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Engineering Alternatives Report – Revised Hydraulic Criteria Appendix for St. Bernard Parish Hurricane Protection System

St. Bernard Parish, LPV 144 – 149

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St. Bernard Parish,
LPV 144 – 149

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Executive Summary

The purpose of this Engineering Alternatives Report – Revised Hydraulic Criteria Appendix (EAR-RHCA) is to evaluate and determine the major items of work required to provide hurricane protection against a storm event with a 1 percent probability of occurrence (100-year) for Reaches LPV 144 through LPV 149 of the St. Bernard Parish, Louisiana. Under this Task Order, the tasks included in this EAR-RHCA are:

- Briefly describe existing hurricane protection features of LPV 144 through LPV 149;
- Describe alternatives for providing the required level of protection;
- Provide cost estimates for the alternatives;
- Identify potential right-of-way (ROW) and relocation requirements for the alternatives;
- Evaluate and recommend an alternative from those presented in the final scope of work to be carried forward for each reach;
- Perform planning for completion of the design and construction of the project; and
- Identify any deficiencies in survey, subsurface, hydraulics and hydrology, or other data that would impede progress on plans and specifications.

Existing Hurricane Protection System

The St. Bernard Parish Hurricane Protection System (HPS) in St. Bernard Parish, Louisiana, is comprised of earthen levee and floodwall reaches between the Bayou Bienvenue Control Structure and the Mississippi River at Caernarvon. The reaches are divided into sections and designated by the term Lake Pontchartrain and Vicinity (LPV) as listed below:

- LPV 144 - Bayou Dupre Control Structure between LPV 145 and 146;
- LPV 145 - Chalmette Loop Levee - Bayou Bienvenue to Bayou Dupre;

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- LPV 146 - Chalmette Loop Levee - Bayou Dupre to Highway 46;
- LPV 147 - Chalmette Loop Levee - Highway 46 and the Bayou Road Floodgate;
- LPV 148 - Chalmette Loop Levee - Verret to Caernarvon (including St. Mary's Pump Station); and
- LPV 149 - Chalmette Loop Levee - Caernarvon Floodwall and Floodgates.

The Inner Harbor Navigation Canal Barrier Project is proposed to cross over the Gulf Intracoastal Waterway and Mississippi River Gulf Outlet (MRGO) tying into the St. Bernard Parish HPS southeast of Bayou Bienvenue. Therefore, the proposed hurricane protection begins in LPV 145 within St. Bernard Parish and potentially eliminates the need to raise the Bayou Bienvenue Control Structure. If this assumption proves to be invalid, detailed designs will be developed to raise the structure to the appropriate 100-year elevation.

The existing hurricane protection features including reach lengths and approximate top of levee elevations in each of the reaches are shown below. All elevations shown in the table are in North American Vertical Datum of 1988 – epoch 2004.65.

Reach	Station Limits	Length (feet)	Existing Hurricane Protection Feature	Approximate Crest Elevation (feet NAVD88)
LPV 145	370+00.00 to 703+98.00	33,398.00	Earthen Levee	17.0~21.0
LPV 144	703+98.00 to 708+65.00	467.00	Sector Gate	15.4
LPV 146	708+65.00 to 1115+00.00	40,635.00	Earthen Levee	17.5~21.5
LPV 147	1115+00.00 to 1125+00.00	1,000.00	Earthen Ramp	15.0
LPV 148.01	1125+00.00 to 1195+16.50	7,016.50	Earthen Levee	19.0*
St. Mary's Pump Station	1195+16.50 to 1197+63.50	247.00	Floodwall	17.0

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Reach	Station Limits	Length (feet)	Existing Hurricane Protection Feature	Approximate Crest Elevation (feet NAVD88)
LPV 148.02	1197+63.50 to 1560+25.00	36,261.50	Earthen Levee	14.5~19.0*
LPV 149	1560+25.00 to 1581+60.00	2,135.00	Floodwall	14.0

* Existing levee crest elevation of LPV 148.01 and LPV 148.02 is assumed as 0.5-foot lower than Post-Katrina design grade due to settlement.

Project History

This EAR-RHCA is a revision to the original EAR submitted in October 2007. When the project began in April 2007, various alternatives were initially considered for reaches LPV 144 to 149. In LPV 144, three sector gate replacement/retrofit options were evaluated: replacement structure on the flood side or protected side; replacement structure at the existing location; or retrofitting the existing gate. In LPV 145, 146, and 148, T-walls and earthen levees with stability berms for existing (2010/2011) elevations of 22.5 and future design (2057/2060) up to 26.0 for levees and 31.0 for structures were considered. LPV 147 consisted of constructing a roadway ramp at Highway 46 and replacing the Bayou Road Floodgate. In LPV 149, three alignments were evaluated for the Caernavon floodwall: the existing alignment; flood-side shift and protected-side shift. During the initial design process, the levee alternatives for LPV 145, 146, and 148 were found to be quite extensive; therefore, new levee alternatives with foundation improvements were added to the EAR. Earthen levees using geotextiles, landside shifts, staged construction with and without wick drains, and deep soil mixing were considered along with those alternatives originally scoped. All alternatives up to this point used historical soils information obtained from the Design Memoranda.

During the finalization of the original EAR, new geotechnical data and a revised hydraulic model became available in September and October of 2007. The geotechnical data included undisturbed borings, laboratory testing, and cone penetrometer tests (CPT) in LPV 145, 146, and 148. Some strength gain was recognized in certain reaches according to the results of these borings while some reaches were similar to the historical data. The revised hydraulic model indicated higher design elevations than used in the original EAR in all of the reaches. Therefore, in LPV 144, the design was altered to accommodate the new top of structure elevations. In LPV 145, 146, and 148, the four most feasible alternatives (T-walls, earthen levees with stability berms, earthen levees with deep soil mixing,

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and earthen levees with staged construction) were carried forward and incorporated the new soils and hydraulic design input. In LPV 147, a second alternative, construction of a bridge over a T-wall, was considered in addition to the Highway 46 roadway ramp. The Bayou Road Floodgate would be removed and replaced with a T-wall or earthen option from LPV 148 rather than replacing the gate. Reasons to close off the gate included the reduction in traffic on Bayou Road as a result of its roadway closure to the southwest of the gate and maintenance issues associated with the gate as communicated by the local sponsors. In LPV 149, the alignments were revised for the new design elevations.

In November 2007, during discussions related to the alternatives in LPV 145, 146, and 148, new alternatives were decided to be studied considering the effect of wave berms and scour protection on design elevations. The presence of wave berms and scour protection can lower design elevations; however, the additional flood-side weight from them can cause greater stability issues for both the flood side and protected side. Therefore, various T-walls and levee sections using combinations of different size wave berms with and without scour protection were developed by the U.S. Army Corps of Engineers (USACE) in January 2008 as a supplement to the October 2007 Hydraulic Criteria Report. Because many new alternatives were being considered, an interim report was developed in March 2008 in order to eliminate the alternatives that were not feasible to carry forward to the EAR-RHCA stage. Limited and conceptual analyses were performed based on previous analysis, and several alternatives were eliminated. A joint selection meeting was held to determine the most feasible alternatives to carry forward. Two alternatives, T-walls with armoring but without a wave berm on the existing levee ("B2" alternative) and earthen levees with short, slope berm (Oct 07 alternative), were selected to be included in the final EAR-RHCA along with the earthen levees with deep soil mixing and staged construction with wick drains for LPV 145, 146, and 148. For LPV 149, a fourth alignment was added during this time frame, which was preferred by the local community. At the joint selection meeting, only two of the four alignments, Alignments 2 and 4, were to be evaluated further for the EAR-RHCA. The final elevations used for both the T-walls and the levees are as follows:

T-wall Design Elevations

Hydraulic Reach Segment	Description	2057/2060 1% Design Elevation	2057/2060 Flood-Side Toe Design Elevation
SB11	T-wall on levee, armored, no berm	32.0	19.9
SB15	T-wall on levee, armored, no berm	27.5	17.1
SB16	T-wall on levee, armored, no berm	29.5	19.0

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Hydraulic Reach Segment	Description	2057/2060 1% Design Elevation	2057/2060 Flood-Side Toe Design Elevation
SB19	Bayou Dupre Control Structure	31.0	NA
SB20	St. Mary Pump Station	30.5	NA
SB21	Caernarvon Floodwall	24.0	19.5

Information regarding Hydraulic Reaches SB12, SB13, and SB17 was not provided in revised Hydraulic Report HPO-LFA-0091-07. Therefore, the USACE suggested the use of SB11, SB15, and SB16 for SB12, SB13, and SB17, respectively, for the T-wall alternative study.

For levees, both the 2010/2011 and 2057/2060 elevations were present as shown in the table below, but do not include overbuild. Overbuild was determined from settlement analysis and discussed in the individual results sections of this report.

Earthen Levee Design Elevations

Hydraulic Reach	2057/2060 1% Design Elevation	2057/2060 Still Water Elevation	2010/2011 1% Design Elevation	2010/2011 Still Water Elevation	Normal Water Elevation for Flood-Side Checks
SB11	31.5	21.2	29.0	19.7	-1.0
SB12	30.0	20.2	27.5	18.7	-1.0
SB13	29.0	19.3	26.5	17.8	-1.0
SB15	29.0	18.6	26.5	17.1	-1.0
SB16	29.0	20.4	26.5	18.9	-1.0
SB17	29.0	21.0	26.5	19.5	-1.0

The Draft EAR-RHCA was submitted in June 2008. This document was reviewed by the Government as well as the Independent Technical Review (ITR) team. Comments, resolutions, and final recommendations are included in this EAR-RHCA. In addition, real estate costs, not part of the original EAR, and a detailed risk analysis of the cost and construction schedule on the selected alternatives are included in this EAR-RHCA.

Also in June 2008, the Final ITR for Existing Soils Reports in LPV 145 and 146 was completed. This effort was performed between February and June 2008 concurrent

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with the EAR. Because different entities had conducted the drilling and testing of the data used for LPV 145 and 146, there was general concern regarding the consistency and quality of the data as it relates to its use in design. The purpose of the ITR of the Existing Soils Reports was to review the quality assurance (QA) reports and assess the status of the QA comment resolutions, summarize the results of the ITR, and determine the tasks necessary to complete the Existing Soils Reports for LPV 145 and 146 including recommending additional testing if necessary to meet Mississippi Valley Division and USACE standards. The results of the ITR indicated that some of the data should not be used for design, while other data just need modifications to the lab reports. The data that were not useful were not used in the shearlines for this EAR-RHCA. Select CPTs were also evaluated with the shearlines to validate the strengths used to make decisions for this EAR-RHCA. Additional borings and testing will be obtained as a result of the ITR and the final recommendations of this EAR-RHCA.

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Proposed Alternatives for EAR-RHCA

Alternatives were investigated for the levees and floodwalls within these reaches to provide recommendations on the most feasible solution to achieve this level of protection plus overbuild. The USACE provided recently completed field and laboratory testing data, historical data via original design memoranda, construction plans, as-built plans, Task Force Guardian construction plans, and existing survey data. All analysis and design were based on this information.

The levees and floodwalls within the St. Bernard Parish HPS are proposed to be raised to a design protection level of elevation (EL) between EL 24.0 and 31.5 (North American Vertical Datum of 1988 epoch 2004.65 – all references to elevations will be to this datum) depending upon the reach. These reaches were broken up by geometry (LPV), hydraulic conditions (Hydraulic Reach), and geotechnical considerations (Soils Reach), which are summarized in the table below:

LPV Reach	Soils Reach	Hydraulic Reach	Station Limits	Length (feet)
LPV 145	A	SB11	370+00.00 to 475+04.50	10,504.50
	A	SB12	475+04.50 to 525+82.82	5,078.32
	A	SB13	525+82.82 to 703+98.00	17,815.18
LPV 144	AB	SB19	703+98.00 to 708+65.00	467.00
LPV 146	B	SB13	708+65.00 to 821+22.61	11,257.61

LPV Reach	Soils Reach	Hydraulic Reach	Station Limits	Length (feet)
	B	SB15	821+22.61 to 965+00.00	14,377.39
	C	SB15	965+00.00 to 1008+00.00	4,300.00
	D	SB15	1008+00.00 to 1070+82.23	6,282.23
	D	SB16	1070+82.23 to 1115+00.00	4,417.77
LPV 147	D	SB16	1115+00.00 to 1125+00.00	1,000.00
LPV 148	E	SB16	1125+00.00 to 1354+38.88*	22,938.88
	E	SB17	1354+38.88 to 1537+00.00	18,261.12
	F	SB17	1537+00.00 to 1560+25.00	2,325.00
LPV 149	G	SB21	1560+25.00 to 1581+60.00	2,135.00

*St. Mary's Pump Station is located between Station 1195+16.50 and Station 1197+63.50.

The following is a summary of the alternatives that were investigated for this EAR-RHCA:

LPV 144 – Bayou Dupre Control Structure - The proposed work for this reach considered a new sector gate structure located on the flood side of the existing structure. The new top of structure is EL 31.0 based on the revised hydraulic criteria. The quantities that were developed for the original EAR were extrapolated from the design elevation of 26.0 to the new elevation. The cost estimate and construction schedule were revised to reflect these modifications. The pontoon bridge design and quantities remained unchanged.

LPV 145 – Chalmette Loop Levee – Bayou Bienvenue to Bayou Dupre - For this reach, the existing flood protection consists of earthen levees only. For the proposed work, the following flood protection alternatives were considered for Hydraulic Reaches SB11, SB12, and SB13:

(1) T-wall alternative on existing levee;

(2) Earthen levees using stability berms (Oct 07 Alternative);

(3) Earthen levees using deep soil mixing and landside shift; and

(4) Earthen levees using stability berms with staged construction and wick drains.

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The above hydraulic reaches (SB11, SB12, and SB13) have design elevations for both existing (2010/2011) and future (2057/2060) conditions. Alternative 1, the T-wall alternative on existing levee, is designed for future conditions, because T-walls cannot be modified as easily as levees. Alternative 2, earthen levees using stability berms, was initially designed for the existing condition and then has subsequent lifts to reach future conditions by 2057/2060. Both Alternatives 3 and 4 have elevations corresponding to future conditions by 2057/2060. Because of the nature of Alternatives 3 and 4, the foundation must be designed to accommodate the future design elevations and was the reason for investigating these alternatives up to this level; however, interim or lower height lifts for the existing condition could be investigated after the foundation is prepared for future conditions. All these alternatives used one soils reach (Reach A: Stations 370+00 to 703+98) based on the updated geotechnical undisturbed borings, CPT, and field and laboratory testing. One soils reach is considered appropriate for this level of study and the similarity of soil profile in LPV 145.

<u>LPV 146 – Chalmette Loop Levee – Bayou Dupre to Highway 46</u> - For this reach, the existing flood protection consists of earthen levees only. For the proposed work, the following flood protection alternatives were considered for Hydraulic Reaches SB13, SB15, and SB16:

(1) T-wall alternative on existing levee;

(2) Earthen levees using stability berms (Oct 07 Alternative);

(3) Earthen levees using deep soil mixing and landside shift; and

(4) Earthen levees using stability berms with staged construction and wick drains.

Similar to LPV 145, the above hydraulic reaches (SB13, SB15, and SB16) have design elevations for both existing (2010/2011) and future (2057/2060) conditions. The design elevations used for the above alternatives followed the same logic as described for LPV 145. Also, for all alternatives, three soils reaches (Reach B: Stations 708+65 to 965+00, Reach C: Stations 965+00 to 1008+00, and Reach D in LPV 146: Stations 1008+00 to 1115+00) were considered based on the updated geotechnical undisturbed borings, CPT, and field and laboratory testing. LPV 146 was broken up into three soils reaches because of the difference in soil properties along the MRGO and then after the levee turn.

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LPV 147 – Chalmette Levee Loop – Highway 46 and Bayou Road Floodgate - This reach consists of the levee section at the Highway 46 Crossing and Bayou Road Floodgate. For Highway 46, two alternatives were considered:

(1) Highway 46 Ramp (Earthen Levee); and

(2) Bridge over T-wall.

The ramp alternative would be grouped with the earthen levee alternative used for LPV 146 and LPV 148. For the bridge alternative, the structure would span the T-wall alternative used from LPV 148. The Bayou Road Floodgate would be demolished and replaced with a T-wall. The Bayou Road traffic will be rerouted onto Highway 46 on either side of the protection with the use of new access roads.

LPV 148 – Chalmette Loop Levee – Verret to Caernarvon - For this reach, the existing flood protection consists of an earthen levee, St. Mary's Pump Station, and the Creedmore Drainage Structure. For the existing earthen levees, the following flood protection alternatives were considered for Hydraulic Reaches SB16 and SB17:

(1) T-wall alternative on existing levee;

(2) Earthen levees using stability berms (Oct 07 Alternative);

(3) Earthen levees using deep soil mixing and landside shift; and

(4) Earthen levees using stability berms with staged construction and wick drains.

Similar to LPV 145 and LPV 146, the above hydraulic reaches (SB16 and SB17) have design elevations for both existing (2010/2011) and future (2057/2060) conditions. The design elevations used for the above alternatives followed the same logic as described for LPV 145 and 146. Two soils reaches (Reach E: Stations 1125+00 to 1537+00 and Reach F: Stations 1537+00 to 1560+25) were considered for this EAR-RHCA based on the updated geotechnical undisturbed borings, CPT, and field and laboratory testing. The soils reaches were broken into two for LPV 148 because of the differences in soil profile.

The fronting protection at St. Mary's Pump Station was evaluated using Hydraulic Reach SB20. Due to the significant rise in water level from the existing design,

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demolition and replacement of the existing T-walls were considered. For the Creedmore Drainage Structure, the existing structure will be demolished as requested by the sponsor because it is not being used.

LPV 149 – Chalmette Loop Levee – Caernarvon Floodwall and Floodgates - For this reach, the existing flood protection consists of a combination of concrete-capped I-walls, one railroad gate, and the Highway 39 vehicular gate.

The final modifications to the scope involved alternatives to be carried forward to the EAR stage that can be summarized as follows:

- Alignment 2 T-wall: Constructed to EL 24.0 along an offset (approximately 25 feet) alignment west of existing; and

- Alignment 4 T-wall: Constructed to EL 24.0 along an offset alignment that crosses the Caernarvon Canal (west of Elevated Boat Industries) and ties into the Mississippi River Levee.

All geotechnical analysis for the alternatives in most reaches was performed using the most recent and acceptable subsurface information provided by the USACE in September 2007. Design levee elevations were altered to include overbuild over the design life of 50 years. The geotechnical analysis for the levee reaches resulted in overbuilds on the order of 1.8 to 5.1 feet in some areas. The wide range in overbuild is due to alternative, lift schedule over 50 years, and where the centerline of the levee is placed with respect to the existing levee. Weak foundation soils, reach location, depth of the deep soil-mixing regions, landside shifts, and the effects of the wick drains are all contributors and variables to settlement and overbuild values. With landside shifts and alignments where an existing levee was not present, settlements were greater than if the new levee was raised at the centerline of the existing levee unless deep soil mixing was considered. A full description of the analysis results is provided in detail in later sections.

Variations in levee slopes and dimensions are based on slope stability, seepage, and settlement of improved and unimproved sections (berm enlargement, deep soil mixing, wick drains, staged construction, and guidance included in the scope of work). For scour protection, specific direction from hydraulics was not available at the time of this report and is still being investigated by the USACE. Therefore, the assumption of 18 inches of riprap from protected-side levee toe to flood-side levee toe was used as discussed with the USACE to include for costing and analysis

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purposes for this EAR-RHCA. The initial sections as shown in the Scope of Work were modeled first. In the majority of cases, additional stability berms or deeper soil-mixed regions were required to achieve acceptable geotechnical factors of safety. The calculated factors of safety associated with varying conditions (alternatives, water levels, soil reaches, etc.) were summarized. Detailed explanations of each levee system are presented in greater depth in subsequent sections.

From a program perspective, it is known that the sub-issues of ROW acquisition, borrow material, and cost sharing requirements are at the forefront of the decision-making process for the USACE. ROW acreages were estimated based on the requirements of the footprint of each alternative.

Borrow material source, proximity, and cost are program-wide concerns. The cost estimates account for the challenges associated with these issues. The recommended alternatives presented below will require minimal borrow as compared to more traditional methods of levee enlargement.

Results & Conclusions

Detailed narratives for each reach are found in the report including structural, geotechnical, civil, utility relocations, ROW requirements, drawings, cost estimates, construction schedule summaries, and rationale for the selection of the recommended alternatives. A summary of the alternatives (not including escalation) considered and the associated estimates of probable cost, based on the final revised Scope of Work, is as follows:

Description	Real Estate ($k)	Environmental Mitigation Including Contingency ($k)	Contract Cost Including Contingency ($k)	Engineering and Design Cost Including Contingency ($k)	Supervision and Administration Cost Including Contingency ($k)	Project Cost ($k)	Operation and Maintenance ($k)	Total Present Values ($k)
	1	2	3	4	5	1+2+3+4+5 A	B	= A + B
LPV 144, Bayou Dupre Control Structure to EL 31.0	-	-	$62,709	$7,838	$5,957	$76,504	$1,622	**$78,126**
LPV 145, T-wall	$485	$107	$479,681	$59,960	$45,570	$585,802	$9,867	**$595,670**

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Description	Real Estate ($k)	Environmental Mitigation Including Contingency ($k)	Contract Cost Including Contingency ($k)	Engineering and Design Cost Including Contingency ($k)	Supervision and Administration Cost Including Contingency ($k)	Project Cost ($k)	Operation and Maintenance ($k)	Total Present Values ($k)
	1	2	3	4	5	1+2+3+4+5 A	B	= A + B
LPV 145, Deep Soil Mixing	-	-	$746,110	$93,264	$70,880	$910,254	$45,594	**$955,847**
LPV 145 Staged with Wick Drain – All Stages	$3,492	$9,195	$858,078	$107,260	$81,517	$1,059,542	$69,328	**$1,128.870**
LPV 145, Oct 07 All Stages	$6,305	$4,264	$644,044	$80,505	$61,184	$796,302	$82,257	**$878,560**
LPV 146, T-wall	-	-	$449,798	$56,225	$42,731	$548,754	$8,294	**$557,048**
LPV 146, Deep Soil Mixing	-	-	$706,146	$88,268	$67,084	$861,499	$70,586	**$932,085**
LPV 146, Staged with Wick Drain All Stages	-	-	$701,100	$87,638	$66,605	$855,342	$56,399	**$911,741**
LPV 146, Oct 07 All Stages	$1.34	$110	$788,028	$98,503	$74,863	$961,505	$74,745	**$1,036,250**
LPV 147, Highway 46 Ramp	$2,527	-	$55,968	$6,393	$5,317	$70,205	-	**$70,205**
LPV 147, Bridge	-	-	$22,029	$2,202	$1,674	$26,406	$500	**$26,906**
LPV 148, T-wall	$0.65	$53	$643,685	$80,461	$61,150	$785,349	$11,403	**$796,752**
LPV 148, Deep Soil Mixing	$327	$7,297	$1,060,760	$132,595	$100,772	$1,301,752	$69,071	**$1,370,823**
LPV 148, Staged with Wick Drain All Stages	$622	$31,462	$1,025,171	$128,146	$97,391	$1,282,793	$75,915	**$1,358,708**
LPV 148, Oct 07 All Stages	$594	$29,148	$1,132,529	$141,566	$107,590	$1,411,426	$106,976	**$1,518,403**

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Description	Real Estate ($k)	Environmental Mitigation Including Contingency ($k)	Contract Cost Including Contingency ($k)	Engineering and Design Cost Including Contingency ($k)	Supervision and Administration Cost Including Contingency ($k)	Project Cost ($k)	Operation and Maintenance ($k)	Total Present Values ($k)
	1	2	3	4	5	1+2+3+4+5 A	B	= A + B
LPV 149 – Alt 2 Mod 4, T-wall	$278	$8,285	$40,713	$6,125	$4,655	$60,055	$2,042	**$62,097**
LPV 149 – Alt 4 Mod 4, T-wall	$6.32	$8,808	$100,551	$13,605	$10,340	$133,309	$2,729	**$136,038**

NOTES:
1. Values for second and third construction stages included in multi-lift alternative LPVs are 2008 dollars.
2. Escalation to midpoint of construction is not included in these calculations.
3. "Present Value" calculations were only considered in the Operation and Maintenance cost.
4. See project summary sheets for additional details.

Upon evaluation of these alternatives, the following are recommended to be carried forward into final design:

- LPV 144: Sector Gate to EL 31.0 on flood side of the existing;
- LPV 145: T-walls;
- LPV 146: T-walls;
- LPV 147: Bridge over T-wall;
- LPV 148: T-walls; and
- LPV 149: T-wall Alignment 2.

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Estimates of probable cost for the final recommended alternatives including **contingencies and escalation** are provided below:

Description	Real Estate ($k) 1	Environmental Mitigation ($k) 2	Contract Cost ($k) 3	Engineering and Design Cost ($k) 4	Supervision and Administration Cost ($k) 5	Project Cost ($k) 1+2+3+4+5 A	Operation and Maintenance ($k) B	Total Values ($k) A+B
LPV 144, Bayou Dupre	-	-	$62,709	$7,839	$5,957	$76,506	$1,622	$78,127
LPV 145, T-wall	$506	$111	$551,008	$62,680	$52,346	$666,651	$9,867	$676,518
LPV 146, T-wall	-	-	$510,693	$58,775	$48,516	$617,984	$8,294	$626,278
LPV 147, Bridge	-	-	-	-	-	-	-	$26,406
LPV 148, T-wall	$0.679	$55	$750,154	$84,110	$71,264	$905,585	$11,402	$916,988
LPV 149, Alt 2/Mod 4 T-wall	$289	$9,165	$45,033	$6,402	$5,149	$66,038	$2,042	$68,080
					TOTAL	**$1,589,607**	**$21,738**	**$2,392,397**

A summary of the total design and construction durations for the recommended alternatives is listed in the table below. Full Gantt chart schedules for the recommended alternatives are provided in this report.

Description	Project Start Date (Plans and Specifications)	Construction Start Date	Project Acceptance Date	Total Construction Duration (Days)
LPV 144, Sector Gate to EL 31.0	6/11/08	7/29/09	5/27/11	510
LPV 145, T-wall	12/18/08	12/31/09	9/25/12	937
LPV 146, T-wall	12/18/08	12/31/09	2/22/12	721
LPV 147, Bridge	12/18/08	11/9/09	6/20/11	394

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Description	Project Start Date (Plans and Specifications)	Construction Start Date	Project Acceptance Date	Total Construction Duration (Days)
LPV 148, T-wall	12/18/08	12/31/09	8/9/13	1255
LPV 149 – Alt 2/ Mod 4, T-wall	12/18/08	11/7/09	1/12/11	279

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The following conclusions can be reached from the evaluation of the alternatives presented in this EAR-RHCA for the St. Bernard Parish Hurricane Protection System – LPV 144 through LPV 149:

- Based on topography and subsurface soil conditions (using historical data), a variety of alternatives were recommended for the study reaches;
- Optimization of the recommended alternative, including the estimates of probable cost, will be possible during a final design phase and shall be based on recent survey and subsurface data;
- The subsurface conditions in this region present challenging scenarios to provide flood protection to the required 100-year flood elevations;
- Additional topographic and hydrographic survey information will be necessary for all recommended alternatives;
- The revised alternatives were an improvement over the original study. Some cost savings were realized as the alternatives were refined; and
- Additional subsurface investigation will be necessary to complete the final design for any of the alternatives in LPV 145 through LPV 149. Additional investigations should incorporate geophysical methods, CPT, and undisturbed borings. The USACE has been conducting subsurface investigations along the Federal levees. However, for this report, some of the results of these investigations were not available to the design team and thus were not incorporated in the studies.

In conclusion, while this EAR-RHCA presents viable and constructible alternatives for the St. Bernard Parish HPS, further refinement/optimization of the recommended or

final government selected alternatives may be realized during the plans and specifications phase. As noted in the Executive Summary, from a program perspective, it is known that the sub-issues of ROW acquisition, borrow material, and cost sharing requirements are at the forefront of the decision-making process for the USACE. Borrow material source, proximity, and cost are program-wide concerns. The design team is ready to assist the USACE in all aspects of the final stages of planning and the decision-making process.

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1. Introduction

St. Bernard Parish is located in southeastern Louisiana, east of and adjacent to the city of New Orleans. Approximately two-thirds of the parish is surrounded by water and consists primarily of marshlands formed by the Mississippi River Delta. This is one of the largest and richest wetland ecosystems on the North American continent. The area of the parish along the western portion on the northern shore (east bank) of the Mississippi River has the highest elevation and heaviest population. Eastern portions of St. Bernard Parish consist of largely fresh, brackish, and saline marshes. Remnants of natural topographic ridges can be found along the existing or abandoned courses of river distributaries, or bayous. Other ridges, or cheniers, are found along abandoned coastlines in isolated areas of the marshes. Barrier islands and beaches, in particular the Chandeleur and Breton islands, can be found at the edge of the Old St. Bernard delta.

Portions of the Lake Pontchartrain and Vicinity (LPV) Hurricane Protection System (HPS) fall within St. Bernard Parish as shown in the Overall Site Plan (Figure 1). The St. Bernard Parish HPS, which is comprised of approximately 25 miles of levees and floodwalls, extends from the Bayou Bienvenue Control Structure, continuing along the Mississippi River Gulf Outlet (MRGO) southeasterly, then turns generally to the west, where it ties into the Mississippi River Levee System at Caernarvon. The LPV reaches are divided into sections and designated as listed below:

- LPV 144 - Bayou Dupre Control Structure;
- LPV 145 - Chalmette Loop Levee - Bayou Bienvenue to Bayou Dupre;
- LPV 146 - Chalmette Loop Levee - Bayou Dupre to Highway 46;
- LPV 147 - Chalmette Loop Levee - Highway 46 and Bayou Road Floodgate;
- LPV 148 - Chalmette Loop Levee - Verret to Caernarvon (including St. Mary's Pump Station); and
- LPV 149 - Chalmette Loop Levee - Caernarvon Floodwall and Floodgates.

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The Inner Harbor Navigation Canal Barrier Project is proposed to cross over the Gulf Intracoastal Waterway and Mississippi River Gulf Outlet (MRGO) tying into the St. Bernard Parish HPS southeast of Bayou Bienvenue. Therefore, the proposed hurricane protection begins in LPV 145 within St. Bernard Parish and potentially eliminates the need to raise the Bayou Bienvenue Control Structure. If this assumption proves to be invalid, detailed designs will be developed to raise the structure to the appropriate 100-year elevation.

A summary of the existing hurricane protection by reach is presented in the following table. All elevations shown in the table are in North American Vertical Datum of 1988 – epoch 2004.65.

