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Executive Summary

Introduction

The current authorized plan to increase the level of hurricane protection in the Westbank and vicinity area of New Orleans along the Harvey and Algiers Canals to the 1% annual exceedance probability storm is a parallel protection system of projects. In an earlier study for the USACE by URS Corporation dated April 2007, several alternatives to the parallel protection were evaluated. The URS report recommended feasible alternatives that would provide the level of protection sooner and at a lesser cost than the parallel protection. Following the URS study, the USACE in conjunction with the ARCADIS Bioengineering Joint Venture team identified several additional alternatives to consider for further study. During the development of report, several alternatives were both added and removed from consideration. In the end only three alternatives were selected by the USACE to be developed to completion in this detailed study and to be compared to the authorized parallel protection system.

The purpose of this project was to conduct further analyses, designs, and layouts; to develop schedules and cost estimates of selected alternatives; and to provide a comparative analysis of the cost and schedule estimates of the design alternatives to the parallel protection system. This Engineering Alternatives Report (EAR) addresses four closures to provide the required 100-Year level of protection (LOP), the authorized parallel protection system of projects as one alternative and the three selected alternatives being designed under this project. This report presents the design results of the three alternatives and a schedule and cost comparison to the authorized parallel protection system projects being developed by others.

Existing Hurricane Storm Surge Protection System

Existing protection for the project area consists of levees and floodwalls adjacent to and on the banks of the Harvey Canal, the Algiers Canal and the Gulf Intracoastal Waterway (GIWW). This existing flood protection is approximately elevation (El.) 9.5, but in some industrialized areas along the canal, the flood protection is much lower than El. 9.5. Protected side areas are drained by a system of canals and pumping stations of various capacities.

Alternatives Evaluated in this Study

The following provides a description of the alternatives evaluated in this report. Plates of the final study alternatives are provided in this report as Appendix G, Final Study Alternatives (Alt. 2D, 2E, 3B). The relative location of the alternatives is shown on Plate C-01, Overall Site Plan in Appendix G to this report.

Alternative 1 Parallel Protection

Raise the parallel protection over a large area of the Westbank along both Harvey and Algiers Canals. This alternative is referred to in this report as the Parallel Protection Alternative 1. The Alternative 1 includes ten projects that are listed in Section 4.2.1 and shown on Plate C-01, Overall Site Plan.

Alternatives 2D and 2E GIWW South Gates

Construct a 225-foot wide gate closure in the GIWW , a 25,000 cubic foot per second (cfs) pumping station on the east side of the GIWW to be used during hurricane events and a 110-foot wide velocity reduction channel and sector gate on the east side of the GIWW. A T-wall would be constructed from the west side of the gate and along the west bank of the GIWW. The navigation gate and pumping station structures are located downstream or south of the confluences of Algiers and Harvey Canals, and upstream of the Hero Canal. Alternative 2D would have a 225-foot wide pocket gate and a secondary 110-foot wide sector gate. Alternative 2E would have a 225-foot wide sector gate and a secondary 110-foot wide sector gate. The pumping station will be used only during hurricane events.

Alternative 3B Algiers Gate

Constructing a 225-foot sector gate in the Algiers Canal that would close the Algiers Canal during a hurricane event, and a 16,000 cfs pumping station on the east side of the canal. The structures are located upstream of the confluence of Algiers and Harvey Canals. The pumping station will be used only during hurricane events.

Design Criteria and Data

The design criteria for the structures and systems studied and reported herein can be basically described as providing for hurricane storm surge protection which has a 1% annual exceedance probability within a 90% confidence limit. This is defined for the Westbank vicinity as a still water surge elevation of El. 11.0, plus wave run-up, plus freeboard, and handling the interior drainage associated with the storm.

The detailed design criteria includes: hydraulic, civil and geotechnical, structural, electrical, and mechanical for USACE designed civil works structures and systems. Data was provided by the USACE for soils and existing facilities.

Earthwork Requirements

To the extent possible the completed facilities can be constructed by balancing excavations and fills. Temporary retaining structures (TRS) or cofferdams may require importing significant sand depending upon method selected.

Real Estate – Right of Way Requirements

Depending upon which alternative is selected, as much as 161.6 acres of Perpetual Flood Protection Servitude Easement, 15.9 acres of Required Perpetual Road Easement, 4.5 acres of Pile Easement, and 11.3 acres of Temporary Staging Area Easement are required for construction. Alternative 2D requires the more servitude and easement area than Alternatives 2E or 3B.

Relocations

GIWW South Alternatives 2E and 2D require 6,443 feet of Bayou Road realignment and relocation. For the Algiers Alternative 3B, 4,451 feet of Bayou Road will require realignment and relocation.

Facilities requiring relocation include 10-inch buried natural gas pipelines for Alternatives 2D and 2E, electric distribution lines, transmission lines and towers, a shooting range, and residences. GIWW Alternative 2E and 2D affect the 995 feet and 1,882 feet respectively of a 10-inch diameter natural gas pipeline. Algiers Alternative 3B will not affect this natural gas pipeline. Alternatives 2E and 2D will be less disruptive than 3B on the other

utilities and structures, only affecting, 3,299 feet of electric distribution line, and 2 acres of a shooting range. Algiers Alternative 3B will affect only 2,669 feet of electric distribution lines, but will affect 2 utility towers and associated 3,515 feet of overhead electric distribution lines and will relocate nine houses and four trailers.

Cost Estimating and Construction Scheduling

The cost estimates do not include any right of way or relocation costs. These construction duration estimates do not include the fast track capabilities associated with the Early Contractor Involvement (ECI) acquisition method that the PDT is expected to utilize during P&S phase. For the selected alternative, further refinement of the construction schedule and sequencing by the PDT in cooperation with the ECI Contractor is anticipated to provide that interim protection will be completed by June 2011.

The cost estimates and completion dates for Alternatives 1, 2D, 2E, and 3B are as summarized follows:

Alternative	Total Construction Cost	Interim Date	Completion Date
• Atl. 1	$962.0M	September 27, 2011	September 27, 2011
• Alt. 2D	$681.3M	March 16,2012	October 5, 2012
• Alt. 2E	$665.2M	June 8, 2012	January 21, 2014
• Alt. 3B	$582.2M	September 15, 2012	May 26, 2014

Quality Implementation

This study and report were produced utilizing the ARCADIS Bioengineering Joint Venture and HNTB Corporation quality control procedures. Independent technical reviews were conducted on the 65% Submittal, the Modification 65% Submittal, and at the 95% Submittal.

Pros and Cons for the Alternatives

- Alternative 1 parallel protection system has the highest initial capital cost, lowest O&M cost, and shortest construction duration. A significant number of businesses and home will be affected. Alternative 1 has the highest risk of levee failure due to the longest length of the first line protection levees that are exposed.

- Alternative 2D with the sliding pocket gate has a reduced capital cost, higher O&M cost, long construction duration, and is an untested gate type for the area. Alternative 2D and 2E levees pose the least risk of failure than as they have the least length of levee exposed.

- Alternative 2E with the sector gates has a reduced capital cost, higher O&M cost, the longest construction duration, but the lowest risk of future failure. Alternative 2D and 2E levees pose the least risk of failure than as they have the least length of levee exposed.

- Alternative 3B with the sector gate has the lowest capital cost, slightly lower O&M cost than 2E, and the same construction duration as 2E. Alternative 3B poses a greater risk of failure than 2D and 2E but less risk than Alternative 1 due to the intermediate length of first line of defense levees along Harvey Canal that are open to vulnerability and failure.

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1. Introduction

This report summarizes results of a study conducted by the ARCADIS Bioengineering Joint Venture team under Contract Number W912P8-07-D-0062, Task Order 3 with the US Army Corps of Engineers (USACE), New Orleans District (MVN).

The study title is:

Westbank and Vicinity, New Orleans, Louisiana, Hurricane Protection Project
Westwego to Harvey, West of Algiers Canal, East of Algiers Canal
Sector Gate South – Detailed Alternatives Study
Jefferson, Orleans and Plaquemines Parishes, Louisiana

The USACE has two primary goals for hurricane protection for the Westbank and vicinity area: to provide the original authorized level of protection (LOP) in the Westbank and vicinity area of New Orleans along the Harvey and Algiers Canals as soon as practical, and to provide recently authorized 100-year (1% annual exceedance probability) LOP by June 2011 or as soon as practical.

The current authorized plan to provide the 100-year LOP, herein described as Parallel Protection (Alternative 1) is to raise the flood protection system (including floodwalls, gates, levees, pumping station fronting protection, etc.) along both the Algiers Canal and the southern portion of the Harvey Canal below the new sector gate at Lapalco Boulevard.

The USACE evaluated an earlier preliminary study report conducted by URS Corporation dated April 2007 that describes alternatives to the current authorized plan to provide the 100-year LOP. The URS report recommends feasible alternatives to provide the level of protection sooner and at a lesser cost than the authorized parallel protection plan.

Following the URS study, the USACE identified several additional alternatives to consider for this subsequent Engineering Report (EAR) project. During the development of this detailed report, the USACE added and removed several alternatives from consideration in the study. In the end, only three alternatives were selected by the USACE to be

developed to completion and to be compared to the parallel protection system (Alternative 1) in this report.

In total, this study report considers four different alternatives that provide the 100-year LOP, three selected alternatives were designed under this project, and the Parallel Protection system alternative is being developed by others under other ten projects. This report presents the study results of the three selected alternatives and provides a schedule and cost comparison to the authorized Parallel Protection system of projects.

2. Purpose and Scope of Study

The purpose of the study is to provide a definitive basis for comparison of the construction costs and construction durations of the three selected design alternatives to the authorized Parallel Protection alternative that is to provide hurricane protection on the Westbank area of the Mississippi River in the parishes of Jefferson, Orleans, and Plaquemines. The work for this study included conducting further analyses, and developing designs and layouts for selected alternatives identified by the USACE after analysis of the URS study dated April 2007.

This study also includes a comparative analysis of the selected alternatives with the authorized Parallel Protection system to determine the most economical alternative to obtain the desired 100-year LOP, and to determine the ability to achieve the desired protection by June 2011 or sooner. For this analysis, schedules and cost estimates for the selected alternatives were developed based on the designs developed for this project. For the authorized Parallel Protection system, schedules and cost estimates were developed based on projects being developed and designed by others.

Seven alternatives that were under design were eliminated from the study by two scope modifications. Only three selected alternatives continue to be considered as alternatives and were carried forward in order to compare the costs and durations to the authorized Parallel Protection Alternative 1 solution.

The Alternative 1 Parallel Protection is being designed by others under ten projects. At the time of this study the Alternative 1 projects were at different stages of design. The USACE provided Alternative 1 project data and assumptions needed to prepare the cost and construction duration estimate included in this report.

This report presents the essential data, assumptions, computations and criteria used in the design of sector gates, pumping stations and their associated structures. The purpose of this report is to provide an adequate basis for future preparation of construction plans, specifications, cost estimates and schedules.

2.1 Brainstorming Session and Innovation Study

In November 2007, three alternatives were being considered to provide the authorized level of protection (LOP) in the Westbank and vicinity area, Alternative 1 Parallel Protection System, Alternative 2 GIWW South Sector Gate, and Alternative 3 Algiers Sector Gate. A brainstorming session was held between the team and the USACE to determine if a fourth alternative should be considered that had not yet been identified by the USACE. The results of the brainstorming session did not identify a fourth alternative to study, but did result in recommendations to evaluate several modifications to the existing study alternatives.

The brainstorming session recommendations included:

- Using alternative pump configurations including vertical, flowerpot, or bow thruster type pumps, and alternative construction methods including a float-in construction alternative of the pumping station.
- Using alternative gate types and sizes. Alternative gate types evaluated include a "Dutch Sector Gate" and a sliding/rolling pocket gate.
- Revised locations of the sector gate, pumping stations, and bypass channel.
- Alternative floodwall tie-ins to the environmentally sensitive Clean Water Act Section 404(c) wetlands area.

The results of brainstorming session were presented at a 17 December 2007 meeting at the USACE New Orleans District Office. This Brainstorming Study report is incorporated into this report as Appendix K.

As a result of the Brainstorming Session, the USACE began a separate innovation study project that addressed different gate openings, gate types, pump types, and alternative construction methodologies for hurricane protection for the Westbank and vicinity area. The Innovation Study was conducted concurrent with the EAR project, and it produced designs for gates and pumping stations that were at a level of design between a reconnaissance and a feasibility study and estimated at 25%. This EAR was modified to incorporate some of the Innovation Study. The results of the Innovation Study are summarized in the "Westbank and Vicinity, New Orleans, Louisiana, Hurricane Protection,

Sector Gate South Innovation Study - Final Report" dated August 2008, incorporated herein by reference and noted as "Innovation Study Report."

2.2 Modifications to Study Alternatives

The initial the scope of work included one gate design and associated pumping stations for each general closure site, one on the GIWW and one on the Algiers Canal. The GIWW south site located on the GIWW to the south of the confluence of the Harvey and Algiers Canals. The Algiers closure site is located on the Algiers Canal north of the confluence of the Harvey and Algiers Canals. Alternatives added to the project were all located in one of these two general locations. An alternative pumping station location to supplement the station adjacent to the closure gate was located adjacent to the Algiers / Mississippi River lock.

The scope of work and was modified three times to both add and to eliminate several alternatives. At one point, all the current study alternatives under design were eliminated and replaced with three new alternatives that were derived and refined from previous alternatives. There were several ongoing studies, discussions and investigations that affected the study. There were many issues driving the addition and elimination of the design alternatives:

- Incorporated ideas from the Brainstorming Session and Innovation Study including different gate types and openings, different pump types, and alternative construction methodologies,
- Improved navigation by revising alignments and eliminating temporary bypass channel at the Algiers closure,
- Navigation concerns of the floodwall under design adjacent to the navigation channel,

- Concerns of the National Park Service and the Environmental Protection Agency about construction impacts and the proposed floodwall across an area classified as a Section 404(c) wetlands area under the Clean Waters Act. To minimize impacts, an at-grade service road vs. a road on top of the floodwall was studied. Also an alternate location of the floodwall closure was studied to locate it along but not traversing the 404(c) wetland area.

- Changed pumping station capacities due to ongoing USACE hydraulic studies and analysis, and

- Revised the relative layout of pumping station and gates to shorten construction durations,,

The final scope modification eliminated all alternatives that were under design and added three new design alternatives derived from the previous alternatives. Only these final Alternatives 2D, 2E and 3B were completed and compared to the authorized parallel protection Alternative 1. The following table lists the alternatives eliminated during the development of this report.

Table 2.2: Eliminated Study Alternatives

Alternative	Description Summary	Reason Eliminated
GIWW South 2	Pumping station across GIWW, relocated navigation channel through sector gate, tainter gate with secondary channel, and floodwall across Section 404(C). Sub alternative for supplemental pumping station adjacent to Algiers / Mississippi River lock.	•Unnamed sub-alternative for supplemental pumping station adjacent to Algiers / Mississippi River lock was eliminated very early in the project due to USACE hydraulic analysis. •Alt. 2: T-wall crossing the Section 404(c) wetlands area replaced with preferred 2D along but not crossing 404(c) boundary.
GIWW South 2A, 2A.1	Pumping station partially in GIWW but shifted for more construction in dry, relocated navigation channel through sector gate, tainter gate with secondary channel, and floodwall across Section 404(C). At-grade access road across Section 404(c) wetland area and sub alternative 2A.1 for elevated road access road on Section 404(c) floodwall.	•T-wall crossing the Section 404(c) wetlands area replaced with preferred 2D along but not crossing 404(c) boundary.
GIWW South 2B, 2B.1	Sector gate partially in GIWW for more construction in dry, relocated navigation channel, pumping station to east, and floodwall across Section 404(C). At-grade access road across Section 404(c) wetland area and sub alternative 2B.1 for elevated road access road on Section 404(c) floodwall.	•T-wall crossing the Section 404(c) wetlands area replaced with preferred 2D along but not crossing 404(c) boundary.
GIWW South 2C, 2C.1	Both pumping station and sector gate shifted further east for more construction in dry, sector gate to east of pumping station, relocated navigation channel and floodwall across Section 404(c). At grade access road across Section 404(c) wetland area and sub alternative 2C.1 providing elevated access road on Section 404(c) floodwall.	•Expected to have large and deep excavation proximate to the Mississippi River levees. Cost and reliability/dependability of large fill area across channel.
Algiers 3	Sector gate across Algiers canal, pumping station to east, temporary relocated navigation channel to bypass Algiers Canal	•Temporary bypass replaced with permanent bypass alignment preferred for navigation.
Algiers 3A	Pumping station across Algiers canal, sector gate to east, permanently relocated navigation channel of Algiers Canal	•Lengthy construction durations with continuous navigation. Construction of the pumping station and sector gates construction could not be concurrent.

