US Army Corps of Engineers

Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System

Draft Final Report of the Interagency Performance Evaluation Task Force

Volume III – The Hurricane Protection System

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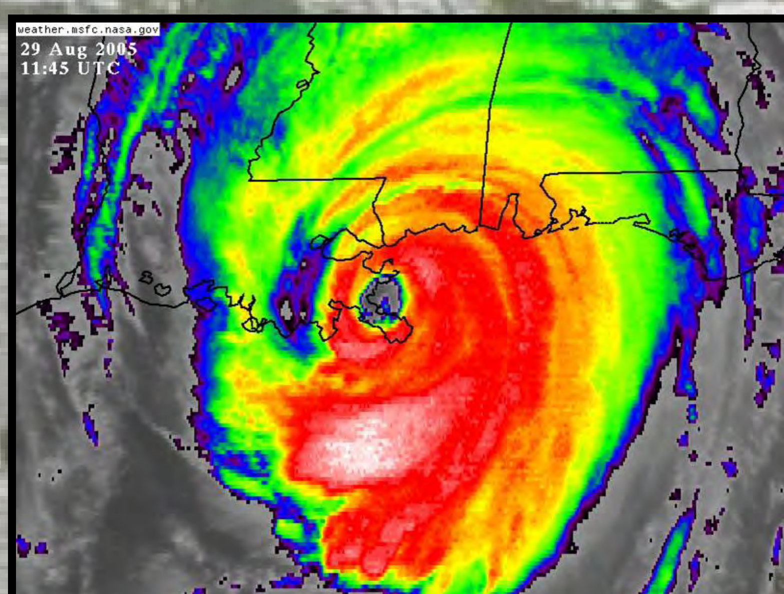

FINAL DRAFT

(Subject to Revision)

Volume III
The Hurricane Protection System

This report is not intended as a final expression of the findings or conclusions of the United States Army Corps of Engineers, nor has it been adopted by the Corps as such. Rather, this is a preliminary report summarizing data and interim results compiled to date. As a preliminary report, this document and the information contained therein are subject to revisions and changes as additional information is obtained.

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Appendix 1: History of Hurricane Protection System

3.1 Executive Summary

There are nine volumes in the Interagency Performance Evaluation Task force (IPET) performance evaluation of the New Orleans and Southeast Louisiana hurricane protection system report. Volume I provides an overview of the findings and lessons learned for the broad multi-disciplinary evaluation performed. This volume, Volume III, of the report addresses the design criteria for the pre-Katrina hurricane protection system, any changes that have occurred during construction, and the operation and maintenance of the system after construction. The purpose of this volume is to synopsize and appropriately summarize this information and not to draw recommendations on the information. This volume is also intended to provide insight and direction on where additional information relating to the design, constructed, and maintained condition of the hurricane protection system can be found. Documents referenced in this volume can be obtained from the IPET web site at https://ipet.wes.army.mil. Volume IX of the IPET Report contains a listing of approximately 4000 documents currently on the IPET web site relating to the hurricane protection system. The specific scope of this volume is to address the first IPET objective as presented in Volume I. That objective is to answer the following question:

1. **What were the design criteria for the pre-Katrina hurricane protection system, and did the design, as-built construction, and maintenance condition meet these criteria?**

The volume presents information in response to this question based on the three Corps of Engineers authorized hurricane protection projects in the area – Lake Pontchartrain and Vicinity, West Bank and Vicinity, and New Orleans to Venice. Within each project, the volume is further divided into sections arranged by parish, basin, subarea, and/or reach of construction. Each section contains a description of the project or reach of project, the pre-Katrina status of construction, and the original design criteria organized by technical discipline. This is followed by construction quality control, as-built conditions, and subsequent inspection and maintenance of the completed works. The sections also include a brief presentation of criteria for interior drainage, pumping stations, the Mississippi River Levee Flood Protection System, and non-Corps of Engineers levee features located within each project area. While these other features are not directly affiliated with the Corps of Engineers hurricane flood protection system, they are an integral part of the overall drainage and hurricane protection system as demonstrated in Hurricane Katrina.

Volume III concludes with a section describing the post-Katrina changes to the hurricane protection system and a listing of all references used for the volume. An appendix contains a detailed history of the hurricane protection system canals.

To broadly summarize the information in this volume, the following four sub-questions to IPET Question 1 with responses are listed below. This volume contains a voluminous amount of technical data. It would not be possible to provide a thorough summary in this concise executive summary. For this purpose, Section 3.2 of this volume, The Hurricane Protection System, contains a more comprehensive design overview by technical discipline of the report information.

a. **What were the design assumptions and as-built characteristics of the primary components of the flood protection system?**

For each of the three hurricane protection projects, a design hurricane was selected that served as the basis for the hydraulic and hydrology (H&H) design of the plan of the project. It was assumed that the design hurricane would approach a given project site from a critical path, and at such rate of movement, to produce the highest hurricane surge hydrograph for that characteristic storm, considering pertinent hydraulic characteristics of the area.

For Lake Pontchartrain and Vicinity and West Bank and Vicinity projects, the standard project hurricane (SPH) was selected as the design hurricane because of the urban nature of the project area. For New Orleans to Venice project, the design hurricane was a hurricane that would produce a 100-year surge elevation; the meteorological parameters used for this storm were derived from SPH parameters. Meteorological parameters for the SPH storm were developed by the U.S. Weather Bureau; the parameters valid at the time of project authorization were used to calculate wind speeds for the design storms. A general wind tide equation, calibrated to observed high water marks from three storms along the Mississippi gulf coast, was used to compute wind tide levels, after verifying with data from two storms along the Louisiana coast, including Hurricane Betsy. Adjustments were made to the wind tide levels to consider surge, setup, tide, and runoff from rainfall in lakes, the effects of marshland where protection systems were a considerable distance from the coastline and, in the case of the three outfall canals in Orleans East Bank, the effects of bridges and runoff that is pumped into the canals.

For surge propagating up the Mississippi River, a bathystrophic storm surge technique was used to compute surge along the river. The procedure was validated with observed data from Hurricane Betsy.

Where wave runup was considered to be present, wave runup for the significant wave was computed using methodology available at the time of project authorization. For Lake Pontchartrain and Vicinity and New Orleans to Venice projects, the methodology was based on interpolation of model study data developed by Saville. For the West Bank and Vicinity project, wave runup was computed using procedures found in the 1984 *Shore Protection Manual*. Wave runup was added to the wind tide level to get the design elevation. Where wave runup was not computed, freeboard levels of 1, 2, or 3 feet were added to the wind tide level to get the design elevation.

The Hurricane Protection System consists predominantly of either levee or a combination of levee and cantilevered I-type floodwall. In addition, where limited rights-of-way did not permit a levee/I-wall footprint and at gated closure structures where the levee alignment crosses vehicular roads or railroads, there are segments of T-walls. T-walls are inverted, T-shaped concrete structures supported on precast prestressed concrete or steel H-pile bearing piles. A continuous steel sheet pile wall is embedded into the bottom of the T-wall base slab to reduce seepage under the wall. I-type floodwalls consist of a cantilever floodwall comprised of steel sheet piling driven through an existing levee and/or fill and either capped with a concrete wall or left uncapped. Gated closure structures consist of a swing gate, miter swing gate, bottom roller gate, or top roller gate supported on a concrete monolith with a bearing pile foundation. I-wall, T-wall, and various gated closures are often combined to form a single wall. When this is the case, the steel sheet pile and capped I-wall and T-wall stem serve as continuous seepage cut-off. At I-wall/T-wall monolith joints, a fabricated sheet pile section that allows independent movement of the two types of walls is used.

All of the projects are constructed over weak and compressible soils. Stability and settlements of the structures are generally critical design issues. The weak and compressible foundation soils generally require that all levees be constructed using staged construction procedures. Consequently, relatively long periods of time were frequently required to achieve project grades for levees. The levees were generally analyzed for stability using short-term or undrained shear strengths, because the weak undrained strengths of the foundation soils had been proven to control design. The cantilevered I-walls were generally designed considering both short-term (undrained) and long-term (drained) strengths.

The structural designs of these features follow industry codes and criteria as modified by the Corps of Engineers more conservative criteria for hydraulic structures. Some variations did occur amongst projects in the loading conditions, factors of safety, and sheet pile penetration ratios for I-wall designs. A dynamic wave impact loading case was included in the designs for I-walls and T-walls considered exposed to wave conditions, such as along lake front areas, as opposed to walls paralleling canal areas where a wave loading case was not part of the analysis. Also there was change in the criteria to determine the penetration of I-walls for designs prepared after December 1987 based on a sheet pile wall field load test, commonly referred to as the E-99 test. This change is discussed in more detail in the design overview section along with other criteria and material changes that occurred from early to later designs.

The interior drainage system consists of overland flow, storm sewers, roadside ditches, flow down roadways, collector ditches, interior canals, interior pump (lift) stations, outfall pump stations, and outfall canals. The interior drainage system is designed for removing stormwater from rainfall events, not for removing water that enters the area from levee or floodwall overtopping or breaches.

The current design criterion for new storm drainage facilities in most of the parishes is the 10% probability (10-year frequency). Generally, the capacity of the older parts of the storm drain systems are approximately the 50% probability (2-year frequency) event, and in some cases less. The functional capacity of the interior canals and pump stations varies from 0.25 to 0.7 inches per hour.

The interior drainage system was in good condition and prepared for high inflows from rainfall prior to August 29, 2005, Katrina landfall.

For design of pumping stations, each of the four parishes is divided into drainage basins. The basins usually follow natural topographical lines. They are often bordered by levees or ridges of relatively higher elevations. Pump stations are located throughout the drainage basins. The function of the pump stations is to remove excess water accumulated from rainfall and seepage from the surrounding bodies of water. New Orleans area is surrounded by several bodies of water, including the Gulf of Mexico, Lake Pontchartrain, and the Mississippi River. The natural elevation of most of the land is lower than the surrounding bodies of water. Levees and floodwalls are designed to prevent the surrounding bodies of water from freely flowing into the area. They also keep water from flowing out. Flooding will occur if accumulated precipitation and seepage from surrounding bodies of water are not removed. An elaborate system of canals directs the accumulated water to the pump stations. The pump stations remove the accumulated water by discharging the water to other side of the levees and floodwalls. The pump stations are designed to keep up with natural rainfall and seepage.

Historically, the pumping stations have not been considered to be part of the hurricane protection system except in a few instances where the buildings are a structural part of a levee or floodwall. Since much of the area is below the level of Lake Pontchartrain, sea level, and the Mississippi River, the pumping stations are needed to prevent flooding caused by accumulated rainfall and seepage, and (as in the case of Katrina) to evacuate floodwaters after a failure of the hurricane protection system. These stations would have performed as designed during Katrina to dewater their respective drainage basins had the hurricane protection system not failed.

There are nearly 100 pumping stations in the greater New Orleans area. Some have been recently completed. Others are approaching 100 years old. Most of the pumping stations have significant variations in their design, construction, and capacity. Station designs range from large plants built of reinforced concrete to small capacity stations housed in light gage metal frame buildings.

Operational power is provided by various means. Some stations use pumps directly connected to diesel engines. For many stations, power is normally provided by the electrical grid with backup diesel generators or direct drive diesel engines available when the electrical grid is out of service. Some of the older stations utilize 25-Hz power provided by a central generating plant to run the pumps. These stations use frequency changers to change 25-Hz power to 60-Hz power for the operation of their station service. Some prime movers use gearboxes and a few use hydraulic motors and pumps to transmit the power from the motor or engine to the pump shaft.

Prior to Katrina, the pumping stations in the Greater New Orleans area were operational and prepared for removal of runoff from high rainfall events.

b. **What records of inspection and maintenance of original construction and post Katrina repairs are available that documents their conditions.**

Once the construction contract is completed and release of claims is granted by the Contractor, the records of the project are boxed up and sent to off-site storage where they remain for six years. After 6 years, they are destroyed.

A copy of the Completion Report and the as-built drawings are sent to the Corps of Engineers New Orleans District Engineering Division where they are maintained. The documents that are maintained in the construction files for 7 years include the Completion Report and files on modifications and claims.

On construction contracts, the Government Quality Assurance (QA) reports and Contractor Quality Control (QC) reports are normally filed and stored together. QC reports normally follow a Government suggested format; therefore, they usually cover the same items. Those items are general information about the weather conditions for that day, the numbers of laborers and supervisors on the job, hours worked, and the operating equipment that is on the job. There is a statement as to what work was performed that day. There are paragraphs to cover the results of the controlled activities, such as preparatory, initial, and follow-up meetings and inspections; and for tests performed that day, as required in the plans and specifications. There are paragraphs for materials received, submittals reviewed, off-site surveillance activities, job safety, environmental protection, and a general remarks paragraph.

Much of the same information is covered in the QA reports. The items /sections listed on the QA reports usually are as follows: general information about the weather conditions for that day, the number of contractor and government employees on the job, the prime contractor and the subcontractors on the job and their responsibilities, and description of the work performed that day. There are sections for days of no-work and reasons for the no-work, and progress of the work. There is also information on Contractor Quality Control (CQC) inspection phases attended, instructions given, and results of QA inspections and tests, deficiencies observed and actions taken, and corrective action of the contractor. There are sections for verbal instructions given the contractor that day; for controversial matters that may have arisen; for information, instructions, or actions taken not covered in QC reports or disagreements; for safety; and a section for remarks.

The construction documents that were reviewed are summarized by project based on a review of the documents available. The project summary can be found under the as-built paragraph of each project.

Completed Federal Civil Works projects are inspected by the Corps of Engineers under the Periodic Inspection Program. Most structures are inspected at 5-year intervals, certain selected local interest structures are inspected at 3-year intervals, and federal bridges inspected at 2-year intervals. Three structures are inspected under this program, namely Bayou Bienvenue Control Structure and Bayou Dupre Control Structure in the St. Bernard area, and Empire Floodgate in the Plaquemines area. Detailed information on these inspections can be found in Volume III. Prior to Hurricane Katrina, these structures were found to be generally in good operating condition. As part of the Periodic Inspection Program, components of the HPS were systematically reviewed under the current design criteria.

Federally constructed structures, turned over to local interests for operation and maintenance, are inspected annually by the federal government as required by Engineer Regulation (ER) 1130-2-530. The local entity is required to follow the requirements of 33 CFR 208.10. All ratings for hurricane protection systems were at least acceptable, and sometimes outstanding. These ratings are general in nature, and do not address detail features of the project. Actual observed conditions demonstrate that trees, hot tubs, swimming pools, and other encroachments have been allowed to accumulate over time. The program for annual inspections has not historically been structured to accommodate a rigorous inspection, findings, and documentation process. The ratings do occasionally address status of completion for specific hurricane reaches. More information on these inspections is included in Volume III.

c. What subsurface exploration and geotechnical laboratory testing information was available as the basis of design, and were these conditions verified during construction?

The subsurface exploration and geotechnical laboratory testing information that were available as the basis of design are included in the more than 60 Design Documents included in the reference list. Copies of the individual boring logs and laboratory test results in the form of individual soil tests data sheets were provided in each design document along with plates depicting the cross-sections analyzed and the selected design shear strengths. The standard practice was to use all boring and test data from previous and current investigations as part of the site characterization. The borings were taken at spacings ranging from 350 to 1500 feet apart and were usually 50 to 80 feet deep with a few extending to a depth of 100 feet. Generally, the borings were taken at 350 to 650 feet apart in the areas where floodwalls were to be constructed and 700- to 1500-foot spacings in the more remote levee reaches. As part of staged construction for each levee enlargement, additional borings would be made to evaluate the strength gain since the last enlargement.

The conditions were essentially confirmed by the fact that after reviewing more than 50 sets of contract documents, five of the contracts reviewed showed modifications or changes. Four would be considered as changed conditions. (Paragraph 3.2.1.5.4.1.10) 17th Street Canal East Side Stations 0+96.27 to Station 7+00 cut off 4′ 3″ of sheet pile because of unanticipated hard driving. The second is described in paragraph 3.2.1.5.4.1.22 Orleans East Bank. During dewatering of the steel sheet pile cofferdam for the 17th Avenue Outfall Canal, Hammond Highway Complex, excessive settlement of the cofferdam occurred on one side because the borings did not identify a layer of extremely soft soils. The third is described in paragraph 3.2.1.6.4.1.3 New Orleans East, South Point to GIWW. During construction of a floodside berm, it began to slide and crack. A modification was issued to change the berm configuration by lowering the height of the berm and making it wider. The fourth instance (paragraph 3.2.1.9.4.1.1) St. Charles Parish North of Airline Highway required modifications to remove pile driving obstructions. The fifth occurrence is described in paragraph 3.2.1.8.4.1.1 Jefferson Parish Lakefront Levee Pump Station No. 2 where, because of a survey error, the breakwater was realigned by moving it 70 feet to the west to obtain better alignment. Also, an obstruction was encountered while driving sheet piles for the breakwater that required cutting off some sheet piles.

d. Were the subsurface conditions at the locations of levee failures unique, or are there same conditions found elsewhere?

Based on the geology of the area and the various environments of deposition of the Holocene age, it is possible that the same conditions could be found elsewhere. In areas where suspected foundation failures have occurred, the soils involved have consisted of varying thicknesses of peat and/or weak clays overlying sand and/or clay layers. The peat and/or weak clays have generally been marsh/swamp deposits and the clay layers have been lacustrine/interdistributary deposits. These conditions exist over larger areas of the project. The general spacing of the borings could possibly miss some areas of weaker soils.

Hurricane Protection System Findings

There are no findings that indicated government or contractor negligence or malfeasance. The system was generally built as designed, with the exceptions noted below, using design approaches that were consistent with industry and local practices. Due to an inaccurate relationship between geodetic datum and mean sea level much of the system was built below specified design elevations. Parts of the system have yet to be fully constructed. The majority of the pump stations are not designed to provide capability during large storms. The lack of a CSX closure gate prevented the system from being operated as designed. While the presence of trees and other features on the levees were not obvious causes of breaching, it is possible that they were enablers in the overall breaching process.

Hurricane Protection System Lessons Learned

Design methods and designs need periodic review to determine whether they represent best practice and knowledge. Designs for hurricane protection systems need to include consideration of resilience, adaptation, and redundancy to accommodate unanticipated conditions or structural behaviors. Designs should be based on a system-wide understanding of the processes affecting the system and the interaction and interdependencies of the system components.

Participants

This Volume was developed as a joint effort either directly or indirectly by the individuals listed below:

Name	Organization
Carol Burdine	USACE
Charles Camillo	USACE
Shung Chiu	USACE
Larry A. Cooley	Burns, Cooley, Dennis
Angela Desoto-Duncan	USACE
Steve Fitzgerald	Harris County Flood Control District
Robert L. Fleming	Burns, Cooley, Dennis

Bob Howard	South Florida Water Management District
Mark Huber	USACE
Joseph Jacobazzi	USACE (on loan from the Metropolitan Water Reclamation District of Greater Chicago)
John Jaeger	USACE
Reuben Mabry	USACE
Denise Martin	USACE
James Morgan	Burns, Cooley, Dennis
Alfred Naomi	USACE
Nancy Powell	USACE
James A. Young	USACE

3.2 The Hurricane Protection System

Southeastern Louisiana is subject to heavy property damage and high risk to human life from hurricane-induced flooding. The first attempt to address this problem occurred when Congress authorized Lake Pontchartrain, LA, in the flood control Act of 1946. This project was completed in 1965 and its purpose was to protect Jefferson Parish from storm-induced flooding from Lake Pontchartrain for 30-year (yr) frequency storms. Since that time, Congress has authorized additional projects at various locations in southeast Louisiana.

History

Since its founding in 1718, the city of New Orleans has struggled against the annual flooding of the Mississippi River and the occasional storm surge flooding brought by tropical cyclones. Private river levees were constructed almost from the beginning. Federal participation in these efforts included establishment of the Mississippi River Commission in 1879 and the Mississippi River and Tributaries project in 1928.

Originally situated on the relatively high ground near the river, the city continued to grow and expand through the 20th century. Marsh land north of the city was drained for development up to the shore of Lake Pontchartrain. Protected from the seasonal floods of the river, attention shifted to building levees along Lake Pontchartrain to the north and Lake Borgne to the east of the city.

Surrounding parishes experienced collateral growth, particularly with the development of the fossil fuel extraction and processing industry, and the growing prominence of Louisiana's seafood industry. During the past 40 years, as infrastructure and population expanded, the hurricane protection system was expanded to include protection for national economic assets.

Interior Drainage

While levees provided protection from rising tides and storm surges, they also hydraulically isolated urban and industrialized areas. Rainfall runoff can be significant in southeast Louisiana even without the fuel of a tropical cyclone. The average annual rainfall for the New Orleans area is 60 inches. Nearly all runoff must be pumped out of the protected area, or basin, to prevent flooding. The interior drainage system is designed for removing stormwater from rainfall events, not removing water that enters the area from levee or floodwall overtopping or breaches.

Drainage systems in the New Orleans area are currently designed to convey, pump and store runoff for the 10-year rainfall event. Recent federal projects such as the Southeast Louisiana (SELA) Urban Drainage project have significantly improved capacity in some areas, but problems persist. As parishes such as St. Charles and Plaquemines are developed, the quantity of runoff increases and the time to peak flows decreases, which strains the existing municipal pumping systems.

Geology

As southeast Louisiana is made ideal for commerce by its access to the Gulf of Mexico via the Mississippi River, its largely sedimentary geology poses major challenges to engineers and urban planners. Soils in the area consist of geologically young sedimentary layers deposited by riverine flows during the last 10,000 years. Geologically, the area is predominately classified as Holocene alluvium and Holocene coastal marsh. Soils generally contain a high percentage of organics and voids and thus tend to compact when loads are applied. Except in select locations where firmer soils were deposited, structures are built with substantial pile foundations to resist settlement or must be designed to tolerate long-term settlement that could be in the range of several feet.

Hydrology

The hydrology of southeast Louisiana is dominated by several major features including the Mississippi River, Lake Pontchartrain, Lake Borgne, Lake Cataouatche, Lake Salvador and the Gulf of Mexico.

Bisecting Jefferson, Orleans, St. Charles and Plaquemines Parishes, the Mississippi River drains 64% of the continental U.S. The river stage at New Orleans peaked at 21.27 feet in 1922 and dropped to its lowest at minus 1.60 feet in 1872. The river's average elevation at the Carrollton gauge in New Orleans is about 6.8 ft.

Lake Pontchartrain covers about 630 square miles with depths of mostly 12 to 14 feet. During tropical events, storm surges can fill the lake and wind can push water up and over the existing hurricane protection system.

The Gulf of Mexico has a mean tide range of about 1 foot through most of coastal Louisiana. Tropical cyclones can push storm surges of significant height into Lake Borgne, Lake Pontchartrain and other lakes and bays with catastrophic results. Surges also propagate up the Mississippi River and have resulted in measured increases in river stages as far north as Baton Rouge.

Currently Authorized Projects

There are currently three authorized hurricane and flood protection projects in the report area of interest: Lake Pontchartrain, LA & Vicinity; New Orleans to Venice, LA; and West Bank & Vicinity, New Orleans, LA. While these projects have provided substantial protection, they are not designed to protect against storm surges produced by the most extreme hurricanes.

The area protected by these projects comprises only 5% of the land area of Louisiana, but includes 24% of the state's population according to the 2000 Census. The majority of the parishes of Orleans, Jefferson, St. Bernard, St. Charles and Plaquemines lie within the hurricane protection system. Approximately 458,000 housing units and 26,000 places of business are sheltered by the three currently authorized projects. Detailed information is provided below.

Parish	Land Area (square miles)	2000 Census Population	Housing Units	Places of Business
Orleans	181	484,674	213,134	10,628
Jefferson	307	455,466	189,539	12,694
St. Charles	284	48,072	17,835	885
Plaquemines	845	26,757	10,805	744
St. Bernard	465	67,229	27,078	1,191
Total	2,082	1,082,198	458,391	26,142
Louisiana	43,562	4,468,976	1,880,122	100,780
Percent of LA	5%	24%	24%	26%

The existing hurricane protection projects are shown in Figure 1 below. More detailed information on these projects may be found in this report within the individual sections pertaining to the projects.

Design Overview

The following is a more comprehensive summary of the Volume III technical information to supplement the Executive Summary found in Section 3.1. This information is organized by technical discipline. The specific design criteria by parish, basin, subarea, and/or reach of construction may be found within the individual sections of this volume.

Hydrology and Hydraulics

For each of the three hurricane protection projects, a design hurricane was selected that served as the basis for the hydrology and hydraulic design of the plan of each project. It was assumed that the design hurricane would approach a given project site from a critical path and at such rate of movement, to produce the highest hurricane surge hydrograph for that characteristic storm, considering pertinent hydraulic characteristics of the area. Critical paths were selected giving consideration to the paths of historical storms.

Figure 1. Hurricane Protection Projects in the New Orleans and Vicinity Area

For Lake Pontchartrain and Vicinity and West Bank and Vicinity, the Standard Project Hurricane (SPH) was selected as the design hurricane because of the urban nature of the project area. The Standard Project Hurricane was defined as a hypothetical hurricane intended to represent the most severe combination of hurricane parameters that is reasonably characteristic of a specified region. For New Orleans to Venice, the design hurricane was a hurricane that would produce a 100-year surge elevation or stage.

The U.S. Weather Bureau, now the National Weather Service, developed the SPH parameters based on historic hurricanes. The original U.S. Weather Bureau document "NHRP Report No. 33, Meteorological Considerations Pertinent to Standard Project Hurricane, Atlantic and Gulf Coasts of the United States, November 1959," covered a period of 57 years, 1900-1956. After Hurricane Betsy in 1965, the Weather Bureau revised the wind field parameters, but did not change the other characteristics of the SPH. These SPH meteorological parameters were used to design the Lake Pontchartrain and Vicinity project. Information from the SPH was also used to formulate meteorological parameters representative of the design storm utilized for the New Orleans to Venice project.

In 1979, a new report, NOAA Technical Report NWS 23, was published containing revised criteria for the SPH. The meteorological parameters of this SPH were used to design the West Bank and Vicinity project.

The concept of SPH, the derivation of the SPH meteorological parameters, the frequency assigned to SPH parameters and surge elevations, and the level of protection provided by the hurricane protection projects have been a source of confusion throughout the project history. The SPH hurricane is a steady state hurricane. The SPH index is based on an analysis of meteorological parameters of past hurricanes of record. Hurricane characteristics are correlated with intensity criterion, location, and other features. The central pressure index (CPI) was the principal intensity criterion for defining the SPH index. The 1% recurrence interval CPI was selected to define the SPH index.

The SPH storm was considered to have a recurrence interval of once in 100 years (1%) anywhere within Zone B. The probability of the SPH storm striking a smaller subzone within Zone B, such as the Lake Pontchartrain lakefront or Reach B2, would be less. A methodology was utilized to develop surge frequency curves that took into account the smaller geographic subzone area, historic observed surge data, and statistics on the direction of approach of a hurricane. It is from this methodology that the recurrence interval of the surge elevation was developed. This surge elevation recurrence interval was used to describe the frequency of occurrence of the SPH storm at the smaller subzone. Further clarification will be presented in the IWR report.

Maximum wind speeds were computed from the meteorological parameters using equations that included central pressure, radius of maximum winds, forward speed, and asymptotic pressure. A general wind tide equation was used to compute setup; this equation included wind speed, fetch length, average depth of fetch, angle between the direction of wind and fetch, a planform factor generally equal to unity, and a surge adjustment factor. This procedure was developed for an area along the Mississippi gulf coast where reliable data was available for several hurricanes to validate the methodology. Two historical storms, the September 1915 and September 1947 hurricanes, were used to establish and verify the procedure. To establish agreement between computed maximum surge height and observed high water marks, a calibration coefficient called the surge adjustment factor, was introduced into the equation. When the procedure was applied to the Louisiana coast, a third hurricane, occurring in 1956, and Hurricane Betsy, occurring in 1965, were used to verify the procedure.

For lakes such as Lake Pontchartrain and Lake Cataouatche, the wind tide level would be the sum of the surge, setup, tide, and runoff from rainfall. A method was developed to compute the water level associated with each factor. For Lake Pontchartrain, the method was validated using the 1947 hurricane. Moderate rainfall was assumed to be coincident with the storm. Mean normal tide was assumed to occur at the time of the storm. Setup and setdown were computed using modified step-method formulas.

For protection systems for Chalmette Extension and St. Charles East Bank portions of Lake Pontchartrain and Vicinity, and West Bank and Vicinity, marshlands were present that would be inundated for considerable distances from the coastline. A study was performed of available observed high water mark data along the Louisiana coastline for several storms from 1909 through 1965. The data indicated a consistent simple relationship between the maximum surge height and the distance inland from the coast. The weighted mean decrease in surge heights per mile was used to adjust the surge height at the inland locations.

For the three outfall canals in Orleans East Bank, design water levels were computed with steady state step-backwater calculations using HEC-2 Water Surface Profile computer program. The design flowline was based on existing channel geometry and assumed pump capacities that took into account future capacity and the stations' ability to pump during a hurricane. The starting water surface elevation for the models was the still water level in Lake Pontchartrain for the SPH condition. The HEC-2 models also incorporated modifications to some of the bridges at London Avenue and 17th Street Outfall Canals, such as raising bridge decks and constructing floodgates.

For surge along the Mississippi River, a bathystrophic storm surge technique was used to compute surge along the river. Surge hydrographs were computed for Hurricane Betsy and used to validate the procedure. A hypothetical hurricane isovel pattern based on 96 percent of the SPH winds was developed, transposed, rotated, and moved along tracks considered critical to five points along the river. Using these winds with the other SPH parameters, hurricane surge elevations were computed at the five points and used to construct a surge profile.

The design elevation for protective structures exposed to wave runup was an elevation sufficient to prevent all overtopping from the significant wave and waves smaller than the significant wave. Wave runup was computed and added to the maximum surge or wind tide level to get the design elevation. For Lake Pontchartrain and Vicinity and New Orleans to Venice projects, wave runup was calculated by methodology based on the interpolation of model study data developed by Saville, which related relative runup, wave steepness, relative depth, and structure slope. A modification to the methodology was made in some areas due to the presence of features that would modify the runup. For several protection system reaches, such as the South Point to Highway 90 reach of the New Orleans East protection system and lateral levee portions of the New Orleans to Venice project, no wave runup was considered. During the peak hour of the storm as the winds would be parallel to the protection system, and no wave runup would occur. For IHNC, and the portion of the GIWW west of Paris Road, waves were not considered a factor due to insufficient open water areas from which waves could be generated. For most of the outfall canal protection system, waves were not considered a factor due to entrance conditions, or, in the case of the 17th Street Outfall Canal, the recommendation to construct a breakwater. Where waves were not considered a factor, one, two, or three foot of freeboard was added to the maximum surge or wind tide level to get the design elevation.

For West Bank and Vicinity, some levees and floodwalls would be sheltered from storm generated wave runup; only small waves would be likely to occur. A small runup height was applied to these locations. Wave runup was calculated using methodology described in the 1984 Shore Protection Manual.

Geotechnical

General. The Hurricane Protection System included new and enlarged levees and floodwalls as well as numerous structures. To address the geotechnical design criteria for Volume III, numerous design and construction documents were reviewed. The geotechnical design criteria presented under each project is taken directly from the Design Memoranda and Soil Reports. As would be expected for documents prepared over a period of 40 years, the level of detail and the design emphasis on various aspects of design tended to vary with time. The information from the various construction documents reviewed is summarized under the individual projects.

Geology. The geological history and principal physiographic features of the New Orleans area as well as the surface and subsurface geology are described in Volume V. As stated in Volume V, the soils in the New Orleans area consist of Holocene age deposits of the Mississippi River deltaic plain underlain by sediments of the Pleistocene age from a much older deltaic plain. The Pleistocene is generally encountered 50 to 100 feet below sea level. The Holocene deposits generally have low to very low cohesive strengths, high to very high water contents and high to very high settlement potential. Pleistocene sediments in contrast have higher shear strengths, and lower water contents and settlement potential.

A map showing the surface geology in the general vicinity of New Orleans is presented in Figure 2. The surface deposits include a natural levee and point bar deposits (which are associated with the present course of the Mississippi River), inland swamp, fresh marsh, interdistributary and abandoned distributary channel. The point bar and abandoned distributary soils were deposited in a high energy environment and generally contain more coarse-grained sediments. Low energy environment deposits are composed primarily of clays and include inland swamp, fresh marsh and interdistributary.

The subsurface sediments in the New Orleans area include the following environments of deposition of the Holocene age: marsh/swamp, lacustrine/interdistributary, buried beach, abandoned distributary, prodelta, intradelta, near-shore gulf, estuarine and bay sound. The Pleistocene age deposits consist of clay top stratum and substratum sands and gravels.

The buried barrier beach ridge found in the New Orleans area was formed approximately 4,500 to 5,000 years ago and extends in the subsurface along the southern shore of Lake Pontchartrain. As shown on Figure 3, the buried barrier beach extends in an east – west direction across the entire Orleans East Bank and New Orleans East projects. Major project features that are crossed by the buried barrier beach include the 17th Street Outfall Canal, the Orleans Avenue Outfall Canal, the London Avenue Outfall Canal and the IHNC. The buried beach is encountered directly beneath the marsh/swamp deposits in some areas as near the surface as Elevation-10 feet and beneath lacustrine or prodelta deposits in some areas as deep as Elevation -30 to -35 feet. The thickness of the buried beach typically ranges from about 10 feet up to 30 to 40 feet, with the lesser thickness generally encountered in reaches where the upper surface of the buried beach is deeper.

In areas where suspected foundation failures have occurred, the soils involved have consisted of varying thicknesses of peat and/or weak clays overlaying sand and/or clay layers. The peat and/or weak clays have generally been marsh/swamp deposits and the clay layers have generally been lacustrine/ interdistributary deposits.

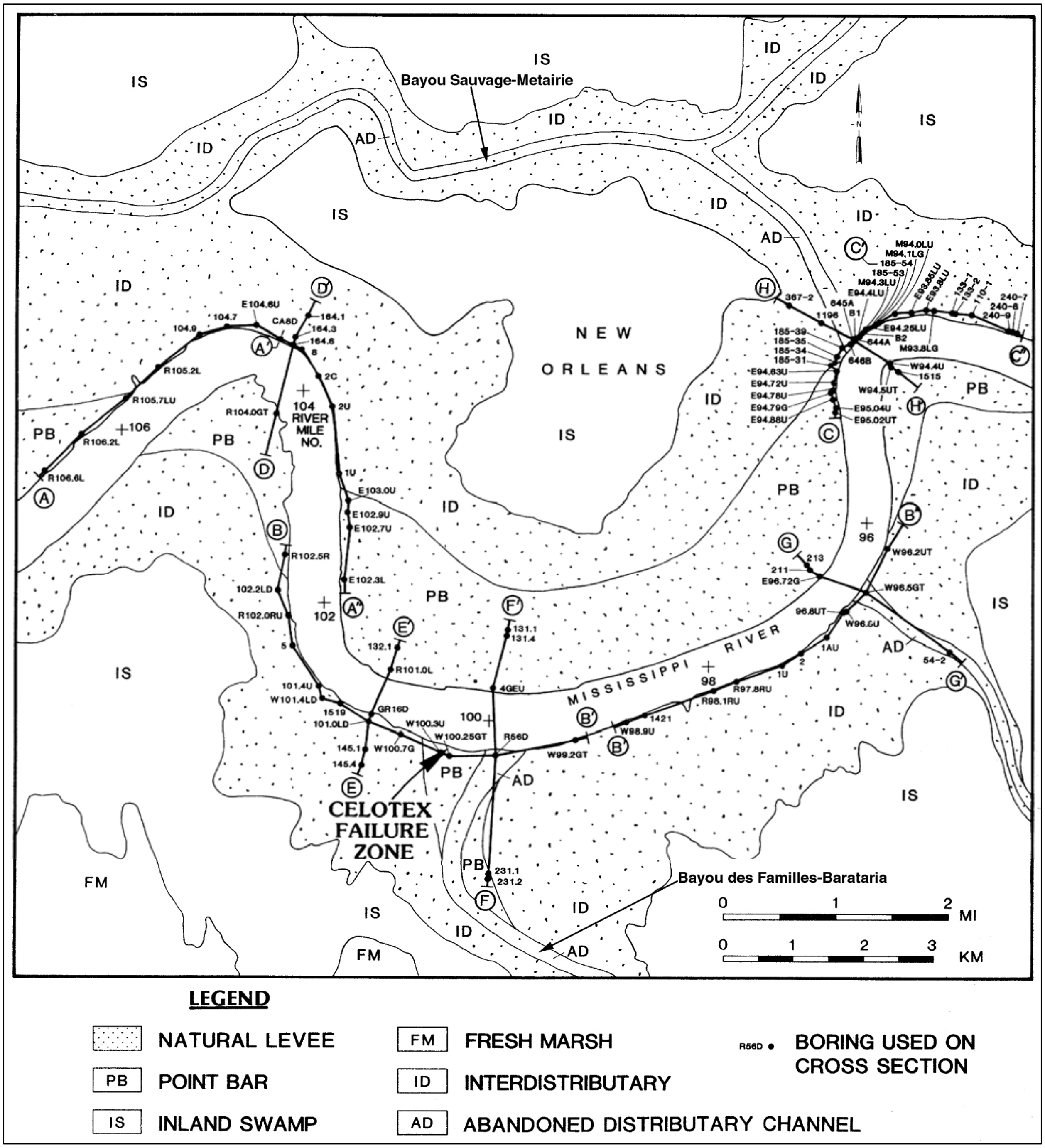

Figure 2. Surface Geology in the General Vicinity of New Orleans (from Figure 2-4, Volume V)

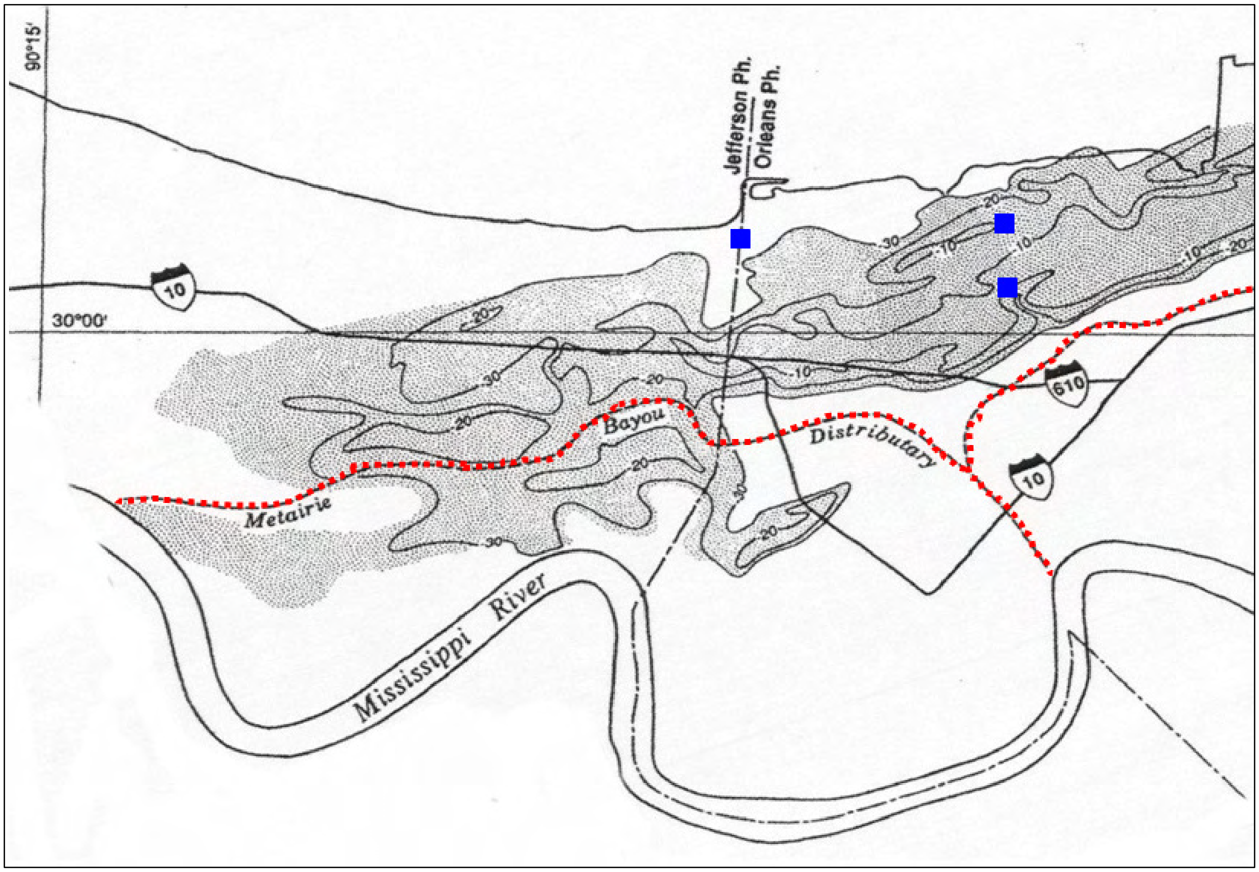

Figure 3. Close-up view of the buried beach ridge, and the locations of the canal breaches to the buried beach (after Saucier 1994). The 17th Street breach is located behind the axis of the beach ridge while the London Canal breaches are located on the axis of the ridge. Bayou Metairie is identified in red and forms the Bayou Sauvage distributary course (No. 11) in Figure 1-2 (from Figure 1-4b, Volume V)

Foundation Conditions. All of the hurricane protection projects were designed and constructed over relatively weak and compressible Holocene age deposits. In several areas, the natural ground surface was overlain by fill materials that had been placed in years past for various purposes. For the portions of the projects located nearest the Mississippi River, the natural soils directly beneath the ground surface consist of natural levee deposits of the Mississippi River alluvium. All of the remaining natural soils consist of deltaic deposits. The deltaic deposits are underlain by Pleistocene deposits. Brief descriptions of the various soil types follow:

a. *Fill Materials.* Where encountered, the fill materials are variable with respect to soil type and thickness. The soil types range from sands to clays, and some of the fill materials are indicated to have been hydraulically placed. As an example, fill materials are located on the northern 3,000 ft of the Orleans Avenue Outfall Canal where up to 20 ft of hydraulic fill materials were placed in the 1920's and early 1930's. The soil types included clays (CL & CH) silts (ML) and sands (SP, SM and SC).

b. *Natural Levee Deposits.* The natural levee deposits were encountered in the project areas nearest the present course of the Mississippi River. The deposits are thickest near the river and thin with distance from the river, and typically range in thickness up to about 20 ft. The ground surface in the areas where the natural levee deposits are thickest is generally the highest within the project areas. The soil types are variable and include clays (CL and CH), silts (ML) and silty sands (SM).

c. *Swamp/Marsh Deposits.* The swamp/marsh deposits generally comprise the surface layer except where overlain by fill and/or natural levee. The thickness averages about 10 ft but can range from as little as 2 or 3 ft to as much as 20 ft. The soil types generally include clays (CH) with organic matter, organic clays (OH) and peat. The consistencies are generally very soft to soft.

d. *Lacustrine, Interdistributary, Intradelta and Prodelta Deposits.* The swamp/marsh deposits are most commonly underlain by lacustrine, interdistributary, or prodelta deposits that can range in thickness from less than 5 ft to more than 20 or 30 ft. These deposits generally consist of high plasticity clays (CH), but occasionally have layers of lean clays, silts and silty sands. The consistency of the clays is generally very soft to soft except with depth where they can increase to medium stiff.

e. *Buried Beach Deposits.* The buried beach deposits are encountered primarily in the areas of the Orleans East Bank and New Orleans East projects as shown on Figure 3 and discussed in the Geology paragraph above. The buried beach sands are fairly pervious and depending on their depth below the surface can have a significant influence on underseepage.

f. *Abandoned Distributary Deposits.* The abandoned distributaries are also frequently encountered beneath the swamp/marsh deposits. These deposits can be fairly variable with respect to stratification and generally include clays (CL and CH), silts (ML) and silty sands (SM). The abandoned distributary deposits can range in thickness from 30 or 40 ft up to as much as 100 ft.

g. *Miscellaneous Other Deposits.* The above deposits are underlain by various other deposits including bay sound, nearshore gulf, intradelta, and estuarine. The prodelta deposits can also be encountered beneath the above listed deposits. The intradelta and nearshore gulf deposits are generally coarse-grained and the estuarine and bay sound deposits are generally fine-grained. The consistencies of the clays increase with depth from soft or medium to stiff depending on their depths.

h. *Pleistocene Deposits*. The Pleistocene deposits are variable with respect to soil types and include clays (CL and CH), silts (ML) and silty sands (SM). The consistencies of the clays are generally stiff to very stiff, and the relative densities of the sands are generally medium dense to dense. The Pleistocene deposits are encountered as shallow as Elev. -50 feet along the south shore of Lake Pontchartrain as deep as Elev. - 210 feet on the southern end of the New Orleans to Venice project.

The deposits that generally have the most influence on the shear stability of the levees and floodwalls consist of the swamp/marsh and the directly underlying lacustrine/interdistributary or prodelta deposits. All of these deposits generally have very soft to soft consistencies and are highly compressible. The natural deposits that generally involve underseepage considerations consist of the buried beach sands, particularly where they are encountered directly beneath the swamp/marsh deposits. Sand fill materials also require underseepage considerations. Figure 4 shows a cross-section of the more significant units that control the foundation, seepage and stability conditions.

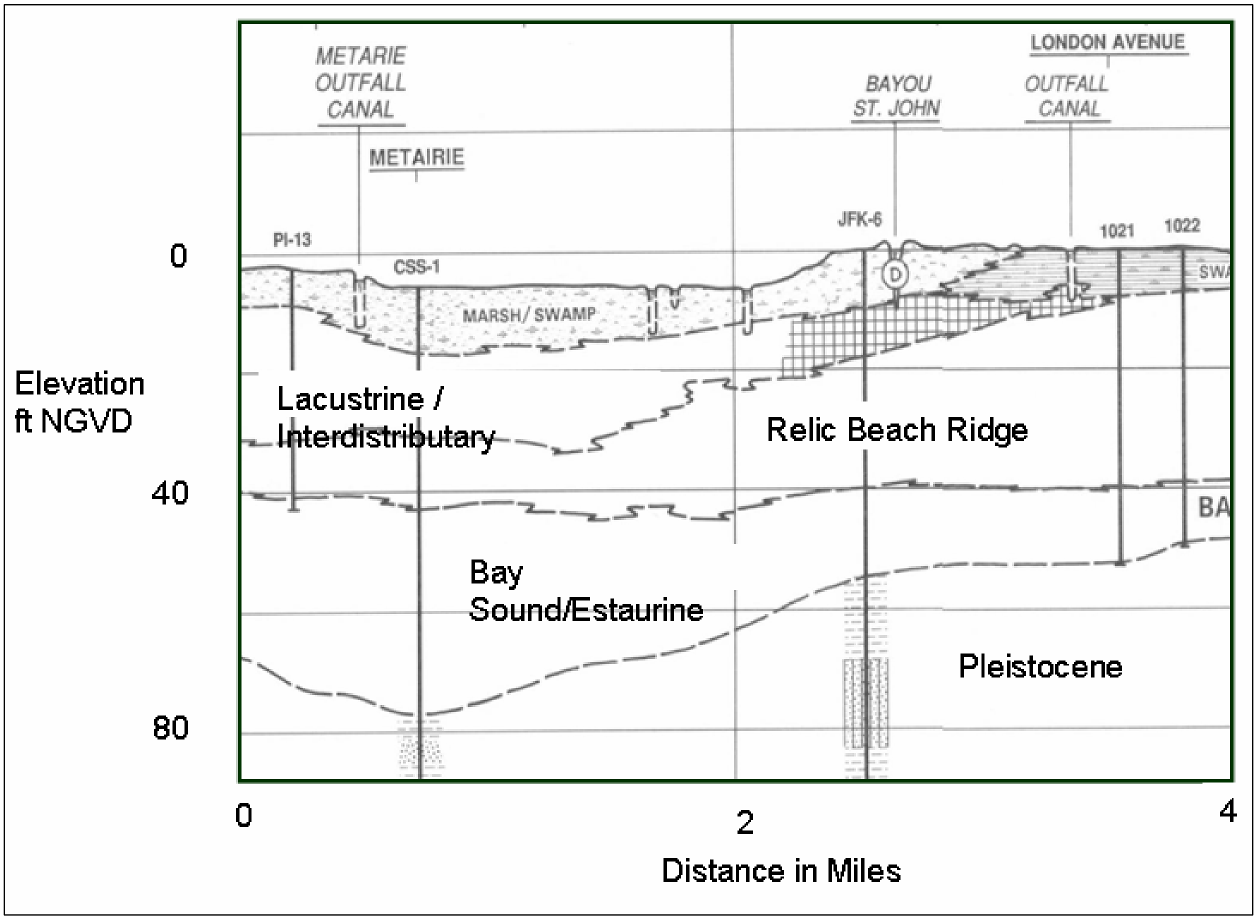

Figure 4. Portion of cross section C-C" from the Spanish Fort Quadrangle which extends through the 17th and London Canal breaches and identifies the stratigraphic environments in the subsurface (from Dunbar and others 1995) (from Figure 1-6, Volume V)

Field Exploration. Field exploration generally consisted of borings taken by both the U.S. Army Corps of Engineers and Architect-Engineers (A-Es). The A-Es were working for both the Corps and for the various local interest entities such as the Levee Boards and the New Orleans Sewerage and Water Board. The types of borings taken by Corps drilling crews or by an A-E working directly for the Corps usually consisted of 5-in. diameter continuous undisturbed borings taken with a 5 ft long fixed piston sampler and general-type borings using a 1 7/8 in ID core barrel or 1 3/8 in split spoon sampler. A-Es working directly for local entities usually obtained undisturbed samples using a 3-inch diameter Shelby tube sampler and general samples using a split-spoon sampler. The borings were taken at various spacings ranging from 350 to 1500 feet and were usually 50 to 80 feet deep with a few borings extending to a depth of 100 ft. Generally the borings were taken at spacings of 350 to 650 feet in the areas where floodwalls were to be constructed and on 700- to 1500-foot spacings in the more remote levee reaches. Borings were also taken during various stages of design. A number of the levees required building in stages to allow consolidation and strength gain in the foundation. During the subsequent stages of construction, additional borings would be made to confirm the predicted strength gains before an additional stage was constructed.

A number of projects involved design and construction of a Corps project over an existing levee or floodwall that had been designed and constructed by local interests. On these projects, the Corps would take the existing soils data that had been developed by an A-E working for the local interest and supplement the soils data with Corps borings. The Corps borings would generally be made both through the existing levee and at the levee toe, if right-of-entry were available. As an example, 14 continuous undisturbed 5-inch diameter soil borings and two general sample borings were made by the Corps for Design Memorandum No. 20 (Reference 2) for the 17th Street Outfall Canal. Seven of the Corps borings were made at the toe of the existing levee.

The Corps borings were supplemented by 77 borings that had been made for previous designs by an A-E employed by the New Orleans Sewerage and Water Board. The majority of the A-E borings were located along the levee centerline.

Laboratory Tests.

a. *Corps of Engineers Designs.* All samples obtained from-the-borings were visually classified. Water content determinations are made on all cohesive soil samples. Unconfined Compression (UC) tests, Atterberg limit tests and grain size analyses were then made on selected samples of cohesive and granular soils, respectively. Unconsolidated-Undrained (Q), Consolidated-Undrained (R), Consolidated- Drained (S) shear tests and Consolidation (C) tests were performed on representative undisturbed samples. Testing for Corps projects were performed both by the Corps and by A-Es working for the Corps. All of the laboratory test results were presented in the design documents until the middle 1990s, Q, R and S tests were performed at the LMVD (Division) laboratory at WES (now ERDC). After the Division laboratory downsized, the (Q), (R) and (S) tests and consolidation tests were performed at local A-E labs.

b. *Designs for Local Interests.* The laboratory testing performed by A-Es for local interest projects was generally fairly comparable in scope to the testing that would be performed by the Corps. The results of the laboratory tests were provided in the A-E-prepared design documents.

c. *Selection of Shear Strengths.* When a geotechnical analysis or soils report was prepared for a project, the project would be divided into design reaches based on similar geology and soil stratification. The undrained test results, UC and Q were plotted versus depth. The three point unconsolidated-undrained (Q) results were relied upon most heavily to select a design envelope for the specific reach. The designer would usually select a conservative envelope based on the test results, past experience and judgment. The stability analyses were performed using the undrained or (Q) case design envelope for the CH and CL clays. For sands (SP and SM), the values of $\phi = 30^\circ$ and C = 0 psf were normally used. For silts (ML), the values of $\phi = 15^\circ$ and C = 200 psf or $\phi = 28^\circ$ and C = 0 psf were normally used. The soil classifications of CH, CL, SP, SM and ML are from the Unified Soil Classification System.

Underseepage. Design for protection against underseepage was not required for most of the reaches of the Hurricane Protection System because the majority of the foundation soils consist of high plasticity clays. However, two types of foundation conditions were encountered in the New Orleans Projection System that presented potential underseepage problems and these conditions were addressed in the designs performed by the Corps. The two foundation conditions requiring underseepage designs were: (1) reaches underlain by the buried barrier beach ridge and (2) a few reaches where enlargements were designed by the Corps for existing local interest levees that were constructed with dredged sand bases, or over existing sand fill materials. The project features that cross the buried barrier beach ridge are the 17th Street Outfall Canal, Orleans Ave Outfall Canal, London Ave Outfall Canal, Orleans Parish Lakefront Levee and the Inner Harbor Navigation Canal (IHNC) East and West Levees and Floodwalls. A reach of the Citrus Lakefront Levee was built on an existing sand fill base, and a reach of the London Avenue Outfall canal was constructed over hydraulic fill that included sands.

The field investigation generally included piezometers to determine whether or not the piezometric pressures in the sands indicated a direct connection with the water level in the adjacent canal or channel. In a few instances, the piezometers had been previously set and data had been collected by an A-E working for the local entity. For most of these projects, the Corps set piezometers or collected data from existing A-E piezometers. It was generally concluded that the design for underseepage protection should consider that the piezometric pressures in the sands were directly influenced by the water level in the adjacent canal or channel. A piezometric grade line was then computed for the sands that reflected the S. W. L. However, in at least one instance it was concluded from the piezometer data that there was not a direct connection between the sands and the canal due to sedimentation in the canal.

The primary design criteria used in underseepage analyses was obtained from WES Technical Manual No. 3-424 (Ref 80). The method of underseepage analysis typically utilized was the creep ratio method using Lane's weighted creep ratio (Reference 80). Some analyses were also performed using flow nets and the method of fragments. Based on the underseepage analyses performed, the sheet piling were extended to obtain an adequate creep length or factor of safety against piping. In several instances, the penetrations of sheet piling were extended as required through the pervious zone to provide a cutoff. On the Citrus Lakefront Levees sheet piling were used to provide a partially penetrating cutoff to decrease the uplift at the landside toe of the levee as required to provide a computed factor of safety of 1.3.

Relief wells were used on the East and West Levees and Floodwalls of the IHNC. The relief well design was performed using the projected piezometric heads for the design hurricane in conjunction with the criteria that is found in EM 1110-2 – 1905 dated 1 March 1965 (Ref 15) to design the well spacing and discharge.

Pile Foundations. The typical procedures for design of pile foundations was to develop curves providing ultimate compression and tension capacities versus tip elevation for the piles expected to be used, i.e. 12 in. square pre-stressed piles, 12 in. steel H-piles, etc. The pile curves were developed based on the boring data. For calculating unit side (skin) resistance in sands, effective overburden stresses were assumed to reach a maximum value at a normalizing depth of D/B = 15. For the S Case analyses in clays, the unit side resistance was limited to a maximum value of 2.0 ksf. The design pile loads versus tip elevations presented in the DMs were computed for cost estimating purposes and were normally based on a factor of safety of 3.0 if no pile load test data were available and a factor of safety of 2.0 if pile load test data were available from previous pile load tests on the project. During construction, test piles were generally driven in the project area and tested, and the pile capacities used in final design were based on a factor of safety of 2.0. Subgrade moduli curves for estimating lateral resistance of the soil beneath the structure were generally developed and provided in the DMs.

Slope Stability. The method of slope stability analysis preferred by the New Orleans District is referred to as the Lower Mississippi Valley Division (LMVD) Method of Planes. The LMVD Method of Planes resolves the forces into horizontal components and computes the factor of safety as the ΣDriving Forces ÷ ΣResisting Forces. The LMVD Method of Planes is a wedge method that was developed in the 1950's and is based on equilibrium of a soil mass above a slip surface that consists of an active wedge, a neutral block and a passive wedge. Although the method does not satisfy all conditions of static equilibrium, it has been demonstrated to be a conservative method of slope stability analysis that results in safety factors as low or lower than more modern methods that do satisfy all conditions of static equilibrium.

The slope stability analyses for the Hurricane Protection project were performed using short-term or unconsolidated-undrained (Q) shear strengths with a minimum acceptable factor of safety of 1.3. The only exception encountered in the design documents where the LMVD Method of Planes was not used occurred where high strength geotextile was used to reinforce the embankment and shorten the time for stage construction. Spencer's Method with the PC-slope computer program was used to check the critical failure surface. This was done because Spencer's Method considered the location of the geotextile in determining the required geotextile tensile strength.

For both I-walls and T-walls on piling, the global stability of the walls was analyzed using the LMVD Method of Planes and a minimum factor of safety of 1.3 for the short-term (undrained) (Q) case.

Sheet Pile Wall/Cantilevered I-Type Floodwall Design Criteria. The cantilever/I-type floodwalls were analyzed using the Method of Planes and developed shear strengths. Net lateral water and earth pressure diagrams were determined for movement toward each side of the sheet pile using the developed shear strengths. Using these distributions of pressure, the summation of horizontal forces was equated to zero for various tip elevations. At these penetrations, summations of overturning moments about the bottom of the sheet pile were computed. The required depths of penetration to satisfy the stability criteria were determined as those where the summation of moments was equal to zero. The water level on the hurricane side was set per the design storm and water level criteria. The water level on the protected side considered a water level equal to the ground water table assuming the water table at the ground surface. Where the ground surface was below Elev. zero, the water table was taken at Elev. zero. In those areas where the buried beach sand was near the surface, Factors of Safety were determined for the headwater level at the top of the wall and for a high tail water conditions representing the design piezometric grade line in the buried beach sand reach.

Levee Borrow Material. Except where dredged or hauled sand was used as a base or working platform along the levee alignment, the standard practice was to specify that all levee fill consist of CH, CL or ML as classified by the Unified Soil Classification System. In those cases where dredged sand or sand bases were used, it was encapsulated with a 2-ft to 4-ft thick clay blanket. Locating a close-by source of the CH, CL or ML materials was frequently difficult. Many of the earlier DM's stated that borrow would be available from a pit in the bottom of Lake Pontchartrain on the North Shore known as the Howze Beach pit. One contractor did try to use that pit and had considerable difficulty. The majority of the levee borrow material on the Lake Pontchartrain and Vicinity projects (Orleans East Bank, New Orleans East, St. Bernard, Jefferson East Bank and St. Charles East Bank) came from either a government-furnished pit in the Bonnet Carré spillway or a contractor-owned pit in New Orleans East known as the Highway 90 pit. The borrow material for New Orleans to Venice and West Bank and Vicinity came from closer sources.

There were a number of gap closures in the levees such as at Bayou Bienvenue and Bayou Dupre. These gap closures were made with small oyster shells that were lightweight and easy to use in a closure fill. They were then capped with 2 feet to 4 feet of clay.

Erosion Protection. The design for erosion protection anticipated short duration hurricane floods and some wave over topping, but nothing to the degree that occurred along the MRGO – GIWW and in the New Orleans to Venice area. The designers anticipated on the lake shore and along the Mississippi River that wave protection would be required, but that the resistant nature of the clayey soils would limit the need for erosion protection elsewhere.

Independent Technical Review of Design. Until the middle 1990s, design documents such as Design Memoranda and Detailed Soil Reports were submitted to LMVD for an Independent Technical Review of the designs. The reviews were documents in a series of endorsements between the District, Division and in some cases, the Office Chief of Engineers (OCE). Independent Technical Reviews were handled after that by the Districts either with In-House assets, by other Districts, or through A-Es. This was done because of the changing mission of the Divisions and OCE and the implementation of the Project Management Business Process.

Structural

The structural design of the hurricane protection structures followed the current applicable industry codes such as American Institute of Steel Construction (AISC), American Concrete Institute (ACI), and American Association of State Highway and Transportation Officials (AASHTO) pertaining to the various project features. These code provisions were supplemented by the current Corps of Engineers more conservative criteria for hydraulic structures applicable at the time of design as promulgated in published engineering manuals and other Corps of Engineers design guidance documents. Also, the design work used generally consistent assumed unit weights of materials and dead loads, wind loads, and vertical live loads, where applicable in the design of gate closure structures.

There were some differences in the materials specified and used in construction from early to later projects, resulting from the evolution of materials available to the designer and construction contractor. Domestic hot rolled steel sheet piling, conforming to ASTM A-328, was commonly used in construction on hurricane protection projects before the 1990's. The exception was foreign hot rolled sheet piling on projects constructed by local interests. Beginning in the 1990's, domestic cold and hot rolled sheet piling, conforming to ASTM A328 or ASTM A572, Grade 50, was allowed as a substitute. This substitution began several years earlier on Atchafalaya Basin and Mississippi River projects. Initially, concern about the thickness, width, and depth of cold rolled as compared to hot rolled sheet piling existed. However, it was determined that the loss of section due to corrosion of the slightly thinner cold rolled piles had negligible impact on the life expectancy of the project. Limiting variance in width and depth of the sheet pile was adopted to ensure ease of handling and also to maintain concrete dimensions. Many projects, most notably along London and Orleans canals, were constructed using the cold rolled sheet piling substitution. In the late 1990's, Federal constructed projects began using hot rolled foreign steel because domestic hot rolled sheet pile was no longer being produced. Recently, as domestic hot rolled mills resumed operations, foreign steel sheet piling is used only under special circumstances. Although sheet piling conforming to ASTM A328 is still permitted, piling conforming to ASTM A572, Grade 50 steel is currently more commonly used.

Early projects were designed using the then concrete industry standard, $f'c = 3{,}000$ psi with Grade 40 reinforcement. In the later projects, as higher strength materials became more common in the industry, designs for hurricane protection projects transitioned to $f'c = 4{,}000$ psi strength concrete with Grade 60 reinforcement, although the higher strength concrete was not universally adopted in the later designs. Reinforced concrete designs transitioned from Allowable Working Stress method of analysis to Load Factored Design after this became the standard in the American Concrete Institute design code. Typically, concrete with a strength of $f'c = 5{,}000$ psi was used for prestressed concrete piles with either Grade 250 or Grade 270 prestressing strands. In addition, design of prestressed concrete piles changed from stress-relieved strands to low-relaxation strands consistent with industry practice.

The designs of the steel sheet piling for I-walls to determine a depth of penetration, bending moment and defection followed the classical cantilever limit equilibrium fixed end method. This classical method is based on the premise that equilibrium of the wall requires that the sum of horizontal forces and the sum of moments as a result of lateral pressures on the wall about any point must both be equal to zero. Pile foundations for T-walls were analyzed using methods outlined in "Analysis of Pile Foundations with Batter Piles," by Hrennikoff. (Reference 56)

Some variations did occur in the loading conditions, factors of safety and sheet pile penetration ratios for I-wall designs. To calculate bending moments and flexural deflections, soil pressures were calculated using unfactored soil strengths in most cases, to provide the most realistic estimate of actual loads on the steel. However, there were some instances, particularly in the earlier designs, where a factored soil pressure was used in these calculations. A dynamic wave impact loading case was included in the designs for I-walls and T-walls considered exposed to wave conditions, such as along lake front areas, as opposed to walls paralleling canal areas where a wave loading case was not part of the analysis. In design work prior to December 1987, sheet piling for I-walls, with anticipated exposure to wave conditions, was analyzed for a dynamic wave impact load case with a FS = 1.25, to determine penetration of sheet piling either as a single load case or in combination with a load case at the SWL and a FS = 1.5. On other design work, a FS = 1.5 was used for a case with static water to SWL plus freeboard, static water to top of wall condition, or static water to 6 inches below top of wall condition. Typically, sheet pile penetrations for I-walls designed prior to December 1987 were determined on the basis of the "S" shear strengths with a FS = 1.5.

On 23 December 1987, the Mississippi River Commission (MRC) issued criteria guidance (Reference 76) to the New Orleans District on sheet piling design based on a sheet pile wall field load test, commonly referred to as the E-99 test. The field load test is documented in Technical Report No. 1, E-99 Sheet Pile Wall Field Load Test Report. (Reference 79). This Technical Report concludes that the sheet pile penetration design procedure, which is based on the S-case analysis and a factor of safety of 1.50, would be too conservative for design of the test section wall. It further stated that sheet pile penetrations determined using the S-case analysis (FS = 1.2) should be adequate to provide satisfactory limit equilibrium stability and to avoid excessive deflections. It also recommended that the New Orleans District's arbitrary limiting deflection criteria of 3 inches of estimated lateral flexural deflection be re-evaluated noting that actual lateral movement will most likely be in the levee foundation not flexural deflection.

On the basis of the test data, the 23 December 1987 guidance recommended the following design criteria:

Q-Case

- F.S. = 1.5 with water to flowline or SWL
- F.S. = 1.25 with water to freeboard (net levee grade) for river levees or with SWL and waveload for hurricane protection levees

S-Case

- F.S. = 1.2 with water to flowline or SWL + wave load (if applicable) for hurricane protection levees
- F.S. = 1.0 with water to freeboard (net levee grade) for river levees

In addition, the 23 Dec 87 guidance stated that if the penetration to head ratio is less than about 3:1 increase it to 3:1 or to that required by the S-case, F.S. = 1.5, whichever results in the least penetration.

The MRC restated essentially the same criteria in a 24 July 1989 memorandum to the New Orleans District (Reference 77) as follows:

Q-Case

- F.S. = 1.5 with water to flowline or SWL
- F.S. = 1.25 with water to flowline plus approved freeboard for river levees or with SWL and waveload for hurricane protection levees
- F.S. = 1.0 with water to SWL +2.0 ft. freeboard for hurricane protection levees

S-Case

- F.S. = 1.2 with water to flowline or SWL and waveload. If a hurricane protection floodwall has no significant waveload, determine the penetration using the Q-case criteria only.
- F.S. = 1.0 with water to flowline plus approved freeboard for river levees

To ensure adequate penetration to account for unknown variations in ground surface elevations, the July 1989 guidance stated that penetrations should be arbitrarily increased, as necessary, to achieve a penetration to head ratio (for flowline or SWL) of about 2.5 to 3:1. Also, it stated that the estimated sheet pile flexural deflection should no longer control selection of the sheet pile section for walls in soft clays.

There are three major differences between the two documents. Table 1 contains a summary of the sheet piling penetration criteria from the two documents. The 24 Jul 89 guidance:

1. Added that if a hurricane protection levee has no significant waveload, determine the penetration using Q-case criteria only.
2. Changed the penetration to head ratio criteria to “about 2.5 to 3:1”.
3. Stated that sheet pile flexural deflections should no longer control selection of sheet pile sections in soft clays.

The subsequent I-wall designs appear to comply with either the Dec 87 criteria or the Jul 89 criteria concerning penetration to head ratio criterion. The design information for the Orleans Canal and London Canal Parallel Protection Plans as well as recent design in St. Charles Parish and the West Bank and Vicinity Project include a check of the penetration to head ratio of at least 3:1 unless this exceeds the penetration required by a S-case, F.S. = 1.5. This follows the Dec 87 criterion, whereas the design for the 17th Street Parallel Protection incorporated a penetration to head ratio of 2.5 to 1 as recommended in the Jul 89 criteria.

Table 1
Sheet Piling Penetration Criteria Summary

	Penetration Factor of Safety		
	Q-Case	**S-Case**	**P / H Ratio**
Ref 76 - CEMRC-ED-GS Memorandum For: Commander New Orleans District. ATTN: CELMV-ED-F, Subject, “Sheet Pile Wall Design Criteria” dated 23 December 1987.	FS = 1.5 water to SWL FS = 1.25 water to SWL and waveload	FS = 1.2 water to SWL and waveload FS = 1.0 water to fbd	3:1 or that required by S-Case, FS = 1.5, whichever results in least penetration
Ref 77 - CEMRC-ED-GS Memorandum For: Commander New Orleans District. ATTN: CELMV-ED-F, Subject, “Sheet Pile Wall Design Criteria” dated 24 July 1989.	FS = 1.5 water to SWL FS = 1.25 water to SWL and waveload FS = 1.0 water to SWL plus 2 ft fbd	FS = 1.2 water to SWL and waveload FS = 1.0 water to fbd	2.5 to 3:1

Table 2 summarizes how this criteria was incorporated into the I-wall designs for the various project components to determine depth of penetration and, where used, penetration to head ratio.

Table 2
I-Wall Design Criteria

Location	Penetration Factor of Safety	P / H Ratio
Orleans East Bank		
Orleans Lakefront - Orleans Marina	Q & S 1.5	
Orleans Lakefront - West of IHNC	Q & S 1.5, 1.25 (w/ waves)	Lane's Creep 3.0 to 8.5
17th Street Outfall Canal	Ref 77 (no waves)	Ref 77
Orleans Ave Outfall Canal	Ref 77 (no waves)	Ref 76
London Ave Outfall Canal	Ref 77 (no waves)	Ref 77
Pontchartrain Beach Levee and Floodwall	S 1.5, 1.25 (w/ waves)	
Bayou St. John Closure	S 1.5, 1.25 (w/ waves)	
IHNC West - Remaining Levees	S 1.5	
IHNC - France Road Terminal	Q 1.5 S 1.3 (SWL) , Q 1.5 S 1.0 (w / 2ft fb)	3:1
IHNC West - Florida Avenue to IHNC Lock	S 1.5	
IHNC - Florida Avenue Complex	S 1.5	
New Orleans East		
Citrus Lakefront IHNC to Paris Road	S 1.5	Lane's Creep 7.0
New Orleans East Lakefront Paris Road to South Point	S 1.25 (w/ waves)	Lane's Creep 2.5
New Orleans East South Point to GIWW	S 1.5, 1.25 (w/ waves)	
New Orleans East Back Levee	S 1.5, 1.25 (w/ waves)	
Citrus Back Levee - West of Paris Road	S 1.5	
Citrus Back Levee - East of Paris Road	S 1.5, 1.25 (w/ waves)	
IHNC East - Remaining Levees	S 1.5	
St. Bernard Parish		
Chalmette Area Plan	S 1.5	
Chalmette Area Plan - Bayou Bienvenue and Bayou Dupre Control Structures	Q & S 1.5, 1.25 (w/ waves)	
Chalmette Area Plan - Chalmette Extension	Q & S 1.5, 1.25 (w/ waves)	
Jefferson East Bank		
Jefferson Lakefront	Q & S 1.5, 1.25 (w/ waves)	
Jefferson Return Levee	Q & S 1.5, 1.25 (w/ waves)	
St. Charles East Bank		
St. Charles - North of Airline Highway	Q 1.5, 1.0 (w / 2ft fb) S 1.2	3:1 S case
New Orleans to Venice		
Reach A - City Price to Empire (Tropical Bend)	S 1.5	
Reach B-1 - Empire (Tropical Bend) to Fort Jackson - Floodgate at Empire	S 1.25 (w/ waves)	
West Bank Mississippi River Levee - City Price to Venice	S 1.5 (SWL w/ waves)	
Reach B-2 - Fort Jackson to Venice - Floodwall	S 1.5, 1.25 (w/ waves)	
Reach C - Phoenix to Bohemia - Floodwall	No design analyses	
West Bank & Vicinity		
Lake Cataouatche - Adjacent to Lake Cataouatche Pumping Stations 1 and 2	Ref 77 (w/ waves)	3:1
Lake Cataouatche - Station 518+00 to Bayou Segnette floodwall	Ref 77 (no waves)	3:1
Westwego to Harvey Canal Area	Ref 77 (w/ waves)	Ref 76
Westwego to Harvey Canal Area, Cousins Pumping Station	Ref 77 (no waves)	
East of Harvey Canal Area - East and West of Algiers Canal	Ref 77 (no waves)	3:1

Levee Construction Overview

Construction Overview

General. The construction overview is an attempt to give the reader an overview of the information on the compliance with the specifications and as-built conditions, as well as other criteria related to the construction of levees and floodwalls of the Hurricane Protection System.

Construction Documents. During the execution of the construction contract, the contractor maintains daily quality control (QC) reports. The Constructor is responsible for providing such records as form checkout sheets for concrete structures, site testing data for concrete, pile driving records, in-place density tests, minutes of preparatory inspection meetings and daily dewatering reports. These records, if applicable, are attached to the Daily QC Report. The Corps Construction representative prepares the Daily Government Quality Assurance (QA) reports. These reports are normally filed and stored together. The QC reports normally follow a government suggested format. The QC and QA Reports usually cover the same items. The general information about weather conditions for that day, the numbers of laborers and supervisors on the job, hours worked and the operating equipment on the job. There is also a statement of what work was performed on the job that day. There are paragraphs to cover the results of the controlled activities, such as preparatory, initial, and follow-up meetings and inspections; and for tests performed that day, as required in the plans and specifications. There are paragraphs for materials received, submittals reviewed, off-site surveillance activities, job safety, environmental protection, and a general remarks paragraph.

A lot of the same information is covered in the QA Reports. The items /sections listed on the QA report usually are as follows: general information about the weather conditions for that day, the number of contractor and government employees on the job, the prime contractor and the subcontractors on the job and their responsibilities, and description of the work performed that day. There are sections for days of no-work and reasons for the no-work, and progress of the work. There is information on CQC inspection phases attended, instructions given, and results of QA inspections and tests, deficiencies observed and actions taken, and corrective action of contractor. There are sections for verbal instructions given the contractor that day, for controversial matters that may have arisen, for information, instructions, or actions taken not covered in QC reports or disagreements, safety, and a section for remarks.

At the end of the contract, a completion report is prepared that lists among other things all modifications, changes and claims related to the contract. Once the contract is completed and release of claims is granted by the contractor, the records of the project are boxed up and sent to off-site storage where they remain for six years. After six years, they are destroyed. At the same time the records are being boxed and sent to storage, a copy of the completion report along with a marked up set of as-built drawings are forwarded to Engineering Division to maintain.

Review of Construction Documents. As part of Vol. III, the construction files on as many as 50 construction contracts were reviewed. The review identified what records were available and if any modifications, changes or claims were documented that showed if the design intent was changed or whether there were changed conditions claims that show the soil conditions as fundamentally different than what was presented in the construction documents. The results of this review are shown in the report under the individual projects. Of the over 50 construction documents reviewed, five showed modifications or changes. Four could be considered as different site conditions. (paragraph 3.2.1.5.4.1.10) 17th Street Canal East Side Stations 0+96.27 to Station 7+00 cut off 4′ 3″ of sheet pile because of unanticipated hard driving. The sheet pile ended up short of the desired penetration. The second instance (paragraph 3.2.1.5.4.1.22) 17th Street Outfall Canal, Hammond Highway Complex required modification of sheet pile cofferdam because of excessive settlement of one side because of encountering an extremely soft layer of clay. The third instance (paragraph 3.2.1.6.4.1.3) South Point to GIWW Levee required a redesign of the levee and berm configuration due to sliding. The fourth instance (paragraph 3.2.1.9.4.1.1) St. Charles Parish North of Airline Highway required modifications to remove pile driving obstructions. The fifth instance (paragraph 3.2.1.8.4.1.1) was Jefferson Parish Lakefront Levee, Pump Station No. 2. Because of a survey error, the breakwater was realigned 70 feet to the west. In driving the sheet pile for the breakwater, an obstruction was encountered and the sheet piles were cut off.

Levee Construction. Prior to the late 1980’s, all HPS levees were constructed using semi-compaction and a specified moisture content range based on the soil type in the borrow area. The semi-compaction specification was generally referred to as a performance specification. This specification required spreading the borrow materials in 12-inch maximum thickness lifts and compacting with three passes of a dozer. After the late 1980’s, an end result type specification was used. The end result specification required that the levee materials be spread in 12-inch maximum thickness lifts and compacted to not less than 90 percent of standard Proctor (ASTM D 698) maximum dry density at moisture contents not greater than 5 percent above nor less than 3 percent below the optimum moisture content as determined from the compaction tests.

All seepage berms and stability berms, with some exceptions, were constructed as uncompacted fill. The uncompacted specifications require the borrow materials to be spread in lifts not greater than 3 feet in thickness. No specific compaction is required.

3.2.1. Lake Pontchartrain & Vicinity

3.2.1.1. General Description

The Lake Pontchartrain, LA and Vicinity Hurricane Protection Project (HPP) covers St. Bernard, Orleans, Jefferson and St. Charles Parishes in southeast Louisiana, generally in the vicinity of the city of New Orleans, and between the Mississippi River and Lake Pontchartrain. The Orleans East Bank portion of the project includes the east bank of the Mississippi River between the 17th Street Outfall Canal and Inner Harbor Navigational Canal (IHNC). Figure 5 is an index map showing the individual polders or sub-basins within the Lake Pontchartrain, LA and Vicinity HPP.

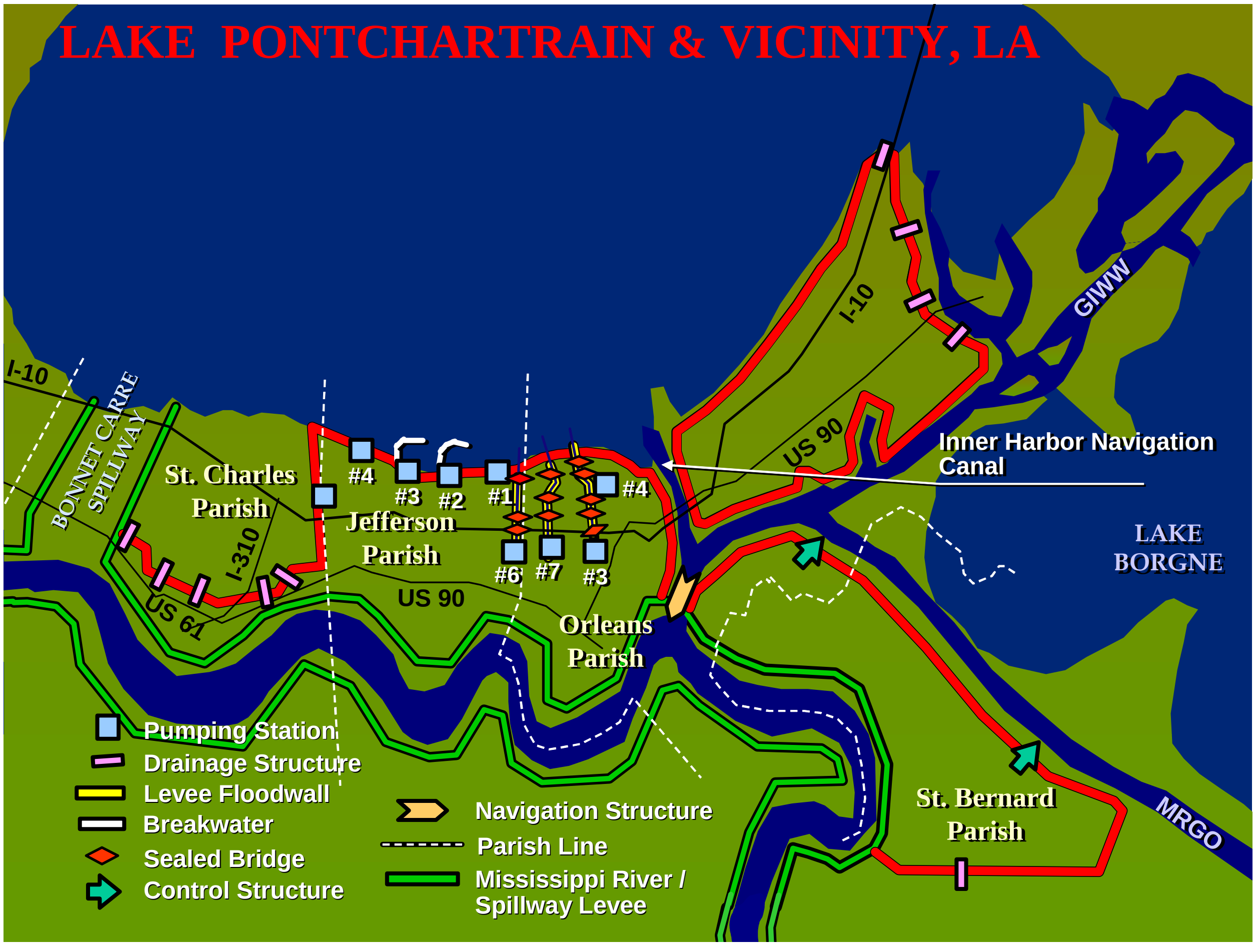

Figure 5. Index Map to Lake Pontchartrain, LA, and Vicinity Hurricane Protection Project

The Pre-Katrina project condition or construction status for each polder is described within their respective section of this report. However, in general, the relatively poor soils in southeast Louisiana impact the methods used to construct levees. Construction of levees usually requires several lifts. A lift is simply a reconstruction or raising of a previously constructed levee to account for localized subsidence and compaction of the earthen structure over time. In some cases the first lift cannot be constructed to the design elevation because of the underlying soil conditions. In that case, the levee is constructed to an interim elevation to load and compact the subsurface soils. Subsequent lifts would be constructed to the design elevation. In areas where the first lift can be constructed to the design elevation, subsequent lifts will be constructed to restore this elevation once settlement has occurred.

During the design phase of a project, geotechnical engineers estimate the number of lifts required for various reaches of the project. This estimate is based on soil conditions in the area and the information is used to estimate the cost of the project. For the Lake Pontchartrain and Vicinity project, it was estimated that as many as four lifts would be required for many reaches of the levee system. Once the initial lift is constructed, profiles are taken of the levee usually on an annual basis to determine the rate of subsidence. While there is no definitive yardstick for deciding when another lift will be constructed, the general policy has been that if a foot of settlement has occurred then preparation of plans and specifications should begin.

As the lifts are constructed over time, the rate of subsidence generally tends to decrease. This means that the length of time between levee lift may increase, and the amount of material required to raise the levee will decrease. For the Lake Pontchartrain and Vicinity project, the initial through the final lift includes an additional amount of fill called overbuild. This material is used to raise the levee somewhat higher than the design elevations to account for shrinkage and long-term subsidence

3.2.1.2. History

The Lake Pontchartrain, Louisiana, and Vicinity Hurricane Protection project originated from an act of Congress, approved June 15, 1955, that authorized examinations and surveys of the coastal and tidal areas of the eastern and southern United States susceptible to damage from hurricanes.

Subsequently, the New Orleans District submitted the Interim Survey Report, Lake Pontchartrain, Louisiana, and Vicinity on November 21, 1962. To prevent large tidal surges from entering Lake Pontchartrain during the approach of hurricanes from the Gulf of Mexico, the survey report advocated a hurricane protection plan that consisted of a barrier at the eastern end of the lake, complete with tidal and navigation structures in the Rigolets and Chef Menteur Pass and a dual purpose navigation lock in the Inner Harbor Navigation Canal (IHNC) at Seabrook. The plan posited in the survey report also recommended new or enlarged protective works fronting the developed or potentially developable areas along the lakefront.

The survey report served as the genesis for what came to be known as the Barrier Plan. On March 4, 1964, the Chief of Engineers sent a report to the Secretary of the Army that recommended the construction of the eastern lake barrier and barrier complexes, as well as new lakeshore levees in St. Charles Parish, Citrus, and New Orleans East, and the enlargement of existing protective works in Jefferson Parish, Orleans Parish, and at Mandeville. The report also recommended the authorization of a separate plan for the Chalmette area that included improvements to the levees flanking the IHNC and the construction of new levees along the south side of the Mississippi River-Gulf Outlet (MRGO) from the IHNC to Bayou Dupré and on toward Violet.

The 1965 Flood Control Act (Public Law 89-298) authorized the Lake Pontchartrain, Louisiana, and Vicinity Hurricane Protection project generally in accordance with the recommendations contained within the report of the Chief of Engineers. Upon the receipt of funds in 1966, construction of the hurricane protection project began. In accordance with the National Environmental Policy Act of 1969, the New Orleans District completed and submitted an environmental impact statement for the project. The adequacy of the environmental impact statement was challenged in court, and, on December 30, 1977, the U.S. District Court, Eastern District of Louisiana, enjoined the New Orleans District from constructing the barrier complexes, the New Orleans East levee system, and the Chalmette area plan (which had since been extended southward along the MRGO) pending acceptance of a revised environmental impact statement. The following March, the injunction was modified to allow the continuation of all project components, with the exception of the barrier complexes at the Rigolets and Chef Menteur Pass.

In response to the court injunctions against the barrier complexes, the New Orleans District initiated an effort to pursue a fast-track study to recommend a path forward for the project. This effort culminated in a July 1984 reevaluation study of the Lake Pontchartrain, Louisiana, and Vicinity Hurricane Protection project. The reevaluation study examined the continued feasibility of the barrier plan and the feasibility of providing hurricane protection solely by the means of raising and strengthening levees and floodwalls, more common known as High Level plans. The study concluded that a High Level Plan represented the most feasible plan of protection for the study area from the Standard Project Hurricane—the most severe hurricane reasonable expected to occur from a combination of meteorological and hydrologic events characteristic of the area. The plan recommended improved hurricane protection levee systems in Orleans Parish, St. Bernard Parish, and the east bank of Jefferson Parish; repairing and rehabilitating the Mandeville Seawall in St. Tammany Parish; constructing a new levee on the east bank of St. Charles Parish north of US Highway 61; and raising and strengthening the levee along the Jefferson and St. Charles Parish boundary.

The reevaluation study, however, did not address lingering concerns on the treatment of the 17th Street, London Avenue, and Orleans Avenue outfall canals. Subsequent to the 1965 Flood Control Act, the New Orleans District determined that the levees flanking the outfall canals were inadequate in terms of grade and stability. The reevaluation study did set forth five potential solutions, ranging from higher and stronger levees to floodgates at the entrances to auxiliary pumping stations at the canal openings; but left the final determination for alternative selection to future design memorandums.

On February 7, 1985, the Director of Civil Works for the Corps of Engineers, after reviewing the reevaluation study and the final supplement to the environmental impact statement, approved the post-authorization change for the Lake Pontchartrain, Louisiana, and Vicinity Hurricane Protection project, thereby formalizing the High Level Plan. In turn, the New Orleans District commenced examining two alternative plans for providing "high level" standard project hurricane protection for the outfall canals—fronting protection in the form of gated structures at the canal entrances from the lake, and parallel protection in the form of floodwalls and flood proofing of bridges. The plans and designs for the outfall canals called for gated control structures at or near the canal entrances to the lake, but the local sponsor, the Orleans Levee Board, indicated its preference for parallel protection. Congress settled the dispute through the 1992 Energy and Water Development Appropriations Act, which mandated construction of the parallel protection plan.

3.2.1.3. Datum – Subsidence and Vertical Datum Problems in New Orleans, LA

Because of technological gains, the U.S. Army Corps of Engineers is able to more accurately track subsidence of projects – something that could not be done as reliably in the past. Based on a recent study, we can now estimate that the New Orleans area is subsiding at a rate of 6-17 mm/yr or 2-5½ feet per century. In the city itself, it's about 3 feet per century and as much as 10 feet per century in Venice, if recent trends continue.

The Interagency Performance Evaluation Task Force (IPET), an independent group activated by the Corps of Engineers to study the response of the hurricane protection system during Hurricane Katrina, identified problems with using the previous vertical datum to which survey benchmarks were referenced. IPET's ability to accelerate analysis of this issue, which was ongoing by the Corps' New Orleans District and the National Oceanic and Atmospheric Administration (NOAA)'s National Geodetic Survey (NGS), led to the identification of two major problems with elevations in the New Orleans area: subsidence and the use of the old vertical datum elevations as equal to local mean sea level, a common misunderstanding in the engineering community up until the 1990s.

Benchmarks serve as the reference or starting elevation when measuring levee heights, relationships to the water surface (local mean sea level), structure and levee elevations, etc. It has been known since 1985 that the elevations of benchmarks in and around New Orleans were inaccurate, due to subsidence, and needed to be updated. The exact amount of subsidence was not known until a 2004 survey conducted by the NGS in cooperation with the Louisiana Spatial Reference Center, the Corps of Engineers and state and local governments was performed on some 86 benchmarks in southern Louisiana.

The 2004 survey pointed out inaccuracies due not only to subsidence, but also to distortions and errors in elevations of benchmarks that were assumed to be stable in the past, but had in fact subsided themselves. Based on the 2004 survey, the Corps of Engineers has revised the elevations of survey benchmarks used to establish heights of structures, such as levees and floodwalls, in Southern Louisiana. Use of the new 2004 survey assures consistency for all elevation surveys performed in the southern Louisiana area.

The IPET has developed a new relationship between the current local mean sea level and the 2004 survey, which is referred to as the North American Vertical Datum of 1988 (2004.65 Adjustment). Local mean sea level in the city itself is about ½ foot above the 2004 datum. The Corps will use the 2004 elevations and their varied relationship to the local mean sea level throughout the area to precisely determine the elevations of levees and other critical flood protective structures. This datum will also be used by the construction industry and others in southern Louisiana for a wide variety of projects that rely on elevations relative to the local water surface.

More information can be found in the "Geodetic and Water Level Datum" report.

3.2.1.4. Design Hurricane

Because of the urban nature of the project area, the standard project hurricane was selected as the design hurricane.

3.2.1.4.1. Standard Project Hurricane. The standard project hurricane (SPH) is one that may be expected from the most severe combination of meteorological conditions that are considered "reasonably characteristic" of the region. Guidance on the selection of site-specific storm meteorological parameters was initially given in National Hurricane Research Project Report No. 33 (U.S. Weather Bureau, Nov 1959). The Weather Bureau and USACE jointly derived the specifications, criteria, procedures, and methods. The specifications for SPH were reviewed several times after 1959, and the Weather Bureau issued updates. After Hurricane Betsy in 1965, the Weather Bureau revised the wind field parameters, but did not change the other characteristics of the SPH (U.S. Weather Bureau, Aug 1965, Nov 1965, Feb 1966). The post Betsy SPH parameters were used in the hydraulic analysis. An additional update was published by NOAA in 1979 (Sep 1979).

The Central Pressure Index (CPI) was the principal intensity criterion for defining the SPH index. As defined in Report No. 33, the CPI is the estimated minimum pressure for individual hurricanes in Zone B. The 1% recurrence interval CPI was selected to define the SPH index. Three Gulf coast zones were identified; most of coastal Louisiana was contained within Zone B, a 400-mile zone extending from Cameron, LA, to Pensacola, FL. For each zone, an analysis was performed on the central pressure index of all storms with a CPI less than or equal to 29 inches that passed through the zone during the period of record 1900-1956. The CPI was determined from observations of minimum pressure at a given location; computations based on observational data; or by estimate in event that the hurricane passed through a zone where there were insufficient pressure observations to complete a computation but enough evidence to warrant an estimate. Frequency of occurrence was computed using the following equation

$$P = \frac{100(M - 0.5)}{Y}$$

where M is the rank, Y is the period of record, and P is the frequency of occurrence per 100 years.

A SPH storm was considered to have a recurrence interval of once in 100 years (1%) anywhere within Zone B. The probability of the SPH storm striking a smaller subzone within Zone B, such as the Lake Pontchartrain lakefront, would be less. The frequency of the SPH at the site of a protective structure was assumed to be dependent upon its exposure and the direction of approach of the storm. It was assumed that a hurricane whose track is perpendicular to the coast would cause high tides and inundation for a distance of about 50 miles along the coast. Thus, the number of occurrences in a 50-nautical mile subzone of Zone B would be 50/400 or 1/8 or 12.5 percent of the number of occurrences in the zone, provided that all hurricanes traveled in a direction normal to the coast.

However, the usual hurricane track is oblique to the shoreline, as shown in U.S. Weather Bureau, Memorandum HUR 2-4, “Hurricane Frequency and Correlations of Hurricane Characteristics of the Gulf of Mexico Area”, dated August 30, 1957. The average projection along the coast of this 50-nautical mile swath for the azimuths of 42 Zone B hurricanes is 80 nautical miles. The ratio of 80/50 = 1.6. Thus, the probability of occurrence of any hurricane in the 50 nautical mile subzone would be 1.6 times the 12.5 percent, or 20 percent of the probability for the entire Zone B. Therefore, 20 percent of the frequencies on the frequency curve were used to represent the CPI frequencies in the 50 nautical mile subzone that is critical for each study locale.

Using observed high water mark and stage data, combined with computed wind tide levels using different central pressure indices, a surge frequency curve was constructed representative of reaches of the hurricane protection system. The frequency curve also considered statistics on the critical direction of approach. The frequency of the computed wind tide levels was adjusted based on the percentage of each direction followed by historic hurricanes. The probabilities of equal stages for both groups of tracks were then added arithmetically to develop a curve representing a synthetic probability of recurrence of maximum wind tide levels for hurricanes from all directions.

3.2.1.4.2. Probable Maximum Hurricane. The probable maximum hurricane is one that may be expected from the most severe combination of critical meteorological conditions that are “reasonably possible” for the region. The Weather Bureau recommended a CPI of 26.9 inches (U.S. Weather Bureau, Aug 1959, Nov 1961). It was considered to have an infinite recurrence period. All other meteorological parameters were the same as the SPH parameters. Surge estimates using PMH meteorological parameters were not used in the design of the project. The PMH surge estimates were used in the development of surge frequency curves.

3.2.1.5. Orleans East Bank

3.2.1.5.1 Orleans East Bank – HPP Features. This portion of the project that protects the city of New Orleans was designed to protect 28,300 acres of urban and industrial lands. The levee portion is constructed with a 10-foot crown width with side slopes of 1 on 3. Along Lake Pontchartrain Lakefront the top elevation of the earthen levees range between elevation +13 and +20 ft National Geodetic Vertical Datum (NGVD). Figure 6 below shows general elevations for the protection system in Orleans East Bank. There are variations in the system, listed on Table 4, that are not shown in the figure. Floodwalls were designed to provide lines of protection on the east side of the 17th Street Canal, both sides of Orleans Avenue Canal and London Avenue Canal, and the west side of the IHNC. Floodwalls consist of reinforced concrete T-wall floodwalls and reinforced concrete I-wall floodwalls constructed on the top of sheet-pile, and sheet piling without a concrete section. Top elevations of the floodwalls vary between elevation +13 and +15 ft. Also, there are floodwalls along the lakefront, at Seabrook, American Standard, Pontchartrain Beach, and Orleans Marina. The American Standard floodwall has a design elevation of 20 ft NGVD. The other three locations are exposed to reduced or negligible wave runup, and the floodwall design elevations are between 13 and 15 ft NGVD.

Orleans East Bank Lakefront. This protection system segment is located in southeastern Louisiana in Orleans Parish and roughly parallels the shoreline of Lake Pontchartrain between the IHNC on the east and 17th Street Canal on the west.

IHNC Canal (West Bank). The Inner Harbor Navigation Canal is located in the western portion of Orleans Parish and is described in the IHNC section of this report. It forms the eastern border of the Orleans East Bank area.

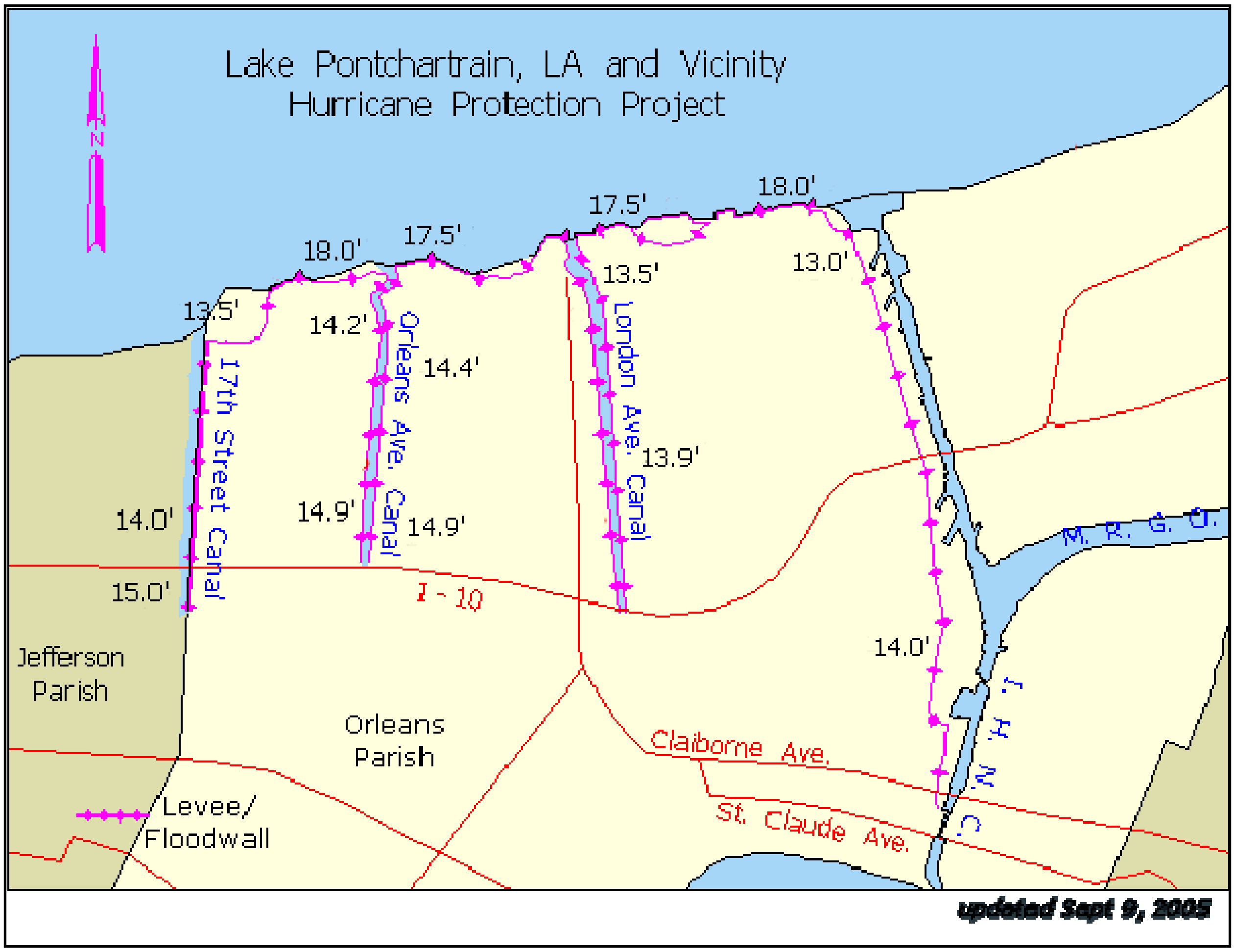

Figure 6. HPP features – New Orleans East Bank

Table 4 **New Orleans East Bank Hurricane Protection System**	
19.2 miles	levee and floodwall
13	pump stations (owned by local agencies)
15	roadway floodgates

17th Street Outfall Canal (Metairie Relief). The 17th Street Outfall Canal lies in Jefferson Parish immediately west of the Orleans Parish boundary line. The canal extends approximately three miles from Pump Station No. 6 near Interstate Highway 10 to its confluence with Lake Pontchartrain.

Orleans Avenue Canal. The Orleans Avenue Canal, located to the east of the 17th Street Outfall Canal, extends about 2.4 miles from Pumping Station No.7 in the vicinity of I-610 to its confluence with Lake Pontchartrain.

London Avenue Outfall Canal. The London Avenue Outfall Canal is located on the south side of Lake Pontchartrain in Orleans Parish. The London Avenue Outfall Canal lies to the east of 17th Street Canal and Orleans Avenue Canal and west of IHNC. It extends approximately 3.2 miles from Pump Station No. 3 to its confluence with Lake Pontchartrain.

3.2.1.5.2. Pre-Katrina - The Orleans Parish portion of the Lake Pontchartrain and Vicinity project is under construction. As of August 29, 2005, the remaining work consisted of the following:

- A floodproofed bridge for Robert E. Lee Blvd over the London Avenue Outfall Canal.
- Fronting protection for Pumping Station No. 3 on the London Avenue Outfall Canal.
- Fronting protection for Pumping Station No. 7 on the Orleans Avenue Outfall Canal.
- A levee enlargement along the London Avenue Outfall Canal between Robert E. Lee Blvd and the lakefront levee.

Construction is underway on temporary closure structures for the three outfall canals. Legislation is pending that would permit the construction of structures that would permanently keep storm surges out of the outfall canals. If this happens, the floodproofed Robert E. Lee Blvd bridge, and the two Fronting Protection contracts would not be required. The levee enlargement along the London Avenue Outfall Canal may not be required depending on the location of the proposed permanent structure.

3.2.1.5.3. Design Criteria and Assumptions - Functional design criteria.

3.2.1.5.3.1. Hydrology and Hydraulics. For Orleans East Bank, the design hurricane characteristics utilized in the design memoranda are shown in Table 5; the design tracks are shown on Figure 7. Maximum wind speed, V_x, was computed using the following equations:

$$V_{gx} = 73(P_n - P_0) - R(0.575f)$$
$$V_x = 0.885V_{gx} + 0.5T$$

where

V_{gx} = maximum gradient wind speed, mph

P_0 = CPI, inches

P_n = asymptotic pressure, inches

R = radius of maximum winds, nautical miles

f = Coriolis parameter in units of hour $^{-1}$

T = the average speed of translation of the hurricane center, mph.

For each project area, the track and forward speed were selected to produce maximum surge or wind tide level. Wind tide levels are defined here as the elevation of the water surface without waves; it can also be referred to as stillwater level. In Lake Pontchartrain, the wind tide level is the sum of the surge, setup, tide, and runoff from rainfall.

Table 5 Design Hurricane Characteristics						
Location	**Track**	**CPI, Inches**	**Radius of Maximum Winds, Nautical miles**	**Forward Speed, Knots**	**Maximum Wind Speed[1], MPH**	**Direction of Approach**
Lake Pontchartrain Southshore	A	27.6	30	6	100	South
Lake Borgne, Rigolets, and Chef Menteur Pass	F	27.6	30	11	100	East

[1] Windspeeds represent a 5 minute average 30 feet above ground level

3.2.1.5.3.1.1. Surge

IHNC. Stillwater levels or wind tide levels were computed using methods described in DM1, Hydrology and Hydraulic Analysis, Part 1, Chalmette, dated August 1966. All computations were made using MSL datum. The Weather Bureau provided frequency data, isovel and rainfall patterns, pressure profiles, hurricane paths and other parameters required for the hydraulic computations. For historical storms used to calibrate and validate methodologies, the Weather Bureau provided historical meteorological and hydrological data. For the synthetic SPH and PMH, generalized estimates of hurricane parameters were provided, based on the latest research and concept of hurricane theory.

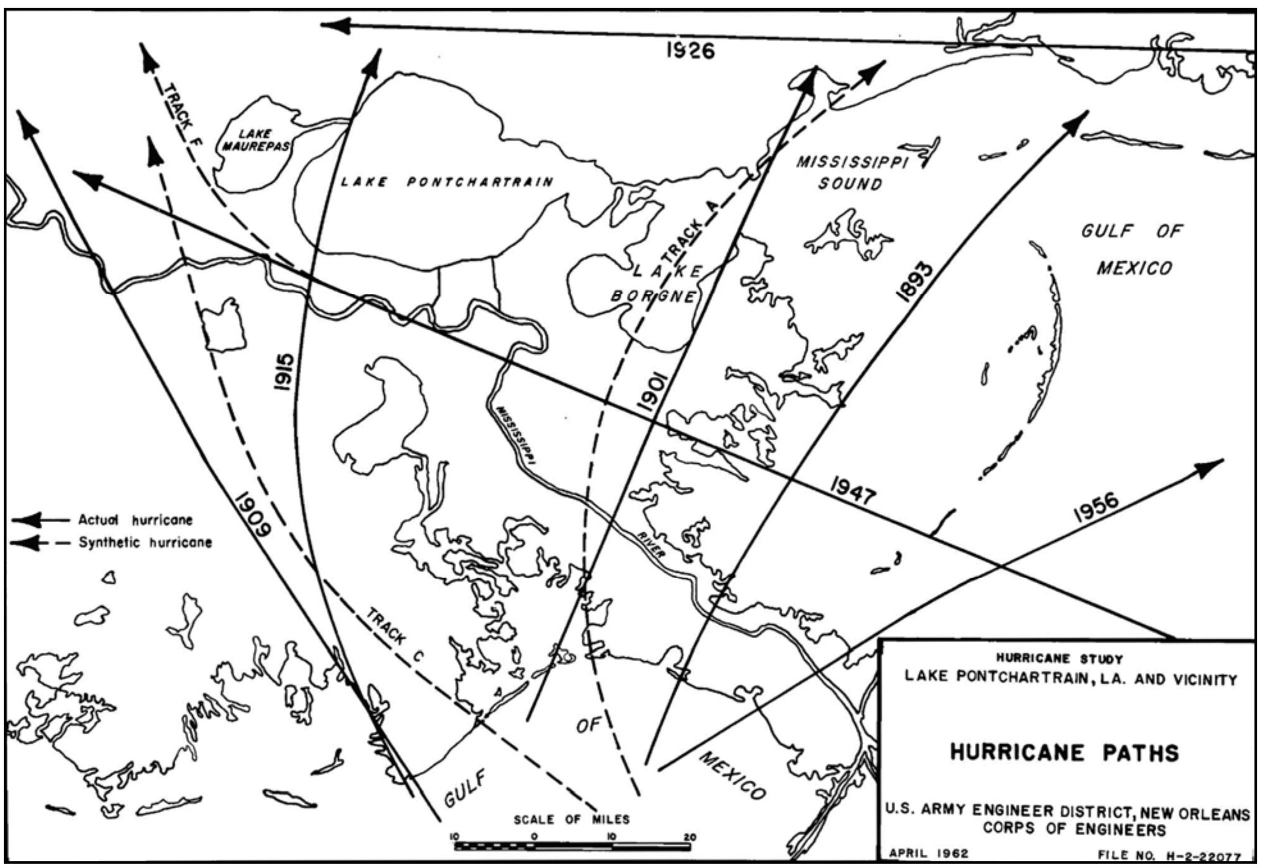

Figure 7. Hurricane Paths, Lake Pontchartrain and Vicinity

Hurricane surge height, defined as the elevation of the stillwater level at a given point resulting from hurricane surge action, is the sum of tide, pressure setup, set up due to winds over the continental shelf, and buildup. Where appropriate, the wind tide level was used in lieu of the stillwater level. Mean normal predicted tide from the Weather Bureau was used. For the pressure setup, a normal pressure of 30.14 inches was used.

The setup due to winds was computed using a general wind tide equation that is based on the steady state conception of water superelevation.

$$S = 1.165x10^{-3}\frac{V^2F}{D}NZ\cos\theta$$

where

- S = wind setup in feet
- V = windspeed in statute miles per hour
- F = fetch length in statute miles
- D = average depth of fetch in feet
- Θ = angle between direction of wind and the fetch
- N = planform factor, generally equal to unity
- Z = surge adjustment factor

The project area was divided into ranges. Water surface elevations along a range were determined by summing the wind setup above the water elevation at the gulf end of a range. The low strip of marshland between Lake Borgne and the Gulf of Mexico was considered already submerged prior to the time of maximum elevation at shore. Initial elevation at the beginning of a range was determined from the predicted normal tide and the setup due to the difference between the central pressure and atmospheric pressure. An adjustment was made at the shoreward end of a range to compensate for the difference in pressure setup between both ends of the range.

