WESTBANK AND VICINITY
NEW ORLEANS, LOUISIANA
HURRICANE PROTECTION PROJECT

JEFFERSON PARISH, LOUISIANA

WBV 14a.2
HARVEY CANAL WEST BANK LEVEES – PHASE 2

ENGINEERING ALTERNATIVE REPORT

U. S. Army Corps
of Engineers
New Orleans District

Prepared by:

BCG Engineering & Consulting, Inc.
Excellence in Engineering

2701 Kingman Street
Metairie, Louisiana

In Association with:

Eustis Engineering Company, Inc.

C & C Technologies, Inc.

May 2008

WESTBANK AND VICINITY, NEW ORLEANS, LOUISIANA

HURRICANE PROTECTION PROJECT

WBV 14a.2
HARVEY CANAL WEST BANK LEVEES – PHASE 2

ENGINEERING ALTERNATIVE REPORT

Prepared for

U.S. Army Corps of Engineers

New Orleans District

New Orleans, Louisiana

May 2008

TABLE OF CONTENTS

TABLE OF CONTENTS
(continued)

INDEX TO DRAWINGS

WBV 14a.2

INDEX TO DRAWINGS
(continued)

WESTBANK AND VICINITY
HURRICANE PROTECTION PROJECT
ENGINEERING ALTERNATIVE REPORT

Harvey Canal West Bank Levees – Phase 2
WBV 14a.2

1.0 Introduction. This report presents a feasibility level study of alternative engineering solutions to raise the hurricane protection system for the Westwego to Harvey Canal Hurricane Protection in Jefferson Parish, Louisiana. The Westwego to Harvey Canal Hurricane Protection Project provides protection from Westwego, Louisiana to the Harvey Canal as described in the Design Memorandum (DM) No. 1 General Design Supplement No. 2, dated February 1990. The general area of study covered by this report is bounded by New Estelle Pump Station at Station 872+00 of the Harvey Canal Levee and Lapalco Boulevard near Station 1013+38, roughly paralleling the GIWW and Harvey Canals (see Plates). The report presents designs to prevent future floods and devastating effects of hurricanes such as Katrina and Rita. All elevations within this report reference NAVD 88 (2004.65).

2.0 Executive Summary. This engineering alternative report examines four different engineering solutions to raise the hurricane protections system for the Westwego to Harvey Canal Hurricane Protection Project to 2057 levels (elevation 14.0). This levee parallels the west bank of Harvey Canal from New Estelle Pump Station to Lapalco Boulevard and covers approximately 2.6 miles of levee. The alternatives include a conventional un-reinforced earthen levee, a geotextile reinforced levee, a floodwall, and an earthen levee reinforced with deep soil mixing columns. An additional alternative is included in the report which examines the design, right of way and cost needed to raise the existing levee/floodwall system to elevation 10.00 in support of the Sector Gate South Study. Feasibility level designs are presented for each alternative along with an estimate of the rights of way, relocations and cost associated with each plan. If the Sector Gate South Option is not used, the recommended alternative considering all factors is the Geotextile Reinforced Levee.

3.0 Purpose and Scope of Study.

3.1 Objectives. The purpose of this report is to present the results of an analysis of Hurricane Damage Reduction System alternatives and to recommend the most feasible alternative based on engineering investigations for the Harvey Canal West Bank Levees between New Estelle Pump Station and Lapalco Boulevard.

3.2 Level of Detail. This report presents feasibility level designs for four alternative methods of raising the hurricane protection to 100-year levels between New Estelle Pump Station and Lapalco Boulevard. Sufficient details are to be provided to allow for selection of a recommended plan followed by detailed design and preparation of plan and specifications. An additional feasibility level plan is presented for raising the existing protection to old authorized levels which also coincides with the proposed retention basin elevation required for the Sector Gate South Study.

4.0 Description of Existing Protection.

4.1 Type of Protection. The existing protection system includes both full earthen levee sections and two reaches with sheet pile I-walls near the northern end of the study area. The existing earthen levee varies in elevation from approximately 7.0 to 9.0 (see Plates). The existing levee has a 10 foot crown, roughly one on four side slopes and represents approximately 1.9 miles of the total 2.6 miles of study area. The first reach of existing I-wall is located between Station 907+85.12 and Station 913+49.26 (564 ft) and the second reach of existing I-wall is located between Station 936+74.61 and Station 971+75.50 (3,500 ft). The two reaches of I-wall were raised as part of Phase 1 contracts to elevation 9.0. Grouted rip rap scour protection and stability berms were also added to the existing I-wall sections as part of Phase 1 contracts in 2007. No other Phase 1 contracts have been accomplished in the reach covered by this report at the time of this report's printing. However, a Phase 1 levee lift is planned for Spring 2008.

The two completed sheet pile reaches of the Phase I work were only raised to elevation 9.0 due to the constraints of remaining within the existing rights of way and the need to obtain a factor of safety of 1.3. The Phase I levee data is as follows: Harvey Canal Levee Station 872+00 to Station 1009+78 COE Baseline (non-continuous) Design Grade Elevation 9.5, 1V on 3H side slopes and 10-foot wide crown.

4.2 Alignment. The alignment of the existing levee system parallels the west bank of Harvey Canal for the full study reach. A plan of the existing levee alignment and typical sections of the existing levee are both included in the plates.

4.3 Limits of Right of Way. The existing flood side right of way follows the waters edge for Harvey canal and the existing protected side right of way, roughly speaking, varies from 70 feet to 80 feet beyond the current levee centerline. Detailed descriptions of the existing rights of way are shown on the plates.

4.4 Level of Existing Protection. The level of the existing protection varies from approximately 7.0 to 9.0. The "2057" top of structure and levee elevation is 14.0. A profile of the existing levee protection is shown on the plates.

5.0 Description of Proposed Alternatives. Within the project study area four alternatives were considered for raising the protection to 100-year levels. These alternatives are listed below. An additional alternative referred to as "Alternative 4 – Levee and Capped Sheet Pile (Elevation 10.00)" is included which examines measures needed to raise the existing protection to the old authorized Sector Gate South retention basin top of wall/levee elevation. More detailed explanations of the alternatives are provided below:

5.1 Alternative 1 – Levee (Unreinforced) (2057 Elevation). This alternative consists of a conventional earthen levee without any type of reinforcement. The new enlarged levee parallels the existing levee with a centerline which is offset 150-feet towards the protected side of the existing levee/sheet pile centerline to provide a safe distance from Harvey Canal. See Appendix A for the geotechnical considerations for the selection of this offset. This alternative produced the largest foot print and requires the most right-of-way. Of the alternatives to be constructed to the 2057 elevation, this alternative is the second least expensive but requires the most borrow and the estimated construction duration when compared to the other alternatives is average.

Three lifts will be required to obtain the project grade of elevation 14.0. The first lift will be to elevation 12.0. The second lift will be required roughly 3 years later to elevation 13.0 and a third lift to elevation 14.0 (or maybe slightly above elevation 14 due to gain in strength will be needed roughly 20 years after the initial lift. Additional lift discussion is presented in Appendix A – Geotechnical Report.

5.2 Alternative 2 – Levee (Geotextile Reinforced) (2057 Elevation). The geotextile alternative consists of an earthen levee with geotextile reinforcement placed on the levee foundation at approximate elevation 0.5, which is approximately the natural ground surface elevation. The reinforcement will allow for a smaller levee section than a conventional levee. The alignment of the geotextile reinforced levee parallels the existing levee and has the centerline offset 105-feet towards the protected side from the existing levee/sheet pile centerline to provide a safe distance from Harvey Canal. See Appendix A for the geotechnical considerations for the selection of this offset. Three lifts similar to the conventional levee (discussed in above section) will be required to obtain the project grade of elevation 14.0. Of the alternatives to be constructed to the 2057 Elevation this alternative has the lowest estimated construction cost and the lowest estimated construction duration. However, it requires the second most borrow material and second most required right of way.

5.3 Alternative 3 – Floodwall (2057 Elevation). This alternative consists of a concrete T-wall founded on steel H-piles. There are two basic sections of floodwall. In those reaches of levee where the existing protection consists of an earthen levee (i.e. no I-wall) the new floodwall section is located on the protected side slope of the existing levee so that the existing levee can serve as a barrier against possible boat impact. This section covers approximately 1.9 miles of the total 2.6 miles of project reach. The second type floodwall section consists of a similar T-wall founded on steel H-piles located on the protected side of the existing sheet pile I-wall sections. After completion of the new T-wall the existing I-wall will be pulled and the existing small embankments on each side of the I-wall will be graded to elevation 5.0 as shown on the plates. A steel dolphin will be required for this type section since there is no levee to provide protection from boat impact. The steel dolphins will be located along the top bank of Harvey Canal and will be placed at approximately 30 foot intervals to provide protection for the T-wall.

For the reaches with existing sheet pile, the centerline of the stem of the new floodwall will need to be offset 21-feet from the existing sheet pile centerline to maintain a safe distance from Harvey Canal. For the reaches with existing levee, the centerline of the stem of the new floodwall will need to be offset 41-feet from the existing levee centerline to maintain a safe distance from Harvey Canal. See Appendix A for the geotechnical considerations for the selection of these offsets. Of the four 2057 Elevation alternatives, this alternative requires the least amount of right of way taking and borrow material. However, it has the second highest estimated construction cost and the estimated construction duration is also the second longest.

5.4 Alternative 4 – Levee and Capped Sheet Pile (Elevation 10.0). This alternative consists of keeping the existing alignment and raising the protection to elevation 10.0. In areas where there is an existing levee the protection will be raised as a conventional protected-side earthen levee enlargement and offset 35-feet to the protected side to maintain a safe distance from Harvey Canal. In areas where there is an existing I-wall the protection will be raised by adding a concrete cap to the I-wall and a protective side berm is to be added to provide the necessary safety factors to meet current criteria. This alternative is included to raise the protection to old authorized levels which also coincide with the proposed retention basin elevation required for the Sector Gate South Study.

5.5. Alternative 5 – Levee (Soil Mixing Columns) (2057 Elevation). The deep soil mixing alternative consists of an earthen levee section reinforced with deep soil mixing columns drilled under the foundation of the new levee. The deep soil columns will strengthen the foundation of the levee and allow for a smaller levee section. Additionally, this section should not require the multiple lifts required for Alternatives 1 and 2. The project grade of elevation 14.0 should be obtainable with the initial lift. The alignment of this alternative parallels the existing levee alignment and has a 60-foot centerline offset towards the protected side from the existing levee/sheet pile centerline to provide a safe distance from Harvey Canal. The location and spacing of the soil columns is shown on the plates. The estimated construction cost and construction duration for this alternative are the highest and longest respectively of the four 2057 Elevation alternatives considered. Contractors and construction crews with experience installing the soil mixing columns is limited within the project area which may limit the number of bids and could cause even higher project costs.

