This appendix describes the input, assumptions, design criteria, analysis, and limitations used to develop the recommendations of this report.

Survey Data

- **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. Bathymetric surveys 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 extended 450 feet and 550 feet from the centerline of the levee on the protected side and flood sides, 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. Bathymetric surveys 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.

  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) 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 discernable 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 National 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.

Existing Plan Documentation

- DM No. 3 (1966);
- DM No. 5 (1968);
- Hurricane Katrina Repairs and Modifications for Bayou Dupre Control Structure (October 2005) by Brown Cunningham Gannuch (BCG);
- Construction Drawings for Flood Protection Design including Borrow Pits and Additional Fill Placement (June 2006) by ARCADIS;
- Levee Restoration East of Bayou Dupre (October 2005) Construction Drawings by USACE;
- Bayou Road Floodgate construction drawings or as-builts were not obtained in time for this study. However, if it can be determined that Bayou Road was Highway 46 in 1982 before a realignment, then construction plans can be found as part of the Creedmore Drainage Structure Drawings as described next;

- Creedmore Drainage Structure Construction Drawings, labeled, “Verret Closure, Levee Shaping and Creedmore Drainage Structure,” dated July 1982, designed by the USACE;

- St. Mary’s Pump Station Construction Drawings, labeled, “Plans of Proposed Drainage Pumping Station” or “Caernarvon Pumping Station (Pumping Station No. 8)” (unsigned DGN cad files), dated July 1994, designed by Burk-Kleinpeter, Inc. (also dated 5/6/97 as “Record Drawings” in the revision block);

- Highway 39 Construction Drawings, labeled, “Caernarvon Floodwall and Levee,” dated February 1975, designed by the USACE;

- Chalmette Area Plan Verret to Caernarvon (September 2006);

- 1418 & 1420 Bayou Road Borrow Investigation (November 2006);

- Conceptual Design Modifications Bayou Bienvenue (September 2006);

- Chalmette Loop Verret to Caernarvon Levee Reach Right-of-Way Drawings (August 2006) by USACE;

- Chalmette Loop Verret to Caernarvon Levee Reach (September 2006) Construction Drawings by Burk-Kleinpeter;

- Inner Harbor Navigational Canal National Environmental Policy Act Environmental Compliance;

- Preliminary Hurricane Protection (May 2007);

- LPV 144-149 Preliminary Hurricane Protection (May 2007); and

- Emergency Restoration B/L Station 383+00 to B/L Sta 704+00 (October 2005) Construction Drawings by the USACE.

Subsurface Conditions

Overall, the geology of St. Bernard Parish is described in DM No. 3 and concisely described herein. St. Bernard Parish lies within outcrops of the Quaternary natural levee and delta plain deposits. The natural levee borders the Mississippi River and forms the higher areas of St. Bernard Parish as it slopes down away from the river. The natural levee deposits are composed of normally consolidated clays and sandy silts. The delta plain deposits are composed of fat and lean clays with varying organic content formed from the swamps, marshes, and distributary bays.

Underlying these recent deposits is a relic delta lobe of the Mississippi River. The St. Bernard delta lobe, formed approximately 2,800 years ago, contains the sequence of a prodelta overlain by a mixed layer of intradelta and interdistributary deposits. The prodelta contains fat clays overlying a marine sand representing the toe of the delta building out into the Gulf of Mexico. The intradelta deposits are normally consolidated silty and sandy clays deposited in the advancing delta lobe. The interdistributary unconsolidated clays were formed by fines falling from suspension of low energy flood waters in the delta plain.

Besides reviewing the existing data and geology, the design shear strengths as well as other soil properties were primarily based upon recently completed soil borings, CPT and lab testing data in the following soil reports provided by the USACE:

- Geotechnical Investigation for the Chalmette Loop Levee (Bayou Bienvenue to Highway 46 Extension), St. Bernard Parish, LA, LPV 145 and 146 conducted by Omaha District of USACE; and

- Geotechnical Investigation of LPV 148 completed by Eustis Engineering, Inc.

- 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. Subsurface information used in the LPV 149 analyses were derived from Plates 45 and 49 of the referenced DM.