This procedure was developed for an area along the Mississippi gulf coast where reliable data was available for several hurricanes to validate the methodology. Two historical storms, the September 1915 and September 1947 hurricanes, were used to establish and verify procedure. In order to reach agreement between computed maximum surge height and observed high water marks, a calibration coefficient or surge adjustment factor, Z, was introduced into the wind tide equation. The procedure was then applied to the Louisiana coast. A third hurricane, occurring in 1956, was used to verify the process. Table 6 shows the surge computations and the comparison with observed high water marks from the three hurricanes.

Table 6 **Verification of Hurricane Surge Heights**							
Location	**Surge adjustment factor, Z**	**Sep 1915**		**Sep 1947**		**Sep 1956**	
		Observed, ft MSL	**Computed, ft MSL**	**Observed, ft MSL**	**Computed, ft MSL**	**Observed, ft MSL**	**Computed, ft MSL**
Shell Beach	0.30	8.3	8.4	11.2	10.5	10.9	10.7
Violet	0.30	-	-	7.3	7.9	6.5	7.7
Michoud	0.30	11.0	11.4	-	-	-	-
Long Point	0.21	9.8	9.6	10.0	10.1	-	-

Computed surge heights for Hurricane Betsy using the same Z factors averaged about 2.2 feet higher than observed surge heights. This was attributed to the effect of the high forward speed of Hurricane Betsy. The September 1915, September 1947, and September 1956 hurricanes had a slower forward speed. A fast moving hurricane would not allow enough time for the surge heights to approach the steady state of water superelevation. For design purposes, Z factors derived from the slow moving hurricanes were used.

Lakefront. Lake Pontchartrain wind tide levels were computed using methodologies contained in DM1, Hydrology and Hydraulics, Part 3 – Lakeshore, dated September 1968. All computations were made using the MSL datum. After the high level plan was authorized in the 1980s, the surge elevations in Lake Pontchartrain were not recomputed; values contained in DM1 were presented in subsequent DMs. It was assumed that the MSL datum and NGVD datum were the same.

In Lake Pontchartrain, the wind tide level is the sum of the surge, setup, tide, and runoff from rainfall. A method was developed to compute the water level associated with each factor and validated using the 1947 hurricane and Hurricane Esther (1957). This method started with a surge hydrograph at Long Point in Lake Borgne, which was developed using a method developed by R.O. Reid. The hydrograph was modified so that the peak of the hydrograph coincided with the maximum surge elevation computed at this location using the general wind tide equation. The resulting hydrograph did not compare well with data from the two storms because of offshore wind directions prevailing after the peak stage; the recession side of the hydrograph was estimated to achieve a more comparable hydrograph.

There are three passes and one canal that could convey water from Lake Borgne into Lake Pontchartrain. Head vs. flow rating tables, using reverse routings of observed storms, were developed for the three passes and one canal to route flow from Lake Borgne into Lake Pontchartrain. Runoff from rainfall associated with the storms was calculated using methods from NWS documents. It was assumed that moderate rainfall would be coincident with the storm. Mean normal tide was assumed to occur at the time of the storm. Lake Pontchartrain stage storage curves were developed and storage from included adjacent wetland areas. Adjustments were made in the routing procedure to account for overtopping shore protective structures.

Next, setup and set down were computed. Lake Pontchartrain was divided into parallel segmental regions. The average windspeeds and depths were determined from the isovel patterns and hydrographic charts. Setup and setdown were computed using step-method formulas that were modified as follows

$$setup = d_t \left[\sqrt{\frac{0.00266U^2FN}{\left(d_t\right)^2} + 1} - 1 \right]$$

$$setdown = d_t \left[1 - \sqrt{1 - \frac{0.00266U^2FN}{\left(d_t\right)^2}} \right]$$

d_t = average depth of fetch in feet below mean water level

U = windspeed in mph over fetch

F = fetch length in miles, node to shoreline

N = planform factor, equal generally to unity

Maximum computed and observed setup elevation for the 1947 hurricane were 4.9 ft and 5.4 ft at West End. Computed stages for the 1915 hurricane compared favorably with observed high water marks.

Outfall Canals. For the Outfall Canals, design water levels were initially computed in the Hurricane Protection Project Reevaluation Study, dated Jul 1984. These values were revised in subsequent DMs.

For the 17th Street Outfall Canal parallel protection, the Corps of Engineers performed steady state step-backwater calculations using HEC-2 Water Surface Profile computer program. Special bridge routine was used to model weir and pressure flow at bridges. Channel cross sections were developed from information provided in Modjeski and Masters drawings dated December 1981. It was assumed that the canal would be dredged according to Sewage and Water Board Base Project, with the exception of the area under bridges. Flow rate was initially based on pump capacities provided by the Sewage and Water Board; nominal capacity of 6,650 cfs for Pump Station No. 6 and a future capacity of 9.630 cfs for Pump Station No. 6 were modeled. Manning’s n values selected were 0.024 for channel and 0.060 overbank. A starting water surface elevation of 11.5 ft NGVD at Lake Pontchartrain was used, which is the still water level in Lake Pontchartrain for SPH condition.

The design flowline is based on 9,630 cfs pump station capacity, floodproofing Veterans Ave bridges, raising I-10 and I-610 bridges, and floodgates at Hammond Highway and Southern Railroad bridges. This flowline is not presented in DM20. Table 7 shows the surge elevations and design elevation for a similar alternative: 9,630 cfs pump station capacity, floodproofing Hammond Highway bridge, raising I-10 and I-610 bridges, and floodgates at Veterans Highway and Southern Railroad bridges.

For the Orleans Avenue Outfall Canal parallel protection, the Corps of Engineers performed steady state step-backwater calculations using HEC-2 Water Surface Profile computer program. Special bridge routine was used to model weir and pressure flow at bridges. Bridge data were taken from available as-builts and field observations. Channel cross-sections were developed from 1971 surveys. Flow rate was based on pump capacities for Pump Station No. 7, provided by the Sewage and Water Board; nominal capacity of 3,250 cfs and future capacity of 4,550 cfs were modeled. Manning's n values selected were 0.03 for channel and 0.035 for overbank. A starting water surface elevation of 11.5 ft NGVD at Lake Pontchartrain was used, which is the stillwater level in Lake Pontchartrain for SPH condition. Five scenarios involving bridge modifications were modeled.

The design flowline is based on existing conditions, with future pump capacity, and no changes in bridges.

For the London Avenue Outfall Canal parallel protection, the Corps of Engineers performed steady state step-backwater calculations using HEC-2 Water Surface Profile computer program. Special bridge routine was used to model weir and pressure flow at bridges. Bridge data were taken from available as-builts and field observations. Channel cross sections were developed from information provided in Burk and Associates hydraulic study dated January 1986. Flow rate was initially based on pump capacities provided by the Sewage and Water Board; nominal capacity of 4,300 cfs for Pump Station No. 3 and 3,980 cfs for Pump Station No. 4 were modeled. For future conditions, it was assumed a third pump station, with a capacity of 1,000 cfs was present. A third pump scenario was modeled. The capacity at Pump Station No. 3 was reduced to 0 cfs and the capacity at Pump Station No. 4 was reduced to 2,475 cfs to represent the stations' ability to pump during the peak of the design hurricane. It was assumed the new station could pump during the peak of the design hurricane. Manning's n values selected were 0.015 to 0.021 for channel and 0.015 to 0.027 for overbank. A starting water surface elevation of 11.5 ft NGVD at Lake Pontchartrain was used, which is the still water level in Lake Pontchartrain for SPH condition.

The design flowline is based on 3,475 cfs pump station capacity, floodproofing of the bridges at Gentilly Blvd, Mirabeau Ave, Filmore Ave, Robert E. Lee Blvd, and Leon C. Simon Blvd, and floodgates at Benefit Street and Southern Railroad bridges.

3.2.1.5.3.1.2. Waves

IHNC. Insufficient open water areas existed for wave generation. Wave runup was considered to be practically nonexistent for the floodwalls and levees.

Lakefront. Wave runup was calculated by the interpolation of model study data developed by Saville (Apr 1956, Oct 1955, Jul 1958), which relates relative runup, wave steepness, relative depth, and structure slope. Wave runup calculations were made for all structures exposed to waves.

The design elevation chosen for protective structures exposed to wave runup was an elevation sufficient to prevent all overtopping from the significant wave and waves smaller than the significant wave. Significant wave heights and periods were determined from prediction curves developed by C. L. Bretschneider (Aug 1954). Waves larger than the significant wave would overtop the protective structures; 14 percent of the waves are higher than the significant wave, and the maximum wave height is about 1.87 times higher than the significant wave. However, such overtopping was not considered a danger to the security of the structures or would not cause material interior flooding. In cases of levees with berms, runup was computed for waves breaking at the toe of each berm to determine the required levee elevation.

Along the seawall segment, a modification to the methodology was made because the land behind the seawall is generally lower in elevation than the seawall crest, approximately 8 ft MSL, and the levee is located approximately 250 ft behind the seawall. When the wind tide is of sufficient height to allow waves to overtop the seawall, water would pond behind the seawall, increasing the stage at the levee, causing wave setup to occur in addition to wind setup. Model study data developed by the Beach Erosion Board was used to compute wave setup, and wave setup was added to the maximum computed wind tide before wave runup was determined. Only the significant wave expected within the ponded area was used to compute the wave runup because smaller waves cause less runup than the significant wave when they break on the same slope.

For Bayou St. John closure and floodgate, 6.5 ft and 5.0 ft was added to the wind tide level, respectively, to account for wave runup.

Outfall Canals. The entrance to 17th Street Outfall Canal is normal to Lake Pontchartrain; waves from Lake Pontchartrain could propagate into the canal. For the 17th Street Outfall Canal, a breakwater was proposed for the canal entrance.

For a short reach of Orleans Ave Outfall Canal extending from the lakefront to about 600 ft upstream of Lakeshore Drive, 6.5 ft was added to the Lake Pontchartrain wind tide level to account for wave runup. London Ave Outfall Canal would have a transition zone between the lakefront levee height and the floodwall height, which would account for wave runup.

3.2.1.5.3.1.3. Summary. Table 7 contains maximum surge or wind tide level, wave, and design elevation information.

3.2.1.5.3.2. Geotechnical

3.2.1.5.3.2.1. 17th Street Outfall Canal (Metairie Relief) (Reference 2)

3.2.1.5.3.2.1.1. Geology. The geologic history and principal physiographic features of the New Orleans area and also the surface and subsurface geology of the New Orleans area are described in Volume V. The breach location is also described in Volume V and is located on the east levee between Stations 560+50 and 564+50.

Table 7
Wave Runup and Design Elevations (transition zones not tabulated – governing DM is listed)

Location	DM	Average Depth of fetch, ft	Significant Wave Height Hs, ft	Wave Period, T, sec	Maximum Surge or Wind Tide Level, Ft	Runup Height, Ft	Freeboard, Ft	Design Elevation Protective Structure, ft
IHNC, Seabrook to L&N Railroad Bridge	DM2 Sup8, Feb 1968	-	-	-	11.4 – 12.9 MSL	0	1.0	13.0 – 14.0 MSL
IHNC, L&N Railroad Bridge to Mississippi River	DM2, Sup8, Feb 1968	-	-	-	12.9 – 13.0 MSL	0	1.0	14.0 MSL
Orleans Lakefront, 29+25.54 to 42+10	DM13, Nov 1984	4.9	1.46	7.3	12.9 NGVD	4.7	-	18.0 NGVD
Orleans Lakefront, 43+10 to 78+59.24	DM13, Nov 1984	5.6	1.8	7.3	12.8 NGVD	5.5	-	18.0 NGVD
Orleans Lakefront, 78+59.24 to 88+24	DM13, Nov 1984	24.4	7.8	7.3	11.5 NGVD	8.5	-	20.0 NGVD
Orleans Lakefront, 88+24 to 94+60	DM13, Nov 1984	24.4	7.8	7.3	11.5N GVD	8.2	-	19.5 NGVD
Orleans Lakefront, 94+60 to 102+23.16	DM13, Nov 1984	4.6	1.33	7.3	12.9 NGVD	4.0	-	17.0 NGVD
Orleans Lakefront, 136+13.19 to 159+70	DM13, Nov 1984	4.9	1.48	7.3	12.8 NGVD	4.7	-	17.5 NGVD
Orleans Lakefront, 164+98.15 to 196+50	DM13, Nov 1984	5.6	1.8	7.3	12.8 NGVD	5.5	-	18.0 NGVD
Orleans Lakefront, 199+41.52 to 246+37.17	DM13, Nov 1984	4.9	1.48	7.3	12.8 NGVD	4.7	-	17.5 NGVD
Orleans Lakefront, 250+72.09 to 289+49	DM13, Nov 1984	5.6	1.8	7.3	12.8 NGVD	5.5	-	18.0 NGVD
Orleans Lakefront, 289+49 to 303+51.39	DM13, Nov 1984	6.2	2.06	7.3	12.8 NGVD	5.9	-	18.5 NGVD
Orleans Lakefront, 303+51.39 to 305+41.96	DM13, Nov 1984	6.2	2.06	7.3	12.8 NGVD	5.9	-	18.5 NGVD
Bayou St. John Closure	DM22, Apr 1993	-	7.8	7.3	11.5 NGVD	6.5	-	18.5 NGVD
Bayou St. John Structure	DM22, Apr 1993	-	2.1	7.3	11.5 NGVD	5.0	-	16.5 NGVD
Pontchartrain Beach Levee and Floodwall	DM22, Apr 1993	-	6.1	7.3	11.5 NGVD	8.5	-	20.0 NGVD
17th Street Outfall Canal, Hammond Highway	DM20, Mar 1990	-	-	-	11.66 NGVD	-	2.0	13.66 NGVD
17th Street Outfall Canal, Southern Railroad Bridge	DM20, Mar 1990	-	-	-	12.63 NGVD	-	2.0	14.66 NGVD
Orleans Ave Outfall Canal, lakefront to 118+00	DM19, Aug 1988	-	-	-	11.5 NGVD at Lake Pontchartrain	6.5	-	18.0 NGVD
Orleans Ave Outfall Canal, 118+00 to 90+86	DM19, Aug 1988	-	-	-	11.64 NGVD	-	2.0	13.64 NGVD

(Continued)

Table 7 (Concluded)								
Location	**DM**	**Average Depth of fetch, ft**	**Significant Wave Height Hs, ft**	**Wave Period, T, sec**	**Maximum Surge or Wind Tide Level, Ft**	**Runup Height, Ft**	**Freeboard, Ft**	**Design Elevation Protective Structure, ft**
Orleans Ave Outfall Canal, 90+86 to 64+14	DM19, Aug 1988	-	-	-	11.80 NGVD	-	2.0	13.80 NGVD
Orleans Ave Outfall Canal, 64+14 to 36+64	DM19. Aug 1988	-	-	-	11.97 NGVD	-	2.0	13.97 NGVD
Orleans Ave Outfall Canal, 36+64 to PS No. 7	DM19, Aug 1988	-	-	-	12.21 NGVD	-	2.0	14.21 NGVD

3.2.1.5.3.2.1.2. Project Foundation Conditions. The engineering properties of the sediment beneath the project vary greatly. Generally, the subsurface consists of Holocene deposits varying in depth to approximately 60 feet and underlain by Pleistocene deposits. Specifically from Station 670+00 to Station 540+00 the surface is comprised of marsh-swamp deposits which vary in thickness between 5 and 10 feet. The marsh-swamp deposits are characterized by high wood and organic material contents and high water contents. Beneath the marsh-swamp deposits is a sequence of deposits which include bay-sound, lacustrine, beach and prodelta deposits. From Station 672+00 to Station 660+00, the marsh-swamp deposits are underlain by prodelta deposits which vary up to 10 feet in thickness. The prodelta deposits are comprised predominantly of fat clays. Between Station 617+00 and Station 540+00 the marsh-swamp deposits are underlain by lacustrine deposits which vary in thickness to 20 feet. This is the area of the breech. These lacustrine deposits are comprised predominantly of fat clays. Underlying the marsh-swamp deposits from Station 660+00 to Station 617+00 are beach deposits which vary in thickness up to 40 feet or more. These beach deposits consist of sands and silty sands and extend beneath the prodelta deposits to the south and the lacustrine deposits to the north. The thickness of the beach deposits remains constant towards the south; however, the thickness of the beach deposits decreases to the north until they terminate near Station 540+00. Underlying the beach deposits throughout the project are bay-sound deposits which vary in thickness from 15 to 20 feet. The bay-sound deposits consist generally of fat clays with some lean clays. Underlying the Holocene deposits in the project area are the Pleistocene lean clays, fat clays, silty sands and sands.

3.2.1.5.3.2.1.3. Field Exploration. Fourteen (14) continuous sample, 5-inch diameter soil borings were made by the USACE, New Orleans District in the project area. Six of the undisturbed borings were made at the levee centerline, and five of the undisturbed borings were made along the levee protected side toe. The two general type borings were made at the flood side and protected side toe of the canal levee. In addition, 77 borings made by an A-E for the New Orleans Sewerage and Water Board (NOS & WB) were used in conjunction with the USACE borings in the foundation design. Nineteen of the borings by the A-E were sampled using a 5-inch diameter Shelby tube sampler, and 58 borings were sampled using a 3-inch diameter Shelby tube sampler.

3.2.1.5.3.2.1.4. Underseepage. Underseepage analyses were performed following EM 1110-2-2501 (Reference 3) using Lanes Creep ratio.

3.2.1.5.3.2.1.5. Hydrostatic Pressure Relief. The buried beach sand is highest between B/L Station 614+00 and B/L Station 663+00. A piezometric grade line of Elevation -2.4 feet was used for the buried beach sand and was considered to be independent of the canal water level elevation. The only exception was at the Lake Pontchartrain end of the project where a piezometric level of Elevation 0.0 was used in the buried beach sand. This design decision was based on the information obtained from a test section that was dredged in 1983 to expose the buried beach sand in the slopes and bottom of the canal. Piezometers were installed for the test section with their tips in the buried beach sand. The piezometers indicated that neither the water level in the canal nor the dredging of the test section affected the piezometric levels.

3.2.1.5.3.2.1.6. Pile Foundation. The pile foundations were designed for the following factors of safety:

Factors of Safety For Pile Capacity Curves

	With Pile Load Test	**W/O Pile Load Test**
Q-Case	2.0	3.0
S-Case	2.0 (Dead Load only)	3.0 (Dead Load only)
	1.0 (Total Load)	1.5 (Total Load)

Pile load tests were furnished by representatives of the NOS & WB for review of their projects. Pile load tests for Class B timber piles (Tested 1984), Steel H 12x53 piles (Tested 1986) and 12" square prestressed concrete piles (tested 1986) were conducted by the New Orleans Sewerage and Water Board's contractors.

3.2.1.5.3.2.1.7. Levee Stability. Stability of the levees was analyzed y the LMVD Method of Planes for a minimum factor of safety of 1.30 with respect to the design shear strength. The analyses considered potential failure surfaces to the flood side and the protected side of the levee. Analyses to the protected side considered the Channel water level at the Standing Water Level (SWL). Analyses to the channel side considered the channel water level at Elevation -5.0 feet which would be associated with a minimum water level in Lake Pontchartrain due to a hurricane having winds blowing from south to north.

3.2.1.5.3.2.1.8. I-Wall Design. The required penetration of the steel sheet piling was determined by the Method of Planes using "Q" case design shear strengths. The factors of safety were applied to the design shear strengths. Following are I-wall design criteria used for this hurricane protection project levee:

Tip Penetrations

Q-Case

F.S. = 1.5 with water to SWL
F.S. = 1.25 with water to SWL and waveload
F.S. = 1.0 with water to SWL + 2 feet freeboard

S-Case

F.S. = 1.2 With water to SWL and waveload (if applicable)

Deflections

Q-Case, F.S. = 1.0 with water to SWL = 2 feet freeboard

Bending Moments

Governing Tip Penetration Case
If the penetration to head ratio was less than 2.5 to 1, the penetration was to be increased to 2.5 to 1. The SWL was used to calculate head for penetration to head ratio.

Endorsement 1 by LMVD stated that a minimum penetration to head ratio of 3 to 1 should be used for sheet pile design for this project. Also, all walls retaining soil should be analyzed as permanent bulkheads using “S” soil strengths, a factor of safety of 1.5 and a canal level of Elevation 0.0. The District disagreed with the comment to increase the penetration ratio to 3 to 1 based on the detailed information available for the project with respect to surveys and the number of soil borings. The walls retaining soils were reanalyzed to comply with the comment. The Division approved the District response.

3.2.1.5.3.2.2. Orleans Avenue Outfall Canal (Reference 4)

3.2.1.5.3.2.2.1. Geology. The geologic history and principal physiographic features of the New Orleans area and also the surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.5.3.2.2.2. Project Foundation Conditions. The ground surface within the project area is directly underlain by marsh-swamp deposits ranging in thickness from about 5 to 15 feet. Hydraulic fill sands ranging in thickness from about 10 feet to 20 feet were encountered directly beneath the surface in the borings north of Robert E. Lee Boulevard. Lacustrine clays underlie the march-swamp deposits and generally average about 10 feet in thickness. The beach deposit underlies the lacustrine clays and ranges in thickness from about 40 feet on the southern end of the project to about 10 feet near Lake Pontchartrain. The base elevation of the beach deposit remains a fairly constant 45 feet. Bay-Sound deposits underlie the beach deposit. The Pleistocene is encountered at an average depth of about 65 ft, but varies in depth from about 40 to 85 feet.

3.2.1.5.3.2.2.3. Field Exploration. A total of 16 undisturbed 5-inch diameter soil borings and four general type borings were made in the project area by the USACE. In addition, 52 borings made by an A-E working for the Orleans Levee Board were used in conjunction with the USACE borings for foundation design.

3.2.1.5.3.2.2.4. Underseepage. One reach was analyzed by flow net due to the presence of silt and sand layers in the levee section. The remaining reaches were analyzed by Harr’s Method. No criteria were given.

3.2.1.5.3.2.2.5. Hydrostatic Pressure Relief. Piezometers installed by the Levee Board and also by the USACE indicated the buried beach sand was connected to the canal in one reach and was not connected in another reach. In the reach where the canal and buried beach sand were indicated to be connected, a gradient was determined from the piezometer readings and a piezometric gradeline was determined for a S.W.L. of Elevation 11.6, NGVD. The piezometric grade line was used in the stability analyses and uplift analyses. In the reach where the piezometers indicated there was not a hydraulic connection between the buried beach sand and the canal, the piezometers were used to develop a piezometric grade line at Elevation -3.0 feet, NGVD, for the analyses.

3.2.1.5.3.2.2.6. Pile Foundation. The estimated pile lengths from the pile capacity analyses were based on a factor of safety of 2.0 for both compression and tension. The results of the analyses presented in the DM were to be used for estimating purposes only. The final design pile lengths were to be based on the results of the pile load tests performed during construction.

3.2.1.5.3.2.2.7. Slope Stability. The stability of the levees along the Orleans Avenue Outfall Canal from the lakefront levees to the pumping station was determined by the Method of Planes analyses. The Method of Planes analysis was based on a minimum factor of safety of 1.3 with respect to the Q design shear strengths.

3.2.1.5.3.2.2.8. I-Walls. The required penetration of the steel sheet piling was analyzed using both Q and S strengths. The factors of safety were applied to the design shear strengths. The following is sheet pile wall design criteria for the hurricane protection levees:

Q-Case

F.S. = 1.5 with water to SWL
F.S. = 1.25 with water to SWL and waveload
F.S. = 1.0 with water to SQWL plus 2 ft of freeboard

S-Case

F.S. = 1.2 with water to SWL and waveload (if applicable)

If the penetration to head ratio were less than about 3:1, it was to be increased to 3:1 or to that required by the S-Case F.S = 1.5, whichever resulted in the least penetration. The SWL was used to calculate head for penetration to head ratio.

3.2.1.5.3.2.2.9. T-Walls. A deep-seated analysis utilizing a 1.3 factor of safety incorporated into the soil properties was performed for various potential failure surface beneath the T-walls. The summation of horizontal driving and resisting forces results in a value that is positive indicating that the load on the base must be equal to or greater than the load on the failure critical surface. The base of the T-wall was lowered until the at-rest force equaled or was greater than the positive unbalanced load on the critical failure surface.

3.2.1.5.3.2.3. London Avenue Outfall Canal (Reference 5)

3.2.1.5.3.2.3.1. Geology. The geologic history and principal physiographic features of the New Orleans area and also the surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.5.3.2.3.2. Project Foundation Conditions. The project represents approximately six miles of levee improvement. Subsurface soils encountered in the field investigation include hydraulic fill, Holocene surficial marsh and subsurface beach, intradelta abandoned distributary, lacustrine, prodelta and marine deposits. Borings north of Leon C. Simon Boulevard encountered 10 to 20 feet of hydraulic fill placed in the 1920's and early 1930's. The surficial marsh veneer was encountered over the majority of the project area and ranges in thickness from 5 ft to 15 ft. The remaining Holocene soils were variable with respect to thickness and aerial extent. The Pleistocene was encountered at an average elevation of about -60 feet, but varies from about -55 feet to -70 feet.

3.2.1.5.3.2.3.3. Field Exploration. Thirteen continuous undisturbed 5-inch diameter soil borings and four general type soil borings using a 1 7/8-inch core barrel or a 1 3/8-inch split-spoon sampler were made by the USACE, New Orleans District. The borings were located both along the centerline and along the toes of the levees. Sixty-nine (69) borings taken by an A-E for the Orleans Levee Board were used in conjunction with the USACE borings in the foundation design. Three of the borings made by the A-E were made using a 5-inch diameter Shelby tube sampler and the remaining 66 borings were made using a 3-inch diameter Shelby tube sampler.

3.2.1.5.3.2.3.4. Underseepage Control Measures. The existing bridge cutoff sheet pile walls were to be utilized at Mirabeau Avenue, Robert E. Lee Boulevard, and Leon C. Simon Boulevard. At the Benefit Street and the Southern Railroad bridges, floodgates were to be installed with sheet pile cutoff walls. At Gentilly Boulevard, a new sheet pile cutoff wall was to be installed. At Fillmore Avenue a new sheet pile cutoff wall was to be installed unless the existing sheet pile cutoff wall could be verified.

3.2.1.5.3.2.3.5. Hydrostatic Pressure Relief. A piezometric gradeline based on the ground surface and past piezometer readings was used. The USACE had installed 12 piezometers in 1970 at the Mirabeau Avenue Bridge at A-E B/L Station 69+40. Readings from these piezometers were obtained in 1970 and 1971. In addition, two piezometers were installed by the Orleans Levee Board's A-E at Station 101+00, A-E B/L west levee toe and 75 feet west of the west levee toe. No top of pipe elevation had been obtained for the two A-E piezometers. The piezometric gradeline assumed by the USACE for design was to be verified after the Orleans Levee Board collected readings from their piezometers.

3.2.1.5.3.2.3.6. Pile Foundations. The pile capacity analyses were performed for both the Q and S cases. Overburden stresses were limited to D/B = 15 in the sands or a maximum limiting resistance of less than 2.0 ksf in S-case clays. The estimated pile lengths from the pile capacity analyses were based on the following criteria:

Factors of Safety
For Pile Capacity Curves

With Pile Load Test*	W/O Pile Load Test
2.0	3.0

* A pile load test was to be conducted and F. S. = 2.0 was to be used for both the Q and S cases.

During construction, test piles were to be driven and load tested. The results of the pile load tests were to be used to determine the lengths of the service piles.

3.2.1.5.3.2.3.7. Slope Stability. The slope stability analyses for the levee sections were performed using the Method of Planes and a minimum factor of safety of 1.3. Since the Parallel Protection Plan was not the recommended plan by GDM 19A, only a few slope stability analyses were presented with the GDM. Division comments cited certain weak clay layers that should be considered in the final design.

3.2.1.5.3.2.3.8. I-Wall. The required penetration of the steel sheet piling was determined by the Method of Planes. Following is sheet pile wall design criteria for hurricane protection levees:

Tip Penetrations

Q-Case

F.S. = 1.5 with water to SWL
F.S. = 1.25 with water to SWL and waveload
F.S. = 1.0 with water to SWL + 2 feet freeboard

Deflections

Q-Case, F.S. = 1.0 with water to SWL = 2 feet freeboard

Bending Moments

Governing Tip Penetration Case

If the penetration to head ratio was less than 3:1, it was to be increased to 3:1 or to that required by the S-case, F.S. = 1.5, whichever results in the least penetration. The SWL was used to calculate head for penetration to head ratio.

As for the slope stability analyses, only a few I-wall analyses were performed because the Parallel Protection Plan was not the approved plan.

Division comments stated that certain conditions involving soil stratification and selected shear strengths should be reevaluated in the DDM.

3.2.1.5.3.2.4. Pontchartrain Beach Floodwall/Levee Project (References 7 and 8)

3.2.1.5.3.2.4.1. General. Reference 8 provides the geotechnical investigation used in preparation of Reference 7. The results of the 1985 geotechnical investigation are summarized in Reference 7. The following criteria review was obtained from Reference 8.

Flood protection for the Pontchartrain Beach area was to be provided by either earthen levee or a combination levee and I-wall. Access ramps were to be provided at three locations and gated structures were to be provided at these ramps for flood protection.

3.2.1.5.3.2.4.2. Geology. The geologic history and principal physiographic features of the New Orleans area and also the surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.5.3.2.4.3. Project Foundation Conditions. A topographic survey indicated that the existing elevations along the levee alignment generally vary between 5.0 and 7.0 NGVD. Near surface fill materials at the locations of six borings are generally comprised of medium stiff to stiff gray and tan clay and silty clay with sand and shells and generally encountered to depths varying from Elev. 3 feet and Elev. 2 feet. At one boring location, this near surface fill material is underlain by a layer of soft gray clay with sand pockets and shell fragments to Elev. -5.0 feet. Beneath these materials and from the ground surface at three other boring locations, strata of very loose to medium dense and medium compact gray sand, silty sand, sandy silt, clayey sand and clayey silt with clay layers and shell fragments are interbedded to depths varying between Elev. -20.0 feet and Elev. -26.0 feet. At these elevations and continuing to elevations varying between Elev. -30 feet and Elev. -35 feet are strata of soft to medium stiff gray clay and sandy clay with sand layers and shell fragments. Beneath this clay stratum and continuing to depths ranging between Elev. -41 feet and Elev. -46 feet are strata of very loose to medium dense gray silty sand, clayey sand and sand with clay layers and shell fragments. These strata overlie a stratum of medium stiff to stiff gray clay and sandy clay with shell fragments and sand pockets encountered to the Pleistocene surface that varies between Elev. -50 feet and Elev. -55 feet.

3.2.1.5.3.2.4.4. Field Exploration. A total of ten undisturbed soil test borings were drilled for this investigation to depths of 80 feet and 100 feet. Two of the borings were sampled using a 5-inch diameter Shelby tube sampler and the remaining eight borings were sampled using a 3-inch diameter Shelby tube sampler. The samples from one of the 5-inch diameter holes were delivered to the USACE, New Orleans District. In addition to the soil borings, four piezometers were installed in near surface sands at depths ranging from about 11 feet to 16 feet. The piezometers were installed to provide groundwater data for use in establishing any correlation between stages in Lake Pontchartrain and piezometric levels in the near surface sands. At the time of the field investigation, the groundwater level was generally 3 to 5 feet below the ground surface.

3.2.1.5.3.2.4.5. Design Conditions. The design static water level (SWL) was taken as Elevation 11.5 feet. Dynamic wave loads as furnished to URS Engineers and in turn Eustis Engineering Company by the U.S. Army Corps of Engineers are tabulated below.

I-Wall Elevation in Feet	Levee Crown Elevation In Feet	Dynamic Wave Load Pounds/Foot	Elevation of Wave Load Resultant in Feet
17.5	10.5	5401	14.2
20.0	13.0	5362	16.2

3.2.1.5.3.2.4.6. Levee Analyses

Slope Stability. Levees were designed for a minimum factor of safety with respect to slope stability of 1.3 using the LMVD Method of Planes analyses.

Underseepage. Seepage beneath the all-earth levee section was evaluated by Bligh's Creep Method of Analysis. The minimum creep ratio considered adequate was a value of 18.5 for very fine or silty sand. Recommendations were made that the piezometers be read on a periodic basis so that the piezometric levels could be correlated with lake stage.

3.2.1.5.3.2.4.7. I-Wall Analyses

Cantilever I-wall Analyses. The I-wall analyses were performed for two conditions: (1) the static water level loading with a factor of safety of 1.5 factored into the soil shear strength parameters, and (2) the dynamic wave load with a factor of safety of 1.25 factored into the soil shear strength parameters considering floodside water at the static water level. The analyses were performed using "S" shear strengths. Dynamic wave loadings with a factor of safety of 1.25 governed the required penetration of the sheet piling. A factor of safety of 1.0 for the same loading condition was used to determine the maximum anticipated bending moment.

Slope Stability Analyses. The combination levee/sheet pile wall sections were analyzed for slope stability using the LMVD Method of Planes and a minimum factor of safety of 1.3. The results of the analyses indicated factors of safety greater than the minimum 1.3 factor of safety.

Underseepage. Underseepage for the combination I-wall/levee section was evaluated based on Lane's Weighted Creep Ratio Method of Analyses. Weighted creep ratios varying between approximately 10.2 and 12.3 were computed for the sheet pile penetrations required for cantilever stability. These values all exceeded the minimum creep ratio of 8.5 recommended in Lane's analyses for very fine or silty sand.

3.2.1.5.3.2.4.8. Gated Structures

Deep Seated Stability Analyses. The potential for deep seated failure of the T-wall and gated structures was evaluated by slope stability analyses using the LMVD Method of Planes. The analyses indicated that the active driving forces for all failure surfaces analyzed did not exceed the summation of the resisting forces and the passive driving forces. Therefore, it was concluded there was no potential for a deep seated stability failure beneath the gated structures.

Underseepage. Based on Lane's weighted Creep Ratio of 8.5, the sheet pile cutoff beneath the gated structures was extended to Elev. -11.0 feet.

Allowable Pile Load Capacities. The allowable load capacities for various lengths, sizes, and types of piling were computed and presented as curves of allowable load versus penetration. The allowable load curves included a factor of safety of 2.0 for both tension and compression. No mention was made of whether the analyses were performed using (Q) strengths or (S) strengths. It was pointed out in the report that the factor of safety of 2.0 would only be applicable if the USACE conducted a test pile program to determine final pile design lengths. If the USACE did not conduct a test pile program, a factor of safety of 3.0 would be required. The curves presented in the report could be adjusted to reflect a factor of safety of 3.0 by multiplying the capacities on the curves by a factor of two-thirds.

3.2.1.5.3.2.4.9. Levee Construction Recommendations. The geotechnical report recommended that site preparation, levee fill and compaction be accomplished in accordance with the Department of the Army, Mississippi River Commission, Lower Mississippi Valley Division, Corps of Engineers Standard Specifications for Levee Construction. The levee fill was to be either a CH or CL material as classified by the Unified Soil Classification System and compacted by semi-compaction methods. Material for levee fill was to be compacted within the following moisture content ranges.

Material	Moisture Content	
	Minimum	Maximum
CL	18	32
CH	20	50

The intent of these specifications was to construct a relatively uniform embankment free of large gaps, voids and loose materials. To accomplish this, it was recommended that the backfill be spread in 8- to 10-inch lifts and each lift compacted with a minimum of three passes of a D-5 dozer, or equivalent. After proper compaction was achieved, it was stated that a D-5 dozer should be able to "walk-out" without fill material sticking to the treads or otherwise disturbing the lifts. If this could not be achieved, "moisture control," such as disking to dry back material or spraying to wet the materials was recommended.

3.2.1.5.3.2.5. Modification of Protective Alignment and Pertinent Design Information IHNC Remaining Levees West Levee Vicinity France Road and Florida Avenue Containerization Complex (Reference 10)

3.2.1.5.3.2.5.1. Geology. The geologic history and principal physiographic features of the New Orleans area and also the surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.5.3.2.5.2. Project Foundation Coordination. The subsurface along the alignment presented herein consists of approximately 8 to 15 feet of fill material overlying about 60 feet of Recent deposits. These Recent deposits generally consist of clays with varying amounts of organic materials, some silts, and sand. The top of the Pleistocene soil is located at approximately Elev. -63 feet at the northern end of the alignment near France Road, and at Elev. -70 feet at the southern end near Florida Avenue.

3.2.1.5.3.2.5.3. Field Exploration. Twelve borings made previously along this reach were utilized for all analyses presented in the referenced report. The 12 borings were originally made for Reference 9.

3.2.1.5.3.2.5.4. Cantilever I-Wall. The stability and required penetration of the steel sheet pile were determined by the Method of Planes for both the (Q) and (S) shear strength cases. The latter governed the design. A factor of safety of 1.50 was applied to the design shear strengths. The required depths of penetration were determined for a hurricane water level 6 inches below the top of wall on the flood side, and a water level equal to the water table on the protected side.

3.2.1.5.3.2.5.5. Levee and Levee/I-Wall Stability. Stability of the earthen levee and the levee/I-wall was investigated by the Method of Planes based on a minimum factor of safety of 1.3 with respect to shear strength using the (Q) design shear strengths. The stability of a road ramp was also analyzed using these criteria. Analyses were run for both the flood side and the protected side. All analyses yielded factors of safety equal to 1.3 or greater.

3.2.1.5.3.2.5.6. Pile Foundations. Pile bearing capacities for the gated structures and I-walls were determined from the pile test performed at site 1 of the IHNC West Levee, Florida Avenue to IHNC Lock project, where subsurface conditions were similar to those at the proposed site of the T-wall and gates.

3.2.1.5.3.2.6. Inner Harbor Navigation Canal Remaining Levees (Reference 9)

3.2.1.5.3.2.6.1. Geology. The geologic history and principal physiographic features of the New Orleans area and also the surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.5.3.2.6.2. Project Foundation Conditions. The subsurface consists of Recent Deposits varying in thickness from about 50 feet at the north or Lake Pontchartrain end of the project on both sides of the Inner Harbor Navigation Canal, to about 70 feet near Florida Avenue along the west levee. Exceptions to this are in the vicinity of Station 130+00 along the east levee and Station 126+00 along the west levee where the ancient Bayou Metairie Distributary has incised into the Pleistocene surface, and south of Station 133+00 on the east levee and Station 165+00 on the west levee where an ancient reentrant exists on the Pleistocene surface. The Recent deposits are underlain by Pleistocene (Prairie Formation) deposits. Generally, the Recent at the northern end of the project consists of a discontinuous layer of very soft marsh clays with organic matter and peat, and soft to stiff natural levee clays with lenses and layers of silt, underlain by a thick sequence of buried beach sands with shells and shell fragments that overlie thin medium to stiff prodelta clays. South of Station 80+00 to the vicinity of Station 124+00, a wedge of very soft to soft interdistributary clays with lenses and layers of silt and sand exists between the upper marsh and natural levee deposits and the underlying buried beach sands. In the vicinity of Station 165+00, an abandoned distributary consisting of silt and silty sands with layers of clay exists to a depth of at least 100 feet. South of the abandoned distributary deposit, the Recent consists of a discontinuous layer of marsh and natural levee deposits underlain by a thick sequence of interdistributary deposits and estuarine clays, silts, and sands with shells and shell fragments. The fill material, marsh, natural levee, interdistributary, abandoned distributary, buried beach, prodelta, and estuarine deposits are underlain by Pleistocene deposits along the entire east and west levee.

3.2.1.5.3.2.6.3. Field Exploration. Ten 5-inch diameter undisturbed soil borings were made along the levee alignment. Twenty-eight 1-7/8-inch ID general-type (GT) soil borings were made on the west side of which 26 were made along the levee alignment and two on an abandoned alignment. Borings were made generally along the project alignment at intervals varying from 350 to 1,500 feet through existing levees, at the toe of the levees at selected locations, and along the centerline of protection works between existing levees. The boring depths extended to Elev. -15.0 feet to Elev. -98.0 feet. Three piezometers were installed in the buried beach sands, along each of two ranges along the west bank extending from the canal to landside of the levees. The piezometers were read at frequent intervals to determine existing piezometric conditions in the buried beach sand.

3.2.1.5.3.2.6.4. Type of Protection. Because of the limited space available due to the nearness of dwellings, roads, railroads, and industrial plant facilities; the necessity to cut off seepage in the sandy levee fill in the buried beach area; and the economical advantage of walls over the cost of right-of-way for the large levees and berms required, the protection was to consist predominantly of a cantilever I-type floodwall of steel sheet piling driven through existing levees, and/or fill, and capped with a concrete wall. T-type floodwalls supported by bearing piles were to provide the protection in the more congested areas in the vicinity of road and railroad crossings. Conventional earthen levees were to be used in the less congested areas.

3.2.1.5.3.2.6.5. Cantilever I-type Floodwall. The stability and required penetration of the steel sheet pile below the earth surface were determined by the Method of Planes using (S) shear strengths. A factor of safety of 1.5 was applied to the design shear strengths. The stability of I-type floodwalls was determined for a hurricane water level 6 inches below the top of the wall on the floodside; and on the protected side, for a water level equal to the water table assuming the water table at the average ground surface where the ground surface is below elevation zero and for a water level at elevation zero where the ground surface is above zero. Factors of safety (FOS) were also determined for the headwater level at the top of the walls, and for high tail water conditions in the sandy fill along the buried beach sand reach. Where I-walls serve as floodwalls and earth retaining bulkheads the stability condition that governed for design penetration was used in setting the pile tip elevation.

3.2.1.5.3.2.6.6. Levees and Road Ramps. Using sections representative of existing conditions along the protection alignment, the slopes and berm distances for the recommended levees and ramps were designed for a hurricane water condition 1.5 feet above still water level for the project hurricane and for assumed failure toward the landside. The stability of the levees and ramps was determined by the Method of Planes using the design (Q) shear strengths and applying a minimum factor of safety with respect to the shear strength of approximately 1.3. In the stability analyses for the levees in the buried beach sand reaches, hydrostatic uplift was applied on the base of the clay, from the top of the sands to the midwell piezometric head, determined by the relief well analysis, and dissipating to the water surface at the landside along the passive earth wedge.

3.2.1.5.3.2.6.7. Seepage and Hydrostatic Uplift Relief

General. Because of the sandy levee and foundation in the buried beach area, interception of seepage through the levee and reduction of hurricane piezometric heads in the foundation sands were considered necessary to maintain stability. The I-wall sheet pile was extended in depth below that required for stability where necessary to cut off the upper sand fill strata.

Relief Wells. Permanent hydrostatic pressure relief wells were to be provided along the west levee in the buried beach sand area. The piezometers installed with tips in the buried beach sand were read at frequent intervals to determine existing piezometric conditions in the vicinity of the levees. In addition to intermittent readings, a series of continuous observations were made for periods of 45 and 31 hours during periods when a maximum tide change was expected.

To determine the relationship between the piezometric level in the beach sand and the IHNC water level; and to determine the effective canal side entrance and landside exit drainage distances, the mean high stage readings from the compilation of piezometer data were plotted on the levee sections at the piezometers locations. This information indicates that the effective landside exit drainage is governed by the subsurface drainage along Pauline Drive. Using the hydraulic gradients established by these existing piezometric conditions, effective entrance and exit drainage distances were determined. The design piezometric heads at the exit distances were based on the following reasoning: Information from inhabitants in the developed area on the west side indicates that during hurricanes the excess heads in the foundation sands caused severe “boiling” in the subsurface drainage manholes. Since the design hurricane is more severe than those previously experienced, an elevation of zero was used at the exit point for the design hurricane condition.

The projected piezometric heads for the design hurricane conditions were based on the canal water level at the effective entrance distance and the assigned piezometric and/or water surface elevations at the effective exit distance from the well line.

To determine the possible effect of feeding from Lake Pontchartrain, the soil profiles along the west levee was extended by utilizing boring data made for the authorized Seabrook Lock and the local interest sponsored Seabrook Bridge. This information indicated that the buried sand beach terminates in the immediate vicinity of the ends of the recommended well lines. The water level in the lake, concurrent with the design hurricane condition in the IHNC, is elevation 3.0 and feedback was determined to not be an influencing factor for design of the wells at the lake end of the project. The well line, however, was to be extended along the tie-in levees at the lake end of the project as part of the Seabrook Lock construction.

Using the water level design data, the piezometric conditions derived from the data, the grain size gradations, the permeability, the well details, and procedures in accordance with EM 1110-2-1905, 1 March 1965, “Design of Finite Relief Well System”, well spacings and discharges were determined for a line of landside relief wells along the levee in the buried sand beach area.

Permanent Piezometers. Additional piezometers were to be installed in the beach sand to obtain readings on piezometric conditions before, during and after high flood heads in the IHNC. The data were to be used in evaluating the effectiveness of the relief well system and remedial measures were to be initiated if found to be necessary.

3.2.1.5.3.2.6.8. Pile Foundations. Pile bearing capacities and lengths for the gated structures and T-walls were determined by use of the following criteria:

- Skin friction disregarded above bottom of marsh deposit and/or above upper one-third of Recent deposit.

- Applied factors of safety 1.75 in compression and 2.0 in tension

- Applied conjugate stress ratios K = 1.00 in compression and 0.7 in tension. (S) case governed.

- Bearing pile subgrade modulus for estimating lateral restraint of the soil were determined by use of Reference 16.

3.2.1.5.3.2.6.9. Erosion Protection. Due to the short duration of hurricane floods, the resistant nature of the clayey soils, and the limited conditions for wave generation; no erosion protection was considered necessary along the major portion of the line of protection. However, where the levees and walls were near the canal proper in the vicinity of U.S. Highway 90 and Florida Avenue, erosion protection was to be provided where required. A concrete strip was to be provided around the relief wells and extend into the sodded discharge collection ditch.

3.2.1.5.3.2.6.10. Methods and Sequence of Construction. The earthwork required along the project consisted of degrading, shaping, and rehandling of existing fill on the west levee in the buried beach area; raising the conventional levees and ramps constructed by local interests; and constructing the two remaining road ramps. The structural work consisted of completing the existing and constructing the new I-walls; and constructing the T-walls and gates. Work pertinent to hydrostatic uplift relief in the buried beach area consisted of installing the relief wells and piezometers; and constructing the collection facilities for the disposal of the discharge from the wells.

The sequence of construction in the buried beach area was to be as follows: install steel sheet piling, degrade, re-handle, and shape the existing fill, install pressure relief wells, piezometers, and collector systems; construct the concrete I-wall on the steel sheet piling; and fill and dress the levee crowns to grade and section. Semi-compacted fill methods of construction were to be used in placing the earth fill.

Where earth filling was to be required along the levees in which the steel sheet pile had not been installed, the fill was to be placed using semi-compacted methods in advance of installation of the steel sheet piling and wall construction to reduce the ultimate settlement of the walls. For the same reasons, the fill for road ramps was to be placed ahead of the tie-in wall construction.

3.2.1.5.3.2.7. Supplemental Design Information – IHNC Remaining Levees, West Levee Vicinity France Road and Florida Avenue (Reference 11). (This report presents the information required to support the design of a reach of alignment between Stations 210+75 and 237+44.51 that has been revised since Reference 9 was submitted.)

3.2.1.56.3.2.7.1. Geology. The geologic history and principal physiographic features of the New Orleans area and also the surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.5.3.2.7.2. Project Foundation Conditions. The subsurface soils along this reach consist of 8 to 14 feet of fill materials underlain by about 5 ft of recent marsh deposits, 20 feet of interdistributary clays and 10 to 20 feet of estuarine. The Recent soils are in turn underlain by Pleistocene age deposits below elevations ranging from about -70 to -75 feet.

3.2.1.5.3.2.7.3. Field Investigation. In addition to the borings given in Reference 9, four 5-inch diameter undisturbed borings and five 1-7/8 inch ID general-type borings were made for the protective works on the revised alignment.

3.2.1.5.3.2.7.4. Cantilever I-Type Floodwalls. The stability and required penetration of the steel sheet pile below ground surface were determined by the Method of Planes for both the (Q) and (S) shear strength cases. A factor of safety of 1.50 was applied to the design shear strengths. The required depths of penetration were determined for hurricane water level 6 inches below the top of the floodside, and water level equal to the water table on the protected side. Factors of safety were also determined for the headwater level at the top of the walls.

3.2.1.5.3.2.7.5. Slope Stability. Stability analyses of the levee, with the I-wall, were made for the (Q) condition using the Method of Planes.

3.2.1.5.3.2.7.6. Pile Foundation. Pile bearing capacities for the gated structures and T-walls were determined from the pile test performed at site 1 of the IHNC West Levee, Florida Avenue to IHNC Lock project, where subsurface conditions are similar to those at the proposed site of the T-wall and gates. Results of this test were taken from the Pile Test Report, September 1967. Results of the load test were given in terms of ultimate load versus tip elevation. Design loads for this project needed to be multiplied by the proper safety factor, 1.75 for compression and 2.0 for tension, before using the graph. A minimum penetration elevation of -54.0 feet was required to assure adequate seating into the sand.

3.2.1.5.3.2.8. Design Memorandum No. 2, General, Advance Supplement, IHNC West Levee Florida Avenue to IHNC Lock (Reference 12)

3.2.1.5.3.2.8.1. Geology. The geologic history and principal physiographic features of the New Orleans area and also the surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.5.3.2.8.2. Project Foundation Conditions. The subsurface consists of Recent deposits varying in thickness from 60 to 70 feet overlain by 6 to 16 feet of fill materials. The Recent deposits are underlain by Pleistocene deposits (Prairie Formation). Generally, the Recent consists of a discontinuous layer of soft to stiff natural levee clays underlain by very soft marsh clays with organic matter and peat. Underlying the marsh and natural levee deposits are very soft to soft interdistributary clays with lenses and layers of silt and sand. Estuarine deposits of sand, clay, and silt with shell fragments underlie the interdistributary deposits and lie unconformably on top of the Pleistocene deposits.

3.2.1.5.3.2.8.3. Field Exploration. Four 5-inch diameter undisturbed soil borings and eighteen 1-7/8 inch ID general-type core soil borings were made along the project alignment. The borings were made at intervals varying from about 100 to 600 feet along the project location. The borings extended in depth to Elev. -48 feet to Elev. -75 feet. Four piezometers were installed on a range located at the floodwall centerline Station 43+37 to obtain existing pore pressures in the foundation clays for estimating residual settlement beneath the fill material. One 5-inch diameter undisturbed soil boring and seven 7-1/8-inch ID general-type soil boring were made along an alternate alignment later rejected.

3.2.1.5.3.2.8.4. Type of Protection. Because of the limited space available due to the proximity of roads, railroads, and existing industrial plant facility, the necessity for providing protection against seepage and potential erosion, the protective works were to consist of cantilever I-type floodwalls of steel sheet piling capped with a concrete wall where the wall height was less than 10 feet, and T-type concrete floodwalls with steel sheet pile cutoffs supported by 12-inch by 12-inch square prestressed concrete bearing piles where the wall height was more than 10 feet.

3.2.1.5.3.2.8.5. Seepage. The steel sheet piling associated with the “I” and “T” walls and gated structures were to provide protection against hazardous seepage. The minimum depth of cutoff was that required to penetrate the upper marsh deposit, and where the I-wall sheet pile penetration required for stability did not meet the requirement for cutoff, the necessary extension was made.

3.2.1.5.3.2.8.6. Cantilever I-Type Walls. Cantilever I-type floodwalls in levee fill were designed for the following loading conditions: top of wall at Elev. 15.0; water level on the floodside 6 inches below the top of the wall (1.5 feet above Stillwater level at Elev. 13.0) and groundwater on the protected side at elevation 0.0. The remaining I-type walls, with top at elevation 14.5 were designed with water 6 inches below top on floodside (1.0 foot above Stillwater level at Elev. 13.0), and groundwater on the projected side at Elev. 0.0. In the vicinity of the Chase Bag Company warehouse, an I-type wall analysis was performed for a reverse loading condition on the protected side due to a 200 psf load on the warehouse platform, with groundwater at elevation 0.0 on both sides of the wall. The stability and required penetrations of the steel sheet piles below the surface were determined by the Method of Planes using the (S) shear strengths. In determining the minimum penetration required for stability, a factor of safety of 1.5 was applied to the design shear strengths. Using the required penetrations, factors of safety were also determined for the water surface at the top of the walls. The foregoing procedures also were used in determining the penetrations and loading diagrams for analyzing the structural member by applying a factor of safety of 1.0 to the (S) soil shear strengths.

During review, the Division directed that the I-walls should also be analyzed for the Q case in five different reaches where the undrained shear strength varied from about 250 psf to 400 psf.

3.2.1.5.3.2.8.7. Levees. Using sections representative of existing conditions along the leveed portion of the wall alignment, the slopes and berm distances for the recommended levee were designed with the I-type wall in place for a hurricane water condition with water to elevation 14.5 on the flood side and varying from Elev. 0.0 to Elev. -6.0 feet on the protected side with assumed failure toward the protected side. The stability of the levee was determined by the Method of Planes using the design (Q) shear strengths and assigned piezometric conditions. A design levee section was determined by the Method of Planes for a minimum factor of safety of 1.3 based on the (Q) shear strengths.

3.2.1.5.3.2.8.8. Structure Foundations. Design bearing and tension capacities versus tip elevations were determined for four representative foundation conditions along the project alignment. Design data were determined for the (Q) and (S) shear strengths, disregarding the skin friction above the bottom of the Recent marsh deposit. A factor of safety of 1.75 was applied to the shear strengths in compression, and a factor of safety of 2.0 was applied to the shear strength in tension. Steel sheet pile seepage cutoffs were to be provided beneath the T-type walls and gated structures. Prior to construction, three 12-inch by 12-inch precast prestressed concrete piles of different lengths were to be driven at three locations. At each site, the short pile and the intermediate pile were to be tested in compression. If test results showed that either of these two piles could safely support twice the design loads, the long piles would not be tested. One pile at each site was to be tested in tension.

3.2.1.5.3.2.9. Orleans Parish Lakefront Levee, Orleans Marina (Reference 13). (This section covers the soil and foundation investigations and design for approximately 1,500 feet of floodwall (I-wall, T-wall, and road gates) along Lake Avenue and adjacent to the Orleans Marina, New Orleans, Louisiana. This is a portion of the hurricane protection plan that is contained in the larger project feature, Lake Pontchartrain, Louisiana & Vicinity, Orleans Parish Lakefront Levees, West of IHNC, GDM No. 2, Supplement No. 5. The proposed floodwall ties into the existing Lake Avenue ramp which is also part of the hurricane protection in the area. Design analyses for the Lake Avenue ramp are also included in this section.)

3.2.1.5.3.2.9.1. Geology. The geologic history and principal physiographic features of the New Orleans area and also the surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.5.3.2.9.2. Project Foundation Conditions. The subsurface consists of Holocene deposits approximately 60 feet thick underlain by sediments of Pleistocene age. Generally, the Holocene sediments consist of a surface layer approximately 6 to 10 feet thick of fill material underlain by a 5- to 10-foot thick layer of soft marsh clays and organic material. The marsh deposits are underlain by a layer of interdistributary clays approximately 20 to 25 feet thick which are in turn underlain by a layer of sand representing a buried beach approximately 3 to 6 feet thick. At the base of the Holocene deposits is a layer of prodelta clays between 15 and 20 feet thick.

3.2.1.5.3.2.9.3. Field Exploration. Undisturbed 5-inch diameter borings were made at two locations along the alignment. One additional undisturbed boring was located immediately outside of the project area. A general-type boring, 1-7/8-inch ID was also located in the vicinity of the project.

3.2.1.5.3.2.9.4. Cantilevered I-Type Walls. The stability and required penetration of the steel sheet pile below the surface was determined by the Method of Planes using (S) shear strengths. The (Q) analysis was performed to confirm that the (S) case governed for design. A factor of safety of 1.5 was applied to the design shear strengths. The required depths of penetrations to satisfy the stability criteria were determined as those where the summation of moments were equal to zero.

3.2.1.5.3.2.9.5. Levee/I-Wall and Ramp Slope Stability. The stability of the levees with I-walls was determined by the Method of Planes using the design (Q) shear strengths and conditions shown on the stability plate and applying a minimum factor of safety of approximately 1.3. The road ramp was also designed for the most critical conditions with the shear stability being determined by the Method of Planes and minimum factor of safety of 1.3.

3.2.1.5.3.2.9.6. T-Walls and Gates. T-type floodwalls supported by bearing piles were to provide the protection adjacent to the inverted T-type gates supported by bearing piles to provide access to the Orleans Marina.

A steel sheet pile cutoff was to be used beneath the gates and T-walls to provide protection against hazardous seepage during a hurricane. The sheet pile penetration required to satisfy Lane's weighted creep ratio (LWCR) of 3 was determined for the gates and the T-wall sections.

A conventional stability analysis utilizing a 1.30 factor of safety incorporated into the soil parameters was performed for various failure surfaces beneath the T-wall sections. In all cases below the base, the summation of horizontal driving and resisting forces indicated excess resistance. Therefore, the bearing piles are not required to carry any additional lateral load resulting from unbalanced loads transmitted to the structures.

Ultimate compression and tension capacities versus tip elevations were developed for both the (Q) and (S) cases. Values of adhesion and soil to pile frictional resistance shown in EM 1110-2-2906 were used in computing the pile capacities. The recommended tip elevations for cost estimating purposes were based on applying factors of safety of 2.0 in compression and tension.

During construction, test piles were to be driven and tested along the project alignment. The results of the pile tests were to be used to determine the length of the service piles.

3.2.1.5.3.2.10. Orleans Parish Lakefront Remaining Work (Reference 14). The portion of the DM covered here considers plans for modifying the existing Orleans Marina floodwall to provide high level plan protection. The existing Orleans Marina floodwall was constructed under the barrier plan.

3.2.1.5.3.2.10.1. Geology. The geologic history and principal physiographic features of the New Orleans area and also the surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.5.3.2.10.2. Project Foundation Considerations. A description of the project foundation conditions along the Orleans Marina floodwall is provided in Reference 13.

3.2.1.5.3.2.10.3. Field Exploration. Existing borings presented in Reference 13 were used in the design.

3.2.1.5.3.2.10.4. Cantilevered I-Type Wall Analyses. The required penetration of the steel sheet piling below ground surface was determined by the Method of Planes using either (S) case shear strengths or (Q) case design strengths. The factors of safety were applied to the design shear strengths. The required depth of penetration to satisfy the stability criteria was determined where the summation of moments was equal to zero.

Tip Penetrations

Q-Case

F.S. = 1.5 with water to SWL
F.S. = 1.25 with water to SWL and waveload
F.S. = 1.0 with water to SWL + 2 feet freeboard

S-Case

F.S. = 1.2 with water to SWL and waveload (if applicable)
F.S. = 1.25 with water to SWL and waveload for Pontchartrain Beach (Special case – defines existing criteria when constructed)

Bending Moments

Governing Tip Penetration Case

3.2.1.5.3.2.10.5. Stability of I-Wall/Levee. The stability of the I-wall in levee or natural ground was determined by the LMVD Method of Planes using the design (Q) shear strengths and applying a minimum factor of safety of approximately 1.3.

3.2.1.5.3.2.10.6. Pile Foundation. Pile load tests performed by the USACE during original construction of the Orleans Marina floodwall (Reference 13) were utilized in performing the pile analyses. Based on the pile load test data, the analyses used a factor of safety of 2.0. The resulting pile curves were to be used for cost estimating purposes.

3.2.1.5.3.2.10.7. Underseepage Beneath I-Walls. No mention was made in the DM regarding underseepage beneath the I-walls.

3.2.1.5.3.2.11. Orleans Parish Lakefront Levee West of IHNC (Reference 6)

3.2.1.5.3.2.11.1. Geology. The geologic history and principal physiographic features of the New Orleans area and also the surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.5.3.2.11.2. Project Foundation Conditions. The subsurface consists of Holocene deposits approximately 50 to 60 feet thick underlain by sediment of Pleistocene age. The Pleistocene sediments encountered in the borings consist of clays, silts, and silty sands. The contact surface varies from an elevation of -50 feet to -80 feet. Overlying the Holocene sediment is a surface layer of fill material approximately 6 to 15 feet thick. From baseline Station 313+00 to 351+00, a 10- to 12-foot thick layer of clays and organic marsh sediments underlies the fill materials. Underlying the marsh deposit and in the remaining portion of the project area underlying the fill material is a 25- to 30-foot thick layer of sediment deposited in a lacustrine environment. These deposits are clays and silts in the western and middle portion of the project area and grades laterally into silts, silty sands, and sands in the eastern portion of the project area. A 6- to 12-foot thick layer of sand representing a buried beach underlies the lacustrine deposits. This sand deposit thickens to 22 to 25 feet from Station 50+00 to the eastern limits of the section. It is composed of fine to medium grained sand, silty sand, and numerous shell fragments. At the base of the Holocene is a layer of baysound clays. This layer is 12 to 18 feet thick, and thins eastward to 6 to 8 feet.

3.2.5.3.2.11.3. Field Exploration. A total of 13 new 5-inch undisturbed borings were made for this GDM by the USACE, New Orleans District. Eight of the borings were made along the centerline of the levee and five borings were made at distances ranging from 50 to 105 ft lakeside of the baseline. In addition, 43 old borings that had previously been made at various times by the District were also considered in the design.

3.2.1.5.3.2.11.4. Levee

General. A conventional earthen levee was considered for the design to be the main protective feature for the project. The levee was to be constructed by enlarging the existing levee which was built by the Orleans Levee Board. The levee addition was to be constructed by placing semi-compacted clay fill to the design grades and section.

Slope Stability. Using cross sections representative of existing conditions along the levee, the stability of the levees and the levees with I-walls was determined for the most critical conditions by the Method of Planes using the design (Q) shear strengths and applying a minimum factor of safety of approximately 1.3.

Seepage Control. Seepage analyses were performed to determine the need for landside seepage berms. The analyses were performed using a maximum Bligh's creep ratio of 18 (very fine sand). Based on the analyses, seepage berms were required for four reaches, a clay cutoff was required for one levee reach of levee embankment, and a sheetpile cutoff was used in one reach to penetrate through the previous stratum. The following references were used for the analyses:

AD-A012-771 – Investigation of Underseepage and its Control, Lower Mississippi River Levees, Volume I, Army Engineer, Waterways Experiment Station, Vicksburg, Mississippi, October 1956. (I have it as TM No. 3-424.)

DIVR 1110-1-400, Section 8, Part 6, Item I, 30 Nov 76.

3.2.1.5.3.2.11.5. I-Walls.

General. The I-walls consist of a cantilever floodwall of sheet piling driven through existing levees and/or fill and capped with a concrete wall.

Cantilever I-Wall Analyses. The required penetrations for the stability of the cantilever walls were determined by the Method of Planes analysis. The walls were analyzed for both the short term (Q) case and the long-term (S) case. The factor of safety was applied to the design shear strengths. The following factors of safety (FS) were used in the analyses with the corresponding loading conditions:

For confined areas at Seabrook and Orleans Marina, FS used = 1.5 with static water at the top of the wall (Still water level (SWL) plus freeboard) and no dynamic wave force.

For unconfined areas along the lakefront with adjacent open water, FS used = 1.5 with static water at the SWL (and no dynamic wave force) and FS used = 1.25 with static water at the SWL and a dynamic wave force.

Sheet Pile Penetration. The sheet pile penetration required to satisfy Lane's weighted creep ratio of 3.0 to 8.5 depending on soil type was determined for various I-wall sections. The deeper penetration of the two analyses (cantilever I-wall or creep ratio) was selected as the recommended tip elevation of the sheet pile floodwall except where the soil boring data indicated that a slightly deeper penetration would be preferable.

Slope Stability. The stability of the levees with I-walls was determined by the Method of Planes using the design shear strengths and appropriate hydraulic loading, and applying a minimum factor-of-safety of approximately 1.3.

3.2.1.5.3.2.11.6. Anchored Bulkhead. Lateral soil pressures used for the analysis of the anchored sheet pile bulkhead portion of the floodwall were developed by a Method of Planes analysis. For determination of the required sheet pile penetration, a factor of safety of 1.5 was applied to the "S" soil parameters. For determination of maximum bending moment and required anchor force, a factor of safety of 1.0 was applied to the soil parameters.

3.2.1.5.3.2.11.7. T-Walls and Gates

General. T-type floodwalls supported by bearing piles were designed to provide the protection at road gates.

Sheet Pile Cutoff. A steel sheet pile cutoff was to be used beneath the gates and T-walls to provide protection against seepage during a hurricane. The sheet pile penetration required to satisfy Lane's weighted creep ratio (LWCR) of 3.0 to 8.5 depending on the soil type was determined for the gates and the T-wall sections.

Deep Seated Stability Analysis. A conventional slope stability analysis utilizing a 1.30 factor of safety incorporated into the soil parameters was performed for various failure surfaces beneath the gates and the T-wall sections. In all cases below the base, the summation of horizontal driving and resisting forces indicated decreasing unbalanced loads. Therefore, the bearing piles were determined to not carry any additional lateral load resulting from unbalanced loads transmitted to the structures.

Bearing Pile Foundation. Ultimate compression and tension pile capacities versus tip elevations were developed for the (Q) case and (S) case. During construction, selected piles were to be driven and tested at some locations along the project alignment. The results of the pile load tests were to be used to determine the length of the service piles by applying a factor of safety of 2.0. In areas where no pile tests were to be performed, the service length of the pile was to be determined by incorporating a factor of safety of 3.0.

3.2.1.5.3.2.11.8. Slope Stability at Road Ramps. Slope stability analyses were performed on the ramp sections determined to be most critical with respect to slope stability. The analyses were performed using the Method of Planes and a minimum factor of safety of 1.3 with respect to shear strength.

3.2.1.5.3.2.11.9. Division Review. No criteria changes resulted from Division review.

3.2.1.5.3.3. Structural.

3.2.1.5.3.3.1. Orleans East Bank Lakefront - Pontchartrain Beach Floodwall – Reference 7

General. As constructed, the Pontchartrain Beach Floodwall consists of earthen levee, combination earthen levee and capped cantilevered I-wall, and three pile-founded swing gates.

Design Loads.

Design static water level is El. 11.5 NGVD

I-walls Top El. 20.0 NGVD and Levee Crown 13.0 NGVD, the dynamic wave force is 5,632 pounds per foot.

Levee / Floodwall Sections

Slope Stability

Levees and levee I-wall combinations designed for a Factor of Safety of 1.3 using LMVD Method of Planes

Cantilever Analysis

Cantilever Analysis used with a Factor of Safety of 1.5 factored into the soil shear strength parameters for the static water level loading.

Cantilever Analysis used with a Factor of Safety of 1.25 factored into the soil shear strength parameters for the dynamic water level loading

Factor of Safety of 1.0 for the second case used to determine maximum bending moment.

This resulted in a tip elevation of -14.0 NGVD.

Maximum desirable deflection of the wall was 1.5 inches.

Gate Structures

Allowable Pile Load Capacity

Recommended loads for 14” square, precast, prestressed concrete piles based on a factor of safety of 3. No load test performed.

Pile Loads

Hrennikoff method of analysis used to analyze distribution of loads to piles. Coefficient of horizontal subgrade reaction computed using in-situ field tests and laboratory test data.

3.2.1.5.3.3.2. Orleans East Bank Lakefront - Orleans Parish Lakefront Levees Orleans Marina – Reference 13

General. As constructed, the Orleans Marina hurricane protection system consists of combination earthen levee and capped cantilevered I-wall tying into the 17th Street Canal and Orleans East Bank Lakefront Levee hurricane protection systems; four pile-founded roller gates (Lake Marina Drive, two at the entrances to the New Orleans Municipal Yacht Harbor, and Lakeshore Drive); one pile-founded swing gate at Pontchartrain Boulevard; pile-founded T-wall; and capped cantilevered I-wall with tie-back system.

Structural Design Criteria

Basic Data

Water Elevations

Item	*Elevation (ft msl)*
Wind Tide Level (IHNC)	13.0
Wind Tide Level (Lake Pontchartrain)	8.5
Landside of floodwall	0.0

Floodwall Gross Grades

Item	*Top Elevation (ft msl)*
I-walls	11.0
T-walls and Gates	10.5

Unit Weights

Item	*lb per cu ft*
Water	62.5
Concrete	150
Steel	490

Design Loads

Wind	50 psf
Water	62.5 pcf

Allowable Working Stresses. The allowable working stresses for concrete and structural steel are in accordance with those recommended in "Working Stresses for Structural Design, EM 1110-1-2101, dated 1 November 1963 and Amendment No. 1 dated 14 April 1965. The basic minimum 28-day compressive strength for concrete will be 4,000 psi. except for prestressed concrete piling where the minimum strength will be 5,000 psi. Steel for steel sheet piling will meet the requirements of ASTM A328-69, "Standard Specification for Steel Sheet Piling". Pertinent allowable stresses are tabulated as follows:

Reinforced Concrete	
fc'	4,000 psi
fc	1,400 psi.
v_c (without web reinforcement)	60 psi
v_c (with web reinforcement)	274 psi
fs	20,000 psi
Minimum area steel	0.0025 bd
Shrinkage and temperature steel area	0.0020 bt
Structural Steel (ASTM A-36)	
Basic working stress	18,000 psi

In the design of the I-wall, one loading case was considered:

Case I Static water at top of wall, no wind, no dynamic wave force

Depth of penetration was determined by applying a factor of safety of 1.5 to the "S" Case soil shear strengths. The "Q" analysis was performed but the "S" Case governed.

Gates. The gates and gate monoliths were designed for the following cases:

Case I	Gate closed, water at top of wall, no wind, impervious sheet pile cutoff
Case II	Gate closed, water at top of wall, no wind, pervious sheet pile cutoff
Case III	Gate opened, no water, no wind, truck on edge of slab on floodside
Case IV	Gate opened, no water, no wind, truck on edge of slab on protected side
Case V	Gate opened, no water, wind from protected side, truck on edge of slab on floodside, 33-1/3 percent increase in allowable stresses
Case V	Gate opened, no water, wind from floodside, truck on edge of slab on protected side, 33-1/3 percent increase in allowable stresses

3.2.1.5.3.3.3. Orleans East Bank Lakefront - Orleans Parish Lakefront Levee West of IHNC – Reference 6

General. As constructed, the project consists of primarily earthen levee. At Topaz Street, Marconi Drive, and Leroy Johnson Drive, the protection consists of one pile-founded double swing gate and combination levee and capped cantilevered I-wall. In the vicinity of Rail Street, and at Bayou St. John, there is a short reach of combination earthen levee and capped canti-levered I-wall. At the American Standard plant at the end of Franklin Avenue, the protection consists of capped cantilevered I-wall and pile-founded T-wall. At its eastern end, there is a short stretch of capped cantilevered I-wall with a railroad swing gate. The project ties into the Orleans Marina hurricane protection at its western end, the parallel protection of Orleans Avenue Outfall Canal, Bayou St. John, and London Avenue Outfall Canal, Pontchartrain Beach Floodwall, and the IHNC hurricane protection at its eastern end.

Structural Design Criteria

Basic Data

Water Elevations

Water Elevation	*Elevation (ft NGVD)*
Wind Tide Level (IHNC)	13.0
Wind Tide Level (Lake Pontchartrain)	8.5
Landside of floodwall	0.0

Floodwall Gross Grades

Item	*Top Elevation (ft NGVD)*
I-walls	Vary from 14.0 to 20.5
T-walls and Gates	Vary from 13.5 to 20.75

Unit Weights

	lb per cu ft
Water	64.0
Concrete	150
Steel	490

Design Loads

Wind	50 psf

Design methods

Structural steel. The design of steel structures is in accordance with the requirements in "Working Stresses for Structural Design", EM 1110-1-2101, dated 1 November 1963 and Amendment No. 2 dated 17 January 1972. The basic working stress for ASTM A-36 steel is 18,000 psi. Steel for steel sheet piling will meet the requirements of ASTM A328, "Standard Specification for Steel Sheet Piling."

Reinforced Concrete. The design of reinforced concrete structures is in accordance with the requirements of strength design of the strength design method of the current ACI Building Code, as modified by the guidelines of "Strength Design Criteria for Reinforced Concrete Hydraulic Structures," ETL 1110-2-265 dated 15 September 1981. The basic minimum 28-day compressive strength for concrete will be 3,000 psi except for prestressed concrete piling where the minimum strength will be 5,000 psi. Pertinent allowable stresses are tabulated as follows.

Reinforced Concrete	
f'c	3,000 psi
fy (grade 40 steel)	40,000 psi
Maximum flexural reinforcement	0.25 x balance ratio.
Minimum flexural reinforcement	200/fy
f'c (for prestressed concrete piles)	5,000 psi
Fu (prestressing strands, Gr 250)	250,000 psi

I-Walls. In the design of the I-walls, two loading cases were considered:

Case I	(1) For confined areas, FS used = 1.5. Static water at top of wall (still water level (SWL) plus freeboard), no dynamic wave force (2) For unconfined areas: FS used = 1.5. Static water at top of wall (still water level (SWL) plus freeboard) and no dynamic wave force FS used = 1.25. Static water at top of wall (still water level (SWL) plus freeboard) and a dynamic wave force
Case II	No water, lateral earth pressure

Both the short term (Q) case and long term (S) case were used in analyzing the walls.

T-Walls. In the design of the T-walls, the following load cases were considered:

Case I	Static water pressure, no wind, impervious sheet pile cutoff, no dynamic wave force
Case II	Static water pressure, no wind, pervious sheet pile cutoff, no dynamic wave force
Case III	Still water pressure to El. 11.5, dynamic wave force, no wind, impervious sheet pile cutoff (75% forces used)
Case IV	Still water pressure to El. 11.5, dynamic wave force, no wind, pervious sheet pile cutoff (75% forces used)
Case V	No water, no wind
Case VI	No water, wind from protected side (75% forces used)
Case VII	No water, wind from flood side (75% forces used)

Swing Gates, Miter Swing Gates, and Bottom Roller Gates

Case I	Gate closed, still water pressure to El. 11.5, dynamic wave force, impervious sheet pile cutoff (75% forces used)
Case II	Gate closed, still water pressure to El. 11.5, dynamic wave force, pervious sheet pile cutoff (75% forces used)
Case III	Gate open, no wind, truck or train on protected edge of base slab
Case IV	Gate open, no wind, truck or train on floodside edge of base slab
Case V	Gate open, wind from protected side, truck or train on edge of slab on floodside edge of base slab, (75% forces used)
Case VI	Gate open, wind from flood side, truck or train on edge of slab on protected side edge of base slab, (75% forces used)

Vertical Lift Roller Gate

Case I Gate closed, water to top of gate on flood side, no water on protected side

3.2.1.5.3.3.4. Orleans East Bank Lakefront Orleans Avenue Outfall Canal - Reference 4

General. As constructed, the east side of Orleans Avenue Outfall Canal hurricane protection system consists of combination earthen levee and uncapped cantilever I-wall tying into the existing non-Federal levee at New Orleans Sewerage and Water Board Drainage Pumping Station #7; combination of earthen levee and capped cantilevered I-wall; and earthen levee tying into the Orleans East Bank Lakefront Levee. The west side consists of combination earthen levee and uncapped cantilevered I-wall tying into the existing non-Federal earthen levee at the New Orleans Sewerage and Water Board Drainage Pumping Station #7, combination earthen levee and capped cantilevered I-wall from New Orleans Sewerage and Water Board Drainage Pumping Station #7 to French Street, and at bridge crossings; combination earthen levee and pile-founded T-wall from French Street to Robert E. Lee Boulevard and in the vicinity of Germain Street; and combination earthen levee capped cantilevered I-wall tying into the Orleans Lakefront Levee hurricane protection system. At the Marconi Drive double swing gate. In addition there are three pile–founded floodproofed bridges (Robert E. Lee Boulevard, Filmore Avenue, and Harrison Avenue). Fronting protection of New Orleans Sewerage and Water Board Drainage Pumping Station #7 has not been constructed.

Structural Design

Design Criteria

I-Walls.

Q–Case

F.S. = 1.5 with water to SWL
F.S. = 1.25 with water to SWL and wave load
F.S. = 1.0 with water to SWL + 2 feet freeboard

S–Case

F.S. = 1.2 with water to SWL and wave load (if applicable)

Wave loading not applied to design of the canal parallel protection

If the penetration to head ratio is less than 3:1, it is increased to 3:1 or that required by the S – Case with a FS = 1.5, whichever results in the least penetration. The SWL is used to calculate the head ratio.

The following criteria was contained in a letter from Frederic M. Chatry, Chief Engineering Division, New Orleans District, Army Corps of Engineers, dated 11 April 1985, to Mr. John Holtgreve, Design Engineering, Metairie, Louisiana in regards to the design requirements for flood protection along the Orleans Avenue Canal. This letter was contained in above referenced GDM No. 19 Volume II, pages B-3 through B-5.

Concrete Design based on ETL 1110-2-265 dated 15 September 1981.

Design Criteria and Standards for Floodgates. Gates are designed by the working stress method using an allowable bending stress Fb = 0.55 Fy using A36 steel.

Load Cases

Case I	Water to top of gate
Case II	Wind load of 50 pounds per square foot on the gate

3.2.1.5.3.3.5. INHC Canal (West Bank) – Reference 9. The structural features consist predominantly of cantilever I-type floodwalls of steel sheet piling driven through existing levees, and/or fill, and capped with a concrete wall. T-type floodwalls supported by bearing piles will provide the protection in the more congested areas in the vicinity of road and railroad crossings.

Basic Data Maximum wind tide levels along the IHNC resulting from the design hurricane vary from elevation 11.4 at Seabrook to 12.9 at the L&N Railroad Bridge and then to 13.0 at the IHNC Lock. Water elevations landside of the floodwall vary from elevation zero to elevation -3.0. The elevation of the top of an I-wall in a levee is 2.0 feet above the wind tide level. The elevation of the top of T-type walls and gates are 1.0 foot above the wind tide level.

Unit Weights	lb. per cu ft
Water	62.5
Concrete	150
Steel	490

Water Loads

- No wave forces will occur
- One foot freeboard

I-type Floodwall. Bending moments and deflections for structural design of sheet piles were based on a factor of safety of 1.5 applied to the soils. The strength of the wall was checked for the case with water at the top of the wall and found to be adequate.

Design of T-type Wall for West Levee. The T-type floodwalls for the west levee were designed for the following conditions:

- Case 1 - Water at elevation 14.0 on the floodside and elevation zero on the protected side. Steel sheet pile cutoff impervious. Uplift with full head on floodside of cutoff and tailwater on the protected side. Earth fill to elevation 5.0.

- Case 2 - Same as Case 1 except steel sheet pile cutoff pervious. Uplift varies uniformly from full head on floodside to tailwater on the protected side.

- Case 3 - Water at elevation 11.0 on floodside and water at elevation zero on protected side. Impervious cut off. . Uplift as in Case 1.

- Case 4 - Same as Case 3 except cutoff pervious and uplift as in Case 2.

- Case 5 - Water at elevation 10.0 on floodside and at elevation zero on protected side. Impervious cutoff. Uplift as in Case 1.

- Case 6 - same as Case 5 except cutoff pervious and uplift as in Case 2.

In all cases, the at rest earth pressure was assumed to be 75% of the submerged unit weight of earth (55#/cu. ft.) on the floodside and cracked section assumed on protected side.

3.2.1.5.3.3.6. IHNC West Levee - Florida Avenue to IHNC Lock –Reference 12. The protective works covered herein consist of approximately 2,150 feet of “I”-type cantilever floodwall and 4,900 feet of inverted “T”-type floodwall. Eleven overhead roller gates and three swing gates are provided where the alignment crosses vehicular roads and railroads, and a flap gate is provided a t the loading platform of the Jones & Laughlin Steel Company warehouse.

Structural Design

Design Criteria

Basic data

Water Elevations	*Elevations*
Project flow line (surge elevation from design hurricane)	13.0
Landside of floodwall	0.0

Floodwall grades	*Elevations*
Net grade (one foot freeboard over project flow line)	14.0
Top of wall, I-type wall in levee (as constructed)	15.0
Top of wall, I-type wall in natural ground (as constructed)	14.5
Top of wall, T-type wall in levee (as constructed	14.0
Top of access gates (as constructed)	14.0

Unit Weights

Item	*lb per cu ft*
Water	62.5
Concrete	150.0
Steel	490.0

Design Loads

Water loads

No wave force
One foot freeboard
Design Water Elevations as follows

Design Water Elevations

	Flood side	*Protected side*
I-wall in levee	14.5	0
T-wall in natural ground	14.0	0
T-wall	14.0	0

Wind loads

On walls ....30 psf
On overhead beams...50 psf

Allowable Working Stresses. The allowable working stresses for concrete and structural steel are in accordance with those recommended in "Working Stresses for Structural Design", EM 1110-1-2101, dated 1 November 1963 and Amendment No. 1 dated 14 April 1965. The basic minimum 28-day compressive strength for concrete will be 4,000 psi. except for prestressed concrete piling where the minimum strength will be 5,000 psi. Steel for steel sheet piling will meet the requirements of ASTM A328-69, "Standard Specification for Steel Sheet Piling". Pertinent allowable stresses are tabulated as follows:

Reinforced Concrete	*Stress (psi)*
fc'	3,000
fc	1,050
v_c (without web reinforcement)	60
v_c (with web reinforcement)	274
fs	20,000
Minimum tensile steel	0.0025 bd
Shrinkage and temperature steel area	0.0020 bt
Structural Steel (ASTM A-36)	
Basic working stress	18,000

I-Wall Design. The penetration for I-walls was determined based on the "S" case and with a factor of safety equal to 1.5. Water was assumed at 6 inches below the top of wall on the flood side and at 0.0 on the protected side. Bending moments and deflections were determined by applying a factor of safety of 1.0 to the soil parameters. LMVD in the 1st Ind of 13 Apr 67 stated that bending moments, stresses and wall deflections for I-walls should be computed using the same earth and water pressure diagrams as those used in determining the pile penetration. However, where the sheet piling is in clay and the "S" case governs, LMVD permitted a 1/3 overstress. Where the sheet piling is in clay and the "Q" case governs, no overstress was permitted by LMVD. In the 2d Ind of 31 May 67, NOD concurred with using the same earth and water pressure diagrams as those used in determining pile penetration for computing bending moments and stresses, but further stated that an overstress should be permitted for either shear strength that governs the design.

T-Wall Monoliths. The T-type floodwalls were designed for the following conditions:

Case I	Water at elevation 14.0 on flood side and water at elevation 0.0 on protected side. Sheet pile cutoff pervious. Uplift varies uniformly from full head on flood side to tailwater on protected side.
Case II	Same as Case I except sheet pile cutoff impervious. Uplift full head on flood side of cutoff and tailwater on protected side.
Case III	Water at elevation 10.0 on flood side and water at elevation 0.0 on protected side. Pervious cutoff. Uplift as in Case I.
Case IV	Same as Case III except sheet pile cutoff impervious. Uplift as in Case II.
Case V	Water at elevation 7.5 on flood side and water at elevation 0.0 on protected side. Pervious cutoff. Uplift as in Case I.
Case VI	Same as Case V except sheet pile cutoff impervious and uplift as in Case II.
Case VII	No water, wind from canal side (75% forces used)

In all cases, the earth pressure was assumed to be balanced.

Three methods of analysis were used to check the pile foundations. They are as follows:

- “Analysis of Pile Foundations with Batter Piles”, by A. Hrennikoff, Transactions, ASCE Vol. 115 (1950). Used for checking all layouts.)

- “Design of Pile Foundations”, by G. Vetter, Transactions, ASCE Vol. 104 (1939). (Used for checking the layout with two batter piles.)

- “Culmann’s method for the Design of Pile Foundations” from “Theoretical Soil Mechanics” by K. Terzaghi. (Used for checking the two layouts with one vertical and two batter piles.)

These studies indicate that a foundation consisting of two piles battered in opposite directions is the most suitable and economical for the for the T-type walls.

3.2.1.5.3.3.7. 17th Street Outfall Canal (Metairie Relief) – Reference 2

General. As constructed, the 17th Street Outfall Canal hurricane protection system consists of earthen levee with capped cantilevered I-wall tying into geotextile-reinforced earthen levee of the Jefferson Lakefront Levee hurricane protection on the west side of the canal and the Orleans Marina hurricane protection on the east side of the canal. In addition, there is one pile founded swing gate at Orpheum Avenue; two pile-founded floodproofed bridges (Veterans Boulevard and Old Hammond Highway); and fronting protection of New Orleans Sewerage and Water Board Drainage Pumping Station #6. All hurricane protection construction was completed along the canal.

Fronting protection of the Canal Street Pumping Station in Jefferson parish consists of butterfly valves to prevent water backflow through the pumps when pump operation ceases due to high water levels in Lake Pontchartrain. Fronting protection of the I-10/Metaire Road Underpass Pump Station consists of a combination siphon and valves to prevent water backflow through the pumps when pump operation ceases due to high water levels in Lake Pontchartrain. Work at both stations was performed by local interests and is not covered by design documents.

The basic data relevant to the design of the protective works are shown in the following table:

Water Elevations	**Elevations (feet NGVD)**
Wind tide level (Lake Pontchartrain)	11.50
Wind tide level (17th Street Outfall Canal)	11.50 to 12.50

Unit Weight	**lb. per cu ft**
Water	64
Steel	490
Concrete	150

Reinforced Concrete: The design of reinforced concrete structures is in accordance with the requirements of the strength design method of the current ACI Building Code, as modified by the guidelines of "Strength Design Criteria for Reinforced Concrete Hydraulic Structures", ETL 1110-2-312 dated 10 March 1988. The basic minimum 28-days compressive strength concrete will be 3,000 psi except for bridge superstructure and for prestressed concrete piling, where the minimum will be 4,000 psi and 5,000 psi, respectively. For convenient reference, pertinent stresses are tabulated below:

fc	3,000 psi
fy (Grade 60)	48,000psi
Maximum flexural reinforcement	0.25 x balanced ratio
Minimum flexural reinforcement	200/fy
fc (for bridge superstructure concrete)	4,000 psi
fc (for prestressed concrete piles)	5,000 psi
fy (for prestressed strand grade 250)	250,000
fy (for prestressed strand grade 270)	270,000

I-Type Floodwall The following loading cases were considered.

Flood Analysis

- Case I: Water to SWL, Q case F. S. = 1.5
- Case II: Water to SWL + 2 feet free board, Q case F. S. = 1.0
- Penetration to head ratio equal to 2.5:1

Bulkhead Analysis

- Canal Water at El 0.0, S case F.S. = 1.5

Floodgates and Gate Monoliths The foundation piles for the gate monoliths were designed with a factor of safety (F.S. = 3). Because of the small number of piles, pile tests were not considered to be economical for this work. The following load cases were used for the preliminary design of these gates.

- Case I: Gate closed static water pressure to SWL, no wind, impervious sheet pile cutoff, dynamic wave force (100 % forces used).
- Case II: Gate closed static water pressure to SWL, no wind, pervious sheet pile cutoff, no dynamic wave force (100% forces used).
- Case III: Gate closed static water pressure with water level 2 feet above. SWL, no wind, impervious sheet pile cutoff no dynamic wave force (75% forces used).
- Case IV: Gate closed static water pressure with water level 2 feet above SWL, no wind, pervious sheet pile cutoff, no dynamic wave force (75% forces used).
- Case V: Gate open, no wind, truck on protected side edge of base slab (100 % forces used)
- Case VI: Gate open, no wind, truck on flood side edge of base slab (100% forces used).
- Case VII: Gate open, wind from protected side, truck on floodside edge of base slab (75% forces used).
- Case VIII: Gate open, wind from floodside, truck on protected edge of base slab (75% forces used).

3.2.1.5.3.3.8. Improvements to the Fronting Protection at Pump Station No. 6, 17th Street Outfall Canal – Reference 34

General. As constructed, fronting protection of PS#6 consists of pile-founded T-wall, pile-founded sluice gates for horizontal and wood screw pumps, and butterfly valves for vertical pumps to prevent water backflow through the pumps when pump operations cease due to high water levels in Lake Pontchartrain.

Sheet Pile Penetration Analysis Sheet pile penetration was determined using the USACE's program CWALSHT. A F.S. of 1.5 was used for permanent I-walls and temporary cofferdams.

H- Pile Capacity Computations The ultimate pile capacities were divided by the following factors of safety to determine the design pile capacity for axial loading:

	Minimum Factor of Safety	
Loading Condition	**W/Pile Load Test**	**W/Out Pile Load Test**
Construction Case	1.5	2.25
Water to Still Water Level	2.0	3.0
Normal Operating Case	2.0	3.0
Maintenance Case	1.5	2.25
Water to 2' Above Still Water Level	1.5	2.25
Flood Water on Protected Side (for east monolith only)	1.5	2.25

Material Weights The following material weights were used in the calculations:

Item	Lbs. per cubic foot
Water (Brackish)	64.0
Concrete	150.0
Steel	490.0
Rip-rap	165.0
Saturated Sand	122.0
Saturated Clay	110.0
Saturated Random Backfill	120.0

Design Stresses

Structural Steel. The basic stresses for structural steel are according to the 9th Edition of the AISC Manual of Steel Construction as modified by EM 1110-2-2101. This EM requires that all AISC allowable stresses be reduced by 17%, as a basis for design.

Reinforced Concrete. The design of concrete is in accordance with the strength design methods and criteria established in EM 1110-2-2104 including a durability factor of 1.3 (H_f)

f'c	3,000 psi
Maximum flexural reinforcement	0.25 x balance ratio
Minimum flexural reinforcement	200/fy OR 1.3 x Design Requirement
Temperature Reinforcement	.0028(Ag)

Reinforcement. The design strength of reinforcement is based on the use of ASTM A-615 Grade 60 steel, having yield strength of 60,000 psi. Strength design is based on yield strength of 48,000 psi according to EM 1110-2-2104. Development lengths are based on the full yield strength of 60,000 psi.

Steel H-Piles. The allowable stress used for H-piling is 18 ksi for A-36 which is in accordance with EM1 110-2-2906.

Sheet Piles. Allowable stresses for sheet piling used is based on an allowable stress of 18,000 psi plus allowable over stress if applicable.

OverStresses. An allowable overstresses of 331/3 is permitted for construction, maintenance, 2' above still water, and flood on protected side conditions.

Uniform Live Loads. The following uniform live loads are used in the calculations:

Item	**Lbs. per Sq. Ft.**
Construction LL	20
Operating Floor	60

Loading Conditions The following load cases were considered when designing the structural components of the proposed structures. Headwater (H.W.) represents stages on the flood side of the structure and tailwater (TW1) represents stages on the protected side of the structure. TW2 indicates water level inside discharge tube equal to the highest invert elevation of the tube when gate is closed.

East Sluice Gate Monolith

- Case I (Construction) Site Dewatered Dead Load, Construction Live Load, Wind Load, Backfill on Monolith (75% forces used).

- Case II (Still Water Level) HW Elevation = 12.6', Gate Closed with Water in Tube , TW2 Elevation = 3.9**,** Dead Load, Live Load, Wind Load, Backfill on Monolith, Impervious Cut-Off Wall (100% forces used).

- Case III (Still Water Level) HW Elevation = 12.6', Gate Closed with Water in Tube, TW2 Elevation =3.9**,** Dead Load, Live Load, Wind Load, Backfill on Monolith, Impervious Cut-Off Wall (100% forces used).

- Case IV (Normal Operating) HW Elevation = 2.0', Gate Open, Dead Load, Live Load, Backfill on Monolith, Impervious Cut-off Wall (100% force used).

- Case V (Maintenance) HW Elevation = 2.0, Stop Logs in Place, Monolith Dewatered, Dead Load, Live Load, Backfill on Monolith, Impervious Cut-Off Wall (75% forces used).

- Case VI (2' Above Still Water Level) HW Elevation = 14.6', Gate Closed with Water in Tube, TW2 Elevation =3.9**,** Dead Load, Live Load, Wind Load, Backfill on Monolith, Pervious Cut-Off Wall (75% forces used).

- Case VII (2' Above Still Water Level) HW Elevation = 14.6', Gate Closed with Water in Tube, TW2 Elevation =3.9'**,** Dead Load, Live Load, Wind Load, Backfill on Monolith Impervious Cut-Off Wall (75% forces used).

- Case VIII (Flood on Protected Side) HW Elevation = -5.01**,** TWI Elevation = 14.6', Dead Load, Live Load, Backfill on Monolith, Impervious Cut-Off Wall (75% forces used).

Groundwater elevation on protected side is below invert of structure for east monolith.

West Sluice Gate Monoliths

- Case I (Construction) Site Dewatered, Dead Load, Construction Live Load, Wind Load, Backfill on Monolith, Uniform Uplift Pressure (75% forces used).
- Case II (Still Water Level) HW Elevation = 12.6, TW1 Elevation = 12.6, Gate Closed with Water in Tube, TW2 Elevation = 5.0', Dead Load, Live Load, Wind Load, Backfill on Monolith, Uniform Uplift Pressure (100% forces used).
- Case III (Normal Operating) HW Elevation = 2.0', TW1 Elevation = 2.0', Gate Open, Dead Load, Live Load, Backfill on Monolith, Uniform Uplift Pressure (1 00% force used).
- Case IV (Maintenance) HW Elevation = 2.0', TW1 Elevation = 2.0', Stop Logs in Place, Monolith Dewatered, Dead Load, Live Load, Backfill on Monolith, Uniform Uplift Pressure (75% forces used).
- Case V (2' Above Still Water Level) HW Elevation = 14.6', TW1 Elevation = 14.6', Gate Closed with Water in Tube, TW2 Elevation = 5.0', Dead Load, Live Load, Wind Load, Backfill on Monolith, Uniform Uplift Pressure (75% forces used).

The ground water elevation causing uplift at the west monoliths shall be the same as the flood side since flood waters are allowed to surround these monoliths.

East I-Wall at East Monolith (2' Above SWL). HW = 14.6', TW1 = 3.8' Ground Elevation 3.8' on FS and PS

West I-Wall at East Monolith (Min. Water with Backfill). HW = 3.0' Ground Elevation 3.0' on FS, Ground Elevation 14.0 on PS

3.2.1.5.3.3.9. London Avenue Outfall Canal - Reference 5, Supplemented by References 35 and 75

General. As constructed, the London Avenue Outfall Canal hurricane protection system consists of earthen levee which ties into the Orleans Lakefront Levee hurricane protection project on both sides of the canal, and earthen levee with capped cantilevered I-wall tying into In addition, there are two pile-founded railroad swing gates; four pile-founded floodproofed bridges (Gentilly Boulevard, Mirabeau Avenue, Filmore Avenue, Leon C. Simon Avenue); and fronting protection of New Orleans Sewerage and Water Board Drainage Pumping Station #4. Fronting protection of PS#4 consists of pile-founded sluice gates to prevent water backflow through the pumps when pump operations cease due to high water levels in Lake Pontchartrain. Floodproofing of the Robert E. Lee Bridge and fronting protection of New Orleans Sewerage and Water Board Drainage Pumping Station #3 have not been constructed.

Structural Design

Design Criteria

Structural steel. The design of steel structures is in accordance with the requirements in "Working Stresses for Structural Design", EM 1110-1-2101, dated 1 November 1963 and Amendment No. 2 dated 7 January 1972. The basic working stress for ASTM A-36 steel is 18,000 psi. Steel for steel sheet piling will meet the requirements of ASTM A328, "Standard Specification for Steel Sheet Piling".

Reinforced Concrete. The design of reinforced concrete structures is in accordance with the requirements of the strength design method of the current ACI Building Code, as modified by the guidelines of "Strength Design Criteria for Reinforced Concrete Hydraulic Structures", ETL 1110-2-312 dated 10 March 1988, which supersedes ETL 1110-2-265 dated 15 September 1981. Pertinent stresses are tabulated as follows:

Pertinent Stresses for Reinforced Concrete Design

fc'	3,000 psi
fy (Grade 60 steel)	48,000 psi
Maximum Flexural Reinforcement Ratio	0.25 x Balance Ratio
Minimum Flexural Reinforcement Ratio	200/fy
fc'(Prestressed Concrete Piles)	5,000 psi
fu (Prestressing Strands Grade 270)	270,000 psi

Unit Weights

Item	*lb per cu ft*
Water	62.5
Concrete	150.0
Steel	490.0
Gravel	110.0
Riprap	132.0
Saturated Sand	122.0
Saturated Clay	110.0
Saturated Shell	117.0
Saturated Silt	117.0

Design Grade Elevations, tabulated below, are based on the still water level (SWL), plus 2 feet of freeboard and 6 inches of projected settlement.

Station Limits	*Design Grade*
0 + 00 to 120 + 08	El. 14.4
120 + 42 to 127 + 15	El. 14.1
127 + 85 to 152 + 50	El. 14.0
152 + 50 to 158 + 50	Transition from EL. 14.0 to El.18.5, West Side Transition from EL. 14.0 to El.18.0, East Side
158 + 50 to 159 + 70	El.18.5, West Side El.18.0, East Side

I-Walls. In the design of the I-walls, the following load cases were considered:

Case I	Q-Case with water to still water level and a factor of safety, FS = 1.5
Case II	Q-Case with water to still water level plus 2 feet of freeboard (top of wall) and a factor of safety, FS = 1.0
Case III	S-Case with water to still water level and wave load (where applicable) with a factor of safety, FS = 1.2
Case IV	Q-Case with water to still water level and wave load with a factor of safety, FS = 1.25
Case V	Water at low pool level with lateral earth pressure, where applicable

Note: In Foundations Investigation and Design Section of GDM, Para 31, the sheet pile wall design criteria was summarized as follows:

Q Case

F.S. = 1.5.. With water to SWL
F.S. = 1.25.......... With water to SWL and wave load (if applicable)
F.S. = 1.0.................................. With water to SWL + 2 ft freeboard

S Case

F.S. = 1.2.................................... With water to SWL and wave load (if applicable)

Wave loading not applied to design of the canal parallel protection

Deflections

Q case

F.S. = 1.0.................................. With water to SWL + 2 ft freeboard

Bending Moments

Governing Tip Penetration Case

Additionally, if the penetration to head ratio is less than about 3:1, it is increased to 3:1 or to that required by the S case, FS = 1.5, whichever results in the least penetration. The SWL is used to calculate head for the penetration to head ratio.

T-Wall Monoliths In the GDM, T-wall protection is shown as extending from Station 59+00 to Robert E. Lee Blvd. (Station 120+00). However, I-wall was installed instead. There is no T-wall, other than swing gate monoliths, on the canal proper.

Gate Monoliths

Case I	Gate closed, static water pressure to SWL, no wind, impervious sheet pile cutoff, no dynamic wave force (100% forces used)
Case II	Gate closed, static water pressure to SWL, no wind, pervious sheet pile cutoff, no dynamic wave force (100% forces used)
Case III	Gate closed , static water pressure with water level 2 feet above SWL, no wind, impervious sheet pile cutoff, no dynamic wave force (75% forces used
Case IV	Gate closed , static water pressure with water level 2 feet above SWL, no wind, pervious sheet pile cutoff, no dynamic wave force (75% forces used
Case V	Gate open, no wind, truck or train on land side edge of base slab (100% forces used)
Case VI	Gate open, no wind, truck or train on canal side edge of base slab (100% forces used)
Case VII	Gate open, wind from flood side, train on canal side edge of base slab , (75% forces used)
Case VIII	Gate open, wind from flood side, train on land side edge of base slab , (75% forces used)

London Avenue Outfall Canal, Design Memorandum No. 19A, General Design, Supplement No. 1, Fronting Protection, Pumping Station No. 4, Dec 94

Material weights

Item	*lb per cu ft*
Water	62.5
Concrete	150.0
Steel	490.0
Saturated Sand	122.0
Saturated Clay	110.0
Saturated Random Backfill	115.0
Riprap	132.0

Design stresses

Structural steel. The basic stresses for structural steel shall be in accordance with the American Institute of Steel Construction (AISC), *Manual of Steel Construction*, Allowable Stress Design, as modified by EM 1110-1-2101. EM 1110-1-2101 requires that AISC allowable stresses be reduced by 17%, as a basis for design. The structural steel shall be in accordance with ASTM A36.

Welds. The allowable stresses for the design of welds shall be in accordance with the American Welding Society, *Structural Welding Code*, Steel, as modified by EM 1110-2-2101.

Steel sheet piling. The basic stress for steel sheet piling used in the cantilevered I-walls and temporary cofferdam shall be in accordance with EM 1110-2-2504. The steel sheet piling for permanent construction shall be in accordance with ASTM A328. The grade of steel sheet piling used for the temporary cofferdam system shall be as required for the selected cofferdam design. Allowable stresses for the cofferdam shall be increased due to the temporary nature of the structure.

Reinforced concrete. The design of reinforced concrete shall be by strength design methods and criteria established in EM 1110-2-2104.

fc'	3,000 psi
Maximum Flexural Reinforcement Ratio	0.25 x Balance Ratio
Minimum Flexural Reinforcement Ratio	200/fy

Steel H-piling. The design stresses for steel H-piles are in accordance with EM 1110-2-2906. Steel is in accordance with ASTM A36. The allowable stresses for the steel H-piles are as follows:

Axial compression or tension – lower region: 10.0 ksi

Combined bending and axial compression – upper region:

$$Fa/Fa + fbx/Fb + fby/Fb <= 1.0$$

where:

fa = computed axial unit stress
Fa = (0.833) * (0.600) * fy = 18.0 ksi (ASTM A36)
fbx, fby = computed bending unit stress
Fb = (0.833) * (0.600) * fy = 20.0 ksi (ASTM A36; compact)

Loading Conditions

General. The Standard Project Hurricane (SPH) level is El. 11.9 NGVD. For the I-wall, T-wall, and gated monoliths, usual loading conditions include a canal stage at the SPH level. Unusual loading conditions include a canal stage at the top of protection, El. 13.9 NGVD. An extreme loading condition was used only for the 1000 cfs pumps and is discussed below. For all hydraulic conditions, i.e. conditions including hydrostatic loads, two uplift conditions are used to account for the effectiveness of the sheet pile cutoff under the monoliths.

Gated monolith for 1000 cfs pumps. Structural and foundation designs are based on the following load cases:

Usual conditions:

- Gate closed, canal SWL El 11.9 NGVD, SWL inside discharge culverts El 8.0 NGVD, storm wind load, impervious sheet pile cutoff

- Gate closed, canal SWL El 11.9 NGVD, SWL inside discharge culverts El 8.0 NGVD, storm wind load, pervious sheet pile cutoff

Unusual conditions[1]

- Gate closed, canal SWL El 13.9 NGVD, SWL inside discharge culverts El 8.0 NGVD, storm wind load, impervious sheet pile cutoff

- Gate closed, canal SWL El 13.9 NGVD, SWL inside discharge culverts El 8.0 NGVD, storm wind load, pervious sheet pile cutoff

Maintenance dewatering conditions

- Dewatering stop logs installed, canal SWL El 4.0 NGVD, SWL inside discharge culverts El -11.0 NGVD, operating wind load, impervious sheet pile cutoff

- Dewatering stop logs installed, canal SWL El 4.0 NGVD, SWL inside discharge culverts El -11.0 NGVD, operating wind load, pervious sheet pile cutoff

Construction condition No hydrostatic load, no wind load. This case considered the completed structural components prior to watering.

Extreme condition gate closed, canal SWL El 11.9 NGVD, SWL inside discharge culverts El 1.0 NGVD, storm wind load, impervious sheet pile cutoff.

Gated discharge basin for 320 cfs pumps

Usual conditions:

- Gate closed, canal SWL El 11.9 NGVD, SWL inside discharge culverts El 3.57 NGVD, storm wind load, impervious sheet pile cutoff

- Gate closed, canal SWL El 11.9 NGVD, SWL inside discharge culverts El 3.57 NGVD, storm wind load, pervious sheet pile cutoff

Unusual conditions[2]

- Gate closed, canal SWL El 13.9 NGVD, SWL inside discharge culverts El 3.57 NGVD, storm wind load, impervious sheet pile cutoff

- Gate closed, canal SWL El 13.9 NGVD, SWL inside discharge culverts El 3.57 NGVD, storm wind load, pervious sheet pile cutoff

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[1] For the foundation analyses only, the total monolith loads were reduced by 25% and no overstressing in the foundation piles was allowed. This method was used in lieu of no load reduction and 33-1/3% overstress.
[2] For the foundation analyses only, the total monolith loads were reduced by 25% and no overstressing in the foundation piles was allowed. This method was used in lieu of no load reduction and 33-1/3% overstress.

Maintenance dewatering conditions

- Dewatering stop logs installed, canal SWL El 4.0 NGVD, SWL inside discharge culverts El -8.5 NGVD, operating wind load, impervious sheet pile cutoff
- Dewatering stop logs installed, canal SWL El 4.0 NGVD, SWL inside discharge culverts El -8.5 NGVD, operating wind load, pervious sheet pile cutoff

Construction condition No hydrostatic load, no wind load. This case considered the completed structural components prior to watering.

T-Wall monoliths.

Usual conditions:

- Canal SWL El 11.9 NGVD, storm wind load, impervious sheet pile cutoff
- Gate closed, canal SWL El 11.9 NGVD, storm wind load, pervious sheet pile cutoff

Unusual conditions

- Gate closed, canal SWL El 13.9 NGVD, storm wind load, impervious sheet pile cutoff
- Gate closed, canal SWL El 13.9 NGVD, storm wind load, pervious sheet pile cutoff

Construction conditions

- No hydrostatic load, operating wind load from flood side
- No hydrostatic load, no wind load, dead load only

I-Wall monoliths

Usual conditions: Q-Case (soil FS = 1.5) with SWL El 11.9 NGVD

Unusual conditions: Q-Case (soil FS = 1.0) with SWL El 13.9 NGVD

Piling was analyzed as a cantilever with program BEAMS, based on net pressure diagrams produced with program NODWAL. The required tip elevation was based on a soil strength factor safety factor of 1.5, while the required sheet pile section was based on a soil strength factor of 1.0 and a steel strength safety factor of 2.0.

3.2.1.5.3.3.10. IHNC West Levee, France Road Terminal Relocation of IHNC Flood Protection Reference 36

General. Wall types consist of I- and T-walls, the former being limited in unsupported height from 8 to 8.5 ft. and the latter to be used when I-walls are not feasible. Eight steel bottom roller or swing-type floodgates will be provided as necessary for required access, one at the Boh

Brothers Construction lease site, one at the Pontchartrain Materials Corporation lease site, one at the MECO lease site, and one at each of the five existing ramps at the ship berths.

Structural Design

Design Criteria

Basic data

Water Elevations	*Elevations (ft NGVD)*
Net Design Grade	+15.0
Still Water Elevation	+13.0

Grades

Floodwall Gross Grade	*Elevations (ft NGVD)*
I-Wall	+15.5
T-Wall	+15.0
Cofferdam	+16.0

Unit Weights

Water	62.4 pcf
Concrete	150 pcf
Steel	490 pcf
Saturated Soil	115 pcf

Design Loads

Wind Loads	50 psf
Live Loads	AASHTO
	Special Forklift Loads

Design Methods. Design of reinforced concrete is in accordance with the strength design method of the current ACI Building Code, as modified by the guidelines of "Strength Design Criteria for Reinforced Concrete Hydraulic Structures", EM 1110-2-2104 dated June 30, 1992, except prestressed concrete piling for which the minimum is 5,000 psi. Pertinent stresses are tabulated below:

fc’	3,000 psi
fy (Grade 60)	60,000 psi.
Maximum Flexural Reinforcement Ratio	0.375 x balance ratio
Minimum Flexural Reinforcement Ratio	200/fy
fc’(for Prestressed Concrete Piles)	5,000 psi
fy (for Prestressing Strand Grade 250)	250,000 psi
fy (for Prestressing Strand Grade 270)	270,000 psi

Live Loads

Basic Uniform Live Load = 850 pounds per square foot
Truck Loading – HS20-16, latest AASHTO Specifications
Forklift Loading – KALMAN LMV
Crane Load – BUCKNES 88B
Wind Load – 50 psf (for structures within 100 miles of a hurricane shoreline)

I-Type Floodwall. In the design of the I-wall, the following loading cases were considered:

Case I	Water at Still Water Level (Elev +13.0 NGVD), Q case FS = 1.5; S case FS = 1.3
Case II	Water at Net Design Grade (Elev +15.0 NGVD), Q case FS = 1.0; S case FS = 1.0

Minimum penetration to head ratio of 3 to1 used, where the head is at still water level.