The designs of the eliminated alternatives were at various levels of completeness when eliminated. It should be noted that the structural, mechanical, electrical and civil designs of the eliminated alternative were at various stages of design when halted and their plates, designs, costs and construction duration cannot be compared directly the analysis results of the final three selected alternatives. The design and analysis data for the eliminated alternatives may be found in the following appendices:

Appendix L	Eliminated Alternatives Status Design Calculations
Appendix M	Detailed Quantity and Cost Estimates, Eliminated Alternatives
Appendix N	Construction Durations, Eliminated Alternatives
Appendix O	Plates of Eliminated Alternatives
App. B, Sub-Appendix A	Geotechnical Report

2.3 Objectives

The objective is to prepare designs, calculations, cost estimates, construction duration estimates and analysis of alternatives that support the feasibility of each alternative studied. The USACE will determine the preferred alternative and move forward into final Plans and Specifications (P&S) if the current Parallel Protection System solution is not selected.

This report includes the analysis and designs for only the three remaining design alternatives being considered, Alternative 2D GIWW South sliding gate, Alternative 2E GIWW South sector gate and Alternative 3B Algiers sector gate, and includes a cost and construction duration estimate comparing the three remaining alternatives to the Alternative 1 designs by others. The civil designs, construction estimates and durations for Alternatives 2D, 2E and 3B were prepared specifically for this report. Appendix G contains the plates for the selected alternatives, including the plate C-01, Overall Site Plan.

This report includes comparisons of the Pros and Cons of the alternatives based on the EAR and Innovation Study designs, costs, construction durations, and other engineering considerations.

All elevations used in this report are referenced to North American Vertical Datum 88 (Epoch 2004.65) (NAVD88). This report is based upon the estimated one percent (1%) exceedence still water level (SWL) elevation (El.) 11.0 and preliminary protected side allowable water surface elevations provided by the USACE at the onset of this study. The USACE will assess the need to re-evaluate the results presented here upon issuance of their final interior drainage study hydraulic report.

2.4 Level of Detail

The level of detail for this report supports the cost estimates within a 25% general contingency. Calculations and drawings included in this report support this level of detail for the cost estimates and are developed approximately to the 35% design level.

The designs completed under this EAR project were completed to a 35% level. The mechanical, electrical and structural designs of the navigation gates and pumping stations completed under the Sector Gate South Innovation Study were completed to a 25% level.

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3. Description of Existing Protection

3.1 Project Location

This project is located on the Westbank and vicinity area of the Mississippi River in the parishes of Jefferson, Orleans, and Plaquemines. The proposed study area is located mainly along Harvey Canal, Algiers Canal and the GIWW. The project's components affect three of the Westbank Vicinity (WBV) Hurricane Protection System project areas: Westwego - Harvey, Gretna - Algiers, and Belle Chasse. The northern limits of this project are the Cousins Pump Station on the Harvey Canal and the Algiers /Mississippi River Lock. The southern limit is south of the confluence of the Harvey and Algiers Canals on the GIWW, and north of the Hero Canal.

3.2 Type and Level of Protection

Existing protection for the project area consists of levees and floodwalls adjacent to and on the banks of the Harvey Canal, Algiers Canal, and the GIWW. This existing flood protection is approximately El. 9.5., but in some industrialized areas along the Harvey Canal, the flood protection is much lower than El. 9.5. Protected side areas are drained by a system of canals and pumping stations. The objective of this project is to provide alternative designs that raise the LOP to the 1% "chance" annual exceedence level designated for the project area. This equates to a still water level El. 11.0 and would mean raising the protection, including structural superiority, between El. 14.0 and El. 16.0, depending upon structure type, and cutting off potential storm surge through the Algiers and Harvey Canals.

3.3 Limits of Right of Way

The limits of the existing and proposed right of way (ROW) servitudes are shown in the ROW plates provided in Appendix G to this report. Existing ROW information was collected from as-built drawing files, MicroStation files and site information provided by the USACE. For the GIWW, the USACE directed to assume the existing right of way line to be the bank and tree line along the Section 404(c) wetlands area. This line was determined utilizing aerial photograph files and LIDAR surfaces provided by the USACE. During P&S, additional survey data will allow for refinement of this existing ROW line between the Section 404(c) wetlands area and the GIWW.

The proposed perpetual flood protection servitude easements were calculated for Operations and Maintenance (O&M) by developing the construction limits of the proposed alignments and then adding a 25-foot vegetation-free zone between the perimeter and the levee toe, edge of the floodwall, or grading limits. P&S Levee designs will include tree removal, sloping, grading, placing fill, etc. as necessary to achieve a maintainable 15-foot vegetation free zone from the toe of the levee on both the flood and protected sides. All P&S for Hurricane and Storm Damage Risk Reduction System (HSDRRS) levee contracts will ensure standards are met with respect to maintenance corridors.

The proposed ROW design contains all proposed temporary and perpetual servitudes and easements for the final three selected alternatives. ROW design plates for Alternative 1 projects are not included in this report.

4. Description of Proposed Alternative Protection Systems

This Detailed Alternatives Study Report considers the following locations and alternatives.

4.1 Location of Alternatives

The study area for this EAR is in the Westbank area of the Mississippi River in the parishes of Jefferson, Orleans, and Plaquemines. The southern limit of the study area of this project is the confluence of the Algiers and Harvey Canals on the Gulf Intracoastal Waterway (GIWW). The northern limits of the study area are the Cousins Pumping Station on Harvey Canal and the Algiers /Mississippi River lock on the Algiers Canal.

The general locations of the four alternatives that are addressed by this EAR report, and the parallel protection system projects of Alternative 1 that are currently being performed by others are shown on Plate C-01, Overall Site Plan in Appendix G of this report. The USACE project numbers of the ten projects of Alternative 1 are listed on the right side of the plate. Detailed plans of the three design alternatives are included in Appendix G.

4.1.1 Location of Alternative 1 Parallel Protection Projects

This is the current authorized protection system under design to be raised to meet the new LOP. The design work by others consists of ten projects including raising a system of levees and floodwalls adjacent and parallel to the existing Harvey and Algiers Canals, and providing fronting protection at nine existing pumping stations.

4.1.2 Location of Alternatives 2D and 2E GIWW South Gate

Alternatives 2D and 2E are located on the GIWW just south of the confluence of the Harvey and Algiers Canals. The township/range/sections for these alternatives are T14S/R11E/47, T14S/R24E/47, T15S/R11E/47 and T15S/R24E/47. These alternatives consist of construction of a new 225-foot navigation gates and a 25,000 cubic foot per second (cfs) pumping station on the GIWW south of the confluence of the Harvey and Algiers Canals. In addition, these alternatives also considered realignment of the GIWW channel to create a secondary GIWW channel adjacent to the existing, construction of a

floodwall along the Section 404(c) wetlands area to tie into the Westwego to Harvey levee system at the end of the Estelle discharge canal, and considered a system of parallel protection constructed to interior drainage 2007 1% annual exceedence event elevation levels to provide a retention basin on the protected side of the new structure. These alternatives would permanently realign the GIWW channel through a 225-foot navigation gate structure and create a secondary channel alignment through a 110-foot sector gate. The GIWW South pumping station includes vertical type pumping station designed for a total capacity of 25,000 cfs. The floodwall paralleling Section 404(c) wetlands area is constructed such that no more than a 100-foot width of Section 404(c) wetlands area is impacted by the construction.

Alternative 2D involves construction of a 225-foot sliding pocket gate in conjunction with an 110-foot sector gate and adjacent GIWW 25,000 cfs pumping station.

Alternative 2E involves construction of a 225-foot sector gate in conjunction with a 110-foot sector gate and adjacent 25,000 cfs pumping station.

4.1.3 Location of Alternative 3B Algiers Sector Gate

Alternative 3B is located on the Algiers Canal just north of its confluence of the GIWW and Harvey Canal. The township/range/sections for this alternative are T14S/R11E/47 and T15S/R11E/47. This alternative consists of construction of a new navigation gate and a pumping station in the Algiers Canal north of the confluence of the Harvey Canal. The alternative also considered a floodwall tying into the existing levee and floodwall alignments along the Harvey Canal extending from the Algiers Sector Gate location to the southern project limit and considered a permanent bypass channel for navigation.

Alternative 3B involves construction of a 225-foot sector gate structure and an adjacent 16,000 cfs pumping station. This alternative maintains the existing Algiers Canal channel alignment through the new sector gate structure. The pumping station includes vertical type pumps with a total capacity of 16,000 cfs.

4.2 Details of Alternatives

The following description provides the details of the four alternatives to provide the required 100-Year LOP, including the parallel protection system and the three selected design alternatives that were studied to provide a schedule and cost comparison to parallel protection system.

4.2.1 Alternative 1 Parallel Protection Projects Details

Alternative 1 Parallel Protection consists of raising the present levee and floodwall system adjacent to and parallel to both the Harvey and Algiers Canals up to the 100-year (1% annual exceedence probability) level of protection and providing fronting protection at nine existing pumping stations. The protected elevation is being designed by others for a SWL El. 11.0, with provisions for waves and freeboard

The ten projects in the Alternative 1 Parallel Protection are defined as:

- WBV-14h.2 V - Line Levee South,
- WBV-14.g2 Old Estelle to New Estelle,
- WBV-14a.2 Harvey Canal West Bank Levees,
- WBV-39b.2 Cousins Discharge Canal Floodwalls,
- WBV-46.2 Lapalco Sector Gate,
- WBV-6a.2 Belle Chasse Hwy. to Hero Cutoff (West),
- WBV-47.2 Algiers Lock to Belle Chasse Highway (West),
- WBV-48.2 Belle Chasse to Algiers Lock (East),
- - Belle Chasse Tunnel, and
- WBV-49.2 Hero Levee to Belle Chasse Highway (East).

Designs, quantity estimates and schedules for these Alternative 1 projects are being developed by others. The USACE provided this project team with the preliminary and final quantity estimates and schedules, or provided the necessary criteria to develop the construction duration and cost estimates for Alternative 1. The cost estimates for Alternative 1 and the final three selected alternatives are included in Appendix E of this report.

4.2.2 Alternative 2D GIWW South Gate Details

Alternative 2D consists of the construction of a 225-foot sliding pocket gate, a 110-foot sector gate, and a 25,000 cfs pumping station at the GIWW south sector gate location, the southern most location of the alternatives. The main features of Alternative 2D are:

- 225-foot sliding pocket gate,
- 110-foot sector gate,
- Pumping station with 25,000 cfs discharge capacity, to be used only during hurricane events,
- Secondary GIWW channel,
- Flow separator wall between the 110-foot sector gate and pumping station,
- Temporary retaining systems (TRS) and dewatering system,
- Floodwall paralleling Section 404(c) wetlands area. This wall is to be constructed such that no more than a 100-foot width of Section 404(c) wetlands area is impacted by the construction of the floodwall. A protective berm is to be constructed on the protected side of the floodwall, the GIWW channel side constructed with top of wall at El. 8.0. This berm will protect the wall from barge impact and provide concrete scour protection and a maintenance access road to the west side of the 225-foot pocket gate,
- T-walls between the GIWW channel and Section 404(c) wetlands area, between both proposed navigation gates, and between proposed pumping station and existing GIWW (East) levee system,
- Realigned existing GIWW Canal (East) levee system and realigned East Bayou Road, and
- Filling the Old Estelle discharge canal to tie-in to the levee system on the west side of the channel.

4.2.3 Alternative 2E GIWW South Gate Details

Alternative 2E consists of the construction of a 225-foot sector gate, a 110-foot sector gate, and a 25,000 cfs pumping station at the GIWW south sector gate location. The main features of this Alternative 2E are:

- 225-foot sector gate,
- 110-foot sector gate,
- Pumping station with 25,000 cfs discharge capacity, to be used only during hurricane events,
- Secondary GIWW Channel,
- Floodwall paralleling Section 404(c) wetlands area. This wall is constructed such that no more than a 100-foot width of Section 404(c) wetlands area is impacted by the construction of the floodwall. A protective berm is to be constructed protected side of the floodwall, the GIWW channel side constructed with top of the wall at El. 8.0. This berm will protect the wall from barge impact and provide concrete scour protection and a maintenance access road to the west side of 225-foot sector gate,
- Temporary retaining systems (TRS) and dewatering system,
- T-walls between the GIWW Channel and Section 404(c) wetlands area, between both proposed navigation gates, and between proposed pumping station and existing Algiers Canal (East) levee system,
- Realigned existing Algiers Canal (East) levee system and realigned East Bayou Road, and
- Filling the Old Estelle discharge canal to tie-in to the levee system on the west side of the channel.

4.2.4 Alternative 3B Algiers South Gate Details

Alternative 3B consists of the construction a 225-foot sector gate structure and an adjacent 16,000 cfs pumping station located in the Algiers Canal north of the confluence of Harvey and Algiers Canals. The main features of Alternative 3B are:

- 225-foot sector gate,
- Pumping station with 16,000 cfs discharge capacity, to be used only during hurricane events,
- Realigned Algiers canal with levees,
- Temporary retaining systems (TRS) and dewatering system,
- T-walls across the Algiers Canal between the existing Algiers Canal (West) levee system and the proposed 225-foot sector gate, and between the proposed pumping station and existing Algiers Canal (East) levee system, and
- Realigned existing Algiers Canal (East) levee system and realigned East Bayou Road.

4.3 Types of Protection

All new project features and existing systems detailed in each alternative are designed for a 1% annual exceedence storm event The main components of the alternatives outlined in the report include sector and sliding pocket navigation gates, pumping stations, flow separators, earthen levees, bypass channels, and concrete T-walls. These components are described below. In addition the gate openings and flow separators were designed for non-hurricane floods and navigation.

4.3.1 Gate Structures

Initially the proposed sector gates for this project were configured for a 150-foot wide opening. However, due to USACE revised hydraulic modeling and navigation considerations, larger and multiple gates options are being proposed.

4.3.1.1 Sector Gate Structures

The top of the sector gate structures was designed to El. 16.0 to provide for the 100-year protection including structural superiority. The top of the base slab or sill will be El. -16.0 as required by deep draft navigation requirements. The gate structures will be comprised of cast-in-place reinforced concrete (CIPC) gate bay walls and pile supported CIPC base slab. The framework for the sector gate will consist of rolled wide flanged steel sections for the 110-foot gate and a combination of hollow pipe sections and rolled wide flanged steel sections for the 225-foot gate. The steel sections make up three vertical trusses and three horizontal frames for the 110-foot gate. The 225-foot gate is comprised of four vertical steel trusses and three horizontal frames. The sector gate structures will be founded on steel pipe piles with a grid-type layout. T-wall tie-ins will connect the sector gate structures with the pumping stations and flow control structures as necessary. These T-wall sections will be CIPC stems and footings founded on steel H-piles.

4.3.1.2 Sliding Pocket Gate Structures

The 225-foot sliding pocket gate requires that a floating sliding gate be designed to sit in a recessed pocket off of the GIWW to the east of the navigation channel. The gate is required to float across a 225-foot wide opening within the 400-foot wide channel and be sunk into place. The recessed pocket is to be designed to allow maintenance in dry conditions. The steel-framed gate is 50-foot wide by 36-foot high by 258-foot long. The gate sits in a 282-foot long recessed concrete pocket. During construction, the gate will be ballasted with concrete and floated into place within the pocket.

