CPRA
Coastal Protection and
Restoration Authority of Louisiana

COASTAL PROTECTION AND RESTORATION AUTHORITY

GEOTECHNICAL INVESTIGATION AND 95% ENGINEERING REPORT LAKE MAUREPAS DIVERSION CANAL AND HEADWORKS STRUCTURE ST. JOHN THE BAPTIST PARISH, LOUISIANA

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TABLE OF CONTENTS

TABLE OF CONTENTS

FIGURES

TABLE OF CONTENTS

TABLES

APPENDICES

1.1 INTRODUCTION

The Maurepas Swamp is a generally freshwater cypress-tupelo forested landscape located at the upper tidal margin of the Lake Pontchartrain/Lake Maurepas estuary system. As an unintended consequence of regional flood control measures, the swamp has been isolated from the periodic nutrient-rich sediment nourishment from Mississippi River overbank floods for over a century. As a result, the swamplands are currently threatened by episodic brackish water intrusion from Lake Maurepas, long-term subsidence, and the elimination of nutrient inputs that historically built and maintained the wetlands. In order to alleviate this problem, this project has been designed to divert freshwater from the Mississippi River back into the Maurepas Swamp as part of the Federal and State cooperative initiative, through the Coastal Wetlands Planning, Protection and Restoration Act.

1.2 PURPOSE

The purpose of this report is to evaluate the subsurface conditions at the site of the proposed Lake Maurepas Diversion Canal and provide results of the geological and geotechnical investigation, laboratory test program, geotechnical engineering analyses, and recommendations for foundations systems and considerations for construction. This final report and recommendations presented are based upon the updated design information. This report supersedes our February 2008 report.

1.3 OBJECTIVES

Based on the requirements of the project and the geotechnical conditions across the site, the primary geotechnical challenges that will potentially impact the project design are stability of individual components during construction, construction sequencing, settlement and final stability of the constructed facilities. In addition, construction of the diversion canal will traverse through a Mississippi River flood protection levee, two operating railroad lines, two state highways and a federal highway, some of which must continue operation during project construction. Therefore, careful planning and phasing of construction will be required, along with construction of temporary structures that will be necessary to maintain operability of these systems. The purpose of this geotechnical evaluation is to address these issues and make geotechnical related recommendations for design and construction of both temporary and permanent project structures.

1.4 PROJECT DESIGN CONSIDERATIONS

As previously discussed, this project provides a means of diverting flow from the Mississippi River into the Maurepas Swamp. The project intake will be located on the flood side of the levee along the Mississippi River near Garyville, LA in St. John the Baptist Parish and will extend 5½ miles to the an area north of Interstate Highway I-10. The Louisiana Department of Natural Resources (LNDR) and the United States Environmental Protection Agency have requested that a design flow of 2,000 cubic feet per second (cfs) be obtained for as much of the year as possible. Based on Mississippi River stage data, the intake structure and conveyance channel will be capable of conveying this design flow for approximately six months each year.

1.4.1 Primary Structures

The project will consist of the following primary components:

1.4.1.1 Intake Structure and Headworks

A 135-foot long pile-supported, U-shaped, concrete channel will traverse the area from the inlet in the Mississippi River across the batture area to a gated intake structure at the flood protection levee with diversion flow controlled by three hydraulically actuated, rising-stem sluice gates. The entire intake structure must be securely anchored to resist the hydraulic forces from the flow of the river. Further, it must have sufficient elevation and a low enough flow rate such that smaller riverine species are not swept into the diversion.

1.4.1.2 Levee Culvert Section

An approximately 140-foot section of three (3), side-by-side, 10’ x 10’ pile-supported, concrete box culverts will be used to cross beneath the river levee and adjacent Louisiana Highway 44 (River Road) to the protected side of the flood protection system.

1.4.1.3 Sedimentation Basin

From the culvert section, 135 feet of pile-supported U-shaped concrete channel extend to the inlet weir of the Sedimentation Basin. In this part of the channel, the width transitions from approximately 40 feet to 153 feet to match the basin, while the channel bottom slopes upward from the invert of the culvert at EL -7.00 to the crest of the weir at EL +2.50. The Sedimentation Basin is a trapezoidal shaped channel extending approximately 485 feet to an exit weir, essentially identical to the inlet weir. The bottom of this basin is approximately 36 feet wide, with side slopes of 3H:1V up to the crown of the channel at EL +9.00 . The total width of the basin varies between 156 and 167 feet, the wider portion designed to accommodate an access road along the east side of the channel which will allow for periodic sediment removal. The design of the basin is based on having suitable cross sectional area such that flow is reduced sufficiently to allow sediments to settle out, as well as have adequate volume to store a six-month accumulation of sediment. Rip rap will be placed along the sides and the bottom of the basin to provide a stabilized surface.

1.4.1.4 Conveyance Channel

A conveyance channel will deliver the flow of water into the Maurepas Swamp at a point approximately 1,000 feet north of US Interstate Highway No. 10 (I-10). From the Sedimentation Basin, the channel will be 40 feet wide with side slopes of 4H:1V and will traverse under the alignments of the Canadian National Railway (CN) and the Kansas City Southern Railway (KCS). Beyond the KCS railway, the channel width increases to 60 feet in order to lower the headwater elevation due to height restrictions at the Airline Highway (US 61) crossing. In addition, side slopes will be flattened to 5H:1V. At a point approximately 12,000 feet from the outlet weir of the Sedimentation Basin, the channel will be connected to the existing Hope Canal, which will be widened to increase flow.

1.4.1.5 Pump Station

Since the Conveyance Channel will be connected to the alignment of the existing Hope Canal, the guide levees will prohibit water from the Hope and Bourgeois Canals to flow naturally to the drainage basin in the Maurepas Swamp. To maintain adequate drainage, a pump station will be constructed at the confluence of the Hope Canal and the proposed alignment of the Conveyance Channel to transfer these flows into the channel. The three-bay station will be constructed over a concrete intake section and supported on pile foundations. The bottom of the concrete intake bays are at EL -7.0, with the top concrete slab, supporting the 52' x 60' insulated metal building, at EL 12.5. At the entrance to the station, wingwalls constructed from a combined wall system of steel pipe piles interconnected with steel sheet piling will direct flow to the intake bays. As water enters the inlet section, it flows through a trash rack screen at the entrance of a vortex-suppressed 12' x 18' bay containing a 48-inch diameter suction pipe. In each of the three pipelines the flow is maintained by an in-line pump powered by a natural gas-fired engine which discharges through 48-inch diameter pipes to the channel. An eccentric diffuser discharging over a bed of rip rap will minimize erosion and disturbance of the channel bottom.

1.4.1.6 CN Railroad Culvert Section

An approximately 226-foot section of four (4), side-by-side, pile-supported, 8' x 12' concrete box culverts will be used to carry the channel beneath the alignment of the CN railroad.

1.4.1.7 KCS Railroad Bridge

In lieu of a culvert section, a pile-supported, precast concrete bridge will be constructed over the channel crossing for the KCS railroad.

1.4.1.8 US 61 Culvert Section

An approximately 410-foot section of six (6), side-by-side, pile-supported, 9' x 9' concrete box culverts will be used to cross the area under the alignment of US 61.

1.4.2 Temporary Structures

As previously mentioned, the construction will cross through an existing levee system, as well as railroads and highways. In order to maintain these systems in operation, temporary structures will be required to be constructed at various points along the alignment. These configurations, as well as considerations for construction sequencing, are outlined as follows:

1.4.2.1 Mississippi River Flood Protection

An earthen cofferdam will be used as temporary flood protection while the existing levee is removed for construction of the culverts. In Phases I and II of this operation, the cofferdam and associated sheet-pile cutoff walls will be constructed south of the levee in the batture area. In Phase III, the existing levee is removed, a bypass is constructed for River Road (LA Highway 44) and the original alignment and adjacent utilities are re-routed. Part of the concrete inlet channel and three box culvert sections are then constructed in the existing levee footprint and inlet to the Sedimentation Basin. Phase IV addresses the reconstruction of LA 44, the realignment of the utilities, and the removal of the temporary bypass. The remaining culvert sections and the inlet channel to the Sedimentation Basin are then constructed in Phase V. In Phase VI, the levee is reconstructed and the cofferdam is removed. The final Phases, VII and VIII, will include construction of the remaining portions of the intake channel replacement of levee slope paving.