6.0 Design Criteria.

6.1 Assumptions. The designs included in this report comply with the Hurricane and Storm Damage Risk Reduction System Design Guidelines (HSDRRSDG) dated October 2007 and with applicable USACE Engineering Manuals published by the Office of the Chief of Engineers. All designs are also based on the Post Katrina Hurricane Protection T-wall Design Criteria and L-wall/Kicker Pile Design Criteria dated 20 April 2006 which have been incorporated into the HSDRSDG. Details on the design guidance used and assumptions associated with each of the technical disciplines of Hydraulics, Geotechnical, Structural, Mechanical and Electrical are covered in the following paragraphs of this section. In general the geotechnical and topographical data was examined for the study reach and the data considered to be the most conservative was used to prepare the designs for the entire study reach.

6.2 Field Data Collection. Site reconnaissance was made of the project area prior to preparation of designs. Also numerous on site inspections were made of the Phase 1 contracts for the two reaches of I-wall which are part of this project and were both subjects of Phase 1 contracts. Surveys taken consisted of cross sections taken at 600 foot intervals and extended from roughly the west top bank of Harvey Canal to 100 feet into the wood line on the protected side of the levee utilizing GPS equipment. Boring and testing data used are covered in detail in the soils report which is part of this document. On site investigations were made to identify possible relocations and these are covered in Paragraph 6.06. More details on the horizontal and vertical controls used are provided in the Survey Plan in Appendix C.

Surveys conform to the requirements stated in Section 9 of the latest version of the HSDRRSDG. This includes identifying a minimum of three (3) permanent benchmarks (new or existing) on design and construction drawings for all flood control projects (see Plates 1 and 2). The benchmarks were established relative to existing NAVD88 control established by the NGS, using either conventional differential leveling and/or the latest NGS-approved differential GPS network observations, with appropriate corrections to the local hydraulic design surface. Prior to and during actual construction stake out, these primary reference marks shall be verified externally and internally and field records of these survey verifications shall be permanently archived. A complete reevaluation of the vertical datum shall be conducted at each scheduled periodic inspection. The survey report and ITR have been completed and are shown in Appendices C and E respectively.

6.3 Hydraulic Design Criteria. The hydraulic design data was provided as described in the paragraphs below. Elevations shown are based on NAVD 88(2004.65) and assume a flood side levee slope of 1V:5H.

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

The Hurricane and Storm Damage Risk Reduction System (HSDRRS) includes features that provide protection from a hurricane event that would produce a 1 percent exceedance surge elevation and associated waves. Hydraulic modeling and analyses performed to calculate the surge elevation and wave characteristics are described in the October 9, 2007 report.

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

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

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

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

The gage at the New Estelle Pump Station is located within the contract reach and will be used for determining the tidal datum local mean sea level (LMSL) prior to construction. Additional temporary gages may be required depending on vertical accuracy requirements. The gage(s) can also be used to monitor future hydrologic conditions in the area. The datum of the gage(s) has been established to comply with criteria contained in the Vertical Control Requirements for Engineering, Design, Construction, and Operation of Flood Control, Shore Protection, Hurricane Protection, and Navigation Projects (Engineering Division Policy Memo #2).

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

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

6.3.1 <u>100-Year - 2057 Design Elevations</u>. The 2057 design elevations based on a flood side levee slope of 1V:5H are:

- Top of Structure and Levee: Elevation 14.0 and
- Stillwater Level: Elevation 11.0.

6.3.2 <u>Alternative 4 - Design Elevations</u>. The design elevations used for evaluation of Alternative 4 are as follows:

- Top of Structure and Levee: Elevation 10.0 and
- Stillwater Level: Elevation 9.0.

6.3.3 <u>Wave Loads</u>. A linear wave load of 2.34 kips per foot acting on the floodwall stem at elevation 8.84 was used in design. The basis for this load and the distribution of the wave load is shown on the figures included in the sample calculations in Appendix B.

6.4 <u>Geotechnical Design Criteria.</u>

6.4.1 <u>General</u>. The following represents the typical procedures used for the geotechnical design and analysis of levee embankments. The procedures stated herein, although considered typical, did not eliminate engineering judgment. The provisions of this paragraph supersede any conflicting requirements specified in Paragraphs 6.4.2 – 6.4.11.

Method of Planes was utilized for levee designs. Spencer's Method was utilized for L-wall stability. The combined methodology outlined by the following table and Paragraph 6.4.4, below, was used for T-walls.

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

EAR STABILITY ANALYSIS REQUIRED FACTORS OF SAFETY		
EMBANKMENT DESIGN		
	Method of Planes	Spencer's Method
Still Water Level	1.40	N/A
Top of Levee	N/A	N/A
Low Water (Flood Side)	1.35[1]	N/A
L-WALL DESIGN		
	Method of Planes	Spencer's Method
Still Water Level	N/A	1.50
Top of Wall	N/A	1.40
Low Water (Flood Side)	N/A	1.40
Still Water Level	N/A	1.50
Top of Wall	1.30	1.40
Low Water (Flood Side)	1.30	1.40

NOTES:

1. The flood side levee slope shall have a minimum slope of 1V on 4H. If stability analyses show a berm is required, a 1V on 4H slope shall be used from the crown of the levee to the start of the flood side stability berm.

2. The initial stability analyses for T-walls were conducted utilizing Spencer's Method. Designers attempted to achieve the required factors of safety, or as close to the required factors of safety as possible, by incorporating stability berms in the design. If unbalanced loads still exist, see Note 3. Sheet piles were designed for seepage only.

3. Final stability analyses, determination of unbalanced loads, and final floodwall design were accomplished by utilizing the Method of Planes (MOP). Designers followed the 20 April 2006 criteria with MOP and the structural software program CPGA. To replicate the new analysis, all steel piles (H-piles or pipe piles) were used and only PZ-22 was used as the sheet pile cutoff, extending the piling 10 feet past the critical failure plane.

6.4.2 Design References. The following Corps EM's, ETL's, EC's (all available on the internet at http://www.usace.army.mil/pubtypes.html):

USACE Publications:

- EM 1110-2-1902, Slope Stability, October 2003.
- EM 1110-2-1913, Design and Construction of Levees, April 2000.
- EM 1110-2-1901, Seepage Analysis and Control for Dams, April 1993.
- DIVR 1110-1-400, Soil Mechanic Data, December 1998,
- (https://inet.mvk.usace.army.mil/offices/im/private/cis/publications/mvdpubs.htm).
- ETL 1110-2-569, Design Guidance for Levee Underseepage, May 2005.

Computer Software:

- Slope Stability Program based on "MVD Method of Planes" (Method of Planes Program and plotting program is available by contacting the New Orleans District).
- Slope Stability Programs based on "Spencer's Procedure".

6.4.3 Levee Embankment Design.

A. Using centerline borings, toe borings, CPTs, applicable test results, and geologic profiles, determine stratification, shear strength, and unit weights of materials and separate alignment into soils and hydraulic reaches. Soil parameters, stratification, and geologic profiles to be used for design were submitted to the New Orleans District for assessment before design commenced.

B. Using cross sections of existing conditions, determine minimum composite sections for similar topography for each reach. Use of a typical section is not acceptable.

C. Settlement calculations were made to determine a levee lift construction schedule. The lift construction schedule was determined to maintain the levee to net design grade during the life of the project. It was assumed that the second lift will take place in three to five years.

D. Using the method of planes (stability with uplift program which were provided by the Government and design un-drained shear strengths, the factor of safety of the gross section was determined.

E. Typical assumed values (in lieu of test results) for un-drained soil parameters are shown in Tables 3 and 4.

Table 3 – Typical Values For Embankment Fill

Soil Type	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)
Compacted Clay (90%)	110	400	0
Compacted Clay from Bonnet Carré (from dry borrow pit placed on land)	115	600	0
Uncompacted Clay (from dry borrow pit placed on land)	100	200	0

Table 4 - Typical Values For Silts, Sands, and Riprap

Soil Type	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)
Silt	117	200	15
Silty Sand	122	0	30
Poorly Graded Sand	122	0	33
Riprap	132	0	40

Note: Weight of riprap may vary based on the filling of the riprap voids over time.

G. At pipeline crossings, the MOP allowable factor of safety is 1.5 with the flood side water at the still water level and a 1.4 with the flood side water to the top of the levee crown. This analysis was performed for the gross section for a distance of 150 feet on either side of the centerline of the pipeline.

6.4.4 Corps of Engineers Deep-Seated Stability Design Criteria. Deep-seated stability design criteria is included in the floodwall design criteria. The latest floodwall criteria (i.e. Spencer's method of analysis) were used to determine if there is an unbalanced load on the foundation. If there is an unbalanced load, utilize the LMVD Method of Planes analysis (traditional) to determine the anchor force on the foundation. The foundation analysis may utilize traditional pile group analysis programs (i.e. CPGA, ENSOFTs GROUP, etc.). If unbalanced loads exist, only steel piles were allowed. Sheet pile tips and sizes shall be as specified in Paragraph 5.4.1.

6.4.5 Pile Foundation (USACE Criteria). Design of the pile foundation is generally in accordance with Corps of Engineers Engineering Manual, EM 1110-2-2906. Theoretical pile capacities were calculated for both the undrained and drained soil conditions and the deepest tip penetration for the design load were used. New Orleans District limits the vertical stress in the subsurface foundation to 3,500 psf for determining both the undrained and drained pile capacity curves. Typical minimum factor-of-safety for pile capacity of the compression and tension piles is as follows for the loading conditions.

Loading Condition	Factor-of-Safety Without a Pile Test	Factor-of-Safety With a Pile Test
Q-case	3.0	2.0
S-case	1.5	1.5

The design for the piles included the type of material of the piles (steel vs. timber or concrete), method of pile installation (impact hammer vs. vibratory hammer), and any other pertinent data. It is common practice to reduce the frictional resistance of the granular soils against steel, to reduce frictional resistance of granular soil on piles in tension (K_t), and to reduce the load capacity of piles installed by vibratory hammer.

6.4.6 Floodwalls. Floodwall design criteria discussed in the HSDRSDG were used for this report, except as noted herein for unbalanced load determination.