Reach	Station Limits	Length (feet)	Existing Hurricane Protection Feature	Approximate Crest Elevation (feet NAVD88)
LPV 145	370+00.00 to 703+98.00	33,398.00	Earthen Levee	17.0~21.0
LPV 144	703+98.00 to 708+65.00	467.00	Sector Gate	15.4
LPV 146	708+65.00 to 1115+00.00	40,635.00	Earthen Levee	17.5~21.5
LPV 147	1115+00.00 to 1125+00.00	1,000.00	Earthen Ramp	15.0
LPV 148.01	1125+00.00 to 1195+16.50	7,016.50	Earthen Levee	19.0*
St. Mary's Pump Station	1195+16.50 to 1197+63.50	247.00	Floodwall	17.0
LPV 148.02	1197+63.50 to 1560+25.00	36,261.50	Earthen Levee	14.5~19.0*
LPV 149	1560+25.00 to 1581+60.00	2,135.00	Floodwall	14.0

* Existing levee crest elevation of LPV 148.01 and LPV 148.02 is assumed as 0.5 foot lower than Post-Katrina design grade due to settlement.
NAVD88 North American Vertical Datum, 1988

The majority of levees in this system were overtopped or breached during Hurricane Katrina. Task Force Guardian (TFG) repaired the Bayou Dupre Control Structure in LPV 144 and the levees in LPV 145 and LPV 146. The remaining reaches were inspected in April 2006. Several deficiencies were discovered and presented in a levee assessment report prepared by ARCADIS in May 2006. These deficiencies ranged in nature from scour holes and desiccation cracks to areas

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sloughed below design grade and rutting. Upon completion of the levee assessments and the profile surveys, the bulk of the levee system in reaches LPV 147 through LPV 149 was found to be greater than 1 foot below design grade, which led to a “not acceptable” rating. In October 2007, construction of an additional lift began in LPV 148.01, which is currently ongoing, to raise the levee to an acceptable level. The construction plans have been included in the design of the proposed hurricane protection.

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2. Purpose and Scope of Study

The purpose of the original scope for this study was to evaluate and determine the major items of work required to provide hurricane protection against a storm event with a 1 percent probability of occurrence (100-year) for reaches LPV 144 through LPV 149 of the St. Bernard Parish HPS. The 1 percent probability of occurrence accounts for regional subsidence and sea level rise. Under this Task Order, the tasks included in this Engineering Alternatives Report – Revised Hydraulic Criteria Appendix (EAR-RHCA) are:

- Briefly describe existing hurricane protection features of LPV 144 through LPV 149;
- Describe alternatives for providing the required level of protection;
- Provide cost estimates for the alternatives;
- Identify potential right-of-way (ROW) and relocation requirements for the alternatives;
- Evaluate and recommend an alternative from those presented in the final scope of work to be carried forward for each reach;
- Perform planning for completion of the design and construction of the project; and
- Identify any deficiencies in survey, subsurface, hydraulics and hydrology, or other data that would impede progress on plans and specifications.

The U.S. Army Corps of Engineers (USACE) provided access to recently completed field and laboratory testing data, historical data such as design memoranda (DMs), construction plans, as-built plans, TFG construction plans, and existing survey data. All other references used in this study are provided in later sections. All analyses and designs were based on this information. The soil design parameters were primarily based on recently available testing data.

By developing this EAR-RHCA, the USACE will be able to make a sound decision on a path forward to develop final designs and produce plans and specifications for the preferred alternatives. Recently, the USACE has undertaken an extensive survey

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and subsurface exploration program to provide new data for the topographic features, existing contours, channel geometry, and soil strata. In a final design phase, this new soil and survey information will be used to refine the designs completed for the EAR-RHCA.

2.1 Project History

This EAR-RHCA is a revision to the original EAR submitted in October 2007. When the project began in April 2007, various alternatives were initially considered for reaches LPV 144 to 149. In LPV 144, three sector gate replacement/retrofit options were evaluated: replacement structure on the flood side or protected side; replacement structure at the existing location; or retrofitting the existing gate. In LPV 145, 146, and 148, T-walls and earthen levees with stability berms for existing (2010/2011) elevations of 22.5 and future design (2057/2060) elevations up to 26.0 for levees and 31.0 for structures were considered. LPV 147 consisted of constructing a roadway ramp at Highway 46 and replacing the Bayou Road Floodgate. In LPV 149, three alignments were evaluated for the Caernavon floodwall: the existing alignment; flood-side shift and protected-side shift. During the initial design process, the levee alternatives for LPV 145, 146, and 148 were found to be quite extensive; therefore, new levee alternatives with foundation improvements were added to the original EAR. Earthen levees using geotextiles, landside shifts, staged construction with and without wick drains, and deep soil mixing were considered along with those the alternatives originally scoped. All alternatives up to this point used historical soils information obtained from the DM.

During the finalization of the original EAR, new geotechnical data and a revised hydraulic model became available in September and October 2007. The geotechnical data included undisturbed borings, laboratory testing, and cone penetrometer tests (CPT) for LPV 145, 146, and 148. Some strength gain was recognized in certain reaches according to the results of these borings while some reaches were similar to the historical data. The revised hydraulic model indicated higher design elevations than used in the original EAR in all of the reaches. Therefore, in LPV 144, the design was altered to accommodate the new top of structure elevations. In LPV 145, 146, and 148, the four most feasible alternatives (T-walls, earthen levees with stability berms, earthen levees with deep soil mixing, and earthen levees with staged construction) were carried forward and incorporated the new soils and hydraulic design input. Design levee elevations increased from 26.0 up to 31.0. In LPV 147, a second alternative, construction of a bridge over a T-wall, was considered in addition to the Highway 46 roadway ramp. The Bayou Road Floodgate would be removed and replaced with a T-wall or earthen option from LPV 148 instead of replacing the gate. Reasons to close off the gate included the reduction in traffic on Bayou Road as a result of its roadway closure to

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the southwest of the gate and maintenance issues associated with the gate as communicated by the local sponsors. In LPV 149, the alignments were revised for the new design elevations.

In November 2007, during discussions related to the alternatives in LPV 145, 146, and 148, new alternatives were decided to be studied considering the effect of wave berms and scour protection on design elevations. The presence of wave berms and scour protection can lower design elevations; however, the additional flood-side weight from them can cause greater stability issues for both the flood side and protected side. Therefore, various T-walls and levee sections using combinations of different size wave berms with and without scour protection were developed by the USACE in January 2008 as a supplement to the October 2007 Hydraulic Criteria Report (USACE). Because many new alternatives were being considered, an interim report was developed in March 2008 in order to eliminate the alternatives that were not feasible to carry forward to the EAR-RHCA stage. Limited and conceptual analyses were performed based on previous analysis, and several alternatives were eliminated. A joint selection meeting was held to determine the most feasible alternatives to carry forward. Two alternatives, T-walls with armoring but without a wave berm on the existing levee ("B2" alternative) and earthen levees with short, slope berm (Oct 07 alternative), were selected to be included in the EAR-RHCA along with the earthen levees with deep soil mixing and staged construction with wick drains for LPV 145, 146, and 148. For LPV 149, a fourth alignment was added during this time frame, which was preferred by the local community. At the joint selection meeting, only two of the four alignments, Alignments 2 and 4, were to be evaluated further for the EAR-RHCA.

The Draft EAR-RHCA was submitted in June 2008. This document was reviewed by the Government as well as the Independent Technical Review (ITR) team. Comments, resolutions, and final recommendations are included in this EAR-RHCA. ITR comment certifications are presented in Appendix A. In addition, real estate costs, not part of the original EAR, and a detailed risk analysis of the cost and construction schedule on the selected alternatives (Appendix J) are included in this EAR-RHCA.

Also in June 2008, the Final ITR for Existing Soils Reports in LPV 145 and 146 was completed. This effort was performed between February and June 2008 concurrent with the EAR. Because different entities had conducted the drilling and testing of the data used for LPV 145 and 146, there was general concern regarding the consistency and quality of the data as it relates to its use in design. The purpose of the ITR of the Existing Soils Reports was to review the quality assurance (QA) reports and assess the status of the QA comment resolutions, summarize the results of the ITR, and determine the tasks necessary to complete the Existing Soils Reports for LPV 145 and

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146 including recommending additional testing if necessary to meet Mississippi Valley Division and USACE standards. The results of the ITR indicated that some of the data should not be used for design, while other data just need modifications to the lab reports. The data that were not useful were not used in the shearlines for this EAR-RHCA. Select CPTs were also evaluated with the shearlines to validate the strengths used to make decisions for this EAR. Additional borings and testing will be obtained as a result of the ITR and the final recommendations of this EAR-RHCA.

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2.2 Description of Alternatives

Various alternatives were investigated within each LPV reach, which were broken up to consider the geometric, hydraulic, and geotechnical considerations. The locations of these LPV, hydraulic, and soils reaches are presented on the Overall Site Plan (Figure 1) and are summarized in the following table:

LPV Reach	Soils Reach	Hydraulic Reach	Station Limits	Length (feet)
LPV 145	A	SB11	370+00.00 to 475+04.50	10,504.50
	A	SB12	475+04.50 to 525+82.82	5,078.32
	A	SB13	525+82.82 to 703+98.00	17,815.18
LPV 144	AB	SB19	703+98.00 to 708+65.00	467.00
LPV 146	B	SB13	708+65.00 to 821+22.61	11,257.61
	B	SB15	821+22.61 to 965+00.00	14,377.39
	C	SB15	965+00.00 to 1008+00.00	4,300.00
	D	SB15	1008+00.00 to 1070+82.23	6,282.23
	D	SB16	1070+82.23 to 1115+00.00	4,417.77
LPV 147	D	SB16	1115+00.00 to 1125+00.00	1,000.00
LPV 148	E	SB16	1125+00.00 to 1354+38.88*	22,938.88
	E	SB17	1354+38.88 to 1537+00.00	18,261.12
	F	SB17	1537+00.00 to 1560+25.00	2,325.00
LPV 149	G	SB21	1560+25.00 to 1581+60.00	2,135.00

* St. Mary's Pump Station is located between Station 1195+16.50 and Station 1197+63.50.

A general description of the alternatives considered for this EAR-RHCA in each of the LPV reaches is presented below:

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LPV 144 – Bayou Dupre Control Structure - The proposed work for this reach considered a new sector gate structure located on the flood side of the existing structure. The new top of structure is elevation (EL) 31.0 based on the revised hydraulic criteria. The quantities that were developed for the original EAR were extrapolated from the design elevation of 26.0 to the new elevation. The cost estimate and construction schedule were revised to reflect these modifications. The pontoon bridge design and quantities remained the unchanged.

LPV 145 – Chalmette Loop Levee – Bayou Bienvenue to Bayou Dupre - For this reach, the existing flood protection consists of earthen levees only. For the proposed work, the following flood protection alternatives were considered for Hydraulic Reaches SB11, SB12, and SB13, which have design elevations for both existing (2010/2011) and future (2057/2060) conditions:

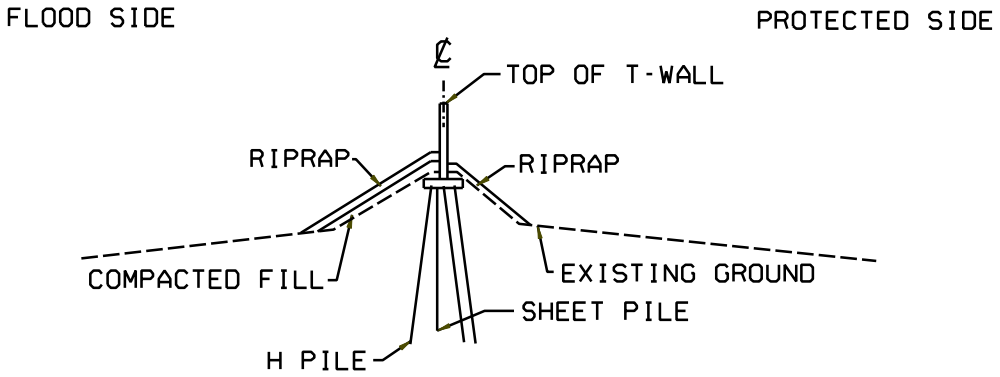
(1) T-wall alternative on existing levee;

(2) Earthen levees using stability berms (Oct 07 Alternative);

(3) Earthen levees using deep soil mixing and landside shift; and

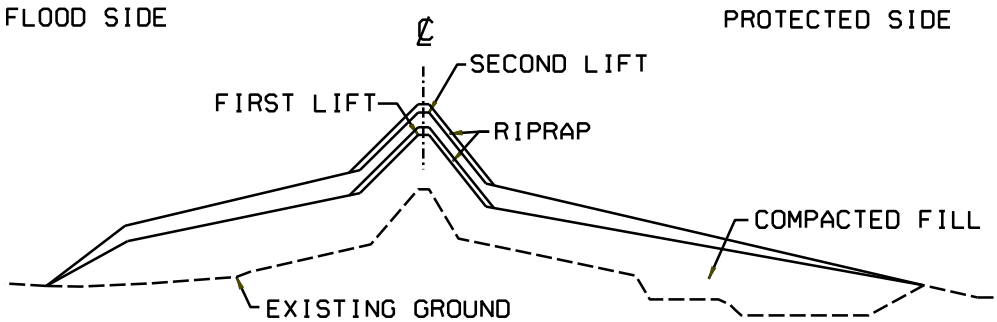
(4) Earthen levees using stability berms with staged construction and wick drains.

In general, Alternative 1 consists of a new reinforced concrete T-wall supported by steel H-piles, built on the existing levee. The existing levee may be slightly degraded in order to drive H-piles, drive steel sheet pile for seepage cut-off, and compact a stone pad for pouring of the concrete footing. From the flood-side toe to protected-side toe, 18 inches of riprap was assumed to be placed for scour protection and provide a reasonable cost for this protection. At the time of this report design, the USACE is still evaluating the preferred scour protection for St. Bernard. Once additional information is available regarding the preferred scour protection, this would be included in the final design. Alternative 1 is designed for future (2057/2060) conditions only, because T-walls cannot be modified as easily as levees. A general section of the T-wall alternative is shown below:



Alternative 1: T-wall alternative on existing levee

Alternative 2, earthen levees using stability berms, consists of constructing additional earthen levee lifts along the centerline of the existing levees. Scour protection, consisting of 18 inches of riprap, was used for cost estimation purposes from toe to toe of the entire cross section. The initial section, taken from the October 2007 Hydraulic Report, was altered to meet the required geotechnical factors of safety. Overbuild was included in all lifts. Alternative 2 was initially designed for the existing condition and then has subsequent lifts to reach future conditions in 50 years. The number of lifts and the timeline required for these lifts was optimized over this 50-year time period. Typically, the second lift is in the first 2 to 5 years. A section showing the general design features of Alternative 2 is presented below:



Alternative 2: Earthen levees using stability berms (Oct 07 Alternative)

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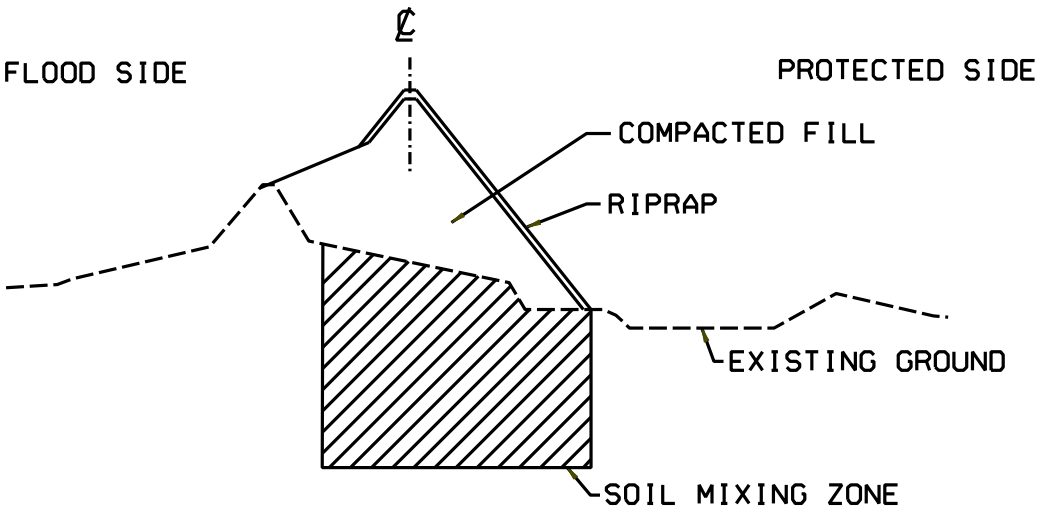
Alternative 3 consists of an earthen levee using deep soil mixing, which is constructed by first installing deep soil-mixed panels on the protected side of the existing levee toe and then placing earthen embankment on top of this improved foundation soil. The centerline of the proposed levee is offset from the existing levee and is positioned on the deep soil-mixed panels. Scour protection, consisting of 18 inches of riprap, was used for cost estimation purposes from toe to toe of the entire cross section. This alternative was investigated with elevations corresponding to the future conditions by 2057/2060. Because of the nature of this alternative, the foundation of the deep soil mixing must be designed to accommodate the future design elevations and was the reason for investigating this alternative up to this level; however, interim or lower height lifts for the existing condition could be investigated after the foundation is prepared for future conditions. The width of the soil-mixing region was extended the full width of the new levee to accommodate differential settlement along the deep soil-mixing region. A section showing the general design features of Alternative 3 is presented below:



Alternative 3: Earthen levees using deep soil mixing and landside shift

Alternative 4 is an earthen levee using staged construction and wick drains. This alternative is constructed by first placing a sand blanket on top of the existing levee and installing wick drains. An earthen levee is placed on top of the sand blanket to the highest elevation possible for the existing soil strength, and a steel sheet pile or other seepage cut-off is installed down the center to prevent water from

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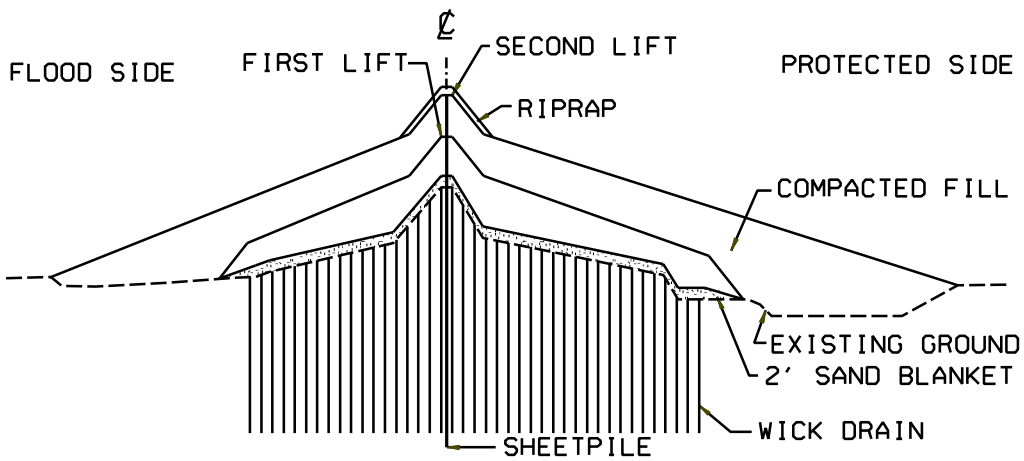
flowing through the levee. A time frame is then required before placement of the second lift of earthen embankment and 18 inches of riprap scour protection. The time frame is to allow the water to dissipate from the foundation soils so that the shear strength of the soils increases. A greater time frame results in greater shear strength, thus allowing higher levee elevation and a reduced footprint. For this design, a minimum of 1 year must pass before the second lift can be placed. Alternative 4 was evaluated for future conditions for the same reasons as stated for Alternative 3. A section showing the general design features of Alternative 4 is presented below:



Alternative 4: Earthen levees using stability berms with staged construction and wick drains

All these alternatives used one soils reach for LPV 145 (Reach A: Stations 370+00 to 703+98) based on the updated geotechnical undisturbed borings, CPT, and field and laboratory testing. One soils reach is considered appropriate for this level of study and the similarity of soil profile in LPV 145.

<u>LPV 146 – Chalmette Loop Levee – Bayou Dupre to Highway 46</u> - For this reach, the existing flood protection consists of earthen levees only. For the proposed work, the following flood protection alternatives were considered for Hydraulic Reaches SB13, SB15, and SB16:

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(1) T-wall alternative on existing levee;

(2) Earthen levees using stability berms (Oct 07 Alternative);

(3) Earthen levees using deep soil mixing and landside shift; and

(4) Earthen levees using stability berms with staged construction and wick drains.

These are the same alternatives evaluated for LPV 145, and general descriptions can be found above. Three soils reaches (Reach B: Stations 708+65 to 965+00, Reach C: Stations 965+00 to 1008+00, and Reach D in LPV 146: Stations 1008+00 to 1115+00) were considered for this EAR-RHCA based on the updated geotechnical undisturbed borings, CPT, and field and laboratory testing. LPV 146 was broken up into three soils reaches because of the difference in soil properties along the MRGO and then after the levee turn, as shown on the Overall Site Plan (Figure 1).

<u>LPV 147 – Chalmette Levee Loop – Highway 46 and Bayou Road Floodgate</u> - This reach consists of the levee section at the Highway 46 Crossing and Bayou Road Floodgate. For Highway 46, two alternatives were considered using the hydraulic information of SB16:

(1) Highway 46 Ramp (Earthen Levee); and

(2) Bridge over T-wall.

The ramp alternative would be grouped with the earthen levee alternative used for LPV 146. For the bridge alternative, the structure would span the T-wall option used from LPV 148. The Bayou Road Floodgate and T-wall would be demolished and replaced with a T-wall. The Bayou Road traffic will be re-routed onto Highway 46 on either side of the protection using access roads. As stated previously, the decision to close off the Bayou Road Floodgate was based on reduced traffic volume along Bayou Road, maintenance issues, and preference by the local sponsor.

<u>LPV 148 – Chalmette Loop Levee – Verret to Caernarvon</u> - For this reach, the existing flood protection consists of an earthen levee, St. Mary's Pump Station, and the Creedmore Drainage Structure. For the existing earthen levee sections, the following flood protection alternatives were considered for Hydraulic Reaches SB16 and SB17:

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(1) T-wall alternative on existing levee;

(2) Earthen levees using stability berms (Oct 07 Alternative);

(3) Earthen levees using deep soil mixing and landside shift; and

(4) Earthen levees using stability berms with staged construction and wick drains.

These are the same alternatives evaluated for LPV 145, and general descriptions and sections can be found above. Two soils reaches (Reach E: Stations 1125+00 to 1537+00 and Reach F: Stations 1537+00 to 1560+25) were considered for this EAR-RHCA based on the updated geotechnical undisturbed borings, CPT, and field and laboratory testing. The soils reaches were broken into two for LPV 148 because of the differences in soil profile.

The fronting protection at St. Mary's Pump Station was evaluated using Hydraulic Reach SB20. Due to the significant rise in water level from the existing design, demolition and replacement of the T-walls was considered. For the Creedmore Drainage Structure, the existing structure will be demolished, as requested by the sponsor, because it is not being used.

<u>LPV 149 – Chalmette Loop Levee – Caernarvon Floodwall and Floodgates</u> - The final modifications to the scope involved alternatives to be carried forward to the EAR stage that can be summarized as follows:

- Alignment 2 T-wall: Constructed to EL 24.0 along an offset (approximately 25 feet) alignment east of existing; and
- Alignment 4 T-wall: Constructed to EL 24.0 along an offset alignment that crosses the Caernarvon Canal (west of Elevated Boat Industries [EBI]) and ties into the Mississippi River Levee.

Both of these alternatives utilize T-wall sections that are full height or constructed at grade. Each of these alignments will also require a closure gate at the railroad and Highway 39. Alignment 4 will require a floodgate structure in Caernarvon Canal. For this report, a sector gate structure similar to Bayou Dupre was used.

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2.3 Design Criteria Summary

Detailed information with regard to design criteria, input data, and assumptions to complete the analysis and recommendations is discussed in Appendix B. A concise summary is presented below:

2.3.1 Site Reconnaissance Notes

A site reconnaissance was performed by the team on April 5 and April 25, 2007, at all of the reaches with the exception of LPV 145 due to access issues. The purpose of the site reconnaissance was to become familiar with the existing structures and levees and identify any potential issues that may affect the analysis and/or design alternatives. The site reconnaissance was not intended to inspect and evaluate existing conditions. A levee assessment was previously completed by others. Site descriptions for each structure are provided in the corresponding sections of this report.

2.3.2 Survey Data

The baseline used in this report was created from existing survey data provided by the USACE. No new surveys were completed as part of this project. Survey information provided by the USACE which includes Reaches LPV 144 through 149 is discussed below as well as in Appendix B.

- **LPV 144:** Aerial images were primarily used in conjunction with the 1968 DM.
- **LPV 145:** Aerial images, baseline data (May 1987), bathymetry of the MRGO, and construction drawings with typical cross sections were reviewed. Detailed topography was not available at the time of the project. Therefore, cross sections used for analysis were generated by using the construction drawings, DMs, aerials, MRGO bathymetry, and engineering judgment. For future engineering reports, survey information conducted by the USACE during the report development will be incorporated. Topographic surveys were obtained between the MRGO and the borrow pits located on the protected side of the levees. Construction plans of the borrow pits were also utilized. Horizontal and vertical control points were located along the levee crest at 2,000-foot intervals.
- **LPV 146:** Aerial images, detailed topography from October 2005, baseline data, and bathymetry of the MRGO were reviewed. The detailed topography generally

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extended 450 feet and 550 feet from the centerline of the levee on the protected side and flood side, respectively. Survey information was used in conjunction with aerial images and MRGO bathymetry to model cross sections for analysis. For future engineering reports, survey information recently conducted by the USACE during the report development will be incorporated. Topographic surveys were obtained between the MRGO and the borrow pits located on the protected side of the levees. Constructions plans of the borrow pits were also utilized. Horizontal and vertical control points were located along the levee crest at 2,000-foot intervals.

- **LPV 147:** Aerial images, baseline data, and limited topography that extended approximately 22 feet from the centerline were reviewed. Additional information needed to complete design sections was obtained from construction drawings and as-built plans of Highway 46 and the Bayou Road Floodgate.

- **LPV 148:** Aerial images, baseline data, and cross sections, taken every 300 feet, were reviewed for the levees. Cross sections generally extended approximately 200 feet from the centerline of the levee. The cross sections did not extend far enough to capture the flood-side and protected-side canals at the toes of the levees. Therefore, DMs, in conjunction with the survey information, were used to model the cross sections used for analysis. In addition, the design sections as shown on the construction plans for LPV 148.01, which is currently underway, were included in the analysis, quantities, and cost.

  For the structures within this reach (Creedmore Drainage Structure and St. Mary's Pump Station), only aerial images and baseline data linked to the levee were available. Detailed topography and structure profiles were not available at the time of the report. Therefore, construction drawings, DMs, and as-built information had to be used to reconstruct information at the Creedmore Drainage Structure and St. Mary's Pump Station.

- **LPV 149:** Aerial images, baseline data, and limited topography were reviewed. Additional information needed to complete design sections was obtained from *Conceptual Design Modifications, Caernarvon Floodwall and Gate*, 2006. Detailed topography was not provided or available at the time of the report. Therefore, cross sections used for analysis were generated by using the construction drawings, DMs, aerial images, and engineering judgment. Although survey data from the USACE were not available for the gates and floodwalls at Caernarvon (LPV 149), alternate sources of Light Detection and Ranging (LIDAR)

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survey data were used to define the existing ground surface. Following Hurricane Katrina, LIDAR data were collected by the Interagency Performance Evaluation Taskforce (IPET) to establish the post-storm levee surface conditions; however, the data were limited to a narrow corridor along the existing levee alignment. Two of the alternatives for LPV 149 go beyond the limits of these data; therefore, another source of topographic data was needed. The Louisiana Oil Spill Containment Office (LOSCO) has collected LIDAR survey data for the entire state of Louisiana. The LOSCO data in the Caernarvon area dates to 2003, before Hurricane Katrina; however, it appears to be in reasonable conformance with the LIDAR data collected after the storm. The LOSCO data were used instead of the IPET data because the data covers the entire project area and there is no discernible difference between the two data sets.

Due to the multiple sources of information, the survey datum was not consistent between the data obtained. All elevations used for this report were converted to North American Vertical Datum of 1988 (NAVD88) using the latest epoch (currently 2004.65). All elevations were based upon NAVD88 epoch 2004.65. It should be noted that as of June 21, 2007, the National Geodetic Survey has republished 340 survey benchmarks in the 27 parishes affected by Hurricanes Katrina and Rita, which are to be used for all pending and recently designed projects. These elevations are referred to as NAVD88 epoch 2006.81. However, any survey data to be used for the final design will be referenced to the NAVD88 2004.65 epoch unless directed otherwise.

2.3.3 Existing Plan Documentation

Available existing plan information is listed in Appendix B. This information was obtained either by research of the design team or provided by the USACE, St. Bernard Parish, and the Louisiana Department of Transportation and Development (LADOTD). The relevant information was utilized by the design team to assist with modeling cross sections and developing design alternatives.

2.3.4 Utilities and Relocations

No utility owners were contacted, per the USACE request; therefore, the location, quantity, and size of any utilities described in this report are subject to correction upon verification with the utility owners. Utility and relocation related assumptions can be found in Appendix B.

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2.3.5 Subsurface Conditions

As directed by the USACE, recent subsurface information was used for the design analysis in this report. The design shear strengths as well as other soil properties were primarily based upon recently completed soil borings, CPTs, and laboratory testing data from early 2007. A separate task order was conducted to independently review the recent data. Data that were not considered acceptable were removed from the shear lines used for this EAR-RHCA. A few of the CPTs were also used to verify the shear lines. Historical subsurface information obtained from DMs, construction drawings, engineering judgment, and typical values of area soils were also referenced to help model the cross sections, develop shear lines, and determine other soil properties where required. The DMs included soil borings, subsoil investigation reports, shear strength data stratification, and consolidation testing. The specific soil profiles, shear lines, and associated references for each reach are described in their appropriate sections. A brief introduction to the geology conditions at the site is given in Appendix B.

2.3.6 Hydraulic Information

Based on the hydraulic reach information received from the USACE, the design water elevations associated with the alternatives are summarized in the tables below.

As stated before, the Interim Selection Report, completed in March 2008, had been prepared to compare different options (with or without wave berm, full- or half-length of wave berm, armored or unarmored, etc.) for the T-wall alternative. The T-wall option “B2 (T-wall on levee, armored, no berm)” was selected to carry forward for this report. The design elevations for the “B2” T-wall alternative were obtained from the USACE report HPO-LFA-0091-07, which include structural superiority and are summarized in the table below. The differences in elevations are a result of the location of the hydraulic reach with respect to the modeled hurricanes. In order to check the flood-side stability, a Normal Water Elevation of -1 was assumed for all elevations in the MRGO, Jorda Canal, and Caernarvon Canal, as provided by local agencies and approved by the USACE prior to our analysis.

