US Army Corps of Engineers®

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

Final Report of the Interagency Performance Evaluation Task Force

Volume III – The Hurricane Protection System

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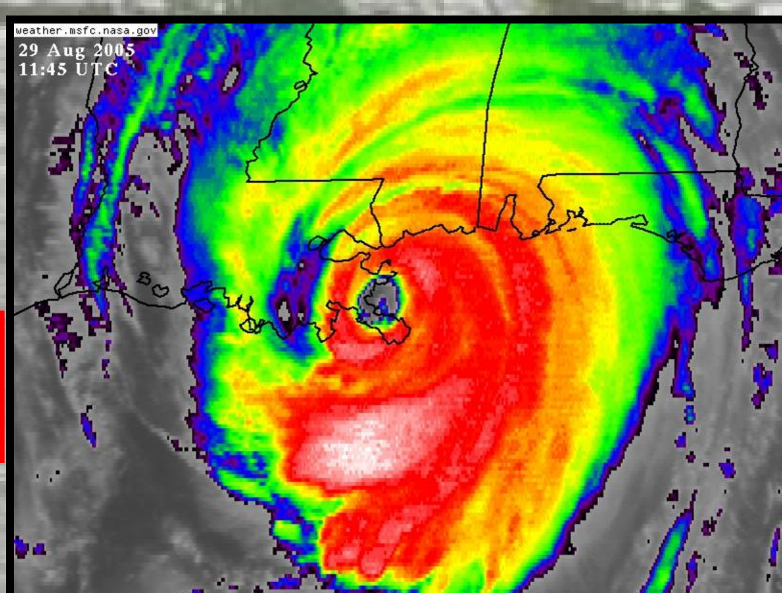

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3.1 Executive Summary

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 multidisciplinary 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 4,000 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:

> **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 as shown in Figure 1.

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 projects, 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 projects and a listing of all references used for the volume. An appendix contains a detailed history of the hurricane protection 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 (now the National Weather Service); 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 projects 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 with 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 ft were added to the wind tide level to get the design elevation.

The hurricane protection projects consist 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 cutoff. At I-wall/T-wall monolith joints, a fabricated sheet-pile section that allows independent movement of the two types of walls is used. The I-walls were not designed for a water-filled gap forming at the base of the wall on the canal side as the water level rises. In addition, levees and I-walls were not designed to prevent erosion upon being overtopped. IPET Volume V contains extensive information on the various types of levees and floodwalls used on the hurricane protection projects and their performance during Hurricane Katrina.

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 with 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 available at the time of design, as modified by the Corps of Engineers more conservative criteria for hydraulic structures. Some variations did occur among 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.

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. Many of the pump stations were not designed for hurricane loads as experienced during Hurricane Katrina. Interior drainage and pumping is evaluated in considerable depth in IPET Volume VI.

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.

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 in./hr.

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

For design of pumping stations in the New Orleans area, each of the four parishes is divided into drainage basins. The basins usually follow natural topographical lines. They are typically 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. The pump stations were not intended to remove water resulting from overtopping or breaching. 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. Consequently 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 projects 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. While there would have been local flooding, these stations would have performed as designed during Katrina to dewater their respective drainage basins had the hurricane protection system not failed.

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 and sent to offsite storage where they remain for 6 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; tests performed that day, as required in the plans and specifications; materials received, submittals reviewed, offsite surveillance activities, job safety, environmental protection, and general remarks.

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 QC inspection phases attended, instructions given, 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; controversial matters that may have arisen; information, instructions, or actions taken not covered in QC reports or disagreements; safety; and remarks.

The construction documents that were reviewed by IPET 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 are 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 under the periodic inspection paragraphs of each project in this volume. 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 hurricane protection projects 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 projects were at least acceptable, and some were 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 occasionally address status of completion for specific hurricane reaches. More information on these inspections is included under the annual compliance inspection paragraphs of each project within this volume..

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 1,500 ft apart and were usually 50 to 80 ft deep with a few extending to a depth of 100 ft. Generally, the borings were taken at 350 to 650 feet apart in the areas where floodwalls were to be constructed and 700- to 1,500-ft 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 confirmed during construction 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. The first instance is described on page III-132, 17th Street Canal East Side Stations 0+96.27 to Station 7+00. About 4 ft 3 in. of sheet pile were cut off because of unanticipated hard driving. The second is described on page III-134, 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 on page III-196, New Orleans East, South Point to Gulf Intracoastal Waterway (GIWW). During construction of a floodside berm, the berm 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, described on page III-299, St. Charles Parish North of Airline Highway, required modifications to remove pile driving obstructions. The fifth occurrence is described on pages III-271 through III-272, Jefferson Parish Lakefront Levee Pump Station No. 2, where, because of a survey error, the breakwater was realigned by moving it 70 ft 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 miss some areas of weaker soils.

Hurricane Protection System Findings

The system was generally built as designed, with the exceptions noted below, using design approaches that were consistent with industry and local practices at the time of design. Due to an inaccurate relationship between geodetic datum and mean sea level much of the system was built below specified design elevations. The hurricane protection projects have also subsided as a result of regional subsidence. 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 without local flooding. The lack of a CSX closure gate due to being damaged prior to Katrina also 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. The I-walls were not designed for a water-filled gap forming at the base of the wall on the canal side as the water level rises. Levees and I-walls were not designed to prevent erosion upon being overtopped. In addition, risk was not well understood or communicated effectively to the public.

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.

1. System Based Approach - Hurricane protection systems must have a unified, integrated, comprehensive system based approach. Planning, design, construction, operation and maintenance should be based on a system-wide understanding of the processes affecting the system and the interaction and interdependencies of the system components. Variations in the system levee and floodwall heights can compromise system performance. Special performance consideration must be given where transitions occur between features and components that comprise the system. Consistent operation and maintenance standards must be employed throughout the system to sustain a reliable system. A comprehensive integrated water control plan for the system that is effectively coordinated and managed is imperative to the success of the system. Construction phasing and scheduling must take a comprehensive systems based approach to ensure public safety at all times. Redundancy and resilience must be considered throughout the system allowing for uncertainty and the interdependency of the system reaches and features. The system based approach must provide a balanced evaluation of all relevant issues integrating hurricane protection, land use, and emergency response. Employing an integrated, comprehensive system based approach will result in safer, more reliable projects working together as a system with increased economic and environmental benefits.

2. Risk-Based Concepts - Risk-based concepts employed throughout the planning, design, construction, operations and maintenance phases of a hurricane protection system will provide knowledge of what can go wrong, how likely is it to occur, and what are the consequences to enable risk-informed decisions. Risk and reliability assessments also provide a unified way for on-going evaluation and reporting of system conditions. Through a unified system based risk assessment, using formal analytical quantitative risk and reliability evaluations, implications of tradeoffs from technical decisions can more effectively be made. Hazard recognition from future storms will be enhanced by more accurately predicting recurrence rates and magnitudes of water levels and forces associated with hurricanes. Including failure modes that resulted from Hurricane Katrina in the risk based methods will enhance the effectiveness of the assessments including understanding the resiliency of the components and features. Risk associated with flooding caused by storms must be communicated in ways stakeholders and the public can understand. Especially interim risk while the system is under construction, and residual risk which is the portion of total risk that will always remain after avoiding, preventing, mitigating, and assuming all other risk. Risk based methods will improve assessment of the cumulative impacts of incremental decisions on residual risk throughout the system life cycle. Risked based concepts will enable an appropriately balanced systems based approach to risk management and integrating hurricane protection, land use, and emergency response in providing public safety.

3. Policy, Guidance, and Standards - Policy for program development, planning guidance, and design and construction standards must be continuously reassessed, updated, and reviewed for applicability throughout the system life cycle. Engineering models to accurately predict the water levels and forces associated with hurricanes is essential and those models must be maintained through the life cycle of the system. Policy and guidance must clearly address the importance of resilient engineering components in providing safe and reliable hurricane protection systems. In addition, policy and guidance must clearly articulate the need for redundancy in critical components of the system. Risk based policy and guidance will enable more informed decisions on the part of public officials to reduce risk and a more inform public so that personal decisions are based on the best knowledge available. Policy and guidance must be updated to include failure modes that resulted from Hurricane Katrina and were confirmed by the IPET analysis and tests. As knowledge and information changes planning and engineering guidance reflecting state-of-the-art technology and methods must be continuously updated and comprehensively applied to water resource systems. Safe and reliable hurricane protection systems are achievable through up-to-date policy, guidance, and standards that are implemented by capable and competent professionals using state-of-the-art technology and methods.

4. Dynamic Independent Review - Dynamic independent reviews throughout the system life cycle will result in safe, reliable systems with greater economic and environmental benefits. Planning, design, construction, operation and maintenance need periodic review to determine whether they represent best practice and knowledge. Dynamic independent reviews that emphasize temporal and spatial incremental changes occurring over decades in a system, as well as changing knowledge, will effectively contribute to assuring public safety remains paramount. Cross functional and discipline dynamic independent reviews that evaluate the influence of incremental decision-making will help assure public safety and environmental benefits. Dynamic independent review requires more then an individual project or component perspective. The independent review must include the complete group of projects, features, components, environment, policies, plans, and practices that collectively in a unified way serve a common purpose.

5. Adaptive Systems - Employing adaptive planning and engineering approaches will build flexibility into the system to respond to future dynamic conditions and nonlinear processes of nature that can place sudden demands on the system. Planning, design, operation and maintenance for hurricane protection systems need to include consideration of resilience, adaptation, and redundancy to accommodate unanticipated conditions or structural behaviors. There are inherent uncertainties with information particularly hydrologic, geologic, and environmental. Recognizing system uncertainties then finding ways to achieve system objectives within the band of uncertainties will avoid inadvertent mistakes that could lead to irreversible unsatisfactory results. Adaptive management recognizes knowledge about future conditions is uncertain and enables deliberate planning considering uncertainty. The lack of adaptive management contributes to crisis planning reacting to uncertainty.

6. Sustainability - Effective integration of Environmental Operating Principles and asset management will result in a sustainable environment with reliable assets. The environment is an integral part of the system. The infrastructure must be managed in an environmentally sustainable manner through explicit risk management. Sustainability is critical for infrastructure where failure would have major consequences. Comprehensive system asset management enables prioritization of investments throughout the life cycle of the system. Sustainability is enhanced through integrated engineering and ecosystem analyses considering a system approach. A cyclic model approach to plan, design, and construct, then monitoring and evaluating system performance over time enables appropriate modification of operation and maintenance to achieve a sustainable system. With the environment an integral component of the system, and asset management used to evaluate the functional reliability of the system infrastructure, safe and sustainable systems will evolve.

7. Inspections and Assessments - Comprehensive inspections and risk assessments of completed works will provide knowledge of the condition and reliability of the system to perform.

Continuous technology transfer from research and development activities will help to ensure assessments of completed works are relevant and utilize current knowledge. Completed works must be designed, operated, and maintained with flexibility to adapt as new information and knowledge becomes available throughout the hurricane protection system life cycle. Completed works must be sustainable and resilient to provide public safety. Effective inspections and assessments utilize monitoring and instrumentation information throughout the life cycle of the system to understand the effects of incremental changes. Completed works must be constructed, operated, and maintained in accordance with the design intent. Comprehensive inspections and assessments will result in more reliable and safer systems that are adaptable and sustainable.

8. Research and Development – Research and development directed towards an increased understanding of the forces acting on the hurricane protection system and the response of the system to those forces is vital. A greater understanding of effective ways to achieve adaptability, sustainability, resiliency, and redundancy throughout the system is possible through research and development. Research and development focused on enhancing the current state of applied risk and reliability methodology will improve public safety and the nation's flood and coastal storm damage reduction systems. Research and development is essential to advancing the understanding of the physics of breaching mechanisms in levees, floodwalls, and shore protection. Advancing the understanding of the physics of creating fragility curves to model the risk and reliability effects of surge, waves, and overtopping is possible through research and development. Technology transfer from research and development to implementation is effectively performed through the development of training, tools, guidance, procedures, policy, and behavior, in collaboration with those planning, designing and operating the system.

Participants

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

Name	Organization
Carol Burdine	U.S. Army Corps of Engineers
Charles Camillo	U.S. Army Corps of Engineers
Shung Chiu	U.S. Army Corps of Engineers
Larry A. Cooley	Burns, Cooley, Dennis
Angela Desoto-Duncan	U.S. Army Corps of Engineers
Steve Fitzgerald	Harris County, TX, Flood Control District
Robert L. Fleming	Burns, Cooley, Dennis
Bob Howard	South Florida Water Management District

Name	Organization
Mark Huber	U.S. Army Corps of Engineers
Joseph Jacobazzi	U.S. Army Corps of Engineers (on loan from the Metropolitan Water Reclamation District of Greater Chicago)
John Jaeger	U.S. Army Corps of Engineers
Reuben Mabry	U.S. Army Corps of Engineers
Denise Martin	U.S. Army Corps of Engineers
James Morgan	Burns, Cooley, Dennis
Alfred Naomi	U.S. Army Corps of Engineers
Nancy Powell	U.S. Army Corps of Engineers
James A. Young	U.S. Army Corps of Engineers

3.2 The Hurricane Protection System

Southeast 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 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 (MRC) in 1879 and the Mississippi River and Tributaries (MR&T) project in 1928.

Originally situated on the relatively high ground near the river, the city continued to grow and expand through the 20th century. Marshland 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 in. 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. Interior drainage performance following Hurricane Katrina is evaluated in Volume VI.

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

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 percent of the continental United States. The river stage at New Orleans peaked at 21.27 ft in 1922 and dropped to its lowest at minus 1.60 ft in 1872. The river's average elevation at the Carrollton gauge in New Orleans is about 6.8 ft MSL (mean sea level).

Lake Pontchartrain covers about 630 square miles with depths of mostly 12 to 14 ft. 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 ft 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, and Vicinity; New Orleans to Venice, LA; and West Bank and 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 percent of the land area of Louisiana, but includes 24 percent 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 Louisiana	5%	24%	24%	26%

The existing hurricane protection projects are shown in Figure 1. 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 SPH 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, National Oceanic and Atmospheric Administration (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 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 percent 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 percent) 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 Institute for Water Resources (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 were 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 the HEC-2 Water Surface Profile computer program. The design flow line 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 the Inner Harbor Navigation Canal (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, 1, 2, or 3 ft 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 (DMs) 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 ft 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, nearshore 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 ft MSL and beneath lacustrine or prodelta deposits in some areas as deep as elevation -30 to -35 ft MSL. The thickness of the buried beach typically ranges from about 10 ft to 30 or 40 ft, 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.

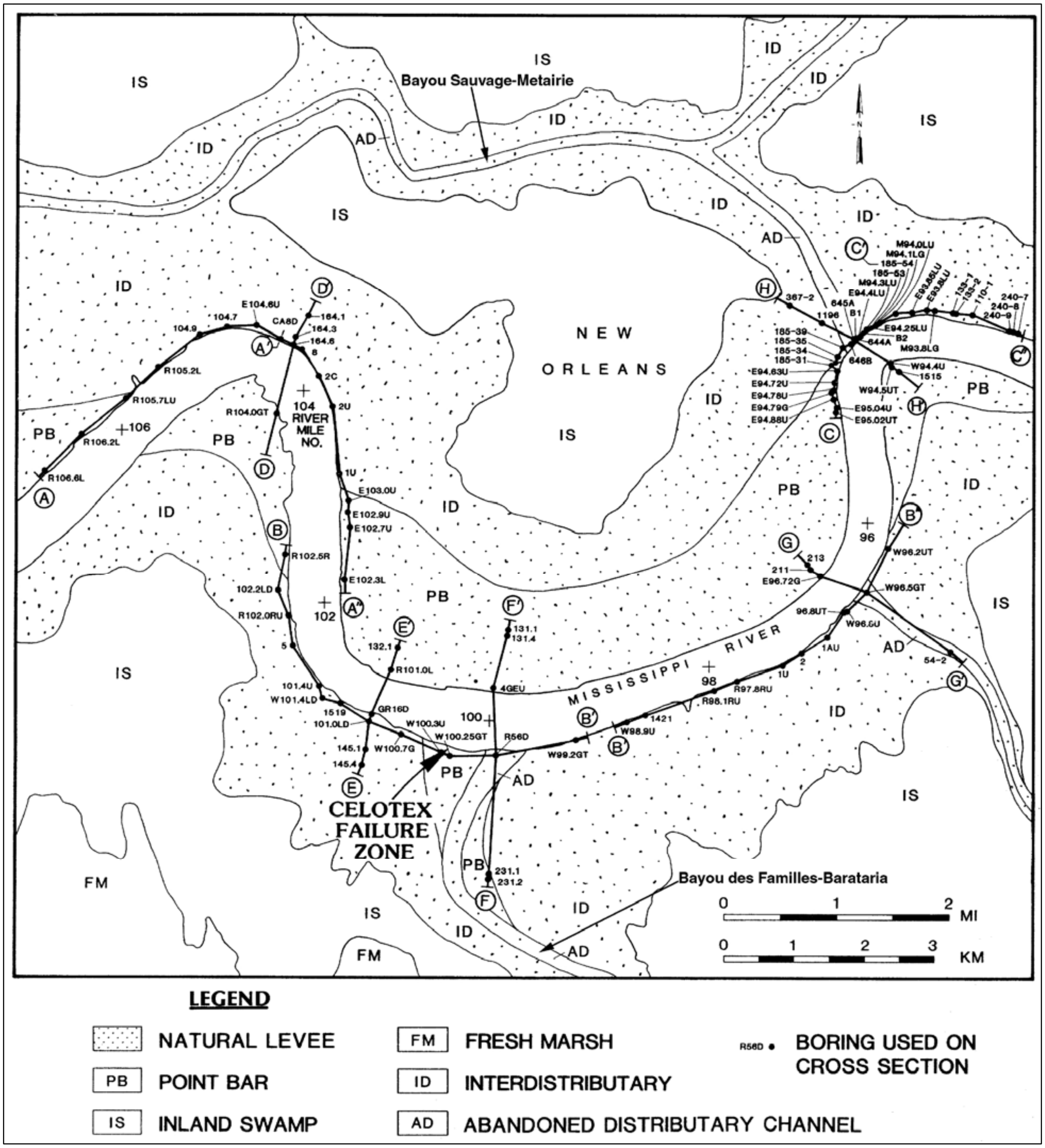


Figure 2. Surface geology in the general vicinity of New Orleans (from Figure 2-4, Volume V).

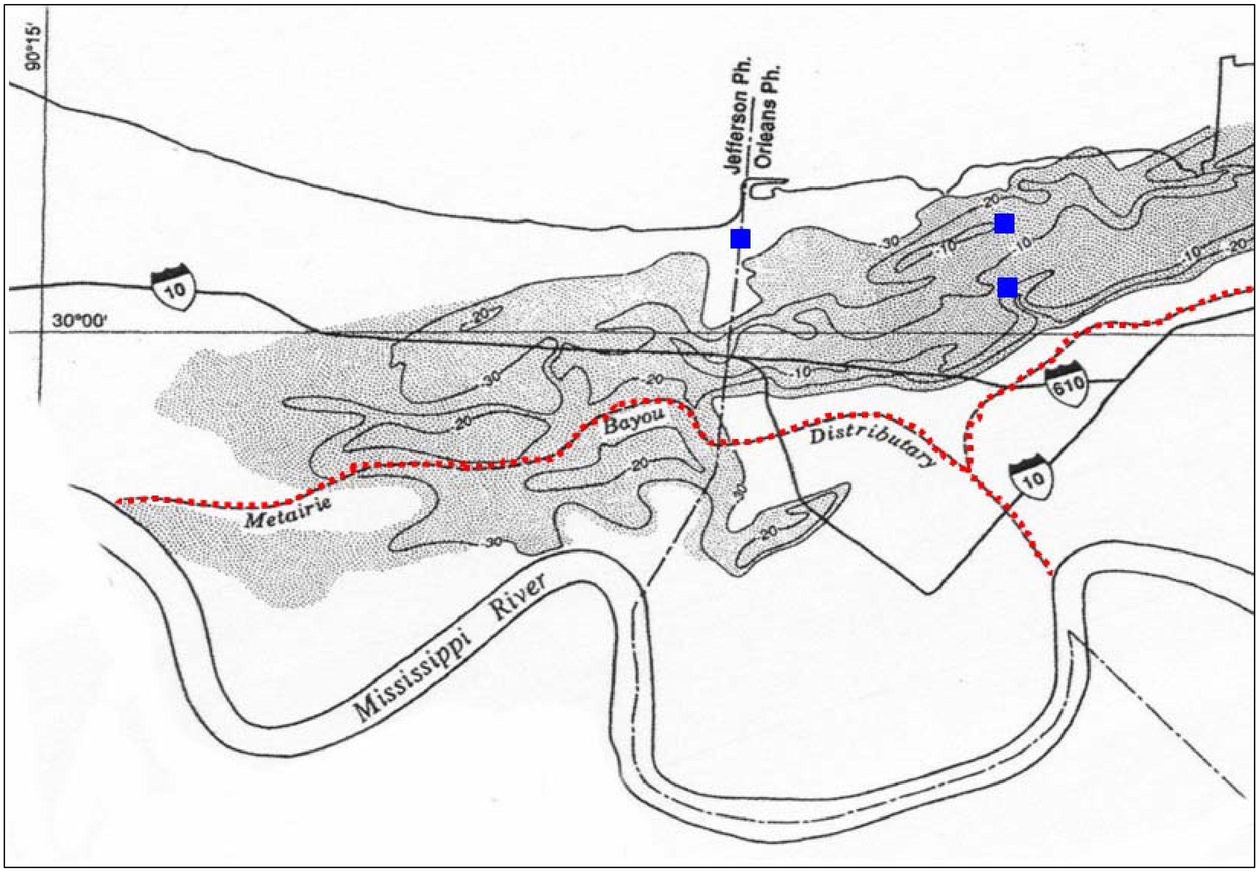


Figure 3. Close-up view of the buried beach ridge, and the locations of the canal breaches to the buried beach (after Saucier 1994). The 17th Street breach is located behind the axis of the beach ridge whereas the London Canal breaches are located on the axis of the ridge. Bayou Metairie is identified 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 1920s and early 1930s. The soil types included clays (CL and 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 to 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 elevation -50 ft MSL along the south shore of Lake Pontchartrain as deep as elevation -210 ft MSL 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.

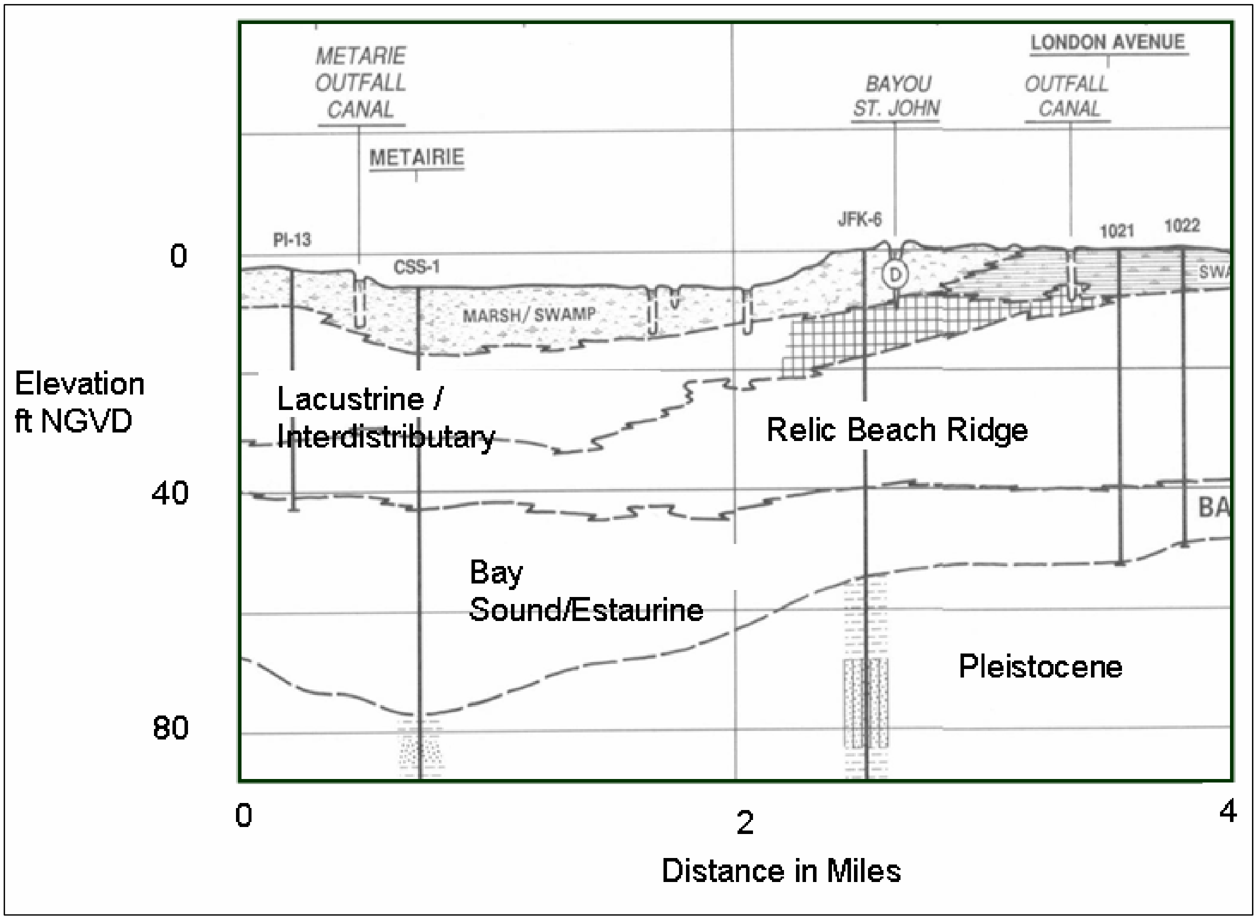


Figure 4. Portion of cross section C-C′ from the Spanish Fort Quadrangle which extends through the 17th Street and London Canal breaches and identifies the stratigraphic environments in the subsurface (from Dunbar and others 1995) (from Figure 1-6, Volume V).

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 (NOS & WB). 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 1⅞-in.-ID core barrel or 1-⅜-in. split spoon sampler. A-Es working directly for local entities usually obtained undisturbed samples using a 3-in.-diameter Shelby tube sampler and general samples using a split-spoon sampler. The borings were taken at various spacings ranging from 350 to 1,500 ft and were usually 50 to 80 ft deep with a few borings extending to a depth of 100 ft. Generally the borings were taken at spacings of 350 to 650 ft in the areas where floodwalls were to be constructed and on 700- to 1,500-ft 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-in.-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 NOS & WB. 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 was 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 (Lower Mississippi Valley 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 laboratories.

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 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 Q-Case design envelope for the CH and CL clays. For sands (SP and SM), the values of $\phi = 30°$ and C = 0 psf were normally used. For silts (ML), the values of $\phi = 15°$ and C = 200 psf or $\phi = 28°$ 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 Protection 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 Avenue Outfall Canal, London Avenue Outfall Canal, Orleans Parish Lakefront Levee, and the 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 still-water level (swl). 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 (Reference 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 was 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 (FS) 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 (Reference 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 design memoranda were computed for cost estimating purposes and were normally based on FS of 3.0 if no pile load test data were available and a FS 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 FS 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 LMVD Method of Planes—hereafter, simply the Method of Planes. The Method of Planes resolves the forces into horizontal components and computes the factor of safety as the ΣDriving Forces ÷ ΣResisting Forces. The Method of Planes is a wedge method that was developed in the 1950s 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 FSs 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 Q shear strengths with a minimum acceptable FS of 1.3. The only exception encountered in the design documents where the 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 Method of Planes and a minimum FS of 1.3 for the short-term 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 elevation 0 ft MSL, the water table was taken at elevation 0 ft MSL. In those areas where the buried beach sand was near the surface, FSs were determined for the headwater level at the top of the wall and for high tailwater 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- 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 DMs 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 to 4 ft 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 Mississippi River Gulf Outlet (MRGO) – GIWW and in the New Orleans to Venice area. The designers anticipated on the lakeshore 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' more conservative criteria for hydraulic structures applicable at the time of design as promulgated in published engineering manuals and other Corps design guidance documents. Also, the design work used generally consistently 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 1990s. The exception was foreign hot rolled sheet piling on projects constructed by local interests. Beginning in the 1990s, 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 1990s, 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 was 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 ACI 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, FSs, 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 of 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 of 1.5. On other design work, a FS of 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 in. 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 of 1.5.

On 23 December 1987, the 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 FS 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 of 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 in. of estimated lateral flexural deflection be reevaluated 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

- FS = 1.5 with water to flow line or swl
- FS = 1.25 with water to freeboard (net levee grade) for river levees or with swl and wave load for hurricane protection levees

S-Case

- FS = 1.2 with water to flow line or swl + wave load (if applicable) for hurricane protection levees
- FS = 1.0 with water to freeboard (net levee grade) for river levees

In addition, the 23 December 1987 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, FS = 1.5, whichever results in the least penetration.

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

Q-Case

- FS = 1.5 with water to flow line or swl
- FS = 1.25 with water to flow line plus approved freeboard for river levees or with swl and wave load for hurricane protection levees
- FS = 1.0 with water to swl +2.0 ft freeboard for hurricane protection levees

S-Case

- FS = 1.2 with water to flow line or swl and wave load. If a hurricane protection floodwall has no significant wave load, determine the penetration using the Q-Case criteria only
- FS = 1.0 with water to flow line 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 flow line 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 July 1989 guidance:

1. Added that if a hurricane protection levee has no significant wave load, 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 23 December 1987 criteria or the 24 July 1989 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 (P/H) of at least 3:1 unless this exceeds the penetration required by a S-Case, FS = 1.5. This follows the 23 December 1987 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 24 July 1989 criteria.

Table 1
Sheet-Piling Penetration Criteria Summary

	Penetration Factor of Safety		
	Q-Case	**S-Case**	**P / H Ratio**
Reference 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 wave load	FS = 1.2 water to swl and wave load FS = 1.0 water to fbd	3:1 or that required by S-Case, FS = 1.5, whichever results in least penetration
Reference 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 wave load FS = 1.0 water to swl plus 2 ft fbd	FS = 1.2 water to swl and wave load FS = 1.0 water to fbd	2.5 to 3:1

Table 2 summarizes how these criteria were 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	Reference 77 (no waves)	Reference 77
Orleans Avenue Outfall Canal	Reference 77 (no waves)	Reference 76
London Avenue Outfall Canal	Reference 77 (no waves)	Reference 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 / 2 ft 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 / 2 ft 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	
		(Continued)

Table 2 (Concluded)		
Location	**Penetration Factor of Safety**	**P / H Ratio**
West Bank and Vicinity		
Lake Cataouatche - Adjacent to Lake Cataouatche Pumping Stations 1 and 2	Reference 77 (w/ waves)	3:1
Lake Cataouatche - Station 518+00 to Bayou Segnette floodwall	Reference 77 (no waves)	3:1
Westwego to Harvey Canal Area	Reference 77 (w/ waves)	Reference 76
Westwego to Harvey Canal Area, Cousins Pumping Station	Reference 77 (no waves)	
East of Harvey Canal Area - East and West of Algiers Canal	Reference 77 (no waves)	3:1

Levee Construction Overview

Construction Overview

General. The construction overview is intended 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 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 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, offsite surveillance activities, job safety, environmental protection, and general remarks paragraph.

Much 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 QC 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 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 and sent to offsite storage where they remain for 6 years. After 6 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 Volume 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 sets of construction documents reviewed, five showed modifications or changes. Four could be considered as different site conditions. The first instance is described on page III-132, 17th Street Canal East Side Stations 0+96.27 to Station 7+00. About 4 ft 3 in. of sheet pile was cut off because of unanticipated hard driving. The sheet pile ended short of the desired penetration. The second instance (pages III-134 through III-135), 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 (page III-196), South Point to GIWW Levee, required a redesign of the levee and berm configuration due to sliding. The fourth instance, described on page III-299, St. Charles Parish North of Airline Highway, required modifications to remove pile driving obstructions. The fifth instance (pages III-271 through III-272) was Jefferson Parish Lakefront Levee, Pump Station No. 2. Because of a survey error, the breakwater was realigned 70 ft 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 1980s, 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-in. maximum thickness lifts and compacting with three passes of a dozer. After the late 1980s, an end result type specification was used. The end result specification required that the levee materials be spread in 12-in.-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.

The 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 ft in thickness. No specific compaction is required.

3.2.1. Lake Pontchartrain and 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 parishes within the Lake Pontchartrain, LA, and Vicinity HPP.

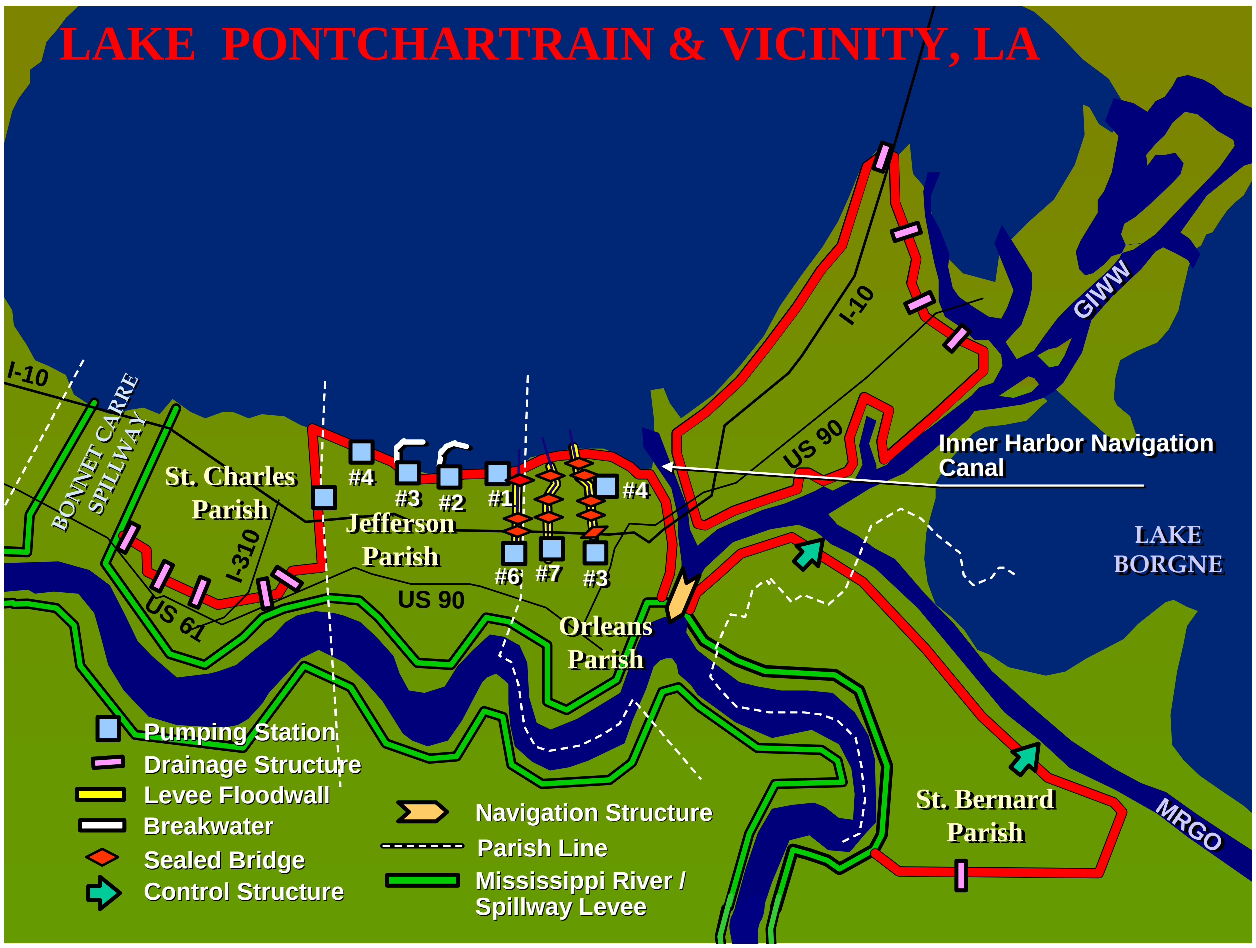


Figure 5. Index map to Lake Pontchartrain, LA, and Vicinity Hurricane Protection Project (includes St. Bernard, Orleans, Jefferson, and St. Charles Parishes as shown)

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, LA, and Vicinity HPP originated from an act of Congress, approved 15 June 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 21 November 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 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 4 March 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, LA, and Vicinity HPP 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 30 December 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, LA, and Vicinity HPP. 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 U.S. 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 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 7 February 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, LA, and Vicinity HPP, thereby formalizing the High Level Plan. In turn, the New Orleans District commenced examining two alternative plans for providing "high level" SPH 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 Corps 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 to 17 mm/yr or 2 to 5½ ft per century. In New Orleans itself, subsidence is about 3 ft per century. Subsidence is as much as 10 feet per century in Venice.

The Interagency Performance Evaluation Task Force (IPET), an independent group activated by the Corps 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 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 until the 1990s.

Benchmarks serve as the reference or starting elevation when measuring levee heights, relationships to the water surface (local mean sea level), and structure and levee elevations. 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, and state and local governments was performed on 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 has revised the elevations of survey benchmarks used to establish heights of structures, such as levees and floodwalls, in southern Louisiana. Use of the 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 of New Orleans is about ½ ft 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 variety of projects that rely on elevations relative to the local water surface.

Geodetic and water level datum is evaluated in extensive detail in Volume II.

3.2.1.4. Design Hurricane

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

Standard Project Hurricane

The 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, November 1959). The Weather Bureau and Corps 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, August 1965, November 1965, February 1966). The post-Betsy SPH parameters were used in the hydraulic analysis. An additional update was published by NOAA in 1979 (September 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-percent 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 CPI of all storms with a CPI less than or equal to 29 in. 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 estimates 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 P is the frequency of occurrence per 100 years, M is the rank, and Y is the period of record.

A SPH storm was considered to have a recurrence interval of once in 100 years (1 percent) 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 30 August 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 CPIs, 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.

Probable Maximum Hurricane

The probable maximum hurricane (PMH) 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 in. (U.S. Weather Bureau, August 1959, November 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

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-ft crown width with side slopes of 1 on 3. Along Lake Pontchartrain Lakefront the top elevation of the earthen levees ranges from 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 3, 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 range from elevation 13 to 15 ft NGVD. 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.

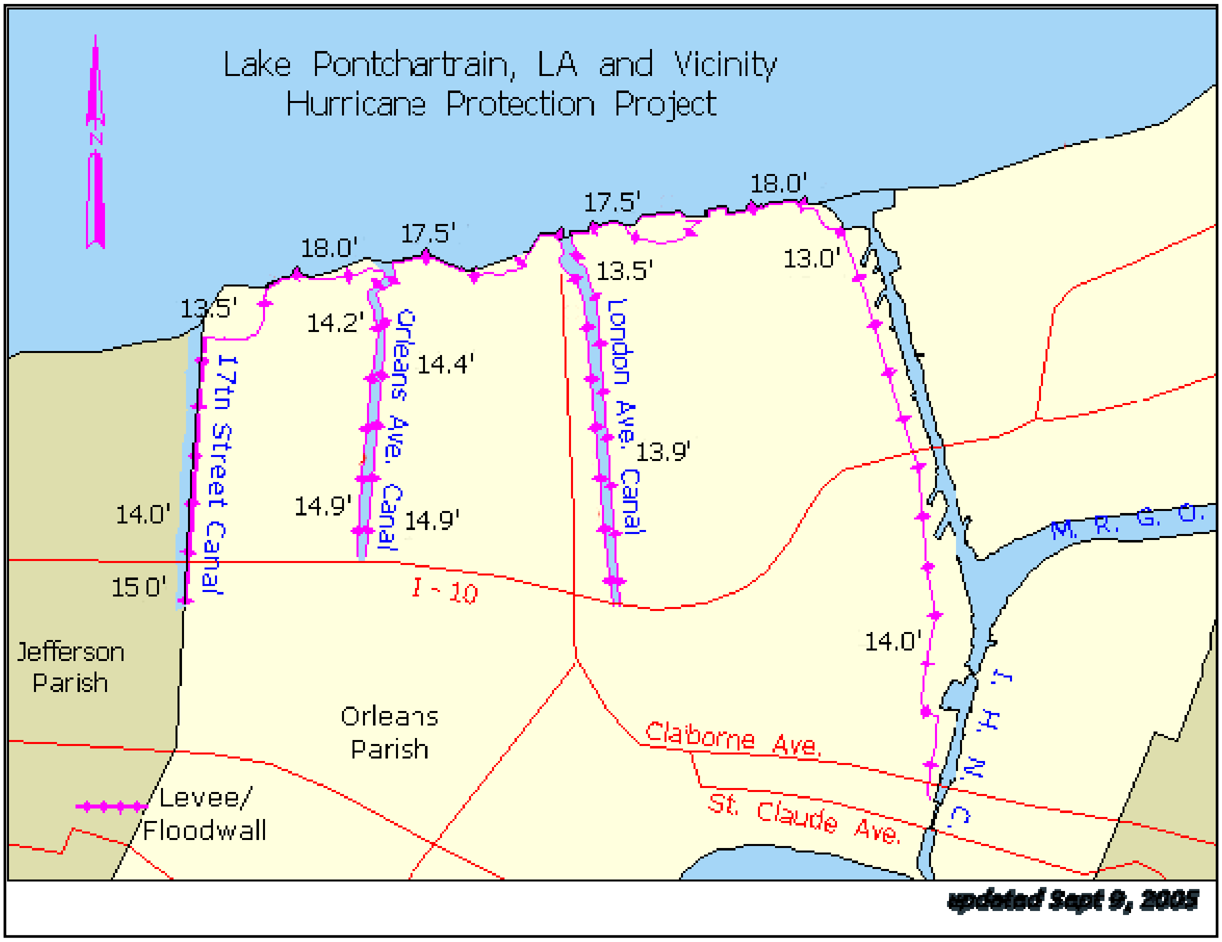


Figure 6. HPP features – New Orleans East Bank.

Table 3 **New Orleans East Bank Hurricane Protection System**	
19.2 miles	levee and floodwall
13	pump stations (owned by local agencies)
15	roadway floodgates

Orleans East Bank Lakefront. This protection system segment is located in southeast 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.

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 3 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.

Pre-Katrina

The Orleans Parish portion of the Lake Pontchartrain and Vicinity project is under construction. As of 29 August 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 Boulevard 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 Boulevard 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.

Design Criteria and Assumptions - Functional Design Criteria

Hydrology and Hydraulics.

For Orleans East Bank, the design hurricane characteristics utilized in the design memoranda are shown in Table 4; 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, miles per hour

P_n = asymptotic pressure, inches

P_0 = CPI, inches

R = radius of maximum winds, nautical miles

f = Coriolis parameter, hour $^{-1}$

T = the average speed of translation of the hurricane center, miles per hour

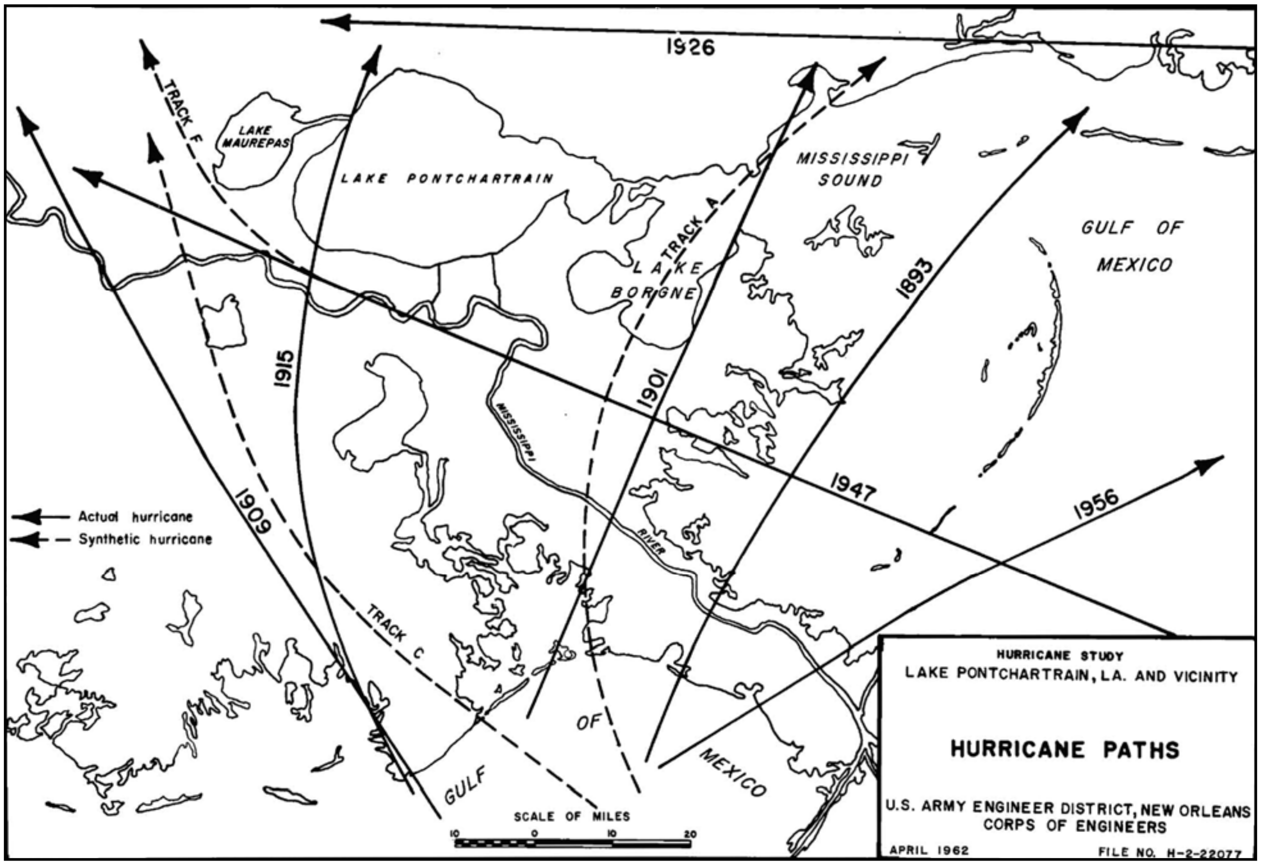


Figure 7. Hurricane paths, Lake Pontchartrain and Vicinity.

Table 4
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] Wind speeds represent a 5-minute average 30 ft above ground level.

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 still-water level. In Lake Pontchartrain, the wind tide level is the sum of the surge, setup, tide, and runoff from rainfall.

Surge.

IHNC. Stillwater levels or wind tide levels were computed using methods described in DM1, "Hydrology and Hydraulic Analysis, Part 1" (Chalmette, 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.

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

V = wind speed, statute miles per hour

F = fetch length, statute miles

D = average depth of fetch, feet

N = planform factor, generally equal to unity

Z = surge adjustment factor

θ = angle between direction of wind and the fetch

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 were 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 5 shows the surge computations and the comparison with observed high water marks from the three hurricanes.

Table 5
Verification of Hurricane Surge Heights

Location	Surge adjustment factor, Z	September 1915		September 1947		September 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 ft 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 versus 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 setdown were computed. Lake Pontchartrain was divided into parallel segmental regions. The average wind speeds 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

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

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

where

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

U = wind speed, miles per hour over fetch

F = fetch length, miles, node to shoreline

N = planform factor, equal generally to unity

Maximum computed and observed setup elevations for the 1947 hurricane were 4.9 ft and 5.4 ft, respectively, 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 July 1984 (References 103 and 104). These values were revised in subsequent DMs.

For the 17th Street Outfall Canal parallel protection, the Corps 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 the NOS & WB base project, with the exception of the area under bridges. Flow rate was initially based on pump capacities provided by the NOS & WB; 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 flow line is based on 9,630 cfs pump station capacity, floodproofing Veterans Avenue bridges, raising I-10 and I-610 bridges, and floodgates at Hammond Highway and Southern Railroad bridges. This flow line is not presented in DM20. Table 6 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 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 NOS & WB; 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 still-water level in Lake Pontchartrain for SPH condition. Five scenarios involving bridge modifications were modeled.

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

For the London Avenue Outfall Canal parallel protection, the Corps 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 NOS & WB; 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 flow line is based on 3,475 cfs pump station capacity, floodproofing of the bridges at Gentilly Boulevard, Mirabeau Avenue, Filmore Avenue, Robert E. Lee Boulevard, and Leon C. Simon Boulevard, and floodgates at Benefit Street and Southern Railroad bridges.

Table 6
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, November 1984	4.9	1.46	7.3	12.9 NGVD	4.7	-	18.0 NGVD
Orleans Lakefront, 43+10 to 78+59.24	DM13, November 1984	5.6	1.8	7.3	12.8 NGVD	5.5	-	18.0 NGVD
Orleans Lakefront, 78+59.24 to 88+24	DM13, November 1984	24.4	7.8	7.3	11.5 NGVD	8.5	-	20.0 NGVD
Orleans Lakefront, 88+24 to 94+60	DM13, November 1984	24.4	7.8	7.3	11.5N GVD	8.2	-	19.5 NGVD
Orleans Lakefront, 94+60 to 102+23.16	DM13, November 1984	4.6	1.33	7.3	12.9 NGVD	4.0	-	17.0 NGVD
Orleans Lakefront, 136+13.19 to 159+70	DM13, November 1984	4.9	1.48	7.3	12.8 NGVD	4.7	-	17.5 NGVD
Orleans Lakefront, 164+98.15 to 196+50	DM13, November 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, November 1984	4.9	1.48	7.3	12.8 NGVD	4.7	-	17.5 NGVD
Orleans Lakefront, 250+72.09 to 289+49	DM13, November 1984	5.6	1.8	7.3	12.8 NGVD	5.5	-	18.0 NGVD
Orleans Lakefront, 289+49 to 303+51.39	DM13, November 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, November 1984	6.2	2.06	7.3	12.8 NGVD	5.9	-	18.5 NGVD
Bayou St. John Closure	DM22, April 1993	-	7.8	7.3	11.5 NGVD	6.5	-	18.5 NGVD
Bayou St. John Structure	DM22, April 1993	-	2.1	7.3	11.5 NGVD	5.0	-	16.5 NGVD
Pontchartrain Beach Levee and Floodwall	DM22, April 1993	-	6.1	7.3	11.5 NGVD	8.5	-	20.0 NGVD
17th Street Outfall Canal, Hammond Highway	DM20, March 1990	-	-	-	11.66 NGVD	-	2.0	13.66 NGVD
17th Street Outfall Canal, Southern Railroad Bridge	DM20, March 1990	-	-	-	12.63 NGVD	-	2.0	14.66 NGVD
Orleans Ave Outfall Canal, lakefront to 118+00	DM19, August 1988	-	-	-	11.5 NGVD at Lake Pontchartrain	6.5	-	18.0 NGVD
Orleans Ave Outfall Canal, 118+00 to 90+86	DM19, August 1988	-	-	-	11.64 NGVD	-	2.0	13.64 NGVD
Orleans Ave Outfall Canal, 90+86 to 64+14	DM19, August 1988	-	-	-	11.80 NGVD	-	2.0	13.80 NGVD
Orleans Ave Outfall Canal, 64+14 to 36+64	DM19, August 1988	-	-	-	11.97 NGVD	-	2.0	13.97 NGVD
Orleans Ave Outfall Canal, 36+64 to PS No. 7	DM19, August 1988	-	-	-	12.21 NGVD	-	2.0	14.21 NGVD

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 (April 1956, October 1955, July 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 (August 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, respectively, were added to the wind tide level 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 Avenue 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 Avenue Outfall Canal would have a transition zone between the lakefront levee height and the floodwall height, which would account for wave runup.

Summary. Table 6 contains maximum surge or wind tide level, wave, and design elevation information.

Geotechnical.

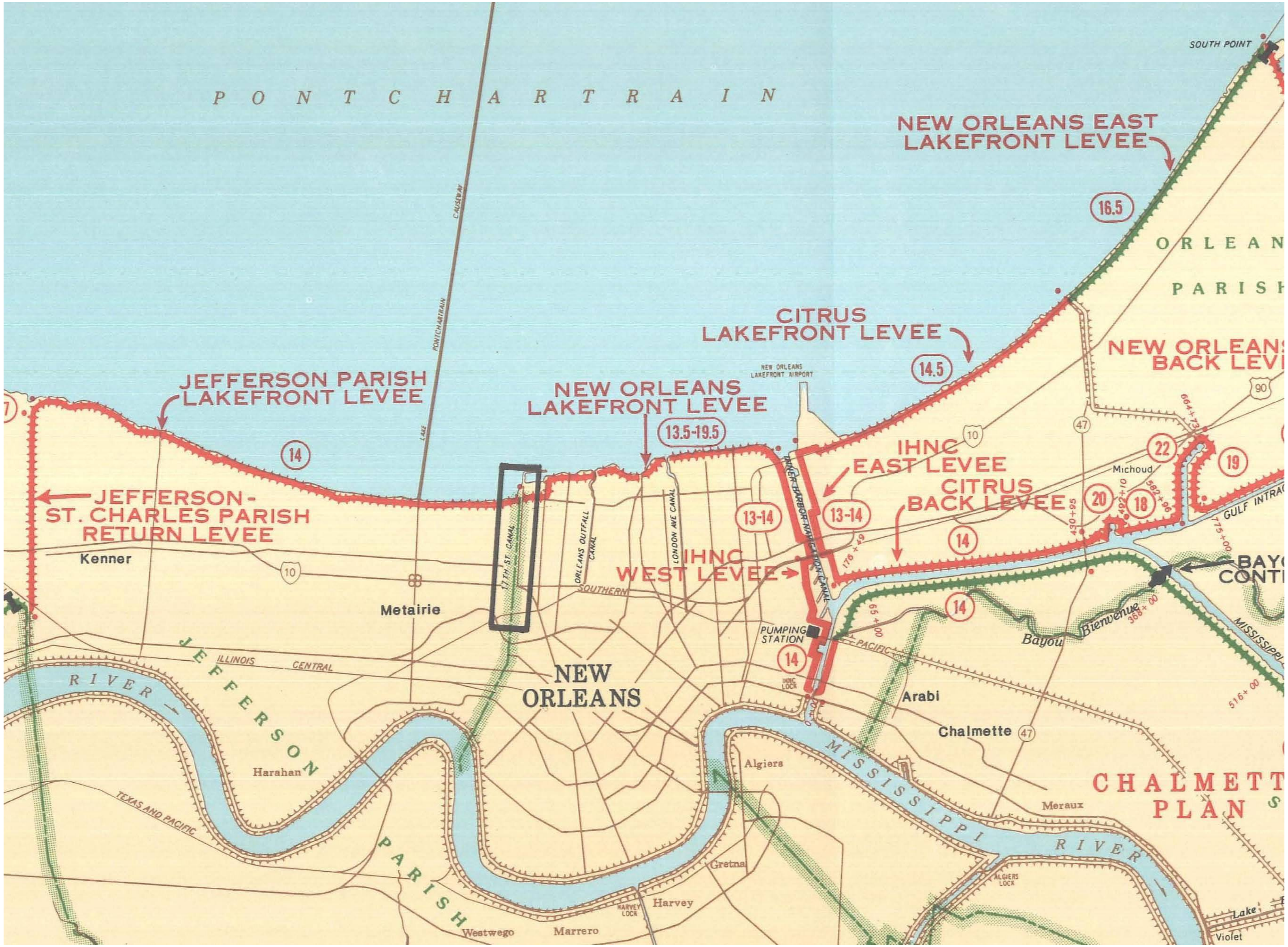
<u>17th Street Outfall Canal (Metairie Relief) (Reference 2).</u> This area shown in boxed area in image on following page.