6.4.7 Lateral Earth Pressure. At-rest soil pressure diagram were used behind the retaining walls. At-Rest soil coefficients commonly used by New Orleans District are 0.8 for a clay backfill and 0.5 for sand backfill when utilizing the general wedge method for computing earth pressures.

6.4.8 Bearing Capacity. Factor of safety of 3.0.

6.4.9 Dewatering. Design was such that groundwater drawdown outside the construction easement was not affected. (The dewatering system used during construction will be Contractor designed, however, a realistic design sufficient to develop a reasonable cost estimate was performed.)

6.4.10 Cantilever Retaining Walls and Braced Walls. Design guidance is included. Wall stability, slope stability and seepage requirements were determined as directed in the guidance. Wall stability and required penetration are determined by limit stresses with a Factor of Safety applied to the soil parameters. The F.S. is applied as follows:

- Cohesion Developed = Cohesion/Factor of Safety.
- Developed ϕ = Arctan (Tan $\phi_{Available}$/Factor of Safety).

The developed friction angle is used to determine the lateral earth pressure coefficients.

6.4.11 Seepage. It is the intent of these criteria to provide requirements that result in a safe design for seepage and uplift based on loading to the top of the barrier at any stage in the life of the project. In support of that, the following criteria are based on steady state seepage conditions in coarse grained soils. Due to their permeability it is unlikely that steady state conditions will develop in fine grained soils within the relatively short duration of a hurricane storm surge. However, open seepage entrances and non-continuity in blanket materials may allow steady state conditions to occur in coarser strata.

The following criteria are based on ETL 1110-2-569 except that factors of safety are presented instead of seepage gradients. Factors of safety are used because of the lighter weight blanket materials that may be encountered in the local region. If the criteria presented in the following table were not met, at the levee toe, seepage berms or remediation measures designed in accordance with EM 1110-2-1901, DIVR 1110-1-400 (for material properties where site specific information is not available), and ETL 1110-2-569. HPS seepage berms were designed for a 1.6 safety factor at the levee toe and 1.0 at the berm toe. Relief wells or other seepage control measures were designed to limit the factor of safety to 1.6 along the levee toe. The factors of safety for seepage are computed using effective stresses (defined by gradient) as:

$$FS_g = \frac{\gamma' \times z_t}{\gamma_w \times h_o} \quad \text{same as} \quad FS_g = \frac{I_{cr}}{I_e}$$

γ' = Effective Unit Weight Soil (or Average Effective Unit Weight of Soil).
γ_w = Unit Weight of Water.
z_t = Landside Blanket Thickness.
h_o = Excess Head (Above Hydrostatic) at Toe.
I_{cr} = Critical Exit Gradient.
I_e = Exit Gradient.

SEEPAGE AND UPLIFT DESIGN CRITERIA		
	Minimum Factor of Safety at Levee or Wall Toe(1)	
Levee/Wall Application	**Authorized Water Surface Elevation (AWSE)**	**Top of Protection(2)**
Riverine	1.6	1.3
Coastal (Top of Protection < 5 ft above AWSE)	1.6	1.3
Coastal (Top of Protection > 5 ft above AWSE)	1.6	1.2

NOTES:

1. Minimum factors of safety at the levee toe are based on steady state seepage conditions. Loading in excess of the "Top of Protection" is considered sufficiently short term that steady state conditions do not fully develop and safety is adequately addressed by the steady state factors of safety.

2. The top of protection includes increases above the authorized water surface elevation to account for run-up and/or grade elevations for other reasons minus overbuild for primary consolidation.

6.5 Structural Design Criteria.

6.5.1 General. The structural design for the hurricane protection features complies with standard engineering practice and criteria set forth in Engineering Manuals, Regulations and Technical Letters for civil works construction published by the Office of Chief of Engineers. Note, however, that the design criteria for floodwalls included in the Post Katrina Hurricane Protection T-wall Design Criteria and L-wall/Kicker Pile Design Criteria (20 April 2006) supersedes all references below.

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

6.5.2 References.

Applicable COE Publications:

- EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures, June 1992 (including change August 1, 2003).
- EM 1110-2-2105, Design of Hydraulic Steel Structures (including change May 1, 1994).
- EM 1110-2-2502, Retaining and Flood Walls, September 1989.

- EM 1110-2-2906, Design of Pile Foundations, January 1991.
- EM 1110-2-2503, Design of Sheet Pile Cellular Structures Cofferdams and Retaining Structures, September 1989.
- EM 1110.2.2504, Design of Sheet Pile Walls, March 1994.
- EM 1110-2-1913, Design and Construction of Levees, April 2000.
- EM 1110-2-1901, Seepage Analysis and Control for Dams, April 1993.
- EM 1110-2-2100, Stability Analysis of Concrete Hydraulic Structures, December 2005.
- DIVR 1110-1-400, Soil Mechanic Data, December 1998.

Applicable Technical Publications:

- American Concrete Institute, Building Code and Commentary, ACI 318-99.
- American Institute of Steel Construction, Manual of Steel Construction (9^{th} Ed.).
- American Welding Society, AWS D1.1 (2006).
- American Welding Society, AWS D1.5 (2002).
- ASCE 7, Minimum Design Loads for Buildings and Other Structures.

Applicable Computer Software:

- CE Pile Group Analysis Program, "CPGA".
- CE Structural Analysis Program, "C-Frame".
- CE Strength Analysis of Concrete Structural Elements, "CGSI".
- CE Sheet Pile Wall Design/Analysis Program, "CWALSHT".
- Structural Analysis and Design Software, "STAAD".
- Slope Stability Program Based on "MVD Method of Planes".
- Additional Approved COE Programs.

6.5.3 Unit Weights and Earth Pressure Coefficients.

Material	Unit Weight (lbs./cubic feet)
Water	64
Concrete	150
Steel	490
Granular Fill	*120
Clay Fill	*110

*Note: The submerged unit wt. equals the moist weight shown above minus 62 lbs./cu. ft.

Material	At Rest (Ko)	Active (Ka)
Granular	0.55	0.33
Clay	0.80	0.44 (long term) - 1.0 (short term)

6.5.4 Reinforced Concrete Criteria. The design of reinforced concrete structures is in accordance with EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures, June 1992 (including change August 1, 2003). A single load factor of 1.7 was applied to moments and shears and an additional hydraulic load factor of 1.3 was applied. For convenient reference pertinent design data are tabulated below:

fi (28-day strength)	3,000 psi
fy (Grade 60 steel)	60 ksi
Maximum Flexural Reinforcement	0.25 Pb
Minimum Flexural Reinforcement	ACI Code
Temperature Reinforcement (1/2 each face)	.0028Ag
Clear Cover (less than 12 inches thickness)	2 inch minimum
Clear Cover (greater than 12 inches and less than 24 inches thickness)	3 inch minimum.
Clear Cover (equal to or greater than 24 inches)	4 inch minimum
0 Bending	0.90
0 Shear	0.85

6.5.5 Steel Design. The allowable stress design (ASD) was used for steel design. In general the allowable stresses were limited to 5/6 of those allowed by AISC. The minimum thickness of steel was 5/16 inch to allow for corrosion control. Weld designs for fracture critical members were based on AWS D 1.5 code.

6.5.6 Steel Sheet Pile Design Criteria. The steel sheet pile was designed for allowable stress in accordance with EM 1110-2-2504.

The allowable stress for bending, Fb, was limited to 0.50 Fy. The allowable stress for shear, Fv, was limited to 0.33 Fy. When investigating conditions with unbalanced load the allowable stresses were Fb = 0.75 Fy and Fv = 0.45 Fy.

6.5.7 Bearing Piles. Piles were designed in accordance with EM 110-2-2906. Pile capacities were developed as described in, the "Geotechnical" section of this report. The maximum deflections allowed were as follows:

Case	Vertical (inches)	Horizontal (inches)
Normal (0 percent)	0.50	0.75
Overstress (16 2/3)	0.583	0.875
Overstress (33 1/3)	0.67	1.0

6.5.8 General T-wall, L-wall Criteria. I-walls were not considered for this study reach because the levees are located along a navigable waterway and are subject to boat impact. L-walls were not considered where wall heights exceeded eight (8) feet or where unbalanced loads were present. T-walls were considered as acceptable for all height requirements and were designed for boat impact or unbalanced load where applicable.

6.5.9 Load Cases Considered. For the structural analysis of the floodwalls the following load cases were considered.

Case 1 - Construction Condition with wind, surcharge, and drag considered in applicable directions. 16-2/3 percent overstress was permitted.

Case 2 - Stillwater at El 11.0 plus wave load. 33-1/3 percent overstress permitted. (Note: The Stillwater elevation was not available at the start of this EAR and the value of 11.6 was utilized).

Case 3 - Stillwater at El 11.0 plus unbalanced load applied at the base. No overstress was permitted. (Note: The Stillwater elevation was not available at the start of this EAR and the value of 11.6 was utilized).

Case 4 - Stillwater plus wave plus Boat Impact. 50 percent overstress permitted.

Case 5 -.Water to top of wall. 33-1/3 percent overstress permitted.

Case 6 -.Water to top of wall plus unbalanced load applied at base. 50 percent overstress permitted.

Case 7 - Water to top of wall plus unbalanced load applied at base plus Boat Impact. 67 percent overstress permitted.

Two T-wall situations were analyzed. One situation was with T-wall sitting on top of embankment with footing backfill to El 5.0. The other situation was with T-wall sitting on the protected side of embankment with footing backfill to El. 7.0. For both situations, 100 Kip Boat Impact was considered and the impact load was distributed uniformly over the full width of the 49.5-ft monolith. Although, the impact loading contributed to some of the largest pile loads, they turned out to be non-critical when the overstress correction was made. The 2-ft difference in backfill height for the two schemes had negligible influence on pile load results. Therefore, the same pile layout works equally well for both T-wall situations.

Summary of T-wall pile design loads (kips) (with overstress corrections incorporated):

NOTE: Case 3 is the Critical Loading for Both Reaches.

Un-protected Reach *			**Protected Reach**	
Compression	Tension		Compression	Tension
32.1	-2.00	Case 1	28.4	0.00
44.3	-11.35	Case 2	46.4	-10.50
123.0	-77.4	Case 3	122.7	-75.20
54.3	-21.13	Case 4	56.2	-20.4
39.6	-7.14	Case 5	41.7	-6.30
104.7	-68.13	Case 6	109.2	-71.0
101.9	-58.6	Case 7	105.4	-60.6

*Levee not available to protect floodwall from boat impact.