Specific details about the subsurface conditions encountered for each reach are described in their corresponding sections.

Overall, the recommendations and conclusions in this report are dependent on the soil parameters used. It should be noted that soil properties and profiles can change over time (e.g., increase in strength due to consolidation or lose strength due to disturbance); therefore, the results of the analysis and design sections may not be representative of actual recent conditions. Additional information may be required for further design of the selected alternative or alternatives.

Additional Assumptions for Modeling Cross Sections

- For options not considering a landside shift or new alignment, the centerline of the proposed alternatives coincides with the centerline of existing levee.

- The elevation of the top of the levees was increased to account for overbuild and 18 inches of slope protection. Slope protection consisted of riprap from toe to toe of the levee. The amount of overbuild accounted for was computed by determining the amount of primary consolidation settlement in a given time frame. Secondary consolidation settlement and regional subsidence were already accounted for in the design elevations.

- Piezometric head for levees was drawn linearly through the levee from the water level on the flood side to the toe of the levee on the protected side.

- Wave berms, as detailed in the scope, were modeled on the flood side for all earthen levee alternatives. However, in most cases for the stability berm alternative, additional fill above the wave berm was required. To account for settlement over time, the wave berm elevations as shown in the scope were raised concurrently in the same manner as the levee. For instance, if the top of the levee required a 3-foot raise, then the minimum wave berm configurations were raised 3 feet.

- Based on the provided hydraulic report, the proposed wave berm configurations associated with those earthen levee alternatives in different hydraulic reaches were as follows:

Hydraulic Reach	Condition	Design Elevation	Levee Slope	Top of Slope Elevation	Breaker Slope	Toe of Slope Elevation
SB-11	Existing	29.0	5H:1V	18.4	10H:1V	10.5
	Future	31.5	5H:1V	19.9	10H:1V	12.0
SB-12	Existing	27.5	4H:1V	17.3	8H:1V	6.5
	Future	30.0	4H:1V	18.8	8H:1V	8.0
SB-13	Existing	26.5	4H:1V	16.4	8H:1V	6.5
	Future	29.0	4H:1V	17.9	8H:1V	8.0
SB-15	Existing	26.5	5H:1V	15.6	10H:1V	6.5
	Future	29.0	5H:1V	17.1	10H:1V	8.0
SB-16	Existing	26.5	5H:1V	17.5	12H:1V	7.5
	Future	29.0	5H:1V	19.0	12H:1V	9.0
SB-17	Existing	26.5	5H:1V	18.0	12H:1V	7.5
	Future	29.0	5H:1V	19.5	12H:1V	9.0

- There's no actual proposed "wave berm" for the T-wall alternative. The design elevation of the levee associated with the T-wall alternative is at El 19.9 for SB-11 and SB-12, El 17.1 for SB-13 and SB-15 and El 19.0 for SB-16 and SB-17. The levee slope is to be at 4H:1V for the T-wall alternative. If the existing levee elevation is below the above required one, raising of the existing levee will be necessary. However, if the existing levee is higher than the design elevation, no degrade of existing levee was proposed since it exceeds the required protection level. Specific assumptions beyond those outlined here are described in corresponding sections.

Hydraulic Design Criteria

Based on the hydraulic reach information received from USACE, the design water elevations associated with earthen levee alternatives were summarized below. Other hydraulic information necessary for analysis was assumed and also summarized in the table below. The 2057 design elevations associated with structural T-wall options are at elevation (El) 32.0 for SB11 and SB-12, El 27.5 for SB-13 and SB-15, El 29.5 for SB-16 and SB-17, El 31.0 for SB-19 and El 30.5 for SB-20, respectively.

Hydraulic Reach	2057 High Water Elevation	2057 Still Water Elevation	2010 High Water Elevation	2010 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

Geotechnical Design Criteria

The following design criteria were provided by the U.S. Army Corps of Engineers (USACE) for use as the basis of design:

- T-Wall/L-Wall/Kicker Pile Wall Design Criteria (20 April 2006); and
- DRAFT Hurricane Protection System (HPS) Guidelines (March 2007).