Geology. The geologic history, principal physiographic features, and 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.

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 ft and underlain by Pleistocene deposits. Specifically, from Station 670+00 to Station 540+00 the surface is comprised of marsh-swamp deposits which range from 5 to 10 ft in thickness. 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 ft 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 to 20 ft in thickness. 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 up to 40 ft or more in thickness. 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 the deposits terminate near Station 540+00. Underlying the beach deposits throughout the project are bay-sound deposits which range in thickness from 15 to 20 ft. 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.

Field Exploration. Fourteen continuous sample, 5-in.-diameter soil borings were made by the Corps, 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 floodside and protected side toe of the canal levee. In addition, 77 borings made by an A-E for the NOS & WB were used in conjunction with the Corps borings in the foundation design. Nineteen of the borings by the A-E were sampled using a 5-in.-diameter Shelby tube sampler, and 58 borings were sampled using a 3-in.-diameter Shelby tube sampler.

Underseepage. Underseepage analyses were performed following EM 1110-2-2501 (Reference 3) using Lane's creep ratio.


P O N T C H A R T R A I N
NEW ORLEANS EAST LAKEFRONT LEVEE
16.5
ORLEANS
PARISH
CITRUS LAKEFRONT LEVEE
14.5
NEW ORLEANS BACK LEVEE
JEFFERSON PARISH LAKEFRONT LEVEE
NEW ORLEANS LAKEFRONT LEVEE
13.5-19.5
NEW ORLEANS LAKEFRONT AIRPORT
14
IHNC EAST LEVEE
CITRUS BACK LEVEE
22
19
Michoud
20
18
13-14
JEFFERSON-ST. CHARLES PARISH RETURN LEVEE
Kenner
17TH ST. CANAL
ORLEANS OUTFALL CANAL
LONDON AVE CANAL
IHNC WEST LEVEE
INNER HARBOR NAVIGATION CANAL
GULF INTRACOASTAL
BAYOU CONTI
Metairie
SOUTHERN
PUMPING STATION
Bayou Bienvenue
ILLINOIS CENTRAL
PACIFIC
RIVER
JEFFERSON
NEW ORLEANS
IHNC LOCK
Arabi
Chalmette
MISSISSIPPI
Harahan
Algiers
MISSISSIPPI RIVER
CHALMETTE PLAN
TEXAS AND PACIFIC
Meraux
PARISH
Gretna
ALGIERS LOCK
Harvey
HARVEY LOCK
Westwego
Marrero
Violet
SOUTH POINT
CAUSEWAY
PONTCHARTRAIN
LAKE

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 ft MSL 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 ft MSL 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. Neither the water level in the canal nor the dredging of the test section affected the piezometric levels.

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

Factors of Safety for Pile Capacity Curves

	With Pile Load Test	Without 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-in. square prestressed concrete piles (tested 1986) were conducted by the NOS & WB's contractors.

Levee Stability. Stability of the levees was analyzed by the Method of Planes for a minimum FS of 1.30 with respect to the design shear strength. The analyses considered potential failure surfaces to the floodside and the protected side of the levee. Analyses to the protected side considered the channel water level at the swl. Analyses to the channel side considered the channel water level at elevation -5.0 ft MSL which would be associated with a minimum water level in Lake Pontchartrain due to a hurricane having winds blowing from south to north.

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 FSs were applied to the design shear strengths. Following are I-wall design criteria used for this hurricane protection project levee:

Tip Penetrations

<u>Q-Case</u>

FS = 1.5 with water to swl
FS = 1.25 with water to swl and wave load
FS = 1.0 with water to swl + 2 ft freeboard

<u>S-Case</u>

FS = 1.2 with water to swl and wave load (if applicable)

<u>Deflections</u>

Q-Case, FS = 1.0 with water to swl + 2 ft freeboard

Bending Moments

Governing Tip Penetration Case
If the penetration to head ratio was less than 2.5:1, the penetration was to be increased to 2.5:1. 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 3:1 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 FS of 1.5, and a canal level of elevation 0.0 ft MSL. The District disagreed with the comment to increase the penetration ratio to 3:1 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.

Orleans Avenue Outfall Canal (Reference 4). Outfall Canal image is shown in the boxed area of map on the following page.

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

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 ft. Hydraulic fill sands ranging in thickness from about 10 to 20 ft 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 ft in thickness. The beach deposit underlies the lacustrine clays and ranges in thickness from about 40 ft on the southern end of the project to about 10 ft near Lake Pontchartrain. The base elevation of the beach deposit remains a fairly constant 45 ft. Bay/sound deposits underlie the beach deposit. The Pleistocene is encountered at an average depth of about 65 ft, but ranges in depth from about 40 to 85 ft.

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

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.

Hydrostatic Pressure Relief. Piezometers installed by the Levee Board and also by the Corps 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 grade line was determined for a swl of elevation 11.6 ft NGVD. The piezometric grade line was used


LAKE PONTCHARTRAIN
LAKESHORE DR
ROBERT E. LEE BLVD
PLATE 2
UNIVERSITY OF NEW ORLEANS
FILMORE AVE
HARRISON AVE
CITY PARK
ORLEANS AVE. OUTFALL CANAL
METAIRIE RELIEF OUTFALL CANAL
LONDON AVE. OUTFALL CANAL
VETERANS MEMORIAL HWY
WISNER BLVD
New Orleans
DILLARD UNIVERSITY
JEFFERSON PARISH
ORLEANS PARISH
ARKANSAS
LOUISIANA
MISSISSIPPI
TEXAS
LOCATION OF WORK
GULF OF MEXICO
VICINITY MAP
N
INDEX MAP
SCALE IN FEET
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19
GENERAL DESIGN
ORLEANS AVE. OUTFALL CANAL
INDEX AND VICINITY MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
H-2-30290
PLATE 1

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 ft NGVD for the analyses.

Pile Foundation. The estimated pile lengths from the pile capacity analyses were based on a FS 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.

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 FS of 1.3 with respect to the Q design shear strengths.

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

Q-Case

FS = 1.5 with water to swl
FS = 1.25 with water to swl and wave load
FS = 1.0 with water to swl + 2 ft of freeboard

S-Case

FS = 1.2 with water to swl and wave load (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 FS = 1.5, whichever resulted in the least penetration. The swl was used to calculate head for penetration to head ratio.

T-Walls. A deep-seated analysis utilizing a FS of 1.3 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.

London Avenue Outfall Canal (Reference 5). London Avenue Outfall Canal is shown in boxed area of image on following page.

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


LAKE PONTCHARTRAIN
LAKESHORE DR.
ROBERT E. LEE BLVD.
UNIVERSITY OF NEW ORLEANS
PLATE 2
BONNABEL BLVD.
METAIRIE RELIEF OUTFALL CANAL
PONTCHARTRAIN BLVD.
WEST END BLVD.
CANAL BLVD.
FILMORE AVE.
HARRISON AVE.
ORLEANS AVE. OUTFALL CANAL
MARCONI DR.
CITY PARK
BAYOU ST. JOHN
WISNER BLVD.
LONDON AVE. OUTFALL CANAL
ELYSIAN FIELDS AVE.
FRANKLIN AVE.
VETERANS MEMORIAL HWY.
METAIRIE RD.
JEFFERSON PARISH
ORLEANS PARISH
10
610
90
New Orleans
DILLARD UNIVERSITY
N
INDEX MAP
SCALE IN FEET
LOCATION OF WORK
LOUISIANA
MISSISSIPPI
TEXAS
VICINITY MAP
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A
GENERAL DESIGN
LONDON AVE. OUTFALL CANAL
INDEX AND VICINITY MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
JAN. 1969
H-2-30288
PLATE 1

Project Foundation Conditions. The project represents approximately 6 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 ft of hydraulic fill placed in the 1920s and early 1930s. The surficial marsh veneer was encountered over the majority of the project area and ranges in thickness from 5 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 ft, but varies from about -55 to -70 ft in elevation.

Field Exploration. Thirteen continuous undisturbed 5-in.-diameter soil borings and four general type soil borings using a 1⅞-in. core barrel or a 1 3/8-in. split-spoon sampler were made by the Corps, New Orleans District. The borings were located both along the centerline and along the toes of the levees. Sixty-nine borings taken by an A-E for the Orleans Levee Board were used in conjunction with the Corps borings in the foundation design. Three of the borings made by the A-E were made using a 5-in.-diameter Shelby tube sampler and the remaining 66 borings were made using a 3-in.-diameter Shelby tube sampler.

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.

Hydrostatic Pressure Relief. A piezometric grade line based on the ground surface and past piezometer readings was used. The Corps 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 ft west of the west levee toe. No top of pipe elevation had been obtained for the two A-E piezometers. The piezometric grade line assumed by the Corps for design was to be verified after the Orleans Levee Board collected readings from their piezometers.

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*	Without Pile Load Test
2.0	3.0

* A pile load test was to be conducted and a FS of 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.

Slope Stability. The slope stability analyses for the levee sections were performed using the Method of Planes and a minimum FS 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.

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

<u>Q-Case</u>

FS = 1.5 with water to swl
FS = 1.25 with water to swl and wave load
FS = 1.0 with water to swl + 2 ft freeboard

<u>Deflections</u>

Q-Case, FS = 1.0 with water to swl + 2 ft freeboard

<u>Bending Moments</u>

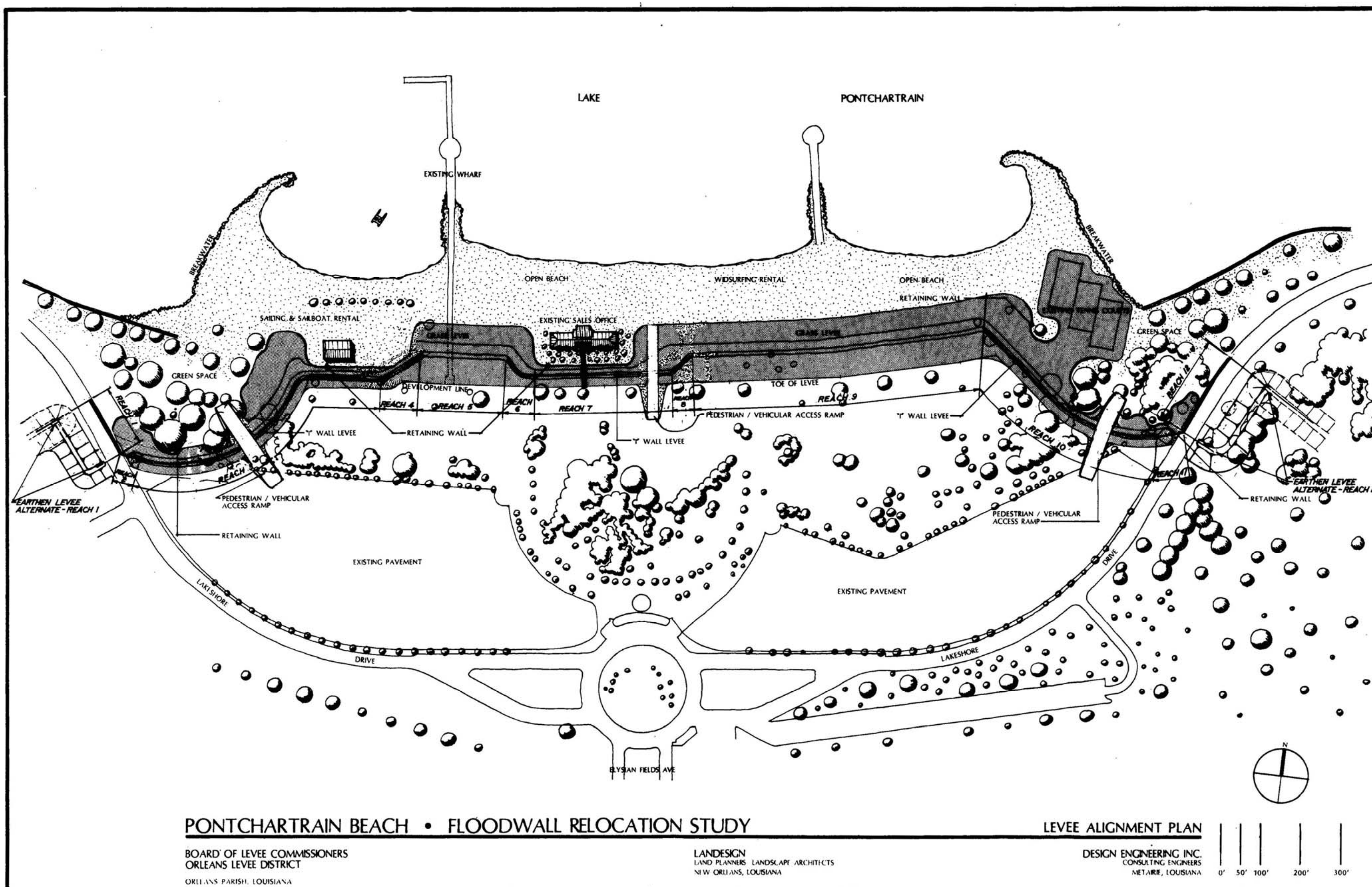
Governing Tip Penetration Case
If the penetration to head ratio were less than 3:1, it was to be increased to 3:1 or to that required by the S-Case, FS = 1.5, whichever results in the least penetration. The swl was used to calculate head for penetration to head ratio.

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.

<u>Pontchartrain Beach Floodwall/Levee Project (References 7 and 8).</u> Image on following page.

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.



LAKE
PONTCHARTRAIN
EXISTING WHARF
BREAKWATER
OPEN BEACH
WINDSURFING RENTAL
OPEN BEACH
RETAINING WALL
SAILING & SAILBOAT RENTAL
EXISTING SALES OFFICE
GRASS LEVEE
GREEN SPACE
DEVELOPMENT LINE
TOE OF LEVEE
REACH 1
REACH 2
REACH 3
REACH 4
REACH 5
REACH 6
REACH 7
REACH 8
REACH 9
REACH 10
REACH 11
REACH 12
"T" WALL LEVEE
RETAINING WALL
"T" WALL LEVEE
PEDESTRIAN / VEHICULAR ACCESS RAMP
"T" WALL LEVEE
EARTHEN LEVEE ALTERNATE - REACH 1
PEDESTRIAN / VEHICULAR ACCESS RAMP
RETAINING WALL
EARTHEN LEVEE ALTERNATE - REACH 12
RETAINING WALL
PEDESTRIAN / VEHICULAR ACCESS RAMP
EXISTING PAVEMENT
EXISTING PAVEMENT
LAKESHORE
DRIVE
LAKESHORE
DRIVE
ELYSIAN FIELDS AVE
N
PONTCHARTRAIN BEACH • FLOODWALL RELOCATION STUDY
LEVEE ALIGNMENT PLAN
BOARD OF LEVEE COMMISSIONERS
ORLEANS LEVEE DISTRICT
ORLEANS PARISH, LOUISIANA
LANDESIGN
LAND PLANNERS LANDSCAPE ARCHITECTS
NEW ORLEANS, LOUISIANA
DESIGN ENGINEERING INC.
CONSULTING ENGINEERS
METAIRIE, LOUISIANA
0' 50' 100' 200' 300'

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.

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

Project Foundation Conditions. A topographic survey indicated that the existing elevations along the levee alignment generally range from 5.0 to 7.0 ft 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 ranging from 2 to 3 ft NGVD. 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 -5.0 ft NGVD. 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 ranging from -20.0 to -26.0 ft NGVD. At these elevations and continuing to elevations ranging from -30 to -35 ft are strata of soft to medium stiff gray clay and sandy clay with sand layers and shell fragments. Beneath these strata to depths ranging from -41 to -46 ft 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 ranges from -50 to -55 ft NGVD.

Field Exploration. A total of ten undisturbed soil test borings were drilled for this investigation to depths of 80 and 100 ft. Two of the borings were sampled using a 5-in.-diameter Shelby tube sampler and the remaining eight borings were sampled using a 3-in.-diameter Shelby tube sampler. The samples from one of the 5-in.-diameter holes were delivered to the Corps, New Orleans District. In addition to the soil borings, four piezometers were installed in near surface sands at depths ranging from about 11 to 16 ft. 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 ft below the land surface.

Design Conditions. The design still-water level (swl) was taken as elevation 11.5 ft NGVD. Dynamic wave loads as furnished to URS Engineers and, in turn, Eustis Engineering Company by the Corps are tabulated below.

I-Wall Elevation ft	Levee Crown Elevation ft NGVD	Dynamic Wave Load lb/ft	Elevation of Wave Load Resultant ft NGVD
17.5	10.5	5,401	14.2
20.0	13.0	5,362	16.2

Levee Analyses.

Slope Stability. Levees were designed for a minimum FS with respect to slope stability of 1.3 using the 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.

I-Wall Analyses.

Cantilever I-wall Analyses. The I-wall analyses were performed for two conditions: (1) the still-water level loading with a FS of 1.5 factored into the soil shear strength parameters, and (2) the dynamic wave load with a FS 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 FS of 1.25 governed the required penetration of the sheet piling. A FS 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 Method of Planes and a minimum FS of 1.3. The results of the analyses indicated FSs greater than the minimum 1.3..

Underseepage. Underseepage for the combination I-wall/levee section was evaluated based on Lane's weighted creep ratio method of analyses. Weighted creep ratios ranging from approximately 10.2 to 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.

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 elevation -11.0 ft NGVD.

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 an FS of 2.0 for both tension and compression. No mention was made of whether the analyses were performed using Q or S strengths. It was pointed out in the report that the FS of 2.0 would only be applicable if the USACE conducted a test pile program to determine final pile design lengths. If the Corps did not conduct a test pile program, an

FS of 3.0 would be required. The curves presented in the report could be adjusted to reflect an FS of 3.0 by multiplying the capacities on the curves by a factor of two-thirds.

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 (percent)	
	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-in. 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.

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

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

Project Foundation Coordination. The subsurface along the alignment presented herein consists of approximately 8 to 15 ft of fill material overlying about 60 ft 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 elevation approximately -63 ft MSL at the northern end of the alignment near France Road, and at -70 ft MSL at the southern end near Florida Avenue.

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.

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 FS 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 floodside, and a water level equal to the water table on the protected side.

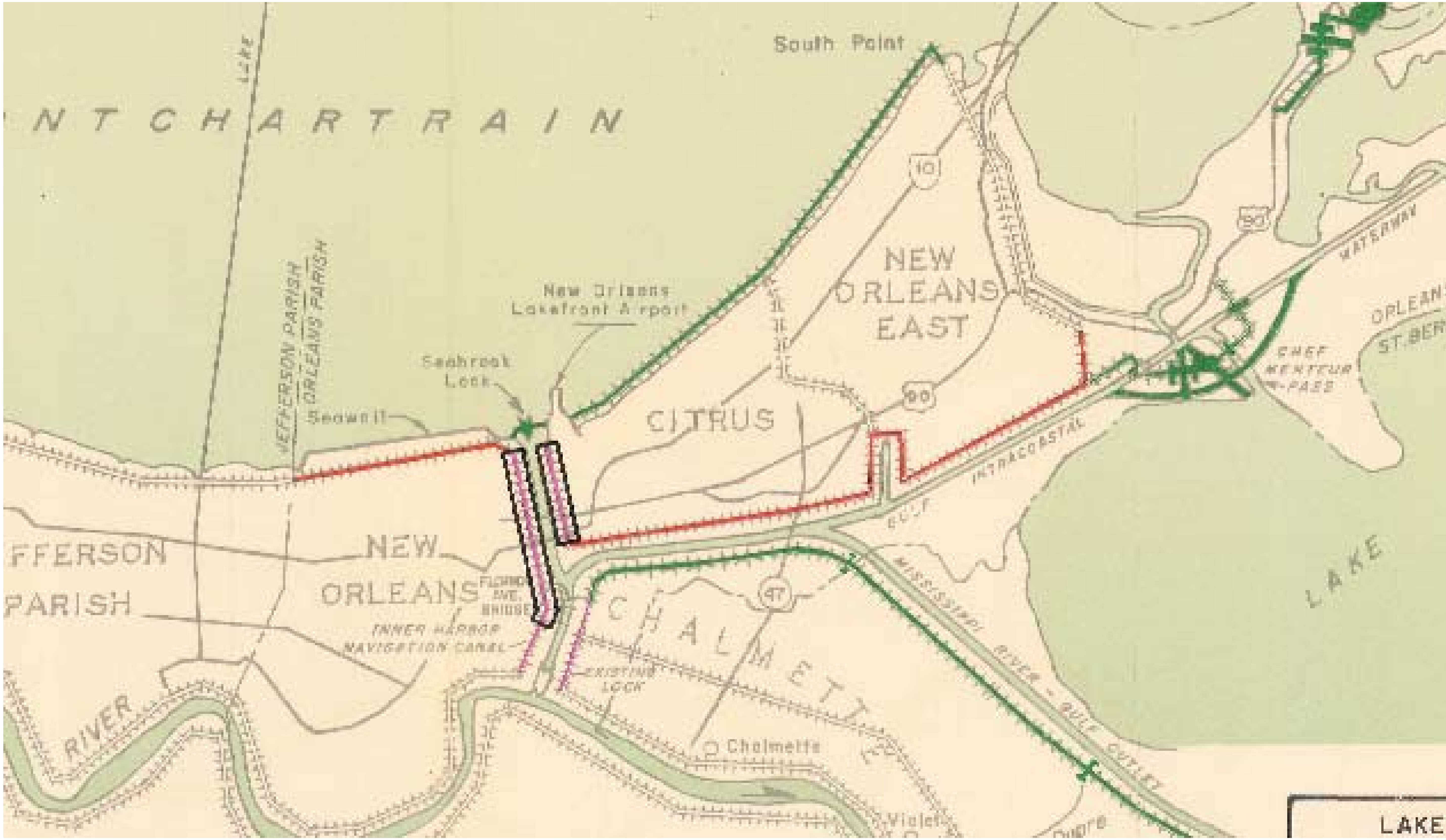
Levee and Levee/I-Wall Stability. Stability of the earthen levee and the levee/I-wall was investigated by the Method of Planes based on a minimum FS of 1.3 with respect to shear strength using the Q design shear strengths. The stability of a road ramp was also analyzed using these criteria. Analyses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

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.

Inner Harbor Navigation Canal Remaining Levees (Reference 9). Area shown on following page.

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

Project Foundation Conditions. The subsurface consists of recent deposits ranging in thickness from about 50 ft at the north or Lake Pontchartrain end of the project on both sides of the IHNC, to about 70 ft 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 ft. South of the abandoned distributary deposit, the Recent deposits consist 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.


NTCHARTRAIN
South Point
NEW
ORLEANS
EAST
New Orleans
Lakefront Airport
Seabrook
Lock
Seawall
JEFFERSON PARISH
ORLEANS PARISH
CITRUS
FFERSON
PARISH
NEW
ORLEANS
INNER HARBOR
NAVIGATION CANAL
EXISTING
LOCK
CHALMETTE
Chalmette
RIVER
GULF INTRACOASTAL WATERWAY
MISSISSIPPI RIVER - GULF OUTLET
CHEF
MENTEUR
PASS
ORLEANS
ST. BERN
LAKE
Violet
LAKE

Field Exploration. Ten 5-inch diameter undisturbed soil borings were made along the levee alignment. Twenty-eight 1⅞-in.-ID general type 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 ranging from 350 to 1,500 ft 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 elevation -15.0 to -98.0 ft. 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.

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.

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 FS of 1.5 was applied to the design shear strengths. The stability of I-type floodwalls was determined for a hurricane water level 6 in. 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. FSs were also determined for the headwater level at the top of the walls, and for high tailwater 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.

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 ft 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 FS 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.

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 were 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 ft MSL 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.

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 FSs of 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.

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.

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 I-walls and constructing the new I-walls, 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.

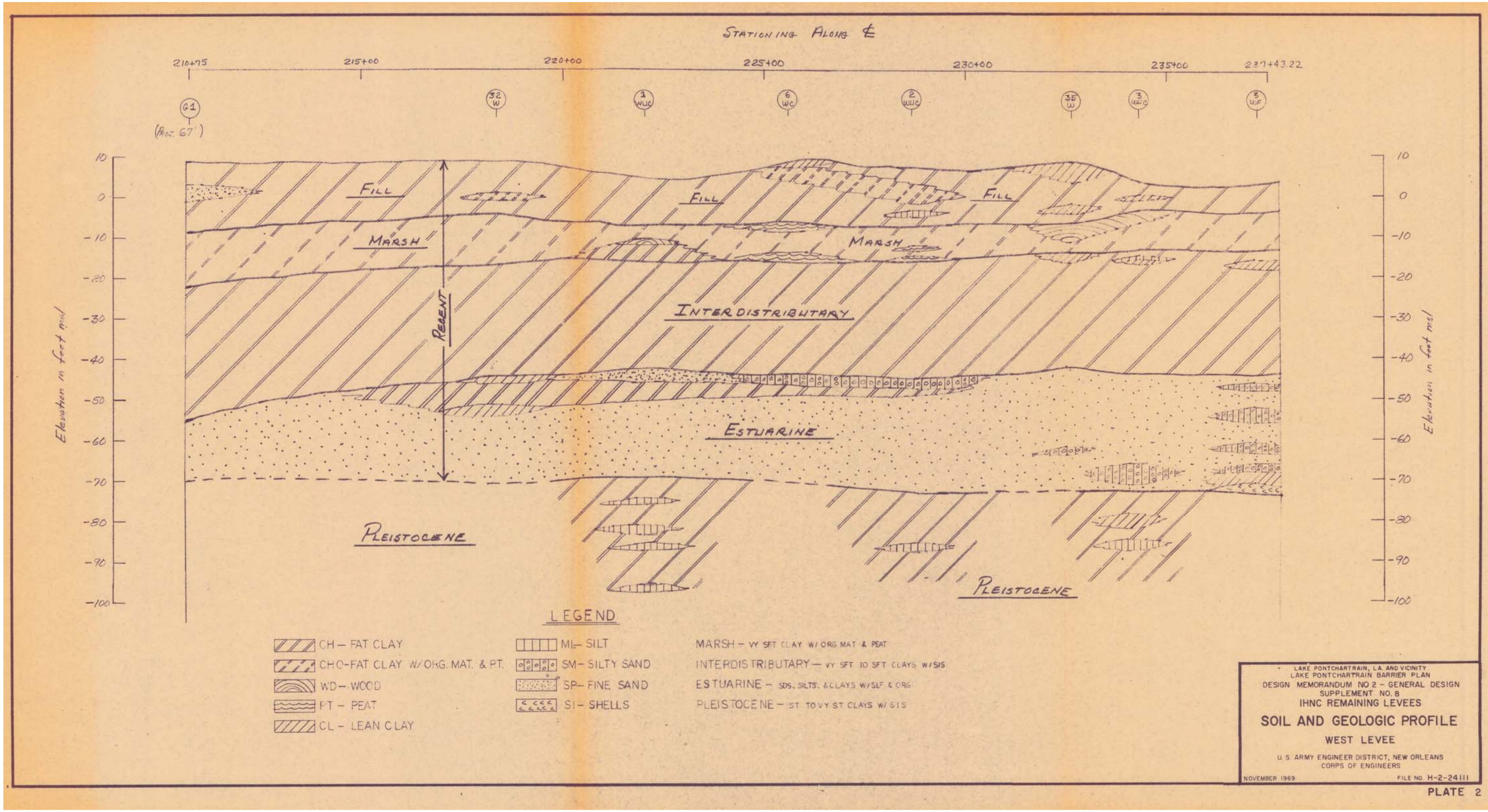
The sequence of construction in the buried beach area was to be as follows: install steel sheet piling, degrade, rehandle, and shape the existing fill, install pressure relief wells, piezometers, and collector systems; construct the concrete I-wall on the steel sheet piling; and fill and dress the levee 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.

Supplemental Design Information – IHNC Remaining Levees, West Levee Vicinity France Road and Florida Avenue (Reference 11). Area shown in figure on following page. 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.

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

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



STATIONING ALONG ℄
210+75
215+00
220+00
225+00
230+00
235+00
237+43.22
(Proj. 67')
Elevation in feet msl
FILL
MARSH
INTERDISTRIBUTARY
RECENT
ESTUARINE
PLEISTOCENE
LEGEND
CH – FAT CLAY
CHO – FAT CLAY W/ORG. MAT. & PT.
WD – WOOD
PT – PEAT
CL – LEAN CLAY
ML – SILT
SM – SILTY SAND
SP – FINE SAND
SI – SHELLS
MARSH – VY SFT CLAY W/ ORG MAT & PEAT
INTERDISTRIBUTARY – VY SFT TO SFT CLAYS W/SIS
ESTUARINE – SDS, SILTS, & CLAYS W/SLF & ORG
PLEISTOCENE – ST TO VY ST CLAYS W/ SIS
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO 2 – GENERAL DESIGN
SUPPLEMENT NO. 8
IHNC REMAINING LEVEES
SOIL AND GEOLOGIC PROFILE
WEST LEVEE
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NOVEMBER 1969
FILE NO. H-2-24111
PLATE 2

Field Investigation. In addition to the borings given in Reference 9, four 5-in.-diameter undisturbed borings and five 1⅞-in.-ID general type borings were made for the protective works on the revised alignment.

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 FS of 1.50 was applied to the design shear strengths. The required depths of penetration were determined for hurricane water level 6 in. below the top of the floodside, and water level equal to the water table on the protected side. FSs were also determined for the headwater level at the top of the walls.

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

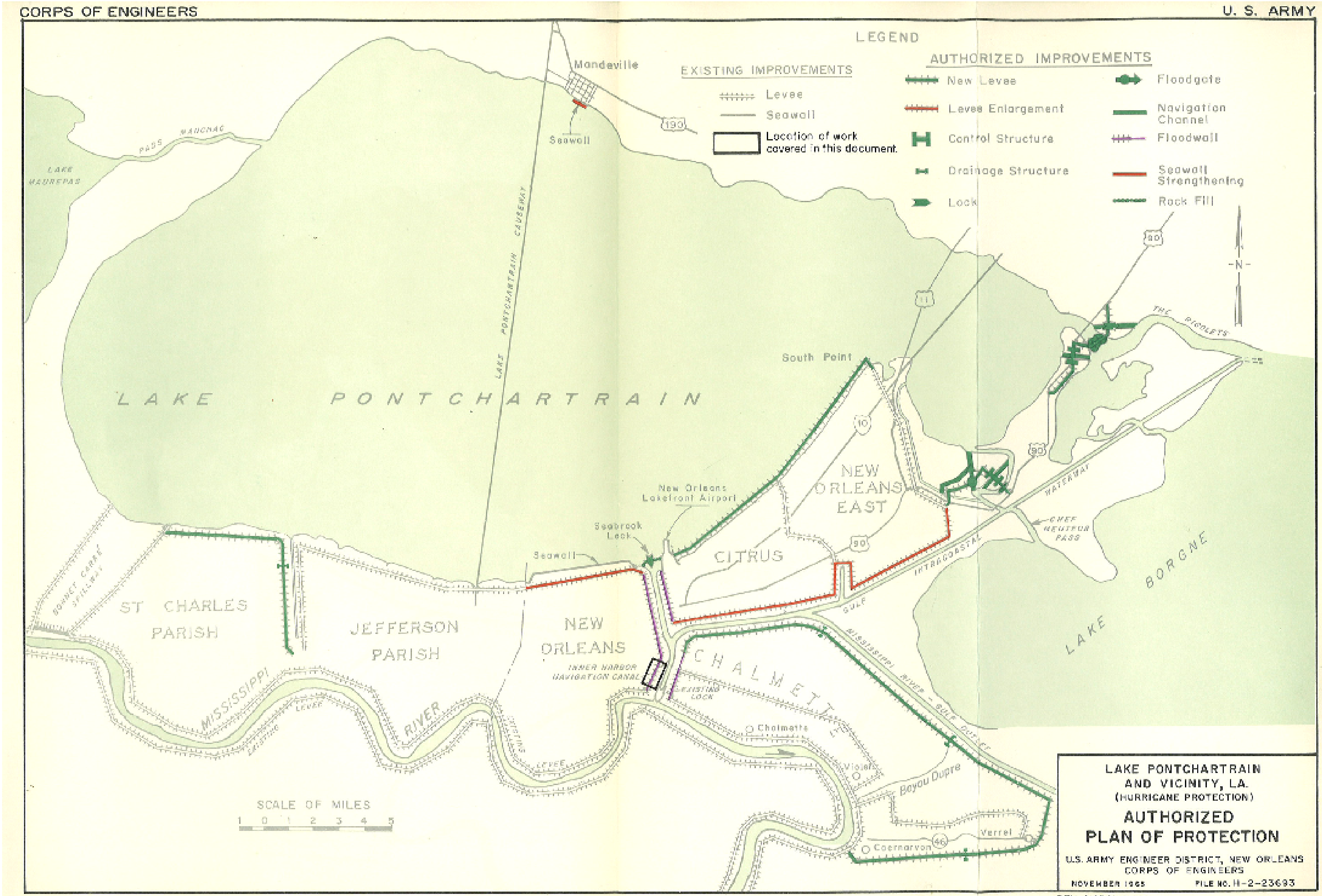
Pile Foundation. Pile bearing capacities for the gated structures and T-walls were determined from the pile test performed at site 1 of the IHNC West Levee, Florida Avenue to IHNC Lock project, where subsurface conditions are similar to those at the proposed site of the T-wall and gates. Results of this 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 FS, 1.75 for compression and 2.0 for tension, before using the graph. A minimum penetration elevation of -54.0 ft was required to assure adequate seating into the sand.

Design Memorandum No. 2, General, Advance Supplement, IHNC West Levee Florida Avenue to IHNC Lock (Reference 12). Area map shown on following page.

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

Project Foundation Conditions. The subsurface consists of Recent deposits ranging in thickness from 60 to 70 ft overlain by 6 to 16 ft of fill materials. The Recent deposits are underlain by Pleistocene deposits (Prairie Formation). Generally, the Recent deposits consist 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.

Field Exploration. Four 5-in.-diameter undisturbed soil borings and eighteen 1⅞-in.-ID general type core soil borings were made along the project alignment. The borings were made at intervals ranging from about 100 to 600 ft along the project location. The borings extended in depth to elevation -48 to -75 ft. 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-in.-diameter undisturbed soil boring and seven 1⅞-in.-ID general type soil boring were made along an alternate alignment later rejected.


CORPS OF ENGINEERS
U. S. ARMY
LEGEND
EXISTING IMPROVEMENTS
Levee
Seawall
Location of work covered in this document.
AUTHORIZED IMPROVEMENTS
New Levee
Levee Enlargement
Control Structure
Drainage Structure
Lock
Floodgate
Navigation Channel
Floodwall
Seawall Strengthening
Rock Fill
N
Mandeville
Seawall
LAKE MAUREPAS
PASS MANCHAC
LAKE PONTCHARTRAIN CAUSEWAY
LAKE PONTCHARTRAIN
South Point
THE RIGOLETS
CHEF MENTEUR PASS
LAKE BORGNE
NEW ORLEANS EAST
CITRUS
New Orleans Lakefront Airport
Seabrook Lock
Seawall
INTRACOASTAL
GULF
WATERWAY
ST CHARLES PARISH
BONNET CARRE SPILLWAY
JEFFERSON PARISH
NEW ORLEANS
INNER HARBOR NAVIGATION CANAL
EXISTING LOCK
CHALMETTE
Chalmette
Violet
Bayou Dupre
Verret
Caernarvon
MISSISSIPPI RIVER - GULF OUTLET
MISSISSIPPI RIVER
EXISTING LEVEE
SCALE OF MILES
1 0 1 2 3 4 5
LAKE PONTCHARTRAIN AND VICINITY, LA.
(HURRICANE PROTECTION)
AUTHORIZED PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NOVEMBER 1965
FILE NO. H-2-23693
REV. MARCH 1967

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 ft, and T-type concrete floodwalls with steel sheet-pile cutoffs supported by 12-in. by 12-in. square prestressed concrete bearing piles where the wall height was more than 10 ft.

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.

Cantilever I-Type Walls. Cantilever I-type floodwalls in levee fill were designed for the following loading conditions: top of wall at elevation 15.0 ft; water level on the floodside 6 in. below the top of the wall (1.5 ft MSL above still-water level at elevation 13.0 ft MSL) and groundwater on the protected side at elevation 0.0 ft MSL. The remaining I-type walls, with top at elevation 14.5 ft were designed with water 6 in, below top on floodside (1.0 ft MSL above still-water level at elevation 13.0 ft), and groundwater on the projected side at elevation 0.0 ft MSL. 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 ft MSL 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 FS of 1.5 was applied to the design shear strengths. Using the required penetrations, FSs 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 FS 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 ranged from about 250 to 400 psf.

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 ft on the floodside and ranging from elevation 0.0 to -6.0 ft 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 FS of 1.3 based on the Q shear strengths.

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 FS of 1.75 was applied to the shear strengths in compression, and a FS 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-in. by 12-in. 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.

Orleans Parish Lakefront Levee, Orleans Marina (Reference 13). Image of area is shown on following page. This section covers the soil and foundation investigations and design for approximately 1,500 ft of floodwall (I-wall, T-wall, and road gates) along Lake Avenue and adjacent to the Orleans Marina, New Orleans, LA. This is a portion of the HPP that is contained in the larger project feature, Lake Pontchartrain, LA, and 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.

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

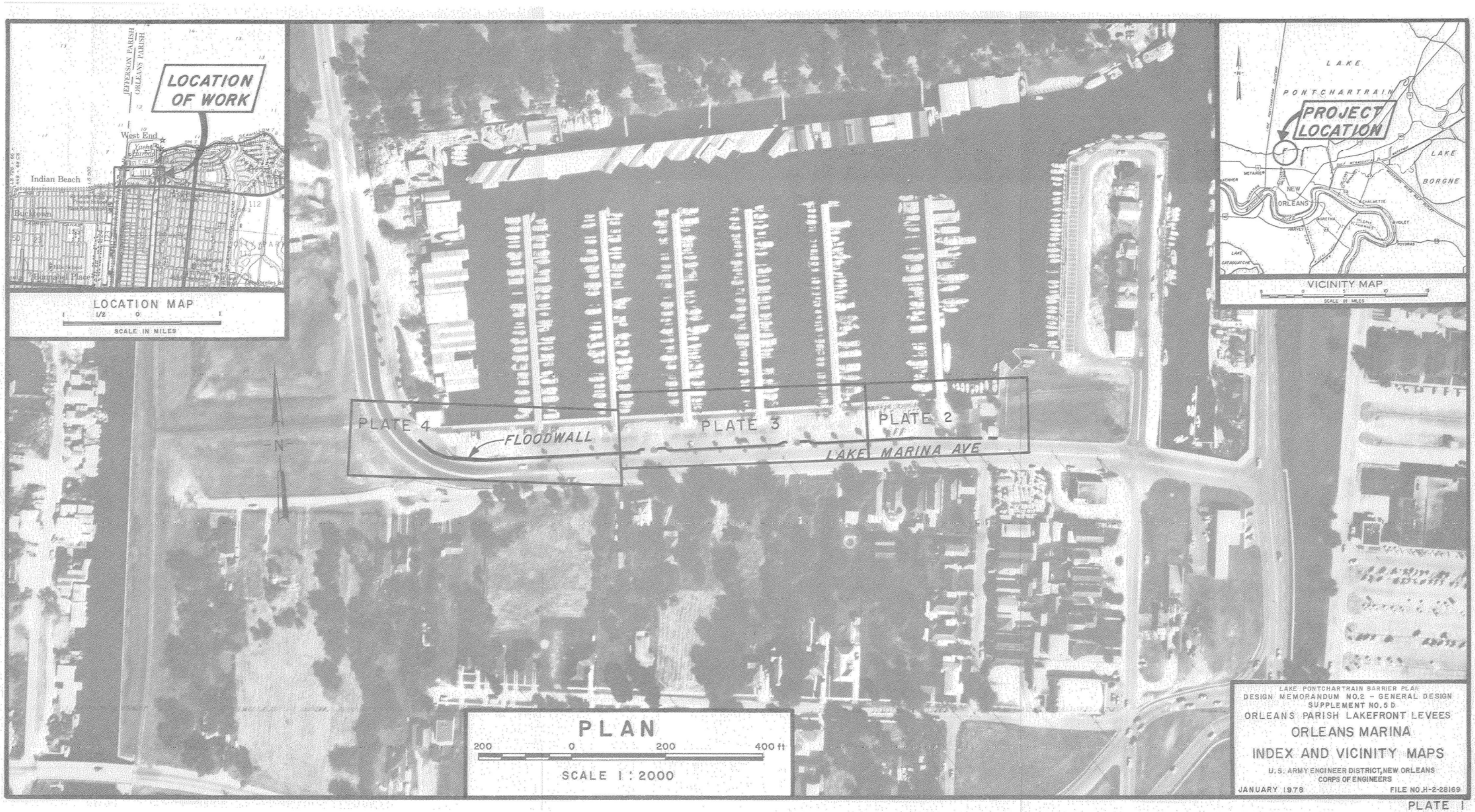
Project Foundation Conditions. The subsurface consists of Holocene deposits approximately 60 ft thick underlain by sediments of Pleistocene age. Generally, the Holocene sediments consist of a surface layer approximately 6 to 10 ft thick of fill material underlain by a 5- to 10-ft-thick layer of soft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

Field Exploration. Undisturbed 5-in.-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, 1⅞-in.-ID was also located in the vicinity of the project.

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 FS 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 was equal to zero.

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 FS 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 FS of 1.3.

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.


LOCATION OF WORK
JEFFERSON PARISH
ORLEANS PARISH
West End
Indian Beach
LOCATION MAP
1/2 0 1
SCALE IN MILES
LAKE
PONTCHARTRAIN
PROJECT LOCATION
LAKE
BORGNE
NEW
ORLEANS
VICINITY MAP
SCALE IN MILES
N
PLATE 4
PLATE 3
PLATE 2
FLOODWALL
LAKE MARINA AVE
PLAN
200 0 200 400 ft
SCALE 1 : 2000
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO.2 - GENERAL DESIGN
SUPPLEMENT NO.5 D
ORLEANS PARISH LAKEFRONT LEVEES
ORLEANS MARINA
INDEX AND VICINITY MAPS
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
JANUARY 1978
FILE NO. H-2-28169
PLATE 1

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 of 3 was determined for the gates and the T-wall sections.

A conventional stability analysis utilizing a 1.3 FS 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 FSs 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.

Orleans Parish Lakefront Remaining Work (Reference 14). Image of area shown on the following page. 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.

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

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

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

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 FSs 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

FS = 1.5 with water to swl
FS = 1.25 with water to swl and wave load
FS = 1.0 with water to swl + 2 ft freeboard


LAKE PONTCHARTRAIN
NEW ORLEANS
RIVER
LOCATION AND INDEX MAP
SCALE OF MILES

S-Case

FS = 1.2 with water to swl and wave load (if applicable)
FS = 1.25 with water to swl and wave load for Pontchartrain Beach (special case – defines existing criteria when constructed)

Bending Moments

Governing Tip Penetration Case

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

Pile Foundation. Pile load tests performed by the Corps 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 FS of 2.0. The resulting pile curves were to be used for cost estimating purposes.

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

<u>Orleans Parish Lakefront Levee West of IHNC (Reference 6).</u>

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

Project Foundation Conditions. The subsurface consists of Holocene deposits approximately 50 to 60 ft thick underlain by sediment of Pleistocene age. The Pleistocene sediments encountered in the borings consist of clays, silts, and silty sands. The contact surface ranges from an elevation of -50 to -80 ft. Overlying the Holocene sediment is a surface layer of fill material approximately 6 to 15 ft thick. From baseline Station 313+00 to 351+00, a 10- to 12-ft 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-ft-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 grade laterally into silts, silty sands, and sands in the eastern portion of the project area. A 6- to 12-ft thick layer of sand representing a buried beach underlies the lacustrine deposits. This sand deposit thickens to 22 to 25 ft 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 bay/sound clays. This layer generally is 12 to 18 ft thick, and thins eastward to 6 to 8 ft.

Field Exploration. A total of 13 new 5-inch undisturbed borings were made for this GDM by the Corps, 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 lake side 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.

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 FS 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 sheet-pile 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.

DIVR 1110-1-400, Section 8, Part 6, Item I, 30 November 1976.

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 FS was applied to the design shear strengths. The following FSs 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 (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 FS of approximately 1.3.

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 FS of 1.5 was applied to the S soil parameters. For determination of maximum bending moment and required anchor force, a FS of 1.0 was applied to the soil parameters.

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 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 FS 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.

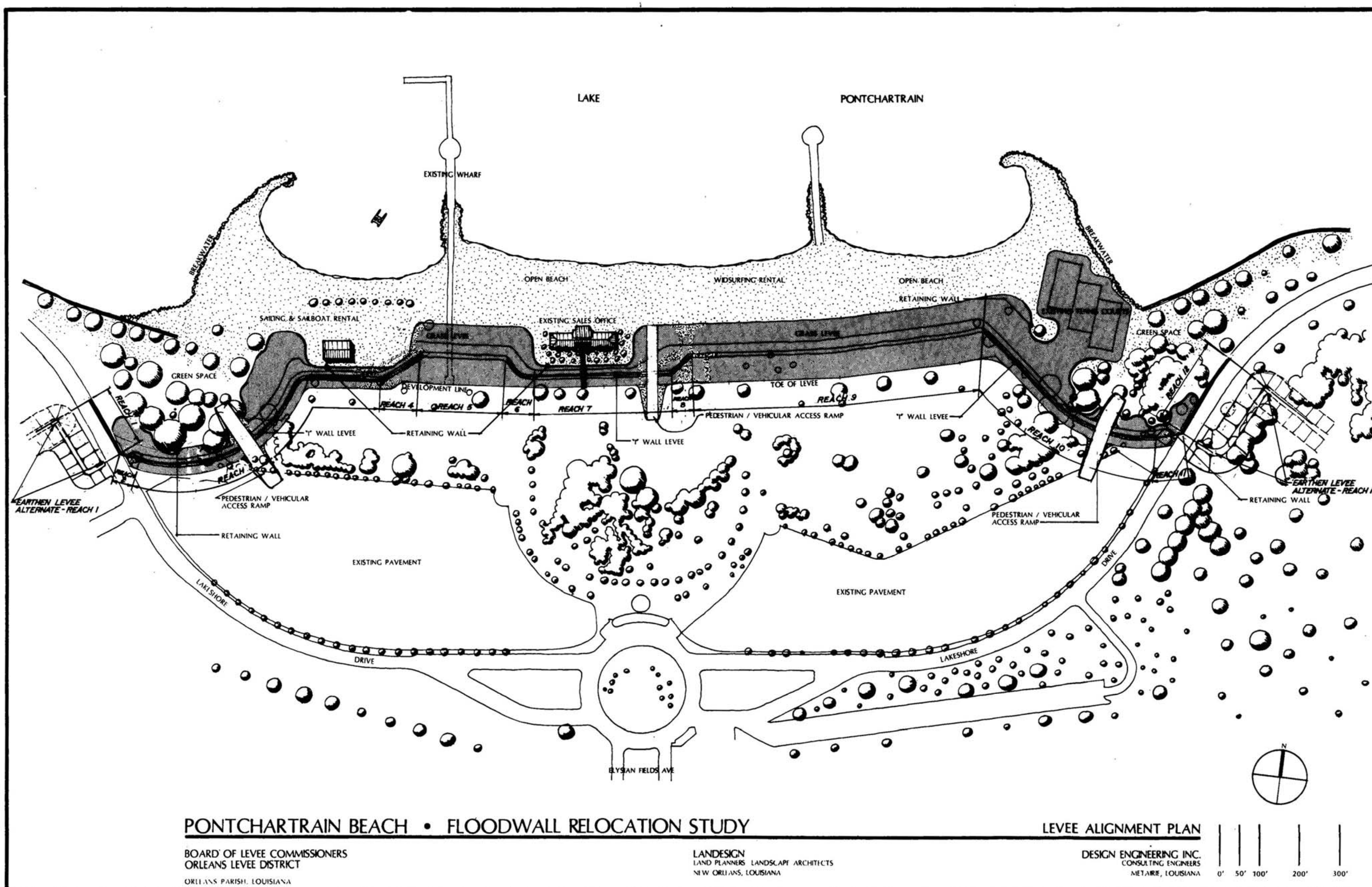
Bearing Pile Foundation. Ultimate compression and tension pile capacities versus tip elevations were developed for the Q-Case and S-Case. During construction, selected piles were to be driven and tested at some locations along the project alignment. The results of the pile load tests were to be used to determine the length of the service piles by applying a FS 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 FS of 3.0.

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 FS of 1.3 with respect to shear strength.

Division Review. No criteria changes resulted from Division review.

Structural.

<u>Orleans East Bank Lakefront - Pontchartrain Beach Floodwall –Reference 7.</u> Image of area shown on following page.


LAKE
PONTCHARTRAIN
EXISTING WHARF
BREAKWATER
OPEN BEACH
WINDSURFING RENTAL
OPEN BEACH
RETAINING WALL
SAILING & SAILBOAT RENTAL
EXISTING SALES OFFICE
GRASS LEVEE
GREEN SPACE
GREEN SPACE
TOE OF LEVEE
DEVELOPMENT LINE
REACH 4
REACH 5
REACH 6
REACH 7
REACH 8
REACH 9
REACH 10
REACH 11
REACH 12
"T" WALL LEVEE
RETAINING WALL
"T" WALL LEVEE
PEDESTRIAN / VEHICULAR ACCESS RAMP
"T" WALL LEVEE
EARTHEN LEVEE ALTERNATE - REACH 1
PEDESTRIAN / VEHICULAR ACCESS RAMP
RETAINING WALL
PEDESTRIAN / VEHICULAR ACCESS RAMP
EARTHEN LEVEE ALTERNATE - REACH 12
RETAINING WALL
EXISTING PAVEMENT
EXISTING PAVEMENT
LAKESHORE
DRIVE
LAKESHORE
DRIVE
ELYSIAN FIELDS AVE
N
PONTCHARTRAIN BEACH • FLOODWALL RELOCATION STUDY
LEVEE ALIGNMENT PLAN
BOARD OF LEVEE COMMISSIONERS
ORLEANS LEVEE DISTRICT
ORLEANS PARISH, LOUISIANA
LANDESIGN
LAND PLANNERS LANDSCAPE ARCHITECTS
NEW ORLEANS, LOUISIANA
DESIGN ENGINEERING INC.
CONSULTING ENGINEERS
METAIRIE, LOUISIANA
0' 50' 100' 200' 300'

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 elevation 11.5 ft NGVD.

I-walls top elevation 20.0 ft NGVD and levee crown 13.0 ft 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 FS of 1.3 using Method of Planes.

Cantilever Analysis

Cantilever analysis was used with a FS of 1.5 factored into the soil shear strength parameters for the static-water level loading.

Cantilever analysis was used with a FS of 1.25 factored into the soil shear strength parameters for the dynamic water level loading.

A FS of 1.0 for the second case was used to determine maximum bending moment.

This resulted in a tip elevation of -14.0 ft NGVD.

Maximum desirable deflection of the wall was 1.5 in..

Gate Structures

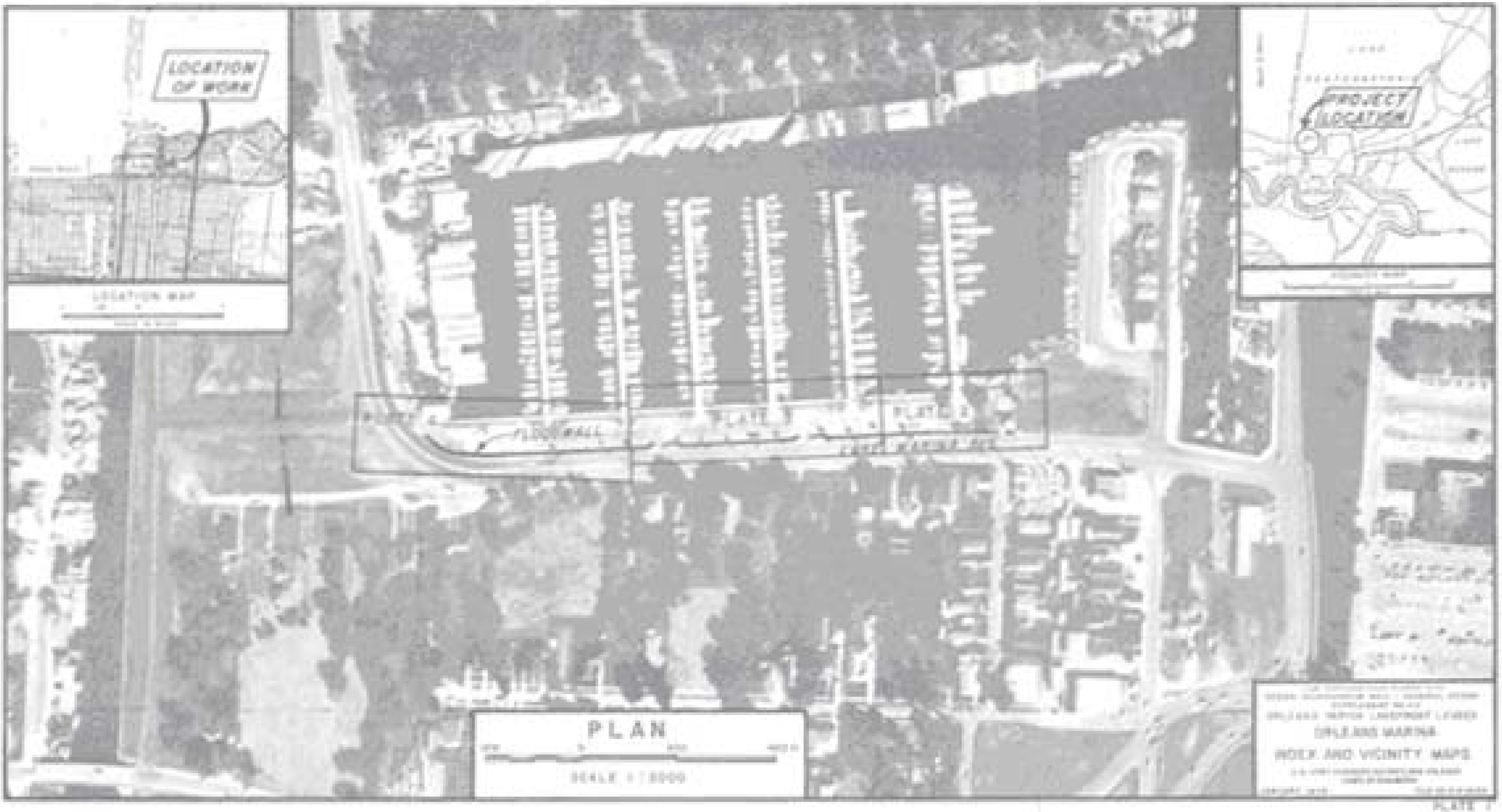
Allowable Pile Load Capacity

Recommended loads for 14-in. square, precast, prestressed concrete piles based on a FS of 3. No load test was performed.