1.4.2.2 CN Railway

In order to cross beneath the CN railway, the alignment must be temporarily re-routed to allow installation of the culvert. In Phase I of this part of the project, a “shoo-fly” embankment along with a temporary mainline and spur track will be constructed to allow the rail line to bypass the construction area. In Phase II, a sheet-pile wall will be installed to support the sides of the necessary excavation, and most of the culvert sections will be installed beneath the footprint of the original railroad. Phase III will involve re-installing the rail line and spur tracks to their original locations and removing the temporary tracks and embankment. The transition is completed in Phases IV and V, which involves installation of the final portion of the culvert section, removal of temporary sheet piling and construction of the headwalls.

1.4.2.3 Pump Station

In order to construct the pumping station, flows from the Hope and Bourgeois Canals will be diverted around the proposed footprint by excavating a temporary by-pass channel and directing the flow of water with a temporary earthen berm and associated sheet-pile wall. In addition, a Temporary Restraining System (TRS) of braced sheet piling will be required for the construction of the intake section and the foundation of the station.

1.5 SCOPE OF WORK

The geotechnical engineering evaluation scope of work was provided to URS by the Coastal Protection and Restoration Authority (CPRA). The scope of work includes the following technical approach:

- Review information for the project area, including geologic maps, design memoranda, and available geotechnical data
- Perform a subsurface investigation and laboratory testing program to investigate the site conditions and define the behavior of the soils
- Perform engineering studies and analyses to determine
  - Behavior of potentially compressible layers under load
  - Strength properties of soils with respect to capacity of foundation systems
  - Stability of temporary and final constructed facilities
- Provide recommendations for foundation systems, temporary structures, and construction preparation

2.1 GEOLOGIC SETTING

2.1.1 Landscape and Geographic Setting

This site is located in the Mississippi River Delta portion of the Mississippi River Alluvial Plain, which extends from the Louisiana-Arkansas border in the north to the Gulf of Mexico in the south and parallels the main channel of the Mississippi River. In Louisiana, the region is commonly referred to as “the Delta,” a term that, in local usage, is not confined to the delta at the mouth of the Mississippi River. Largely a low-lying and swampy area, the Mississippi Alluvial Plain has an average width of about 50 miles and slopes gently southward from EL 115 ft. on the Louisiana-Arkansas border to sea level at South Pass, one of the delta’s chief channels at the mouth of the Mississippi River. Near the city of New Orleans, parts of the plain lie below sea level.

Along the banks of the Mississippi River are natural levees, which have been built up from river silts deposited by floods. The levees rise as much as 15 ft. above the general level of the surrounding plain, although most are about 6 to 10 ft. high. Over time, these have been improved as part of the federal flood protection system. The levees, some of which are very wide, include some of the state’s best farmland.

At various times in our geologic past, all of the surrounding Gulf Coastal Plain and the alluvial plain have been under the sea on one or more occasions. The geological materials present on the surface reflect this history of intermittent advances and retreats of ancient seas. Some of these sediments were deposited under marine conditions during periods of inundation. Thus, in some areas, we see outcrops of marl, limestone and even sandstone. Sediment in lower areas was deposited in swamps, along streams, and at the mouths of rivers (deltaic deposits).

2.1.2 Geologic History

The Pleistocene Prairie Complex was deposited in a coastal-plain setting approximately 135,000 to 150,000 years before present. During the late-Pleistocene (Wisconsin Stage) glaciation between 120,000 and approximately 10,000 years before present (b.p.), the Prairie Complex was exposed to weathering and erosion due to the low stand of sea level that accompanied glaciation. During the low-stand period, the Prairie Complex sediments were oxidized and desiccated resulting in over-consolidation of the soil and the development of soil-weathering features such as iron oxidation and precipitation of calcium carbonate nodules. The erosion surface of the Pleistocene sediments is a distinct contact that generally can be recognized by the contrast between the Holocene and Pleistocene sediments in color, soil consistency and strength, and water content. The overlying Holocene sediments typically are dark gray or greenish gray in color. The upper portion of the Pleistocene generally is tan, reddish brown, or brown in color as a result of the soil oxidation accompanying weathering during the sea-level low stand. Where the Pleistocene contact is deeper than 50 feet below sea level, the color of the Pleistocene sediment can be mottled tan, orange, and greenish gray and have a smaller contrast with the Holocene sediments. The soil cohesive strengths in the upper portion of the Pleistocene clay range from 0.5 to more than 2.0 tons per square foot (tsf). The immediately overlying Holocene near-shore clays or alluvial swamp clays have much lower cohesive strengths. The high soil strengths of the majority of the Pleistocene soils are due to cementation by hydrous iron oxides, calcium carbonate, siderite, and manganese carbonate resulting from the exposure and weathering that took place during the glacial low stand of sea level. The water content of the overlying Holocene sediments is generally much higher than the Pleistocene sediments, which have water contents less than 50 percent.

The Holocene transgression (rise) of sea level started approximately 18,000 years b.p. and was at an elevation of approximately -100 feet MSL by 9,000 years ago. The shoreline was located in the Maurepas Diversion project area by approximately 6,000 to 4,500 years b.p. (Saucier, 1994). At this time, the elevation of sea level was approximately 10 to 15 feet below its present level. As sea level continued to rise, clay and sand was deposited across the area in nearshore Gulf environments. The St. Bernard delta complex of the Mississippi River delta began to form approximately 4,700 years b.p. and fine-grained sediments were deposited in interdistributary bays adjacent to river channels. The bay-sound deposits graded upward into the interdistributary deposits as the depositional environment shallowed.

The Mississippi River established its present course in this area between approximately 4,500 to 3,000 years ago and has deposited clay, silty clay, and silt in the aforementioned natural-levee deposits adjacent to the river during periods of river flooding. This portion of the Mississippi River channel has been active during the St. Bernard delta complex and during the subsequent development of the Lafourche delta complex and the present-day Plaquemines delta complex. The natural-levee deposits are up to approximately 20 feet in thickness near the Mississippi River and generally become thinner or absent away from the river. In the area near I-10, the Holocene sediments can consist of soft to very soft clays that have been deposited in fresh-water swamp environments since the Mississippi River established its present course.

2.1.3 Geology and Geomorphology

The Lower Mississippi River area is underlain by Pleistocene and Holocene alluvial deposits of the Mississippi River alluvial and deltaic plain. In the area of the Maurepas Diversion project (western part of St. John the Baptist Parish), the Holocene deposits generally range from 25 to 50 feet thick and are underlain by undifferentiated alluvial deposits of the Pleistocene-age Prairie Formation (Prairie Complex). Along the alignment of the proposed diversion channel, the Holocene sediments consist of fine-grained sediments deposited in deltaic and alluvial flood-plain environments. The Holocene sediments also include silt and fine sand that were deposited in natural-levee sedimentary environments. The Pleistocene Prairie Complex consists of undifferentiated alluvial deposits overlying the Pleistocene-age major aquifer sands of the Lower Mississippi River area (Gramercy aquifer, Norco aquifer, and Gonzales-New Orleans aquifer). In the Maurepas Diversion project area, the Prairie Complex ranges from 130 to 150 feet in thickness and consists of clay, silty clay, silt, and sand deposited in alluvial environments. The sand zones within the Prairie Complex are identified as shallow sands that overlie the major aquifer zones.