4.3.2 Pumping Stations

Pumping stations will be CIPC substructures with a steel moment frame supporting precast concrete skin panels. The pumping station operating floor will be CIPC. The entire structure will be founded on reinforced concrete piles. The pumps will be vertical column pump propeller units with speed reducers and diesel drive motors, capable of pumping an average discharge rate of approximately 1566 cfs over the operating range of head. The pumps will be controlled and supported by auxiliary equipment including: engine radiator systems, compressed air system, fuel, waste oil, water supply, trash rakes

and racks, cranes, drains and unwatering, sewage, HVAC, fire protection, controls, lights, and emergency power.

4.3.3 Floodwalls

The floodwall paralleling Section 404(c) wetlands area is designed such that no more than a 100-foot wide section of Section 404(c) wetlands area, measured from the existing right of way line is impacted by the construction. A protective berm will be placed on the protected side, the GIWW channel side of the wall up to El. 8.0 to protect the wall from barge impact. The T-walls tying in between the sector gates, pumping stations, and levees are incorporated from the previous design used in an earlier phase of this project.

4.3.4 Flow Separator

The flow separator between the sector gates and the pumping stations will be either concrete walls founded on the channel bed at El. -16 or will be an earthen embankment. The flow separator will extend upstream and downstream from the gate structure.

4.3.5 Temporary Retaining Systems (TRS)

Temporary restraining and dewatering systems are detailed in the Geotechnical Report in Appendix B to this report. There were four systems evaluated; a circular cellular cofferdam, a braced double wall internally earth filled cofferdam, an A-frame braced single wall cofferdam, and an internally braced single wall cofferdam. The internally braced, A-frame braced and circular cellular cofferdams were designed and their costs and schedules were estimated. The two construction phase internally braced single wall cofferdam was ultimately chosen for its ability to meet the navigation and schedule requirements. Design details of the single wall systems are included in the structural appendix. During preliminary analysis of the braced double wall system, it was eliminated based on the geotechnical analysis. It was determined that there would be no real cost or schedule savings beyond the other three systems. However, during bidding of the work, all four systems should be provided to the Contractor as options for consideration. The required temporary retaining structures in the canal will be built to El. 11, the 2011 1% elevation.

4.4 Channel Alignments

Channel alignments for each alternative were provided by the USACE and are detailed in the plates provided in Appendix G to this report.

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5. Design Data and Criteria

5.1 Assumptions

The design criteria for the structures and systems studied and reported herein can be basically described as providing hurricane storm surge protection which has a 1% annual exceedance probability within a 90% confidence limit. This is defined for the Westbank area as a still water surge elevation of El. 11.0, plus wave run-up, plus freeboard, and handling the interior drainage associated with the storm.

The detailed design criteria includes: survey, hydraulic, civil and geotechnical, structural, electrical, and mechanical for USACE designed civil works structures and systems. Data was provided by the USACE for soils and existing facilities. Assumptions made for each discipline are located in the design criteria narratives for each section, as well as the project feature narratives of this report.

5.2 Field Data Collection

Field Data collected for this project included site reconnaissance and a site survey. Both items are detailed below.

5.2.1 Site Reconnaissance

An initial site visit was conducted on October 31, 2007. The purpose of this site visit was to observe the existing site conditions, residential areas, utilities, and industrial areas. Several site visits were made after the initial site visit to gather additional information and confirm project elements.

5.2.2 Survey Data

John Chance Land Surveys (JCLS) completed the survey data collection and processing. JCLS delivered digital terrain models from the processed survey data. The design team combined the JCLS digital terrain models with USACE-provide LIDAR and hydrographic surveys.

Surveys conform to the requirements stated in Section 9 of the latest version of the "Hurricane and Storm Damage Risk Reduction System Design Guidelines". This includes identifying a minimum of three permanent benchmarks (new or existing) on design and construction drawings for all flood control projects (see Appendix J, for monument locations). The benchmarks were established relative to existing NAVD88 control established by the NGS, using either conventional differential leveling and/or the latest NGS-approved differential GPS network observations, with appropriate corrections to the local hydraulic design surface. Prior to and during actual construction stake out, these primary reference marks shall be verified externally and internally and field records of these survey verifications shall be permanently archived. A complete reevaluation of the vertical datum shall be conducted at each scheduled periodic inspection. The survey report has been completed and is attached to this report as Appendix J, Surveys.

The data collected by John Chance Land Surveys include the following:

- Topographic survey at the gate and pumping station locations and bypass areas.
- Cross sections on 500-foot centers for the length of the channel relocation.
- Topographic survey of the construction area with intermittent shots and swing topography taken to establish grade. Topographic features such as fences, utilities, power poles, roadways, bridges, sidewalks, ditches, canals, existing pumping stations structures and floodwalls and discharge pipes, and culverts.
- Site survey at the Algiers / Mississippi River lock location was collected in a grid system with shots at a maximum distance of 50 feet in any direction.
- All data tied to the adjacent baseline.

All horizontal data was tied to the State Plane Coordinate System using North American Datum of 1983 (NAD83). Vertical data was tied to the North American Vertical Datum of 1988 (NAVD88-2004.65).

TBM RED was established by performing Real-Time Kinematic (RTK) surveys based from PBM AC-3 as shown in Appendix J. Quality Assurance checks were performed from TBM RED back to PBM AC-3 on multiple days to verify that RTK corrections were being

delivered properly. For an additional check, Static GPS files for those setups were submitted to NGS OPUS for an independent Solutions check. Differences were observed and a report provided to USACE. A checklist is included in the Survey Appendix J.

JCLS provided the project survey plan, field books, EM file, kml file, codes list, baseline PI's and Echotrac Scrolls.

5.2.3 Borings, Cone Penetrometer Tests (CPTs) and Testing

The borings, CPTs, and test results used in the geotechnical analysis were provided by the USACE. The specific borings, CPTs, and test results used for each of the structure locations are presented in Section II of the Geotechnical Report attached to this report as Appendix B.

The borings, CPTs and test results data came from the following:

- Intracoastal Waterway Gulf Section Alternate Connection Vicinity, Algiers Lock and Canal showed boring locations near the Algiers Lock, by the USACE, June 1948,
- Draft Geotechnical Report Hero Canal Reach 1 Westbank and Vicinity Hurricane Protection Project, East of Algiers Canal, Plaquemines Parish, Louisiana, HNTB, September 2007,
- Boring Logs, CPTs and Laboratory Results along the Algiers Canal, 2007, and
- Boring Logs, CPTs and Laboratory Results along the Westwego to Harvey Canal, 2007.

5.2.4 Identify Potential Relocations

Potential facility and utility relocations were identified during the initial site visit on October 31, 2007 and during two subsequent site visits in November 2007. Utility owners were not contacted as part of this evaluation as stipulated in the scope of work for this study. These visits focused primarily on the initial Alternatives 2 and 3. In addition to the physical site reconnaissance, the survey data gathered by John Chance Surveys was

utilized to define the potential relocations effort for this project. Details of the relocations are listed in Section 8 of this report.

5.3 Hydraulic Design Criteria

5.3.1 General

Hydraulic design data used in this study were provided by the USACE and summarized below. The USACE are currently conducting hydraulic studies to complete the design discharge at the pumping stations for the various alternatives. They are performing flood routing simulations with the existing channel acting as a reservoir for storage. Determining how much water the channels can store then dictates the required discharge necessary. These studies have not been completed to date.

Hydrology and hydraulics design calculations for this project are provided in Appendix A to this report.

5.3.2 Modeling and the Design Elevations

The source of the hydraulic elevations in this EAR is the USACE MVN, October 9, 2007 report: "*Elevations for Design of Hurricane Protection Levees and Structures, Lake Pontchartrain and Vicinity Hurricane Protection Project; Westbank and Vicinity Hurricane Protection Project*", and subsequent addenda. All elevations are in Feet NAVD88 2004.65.

The HSDRRS includes features that provide protection from a hurricane event that would produce a 1% annual exceedence surge elevation and associated waves. Hydraulic modeling and analyses performed to calculate the surge elevation and wave characteristics are described in the October 9, 2007 report.

After construction is complete, the HSDRRS will meet the hydraulic requirements for levee certification as documented in draft Engineering Technical Letter (ETL), Engineering and Design, Certification of Levee Systems, for the National Flood Insurance Program (NFIP).

The hydraulic elevations presented in this EAR should be considered initial elevations. Additional, more thorough engineering investigations may follow to determine final construction elevations.

This EAR considers different configurations of levees and structures that may have different design elevations. The selected alternative may have effects on design elevations in adjacent contract reaches. To assure continuity of design methodology, consistency of designs across contract reaches, and provide close quality management, final design elevations utilized throughout the New Orleans area will be reviewed by the New Orleans District Engineering Division Chief of Hydraulics and Hydrologic Branch.

5.3.3 Future Analysis

As noted in the October 9, 2007 report, in the future, subsidence and sea level rise will affect elevations required for levee certification, and an analysis was performed to project the effect of these parameters on future surge elevations and wave characteristics. The New Orleans District will perform regular reassessments of these and other hydrologic parameters to assure the effectiveness of the system in future years. The system will undergo a reassessment after major events, significant changes in design and analysis methodologies, or no less than once every 10 years.

5.3.4 Gages

JCLS installed an Algiers GIWW staff gage located within the contract reach. This staff gage will be used for determining the tidal datum local mean sea level (LMSL) prior to construction. An exhibit of the gage location is shown on the last page of Appendix J to this report. The gage was attached to the piling of a private dock. Benchmark AC3 was used to calibrate the staff. The limits of the 5-foot graduated scale are El. 3.00 to El. -2.00 with 0.01-inch increments.

Additional temporary gages may be required during P&S depending on vertical accuracy requirements. The gage(s) can also be used to monitor future hydrologic conditions in the area. The datum of the gage(s) has been established to comply with criteria contained in the Vertical Control Requirements for Engineering, Design, Construction,

and Operation of Flood Control, Shore Protection, Hurricane Protection, and Navigation Projects (Engineering Division Policy Memo #2).

The relationship between NAVD88 2004.65 and LMSL for the gage(s) will be reevaluated and reviewed by NOAA every 5 years (or more frequently if warranted based upon rate of subsidence)

The "Vertical Datum Report" for the Westwego to Harvey Polder contains specific information on the gage network and the relationship between LMSL and NAVD 88 2004.65 for the project area.

5.3.5 Riprap Criteria

Thickness of riprap (24 inches) and limits of riprap placement for the pumping stations and sector gates were provided by the USACE on August 18, 2008. Final gradation will be provided by the Chief of Hydrology and Hydraulics (H&H) Branch during P&S.

5.3.6 New Pumping Station Capacities

Pumping station capacities required to prevent the "Retention Basin" (Channel) from rising above El. 8 .0 NAVD88 (2004.65) are shown in the table below. These capacities were provided by the USACE and are expected to be modified for the P&S. The P&S capacities are expected to be 20,000 cfs for Alternative 2 and 15,000 cfs for Alternative 3. However for the purpose of this report, the guidance provided consists of the following table.

Table 5.3.6: Pumping Station Capacities

Alternative		Location	Required Capacity (cfs)
GIWW South Gate on the GIWW	2D	East of the secondary channel	25,000
	2E	East of the secondary channel	25,000
Algiers Gate on the Algiers Canal	3B	East side of existing channel	16,000

5.3.7 Design Stages and Structure Elevations

Design stages and top elevations of structures are provided below. All elevations are in NAVD88 (2004.65).

Table 5.3.7a: 1 % Exceedence Design Stages [1, 2]

Description	Protected Side El.	Flood Side El.
Normal operation (no hurricane)	0.75	0.75
Maximum reverse head (allowing gate operation)	3.5	-1.5
Maximum direct head (allowing gate operation)	-1.5	2.5
Maximum direct head (no hurricane)	-0.5	4.5
Hurricane – water to top of wall/gate	-1.5 to 8.0	16
Hurricane – water to SWL (no wave)	-1.5 to 8.0	11.0
Maintenance unwatering	3.5	4.5

[1] Design Stage for "Retention Basin" is El. 8.0

[2] Old Authorized SWL is El. 8.0.

Table 5.3.7b: 1% Exceedence 2007 Design Elevations

Description	Top of Feature (2007 El.)
Existing sector gates, fronting protection walls, and all walls requiring cofferdams	12.0*
Walls not requiring cofferdams	10.5
Levees	10.5

*Includes 2 feet of Structural Superiority.

Note: For all features, 2007 Elevation is El. 10.5.

Table 5.3.7c: 1 % Exceedence 2057 Design Elevations

Description	Top of Feature (2057 El.)
New sector gates and adjacent pumping station fronting protection walls	16.0*
Walls not requiring cofferdams	14.5 (Algiers) 14.0 (Harvey, GIWW)
Levees @ gates	14.5 (Algiers) 14.0 (Harvey, GIWW)

*Includes 2 feet of Structural Superiority

5.3.8 Critical Structure Dimensions

Critical gate structure dimensions (all options) and elevations are listed below.

Sill Elevation	El. -16.0
Opening Width	110 feet and 225 feet
Top of Fenders and Guide Walls	9 feet above Normal Water Surface Elevation (WSE)

5.4 Civil and Geotechnical Design Criteria

5.4.1 General Civil Design Criteria

Prior to construction, borrow pits must be located, tested and acquired to provide the necessary fill material for any design alternative. Estimates of the required borrow area are based on an assumed 20-foot pit depth and 3:1 side slopes and 75% usable materials.

Pipeline crossings are being identified by the USACE Relocations Section in MVN. The Armoring Team is reviewing their detail drawings and requirements to include armoring features. These drawings will need ITR and should be forwarded to those utility owners that are ultimately responsible for the work.

No utility owners were contacted, as requested by the USACE; therefore, the location, quantity, and size of any utilities described in the report are subject to correction upon verification with the utility owner.

All overhead electrical crossings are assumed to be currently at or near the required vertical clearance based on voltage. Vertical clearance above levee crossings shall be maintained in the proposed condition and raising all transmission poles a distance equivalent to the changed levee elevation.

In alternatives 2D and 2E, maintenance for access to the east side proposed levees, pumping station and 110-foot sector gate is to be provided by two 20-foot wide access ramps from realigned East Bayou Road and a 950-foot concrete span bridge along the intake side of the pumping station building. Access to the west side of the 225-foot sector gate or pocket gate is to be provided by a 12-foot wide maintenance access road on top of the protective berm located along the protected side of Section 404(c) wetlands area floodwall. It is assumed that the maintenance crew can access the 404(c) wetland area, the flood side of the floodwall via the Old Estelle Canal pumping station's access road and the existing Westwego – Harvey levee, or by ladder over the floodwall. To minimize the impact to the 404(c) wetland area, access to the flood side will be reviewed in the P&S.

In alternative 3B, maintenance access to the proposed levees, pumping station and 225-foot sector gate is to be provided by two 20-foot wide access ramps from realigned East Bayou Road and a 760-foot concrete span bridge along the intake side of the pumping station building.

Alternatives 2D, 2E and 3B would require realignment of the existing Algiers (East) Levee and the East Bayou Road. The realigned road is assumed to be 20-foot wide gravel road with a 2-foot wide shoulder on each side. The construction for the road is assumed to have 4 inches of gravel surface on 9 inches of compacted base fill separated by geotextile fabric. Provisions to include lands and damages in the cost estimates were beyond this project's scope of work. Quantity estimates for relocations and approximate acreages for additional servitudes are provided for the USACE's use.

5.4.1.1 Civil Design Criteria References

The following design criteria provided by the USACE serve as the civil design basis:

- American Association of State Highway and Transportation Officials, A Policy on Geometric Design of Highways and Streets, 4th Edition (The Green Book), 2001
- American Association of State Highway and Transportation Officials, Manual on Uniform Traffic Control Devices, 2001
- American Association of State Highway and Transportation Officials, Standard Specifications for Highway Bridges, 17th Edition
- Louisiana Department of Transportation and Development, Bridge Design Manual (English)
- Louisiana Department of Transportation and Development, Road Design Manual
- Louisiana Department of Transportation and Development, Standard Specifications for Roads and Bridges (LSSRB), 2000
- EM 1110-2-1611, Engineering and Design – Layout and Design of Shallow Draft Waterways, December 1980

5.4.1.2 Armoring

Armoring will be provided for critical areas of the Hurricane and Storm Damage Risk Reduction System (HSDRRS) features described in this report. The design criteria for determining the overtopping rates and armoring methods are still under investigation. Therefore, a detailed description of the armoring criteria for the features in this report is not available. This work will continue in parallel with other pre-award activities until complete.