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T-wall Design Elevations

Hydraulic Reach Segment	Description	2057/2060 1% Design Elevation	2057/2060 Flood-Side Toe Design Elevation
SB11	T-wall on levee, armored, no berm	32.0	19.9
SB15	T-wall on levee, armored, no berm	27.5	17.1
SB16	T-wall on levee, armored, no berm	29.5	19.0
SB19	Bayou Dupre Control Structure	31.0	N.A.
SB20	St. Mary Pump Station	30.5	N.A.
SB21	Caernarvon Floodwall	24.0	19.5

Information regarding Hydraulic Reaches SB12, SB13, and SB17 was not provided in Hydraulic Report HPO-LFA-0091-07. Therefore, the USACE suggested the use of SB11, SB15, and SB16 for SB12, SB13, and SB17, respectively, for the T-wall alternative study.

For the levee alternatives, the design elevations were obtained from Chapter 3 (St. Bernard Parish) of the Hydraulic Report, dated October 2007. Both the 2010/2011 and 2057/2060 elevations were present as shown in the table below, but do not include overbuild. Overbuild was determined from settlement analysis and discussed in the individual results sections of this report.

Earthen Levee Design Elevations

Hydraulic Reach	2057/2060 1% Design Elevation	2057/2060 Still Water Elevation	2010/2011 1% Design Elevation	2010/2011 Still Water Elevation	Normal Water Elevation for Flood-Side Checks
SB11	31.5	21.2	29.0	19.7	-1.0
SB12	30.0	20.2	27.5	18.7	-1.0
SB13	29.0	19.3	26.5	17.8	-1.0
SB15	29.0	18.6	26.5	17.1	-1.0
SB16	29.0	20.4	26.5	18.9	-1.0
SB17	29.0	21.0	26.5	19.5	-1.0

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2.3.7 Additional Assumptions for Modeling Cross Sections

Additional assumptions with regard to modeling cross sections in the analysis are also addressed in Appendix B.

2.3.8 Geotechnical Design Criteria

Geotechnical analyses include slope stability, seepage, pile capacities, settlement analyses, bearing capacity, earth pressures, and shear strength gain in staged construction as appropriate for each reach and alternative. The design criteria for this project were discussed and agreed upon by the USACE during the kickoff meeting and adjusted if necessary during the course of the analysis and design. The details regarding geotechnical design criteria and assumptions used in this report are discussed in Appendix B. The change in design criteria from Method of Planes (MOP) to Spencers analysis for T-walls and to optimization of levees for the Spencers method was made after the analysis and design of this project. Prior to finalization of this EAR-RHCA, select analysis was performed to check Spencers in T-walls and optimization in levees to confirm the final selections were still relatively correct.

2.3.9 Structural Design Criteria

The structural analysis included the following structures: T-walls, floodgates, and a sector gate. Preliminary 1 percent probability of occurrence (100-year) design elevations for future (2057/2060) hydraulic conditions were used for analysis. Please refer to Appendix B for a more detailed discussion.

2.3.10 Civil/Right-of-Way/Relocations/Utilities 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 with 3:1 side slopes and 75 percent usable material. Additional design criteria and assumptions associated with Civil/Right-of-Way/Relocations/Utilities are also discussed in Appendix B of this report.

2.3.11 Limitations

The geotechnical and structural analyses were performed using soil boring and laboratory data, CPT results, as-built structure information, limited survey data, and

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computer programs as directed by the USACE. The soil properties such as strength, compressibility, and permeability have been carefully selected and reviewed based upon the widely spaced testing data. Some of the soil properties, such as the design shear lines, had been reviewed by the USACE before the analysis started in October 2007 and during review of the Draft EAR-RHCA in July 2008, respectively. The results of the analyses contained herein, particularly the safety factors generated, were very sensitive to the soil parameters. Given the natural variation of soil properties over space, the actual safety factors that exist in the field may differ from those presented in this report.

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2.3.12 Contingency

A cost estimating contingency of 30 percent was applied to all alternatives evaluated in each reach. The design team met and discussed each alternative and determined that the amount of uncertainty was essentially equivalent in magnitude (within 5 percent); therefore, the team agreed that one overall contingency factor of 30 percent was appropriate for this level of study and the available information. The percentage was determined by evaluating the amount and quality of the available data (geotechnical, survey, and plans) that were used in the analyses.

For all of the reaches, there was limited existing plan and survey information as described in earlier sections. For instance, some survey information was provided for LPV 145 and LPV 146; however, the extent of the information did not provide enough coverage for the design levee footprints required. Assumptions regarding the existing surface grade were required beyond these cross sections based on DMs, aerials, and engineering judgment. Another instance of limited information includes the construction drawings and as-built plans for Bayou Road and Highway 46. These plans were obtained; however, these drawings were all pre-Hurricane Katrina and not always clear in their designations of the roadways.

In summary, it is the design team's opinion that a 30 percent cost estimating contingency is appropriate for this alternatives study.

3. Project Reaches

3.1 LPV 144 – Bayou Dupre Control Structure – Modifications Based on Revised Hydraulic Criteria

As part of the original scope of work for this Task Order, preliminary designs and locations for the new structure were investigated for the Sector Gate Control Structure at Bayou Dupre. These evaluations were designed to EL 26.0, which was the assumed 2057 elevation at the time this project was initiated. As part of the revised scope of work, the revised hydraulic criteria were changed to reflect a new 2057 elevation of 31.0 feet. As directed by the new scope, the preliminary designs completed in the original EAR were evaluated and modified and the quantities were extrapolated for this higher elevation. A new cost estimate was developed to reflect these new quantities. A brief discussion of the assumptions and methods can be found in the following paragraphs.

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LPV 144 – Aerial view of Bayou Dupre Sector Gate

3.1.1 Structural and Geotechnical - General

The sector gate elements were evaluated for a taller structure set at EL 31.0. Potential increases to member sizes, thicknesses, and piles were anticipated by this modification. Engineering judgment including modification of the original calculations was used to extrapolate these changes.

3.1.1.1 Steel Sector Gate

For the original design, STAAD models were developed to estimate the gate framing and members using three-dimensional analysis. The STAAD output files were reviewed to determine the member types and stresses induced based on the original analysis. Generally, it was determined that an approximate increase of 20 percent for the total quantity of steel in gate frame would be adequate to account for the increased height. This 20 percent increase accounts for slightly heavier members as well as the additional physical member length to achieve the new height. The original estimated quantity for structural steel was 200 tons. The revised estimate is approximately 240 tons.

3.1.1.2 Gate Bay Walls

For this analysis, a more generalized approach was used because of the more complicated analysis and computer programs. Because the water height was raised by 5 feet, from 26 feet to 31 feet, increased moments were applied by a similar ratio. Similar calculations were performed to check the capacity requirements for the member thicknesses and proposed reinforcing. Based upon this limited analysis, the wall thickness increased approximately 1 foot with minor increases in the anticipated reinforcing.

3.1.1.3 Foundation for Gate Structure

Using the new dimensions determined from the above-mentioned analysis, a pile capacity check was performed in order to determine the new requirements due to overall axial and uplift loads. In the original design, 350 HP 14x89 piles were required. It was determined that an additional 25 piles would be required to resist the additional axial and uplift loads. It was approximated that the additional piles could be incorporated into the base slab without any modifications to the overall size of the slab. The original base slab thickness was deemed to be appropriate and no changes were

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necessary. The sheet pile cut-off length was increased approximately 20 percent based on the revised structure height.

3.1.1.4 T-Walls

The existing hand calculations were reviewed and modified to account for the wall height increase from 26 feet to 31 feet. The associated moments were also revised. Based on an area of steel requirement and other calculations including ratio of tension reinforcement, etc., bar sizes were calculated to determine a usable concrete thickness based on the aforementioned requirements. The stem was increased from 2.5 feet to 3.0 feet in thickness. The footing base increased from 3.5 feet to 4.0 feet in thickness. Based on these increases, engineering judgment was applied to modify the H-pile and sheet pile lengths using a 20 percent increase.

3.1.1.5 Other General Quantity Items

Because the footprint remained the same, there were several items that required no additional changes. Some examples of other contributing items that remained the same are:

- Excavation;
- Cofferdam Materials;
- Riprap;
- Mechanical/Electrical;
- Guidewalls; and
- Shell Backfill.

3.1.2 Pontoon Bridge

The conceptual layout of the proposed pontoon bridge remained unchanged from the original EAR submittal. The remaining issue of the location of the pontoon bridge in relation to the sector gate will be determined during development of the construction plans and specifications. The revised hydraulic elevation changes that impacted the

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sector gate have no bearing on the overall estimated cost of the pontoon bridge structure. This cost remained as a lump sum line item in the M2 estimate.

3.1.3 Final Estimated Cost and Conclusion

Based on the revised quantities that were developed using the methods described above, a revised cost estimate was developed. The total estimate of probable cost for the Bayou Dupre Control Structure at EL 31.0 is **$76,504,000**.

Because the final structure location will be determined during the development of the construction plans and specifications, it is anticipated that the total cost will change slightly based on the final location. Although the overall footprint of the structure will be very similar to that shown in the original plates, the T-wall tie-in lengths contribute (whether increasing or decreasing) significant costs to the overall total. As in this study, care will be taken during the plans and specifications (P&S) stage to minimize the overall cost for the T-walls during final site selection.

3.1.4 Design and Construction Schedules

The proposed schedule summarizes the time required for design and construction of the preferred alternative for LPV 144. The schedule is based on a notice to proceed date of June 11, 2008. Approximately 2 months are scheduled for geotechnical testing, which should encompass four to six borings for the structural design. Final design will require approximately 9 months and will be submitted in March 2009. Real estate acquisition and environmental permitting are to be completed by the USACE and are assumed not to delay construction for the sake of this schedule.

Construction of the floodgate will commence on October 29, 2009, and is slated for final acceptance on May 27, 2011. The fabrication of the gate leaf is not included in the schedule, but is planned to last 210 days. Fabrication will commence subsequent to the construction of the cofferdam. The demolition of the existing floodgate structure will not initiate until the completion of the gate leaf installation and tie-in to adjacent levee transpire.

All work is estimated using 10-hour workdays on a 7-day per week work schedule. All work is scheduled to be completed prior to the 2011 hurricane season.

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3.1.5 Report Plates – LPV 144

Summary tables of the geotechnical analysis along with the plan and profile and typical sections are combined for the Bayou Dupre Control Structure and presented following this report.

<u>Bayou Dupre Control Structure to EL 31.0 (Plates 3.1.0 – 3.1.15)</u>

- LPV 144 General Layout Sheets for Bayou Dupre (Plates 3.1.0 – 3.1.4)
- Sector Gate Details for Bayou Dupre (Plates 3.1.5 – 3.1.11)
- Pontoon Bridge Details (Plates 3.1.12 – 3.1.15)

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3.2 LPV 145 – Chalmette Loop Levee – Bayou Bienvenue to Bayou Dupre

LPV 145 is a levee segment located between the Bayou Bienvenue and Bayou Dupre Control Structures from USACE Baseline Station 370+00 to Station 703+98 (33,398 feet in length). This portion of the loop levee is only accessible by boat and could not be viewed during the site visits; therefore, no photographs were taken. Based on the typical sections and the aerial images, borrow pits were indicated near the protected-side levee toes. On the flood side, the MRGO is located between 160 feet to more than 640 feet from the centerline of the levee.

3.2.1 Design Evaluations

There are three hydraulic reaches (SB11, SB12, and SB13) in LPV 145. For each hydraulic reach, four engineering alternatives were evaluated. Initially, analysis of the levee, with overbuild and scour protection, was completed without stability berms. In all cases, the safety factor for the existing condition was significantly less than required and additional measures were necessary. The four evaluated engineering alternatives included:

(1) T-wall alternative on existing levee;

(2) Earthen levees using stability berms (Oct 07 Alternative);

(3) Earthen levees using deep soil mixing and landside shift; and

(4) Earthen levees using stability berms with staged construction and wick drains.

Geometric Section

For this reach, one representative section (Station 586+00) was selected for analysis. Station 586+00 followed Design Section 2 shown on the construction drawings dated September 2005. The surface extended approximately 600 feet from the centerline of the levee on the protected side with a bottom of borrow area at EL -20.0, as noted on the proposed construction drawings. However, the bottom of borrow area was changed to EL -3.0 in our analysis as instructed by the USACE because the pits were actually excavated much shallower than on the proposed plan. On the flood side, the surface was shown approximately 170 feet from the centerline of the levee. For the purpose of slope stability analysis, the surface geometry was

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further extended to include the MRGO on the flood side based on aerial images, MRGO bathymetry, and engineering judgment. From the aerial images, it was noted that the bank of the MRGO is on average about 350 feet from the levee centerline in reach LPV 145. As a result, a typical survey section of the MRGO where the bank is about 350 feet from the levee centerline was utilized to extend the flood-side geometry to include the MRGO. The ground surface was assumed to be flat (same elevation as the last known point) on the protected side if further extension was needed in slope stability analysis.

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Soils Reach

Subsurface explorations indicated that underlying soils consist mainly of cohesive soils. Based on the boring results, it appears that the soil properties do not vary significantly along the levee alignment in LPV 145 and that one soils reach (Reach A from Stations 370+00 to 703+98) may be justified. Laboratory tests indicated that the existing levee fill materials may have undrained shear strengths on the order of 800 pounds per square foot (psf). A peat layer appeared to present at approximately EL -10.0 to EL -20.0. However, the laboratory strength tests showed relatively high shear strength for those peat materials under the crown of the existing levee, ranging from 600 to 700 psf, while the shear strengths ranged from 250 to 310 psf in the toe areas. Below the peat layer, the shear strength increased from 500 to 1,230 psf at EL -77 at an approximate rate of 12.8 psf per foot under the centerline, while at the toes the shear strength increased from 310 to 1,230 psf at EL -77 at an approximate rate of 16.1 psf per foot. The shear strength difference between the centerline and the toe decreases as the depth increases, and the two shear lines coincide with each other below EL -77.

Shear lines were determined based on the newly completed soil borings (1U through 34U completed in February 2007) and laboratory tests in LPV 145. A centerline shear line was generated using Borings 1U through 17U. A toe shear line was also generated using Borings 18U through 34U. These shear line diagrams can be found in Appendix C.

Limited consolidation test data were available to determine the design consolidation parameters: compression index; coefficient of consolidation; and initial void ratio. Given the fact that the settlement analysis is very sensitive to the values of compression indices, Skempton's empirical relationship between liquid limit and compression index was also used to help judge the design compression index versus depth because only limited laboratory data are available in some boring locations at

various depths. It is also assumed that all the soils in settlement analysis were normally consolidated. As a result, the recompression index was forced to be the same value as the virgin compression index for the same soil layer for settlement calculation purposes. Coefficient of consolidation and initial void ratio were determined in a similar way as compression index except that no empirical correlations were used. Those design consolidation parameters versus depths were also presented in Appendix C. It should be noted that the coefficient of consolidation points shown on the figures were the average values from laboratory tests under 2,000 and 4,000 psf incremental loads, reported from both Logarithm-of-Time Method and Square-Root-of-Time Method. Again, similarly to shear strength design line, centerline and toe design lines were separated from each other for all the consolidation parameters.

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3.2.2 Alternative 1 – T-wall Alternative on Existing Levee (Plates 3.2.1 – 3.2.11)

In general, Alternative 1 consists of a new reinforced concrete T-wall supported by steel H-piles to be built on the existing levee. The existing levee would be slightly degraded in order to drive H-piles, drive steel sheet pile for seepage cut-off, and place a stone pad for placement of the concrete footing. The foundations were typically embedded approximately 3.0 feet into the crown for erosion protection. From the flood-side toe to protected-side toe, 18 inches of riprap was assumed to be placed for scour protection and provide a reasonable cost for this protection. Stability berms are not used in this alternative. Alternative 1 is designed for future (2057/2060) conditions only.

3.2.2.1 Geotechnical and Structural Results

For this option, the existing levee would be slightly raised in Hydraulic Reaches SB11 and SB12 (Stations 370+00 to 525+82.8) to meet the hydraulic design elevations while in SB13 (Stations 525+82.8 to 703+98) the existing levee would be kept in place. Unbalanced loads that result from global instability are transferred into the driven piles and are reported in the tables below. For these walls, HP 14x89 piles are used. Typical sections illustrating this option are provided at the end of this report section along with the required construction limits. The results of the geotechnical and structural analysis are summarized below:

T-wall Analysis Structural Summary							
Hydraulic Reach	**Station Limits**	**Top of Wall Elevations**	**Stem Height (feet)**	**Stem/Base Thickness (feet)**	**Base Width**	**Pile Tip Elevation**	**No. of Piles in Cross Section**
SB11	370+00 to 475+04.5	32.0	12.5	2.0/2.5	13.0	-105.0	3
SB12	475+04.5 to 525+82.8	32.0	12.5	2.0/2.5	13.0	-105.0	3
SB13	525+82.8 to 703+98	27.5	8.0	2.0/2.5	10.0	-80.0	2

Assumptions: 4,000 pounds per square inch compression strength; 5-foot pile center spacing along wall; and steel HP 14x89 piles on a 1H:3V batter perpendicular to wall.

T-wall Analysis Geotechnical Summary					
Station Limits	**Hydraulic Reach**	**Sheet Pile Tip Elevation**	**Fictitious Surface Elevation**	**Maximum Anchor Force (lbs/ft)**	**Maximum Moment (lbs-ft/ft)**
370+00 to 525+82.8	SB11 and SB12	-83.4	-56.6	8,810	157,960
525+82.8 to 703+98	SB13	-32.0	NA	NA	NA

lbs/ft Pounds per foot.
lbs-ft/ft Pounds per foot per foot.

In the above table, “NA” stands for “not applicable”, which typically means that the T-wall is not subjected to unbalanced forces from slope stability analysis. Some benching of the existing levee may be necessary in order to drive the piles at the top of the levee due to the steep existing slopes and levee height. Because no dense sands were identified from the borings, steel piles are expected to be driven easily into the ground. Seepage will be controlled by driving sheet piles into the ground to the designed elevations as shown in the table above.

The use of stability berms and/or deep soil mixing may be applied here to reduce unbalanced loads, fictitious surfaces, and thus the pile lengths. However, adding stability berms will likely require additional ROW and local stability issues as discussed in other alternatives within this reach. Deep soil mixing will also add to the cost but may reduce pile loads and lengths. These alternatives could be considered

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in the future should the T-wall need to be raised with minimal impacts to the foundations.

3.2.2.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

The T-wall alternative in LPV 145 may not require much additional ROW or temporary construction easement because the construction will occur within the known existing ROW. Because the existing ROW for the entire reach is not known, a small amount of ROW and easement could potentially be required for this reach. Research of all available information does not include existing ROW for Station 370+00 to Station 383+00.

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following will require relocation in LPV 145 for Alternative 1.

Utility Description	Station	Length	Orientation	Owner
Gas Pipeline (24")	390+00	877 feet	Crossing	unknown
Gas Pipeline (24")	426+00	875 feet	Crossing	unknown
Gas Pipeline (24")	436+00	875 feet	Crossing	unknown

The flood side of the levee along the MRGO is populated by grasses, most of which appear to be in poor condition. This area provides little habitat value. Sugarberry (*Celtis laevigata*) is found in small, isolated stands along the flood face of the levee and more so in the northwest portion approaching Bayou Bienvenue. On the protected side of the levee, there appears to be an expansive scrub-shrub marsh community with substantial stands of sugarberry *(Celtis laevigata),* and perhaps tupelo (*Nyssa*) and some cypress (*Taxodium*). This well-stratified habitat is of high value to many wildlife species common to the project area and likely represents the most significant ecological impacts resulting from any possible widening of the levee base. The T-wall alternative does not fill any of the submerged wetlands and will only cause temporary disturbance during construction. Some clearing and grubbing may be required to provide access to working sites; however, the habitat area would return quickly to its current state following construction completion. Alternative 1 would result in the least ecological impact among all the alternatives considered.

The preliminary ROW plates are included in Appendix I.

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3.2.3 Alternative 2 – Earthen Levees Using Stability Berms (Oct 07 Alternative) (Plates 3.2.12 – 3.2.30)

Alternative 2, earthen levees using stability berms, consists of constructing additional earthen levee lifts along the centerline of the existing levees. Scour protection, consisting of 18 inches of riprap, was used for cost estimation purposes from toe to toe of the entire cross section. Final recommendations from the USACE Armor Team are under development. The initial section, taken from the Hydraulic Report, was altered to meet the required geotechnical factors of safety.

3.2.3.1 Geotechnical Results

For this option, complete analysis was conducted for the existing condition (2010/2011 design elevations) while only limited slope stability analysis was performed for future conditions (2057/2060 design elevations). Additional lifts were proposed to reach 2057/2060 design elevations based on time-rate settlement analysis. The scheduling of the future lift(s) is provided with the typical sections attached to this report. Overbuild was also included with these lifts to account for settlement in the proposed levee. Design information used to model this alternative is summarized below.

Summary of Overbuild for 2010/2011 Design Elevations					
Station Limits	**Hydraulic Reach**	**2010/2011 Levee Design Elevation**	**Proposed Top of Levee Elevation**	**Proposed Top of Riprap Elevation**	**Overbuild Thickness (feet)**
370+00 to 475+04.5	SB11	29.0	33.0	34.5	4.0
475+04.5 to 525+82.8	SB12	27.5	30.5	32.0	3.0
525+82.8 to 703+98	SB13	26.5	29.0	30.5	2.5

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Summary of Overbuild for 2057/2060 Design Elevations					
Station Limits	**Hydraulic Reach**	**2057/2060 Levee Design Elevation**	**Proposed Top of Levee Elevation**	**Proposed Top of Riprap Elevation**	**Overbuild Thickness (feet)**
370+00 to 475+04.5	SB11	31.5	34.0	35.5	2.5
475+04.5 to 525+82.8	SB12	30.0	33.0	34.5	3.0
525+82.8 to 703+98	SB13	29.0	32.0	33.5	3.0

For both 2010/2011 and 2057/2060 design elevations, the levee has a crown approximately 10 feet wide and slopes down at a 4 horizontal (H):1 vertical (V) (4H:1V) or 3H:1V until intersecting the stability berms on the protected side. The flood side has a wave berm as proposed by the hydraulics section of the USACE, which may be altered in order to meet the required factor of safety for the flood-side stability. Typical sections illustrating this option are provided at the end of this report section along with the required construction limits. Based on the geotechnical analysis results, the approximate berm dimensions on both protected and flood sides are summarized below:

Summary of the Stability Berm Dimensions (2010/2011 Design Elevations)				
Station Limits	**Hydraulic Reach**	**Distance of Toe from Levee Centerline (PS/FS) (feet)**	**Length (PS/FS) (feet)**	**Average Depth (PS/FS) (feet)**
370+00 to 475+04.5	SB11	455/457	398/399	10/11
475+04.5 to 525+82.8	SB12	455/374	407/328	9/7
525+82.8 to 703+98	SB13	455/374	407/328	8/6

PS Protected Side.
FS Flood Side.

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Summary of the Stability Berm Dimensions (2057/2060 Design Elevations)				
Station Limits	**Hydraulic Reach**	**Distance of Toe from Levee Centerline (PS/FS) (feet)**	**Length (PS/FS) (feet)**	**Average Depth (PS/FS) (feet)**
370+00 to 475+04.5	SB11	455/457	398/399	12/14
475+04.5 to 525+82.8	SB12	455/374	407/328	12/10.5
525+82.8 to 703+98	SB13	455/374	407/328	11.5/10

PS Protected Side.
FS Flood Side.

The size of the berm decreases from SB11 to SB12 to SB13 due to the decrease of the levee design elevations. Typical ROW distance from the centerline of the levee is 525 feet on the protected side and 350 feet on the flood side in LPV 145. Based on the table above, the protected-side berms will likely stay within ROW while additional ROW on the order of 24 to 107 feet will be needed on the flood side for both 2010/2011 and 2057/2060 design elevations.

Compacted clay fill was used for both the raised levee and stability berms because of the increased strength, more stringent testing requirements, and heavier unit weight. However, for erosion purposes, riprap fill was used for the small portion at the toe of the flood-side berm where the berm intersects with existing ground surface near the MRGO banks. On the flood side, it is expected that the distance from the MRGO to the centerline of levee varies significantly along the levee alignment in LPV 145. As a result, riprap or sand fill should be used for any filling underwater.

Seepage analysis was only performed for 2010/2011 design elevations. The berm on the protected side was mainly proposed for stability purposes. At the same time, it also acts as a seepage berm by providing additional weight to counteract those upward seepage forces. The critical exit gradient at the berm toe is 0.20, 0.26, and 0.24 for SB11, SB12, and SB13, respectively, which are all less than 0.5. In addition, because the length of the berm on the protected side is on the order of 400 feet, the possible seepage effect beyond the berm can be ignored based on the current seepage criteria.

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3.2.3.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

The plates following this report present the construction limit and proposed ROW. All staging and stockpiling are proposed to occur within the construction limits presented in these plates.

Real Estate Description	Quantity	Units
Additional Right-of-Way – 2010/2011 and 2057/2060	76.83	Acre
Construction Easement*	0.28	Acre

*Construction Easement is in addition to Right-of-Way quantity.

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following utilities will require relocation:

Utility Description	Station	Length	Orientation	Owner
Gas Pipeline (8")	399+15	877 feet	crossing	Air Products
Gas Pipeline (12”)	399+15			Air Products
Gas Pipeline (12.75")	424+00	875 feet	crossing	Enbridge
Gas Pipeline (16")	447+97	875 feet	crossing	Collins
Gas Pipeline (20")	590+49	875 feet	crossing	Southern Natural Gas
Gas Pipeline (20")	590+79	875 feet	crossing	Southern Natural Gas
Gas Pipeline (20")	600+70	875 feet	crossing	Southern Natural Gas

The flood side of the levee along the MRGO is populated by grasses, most of which appear to be in poor condition. This area provides little habitat value. Sugarberry (*Celtis laevigata*) is found in small, isolated stands along the flood face of the levee, more so in the northwest portion approaching Bayou Bienvenue. On the protected side of the levee, there appears to be an expansive scrub-shrub marsh community with substantial stands of sugarberry *(Celtis laevigata),* and perhaps tupelo (*Nyssa*) and some cypress (*Taxodium*). This well-stratified habitat is of high value to many wildlife species common to the project area, and likely represents the most significant ecological impacts resulting from a widening of the levee base. Construction of Alternative 2 will fill the open channels and will eliminate portions of this high-value wetland habitat.

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3.2.4 Alternative 3 – Earthen Levees Using Deep Soil Mixing and Landside Shift (Plates 3.2.31 – 3.2.41)

Alternative 3 is the earthen levee using deep soil mixing, which is constructed by first installing deep soil-mixed panels on the protected side of the existing levee toe and then placing earthen embankment on top of this improved foundation soil. The centerline of the proposed levee is offset from the existing levee and positioned near the center of the deep soil-mixed panels. Scour protection, consisting of 18 inches of riprap, was assumed (for cost estimation purposes) from toe to toe of the entire cross section.

3.2.4.1 Geotechnical Results

Because of the nature of this alternative, the foundation of the deep soil mixing must be prepared to handle future (2057/2060) design elevations and was the reason for investigating this alternative up to this level; however, interim or lower height lifts for the existing condition could be investigated after the foundation is prepared for future conditions. Overbuild was also included to accommodate settlement over time. Design information used to model this alternative is summarized in the table below.

Summary of Overbuild for 2057/2060 Design Elevations					
Station Limits	**Hydraulic Reach**	**2057/2060 Levee Design Elevation**	**Proposed Top of Levee Elevation**	**Proposed Top of Riprap Elevation**	**Overbuild Thickness (feet)**
370+00 to 475+04.5	SB11	31.5	33.8	35.3	2.3
475+04.5 to 525+82.8	SB12	30.0	32.0	33.5	2.0
525+82.8 to 703+98	SB13	29.0	30.8	32.3	1.8

The levee has a crown approximately 10 feet wide and slopes down at a 4H:1V until intersecting the existing ground surface on the protected side. The flood side has a proposed wave berm as instructed by the USACE. The existing levee crown may need to be partially excavated to accommodate the proposed wave berm on the flood side. The centerline of the proposed levee was shifted toward landside from the existing levee centerline with the deep soil-mixing option in order to construct the columns/panels. The location of the deep soil mixing was between the existing levee

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protected-side toe and the proposed new levee protected-side toe. Based on the results of the analysis, the approximate soil-mixing regions are as follows:

Soil-Mixing Regions				
Station Limits	**Hydraulic Reach**	**Width (feet)**	**Average Depth (feet)**	**Bottom Elevation of Deep Soil-Mixing Region**
370+00 to 475+04.5	SB11	215	64.5	-57
475+04.5 to 525+82.8	SB12	203	64.5	-57
525+82.8 to 703+98	SB13	190	64.5	-57

The lengths and depths were altered until all design cases were met for the most appropriate section. Due to the improved shear strength of the soil mass, failure surfaces were forced deeper or longer to reach weaker material. A maximum shear strength of 2,300 psf was used for the improved soil mass; however, the factor of safety for the shallower layers likely could be designed with less strength depending on the internal design and numerical analysis required for future design.

Typical sections illustrating this option are provided at the end of this report section along with the required construction limits.

Seepage analysis indicated that the soil-mixing region has provided enough “cut-off” effect and that an acceptable exit gradient can be achieved without additional seepage control measures.

3.2.4.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

The soil-mixing alternative in LPV 145 may require very little additional ROW or temporary construction easement. All construction will occur within the known existing ROW. Because the existing ROW for the entire reach is not known, a small amount of ROW and easement may be needed for this reach. Research of all available information does not include existing ROW for Station 370+00 to Station 383+00. All staging and stockpiling are proposed to occur within the construction limits.

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Real Estate Description	Quantity	Unit
Additional Right-of-Way	9.17	Acre

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following utilities will require relocation based on the soil-mixing alternative:

Utility Description	Station	Length	Orientation	Owner
Gas Pipeline (8")	399+15	877 feet	crossing	Air Products
Gas Pipeline (12”)	399+15			Air Products
Gas Pipeline (12.75")	424+00	875 feet	crossing	Enbridge
Gas Pipeline (16")	447+97	875 feet	crossing	Collins
Gas Pipeline (20")	590+49	875 feet	crossing	Southern Natural Gas
Gas Pipeline (20")	590+79	875 feet	crossing	Southern Natural Gas
Gas Pipeline (20")	600+70	875 feet	crossing	Southern Natural Gas

The flood side of the levee along the MRGO is populated by grasses, most of which appear to be in poor condition. This area provides little habitat value. Sugarberry (*Celtis laevigata*) is found in small, isolated stands along the flood face of the levee, more so in the northwest portion approaching Bayou Bienvenue. On the protected side of the levee, there appears to be an expansive scrub-shrub marsh community with substantial stands of sugarberry *(Celtis laevigata),* and perhaps tupelo (*Nyssa*) and some cypress (*Taxodium*). This well-stratified habitat is of high value to many wildlife species common to the project area and likely represents the most significant ecological impacts resulting from a widening of the levee base. The soil-mixing alternative does not fill any of the submerged wetlands and will only cause temporary disturbance during drilling. Some clearing and grubbing may be required to provide access to drilling sites; however, the habitat area would return quickly to its current state following drilling. Alternative 3 would result in less ecological impact than Alternative 2.