6.6 Mechanical and Electrical Design Criteria.

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

6.6.1 Mechanical. Any mechanical features required were designed in accordance with the applicable portions of USAEC engineering manuals for civil works construction referenced below and applicable portions of industry codes. If a conflict exist between the codes the more stringent criteria was applied.

- EM 1110-2-1424, Lubrication and Hydraulic Fluids (February 1999, July 2006).
- EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations (March 1994, August 1994, November 1999).
- NFPA.

6.6.2 Electrical. Any electrical features required were designed in accordance with the applicable portions of code and USACE engineering manuals for civil works construction referenced below. Cathodic protection systems, if required, were designed by a NACE certified corrosion specialists.

- EM 1110-2-2704, Cathodic Protection Systems for Civil Works Structures, January 1, 1999.
- EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations, March 1994, August 1994, November 1999.
- NFPA 70.
- National Electric Code, 2005.

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

6.7 Relocations (Existing Utilities). There is one utility located within the project limits. There is a transmission line tower which crosses the levee at Station 958+69± B/L. More specific information about the utility located in the project area is summarized below.

EXISTING UTILITIES				
Description	**B/L Station**	**Owner**	**Address**	**Phone No.**
Transmission Line Tower	958+69±	Entergy	P.O. Box 61000 New Orleans, LA 70161	504-365-3625

6.8 Borrow Requirements. An estimate of the borrow requirements for each alternative is shown in the shown in the table below. Borrow was assumed to be obtained within 10-miles of the job site.

Alternative	Estimated Borrow (CY)
Alternative 1 – Levee (Un-reinforced)	1,390,607 Cubic Yards
Alternative 2 – Levee (Geotextile Reinforced)	1,015,830 Cubic Yards
Alternative 3 – Floodwall	150,528 Cubic Yards
Alternative 4 – Levee and Capped Sheet Pile	257,584 Cubic Yards
Alternative 5 – Levee (Soil Mixing Columns)	307,158 Cubic Yards

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

The Armoring Team is tasked to provide research and planning for the use of armoring against erosion and scour on the protected side of selected critical portions of levees and floodwalls in the HSDRRS. These critical areas include: transition points (where levees and floodwalls transition into any hardened feature such as other levees, floodwalls, pump stations, etc.), utility pipeline crossings, floodwall protected side slopes, and earthen levees that are exposed to wave and surge overtopping during a 500-year surge elevation. The Armoring Team will be guiding the design PDT in this process by providing an Armoring Manual for design guidance and criteria. This manual will be the basis for decisions on what should be armored and how armoring should take place.

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

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

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

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

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

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

7.0 <u>Real Estate</u>. The existing right of way for this study reach is shown on the plates which accompany this report. The right of way required for each of the alternatives (including construction access) is shown on the plates with ties to the existing baseline. Permanent rights of way, easements, underground servitudes, and any other type rights of way required such as temporary access easements are differentiated by line type. The location of P.I.'s on all right of way lines are also identified by state plane coordinates. The temporary work area easement figures do not include construction access and are limited to staging areas at both ends of the project and near the Entergy transmission tower. These staging areas will aid in the utilization of multiple crews implementing various items of the construction and the same value of 4.71 acres is proposed for each alternative. The acreage of new perpetual flood protection servitude required for each of the alternatives is summarized in the following table:

<u>New Perpetual Flood Protection Servitude.</u>

Alternative	Permanent Flood Protection Servitude	Temporary Access Easement & Staging Areas
Alternative 1 – Levee (Un-reinforced)	105.52 Acres	4.71 Acres
Alternative 2 – Levee (Geotextile Reinforced)	73.90 Acres	4.71 Acres
Alternative 3 – Floodwall	10.36 Acres	4.71 Acres
Alternative 4 – Levee & Capped Sheet Pile	27.08 Acres	4.71 Acres
Alternative 5 – Levee (Soil Mixing Columns)	26.68 Acres	4.71 Acres

Vegetation Free Zone for Operations and Maintenance:

The all earthen levee alternative, which has the maximum footprint, has adequate clearance to provide a 15' vegetation free zone on both the protected and flood sides and will thus be in compliance with current guidance and policy. Levee designs will include tree removal, sloping, grading, placing fill, etc. necessary to achieve a maintainable 15-ft vegetation free zone from the toe of the levee on both the flood and protected sides. All plans and specifications (P&S) for HSDRRS levee contracts will ensure standards are met with respect to maintenance corridors

8.0 <u>Relocations Required</u>. There is one utility, consisting of a high voltage transmission line, located within the project limit as described in Paragraph 6.0 above. The proposed improvements of Alternatives 1, 2 and 5 conflict with the Entergy transmission tower and for these three alternatives both a "Do Not Disturb" and a "Relocation" option have been provided.

The "Do Not Disturb Option" requires a floodwall aligned around the transmission tower in the shape of a "U" and carries a construction cost of roughly \$10,256,010 million. The construction cost to move the transmission tower such that the full protection improvement can be constructed was provided by COE as roughly \$3 million. The official cost that Entergy would require for the relocation of the tower has not been determined as an official inquiry has not been solicited. Relocation of the transmission tower may have impacts on adjacent towers or other impacts that will not be fully known until an official inquiry is made and an impact analysis is conducted by the utility company. These items are beyond the BCG's scope of work for this EAR. There is a canal adjacent to the Entergy easement that might require backfilling (these costs are not included in the cost estimate); however, based on the differential cost between the two options, the relocation of the transmission tower is assumed as the better solution and that figure was used in the cost analysis.

9.0 Cost Engineering.

9.1 Quantities and Cost Per Alternative. See the following pages.

9.1.1 Alternative 1 – Levee (Un-reinforced) (2057 Elevation).

Item	Quantity	Unit	Unit Cost	Cost
Mob/Demob (5 %)	Lump Sum	Lump Sum	Lump Sum	$2,693,906
Clearing & Grubbing	149	Acres	$2,500.00	$372,500
Excavation	133,000	Cubic Yards	$8.00	$1,064,000
Berm Embankment	578,870	Cubic Yards	$31.00	$17,944,970
Levee Embankment	811,737	Cubic Yards	$31.00	$25,163,847
Geotextile Fabric	0	Square Yards	$3.00	-0-
T-wall Tie-in to Sector Gate	Lump Sum	Lump Sum	Lump Sum	$4,234, 000
Concrete Cap	0	Cubic Yards	$800.00	-0-
Soil Mixing Columns	0	Linear Feet	$80.00	-0-
Pull Existing Sheet Pile	3,944	Linear Feet	$200.00	$788,800
Transmission Tower Relocation	Lump Sum	Lump Sum	Lump Sum	$3,000,000
Dolphins	9	Each	$105,000.00	$945,000
Fertilizing, Seeding & Mulching	146	Acres	$2,500.00	$365,000
Subtotal				$56,572,023
Contingencies at 25%				$14,143,006
Initial Construction Cost Total				**$70,715,029**
Lift Construction				
Initial Lift	Included in Above Estimate			
Second Lift				
Mob/Demob (5%)	Lump Sum	Lump Sum	Lump Sum	$372,959
Clearing & Grubbing	122.6	Acres	$500.00	$61,300
Embankment	228,754	Cubic Yards	$31.00	$7,091,374
Fertilizing, Seeding & Mulching	122.6	Acres	$2,500	$306,500
Subtotal				$7,832,133
25% Contingency				$1,958,033
Total Cost Second Lift				**$9,790,166**
Third Lift				
Mob/Demob (5%)	Lump Sum	Lump Sum	Lump Sum	$550,243
Clearing & Grubbing	122.6	Acres	$500.00	$61,300
Embankment	343,131	Cubic Yards	$31.00	$10,637,061
Fertilizing, Seeding & Mulching	122.6	Acres	$2,500	$306,500
Subtotal				$11,555,104
25% Contingency				$2,888,776
Total Cost Third Lift				**$14,443,880**
O&M Costs	2.63	Miles	$9,000	$23,600/Yr.

9.1.2 Alternative 2 – Levee (Geotextile Reinforced) (2057 Elevation).

Item	Quantity	Unit	Unit Cost	Cost
Mob/Demob (5 %)	Lump Sum	Lump Sum	Lump Sum	$1,948,699
Clearing & Grubbing	119	Acres	$2,500.00	$297,500
Excavation	133,000	Cubic Yards	$8.00	$1,064,000
Berm Embankment	282,630	Cubic Yards	$31.00	$8,761,530
Levee Embankment	733,200	Cubic Yards	$31.00	$22,729,200
Geotextile Fabric	249,650	Square Yards	$3.00	$748,950
T-wall Tie in to Sector Gate	Lump Sum	Lump Sum	Lump Sum	$3,329,000
Concrete Cap	0	Cubic Yards	$800.00	-0-
Soil Mixing Columns	0	Linear Feet	$80.00	-0-
Pull Existing Sheet Pile	3944	Linear Feet	$200.00	$788,800
Transmission Tower Relocation	Lump Sum	Lump Sum	Lump Sum	
Dolphins	9	Each	$105,000.00	$945,000
Fertilizing, Seeding & Mulching	124	Acres	$2,500.00	$310,000
Subtotal				$40,922,679
Contingencies at 25%				$10,230,670
Initial Construction Cost Total				**$51,153,349**
Lift Construction				
Initial Lift	Included in Above Estimate			
Second Lift				
Mob/Demob (5%)	Lump Sum	Lump Sum	Lump Sum	$318,784
Clearing & Grubbing	92.6	Acres	$500.00	$46,300
Embankment	196,706	Cubic Yards	$31.00	$6,097,886
Fertilizing, Seeding & Mulching	92.6	Acres	$2,500	$231,500
Subtotal				$6,694,470
25% Contingency				$1,673,618
Total Cost Second Lift				**$8,368,088**
Third Lift				
Mob/Demob (5%)	Lump Sum	Lump Sum	Lump Sum	$471,231
Clearing & Grubbing	92.6	Acres	$500.00	$46,300
Embankment	295,059	Cubic Yards	$31.00	$9,146,829
Fertilizing, Seeding & Mulching	92.6	Acres	$2,500	$231,500
Subtotal				$9,895,860
25% Contingency				$2,473,965
Total Cost Third Lift				**$12,369,825**
O&M Costs	2.63	Miles	$9,000	$23,600/Yr.