The USACE Armoring Team is tasked to provide research and planning for the use of armoring against erosion and scour on the protected side of selected critical portions of levees and floodwalls in the HSDRRS. These critical areas include: transition points (where levees and floodwalls transition into any hardened feature such as other levees,

floodwalls, pumping stations, etc.), utility pipeline crossings, floodwall protected side slopes, and earthen levees that are exposed to wave and surge overtopping during a 500-year hurricane storm event. The Armoring Team will be guiding the design Project Delivery Teams (PDT) in this process by providing an Armoring Manual for design guidance and criteria. This manual will be the basis for decisions on what should be armored and how armoring should take place.

The Armoring Team defines resiliency as the capacity of the levee/floodwall to resist, with out catastrophic failure, overtopping (wave and surge) caused by a storm which is greater than the design event. A Resilience Team has been formed to validate the Armoring Team's initial focus. MVN Engineering Division is leading the Resiliency effort to certify the practicality and applicability of using the 500 year storm event for armoring. The armoring methods to be implemented in the final design are anticipated to provide erosion protection such that the structure will be resilient to the 500-year event, or more defined as the ability of the structure to provide protection during events greater that the design event without catastrophic failure.

The following armoring methods are under consideration and the appropriate combination of methods will be applied throughout the earthen levee projects included in the HSDRRS:

- ACB – Articulated Concrete Blocks,
- ACB/TRM – The physical conditions or hydraulic parameters are such that small modifications could allow a reduction to a TRM (Turf Reinforcement Mattress),
- TRM,
- TRM/Grass – The physical conditions or hydraulic parameters are such that small modifications could allow a reduction to a surface with good grass cover only, and
- Good grass cover.

The armoring required for floodwalls will be a hybrid of materials to accomplish the require level of armoring. For instance, the interim floodwall repairs curtailed the concrete splash pads midway down the levee slope. The Armoring Team suggests that these pads be extended down the entire slope of levee and be curtained at the toe in order to eliminate a transition in a critical part of the levee section.

Transitions have been a significant part of the Armoring Team's effort to date. The transitions from structures to floodwalls to sheet piles are being addressed with detailed design drawings and will be forwarded to the individual design PDTs to aid them in their site-specific designs.

5.4.2 Geotechnical Design Criteria

The geotechnical analyses were completed at each structure to determine preliminary design sections and cost estimates. This section presents the design criteria and analysis used to determine the recommendations of this report. Geotechnical analyses included slope stability, seepage, pile capacities, and settlement. The Geotechnical Report, attached to this report as Appendix B under separate volumes, presents the methods of analyses, design criteria and assumptions used for each structure.

A complete geotechnical analysis will be performed on the selected alternative during the preparation of P&S. This analysis will conform to the guidelines included in the latest version of the "Hurricane and Storm Damage and Risk Reduction System Design (HSDRS) Guidelines". We do not expect this further design work to affect the selection of the preferred alternative.

The following design criteria, provided by the USACE, serve as the geotechnical design basis:

- Hurricane Protection System Slope Stability Design Criteria and Interim Design Criteria for Earthen Embankments as provided in Section 5.4.3 of the SOW dated October 12, 2007,
- New Orleans Hurricane and Storm Damage Reduction System Draft T-Wall Design Procedure dated July 31, 2007 and modified as provided in Section 5.4.4 of the SOW dated October 12, 2007,
- Hurricane and Storm Damage System Design Guidelines, Draft, March 16 2007, and
- New wave loadings, wave berm requirements, still water and top of wall elevations are from USACE e-mail correspondence dated November 14 and 15, 2007.

Other applicable USACE Publications included:

- EM 1110-1-1904, Settlement Analysis, September 1990,
- EM 1110-2-2502, Retaining and Flood Walls, September 1989,
- EM 1110-2-2906, Design of Pile Foundations, January 1991,
- EM 1110-2-2504, Design of Sheet Pile Walls, March 1994,
- EM 1110-2-1913, Design and Construction of Levees, April 2000,
- EM 1110-2-1901, Seepage Analysis and Control for Dams, April 1993,
- EM 1110-2-2100, Stability Analysis of Concrete Hydraulic Structures, December 2005,
- DIVR 1110-1-400, Soil Mechanic Data, December 1998, and
- EM 1110-2-2503, Design of Sheet Pile Cellular Structures Cofferdams and Retaining Walls, September 1989.

Other applicable Publications included:

- NAVFAC Design Manual DM-7, Soil Mechanics, Foundations, and Earth Structures, 1971.

Computer software used:

- SLOPE/W Version 2004 and 2007 by GeoStudio for slope stability using Spencer's Method,
- "Stability with Uplift" for Slope Stability using MVD Method of Planes,
- CE Sheet Pile Wall Design/Analysis Program, "CWALSHT",
- CSETT for settlement estimates,
- GROUP for Windows by ENSOFT, Inc. for group lateral pile analysis, and
- SEEP/W Version 2004 and 2007 by GeoStudio for Seepage.

5.5 Structural Design Criteria

5.5.1 General Structural Design Criteria

The scope of the structural analyses and design is limited to the layout and design of all pile foundations and major elements of sector gates, pumping stations, floodwalls, floodwall sluice-gate structures, flow control structures and cofferdams. The 1% annual exceedence probability of occurrence (100-year) design elevations was used for analysis. Sufficient analysis was performed to support the major member sizes and layouts shown in the plates.

Structural details and computations for the three selected alternatives are provided in this report as Appendix C, Structural Design Calculations, Final Study Alternatives (Alternatives 2D, 2E and 3B) of this report, except as noted below.

Structural analyses for the sector gate were carried out using STAAD Pro software. AISC beam coefficients for determining bending moments and shears in continuous structures were used in lieu of resorting to detailed analytical solutions. The structural design work includes the layout and design of the major structural elements that comprise each gate,

including the gate truss members, the gate face, the sill, the side enclosure for the gate, the hinge connection, connection to mechanical operating equipment, the pile foundations, and appurtenances. Analysis was performed as sufficient to develop the major member sizes and layouts. Details and computations are shown in Appendix C to this report, "Structural Design Calculations, Final Study Alternatives 2D, 2E, 3B" and in Appendix D, "Structural Design Calculations" of the "Innovation Study Report."

In general, the basis of design for the structural member sizing follows applicable guidance and criteria from documents tabulated below; information developed from previous studies/designs; and acceptable engineering practice.

The following design criteria were provided by the USACE for use as the basis of design:

- DRAFT Hurricane and Storm Damage Reduction System (HSDRS) Guidelines, March 2007, and revisions – July 2007, October 2007, and
- 1% Future Wave Forces (at Hero Canal and Algiers Canal).

Other applicable USACE publications included:

- EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures, June 1992, including Change 1, August 2003,
- EM 1110-2-2105, Design of Hydraulic Steel Structures, including Change 1, May 1994,
- EM 1110-2-2502, Retaining and Floodwalls, September 1989,
- EM 1110-2-2906, Design of Pile Foundations, January 1991,
- EM 1110-2-2703, Lock Gates and Operating Equipment, June 1994,
- EM 1110-2-2503 Design of Sheet Pile Cellular Structures Cofferdams & Retaining Structures (1989),
- EM 1110-2-2702 Design of Spillway Tainter Gates (2000),
- EM 1110-2-3104 Structural and Architectural Design of Pumping Stations (1989), and
- TI 809-02 Structural Design Criteria for Buildings (1999).

Technical publications included:

- AASHTO Standard Specifications for Highway Bridges, 17th Edition (2002),
- ACI 318-05 Building Code Requirements for Structural Concrete and Commentary (2005),
- ACI 350-06 Code Requirements for Environmental Engineering Concrete Structures (2006),
- AISC Steel Construction Manual, 13th Edition (2006) and Load & Resistance Factor Design, 2nd Edition (2001),
- ASCE 7-05 Minimum Design Loads for Buildings and Other Structures (2006),
- SDI Diaphragm Design Manual, 3rd Edition (2004), and
- Engineering Monograph No. 27 – Moments and Reactions for Rectangular Plates by W.T. Moody.

Design aids:

- Steel Roof and Floor Deck, Vulcraft Corp. (2001),
- Moments and Reactions for Rectangular Plates, United States Department of the Interior Bureau of Reclamation (1963),
- PCI Bridge Design Manual, Precast/Prestressed Concrete Institute,
- PCI Design Handbook, 6th Edition (2004), and
- Wood Handbook - General Technical Report FPL-GTR-113.

Computer software utilized:

- MathCAD Version 14,
- CE Pile Group Analysis (CPGA),
- Structural Analysis and Design Software (STAAD-Pro),
- Group 7.0,
- Microsoft® Office Excel 2002, Microsoft Corporation (version 10), and
- T187 matrix structural analysis program, HNTB Corporation (version 2.1012).

5.5.2 Structural Steel Design – Gate Structures

Top of Gate Walls	El. 16.0
Sill	El. -16.0
SWL	El. 11.0
Width of Gate Channel Opening	110 and 225 feet

5.5.2.1 Maximum Differential Head

Flood Side	El. 16.0
Protected Side	El. -1.5

5.5.2.2 Wave Forces

(90% Non-Exceedance Value, Hero Canal)	Fwave = 5.28 kips/foot [acting at 1.31 ft. with respect to (w.r.t.). Datum]
(90% Non-Exceedance Value, Algiers Canal)	Fwave = 4.82 kips/foot [acting at 5.28 ft. w.r.t. Datum]

5.5.2.3 Barge Impact

Barge Impact	125 kips

5.5.2.4 Unit Weights

Water	64.0 pounds/cubic foot (pcf)
Concrete	150.0 pcf
Earth	120.0 pcf
Steel	490.0 pcf

5.5.2.5 Concrete Design Parameters

28 Day Concrete Compressive Design Strength (f'c):	f'c = 3000 – 4000 pounds/square inch (psi)
Reinforcing Steel Strength (fy):	fy = 60 kips/square inch (ksi)
Maximum Flexural Reinforcement ρb:	0.25 ρb (Recommended) 0.375 ρb (Permitted w/o special studies)
Minimum Flexural Reinforcement:	ACI Code
Minimum Thickness:	T-walls = 18″ Walls subject to barge impact = 24"
Minimum Concrete Cover:	3″ minimum for sections, 12″ < t < 24″ 4″ minimum for sections, t ≥ 24″

5.5.2.6 Structural Steel Design Parameters

All Plate and shapes are considered as A 572 GR 50 Structural Steel.

Yield Strength of Structural Steel (Fy): Fy = 50 ksi

Considering Type-B, Hydraulic Steel Structure, and the Allowable Stress is considered 5/6 of the AISC Allowable Stress.

5.5.2.7 Piles

Unfactored service loads, including the applicable allowable overstress factors, were used in all pile analysis. The USACE-provided CPGA, as well as Group 6.0, was the foundation software used to determine the pile loads and deflection. The maximum structural deflection at pile heads could not exceed 0.5 inch vertically and 0.75 inch horizontally, based on service loads reduced by the applicable allowable overstress. For deflection purposes, the maximum allowable overstress for high water checks was assumed not to exceed 33.3%.

Pile embedment was determined to be 9 inches into the base slab.

5.5.3 Structural Design Criteria – Pumping Station and T-Walls

Top of Operating Floor	El. 21.5
Intake	El. -16.0
SWL	El. 11.0 (flood side) El. 8.0 (protected side)

5.5.3.1 Wave Forces

(90% Non-Exceedance Value, Hero Canal)	Fwave = 5.28 kips/foot [acting at 1.31 ft. w.r.t. Datum]
(90% Non-Exceedance Value, Algiers Canal)	Fwave = 4.82 kips/foot [acting at 5.28 ft. w.r.t. Datum]

5.5.3.2 Unit Weights

Water	64.0 pcf
Normal Weight Concrete	150.0 pcf
Steel	490.0 pcf
Saturated Sand / granular backfill	120.0 pcf
Uncompacted fill	100.0 pcf

5.5.3.3 Concrete and Steel Design Parameters

28 Day Concrete Compressive Design Strength (f′c):

Pumping Station	f'c = 4,000 psi
Floodwalls	f'c = 4,000 psi

Reinforcing Steel Min. Yield Strength (fy): fy = 60 ksi

Structural Steel Min. Yield Strength (Fy):

W sections	Fy = 50 ksi
M, S sections, and Angles	Fy = 36 ksi
Plates	Fy = 50 ksi
Anchor Rods	Fy = 55 ksi

5.5.3.4 Piles

Unfactored service loads, including the applicable allowable overstress factors, were used in all pile analysis. The USACE-provided CPGA was used to determine the pile loads and deflection.

5.5.3.5 Reinforced Concrete Design

Reinforced concrete hydraulic structures were designed according to the USACE criteria (EM 1110-2-2104).

A hydraulic factor, Hf = 1.3, was applied to factored loads for moment and shear to improve crack control by increasing reinforcement requirements, thus reducing steel stress. A single load factor of 1.7 for dead and live loads was used to factor loads.

The strength reduction factor for bending was 0.9. A maximum strength reduction factor used for shear was 0.85; however, 0.75 was acceptable according to the current ACI requirements. The following criteria were implemented as per ACI code and the pertinent Engineering Manuals:

Concrete Design Strength:	f′c = 4000 psi
Reinforcing Steel Strength:	fy = 60 ksi
Maximum Flexural Reinforcement:	0.25 ρb (Recommended) 0.375 ρ_b (Permitted w/o special studies)
Minimum Flexural Reinforcement:	ACI Code
Minimum Thickness:	T-walls = 18 inches (“) Walls subject to barge impact = 24"
Minimum Concrete Cover:	3″ minimum for sections, 12″ < t < 24″ 4″ minimum for sections, t ≥ 24″

5.5.4 Structural Steel Design

5.5.4.1 Gate Structures

The minimum steel thickness considered for corrosion control was 5/16 inch. The Allowable Stress Design method was used for structural design. The Load Resistance Factor Design method was not used for any structural steel design.

5.5.4.2 Pumping Station and Flow Control Structures

Member strengths were determined in accordance with AISC, Manual of Steel Construction, Load Factor Design, as modified by EM 1110-2-2105.

5.5.5 Design Loads

5.5.5.1 Design Loads – Gate Structures

The following loads were considered for the preliminary analysis and design of the sector gate structures:

- Dead Load (including self-weight and fender loads),
- Wind (as per ASCE 7-05),
- Hydrostatic Pressure to Still Water Level (SWL),
- Hydrostatic Pressure to High Water Level (HWL) or Top of Wall (TOW) Wave Forces (as provided by USACE 14 November 2007). For the GIWW and Algiers locations; two sets of wave loads were provided by USACE. Conservatively the GIWW location (Hero Canal) wave-load was used in the design as the governing wave load,
- Uplift Forces (hydrostatic pressure under the base slab at various water elevations assuming pervious and impervious sheet pile cutoff walls),
- Earth Pressure (based on various assumed heights using 120 pcf – active earth coefficient = 0.8),

- Barge Impact (taken as 125 kips as per EM 1110-2-2703), and
- Reverse Head (hydrostatic pressure using water levels provided in the scope of work).

5.5.5.2 Load Cases – Gate Structures

Due to the nature of the scope of work (preliminary design), the load cases that were considered in the analysis and design process were limited to a minimum number of cases that were either provided by the USACE or determined by engineering judgment based on review of the pertinent Engineering Manuals (EM).

The load cases and the pertinent allowable overstress percentages that were considered in the sector gate analysis were as follows:

- SWL – (No overstress),
- SWL + Barge Impact (50% overstress),
- SWL + Wave (33.33% overstress),
- SWL + Wave + Barge Impact (75% overstress),
- TOW (No Barge Load, 50% overstress),
- Dead Load (16.67% overstress),
- Reverse Head (No overstress), and
- Wind + Reverse Head (No overstress).