Pile Loads

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

Orleans East Bank Lakefront - Orleans Parish Lakefront Levees Orleans Marina – Reference 13. Image of area is shown on following page.


LOCATION OF WORK
LOCATION MAP
PROJECT LOCATION
FLOODWALL
PLATE 3
PLATE 2
PLAN
ORLEANS MARINA
INDEX AND VICINITY MAPS
PLATE 1

General. As constructed, the Orleans Marina Hurricane Protection System consists of a 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/cu ft*
Water	62.5
Concrete	150
Steel	490

Design Loads

Wind	50 lb/sq ft
Water	62.5 lb/sq ft

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 pre-stressed 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	
f'_c	4,000 psi
f_c	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 FS 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.

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 cantilevered 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 and 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	Range from 14.0 to 20.5
T-walls and gates	Range from 13.5 to 20.75

Unit Weights

	lb/ cu ft
Water	64.0
Concrete	150
Steel	490

Design Loads

Wind	50 lb/sq ft

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, Grade 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 (swl plus freeboard), no dynamic wave force (2) For unconfined areas: FS used = 1.5. Static water at top of wall (swl plus freeboard) and no dynamic wave force FS used = 1.25. Static water at top of wall (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 level, no wind, impervious sheet-pile cutoff, no dynamic wave force
Case II	Static water level, no wind, pervious sheet-pile cutoff, no dynamic wave force
Case III	Still-water level to El. 11.5 ft, dynamic wave force, no wind, impervious sheet-pile cutoff (75 percent forces used)
Case IV	Still-water level to El. 11.5 ft, dynamic wave force, no wind, pervious sheet-pile cutoff (75 percent forces used)
Case V	No water, no wind
Case VI	No water, wind from protected side (75 percent forces used)
Case VII	No water, wind from floodside (75 percent forces used)

Swing Gates, Miter Swing Gates, and Bottom Roller Gates

Case I	Gate closed, still-water pressure to elevation 11.5 ft, dynamic wave force, impervious sheet-pile cutoff (75 percent forces used)
Case II	Gate closed, still-water pressure to elevation 11.5 ft, dynamic wave force, pervious sheet-pile cutoff (75 percent 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 percent forces used)
Case VI	Gate open, wind from floodside, truck or train on edge of slab on protected side edge of base slab (75 percent forces used).

Vertical Lift Roller Gate

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

Orleans East Bank Lakefront Orleans Avenue Outfall Canal – Reference 4. Image of area is shown on following page.

LAKE PONTCHARTRAIN
LAKESHORE DR
ROBERT E. LEE BLVD
UNIVERSITY OF NEW ORLEANS
LOCATION OF WORK
VICINITY MAP
FILMORE AVE
HARRISON AVE
VETERANS MEMORIAL HWY
METAIRIE RELIEF OUTFALL CANAL
ORLEANS AVE. OUTFALL CANAL
LONDON AVE. OUTFALL CANAL
CITY PARK
WISNER BLVD
New Orleans
DILLARD UNIVERSITY
JEFFERSON PARISH
ORLEANS PARISH
INDEX MAP
SCALE IN FEET
N
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19
GENERAL DESIGN
ORLEANS AVE. OUTFALL CANAL
INDEX AND VICINITY MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
AUG. 1988
H-2- 30290
PLATE 1

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 NOS & WB Drainage Pumping Station No.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 NOS & WB Drainage Pumping Station No. 7, combination earthen levee and capped cantilevered I-wall from NOS & WB Drainage Pumping Station No.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 NOS & WB Drainage Pumping Station No. 7 has not been constructed.

Structural Design

Design Criteria

I-Walls

Q–Case

FS = 1.5 with water to swl
FS = 1.25 with water to swl and wave load
FS = 1.0 with water to swl + 2 ft freeboard

S–Case

FS = 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 of 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, LA, 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 F_y using A36 steel.

Load Cases

Case I	Water to top of gate
Case II	Wind load of 50 lb/sq ft on the gate

<u>INHC Canal (West Bank) – Reference 9.</u> Map of area shown on following page. 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 range from elevation 11.4 ft at Seabrook to 12.9 ft at the L&N Railroad bridge and to 13.0 ft at the IHNC Lock. Water elevations landside of the floodwall range from elevation 0.0 to -3.0 ft. The elevation of the top of an I-wall in a levee is 2.0 ft above the wind tide level. The elevations of the top of T-type walls and gates are 1.0 ft above the wind tide level.

Unit Weights	lb/cu ft
Water	62.5
Concrete	150
Steel	490

Water Loads

- No wave forces will occur
- One foot freeboard

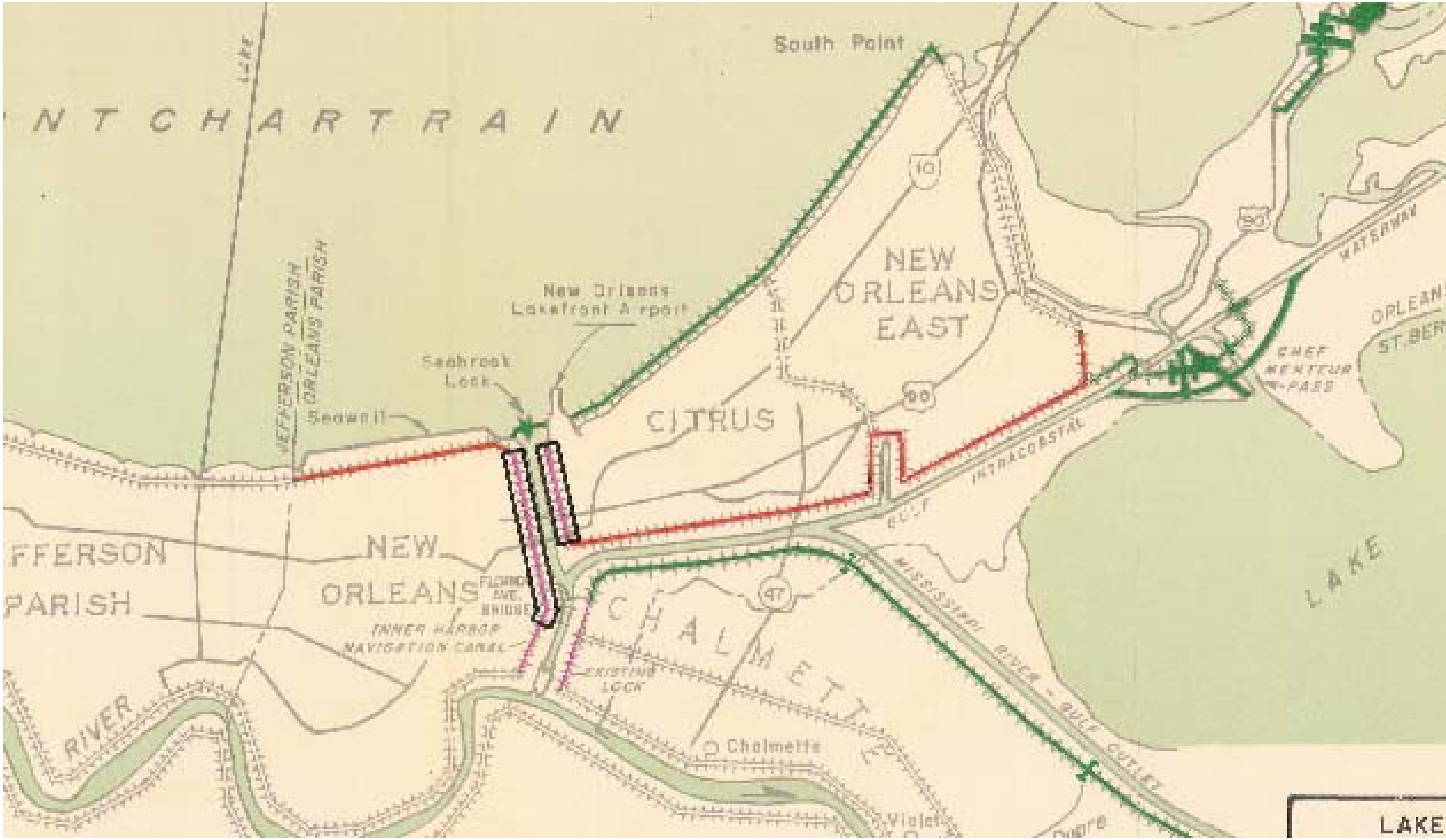
I-type Floodwall. Bending moments and deflections for structural design of sheet piles were based on a FS of 1.5 applied to the soils. The strength of the wall was checked for the case with water at the top of the wall and found to be adequate.

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 ft on the floodside and elevation 0.0 ft 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 ft.

- 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 ft on floodside and water at elevation 0.0 ft on protected side. Impervious cut off. Uplift as in Case 1.


NTCHARTRAIN
South Point
NEW ORLEANS EAST
New Orleans Lakefront Airport
Seabrook Lock
Seawall
JEFFERSON PARISH
ORLEANS PARISH
CITRUS
FFERSON PARISH
NEW ORLEANS
FLORIDA AVE. BRIDGE
INNER HARBOR NAVIGATION CANAL
EXISTING LOCK
CHALMETTE
Chalmette
RIVER
GULF INTRACOASTAL WATERWAY
MISSISSIPPI RIVER - GULF OUTLET
CHEF MENTEUR PASS
ORLEANS
ST. BERN
LAKE
Violet
LAKE

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

- Case 5 - Water at elevation 10.0 ft on floodside and at elevation 0.0 ft 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 percent of the submerged unit weight of earth (55 lb/cu. ft.) on the floodside and cracked section assumed on protected side.

IHNC West Levee - Florida Avenue to IHNC Lock –Reference 12. Map of area shown on following page. The protective works covered herein consist of approximately 2,150 ft of I-type cantilever floodwall and 4,900 ft 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 at the loading platform of the Jones & Laughlin Steel Company warehouse.

Structural Design

Design Criteria

Basic data

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

Floodwall grades	*Elevations* (ft NGVD)
Net grade (1 ft 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

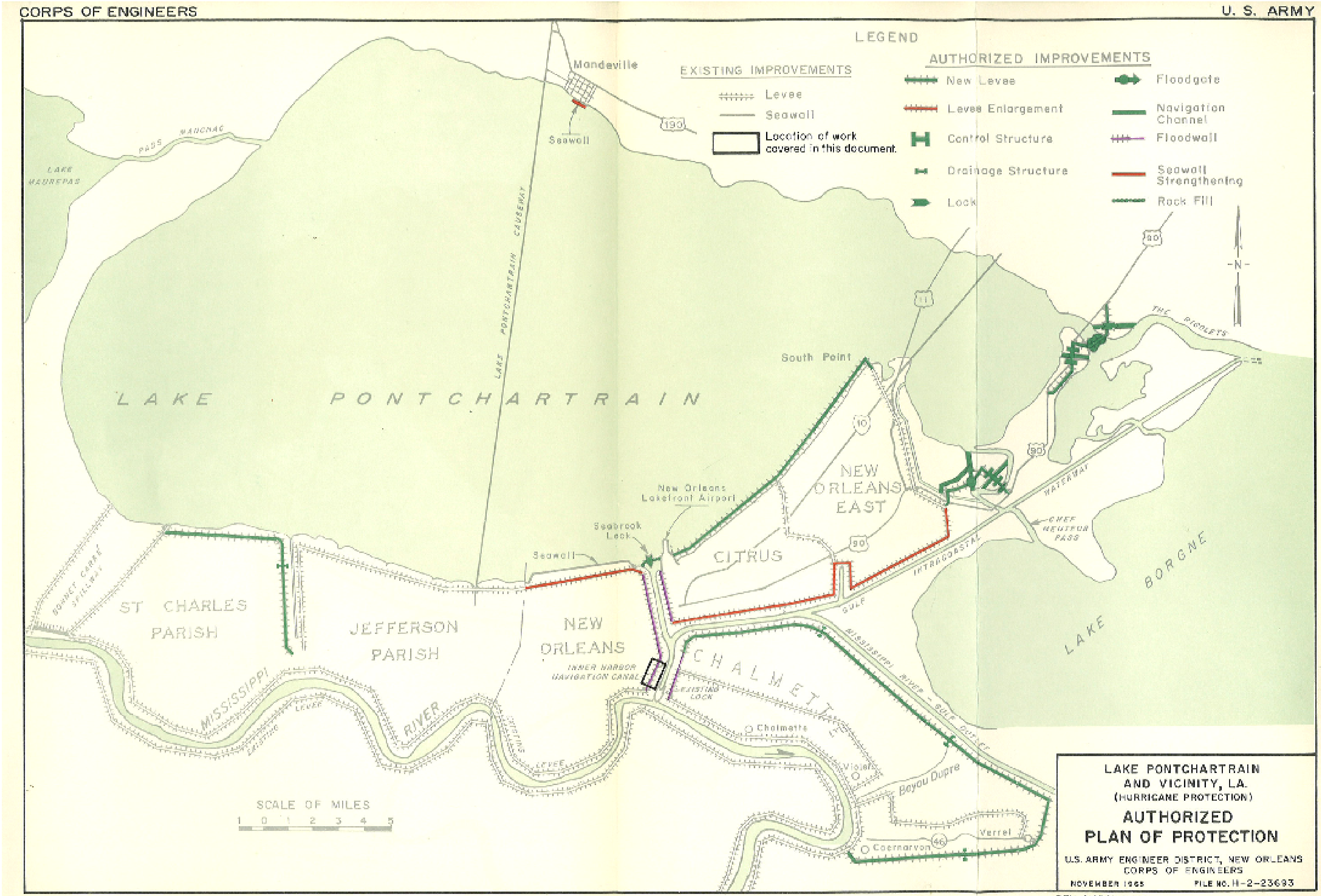
Unit Weights

Item	*lb/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


CORPS OF ENGINEERS
U. S. ARMY
LEGEND
EXISTING IMPROVEMENTS
Levee
Seawall
Location of work covered in this document.
AUTHORIZED IMPROVEMENTS
New Levee
Levee Enlargement
Control Structure
Drainage Structure
Lock
Floodgate
Navigation Channel
Floodwall
Seawall Strengthening
Rock Fill
LAKE PONTCHARTRAIN
LAKE MAUREPAS
PASS MANCHAC
Mandeville
Seawall
LAKE PONTCHARTRAIN CAUSEWAY
South Point
NEW ORLEANS EAST
New Orleans Lakefront Airport
Seabrook Lock
Seawall
CITRUS
ST CHARLES PARISH
JEFFERSON PARISH
NEW ORLEANS
INNER HARBOR NAVIGATION CANAL
EXISTING LOCK
CHALMETTE
Chalmette
Violet
Bayou Dupre
Verret
Caernarvon
BONNET CARRE SPILLWAY
MISSISSIPPI RIVER
GULF INTRACOASTAL WATERWAY
MISSISSIPPI RIVER - GULF OUTLET
CHEF MENTEUR PASS
THE RIGOLETS
LAKE BORGNE
-N-
SCALE OF MILES
1 0 1 2 3 4 5
LAKE PONTCHARTRAIN AND VICINITY, LA.
(HURRICANE PROTECTION)
AUTHORIZED PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS
NOVEMBER 1965
FILE NO. H-2-23693
REV. MARCH 1967

Design Water Elevations

	Floodside	*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 lb/sq ft
On overhead beams.. 50 lb/sq ft

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	
f'_c	3,000 psi
f_c	1,050 psi
v_c (without web reinforcement)	60 psi
v_c (with web reinforcement)	274 psi
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 psi

I-Wall Design. The penetration for I-walls was determined based on the S-Case and with a FS of 1.5. Water was assumed at 6 in. below the top of wall on the floodside and at 0.0 ft NGVD on the protected side. Bending moments and deflections were determined by applying a FS of 1.0 to the soil parameters. LMVD in the 1st Ind of 13 April 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 2nd Ind of 31 May 1967, New Orleans District 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 ft NGVD on floodside and water at elevation 0.0 ft NGVD on protected side. Sheet-pile cutoff pervious. Uplift varies uniformly from full head on floodside to tailwater on protected side.
Case II	Same as Case I except sheet-pile cutoff impervious. Uplift full head on floodside of cutoff and tailwater on protected side.
Case III	Water at elevation 10.0 ft NGVD on floodside and water at elevation 0.0 ft NGVD 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 ft NGVD on floodside and water at elevation 0.0 ft NGVD 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 percent 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.)

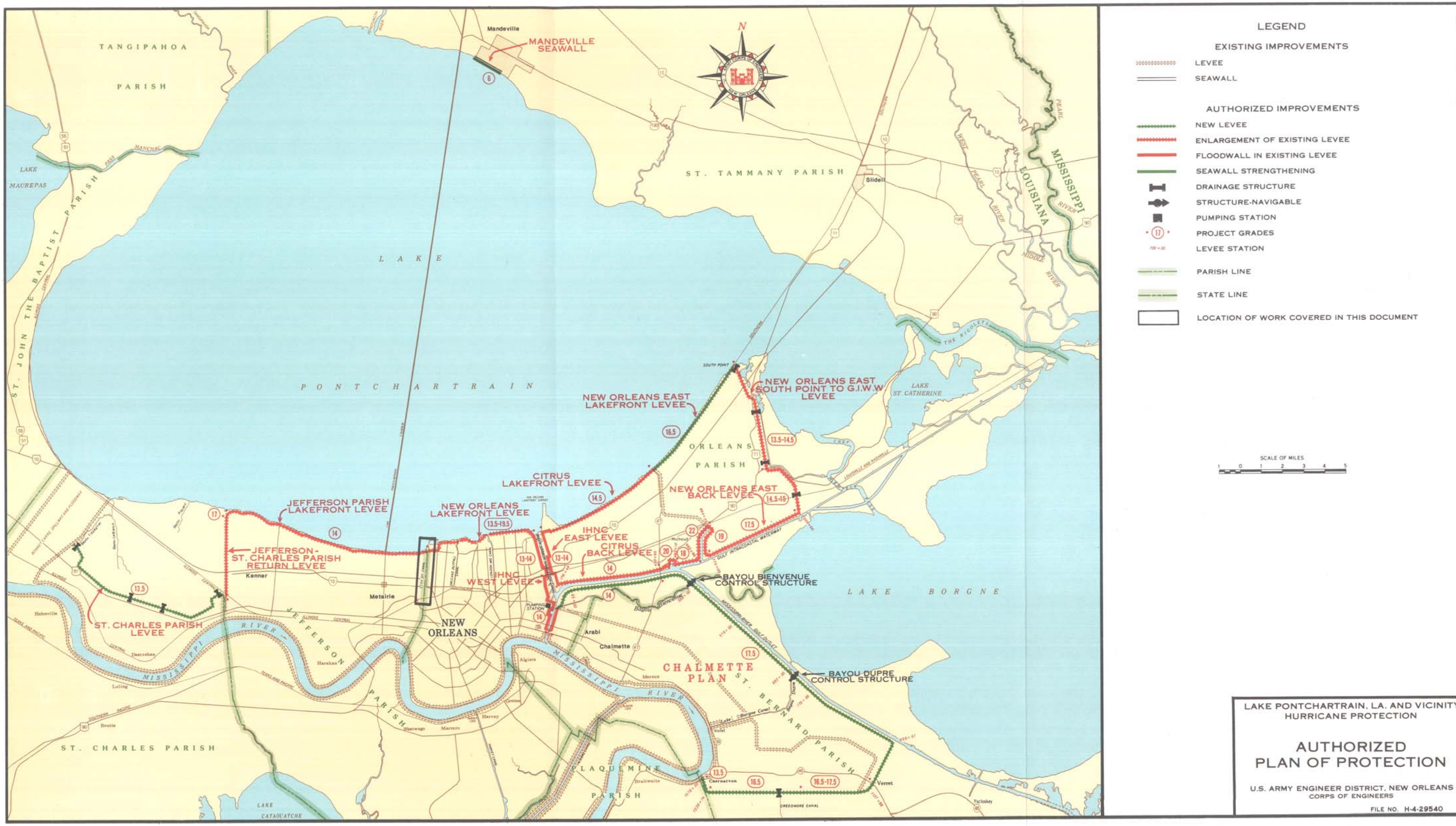
- "Design of Pile Foundations," by G. Vetter, *Transactions*, ASCE Vol. 104 (1939). (Used for checking the layout with two batter piles.)

- "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 T-type walls.

17th Street Outfall Canal (Metairie Relief) – Reference 2. Map of area is shown on following page.

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 NOS & WB Drainage Pumping Station No. 6. All hurricane protection construction was completed along the canal.


LEGEND
EXISTING IMPROVEMENTS
LEVEE
SEAWALL
AUTHORIZED IMPROVEMENTS
NEW LEVEE
ENLARGEMENT OF EXISTING LEVEE
FLOODWALL IN EXISTING LEVEE
SEAWALL STRENGTHENING
DRAINAGE STRUCTURE
STRUCTURE-NAVIGABLE
PUMPING STATION
PROJECT GRADES
LEVEE STATION
PARISH LINE
STATE LINE
LOCATION OF WORK COVERED IN THIS DOCUMENT
SCALE OF MILES
LAKE PONTCHARTRAIN, LA. AND VICINITY HURRICANE PROTECTION
AUTHORIZED PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS CORPS OF ENGINEERS
FILE NO. H-4-29540
MANDEVILLE SEAWALL
NEW ORLEANS EAST LAKEFRONT LEVEE
NEW ORLEANS EAST SOUTH POINT TO G.I.W.W LEVEE
CITRUS LAKEFRONT LEVEE
NEW ORLEANS EAST BACK LEVEE
JEFFERSON PARISH LAKEFRONT LEVEE
NEW ORLEANS LAKEFRONT LEVEE
JEFFERSON-ST. CHARLES PARISH RETURN LEVEE
IHNC EAST LEVEE
CITRUS BACK LEVEE
IHNC WEST LEVEE
BAYOU BIENVENUE CONTROL STRUCTURE
ST. CHARLES PARISH LEVEE
BAYOU DUPRE CONTROL STRUCTURE
CHALMETTE PLAN
NEW ORLEANS
LAKE PONTCHARTRAIN
LAKE BORGNE
LAKE MAUREPAS
LAKE ST CATHERINE
TANGIPAHOA PARISH
ST. TAMMANY PARISH
ST. JOHN THE BAPTIST PARISH
ORLEANS PARISH
JEFFERSON PARISH
ST. CHARLES PARISH
ST. BERNARD PARISH
PLAQUEMINE PARISH
MISSISSIPPI
LOUISIANA
MISSISSIPPI RIVER
Mandeville
Slidell
Kenner
Metairie
Arabi
Chalmette
Hahnville
Luling
Boutte
Harahan
Algiers
Gretna
Harvey
Westwego
Marrero
Verret
Caernarvon
Ycloskey
LAKE CATAOUATCHE
THE RIGOLETS
PEARL RIVER

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/cu ft**
Water (brackish)?	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-day 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:

f'_c	3,000 psi
fy (Grade 60)	48,000 psi
Maximum flexural reinforcement	0.25 x balance ratio
Minimum flexural reinforcement	200/fy
f_c (for bridge superstructure concrete)	4,000 psi
f_c (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 FS = 1.5
- Case II: Water to swl + 2 ft freeboard, Q-Case FS = 1.0

- Penetration to head ratio equal to 2.5:1

Bulkhead Analysis

- Canal water at elevation 0.0 ft, S-Case FS = 1.5

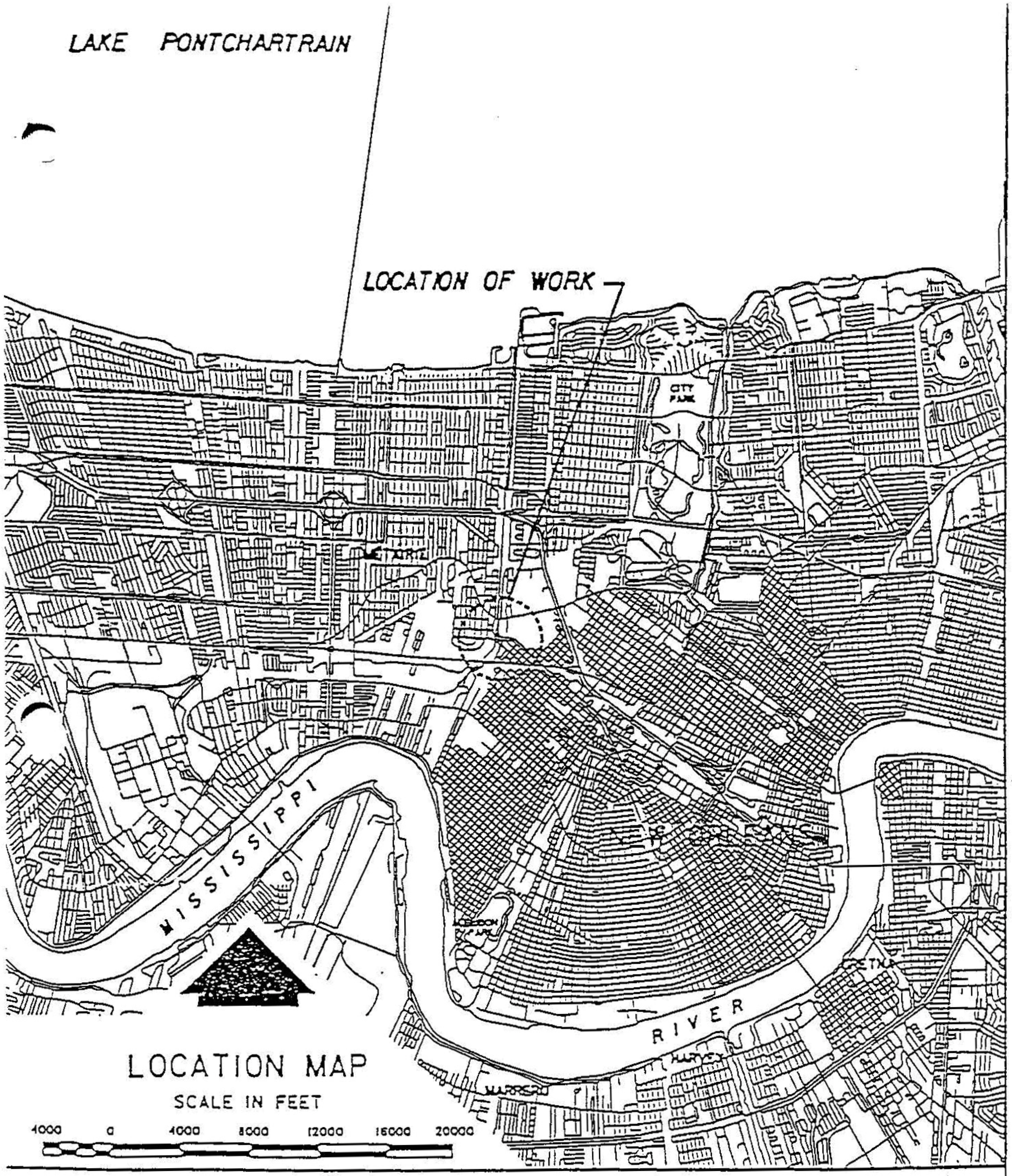
Floodgates and Gate Monoliths The foundation piles for the gate monoliths were designed with a FS of 3). Because of the small number of piles, pile tests were not considered to be economical for this work. The following load cases were used for the preliminary design of these gates.

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

Improvements to the Fronting Protection at Pumping Station No. 6, 17th Street Outfall Canal – Reference 34. Map of area and plan view shown on following pages.

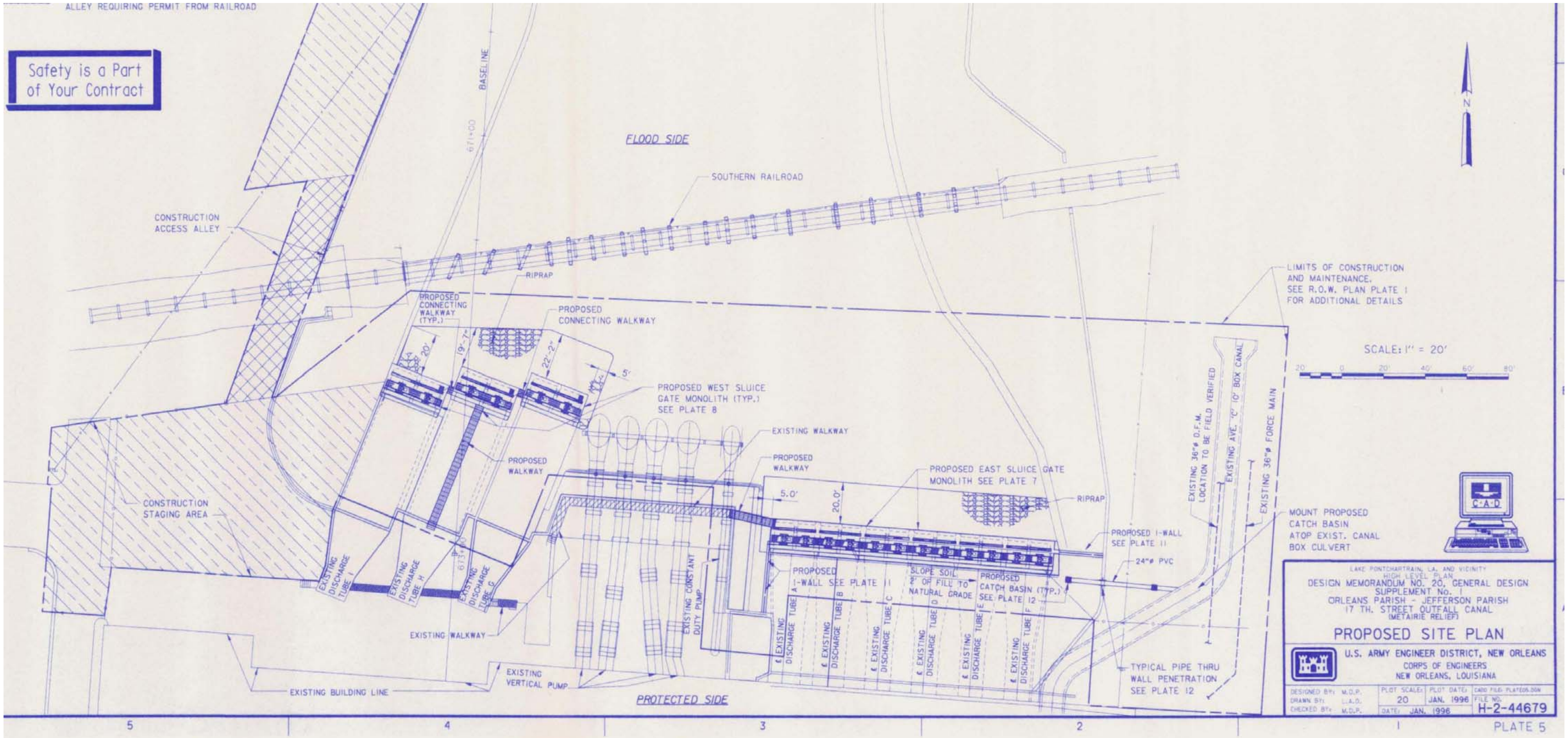
General. As constructed, fronting protection of Pumping Station No. 6 consists of pile-founded T-wall, pile-founded sluice gates for horizontal and wood screw pumps, and butterfly valves for vertical pumps to prevent water backflow through the pumps when pump operations cease due to high water levels in Lake Pontchartrain.

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



LOCATION MAP
FRONTING PROTECTION AT NOS&WB PUMP STA. NO. 6
17TH. STREET OUTFALL CANAL

FIG. 11



ALLEY REQUIRING PERMIT FROM RAILROAD
Safety is a Part of Your Contract
BASELINE
FLOOD SIDE
SOUTHERN RAILROAD
CONSTRUCTION ACCESS ALLEY
RIPRAP
PROPOSED CONNECTING WALKWAY (TYP.)
PROPOSED CONNECTING WALKWAY
LIMITS OF CONSTRUCTION AND MAINTENANCE. SEE R.O.W. PLAN PLATE 1 FOR ADDITIONAL DETAILS
SCALE: 1" = 20'
PROPOSED WEST SLUICE GATE MONOLITH (TYP.) SEE PLATE 8
EXISTING WALKWAY
PROPOSED WALKWAY
PROPOSED EAST SLUICE GATE MONOLITH SEE PLATE 7
EXISTING 36"φ D.F.M. LOCATION TO BE FIELD VERIFIED
EXISTING AVE. "C" 10' BOX CANAL
EXISTING 36"φ FORCE MAIN
CONSTRUCTION STAGING AREA
MOUNT PROPOSED CATCH BASIN ATOP EXIST. CANAL BOX CULVERT
PROPOSED I-WALL SEE PLATE 11
24"φ PVC
EXISTING DISCHARGE TUBE I
EXISTING DISCHARGE TUBE H
EXISTING DISCHARGE TUBE G
EXISTING CONSTANT DUTY PUMP
PROPOSED I-WALL SEE PLATE 11
SLOPE SOIL 2' OF FILL TO NATURAL GRADE
PROPOSED CATCH BASIN (TYP.) SEE PLATE 12
EXISTING DISCHARGE TUBE A
EXISTING DISCHARGE TUBE B
EXISTING DISCHARGE TUBE C
EXISTING DISCHARGE TUBE D
EXISTING DISCHARGE TUBE E
EXISTING DISCHARGE TUBE F
EXISTING WALKWAY
EXISTING VERTICAL PUMP
EXISTING BUILDING LINE
PROTECTED SIDE
TYPICAL PIPE THRU WALL PENETRATION SEE PLATE 12
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 20, GENERAL DESIGN
SUPPLEMENT No. 1
ORLEANS PARISH - JEFFERSON PARISH
17 TH. STREET OUTFALL CANAL
(METAIRIE RELIEF)
PROPOSED SITE PLAN
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
DESIGNED BY: M.D.P.
DRAWN BY: L.A.D.
CHECKED BY: M.D.P.
PLOT SCALE: 20
PLOT DATE: JAN. 1996
DATE: JAN. 1996
H-2-44679
PLATE 5

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

	Minimum Factor of Safety	
Loading Condition	**With Pile Load Test**	**Without 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 ft 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	lb/cu ft
Water (brackish)	64.0
Concrete	150.0
Steel	490.0
Riprap	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 percent, 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 stress for sheet piling used is based on an allowable stress of 18,000 psi plus allowable overstress if applicable.

Overstresses. An allowable overstress of 33-1/3 percent is permitted for construction, maintenance, 2 ft above still water, and flood on protected side conditions.

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

Item	lb/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 (HW) represents stages on the floodside 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 percent forces used)
- Case II (swl) HW elevation = 12.6 ft, gate closed with water in tube , TW2 elevation = 3.9***, d***ead load live load, wind load, backfill on monolith, impervious cutoff wall (100 percent forces used)
- Case III (swl) HW elevation = 12.6 ft, gate closed with water in tube, TW2 elevation = 3.9**,** dead load, live load, wind load, backfill on monolith, impervious cutoff wall (100 percent forces used)
- Case IV (normal operating) HW elevation = 2.0 ft, gate open, dead load, live load, backfill on monolith, impervious cutoff wall (100 percent force used)
- Case V (maintenance) HW elevation = 2.0 ft, stop logs in place, monolith dewatered, dead load, live load, backfill on monolith, impervious cutoff wall (75 percent forces used)
- Case VI (2 ft above swl) HW elevation = 14.6 ft, gate closed with water in tube, TW2 elevation =3.9 ft**,** dead load, live load, wind load, backfill on monolith, pervious cutoff wall (75 percent forces used)

- Case VII (2 ft above swl) HW elevation = 14.6 ft, gate closed with water in tube, TW2 elevation =3.9 ft**,** dead load, live load, wind load, backfill on monolith impervious cutoff wall (75 percent forces used)
- Case VIII (flood on protected side) HW elevation = -5.01 ft, TWI elevation = 14.6 ft, dead load, live load, backfill on monolith, impervious cutoff wall (75 percent 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 percent forces used)
- Case II (swl) HW elevation = 12.6 ft, TW1 elevation = 12.6 ft, gate closed with water in tube, TW2 elevation = 5.0 ft, dead load, live load, wind load, backfill on monolith, uniform uplift pressure (100 percent forces used)
- Case III (normal operating) HW elevation = 2.0 ft, TW1 elevation = 2.0 ft, gate open, dead load, live load, backfill on monolith, uniform uplift pressure (100 percent force used)
- Case IV (maintenance) HW elevation = 2.0 ft, TW1 elevation = 2.0 ft, stop logs in place, monolith dewatered, dead load, live load, backfill on monolith, uniform uplift pressure (75 percent forces used)
- Case V (2 ft above swl) HW elevation = 14.6ft, TW1 elevation = 14.6 ft, gate closed with water in tube, TW2 elevation = 5.0 ft, dead load, live load, wind load, backfill on monolith, uniform uplift pressure (75 percent forces used)

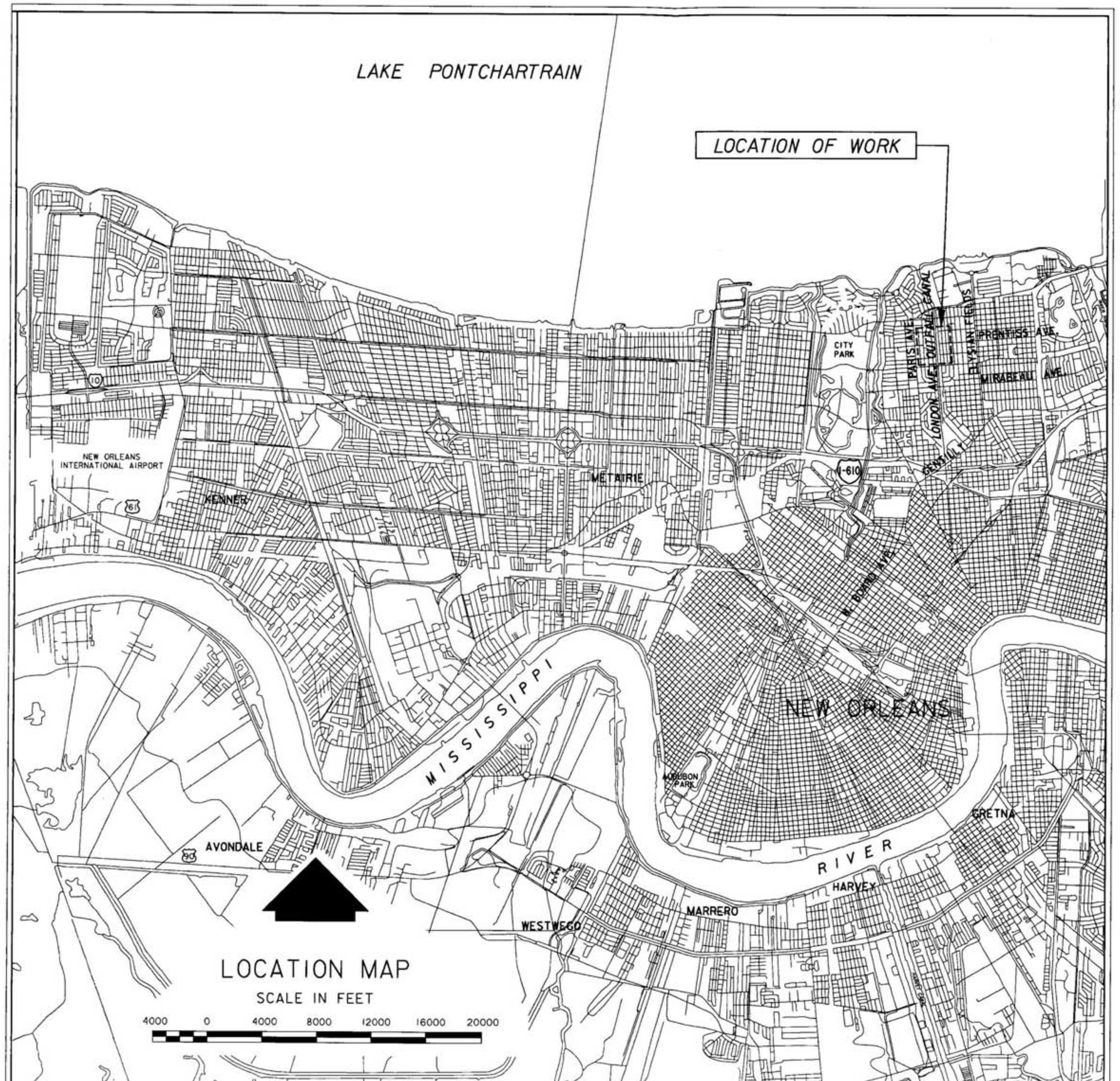
The groundwater elevation causing uplift at the west monoliths shall be the same as the floodside since floodwaters are allowed to surround these monoliths.

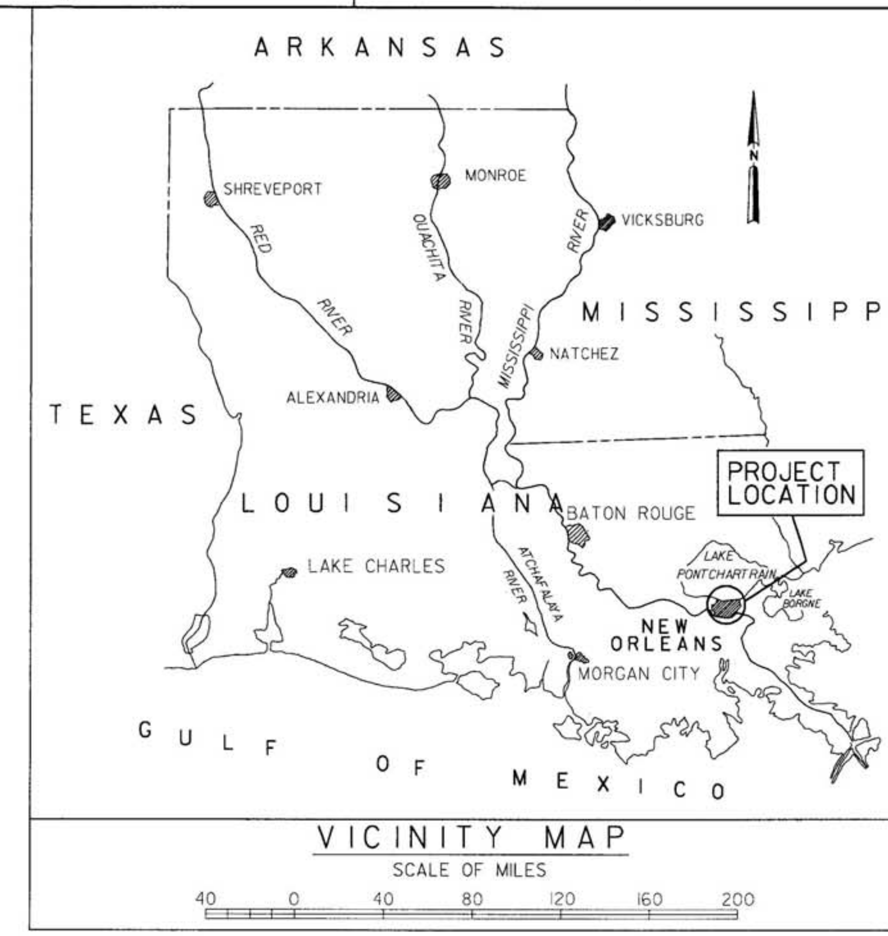
East I-Wall at East Monolith (2 ft above swl). HW = 14.6 ft, TW1 = 3.8 ft, ground elevation 3.8 ft on floodside and protected side.

West I-Wall at East Monolith (Min. Water with Backfill). HW = 3.0 ft, ground elevation 3.0 ft on floodside, ground elevation 14.0 ft on protected side.

London Avenue Outfall Canal - Reference 5, Supplemented by References 35 and 75. Images of area shown on following pages.


LAKE PONTCHARTRAIN
LAKESHORE DR.
UNIVERSITY OF NEW ORLEANS
PLATE 2
ROBERT E. LEE BLVD.
MARCONI DR.
FILMORE AVE.
HARRISON AVE.
BONNABEL BLVD.
METAIRIE RELIEF OUTFALL CANAL
PONTCHARTRAIN BLVD.
WEST END BLVD.
CANAL BLVD.
ORLEANS AVE. OUTFALL CANAL
BAYOU ST. JOHN
WISNER BLVD.
CITY PARK
LONDON AVE. OUTFALL CANAL
ELYSIAN FIELDS AVE.
FRANKLIN AVE.
VETERANS MEMORIAL HWY.
10
610
90
METAIRIE RD.
JEFFERSON PARISH
ORLEANS PARISH
New Orleans
DILLARD UNIVERSITY
N
INDEX MAP
SCALE IN FEET
ARKANSAS
LOUISIANA
MISSISSIPPI
TEXAS
LOCATION OF WORK
GULF OF MEXICO
VICINITY MAP
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A
GENERAL DESIGN
LONDON AVE. OUTFALL CANAL
INDEX AND VICINITY MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
DATE JAN. 1989
FILE NO. H-2-30288
PLATE 1





NOTE:
DRAWINGS IN THIS FOLIO HAVE BEEN REDUCED ONE HALF THE ORIGINAL SCALE

LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 19A - GENERAL DESIGN
LONDON AVE. OUTFALL CANAL - SUPPLEMENT NO. 1
FRONTING PROTECTION PUMPING STATION NO. 4
ORLEANS PARISH

LOCATION AND VICINITY MAP

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY:	M.W.W.	PLOT SCALE: 4800	PLOT DATE: 9 JAN 95	CADD FILE: 4051	NOLDGN
DRAWN BY:	V.A.C.			FILE NO.
CHECKED BY:	W.O.B.	DATE: JAN 1995		H-2-40511

PLATE 1

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. 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 NOS & WB Drainage Pumping Station No. 4. Fronting protection of Pumping Station No. 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 NOS & WB Drainage Pumping Station No. 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

f'_c	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
f'_c (prestressed concrete piles)	5,000 psi
fu (prestressing strands Grade 270)	270,000 psi

Unit Weights

Item	*lb/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 swl, plus 2 ft of freeboard and 6 in. of projected settlement.

Station Limits	***Design Grade***
0 + 00 to 120 + 08	elevation 14.4
120 + 42 to 127 + 15	elevation 14.1
127 + 85 to 152 + 50	elevation 14.0
152 + 50 to 158 + 50	Transition from elevation 14.0 to 18.5 ft, West Side
	Transition from elevation 14.0 to 18.0 ft, East Side
158 + 50 to 159 + 70	elevation18.5 ft, West Side
	elevation18.0 ft, East Side

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

Case I	Q-Case with water to swl and FS = 1.5
Case II	Q-Case with water to swl + 2 ft of freeboard (top of wall) and FS = 1.0
Case III	S-Case with water to swl and wave load (where applicable) with FS = 1.2
Case IV	Q-Case with water to swl and wave load with 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, Paragraph 31, the sheet-pile wall design criteria was summarized as follows:

Q-Case

FS = 1.5 .... with water to swl
FS = 1.25 .... with water to swl and wave load (if applicable)
FS = 1.0 .... with water to swl + 2 ft freeboard

S-Case

FS = 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

FS = 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 Boulevard. (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 percent forces used)
Case II	Gate closed, static water pressure to swl, no wind, pervious sheet-pile cutoff, no dynamic wave force (100 percent forces used)
Case III	Gate closed , static water pressure with water level 2 ft above swl, no wind, impervious sheet-pile cutoff, no dynamic wave force (75 percent forces used
Case IV	Gate closed , static water pressure with water level 2 ft above swl, no wind, pervious sheet-pile cutoff, no dynamic wave force (75 percent forces used
Case V	Gate open, no wind, truck or train on land side edge of base slab (100 percent forces used)
Case VI	Gate open, no wind, truck or train on canal side edge of base slab (100 percent forces used)
Case VII	Gate open, wind from floodside, train on canal side edge of base slab (75 percent forces used)
Case VIII	Gate open, wind from floodside, train on land side edge of base slab (75 percent forces used).

London Avenue Outfall Canal, Design Memorandum No. 19A, General Design, Supplement No. 1, Fronting Protection, Pumping Station No. 4, December 1994

Material weights

Item	*lb/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 percent, 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.

f'_c	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:

$$f_a/Fa + f_{bx}/Fb + f_{by}/Fb <= 1.0$$

where:

f_a = computed axial unit stress
Fa = (0.833) * (0.600) * fy = 18.0 ksi (ASTM A36)
f_{bx}, f_{by} = computed bending unit stress
Fb = (0.833) * (0.600) * fy = 20.0 ksi (ASTM A36; compact)

Loading Conditions

General. The SPH level is elevation 11.9 ft 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, elevation 13.9 ft NGVD. An extreme loading condition was used only for the 1,000-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 elevation 11.9 ft NGVD, swl inside discharge culverts elevation 8.0 ft NGVD, storm wind load, impervious sheet-pile cutoff
- Gate closed, canal swl elevation 11.9 ft NGVD, swl inside discharge culverts elevation 8.0 ft NGVD, storm wind load, pervious sheet-pile cutoff

Unusual conditions:[1]

- Gate closed, canal swl elevation 13.9 ft NGVD, swl inside discharge culverts elevation 8.0 ft NGVD, storm wind load, impervious sheet-pile cutoff
- Gate closed, canal swl elevation 13.9 ft NGVD, swl inside discharge culverts elevation 8.0 ft NGVD, storm wind load, pervious sheet-pile cutoff

Maintenance dewatering conditions:

- Dewatering stop logs installed, canal swl elevation 4.0 ft NGVD, swl inside discharge culverts elevation -11.0 ft NGVD, operating wind load, impervious sheet-pile cutoff
- Dewatering stop logs installed, canal swl elevation 4.0 ft NGVD, swl inside discharge culverts elevation -11.0 ft 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 elevation 11.9 ft NGVD, swl inside discharge culverts elevation 1.0 ft NGVD, storm wind load, impervious sheet-pile cutoff.

Gated discharge basin for 320-cfs pumps

Usual conditions:

- Gate closed, canal swl elevation 11.9 ft NGVD, swl inside discharge culverts elevation 3.57 ft NGVD, storm wind load, impervious sheet-pile cutoff
- Gate closed, canal swl elevation 11.9 ft NGVD, swl inside discharge culverts elevation 3.57 ft 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 percent and no overstressing in the foundation piles was allowed. This method was used in lieu of no load reduction and 33-1/3 percent overstress.

Unusual conditions:[1]

- Gate closed, canal swl elevation 13.9 ft NGVD, swl inside discharge culverts elevation 3.57 ft NGVD, storm wind load, impervious sheet-pile cutoff

- Gate closed, canal swl elevation 13.9 ft NGVD, swl inside discharge culverts elevation 3.57 ft NGVD, storm wind load, pervious sheet-pile cutoff

Maintenance dewatering conditions:

- Dewatering stop logs installed, canal swl elevation 4.0 ft NGVD, swl inside discharge culverts elevation -8.5 ft NGVD, operating wind load, impervious sheet-pile cutoff

- Dewatering stop logs installed, canal swl elevation 4.0 ft NGVD, swl inside discharge culverts elevation -8.5 ft 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 elevation 11.9 ft NGVD, storm wind load, impervious sheet-pile cutoff

- Gate closed, canal swl elevation 11.9 ft NGVD, storm wind load, pervious sheet-pile cutoff

Unusual conditions:

- Gate closed, canal swl elevation 13.9 ft NGVD, storm wind load, impervious sheet-pile cutoff

- Gate closed, canal swl elevation 13.9 ft NGVD, storm wind load, pervious sheet-pile cutoff

Construction conditions:

- No hydrostatic load, operating wind load from floodside

- No hydrostatic load, no wind load, dead load only

I-Wall monoliths

Usual conditions: Q-Case (soil FS = 1.5) with swl elevation 11.9 ft NGVD

---

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

Unusual conditions: Q-Case (soil FS = 1.0) with swl elevation 13.9 ft 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 FS of 1.5, while the required sheet-pile section was based on a soil strength factor of 1.0 and a steel strength FS of 2.0.

IHNC West Levee, France Road Terminal Relocation of IHNC Flood Protection, Reference 36. Image of area shown on following page.