T-Type Floodwall. T-walls, including the monolithic base slabs of the floodgate sections, will consist of reinforced concrete walls (columns for gate monoliths) and base slabs supported on 14” square prestressed concrete piles with steel sheetpiling for seepage control. Pile load tests have been performed, with the result that a factor of safety of two (FS = 2) was used for design. An exception is the existing T-wall sections through which the discharge pipelines of the existing pump station pass. In this case, the number of piles is small and the walls were constructed prior to pile load testing, with the result that a factor of safety of three (FS = 3) was used.

Loading Cases. In the design of the T-wall, the following loading cases were considered:

T-Wall

Case I	Wall Dead Load (DL) + Water to Elev 13.0 + Soil Load + Uplift Load (impervious cut-off wall)
Case II	{Wall DL + Water to Elev 15.0 (low probability head)+ Soil Load + Uplift Load (impervious cut-off)} x 0.75
Case III	{Wall DL + Floodside Wind Load @ 50 psf + Soil} x 0.75
Case IV	{Wall DL + Protected side Wind Load @ 50 psf + Soil} x 0.75
Case V	{Wall DL + Soil} x 0.75
Case VI	Case I with pervious cut-off
Case VII	Case II with pervious cut-off

For Load Case VI and Load Case VII above, note the following:

- Soil pressures at rest were used for analyses as the wall movement was to be minimized. (kr = 0.50) Passive pressures were neglected.

- Wind load was based upon AASHTO load as per requirement of U.S. Army Corps of Engineers.

Gate Monoliths

Case I	Gate closed; water to Elev 13.0
Case II	Gate closed; water to Elev 15.0 (75 % forces)
Case III	Gate open; 2 fork lifts or 2 HS20-16 trucks on protected side edge of base slab
Case IV	Gate open; 2 fork lifts or 2 HS20-16 trucks on flood side edge of base slab
Case V	Gate closed; wind from flood side (75% forces)
Case VI	Gate closed; wind from protected side (75% forces)
Case VII	Gate open; no wind, no water

Floodgates. Swing and roller gates will be constructed of structural steel. Floodgates were designed in accordance with EM 1110-2105, "Design of Hydraulic Steel Structures" and EM 1110-2-2705, "Structural Design of Closure Structures for Local Flood Protection Projects". Deflections were limited to 1/400 of the span length. Gate No. 6 was designed to span between the concrete end posts, with the latching eye bolts in place, within the above deflection limitations. It was also designed to span between the two concrete end posts without the latching eye bolts but with no deflection limitation.

3.2.1.5.3.3.11. IHNC West Levee, Florida Avenue to IHNC Lock – Reference 12. The protective works covered herein consist of approximately 2,150 feet of "I"-type cantilever floodwall and 4,900 feet of inverted "T"-type floodwall. Eleven overhead roller gates and three swing gates are provided where the alignment crosses vehicular roads and railroads, and a flap gate is provided a t the loading platform of the Jones & Laughlin Steel Company warehouse

Structural Design

Design Criteria

Basic Data

Water Elevations	*Elevations*
Project flow line (surge elevation from design hurricane)	13.0
Landside of floodwall	0.0
Floodwall grades	*Elevations*
Net grade (one foot freeboard over project flow line)	14.0
Top of wall, I-type wall in levee (as constructed)	15.0
Top of wall, I-type wall in natural ground (as constructed)	14.5
Top of wall, T-type wall in levee (as constructed	14.0
Top of access gates (as constructed)	14.0

Unit Weights

Item	*lb per cu ft*
Water	62.5
Concrete	150.0
Steel	490.0

Design Loads

Water loads

No wave force
One foot freeboard
Design Water Elevations as follows

	Flood side	*Protected side*
I-wall in levee	14.5	0
T-wall in natural ground	14.0	0
T-wall	14.0	0

Wind loads

On walls ....30 psf
On overhead beams..................50 psf

Allowable Working Stresses. The allowable working stresses for concrete and structural steel are in accordance with those recommended in "Working Stresses for Structural Design", EM 1110-1-2101 of 6 January 1958 revised August 1963. The basic minimum 28-day compressive strength for concrete will be 3,000 psi except for prestressed concrete piling which shall be designated 5,000 psi concrete. Steel for steel sheet piling will meet the requirements of ASTM A328-54, "Standard Specification for Steel Sheet Piling". Pertinent allowable stresses are tabulated as follows:

Reinforced Concrete	*Stress (psi)*
fc'	3,000
fc	1,050
v_c (without web reinforcement)	60
v_c (with web reinforcement)	274
fs	20,000
Minimum tensile steel	0.0025 bd
Shrinkage and temperature steel area	0.0020 bt
Structural Steel (ASTM A-36)	
Basic working stress	18,000

I-Wall Design. The penetration for I-walls was determined based on the “S” case and with a factor of safety equal to 1.5. Water was assumed at 6 inches below the top of wall on the flood side and at 0.0 on the protected side. Bending moments and deflections were determined by applying a factor of safety of 1.0 to the soil parameters. LMVD in the 1st Ind of 13 Apr 67 stated that bending moments, stresses and wall deflections for I-walls should be computed using the same earth and water pressure diagrams as those used in determining the pile penetration. However, where the sheet piling is in clay and the “S” case governs, LMVD permitted a 1/3 overstress. Where the sheet piling is in clay and the “Q” case governs, no overstress was permitted by LMVD. In the 2d Ind of 31 May 67, NOD concurred with using the same earth and water pressure diagrams as those used in determining pile penetration for computing bending moments and stresses, but further stated that an overstress should be permitted for either shear strength that governs the design.

T-Wall Monoliths. The T-type floodwalls were designed for the following conditions:

Case I	Water at elevation 14.0 on flood side and water at elevation 0.0 on protected side. Sheet pile cutoff pervious. Uplift varies uniformly from full head on flood side to tailwater on protected side.
Case II	Same as Case I except sheet pile cutoff impervious. Uplift full head on flood side of cutoff and tailwater on protected side.
Case III	Water at elevation 10.0 on flood side and water at elevation 0.0 on protected side. Pervious cutoff. Uplift as in Case I.
Case IV	Same as Case III except sheet pile cutoff impervious. Uplift as in Case II..
Case V	Water at elevation 7.5 on flood side and water at elevation 0.0 on protected side. Pervious cutoff. Uplift as in Case I.
Case VI	Same as Case V except sheet pile cutoff impervious and uplift as in Case II..
Case VII	No water, wind from canal side (75% forces used)

In all cases, the earth pressure was assumed to be balanced.

Three methods of analysis were used to check the pile foundations. They are as follows:

- “Analysis of Pile Foundations with Batter Piles”, by A. Hrennikoff, Transactions, ASCE Vol. 115 (1950). Used for checking all layouts.)

- “Design of Pile Foundations”, by G. Vetter, Transactions, ASCE Vol. 104 (1939). (used for checking the layout with two batter piles.)

- “Culmann’s method for the Design of Pile Foundations” from “Theoretical Soil Mechanics” by K. Terzaghi. (Used for checking the two layouts with one vertical and two batter piles.

These studies indicate that a foundation consisting of two piles battered in opposite directions is the most suitable and economical for the for the T-type walls.

3.2.1.5.3.3.12. IHNC West Levee and East Levee, Florida Avenue Complex, IHNC (Reference 37). Floodwall (T- and I- type) will be located along both the west and east sides of the Inner Harbor Navigation Canal (IHNC) in the vicinity of Florida Avenue. The floodwall along the west side of the I.H.N.C extends from a tie- in with the existing I-wall at wall line Station 99+06.49 to a tie-in with the existing T-wall near the south end of the France Road Terminal at wall line Station 108+31.54. This feature of the project will also include installation of two steel overhead roller gates (at Florida Avenue and Harbor Road); two steel swing gates (at the existing double track railroad and at the future spur track of the New Orleans Dock Board); a dual vertical lift gate structure at the Florida Avenue Canal and modification of the existing canal by installation of a covered concrete box structure and headwall.

Structural Design

Design Criteria

Basic data

Water elevations

	West Side IHNC *WS Elev – Ft. NGVD*		*East Side IHNC* *WS Elev – Ft, NGVD*	
Case	Flood Side	Protected Side	Flood Side	Protected Side
I	14.0	-8.5	14.0	-8.5
II	4.0	-14.5	4.0	-14.5
III	-14.0	-3.0	-14.0	-3.0

Grades

Floodwall Gross Grade	*Elevations (ft NGVD)*
West Side IHNC	
I-Wall	+14.5
T-Wall	+14.0
East Side IHNC	
I-Wall	+16.0 (North) +15.0(South)
T-Wall	+14.0

Unit Weights

Water	62.5 pcf
Concrete	150 pcf
Steel	490 pcf

Design Loads

Wind Loads	50 psf
Water Loads	62.5 pcf

Allowable Working Stresses. The allowable working stresses for concrete and structural steel are in accordance with those recommended in “Working Stresses for Structural Design”, EM 1110-1-2101 dated 1 November 1963 and Amendment No. 2 dated 17 January 1972. The basic minimum 28-day compressive strength for concrete will be 3,000 psi, except for prestressed concrete piling where the minimum strength will be 5,000 psi. Steel for steel sheet piling will meet the requirements of ASTM A328-75a, “Standard Specification for Steel Sheet Piling”. Pertinent allowable stresses are tabulated below:

Reinforced Concrete

fc’	3,000 psi
fc’ (Florida Avenue Canal gates and conduits)	4,000 psi
fc	1,050 psi
fc (Florida Avenue Canal gates and conduits)	1,400 psi
Minimum area steel	0.0025 bd
Shrinkage and temperature steel	0.0020 bt
fc’(for Prestressed Concrete Piles)	5,000 psi
fy (for Prestressing Strand Grade 250)	250,000 psi
fy (for Prestressing Strand Grade 270)	270,000 psi

Structural Steel (ASTM A-36)

Basic working stress ....18,000 psi

I-Type Floodwall In the design of the I-wall, one loading cases was considered:

Case I Static water at top of wall, no wind, no dynamic wave force

Depth of penetration was determined by applying a factor of safety of 1.5 to the “S” case soil shear strengths.

T-Type Floodwall In the design of the T-wall, two loading cases were considered as follows:

Case I	Water at the top of the wall floodside, water at the top of the base slab monolith protected side, no wind, no dynamic wave force, impervious sheet pile cutoff
Case II	Water at the top of the wall floodside, water at the top of the base slab monolith protected side, no wind, no dynamic wave force, pervious sheet pile cutoff

Gates and Gate Monoliths

Swing Gates

Load Cases

Case I	Gate closed, no wind, ballast saturated
Case II	Gate closed, water at the top of the wall floodside, water at the top of the base slab protected side, no wind, no dynamic wave force, impervious sheet pile cutoff
Case III	Gate closed, water at the top of the wall floodside, water at the top of the base slab protected side, no wind, no dynamic wave force, pervious sheet pile cutoff
Case IV	Gate opened, ballast saturated, no wind, train on edge of slab on floodside
Case V	Gate opened, ballast saturated, no wind, train on edge of slab on protected side

Overhead roller gates

Load Cases

Below elevation of top of wall

Case I	Gate closed, water at the top of the wall floodside, water at the top of the base slab protected side, no wind, no dynamic wave force, impervious sheet pile cutoff
Case II	Gate closed, water at the top of the wall floodside, water at the top of the base slab protected side, no wind, no dynamic wave force, pervious sheet pile cutoff
Case III	Gate opened, ballast saturated, no wind, truck on edge of slab on floodside
Case IV	Gate opened, ballast saturated, no wind, truck on edge of slab on protected side
Case V	Gate opened, no water, wind from protected side, 33-1/3 percent increase in allowable stresses
Case VI	Gate opened, no water, wind from flood side, 33-1/3 percent increase in allowable stresses

Superstructure above top of wall

Case I	Gate open, no water, no wind
Case II	Gate closed, no wind
Case III	Gate open, wind from right, 33-1/3 percent increase in allowable stresses
Case IV	Gate closed, wind from right, 33-1/3 percent increase in allowable stresses
Case V	Gate closed, wind from left, 33-1/3 percent increase in allowable stresses
Case VI	Gate open, no wind, hangar loads centered on middle column
Case VII	Gate open, no wind, one hangar load near center of span, one hangar load 1 foot (plus or minus) from end column

Vertical Lift Gates (Sluice Gates)

Case I	Construction case, no backfill, gates raised, no water
Case II	Water level at El 14 flood side, at El -8.5 protected side, impervious cutoff
Case III	Water level at El 14 flood side, at El -8.5 protected side, pervious cutoff
Case IV	Water level at El 4 flood side, at El -14.5 protected side, impervious cutoff
Case V	Water level at El 4 flood side, at El -14.5 protected side, pervious cutoff
Case VI	Water level at El -14 flood side, at El -3 protected side, impervious cutoff
Case VII	Water level at El -14 flood side, at El -3 protected side, pervious cutoff

Concrete Box Structure in Florida Avenue Canal

West Side of IHNC

Load Cases

Case I	Dry inside, water at El -4.5 outside, full surcharge
Case II	Water inside, water at El -4.5 outside, full surcharge
Case III	Dry inside, water at El 14 outside, full surcharge
Case IV	Water inside, water at El 14 outside, full surcharge

East Side of IHNC

Load Cases

Case I	Water at El -14, full surcharge
Case II	Water at El +14, full surcharge
Case III	Dry inside, water at top of wall outside, full surcharge

3.2.1.5.3.3.13. Orleans Parish Remaining Work – Reference 14. The project consists of raising the remaining flood protection along the south shore of Lake Pontchartrain in Orleans Parish, Louisiana to the design hurricane project elevation as approved in Design Memorandums Nos. 13 and 14. With the exception of the Lake Pontchartrain Beach Floodwall, the reaches shown below required improvements to the existing protection

Floodwall	Existing Top Elevation	Proposed Top Elevation
Marina Floodwall	10.9	13.5 to 14.0
New Basin Canal Sluice Gate	N/A	13.5
Bayou St. John Earthen Closure	N/A	18.0
Pontchartrain Beach Floodwall & Levee	Varies	Varies
Lakefront Airport Floodwall	Varies	13.5
Lincoln Beach Floodwall	Varies	13.5

Structural Improvements. The types of structural improvements include: extending the tops of existing floodwalls, constructing new concrete capped portion of I-walls, removing T-type wall stem portions and replacing with a new higher level stem, or providing metal flip gates atop existing I-walls to achieve a higher flood profile in areas where restrictions will not allow fixed structures, such as within an airport runway's flight path. At the Marina Floodwall, three existing swing gates will be extended from Elevation 10.4 t o Elevation 13. 5 and at New Basin Canal Floodgate, a 4-gated sluice gate structure will be provided.

Bayou St. John Closure Structure - To provide ingress and egress of water within Bayou St. John, a sluice and sector gated structure will be provided between Station 200+06.50 W/L and Station 201+60.00 W/L with the centerline of the sector gated structure at Station 201+00.00 W/L.

Constructed by the non-Federal local sponsor, the Bayou St. John closure structure is a pile-founded sector gate with adjacent pile-founded sluice gates, pile-founded T-wall, and capped cantilevered I-wall. Daily tidal flow on Bayou St. John is maintained through the sluice gates. Navigation on Bayou St. John, an old portage route, is possible through the sector gate.

Structural Design Criteria The design criteria for pertinent portions of the above structures is as follows:

Water elevations	**Elevation (ft NGVD)**
Lake Pontchartrain Wind Tide Level Elevation 11.5 plus 2 f t freeboard	13.5
Landside of Floodwall	0.0

I-Walls. The I-type wall will consist of steel sheetpiling driven into the existing ground. The upper portion of the sheetpiling will be capped with concrete. The sheetpiling will be driven to the required depth with 1 foot of the sheet piling extending above the finished ground elevation. The concrete portion of the floodwall will extend from 2 feet below the finished ground elevation to the required protection height. For the I-type wall requiring extensions from an existing elevation to a higher elevation, the sheetpiling will not be disturbed.

Loading cases. In the design of the I-walls, two loading cases were considered:

- Case I –
  - For confined area, the factor of safety (FS) used = 1.5 with static water at the top of the wall (still water level (SWL) plus freeboard)) and no dynamic wave force.
  - For unconfined areas along the lakefront adjacent to open water, such as Bayou St. John, the FS used = 1.5 with static water at the SWL (and no dynamic wave force) and FS= 1. 25 with static water a t the SWL and a dynamic wave force
- Case II - No water, lateral soil pressure (where applicable)

T-Walls. T-walls consist of a reinforced concrete stem on a monolithic concrete base supported on either precast prestressed concrete piling or steel H-piles. The T-wall extensions will consist of modifying the stem of the T wall, with either removing a portion of the existing T-wall stem and replacing it with a new stem or attaching a metal extension on top of the stem. The T- walls will be designed for the following load conditions

- Case I - Static water pressure, no wind, impervious sheetpile cutoff, no dynamic wave force.
- Case II - Static water pressure, no wind, pervious sheetpile cutoff, no dynamic wave force.
- Case III - Still water pressure t o Elevation 11.5, no wind, impervious sheetpile cutoff, dynamic wave force (75 % forces used)
- Case IV - Still water pressure t o Elevation 11.5, no wind, pervious sheetpile cutoff, dynamic wave force (75% forces used).
- Case V - No water, no wind.
- Case VI - No water, wind from protected side (75 % forces used.
- Case VII - No water, wind £ran flood side (75% forces used).

Gates and Gate Monoliths. Gate monoliths and two swing gates will be constructed on the lakeside end of the Marina Drive ramp. The gates will be designed and or analyzed for the following load conditions:

- Case I - Gate closed, still water pressure to elevation 11.5, no wind, impervious sheetpile cutoff, dynamic wave force (75% forces used)
- Case II - Gate closed, still water pressure to Elevation 11.5, no wind, pervious sheetpile cutoff, dynamic wave force (75% forces used)
- Case III - Gate open, truck on protected side of base slab, no wind
- Case IV - Gate open, truck on flood side of base slab, no wind.
- Case V - Gate open, truck on protected side of base slab, wind from protected side (75% forces used).
- Case VI - Gate open, truck on protected side of base slab, wind from floodside (75% forces used).

3.2.1.5.3.3.14. Lake Pontchartrain and Vicinity, Floodproofed Bridge Design Criteria Strength Design Method

General. This section addresses the general criteria used to floodproof the bridges which cross the 17th Street, London, and Orleans Canals accomplished as part of the parallel protection plan for these canals. The bridge structures were evaluated under loads imposed as a hydraulic structure. Both precast prestressed concrete slab type girders and cast in place reinforced concrete slab spans have been used in the construction of floodproofed bridges.

Design Approach. In brief summary: the bridge barrier wall is designed as a floodwall, the edge girder/slab is designed to resist the torsion applied by the barrier wall as it functions as a floodwall, the girder/slab is designed for uplift pressures, and the girder/slab connection to the pile bents is designed for tension forces. The following is a brief synopsis of the specific criteria based on EM 1110-2-2104:

Cover.

Hydraulic Structure. The bottom face of the bridge girder/slab, the outer face of the barrier/flood wall and the pile bents, where hydraulic loading is applied, follow the more conservative requirements of the COE criteria for hydraulic structures. EM 1110-2-2104, states that concrete sections with a thickness greater than 12 inches but less than 24 inches have a clear cover of 3 inches. Concrete sections with a thickness equal to or greater than 24 inches should have a clear cover of 4 inches. However, in special circumstances, a clear cover of 3 inches for concrete sections equal to 24 inches has been allowed.

Highway Bridge. AASHTO criteria should control the bridge deck, the top face of the bridge girder/slab and the inner face of the barrier/flood wall where highway loading is applied.

Load Factors.

Hydraulic Loading. The portion of the bridge that will be submerged is considered a hydraulic structure, and therefore, designed in accordance with the requirements of EM 1110-2-2104.

In accordance with the COE strength design method, the service loads are multiplied by their appropriate load factor. Typically, a single load factor approach is used for both the dead load and live load.

$$U = 1.7 (D + L)$$

For hydraulic structures, the factored loads are then multiplied by an additional hydraulic factor, $\mathbf{H_f = 1.3}$.

$$U_h = 1.3 [1.7 (D + L)]$$

For short duration loads, during construction of hydraulic structures, the load factors may be reduced by 14% (equivalent to 16 2/3 % increase in allowable stress, or 1 / 1.1667 = 0.86).

$U_h = 0.86 [H_f (U)]$

For long duration loads, during construction of hydraulic structures, no reduction in load factors is allowed.

For resistance to the effects of wind or other forces of short duration, with low probability of occurrence and for unusual or extreme hydraulic conditions, such as water to the top of wall (water levels above the Standard Project Hurricane or still water levels), the load factors may be reduced by 25% (equivalent to a 1/3 increase in allowable stress or 1 / 1.3333 = 0.75).

$U_h = 0.75 [H_f (U)]$

Highway Loading. The portion of the bridge that will not become submerged is considered a bridge structure, and therefore, designed in accordance with the load factors prescribed by AASHTO.

The hydraulic factor should is not combined with the AASHTO load factors.

Future wearing surface and highway loads are not be used to reduce the effects of hydraulic loading.

Prestressing strands are not be used as tension connectors.

3.2.1.5.3.4. Sources of Construction Materials -

3.2.1.5.3.4.1. Sheet Pile. Generally, the sheet pile sections specified during advertisement were used for construction. However, sheet pile section substitutions conforming to the minimum required section modulus was allowed, primarily in contracts constructed after 1990. Below, is a table of sheet pile sections for Orleans East Bank, broken down by DM.

Orleans East Bank	
17th Street Canal	
PS#6 to Hammond Hwy	Hoesch 12
Orleans Marina	
17th Street Canal to	PZ-38, PMA-22, PZ-35
Lakeshore Drive	
Lake Marina Dr.	PZ-27
Orleans Outfall Canal	
West Side	
I-610 to French St	Syro SPZ-22
Harrison Ave Bridge Tie-Ins	Casteel CZ-114
Filmore Ave Bridge Tie-Ins	Casteel CZ-114
Robert E Lee Bridge Tie-Ins	Casteel CZ-101, CZ-114

East Side	
I-610 to Robert E Lee	PZ-22
Harrison Ave Bridge Tie-Ins	Casteel CZ-114
Filmore Ave Bridge Tie-Ins	Casteel CZ-114
Robert E Lee Bridge Tie-Ins	Casteel CZ-101, CZ-114
London Ave. Outfall Canal	
PS#3 to Mirabeau, Both Sides	Syro SPZ-22
Mirabeau to Robert E. Lee, West Side	Casteel CZ-101
Mirabeau to Filmore, East Side	Casteel CZ-101
Filmore to PS#4, East Side	Arbed AZ-18
PS#4 to Leon C. Simon, East Side	Casteel CZ-101
Lakefront Levee	
Topaz Street Swing Gate Tie-In	PZ-35
Marconi Dr. Swing Gate Tie-In	PZ-27
Rail St.	**
Bayou St. John Sector Gate East Side	Arbed BU-32
Bayou St. John Sector Gate West Side	PZ-40
American Standard (Franklin Ave)	PZ-27
Leroy Johnson Dr. Swing Gate Tie-In	PZ-27
Pontchartrain Beach Floodwall	PZ-22*
IHNC	
East Side	
North of Fla. Ave. to Chalmette Back Levee	PZ-27
North of US 90	PZ-27
IHNC to Florida Ave	PZ-27
Hayne Blvd to Dwyer Rd	PZ-27
Dwyer Rd to Hwy 90	MA-22*, Z-27, PZ-32*, M27*
West Side	
IHNC to Florida Ave	PZ-27
France Rd to Florida Ave	PZ-27*
North of US 90	PZ-27*
Hayne Blvd to Hwy 90	PZ-27
Hwy 90 to Almonaster Blvd.	PZ-27, MA-22, SA-23
Almonaster Blvd to Florida Ave	PZ-27
* As-advertised – Not confirmed as-built ** Information not located at the time of publication	

3.2.1.5.3.4.2. Levee material

3.2.1.5.3.4.2.1. Levee Materials (17th Street Canal). No mention made in DM of Materials to be used for levee construction.

3.2.1.5.3.4.2.2. Construction Materials. No mention was made in the DM of the materials to be used for levee construction.

3.2.1.5.3.4.2.3. Levee Construction Materials. GDM 19A stated that the levee fill material was to consist of clay, and was to be hauled in by dump trucks from the Bonnet Carré Spillway. Because of the high percentage of fines in the existing levee, it was assumed that only 50 percent of the existing levee material could be reused in the construction of the new realigned levee.

3.2.1.5.3.4.2.4. Levee Material (Pontchartrain Beach Floodwall/Levee). Reference Nos. 7 and 8 make no mention of source of borrow.

3.2.1.5.3.4.2.5. Levee Material (IHNC Remaining West Levee, France Road and Florida Avenue Complex). Borrow for levee construction was to be hauled from the Bonnet Carré Spillway.

3.2.1.5.3.4.2.6. Levee Materials (IHNC Remaining Levees). The earth fill for completing the road ramps and levee portion of the protection was to be obtained from excess material cut from some of the reshaped existing levees and from a borrow area in the bottom and along the north shore of Lake Pontchartrain. The borrow material from the lake area consisted primarily of stiff Pleistocene clays and was to be transported to the project on barges.

During a subsequent study based on a Division review comment, it was disclosed that the only sources of suitable material were the Mississippi River batture, the Bonnet Carré Spillway, and the bottom of Lake Pontchartrain. Comparable cost estimates revealed that the Lake Pontchartrain source would be the most economical if the quantities of borrow to be hauled were large. The studies also revealed that if the quantities to be hauled were relatively small, as is the case for this project, the Bonnet Carré Spillway would be the most advantageous source, and consequently the Bonnet Carré was recommended as the borrow source.

3.2.1.5.3.4.2.7. Levee Materials (Reference No. 11). The levee which supports the I-wall along the Florida Avenue drainage canal was to be constructed by reshaping the existing levee and berms whenever possible. All sections of I-wall levee with insufficient material for reworking, and other levee sections where raising was required, were to be completed with haul fill. Where earth filling was required, the fill was to be placed using semi-compacted methods in advance of installation of the steel sheet piling and wall construction to reduce the ultimate settlement of the walls. Since the required amount of haul fill was small, the Bonnet Carré Spillway borrow source was to be used.

3.2.1.5.3.4.2.8. Levee Materials (Reference No. 12). After re-shaping the existing fill along the leveed portion of the project, additional fill consisting of stiff Pleistocene clays for completing levees to design grade and section was to be obtained from a borrow area in the bottom of Lake Pontchartrain along its north shore and barged to the construction site, inasmuch as satisfactory borrow was not available in the immediate vicinity of the project. The fill was to be placed using semi-compacted methods well in advance of installation of the steel sheet piling and wall construction to reduce the ultimate settlement of the wall.

3.2.1.5.3.4.2.9. Levee Fill (Orleans Parish Lakefront Levee West of IHNC). The levee fill and structural backfill was to be hauled clay from a borrow area of Pleistocene clays located in Lake Pontchartrain near Howze Beach along the north shore. The material was to be transported to the project on barges.

3.2.1.5.4. As-built Conditions

3.2.1.5.4.1. Changes between design and construction (i.e. cross sections, alignment, sheet pile tip el, levee crest el.)

3.2.1.5.4.1.1. The Board of Commissioners of the Orleans Levee District, Contract 2043-0489, Excavation and Flood Protection - 17th Street Canal, Phase IB, Hammond Hwy. To Southern Railway.

Reviewed As Builts, No Modifications or Changes Found.

3.2.1.5.4.1.2. No Bid or Contract No. Lake Pontchartrain, Louisiana and Vicinity, Lake Pontchartrain Barrier Plan, Inner Harbor Navigation Canal, West Levee, Florida Avenue to IHNC Lock Floodwall.

Reviewed As Builts, No Modifications or Changes Found

3.2.1.5.4.1.3. DACW29-68-B-0141. Lake Pontchartrain, Louisiana and Vicinity, Lake Pontchartrain Barrier Plan, Orleans Parish, LA., Inner Harbor Navigation Canal, West Levee, Hayne Blvd. To U.S. Hwy 90 (Station 30 + 00 to Station 105 + 66), Almonaster Ave. to Florida Ave. (Station 144 +43 to Station 206 + 47) Plans for Levee and Floodwall Capping.

Reviewed As Builts, No Applicable Modifications or Changes Found.

3.2.1.5.4.1.4. DACW29-70-B-0126. Lake Pontchartrain, Louisiana and Vicinity, Lake Pontchartrain Barrier Plan, Orleans Parish, LA., Inner Harbor Navigation Canal, West Levee, U.S. Hwy 90 to Almonaster Ave. (Station 105 + 66 to Station 167 + 00) Plans for Levee and Floodwall.

Reviewed As Builts, No Applicable Modifications or Changes Found.

3.2.1.5.4.1.5. DACW29-82-B-0033. Lake Pontchartrain, Louisiana and Vicinity, Hurricane Protection, Orleans Parish, LA., Floodwall and Levee, I. H. N. C., North of Florida Ave.

Reviewed As Builts, No Applicable Modifications or Changes Found

3.2.1.5.4.1.6. DACW29-93-C-0071. Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, New Orleans Lakefront Levee West of Inner Harbor Navigation Canal, Orleans Avenue Canal Floodwall, West Side Phase II-B, Station 64+51.53 B/L to Station 90+26.91 B/L, Orleans Levee District, Orleans Parish, Louisiana

Reviewed Completion Report, No Applicable Modifications or Changes Found

3.2.1.5.4.1.7. DACW29-93-C-0077. Lake Pontchartrain, Louisiana & Vicinity, High Level Plan, New Orleans Lakefront Levee, West of IHNC, Orleans Avenue Canal Flood Protection Improvement, Phase II-D, New Orleans, LA

Reviewed Completion Report, No Applicable Modifications or Changes Found

3.2.1.5.4.1.8. DACW29-93-C-0071. Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, New Orleans Lakefront Levee West of Inner Harbor Navigation Canal, Orleans Avenue Canal Floodwall, West Side Phase II-B, Station 64+51.53 B/L to Station 90+26.91 B/L, Orleans Levee District, Orleans Parish, Louisiana

Reviewed Completion Report, No Applicable Modifications or Changes Found

3.2.1.5.4.1.9. DACW29-93-C-0077. Lake Pontchartrain, Louisiana & Vicinity, High Level Plan, New Orleans Lakefront Levee, West of IHNC, Orleans Avenue Canal Flood Protection Improvement, Phase II-D, New Orleans, LA

Reviewed Completion Report, No Applicable Modifications or Changes Found

3.2.1.5.4.1.10. DACW29-93-C-0081. Lake Pontchartrain and Vicinity, Hurricane Protection Improvements, 17th Street Canal, East Side Floodwall Capping, Orleans Parish, Louisiana

Reviewed Completed Report, Found the Following Modifications and Changes:

Eleven monoliths placed out of the tolerance.

This contract was for construction of East Bank floodwalls along the 17th Street Canal. There was a claim and dispute on the contract centering around the out of tolerance monoliths that were placed. It was resolved through an Alternative Disputes Resolution (ADR) process, and the contractor lost on all counts that he had based his claim on. He was seeking $809,659 and 80 days time based on defective specifications, superior knowledge of the Government, commercial impracticability, alleged differing site conditions, contract interpretation, and the Government’s failure to cooperate.

Between Veterans Boulevard and I-10 - Monolith numbers 18, 20, 22, and 24. W/L Stations 81+74.95 to 89+20.75.

Between Hammond Highway and W. Harrison Avenue - Monolith numbers 3, 4, 5, 7, 8, 9, and 16. W/L Stations 9+25.8 to 79+87.69.

Mod 0005 - Because of hard pile driving between W/L Stations 0+96-27 and 7+00, they ended up short of the designated plan tip elevation and by cutting the 4’-3” off the sheet pile, 9.25’ was the top of pile elevation. Contract Correspondence

3.2.1.5.4.1.11. DACW29-94-C-0003. Lake Pontchartrain, Louisiana & Vicinity, High Level Plan, London Avenue Outfall Canal, Parallel Protection, Pumping Station No. 3 to Mirabeau Avenue Floodwall, Orleans Parish, Louisiana

Reviewed Completion Report, No Applicable Modifications or Changes Found

3.2.1.5.4.1.12. DACW29-95-C-0093. Lake Pontchartrain and Vicinity, Floodproofing Veterans Boulevard Bridges over 17th Street Canal, Orleans and Jefferson Parishes, Louisiana, Modification A00022, CIN 20

Reviewed As Builts, No Applicable Modifications or Changes Found

3.2.1.5.4.1.13. DACW29-98-C-0022. Lake Pontchartrain, LA and Vicinity, New Orleans Lakefront Levee, Orleans Parish Lakefront - East and West of IHNC, Miscellaneous Floodwall Capping, Lake Marina Avenue to Collins Pipeline, Orleans Parish, Louisiana

Reviewed As Builts, No Applicable Modifications or Changes Found

3.2.1.5.4.1.14. DACW29-98-C-0043. Southeast Louisiana Urban Flood Control Project, Keyhole Canal, 4th Street to LaPalco Blvd.

Reviewed As Builts, No Applicable Modifications or Changes Found

3.2.1.5.4.1.15. DACW29-98-C-0082. Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, London Avenue Outfall Canal, Parallel Protection, Floodproofing at Leon C. Simon Boulevard Bridge, Orleans Parish, LA

Reviewed Contract/Modification Documents, No Major Modifications or Changes Found

3.2.1.5.4.1.16. DACW29-99-C-0005. Lake Pontchartrain, Louisiana and Vicinity, London Avenue Outfall Canal, Parallel Protection, Floodproofing of Gentilly Boulevard Bridge, Orleans Parish, Louisiana

Reviewed Completion Report & As Builts, No Major Modifications or Changes

3.2.1.5.4.1.17. DACW29-99-C-0012. Lake Pontchartrain, Louisiana and Vicinity, Hurricane Protection, High Level Plan, Orleans Parish, London Avenue Outfall Canal, Parallel Protection Fronting Protection of Pumping Station No. 4, Orleans Parish, Louisiana

No Modifications or Contract Document Folder Found

3.2.1.5.4.1.18. DACW29-99-C-0018. Lake Pontchartrain, Louisiana and Vicinity, Hurricane Protection, High Level Plan, Fronting Protection at Pumping Station No. 6, Orleans Parish - Jefferson Parish, 17th Street Outfall Canal (Metairie Relief)

Reviewed Contract/Modification Documents, No Applicable Modifications or Changes Found

3.2.1.5.4.1.19. DACW29-99-C-0025. Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Orleans Avenue Outfall Phase I-C, Filmore and Harrison Avenue Bridges, Orleans Parish, LA

Reviewed Contract/Modification Documents, no Applicable Modifications or Changes Found

3.2.1.5.4.1.20. DACW29-00-C-0073. Lake Pontchartrain and Vicinity, High Level Plan, Orleans Outfall Canal, Phase 1-B, Robert E. Lee Boulevard Bridge, Orleans Parish, Louisiana

Reviewed As Builts & Contract Documents, No Applicable Modifications or Changes Found

3.2.1.5.4.1.21. DACW29-02-C-0013. Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, London Avenue, Outfall canal, Parallel Floodproofing Protection of Mirabeau and Filmore Ave. Bridges, Orleans Parish, Louisiana

Reviewed Contract/Modification Documents, no Applicable Modifications or Changes Found

3.2.1.5.4.1.22. DACQ29-02-C-0016. Lake Pontchartrain, Louisiana and Vicinity, Hurricane Protection Project, High Level Plan, 17th Street Outfall Canal, Hammond Highway Complex, Orleans and Jefferson Parishes, Louisiana

Reviewed Contract/Modification Documents, Found that during dewatering of steel sheet pile cofferdam, excessive settlement of the cofferdam occurred on one side. Reason: Borings did not identify a layer of extremely soft soils in the area. A modification required rewatering and changed the requirements for the bottom of the cofferdam from tremie concrete to a layer of bedding with a four-inch stabilization slab. It also required beefing up the sheet pile cofferdam bracing.

3.2.1.5.4.2. Inspection during original construction, QA/QC, state what records are available. On construction contracts, the Government Quality Assurance (QA) Reports and Contractor Quality Control (QC) Reports are normally filed and stored together. QC reports normally follow a Government suggested format; therefore, they usually cover the same items. Those items are general information about the weather conditions for that day, the numbers of laborers and supervisors on the job, hours worked, and the operating equipment that is on the job. There is a statement as to what work was performed that day. There are paragraphs to cover the results of the controlled activities, such as preparatory, initial, and follow-up meetings and inspections; and for tests performed that day, as required in the plans and specifications. There are paragraphs for materials received, submittals reviewed, off-site surveillance activities, job safety, environmental protection, and a general remarks paragraph.

A lot of the same information is covered in the QA Reports. The items/sections listed on the QA report usually are as follows: general information about the weather conditions for that day, the number of contractor and government employees on the job, the prime contractor and the subcontractors on the job and their responsibilities, and description of the work performed that day. There are sections for days of no-work and reasons for the no-work, and progress of the work. There is information on CQC inspection phases attended, instructions given, and results of QA inspections and tests, deficiencies observed and actions taken, and corrective action of contractor. There are sections for verbal instructions given the contractor that day, for controversial matters that may have arisen, for information, instructions, or actions taken not covered in QC reports or disagreements, safety, and a section for remarks.

QA and QC reports are available on the following contracts unless stated otherwise. Therefore, only the information that is attached to the QA or QC Reports on each contract is noted, based on a cursory review of the records.

Once the contract is completed and release of claims is granted by the Contractor, the records of the project are boxed up and sent to off-site storage where they remain for six years. After six years, they are destroyed.

3.2.1.5.4.2.1. DACW29-93-C-0071 – NO LKFNT LEV. ORL AVE CANL, PHS II-B, ORL PAR

The contractor has attached the form checkout sheets for reinforced concrete that also contains concrete test data. These sheets are checklists for inspecting the forms before placing concrete. Pile driving reports are attached and also minutes of preparatory meetings.

3.2.1.5.4.2.2. DACW29-93-C-0081 – 17TH ST OUTFAL CNL CAPPING OF FLDWL, ORL PAR

The contractor has attached the form checkout sheets for reinforced concrete floodwalls, sheets documenting preparatory, initial, or follow-up inspections, and percent of the job completed.

3.2.1.5.4.2.3. DACW29-95-C-0093 – FLOODPROOFING VETS BRIDGE – 17TH ST, JEF & ORL PAR

The contractor attached form checkout sheets for concrete structures, which includes site testing data for the concrete. Pile driving records, in-place density tests, minutes of preparatory inspection meetings, and daily de-watering reports are also included.

3.2.1.5.4.2.4. DACW29-96-C-0080 – ORLEANS MARINA FLDWALL – PHASE IV, ORL PAR

Attached to the reports are concrete curing records, form checkout sheets, records of preparatory inspections/meetings, and pile driving records.

3.2.1.5.4.2.5. DACW29-97-C-0066 – L PONT PONT BEACH, STA 10+03 – 39-78, ORL PAR

Attached are soil tests and preparatory inspection reports.

3.2.1.5.4.2.6. DACW29-97-C-0029 – ORLEANS AVE PHASE II-AFLOODWALL, ORL PAR

The form checkout sheets for concrete structures, on-site concrete tests, and mix design data, reports on concrete compression tests, and in-place density tests are attached.

3.2.1.5.4.2.7. DACW29-98-C-0022 – L PONT, NO E., FLDWL CAP, MARINA-COLLIN, ORL PAR

The contractor has attached such documents as the seed certifications, soil classification, and in-place density tests, batch plant certification of concrete mix design, concrete testing reports, and also Form checkout sheets for structures.

3.2.1.5.4.2.8. DACW29-98-C-0050 – ORLEANS MARINA, PH V, SLUICE GATE, ORL PAR

Attached are piezometer readings, and monolith reference readings.

3.2.1.5.4.2.9. DACW29-98-C-0082 – L PONT LON CNL, FLDPROOF LEON C. SIMON, ORL PAR

Form checkout sheets for concrete structures, pile driving records, minutes of preparatory and initial meetings, girder prestress data, concrete testing data, and in-place density tests are attached.

3.2.1.5.4.2.10. DACW29-99-C-0005 – L PONT LON CNL, F/PROOF GENT BRDG, ORL PAR

Attached to the contractor’s quality control reports are such items as preparatory inspection meetings, crane tests and certification, vibration monitoring reports, resteel tests, pile driving reports, dive reports, in-place density tests, and concrete mix design reports.

3.2.1.5.4.2.11. DACW29-99-C-0012 - LONDON AVE O/FALL, CNL, PS 4, ORL PAR

Load tests on the cranes and dewatering daily reports are attached.

3.2.1.5.4.2.12. DACW29-99-C-0025 – FILMORE AND HARRISON BRIDGES, ORL PAR

Pile driving records are attached to the reports.

3.2.1.5.4.2.13. DACW29-99-C-0046 – L PONT BREAKWATERS PS 2 & 3, ORL PAR

Contractor pile driving logs and dredging records are attached to the reports.

3.2.1.5.4.2.14. DACW29-00-C-0073 – L PONT ORL O/FALL, PH. 1B, (FLDPRF R.E.LEE BRDG), ORL PAR

The in-place density tests and documentation of preparatory or initial phase inspections are attached.

3.2.1.5.4.2.15. DACW29-02-C-0013 – L PONT, LONDON CNT 5 (MIRABEAU/FILMORE), ORL PAR

Attached are percent complete reports, pile driving records, concrete test specimen data, concrete field data, batch certifications, and records of preparatory meetings/inspections.

3.2.1.5.4.2.16. DACW29-98-C-0003 – L PONT L/S R/O RCH 2, STA 167-209, JEF PAR

QA/QC Reports available, nothing attached.

3.2.1.5.5. Inspection and maintenance of original construction. The inspection of hurricane protection features of the East Bank polder fall under the following categories:

3.2.1.5.5.1. Annual Compliance inspection. Annual Compliance Inspections for the East Bank polder were conducted by the U.S. Army Corps of Engineers, New Orleans District, Operations Division in conjunction with the Orleans Levee District. This district is responsible for maintaining 98.7 miles of protection works along the shore of Lake Pontchartrain and canals. The rating for these protection works was “Outstanding” through 2001, at which time the condition ratings system changed. The ratings from that time on were “Acceptable”, but corresponded to the “Outstanding” rating under the previous rating system.

3.2.1.5.5.2. Periodic inspections. The Orleans East Bank polder contains no structures which are inspected under the Periodic Inspection Program at this time.

3.2.1.5.6. Other Features

3.2.1.5.6.1. Brief Description

The primary components of the hurricane protection system for the Orleans East Bank basin are described above, namely the levees and floodwalls designed and constructed by the Corps of Engineers. However, other drainage and flood control features that work in concert with the Corps of Engineers levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section will describe and present the criteria and pre-Katrina conditions of the interior drainage system, pump stations, and the Mississippi River Flood Protection System. There are currently no non-Corps levees or floodwalls in this basin. Even though the stormwater pump stations are part of the interior drainage system, they are a significant part of the system and warrant their own section.

3.2.1.5.6.2. Pre-Katrina Conditions -

According to the local jurisdictions responsible for interior drainage, the storm drain system, interior canals, interior pump (lift) stations, outfall pump stations, and outfall canals were in good condition and prepared for high inflows from rainfall prior to August 29, 2005, Katrina landfall.

The Mississippi River Flood Protection System was in good condition prior to Katrina landfall.

3.2.1.5.6.3. Interior drainage system

Overview. The Orleans East Bank basin contains about 40 square miles and generally slopes south to north from the Mississippi River to Lake Pontchartrain. It is fully developed except for the 2.5 square mile City Park. The initial settlement of New Orleans began on the banks of the Mississippi River and progressed northward to the lake. Many features are typical of large urban cities in the United States, and some features that are unique because much of the area is below sea level. Catch basins and inlets collect surface runoff from yards and streets into storm sewers. Excess runoff flows down streets and/or overland to lower areas. Enclosed and open canals collect the stormwater and carry it to outfall pump stations that pump directly into outfall canals or the Inner Harbor Navigation Canal. The outfall canals flow into Lake Pontchartrain. No stormwater is pumped into the Mississippi River. When it is not raining, dry weather flow from the entire basin can be pumped into the Mississippi River.

Flood water can overflow into Jefferson East Bank when flooding reaches a certain elevation. The adjacent area impacted is referred to as old Metairie or Hoey’s basin.

The entity responsible for local drainage in the Orleans East Bank basin is the Sewerage and Water Board of New Orleans. In addition to local drainage, they also provide potable water and sanitary sewerage service. The Louisiana Department of Transportation and Development highways are also a part of the local drainage system.

System Components. Local drainage begins with overland flow which follows the ground topography. Figure 5 in Volume VI shows the topographic layout of Orleans East Bank. The land generally falls from the Mississippi River to Lake Pontchartrain with an elevation difference of about 20-25 feet. A land feature visible on the topographic layout that affects the local drainage is the Metairie or Gentilly Ridge. It runs east-west between the river and the lake. The locations of the three major pump stations that pump into the 17th Street, Orleans Avenue, and London Street canals were influenced by this ridge.

Based on land topography and the drainage system, the basin is divided into 20 subbasins, including the Hoey’s basin in Jefferson Parish. Pump station information is presented in Section 3.2.1.5.6.4 of this volume.

Most of the local drainage is collected by underground storm drains that have been installed over many years. There are very few open ditches in this highly urbanized basin. Photos 1 and 2 show typical inlets and streets.

The land topography also influences the canal and pump station layout. With the relatively flat topography, development sequence, and location of outfall pump stations in this basin, interconnecting canals and interior pump (lift) stations were constructed to accommodate the interior drainage. Photo 3 shows an enclosed interior canal entering a pump station.

Photos 1 and 2. Typical Streets and Inlet – Orleans East Bank

Photo 3. Broad Street Canal Entering Pump Station #1

All but three of the interior canals are enclosed and most are under roadways. The enclosed canals are very large rectangular brick or concrete structures (Photo 4). The open canals are concrete lined or have sheet pile rectangular bottom sections with grass lined side slopes (Photos 5 and 6). The interior canals not only collect stormwater from streets and storm sewers and covey it to the pump stations, they also are storage areas that work in conjunction with the pump stations.

Photo 4. Claiborne Canal after Construction

Photo 5. Florida Avenue Canal from Louisa Street

Photo 6. Palmetto Canal from Jefferson Davis Parkway

Design Criteria. The current design criterion for new storm drainage facilities in Orleans East Bank is the 10 % probability (10 year frequency). The capacity of the older parts of the storm drain system is not known since improvements were made over many years. The functional capacity of the interior canals and pump stations is 0.5 inches per hour. Rainfall in excess of this amount goes into temporary storage in the storm sewers and streets. There are no criteria for redevelopments to use stormwater detention because the impervious cover wouldn't change significantly and delaying runoff to an outfall pump is counter productive.

Where local drainage is considered poor, the Sewerage and Water Board is working to improve the drainage. In some cases, the Sewerage and Water Board and Corps of Engineers are working together on projects, as presented below in the Southeast Louisiana (SELA) Urban Flood Control Projects section.

Southeast Louisiana Urban Flood Control Projects. As a result of the extensive flooding in May 1995, Congress authorized the SELA Urban Flood Control Project with enactment of the Energy and Water Development Appropriations Act for Fiscal Year 1996 and the Water Resources Development Act (WRDA) of 1996 to provide for flood control and improvements to rainfall drainage systems in Jefferson, Orleans, and St. Tammany Parishes. The Sewerage and Water Board of New Orleans is the local, cost sharing sponsor for the Orleans Parish work.

The project includes channel and pump station improvements in the three parishes. The channel and pumping station improvements in Orleans and Jefferson Parishes support the parishes' master drainage plans and generally provide flood protection on a level associated with a 10-year rainfall event, while also reducing damages for larger events.

Most of the work in Orleans Parish is in the Orleans East Bank basin. It consists of projects in three areas – Uptown/Broadmoor, Hollygrove, and Peoples Triangle, as shown in Figure 8.

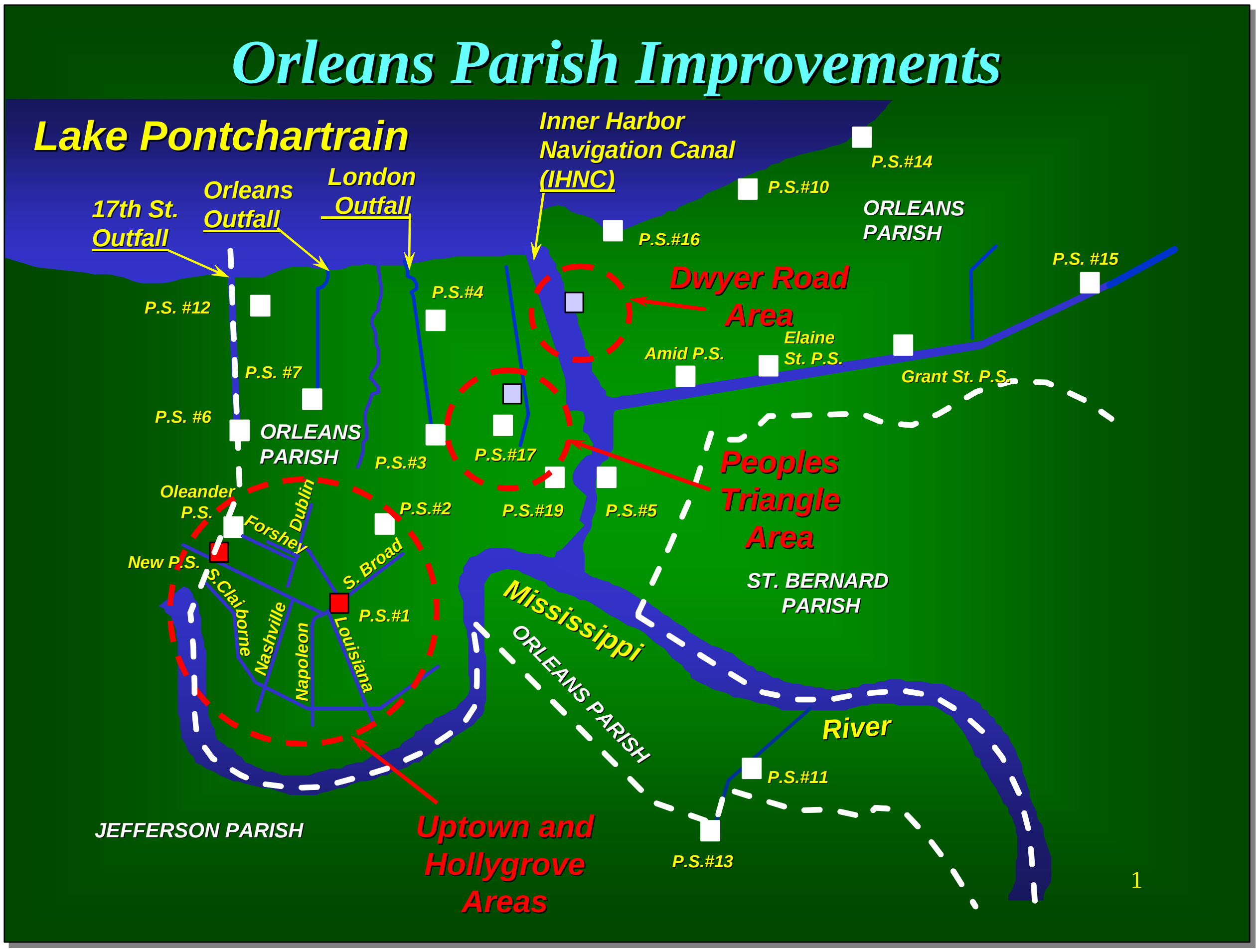


Figure 8. SELA Urban Flood Control Projects in Orleans East Bank

The Uptown/Broadmoor area work consists of additional enclosed canal capacity along Napoleon Avenue and South Claiborne; and increasing the pumping capacity of Drainage Pumping Station No. 1. This work was complete prior to Hurricane Katrina.

The Hollygrove area work consists of additional enclosed canal capacity along Forshey, Dublin, and Eagle streets; and a new pump station - Prichard Place Drainage Pumping Station. This work was complete prior to Hurricane Katrina.

The Peoples Avenue area work consists of additional enclosed canal capacity along Florida Ave. canal, Peoples Ave. canal, and a new pump station. This work was not started prior to Hurricane Katrina. It is waiting for Federal funding.

3.2.1.5.6.4. Pumping stations - Orleans Parish Summary. Figure 9 is a map showing the Orleans Parish pump stations that were used in this report. The locations of the pump stations were verified by Global Positioning System (GPS) and/or by using Google Earth Pro. The GPS coordinates were then input into Microsoft Streets and Trips (shown below).

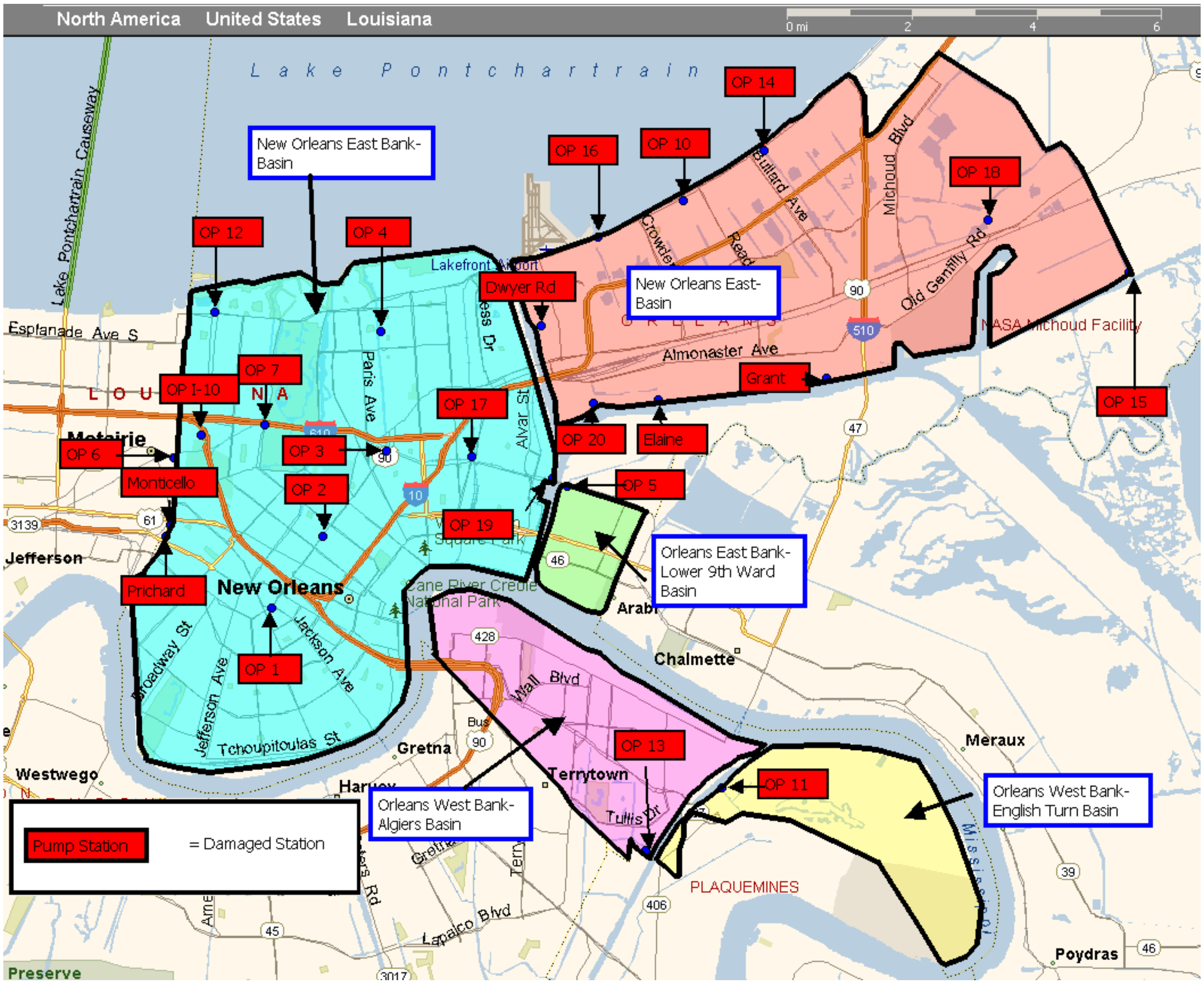


Figure 9. Orleans Parish Pump Station Locations

Table 8 contains a summary of Orleans Parish pump stations by drainage basin. The list is composed of information that was collected in the field. Not all information was available for each pump and was left blank or highlighted.

Table 8
Summary of Orleans Parish Pump Stations by Drainage Basin

Basin	East Bank	East	East Bank-Lower 9th Ward	West Bank-Algiers	West Bank-English Turn	Total
Number of pump stations	12	9	1	1	1	24
Number of pumps	68	24	7	7	5	111
Total rated capacity (cfs)	36,615	4,852	1,850	4,700	1,690	49,707
Estimated cost of damages	n/a	n/a	n/a	n/a	n/a	n/a

3.2.1.5.6.4.1. Drainage Basins. Orleans Parish consists of five drainage basins. The majority of the pump stations are in the East Bank and East basins. The Lower Ninth Ward, Algiers, and English Turn Basins have one pump station each. The Orleans Parish pump stations are listed below under their appropriate basins. Details for each pump station are listed in Volume VI.

3.2.1.5.6.4.1.1. East Bank. The East Bank Drainage Basin has 12 pump stations. It is bordered by Lake Pontchartrain on the north, and the Mississippi River on the south. Its drainage system includes the surrounding bodies of water, as well as the Melpomene, Broad Ave., Broad Street, Prentiss Ave., St. Anthony, Palmetto, Peoples, Florida, Monticello, 17th Street, Industrial, and Lake Canals. Below is a brief summary of each of the 12 pump stations. Volume VI provides more detailed information.

OP 1
Intake location:....................................Melpomene and Broad Ave Canals
Discharge location: ....Palmetto Canal
Nominal capacity: ....6825 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
A	550	1929	Electric 25 Hz	Horizontal
B	550	1929	Electric 25 Hz	Horizontal
C	1000	1929	Electric 25 Hz	Horizontal
D	1000	1929	Electric 25 Hz	Horizontal
E	1000	1929	Electric 25 Hz	Horizontal
F	1100	1991	Electric 60 Hz	Horizontal
G	1100	1991	Electric 60 Hz	Horizontal
V1	225	n/a	Electric 25 Hz	Vertical
V2	225	n/a	Electric 25 Hz	Vertical
CD1	60	n/a	Electric 25 Hz	Vertical
CD2	15	n/a	Electric 25 Hz	Centrifugal

OP 2
Intake location: ....Broad Street Canal
Discharge location: .... OP 3 & 7
Nominal capacity: ....3150 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
A	550	1914	Electric 25 Hz	Horizontal
B	550	1914	Electric 25 Hz	Horizontal
C	1000	1914	Electric 25 Hz	Horizontal
D	1000	1914	Electric 25 Hz	Horizontal
CD2	25	1974	Electric 25 Hz	Centrifugal
CD3	25	1974	Electric 25 Hz	Centrifugal

OP 3

Intake location: .... OP 2
Discharge location: .... London Ave Canal
Nominal capacity: .... 4340 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
A	590	1916	Electric 25 Hz	Horizontal
B	590	1916	Electric 25 Hz	Horizontal
C	1000	1930	Electric 25 Hz	Horizontal
D	1000	1930	Electric 25 Hz	Horizontal
E	1000	1930	Electric 25 Hz	Horizontal
CD 1	80	1916	Electric 25 Hz	Centrifugal
CD 2	80	1916	Electric 25 Hz	Centrifugal

OP 4

Intake location: .... Prentiss Ave and St. Anthony Canals
Discharge location: .... London Ave Canal
Nominal capacity: .... 3720 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
1	320	1938	Electric 60 Hz	Centrifugal
2	320	1938	Electric 60 Hz	Centrifugal
C	1000	1957	Electric 25 Hz	Horizontal
D	1000	1957	Electric 25 Hz	Horizontal
E	1000	1957	Electric 25 Hz	Horizontal
CD1	80	n/a	Electric 25 Hz	Vertical

OP 6

Intake location: .... Palmetto Canal
Discharge location: .... Forcemain and 17th Street Canal
Nominal capacity: .... 9480 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
A	550	1914	Electric 25 Hz	Horizontal
B	550	1914	Electric 25 Hz	Horizontal
C	1000	1928	Electric 25 Hz	Horizontal
D	1000	1928	Electric 25 Hz	Horizontal
E	1000	1928	Electric 25 Hz	Horizontal

F	1000	1928	Electric 25 Hz	Horizontal
G	1000	1984	Electric 25 Hz	Horizontal
H	1100	1984	Electric 60 Hz	Horizontal
I	1100	1984	Electric 60 Hz	Horizontal
CD 1	90	1984	Electric 60 Hz	Vertical
CD 2	90	1984	Electric 60 Hz	Vertical
1	250	1983	Electric 60 Hz	Vertical
2	250	1983	Electric 60 Hz	Vertical
3	250	1983	Electric 60 Hz	Vertical
4	250	1983	Electric 60 Hz	Vertical

OP 7

Intake location: .... OP 2
Discharge location: .... Lake Canal
Nominal capacity: .... 2690 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
A	550	1931	Electric 25 Hz	Horizontal
C	1000	1908	Electric 25 Hz	Horizontal
D	1000	1908	Electric 60 Hz	Horizontal
CD 1	70	n/a	Electric 25 Hz	Vertical
CD 2	70	n/a	Electric 25 Hz	Vertical

OP 12

Intake location: .... Robert E. Lee and Fluer De Lis Canals
Discharge location: .... Lake Pontchartrain
Nominal capacity: .... 1000 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
D	1000	1961	Electric 25 Hz	Horizontal

OP 17 (Station D)

Intake location: .... Peoples and Florida Ave. Canals
Discharge location: .... Mississippi River
Nominal capacity: .... 160 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
A	40	1975	Electric 60 Hz	Centrifugal
B	40	1975	Electric 60 Hz	Centrifugal
C	40	1975	Electric 60 Hz	Centrifugal
D	40	1975	Electric 60 Hz	Centrifugal

OP 19

Intake location: .... Florida Ave Canal
Discharge location: .... Industrial Canal (Lake Pontchartrain)
Nominal capacity: .... 3920 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
H1	1100	1975	Electric 60 Hz	Horizontal
H2	1100	1975	Electric 60 Hz	Horizontal
H3	1100	1975	Electric 60 Hz	Horizontal
V1	310	1975	Electric 60 Hz	Vertical
V2	310	1975	Electric 60 Hz	Vertical

I 10

Intake location: .... Railroad Underpass
Discharge location: .... 17th Street Canal
Nominal capacity: .... 850 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
1	250	n/a	Electric 60 Hz	Vertical
2	250	n/a	Electric 60 Hz	Vertical
3	250	n/a	Electric 60 Hz	Vertical
CD1	100	n/a	Electric 60 Hz	Centrifugal

Prichard

Intake location: .... Carrollton Drainage
Discharge location: ....Monticello Canal
Nominal capacity: ....250 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
1	125	n/a	Electric 60 Hz	Vertical
2	125	n/a	Electric 60 Hz	Vertical
CD1	n/a	n/a	Electric 60 Hz	Vertical

Monticello

Intake location: .... Carrollton Drainage
Discharge location: ....Monticello Canal
Nominal capacity: ....99 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
1	33	1979	Electric 60 Hz	Vertical
2	33	1979	Electric 60 Hz	Vertical
3	33	1979	Electric 60 Hz	Vertical

3.2.1.5.6.4.1.2. East Bank – Lower Ninth Ward. The Lower Ninth Ward drainage basin is bordered by the IHNC on the west, and the Mississippi River on the south. It only has one significant pump station, which is described below. Volume VI provides more detailed information.

OP 5

Intake location: ....Florida and Jourdan Ave. Canals
Discharge location: ....Lake Borgne
Nominal capacity: ....2260 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
A	550	1914	Electric 25 Hz	Horizontal
B	550	1914	Electric 25 Hz	Horizontal
D	1000	1961	Electric 25 Hz	Horizontal
CD1	40	n/a	Electric 25 Hz	Centrifugal
CD2	40	n/a	Electric 25 Hz	Centrifugal
CD3	40	1975	Electric 25 Hz	Centrifugal
CD4	40	1975	Electric 25 Hz	Centrifugal

3.2.1.5.6.5. Levees and floodwalls

3.2.1.5.6.5.1. Mississippi River Levees (Reference Nos. 44, 74, 78)

3.2.1.5.6.5.1.1. Geology. The study area is located within the Central Gulf Coastal Plain. Specifically, the area is located on the modern subdelta which projects gulfward from the deltaic plain of the Mississippi River. It is a region of extremely low relief. Dominant physiographic features are the natural levees of the Mississippi River and abandoned distributaries, and the marshlands and inland bodies of water that lie between the natural levee ridges. Elevations range from a maximum of about 5 feet along the crests of the natural levees to a minimum of sea level or slightly lower in the marshlands between the natural levee ridges. The numerous inland bodies of water vary in depth from 1 to 6 feet. The Mississippi River channel varies in depth from 65 to 190 feet below sea level. At present, the rate of subsidence in the study area varies between 0.5 and 1.0 feet per century.

3.2.1.5.6.5.1.2. Design Criteria. Reference 74 established the code for utilization of soils data for levees. The Mississippi River Levees were built under the 1947 CODE; however, EM 1110-2-1913 dated 30 April 2000 addresses the present day design criteria for slope design and settlement, as well as design of seepage berms for levees. Reference No. 74 spells out the general policies that were used to design the MRL levees with respect to planning, exploration, testing design and construction of Mainline Mississippi River levees. In addition to spelling out policy for exploration and testing, it spells out the design criteria. The design criteria call for three types of cross-sections. The three types of cross-sections recommended are as follows:

Type 1. Slightly smaller than the present compacted cross section. Compacted to maximum density at optimum moisture content. Comparable to earth dam construction.

Type 2. Intermediate in size between compacted and uncompacted fill Embankments having a moderate degree of compaction at natural moisture content. Uncompacted fills of material which are too wet for compaction.

Type 3. Moderately larger than the present uncompacted cross-section. Uncompacted emergency construction of relative dry material, that is, material sufficiently dry for a moderate degree of compaction. The reasons for adopting the three cross-sections as standards and more detailed description of their characteristics are given in the following paragraphs.

The three types of levee sections have been predicated on a stable foundation. In cases where unstable foundations exist, or detrimental underseepage conditions prevail, adequate corrective measures will be designed as a secondary consideration to take care of the special critical conditions which exist.

New Levees. The recommended net dimensions of the three types of sections for mainline levees are as follows:

Type	Riverside Slope	Crown Width	Landside Slope
1	1:3.5	10 feet	1:4.5
2 less than 25 ft in height	1:4.0	10 feet	1:5.5
2 25-ft and higher	1:4.0	10 feet	1:6.0
3	1:4.5	10 feet	1:6.5

The main line levees below New Orleans are generally less than 15 feet in height and are built to the tributary standard levee section of 1-on-3 RS, 1-on-4LS with a 10-foot crown. Underseepage and through seepage are not significant problems with these levees due to their relatively low height and the fact that both the levees and their foundations consist mainly of clays. On the lower reaches of the river, primarily below New Orleans, the levees are close to the river and are exposed to waves caused by wind and passing ships during high water. For this reason, these levees are provided with 4-in.-thick concrete slope paving on the riverside levee slope.

Enlargements. For enlargements, the existing levee should in all cases be considered as semi-compacted fill. Where the existing landside slope of the levee is flatter than 1:5.5 for levees less than 25 feet in height and1:6.0 for levees greater than 25 feet, the landside slope of a riverside enlargement should commence at the landside edge of the crown of the existing levee. Where the existing landside slope of the levee is steeper than 1:5.5 for levees less than 25 feet in height and 1:6.0 for levees greater than 25 feet, the landside slope of the enlargement should commence at a point where a 1:5.5 or 1:6.0 slope, whichever is applicable commencing at the landside toe, intersects the surface of the levee. From either of these commencement points, the net landside slope of the enlargement should be that applicable to the type of construction, i.e., 1:4.5 for Type 1, 1:5.5 or 1:6.0, whichever is applicable for Type 2, and 1:6.5 for Type 3. Crown widths and riverside slopes should be the same as for new levees. Where control of through seepage is required, the minimum thickness of enlargement .should' be 5 feet, measured normal to the riverside slope of the existing levees.

Seepage

General. In the location and design of levees, seepage conditions are considered to be one of the paramount features of design. Seepage correction at critical points on presently built levees is now of primary importance in the work of the three Districts.

Through Seepage. Through seepage in the present main line levee system seldom offers a serious problem. For those cases where it is a problem, an impervious core placed through the center of the levee section should be more effective against through seepage than a thin riverside blanket of impervious material.

Underseepage. Where a relatively thin (with respect to levee height) impervious topstratum exists above a highly pervious substratum of sand, conditions favorable to dangerous underseepage are present. Where river stages result in a hydraulic gradient or ratio of head to thickness of topstratum approaching unit, dangerous boils are likely to be encountered. A hydraulic gradient as low as 0.7 approaches the critical gradient for silts and fine sands.

Stability. Certain minimum stability requirements, previously stated, imply certain shear strength values. These values can be approximated by field moisture contents and Atterberg limits. The stability of the embankment and foundation should be such that on sudden drawdown (where applicable) the factor of safety should be at least 1. In other cases where drawdown does not apply, the minimum factor should be 1.3, except that where extremely weak materials make this impractical, a lower safety factor may be used. Where feasible and necessary, the procedure for constructing levees over very weak foundations in two or more stages is warranted and should be continued. On occasion, it may be necessary to use two or more stage construction, due to very wet levee materials.

3.2.1.5.6.5.1.3. Floodwalls. There are some existing floodwalls in downtown New Orleans that are part of the MRL system. These floodwalls run from the IHNC Loc to Audubon Park in New Orleans East Bank. The criteria for design are spelled out in Reference 77.

3.2.1.5.6.5.1.4. Levee Materials, MRL. The standard practice was to first obtain levee borrow from the batture area located on the riverside. The first levee sections were built using uncompacted fill placed with a tower machine. Later enlargements were constructed using hauling equipment and semi-compaction. The materials came from the batture area. In areas where there was no batture, borrow was obtained from off site.

3.2.1.5.6.5.2. Non Corps. Several local interest and/or private levees are located within the project area. No design criteria for these levees have been made available to the Corps.

3.2.1.6. New Orleans East

3.2.1.6.1. Introduction. The hurricane protection system for the New Orleans East (NOE) Basin was designed as part of the Lake Pontchartrain, LA and Vicinity Hurricane Protection Project. The NOE portion of the project protects 45,000 acres of urban, industrial, commercial, and industrial lands. Figure 10 illustrates the boundaries and basic flood protection components within the NOE Basin. The levee is constructed with a 10-ft crown width with side slopes of 1 on 3. The height of the levee varies from 13 to 19 ft. There are floodwall segments along the line of protection that consists of sheet-pile walls or concrete I-walls constructed on top of sheet-pile. The line of protection was designed to provide protection from the Standard Project Hurricane.

Figure 10 is used by the New Orleans District for planning and design, specifically because it shows as-built levee and floodwall elevations. The western border coincides with the Inner Harbor Navigation Canal (IHNC) and the eastern boundary of the Orleans Basin. It is bounded by the east bank of the IHNC, the Lake Pontchartrain shoreline (between the IHNC and South-point), the eastern boundary of the Bayou Savage National Wildlife Preserve, and the north side of the Gulf Intracoastal Waterway (GIWW) (between the IHNC and eastern edge of the Bayou Savage National Wildlife Preserve). The main components are described in the next section moving clockwise through the basin, beginning at the Lakefront Airport and ending at the western end of the GIWW.

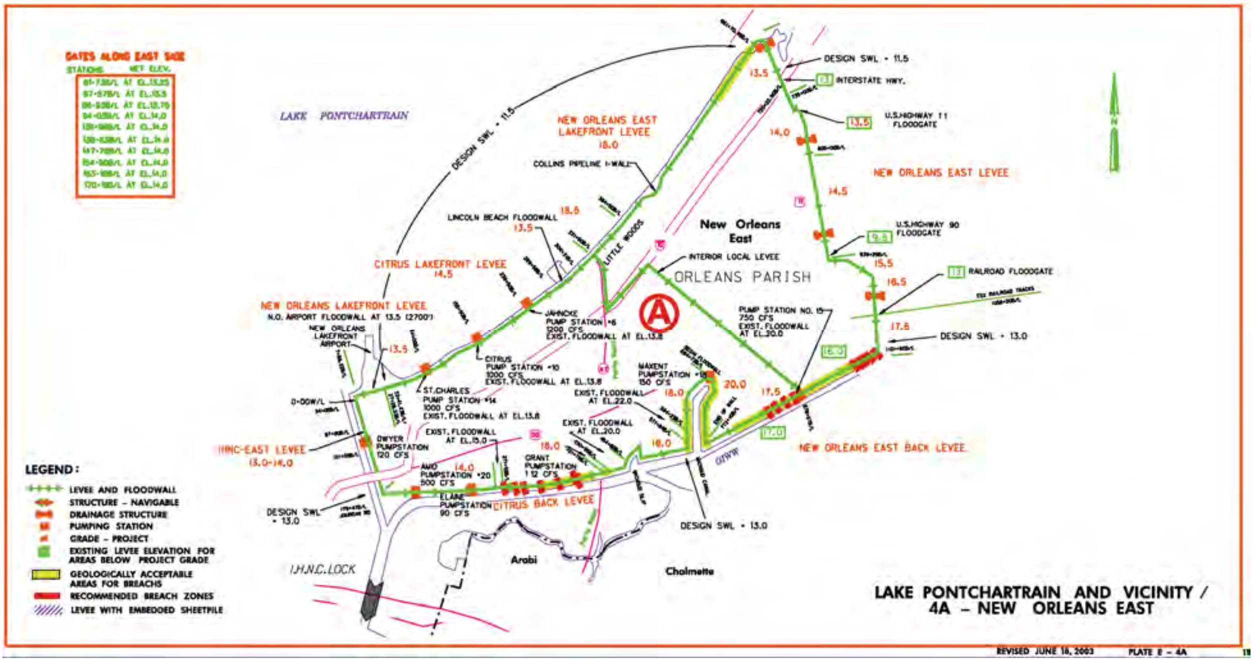


Figure 10. NOE Basin general components and top of levee/floodwall as-built elevations (feet) (source USACE, New Orleans District (Wayne Naquin)

Hurricane Protection Features New Orleans East Basin, Orleans Parish.

New Orleans East Lakefront includes the Citrus Lakefront Levee and New Orleans East Lakefront Levee consisting of 12.4 miles of earthen levee paralleling the Lakefront from the IHNC to Southpoint. It also includes floodwalls at the Lakefront Airport and Lincoln Beach.

The New Orleans East Levee consists of 8.4 miles of earthen levee from Southpoint to the GIWW along the eastern boundary of the Bayou Savage National Wildlife Preserve.

GIWW. The basin includes the Citrus Back Levee and New Orleans East Back Levee which consisting of approximately 17.5 miles of earthen levees and concrete floodwalls along the northern edge of the GIWW.

IHNC. The basin protection includes approximately 2.8 miles of levee and concrete floodwall along the eastern side of the IHNC. The IHNC is described in a separate report.

Pump Stations. Eight pump stations and numerous drainage structures, pipe crossings and culverts also lay on the boundaries.

Table 9 Summary of NOE Basin Hurricane Protection Features	
Exterior levee and floodwall (I wall)	39 miles
Drainage Structures	4
Pump Stations (local agencies)	8
Highway Closure Structures	2
Railroad Closure Structure	1

West and East Sides, IHNC, Orleans Parish. The Inner Harbor Navigation Canal (IHNC) HPP contains approximately 10 miles of levee and floodwalls along the Inner Harbor Navigation Canal in a heavily industrialized area.

3.2.1.6.2. Pre-Katrina - The Orleans Parish portion of the Lake Pontchartrain and Vicinity project is under construction. As of August 29, 2005, the remaining work consisted of the following:

- A levee enlargement along the New Orleans East Back levee.
- Rehabilitation of 4 small drainage structures in the South Point to GIWW reach in East New Orleans

Legislation is pending that would construct navigable closure structures at Seabrook and near the Paris Road Bridge. These structures would keep storm surges out of the IHNC area. If these are constructed, then the levee enlargement between Paris Road and the IHNC would no longer be required.

3.2.1.6.3. Design Criteria and Assumptions - Functional design criteria

3.2.1.6.3.1. Hydrology and Hydraulics. For New Orleans East, the design hurricane characteristics utilized in the design memoranda are shown in Table 10; the design tracks are shown on Figure 11. The maximum wind speed was computed using the same equations as for Orleans East Bank. For each project area, the track and forward speed were selected to produce maximum wind tide levels.

Table 10
Design Hurricane Characteristics

Location	Track	CPI, Inches	Radius of Maximum Winds, Nautical miles	Forward Speed, Knots	Maximum Wind Speed,[1] MPH	Direction of Approach
Lake Pontchartrain Southshore	A	27.6	30	6	100	South
Lake Borgne, Rigolets, and Chef Menteur Pass	F	27.6	30	11	100	East

[1] Windspeeds represent a 5 minute average 30 feet above ground level.

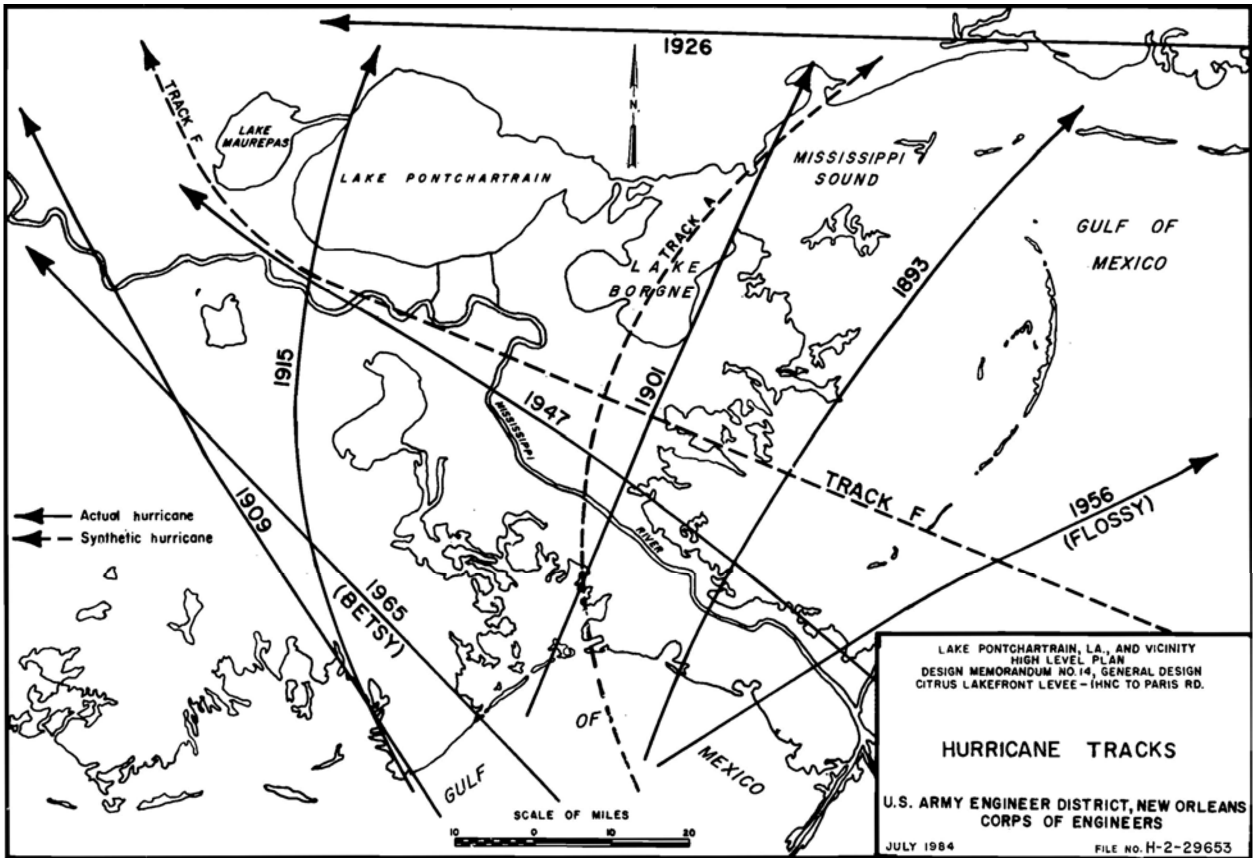


Figure 11. Hurricane tracks, New Orleans East Protection System

3.2.1.6.3.1.1. Surge. For Citrus Lakefront and New Orleans East Lakefront, wind tide levels were computed using the same methodology as used for Lake Pontchartrain lakefront for Orleans East Bank. For Citrus Back Levee, New Orleans East Back Levee, and IHNC from Seabrook to Citrus Back Levee, surge elevations were computed using the same methodology as used for IHNC for Orleans East Bank. For the New Orleans East levee from South Point to GIWW, wind tide levels were computed using the same methodology as used for Lake Pontchartrain lakefront for Orleans East Bank.

3.2.1.6.3.1.2. Waves. Wave runup along the Lake Pontchartrain shoreline was calculated using the methodology described in Orleans east bank. On the Citrus Lakefront Levee, foreshore protection on the floodside of the levee was considered to reduce wave runup, allowing for the levee crest height to remain lower than if no revetment was present. For New Orleans East Lakefront Levee, from 331+50 to 364+50, the presence of camps and land along this reach was considered to provide protection from normal wave activity. For the reach 364+50 to 661+70, a foreshore protection at the toe was required to prevent erosion due to normal wave activity. The runup height was reduced by 0.5 ft in this reach.

The levee from South Point to Highway 90 was not considered to be subject to waves during the peak hour of the design storm; the winds would be parallel to the levee, so wave runup would not occur. The levee from Highway 90 to the GIWW would be subject to waves generated in Lake Borgne. Wave runup was calculated using the methodology described in Orleans East Bank.

Along the Citrus Back Levee, waves were not considered a factor for the reach between IHNC and Paris Road. East of Paris Road, along the Citrus Back Levee and New Orleans East Back Levee, wave runup was calculated using the methodology described in Orleans East Bank.

Along the IHNC, waves were not considered a factor.

3.2.1.6.3.1.3. Summary. Table 11 contains maximum surge or wind tide level, wave, and design elevation information.

Table 11
Wave Runup and Design Elevations (transition zones not tabulated – governing DM is listed)

Location	DM	Average Depth of fetch, ft	Significant Wave Height Hs, ft	Wave Period, T, sec	Maximum Surge or Wind Tide Level, ft	Runup Height, ft	Freeboard, ft	Design Elevation Protective Structure, ft
Citrus Lakefront, 28+31 – 64+00*	DM14, Jul 1984	-	-	-	11.5 NGVD	-	3.0	14.5 NGVD
Citrus Lakefront, 64+00 to 331+50	DM14, Jul 1984	24.4	7.8	7.3	11.5 NGVD	3.0**	-	14.5 NGVD
New Orleans East Lakefront, 331+50 to 364+50	DM15, Apr 1985	24.4	7.8	7.3	11.5 NGVD	7.0	-	18.5 NGVD
New Orleans East Lakefront, 364+50 to 661+70	DM 15, Apr 1985	24.4	7.8	7.3	11.5 NGVD	6.5	-	18.0 NGVD
New Orleans East South Point to Highway 90	DM16, Sep 1987	-	-	-	11.5-12.2 NGVD	-	2.0	13.5-14.5 NGVD
New Orleans East, Highway 90 to Station 1030+00	DM16, Sep 1987	11.0	4.7	5.4	12.2 – 12.8 NGVD	4.5	-	15.5 - 17.5 NGVD
New Orleans East, Station 1030+00 to GIWW	DM16, Sep 1987	11.0	4.7	5.4	12.8 NGVD	4.5	-	17.5 NGVD
New Orleans East back levee, levee	DM2, Sup 04, Mar 1971	12.7	4.9	5.5	13.0 MSL	4.5	-	17.5 MSL
New Orleans East back levee, floodwall	DM2, Sup 04, Mar 1971	12.7	4.9	5.5	13.0 MSL	6.0	-	19.0 MSL
New Orleans East back levee, floodwall at PS 15	DM2, Sup 04, Mar 1971	12.7	4.9	5.5	13.0 MSL	10.0	-	23.0 MSL
Citrus Backlevee, west of Paris Road	DM2, Aug 1967	-	-	-	13.0 MSL	-	1.0	14.0 MSL
Citrus back levee, east of Paris Road	DM2, Aug 1967	13.1	4.7	5.4	13.0 MSL	5.0**	-	18.0 MSL
IHNC Seabrook to L&N Railroad Bridge	DM02, Sup 8, Feb 1968	-	-	-	11.4 – 12.9 MSL	-	1.0	13.0 – 14.0 MSL
IHNC L&N Railroad Bridge to Citrus Backlevee	DM02, Sup 8, Feb 1968				12.9 MSL	-	1.0	14.0 MSL

* At New Orleans Lakefront Airport, assume no waves.
**Foreshore protection reduces wave runup.

3.2.1.6.3.2. Geotechnical. The projects that make up the New Orleans East Levee are Citrus Lakefront Levee, New Orleans East Lakefront Levee, South Point to GIWW, New Orleans East Back Levee, Citrus Back Levee and IHNC East Levee from New Orleans Lakefront A.P. to Intersection of MRDG/GIWW with IHNC.

3.2.1.6.3.2.1. Citrus Lakefront Levee. The Citrus Lakefront Levee extends from the IHNC to Paris Road and includes 5.5 miles of earthen levee and 0.9 mile of I-wall. (Reference Nos. 17 and 18).

3.2.1.6.3.2.1.1. Geology. The geologic history and principle physiographic features of the New Orleans area and also surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.6.3.2.1.2. Foundation Conditions. The soil types and stratifications along the project alignment consist of 10 to 15 feet of artificial levee fill (natural material) underlain by the deposits of clays, silts and sands which exist down to -12.0 to -17.0 NGVD. The clays, silts and sands are underlain by sand deposits to -40.0 NGVD, the top of the Pleistocene surface.

3.2.1.6.3.2.1.3. Field Exploration. Undisturbed 5-inch diameter borings were made at 16 locations. General-type core borings 1-7/8-inch ID were made at 41 locations. Additional undisturbed borings were taken and tested by USACE along the centerline and 50 feet lakeside of the baseline.

3.2.1.6.3.2.1.4. Underseepage. Calculations were made to investigate the amount of seepage, uplift pressure and upward exit grades. Assumptions for the analyses are contained on Page 15 of DM 14 and Page 17 of DM No. 2, Supplement 54.

3.2.1.6.3.2.1.5. Pile Foundation. There were no pile foundations shown on the levee project.

3.2.1.6.3.2.1.6. Slope Stability. Using cross-sections representative of existing conditions along the levee, the stability of the levee was investigated by the method of plane analyses, using design Q shear strengths, the trends assigned to various levee sections and applying a minimum factor of safety with respect to shear strengths of 1.3.

3.2.1.6.3.2.1.7. I-Walls. DM 2 shows 0.9 miles of floodwall along the east bank IHNC, along the New Orleans Lakefront Airport and landside of Lincoln Beach. The stability and required penetration of steel sheet pile below the ground surface was determined by the Method of Planes using S shear strengths. Sufficient Q stability analyses were performed to insure S case governed. A factor of safety of 1.5 was applied to design shear strengths. The sheet pile penetration of required to satisfy a Lane's creep ratio of 7 was used. The deeper penetration of the two analyses was used to select the tip elevation.

3.2.1.6.3.2.1.8. T-Walls. The T-type floodwalls supported on bearing piles will provide protection adjacent to T-type gates supported by bearing piles.

3.2.1.6.3.2.1.9. Erosion Protection. Thirty-six-inch derrick stone will be placed on a 12-inch rip rap blanket to cover the lakeside slope of the existing railroad embankment.

3.2.1.6.3.2.2. New Orleans East Lakefront Levee. The New Orleans East Lakefront Levee consists of 6.3 miles of earth levee and 463 feet of I-wall. (Reference Nos. 19 and 20).

3.2.1.6.3.2.2.1. Geology. The geologic history and principle physiographic features of the New Orleans area and also surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.6.3.2.2.2. Foundation Conditions. Generally the area consists of Holocene deposits varying from elevation approximately -40.0 feet NGVD to -25.0 NGVD. These deposits are predominantly unconsolidated, saturated, low strength clays with some silts and silty sands. Artificial fill consisting of sand core overlain by a semi-compacted clay cap was placed along the levee centerline from -10.0 to 15.0 NGVD.

3.2.1.6.3.2.2.3. Field Investigation. For DM 15, a total of 15 undisturbed borings were taken and tested. Eleven along the centerline and 120 feet land side of the levee. Additional old borings were considered.

3.2.1.6.3.2.2.4. Underseepage. Not used.

3.2.1.6.3.2.2.5. Hydrostatic Pressure Relief. Not used.

3.2.1.6.3.2.2.6. Pile Foundation. Not used.

3.2.1.6.3.2.2.7. Slope Stability. The stability of the levee was determined by the Method of Planes using the design (Q) shear strengths and applying a minimum factor of safety of approximately 1.3. To preclude potential movement of pipelines conveying high pressure volatile liquids and/or gases, a factor of safety of 1.5 was utilized to design the levee at pipeline relocations. The cases analyzed were:

(1) Water level to Elev. 1.0 with anticipated failure into the demucked canal (floodside)

(2) Water level to project Hurricane wind tide level (WTL) Elev. 8.5 on floodside and Elev. 0.0 on protected side.

(3) Water to Elev. 0.0 on both sides and failure to the floodside.

The results of (Q) triaxial shear tests from two borings were used to develop a composite shear strength ____ depth profile.

3.2.1.6.3.2.2.8. I-walls. A short section (463 feet) of I-wall was used at the Collins pipe crossing. Factor of safety of 1.25 w/ static water at the wind tide level of 11.5 NGVD and a dynamic levee force. The wall was analyzed for both the (Q) and (S) cases, but the (S) case governed. The Lane's creep ratio of 2.5 was used as well. The creep ratio analysis controlled and a tip penetration of -13.0 NGVD was used to penetrate the sand core.

3.2.1.5.3.2.2.9. T-Walls. Not used.

3.2.1.6.3.2.3. New Orleans East Levee South Point to GIWW (Reference Nos. 21 and 22). There was an existing levee along the alignment which was constructed in 1956 by the Orleans Parish Levee District. This initial levee construction consisted of demucking the organic clay along the levee centerline, constructing retaining dikes and pumping hydraulic fill between the dikes and shaping to Elev. 11.5 NGVD. The new levee enlargement for this levee section consisted of a straddle enlargement done with clay fill material.

3.2.1.6.3.2.3.1. Geology. The geologic history and principle physiographic features of the New Orleans area and also surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.6.3.2.3.2. Foundation Conditions. The subsurface along this project consists generally of 16 to 20 feet of artificial levee fill. Below this artificial fill, 13 to 22 feet of Recent deposits of clays, silts, and sands with a Pleistocene deposit encountered at approximate elevations of -23.0 to -32.0. Also contained in this area are two abandoned distributaries. Between Station 939+60 and Station 1102+98 underneath the artificial fill exists 6 to 40 feet of Recent deposits of clay with layers and lenses of silt and sand which are underlain by a sand deposit at approximate elevations -14.0 to -50.0.

3.2.1.6.3.2.3.3. Field Exploration. Twelve 5-inch undisturbed borings and twenty-two general type borings were taken along the levee alignment for DM No. 2 General Design Supplement 9 and an additional nine new 5-inch diameter undisturbed borings were taken for Design Memorandum No. 16 General Design.

3.2.1.6.3.2.3.4. Underseepage. A sheet pile cutoff was used beneath the railroad gate to control underseepage.

3.2.1.6.3.2.3.5. Hydrostatic Uplift. Not used.

3.2.1.6.3.2.3.6. Pile Foundation. Design compression and tension capacities versus tip elevation were developed for 12-inch square prestressed concrete piles. The piles will support the railroad swing gate monolith. In compression, a factor of safety of 1.75 with a $K_0 = 1.0$ for the (S) case. In tension, a factor of safety of 2.0 was applied with a K_0 for the (S) case. The Q case governed the design.

3.2.1.6.3.2.3.7. Slope Stability. Shear stability was determined using cross-sections represent of existing conditions along the levee alignment, for the condition of water to elevation 0.0 on both sides of the levee and assumed failure towards the flood side and for the conditions with water to elevation 11.5 (WTL) on the flood side and to elevation 0.0 on the protected side and assumed failure toward the protected side. A minimum factor of safety of 1.3 was used.

3.2.1.6.3.2.3.8. I-Walls. I-type floodwalls were used to tie the railroad swing gate to the earthen levees. The stability and required penetration of steel sheet pile below the earth surface were determined by the Method of Planes using the (S) shear strength, i.e., $c = 0$, $\phi = 23$ degrees. An I-type floodwall will be constructed on the levee crown in the vicinity of the drainage structure located at Station 105+55.

3.2.1.6.3.2.3.9. T-Walls. Not used.

3.2.1.6.3.2.3.10. Review Comments. First Endorsement Comment 2i(1) as to whether a wave wash clay blanket thickness of 2 feet was adequate. District Response was that 2 feet of riprap plus 2 feet of clay blanket was adequate.

3.2.1.6.3.2.4. New Orleans East Back Levee (Reference No. 23). The New Orleans East Back Levee is located on the north bank of the MRGO and GIWW and extends from the Michaud Assembly Facility (NASA) to Intersection with the south point to GIWW New Orleans East Levee. This project consists of 4.4 miles of levee to elevation 17.5 NGVD and 2 miles of I-wall and inverted T-wall to elevations between 19.0 and 23.0 NGVD.

3.2.1.6.3.2.4.1. Geology. The geologic history and principle physiographic features of the New Orleans area and also surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.6.3.2.4.2. Project Foundation Conditions. The subsurface along this project generally consists of 10 to 15 feet of artificial levee fill overlying 40 to 60 feet of Recent deposits of clays, silts and sands which are underlain by Pleistocene deposits encountered at approximate elevations of -40.0 NGVD to -60.0 NGVD.

3.2.1.6.3.2.4.3. Field Investigation. A total of eight 5-inch undisturbed borings along with 15 1-7/8 inch general-type core borings were made along the levee alignment.

3.2.1.6.3.2.4.4. Underseepage. Not addressed.

3.2.1.6.3.2.4.5. Hydrostatic Uplift. Not addressed.

3.2.1.6.3.2.4.6. Pile Foundation. The pile foundation for T-type floodwalls would be 12-inch prestressed concrete piles. Design compression and tension capacities vs. tip elevations were developed using (Q) and (S) shear strengths. In compression, a factor of safety of 1.75 with a (K_0) = 1.0 and for tension a factor of safety of 2.0 with (K_0) = 0.7 were used. The (Q) case governed the design so capacities for tip elevation vs. capacity were presented for 14-inch and 16-inch piles for the (Q) case. During construction, bearing pile tests will be conducted at two sites.

3.2.1.6.3.2.4.7. Levee Stability. Slope stability analyses were run for the following conditions:

(1) Hurricane floodwater to the WTL (Elevation 13.0 NGVD) on the floodside and water to natural ground on the protected side.

(2) Water to elevation 0.0 on both sides.

(3) For levees with T-type floodwalls, the water elevation was considered to be equal to the top of the design hurricane wave on the flood side and 0.0 on the protected side.

The stability analyses were determined by the Method of Planes using (Q) strengths and a minimum factor of safety with respect to shear strength of approximately 1.3.

3.2.1.6.3.2.4.8. I-Wall Design. Stability and the required penetration of steel sheet piles below the earth surface were determined by the Method of Planes using (S) shear tests. Sufficient (Q) case analyses were run to confirm the (S) case governed. (see Division Comments on District Response).

3.2.1.6.3.2.4.9. T-Type Floodwall. Inverted T-type floodwalls on bearing piles will be utilized in lieu of I-type floodwalls at overland utility crossings, gate monoliths and pumping stations. A steel sheet pile cutoff will be used beneath the wall to control underseepage.

3.2.1.6.3.2.4.10. Endorsement Comments on I-wall Design. 1st Ind Comment 19. The shallowest failure surface intersects the tips of the sheet pile I-walls. This implies that either this is the location of a critical failure surface or that no critical failure surface at high elevations or some value of shear strength is analyzed as if the sheet piling are not existent. Any sections where this change in concept caused an appreciable change in factor of safety should be reanalyzed.

4th Ind. Response to Comment 19. The levee embankment containing I-wall sections referred to in this paragraph were analyzed for the change of concept, i.e., considering the sheet piling as nonexistent, and there were no appreciable changes in factors of safety. We can really understand and appreciate the value of this type of concept for establishing a conservative design for cursory review; however, we feel that general use of this procedure for final design will result in ultra conservative designs and would prevent the use of I-type walls.

For purposes of maintaining currently approved designs, inviolate and establishing mutually acceptable guidelines for future design, we proposed a modified approach as follows:

(1) Using the construction (Q) shear strengths and conventional I-type wall analyses, determine the minimum sheet pile penetration needed to retain the differential water head (FS – 1.0) and the maximum penetration required for design.

(2) Disregard the sheet pile below the minimum penetration and conduct conventional levee stability analyses, deducting the lateral load of the differential head caused by the wedge of water supported by the I-wall adding, however, the effects of the weight of all other water overlying the active wedges.

(3) The stability of the levee containing the I-type wall will be considered acceptable if the factors of safety for assumed failure surfaces between the minimum tip penetration and required penetration are above 1.0 and the FS below the tip penetration is 1.3 or higher. Further, for subsurface conditions having a soft stratum (low shear strengths) overlying a firm stratum (high shear strength), the pile penetration into the firm stratum will be to sufficient depth to prevent the pile tip from kicking up into the soft stratum.

3.2.1.6.3.2.5. Citrus Back Levee (Reference No. 24). The Citrus Back Levee is located on the north bank of the MRGD and GIWW and extends from a junction with protective works on the east bank of the IHNC to and through the site occupied by the Michaud Assembly Facility (NASA). It includes 8 miles of levee and 1 mile of floodwall.

3.2.1.6.3.2.5.1. Geology. The geologic history and principle physiographic features of the New Orleans area and also surface and subsurface geology of the New Orleans area are described in Volume V.

3.2.1.6.3.2.5.2. Project Foundation Conditions. The subsurface along the project consists generally of 10 to 15 feet of artificial levee fill overlying 45 to 60 feet of Recent deposits of clays, silts, and sands which are underlain by a Pleistocene deposit encountered at elevation -50 NGVD on the west end to -60 NGVD on the east end.

3.2.1.6.3.2.5.3. Field Investigation. Four 5-inch diameter undisturbed soil borings were made along the levee alignment. Fifteen 1-7/8 inch ID general type core borings were also made. In order to insure adequate design additional soil borings were scheduled between successive lifts.

3.2.1.6.3.2.5.4. Underseepage. A steel sheet pile cutoff will be used beneath the short reach of T-wall to protect from underseepage.

3.2.1.6.3.2.5.5. Hydrostatic Uplift. Not addressed in this DM.

3.2.1.6.3.2.5.6. Pile Foundation. Twelve inch square prestressed concrete piles will be used to support the T-type walls and gated structures. Bearing and tension values vs. tip elevation will be computed using the (Q) and (S) shear strengths. In compression, a factor of safety of 1.75 was applied with a (K_0) – 1.0. In tension, a factor of safety of 2.0 and (K_0) = 1.7 was used. The (S) case governed the design.

3.2.1.6.3.2.5.7. Levee Stability. Staged construction was used to get the levees to their design height. The slope and beam distance for each stage was based on the following conditions: hurricane water condition at still water level + 13.0 NGVD with landside failure and mean low water canal side with a canal-side failure. The stability of the levee was determined using the Method of Planes using (Q) design strengths and applying a minimum factor of safety of 1.3 with respect to shear strengths.

3.2.1.6.3.2.5.8. I-Wall Design. The penetrations for the sheet piles were computed for a dynamic water force and a minimum factor of safety of 1.25 and a static water level of 14.5 feet NGVD with a factor of safety of 1.5. The stability and sheet pile penetration were determined by the Method of Planes using (S) shear strengths sufficient (Q) stability analyses were performed to insure the (S) case governed.

3.2.1.6.3.2.5.9. Erosion Protection. Due to short duration of hurricane stages and the resistant nature of the clays, no erosion protection is considered necessary.

3.2.1.6.3.2.5.10. Review Comments. First Endorsement Comment paragraph 2 questions the depth of penetration in reach 430+95 to 454+80 and I-wall analyses for Stations 571+55 to 584+23.6 for dynamic load case

Paragraph 24 questions number of undisturbed borings as adequate. Fourth Endorsement and response not in document.

3.2.1.6.3.2.6. New Orleans East, IHNC Floodwall/Levee (Reference No. 9). This project included all the protection works between the end of the Citrus Lakefront Levee and the end of the Citrus Back Levee. I-type floodwalls will be used for all height above grade is less than 10 feet and a bearing pile supported T-type floodwall for all heights greater than 10 feet.

3.2.1.6.3.2.6.1. Geology. The geologic history and principle physiographic features of the New Orleans area are described in Volume V.

3.2.1.6.3.2.6.2. Project Foundation Conditions. The subsurface along the project consists of generally 6 to 10 feet of artificial fill overlying 40 to 50 feet of Recent deposits of sand, silts, and clays which are underlain by the Pleistocene soils. The Pleistocene surface is encountered at elevation -50 NGVD on the Citrus Lakefront end to -70 NGVD at various locations.

3.2.1.6.3.2.6.3. Field Investigation. Nine 5-inch diameter undisturbed soil borings were made along the east alignment. Twenty-three general type borings were also made. Borings were generally made along the levee alignment at intervals ranging from 350 to 1,500 feet.

3.2.1.6.3.2.6.4. Underseepage. Because of the sandy levee and foundation in the buried beach area, interception of seepage through the levee and reduction of piezometric heads in the foundation sands are necessary to maintain stability. The I-wall sheet pile was extended in depth below that required for stability when necessary to cut of the upper sand fill strata.

3.2.1.6.3.2.6.5. Hydrostatic Uplift. Permanent hydrostatic pressure relief wells will be provided in the buried beach sand area. Piezometers were installed to determine the existing hydrostatic conditions.

3.2.1.6.3.2.6.6. Pile Foundations. Twelve-inch thick prestressed concrete pile will be used. Prior to construction, bearing pile tests will be conducted at selected locations along the line of protection for selecting pile lengths. The following design criteria were used:

FOS Compression	1.75	$K_0 = 1.0$
FOS Tension	2.0	$K_0 = 0.7$
(S) case governed		
(S) strength	Recent clays	$\phi = 23$ degrees, c = 0
	Pleistocene Clays	$\phi = 25$ degrees, c = 0
	Sand	$\phi = 30$ degrees, c = 0

Notes: Skin friction disregarded above bottom of the marsh deposit and or upper one-third of Recent deposits.

3.2.1.6.3.2.6.7. Levee Stability. Levee stability analyses were incorporated into the stability analyses of the cantilever I-type floodwalls. The levees were also checked for the (Q) case using the Method of Planes and a factor of safety with respect to shear strength of 1.3. For those stability analyses for levees in the buried beach sand reaches, hydrostatic uplift was applied on the base of the clay from top of the sand to the midwell piezometric heads determined from the relief well analyses.

3.2.1.6.3.2.6.8. I-Type Floodwalls. The stability of the levee and floodwalls section and the required penetration of the steel sheet pile below the earth surface were determined by the Method of Planes using (S) shear strengths. A factor of safety of 1.5 was applied to the design shear strengths. The stability of the floodwall was determined for a hurricane water level 6 inches below the top of the wall on the floodside and for the groundwater level at the ground surface on the protected side.

3.2.1.6.3.2.6.9. Erosion Protection. Due to the short duration of hurricane floods, the resistant nature of clayey soils and limited conditions for wave generation, erosion protection was not considered necessary.

3.2.1.6.3.2.6.10. Review Comments. First Endorsement Comment paragraph 5 questioned the use of a 9-foot I-wall to hold back earth fill at the Chef Menteur Bridge with a compiled deflection of 4 inches.

Paragraph 6 questions excessive recommended depth of sheet pile Station 115+65 to 117+50 and 119+59 to 132+00.

Paragraph 9 questions use of I-walls between 8 and 10 feet high. Fourth Endorsement changes method of computing net pressure diagram on sheet piles cutoff walls, concurrent with comment paragraph 6 and did not respond to paragraph 9.

3.2.1.6.3.3. Structural

3.2.1.6.3.3.1. New Orleans East Lakefront

Citrus Lakefront Levee (Reference 17)

General. The structural features include floodwalls to replace the levee from the tie-in to the floodwall along Jourdan Road to B/L Station 28+31 and in the vicinity of Lincoln Beach. Within the floodwall reaches two steel overhead roller gates and three steel swing gates were constructed. The overhead roller gates are located across Hayne Blvd. at Jourdan Road and across the entrance to Lincoln Beach. The swing gates are located across the Southern railroad track near the IHNC, across the New Orleans Lakefront Airport service road near Seabrook Bridge, and across an entrance to the New Orleans Lakefront Airport.

The basic data relevant to the design of the protective works are shown in the following tabulation:

Water Elevations	Elevation (feet m.s.l.)
Wind tide level (IHNC)	13.0
Wind tide level (Lake Pontchartrain)	8.5
Landside of Floodwall	0.0
Unit Weights	**Lb. per cu ft**
Water	62.5
Concrete	150
Steel	490
Design Loads	**Lb. per cu ft**
Wind load	50 p.s.f.

Allowable Working Stresses. The allowable working stresses for concrete and structural steel are in accordance with those recommended in "Working Stresses for Structural Design," EM1110-1-2101 dated 1 November 1963 and amendment No. 1 dated 14 April 1965. The basic minimum 28-day compressive strength for concrete is 3,000 p.s.i., except for prestressed concrete piling where the minimum is 5, 000 p.s.i. Steel for sheet piling met the requirements of ASTM A328-69, "Standard Specification for Steel Sheet Piling." Pertinent allowable stresses are tabulated below:

Reinforced Concrete	
f'_c	3,000 p.s.i.
f_c	1,050 p.s.i.
V_c (without web reinforcement)	60 p.s.i.
V_c (with web reinforcement)	274 p.s.i.
f_s	20,000 p.s.i.
Minimum tensile steel	0.00025 bd sq. in.
Shrinkage and temperature steel area	0.0020 bt
Structural Steel (ASTM A-36)	
Basic working stress	18,000 p.s.i.

I-type floodwall. The I-wall consists of sheet piling driven into the existing ground and in some cases into a new embankment and the upper portion of the sheet piling will be capped with concrete. In the design of the I-wall, one loading case was considered.

- Case I. Static water at 6 inches below top of wall, no wind, no dynamic wave force.

T-type floodwall. Four T-wall monoliths were constructed along the eastside of Jourdan Road adjacent to gate monoliths No. 1 and No. 2, and four other T-wall monoliths were constructed adjacent to both sides of gate No. 5 at Lincoln Beach. These walls were designed for the following load conditions.

- Case I. Static water to top of wall, no wind, impervious sheet pile cutoff, no dynamic wave force.

- Case II. Static water to top of wall, no wind, pervious sheet pile cutoff, no dynamic wave force.

- Case III. No water, no wind.
- Case IV. No water, wind (75 percent forces used).

Piling. Prestressed 12-inch square concrete piles were used meeting the requirements of the joint AASHO and PCI committee standard specifications for “square concrete prestressed piles.”