The Holocene sediments in the area adjacent to the Mississippi River consist of medium to stiff gray clay and silty clay with interbeds of silt and silty sand. The Holocene sediments in this area were interpreted to consist of approximately 10 feet of bay-sound deposits and approximately 20 feet of interdistributary deposits that accumulated in deltaic environments. The overlying sequence of point bar and natural levee deposits accumulated adjacent to the present course of the Mississippi River. The natural levee deposits range from 10 to 20 feet in thickness in the area adjacent to the Mississippi River. The geologic profile shows the approximate division between the natural-levee deposits and the underlying deposits, which include back-swamp and deltaic deposits.

The underlying Pleistocene deposits consist principally of stiff gray clay. Discontinuous silt and sand zones occur within the Pleistocene as shown by a silt zone from depths of 137 to 142 feet in soil boring B-02A and a sand interval from 118 to 125 feet in CPT-16. A silt and sand zone is shown at an elevation of approximately -125 feet NGVD adjacent to the Mississippi River. The top of the Gramercy aquifer occurs at an elevation of approximately -200 feet NGVD in this area and is approximately 75 feet thick.

3.1 GEOTECHNICAL INVESTIGATION

3.1.1 Site Description

The site is located north of the Mississippi River in St. John the Baptist Parish, LA. The project begins along the batture of the river and extends northward to a point approximately 1,000 feet north of I-10. The site is bound by the marshes of Maurepas Swamp to the north, the Mississippi River to the south, the town of Garyville, La to the east and industrial and agricultural properties to the west. The area is generally unimproved and heavily vegetated. With the exception of the river levee, the ground is generally flat and at about the same elevation as the surrounding properties. The alignments of CN and KCS Railroads, LA 44, US 61 and I-10 all traverse the site in an east-west direction and are higher than the surrounding grade.

3.1.2 Subsurface Exploration

A soil strength profile had been previously developed for levee reach VII (see Plate 37, attached in Appendix C) by the USACE for geotechnical design of the original Mississippi River levee system. After the conducting initial fieldwork in 2007 for the Lake Maurepas project, and after reviewing the historical boring information of the site (levee reach VII) obtained from the USACE New Orleans District, it was determined that a higher soil strength profile for geotechnical design could be applicable for the project. The higher strength profile would allow for the possible use of an earthen construction cofferdam which would provide considerable project cost savings.

After presentation of this information to the USACE, URS subsequently performed an additional geotechnical field investigation in 2011 in order to collect sufficient subsurface information to confirm the higher soil strength profile for design. This field investigation included one (1) soil test boring (5-inch diameter samples) and two (2) CPT soundings to a depth of 130 feet below ground surface in the batture area near the proposed intake structure. This boring and associated laboratory tests was used to confirm the higher soil strength profile suggested by the historical subsurface information, and to obtain suitable settlement data for design of the earthen cofferdam. In addition, URS conducted three (3) soil test borings (5-inch diameter samples) to the east of River Road to a depth of 40 feet for geotechnical design of the proposed Sedimentation Basin. These borings and associated laboratory tests were used to determine the suitability of the materials excavated from this area for possible reuse as fill for construction of the earthen cofferdam. The soil boring and CPT sounding locations are shown on the attached Test Location Plan in Appendix A.

After the 2011 geotechnical field investigation was completed and submitted for review, the USACE approved the proposed higher strength soil profile for geotechnical design. Subsequent engineering studies for the proposed earthen cofferdam, indicated that the structure footprint would encroach upon an existing pond (previous borrow pit), requiring this area to be filled in order to facilitate cofferdam construction. Additionally, the KCS Railroad directed that a bridge would be used rather than the box culvert design at their crossing location. In order to obtain suitable geotechnical information for the design and analysis for these structures and other project components, an additional boring and two CPT soundings were performed in 2013.

The soil test borings were performed using BK-66, Diedrich-D-50 and CME-750 drilling equipment. The boreholes were advanced using mud-rotary drilling techniques and all drilling and sampling was performed in accordance with ASTMs D1586 and D1587. Samples obtained in the field were returned to the soil laboratory for laboratory testing and visual classification in accordance with the Unified Soil Classification System. Boring logs are attached in Appendix B showing the subsurface conditions encountered.

The CPT soundings were advanced using a Hogentogler 20-ton electronic CPT rig operated in accordance with ASTM D-5778. CPT log sheets are attached in Appendix B which graphically show the cone tip resistance, local friction, pore water pressure, equivalent N_{60} values and interpreted soil types at each sounding location. Soil classifications were interpreted from methods recommended by Robertson and Campanella. Correlations between cone resistance and Standard Penetration Test "N" values were performed according to the methods developed by Robertson, Campanella and Wightman.

After completion, all boreholes and soundings were grouted to full depth in accordance with LADOTD specifications.

3.1.2.1 Initial Investigation

The initial geotechnical field investigation was performed between the dates of August 26 and September 15, 2007. During this exploration, a total of seventeen (17) soil test borings and seventeen (17) Cone Penetration Test (CPT) soundings were conducted to depths of 50 to 150 feet along the proposed alignment. Two of the borings were conducted at the proposed inlet structure on the Mississippi River levee, and were 5-inch diameter in accordance with USACE guidelines. Two of the CPT soundings were also performed on the river levee for the proposed inlet structure. Five (5) 3-inch diameter borings and the remaining fifteen (15) CPT soundings were conducted north of the river levee to approximately US 61. The other ten (10) 3-inch diameter soil test borings were conducted along the existing canal extending north to I-10 using airboat mounted drilling equipment.

3.1.2.2 Supplemental Investigation

As previously mentioned, it was desirable to collect sufficient subsurface information to confirm the higher soil strength profile for the design of the earthen cofferdam. On July 14, 2011, URS conducted two (2) CPT soundings to a depth of 130 feet in the batture area near the proposed intake structure. On August 4 through 9, 2011, a single soil test boring (5-inch diameter samples) was also performed in this area to the same depth. In addition, On August 16 through 18, 2011, URS conducted three (3) soil test borings (5-inch diameter samples) to the east of River Road to a depth of 40 feet for geotechnical design of the proposed Sedimentation Basin. These three supplementary borings and associated laboratory tests were used to determine the suitability of the materials excavated from this area for possible reuse as fill for construction of the earthen cofferdam.

3.1.2.3 Final Investigation

To determine the subsurface conditions in the pond area, URS conducted two (2) additional CPT soundings on April 9 and 10, 2013 to a depth of 180 feet along the batture area. On January 22 through 24, 2013, an additional soil test boring was performed to a depth of 155 feet north of the KCS RR alignment in order to provide data for the design of the deep foundations that would be required for the proposed bridge.

3.1.3 Laboratory Testing

To more closely define the characteristics of the soils, representative soil samples were selected for testing to determine their approximate strengths, compression characteristics, as well as index properties. Test procedures included:

- ASTM D2216 – Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- ASTM D4318 – Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- ASTM D7263 – Standard Test Methods for Laboratory Determination of Density (Unit Weight) of Soil Specimens
- ASTM D2166 – Standard Test Method for Unconfined Compressive Strength of Cohesive Soil
- ASTM D2850 – Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils
- ASTM D422 – Standard Test Method for Particle-Size Analysis of Soils
- ASTM D2435 – Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading
- ASTM D2974 – Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils

Results of the laboratory testing are shown on the individual boring logs and on summary sheets in Appendix D.

4.1 GEOTECHNICAL ANALYSES

4.1.1 Soil Analysis

The geologic interpretation of the subsurface soils for this project has been based on the soil boring logs, laboratory test results, and Cone Penetrometer Test (CPT) measurements that were acquired during the geotechnical investigation. A description of the materials encountered during the geotechnical field investigation is included on the Boring Log Sheets and CPT Log Sheets in Appendix B.

As previously mentioned, historical soil boring data was acquired from the USACE and was used to develop the soil profile for the levee area. Subsequent to our 2011 supplemental investigation, an updated soil strength profile was presented to the USACE for review. After an extended review process, the USACE approved the proposed soil strength profile for geotechnical design (see USACE letter dated October 5, 2012 in Appendix C). The borings used for the updated soil profile are listed in Table 4-1 below and copies of the boring logs are included in Appendix B.