The sector gate foundation used similar, but slightly different load cases for analysis and design. They are as follows:

- SWL,
- SWL + Wave,
- TOW,
- Reverse Head, and
- Maintenance Unwatering.

5.5.5.3 Design Loads – Pumping Stations

- Dead Load (D): The loads from the supporting structure and permanent non-removable items are considered dead loads.
- Fluid Load (F): Unit weight of water calculated as 64 pcf for hydrostatic and buoyant forces. Wave forces have been supplied by USACE, dated 14 November 2007 and are included in the design calculations.
- Floor Live load (L) and Roof Live Load (Lr): Live loads will consist of gravity loads other than dead loads that must be considered in the design. The live loads listed include an allowance for installed equipment such as generators, pumps and control cabinets.

Roofs, except safe house	50 pounds/square foot (psf)
Safe house roof	100 psf
Stairways and corridors	100 psf
Control rooms	300 psf
Operating areas	500 psf
Erection areas	1,000 psf
Vehicular areas	300 psf or H15 with impact
Safe house floor	300 psf
Earth surcharge	2.0 feet of fill

- Live loads may be reduced 20% for the design of a girder, truss, column, or footing supporting more than 300 square feet of slab, except that operating and erection areas will be reduced by 20% only where the member supports more than 500 square feet of slab.
- Wind load (W): Wind loads are per ASCE 7-05 method 2. The basic wind speed is 125 miles per hour (mph) with exposure category C. The safe house is designed for a 480 per square foot (psf) wind load applied in any direction on the walls and roof.

- Lateral earth pressure (H): The pumping station and floodwalls are designed for at-rest soil coefficient of 0.8. The structure will be designed to permit the excavation to be backfilled after the construction of the ground-level slabs. Walls will be designed for ground water at the ground elevation and a live load surcharge equal to 2.0 feet of fill.
- Barge impact (BI): The barge impact force is 100 kips where barge impact can occur now or in the future. On walls, the load is distributed over a 5-foot width plus the width gained along a 45-degree angle. On foundations, the load is distributed over the full width of a monolith. A minimum boat impact load of 0.5 kips/ft is applied on floodwalls. For still water cases, the BI is applied 3 feet above SWL or at the top of the wall, whichever is lower. For water to top of wall, the BI is applied at top of wall. The minimum thickness for walls subject to boat impact is 24 inches.
- Earthquake (E): No earthquake loads.
- Crane Loads (C): The maximum wheel load on crane girders and supports shall be the capacity of the hoist plus an additional 20% dead load allowance for the hoist and bridge. Crane rail dead load is 100 pounds per yard (33.3 lbs/ft). Impact loads on crane beams and supports are listed below.

Vertical	10% of the hoist capacity
Lateral	20% of the hoist capacity
Longitudinal	10% of the hoist capacity

- Construction Loads: The pumping station will be designed for a construction load of 200 psf applied over the entire area of the structure in plan view.

5.5.5.4 Load Cases – Pumping Station

The following load cases were investigated for the stability, foundation and structural design of the structures: usual conditions, unusual conditions, maintenance conditions, and construction conditions.

5.5.5.4.1 Usual Conditions

- Non-pumping operation: water to El. 2.0 at intake and discharge; water in intake tube; wind and earth loads.
- Pumping operation: water to El. 8.00 at intake, El. 11.0 at discharge; water in intake tube; wind and earth loads.

5.5.5.4.2 Unusual Conditions

- Water to El. 10.0 at intake, El. 11.0 at discharge; water in intake tube; wind and earth loads.
- Water to El. 8.0 at intake, El. 11.0 at discharge; no water in intake tube; wind and earth loads.

5.5.5.4.3 Maintenance Conditions

- Water to El. 2.0 at intake and discharge; no water in intake tube; wind and earth loads.

5.5.5.4.4 Construction Conditions

- Base slab and walls in place prior to watering; no hydrostatic loads; no wind and earth loads.
- Entire structure in place; water to El. 8.0 at intake and El. 2.0 at discharge; no water in intake tube; no wind and earth loads.

5.5.5.5 Load Cases – T-Walls

Typical T-walls were designed for the standard load cases and design checks from the Hurricane and Storm Damage System Design Guidelines.

5.5.6 Structural Design

5.5.6.1 Sector Gates

Two sector gates were designed for channel widths of 110 feet and 225 feet. These gates are regular steel sector gates which follow the USACE specifications. Removable buoyancy tanks were used in the analysis, which tends to relieve dead loads during mechanical operation of the gate. In addition to the steel sector gates, the structural components required to house a sector gate within the channel were also designed. These include gate bay walls, thrust blocks, base slab and foundation. Design of these components includes special emphasis on de-watered load conditions which is achieved using a needle girder system/dewatering bulkheads.

A comparison table is presented below to highlight the major differences in the design of these two sector gates.

Table 5.5.6.1: Comparison Between 110-Foot and 225-Foot Sector Gates

Item	110-Foot Gate	225-Foot Gate
Vertical Truss	3	4
Horizontal Frame	3	3
Truss Members	Wide Flange	Pipe
Horizontal Girder	Wide Flange	Plate Girder

5.5.6.2 Sliding Pocket Gate Structure

Two sets of loading conditions were prescribed by USACE that were applied to the sector gates and the rest of the structural components. These load cases are mentioned in Section 6.1.16.2. Apart from the loads specified by USACE, some additional loads were calculated and applied to the sector gate models. These include dead loads from vertical ribs, skin plates, walkway, fender etc. and live load on walkways. During the analysis,

125 kips of barge load was applied extensively at different panel points on the vertical face of the structure. Water loads for different loading conditions were applied on girders on the three horizontal frames. Separate analyses were performed to obtain the tributary loads on each girder for different water heads.

5.5.6.2.1 Truss Members Design Summary

For optimization purposes and based on the analysis results, the truss members were grouped in terms of the maximum tension/compression forces in them. They were also grouped based on their location in the entire structure. Based on these member groups, six different ranges of forces were identified for the 225-foot sector gate. Hollow core pipe cross sections were then selected for these forces and were checked for adequacy for each and every truss member. Although this checking was performed in STAAD, additional spot checks were performed by hand. The following is a table showing the different hollow pipe sections that were selected.

Table 5.5.6.2.1: List of Hollow Pipe Sections Used in 225-Foot Sector Gate

Section Name	Outside Diameter (in)	Inside Diameter (in)
A	18.0	15.0
B	12.0	11.0
C	12.0	10.5
D	10.0	8.0
E	10.0	9.0
F	8.0	7.0

For the 110-foot sector gate, optimization was performed by selecting five different wide flange sections that represent the truss members. The heights of all the member cross-sections were kept the same to facilitate joint connections. Further optimization was performed by removing members from the three horizontal frames. Care was taken not to

introduce any instability in the structure because of these removals. Also the truss members at the top frame were designed to have different sections than the bottom ones.

5.5.6.2.2 Horizontal and vertical Girders Design Summary

Both the horizontal and vertical girders were assigned the same cross-section. This ensured ease of connection in between the two elements. Based on their position and because of the load applied these elements experienced combined axial and flexural stresses and were designed accordingly. Since the barge loads were directly applied on the horizontal girders in some of the load cases, they experienced maximum stress and consequently needed the heaviest cross-section of all the structural elements.

In the 225-foot sector gate, the girders were designed as 4'-8" high plate girders, which had flange width of 1 foot. Web and flange thickness is kept at ¾ inch. For the 110-foot gate, a standard W24X94 section was deemed adequate for the girder.

5.5.6.2.3 Vertical Girder Design Summary

Same cross sections as the horizontal girders were used for vertical girders. Combined axial and bending effect was checked for the girders.

5.5.6.2.4 Skin Plate Design Summary

For the Sector Gates, skin plates are incorporated as ½ inch thick steel plates within the models. Based on the load applied and the corresponding stress output, it was found that all the limiting stresses (maximum bending, maximum shear, maximum principal and shear stress as well as Tresca and Von Mises stresses) were within the AISC allowable limit.

5.5.6.2.5 Vertical Ribs Design Summary

Vertical ribs were not directly incorporated in the STAAD models. Their self weights, however, were applied as dead loads. The vertical ribs were designed separately using the maximum water pressure from one of the load cases. Standard WT10.5x41.5 sections were deemed adequate.

5.5.6.2.6 Summary of Steel Requirement

Total amount of steel used in each sector gate model was calculated by STAAD. The calculated steel amount was then increased by 10% to take into account the steel requirement of the buoyancy tank. The total weight of steel required for the 110-foot gate is 259 tons while the amount of steel required for the 225-foot gate is 1,061 tons.

5.5.6.2.7 Substructure – Gate Bay Walls Design Summary

There are two sets of walls that house the sector gate in the recess position. Two of these walls are located on the flood side and the other two are located on the protected side. These walls experience different amount of water heads and therefore were designed separately. Moments were calculated for these walls at the sill elevation as these walls act as cantilevers supported at the base slab. Steel reinforcement for these walls was calculated per unit width of the wall. The steel reinforcement ratio for all the designs was kept under the USACE specified allowable limit of 0.25ρ balanced.

5.5.6.2.8 Substructure – Thrust Block Design

The thrust block (as referenced in the USACE provided Harvey Lock Design Memorandum) is the portion of the gate bay wall where the sector gate is anchored. The thrust block needs to withstand a number of forces making it a very critical element. The loads that are transferred to the thrust blocks are:

- Dead load reaction from the gate;
- Hydrostatic force acting on the block;

- Reaction from the gate bay wall which also experiences heavy loading from hydrostatic and soil pressure; and
- Force from the needle girder assembly during maintenance dewatering.

The first load was obtained from the SATTD analysis. The reaction from the gate bay wall was calculated considering the wall is supported on three edges. "Moments and Reactions for Rectangular Plates" by Moody provides monographs for calculating reactions for such cases. This was used to determine the reaction and corresponding moment on the thrust block due to hydrostatic and soil pressure on the gate bay walls. Needle girder forces were calculated for un-watering load cases when it was assumed that the total gate area was sealed off from water.

All four forces on the thrust block will be active during maintenance un-watering which proved to be the governing load case for this element. The thrust block was also designed as a cantilever with moments calculated at the base slab sill elevation. The dimension of the thrust block was determined to be 18 feet square for the 225-foot gate and 18 feet by 15 feet for the 110-foot gate. Both the thrust blocks were also checked for column action and were designed with heavy steel reinforcement.

5.5.6.2.9 Substructure – Base Slab Design

The base slab provides a cap for the foundation and also acts as a huge weight against the uplift of the water. The base slabs for the current study were designed to have minimum footprint beyond the sector gate. For the 225-foot wide gate, a 15-foot high base slab was provided, while for the 110-foot wide gate, it was limited to 10 feet. The weight of the base slab in addition to the gate bay walls provides a factor of safety of 1.2 against uplift. Also the friction from the 9-inch embedment of piles within the base slab provides an extra factor of safety.

At this level of study, the adequacy of the base slab height was verified in two ways. The first approach involves selecting a base slab width equal to the pile strip spacing width. The base slab strip was then analyzed using pile loads only. This was conservative and can be justified considering the fact during the early phases of construction only the base

slab will be there and there is no other overburden pressure to resist the uplift. The required reinforcement was calculated and the section depth was found adequate by ensuring that the provided reinforcements are within the permissible limits.

Based on previous construction (East of Harvey Sector Gate) it was observed that the critical moments are encountered if a strip of the base slab is taken through the region of the thrust block. Such strips were considered in each direction and were analyzed for critical moments. Steel reinforcements corresponding to such moments and designed within the allowable limit provided the adequacy of the base slab height.

All gate structural calculations are provided in Appendix D of the “Innovation Study Report.”

5.5.6.3 Structural Design – Sliding Pocket Gate

The Sliding Pocket Gate is a steel-framed, 50-foot wide by 36-foot high by 258-foot long conceptual gate that sits in a recessed pocket off of the GIWW to the east of the navigation channel. The gate is required to float across a 225-foot wide opening within the 400-foot wide channel and be sunk into place. The conceptual structural frame design was based on established engineering practices, preliminary calculations, and reference information from similar projects. Advanced technology demonstrating safety and efficiency is incorporated where possible. Specific design criteria for this concept can be found in Section 5 and Appendix C of the "Innovation Study Report."

5.5.6.4 Structural Design-T-Walls

The T-walls paralleling the Section 404(c) wetlands area is constructed such that no more than a 100-foot width of the Section 404(c) wetlands area, measured from existing right of way line is impacted by the construction of the floodwall. A protective berm will be placed on the protected side (GIWW channel side) of the wall up to El. 8.0 to protect the wall from barge impact. T-wall tie-ins between the gates, pumping stations and flow control structures are incorporated from the earlier designs of the eliminated alternatives. The T-walls were roughly sized based on preliminary analysis and design following the criteria set forth in the HSDRRS Guidelines. HP 14 x 89 steel piles (1H to 3V batter) were assumed for the foundations with sheet piling for seepage cutoff. The load cases

specified in the HSDRRS Guidelines were used for the development of the forces and moments.

5.5.6.5 Structural Design – Pumping Station

The design was performed for one pumping bay of a single structure since all pumping bays are identical and both alternatives have identical pumping station elevations. The two locations have similar pile capacity curves so a single pile layout was used for both alternatives and separate pile tip elevations were determined for each alternative.

Different components of the pumping station were designed as follows:

5.5.6.5.1 Intake Channel

The access road slab over the intake was designed as a continuous slab supported on the inlet channel walls. The dividing walls at the inlet were designed for bending and axial loads, assuming the wall to act as a beam fixed at the base and pinned at the top with a 5-foot water head difference between adjacent channels.

The piles layout in the intake channel was controlled by the construction loading when the site was dewatered.

5.5.6.5.2 Wall at Intake Opening

The wall was assumed to span simply from the top of the operating floor slab to the top of the intake opening. The wall was sized for forces from lateral loads, mainly hydrostatic, and axial loads from the superstructure and operating floor.

A beam on the top opening to support the lateral flood loads from the intake wall spanned the intake opening. Vertical loads in the wall were assumed to span by deep beam / diaphragm action because the clear span / depth ratio of the wall is 1.6.

5.5.6.5.3 Back-bay Wall

The deep wall under the back-bay was designed as a two-way flat plate fixed on the bottom and both sides, and free at the top. The lateral pressure on the wall included passive earth pressure due to the hydrostatic thrust force of the bend in the intake tube attempting to push the structure into the earth.

5.5.6.5.4 Intake Tube

The intake tube was designed as a three-sided box section, fixed at the bottom where it joins the thick bottom slab. The tube was designed for internal hydrostatic pressure assuming the remainder of the pumping station interior is dewatered.

Walls connecting the intake tube to the back-bay wall and the base slab resisted the hydrostatic thrust force of the bend in the intake tube.

The walls were analyzed as rectangular plates fixed on three sides and free on the top, subjected to a horizontal load in the plane of the walls.

Above the bend in the intake tube, the intake is a circular opening inside a rectangular cast-in-place concrete tube. It was analyzed as a cylindrical shell using a wall thickness equal to the minimum wall thickness of the intake tube.

5.5.6.5.5 Base Slab

The base slab thickness was determined by flotation stability requirements. Bending and shear in the slab were not a design consideration.

The pile layout was controlled by the construction loading when the site was dewatered. After the excavation is backfilled and flooded, the pile forces will be reduced. The piles will not be in tension during usual operation or maintenance load conditions.

5.5.6.5.6 Floors

The operating floor was designed as a one-way slab supported on one way beams. The beams are supported by the intake wall and a deep beam located between the front bay and back bay areas.

5.5.6.5.7 Superstructure

The roof deck was analyzed as continuous over the roof beams. The roof beams were analyzed as simple spans between the moment frames. The moment frames were designed as beam-columns supporting gravity, wind and crane loads.

The precast cladding was analyzed as a simple span beam supported at the floor slab and the top of the moment frame.

5.5.6.5.8 Overhead Bridge Cranes

Crane girders were designed as simple spans between the moment frame supports, with the live load applied at any point along the span.

5.5.6.5.9 Safe House

The safe house walls, roof and floor were designed as simple spans to resist a uniform lateral wind pressure in either direction. The roof and walls were designed as a shear wall system to transmit loads to the floor and the piles. The piles were analyzed as a pile group cantilevering above the ground line.