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

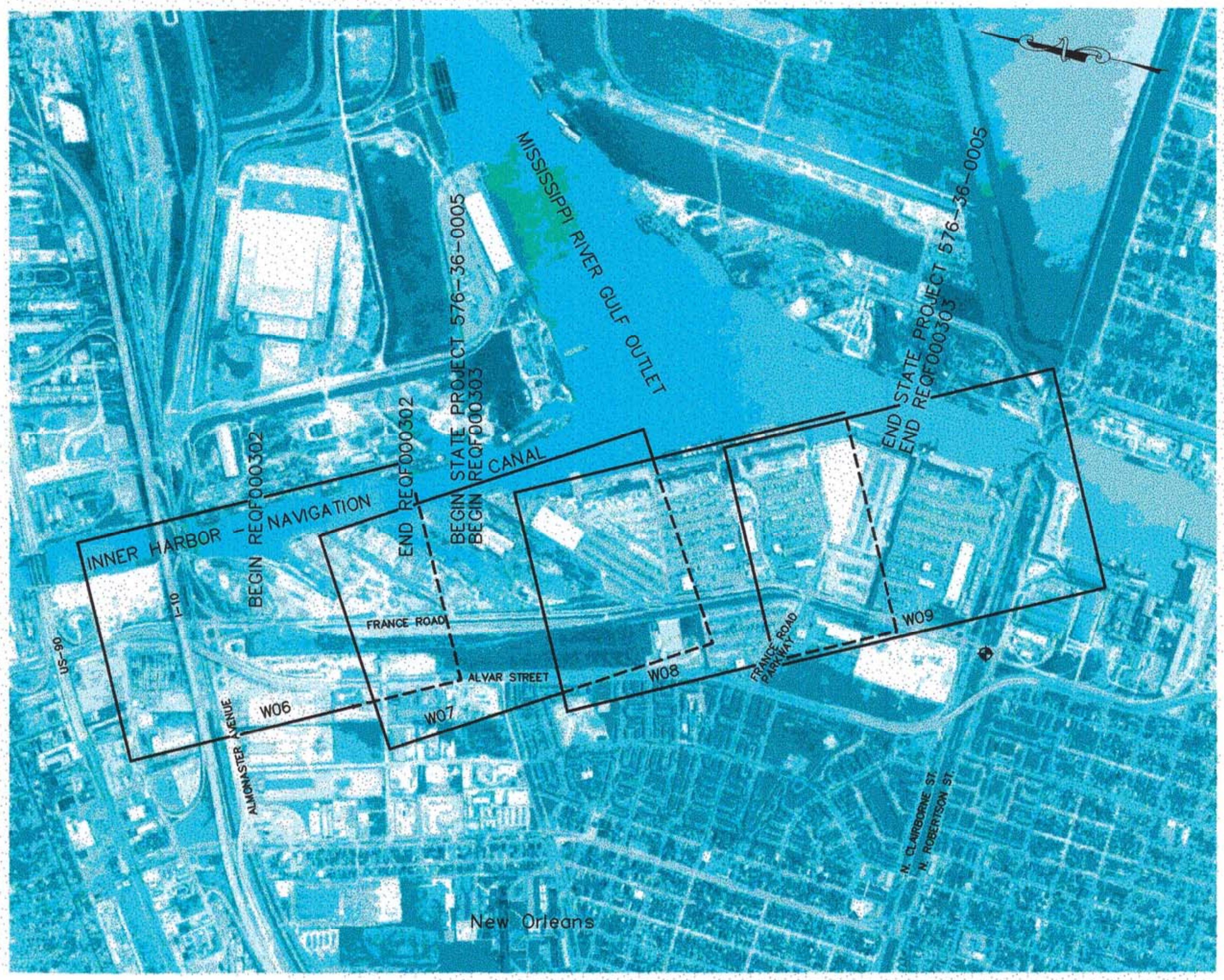
Floodwall Gross Grade	*Elevations (ft NGVD)*
I-Wall	15.5
T-Wall	15.0
Cofferdam	16.0

Unit Weights

Water	62.4 lb/cu ft
Concrete	150 lb/cu ft
Steel	490 lb/cu ft
Saturated soil	115 lb/cu ft

Design Loads

Wind loads	50 lb/sq ft
Live loads	AASHTO Special forklift loads



PROJECT BENCHMARK
S-189 (1991)
EL. 2.110 N.G.V.D.

NOTE:
ALL ELEVATIONS OF EXISTING FEATURES MAY BE OFF BY (2.247−2.110 =0.137) 0.14 DUE TO A CHANGE IN THE B.M. ELEV. WHEN THE SURVEY WAS MADE AND THE PRESENT B.M. ELEV. 2.110.

NOTE:
ELEVATIONS FOR ALL I-WALLS AND BERMS SHOWN IN THIS MEMORANDUM ARE CONSTRUCTION ELEVATIONS, DESIGN ELEVATIONS ARE 0.5' LOWER.

SCALE: 1"=600'
600' 0 600' 1200' 1800'

LAKE PONTCHARTRAIN, LOUISIANA AND VICINITY
HURRICANE PROTECTION PROJECT
DESIGN MEMORANDUM NO. 2 GENERAL DESIGN
SUPPLEMENT NO. 8A
RELOCATION OF I.H.N.C. FLOOD PROTECTION
FRANCE ROAD TERMINAL
NEW ORLEANS, LOUISIANA

ALIGNMENT DWG INDEX SHEET

SUBMITTED TO: U.S. ARMY CORPS OF ENGINEERS, NEW ORLEANS, DISTRICT
AND
BOARD OF COMMISSIONERS OF THE ORLEANS LEVEE DISTRICT
SUBMITTED BY: BOARD OF COMMISSIONERS, PORT OF NEW ORLEANS, LA
PREPARED BY: PYBURN & ODOM, INC., BATON ROUGE, LA.

DESIGNED BY: H.C.D.	PLOT SCALE: 1"=600'	PLOT DATE: 10/97	CADD FILE: INDEX
DRAWN BY: L.P.C.			FILE NO. 504-005
CHECKED BY: H.C.D.	DATE: 10/97		

PLATE W1

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 30 June 1992, except prestressed concrete piling for which the minimum is 5,000 psi. Pertinent stresses are tabulated below:

f'_c	3,000 psi
fy (Grade 60)	60,000 psi
Maximum flexural reinforcement ratio	0.375 x balance ratio
Minimum flexural reinforcement ratio	200/fy
f'_c (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 lb/sq ft
Truck loading – HS20-16, latest AASHTO specifications
Forklift loading – KALMAN LMV
Crane load – BUCKNES 88B
Wind load – 50 lb/sq ft (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 swl (elevation +13.0 ft NGVD), Q-Case FS = 1.5; S-Case FS = 1.3
Case II	Water at net design grade (elevation +15.0 ft NGVD), Q-Case FS = 1.0; S-Case FS = 1.0

Minimum penetration to head ratio of 3:1 used, where the head is at swl.

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-in. square prestressed concrete piles with steel sheet piling for seepage control. Pile load tests have been performed, with the result that a FS of 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 FS of 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 elevation 13.0 ft + soil load + uplift load (impervious cutoff wall)
Case II	Wall DL + water to elevation 15.0 ft (low probability head) + soil load + uplift load (impervious cutoff) x 0.75
Case III	Wall DL + floodside wind load at 50 lb/sq ft + soil load? x 0.75
Case IV	Wall DL + protected side wind load at 50 lb/sq ft + soil load? x 0.75
Case V	Wall DL + soil load x 0.75
Case VI	Case I with pervious cutoff
Case VII	Case II with pervious cutoff

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

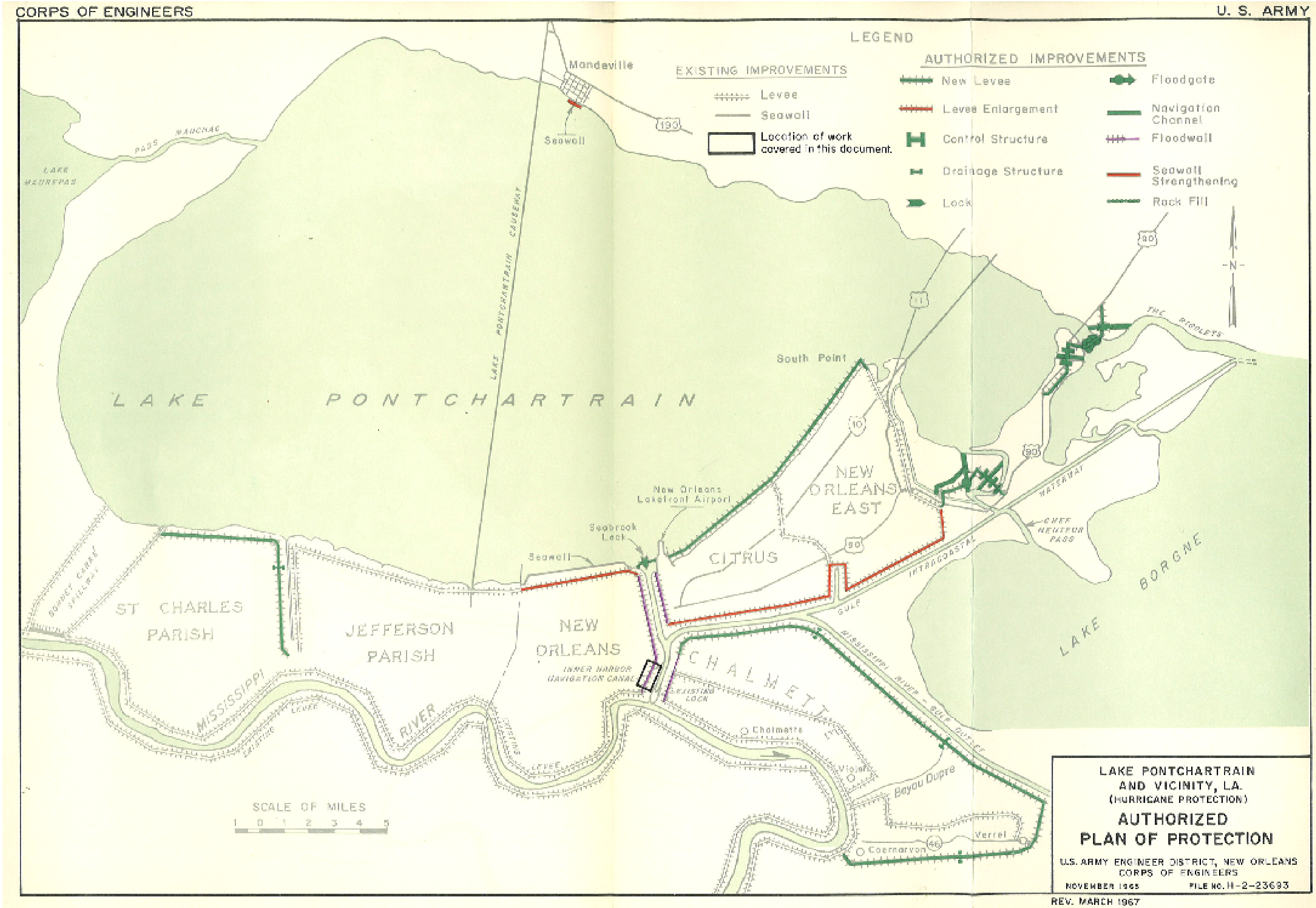
- Wind load was based upon AASHTO load as per requirement of U.S. Army Corps of Engineers.

Gate Monoliths

Case I	Gate closed; water to elevation 13.0 ft
Case II	Gate closed; water to elevation 15.0 (75 percent forces)
Case III	Gate open; two fork lifts or two HS20-16 trucks on protected side edge of base slab
Case IV	Gate open; two fork lifts or two HS20-16 trucks on floodside edge of base slab
Case V	Gate closed; wind from floodside (75 percent forces)
Case VI	Gate closed; wind from protected side (75 percent 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.

IHNC West Levee, Florida Avenue to IHNC Lock – Reference 12. Map of area shown on following page. The protective works covered herein consist of approximately 2,150 ft of “I”-type cantilever floodwall and 4,900 ft 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 at the loading platform of the Jones & Laughlin Steel Company warehouse.


CORPS OF ENGINEERS
U. S. ARMY
LEGEND
EXISTING IMPROVEMENTS
Levee
Seawall
Location of work covered in this document.
AUTHORIZED IMPROVEMENTS
New Levee
Levee Enlargement
Control Structure
Drainage Structure
Lock
Floodgate
Navigation Channel
Floodwall
Seawall Strengthening
Rock Fill
Mandeville
Seawall
LAKE MAUREPAS
PASS MANCHAC
LAKE PONTCHARTRAIN
LAKE PONTCHARTRAIN CAUSEWAY
South Point
NEW ORLEANS EAST
New Orleans Lakefront Airport
Seabrook Lock
Seawall
CITRUS
THE RIGOLETS
CHEF MENTEUR PASS
INTRACOASTAL WATERWAY
GULF
LAKE BORGNE
MISSISSIPPI RIVER - GULF OUTLET
ST CHARLES PARISH
JEFFERSON PARISH
NEW ORLEANS
INNER HARBOR NAVIGATION CANAL
EXISTING LOCK
CHALMETTE
Chalmette
Violet
Bayou Dupre
Verret
Caernarvon
BONNET CARRE SPILLWAY
MISSISSIPPI RIVER
EXISTING LEVEE
SCALE OF MILES
1 0 1 2 3 4 5
LAKE PONTCHARTRAIN AND VICINITY, LA.
(HURRICANE PROTECTION)
AUTHORIZED PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NOVEMBER 1965
FILE NO. H-2-23693
REV. MARCH 1967

Structural Design

Design Criteria

Basic Data

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

Floodwall grades	*Elevations (ft NGVD)*
Net grade (1 ft 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/cu ft*
Water	62.5
Concrete	150.0
Steel	490.0

Design Loads

Water loads

No wave force
1 ft freeboard
Design Water Elevations as follows

	Floodside (ft NGVD)	*Protected side ft NGVD*
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 lb/sq ft
On overhead beams........... 50 lb/sq ft

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

I-Wall Design. The penetration for I-walls was determined based on the S-Case and with a FS of 1.5. Water was assumed at 6 in. below the top of wall on the floodside and at elevation 0.0 ft on the protected side. Bending moments and deflections were determined by applying a FS of 1.0 to the soil parameters. LMVD in the 1st Ind of 13 April 1967 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 2nd Ind of 31 May 1967, New Orleans District 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 ft on floodside and water at elevation 0.0 ft on protected side. Sheet-pile cutoff pervious. Uplift varies uniformly from full head on floodside to tailwater on protected side.
Case II	Same as Case I except sheet-pile cutoff impervious. Uplift full head on floodside of cutoff and tailwater on protected side.
Case III	Water at elevation 10.0 ft on floodside and water at elevation 0.0 ft 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 ft on floodside and water at elevation 0.0 ft 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 percent 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 T-type walls.

IHNC West Levee and East Levee, Florida Avenue Complex, IHNC (Reference 37). Images of area are shown on following pages. Floodwall (T- and I-type) will be located along both the west and east sides of the IHNC in the vicinity of Florida Avenue. The floodwall along the west side of the IHNC 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

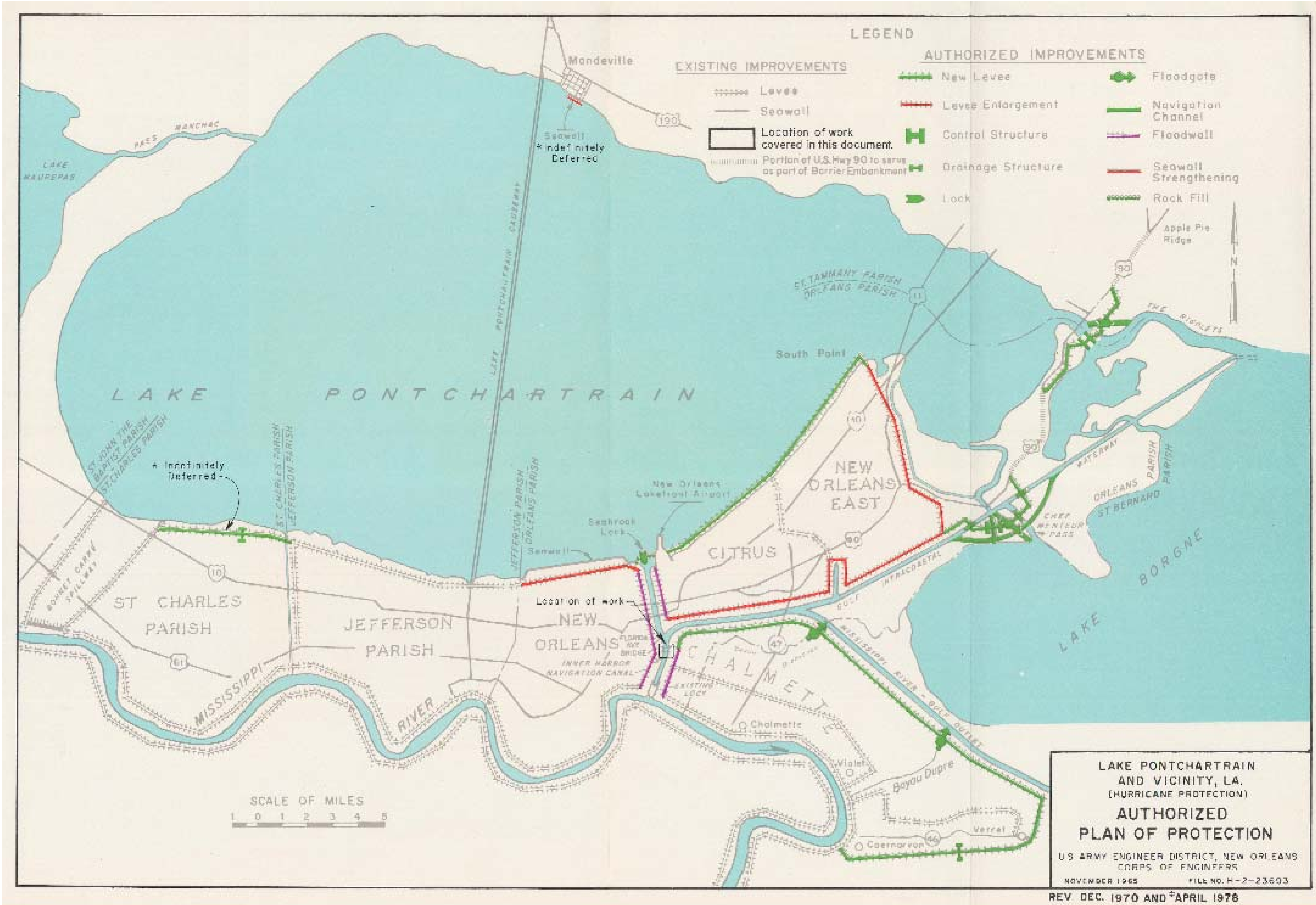
Basic data

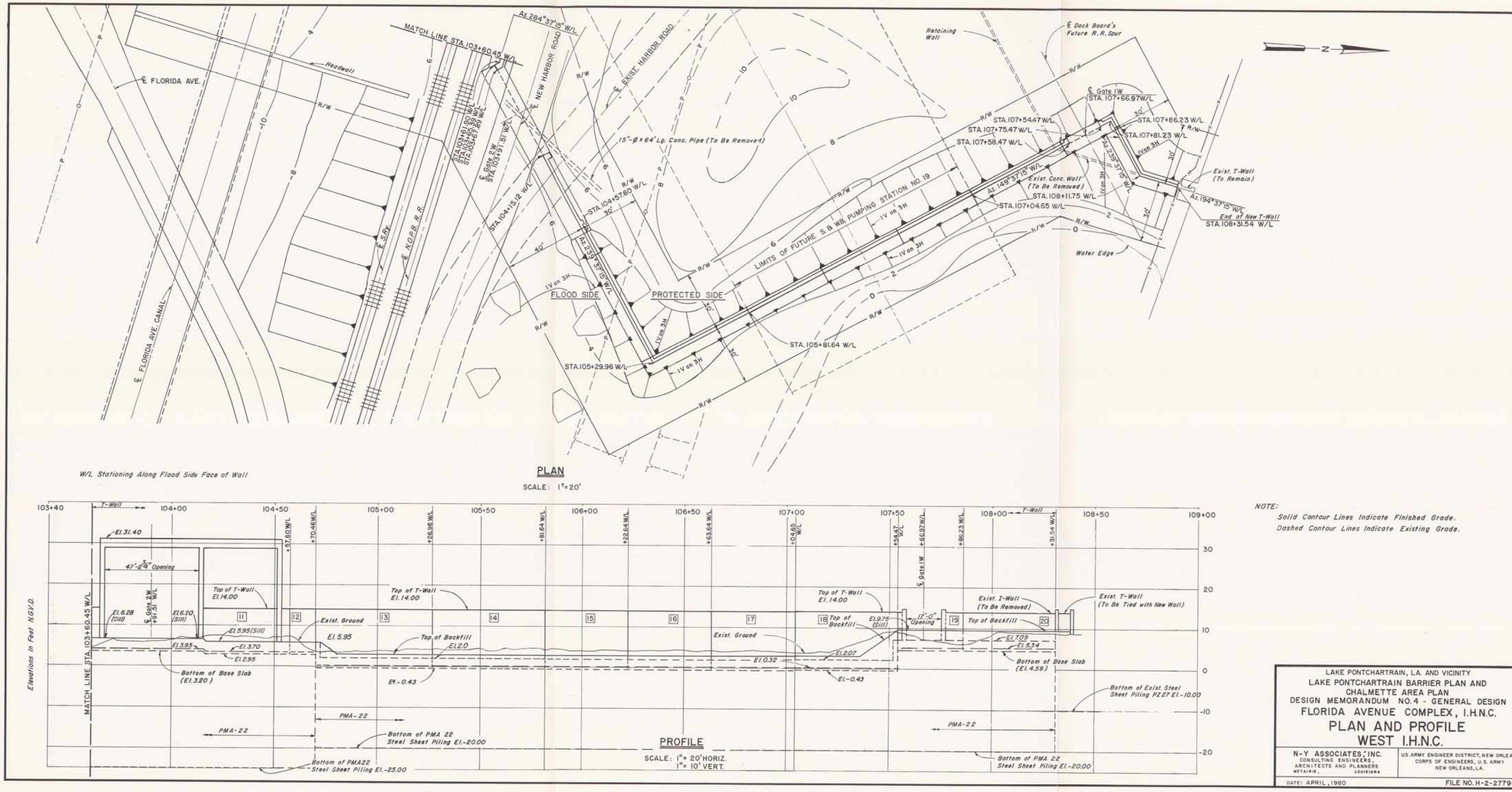
Water elevations

	West Side IHNC Elevations (ft NGVD)		*East Side IHNC Elevations (ft NGVD)*	
Case	Floodside	Protected Side	Floodside	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

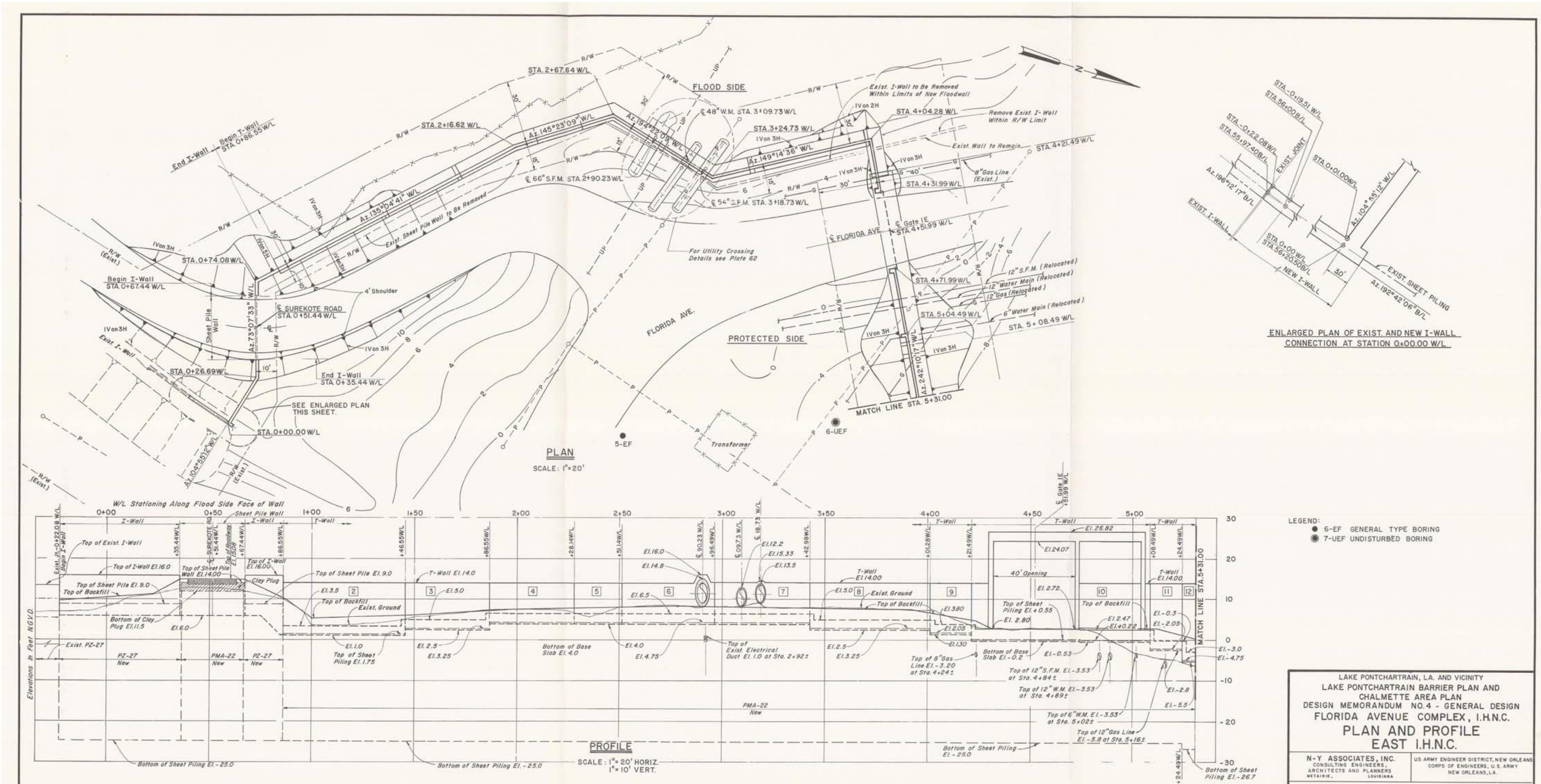
Floodwall Gross Grade	*Elevations (ft NGVD)*
West Side IHNC	
I-Wall	14.5
T-Wall	14.0
East Side IHNC	
I-Wall	16.0 (North) +15.0 (South)
T-Wall	14.0


LEGEND
EXISTING IMPROVEMENTS
Levee
Seawall
Location of work covered in this document.
Portion of U.S. Hwy 90 to serve as part of Barrier Embankment
AUTHORIZED IMPROVEMENTS
New Levee
Levee Enlargement
Control Structure
Drainage Structure
Lock
Floodgate
Navigation Channel
Floodwall
Seawall Strengthening
Rock Fill
LAKE PONTCHARTRAIN
LAKE MAUREPAS
PASS MANCHAC
Mandeville
Seawall
* Indefinitely Deferred
LAKE PONTCHARTRAIN CAUSEWAY
ST. TAMMANY PARISH
ORLEANS PARISH
South Point
Apple Pie Ridge
THE RIGOLETS
N
NEW ORLEANS EAST
CITRUS
New Orleans Lakefront Airport
Seabrook Lock
Seawall
Location of work
NEW ORLEANS
INNER HARBOR NAVIGATION CANAL
EXISTING LOCK
CHALMETTE
Chalmette
Violet
Bayou Dupre
Caernarvon
Verret
GULF INTRACOASTAL WATERWAY
MISSISSIPPI RIVER - GULF OUTLET
CHEF MENTEUR PASS
ORLEANS PARISH
ST BERNARD PARISH
LAKE BORGNE
* Indefinitely Deferred
ST JOHN THE BAPTIST PARISH
ST CHARLES PARISH
ST. CHARLES PARISH
JEFFERSON PARISH
JEFFERSON PARISH
ORLEANS PARISH
BONNET CARRE SPILLWAY
ST CHARLES PARISH
JEFFERSON PARISH
MISSISSIPPI RIVER
SCALE OF MILES
1 0 1 2 3 4 5
LAKE PONTCHARTRAIN AND VICINITY, LA.
(HURRICANE PROTECTION)
AUTHORIZED PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NOVEMBER 1965
FILE NO. H-2-23693
REV DEC. 1970 AND *APRIL 1978



PLAN
SCALE: 1"=20'
PROFILE
SCALE: 1"= 20' HORIZ. 1"= 10' VERT.
W/L Stationing Along Flood Side Face of Wall
NOTE:
Solid Contour Lines Indicate Finished Grade.
Dashed Contour Lines Indicate Existing Grade.
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN AND CHALMETTE AREA PLAN
DESIGN MEMORANDUM NO. 4 - GENERAL DESIGN
FLORIDA AVENUE COMPLEX, I.H.N.C.
PLAN AND PROFILE
WEST I.H.N.C.
N-Y ASSOCIATES, INC.
CONSULTING ENGINEERS, ARCHITECTS AND PLANNERS
METAIRIE, LOUISIANA
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS, U.S. ARMY
NEW ORLEANS, LA.
DATE: APRIL, 1980
FILE NO. H-2-27790

PLATE 3


FLOOD SIDE
PROTECTED SIDE
FLORIDA AVE.
PLAN
SCALE: 1"=20'
ENLARGED PLAN OF EXIST. AND NEW I-WALL CONNECTION AT STATION 0+00.00 W/L.
PROFILE
SCALE: 1"= 20' HORIZ. 1"= 10' VERT.
W/L Stationing Along Flood Side Face of Wall
Elevations in Feet N.G.V.D.
LEGEND:
6-EF GENERAL TYPE BORING
7-UEF UNDISTURBED BORING
MATCH LINE STA. 5+31.00
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN AND
CHALMETTE AREA PLAN
DESIGN MEMORANDUM NO. 4 - GENERAL DESIGN
FLORIDA AVENUE COMPLEX, I.H.N.C.
PLAN AND PROFILE
EAST I.H.N.C.
N-Y ASSOCIATES, INC.
CONSULTING ENGINEERS, ARCHITECTS AND PLANNERS
METAIRIE, LOUISIANA
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS, U.S. ARMY
NEW ORLEANS, LA.
DATE: APRIL, 1980
FILE NO. H-2-27790
PLATE 43

Unit Weights

Water	62.5 lb/cu ft
Concrete	150 lb/cu ft
Steel	490 lb/cu ft

Design Loads

Wind loads	50 lb/sq ft
Water loads	62.5 lb/sq ft

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 pre-stressed 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

f'_c	3,000 psi
f'_c (Florida Avenue Canal gates and conduits)	4,000 psi
f_c	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
f_c(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 FS 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 floodside, 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 ft (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 elevation 14 ft floodside, at elevation -8.5 ft protected side, impervious cutoff
Case III	Water level at elevation 14 ft floodside, at elevation -8.5 ft protected side, pervious cutoff
Case IV	Water level at elevation 4 ft floodside, at elevation -14.5 ft protected side, impervious cutoff
Case V	Water level at elevation 4 ft floodside, at elevation -14.5 ft protected side, pervious cutoff
Case VI	Water level at elevation -14 ft floodside, at elevation -3 ft protected side, impervious cutoff
Case VII	Water level at elevation -14 ft floodside, at elevation -3 ft protected side, pervious cutoff

Concrete Box Structure in Florida Avenue Canal

West Side of IHNC

Load Cases

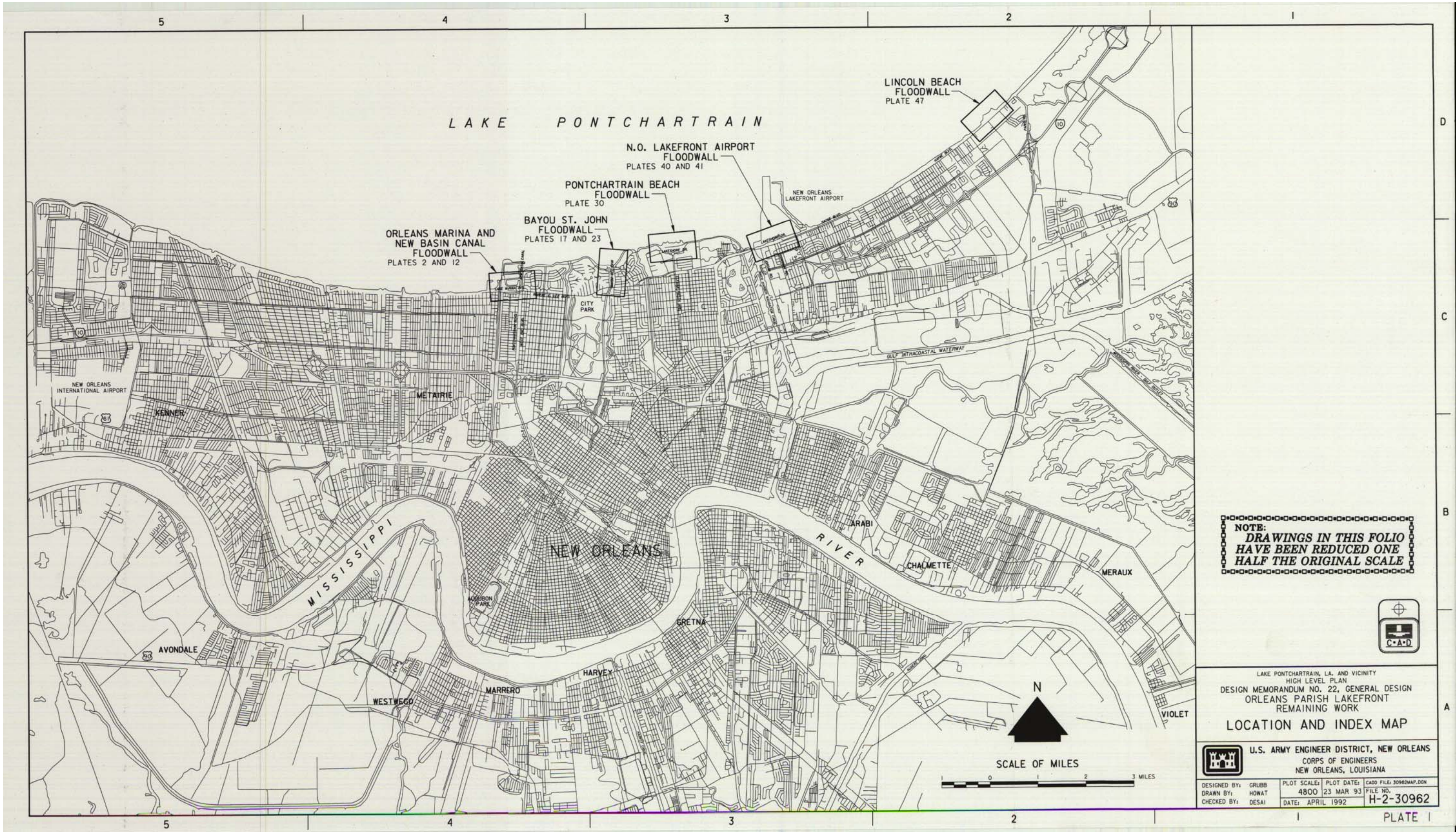
Case I	Dry inside, water at elevation -4.5 ft outside, full surcharge
Case II	Water inside, water at elevation -4.5 ft outside, full surcharge
Case III	Dry inside, water at elevation 14 ft outside, full surcharge
Case IV	Water inside, water at elevation 14 ft outside, full surcharge

East Side of IHNC

Load Cases

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

Orleans Parish Remaining Work – Reference 14. Image of area shown on following page. The project consists of raising the remaining flood protection along the south shore of Lake Pontchartrain in Orleans Parish, LA 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.


LAKE PONTCHARTRAIN
LINCOLN BEACH FLOODWALL PLATE 47
N.O. LAKEFRONT AIRPORT FLOODWALL PLATES 40 AND 41
PONTCHARTRAIN BEACH FLOODWALL PLATE 30
BAYOU ST. JOHN FLOODWALL PLATES 17 AND 23
ORLEANS MARINA AND NEW BASIN CANAL FLOODWALL PLATES 2 AND 12
NEW ORLEANS LAKEFRONT AIRPORT
CITY PARK
NEW ORLEANS INTERNATIONAL AIRPORT
METAIRIE
KENNER
GULF INTRACOASTAL WATERWAY
NEW ORLEANS
MISSISSIPPI
RIVER
ARABI
CHALMETTE
MERAUX
AUDUBON PARK
GRETNA
AVONDALE
HARVEY
MARRERO
WESTWEGO
VIOLET
NOTE: DRAWINGS IN THIS FOLIO HAVE BEEN REDUCED ONE HALF THE ORIGINAL SCALE
C•A•D
N
SCALE OF MILES
1 0 1 2 3 MILES
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 22, GENERAL DESIGN
ORLEANS PARISH LAKEFRONT
REMAINING WORK
LOCATION AND INDEX MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
DESIGNED BY: GRUBB
DRAWN BY: HOWAT
CHECKED BY: DESAI
PLOT SCALE: 4800
PLOT DATE: 23 MAR 93
DATE: APRIL 1992
FILE NO. H-2-30962
PLATE 1

Floodwall	Existing Top Elevation (ft NGVD)	Proposed Top Elevation (ft NGVD)
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 and 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 to 13. 5 ft and at New Basin Canal Floodgate, a four-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	13.5
Elevation 11.5 ft + 2 ft freeboard	
Landside of floodwall	0.0

I-Walls. The I-type wall will consist of steel sheet piling driven into the existing ground. The upper portion of the sheet piling will be capped with concrete. The sheet piling will be driven to the required depth with 1 ft of the sheet piling extending above the finished ground elevation. The concrete portion of the floodwall will extend from 2 ft 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 sheet piling will not be disturbed.

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

- Case I –
    - For confined area, the FS used = 1.5 with static water at the top of the wall (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 at 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 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 elevation 11.5 ft, no wind, impervious sheet-pile cutoff, dynamic wave force (75 percent forces used)
- Case IV – Still-water pressure to elevation 11.5 ft, no wind, pervious sheet-pile cutoff, dynamic wave force (75 percent forces used)
- Case V - No water, no wind
- Case VI - No water, wind from protected side (75 percent forces used
- Case VII - No water, wind from floodside (75 percent forces used).

Gates and Gate Monoliths. Gate monoliths and two swing gates will be constructed on the lake side 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 ft, no wind, impervious sheet-pile cutoff, dynamic wave force (75 percent forces used)
- Case II - Gate closed, still-water pressure to elevation 11.5 ft, no wind, pervious sheet-pile cutoff, dynamic wave force (75 percent forces used)

- Case III - Gate open, truck on protected side of base slab, no wind
- Case IV - Gate open, truck on floodside of base slab, no wind
- Case V - Gate open, truck on protected side of base slab, wind from protected side (75 percent forces used)
- Case VI - Gate open, truck on protected side of base slab, wind from floodside (75 percent forces used).

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/floodwall and the pile bents, where hydraulic loading is applied, follow the more conservative requirements of the Corps' criteria for hydraulic structures. EM 1110-2-2104 states that concrete sections with a thickness greater than 12 in. but less than 24 in. have a clear cover of 3 in. Concrete sections with a thickness equal to or greater than 24 in. should have a clear cover of 4 in. However, in special circumstances, a clear cover of 3 in. for concrete sections equal to 24 in. 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/floodwall 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 Corps' 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, $H_f = 1.3$.

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

For short duration loads, during construction of hydraulic structures, the load factors may be reduced by 14 percent (equivalent to 16 2/3 percent 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 SPH or swl), the load factors may be reduced by 25 percent (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 is not combined with the AASHTO load factors.

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

Prestressing strands are not to be used as tension connectors.

Sources of Construction Materials.

<u>Sheet Pile.</u> 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.

Orleans East Bank	
17th Street Canal	
PS No. 6 to Hammond Hwy	Hoesch 12

Orleans Marina	
17th Street Canal to	PZ-38, PMA-22, PZ-35
Lakeshore Drive	
Lake Marina Drive	PZ-27
Orleans Outfall Canal	
West Side	
I-610 to French Street	Syro SPZ-22
Harrison Avenue Bridge Tie-Ins	Casteel CZ-114
Filmore Avenue 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 Bridge	PZ-22
Harrison Avenue Bridge Tie-Ins	Casteel CZ-114
Filmore Avenue Bridge Tie-Ins	Casteel CZ-114
Robert E. Lee Bridge Tie-Ins	Casteel CZ-101, CZ-114
London Avenue Outfall Canal	
PS#3 to Mirabeau, Both Sides	Syro SPZ-22
Mirabeau to Robert E. Lee Bridge, West Side	Casteel CZ-101
Mirabeau to Filmore, East Side	Casteel CZ-101
Filmore to PS No. 4, East Side	Arbed AZ-18
PS No. 4 to Leon C. Simon, East Side	Casteel CZ-101
Lakefront Levee	
Topaz Street Swing Gate Tie-In	PZ-35
Marconi Drive Swing Gate Tie-In	PZ-27
Rail Street	**
Bayou St. John Sector Gate East Side	Arbed BU-32
Bayou St. John Sector Gate West Side	PZ-40
American Standard (Franklin Avenue)	PZ-27
Leroy Johnson Drive Swing Gate Tie-In	PZ-27
Pontchartrain Beach Floodwall	PZ-22*
IHNC	
East Side	
North of Florida Avenue to Chalmette Back Levee	PZ-27
North of US 90	PZ-27
IHNC to Florida Avenue	PZ-27
Hayne Boulevard to Dwyer Road	PZ-27
Dwyer Road to Hwy 90	MA-22*, Z-27, PZ-32*, M27*

West Side	
IHNC to Florida Avenue	PZ-27
France Road to Florida Avenue	PZ-27*
North of US 90	PZ-27*
Hayne Boulevard to Hwy 90	PZ-27
Hwy 90 to Almonaster Boulevard	PZ-27, MA-22, SA-23
Almonaster Boulevard to Florida Avenue	PZ-27
* As-advertised – Not confirmed as-built. ** Information not located at the time of publication.	

<u>Levee material.</u>

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

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

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.

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

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.

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.

Levee Materials (Reference No. 11). Image of profile shown on following page. 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.

Levee Materials (Reference No. 12). Map of area shown on following page. 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.

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.

As-built Conditions.

<u>Changes Between Design and Construction (i.e. cross sections, alignment, sheet-pile tip elevation, levee crest elevation).</u>

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.

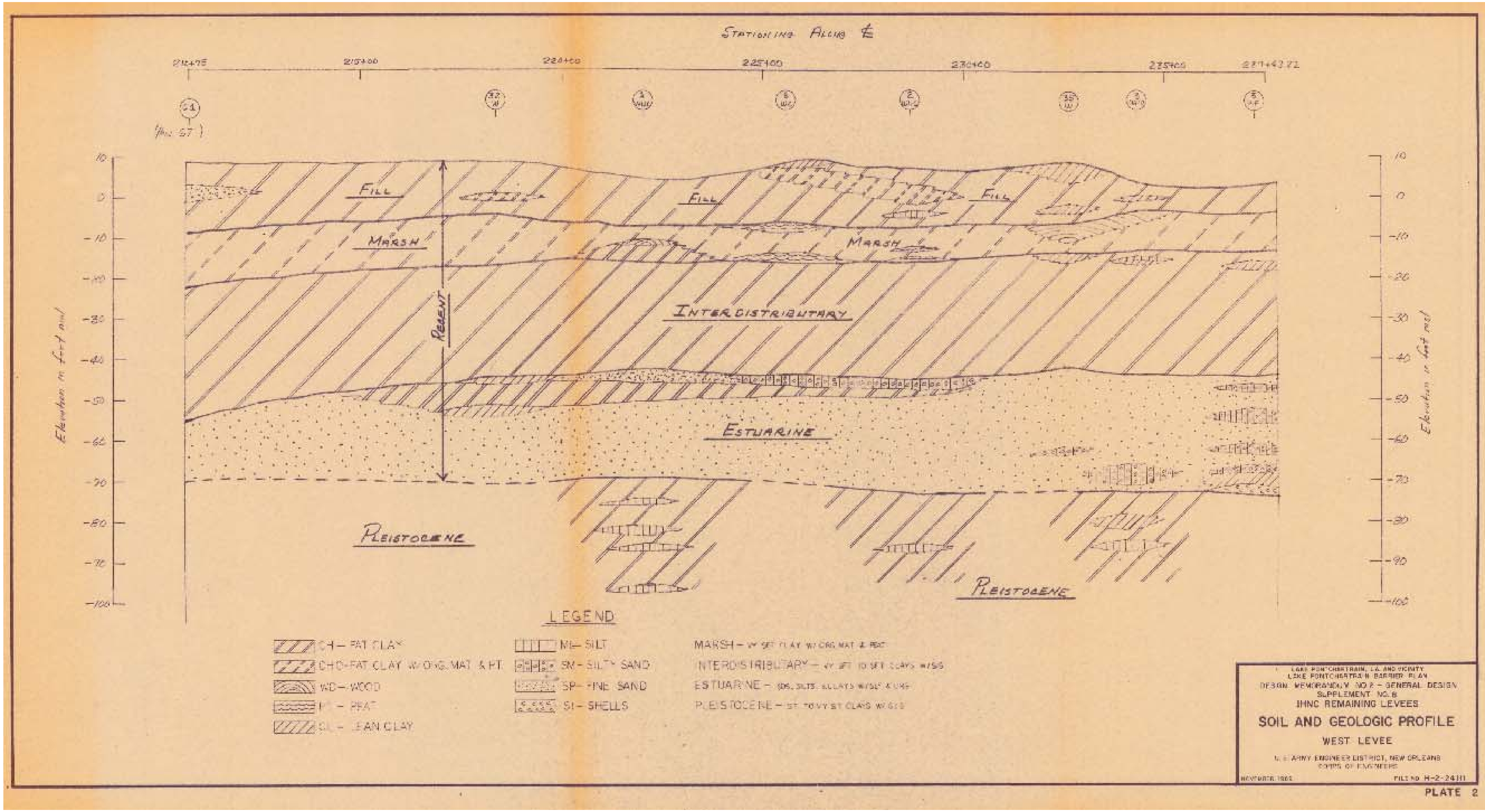
Reviewed as- builts; no modifications or changes were found.

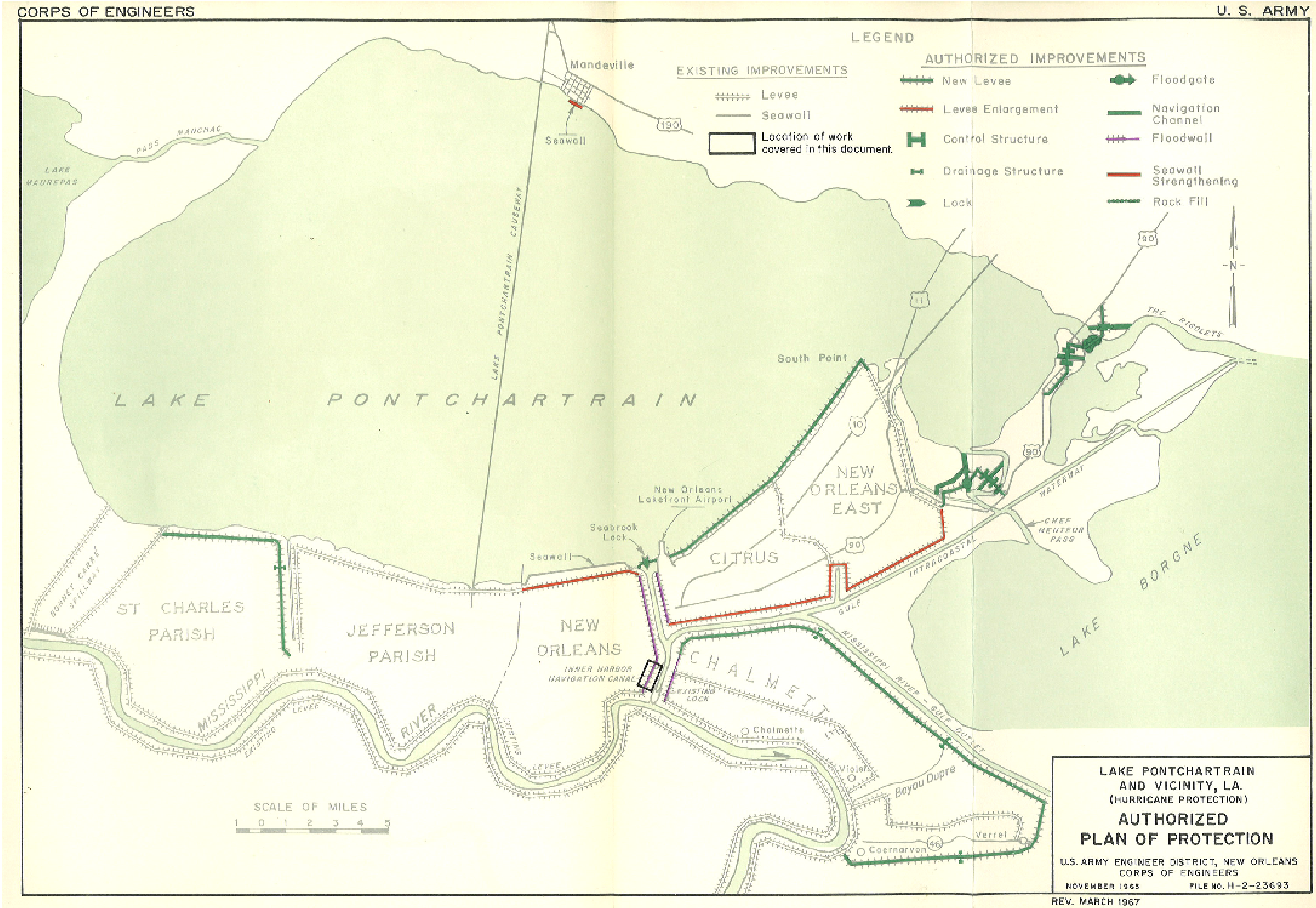
No Bid or Contract No. Lake Pontchartrain, LA, and Vicinity, Lake Pontchartrain Barrier Plan, Inner Harbor Navigation Canal, West Levee, Florida Avenue to IHNC Lock Floodwall.

Reviewed as-builts; no modifications or changes were found.

DACW29-68-B-0141. Lake Pontchartrain, LA, 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 were found.


STATIONING ALONG ℄
215+00
220+00
225+00
230+00
235+00
FILL
MARSH
INTERDISTRIBUTARY
ESTUARINE
PLEISTOCENE
REGENT
LEGEND
CH – FAT CLAY
CHO–FAT CLAY W/ ORG. MAT. & PT.
WD – WOOD
SL – LEAN CLAY
ML – SILT
SM – SILTY SAND
SP – FINE SAND
SI – SHELLS
MARSH
INTERDISTRIBUTARY
ESTUARINE
PLEISTOCENE
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 – GENERAL DESIGN
SUPPLEMENT NO. 8
IHNC REMAINING LEVEES
SOIL AND GEOLOGIC PROFILE
WEST LEVEE
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FILE NO. H-2-24111
PLATE 2


CORPS OF ENGINEERS
U. S. ARMY
LEGEND
EXISTING IMPROVEMENTS
Levee
Seawall
Location of work covered in this document.
AUTHORIZED IMPROVEMENTS
New Levee
Levee Enlargement
Control Structure
Drainage Structure
Lock
Floodgate
Navigation Channel
Floodwall
Seawall Strengthening
Rock Fill
Mandeville
Seawall
LAKE MAUREPAS
PASS MANCHAC
LAKE PONTCHARTRAIN
LAKE PONTCHARTRAIN CAUSEWAY
South Point
THE RIGOLETS
NEW ORLEANS EAST
New Orleans Lakefront Airport
Seabrook Lock
Seawall
CITRUS
CHEF MENTEUR PASS
INTRACOASTAL WATERWAY
GULF
LAKE BORGNE
ST CHARLES PARISH
BONNET CARRE SPILLWAY
JEFFERSON PARISH
NEW ORLEANS
INNER HARBOR NAVIGATION CANAL
EXISTING LOCK
CHALMETTE
Chalmette
MISSISSIPPI RIVER - GULF OUTLET
MISSISSIPPI RIVER
EXISTING LEVEE
Violet
Bayou Dupre
Verret
Caernarvon
SCALE OF MILES
1 0 1 2 3 4 5
LAKE PONTCHARTRAIN AND VICINITY, LA.
(HURRICANE PROTECTION)
AUTHORIZED PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NOVEMBER 1965
FILE NO. H-2-23693
REV. MARCH 1967

DACW29-70-B-0126. Lake Pontchartrain, LA, 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 were found.

DACW29-82-B-0033. Lake Pontchartrain, LA, and Vicinity, Hurricane Protection, Orleans Parish, LA, Floodwall and Levee, IHNC, North of Florida Ave.

Reviewed as-builts; no applicable modifications or changes were found.

DACW29-93-C-0071. Lake Pontchartrain, LA, 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, LA.

Reviewed completion report; no applicable modifications or changes were found.

DACW29-93-C-0077. Lake Pontchartrain, LA, and 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 were found.

DACW29-93-C-0071. Lake Pontchartrain, LA, 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, LA.

Reviewed completion report; no applicable modifications or changes were found.

DACW29-93-C-0077. Lake Pontchartrain, LA, and 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 were found.

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

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 short of the designated plan tip elevation, and by cutting the 4 ft 3 in. off the sheet pile, 9.25 ft was the top of pile elevation.

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

Reviewed completion report; no applicable modifications or changes were found.

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

Reviewed as-builts; no applicable modifications or changes were found.

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, LA.

Reviewed as-builts; no applicable modifications or changes were found.

DACW29-98-C-0043. Southeast Louisiana Urban Flood Control Project, Keyhole Canal, 4th Street to LaPalco Boulevard, Jefferson Parish, LA.

Reviewed as-builts; no applicable modifications or changes were found.

DACW29-98-C-0082. Lake Pontchartrain, LA, 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 were found.

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

Reviewed completion report and as-builts; no major modifications or changes were found.

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

No modifications or contract document folder was found.

DACW29-99-C-0018. Lake Pontchartrain, LA, 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 were found.

DACW29-99-C-0025. Lake Pontchartrain, LA, 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 were found.

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

Reviewed as-builts and contract documents; no applicable modifications or changes were found.

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

Reviewed contract/modification documents; no applicable modifications or changes were found.

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

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 4-in. stabilization slab. It also required increasing the sheet-pile cofferdam bracing.

<u>Inspection During Original Construction, QA/QC, State What Records Are Available.</u> On construction contracts, the government QA reports and contractor 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, offsite surveillance activities, job safety, environmental protection, and remarks.

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 QC 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 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 and sent to offsite storage where they remain for 6 years. After 6 years, they are destroyed.