Table 4-1 USACE Borings

Boring ID	*Date*
R-144.25-LU	1973
R-144.2-LUT	1973
R-144.2-LU	1969

USACE approved stratigraphy and strength profiles for the levee and batture are included in Appendix C. Summaries of the soil strength profiles are shown in Table 4-2 and Table 4-3 below:

Table 4-2 Design Soil Parameters (Q-Case) - Levee Area

Soil Type	Elevation (feet)		Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)	K_0	K_a	K_p
	Top	Bottom						
Clay	31	20	112.5	600	0	0.95	1	1
Clay	20	0	112.5	600 - 900	0	0.95	1	1
Clay	0	-10	112.5	900 - 950	0	0.95	1	1
Clay	-10	-22	107	950	0	0.95	1	1
Silt	-22	-27.5	107	200	15	0.74	0.59	1.70
Clay	-27.5	-35	107	950 - 1070	0	0.95	1	1
Clay	-35	-52	120	1070 - 1250	0	0.95	1	1
Clay	-52	-60	120	1250 - 1334	0	0.95	1	1
Clay	-60	-73	112.5	1334 - 1470	0	0.95	1	1
Clay	-73	-92	112.5	1470 - 1600	0	0.95	1	1
Clay	-92	-120	112.5	1600 - 2210	0	0.95	1	1
Sand	-120	-140	112.5	0	32	0.47	0.31	3.25
Clay	-140	-160	112.5	2430 - 2650	0	0.95	1	1

Table 4-3 Design Soil Parameters (Q-Case) - Batture Area

Soil Type	Elevation (feet)		Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)	K_0	K_a	K_p
	Top	Bottom						
Clay	31	20	115	600	0	0.95	1	1
Clay	20	0	112.5	600 - 743	0	0.95	1	1
Clay	0	-10	107	743 - 814	0	0.95	1	1
Clay	-10	-22	107	814 - 900	0	0.95	1	1
Silt	-22	-27.5	107	200	15	0.74	0.59	1.70
Clay	-27.5	-35	107	950 - 1070	0	0.95	1	1
Clay	-35	-52	120	1070 - 1250	0	0.95	1	1
Clay	-52	-60	120	1250 - 1334	0	0.95	1	1
Clay	-60	-73	112.5	1334 - 1470	0	0.95	1	1
Clay	-73	-92	112.5	1470 - 1600	0	0.95	1	1
Clay	-92	-120	112.5	1600 - 2210	0	0.95	1	1
Clay	-120	-160	112.5	2210 - 2650	0	0.95	1	1

4.1.2 Seismic Considerations

4.1.2.1 Seismic Site Class

Based on the subsurface conditions encountered, with reference to Table 1613.5.2 of the 2006 Edition of the International Building Code (IBC), this site would best be categorized as Site Class E.

4.1.2.2 Site Coefficients

In accordance with Section 1613.5 of the IBC, design parameters were calculated for an earthquake having a 2% probability of exceedance in a 50-year period. The results of these calculations, expressed as a percent of the gravitational force (g) are as follows:

Five-Percent Damped Design Spectral Response Acceleration Parameters

Short Periods (0.2 sec) S_{DS} = 19.3 %g

1-Second Periods S_{D1} = 11.4 %g

5.1 RECOMMENDATIONS

5.1.1 Inlet Structure and Monoliths

The intake structure must convey the flow from the Mississippi River into the diversion channel with a minimum of head loss and must 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 (East bank of the Mississippi River near the St. Bernard/Plaquemines Parish line) and Davis Pond (West bank of St. Charles Parish).

The proposed intake structure will be located approximately 100 feet south of the crown of the levee, as placing the structures close to the levee provides a solid foundation and minimizes the required length of the culverts. The platform of the culvert will support a control house at EL 33.5 (NAVD88-LDNR) to protect against high river stages. The sluice gate and culvert elevations were set as high as possible to minimize excavation costs, and the culverts will be installed flat, since they will operate under outlet control and slope is irrelevant to their hydraulic performance. The top-of-culvert elevation of is set at EL 6.0 (NAVD88-LDNR) in order to pass under the roadside drainage ditch along River Road (LA 44).

The driving force for delivering the target flow to the conveyance channel is provided by the stage of the Mississippi River, therefore the water levels in the river and the channel are 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.

Refinements in the design over the course of the project have allowed the intake features to retain their 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. However, the intake U-channels have been re-designed to include three sections instead of two to facilitate constructability. The riverside wingwalls have also been re-configured to be straight instead of curved to reduce the complexity of construction as well.

5.1.1.1 Axial Pile Capacities

Pile capacity calculations were performed various size HP-section, Pre-cast Pre-stressed Concrete Piles (PPCP) and open-ended steel pipe piles. The pile capacity curves for the various pile sizes for use in structural design of the inlet structure and monoliths are presented in Appendix E.

5.1.1.2 Lateral Analysis

A lateral load capacity analysis was performed using LPILE® software, which employs p-y analysis to determine the deflections at the ground surface of a single pile under specific loading conditions. All lateral loads are assumed to be applied at the top of the pile under a free head condition. The soil parameters used in the analyses are shown below in Table 5-1 and Table 5-2. The complete LPILE analysis results for the various pile types and sizes considered for the inlet structure and monoliths are presented in Appendix F.

Table 5-1 LPILE Design Soil Parameters - Levee Area

Soil Type	*Elevation (feet)*		*Unit Weight (pcf)*	*Cohesion (psf)*	*Friction Angle (deg)*	*k_s (pci)*	*ε_{50}*
	Top	*Bottom*					
Clay	31	20	112.5	600	0	500	0.010
Clay	20	0	112.5	600 - 900	0	500	0.010
Clay	0	-10	112.5	900 - 950	0	500	0.010
Clay	-10	-22	107	950	0	500	0.010
Silt	-22	-27.5	107	200	15	500	0.02
Clay	-27.5	-35	107	950 - 1070	0	500	0.007
Clay	-35	-52	120	1070 - 1250	0	500	0.007
Clay	-52	-60	120	1250 - 1334	0	500	0.007
Clay	-60	-73	112.5	1334 - 1470	0	60	0.007
Clay	-73	-92	112.5	1470 - 1600	0	500	0.007
Clay	-92	-120	112.5	1600 - 2210	0	1000	0.007
Sand	-120	-140	112.5	0	32	-	-
Clay	-140	-160	112.5	2430 - 2650	0	1000	0.005

Table 5-2 LPILE Design Soil Parameters - Batture Area

Soil Type	Elevation (feet)		Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)	k_s (pci)	ε_{50}
	Top	Bottom					
Clay	31	20	115	600	0	500	0.01
Clay	20	0	112.5	600 - 743	0	500	0.01
Clay	0	-10	107	743 - 814	0	500	0.01
Clay	-10	-22	107	814 - 900	0	500	0.01
Silt	-22	-27.5	107	200	15	500	0.02
Clay	-27.5	-35	107	950 - 1070	0	500	0.01
Clay	-35	-52	120	1070 - 1250	0	500	0.007
Clay	-52	-60	120	1250 - 1334	0	500	0.007
Clay	-60	-73	112.5	1334 - 1470	0	500	0.007
Clay	-73	-92	112.5	1470 - 1600	0	500	0.007
Clay	-92	-120	112.5	1600 - 2210	0	1000	0.007
Clay	-120	-160	112.5	2210 - 2650	0	1000	0.005

5.1.1.3 Unbalanced Loads

If a computed safety factor is less than required by the HSDRRD Guidelines, additional analyses must be completed. The most critical failure surface produced by the search procedure is fixed in place, and a horizontal force is added until the required minimum safety factor is produced. The horizontal force required to produce the required minimum safety factor is computed and becomes the "unbalanced load". Based upon our evaluation, no unbalanced loads were determined for the inlet structure.