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3.2.5 Alternative 4 – Earthen Levees Using Stability Berms with Staged Construction and Wick Drains (Plates 3.2.42 – 3.2.52)

Alternative 4 is an earthen levee using staged construction and wick drains. This alternative is constructed by first placing a sand blanket on top of the existing levee and installing wick drains. Earthen levee is then placed on top of the sand blanket to the highest elevation possible based on the existing soil strength, and then a steel sheet pile or other seepage cut-off is installed down the center to prevent water from flowing through the levee. A time frame of approximately 1 year is required prior to the placement of the second lift of earthen embankment and 18 inches of riprap scour protection. The time frame is to allow the water to dissipate from the foundation soils so that the shear strength of the soils increases, thus allowing higher levee elevations.

3.2.5.1 Geotechnical Results

Because of the nature of this alternative, the foundation of the soil must be prepared to handle future (2057/2060) design elevations and was the reason for investigating this alternative up to this level; however, interim or lower height lifts for the existing condition could be investigated after the foundation is prepared for future conditions. Overbuild was also included to accommodate settlement over time.

The staged construction was anticipated to be constructed in a total of two lifts. The first lift will be raised to EL 29.0 and allowed to settle for 1 year. At the end of 1 year, the strength gain for each soil layer due to surcharge is calculated based on the degree of consolidation in that soil layer, considering the effect of wick drains. The second lift will then be placed based on future design elevations, as summarized in the table below.

Summary of Overbuild for 2057/2060 Design Elevations

Station Limits	Hydraulic Reach	2057/2060 Levee Design Elevation	Proposed Top of Levee Elevation	Proposed Top of Riprap Elevation	Overbuild Thickness (feet)
370+00 to 475+04.5	SB11	31.5	36.5	38.0	5.0
475+04.5 to 525+82.8	SB12	30.0	34.5	36.0	4.5
525+82.8 to 703+98	SB13	29.0	33.2	34.7	4.2

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The levee has a crown approximately 10 feet wide and slopes down at a 4H:1V until intersecting the stability berms. The centerline of the proposed levee coincides with the existing levee centerline for this option. Stability berms are required for this alternative with the future design elevation. Compacted clay fill was used for both the levee and stability berms because of the increased strength, more stringent testing requirements, and heavier unit weight. Based on the results of the analysis, the stability berms on both the protected and flood sides have the approximate dimensions shown in the table below:

Summary of the Stability Berm Dimensions (2057/2060 Design Elevations)				
Station Limits	**Hydraulic Reach**	**Distance of Toe from Proposed Levee Centerline (PS/FS) (feet)**	**Length (PS/FS) (feet)**	**Average Depth (PS/FS) (feet)**
370+00 to 475+04.5	SB11	455/621	422/500	10/11
475+04.5 to 525+82.8	SB12	455/354	422/322	9/9
525+82.8 to 703+98	SB13	455/347	422/230	7/6

PS Protected Side.
FS Flood Side.

Typical sections illustrating this option are provided at the end of the section along with the required construction limits. As mentioned before, the bank of the MRGO is typically 350 feet away from the levee centerline. Based on the table above, in SB12 and SB13, the stability berm on the flood side will stay in the ROW and will only go into the MRGO at limited stations. However, in SB11, additional ROW may have to be acquired and the MRGO needs to be filled partially for the sake of slope stability. Failure surfaces tended to exit near where the berms end due to significant decrease in overburden.

For seepage, the stability berms do not provide enough protection against excessive exit gradients because of the wick drains; therefore, a sheet pile cut-off wall or other acceptable seepage cut-off is required. The tip of the sheet pile wall is anticipated to be located at EL -44.5 in order to reduce the exit gradient to within allowable limits.

3.2.5.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

The stability berms with staged construction and wick drains alternative require acquisition of an additional 106.25 acres of ROW. The wide footprint of the stability

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berm is the reason for most of the new real estate requirements associated with this alternative. Plates provided at the end of this report present the construction limit and proposed ROW. All staging and stockpiling are proposed to occur within the construction limits presented in these plates.

The stability berms with staged construction and wick drains alternative requires 1,200 acres of borrow pit area to satisfy fill requirements.

Real Estate Description	Quantity	Unit
Additional Right-of-Way	106.25	Acre

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following utilities/facilities will require relocation based on the stability berm alternative:

Utility Description	Station	Length	Orientation	Owner
Gas Pipeline (8")	399+15	877 feet	crossing	Air Products
Gas Pipeline (12”)	399+15			Air Products
Gas Pipeline (12.75")	424+00	875 feet	crossing	Enbridge
Gas Pipeline (16")	447+97	875 feet	crossing	Collins
Gas Pipeline (20")	590+49	875 feet	crossing	Southern Natural Gas
Gas Pipeline (20")	590+79	875 feet	crossing	Southern Natural Gas
Gas Pipeline (20")	600+70	875 feet	crossing	Southern Natural Gas

The flood side of the levee along the MRGO is populated by grasses, most of which appear to be in poor condition. This area provides little habitat value. Sugarberry (*Celtis laevigata*) is found in small, isolated stands along the flood face of the levee, more so in the northwest portion approaching Bayou Bienvenue. On the protected side of the levee, there appears to be an expansive scrub-shrub marsh community with substantial stands of sugarberry *(Celtis laevigata),* and perhaps tupelo (*Nyssa*) and some cypress (*Taxodium*). This well-stratified habitat is of high value to many wildlife species common to the project area, and likely represents the most significant

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ecological impacts resulting from a widening of the levee base. Construction of Alternative 4 will fill the open channels and will eliminate portions of this high-value wetland habitat.

3.2.6 Summary of Cost Analysis

Below is a table summarizing the cost of each of the alternatives (not including escalation). Detailed cost estimates for each alternative can be found in Appendix H.

Description	Real Estate ($k)	Environmental Mitigation Including Contingency ($k)	Contract Cost Including Contingency ($k)	Engineering and Design Cost Including Contingency ($k)	Supervision and Administration Cost Including Contingency ($k)	Project Cost ($k)	Operation and Maintenance ($k)	Total Values ($k)
	1	2	3	4	5	1+2+3+4+5 A	B	= A+B
LPV 145 Alternative 1: T-wall	$485	$107	$479,681	$59,960	$45,570	$585,802	$9,867	$595,670
LPV 145 Alternative 2: Oct 07 All Stages	$6,305	$4,264	$644,044	$80,505	$61,184	$796,302	$82,257	$878,560
LPV 145 Alternative 3: Deep Soil Mixing	-	-	$746,110	$93,264	$70,880	$910,254	$45,594	$955,847
LPV 145 Alternative 4: Staged with Wick Drain All Stages	$3,492	$9,195	$858,078	$107,260	$81,517	$1,059,542	$69,328	$1,128,870

For comparison purposes, the tables below provide the quantities of the major work items for each alternative presented.

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LPV 145 – Alternative 1: T-wall		
Summary of Major Quantities		
	Quantity	**Unit**
Riprap	161,136	Cubic Yard
T-wall Concrete	60,238	Cubic Yard
T-wall Sheet Pile	273,612	Square Yard
T-wall Pile	1,793,792	Foot

LPV 145 – Alternative 2: Earthen Levee using Stability Berms		
Summary of Major Quantities		
	Quantity	**Unit**
Embankment Compacted Fill	11,433,224	Cubic Yard
Riprap	666,970	Cubic Yard
Clearing and Grubbing	1,345	Acre

LPV 145 – Alternative 3: Earthen Levees using Deep Soil Mixing		
Summary of Major Quantities		
	Quantity	**Unit**
Embankment Compacted Fill	3,675,896	Cubic Yard
Riprap	320,828	Cubic Yard
Soil Mixing*	15,531,913	Cubic Yard
Clearing and Grubbing	215.3	Acre

*Quantity for soil mixing is for the composite region. A 40 percent replacement ratio was applied to determine the estimated quantity of soil-mixed columns and/or panels.

LPV 145 – Alternative 4: Earthen Levees using Wick Drains and Staged Construction		
Summary of Major Quantities		
	Quantity	**Unit**
Embankment Compacted Fill	10,716,924	Cubic Yard

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LPV 145 – Alternative 4: Earthen Levees using Wick Drains and Staged Construction		
Summary of Major Quantities		
	Quantity	**Unit**
Riprap	628,665	Cubic Yard
Clearing and Grubbing	1073.6	Acre
Sheet Pile	292.921	Square Yard
Wick Drains	27,052,380	Foot

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3.2.7 Recommended Alternative

Based on the four options considered in this reach, the recommended alternative is Alternative 1 – T-walls on Existing Levee. This alternative is recommended based on the limited environmental impacts, ease of construction, reduced need for borrow, lowest cost, and no requirements for additional temporary easements and very little ROW. This alternative would not impact the MRGO. Also, this alternative will tie-in directly to the new T-walls tie-ins for LPV 144.

Alternatives 2 and 4 require extensive stability berms and some placement of fill in the MRGO. The cost of Alternatives 2 and 4 is largely because of the riprap, embankment fill, and geotextile separator fabric required. Geotextile fabric is used only as a separator between the any sand or stone required underwater in the MRGO. The fabric is not required for slope stability.

Alternative 2 requires the largest stability berms and amount of borrow. The availability of borrow is unknown at this time, which could adversely impact the construction costs and schedule if these resources are not available for the earthen alternatives.

Advantages and disadvantages to all the alternatives are shown in the table below:

Alternative	Advantages	Disadvantages
T-wall on Existing Levee	• Construction with ROW • No stability berms required • Materials more readily available • Reduced environmental impacts • No additional interim measures required • Built to future elevation • Reduced life cycle cost • Additional levee lifts not required	• Hard to modify in future and major reconstruction if altered • Political/public perception of walls after Hurricane Katrina • Poor aesthetics • Routine inspections
Earthen Levee with Stability Berms – Oct 07	• Can modify for future lifts • No interim measures required • Greater vehicular access • Aesthetically pleasing • More eco-friendly	• Source of borrow • Additional scour protection (heavier than current design) required • Additional ROW may be required • Significant cost up to future elevation • Fill in portions of the MRGO in LPV 145

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Alternative	Advantages	Disadvantages
Earthen Levee with Deep Soil Mixing	• Can reduce lift heights to existing design elevation • Improved subgrade/ foundation up to future design elevation § No interim measures required • Greater vehicular access • Aesthetically pleasing • More eco-friendly • No to very little ROW • No canal relocations • Less borrow required	• Landside shift • Difficult to excavate foundation in the future • Wet mixing produces excess spoil • Cost may be more significant • Additional scour protection required (heavier than current design) • Specialty contractors
Earthen Levee using Staged Construction & Wick Drains	• Reduced protection in 1st year • Can modify for future lifts • No interim measures required • Greater vehicular access • Aesthetically pleasing • More eco-friendly • Improved foundation strength with less time	• Source of borrow • Additional scour protection required (heavier than current design) • Additional ROW may be required • Seepage cut-off required • Installation of wick drains • Fill in portions of the MRGO in LPV 145

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Additional Design Information

Additional survey and subsurface information will be required for preparation of final plans, quantities, and specifications. Currently, the USACE has obtained some survey information during 2006 and 2007 as a component of the geotechnical investigations. This information will be incorporated into future design plans. Topographic surveys were conducted for LPV 145 between the MRGO and the borrow pit ponds located on the protected side of the levees. Construction plans of the borrow pits were also utilized. Horizontal and vertical control points were located along the

levee crest at 2,000-foot intervals. The topographic surveys should be tied to the USACE baseline.

Based on this recommended alternative and the new information obtained, the following minimum survey information is required for preparation of final design plans, quantities, and specifications:

- Re-establish the USACE baseline;
- Verify the profile of the top of levee every 50 feet and identify any intermediate grade breaks;
- Verify the 200-foot cross sections extending from the protected-side ROW line to the bank of the MRGO;
- Develop hydrographic cross sections of the MRGO every 600 feet to the midpoint of MRGO;
- Perform topographic survey of the proposed borrow pits; and
- Perform topographic survey of features such as fences, utilities, power poles, ditches, borrow pits, canals, and culverts.

Geotechnical investigations were recently completed on LPV 145 by the USACE. The primary components of the investigations included drilling undisturbed borings, in-situ field vane shear tests, in-situ cone penetrometers (piezocones/penetrometer test [CPTu]), and multiple electrode resistivity (MER) surveys.

For the undisturbed borings, a total of 55 were drilled. There were 17 pairs of undisturbed borings (34 total undisturbed borings) spaced at approximately 2,000-foot intervals along the levee. Each undisturbed boring pair consisted of a centerline boring with an adjacent boring located on the unprotected side between the MRGO and the existing levee berm on the flood side. Crest borings were about 80 feet deep. The MRGO side borings are about 60 feet deep. Borings were drilled in pairs in order to compare natural soil strengths against the improved soil strengths under the levee. The other borings were drilled for either QA, quality control (QC) or to investigate soft spots (as identified by CPT). Most ranged from 32 to 60 feet deep, with the exception of one 150-foot deep boring and a nearby 150-foot CPT pushed near the Bayou Bienvenue Control Structure.

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Samples obtained from the drilled borings were collected continuously using 5-inch diameter Shelby tubes that were 54 inches long. The samples were taken to a temporary laboratory located in Belle Chasse, Louisiana. A continuous visual log of all the tubes for each boring was visually classified in the laboratory. The following strength tests were performed on selected samples: triaxial compression (3-point UU or Q) was coupled with an unconfined compression test, triaxial compression (CU or R bar), direct simple shear, mini-vane, and hand-held tore vanes. A consolidation test was performed on a sample from half of the borings. Soil classification tests performed included Atterberg limits, sieve analysis, specific gravity, hydrometer, and organic content. Sample density and moisture content were determined.

At each undisturbed boring location, a CPTu was pushed. CPTu is an in-situ technology that is used to determine stratigraphy and to estimate geotechnical parameters such as shear strength and permeability. A field vane shear was also pushed at each undisturbed boring location. A field vane shear is used to determine in-situ shear strength and remolded shear strength. The data obtained from the in-situ methods are compared and correlated with the laboratory results. CPTu were pushed along the protected-side berm at approximately 500-foot intervals and along the levee crest and MRGO side berm at about 1,000-foot intervals. Dissipation tests were performed to determine substrata permeability. The MER survey was performed along the entire length of LPV 145 along the levee crest. Approximately 45 CPTu were pushed to characterize MER survey anomalies.

For the recommended alternative, the borings along the centerline would need to be extended to reach pile depths. Centerline borings spaced at approximately 2,000-foot centers and drilled up to 150 feet are proposed in combination with the existing centerline CPTs. The depths of the CPT should be confirmed and possibly extended. Also, verification of the subsurface requirements on the protected side should be confirmed. At this time, CPTs, spaced at approximately 250-foot spacings, were performed; however, no undisturbed borings on the protected side were drilled.

Another consideration for final design would be to include static pile load tests for both axial and tension loading conditions. By completing these tests, the factor of safety for pile design decreases thus reducing overall pile lengths and resulting in cost savings. It is recommended that these be included in the design process.

The availability for borrow was unknown at the time of this report. If new borrow sites are required, additional subsurface and environmental investigations will be required at

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the proposed borrow pits. The quality of the borrow needs to follow specifications for the compacted fill because this material will be used for the new levee. The availability for the required borrow for the preferred alternative may adversely affect the proposed schedule if resources are limited.

3.2.8 Design and Construction Schedules

The proposed schedule summarizes the time required for design and construction of the preferred alternative for LPV 145. The schedule is based on a notice to proceed date of December 18, 2008. A December 2008 notice to proceed is assumed based upon ongoing design work and allowing time for negotiations. Approximately 5 months are scheduled for geotechnical testing. Final design will require approximately 10 months and will be submitted on December 21, 2009. Real estate acquisition and environmental compliance have not been completed.

Construction of the T-wall will commence on April 8, 2010, and is slated for final acceptance on September 25, 2012. Construction access for LPV 145 will only be possible via a temporary bridge or by barge. This adds additional construction time. The potential to cut back construction time still exists, because the reach may be accessible via the MRGO. However, this is subject to change based on the proposed decommissioning of the MRGO to navigable traffic. The T-wall structure will be built using the existing levee and, therefore, will lag behind the soil amendment task. The schedule also depicts a lag time between the commencement of the pile driving crew and actual T-wall installation.

Access to the site is a major constraint. There is no current access point due to being located between two floodgates. The potential for water access by barge along the MRGO may be hindered due to the plan to close the MRGO near the junction with the Intracoastal Waterway.

All work is estimated using 10-hour workdays on a 7-day-per-week work schedule. Multiple crews for the pile and sheet pile driving are assumed to work concurrently to meet the deadline.

Work on LPV 145 will not be completed prior to the 2011 hurricane season based on the assumed production rates. Additional support, improved construction access, or revised scheduling will be required to complete this work by the mandated deadline.

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3.2.9 Report Plates – LPV 145

Summary tables of the geotechnical analysis along with the plan and profile and typical sections are combined for each of the alternatives and presented by alternative following this report.

<u>Alternative 1 – T-wall Alternative on Existing Levee (Plates 3.2.1 – 3.2.11)</u>

- LPV 145 Plan and Profile Sheets for T-wall Alternative
- Typical Sections for T-wall Alternative

<u>Alternative 2 – Earthen Levees Using Stability Berms (Plates 3.2.12 – 3.2.30)</u>

- LPV 145 Plan and Profile Sheets for Stability Berms Alternative
- Typical Sections for Stability Berms Alternative

<u>Alternative 3 – Earthen Levees Using Deep Soil Mixing and Landside Shift (Plates 3.2.31 – 3.2.41)</u>

- LPV 145 Plan and Profile Sheets for Soil-Mixing Alternative
- Typical Sections for Soil-Mixing Alternative

<u>Alternative 4 – Earthen Levees Using Stability Berms with Staged Construction and Wick Drains (Plates 3.2.42 – 3.2.52)</u>

- LPV 145 Plan and Profile Sheets for Stability Berms with Staged Construction and Wick Drains
- Typical Sections for Stability Berms with Staged Construction and Wick Drains

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3.3 LPV 146 – Chalmette Loop Levee – Bayou Dupre to Highway 46

LPV 146 is a levee segment located between the Bayou Dupre Control Structure and Highway 46 between Stations 708+65 and 1115+00 (40,635 feet in length). The majority of the levee is vegetated with the exception of the dirt road at the protected-side toe. Borrow pits were observed near the levee toes with light woods and brush nearby. On the flood side, the MRGO was located between approximately 300 feet to more than 680 feet from the centerline of the levee between Stations 708+65 and 1008+00. The remainder of the segment had water located a substantial distance from the centerline. Relocation of utilities was not considered throughout this reach as per USACE direction. Any utility relocation in this reach will be addresses during the plans and specifications phase.

LPV 146 - Borrow pits adjacent to protected side of existing levee

3.3.1 Design Evaluations

There are three hydraulic reaches (SB13, SB15, and SB16) in LPV 146. For each hydraulic reach, four engineering alternatives were evaluated. Initially, analysis of the levee, with overbuild and scour protection, was completed without stability berms up to the 2010/2011 hydraulic condition. In all cases, the safety factor for the existing

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condition was significantly less than required and additional measures were necessary. The four evaluated engineering alternatives included:

(1) T-wall alternative on existing levee;

(2) Earthen levees using stability berms (Oct 07 Alternative);

(3) Earthen levees using deep soil mixing and landside shift; and

(4) Earthen levees using stability berms with staged construction and wick drains.

Geometric Sections

For this reach, two representative sections were selected for the analysis. Station 1003+00 was used to represent the part of LPV 146 from Stations 708+65 to 1008+00 where the MRGO presents on the flood side. Station 1040+00 was used to represent the part from Stations 1008+00 to 1115+00 where the sections do not intersect with the MRGO.

Station 1003+00

The survey cross section at Station 1003+00 extended about 350 feet on the flood side and 1,400 feet on the protected side. The surface geometry was further extended to include the MRGO on the flood side based on aerial images, MRGO bathometry, and engineering judgment. This section was selected because it is representative of the typical flood-side condition with regard to the distance to the MRGO from the existing levee. The distance from the centerline of the existing levee to the MRGO was approximately 350 to 450 feet. This geometry section was used in combination with design shear strengths in the first two soils reaches in LPV 146 (Reach B: Stations 708+65 to 965+00 and Reach C: Stations 965+00 to 1008+00).

Station 1040+00

The survey cross section at Station 1040+00 extends approximately 600 feet on the protected side and 380 feet on the flood side. The cross section is located after the sharp turn of the levee alignment and the flood side does not intersect with the MRGO. This section was chosen because of its relative low elevations on the protected side. The ground surface is assumed to be flat (same elevation as the last known point) on both protected and flood sides if any further extension was needed

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in slope stability analysis. This geometry section was used in combination with design shear line in the last soils reach in LPV 146 (Reach D: Stations 1008+00 to 1115+00).

Soils Reaches

Based on soils results, it appears that the soils are much weaker between Stations 965+00 to 1008+00, where underlying soils consist mainly of cohesive soils. A sand layer appears to be present at EL -21 to -34 and EL -20 to -38 from Stations 708+65 to 965+00 and Stations 1008+00 to 1115+00, respectively. To take into account for the varying soil properties in LPV 146, LPV 146 was further divided into three soils reaches with different design shear strength characteristics: Soils Reach B: Stations 708+65 to 965+00, Soils Reach C: Stations 965+00 to 1008+00, and Soils Reach D: Stations 1008+00 to 1115+00. The shear line diagrams can be found in Appendix D, which were determined based on newly completed borings and laboratory test results, as well as field CPT data. A centerline design shear line was separated from the toe design shear line in each soils reach. A brief discussion of the shear strengths in each soils reach is presented below:

Soils Reach B (Stations 708+65 to 965+00)

Based on the laboratory test results, the existing levee fill materials have an undrained shear strength on the order of 600 psf. No continuous peat layer was justified based on the boring information. In the centerline, the shear strength increased from 400 psf at EL 4 to 600 psf at EL -21 at a rate of 8 psf per foot. This is the clay layer between the bottom of the existing levee fill to the top of the sand layer. In contrast, in the toe area of this layer, the shear strength increased from 200 psf at EL 4 to 420 psf at EL -21 at a rate of 8.8 psf per foot. The sand layer, which is located between EL -21 to -34, was modeled with an internal friction angle of 30 degrees in accordance with the USACE local experience in this area. In the clays below the sand layer, the shear strength increased from 700 psf at EL -34 to 1,200 psf at EL -80 at a rate of 10.9 psf per foot in the centerline area. In the toe area, the shear strength increased from 540 psf at EL -34 to 1,000 psf at EL -80 at a rate of 10 psf per foot.

Soils Reach C (Stations 965+00 to 1008+00)

As stated previously, subsurface explorations indicated cohesive soils predominantly present in this soils reach. Similar to Soils Reach B, the existing levee fill materials

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have an undrained shear strength on the order of 600 psf. A peat layer was present between EL -18 to EL -23 at the centerline, while the peat layer was located between EL -12 to EL -18 at the toe. However, based on the laboratory tests, the shear strength of the peat layer was similar to that of the surrounding soils and hence did not warrant being separated as another layer in terms of shear strength characteristics. In the centerline, the shear strength increased from 350 psf at EL 5 to 650 psf at EL -40 at a rate of 6.7 psf per foot, then the shear strength increased from 500 psf at EL -40 to 900 psf at EL -80 at a rate of 10 psf per foot. In the toe area, the shear strength was almost constant at 300 psf between EL 5 to EL -18 and then increased from 300 at EL -18 to 900 psf at EL -80 at a rate of 9.7 psf per foot.

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Soils Reach D (Stations 1008+00 to 1115+00)

Overall, this soils reach has a soil property characteristic that is comparable to Soils Reach B. Similar to the previous two soils reaches, the existing levee fill materials have an undrained shear strength on the order of 600 psf. A peat layer was identified between EL -14 to EL -20. Again, because the testing results of shear strengths do not indicate significant changes between the peat and the surrounding soils, this peat layer was not separately discussed in terms of shear lines. In the centerline, the shear strength increased from 400 psf at EL 5 to 600 psf at EL -20 at a rate of 8 psf per foot. This is the clay layer between the bottom of the existing levee fill to the top of the sand layer. In contrast, in the toe area of this layer, the shear strength increased from 250 psf at EL 5 to 500 psf at EL -20 at a rate of 10 psf per foot. The sand layer, which is located between EL -20 to EL -38, was modeled with an internal friction angle of 30 degrees in accordance with USACE local experience in this area. In the clays below the sand layer, laboratory test results showed very similar shear strength behaviors between the centerline and the toe. As a result, the modeled shear line coincided below EL -38. The shear strength increased from 700 psf at EL -38 to 1,200 psf at EL -80 at a rate of 11.9 psf per foot.

Limited consolidation test data were available to determine the following design consolidation parameters: compression index; coefficient of consolidation; and initial void ratio. Given the fact that the settlement analysis is very sensitive to the values of compression indices, Skempton's empirical relationship between liquid limit and compression index was also used to help judge the design compression index versus depth because only limited laboratory data were available in some boring locations at various depths. It is also assumed that all soils in the settlement analysis were normally consolidated. As a result, the recompression index was forced to be the same value as the virgin compression index for the same soil layer. Coefficient of

consolidation and initial void ratio were determined in a similar way as compression index except that no empirical correlations were used. Those design consolidation parameters versus depths were also presented in Appendix D. It should be noted that those coefficient of consolidation points shown on the figures were the average values from laboratory tests under 2,000 and 4,000 psf incremental loads, reported from both Logarithm-of-Time Method and Square-Root-of-Time Method. Not like the design shear strength lines, due to limited data points in consolidation parameters, centerline and toe design lines were not distinguished for compression index, coefficient of consolidation, and initial void ratio.

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3.3.2 Alternative 1 – T-wall Alternative on Existing Levee (Plates 3.3.1 – 3.3.14)

In general, Alternative 1 consists of a reinforced concrete T-wall supported by steel H-piles to be built on the existing levee. The existing levee may be slightly degraded in order to drive H-piles, drive steel sheet pile for seepage cut-off, and place a stone pad for placement of the concrete footing. The foundations were typically embedded approximately 3.0 feet into the crown for erosion protection. From the flood-side toe to protected-side toe, 18 inches of riprap was assumed to be placed for scour protection and provide a reasonable cost for this protection. Stability berms are not used or required in this alternative. Alternative 1 is designed for future (2057/2060) conditions only.

3.3.2.1 Geotechnical and Structural Results

For this option, the existing levee would be slightly raised in Hydraulic Reach SB16 (Stations 1070+82.2 to 1115+00) to meet the hydraulic design elevations while in other portions of LPV 146 the existing levee would be kept in place. Unbalanced loads that result from global instability are transferred into the driven piles and are reported in the tables. For these walls, HP 14x89 piles are used. Typical sections illustrating this option are provided at the end of this report section along with the required construction limits. The results of the geotechnical and structural analysis are summarized below:

T-wall Analysis Structural Summary								
Hydraulic Reach	**Soils Reach**	**Station Limits**	**Top of Wall Elevation**	**Stem Height (feet)**	**Stem/Base Thickness (feet)**	**Base Width**	**Pile Tip Elevation**	**No. of Piles in Cross Section**
SB13	B	708+65 to 821+22.6	27.5	9.0	2.0/2.5	10.0	-80.0	2
SB15	B	821+22.6 to 965+00	27.5	9.0	2.0/2.5	10.0	-80.0	2
SB15	C	965+00 to 1008+00	27.5	9.0	2.0/2.5	11.0	-95.0	3
SB15	D	1008+00 to 1070+82.2	27.5	9.3	2.0/2.5	10.0	-90.0	2
SB16	D	1070+82.2 to 1115+00	29.5	10.5	2.0/2.5	10.0	-90.0	2

Assumptions: 4,000 pounds per square inch compression strength; 5-foot pile center spacing along wall; and steel HP 14x89 piles on a 1H:3V batter perpendicular to wall.

T-wall Analysis Geotechnical Summary						
Station Limits	**Hydraulic Reach**	**Soils Reach**	**Sheet Pile Tip Elevation**	**Fictitious Surface Elevation**	**Maximum Anchor Force (lbs/ft)**	**Maximum Moment (lbs-ft/ft)**
708+65 to 965+00	SB13 and SB15	B	-39.0	NA	NA	NA
965+00 to 1008+00	SB15	C	-68.9	-52.2	3,904	61,415
1008+00 to 1115+00	SB15 and SB16	D	-43.0	NA	NA	NA

lbs/ft Pounds per foot.
lbs-ft/ft Pounds per foot per foot.

In the above table, “NA" stands for “not applicable”, which typically means that the T-wall is not subjected to any unbalanced forces from slope stability analysis. Some benching of the existing levee may be necessary in order to drive the piles at the top of the levee due to the steep existing slopes and levee height. Because no dense sands were identified from the borings from Stations 965+00 to 1008+00, steel piles are expected to be driven easily into the ground. Seepage will be controlled by

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driving sheet piles into the ground to the designed elevations as shown in the table above.

The use of stability berms and/or deep soil mixing may be applied here to reduce unbalanced loads, fictitious surfaces, and thus the pile lengths. However, adding stability berms will likely require additional ROW and local stability issues as seen in other alternatives within this reach. Deep soil mixing will also add to the cost but may reduce pile loads and lengths. These options could be considered in the future should the T-wall need to be raised with minimal impacts to the foundations.

3.3.2.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

All construction for Alternative 1 occurs within the existing ROW and will not require any regrading of the levee section. Therefore, the T-wall alternative requires no additional ROW. Plates following this report present the construction limits. All staging and stockpiling are proposed to occur within the construction limits presented in these plates.

Real Estate Description	Quantity	Unit
Additional Right-of-Way	0	Acre
Additional Borrow Pit Area	0	Acre

Borrow pit excavations noted along the toe of the levee are located on both sides of the levee from Station 1011+00 to Station 1113+00. A thin scrub-shrub buffer exists between the borrow pits and the levee. This buffer is comprised of cordgrass (*Spartina*) and other marsh grass and scrub-shrub vegetation. Farther on the protected side beyond the bayou is an expansive swamp, with a mixture of common grasses and trees. This is high-value habitat. There is little marsh vegetation along the MRGO face of the levee, with the exception of a thin edge buffer of cordgrass (*Spartina*) and some other scrub-shrubs. Grasses populate the sides of the levee in varying thickness. The end of the levee approaching Highway 46 is comprised of a more viable marsh habitat, with more lush vegetation and higher species diversity than the thin buffer strips growing along either side of the levee.