Overhead roller gates. Overhead roller gates were planned at Hayne Blvd. and at Lincoln Beach designed to meet the following loading criteria:

- Case I. Water at top of wall, no wind, impervious sheet pile cutoff.
- Case II. Water, at top of wall, no wind, pervious sheet pile cutoff.
- Case III. Water at el 9.75 of wall, no wind, impervious sheet pile cutoff.
- Case IV. Water at el. 9.75 of wall, no wind, pervious sheet pile cutoff.
- Case V. No water, no wind, truck on edge of slab, flood side
- Case VI. No water, no wind, truck on edge of slab, protected side.
- Case VII. No water, wind from flood side, truck on edge of slab, protected side, 33 1/13 percent increase in allowable stresses.
- Case VIII. No water, wind from protected side, truck on edge of slab, flood side, 33 1/3 percent increase in allowable stresses.

Swing Gates. Three swing gates will be constructed in the vicinity of the New Orleans Lakefront Airport designed with the following loading cases:

- Case I. Gate closed water at top of wall, no wind.
- Case II. Gate closed, water at top of wall, wind from flood side 33 1/13 percent increase in allowable stresses.
- Case III. Gate opened (parallel to wall), no water, no wind.
- Case IV. Gate opened (perpendicular to wall), no water, no wind.

3.2.1.6.3.3.2. Paris Road to South Point (Reference 20).

General. As constructed, Paris Road to South Point consists of earthen levees with uncapped cantilevered I-wall in the vicinity of the Collins Pipeline Company’s 16-inch pipeline crossing, and one soil-founded, sluice gated drainage structure at the South Point edge of the project.

The basic data relevant to the design of the protective works are shown in the following tabulation:

Water Elevations	**Elevation (feet N.G.V.D.)**
Wind tide level (Lake Pontchartrain)	11.5
Landside of Floodwall	0.0
Unit Weights	**Lb. per cu ft**
Water	64.0
Concrete	150
Design Loads	**Lb. per cu ft**
Wind load	50 p.s.f.

Design Methods.

Reinforced concrete. The design of reinforced concrete structures were performed in accordance with the strength design method of the ACI Building Code, as modified by the guidelines of “Strength Design Criteria for Reinforced Concrete Hydraulic Structures”, ETL 1110-2-265 dated 15 September 1981. The basic minimum 28-day compressive strength concrete is 3,000 psi. Pertinent stresses are tabulated below:

f’c	3,000 psi
fy (grade 40 steel)	40,000 psi
Maximum flexural reinforcement	0.25 x balanced ratio
Minimum flexural reinforcement	200/fy

I-type Floodwall.

General. The I-wall consists of steel sheet piling driven into the existing ground and, in some cases, into a new embankment. The upper portion of the sheet piling will be capped with concrete. The sheet piling was driven to the required depth with 1 foot of the sheet piling extending above the finished ground elevation. The concrete portion of the flood wall extended from 2 feet below the finished ground elevation to the required protection height.

Loading Cases. In the design of the I-wall, one loading case was considered as follows:

FS used = 1.25 with static water at the SWL and a dynamic wave force.

3.2.1.6.3.3.3. New Orleans East Levee. New Orleans East Levee, South Point to GIWW (Reference 21).

General. As constructed, South Point to GIWW consists of earthen levee with three soil-founded sluice-gated drainage structures one of which is straddled by a pile-supported uncapped I-wall, and one pile-founded railroad swing gate with two adjacent capped cantilevered I-wall Monoliths.

The basic data relevant to the design of the relocated drainage structures and the swing gate are as follows:

Water elevation	**Elevations, feet m.s.l.**
Wind tide level (WTL)	
South Point to Highway 90	8.5-11.5
Highway 90 to GIWW	11.5-12.8
Landside of Structure	0.0
Unit weights	**Lb. per cubic foot**
Water	62.5
Concrete	150
Steel	490
Earth	117
Design loads	
Earth pressure (lateral)	
Clay	90 p.s.f.
Sand	65 p.s.f.
Wind pressure (lateral	50 p.s.f.

Allowable Working Stresses. The allowable working stresses for concrete and structural steel are in accordance with those recommended in "Working Stresses for Structural Design," EM1110-1-2101 dated 14 April 1965. The basic minimum 28-day compressive strength for concrete is 3,000 p.s.i., except for prestressed concrete piling where the minimum is 5, 000 p.s.i. Steel for sheet piling met the requirements of ASTM A328-69, "Standard Specification for Steel Sheet Piling." Pertinent allowable stresses are tabulated below:

Reinforced Concrete	
f'_c	3,000 p.s.i.
f_c	1,050 p.s.i.
V_c (without web reinforcement)	60 p.s.i.
V_c (with web reinforcement)	274 p.s.i.
f_s	20,000p.s.i.
Minimum tensile steel	0.00025 bd sq. in.
Structural Steel (ASTM A-36)	
Basic working stress	18,000 p.s.i.

Swing Gate Structure. A swing gate structure was constructed where the L&N Railroad crosses the levee. The gate monolith was designed for the following load conditions:

- Case I – Water to top of wall at elevation 13.0, no wind, impervious soil
- Case II – Water to top of wall at elevation 13.0, no wind, pervious soil
- Case III – No water, no wind, one train wheel axle on edge of slab, flood side
- Case IV – No water, no wind, one train axle on edge of slab, protected side

- Case V – No water, no wind, two train wheel axles on edge of slab, flood side
- Case VI – No water, no wind, two train wheel axles on edge of slab, protected side

I-Type Wall Structure. (Reference 22)

In the design of the I-wall, two loading cases were considered as follows:

- Case I – FS used = 1.5 with static water at the SWL and no dynamic wave force.
- Case II – FS used = 1.25 with static water at the SWL and a dynamic wave force.

Water Elevations for Design of the I- Wall	Elevations feet (NGVD)
Wind tide level (WTL)	
Drainage Structure	13.0
Landside of Structures	0.0

3.2.1.6.3.3.4. GIWW. New Orleans East Back Levee (Reference 23)

General. The structural features of this reach include 2 miles of floodwall (I-type and inverted T-type) constructed to an elevation of 20.0 feet. The plan also provides for constructing four T-walls and eight gate closures and modifying eight road ramps, eight pipeline and four electric cable crossings, and the floodwall at an existing pumping station.

The basic data relevant to the design of the protective works are shown in the following tabulation:

Water Elevations	Elevation (feet m.s.l.)
Still Water Level	13.0
Landside of Wall	0.0
Unit Weights	**Lb. per cu ft**
Water	62.5
Concrete	150
Steel	490
Design Loads	**Lb. per cu ft**
Wind loads	50

I-Wall Loading Cases. In the design of the I-wall, two loading cases were considered:

- Case I – Static water to the wind tide level, elevation 13.0, 1.5 factor of safety in the soil, no dynamic wave force
- Case II – Static water to top of broken wave, 1.25 factor of safety in the soil, dynamic wave load from broken wave

T-type floodwall – Vicinity of Michoud Canal. Walls were designed assuming the sheet pile cutoff to be impervious and under the following conditions:

- Case I – Water at WTL El. 13.0, no wave, no wind
- Case II – Water at WTL El. 13.0, no wave, wind from flood side. 33 1/3 percent increase in allowable stresses.
- Case III – Broken waves to elevation 18.8, wave force, wind from flood side. 33 1/3 percent increase in allowable stresses.
- Case IV – No water or wave force, wind from protected side. 331/3 percent increase in allowable stresses.

T-type floodwall – Vicinity of Pumping Station. The walls were designed assuming the sheet pile cutoff to be impervious and for the following loading conditions:

- Case I - Water at WTL el. 13.0, no wave, no wind, discharge pipes filled with water.
- Case II - Water at WTL el. 13.0, no wave, no wind, discharge pipes empty.
- Case III - Water at WTL el. 13.0, no wave, wind from flood side, discharge pipes filled with water. 33 1/3 percent increase in allowable stresses.
- Case IV - Water at WTL el. 13.0, no wave, wind from flood side, discharge pipes empty. 33 1/3 increase in allowable stresses.
- Case V - Broken waves to el. 17.4, wave force, wind from flood side, discharge pipes filled with water. 33 1/3 percent increase in allowable stresses.
- Case VI - Broken waves to el. 17.4, wave force, wind from flood side, discharge pipes empty. 33 1/3 percent increase in allowable stresses.
- Case VII - No water or wave force, wind from protected side, discharge pipes empty. 33 1/3 percent increase in allowable stresses.

Gates. Eight gate monoliths were constructed for access roads in lieu of I-wall between Station 664+73.3 and Station 772+00. The gate monoliths were designed for the following load conditions:

- Case I - Water at WTL el. 13.0, no wave, no wind.
- Case II - No water, no wave, no wind, truck loading on edge of slab at protected side.
- Case III - No water, no wave, no wind, truck loading on edge of slab at flood side.

- Case IV - Water at WTL el. 13.0, no wave, wind from flood side. 33 1/3 percent increase in allowable stresses.

- Case V - Broken waves to el. 18.8, wave force, wind from flood side. 33 1/3 percent increase in allowable stresses.

- Case VI - No water, no wave, wind from flood side, truck loading on edge of slab at protected side. 33 1/3 percent increase in allowable stresses.

- Case VII - No water, no wave, wind from protected side, truck loading on edge of slab at flood side. 33 1/3 percent increase in allowable stresses.

3.2.1.6.3.3.5. Citrus Back Levee (Reference 24)

General. Floodwalls are required in three locations along the Citrus Back Levee. Typically, these will consist of an I-type floodwall constructed on an enlarged levee cross-section to achieve the required gross grade elevation. Where space limitations preclude the construction of the embankment required with I-type floodwall, inverted T-walls supported by concrete bearing piles will be provided. In addition, three gates will be provided in the floodwall alignment passing through the NOPSI electric generating plant. Each gate will consist of a single leaf overhead roller gate riding on an I-beam suspended from a reinforced concrete beam supported by three concrete columns.

The basic data relevant to the design of the protective works are shown in the following tabulation:

Water Elevations	**Elevation (feet N.G.V.D.)**
Still Water Level	13.0
Landside of Wall	0.0
Unit Weights	**Lb. per cu ft**
Water	62.5
Concrete	150
Steel	490
Design Loads	**Lb. per cu ft**
Wind loads	
On Walls	50 p.s.f.
On Overhead Beams	30 p.s.f.

Allowable Working Stresses. The allowable working stresses for concrete and structural steel are in accordance with those recommended in “Working Stresses for Structural Design,” EM1110-1-2101 dated, 1 November 1963. The basic minimum 28-day compressive strength for concrete is 3,000 p.s.i., except for prestressed concrete piling where the minimum is 5, 000 p.s.i. Steel for sheet piling met the requirements of ASTM A328-54, “Standard Specification for Steel Sheet Piling.” Pertinent allowable stresses are tabulated below:

Reinforced Concrete	
f'_c	3,000 p.s.i.
f_c	1,050 p.s.i.
V_c (without web reinforcement)	60 p.s.i.
V_c (with web reinforcement)	274 p.s.i.
f_s	20,000p.s.i.
Minimum tensile steel	0.00025 bd sq. in.
Structural Steel (ASTM A-36)	
Basic working stress	18,000 p.s.i.

I-Wall Design. The I-wall was designed for two different conditions. The floodwall west of Paris Road was not considered subject to wave loads and was designed using a factor of safety in the soil of **1.5**, for a floodside water elevation of 14.5 and checked for water to the top of the wall at elevation 15.0. The I-wall east of Paris Road is designed for the following loading cases:

- Case I – Static water to top of broken wave (elevation 18.8), 1.5 factor of safety in the soil, no dynamic wave force

- Case II – Static water to top of broken wave, 1.25 factor of safety in the soil, dynamic wave load from broken wave

T-Wall Design. Inverted T-wall sections on concrete bearing pile foundations were designed for the following conditions:

- Case I – Water at elevation 15.0 on the floodside and water at elevation 5.5 on the protected side. Sheet pile cutoff pervious. Uplift varies by deceasing uniformly from full head uplift on floodside to tailwater uplift on protected side.

- Case II – Same as Case I except sheet pile cutoff impervious. Full head uplift on floodside of cutoff, and tailwater uplift on protected side of cutoff.

- Case III – Water at elevation 12.5 on floodside and water at elevation 5.5 on protected side. Sheet pile cutoff pervious. Uplift varies by decreasing uniformly from full head uplift on floodside to tailwater uplift on protected side.

- Case IV – Same as Case III except sheetpile cutoff impervious. Full head uplift on floodside of cutoff, and tailwater uplift on protected side of cutoff.

Gates. The loading cases used to design the gates are as follows:

- Case I – Water to elevation 18.8 on the floodside (top of broken wave), elevation 10.5 on protected side, no dynamic wave load, normal working stresses.

- Case II – Water to elevation 18.8 on floodside (top of broken wave), elevation 10.5 on the protected side, dynamic wave load, 1/3 increase in allowable working stresses. With the gates open, the base is designed to support an H-20 highway loading.

3.2.1.6.3.3.6. IHNC (Reference 9)

General. The structural features consist predominantly of a cantilever I-type floodwall of steel sheet piling driven through existing levees, and/or fill, and capped with a concrete wall. T-type floodwalls supported by bearing piles were provide the protection in the more congested areas in the vicinity of road and railroad crossings.

Basic Data. Maximum wind tide levels along the IHNC resulting from the design hurricane vary from elevation 11.4 at Seabrook to 12.9 at the L&N Railroad Bridge and then to 13.0 at the IHNC Lock. Water elevations landside of the floodwall vary from elevation zero to elevation -3.0. The elevation of the top of an I-wall in a levee are 2.0 feet above the wind tide level. The elevation of the top of T-type walls and gates are 1.0 foot above the wind tide level.

Unit Weights	**Lb. per cu ft**
Water	62.5
Concrete	150
Steel	490

Water Loads

- No wave forces will occur
- One foot freeboard

I-type Floodwall. Bending moments and deflections for structural design of sheet piles were based on a factor of safety of 1.5 applied to the soils. The strength of the wall was checked for the case with water at the top of the wall and found to be adequate.

Design of T-type wall for the East Levee. The T-type floodwalls were designed for the following typical conditions:

- Case 1 – Water at elevation 13.25 on floodside and at bottom of base (elevation 2.5) on protected side. Steel sheet pile cutoff at center of base and impervious. Uplift with full head on floodside of cutoff and tailwater on the protected side. No earth load.
- Case 2- Same as case 1 except steel sheet pile cutoff pervious. Uplift varies uniformly from full head on floodside to tailwater on the protected side.
- Case 3 – water at elevation 11.0 on the floodside and at the bottom of the base on the protected side. Impervious cutoff. Uplift as in Case 1. No earth load.
- Case 4- Same as Case 3 except cutoff pervious and uplift as in Case 2.
- Case 5 – Water at elevation 10.5 on the flood and at the bottom of the base on the protected side. Impervious cutoff. Uplift as in case 1. No earth load.

- Case 6 – Same as case 5 except cutoff pervious and uplift as in Case 2.

Allowable axial and transverse pile loads and the computed pile loads were obtained using Hrennikoff's method. In the determination of the allowable transverse pile loads, the soil was considered to have a constant modulus of subgrade reaction. (K) with depth.

3.2.1.6.3.4. Sources of Construction Materials

3.2.1.6.3.4.1. Sheet Pile. Generally, the sheet pile sections specified during advertisement were used for construction. However, sheet pile section substitutions conforming to the minimum required section modulus was allowed, primarily in contracts constructed after 1990. Below, is a table of sheet pile sections for New Orleans East, broken down by DM.

New Orleans East	
Citrus Lakefront Levee	
Jourdan Rd. to Lakefront Airport	PZ-27
Lincoln Beach	PZ-27
Paris Rd. to South Point	
Vicinity of Collins Pipeline	PZ-22
South Point to GIWW	
Railroad Swing Gate Tie-In	PZ-27
Drainage Structure	**
New Orleans East Back Levee	
Floodwall at Intracoastal Pump Station	PZ-27*
Citrus Back Levee	
Michoud Canal (Station 624+17 to 664+73)	PZ-27
Paris Rd. to NOPSI	PZ-27
* As-advertised ** Information not located at the time of publication	

3.2.1.6.3.4.2. Levee material - The descriptions of proposed levee construction materials in the following paragraphs were taken directly from the referenced DM's. Numerous times in the various DM's, reference is made to taking borrow material from a pit on the bottom of the north shore of Lake Pontchartrain. What actually happened according to personnel in LMVN is one contractor attempted to use the Howze Beach pit and it did not work. We were told that the borrow material specs on the levee construction for the Lake Pontchartrain and vicinity projects required CH, CL or ML classified by the Unified Soil Classification System and that it came from either a government furnished pit in the Bonnet Carré spillway or a contractor-furnished pit in New Orleans East known as the Highway 90 pit. Some borrow material for the levees in the New Orleans East projects may have also come from the Geohagan Canal near Slidell.

3.2.1.6.3.4.2.1. Source of Borrow Materials (Citrus Lakefront Levee). The levee will be constructed of semi-compacted clay fill which will be obtained from Borrow area of Pleistocene clays in the bottom of Lake Pontchartrain along the north slope.

3.2.1.6.3.4.2.2. Sources of Borrow Material (New Orleans East Lakefront Levee). The levee will be constructed of semi-compacted clay fill from a borrow area of Pleistocene clays in the bottom of Lake Pontchartrain along the north shore.

3.2.1.6.3.4.2.3. Sources of Construction Materials (South Point to GIWW). Borrow material for the levees is available in a borrow pit located in the Bonnet Carré Spillway.

3.2.1.6.3.4.2.4. Source of Fill Materials (New Orleans East Back Levee). The levees will be built of hydraulic fill from adjacent GIWW and Michaud Canal. In order to be utilized, the maximum amount of Pleistocene materials borrow will come from the deepest parts of the borrow pits. The material for construction of the levee in the floodwall areas will come from a borrow pit in the bottom of Lake Pontchartrain along the north shore.

3.2.1.6.3.4.2.5. Source of Fill Materials (Citrus Back Levee). The fill for completing the levee portion of the project will come from adjacent borrow and if required from a borrow area in the bottom of Lake Pontchartrain along the north shore.

3.2.1.6.3.4.2.6. Source of Fill Materials (IHNC Floodwall/Levee). The earth fill for completing the levee portions of the protection will be obtained from excess material cut from the reshaped existing levees, and from a borrow area in Lake Pontchartrain on the north shore.

3.2.1.6.4. As-built Conditions

3.2.1.6.4.1. Changes between design and construction (i.e. cross sections, alignment, sheet pile tip el, levee crest el.)

3.2.1.6.4.1.1. DACW29-68-B-0148. Lake Pontchartrain, Louisiana and Vicinity, Lake Pontchartrain Barrier Plan, Orleans Parish, LA., Inner Harbor Navigation Canal, East Levee, Hayne Blvd. To Dwyer Road (Station 33 + 95 to Station 83 + 00) Plans for Levee and Floodwall Capping

Reviewed As Builts, No Applicable Modifications or Changes Found

3.2.1.6.4.1.2. DACW29-83-R-0056. IHNC - East and West Levee and Citrus Back Levee Capping Floodwalls, Paris Rd. through N.O.P.S.I., Orleans Parish, LA.

Reviewed As Builts, No Applicable Modifications or Changes Found

3.2.1.6.4.1.3. DACW29-93-C-0096. Reprocurement of Lake Pontchartrain High Level Plan, New Orleans East Levee, south Point to GIWW, Orleans Parish, LA

Part of the work required on this contract was constructing a flood side berm on the existing levee. After constructing a small portion of it in accordance with the plans, apparently the berm slid in places and was cracking. A modification was issued to change the configuration which lowered the top elevation of the berm and made it wider in order to keep it from sliding. They were in fear of losing the whole berm if they didn't do something. This mod was to change the configuration of the flood side berm between Stations 404 + 79.23 and 437 + 00.00 to prevent its eventual failure. This resulted in a decrease in the quantity of semi-compacted material to be placed, which resulted in a credit to the government.

3.2.1.6.4.1.4. DACW29-98-C-0002. Lake Pontchartrain, Louisiana and Vicinity, Hurricane Protection Project, High Level Plan, Orleans Parish, Lakefront Airport, South Airport Floodwall Modifications, Orleans Parish, LA

Reviewed As Builts, No Applicable Modifications or Changes Found

Reviewed Narrative Completion Report and Modification Documents, no applicable modifications or changes found.

3.2.1.6.4.1.5. DACW29-89-C-0134. Lake Pontchartrain High Level Plan, New Orleans East Levee, South Point to GIWW, Orleans Parish, Louisiana

3.2.1.5.6.1.6. DACW29-96-C-0080. Lake Pontchartrain, Louisiana and Vicinity, Hurricane Protection, Orleans Marina Floodwall **–** Phase IV, Orleans Parish, LA

Reviewed Completion Report, no applicable modifications or changes found.

3.2.1.5.6.1.7. DACW29-97-C-0066. Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, Orleans Parish Lakefront Levee/Floodwall, Pontchartrain Beach Wave Berm, Station 10+03.45 to Station 39+78.39 W/L, Orleans Parish, LA

Reviewed Narrative Completion Report, no applicable modifications found; however, contractor provided and used an alternate borrow pit which was material from a city owned (Slidell) detention pond.

3.2.1.6.4.2. Inspection during original construction, QA/QC, state what records are available. See paragraph 3.2.1.5.4.2. New Orleans East Bank for description of how records are kept.

3.2.1.6.4.2.1. DACW29-89-C-0134 – SOUTH POINT TO GIWW, ORL PAR

Attached to QA/QC Reports are moisture test records and records of preparatory inspections/meetings for identifiable features of work.

3.2.1.6.4.2.2. DACW29-98-C-0002 – L PONT AIRPORT FLOODWALL MODS, ORL PAR

Attached are records of preparatory meetings, concrete sampling and testing reports, and form checkout sheets for reinforced concrete floodwalls.

3.2.1.6.5. Inspection and maintenance of original construction - Inspections of Civil Works projects in the New Orleans district fall primarily under two programs, not including local sponsor inspections:

Periodic Inspections - Inspections of Federal Civil Works structures, owned and operated by the federal government, are done under the Periodic Inspection Program, as defined by **ER 1130–2–100**, entitled *Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures.* Bridges are inspected under **ER 1110-2-111**, entitled, *Periodic Safety Inspection and Continuing Evaluation of USACE Bridges.* These inspections are funded by the appropriate projects under Construction, General (CG) appropriation, during the final phases of construction, and Operations and Maintenance (O&M), General (O&M,G) appropriation during the O&M phase.

Annual Compliance Inspections - Certain provisions for these inspections are codified under 33 CFR 208.10. Inspections of federal flood control projects, operated and maintained by non-federal sponsors, are inspected under the Inspection of Completed Works program, under **ER 1130–2–530**, entitled *Flood Control Operations and Maintenance Policies,* dated October 30, 1996. (This engineering regulation supersedes the previous regulation **ER 1130–2–339**, entitled *Inspection of Local Flood Protection Projects.* These projects are funded by the Inspection of Completed Works Project, under both the (O&M,G) and the Flood Control, Mississippi River and Tributaries (FCMR&T) appropriations.

3.2.1.6.5.1. Annual Compliance Inspections - Annual inspections were conducted by Operations Division for projects under the Inspection of Completed Works Project for the New Orleans East polder which is a part of the Lake Pontchartrain and Vicinity Hurricane Protection Project. These inspections, which were general in nature, primarily defined the status of existing project work, and a general condition rating.

For the last 6 years, 1998 through 2004, the ratings for the Orleans Levee District, which includes the New Orleans East polder, were “OUTSTANDING” through year 2001, and “ACCEPTABLE” each year thereafter, at which time there was a change in the Project Rating Scale. The project rating scale was then redefined, and “ACCEPTABLE” became the highest rating.

There was no specific mention of deficiencies for the hurricane protection system. (i.e., trees, etc.).

3.2.1.6.5.2. Periodic Inspections – There are no structures under the Periodic Inspection Program in this polder.

3.2.1.6.6. Other Features

3.2.1.6.6.1. Brief Description. The primary components of the hurricane protection system for the New Orleans East basin are described above, namely the levees and floodwalls designed and constructed by the Corps of Engineers. However, other drainage and flood control features that work in concert with the Corps of Engineers levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section will describe and present the criteria and pre-Katrina conditions of the interior drainage system, pump stations, and the one non-Corps levee. Even though the stormwater pump stations are part of the interior drainage system, they are a significant part of the system and warrant their own section.

3.2.1.6.6.2. Pre-Katrina Conditions. According to the local jurisdictions responsible for interior drainage, the storm drain system, interior canals, interior pump stations, outfall pump (lift) stations, and outfall canals were in good condition and prepared for high inflows from rainfall prior to August 29, 2005.

The pre-Katrina condition of the non-Corps levee on the eastern limits of the urban area adjacent to the Bayou Sauvage National Wildlife Refuge was not obtained by the IPET team.

3.2.1.6.6.3. Interior Drainage System.

Overview. The developed area of the New Orleans East basin is 32 square miles and the undeveloped area is 22 square miles. The land generally slopes south to north from the Intracoastal Waterway to Lake Pontchartrain. It is primarily a fully developed residential area north of the Chef Menteur Highway and a nearly fully developed industrial area south of the highway. Many features are typical of large urban cities in the United States, and some features that are unique because much of the area is below sea level. Catch basins and inlets collect surface runoff from yards and streets into storm sewers. Excess runoff flows down streets and/or overland to lower areas. Open canals collect the stormwater and carry it to outfall pump stations that pump directly into Lake Pontchartrain, the Inner Harbor Navigation Canal, or the Intracoastal Waterway.

The entity responsible for local drainage in the Orleans East Bank basin is the Sewerage and Water Board of New Orleans. In addition to local drainage, they also provide potable water and sanitary sewerage service. The Louisiana Department of Transportation and Development highways are also a significant part of the local drainage system.

System Components. Local drainage begins with overland flow which follows the ground topography. Figure 5 in Volume VI shows the topographic layout of New Orleans East. The land generally falls from the Intracoastal Waterway to Lake Pontchartrain. A land feature visible on the topographic layout that affects the local drainage is the Gentilly Ridge. It runs east-west between the Intracoastal Waterway and Lake Pontchartrain. The Chef Menteur Highway is built on the ridge.

The local drainage is collected by underground storm drains and roadside ditches which carry the water to the canals. Photos 1 and 2 show typical inlets and streets. Photo 3 shows a typical storm sewer outfall into a canal.

The land topography and development sequence influenced the storm sewer, canal, and pump station layout. Based on land topography and the drainage system, the basin is divided into 62 subbasins. Pump station information is presented in Section 3.2.1.6.6.4 of this volume.

Photos 1 and 2. Typical Streets and Inlet – New Orleans East

Photo 3. Storm Sewer Outfall into Dwyer Canal

The primary interior canals are open and either concrete-lined or grass-lined (Photos 4, 5, and 6). The interior canals not only collect stormwater from streets and storm sewers and covey it to the pump stations, they also are storage areas that work in conjunction with the pump stations. Because of their size, they have a considerable storage volume compared to Orleans East Bank.

Photo 4. St. Charles Canal from Dwyer Road

Photo 5. Benson Canal from Dwyer Road

Photo 6. Dwyer Canal near Crowder Blvd.

Design Criteria. The current design criterion for new storm drainage facilities in New Orleans East is the 10 % probability (10 year frequency). The capacity of the older parts of the storm drain system is not known since improvements were made over many years. The functional capacity of the interior canals and pump stations is a little less than 0.5 inches per hour. However, the level of protection is similar to Orleans East Bank because of the additional storage in the open canals. Rainfall in excess of this amount goes into temporary storage in the canals, storm sewers, open areas, and streets. There are no criteria for redevelopments to use stormwater detention because the impervious cover wouldn't change significantly and delaying runoff to an outfall pump is counter productive.

Where local drainage is considered poor, the Sewerage and Water Board is working to improve the drainage. In some cases, the Sewerage and Water Board and Corps of Engineers are working together on projects, as presented below in the Southeast Louisiana (SELA) Urban Flood Control Projects section.

Southeast Louisiana Urban Flood Control Projects. As a result of the extensive flooding in May 1995, Congress authorized the SELA Urban Flood Control Project with enactment of the Energy and Water Development Appropriations Act for Fiscal Year 1996 and the Water Resources Development Act (WRDA) of 1996 to provide for flood control and improvements to rainfall drainage systems in Jefferson, Orleans, and St. Tammany Parishes. The Sewerage and Water Board of New Orleans is the local, cost sharing sponsor for the Orleans Parish work.

The project includes channel and pump station improvements in the three parishes. The channel and pumping station improvements in Orleans and Jefferson Parishes support the parishes' master drainage plans and generally provide flood protection on a level associated with a 10-year rainfall event, while also reducing damages for larger events.

One of the areas in Orleans Parish is in the New Orleans East basin. It is in the Dwyer Road area and is shown in Figure 12. The work consists of additional enclosed canal capacity along Dwyer Road from the Dwyer Road Pumping Station to the St. Charles Canal, replacement of the existing Dwyer Road Pumping Station, and an outfall canal (enclosed and open) into the Inner Harbor Navigation Canal. Prior to Hurricane Katrina, the outfall canal was complete, the pump station was under construction, and the Dwyer Road canal was not started.

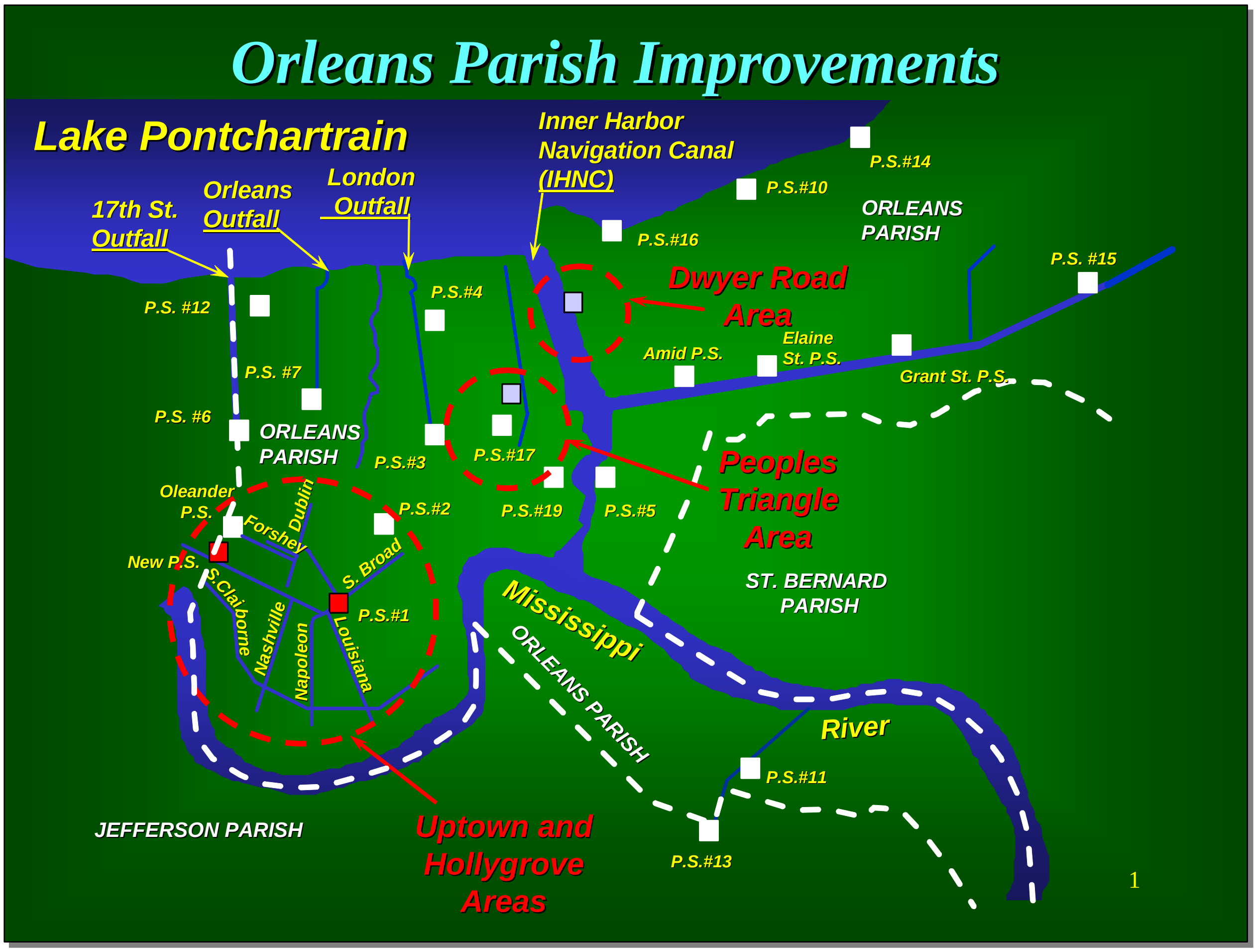


Figure 12. SELA Urban Flood Control Projects in New Orleans East

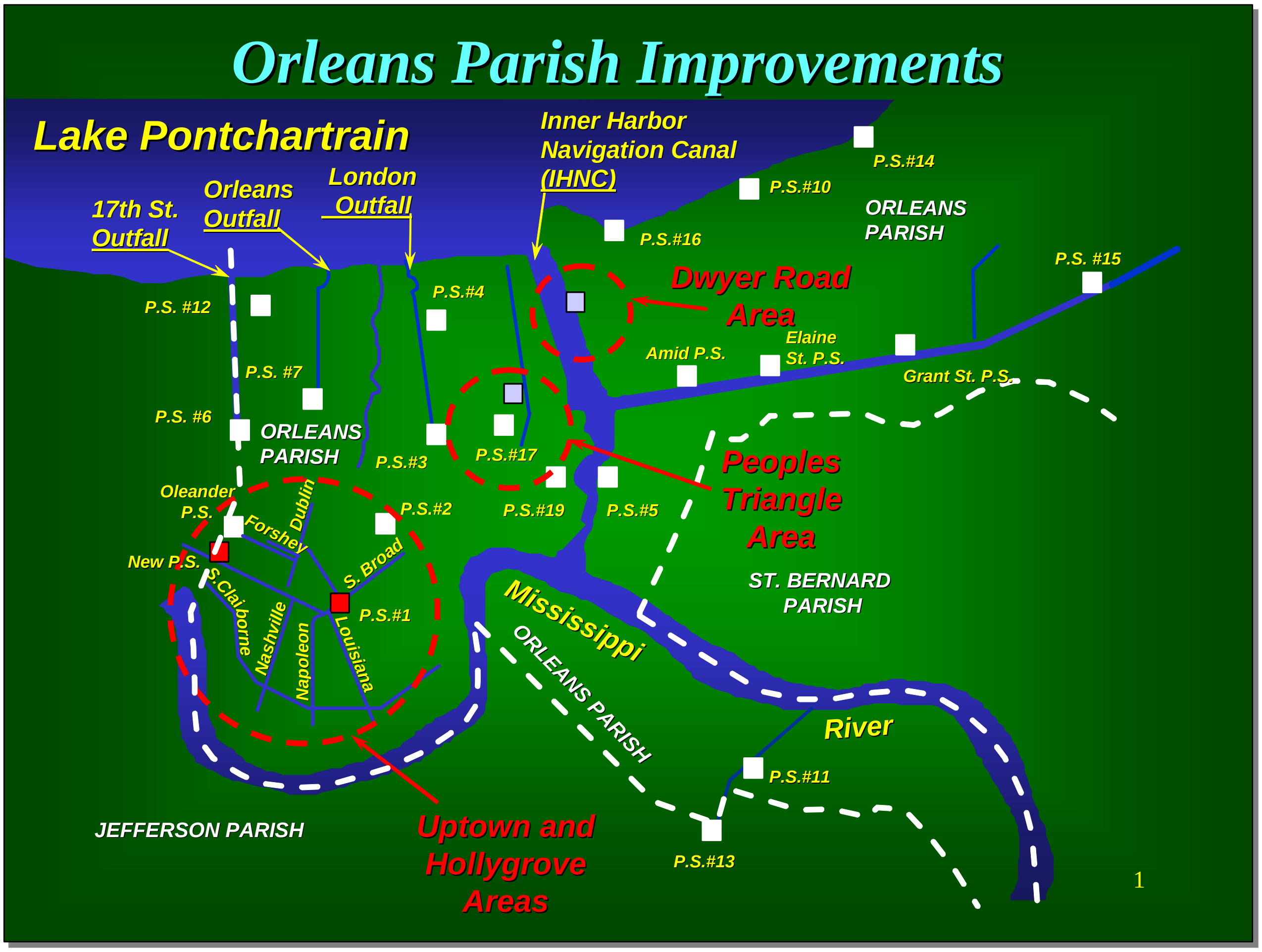


Figure 13. SELA Urban Flood Control Projects in New Orleans East

3.2.1.6.6.4. Pumping stations – Orleans Parish Summary. Figure 14 is a map showing the Orleans Parish pump stations that were used in this report. The locations of the pump stations were verified by Global Positioning System (GPS) and/or by using Google Earth Pro. The GPS coordinates were then input into Microsoft Streets and Trips (shown below).

Table 12 contains a summary of pump stations by drainage basin in Orleans Parish. The list is composed of information that was collected in the field. Not all information was available for each pump and was left blank or highlighted.

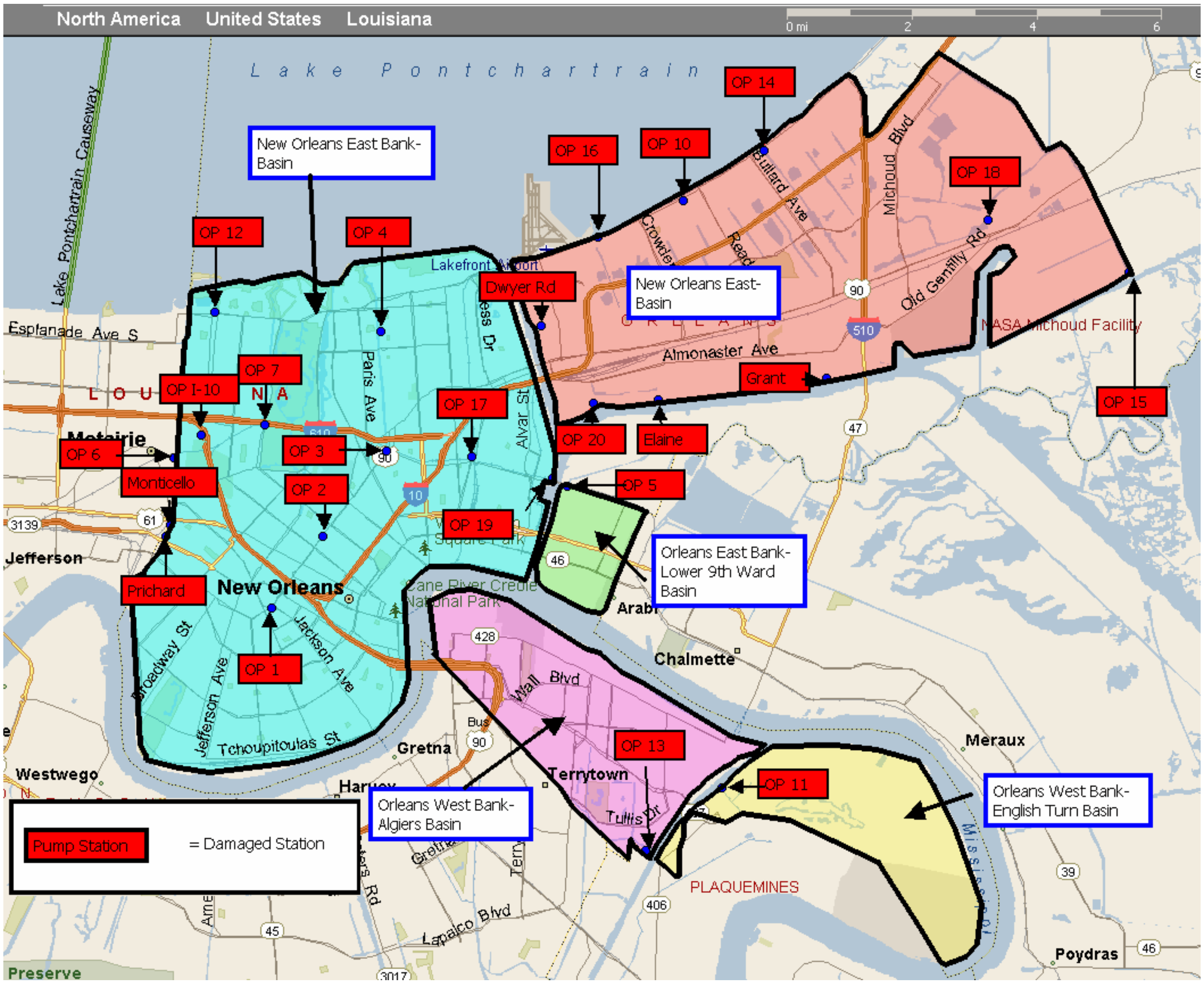


Figure 14. Orleans Parish Pump Station Locations

Table 12
Summary of Orleans Parish Pump Stations by Drainage Basin

Basin	East Bank	East	East Bank-Lower 9th Ward	West Bank-Algiers	West Bank-English Turn	Total
Number of pump stations	12	9	1	1	1	24
Number of pumps	68	24	7	7	5	111
Total rated capacity (cfs)	36,615	4,852	1,850	4,700	1,690	49,707
Estimated cost of damages	n/a	n/a	n/a	n/a	n/a	n/a

Drainage Basins. Orleans Parish consists of five drainage basins. The majority of the pump stations are in the East Bank and East basins. The Lower Ninth Ward, Algiers, and English Turn Basins have one pump station each. The Orleans Parish pump stations are listed below under their appropriate basins. Details for each pump station are listed in Volume VI.

Orleans East

The East Drainage Basin consists of eight pump stations, and a ninth station (Dwyer Street) is being built. It is bordered by Lake Pontchartrain on the north, the Intracoastal Waterway on the South, and the IHNC on the west. Its drainage system includes the surrounding bodies of water, as well as the Citrus, Morrison, Jahncke, St. Charles, Amid, Grant St., Elaine St., and Maxent Canals, and the Village de'l East Lagoon. Below is a brief summary of each of the 9 pump stations. Volume VI provides more detailed information.

OP 10 – Citrus
Intake location: .... Citrus Canal
Discharge location: .... Lake Pontchartrain
Nominal capacity: .... 1000 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
1	250	1984	Electric 60 Hz	Vertical
2	250	1984	Electric 60 Hz	Vertical
3	250	1984	Electric 60 Hz	Vertical
4	250	1984	Electric 60 Hz	Vertical

OP 14 – Jahncke
Intake location: .... Morrison and Jahncke Canals
Discharge location: .... Lake Pontchartrain
Nominal capacity: .... 1200 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
1	300	n/a	Electric 60 Hz	Vertical
2	300	n/a	Electric 60 Hz	Vertical
3	300	n/a	Electric 60 Hz	Vertical
4	300	n/a	Electric 60 Hz	Vertical

OP 16 – St. Charles
Intake location: ....St. Charles Canal
Discharge location: ....Lake Pontchartrain
Nominal capacity: ....1000 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
1	250	1966	Electric 60 Hz	Vertical
2	250	1966	Electric 60 Hz	Vertical
3	250	1966	Electric 60 Hz	Vertical
4	250	1966	Electric 60 Hz	Vertical

OP 18 – Maxent
Intake location: .... Village de'l East Lagoon
Discharge location: ....Maxent Canal
Nominal capacity: ....60 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
1	30	1983	Electric 60 Hz	Vertical
2	30	1983	Electric 60 Hz	Vertical

OP 20 – Amid
Intake location: .... Amid Canal
Discharge location: .... Intracoastal Waterway
Nominal capacity: ....500 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
1	250	1989	Electric 60 Hz	Vertical
2	250	1989	Electric 60 Hz	Vertical

Grant St

Intake location: ....Grant Street Canal
Discharge location: .... Intracoastal Waterway
Nominal capacity: ....192 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
1	8	n/a	Electric 60 Hz	Vertical
2	8	n/a	Electric 60 Hz	Vertical
3	8	n/a	Electric 60 Hz	Vertical
4	8	n/a	Electric 60 Hz	Vertical
5	80	1990	Electric 60 Hz	Vertical
6	80	1990	Electric 60 Hz	Vertical

Elaine St

Intake location: ....Elaine Street Canal
Discharge location: .... Intracoastal Waterway
Nominal capacity: ....90 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
1	45	1975	Electric 60 Hz	Vertical
2	45	1975	Electric 60 Hz	Vertical

OP 15

Intake location: ....Maxent Canal
Discharge location: .... Intracoastal Waterway
Nominal capacity: ....750 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
1	250	n/a	Electric 60 Hz	Vertical
2	250	1997	Diesel	Vertical
3	250	1997	Diesel	Vertical

DWYER

Intake location: ....

Discharge location: ....Inner Harbor Navigation Channel

Nominal capacity: .... cfs

Pump	Capacity (cfs)	Installe d (year)	Driver Electric /Diesel	Pump Configuration
1	0			Vertical
2	0			Vertical
3	0			Vertical

3.2.1.6.6.5. Levees and floodwalls

3.2.1.6.6.5.1. MRL. There are no MRL levees and floodwalls as a part of the New Orleans East Project.

3.2.1.6.6.5.2. Non Corps. Several local interest and/or private levees are located within the project area. No design criteria for these levees have been made available to the Corps

3.2.1.7. St. Bernard Introduction

St. Bernard Parish Basin. The St. Bernard Basin hurricane protection system includes the levee/floodwall extending from the Inner Harbor Navigation Channel (IHNC) easterly, along the Gulf Intracoastal Waterway (GIWW), to the Bayou Bienvenue Control Structure, continuing along the Mississippi River Gulf Outlet (MRGO) southeasterly, then turns generally to the west, where it ties into the Mississippi River Levee at Caernarvon, as shown on the map below. A portion of the hurricane protection system in this area also provides hurricane protection to the Lower 9th Ward area in Orleans Parish.

The pertinent data for the Chalmette area plan (Orleans and St. Bernard's Parishes) was 1.51 miles of floodwall along the IHNC and 19.95 miles of levee which extend to the lower end of St. Bernard Parish. Also included in the plan are the Bayou Bienvenue and Bayou Dupre structures.

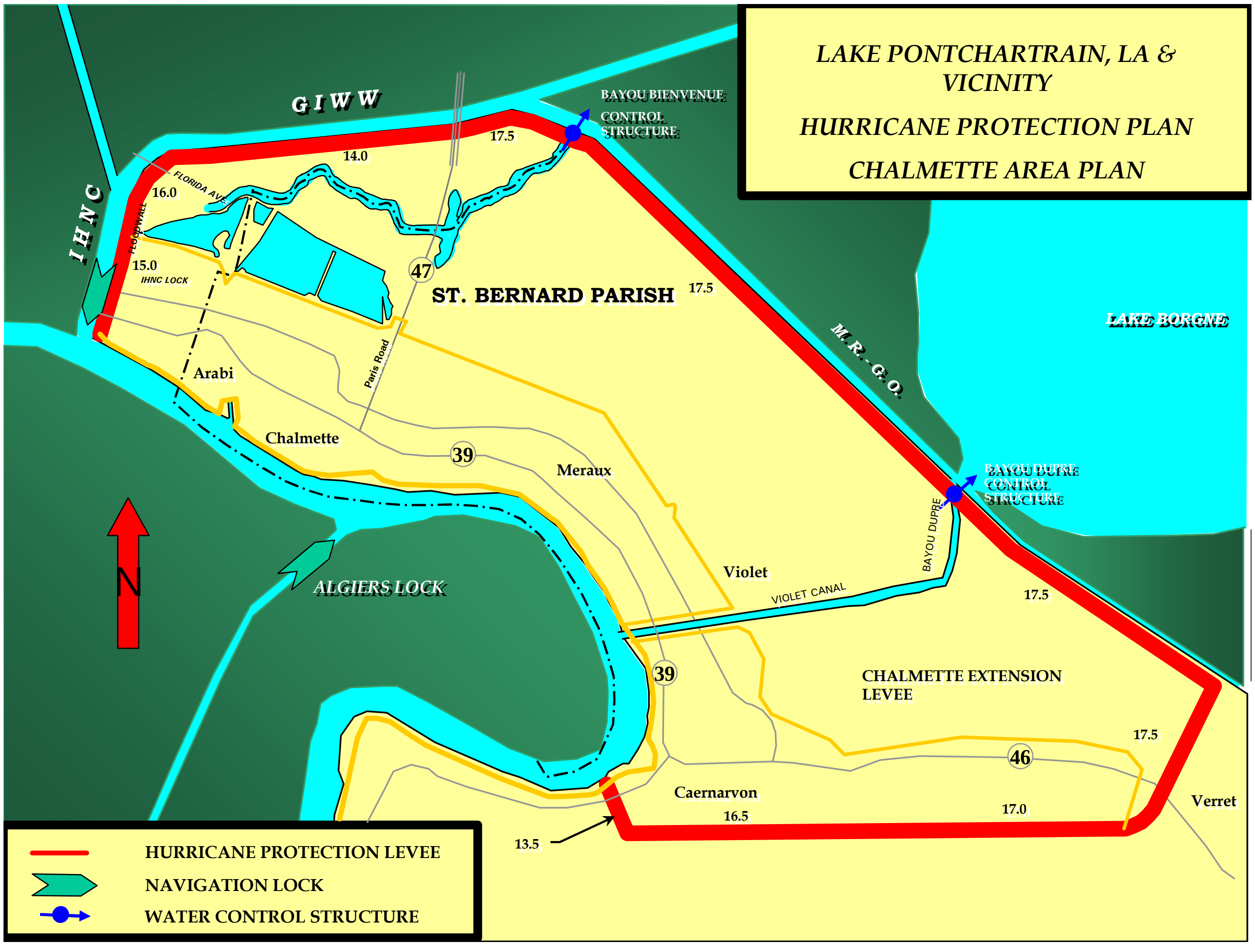


Figure 15. Hurricane Protection Project St. Bernard Perish

Table 13 **Summary of St. Bernard Basin Hurricane Protection Features**	
Levees and Floodwalls	157,800 ft
Road Closure Structures	6
Water Control Structures	2
Gravity Drainage Structure	1

3.2.1.7.1. Pre-Katrina - The St. Bernard Parish portion of the Lake Pontchartrain and Vicinity project is under construction. As of August 29, 2005, the remaining work consisted of the following:

- A levee enlargement between the Bayou Bienvenue and Bayou Dupre Structures
- A levee enlargement between Verret and Caernarvon
- A levee enlargement in the Orleans Parish portion of the Chalmette area plan. That levee was located between Paris Road and the IHNC.

Preparation of plans and specifications had begun prior to the storm but had been halted due to lack of funding. Because of damages due to Hurricane Katrina, the Corps through its Task Force Guardian has constructed the levee enlargement between the Bayou Bienvenue and Bayou Dupre Structures. Plans are being developed to construct the Verret to Caernarvon reach. A review is underway to determine if the levees, floodwalls and structures will have to be redesigned based on the results of the Interagency Performance Evaluation Team analysis and based on a reanalysis of design storm calculations. Additional contracts may be required as a result of this analysis.

3.2.1.7.2. Design Criteria and Assumptions - Functional design criteria

3.2.1.7.2.1. Hydrology and Hydraulics.

For St. Bernard, the design hurricane characteristics utilized in the design memoranda are shown in Table 14; the design tracks are shown on Figure 16. The maximum wind speed was computed using the same equations as for Orleans East Bank. For each project area, the track and forward speed were selected to produce maximum wind tide levels.

Table 14
Design Hurricane Characteristics

Location	Track	CPI, Inches	Radius of Maximum Winds, Nautical miles	Forward Speed, Knots	Maximum Wind Speed[1], MPH	Direction of Approach
Chalmette Area and Extension along the MRGO	F	27.6	30	11	100	East
IHNC East	F	27.6	30	11	100	East
Chalmette Extension	C	27.6	30	5	100	SSE

[1] Windspeeds represent a 5 minute average 30 feet above ground level.

3.2.1.7.2.1.1. Surge. For Chalmette Area, IHNC East, and Chalmette Extension along the MRGO, surge elevations were computed using the same methodology as used for IHNC for Orleans East Bank.

For the Verret and Toca reach of the Chalmette Extension, surge elevations were computed using the same methodology as used for IHNC for Orleans East Bank, with an additional step. For the purpose of surge routing, maximum surge heights would be observed along a line representing the coastline, called the surge reference line. Marshlands that fringe the study area would be inundated for considerable distances inland of this surge reference line. A study of available observed high water marks, at the coastline and inland, indicated a consistent simple relation between the maximum surge height and the distance inland from the coast (Figure 17). This relationship was considered independent of hurricane forward speed, windspeed, or direction. The data indicated that the weighted mean decrease in surge heights inland would be at the rate of 1.0 ft per 2.75 miles. For the Verret and Toca reaches, the maximum surge height at the surge reference line was computed, then reduced to obtain the surge height at the inland locations. Table 15 shows the wind tide levels at the surge reference line and at the levee location.

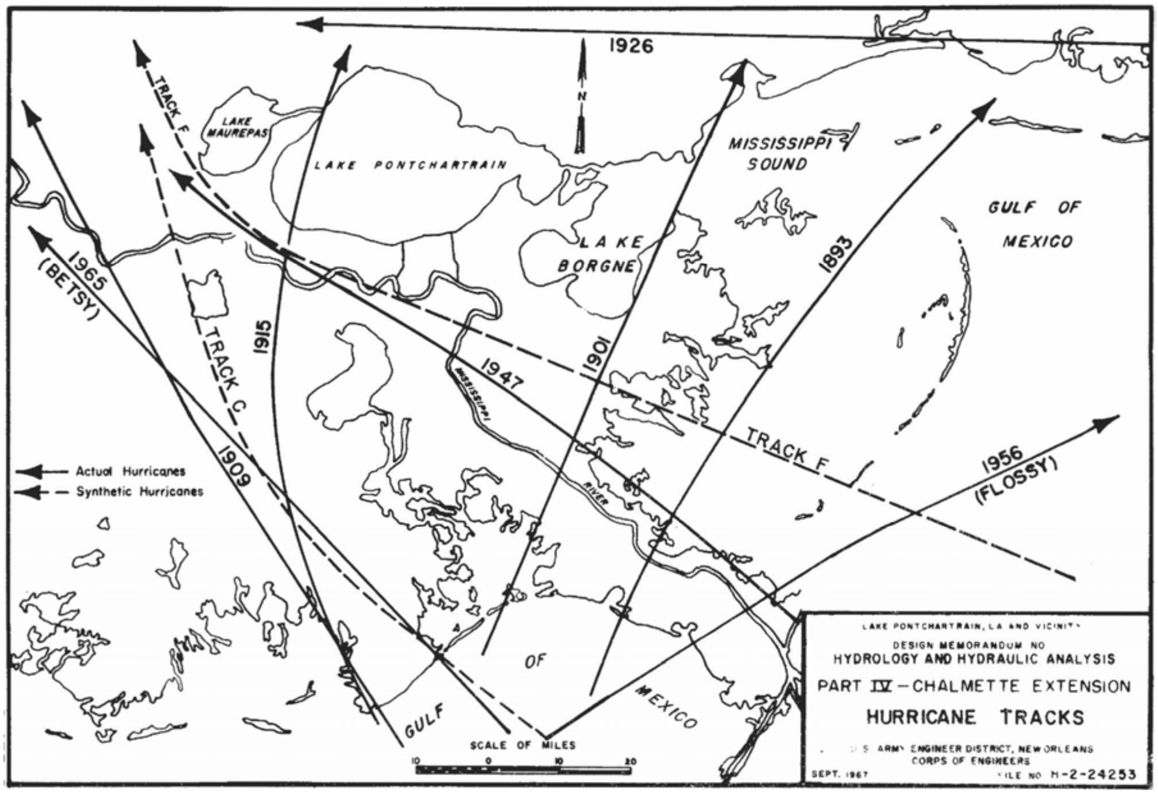


Figure 16. St. Bernard Hurricane Tracks

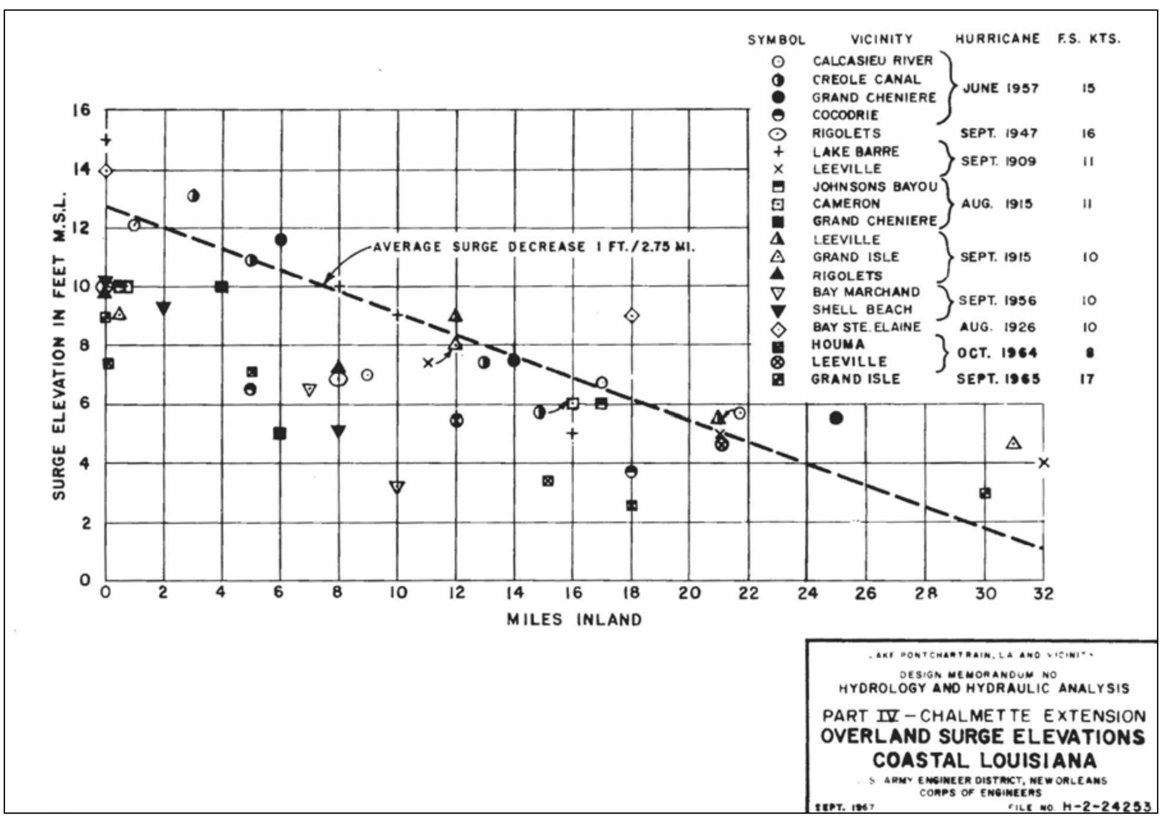


Figure 17. Overland Surge Elevations

Table 15 Wind Tide Levels			
Location	**Surge Adjustment Factor, Z**	**Wind Tide Level, Surge Reference Line, FT MSL**	**Wind Tide Level at Levee Location, FT MSL**
Verret	0.48	15.1	12.2
Toca	0.52	15.8	11.8

3.2.1.7.2.1.2. Waves. Wave runup was calculated using the methodology described in Orleans East Bank. Along the IHNC and the portion of the Chalmette Area west of Paris Road to IHNC, waves were not considered a factor.

3.2.1.7.2.1.3. Summary. Table 16 contains maximum surge or wind tide level, wave, and design elevation information.

Table 16 Wave Runup and Design Elevations (transition zones not tabulated – governing DM is listed)								
Location	**DM**	**Average Depth of fetch, ft**	**Significant Wave Height Hs, ft**	**Wave Period, T, sec**	**Maximum Surge or Wind Tide Level, ft**	**Runup Height, ft**	**Freeboard, ft**	**Design Elevation Protective Structure, ft**
IHNC L&N Railroad Bridge to Mississippi River	DM1, Part 1, Aug 1966	-	-	-	12.9 – 13.0 MSL	-	1.0	14.0 MSL
Chalmette West of Paris Road	DM1, Part 1, Aug 1966	-	-	-	13.0 MSL	-	1.0	14.0 MSL
Chalmette East of Paris Road to Bayou Lawler	DM1, Part 1, Aug 1966	16.3	7.0	6.4	13.0-12.5 MSL	4.7	-	17.5 MSL
Chalmette Bayou Lawler to Violet	DM1, Part 1, Aug 1966	9.7	4.6	5.2	12.5-13.0 MSL	4.3	-	17.5 MSL
Chalmette Extension Bayou Dupre to Verret	DM1, Part 4, Aug 1966	16.3	6.6	6.2	12.5 MSL	4.6	-	17.5 MSL
Chalmette Extension Verret to Toca	DM1, Part 4, Aug 1966	10.1	4.4	5.1	12.2 MSL	4.8	-	17.5 – 16.5 MSL
Chalmette Extension Toca to Caernarvon	DM1, Part 4, Aug 1966	9.7	4.5	5.1	11.8 MSL	4.4	-	16.5 MSL

3.2.1.7.2.2. Geotechnical

3.2.1.7.2.2.1. Chalmette Area Levees and Floodwall. For the Chalmette Area levees and floodwalls, the general design is included in Reference 42. The detailed design for Bayou Bienvenue and Bayou Dupree Structures are included in Reference 43. Additional soils reports and analyses were done to supplement the data and analyses for the Chalmette Area Plan. Reference 63 dated 1984 covers the 2nd, 3rd and final lifts for the Paris Road to Bayou Bienvenue segments (Station 277+75 to 359+00). Reference 63 was not a formal report, but consisted of in house working notes, comps and sketches. The results were transmitted from Chief of Foundations and Materials Branch to Chief of Design Branch in two memoranda dated 22nd April 1991 and 4th February 1991. Reference 64 dated 2001 covers the Bayou Bienvenue to Bayou Dupree levee (Stations 359+00 to 740+00). This report covers the soils, foundation investigation and conditions and the design for raising the subject levee. For this contract the earthen levee section was to be constructed to El. 20.0 feet. The Standard Project Hurricane (SPH) elevation for the levee between Bienvenue and Bayou Dupree was elevation 17.5 feet. Reference 67 was published in 1982 and includes additional soils data and analysis for Station 208+93 to Station 945+00. Reference 68 is a soils report that covers the extreme lower end of the Chalmette area plan (Verret to Caernarvon levee). This levee ties to the main line Mississippi River levee.

3.2.1.7.2.2.1.1. Geology. The Chalmette Area project is located within the Central Gulf Coastal Plain. Specifically, the area was located on the eastern flank of the Mississippi River Deltaic Plain. The dominant physiographic features are swamps, marshes, natural levees and abandoned distributaries. Elevations of about 4 feet are found at the southern end at the distal edge of the slope of the natural levee of the Mississippi River near the IHNC Lock. Minimum elevations of -2 feet are found in the area near 90+25 = 7+52.9. The Chalmette area slopes from the alluvial ridge along the Mississippi River to the Lake Borgne Basin, which was a part of the Lake Pontchartrain Basin. The land adjacent to the Mississippi River ranges in Elev. from 4 to 10 feet and slopes away from the river at about 1 foot per 1,000 feet. The area adjacent to the Mississippi River, comprising about 10,000 acres, is presently protected from tidal inundation by back levees approximately paralleling the Mississippi River levee. The central part of the Chalmette area, comprising about 13,200 acres of marsh land, was only a foot or two above sea level and was subject to the-tidal flooding from the MR-GO through connecting canals and bayous. The balance of the Chalmette area along the MR-GO and bounded by Bayou Bienvenue and a line about 4,000 feet landward of the MR-GO right-of-way had been filled hydraulically to elevations 4 feet to 10 feet with material excavated from the MR-GO Channel.

3.2.1.7.2.2.1.2. Foundation Conditions. From the IHNC Lock to Station 90+25 = 7+52.9, the subsurface consists of Recent deposits varying in thickness from about 63 feet at the northeastern end of this portion of the project to about 75 feet at the southern or IHNC Lock end. Underlying the Recent are deposits of Pleistocene Age (Prairie Formation). The Recent consists generally of a 4- to 14-foot stratum of very soft organic clays underlying 7 to 20 feet of fill material except between the IHNC Lock and Station 30 + 00 where a thin layer of natural levee material 5 to 10 feet thick underlies the fill material. Underlying the marsh deposits is a 20- to 35-foot stratum of very soft to soft interdistributary clays containing lenses of silt and silty sand. From the IHNC Lock to approximately Station 64+92, a 20- to 40-foot layer of estuarine deposits, consisting generally of soft to medium clays with silt and sand lenses and shell fragments, directly underlies the interdistributary clays. From Station 50+00 to Station 64+92, a wedge of sand 7 to 15 feet thick, exists within the estuarine deposits. Along the Outfall Canal from Station 74+85 to Station 90+25, the estuarine deposits grade into near-shore gulf sands containing shell sand shell fragments. The estuarine and near-shore gulf lie directly over the stiff Pleistocene. From Station 90+25 = 7+52.9 to the end of the project at Station 1050 + 57.7, the soft Recent soils overlying the stiffer Pleistocene clays which occur from Elev. -55 to Elev. -65 consist generally of organic clays, peat, fat clays, some lean clays, some clayey sands, and rare spots of sand. Along the project alignment paralleling the MR-GO., the natural Recent soils have been covered with hydraulic spoil from the excavation of the MR-GO Channel.

3.2.1.7.2.2.1.3.1. Preliminary Field Exploration. (Ref 42) From Station 1+82 to Station 0+00 = 1+46.6 to Station 90+25 = 7+52.9, five 5-inch diameter undisturbed soil borings and sixteen 1 and 7/8 inch I.D. core barrel general type borings were made at intervals varying from about 200 to 1,000 feet along this project location. The borings were made through the existing levee and at the toe of the levee at selected locations, and extended to elevations -40 feet and -88 feet. From Station 90+25 = 7+52.9 to the project end at Station 1050+57.7, 103 3-inch ID core barrel general type soil test borings extending to a depth of 60 feet below existing ground surface were made at l,000 foot intervals along the proposed levee location. In addition to these 60-foot borings, two 5-inch diameter undisturbed type soil test borings 100 feet deep were made along the levee alignment; one-in the section adjacent-to-the MR-GO and one along the Bayou Dupre-Violet alignment. Two additional 5-inch diameter undisturbed type soil test borings were made, one at the- Bayou Bienvenue control structure site and one at the Bayou Dupre control structure site. The following field exploration paragraphs describe the additional borings taken for additional data

3.2.1.7.2.2.1.3.2. Field Exploration Bayou Bienvenue to Bayou Dupre (Ref 64). A total of 22 undisturbed type borings were made along the levee alignment between 1976 and 2001 for various purposes.

Four undisturbed soil borings were drilled for the current project in January and February 2001. These borings were drilled to a depth of 90 feet and tested through an A/E/ contract. Nine borings were made in 1976, two borings in 1986, and one in 1991. The 1984, 1986 and 1991 borings were drilled to depths ranging from 60 ft to 70 ft and were tested by the COE.

3.2.1.7.2.2.1.3.3. Field Exploration Chalmette Extension (Ref 67). Additional undisturbed borings were taken and tested by the Corps of Engineers along the centerline and 150 feet landside from centerline. Five borings were mentioned in Reference No. 67. No mention was made of other borings, or borings that may have been made during previous investigations.

3.2.1.7.2.2.1.3.4. Field Investigation Verret to Caernarvon Levee (Ref 68). A total of 30 undisturbed soil borings were taken along the levee alignment between 1967 and 2000 for various purposes. Eight (8) undisturbed soil borings were drilled for this project in April and May 2000. The soil borings were taken by the Corps of Engineers, New Orleans District. The centerline borings and the toe borings were taken to depths of 70 feet and 55 feet below the existing ground surface, respectively, and tested through an A/E contract. The other 22 undisturbed soil borings were taken during previous studies and were made by and tested by the Corps of Engineers, New Orleans District.

3.2.1.7.2.2.1.4. Underseepage. Based on the soil conditions along this part of the project and the short duration of hurricane floods, hazardous seepage or hydrostatic uplifts on the protected side was not anticipated.

3.2.1.7.2.2.1.5. Pile Foundation. Will be addressed in the Bayou Bienvenue and Bayou Dupre paragraphs**.**

3.2.1.7.2.2.1.6. Slope Stability (Ref 42, 64, 67&68). The stability of the levees and I Walls was determined by the Method of Planes using the design (Q) shear strengths and applying a minimum factor of safety of 1.3 with respect to shear strength. The minimum factor of safety at pipeline crossings was taken as 1.5. The levee slopes and berm distances were designed for a hurricane water condition at still water level for the project hurricane and assumed failure toward the protected side and a mean low water condition and failure toward the flood side. The project was divided into seven design reaches.

Preliminary stability analyses were conducted to compare stabilities of various trial levee sections and also to consider the feasibility of mucking out and backfilling for the levee base. For the preliminary analyses, shear strengths from unconfined compression tests on samples from the general-type borings were utilized. The Method of Planes was employed for the analyses. The analyses indicated that the shear strengths "in situ" were inadequate for proper stability if the levee were constructed to final section in one operation. They also indicated that the mucking out and backfilling scheme did not increase stability results sufficiently to justify the construction expense. They further indicated that a "stage" or "lift" construction scheme was necessary so that gains in subsoil shear strength could be made through consolidation under the intermittent "lifts" of embankment material so as to arrive at proper stability for the final levee section. It should be noted that the levee construction from Station 7+52.9 to 807+00 was over an area where spoil from the dredging of the MR-GO had been placed over a 5- to 7 -year period to an Elev. varying from 5.0 to 12.0 feet. This area was from 2,000 to 4,000 feet in width. There has been considerable consolidation of the underlying strata as evidenced by the general borings which indicated that the original ground has been depressed from 5 to 10 feet by the surcharge of the spoil. Furthermore, a statistical analysis of over 30 general borings made along the levee centerline at 3,000 foot intervals indicated average subsurface strata strength 15 percent higher. In areas with overburden as compared to areas without overburden and, in the top 30 feet of strata, this strength increase was approximately 25 percent. The increase in strength was further evidenced by the spoil bank itself which was standing, in some cases, up to Elev. 10.0 or 11.0 feet on slopes steeper than 1 on 3.

3.2.1.7.2.2.1.7. Cantilever I-Type Floodwall. (Ref 23, 45, 48 & 105) The cantilever I-type floodwall from Station 479+95 to Station 487+18, from Station 579+90 to Station 604+15, and from Station 621+6- to Station 656+45 were designed for a water level at El. 123.5 NGVD. The required penetration for the stability of the walls were determined by the4 Method of Planes analysis for both the short-term (Q) and the long term (S) cases. The wall was analyzed for the S-case using the shear strengths of c = 0 and $\phi=23°$ for clay strata. The following criteria was applied:

Case	Factor of Safety	Criteria
Q	1.5	Water at SWL
Q	1.25	Water at SWL and waveload
S	1.2	Water at SWL and waveload

The factors of safety were applied to the design shear strengths. Using the resulting shear strength, net horizontal water and earth pressure diagrams were determined for movement toward each side of the sheet pile. Using these distributions of pressure, summations of horizontal forces were equated to zero for various tip penetrations. At these penetrations, summations of overturning movements about the bottom of the pile were determined. The required depth of penetration to satisfy the stability criteria was determined where the summation of movements was equal to zero. The S-case was the governing case.

3.2.1.7.2.2.1.8. T-Walls. The foundation design for the T-type floodwall at the control structures were to be presented in a detailed design memorandum. (Reference 43).

3.2.1.7.2.2.1.9. Erosion Protection. Due to the short duration of hurricane floods and the generally erosion-resistant nature of the soil along this project, no erosion protection was considered necessary along the leveed portion of the project. Riprap protection was considered necessary around the structures at Florida Avenue.

3.2.1.7.2.2.1.10. Review Comments. First Endorsement comments paragraph 3. Questioned using borrow material below Elev. 0.0 because material between 0.0 and bottom of the pit is unsuitable for levee construction.

Paragraph 7 of 2nd Endorsement recommends a new procedure in design of I-walls under Hurricane conditions (i.e., where to put landside saturation line). The higher the saturation line, the lower the passive resistance and the use of a net pressure diagram with a factor of safety of 1.3 instead of 1.0.

Fourth Endorsement has considerable discussion but says "In the future, design of floodwalls will conform to the criteria spelled out in 2nd Endorsement paragraph 7. There are considerable comments in all endorsements about this subject.

3.2.1.7.2.2.2. Bayou Bienvenue and Bayou Dupre Control Structures (Ref 43)

3.2.1.7.2.2.2.1. General. The general design for Bayou Bienvenue and Bayou Dupre Control structures is presented in Reference 41 and the detailed design for the two structures is presented in Reference 43. The criteria review presented herein was obtained from Reference 43.

3.2.1.7.2.2.2.2. Geology. The general geology within the Chalmette area was presented in paragraph 3.2.7.2.2.1.1.

3.2.1.7.2.2.2.3. Project Foundation Conditions

a. Bayou Dupre Control Structure. The upper stratum from existing ground at Elev. 8 feet to Elev. -3 feet consists of very soft gray clays (CH) with roots, peat and organic matter and had water contents varying from 43 percent to 85 percent. From Elev. -3 feet to Elev. -23 feet, the soils consisted of alternating layers of soft clays (CL) and soft silty clay with clayey silt layers. Water contents in these layers ranged from 34 to 45 percent. From Elev. -23 to Elev. -26 and from Elev. -31 feet to Elev. -58 feet, layers of gray clays (CH) with some fine sand, sand lenses and shell fragments were encountered. Consistencies varied from stiff to medium stiff and water contents varied from 46 to 65 percent. Between Elev. -26 feet and Elev. -31 feet a stratum of sand was encountered. From Elev. -58 feet to Elev. -64 feet, a medium dense fine gray sand stratum occurs over the Pleistocene formation at Elev. -64 feet. Except for a soft dark gray clay stratum occurring between Elev. -65.5 feet and Elev. -67.5 feet having an average water content of 30 percent, the remainder of the soils down to Elev. -91 feet consisted of layers of medium stiff to stiff clays (CL and CH) with water contents varying from 23 to 45 percent.

One boring made 50 ft landside of the transverse centerline of the Bayou Dupre control structure indicated similar soft clays as found in the two borings made within the structure site except that greater amounts of silts and sands were encountered. A comparison of the stratification indicated an increase of silts and sands in the landward direction from the structure.

b. Bayou Bienvenue Control Structure. From existing ground at Elev. 5.5 feet to about Elev. -8 feet, the soil was found to be very soft dark gray and dark brown clay with peat, wood, and fine rootlets with water contents that range up to 310 percent. From Elev. -8 feet to Elev. -28 feet, the stratum consisted of soft to very soft gray clay with silt pockets, sandy silt pockets and shell fragments with water contents varying between 50 and 80 percent. From Elev. -28 feet to Elev. -35 feet, a stratum of gray sand was encountered ranging from loose to dense. From Elev. -35 feet to Elev. -63 feet, there was encountered soft to stiff gray clays with silt pockets and occasional small shell fragments. From Elev. -63 feet, which was the top of the Pleistocene, to Elev. -78 feet, the limit of the boring, the soil consisted of soft to medium gray clays and green clays with silt and sand lenses.

3.2.1.7.2.2.2.4. Field Exploration. Four 3-inch I.D. general type borings were made by the Corps of Engineers at each structure location. One 5-inch I.D. undisturbed boring designated as B-IU was made by the Corps of Engineers at Bayou Bienvenue. At Bayou-Dupre, one 5-inch ID undisturbed boring was made by the Engineer-Architect during the preparation of Reference 41 and was used in Reference 43. An additional undisturbed boring designated as D-IU (5-inch ID) was made by the Corps of Engineers at Bayou Dupre.

3.2.1.7.2.2.5. Seepage, Dewatering and Pressure Relief.

a. Both sites for the structures had received hydraulic spoil from the excavation of the MR-GO. Underlying the spoil were very soft clays with sand and silt lenses and organic matter typical of the marshy areas of this locality. Underlying these strata was a sand stratum between Elev. -26 feet and Elev. -31 feet at Bayou Dupre and between Elev. -28 feet and Elev. -35 feet at Bayou Bienvenue. Study of this condition indicated the need for some form of pressure relief during construction at both structures.

b. Dewatering and pressure relief during construction were studied and analyzed for two methods of excavation: (1) Plan No. 1 Open Excavation and 2) Plan No. 2, Open Excavation with Steel Sheet Pile Enclosure of Foundation Mat Area. The procedure used in design of pressure relief was obtained from Reference 55. The coefficients of permeability (k) were estimated using Figure 3-39 of Reference 55. Design calculations were performed for the pressure relief system for Plan No. 1 and for the design of sand drains required for Plan No. 2. All analyses were performed following procedures presented in Reference 55.

(1) Slope stability analyses for Plan No. 1 indicated that during the initial dewatering to Elev. -5 there would be a potential heave of the excavation bottom if the tide level in the MR-GO exceeded Elev. 2 feet. An outer ring of well points installed in the upper berm, Elev. 2 feet at Bayou Dupre and Elev. 3 feet at Bayou Bienvenue, was designed to relieve the hydrostatic pressure in the underlying sands. With the tide in the MR-GO at Elev. 8 feet and the outer well point system operating, the piezometric pressure reading in the sands would be at Elev. -5 feet at the center of the excavation. At this point there would be a factor of safety of 1.3 against bottom heave when dewatered to Elev. -5 feet.

Division comments directed that the well point screens fully penetrate the pervious stratum or either stagger the well screens at various elevations between the top and bottom of the pervious stratum to yield an effective 100 percent well penetration. The District elected to stagger the well screen tip elevation and use 3-foot long screens.

(a) With the excavation continuing and the berm at Elev. -5 feet exposed, the inner ring of well points would be installed and would completely encircle the excavation. The Bayou Dupre location would have well point tips at Elev. -31 feet and Bayou Bienvenue would have well point tips at Elev. -31 feet. With a tide level of Elev. 5 feet in the MR-GO, the piezometric pressure reading in the underlying sands would be at Elev. -21 feet in the center of the excavation with the inner ring of well points in operation. The outer ring could be removed when the inner ring began pumping. Boring D-lU at Bayou Dupre indicated a silty clay stratum between Elev. -10 feet and Elev. -23 feet. Pressures in this stratum would be relieved by encasing the well points with sand from Elev. -10 feet to the sand stratum that water was being withdrawn from. Above Elev. -10 feet to the berm at Elev. -5 feet, the well points would be encased in clay.

(b) Based on the pressure relief analysis it was estimated that the inner ring could pump approximately 500 gallons per minute at Bayou Dupre and 265 gallons per minute at Bayou Bienvenue.

(c) Sump pumps would also be required to remove some seepage which could be expected from the side slopes of the excavation, and to remove surface runoff within the excavation from rainfall of a 25-year frequency rain storm.

(2) Plan No. 2 proposed a combination of open excavation and a steel sheet pile enclosure of the foundation mat area with all excavation being done in the “dry”.

(a) When the initial excavation was completed to Elev. -5 feet, steel sheet pile would be driven to completely enclose the foundation mat area and the area to receive the derrick stone. Sheet pile would be driven to a tip elevation of -55 feet at Bayou Bienvenue and -60 feet at

Bayou Dupre. It was assumed that some seepage would enter the enclosure through sheet pile joints where the sand stratum had been penetrated. Based on the analyses, it was recommended that 12-inch diameter sand drains be placed every 20 feet around the periphery, inside the sheet pile enclosure. The sand drains and piezometers were to be installed when the excavation was at Elev. -5 feet after the sheet pile wall was installed. The drains would extend through the underlying sand to approximate Elev. -40 feet at Bayou Dupre and Elev. -37 feet at Bayou Bienvenue. This would allow a self-relieving condition as the excavation was carried down and with the use of sump pumps the working area could be kept dry. Sufficient pumping capacity was to be provided to remove surface runoff within the excavation from rainfall of a 25-year frequency rain storm.

As a result of Division comments, the sand drains were eliminated and replaced with well points.

c. During an unwatered condition it was assumed that the water on the MR-GO side would be at Elev. 5.0 feet and the water on the land side would be at Elev. 2.0 feet. Under these conditions and with the structure completely dewatered, a factor of safety of 1.16 against uplift was computed that disregarded the hold down straps on the piles. Assuming the cut off wall impervious and the same water heights as above, a factor of safety of 1.07 against uplift was computed that disregarded the hold down straps on the piles. Therefore, no pressure relief was considered to be was required.

d. During the normal operating condition, with the gates open, no pressure relief was computed to be required.

e. A steel sheet pile cut-off wall was to be driven below the base slab and inverted T-type wall. This cut-off wall was to effectively stop piping action in the event that roofing occurred.

f. During review, the Division directed that the influence of the slope back of the sheet piles be considered in the stability analyses. The Division also directed that the analyses be performed using (S) strengths. The District concurred.

3.2.1.7.2.2.6. Temporary Protection Levee. Protection from flooding of the construction area was to be provided by placing a temporary levee around the excavation. The construction areas would be protected from normal tidal waters and also from high tides in conjunction with adverse winds without the temporary levees. The temporary levee would be constructed to Elev. 8 feet to protect the construction areas from storms of less than design size. This elevation would protect the construction areas from high waters resulting from the majority of the storms experienced in this locality. However, the temporary levee would be subject to over-topping by severe storms approaching design intensity. The frequency of such storms was not considered to warrant raising the levee any higher.

3.2.1.7.2.2.7. Slope Stability

a. Construction slopes and permanent slopes for both structure locations were analyzed by the Method of Planes for stability with a minimum factor of safety of 1.3 using (Q) shear strengths. Values of increased shear strengths due to consolidation were based on procedures developed during the analyses of levee stability for the preparation of Reference 41.

b. The following sections were analyzed for stability:

(1) Stream closure of Bayou Bienvenue and Bayou Dupre: The analyses indicated that a shell core would be required for stability.

(2) A stability analysis was made for a high bank section adjacent to the approach channel at Bayou Dupre. Results of this study indicated the need for degrading high areas adjacent to the approach channel to Elev. 6 feet and sloping to drain towards the channel. Water in the approach channel was assumed as Elev. 0.0 in the study.

(3) Stability analyses were performed for a section taken from the end of the levee to the approach channel. Water surface in the approach channel was assumed at Elev. 0.0 feet. The full levee height and increased shear strengths were used. This study determined the location of the toe of the levee with relation to the top of bank of the approach channel. These analyses also defined definite lengths of floodwall for each location.

(4) Stability analyses for the open excavation at Bayou Bienvenue and Bayou Dupre were performed for three conditions at each structure as follows:

Condition No.1: Initial dredge excavation completed with the bottom at Elev. -16 feet. Water behind the temporary protection levee at Elev. 2 feet and the water in the excavation at Elev. -5 feet with outer ring of well points operating.

Condition No. 2: This condition would normally be experienced during construction. Completed excavation to Elev. -19.28 feet. Water behind the temporary protection levee at Elev. 2 feet and in the excavation at Elev. -19.28 feet, with the well point system operating.

Condition No. 3: This was a storm condition with water behind the temporary protection levee at Elev. 8 feet and water at Elev. -19.28 feet with the well point system operating.

(5) The alternate method of excavation, open excavation with sheet pile enclosure of the foundation mat, was also analyzed for stability for each structure for Condition 2 of paragraph (4) above without the well point system.

(6) Retention dikes for the spoil areas were also analyzed for stability.

(7) The Division commented on the procedures used by the District to estimate strength gains due to consolidation. After much discussion, the procedure used by the District was not modified by the Division.

3.2.1.7.2.2.8. Stability of Floodwalls and Wing Walls

a. **General.** Floodwalls were required to connect the control structures to the location where the full levee section would begin. Adjacent to the structures, an inverted T-type wall would be constructed and an I-type wall would make the transition between the inverted T-type wall and the full levee section.

b. **T-Walls.** The inverted T-wall of reinforced concrete was to be supported by prestressed concrete bearing piles driven at a batter and a steel sheet pile cut-off wall. A factor of safety of 1.75 was used for determining compressive pile penetration and 2.0 for tension piles. Two methods of analysis were used in the stability study of the inverted T-wall as follows.

(1) The first method used was that presented by References 56 and 57. Analysis based on the above references was performed for each of the loading conditions for each location. A group of curves was developed showing actual and allowable stresses and deflections of the battered piles for various assumed modulus of subgrade reaction (K) values. Approximate values of K were obtained from unconfined compression test results based on methods presented in References 16 and 58. Positions of the values determined from these references on the above mentioned group of curves indicated that the battered pile foundation of the inverted T-wall was satisfactory.

(2) The second method of analysis was based on the "Method of Elastic Centers" as presented in the book titled "Substructure Analysis and Design", by Paul Andersen.

c. **I-Walls.** The I-type flood wall was to be constructed from precast prestressed concrete sheet piles driven in place and capped by a concrete walkway. Stability analyses were performed using the Method of Planes. The floodwall was analyzed for a hurricane condition with a still water elevation of 13 feet and a 5-ft broken wave on the flood side and ground water at Elev. 2 feet on the protected side. The wall was investigated for both (Q) and (S) design shear strengths for a factor of safety of 1.5 with static water level at the top of the wave and a factor of safety of 1.25 with the dynamic force of the wave added. The effect of drag force on the wall was investigated and found not to be critical.

d. **Anchored Sheet Pile Walls.** At each end of the gate bay there was to be an anchored precast prestressed concrete sheet pile retaining wing wall. The wing walls were analyzed for stability using both (Q) and (S) shear strengths. The water was assumed to be at Elev. 0.0 feet on the channel side and behind the wall. A factor of safety of 1.5 was used in both analyses.

e. The Division directed that the stability of the "I" and "T" type floodwalls and wingwalls be analyzed for conditions of maximum reverse differential head (MR-GO side Elev. 0.00, landside Elev. 5.0 feet) using (Q) and (S) design shear strengths and a factor of safety of 1.5.

3.2.1.7.2.2.9. Pile Capacity Analyses. Pile lengths were determined by using the (Q) values obtained from the soil boring laboratory results applied to the full length of the pile. A factor of safety of 1.75 was used for compression piles and a factor of safety of 2.00 was used for tension piles. Pile penetrations were also determined by using (S) values obtained from laboratory results of the soil samples, applied to the lower two-thirds of the pile length. Pile lengths using the appropriate (Q) or (S) curve were determined for each structure. Steel sheet pile cut-off walls were provided beneath the gate bay structure and beneath the inverted T-type floodwall with tips at Elev. -26 feet. The cut-off walls were provided to prevent piping beneath the structure in the event roofing occurs below the slab. Pile lengths shown in the DM were for estimating purpose only and final pile lengths were to be determined after pile load tests are performed at each structure location during construction.

3.2.1.7.2.2.10. Erosion Control and Protection. Erosion protection for the access channel bottom adjacent to the control structure on the flood side was to consist of 3 feet of derrick stone on 1 foot of shell extending 100 feet from the gate bay and 2 feet of riprap on one foot of shell extending an additional 100 feet. The erosion protection on the protected side of the structure was to consist of 2 feet of riprap on 1 foot of shell extending 150 feet from the gate structure. The channel side slope within the above limits was to have 2 feet of riprap on 1 foot of shell to Elev. 5 feet. Erosion protection beyond the above limits was not included in this Detail Design Memorandum and would be placed by local interests as required. Erosion protection as described was considered to be required as protection against high velocities that would occur under certain conditions. Under normal operations (gates open) velocities of 7 feet per second could be anticipated approximately 1 percent of the time. An abnormal condition could occur where there would be a reverse head resulting from closure of the gates for hurricane approach and abnormal rainfall ponded within the area and delay in re-opening of the gates and a rapid drop in tide in the MR-GO could result in considerable run-out. In cases such as this, eroding velocities would occur dictating the need for erosion protection. The shell beneath the derrick stone and riprap was required to form a supporting blanket, otherwise the stone would eventually sink into the soft channel bottom. Ground elevation in the area of the structures and adjacent levees range between 5 and 6 feet and the structures and levees are located approximately 500 feet from the edge of the MR-GO. Erosion protection of the structure backfill and adjacent levees was not indicated for the condition of high tides and wave wash from passing vessels since the ground elevation and the distance from the MR-GO would eliminate the above problem.

3.2.1.7.2.2.12. Engineering Observations. Bearing pile tip elevations given in this section were for estimating purpose only. Upon completion of excavation, three Class B treated timber piles of different lengths was to be driven at each structure site as part of the base slab foundation. At each site, the short pile and the intermediate pile were to be tested in compression. If test results show that either of these two piles can carry twice the design loads, the long pile would not be tested. One pile at each site would be tested in tension. At Bayou Bienvenue, the test piles would be driven to the following tip Elev. -65 feet, -70 feet and -75 feet. At Bayou Dupre, the test piles would be driven to the following tip elevations: -60 feet, -65 feet and -70 feet. The results of the load tests would be evaluated to confirm tip elevations of the 12x12 concrete pile supporting the inverted T-wall.

Settlement observations of the structure were to be made frequently during construction. Settlement plates were to be placed in the surcharged area and observed frequently during and after pre-loading so as to determine the time rate of settlement. This data was to be used in determining the required gross elevation of the concrete cap of the “I” floodwall. Elevation measurements were to be taken prior to each concrete pour of the base slab and gate bay walls. Permanent reference markers were to be placed on the structure and the floodwalls.

Settlement and lateral movement observations were to be made quarterly for the first two years after completion of construction, and annually thereafter. A periodic examination of this schedule for adequacy was to be made as the data are obtained. Scour surveys were to be made, at the same time settlement measurements are made, at each end of the gate structure and in the area adjacent to the riprap until it has been determined that the channel bottom has become stabilized.

3.2.1.7.2.3. Structural

3.2.1.7.2.3.1. Chalmette Area Plan (Reference 41). As constructed, the Chalmette Area plan consists of primarily of earthen levee; with segments of combination levee and capped cantilevered I-wall, and T-wall; one swing gate at Paris Road; and one swing gate at a railroad crossing. The hurricane protection ties into the IHNC hurricane protection on the western end and the Chalmette Extension hurricane protection on the eastern side.

- **Structural Design**
- **Design Criteria**
    - **Unit Weights**

Item	*lb per cu ft*
Water	62.5
Concrete	150.0

- **Design Loads**
    - Earth Pressure (lateral)
    - Water loads
        - No wave force
        - Surge to within 6 inches of the top of the wall
- **Allowable Working Stresses.** The allowable working stresses for concrete and structural steel are in accordance with those recommended in "Working Stresses for Structural Design", EM-1110-1-2101 of 6 January 1958, revised August 1963. Concrete will be designated by basic minimum strength 3,000 pound concrete. Steel sheet piling meeting the requirements of ASTM A328-54, "Standard Specification for Steel Sheet Piling" will be used.

- **I-type floodwalls were used with the following observations**
    - Bending moments and deflections are based on a factor of safety of 1.0 applied to the soils, since the structural steel has an inherent safety factor of about 2.0.
        - In the 3d Ind of 23 Feb 67, LMVD stated that the pile section should be selected on the basis produced for a loading with a factor of safety 1.5 with low tailwater and a factor of safety of 1.3 with high tailwater due to saturation from rainfall.
        - In the 4th Ind of 10 Mar 67, NOD stated that DIVR 1110-1-400 dated November 1966 which specifies the use of a soil shear strength factor of safety of 1.0 in evaluating deflections and stresses is being revised, at the direction of OCE, to require the use of a factor of safety of 1.3.

- In the 5th Ind of 22 Mar 67, LMVD stated that DIVR 1110-1-400 was revised in March 67 and that the revision indicates that bending moments, stresses and wall deflections for I-type floodwalls should be computed using the same earth and water pressure diagrams as those used in determining pile penetration. In the case of Lake Pontchartrain and Vicinity, earth pressures computed from the S shear strength are governing the design. LMVD permitted a 1/3 overstress for the sheet piling in clay because the duration of loading is very short. However, for piling in clean sand, LMVD stipulated normal stresses should be used.

- The strength of the wall was checked for the case with water at the top of the wall, as initially constructed and found to be adequate.

- **T-Wall Monoliths**. Due to the complex nature of the design, a detailed design memorandum was proposed to cover this aspect of the design. This detailed design memorandum could not be located.

  - Bayou Bienvenue and Bayou Dupre control structures are covered in Design Memorandum No. 5 – Detail Design

3.2.1.7.2.3.2. Chalmette Area Plan. Chalmette Extension (Reference 42)

General. As constructed, the Chalmette Extension hurricane protection consists of primarily unreinforced levee. In addition, there is a segment of capped cantilevered I-wall at the Verret Gap; a capped cantilevered I-wall with a roller gate crossing Louisiana Highway 46 (as shown on drawings, Hwy 39 in DM) at Caernarvon; and a soil-founded drainage structure at Creedmore.

- **Structural Design**
- **Design Criteria**
  - Unit Weights

Item	*lb per cu ft*
Water	62.5
Concrete	150.0

- **Design Loads**
  - Earth Pressure (lateral)
  - Water loads:
    - Design Still Water Elevation as follows:
      Verret Gap Closure = El 12.2
      Caernarvon Gap Closure = El 11.8
  - Wind Loads:
    A 60 mph wind was applied to both gap closure gates

- **Allowable Working Stresses.** The allowable working stresses for concrete and structural steel are in accordance with those recommended in "Working Stresses for Structural Design", EM 1110-1-2101 of 6 January 1958, revised August 1963. Concrete will be designated by basic minimum strength of 3,000 psi. Steel sheet piling meeting the requirements of ASTM A328-54, "Standard Specification for Steel Sheet Piling" will be used. Pertinent allowable stresses are tabulated below:

Reinforced Concrete	*Stress psi*
fc'	3,000
fc	1,050
v (without web reinforcement)	60
v (with web reinforcement)	274
fs	20,000
Minimum tensile steel	0.0025 bd
Shrinkage and temperature steel	0.0020 bt
Structural Steel (ASTM – A36)	
Basic stress	18,000

The allowable stresses are increased 33-1/3% for Group 2 loading

- Gap Closure Structures
  - Verret gap closure – Bottom Roller Gate
  - Caernarvon gap closure – Two Bottom Roller Gates (One at Highway 39 and one at Southern Railroad Company tracks)
  - Allowable Bearing Pile Loads
    Verret – 34 kips
    Caernarvon – 47 kips
  - **I – Wall Criteria**
    - Load Cases
      S case and Q case
      FS=1.5 SWL at top of wall
      FS= 1.25 with dynamic wave force
    - Verret
      SWL at El 12.2
      Five foot broken wave
      Ground water at El 1.0 protected side
    - Caernarvon
      SWL at El 11.8
      Five foot broken wave
      Ground water at El 2.0 protected side

Drainage Structure – Two 72 inch diameter corrugated metal pipe culverts

3.2.1.7.2.3.3. Chalmette Area Plan - (Reference 43). Bayou Bienvenue and Dupre Control Structures

General. As constructed, both Bayou Bienvenue and Bayou Dupre Control Structures consist of a pile-founded sector gate structure with adjacent pile-founded T-wall and cantilevered I-wall of precast concrete sheet piling. The navigable width of both structures is 56-feet. Both structures have timber guide walls and dolphins.

Structural Design Criteria

Basic Data

Top of Gate Walls	Elev 13.0
Sill	Elev -10.0
Width of Gate Channel	Elev 0.0
Maximum Water Surface MR-GO Side	Elev 2.0
Maximum Differential Head MR-GO Side	Elev 13.0
Land Side	Elev 2.0
Maximum Differential Head MR-GO Side (Reverse)	Elev 0.0
Land Side	Elev 5.0

Unit Weights

Unit Weight	*lb per cu ft*
Water	62.5
Concrete	150
Shell Backfill	98

Lateral Pressure

Shell Backfill (Ǿ= 40º)	Equivalent Fluid Pressure
Select Backfill	Sat unit wt = 110 pcf
	Cohesion = 120, Ǿ = 0
Active (Above Water)	21.3 lbs
Active (Submerged)	8 lbs
At Rest (Above Water)	54 lbs
At Rest (Submerged)	20 lbs

Allowable Working Stresses. The allowable working stresses for structural steel and concrete are in accordance with those recommended in “Working Stresses for Structural Design”, EM 1110-1-2101, dated 1 November 1963. The basic minimum 28-day compressive strength for concrete will be 4,000 psi except for prestressed concrete piling where the minimum strength will be 5,000 psi. Steel for steel sheet piling will meet the requirements of ASTM A328-69, “Standard Specification for Steel Sheet Piling

Allowable Working Stress Structural Steel, ASTM A-36

	Group 1 Loading	Group 2 Loading
Basic Stress	18,000 psi	24,000 psi

Allowable Working Stresses Concrete (3,000 psi, 28 days). Concrete which will be subjected to submergence, wave action and spray will be designed with working stresses in accordance with ACI Building Code with the following modifications:

	Stress (psi)
Flexure (fc):	0.35 f'c.
Extreme fiber in tension (Plain concrete for footings and walls but not for other portions of gravity structures)	$1.2 \sqrt{f'c}$
Extreme fiber in tension (For other portions of gravity structures)	$0.6 \sqrt{f'c}$
Allowable stresses in reinforcement in tension for deformed bars with a yield strength of 60,000 psi or more.	20,000 Group 1 Loading
For Group 2 loading, the above stresses may be increased by 33 1/3%.	
Minimum tensile steel	0.0025 bd
Shrinkage and temperature steel area	0.0020 bt

Structural Steel (ASTM A-36)

Basic working stress	18,000

Application of Working Stresses

Group 1 Loading	Allowable working stresses as listed for structural steel and for reinforced concrete will be applied to the following loads: Dead Load Live Load Buoyancy Earth Pressure Water Pressure
Group 2 Loading	Allowable working stresses as listed for structural steel and for reinforced concrete will be applied to the following loads when combined with Group 1 Loads: Wind Loads Wave Loads Boat Loads Erection Loads

Design Loading Conditions

Base Slab

Case 1	Gate open, backfill not in place, no buoyancy
Case 1A	Gate open, backfill in place, no buoyancy
Case 2	Structure complete, backfill in place, water at Elev 0.0, buoyancy active
Case 3	Needle dams in place, structure dewatered, gates removed, water at Elev 5.0, buoyancy active
Case 4	Hurricane condition, gate closed, water in MR-GO at Elev 13,0, water on land side at Elev 2.0, buoyancy active
Case 5	Gate closed, water in MR-GO at Elev 13.0, water on land side at Elev 0.0, water on land side at Elev 5.0, buoyancy active
Case 5A	Case 5 above, cutoff wall assumed pervious

All of the above conditions are considered as Group 1 Loadings

Case 6	Case 4 above, and wave loading (Group 2 Loading)
Case 7	Case 6 above, cutoff wall assumed pervious (Group 2 Loading)

Sector Gates

Case 1	Dead load only which includes truss members, skin plate, skin plate supports, fender system and fender system supports
Case 2	Dead load, water in MR-GO at Elev 13.0, water on land side at Elev 2.0
Case 3	Dead load, water in MR-GO at Elev 0.0, water on land side at Elev 5.0

Cases 1, 2, and 3 are considered as Group 1 Loadings

Case 4	Case 3 with a boat load of 120 kips acting at right angle to canal truss
Case 5	Dead load, water at Elev 13.0 in MR-GO and a wave loading on MR-GO side and water on land side at Elev 2.0.

3.2.1.7.2.4. Sources of Construction Materials

3.2.1.7.2.4.1. Sheet Pile. Generally, the sheet pile sections specified during advertisement were used for construction. However, sheet pile section substitutions conforming to the minimum required section modulus was allowed, primarily in contracts constructed after 1990. Below, is a table of sheet pile sections for St. Bernard, broken down by DM.

St. Bernard	
Chalmette Extension	
Verret Gap Roller Gate Tie-In	**
Hwy 46/39 Roller Gate South Tie-In (Caernarvon)	PZ-27
Creedmore Tie-In	**

Chalmette Area Plan	
Paris Rd. Floodgate Tie-In	PZ-27 & PMA-22
Paris Rd. Floodwall	PZ-27
RR Swing Gate Tie-In	PZ-22
Bayou Bienvenue Control Structure	
Floodgate Tie-In	
East Tie-In	PZ-22
West Tie-In	Syro SPZ-16
Bayou Dupre Control Structure	
East Tie-In	precast concrete sheet pile
West Tie-In	precast concrete sheet pile
As-advertised – Not confirmed as-built ** Information not located at the time of publication	

3.2.1.7.2.4.2. Levee material

3.2.1.7.2.4.2.1. Sources of Borrow Material (Chalmette Area Levees). The earth fill for completing the existing levee portion of the protection between the IHNC Lock and Florida Avenue was to be obtained from a borrow area in the bottom of Lake Pontchartrain along the north shore. This material, consisting of stiff Pleistocene clays, was to be transported to the project on barges. A borrow area located on the floodside of the new levee between Station 81+38 and Station 89+45 was expected to provide all the earth fill required for the construction of the new levee between Florida Avenue and the north bank of the Outfall Canal (Station 67+94 to Station 79+62) and the first lift earth fill for the new levee along the north bank of the Outfall Canal (Station 79+62 to Station 90+25). For the former section of levee, no borrow was to be taken below Elev. 0.0. The borrow area was to be refilled by hydraulic methods during construction of the levee north of Station 90+25. Earth fill for the second and third lifts of the latter section of levee was to be obtained from the bottom of Lake Pontchartrain and transported to the work site on barges.

3.2.1.7.2.4.2.2. Embankment and Berm Fill. The clay embankment and berms were to consist of earth materials naturally occurring or Contractor blended, and to be classified in accordance with ASTM D 2487 (Reference No. 65) as CL, CH, or ML.

Compacted Fill. Compacted fill was not to be placed in water. The materials for compacted fill were to be placed or spread in layers. The first layer was to be 6 inches thick and the succeeding layers not more than 12 inches in thickness prior to compaction. The first and each succeeding layer of compacted fill was to be compacted to at least 90 percent of maximum dry density as determined by ASTM D 698 (Standard Proctor Density) (Reference No. 66) at a moisture content within the limits of plus 5 to minus 3 percent of optimum.

Uncompacted Fill. Uncompacted fill (berms) was to be placed in approximately horizontal layers not exceeding 3 feet in thickness. Moisture content control of uncompacted fill was not required.

3.2.1.7.2.4.2.3. Structure Backfill. The excavation adjacent to the gate bay walls and sheet pile wing walls was to be backfilled with clam shell to elevation zero. The remainder of the backfill to Elev. 6.0 feet was to be made utilizing selected material from the spoil area.

3.2.1.7.2.4.2.4. Backfill of Existing Bayou Channels. Upon completion of the gate control structures, floodwalls, levee tie-in, and access channels the closure of Bayou Bienvenue and Bayou Dupre was to be made at the location of the levee centerline. The closure at each location was to be made in three stages. This would allow underlying clays to gain shear strengths during the period between stages of construction. The first stage was to be the placement of a clam shell core and hydraulic fill. The shell core was required as a back-up for the hydraulic fill. The second stage was to consist of additional shell and hydraulic fill. The third stage would be the final shaping and a clay blanket.

3.2.1.7.3. As-built Conditions

3.2.1.7.3.1. Changes between design and construction (i.e. cross sections, alignment, sheet pile tip el, levee crest el.)

3.2.1.7.3.1.1. DACW29-02-C-0044. Lake Pontchartrain and Vicinity, High Level Plan, Hurricane Protection Plan, Chalmette Levee IHNC to Paris Rd. Station 157+00 to Station 282+37, St. Bernard Parish, LA

Reviewed Mod Log Report, no applicable modifications or changes found.

3.2.1.7.3.2. Inspection during original construction, QA/QC, state what records are available. See paragraph 3.2.1.5.4.2 New Orleans East Bank for description of how records are kept.

3.2.1.7.3.2.1. DACW29-02-C-0044 – L PONT, IHNC – PARIS RD, LEVEE ENLARG, ST BER PAR

Attached to QA/QC Reports are in-place density tests and preparatory phase inspection checklists.

3.2.1.7.4. Inspection and maintenance of original construction. Inspections of Civil Works projects in the New Orleans district fall primarily under two programs, not including local sponsor inspections:

Periodic Inspections - Inspections of Federal Civil Works structures, owned and operated by the federal government, are done under the Periodic Inspection Program, as defined by ER 1130–2–100, entitled *Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures.* Bridges are inspected under ER 1110-2-111, entitled, *Periodic Safety Inspection and Continuing Evaluation of USACE Bridges.* These inspections are funded by the appropriate projects under Construction, General (CG) appropriation, during the final phases of construction, and Operations and Maintenance (O&M), General (O&M,G) appropriation during the O&M phase.

Annual Compliance Inspections - Certain provisions for these inspections are codified under 33 CFR 208.10. Inspections of federal flood control projects, operated and maintained by non-federal sponsors, are inspected under the Inspection of Completed Works program, under ER 1130–2–530, entitled *Flood Control Operations and Maintenance Policies,* dated October 30, 1996. (This engineering regulation supersedes the previous regulation ER 1130–2–339, entitled *Inspection of Local Flood Protection Projects.* These projects are funded by the Inspection of Completed Works Project, under both the (O&M,G) and the Flood Control, Mississippi River and Tributaries (FCMR&T) appropriations.

3.2.1.7.4.1. Annual Compliance inspection (i.e. trees, etc.). Annual inspections were conducted by Operations Division for projects under the Inspection of Completed Works Project for the St. Bernard polder which is a part of the Lake Pontchartrain and Vicinity Hurricane Protection Project. These inspections, which were general in nature, primarily defined the status of existing project work, and a general condition rating.

For the last 6 years, 1998 through 2004, the ratings for the Orleans Levee District, and the Lake Borgne Basin Levee District, which covers the St. Bernard polder were "OUTSTANDING" through year 2001, and "ACCEPTABLE" each year thereafter, at which time there was a change in the Project Rating Scale. The project rating scale was then redefined, and "ACCEPTABLE" became the highest rating.

There was no specific mention of deficiencies for the hurricane protection system.

3.2.1.7.4.2. Periodic Inspections. The St. Bernard polder contains two structures, Bayou Bienvenue Control Structure, under the authority of Orleans Levee District, and Bayou Dupre Control Structure, under the Lake Borgne Basin Levee District. Both of these structures are inspected under the New Orleans District Periodic Inspection Program. The following info summarizes the inspection and repairs history for these structures.

3.2.1.7.4.2.1. Periodic Inspections of Bayou Bienvenue Control Structure (Reference 60)

3.2.1.7.4.2.1.1. Historical Deficiencies Reported During and Related to Periodic Inspections

Date	**Description of Observations**
Oct 1974	During Periodic Inspection No. 1, the structure was still under construction. The concrete sheet pile I-wall was recommended for replacement by modification to the construction contract. This was the only deficiency documented in Report No. 1.
July 1979	Spot rusting of the Sector gate members and corroded surfaces of the sector gates and embedded steel members required cleaning and treatment with a corrosion preventative material. The electronic gate monitor was non-operational. Both sides of the approach channels were missing rip rap.