5.1.1.4 Horizontal and Vertical Pressure on Monoliths

In order to determine vertical and horizontal pressures due to surcharge loadings for design of the concrete monoliths, the procedures outlined in EM 1110-2-2902 Section 2.4 should be followed, using an embankment Condition III. Additionally, a 600 psf adhesion loading should be applied to the vertical sides of earth retaining structures. This design guidance was provided through email correspondence from the USACE dated July 25, 2013.

5.1.1.5 Earth Retaining Structures

Excavations should be designed to consider lateral earth pressures plus any surcharge loadings. An additional component for hydrostatic pressures should also be included if applicable.

Recommended coefficients for calculating lateral earth pressures are summarized below in Table 5-3 and Table 5-4.

Table 5-3 Coefficient of Later Earth Pressure - Clean Sand

Case	Coefficient
Ka (active)	0.31
Kp (passive)	3.25
Ko (at-rest)	0.47

Table 5-4 Coefficient of Lateral Earth Pressure - Clay

Case	Coefficient	
	Q-Case	S-Case
Ka (active)	1.0	0.44
Kp (passive)	1.0	2.28
Ko (at rest)	0.95	0.61

5.2 CONSTRUCTION COFFERDAM

As mentioned previously, with respect to construction of the culvert section within the existing flood protection levee, URS has determined that an earthen cofferdam with a sheet-pile cutoff wall to be an economical alternative for a temporary structure. Stability analyses were performed for the earthen cofferdam structure based upon the USACE approved soil strength profile (Appendix C). The results of these cofferdam analyses were presented in a separate report, dated December 19, 2012, submitted to the USACE NOD for review and comment.

The original design of the River Road (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. Subsequent to our December 19, 2012 submittal to the USACE for the earthen cofferdam design, 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 these headworks facilities.

A 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 seven phases of the revised design to insure that the USACE's factors of safety are met for each stage of construction.

A subsequent submittal to the USACE for the earthen cofferdam dated July 2, 2013 also presents the construction sequencing and TRS analyses. At the time of this report, no comments pertaining to these submittals had yet been received from the USACE. Accordingly, the cofferdam analyses results, drawings, and subsequent submittal information are not included as part of this report. Once the USACE review process has been completed and any comments have been addressed, the results of the analyses will be provided under separate cover.

5.2.1 Canadian Northern (CN) Railroad Crossing

The CN Railroad directed that the crossing shall incorporate a reinforced concrete box (RCB) culvert and 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 feet and from the base of rail to the top of the RCB it shall be 3 feet. A permit from the railroad will be required and the review process is expected to take as long as 6 months.

The CN Railroad subsequently 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 Railroad with an additional 1260 feet of siding (shoo-fly). This change required the design of the siding, including its horizontal and vertical geometry, along with 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, requiring that the pile foundations be designed for this added capacity.

5.2.1.1 Axial Pile Capacities

Pile capacity calculations were performed for various size PPCP piles. The design strength parameters used in the pile capacity analysis for the CN Railroad are presented below in Table 5-5. The complete pile capacity analyses results are presented in Appendix G.

Table 5-5 Design Soil Parameters - CN Railroad

Soil Type	Elevation (feet)		Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)	K_0	K_a	K_p
	Top	Bottom						
Clay	6	1	110	750	0	0.95	1	1
Clay	1	-24	107	500	0	0.95	1	1
Clay	-24	-44	115	500 - 1000	0	0.95	1	1
Clay	-44	-84	115	1000	0	0.95	1	1

5.2.1.2 Settlement Analysis

A settlement analysis was performed for the shoo-fly system based upon the subsurface conditions estimated along the proposed alignment, an average placement of 4.5 feet of embankment fill and 2 feet of ballast/sub ballast. Due to anticipated settlements, we recommend that a layer of Tensar TX-140 geogrid (or approved equivalent) be incorporated beneath the railroad embankment materials in order to reduce differential settlements along the railroad alignment. Results of the consolidation settlement analyses are presented in Appendix G.

5.2.2 Kansas City Southern (KCS) Railroad

At the request of the KCS Railroad, the crossing was designed as a group of pile-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 facilitated by the use of a temporary false-work bridge, for which a detailed set of design drawings, incorporating a phased construction process, was developed. Upon review of the design, KCS Railroad changed their endorsement and requested that a bridge structure be used instead, and accordingly the crossing design was changed to a railroad bridge. Since the existing subsurface data was not sufficient to design the deep pile foundations that would be required, an additional soil boring was performed at the location of the crossing.

As requested by the URS structural design team, pile capacity calculations were performed for various size open ended steel pipe piles. The design strength parameters used in the pile capacity analysis for the KCS Railroad are presented below in Table 5-6. The complete pile capacity analyses results are presented in Appendix H.

Table 5-6 Design Soil Parameters - KCS Railroad

Soil Type	Elevation (feet)		Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)	K_0	K_a	K_p
	Top	Bottom						
Clay	3	-7	107	250	0	0.95	1	1
Clay	-7	-20	92	250	0	0.95	1	1
Clay	-20	-25	103	500	0	0.95	1	1
Clay	-25	-45	115	1150	0	0.95	1	1
Clay	-45	-55	124	1500	0	0.95	1	1
Clay	-55	-80	108	1250	0	0.95	1	1
Clay	-80	-102	108	1400	0	0.95	1	1
Clay	-102	-120	122	1630	0	0.95	1	1
Clay	-120	-135	110	2020	0	0.95	1	1
Sand	-135	-152	125	0	32	0.47	0.31	3.25

5.2.3 US Highway 61 Crossing

The US 61 crossing will consist of a 410-foot, six-barrel, 9-ft x 9-ft, reinforced concrete box culvert constructed per LDOTD standards. The culvert may be either pre-cast or cast-in-place 4,000 psi reinforced concrete and will be pile-supported. An earthen levee will be required to contain the water within the upstream and downstream channel sections. The levee side slopes should have a slope of 3H:1V or flatter on the south side and 5H:1V or flatter on the north side of the crossing. The RCB and diversion channel shall be centered within the proposed 300-ft right-of-way.

In order to maintain the highway in operation, the installation of the culvert will be conducted in two phases. In Phase I, the westbound lanes will be shut down and the resulting two-way traffic redirected to the eastbound lanes. A sheet pile will be driven in the median of the highway to allow for excavation on the north side of the alignment and subsequent installation of the culverts. After restoration of the westbound lanes, traffic will be diverted to them and the sheet pile will be removed. The culverts will then be installed on the south side of the alignment and the eastbound lanes restored..

Pile capacity calculations were performed various size PPCP piles. The design strength parameters used in the pile capacity analysis for the US 61 crossing are presented below in Table 5-7. The complete pile capacity analyses results are presented in Appendix I.

Table 5-7 Design Soil Parameters - US 61

Soil Type	*Elevation (feet)*		*Unit Weight (pcf)*	*Cohesion (psf)*	*Friction Angle (deg)*	K_0	K_a	K_p
	Top	*Bottom*						
Clay	3.7	-3.3	102	640	0	0.95	1	1
Clay	-3.3	-6.3	102	300	0	0.95	1	1
Clay	-6.3	-13.3	120	300	0	0.95	1	1
Clay	-13.3	-31.3	120	300 - 1375	0	0.95	1	1
Silt	-31.3	-46.3	120	200	15	0.74	0.59	1.70
Clay	-46.3	-76.3	120	1375	0	0.95	1	1

5.2.4 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 or they might settle in the downstream reaches requiring removal by dredging; hence, 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 and have adequate storage capacity to accumulate six months of sediment without requiring cleaning. The maximum settling velocity of a 0.2 mm particle of sand in water is approximately 4 feet per minute. The cross-sectional area of the basin was then established calculated to achieve a flow velocity of approximately 1 ft/s, which would prevent re-suspension of the settled solids due to turbulence, while maintaining the desired flow rate of 2,000 cfs.