The T-wall alternative does not require drilling along the existing levee, filling of wetlands, or clearing of vegetation outside the existing levee landform. This alternative has a negligible effect on the surrounding environment, and the impact that does occur

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will be along the levee section, which is not an important habitat area. For these reasons, the T-wall alternative has the least environmental impact of any of the proposed alternatives. This alternative will not affect the MRGO channel.

3.3.3 Alternative 2 – Earthen Levees Using Stability Berms (Oct 07 Alternative) (Plates 3.3.15 – 3.3.38)

Alternative 2, earthen levees using stability berms, consists of constructing additional earthen levee lifts along the centerline of the existing levees. From flood-side levee toe to protected-side levee toe, 18 inches of riprap was assumed for scour protection as noted in previous paragraphs. The initial section, taken from the Hydraulic Report, was altered to meet the required geotechnical factors of safety.

3.3.3.1 Geotechnical Results

Similar to LPV 145, complete analysis was conducted for existing hydraulic conditions (2010/2011 design elevations) while only limited slope stability analysis was performed for future conditions (2057/2060 design elevations). Additional lifts were proposed to reach 2057/2060 design elevations by 2057/2060 based on time-rate settlement analysis. The scheduling of the future lift(s) is provided with the typical sections attached to this report. Due to settlement, a certain amount of overbuild has to be included for the proposed levee and the information is summarized in the tables below.

Summary of Overbuild for 2010/2011 Design Elevations

Station Limits	Hydraulic Reach	Soils Reach	2010/2011 Levee Design Elevation	Proposed Top of Levee Elevation	Proposed Top of Riprap Elevation	Overbuild Thickness (feet)
708+65 to 821+22.6	SB13	B	26.5	30.0	31.5	3.5
821+22.6 to 965+00	SB15	B	26.5	30.0	31.5	3.5
965+00 to 1008+00	SB15	C	26.5	30.0	31.5	3.5
1008+00 to 1070+82.2	SB15	D	26.5	28.5	30.0	2.0
1070+82.2 to 1115+00	SB16	D	26.5	28.5	30.0	2.0

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Summary of Overbuild for 2057/2060 Design Elevations						
Station Limits	**Hydraulic Reach**	**Soils Reach**	**2057/2060 Levee Design Elevation**	**Proposed Top of Levee Elevation**	**Proposed Top of Riprap Elevation**	**Overbuild Thickness (feet)**
708+65 to 821+22.6	SB13	B	29.0	32.5	34.0	3.5
821+22.6 to 965+00	SB15	B	29.0	32.5	34.0	3.5
965+00 to 1008+00	SB15	C	29.0	34.0	35.5	5.0
1008+00 to 1070+82.2	SB15	D	29.0	32.5	34.0	3.5
1070+82.2 to 1115+00	SB16	D	29.0	32.5	34.0	3.5

For both 2010/2011 and 2057/2060 design elevations, the levee has a crown approximately 10 feet wide and slopes down at a 4H:1V until intersecting the stability berms on the protected side. The flood side has a wave berm, as proposed by hydraulic engineers at the USACE, which was altered in order to meet the required factor of safety for the flood-side stability. Typical sections illustrating this option are provided at the end of this report section along with the required construction limits. Based on the geotechnical analysis results, the approximate berm dimensions on both protected and flood sides are summarized below:

Summary of the Stability Berm Dimensions (2010/2011 design elevations)					
Station Limits	**Hydraulic Reach**	**Soils Reach**	**Distance of Toe from Levee Centerline (PS/FS) (feet)**	**Length (PS/FS) (feet)**	**Average Depth (PS/FS) (feet)**
708+65 to 821+22.6	SB13	B	312/269	253/197	11/12
821+22.6 to 965+00	SB15	B	312/269	253/169	11/11
965+00 to 1008+00	SB15	C	403/345	350/271	13/13
1008+00 to 1115+00	SB15 and SB16	D	231/173	168/114	4.5/5.5

PS Protected Side.
FS Flood Side.

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Summary of the Stability Berm Dimensions (2057/2060 design elevations)					
Station Limits	**Hydraulic Reach**	**Soils Reach**	**Distance of Toe from Levee Centerline (PS/FS) (feet)**	**Length (PS/FS) (feet)**	**Average Depth (PS/FS) (feet)**
708+65 to 821+22.6	SB13	B	357/298	298/222	13/13
821+22.6 to 965+00	SB15	B	357/298	298/233	13/13
965+00 to 1008+00	SB15	C	473/395	419/330	15/14
1008+00 to 1115+00	SB15 and SB16	D	258/199	198/140	8/11

PS Protected Side.
FS Flood Side.

The size of the berm is the largest in Soils Reach C (Stations 965+00 to 1008+00) because it has the lowest shear strengths. Typical ROW distance from centerline of levee is 525 feet on the protected side and 350 feet on the flood side between Stations 708+65 to 1008+00. Between Stations 1008+00 and 1115+00, the typical ROW distance from centerline of levee is 400 feet on both the protected side and the flood side. Based on the table above, the proposed berms will stay within ROW in LPV 146 for 2010/2011 and 2057/2060 design elevations with the exception of the flood-side berm in Soils Reach C (Stations 965+00 to 1008+00), which will go beyond the ROW on the order of 45 feet for 2057/2060 design elevations.

Compacted clay fill was used for both the raised levee and stability berms because of the increased strength, more stringent testing requirements, and heavier unit weight. However, for erosion purposes, riprap fill was used for the small portion of the flood-side berm where the berm intersects with existing ground surface between Stations 708+65 to 1008+00. On the flood side, it is expected that the distance from the MRGO to the centerline of the levee varies significantly along the levee alignment in LPV 146 between Stations 708+65 to 1008+00. As a result, riprap should be used for any filling underwater.

Seepage analysis was only performed for 2010/2011 design elevations. The berm on the protected side was proposed primarily for stability purposes. At the same time, the stability berms also act as seepage berms by providing additional weight to counteract those upward seepage forces. However, as compared to LPV 145, the size of the

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berm in LPV 146 is relatively small and the critical exit gradient is greater than 0.5 without any additional measures. As a result, sheet pile cut-offs were proposed for seepage control purposes as summarized below:

Seepage Analysis Summary			
Station Limits	**Sheet Pile Location**	**Sheet Pile Tip Elevation**	**Sheet Pile Depth (feet)**
708+65 to 965+00	C/L	-39.0	69
965+00 to 1008+00	C/L	-28.0	58
1008+00 to 1115+00	PS Toe	-43.0	49

C/L Centerline
PS Protected Side.

Depending on the effectiveness of blocking seepage flows, the sheet pile is to be located at the levee center in Soils Reaches B and C and to be located near the protected-side berm toe in Soils Reach D. The sheet pile cut-off is just one alternative to remedy the seepage criteria. Slurry cut-offs and toe drains or relief wells could be used as other measures. Determination of the most effective seepage cut-off will be addressed during final design. Sheet pile was used for cost purposes only.

3.3.3.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following utilities/facilities will require relocation based on LPV 146 Oct 07, 2010, alternative:

LPV 146 Oct 07 Alternative 2			
Right-of-Way			
Description		**Quantity**	**Units**
Additional Right-of-Way required		1.03	Acre
Facility Relocation			
Description		**Quantity**	**Units**
1115+00	Existing fence	50	Foot

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A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following utilities/facilities will require relocation based on LPV 146 Oct 07, 2057, alternative:

LPV 146 Oct 07 Alternative 2			
Right-of-Way			
Description		**Quantity**	**Units**
Additional Right-of-Way required		6.53	Acre
Facility Relocation			
Description		**Quantity**	**Units**
1115+00	Existing fence	50	Foot

Borrow pit excavations noted along the toe of the levee are located on both sides of the levee from Station 1011+00 to Station 1113+00. A thin scrub-shrub buffer exists between the borrow pits and the levee. This buffer is comprised of cordgrass (*Spartina*) and other marsh grass and scrub-shrub vegetation. Farther on the protected side beyond the bayou is an expansive swamp, with a mixture of common grasses and trees. This is high-value habitat. There is little marsh vegetation along the MRGO face of the levee, with the exception of a thin edge buffer of cordgrass (*Spartina*) and some other scrub-shrubs. Grasses populate the sides of the levee in varying thickness. The end of the levee approaching Highway 46 is comprised of a more viable marsh habitat, with more lush vegetation and higher species diversity than the thin buffer strips growing along either side of the levee.

The stability berm alternative for the 2057/2060 design will extend 258 to 473 feet on the protected side and 199 to 395 feet on the flood side from the centerline to the levee toe, completely filling the borrow pits and the buffer on the protected side of the levee. The construction will eliminate, to some extent, a high-value wetland on the protected side of the levee. The low-value habitat between the levee centerline and the MRGO between Stations 708+65 and 1008+00 will be filled in completely. In addition, viable marsh habitat on both sides of the levee approaching Highway 46 will be eliminated.

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3.3.4 Alternative 3 – Earthen Levees Using Deep Soil Mixing and Landside Shift (Plates 3.3.39 – 3.3.51)

Alternative 3 is the earthen levee using deep soil mixing, which is constructed by first installing deep soil-mixed panels on the protected side of the existing levee toe and then placing earthen embankment on top of this improved foundation soil. The centerline of the proposed levee is offset from the existing levee and positioned on the deep soil-mixed panels. Scour protection, consisting of 18 inches of riprap, was assumed (for cost estimation purposes) from toe to toe of the entire cross section.

3.3.4.1 Geotechnical Results

Because of the nature of this alternative, the foundation of the deep soil mixing must be prepared to handle future (2057/2060) design elevations and was the reason for investigating this alternative up to this level; however, interim or lower height lifts for the existing condition could feasibly be investigated after the foundation is prepared for future conditions. Overbuild was also included to accommodate settlement over time. Design information used to model this alternative is summarized in the table below.

Summary of Overbuild for 2057/2060 Design Elevations

Station Limits	Hydraulic Reach	Soils Reach	2057/2060 Levee Design Elevation	Proposed Top of Levee Elevation	Proposed Top of Riprap Elevation	Overbuild Thickness (feet)
708+65 to 821+22.6	SB13	B	29.0	32.5	34.0	3.5
821+22.6 to 965+00	SB15	B	29.0	32.5	34.0	3.5
965+00 to 1008+00	SB15	C	29.0	31.0	32.5	2.0
1008+00 to 1070+82.2	SB15	D	29.0	33.0	34.5	4.0
1070+82.2 to 1115+00	SB16	D	29.0	33.0	34.5	4.0

The levee has a crown approximately 10 feet wide and slopes down at a 4H:1V or 5H:1V until intersecting the existing ground surface on the protected side. The flood side has a proposed wave berm as suggested by the USACE. The existing levee crown may need to be partially excavated to accommodate the proposed wave berm

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on the flood side. The centerline of the proposed levee was shifted toward landside from existing levee centerline with the deep soil-mixing option in order to construct the columns/panels. The location of the deep soil mixing was placed between the existing levee protected-side toe to the proposed new levee protected-side toe. Based on the results of the analysis, the approximate soil-mixing regions are as follows:

Soil-Mixing Regions					
Station Limits	**Hydraulic Reach**	**Soils Reach**	**Width (feet)**	**Average Depth (feet)**	**Bottom Elevation of DSM Region**
708+65 to 965+00	SB13 and SB15	B	201	26	-21
965+00 to 1008+00	SB15	C	224	65	-60
1008+00 to 1115+00	SB15 and SB16	D	227	23	-20

DSM Deep Soil Mixing.

The lengths and depths were altered until all design cases were met for the most appropriate section. Due to the improved shear strength of the soil mass, failure surfaces were forced deeper or longer to reach weaker material. A maximum shear strength of 2,300 psf was used for the improved soil mass; however, the factor of safety for the shallower layers likely could be designed with less strength depending on the internal design and numerical analysis required for future design.

Typical sections illustrating this option are provided at the end of this report section along with the required construction limits.

For seepage, the soil-mixing block did not provide enough protection against excessive exit gradients and a sheet pile cut-off wall is required. As stated previously, sheet pile is just one alternative to cut-off seepage issues. Slurry walls or relief wells could also be considered during final design. Sheet piles were used for costing purposes for this EAR-RHCA, and the results are summarized below:

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Seepage Analysis Summary			
Station Limits	**Sheet Pile Location**	**Sheet Pile Tip Elevation**	**Sheet Pile Depth (feet)**
708+65 to 965+00	PS Toe	-39.0	40
965+00 to 1008+00	PS Toe	-24.0	26
1008+00 to 1115+00	C/L	-43.0	76

C/L Centerline
PS Protected Side.

3.3.4.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following utilities/facilities will require relocation based on LPV 146 Soil-Mixing Alternative 3:

LPV 146 Soil-Mixing Alternative 3			
Right-of-Way			
Description		**Quantity**	**Units**
Additional Right-of-Way required		0.0	Acre
Facility Relocation			
Description		**Quantity**	**Units**
1115+00	Existing fence	50	Foot

When compared with the stability berm alternatives, Alternative 3 minimizes impacts on the valuable surrounding wetland habitats because it does not require filling of any area outside the existing landform. The soil-mixing alternative requires drilling along the existing levee as well as in the borrow pits; however, this is a temporary disruption to the wetland system and will not extend outside the limits of the borrow pits. Neither filling of wetland nor clearing of vegetation outside the existing levee landform is required for this soil-mixing alternative. The soil-mixing alternative represents a smaller impact than any of the stability berm alternatives. This alternative will not affect the MRGO channel.

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3.3.5 Alternative 4 – Earthen Levees Using Stability Berm with Staged Construction and Wick Drains (Plates 3.3.52 – 3.3.64)

Alternative 4 is an earthen levee using staged construction and wick drains. This alternative is constructed by first placing a sand blanket on top of the existing levee and installing wick drains. An earthen levee is placed on top of the sand blanket to the highest elevation possible for the existing soil strength, and a steel sheet pile or other seepage cut-off is installed down the center to prevent water from flowing through the levee. A time frame of approximately 1 year is required prior to the placement of the second lift of earthen embankment and 18 inches of riprap scour protection. The time frame is to allow the water to dissipate from the foundation soils so that the shear strength of the soils increases, thus allowing higher levees and less footprint is required.

3.3.5.1 Geotechnical Results

Because of the nature of this alternative, the foundation of the soil must be prepared to handle future (2057/2060) design elevations and was the reason for investigating this alternative up to this level; however, interim or lower height lifts for the existing condition could be investigated after the foundation is prepared for future conditions. Overbuild was also included to accommodate settlement over time.

The staged construction is anticipated to be constructed in a total of two lifts. The first lift will be raised to top of levee EL 26.0, EL 27.0, and EL 28.6 for Soils Reaches B, C, and D, respectively. The first lift will be allowed to settle for 1 year. At the end of 1 year, the strength gain for each soil layer due to surcharge is calculated based on the degree of consolidation in that soil layer, considering the effect of wick drains. The second lift will be raised to the future design elevation, which is summarized in the table below.

Summary of Overbuild for 2057/2060 Design Elevations

Station Limits	Hydraulic Reach	Soils Reach	2057/2060 Levee Design Elevation	Proposed Top of Levee Elevation	Proposed Top of Riprap Elevation	Overbuild Thickness (feet)
708+65 to 821+22.6	SB13	B	29.0	33.0	34.5	4.0
821+22.6 to 965+00	SB15	B	29.0	33.0	34.5	4.0

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Summary of Overbuild for 2057/2060 Design Elevations						
Station Limits	**Hydraulic Reach**	**Soils Reach**	**2057/2060 Levee Design Elevation**	**Proposed Top of Levee Elevation**	**Proposed Top of Riprap Elevation**	**Overbuild Thickness (feet)**
965+00 to 1008+00	SB15	C	29.0	33.5	35.0	4.5
1008+00 to 1070+82.2	SB15	D	29.0	32.5	34.0	3.5
1070+82.2 to 1115+00	SB16	D	29.0	32.5	34.0	3.5

The levee has a crown approximately 10 feet wide and slopes down at a 4H:1V until intersecting the stability berms. The centerline of the proposed levee coincides with the existing levee centerline for this option. Compacted clay fill was used for both the levee and stability berms because of the increased strength, more stringent testing requirements, and heavier unit weight. Based on the results of the analysis, stability berms are required. The stability berms on both the protected and flood sides have the approximate dimensions shown in the table below:

Summary of the Stability Berm Dimensions (2057/2060 design elevations)					
Station Limits	**Hydraulic Reach**	**Soils Reach**	**Distance of Toe from Proposed Levee Centerline (PS/FS) (feet)**	**Length (PS/FS) (feet)**	**Average Depth (PS/FS) (feet)**
708+65 to 965+00	SB13 and SB15	B	317/268	290/241	13/13
965+00 to 1008+00	SB15	C	439/350	396/323	9/10
1008+00 to 1115+00	SB15 and SB16	D	197/176	136/149	8/15

PS Protected Side.
FS Flood Side.

Typical sections illustrating this option are provided at the end of the section along with the required construction limits. Based on the table above, it appears that Soils Reach C requires the largest size of berms on the flood side. Because the ROW is approximately 350 feet on the flood side in this soils reach, it is likely no additional

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ROW purchase will be necessary for LPV 146. Failure surfaces tend to exit near where the berms end due to significant decrease in overburden.

For seepage, the stability berms do not provide enough protection against excessive exit gradients, and a sheet pile cut-off wall is required. In order to reduce the exit gradient to within allowable limits, the tip of the sheet pile wall is anticipated to be located at EL -39, EL -50, and EL -43 for Soils Reaches B, C, and D, respectively. The effect of wick drains was modeled in the seepage analysis which contributed to the relative long sheet pile penetration depth in Soils Reach C.

3.3.5.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following utilities/facilities will require relocation based on LPV 146 Alternative 4 - Stability Berms with Staged Construction and Wick Drains:

LPV 146 Alternative 4 - Stability Berms with Staged Construction and Wick Drains			
Right-of-Way			
Description		**Quantity**	**Units**
Additional Right-of-Way required		2.14	Acre
Facility Relocation			
Description		**Quantity**	**Units**
1115+00	Existing fence	50	Foot

As described in Alternative 1, LPV 146 is bounded on the protected side by borrow pits and high-value wetlands beyond these. The flood side is characterized by the MRGO channel for much of the reach (Station 708+65 to Station 1008+00) and a similar borrow pit/wetland complex for the remainder of the reach. The wick drain alternative will impact approximately 315 acres of wetland and bottomland forest, which represents the smallest environmental impact of the stability berm alternatives. This alternative will eliminate the scrub-shrub buffer, submerged borrow pit, and the wetland area beyond these. The wick drain alternative has no impact on the MRGO channel.

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3.3.6 Summary of Cost Analysis

Below are tables summarizing the cost for each of the alternatives (not including escalation). The first table summarizes the costs for the two alternatives analyzed at EL 22.5. Detailed cost estimates can be found in Appendix H.

Description	Real Estate ($k)	Environmental Mitigation Including Contingency ($k)	Contract Cost Including Contingency ($k)	Engineering and Design Cost Including Contingency ($k)	Supervision and Administration Cost Including Contingency ($k)	Project Cost ($k)	Operation and Maintenance ($k)	Total Values ($k)
	1	2	3	4	5	1+2+3 +4+5 A	B	= A + B
LPV 146 Alternative 1: T-wall	-	-	$449,798	$56,225	$42,731	$548,754	$8,294	$557,048
LPV 146 Alternative 2: Oct 07 All Stages	$1.34	$110	$788,028	$98,503	$74,863	$961,505	$74,745	$1,036,250
LPV 146 Alternative 3: Deep Soil Mixing	-	-	$706,146	$88,268	$67,084	$861,499	$70,586	$932,085
LPV 146 Alternative 4: Staged with Wick Drain All Stage	-	-	$701,100	$87,638	$66,605	$855,342	$56,399	$911,741

For comparison purposes, the tables below provide the quantities of the major work items for each alternative presented above.

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LPV 146 – Alternative 1		
Summary of Major Quantities		
	Quantity	**Unit**
Riprap	197,061	Cubic Yard
T-wall Concrete	66,540	Cubic Yard
T-wall Sheet Pile	270,788	Square Yard
T-wall Pile	1,326,664	Foot

LPV 146 – Alternative 2		
Summary of Major Quantities		
	Quantity	**Unit**
Embankment Compacted Fill	10,448,448	Cubic Yard
Riprap	1,593,227	Cubic Yard
Clearing and Grubbing	1,147.1	Acre

LPV 146 – Alternative 3		
Summary of Major Quantities		
	Quantity	**Unit**
Embankment Compacted Fill	5,125,307	Cubic Yard
Riprap	389,978	Cubic Yard
Soil Mixing*	9,792,866	Cubic Yard
Clearing and Grubbing	256.9	Acre

*Quantity for soil mixing is for the composite region. A 40 percent replacement ratio was applied to determine the estimated quantity of soil-mixed columns and/or panels.

LPV 146 – Alternative 4		
Summary of Major Quantities		
	Quantity	**Unit**
Embankment Compacted Fill	9,874,802	Cubic Yard
Riprap	164,478	Cubic Yard

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LPV 146 – Alternative 4		
Summary of Major Quantities		
	Quantity	**Unit**
Clearing and Grubbing	875.1	Acre
Sheet Pile	329,039	Square Yard
Wick Drains	12,488,655	Foot

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3.3.7 Recommended Alternative

For this reach, the recommended alternative is Alternative 1 – T-wall Structure. This recommendation is based on the limited environmental impacts, reduced need for borrow, least cost, and no requirements for additional temporary easements or ROW. The proposed construction would not impact the MRGO in either case. The costs of the T-wall are significantly less than those of the levee Alternatives 2 through 4. Also, this alternative will tie-in directly to the new T-wall tie-ins for LPV 144.

For Alternatives 2 and 4, additional ROW will also be required, which will further increase these costs. The cost difference is largely due to the extensive fill. The availability and need for borrow are unknown at this time, which could also adversely impact the construction costs and schedule. Alternative 3 has similar advantages to Alternative 1 in comparison to Alternatives 2 and 4, and it also provides the advantages of a levee. However, the cost appears to significantly outweigh these advantages.

Advantages and disadvantages for all of the alternatives considered are presented below:

Alternative	Advantages	Disadvantages
T-wall on Existing Levee	• Construction with ROW • No stability berms required • Materials more readily available • Reduced environmental impacts • No additional interim measures required • Built to future elevation • Reduced life cycle cost • Additional levee lifts not required	• Hard to modify in future and major reconstruction if altered • Political/public perception of walls after Hurricane Katrina • Poor aesthetics • Routine inspections
Earthen Levee with Stability Berms – Oct 07	• Can modify for future lifts • No interim measures required • Greater vehicular access • Aesthetically pleasing • More eco-friendly	• Source of borrow • Additional scour protection required (heavier than current design) • Additional ROW may be required • Significant cost to implement protection up to future elevation • Fill in portions of the MRGO in LPV 145

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Alternative	Advantages	Disadvantages
Earthen Levee with Deep Soil Mixing	• Can reduce lift heights to existing design elevation • Improved subgrade/ foundation up to future design elevation • No interim measures required • Greater vehicular access • Aesthetically pleasing • More eco-friendly • No to very little ROW • No canal relocations • Less borrow required	• Landside shift • Difficult to excavate foundation in the future • Wet mixing produces excess spoil • Cost may be more significant • Additional scour protection required (heavier than current design) • Specialty contractors
Earthen Levee using Staged Construction & Wick Drains	• Reduced protection in 1st year • Can modify for future lifts • No interim measures required • Greater vehicular access • Aesthetically pleasing • More eco-friendly • Improved foundation strength with less time	• Source of borrow • Additional scour protection required (heavier than current design) • Additional ROW may be required • Seepage cut-off required • Installation of wick drains • Fill in portions of the MRGO in LPV 145

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Additional Design Information

In preparation of final design plans, quantities, and specifications, additional survey and subsurface information will need to be obtained. Topographic surveys were conducted for LPV 145 between the MRGO and the borrow pit ponds located on the protected side of the levees. Construction plans of the borrow pits were also utilized. Horizontal and vertical control points were located along the levee crest at 2,000-foot intervals. The topographic surveys should be tied to the USACE baseline.

Based on the recommended alternatives and the recent survey information, the following minimum survey information is required for preparation of final design plans, quantities, and specifications:

- Re-establish the USACE baseline;
- Verify the profile at the top of levee every 50 feet and identify any intermediate grade breaks;
- Verify 200-foot cross sections extending from the protected-side ROW line to the bank of the MRGO;
- Develop hydrographic cross sections of the MRGO every 600 feet to the midpoint of MRGO;
- Perform topographic survey of the proposed borrow pits; and
- Perform topographic survey of features such as fences, utilities, power poles, ditches, borrow pits, canals, and culverts.

Additional geotechnical investigations will also be required for development of final design beyond what was recently obtained by the USACE for LPV 146. The primary components of the investigations include drilling undisturbed borings, in-situ field vane shear tests, in-situ cone penetrometers (piezocones/CPTu), and MER surveys.

For the undisturbed borings, a total of 64 were drilled. There were 20 pairs of undisturbed borings (40 total undisturbed borings) spaced at approximately 2,000-foot intervals along the levee. Each undisturbed boring pair consisted of a centerline boring with an adjacent boring located on the unprotected side between the MRGO and the existing levee berm. Crest borings were about 80 feet deep. The MRGO side borings were about 60 feet deep. Borings were drilled in pairs in order to compare natural soil strengths against the improved soil strengths under the levee. The other borings were drilled for either QA/QC to investigate soft spots (as identified by CPT) or to investigate MER survey anomalies and ranged from 32 to 80 feet deep.

Samples from drilling were collected continuously using 5-inch-diameter Shelby tubes that were 54 inches long. The samples were taken to a temporary laboratory located in Belle Chasse, Louisiana. A continuous visual log of all the tubes for each boring

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was visually classified in the laboratory. The following strength tests were performed on selected samples: triaxial compression (3-point UU or Q) was coupled with an unconfined compression test, triaxial compression (CU or R bar), direct simple shear, mini-vane, and hand-held tore vanes. A consolidation test was performed on a sample from half of the borings. Soil classification tests performed included Atterberg limits, sieve analysis, hydrometer, specific gravity, and organic content. Sample density and moisture content were determined.

A CPTu was pushed adjacent to each undisturbed boring location. CPTu is an in-situ technology that is used to determine stratigraphy and to estimate geotechnical parameters such as shear strength and permeability. A field vane shear was also pushed at each undisturbed boring location. A field vane shear is used to determine in-situ shear strength and remolded shear strength. The data obtained from the in-situ methods were compared and correlated with the laboratory results. CPTu were pushed along the protected-side berm at approximately 250-foot intervals, along the levee crest at approximately 1,000-foot intervals, and along the MRGO side berm at about 500-foot intervals. Dissipation tests were performed to determine substrata permeability. The MER survey was performed along the entire length of LPV 146 along the levee crest. Additional CPTu were pushed and several borings were drilled to characterize MER survey anomalies.

In addition, a correlation study was conducted at LPV 146 near MRGO Station 975+00 at Boring 14U, which included undisturbed borings, CPT, seismic CPT, dilatometer, pressuremeter, and MER testing. QA/QC borings were also drilled, and samples were tested from each tube for the entire depth of the hole. The laboratory data were compared to CPT pushes with the Terracon 2.5- and 5-ton cones and the USACE Savannah District 5-ton CPTu. Seismic cone tests were conducted in the correlation study area. The field vane shear was used to determine in-situ soil shear strength at this location. In addition, the flat blade dilatometer and a pressuremeter were used at this location. The MER analysis was also conducted through the correlation study area.

For the recommended alternative, the borings along the centerline would need to be extended to reach pile depths. Centerline borings spaced at approximately 2,000-foot centers and drilled up to 150 feet are proposed in combination with the existing centerline CPTs. The depths of the CPT should be confirmed and possibly extended. Also, verification of the subsurface requirements on the protected side should be confirmed. At this time, CPTs, spaced at approximately 250-foot spacings, were performed; however, no undisturbed borings on the protected side were drilled.

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Another consideration for final design would be to include static pile loads tests for both axial and tension loading conditions. By completing these tests, the factor of safety for pile design decreases thus reducing overall pile lengths and resulting in cost savings. It is recommended that these be included in the design process.

The availability for borrow was unknown at the time of this report. If new borrow sites are required, additional subsurface and environmental investigations will be required at the proposed borrow pits. The quality of the borrow needs to follow specifications for the compacted fill because this material will be used for the new levee. The availability for the required borrow for the preferred alternative may adversely affect the proposed schedule if resources are limited.

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3.3.8 Design and Construction Schedules

The proposed schedule summarizes the time required for design and construction of the preferred alternative for LPV 146. The schedule is based on a notice to proceed date of December 18, 2008. A December 2008 notice to proceed is assumed based upon ongoing design work and allowing for negotiations. Approximately 5 months are scheduled for additional geotechnical testing along the levee. Final design will require approximately 12 months and will be submitted on December 21, 2009. Real estate acquisition and environmental compliance have not been completed.

Construction of the T-wall will commence on April 8, 2010, and is slated for final acceptance on approximately February 22, 2012. The T-wall structure will be built using the existing levee and, therefore, will lag behind the soil amendment task. The schedule also depicts a 70-day lag time between the commencement of the pile driving crew and actual T-wall installation.

Access to the site is a major constraint. There is one overland access point at Highway 46 and a potential for water access by barge along the MRGO. LPV 146 is considered the critical path for this Task Order.

All work is estimated using 10-hour workdays on a 7-day-per-week work schedule. Two crews for the pile and sheet pile driving are assumed to work concurrently. Work on LPV 146 will not be completed prior to the 2011 hurricane season based on the assumed production rates. Additional support, improved construction access, or revised scheduling will be required to complete this work by the mandated deadline.

3.3.9 Report Plates – LPV 146

Summary tables of the geotechnical and structural along with the plan and profile and typical sections are combined for each of the alternatives and presented by alternative following this report.