March 1983	Rip rap along the approach channels was missing again. Additional rip rap, 275 feet north and 200 feet south of the structure, was recommended to be placed to assist in erosion control. A 1-inch gap was noted between the gate seals. Corrosion in the areas of tidal fluctuation and separation of expansion joints on the wing walls was noted. Heavy vegetative growth was noted. The expansion joint between the west wing wall and the structure on the protected side needed repair and backfill.
March 1985	The floodwall and wing wall joints were not watertight. Vegetation was noted in one of the expansion joints in the northwest floodwall. Sinkholes and voids were noted behind the wing walls. Missing rip rap was noted again on both the north and south approach channels. Broken handrails and safety chains required repair. Staff gages required cleaning and repair.
March 1988	Missing rip rap in the approach channels continued to be a deficiency. Navigation lights were frequently found to be broken due to vandalism. Rust and corrosion was noted on steel members, ladders and steel plates. Staff gages required cleaning and repair.
July 1991	Missing riprap in the approach channels continued to be a deficiency. Metal pile caps were rusted and the timber guide wall was termite infested. Rust and corrosion was noted on steel members, ladders and steel plates.
March 1994	Deficient riprap, rusting steel members, and the termite infested guidewall/missing timbers noted in the last inspection continued to exist. Missing safety chains noted. A hazardous electrical conduit and loose cables/frozen sheave in the machinery room was noted. The staff gages were unreadable. Small concrete spalls were noted. A depression behind the wing wall in the northwest corner was noted.
March 1999	Small spalls and hairline cracks noted in the concrete surfaces. Upward seepage through a small crack/hole in the sill slab was noted. Corroded areas on embedded metals were noted. Wire ropes used to activate the gate sectors were loose. Equipment and sheaves in the equipment recesses required cleaning. The frequency meter on the generator set was improperly operating. Defective load side conductors for the east gate sector required replacement. The east gate indicator light system required repair/replacement. The lights in the control room required cleaning. The fluorescent fixtures in the machinery recess required replacement. A broken weatherproof cover on the receptacle near the access stair required replacement. All receptacles required replacement with GFCI units. Guidewalls were noted to be in poor condition. An evaluation was recommended to determine if major repair and or replacement of the guidewalls was necessary. Settlement markers needed repainting. A reliable benchmark was required.

3.2.1.7.4.2.1.2. Historical Repairs/Construction Work Bayou Bienvenue Structure

Date	**Description**
July 1979	Damage due to vandalism was repaired as part of regular maintenance. Rip rap was placed along the landside channel banks to prevent erosion. Ladders were installed on the protected side of the structures to provide access from ground level to the top of the structure. The concrete sheet pile I-wall was pulled and stockpiled for future placement after the levee adjacent to the structure settles.
March 1985	The north and south channels received scour repair. The 3/4-inch gap between the gate seals, the corrosion in the areas of tidal fluctuation, and the separation of expansion joints on wing walls were repaired during dewatering. The vegetative growth and debris was cleaned-up. Siltation and accumulated oyster shells were removed from the gatebays. Corrosion in the area of tidal fluctuation was removed and the gates were sandblasted and repainted. The cathodic protection anodes on the skin plate and structural members were replaced. The floodwall and wing wall joints were repaired and made water tight. Vegetation was removed from one of the expansion joints in the northwest floodwall. Sinkholes and voids behind the wing walls were backfilled. Broken handrails and safety chains were replaced. Staff gages were cleaned and repaired. Reference marks were repaired and grouted.
July 1991	Minor deficiencies were repaired as necessary under routine maintenance program including navigation light repairs, corrosion monitoring, cleaning, re-painting and staff gage repair/cleaning.
FY 1993	Steel sheetpile floodwalls were installed at the end of each “T” Floodwall to tie into the levee on either side of the structure. The sheetpile tie-in brought the structure up to hurricane protection elevation.
March 1994	Rusting metal pile caps and termite infested guidewall members were replaced as necessary.
Mar 1994-97	Deficient rip rap, rusting steel members, termite infested timbers and missing timbers were replaced. Missing safety chains, hazardous electrical conduits, loose cable/frozen sheave in machinery room and unreadable staff gages were repaired/replaced as necessary.
Feb 1997	On the west side of the structure, the steel sheetpile floodwall levee tie-in (installed in 1993) was cutoff at ground level and a new concrete I-wall section was constructed to tie-into the levee.
July 1999	The wire ropes used to activate the gate sectors were tightened. The defective load side conductors for the east sector gate was replaced. The broken weather proof cover on the receptacle near the access stairs was replaced.

3.2.1.7.4.2.2. Period Inspections of Bayou Dupre Control Structure (Reference 61)

3.2.1.7.4.2.2.1. Historical Deficiencies Reported During and Related to Periodic Inspections

Date	Description of Observations
13 Feb 1973	A sand boil was noted at Station 12+35 centerline of the structure and Station 699+97 centerline of the levee while the timber piles for the foundation were being driven. Sand boil disappeared after the contractor finished driving the piles.
22 Feb 1974	The following items were noted during Periodic Inspection No. 1: (1) Two rectangular stiffener plates used to stiffen the vertical girder-horizontal rib connection of the sector gates formed a reservoir trapping water and dirt that would accelerate deterioration of the protective paint; and (2) Derrick stone placed in the bottom of the MRGO approach channel was small and poorly graded.
12 Mar 1980	The following items were noted during Periodic Inspection No. 2: (1) The first two pile bents on the northwest timber guide wall had been damaged near the end; (2) The east concrete sheetpile wall had settled 0.6 to 0.9 feet since construction; (3) Severe scour action was noted on the west bank between Stations 5+00 and 9+62 on the north approach channel and moderate scour action was noted between Stations 14+00 and 17+00 on the south approach channel; (4) Vertical joints on the east side T-wall monoliths showed separation from 0.25 inches at the top of wall to 0.0 inches at the bottom; (5) Sector gates had heavy corrosion within the tidal fluctuation area; (6) The alternator belt was loose on the diesel engine for the generator; (7) Some indicator lights on the control panel were burned out; (8) There was a loose coupling on the # 1 side of the electric motor; (9) Riprap bank protection was weathering and breaking down in the tidal fluctuation area due to poor quality.
Dec 1982	Minor damage occurred to the service wharf on the protected side of the structure.
1 Dec 1983	The following items were noted during Periodic Inspection No. 3: (1) A segment of Range 17+00 (300-feet from the east bank) had scoured approximately 10 feet; (2) Concrete sheetpile walls adjacent to the T-walls had experienced differential settlement between piles, with no apparent separation of the joints; (3) Separation of the joint where the concrete sheetpile wall ties into the west T-wall; (4) Sector gates had corrosion within the tidal fluctuation area; (5) A hole 10 feet wide by 30 feet long by 5.5 feet deep was noted behind the riprap on the east bank of the north channel approach; (6) The west gate had to be opened manually because a limit switch was damaged due to over closure during the inspection; (7) The damage to the first two pile bents on the northwest timber guide wall noted at last inspection had not yet been repaired; and (8) Wire rope for the sector gates lacked adequate lubrication.

25 Jun 1986 The following items were noted during Periodic Inspection No. 4: (1) A thin sheet of concrete had started to separate from the east side sector gate bay wall near the gate's top hinge recess; (2) The filler material between the gate bay monolith and the "T" walls was desiccated on both sides of the structure; (3) Form tie rod patches on the walls of the structure had begun to separate from the walls; (4) Some walls appeared to be covered in a white powdery substance believed to be curing compound; (5) Steel members of the sector gates located in the tidal fluctuation zone were corroded; (6) The landing dock on the west bank of the structure's south approach channel had been redamaged and not usable; (7) The 10-foot by 30-foot by 5-foot deep hole was apparently the remnants of a drainage ditch and had been closed by a small stone dike; (8) The L-shaped waterstop and filler material had separated at the joint between the east gate monolith and the wingwall on the MRGO side of the structure; (9) The tidal current warning light was not functioning as designed; (10) The disconnect/transfer switch from commercial to emergency power was not labeled to indicate the purpose, position and status of the switch; (11) Timber dolphins on the east side of the structure had been damaged by tows; (12) The staff gages at the control structure were corroded; and (13) The east concrete "I" wall had settled more and also at an accelerated rate as compared to the west "I" wall.

18 Mar 1987 The following items were noted during Periodic Inspection No. 5 (Phase I): (1) Severe corrosion of three steel girders on the west gate in the tidal fluctuation zone was evident. Steel members of the east sector gate were in very good condition with only surface corrosion; (2) Elements of the sector gates were overgrown with barnacles and other marine life; (3) Holes in the PVC surrounding the sacrificial anodes were in some cases completely blocked by the growth of barnacles and the anodes in the pipes were barely consumed; (4) Concrete surfaces below the water were completely covered in barnacles and other marine growth; and (5) Surface corrosion was observed on most miscellaneous steel members. Steel ladders were completely destroyed by corrosion and removed by the contractor.

8 Apr 1987 The following items were noted during Periodic Inspection No. 5 (Phase 2): No new deficiencies were noted. Those deficiencies noted on the 18th of March had been corrected. A coal tar epoxy paint system was used instead of a vinyl paint system (original).

25 Apr 1990 The following items were noted during Periodic Inspection No. 6: (1) A shrinkage crack with efflorescence was noted at the lower corner of the west gate hinge recess; (2) A thin sheet of concrete had separated off the wall near the east gate hinge; (3) The vertical crack with minor efflorescence at the cable assembly of the east gate had not changed from the last inspection; (4) The exposed reinforcing steel near the "A-NE" mark on the east gate monolith had not changed from the last inspection; (5) Some tie rod patches on the walls appeared to have been repaired – remaining original patches had not deteriorated any further; (6) A white substance (curing compound) was observed on the walls; (7) The metal

hatch on top of the east gate monolith had corrosion around its perimeter - causing minor concrete spalling; (8) Minor spalls were observed on the top of the concrete sheetpile due to differential settlement; (9) Between the two west side “T”-type floodwall monoliths, a 1/2-inch gap was found at the top with no gap at the bottom, and the east side “T”-type floodwall monoliths had 1-inch gaps; (10) The sealant in the expansion joints had desiccated, shrunk on the top and side and some sealant was missing from the top joint; (11) Gaps of 1 inch on the east side and 1-1/2 inch on the west side were found between the top of the “T”-type floodwall and the structure monolith; (12) A gap of 1 inch to 1-1/2 inches was observed between each of the four wingwalls and the structure monolith; (13) The expansion joint between the “T”-wall and the concrete sheet pile wall on the west side of the structure had separated considerably - approximately 6 inches; (14) The engine generator exhaust had a leak where the engine manifold and flexible exhaust meet; (15) The east and west concrete sheet piles continued to settle and some were pulling apart (1/2 inch to 1-1/2 inches) due to the embankment fill adjacent to these structures; (16) Two additional staff gages were noticed on the structure and should be removed in order to avoid confusion in data collection; (17) The metal caps on the piles were rusted, but otherwise still functional; (18) The fender system had minor nicks from marine traffic; and (19) The two timber pile dolphins on the south side of the structure were damaged.

29 Apr 1993 The following items were noted during Periodic Inspection No. 7: (1) The east and west concrete sheetpile walls were still settling; (2) Minor weathering effects were noted on the handrails; (3) The concrete sheetpile on the west side had separated from the T-wall toward the west and created a gap where there was no sealant left between the joints; (4) Vegetation was noted in the joint between the concrete sheetpile and the T-wall on the east side; (5) Small spalls were noted in the surface of the gatebay monoliths and the channel walls on both the east and west sides; (6) Hairline cracks were noted on top of the gatebay monoliths and T-walls on both east and west sides; (7) A small diagonal crack was observed on the west side near the edge of the gatebay structure and handrail; and (8) Efflorescence was noted on the channel walls and on both west and east sides of the gatebay structures.

3 Sep 1997 The following items were noted during Periodic Inspection No. 8: (1) Minor hairline cracks and small spalls that were noted in previous inspections did not appear to have changed or increased in number; (2) The T-wall/gatebay expansion joint material at the east and west side joints had deteriorated - the waterstop was exposed; (3) The T-wall sheetpile joint on the west side had an excessive opening and exposed reinforcing steel in the T-wall concrete on the south side of the joint; (4) The concrete sheetpile alignment was not straight and several small spalls were noted at the tops at the joints - some deterioration of the plastic interlocks; (5) The northwest wing wall had separated about 2-1/4 inches from the gatebay structure at the top of the wall - “L” shaped waterstop barely spans the opening, and a depressed area in the backfill behind this joint was noted; (6) The corresponding openings in the three other wingwalls were smaller,

but depressed areas behind in the backfill at these locations were also noted; (7) Minor corrosion and marine crustaceans were observed on the sector gates above the normal splash zone; (8) Embedded metal at the needle girder recesses and the corner protection had corroded near and slightly above the splash zone; (9) The east side gate operating machinery brake enclosure was rubbing on the motor shaft where it passes through the brake enclosure; (10) The exterior of the machinery enclosures were corroding; (11) The west side “gate closed” limit switch did not function as it was misaligned and in a position that did not mate with the toggle arm; (12) The 12 volt D.C. current wiring serving the navigation light was not enclosed in a protective conduit; (13) The batteries in the control house were not in protective enclosures; (14) The Tidal Current Warning System was inoperative; (15) The PVC sleeves housing the cathodic protection anodes had filled with oysters and clams and could not be removed; (16) A few rotten and damaged timbers were noted on the guide walls and gate fenders; (17) Timber dolphins at the end of the northeast, southeast and southwest guide walls were damaged and leaning badly.

3.2.1.7.4.2.2.2. Historical Repairs/Construction Work Bayou Dupre Structure

Date	Description
Apr 1974	The following work was accomplished: (1) Holes were drilled in the bottom of the stiffeners to allow drainage; (2) Additional riprap was placed on top of the derrick stone in the wet – soundings verified quantity to be placed; (3) Concrete sheetpile “I” walls were installed; and (4) Remaining 4 feet of earth fill was placed from the west “I” wall to the shell closure across Bayou Dupre.
Oct 1975	Repairs were made to the north approach channel. Rip rap was placed on the west bank extending the entire length of the north approach channel from Station 5+00 to 10+62. Lost derrick stone was placed from Station 10+62 to the structure.
Apr 1976	The Coast Guard had repaired navigational lights, lens, and batteries.
Jul 1977	Seven hundred (700) linear feet of trench was dug for running electrical lines/conduits to the control house and control panel boxes.
9 Feb 1981	Completed scour repairs on both sides of the structure. The channel required 45,000 tons of rip rap, 23,000 tons of Class “C” stone and 10,800 C.Y. of shell.
2 Qtr 1981	The Levee Board added fill and made general repairs to the tie-in levees.
Prior Dec ’83	The following was performed: (1) The alternator belt was tightened; (2) All burned indicators lights were replaced; (3) The loose coupling on the electric motor was adjusted and tightened.

Prior Jun '86	The following was performed: (1) The service wharf was repaired by local interests; (2) Local interests had completed repairs to the northwest wall; and (3) Local interests had been lubricating the wire rope for the sector gates.
Mar-Apr '87	The following was performed: (1) Blasting and painting of sector gates, and structural steel members and skin plates and other miscellaneous metals were thoroughly cleaned and professionally painted with a coal tar epoxy paint; (2) Ladders and other metal items damaged by corrosion were repaired; (3) The three badly corroded sections of the west sector gate were replaced; (4) Replacement of the timber fender system on both gates; (5) Repairs to the dolphins and their navigation lights on the east side of the structure; (6) Repair of the tidal current warning system; (7) Replacement of staff gages; (8) The cathodic protection system was completely replaced, three rows of ship hull anodes were installed on each skin plate, new 60-inch, 250-pound anodes were located within PVC protection tubes, and the contractor drilled 4 holes, at 90 degrees to each other, instead of three holes at 120 degrees to each other; and (9) The concrete surfaces were cleaned.
Prior Apr '93	The following was performed: (1) The rusted portion of the metal hatch on top of the east gate monolith was cleaned and painted and the concrete repaired; (2) Wood blocking was installed to hold the expansion joint material in place at the gap on top of the T-type floodwall and structure monolith on the east and west sides; and (3) The spalled area at the joint of the concrete sheetpile was repaired.

3.2.1.7.5. Other Features

3.2.1.7.5.1. Brief Description. The primary components of the hurricane protection system for the St. Bernard basin are described above, namely the levees and floodwalls designed and constructed by the Corps of Engineers. However, other drainage and flood control features that work in concert with the Corps of Engineers levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section will describe and present the criteria and pre-Katrina conditions of the interior drainage system, pump stations, non-Corps levees, and the Mississippi River Flood Protection System. Even though the stormwater pump stations are part of the interior drainage system, they are a significant part of the system and warrant their own section.

3.2.1.7.5.2. Pre-Katrina Conditions. According to the local jurisdictions responsible for interior drainage, the storm drain systems, interior canals, outfall pump stations, and outfall canals were in good condition and prepared for high inflows from rainfall prior to August 29, 2005, Katina landfall.

The St. Bernard back levee along the Forty Arpent Canal was in good condition prior to Katrina landfall.

The Mississippi River Flood Protection System was in good condition prior to Katrina landfall.

3.2.1.7.5.3. Interior Drainage System.

Overview. The developed area of the St. Bernard basin contains about 32 square miles and the undeveloped area is 45 square miles. This section only addresses the developed area. The land generally slopes south to north from the Mississippi River to marsh adjacent to the Intracoastal Waterway and Mississippi River Gulf Outlet (MRGO). The northern subbasin (Chalmette to Violet) is highly developed while the southern subbasin (Poydras and St. Bernard) is partially developed. Many features are typical of large urban cities in the United States, and some features that are unique because much of the area is below sea level. Catch basins and inlets collect surface runoff from yards and streets into storm sewers. Excess runoff flows down streets and/or overland to lower areas. Open canals collect the stormwater and carry it to outfall pump stations along the non-Corps back levee that pump directly into the marsh. No stormwater is pumped into the Mississippi River. There are two entities responsible for local drainage in the St. Bernard basin. St. Bernard Parish is responsible for the local streets, storm sewers, ditches, and small canals. The Lake Borne Levee District is responsible for the large interior canals and pump stations.

The Lower 9th Ward in Orleans Parish is contained in the far western end of this basin. The local stormwater collects in large enclosed conduits and carried to Pump Station #5 which pumps into the marsh. The system components have been described in the Interior Drainage System section for Orleans East Bank.

System Components. Local drainage begins with overland flow which follows the ground topography. Figure 5 in Volume VI shows the topographic layout of St. Bernard. The land generally falls from the Mississippi River to the marsh adjacent to MRGO. The southern subbasin does have a ridge that runs east-west.

The local drainage is collected by underground storm drains and roadside ditches which carry the water to the canals. Photos 1 and 2 show typical inlets and streets in St. Bernard. Photo 3 shows a typical roadside ditch collector.

The land topography and development sequence influenced the storm sewer, canal, and pump station layout. Based on land topography and the drainage system, the basin is divided into 67 subbasins. Pump station information is presented in Section 3.2.1.7.5.4 of this volume.

The interior canals are grass-lined and carry the water to the canals that parallel the non-Corps back levee where the outfall pumps are located. (Photos 4, 5, and 6).The interior canals not only collect stormwater from streets and storm sewers and covey it to the pump stations, they also are storage areas that work in conjunction with the pump stations.

Photos 1 and 2. Typical Streets and Inlets – St. Bernard

Photo 3. Typical Roadside Ditch Collector

Photo 4. Interior Canal from Judge Perez Dr.

Photo 5. Interior Canal near Judge Perez Dr.

Photo 6. Forty Arpent Canal and at Pump Station #3, Bayou Villere

Design Criteria. The current design criterion for new storm drainage facilities in St. Bernard is the 10% probability (10 year frequency) for the collection system and 4% probability (25 year frequency) for the interior canals and pump stations. The interior drainage systems in the older and rural areas have a capacity of about a 50% probability (2 year frequency) event. Where canal or pump capacity is not available downstream, new commercial developments are required to put in stormwater detention facilities so there is no impact for the 10 year frequency event (Photo 7). The calculated capacity of the interior canals and pump stations is 0.4 inches per hour. Rainfall in excess of this amount goes into temporary storage in the canals, storm sewers, open areas, and streets

There are no Southeast Louisiana (SELA) Urban Flood Control Projects in this basin.

Photo 7. Onsite Detention Basin on Judge Perez Dr.

3.2.1.7.5.4. Pumping stations - St. Bernard Parish Summary. St. Bernard Parish is located east of the city of New Orleans and borders the east side of Orleans Parish. Figure 18 is a map of St. Bernard Parish with the pump stations that were studied identified by red dots. St. Bernard Parish is located on the east bank of the Mississippi River. To alleviate flooding from rainfall, pumps drain the area. The Lake Borgne Basin Levee District owns and operates eight pump station located along the interior back levee. Rainfall runoff is collected through a system of culverts, canals, and ditches delivering the storm water runoff to the pump stations. The pump stations discharge the runoff over the interior back levee into the marsh north and east of the levee. This report examined the 8 Parish pump stations with a total of 28 pumps. The locations of the pump stations were verified by Global Positioning System (GPS) and/or by using Google Earth Pro. The GPS coordinates were then input into Microsoft Streets and Trips (shown below).

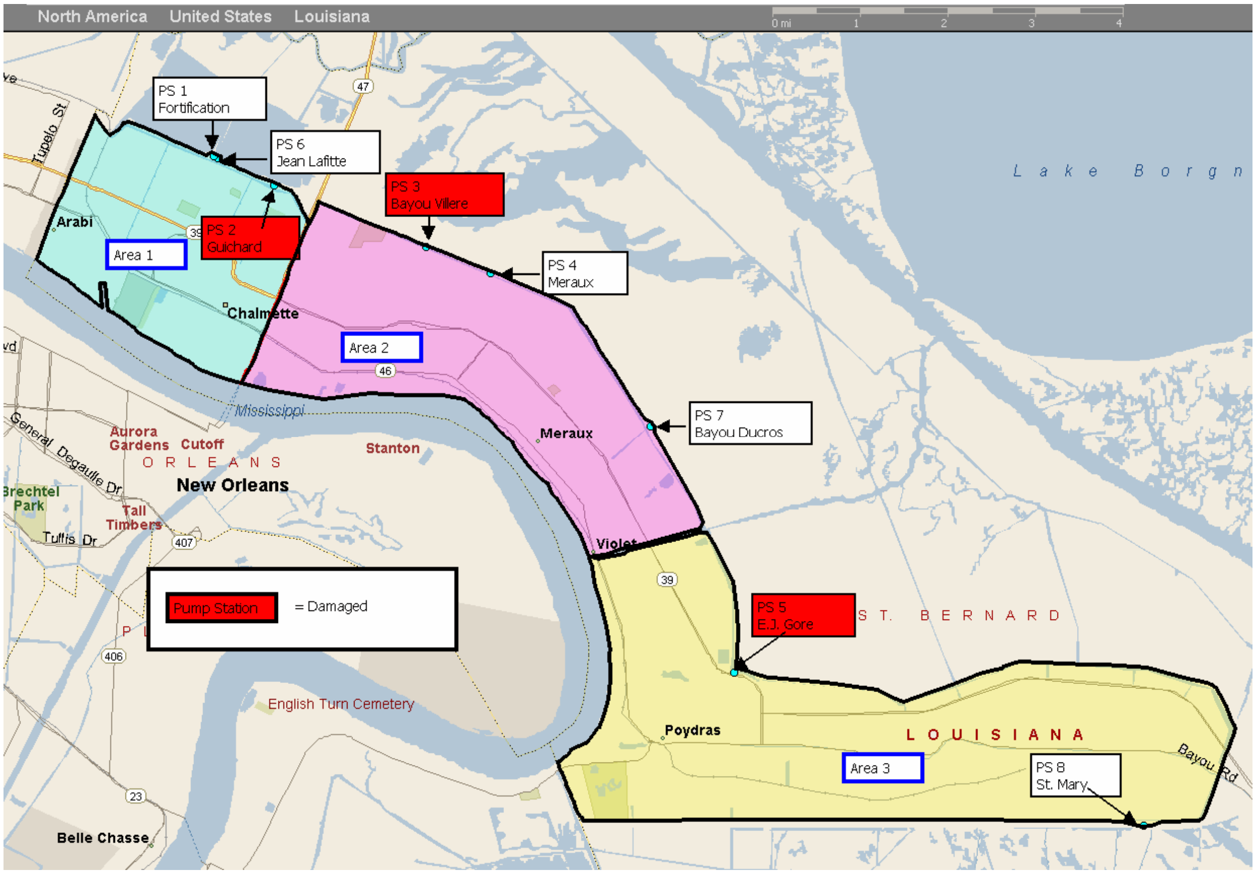


Figure 18. St. Bernard Parish Pump Station Locations

Drainage Basins. St. Bernard Parish consists of three drainage basins. All of the pump stations lay on the borders of the drainage basins. The stations are evenly distributed through the parish, with area three having two pump stations while area one and two each have three pump stations. All the pump stations have a suction basin from a canal and discharge into various bayous and lakes in the surrounding area. The pump stations vary between vertical and horizontal pump configurations. Details for each pump station are listed in Volume VI.

Area 1

PS 1 – Fortification

Intake location: ....Florida Walk Canal
Discharge location: ....Bayou Bienvenue
Nominal capacity: ....980 cfs

Pump	Capacity (cfs)	Year Installed	Driver Electric /Diesel	Pump Configuration
1	445	1972	Diesel	Vertical
2	90	1972	Electric 60 Hz	Vertical
3	445	1972	Diesel	Vertical

PS 2 – Guichard

Intake location: .... Florida Walk Canal
Discharge location: .... Bayou Bienvenue
Nominal capacity: .... 825 cfs

Pump	Capacity (cfs)	Year Installed	Driver Electric /Diesel	Pump Configuration
1**	111	1950's	Diesel	Horizontal
2**	267	1950's	Diesel	Horizontal
3	180	1950's	Diesel	Horizontal
4**	267	1950's	Diesel	Horizontal

PS 6 – Jean Lafitte

Intake location: .... Forty Arpent Canal
Discharge location: .... Bayou Bienvenue
Nominal capacity: .... 1,000 cfs

Pump	Capacity (cfs)	Year Installed	Driver Electric /Diesel	Pump Configuration
1	333	1990	Diesel	Vertical
2	333	1990	Diesel	Vertical
3	333	1990	Diesel	Vertical

Area 2

PS 3 – Bayou Villere

Intake location: .... Forty Arpent Canal
Discharge location: .... Bayou Villere
Nominal capacity: .... 800 cfs

Pump	Capacity (cfs)	Year Installed	Driver Electric /Diesel	Pump Configuration
1**	267	1950's	Diesel	Horizontal
2**	267	1950's	Diesel	Horizontal
3***	267	1950's	Diesel	Horizontal

PS 4 – Meraux
Intake location: .... Forty Arpent Canal
Discharge location: .... Bayou Dupre
Nominal capacity: .... 980 cfs

Pump	Capacity (cfs)	Year Installed	Driver Electric /Diesel	Pump Configuration
1	445	1972	Diesel	Vertical
2	90	1972	Electric 60 Hz	Vertical
3	445	1972	Diesel	Vertical

PS 7 – Bayou Ducros
Intake location: .... Forty Arpent Canal
Discharge location: .... Bayou Ducros
Nominal capacity: .... 945 cfs

Pump	Capacity (cfs)	Year Installed	Driver Electric /Diesel	Pump Configuration
1	315	1992	Diesel	Vertical
2	315	1992	Diesel	Vertical
3	315	1992	Diesel	Vertical

Area 3

PS 5 – E.J. Gore
Intake location: .... Forty Arpent Canal
Discharge location: .... Bayou Dupre
Nominal capacity: .... 665 cfs

Pump	Capacity (cfs)	Year Installed	Driver Electric /Diesel	Pump Configuration
1	111	1980's	Diesel	Horizontal
2	111	1980's	Diesel	Horizontal
3	111	1980's	Diesel	Horizontal
4	111	1980's	Diesel	Horizontal
5	111	1980's	Diesel	Horizontal
6	111	1980's	Diesel	Horizontal

PS 8 – St. Mary
Intake location: .... Forty Arpent Canal
Discharge location: ....Lake Lery
Nominal capacity: ....780 cfs

Pump	Capacity (cfs)	Year Installed	Driver Electric /Diesel	Pump Configuration
1	260	1996	Diesel	Vertical
2	260	1996	Diesel	Vertical
3	260	1996	Diesel	Vertical

3.2.1.7.5.5. Levees and floodwalls

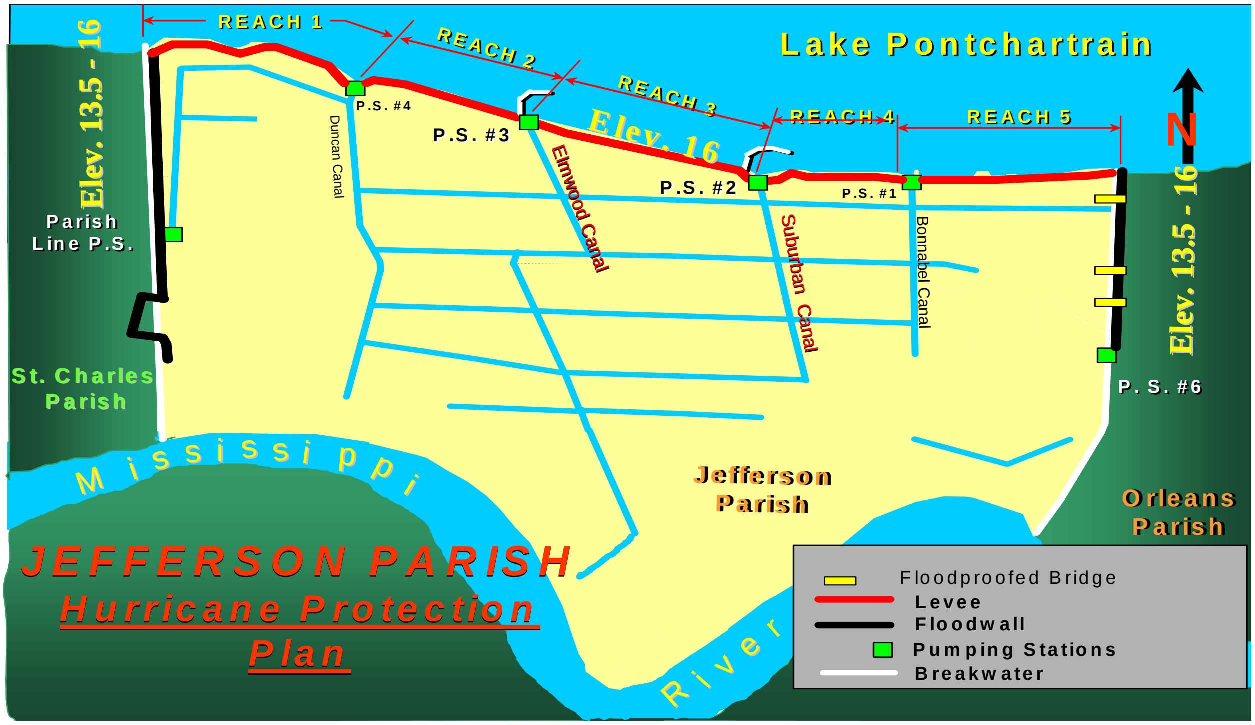
3.2.1.7.5.5.1. MRL. MRL levees and floodwalls are addressed in Paragraph 3.2.1.5.6.5.1 New Orleans East Bank MRL. There are some short reaches of floodwall near the IHNC and the Chalmette Battlefield that are part of the MRL.

3.2.1.7.5.5.2. Non Corps. Several local interest and/or private levees are located within the project area. No design criteria for these levees have been made available to the Corps

3.2.1.8. Jefferson East Bank

3.2.1.8.1. Introduction. The Lake Pontchartrain and Vicinity hurricane protection project in East Jefferson consists of levees, floodwalls, floodgates, and pump station fronting protection on its north (lakefront) shore, and portions of its west and east boundaries.

3.2.1.8.2. Pre-Katrina. The Jefferson Parish portion of the Lake Pontchartrain and Vicinity project is under construction. As of August 29, 2005, all of the levees had 2 lifts completed with the exception of Reach 4 which was under construction. An additional two lifts are planned for the Jefferson Parish part of the project. The plans and specifications for Reach 1 were completed prior to Katrina, however construction funds were not available to initiate construction. All structures were completed with the exception of floodwall tie-ins to the hurricane levee extending from Pumping stations 2 and 3. The plans and specifications for the tie-ins walls were under development pre-Katrina and are nearing completion. A review is underway to determine if the levees floodwalls and structures will have to be redesigned based on the results of the Interagency Performance Evaluation Team analysis and based on a reanalysis of design storm calculations. Preliminary indications are that the entire floodwall along the St. Charles and Jefferson Parish boundary will have to be replaced. Most of this wall is T-wall construction. Approximately 1500 feet of the wall is I-wall and an interim repair of that structure is underway. Additional contracts may be required as a result of this analysis.



3.2.1.8.3. Design Criteria and Assumptions - Functional design criteria and revisions / deviations

3.2.1.8.3.1. Hydrology and Hydraulics. For Jefferson East Bank, the design hurricane characteristics utilized in the design memoranda are shown in Table 17; the design track is shown on Figure 19. The maximum wind speed was computed using the same equations as for Orleans East Bank. For each project area, the track and forward speed were selected to produce maximum wind tide levels.

Table 17
Design Hurricane Characteristics

Location	Track	CPI, Inches	Radius of Maximum Winds, Nautical miles	Forward Speed, Knots	Maximum Wind Speed[1], MPH	Direction of Approach
Lake Pontchartrain South Shore	A	27.6	30	6	100	South

[1] Windspeeds represent a 5 minute average 30 feet above ground level.

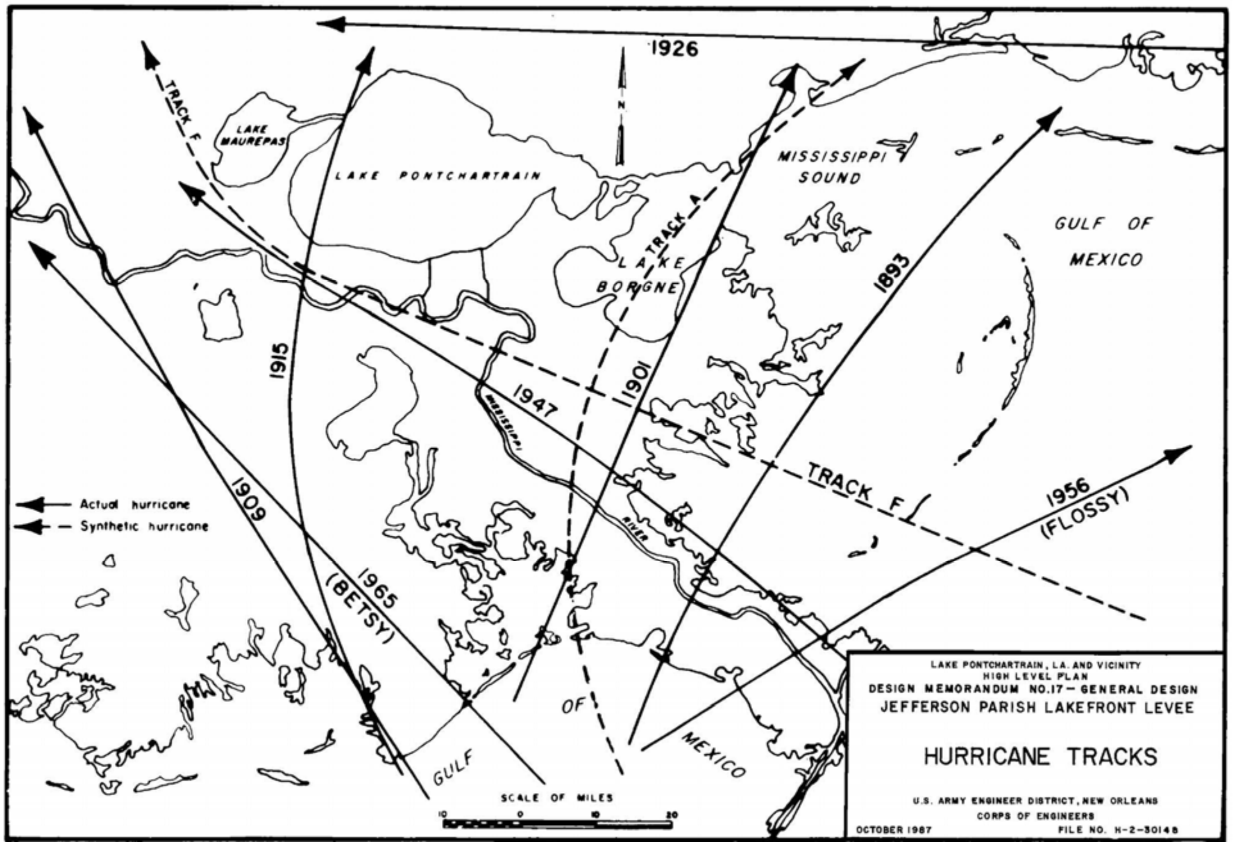


Figure 19. Hurricane tracks, Jefferson Parish Protection System

3.2.1.8.3.1.1. Surge. Surge elevations were computed using the same methodology as used for lakefront for Orleans East Bank. For the Jefferson/St. Charles Parish return levee, the height of protection required decreased southward from the lakefront as the height of the wind time would drop with distance due to the friction over the marsh. At the lakefront, the return levee height would match the lakefront design elevation.

3.2.1.8.3.1.2. Waves. Wave runup along the Lake Pontchartrain shoreline was calculated using the methodology described in Orleans East Bank. For Pump Stations No. 2 and No. 3, breakwaters were included in the design analysis. The wave runup was reduced, resulting in a floodwall height lower than the floodwall height for Pump Stations No. 1 and No. 4.

For the Jefferson/St. Charles Parish return levee, during the time the maximum wind tides are against the protection structure, the winds would be parallel to or leeward of the levee. These winds would generate waves that travel along the levee parallel to its alignment, and no wave runup would occur. The passage of the crest and trough of the waves would cause the water level to rise and fall on the protection structure. During the critical hour, the top of the wave would not be more than 3 ft above the still water level. After the critical hours, the winds would begin to flow more nearly perpendicular to the alignment. The waves could strike the levee at a highly oblique angle and cause wave runup. The height of this runup would not exceed the design grade.

3.2.1.8.3.1.3. Summary. Table 18 contains maximum surge or wind tide level, wave, and design elevation information.

Table 18
Wave Runup and Design Elevations (transition zones not tabulated – governing DM is listed)

Location	DM	Average Depth of fetch, ft	Significant Wave Height Hs, ft	Wave Period, T, sec	Maximum Surge or Wind Tide Level, Ft	Runup Height, Ft	Freeboard, Ft	Design Elevation Protective Structure, ft
Jefferson Lakefront	DM17, Vol 1, Nov 1987	24.6	7.9	7.2	11.5 NGVD	4.5	-	16.0 NGVD
Return Levee, Sta. 181+35.5 to 173+04.7	DM17A, Jul 1987				11.5 NGVD	-	3.0	14.5 NGVD and greater[1]
Return Levee, Sta. 173+04.7 to 130+70	DM17A, Jul 1987				11.5 NGVD	-	3.0	14.5 NGVD
Return Levee, Sta. 130+70 to 65+20.4	DM17A, Jul 1987				11.0 NGVD	-	3.0	14.0 NGVD

[1] transitions to Lakefront design elevation

3.2.1.8.3.2. Geotechnical

3.2.1.8.3.2.1. Jefferson Parish Lakefront Levee

The project extends from the Jefferson and St. Charles Parishes boundary line at the lakefront to the Jefferson and Orleans Parishes boundary line at 17th Street Outfall Canal (a distance of approximately 10.4 miles). The proposed levee generally follows the alignment of the 1950’s project.

The Jefferson Lakefront levee was divided into three soils reaches:

(1) Station 0+00 to 185+00 {Reach “A”)

(2) Station 185+00 to 343+95 (Reach “B”)

(3) Station 343+95 to 549+42.9 (Reach “C”)

The recommended design presented is a full earthen levee section with geotextile reinforcement, crown Elev. 18.0 feet (net 16.0 feet) for Reach A and, Elev. 16.0 feet (net 16.0 feet) for Reaches B and C, respectively.

There are five pumping stations along the lakefront. I-walls and T-walls were designed adjacent to Pumping Stations Nos. 1 and 4. A floodgate was also designed at Pumping Station No. 4 for access to the “bike path”. Design for hurricane protection at Pumping Stations Nos. 2 and 3 will be accomplished as a supplement to this DM. Two floodgates and the associated floodwalls were designed at Causeway Boulevard.

3.2.1.8.3.2.1.1. Geology. The project is confined to that portion of the Jefferson Parish levee that runs parallel to the Lake Pontchartrain shoreline from Orleans Parish to St. Charles Parish. This represents approximately 10 miles of levee. The project alignment is nearly parallel to the regional geologic strike and traverses Holocene surficial deltaic and subsurface lacustrine and marine deposits. Subsurface elevations at top of Pleistocene average -50 feet, but vary from -45 to -100 feet.

A surficial marsh veneer, 5 to 15 feet thick throughout the project, represents the last stage of sedimentation in the area. Marsh-type sediments are a result of annual Mississippi River overbank flooding and subsequent deposition of clay and silt size particles landward of the natural levees. A review of borings in the vicinity of the artificial levee indicates that the additional overburden acts as a surcharge, consolidating the underlying marsh deposits as much or more than 50 percent its original thickness. Along the centerline of the artificial levee, the additional loading of soil has, to a lesser extent, similarly affected the underlying lacustrine and bay-sound deposits.

3.2.1.8.3.2.1.2. Foundation Conditions. The upper 20 feet of materials adjacent to the shoreline generally consist of artificial fills on the south and marsh deposits on the north. Further north into the lake, the upper 20-30 feet consist of natural lake deposits. The marsh deposits generally-consist of very soft organic clays, clays and peat. Subsurface elevations of the top of the Pleistocene formation are approximately Elev. -50 feet. These are the dominant features in the design of the foundation works. The foundation conditions are essentially the same throughout the Jefferson Lakefront project. Reach A possesses a better shear strength with C and B progressively worse.

3.2.1.8.3.2.1.3. Field Exploration.

a. A total of thirty-eight (38) 5-in diameter undisturbed and fifty (50) general type soil borings were made for design and borrow in association with the Jefferson Lakefront project. The Reevaluation Report recommended that the levee centerline be located approximately 130 feet to the floodside of the existing levee centerline. Therefore, the borings were concentrated along the expected centerline. Due to the utilization of the geotextile reinforcement, the proposed centerline as presented herein was shifted landward to optimize the design section.

b. Borrow borings for hydraulic fill were taken in the area as stated in the feasibility study report. However, alternatives using hydraulic fill were eliminated during the design phase. Prior to preparation of plans and specifications, additional general type borrow borings will be taken in Bonnet Carré Spillway for hauled clay.

3.2.1.8.3.2.1.4. Seepage. The analyses for required penetration for seepage cut-off were performed by utilizing Lane's weighted creep ratio. The weighted creep distance was calculated as the sum of the vertical creep path distance plus one-third the horizontal creep path distance. Lane's weighted creep ratio is the ratio of the weighted creep distance to the maximum differential head and varies depending on soil type. The deeper penetration of the two analyses (stability and creep ratio) was selected as the recommended tip elevation of the sheet pile. All analyses showed that the stability analyses governed the penetration.

3.2.1.8.3.2.1.5. Pile Foundation. Ultimate compression and tension pile capacities versus tip elevations were developed for 12-, 14- and 16-inch square prestressed concrete and 12-inch timber piles. Overburden stress in the soft clay material was limited to D/B = 15 in the "S" case. The design parameters used are shown in Tables 19 and 20. The estimated tip elevations are based on the factors of safety presented in Table 21.

Table 19
Concrete Piles

	Q-Case						S-Case					
	ϕ	**K_C**	**K_T**	**N_C**	**N_Q**	**δ**	**ϕ**	**K_C**	**K_T**	**N_C**	**N_Q**	**δ**
CH	0°	1.0	0.7	9.0	1.0	0°	23°	1.0	0.7	0	10	23°
SM	30°	1.5	0.75	0	22	30°	30°	1.5	.75	0	22	30°

Table 20
Timber Piles

Q-Case							S-Case					
ϕ	K_C	K_T	N_C	N_Q	δ		ϕ	K_C	K_T	N_C	N_Q	δ
0°	1.0	0.7	9.0	1.0	0°		23°	1.0	0.7	0	10	23°
30°	1.25	0.5	0	22	28°		30°	1.25	.5	0	22	28°

Table 21
Recommended Factor of Safety

With Pile Load Test	Without Pile Load Test
Q-Case2.0	3.0
S-Case 2.0 (Dead load only)	3.0 (Dead load only)
1.0 (Total load)	1.5 (Total load)

It is anticipated that during construction, test piles will be driven and load tested in the project area. The results of the pile load tests will be used to determine the length of the service piles.

3.2.1.8.3.2.1.6. Shear Stability.

a. Bearing Capacity of the Geotextile Reinforced Levee. Since the reinforced embankment acts as a unit, bearing capacity has to be checked to insure that the embankment will not punch into the foundation soil. All geotextile reinforced sections have been analyzed, based on ASTM Special Technical Publication 952, Geotextile Testing and the Design Engineer, Joseph E. Fluet, Jr., editor, 1987, and were found to be adequate.

b. Shear Stabilities of the Earthen Levee with Geotextile Reinforcement. The stability of the levee was determined by the method of planes using the design "Q" shear strengths with appropriate hydraulic loading. The basic sections were set to fulfill hydraulic requirements during hurricane conditions. Thus levee centerlines had to be relocated landward and constricted because of limited right-of-way. Geotextile was introduced to stabilize the levee section. The levee section at Williams Boulevard boat launch provides adequate berm for wave runup hence the centerline was not moved and geotextile was not required.

(1) To overcome weak foundation soil strengths geotextile reinforcement was designed to obtain the required factor-of-safety of 1.3. The following equation was used to determine the critical wedges which required the maximum tensile strength needed in the geotextile:

$$T = \frac{(D_a - D_p)1.3 - R_a - R_b - R_p}{12}$$

Where T = tensile strength in lbs/in. at 5 percent strain and less than 40 percent of ultimate.

D_a = Drive active

D_P = Drive passive

R_a = Resistance active

R_b = Resistance neutral block

R_P = Resistance passive

(2) Once the critical wedges were determined by the LMVD method of planes analysis, this failure surface was checked by the Spencer Method in the PC-SLOPE micro computer program. The result of this analysis was used to determine the location of the geotextile and the corresponding tensile strength according to "Design with Geosynthetic" by Robert M. Koerner 1986. The embedment length "L" for pull-out was calculated by the following equation:

$$L = \frac{T}{\left[\gamma_1 H_1 Tan\phi_1 + C1\right] + \left[\gamma_2 h_2 Tan\phi_2 + C_2\right]}$$

Subscript 1 denotes soil parameter above geotextile. Subscript 2 denotes soil parameter below geotextile. "L" was measured from the critical active wedge into the anchorage zone and an equal length also placed in the active wedge zone. The designer intends to perform further refinement of the geotextile design during the preparation of the plans and specifications.

(3) *Shear Stability*. The stability of the levees at Williams Boulevard and the I-wall and T-wall levees at the pumping stations were determined by the method of planes analysis using the design "Q" strengths with appropriate hydraulic loading and were based on a minimum factor-of-safety of 1.3.

3.2.1.8.3.2.1.7. Cantilever I-Wall. The required penetration of the steel sheet piling ground surface was determined by the method of planes for both "S" and "Q" cases. Factors of safety of 1.5 for static water and 1.25 for static water plus dynamic wave force were applied to design shear strengths as follows: 0 developed = arctan (tan ϕ of available factor of safety) and cohesion/factor of safety. Using the resulting shear strengths, net lateral soil and water pressure diagrams were developed for movement toward each side of the sheet pile. With these pressure distributions, the summation of horizontal forces was equated to zero for various tip penetrations, and the overturning moments about the tip of the sheets were determined. The required depth of penetration to satisfy the stability criteria was determined where the summation of the moments was equal to zero.

3.2.1.8.3.2.1.8. T-Walls. *Deep Seated Stability Analysis.* A conventional stability analysis utilizing a 1.30 factor of safety incorporated into the soil parameters was performed for various potential failure surfaces beneath the T-wall sections. Analyses were performed for all T-wall sections. The summation of horizontal driving and resisting forces results in a value that is negative for all failure surface, indicating that no additional load need be carried by the structure.

3.2.1.8.3.2.1.9. Erosion Protection. Due to the short duration o flood stage and the resistant nature of the clayey soils; no erosion protection other than sodding is considered necessary on the levee slopes along most of the levee alignment. The existing foreshore protection is adequate to protect the shoreline during “normal” wave wash conditions. The foreshore riprap has been in place for more than 25 years and currently is in good condition. Therefore, no additional foreshore work to provide erosion control is necessary.

3.2.1.8.3.2.1.10. Review Comments. No comments to change design criteria.

3.2.1.8.3.2.2. Jefferson Parish, St. Charles Parish Return Levee. This project is 3.4 miles in length and includes all but 225 feet of T-wall on piling, under Interstate 10. A levee/I-wall was used.

3.2.1.8.3.2.2.1. Geology. The project is confined to that portion of the Jefferson Parish levee that runs parallel to the St. Charles Parish boundary and north from New Orleans International Airport to Lake Pontchartrain. This represents approximately 3.5 miles of levee. The project alignment is nearly normal to the regional geologic strike and traverses Holocene surficial deltaic and subsurface deltaic, lacustrine, and marine deposits. Subsurface elevations at the top of Pleistocene average -65 feet, but vary from - 45 to approximately -105 feet.

A surficial marsh veneer, 5 to 15 feet thick throughout the project, represents the last stage of sedimentation in the area. Marsh type sediments are a result of annual Mississippi River overbank flooding and subsequent deposition of clay and silt size particles landward of the natural levees. A review of borings in the vicinity of the artificial levee indicates that the additional overburden acts as a surcharge, in some instances consolidating the underlying marsh deposits to less than half its original thickness. Along the centerline of the artificial levee, the additional loading of soil has, to a lesser extent, similarly affected the underlying lacustrine, deltaic, and bay-sound deposits.

3.2.1.8.3.2.2.2. Foundation Conditions. The stratigraphy is basically tabular throughout except for minor entrenchments and undulations, created by artificial sediment loads and differential settling. Potential for additional differential settlement, structural uplift, or need of construction dewatering and its effect on foundation conditions must be addressed.

3.2.1.8.3.2.2.3. Field Investigation. A total of 12 general type and 14 undisturbed soil borings were taken and tested by the Corps of Engineers along the alignment of the existing levee/I-wall. The 11 general type soil borings, 1-G through 11-G and 5-GA extend to an approximate elevation of -100 feet NGVD and the 14 undisturbed soil borings, 1-U through 14-U, extend to an elevation between -80 feet and -100 feet NGVD.

3.2.1.8.3.2.2.4. Underseepage

a. I-Wall. The sheet pile penetration required to satisfy Lane's weighted creep ratio (LWCR) of 3.0 for soft clays was determined for the I-wall section. The deeper penetration of the two analyses (cantilever I-wall or creep ratio) was selected as the recommended tip elevation o the sheet pile floodwall except where the soil boring data indicated that a slightly deeper penetration would be preferable. The I-wall stability penetration elevation of -16.0 governed the required penetration.

b. T-Wall. A steel sheet pile cut-off will be used beneath the T-walls to provide protection against hazardous seepage during a hurricane. The sheet pile penetration required to satisfy Lane's weighted creep ratio (LWCR) of 3.0 for soft clays was determined for the T-wall sections. The required penetration for seepage cut-off is Elev. -12.0 feet. The steel sheet pile construction tip elevation is Elev. -18.75 feet, since the existing 20-foot long steel sheet piling in the existing levee will be used.

3.2.1.8.3.2.2.5. Pile Foundation. Ultimate compression and tension pile capacities versus tip elevations were developed for 12 and 14-inch square prestressed concrete piles. Overburden stress in the soft clay material was limited to approximately 1,000 psf in the (S) case. In determining the normal pressure on the pile surface for the (Q) case and (S) case, lateral earth pressure coefficients of 1.0 and 0.7 were used in compression and tension, respectively. The estimated tip elevations are based on the factors-of-safety presented in Table 22.

Table 22
Recommended Factors-of-Safety for Pile Capacity Curves

	With Pile Load Test	**W/O Pile Load Test**
Q-Case	2.0	3.0
S-Case	2.0 (Dead Load Only)	3.0 (Dead Load Only)
	1.0 (Total Load)	1.5 (Total Load)

It is anticipated that during construction, tests piles will be driven and load tested in the project area. The results of the pile load tests will be used to determine the length of the service piles.

3.2.1.8.3.2.2.6. Slope Stability

a. The stability of the levee with the I-wall was determined by the Method of Planes using the design (Q) shear strengths with appropriate hydraulic loading and were designed for a minimum factor-of-safety of 1.3.

b. For the I-Walls. A conventional stability analysis utilizing a 1.30 factor of safety incorporated into the soil parameters was performed for various potential failure surfaces beneath the T-wall sections. The summation of horizontal driving and resisting forces is negative for all failure surfaces, indicating that no additional load need be carried by the structure.

3.2.1.8.3.2.2.7. I-Walls. The required penetration for the stability of the sheet pile wall was determined by the method of planes analysis for both the short-term (Q) and long-term (S) cases. The wall was analyzed for the short term (Q) case, using the (Q) soil design parameters and the long term (S) case, using the (S) shear strengths of $C = 0$ and $\phi = 23°$ for clay strata. A factor of safety of 1.5 was applied to the design shear strengths as follows: 0 developed = arctan (tan 0 available/factor-of-safety) and cohesion value/factor-of-safety. Using the resulting shear strengths, net lateral soil and water pressure diagrams were developed for movement toward each side of the sheet pile. With these pressure distributions, the summation of horizontal forces was equated to zero for various tip penetrations, and the overturning moments about the tip of the sheets were determined. The required depth of penetration to satisfy the stability criteria was determined where the summation of the moments were equal to zero. The (S) case governed the required penetration.

3.2.1.8.3.2.2.8. T-Walls. The stability of the levee with the T-wall was determined by the Method of Planes using the design (Q) shear strengths with appropriate hydraulic loading and were designed for a minimum factor of safety of 1.3.

3.2.1.8.3.2.2.9. Erosion Control. Due to the short duration of the hurricane flood states, no erosion protection is considered necessary along most of the T-wall alignment. However, foreshore protection will be constructed on the flood side of the T-wall in areas where damages could occur from waves generated by other than hurricane winds. The foreshore protection will consist of 24 inches of riprap or gabions on a 6-inch thick shell bedding.

3.2.1.8.3.2.2.10. Review Comments. Fourth endorsement concludes that future I-wall designs will follow the criteria furnished in CEMRC-ED-GS letter dated 23 December 1987 and any future guidance that may be forthcoming.

3.2.1.8.3.3. Structural

Jefferson Parish Lakefront Levee – Reference 45. As constructed the Jefferson Parish Lakefront Levee hurricane protection system consists primarily of geotextile-reinforced earthen levee. Structures include with the levee are pile-founded T-wall tying into the Jefferson Parish/St. Charles Parish Return levee, one vehicular swing gate, re-entrant cantilevered I-wall, and capped cantilevered I-wall at the western end of the protection; one roller gate and capped cantilevered I-wall at PS#4 (Duncan); one roller gate at Williams Boulevard; uncapped cantilevered I-wall and capped cantilevered I-wall with tie-backs at Causeway Boulevard; capped cantilevered I-wall at PS#1(Bonnabel); two swing gates at the Bonnabel boat launch; and capped cantilevered I-wall tying into the hurricane protection at the 17th Street Canal. With the exception of the pumping stations themselves, all structures are in combination with unreinforced levee. Fronting protection at pumping Stations #1 (Bonnabel) and #4 (Duncan) consists of pile-founded floodwalls incorporated into the discharge tubes of the stations and a suppressed air system to prevent water backflow through the pumps when pump operation ceases due to high water levels in Lake Pontchartrain.

A supplemental design memorandum was proposed to cover hurricane protection at pumping Stations #2 (Suburban) and #3 (Elmwood). However fronting protection constructed at both stations is similar to that of PS#1 and PS#4. In addition, pile-founded concrete and sheet pile breakwaters are constructed in Lake Pontchartrain protecting their discharge channels. The existing uncapped I-walls tying the station protection to the levee in these areas are non-Federal.

Structural Design Design Criteria	
Water Elevations	
Water Elevations	*Elevations (feet NGVD)*
Wind Tide Level (Lake Pontchartrain)	11.5
Landside of Floodwall	0.0
Floodwall Gross Grades	
	Elevations (feet NGVD)
T-Wall	17.0 to 22.57 (at Pumping Stations 1 and 4)
I-Wall (along Parish Line Canal)	17.0 to 20.0 (at Pumping Stations)
Unit Weights	
Item	*lb per cu ft*
Water	64.0
Concrete	150.0
Steel	490
Design Loads	
Earth Pressure (lateral) Water loads Wind Loads	50psf

Design Methods. Design of reinforced concrete is in accordance with the requirements of the strength design method of the current ACI Building Code, as modified by the guidelines of "Strength Design Criteria for Reinforced Concrete Hydraulic Structures", ETL 1110-2-265 dated 15 September 1981. The basic minimum 28-day compressive strength will be 3,000 psi, except for prestressed concrete piling where the minimum is 5,000 psi. Pertinent stresses are tabulated below:

Pertinent Stresses for Reinforced Concrete Design	
fc'	3,000 psi
fy (Grade 40)	40,000 psi
Maximum Flexural Reinforcement Ratio	0.25 x Balance Ratio
Minimum Flexural Reinforcement Ratio	200/fy
fc'(for Prestressed Concrete Piles)	5,000 psi
fu (for Prestressing Strands Grade 250)	250,000 psi

I – Type Floodwall.

Loading Cases. In the design of the I-wall, two loading cases were considered

Case I. For unconfined areas along the lakefront with adjacent open water, FS used = 1.5 with static water at the SWL (and no dynamic wave force) and FS used = 1.25 with static water at the SWL and a dynamic wave force

Case II. No water, lateral soil pressure (where applicable)

Note: In Soils and Foundations Investigation and Design Section of GDM, Para 40d(1), it is noted penetration was determined for both "S" and "Q" cases. Factors of safety of 1.5 for static water and 1.25 for static water plus dynamic wave force were applied to the design shear strengths.

T – Type Floodwall.

Loading Cases

Case I	Static water pressure, no wind, impervious sheet pile cutoff, no dynamic wave force
Case II	Static water pressure, no wind, pervious sheet pile cutoff, no dynamic wave force
Case III	Stillwater pressure to elevation 11.5, dynamic wave force, impervious sheet pile cutoff (75% forces used)
Case IV	Stillwater pressure to elevation 11.5, dynamic wave force, pervious sheet pile cutoff (75% forces used)
Case V	No water, no wind
Case VI	No water, wind from protected side (75% forces used)
Case VII	No water, wind from flood side (75% forces used)

Gates

Loading Cases

Case I	Gate closed, stillwater to elevation 11.5, dynamic wave force, impervious sheet pile cutoff (75% forces used)
Case II	Gate closed, stillwater to elevation 11.5, dynamic wave force, pervious sheet pile cutoff (75% forces used)
Case III	Gate open, no wind, truck on protected side edge of base slab
Case IV	Gate open, no wind, truck on flood side edge of base slab
Case V	Gate open, wind from protected side, truck on flood side edge of base slab (75% forces used)
Case VI	Gate open, wind from flood side, truck on protected side edge of base slab (75% forces used)

3.2.1.8.3.3.2. Jefferson Parish / St. Charles Parish Return Levee –Reference 46. As constructed, the Jefferson Parish/St. Charles Parish Return Levee (commonly referred to as the St. Charles Return Levee or more simply, as the Return Levee) hurricane protection system consists of primarily combination levee and pile-founded T-wall tying into the Jefferson

Lakefront Levee hurricane protection system with two reaches of combination levee and capped cantilevered I-wall and one reach of combination levee and uncapped cantilevered I-wall tying into the St. Charles Parish hurricane protection system.

Structural Design

Design Criteria

Water Elevations

Water Elevations	*Elevations (feet NGVD)*
Wind Tide Level (Lake Pontchartrain)	11.5
Wind Tide Level (Parish Line Canal)	9.51 to 11.5
Landside of Floodwall	0.0 to -5.0

Floodwall Gross Grades

	Elevations (feet NGVD)
T-Wall (along Parish Line Canal)	13.0 to 14.5
T-Wall (along Lake Pontchartrain)	20.0
I-Wall (along Parish Line Canal)	11.5 to 13.5
I-Wall (along Lake Pontchartrain)	20.5

Unit Weights

Item	*lb per cu ft*
Water	64.0
Concrete	150.0
Steel	490

Design Loads

Earth Pressure (lateral)	
Water loads	
Wind Loads	50psf

Design Methods. Design of reinforced concrete is in accordance with the strength design method of the current ACI Building Code, as modified by the guidelines of “Strength Design Criteria for Reinforced Concrete Hydraulic Structures”, ETL 1110-2-265 dated 15 September 1981. The basic minimum 28-day compressive strength will be 3,000 psi, except prestressed concrete piling for which the minimum is 5,000 psi. Pertinent stresses are tabulated below:

Pertinent Stresses for Reinforced Concrete Design

fc’	3,000 psi
fy (Grade 40)	40,000 psi.
Maximum Flexural Reinforcement Ratio	0.25 x balance ratio
Minimum Flexural Reinforcement Ratio	200/fy
fc’(for Prestressed Concrete Piles)	5,000 psi
fu (for Prestressing Strands Grade 250)	250,000 psi

I – Type Floodwall.

Load Cases

Along Parish Line Canal

FS = 1.5

Static Water at the top of wall

No dynamic wave force

Along Lake Pontchartrain

FS = 1.25

Static Water to El 11.5

Dynamic wave force applied

Note: In Foundation Investigation and Design Section of GDM, Para 40a(1), it is noted that both “S” and “Q” cases were investigated for a factor of safety of 1.5 with the “S” case governing. No mention was made of a factor of safety of 1.25 being used along Lake Pontchartrain.

T – Type Floodwall.

Load Cases

Along Parish Line Canal

Case I	Static water pressure, no wind, impervious sheet pile cutoff, no dynamic wave force
Case II	Static water pressure, no wind, pervious sheet pile cutoff, no dynamic wave force
Case III	No water, no wind
Case IV	No water, wind from protected side (75% forces used)
Case V	No water, wind from flood side (75% forces used)

Along Lake Pontchartrain

Case VI	Stillwater pressure to El 11.5, dynamic wave force, impervious sheet pile cutoff (75% forces used)
Case VII	Stillwater pressure to El 11.5, dynamic wave force, pervious sheet pile cutoff (75% forces used)

3.2.1.8.3.4. **Sources of Construction Materials**

3.2.1.8.3.4.1. Sheet Pile. Generally, the sheet pile sections specified during advertisement were used for construction. However, sheet pile section substitutions conforming to the minimum required section modulus was allowed, primarily in contracts constructed after 1990. Below, is a table of sheet pile sections for Jefferson Parish, broken down by DM.

Jefferson Parish	
St. Charles Return Levee	
Vicinity of the Airport	PZ-22 (East-West)
	Frodingham 1B (North-South)
Under I-10	Frodingham 1B
Vicinity of Vintage Dr.	PZ-22*
Jefferson Lakefront Levee	
Recurve Wall	PZ-22*
PS#4 (Duncan) Tie-In	**
Williams Blvd. Roller Gate Tie-In	**
Causeway Blvd.	PZ-22 & Concrete Tie-Back System
PS#1 (Bonnabel) Tie-In	BZ-7 & PZ-35*
Bonnabel Boat Launch Swing Gate Tie-In	**
17th Street Canal	
PS#6 to Hammond Hwy	Hoesch 12

*As-advertised – Not confirmed as-built

** Information not located at the time of publication

3.2.1.8.3.4.2. Levee material

3.2.1.8.3.4.2.1. Sources of Borrow Materials. The planned source of borrow for this project is to haul clay fill from the Bonnet Carre' spillway.

3.2.1.8.3.4.2.2. Source of Borrow Materials. No borrow is required since the placement of the T-wall required degrading of the existing levee.

3.2.1.8.4. As-built Conditions

3.2.1.8.4.1. Changes between design and construction (i.e. cross sections, alignment, sheet pile tip el, levee crest el.)

3.2.1.8.4.1.1. DACW29-99-C-0046. Lake Pontchartrain, La., and Vicinity, High Levee Plan, Jefferson Parish Lakefront Levee, Breakwaters at Pump Station No. 2 and No. 3, Jefferson Parish, La.

At Pumping Station No. 2, the breakwater was realigned by moving it 70 feet west to provide adequate outflow channel width. Also, while driving sheet piles for the breakwater they encountered an obstruction, and some of the sheetpiles were cut off.

3.2.1.8.4.1.2. DACW29-98-C-0003. Lake Pontchartrain, Louisiana and Vicinity, Jefferson Parish Lakefront Levee, Landside Runoff Control, Reach 2, Station 167+75 B/L to Station 209+15.2 B/L, Jefferson Parish, Louisiana.

Reviewed Mod Log Report, no applicable modifications or changes found.

3.2.1.8.4.1.3. DACW29-98-C-0012. Lake Pontchartrain, Louisiana and Vicinity, Jefferson Parish Lakefront Levee, Jefferson Parish, Louisiana, Landside Runoff Control, Reach 1

Reviewed Modification Documents, no applicable modifications or changes found.

3.2.1.8.4.1.4. DACW29-98-C-0031. Lake Pontchartrain, Louisiana and Vicinity, Jefferson Parish Lakefront Levee, Landside Runoff Control, Reach 4, B/L Station 337+71 to B/L Station 419+96.77, Jefferson Parish, Louisiana

Reviewed Modification Documents, no applicable modifications or changes found.

3.2.1.8.4.1.5. DACW29-98-C-0068. Lake Pontchartrain, Louisiana and Vicinity, Jefferson Parish Lakefront Levee, Landside Runoff Control, Reach 5, B/L Station 422+00 to B/L Station 509+75, Jefferson Parish, Louisiana

Reviewed Mod Log Report, no applicable modifications or changes found.

3.2.1.8.4.2. Inspection during original construction, QA/QC, state what records are available. See paragraph 3.2.1.5.4.2 New Orleans East Bank for description of how records are kept.

3.2.1.8.4.2.1. DACW29-98-C-0012 – JEF PAR, LKEFRNT LEV, LS/RO, RCH 1

Attached are in-place density tests, pile test reports, and daily material quantities.

3.2.1.8.4.2.2. DACW29-98-C-0031 – JEF PAR LS/RO, RCH4, STA 337-419

No QA/QC Reports found.

3.2.1.8.4.2.3. DACW29-98-C-0068 – L PONT JEF PAR L/S R/O, RCH 5, STA 422 + 509

Attached are concrete field tests, stored material lists, percent complete list, and preparatory inspection reports.

3.2.1.8.4.2.4. DACW29-02-C-0016 – L PONT BDG @ HAMMOND HWY, JEF & ORL PAR

The contractor has included the pile driving records, pile test data, pile logs, progress summary sheets, survey data, in-place density test data, concrete field test and mix design data, and compression test data.

3.2.1.8.4.2.5. DACW29-95-C-0103 – WW – HC, ESTELLE PUMP STA – LP&L POWERLINES, 1^{ST} LFT JEF PAR LA

Attached are moisture analysis records and percent complete lists.

3.2.1.8.5. Inspection and maintenance of original construction

3.2.1.8.5.1. Annual Compliance inspection (i.e. trees, etc.). Annual Compliance inspection (i.e. trees, etc.) **–** Annual inspections were conducted by Operations Division for projects under the Inspection of Completed Works Project for the Jefferson East Bank which is a part of the Lake Pontchartrain and Vicinity Hurricane Protection Project. These inspections, which were general in nature, primarily defined the status of existing project work, and a general condition rating. For the last 6 years, 1998 through 2004, the ratings for the East Jefferson Levee District were "OUTSTANDING" through year 2001, and "ACCEPTABLE" each year thereafter, at which time there was a change in the Project Rating Scale. The project rating scale was then redefined, and "ACCEPTABLE" became the highest rating. There was no specific mention of deficiencies for the hurricane protection system.

3.2.1.8.5.2. Periodic inspections. There are no structures which fall under the Periodic Inspection Program in the Jefferson East Bank area

3.2.1.8.6. Other Features

3.2.1.8.6.1. Brief Description. The primary components of the hurricane protection system for the Jefferson East Bank basin are described above, namely the levees and floodwalls designed and constructed by the Corps of Engineers. However, other drainage and flood control features that work in concert with the Corps of Engineers levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section will describe and present the criteria and pre-Katrina conditions of the interior drainage system, pump stations, and the Mississippi River Flood Protection System. There are currently no non-Corps levees or floodwalls in this basin. Even though the stormwater pump stations are part of the interior drainage system, they are a significant part of the system and warrant their own section.

3.2.1.8.6.2. Pre-Katrina Conditions. According to the local jurisdictions responsible for interior drainage, the storm drain system, interior canals, interior pump stations, outfall pump stations, and outfall canals were in good condition and prepared for high inflows from rainfall prior to August 29, 2005.

The Mississippi River Flood Protection System was in good condition prior to Katrina landfall.

3.2.1.8.6.3. Interior Drainage System.

Overview. The Jefferson East Bank basin contains about 47 square miles and generally slopes south to north from the Mississippi River to Lake Pontchartrain. The area is fully developed. The initial settlement of New Orleans began on the banks of the Mississippi River and progressed northward and westward to the lake. Many features are typical of large urban cities in the United States, and some features that are unique because much of the area is below sea level. Catch basins and inlets collect surface runoff from yards and streets into storm sewers. Excess runoff flows down streets and/or overland to lower areas. Ditches and open canals collect the stormwater and carry it to outfall pump stations that pump the water directly into Lake Pontchartrain, the 17th Street outfall canal, or the Duncan outfall canal. The outfall canals flow into Lake Pontchartrain. No stormwater is pumped into the Mississippi River.

Pump Station No. 6 in Orleans East Bank basin evacuates runoff from Hoey's basin, a portion of the Jefferson East Bank basin. Flood water can overflow from Orleans East Bank into the old Metairie or Hoey's basin area of Jefferson East Bank when flooding reaches a certain elevation .

The entity responsible for local drainage in the Jefferson East Bank basin is Jefferson Parish. The Louisiana Department of Transportation and Development highways are also a part of the local drainage system.

System Components. Local drainage begins with overland flow which follows the ground topography. Figure 5 in Volume VI shows the topographic layout of Jefferson East Bank. The land generally falls from the Mississippi River to Lake Pontchartrain with an elevation difference of about 20-25 feet. A land feature visible on the topographic layout that affects a portion of the local drainage is the Metairie or Gentilly Ridge. It runs east-west between the river and the lake.

Based on land topography and the drainage system, the basin is divided into 105 subbasins. Pump station information is presented in Section 3.2.1.8.6.4 of this volume.

Most of the local drainage is collected by underground storm drains and ditches in this urbanized basin. Photos 1, 2, and 3 show typical inlets and streets. Photos 4 and 5 show storm sewer pipe outfalls into the canals.

The interior canals are open and are grass-lined, concrete-lined, or both (Photos 4 and 5). The interior canals not only collect stormwater from streets and storm sewers and covey it to the pump stations, they also are storage areas that work in conjunction with the pump stations.

The land topography also influences the ditch, canal, and pump station layout. With the relatively flat topography, development sequence, and location of outfall pump stations in this basin, interconnecting canals and interior pump (lift) stations were constructed to accommodate the interior drainage. Photo 6 shows an interior canal and interior pump (lift) station.

Photos 1, 2, and 3. Typical Streets and Inlet – Jefferson East Bank

Photos 4 and 5. Storm Sewer Outfalls into Canals

Photo 6. N. Cumberland Interior Pump at Interior Canal #4, W. Napoleon Ave.

Photo 7. Interior Canal # 3 from Bissonet Dr. near Veterans Blvd.

Photo 8. Elmwood Canal (interior) from Kawanee Ave.

Design Criteria. The current design criterion for Jefferson East Bank is the 10% storm event for all storm drainage system components. Older parts of the stormwater collection system have approximately a 2-year frequency or less capacity. The functional capacity of the interior canals and pump stations is 0.5 inches per hour. It will increase to 0.7 inches per hour after the SELA projects are complete (see status below). Rainfall in excess of this amount goes into temporary storage in the streets, storm sewers, ditches, and canals. There are criteria for new developments to use stormwater detention to offsite downstream impacts.

Where local drainage is considered to need improvement, Jefferson Parish is working to improve the drainage. In some cases, Jefferson Parish and Corps of Engineers are working together on projects, as presented below in the Southeast Louisiana (SELA) Urban Flood Control Projects section.

Southeast Louisiana Urban Flood Control Projects. As a result of the extensive flooding in May 1995, Congress authorized the SELA Urban Flood Control Project with enactment of the Energy and Water Development Appropriations Act for Fiscal Year 1996 and the Water Resources Development Act (WRDA) of 1996 to provide for flood control and improvements to rainfall drainage systems in Jefferson, Orleans, and St. Tammany Parishes. Jefferson Parish is the local, cost sharing sponsor.

The project includes channel and pump station improvements in the three parishes. The channel and pumping station improvements in Orleans and Jefferson Parishes support the parishes' master drainage plans and generally provide flood protection on a level associated with a 10-year rainfall event, while also reducing damages for larger events.

The SELA projects in Jefferson East Bank basin are shown in Figure 20. The work consists of adding capacity to 4 canals and increasing pumping capacity at Elmwood Pump Station #3 and the Suburban Pump Station #2. Prior to Hurricane Katrina, the pump stations were under construction, the Suburban Canal and Canal #3 were complete, Elmwood Canal was nearly complete, and Soniat Canal was partially complete.

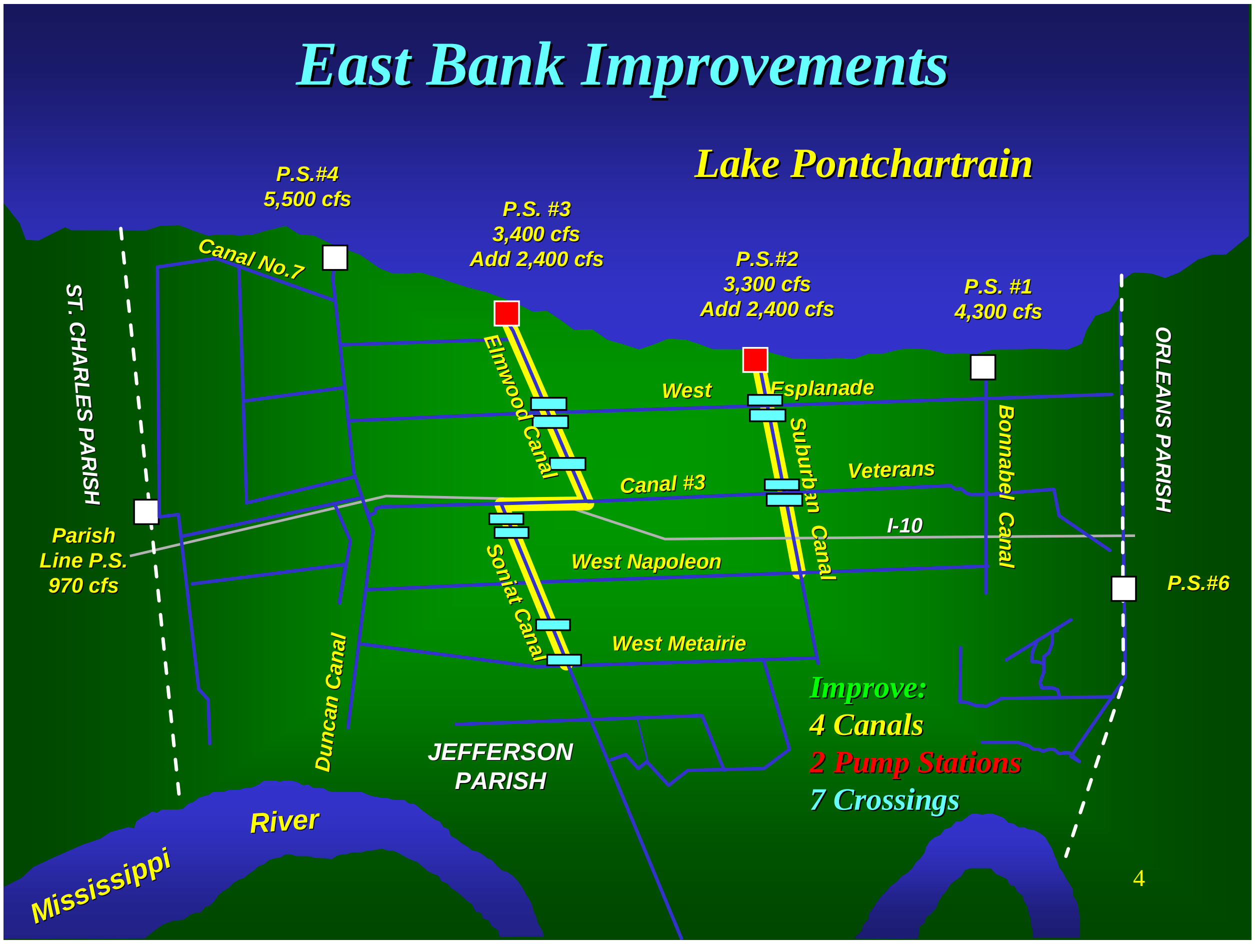


Figure 20. SELA Urban Flood Control Projects in Jefferson East Bank

3.2.1.8.6.4. Pumping stations - Jefferson Parish East Bank. Jefferson Parish is located west of the city of New Orleans and borders the west side of Orleans Parish. Figure 21 is a map of Jefferson Parish with the pump stations that were studied identified by red dots. Jefferson Parish is separated by the Mississippi River into East and West Banks. The East Bank pump stations are connected by a grid of canals. The canals running east and west serve to equalize flow between the major outfall canals, allowing rain water to flow in different directions depending on the rainfall patterns and available capacities at the pump stations. The West Bank is subdivided into sub-basins that, for smaller rainfall events, operate independently. However, over-bank flow does occur between adjacent sub-basins for a 10-year event. This report examined 6 pump stations on the East Bank with a total of 36 pumps and 17 pump stations on the West Bank with a total of 65 pumps. The locations of the pump stations were verified by Global Positioning System (GPS) and/or by using Google Earth Pro. The GPS coordinates were then input into Microsoft Streets and Trips (shown below).

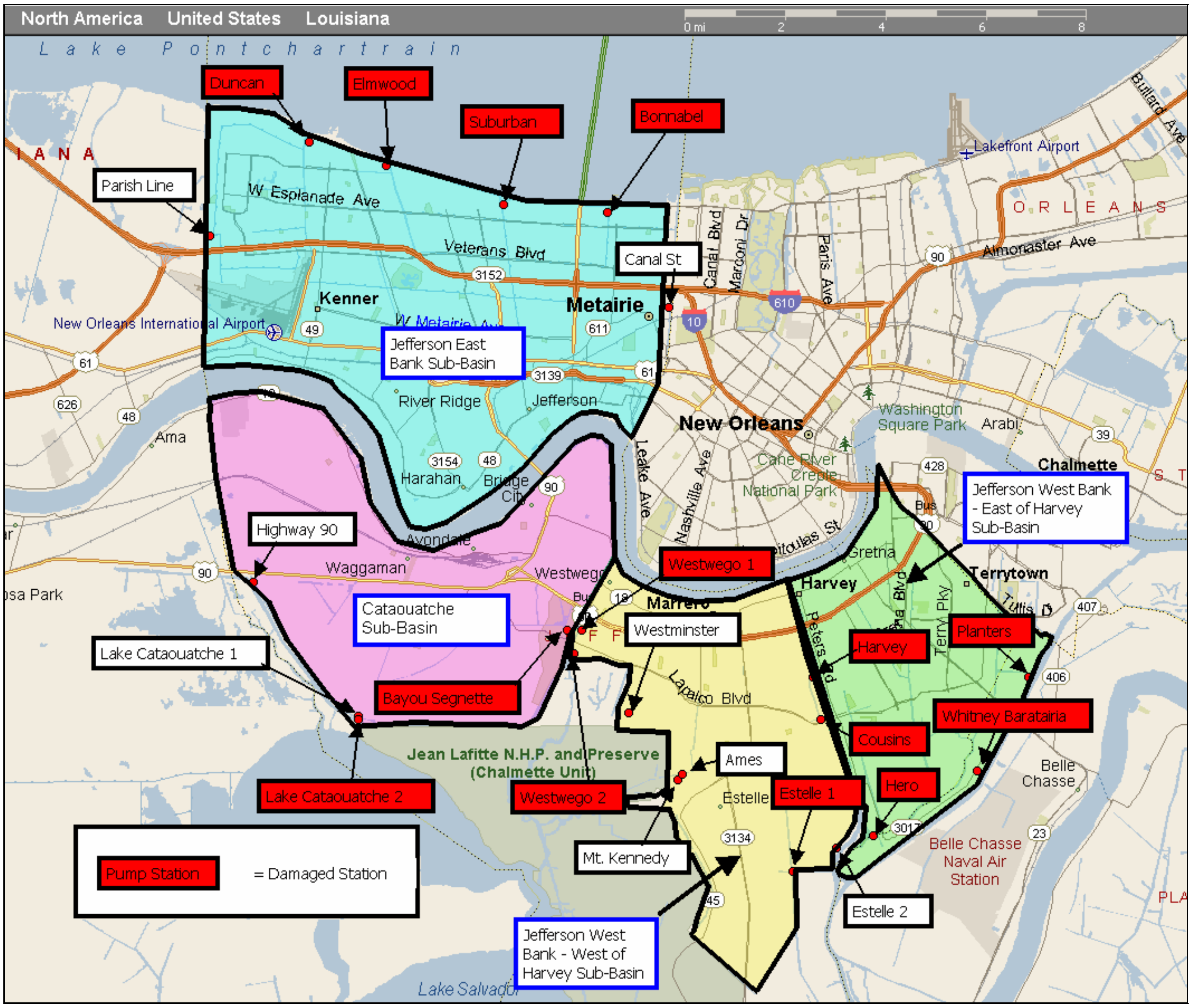


Figure 21. Jefferson Parish Pump Station Locations

Table 23 contains information about each individual pump at each of the examined pump stations in Jefferson Parish. The list is composed of information that was collected in the field. Not all information was available for each pump and was left blank or highlighted.

Table 23
Summary of Jefferson Parish Pump Stations by Drainage Basin

Basin	East Bank	Cataouatche	West Bank – West of Harvey	West Bank-East of Harvey	Total
Number of pump stations	6	4	9	3	22
Number of pumps	36	24	29	15	104
Total rated capacity (cfs)	20,662	3,346	10,695	9,958	44,661
Estimated cost of damages	$558,000	$3,000	$136,000	$61,000	$758,000

East Bank Drainage Basin. The East Bank Drainage Basin is bordered by Lake Pontchartrain on the north, and the Mississippi River on the south. The drainage system includes the surrounding bodies of water, as well as Bonnabel, Suburban, Elmwood, Duncan, Canal, and17th Street Canals. The basin has six significant pump stations, which are summarized below. Volume VI provides more detailed information.

Bonnabel
Intake location: .... Bonnabel
Discharge location: .... Lake Pontchartrain
Nominal capacity: 3750

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	300	1986	Electric 60 HZ	Vertical
2	300	1986	Electric 60 HZ	Vertical
3	1050	1986	Diesel	Horizontal
4	1050	1986	Diesel	Horizontal
5	1050	1986	Diesel	Horizontal

Suburban
Intake location: .... Suburban
Discharge location: .... Lake Pontchartrain
Nominal capacity: .... 5155 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	1050	1983	Diesel	Horizontal
2	1050	1970	Diesel	Horizontal
3	55	1970	Electric 60 HZ	Vertical
4	300	1970	Diesel	Vertical
5	300	1970	Diesel	Vertical
6	300	1983	Electric 60 HZ	Vertical
7	1050	2005	Diesel	Horizontal
8	1050	2005	Diesel	Horizontal

Elmwood

Intake location: ....Elmwood Canal
Discharge location: ....Lake Pontchartrain
Nominal capacity: ....5912 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	303	1981	Diesel	Vertical
2	303	1981	Diesel	Vertical
3	550	1981	Diesel	Vertical
4	550	1981	Diesel	Vertical
5	550	1981	Diesel	Vertical
6	550	1981	Diesel	Vertical
7	303	1981	Diesel	Vertical
8	303	1981	Diesel	Vertical
9	1250	2004	Diesel	Horizontal
10	1250	2004	Diesel	Horizontal

Duncan

Intake location: ....Duncan Canal
Discharge location: ....Lake Pontchartrain
Nominal capacity: ....4800 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	300	1986	Electric 60 HZ	Vertical
2	300	1986	Electric 60 HZ	Vertical
3	1050	1986	Diesel	Horizontal
4	1050	1986	Diesel	Horizontal
5	1050	1986	Diesel	Horizontal
6	1050	1986	Diesel	Horizontal

Parish Line
Intake location: .... 16th & 17th Street Canal
Discharge location: ....Lake Pontchartrain
Nominal capacity: ....885 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	295	1987	Electric 60 HZ	Vertical
2	295	1987	Electric 60 HZ	Vertical
3	295	1987	Electric 60 HZ	Vertical

Canal Street
Intake location: .... Canal
Discharge location: ....17th Street Canal
Nominal capacity: ....160 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	40	1998	Electric 60 HZ	Vertical
2	40	1998	Electric 60 HZ	Vertical
3	40	1998	Electric 60 HZ	Vertical
4	40	1998	Electric 60 HZ	Vertical

3.2.1.8.6.5. Levees and floodwalls

3.2.1.8.6.5.1. MRL. MRL levees and floodwalls are addressed in Paragraph 3.2.1.5.6.4.1, New Orleans East Bank MRL. There are no floodwalls that are part of the MRL Project.

3.2.1.8.6.5.2. Non-Corps. Several local interest and/or private levees are located within the project area. No design criteria for these levees have been made available to the Corps.

3.2.1.9. St. Charles East Bank

3.2.1.9.1. Introduction.

Hurricane Protection Features St. Charles Parish East Bank

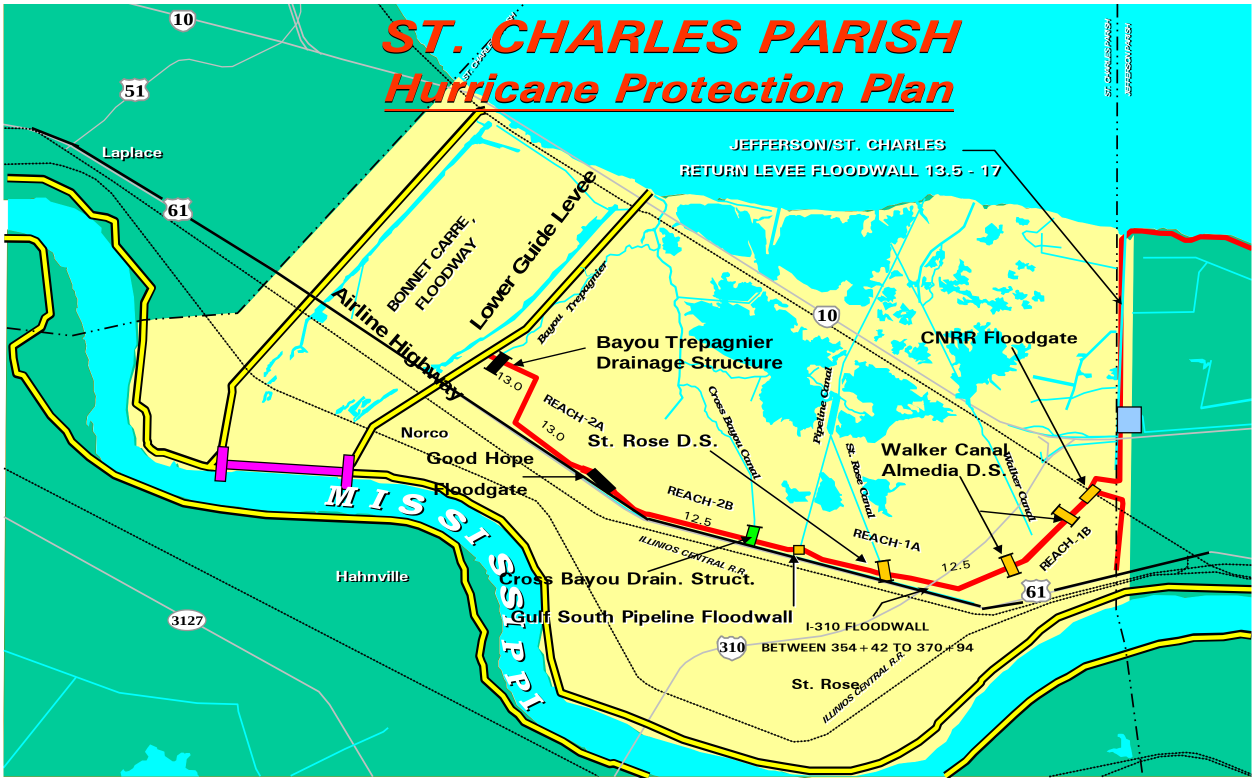


Figure 22.

Features. The Lake Pontchartrain and Vicinity hurricane protection project in St. Charles Parish consists of approximately 10 miles of levees and floodwalls north of Airline Highway (US Hwy. 61) from the Bonnet Carré Spillway East Guide Levee to the Jefferson-St. Charles Parish boundary at the New Orleans Airport East-West runway terminus. Five drainage structures are included to allow intercepted drainage to flow north into the adjacent bayous and drainage canals and ultimately into Lake Pontchartrain. Floodwalls are located at I-310, Goodhope, and at the Gulf South Pipeline Crossing. A double track railroad floodgate is located near the eastern end of the project where the Canadian National Railroad crosses through the protection system.

Table 24
Summary of St. Charles Parish East Bank Hurricane Protection Features

Exterior levee and floodwall (I wall and T-wall)	10 miles
Drainage Structures	5
Highway Closure Structures	1
Railroad Closure Structure	1

3.2.1.9.2. Pre-Katrina - The levees in the St. Charles Parish portion of the Lake Pontchartrain and Vicinity project are under construction. All of the levees have first lift construction completed. Prior to Hurricane Katrina, plans had been developed to construct the second lift of Reach 2B. Lack of funding had prevented construction of this contract for three years. Plans were also being developed for Reach 2A and initial surveys had been taken for Reach 1B. Pre-Katrina funding levels precluded completion of P&S development.

The project in St. Charles Parish includes five gravity drainage structures. These are all completed. A railroad floodgate for the Canadian National Railroad is currently nearing completion. The construction was performed by the New Orleans International Airport for the Corps of Engineers and the Pontchartrain levee district as a part of the rehabilitation of the airport's east-west runway. This was required because of the position of the floodgate near the end of the runway. Floodgate construction by the Corps would have required the runway to shut down for at least six months due to clearance and safety issues. Since the Corps could not fund the floodgate construction, the airport elected to fund the work while the runway was shut down for rehabilitation, thus avoiding another shut down when corps funding was secured. All other floodgate and floodwalls in St. Charles Parish were completed with the exception of the I-310 floodwall. Pre-Katrina, the I-310 floodwall consisted of a single row of sheet piling. Ultimately, the sheet piling will form a base for a concrete I-wall and T-wall combination. These have not been constructed as yet. A review is underway to determine if the levees floodwalls and structures will have to be redesigned based on the results of the Interagency Performance Evaluation Team analysis and based on a reanalysis of design storm calculations. Additional contracts may be required as a result of this analysis.

3.2.1.9.3. Design Criteria and Assumptions - Functional design criteria

3.2.1.9.3.1. Hydrology and Hydraulics. For St. Charles East Bank, the design hurricane characteristics utilized in the design memoranda are shown in Table 25; the design track is shown on Figure 23. The maximum wind speed was computed using the same equations as for Orleans East Bank. For each project area, the track and forward speed were selected to produce maximum wind tide levels.

Table 25
Design Hurricane Characteristics

Location	Track	CPI, Inches	Radius of Maximum Winds, Nautical miles	Forward Speed, Knots	Maximum Wind Speed[1], MPH	Direction of Approach
Lake Pontchartrain South Shore	A	27.6	30	6	100	South

[1] Windspeeds represent a 5 minute average 30 feet above ground level

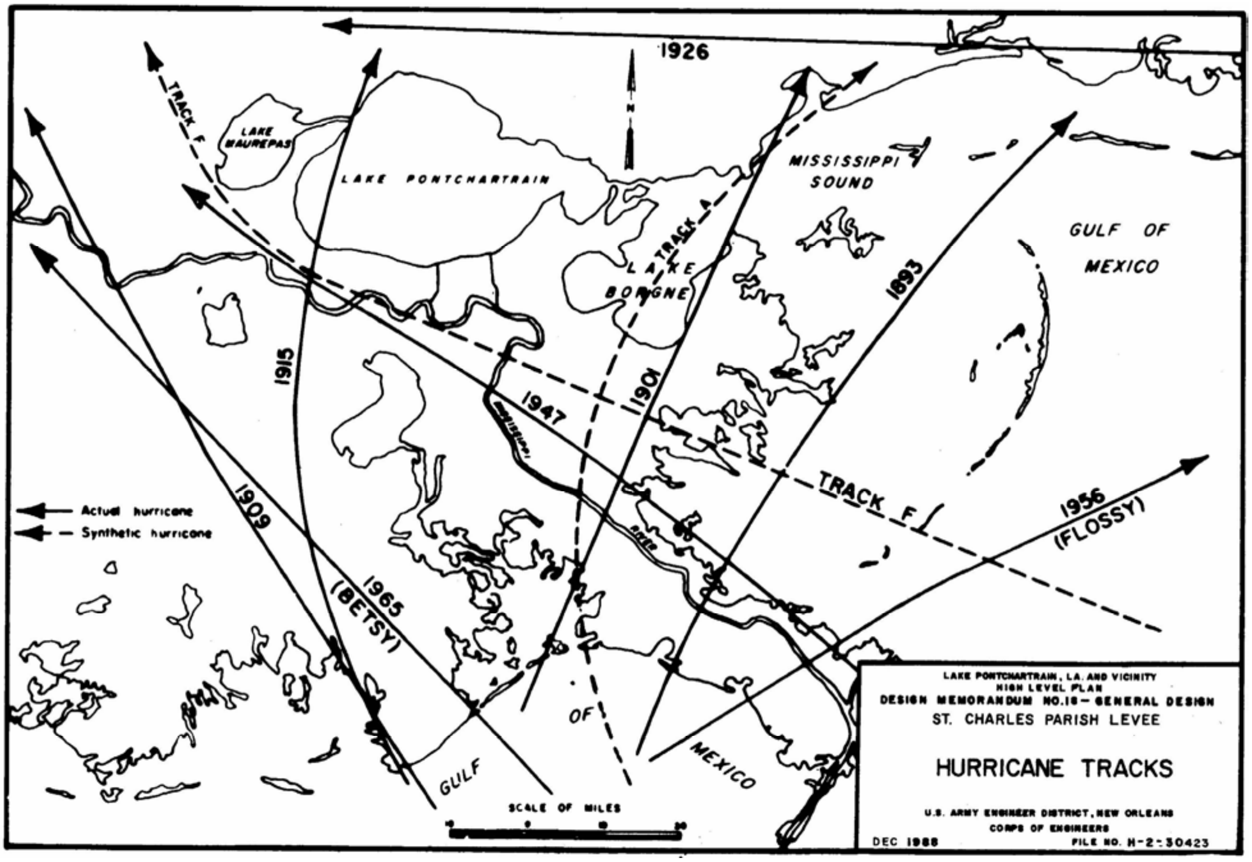


Figure 23. Hurricane tracks, St. Charles Parish Levee

3.2.1.9.3.1.1. Surge. Surge elevations were computed using the same methodology as used for the Lake Pontchartrain lakefront for Orleans East Bank, with an additional step. The shoreline of Lake Pontchartrain was considered the location of the surge reference line. The same methodology used to adjust surge heights for the Chalmette Extension was applied, with a modification of the drop-off rate away from the surge reference line; for the swamp condition, the average drop-off rate applied was 1 foot per 2 miles. Table 26 shows the wind tide level at the surge reference line and at the levee location.

Table 26
Wind Tide Levels

Location	Wind Tidelevel, surge reference line, FT NGVD	Wind Tide level at levee location, FT NGVD
NORCO to New Sarpy	13.0	11.0
New Sarpy to Pipeline Canal	12.7	10.5
Pipeline Canal to Almedia	12.1	10.0
Almedia to T. L. James	11.8	10.0
T. L. James to Kenner	11.5	10.0

3.2.1.9.3.1.2. Waves. Waves were not considered a factor for the protection structure. The levee is fronted by a wooded swamp that would affect the translation of the waves from Lake Pontchartrain toward the levee. A freeboard of 2 ft was recommended. Future subsidence and sea level rise were considered in the analysis. By the year 2040, the changed conditions fronting the levee could require a wave berm to be added to the flood side of the levee and raising the levee elevation one foot.

3.2.1.9.3.1.3. Summary. Table 27 contains maximum surge or wind tide level, wave, and design elevation information.

Table 27
Wave Runup and Design Elevations (transition zones not tabulated – governing DM is listed)

Location	DM	Average Depth of fetch, ft	Significant Wave Height Hs, ft	Wave Period, T, sec	Maximum Surge or Wind Tide Level, Ft	Runup Height Ft	Free board, Ft	Design Elevation Protective Structure, ft
NORCO to New Sarpy	DM18, Feb 1989	-	-	-	11.0 NGVD	-	2.0	13.0 NGVD
New Sarpy to Pipeline Canal	DM18, Feb 1989	-	-	-	10.5 NGVD	-	2.0	12.5 NGVD
Pipeline Canal to Almedia	DM18, Feb 1989	-	-	-	10.0 NVGD	-	2.0	12.0 NGVD
Almedia to T. L. James	DM18, Feb 1989	-	-	-	10.0 NGVD	-	2.0	12.0 NGVD
T. L. James to Kenner	DM18, Feb 1989	-	-	-	10.0 NGVD	-	2.0	12.0 NGVD

3.2.1.9.3.1.4. Interior Drainage. The hurricane protection system would have an impact on interior drainage. Five gated culverts would be constructed along the levee, as shown in Figure 24. The culverts were designed to have sufficient capacity to evacuate runoff from high intensity storms without excessive overflow of lands and to provide for prompt evacuation of impounded runoff following periods of gate closures.

The basis for design was a rainfall event with a return period of 25 years and a duration of 24 hours, coincident with a Lake Pontchartrain stage of 1.6 ft NGVD. The lake stage was based on a 50 percent duration elevation of 1.2 ft NGVD with 0.4 ft tidal influence. The maximum headwater, 2.9 ft NGVD, was based on an interior sump damage elevation of 2.4 ft NGVD and an assumed loss of 0.5 ft through the Airline Highway embankment. Design head was 1.3 ft.

Runoff data was developed using the Corps of Engineers software, HEC-1, Flood Hydrograph Package (Revised 1985). Infiltration rates were calculated using the Soil Conservation Service (SCS) curve number and the percent of the area that is impervious. Synthetic rainfall values from U.S. Weather Bureau Technical Paper No. 40, "Rainfall Frequency Atlas of the United States," were used. Storage curves and flow lengths were developed from topographic maps.

A submerged outlet condition was assumed. Flow through the culvert was computed by use of the equation

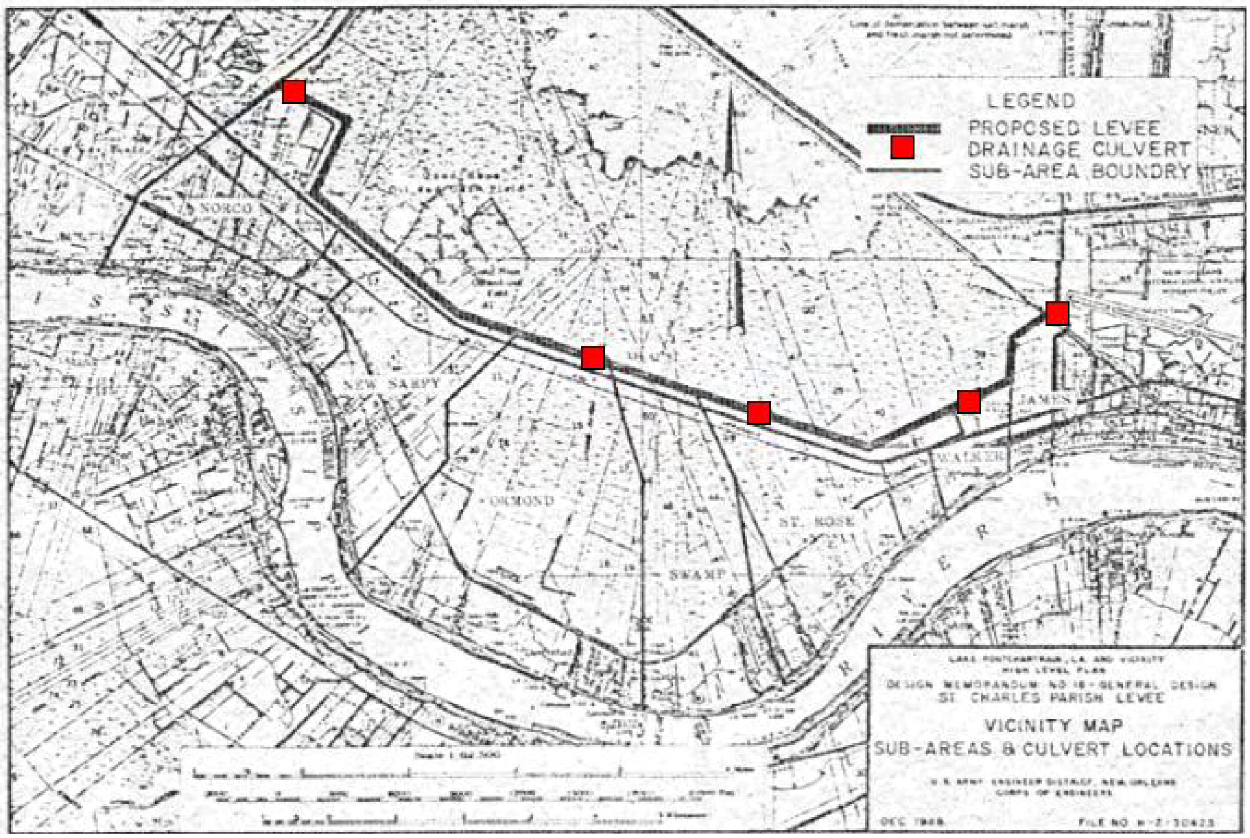


Figure 24. Culvert locations

$$Q = CA(2gh)^{0.5}$$

where

Q = Discharge, cfs

C = coefficient of discharge, 0.80

A = clear structure area, square ft

g = acceleration due to gravity

h = difference in inside and outside water levels, ft

The pertinent design data is shown on Table 28.

Table 28
Design Data — Gated Culverts

Subdrainage Area Characteristics							Culvert Design Data					
Subdrainage Area	Area, Sq Mi	SCS CN	% Area Impervious	Flow Length, Ft	Tc Hour	Rainfall Excess, In	Culvert Invert, FT NGVD	Number of Culverts	Size, ft	Maximum Design Inflow, CFS	Maximum Design Outflow, CFS	Headwater Stage, FT NGVD
NORCO	3.7	91	73	19,000	4.9	9.2	-3.5	3	5 x 5	1,369	532	2.8
NSARPY	2.7	89	70	15,800	4.3	9.2	-5.3	6	6 x 6	4,001	1,441	2.7
ORMDES	6.3	88	69	21,300	8.1	9.3						
SWAMP	2.9	86	81	18,000	3.3	9.6						
STROSE	3.5	88	69	16,600	5.0	9.4	-5.0	2	6 x 6	1,263	510	2.8
WALKER	0.5	88	61	4,900	4.5	9.1	-3.5	1	4 x 4	300	111	2.8
JAMES	0.5	92	66	4,300	4.4	9.3	-3.0	1	4 x 4	304	111	2.8

3.2.1.9.3.2. Geotechnical.

3.2.1.9.3.2.1. St. Charles Parish Frontline Levee. St. Charles Parish Front Line Levee consists of 5.7 miles of earthen levee except in the vicinity of the Bayou Piquant Drainage Structure where I-walls and T-walls were used to tie to the structure.

3.2.1.9.3.2.1.1. Geology. The project area is located within the Gulf Coastal Plain. Specifically, the area is located at the western edge of the Pontchartrain Basin between the alluvial ridge of the present Mississippi River and the southwest shoreline of Lake Pontchartrain. Dominant physiographic features of the area are the marshes, the natural levees of the Mississippi River, and Lake Pontchartrain. Relief in the project area is slight with a maximum of about 12 feet between the natural levee ridge of the present Mississippi River and the marshes adjacent to Lake Pontchartrain. Maximum elevations of 12 feet are found along the natural levee ridges of the present Mississippi River. Minimum elevations of mean sea level or slightly below are found in the marsh area adjacent to Lake Pontchartrain.

3.2.1.9.3.2.1.2. Foundation Conditions. The subsurface consists of Recent deposits varying in thickness from about 50 feet between Stations 25+00 and 130+00 to over 100 feet between Stations 155+00 and 298+61.07 (the western limit of the project). Underlying the Recent are sediments of Pleistocene (Prairie formation) age. Generally, the Recent consists of a surface layer, 12 feet to 20 feet thick, of very soft marsh clays with peat and organic matter and have moisture contents averaging about 360 percent. At the western end of the project, the marsh deposits are overlain by a surface veneer of fill material consisting primarily of silts and lean clays. The marsh deposits are underlain by very soft lacustrine clays, interspersed with lenses and layers of silt and shell fragments, and have moisture contents of about 60 to 80 percent. The lacustrine deposits vary in thickness from about 36 feet between Stations 30+00 and 130+00 to at least 60 feet west of Station 130+00. From Station 20+00 to 141+00, the lacustrine deposits are underlain by stiff to very stiff Pleistocene clays with interspersed lenses of silt.

3.2.1.9.3.2.1.3. Field Exploration. Undisturbed borings 5 inches in diameter extending to approximate Elev. -80.0 were made at four locations along the levee baseline (Stations 5+00, 105+00, 205+00, and 296+50). General-type core borings, 1-7/8-inch I.D., extending to approximate Elev. -60.0 were made at-ten locations along the levee baseline (Stations 1+85, 30+00, 55+00, 80+00, 130+00, 155+00, 180+00, 230+00, 255+00, and 280+00). Twelve general-type core borings, 1-7/8-inch I.D., extending to approximate Elev. -70.0 feet were made in the bottom of Lake Pontchartrain in the recommended borrow area opposite the levee alignment.

3.2.1.9.3.2.1.4. Seepage. Not addressed.

3.2.1.9.3.2.1.5. Pile Foundations. Twelve-inch square prestressed concrete piles will be used to support the T-type walls and the drainage structure. Design compression and tension capacities versus tip elevations were developed for treated timber and 12-inch square concrete piles. Design data were determined for the (Q) and (S) shear strengths. In compression, a factor of safety of 1.75 was applied to the shear strengths and a conjugate stress ratio (K_0) = 1.0 was used in the (5) case for determining the normal pressure on the pile surface. In tension, a factor of safety of 2.0 was applied to the shear strengths and a conjugate stress ratio (Ka) = 0.70 was used in the (5) case. Further, pile design loads versus tip elevations are presented for 16-inch square concrete piles for the (5) case only, inasmuch as the (S) case governed for design. The stability of the drainage structure relative to failure of the soils foundation for the hurricane condition with water to Elev. 10.5 feet on the flood side and to Elev. -1.5 feet on the protected side was determined using the design (Q) shear strengths.

3.2.1.9.3.2.1.6. Slope Stability.

a. **Levees.** The slopes and berm distances for the recommended levee, using cross sections representative of existing conditions along the levee alignment, were designed to resist the following conditions: project hurricane still water level (Elev. 10.0 feet from Stations 0+00 to 140+00 and Elev. 10.5 feet from Stations 140+00 to 298+61.07) and assumed failure toward the landside. The stabilities of the first lifts were determined by the method of planes using the design (Q) shear strengths and applying a minimum factor of safety with respect to strength of approximately 1.3. The stabilities of subsequent lifts were determined by the method of planes utilizing an assumed gain in shear strength based on the consolidated-undrained (R) test trend, i.e., $S = C + P \tan 13°$, where S = design shear strengths, C = cohesion based on (Q) test, P = increase in intergranular pressure in the strata (based on the percent consolidation at the time) due to the overburden, and 13° = friction angle based on the (R) tests.

b. **Stream Closures.** The slope and berm distances for the recommended first lift of the stream closures were designed for water at Elev. 0.0 feet and to resist assumed failure towards the flood side for the construction period. Even though the SPH could occur during construction, it would be more economical to repair the failure, if one should occur, than to build the closure wide enough to provide a factor of safety of 1.3 with the water at Elev. -6.0 feet on the flood side. However, the ultimate stream closure configuration was designed for the most critical design hurricane condition, i.e., water at Elev. -6.0 feet on the lakeside and the prevention of assumed failure towards the lakeside.