The percent sand in the river water at Maurepas was derived by interpolating data recorded at St. Francisville and Belle Chasse, which are upstream and downstream of the site, respectively. A review of the data from a similar project in Plaquemines Parish, LA (Caernarvon Diversion Outfall) indicated a ratio of the percent sand in a diversion to that in the adjacent river water. Using that ratio as a guideline, the percentage of sand in the influent to the Maurepas diversion was estimated and utilized to calculate the approximate mass and volumetric accumulation rate of sand. The Sedimentation Basin will have a central section 265-ft long by 66-ft wide, with 3H:1V side slopes adding 60-ft of width on each side.

5.2.4.1 Stability Analysis

Due to hydraulic considerations, maintenance issues, and confinements of the project right-of-way, the Sedimentation Basin is designed to incorporate side slopes of 3H:1V. Additionally, heavy excavation equipment and dump trucks will be used along the Sedimentation Basin area for ongoing maintenance and clean-out operations, imparting considerable traffic loadings that can affect overall slope stability.

Global stability analyses were performed on several cross sections of the basin. The design soil properties used in the stability analyses are shown below in Table 5-8.

Table 5-8 Stability Design Soil Parameters - Sedimentation Basin

Soil Type	Elevation (feet)		Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)	K_0	K_a	K_p
	Top	Bottom						
Clay	9	7	115	600	0	0.95	1	1
Clay	7	3	112	800	0	0.95	1	1
Clay	3	-3	110	400	0	0.95	1	1
Clay	-3	-12.5	106	635	0	0.95	1	1
Clay	-12.5	-20	106	440	0	0.95	1	1
Clay	-20	-31	106	330 - 440	0	0.95	1	1
Clay	-31	-41	115	850	0	0.95	1	1
Clay	-41	-61	115	1300	0	0.95	1	1
Clay	-61	-73	114	975	0	0.95	1	1

Results of the stability analyses are shown below in Table 5-9, Table 5-10, Table 5-11 and Table 5-12.

Table 5-9 Design Factors of Safety – Sedimentation Basin – Station 18+20

Analysis No.	Station	Method		Berm Side	Load	
		Circular	Block		Location	(psf)
1	18+20	2.541	2.181	Right		0
2	18+20	2.293	1.962	Right	Crest	300
5	18+20	-	2.177	Left		0
6	18+20	-	1.968	Left	Crest	300

Table 5-10 Design Factors of Safety – Sedimentation Basin – Station 19+84

Analysis No.	Station	Method		Berm Side	Load	
		Circular	Block		Location	(psf)
1	19+84	1.245	1.314	Right	Crest	550
2	19+84	-	1.214	Left	Crest	550
3	19+84	-	1.251	Right	Ramp	550

Table 5-11 Design Factors of Safety – Sedimentation Basin – Station 21+00

Analysis No.	Station	Method		Berm Side	Load	
		Circular	Block		Location	(psf)
1	21+00	1.352	-	Right		0
2	21+00	1.368	1.342	Right	Ramp	550
3	21+00	-	1.335	Left		0
4	21+00	-	1.215	Left	Crest	550
5	21+00	-	1.342	Right		0

Table 5-12 Design Factors of Safety – Sedimentation Basin – Station 26+00

Analysis No.	Station	Method		Berm Side	Load	
		Circular	Block		Location	(psf)
1	26+00	1.918	1.929	Right		0
2	26+00	1.780	1.803	Right	Crest	300
5	26+00	-	1.945	Left		0
6	26+00	-	1.815	Left	Crest	300

5.2.5 Conveyance Channel

The alignment of the proposed conveyance channel was selected to divert the river flow to the targeted discharge location within the Maurepas wetlands at a minimum cost, using a 300-ft wide right-of-way along a 5½-mile long strip 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, 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 beneath I-10 and terminates 1,000 feet north of the interstate highway.

As part of a refinement of the project design, the conveyance channel has been widened to provide additional freeboard between the top of the guide levees and the water surface elevation and the side slopes have been adjusted to minimize potential sloughing. South of the KCS railroad crossing, the channel will have an adjusted typical bottom width of 40 feet with a flattened water-side slope of 4H:1V and 3H:1V slope on the land side. North of the KCS railroad, the bottom width has been widened to 60 feet and the water-side slope will remain 5H:1V while the land-side slope will be changed to 3H:1V.

5.2.5.1 Stability Analysis

Global stability analyses were performed on several cross sections of the conveyance channel, both north and south of US 61. In the analyses, traffic loading was applied to the top of the guide levee in the form of a lightweight pickup truck with a maximum axle load of 5,000 lbs. The design soil properties used in the stability analyses are shown below in Table 5-13 and Table 5-14.

Table 5-13 Stability Design Soil Parameters - Conveyance Channel South of US 61

Soil Type	Elevation (feet)		Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)	K_0	K_a	K_p
	Top	Bottom						
Clay	9	3.5	115	600	0	0.95	1	1
Clay	3.5	-11.5	98	450	0	0.95	1	1
Clay	-11.5	-26.5	115	325	0	0.95	1	1
Clay	-26.5	-39.5	117	1150	0	0.95	1	1
Sand	-39.5	-49.5	120	0	30	0.50	0.33	3
Clay	-49.5	-76.5	119	1100	0	0.95	1	1

Table 5-14 Stability Design Soil Parameters - Conveyance Channel North of US 61

Soil Type	Elevation (feet)		Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)	K_0	K_a	K_p
	Top	Bottom						
Clay	7	0	115	600	0	0.95	1	1
Clay	0	-5	88	400	0	0.95	1	1
Clay	-5	-16	88	200	0	0.95	1	1
Clay	-16	-20	120	400	0	0.95	1	1
Clay	-20	-24	120	800	0	0.95	1	1
Clay	-24	-30	125	800	0	0.95	1	1
Clay	-30	-40	120	800	0	0.95	1	1
Clay	-40	-50	120	1300	0	0.95	1	1

Results of the stability analyses are shown below in Table 5-15 and Table 5-16. The complete analysis results as well as the design shear strengths and typical cross sections used in the analysis are included in Appendix K.

Table 5-15 Design Factors of Safety - Conveyance Channel South of US 61

Analysis	Channel Width (ft)	Method		Levee Side
		Circular	Block	
1	40	-	1.55	Left
2	40	-	1.59	Right
3	60	-	1.47	Left
4	60	-	1.54	Right

Table 5-16 Design Factors of Safety - Conveyance Channel North of US 61

Analysis	Channel Width (ft)	Method		Levee Side
		Circular	Block	
1	140+50	-	1.49	Left
2	140+50	-	1.36	Right
3	210+00	-	1.49	Left
4	210+00	-	1.4	Right
5	211+50	-	1.49	Left
6	211+50	-	1.38	Right

5.2.5.2 Settlement Analysis

The conveyance channels guide levees will have a final grade elevation varying from EL 6.5 to EL 9.0. The area south of US 61 will require an average of three (3) feet of fill, while the area north of US 61 may require as much as 9 feet of fill to achieve the desired subgrade elevation. A settlement analysis was conducted to determine the amount of total settlement as well as the rate at which it will occur over time.

During the estimated construction period of 3 years, estimated consolidation settlements could be on the order of 44 inches due to the 9 feet of fill placement. Lesser fill heights will result in lower corresponding consolidation settlements during the construction period. The settlement estimates presented are based upon consolidation of the underlying supporting soils. Settlements may also be expected in extremely soft soil conditions (swamp areas) due to shear failure of the supporting soils from fill placement. However, these shear failure type settlements may be expected occur rather quickly during initial fill placement operations. Anticipated settlements should be considered when determining guide levee overbuild elevation.

In order to monitor settlement of the subgrade over time, settlement plates should be installed at 1,000-foot intervals on alternating sides of the channel.

The consolidation settlement analyses results for varying fill heights and time periods are presented in Appendix K.

5.2.6 Pump Station

As the Conveyance Channel will be connected to the existing Hope Canal north of US 61, the guide levees will prevent the natural drainage in this region from reaching the Maurepas Swamp. In order to provide flow from the Hope and Bourgeois Canals into the Conveyance Channel, a pump station will be constructed to maintain the drainage in this area.