9.1.3 Alternative 3 – Floodwall (2057 Elevation).

Item	Quantity	Unit	Unit Cost	Cost
Mob/Demob (5 %)	Lump Sum	Lump Sum	Lump Sum	$10,223,449
Clearing & Grubbing	72	Acres	$2,500.00	$180,000
Excavation	13,550	Cubic Yards	$8.00	$108,400
Berm Embankment	150,528	Cubic Yards	$31.00	$4,666,368
Levee Embankment	0	Cubic Yards	$31.00	-0-
Geotextile Fabric	0	Sq. Yards	$3.00	-0-
Concrete Stem	10,350	Cubic Yards	$800.00	$8,280,000
Concrete Base	18,354	Cubic Yards	$600.00	11,012,400
Concrete Scour Slab	2,208	Cubic Yards	$600.00	$1,324,800
Stabilization Slab	3,067	Cubic Yards	$500.00	$1,533,500
Sheet Pile (PZ-22)	1,076,400	Square Feet	$40.00	$43,056,000
Piles (HP 14x73)	1,092,900	Linear Feet	$107.00	$116,940,300
Waterstop	6,400	Linear Feet	$6.00	$38,400
Joint Material	7,728	Square Feet	10.00	$77,280
T-wall Tie in to Sector Gate	Lump Sum	Lump Sum	Lump Sum	$952,740
Concrete Cap	0	Cubic Yards	$800.00	-0-
Soil Mixing Columns	0	Linear Feet	$80.00	-0-
Pull Existing Sheet Pile	3,944	Linear Feet	$200.00	$788,800
Transmission Tower Relocation	Lump Sum	Lump Sum	Lump Sum	-0-
Dolphins	146	Each	$105,000.00	$15,330,000
Fertilizing, Seeding & Mulching	72	Acres	$2,500.00	$180,000
Subtotal				$214,692,437
Contingencies at 25%				$53,673,109
Initial Construction Cost Total				**$268,365,547**
Lift Construction (Design Grade in Initial Lift)	Lump Sum	Lump Sum	Lump Sum	$0
O&M Costs	Lump Sum	Lump Sum	Lump Sum	$10,000/Yr.

9.1.4 Alternative 4 – Levee and Capped Sheet Pile (Elevation 10.0).

Item	Quantity	Unit	Unit Cost	Cost
Mob/Demob (5 %)	Lump Sum	Lump Sum	Lump Sum	$499,485
Clearing & Grubbing	72	Acres	$2,500.00	$180,000
Excavation	58,550	Cubic Yards	$8.00	$468,400
Berm Embankment	114,991	Cubic Yards	$31.00	$3,564,721
Levee Embankment	142,593	Cubic Yards	$31.00	$4,420,383
Geotextile Fabric	0	Square Yards	$3.00	-0-
Concrete Stem	0	Cubic Yards	$800.00	-0-
Concrete Base	0	Cubic Yards	$600.00	-0-
Concrete Scour Slab	0	Cubic Yards	$600.00	-0-
Stabilization Slab	0	Cubic Yards	$500.00	-0-
Sheet Pile (PZ-22)	0	Square Feet	$40.00	-0-
Piles (HP 14x73)	0	Linear Feet	$107.00	-0-
Waterstop	Lump Sum	Lump Sum	Lump Sum	$5,000
Concrete Cap	1,464	Cubic Yards	$800.00	$1,171,200
Soil Mixing Columns	0	Linear Feet	$80.00	-0-
Pull Existing Sheet Pile	0	Linear Feet	$200.00	-0-
Transmission Tower Relocation	Lump Sum	Lump Sum	Lump Sum	-0-
Dolphins	0	Each	$105,000.00	-0-
Fertilizing, Seeding & Mulching	72	Acres	$2,500.00	$180,000
Subtotal				$10,489,189
Contingencies at 25%				$2,622,297
Initial Construction Cost Total				**$13,111,486**
Lift Construction (Design Grade in Initial Lift)	Lump Sum	Lump Sum	Lump Sum	$0
O&M Costs	Lump Sum	Lump Sum	Lump Sum	$5,000/Yr.

9.1.5 Alternative 5 – Levee (Soil Mixing Columns) (2057 Elevation).

Item	Quantity	Unit	Unit Cost	Cost
Mob/Demob (5 %)	Lump Sum	Lump Sum	Lump Sum	$14,287,280
Clearing & Grubbing	72	Acres	$2,500.00	$180,000
Excavation	133,000	Cubic Yards	$8.00	$1,064,000
Berm Embankment	0	Cubic Yards	$31.00	-0-
Levee Embankment	307,158	Cubic Yards	$31.00	$9,521,898
Geotextile Fabric	0	Square Yards	$3.00	-0-
Concrete Stem	0	Cubic Yards	$800.00	-0-
Concrete Base	0	Cubic Yards	$600.00	-0-
Concrete Scour Slab	0	Cubic Yards	$600.00	-0-
Stabilization Slab	0	Cubic Yards	$500.00	-0-
Sheet Pile (PZ-22)	0	Square Feet	$40.00	-0-
Piles (HP 14x73)	0	Linear Feet	$107.00	-0-
Waterstop	0	Linear Feet	$6.00	-0-
Joint Material	0	Joint Material	10.00	-0-
T-wall Tie in to Sector Gate	Lump Sum	Lump Sum	Lump Sum	$2,423,500
Concrete Cap	0	Cubic Yards	$800.00	-0-
Soil Mixing Columns	3,357,500	Linear Feet	$80.00	$268,600,000
Pull Existing Sheet Pile	3944	Linear Feet	$200.00	$788,800
Transmission Tower Relocation	Lump Sum	Lump Sum	Lump Sum	$3,000,000
Dolphins	9	Each	$105,000.0 0	$945,000
Fertilizing, Seeding & Mulching	72	Acres	$2,500.00	$180,000
Subtotal				$300,032,878
Contingencies at 25%				$75,008,219
Initial Construction Cost Total				**$376,041,097**
Lift Construction (Design Grade in Initial Lift)	Lump Sum	Lump Sum	Lump Sum	$0
O&M Costs	2.63	Miles	$9,000	$23,600/Yr.

9.1.6 Floodwall at Transmission Tower Option.

Item	Quantity	Unit	Unit Cost	Cost
Mob/Demob (5%)	Lump Sum	Lump Sum	Lump Sum	
Clearing & Grubbing	0.5	Acres	$2,500	$1,250
Excavation	3,500	Cubic Yard	$8.00	$28,000
Berm Embankment	2,950	Cubic Yard	$31,00	$91,450
Concrete	1,144	Cubic Yard	$700	%800,800
Sheet Pile (PZ-22)	40,150	Square Feet	$40.00	$1,606,000
Piles (HP 14x73)	40, 656	Linear Feet	$107,00	$4,350,192
Stabilization Slab	122	Cubic Yards	$500.00	$61,000
Waterstop	506	Linear Feet	$6.00	$3,036
Joint Material	308	Square Feet	$10.00	$3,080
Dolphins	12	Each	$105,000.00	$1,260,000
Subtotal				$8,204,808
Contingencies at 25%				$2,051,202
Total				$10,256,010

9.1.6.1 T-wall Tie-in to Sector Gate.

Item	Quantity	Unit	Unit Cost	Cost
Alternative 1				
Concrete Stem	219.8	Cubic Yards	$800.00	$175,840
Concrete Base	395.1	Cubic Yards	$600.00	$237,060
Concrete Scour Slab	44.6	Cubic Yards	$600.00	$26,760
Stabilization Slab	44.6	Cubic Yards	$500.00	$22,300
Sheet Pile (PZ-22)	22,877	Square Feet	$40.00	$915,080
Piles (HP 14x73)	26,145	Linear Feet	$107.00	$2,797,515
Joint Material & Water Stop	Lump Sum	Lump Sum	Lump Sum	$59,500
Total T-wall Tie-in to Sector Gate Alternative 1				**$4,234,000**

Item	Quantity	Unit	Unit Cost	Cost
Alternative 2				
Concrete Stem	172.8	Cubic Yards	$800.00	$138,240
Concrete Base	310.6	Cubic Yards	$600.00	$186,360
Concrete Scour Slab	35.0	Cubic Yards	$600.00	$21,000
Stabilization Slab	35.0	Cubic Yards	$500.00	$17,500
Sheet Pile (PZ-22)	17,980	Square Feet	$40.00	$719,200
Piles (HP 14x73)	20,548	Linear Feet	$107.00	$2,198,636
Joint Material & Water Stop	Lump Sum	Lump Sum	Lump Sum	48,000
Total T-wall Tie-in to Sector Gate Alternative 2				**$3,329,000**

Item	Quantity	Unit	Unit Cost	Cost
Alternative 3				
Concrete Stem	49.3	Cubic Yards	$800.00	$39,440
Concrete Base	88.6	Cubic Yards	$600.00	$55,160
Concrete Scour Slab	10.0	Cubic Yards	$600.00	$6,000
Stabilization Slab	10.0	Cubic Yards	$500.00	$5,000
Sheet Pile (PZ-22)	5,128	Square Feet	$40.00	$205,120
Piles (HP 14x73)	5,860	Linear Feet	$107.00	$627,020
Joint Material & Water Stop	Lump Sum	Lump Sum	Lump Sum	$15,000
Total T-wall Tie-in to Sector Gate Alternative 3				**$952,740**

Item	Quantity	Unit	Unit Cost	Cost
Alternative 5				
Concrete Stem	125.7	Cubic Yards	$800.00	$100,560
Concrete Base	226.0	Cubic Yards	$600.00	$135,600
Concrete Scour Slab	25.5	Cubic Yards	$600.00	$15,300
Stabilization Slab	25.5	Cubic Yards	$500.00	$12,750
Sheet Pile (PZ-22)	13,082	Square Feet	$40.00	$523,280
Piles (HP 14x73)	14,951	Linear Feet	$107.00	$1,599,760
Joint Material & Water Stop	Lump Sum	Lump Sum	Lump Sum	$36,300
Total T-wall Tie-in to Sector Gate Alternative 5				**$2,423,500**

9.1.7 Relocation of Transmission Tower Option. A lump sum price of $3 million was provided by COE. For Alternatives 1, 2 and 5 this Relocation Option was considered to be the better solution and included in the Cost Estimates.

9.1.8 Dolphin. Dolphin unit price of $105,000.00 per each is based on Lump Sum Price provided by COE in 65% review comments.

9.2 Level of Contingencies. The quantities are based on feasibility level designs. A twenty-five percent contingency is added to the cost estimate for each alternative.