Other applicable USACE publications included:

- Engineer Manual (EM) 1110-2-2502, Retaining and Floodwalls, September 1989;
- EM 1110-2-2906, Design of Pile Foundations, January 1991;
- EM 1110-2-2504, Design of Sheet Pile Walls, March 1994;
- EM 1110-2-1913, Design and Construction of Levees, April 2000;

- EM 1110-2-1901, Seepage Analysis and Control for Dams, April 1993;
- EM 1110-2-2100, Stability Analysis of Concrete Hydraulic Structures, December 2005;
- Engineering Technical Letter 1110-2-569, Design Guidance for Levee Underseepage, May 2005; and
- Division Regulation 1110-1-400, Soil Mechanic Data, December 1998.

The following computer software programs were utilized for the geotechnical analysis of this study:

- "Stability with Uplift" for Slope Stability using USACE Mississippi Valley Division (MVD) Method of Planes;
- SLOPE/W Versions 2004 or 2007 by GeoStudio for Slope Stability using Spencer's Method;
- CE Sheet Pile Wall Design/Analysis Program, "CWALSHT";
- CSETT for settlement estimates; and
- SEEP/W Versions 2004 or 2007 by GeoStudio for Seepage.
- GROUP 7 for Windows by ENSOFT, Inc. for group lateral pile analysis

Geotechnical Analysis

Geotechnical analyses include limited slope stability and pile capacities; seepage and settlement were based on engineering judgment as appropriate for each reach and alternative. This section briefly describes the criteria used to model the analysis.

Cross-Section & Soil Parameters

Existing soil parameters and shear lines were determined using the latest subsurface exploration data as well as using old DMs, engineering judgment, and typical values used in the area. For materials used in construction of the new levees and structures, the following parameters were used:

• Uncompacted Clay Fill	γ = 110 pcf	c = 200 psf
• Semi-Compacted Clay Fill (90 Percent)	γ = 110 pcf	c = 400 psf
• Compacted Clay Fill	γ = 115 pcf	c = 600 psf
• Riprap	γ = 130 pcf	ϕ = 30 degrees
• Limestone	γ = 120 pcf	ϕ = 34 degrees
• Concrete	γ = 150 pcf	

Both flood side and protected stability berms consisted of compacted clay fill.

Slope Stability

For T-wall alternatives, Method of Planes (MOP) was conducted under HWL condition and the unbalanced forces, if any, were determined and used for structural design. Only the most critical failure surface was used to determine the anchor force in CWALSHT. For the levee alternatives, both Method of Planes (MOP) and Spencer's Method were utilized to determine the required cross section for design. High water level (HWL), Still Water Level (SWL), and Flood side cases were completed for all sections for the 2010 design elevations. Most critical cases were checked for the 2057 design elevations. The following FS were used as taken from the latest HPS design guidelines (March 2007):

Case	MOP FS	Spencer's FS
HWL – Levees	1.2	1.4
HWL – T-Walls	1.3	NA
SWL – Levees	1.3	1.5
SWL – T-Walls	1.3	NA
Flood-Side Check	1.3	1.4

*Note: Spencer's Method was not required to be performed for T-wall structures.
FS Factors of Safety.
HWL High Water Level.
MOP Method of Planes.
NA Not Available.
SWL Still Water Level.

Levees

Global instability was first checked by using the USACE MVD's Method of Planes Slope Stability with Uplift Computer Program, as instructed by the USACE. This program uses wedge-type failures with force equilibrium. The soil strength and unit weight was modeled varying vertically by inputting the average and bottom cohesion of each layer and varying horizontally by inserting centerline and toe verticals. Searches were performed at the bottom of each layer with the lowest cohesion at that elevation. The locations of the most critical active and passive wedges were searched (alternative wedge geometries tried) until the lowest factor of safety was obtained for each stratum. Once the most critical wedge was determined, the user adjusted the input file to plot the most critical wedges for each soil strata. The computed factor of safety (FS) was determined by the following equation:

$$FS = \frac{RA + RB + RP}{DA - DP}$$

The numerator of this equation includes all the resisting forces and the denominator includes all the driving forces. The distance between the active and passive wedges was located a minimum of 0.7H from the active point, where H is the height of the active wedge as it exits the ground surface. Failure surfaces less than 0.7H were ignored as directed by the USACE.