The pump station will be constructed in the existing alignment of the Hope Channel; therefore the existing flow must be diverted around the proposed station by excavating a temporary by-pass channel around the station footprint. The existing grade in the station area will raised to approximately EL 4.0 and a temporary sheet-pile wall will be used to maintain the flow along the west side of the pump station. Since the soils in this area are generally weak and highly compressible, the addition of fill would be expected to cause consolidation and settlement, and possibly result in localized shear failures along the areas where the elevation changes due to placement of fill.

5.2.6.1 Deep Mixing Method (DMM)

The Deep Mixing Method is an admixture stabilization method that uses cement, lime, slag, and other pozzolonic materials, and combinations of these stabilizers to increase the strength and stiffness of soft or loose ground. These stabilizers are blended into the ground using a variety of mixing tools, such as vertical rotating shafts or paddles that create continuous shear panels of overlapping columns. A typical DMM section is shown below in Figure 5-1.

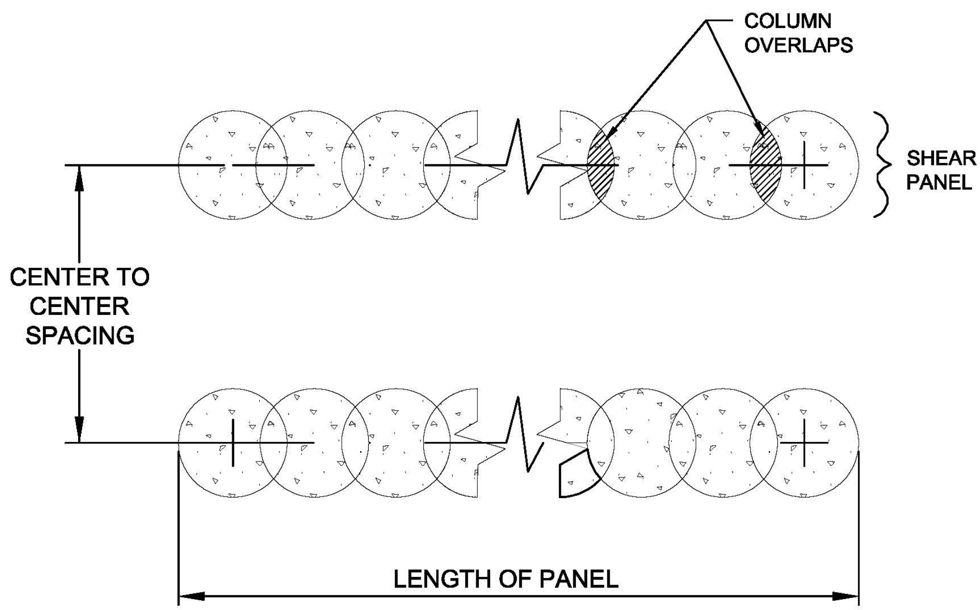

Figure 5-1 Typical DMM Shear Panel

To mitigate the aforementioned problems with weak soils, DMM will be utilized to create shear panels in the subgrade which will reduce the settlements and increase the stability of the subgrade. Further, to stabilize the area at the inlet to the pump station, DMM will also be employed to reduce settlement, erosion, and strengthen the area between the wingwalls to minimize stresses and consequent deflections of the wingwalls.

The amount of stabilization used in a given zone is described by the term "replacement ratio", which is defined as the effective width of a shear panel divided by the center-to center spacing. For the DMM across the area of the pump station, rows of nominal 3-foot diameter columns with an approximately 9-inch overlap will be spaced 9 feet apart, center-to-center, resulting in a replacement ratio of 30%. At the inlet of the station, the rows will be 3 feet apart, center-to-center, resulting in a replacement ratio of 100%.

5.2.6.2 Axial Pile Capacities

Analyses for the deep foundations were performed and the results submitted in a previous URS report, dated September 16, 2008. A copy of the report is attached in Appendix M.

5.2.6.3 Inlet Wingwalls

Intake basin wingwalls will channel water to the intake sump of the pump station, and will consist of a combined wall system, also referred to as a "pipe-z combination" or "combo" wall. The pipe-Z system is composed of large diameter pipe, C-type connectors, and AZ intermediary steel sheet piling. The AZ sheets transfer pressure to the pipe which carries most of the load. A typical pipe-Z wall system is shown below in Figure 5-2.

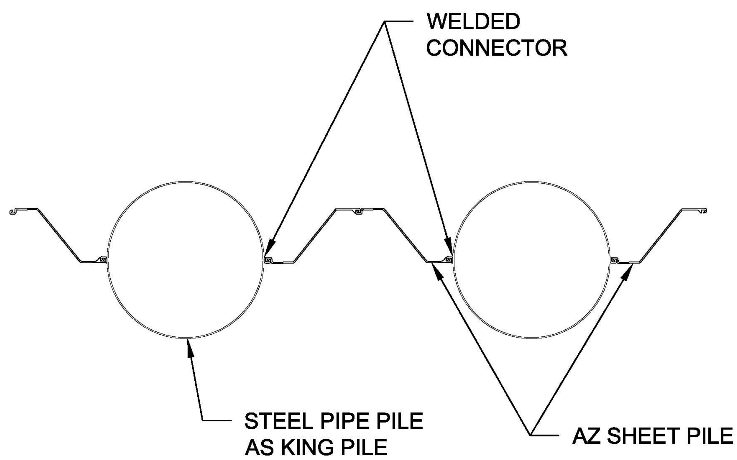

Figure 5-2 Typical Pipe-Z Wall System

For the wingwalls, a pipe-z system of 24-inch diameter PAZ24 steel pipe will be used with AZ-19-700 steel sheets. The top of wall will be at EL +5.5 with the pile tips at EL -40.0. An analysis for the wall is included in Appendix M.

5.2.6.4 TRS System

As previously mentioned, a TRS system will be required to construct the intake structure of the pump station. Analyses for the TRS are also included in Appendix M.

6.1 GEOTECHNICAL DESIGN CONSIDERATIONS

6.1.1 Elevations

Unless otherwise noted, all elevations referred to in this report are in US feet and are based on the NAVD88 (2004.65) datum.

6.1.2 Soil Properties

1. Where a range of shear strength is provided in the tables, the strength varies from the top to bottom elevation of the layer
2. Drained soil parameters for the S-Case are:
   a. Clay – Cohesion = 0, $\phi^o = 23$
   b. Sand – same as Q-case
3. Calculation of soil pressure coefficients are as follows:
   a. At rest $K_0 = 1 - Sin\emptyset$
   b. Active $K_a = Tan^2\left(45 - \frac{\emptyset}{2}\right)$
   c. Passive $K_p = Tan^2\left(45 + \frac{\emptyset}{2}\right)$

6.1.3 Axial Pile Capacities

The pile capacity analyses follow the procedures in EM 1110-2-2906, Design of Pile Foundations, with additional guidance from Section 3.3 of the HSDRRS Guidelines June 2012). The factor of safety requirements from the HSDRRS Guidelines are shown below in Table 6-1.

Table 6-1 Factor of Safety Requirements for Pile Foundations

Soil Strength	*With Pile Load Test*	*w/o Pile Load Test*
Q-Case	2.0	3.0
S-Case	1.5	1.5

6.1.3.1 General Comments – Pile Foundations

1. The structural capacity of the pile to withstand the allowable axial and lateral loads and deflections are not part of our studies and must be determined by others.
2. The pile capacities presented are based upon pile/soil interaction and do not consider the structural aspects or the weight of the pile.