9.3 Construction Durations. Levee/Berm Fill - The production rate for placing levee and berm fill utilized was 1,100 cubic yards per day based on guidance provided by the COE in the 65% comments.

Sheet Pile Placement - A review was made of the construction records for the contract which placed the sheet piles in Reach 3 and the first portion of sheet piles in Reach 4. This contract involved degrading the levee and placing 907 feet of PZ-27 piling with a top elevation of +10.0 and a bottom tip elevation of -35.0 feet (40,500 sq. ft.). The contract duration was 110 days. The actual driving time for sheet pile was approximately 45 days. The performance time for all work including mobilization, earthwork, turfing and driving was approximately

12 feet per day (from notice to proceed). The driving rate was 20 linear feet of wall per day. Based on this information, a driving rate of 900 square feet per day was utilized.

Bearing Pile Placement – The bearing piles for this project will be HP 14x73 steel piles driven approximately 120 feet. Using a production rate of 50 feet per hour we estimate 5 to 7 piles per day.

Concrete Placement – We estimate an average placement rate of 7 to 10 cubic yards of concrete per hour. It is assumed that concrete will be brought in by trucks capable of hauling 8 to 9 cubic yards of concrete.

Soil Column Mixing Production – Based on information obtained from Hayward Baker a production rate for drilling 31.5 inch diameter columns 68 ft. deep will be roughly 20 to 25 shafts per drill rig per day. This will require more than one drill rig to provide for a reasonable completion time for the soil column mixing alternatives.

When computing the construction durations of the alternatives, consideration was made of the availability of three access points to the work (each end of the project and near the project middle along the Entergy easement. Staging areas are proposed at each of these locations for multiple crew utilization. Construction durations were shortened by the assumption that multiple crews could be utilized for the various items of work with longer implementation times.

Construction Schedules for Alternatives.

Using the information above a performance time for each alternative are summarized below:

Alternative	Estimated Construction Time
1. Un-reinforced Levee	31 Months
2. Geotextile Levee	18 Months*
3. Floodwall	35 Months
4. Existing Alignment Elevation 10.0	12 Months
5. Soil Column Mixing Levee	39 Months

* Since the Geotextile Reinforced Levee Alternative is the recommended plan, means of expediting the construction were investigated. The performance time shown in the table is based on multiple construction crews working concurrently utilizing the three site access easements shown on the plans.

9.4 Construction Lift Schedules. The geotechnical recommendation for achieving the project grade of 14.0 for the earthen levee configurations is for the initial lift to be constructed to elevation 12.0. The second lift, three years after the initial lift should be constructed to elevation 13.0 and should require placement of approximately 2 foot of fill due to settlement. The third lift would occur after approximately 20 years and would be to the Project Grade at elevation 14.0. This third lift would also require placement of approximately 2 feet of fill due to settlement. Either a forth lift would be required after approximately 45 years to maintain el 14.0, or a gain-in strength would need to be considered such that the third lift could exceed elevation 14.0 (by placing more than 2 feet of fill). This second option was utilized in the cost estimates and the fill placement assumed for the third lift was increased to 3 feet. The preliminary estimates of lift construction presented here are provided as Tab 12 of Appendix A – Geotechnical Report.

Per the geotechnical recommendations, lift construction is required for the levees and berms of the conventional levee (Alternative 1) and the geotextile reinforced levee (Alternative 2). The geotextile reinforced levee was considered to have the same lift height requirements as the conventional un-reinforced levee in the lift quantity computations. T he strength gain afforded by the soil mixing columns of Alternative 5 should allow the levee to be constructed to the project grade elevation of 14.0 in a single lift.

10.0 Quality Implementation.

10.1 Quality Control Plan. A copy of the approved Quality Control Plan for this project is provided as Appendix D.

10.2 Technical Review Documentation. Copies of the review documentation, comments, comment resolutions and ITR Certification is provided as Appendix E. Also provided in Appendix E are copies of the review comments, evaluation responses and back check comments for both the 95% EAR QA Review and the 65% EAR QA Review.

11.0 Recommendations. We recommend that Alternative 2 – Levee (Geotextile Reinforced) (2057 Elevation) be implemented if the Sector Gate South option is not used since it has the lowest construction cost.

12.0 Operations and Maintenance Requirements. Operations and maintenance requirements for all the levee alternatives will consist of grass cutting of all levees and berms and maintenance of the turf. The alternative involving a floodwall will require maintenance of the floodwall joints with occasional repair of joint material. Maintenance will be the responsibility of the West Jefferson Levee District.

13.0 Appendices. (See Following Pages).

Appendix A Geotechnical Report (Separate Volume)
Appendix B Sample Calculations (Separate Volume)
Appendix C Survey Plan
Appendix D Design Quality Control Plan
Appendix E Technical Review Documentation

APPENDIX A

GEOTECHNICAL REPORT
(Separate Volume)

APPENDIX B

SAMPLE CALCULATIONS
(Separate Volume)

APPENDIX C

SURVEY PLAN

SURVEY PLAN

1. **Job#:** T.O.0012, Mod 04

2. **Contract #:** W912P8-06-D-0032

3. **Lat./Long**: 29°52'33"N   090°06'55"W

4. **Job Title**: Alternative Selection Report, Hurricane Protection Project, Old Estelle Pump Station to Lapalco Boulevard (14a.2 and 14g.2).

5. **General Approach**: All levee profiles/sections will be surveyed using GPS RTK procedures and conventional survey methods with OPUS checks on base station positions.

6. **Horizontal Positioning**:
   - 6.1 Datum: NAD 83.
   - 6.2 Control: Corps Baseline. Filename: T00-101A.83.
   - 6.3 Equipment: Leica 500/1200, Leica 800 Series Total Station.
   - 6.4 Methodology: RTK with ties to existing traverse and Total station topo under thick canopy of trees.

7. **Vertical Positioning:**
   - 7.1 Datum: NAVD88.
   - 7.2 Epoch: 2004.65.
   - 7.3 Control: US COE Monuments "**07-065C_GPS-1**", "**07-065C_GPS 2**" provided M. Huber of New Orleans COE office, with checks on existing monument "**BAFS-SM-02H**" and "**Q368**".
   - 7.4 Equipment: Seco Automatic Level.
   - 7.5 Methodology: RTK topo ties along levee profiles/sections or Total station topo under thick canopy of trees.

8. **US ACE FTL**: Christopher Dunn   **Phone:** 504-862-1799

9. **A/E Survey POC**: Rex Jones, C&C   **Phone**: 337-261-0660

10. **Approved:** Mark Huber   **Phone:** 504-862-1852   **Date:** 12-11-2007

APPENDIX D

DESIGN QUALITY CONTROL PLAN

DESIGN QUALITY CONTROL PLAN

ENGINEERING ALTERNATIVE REPORT

OLD ESTELLE PUMP STA. TO LAPALCO BLVD.
(B/L STA. 809+03.8 to B/L STA. 1010+00)

West Bank and Vicinity

Jefferson Parish, Louisiana

U. S. Army Corps
of Engineers
New Orleans District

W912P8-06-D-0032

Prepared by

BCG Engineering & Consulting, Inc.
Excellence in Engineering

2701 Kingman Street
Metairie, Louisiana

2008

DESIGN QUALITY CONTROL PLAN
FOR
ENGINEERING ALTERNATIVE REPORTS
OLD ESTELLE PUMP STATION TO NEW ESTELLE PUMP STATION
AND
NEW ESTELLE PUMP STATION TO LAPALCO BOULEVARD

1. Project Information:

   a) Project Names:

      (1) Westbank and Vicinity, New Orleans, Louisiana
      Hurricane Protection Project
      Westwego to Harvey Canal
      WBV 14a.2: Harvey Canal West Bank Levees – Phase 2
      Engineering Alternative Report

      (2) Westbank and Vicinity, New Orleans, Louisiana
      Hurricane Protection Project
      Westwego to Harvey Canal
      WBV 14g.2: Estelle Pump Station Vicinity Floodwalls
      Engineering Alternative Report

   b) Project Location: The work is located in Jefferson Parish, Louisiana, and is part of the Westwego to Harvey Canal, LA Hurricane Protection Project. The Westwego to Harvey Canal, LA Hurricane Protection Project provides Standard Project Hurricane (SPH) protection from Westwego Louisiana to the Harvey Canal as described in Design Memorandum (DM) No. 1 General Design Supplement No. 2, dated February 1990 and the Lake Cataouatche Area Post Authorization Change Report and Environmental Impact Statement (EIS). The area is bounded by the Old Estelle Pump Station at the southernmost limit and Lapalco Boulevard at the northernmost limit, roughly paralleling the GIWW and Harvey canals.

   c) Project Description: This project consist of the preparation of two reports outlining feasibility level designs for alternative methods of raising the existing hurricane protection to 100 year levels. The first report identified as WBV14a.2 will cover the protection from New Estelle Pump Station to Lapalco Boulevard. The second report identified as WBV14g.2 will cover the protection from Old Estelle Pump Station to New Estelle Pump Station. Multiple alternatives will be investigated in each report consisting of un-reinforced earthen levees, reinforced earthen levees, floodwalls and alternate alignments. Sufficient design work will be provided to support each plan and the associated costs and right of way requirements.

A copy of the Project Management Plan can be found at:
https://mvn-fshpo01.mvn.ds.usace.army.mil/HPSDocs/PDT/PROPDT/Floodwalls/PMP

The project may require a Value Engineering Study, which will be performed in the P&S Phase.

d) Project Work: Project work will include site investigation to review existing conditions, preparation of surveys by C&C Technologies Inc., geotechnical analysis by Eustis, and general, civil, structural and project management by Brown, Cunningham & Gannuch, Inc.

2. Purpose and Scope of DQCP:

a) Purpose: This Design Quality Control Plan (DQCP) outlines the technical expertise, technical criteria, and technical review processes that will be used to produce a quality product satisfying technical, functional, legal, safety and environmental requirements.

b) Scope of Reviews: The project will provide hurricane protection to a large part of the West Bank, and therefore the consequence of a failure would be substantial loss of property and potential loss of lives. The nature of the work requires complex engineering solutions; however, the work is similar to much of the hurricane protection work in the New Orleans area. There are inherent risks associated with weak foundation conditions in the area which will require careful consideration of all available soils data. The location of the project is along a navigable waterway and carries the added risk of damage from boat impact. All of these factors were considered in defining the scope of review effort. Detail checks of calculations will be performed to ensure that no computational errors are made and that standard practice is being used in performing the calculations. The detailed check of the plates will be used to eliminate obvious errors, check for proper references between drawings, ascertain whether adequate information was provided, and to review drawing standards. The Independent Technical Review (ITR) will be performed to ensure the quality of design and to substantiate that all services conform to contract requirements. A 65 percent and 95 percent review will be conducted and all comments generated from these reviews will be resolved thru the Dr. Checks System.