5.6 Mechanical Design Criteria

Mechanical design calculations for this project are provided in Appendix D to this report.

5.6.1 Gate Structures

5.6.1.1 Sector Gate Structures

In general, the basis of design for the mechanical and electrical systems for the gate alternatives follow applicable guidance and criteria from the USACE engineering manuals, referenced information from previous studies/designs and standard engineering practice. The following USACE Engineering Manuals have been referenced for this purpose.

- EM 1110-2-2703 – Lock Gates and Operating Equipment
- EM 1110-2-2610 – Lock and Dam Gate Operating and Control Systems

The USACE has also provided construction plans, as-built plans, and design memoranda to be used as example models for all aspects of design. This grouping of references was used throughout the preliminary design process.

Mechanical systems must support an operating opening or closing in less than one hour. As a failsafe in the event of mechanical failure, each gate design is to have the capability to disengage from the mechanisms so that the gate can operate manually. Additionally, machinery is to be designed to operate the gates for all identified load cases and electrical power is to be designed for utility service and emergency power systems.

More specific criteria regarding the mechanical design of the sector gate alternatives are provided in Sections 3.3.1 of the “Innovation Study Report.”

5.6.1.2 Sliding Pocket Gate Structure

The mechanical components associated with the sliding pocket gate are based on a conceptual design. The conceptual design is developed from preliminary calculations where possible, referenced information from similar projects and good engineering practices. Section 5.3.1 of the "Innovation Study Report" describes the mechanical components for the design concept.

5.6.2 Pumping Stations

5.6.2.1 Pumping Station Structure

The pumping station mechanical systems were originally arranged and designed to accommodate typical groups of four surge pumps based on the original design flow. However, the design flow has been adjusted which does not necessarily allow for a consistent group of four pump groups. However, to keep things simplified the pump systems have been left in this four-pump system grouping. The GIWW entire pumping station will consist of 17 surge pumps including one spare. The Algiers Canal entire pumping station will consist of 12 surge pumps including one spare. Each of the surge pump groups will operate independent of the adjacent group. This arrangement will result in smaller mechanical equipment sizes and minimize loss of station pumping capacity in case of equipment failure.

The fresh water supply system, waste lubrication oil system, and bridge cranes are designed to serve the entire pumping station. The safe house will have separate mechanical systems.

5.6.2.2 Surge Pumps

The pumping station is sized assuming a pumping unit average discharge capacity of 1556 cfs over the operating range of heads. The pumping units design conditions are the total dynamic head (TDH) for a canal or sump side water surface elevation varying from El. 2.0 to El. 8.0, and a still water flood side water surface El. 11.0, vertical formed suction intake pumps, with diesel engines. TDH will consider appropriate head losses for

trashracks, intake, friction, bends, and exit. One spare pumping unit will be provided. The total pumping capacity for the GIWW South alternatives shall be 25,000 cfs and 16,000 cfs for the Algiers alternative.

5.6.2.3 Surge Pump Engines

Surge pumps shall be driven by 4,000 horse power (HP) diesel engines to account for the loads described above in Surge Pumps.

5.6.2.4 Engine Speed Reducers

Engine speed reducers shall be sized to reduce the rated engine RPM to that of the rated pump RPM. Speed reducers shall be acceptable to both engine and pump manufacturers.

5.6.2.5 Engine Cooling Systems

Each pump engine cooling system will consist of a submerged water-cooled heat exchanger, piping and valves designed to cool the engine. Each gear reducer will also be cooled by a submerged heat exchanger. The design ambient air temperature is 100°F and the engine-jacket water temperature will be cooled to 180°F. Heat exchanger construction will be capable of withstanding hydraulic loads in the discharge flume of the pumping station.

5.6.2.6 Compressed Air System

The compressed air system will be designed to provide surge pump engine starting air, instrument air and general maintenance air. The system will consist of two industrial-grade compressors rated at 7.0 actual cubic feet per minute (ACFM) @ 200 pounds per square inch gauge (psig), two 525-gallon air receiver tanks, after-cooler, regulators, relief valves, air/water separators, instrumentation and controls.

5.6.2.7 Diesel Fuel System

The diesel fuel system will be designed to supply fuel to the surge pump engines and standby generators for 72 hours of continuous operation. There is one standby generator for four pumps system with a second for back-up. However, when there are an odd number of pumps and the last pump section contains less than four pumps, the two standby generators for this group would be smaller. There will be essentially one 12,000-gallon horizontal storage tank for each pump, and one 1,000-gallon horizontal day tank for each pump, fuel transfer pumps, filters, strainers, valves, instrumentation and controls.

The storage tanks will be double wall, ½ inch-thick steel and meet the requirements of UL 142 and National Flood Processors Association (NFPA) 30. The tanks will be located outdoors in a contained area and be easily accessed by fuel tanker trucks. Containment area will be constructed of reinforced concrete for additional redundancy.

The fuel transfer pumps will be rated at 20 gallons per minute (gpm) each capable of filling the associated day tank in approximately 60 minutes. The pumps will be located indoors adjacent to the day tanks and be automatically alternated.

The day tanks will be located adjacent to each engine.

5.6.2.8 Waste Lube Oil System

The waste-lubrication oil system will be designed to collect and store waste oil from the surge pump engines, engine gear reducers and standby generators. The collection system will consist of steel pipe directly connected to each oil case drain fitting and routed to drain by gravity to a 6,400-gallon horizontal steel storage tank located at the end of the pumping station. Each drain line will be provided with a ball valve to isolate the equipment during normal operation.

The waste-oil storage tank will be provided with level sight glass and be located below the surge pump level in a spill containment area. The spill containment area will be designed in accordance with the Environmental Protection Authority (EPA) Spill Prevention, Control and Countermeasure (SPCC) regulations. The tank outlet line will be routed to the

exterior of the pumping station and terminate with a camlock fitting suitable for connection to a tank truck. The waste oil will be transferred by a manually actuated electric pump.

5.6.2.9 Water Supply System

The pumping station supply system will be designed to rely on potable city water as the main source and emergency non-potable water well as the standby source. Further research and evaluation is required to confirm the availability of these water sources.

The water supply system will be designed to accommodate mechanical equipment and domestic water requirements. The distribution system serving the mechanical equipment will provide surge pump engine make-up water, speed reducer gear cooler water, surge pump seal water and general pumping station maintenance water to hose bibs. Domestic water needs will include safe house plumbing fixtures and bathroom facilities and will be served by a separate branch of the distribution system protected by a backflow preventer.

The supply system will include a booster pump to maintain station pressure requirements for the mechanical equipment distribution system. The emergency water well will be connected to the mechanical equipment loop of the distribution system.

5.6.2.10 Trash Racks and Rakes

Each surge pump suction inlet structure will be provided with trash racks. Each trash rack will be 35 feet wide and extend from the channel bottom at El. -16.0 to the discharge height at El. 16.0. The trash rack will be hot-dip galvanized ASTM A36 steel.

Two rail mounted elbow arm-raking mechanisms will be provided to clean all trash racks. Each mechanism will be designed to have all permanent moving parts above the water level. The rake is manually operated with an equipment operator during non hurricane events. The rakes will have the capability of removing a 20-foot long, 2-foot diameter tree trunk. The trash will be placed on the adjacent access road across the pumping station and moved with a bucket mounted bobcat to the end of the road for further loading and

offsite disposal. The rake arms and teeth, links, bolts, nuts and washers will be stainless steel and the rake frame will be hot-dip galvanized A36 steel.

5.6.2.11 Bridge Cranes

A 70-ton indoor bridge crane will serve the pumping station. The crane will be able to access all surge pumps, auxiliary systems and electrical equipment. The crane will be designed and constructed in accordance with CMAA 70 standards and have a Class A, standby service classification

5.6.2.12 Drain System

The pumping station lower level at El. -10.0 in each group of surge pumps will be provided with an automatic duplex sump system. Each sump pump will be rated at 30 gpm and will include automatic float switches. The drain system will operate on 110-volt, single-phase, 60-Hertz power.

5.6.2.12 Sump Dewatering System

Two portable submersible dewatering pumps will be provided to dewater the surge pump sumps. Each pump will be rated at 500 gpm and will be able to dewater each sump in approximately 3 hours. Each sump will be provided with permanent guide rails and means to attach a portable jib crane to raise and lower the dewatering pump. When placed into position the dewatering pump will rest in the bottom of the sump within a 12-inch deep box out.

A portable control panel with starter, overload protection, level-switch controls and Hubbell-type receptacle will be provided for each dewatering pump. A disconnect switch with Hubbell-type connector will be provided at each pump sump. Two portable jib cranes each rated at 1,000 pounds capacity will be provided.

5.6.2.14 Sewage Handling System

Connection to an existing sanitary sewer is not readily available based on initial investigations. Therefore, an individual onsite sewage system is proposed. The sewage handling system for the pumping station will treat the safe house generated wastes, pumping station wash down water, and the seal water for various mechanical systems. It will consist of a package plant with 1000 gallon per day (gpd) capacity. All pumping station waste lines will be able to convey flow to the system by gravity.

5.6.2.15 HVAC and Plumbing

The HVAC design criteria is based on the following:

- Outdoor winter design temperature = 30°F.
- Outdoor summer design temperature = 93°F.

5.6.2.15.1 Pumping Station Heating and Ventilation

The ventilation arrangement and operation principles for the pumping station will be in accordance with the American Society of Heating, Refrigeration and Air Conditioning Engineers publication standards.

The pumping station will be ventilated to remove excess heat generated by the operation of the surge pump engines, standby generators and motors within the space. The ventilation will operate to maintain a space temperature below 100°F at outdoor summer design conditions. Wall-mounted exhaust fans interlocked with dampers at intake louvers will provide the ventilation required. Automatic controls provided will energize the ventilation fans as required to maintain the space temperature.

The pumping station will be provided with electric heaters to maintain a 45°F space temperature at outdoor winter design conditions. Wall-mounted controls provided will energize the heaters as required to maintain the space temperature. Also 208 or 240 VAC electrical outlets will be provided to accommodate portable heating units.

5.6.2.15.2 Safe House Comfort Heating and Air Conditioning

The pumping station safe house will be heated and air conditioned for operator use during emergency pumping operations. Heating will be provided by electric coils within the air-handling unit and sized to maintain 70°F space temperature at outdoor winter design conditions. Cooling will be provided by the air-handling unit and sized to maintain 74°F space temperature at outdoor summer design conditions.

5.6.2.15.3 Plumbing

The safe house will be provided with separate men's and women's bathroom facilities that include toilets, urinals, lavatories and showers. In addition, the safe house will be provided with a utility sink and an electric water cooler for operation staff to utilize during operation of the pumping station.

Plumbing fixtures will be of the water conservation type and will meet local code requirements.

5.6.2.16 Fire Protection

Fire protection for the pumping station and safe house will be provided with wall-mounted fire extinguishers in accordance with NFPA and Factory Mutual requirements. The diesel-fuel-storage tanks will be designed in accordance with NFPA 30 requirements. The indoor fuel and waste lube oil system tanks will be designed in accordance with NFPA 31 requirements. A control-station alarm system will be provided, which will automatically notify the local fire department.

5.7 Electrical Design Criteria– Pumping Station

Electrical design calculations for this project are provided in Appendix D to this report.

5.7.1 Electrical Design Criteria – Pumping Station

The electrical design criteria are essentially the same for each alternative. The pumping station is the same for both Alternatives 2D and 2E. The difference for Alternative 3B pumping station is that there are fewer pumps and thereby a smaller station.

5.7.1.1 Electrical Service

The electrical service will consist of a 4000 amp, 480Y/277 volt, 3-phase, 4-wire, grounded secondary service from the local utility company, supplying “normal” power into a service-entrance rated main switchboard “SBM1”. Electric service metering will be designed in accordance with the local electric utility requirements.

5.7.1.2 Electrical Power Distribution

Due to the large size of the overall building, the building will be broken down into sections, with each building section consisting of four large flood pumps and associated ancillary equipment. Each of the four building sections will contain its’ own sub-distribution power system, containing a transfer switchboard, two standby generators, a motor control center, and power and lighting panel boards. For the GIWW Alternatives it is anticipated that there will be one section with only one pump to account for the uneven pump number requirements.

5.7.1.3 Main Switchboard “SBM1”

A 4000 amp, 480Y/277 volt, 3-phase, 4-wire switchboard will be provided to distribute power to each section of the overall pumping station building. The main switchboard will consist of a 4000 amp main circuit breaker and one 1200 amp distribution feeder breakers per section, with each of the feeder breakers supplying “normal” power to one of

the transfer switchboards. The main breaker will be a 100% rated solid-state type circuit breaker with LSIG (long time trip, short time trip, instantaneous trip, and ground fault) trip settings. Each of the distribution feeder breakers will be an 80% rated solid-state type circuit breaker with LSIG trip settings. Space will be allocated to add one additional distribution section to the switchboard in the future if desired.

5.7.1.3.1 Transfer Switchboards “SBT#” – “SBT#”

There will be one transfer switchboard for each building section. Each transfer switchboard will consist of an automatic transfer switch (ATS) and two emergency circuit breakers. “Normal” power will be fed to the transfer switchboard from one of the main switchboard feeder breakers, feeding into the “normal” source of the ATS. “Emergency” power will be fed to the transfer switchboard from two standby generators with each generator feeding into one of the emergency circuit breakers. The emergency circuit breakers will be “Kirk Key” interlocked, to allow only one of the emergency circuit breakers to be closed at any given time, and will feed into the “emergency” source of the ATS. The “load” output of the ATS will feed the motor control center (MCC). All transfer operations will be open-transition type to ensure the generator and utility cannot be paralleled together.

5.7.1.3.2 Motor Control Centers “MCC#” – “MCC#”

There will be one motor control center for each building section. Each MCC will be rated 1200 amp, 480Y/277 volt, 3-phase, 4-wire will be provided to supply power to all 480-volt process and HVAC equipment loads. The MCC will generally consist of full-voltage, non-reversing motor starters and distribution feeder breakers feeding their respective loads. The determination as to the starter type will be based on the relative horsepower ratings, starting current loads and operation criteria required for the equipment being served. The MCC will be constructed with tin-plated copper bus, in a NEMA 1 gasketed enclosure. Each MCC starter will be equipped with an integral control power transformer, sized for at least 50% spare capacity for future control modifications. All starter pilot lights will be push-to-test, LED type for long life.

5.7.1.3.3 Power Panelboards “H#” – “H#”

There will be one power panelboard for each building section. Each power panelboard will be rated 100 amp, 480Y/277 volt, 3-phase, 4-wire, and will supply 277 volt lighting throughout the facility.

5.7.1.3.4 Lighting Panelboards “L#” – “L#”

There will be one lighting panel board for each building section. The lighting panels shall be 208Y/120 volt, 3-phase, 4-wire systems to provide power to all 120 and 208 volt equipment. These will be provided with copper bus and bolt-on type circuit breakers.

5.7.1.3.5 Lighting Transformers “TR-L#” – “TR-L#”

There will be one dry-type lighting transformer for each building section. The transformers will be provided to step down the voltage from 480 volts to 208/120 volts, 3-phase, and 4-wire. The transformers will be provided with copper windings, and a minimum of two taps above rated voltage and four taps below rated voltage. The taps will be in 2.5% increments.

5.7.1.4 Emergency Power

There will be two diesel-driven standby generators for each building section. One of the two generators will be a “redundant” spare, which can be manually connected via the “Kirk Key” interlock switches at the transfer switchboard. Each generator will be provided with a UL-Listed, double-walled, sub-base type day-tank, which will be connected to the overall fuel supply system for the engine-driven pumps. Two of the generators have been preliminarily sized at 750 kW (kilowatt) / 937 kVA (kilovolt-ampere).

The remaining generators have been preliminarily sized at 650 kW / 812 kVA.