The second slope stability method used was Spencer's Method using SLOP/W Versions 2004 or 2007 program created by GeoStudio. Both the block and circular searches were performed. For the block search, wedge-shaped failure surfaces were generated using force and moment equilibrium. Each layer was investigated to determine and narrow down the position where the lowest FS resulted. The FS was determined by using the cohesion at that elevation. For the circular search, the program automatically searched for the lowest FS regardless of elevation using force and moment equilibrium. Regions are used to model the soil strengths and unit weights, which can vary vertically within each layer, but horizontally they are limited by the number of regions between the centerline and the toe.

For the earthen levee alternatives with stability berms and with staged construction, the levee construction was divided into two or more lifts. Only shear strength gain was determined for the staged construction alternative by considering the effect of wick drains on the first lift. The final configuration was modified until all minimum FS were met. For soil mixing, a soil-mixed region, consisting of soil with a maximum in-situ shear strength of 2,300 pounds per square foot (psf), was altered in width and depth until all minimum FS were met. The soil-mixed regions were easily modeled with the Spencer's method. However, in the MOP, due to the limited number of verticals within the program, the centerline cohesions were eliminated so that the analysis could be run. Unit weights of the soil-mixed regions remain the same as the in-situ because of the disturbance resulting from the mixing process.

T-Walls

Global instability for T-wall structures was only checked by using the MOP Slope Stability with Uplift Computer Program, as instructed by the USACE. The soil strengths were factored by an FS of 1.3, and the locations of the most critical passive wedges were searched while fixing the active wedge at the base of the T-wall footing until the lowest FS was obtained for each stratum.

Failure surfaces located beneath the bottom of the footings were checked to determine if any unbalanced forces were present. Free water was deducted from the net force if present in the active wedge and was added to the net force if present in the passive wedge. Free water is defined as the water that is above the ground surface but is no lower than the bottom of the wall base slab.

If the results indicate that unbalanced loads were not present, the structure is required to carry only the net at-rest loads acting above the base. If unbalanced loads are encountered, the structure is required to carry the net at-rest loads acting above the base plus the anchor load transferred from the sheet pile. The anchor force is determined by performing an anchored bulkhead analysis using CWALSHT program. The structure base is modeled as the anchor at the top of the sheet pile cutoff wall, which is embedded below a fictitious ground surface. The fictitious ground surface is the layer where no more unbalanced loads is encountered. The sheet

pile cutoff is then subjected to lateral forces from the unbalanced loads above the fictitious surface in order to determine the anchor force and sheet pile tip. An FS of 1.0 was used to compute the anchor force, and an FS of 1.5 was incorporated while finding the minimum sheet pile tip.

Pile Capacity

Pile capacity analysis was required for structures supported on deep foundations. Analysis criteria used by the team was outlined in the Scope of Work and detailed as part of EM 1110-2-2906, Design of Pile Foundations, and T-Wall Design Criteria. Allowable pile capacities and allowable uplift curves were generated by using hand calculations and spreadsheets. If unbalanced loads were present based on the slope stability, pile capacity was neglected above the fictitious surface. Factors of safety of 3.0 for the Q-case (short-term) and 1.5 for the S-case (long-term) were assumed for those loads without pile load tests assumed for LPV 145, 146, 147 and 148. Pile load tests were assumed to be performed before construction of the LPV149 T-Walls, thus the factor of safety for the Q-case was reduced to 2.0 while the S-case remained at 1.5. The more critical pile length of two cases was used.

Seepage

Seepage of both levees and T-walls was checked. Analysis criteria used by the team were outlined as part of the EMs for levees and T-Wall Design Criteria for structures. For levees, SEEP/W was used to determine exit gradients, which were limited to less than 0.5 within 400 ft from the levee toe on the protected side, which translates to an FS of 1.6 for the typical unit weights used here. Also, the FS at the levee toe and berm toe should not exceed 1.5 and 1.1, respectively. If the exit gradients were greater than required, methods of cutting off seepage were investigated such as cut-off trenches, pervious toe trenches, sheet pile walls, or revised soil mixed regions.