3.2.1.9.3.2.1.7. I-Walls.

a. The stability and required penetration of the steel sheet pile below the ground surface were determined by the method of planes using the consolidated-drained (5) shear test results, i.e., C = 0, = ϕ_a = 23°. A factor of safety of 1.25 was applied to the friction angle as follows:

$$\phi_d = \tan^{-1}\left(\frac{\tan\phi \text{ available}}{\text{Factor of Safety}}\right)$$

The developed friction angle was used to determine K_A and K_P values as follows:

$$K_A = \tan^2\left(45° - \frac{\phi_d}{2}\right); K_P = \tan^2\left(45° + \frac{\phi_d}{2}\right)$$

Using K_A and K_P values and the effective unit weights, net horizontal water and earth pressure diagrams were determined for movement toward each side of the sheet pile. The summation of the horizontal forces on the protected side was equated to the summation of the horizontal forces on the flood side for various tip penetrations. At these various tip penetrations, summations of overturning moments were determined. The required depths of penetration were determined, as those where the summation of moments was equal to zero. Sufficient (Q) stability analyses were performed to confirm that the (S) case governed for design.

b. The results of tidal hydraulic analyses indicate that the floodwalls will be subjected to the pressure and forces imparted by-broken and breaking waves. In the stability analyses, the wave effect was applied as a line force acting through the centroid of the dynamic wave pressure distribution diagram. The static water pressure diagram resulting from wave action was considered effective only to the top of the impervious clay layer, inasmuch as the period of time the wave will exist is too short to allow water pressures to become effective in the impervious soil layer. The aforementioned analyses were used for design. However, tip penetrations were also determined for the static water pressure diagram resulting from wave action effective through the clay fill to the tip of the sheet pile.

3.2.1.9.3.2.1.8. T-Wall. Inverted T-type floodwalls on bearing piles will be utilized in lieu of I-type floodwalls where the height of the wall above ground and the magnitude of the dynamic wave force render the I-type floodwall impracticable. A steel sheet pile cutoff will be used beneath the T-wall to provide protection against seepage. The drainage structure will be a concrete structure supported on prestressed concrete bearing piles with steel sheet pile cutoff.

3.2.1.9.3.2.1.9. Erosion Protection. Erosion protection will not be provided for damage from hurricane flood stages because of the relatively short duration of hurricane flood stages and the resistant nature of the clayey soils. However, because of the frequency and duration of waves generated in Lake Pontchartrain by other than hurricane winds and because of the proximity of the levee to Lake Pontchartrain, erosion protection will be provided for damage which could occur from waves generated by other than hurricane winds. The erosion protection for the levee will consist of 2 feet of riprap placed on 0.75 foot of shell extending from Elev. 6.5 feet to Elev. -2.8 feet along the lakeside slope of the levee. In addition to the levee slope protection, erosion protection will also be provided on the flood side slopes of stream closures and will extend from Elev. 0.0 feet to the bottom of the streams. Further, 2 feet of riprap on 1 foot of shell will be placed 20 feet on each side of the floodwall and will extend from Elev. 8.0 feet at the earth levee to Elev. -6.0 feet at the drainage structure.

3.2.1.9.3.2.1.10. Review Comments. None provided with Reference No. 94.

3.2.1.9.3.2.2. St. Charles Parish North of Airline Highway (reference Nos. 48, 48). The St. Charles Parish north of Airline Highway consists of approximately 10 miles of levee. Approximately 9 miles of the levee will be full earthen levee sections with geotextile reinforcement over a sand working base. Approximately 1 mile of unreinforced earth levee along with a short reach of I-wall and T-wall under the I-310 Interchange was used.

3.2.1.9.3.2.2.1. Geology. The project site is located on the Deltaic Plain portion of the Mississippi River Alluvial Plain. Specifically, the project is located on the southern edge of the Lake Pontchartrain Basin and east of the Mississippi River. Dominant physiographic features include natural levee ridges, crevasse splay deposits, marsh, swamps and lakes. Elevations vary from approximately +10 to +15 feet NGVD along the natural levee of the Mississippi River to Elev. 0 ft. NGVD in the backswamp and marsh areas.

3.2.1.9.3.2.2.2. Foundation Conditions. Engineering properties of the sediment beneath the project vary greatly. Generally, the subsurface consists of Holocene deposits varying in depth from, 55 feet to 80 feet and underlain by Pleistocene deposits. Specifically, from Station 0+00 to Station 27+00, the Holocene is between 55 and 80 feet thick and from Station 27+00 to Station 505+00, the Holocene sequence is comprised of marsh-swamp deposits throughout the project except between Station 0+00 and Station 205+00 and between Station 360+00 and Station 480+00, where natural levee deposits overlie the marsh-swamp deposits. The marsh-swamp deposits are characterized by high wood and organic material contents and high water contents. Underlying the marsh-swamp deposits is a sequence of deposits which include crevasse-splay deposits, interdistributary deposits and lacustrine deposits which vary in thickness. From Station 0+00 to Station 240+00, this sequence is between 12 and 27 feet thick and from Station 240+00 to Station 505+00, the sequence is between 30 and 40 feet thick. These materials consist of clays, silts and sands which exhibit lower wood and organic material contents and lower water contents than the deposits above or below. Beneath the sequence of crevasse-splay, interdistributary and lacustrine deposits, prodelta clays are found from Station 0+00 to Station 310+00 and vary in thickness between 5 and 20 feet. The bottom of the Holocene sequence is formed by Bay-sound deposits which vary in thickness from 5 to 20 feet and extend throughout the project. Underlying the Holocene in the project are the Pleistocene lean clays, fat clays and silty sands. These Pleistocene deposits are oxidized and exhibit a marked decrease in water content when compared to the overlying Holocene deposits. Moreover, the Pleistocene deposits, which vary in consistency from stiff to very stiff, normally yield unconfined compressive strengths that exceed those in the Holocene deposits.

3.2.1.9.3.2.2.3. Field Exploration.

a. A total of eleven (11) 5-inch diameter undisturbed and forty-six (46) general type soil borings were taken and tested by the Corps of Engineers for the design of the St. Charles project. The general type borings, 1-GSC through 48-GSC (note borings 4-GSC & 42 GSC were not taken), extend to an elevation between Elev. -60 feet and Elev. -70 feet NGVD; and 11 undisturbed soil borings, 1-SCU thru 11-SCU, extend to an approximate elevation of -80 feet NGVD.

b. Twenty-eight (28) general-type borrow borings were taken in the Bonnet Carré Spillway to classify proposed borrow material. Prior to preparation of plans and specifications, general type borrow borings will be taken in the Mississippi River to locate the required sand source.

3.2.1.9.3.2.2.4. Seepage.

a. **Seepage Blanket.** A seepage blanket over the landfills is required. A minimum three (3)-foot thick clay cover was used for the seepage blanket. The required seepage blanket length was analyzed by Lane's Weighted Creep Ratio Method utilizing a LWCR valve of 8.5. Lane's Weighted Creep Ratio is the ratio of the weighted creep distance to maximum differential head. The weighted creep distance was calculated as one-third (1/3) of the horizontal creep path distance.

b. **Seepage Cutoff for I-Walls and T-Walls.** The required penetration for seepage cutoff was analyzed by utilizing Lane's Weighted Creep Ratio Method. The weighted creep-distance was calculated as the sum of the vertical creep path distance plus one-third of the horizontal path distance. Lane's Weighted Creep Ratio is the ratio of the weighted creep distance to the maximum differential head. The deeper penetration of the two analyses (stability and creep ratio) was selected as the recommended tip elevation of the sheet pile. The cantilever stability analyses governed the penetration.

3.2.1.9.3.2.2.5. Pile Foundation. A pile foundation structure was the recommended alternative. T-walls would also be founded on piles.

a. Typical ultimate compression and tension pile capacities versus tip elevations were developed for 12 and 14 inch square prestressed concrete piles and for HP 12x53 steel H-Pile. Overburden stress in the soft clay material was limited to D/B-15 in the (S) case. Negative skin friction (Q) case was calculated for the piles when stability berms are constructed above the T-wall base. The design parameters used are shown in Tables 29 and 30.

Table 29
Concrete Piles

	Q-case						S-case					
	ϕ	K_c	K_t	N_c	N_q	δ		ϕ	K_c	K_t	N_c	N_q	δ
Clay	0°	1	1.7	9	1.0	0	Clay	23°	1	0.7	0	10.0	23°

Table 30 Steel H-Piles													
	Q-case							**S-case**					
	ϕ	**K_c**	**K_t**	**N_c**	**N_q**	**δ**		**ϕ**	**K_c**	**K_t**	**N_c**	**N_q**	**δ**
Clay	0°	1	1	9	1	0	Clay	23°	1	0.7	0	10.0	15°

The recommended pile tip elevations for cost estimating purposes are based on applying a factor of safety of 2.0 in both compression and tension since pile loads tests will be performed. For piles with negative skin friction, the following equation should be used:

$$Q(All) \qquad \frac{Quit}{- F.S.} - NEG\ Skin\ Friction$$

b. For T-walls with positive resultant forces determined from the deep-seated stability analysis, the design loads plus these additional loads must be carried by the piles below the critical slip plane. Positive resultant earth forces are applied to the sheet pile cutoff wall beneath the structure. The cutoff wall is, in turn, designed to transfer the earth loads to the base of the structure and thus to the pile foundation. From the positive resultant forces, a net pressure diagram is applied to the sheet pile from the base of the structure to the critical slip plane elevation. The pressure diagram was calculated by taking the difference between the resultant force at the base of the structure and the resultant force at each stratum.

c. During construction, test piles will be driven and load tested in the project area. The results of the pile load tests will be used to determine the length of the service piles.

3.2.1.9.3.2.2.6. Slope Stability.

a. The stability of the levee was determined by the-LMVD Method of Planes using the design (Q) shear strengths with hydraulic loading. To overcome the weak foundation soil strengths, geotextile reinforcement was introduced to stabilize the levee section. The required geotextile tensile strength for a factor of safety of 1.3 was based on the <u>larger value</u> of the following two analyses:

(1) From the LMVD Method of Planes analyses, the following equation was used to determine the critical wedges which required the maximum tensile strength for the geotextile:

$$T = \frac{\left(D_a - D_p\right)E.S. - \left(Ra - Rb - Rp\right)}{12}$$

Where T = tensile strength in lbs/in. at 5 percent strain and less than 40 percent of ultimate F.S. = factor of safety.

b. Once the critical wedges were determined by the LMVD Method of Planes, these failure surfaces were checked by the Spencer method with the PC-SLOPE microcomputer program. The

Spencer method considered the location of the Geotextile in determining the required geotextile tensile strength. For geotextile tensile strength requirements larger than 1600 lb/in, a two-layer system was used with two-thirds (2/3) of the required tensile strength in the bottom layer and one-third (1/3) in the upper layer with a minimum of 3 feet of fill between and over the fabric layers.

The embedment length (L) of the fabric for pull-out was calculated by the following equation:

$$L = \left[\frac{\left(D_a - D_p\right) E.S. - \left(Ra - Rb - Rp\right)}{\left(\lambda_1 h_1 \tan\Phi_1 + C_1\right) + \left(\lambda_2 h \tan\Phi_2 + C_2\right)} \right]$$

$_1$ denotes soil parameter above geotextile
$_2$ denotes soil parameter below geotextile

"L" was measured from the critical active wedge into the anchorage zone and an equal length was placed in the active wedge zone. Also, the bottom layer of fabric was extended gait the anchorage embedment requirement to attain a factor of safety of 1.3 of the levee berm in certain cases.

For the pipeline crossings, the levee was designed by the LMVD Method of Planes for a minimum factor of safety of 1.3 without the geotextile reinforcement, and the reinforcement was used to attain a factor of safety of 1.5 for the pipeline crossings.

c. **Bearing Capacity of the Geotextile Reinforced Levee.** Since the reinforced embankment acts as a unit, overall bearing capacity has to be checked to insure that the embankment will not punch into the foundation soil. All geotextile reinforced sections have been analyzed, based on a report by R. K. Rowe and K. L. Soderman for reinforced levees, and were found to be adequate. The Rowe and Soderman report presents design bearing capacity factors for rigid footings. The design bearing capacity factors consider the effect of increasing undrained strength with depth as well as the effect of the relative thickness of the soil deposit.

d. **Shear Stability of Unreinforced Earthen Levee and I-wall Levee.** The stability of the levee and levee with I-wall was determined by the LMVD Method of Planes using the design (Q) strengths with appropriate hydraulic loading and was designed for a minimum factor of safety of 1.3.

3.2.1.9.3.2.2.7. I-Walls. The required penetration for the stability of the sheet pile wall was determined by the Method of Planes analysis for both the short term (Q) and long-term (S) cases. The wall was analyzed for the short term case using the soil design (Q) strengths and for the long term (S) case using the (S) shear strengths of C = 0 and 0 = 23° for the clay strata. Factors of safety of

a. Short term (Q) Case 1.5 for static water 1.0 for static water plus 2 feet of freeboard.

b. Long term (S) Case 1.2 for static water.

were applied to the design shear strength as follows: 0 developed = arctan (tan 0 available/factor of safety) and cohesion/factor of safety. Using the resulting shear strength, net lateral soil and water pressure diagrams were developed for movement toward each side of the sheet pile. With these pressure distributions, the summation of horizontal forces was equated to zero for various tip penetrations, and the overturning moments about the tip of the sheets were determined. The required depth of penetration to satisfy the stability criteria was determined where the summation of the moments were equal to zero. Both (Q) and (S) Cases were analyzed. Additionally, the governing tip penetrations were checked to satisfy the minimum tip to headwater ratio of 3 to 1 in the (S) case. The sheet pile was extended if required.

3.2.1.9.3.2.2.8. T-Walls. A conventional stability analysis utilizing a 1.30 factor of safety incorporated into the soil parameters was performed for various potential failure surfaces beneath the T-wall sections. Negative resultant forces for all failure surfaces indicate that no additional load needs to be carried by the structure. Positive resultant forces greater than the positive resultant at the base of the structure indicate that this additional load must be carried by the structure and by the pile below the slip plane.

3.2.1.9.3.2.2.9. Erosion Protection. Due to the short duration of the hurricane flood stage and the resistant nature of the clayey soils, no erosion protection other than sodding is considered necessary on the levee slopes.

3.2.1.9.3.2.2.10. Review Comments. LMVD Comments (First endorsement) to limit the strain in geotextile to 5 percent instead of 7 to 8 percent were concurred with.

3.2.1.9.3.3. Structural.

St. Charles Parish - North of Airline Highway (Reference 48).

General. As constructed, the St. Charles Parish hurricane protection consists of primarily geotextile reinforced earthen levee. At its eastern edge, it ties into the Bonnet Carré West Levee which is part of the Mississippi River Levee system. At its western edge it ties into the Jefferson/ St. Charles Return Levee hurricane protection system at the Louis Armstrong Airport with a combination levee and uncapped cantilevered I-wall, capped cantilevered I-wall, T-wall and one railroad swing gate. In between, there are three pile-founded sluice gate structures with pile-founded T-wall and capped cantilevered I-wall (Bayou Trepagnier, St. Rose, Cross Bayou; two pile-founded sluice gate structures with capped cantilevered I-wall (Almedia, Walker); pile founded T-wall at the Gulf South Pipeline, uncapped cantilevered I-wall, pile-founded T-wall pipeline crossing, and one pile-founded swing gate at Good Hope; uncapped cantilevered I-wall and one pile-founded swing gate under I-310.

Structural Design

Design Criteria

<u>Basic data</u>

<u>Water elevations</u>

	Elevations (feet NGVD)
Wind Tide Level (Lake Pontchartrain)	11.5
Wind Tide Level (Bayou Trepagnier)	11.0
Wind Tide Level (Cross Bayou)	10.50
Wind Tide Level (St. Rose)	10.00
Wind Tide Level (Parish Line Canal)	10.00
Land side of floodwall	0.00 to -0.50

<u>Grades</u>

Floodwall Gross Grade	*Elevations (ft NGVD)*
I-Wall	12.00 to 13.50
T-Wall	12.00 to 13.00

<u>Unit Weights</u>

	pcf
Water	64.00
Concrete	150.00
Steel	490.00
Riprap	132.00
Saturated Sand	122.00
Saturated Clay	110.00
Saturated Shell	117.00

<u>Uniform Live Loads</u>

Item/Description	*psf*
Floors for Vertical Lift Gate Machinery	100

<u>Design Loads</u>

Wind Loads	50 psf

Design Methods. Design of reinforced concrete structures is in accordance with the requirements of the strength design method of the current ACI Building Code, as modified by the guidelines of “Strength Design Criteria for Reinforced Concrete Hydraulic Structures”, ETL 1110-2-312 dated 10 March 1988. The basic minimum 28-days compressive strength is 3,000 psi, except for prestressed concrete piling where the minimum is 5,000 psi. Pertinent stresses are tabulated below:

Pertinent Stresses for Reinforced Concrete Design	
fc’	3,000 psi
fy (Grade 40)	40,000 psi.

Maximum Flexural Reinforcement Ratio	0.25 x Balance Ratio
Minimum Flexural Reinforcement Ratio	200/fy
fc'(for Prestressed Concrete Piles)	5,000 psi
fu (for Prestressing Strands Grade 250)	250,000 psi

Drainage Structures

General. The drainage structures consist of reinforced concrete box culverts supported on precast, prestressed concrete piles with a sheet pile cutoff. The structures contain vertical lift gates. A reinforced concrete one-lane bridge is included at each of the structures to provide access across the structure.

Loading Cases. The pile designs for the drainage structures, based on the use of a pile test, are designed with a factor of safety = 2.0.The following load cases were used for the design of the drainage structures:

Case I	Dead loads only, no backfill or water loads, no wind, impervious sheet pile cutoff, no dynamic wave force (100% forces used)
Case II	Static water pressure to SWL, no wind, pervious sheet pile cutoff, no dynamic wave force (100% forces used)
Case III	Static water pressure to SWL, no wind, impervious sheet pile cutoff, no dynamic wave force (100% forces used)
Case IV	Static water pressure with water level 2 feet above SWL, no wind, impervious sheet pile cutoff, no dynamic wave force (75% forces used)
Case V	Static water pressure with water level 2 feet above SWL, no wind, pervious sheet pile cutoff, no dynamic wave force (75% forces used)
Case VI	No water, wind from flood side (75% forces used)

Bridge at Drainage Structures. The drainage structures include a one-lane bridge designed in accordance with AASHTO requirements for an H-10 loading for a single truck.

Bridge at Vicinity of Cross Bayou Drainage Structure. The one-lane bridge was designed in accordance with AASHTO requirements for an H-20 loading for a single truck. The bridge serves as access to the construction site for the Cross Bayou Drainage Structure with US Hwy 61.

I-Type Floodwall In the design of the I-wall, the following loading cases were considered:

Case I	Water to SWL, Q-case, FS = 1.5
Case II	Water to SWL + 2 feet freeboard, Q-case, FS = 1.0
	Water to SWL, S-case, FS = 1.2

Note: In Soils and Foundations Investigation and Design Section of GDM, Para 27d(1), it is noted penetration was determined for both the short term "Q" and long term "S" cases. Factors of safety were itemized as follows:

Short term (Q) Case
1.5 for static water
1.0 for static water plus 2 feet of freeboard
Long term (S) Case
1.2 for static water

Additionally, the governing tip penetrations were checked to satisfy the minimum tip to headwater ratio of 3 to 1 in the "S" case.

T-Type Floodwall The pile designs for the T-walls, based on the use of a pile test, are designed with a factor of safety = 2.0.The following load cases were used for the design of the T-walls:

Case I	Dead loads only, no backfill or water loads, no wind, impervious sheet pile cutoff, no dynamic wave force (100% forces used)
Case II	Static water pressure to SWL, no wind, pervious sheet pile cutoff, unbalanced soil load applied to sheet pile cutoff, no dynamic wave force (100% forces used)
Case III	Static water pressure to SWL, no wind, impervious sheet pile cutoff, no dynamic wave force (100% forces used)
Case IV	Static water pressure with water level 2 feet above SWL, no wind, impervious sheet pile cutoff, no dynamic wave force (75% forces used)
Case V	Static water pressure with water level 2 feet above SWL, no wind, pervious sheet pile cutoff, no dynamic wave force (75% forces used)
Case VI	No water, wind from land side (75% forces used)
Case VII	No water, wind from canal side (75% forces used)

Swing Gates and Gate Monoliths The pile designs for the swing gate monoliths, based on the use of a pile test, are designed with a factor of safety = 2.0. The following load cases were used for the design of the swing gate monoliths:

Case I	Gate closed, no wind, static water pressure to SWL, no wind, impervious sheet pile cutoff, no dynamic wave force (100% forces used)
Case II	Gate closed, no wind, static water pressure to SWL, no wind, pervious sheet pile cutoff, no dynamic wave force (100% forces used)
Case III	Gate closed, static water pressure to with water level 2 feet above SWL, no wind, impervious sheet pile cutoff, no dynamic wave force (75% forces used)
Case IV	Gate closed, static water pressure with water level 2 feet above SWL, no wind, pervious sheet pile cutoff, no dynamic wave force (75% forces used)
Case V	Gate open, no wind, truck or train on protected side edge of base slab
Case VI	Gate open, no wind, truck or train on flood side edge of base slab

Cases VII and VIII were shown in the GDM but they appear to be identical to Cases V and VI respectively

Case IX	Gate open, wind from protected side, truck or train on flood side edge of base slab
Case X	Gate open, wind from flood side, truck or train on protected side edge of base slab

3.2.1.9.3.4. Sources of Construction Materials

3.2.1.9.3.4.1. Sheet Pile. Generally, the sheet pile sections specified during advertisement were used for construction. However, sheet pile section substitutions conforming to the minimum required section modulus was allowed, primarily in contracts constructed after 1990. Below, is a table of sheet pile sections for St. Charles Parish.

St. Charles Parish	
Cross Bayou	unknown
Bayou Trepagnier Drainage Structure Tie-In	**
Almedia Drainage Structure Tie-In	**
Walker Drainage Structure Tie-In	**
Good Hope	SPZ-22*
Under I-310	PZ-22
St. Rose Drainage Structure Tie-In	PZ-22
Gulf South Pipeline Tie-In	**
Canadian National Swing Gate Tie-In	**

*As-advertised – Not confirmed as-built

** Information not located at the time of publication

3.2.1.9.3.4.2. Levee material

3.2.1.9.3.4.2.1. Source of Fill Materials. The levee will be constructed of hydraulic fill material obtained from an adjacent borrow area located in Lake Pontchartrain. Shell to be utilized at the structure site is also available from Lake Pontchartrain. Haul material is available from the Bonnet Carré Spillway to repair damage which may occur to the final levee and to construct the Bonnet Carré Spillway east guide levee enlargement.

3.2.1.9.3.4.2.2. Sources of Fill Material. The recommended plan of construction consists of hydraulically pumping sand from selected sites in the Mississippi River for use as a haul road and a base for the high strength geotextile to reinforce the hauled clay fill. Since there are ten soil reaches along the length of the alignment, each reach varies slightly in length of fabric, strength of fabric and number of layers of fabric. The clay will be hauled from selected borrow areas in the Bonnet Carré Spillway. After time has elapsed for required settlement and consolidation, subsequent semicompacted lifts will be constructed by hauling material from the borrow areas in Bonnet Carré Spillway.

3.2.1.9.4. As-built Conditions

3.2.1.9.4.1. Changes between design and construction (i.e. cross sections, alignment, sheet pile tip el, levee crest el.)

3.2.1.9.4.1.1. DACW29-98-C-0064. Lake Pontchartrain, Louisiana and Vicinity, High Level Plan, St. Charles Parish North of Airline Highway, Floodwall at I-310 Interchange, St. Charles Parish, Louisiana

Modification Nos. A00008 and A00009 were issued to allow excavation to remove pile driving obstructions, backfill with sand, rebuild levee, and drive sheetpiles.

3.2.1.9.4.2. Inspection during original construction, QA/QC, state what records are available. See paragraph 3.2.1.5.4.2 New Orleans East Bank for description of how records are kept.

3.2.1.9.5. Inspection and maintenance of original construction. Inspection and maintenance of original construction in the St. Charles East bank area is limited to the Annual Compliance Inspections since for structures have been brought under the Periodic Inspection Program

3.2.1.9.5.1. Annual Compliance inspection. Annual Compliance Inspections for the East Bank polder were conducted by Operations Division in conjunction with the Orleans Levee District. This district is responsible for maintaining 98.7 miles of protection works along the shore of Lake Pontchartrain and canals, which is inclusive of the St. Charles parish polder. The rating for these protection works, was “Outstanding” through 2001, at which time the condition ratings system changed. The ratings from that time on were “Acceptable”, but corresponded to the “Outstanding” rating under the previous rating system.

3.2.1.9.5.2. Periodic inspections. There are no structures under the Periodic Inspection Program in this polder.

3.2.1.9.6. Other Features.

3.2.1.9.6.1. Brief Description. The primary components of the hurricane protection system for the St. Charles East Bank basin are described above, namely the levees designed and constructed by the Corps of Engineers. However, other drainage and flood control features that work in concert with the Corps of Engineers levees are also an integral part of the overall drainage and flood damage reduction system. This section will briefly describe and present the criteria and pre-Katrina conditions of the interior drainage system, pump stations, and the Mississippi River Flood Protection System. There are currently no non-Corps levees or floodwalls in this basin. Even though the stormwater pump stations are part of the interior drainage system, they are a significant part of the system and warrant their own section.

3.2.1.9.6.2. Pre-Katrina Conditions. According to the local jurisdictions responsible for interior drainage, the storm drain system, interior canals, interior pump (lift) stations, and outfall pump station, were in good condition and prepared for high inflows from rainfall prior to August 29, 2005.

The Mississippi River Flood Protection System was in good condition prior to Katrina landfall.

3.2.1.9.6.3. Interior Drainage System.

Overview. The St. Charles East Bank basin contains about 20 square miles and generally slopes south to north from the Mississippi River to marshland on Lake Pontchartrain. Areas along the Mississippi River and the western end of the basin have residential and industrial development. A large area on both sides of Interstate 310 is undeveloped. Many features are typical of cities in the United States, and some features that are unique because much of the area is below sea level. Surface runoff from yards and streets flows into roadside ditches or into inlets and storm sewers. Excess runoff flows down streets and/or overland to lower areas. Open ditches collect the stormwater and carry it to stormwater pump stations that pump the water into interior canals that flow into the marsh next to Lake Pontchartrain through drainage structures in the Corps levee. No stormwater is pumped into the Mississippi River.

The entity responsible for local drainage in the St. Charles East Bank basin is St. Charles Parish. The Louisiana Department of Transportation and Development highways are also a part of the local drainage system.

System Components. Local drainage begins with overland flow which follows the ground topography. The land topography and development sequence influenced the roadside ditch, storm sewer, canal, and pump station layout Pump stations, located north of the developments, pump into canals that flow north to the marsh. The flow gets past the Corps levee through one of the five gated structures or the Bayou Trepagnier outfall pump station at the western end of the basin.

Design Criteria. The current design criteria for St. Charles East Bank is a 10% probability (10 year frequency) storm event for roadside ditches and storm sewers. The interior canals and pump stations for the larger developments west of Interstate 310 have a 10% probability (10-year frequency) capacity, while the smaller systems east of Interstate 310 have less capacity.

There are no Southeast Louisiana (SELA) Urban Flood Control Projects in this basin.

3.2.1.9.6.4. Pumping stations - St. Charles Parish East Bank. Pump stations for St. Charles Parish were not evaluated. A general description of the system is provided in Interior Drainage Summary.

3.2.1.9.6.5. Levees and floodwalls –

3.2.1.9.6.5.1. MRL - MRL Levees and floodwalls are addressed in paragraph 3.2.1.5.6.4.1 New Orleans East Bank MRL. There are no floodwalls that are part of the MRL Project in this reach.

3.2.1.9.6.5.2. Non Corps. Several local interest and/or private levees are located within the project area. No design criteria for these levees have been made available to the Corps.

3.2.2. New Orleans to Venice

3.2.2.1. General Description

The project is located along the east bank of the Mississippi River from Phoenix, Louisiana, (approximately 28 miles southeast of New Orleans) down to Bohemia, Louisiana, and along the west bank of the river from St. Jude, Louisiana, (approximately 39 miles southeast of New Orleans) down to the vicinity of Venice, Louisiana.

Project Purpose. The project will provide protection from hurricane tidal overflow for 100-year frequency storms. The protected area encompasses approximately 75% of the population and 75% of the improved lands in the lower Mississippi River delta region.

Project Features. The project consists of the following:

West Bank

St. Jude to City Price - 3 miles of enlarged back levees from St. Jude to City Price

Reach A - 13 miles of enlarged back levees from City Price to Tropical Bend and two 54" flap-gated culverts

Reach B1 – 12 miles of enlarged back levees from Tropical Bend to Fort Jackson and a floodgate at Empire

Reach B2 – 9 miles of enlarged back levees from Fort Jackson to Venice

West Bank River Levee (WBRL) – 34 miles of enlarged west bank Mississippi River levees from City Price to Venice

East Bank

Reach C – 16 miles of enlarged back levees from Phoenix to Bohemia and 10 flap-gated culverts

Figure 25. Extent of NOV Hurricane Protection in Plaquemines Parish. The NOV consists of six distinct reaches; Reach C, Reach St. Jude to City Price, Reach A, Reach B1 and Reach B2.

Pre-Katrina Conditions.

<u>St. Jude to City Price Pre-Katrina Status</u>

- Construction in this area started in 1993. Before Hurricane Katrina, the one 1st enlargement levee construction contract was completed in this area. Remaining work in this area consists of a 2nd enlargement levee contract.

<u>Reach A Pre-Katrina Status</u>

- Construction in this area started in 1986. Before Hurricane Katrina, all of the 2nd enlargement levee construction contracts and floodwall contracts had been completed. There were a total of 15 construction contracts that were completed in this reach.

Remaining work in this area (pre-Katrina) consists of I-wall cappings. However, some of this area may have settled below design grade.

Reach B1 Pre-Katrina Status

- Construction in this area started in 1968. Before Hurricane Katrina, all of the required levee enlargement construction contracts and floodwall contracts had been completed. There were a total of twenty-nine construction contracts that were completed in this reach. Remaining work in this area (pre-Katrina) consists of I-wall cappings. However, some of this area may have settled below design grade.

Reach B2 Pre-Katrina Status

- Construction in this area started in 1974. Before Hurricane Katrina, all of the 4^{th} enlargement levee construction contracts and floodwall contracts had been completed. There were a total of nine construction contracts that were completed in this reach. Remaining work in this area (pre-Katrina) consists of I-wall cappings. However, some of this area may have settled below design grade.

Reach C Pre-Katrina Status

- Construction in this area started in 1972. Before Hurricane Katrina, the 3rd enlargement levee construction contracts had been completed. There were a total of seven construction contracts that were completed in this reach. Remaining work in this area consists of a 4^{th} enlargement levee contract.

West Bank River Levees (WBRL) Pre-Katrina Status

- Construction in this area started in 1989. Before Hurricane Katrina, the 1st enlargement levee construction contracts and floodwalls had been completed. There was also one 2^{nd} enlargement levee construction contract that had been completed. There were a total of sixteen construction contracts that were completed in this reach. Remaining work in this area (pre-Katrina) consists of three 2nd enlargement levee contracts and I-wall cappings.

3.2.2.2. History

On July 30, 1962, the Chief of Engineers submitted a report that recommended improvements along the Mississippi River below New Orleans to prevent damages to the developed areas of St. Bernard and Plaquemines parishes from hurricane tidal surges and overflow. The plan recommended increasing the heights of existing back levees and modifying existing drainage facilities at four primary reaches: Reach A on the west bank between City Price and Empire (Tropical Bend); Reach B on the west bank between Empire (Tropical Bend) and Venice; Reach C on the east bank between Phoenix and Bohemia; and Reach E on the east bank for about 8 miles between Violet and Verret.

Three months later, the 1962 Flood Control Act (Public Law 87-874) authorized the project for hurricane-flood protection on the Mississippi River Delta at and below New Orleans, Louisiana, in accordance with the recommended plan submitted by the Chief of Engineers. Following the authorization, the hurricane protection project was officially named the New Orleans to Venice, Louisiana, Hurricane Protection project.

Despite congressional authorization, the project plans were far from finalization. On February 5, 1964, the Plaquemines Parish Commission Council requested the division of Reach B into two separate units: Reach B1, between Empire (Tropical Bend) and Fort Jackson; and Reach B2, between Fort Jackson and Venice. On March 25, 1964, the Chief of Engineers approved the division of the Reach B, subject to the proviso that the Plaquemines Parish Commission Council pay for a closure levee at Fort Jackson that would be required to complete the independently constructed Reach B1 loop.

A second post-authorization change to the project emanated from dissatisfaction with the project features covering St. Bernard Parish. On May 8, 1964, the House Committee on Public Works adopted a resolution directing a restudy of the hurricane protection for St. Bernard Parish. The restudy, completed on November 29, 1966, recommended the enlargement of the Chalmette Area Plan feature of the Lake Pontchartrain, Louisiana, and Vicinity Hurricane Protection project, which had been authorized by the 1965 Flood Control Act. The extension of the Chalmette Area Plan to all of the area in St. Bernard Parish, for which hurricane protection could be economically justified, subsequently encompassed the proposed protected area under Reach E of the New Orleans to Venice Hurricane Protection project. The House committee resolution was then closed out with a negative report recommending the deauthorization of Reach E from the project.

Upon the appropriation of funds, construction of the project began in 1966; however, another hurdle remained concerning the treatment of the main river levees that were authorized, constructed, and maintained under the Mississippi River and Tributaries (MR&T) project. Ever since the inception of plans for the New Orleans to Venice Hurricane Protection project, the Chief of Engineers and the Mississippi River Commission recognized the possible necessity to modify the main river levees to accomplish the level of hurricane protection envisioned. On October 29, 1969, the Corps of Engineers initiated review of the New Orleans to Venice Hurricane Protection project. As part of this review, two alternate plans were developed for protecting the west bank project areas from hurricane tidal surges from Breton Sound. The first option consisted of raising the west bank river levee to a grade sufficient enough to prevent overtopping by tidal surges from the east. The second option consisted of a barrier levee on the east bank from Bohemia to a point 10 river miles above the Head of Passes, coupled with minor enlargement of the west bank levees from Fort Jackson to Venice. The project review determined that the latter option was both more feasible and economical at that time. Preparation of a design memorandum for the east bank barrier levee was authorized on July 2, 1970.

In February 1985, the Plaquemines Parish Commission Council requested that further work on Reach A be deferred, and that designs for the east barrier levee or the west bank river levee be undertaken. As a result of this request, a restudy of the two alternate plans was conducted. The restudy, in turn, recommended the west bank river levee plan, which necessitated the enlargement of 33.8 miles of river levee from City Price to Venice. The preparation for the design memorandum for the west bank river levee was authorized on July 24, 1986 and later approved in March 1987.

3.2.2.3. Datum - Subsidence and Vertical Datum Problems in New Orleans, LA

Because of technological gains, the U.S. Army Corps of Engineers is able to more accurately track subsidence of projects – something that could not be done as reliably in the past. Based on a recent study, we can now estimate that the New Orleans area is subsiding at a rate of 6-17 mm/yr or 2-5½ feet per century. In the city itself it's about 3 feet per century and as much as 10 feet per century in Venice, if recent trends continue.

The Interagency Performance Evaluation Task Force (IPET), an independent group activated by the Corps of Engineers to study the response of the hurricane protection system during Hurricane Katrina, identified problems with using the previous vertical datum to which survey benchmarks were referenced. IPET's ability to accelerate analysis of this issue, which was ongoing by the Corps' New Orleans District and the National Oceanic and Atmospheric Administration (NOAA)'s National Geodetic Survey (NGS), led to the identification of two major problems with elevations in the New Orleans area: subsidence and the use of the old vertical datum elevations as equal to local mean sea level, a common misunderstanding in the engineering community up until the 1990s.

Benchmarks serve as the reference or starting elevation when measuring levee heights, relationships to the water surface (local mean sea level), structure and levee elevations, etc. It has been known since 1985 that the elevations of benchmarks in and around New Orleans were inaccurate, due to subsidence, and needed to be updated. The exact amount of subsidence was not known until a 2004 survey conducted by the NGS in cooperation with the Louisiana Spatial Reference Center, the Corps of Engineers and state and local governments was performed on some 86 benchmarks in southern Louisiana.

The 2004 survey pointed out inaccuracies due not only to subsidence, but also to distortions and errors in elevations of benchmarks that were assumed to be stable in the past, but had in fact subsided themselves. Based on the 2004 survey, the Corps of Engineers has revised the elevations of survey benchmarks used to establish heights of structures, such as levees and floodwalls, in Southern Louisiana. Use of the new 2004 survey assures consistency for all elevation surveys performed in the southern Louisiana area.

The IPET has developed a new relationship between the current local mean sea level and the 2004 survey, which is referred to as the North American Vertical Datum of 1988 (2004.65 Adjustment). Local mean sea level in the city itself is about ½ foot above the 2004 datum. The Corps will use the 2004 elevations and their varied relationship to the local mean sea level throughout the area to precisely determine the elevations of levees and other critical flood protective structures. This datum will also be used by the construction industry and others in southern Louisiana for a wide variety of projects that rely on elevations relative to the local water surface.

More information can be found in the "Geodetic and Water Level Datum" report.

3.2.2.4. Design Hurricane

3.2.2.4.1. 100-year storm. The design hurricane is a hurricane that would produce a 100-year stage. A hurricane of lesser intensity which would indicate a lower levee grade and an increased frequency would expose the protected areas to hazards to life and property that would be disastrous in the event of a design hurricane.

The characteristics of the 100-year storm were derived from the SPH parameters. The 100-year storm meteorological parameters differed from the SPH only in wind velocities and CPI. A SPH storm was considered to have a recurrence interval of once in 100 years anywhere within Zone B. The probability of the SPH storm striking a smaller subzone, such as along Reach B2, would be less. The frequency of the SPH at the site of a protective structure was assumed to be dependent upon its exposure and the direction of approach of the storm.

Using observed high water mark and stage data, combined with computed wind tide elevations using different central pressure indices, a surge frequency curve was constructed representative of a portion of the hurricane protection system. The frequency curve also considered statistics on the critical direction of approach. The frequency of the computed wind tide elevations was adjusted based on the percentage of each direction followed by historic hurricanes. The probabilities of equal stages for both groups of tracks were then added arithmetically to develop a curve representing a synthetic probability of recurrence of maximum wind tide levels for hurricanes from all directions. From this curve, the 100-year stage was identified.

3.2.2.4.2. Design Criteria and Assumptions - Functional design criteria.

3.2.2.4.2.1. Hydrology and Hydraulics. The design hurricane characteristics are shown in Table 31; the design tracks are shown in Figures 26–28. The maximum wind speed was computed using the same equations as for Orleans East Bank. For each project area, the track and forward speed were selected to produce maximum wind tide levels.

Table 31
Design Hurricane Characteristics

Location	Track	CPI, Inches	Radius of Maximum Winds, Nautical miles	Forward Speed, Knots	Maximum Wind Speed[1] MPH	Direction of Approach
Reach A	B	28.0	30	11	85	South
Reach B1	B	28.0	30	11	91	South
Reach B2	B	28.0	30	11	91	South
Reach C	Des H	28.0	30	11	96	Southeast
Mississippi River Levees, Mile 49	1	27.6	30	11	96	Southeast
Mississippi River Levees, Mile 40	1	27.6	30	11	96	Southeast
Mississippi River Levees, Mile 25	1	27.6	30	11	96	Southeast
Mississippi River Levees, Mile 15	1	27.6	30	11	96	Southeast
Mississippi River Levees, Mile 10	1	27.6	30	11	96	Southeast

[1] Windspeeds represent a 5 minute average 30 feet above ground level.

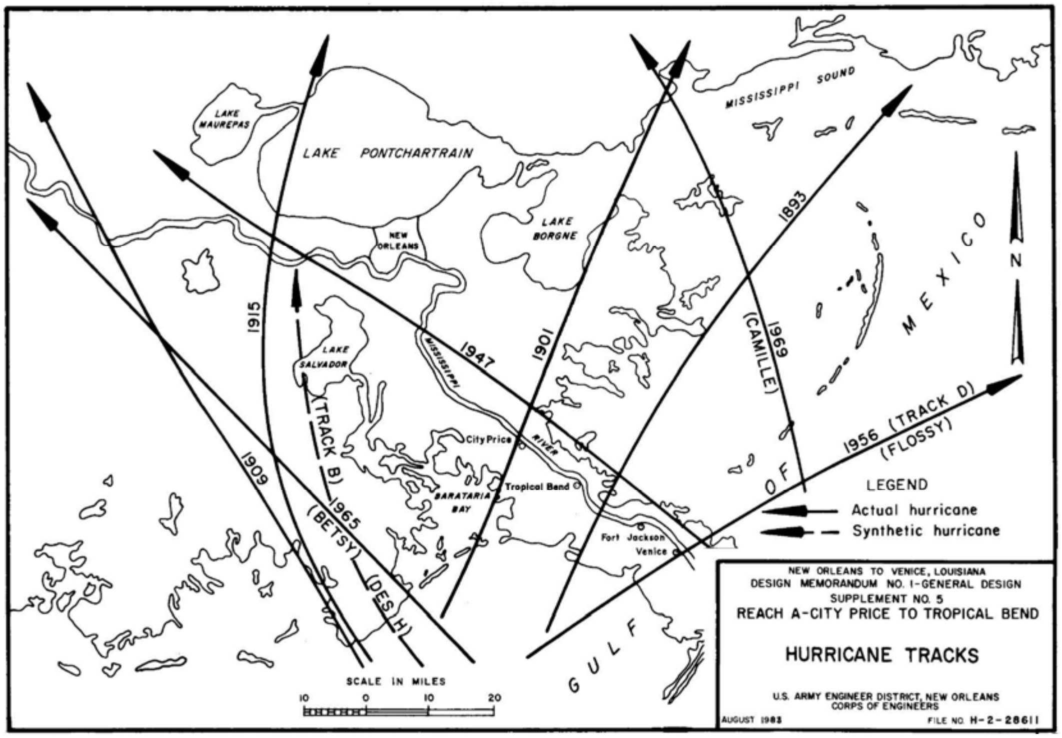


Figure 26. Hurricane tracks, Reach A, B1, and B2

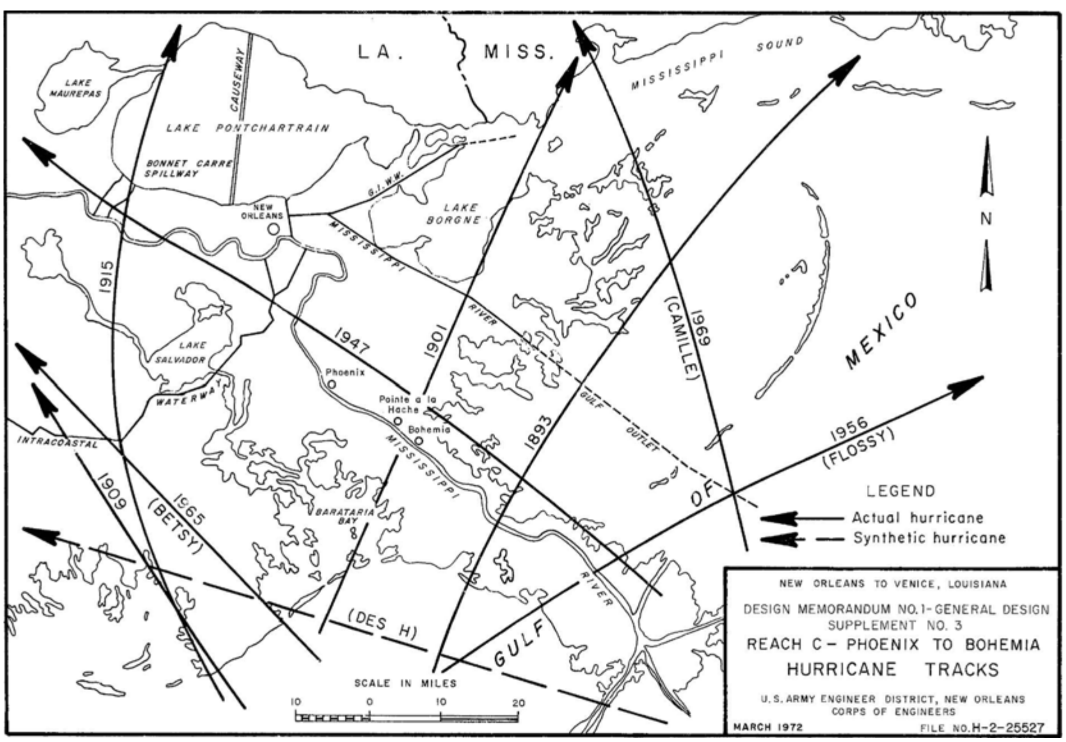


Figure 27. Hurricane tracks, Reach C

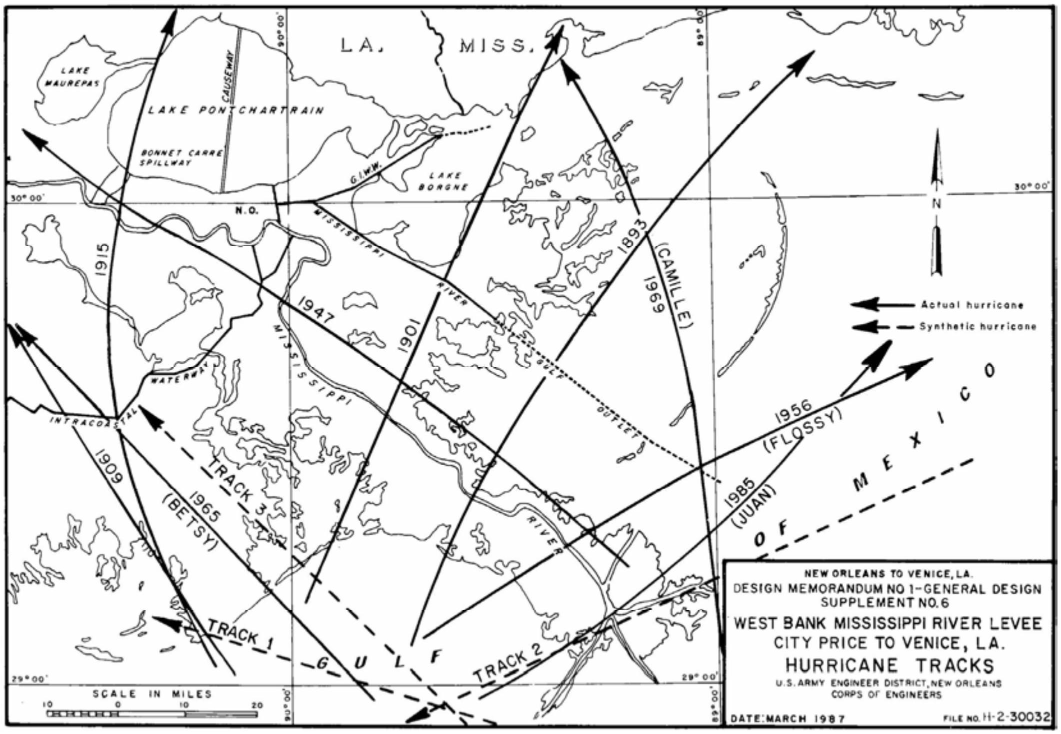


Figure 28. Hurricane track, West Bank Mississippi River Levee

3.2.2.4.2.1.1. Surge. Surge elevations were computed using the same methodology as used for IHNC for Orleans East Bank, with an additional step. Surge heights were verified for the Sep 1915 hurricane and the Sep 1956 hurricane. For Reach B1 and B2, surges were also verified for Hurricane Betsy, in Sep 1965. Computed surge heights for Hurricane Betsy using the same Z factors averaged about 2.9 feet higher than observed surge heights. This was attributed to the effect of the high forward speed of Hurricane Betsy. A fast moving hurricane does not allow enough time for the surge heights to approach the steady state of water superelevation. For design purposes, Z factors derived from the slow moving hurricanes were used.

Table 32
Verification of Hurricane Surge Heights

Location	Surge adjustment factor, Z	Sep 1915		Sep 1956	
		Observed, ft MSL	Computed, ft MSL	Observed, ft MSL	Computed, ft MSL
Belair	0.52	-	-	5.3	6.2
Phoenix	0.52	-	-	8.5	7.8
Pointe a la Hache	0.52	12.0	12.4	10.3	10.2
Ostrica	0.64	-	-	12.1	12.2
Buras	0.80	7.9	8.7	-	-
Grand Isle	0.80	9.0	8.8	3.9	4.1

Next, the same methodology used to adjust surge heights for the Chalmette Extension was applied. The surge reference line was located approximately 6.3 miles from the upper end of Reach A, near City Price, extending to approximately 2.5 miles from the lower end of Reach A, near Tropical Bend. On the east bank, the surge reference line extended up to approximately 7 miles from Phoenix and to about 2 miles from Davant. Table 33 shows the wind tide level at the surge reference line and at the levee location for Reach A; similar information was not contained in the DM for Reach C.

Table 33
Wind Tide Levels

Location	Wind Tide Level, surge reference line, FT NGVD	Wind Tide Level at levee location, FT NGVD
Segment 1, Reach A, 0+00 – 83+30	11.2	8.9
Segment 2, Reach A, 83+30 – 315+00	10.4	8.2
Segment 3, Reach A, 315+00 – 477+00	10.8	9.6
Segment 4, Reach A, 477+00 – 613+00	11.0	10.1
Segment 5, Reach A, 613+00 – 681+67.45	11.2	10.3

For the surge along the Mississippi River, National Engineering Science Company was contracted to evaluate surge along the river. NESCO used a bathystrophic storm surge technique to compute surge at Nairn, LA, a community approximately 15 miles south of Pointe a la Hache. The equations used to determine the behavior of a hurricane surge hydrograph as it propagates upstream were rearranged in terms of discharge. The modified equations are:

$$\frac{d(Q/A)}{dt} = -g\frac{\partial H}{\partial x} - g\frac{Q|Q|}{C_h{}^2 A^2 (H + d_0)} - g(I_0 - I_b)$$

$$B_s \frac{\partial H}{\partial t} + \frac{\partial Q}{\partial x} = 0$$

where

Q= initial steady state discharge in the downstream direction, cfs

H= surge height above the initial water surface profile in the river, ft

g= acceleration due to gravity

C_h = the Chezy coefficient

d_0 = the hydraulic depth for irregular cross sections, ft

I_0= the slope of the water surface under the initial condition, ft/ft

I_b= the bottom slope of the channel, ft/ft

Bs = the surface width of the river, ft

A= the cross sectional area of the river, ft2

t= the routine time, seconds

x= the distance upstream from some initial point of surge input, ft

These equations were rewritten in a form suitable for application of fourth order techniques. The variation of *Q* and *H* was evaluated by finite differences and the integrations in time were performed using a fourth order Runge-Kutta method. The initial boundary conditions were:

a. the initial discharge in the river at time = 0 was taken as a constant for all stations (*x*) along the river.

b. the initial surge height in the river (H_0) was zero at $t = 0$ for all stations.

c. the input surge at $x = 0$ was taken as a prescribed stage hydrograph ($H_0(t)$).

A surge hydrograph at Nairn was generated using isovels from Hurricane Betsy, and hydrograph was extrapolated across the Pointe-a-la-Hache Relief Outlet to Bohemia, LA, and West Pointe-a-la-Hache. The hydrograph at West Pointe-a-la-Hache was compared to a partial stage record at the location that also included the peak stage.

Maximum water surface elevations for the Mississippi River between Venice and Baton Rouge were developed for three hurricane conditions, Hurricane Betsy, SPH, and PMH, and for four different Mississippi River water surface profiles. The water surface profiles corresponded to stages of 2.7 ft, 6.0 ft, 13.0 ft, and 20.0 ft MSL at Carrollton. A moderate speed of translation of 5 to 15 knots was used to generate additional surge hydrographs.

Combining stage-frequency data for the Mississippi River and the hypothetical parameters for different frequency hurricanes, a hypothetical hurricane isovel pattern based on 96 percent of the SPH wind speeds was derived. The isovel pattern was transposed, rotated and moved along three tracks considered critical to five points along the river, mile 49, mile 40, mile 25, mile 15, and mile 10 Above Head of Passes. Using the new winds with the SPH forward speed and radius to maximum winds, hurricane surge elevations were computed at five points along the river.

3.2.2.4.2.1.2. Waves. For Reach A, Reach B1, Reach B2, and Reach C, wave runup was calculated using the methodology described in Orleans East Bank. In this case, the deep water wave is slightly larger than the significant wave. For the lateral levee portion of Segment 1, Reach A, no design runup was considered; approximately 2 feet of freeboard was added to the surge elevation to achieve the design elevation. For the St. Jude to City Price levee, the designers indicated that the wave runup was calculated using the criteria contained in the 1984 Shore Protection Manual.

A large expanse of marsh was present between the Mississippi River and Breton Sound. The marsh varied in elevation from near zero feet at Breton Sound to almost 10 ft along the alluvial ridge adjacent to the Mississippi River. Although the ridge and marsh would be submerged during the design storm, the decrease in water depth would have a pronounced effect on the characteristics of the waves propagating from Breton Sound. Wave heights and periods would be attenuated in the lesser depths over the marsh and ridge and grow in height as they propagate across the river toward the Mississippi River Levee on the west bank.

The wave characteristics for Breton Sound were determined according to procedures outline in the U.S. Army Corps of Engineers Shore Protection Manual. To determine the characteristics of the wave at the river levee, a method outlined in the 1984 Shore Protection Manual was used. Wave runup was computed using model study data developed by Saville that is presented in the 1984 Shore Protection Manual.

The MR&T levee on the west bank above mile 44 is protected from significant wave activity by the east bank MR&T and Reach C levees; therefore, a wave berm was not required. Above Mile 44, the presence of small wind-generated waves in the river necessitate the addition of 2 ft of freeboard to the existing levee above the design still water level.

Information regarding wave characteristics for the MR&T levee on the east bank was not available at the time of preparation of this report.

3.2.2.4.2.1.3. Summary. Table 34 contains maximum surge or wind tide level, wave, and design elevation information. The Mississippi River West Bank Levee requirement is for the levee to slope in a straight line from elevation 17 at Mile 44 into the existing MR&T levee height at Mile 48. Information on the design elevation for the Mississippi River East Bank Levee was not available at the time of preparation of this report.

3.2.2.4.2.1.4. Interior Drainage. The protection system in Reach A would have an impact on the interior drainage of two small areas totaling 115 acres. The first area, 75 acres, would be drained by an existing drainage facility. The second area, 40 acres, would be drained into the Plaquemines Parish Drainage Canal. Local interests would determine exactly how water from the second area would reach this canal.

In Reach B1, in the vicinity of Empire, the protection system would intercept drainage of an area of about 365 acres. To meet the requirement of navigation, a floodgate would be constructed with an 84 ft width and a sill elevation of -14 ft MSL. This floodgate would be more than adequate to dispose of runoff from intense storms.

In Reach B2, the discharge pipes of the Venice pump station would require modification to accommodate construction of a floodwall at the site. In addition, the flotation channel to the Venice pump station would serve as an outfall to allow drainage flow into open water.

In Reach C, five gravity structures with flap gates were constructed by the local interests prior to the construction of the federal protection system. The system was determined to have sufficient capacity to dispose of runoff from a 25-year, 24-hour storm with an average stage of 0.5 ft on the gulfside so that the sump pool elevation remains below 2.0 ft MSL and storage equivalent to about 3 inches of runoff below elevation 2.0 ft MSL would be available within 24 hours after cessation of runoff.

Table 34
Wave Runup and Design Elevations. (Transition zones not tabulated – governing DM is listed)

Location	DM	Average Depth of Fetch, ft	Significant Wave Height Hs, ft	Wave Period, T, sec	Deep Water Wave Height, ft	Maximum Surge or Wind Tide Level, ft	Runup Height ft	Free-board ft	Design Elevation Protective Structure, ft
Segment 1, Reach A, 0+00 – 4+50	DM01, Sup 05, Nov 1987	6.6	3.20	4.35	3.28	8.9 NGVD	0.0	2.1	11.0 NGVD
Segment 1, Reach A, 4+50 – 83+30	DM01, Sup 05, Nov 1987	6.6	3.20	4.35	3.28	8.9 NGVD	3.5	-	12.5 NGVD
Segment 2, Reach A, 83+30 – 315+00	DM01, Sup 05, Nov 1987	6.9	3.26	4.40	3.35	9.2 NGVD	3.8	-	13.0 NGVD
Segment 3, Reach A, 315+00 – 477+00	DM01, Sup 05, Nov 1987	7.3	3.49	4.50	3.60	9.6 NGVD	3.7	-	13.5 NGVD
Segment 4, Reach A, 477+00 – 613+00	DM01, Sup 05, Nov 1987	7.8	3.65	4.65	3.76	10.1 NGVD	3.8	-	14.0 NGVD
Segment 5, Reach A, 613+00 – 681+67.45	DM01, Sup 5, Nov 1987	8.0	3.71	4.70	3.82	10.3 NGVD	4.0	-	14.5 NGVD
Reach B1, levee	DM01, Aug 1971	6.7	3.1	4.2	3.2	12.0 MSL	3.0	-	15.0 MSL
Reach B1, floodwall	DM01, Aug 1971	6.7	3.1	4.2	3.2	12.0 MSL	6.5 – 7.7 [1]	-	18.5 – 20.0 [2] MSL
Reach B2, levee	DM01, Sup 04, Aug 1972	7.2	3.3	4.4	3.41	11.5 MSL	3.5	-	15.0 MSL
Reach B2, floodwall	DM01, Sup 04, Aug 1972	7.2	3.3	4.4	3.41	11.5 MSL	7.5	-	19.0 MSL
West Pointe a la Hache Back Levee	None – memo dated 21 Feb 1991	NA	3.53	3.89	NA	8.1 NGVD	3.9	-	12.0 NGVD
Return Levee to Mississippi River Levee	None – memos dated 21 Feb 1991 and 20 Jun 1991	NA	NA	NA	NA	NA	-	NA 2 est.	12.0 – 10.0 NGVD
Mississippi River Mile 10.8 – 20.0	DM01, Sup 06, Mar 1987	15.6 [3]	5.2	4.7	5.7	12.6 NGVD	3.4	-	16.0 NGVD
Mississippi River Mile 20.0 – 30.0	DM01, Sup 06, Mar 1987	14.9 [4]	5.4	4.5	-	13.5 NGVD	3.5	-	17.0 NGVD
Mississippi River Mile 30.0 – 44.0	DM01, Sup 06, Mar 1987	13.7 [5]	5.1	4.5	-	13.5 NGVD	3.4	-	17.0 NGVD
Mississippi River Mile 44.0 – Mile 48.0	None – memo dated 30 May 91	NA	NA	NA	NA	14.0 – 13.5 NGVD	-	2.0	17.0 – 15.5 NGVD
Reach C, Phoenix to Davant, Levee	DM01, Sup 04, May 1972	11.5	5.45	5.35	5.70	13.0 MSL	4.0	-	17.0 MSL
Reach C, Phoenix to Davant, Floodwall	DM01, Sup 03, May 1972	11.5	5.45	5.35	5.70	13.0 MSL	7.0	-	20.0 MSL
Reach C, Davant to Bohemia, Levee	DM01. Sup 04, May 1972	12.5	5.45	5.65	5.68	14.0 MSL	3.0	-	17.0 MSL
Reach C, Davant to Bohemia, Floodwall	DM01, Sup 04, May 1972	12.5	5.45	5.65	5.68	14.0 MSL	6.0	-	20.0 MSL

1 Height of floodwall would be dependent on levee configuration on floodside of structure.
2 Height of floodwall would be dependent on levee configuration on floodside of structure.
3 In Breton Sound
4 In Breton Sound
5 In Breton Sound

3.2.2.4.2.2. Geotechnical

3.2.2.4.2.2.1. Reach A - City Price to Tropical Bend. Approximately 12.8 miles of Hurricane Protection Levees and Floodwalls.

3.2.2.4.2.2.1.1. Geology (Reference No. 52). The project area is located within the Gulf Coastal Plain. More specifically, the area is located on the deltaic plain of the Mississippi River in a region of extremely low relief. The dominant physiographic features are the natural levees of the Mississippi River and its abandoned distributaries, and the marshlands and bodies of water that lie between the natural levees. Elevations range from a maximum of approximately 6 feet along the natural levees to a minimum elevation of 0.0 feet in the area between the natural levees.

3.2.2.4.2.2.1.2. Foundation Conditions

The subsurface consists of Holocene deposits of variable thickness underlain by Pleistocene material. Generally, the Holocene deposits consist of a surface layer of natural levee and/or marsh deposits underlain by interdistributary, intradelta, prodelta, and abandoned distributary deposits.

The marsh deposits, which vary in thickness from 2 feet to 12 feet, consist of very soft to soft clays with peat and organic matter. Natural levee deposits overlie the marsh deposits between: Station 24+90 and Station 113+64, Station 234+00 and Station 329+53, and Station 445+00 and Station 501+12. These natural levee deposits vary in thickness up to 12 feet and consist of fat and lean clays, silts and silty sands.

Abandoned distributary deposits are located in the vicinity of Stations 5+00, 70+00, 140+00, 170+00, 290+00, and 385+00. These abandoned distributary deposits consist of very soft to soft clays, silts, silty sands and sands. The depths of the distributary deposits cannot be determined from available boring data; however, depths of from 40 feet to 100 feet are indicated.

Underlying the marsh deposits between the abandoned distributaries are interdistributary deposits. These interdistributary deposits vary in thickness from 25 feet to 65 feet and consist predominantly of fat clays. Occasionally lean clay, silt, silty sand and sand lenses are found within the interdistributary deposits.

3.2.2.4.2.2.1.3. Field Investigation. A total of 30 general type and 36 undisturbed soil borings were made for design in association with the Reach A project. The bottom elevations of these borings range from -40 to -189. In addition to the above, borings P2-U, PI-G, and I-2-U were taken for the geotextile reinforced levee test section. No changes were made to the geologic profile or the shear strength lines to reflect the information from these borings. These borings are shallow and were taken to determine the local conditions and to find what type of materials the piezometers were going to be installed in.

3.2.2.4.2.2.1.4. Levee Improvements

a. *Existing Levee*. Throughout the project area, the existing hurricane levee has a factor of safety slightly above one against a slope failure into the drainage canal which is on the protected side of the levee. Using conventional construction techniques, an enlargement of the existing levee gulfward from its present toe would result in a levee having a factor of safety (F.S.) = 0.80 for a protected side analysis and 0.85 for a flood side analysis. This analysis applies where the levee crown elevation is 14.5. The critical slip surface for a failure into the canal is much deeper than the slip surface for a gulf side failure; it ranges from Elev. -25 feet to Elev. -40 feet. The design reaches are described in Table 35 below:

b. *Geotextile Reinforced Levee Option*. Recent developments in high - molecular - weight polymers and weaving techniques have made it possible to enlarge the present levee in place. Geotextiles are textiles in a traditional sense, but consist of synthetic fibers rather than natural ones like cotton, wool, and silk. Thus., biodegradation is not a problem. The fibers are made into a flexible, porous fabric by standard weaving machinery. Geotextiles are designed to provide a wide range of porosity. The new generation of reinforcing geotextiles are made from polyester, nylon, aramid, or fiberglass fibers. An extremely strong single layer fabric can be manufactured from these fibers with tensile strain characteristics that are compatible with soft clay soils. Geotextiles made from other fibers generally exhibit excessive creep properties under a lower percentage of their ultimate load strength. Excessive creep can destroy any reinforced soil structure. In order for reinforcement to be effective, it must provide the required tensile force at levels of strain that are compatible with the soils at the site. The soft clay soils throughout this site reach maximum deviator stresses between 3 percent and 5 percent strain. A polyester geotextile is recommended, and the maximum recommended strain is 5 percent. Polyester is presently the most economical geotextile within the high strength group. At present, geotextiles provide the most viable alternative for raising the existing levee in place to design grade.

3.2.2.4.2.2.1.5. Pile Foundation

The T-wall will be supported by piling, battered as required, to provide stability against the unbalanced lateral waterloads. In compression, a factor of safety of 2.0 was applied to the shear strength and a lateral earth pressure coefficient of $K_0 = 1.0$ was used for determining the normal pressure on the pile surface. In tension, a factor of safety of 2.0 was applied to the shear strengths and coefficients of $K_0 = 0.7$ (S-case) and Ko = 1.0 (Q-case) were used. Design of the T-wall pile foundation was performed for both the (Q) and (S) cases. In these two designs, the (Q) case shear strengths governed. Pile design loads vs. tip elevations, and subgrade moduli vs. tip elevations were computed. Settlement of the piles due to consolidation during maximum loading is not expected since the major loads are caused by hurricane-induced stages of insufficient duration for consolidation of the foundation clays to ensue.

It is recommended that pile load tests be performed at the Homeplace (Gainard Woods) pumping station prior to preparation of the plans and specifications. A minimum of two piles would be load tested: one at the design tip elevation and another 10 feet below the design tip elevation. The piles would be tested in both compression and tension, allowing a minimum of 14 days between tests.

3.2.2.4.2.2.1.6. Slope Stability

a. *Geometry*. The design section consists of a IV on 3H slope on the protected side, an 8-ft. crown, and a IV on 3H slope from the crown to the wave berm. Specifics for each reach are presented in Table 35.

Table 35
Reach Geometry

Reach Number	Levee Stations	Crown Elevations	Top of Berm el.	Berm Slope	Bottom of Berm, el.
1*	0+00 to 4+00	11.0	not req'd	not req'd	
1**	4+50 to 83+30	12.5	7.5	IV on 11H	4.5
2	83+80 to 314+50	13.0	7.5	IV on 12H	5.0
3	315+00 to 477+00	13.5	8.0		5.0
4	477+50 to 613+00	14.0	8.5		5.5
5	613+50 to 681+90.791	14.5	8.5		5.5

* Pertains to upper return levee.
** Pertains to back levee.

The slope from the lower berm elevation to the existing ground is IV on 3H.

b. *Factors of Safety*. The geotextile requirements to develop factors of safety of 1.3 or 1.5, as appropriate were computed. A factor of safety of 1.5 is used in the vicinity of pipelines and other structures. Two layers of geotextile will be used in reaches where a safety factor of 1.5 is required.

c. *Construction Fill.* A sand core is used in the gulf side enlargement of the existing levee. Sand has several advantages in this type of construction, especially in this area.

(1) Sand improves the frictional resistance between the geotextile and the fill.

(2) Sand provides a more stable foundation to place clay fill on, and also reduces the chances of a failure within the fill.

Sand helps relieve the pore pressure at the soil/fabric interface caused by the foundation loading.

Clay will be placed over the sand blanket to provide a seepage barrier, erosion control and a medium in which grass will grow. A minimum of 2 feet of clay will be placed on the sand blanket over the wave berm. A much thicker clay cover will be placed under the centerline of the new levee. An impermeable core is provided by the existing levee, which will effectively prevent flow through the section.

d. *Geotextile Design Methodology.* Stability analyses were performed using the sliding wedge method and the results were compared to values obtained from circular arc analyses for the section to Elev. 14.5 feet design grade. For this job, at shallow depths, the wedge method of analysis is more conservative and requires a stronger fabric to achieve the same factor of safety; at greater depths, the geotextile requirements are approximately the same for both methods. The geotextile will provide the required tensile force to reinforce the soil and increase the factor of safety to 1.3 or 1.5 where required against failure.

A geotextile that provides the necessary tension for a chosen factor of safety.

(1) Tensile Requirements. Tensile requirements were computed using the following equation:

$$T = F.S.\,(D) - (D) - R$$

$$D = D_a - D_p$$

$$R = R_a + R_b + R_p$$

F.S. = required factor of safety

Since it is customary to report fabric strength in lbs/in, the T value is divided by 12.

Sufficient embedment length is available to develop the necessary tensile force.

(2) Embedment Length. The embedment length required to provide the frictional (cohesive) components to develop T, is calculated by combining the contributions from the top and bottom surfaces of the geotextile strip.

$$L = \frac{T}{[\gamma h \tan\phi + c]^{*} + [\gamma h \tan\phi + c]^{**}}$$

where

L = ft

T = lbs/ft

ϕ = friction angle between soil and geotextile

* = top surface

** = bottom surface

A length equal to or greater than L has to be available from the intersection of the active wedge and geotextile and into the stable portion of the slope.

3.2.2.4.2.2.1.7. I-Walls. The stability and required penetration of the steel sheet piling below the ground surface was determined by the method of planes. The long-term (S) shear strengths. (c = 0) governed for design. A factor of safety of 1.5 was applied to the friction angle as follows:

ϕ_d (developed friction angle) = $\tan^{-1}$ $\underline{(\tan \phi_a)}$/F.S.

This developed angle was used to determine $K_a = \tan^2 (45° - \phi_d/2)$, and $K_p = 1/K_a$. Using the two resulting developed shear strengths and net horizontal static eater pressure, the earth pressure diagrams were determined for movement toward each side of the sheet pile. Using. these pressure diagrams and the wave force, the summation of horizontal forces was equated to zero for various tip penetrations. The tip penetration required for stability was determined as that elevation at which the summation of overturning moments about the bottom of the sheet piling approached zero.

3.2.2.4.2.2.1.8. Floodwalls. Floodwalls are proposed for use in areas where an earth levee cannot be economically built. A new levee to Elev. 10 feet will be used to make the transition from the sand core levee alignment to the existing back levee. This new levee is designed for an initial F.S. of 1.2 and a final F.S. of 1.3 (after settlement to Elev. 7 feet). The existing levee I-wall composite section is designed for an F.S. of 1.3 against shear failure. For the high water hurricane loading case, with water to still water level, the I-wall sections are designed for an F.S. of 1.2 against shear failure. In all cases, the penetration of the sheet pile is designed for an F.S. of 1.5. The wave effect was applied as a line force acting at the centroid of the wave pressure diagram. At the site of the two pumping stations, the existing levee across each station will be degraded to Elev. 4 feet and an inverted T-wall construction on top of it. The I-wall will be tied into each end of the T-wall.

3.2.2.4.2.2.1.9. Erosion Protection. Due to the short duration of hurricane flood stages and the resistant nature of the clayey soils, no erosion protection, other than sodding, is considered necessary on the levee slopes along most of the levee alignment. However, foreshore protection will be constructed on the flood side levee toe in areas where damages could occur from waves generated by other than hurricane winds. This will be any berm or levee slope which is constructed into the open bays and bayous. The foreshore protection will consist of 24 inches of riprap on a 9-inch thick shell bedding. At the pumping stations, protection against erosion will consist of 18 inches of riprap over a 9-inch thick shell bedding.

3.2.2.4.2.2.2. Reach B1 to Tropical Bend to Fort Jackson – Approximately 12 miles of levee built by Hydraulic Dredge with shape-ups (Reference No. 53).

3.2.2.4.2.2.2.1. Geology. The project area is located within the Central Gulf Coastal Plain. Specifically, the area is located on the modern subdelta which projects gulfward from the deltaic plain of the Mississippi River. It is a region of extremely low relief. Dominant physiographic features are the natural levees of the Mississippi River and abandoned distributaries, and the marshlands and inland bodies of water that lie between the natural levee ridges. Elevations range from a maximum of +5 along the crests of the natural levees to a minimum approaching mean sea level in. the marshlands between the natural levee ridges. The numerous inland bodies of water vary in depth from 1 to 10 feet. The Mississippi River channel varies in depth from 70 to 190 feet below sea level.

3.2.2.4.2.2.2.2. Foundation Conditions. A generalized soil profile delineating the subsurface conditions along the project alignment shows that the subsurface consists of Recent deposits of very soft to medium clay soils with peat, silt, and sand layers. The upper 10 to 20 feet of marsh deposits generally consist of very soft organic clays, clays, and peat. Between Stations 0+00 and 399+00 the marsh deposit is underlain by interdistributary deposits of approximately 8 to 20 feet of layers of silt, silty sand, and sand. Below these layers is fat clay with layers of silt, silty sand, and sand. Between Stations 417+00 and 635+72 the marsh deposits are underlain by predominantly fat clay with intermittent thin layers of silt, sandy silt, and sand. Four abandoned distributaries are located below the marsh deposits between the following stations: 92+20 109+60, 398+50 - 417+50, 532+40 -551+90, and 610+50 - 615+30. These abandoned distributaries are composed of alternate layers of clay, silt, silty sand, and sand. The dominant feature in the design of all the levee sections is the very soft foundation condition between Elev. 0 and Elev. -12 feet.

3.2.2.4.2.2.2.3. Field Exploration. A total of 112 general-type and 17 undisturbed borings was made in conjunction with the project. Eight general-type borings were made by the Louisiana Department of Highways to locate a source of sand for borrow in the Mississippi River. Twenty-seven general-type and two 3-inch diameter undisturbed borings were made by the Louisiana Department of Public Works along the authorized levee alignment at the request of the Commission Council. Seventy-seven 1-7/8-inch I.D. core barrel and fifteen 5-inch diameter undisturbed borings were taken by the Corps of Engineers. The bottom elevations of the general-type and undisturbed borings range from -40 to -50 and -77 to -242, respectively

3.2.2.4.2.2.2.4. Levee Improvements. In general, the protection will consist of a levee. Between Stations 0+00 and 98+81 the protection will consist of a conventional hydraulic clay fill levee. From Station 104+81 to Station 635+72 the protection will consist of a hydraulic clay fill levee with a core composed of sand. A floodgate will be located where the Empire Waterway crosses the project alignment between Stations 98+71 and 104+91. Cantilever I-type and T-type walls will be used in the vicinity of the Sunrise (Station 232+31 to Station 242+41) and Grand Liard (Station 532+76 to Station 539+81) pumping stations to avoid relocations or major modifications to these facilities.

3.2.2.4.2.2.2.5. Pile Foundation

The T-walls will be supported by piling, battered as required, to provide stability against the unbalanced lateral waterloads. The inverted T-type floodwalls will be used in lieu of the I-type for reasons mentioned above. In compression, a factor of safety of 1.75 was applied to the shear strength and a lateral earth pressure coefficient of $K_0 = 1.0$ was used for determining the normal pressure on the pile surface. In tension, a factor of safety of 2.0 was applied to the shear strengths and a coefficient of $K_O = 0.7$ was used. One design was performed for both the (Q) and (S) cases for the Bayou Grand Liard pumping station and is applicable to the Sunrise pumping station since it was considered more conservative. The (Q) case governed. Settlement of the piles due to consolidation will not be a problem since the major loads are caused by hurricane waterheads of insufficient duration for consolidation of the foundation clays to ensue.

During construction, one 12-inch square concrete pile will be driven at the Bayou Grand Liard pumping station. The pile will be tested in compression to twice the design load (35 tons). If the pile fails before this load is reached the spacing will be adjusted accordingly. Since spacings of greater than 10 feet on the tension piles are not desirable, the tension piles will be working well below the design load, and no pile test will be performed in tension. Because of the small number of piles at the Sunrise pumping station, there will be no test piles at this site. In the interest of avoiding a tension pile test and having only one form for casting concrete piles, tension piles will be the same length as compression piles and spaced a maximum of 10 feet on centers, thus reducing the design load to well below the theoretical allowable tension load.

3.2.2.4.2.2.2.6. Levee Stability

Levees and dikes. In the interim between the publication of the GDM dated March 1967 and this GDM dated August 1971, plans and specifications were prepared for the first lift construction on two reaches of the project from Stations 0+00 to 98+55.3 and Stations 104+70 to 340+20. An additional set of plans and specifications were prepared by an A-E for the Commission Council and approved by the District Engineer for a reach of levee from Station 340+20 to Station 377+50. Plans and specifications for the remaining section between Stations 377+50 and 635+72 will be prepared after approval of this general design memorandum. Stability plates 89 through 116 are divided to reflect the above segments as follows:

Stations	Segments
0+00 to 98+81	Tropical Bend to Empire
104+81 to 337+72	Empire to Buras
337+72 to 635+72	Buras to Fort Jackson

(Q) shear stability analyses were performed for these segments using four different shear strength criteria as shown on plates 87 and 88. Using sections and (Q) shear strengths representative of the existing conditions along the alignment, the slopes and minimum berm distances for the levee and dike sections were determined the minimum factor of safety of the levee with respect to shear failure in the levee and foundation was 1.3 and 1.5 for failure into the adjacent borrow pit. The retaining and ponding dike sections were designed for a minimum factor of safety of 1.2 for failure into the sand core trench and interior dike borrow, respectively, and a minimum factor of safety of 1.3 for failure into the ponding area and borrow area, respectively. Borings 1-DU-1 and 2-DU-1 which were taken for design of the second lift showed no gain of shear strength. However, the second lift sections from 0+00 to 46+00 are somewhat larger than the first lift sections because the spoil from the first lift is serving as a flood side berm for the retaining dike. Since there was no spoil from 46+00 to 98+71 the second lift section is the same as the first lift section.

3.2.2.4.2.2.2.7. I-Walls. The stability and required penetration of the steel sheet piling below the fill surface was determined by the method of planes. The long-term (S) shear strengths (b = 0) governed the design. Prior to the preparation of plans and specifications for the I-type floodwall tying the final levee section to the I-wall in the existing back levee at Sunrise and the T-wall at Grand Liard, additional borings and analyses will be performed. A factor of safety of 1.25 was applied to the friction angle as follows:

ϕ_d -(developed friction angle) = $\tan \frac{-1(\tan \phi_A)}{F.S.}$

This developed angle was used to determine K_A and K_P lateral earth pressure coefficients as follows: $K_A - \tan_2 (45° - \phi_d/2)$ and $KP - 1/K_A$.

Using the resulting shear strengths and net horizontal static water, the earth pressure diagrams were determined for movement toward each side of the sheet pile. Using these pressure diagrams and the wave force, the summation of horizontal forces was equated to zero for various tip penetrations. The tip penetrations required for stability were determined as those where the summation of moments approached zero.

3.2.2.4.2.2.2.8. T-Wall. A steel sheet pile cutoff will be used beneath the T-walls to provide protection against seepage. The recommended tip elevations of the cutoff below the T-walls are shown on plates 25 and 26. No sheet pile analysis was performed for the Sunrise pumping station since the unbalanced waterload is negligible. The analysis for the Bayou Grand Liard pumping. station is shown on plate 120 and was analyzed under the following design assumptions:

a. Conventional (Q) shear stability analyses utilizing a F.S. of 1.5 applied to the soil strength parameters were performed at 1-foot intervals.

b. Net driving force = $D_p + R_A + R_B + R_p - D_{A.}$

c. The driving force above the base of the structure and the horizontal hydrostatic load were carried by the structure.

d. If the net driving force is positive there is available horizontal soil resistance in excess of the unbalanced waterload and therefore the bearing piles are not required to carry any additional lateral load acting on the sheet pile cutoff.

3.2.2.4.2.2.2.9. Erosion Protection. Due to the short duration of hurricane flood stages and the resistant nature of the clayey soils, no erosion protection is considered necessary on the levee slopes along most of the levee alignment other than sodding. However, foreshore protection will be placed along the bank of Adams Bay from Station 57+50 to the Empire floodgate, along the bank of the Empire to Gulf Waterway from Station 62+00 to the Empire floodgate, and on the land side and flood side of the canal closures between Stations 46+50 and 87+00 to protect the levee from damages which could occur from waves generated by other than hurricane winds. Design sections for the foreshore protection are shown on plate 22. At the Sunrise and Bayou Grand Liard pumping stations the erosion protection will consist of 18 inches of riprap over a 6-inch thick shell bedding. Erosion protection at the Empire floodgate will consist of 2 feet of riprap on a minimum 1-foot blanket of clamshell.

3.2.2.4.2.2.3. Reach B2 – Ft. Jackson to Venice. Nine miles of levee constructed by hydraulic lifts and shaping to Elev. 50 feet with 8-ft crown with (Reference 50)

3.2.2.4.2.2.3.1. Geology. The project area is located within the Central Gulf Coastal Plain. More specifically, the area is situated on the deltaic plain of the Mississippi River, a region of extremely low relief. Dominant physiographic features are the natural levees of the Mississippi

River and its abandoned distributaries, and the marshlands and inland bodies of water that lie between the natural levee ridges. Elevations range from a maximum of approximately 5 along the crests of the natural levees to a minimum approaching mean sea level in the marshlands between the natural levee ridges.

3.2.2.4.2.2.3.2. Foundation Conditions. The subsurface consists of recent deposits of very soft to medium clay soils with peat, silt, and sand layers. The upper 5 to 18 feet of marsh deposits generally consist of very soft organic clays, clays, and peat. Between Stations 0+00 and 4+50 the marsh deposits are underlain by interdistributary deposits of soft clay with layers of silt. Between Station 4+50 and 480+31 the marsh deposits are underlain by 20 to 40 feet of intradelta deposits consisting primarily of very soft to medium clays with alternating lenses and layers of silt, sand, and silty sand. These deposits are in turn underlain by interdistributary deposits consisting of soft to medium clays with very few lenses and layers of silt. Two abandoned distributaries are located below the marsh deposits – one between Stations 457+60 and 460+60 and the other between Stations 466+00 and 469+00. These abandoned distributaries are composed of alternate layers of clay, silt, silty sand, and sand. The dominant feature in the design of all the levee sections is the very soft marsh deposits in the upper 5 to 18 feet of the foundation.

3.2.2.4.2.2.3.3. Field Exploration. A total of 39 general type and nine undisturbed borings were made in association with the Reach B2 project. Twenty general type borings were made by the Louisiana Department of Highways to locate a source of sand for borrow between mile 12 and mile 18.5 in the Mississippi River. Nineteen 1-7/8 inch I.D. core barrel and nine 5-inch diameter undisturbed borings were taken by the Corps of Engineers, New Orleans District. The bottom elevations of the general type and undisturbed borings ranged from -45 to -79 and -71 to -237, respectively. Prior to the preparation of plans and specifications, additional general type borings will be taken in the sand and clay borrow areas.

3.2.2.4.2.2.3.4. Levee Improvements

The Reach B2 project will consist of a sand core hydraulic clay fill levee, and extend from a junction with the terminus of the proposed Reach B1 project levee in the vicinity of Fort Jackson (Station 0+00) for about 9 miles southeast to a junction with the proposed highway ramp for relocation of Louisiana Highway 23 at Venice (Station 475+33). The proposed realigned Mississippi River levee will join the opposite side of the highway ramp to complete the Reach B2 project. The Reach B2 levee, the realigned river levee, and the highway ramp will be constructed to Elev. 15 feet, thereby forming a uniform net grade for the Reach B2 levee system. The Reach B2 levee centerline will be approximately 190 feet marshward and generally parallel to the existing non-Federal back levee. Minor changes in levee centerline location will be permitted in the field where the changes will result in a more favorable alignment.

Floodwalls at Venice Pumping Station. The Venice pumping station is located on the protected side of the existing back levee with discharge pipes passing through the levee just below the road surface on the levee crown. To provide continuous protection at minimum cost, the new levee will tie into the existing back levee approximately 100 feet to each side of the discharge pipe crossings. Inverted T-type floodwall in the existing levee and I-type floodwall in the tie-in levees will be constructed to Elev. 19 feet. The tie-in levees will have an 8-foot crown width at Elev. 10 feet. Stability of the existing levee requires that it be degraded to Elev. 5 feet and the slopes be regraded to 1 on 3. Where the discharge pipes pass through the floodwall, provisions to accommodate settlement or deflection of the wall or any small movements of the pipes will be provided.

3.2.2.4.2.2.3.5. Pile Foundations

The T-wall will be supported by piling, battered as required, to provide stability against the unbalanced lateral waterloads. In compression, a factor of safety of 1.75 was applied to the shear strength and a lateral earth pressure coefficient of $K_0 = 1.0$ was used for determining the normal pressure on the pile surface. In tension, a factor of safety of 2.0 was applied to the shear strengths and a coefficient of $K_0=0.7$ was used. Design of the T-wall pile foundation was performed for both the (Q) and (S) cases. The (Q) case governed. Pile design loads vs. tip elevations, and subgrade moduli vs. tip elevations are shown on plate 72. Settlement of the piles due to consolidation is not expected since the major loads are caused by hurricane-induced stages of insufficient duration for consolidation of the foundations clays to ensue.

During construction, one 12-inch square concrete pile will be driven to the design tip elevation (-50.4) in the vicinity of the Venice pumping station. The test site will be located in the vicinity of boring 28-B2UC. The pile will be tested in compression to 78 tons (twice the design load). If the pile fails before this load is reached, the spacing will be appropriately adjusted. To eliminate a tension pile test and have only one form for casting concrete piles, tension piles will be the same length as compression piles (60 feet) and spaced a maximum of 10 feet on centers thereby reducing the design load to 22.5 tons which is well below the theoretical allowable tension load of 30 tons. If the spacing of compression piles has to be reduced, the spacing of tension piles will be reduced by the same ratio.

3.2.2.4.2.2.3.6. Slope Stability

a. *Levees and Dikes.* Using levee sections and (Q) shear strengths representative of conditions along the project alignment, slopes and minimum berm distances for the levee and dike sections were determined by the method of planes. Levee sections were designed for a minimum factor of safety of 1.3 with respect to shear failure in the levee and foundation and 1.5 for failure into the adjacent borrow pit. The retaining dike sections were designed for a minimum factor of safety of 1.3 for failure into either the sand core trench or the retaining dike borrow pit. The pending dike sections were designed for a minimum factor of safety of 1.2 for failure into the interior dike borrow pit and a minimum factor of safety of 1.3 for failure into the marsh borrow area.

b. *Floodwalls.* A combination of I-type and inverted T-type floodwalls will be used at the Venice pumping station. The use of I-wall along the existing back levee at this location was not feasible because a minimum levee crown elevation of 10.0 would be required to prevent excessive deflection of the wall. A stability analysis was performed with the levee crown at Elev. 10.0 feet and the I-wall in place. In order to maintain the required factor of safety of 1.30, large stability berms would be necessary in both the landside and flood-side drainage pits resulting in either relocation or major modifications to the pumping station. Therefore, a 365-foot length of 7-wall with the levee degraded to Elev.

5.0 feet will be used along the existing back levee with I-wall joining the T-wall to the full earthen levee section. For the stability analyses, the wave effect was applied as a line force acting at the centroid of the wave pressure diagram. The water pressure diagram resulting from the wave action alone was considered effective only to the levee crown.

3.2.2.4.2.2.3.7. I-Walls

Cantilever I-wall. The stability and required penetration of the steel sheet piling below the fill surface was determined by the Method of Planes. The long-term (S) shear strengths (c = 0) governed for design. Prior to the preparation of plans and specifications for the I-wall tying the full earthen levee section to the T-wall at the Venice pumping station, additional borings and analyses will be performed. A factor of safety of 1.25 was applied to the friction angle as follows: ϕd (developed friction angle) = tan -1 $\left(\frac{\tan\phi_A}{F.S.}\right)$. This developed angle was used to determine KA and KP (lateral earth pressure coefficients) as follows:

$$K_A = \tan^2\left(45° - \frac{\phi_d}{2}\right)$$

and KP = $\frac{1}{K_A}$**.** Using the resulting shear strengths and net horizontal static water pressure, the earth pressure diagrams were determined for movement toward each side of the sheet pile. Using these pressure diagrams and the wave force, the summation of horizontal forces was equated to zero for various tip penetrations. The tip penetration required for stability was determined as that elevation at which the summation of overturning movements about the bottom of the sheet piling approached zero.

3.2.2.4.2.2.3.8. T-Walls

Steel sheet pile cutoff. A steel sheet pile cutoff will be used beneath the T-wall to provide protection against seepage. The stability analysis of the T-wall was based on the following:

Conventional (Q) shear stability analyses were performed at 1-foot intervals from the bottom of the structure base to the sheet pile tip, utilizing a factor of safety of 1.3 applied to the soil strength parameters.

The value of R_b at the bottom of the base of the-structure was assumed equal to zero.

The net force equals D_a - (D_p + R_a + R_b + R_p) and was determined at each increment of depth.

The driving force above the base of the structure and the horizontal hydrostatic load are carried by the structure.

The algebraic difference in the net forces at the top and bottom of each 1-foot interval was used to develop the pressure diagram."

If the algebraic difference is negative, the available horizontal soil resistance is in excess of the unbalanced waterload, and the bearing piles are not required to carry any additional lateral load acting on the sheet pile cutoff.

3.2.2.4.2.2.3.9. Erosion Protection. Due to the short duration of hurricane flood stages and the resistant nature of the clayey soils, no erosion. protection, other than sodding, is considered necessary on the levee slopes along most of the levee alignment. However, foreshore protection will be constructed on the floodside levee toe in the Bay Carrion Crow area from Station 232+00 to Station 263+00 to protect the levee from damages which could occur from waves generated by other than hurricane winds. The foreshore protection will consist of 21 inches of riprap on a 9-inch thick shell bedding. At the Venice pumping station, protection against erosion will consist of 18 inches of riprap over a 9-inch thick shell bedding.

3.2.2.4.2.2.4. Reach C Phoenix to Bohemia. The existing 16 miles of interior earth fill will be enlarged and raised from an elevation of approximately 14 to a net elevation of 17. **(Reference No. 51)**

3.2.2.4.2.2.4.1. Geology. The project area is located within the Central Gulf Coastal Plain. More specifically, the area is situated on the deltaic plain of the Mississippi River, a region of extremely low relief. Dominant physiographic features are the natural levees of the Mississippi River and its abandoned distributaries, and the marshlands and inland bodies of water that lie between the natural levee ridges. Elevations range from a maximum of approximately 5 along the crests of the natural levees to a minimum approaching man sea level in the marshlands between the natural levee ridges. The numerous inland bodies of water vary in depth from 1 to 6 feet. The Mississippi River channel in the vicinity of the project area varies in depth from 70 to 190 feet below mean sea level.

3.2.2.4.2.2.4.2. Foundation Conditions. The subsurface consists of Recent deposits that vary in depth from approximately 112 feet at the upstream end of the project to about 131 feet at the downstream end. The Recent deposits are underlain by Pleistocene (Prairie formation) deposits. Generally, the Recent consists of a 6- to 12-foot surface layer of very soft to soft marsh deposits with organic material and peat. The marsh deposits consist generally of clays with organic matter and peat, underlain by interdistributary deposits of very soft to soft clays containing lenses and layers of silt and silty sands. The interdistributary deposits vary in thickness from 38 feet in the vicinity of Station 10+00 to about 50 feet at approximate Station 650+00. Underlying the interdistributary clays at elevations varying between -110.0 in the vicinity of Station 10+00, and -119.0 near Station 800+00 are medium to stiff prodelta clays. The prodelta clays overlie a thin wedge of nearshore sands with shell and shell fragments which thickens from a minimum of about 2 feet at Station 10+00 to a maximum of about 12 feet near Station 800+00. The entire sequence of Recent sediments is underlain by stiff to very stiff Pleistocene clays at elevations ranging from -112.0 at Station 10+00 to -131.0 at Station 800+00.

3.2.2.4.2.2.4.3. Field Investigation. Two 5-inch diameter undisturbed borings approximately 110 feet in depth were made. Nine additional undisturbed borings approximately 100 feet in depth were made. The nine additional undisturbed borings were equally divided between three locations, each location having a levee centerline boring and a boring at the levee toe on each side of the centerline. A total of 31 general type core borings, 1-7/8-inch I.D. were made. Twenty-four of the general type borings extended approximately 50 feet in depth. The remaining seven borings extended to 80 feet in depth. Two general type borings were made in the recommended borrow area in the Pointe a la Hache Relief Outlet.

3.2.2.4.2.2.4.4. Seepage. Approximately 10 feet of clay cover above the sand core will be provided on the flood side of the levee. Due to the relatively short duration of hurricane headwaters, this is considered sufficient to prevent seepage.

3.2.2.4.2.2.4.5. Pile Foundations. Not Used.

3.2.2.4.2.2.4.6. Sliding Stability

a. *Levees.* Based on varying soil conditions, the Reach C levee was divided into three subreaches - Station 0+00 to Station 159+00, Station 159+00 to Station 495+00, and Station 495+00 to Station 834+85.0 (end of project). Undisturbed borings were made at Stations 14+06 (borings 30 CUT, 30 CU, and 30 CUTP), 303+05 (borings 8 CUT, 8 CU, and 8 CUTP), and 687+50 (borings 21 CUT, 21 CU, and 21 CUTP). Stability of the proposed levee sections was investigated for each subreach using soil properties and strengths derived from the appropriate set of undisturbed borings. Stability was determined by the Method of Planes based on a minimum factor of safety of 1.3 with respect to shear strength. Stability was investigated at various depths in the foundation, and factors of safety with respect to shear strength were determined for various assumed failure planes. Berms on the flood side of the levee are not necessary for levee shear stability, but are provided as a means of dissipating a portion of the wave energy and thus reducing the required levee grade.

b. *Bohemia spillway canal.* Between Stations 801+09.5 and 834+85 at the lower end of Reach C, the spillway canal parallels the levee. Because of this condition, a stability computation was performed in addition to the ones calculated as generally representative of the subreach between Stations 495+00 and 834+85.0. The results shown on plate 42 indicate factors of safety in excess of the required minimum of 1.3. The assigned foundation stratification and design shear strengths were those determined from the set of undisturbed borings at Station 687+50.

c. *Gravity drainage structures.* Five drainage structures have been constructed by local interests along the Reach C alignment: Conventional stability computations were made for three structures, one in each subreach which represented the most critical condition in the respective subreach. The stability analyses were based on the foundation stratifications, design shear strengths, and water conditions appropriate for the respective subreaches. In addition, a surcharge was added to the passive wedges. The surcharge was computed by dividing the total weight of concrete and sacked riprap above the soil by an assumed failure width. Routine stability computations were made assuming the culverts were non-existent, which is a conservative assumption. Two of the structures, one at Station 89+50 and the other at Station 425+50, satisfied the requirement for a 1.3 factor of safety. The third structure, at Station 548+60, did not meet the 1.3 criteria by conventional stability computations, assuming an infinite width. It was necessary, therefore, to perform a mass stability analysis at this location. The result of this analysis indicates a factor of safety in excess of 1.3 with respect-to shear stability (see plate 45).

d. *Pumping Stations.* Two pumping stations have been recently constructed along the project alignment by local interests. One is located at Station 241+77 near Bellevue, the other at Station 551+38 near Pointe a la Hache. Stability of the levee sections adjacent to the connecting cantilever I-type floodwalls was analyzed. The most critical condition was found at the Pointe a la Hache pumping station. Computations for this location indicate a factor of safety of 1.42, assuming levee failure into the discharge canal. The assigned foundation stratification and design shear strengths used in the above computations are those appropriate to the subreaches containing the pumping stations.

3.2.2.4.2.2.4.7. I-Walls. The floodwalls between Stations 240+72 and 242+82 and Stations 550+33 and 552+43 are I-type cantilever sheet pile walls consisting of P2-27 steel sheet pile capped with concrete. The sheet piling extends from Elev. 13.0 feet to Elev. -20.0 feet, and the concrete cap is provided between Elev. 10.0 feet and Elev. 20.0 feet. In order to provide a seepage cutoff, the sheet piling was extended below the elevation required for stability purposes. Seepage cutoff is also provided under the discharge basin side and backwalls to obtain a continuous diaphragm. A wave berm has been constructed along the floodwall alignment to dissipate hurricane wave forces on the floodwalls. The floodwalls are designed to withstand loading from an 8-foot broken wave.

3.2.2.4.2.2.4.8. T-Walls. Not Used.

3.2.2.4.2.2.4.9. Erosion Protection. Because of the relatively short duration of hurricane flood stages and the resistant nature of the clayey soils, erosion protection other than sodding is not considered necessary along the major length of the levee. In the vicinity of the two pumping stations, adequate erosion protection is provided by riprap along the flood side of the cantilever I-type floodwalls. Erosion protection at the existing gravity drainage structures consists of sacked concrete riprap.

3.2.2.4.2.2.5. Empire Floodgate New Orleans to Venice (Reference 69)

3.2.2.4.2.2.5.1. Geology. The geology within the general area of the Empire Floodgate is presented in Reference 50.

3.2.2.4.2.2.5.2. Project Foundation Conditions. The foundation soils consist predominantly of Recent backswamp clays having soft to medium consistencies, and extending to depths of approximately 90 feet below the natural ground surface. The Recent clays contain 3- to 5-foot thick layers of silts and sands at approximate Elev. -20 feet, Elev. -30 feet, and Elev. -50 feet. The 5- to 10-foot thick clay layer extending from the ground surface contains organic matter with some peat.

3.2.2.4.2.2.5.3. Field Exploration. One 5-inch diameter undisturbed boring and four general-type disturbed core borings were made for this investigation. The undisturbed and general-type borings extended in depth to approximate elevations of -90 feet and -80 feet, respectively.

3.2.2.4.2.2.5.4. Unwatering and Hydrostatic Pressure Relief During Construction. In order to construct the floodgate and floodwalls in the dry and to insure stability of the structure excavation during construction, hydrostatic pressure relief was to be provided in the silt and sand layers within the soil foundation. The pressure relief was to be accomplished by vertical sand drains and well points. To allow time for pore pressure relief, the rate of unwatering of the working area was to be maintained at a maximum of 2 feet per day for the first 10 feet, and 1 foot per day thereafter until completely unwatered. Temporary construction piezometers were to be installed in the pervious layers to monitor the pore pressure during the unwatering and pressure relief-period. After the structure was complete and operating, the sand drains were expected to discharge into the shell backfill and provide a degree of permanent pressure relief. Conventional sumps and pumps were to maintain the area free of surface water during construction.

3.2.2.4.2.2.5.5. Slope Stability

a. *Construction Slopes.* The stability of the excavation, dike and closures, existing first lift levee, and berm distances were determined by the Method of Planes based on a minimum factor of safety of 1.3 with respect to shear strength and the (Q) design shear strengths. Stability was investigated at various depths in the foundation, and factors of safety with respect to shear strength were determined for various assumed failure planes. The relief facilities were to provide the required pressure reduction in the pervious layers for stability.

b. *Final Slopes.* The (Q) stability governed for design of the final slopes. The final slopes were to be constructed by clamshell backfilling. In the vicinity of the structure, the inclinations of the rebuilt slopes were determined by the requirement that the length of the floodwalls be as short as possible without sacrificing stability of the tie-in levee into the inlet and outlet channels. The remaining rebuilt slopes were designed to be stable with a minimum of backfilling.

3.2.2.4.2.2.5.6. Cantilevered I-Wall. The results of tidal hydraulic analyses indicated that the I-wall would be subjected to the pressure and forces imparted by breaking waves. In the stability analyses, the dynamic wave effect was applied as a line force acting through the centroid of the dynamic wave pressure distribution diagram. The static water pressure diagram resulting from wave action was considered effective only to the top of the impervious clay, inasmuch as the period of time the wave would exist was considered too short to allow water pressure to become effective in the impervious clays. The stability and required penetration of the steel sheet piling below the fill surface were determined by the Method of Planes. The long-term (S) shear strengths ($C = 0$) governed for design. A factor of safety of 1.25 was applied to the friction angle. This developed angle was used to determine K_A and Kp lateral earth pressure coefficient values.

Using the resulting shear strengths, net horizontal water and earth pressure diagrams were determined for movement toward each side of the sheet pile. The depths of penetration required for stability were determined as those where the summation of moments was equal to zero.

3.2.2.4.2.2.5.7. Control Structures and T-Walls

a. *Steel sheet pile cutoff.* A steel sheet pile cutoff was to be used beneath the floodgate and T-walls to provide protection against hazardous seepage. The net pressure diagram along the sheet pile cutoff-was determined as follows:

(1) Conventional stability analysis by the method of planes, utilizing a factor of safety of 1.3 incorporated in the soil strength parameters, was performed to determine the stability against rotational failure. The analysis was performed at 1-foot vertical intervals with the active wedge located at the flood side edge of the structure and the passive wedge located at the protected side edge of the structure.

(2) The assumption was made that the value of (R_B) at the bottom of the base of the structure was zero.

(3) For each analysis, the net driving force, i.e., (D_A - Dp) - (R_A + R_B + Rp) was determined. The value of D_A included the weight of water between the tailwater elevation and the SWL elevation located above the active wedge.

(4) The assumption was made that the net driving force above the bottom of the base of the structure was carried by the structure.

(5) Considering driving (D_A) positive and all resistance negative (Dp, Rp, R_E, and R_A) in the expression $D = D_A - D_p - R_p - R_B - R_A$, using the method of planes stability analyses, $\sum D$ was determined by assuming failure at the bottom of the base of the structure and at each foot in depth thereafter. The value of the algebraic difference in $\sum D$, between 1-foot intervals, was used to develop the pressure diagram. If the incremental difference were negative, the pressure diagram indicated an available horizontal resistance in excess of that required, and if the incremental difference were positive, the pressure diagram indicated an unbalanced horizontal pressure in excess of the available soil resistance. It was considered that such an excess must be carried by the sheet pile cutoff.

(6) The net pressure diagrams indicated that the total available horizontal resistance was in excess of the total horizontal waterload. Therefore, the analyses indicated that the bearing piles were not required to carry any additional lateral load acting on the sheet pile cutoff.

b. *Bearing pile foundations.*

(1) The floodgate and T-walls were to be supported by piling, battered as required, to provide stability against the unbalanced lateral waterloads. The inverted T-type floodwalls were to be used in lieu of I-type floodwalls where the height of the I-wall above ground and the magnitude of the dynamic wave force rendered the I-type floodwall impracticable. In compression, a factor of safety of 1.75 was applied to the shear strengths, and a lateral earth pressure coefficient (Ko) = 1.0 was used for determining the normal pressure on the pile surface.- In tension, a factor of safety of 2.0 was applied to the shear strengths and a coefficient (K0) = 0.7 were used. Settlement of the piles due to consolidation were not indicated to be a problem since the major loads were caused by hurricane water heads of insufficient duration for consolidation of the foundation clays to ensue.

(2) During construction, three 12-inch diameter class B untreated timber piles of different lengths were to be driven. The intermediate pile was to be tested in compression. If test results showed that the pile could safely carry twice the design load, the pile would be tested in tension. If the intermediate pile failed before the required capacity was attained in compression, the long pile would be tested in compression and in tension. If the intermediate pile safely carried compression loads significantly in excess of that required, the short pile would be tested in compression and in tension. Pile test loads were to be 15 tons in tension and 40 tons in compression.

c. *Shell backfill.* Clamshell was to be used as backfill around the structure to reduce lateral pressures, and to keep the settlement of the riprap protection and the heights and lengths of the floodwalls to a minimum.

d. *Impervious levee and berm fill.* After the floodgate and floodwalls were completed and protection against flooding was no longer necessary, the material in the temporary protection dike was to be used in the levees and berms at the end of the tie-in walls.

e. *Erosion protection.* To protect against loss of channel and backfill material due to erosion and subsequent undermining of the floodgate and floodwalls, 2 feet of riprap on a minimum 1-foot blanket of clamshell was to be provided.

f. *Settlement observations.* Settlement observations were to be made along the structure and floodwalls promptly after construction and yearly thereafter.

3.2.2.4.2.2.5.8. Spoil Disposal. The major portion of the first stage excavation material was to be used to construct the land dikes and a significant portion of the second stage excavation material was to be used to construct the inside berms for the stream closures. The material remaining to be excavated was to be deposited in the tie-in levee areas. A portion of the material was also to be stockpiled in certain areas outside of the protection dike for use in selective backfilling of the excavation in the vicinity of the structure.

3.2.2.4.2.3. Structural

3.2.2.4.2.3.1. Reach A - City Price to Tropical Bend (Reference 52)

General. The project plan consists of protective levees and appurtenant features. The levee system is approximately 12.8 miles in length, with a net elevation ranging from 12.5 feet NGVD at the beginning near City Price to 14.5 feet NGVD at the lower end near Tropical Bend. Structural features include floodwalls at the City Price drainage structure, Hayes Canal pumping station, Freeport Sulphur unloading dock, and Gainard Woods pumping station. The pumping station discharge pipes will pass through the floodwall, but will be modified t o prevent potential backflow during high outside stages.

Structure Elevations		
Location	**Top of Wall El.**	**Design Water Surface El. ft. NGVD**
City Price Floodwall	12.5 16.0	8.9 8.9
Hayes Canal Pumping Station Floodwall	16.0	9.2
Freeport Sulphur Floodwall	16.0	9.2
Gainard Woods Pumping Station Floodwall	17.0	9.6

I-Type Floodwalls. I-type floodwalls are constructed at Hayes Canal and Gainard Woods Pumping Stations, City Price, and at Freeport Sulphur. The load case which controls design is water load to the still water level (see above) plus the wave loads computed from guidelines outlined in “Shore Protection Manual”, Volume II, 1971. The required factor of safety is 1.5, S-case soil conditions.

T-Type Floodwall. T-type floodwalls are constructed at the Hayes Canal Pumping Station and at the Gainard Woods Pumping Station. Load cases for the T-walls are as follows:

Load Case		Symbol
I	Dead Load	DL & WL
II	Water Load and Impervious Uplift	UI
III	Pervious Uplift	UP
IV	Wave Load	WL

For pile design, no load factors were used (working stress) and the following load cases were considered:

No.	LD Combination
1	DL + WL + UI
2	DL + WL + UP
3	.75 (DL + WL + UI + WL)
4	.75 (DL + WL + UP + WL)

3.2.2.4.2.3.2. Reach B1 Tropical Bend to Fort Jackson – Floodgate at Empire - (Reference 54)

General. The floodgate consists of a reinforced concrete U-frame gate bay with a clear opening of 84 feet and sill elevation of -14.0. A steel gate is hinged at the bottom of the structure. The entire structure is supported on untreated timber piling. The total structure width is 106 feet and the top of the walls are at elevation 15.0. A control house is provided above one wall for operation of the gate, and needle dams are provided for unwatering the gate while the gate is in the closed position.

A 300-foot timber guide wall and a 100-foot long timber fender are located on each side of the gate structure. The guide wall is on the west side of the channel and the fender is on the east side of the channel.

An inverted T-type reinforced concrete floodwall abuts the structure wall and extends for a distance of 150 feet on each side of the structure, at which point I-type reinforced concrete flood-walls extends an additional 105 feet on each side of the structure. The top of the floodwalls will be at elevation 15.0.

Design Water Elevations (feet m. s. l.)		
	Gulf side	**Landside**
Direct head from hurricane	+12.1	+2.0
Reverse head from hurricane	-2.0	+6.3
Direct head for maintenance	+5.0	-1.0
Reverse head for maintenance	-2.0	+5.0

Structure Elevations (feet m. s. l.)	
Top of wall	**+15.0**
Top of timber guide walls & fenders	+ 9.5
Top of sill	-17.5/-14.0
Centerline of gate hinges	-15.54
Centerline of hoist wildcat	+17.75
Centerline of cwt, wildcats	+15.0/+21.0
Centerline of needle girders	+5.0
Bottom of channel outside limits of riprap	-12.0

Design Loads. The assumed design loads used in the design of the structure, gate, and abutment walls are tabulated below:

Lateral pressures (p.s.f. / f t.)	**Submerged**	**Saturated**
Earth	25.8	54.0
Shell	13.5	41.6
Riprap	28.4	56.5

Uniform live loads	**Lbs. per sq. ft.**
Walkways & stairs	100
Control building floor	200
Control building roof	20

Wind loads. Wind loads on exposed vertical surfaces and projected area of sloped surfaces. (Allowable stresses increased one-third) - 30 p.s. f.