3. Deliverables:

Deliverables will include a DQCP, Proposed Benchmark Description Forms and advanced right of way plates. The primary deliverable will be an Engineering Alternative Report which will include right of way plates, design calculations, quantities, costs and a Survey Report Summary. The Engineering Alternative Report will be submitted at the 65 percent, 95 percent and 100 percent completion stages. The EAR will complement the Individual Environmental Report for this area and both will serve as input into the formal Project Decision Document.

4. Customer Involvement:

In addition to the U.S. Army Corps of Engineers the following authorities will be given an opportunity to review and comment on this study. Involvement will include review, meetings, e-mails and discussions as needed.

West Jefferson Levee District
7001 River Road
Marrero, LA 70072
Gerald Spohrer
Executive Director
P: 504-340-0318 x208
Wjld@wjld.com

Louisiana Department of Transportation & Development
Public Works and Hurricane Flood Protection
8900 Jimmy Wedell
Baton Rouge, LA 70807
Bill Feazel, P.E., P.L.S.
Director, Federal Programs
williamfeazel@ldotd.la.gov

Southeast Louisiana Flood Protection Authority – West
7001 River Road
Marrero, LA 70072
David J. Bindewald, Sr. President
P: (504)340-0318
david.bindewald@slfpaw.org

Jefferson Parish Department of Engineering
1221 Elmwood Parkway Boulevard, Suite 802
Jefferson, LA 70123
Mark Drewes, Director
P: (504) 736-6511
JPEngineering@jeffparish.net

Jefferson Parish Drainage Department
1221 Elmwood Parkway Boulevard
Suite 907
Jefferson, LA 70123

Kazem Alikhani (Director)	or	Ali Pirsalehy (Asst. Director)
P: 504-736-6751		P: 504-736-6730
JPDrainage@jeffparish.net		APirsalehy@jeffparish.net

As needed, additional points of contact will be verified and the DQCP will be updated to include other customers.

5. Metric System:

The existing project was designed and constructed using the inch-pound system of measurement. It is not practicable to use metric on this project.

6. Technical Criteria:

The following technical criteria will be used on this project.

ER 1110-1-12, Quality Management, 21 July 2006.

American Concrete Institute, Building Code Requirements for Structural Concrete and Commentary (ACI 318-05/318R-05; ACI 318-99/318R-99 to be used in conjunction with Corps of Engineers EM).

American Institute of Steel Construction (AISC), Manual of Steel Construction, Allowable Stress Design, 9th Edition

American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)

American Welding Society, Structural Welding Code, Steel (AWS-D1.1-02)

WES Technical Report H70-2 including Appendix A, Operating Forces on Sector Gates Under Reverse Heads, March 70 and Dec. 71

National Electric Code, 2005

ETL 1110-2-569	Design Guidance for Levee Underseepage (May 05)
EM 385-1-1	Safety and Health Requirements Manual, ENG Form 5044-R (Nov. 03).
EM 1110-2-1424	Lubricants and Hydraulic Fluids (Feb 99, Jul 06)
EM 1110-2-1902	Slope Stability (Oct. 03)
EM 1110-2-1901	Seepage Analysis and Control for Dams (Apr 93)
EM 1110-2-1913	Design and Construction of Levees (Apr. 00)
EM 1110-2-2000	Standard Practice for Concrete for Civil Works Structures Change 2 (Mar 01).
EM 1110-2-2100	Stability Analysis of Concrete Structures (Dec 05)
EM 1110-2-2102	Waterstops and Other Joint Materials (Sep 95).
EM 1110-2-2104	Strength Design Criteria for Reinforced Concrete Hydraulic Structures (Jun 92, Aug 03).
EM 1110-2-2105	Design of Hydraulic Steel Structures Change 1 (May 94).
EM 1110-2-2400	Structural Design and Evaluation of Outlet Works (Jun 03)
EM 1110-2-2502	Retaining and Floodwalls (Sep 89).
EM 1110-2-2503	Design of Sheet Pile Cellular Structures Cofferdams & Retaining Structures (Sep 89)

EM 1110-2-2504	Design of Sheet Pile Walls (Mar 94).
EM 1110-2-2704	Cathodic Protection Systems for Civil Works Structures, 1 Jan 99
EM 1110-2-2906	Design of Pile Foundations (Jan 91).
EM 1110-2-3102	General Principles of Pumping Station Design and Layout (Feb 95)
EM 1110-2-3104	Structural and Architectural Design of Pumping Stations (Jun 89)

Hurricane and Storm Reduction System Design Guidelines, dated 23 Oct 07
http://www.mvn.usace.army.mil/ED/edsp/MVN-ED_HSDRS_Design_Guidelines_2007-10.pdf

DIVR 1110-1-400, Soil Mechanic Data, Dec. 98
https://inet.mvk.usace.army.mil/offices/im/private/cis/publications/mvdpubs.htm

The Government furnished information included in this work is listed in the Scope of Work for this effort.

7. Horizontal and Vertical Datums:

The horizontal datum to be used for this project is tied to the State Plane Coordinate System using North American Datum of 1983 (NAD83). The vertical datum to be used for this project is tied to the North American Vertical Datum of 1988 (NAVD88-2004.65). The establishment and use of vertical datums in the design work will follow the guidance provided in CECW-CE, INTERIM GUIDANCE FOR A PRELIMINARY EVALUATION OF VERTICAL DATUMS ON FLOOD CONTROL, SHORE PROTECTION, HURRICANE PROTECTION, AND NAVIGATION PROJECTS, dated 31 October 2006. Information relating to the location and determination of elevations of all vertical datums used in the project design will be provided, in the form of a Survey Documentation Report, for review and validation. When completed, the Survey Documentation Report will be included as an attachment to the DQCP (attachment 4).

a. All surveys shall be conducted in accordance with CEMVN-ED-SS-06-01, "USACE New Orleans District Guide for Minimum Survey Standards for Performing Hydrographic, Topographic, and Geodetic Surveys". The guidance is available at http://www.mvn.usace.army.mil/ed/edss/surveyingguidelines.asp.

b. A Survey Report Summary will be completed by Engineering Division, Survey Section for Independent Technical Review (ITR) within two weeks of completing the surveying activities and office processing.

c. Minimum survey deliverables shall include: Survey Report Summary, PDF file of all field books and logs, ASCII coordinate file containing pertinent metadata records, and Benchmark Description Forms.

d. Hurricane protection projects shall be referenced to both NAVD88 and Local Mean Sea Level (LMSL). Where the relationship between NAVD88 and the LMSL does not exist, a tidal study is necessary to establish the local sea level datum.

e. All geospatial data shall contain metadata which defines the relationship between NAVD88 and the local tidal datum (LMSL, MLLW, etc) using the latest epochs.

f. All projects shall reference a minimum of three Permanent Bench Marks (PBM). Ideally these PBMs shall be located in the middle and at each end of the project. All surveys shall tie into a minimum of three benchmarks to determine the reliability of the project's control. The three permanent bench marks to be used are shown below:

Name	Description	Elevation
Q368	1.7 kw (1.05 miles) west along Lapalco Boulevard from the junction of State Highway 45 (Barataria Boulevard) in Marrero, at the entrance to the Marrero sewer treatment plant which is on the west side of outfall canal, 21.48 meters (70.5 feet) north of the centerline of Lapalco Boulevard 6.94 meters (22.8 feet) east of the center of the double security entrance gate to the sewer plant, 14.93 meters (49.0 feet) west and 0.45 meters (1.5 feet) south of the security fence. Note, driving rate Metairie Road anchored. The mark is 0.42 meters (1.4 feet) south from a witness post. The mark is 0.46 meters (1.5 feet) M below Lapalco Boulevard.	2.329 (NAVD88)
T368	2.4 kw (1.05 miles) northerly along State Highway 45 (Barataria Boulevard) from the junction of State Highway 3134 in Marrero, thence 2.5 kw (1.55 miles) easterly along Lapalco Boulevard, in top of and 0.6 M (2.0 feet) north of the south end of the south concrete curb of the Boulevard bridge spanning Harvey Canal, 5.0 M (16.4 feet) south of the centerline of the eastbound lanes of the Boulevard, 1.2 M (3.9 feet) east of the west end of the curb, and 0.3 M (1.0 feet) above the level of the Boulevard.	5.31 (NAVD88 2004.65)
U368	2.4 kw (1.05 miles) northerly along State Highway 45 (Barataria Boulevard) from the junction of State Highway 3134 in Marrero, thence 4 kw (2.10 miles) easterly along Lapalco Boulevard, in top of and 0.7 M (2.3 feet) north of the south end of the east concrete abutment of the Boulevard bridge spanning Murphy Canal, 8.7 M (28.5 feet) south of the centerline of the eastbound lanes of the Boulevard, 0.9 M (3.0 feet) west of the east edge of the abutment, and 0.3 M (1.0 feet) above the level of the Boulevard.	-0.24 (NAVD88 2004.65)

8. Project Delivery Team (PDT):

a) Project Delivery Team: The PDT will be led by an experienced team leader as shown in the table below. The other PDT members also have considerable experience as described in the table below. Should future requirements require the application of different skills appropriate personnel will be added to the PDT.

Project Role/Responsibility	**Name/Registration**	**Company**	**Experience**
Project Principal	Rodney J. Gannuch/P.E.	BCG	30+ years
PDT Leader	C.C. Hamby/P.E.	BCG	30+ years
Structural/Civil	C.C. Hamby/P.E.	BCG	30+ years
Structural/Civil	Terry Cox/P.E.	BCG	40+ years
Civil	David Dodgen/P.E.	BCG	25+ years
Hydraulics	Cecil Soileau/P.E.	BCG	30+ years
Mechanical	Carlos Hernandez/P.E.	BCG	40+ years
Geotechnical	Gwen Sanders/P.E.	Eustis	15 years
Geotechnical	Jim Hance/P.E.	Eustis	8 years
Surveys	Frank Lipari/P.E.	C&C	30+ years
Surveys	Rex Jones/P.L.S.	C&C	28 years

b) QC Review: The review procedures for this project will be conducted in accordance with the DQCP developed for these projects as a group. The DQCP follows the BCG Quality Assurance Program guidelines and USACE guidelines for Quality Control Plans, and incorporates the applicable sections into this work.