For T-walls, seepage requirements indicated that the sheet pile depth should be a minimum of 10 feet or the depth required by Lane's Weighted Line of Creep, whichever was deeper, if no unbalanced loads were present. If unbalanced loads were present, the sheet pile depth was determined by anchored bulkhead analysis.

The following hydraulic conductivity values were typically assumed:

Material Type	Horizontal Permeability cm/sec	Vertical Permeability cm/sec
Fill	$1x10^{-7}$	$1x10^{-7}$
CH	$1x10^{-8}$	$1x10^{-8}$
PT	$1x10^{-5}$	$1x10^{-7}$
SM / SP	$1x10-^{3}$	$1x10^{-3}$
Sheet Pile	$3x10^{-11}$	$3x10^{-11}$

Settlement

Settlement was evaluated to determine the amount of overbuild required from primary consolidation of newly placed fill for the anticipated lift schedule and full service time of the levees. Detailed laboratory information from the recent subsurface investigation was available for LPV 145 and LPV 146 for determining the settlement whereas, for LPV 147, LPV 148 and LPV 149 very limited laboratory information was available from previous investigations and DMs. Most of the soil parameters required for settlement calculations for LPV 147 and LPV 148 were derived based on the LPV 146 laboratory data. Local area values were also considered in judging the consolidation properties of the soil below the levees. The USACE program CSETT was used to model settlement under the levee embankment. The time-rate primary consolidation settlement was determined and considered for overbuild purposes according to anticipated lift schedule. For the staged construction option, the remaining settlement of the first lift after 1 year was added to the ultimate settlement caused by the final lifts. For the option of stability berms with staged construction, wick drains were considered to be included during the 1st lift construction. As a result, in each soil layer penetrated by the wick drains, the horizontal hydraulic conductivity was assumed to be twice the vertical hydraulic conductivity in that layer determined based on laboratory tests. Consequently the settlement of the 1st lift after one year can be properly estimated considering the effect of wick drains.

Secondary subsidence was not calculated because these estimates were built into the design elevations originally given. Fill was also added to the wave berms to account for settlement over time. Settlement taking place during construction is accounted for in the overbuild quantities. Actual top of levee elevations on future design plans would include the anticipated settlement during construction.

Shear Strength Gain with Staged Construction

The analysis of shear strength gain in soft clay material was evaluated for staged construction alternatives. Shear strength gain occurs in soft clays over time as a result of the consolidation process from the additional fill placement. The amount of strength gain is dependent on many factors including the amount of fill placement, the time allowed to settle, and how quickly the pore water can dissipate. The greater amount of fill, the greater the time allowed to settle, and the least time it takes to dissipate the water all contribute to greater shear strength gain.

Wick drains were used in one of the staged alternatives in order to speed up pore water dissipation. The benefit of adding wick drains is that it decreases the time required to consolidate and thus increases the shear strength gain given a specific time period. The wick drains are artificially created drainage paths that can be installed by one of several methods and which can have a variety of physical characteristics. The use of wick drains along with pre-compression (surcharge) has the sole purpose of shortening the drainage path (distance to a drainage boundary) of the pore water, thereby accelerating the rate of primary consolidation. Procedures from Prefabricated Vertical Drains Volume 1: Engineering Guidelines FHWA/RD-86-168 were used in determining the spacing and requirements for design.

To estimate the shear strength gain with or without wick drains, the design procedures documented in DM No. 3 and DM No. 5 were slightly modified and used. To begin, a trial first lift was proportioned so that slope stability met the required FS when using the same shear strengths as the other alternatives without staged construction, or "Q" strengths. Secondly, the consolidations in each stratum were calculated for some time interval, not to exceed 1 year. If the soils were allowed to settle to 100 percent consolidation regardless of time, the amount of shear strength gain would be equivalent to "R" strengths at a certain overburden pressure. Up to 1 year, only a percent of the total consolidation was met. Therefore, the percent increase in shear strength from the "Q" strength to the "R" strength was based on the percent consolidation under the first lift load. This increase in strength gain was used when the slope stability of the second lift, including overbuild, was evaluated.