<u>Alternative 1 – T-wall Alternative on Existing Levee (Plates 3.3.1 – 3.3.14)</u>

- LPV 146 Plan and Profile Sheets for T-wall
- Typical Sections for T-wall
- Detailed Structural Section of T-wall

<u>Alternative 2 – Earthen Levees Using Stability Berms (Plates 3.3.15 – 3.3.38)</u>

- LPV 146 Plan and Profile Sheets for Stability Berm Alternative
- Typical Sections for Stability Berm Alternative

<u>Alternative 3 – Earthen Levees Using Deep Soil Mixing and Landside Shift (Plates 3.3.39 – 3.3.51)</u>

- LPV 146 Plan and Profile Sheets for Soil-Mixing Alternative
- Typical Sections for Soil-Mixing Alternative

<u>Alternative 4 – Earthen Levees Using Stability Berms with Staged Construction and Wick Drains (Plates 3.3.52 – 3.3.64)</u>

- LPV 146 Plan and Profile Sheets for Staged Construction and Wick Drains Alternative
- Typical Sections for Staged Construction and Wick Drains Alternative

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3.4 LPV 147 – Chalmette Loop Levee – Highway 46 and Bayou Road Floodgate

LPV 147 consists of the levee section at the Highway 46 Crossing and Bayou Road Floodgate. Highway 46 and Bayou Road run parallel to each other and are approximately 400 feet apart where the roads cross the hurricane protection levee. Between Highway 46 and Bayou Road on the protected side, a fire station and several side roads were observed. Multiple utilities (such as power poles, water, and sewer) were noted along the roads or leading to the fire station. Along the flood side, a marsh, heavily vegetated area, and water are present near the surface of Bayou Road, which is at the same elevation as the levee toes. Highway 46 is a four-lane divided highway with gravel shoulders, asphalt paving, and a depressed grass median.

Bayou Road is a two-lane undivided highway without shoulders. There is an existing floodgate where Bayou Road crosses the line of protection. The floodgate consists of a steel swing gate supported by adjacent concrete T-walls. At the levee tie-ins, uncapped steel sheet pile I-walls connected the T-walls to the levees. Scour protection consisting of concrete slope paving and stone slushed with grout was noted on either side of the sheet pile I-walls. On the northern protected-side toe, a short concrete T-wall and gravel road are located at the toe of the levee.

LPV 147 - Flood side of Bayou Road Floodgate

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3.4.1 Design Evaluations

For Highway 46, two alternatives were considered:

(1) Ramp; and

(2) Bridge over T-wall.

The ramp alternative would be grouped with the earthen levee alternative used for LPV 146. For the bridge alternative, the structure would pass over the T-wall option used from LPV 148. Hydraulic Reach SB16 was evaluated for Highway 46. The slope stability analysis indicates that the proposed roadway over the levee with overbuild met all required safety factors.

3.4.2 Highway 46 Ramp (Plates 3.4.1 – 3.4.3)

The shear line and the consolidation parameters used for Highway 46 is the same as used for the adjacent section (Station 1040+00) of LPV 146, as shown in Appendix E. The section required 4 feet of overbuild to account for settlement over the service life of the levee. The side slopes were modeled at ±-3.33 percent. Therefore, the fill section of the road extends approximately 1,600 to 1,800 feet from the centerline in either direction due to design requirements at this elevation of the levee. The road section on top of the overbuild levee consists of 4 inches of asphalt over 9 inches of rock base. Therefore, the top of the roadway will be located at approximately EL 34.2.

3.4.2.1 Geotechnical Results

Based on the results of the analysis, stability berms were not required because all minimum factors of safety were met. For seepage, the levee section provides enough protection against excessive exit gradients. For the earthen levee between Highway 46 and the Bayou Road Floodgate, earthen levee alternatives used for LPV 148 will be applied.

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LPV 147 - Highway 46 at centerline of levee

3.4.2.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

No as-built plans were provided by the USACE for the Highway 46 levee crossing section; therefore, the LADOTD Highway 46 project plans were used to model the existing condition. To reflect the settlement over time a 5-foot decrease in elevation from the project plans was assumed for the existing condition.

An additional 25 acres of ROW are required for the ramp at LPV 147. This additional area is located on either side of the existing levee between Bayou Road and Highway 46. Plates following this report present the construction limits and proposed ROW. All staging and stockpiling are proposed to occur within the construction limits presented in these plates. The levee requires 12 acres of borrow pit area to satisfy fill requirements.

Real Estate Description	Quantity	Unit
Additional Right-of-Way	25	Acre
Additional Borrow Pit Area	12	Acre

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After a preliminary settlement analysis of the section it was determined that, in order to get a protection design height of 29.0 feet, the roadway will need to be raised to EL 34.2. Entrance and exit grades of ±3.33 percent and embankment side slopes of 4H:1V to the clear zone are used to reach the 34.2-foot roadway height. The new vertical curve information is shown on Plate 3.4.1 and conforms to LADOTD standards. A temporary access road on each side of the ramp is proposed to maintain traffic flow during construction. The construction of the Highway 46 ramp can also be staged to maintain one lane of traffic in each direction during construction, thus negating the need for temporary roadways.

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicate the following facilities will require relocation based on improvements in LPV 147:

Facility Description	Station
Chainlink fence (760 feet)	1117+00
St. Bernard Fire Station #10, 3901 Bayou Road	1121+00
Antenna	1121+13
Home, 3840-A Bayou Road	1122+00

The fire station will be demolished and will require real estate acquisition prior to levee construction. The chainlink fence along the boundary of LPV 148 will be removed and replaced in kind following construction. The antenna associated with the fire station will be removed from the limits of construction. For the purposes of this study, it is assumed that this antenna will be demolished.

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LPV 147 - Fire Station on protected side of the Bayou Road Floodgate

The following utilities will require relocation based on improvements in LPV 147:

Utility Description	Station	Orientation	Owner
Water Pipeline	1123+05	Crossing	NA
Gas Pipeline	1123+05	Crossing	NA
Drainage Pipeline	1122+76	Crossing	NA
Drainage Pipeline	1123+56	Parallel to Design	NA
Drainage Pipeline	1123+56	Parallel to Design	NA
Overhead Power Lines	1123+29 and 1118+96	Crossing	NA

NA Not available.

The water, gas, and storm drainage pipelines are all mains providing service connections to the homes located adjacent to Bayou Road. These pipelines are located beneath Bayou Road, which will be unaffected in the proposed design. All pipes will be abandoned in place, provided that the homes to which they are providing service are demolished based on the preferred alternative in LPV 147 and LPV 148. Two drain pipes with sluice gates are located on the flood side of the levee and connect drainage canals on either side of Bayou Road. These pipes and sluice

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gates will be relocated toward the flood side, outside the levee section. Overhead electric lines follow Bayou Road and Highway 46. Each of these lines, including poles, will be relocated and raised to ensure adequate vertical clearance. The ramp alternative for LPV 147 will require relocation of the access between Highway 46 and Bayou Road.

The levee sides in the vicinity of the Bayou Road Floodgate are vegetated by mostly grasses and appear to be regularly maintained (mowed). On the protected side of the levee, there are houses in close proximity to the levee, and as such, there is minimal evidence of habitat types of any appreciable value. Much of the landscape is maintained. The area on the flood side of the levee consists of a wooded bottomland, comprised primarily of scrub-shrub species with a large amount of tree coverage, mainly sugarberry *(Celtis laevigata)*. Adjacent to the wooded bottomland is a submerged wetland, functioning as a drainage canal. This is a high-value habitat, and a portion of it will be cleared and filled if the levee is widened using an earthen levee section between Bayou Road and Highway 46.

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3.4.3 Bridge over T-wall Alternative (3.4.4 – 3.4.17)

The second alternative evaluated at LPV 147 consists of using a T-wall between LPV 146 and LPV 148 to provide flood protection and minimize environmental impacts. It is not desirable to utilize floodgates across Highway 46 due to the excessive size of the gate that would be required and the additional danger accompanying the presence of gate storage monoliths adjacent to the traveled way. Therefore, continuous T-wall is proposed across both Highway 46 and Bayou Road. This will necessitate a bridge over the T-wall along Highway 46 as well as the rerouting of local traffic from Bayou Road and Fire Station Road to Highway 46 via new connector roadways. The T-wall section for LPV 148 is also proposed for LPV 147, and the roadways and bridge are designed in accordance with LADOTD requirements.

3.4.3.1 Geotechnical Results

No slope stability analysis was performed on the abutment approaches because of the relative smaller height. For this option, the Bayou Road Floodgate will be permanently closed and replaced with either T-wall or levee as analyzed in LPV 148 Soils Reach SB16. The pile capacity curves were generated for 14X14, 24X24, and 30X30 precast concrete piles.

3.4.3.2 Structural Analysis

The bridge is designed as a precast/prestressed concrete girder span bridge with American Association of State Highway and Transportation Officials Type III girders spanning a maximum of 80 feet. A single bridge will be used to connect the protected-side divided highway to the flood-side divided highway. The distance between opposing bridge abutments is 1,640 feet to accommodate the vertical geometric requirements of a rural collector (RC-3) highway with a design speed of 60 miles per hour, while minimizing the height of embankment for the bridge approaches. Twenty-one spans are required to provide this distance. These spans are supported with pile bents, most of which are oriented perpendicular to the bridge alignment. Two pairs of skewed bents are located on either side of the proposed T-wall and skewed 27 degrees from the bridge alignment to run parallel to and accommodate the T-wall alignment. Two fixity bents are provided to divide the bridge in thirds leaving seven spans between fixity bents. The clearance below the low chord of the outside girder will vary from approximately 18 inches at the abutment to more than 16 feet at the bridge's highest point. The clearance above the T-wall is 5 feet above the stem and at least 15 feet above the adjacent grade in accordance with current USACE standards.

3.4.3.3 Civil/Right-of-Way/Relocations/Utilities/Environmental

The bridge approaches were designed to accommodate the horizontal curvature requirements for a rural collector (RC-3) highway with a design speed of 60 miles per hour. For this study, no superelevation was used to shorten the transition lengths from divided highway to undivided median barrier separated highway bridge. However, should superelevation with the necessary cross-section runoff/runout lengths prove to significantly shorten the horizontal transition distances, it should be considered for implementation in the final roadway design. Currently, the transition lengths are 1,100 feet and 1,300 feet for the north and south approaches, respectively.

The new roadways connecting Bayou Road to Highway 46 on both the flood side and protected side are designed as rural local (RL-1) roads with a design speed of 30 miles per hour. These roads will connect to Highway 46 prior to the bridge approach transition and will be accessible from either direction of Highway 46. These roads will also be utilized for construction access because Highway 46 will necessarily experience closures during the construction of the bridge. Careful construction phase planning will be paramount to providing continuous access to and from the town of Reggio and other locations outside of the flood protection system. ROW acquisitions

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will be required for these two access roads. The north and south connectors will require 1.64 acres and 0.72 acre, respectively.

3.4.4 Summary of Cost Analysis

Below is a table summarizing the costs for each of the alternatives (not including escalation) studied for this reach. The detailed cost estimate for each component can be found in Appendix H.

Description	Real Estate ($k)	Environmental Mitigation Including Contingency ($k)	Contract Cost Including Contingency ($k)	Engineering and Design Cost Including Contingency ($k)	Supervision and Administration Cost Including Contingency ($k)	Project Cost ($k)	Operation and Maintenance ($k)	Total Values ($k)
	1	2	3	4	5	1+2+3 +4+5 A	B	= A+B
LPV 147, Ramp	$2,527	-	$55,968	$6,393	$5,317	$70,205	-	$70,205
LPV 147, Bridge	-	-	$22,029	$2,202	$1,674	$26,406	$500	$26,906

For comparison purposes, the tables below provide the quantities of the major work items for each alternative presented above.

LPV 147 – Highway 46 Ramp Alternative		
Summary of Major Quantities		
	Quantity	**Unit**
Embankment Compacted Fill	671,871	Cubic Yard
Bituminous Wearing Course	2,023	Ton
Bituminous Binder Course	5,394	Ton
Bituminous Base Course	2,724	Cubic Yard
Aggregate Base Course	36,780	Cubic Yard
Clearing and Grubbing	46.9	Acre

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LPV 147 – Bridge over T-wall Alternative		
Summary of Major Quantities		
	Quantity	**Unit**
Precast Concrete Piles – 24”	19,380	Feet
Class A Concrete (Bents)	720	Cubic Yard
Class AA Concrete (Deck)	3,043	Cubic Yard
Prestressed Precast Concrete Girders (Type III)	16,400	Feet

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3.4.5 Recommended Alternative

The final recommendation for the LPV 147 reach is the bridge over T-wall alternative. This alternative has many advantages over the ramp and can be summarized as follows:

Alternative	Advantages	Disadvantages
Earthen Ramp	• All earthen construction • Can be modified more readily in the future • Reduced Inspection • Safer traffic alternative	• Significant fill quantity • Greater impact to adjacent property • Greater ROW required • Future repairs due to settlement
Bridge	• Reduced cost • Reduced ROW • Reduced utility relocation • Reasonable construction duration	• Construction sequencing • Future modifications • Reduced traffic section

Additional Design Information Required

Currently, there is no survey information for Highway 46 and limited survey information for Bayou Road. For these alternatives, the following minimum survey

information is required for preparation of final design plans, quantities, and specifications:

- Re-establish the USACE baseline;
- Develop baseline data for Highway 46 for a minimum of 2,000 feet on either side of the USACE levee baseline;
- Develop baseline data for Bayou Road for a minimum of 300 feet on either side of the wall alignment;
- Incorporate centerline and edge of roadway pavement every 50 feet from 500 feet before the beginning of the roadway raise section to 500 feet past the end of the roadway raise section;
- Develop roadway cross sections at 100-foot intervals within the limits of the proposed roadway work;
- Profile top and bottom of the existing Bayou Road Floodgate, structure corners, and grade on either side of the wall every 25 feet or at any intermediate grade points;
- Develop three cross sections perpendicular to the proposed alignment for analysis;
- Perform full topographic survey of the features such as gates, fences, utilities, power poles, ditches, borrow pits, canals, buildings, signs, appurtenances, and culverts within the proposed ROW; and
- Perform topographic survey of the proposed borrow pits.

The following geotechnical information will be required for the design of the bridge and T-wall section during the plans and specifications phase:

- Five-inch diameter undisturbed borings should be drilled to approximately 60 feet in depth for boreholes located greater than 500 feet from the centerline and to a depth of 80 feet for boreholes located less than 500 feet of the centerline. The borings should be spaced every 1,000 feet on center in both the westbound and

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eastbound lanes of Highway 46. The borings should be offset so that the profile is characterized every 500 feet;

- Deep borings will be required to ensure the stability of the T-wall foundations as well as development of the design of the bridge foundations. The proposed bridge superstructure will be supported by 24-inch-square concrete pile bents. A bridge structure of this type and length will need a minimum of two deep borings (approximately 120 feet each) in order to determine the pile capacities. The borings for the T-wall alignment should be of similar depth;

- Sampling, field procedures, and laboratory testing should follow general USACE guidelines and similar programs already conducted for LPV 145 and 146; and

- Coring of the existing pavement may be required to quantify milling of the existing asphalt pavement and excavation of base material. Pavement cores located every 500 feet in each direction is recommended.

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3.4.6 Design and Construction Schedules

The proposed schedule summarizes the time required for design and construction of the preferred alternative for LPV 147. The schedule is based on a notice to proceed date of December 18, 2008. A December 2008 notice to proceed is assumed based upon ongoing design work and allowing for negotiations. Approximately 2 months are scheduled for geotechnical testing. Final design will require approximately 10 months and will be submitted on October 23, 2009. Real estate acquisition and environmental compliance have not been completed.

Construction of the Highway 46 ramp and Bayou Road Floodgate will commence on February 17, 2010, and is slated for final acceptance on June 20, 2011. This process is described in more detail in Appendix E. The Highway 46 ramp must be raised in order to facilitate the increased levee elevation. The existing Bayou Road Floodgate will be demolished before the new gate will be constructed. Work on LPV 147 will not be completed prior to the start of the 2011 hurricane season based on the assumed production rates. Additional support, improved construction access, or revised scheduling will be required to complete this work by the mandated deadline.

3.4.7 Report Plates – LPV 147

The plans and profile sheets, typical section sheets, and preliminary bridge details are combined and presented by alternative following this report.

<u>Highway 46 Ramp Alternative (Plates 3.4.1 – 3.4.3)</u>

- LPV 147 Plan and Profile Sheets for Highway 46 Ramp Alternative
- Typical Sections of Highway 46 Ramp and Temporary Access Road

<u>Bridge over T-wall Alternative (Plates 3.4.4 – 3.4.17)</u>

- LPV 147 Plan and Profile Sheets for Roadway Approaches and Bridge
- Bridge Details

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3.5 LPV 148 – Chalmette Loop Levee – Verret to Caernarvon

LPV 148 is a levee segment located between Verret and Caernarvon Floodwall between Stations 1125+00 and 1560+25 (43,525 feet in length). The majority of the levee is vegetated with the exception of the dirt road at the protected-side toe. Drainage canals were also observed near the levee toes with light woods and brush beyond these. Several sets of utility (gas) lines were readily observed during the site visit. The pipelines are located either up and over the levee or through the levee core. One set of overhead electric lines was also noted.

Two additional features are noted along this reach: St. Mary's Pump Station (#8) and Creedmore Drainage Structure. Hurricane protection for St. Mary's Pump Station consists of a concrete T-wall that directly ties into the levees. To gain access to either side of the wall, ladders have been installed. Three pipes coming from the pump station building penetrated directly through the wall and discharge the water into a nearby canal running parallel to the levee. No analysis of the Creedmore Drainage Structure was performed because it will be demolished as requested by the sponsor due to its lack of use.

3.5.1 Design Evaluations

There are three hydraulic reaches (SB16, SB17, and SB20) in LPV 148. For Hydraulic Reaches SB16 and SB17, all four engineering alternatives were evaluated. Initially, analysis of the levee, with overbuild and scour protection, was completed for the 2010/2011 design elevation without stability berms. In all cases, the safety factor for existing conditions was significantly less than required and additional measures were necessary. The four evaluated engineering alternatives included:

(1) T-wall alternative on existing levee;

(2) Earthen levees using stability berms (Oct 07 Alternative);

(3) Earthen levees using deep soil mixing and landside shift; and

(4) Earthen levees using stability berms with staged construction and wick drains.

For the T-wall fronting protection at St. Mary's Pump Station, hydraulic condition SB20 was applied, and replacing and/or retrofitting options were considered. Station 1193+00 was used for St. Mary's Pump Station analysis.

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Geometric Sections

For Hydraulic Reaches SB16 and SB17, two representative sections were selected for the analysis. Station 1511+00 was used to represent the part from Stations 1125+00 to 1537+00, and Station 1556+00 was used to represent the part from Stations 1537+00 to 1560+25. For SB20 at St. Mary's Pump Station, one representative section was chosen for Station 1193+10.

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Station 1511+00

The survey cross section provided by the USACE extended 200 feet on either side of the levee centerline and does not include the drainage channels present on both sides of the levee. The as-built plans, construction drawings (LPV 148.01), and aerial images were used to obtain further information on the drainage features. Based on the as-built information and boring data, the existing levee top is at EL 14.0. The reason this station is more critical in comparison to other sections is because of the low levee elevation and drainage channels present on both sides of the levee. Based on the DMs, the bottom of the channel is assumed at EL -14.0.

Station 1556+00

This levee section is located close to the Caernarvon Canal. The provided cross section extends 200 feet on either side of the levee centerline and does not include the Caernarvon Canal present on the flood side of the levee. The as-built plans, construction drawings (LPV 148.01), and aerial images were used to obtain further information on the canal. Based on the as-built information and boring data, the existing levee top is at EL 13.5 and rests on a sand core extending from EL 4.0 to EL -11.0. This station is more critical in comparison to other sections in the vicinity because of the low levee elevation, lower shear strengths, and canal in close proximity on the flood side of the levee. Based on the DMs, the bottom of the channel is assumed at EL -14.0.

Station 1193+10 – St. Mary's Pump Station

Recent cross-section information was not available for this structure. The as-built plans were used to obtain additional information regarding the pump station. The ground profile was generated by using cross section and contour sheets from the as-built plans. The existing T-wall is at EL 17.0 with the base of the footing at

EL -1.0 and an unsupported height of 14 feet. The bottom of the suction basin is at EL -8.5 and the discharge basin is at EL -13.5.

Soils Reaches

Two soils reaches were developed for LPV 148 based on different soil properties indicated in soil borings. The shear lines, which can be found at the front of Appendix F, were determined based on newly completed borings and associated laboratory test results. A centerline design shear line was separated from the toe design shear line in each soils reach. A brief discussion of the shear strengths in each soils reach is presented below:

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Soils Reach E (Stations 1125+00 to 1537+00)

Similar to LPV 146, the existing levee fill materials have an undrained shear strength on the order of 600 psf. No continuous peat layer was justified based on the boring information. The soils consist primarily of clays except that a sand core exists from EL 4.0 to EL -16.0 underneath the existing levee crown. An internal friction angle of 33 degrees was assigned to the sand core to be consistent with the values used in the DMs. Other than this sand core, the shear strengths tend to increase linearly with depths. Below the existing levee fill, the shear strengths in the centerline and toes are so close to each other that only one shear line (for both center and toe) was used below EL -16. The shear strength increased from 400 psf at EL -16 to 1,068 psf at EL -80 at a rate of 10.4 psf per foot. In the toe area clays above EL -16, the shear strength increased from 200 psf at EL 4 to 400 psf at EL -16 at a rate of 10 psf per foot.

Soils Reach F (Stations 1537+00 to 1560+25)

Similar to Soils Reach E, the existing levee fill materials have an undrained shear strength on the order of 600 psf. There is a similar sand core in the centerline located between EL 1 to EL -11. In the toe area, the shear strength is approximately constant at 385 psf from EL 1 to EL -11. Based on the borings, a sand layer appears to present from EL -25 to EL -29 and a silt layer is located from EL -73 to EL -85. The sand layer has an internal friction angle of 33 degrees and the silt layer has 200 psf and 15 degrees for cohesion and internal friction angle, respectively. Those properties were consistent with the DMs. From EL -11 to EL -25, the shear strength was constant at 540 psf in the centerline while the shear strength increased from 300 to 425 psf at a rate of 8.9 psf per foot. The shear line was the same between

centerline and the toe for the clay layer located between EL -29 and EL -73, which increased from 500 to 920 psf at a rate of 9.5 psf per foot.

Because there were no consolidation tests performed in LPV 148, all the consolidation properties were assumed to be the same as those in adjacent LPV 146. The various design consolidation parameters versus depths are also presented in Appendix F.

St. Mary's Pump Station

The shear lines used for Station 1193+10 at the pump station are the same shear lines used for Station 1511+00. The shear line diagrams can be found in Appendix F.

LPV 148 - Flood side of existing T-wall at St. Mary's Pump Station

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3.5.2 Alternative 1 – T-wall Alternative on Existing Levee (Plates 3.5.1 – 3.5.12)

In general, Alternative 1 consists of a reinforced concrete T-wall supported by steel H-piles to be built on the existing levee. The existing levee would be slightly degraded in order to drive H-piles, drive steel sheet pile for seepage cut-off, and placement of a stone pad for placement of the concrete footing. The foundations were typically embedded approximately 3.0 feet into the crown for erosion protection. Scour protection, consisting of 18 inches of riprap, was assumed (for cost estimation purposes) from toe to toe of the entire cross section. Alternative 1 is designed for future (2057/2060) conditions only, because T-walls cannot be modified as easily as levees.

3.5.2.1 Geotechnical and Structural Results

For this option, the existing levee would be slightly raised in both Hydraulic Reaches SB16 and SB17 to meet the hydraulic design elevations. Unbalanced loads that result from global instability are transferred into the driven piles and are reported in the table below. A stability berm, approximately 85 feet in length and 3.5 feet in depth, was proposed on the protected side in Soils Reach F (Stations 1537+00 to 1560+25) to reduce the unbalanced force and consequently to avoid excessive pile designs. For these walls, HP 14x89 piles are used. Typical sections illustrating this option are provided at the end of this report section along with the required construction limits. The results of the geotechnical and structural analysis are summarized below:

T-wall Analysis Structural Summary

Hydraulic Reach	Soils Reach	Station Limits	Top of Wall EL	Stem Height (feet)	Stem/Base Thickness (feet)	Base Width	Pile Tip Elevation	No. of Piles in Cross Section
SB16	E	1125+00 to 1354+38.9	29.5	11.5	2.0/2.5	8.0	-95.0	2
SB17	E	1354+38.9 to 1537+00	29.5	11.5	2.0/2.5	8.0	-95.0	2
SB17	F	1537+00 to 1560+25	29.5	11.0	2.0/2.5	8.0	-90.0	2

Assumptions: 4,000 pounds per square inch compression strength; 5-foot pile center spacing along wall; and steel HP 14x89 piles on a 1H:3V batter perpendicular to wall.

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T-wall Analysis Geotechnical Summary						
Station Limits	**Hydraulic Reach**	**Soils Reach**	**Sheet Pile Tip Elevation**	**Fictitious Surface Elevation**	**Maximum Anchor Force (lbs/ft)**	**Maximum Moment (lbs-ft/ft)**
1125+00 to 1537+00	SB16 and SB17	E	-92.7	-60.6	10,753	197,770
1537+00 to 1560+25	SB17	F	-80.3	-59.7	12,425	189,420

lbs/ft Pounds per foot.
lbs-ft/ft Pounds per foot per foot.

Some benching of the existing levee may be necessary in order to drive the piles at the top of the levee due to the steep existing slopes and levee height. Because sand cores ranging from 12 to 20 feet in depths present beneath the existing levee, it is expected some driving efforts may be required to install the steel piles. Seepage will be controlled by driving sheet piles into the ground to the designed elevations as shown in the table above.

The use of additional stability berms and/or deep soil mixing may be applied here to reduce unbalanced loads, fictitious surfaces, and thus the pile lengths. However, adding additional stability berms will likely require additional ROW and local stability issues as seen in other alternatives within this reach. Deep soil mixing will also add to the cost but may reduce pile loads and lengths. These options could be considered in the future should the T-wall need to be raised with minimal impact to the foundations.

3.5.2.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following utilities/facilities will require relocation based on LPV 148 Alternative 1 – T-wall:

LPV 148 Alternative 1 – T-wall		
Right-of-Way		
Description	**Quantity**	**Units**
Additional Right-of-Way required	0.54	Acre

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LPV 148 Alternative 1 – T-wall			
Utility Relocation			
Description		**Quantity**	**Units**
1290+93	Tennessee Gas Pipeline 20"	762	Foot
1291+25	Targa Gas Pipeline 6"	762	Foot
1291+27	Enterprise/Promix Pipeline 6"	762	Foot
1296+19	Southern Natural Gas Pipeline 20”	761	Foot
1296+39	Southern Natural Gas Pipeline 20”	761	Foot
1296+60	Southern Natural Gas Pipeline 20”	761	Foot
1331+26	Enbridge Pipeline Co. Pipeline 16”	1000	Foot
1331+81	Enbridge Pipeline Co. Pipeline 12”	953	Foot
1426+21-1427+52	Entergy Overhead Powerline	760	Foot
	Entergy Overhead Powerline	760	Foot
	Entergy Overhead Powerline	760	Foot
1424+69	British Petroleum Pipeline 24"	760	Foot
1424+69	Gulf South Pipeline Co. Pipeline 20"	760	Foot
1445+00	Gulf South Pipeline Co. Pipeline 16”	760	Foot
1446+00	Gulf South Pipeline Co. Pipeline 16”	760	Foot
Facility Relocation			
Description		**Quantity**	**Units**
1158+57	Existing fence	328	Foot
1196+30	St. Mary's #8 Pump Station - DO NOT DISTURB	1	Each

All pipelines described above will be raised via a pipe bridge to a bottom of pipe elevation of 31 feet with a pipe bridge clear span of 310 feet. Although pipe bridges required by the soil-mixing alternative are identical to those in the stability berm alternative, the length of underground pipe shielding is only 410 feet for each pipe because of the decreased construction limits. The Creedmore Drainage Structure will be demolished per direction from the government. Three electric transmission poles (two H-frame style and one single pole) shall be raised an additional 12 feet on either side of the levee section to ensure adequate vertical clearance.

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The levee section along LPV 148 is vegetated with grasses and white clover (*Trifolium*). Introduction of the clover may be reflective of the current use of the levee for livestock grazing. Both the inland and outer fringes of the levee are populated with healthy stands of sugarberry *(Celtis laevigata),* and other trees, combined with a scrub-shrub understory. Adjacent to the levee on the protected side is an expansive marsh. It appears to be a salt marsh by the preponderance of cordgrass (*Spartina*) and other salinity-tolerant species. Construction of the T-wall alternative will be a temporary disruption to the wetland system and will not extend outside the limits of the drainage canal. No filling of wetland or clearing of vegetation outside the existing levee landform is required for this alternative. For these reasons, the T-wall alternative has significantly lesser impacts on the valuable surrounding wetland habitats.

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3.5.3 Alternative 2 – Earthen Levees Using Stability Berms (Plates 3.5.13 – 3.5.34)

Alternative 2, earthen levees using stability berms, consists of constructing additional earthen levee lifts. The proposed levee centerline will be shifted from the existing location toward landside on the order of 100 to 150 feet. This levee shift will prevent filling of the Creedmore Drainage Structure on the flood side. The Jourda Canal on the protected side, however, will be filled in and offset. Scour protection, consisting of 18 inches of riprap, was assumed (for cost estimation purposes) from toe to toe of the entire cross section. The initial section, taken from the Hydraulic Report (USACE), was altered to meet the required geotechnical factors of safety.

3.5.3.1 Geotechnical Results

Similar to LPV 145 and LPV 146, complete analysis was conducted for the existing condition (2010/2011 design elevations) while only limited slope stability analysis was performed for future conditions (2057/2060 design elevations). Additional lifts were proposed to reach future conditions design elevations by 2057/2060 based on time-rate settlement analysis. The scheduling of the future lift(s) is provided with the typical sections attached to this report. Due to settlement, a certain amount of overbuild has to be included for the proposed levee. The information is summarized in the tables below.

Summary of Overbuild for 2010/2011 Design Elevations						
Station Limits	**Hydraulic Reach**	**Soils Reach**	**2010/2011 Levee Design Elevation**	**Proposed Top of Levee Elevation**	**Proposed Top of Riprap Elevation**	**Overbuild Thickness (feet)**
1125+00 to 1354+38.9	SB16	E	26.5	31.0	32.5	4.5
1354+38.9 to 1537+00	SB17	E	26.5	31.0	32.5	4.5
1537+00 to 1560+25	SB17	F	26.5	31.5	33.0	5.0

Summary of Overbuild for 2057/2060 Design Elevations						
Station Limits	**Hydraulic Reach**	**Soils Reach**	**2057/2060 Levee Design Elevation**	**Proposed Top of Levee Elevation**	**Proposed Top of Riprap Elevation**	**Overbuild Thickness (feet)**
1125+00 to 1354+38.9	SB16	E	29.0	32.0	33.5	3.0
1354+38.9 to 1537+00	SB17	E	29.0	32.0	33.5	3.0
1537+00 to 1560+25	SB17	F	29.0	30.5	32.0	1.5

The levee has a crown approximately 10 feet wide and slopes down at a 5H:1V until intersecting the stability berms on the protected side. The flood side has a proposed wave berm as proposed by hydraulic engineers at the USACE, but it may be altered in order to meet the required factor of safety for the flood-side stability. Typical sections illustrating this option are provided at the end of this report section along with the required construction limits. Based on the geotechnical analysis results, the approximate berm dimensions on both protected and flood sides are summarized below:

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Summary of the Stability Berm Dimensions (2010/2011 design elevations)					
Station Limits	**Hydraulic Reach**	**Soils Reach**	**Distance of Toe from Levee Centerline (PS/FS) (feet)**	**Length (PS/FS) (feet)**	**Average Depth (PS/FS) (feet)**
1125+00 to 1537+00	SB16 and SB17	E	329/294	268/247	18/10
1537+00 to 1560+25	SB17	F	323/270	277/223	14/13

PS Protected Side.
FS Flood Side.