5.7.1.5 Surge Suppression

Surge suppression will be designed for the incoming service entrance using the appropriate level of transient protections as defined in ANSI/IEEE Standard C62.41, Category C requirements. A coordinated transient voltage surge suppression system will be provided to mitigate the damaging effects to plant electrical equipment due to incurred lightning strikes or off-site power line disturbances including switching surge contributions from a nearby capacitor bank located on the utility distribution system line. In addition, downstream transient voltage surge suppression devices will be employed throughout the power distribution system to further protect sensitive process instrumentation power sources and telecommunications data lines from induced transients.

5.7.1.6 Grounding and Lightning Protection

The design will provide for a solidly grounded system in conjunction with both the electrical power distribution system (NFPA 70 - “National Electrical Code”, IEEE 142 Green Book) and the instrumentation system (IEEE 1100 Emerald Book) grounding requirements. A grounding ring (counterpoise) will be installed around the pumping station building, consisting of ground rods and #4/0 grounding electrode conductors. A ground loop will be installed within the operating room floor, and will connect to the ground ring (counterpoise) by at least four grounding conductors. All connections to the ground ring (counterpoise) or the in-floor ground loop will be made with exothermic weld connections. The frames of the large surge pumps and other permanent motors and transformers will be directly connected to the operating floor ground loop.

A lightning protection system will also be provided to protect the entire pumping station from lightning strikes. The system will include a series of interconnected air terminals on the top of the building with down conductors connecting the air terminals to the grounding counterpoise ring around the pumping station building. The system will be designed in accordance with NFPA 780 - Standard for the Installation of Lightning Protection Systems.

5.7.1.7 Conduit and Boxes

All electrical conduits installed above the operating floor will be rigid galvanized steel conduit, except motors and other equipment subject to vibration will be connected with liquid-tight flexible metal conduit. All conduits buried below grade will be Schedule 40 PVC, encased in a steel reinforced red concrete envelope affording a minimum of 3 inches of concrete cover.

5.7.1.8 Power Factor Correction

Power factor correction capacitors will be installed for all motors rated 50 HP and larger, to correct the motor power factor to 92-95%. If large motors are served with a solid-state reduced-voltage starter, the capacitors will be controlled by a contactor to be disconnected from the circuit when the starter is “ramping up” during motor starting or “ramping down” during motor stopping. After the motor has come up to full speed, and the silicon-controlled rectifiers (SCRs) of the starter have been shorted out, the contactor shall close to energize the capacitors.

5.7.1.9 Lighting

Interior lighting in low-ceiling areas will utilize high efficiency, T-8 or T-5 fluorescent lamps with high power factor electronic ballasts, where possible. The operating floor, floodlights, and other fixtures that are use for long periods of time will be high pressure sodium type due to their high efficiency and long life. Fixture types and enclosure designs will be selected based on the physical, environmental and esthetic parameters associated with the area of installation. Special use fixtures will be provided in areas where required. Foot candle illumination levels will be designed based on EM 1110-2-3105 “Mechanical and Electrical Design of Pumping Stations”, and the recommendations of Illuminating Engineering Society of North America (IES) for the areas and tasks associated with each facility.

Exit lights will be internally illuminated LED type in order to provide long life and low energy consumption, and will be located within the facility to comply with the requirements of NFPA 101 “Life Safety Code”.

Emergency means-of-egress lighting will be provided from emergency, battery powered lighting units, integral emergency ballasts installed within select fluorescent fixtures, or a combination of both, based on the final design. Emergency lighting will comply with the requirements of NFPA 101 “Life Safety Code”.

Exterior lighting will be provided for security and general safety of the plant personnel. High pressure sodium type lamp/fixtures will be used due to their high efficiency and long life. In most cases lighting will be provided through the use of building mounted, vandal-proof perimeter fixtures with limited use of pole mounted area lights for those areas where required. Exterior lighting shall be photocell controlled.

5.7.1.10 Controls

Each engine driven pump will have its own control panel that includes a programmable logic controller (PLC) and human machine interface (HMI) display panel. Each panel will provide all the required safety and operational control functions for that engine – pump unit. All of the engine – pump unit PLCs will be connected together via a redundant ring Ethernet network for each area. Redundant ring networks maintain communications even when one link has been severed.

Each of the main pumping areas will include a master PLC system with redundant processors. The master PLC will monitor and control the associated HVAC equipment, common process equipment such as vacuum priming systems, and other equipment as required. These will be connected to the engine-pump redundant ring Ethernet network. This control panel will have a managed Ethernet switch that controls data traffic and access to the lower level network rings.

The main pumping control panels will be connected via a separate redundant ring Ethernet network to share information between areas and to communicate with the station's SCADA system as described below.

A personal computer based supervisory control and data acquisition system (SCADA) system will be in the safe house. The SCADA system will provide data storage, main HMI displays, and serve as the operators overall window into station operations. The SCADA system will provide status reports both on a scheduled basis, e.g. daily, and on demand. It will provide trend charts, alarm reporting, master scheduling, etc. This system will include redundant data servers protected by uninterruptible power supplies (UPS) units, with a minimum of 2 client workstations for operator use.

5.7.2 Electrical Design Criteria – Gate Structures

Specific design criteria and detail regarding the electrical design of the sector gates and sliding pocket are provided respectively in Section 3.3.2 and Section 5.3.2 of the Westbank and Vicinity, New Orleans, Louisiana, Hurricane Protection, Sector Gate South "Innovation Study" Final Report dated August 2008.

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6. Earthwork Requirements

For construction of the parallel protection alternative, there will be a large amount of borrow required. Construction of the sector gates, pumping stations and by-pass channels will generate considerable quantities of excavated materials that could provide valuable borrow to other projects. To the extent possible, suitable and unsuitable materials will be separated during excavation and stockpiled for reuse or disposal off site. Most of the soils appear in the boring logs to be suitable levee materials that will require some reconditioning (moisture control) prior to placement in fill sections. Leaner clays and silts will be preferred for construction of levees and general backfill, especially between any structure and channel side slope. There will be some requirement for select materials that will have to be obtained offsite. The most significant need will be for sand or similar materials to fill the cofferdam coffer cells.

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7. Real Estate – Right of Way Requirements

Right of Way requirements for the parallel protection Alternative 1 projects are not within the scope of this project and are not included in this report. For Alternative 1, some ROW acquisition may result in the construction duration of some projects extending past 2011.

Alternatives 2D and 2E will require additional ROW servitudes to be acquired between the existing ROW that is adjacent to the GIWW up to 25 feet east of the proposed construction limits. Alternative 2D will require 161.6 acres of Perpetual Flood Protection Servitude Easement, 15.9 acres of Required Perpetual Road Easement, 4.5 acres of Pile Easement, and 11.3 acres of Temporary Staging Area Easement. Alternative 2E will require 142.7 acres of Perpetual Flood Protection Servitude Easement, 14.7 acres of Required Perpetual Road Easement, 4.5 acres of Pile Easement, and 11.3 acres of Temporary Staging Area Easement.

Alternative 3B will require additional ROW servitudes to be acquired between the existing ROW that is adjacent to the Algiers Canal up to 25 feet east of the proposed construction limits. To construct the alternative, 67.6 additional acres of Perpetual Flood Protection Servitude Easement, 10.5 acres of Required Perpetual Road Easement and 7.1 acres of Temporary Staging Area Easement will be required.

During P&S, the size of the staging area of the selected alternative will be checked to verify it will accommodate a batch plant, an excavated material processing area, a material staging area, trailers and employee parking.

Table 7a: Summary of Additional ROW

Alternative	Perpetual Flood Protection Servitude Easement	Perpetual Road Easement	Underground Pile Easement Servitude	Temporary Staging Area Easement	Units
Alternative 2D	161.6	15.9	4.5	11.3	Acre
Alternative 2E	142.7	14.7	4.5	11.3	Acre
Alternative 3B	67.6	10.5	0	7.1	Acre

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8. Relocations

Relocation requirements for the Alternative 1 projects are not within the scope of this project and are not included in this report.

For the GIWW South Alternatives 2D and 2E, a portion of Bayou Road will be realigned and relocated in order to facilitate the design and the High Point Shooting Ground facility will be affected. A 10-inch diameter buried gas pipeline owned by ENTERPRISE crosses the proposed ROW and traverses under the GIWW. This pipeline will need to be relocated by directional drilling underneath the GIWW. The method of levee crossing will be developed during final design. Overhead electric distribution lines will also require relocation.

For Alternatives 3B, a portion of Bayou Road must be realigned and relocated in order to facilitate the new design of this alternative. Utility towers for overhead electric transmission lines carrying power to the facilities on Bayou Road must also be removed and relocated. Overhead electric distribution lines will require relocation. Additionally there will be several houses and trailer homes along Bayou Road that will require relocation to facilitate the limits of the proposed servitudes.

Table 8a: Summary of Utility Relocations

Description	Quantity	Units	Cost
ALT 2D and 2E – Relocate 10-inch natural gas pipeline	1,882	FT	$440,000
ALT 2E – Relocate 10-inch natural gas pipeline	995	FT	$250,000
ALT 2D and 2E – Relocate overhead electric distribution lines	3,229	FT	$2.1M
ALT 3B – Relocate overhead electric transmission lines	3,515	FT	$1.2M
ALT 3B – Relocate utility tower	2	EA	$100,000
ALT 3B – Relocate overhead electric distribution lines	2,669	FT	$1.3M

Table 8b: Summary of Facility Relocations

Alt.	Description	Quantity	Units
2D, 2E	High Point Shooting Ground	2	Acres
3B	425 Bayou Road -- House	1	EA
	427 Bayou Road – Trailer House	1	EA
	431 Bayou Road – Trailer House	1	EA
	445 Bayou Road – Trailer House	1	EA
	447 Bayou Road – Trailer House	1	EA
	457 Bayou Road – House	1	EA
	458 Bayou Road -- House	1	EA
	459 Bayou Road -- House	1	EA
	460 Bayou Road -- House	1	EA
	461 Bayou Road -- House	1	EA
	462 Bayou Road -- House	1	EA
	463 Bayou Road -- House	1	EA
	464 Bayou Road -- House	1	EA

Table 8c: Summary of Road Relocations

Alt.	Description	Quantity	Units	Cost
2D	Bayou Road	6,443	LF	$147,000
2E	Bayou Road	6,318	LF	$145,000
3B	Bayou Road	4,451	LF	$101,000

9. Cost Engineering and Construction Scheduling

9.1 Quantities and Cost Estimates (Consistent Method)

Quantities were computed by the design team based on preliminary designs by this EAR team and as provided for the Alternative 1 projects. Preliminary Cost Estimates were completed in Microsoft Excel format utilizing updated material quotes and unit prices available. Final Cost Estimates for the parallel protection Alternative 1 and for the three gate designs Alternatives 2D, 2E, and 3B were computed in Micro-Computer Aided Cost Estimating System (MCACES) MII format and in accordance with USACE work breakdown structure.

Detailed cost breakdown estimates of the three gate Alternatives 2D, 2E and 3B can be found in Appendix E of this report. For reference, detailed cost breakdown estimates of the eliminated alternatives 2.1, 2.2, 2A.1, 2A.2, 2B.1, 2B.2, 2C, 3, and 3A can be found in Appendix L of this report but it should be noted that the designs of those were stopped at various stages of completion. The estimates are based on generally accepted cost estimating practices and standards. Costs were provided in 2008 dollars.

Alternative 1 quantity data was provided by others and the level of completion for the Alternative 1 EAR projects varies. Lump Sum costs provided in EAR reports were included in this estimate as provided. No ROW information was provided for Alternative 1.

Alternatives 2E and 2D closure include the construction of the floodwall along Old Estelle discharge. The flood wall quantities included were determined under the WBV-14g.2 project by others.

9.2 General Assumptions

The general assumptions used for the cost and construction durations presented in this report at this time are listed below.

- The ten Alternative 1 projects are assumed to be performed each by a different contractor and are therefore estimated accordingly utilizing the limited preliminary information provided. It is our opinion that some of the smaller projects can be combined to make a total of 4 or 5 different projects instead of 10 as provided.
- Embankment cost for Alternative 1 was computed assuming a 1-way haul distance of 25 miles and that material would be supplied from a government furnished borrow pit.
- Embankment cost was determined based on the re-use of excavated materials which will have to include adequate space and time for drying and re-working of materials to provide a suitable material.
- At this time, it is uncertain how much excavated material will be suitable for levee construction. However, for estimating purposes, it is assumed that all the material is usable and all the material is are mechanically excavated. Note that it is likely that unsuitable materials will be encountered and that added costs may be incurred to dispose of these materials.
- Items to be performed by prime contractor are as follows: Clearing and Grubbing, Excavation, Embankment, Pile Driving, Reinforced Concrete, Temporary retaining systems (TRS), and Riprap.
- Sub-contractor Items are assumed to be as follows: Erosion Control items, Electrical work, Mechanical work, and Dredging.
- This estimate is based fair market value and is an estimated cost of time and materials and not a prediction of contractor's low bid.
- For alternatives 2D, 2E and 3B, it is assumed that the pile load tests are performed during the design phase of the project. Therefore, these tests do not interfere with production during construction.

- Assumed all the pile driving for the 225-foot' sector gate can be completed prior to installing the Phase 1 TRS. The gate will need to be designed with a stub-out to connect to the T-Wall, but the T-Wall itself should be able to be completed and tied into the TRS of the Gate Bay before 2011.
- Multiple crews are being used where possible and the construction schedule is updated to account for concurrent work as much as possible.

9.3 Construction Cost Estimates

A summary of estimated construction costs is listed in the table below.

Table 9.3: Summary of Estimated Construction Costs

Alternative	Estimated Cost	Contingency	Total Construction Cost
Alt. 1 Parallel Protection	$769.6M	$192.4M	$962.0M
Alt. 2D GIWW 2 Gates (225-Foot Sliding Pocket Gate & 110-Foot Sector Gate) & Pumping Station	$545.0M	$136.3M	$681.3M
Alt. 2E GIWW 2 Gates (225-Foot & 110-Foot Sector Gates) & Pumping Station	$532.2M	$133.0M	$665.2M
Alt. 3B Algiers 225-Foot Sector Gate & Pumping Station	$465.8M	$116.4M	$582.2M

9.3.1 Level of Contingencies Incorporated into Estimates

Several contingencies are considered in the estimate at this time including overtime and productivity price inflations due to the accelerated time schedule, inflationary prices and possible material shortages during the construction of the project and time from completion of this study to time of construction on the selected alternative. A contingency percentage of 25% was applied to the estimates for the 95% Submittal.

9.4 Construction Durations

Construction Schedules and Durations in this report are for the parallel protection Alternative 1 projects, and the gate alternatives designed under this project and the Innovation Study, Alternatives 2D, 2E and 3B. Detailed of these schedules can be found in Appendix F of this report. Detailed schedules of the eliminated alternatives 2.1, 2.2, 2A.1, 2A.2, 2B.1, 2B.2, 2C, 3, and 3A can be found in Appendix N of this report but it should be noted that the designs of those were stopped at various stages of completion.

Alternative 1 construction duration was compiled using data available. Average production rates were applied to quantities provided to determine this duration. The ten Alternative 1 projects are assumed to be multiple contracts that are not dependant on one another and therefore all have the same start date. It was assumed that all projects to be completed under this Alternative can be constructed at the same time utilizing multiple contractors. Details of this Alternative can be found in Appendix F.

Phased construction of the sector gates is necessary to minimize disruption of traffic through the canal. The following paragraphs summarize the TRS and construction sequence of Alternative 2E. For simplicity in this report, similar concepts and costs are assumed for Alternatives 2D and 3B. Detailed phases of construction will be generated in a future study during the development of plans and specification phase for a selected alternative. The required temporary retaining structures in the canal will be built to El. 11, the 2011 1% elevation, while the sector gates and their connecting structures are to be built to El. 16, the 2057 1% design elevation. Closing the entire canal and relying on the 110-foot sector gate is not considered, as rain events may increase the water velocities enough to make navigation unsafe.