Deep Soil Mixing

The purpose of using deep soil-mixing is to increase the shear strength of weak foundation soils by installing soil-binder columns or panels in a grid or wall type pattern. The binders that are mixed with the soil can be lime, cement, or cement-type materials.

The columns or panels are installed by first disturbing the soil on the auger's downward stroke to the designated depth. As the augers withdraw back up, the binder is mixed with the soil. Once the binder combines with the in-situ water (dry soil-mixing procedure) or with water added (wet soil-mixing procedure), the columns and/or panels will strengthen. The unconfined compressive strengths of the soil-mixed columns depend on the soil type. Column strengths under 100 pounds per square inch (psi) are readily obtained in a variety of soils. The strength of the overall soil-mixed region is dependent on the spacing, pattern, and strengths of individual columns, as well as the surrounding in-situ soil. For this report, a composite strength range between 2,000 and 2,300 psf was used. The details such as column sizes, spacing, and pattern are final design issues to be determined at a later date; however, for cost purposes, a 40 percent replacement ratio was assumed.

The centerline of the proposed levee was shifted towards landside from existing levee centerline with the deep soil mixing option. The location of the deep soil mixing was placed between the existing levee protected side toe to the proposed new levee protected side toe for the following reasons: (1) construction equipment would not be able to fit on the existing side slopes with the current configuration; (2) soil-mixing columns are limited in depth to about 90 feet from where the rig rests; and (3) differential settlement may be an issue if only partial soil mixing block sits under the proposed embankment.

Structural Design Criteria

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

- DRAFT Hurricane and Storm Damage Reduction System (HSDRS) Guidelines (March 2007);
- Elevations for Initial Design of Hurricane Protection Levees & Structures within the New Orleans District (September 2006); and
- One Percent Future Wave Forces (at Bayou Bienvenue Floodgate, Bayou Dupre Floodgate, St. Mary Pump Station).

Other applicable USACE publications included:

- EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures, June 1992 (including Change 1, August 2003);
- EM 1110-2-2105, Design of Hydraulic Steel Structures (including Change 1), May 1994;
- EM 1110-2-2502, Retaining and Floodwalls, September 1989;
- EM 1110-2-2906, Design of Pile Foundations, January 1991;
- EM 1110-2-2705, Structural Design of Closure Structures for Local Flood Projects, March 1994;
- EM 1110-2-2703, Lock Gates and Operating Equipment, June 1994; and
- EM 1110-2-2007, Structural Design of Concrete Lined Flood Control Channels.

Technical publications included:

- American Concrete Institute (ACI), Building Code and Commentary, ACI 318-99;
- American Institute of Steel Construction (AISC), Manual of Steel Construction (9th Edition); and
- American Society of Civil Engineers (ASCE), ASCE 7, Minimum Design Loads for Buildings and Other Structures.

Computer software utilized:

- MathCAD Version 14;
- CE Pile Group Analysis (CPGA); and
- Structural Analysis and Design Software (STAAD).

LPV 145, 146, 148 T-wall Alternatives

The LPV 145, 146 and 148 T-Wall structures will require typical USACE concrete T shaped flood walls, which include concrete walls, concrete bases (footings), concrete stabilization slabs, steel sheet piles and pile foundations. The project Scope of Work requires that the T-Wall structural analysis be based on limited structural calculations for this project phase. The limited structural calculations are to include only the High Water Level (HWL) condition with and without unbalanced forces and in combination with the appropriate boat impact (BI) load applied at the top of wall. The project hydraulic recommendation, provided in the HPO-LFA-0091-07 Report, states "the wave forces should not be considered large because the still water level is below the top of fill." For this project phase, the T-Wall structures are not to be analyzed for combinations which include still water level and wave force conditions.