DACW29-93-C-0071. New Orleans Lakefront Levee Orleans Avenue Canal, Phase II-B, Orleans Parish.

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

DACW29-93-C-0081. 17th Street Outfall Canal Capping of Floodwall, Orleans Parish.

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.

DACW29-95-C-0093. Floodproofing Vets Bridge – 17th Street, Jefferson and Orleans Parish.

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 dewatering reports are also included.

DACW29-96-C-0080. Orleans Marina Floodwall – Phase IV, Orleans Parish.

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

DACW29-97-C-0066. Lake Pontchartrain, Pontchartrain Beach, Station 10+03 – 39-78, Orleans Parish.

Attached are soil tests and preparatory inspection reports.

DACW29-97-C-0029. Orleans Avenue Phase II-A Floodwall, Orleans Parish.

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

DACW29-98-C-0022. Lake Pontchartrain, New Orleans East, Floodwall Capping, Marina-Collin, Orleans Parish.

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 form checkout sheets for structures.

DACW29-98-C-0050. Orleans Marina, Phase V, Sluice Gate, Orleans Parish.

Attached are piezometer readings, and monolith reference readings.

DACW29-98-C-0082. Lake Pontchartrain London Canal, Floodproofing Leon C. Simon, Orleans Parish.

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.

DACW29-99-C-0005. Lake Pontchartrain London Canal, Floodproofing Gentilly Bridge, Orleans Parish.

Attached to the contractors' 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.

DACW29-99-C-0012. London Avenue Outfall Canal, Phase 4, Orleans Parish.

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

DACW29-99-C-0025. Filmore and Harrison Bridges, Orleans Parish.

Pile driving records are attached to the reports.

DACW29-99-C-0046. Lake Pontchartrain Breakwaters, Phases 2 & 3, Orleans Parish.

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

DACW29-00-C-0073. Lake Pontchartrain Orleans Outfall, Phase 1B, (Floodproofing Robert E. Lee Bridge), Orleans Parish.

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

DACW29-02-C-0013. Lake Pontchartrain, London CNT 5 (Mirabeau/Filmore), Orleans Parish.

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

DACW29-98-C-0003. Lake Pontchartrain London Street R/O Reach 2, Stations 167-209, Jefferson Parish.

QA/QC reports are available; nothing is attached.

Inspection and Maintenance of Original Construction

The inspection of hurricane protection features of the East Bank polder fall under the following categories:

Annual Compliance Inspection. Annual Compliance Inspections for the East Bank polder were conducted by the Corps, 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.

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

Other Features

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. However, other drainage and flood control features that work in concert with the Corps levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section describes and presents 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.

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 29 August 2005, Katrina landfall.

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

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 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.

Floodwater 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 New Orleans Sewerage and Water Board (NOS & WB). In addition to local drainage, NOS & WB provides 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 to 25 ft. 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.

Photos 1 and 2. Typical streets and inlet – Orleans East Bank.

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.

Photo 3. Broad Street Canal entering Pump Station No. 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) storm. 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 in./hr. 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 would not change significantly, and delaying runoff to an outfall pump is counter productive.

Where local drainage is considered poor, the NOS & WB is working to improve the drainage. In some cases, the NOS & WB and Corps 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 NOS & WB 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.

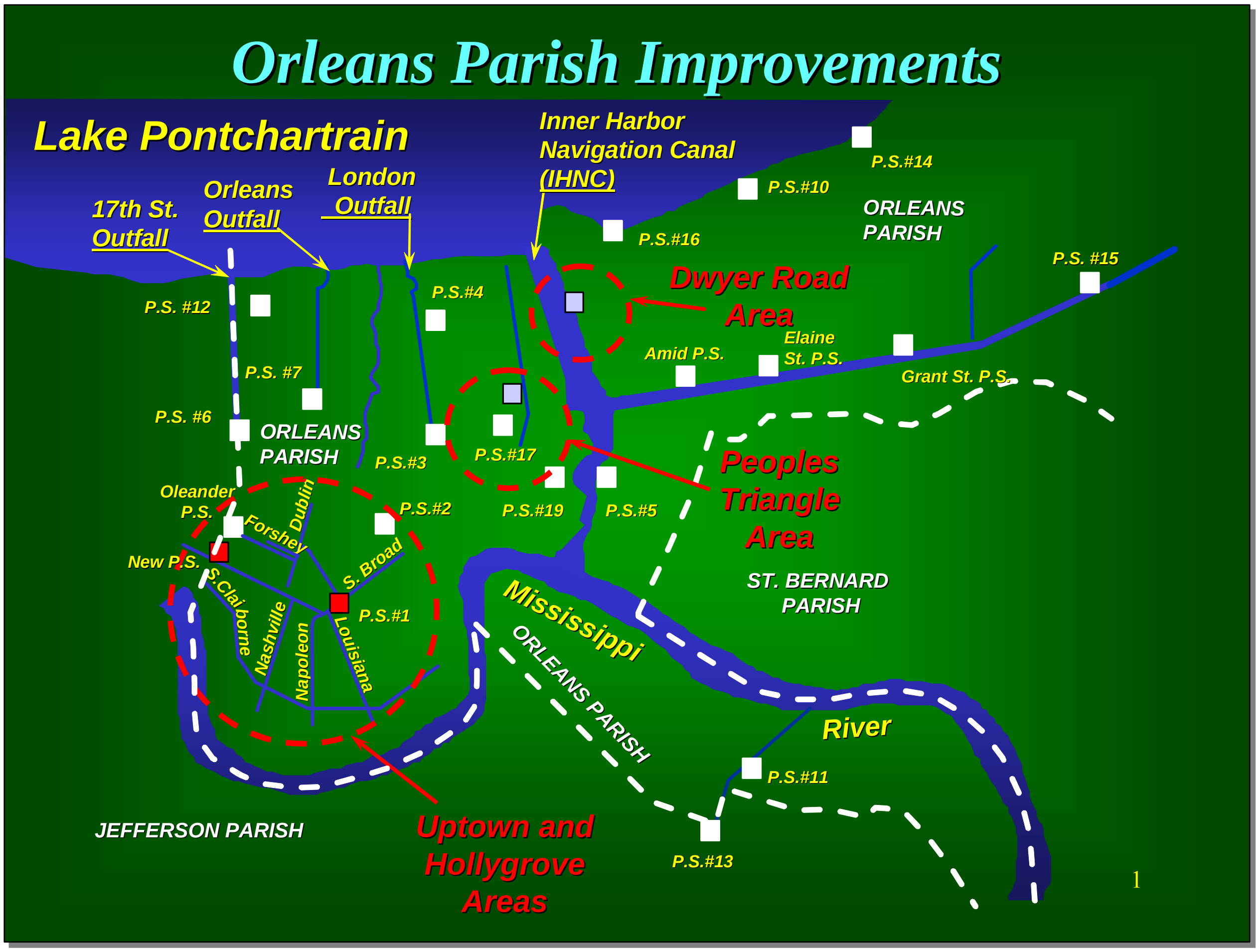


Figure 8. SELA Urban Flood Control Projects in Orleans East Bank.

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 completed 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 completed prior to Hurricane Katrina.

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

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).

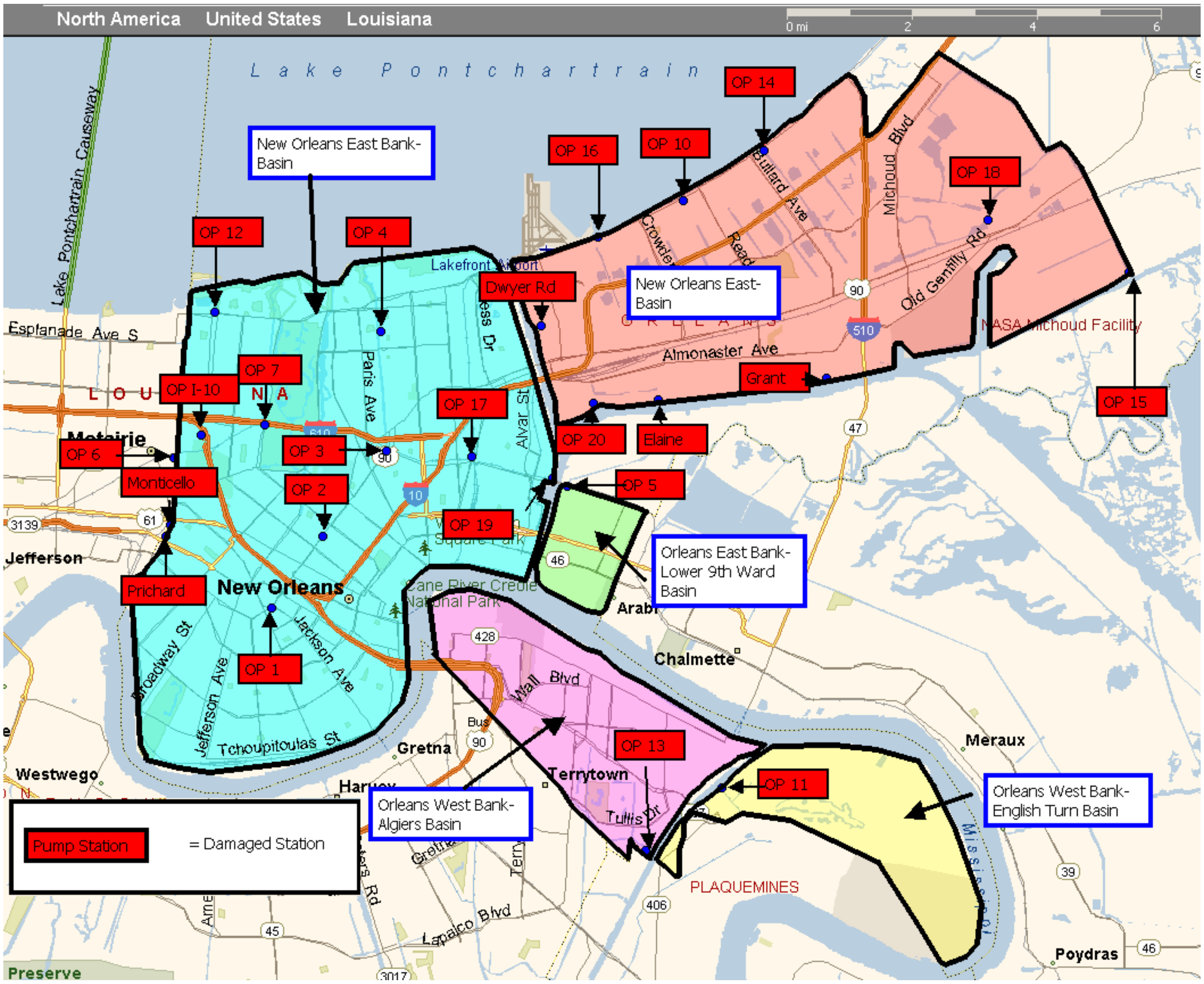


Figure 9. Orleans Parish pump station locations.

Table 7 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; missing information was left blank or highlighted.

Table 7
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.

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 Avenue, Broad Street, Prentiss Avenue, 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 Avenue Canals
Discharge location: ....Palmetto Canal
Nominal capacity: ....6,825 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	1,000	1929	Electric 25 Hz	Horizontal
D	1,000	1929	Electric 25 Hz	Horizontal
E	1,000	1929	Electric 25 Hz	Horizontal
F	1,100	1991	Electric 60 Hz	Horizontal
G	1,100	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 and 7
Nominal capacity: ....3,150 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	1,000	1914	Electric 25 Hz	Horizontal
D	1,000	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 Avenue Canal
Nominal capacity: ....4,340 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	1,000	1930	Electric 25 Hz	Horizontal
D	1,000	1930	Electric 25 Hz	Horizontal
E	1,000	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 Avenue and St. Anthony Canals
Discharge location: .... London Avenue Canal
Nominal capacity: ....3,720 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	1,000	1957	Electric 25 Hz	Horizontal
D	1,000	1957	Electric 25 Hz	Horizontal
E	1,000	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: ....9,480 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	1,000	1928	Electric 25 Hz	Horizontal
D	1,000	1928	Electric 25 Hz	Horizontal
E	1,000	1928	Electric 25 Hz	Horizontal
F	1,000	1928	Electric 25 Hz	Horizontal
G	1,000	1984	Electric 25 Hz	Horizontal
H	1,100	1984	Electric 60 Hz	Horizontal
I	1,100	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:..2,690 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
A	550	1931	Electric 25 Hz	Horizontal
C	1,000	1908	Electric 25 Hz	Horizontal
D	1,000	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: .... 1,000 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
D	1,000	1961	Electric 25 Hz	Horizontal

OP 17 (Station D)
Intake location: .... Peoples and Florida Avenue 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 Avenue Canal
Discharge location: .... Industrial Canal (Lake Pontchartrain)
Nominal capacity: .... 3,920 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
H1	1,100	1975	Electric 60 Hz	Horizontal
H2	1,100	1975	Electric 60 Hz	Horizontal
H3	1,100	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

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 Avenue Canals
Discharge location: .... Lake Borgne
Nominal capacity: .... 2,260 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	1,000	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

Levees and Floodwalls.

Mississippi River Levees (Reference Nos. 44, 74, 78). Map of area shown on following page.

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 ft 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 range in depth from 1 to 6 ft. The Mississippi River channel ranges in depth from 65 to 190 ft below sea level. At present, the rate of subsidence in the study area ranges between 0.5 and 1.0 ft per century.

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 74 lists the general policies that were used to design the Mississippi River Levees (MRL) with respect to planning, exploration, testing design and construction of main stem levees. In addition to spelling out policy for exploration and testing, Reference 74 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.

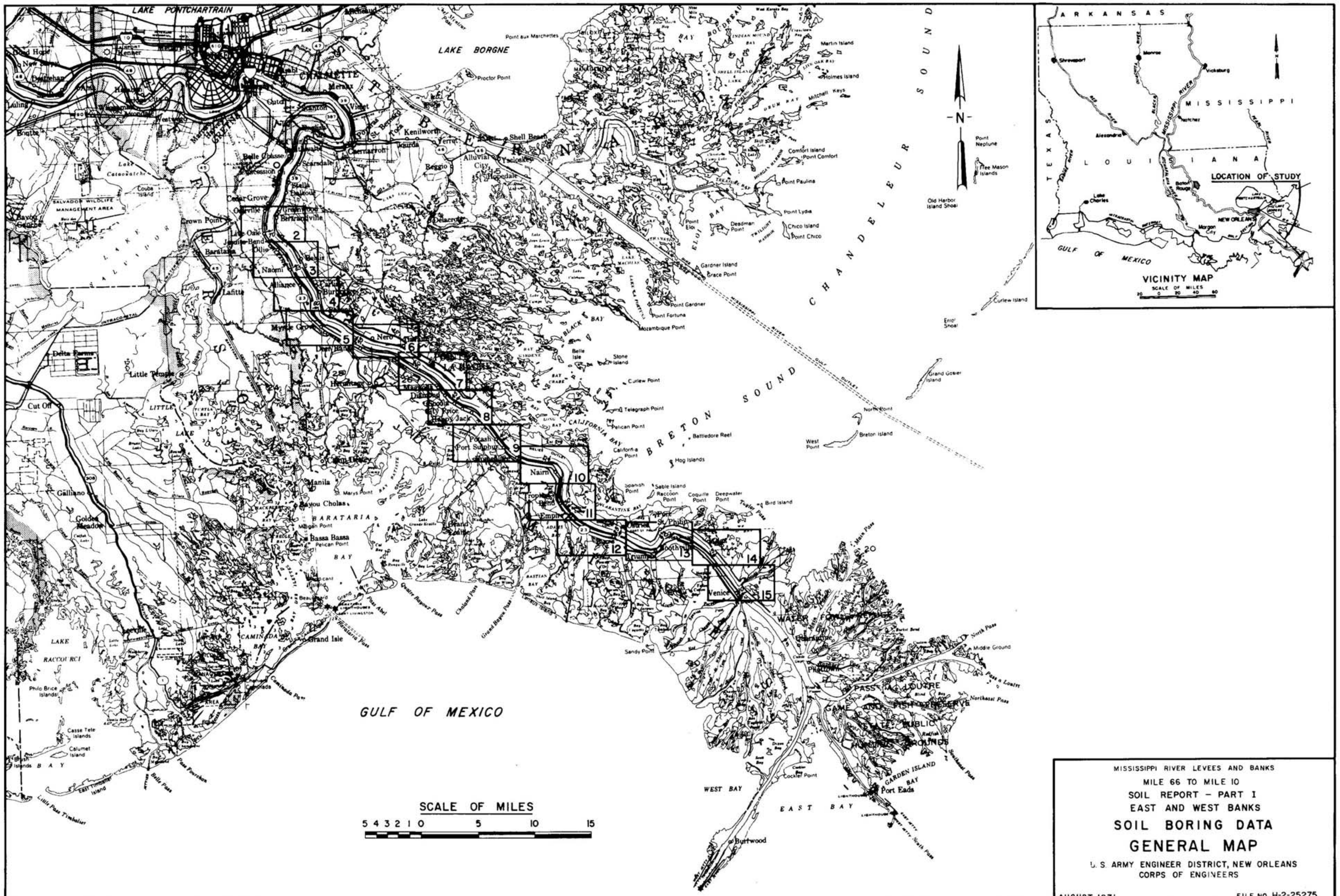

MISSISSIPPI RIVER LEVEES AND BANKS
MILE 66 TO MILE 10
SOIL REPORT – PART I
EAST AND WEST BANKS
SOIL BORING DATA
GENERAL MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
AUGUST 1971
FILE NO. H-2-25275
VICINITY MAP
LOCATION OF STUDY
SCALE OF MILES
GULF OF MEXICO
LAKE PONTCHARTRAIN
LAKE BORGNE
BRETON SOUND
CHANDELEUR SOUND

PLATE I

Type 3. Moderately larger than the present uncompacted cross section. Uncompacted emergency construction of relatively 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 ft	1:4.5
2, less than 25 ft in height	1:4.0	10 ft	1:5.5
2, 25-ft and higher	1:4.0	10 ft	1:6.0
3	1:4.5	10 ft	1:6.5

The main stem levees below New Orleans are generally less than 15 ft in height and are built to the tributary standard levee section of 1-on-3, 1-on-4, with a 10-ft 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 ft in height and flatter than 1:6.0 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 steeper than 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 ft, 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 stem 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 1.0,, dangerous boils are likely to be encountered. A hydraulic gradient as low as 0.7 is approximately 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 FS should be at least 1. In other cases where drawdown does not apply, the minimum FS 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.

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

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.

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

Introduction

The hurricane protection system for the New Orleans East (NOE) Basin was designed as part of the Lake Pontchartrain, LA, and Vicinity HPP. 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 ranges from 13 to 19 ft. There are floodwall segments along the line of protection that consist 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 SPH.

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 Sauvage National Wildlife Preserve, and the north side of the Gulf Intracoastal Waterway (GIWW) (between the IHNC and eastern edge of the Bayou Sauvage 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.

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 South Point. 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 South Point to the GIWW along the eastern boundary of the Bayou Sauvage National Wildlife Preserve.

GIWW. The basin includes the Citrus Back Levee and New Orleans East Back Levee which consists 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 lie within the boundaries.

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.

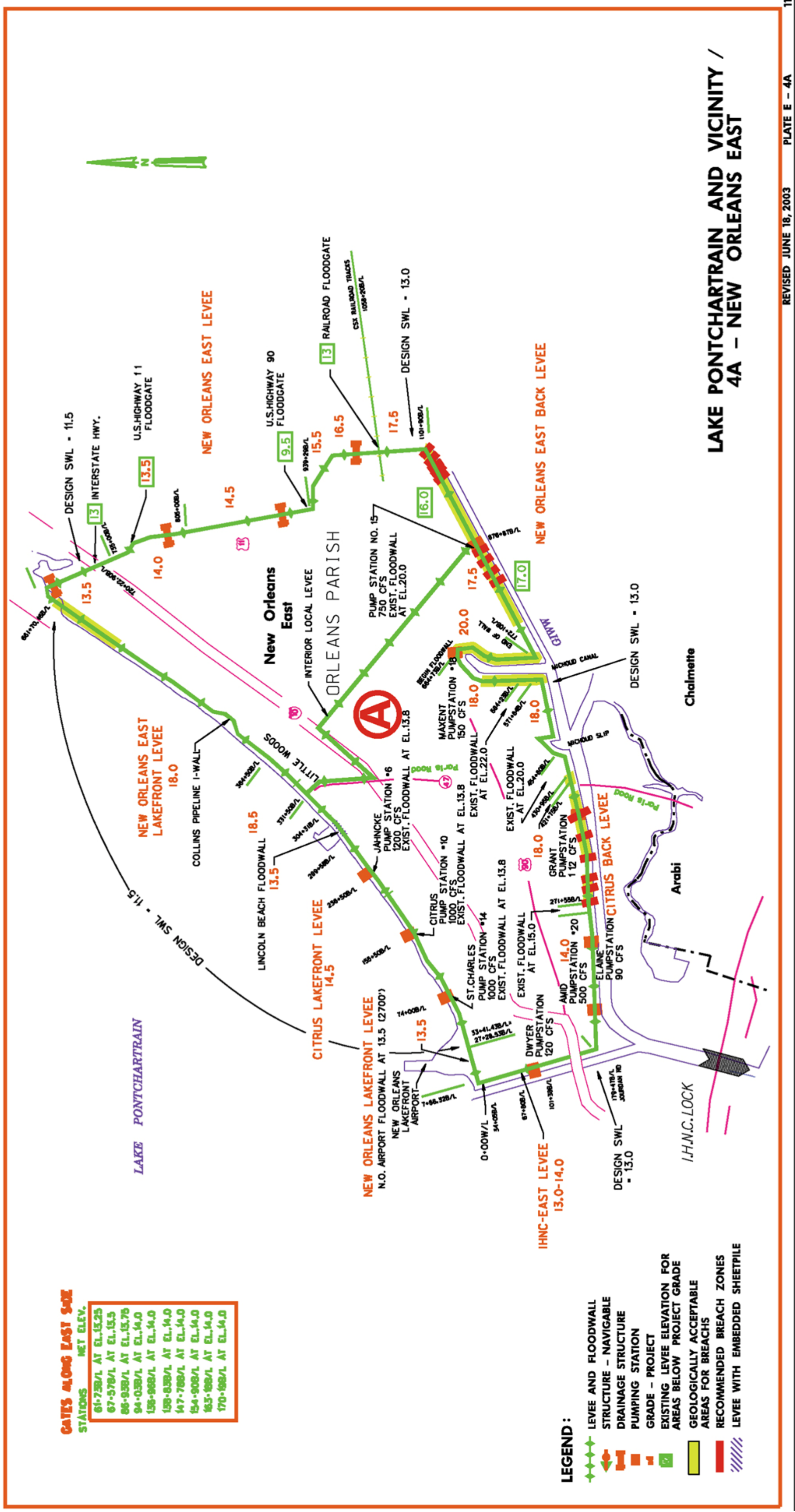


Figure 10. NOE basin general components and top of levee/floodwall as-built elevations (feet) (source New Orleans District (Wayne Naquin).

Table 8 **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

Pre-Katrina

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

- A levee enlargement along the New Orleans East Back Levee.
- Rehabilitation of four 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.

Design Criteria and Assumptions – Functional Design Criteria

Hydrology and Hydraulics.

For New Orleans East, the design hurricane characteristics utilized in the design memoranda are shown in Table 9; 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 9
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] Wind speeds represent a 5-minute average 30 ft above ground level.

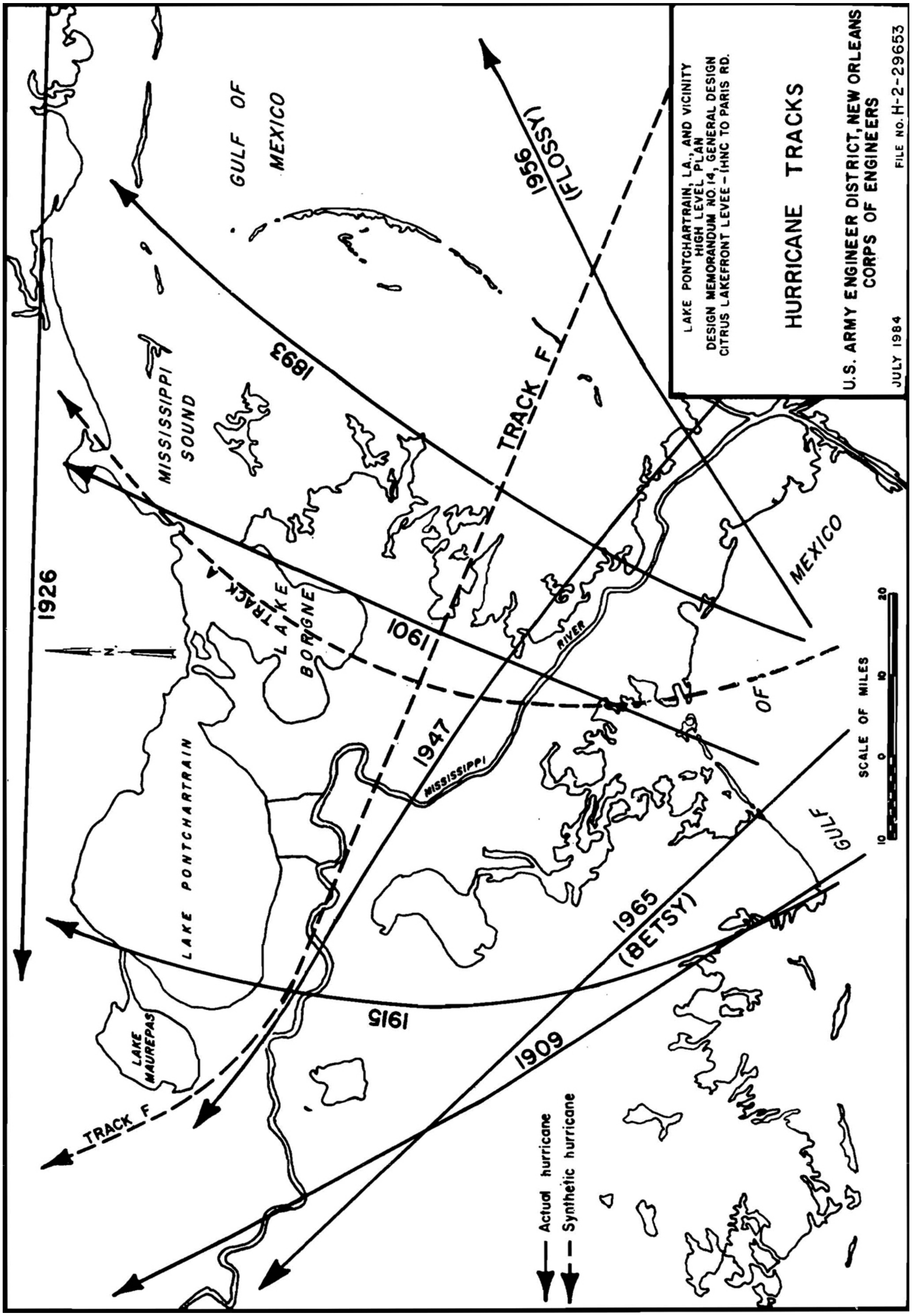


Figure 11. Hurricane tracks, New Orleans East protection system.

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.

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 were 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 U.S. 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.

Summary. Table 10 contains maximum surge or wind tide level, wave, and design elevation information.

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 Airport to Intersection of MRDG/GIWW with IHNC.

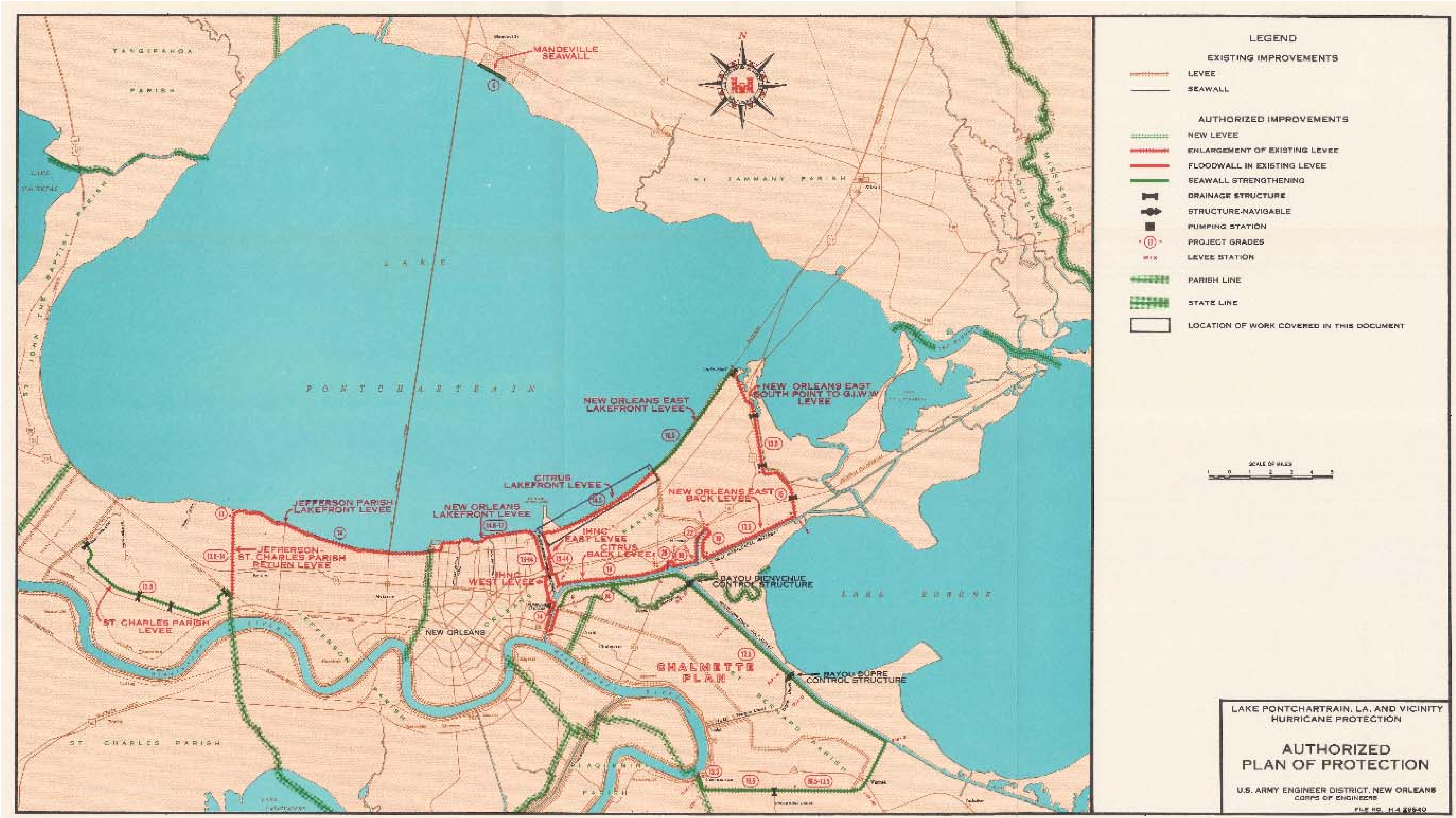
Citrus Lakefront Levee. The Citrus Lakefront Levee extends from the IHNC to Paris Road and includes 5.5 miles of earthen levee and 0.9 mile of I-wall (Reference Nos. 17 and 18). Map of area on following page.

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

Table 10 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, July 1984	-	-	-	11.5 NGVD	-	3.0	14.5 NGVD
Citrus Lakefront, 64+00 to 331+50	DM14, July 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, April 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, April 1985	24.4	7.8	7.3	11.5 NGVD	6.5	-	18.0 NGVD
New Orleans East South Point to Highway 90	DM16, September 1987	-	-	-	11.5-12.2 NGVD	-	2.0	13.5-14.5 NGVD
New Orleans East, Highway 90 to Station 1030+00	DM16, September 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, September 1987	11.0	4.7	5.4	12.8 NGVD	4.5	-	17.5 NGVD
New Orleans East Back Levee, levee	DM2, Sup 04, March 1971	12.7	4.9	5.5	13.0 MSL	4.5	-	17.5 MSL
New Orleans East Back Levee, floodwall	DM2, Sup 04, March 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, March 1971	12.7	4.9	5.5	13.0 MSL	10.0	-	23.0 MSL
Citrus Back Levee, west of Paris Road	DM2, August 1967	-	-	-	13.0 MSL	-	1.0	14.0 MSL
Citrus Back Levee, east of Paris Road	DM2, August 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 Back Levee	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.								

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

Field Exploration. Undisturbed 5-in.-diameter borings were made at 16 locations. General type core borings 1⅞-in.-ID were made at 41 locations. Additional undisturbed borings were taken and tested by the Corps along the centerline and 50 ft lake side of the baseline.


LEGEND
EXISTING IMPROVEMENTS
LEVEE
SEAWALL
AUTHORIZED IMPROVEMENTS
NEW LEVEE
ENLARGEMENT OF EXISTING LEVEE
FLOODWALL IN EXISTING LEVEE
SEAWALL STRENGTHENING
DRAINAGE STRUCTURE
STRUCTURE-NAVIGABLE
PUMPING STATION
PROJECT GRADES
LEVEE STATION
PARISH LINE
STATE LINE
LOCATION OF WORK COVERED IN THIS DOCUMENT
MANDEVILLE SEAWALL
LAKE PONTCHARTRAIN
NEW ORLEANS EAST LAKEFRONT LEVEE
NEW ORLEANS EAST SOUTH POINT TO G.I.W.W LEVEE
CITRUS LAKEFRONT LEVEE
NEW ORLEANS EAST BACK LEVEE
JEFFERSON PARISH LAKEFRONT LEVEE
NEW ORLEANS LAKEFRONT LEVEE
IHNC EAST LEVEE
CITRUS BACK LEVEE
JEFFERSON-ST. CHARLES PARISH RETURN LEVEE
IHNC WEST LEVEE
BAYOU BIENVENUE CONTROL STRUCTURE
LAKE BORGNE
ST. CHARLES PARISH LEVEE
NEW ORLEANS
CHALMETTE PLAN
BAYOU DUPRE CONTROL STRUCTURE
ST. CHARLES PARISH
LAKE PONTCHARTRAIN, LA. AND VICINITY HURRICANE PROTECTION
AUTHORIZED PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS

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.

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

Slope Stability. Using cross sections representative of existing conditions along the levee, the stability of the levee was investigated by the Method of Planes, using design Q shear strengths, the trends assigned to various levee sections, and applying a minimum FS with respect to shear strengths of 1.3.

I-Walls. DM 2 shows 0.9 mile 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 FS of 1.5 was applied to design shear strengths. The sheet-pile penetration 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.

T-Walls. The T-type floodwalls supported on bearing piles will provide protection adjacent to T-type gates supported by bearing piles.

Erosion Protection. Thirty-six-inch derrick stone will be placed on a 12-in. riprap blanket to cover the lake side slope of the existing railroad embankment.

New Orleans East Lakefront Levee. The New Orleans East Lakefront Levee consists of 6.3 miles of earth levee and 463 ft of I-wall (Reference Nos. 19 and 20). Images of map and plan view shown on following pages.

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

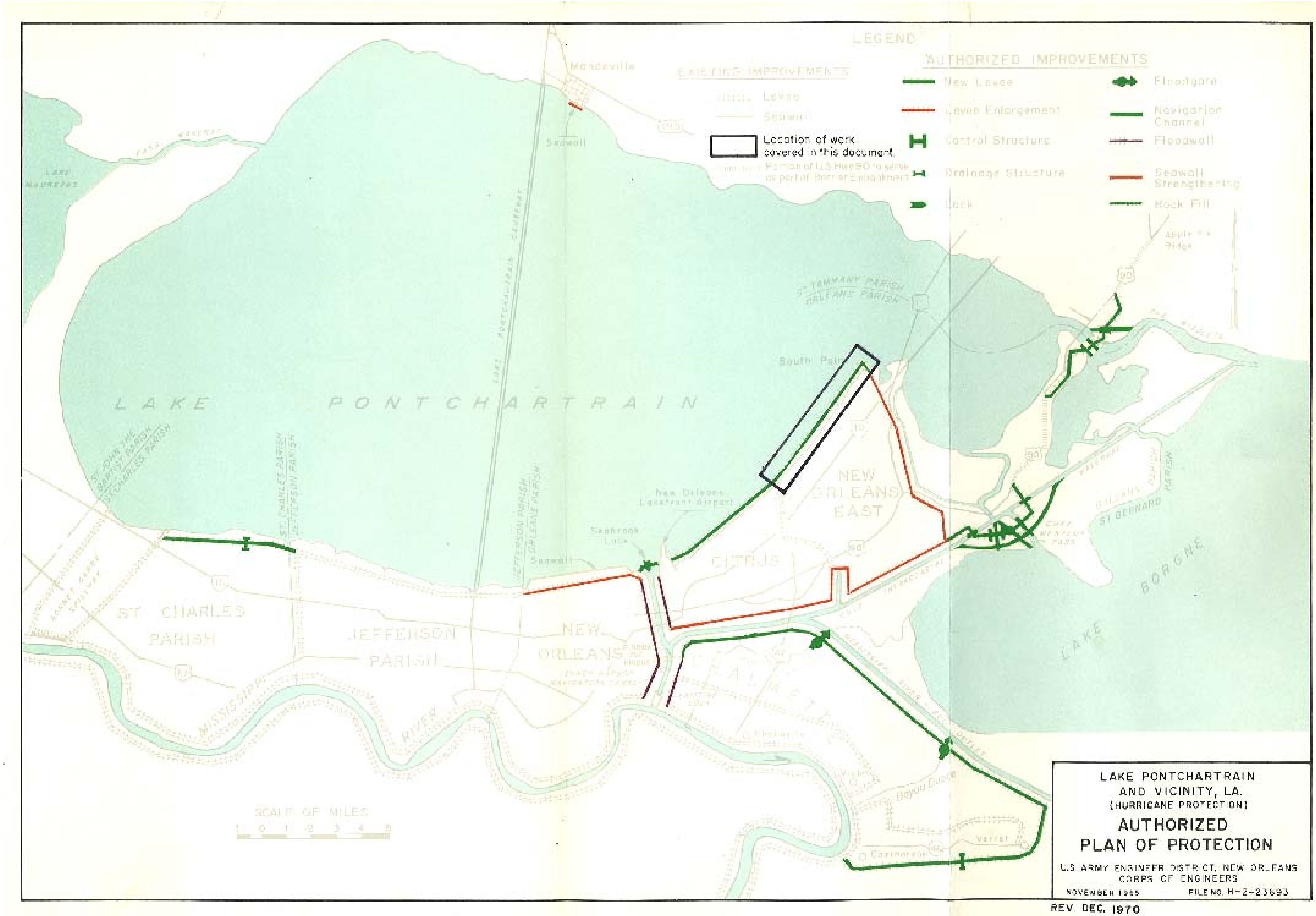
Foundation Conditions. Generally the area consists of Holocene deposits varying from elevation approximately -40.0 to -25.0 ft NGVD. These deposits are predominantly unconsolidated, saturated, low strength clays with some silts and silty sands. Artificial fill consisting of sand core overlain by a semi-compacted clay cap was placed along the levee centerline from -10.0 to 15.0 ft NGVD.

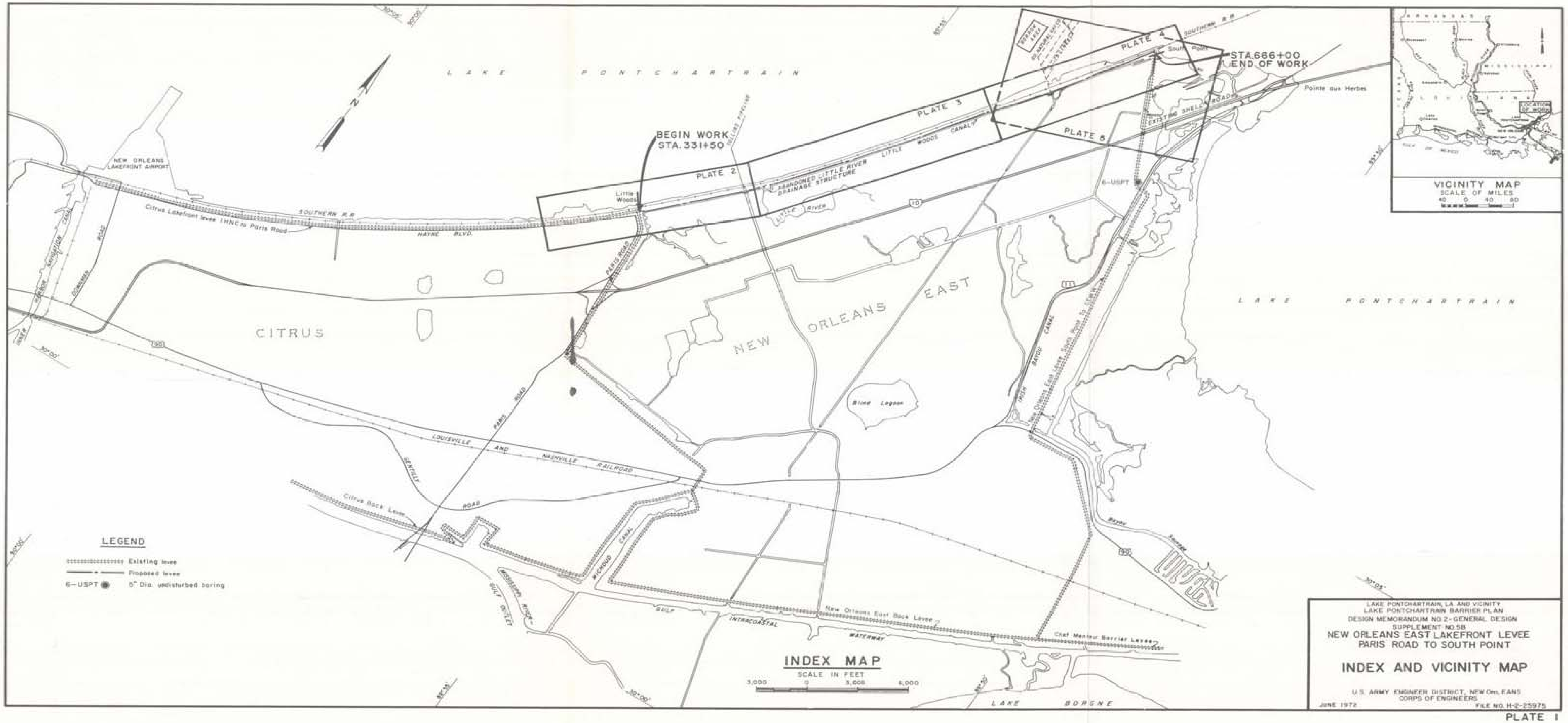
Field Investigation. For DM 15, a total of 15 undisturbed borings were taken and tested. Eleven were along the centerline and 120 ft landside of the levee. Additional old borings were considered.

Underseepage. Not used.

Hydrostatic Pressure Relief. Not used.

Pile Foundation. Not used.


LEGEND
EXISTING IMPROVEMENTS
Levee
Seawall
Location of work covered in this document
Portion of U.S. Hwy 90 to serve as part of Barrier Embankment
AUTHORIZED IMPROVEMENTS
New Levee
Levee Enlargement
Control Structure
Drainage Structure
Lock
Floodgate
Navigation Channel
Floodwall
Seawall Strengthening
Rock Fill
LAKE PONTCHARTRAIN
LAKE BORGNE
Mandeville
Seawall
South Point
New Orleans Lakefront Airport
Seabrook Lock
NEW ORLEANS EAST
CITRUS
NEW ORLEANS
ST. CHARLES PARISH
JEFFERSON PARISH
ST. BERNARD PARISH
MISSISSIPPI RIVER
SCALE OF MILES
1 0 1 2 3 4 5
LAKE PONTCHARTRAIN AND VICINITY, LA.
(HURRICANE PROTECTION)
AUTHORIZED PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NOVEMBER 1965
FILE NO. H-2-23693
REV. DEC. 1970


LAKE PONTCHARTRAIN
BEGIN WORK
STA.331+50
STA.666+00
END OF WORK
PLATE 2
PLATE 3
PLATE 4
PLATE 5
NEW ORLEANS LAKEFRONT AIRPORT
SOUTHERN R.R.
Citrus Lakefront levee IHNC to Paris Road
HAYNE BLVD.
Little Woods
ABANDONED LITTLE RIVER DRAINAGE STRUCTURE
LITTLE WOODS CANAL
LITTLE RIVER
South Point
Pointe aux Herbes
EXISTING SHELL ROAD
6-USPT
CITRUS
NEW ORLEANS EAST
PARIS ROAD
Blind Lagoon
LOUISVILLE AND NASHVILLE RAILROAD
GENTILLY ROAD
Citrus Back Levee
MICHOUD CANAL
GULF INTRACOASTAL WATERWAY
New Orleans East Back Levee
Chef Menteur Barrier Levee
MISSISSIPPI RIVER-GULF OUTLET
LAKE BORGNE
LEGEND
Existing levee
Proposed levee
6-USPT 5" Dia. undisturbed boring
INDEX MAP
SCALE IN FEET
3,000 0 3,000 6,000
VICINITY MAP
SCALE OF MILES
40 0 40 80
LOCATION OF WORK
LAKE PONTCHARTRAIN, LA AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO 2-GENERAL DESIGN
SUPPLEMENT NO.5B
NEW ORLEANS EAST LAKEFRONT LEVEE
PARIS ROAD TO SOUTH POINT
INDEX AND VICINITY MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
JUNE 1972
FILE NO. H-2-25975
PLATE I

Slope Stability. The stability of the levee was determined by the Method of Planes using the design Q shear strengths and applying a minimum FS of approximately 1.3. To preclude potential movement of pipelines conveying high pressure volatile liquids and/or gases, a FS of 1.5 was utilized to design the levee at pipeline relocations. The cases analyzed were:

(1) Water level to elevation 1.0 ft NGVD with anticipated failure into the demucked canal (floodside).

(2) Water level to project hurricane wind tide level elevation 8.5 ft NGVD on floodside and elevation 0.0 ft NGVD on protected side.

(3) Water to elevation 0.0 ft NGVD 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.

I-Walls. A short section (463 ft) of I-wall was used at the Collins pipe crossing. A FS of 1.25 was used with static water at the wind tide level of 11.5 ft 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 ft NGVD was used to penetrate the sand core.

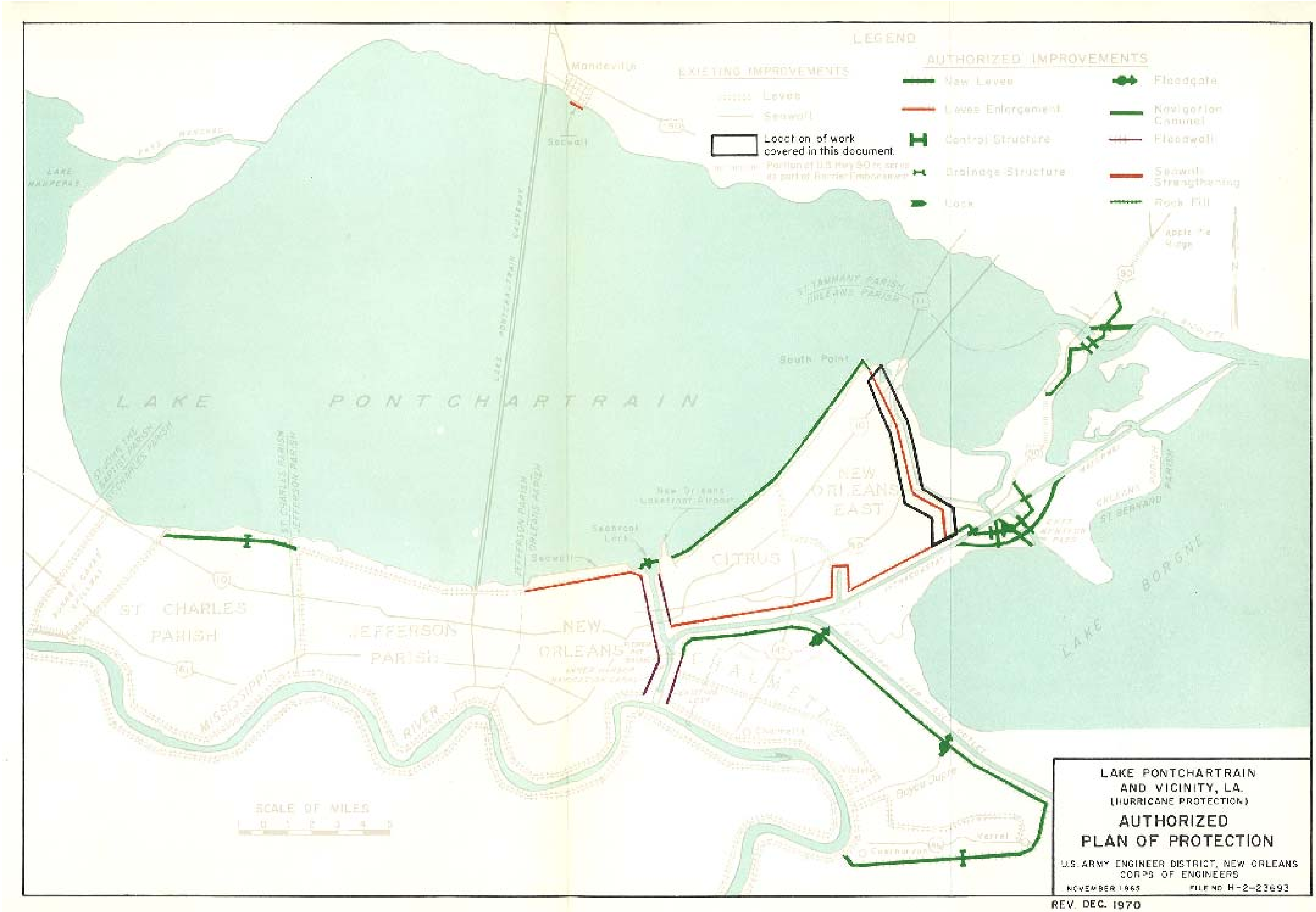
T-Walls. Not used.

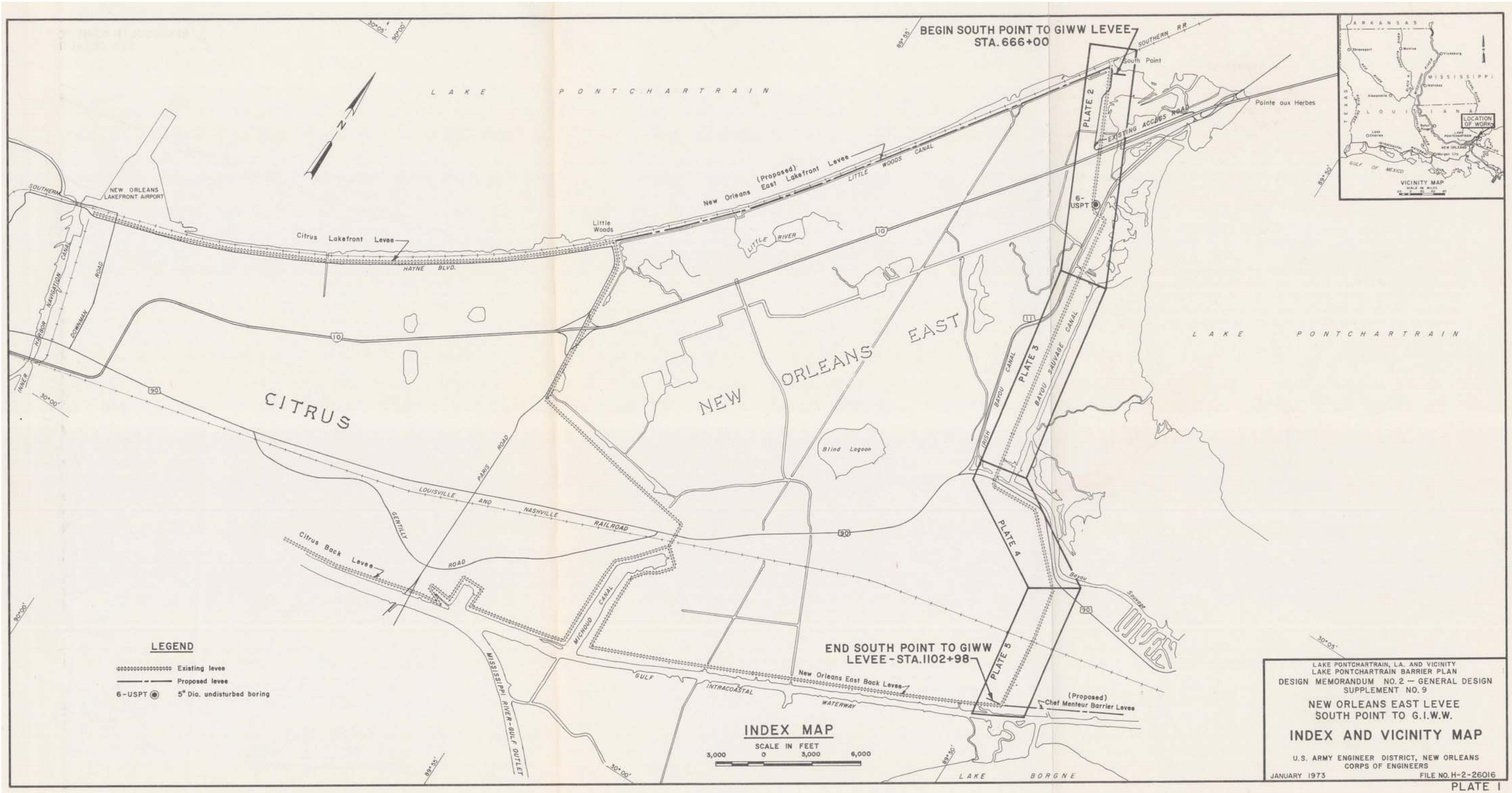
<u>New Orleans East Levee South Point to GIWW (Reference Nos. 21 and 22).</u> There was an existing levee along the alignment which was constructed in 1956 by the Orleans Parish Levee District. This initial levee construction consisted of demucking the organic clay along the levee centerline, constructing retaining dikes, pumping hydraulic fill between the dikes, and shaping to elevation 11.5 ft NGVD. The new levee enlargement for this levee section consisted of a straddle enlargement done with clay fill material. Images of map and plan view shown on following pages.