The reviews for this project will be conducted and documented on appropriate forms and signed by the reviewers and Project Manager. Reviews will consist of calculation checks, both design and quantity calculations, detailed checking and ITRs of the work products.

Calculation checks will consist of detailed checks of engineering design calculations and quantities. Calculations will be checked for correctness of calculation, and computer calculations will be checked for input, output and reasonableness of results. Deficiencies will be discussed with the originator of the calculation and resolved. The calculations and quantities review will be signed and dated and approved by the Project Manager.

Detailed checking will be performed on the plates for submission to the Corps of Engineers. The review will be performed by experienced professional engineers in the discipline of work involved and who may be a member of the Team but did not participate in the preparation of the document(s) reviewed. The comments will be resolved between the originator of the documents and the reviewer with the response noted. The full preparation of the documents will be reviewed.

9. Independent Technical Review (ITR):

The ITR will be conducted by experienced personnel as shown in the table below. The ITR will be performed on all products following the guidance provided in ER 1110-1-12, Engineering and Design, Quality Management, dated September 30, 2005 and in Appendix B of the HPS Quality Management Plan.

An ITR will be conducted by Thomas K. Grant, P.E. (Civil/Structural), Dr. Chris Saucier, P.E. (Geotechnical), and William Gwyn (Geotechnical), who were not involved in the preparation of the design documents and have senior level experience. Mr. Grant is a licensed professional engineer with experience in civil and structural engineering. Mr. Grant has extensive experience in flood control projects and has over 45 years of experience and his entire career has been as a civil/structural engineer with the Corps of Engineers. He is a professional engineer with both civil and structural experience in levee projects, large navigation projects, and numerous types of flood control projects. Dr. Saucier is a licensed professional engineer with over 18 years experience in geotechnical engineering. Dr. Saucier has considerable experience in geotechnical engineering for projects in the New Orleans area and is currently teaching geotechnical engineering at Mississippi State University. Prior to his teaching assignment Dr. Saucier performed geotechnical engineering in the New Orleans area for several years. Mr. Gwyn has over 35 years of geotechnical experience in the design of flood protection projects in the New Orleans area. He has extensive experience with the Corp of Engineers and was formerly employed by the Corps of Engineers, New Orleans District. The ITR will review and evaluate the material requiring interpretation, and verify and validate assumptions, methodologies, and conclusions. The ITR will be a continual process with formal reviews coordinated with the PDT at critical points. It will also verify that the completed work meets the contractual requirements. The scope of the review is given in Paragraph 2 above. Comments will be entered into the Dr. Checks System (reference ER 1110-1-8159, 10 May 2001) with the response noted and differences discussed and resolved with the originator of the documents. Documentation will be provided for all ITR's, consisting of a completed (signed) statement of technical review and certification (reference ER 1110-1-12 and Appendix B of HPS Quality Management Plan), to which is attached to all review comments (identified by the Reviewer) and response of designers to the comment.

ITR Civil/Structural	Thomas K. Grant / P.E.	BCG	45+ years
ITR Geotechnical	Chris Saucier / P.E.	Eustis	18 years
ITR Geotechnical	William Gwyn / P.E.	Eustis	35+ years

10. 65% & 95% Progress Reviews:

1. General: These reviews will follow the guidance and requirements of Section 5 of the HPS QAP and ER 415-1-1. The technical reviews are coordinated reviews by a qualified team to improve how well the alternatives presented in the Engineering Alternative Report (EAR) can be understood, to assure that the report adequately addresses the construction costs and durations, real estate requirements and associated costs/cultural consequences, constructability of the alternatives presented, operations and maintenance costs associated

with the alternatives presented, and any relocations required in conjunction with any specific alternative. The reviews will also include a comprehensive evaluation of correct application of methods, validity of assumptions, adequacy of basic data, correctness of calculations, and completeness of documentation, compliance with guidance and standards, and constructability considerations. These reviews shall include the Review Team listed below, local sponsors and other pertinent stakeholders to assure customer involvement in all major decisions. The ITR, on the other hand, is a comprehensive, holistic review by a qualified person or team not involved in the day-to-day production of a project or product, for the purpose of confirming the proper application of clearly established criteria, regulations, laws, codes, principles, and professional practices. These reviews will be joint MVN and PRO office efforts to serve as the processes that assure the basic product (EAR) submitted meets the intent of Hurricane Protection Project requirements.

2. <u>Structure of Reviews</u>: Dr. Checks shall be utilized for all reviews in compliance with ER 1110-1-8159. At the submittal of the design documentations the review team shall conduct a thorough review as described above and enter comments into Dr. Checks. The design team will periodically inspect the comments being entered and prepare preliminary responses for the comments, but no evaluations will be entered until the conclusion of the review period. At the specified time, the review will be closed to new comments, and the design team will then begin entering comment evaluations. If any review comments provide conflicting guidance, the design team will notify the Review Team Leader, who will work with the individual review team members to determine which comment will govern – the conflicting comment will then be rescinded by its submitter. Upon completion of evaluations, the Review Team Leader will then initiate backchecking and comment closeout. Any outstanding issues will be resolved by the Review Team Leader and the Functional Team Leader.

3. <u>Review Team:</u> The names and disciplines of the reviewers are presented in the table below, but will be updated by the FTL at the time of the review if there are changes. Each team member is knowledgeable about the critical project requirements of all his or her PDT counterparts, understands how his or her own particular project elements and work relates to and affects those requirements, and conducts his or her reviews to insure consistency and effective coordination across all project disciplines. A Technical Manager will act as the lead reviewer for Structures Branch. The anticipated reviewers include the following:

Name	Discipline (Yrs. Exp. In Discipline)	Role (Yrs. Exp. in Role)	Office	Registration
Christopher Dunn	Civil/Structural (9)	Functional Team Leader/Review Team Lead	ED-T	P.E., LA
T. Wade Wright	Civil Tech.	Levees Reviewer	ED-L	
Anh Nguy	Civil (1)	Cost Reviewer	ED-SC	E.I., LA
Gaynell Morrison	Civil Tech.	Relocations Reviewer	ED-SR	

Name	Discipline (Yrs. Exp. In Discipline)	Role (Yrs. Exp. in Role)	Office	Registration
Tim Ruppert	Civil (7)	Engineering Controls Reviewer	ED-E	P.E., LA
Getrisc Coulson	Environmental	Environmental Reviewer	PM-R	
Kim Tullier	Civil/Geotechnical	Geotechnical Reviewer	ED-F	
Rachel Tranchina	Mechanical (6.5)	Mechanical Reviewer	ED-T	E.I., LA
Nancy Powell	Civil/Hydraulics (28)	Hydraulics Reviewer	ED-H	E.I., LA
Jabeen Pasha	Electrical	Electrical Reviewer	ED-T	
Rob Thomson	Real Estate	Real Estate Reviewer	RE	
Jim Montegut	Civil/Construction	Construction Reviewer	CD	
Michael Stack	Civil	Project Management Reviewer	PM-OH	
Carl Balint	Civil/Structural (16)	Structural Reviewer	ED-T	E.I., LA

11. Schedule Checklist:

The attached Quality Control Plan Checklist shows the scheduled and actual dates for ITR, resolution of comments, and other critical checkpoints associated with this contract package. Typical checkpoints are indicated with those specific to this contract indicated by dates.

12. Design Quality Assurance Plan Supplement Updates:

Monthly updates of the DQAP supplement (see attachment 1) will be provided to the Structures Branch QA Coordinator for the project DQAP supplement database of the HPS-QMP.

13. Design Quality Assurance Plan Supplement (DQAPS):

The Structures Branch QA Coordinator will maintain the DQAPS database (https://www.intra.mvn.usace.army.mil/pm/pwp/frmprintreport2.aspx) of the HPS QMP with input from Technical Managers and Functional Team Leaders and will provide monthly updates.

14. Records Maintenance:

Documentation will follow the requirements of section 4.3 of the HPS QAP.

The following QC documentation will be provided.

b. ITR review comments, resolution of comments, and statement of independent technical review and certification (concurrent with final submittal of design product). Reference Appendix F or G of the HSDRRS Quality Management Plan.

c. Resolution of progress review comments.

d. Design Documentation Report, which includes the technical documentation of the design (e.g. calculations, load cases, etc. as required) plus the items above.

All reviewed and accepted documents shall be filed in electronic form in the ProjectWise database by Engineering Control Branch in Engineering Division. POC for Engineering Control is Mike Dupuy, ext. 2612.

The A-E PDT Leader will prepare an After Action Report that includes a lessons learned summary in accordance with MVD guidance and submit to the MVN Technical Manager within 30 days of completion of the P&S.

13. Signatures:

P.E.	13 May '08
A-E PDT Leader	Date
PE	13 May 08
A-E ITR Leader(s)	Date
Rodney J. Hannud, P.E.	13 May '08
A-E Project Manager	Date

ENGINEERING ALTERNATIVE REPORT
V-LINE LEVEE STATION 809+03.8 B/L TO STATION 1010+00 B/L

WEST BANK AND VICINITY
JEFFERSON PARISH, LOUISIANA

DESIGN QUALITY CONTROL PLAN CHECKLIST

WBV14a.2 Report:

ITEM	SCHEDULE DATE	ACTUAL DATE	COMMENTS
DQCP Submitted	12 Dec 07	12 Dec 07	
65% Submittal	23 Jan 08	14 Feb 08	
95% Submittal	16 Feb 08	14 Apr 08	
ITR Complete (With Certification)	16 Feb 08	14 May 08	
Final Document with Calculation Checks Complete	12 Mar 08	14 May 08	

ENGINEERING ALTERNATIVE REPORT
V-LINE LEVEE STATION 809+03.8 B/L TO STATION 1010+00 B/L

WEST BANK AND VICINITY
JEFFERSON PARISH, LOUISIANA

DESIGN QUALITY CONTROL PLAN CHECKLIST

WBV14g.2 Report:

ITEM	SCHEDULE DATE	ACTUAL DATE	COMMENTS
DQCP Submitted	12 Dec 07	12 Dec 07	
65% Submittal	28 Jan 08	21 Mar 08	
95% Submittal	17 Mar 08	14 May 08	
ITR Complete (With Certification)	17 Mar 08		
Final Document with Calculation Checks Complete	28 Apr 08		