The T-Wall structure designs for this study were conducted in accordance with the latest (as of December 2007) applicable COE publications such as EM 1110-2-2104 "Strength Design for Reinforced Concrete Hydraulic Structures", EM 1110-2-2502 "Retaining and Flood Walls", EM 1110-2-2906 "Design of Pile Foundations" and EM 1110-2-2504 "Design of Sheet Pile Walls."

The T-Wall structure design forces are in accordance with the Elevations For Initial Design of Hurricane Protection Levees and Structures Within the New Orleans District September 2006 Edition, as modified by the Hurricane and Storm Damage Reduction System Design Guidelines (HSDRS) March 2007 Edition, except as recommended by the project HPO-LFA-0091-07 Hydraulic Design Recommendations December 2007 Edition. Some of the recommendations of the HSDRS October 2007 Edition are also applied as noted herein.

The T-Wall designs for this study were performed with a combination of hand calculations, project specific Excel spreadsheets, COE CE Pile Group Analysis Program (CPGA) and the Structural Analysis and Design Software (STAAD).

T-wall Fronting Protection for St. Mary's Pump Station in LPV 148

This analysis for conceptual design is in accordance with the structural design criteria presented in the Engineering Alternatives Report and as described herein. The conceptual design at St. Mary 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 two 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 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 final design.

New piles will be located and driven between 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 levee. New I-walls were not to be considered for this study.

The assumptions for conceptual T-wall design consist of the following:

- The design top of wall is at elevation 30.5 feet, NAVD-88, as provided for this study.
- The bottom of footing is taken to be at elevation -1.0 foot, NAVD-88, to match the plans of the existing T-wall.
- The wave force at St. Mary 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 final design.
- Deflection criteria for the T-wall at the discharge pipes is 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.
- The existing construction plans labeled, Caenarvon 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 remains the same distance of 20.5 feet on each side of the wall. (Sheet No. S-2 of the plans incorrectly shows the distance between centerlines as 19.5 feet on the protected side of the T-wall.)

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 Pile Group Analysis program (CPGA), 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 the St. Mary pumping 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.

Floodgates for LPV 144 and 149

The floodgate closure structures in this study consist of a movable steel gate mounted to pile-founded concrete columns/walls and base with a steel sheet pile cut-off. Similar to T-walls, piles are the main components that support the structure. Piles were battered toward protected and flood sides. The steel sheet pile provides both seepage cut-off beneath the wall and resistance to foundation instability.

Openings for roadways and railroads were designed to match the existing structures to be replaced, where applicable, and comply with the requirements of the American Association of State Highway and Transportation Officials (1989) and the American Railway Engineering and Maintenance-of-Way Association (2007).

For ease of closure and practicality, considering the increased gate heights, trolley gates were investigated to replace existing structures.

The proposed sector gate structure at Bayou Dupre consists of a pile-founded concrete base slab, steel sheet pile cut-off, concrete gate bay walls, and a steel-framed sector gate. The following criteria were used in the analysis and design of the alternatives:

<u>Sector Gate (at Bayou Dupre)</u>

Top of Gate Walls	EL 31.0 feet
Sill	EL -13.3 feet
SWL (Still Water Level)	EL 18.9 feet
Width of Gate Channel Opening	56 feet

Maximum Differential Head:	
Flood Side	EL 18.9 feet
Protected Side	EL -0.1 foot
Wave Forces	
(1 Percent Future Case Wave Forces - Bayou Dupre)	P1 = 382 pounds/foot
	P2 = 160 pounds/foot
	P3 = 406 pounds/foot

Unit Weights

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

Design Parameters

Structural Concrete:	F'c = 4 kilo pounds (kips)/square inch (ksi) (Minimum 28-day Concrete Strength)
Reinforcing Steel:	Fy = 60 ksi (Yield Strength of Reinforcing Steel)
Structural Steel:	Fy = 36 ksi (Yield Strength of Structural Steel)
Allowable Steel:	Stress = 0.5*FY + 33.3 percent overstress for extreme event used in design of structural steel hydraulic structures

Piles

Unfactored service loads, including the applicable allowable overstress factors, were used in any pile analysis. The USACE-provided CPGA, in most cases, was the foundation program used to determine the pile loads and deflection. The pile analysis program GROUP 7, by Ensoft, was also utilized. The maximum structural deflection at pile heads could not exceed 0.5 inch vertically and 0.75 inch horizontally, based on service loads reduced by the applicable allowable overstress. For deflection purposes, the maximum allowable overstress for high water checks was assumed to not exceed 33.3 percent.