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

Foundation Conditions. The subsurface along this project consists generally of 16 to 20 ft of artificial levee fill. Below this artificial fill, 13 to 22 ft of Recent deposits of clays, silts, and sands with a Pleistocene deposit encountered at approximate elevations of -23.0 to -32.0 ft NGVD. 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 ft of Recent deposits of clay with layers and lenses of silt and sand which are underlain by a sand deposit at approximately elevations -14.0 to -50.0 ft NGVD.

Field Exploration. Twelve 5-in.-diameter 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-in.-diameter undisturbed borings were taken for Design Memorandum No. 16 General Design.


LEGEND
EXISTING IMPROVEMENTS
Levee
Seawall
Location of work covered in this document.
AUTHORIZED IMPROVEMENTS
New Levee
Levee Enlargement
Control Structure
Drainage Structure
Lock
Floodgate
Navigation Channel
Floodwall
Seawall Strengthening
Rock Fill
Mandeville
Seawall
LAKE MAUREPAS
LAKE PONTCHARTRAIN
South Point
NEW ORLEANS EAST
CITRUS
New Orleans Lakefront Airport
Seabrook Lock
ST. CHARLES PARISH
JEFFERSON PARISH
NEW ORLEANS
MISSISSIPPI RIVER
ST. BERNARD
LAKE BORGNE
SCALE OF MILES
LAKE PONTCHARTRAIN AND VICINITY, LA.
(HURRICANE PROTECTION)
AUTHORIZED PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NOVEMBER 1965
FILE NO. H-2-23693
REV. DEC. 1970



BEGIN SOUTH POINT TO GIWW LEVEE
STA. 666+00
LAKE PONTCHARTRAIN
New Orleans (Proposed) East Lakefront Levee
Citrus Lakefront Levee
NEW ORLEANS LAKEFRONT AIRPORT
CITRUS
NEW ORLEANS EAST
Blind Lagoon
Citrus Back Levee
PLATE 2
PLATE 3
PLATE 4
PLATE 5
6-USPT
South Point
Pointe aux Herbes
END SOUTH POINT TO GIWW LEVEE - STA. 1102+98
New Orleans East Back Levee
(Proposed) Chef Menteur Barrier Levee
LEGEND
Existing levee
Proposed levee
6-USPT 5" Dia. undisturbed boring
INDEX MAP
SCALE IN FEET
3,000 0 3,000 6,000
VICINITY MAP
LOCATION OF WORK
LAKE BORGNE
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 — GENERAL DESIGN
SUPPLEMENT NO. 9
NEW ORLEANS EAST LEVEE
SOUTH POINT TO G.I.W.W.
INDEX AND VICINITY MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
JANUARY 1973
FILE NO. H-2-26016
PLATE 1

Underseepage. A sheet-pile cutoff was used beneath the railroad gate to control underseepage.

Hydrostatic Uplift. Not used.

Pile Foundation. Design compression and tension capacities versus tip elevation were developed for 12-in.-square prestressed concrete piles. The piles will support the railroad swing gate monolith. In compression, a FS of 1.75 with a $K_0 = 1.0$ was used for the S-Case. In tension, a FS of 2.0 was applied with a K_0 for the S-Case. The Q-Case governed the design.

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 floodside and for the conditions with water to elevation 11.5 ft wind tide level on the floodside and to elevation 0.0 ft NGVD on the protected side and assumed failure toward the protected side. A minimum FS of 1.3 was used.

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.

T-Walls. Not used.

Review Comments. First Endorsement Comment 2i(1) as to whether a wave wash clay blanket thickness of 2 ft was adequate. District Response was that 2 ft of riprap plus 2 ft of clay blanket was adequate.

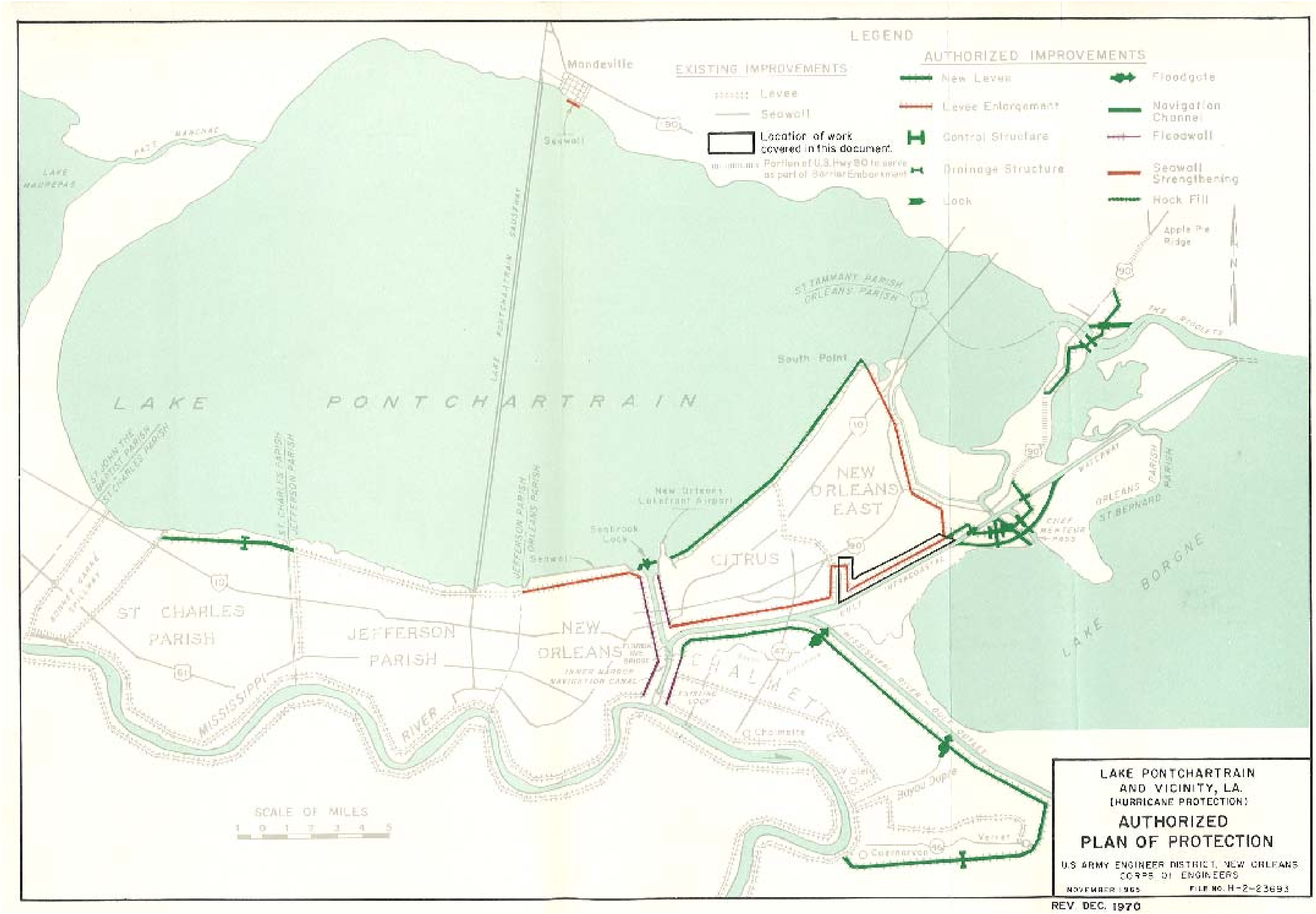
<u>New Orleans East Back Levee (Reference No. 23).</u> Images of map and plan view shown on following pages. The New Orleans East Back Levee is located on the north bank of the MRGO and GIWW and extends from the Michaud Assembly Facility (NASA) to Intersection with the south point to GIWW New Orleans East Levee. This project consists of 4.4 miles of levee to elevation 17.5 ft NGVD and 2 miles of I-wall and inverted T-wall to elevations between 19.0 and 23.0 ft NGVD.

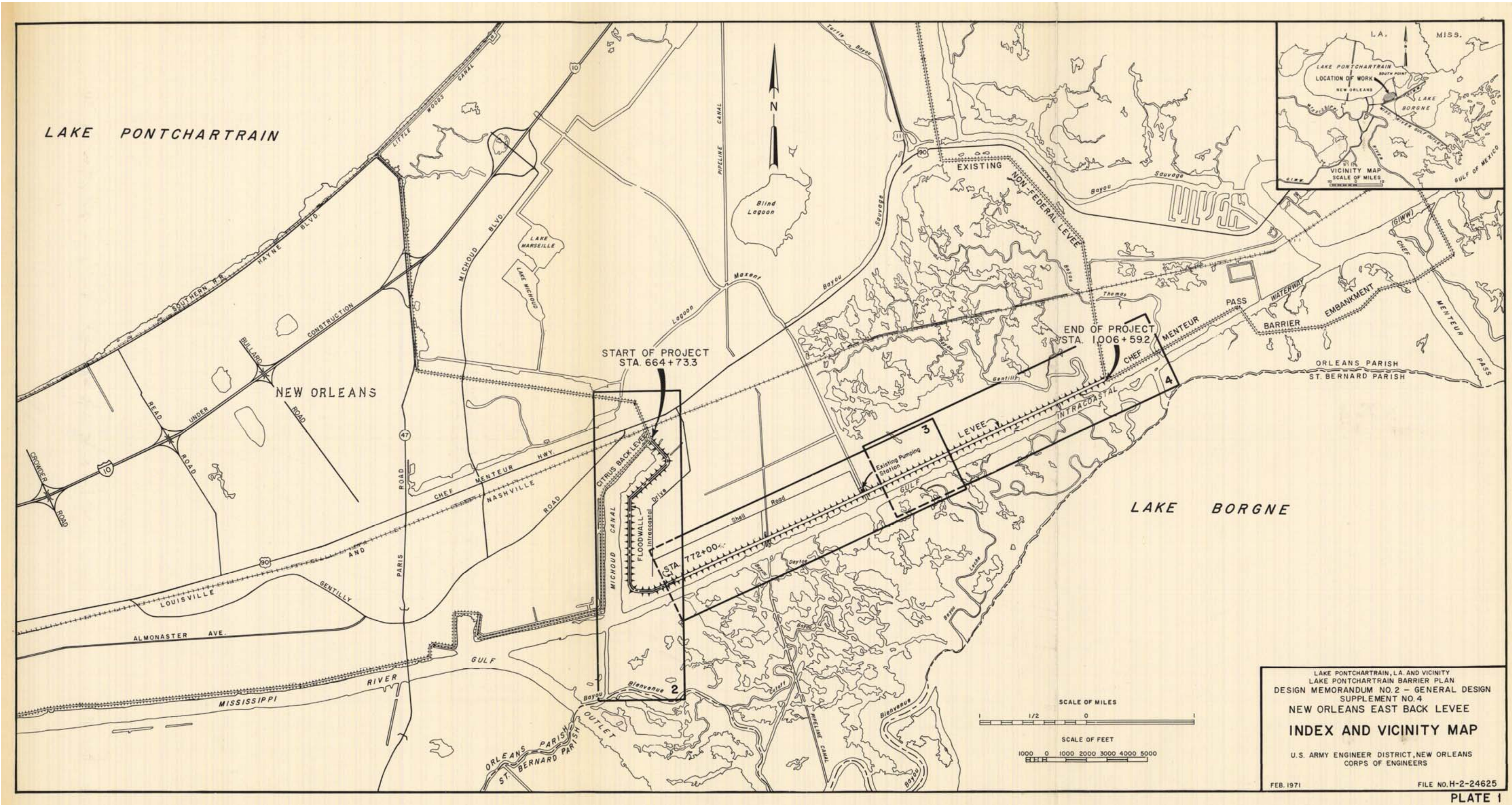
Geology. The geologic history, principal physiographic features, and surface and subsurface geology of the New Orleans area are described in Volume V.

Project Foundation Conditions. The subsurface along this project generally consists of 10 to 15 ft of artificial levee fill overlying 40 to 60 ft of Recent deposits of clays, silts and sands which are underlain by Pleistocene deposits encountered at approximate elevations of -40.0 to -60.0 ft NGVD.

Field Investigation. A total of eight 5-in. undisturbed borings along with 15 1⅞-in. general type core borings were made along the levee alignment.

Underseepage. Not addressed.


LEGEND
EXISTING IMPROVEMENTS
Levee
Seawall
Location of work covered in this document.
Portion of U.S. Hwy 90 to serve as part of Barrier Embankment
AUTHORIZED IMPROVEMENTS
New Levee
Levee Enlargement
Control Structure
Drainage Structure
Lock
Floodgate
Navigation Channel
Floodwall
Seawall Strengthening
Rock Fill
LAKE PONTCHARTRAIN
LAKE MAUREPAS
Mandeville
Seawall
South Point
NEW ORLEANS EAST
CITRUS
NEW ORLEANS
CHALMETTE
ST CHARLES PARISH
JEFFERSON PARISH
LAKE BORGNE
Chalmette
Violet
Bayou Dupre
Caernarvon
Verret
New Orleans Lakefront Airport
Seabrook Lock
INNER HARBOR NAVIGATION CANAL
EXISTING LOCK
MISSISSIPPI RIVER
MISSISSIPPI RIVER - GULF OUTLET
GULF INTRACOASTAL WATERWAY
CHEF MENTEUR PASS
THE RIGOLETS
ST TAMMANY PARISH
ORLEANS PARISH
ST BERNARD PARISH
Apple Pie Ridge
BONNET CARRE SPILLWAY
SCALE OF MILES
1 0 1 2 3 4 5
N
LAKE PONTCHARTRAIN AND VICINITY, LA.
(HURRICANE PROTECTION)
AUTHORIZED PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NOVEMBER 1965
FILE NO. H-2-23693
REV. DEC. 1970


LAKE PONTCHARTRAIN
NEW ORLEANS
LAKE BORGNE
START OF PROJECT
STA. 664+73.3
END OF PROJECT
STA. 1,006+59.2
EXISTING NON-FEDERAL LEVEE
CITRUS BACK LEVEE
FLOODWALL
STA. 772+00
Existing Pumping Station
CHEF MENTEUR PASS
BARRIER EMBANKMENT
ORLEANS PARISH
ST. BERNARD PARISH
MICHOUD CANAL
MISSISSIPPI RIVER GULF OUTLET
INTRACOASTAL WATERWAY
ALMONASTER AVE.
LOUISVILLE AND NASHVILLE
CHEF MENTEUR HWY.
PARIS ROAD
Blind Lagoon
LAKE MARSEILLE
LAKE MICHOUD
VICINITY MAP
SCALE OF MILES
SCALE OF FEET
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 – GENERAL DESIGN
SUPPLEMENT NO. 4
NEW ORLEANS EAST BACK LEVEE
INDEX AND VICINITY MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEB. 1971
FILE NO. H-2-24625
PLATE 1

Hydrostatic Uplift. Not addressed.

Pile Foundation. The pile foundation for T-type floodwalls would be 12-in. prestressed concrete piles. Design compression and tension capacities versus tip elevations were developed using Q and S shear strengths. In compression, a FS of 1.75 with a $K_0 = 1.0$ and for tension a FS of 2.0 with $K_0 = 0.7$ were used. The Q-Case governed the design so capacities for tip elevation versus capacity were presented for 14-in. and 16-in. piles for the Q-Case. During construction, bearing pile tests will be conducted at two sites.

Levee Stability. Slope stability analyses were run for the following conditions:

(1) Hurricane floodwater to the wind tide level (elevation 13.0 ft NGVD) on the floodside and water to natural ground level on the protected side.

(2) Water to elevation 0.0 ft NGVD 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 floodside and 0.0 ft NGVD on the protected side.

The stability analyses were determined by the Method of Planes using Q strengths and a minimum FS with respect to shear strength of approximately 1.3.

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).

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.

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 exists at high elevations, or some value of shear strength is analyzed as if the sheet piling is not existent. Any sections where this change in concept caused an appreciable change in FS 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 FSs. 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 FSs 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.

Citrus Back Levee (Reference No. 24). Images of map and plan view shown on following pages. The Citrus Back Levee is located on the north bank of the MRGO 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.

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

Project Foundation Conditions. The subsurface along the project consists generally of 10 to 15 ft of artificial levee fill overlying 45 to 60 ft of Recent deposits of clays, silts, and sands which are underlain by a Pleistocene deposit encountered at elevation -50 ft NGVD on the west end to -60 ft NGVD on the east end.

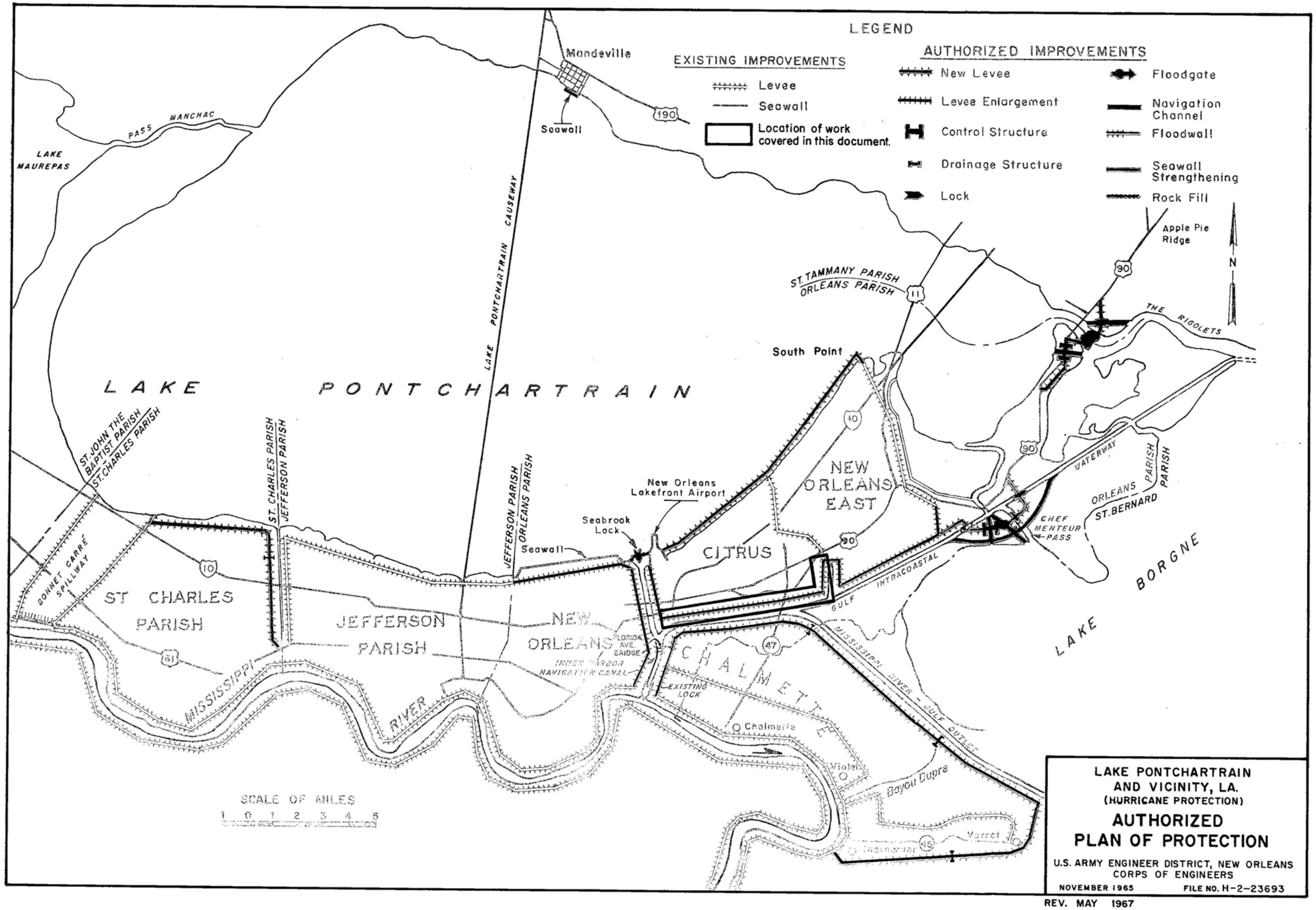
Field Investigation. Four 5-in.-diameter undisturbed soil borings were made along the levee alignment. Fifteen 1⅞-in.-ID general type core borings were also made. In order to insure adequate design, additional soil borings were scheduled between successive lifts.

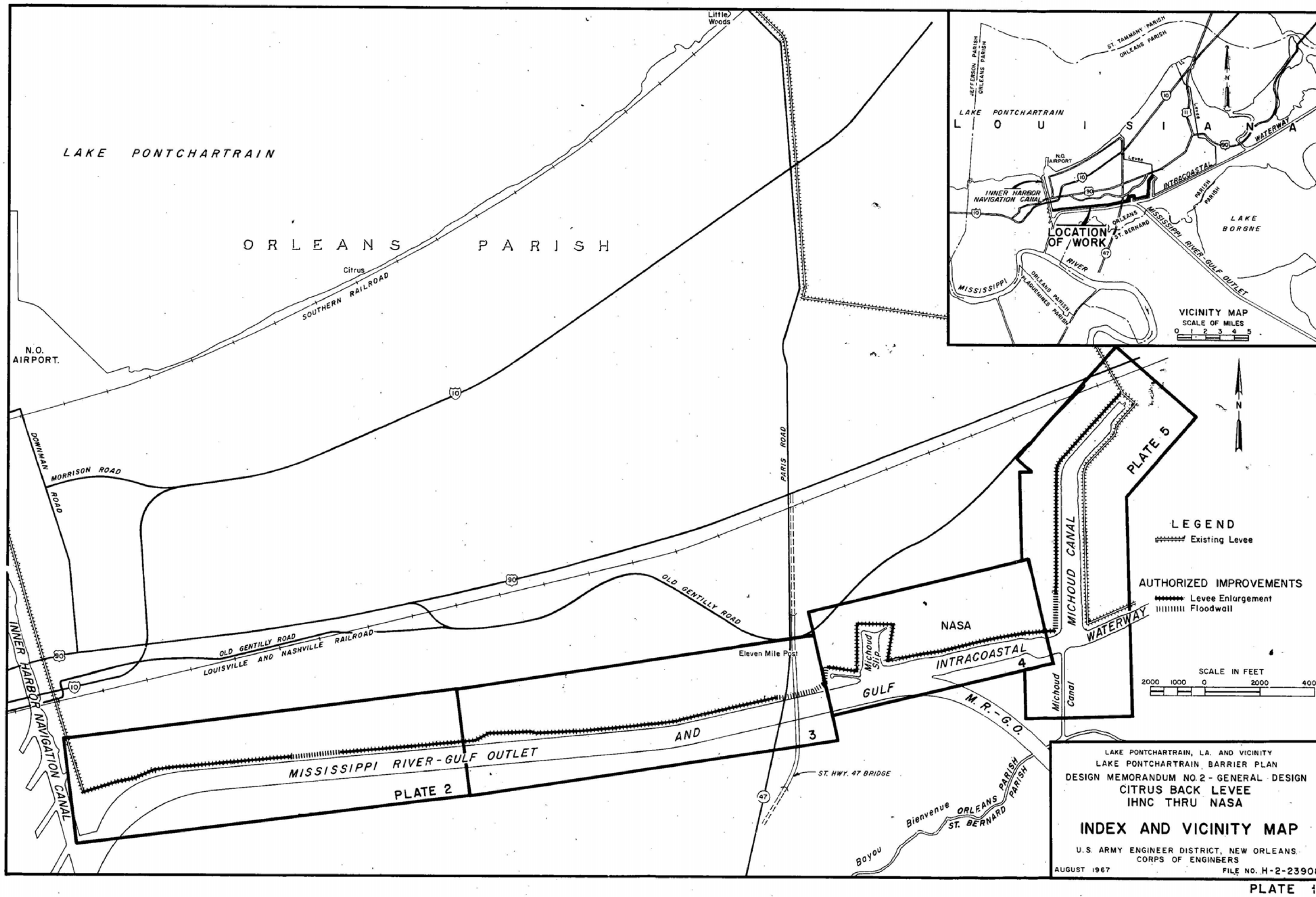
Underseepage. A steel sheet-pile cutoff will be used beneath the short reach of T-wall to protect from underseepage.

Hydrostatic Uplift. Not addressed in this DM.

Pile Foundation. Twelve-inch-square prestressed concrete piles will be used to support the T-type walls and gated structures. Bearing and tension values versus tip elevation will be computed using the Q and S shear strengths. In compression, a FS of 1.75 was applied with a K_0 of 1.0. In tension, a FS of 2.0 and K_0 of 1.7 was used. The S-Case governed the design.

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:


LEGEND
EXISTING IMPROVEMENTS
Levee
Seawall
Location of work covered in this document.
AUTHORIZED IMPROVEMENTS
New Levee
Levee Enlargement
Control Structure
Drainage Structure
Lock
Floodgate
Navigation Channel
Floodwall
Seawall Strengthening
Rock Fill
LAKE PONTCHARTRAIN
LAKE MAUREPAS
PASS MANCHAC
Mandeville
Seawall
LAKE PONTCHARTRAIN CAUSEWAY
ST. TAMMANY PARISH
ORLEANS PARISH
South Point
Apple Pie Ridge
THE RIGOLETS
N
ST. JOHN THE BAPTIST PARISH
ST. CHARLES PARISH
ST. CHARLES PARISH
JEFFERSON PARISH
JEFFERSON PARISH
ORLEANS PARISH
BONNET CARRÉ SPILLWAY
ST CHARLES PARISH
JEFFERSON PARISH
NEW ORLEANS
New Orleans Lakefront Airport
Seabrook Lock
Seawall
CITRUS
NEW ORLEANS EAST
CHEF MENTEUR PASS
ORLEANS PARISH
ST. BERNARD PARISH
WATERWAY
GULF INTRACOASTAL
LAKE BORGNE
FLORIDA AVE. BRIDGE
INNER HARBOR NAVIGATION CANAL
EXISTING LOCK
CHALMETTE
Chalmette
Violet
Bayou Dupre
Verret
MISSISSIPPI RIVER GULF OUTLET
MISSISSIPPI RIVER
SCALE OF MILES
1 0 1 2 3 4 5
LAKE PONTCHARTRAIN AND VICINITY, LA.
(HURRICANE PROTECTION)
AUTHORIZED PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NOVEMBER 1965
FILE NO. H-2-23693
REV. MAY 1967


LAKE PONTCHARTRAIN
ORLEANS PARISH
N.O. AIRPORT.
SOUTHERN RAILROAD
Citrus
Little Woods
MORRISON ROAD
DOWNMAN ROAD
PARIS ROAD
OLD GENTILLY ROAD
LOUISVILLE AND NASHVILLE RAILROAD
Eleven Mile Post
INNER HARBOR NAVIGATION CANAL
MISSISSIPPI RIVER-GULF OUTLET
INTRACOASTAL
GULF
AND
WATERWAY
M. R.-G. O.
NASA
Michoud Slip
MICHOUD CANAL
Michoud Canal
ST. HWY. 47 BRIDGE
Bayou Bienvenue
ORLEANS PARISH
ST. BERNARD PARISH
PLATE 2
3
4
PLATE 5
LEGEND
Existing Levee
AUTHORIZED IMPROVEMENTS
Levee Enlargement
Floodwall
SCALE IN FEET
2000 1000 0 2000 4000
VICINITY MAP
SCALE OF MILES
0 1 2 3 4 5
LOUISIANA
LAKE PONTCHARTRAIN
LAKE BORGNE
LOCATION OF WORK
MISSISSIPPI RIVER
INNER HARBOR NAVIGATION CANAL
N.O. AIRPORT
JEFFERSON PARISH
ORLEANS PARISH
ST. TAMMANY PARISH
PLAQUEMINES PARISH
INTRACOASTAL WATERWAY
MISSISSIPPI RIVER-GULF OUTLET
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
CITRUS BACK LEVEE
IHNC THRU NASA
INDEX AND VICINITY MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
AUGUST 1967
FILE NO. H-2-23908
PLATE 1

hurricane water condition at still-water level + 13.0 ft 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 FS of 1.3 with respect to shear strengths.

I-Wall Design. The penetrations for the sheet piles were computed for a dynamic water force and a minimum FS of 1.25, and a static-water level of 14.5 ft NGVD with a FS 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.

Erosion Protection. Due to short duration of hurricane stages and the resistant nature of the clays, no erosion protection is considered necessary.

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.

<u>New Orleans East, IHNC Floodwall/Levee (Reference No. 9).</u> Map of area shown on following page. 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 less than 10 ft and a bearing pile supported T-type floodwall for all heights greater than 10 ft.

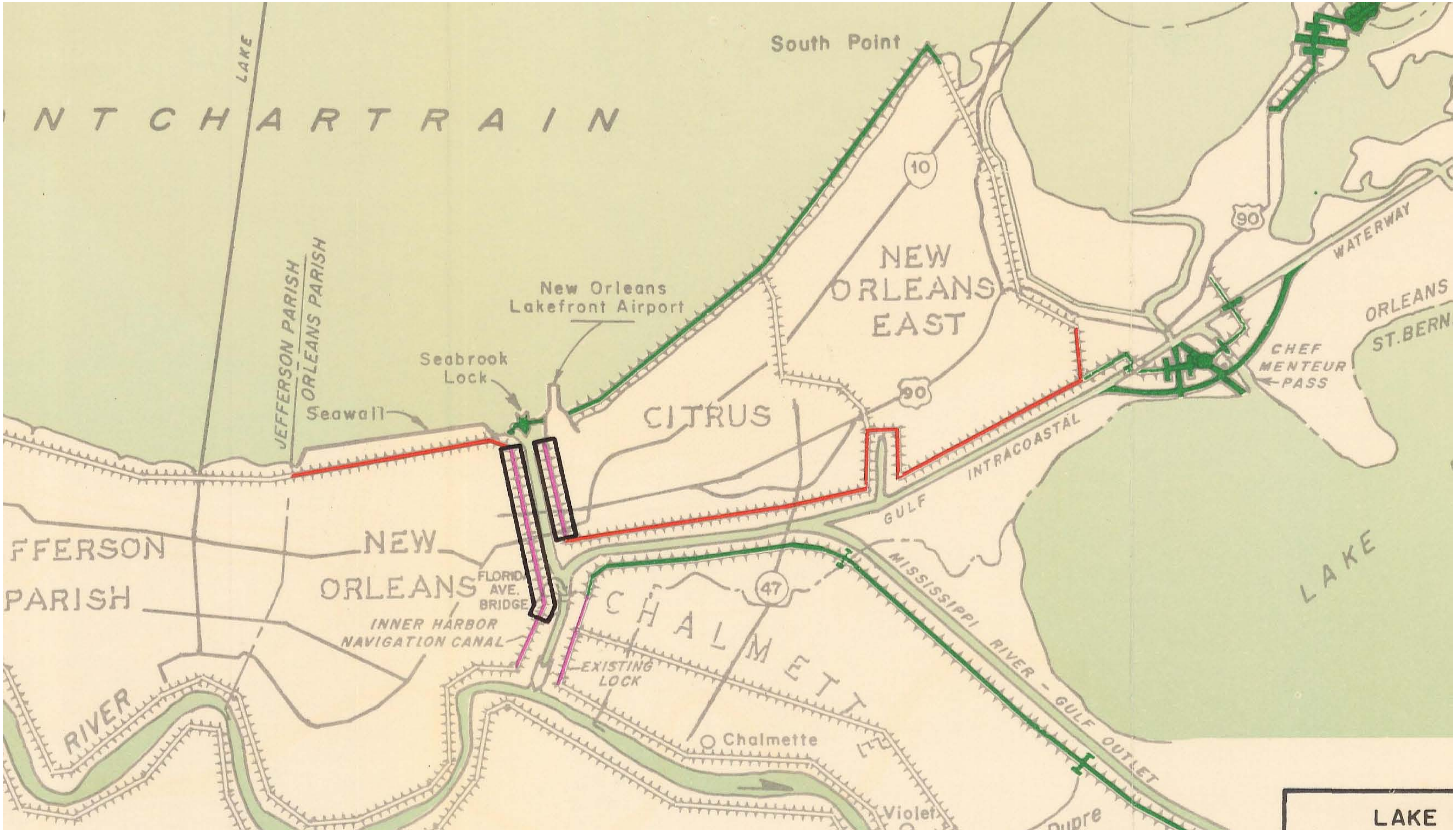
Geology. The geologic history and principal physiographic features are described in Volume V.

Project Foundation Conditions. The subsurface along the project consists of generally 6 to 10 ft of artificial fill overlying 40 to 50 ft of Recent deposits of sand, silts, and clays which are underlain by the Pleistocene soils. The Pleistocene surface is encountered at elevation -50 ft NGVD on the Citrus Lakefront end to -70 ft NGVD at various locations.

Field Investigation. Nine 5-in.-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 ft.

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.

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.


NTCHARTRAIN
LAKE
South Point
10
NEW ORLEANS EAST
90
WATERWAY
ORLEANS
ST.BERN
CHEF MENTEUR PASS
New Orleans Lakefront Airport
Seabrook Lock
Seawall
JEFFERSON PARISH
ORLEANS PARISH
CITRUS
90
INTRACOASTAL
GULF
FFERSON PARISH
NEW ORLEANS
FLORIDA AVE. BRIDGE
INNER HARBOR NAVIGATION CANAL
47
CHALMETTE
EXISTING LOCK
MISSISSIPPI RIVER - GULF OUTLET
LAKE
RIVER
Chalmette
Violet
Dupre
LAKE

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:

FS Compression	1.75	$K_0 = 1.0$
FS Tension	2.0	$K_0 = 0.7$
S-Case governed		
S strength	Recent clays	$\phi = 23°$, $c = 0$
	Pleistocene clays	$\phi = 25°$, $c = 0$
	Sand	$\phi = 30°$, $c = 0$

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

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 FS 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.

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 FS of 1.5 was applied to the design shear strengths. The stability of the floodwall was determined for a hurricane water level 6 in. below the top of the wall on the floodside and for the groundwater level at the ground surface on the protected side.

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.

Review Comments. First Endorsement Comment paragraph 5 questioned the use of a 9-ft I-wall to hold back earth fill at the Chef Menteur Bridge with a compiled deflection of 4 in..

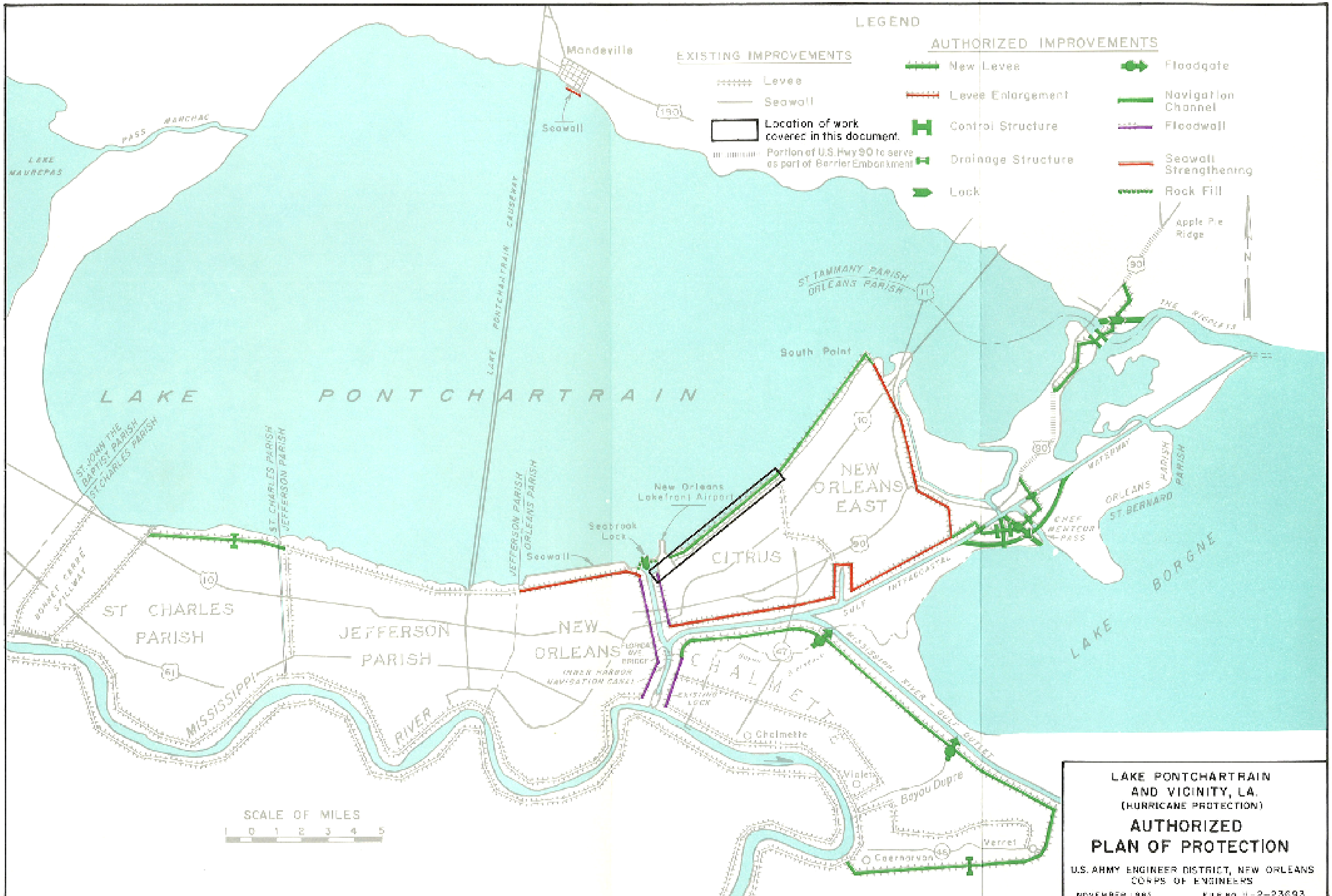
Paragraph 6 questions excessive recommended depth of sheet pile Station 115+65 to 117+50 and 119+59 to 132+00.

Paragraph 9 questions use of I-walls between 8 and 10 ft 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.

Structural.

New Orleans East Lakefront.

Citrus Lakefront Levee (Reference 17). Map of area shown on following page.


LEGEND
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Levee
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Location of work covered in this document.
Portion of U.S. Hwy 90 to serve as part of Barrier Embankment
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LAKE MAUREPAS
PASS MANCHAC
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ST. CHARLES PARISH
JEFFERSON PARISH
BONNET CARRE SPILLWAY
ST CHARLES PARISH
JEFFERSON PARISH
NEW ORLEANS
INNER HARBOR NAVIGATION CANAL
EXISTING LOCK
CHALMETTE
Chalmette
Violet
Bayou Dupre
Verret
Caernarvon
MISSISSIPPI RIVER
MISSISSIPPI RIVER - GULF OUTLET
GULF INTRACOASTAL WATERWAY
SCALE OF MILES
1 0 1 2 3 4 5
LAKE PONTCHARTRAIN AND VICINITY, LA.
(HURRICANE PROTECTION)
AUTHORIZED PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NOVEMBER 1965
FILE NO. H-2-23693

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 Boulevard 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 (ft msl)**
Wind tide level (IHNC)	13.0
Wind tide level (Lake Pontchartrain)	8.5
Landside of Floodwall	0.0
Unit Weights	**lb/cu ft**
Water	62.5
Concrete	150
Steel	490
Design Loads	**lb/cu ft**
Wind load	50 lb/sq ft

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 psi, except for prestressed concrete piling where the minimum is 5, 000 psi. 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 psi
f_c	1,050 psi
V_c (without web reinforcement)	60 psi
V_c (with web reinforcement)	274 psi
f_s	20,000 psi
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 psi

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 in. below top of wall, no wind, no dynamic wave force.

T-type floodwall. Four T-wall monoliths were constructed along the east side 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-in.-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 Boulevard 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 elevation 9.75 ft of wall, no wind, impervious sheet-pile cutoff
- Case IV. Water at elevation 9.75 ft of wall, no wind, pervious sheet-pile cutoff
- Case V. No water, no wind, truck on edge of slab, floodside
- Case VI. No water, no wind, truck on edge of slab, protected side
- Case VII. No water, wind from floodside, truck on edge of slab, protected side, 33 ⅓ percent increase in allowable stresses
- Case VIII. No water, wind from protected side, truck on edge of slab, floodside, 33 ⅓ 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 floodside 33 1/3 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.

Paris Road to South Point (Reference 20). Map of area shown on following page.

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-in.-pipeline crossing, and one soil-founded, sluice-gated drainage structure at the South Point edge of the project.

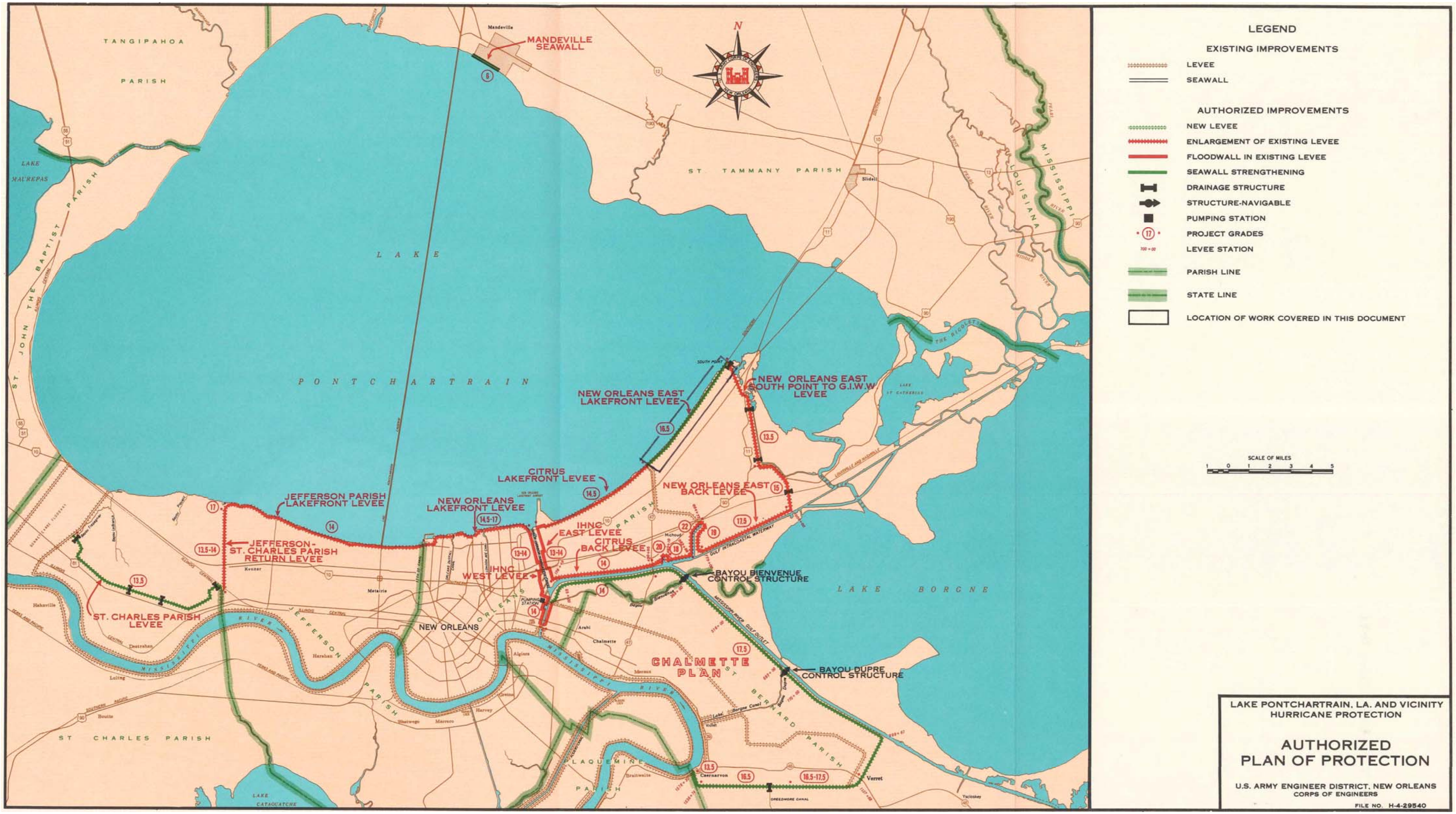
The basic data relevant to the design of the protective works are shown in the following tabulation:

Water Elevations	**Elevation (ft NGVD)**
Wind tide level (Lake Pontchartrain)	11.5
Landside of floodwall	0.0
Unit Weights	**lb/cu ft**
Water	64.0
Concrete	150
Design Loads	**lb/cu ft**
Wind load	50 lb/sq ft

Design Methods.

Reinforced concrete. The design of reinforced concrete structures was 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 balance ratio
Minimum flexural reinforcement	200/fy


LEGEND
EXISTING IMPROVEMENTS
LEVEE
SEAWALL
AUTHORIZED IMPROVEMENTS
NEW LEVEE
ENLARGEMENT OF EXISTING LEVEE
FLOODWALL IN EXISTING LEVEE
SEAWALL STRENGTHENING
DRAINAGE STRUCTURE
STRUCTURE-NAVIGABLE
PUMPING STATION
PROJECT GRADES
LEVEE STATION
PARISH LINE
STATE LINE
LOCATION OF WORK COVERED IN THIS DOCUMENT
SCALE OF MILES
LAKE PONTCHARTRAIN, LA. AND VICINITY
HURRICANE PROTECTION
AUTHORIZED
PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FILE NO. H-4-29540
N
MANDEVILLE SEAWALL
TANGIPAHOA PARISH
ST. TAMMANY PARISH
ST. JOHN THE BAPTIST PARISH
LAKE MAUREPAS
LAKE PONTCHARTRAIN
LAKE BORGNE
NEW ORLEANS EAST LAKEFRONT LEVEE
NEW ORLEANS EAST SOUTH POINT TO G.I.W.W. LEVEE
CITRUS LAKEFRONT LEVEE
NEW ORLEANS EAST BACK LEVEE
JEFFERSON PARISH LAKEFRONT LEVEE
NEW ORLEANS LAKEFRONT LEVEE
IHNC EAST LEVEE
CITRUS BACK LEVEE
JEFFERSON-ST. CHARLES PARISH RETURN LEVEE
IHNC WEST LEVEE
BAYOU BIENVENUE CONTROL STRUCTURE
BAYOU DUPRE CONTROL STRUCTURE
ST. CHARLES PARISH LEVEE
NEW ORLEANS
CHALMETTE PLAN
ST. BERNARD PARISH
JEFFERSON PARISH
ORLEANS PARISH
PLAQUEMINES PARISH
ST. CHARLES PARISH
MISSISSIPPI RIVER
LOUISIANA
MISSISSIPPI
PEARL RIVER
LAKE CATAOUATCHE
PUMPING STATION
GULF INTRACOASTAL WATERWAY
MISSISSIPPI RIVER GULF OUTLET
SOUTH POINT
Mandeville
Slidell
Metairie
Kenner
Arabi
Chalmette
Verret
Caernarvon
Harvey
Marrero
Westwego
Algiers
Gretna
Harahan
Luling
Boutte
Hahnville
Destrehan
LAKE ST. CATHERINE
THE RIGOLETS
6
18.5
13.5
15
17.5
14.5
14
14.5-17
13-14
13.5-14
17
22
19
20
18
16.5
16.5-17.5

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 ft of the sheet piling extending above the finished ground elevation. The concrete portion of the floodwall extended from 2 ft 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:

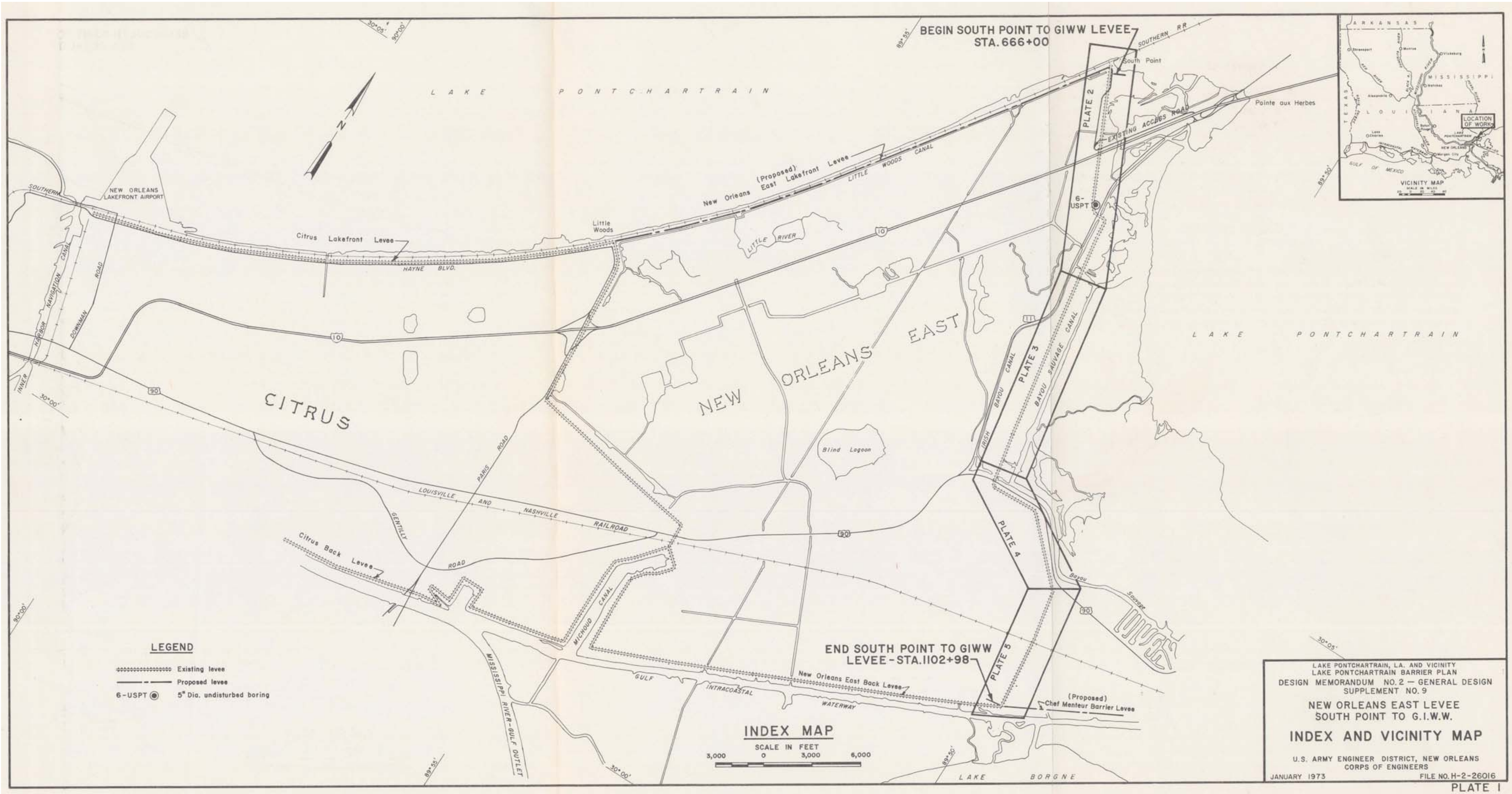
FS used = 1.25 with static water at the swl and a dynamic wave force.

New Orleans East Levee. New Orleans East Levee, South Point to GIWW (Reference 21). Image of area shown on following page.

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 (ft MSL)**
Wind tide level	
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/cu ft**
Water	62.5
Concrete	150
Steel	490
Earth	117
Design loads	
Earth pressure (lateral)	
Clay	90 lb/sq ft
Sand	65 lb/sq ft
Wind pressure (lateral)	50 lb/sq ft



BEGIN SOUTH POINT TO GIWW LEVEE
STA. 666+00
LAKE PONTCHARTRAIN
NEW ORLEANS LAKEFRONT AIRPORT
Citrus Lakefront Levee
New Orleans East Lakefront Levee (Proposed)
CITRUS
NEW ORLEANS EAST
PLATE 2
PLATE 3
PLATE 4
PLATE 5
6-USPT
South Point
Pointe aux Herbes
Blind Lagoon
Citrus Back Levee
END SOUTH POINT TO GIWW LEVEE - STA.1102+98
New Orleans East Back Levee
(Proposed) Chef Menteur Barrier Levee
LAKE BORGNE
LEGEND
Existing levee
Proposed levee
6-USPT 5" Dia. undisturbed boring
INDEX MAP
SCALE IN FEET
3,000 0 3,000 6,000
VICINITY MAP
LOCATION OF WORK
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 — GENERAL DESIGN
SUPPLEMENT NO. 9
NEW ORLEANS EAST LEVEE
SOUTH POINT TO G.I.W.W.
INDEX AND VICINITY MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
JANUARY 1973
FILE NO. H-2-26016
PLATE 1

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 psi, except for prestressed concrete piling where the minimum is 5,000 psi. 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 psi
f_c	1,050 psi
V_c (without web reinforcement)	60 psi
V_c (with web reinforcement)	274 psi
f_s	20,000 psi
Minimum tensile steel	0.00025 bd sq in.
Structural Steel (ASTM A-36)	
Basic working stress	18,000 psi

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:

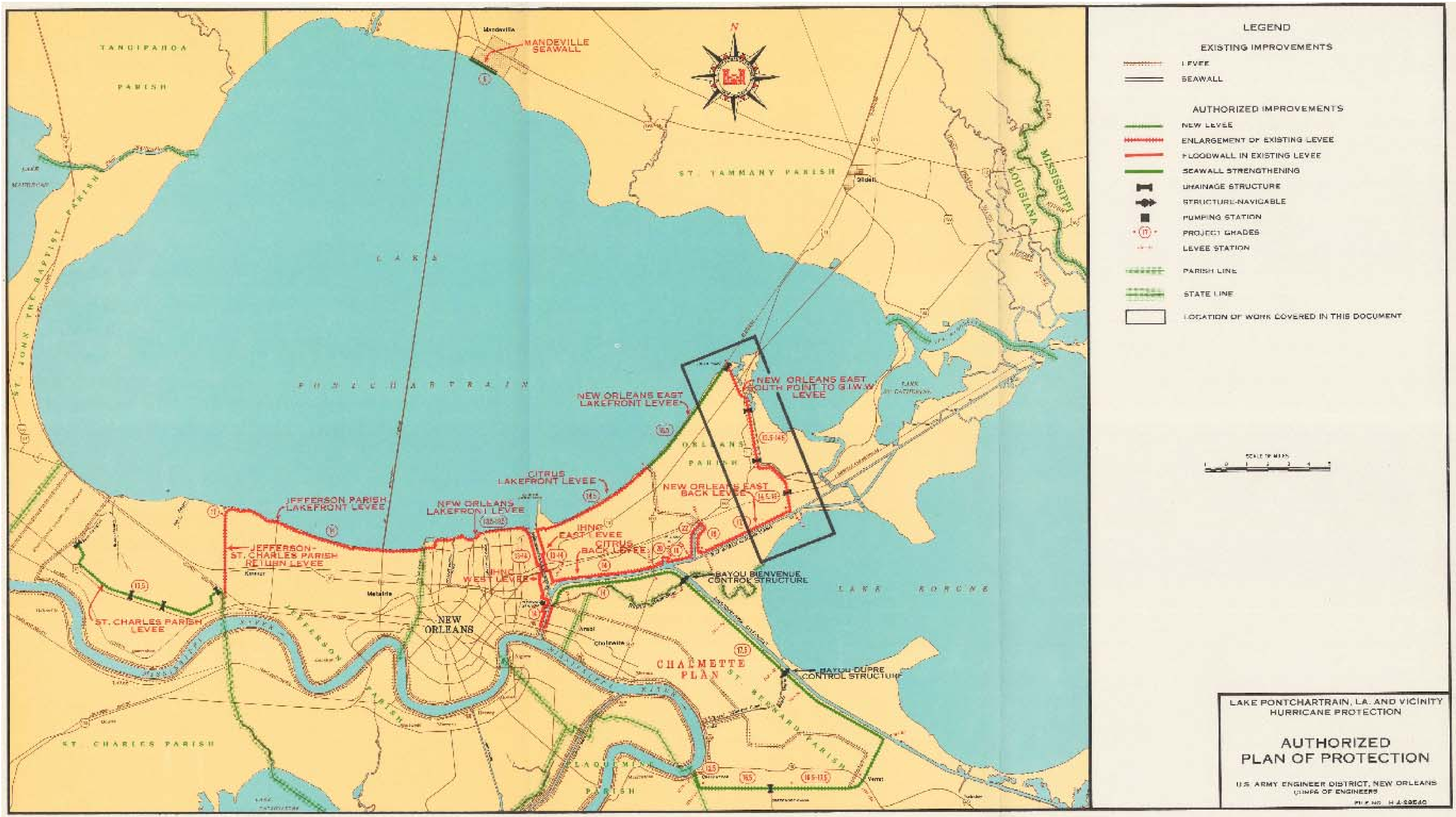
- Case I – Water to top of wall at elevation 13.0 ft NGVD, no wind, impervious soil
- Case II – Water to top of wall at elevation 13.0 ft NGVD, no wind, pervious soil
- Case III – No water, no wind, one train wheel axle on edge of slab, floodside
- Case IV – No water, no wind, one train axle on edge of slab, protected side
- Case V – No water, no wind, two train wheel axles on edge of slab, floodside
- Case VI – No water, no wind, two train wheel axles on edge of slab, protected side.