Wave loads. Net wave pressures have been computed from the hurricane design wave data in accordance with recommendations of "Shore Protection, Planning and Design," Technical Report No. 4, Third Edition, 1966, by the Coastal Engineering Research Center, Corps of Engineers. The hurricane design wave was assumed to approach the structure a t a 90' angle.

Allowable Working Stresses - The allowable working stresses for structural steel and concrete are in accordance with those recommended in "Working Stresses f o r Structural Design," EM 1110-1-2101 of 1 November 1963.

Application of Working Stresses.

Group 1 Loading: Allowable working stresses as listed for structural steel and for reinforced concrete will be applied t o the following loads:

Dead load
Live load
Buoyancy
Earth pressure
Water pressure

Group 2 Loading: Allowable working stresses as listed for structural steel and for reinforced concrete will be applied t o the following loads when combined with Group 1 loads with a general allowance of an increase of 33 1/3 % over allowable stresses:

Wind loads
Wave loads

Pile Foundation and Stability Analysis. The pile foundations were designed in accordance with EM 1110-2-2906, July 1969, "Design of Pile Structures and Foundations." Computed pile - loads were determined from the rational method of pile foundation analysis (method developed by A. Hrennikoff)

Design loading conditions for the Concrete U-Frame Gate structure

- Case I – Operating conditions. Maximum direct head (hurricane). Gate closed; flood side water at elevation +12.1, protected side water at elevation +2.0; uplift with sheetpile cutoff considered impervious--no wave force.
- Case II – Same as Case I, except uplift with sheet pile cutoff considered pervious.
- Case III – Maximum direct head with wave forces (hurricane). Gate closed; flood side water at elevation +12.1, protected side water a t elevation +2.0; uplift with sheet pile cutoff considered impervious.
- Case IV – Same as Case III except uplift with sheet pile cutoff considered pervious.

- Case V – Maximum reverse head. Gate closed; flood side water a t elevation -2.0, protected side water a t elevation +6.3; uplift with sheet pile cutoff considered impervious.
- Case VI – Same as Case V except uplift with sheet pile cutoff considered pervious.

Non-operating conditions

- Case VII – Gate dewatered. Gate removed; needle beams and girders in place; flood side water at elevation +5.0; protected side water a t elevation +5.0; full uplift.
- Case VIII – Construction condition. Gate closed; no uplift.

Cases III and IV are considered Group 2 loadings. All other cases considered Group 1 loadings.

3.2.2.4.2.3.3. West Bank Mississippi River Levee – City Price to Venice (Reference 69)

General. The project plan provides for enlargement of the west bank Mississippi River levees and construction of levee setbacks and floodwalls. The only structure in the project is the existing Empire Lock at Empire, La. It was determined that no modification was necessary to the lock gates. Wave overtopping would be allowed; however, the 30 feet of existing I-wall on both sides of the structure was found to be inadequate to withstand the projected hurricane wave force if capped to project height. The length of the existing sheet piling is inadequate. For structural and constructability reasons, the existing sheet piling shall be removed and replaced with adequate lengths of new PZ-27 sheet piling.

Basic data relevant to water surface elevations, structure elevations, and dimensions are summarized below:

Structure Elevations					
Top of Wall El.	Design SWL Elev. ft. NGVD	Wave Loads			
		fm, psf	hc, ft	psw, psf	ds, ft
21.5	13.5	336	7.3	448	0

Wave forces were computed from guidelines outlined in “Shore Protection Manual”, Volume II, 1971.

Strength Design Criteria. The concrete structures are designed in accordance with ETL 1110-2-265, “Strength Design Criteria for Reinforced Concrete in Hydraulic Structures” dated 15 Sept 1981, and AC I 318-77, “Building Code Requirements for Reinforced Concrete”. Design values used are listed below:

f ‘c	3,000 psi
fy (reinforcement)	40,000 psi

P	.25 P_b
Pmin (flexure)	200/fy o r 1/3 greater than required by analysis
min temp steel	.002 b t - (half in each face)
v_c	2 $(f'c)^{1/2}$
Sheet Pile	ASTM-A328 (19,500 psi allowable)

Design of I-type Floodwalls. The load case which controls design is water load to the stillwater level (see above) plus the wave loads computed from the information given. A factor of safety equal to 1.5 was used in design of the sheet piling assuming a cantilever design under an S Case soil condition.

3.2.2.4.2.3.4. Reach B2 Fort Jackson to Venice (Reference 50)

General. The structural features of the project consist of "I" and "T"-walls at the Venice pumping station. The Venice pumping station is located on the protected side of an existing back levee with discharge pipes passing through the levee just below the road surface on the levee crown. To provide continuous protection at minimum cost, the new levee ties into the existing back levee approximately 100 feet to each side of the discharge pipe crossings Inverted T-type floodwall in the existing levee and I- type floodwall in the tie-in levees were constructed to provide the continuous line of protection.

Criteria for Structural Design. The structural design of the floodwall complies with standard engineering practice and criteria set forth in Engineering Manuals for civil Works Construction published by the Office o f the Chief of Engineers, Wave forces were computed using guidelines outlined in Technical Report No. 4, third edition, 1966, "Shore Protection Planning and Design" published by the U.S. Army Coastal Engineering Research Center with the exception that breaking waves were not considered to act on the total structures (see WES Research Report H-68-2, dated September 1968, "Shock Pressures Caused by Waves Breaking Against Coastal Structures")

Basic Data

Still water level (SWL), flood side	11.5
Assumed water elevation landside of floodwall	-5.0
Unit weight of water	62.5 p.c.f.
Unit weight of reinforced concrete	150.0 p.c.f.

Allowable Working Stresses. The a1lowable working stresses for concrete and structural steel are in accordance with those recommended in "Working Stresses for Structural Design," EM 1110-1-2101, dated 1 November 1963 and amendment 1, dated 14 April 1965.

I-Type Floodwall. The I- wall consists of sheet piling driven into the final levee sections and capped with concrete. For design of the I-wall, two loading cases were considered:

- Case I – Static water to the SWL, elevation 11.5; 1.5 factor of safety in the soil; and no wave force.
- Case II – Static water to SWL, elevation 11.5; 1.25 factor of safety in the soil; and wave load from non-breaking wave.

T- Type Floodwall. A reinforced concrete T-wal1 section will be supported by battered prestressed concrete piles driven into the levee section. The sheet pile cutoff wall below the T-wall base is assumed t o be self- supporting and, therefore, does not cause or resist any load on the T-wall. The T-wall was designed assuming the following loading cases:

- Case I – Static water t o SWL, elevation 11.5; no wave force; and impervious sheet pile cutoff.
- Case II – Static water t o SWL, elevation 11.5; no wave force; and pervious sheet pile cutoff.
- Case III – Static water t o SWL, elevation 11.5; wave load from non-breaking wave; impervious sheet pile cutoff; and 33 1/3 percent increase in allowable stresses.
- Case I V – Static water t o SWL, elevation 11.5; wave load from non-breaking wave; pervious sheet pile cutoff; and 33 1/3 percent increase in allowable stresses.

3.2.2.4.2.3.5. Reach C Phoenix to Bohemia (Reference 51)

General. Two pumping stations one near Bellevue, the other near Pointe a la Hache, recently constructed by local interests; include provisions for protection from design hurricane tides at these locations. A continuous protective system is provided by the floodwalls between the discharge basins and the adjacent levees; the discharge basin sidewalls; and the backwalls of the discharge basins. The critical structure loadings resulting from design hurricane induced stage differentials are transmitted from the discharge basin backwalls through longitudinal shear walls and ultimately distributed to all the structural components of the pumping stations. Therefore, essentially the entire pumping station is used to resist these loads.

In addition, floodwalls between Stations 240+72 and 242+82 and Stations 550+33 and 552+43 are I-type cantilever sheet pile walls consisting of PZ-27 steel sheet pile capped with concrete. The sheet piling extends from elevations 13.0 to -20.0, and the concrete cap is provided between elevations 10.0 and 20.0. In order to provide a seepage cutoff, the sheet piling was extended below the elevation required for stability purposes. Seepage cutoff is also provided under the discharge basin side and backwalls to obtain a continuous diaphragm. A wave berm has been constructed a1ong the f floodwall a1inement to dissipate hurricane wave forces on the floodwalls. The floodwalls are designed to withstand loading from an 8-foot broken wave.

Structural Design Criteria. There are no design analyses prepared by the Corps of Engineers for these structures. However, since the pumping stations and connecting floodwalls are an integral part of the Reach C hurricane protective system, the structural designs prepared by local interests were coordinated with and approved by the New Orleans District, Corps of Engineers.

3.2.2.4.2.4. Sources of Construction Materials

3.2.2.4.2.4.1. Sheet Pile. Generally, the sheet pile sections specified during advertisement were used for construction. However, sheet pile section substitution conforming to the minimum required section modulus was allowed. Below, is a table of sheet pile sections for New Orleans to Venice.

New Orleans to Venice	
West Bank	
St. Jude to City Price	
Diamond Pump Station Tie-In	**
Reach A (City Price to Empire)	
TN Gas Pipeline	PZ-22
Hayes Pump Station Tie-In	unknown cold-rolled sheet pile
Gainard Woods Pump Station Tie-In, Upstream	Frodingham*
Gainard Woods Pump Station Tie-In, Downstream	PZ-22*
Homeplace Marina	unknown cold-rolled sheet pile
Reach B1 (Empire (Tropical Bend) to Ft. Jackson)	
Empire Floodgate Tie-In	PZ-32
Sunrise Pump Station Tie-In	PZ-27*
Bayou Grand Liard	PZ_27*
Reach B2 (Ft. Jackson to Venice)	
Duvic Pump Station Tie-In	unknown cold-rolled sheet pile
East Bank	
Reach C (Phoenix to Bohemia)	
Point a la Hache Pump Station Tie-In	**
Bellview Pump Station Tie-In	**

* As advertised – Not confirmed as built
** Information not located at the time of publication.

3.2.2.4.2.4.2. Levee material

3.2.2.4.2.4.2.1. Sources of Borrow (Reach A City Price to Tropical Bend). Sand fill will be pumped from the river and stockpiled in the batture to be hauled to its final location. The clay will be hauled from local borrow pits. There are three primary sources of borrow for constructing the levees. Two sites are located within the protected area of Reach A and a third just north of City Price.

3.2.2.4.2.4.2.2. Sources of Construction Materials (Reach B1, Tropical Bend to Fort Jackson). Since the levees will be constructed primarily of hydraulic fill with sand and shell core, building materials should present no problems. Hydraulic fill can be pumped from areas immediately adjacent to the proposed alignment; sand can be secured from the Mississippi River nearby; and shell, aggregate, and riprap can be barged and hauled in as required. Suitable materials for topping out the levees can be obtained from the existing earthfill levee.

3.2.2.4.2.4.2.3. Sources of Borrow (Reach B2 – Fort Jackson to Venice, Louisiana). Reach B2 will consist of a sand core hydraulic clay fill levee. A sand core trench will be excavated. Material excavated from the sand core trench will be spoiled in spoil and pending area No. 1, and in the temporary area diked off in spoil and pending area No. 3. Sand will then be pumped from the Mississippi River borrow areas, into the sand core trench and retaining dike base area. Sand will be pumped to elevations that will provide sufficient material for shapeup of the sand core and retaining dike base. A flood side hydraulic clay fill retaining dike will then be constructed from adjacent borrow. Hydraulic clay fill from the clay borrow areas, which will be stripped of the upper 10 feet of poor quality cover material, will then be pumped between the existing back levee and the flood side retaining dike over the shaped sand core fill. When the hydraulic clay fill has sufficiently dried, approximately 2 years after placement, undisturbed borings and shear tests will be made to more accurately design the final levee sections. Where a second lift is not required, the hydraulic clay fill will be shaped to the net section plus some overbuild to compensate for settlement. After the major settlement is essentially complete, approximately 1 year after the first shaping, the levee will be reshaped and the back levee degraded and used as topping material to overbuild the net levee section to allow for any additional settlement. A second hydraulic clay fill lift will be provided where it is anticipated that sufficient material will not be available for the first shaping. Shapeups following the second lift will be essentially the same as those previously described. It is estimated that ultimately, due to settlement, a clay cover of at least 10 feet will be provided on the flood side slope of the levee, including the wave berm.

3.2.2.4.2.4.2.4. Sources of Borrow Materials (Reach C, Phoenix to Bohemia). The levee will be enlarged primarily with hauled fill obtained from the Pointe a la Hache Relief Outlet. In addition, excess material above the levee design section will be used for construction of the final levee section.

3.2.2.4.2.4.2.5. Source of Borrow (West Bank MRL, City Price to Venice, Louisiana). The hurricane protection levee will consist of semi-compacted clay fill in the levee embankment and uncompacted fill in the berms. The borrow material will be obtained almost entirely from the east bank batture directly across from the construction area; this area will be under water when the borrow material is removed. The material will be barged across the river to the construction site. A small portion of material will come from degrading the existing MR&T levee in setback areas.

3.2.2.4.3. As-Built Criteria – Construction documents 3.2.2.4.3.1. Changes between design and construction (i.e., cross sections, alignment, sheet pile tip el, levee crest el.)

3.2.2.4.3.1.1. DACW29-99-C-0052. Narrative Completion Report, New Orleans to Venice, LA, Reach A, Vicinity of Port Sulphur Hurricane Protection Levee, B/L 238+00.4 to 298+00, Final Levee Enlargement & Freeport Canal Closure 2nd Lift, Plaquemines Parish, LA.

Reviewed As Builts, No Major Modifications or Changes Found.

3.2.2.4.3.1.2. DACW29-96-C-0030. New Orleans to Venice, Louisiana, Reach A and B1, Hurricane Protection Levee, (Homeplace to Empire Floodgate) B/L Station 0+00 to B/L Station 467+00, Levee Enlargement, Plaquemines Parish, Louisiana

Reviewed Narrative Completion Report, no applicable modifications or changes found.

3.2.2.4.3.1.3. DACW29-98-C-0039. New Orleans to Venice, Louisiana, Reach A, Hurricane Protection Levee, Hayes Pumping Station to Port Sulphur, Second Enlargement, Plaquemines Parish, LA

Reviewed Narrative Completion Report and Mod Log Report. No applicable modifications or changes. Contractor did provide his own borrow pit for uncompacted fill material.

3.2.2.4.3.1.4. DACW29-01-C-0025. Hurricane Protection Project, New Orleans to Venice, LA, Reach B1, Foreshore Dike at Empire, Plaquemines Parish, Louisiana

Reviewed Mod Log Report and Modification Documents. The area of the West Harbor Dike between WH C/L Stations 0+00 and 5+13.95 was realigned so that it could be constructed using hauling equipment rather than floating plant.

3.2.2.4.3.2. Inspection during original construction, QA/QC, state what records are available. See paragraph 3.2.1.5.4.2 New Orleans East Bank for description of how records are kept.

3.2.1.2.2.4.3.2.1. DACW29-96-C-0030 – NO TO VENICE, REACH A & B1, PLAQ PAR

Attached are moisture analysis reports, percent complete, borrow pit elevations, and daily trucking reports.

3.2.2.2.4.3.2.2. DACW29-98-C-0039 – NO – VEN, RCH A, HAYES PS – PT SULPH, 112 – 239, PLAQ PAR

Attached are records of preparatory inspections/meetings.

3.2.2.2.4.3.2.3. DACW29-99-C-0052 – NO TO VEN, RCH 1, 238 – 298, 2ND LIFT, PLAQ PAR

Records of preparatory inspections/meetings were found.

3.2.2.4.4. Inspection and maintenance of original construction.

3.2.2.4.4.1. Annual Compliance inspection (i.e. trees, etc.). Annual Compliance Inspections were conducted for the New Orleans to Venice Project in conjunction with the Grand Prairie Levee District, the Plaquemines Parish West Bank Levee District, and the Buras Levee District.

These inspections, which were general in nature, primarily defined the status of existing project work, and a general condition rating.

For the last 6 years, 1998 through 2004, the ratings for the Orleans Levee District, which includes the New Orleans East polder, were “OUTSTANDING” through year 2001, and “ACCEPTABLE” each year thereafter, at which time there was a change in the Project Rating Scale. The project rating scale was then redefined, and “ACCEPTABLE” became the highest rating.

There was no specific mention of deficiencies for the hurricane protection system.

3.2.2.4.4.2. Periodic Inspections Empire Floodgates (Ref 62) – The following information presents a summary of the inspection and corrective actions associated with those inspection deficiencies for the Empire Floodgate:

3.2.2.4.4.2.1 Historical Deficiencies Reported During and Related to Periodic Inspections.

Date	Description of Observations
28 August 1975	Geologist inspected in-place riprap and determined that it did not meet the original specification. Contracting Division had approved an alternate gradation on 15 March 1974. The riprap did not appear to meet the revised gradation.
4 September 1975	The following items were noted during Periodic Inspection No. 1: (1) separation at expansion joint between T-4L and T-3L (RM-3 & RM-2); (2) minor shrinkage and temperature cracks on tops of floodwalls and gatebay monolith; (3) lower parts of flap gate including the hinge brackets were rusting and needed touch-up painting; (4) several nuts on hinge anchor bolts were loose; (5) one anode missing on gate; (6) electrical gate control panels not yet operational; (7) west sheet pile wall settled up to 2 feet; (8) reference marks not installed on east side of structure; (9) reaction grillages slightly recessed and skewed horizontally with respect to bearing plate on gate; and (10) locking device retainer plates on each end of the flap gate plate girder were out of position.
5 November 1975	A problem was encountered when lowering the flap gate. The gate was lowered to just below the waterline where it stopped and the hoist chain became slack. After jogging the motors, the gate did go down approximately another foot.
24 November 1975	An underwater inspection revealed that the hoist chain shackle pin was interfering with the concrete chamber wall.
8 December 1975	The modification provided adequate clearance between the lifting chain and the chamber wall. However, the flap gate descended slightly lower than on

previous tests before stopping. Next, the counterweights were dogged off and the gate was nearly lowered to the fully open position. Contractor was instructed to remove some lead weights from the counterweights.

13 January 1977	Field inspection to check on mechanical and electrical features was conducted. Upon arrival, it was noticed that the eastern counterweight chain had failed. The counterweight was not dogged off, but suspended by its chain. When the chain failed, the counterweight fell to the bottom of the slot. The chain failed in the fourth link above the swivel. This link did not pass over the wildcat.
24 January 1977	The western counterweight chain had failed. The chain failed in the eighth link above the swivel. This link did not pass over the wildcat. Tests indicated failure was not caused by faulty material.
8 December 1977	Counterweight chain tests were conducted by WES. The forces acting on the gate hoist and counterweight chains were found to be affected by the wave height within the structure and the position of the gate when a boat traveled through the structure.
FY 1978	Recurring malfunction in the electrical panel that was generally associated with the limit switches.
4 October 1978	The following items were noted during Periodic Inspection No. 2: (1) Enlarged joint between RM-2 & RM-3, RM-4 & RM-5 and RM-18 & RM-19; (2) No noticeable change in minor shrinkage and temperature cracks on top of floodwalls was observed; (3) Wall armor on the gatebay had extensive corrosion; (4) A small crack with efflorescence was noted on the floodside vertical face of the gatebay monolith - west side; (5) Sand deposits in needle beams appeared to be causing corrosion; (6) Settlement of sheetpile I-wall had caused the waterstop to no longer have continuous contact with the T-wall; (7) Paint coating on underside of the gate was discolored; (8) Large amount of soil was trapped on the gate's walkway; (9) Riprap erosion protection was missing from channel slope south of the east wall - shell backfill exposed; and (10) The hoist and counterweight chains were rusted.
29 July 1981	The following items were noted during Periodic Inspection No. 3: (1) No noticeable change in minor shrinkage and temperature cracks on top of floodwalls; (2) Continued movement of joints between RM-2 & RM-3, RM-4 & RM-5 and RM-1 8 & RM-19; (3) Ladders and wall armor on gatebay had extensive corrosion; (4) Counterweight recesses contained entrapped water – indicating that drain holes at Elev. -9.5 feet were not functioning properly; (5) entrapped water in east counterweight recess was flowing through eight small holes on the north face of the concrete wall just above the pump platform; (6) Spalled area of concrete was noted underneath a

handrail anchor plate on the northeast corner of the west side channel wall; (7) Cracked area of concrete was noted underneath a handrail anchor plate on southeast corner of the west side channel wall; (8) Water was noted leaking through form bolt holes on both sides of the T-wall tie-in to the gatebay monolith on the west side of the structure; (9) Sand deposits in the needle beams were causing corrosion; (10) Decaying timber blocking was noted underneath the needle girder; (11) Settlement of the sheetpile I-wall had caused the waterstop to no longer make continuous contact with the T-wall; (12) Corroded sheetpile I-wall was noted; (13) A large amount of soil trapped on the gate's walkway; (14) Two dents on the skin plate at the top of the flap gate was noted near each of the two vertical center ribs; and (15) Missing/corroded handrail anchor bolts were noted.

3 May 1982 — Plaquemines Parish inspected the structure gate several days after it was noted that a boat had hit the gate. Soundings indicated the top of the gate was at Elev. 10.5 feet. Upon raising the gate, there was a dent in the middle. The damage was confined to the top beam and face plate.

31 January 1984 — The following items were noted during Periodic Inspection No. 4: (1) No noticeable change in minor shrinkage and temperature cracks on top of floodwalls since last inspection; (2) Joint readings showed no substantial change between RM-2 & RM-3 and RM-4 & RM-5; (3) Minor concrete popout on top surface of west side wall was noted; (4) missing handrail post anchors were noted; (5) Decaying timber blocking was noted underneath the needle girder; (6) No change was noted in the two dents on the skin plate at the top of the flap gate near each of the two vertical center ribs; (7) Breakwater dike on west bank of south approach channel had lost all riprap; (8) Staff gage number was illegible; and (9) Continued widening of joint between RM-18 & RM-19 (monoliths T-3R and T-4R) was noted.

29 January 1987 — The following items were noted during Periodic Inspection No. 5: (1) No substantial change was noted between RM-2 & RM-3 and RM-4 & RM-5; (2) Entrapped water in east counterweight recess was flowing through a hole on the north face of the concrete wall just above the pump platform; (3) Ladders and wall armor on the gatebay had extensive corrosion; (4) Large amount of soil trapped on the gate's walkway; (5) Moderate damage to the skin plate and channel beam across the top of the gate on the three central interior spans; (6) Section of handrail adjacent to the ladder recess on the west side of the structure was not anchored; (7) No pile caps on the majority of the timber piles; (8) Breakwater dike on the west bank of south approach channel had lost all riprap; (9) Riprap protection was deficient all around the structure; (10) Top of gate resting at approximate Elev. -10.5 feet in lieu of Elev. -14.0 feet and this was due to the accumulation of silt in the gate recess; (11) Cathodic anodes needed to be replaced; and (12) Excessive settlement of the eastern sheetpile I-wall had occurred.

August 1989	At the request of Plaquemines Parish, the structure was inspected by NOD mechanical & electrical engineers. The following were found: (1) A motor casing for ream drive had fractured and loose parts jammed the motor, and (2) electrical problems existed with the transmitter for the synchronizer for the lift motors.
30 January 1990	The following items were noted during Periodic Inspection No. 6: (1) A ½-inch gap was noted between L-type waterstop and T-wall; (2) Entrapped water in east counterweight recess was flowing through a 2-inch hole on the north face of the concrete wall – recently drilled to drain ponded water; (3) Ladders, corner plates and wall armor on gatebay had extensive corrosion; (4) Sacrificial anodes were partially eaten away; (5) The previously reported area of moderate damage to the skin plate and channel beam across the top of the gate on the three central interior spans had sustained further damage since last inspection; (6) South handrail on eastern half of structure had been hit – caused some spalling of concrete adjacent to anchor bolts, but still securely attached; (7) Handrailing on the north side of the access ladder was loose and missing anchor bolts; (8) No pile caps on the majority of the timber piles; (9) Sand deposits in needle beams were causing corrosion; (10) Rotted timber blocking was noted underneath the needle girder; (11) Bottom stair tread to boat dock had rusted completely through; (12) Spall was noted at the expansion joint between T-wall monoliths T-1R and T-2R; (13) Joint filler material was missing from most of the expansion joints; (14) Continued rotation settlement of T-4R away from T-3R was occurring – stretching the waterstop; (15) Breakwater dike on west bank of south approach channel had lost all riprap; (16) Riprap protection was deficient around the entire structure; (17) channel bank lines were receding; (18) Staff gages needed cleaning – had barnacles and algae; and (19) Several chain links in splash zone were severely corroded.
28 January 1993	The following items were noted during Periodic Inspection No. 7: (1) Spalls were noted at handrail anchor plates; (2) Minor popouts were observed on top of T-walls; (3) Protective coating was peeling off east sheet piling, especially just above the ground surface; (4) The breakwater dike was underwater; (5) Lack of riprap on levee nose was noted; (6) 1/2-inch gap between L-type waterstop and west T-wall; (7) 1-1/2- to 3-inch gap between L-type waterstop and east T-wall; (8) Split in the waterstop was noted at the joint between monoliths T-4L and T-3L; (9) Rodent holes along west I-wall, protected side were noted; (10) Channel bank lines were continuing to recede; (11) Riprap protection was slightly deficient all around the structure even though additional riprap had been placed in January 1991; (12) Damage to the channel beam on the flap gate was noted; (13) No change was noted in damage to the south handrail on eastern half of the structure – spalling of concrete adjacent to anchor bolts, but still securely attached; (14) Pump platform ladder had rusted strings and rungs at the bottom; and (15)

Handrailing on the north side of the access ladder on west side of structure was loose and missing anchor bolts.

17 January 1996 & 13 February 1996	The following items were noted during Periodic Inspection No. 8: (1) Gap of 1 inch between "L" type waterstop and T-wall monolith T-4L; (2) Torn waterstop was noted between T-wall monoliths T-4L and T-3L & T-4R and T-3R; (3) Spalls on the gatebay noted from the last inspection had not changed; (4) Corner plates and wall anchor on gatebay had extensive corrosion; (5) Deterioration of paint noted on all exterior miscellaneous metals; (6) Handrails on the boat dock had deteriorated to a point that they no longer provided a safety function; (7) Bottom three treads and attachment plate on boat dock stairway were severely corroded; (8) Pump platform access ladder was badly corroded in the splash zone; (9) Siltation of gate recess continued to be a problem; (10) The top of the flap gate was damaged; (11) Exterior surface of gate operating machinery had deteriorated badly - corrosion noted in many areas; (12) Chain links in splash zone were severely corroded and encrusted with oyster shells/barnacle growth; (13) Steel counterweight cages were badly corroded; (14) Ratchet type load binders for counterweights were not in place; (15) Observed flow distribution in discharge manifold for gate recess flushing system is poor; (16) Pump engine muffler needed replacing; (17) Hydraulic system for safety latching of gate and gate shock absorption were not operable; (18) Emergency generator - old & loose fan belts, old cooling hoses and oil leaks; (19) Diffuser lens needed on two east side pole lights; (20) Exposed wires from removed light poles (one on east side and three on west side); (21) Wire hanging out of junction boxes for gate latching device on east side; (22) Nondestructive testing of needle girders had not been performed; (23) Three timbers on north guidewalls needed replacement in near future; (23) Rotted timber blocking was noted underneath the needle girders; (24) Deck board missing from the boat dock; (25) Breakwater dike on west bank of south approach channel had lost riprap and was well below the design grade; (26) Channel bank lines were continuing to recede due to wave wash; (27) West sheet pile I-wall was up to 3.4 feet below design grade; and (28) Coal tar epoxy protective coating had completely deteriorated off the sheet piling near the ground line.
17-18 Feb 1997	Performed electrical load tests on gate hoist motors.
23 March 1999 & 2 September 1999	The following items were noted during Periodic Inspection No. 9: (1) Spall on the protected side of T-wall at the joint between monoliths T-2R and 3R; (2) Torn waterstop between T-wall monoliths T-4L and T-3L & T-4R and T-3R noted from last inspection had not been repaired; (3) Complete deterioration of joint filler material was observed; (4) Spalls at handrail anchor plates - noted at previous inspections; (5) Entrapped water in east counterweight recess was flowing through numerous small holes on the north face of the concrete wall just above the chambered construction

joint - 1st noted during Periodic Inspection No. 3; (6) Four of the eight piles supporting the pump platform had extensive vertical cracks that extend below the water surface sketches were provided; (7) Crack in support beam of pump platform, southwest corner – sketches were provided; (8) Corner plates and wall armor on gatebay had extensive corrosion; (9) Top of flap gate was damaged; (10) Inoperative gate shock absorbers were noted; (11) HSS evaluation of gate and dewatering components was needed; (12) Wildcat sheaves had appreciable wear on load side; (13) Chains showing some corrosion and barnacle collection at and below the splash zone; (14) Limit switches and synchronizing system were no longer maintained - manual operation of gate was preferred; (15) Lack of redundancy was noted in the event the gate hoist system failed; (16) Pump vanes and diffuser bowl had never been inspected; (17) Hand operated pump lubricating unit for pump bearings was inoperable; (18) Water leak below air vent valves - rupture pipes; (19) Badly corroded attachment ring, nuts and bolts for 24-inch check valve and expansion coupling; (20) Pump engine muffler needs replacing – badly corroded and leaking exhaust through the bottom; (21) Two small leaks noted in generator cooling system; (22) Broken refractors were noted for the two lights on the east side; (23) Several deteriorated timbers on the guidewalls needed replacement in the near future; (24) Breakwater dike on west bank of south approach channel had lost riprap and was well below the design grade; (25) Channel bank lines were continuing to recede due to wave wash; (26) A gap was noted between the east sheet pile I-wall and earthen embankment; and (27) Reference marks on the top of the I-walls were obliterated when the coal tar epoxy was applied.

17 August 2000	Plaquemines Parish made a visual inspection of pump: (1) Needed to replace suction & piping between gate and pump; (2) Needed to replace universal joints between engine and pump angle drive; (3) Needed to replace/rebuild greasing system for pump and pump angle drive; (4) Needed to check components in gear box – replace seals as necessary; (5) Needed to replace/repair relief valves; (6) Needed to replace expansion joint coupling in discharge line; and (7) Needed to replace tie bolts for expansion joint.
14 October 2001	Plaquemines Parish noted that water was passing through sides of gate. Stated that water stops should be changed at next dewatering.
8 September 2003	Plaquemines Parish found a cracked link in the east side hoist chain.

3.2.2.4.4.2.2 Historical Repairs/ Construction Work.

Date	Description of Observations
20 October 1975	The following was completed: (1) Lower portion of the gate was sandblasted and painted, (2) All nuts on the hinge anchor bolts were tightened, (3) Exposed form work anchor bolts were burned off, (4) Reinstalled anode

	that had broken loose and (5) Concrete was chipped out adjacent to grillage bearing face to allow contact between gate bearing plate and grillage bearing face.
Nov – Dec 1975	The detachable link on the hoist chain was attached to the pickup clevis on the gate to provide adequate clearance away from the chamber wall.
2 & 3 May 1977	The broken counter weight chains were removed and new chains were installed by NOD hired labor at a cost of $1,735 (excluding chain cost). The gate was operated to test all components of the system. While lowering the gate, the east chain slipped off the drive wildcat and caused the east end of the gate to drop abruptly (about a foot) before the chain caught again in the wildcat.
24 May 1977	The drive wildcats were ground down by NOD hired labor. A permanent solution was considered to consist of replacing both weldment-type wildcats with steel casting type wildcats.
13-31 March 1978	Two steel casting type replacement wildcats were installed. This work involved the cutting of the pillow blocks to remove the old wildcats. NOD hired labor performed the work for a cost of $26,811 (excluding wildcat cost).
Prior to Aug 1978	The idler drum diameter was increased from 8 to 14 inches and the gate locking device retainer plates were repositioned.
18 May 1979	Replaced two solar panels and repaired damage resulting from theft of panels at a cost of $1,009.
21 May 1979	Repaired and replaced damage done by vandals: broken glass, ripped screens, broken shutters and stolen ratchet jack.
May - June 1982	Plaquemines Parish Commission Council had soil deposits in the gate recesses cleaned out by contract dredging and labor forces. Also replaced lights and shutters.
4 August 1982	Made repairs to damaged portion of the gates for a cost of $15,037.
September 1982	Sandblasted and repainted embedded steel on gatebay and pump pipe.
January 1983	Maintenance work performed on the handrails and miscellaneous cleaning and painting.

3.2.2.4.5. Other Features - New Orleans to Venice - Plaquemines

3.2.2.4.5.1. Brief Description. The primary components of the hurricane protection system for the New Orleans to Venice reach basins are described above, namely the levees and flood-walls designed and constructed by the Corps of Engineers. However, other drainage and flood control features that work in concert with the Corps of Engineers levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section will briefly describe and present the criteria and pre-Katrina conditions of the interior drainage system, pump stations, non-Corps back levees, and the Mississippi River Flood Protection System. This section covers all the basins in Plaquemines Parish from New Orleans to Venice.

3.2.2.4.5.2. Pre-Katrina Conditions. According to the local jurisdictions responsible for interior drainage, the storm drain systems, roadside ditches, interior canals, and outfall pump stations were in good condition and prepared for high inflows from rainfall prior to August 29, 2005.

The Corps and non-Corps back levees were in good condition prior to Katrina landfall.

The Mississippi River Flood Protection System was in good condition prior to Katrina landfall.

3.2.2.4.5.3. Interior Drainage System

Overview. The New Orleans to Venice reach contains 11 separate basins totaling about 60 square miles. The land generally slopes away from the Mississippi River to marshland. It is sparsely developed except for areas along the Mississippi River and there is considerable agricultural and petrochemical activity. Many features in the Belle Chase basin are typical of urban cities in the United States, and some features that are unique because much of the area is below sea level.. Surface runoff from yards and streets flows into roadside ditches or into inlets and storm sewers. Excess runoff flows down streets and/or overland to lower areas. The other basins have more rural type drainage features with many areas below sea level. Stormwater pump stations, located along the back levees, pump the water directly into outfall canals or sloughs in the marsh. No stormwater is pumped into the Mississippi River. The entity responsible for local drainage in the New Orleans to Venice reach is Plaquemines Parish.

System Components. Stormwater flow is influenced by the land topography, roadways, ditches, canals, and pump station layout. Figure 5 in Volume VI shows the topographic layout of the northern end of the reach. With the relatively narrow basins and agricultural influence, the interior drainage systems consist mainly of roadside ditches and canals. The ditches and canals not only collect stormwater from the land and roadways and covey it to the pump stations, they also are storage areas that work in conjunction with the pump stations. Because material for the levees came from the interior canals, there is considerable more storm water storage in the Plaquemines Parish canals than in the other urban areas. Based on land topography and the drainage system, the 11 basins were divided into 37 subbasins. The outfall pump stations are located along the back levees. Pump station information is presented in Section 3.2.2.4.5.4. of this volume.

Design Criteria. The interior drainage systems in the older and rural areas have a capacity of about a 50% probability(2 year frequency) event, while new drainage projects and developments are required to accommodate a 10% probability (10 year frequency) event. Where canal or pump capacity is not available downstream, larger developments are required to put in stormwater detention facilities. The goal for new or upgraded pump stations is to pump one inch per hour for the first two hours and one half inch per hour after that. The current functional capacity of the canals and pump stations is 0.25 inches/hour. The level of protection is similar to the other New Orleans area basins that have a higher pumping capacity because of the additional storage in the Plaquemine Parish open canals.

There are no Southeast Louisiana (SELA) Urban Flood Control Projects in these basins.

3.2.2.4.5.4. Pumping stations - Plaquemines Parish Summary

Figure 29 is a map showing the Plaquemines Parish pump stations that were used in this report. The locations of the pump stations were verified by Global Positioning System (GPS) and/or by using Google Earth Pro. The GPS coordinates were then input into Microsoft Streets and Trips (shown below).

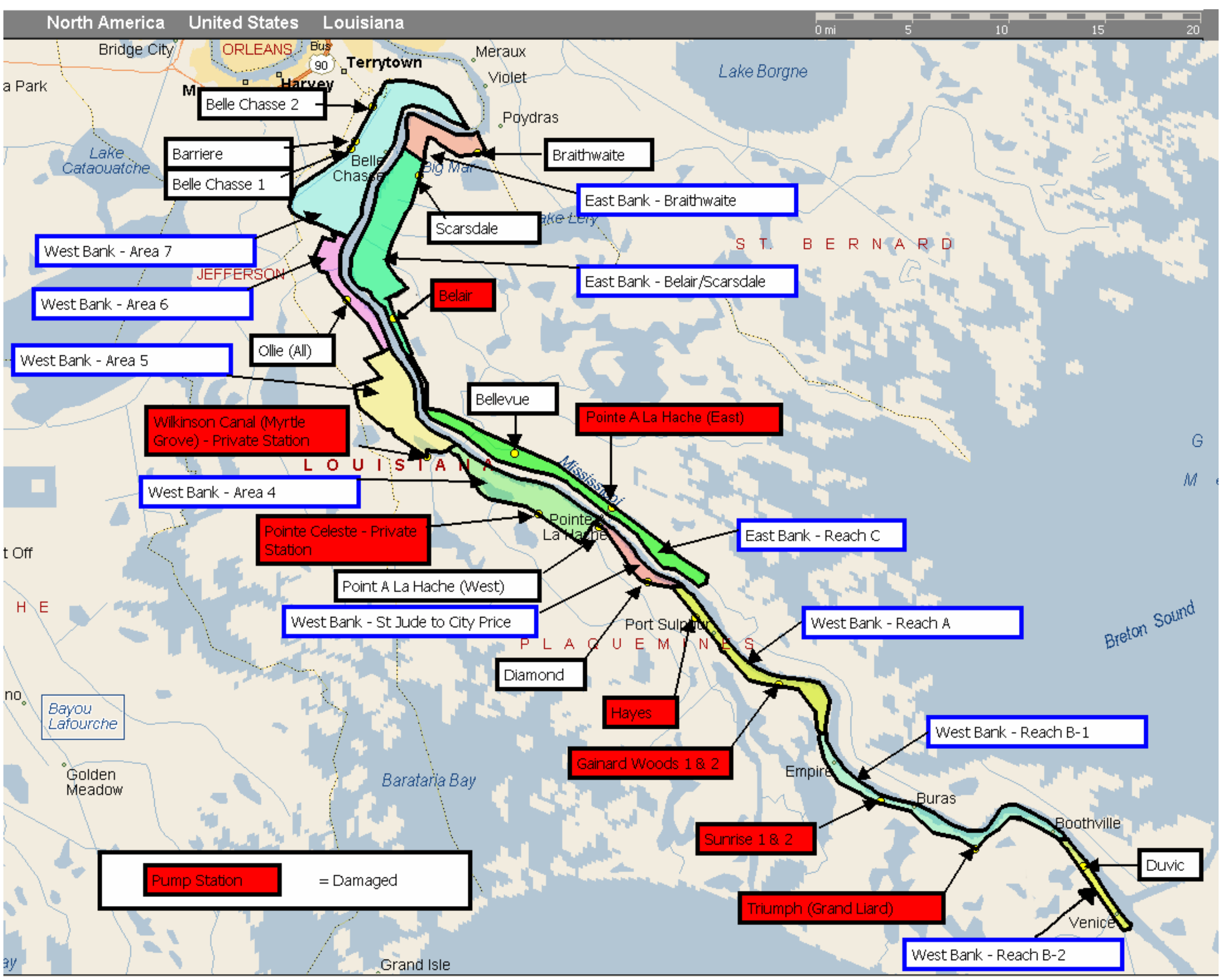


Figure 29. Plaquemines Parish Pump Station Locations

Drainage Basin. Plaquemines Parish consists of ten separate drainage basins. These basins have one or two pump stations, with the exception being the East Bank – Braithwaite, which has three pump stations. Plaquemines parish borders the Mississippi River. The pump stations generally discharge into marshes, although there are exceptions. The pump stations predominantly use diesel driven vertical pumps. Details for each pump station are listed in Volume VI.

East Bank – Braithwaite

Braithwaite

Intake location: ....Braithwaite Pond
Discharge location: .... Marsh
Nominal capacity: ....105 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	40	1974	Diesel	Vertical
2	65	1974	Diesel	Vertical

East Bank – Belair/Scarsdale

Belair

Intake location: ....Pointe A La Hache Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....130 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	130	1950	Diesel	Vertical

Scarsdale

Intake location: ....Scarsdale Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....1,784 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	446	1965	Diesel	Horizontal
2	446	1965	Diesel	Horizontal
3	446	1965	Diesel	Horizontal
4	446	1965	Diesel	Horizontal

East Bank – Reach C

Bellevue

Intake location: .... Pointe A La Hache Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....516 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	258	1972	Diesel	Horizontal
2	258	1972	Diesel	Horizontal

East Point a la Hache

Intake location: .... Pointe A La Hache Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....580 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	290	1972	Diesel	Horizontal
2	290	1972	Diesel	Horizontal

West Bank – Area 7

Belle Chasse 1

Intake location: .... Barriere Canal
Discharge location: .... Intercostal Waterway
Nominal capacity: ....3,556 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	800	1955	Diesel	Horizontal
2	800	1955	Diesel	Horizontal
3	150	1955	Diesel	Vertical
4	903	1963	Diesel	Horizontal
5	903	1963	Diesel	Horizontal

Belle Chasse 2
Intake location: ....Belle Chasse Drainage Canal
Discharge location: .... Intercostal Waterway
Nominal capacity: ....1,050 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	350	n/a	Diesel	Vertical
2	350	n/a	Diesel	Vertical
3	350	n/a	Diesel	Vertical

Barriere Road
Intake location: .... Barriere Pond
Discharge location: .... Intercostal Waterway
Nominal capacity: ....25 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	25	n/a	Diesel	Vertical

West Bank – Area 6

Ollie Lower
Intake location: .... Ollie Canal
Discharge location: ....Ollie Outfall Canal
Nominal capacity: ....440 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	140	n/a	Diesel	Vertical
2	150	1981	Diesel	Vertical
3	150	1981	Diesel	Vertical

Ollie Upper
Intake location: .... Ollie Canal
Discharge location: .... Ollie Outfall Canal
Nominal capacity: .... 140 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	n/a	1964	Diesel	Vertical
2	140	1964	Diesel	Vertical

West Bank – St. Jude to City Price

West Pointe a la Hache
Intake location: .... West Pointe A La Hache Canal
Discharge location: .... Jefferson Lake Canal
Nominal capacity: .... 45 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	15	n/a	Diesel	Vertical
2	15	n/a	Diesel	Vertical
3	15	n/a	Electric	Vertical

Diamond
Intake location: .... Diamond Drainage Canal
Discharge location: .... Marsh
Nominal capacity: .... 256 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	128	1976	Diesel	Vertical
2	128	1976	Diesel	Vertical

West Bank – Reach A

Hayes
Intake location: .... Hayes Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....500 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	250	1963	Diesel	Horizontal
2	250	1963	Diesel	Horizontal

Gainard Woods 1
Intake location: ....Gainard Woods Canal
Discharge location: .... Marsh
Nominal capacity: ....570 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	205	1960	Diesel	Vertical
2	205	1960	Diesel	Vertical

Gainard Woods 2
Intake location: ....Gainard Woods Canal
Discharge location: .... Marsh
Nominal capacity: ....570 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	285	1985	Diesel	Vertical
2	285	1985	Diesel	Vertical

West Bank – Reach B1

Sunrise 1

Intake location: .... Sunrise Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....180 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	90	1960	Diesel	Vertical
2	90	1960	Diesel	Vertical

Sunrise 2

Intake location: .... Sunrise Drainage Canal
Discharge location: .... Marsh
Nominal capacity: ....290 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	145	1979	Diesel	Vertical
2	145	1979	Diesel	Vertical

Grand Liard/Triumph

Intake location: .... Bural Drainage Canal
Discharge location: ....Grand Liard Marsh
Nominal capacity: 840 cfs

West Bank – Reach B2

Duvic

Intake location: .... Venice Drainage Canal
Discharge location: .... Bayou Duvic
Nominal capacity: ....560 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	280	1976	Diesel	Vertical
2	280	1976	Diesel	Vertical

West Bank – Area 5

Wilkinson Canal (Myrtle Grove)
Intake location: ....Unnamed Canal
Discharge location: .... Marsh
Nominal capacity: ....980 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
101	223	n/a	Diesel	Vertical
102	223	n/a	Diesel	Vertical
103	267	n/a	Diesel	Vertical
104	267	n/a	Diesel	Vertical

West Bank – Area 4

Pointe Celeste (Upper and Lower)
Intake location: ....Unnamed Canal
Discharge location: .... Marsh
Nominal capacity: ....895 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
105	223	n/a	Diesel	Vertical
106	223	n/a	Diesel	Vertical
107	223	n/a	Diesel	Vertical
108	223	n/a	Diesel	Vertical

3.2.2.4.5.5. Levees and floodwalls

3.2.2.4.5.6. MRL West Bank Mississippi River Levee City Price to Venice, Louisiana. Approximately 34 miles of levee (Reference Nos. 44, 71).

3.2.2.4.5.6.1. Geology. The study area is located within the Central Gulf Coastal Plain: Specifically, the area is located on the modern subdelta which projects gulfward from the deltaic plain of the Mississippi River. It is a region of extremely low relief. Dominant physiographic features are the natural levees of the Mississippi River and abandoned distributaries, and the marshlands and inland bodies of water that lie between the natural levee ridges. Elevations range from a maximum of about five (5) feet along the crests of the natural levees to a minimum of sea level or slightly lower in the marshlands between the natural levee ridges. The numerous inland bodies of water vary in depth from one to six feet. The Mississippi River channel varies in depth from 65 feet to 190 feet below sea level.

3.2.2.4.5.6.2. Foundation Conditions. The foundation conditions for these 34 miles of levee are very complex because of the geologic history of the area. The subsurface on the West Bank consists of Recent deposits ranging in thickness from about 150 feet at mile 44 to 260 feet at mile 10AHP, the downstream end of the project. The Recent deposits are underlain by Pleistocene age materials. Generally, the Recent deposits consist of a surface layer of soft to stiff natural levee clays with layers and lenses of silt varying in thickness from a minimum of 3 feet to 5 feet to a maximum of 20 feet to 22 feet. The natural levee deposits are underlain by a discontinuous layer of very soft marsh clays with peat and organic matter. The marsh deposits vary in thickness from 2 feet to 7 to 9 feet. The natural levee and marsh deposits are underlain by soft, alternating intradelta clays and silts with layers of silty sand and sand. The thickness of the intradelta deposits varies from 25 feet to 97 feet. Some of the natural levee and marsh deposits are underlain by very soft to soft interdistributary clays with lenses and layers of silt and silty sands. The interdistributary deposits vary in thickness from 20 feet to 78 feet. The remaining reaches of natural levee and marsh deposits are underlain by point bar silts, silt sands and sand with layers of clay.

3.2.2.4.5.6.3.. Field Exploration

Considerable and related data had been obtained in the past from the West Bank of the Mississippi River between miles 10 and 66 to determine the characteristics of the subsurface materials for use in levee and revetment designs. These data are presented in Reference No. 44. The report presents soil data for the West Bank including detailed boring logs, shear test data, consolidation test data, and soil stratification and shear strengths. Of the 247 borings included in Reference No. 44, ten general type and 42 undisturbed borings were considered in the design of the present study area.

a. Additionally, more recent borings considered in the present design are:

(1) Borings 1-UBU, 2-UBU, 3-UBU, 4-UBU, 5-UBU, 6-UBU, 7-UBU, and 8-UBU taken for the Upper Buras Levee Enlargement

(2) Borings R-25.1-U, R-25.1-UT, R-25.5-U, and R-25.5-UT taken for Buras Revetment.

Boring depths were dependent upon the project for which the borings were taken. Borings ranged from ground surface to as shallow as elevation -15 and as deep as Elev. -219, NGVD. Undisturbed borings were taken with a 5-inch diameter steel-tube, piston-type sampler and general types with a 1-7/8-inch I.D. core barrel sampler.

3.2.2.4.5.6.4. Levee Improvements. The most prevalent typical levee section is a landside enlargement of the existing MR&T levee; however, there are riverside enlargements, straddle enlargements, and landside enlargements with underwater rock beans and setbacks. The project basically will consist of raising the existing Mississippi River levee (MR&T) hurricane project height with an appropriate wave berm. The levee embankment will consist of semicompacted clay fill, while the berms will consist of uncompacted fill with riprap armor on the riverside face of the wave berm.

3.2.2.4.5.6.5. Settlement. Consolidation data from borings within a reach were combined and average parameters were determined. Analyses for the enlarged levee sections indicate that the gross grade levee crown will settle from 0.2 to 2.5 feet, depending on the location within the project area. This settlement includes a 10 percent allowance for shrinkage and lateral spread. Due to right-of-way and cost constraints, the enlarged levees will be constructed in multiple lifts in all reaches except Reach T, where a gross levee enlargement is possible. Multiple lift construction will involve enlarging the levee to the desired project net grade for intermediate lifts and grossing the levee in the final lift as required to achieve the desired project net grade. Four years was used as the time between lifts.

Based upon local experience and calculations performed using consolidation data taken from the undisturbed borings, settlement of the setback levees in the first four years due to consolidation, shrinkage, and lateral spread was estimated to be about 4 feet. Again, because of right-of-way and cost constraints, the setback levees will be constructed in multiple lifts.

3.2.2.4.5.6.6. Slope Stability Protected side stability berms were designed, proposed levee and wave berth stability checked, and flood side stability control lines were developed for the following conditions: Water level to the stillwater level (SWL) on the flood side for failure toward the protected side; and low water on the flood side for failure toward the flood side. Minimum slopes were determined by the LMVD Method of Planes analysis, using design shear strengths and a minimum factor of safety of 1.3.

3.2.2.4.5.6.7. I-Walls. The only structure in the project is the existing Empire lock at Empire, Louisiana. It was determined that no modification was necessary to the lock gates. Wave overtopping would be allowed; however, the 30 feet of existing I-wall on both sides of the structure was found to be inadequate to withstand the projected hurricane wave force if capped to project height. The length of the existing sheet piling is inadequate. For structural and constructability reasons, the existing sheet piling shall be removed and replaced with adequate lengths of new PZ-27 sheet piling. All measurable settlement will be allowed to take place prior to I-wall capping. This period is estimated to be 3 years after completion of second enlargement embankment work in the area; however, the area will be monitored regularly to confirm this time period.

3.2.2.4.5.6.8. T-Walls. No T-walls on this project.

3.2.2.4.5.6.9. Erosion Protection. The berms will consist of uncompacted fill with riprap armor on the riverside face of the wave berm.

3.2.2.4.5.6.10. Non-Corps. Several local interest and/or private levees are located within the project area. No design criteria for these levees have been made available to the Corps.

3.2.3. West Bank & Vicinity

3.2.3.1. General Description

The project is located on the west bank of the Mississippi River in the vicinity of New Orleans and in Jefferson, Orleans and Plaquemines parishes.

The project will provide Standard Project Hurricane Protection to residents from storm surges from Lakes Cataouatche and Salvador, and waterways leading to the Gulf of Mexico.

The original project included 22 miles of earthen levee and 2 miles of floodwalls extending from the Harvey Canal down to the V-levee near the Jean Lafitte National Historical Park and back up to the town of Westwego. The Lake Cataouatche area eliminated the west-side closure in Westwego, and added about 10 miles of levee and 2 miles of floodwalls to the project. The East of Harvey Canal area includes a sector floodgate in the Harvey Canal just below Lapalco Boulevard and about 25 miles of levee and 5 miles of floodwalls, including enlargement of the Federal levees along the Algiers Canal.

Figure 30. Hurricane protection project, West Bank and Vicinity, New Orleans, LA

3.2.3.2. History

The modern West Bank and Vicinity, New Orleans, Louisiana, Hurricane Protection project emanated as an outgrowth of the original New Orleans to Venice Hurricane Protection project. In 1965 and 1966, four resolutions—two adopted by the Senate Committee on Public Works and two by the House Committee on Public Works—authorized reviews of the project posited in the 1962 Chief of Engineers report on the Mississippi River Delta at and below New Orleans, (which had been authorized in the 1962 Flood Control Act and later renamed the New Orleans to Venice Hurricane Protection project) to determine feasibility of modifying the project to provide improved hurricane protection and flood control to portions of Jefferson Parish lying between the Mississippi River and Bayou Barataria and Lake Salvador.

In December 1986, the Corps of Engineers completed a feasibility report that examined potential hurricane surge protection measures for the west bank of the Mississippi River in Jefferson Parish between Westwego and the Harvey Canal down to the vicinity of Crown Point. The recommended plan included 22 miles of earthen levees and two miles of floodwalls extending from the canal to the V-levee near Jean Lafitte National Historical Park back north to the town of Westwego. The 1986 Water Resources and Development Act (Public Law 99-662) authorized the Westwego to Harvey Canal Hurricane Protection project and construction was initiated in 1991.

The Westwego to Harvey Canal Hurricane Protection underwent a post-authorization change in the mid 1990s. In February 1992, the Corps of Engineers completed a reconnaissance study regarding hurricane protection for that portion of the west bank of the Mississippi River in Jefferson Parish between Bayou Segnette and the St. Charles Parish line, particularly at Lake Cataouatche where the existing non-federal levee had been deemed structurally unstable. The reconnaissance study recommended a plan based on combination of levees and steel sheet pile floodwalls generally along the existing Lake Cataouatche levee alignment to provide protection from hurricane tidal surges up to a 100-year recurrence interval. This plan also eliminated the need for the west-side closure in Westwego authorized under the Westwego to Harvey Canal Hurricane Protection project. The 1996 Water Resources and Development Act (Public Law 99-662) authorized the Lake Cataouatche area project.

That same act also authorized the East of Harvey Canal Hurricane Protection project. In 1994, the Corps of Engineers completed a feasibility report that examined additional hurricane surge protection to approximately 35,000 acres in portions of Jefferson, Orleans, and Plaquemine parishes bounded by the Harvey Canal to the west, the Mississippi River to the north and east, the Hero Canal to the south, and divided by the Algiers Canal. For the area west of the Algiers Lock, the feasibility report recommended the construction of a navigable floodgate in the Harvey Canal just south of Lapalco Boulevard; a combination of levees and floodwalls on the east side of the canal from the floodgate to the Hero Pumping Station; raised protection from the Hero Pumping Station along both banks of the Algiers Canal to the Algiers Lock; and an outfall canal for the Cousins Pumping Station. For the area to the east of the Algiers Lock, the study recommended raising the existing protection along the Algiers and Hero canals; and the construction of a new levee near Oakville to connect to the Hero Canal levee and to the existing Plaquemines Parish levee. By tying into the line of protection authorized under the Westwego to Harvey

Canal project, the recommended plan presented a continuous line of protection for west bank residents from Westwego to Oakville.

The 1999 Water Resources and Development Act (Public Law 106-53) combined the Westwego to Harvey Canal project, the Lake Cataouatche project, and the East of Harvey Canal project into West Bank and Vicinity, New Orleans, Louisiana, Hurricane Protection project.

3.2.3.3. Datum - Subsidence and Vertical Datum Problems in New Orleans, LA

Because of technological gains, the U.S. Army Corps of Engineers is able to more accurately track subsidence of projects – something that could not be done as reliably in the past. Based on a recent study, we can now estimate that the New Orleans area is subsiding at a rate of 6-17mm/yr or 2-5½ feet per century. In the city itself it's about 3 feet per century and as much as 10 feet per century in Venice, if recent trends continue.

The Interagency Performance Evaluation Task Force (IPET), an independent group activated by the Corps of Engineers to study the response of the hurricane protection system during Hurricane Katrina, identified problems with using the previous vertical datum to which survey benchmarks were referenced. IPET's ability to accelerate analysis of this issue, which was ongoing by the Corps' New Orleans District and the National Oceanic and Atmospheric Administration (NOAA)'s National Geodetic Survey (NGS), led to the identification of two major problems with elevations in the New Orleans area: subsidence and the use of the old vertical datum elevations as equal to local mean sea level, a common misunderstanding in the engineering community up until the 1990's.

Benchmarks serve as the reference or starting elevation when measuring levee heights, relationships to the water surface (local mean sea level), structure and levee elevations, etc. It has been known since 1985 that the elevations of benchmarks in and around New Orleans were inaccurate, due to subsidence, and needed to be updated. The exact amount of subsidence was not known until a 2004 survey conducted by the NGS in cooperation with the Louisiana Spatial Reference Center, the Corps of Engineers and state and local governments was performed on some 86 benchmarks in southern Louisiana.

The 2004 survey pointed out inaccuracies due not only to subsidence, but also to distortions and errors in elevations of benchmarks that were assumed to be stable in the past, but had in fact subsided themselves. Based on the 2004 survey, the Corps of Engineers has revised the elevations of survey benchmarks used to establish heights of structures, such as levees and floodwalls, in Southern Louisiana. Use of the new 2004 survey assures consistency for all elevation surveys performed in the southern Louisiana area.

The IPET has developed a new relationship between the current local mean sea level and the 2004 survey, which is referred to as the North American Vertical Datum of 1988 (2004.65 Adjustment). Local mean sea level in the city itself is about ½ foot above the 2004 datum. The Corps will use the 2004 elevations and their varied relationship to the local mean sea level throughout the area to precisely determine the elevations of levees and other critical flood protective structures. This datum will also be used by the construction industry and others in southern Louisiana for a wide variety of projects that rely on elevations relative to the local water surface.

More information can be found in the “Geodetic and Water Level Datum” report.

3.2.3.4. Design Hurricane

Because of the urban nature of the project area, the selected design hurricane is the standard project hurricane.

3.2.3.4.1. Standard Project Hurricane. The standard project hurricane (SPH) is one that may be expected from the most severe combination of meteorological conditions that are considered reasonably characteristic of the region. Guidance on the selection of site-specific storm meteorological parameters was initially given in National Hurricane Research Project Report No. 33 (U.S. Weather Bureau, Nov 1959). The Weather Bureau and USACE jointly derived the specifications, criteria, procedures, and methods. The specifications for SPH were reviewed several times after 1959, and the Weather Bureau issued updates. An additional update was published by NOAA in 1979 (Sep 1979). This update formed the basis for the SPH meteorological parameters used for this project.

As discussed in the section on the Lake Pontchartrain and Vicinity Project, a SPH storm was considered to have a recurrence interval of once in 100 years anywhere within Zone B. The probability of the SPH storm striking a smaller subzone, such as the Lake Cataouatche area, would be less. The frequency of the SPH at the site of a protective structure was assumed to be dependent upon its exposure and the direction of approach of the storm. Using observed high water mark and stage data, combined with computed wind tide elevations using different central pressure indices, a surge frequency curve was constructed that was representative of a reach of the hurricane protection system. The frequency curve also considered statistics on the critical direction of approach. The frequency of the computed wind tide elevations was adjusted based on the percentage of each direction followed by historic hurricanes. The probabilities of equal stages for both groups of tracks were then added arithmetically to develop a curve representing a synthetic probability of recurrence of maximum wind tide levels for hurricanes from all directions.

3.2.3.4.2. Probable Maximum Hurricane. The probable maximum hurricane was not used in the analysis.

3.2.3.5. Lake Cataouatche (Reference 25), (Reference 26)

3.2.3.5.1. Introduction. This area consists of approximately 10 miles of levee and 2 miles of floodwalls as shown in Figure 31 below.

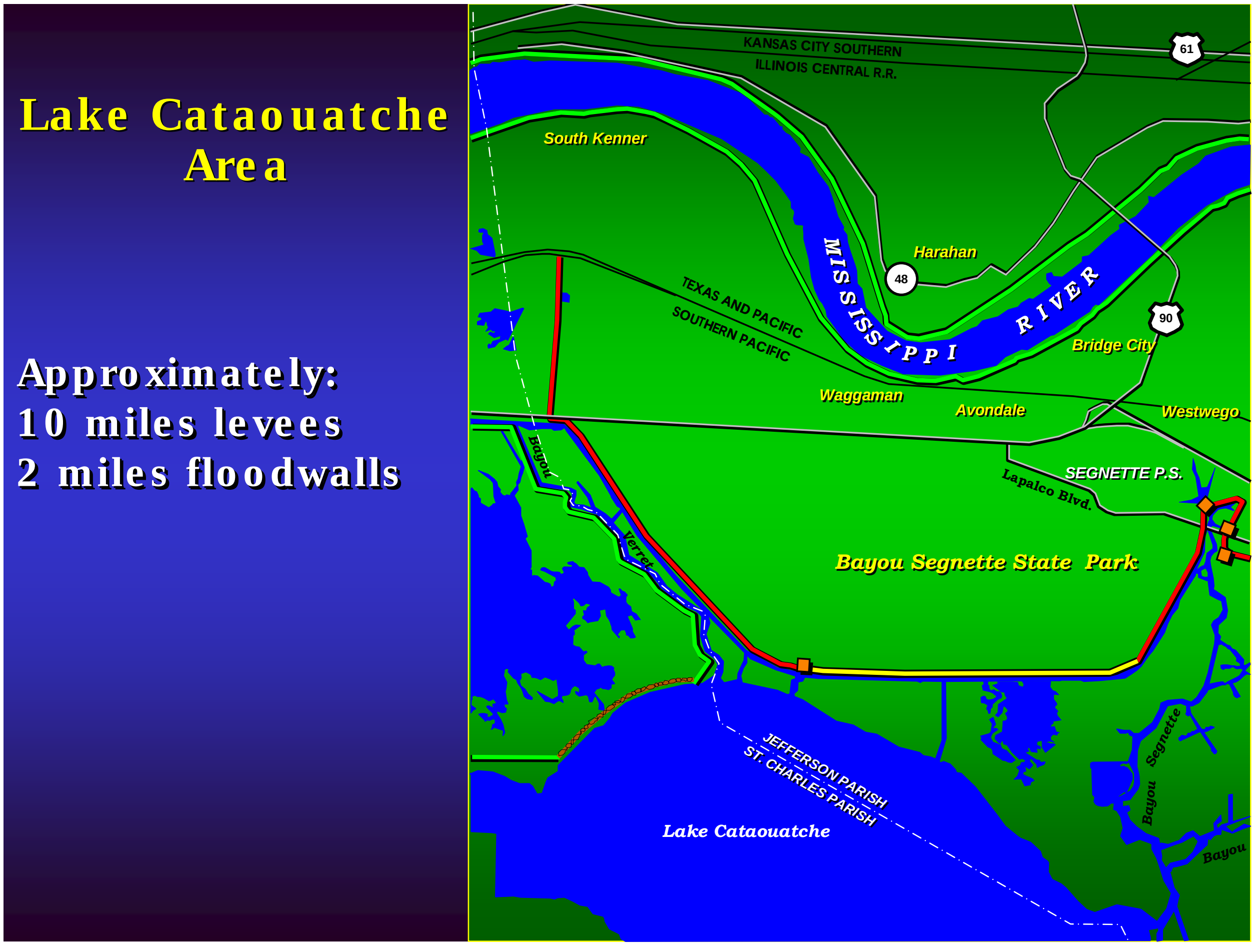


Figure 31. Lake Cataouatche project features

3.2.3.5.2. Pre-Katrina. Construction in this area started in 2000. Before Hurricane Katrina, only one construction contract was completed in this area. Another one was under construction, but the contractor was terminated for default in 2005. Currently, the Corps of Engineers is waiting for the surety to determine their plans regarding the contract takeover. Remaining work in this area consists of 1st enlargement levee or floodwall contracts and future 2nd enlargement levee contracts.

3.2.3.5.3. Design Criteria and Assumptions - Functional design criteria.

3.2.3.5.3.1. Hydrology and Hydraulics. For Lake Cataouatche area, the design hurricane characteristics are shown in Table 36; the design tracks are shown on Figure 32. The maximum wind speed was computed using the same equations as for Orleans East Bank. For each project area, the track and forward speed were selected to produce maximum wind tide levels.

Table 36 Design Hurricane Characteristics						
Location	Track	CPI, Inches	Radius of Maximum Winds, Nautical miles	Forward Speed, Knots	Maximum Wind Speed,[1] MPH	Direction of Approach
Lake Cataouatche	C	27.4	30	11	100	South
[1] Windspeeds represent a 5 minute average 30 feet above ground level.						

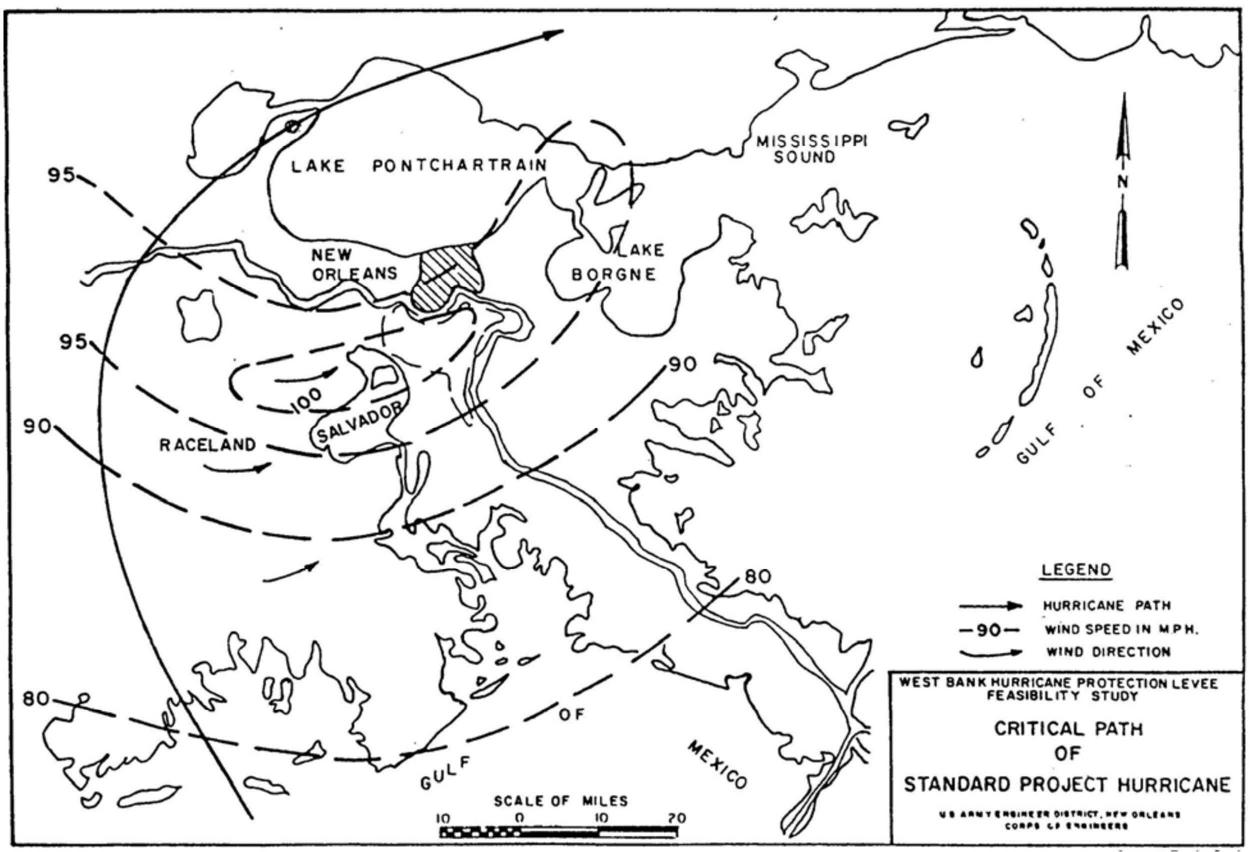


Figure 32. Critical path of standard project hurricane

3.2.3.5.3.1.1. Surge. Wind tide levels for Lake Cataouatche area were computed using the same methodology as used for Lake Pontchartrain lakefront for Orleans East Bank, plus several additional steps.

Where the coastline is characterized by a coastal bay separated from the gulf by an offshore barrier island, such as Grand Isle, or a shoal, an additional step was added to verify hurricane tides. Water surface elevation was transposed from the gulf side to the bay side of the island.

Table 37 Verification of Hurricane Surge Heights							
Location	Surge adjustment factor, Z	Sep 1915		Sep 1947		Sep 1956	
		Observed, ft MSL	Computed, ft MSL	Observed, ft MSL	Computed, ft MSL	Observed, ft MSL	Computed, ft MSL
Grand Isle – front side	0.80	9.0	8.8	3.9	4.1	-	-
Grand Isle – back side	0.80	-	-	8.0	7.8	-	-
Manila Village	0.48	8.0	8.5	-	5.1	-	-

Major hurricane damage would result from induced effects on Lake Salvador and Lake Cataouatche. As with Lake Pontchartrain, the wind tide level for Lake Salvador and Lake Cataouatche is the sum of the surge, setup, tide, and runoff from rainfall. Hurricane surge values were routed from the bay side of the coast to Lake Salvador using the same methodology as that performed for routing surge from Lake Borgne into Lake Pontchartrain. Hourly incremental setup values were computed at Manila Village from general wind tide equation. Stage-area curves were developed for a schematized conveyance channel between Manila Village and Lake Salvador basin, which includes Lake Salvador, Lake Cataouatche, and adjacent marsh. Rainfall calculated by methods described in Weather Bureau reports (15, 16). A moderate rainfall of 8.5 inches over 24 hours was used for the Westwego to Harvey analysis. The surge from Betsy routed by the procedure as a verification of the procedure. Using this procedure resulted in a stage of 3.05 ft, which compared favorably with 3.35 ft, for the location Bayou Barataria at Lafitte. This location was assumed to be representative stage of Lake Salvador Basin.

The average windspeed and average depth were determined from isovel and hydrographic charts furnished by NWS. The same setup and setdown equations for Lake Pontchartrain were used.

The methodology used for Chalmette Extension to route the wind tide level from the surge reference line to the location of the protection system was also used for the Lake Cataouatche portion of the West Bank and Vicinity project. A weighted mean decrease in surge heights inland at a rate of 1 foot per 2.75 miles was used. Table 38 shows the wind tide level at the surge reference line and at the Harvey Canal.

Table 38 Wind Tide Levels		
Location	Wind Tide level, surge reference line, FT NGVD	Wind Tide level at levee location, FT NGVD
Harvey Canal	9.0	7.5

Some additional modeling was also performed; four analytical models were used. HURWIN, Tropical Storm Planetary Boundary Wind Model, was used to generate wind field input into ADCIRC. This model was run to obtain wind fields for Track C, the actual track of the 1915 hurricane. The model was then calibrated to actual results of the 1915 hurricane.

ADCIRC, Advanced Circulation Model, was used to obtain tidal and storm surge condition at the boundary of the WIFM model. PBLWIND, Planetary Boundary Layer Model, was used to interpolate wind fields from ADCIRC to WIFM. WIFM, WES Implicit Flooding Model, was used to compute tidal circulation and storm surge propagation. WIFM was used because ADCIRC did not have wetting and drying capability. WIFM was calibrated to Hurricane Betsy.

The WIFM model was adjusted to account for future land loss due to subsidence and an estimated sea level rise of 0.2 ft per 50 years. Model results indicated an increase in wind tide level of 1.0 ft by the year 2040.

3.2.3.5.3.1.2. Waves. For the Lake Cataouatche, some levees and floodwalls would be sheltered from storm generated runup; small locally generated waves could occur. These small waves would be likely to occur along Bayou Segnette, Bayou Verret, and South Kenner Road. Wave runup for all levees and floodwalls was calculated using methodology described in 1984 Shore Protection Manual.

3.2.3.5.3.1.3. Summary. Table 39 contains maximum surge or wind tide level, wave, and design elevation information.

Table 39
Wave Runup and Design Elevations (Transition zones not tabulated – Governing Report is listed)

Location	Report	Average Depth of fetch, ft	Significant Wave Height Hs, ft	Wave Period, T, sec	Maximum Surge or Wind Tide Level, ft	Runup Height ft	Freeboard, ft	Design Elevation Protective Structure, ft
Lake Cataouatche	PAC Study, Dec 1996	NA[1]	NA	NA	7.5 NGVD	3.0	-	10.5 NGVD
Bayou Segnette	PAC Study, Dec 1996	NA	1.0	2.7	7.0 – 6.0 NGVD	3.0	-	10.0 – 9.0 NGVD
Bayou Verret	PAC Study, Dec 1996	NA	1.0	2.7	7.5 NGVD	2.0	-	9.5 NGVD
South Kenner Road	PAC Study, Dec 1996	NA	1.0	2.7	4.5 NGVD	2.0	-	6.5 NGVD

[1] Not presented in the report.

3.2.3.5.3.2. Geotechnical. The projects that make up the West Bank and vicinity levee are the Lake Cataouatche area, Westwego to Harvey Canal and the east of Harvey Canal.

3.2.3.5.3.2.1. Geology. The study area is located in Jefferson Parish, Louisiana between the Mississippi River and Lake Cataouatche. Surface elevations in the study area range from +10 feet NGVD along the natural levees of the Mississippi River to near 0 feet NGVD in the marshes bordering Lake Cataouatche. The surface and shallow subsurface in the study area is composed of natural levee, marsh, swamp, interdistributary, and prodelta deposits. Specifically, the borings show that the entire area is overlain by 8 to 22 feet of swamp deposits generally consisting of organic clays, wood, peat, with occasional sand and silt layers. Underlying swamp deposits are interdistributary deposits located between elevations +2 and -22 feet NGVD and are up to 40 feet thick. Interdistributary deposits generally consist of very soft, fat clay with occasional lenses of lean clay, silt, and silty sand. Prodelta deposits underlie interdistributary deposits between Stations 149+50 and 455+00. Prodelta deposits are found between elevations -30 and -55 feet NGVD and extend to an unknown depth. Prodelta deposits generally consist of homogeneous, medium clays with occasional lenses of silt, and silty sand. Nearshore gulf deposits underlie interdistributary deposits between Stations 485+00 and 570+00. Nearshore gulf deposits are found at approximately -50 feet NGVD and extend to an unknown depth. These deposits generally consist of silty sand and sand with shell fragments and occasional lenses of silt and clay.

3.2.3.5.3.2.2. Foundation Conditions. The foundation soils are predominantly fat clays (CH) varying in consistency from very soft to medium, with occasional layers of silt (ML) and lean clays (CL). Layers of organic clays, which typically display high moisture contents, exist in many areas from the original ground surface down to approximate elevation -20.

3.2.3.5.3.2.3. Field Exploration. Twelve general type borings were made along parts of the proposed alignment between Mar 91 and Apr 93. Four undisturbed type borings were made along parts of the proposed alignment during Apr 93.

3.2.3.5.3.2.4. Underseepage. Not used.

3.2.3.5.3.2.5. Pile Curves. Pile capacity curves were completed for a 12-inch square concrete pile and a Class B timber pile, respectively. A factor of safety of 3.0 is recommended if no pile tests are performed and with a pile test, a factor of safety of 2.0 is recommended. $K_C = 1.00$ and $K_T = 0.70$ were used to complete data for the curves.

3.2.3.5.3.2.6. Stability of Levees. Existing conditions along the proposed alignment were estimated and the slopes and berm distances for the proposed levee were designed for the (Q) construction case. A factor of safety (F.S.) of 1.3 is required for the levee stability.

3.2.3.5.3.2.7. Cantilever I-Wall. I-wall stability and required penetration were determined by the method of planes. A factor of safety was applied to the soil parameters. For the friction angle, the F.S. was applied as follows:

$$\Phi_d = \tan^{-1}\left(\frac{\tan \Phi_a}{\text{factor of safety}}\right)$$

where

Φ_a = available friction angle

Φ_d = developed friction angle

The developed friction angle was used in determining lateral earth pressure coefficients.

Using the resulting shear strengths, net horizontal water and earth pressure diagrams were determined for movement toward each side of the sheet pile. From the earth pressure diagrams, the summation of horizontal forces were equated to zero and the summation of overturning moments were determined for various tip penetrations. The depth of necessary penetration is the point of zero summation of moments.

Two I-wall designs were analyzed for this project. The first was for an existing sheet pile wall adjacent on either side of the Lake Cataouatche Pumping Stations 1 and 2 between approximate Stations 307+00 to 310+00. Since this reach is subject to wave loads, the sheet pile was analyzed for the following design cases.

Note: There is a significant wave load on the sheet pile wall:

Q-Case

F.S. = 1.5 with static water at still water level (SWL)

F.S. = 1.25 with static water at SWL plus waveload

F.S. = 1.0 with static water at SWL plus 2 feet

S-Case

F.S. = 1.2 with static water at SWL plus waveload

General: If the penetration to head ratio is less than 3:1, then increase it to 3:1.

The S-case was the governing design case for the pumping station sheet pile walls. For design results, see Plate F-6.

The other design reach runs from approximate Station 518+00 B/L to the Bayou Segnette floodwall. In this reach, the top of sheet pile will range from elevation 10.5 (near Station 518+00 B/L) to 9.5 (near Bayou Segnette pump station). The crown of the levee will range from elevation 5.5 (near Station 518+00 B/L) down to 5.0. (near Bayou Segnette pump station).

The following design cases were analyzed for determining required penetration for the levee/I-wall in this reach.

Note: There is no significant wave load on I-wall:

Q-Case

F.S. = 1.5 with static water at still water level (SWL)

F.S. = 1.0 with static water at SWL plus 2 feet

General: If the penetration to head ratio is less than 3:1, then increase it to 3:1.

The 3-to-1 penetration to head ratio was the governing design case for the proposed levee/I-wall. However, to compensate for future flood conditions (general land subsidence and sea level changes), additional sheetpile penetration has been incorporated into the design. See Plate F-7.

3.2.3.5.3.2.8. T-Type Walls. The T-type walls supported on bearing piles will provide protection adjacent to I-type gates and pumping plants

3.2.3.5.3.2.9. Erosion. No protection is considered necessary along the levee other than seeding the levees. Any erosion caused by hurricane floods will be restored under normal maintenance.

3.2.3.5.3.3. Structural – Lake Cataouatche, La. (Reference 39)

General. The structural features in the Lake Cataouatche area consist of approximately 10 miles of levees and 2 miles of floodwalls. The floodwall features include I-walls, I-wall/levee combinations; pile supported inverted T-walls, and swing gate closure structures at street crossings. The following is a summary of the pertinent structural criteria for these structures.

I-Type Floodwalls. In the design of the I-walls, the loading cases that were considered as follows:

- Case I. Q-Case, F.S. = 1.5 with water to SWL
- Case II. Q- Case, F.S = 1.0 with water to SWL = 2 ft.

Minimum penetration to head ratio equal to 3:1

T-Type Floodwalls. The T-wall consists of a reinforced concrete stem on a monolithic concrete base of varying width supported on prestressed concrete piles, except for the Fronting Protection T-wall at the Segnette Pumping Station which will be founded on steel H-piles. The T-walls were designed for the following loading conditions:

- Case I – Static water pressure with water to SWL, no wind, impervious sheet pile cutoff, no dynamic wave force
- Case II – Static water pressure with water to SWL, no wind, pervious sheet pile cutoff, no dynamic wave force
- Case III – Static water pressure with water two feet above SWL, no wind, impervious sheet pile cutoff, no dynamic wave force (75% forces used)
- Case IV – Static water pressure with water two feet above SWL, no wind, pervious sheet pile cutoff, no dynamic wave force (75% forces used)
- Case V – No water, no wind
- Case VI – No water, wind from protected side (75% forces used)
- Case VII – No water, wind from flood side (75% forces used)

Additionally, at the Segnette Pumping Station Front End Protection T-wall, the tension loads from the existing tie rods are included in the above load cases.

Swing Gates and Gate Monoliths. Gate monoliths with swing gates are to be constructed at street crossing in lieu of I-walls. The gate structures are to be designed for the following load conditions:

- Case I – Gate closed, static water pressure with water to SWL, no wind, impervious sheet pile cutoff, no dynamic wave force.
- Case II – Gate closed, static water pressure with water to SWL, no wind, pervious sheet pile cutoff, no dynamic wave force
- Case III – Gate closed, static water pressure with water two feet above SWL, no wind, impervious sheet pile cutoff, no dynamic wave force (75% forces used)
- Case IV – Gate closed, static water pressure with water two feet above SWL, no wind, pervious sheet pile cutoff, no dynamic wave force (75% forces used)
- Case V – Gate open, no water, no wind, truck on protected side edge of base slab
- Case VI – Gate open, no water, no wind, truck on flood side edge of base slab
- Case VII – Gate open, no water, wind from protected side, truck on flood side edge of base slab (75% forces used)
- Case VIII – Gate open, no water, wind from flood side, truck on protected side edge of base slab (75% forces used)

3.2.3.5.3.4. Sources of Construction Materials

3.2.3.5.3.4.1. Sheet Pile. Generally, the sheet pile sections specified during advertisement were used for construction. However, sheet pile section substitutions conforming to the minimum required section modulus was allowed, primarily in contracts constructed after 1990. Below, is a table of sheet pile sections.

Lake Cataouatche	
Bayou Segnette State Park Floodwall	**

** Information not found at the time of publication

3.2.3.5.3.4.2. Levee Material (Lake Cataouatche Area). Borrow for embankment construction will come from multiple sources: (1) will be trucked from David Pond Project, (2) from the excavation of the floodside canal, (3) a 13.6-acre borrow area adjacent to Bayou Segnette State Park and (4) from widening of the interior drainage canal.

3.2.3.5.4. As-built Conditions.

3.2.3.5.4.1. Changes between design and construction (i.e. cross sections, alignment, sheet pile tip el, levee crest el.) – West Bank & Vicinity – Modifications and Changes.

3.2.3.5.4.1.1. DACW29-00-C-0042. Westbank – Vicinity of New Orleans, Hurricane Protection Project, Louisiana, Lake Cataouatche, Segnette State Park Floodwall, Jefferson Parish, Louisiana

Modification was issued to allow the contractor to dress down the levee slopes and crown width in certain reaches in order to give the contractor a borrow source at the job site for required embankment/structural backfill work.

3.2.3.5.4.2. Inspection during original construction, QA/QC, state what records are available.

See paragraph 3.2.1.5.4.2. New Orleans East Bank, for description of how records are kept.

3.2.3.5.4.2.1. DACW29-00-C-0042 – WB, LC, SEGNETTE ST PARK F/WALL, JEF PAR

Attached are preparatory phase reports.

3.2.3.5.5. Inspection and maintenance of original construction – The West Bank and Vicinity Hurricane Protection Project is one of the more recent hurricane protection systems in the New Orleans District. So far, no structures have been brought under the Periodic Inspection Program, but annual compliance inspections have existed for some time for these locals works turned over for operation and maintenance under the West Jefferson Levee District.

3.2.3.5.5.1. Annual Compliance inspection (i.e. trees, etc.) – This district is responsible for maintaining 20.2 miles of levee on the west bank of the Mississippi River, and 40 miles of back levees, which are being upgraded under the West Bank Hurricane Protection Project, in Jefferson Parish. In 2004, it was stated that “Although the hurricane protection levees are not totally complete, we conducted an interim joint inspection of the system on 8 June 2004”. The levees and floodwalls are in excellent condition. An “ACCEPTABLE” rating is assigned.

3.2.3.5.5.2. Periodic inspections. There are no structures under the Periodic Inspection Program in the Cataouatche area, of the West Bank and Vicinity Hurricane protection project.

3.2.3.5.6. Other Features – Jefferson West Bank, Lake Cataouatche

3.2.3.5.6.1. Brief Description. The primary components of the hurricane protection system for the Jefferson West Bank, Lake Cataouatche subarea are described above, namely the levees and floodwalls designed and constructed by the Corps of Engineers. However, other drainage and flood control features that work in concert with the Corps of Engineers levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section will describe and present the criteria and pre-Katrina conditions of the interior drainage system, pump stations, and the Mississippi River Flood Protection System. There are currently no non-Corps levees or floodwalls in this polder. Even though the stormwater pump stations are part of the interior drainage system, they are a significant part of the system and warrant their own section.

3.2.3.5.6.2. Pre-Katrina Conditions. According to the local jurisdictions responsible for interior drainage, the storm drain system, interior canals, and outfall pump stations were in good condition and prepared for high inflows from rainfall prior to August 29, 2005.

The Mississippi River Flood Protection System was in good condition prior to Katrina landfall.

3.2.3.5.6.3. Interior Drainage System

Overview. The Jefferson West Bank, Lake Cataouatche subarea contains about 30 square miles and generally slopes north to south from the Mississippi River. It is sparsely developed except for areas along the Mississippi River that are highly developed. Many features are typical of large urban cities in the United States, and some features that are unique because much of the area is below sea level. Catch basins and inlets collect surface runoff from yards and streets into storm sewers and ditches. Excess runoff flows down streets and/or overland to lower areas. Open canals collect the stormwater and carry it to outfall pump stations that pump the water into the Cataouatche Canal, Lake Cataouatche, and Bayou Segnette. No stormwater is pumped into the Mississippi River.

The entity responsible for local drainage in the Jefferson West Bank polder is Jefferson Parish. The Louisiana Department of Transportation and Development highways are also a part of the local drainage system.

System Components. Local drainage begins with overland flow which follows the ground topography. Figure 5 in Volume VI shows the topographic layout of Jefferson West Bank. The land generally falls south from the Mississippi River.

The land topography and development sequence influenced the storm sewer, ditch, canal, and pump station layout. There are no interior pump (lift) stations. Based on land topography and the drainage system, the subarea is divided into 85 subbasins. Pump station information is presented in Section 3.2.3.5.6.4 of this volume.

The canals are open and most are grass-lined. The canals and ditches not only collect storm-water from streets and storm sewers and covey it to the pump stations, they also are storage areas that work in conjunction with the pump stations.

Design Criteria. The current design criterion for Jefferson West Bank is the 10% storm event for all storm drainage system components. Older parts of the stormwater collection system have approximately a 2-year frequency capacity. The functional capacity of the interior canals and pump stations is 0.4 inches per hour. Rainfall in excess of this amount goes into temporary storage in the streets, storm sewers, and canals. There are criteria for new developments to use stormwater detention to offsite downstream impacts.

Where local drainage is considered to need improvement, Jefferson Parish is working to improve the drainage. There are no Southeast Louisiana (SELA) Urban Flood Control Projects in this subpolder.

3.2.3.5.6.4. Pumping stations

Jefferson Parish Lake Cataouatche

Jefferson Parish is located west of the city of New Orleans and borders the west side of Orleans Parish. Figure 33 is a map of Jefferson Parish with the pump stations that were studied identified by red dots. Jefferson Parish is separated by the Mississippi River into East and West Banks. The East Bank pump stations are connected by a grid of canals. The canals running east and west serve to equalize flow between the major outfall canals, allowing rain water to flow in different directions depending on the rainfall patterns and available capacities at the pump stations. The West Bank is subdivided into sub-basins that, for smaller rainfall events,

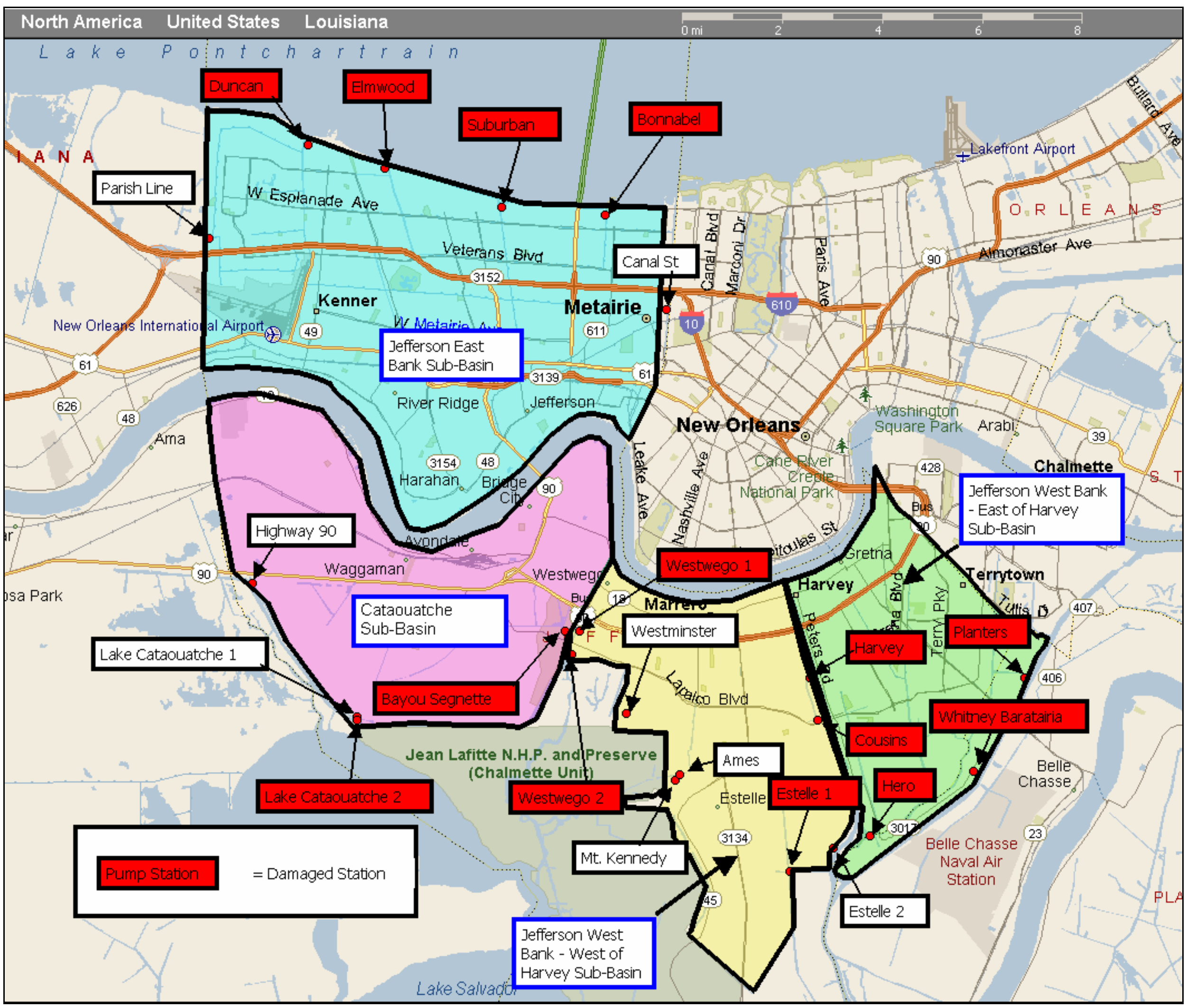


Figure 33. Jefferson Parish Pump Station Locations

operate independently. However, over-bank flow does occur between adjacent sub-basins for a 10-year event. This report examined 6 pump stations on the East Bank with a total of 36 pumps and 17 pump stations on the West Bank with a total of 65 pumps.

Figure 33 is a map showing the Jefferson Parish pump stations that were used in this report. The locations of the pump stations were verified by Global Positioning System (GPS) and/or by using Google Earth Pro. The GPS coordinates were then input into Microsoft Streets and Trips (shown below).

Table 40 contains a summary of pump stations by drainage basin in Jefferson Parish. The list is composed of information that was collected in the field. Not all information was available for each pump and was left blank or highlighted.

Table 40
Summary of Jefferson Parish Pump Stations by Drainage Basin

Basin	East Bank	Cataouatche	West Bank – West of Harvey	West Bank-East of Harvey	Total
Number of pump stations	6	4	9	3	22
Number of pumps	36	24	29	15	104
Total rated capacity (cfs)	20,662	3,346	10,695	9,958	44,661
Estimated cost of damages	$558,000	$3,000	$136,000	$61,000	$758,000

West Bank – West of Harvey (Cataouatche)

The West Bank-West of Harvey Cataouatche drainage basin has four significant pump stations, which are briefly described below. Volume VI provides more detailed information. The basin is bordered by the Mississippi River on the north and east sides. Its drainage system includes the river, Lake Cataouatche, and the Main, Waggaman, and Bayou Segnette Canals.

Lake Cataouatche No. 1
Intake location: ....Main Canal
Discharge location: ....Lake Cataouatche
Nominal capacity: ....500 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	250	n/a	Diesel	Vertical
2	250	n/a	Diesel	Vertical

Lake Cataouatche No. 2
Intake location: ....Main Canal
Discharge location: ....Lake Cataouatche
Nominal capacity: ....600 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	300	1982	Diesel	Vertical
2	300	1982	Diesel	Vertical

Highway 90
Intake location: ....Waggaman Canal
Discharge location: .... Outer Cataouatche Canal
Nominal capacity: ....>90 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	45	n/a	Electric	n/a
2	?	n/a	Electric	n/a
3	45	n/a	Electric	n/a

Bayou Segnette
Intake location: ....Main Canal
Discharge location: .... Bayou Segnette
Nominal capacity: ....2156 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
New 1	610	n/a	Diesel	n/a
New 2	610	n/a	Diesel	n/a
1	156	n/a	Diesel	Vertical
2	156	n/a	Diesel	Vertical
3	156	n/a	Diesel	Vertical
4	156	n/a	Diesel	Vertical
5	156	n/a	Diesel	Vertical
6	156	n/a	Diesel	Vertical

3.2.3.5.6.5. Levees and floodwalls

3.2.3.5.6.5.1. MRL - MRL levees and floodwalls are addressed in Paragraph 3.2.1.5.6.4.1, New Orleans East Bank MRL. There are no floodwalls that are part of the MRL Project in this reach.

3.2.3.5.6.5.2. Non Corps - Several local interest and/or private levees are located within the project area. No design criteria for these levees have been made available to the Corps.

3.2.3.6. Westwego to Harvey

3.2.3.6.1. Introduction. As shown in Figure 34, this area consists of approximately 22 miles of levee and 2 miles of floodwalls in the Westwego area along the existing V-levee alignment to the vicinity of the old Estelle Pumping Station and along the existing Harvey Canal-Bayou Barataria Levee tying into the floodwall at the Cousins Pumping Station, then from the pump station to the navigable sector floodgate complex which is to be constructed in Harvey Canal near the Cousins Pumping Station. This area was the first area of the project authorized, and as such, has the most construction completed.

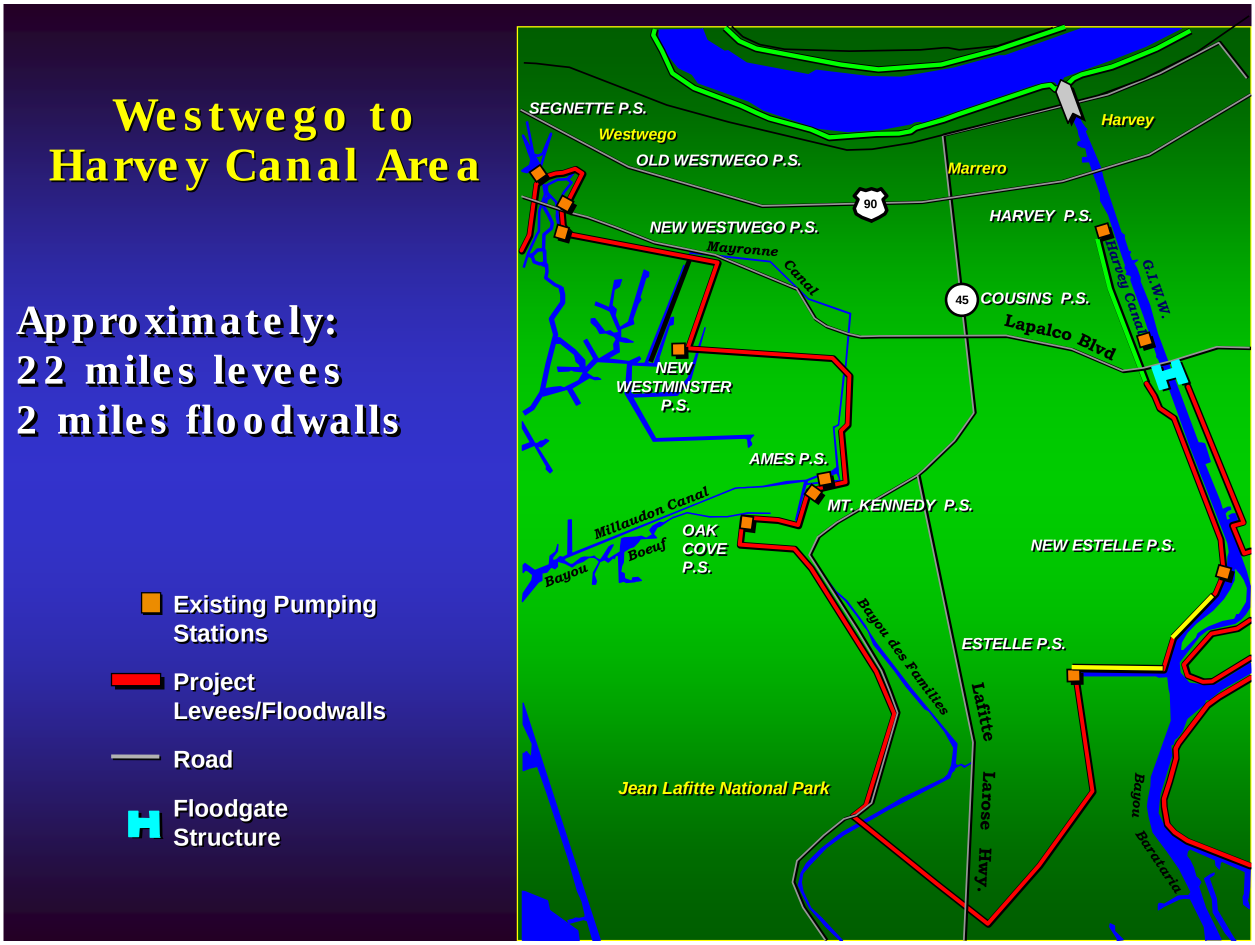


Figure 34. Westwego to Harvey Canal Area project features

3.2.3.6.2. Pre-Katrina. Construction in this area started in 1991. Before Hurricane Katrina, all of the 1st enlargement construction contracts had been completed, except for one, which was under construction. This contract is still ongoing, scheduled for completion this year. One 2nd enlargement contract also had been completed. There were a total of 15 construction contracts that were completed before Hurricane Katrina. Remaining work in this area consists of the future levee enlargements, as most of these contracts have 3rd enlargement levee contracts.

3.2.3.6.3. Design Criteria and Assumptions - Functional design criteria

3.2.3.6.3.1. Hydrology and Hydraulics - For Westwego to Harvey, the design hurricane characteristics are shown in Table 41; the design tracks are shown on Figure 35. The maximum wind speed was computed using the same equations as for Orleans East Bank. For each project area, the track and forward speed were selected to produce maximum wind tide levels.

Table 41
Design Hurricane Characteristics

Location	Track	CPI, Inches	Radius of Maximum Winds, Nautical miles	Forward Speed, Knots	Maximum Wind Speed,[1] MPH	Direction of Approach
Westwego to Harvey	-	27.6	30	6	100	South-Southwest

[1] Windspeeds represent a 5 minute average 30 feet above ground level.

3.2.3.6.3.1.1. Surge. Wind tide levels for Westwego to Harvey area were computed using the same methodology as used for Lake Pontchartrain lakefront for Lake Cataouatche, without the numerical modeling. In addition, a future condition analysis was not performed.

3.2.3.6.3.1.2. Waves. The levee reach from Bayou Segnette to Highway 3134 would be subject to waves generated in Lakes Salvador and Cataouatche. Wave runup was calculated using the methodology described in Orleans East Bank. The remaining reaches in the Westwego to Harvey reach were not considered to be subject to waves.

3.2.3.6.3.1.3. Summary. Table 42 contains maximum surge or wind tide level, wave, and design elevation information.

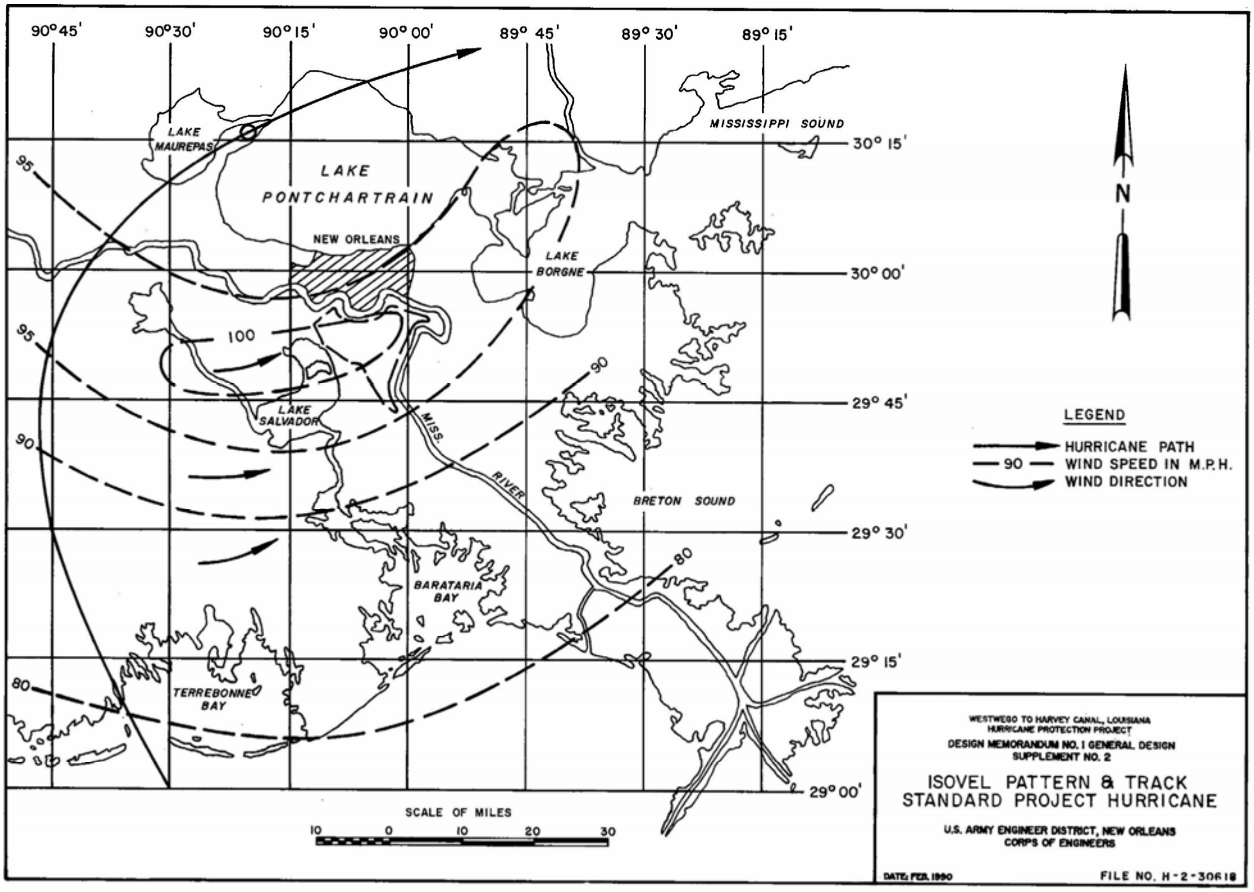


Figure 35. Isovel pattern and track standard project hurricane

Table 42
Wave Runup and Design Elevations (Transition zones not tabulated – Governing DM is listed)

Location	DM	Average Depth of fetch, ft	Significant Wave Height Hs, ft	Wave Period, T, sec	Maximum Surge or Wind Tide Level, ft	Runup Height ft	Freeboard, ft	Design Elevation Protective Structure, ft
Bayou Segnette to Dugues Canal	DM01 Sup02 Feb 1990	5.5	3.0	4.0	7.0 NGVD	3.0	-	10.0 NGVD
Dugues Canal to Estelle Canal	DM01 Sup02 Feb 1990	6.5	3.3	4.0	8.0 NGVD	3.0	-	11.0 NGVD
Estelle Canal to Bayou Des Familles	DM01 Sup02 Feb 1990	7.5	3.7	4.1	9.0 NGVD	3.0	-	12.0 NGVD
Bayou Des Familles to Highway 3134	DM01 Sup02 Feb 1990	2.5	2.1	4.1	9.0 NGVD	3.0	-	12.0 NGVD
Highway 3134 to Apex of V levee	DM01 Sup02 Feb 1990	-	-	-	9.0 NGVD	-	3.0	12.0 NGVD
Apex of V levee to Harvey Canal	DM01 Sup02 Feb 1990	-	-	-	7.5 NGVD	-	2.0	9.5 NGVD

3.2.3.6.3.2. Geotechnical

This report covers the soil investigation and design of approximately 82,000 feet of improved levees and 25,800 feet of floodwalls (**Reference: 33)**

The project alignment has been divided into five design reaches due to a variation in soil strength, stratification and required protection elevations. Many of these design reaches were divided into subreaches because of varying ground elevations, levee crown transitions and soil strength variation. Area and reaches are as follows:

Design Reaches		**Base Line Stations**		**Base Line Stations**
Westwego and Westminster Levee	5 subreaches	0+00	To	256+42
Oak Cove Levee	3 subreaches	256+45	To	420+96
Highway 45 Levee	1 subreach	420+97	To	572+16
V-line Levee	3 subreaches	572+17	To	804+32
Harvey Canal Levee	3 subreaches	804+33	To	1072+00

3.2.3.6.3.2.1. Geology. The project site is located on the Deltaic Plain Portion of the Mississippi River Alluvial Plain. Specifically, the project is located on the northern edge of the Barataria Basin on the western side of the Mississippi River. The Barataria Basin is an inter-distributary basin dominated by features which include natural levee ridges, crevasse-splay deposits, marsh, lake and swamps. The eastern and northern edge of the basin is defined by the natural levee ridge of the Bayou Lafourche natural levee ridge. The Gulf of Mexico constitutes the southern boundary. Elevations vary from approximately +10 to +15 feet NGVD in the back swamp and lake areas.

The foundation soils are predominantly fat clays (CH) varying in consistency from very soft to stiff. In many areas, organic clays (OH) and peat (PT) may be found in the top 20 feet and have a very soft consistency.

The V-Line Levee, from Station 563+00 to Station 588+00 is underlain by large layers of silt (ML) and sand (SM) 5 to 10 feet below the surface. Thin strata of silt and sand are encountered at various other locations in the foundation.

3.2.3.6.3.2.2. Field Exploration. One hundred and fifteen (115) borings were made along the proposed alignment. Of the 115 borings, 102 were obtained in 1976 by Eustis Engineering of Metairie, Louisiana, at the request and authorization of the West Jefferson Levee District, the local project sponsor. The rest of the borings, (13), were obtained in 1988 and 1989 by the Corps of Engineers, the majority of which were check borings. All Eustis borings were undisturbed (5-inch or 3-inch I.D.). Eight (8) C.E. borings were 5-inch undisturbed borings and five were general-type borings.

3.2.3.6.3.2.3. Underseepage. Not Used.

3.2.3.6.3.2.4. Hydrostatic Pressure Relief. Not used.

3.2.3.6.3.2.5. Pile Foundation. Pile capacity curves were generated for various structures along the alignment (Pumping stations, gates, T-walls, etc.).

Design single pile compression and tension capacities versus tip elevations for piles were determined for various locations.

Design data were determined for the (Q) and (S) shear strength. For piles in compression, a factor of safety of 2.0 was applied to the shear strengths, and a conjugate stress ratio K_C of 1.0 was used in the (SC) case for determining the normal pressure on the pile surface. In tension, a factor of safety of 2.0 was applied to the shear strengths, and a K_t of 0.7 was used in the (S) case. In some instances, the (Q) case indicates the least pile capacities, and in other instances, the (S) case yields the least result. The minimum value was used for design.

3.2.3.6.3.2.6. Slope Stability. Using cross-sections representative of existing conditions along the proposed alignment, the slopes and berm distances for the recommended levees were designed with borrow pies on the flood side except in the area of V-Line Levee from Station 660+00 to Station 800+00 where the borrow be will be on the protected side.

The stability of the levees was determined by the Method of Planes using the design (Q) shear strengths as shown on the plots of soil data on Plates F107 through F111. A factor of safety (F.S) of 1.3 was required for the levee stability and a F.S. of 1.5 was the minimum required for failures into borrow pits and canals.