3. The estimated pile capacities presented are for piles driven vertically to a specific tip elevation. For batter piles, the vertical capacities will be equal to the geometric vertical component of the batter pile driven to the same tip elevation.
4. Minimum spacing between piles should be at least four (4) diameters or side dimensions, center-to-center.
5. Piles can be pre-augered to a maximum depth of 5 feet using an auger with a diameter no greater than the smallest side dimension or 80% of the diameter of the pile.
6. To determine driving characteristics, a few probe piles should be driven beneath the proposed structure, preferably in the vicinity of a boring. Probe piles will become working piles, and must be accurately located in accordance with the project's construction drawings.
7. All piles, including probe piles, should be driven under experienced supervision with efficiently operating mechanical equipment, and complete driving records should be maintained.
8. Settlement of the individual piles would not be expected to exceed ¾ inch under full working load.
9. Under conditions of short sustained seismic or wind loading, an allowable design overstress of 15% could be used.

6.1.3.2 Lateral Analysis

The soil profile data used in LPILE® was modeled after the most critical subsurface conditions encountered. Soil parameters inputted into the program were estimated, based on visual inspection of the soil samples in the laboratory, standard penetration test "N" values correlated with accepted geotechnical references, and our knowledge of the soils in the area. These soil strength parameters were factored for conservatism, therefore the lateral loads shown in the graphs would be considered allowable for the condition indicated.

6.1.3.3 Stability Analysis

Global slope stability analyses were conducted using SLOPE/W, Version 7.17, which employs finite element analysis as a design tool. Spencer's Method of Slices was the analysis method used in the program, which satisfies both force and moment equilibrium to derive the factor of safety against failure. The most critical soil properties for each section were used in the analyses.

For the Sedimentation Basin, an area load of approximately 300 psf was used to simulate the transient loading from truck traffic along the road and at the top of the berm. Similar loads were applied to the top of the embankments on either side of the Conveyance Channel to simulate light truck loading from periodic inspection. These loads were based on HS-40 truck loading acting over the area of the truck. In the case of the smaller vehicle, weight of typical pickup of about 6,500 lbs. was used over the area of the truck.

6.1.3.4 Sheet Pile Analysis

Sheet pile walls and TRS structures were analyzed using SPW 911 software as well as CWALSHT, a program supplied by the USACE.

7.1 CONSTRUCTION CONSIDERATIONS

7.1.1 Compaction Adjacent to Vertical Walls

Care should be exercised to brace vertical walls during placement of compacted fill behind the walls, especially when compacting clay soils. Compaction equipment should not be allowed too close to the walls such that large horizontal forces are applied. Fill placed within 18 inches of the wall should be compacted with a small hand tamper, using a lift thickness no greater than 6 inches.

7.1.2 Pile Load Test

We would recommend that a load test be conducted in accordance with ASTM D1143, *Standard Test Methods for Deep Foundations Under Static Axial Compressive Load*. If a number of piles will be used to counteract uplift (tension) loading, a tension load test should also be performed in accordance with ASTM D3689, *Standard Test Methods for Deep Foundations Under Static Axial Tensile Load*. The load tests should be conducted no sooner than 14 days after installation to insure that the piles have developed full capacity in the clay soils. For this project, monitoring the installation of the steel piles with a Pile Driving Analyzer (PDA) could be used in lieu of a full static load test. Monitoring pile installation with a Pile Driving Analyzer© should be used to prevent damage to the piles during installation.

7.1.3 Pile Drivability and Heave Potential

Pile drivability should not be an issue for H-piles, although the piles will be relatively long in order to meet the design loading requirements. Since they are non-displacement type piles, soil heave due to pile driving is not expected to be a concern. High displacement piles such as concrete may have the potential for ground heave which may cause damage to adjacent structures. The required length for concrete piles may also create other negative constructability concerns (splices, multiple pick-up points, transportation and handling issues, etc.).

Prior to pile installation, it is recommended that a drivability analysis be performed using the Wave Equation Analysis of Piles (WEAP) software program. The WEAP program uses the pile hammer system, pile type and size, and soil conditions evaluate the driving resistance needed to obtain pile capacity. The WEAP program also evaluates driving stresses in the pile that can be used to minimize risk of pile damage.

7.1.4 Pile Group Effects

Piles or shafts installed in groups will behave differently than individual foundations. To minimize these effects on compressive capacity, we have recommend a minimum spacing between foundations of four (4) diameters, center-to-center. With respect to the pile foundation layouts provided by the URS structural team, calculations indicate that with the spacing as recommended above group effects would be negligible. Any changes to the provided foundation layout should be re-evaluated for group effects using the HSDRRD Guidelines (June 2012) Section 3.3.6.

In computing lateral capacities of the group, a reduction factor of 0.9 should be used for the front row of foundations and a factor of 0.5 for all follow on rows. As with capacity, this is difficult to predict as the number of piles and their arrangement in the group can have a major effect on the lateral behavior.

7.1.5 Excavation and Trenching

Excavations at this site will require careful preparation as the water tables are near or at grade and seepage into an excavation through soils should be expected. Excavations, even relatively shallow in depth, will be subject to sloughing and constant intrusion of water. In accordance with Appendix A of 29 CFR 1926, Subpart P, soils at this site could be considered as Type C. All relevant safety precautions should be adhered to when working in excavations and dewatering should be anticipated.

Care should be exercised during shored excavations to reduce the potential for excess hydrostatic pressure to build up behind the sheet piling in the event of a heavy rainfall. Any cracks that form between the soils and sheet-pile wall and in the soil itself should be backfilled to reduce infiltration of water behind the sheet piling.

As much as practicable, construction activities should be planned such that excavations are exposed for the least possible amount of time.

7.1.6 Drainage and Dewatering

Prior to construction, a Grading and Drainage Plan should be developed by the contractor. In the initial stages of site development, effective drainage must be established and modified as necessary during construction. In areas where it will be necessary to excavate weaker soils and replace this material with compacted backfill, control of moisture and drainage is vital.

Seepage of water into the excavations can be handled by a system of sumps and pumps. A low-permeability cutoff wall may also be required around some or all of excavations if horizontal groundwater inflow exceeds the capacity of the dewatering sumps. The groundwater level should be re-confirmed prior to excavation. The actual construction dewatering techniques to be implemented shall be determined by the site contractor.

7.1.7 Foundation Bedding

For improving the stability of the subgrade under the concrete foundation elements, it is recommended that a 12-inch layer of compacted aggregate be established beneath these structures prior to construction. This material should meet the requirements of Section 7.1.9.3, and be compacted in place as much as practicable. Preparation of the subgrade in short sections may be more practical, as it will provide a stable working surface and minimize the need for dewatering.

7.1.8 Seepage

Steel sheet-pile cutoff walls will be required beneath the earthen cofferdam and inlet structure systems to provide protection against seepage under the flood protection. Cutoff walls should be driven to a minimum tip elevation of -35.0 to terminate below the silty and sandy stratum underlying the site. Where conduits penetrate embankments, drainage filters are recommended to prevent piping erosion.

7.1.9 Backfill and Fill

7.1.9.1 Structural

Fill placed on the project should be select material free from organics and other deleterious debris and be classified as CL or CH according to the Unified Soil Classification System (USCS). Backfill and fill material should have less than 35% of the soil particles (by weight) retained on the No. 200 mesh sieve and have a Plasticity Index of at least 9. Soils excavated, if free of debris, organics and excessive moisture could be used as backfill and fill. Materials excavated north of US 61 have a high organic content and would not be considered suitable for use.

It should be placed in thin successive layers 8" to 10" loose measurement and each layer should be compacted to at least 90% of its maximum laboratory dry density, within ±2% of its optimum moisture content, in accordance with ASTM D698 (Standard Proctor). In-place field density tests should be performed as this material is being placed and compacted in order to insure that required density is being achieved.

7.1.9.2 Non-Structural

For areas not be designed to support any load and where settlement is not of concern, non-structural backfill and fill can be any clayey soil with a Liquid Limit less than 40. It should be placed in 10" layers loose measurement and compacted as much as practicable.

7.1.9.3 Aggregate

Aggregate material should be a stone or crushed concrete material meeting the requirements of Section 1003 of the Louisiana Standard Specification for Roads and Bridges.

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9.1 COMPUTER PROGRAMS

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