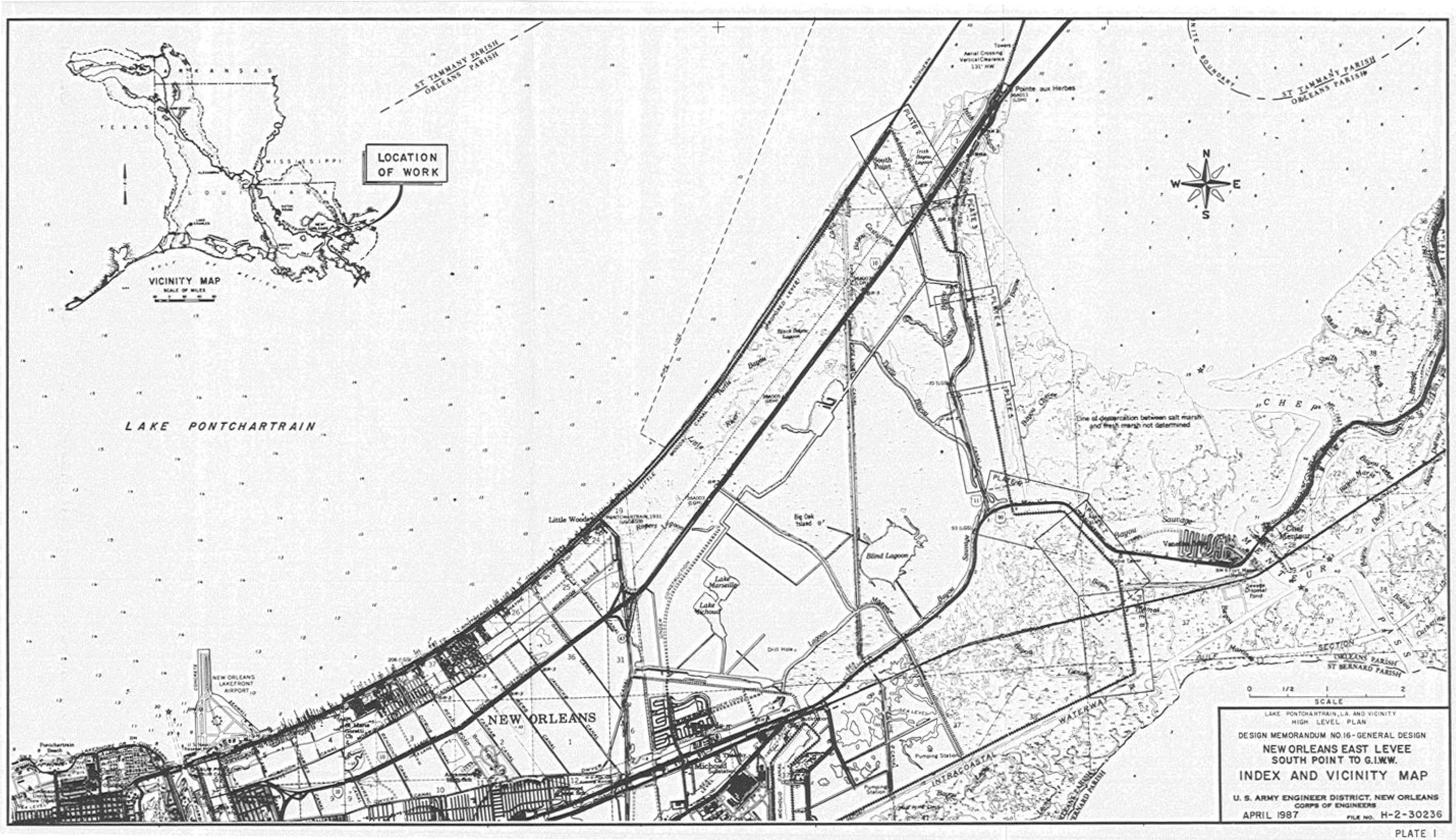
I-Type Wall Structure. (Reference 22) Images of map and plan view are shown on following pages.

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 (ft NGVD)**
Wind tide level (WTL)	
Drainage Structure	13.0
Landside of Structures	0.0


LEGEND
EXISTING IMPROVEMENTS
LEVEE
SEAWALL
AUTHORIZED IMPROVEMENTS
NEW LEVEE
ENLARGEMENT OF EXISTING LEVEE
FLOODWALL IN EXISTING LEVEE
SEAWALL STRENGTHENING
DRAINAGE STRUCTURE
STRUCTURE-NAVIGABLE
PUMPING STATION
PROJECT GRADES
LEVEE STATION
PARISH LINE
STATE LINE
LOCATION OF WORK COVERED IN THIS DOCUMENT
LAKE PONTCHARTRAIN, LA. AND VICINITY
HURRICANE PROTECTION
AUTHORIZED
PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
MANDEVILLE SEAWALL
TANGIPAHOA PARISH
ST. TAMMANY PARISH
LAKE PONTCHARTRAIN
NEW ORLEANS EAST LAKEFRONT LEVEE
NEW ORLEANS EAST SOUTH POINT TO G.I.W.W. LEVEE
ORLEANS PARISH
CITRUS LAKEFRONT LEVEE
NEW ORLEANS EAST BACK LEVEE
JEFFERSON PARISH LAKEFRONT LEVEE
NEW ORLEANS LAKEFRONT LEVEE
IHNC EAST LEVEE
CITRUS BACK LEVEE
JEFFERSON-ST. CHARLES PARISH RETURN LEVEE
IHNC WEST LEVEE
BAYOU BIENVENUE CONTROL STRUCTURE
LAKE BORGNE
ST. CHARLES PARISH LEVEE
NEW ORLEANS
CHALMETTE PLAN
BAYOU DUPRE CONTROL STRUCTURE
ST. BERNARD PARISH
ST. CHARLES PARISH
PLAQUEMINES PARISH
MISSISSIPPI
LOUISIANA
ST. JOHN THE BAPTIST PARISH


ST TAMMANY PARISH
ORLEANS PARISH
LOCATION OF WORK
VICINITY MAP
TEXAS
MISSISSIPPI
LOUISIANA
Pointe aux Herbes
South Point
N
W
E
S
Line of demarcation between salt marsh and fresh marsh not determined
LAKE PONTCHARTRAIN
Little Woods
Big Oak Island
Blind Lagoon
Lake Marseille
Lake Michoud
Drill Hole
NEW ORLEANS LAKEFRONT AIRPORT
NEW ORLEANS
Pumping Station
INTRACOASTAL WATERWAY
ORLEANS PARISH
ST BERNARD PARISH
SCALE
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO.16-GENERAL DESIGN
NEW ORLEANS EAST LEVEE
SOUTH POINT TO G.I.W.W.
INDEX AND VICINITY MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
APRIL 1987
FILE NO. H-2-30236
PLATE I

GIWW. New Orleans East Back Levee (Reference 23). Image of area shown on following page.

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 ft NGVD. 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:

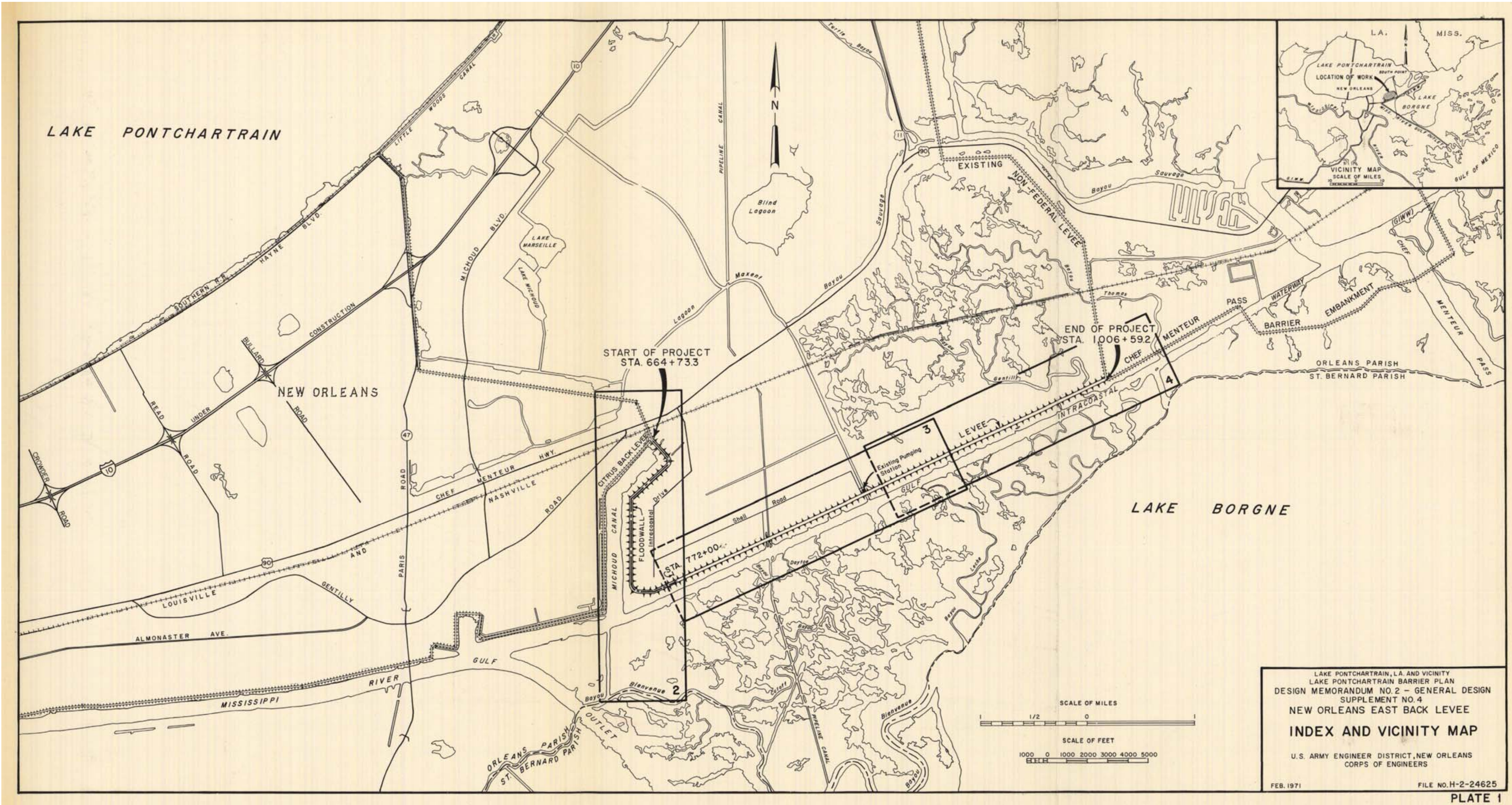
Water Elevations	**Elevation (ft MSL)**
Still-water Level	13.0
Landside of Wall	0.0
Unit Weights	**lb/cu ft**
Water	62.5
Concrete	150
Steel	490
Design Loads	**lb/cu ft**
Wind loads	50

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 ft NGVD, 1.5 FS in the soil, no dynamic wave force
- Case II – Static water to top of broken wave, 1.25 FS 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 wind tide level elevation 13.0 ft, no wave, no wind
- Case II – Water at wind tide level elevation 13.0 ft, no wave, wind from floodside. 33 ⅓ percent increase in allowable stresses
- Case III – Broken waves to elevation 18.8 ft, wave force, wind from floodside. 33 ⅓ percent increase in allowable stresses.
- Case IV – No water or wave force, wind from protected side; 33 ⅓ percent increase in allowable stresses.


LAKE PONTCHARTRAIN
NEW ORLEANS
LAKE BORGNE
N
START OF PROJECT
STA. 664+73.3
END OF PROJECT
STA. 1,006+59.2
STA. 772+00
CITRUS BACK LEVEE
FLOODWALL
MICHOUD CANAL
EXISTING NON-FEDERAL LEVEE
Existing Pumping Station
INTRACOASTAL LEVEE
GULF
CHEF MENTEUR PASS
BARRIER EMBANKMENT
WATERWAY (GIWW)
ORLEANS PARISH
ST. BERNARD PARISH
MISSISSIPPI RIVER GULF OUTLET
ALMONASTER AVE.
GENTILLY
LOUISVILLE
CHEF MENTEUR HWY.
NASHVILLE ROAD
PARIS ROAD
MICHOUD BLVD.
HAYNE BLVD.
SOUTHERN R.R.
LAKE MARSEILLE
LAKE MICHOUD
Blind Lagoon
Maxent Lagoon
PIPELINE CANAL
Bayou Sauvage
Shell Road
Bayou Bienvenue
CROWDER ROAD
BULLARD ROAD
READ ROAD
UNDER CONSTRUCTION
VICINITY MAP
SCALE OF MILES
LAKE PONTCHARTRAIN
LOCATION OF WORK
NEW ORLEANS
LAKE BORGNE
GULF OF MEXICO
LA.
MISS.
SCALE OF MILES
SCALE OF FEET
1000 0 1000 2000 3000 4000 5000
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 – GENERAL DESIGN
SUPPLEMENT NO. 4
NEW ORLEANS EAST BACK LEVEE
INDEX AND VICINITY MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FEB. 1971
FILE NO. H-2-24625
PLATE 1

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 wind tide level elevation 13.0 ft, no wave, no wind, discharge pipes filled with water
- Case II - Water at wind tide level elevation 13.0 ft, no wave, no wind, discharge pipes empty
- Case III - Water at wind tide level elevation 13.0 ft, no wave, wind from floodside, discharge pipes filled with water; 33 ⅓ percent increase in allowable stresses
- Case IV - Water at wind tide level elevation 13.0 ft, no wave, wind from floodside, discharge pipes empty; 33 ⅓ increase in allowable stresses
- Case V - Broken waves to elevation 17.4 ft, wave force, wind from floodside, discharge pipes filled with water; 33 ⅓ percent increase in allowable stresses
- Case VI - Broken waves to elevation 17.4 ft, wave force, wind from floodside, discharge pipes empty; 33 ⅓ percent increase in allowable stresses
- Case VII - No water or wave force, wind from protected side, discharge pipes empty; 33 ⅓ 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 wind tide level elevation 13.0 ft, 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 floodside
- Case IV - Water at wind tide level elevation 13.0 ft, no wave, wind from floodside; 33 ⅓ percent increase in allowable stresses
- Case V - Broken waves to elevation 18.8 ft, wave force, wind from floodside; 33 ⅓ percent increase in allowable stresses
- Case VI - No water, no wave, wind from floodside, truck loading on edge of slab at protected side; 33 ⅓ percent increase in allowable stresses
- Case VII - No water, no wave, wind from protected side, truck loading on edge of slab at floodside; 33 ⅓ percent increase in allowable stresses.

<u>Citrus Back Levee (Reference 24).</u> Image of area shown on following page.

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 New Orleans Public Service, Inc. (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 (ft NGVD)**
Still-water Level	13.0
Landside of Wall	0.0
Unit Weights	**lb/cu ft**
Water	62.5
Concrete	150
Steel	490
Design Loads	**lb/cu ft**
Wind loads	
On Walls	50 lb/sq ft
On Overhead Beams	30 lb/sq ft

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 psi, except for prestressed concrete piling where the minimum is 5,000 psi, 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 psi
f_c	1,050 psi
V_c (without web reinforcement)	60 psi
V_c (with web reinforcement)	274 psi
f_s	20,000 psi
Minimum tensile steel	0.00025 bd sq in.
Structural Steel (ASTM A-36)	
Basic working stress	18,000 psi

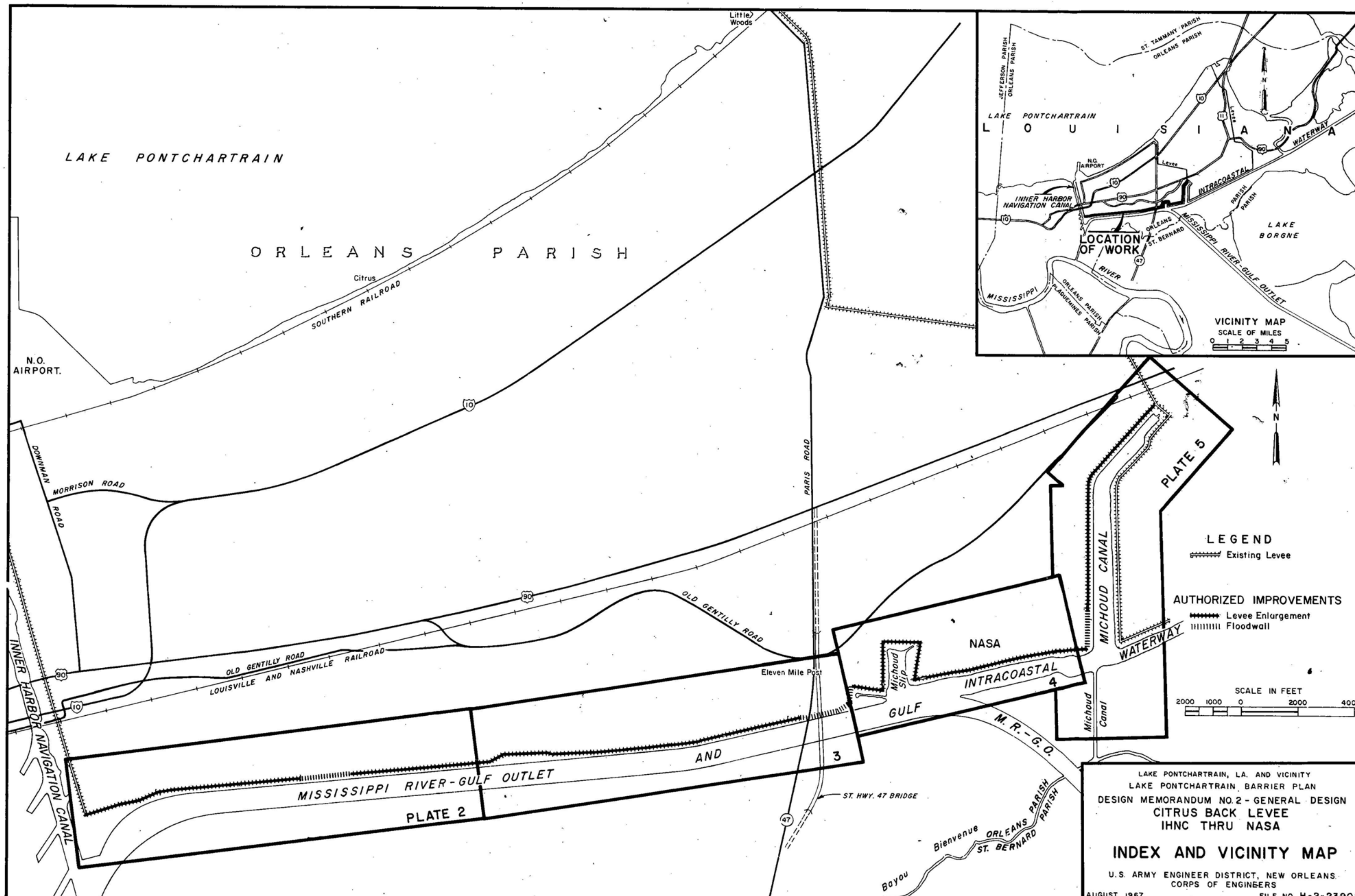

LAKE PONTCHARTRAIN
ORLEANS PARISH
N.O. AIRPORT.
Little Woods
Citrus
SOUTHERN RAILROAD
MORRISON ROAD
DOWNMAN ROAD
PARIS ROAD
OLD GENTILLY ROAD
LOUISVILLE AND NASHVILLE RAILROAD
Eleven Mile Post
INNER HARBOR NAVIGATION CANAL
MISSISSIPPI RIVER-GULF OUTLET
PLATE 2
3
AND
GULF
INTRACOASTAL
WATERWAY
4
NASA
Michoud Slip
MICHOUD CANAL
Michoud Canal
PLATE 5
M. R.-G. O.
ST. HWY. 47 BRIDGE
Bayou Bienvenue
ORLEANS PARISH
ST. BERNARD PARISH
LEGEND
Existing Levee
AUTHORIZED IMPROVEMENTS
Levee Enlargement
Floodwall
SCALE IN FEET
2000 1000 0 2000 4000
N
VICINITY MAP
SCALE OF MILES
0 1 2 3 4 5
ST. TAMMANY PARISH
ORLEANS PARISH
JEFFERSON PARISH
LAKE PONTCHARTRAIN
L O U I S I A N A
N.O. AIRPORT
INNER HARBOR NAVIGATION CANAL
LOCATION OF WORK
INTRACOASTAL WATERWAY
MISSISSIPPI RIVER-GULF OUTLET
LAKE BORGNE
MISSISSIPPI RIVER
ORLEANS PARISH
PLAQUEMINES PARISH
ST. BERNARD
LAKE PONTCHARTRAIN, LA. AND VICINITY
LAKE PONTCHARTRAIN BARRIER PLAN
DESIGN MEMORANDUM NO. 2 - GENERAL DESIGN
CITRUS BACK LEVEE
IHNC THRU NASA
INDEX AND VICINITY MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
AUGUST 1967
FILE NO. H-2-23908

PLATE 1

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 FS in the soil of 1.5, for a floodside water elevation of 14.5 ft and checked for water to the top of the wall at elevation 15.0 ft. 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 ft), 1.5 FS in the soil, no dynamic wave force

- Case II – Static water to top of broken wave, 1.25 FS 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 ft on the floodside and water at elevation 5.5 ft 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 flood-side of cutoff, and tailwater uplift on protected side of cutoff

- Case III – Water at elevation 12.5 ft on floodside and water at elevation 5.5 ft 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 sheet-pile cutoff impervious. Full head uplift on floodside of cutoff, and tailwater uplift on protected side of cutoff.

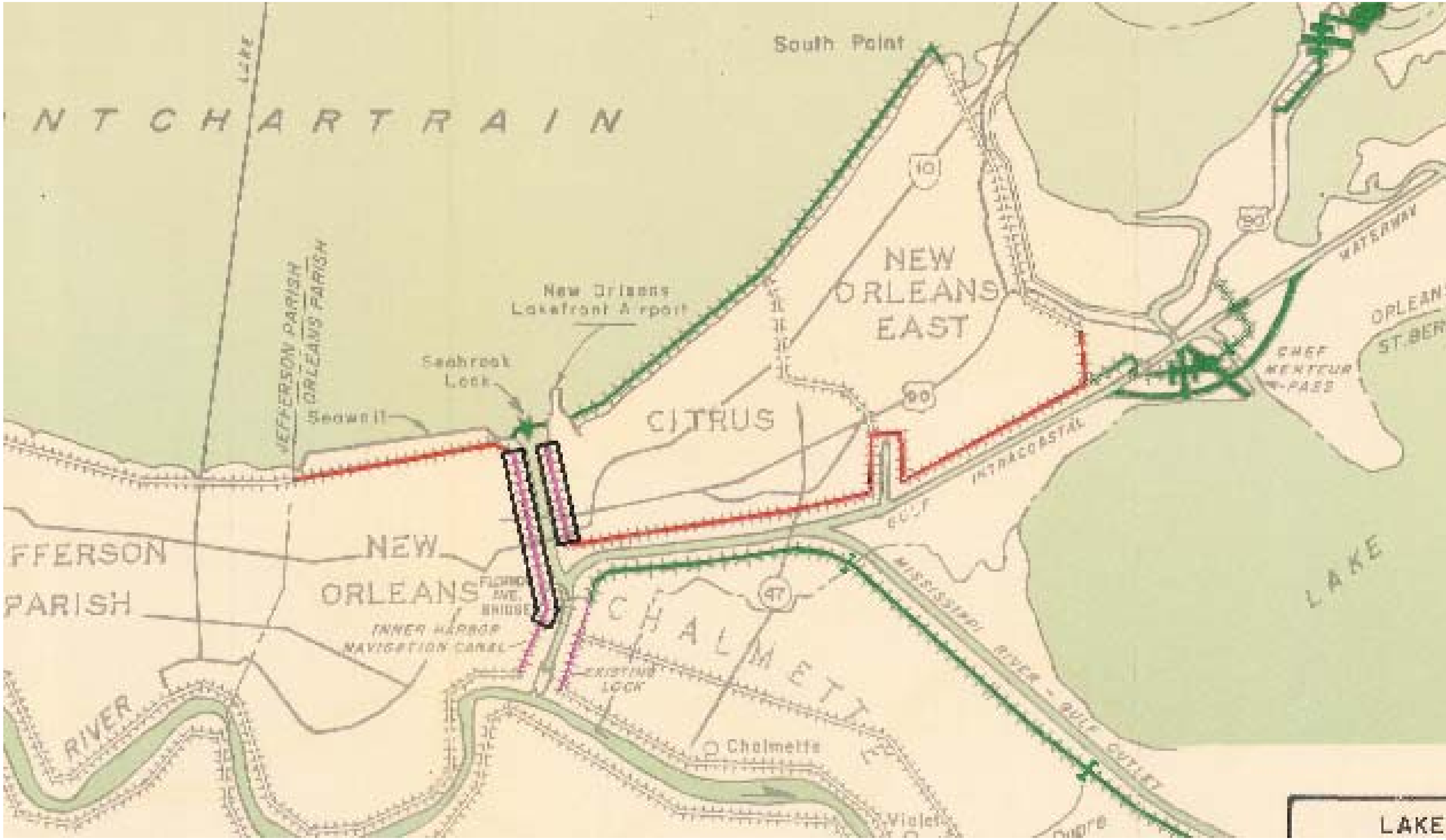
Gates. The loading cases used to design the gates are as follows:

- Case I – Water to elevation 18.8 ft on the floodside (top of broken wave), elevation 10.5 ft on protected side, no dynamic wave load, normal working stresses

- Case II – Water to elevation 18.8 ft on floodside (top of broken wave), elevation 10.5 ft 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.

IHNC (Reference 9). Map of area shown on following page.

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.


NTCHARTRAIN
South Point
NEW ORLEANS EAST
New Orleans Lakefront Airport
Seabrook Lock
Seawall
JEFFERSON PARISH
ORLEANS PARISH
CITRUS
FFERSON PARISH
NEW ORLEANS
FLORIDA AVE. BRIDGE
INNER HARBOR NAVIGATION CANAL
EXISTING LOCK
CHALMETTE
Chalmette
Violet
RIVER
GULF INTRACOASTAL WATERWAY
MISSISSIPPI RIVER - GULF OUTLET
CHEF MENTEUR PASS
ORLEANS ST. BERN
LAKE
LAKE

Basic Data. Maximum wind tide levels along the IHNC resulting from the design hurricane range from elevation 11.4 ft at Seabrook to 12.9 at the L&N Railroad bridge and to 13.0 ft at the IHNC Lock. Water elevations landside of the floodwall range from elevation 0.0 ft to elevation -3.0 ft. The elevation of the top of an I-wall in a levee are 2.0 ft above the wind tide level. The elevation of the top of T-type walls and gates are 1.0 ft above the wind tide level.

Unit Weights	**lb/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 FS 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 ft on floodside and at bottom of base (elevation 2.5 ft) 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 ft 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 ft 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 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.

Sources of Construction Materials.

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	

Levee Material. The descriptions of proposed levee construction materials in the following paragraphs were taken directly from the referenced DMs. Numerous times in the various DMs, 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 was unsuccessful. We were told that the borrow material specifications 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 the material 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.

Source of Borrow Materials (Citrus Lakefront Levee). The levee will be constructed of semi-compacted clay fill which will be obtained from a borrow area of Pleistocene clays in the bottom of Lake Pontchartrain along the north slope.

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.

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.

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 borrow materials 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.

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.

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.

As-built Conditions.

Changes Between Design and Construction (i.e., cross sections, alignment, sheet-pile tip elevation, and levee crest elevation).

DACW29-68-B-0148. Lake Pontchartrain, LA, and Vicinity, Lake Pontchartrain Barrier Plan, Orleans Parish, LA, Inner Harbor Navigation Canal, East Levee, Hayne Boulevard to Dwyer Road (Station 33+95 to Station 83+00) Plans for Levee and Floodwall Capping.

Reviewed as-builts; no applicable modifications or changes were found.

DACW29-83-R-0056. IHNC - East and West Levee and Citrus Back Levee Capping Floodwalls, Paris Road through NOPSI, Orleans Parish, LA.

Reviewed as-builts; no applicable modifications or changes were found.

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 floodside 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 did not do something. This modification was to change the configuration of the floodside 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.

DACW29-98-C-0002. Lake Pontchartrain, LA, 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 were found.

Reviewed narrative completion report and modification documents; no applicable modifications or changes were found.

DACW29-89-C-0134. Lake Pontchartrain High Level Plan, New Orleans East Levee, South Point to GIWW, Orleans Parish, LA.

DACW29-96-C-0080. Lake Pontchartrain, LA, and Vicinity, Hurricane Protection, Orleans Marina Floodwall – Phase IV, Orleans Parish, LA.

Reviewed completion report; no applicable modifications or changes were found.

DACW29-97-C-0066. Lake Pontchartrain, LA, and Vicinity, High Level Plan, Orleans Parish Lakefront Levee/Floodwall, Pontchartrain Beach Wave Berm, Station 10+03.45 to Station 39+78.39, Orleans Parish, LA.

Reviewed narrative completion report. No applicable modifications were found; however, contractor provided and used an alternate borrow pit, which was material from a city-owned (Slidell) detention pond.

Inspection During Original Construction, QA/QC, State What Records Are Available. See pages III-134 through III-135, Orleans East Bank, for description of how records are kept.

DACW29-89-C-0134. South Point to GIWW, Orleans Parish.

Attached to QA/QC reports are moisture test records and records of preparatory inspections/ meetings for identifiable features of work.

DACW29-98-C-0002. Lake Pontchartrain Airport Floodwall Modifications, Orleans Parish.

Attached are records of preparatory meetings, concrete sampling and testing reports, and form checkout sheets for reinforced concrete floodwalls.

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, “Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures.” Bridges are inspected under ER 1110-2-111, “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, “Flood Control Operations and Maintenance Policies,” dated 30 October 1996. (This engineering regulation supersedes the previous regulation ER 1130–2–339, “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.

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 HPP. 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.

Periodic Inspections.

There are no structures under the Periodic Inspection Program in this polder.

Other Features.

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. However, other drainage and flood control features that work in concert with the Corps levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section describes and presents 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.

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 29 August 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.

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 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 NOS & WB. 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 7 and 8 show typical inlets and streets. Photo 9 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 on pages III-203 through III-207 of this volume.

Photos 7 and 8. Typical streets and inlet – New Orleans East.

Photo 9. Storm sewer outfall into Dwyer Canal.

The primary interior canals are open and either concrete-lined or grass-lined (Photos 10, 11, and 12). 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 10. St. Charles Canal from Dwyer Road.

Photo 11. Benson Canal from Dwyer Road.

Photo 12. Dwyer Canal near Crowder Boulevard.

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 in./hr. 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 would not change significantly and delaying runoff to an outfall pump is counter productive.

Where local drainage is considered poor, the NOS & WB is working to improve the drainage. In some cases, the NOS & WB and Corps 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 NOS & WB 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 completed, the pump station was under construction, and the Dwyer Road canal was not started.

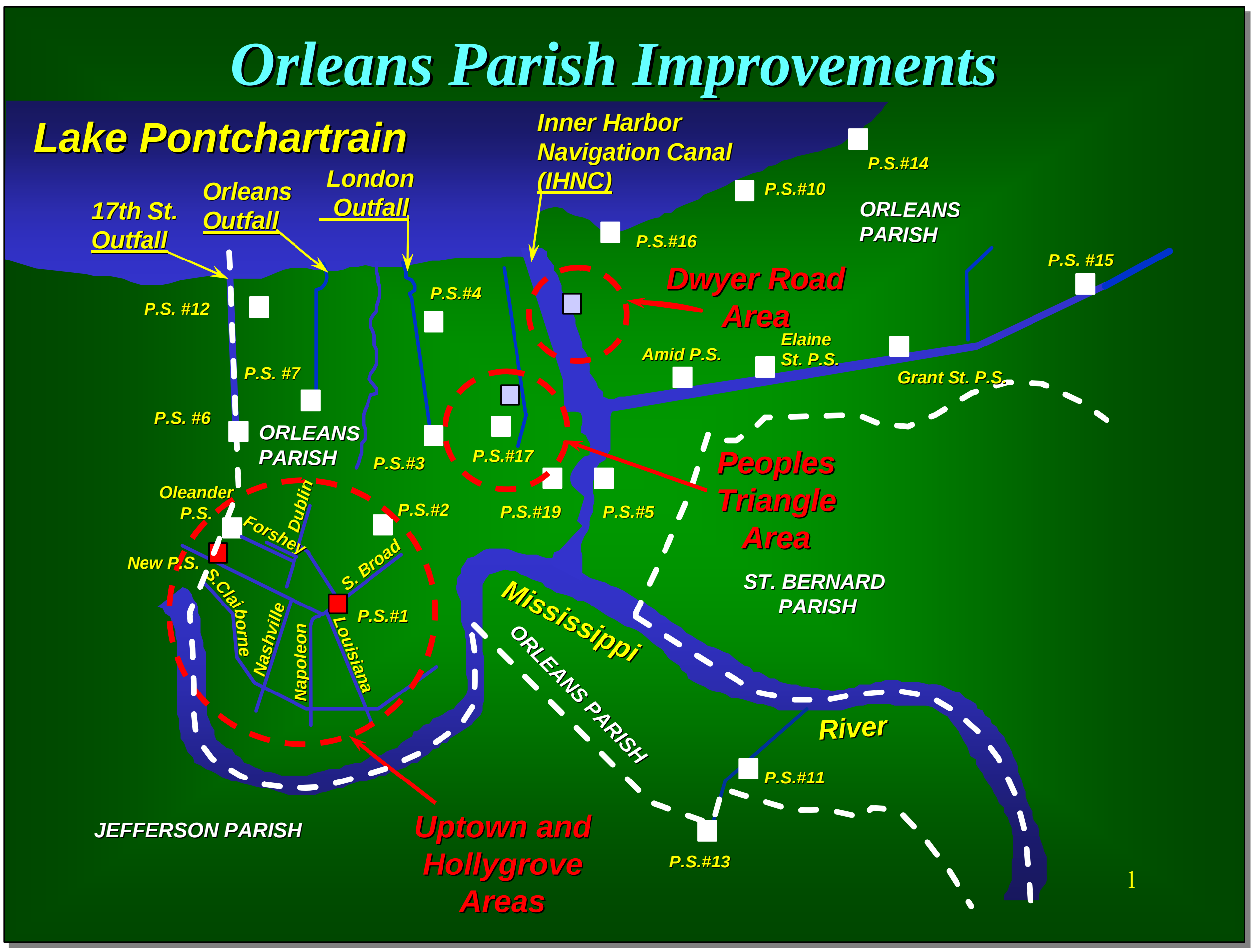


Figure 12. SELA Urban Flood Control Projects in New Orleans East.

Pumping Stations – Orleans Parish Summary.

Figure 13 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).

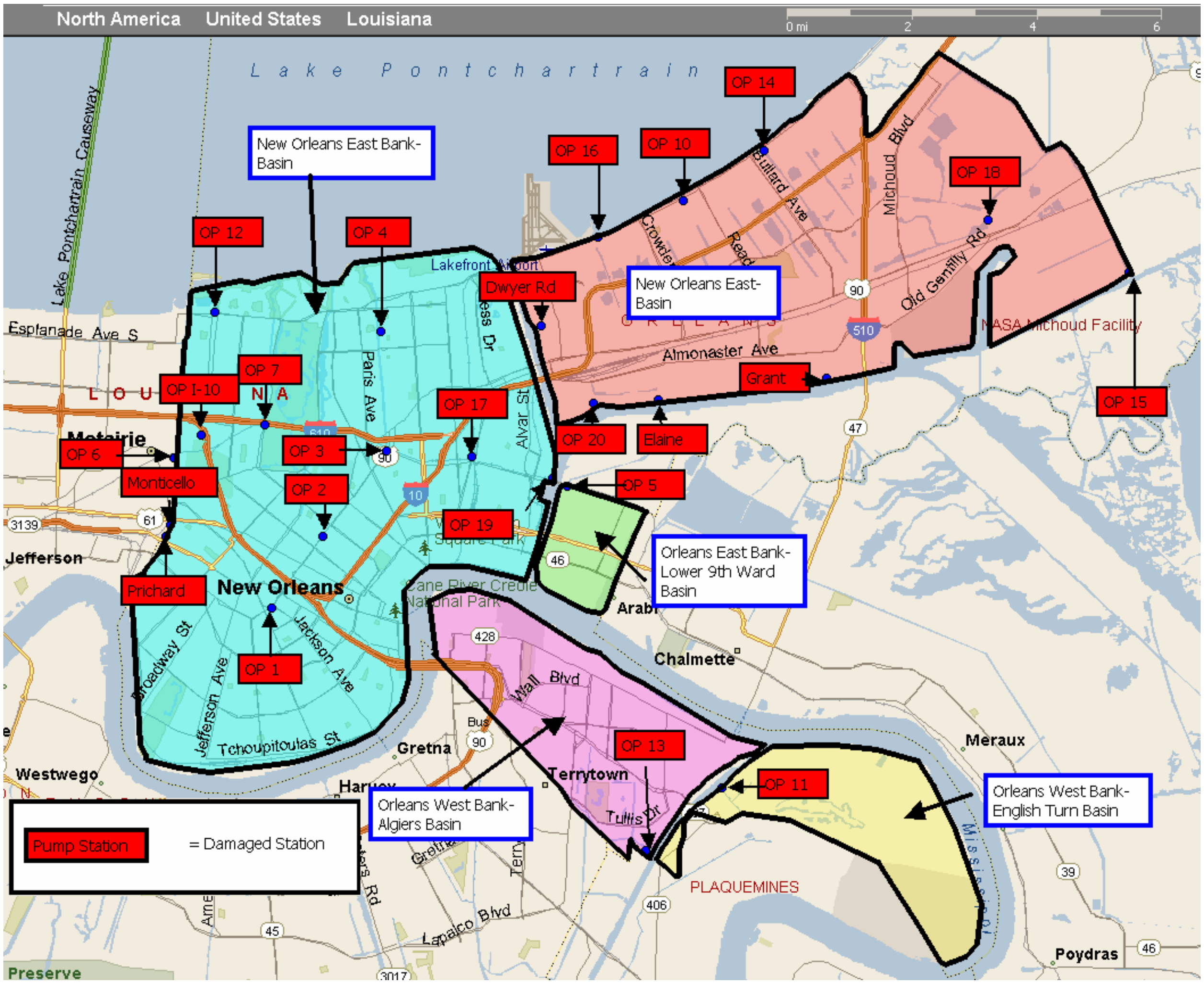


Figure 13. Orleans Parish pump station locations.

Table 11 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; missing information was left blank or highlighted.

Table 11
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 Street, Elaine Street, and Maxent Canals, and the Village de'L East Lagoon. Below is a brief summary of each of the nine pump stations. Volume VI provides more detailed information.

OP 10 – Citrus
Intake location: .... Citrus Canal
Discharge location: .... Lake Pontchartrain
Nominal capacity: .... 1,000 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: .... 1,200 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: .... 1,000 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 Street

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 Street
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: ....0 cfs

Pump	Capacity (cfs)	Installed (year)	Driver Electric /Diesel	Pump Configuration
1	0			Vertical
2	0			Vertical
3	0			Vertical

Levees and Floodwalls.

***MRL*.** There are no MRL levees and floodwalls as a part of the New Orleans East Project.

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 in Figure 14. 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.

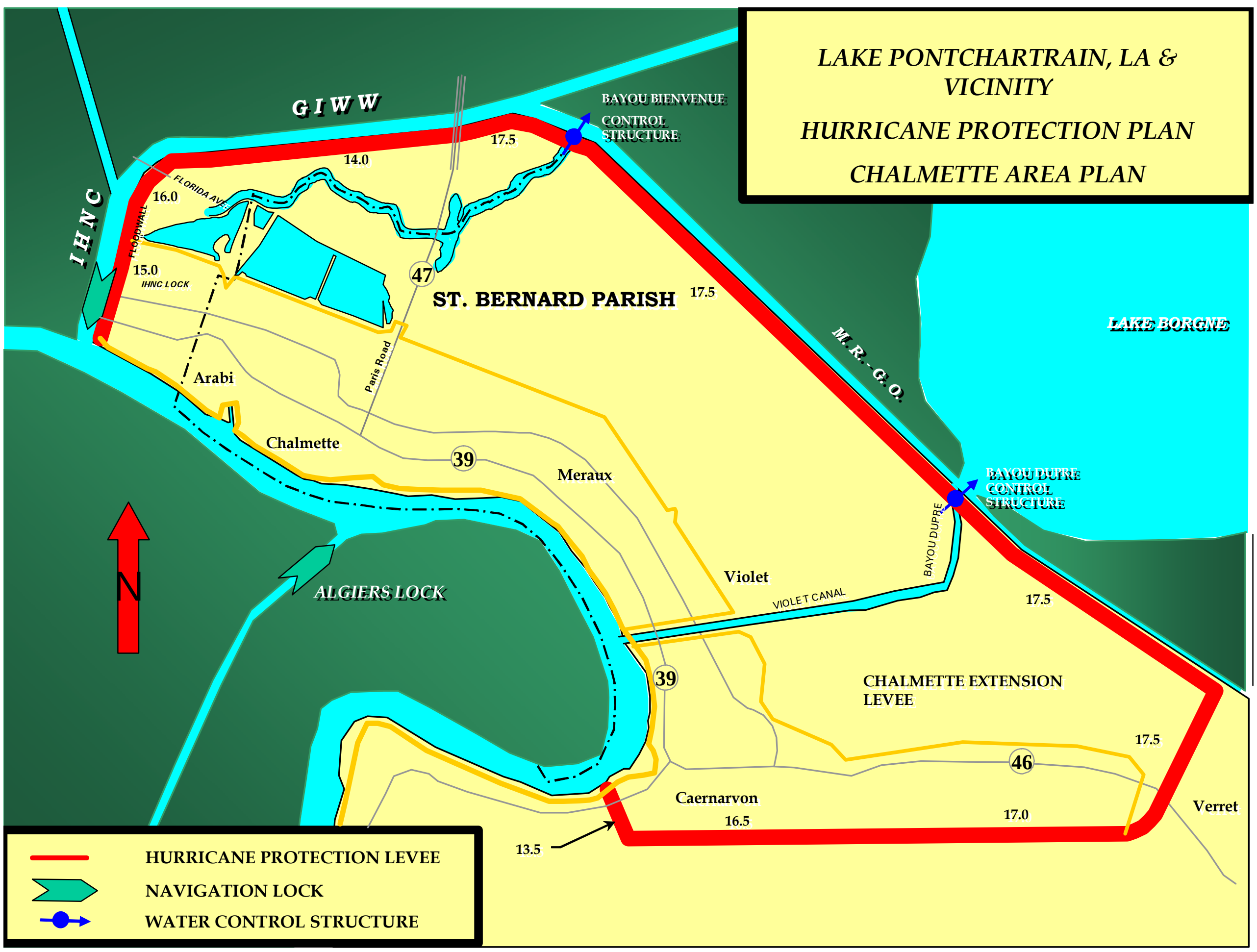


Figure 14. Hurricane protection project, St. Bernard Parish

Table 12 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

Pre-Katrina

The St. Bernard Parish portion of the Lake Pontchartrain and Vicinity Project is under construction. As of 29 August 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 IPET analysis and based on a reanalysis of design storm calculations. Additional contracts may be required as a result of this analysis.

Design Criteria and Assumptions - Functional design criteria.

Hydrology and Hydraulics.

For St. Bernard, the design hurricane characteristics utilized in the design memoranda are shown in Table 13; the design tracks are shown on Figure 15. 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 13
Design Hurricane Characteristics

Location	Track	CPI, in.	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] Wind speeds represent a 5-minute average 30 ft above ground level.

***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 16). This relationship was considered independent of hurricane forward speed, wind speed, 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 14 shows the wind tide levels at the surge reference line and at the levee location.

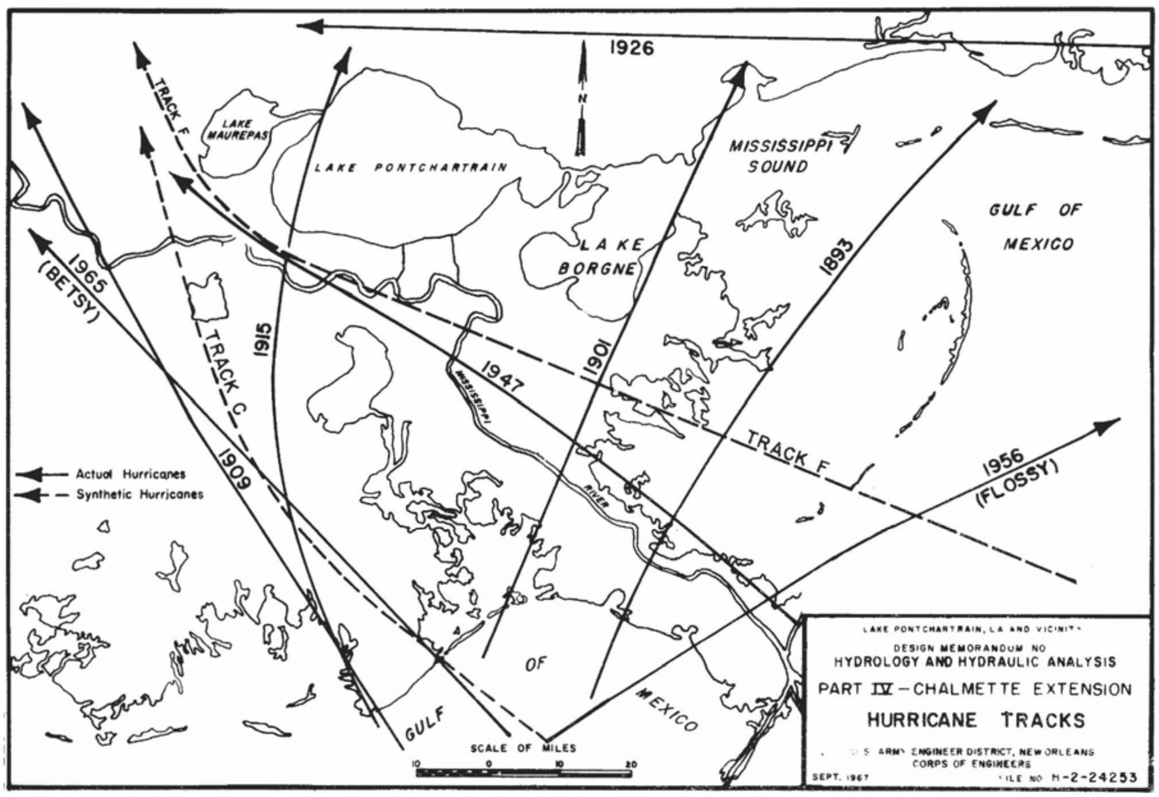


Figure 15. St. Bernard hurricane tracks.

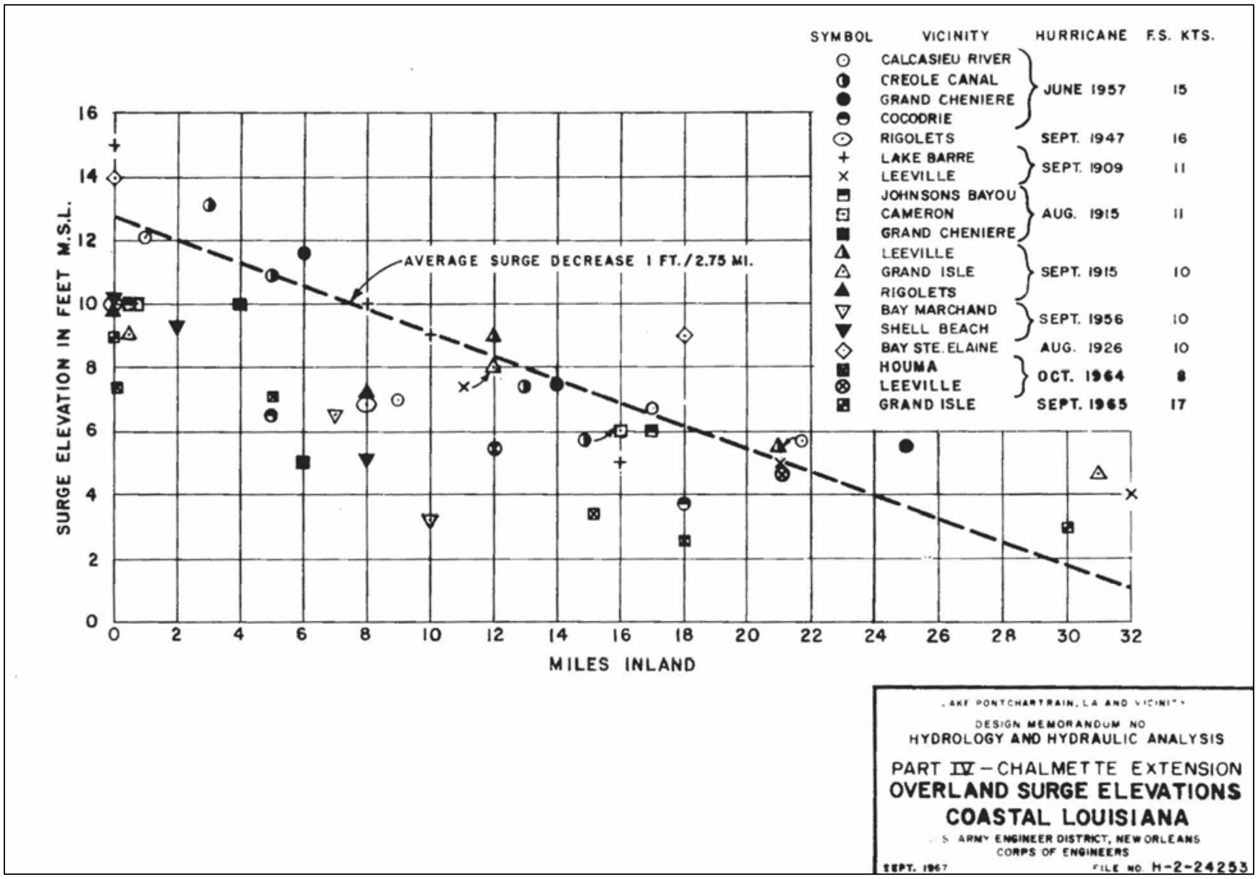


Figure 16. Overland surge elevations.

Table 14 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

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.

Summary. Table 15 contains maximum surge or wind tide level, wave, and design elevation information.