Construction Phase 1 involves the simultaneous construction of the 110-foot sector gate and the pumping station (in the dry) and the eastern half of the 225-foot sector gate in the channel. This will require construction of a TRS on the east side. Canal traffic will pass through the west half of the gate during Phase 1. Once the area is dredged, the piles required by the base of the gate will be driven in the wet. A tremie seal to counteract heave will then be placed at the bottom of the excavation. Once the tremie concrete is set, the TRS will be pumped dry and construction of the gate slab will begin. The

thickness of the tremie seal is roughly calculated for this phase, and in the P&S phase this needs to be verified based on the actual site condition and hydraulic data. The TRS will be removed upon completion of the eastern portion of the 225-foot sector gate. A 12 by 12-foot sacrificial thrust block on the southwest corner of the sill is included to allow emergency closure of the channel in Phase 2.

Construction Phase 2 begins when the eastern TRS has been removed. The first step is the installation of the TRS to allow construction of the remaining western portion of the 225-foot sector gate. This would move navigation to the center of the channel, between the east gate bay and the west TRS wall. The 110-foot sector gate and secondary channel should be completed near the start time of Phase 2, providing an additional route for traffic if needed. Phase 2 construction is adjacent to the sacrificial thrust block that may serve as an emergency support for a series of needle girders. In the event of a predicted storm surge, girders would quickly be added between the sacrificial thrust block and the east recess to block the open channel between the western TRS and the eastern gate bay walls.

A general sequencing for each alternative designed under this project is listed below.

9.4.1 Acquisition Schedule

An Acquisition Schedule was discussed in depth and the following guidelines were provided by USACE and applied to each of the Gate/Pump Station Alternatives in conjunction with the phasing requirements detailed by alternative. The following list is the acquisition schedule provided.

1. Assume pumps ordered 1 Jan 09 (for Feb 2011 delivery). Utilize DOD priority manufacturing rights.
2. Prepare site and construct batch plant at Walker Pits beginning 1 Jan 09.
3. Assume channel excavation begins 1 May 09.
4. Excavate temporary bypass channel through 225-foot West sector gate recess in May 09 while concurrently excavating footprint for sector gates and pumping station.

5. Assume new East side levee construction begins 1 May 09.
6. Fabricate steel gates and bulkheads off-site. Assume work begins in May 09 (Use bulkheads for hurricane protection if sector gates are not in place by June 2011.)
7. Commence fabrication of steel formed suction intake offsite in May 09.
8. Open up navigation bypass through 225-foot West sector gate recess.
9. Drive piling in channel for 225-foot East sector gate recess and center slab while commencing TRS/open cut for pumping station and 110-foot sector gate. Leave existing levee intact as interim protection.
10. Submit TRS design for 225-foot East sector gate recess and center slab for approval. Leave existing levee intact as interim protection.
11. Commence pile driving and concrete work for all structures. Build 225' center slab and 225' east recess as first order of work. Conventional construction within a braced excavation/open cut is assumed. Construct pumping station foundation and walls to operating floor elevation with in-the-dry construction. Steel FSI to be lifted into place. Prefabricate pumping station building above operating floor at El. 6 with off-site construction of precast building or modular construction.
12. Install temporary approach walls and open center 160 feet to navigation traffic as soon as the 225-foot center slab is complete.
13. Construct TRS to act as hurricane protection on 225-foot west sector gate recess and start the 225-foot sector gate western recess.
14. Install gates at both sector gates prior to June 2011 (use bulkheads if gate installation is behind schedule). Test operate gates, one at a time.
15. Complete fronting walls for pumping station to provide interim protection for 2011.
16. Remove TRS for both gates.
17. Complete pumping station.

9.4.2 Alternative 2D GIWW South Gate

- Phase 1 includes excavation and the construction of the pumping station, 110-foot sector gate, cofferdam and the east side of the 225-foot sliding pocket gate. Also Included in this phase is construction of the T-Wall, the dredging for the new channel and the levee and road work associated with this Alternative. Channel traffic will remain on the west side of the existing channel.
- Phase 2 shifts channel traffic through the newly constructed 110-foot sector gate. This phase includes excavation, cofferdams and construction of the west half of the 225-foot sliding pocket gate.

9.4.3 Alternative 2E GIWW South Gate

- Phase 1 includes excavation for the structures, construction of the pumping station and 110-foot sector gate, cofferdam and construction of the east side of the 225-foot sector gate. Also Included in this phase is construction of the T-Wall, dredging for the new channel and the levee and road work associated with this Alternative. Channel traffic will remain on the west side of the existing channel.
- Phase 2 shifts channel traffic through the newly constructed 110-foot sector gate. This phase includes excavation, cofferdams and construction of the remaining portion of the 225-sector gate sill, supports, and gate protection.

9.4.4 Alternative 3B Algiers Sector Gate

- Phase 1 includes excavation and construction of the pumping station and cofferdam and construction of the east side of the 225-foot sector gate. Also Included in this phase is the levee and road work associated with this Alternative. Channel traffic will remain on the west side of the existing channel.
- Phase 2 shifts channel traffic to the east side of the existing channel. This phase includes excavation, cofferdams and construction of the west half of the 225-foot sector gate. Channel traffic will move to the east side of the existing channel.

9.5 Summary of Construction Durations

Below is a summary table of the durations for each alternative. These construction duration estimates do not include the fast track capabilities associated with the Early Contractor Involvement (ECI) acquisition method that the PDT is expected to utilize during P&S phase. For the selected alternative, further refinement of the construction schedule and sequencing by the PDT in cooperation with the ECI Contractor is anticipated to provide that interim protection will be completed by June 2011.

Table 9.5: Summary of Estimated Construction Durations

Alternative	Construction Duration	NTP Date	Interim Protection Date	Project Completion Date
Alt 1 Parallel Protection	2.5 years	May 1, 2009	Sept. 27, 2011	Sept. 27, 2011
Alt 2D GIWW Gates (225-Foot Sliding Pocket Gate & 110-Foot Sector Gate) & Pumping Station	3.5 years	May 1, 2009	March 16, 2012	October 5, 2012
Alt 2E GIWW 2 Gates (225-Foot & 110-Foot Sector Gates) & Pumping Station	4.75 years	May 1, 2009	June 18, 2012	January 21, 2014
Alt 3B Algiers (225-Foot Sector Gate & Pumping Station	5.0 years	May 1, 2009	Sept. 15, 2012	May 26, 2014

10. Operations and Maintenance Requirements and costs

10.1 Pumping Station Operation and Maintenance

This is a standard pumping station arrangement for many owners. It consists of pumps in a vertical column resulting in an efficient use of space. Maintenance will be slightly more difficult for the formed suction intake as opposed to a bellmouth intake because of limited access to the pumps.

Since these pumping stations are expected to run very infrequently, routine maintenance is critical to ensure operation of the system when it will be required. The Operations and Maintenance (O&M) of each pumping station is very similar. The station does not necessarily need to be staffed full time, but a skeleton crew is anticipated for general maintenance, security, inspections, and operation when necessary. Additional staff will be needed as required for various operation/maintenance components.

The following O&M schedule identifies recommended tasks and scheduled frequencies for the pumping station.

Table 10.1: Pumping Station Operations and Maintenance Schedule

Function/Element	Operation/Maintenance	Schedule
surge pump units	operate dry for 4 hours	monthly
	change engine oil	after recommended hours of operation
	change coolant	after recommended hours of operation
	change reducer oil	after recommended hours of operation
	major pump, engine, and reducer inspection	every 5 years
	minor pump, engine, and reducer inspection	annually
	change filters	annually
keel coolers	inspect	monthly
	clean	quarterly
compressed air system	operate	monthly

Function/Element	Operation/Maintenance	Schedule
	change oil	after recommended hours of operation
	change filters	after recommended hours of operation
diesel fuel system	operate fuel system (in conjunction with engines)	monthly
	inspect storage area	monthly
	filter stored fuel	monthly
	test stored fuel	semi-annually
waste oil lube system	inspect	annually
	pump out waste oil	as needed
water supply system	inspect	annually
trash racks and rakes	inspect trash racks	monthly
	clean trash racks	as needed
	operate rake mechanisms	monthly
bridge cranes	inspect	annually
drain system	inspect	annually
sump dewatering system	inspect	annually
sewage handling system	inspect system	annually
	pump holding tank	as needed
pumping station HVAC	inspect	annually
safe house HVAC	inspect	annually
electric panels and gear	inspect	annually
stand-by generators	exercise for 4 hours	monthly
	inspect	monthly
inlet and outlet gates	inspect	monthly
	operate	monthly
pumping station and safe house	routine operation and maintenance	weekly
	clean	monthly
pumping station grounds, access road, fencing and gates, storage tank area	routine maintenance	monthly

The pumping station superstructure is a structural steel frame with an unpainted precast concrete skin, and the substructure below elevation 10.0 is cast-in-place reinforced concrete, providing long-term service with low maintenance costs. Typical superstructure maintenance will include painting the structural steel frames, and replacement of the built-up roof system on an approximate 15-year cycle. Maintenance of the substructure should follow EM 1110-2-2002 Chapter 7, "Evaluation and Repair of Concrete Structures, Maintenance of Concrete." The costs for maintenance activities associated with the structure are based on historical O&M costs for similar structures which equates to 5% of the annual O&M cost. These are included in the O&M detailed estimates.

In addition, a more thorough and detailed inspection should occur on the entire pumping station annually or after every hurricane event. The detailed operation and maintenance requirements and schedule of each system will be detailed during final design.

In developing annual operation and maintenance costs, an event occurrence of once in four years was assumed. Thus, the figure for annual O&M costs represents the sum of the costs for annual routine operation and maintenance plus 25% of the costs of pumping station operation during an event. This results in an estimated annual O&M cost of approximately $3.33 million for alternatives 2D and 2E and $2.62 million for Alternative 3B. The detailed breakdowns are included in Appendix E of this report.

10.2 Gate Structures Operation and Maintenance

A system of scheduled operation and maintenance of the hurricane protection system will be critical to successful operation during hurricane events. The O&M details for the gate structures are described in Section 5.4 of the "Westbank and Vicinity, New Orleans, Louisiana, Hurricane Protection, Sector Gate South Innovation Study Final Report dated August 2008".

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11. Pros and Cons of The Alternatives

The following table contains the comparison of the alternatives presented in a matrix of Pros and Cons. The identified factors of Construction Costs, O&M Costs, ROW servitude acreage, Construction Durations, and Risk for Failure are compared and evaluated.

Table 11: Pros and Cons of Alternatives

Alt.	Pros	Cons
1	• Lowest O&M cost at $196,000. • Shortest construction duration of 2.5 years, not accounting for any delays associated with ROW acquisition. • No new operable structures and therefore most passive operation.	• Highest estimated construction cost alternative totaling $962.0M. • Longest length of protection system and therefore largest potential risk of failure. • Large number of vehicular gates and drainage monoliths that will be necessary for the Algiers Industrial reach. • ROW required acres was not provided to the design team; however, a significant number of homes and businesses could possibly be affected. • Long ROW acquisition as some of the ROW acquisition required may result in projects extending well past 2011. • Massive amount of borrow required which would have to be obtained from offsite borrow sources.
2D	• Shortest length of protection system and therefore smallest risk of potential failure. • Low estimated construction cost totaling $681.3M. • Embankment fill required is far less than excavation for this alternative at 1.3M CY. If all material is suitable this could provide borrow to other projects.	• Largest number of structures results in highest estimated annual O&M costs of $3.3M (Common to all Alts. 2). • Total estimated construction duration of 3.5 years. • Required ROW of 192.7 acres. • Construction impacts to Section 404(c) wetlands area. • Sliding pocket gate has higher risk of failure than sector gates.
2E	• Shortest length of protection system and therefore smallest risk of potential. • Low estimated construction cost totaling $665.2M. • Sector gates have lower risk of failure than sliding pocket gate. • Embankment fill required is far less than excavation for this alternative at 1.3M CY. If all material is suitable this could provide borrow to other projects.	• Largest number of structures results in highest estimated annual O&M costs of $3.3M (Common to all Alts. 2). • Reoccurring O&M cost of $12M for pump replacement every 10 yrs (based on average costs of $750K per 1000 cfs pump). • Relatively long estimated construction duration (1,002 days) compared to other alternatives • Construction impacts to Section 404(c) wetlands area.
3B	• Lowest estimated construction costs of $582.2M. • Intermediate length of protection system and associated risk. • Smallest amount of required ROW required at (86.2 acres). • Embankment fill required is far less than excavation for this alternative at 0.7M CY. If all material is suitable this could provide borrow to other projects.	• High estimated annual O&M cost of $2.6M. • Total estimated construction duration of 5.0 years.

Other considerations in evaluating the alternatives:

- Fill availability: The USACE has noted that availability of local borrow sites may be depleted requiring fill for this project to be barged in from out of state locations.
- Project Life: Each of the alternatives has a gate(s) and pumping station within the protection system. The design life for these components is estimated to be a minimum of 50 years.
- Maintenance Intervals: Sector gate are generally dewatered and inspected every 10 to 15 years. This process requires that the navigation channel is closed for the maintenance period. Maintenance for the gates is generally limited to sandblasting and changing out the seals after 30 years. Similarly, maintenance for pumping stations includes pulling the impellers, welding and changing out light duty gasket hoses after 30 years. Annual maintenance for pumping stations includes changing oil in the pump engines and evaluating the need to replace stored diesel fuel.
- Qualitative Risk: Each of the four alternatives has one or more gate(s) which present a level of risk from failure of operation; however, the risk associated with each alternative is different based on the area that is being protected. Alternative 1 (Lapalco Sector Gate) presents the lowest level of risk due to operational failure as there are more levees and the gate is smaller than the gates in the other alternatives and is protecting the smallest area. However this alternative provides the system with the greatest length. All the Alternatives 2D and 2E present the highest level of risk because there are multiple gates and they are protecting the largest area. The sliding pocket gate in Alternative 2D has a higher risk if failure than the sector gates of Alternative 2E. Alternative 3, located upstream of the Alternatives 2 location, is protecting a smaller area; therefore the risk associated with operational failure of the sector gate is smaller.
- Redundancy: The same amount of electrical and mechanical redundancy is provided for the Alternatives 2D, 2E and 3B. The pumping stations have one spare pump each for redundancy, and to cover maintenance on other pumps. On site power

systems are provided with a back-up generator to the main generator to provide redundancy. For Alternative 1, there will be some redundancy built into some of the pumping stations, but due to limited funding there will not be redundant pumps provided; some pumping stations may get redundant generators.

12. Quality Implementation

12.1 Quality Control Plan

12.1.1 Design Quality Control Plan

This study and report were produced utilizing HNTB Corporation and the ARCADIS Bioengineering Joint Venture quality control procedures. Details of this procedure are found in the plan included in Appendix H to this report.

12.1.2 Independent Technical Review

An Independent Technical Review (ITR) was conducted three times for this EAR project, at the 65% Submittal, the Mod-01 65% Submittal and the 95% Submittal. In interest of meeting the schedule, the 65% reviews were conducted concurrent with the USACE reviews, and the 95% ITR was completed prior to the 95% Submittal. The report of the ITR comments and resolutions and Statement of Technical Review is included in Appendix I to this report.

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13. Appendix List

The following appendices are incorporated in the report. Appendices for the eliminated alternatives will be bound under a separate cover and will be included in the Final Submittal.

Appendix A	Hydraulic and Environmental Design Calculations, Final Study Alternatives 2D, 2E, 3B
Appendix B	Geotechnical Report
Appendix C	Structural Design Calculations, Final Study Alternatives 2D, 2E, 3B
Appendix D	Mechanical and Electrical Design Calculations, Final Study Alternatives 2D, 2E, 3B
Appendix E	Detailed Quantity and Cost Estimates and Supporting Documentation, Final Study Alternatives 2D, 2E, 3B
Appendix F	Construction Durations, Final Study Alternatives 2D, 2E, 3B
Appendix G	Plates, Final Study Alternatives 2D, 2E and 3B
Appendix H	Design Quality Control Plan
Appendix I	DrChecks Reports – ITR & USACE Reviews
Appendix J	Surveys
Appendix K	Brainstorming Results Presentation
Appendix L	Eliminated Alternatives Status Design Calculations
Appendix M	Detailed Quantity and Cost Estimates, Eliminated Alternatives
Appendix N	Construction Durations, Eliminated Alternatives
Appendix O	Plates of Eliminated Alternatives

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