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

Reinforced Concrete Design

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

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

A strength reduction factor for bending was determined to be 0.9.

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

Concrete Design Strength:	f′c = 3000 psi
Reinforcing Steel Strength:	fy = 60 ksi
Maximum Flexural Reinforcement:	0.25 ρ_b (Recommended)
	0.375 ρ_b (Permitted w/o special studies)
Minimum Flexural Reinforcement:	ACI Code
Minimum Thickness:	T-walls = 18″
Minimum Concrete Cover:	3″ minimum for sections, 12″ < t < 24″
	4″ minimum for sections, t ≥ 24″

Structural Steel Design

The minimum steel thickness considered for corrosion control was 5/16 inch. The Allowable Stress Design method was used for structural design. The Load Resistance Factor Design method was not used for any structural steel design. A factor equal to 5/6 of the AISC allowable stress was used for design.

Design Loads

Unit Weights

The following unit weight values were used in the structural design.

Item	Pounds per Cubic Foot
Water	64
Concrete	150
Steel	490

Unbalanced Loads

As determined by the geotechnical analysis, where applicable, the unbalanced load was taken as a lateral force acting at the bottom of the footing in the direction from the flood side to the protected side.

Impact Loads

- A design load of 100 kips was assumed for structures within LPV 144 and LPV 146 where barge impact could occur now or in the future.

- A minimum impact load distribution of 0.5 kip/foot was assumed for structures within LPV 147, LPV 148, and LPV 149.

- The minimum wall thickness for walls subject to impact loading was established to be 18 inches.

- Impact loading was applied 3 feet above the SWL but not greater than the top of the wall. For water to the top of wall, the impact loading was applied at the top of the wall.

- Barge impact loads on the walls were assumed to be distributed over a 5-foot width plus the width gained along a 45-degree angle.

- Foundation load distribution for barge impact was assumed to be distributed over the full width of the monolith.

- Barge impact load distribution at flood gates was assumed at the top girder only over a 5-foot width. A 33.3 percent overstress is permitted for this case.

- Wave load: USACE provided 1 percent Future Case Wave Forces: Wave Forces provided for the Bayou Dupre Floodgate and St. Mary Pump Station are summarized in the appendices and were used accordingly.

Wind Load

ASCE 7, 2002 was used to determine the maximum wind forces.

Uplift Force

Two potential loads cases were assumed as follows:

- Impervious sheet pile cut-off assuming 100 percent effective; and

- Permeable sheet pile cut-off assuming uniformly sloped pressure from flood side to protected side across the base of the slab.

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.

No utility owners were contacted, as per the USACE's 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 pipelines shall be raised above the levee design section and supported on a pipe bridge in accordance with the USACE-MVD Criteria for Crossings of Levees and Floodwalls. Each pipe bridge will have a clear span to ensure that no structural supports are within the levee design section and will have a minimum pipe bottom elevation of 31.0 feet to maintain appropriate vertical clearance for vehicle passage. Pipe bridges will be supported with HP 14x89 piles. Pipes being raised by pipe bridge shall be protected outside the pipe bridge to the limits of construction.

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

Utility pipelines shall be raised above the levee design section and supported on a pipe bridge in accordance with the USACE New Orleans District Criteria for Crossings of Levees and Floodwalls. Each pipe bridge will have a clear span to ensure that no structural supports are within the levee design section and will have a minimum pipe bottom elevation sufficient to maintain appropriate vertical clearance for vehicle passage. Pipe bridges were assumed to be supported with 14x89 H-piles. Pipes being raised by pipe bridge shall be protected outside the pipe bridge to the limits of construction.

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

Limitations

The geotechnical and structural analyses were preformed using soil boring and laboratory data, CPT results, as-built structure information, limited survey data, and 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 USACE before the analysis started. 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.