Summary of the Stability Berm Dimensions (2057/2060 design elevations)					
Station Limits	**Hydraulic Reach**	**Soils Reach**	**Distance of Toe from Levee Centerline (PS/FS) (feet)**	**Length (PS/FS) (feet)**	**Average Depth (PS/FS) (feet)**
1125+00 to 1537+00	SB16 and SB17	E	359/338	298/290	19/12
1537+00 to 1560+25	SB17	F	323/270	277/223	14/13

PS Protected Side.
FS Flood Side.

The ROW varies along the LPV 148 alignment and is typically on the order of 200 feet from centerline on either side of the levee. Due to the significant landside shift of levee, it is likely that the proposed berms will stay within the ROW or very limited additional ROW will be needed on the flood side while additional ROW on the order of 300 to 350 feet may be required on the protected side. The existing Jourda Canal will likely be filled to accommodate the proposed stability berm on the protected side. As a result, it was assumed in the analysis that the Jourda Canal will be shifted more than 30 feet away from the protected-side berm toe.

Compacted clay fill was used for both the raised levee and stability berms because of the increased strength, more stringent testing requirements, and heavier unit weight. However, compacted sand fill was assumed to fill the existing Jourda Canal on the protected side wherever such filling becomes necessary.

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Seepage analysis was only performed for 2010/2011 design elevations. The berm on the protected side was primarily proposed for stability purposes. At the same time, stability berms also act as seepage berms by providing additional weight to counteract those upward seepage forces. The stability berms between Stations 1125+00 and 1537+00 provide enough seepage control, and the maximum exit gradient was less than 0.5 at the berm toe. However, between Stations 1537+00 and 1560+25, the size of the stability berms is not adequate to control the critical exit gradient to less than 0.5 without any additional measures. As a result, sheet pile cut-offs were proposed between Stations 1537+00 and 1560+25. The sheet pile was placed near the protected-side berm toe and the tip of the sheet pile was estimated at EL -34. As stated earlier, sheet pile cut-off is only one alternative. Slurry trenches and relief wells are additional considerations to propose as seepage cut-off options during final design. Sheet pile was used for cost purposes only.

3.5.3.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following utilities/facilities will require relocation based on LPV 148 Oct 07 – 2010 Elevation alternative:

LPV 148 Oct 07 Alternative 2 - 2010/2011			
Right-of-Way			
Description		**Quantity**	**Units**
Additional Right-of-Way required - 2010/2011		276.23	Acre
Utility Relocation			
Description		**Quantity**	**Units**
1290+93	Tennessee Gas Pipeline 20"	986	Foot
1291+25	Targa Gas Pipeline 6"	983	Foot
1291+27	Enterprise/Promix Pipeline 6"	983	Foot
1296+19	Southern Natural Gas Pipeline 20”	1,016	Foot
1296+39	Southern Natural Gas Pipeline 20”	1,015	Foot
1296+60	Southern Natural Gas Pipeline 20”	1,011	Foot
1331+26	Enbridge Pipeline Co. Pipeline 16”	1,142	Foot
1331+81	Enbridge Pipeline Co. Pipeline 12”	1,121	Foot
1426+21-1427+52	Entergy Overhead Powerline	1,179	Foot

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LPV 148 Oct 07 Alternative 2 - 2010/2011			
	Entergy Overhead Powerline	1,179	Foot
	Entergy Overhead Powerline	1,179	Foot
1424+69	B.P. Oil Pipeline 24"	1,000	Foot
1424+69	Gulf South Pipeline Co. Pipeline 20"	1,000	Foot
1445+00	Gulf South Pipeline Co. Pipeline 16”	940	Foot
1446+00	Gulf South Pipeline Co. Pipeline 16"	940	Foot
Facility Relocation			
Description		**Quantity**	**Units**
1124+00-1130+00	Houses to be relocated	unknown	Each
1158+57	Existing fence	328	Foot
1196+30	St. Mary's #8 Pump Station - DO NOT DISTURB	1	Each

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following utilities/facilities will require relocation based on LPV 148 Oct 07 2057/2060 Elevation, alternative:

LPV 148 Oct 07 Alternative 2 - 2057/2060			
Right-of-Way			
Description		**Quantity**	**Units**
Additional Right-of-Way required - 2057/2060		304.07	Acre
Utility Relocation			
Description		**Quantity**	**Units**
1290+93	Tennessee Gas Pipeline 20"	986	Foot
1291+25	Targa Gas Pipeline 6"	983	Foot
1291+27	Enterprise/Promix Pipeline 6"	983	Foot
1296+19	Southern Natural Gas Pipeline 20”	1,016	Foot
1296+39	Southern Natural Gas Pipeline 20”	1,015	Foot
1296+60	Southern Natural Gas Pipeline 20”	1,011	Foot
1331+26	Enbridge Pipeline Co. Pipeline 16”	1,142	Foot

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LPV 148 Oct 07 Alternative 2 - 2057/2060			
1331+81	Enbridge Pipeline Co. Pipeline 12”	1,121	Foot
1426+21-1427+52	Entergy Overhead Powerline	1,179	Foot
	Entergy Overhead Powerline	1,179	Foot
	Entergy Overhead Powerline	1,179	Foot
1424+69	B.P. Oil Pipeline 24"	1,000	Foot
1424+69	Gulf South Pipeline Co. Pipeline 20"	1,000	Foot
1445+00	Gulf South Pipeline Co. Pipeline 16”	940	Foot
1446+00	Gulf South Pipeline Co. Pipeline 16"	940	Foot
Facility Relocation			
Description		**Quantity**	**Units**
1124+00-1130+00	Houses to be relocated	unknown	Each
1158+57	Existing fence	328	Foot
1196+30	St. Mary's #8 Pump Station - DO NOT DISTURB	1	Each

All pipelines described above will be raised via a pipe bridge to a bottom of pipe elevation of 31 feet with a pipe bridge clear span of 310 feet. Each pipe will be shielded underground for an additional 1,095 feet to provide protection during levee construction. The Creedmore Drainage Structure will be demolished per direction from the government. Three electric transmission poles (two H-frame style and one single pole) shall be raised an additional 12 feet on either side of the levee section to ensure adequate vertical clearance.

The levee section along LPV 148 is vegetated with grasses and white clover (*Trifolium*). Introduction of clover may be reflective of the current use of the levee for livestock grazing. Both the inland and outer fringes of the levee are populated with healthy stands of sugarberry *(Celtis laevigata),* and other trees, combined with a scrub-shrub understory. Adjacent to the levee on the protected side is an expansive marsh. It appears to be a salt marsh based upon the preponderance of cordgrass (*Spartina*) and other salinity-tolerant species. The stability berm alternative will extend approximately 740 feet from the levee centerline toward the protected side and 645 feet toward the flood side, completely filling the protected-side Jourda Canal

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and the flood-side Creedmore Drainage Structure and Caernarvon Canal. In addition to filling these canals, which act as drainage structures and valuable habitat, the stability berm alternative will increase the construction limits by 950 acres, which will eliminate the aforementioned tree stands and scrub-shrub understory and fill in valuable marsh habitat.

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3.5.4 Alternative 3 – Earthen Levees Using Deep Soil Mixing and Landside Shift (Plates 3.5.35 – 3.5.46)

Alternative 3 is the earthen levee using deep soil mixing, which is constructed by first installing deep soil-mixed panels on the protected side of the existing levee toe and then placing earthen embankment on top of this improved foundation soil. The centerline of the proposed levee is offset of the existing levee and positioned on the deep soil-mixed panels. Scour protection, consisting of 18 inches of riprap, was assumed (for cost estimation purposes) from toe to toe of the entire cross section.

3.5.4.1 Geotechnical Results

Because of the nature of this alternative, the foundation of the deep soil mixing must be prepared to handle future (2057/2060) design elevations and was the reason for investigating this alternative up to this level; however, interim or lower height lifts for the existing condition could be investigated after the foundation is prepared for future conditions. Overbuild was also included to accommodate settlement over time. Design information used to model this alternative is summarized in the table below.

Summary of Overbuild for 2057/2060 Design Elevations

Station Limits	Hydraulic Reach	Soils Reach	2057/2060 Levee Design Elevation	Proposed Top of Levee Elevation	Proposed Top of Riprap Elevation	Overbuild Thickness (feet)
1125+00 to 1354+38.9	SB16	E	29.0	32.0	33.5	3.0
1354+38.9 to 1537+00	SB17	E	29.0	32.0	33.5	3.0
1537+00 to 1560+25	SB17	F	29.0	34.0	35.5	5.0

The levee has a crown approximately 10 feet wide and slopes down at a 4H:1V until intersecting the existing ground surface on the protected side. The flood side has a

proposed wave berm as instructed by the USACE. The existing levee crown may need to be partially excavated to accommodate the proposed wave berm on the flood side. The centerline of the proposed levee was shifted toward landside from the existing levee centerline with the deep soil-mixing option in order to construct the columns/panels. The location of the deep soil mixing was placed between the existing levee protected-side toe to the proposed new levee protected-side toe. Based on the results of the analysis, the approximate soil-mixing regions are as follows:

Soil-Mixing Regions (Protected Side)					
Station Limits	**Hydraulic Reach**	**Soils Reach**	**Width (feet)**	**Average Depth (feet)**	**Bottom Elevation of DSM Region**
1125+00 to 1537+00	SB16 and SB17	E	221	60	-57
1537+00 to 1560+25	SB17	F	207	32	-29

Soil-Mixing Region (Flood Side)					
Station Limits	**Hydraulic Reach**	**Soils Reach**	**Width (feet)**	**Average Depth (feet)**	**Bottom Elevation of DSM Region**
1537+00 to 1560+25	SB17	F	56	15	-11

DSM Deep Soil Mixing.

The lengths and depths were altered until all design cases were met for the most appropriate section. Due to the improved shear strength of the soil mass, failure surfaces were forced deeper or longer to reach weaker material. A maximum shear strength of 2,300 psf was used for the improved soil mass; however, the safety factor for the shallower layers likely could be designed with less strength depending on the internal design and numerical analysis required for future design.

Typical sections illustrating this option are provided at the end of this report section along with the required construction limits.

Seepage analysis indicated that the soil-mixing region has provided enough “cut-off” effect for Soils Reach E and that an acceptable exit gradient can be achieved without

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additional seepage control measures. However, for Soils Reach F, the soil-mixing block did not provide enough protection against excessive exit gradients, and the tip of the sheet pile cut-off wall, required at the protected-side toe, is anticipated to be located at EL -34.

3.5.4.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following utilities/facilities will require relocation based on LPV 148 Alternative 3 - Soil Mixing:

LPV 148 Alternative 3 - Soil Mixing			
Right-of-Way			
Description		**Quantity**	**Units**
Additional Right-of-Way required		71.85	Acre
Utility Relocation			
Description		**Quantity**	**Units**
1290+93	Tennessee Gas Pipeline 20"	837	Foot
1291+25	Targa Gas Pipeline 6"	837	Foot
1291+27	Enterprise/Promix Pipeline 6"	837	Foot
1296+19	Southern Natural Gas Pipeline 20”	837	Foot
1296+39	Southern Natural Gas Pipeline 20”	837	Foot
1296+60	Southern Natural Gas Pipeline 20”	837	Foot
1331+26	Enbridge Pipeline Co. Pipeline 16”	1,096	Foot
1331+81	Enbridge Pipeline Co. Pipeline 12”	1,049	Foot
1426+21 - 1427+52	Entergy Overhead Powerline	831	Foot
	Entergy Overhead Powerline	831	Foot
	Entergy Overhead Powerline	831	Foot
1424+69	B.P. Oil Pipeline 24"	831	Foot
1424+69	Gulf South Pipeline Co. Pipeline 20”	831	Foot
1445+00	Gulf South Pipeline Co. Pipeline 16”	831	Foot
1446+00	Gulf South Pipeline Co. Pipeline 16"	831	Foot

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LPV 148 Alternative 3 - Soil Mixing			
Facility Relocation			
Description		**Quantity**	**Units**
1158+57	Existing fence	331	Foot
1196+30	St. Mary's #8 Pump Station - DO NOT DISTURB	1	Each

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All pipelines described above will be raised via a pipe bridge to a bottom of pipe elevation of 31 feet with a pipe bridge clear span of 310 feet. Although pipe bridges required by the soil-mixing alternative are identical to those in the stability berm alternative, the length of underground pipe shielding is only 410 feet for each pipe because of the decreased construction limits. The Creedmore Drainage Structure will be demolished per direction from the government. Three electric transmission poles (two H-frame style and one single pole) shall be raised an additional 12 feet on either side of the levee section to ensure adequate vertical clearance.

As stated in the discussion of the stability berm alternative, the levee section is vegetated with grasses and white clover, while the inland and outer fringes of the levee are valuable habitat with healthy tree cover, scrub-shrub understory, and expansive salt marsh. The soil-mixing alternative requires drilling along the existing levee footprint as well as in the drainage canals; however, this will be a temporary disruption to the wetland system and will not extend outside the limits of the drainage canal. No filling of wetland or clearing of vegetation outside the existing levee landform is required for the soil-mixing alternative. For these reasons, the soil-mixing alternative has significantly lesser impacts on the valuable surrounding wetland habitats.

3.5.5 Alternative 4 – Earthen Levees Using Stability Berms with Staged Construction and Wick Drains (Plates 3.5.47 – 3.5.58)

Alternative 4 is an earthen levee using staged construction and wick drains. This alternative is constructed by first placing a sand blanket on top of the existing levee and installing wick drains. An earthen levee is placed to the highest elevation possible for the existing soils on top of the sand blanket, and a steel sheet pile or other seepage cut-off is installed down the center to prevent water from flowing through the levee. A time frame of approximately 1 year is required prior to the placement of the second lift of earthen embankment and 18 inches of riprap scour

protection. The time frame is to allow the water to dissipate from the foundation soils so that the shear strength of the soils increases thus allowing higher levees and less footprint required.

3.5.5.1 Geotechnical Results

Because of the nature of this alternative, the foundation of the soil must be prepared to handle future (2057/2060) design elevations and was the reason for investigating this alternative up to this level; however, interim or lower height lifts for the existing condition could be investigated after the foundation is prepared for future conditions. Overbuild was also included to accommodate settlement over time.

The staged construction is anticipated to be constructed in a total of two lifts. The first lift will be raised to EL 27.5 and EL 26.5 for Soils Reaches E and F, respectively. The first lift will be allowed to settle for 1 year. At the end of 1 year, the strength gain for each soil layer due to surcharge is calculated based on the degree of consolidation in that soil layer, considering the effect of wick drains. The second lift will be raised to the future design elevation, which is summarized in the table below.

Summary of Overbuild for 2057/2060 Design Elevations

Station Limits	Hydraulic Reach	Soils Reach	2057/2060 Levee Design Elevation	Proposed Top of Levee Elevation	Proposed Top of Riprap Elevation	Overbuild Thickness (feet)
1125+00 to 1354+38.9	SB16	E	29.0	32.5	34.0	3.5
1354+38.9 to 1537+00	SB17	E	29.0	32.5	34.0	3.5
1537+00 to 1560+25	SB17	F	29.0	32.0	33.5	3.0

The levee has a crown approximately 10 feet wide and slopes down at a 4H:1V until intersecting the stability berms. The centerline of the proposed levee coincides with the existing levee centerline for this option. Compacted clay fill was used for both the levee and stability berms because of the increased strength, more stringent testing requirements, and heavier unit weight. Based on the results of the analysis, stability berms on both the protected and flood sides have the approximate dimensions shown in the table below:

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Summary of the Stability Berm Dimensions (2057/2060 design elevations)					
Station Limits	**Hydraulic Reach**	**Soils Reach**	**Distance of Toe from Proposed Levee Centerline (PS/FS) (feet)**	**Length (PS/FS) (feet)**	**Average Depth (PS/FS) (feet)**
1125+00 to 1537+00	SB16 and SB17	E	380/311	238/196	10/10
1537+00 to 1560+25	SB17	F	266/224	215/107	7/9

PS Protected Side.
FS Flood Side.

Typical sections illustrating this option are provided at the end of this report along with the required construction limits. Based on the table above, it appears that the majority of the berms on the flood side will fit within the ROW. Failure surfaces tended to exit near where the berms ended due to significant decrease in overburden.

For seepage, the stability berms do not provide enough protection against excessive exit gradients, and a sheet pile cut-off wall is required. In order to reduce the exit gradient to within allowable limits, the tip of the sheet pile wall is anticipated to be located at EL -40 and EL -34 for Soils Reaches E and F, respectively.

3.5.5.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following utilities/facilities will require relocation based on LPV 148 Alternative 4 – Stability Berms with Staged Construction and Wick Drains:

LPV 148 Alternative 4 – Stability Berms with Staged Construction and Wick Drains			
Right-of-Way			
Description		**Quantity**	**Units**
Additional Right-of-Way required		303.42	Acre
Utility Relocation			
Description		**Quantity**	**Units**
1290+23	Tennessee Gas Pipeline 20"	961	Foot
1291+25	Targa Gas Pipeline 6"	961	Foot

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LPV 148 Alternative 4 – Stability Berms with Staged Construction and Wick Drains			
1291+27	Enterprise/Promix Pipeline 6"	961	Foot
1296+19	Southern Natural Gas Pipeline 20”	961	Foot
1296+39	Southern Natural Gas Pipeline 20”	961	Foot
1296+60	Southern Natural Gas Pipeline 20”	961	Foot
1331+26	Enbridge Pipeline Co. Pipeline 16”	1,142	Foot
1331+81	Enbridge Pipeline Co. Pipeline 12”	1,121	Foot
1426+21 - 1427+52	Entergy Overhead Powerline	955	Foot
	Entergy Overhead Powerline	955	Foot
	Entergy Overhead Powerline	955	Foot
1424+69	B.P. Oil Pipeline 24"	955	Foot
1424+69	Gulf South Pipeline Co. Pipeline 20"	955	Foot
1445+00	Gulf South Pipeline Co. Pipeline 16”	940	Foot
1446+00	Gulf South Pipeline Co. Pipeline 16"	940	Foot
Facility Relocation			
Description		**Quantity**	**Units**
1124+00 - 1130+00	Houses to be relocated	unknown	Each
1158+57	Existing fence	328	Foot
1196+30	St. Mary's #8 Pump Station - DO NOT DISTURB	1	Each

3.5.6 St. Mary’s Pump Station – T-wall

3.5.6.1 Geotechnical and Structural Analysis

This analysis for conceptual design is in accordance with the structural design criteria presented in the EAR-RHCA and as described herein. The conceptual design at St. Mary's Pump Station (Pumping Station #8) consists of the removal of the existing T-walls at and near the discharge pipes and the existing I-walls that connect them to the levee. The existing T-wall monoliths are to be removed, except for the existing piles. The existing monolith piles are to be cut off 2 feet below excavation and are to be abandoned in place. In addition, two existing vertical timber piles, 65 feet in length, will be required to be removed to allow for the installation of new piles. These timber

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piles support the stairs for the existing T-wall catwalk. Provisions to accommodate the existing catwalk with the new T-walls are to be determined in the plans and specification phase.

New piles will be located and driven between the existing piles for the proposed T-wall monoliths that will replace the existing T-walls and I-walls. The proposed T-walls will tie directly into the line of protection. New I-walls were not to be considered for this study. Stability and wave berms are not used in this analysis because the existing ground is generally flat at the T-wall location.

The assumptions for conceptual design consist of the following:

- The design top of wall is at EL 30.5, NAVD-88, as provided for this study;
- The bottom of footing is taken to be at EL -1.0, NAVD-88, to match the plans of the existing T-wall;
- The wave force at St. Mary's Pump Station is as provided for this study;
- The boat impact load is taken to be the minimum 500 pounds per foot due to its location;
- A special analysis for consideration of the wall design due to the effects of the discharge pipes though the wall will be performed in the final design;
- Deflection criteria for the T-wall at the discharge pipes are the same as the standard T-wall design criteria provided for this study. That is, a reduction in deflection criteria was not considered based on pipe deflection tolerance, which was not provided for this study;
- Where T-walls tie into the levee, soil on each side of the T-wall aids in stabilizing the T-wall by reducing the overturning effect on the loaded structure. It is conservative to neglect this stabilizing effect and it will therefore be neglected; and
- The existing construction plans, labeled Caernarvon Pumping Station (Pumping Station No. 8), dated 5/6/97 as Record Drawings, are valid, except that it is assumed that the centerline distances between the pipes remain the same distance of 20.5 feet on each side of the wall. (Sheet No. S-2 of the plans

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incorrectly shows the distance between centerlines as 19.5 feet on the protected side of the T-wall.)

The slope stability results indicated that there were unbalanced loads or a fictitious surface. The fictitious surface is noted at EL -31.4 and -4.3 for high water level (HWL) and still water level (SWL). Anchor forces were 4,474 and 4,353 pounds per foot for HWL and SWL, respectively. Moments were -114,880 foot-kips and -52,190 foot-kips for HWL and SWL, respectively. Sheet pile tips are calculated to be EL -87.1 and -52.7 for HWL and SWL, respectively.

Calculations to analyze the proposed T-wall monoliths, excluding the pile foundations, were performed using MathCAD in lieu of hand calculations. The MathCAD calculations were used to determine loads to the structure, including the required input loads for the Computer Pile Group Analysis (CPGA) program, and to determine the structural adequacy of the T-wall stem and footing. From the CPGA output, pile loads, in the global vertical direction were input back into the MathCAD calculations for a starting point in the footing analysis and to complete the analysis of a single monolith.

The pile foundations for the proposed T-wall monoliths were analyzed as a rigid foundation using the USACE library program CPGA, 3d pile group analysis. CPGA was used to determine the structural adequacy of the piles, pile loads, and deflections. Soil capacity was neglected in the CPGA analysis. The resulting pile loads from CPGA were used with geotechnical pile capacity charts that were created in this study to determine the required pile tip elevations.

Two sets of MathCAD/CPGA files were developed for analyzing the T-wall monoliths at St. Mary's Pump Station. The first set was used to analyze the T-wall monolith at the discharge pipes, which is 65 feet in length. The second set was used to analyze the typical T-wall monolith, which is 45.5 feet in length.

All proposed T-wall monoliths have a cross section that includes a T-wall stem 3.5 feet thick and 27.0 feet tall with the base footing 4.5 feet thick and 14.5 feet wide.

Steel H-piles are used for conceptual design due to high tension forces. A three-pile configuration is necessary to satisfy load and deflection criteria. A batter of 1H:3V was required to meet deflection criteria. The pile type HP 14x89 was investigated for this study, and it was shown to be structurally adequate by the CPGA program.

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Based on the pile loadings, one-pile tip elevation result was found in this study for the T-walls at the pumping station. The T-walls require a minimum pile tip elevation of -138 feet for a total length of 147 feet along a 1H:3V batter.

It is noteworthy to mention that, for construction of the proposed T-walls, any lack of accuracy of the placement of the existing piles in the field, according to the existing plans, and/or any lack of accuracy of driving new piles may have undesirable consequences. There are numerous existing piles for T-wall, discharge pipe bents, and stair/platform supports in the area of the T-wall at the existing discharge pipes. Some proposed locations for driving new piles only allow inches for accuracy.

In order to construct the proposed T-wall, there will be existing structural elements requiring temporary removal, such as the discharge pipes, maintenance stairs, and platforms.

Finally, the deflection tolerance of the discharge pipes should be investigated in the final design to ensure that no structural conflict exists between the T-wall and pipe allowable deflection limits.

3.5.6.2 Civil/Right-of-Way/Relocations/Utilities/Environmental

The St. Mary's Pump Station T-wall will not require additional real estate because its footprint is within the existing ROW. Similarly, there will be no additional borrow requirements associated with this section.

St. Mary's Pump Station requires an overhead electric transmission line that follows the levee alignment toward Highway 46. Relocation of this electric line and associated poles is not required by the T-wall construction in the vicinity of St. Mary's Pump Station.

In the vicinity of St. Mary's Pump Station, the levee is well maintained (mowed) and frequently grazed by livestock. The brackish marshes surrounding the levee are separated from the levee by a buffer of sugarberry *(Celtis laevigata)* and other species that create a small upland fringe. This is reflective of the elevation changes between the levee base and the surrounding landscape. The T-wall associated with St. Mary's Pump Station will remain within the existing limits of construction and therefore will only affect the mowed levee area, which does not represent a significant environmental impact.

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Although there is minimal environmental impact associated with the T-wall in the vicinity of St. Mary's Pump Station, it is important to note that filling the drainage canals on either side of the levee will render the pump station ineffective unless new canals are dredged. This represents an important environmental impact with regard to hydraulics and hydrology for Alternative 1.

3.5.7 Summary of Cost Analysis

Below is a table summarizing the cost of each of the alternatives (not including escalation) studied for this reach. The fronting protection for St. Mary's Pump Station is included in each estimate. The detailed cost estimate for each alternative can be found in Appendix H.

Description	Real Estate ($k)	Environmental Mitigation Including Contingency ($k)	Contract Cost Including Contingency ($k)	Engineering and Design Cost Including Contingency ($k)	Supervision and Administration Cost Including Contingency ($k)	Project Cost ($k)	Operation and Maintenance ($k)	Total Values ($k)
	1	2	3	4	5	1+2+3+4+5 A	B	= A+B
LPV 148, T-wall	$0.65	$53	$643,685	$80,461	$61,150	$785,349	$11,403	$796,752
LPV 148, Deep Soil Mixing	$327	$7,297	$1,060,760	$132,595	$100,772	$1,301,752	$69,071	$1,370,823
LPV 148, Staged with Wick Drain All Stages	$622	$31,462	$1,025,171	$128,146	$97,391	$1,282,793	$75,915	$1,358,708
LPV 148, Oct 07 All Stages	$594	$29,148	$1,132,529	$141,566	$107,590	$1,411,426	$106,976	$1,518,403

For comparison purposes, the tables below provide the quantities of the major work items for each alternative presented above.

LPV 148 – Alternative 1		
Summary of Major Quantities		
	Quantity	**Unit**
Riprap	255,163	Cubic Yard
T-wall Concrete	68,839	Cubic Yard

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LPV 148 – Alternative 1		
Summary of Major Quantities		
	Quantity	**Unit**
T-wall Sheet Pile	520,830	Square Yard
T-wall Pile	2,025,631	Foot

LPV 148 – Alternative 2		
Summary of Major Quantities		
	Quantity	**Unit**
Embankment Compacted Fill	19,238,590	Cubic Yard
Riprap	875,723	Cubic Yard
Clearing and Grubbing	2157.8	Acre

LPV 148 – Alternative 3		
Summary of Major Quantities		
	Quantity	**Unit**
Embankment Compacted Fill	6,180,939	Cubic Yard
Riprap	430,733	Cubic Yard
Soil Mixing*	20,458,926	Cubic Yard
Clearing and Grubbing	287.1	Acre

*Quantity for soil mixing is for the composite region. A 40 percent replacement ratio was applied to determine the estimated quantity of soil-mixed columns and/or panels.

LPV 148 – Alternative 4		
Summary of Major Quantities		
	Quantity	**Unit**
Embankment Compacted Fill	10,231,781	Cubic Yard
Riprap	242,726	Cubic Yard
Clearing and Grubbing	1087.3	Acre
Sheet Pile	346,949	Square Yard
Wick Drains	32,470,656	Foot

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St. Mary's Pump Station		
Summary of Major Quantities		
	Quantity	**Unit**
T-wall Concrete	1,461	Cubic Yard
T-wall Sheet Pile	2,384	Square Yard
T-wall Pile	23,523	Foot
Removal of Existing Monoliths	240	Foot
Remove and Replace Existing Drainage Pipes	3	Each

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3.5.8 Recommended Alternative

For this reach, the recommended alternative is Alternative 1 – T-wall Structure. This recommendation is based on the limited environmental impacts, reduced need for borrow, and no requirements for additional temporary easements or ROW. The proposed construction would not impact either the Caernarvon or Jourda canals. The costs of the T-wall are significantly less than the levee Alternatives 2 through 4.

For Alternatives 2 and 4, significant ROW will also be required. The cost difference is largely due to the extensive fill. Also, Alternative 2 requires the Jourda Canal to be relocated for the majority of the reach. The availability and need for borrow are unknown at this time, which could also adversely impact the construction costs and schedule. Alternative 3 has similar advantages to Alternative 1 in comparison to Alternatives 2 and 4, and it also provides the advantages of a levee. However, the cost appears to significantly outweigh these advantages.

Advantages and disadvantages for all the alternatives considered are as follows:

Alternative	Advantages	Disadvantages
T-wall on Existing Levee	• Construction with ROW • No stability berms required • Materials more readily available • Reduced environmental impacts • No additional interim measures required • Built to future elevation • Reduced life cycle cost • Additional levee lifts not required	• Hard to modify in future and major reconstruction if altered • Political/public perception of walls after Hurricane Katrina • Poor aesthetics • Routine inspections
Earthen Levee with Stability Berms – Oct 07	• Can modify for future lifts • No interim measures required • Greater vehicular access • Aesthetically pleasing • More eco-friendly	• Source of borrow • Additional scour protection required (heavier than current design) • Additional ROW may be required • Significant cost up to future elevation • Fill in Jorda Canal

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Alternative	Advantages	Disadvantages
Earthen Levee with Deep Soil Mixing	• Can reduce lift heights to existing design elevation • Improved subgrade/ foundation up to future design elevation • No interim measures required • Greater vehicular access • Aesthetically pleasing • More eco-friendly • No to very little ROW • No canal relocations • Less borrow required	• Landside shift • Difficult to excavate foundation in the future • Wet mixing produces excess spoil • Cost may be more significant • Additional scour protection required (heavier than current design) • Specialty contractors
Earthen Levee using Staged Construction & Wick Drains	• Reduced protection in 1st year • Can modify for future lifts • No interim measures required • Greater vehicular access • Aesthetically pleasing • More eco-friendly • Improved foundation strength with less time	• Source of borrow • Additional scour protection required (heavier than current design) • Additional ROW may be required • Seepage cut-off required • Installation of wick drains • Fill in Jorda Canal

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The two other flood protection structures evaluated in this reach were St. Mary's Pump Station and the Creedmore Drainage Structure. The pump station fronting walls will be replaced, and the Creedmore Drainage Structure will be demolished because it is no longer necessary. The T-walls at St. Mary's Pump Station were not feasible to retrofit because of the inadequacy of the existing foundations. The replacement cost of these walls are included in each of the alternatives.