3.2.3.6.3.2.7. I-Type Floodwalls. I-wall stability and required penetration were determined by the Method of Planes. A factor of safety was applied to the soil parameters. For the friction angle, the F.S. was applied as follows:

$$\Phi_d = \tan^{-1} \frac{\tan \Phi_a}{\text{factor of safety}}$$

where

Φ_a = available friction angle

Φ_d = developed friction angle

The developed friction angle was used in determining lateral earth pressure coefficients.

Using the resulting shear strengths, net horizontal water and earth pressure diagrams were determined for movement toward each side of the sheet pile. From the earth pressure diagrams, the summation of horizontal forces were equated to zero and the summation of overturning moments were determined for various tip penetrations. The depth of necessary penetration is the point of zero summation of moments.

Dynamic wave force was a design factor in the floodwalls design, from New Westwego Pumping Station (Station 69+94} to the V-Vertex of the V-Line Levee (Station 660+06). The results of hydraulic analysis indicate that these walls will be subjected to the pressure and forces implanted by a “broken wave”.

The following design cases were analyzed for determining required penetration:

Case I: No significant waveload on I-call:

Q-Case

F.S. = 1.5 with static water at still water level (SWL)

F.S. = 1.0 with static water at SWL plus 2 feet

General: If the penetration of head ratio is less than 3: 1, increase it to 3:1.

Case II: Significant wave load on I-wall:

Q-Case - Same as above plus

S.S. = 1.25 with static water to SWL plus wave load

S-Case

F.5. = 1.2 with water to SWL plus wave load

General: If the penetration to head ratio is less than 3:1, increase it to 3:1 or to that required by the S-case, F.S. = 1. 5, whichever results in the least penetration.

3.2.3.6.3.2.8. T-Type Floodwalls. T-type floodwalls will be used in some areas. The T-type floodwall will support on a pile foundation of pre-stressed concrete or steel H–piles.

3.2.3.6.3.3. Structural

3.2.3.6.3.3.1. Westwego to Harvey Canal Area (References 32 and 33)

General. I-type and T-type floodwalls will be used to provide protection in congested areas, and to provide a transition between the pumping station fronting protection and the full earthen levee sections. The fronting protection at each pumping station location will be raised to design elevation by either replacement of the existing protection with pile supported T-walls, or by increasing the height of the existing fronting T-walls or sheet pile bulkheads. Gate monoliths will be constructed for street crossings in lieu of I-walls

Structural Steel. The design of steel structures is in accordance with the requirements of the allowable working stresses recommended in “Working Stresses for Structural in EM 1110-1-2101 dated 1 November 1963 and amendment No. 2 dated 17 January 1972. The basic working stress for ASTM A-36 steel is 18,000 psi. Steel for steel sheet piling will meet the requirements of ASTM 328, “Standard Specifications for Steel Sheet Piling”

Reinforced Concrete. The design of reinforced concrete structures is in accordance with the requirements of the strength design method of the current ACI building Code, as modified by ;he guidelines of "Strength Design Criteria for Reinforced Concrete Hydraulic Structures", ETL 1110-2-312 dated 10 March 1988. The basic minimum 28-day compressive strength concrete will be 3,000 psi, except for prestressed concrete piling where the minimum will be 5,000 psi. For convenient reference, pertinent stresses are tabulated below:

f 'c	3,000 psi
fy (Grade 60 Steel)	48,000 psi
Maximum Flexural Reinforcement	0.25 x Balance Ratio
Minimum Flexural Reinforcement	200/ f y
f'c (For Prestressed Concrete Piles)	5,000 psi
fu (Prestressing Strands, Gr. 250)	250,000 psi
(Prestressing Strands, Gr. 270)	270,000 psi

I-Type Floodwall. The I-walls consist of steel sheet piling driven into the existing ground and, in some cases, into a new embankment. In the design of the I- walls, the loading case to be considered will be as follows:

- Q-Case, F.S. =1.5 with water to SWL
- Q-Case, F.S.=1.25 with water to SWL plus waveload
- Q-Case, F.S. = 1.0 with SWL plus 2 ft. freeboard
- S-Case, F.S. = 1.2 with water to SWL plus waveload

No water, lateral soil pressure (where applicable)

Tied-Back Sheet Pile Walls. The tied-back sheet pile walls will consist of steel sheet piling driven in t o existing ground and anchored with tie rods t o a steel pipe, pile, or H- pile dead man. The upper portion o f the sheet piling will be capped with concrete. The required sheet pile penetration and maximum bending moment will be determined by applying a factor- of- safety of 1.2 t o the soil parameters. The required anchor force will be determined by applying a factor-of- safety of 1.0 t o the soil parameters.

T-Type Floodwall. The T-wall will consist of a reinforced concrete stem on a monolithic concrete base of varying width supported on precast, prestressed concrete piles or H-Piles. The base of the T-wall will be constructed on a four-inch concrete stabilization slab. A continuous steel sheet pile wall will be provided beneath the base for seepage cutoff purposes. These walls will be designed for the following load conditions:

- Case I – Static water pressure with water t o SWL, no wind, impervious sheet pile cutoff, no dynamic wave force.
- Case II – Static water pressure with water to SWL, no wind, pervious sheet pile cutoff, no dynamic wave force.

- Case III – Stillwater pressure with water' 2 feet above SWL, dynamic wave force, impervious sheet pile cutoff (75% forces used).
- Case IV – Stillwater pressure with water 2 feet above SWL, dynamic wave force, pervious sheet pile cutoff (75% forces used).
- Case V – Static water pressure to SWL, dynamic waveforce, impervious sheet pile cutoff (75% forces used).
- Case VI – Static water pressure to SWL, dynamic waveforce, pervious sheet pile cutoff (75% forces used).
- Case VII – No water, no wind.
- Case VIII – No water, wind from protected side (75% forces used).
- Case IX – No water, wind from flood side (75% forces used).

Gates and Gate Monoliths. Gate monoliths will be constructed for street crossings in lieu of I-walls. The gate structures were designed for the following load conditions:

- Case I – Gate closed, static water pressure t o SWL, no wind, impervious sheet pile cutoff, no dynamic wave force.
- Case II – Gate closed, static water pressure t o SWL, no wind, pervious sheet pile cutoff, no dynamic wave force.
- Case III – Gate closed, static water pressure with water level 2 feet above SWL, no wind, impervious sheet pile cutoff, no dynamic wave force (75% forces used).
- Case IV – Gate closed, static water pressure with water level 2 feet above SWL, no wind, pervious sheet pile cutoff , no dynamic wave force (75% forces used).
- Case V – Gate closed, static water pressure t o SWL, dynamic wave force, impervious sheet pile cutoff (75% forces used).
- Case VI – Gate closed, static water pressure t o SWL, dynamic wave force, pervious sheet pile cut off (75% forces used).
- Case VII – Gate open, no wind, truck or train on protected edge of base slab.
- Case VIII – Gate open, no wind, truck or train on floodside edge of base slab.
- Case IX – Gate open, wind from protected side, truck or train on flood side edge of base slab (75% forces used).
- Case X – Gate open, wind from flood side, truck or train on protected edge of base slab (75% forces used).

3.2.3.6.3.3.2. Cousins Pumping Station Complex - Reference 40

General. The Cousins Pumping Station will hold two 1,050 cfs horizontal pumps.. The station consists of a concrete suction tube, pump and discharge tube that will ultimately transfer water from Cousins Canal to Harvey Canal. The structure will be supported by a timber pile foundation Other features of the project include 1123 feet of I-type floodwall protection along the banks of the pumping station discharge channel and 184 feet of cantilever sheet pile protection along the station discharge channel. The Destrehan Avenue Bridge will be lengthened by adding a sixty-foot span to accommodate the widening of the Cousins Pumping Station discharge channel. A bottom roller floodgate will be provided at each end of the bridge. The following is a summary of the pertinent structural design criteria for these structures.

Water Elevations	**Elevations (Feet N.G.V.D.)**
Still Water Level (Harvey Canal)	7.5
Still Water Level + 2 feet Freeboard (Harvey Canal)	9.5
Low Water Level (Harvey Canal)	0.0
Cousins Pumping Station	-8.5
Levee and Floodwall Net Grades	
I-Walls, Cantilever Sheet Pile Wall, Bottom Roller Floodgates	11.5
Pumping Station Frontal Protection T- Wall	11.5
Culvert	11.5
Discharge Channel Closure Wall	11.5

Culvert. The proposed culvert consists of a pile-supported, "float-in' type gravity structure. The structure was designed for computed hull stresses foundation loads at water elevations -1.0 NGVD, +9.50 NGVD, and +11.50 NGVD. The structure was also checked for floatation with a required SF of 1.30.

Destrehan Avenue Bridge. The lengthening of the Destrehan Avenue bridge was designed in accordance with AASHTO requirements for HS-20 loading. Group 1 and 3 load cases were considered in the bridge design.

Bottom Roller Gates and Gate Monoliths. The pile designs for the bottom roller gate monoliths, based on pile load tests, are designed with a factor of safety = 2.0. The following load cases were used for preliminary design:

- Case I – Gate closed, static water pressure to SWL, no wind, impervious sheet pile cutoff, no dynamic wave force (100% forces used).
- Case II – Gate closed, static water pressure to SWL, no wind, pervious sheet pile cutoff , no dynamic wave force (100% forces used).
- Case III – Gate closed, static water pressure to SWL +2 feet, no wind, impervious sheet pile cutoff, no dynamic wave force (75% forces used)
- Case IV – Gate closed, static water pressure to SWL +2 feet, no wind, pervious sheet pile cutoff, no dynamic wave force (75% forces used).

- Case V – Gate closed, wind from protected side (75% forces used).
- Case VI – Gate closed, winds from flood side (75% forces used).
- Case VII – gate open, no wind, and truck on protected side edge of base slab (100% forces used).
- Case VIII – Gate open, no wind, and truck on flood side edge of base slab (100% forces used).
- Case IX – gate open, wind from protected side, and truck on flood side edge of base slab (75% forces used).
- Case X – Gate open, wind from flood side, and truck on protected side edge of base slab (75% forces used).

I-Type Floodwall. In the design of the I-walls, the following loading cases were considered:

- Case I – Water SWL, Q-Case , F.S. = 1.5.
- Case II – Water to SWL + 2 feet, Q-Case, F.S. = 1.0.
- Case III – Water to SWL, S-Case, F.S. =1.2.
- Case IV – Water to LPL with lateral earth pressure.

Pumping Station Frontal Protection. The pumping station frontal protection will be provided by a T-wall, supported on prestressed concrete piles. A continuous steel sheet pile wall will be provided beneath the base for seepage cutoff purposes. The T-walls were designed for the following load conditions:

- Case I – Static water pressure to SWL, no wind, impervious sheet pile cutoff (100% of forces used).
- Case II – Static water pressure to SWL, no wind, pervious sheet pile cutoff (100% of forces used).
- Case III – Static water pressure to SWL +2, no wind impervious sheet pile cutoff (75% of forces used).
- Case IV – Static water pressure to SWL +2 feet, no wind, pervious sheet pile cutoff (75% of forces used).
- Case V – Water at low water level, no wind (100% of forces used).
- Case VI – Water at low water level, wind from protected side (75% of forces used).

Closure Wall. The closure wall will consist of two rows of 84 inch diameter piles filled with sand. The floodside of the wall will be lined with steel sheet pile. A 18 feet by 4 feet cast-in-place concrete cap will connect the cylinder piles to the sheet piles. In the design of the closure wall, the following loading cases were considered:

- Case I – Water to SWL, Q-Case , F.S. = 1.5.
- Case II – Water to SWL +2 feet, Q-Case, F.S. = 1.0.

3.2.3.6.3.3.3. West of Algiers Canal Hurricane Protection - Sector Gate Complex – Reference 38

General. This section presents the structural design criteria used to construct the Sector Gate Complex portion of the East Harvey Canal Hurricane Protection features of the West Bank and Vicinity, New Orleans, Louisiana Project. The Sector Gate Complex is located in the Harvey Canal 250 feet downstream of the Lapalco Bridge. It consists of sector gate structure with a 125 foot opening and a sill elevation of -16.0. The east side of the structure will be tied in by a floodwall to a floodwall running along the east side of Harvey Canal. On the west side the structure will be tied by a T-wall to a concrete flume located under Lapalco Bridge. An I-wall will be constructed along the west side of Harvey Canal and will tie into the west side of the concrete flume under Lapalco Bridge. The sector gate structure and tie-in floodwalls will be built to elevation 11.5.

Basic Data. Basic data relevant to the elevations of the water surface, structure elevations and dimensions is as follows:

Design Water Elevations (Feet, NGVD)

Load Case	Gulf Side	Protected Side
Construction(Graving Site)	-	-
Transport Loading	1.3	1.3
Setting Condition (No Backfill)	1.3	1.3
Normal Operation	1.3	1.3
Max. Direct Gate Operation	3.0	-1.0
Max. Reverse Gate Operation	-1.0	4.0
Max. Direct Head - No Hurricane	5.0	0.0
Max. Reverse Head - Hurricane	-1.0	4.0
Direct Head – Hurricane (includes 2’ for subsidence)	9.5	-1.0
Direct Head – Hurricane (includes 2’ for subsidence)	9.5	-1.0
Direct Head - Hurricane – Plus Freeboard	11.5	-1.0
Maintenance Dewatering	5.0	4.0

Structure Elevations (NGVD)

Top of Floodgate	11.5
Top of Fender and Guidewalls	10.5 (9 Ft. above Normal Stage)
Sill	-16.0

Lateral Pressures (At-Rest K_O)

Sand $K_O = 0.50$
Semi-compacted Cohesive Soil $K_O = 0.80$
Stone & Bedding Material $K_O = 0.50$

Structure and Foundation Loadings

a. Loadings. The loads are described in Section 3, of ETL 1110-2-355 and modified as follows:

b. Dead Loads. For draft and buoyancy analysis added 3% to the concrete unit weight to account for swelling and construction tolerances.

Uplift. In lieu of accurate flow nets to determine seepage rates, a limit approach was used for this structure. Relief drains were not considered. Cutoff sheet piling walls are on both sides. The structure was designed for the three uplift conditions:

- Uplift Condition A assumes-uniformly varying pressure between the gulfside and protected side sheet piling cutoffs.

- Uplift Condition B assumes the gulf side sheet pile cutoff is impervious; the uplift pressure equals the protected side pressure head.

- Uplift Condition C assumes the protected side sheet pile cutoff is impervious; the uplift pressure equals the gulf side pressure head.

Thermal. The use of a Nonlinear, Incremental Structural Analysis (NISA) to determine stress concentrations created during construction.

Wave Loads. The wave load refers to loads induced from a design wave when the module is buoyant. Two wave sizes are typically considered; a significant wave and a storm wave. The significant wave is anticipated within the one year construction period. The storm wave is a 50 year event and not considered. The structure shell is designed for inland waterway conditions. The total wave height observed along the GIWW is 3 Ft (trough to crest).

Soil Drag. In lieu of more accurate analysis drag shall be calculated as:

(Psoil-at-rest) x 0.5 x (Tangent of Internal Angle of Friction)

Impact. Boat impact on the gate is a 125 kip point load. This impact is applied to the gates and impact zone of the concrete walls.

Load Case Description

- **Normal Operation**. The gates are open; water stage is at El. 1.3. This is a usual load case with the Ultimate Strength Design (USD) Hydraulic Load Factor equal to 1.3.
- **Maximum Differential Head W/ Gate Operational**. The gates are designed to operate with a 4' head. The maximum stage is at El. 3.0 and the minimum stage is El. -1.0. This is a usual load case with the ultimate Strength Design (USD) Hydraulic Load Factor equal to 1.3.

- **Maximum Reverse Head**. The gates are designed to operate with a 5’ reverse head. The Protected Side stage is at El. 4.0 and the minimum Gulfside stage is El. -1.0.. This is a usual load case with the Ultimate Strength Design (USD) Hydraulic Load Factor equal to 1.3. This is also the maximum reverse operating head.
- **Maximum Direct Head -No Hurricane**. Gates closed with Gulfside at E1.5.0 and the Protected side at El. 0.0. This is a usual load case with the Ultimate Strength Design (USD) Hydraulic Load Factor equal to 1.3.
- **Maximum Direct Head - Hurricane Condition**. Design hurricane, the gates are closed. The Gulfside water stage is at El. 9.3 and the Protected side is at El. 1.0. The El 9.3 includes an allowance for future ground subsidence and sea level rise. This is a usual load case with the Ultimate Strength Design (USD) Hydraulic Load Factor equal to 1.3.
- **Maximum Direct Head Plus Freeboard- Hurricane Condition**. Design hurricane, the gates are closed. The Gulfside water stage is at El. 11.3 and the Protected side is at El. – 1.0. Two feet of freeboard are included. This is an unusual load case with the Ultimate Strength Design (USD) Hydraulic Load Factor reduced to 1.0..
- **Maintenance Dewatering**. Maintenance dewatering condition with the Gulfside needle dam experiencing a water stage at El. 5.0 and the Protected side water stage at El. 4.0. This is a short term loading; the USD Hydraulic Load Factor is reduced to 1.0.

3.2.3.6.3.4. Sources of Construction Materials

3.2.3.6.3.4.1. Sheet Pile. Generally, the sheet pile sections specified during advertisement were used for construction. However, sheet pile section substitutions conforming to the minimum required section modulus was allowed, primarily in contracts constructed after 1990. Below, is a table of sheet pile sections broken down by DM.

Westwego to Harvey	
Company Canal Floodwall	PZ-22*, Casteel CZ-128
Old Westwego to New Westwego PS Floodwall	IN-RU3, Casteel CZ-114
Cousins PS Complex	
Discharge Channel Floodwalls	Arbed AZ-46, AZ-13, PZ-27

* As advertised – not confirmed as built

3.2.3.6.3.4.2. Levee Materials (Westwego to Harvey Canal). The majority of proposed levees will be built of the materials obtained from adjacent borrow pits except for the Harvey Canal portion, which will be obtained from Highway 45 levee borrow area.

Highway 45 levee from BL Station 425+46 to 575+89 will be constructed in one lift with semi-compacted clay fill. All other levees will be constructed in three (3) lifts with an approximate 3-year interval between successive lifts. Lift 1 consists of constructing the levee and berms to full net grade and section, with construction of the berms preceding levee construction. The levees will be restored to net grade in Lift 2: in Lift 3, the levee crown will be built one (1) foot above net grade. The berms will not be rebuilt after initial placement as part of Lift 1.

3.2.3.6.4. As-built Conditions

3.2.3.6.4.1. Changes between design and construction (i.e. cross sections, alignment, sheet pile tip el, levee crest el.)

3.2.3.6.4.1.1. DACW29-95-C-0103. Westwego to Harvey Canal, Louisiana, Westbank Hurricane Protection Levee, Estelle Pump Station to LP&L Powerlines, 1st Lift, Jefferson Parish, Louisiana

Reviewed Mod Log Report and Mod Documents, no applicable modifications or changes found.

3.2.3.6.4.1.2. DACW29-96-C-0032. Westwego to Harvey Canal, Hurricane Protection Plan, New Westwego Pumping Station to Orleans Village, Jefferson Parish, Louisiana

Modification No. P00011 allowed CZ-101 sheetpiles conforming to ASTMA328 with a minimum material thickness of .335 inches and maximum overall width of 27 inches, to be substituted for SZ-20 or PZ-22 sheetpiles.

Modification No. A00001 revised requirement for Atterberg Limit Tests from one for each density test or every 2,000 cu. yds. of semi-compacted fill, to one for each ten density tests, or every 20,000 cu. yds.

Modification No. A00009 extended the sandfill levee base by 218 feet to facilitate the transition between the levee and sheetpile wall.

3.2.3.6.4.1.3. DACW29-99-C-0014. Westwego to Harvey, LA, Hurricane Protection Levee, V-Line Levee, East of Vertex, Second Lift, Louisiana Highway 3134 to Estelle Pumping Station, Jefferson Parish, Louisiana

Modification No. A00001 provided for levee repairs of failed sections of completed work between Station 220+00 B/L and 232.00 B/L and between Station 200+00 B/L and 190+00 B/L, by partially degrading the originally designed levee section and using this material to construct a stability berm on the protected side. This, in essence, reduced the levee cross-section and lowered the crown height from elevation 10.0 to elevation 9.0, and reduced the crown width from 10 feet to 7 feet.

3.2.3.6.4.1.4. DACW29-00-C-0047. Westwego to Harvey Canal, Louisiana, Hurricane Protection Levee, Westwego Seaplane Airport Canal Closure, First Lift, Jefferson Parish, Louisiana

Modification No. A00001 raised the sandbase to El. 2.0 (NGVD) to facilitate placing embankment in the dry.

3.2.3.6.4.2. Inspection during original construction, QA/QC, state what records are available –

3.2.3.6.4.2.1. DACW29-96-C-0032 – WW – HC, NEW WESTWEGO PUMP STA – ORLNS VILLAGE JEF PAR LA

Attached are percent complete lists and grain size analysis records.

3.2.3.6.4.2.2. DACW29-98-C-0043 – SELA, KEYHOLE CANAL 4TH TO LAPALCO, JEF PAR

Attached from time to time are the preparatory inspection reports and the status summary of the job.

3.2.3.6.4.2.3. DACW29-99-C-0014 – WWHC, V-LINE LEV, E OF VTEX, 2ND LIFT, JEF PAR LA

No QA/QC Reports found.

3.2.3.6.4.2.4. DACW29-00-C-0047 – WWHC, SEAPLANE AIRPORT CNL, 1ST LIFT, JEF PAR

Attached are preparatory phase reports.

3.2.1.3.6.4.2.5. DACW29-01-C-0029 – WB, ALGIERS LEV ENL, HERO CNL **–** BEL CHAS PLAQ PAR LA

No QA/QC Reports found.

3.2.3.6.5. Inspection and maintenance of original construction

3.2.3.6.5.1. Annual Compliance inspection (i.e. trees, etc.) –As stated in the Lake Cataouatche Section, this area received a rating of "ACCEPTABLE" for the levee system under the Annual Compliance Program for the West Jefferson Levee District.

3.2.3.6.5.2. Periodic inspections – There are no structures under the Periodic Inspection Program in the Westwego to Harvey area, of the West Bank and Vicinity Hurricane protection project.

3.2.3.6.6. Other Features – Jefferson West Bank, Westwego to Harvey

3.2.3.6.6.1. Brief Description. The primary components of the hurricane protection system for the Jefferson West Bank, Westwego to Harvey subarea are described above, namely the levees and floodwalls designed and constructed by the Corps of Engineers. However, other drainage and flood control features that work in concert with the Corps of Engineers levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section will describe and present the criteria and pre-Katrina conditions of the interior drainage system, pump stations, and the Mississippi River Flood Protection System. There are currently no non-Corps levees or floodwalls in this polder. Even though the stormwater pump stations are part of the interior drainage system, they are a significant part of the system and warrant their own section.

3.2.3.6.6.2. Pre-Katrina Conditions. According to the local jurisdictions responsible for interior drainage, the storm drain system, interior canals, interior pump stations, outfall pump stations, and outfall canals were in good condition and prepared for high inflows from rainfall prior to August 29, 2005.

The Mississippi River Flood Protection System was in good condition prior to Katrina landfall.

3.2.3.6.6.3. Interior Drainage System

Overview. The Jefferson West Bank, Westwego to Harvey subarea contains about 22 square miles and generally slopes north to south from the Mississippi River. It is mostly developed except for some low areas in the southern tip. Many features are typical of large urban cities in the United States, and some features that are unique because much of the area is below sea level. Catch basins and inlets collect surface runoff from yards and streets into storm sewers and ditches. Excess runoff flows down streets and/or overland to lower areas. Open canals collect the stormwater and carry it to outfall pump stations that pump the water into the Harvey Canal, Bayou Segnette, or canals and bayous on the south side of the polder. No stormwater is pumped into the Mississippi River.

The entity responsible for local drainage in the Jefferson West Bank basin is Jefferson Parish. The Louisiana Department of Transportation and Development highways are also a part of the local drainage system.

System Components. Local drainage begins with overland flow which follows the ground topography. Figure 5 in Volume VI shows the topographic layout of Jefferson West Bank. The land generally falls south from the Mississippi River. A land feature visible on the topographic layout that affects the local drainage is a ridge that runs north-south between the Harvey Canal and Bayou Segnette. The locations of the interior ditches, canals, and pump stations were influenced by this ridge.

The land topography and development sequence influenced the storm sewer, ditch, canal, and pump station layout. There are no interior pump (lift) stations. Based on land topography and the drainage system, the subarea is divided into 125 subbasins. Pump station information is presented in Section 3.2.3.6.6.4 of this volume.

The canals are open and most are grass-lined. The canals and ditches not only collect stormwater from streets and storm sewers and covey it to the pump stations, they also are storage areas that work in conjunction with the pump stations.

Design Criteria. The current design criterion for Jefferson West Bank is the 10% storm event for all storm drainage system components. Older parts of the stormwater collection system have approximately a 2-year frequency capacity. The functional capacity of the interior canals and pump stations is 0.4 inches per hour. It will increase to 0.5 inches per hour after the SELA projects are complete (see status below). Rainfall in excess of this amount goes into temporary storage in the streets, storm sewers, ditches, and canals. There are criteria for new developments to use stormwater detention to offsite downstream impacts.

Where local drainage is considered to need improvement, Jefferson Parish is working to improve the drainage. In some cases, Jefferson Parish and Corps of Engineers are working together on projects, as presented below in the Southeast Louisiana (SELA) Urban Flood Control Projects section.

Southeast Louisiana Urban Flood Control Projects. As a result of the extensive flooding in May 1995, Congress authorized the SELA Urban Flood Control Project with enactment of the Energy and Water Development Appropriations Act for Fiscal Year 1996 and the Water Resources Development Act (WRDA) of 1996 to provide for flood control and improvements to rainfall drainage systems in Jefferson, Orleans, and St. Tammany Parishes. Jefferson Parish is the local, cost sharing sponsor for the Jefferson Parish work.

The project includes channel and pump station improvements in the three parishes. The channel and pumping station improvements in Orleans and Jefferson Parishes support the parishes' master drainage plans and generally provide flood protection on a level associated with a 10-year rainfall event, while also reducing damages for larger events.

The SELA projects in the Jefferson West Bank, Westwego to Harvey subpolder are shown in Figure 36. The work consists of adding capacity to 15 canals, increasing pumping capacity at the Cousins Pump Station, and improving two existing pump stations. Prior to Hurricane Katrina, the improvements to the two pump stations were under design, the Cousins Pump Station was under construction, 10 canals were complete, 2 canals were partially complete, and 3 canals were under design.

3.2.3.6.6.4. Pumping stations - Jefferson Parish Westwego to Harvey. Jefferson Parish is located west of the city of New Orleans and borders the west side of Orleans Parish. Figure 37 is a map of Jefferson Parish with the pump stations that were studied identified by red dots. Jefferson Parish is separated by the Mississippi River into East and West Banks. The East Bank pump stations are connected by a grid of canals. The canals running east and west serve to equalize flow between the major outfall canals, allowing rain water to flow in different directions depending on the rainfall patterns and available capacities at the pump stations. The West Bank is subdivided into sub-basins that, for smaller rainfall events, operate independently. However, over-bank flow does occur between adjacent sub-basins for a 10-year event. This report examined 6 pump stations on the East Bank with a total of 36 pumps and 17 pump stations on the West Bank with a total of 65 pumps.

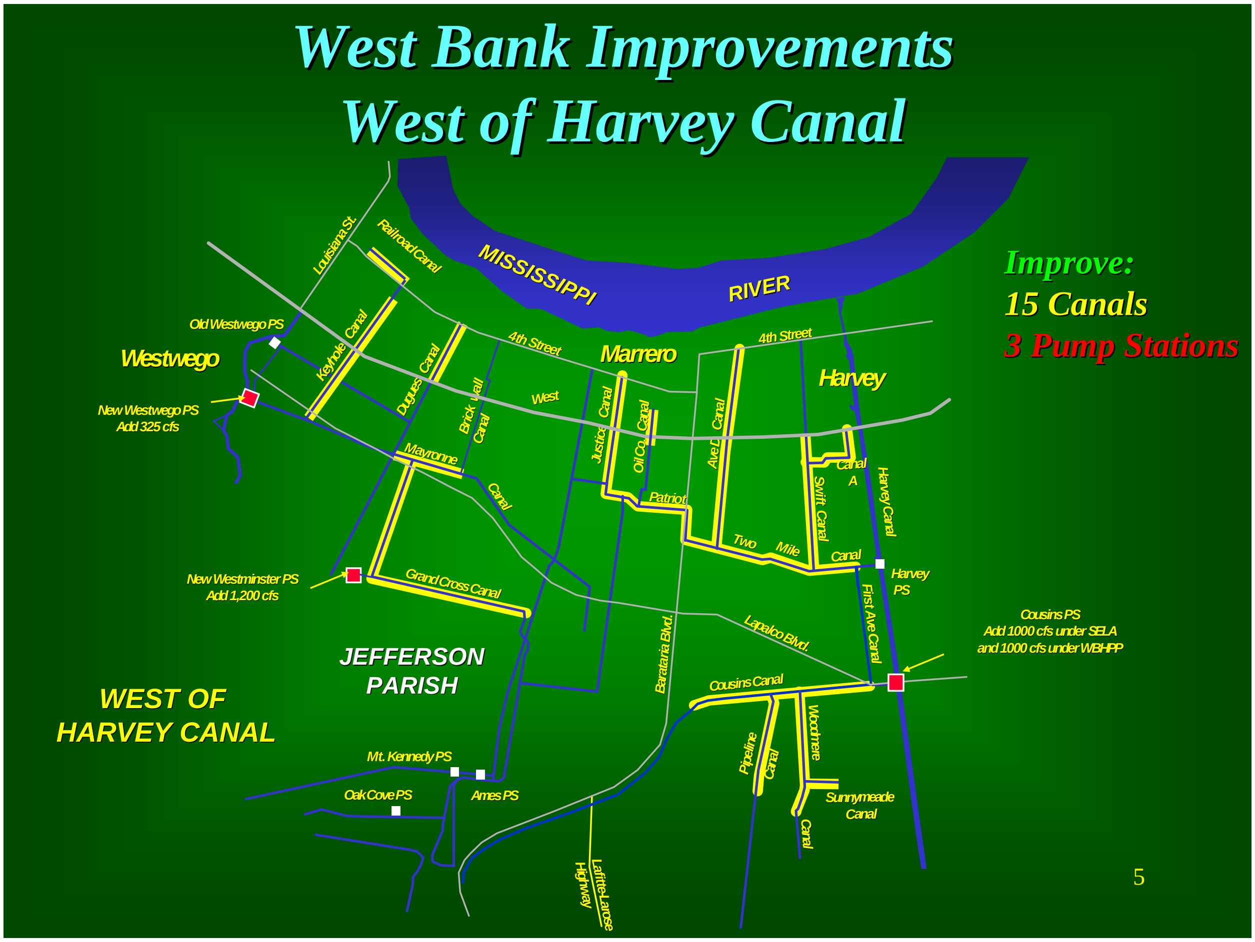


Figure 36. SELA Urban Flood Control Projects in Jefferson West Bank, Westwego to Harvey

Figure 37 is a map showing the Jefferson Parish pump stations that were used in this report. The locations of the pump stations were verified by Global Positioning System (GPS) and/or by using Google Earth Pro. The GPS coordinates were then input into Microsoft Streets and Trips (shown below).

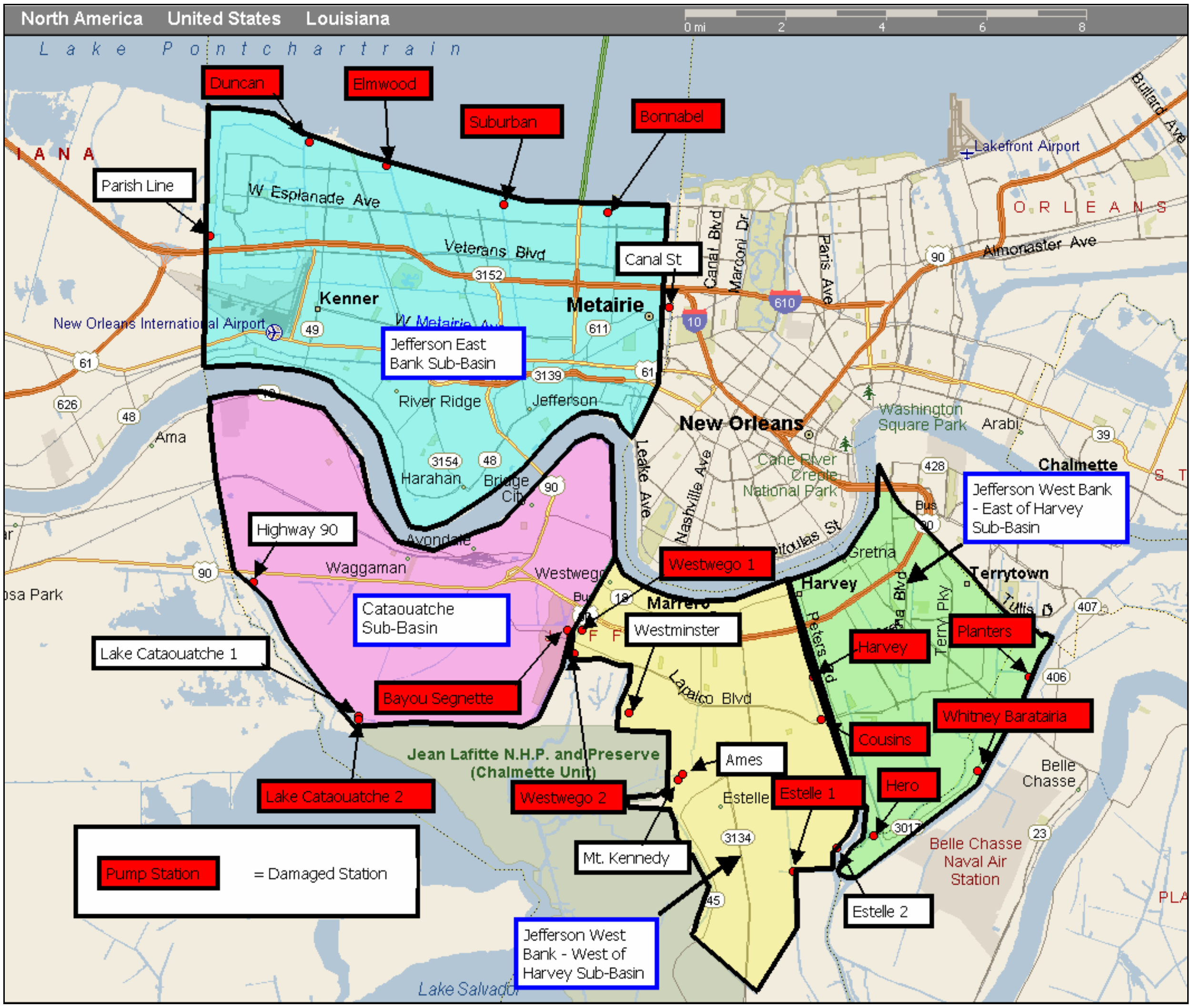


Figure 37. Jefferson Parish Pump Station Locations

Table 43 contains information about each individual pump at each of the examined pump stations in Jefferson Parish. The list is composed of information that was collected in the field. Not all information was available for each pump and was left blank or highlighted.

Table 43
Summary of Jefferson Parish Pump Stations by Drainage Basin

Basin	East Bank	Cataouatche	West Bank – West of Harvey	West Bank-East of Harvey	Total
Number of pump stations	6	4	9	3	22
Number of pumps	36	24	29	15	104
Total rated capacity (cfs)	20,662	3,346	10,695	9,958	44,661
Estimated cost of damages	$558,000	$3,000	$136,000	$61,000	$758,000

Drainage Basin

West Bank – West of Harvey

The West Bank – West of Harvey drainage basin has 8 significant pump stations, which are briefly described below. Volume VI provides more details. The basin is bordered by the Mississippi River on the north. The drainage system includes the Mississippi River, as well as wetlands and the First Ave., Two Mile, Cousins, Harvey, Pipeline, Kenta/Seivers, Grand Cross, Inner Milladoun, Bayou Segnette, WPA, G, and H Canals.

Harvey
Intake location: .... First Ave. & Two Mile Canal
Discharge location: .... First Ave. & Two Mile Canal
Nominal capacity: .... 960 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	320	n/a	Electric	n/a
2	320	n/a	Electric	n/a
3	320	n/a	Electric	n/a

Cousins No. 1
Intake location: .... Cousins Canal & First Ave. Canal
Discharge location: .... Harvey Canal
Nominal capacity: .... 800 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	50	n/a	Electric	Vertical
2	250	n/a	Diesel	Vertical
3	250	n/a	Diesel	Vertical
4	250	n/a	Diesel	Vertical

Cousins No. 2
Intake location: .... Cousins Canal & First Ave. Canal
Discharge location: .... Harvey Canal
Nominal capacity: .... 2200 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	1100	n/a	Diesel	n/a
2	1100	n/a	Diesel	n/a

Estelle

Intake location: .... Pipeline Canal
Discharge location: .... Intercoastal Waterway
Nominal capacity: .... 682 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	150	n/a	Electric	n/a
2	150	n/a	Electric	n/a
3	150	n/a	Electric	n/a
4	232	n/a	Electric	n/a

New Estelle

Intake location: .... Pipeline & Canal G
Discharge location: .... Intercoastal Waterway
Nominal capacity: .... 1140 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	570	n/a	Diesel	n/a
2	570	n/a	Diesel	n/a

Mount Kennedy

Intake location: .... Kenta/Seivers Canal
Discharge location: .... Bayou Segnette
Nominal capacity: .... 500 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	167	n/a	Electric	n/a
	167	n/a	Electric	n/a
3	167	n/a	Electric	n/a

Westminster 1 & 2
Intake location: .... Grand Cross
Discharge location: .... Wetlands
Nominal capacity: .... 1248 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	312	n/a	Electric	n/a
2	312	n/a	Electric	n/a
3	312	n/a	Electric	n/a
4	312	n/a	Electric	n/a

Ames
Intake location: .... Inner Milladoun
Discharge location: .... Bayou Segnette
Nominal capacity: .... 1930 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	390	1982	Electric	Vertical
2	390	1982	Electric	Vertical
3	1150	n/a	Diesel	Horizontal

Westwego No. 1
Intake location: .... WPA Canal
Discharge location: .... Bayou Segnette
Nominal capacity: .... 300 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	300	n/a	Diesel	Vertical

Westwego No. 2
Intake location: ....Ave H Canal
Discharge location: .... Bayou Segnette
Nominal capacity: ....935 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	312	n/a	Diesel	n/a
2	312	n/a	Diesel	n/a
3	311	n/a	n/a	n/a

3.2.3.6.6.5. Levees and floodwalls -

3.2.3.6.6.5.1. MRL - MRL Levees and floodwalls are addressed in paragraph 3.2.1.5.6.4.1 New Orleans East Bank MRL. There are no floodwalls that are part of the MRL in this reach.

3.2.3.6.6.5.2. Non Corps - Several local interest and/or private levees are located within the project area. No design criteria for these levees have been made available to the Corps.

3.2.3.7. East of Harvey Canal Area

3.2.3.7.1. Introduction - This area consists of approximately 25 miles of levee, 5 miles of floodwalls, and a sector floodgate in the Harvey Canal as shown in Figure 38 below. This area consists of work both East and West of Algiers Canal.

3.2.3.7.2. Pre-Katrina – Construction in this area started in 2000. Before Hurricane Katrina, six construction contracts were completed in this area. Another three were under construction and are still ongoing, scheduled for completion in 2007. Remaining work in this area consists of 1st enlargement levee or floodwall contracts, Pump Station Modifications and Fronting Protection contracts, and future 2nd enlargement levee contracts.

3.2.3.7.3. Design Criteria and Assumptions - Functional design criteria

3.2.3.7.3.1. Hydrology and Hydraulics. For the East of Harvey Canal area, the design hurricane characteristics are shown in Table 44; the design tracks are shown on Figure 39. The maximum wind speed was computed using the same equations as for Orleans East Bank. For each project area, the track and forward speed were selected to produce maximum wind tide levels.

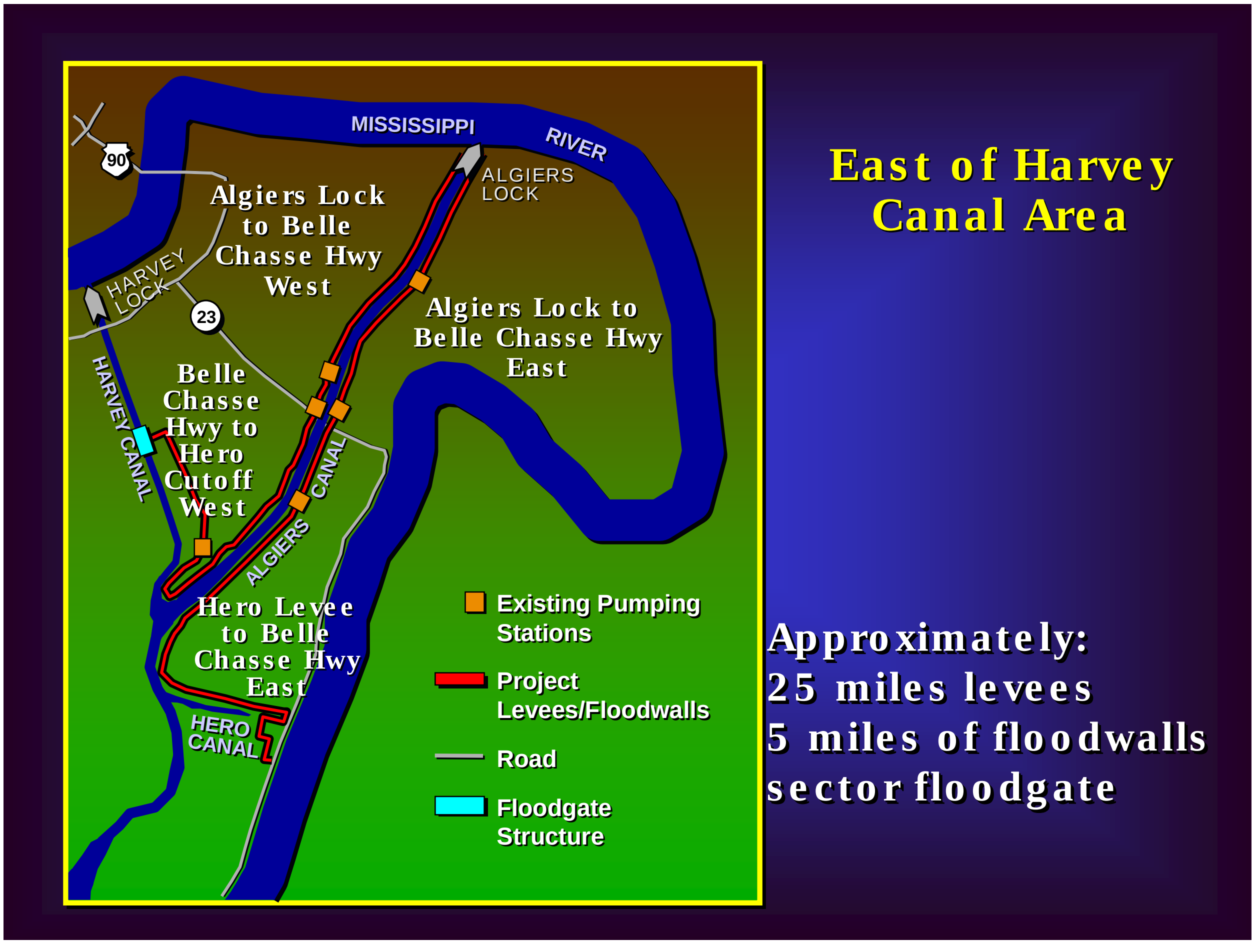


Figure 38. East of Harvey Canal Area project features

Table 44 Design Hurricane Characteristics						
Location	**Track**	**CPI, Inches**	**Radius of Maximum Winds, Nautical miles**	**Forward Speed, Knots**	**Maximum Wind Speed,[1] -Knots**	**Direction of Approach**
East of Harvey	C	27.4	30	11	100	South
[1] Windspeeds represent a 5 minute average 30 feet above ground level.						

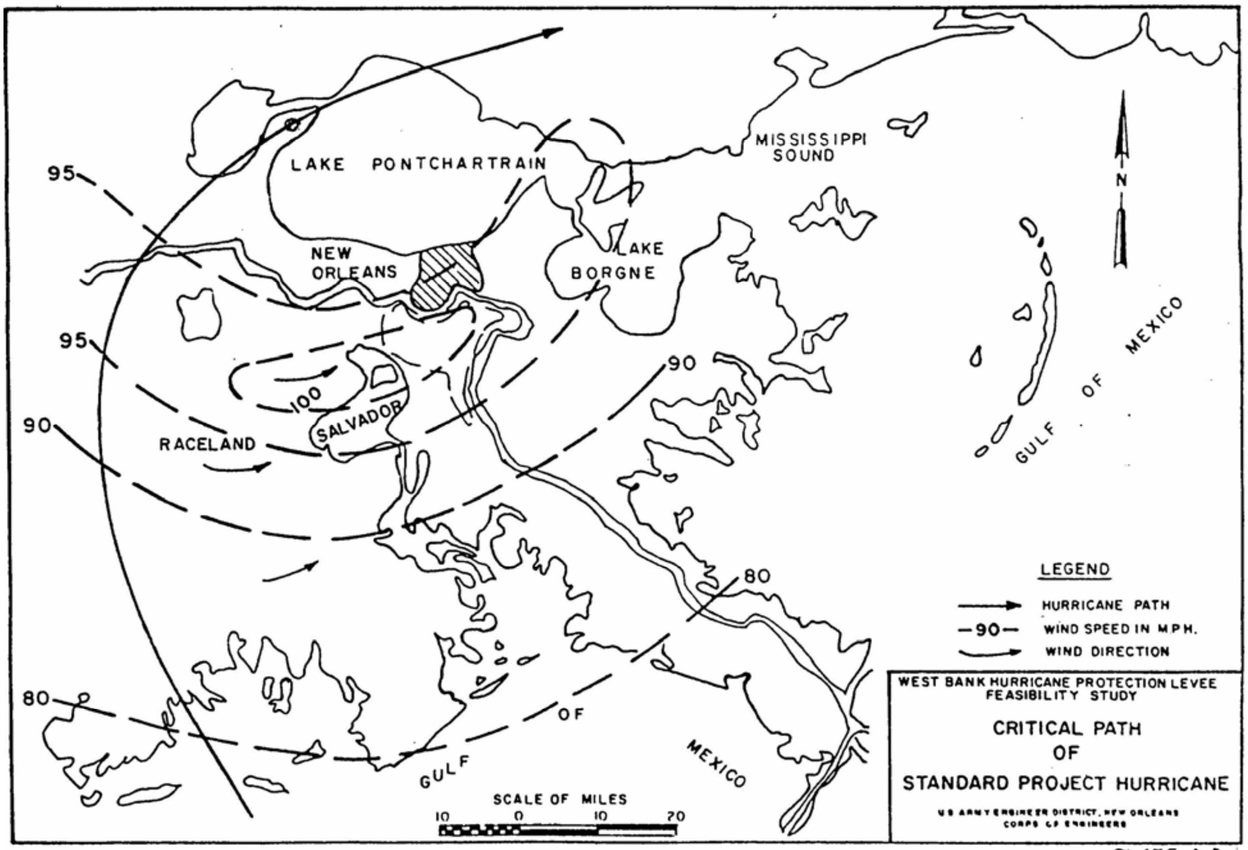


Figure 39. Critical path of standard project hurricane

3.2.3.7.3.1.1. Surge. Wind tide elevations for East of Harvey Canal area were computed using the same methodology as used for Westwego to Harvey area.

A WIFM model was used to evaluate future land loss due to subsidence and an estimated sea level rise of 0.2 ft per 50 years. Model results indicated an increase in wind tide level of 1.0 ft by the year 2040.

3.2.3.7.3.1.2. Waves. For the East of Harvey Canal area, some levees and floodwalls would be sheltered from storm generated runup; small locally generated waves could occur. These small waves would be likely to occur along Oakville levee, Harvey Canal, and Algiers Canal. Wave runup for all levees and floodwalls was calculated using methodology described in 1984 Shore Protection Manual.

3.2.3.7.3.1.3. Summary. Table 45 contains maximum surge or wind tide level, wave, and design elevation information.

Table 45 Wave Runup and Design Elevations (Transition zones not tabulated – Governing Report is listed)								
Location	**Report**	**Average Depth of fetch, ft**	**Significant Wave Height Hs, ft**	**Wave Period, T, sec**	**Maximum Surge or Wind Tide Level, Ft**	**Runup Height Ft**	**Freeboard, Ft**	**Design Elevation Protective Structure, ft**
Oakville Levee	Feasibility Study	NA	1.0	2.7	7.0 NGVD	2.0	-	9.0 NGVD
Harvey Canal	Feasibility Study	NA	1.0	2.7	7.5 NGVD	2.0	-	9.5 NGVD
Algiers Canal	Feasibility Study	NA	1.0	2.7	7.5 NGVD	2.0	-	9.5 NGVD
Hero Canal Reach	Feasibility Study	NA	NA	NA	7.5 NGVD	3.0	-	10.5 NGVD

3.2.3.7.3.1.4. Interior Drainage – The design includes increasing the capacity of the Cousins Pumping Station. When the floodgate structure on the Harvey Canal is closed, the existing Harvey Pumping Station would be shut down, and interior drainage would be diverted to the Cousins Pumping Station. The capacity of this pumping station would be increased by 1,000 cfs, the outfall canal for the pump station enlarged, and the 1st Avenue Canal, which connects the Harvey and Cousins Pumping Stations, would be enlarged to handle the additional drainage. For the feasibility study, Manning's equation was used to size the 1st Avenue Canal. The continuity equation was used to design the outfall canal, with a velocity of 3.5 fps chosen based on erodibility of the channel bottom. The improvements to 1st Avenue Canal were modeled using UNET as part of the SELA project. The increase in capacity to Cousins Pumping Station was also evaluated as part of the SELA project. Improvements to 1st Avenue Canal and Cousins Pumping Station were constructed concurrent with the SELA drainage improvements.

3.2.3.7.3.2. Geotechnical. This report addresses design assumptions and parameters for new levees, enlargement of existing levees and floodwalls. The project consists of three (3) design reaches for approximately 12,000 feet of floodwall and 125,000 feet of levee. Additional information included in Reference 31.

3.2.3.7.3.2.1. Geology. The project site is located on the Deltaic Plain Portion of the Mississippi River Alluvial Plain. Specifically, the area is located on the northern edge of the Barataria Basin on the western side of the Mississippi River between miles 73 to 98 above head of passes. The Barataria Basin is an interdistributary basin dominated by features which include natural levee ridges, crevasse-splay deposits, marsh, lake and swamps. The eastern and northern edge of the basin is defined by the natural levee ridge of the Mississippi River and the western edge of the basin is defined by the Bayou Lafourche natural levee ridge. The Gulf of Mexico constitutes the southern boundary. Elevations vary from approximately +10 to +15 feet NGVD in the back swamp and lake areas to below 0 feet NGVD in areas under pump.

The area is protected from Mississippi River overflows by the mainline levee system. Flooding originating in the Gulf of Mexico can travel across the marsh and through Bayou Barataria to threaten the area from the south. To protect the area from this tidal and storm surge flooding, local interests have constructed a network of levees that provide a limited degree of protection.

3.2.3.7.3.2.2. Foundation Condition. The foundation soils are predominantly fat clays (CH) varying in consistency from very soft to medium. There are occasional layers of silt (ML), silty sand (SM), and lean clays (CL). Layers of organic clays, which typically display high moisture contents, exist in the area near the Intracoastal Waterway form the original ground surface down to approximately elevation -20.

3.2.3.7.3.2.3.Field Exploration. Five general-type borings were taken along parts of the proposed alignment in October 1992, and two borrow borings were taken in the borrow pit in March 1994. Other undisturbed and general-type borings used for design can be found in the following reports stored in the Foundation branch of the New Orleans District.

a. Algiers Lock and Canal – Soils Investigation, June 1948

b. Algiers Lock and Canal – Definite Project Report, June 1948

Raising of the existing Algiers Canal levee will provide part of the flood protection for this project.

Four undisturbed borings and four general-type borings were taken along parts of the proposed alignment in January 1995.

3.2.3.7.3.2.4. Underseepage. Not addressed.

3.2.3.7.3.2.5. Hydrostatic Uplift. Not addressed.

3.2.3.7.3.2.6. Pile Foundation. Not addressed.

3.2.3.7.3.2.7. Slope Stability. The project was divided into three design reaches based on boring data. Reach III was split into subreaches “a” and “b” based on differing surface conditions. The reaches are as follows:

Reach I- .... Floodwall west of Algiers Canal

Reach II - .... Algiers Canal, East and West Bank

Reach IIIa- ....North of Hero Canal

Reach IIIb- ....South of Hero Canal

The still water level (SWL) used for reaches I and II was elevation 7.5 NGVD. Low water level was used as elevation 0.0. The SWL for Reach IIIa was elevation 8.5 and elevation 7.0 for Reach IIIb, both with a low water level of elevation 0.0.

<u>Stability of Levees</u>**.** Existing conditions along the proposed alignment were estimated and the slopes and berm distances for the proposed levee were designed for the (Q) construction case. A factor of safety (F.S.) of 1.3 is required for the levee stability. For Reaches IIIa and IIIb, surveys were taken in January 1995.

Based on historical data from the Larose to Golden Meadow area, shrinkage and settlement of levee fill should be in the range of 20 to 30 percent over the 3 or 4 years between the first and second lift. The final lift will compensate for the expected lifetime settlement of the levee. The Algiers Canal levee should experience minimal settlement sine the centerline of the levee will remain unchanged.

3.2.3.7.3.2.8. I-Type Floodwall. I-wall stability and required sheet pile penetration was estimated using a penetration to head ratio of 3:1 to estimate sheet pile penetration. There is no significant wave load on the I-wall. For Detail Design of the floodwall, the following criteria will be followed:

Q-Case

F.S. = 1.5 with water to flowline or SWL.

F.S. = 1.25 with water to flowline plus approved freeboard for river levees or with SWL and waveload for hurricane protection levees.

F.S. = 1.0 with SWL plus 2 ft freeboard for hurricane protection levees.

S-Case

F.S. = 1.2 with water to flowline or SWL and waveload. If a hurricane protection floodwall has no significant waveload, determine the penetration using Q-case criteria only.

F.S. = 1.0 with water to flowline plus approved freeboard for river levees.

3.2.3.7.3.2.9. T-Type Floodwall. Not used.

3.2.3.7.3.3. Structural – (East and West of Algiers Canal - Reference 59)

General. The structural portions of project that have been completed include three segments of I-wall across the Belle Chasse Tunnel on both the east and west side and adjacent to the railroad on the west side. None of the gate structures or the pump station frontal protection structures has been constructed.

I Walls. Analyses for the cantilevered I-walls were performed using the Corps of engineers CWALSHT program. The analyses were performed by applying a factor of safety of 1.5 to the "Q" soil parameters when considering the design hurricane storm level at el 9.5. A factor of safety of 1.0 was applied to the "Q" soil parameters for a separate analysis evaluating a water level at el. 11.5.

Minimum Penetration. The minimum sheetpile penetration for cantilever sheetpile walls was determined by providing a minimum sheetpile penetration below the ground surface to water head ratio of 3:1

Loading Cases. In the design of the I-walls, the following loading cases were considered:

- Case I – Water to SWL, Q-Case, F.S. = 1.5
- Case II – Water to SWL + 2', Q-Case, F.S. = 1.0

3.2.3.7.3.4. Sources of Construction Materials

3.2.3.7.3.4.1. Sheet Pile. Generally, the sheet pile sections specified during advertisement were used for construction. However, sheet pile section substitutions conforming to the minimum required section modulus was allowed, primarily in contracts constructed after 1990. Below, is a table of sheet pile sections for East of Harvey, broken down by DM..

East of Harvey	
East & West of Algiers Canal	
Belle Chasse PS#1 Tie-In	**
Belle Chasse PS#2 Tie-In	PZ-27
Planters PS Tie-In	PZ-27
S&WB PS#11 Tie-In	**
S&WB PS#13 Tie-In	**

* As advertised – Not confirmed as built

** Information not available at the time of publication

3.2.3.7.3.4.2. Levee material - Borrow material will be hauled from a nearby pit where the limits have been preliminarily established.

3.2.3.7.4. As-built Conditions

3.2.3.7.4.1. Changes between design and construction (i.e. cross sections, alignment, sheet pile tip el, levee crest el.)

3.2.3.7.4.1.1. DACW29-01-C-0029. Westbank and Vicinity, New Orleans, Louisiana, Hurricane Protection Project, Algiers Canal Levee Enlargement and Floodwall, East Side Hero Levee to Belle Chase Highway, Plaquemines Parish, Louisiana

Modification No. P00001 provided for the Government to furnish 347 15-foot SPZ-22 sheetpiles which were substituted for the contract required PZ-22 sheetpiles.

3.2.3.7.4.2. Inspection during original construction, QA/QC, state what records are available –

See paragraph 3.2.1.5.4.2 New Orleans East Bank for description of how records are kept.

3.2.3.7.5. Inspection and maintenance of original construction.

3.2.3.7.5.1. Annual Compliance inspection (i.e. trees, etc.) – As stated in the Lake Cataouatche Section, this area received a rating of “ACCEPTABLE” for the levee system under the West Jefferson Levee District.

3.2.3.7.5.2. Periodic inspections - There are no structures under the Periodic Inspection Program in the East of Harvey Canal area, of the West Bank and Vicinity Hurricane protection project.

3.2.3.7.6. Other Features – Jefferson and Orleans West Bank, East of Harvey

3.2.3.7.6.1. Brief Description. The primary components of the hurricane protection system for the Jefferson and Orleans West Bank, East of Harvey subarea are described above, namely the levees and floodwalls designed and constructed by the Corps of Engineers. However, other drainage and flood control features that work in concert with the Corps of Engineers levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section will describe and present the criteria and pre-Katrina conditions of the interior drainage system, pump stations, and the Mississippi River Flood Protection System. There are currently no non-Corps levees or floodwalls in this polder. Even though the stormwater pump stations are part of the interior drainage system, they are a significant part of the system and warrant their own section.

3.2.3.7.6.2. Pre-Katrina Conditions. According to the local jurisdictions responsible for interior drainage, the storm drain system, interior canals, interior pump stations, outfall pump stations, and outfall canals were in good condition and prepared for high inflows from rainfall prior to August 29, 2005.

The Mississippi River Flood Protection System was in good condition prior to Katrina landfall.

3.2.3.7.6.3. Interior Drainage System.

Overview. The Jefferson and Orleans West Bank, East of Harvey subarea contains about 30 square miles and generally slopes north to south from the Mississippi River. It is mostly developed except for a few tracts near the Harvey Canal in Jefferson Parish and Denver Canal in Orleans Parish. Many features are typical of large urban cities in the U.S., and some features that are unique because much of the area is below sea level. Catch basins and inlets collect surface runoff from yards and streets into storm sewers and ditches. Excess runoff flows down streets and/or overland to lower areas. Open and enclosed canals collect the stormwater and carry it to stormwater pump stations that pump the water into the Intracoastal Waterway. No stormwater is pumped into the Mississippi River.

The entities responsible for local drainage are Jefferson Parish and Orleans Parish in their respective jurisdictions. The Louisiana Department of Transportation and Development highways are also a part of the local drainage system.

System Components. Local drainage begins with overland flow which follows the ground topography. Figure 5 in Volume VI shows the topographic layout of Jefferson and Orleans West Bank. The land generally falls from the Mississippi River to the Intracoastal Waterway.

The land topography and development sequence influenced the storm sewer, ditch, canal, and pump station layout. There are no interior pump (lift) stations. Based on land topography and the drainage system, the subarea is divided into 118 subbasins. Pump station information is presented in Section 3.2.3.7.6.4 of this volume.

The canals are open and most are grass-lined. The interior canals and ditches not only collect stormwater from streets and storm sewers and covey it to the pump stations, they also are storage areas that work in conjunction with the pump stations.

Design Criteria. The current design criterion for Jefferson West Bank is the 10% storm event for all storm drainage system components. Older parts of the stormwater collection system have approximately a 2-year frequency capacity. The functional capacity of the interior canals and pump stations is 0.4 inches per hour. It will increase to 0.5 inches per hour after the SELA projects are complete (see status below). Rainfall in excess of this amount goes into temporary storage in the streets, storm sewers, ditches, and canals. There are criteria for new developments to use stormwater detention to offsite downstream impacts.

Where local drainage is considered to need improvement, the parishes are working to improve the drainage. In some cases, Jefferson Parish and Corps of Engineers are working together on projects, as presented below in the Southeast Louisiana (SELA) Urban Flood Control Projects section.

Southeast Louisiana Urban Flood Control Projects. As a result of the extensive flooding in May 1995, Congress authorized the SELA Urban Flood Control Project with enactment of the Energy and Water Development Appropriations Act for Fiscal Year 1996 and the Water Resources Development Act (WRDA) of 1996 to provide for flood control and improvements to rainfall drainage systems in Jefferson, Orleans, and St. Tammany Parishes. Jefferson Parish is the local, cost sharing sponsor for the Jefferson Parish work.

The project includes channel and pump station improvements in the three parishes. The channel and pumping station improvements in Orleans and Jefferson Parishes support the parishes' master drainage plans and generally provide flood protection on a level associated with a 10-year rainfall event, while also reducing damages for larger events.

The SELA projects in the Jefferson West Bank, East of Harvey subarea are shown in Figure 40. The work consists of adding capacity to 5 canals, increasing pumping and adding a new pump station - Whitney/Barataria Pump Station. Prior to Hurricane Katrina, the pump station was partially completed, 4 canals were complete, and 1 canal was partially complete, but functional when Katrina made landfall.

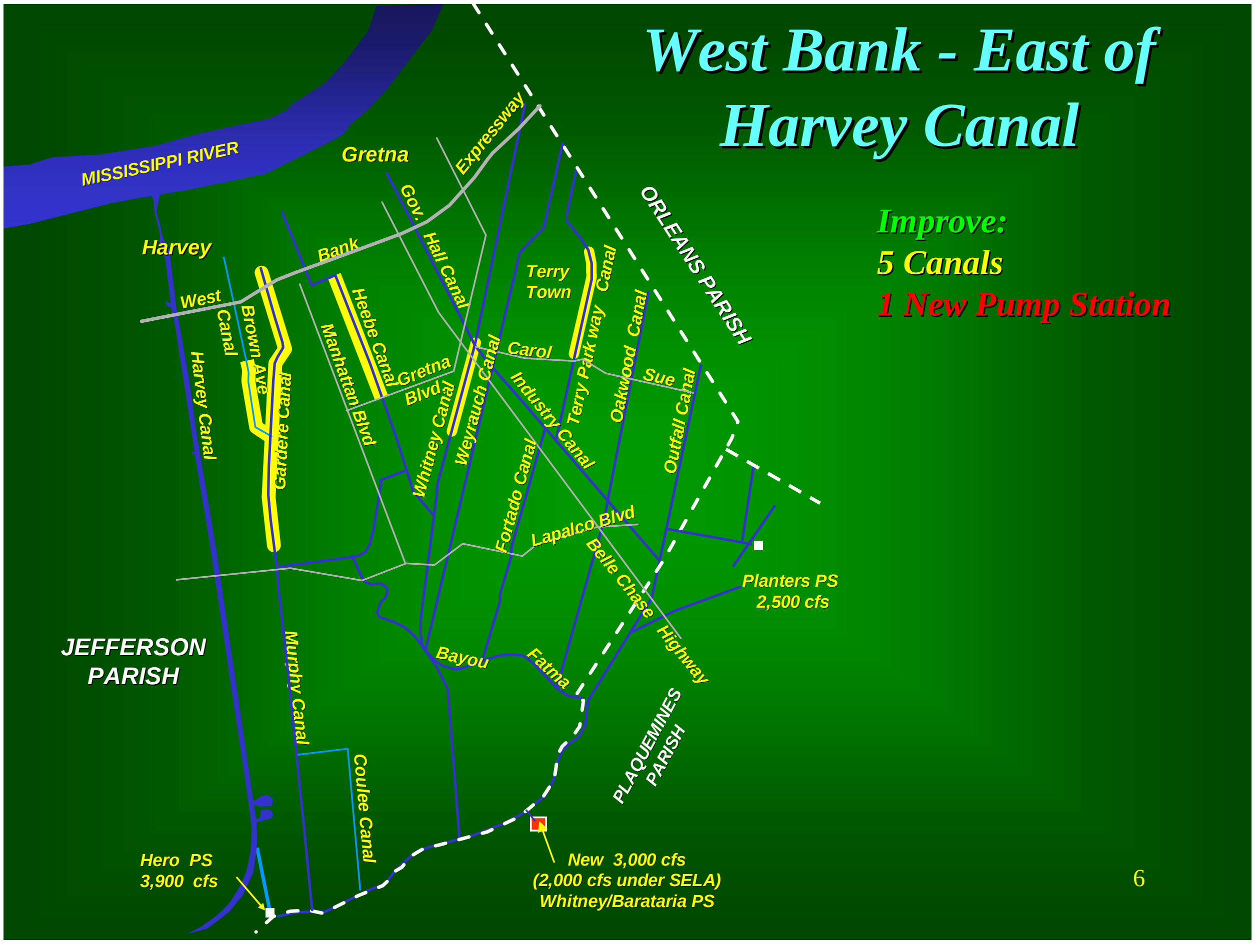


Figure 40. SELA Urban Flood Control Projects in Jefferson West Bank, East of Harvey

3.2.3.7.6.4. Pumping stations - Jefferson Parish West Bank and Orleans Parish West Bank

Figure 41 is a map of Jefferson Parish with the pump stations that were studied identified by red dots. Jefferson Parish is separated by the Mississippi River into East and West Banks. The West Bank is subdivided into sub-basins that, for smaller rainfall events, operate independently. However, over-bank flow does occur between adjacent sub-basins for a 10-year event. This report examined 17 pump stations on the West Bank with a total of 65 pumps. Figure 42 is a map showing the Orleans Parish pump stations that were used in this report. The locations of the pump stations were verified by Global Positioning System (GPS) and/or by using Google Earth Pro. The GPS coordinates were then input into Microsoft Streets and Trips (shown below).

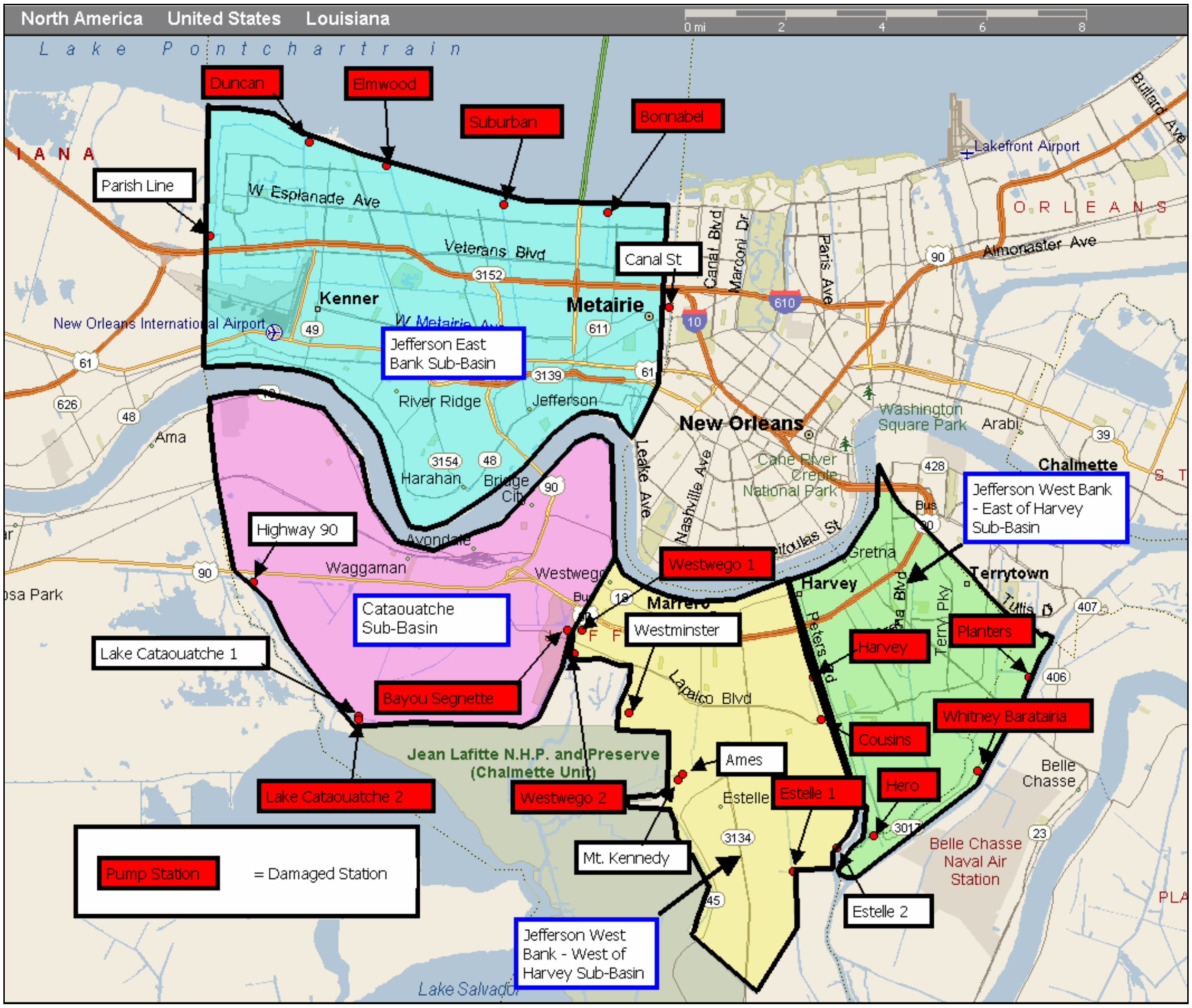


Figure 41. Jefferson Parish Pump Station Locations

Table 46 contains information about each individual pump at each of the examined pump stations in Jefferson Parish. The list is composed of information that was collected in the field. Not all information was available for each pump and was left blank or highlighted.

Table 47 contains information about each individual pump at each pump station in Orleans Parish. The list is composed of information that was collected in the field. Not all information was available for each pump and was left blank or highlighted.

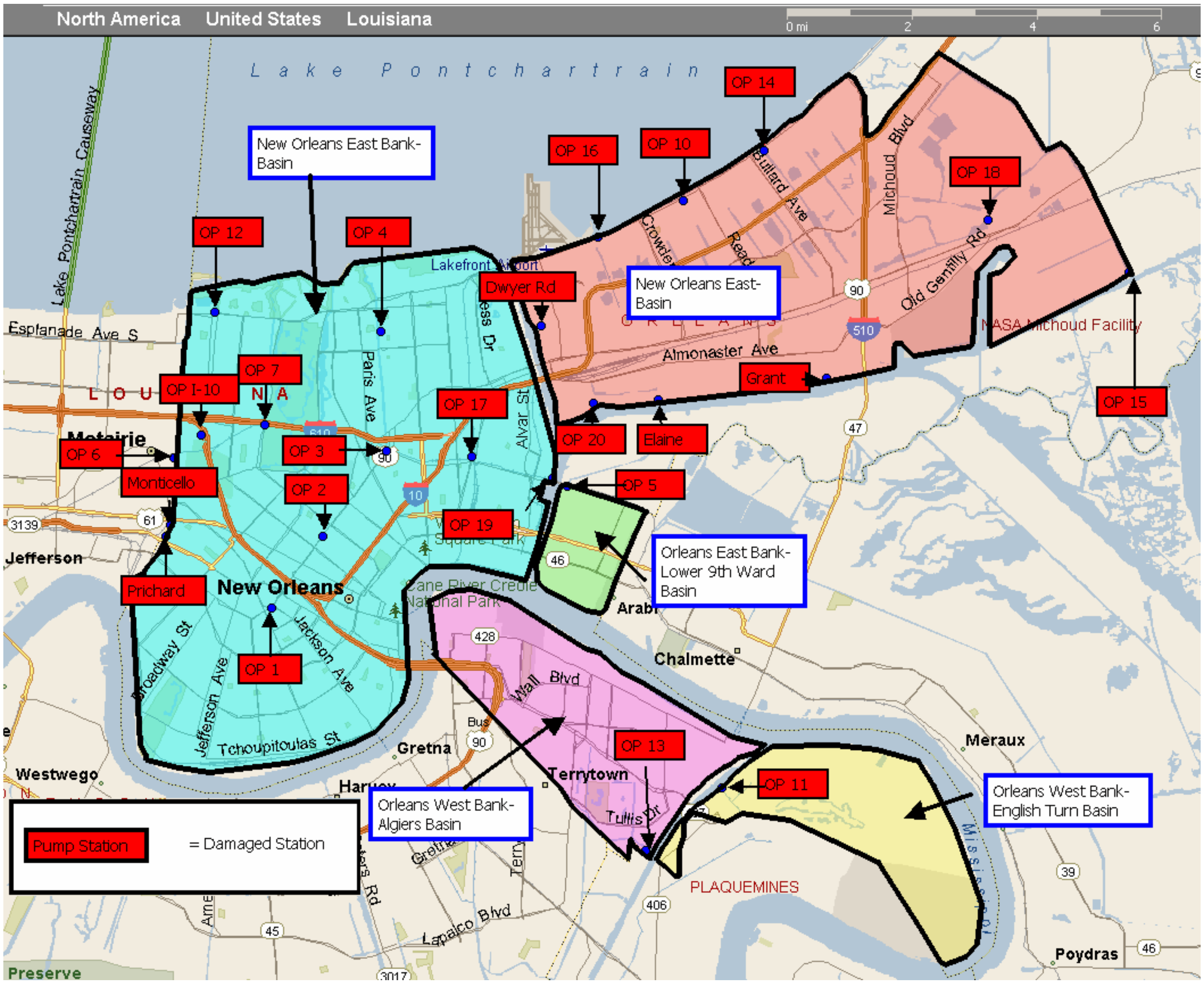


Figure 42. Orleans Parish Pump Station Locations

Table 46
Summary of Jefferson Parish Pump Stations by Drainage Basin

Basin	East Bank	Cataouatche	West Bank – West of Harvey	West Bank-East of Harvey	Total
Number of pump stations	6	4	9	3	22
Number of pumps	36	24	29	15	104
Total rated capacity (cfs)	20,662	3,346	10,695	9,958	44,661
Estimated cost of damages	$558,000	$3,000	$136,000	$61,000	$758,000

Table 47 **Summary of Orleans Parish Pump Stations by Drainage Basin**						
Basin	**East Bank**	**East**	**East Bank-Lower 9th Ward**	**West Bank-Algiers**	**West Bank-English Turn**	**Total**
Number of pump stations	12	9	1	1	1	24
Number of pumps	68	24	7	7	5	111
Total rated capacity (cfs)	36,615	4,852	1,850	4,700	1,690	49,707
Estimated cost of damages	n/a	n/a	n/a	n/a	n/a	n/a

Drainage Basins

West Bank – East of Harvey (Jefferson Parish)

The East of Harvey drainage basin on the West Bank has 3 significant pump stations. The basin is bordered b the Mississippi River on the north, and the Intracoastal Waterway on the southwest. The drainage system consists of the surrounding bodies of water, as well as the Planters Bypass and Hero Outfall Canals. The three pump stations are briefly described below. Volume VI provides more detailed descriptions.

Planters
Intake location: ....Planters Bypass Canal
Discharge location: .... Intercoastal Waterway
Nominal capacity: ....2356 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	288	n/a	Diesel	n/a
2	288	n/a	Diesel	n/a
3	288	n/a	Diesel	n/a
4	288	n/a	Diesel	n/a
5	52	n/a	Electric	n/a
6	288	n/a	Electric	n/a
7	288	n/a	Electric	n/a
8	288	n/a	Electric	n/a
9	288	n/a	Electric	n/a

Hero
Intake location: ....Hero Outfall Canal
Discharge location: .... Intracoastal Waterway
Nominal capacity: ....3852 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	100	n/a	Electric	n/a
2	300	n/a	Electric	n/a
3	300	n/a	Electric	n/a
4	1020	n/a	Diesel	n/a
5	1020	n/a	Diesel	n/a
6	300	n/a	Electric	n/a
7	203	n/a	Diesel	n/a
8	203	n/a	Diesel	n/a
9	203	n/a	Diesel	n/a
10	203	n/a	Diesel	n/a

Whitney Barataria
Intake location: ....n/a
Discharge location: .... Intercoastal Canal
Nominal capacity: ....3750 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	1250	n/a	Electric	n/a
2	1250	n/a	Electric	n/a
3	1250	n/a	Electric	n/a

West Bank – English Turn (Orleans Parish)

The West Bank – English Turn drainage basin is bordered by the Intracoastal Waterway on its northwest side. The Mississippi River wraps around its north and east sides. It only has one significant pump station, which is described below. Volume VI provides more detailed information.

OP 11
Intake location: .... Donner Canal
Discharge location: .... Intracoastal Waterway
Nominal capacity: ....1690 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
A	250	1953	Electric 25 Hz	Horizontal
B	250	1953	Electric 25 Hz	Horizontal
D	570	1990	Electric 60 Hz	Horizontal
E	570	1990	Electric 60 Hz	Horizontal
CD – 3C	50	1953	Electric 25 Hz	Centrifugal

West Bank – Algiers (Orleans Parish)

The West Bank – Algiers drainage basin is bordered by the Intracoastal Waterway on the southeast. The Mississippi River wraps around the west, north, and east sides. It only has one significant pump station, which is described below. Volume VI provides more detailed information.

OP 13
Intake location: .... Nolan and East Donner Canals
Discharge location: .... Intracoastal Waterway
Nominal capacity: ....4700 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
V1	250	1981	Electric 60 Hz	Vertical
V2	250	1981	Electric 60 Hz	Vertical
CD 3	50	1981	Electric 60 Hz	Vertical
D4	1000	1981	Diesel	Horizontal
D5	1000	1981	Diesel	Horizontal
6	1075	1981	Electric 60 Hz	Horizontal
7	1075	1981	Electric 60 Hz	Horizontal

3.2.3.7.6.5. Levees and floodwalls -

3.2.3.7.6.5.1. MRL - MRL levees and floodwalls are addressed in Paragraph 3.2.1.5.6.4.1, New Orleans East Bank MRL. There are no floodwalls that are part of the MRL Project in this reach.

3.2.3.7.6.5.2. Non Corps - Several Local Interest and/or private levees are located in the project area. No design criteria for these levees have been available to the Corps.

3.3 Post Katrina Changes to the Protection System

3.3.1. Documents available

Most documents which support and document the changes to the hurricane protection system can be found in the archives to the Task Force Guardian program. These may include decision documents, assessment documents and solicitation documents for the construction activities.

- Damage Survey Reports, (DSRs), were conducted early after the storm event to quickly describe the conditions to inform federal decision makers.

- Project Information Reports, (PIRs), were the authority reports developed to begin repair and construction activities.

- Construction Plans and Specifications were developed by A-E contractors after a solicitation was conducted by Contracting Division of TF Guardian or the New Orleans District Contracting Office.

This document solely captures the work being performed by Task Force Guardian. The New Orleans District has also performed work on federal and nonfederal components of the hurricane protection system in Jefferson, St. Charles, St. Bernard, and Plaquemines Parishes.

3.3.2. What Exists as of 1 June

3.3.2.1. Orleans East Bank

The Orleans East Bank portion of the program includes the east bank of the Mississippi River between the 17th Street Canal and Inner Harbor Navigational Canal (IHNC). Within these boundaries, project will address damages in the following areas:

- Orleans East Bank Lakefront – 5.2 miles of earthen levee segment located in New Orleans and roughly parallels the shoreline of Lake Pontchartrain between the IHNC on the east and 17th Street Canal on the west. The levee contains seven ramps that traverse the levee profile.

- 17th Street Outfall Canal – The 17th Street Outfall Canal lies in Jefferson Parish immediately west of the Orleans Parish boundary line. The canal extends approximately 2.4 miles from Pump Station No. 6 near Interstate Highway 10 to its confluence with Lake Pontchartrain.

- London Ave. Outfall Canal is located on the south side of Lake Pontchartrain in Orleans Parish, east of the 17th Street and Orleans Ave. Canals. The London Ave. Outfall Canal extends approximately 3.0 miles from Pump Station No. 3 to its confluence with the Lake Pontchartrain.

- Orleans Ave. Outfall Canal is located between 17th Street Outfall Canal and London Ave. Outfall Canal and extends approximately 1.8 miles from Pumping Station No. 7 in the vicinity of I-610 to its mouth at Lake Pontchartrain.

- Damage along IHNC is addressed in a separate section of this document.

Primary damages to the flood protection in the Orleans East Bank basin consists of a 455 foot breach in the east side I-wall along 17th Street Outfall Canal, breaches on both the east side (425 feet) and west side (720 feet) I-wall along London Ave. Outfall Canal, breaches along the west side of IHNC floodwall and damages to all fifteen pumping stations.

In the Orleans East Basin, ***twelve*** separate construction projects have been identified to repair the damaged areas, not including pump stations, and restore flood protection to pre-hurricane Katrina conditions. These projects represent an estimated $182M in construction costs.

Project OEB01 includes construction of a sheet pile cell around the breach area to facilitate replacement of the damaged section that will occur in Phase II of this repair work. This sheet pile wall will be offset 50 feet into the canal and tied into the existing wall to provide interim protection. The length of breach is 455 feet. Stone will be placed in the canal face of the sheet pile for channel stabilization. This will allow the New Orleans Sewerage and Water Board to operate Pump Station No. 6 at full capacity during normal rainfall events.

Project OEB02 includes the continuation and completion of the work begun in Phase I. The temporary breach repair will be removed and replaced with approximately 455 feet of reinforced concrete T-wall. This wall consists of a reinforced concrete base slab with a reinforced concrete wall extending up to elevation +14.0. This wall is supported by steel H-piles and a steel sheet pile cutoff wall embedded in the concrete wall.

Project OEB03 includes construction of a sheet pile cell around the breach area to facilitate replacement of the damaged section that will occur in Phase II of this repair work. This sheet pile wall will be offset 40 feet into the canal and tied into the existing wall to provide interim protection. The length of breach is 425 feet. Stone will be placed in the canal face of the sheet pile for channel stabilization. This protection will allow the New Orleans Sewerage and Water Board to operate Pump Station No. 3 at full capacity during normal rainfall events.

Project OEB04 includes the continuation and completion of the work begun in Phase I. The temporary breach repair will be removed and replaced with approximately 200 feet of reinforced concrete T-wall. This wall consists of a reinforced concrete base slab with a reinforced concrete wall extending up to elevation +14.0. This wall is supported by steel H-piles and a steel sheet pile cutoff wall embedded in the concrete wall.

Project OEB05 includes construction of a sheet pile cell around the breach area to facilitate replacement of the damaged section that will occur in Phase II of this repair work. This sheet pile wall will be offset 20 feet into the canal and tied into the existing wall to provide interim protection. The length of breach is 720 feet. Stone will be placed in the canal face of the sheet pile for channel stabilization. OEB 5 also contains an interim repair for damaged sheet pile wall along the east side of London Ave. Outfall Canal opposite the breach on the west side. This interim repair consists of driving a sheet pile cell to enclose the damaged floodwall.

Project OEB06 includes the continuation and completion of the work begun in Phase I. The temporary breach repair will be removed and replaced with approximately 720 feet of reinforced concrete T-wall. This wall consists of a reinforced concrete base slab with a reinforced concrete wall extending up to elevation +14.0. This wall is supported by steel H-piles and a steel sheet pile cutoff wall embedded in the concrete wall. OEB6 will include the replacement and repair of damaged floodwall along the east side of London Ave. Outfall Canal near Robert E. Lee Blvd. The existing I-wall is rotated several inches at the top for length of approximately 500 feet. Under this project, the wall will be replaced with reinforced concrete L-Wall section. This wall consists of a reinforced concrete base slab with a reinforced concrete wall extending up to elevation +14.0. This wall is supported by steel H-piles and a steel sheet pile cutoff wall embedded in the concrete wall.

Project OEB07 includes intermittent scour repair of 5.2 miles of earthen levee along the Lake Pontchartrain Lakefront. The bulk of the damage is lake side erosion, scour at the base of the floodwalls, and damaged slope paving. The scour repairs will require a small amount of borrow material.

Project OEB09 includes the construction of an interim gated flood control structure at the confluence of 17th Street Outfall Canal and Lake Pontchartrain. This steel structure will have a series of panel gates that will be open under normal conditions and closed during rising Lake Pontchartrain tide or impending tropical storm activity. The structure will include temporary pumping capacity of 3,000 cfs.

Project OEB010 includes the construction of an interim gated flood control structure at the confluence of London Ave. Outfall Canal and Lake Pontchartrain. This steel structure will have a series of panel gates that will be open under normal conditions and closed during rising Lake Pontchartrain tide or impending tropical storm activity. The structure will include temporary pumping capacity of 3,000 cfs.

Project OEB011 includes the construction of an interim gated flood control structure at the confluence of Orleans Ave. Outfall Canal and Lake Pontchartrain. This steel structure will have a series of panel gates that will be open under normal conditions and closed during rising Lake Pontchartrain tide or impending tropical storm activity. The structure will include temporary pumping capacity of 2,500 cfs.

Project OEB012 includes the construction of a levee tie in connecting the interim closure structure to the Lake Pontchartrain Hurricane Protection Levee along Lakeshore Dr. This levee tie-in is a combination of steel sheet pile and earthen levee with back side slope paving for scour protection.

Project OEB013 includes the construction of a levee tie in connecting the interim closure structure to the Lake Pontchartrain Hurricane Protection Levee along Lakeshore Dr. This levee tie-in is a combination of steel sheet pile and earthen levee with back side slope paving for scour protection.

3.3.2.2. IHNC

The Inner Harbor Navigation Canal (IHNC) portion of the program includes the flood protection paralleling the IHNC from the Mississippi River to Lake Pontchartrain.

The IHNC work area contains approximately 10 miles of levee and floodwalls along the Inner Harbor Navigation Canal in a heavily industrialized area.

Overtopping of the hurricane protection by Hurricane Katrina was evident along nearly all portions of the canal. There were four breaches in the protection system, two on the east side and two on the west side. The east side breaches are both located in the lower 9th ward neighborhood and the west side breaches are both in the vicinity of France Road and Benefit Street. Temporary repairs and closures were made in these areas until permanent restoration work is completed. This project under Task Force Guardian will restore the protection back to pre-hurricane Katrina conditions. In the areas of the breaches, this project will replace/repair those walls back to pre-storm project authorized elevations. In the areas of scour, those walls and scour will be repaired accordingly.

In the IHNC area, eight separate construction projects were identified to repair the damaged areas and restore flood protection to pre-hurricane Katrina conditions. These projects represent an estimated $62.7 million in construction costs.

Project IHNC-01 – There is approximately 4,000 lineal feet of concrete I-wall flood barrier along the east side of the IHNC between North Claiborne Avenue and Florida Avenue. The damages in this reach consisted of a breach of the floodwall immediately south of Florida Avenue (250’) and one approximately 100 yards north of Claiborne Ave (850’) with the remaining portions of the floodwall having areas of severe scour and tilting of the I-wall. The work includes replacement of the concrete I-wall with a concrete T-wall, supported on H-piles and sheet piling.

Project IHNC-02 – This section of the project consists of concrete I-wall. The damage in this area consisted of a breach of the floodwall at the container terminal along France Road. There was also heavy scour of the floodwall in this area. The repairs consist of removing approximately 1,300 lineal feet of the damaged concrete I-wall and replacing the damaged section of wall with new concrete L-wall. The new wall will be supported by steel H-piles and longer steel sheet piles.

Project IHNC-03 – There are approximately 2.75 miles of floodwall and levee along the east side of the IHNC between the Gulf Intracoastal Waterway and Lake Pontchartrain and another 1,000 lineal feet of floodwall on the west side of the IHNC between Almonaster Avenue and Highway 90. The damages in this reach consisted of intermittent scour of the levee and scour and damage at the wall/gate closures and at the wall/levee interfaces. The repairs consist of filling in the scour areas, repairing the gate concrete sills and seals, installing new sheet piling, placing rock and ballast, and placing stone erosion protection.

Project IHNC-04 – On the west side from Highway 90 to Lake Pontchartrain and on the east side between Dwyer Street and Hayne Boulevard flood protection consists of concrete I-wall that experienced relatively minor scour damage along its base. The repairs consist of filling in the scour areas, and cleaning existing and installing new relief wells.

Project IHNC-05 – This portion of the project consists of approximately 1,600 feet of existing levee and concrete floodwall that extends from the vicinity of France Road ramp towards the IHNC. This area was breached and experienced severe scour. The repair consists of replacement with a new concrete T-wall.

Project IHNC-07 – There is approximately 1,400 lineal feet of concrete I-wall flood barrier along the east side of the IHNC between the IHNC lock and North Claiborne Avenue. The damages along this reach consisted of intermittent scour along the base of the floodwall. The work includes filling in the scour repairs and providing erosion protection.

Project IHNC-08 – On the west side of the IHNC from 700 feet north of Benefit Street to Highway 90 flood protection consists of concrete I-wall embedded in compacted earthen levee embankment. The damages in this area consisted of scour along the base of the floodwall. The repairs consist of scour repair and erosion protection.

Project IHNC-09 – On the west side of the IHNC from the lock to Florida Avenue flood protection consists of concrete I-wall. The damages in this area consisted of scour along the base of the floodwall. The repairs consist of scour repair and erosion protection.

3.3.2.3. New Orleans East

The New Orleans East portion of the flood protection system is bounded by the east bank of the Inner Harbor Navigational Canal (IHNC), Lake Pontchartrain shoreline between the IHNC and Southpoint, the eastern boundary of the Bayou Savage National Wildlife Preserve, and the north side of the Gulf Intracoastal Waterway (GIWW) between the IHNC and eastern edge of the

Bayou Savage National Wildlife Preserve. Within these boundaries, project will address damages in the following areas:

- New Orleans East Lakefront includes the Citrus Lakefront Levee and New Orleans East Lakefront Levee consisting of 12.4 miles of earthen levee paralleling the Lakefront from the IHNC to Southpoint. It also includes floodwalls at the Lakefront Airport and Lincoln Beach.

- GIWW – The New Orleans East Basin includes the north bank of the GIWW flood protection system from the IHNC to the eastern edge of the Fish and Wildlife Preserve. The system contains the Citrus Back Levee and New Orleans East Back Levee which consisting of approximately17.5 miles of earthen levees and concrete floodwalls.

- The New Orleans East Levee consists of 8.4 miles of earthen levee from Southpoint to the GIWW along the eastern boundary of the Bayou Savage National Wildlife Preserve.

Primary damages to the flood protection in the New Orleans East basin consists of 12,750 feet of levee breach in the New Orleans East Back Levee between Michoud Canal and the CSX Railroad along the GIWW; A couple of floodwall breaches in this reach at Pump Station 15 (800 feet) near the Maxent Levee, and at the Air Products Hydrogen Plant near the Michoud Canal (300 feet); floodgate floodwall and adjacent levee damage at the CSX railroad; and 2000 feet of floodwall damage in the Citrus Back Levee along the GIWW between the IHNC and Paris Road. The other damages consist mostly of levee and floodwall scour at various locations throughout the New Orleans East Basin and damages to all of the eight pump stations.

In the New Orleans East Basin, ten separate construction projects have been identified to repair the damaged areas and restore flood protection to pre-hurricane Katrina conditions. These projects represent an estimated $83 M (not including pump stations) in construction costs.

NOE 01 – Project NOE01 consists of rebuilding approximately 5 miles of the existing levee up to elevation 19.5 with 680,000 cubic yards of earthen material, then seeding and fertilizing. The entire reach of levee was brought up to an interim level of protection of elevation +10 by November 15, 2005.

NOE 02 – Project NOE02 includes removing the damaged steel sheet pile wall, installing a new concrete T-wall, filling in scour holes and bringing the damaged levee back up to pre-hurricane Katrina elevation.

NOE 03 – Project NOE03 includes removing the damaged concrete I-wall and steel sheet pile wall, filling in scour holes, installing new sheetpile and raising the damaged levee to pre-hurricane Katrina elevation and then seeding and fertilizing. The damaged reach was first brought up to an interim level of protection of elevation +10 by November 15, 2005 before final repairs are made.

NOE 04 – Project NOE04 includes removing the damaged concrete I-wall sections, filling in the scour holes, regrading the damaged levee, constructing new concrete L-wall, and putting in slope paving and an earthen stability berm on the landside of the wall. The repaired levee section and stability berm will be seeded and fertilized. The damaged reach was first brought up to an interim level of protection of elevation +10 by December 1, 2005 before final repairs are made.

NOE 05 – Project NOE05 includes the removal of the existing concrete wall and railroad closure gate, filling the scoured areas, constructing a new closure gate and new concrete T-walls and I-walls, placement of rip rap, concrete slope paving and concrete roadway.

NOE 06 – Project NOE06 consists of filling in the scour holes and placing a concrete pavement section next to the concrete wall. It also includes filling in the scour hole and paving the damaged road section with concrete at the interface of the Floodgate L-15 concrete wall and levee.

NOE 07 – Project NOE07 includes intermittent scour repair along approximately 19 miles of earthen levee along the Lake Pontchartrain Lakefront and the eastern boundary of the Bayou Savage National Wildlife Preserve. The work consists of filling in the scour areas with semi-compacted fill, reshaping where needed, and seeding and fertilizing.

NOE 08 – Project NOE08 includes filling in the scour holes and capping with gabion structures around several gated drainage control structures to prevent future erosion. The gabion structures are wire baskets filled with stone interlocked to form a surface erosion barrier.

NOE 09 – Project NOE09 includes filling in the scour holes next to the existing concrete I-wall floodwall with embankment material, installing bedding material, grouted riprap, and concrete slope paving above the scour to prevent future erosion. Also includes adding an earthen stability berm on both flood and protected sides of the wall. The project also consists of intermittent repairs to damaged concrete and various joints and gates in the walls, and the installation of relief wells and sheetpile in selected areas.

NOE 10 – Project NOE10 includes filling in the scour holes next to the floodwalls with embankment material, installing bedding material, and concrete slope paving above the scour to prevent future erosion. These walls are around pump stations and utility lines along Lakefront and New Orleans East Levee systems.

3.3.2.4. St. Bernard Parish

The St. Bernard Basin hurricane protection system includes the levee/floodwall extending from the Inner Harbor Navigation Channel (IHNC) easterly, along the Gulf Intracoastal Waterway (GIWW), to the Bayou Bienvenue Control Structure, continuing along the Mississippi River Gulf Outlet (MRGO) southeasterly, then turns generally to the west, where it ties into the Mississippi River Levee at Caernarvon. A portion of the hurricane protection system in this area also provides hurricane protection to the Lower 9th Ward area in Orleans Parish. Within this area of protection, the Task Force Guardian authorities will address damages to the following project features:

- 8 miles of the 30 total miles of hurricane protection levee were damaged:
    - Most severely damaged levees are along the reach adjacent to the MRGO extending from the Bayou Bienvenue Control Structure to the southeast for 11.8 miles
    - Minor levee scour along GIWW in Orleans Parish
    - Miscellaneous scour on the levee from MRGO to Caernarvon
- Repair of Bayou Dupre Control Structure
- Repair of Bayou Bienvenue Control Structure
- Repair of 5 floodgates, floodwall, and minor levee damages from Bienvenue Control Structure to GIWW lock
- Repair Creedmore Structure

The New Orleans District has performed work on the nonfederal St. Bernard back levee.

In the St. Bernard Parish, nine separate construction projects were identified to repair damaged areas and restore flood protection to pre-hurricane Katrina conditions. These projects represent an estimated $50.2 million in construction costs.

STB 01 – The work for this project included site preparation work in the areas of levee damage between the Bayou Bienvenue and Bayou Dupre Control Structures. The contracted work (rental agreement contract) is complete.

STB 02 – The work for this project included site preparation work in the borrow areas between the Bayou Bienvenue and Bayou Dupre Control Structures. The borrow area is a strip of land adjacent to the levee, which was used as a disposal area during the construction of the MRGO canal. This rental agreement contract is complete.

STB 03 – The 5.6-mile reach of levee along the MRGO extending east from the Bayou Dupre Control Structure was severely damaged from overtopping. The entire levee reach is being restored to the design grade elevation, requiring the placement of an estimated 800,000 cubic yards of fill material. The borrow area for this fill material is a strip of land adjacent to the levee, which was used as a disposal area during the construction of the MRGO canal. Protection is restored when the levee reaches elevation 17.5.

STB 04 – The 6.2-mile reach of levee along the MRGO between the Bayou Bienvenue and the Bayou Dupre Control Structures was also severely damaged from overtopping. The entire levee reach is being restored to the design grade elevation, requiring the placement of an estimated 1,350,000 cubic yards of fill material. The borrow area for this fill material is a strip of land adjacent to the levee, which was used as a disposal area during the construction of the MRGO canal. Clay material is also being barged into the site to supplement to on-site borrow. Protection is restored when the levee reaches elevation 17.5.

STB 05 – Minor scour repairs are needed on the backside of the levee and structural and structural backfill scour adjacent to floodwalls and four closure structures, which are located between the Bayou Bienvenue Control Structure and the Florida Avenue Bridge, with most of the levee reach adjacent to the GIWW.. An estimated 26,000 cubic yards of fill material are required for this work, which are being furnished by the contactor.

STB 06 – The work on this project includes repair of structural damage and loss of structural backfill at the Bayou Dupre Control Structure. A significant scour hole is to be filled with 17,500 cubic yards of granular backfill and protected with grouted riprap. An estimated 22,500 tons of riprap and 13,400 cubic yards of embankment fill are required for the repairs.

STB 07 – The work on this project includes repair of structural damage and loss of structural backfill at the Bayou Bienvenue Control Structure. A significant scour hole is being filled with 28,600 cubic yards of granular backfill and protected with grouted riprap. An estimated 32,100 tons of riprap and 3,400 cubic yards of embankment fill is required for the repairs.

STB 08 – The work includes repair of minor scour on the backside of the levee from the Mississippi River Gulf Outlet (MRGO) to Caernarvon, which is about 10.8 miles in length. An estimated 36,000 cubic yards of fill material are required for this work.

STB 09 – The work includes constructing a cofferdam and removing debris from the structure to permit closure of the gates and inspection of the structure to determine if further repairs are necessary. Contract work is complete.

3.3.2.5. Plaquemines Parish

The Plaquemines Parish Basin includes long, narrow strips of protected land on both sides of the Mississippi River between New Orleans and the Gulf of Mexico. The Mississippi River Levees (MRL) protect the Parish from river flooding. Protection from hurricane-induced tidal surge is achieved by the New Orleans to Venice (NOV) hurricane protection system. The NOV is a system of levees on the gulf side of the protected lands. Additional berms and floodwalls are constructed on top of the MRL in the lower part of the Parish, where hurricane-induced water levels are higher than river flood stages. The distance between the gulf-side levees (back levees), and the MRL is less than a mile in most places.

Altogether the Plaquemines Parish MRL and NOV systems include 162 miles of levee and 7 miles of floodwall. There are fifteen non-federal pump stations for interior drainage. The levees are crossed by numerous oil pipelines, constructed in various manners. Some crossings bridge the levee without touching the embankment; some are constructed on top of the line of protection; and some pass through the line of protection with measures to prevent seepage. There is a wicket gate closure on the back levee at Empire, where a shipping canal connects the Mississippi River to the Gulf of Mexico.

- The Plaquemines Parish East Bank MRL system extends from the Parish line at Braithwaite thirty-five miles downstream to Bohemia. The flood-side slopes have concrete slope pavement from the bottom of the embankment to the design high water level. The crown is surfaced with 9 inches of crushed limestone. The freeboard and protected-side slopes are grassed.

- The east bank NOV back levee runs between Phoenix and Bohemia, a distance of sixteen miles. It is a grass-covered earthen levee.

- The West Bank Plaquemines MRL system extends from the parish line at Belle Chasse, seventy miles downstream to Venice. Its composition is similar to the East Bank MRL with concrete slope pavement, crushed limestone surface course, and the remaining slopes grassed. Below Port Sulphur (twenty-nine miles above Venice) the MRL design grade is lower than the NOV hurricane design grade, so the NOV is constructed as berms or floodwalls on top of the MRL.

- The west bank NOV extends from St. Jude to Venice, a distance of thirty-six miles. The NOV protection along the river includes six miles of floodwalls in thirteen distinct reaches, projecting above the MRL from two to eight feet. The back levee is a grass covered earthen embankment.

All of the levees in Plaquemines Parish sustained damage from Hurricanes Katrina and Rita. There was considerable crown and slope scour along the total length. The MRL slope pavement sustained damage from the hundreds of ships and barges that crashed upon it. There were also several severe breaches, coinciding with pipeline crossings and with some floodwalls. Five of the six miles of NOV floodwall along the Mississippi River were damaged beyond repair. There were major breaches at sheet pile wing walls at two pump stations in the back levee. A major breach occurred at the Shell pipeline crossing near Nairn. And the West Pointe a la Hache pipeline crossing was severely damaged. Wind and water damage from Katrina and Rita severely impacted nearly every structure within the east bank area of protection and on the west bank below Myrtle Grove (50 miles above Venice).

New Orleans District has performed repair work on the nonfederal levees in Plaquemines Parish.

Task Force Guardian has divided the Plaquemines Parish flood protection recovery process into twenty-two projects, labeled P1 through P26, with no projects for P5, P9, P10, or P23 (the work of the missing projects was combined with the other 22):

P01 – This project consisted of preparing the borrow area, at the southern end of the project, for use by levee-construction contractors. The work involved clearing and burning vegetation from 40 acres.

P02 – Walker Road borrow is used for most of the MRL projects because of its high quality material and the speed which it could be brought into production. This work was performed by the Memphis District Corps of Engineers. Altogether, more than 390,000 cubic yards of fill were excavated and processed by the time the MRL projects were completed in March 2006.

P03 – This project was for repair of the Gravolet Canal breach near Bohemia, Louisiana. The 15-foot-deep by 550-foot-wide breach was temporarily closed under Task Force Unwatering, and some of that temporary repair has been incorporated into this permanent project. The work consisted of preparing the borrow area, excavation, and reconstructing the levee by placing fill and armor stone, and restoring the surface by fertilizing and seeding.

P04 – This project, for cleaning and repairing levee scour sites, was executed by the Memphis District Corps of Engineers Revetment Unit.

P06 – This project was executed under a rental contract for equipment and personnel to repair scour sites along a twenty mile reach of river levee. The work included hauling, placing and compacting fill and crushed limestone surfacing, as well as fertilizing and seeding disturbed areas. The project used government-furnished borrow from Walker Road borrow pit (Project P02).

P07 – This project was executed under a rental contract for furnishing equipment and personnel to repair scours along a five mile reach of river levee. The work included hauling, placing and compacting fill and crushed limestone surfacing, as well as fertilizing and seeding disturbed areas. The project used government-furnished borrow from Walker Road borrow pit (Project P02).

P08 – This project was for repair of slope and crown scour along the twenty-mile reach from Port Sulphur to Fort Jackson. Approximately 350,000 cubic yards of government-furnished material from the Walker Road and Buras borrow pits was required. The contract included demolition of the Buras floodwall, which was required before the embankment could be repaired.

P11 – Project P11, for shaping the MRL along the 10-mile reach from Fort Jackson to Venice, was completed by New Orleans District and Memphis District hired labor. They used bulldozers to patch and compact scour sites with material recovered from the toe of the levee and borrowed from the damaged NOV section on top of the MRL. The NOV will be repaired under Project P12.

P12 – This project will reconstruct the NOV hurricane protection levee on the same 10-mile reach that P11 addresses on the MRL. The NOV design grade in this reach is between 2.6 and 3.5 feet higher than the MRL. The project requires 210,000 cubic yards of fill material from new borrow pits in Buras and Triumph.

P13 – This project will replace damaged NOV hurricane protection floodwalls with levees constructed above and behind the MRL levee. The NOV design grade in this reach is between 1.7 and 2.1 feet higher than the MRL. The project requires approximately 145,000 cubic yards of fill from a new borrow pit in Port Sulphur.

P14 – This project will replace damaged NOV hurricane protection floodwalls with levees constructed above and behind the MRL levee between Empire and Buras. The NOV design grade in this reach is between 2.9 and 3.1 feet higher than the MRL. The enlargement requires approximately 600,000 cubic yards of fill.

P15 – This project is for repairs to the Empire Canal flood gate. The gate is currently inoperable and in need of structural, mechanical, and electrical repairs. The structure must be de-watered to accomplish the construction.

P16 – This project is for repair of crown and slope scour that the 10-mile reach, B2, incurred from Hurricane Katrina. This lowest reach of the back levees, from Fort Jackson to Venice, was covered with marsh grass, debris and boats when the flood waters receded but the levee was not severely damaged and there were no breaches.

P17 – This project will replace damaged NOV hurricane protection floodwalls with levees near Buras. The NOV design grade in this reach is 3.1 feet higher than the MRL. The project will utilize approximately 310,000 cubic yards of fill from a new borrow pit in Triumph.

P18 – This project was for repair of levee crown and slope scour along the 11-mile-long B-1 Reach, from Empire to Fort Jackson. The levee was not severely damaged in this reach, so overall quantities were small; only 14,000 cubic yards of fill were required for the entire project.

P19 – This project is for repair of crown and slope scour along the 18-mile-long Reach A, from City Price to Empire. This levee sustained significant damage at several places, including severe crown scour, breaches, and wall tie-in failures. All together more than 300,000 cubic yards of fill material are required.

P20 – This project will repair floodwalls at Sunrise and Hayes pump stations. Emergency sheet pile walls were constructed at Sunrise during Task Force Unwatering, to achieve closure of a deep, wide breach. This project will make those repairs permanent and will construct additional sheet pile wing walls at the Hayes pump station.

P21 – This project will make repairs to floodwalls at Freeport, Home Place Marina, Gainard Woods Pump Station and Diamond Pump Station. Floodwalls are sheet pile I-walls; some are capped with reinforced concrete. Improvements include replacing I-walls with T-walls or embankment, or adding fill to reduce the height of wall stick-up.

P22 – This project was for repair of a levee breach at Woodland, on the west bank of the Mississippi River. The rental contractor was moved from a completed project in St. Bernard Parish in order to complete the repair before December 1.

P24 – This project will replace damaged NOV hurricane protection floodwalls with levees. The NOV design grade in this reach is between 1.7 and 2.1 feet higher than the MRL. The project will require approximately 582,000 cubic yards of fill from a new borrow pit at Myrtle Grove. This borrow pit will eventually be incorporated into the planned Mississippi River Diversion Channel at Myrtle Grove.

P25 – This project was to reconstruct the levee section, extend sheet pile cutoff walls thirty feet upstream and downstream of the siphon, and replace slope pavement on both sides and on the crown of the levee.

P26 – This project is for removing damaged sections of the reinforced concrete slope pavement and filling the holes with riprap to protect the levee. The riprap repairs are an interim remedy, designed to last for several years until concrete sections can be re-cast.

3.3.2.6. St. Charles Parish

No Post Katrina hurricane restoration work has been performed by Task Force Guardian. Construction work has been performed by the New Orleans District.

3.3.2.7. Jefferson East Bank

No Post Katrina hurricane restoration work has been performed by Task Force Guardian. Work has been performed by the New Orleans District.

3.3.2.8. West Bank and Vicinity

No Post Katrina hurricane restoration work has been performed by Task Force Guardian. Construction activities continue under the direction of the New Orleans District.

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