Additional Design Information Required

Currently, there are limited surveyed cross sections in this reach. Based on the recommended alternatives, the following minimum survey information is required for preparation of final design plans, quantities, and specifications of the levee:

- Re-establish the USACE baseline;
- Verify the profile of the top of levee every 50 feet and identify any intermediate grade breaks;
- Verify the 200-foot cross sections extending from the protected-side ROW line to the bank of the protected- and flood-side drainage canals;
- Develop the hydrographic cross sections of the drainage canals every 1,000 feet;
- Perform topographic survey of the proposed borrow pits; and
- Perform topographic survey of features such as fences, utilities, power poles, ditches, borrow pits, canals, and culverts.

Additional information would be required specifically for the T-walls at St. Mary's Pump Station, which include:

- Top and bottom profiles of the existing T-wall at St. Mary's Pump Station, structure corners, and grade on either side of the wall every 25 feet or at any intermediate grade points;
- Survey information along the discharge pipes in addition to the pipe supports;
- Three cross sections perpendicular to the proposed T-wall alignment for analysis. These should extend into both the suction basin and the discharge basin; and
- Full topographic survey of the features such as gates, fences, utilities, power poles, gasoline tanks, ditches, borrow pits, canals, buildings, signs, appurtenances, and culverts within the proposed ROW shown in the EAR-RHCA.

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A recent subsurface investigation program in 2006 and 2007 was completed by the USACE during preparation of this report but did not extend the full length of the reach. Within Soils Reach E, borings were completed along the centerline of levee at approximately 2,000-foot spacings up to 80 feet deep. Toe borings were alternated between the flood side and protected side on 2,000-foot spacings. Within Soils Reach F, only two new borings were completed, one at the centerline and one at the toe.

If Alternative 1 is selected as the preferred alternative, the borings along the centerline of the levee will need to be extended at least 20 feet beyond the pile tip to about to about EL -115. Additional borings and/or CPTs will be required between the existing centerline and toe borings in order to fulfill the current Hurricane and Storm Damage Reduction System (HSDRS) guidelines, which require undisturbed borings at 1,000-foot spacings and a general or CPT between the undisturbed borings at 1,000-foot spacings.

For the T-walls at St. Mary's Pump Station, the following is recommended:

- Five-inch diameter undisturbed borings should be spaced no more than approximately 100 feet along the alignment of the structure, with a minimum of three borings. The depth of the borings should be at least 20 feet greater than the estimated pile length. The pile tips extend to approximately EL -138, so the length of the piles are about 147 feet. A few additional borings are also recommended to be drilled about 100 feet from the wall alignment on both the protected side and the flood side;

- CPTs are recommended to be pushed adjacent to the two borings located 100 feet from the wall alignment. The CPTs could be used for developing pile capacity curves; and

- Overall, all subsurface requirements should follow the current HSDRS guidelines at the time of this report.

3.5.9 Design and Construction Schedules

The proposed schedule summarizes the time required for design and construction of the preferred alternative for LPV 148. The schedule is based on a notice to proceed date of December 18, 2008. A December 2008 notice to proceed is assumed based upon ongoing design work and allowing for negotiations. Approximately 7 months are

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scheduled for additional geotechnical testing along the levee. Final design will require approximately 12 months and will be submitted on December 21, 2009. Real estate acquisition and environmental compliance have not been completed.

Construction of the T-wall will commence on April 8, 2010, and is slated for final acceptance on approximately August 9, 2013. The T-wall structure will be built using the existing levee and, therefore, will lag behind the soil amendment task. The schedule also depicts a 60-day lag time between the commencement of the pile driving crew and actual T-wall installation. There is one overland access point at Highway 46 and another at Highway 39.

All work is estimated using 10-hour workdays on a 7-day-per-week work schedule. Three crews for the pile and sheet pile driving are assumed to work concurrently in order to meet the deadline. Work on LPV 148 will not be completed prior to the 2011 hurricane season based on the assumed production rates. Additional support, improved construction access, or revised scheduling will be required to complete this work by the mandated deadline. LPV 148 is considered the critical path for this Task Order.

3.5.10 Report Plates – LPV 148

Summary tables of the geotechnical and structural along with the plan and profile and typical sections are combined for each of the alternatives and presented by alternative following this report.

<u>Alternative 1 – T-wall Alternative on Existing Levee (Plates 3.5.1 – 3.5.12)</u>

- LPV 148 Plan and Profile Sheets for T-wall Alternative
- Typical Sections for T-wall Alternative

<u>Alternative 2 – Earthen Levees Using Stability Berms (Plates 3.5.13 – 3.5.34)</u>

- LPV 148 Plan and Profile Sheets for Stability Berms Alternative
- Typical Sections for Stability Berms Alternative

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Alternative 3 – Earthen Levees Using Deep Soil Mixing and Landside Shift (Plates 3.5.35 – 3.5.46)

- LPV 148 Plan and Profile Sheets for Soil-Mixing Alternative
- Typical Sections for Soil-Mixing Alternative

Alternative 4 – Earthen Levees Using Stability Berms with Staged Construction and Wick Drains (Plates 3.5.47 – 3.5.58)

- LPV 148 Plan and Profile Sheets for Stability Berms with Staged Construction and Wick Drains
- Typical Sections for Stability Berms with Staged Construction and Wick Drains

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3.6 LPV 149 – Chalmette Loop Levee – Caernarvon Floodwall and Floodgates

LPV 149 is a floodwall segment located at the south end of the Chalmette Levee Loop. It begins at the end of LPV 148 (Station 1560+25) and connects to the Mississippi River levee at Station 1598+22. The existing floodwall consists of capped and uncapped sheet pile I-wall and T-wall monoliths. A swinging floodgate is located at the railroad crossing with a sliding gate at Highway 39.

3.6.1 Design Evaluations – Alternative Alignments 1, 2, 3, and 4 – Design Considerations (Plates 3.6.1 – 3.6.41)

As part of the modified Scope of Work, four alternatives were considered for LPV 149 based on revised hydraulic criteria. These alternatives consisted of T-wall and gates along four different alignments. The plates following this report illustrate the proposed floodwall along each alignment. The table below summarizes the characteristics of each alternative. An interim report was submitted to the USACE which provided a comparison of the alternatives based on preliminary analysis and cost information for the purpose of identifying preferred alternatives to proceed with for this current effort. The table below depicts the selections made during the interim report selection meeting. Conceptual analysis was performed in order to develop these T-wall sections based on engineering judgment and limited load cases. The T-wall stem heights investigated ranged from 16.0 feet to 20.5 feet. Unbalanced forces were encountered in the preliminary analysis and contributed to more conservative designs for the foundations. Because the structures for this study will require a large number of piles, a pile load test was deemed appropriate during construction, reducing the pile capacity factor of safety from 3 to 2 for the Q-case. The more critical pile length of the S-case and Q-case was used for support. All quantities for the closure gates (highway and railroad) and sector gate (Alignment 4) were extrapolated from the information developed during the original EAR submitted in October 2007. The alignments are summarized below with Alignments 1 and 3 included for comparison:

Alternative	Style	Top Elevation	Alignment	Interim Selection
1	T-wall	24.0	Along Existing Wall/Levee	Abandoned
2	T-wall	24.0	Between Existing Wall/Levee and EBI	Proceed

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Alternative	Style	Top Elevation	Alignment	Interim Selection
3	T-wall/Gate	24.0	Between Elevated Boat Industries and Caernarvon Canal	Abandoned
4	T-wall/Gate	24.0	Across Caernarvon Canal and Between Caernarvon Canal and Freshwater Outlet Canal	Proceed

EBI Elevated Boat Industries.

Several factors led to the final decision to carry Alignments 2 and 4 to the Final EAR-RHCA stage. The issues related to Alignments 1 and 3 are as follows:

- Alignment 1: Pile tips encroach past existing ROW. To demolish and rebuild along the existing alignment, interim protection would be necessary during hurricane season. Building the roadway and railroad gate on the existing alignment presented many problems with construction phasing and interim protection considering the limited space in this area, driving piles in proximity to the adjacent homes may cause damage, and construction easements as well as additional ROW would be required.

- Alignment 3: This alignment ran west of the EBI building. This presented several issues with the building features such as the boat slip, loading dock, and crane placement. The boat slip presented a unique challenge in that a small sector gate or similar structure would have to be built. An option was also considered that moved the alignment farther east between the buildings. A floodgate would have been required and would have severely inhibited the operations of EBI. This alignment was also in proximity to the Caernarvon Canal, which presented potential construction problems. Pile driving issues as well as additional ROW would have also been issues for this alignment.

The objective for this phase of the project was to develop the preliminary costs for these alignments using limited structural and geotechnical analyses. Only the typical governing load case(s) were applied to the designs to develop the proposed cross sections. Alignments were refined to accommodate the cross sections required and to eliminate impacts with structures or other features. The proposed T-walls ranged in height from 16.0 feet to 20.5 feet. The at-grade portions of the alignments were assumed to have a top of footing at EL 3.5, based on existing ground profiles generated from LIDAR data. It should be noted that there are several areas of concern

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in regard to Hazardous, Toxic, and Radioactive Waste (HTRW) along the banks of the Caernarvon Canal. Any excavation associated with floodwall construction in the vicinity of the canal banks was considered as hazardous material and was priced accordingly.

Based on the geotechnical and structural analysis, the following typical cross sections were developed.

T-Wall Analysis Summary								
Alternative	**Wall Length (feet)**	**Stem Height (feet)**	**Stem Thickness (feet)**	**Base Thickness (feet)**	**Base Width (feet)**	**No. of Piles**	**Pile Tip Elevation**	**Batter (H:V)**
1	1,700	16.0	2.5	3.5	15/3	3	-97.0	1:4
		20.5 (for 160 feet along highway)						
2	1,820	20.5	2.5	3.5	25/4	4	-95.0	1:4
3	1,735	20.5	2.5	3.5	25/4	4	-92.0	1:4
4	2715	20.5	2.5	3.5	25/4	4	-102	1:4

The plates for the typical sections depict the size and layout of the floodwall elements. The cost estimates and quantities associated with these sections can be found in Appendix H. The designs/quantities for the railroad, highway, and sector gates in the original EAR were extrapolated down from EL 26.0 to EL 24.0. No new analyses or designs were developed for these gates.

The Phase I HTRW Environmental Site Assessment (ESA) was performed in 2008 on the area east and west of Chalmette Loop Levee—Caernarvon Floodwall, LPV 149, in Caernarvon, Louisiana (the “Property”), through personal interviews, visual, drive-by and walking inspections, and aerial photography, topographic maps, past environmental records, and Federal and State database reviews. Based on the ESA, both current and historical known Recognized Environmental Conditions (RECs) and suspected RECs were identified within the Property and were noted to have had or may have had some impact to the soil, gravel, or groundwater quality within the Property. It was recommended that soil and/or groundwater quality samples be taken at all locations where known or suspected RECs, current or historical, were identified. It was also recommended that legal surveyed locations of oil and gas wells, drilling

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pads, and production facilities be obtained. During the Phase I study, examples of known or suspected, current or historical RECs identified were leaking drums and containers containing used oil, petroleum products, and unknown materials dredged from the Caernarvon Canal, several aboveground storage tanks containing diesel, gasoline, and methyl ethyl ketone, and three formerly registered underground storage tanks containing gasoline and diesel. Additionally, other known or suspected historical RECs on adjacent and off-site properties were listed in the Phase I HTRW ESA report. A Phase II HTRW ESA was carried out concurrently with this EAR-RHCA study.

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LPV 149 – Flood side of Highway 39 Floodgate

3.6.2 Geotechnical

3.6.2.1 T-walls to EL 24.0

The geotechnical analyses were completed at each structure to determine preliminary design sections and cost estimates. The design criteria and analyses used to determine the recommendations of this report are documented in Appendix G.

Geotechnical analyses included slope stability, seepage, and pile capacities and are shown in Appendix G.

Subsurface stratigraphy and soil parameters were based on information found in the "U.S. Army Corps of Engineers Lake Pontchartrain, LA. and Vicinity, Chalmette Area Plan, Design Memorandum No. 3, General Design Supplement No. 1 Chalmette Extension" dated September 1968. This document was obtained from the IPET website at https://ipet.wes.army.mil/. Subsurface information used in the analyses is derived from Plates 45 and 49 of the referenced DM.

Allowable pile capacities were generated as described in Appendix G. Factors of safety of 2.0 for the Q-case (short term) and 1.5 for the S-case (long term) were assumed. The more critical pile length of two cases was used. A factor of safety of 2.0 was chosen for the Q-case due to the recommendation of a load test to address the large number of piles expected with the T-walls. Piles considered were 14x89-inch H piles and 24-inch diameter steel pipe piles. Pile capacity calculations and curves for each structure are also presented in Appendix G.

Settlement calculations are not included in the analysis for Alignment 1. This option utilizes the existing levee and it is assumed full foundation settlement has been achieved under the existing load. The existing levee will be raised to final grade with a pile supported T-wall section.

Alignment 2 T-Wall

The proposed concrete T-wall is designed to be placed on natural ground to bring the elevation to 24 feet. The stem averaged 21 feet in height. An initial T-wall and pile design was configured and slope stability analyses were performed using MOP.

The T-wall structure without the piling was modeled at the top-of-wall (EL 24.0) water level. The protected-side water level was assumed to be at ground surface, EL 3.5. The resulting slope stability factor of safety at the top of wall for the MOP wedge failure surfaces failed to meet the design criteria, indicating the presence of unbalanced loads. The resulting anchor force was 3,146 pounds per foot (lbs/ft).

The depth of the sheet pile was based on the bulkhead analysis in CWALSHT. A minimum sheet pile depth of approximately 55 feet below the foundation was required for stability. A seepage analysis was performed using Lanes Weight Creep Ratio

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method and resulted in a sheet depth of 28 feet. Thus, it is recommended that the sheet pile tip extend to EL -55.

Steel 14x89 H piles were chosen for the foundation of the T-wall. The worst load case that included water at the top of wall and a barge impact was analyzed. Only load case DCc was analyzed because wave loads were not available at the time of this report. The pile capacity curves based on the shear lines were used to determine a pile length of 103 feet (EL -98) where the pile capacity was approximately 78 kips tension and 118 kips compression. The greatest resulting axial loads on a pile were 49 kips in tension and 115 kips compression. The piles are spaced 5 feet apart along the length of the wall.

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Alignment 4 T-Wall

The proposed concrete T-wall is designed to be placed on natural ground to bring the elevation to 24 feet. The stem will average 21 feet in height. An initial T-wall and pile design was configured and slope stability analyses were performed using MOP.

The T-wall structure without the piling was modeled at the top-of-wall (EL 24.0) water level. The protected-side water level was assumed to be at the ground surface, EL 3.5. The resulting slope stability factor of safety at the top of wall for the MOP wedge failure surfaces failed to meet the design criteria, indicating the presence of unbalanced loads. The resulting anchor force was 5,055 lbs/ft.

The depth of the sheet pile was based on the bulkhead analysis in CWALSHT. A minimum sheet pile depth of approximately 27.5 feet below the foundation was required for stability. A seepage analysis was performed using Lanes Weight Creep Ratio method and resulted in a sheet depth of 27.3 feet. Thus, it is recommended that the sheet pile tip extend to EL -27.5 based on stability.

Steel 24-inch pipe piles were chosen for the foundation of the T-wall. The worst load case that included water at the top of wall and a barge impact was analyzed. Only load case DCc was analyzed because wave loads were not available at the time of this report. The pile capacity curves based on the shear lines were used to determine a pile length of 115 feet (EL -110) where the pile capacity was approximately 144 kips tension and 146 kips compression. The greatest resulting axial loads on a pile were 54 kips in tension and 140 kips compression. The piles are spaced 6 feet apart along the length of the wall.

3.6.3 Civil/Right-of-Way/Relocations/Utilities/Environmental

A review of pre-Hurricane Katrina surveys provided by the USACE and field verification indicates that the following utilities/facilities will require relocation based on the final modifications to the scope of work for LPV 149 Alignments 2 and 4 alternatives:

LPV 149 Alignment 2			
Right-of-Way			
Description		**Quantity**	**Units**
Additional Right-of-Way required		2.00	Acre
Utility Relocation			
Description		**Quantity**	**Units**
1578+83	Overhead powerline	75	Foot
1578+49	Gas main pipeline - diameter unknown	200	Foot
1578+46	Water main pipeline - diameter unknown	200	Foot
1578+89	Copper/fiber optic cable	75	Foot
1578+28	Hydrants (2)	2	Each
1570+70 and 1577+27	Storm drain pipe (855 feet) - diameter unknown	855	Foot
1564+00-1575+93	Overhead powerline - Deogracias Lane	1,400	Foot
1564+00 - 1578+49	Gas pipeline - Deogracias Lane - diameter unknown	1,400	Foot
1564+00 - 1578+46	Water pipeline - Deogracias Lane - diameter unknown	1,400	Foot
Facility Relocation			
Description		**Quantity**	**Units**
1564+00 - 1576+57	EBI, 900 St. Bernard Parkway	1	Each
1570+67	EBI Floodgate	1	Each
1576+50	Railroad Floodgate	1	Each
1578+20 - 1578+67	Highway 39 Floodgate	1	Each

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The residences and EBI were placed in the relocations estimate to be conservative. Once the plans and specifications design phase is initiated, these acquisitions may not be required.

LPV 149 Alignment 4		
Right-of-Way		
Description	**Quantity**	**Units**
Additional Right-of-Way required	4.86	Acre

3.6.4 Summary of Cost Analysis

The table below shows the total estimated cost of each alternative (not including escalation). Detailed cost estimates can be found in Appendix H.

Description	Real Estate ($k)	Environmental Mitigation Including Contingency ($k)	Contract Cost Including Contingency ($k)	Engineering and Design Cost Including Contingency ($k)	Supervision and Administration Cost Including Contingency ($k)	Project Cost ($k)	Operation and Maintenance ($k)	Total Values ($k)
	1	2	3	4	5	1+2+3+4+5 A	B	= A+B
LPV 149, Alt 2/Mod 4, T-wall	$278	$8,285	$40,713	$6,125	$4,655	$60,055	$2,042	$62,097
LPV 148, Alt 4/Mod 4, T-wall	$6.32	$8,808	$100,551	$13,605	$10,340	$133,309	$2,729	$136,038

For comparison purposes, the tables below provide the quantities of the major work items for each alternative presented above.

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LPV 149 – Alternative 2/MOD 4: T-wall		
Summary of Major Quantities		
	Quantity	**Unit**
T-wall 21' Stem Section	9,209	5.06 CY/LF
T-wall Sheet Pile	101,465	55.75 SF/LF
T-wall Pile	124,973	LF

LPV 149 – Alternative 4/MOD 4: T-wall		
Summary of Major Quantities		
	Quantity	**Unit**
Riprap	270	Tons
T-wall 21' Stem Section	13,738	5.06 CY/LF
T-wall Sheet Pile	76,699	28.25 SF/LF
T-wall Pipe Pile	199,100	LF

3.6.5 Recommended Alternative

For LPV 149, the recommended alternative is Alignment 2 – East of EBI. Each of the alternatives presented unique challenges as noted above. The advantages and disadvantages can be summarized as follows:

Alternative	Advantages	Disadvantages
Alignment 2 T-wall	• Reduced ROW as compared to Alignment 4 • No sector gate required • Reduced wetland/ hardwood impacts • Reduced HTRW impacts • Construction sequencing • Reasonable construction schedule • Reduced cost	• Public opposition • Construction activities encroach on EBI • Potential construction damage to adjacent structures (due to vibration) • Limited construction space

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Alternative	Advantages	Disadvantages
Alignment 4 T-wall	• Public prefers (protects EBI) • Does not encroach on residential or business community • Potentially easier ROW acquisition • Easier construction access • No interim protection plan	• More expensive • Sector gate required in Caernarvon Canal • Potential HTRW issue

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Each alternative had distinct advantages and disadvantages for these evaluation criteria. A distinct disadvantage for Alignment 2 is that EBI will not be inside the new line of protection. This presents potential issues for the community and will likely be considered to carry high importance in the final decision made by the USACE. However based on ranking the criteria above, Alignment 2 remains as the recommendation.

Based on the recommended alternative, the following minimum survey information is required for preparation of final design plans, quantities, and specifications:

- Re-establish USACE baseline;
- Verify the profile at the top of wall every 50 feet and identify any intermediate grade breaks in the monoliths and closure gates;
- Develop cross sections extending from a minimum of 100 feet either side of the existing centerline; and
- Develop topographic survey of features such as fences, utilities, power poles, ditches, borrow pits, canals, and culverts.

Before final design is accomplished, additional geotechnical investigation is required to verify subsurface conditions and design parameters. The LPV 149 alignment is

approximately 2,100 feet and the recommended additional investigation should be based on the following:

- Levee Centerline. Starting at the initial station and including the final station, drill CPT borings on approximate 500-foot centers and 5-inch undisturbed borings on approximate 1,000-foot centers;
- Flood-Side Toe. Starting at the initial station and including the final station, drill CPT borings on approximate 500-foot centers;
- Protected-Side Toe. Starting at the initial station and including the final station, drill CPT borings on approximate 1,000-foot centers;
- Entire Alignment. Within the 2,100-foot length, drill one additional 5-inch undisturbed boring in conjunction with a CPT along the flood-side toe;
- Railroad and Highway Closure Gates. Drill a 5-inch undisturbed boring at each gate location; and
- Assume boring depths of 130 feet.

Undisturbed samples will be utilized for classification and physical testing. Recommended physical testing includes unconfined compression, unconsolidated undrained (Q) shear tests, and consolidated undrained with pore pressure® shear tests.

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3.6.6 Design and Construction Schedules

The proposed schedule summarizes the time required for design and construction of the preferred alternative for LPV 149. The schedule is based on a notice to proceed date of December 18, 2008. A December 2008 notice to proceed is assumed based upon ongoing design work and allowing for negotiations. Approximately 2 months are scheduled for geotechnical testing. Final design will require approximately 10 months and will be submitted on October 23, 2009.

Construction of the T-wall along the existing alignment will commence on February 17, 2010, and is slated for final acceptance on approximately January 12, 2011. The T-wall structure will be built using the existing levee as interim protection. The schedule

depicts a 30-day lag time between the commencement of the pile driving crew and actual T-wall installation.

Real estate acquisition and environmental compliance have not been completed. All work is scheduled to be completed prior to the 2011 hurricane season.

3.6.7 Report Plates – LPV 149

Summary tables of the geotechnical and structural along with the plan and profile and typical sections are combined for each of the alternatives and presented by alternative following this report.

Alternative Alignment 2 T-wall (Plate 3.6.1)

- LPV 149 Plan and Profile Sheet for T-Wall

Alternative Alignment 4 T-wall (Plate 3.6.2)

- LPV 149 Plan and Profile Sheet for T-Wall

Alternative Alignments 2 and 4 (Plate 3.6.3)

- Typical Sections for T-Wall

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4. Conclusions

Based on the preliminary analysis and design performed for this EAR, the estimates of probable cost (not including escalation) for the alternatives for each reach are summarized below:

Description	Real Estate ($k)	Environmental Mitigation Including Contingency ($k)	Contract Cost Including Contingency ($k)	Engineering and Design Cost Including Contingency ($k)	Supervision and Administration Cost Including Contingency ($k)	Project Cost ($k)	Operation and Maintenance ($k)	Total Present Values ($k)
	1	2	3	4	5	1+2+3+4+5 A	B	= A + B
LPV 144, Bayou Dupre Control Structure to EL 31.0	-	-	$62,709	$7,838	$5,957	$76,504	$1,622	**$78,126**
LPV 145, T-wall	$485	$107	$479,681	$59,960	$45,570	$585,802	$9,867	**$595,670**
LPV 145, Deep Soil Mixing	-	-	$746,110	$93,264	$70,880	$910,254	$45,594	**$955,847**
LPV 145 Staged with Wick Drain – All Stages	$3,492	$9,195	$858,078	$107,260	$81,517	$1,059,542	$69,328	**$1,128.870**
LPV 145, Oct 07 All Stages	$6,305	$4,264	$644,044	$80,505	$61,184	$796,302	$82,257	**$878,560**
LPV 146, T-wall	-	-	$449,798	$56,225	$42,731	$548,754	$8,294	**$557,048**
LPV 146, Deep Soil Mixing	-	-	$706,146	$88,268	$67,084	$861,499	$70,586	**$932,085**
LPV 146, Staged with Wick Drain All Stages	-	-	$701,100	$87,638	$66,605	$855,342	$56,399	**$911,741**
LPV 146, Oct 07 All Stages	$1.34	$110	$788,028	$98,503	$74,863	$961,505	$74,745	**$1,036,250**
LPV 147, Highway 46 Ramp	$2,527	-	$55,968	$6,393	$5,317	$70,205	-	**$70,205**

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Description	Real Estate ($k)	Environmental Mitigation Including Contingency ($k)	Contract Cost Including Contingency ($k)	Engineering and Design Cost Including Contingency ($k)	Supervision and Administration Cost Including Contingency ($k)	Project Cost ($k)	Operation and Maintenance ($k)	Total Present Values ($k)
	1	2	3	4	5	1+2+3+4+5 A	B	= A + B
LPV 147, Bridge	-	-	$22,029	$2,202	$1,674	$26,406	$500	**$26,906**
LPV 148, T-wall	$0.65	$53	$643,685	$80,461	$61,150	$785,349	$11,403	**$796,752**
LPV 148, Deep Soil Mixing	$327	$7,297	$1,060,760	$132,595	$100,772	$1,301,752	$69,071	**$1,370,823**
LPV 148, Staged with Wick Drain All Stages	$622	$31,462	$1,025,171	$128,146	$97,391	$1,282,793	$75,915	**$1,358,708**
LPV 148, Oct 07 All Stages	$594	$29,148	$1,132,529	$141,566	$107,590	$1,411,426	$106,976	**$1,518,403**
LPV 149 – Alt 2 Mod 4, T-wall	$278	$8,285	$40,713	$6,125	$4,655	$60,055	$2,042	**$62,097**
LPV 149 – Alt 4 Mod 4, T-wall	$6.32	$8,808	$100,551	$13,605	$10,340	$133,309	$2,729	**$136,038**

NOTES:
1. Values for second and third construction stages included in multi-lift alternative LPVs are 2008 dollars.
2. Escalation to midpoint of construction is not included in these calculations.
3. "Present Value" calculations were only considered in the Operation and Maintenance cost.
4. See project summary sheets for additional details.

A summary of alternatives recommended to be carried forward into final design is as follows:

- LPV 144: Sector Gate to EL 31.0 on flood side of the existing;
- LPV 145: T-walls;
- LPV 146: T-walls;

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- LPV 147: Bridge over T-wall;
- LPV 148: T-walls; and
- LPV 149: T-wall Alignment 2.

Estimates of probable cost for the final recommended alternatives including **contingencies and escalation** are provided below:

Description	Real Estate ($k)	Environmental Mitigation ($k)	Contract Cost ($k)	Engineering and Design Cost ($k)	Supervision and Administration Cost ($k)	Project Cost ($k)	Operation and Maintenance ($k)	Total Values ($k)
	1	2	3	4	5	1+2+3+4+5 A	B	A+B
LPV 144, Bayou Dupre	-	-	$62,709	$7,839	$5,957	$76,506	$1,622	$78,127
LPV 145, T-wall	$506	$111	$551,008	$62,680	$52,346	$666,651	$9,867	$676,518
LPV 146, T-wall	-	-	$510,693	$58,775	$48,516	$617,984	$8,294	$626,278
LPV 147, Bridge	-	-	-	-	-	-	-	$26,406
LPV 148, T-wall	$0.679	$55	$750,154	$84,110	$71,264	$905,585	$11,402	$916,988
LPV 149, Alt 2/Mod 4 T-wall	$289	$9,165	$45,033	$6,402	$5,149	$66,038	$2,042	$68,080
					TOTAL	**$1,589,607**	**$21,738**	**$2,392,397**

A summary of the total design and construction durations for the recommended alternatives is listed in the table below. Full Gantt chart schedules for the recommended alternatives are shown in Appendix I.

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Description	Project Start Date (Plans and Specifications)	Construction Start Date	Project Acceptance Date	Total Construction Duration (Days)
LPV 144, Sector Gate to EL 31.0	6/11/08	7/29/09	5/27/11	510
LPV 145, T-wall	12/18/08	12/31/09	9/25/12	937
LPV 146, T-wall	12/18/08	12/31/09	2/22/12	721
LPV 147, Bridge	12/18/08	11/9/09	6/20/11	394
LPV 148, T-wall	12/18/08	12/31/09	8/9/13	1255
LPV 149 – Alt 2/ Mod 4, T-wall	12/18/08	11/7/09	1/12/11	279

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The following conclusions can be reached from the evaluation of the alternatives presented in this EAR-RHCA for the St. Bernard Parish Hurricane Protection System – LPV 144 through LPV 149:

- Based on topography and subsurface soil conditions (using historical data), a variety of alternatives were recommended for the study reaches;
- Optimization of the recommended alternative, including the estimates of probable cost, will be possible during a final design phase and shall be based on recent survey and subsurface data;
- The subsurface conditions in this region present challenging scenarios to provide flood protection to the required 100-year flood elevations;
- Additional topographic and hydrographic survey information will be necessary for all recommended alternatives;
- The revised alternatives were an improvement over the original study. Some cost savings were realized as the alternatives were refined; and
- Additional subsurface investigation will be necessary to complete the final design for any of the alternatives in LPV 145 through LPV 149. Additional investigations should incorporate geophysical methods, CPT, and undisturbed borings. The

USACE has been conducting subsurface investigations along the Federal levees. However, for this report, some of the results of these investigations were not available to the design team and thus were not incorporated in the studies.

In conclusion, while this EAR-RHCA presents viable and constructible alternatives for the St. Bernard Parish HPS, further refinement/optimization of the recommended or final government selected alternatives may be realized during the plans and specifications phase. As noted in the Executive Summary, from a program perspective, it is known that the sub-issues of ROW acquisition, borrow material, and cost sharing requirements are at the forefront of the decision-making process for the USACE. Borrow material source, proximity, and cost are program-wide concerns. The design team is ready to assist the USACE in all aspects of the final stages of planning and the decision-making process.

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Appendix A

Independent Technical Review

Appendix B

Design Criteria and Final Scope of Work

Appendix C

LPV 145

Appendix D

LPV 146

Appendix E

LPV 147

Appendix F

LPV 148

Appendix G

LPV 149

Appendix H

Cost Estimate Report

Appendix I

Construction Schedules and Right-of-Way Plates

Appendix J

Crystal Ball Risk Analysis