Table 15
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, August 1966	-	-	-	12.9 – 13.0 MSL	-	1.0	14.0 MSL
Chalmette West of Paris Road	DM1, Part 1, August 1966	-	-	-	13.0 MSL	-	1.0	14.0 MSL
Chalmette East of Paris Road to Bayou Lawler	DM1, Part 1, August 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, August 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, August 1966	16.3	6.6	6.2	12.5 MSL	4.6	-	17.5 MSL
Chalmette Extension Verret to Toca	DM1, Part 4, August 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, August 1966	9.7	4.5	5.1	11.8 MSL	4.4	-	16.5 MSL

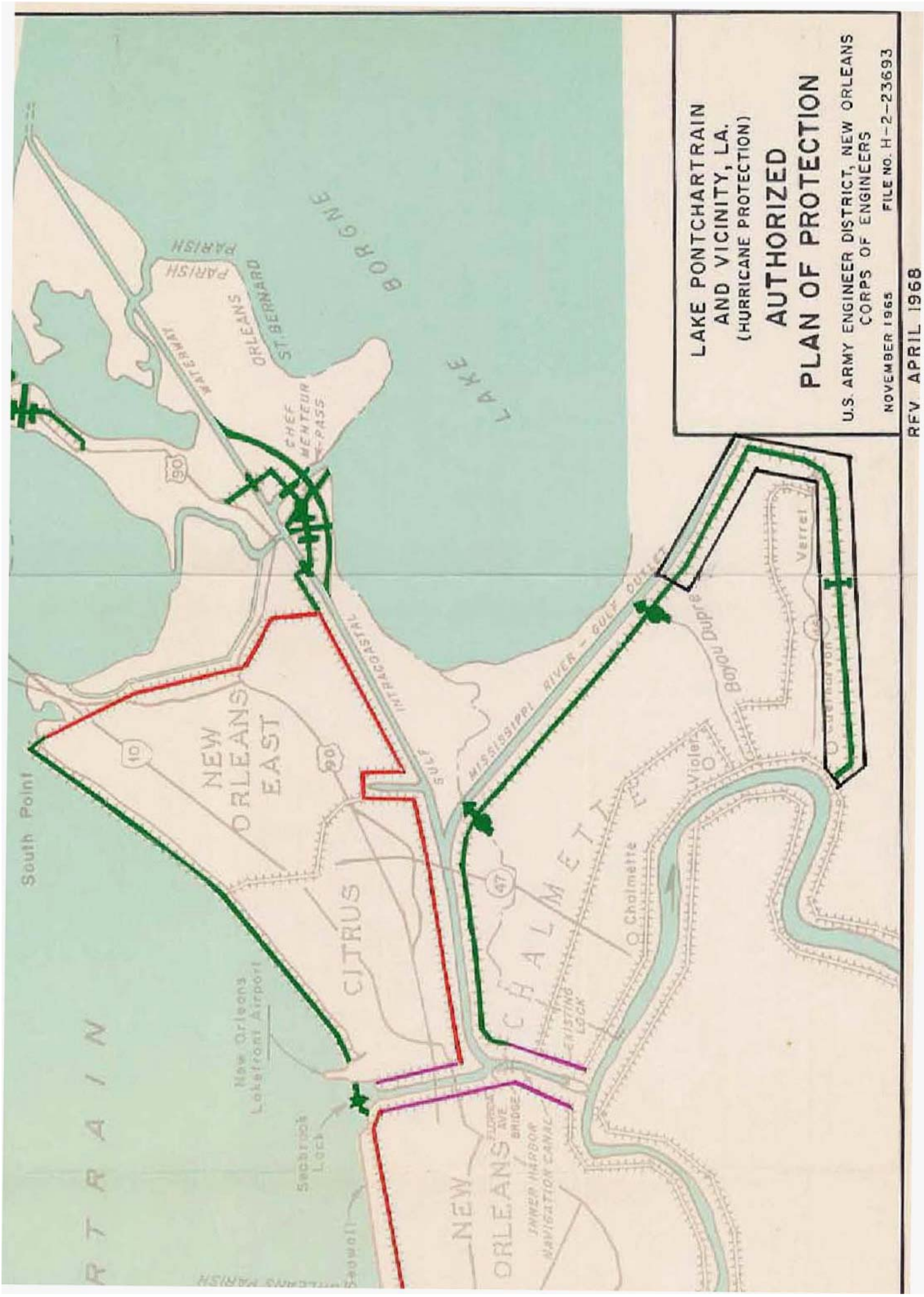
Geotechnical.

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 Dupre 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 second, third, 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, computations, and sketches. The results were transmitted from Chief of Foundations and Materials Branch to Chief of Design Branch in two memoranda dated 22 April 1991 and 4 February 1991. Reference 64, dated 2001, covers the Bayou Bienvenue to Bayou Dupre 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 elevation 20.0 ft. The SPH elevation for the levee between Bienvenue and Bayou Dupre was elevation 17.5 ft. 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 stem Mississippi River Levee.

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 ft 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 ft are found in the area near Station 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 elevation from 4 to 10 ft and slopes away from the river at about 1 ft per 1,000 ft. 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 marshland, was only 1 or 2 ft above sea level and was subject to tidal flooding from the MRGO through connecting canals and bayous. The balance of the Chalmette Area along the MRGO and bounded by Bayou Bienvenue and a line about 4,000 ft landward of the MRGO right-of-way had been filled hydraulically to elevations 4 ft to 10 feet with material excavated from the MRGO channel.

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 ft at the northeastern end of this portion of the project to about 75 ft 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-ft stratum of very soft organic clays underlying 7 to 20 ft of fill material except between the IHNC Lock and Station 30+00 where a thin layer of natural levee material 5 to 10 ft thick underlies the fill material. Underlying the marsh deposits is a 20- to 35-ft 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-ft 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 ft thick exists within the estuarine deposits. Along the Outfall Canal from Station 74+85 to Station 90+25, the estuarine deposits grade into nearshore gulf sands containing sand shell fragments. The estuarine and nearshore 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 elevation -55 to -65 ft consist generally of organic clays, peat, fat clays, some lean clays, some clayey sands, and rare amounts of sand. Along the project alignment paralleling the MRGO, the natural Recent soils have been covered with hydraulic spoil from the excavation of the MRGO channel.

Preliminary Field Exploration (Reference 42). Map of area shown on following page. From Station 1+82 to Station 0+00 = 1+46.6 to Station 90+25 = 7+52.9, five 5-in.-diameter undisturbed soil borings and sixteen 1⅞-in.-ID core barrel general type borings were made at intervals ranging from about 200 to 1,000 ft 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 and -88 ft. From Station 90+25 = 7+52.9 to the project end at Station 1050+57.7, 103 3-in.-ID core barrel general type soil test borings extending to a depth of 60 ft below existing ground surface were made at l,000 ft intervals along the proposed levee location.


LAKE PONTCHARTRAIN
AND VICINITY, LA.
(HURRICANE PROTECTION)
AUTHORIZED
PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NOVEMBER 1965
FILE NO. H-2-23693
REV. APRIL 1968
LAKE BORGNE
NEW ORLEANS EAST
CITRUS
CHALMETTE
NEW ORLEANS
MISSISSIPPI RIVER - GULF OUTLET
INTRACOASTAL
WATERWAY
CHEF MENTEUR PASS
ORLEANS PARISH
ST. BERNARD PARISH
South Point
New Orleans Lakefront Airport
Seabrook Lock
Seawall
FLORIDA AVE. BRIDGE
INNER HARBOR NAVIGATION CANAL
EXISTING LOCK
Chalmette
Violet
Verret
Bayou Dupre

In addition to these 60-ft borings, two 5-in.-diameter undisturbed type soil test borings 100 ft deep were made along the levee alignment; one-in the section adjacent to the MRGO and one along the Bayou Dupre-Violet alignment. Two additional 5-in.-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.

Field Exploration Bayou Bienvenue to Bayou Dupre (Reference 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 ft 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 to 70 ft and were tested by the Corps.

Field Exploration Chalmette Extension (Reference 67). Additional undisturbed borings were taken and tested by the Corps along the centerline and 150 ft landside from centerline. Five borings were mentioned in Reference 67. No mention was made of other borings, or borings that may have been made during previous investigations.

Field Investigation Verret to Caernarvon Levee (Reference 68). A total of 30 undisturbed soil borings were taken along the levee alignment between 1967 and 2000 for various purposes. Eight undisturbed soil borings were drilled for this project in April and May 2000. The soil borings were taken by the Corps, New Orleans District. The centerline borings and the toe borings were taken to depths of 70 and 55 ft 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, New Orleans District.

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.

Pile Foundation. Is addressed in the Bayou Bienvenue and Bayou Dupre paragraphs**.**

Slope Stability (References 42, 64, 67, and 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 FS of 1.3 with respect to shear strength. The minimum FS 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 floodside. The project was divided into seven design reaches.

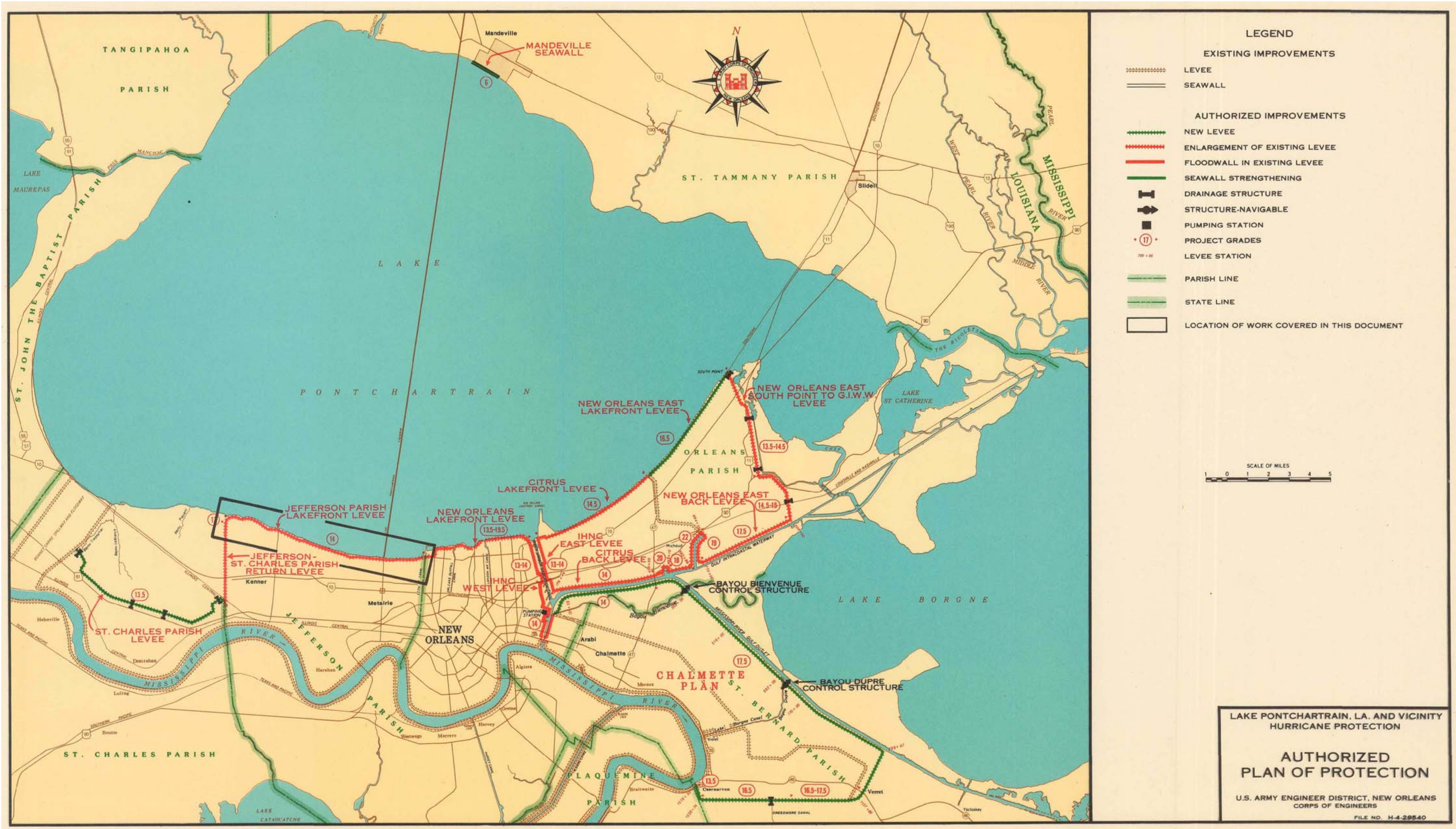
Preliminary stability analyses were conducted to compare stabilities of various trial levee sections and to consider the feasibility of mucking out and backfilling for the levee base. For the preliminary analyses, shear strengths from unconfined compression tests on samples from the general type borings 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 MRGO had been placed over a 5- to 7-year period to an elevation ranging from 5.0 to 12.0 ft. This area was from 2,000 to 4,000 ft 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 ft by the surcharge of the spoil. Furthermore, a statistical analysis of over 30 general borings made along the levee centerline at 3,000-ft intervals indicated average subsurface strata strength 15 percent higher. In areas with overburden as compared to areas without overburden and, in the top 30 ft 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 elevation 10.0 or 11.0 ft on slopes steeper than 1:3.

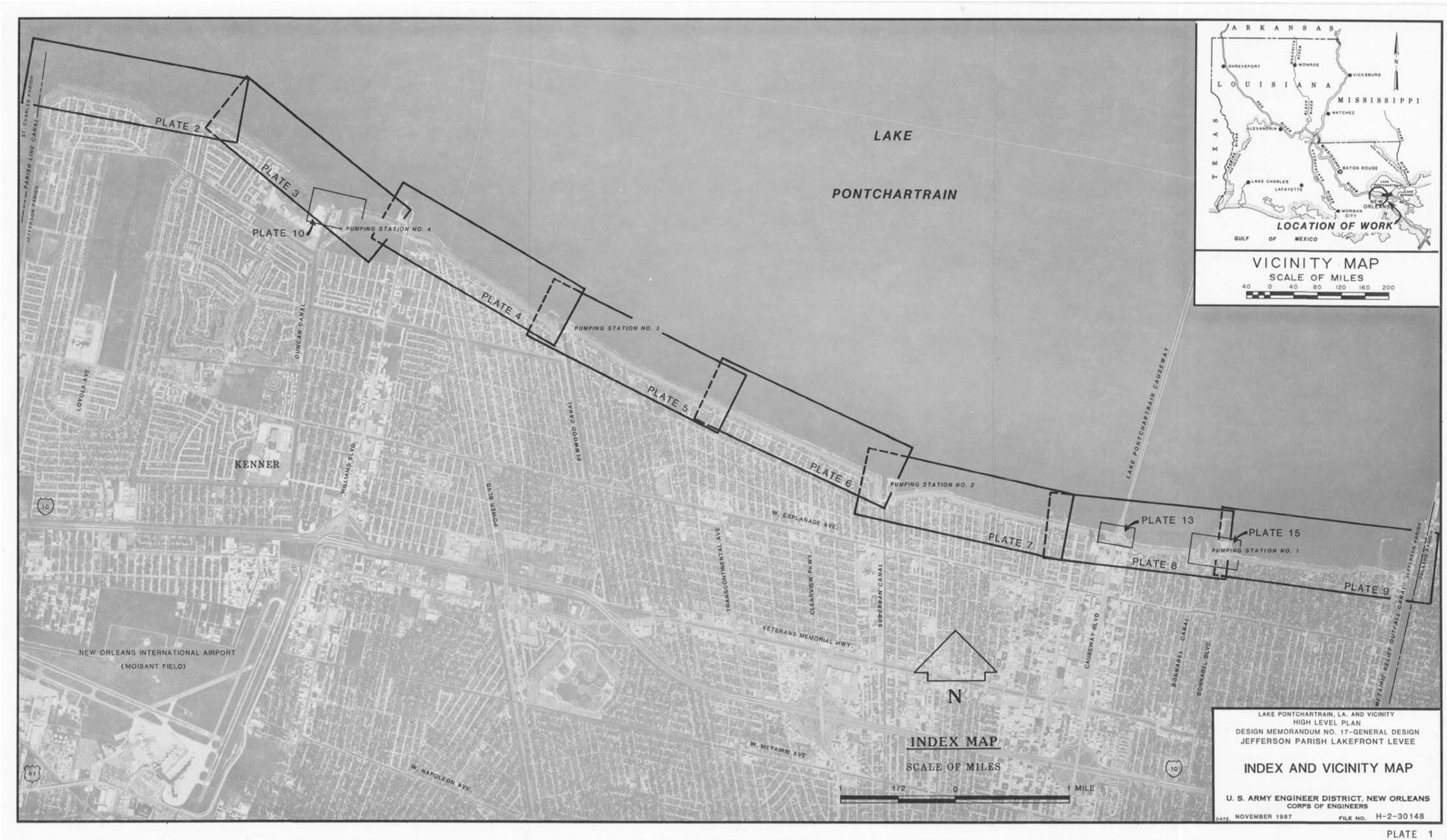
Cantilever I-Type Floodwall (Reference 23, 45, 48, and 105). Images of map and plan view shown on following pages.

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 elevation 123.5 ft NGVD. The required penetration for the stability of the walls were determined by the Method of Planes 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 were applied:

Case	Factor of Safety	Criteria
Q	1.5	Water at swl
Q	1.25	Water at swl and wave load
S	1.2	Water at swl and wave load

The FSs 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.


LEGEND
EXISTING IMPROVEMENTS
LEVEE
SEAWALL
AUTHORIZED IMPROVEMENTS
NEW LEVEE
ENLARGEMENT OF EXISTING LEVEE
FLOODWALL IN EXISTING LEVEE
SEAWALL STRENGTHENING
DRAINAGE STRUCTURE
STRUCTURE-NAVIGABLE
PUMPING STATION
PROJECT GRADES
LEVEE STATION
PARISH LINE
STATE LINE
LOCATION OF WORK COVERED IN THIS DOCUMENT
SCALE OF MILES
LAKE PONTCHARTRAIN, LA. AND VICINITY
HURRICANE PROTECTION
AUTHORIZED
PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FILE NO. H-4-29540
TANGIPAHOA PARISH
ST. JOHN THE BAPTIST PARISH
ST. TAMMANY PARISH
ORLEANS PARISH
JEFFERSON PARISH
ST. CHARLES PARISH
ST. BERNARD PARISH
PLAQUEMINE PARISH
LAKE PONTCHARTRAIN
LAKE MAUREPAS
LAKE BORGNE
LAKE ST CATHERINE
LAKE CATAOUATCHE
MISSISSIPPI RIVER
LOUISIANA
MISSISSIPPI
PEARL RIVER
Mandeville
Slidell
Kenner
Metairie
NEW ORLEANS
Arabi
Chalmette
MANDEVILLE SEAWALL
NEW ORLEANS EAST LAKEFRONT LEVEE
NEW ORLEANS EAST SOUTH POINT TO G.I.W.W. LEVEE
CITRUS LAKEFRONT LEVEE
NEW ORLEANS EAST BACK LEVEE
JEFFERSON PARISH LAKEFRONT LEVEE
NEW ORLEANS LAKEFRONT LEVEE
JEFFERSON-ST. CHARLES PARISH RETURN LEVEE
IHNC EAST LEVEE
CITRUS BACK LEVEE
IHNC WEST LEVEE
ST. CHARLES PARISH LEVEE
BAYOU BIENVENUE CONTROL STRUCTURE
BAYOU DUPRE CONTROL STRUCTURE
CHALMETTE PLAN
GULF INTRACOASTAL WATERWAY



LAKE
PONTCHARTRAIN
PLATE 2
PLATE 3
PLATE 4
PLATE 5
PLATE 6
PLATE 7
PLATE 8
PLATE 9
PLATE 10
PLATE 13
PLATE 15
PUMPING STATION NO. 4
PUMPING STATION NO. 3
PUMPING STATION NO. 2
PUMPING STATION NO. 1
ST. CHARLES PARISH
JEFFERSON PARISH
PARISH LINE CANAL
DUNCAN CANAL
LOYOLA AVE.
WILLIAMS BLVD.
POWER BLVD.
ELMWOOD CANAL
TRANSCONTINENTAL AVE.
CLEARVIEW PKWY.
SUBURBAN CANAL
CAUSEWAY BLVD.
BONNABEL CANAL
BONNABEL BLVD.
METAIRIE RELIEF OUTFALL CANAL
ORLEANS PARISH
LAKE PONTCHARTRAIN CAUSEWAY
W. ESPLANADE AVE.
VETERANS MEMORIAL HWY.
W. METAIRIE AVE.
W. NAPOLEON AVE.
KENNER
NEW ORLEANS INTERNATIONAL AIRPORT
(MOISANT FIELD)
10
61
N
INDEX MAP
SCALE OF MILES
1 1/2 0 1 MILE
ARKANSAS
LOUISIANA
MISSISSIPPI
TEXAS
SHREVEPORT
MONROE
VICKSBURG
NATCHEZ
ALEXANDRIA
BATON ROUGE
LAKE CHARLES
LAFAYETTE
NEW ORLEANS
MORGAN CITY
GULF OF MEXICO
LOCATION OF WORK
VICINITY MAP
SCALE OF MILES
40 0 40 80 120 160 200
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 17-GENERAL DESIGN
JEFFERSON PARISH LAKEFRONT LEVEE
INDEX AND VICINITY MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
DATE NOVEMBER 1987
FILE NO. H-2-30148

PLATE 1

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). Map of area shown on following page.

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.

Review Comments. First Endorsement comments paragraph 3. Questioned using borrow material below elevation 0.0 ft because material between 0.0 ft 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 FS 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.

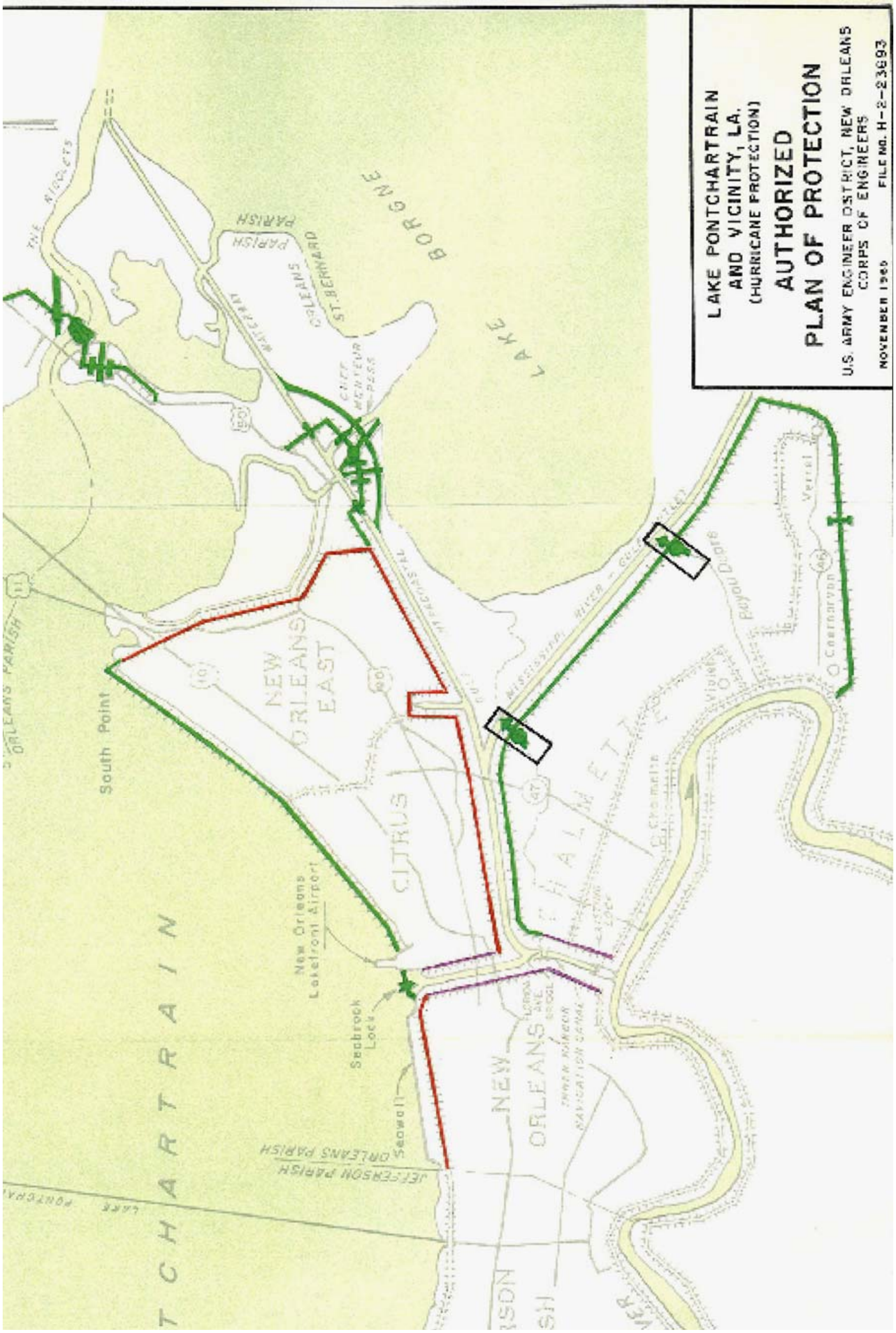
Bayou Bienvenue and Bayou Dupre Control Structures (Reference 43).

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.

Geology. The general geology within the Chalmette Area was presented on page III-212.

Project Foundation Conditions.

a. Bayou Dupre Control Structure. The upper stratum from existing ground at elevation 8 to -3 ft consists of very soft gray clays (CH) with roots, peat, and organic matter and had water contents ranging from 43 to 85 percent. From elevation -3 to -23 ft, 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 elevation -23 to -26 ft and from elevation -31 to -58 ft, 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 ranged from 46 to 65 percent. Between elevation -26 and -31 ft a stratum of sand was encountered. From elevation -58 to -64 ft, a medium dense fine gray sand stratum occurs over the Pleistocene Formation at elevation -64 ft. Except for a soft dark gray clay stratum occurring between elevation -65.5 and -67.5 ft having an average water content of 30 percent, the remainder of the soils down to elevation -91 ft consisted of layers of medium stiff to stiff clays (CL and CH) with water contents ranging from 23 to 45 percent.


LAKE PONTCHARTRAIN
AND VICINITY, LA.
(HURRICANE PROTECTION)
AUTHORIZED
PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NOVEMBER 1966
FILE NO. H-2-23693
LAKE BORGNE
NEW ORLEANS EAST
CITRUS
NEW ORLEANS
CHALMETTE
South Point
New Orleans Lakefront Airport
Seabrook Lock
Seawall
ORLEANS PARISH
JEFFERSON PARISH
ST. BERNARD PARISH
Bayou Dupre
Verret
Caernarvon
Chalmette
INNER HARBOR NAVIGATION CANAL

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 elevation 5.5 ft to about -8 ft, 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 elevation -8 to -28 ft, the stratum consisted of soft to very soft gray clay with silt pockets, sandy silt pockets and shell fragments with water contents ranging from 50 and 80 percent. From elevation -28 to -35 ft, a stratum of gray sand was encountered ranging from loose to dense. From elevation -35 to -63 ft, there was encountered soft to stiff gray clays with silt pockets and occasional small shell fragments. From elevation -63 ft, which was the top of the Pleistocene, to elevation -78 ft, the limit of the boring, the soil consisted of soft to medium gray clays and green clays with silt and sand lenses.

Field Exploration. Four 3-in.-ID general type borings were made by the Corps at each structure location. One 5-in.-ID undisturbed boring designated as B-IU was made by the Corps at Bayou Bienvenue. At Bayou Dupre, one 5-in.-ID undisturbed boring was made by the A-E during the preparation of Reference 41 and was used in Reference 43. An additional undisturbed boring designated as D-IU (5-in. ID) was made by the Corps at Bayou Dupre.

Seepage, Dewatering, and Pressure Relief.

a. Both sites for the structures had received hydraulic spoil from the excavation of the MRGO. 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 elevation -26 and -31 ft at Bayou Dupre and between elevation -28 and -35 ft 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 elevation -5 ft there would be a potential heave of the excavation bottom if the tide level in the MRGO exceeded elevation 2 ft. An outer ring of wellpoints installed in the upper berm, elevation 2 ft at Bayou Dupre and elevation 3 ft at Bayou Bienvenue, was designed to relieve the hydrostatic pressure in the underlying sands. With the tide in the MRGO at elevation 8 ft and the outer well-point system operating, the piezometric pressure reading in the sands would be at elevation -5 ft at the center of the excavation. At this point there would be a FS of 1.3 against bottom heave when dewatered to elevation -5 ft.

Division comments directed that the wellpoint 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-ft-long screens.

(a) With the excavation continuing and the berm at elevation -5 ft exposed, the inner ring of wellpoints would be installed and would completely encircle the excavation. The Bayou Dupre location would have wellpoint tips at elevation -31 ft and the Bayou Bienvenue location would have wellpoint tips at elevation -31 ft. With a tide level of elevation 5 ft in the MRGO, the piezometric pressure reading in the underlying sands would be at elevation -21 ft in the center of the excavation with the inner ring of wellpoints 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 elevation -10 and -23 ft. Pressures in this stratum would be relieved by encasing the wellpoints with sand from elevation -10 ft to the sand stratum that water was being withdrawn from. Above elevation -10 ft to the berm at elevation -5 ft, the wellpoints would be encased in clay.

(b) Based on the pressure relief analysis it was estimated that the inner ring could pump approximately 500 gpm at Bayou Dupre and 265 gpm 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 elevation -5 ft, 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 ft at Bayou Bienvenue and -60 ft 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-in.-diameter sand drains be placed every 20 ft around the periphery, inside the sheet-pile enclosure. The sand drains and piezometers were to be installed when the excavation was at elevation -5 ft after the sheet-pile wall was installed. The drains would extend through the underlying sand to approximately elevation -40 ft at Bayou Dupre and elevation -37 ft 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 wellpoints.

c. During an unwatered condition it was assumed that the water on the MRGO side would be at elevation 5.0 ft and the water on the landside would be at elevation 2.0 ft. Under these conditions and with the structure completely dewatered, a FS of 1.16 against uplift was computed that disregarded the hold down straps on the piles. Assuming the cutoff wall impervious and the same water heights as above, a FS of 1.07 against uplift was computed that disregarded the hold down straps on the piles. Therefore, no pressure relief was considered to be required.

d. During the normal operating condition, with the gates open, no pressure relief was computed to be required.

e. A steel sheet-pile cutoff wall was to be driven below the base slab and inverted T-type wall. This cutoff 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.

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 elevation 8 ft 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.

Slope Stability.

a. Construction slopes and permanent slopes for both structure locations were analyzed by the Method of Planes for stability with a minimum FS 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 elevation 6 ft and sloping to drain towards the channel. Water in the approach channel was assumed as elevation 0.0 ft 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 elevation 0.0 ft. 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 elevation -16 ft. Water behind the temporary protection levee at Elev. 2 feet and the water in the excavation at elevation -5 ft with outer ring of wellpoints operating.

Condition No. 2: This condition would normally be experienced during construction. Completed excavation to elevation -19.28 ft. Water behind the temporary protection levee at elevation 2 ft and in the excavation at elevation -19.28 ft, with the wellpoint system operating.

Condition No. 3: This was a storm condition with water behind the temporary protection levee at elevation 8 ft and water at elevation -19.28 ft with the wellpoint 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 wellpoint 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.

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 cutoff wall. A FS 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 floodwall 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 ft and a 5-ft broken wave on the floodside and ground water at elevation 2 ft on the protected side. The wall was investigated for both Q and S design shear strengths for a FS of 1.5 with static-water level at the top of the wave and a FS 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 elevation 0.0 ft on the channel side and behind the wall. A FS of 1.5 was used in both analyses.

e. The Division directed that the stability of the I- and T-type floodwalls and wing walls be analyzed for conditions of maximum reverse differential head (MRGO side elevation 0.00 ft, landside elevation 5.0 ft) using Q and S design shear strengths and a FS of 1.5.

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 FS of 1.75 was used for compression piles and a FS 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 cutoff walls were provided beneath the gate bay structure and beneath the inverted T-type floodwall with tips at elevation -26 ft. The cutoff 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.

Erosion Control and Protection. Erosion protection for the access channel bottom adjacent to the control structure on the floodside was to consist of 3 ft of derrick stone on 1 ft of shell extending 100 ft from the gate bay and 2 ft of riprap on 1 ft of shell extending an additional 100 ft. The erosion protection on the protected side of the structure was to consist of 2 ft of riprap on 1 ft of shell extending 150 ft from the gate structure. The channel side slope within the above limits was to have 2 ft of riprap on 1 ft of shell to elevation 5 ft. 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 ft/sec 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 MRGO 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 from 5 to 6 ft and the structures and levees are located approximately 500 ft from the edge of the MRGO. 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 MRGO would eliminate the above problem.

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 were 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 tip elevations -65, -70, and -75 ft. At Bayou Dupre, the test piles would be driven to the following tip elevations -60, -65, and -70 ft. The results of the load tests would be evaluated to confirm tip elevations of the 12-in. square 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 rate of settlement. These data were 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 pouring 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 2 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.

Structural.

Chalmette Area Plan (Reference 41). As constructed, the Chalmette Area Plan consists 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/cu ft*
Water	62.5
Concrete	150.0

- **Design Loads**
  - Earth pressure (lateral)

- Water loads
  - No wave force
  - Surge to within 6 in. of the top of the wall

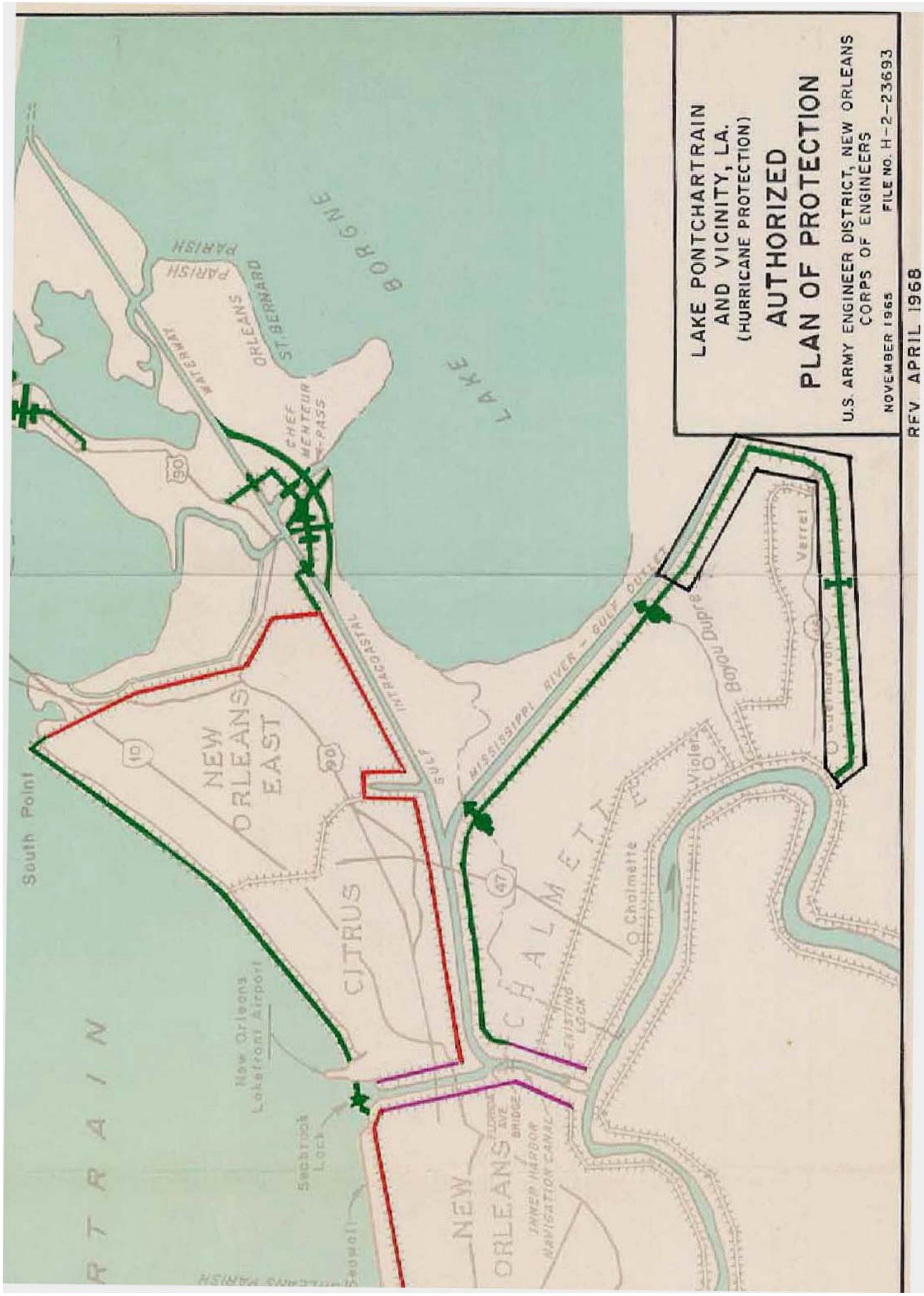
- **Allowable Working Stresses.** The allowable working stresses for concrete and structural steel are in accordance with those recommended in "Working Stresses for Structural Design," EM-1110-1-2101, dated 6 January 1958, revised August 1963. Concrete will be designated by basic minimum strength 3,000 psi 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 FS of 1.0 applied to the soils, since the structural steel has an inherent FS of about 2.0.
    - In the 3d Ind of 23 February 1967, LMVD stated that the pile section should be selected on the basis produced for a loading with a FS 1.5 with low tailwater and a FS of 1.3 with high tailwater due to saturation from rainfall.
    - In the 4th Ind of 10 March 1967, New Orleans District stated that DIVR 1110-1-400, dated November 1966, which specifies the use of a soil shear strength FS of 1.0 in evaluating deflections and stresses is being revised, at the direction of the Office, Chief of Engineers (OCE), to require the use of a FS of 1.3.
    - In the 5th Ind of 22 March 1967, LMVD stated that DIVR 1110-1-400 was revised in March 1967 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.

Chalmette Area Plan. Chalmette Extension (Reference 42)**.** Map of area shown on following page.


LAKE PONTCHARTRAIN
AND VICINITY, LA.
(HURRICANE PROTECTION)
AUTHORIZED
PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NOVEMBER 1965
FILE NO. H-2-23693
REV. APRIL 1968
LAKE BORGNE
NEW ORLEANS EAST
CITRUS
CHALMETTE
NEW ORLEANS
South Point
New Orleans Lakefront Airport
Seabrook Lock
CHEF MENTEUR PASS
ORLEANS PARISH
ST. BERNARD PARISH
INTRACOASTAL WATERWAY
MISSISSIPPI RIVER - GULF OUTLET
INNER HARBOR NAVIGATION CANAL
EXISTING LOCK
Chalmette
Violet
Verret
Bayou Dupre
Caernarvon

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, Highway 39 in DM) at Caernarvon; and a soil-founded drainage structure at Creedmore.

- **Structural Design**
- **Design Criteria**
    - Unit Weights

Item	*lb/cu ft*
Water	62.5
Concrete	150.0

- **Design Loads**
    - Earth pressure (lateral)
    - Water loads:
        - Design still water elevation as follows:
          Verret Gap Closure = 12.2 ft NGVD
          Caernarvon Gap Closure = 11.8 ft NGVD
    - 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	
f'_c	3,000 psi
f_c	1,050 psi
v (without web reinforcement)	60 psi
v (with web reinforcement)	274 psi
fs	20,000 psi
Minimum tensile steel	0.0025 bd sq in.
Shrinkage and temperature steel	0.0020 bt
Structural Steel (ASTM – A36)	
Basic stress	18,000 psi

The allowable stresses are increased 33-1/3 percent 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 elevation 12.2 ft
    5-ft broken wave
    Ground water at elevation 1.0 ft protected side
  - Caernarvon
    swl at elevation 11.8 ft
    5-ft broken wave
    Ground water at elevation 2.0 ft protected side

Drainage structure – Two 72-in.-diameter corrugated metal pipe culverts

Chalmette Area Plan - (Reference 43). Bayou Bienvenue and Bayou 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 ft. Both structures have timber guide walls and dolphins.

Structural Design Criteria

Basic Data

	Elevation (ft NGVD)
Top of gate walls	13.0
Sill	-10.0
Width of gate channel	0.0
Maximum water surface MRGO side	2.0
Maximum differential head MRGO side	13.0
Landside	2.0
Maximum differential head MRGO side (Reverse)	0.0
Landside	5.0

Unit Weights

Unit Weight	*lb/cu ft*
Water	62.5
Concrete	150
Shell Backfill	98

Lateral Pressure

Shell Backfill (Ǿ = 40º)	Equivalent fluid pressure
Select Backfill	Saturated unit weight = 110 lb/cu ft
	Cohesion = 120, Ǿ = 0
Active (above water)	21.3 lb
Active (submerged)	8 lb
At rest (above water)	54 lb
At rest (submerged)	20 lb

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 (f_c):	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 percent.	
Minimum tensile steel	0.0025 bd
Shrinkage and temperature steel area	0.0020 bt

Structural Steel (ASTM A-36)

Basic working stress	18,000 psi

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 elevation 0.0 ft, buoyancy active
Case 3 Needle dams in place, structure dewatered, gates removed, water at elevation 5.0 ft, buoyancy active
Case 4 Hurricane condition, gate closed, water in MRGO at elevation 13.0, water on landside at elevation 2.0 ft, buoyancy active
Case 5 Gate closed, water in MRGO at elevation 13.0, 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 MRGO at elevation 13.0, water on landside at elevation 2.0 ft
Case 3 Dead load, water in MRGO at elevation 0.0, water on landside at elevation 5.0 ft
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 elevation 13.0 ft in MRGO and a wave loading on MRGO side and water on land side at elevation 2.0 ft.

Sources of Construction Materials.

***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, listed by DM.

St. Bernard	DM
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	

Levee Material.

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 elevation 0.0 ft. 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.

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 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 in.-thick and the succeeding layers not more than 12 in. 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 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 ft in thickness. Moisture content control of uncompacted fill was not required.

Structure Backfill. The excavation adjacent to the gate bay walls and sheet-pile wing walls was to be backfilled with clam shell to elevation 0.0 ft. The remainder of the backfill to elevation 6.0 ft was to be made utilizing selected material from the spoil area.

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.

As-built Conditions.

Changes Between Design and Construction **(i.e. cross sections, alignment, sheet-pile tip elevation, levee crest elevation).**

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 Modification Log Report; no applicable modifications or changes were found.

Inspection During Original Construction, QA/QC, State What Records Are Available. See pages III-134 through III-135, Orleans East Bank, for description of how records are kept.

DACW29-02-C-0044. Lake Pontchartrain, IHNC – Paris Road, Levee Enlargement, St. Bernard Parish.

Attached to QA/QC reports are in-place density tests and preparatory phase inspection checklists.

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, “Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures.” Bridges are inspected under ER 1110-2-111, “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, “Flood Control Operations and Maintenance Policies,” dated 30 October 1996. (This engineering regulation supersedes the previous regulation ER 1130–2–339, “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.

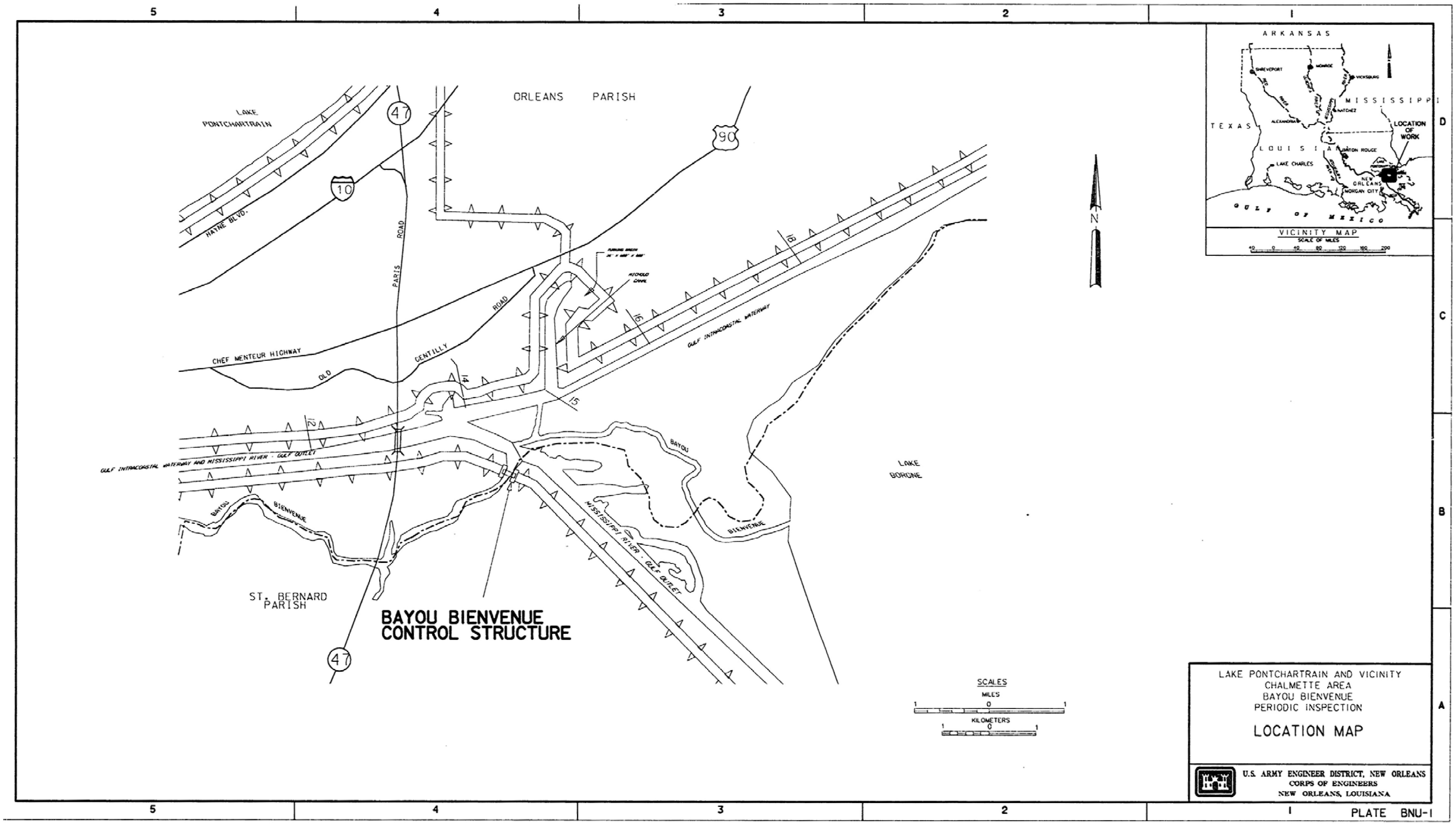
Annual Compliance Inspection (i.e., trees, etc.). Annual inspections were conducted by Operations Division for projects under the Inspection of Completed Works Project for the St. Bernard polder, which is a part of the Lake Pontchartrain and Vicinity HPP. These inspections, which were general in nature, 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.

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 information summarizes the inspection and repairs history for these structures.

Periodic Inspections of Bayou Bienvenue Control Structure (Reference 60). Image of area shown on following page.



LAKE PONTCHARTRAIN
ORLEANS PARISH
HAYNE BLVD.
PARIS ROAD
CHEF MENTEUR HIGHWAY
OLD GENTILLY ROAD
GULF INTRACOASTAL WATERWAY
GULF INTRACOASTAL WATERWAY AND MISSISSIPPI RIVER - GULF OUTLET
MISSISSIPPI RIVER - GULF OUTLET
BAYOU BIENVENUE
LAKE BORGNE
ST. BERNARD PARISH
BAYOU BIENVENUE
CONTROL STRUCTURE
N
VICINITY MAP
ARKANSAS
MISSISSIPPI
TEXAS
LOUISIANA
GULF OF MEXICO
SHREVEPORT
MONROE
VICKSBURG
NATCHEZ
ALEXANDRIA
BATON ROUGE
LAKE CHARLES
NEW ORLEANS
MORGAN CITY
LOCATION OF WORK
SCALES
MILES
KILOMETERS
LAKE PONTCHARTRAIN AND VICINITY
CHALMETTE AREA
BAYOU BIENVENUE
PERIODIC INSPECTION
LOCATION MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
PLATE BNU-1

Historical Deficiencies Reported During and Related to Periodic Inspections.

Date	Description of Observations
October 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 riprap.
March 1983	Riprap along the approach channels was missing again. Additional riprap, 275 ft north and 200 ft south of the structure, was recommended to be placed to assist in erosion control. A 1-in.-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 riprap 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 riprap 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 guide wall/ 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 ground fault circuit interrupt (GFCI) units. Guide walls were noted to be in poor condition. An evaluation was recommended to determine if major repair and or replacement of the guide walls was necessary. Settlement markers needed repainting. A reliable benchmark was required.

Historical Repairs/Construction Work Bayou Bienvenue Structure.

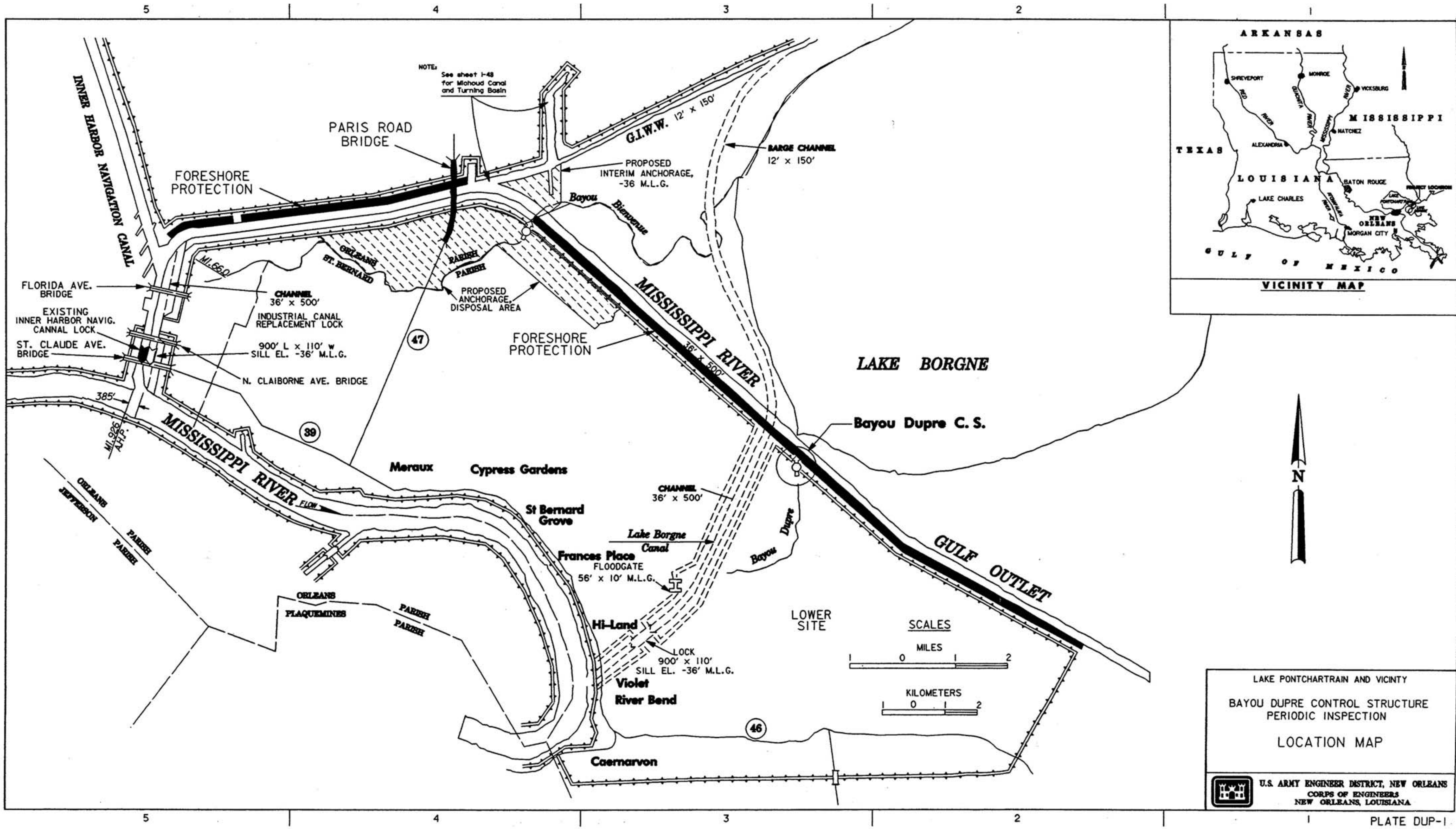
Date	Description
July 1979	Damage due to vandalism was repaired as part of regular maintenance. Riprap was placed along the landside channel banks to prevent erosion. Ladders were installed on the protected side of the structures to provide access from ground level to the top of the structure. The concrete 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-in. 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 gate bays. 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 sheet-pile 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 guide wall members were replaced as necessary.

March 1994-97	Deficient riprap, 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.
February 1997	On the west side of the structure, the steel sheet-pile floodwall levee tie-in (installed in 1993) was cut off 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 were replaced. The broken weather proof cover on the receptacle near the access stairs was replaced.

Period Inspections of Bayou Dupre Control Structure (Reference 61). Image of area shown on following page.

Historical Deficiencies Reported During and Related to Periodic Inspections.

Date	Description of Observations
13 February 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 February 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 March 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 ft 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 in. at the top of wall to 0.0 in. 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.



NOTE: See sheet I-48 for Michoud Canal and Turning Basin
PARIS ROAD BRIDGE
FORESHORE PROTECTION
INNER HARBOR NAVIGATION CANAL
G.I.W.W. 12' x 150'
PROPOSED INTERIM ANCHORAGE, -36 M.L.G.
BARGE CHANNEL 12' x 150'
Bayou Bienvenue
ORLEANS PARISH
ST. BERNARD PARISH
PROPOSED ANCHORAGE DISPOSAL AREA
FLORIDA AVE. BRIDGE
CHANNEL 36' x 500'
EXISTING INNER HARBOR NAVIG. CANNAL LOCK
INDUSTRIAL CANAL REPLACEMENT LOCK
900' L x 110' w SILL EL. -36' M.L.G.
ST. CLAUDE AVE. BRIDGE
N. CLAIBORNE AVE. BRIDGE
FORESHORE PROTECTION
MISSISSIPPI RIVER 36' x 500'
LAKE BORGNE
Bayou Dupre C. S.
MISSISSIPPI RIVER
FLOW
Meraux
Cypress Gardens
St Bernard Grove
Frances Place
FLOODGATE 56' x 10' M.L.G.
CHANNEL 36' x 500'
Lake Borgne Canal
Bayou Dupre
GULF OUTLET
ORLEANS PARISH
JEFFERSON PARISH
ORLEANS PARISH
PLAQUEMINES PARISH
Hi-Land
LOCK 900' x 110' SILL EL. -36' M.L.G.
Violet
River Bend
Caernarvon
LOWER SITE
SCALES
MILES
KILOMETERS
ARKANSAS
MISSISSIPPI
TEXAS
LOUISIANA
SHREVEPORT
MONROE
VICKSBURG
NATCHEZ
ALEXANDRIA
BATON ROUGE
LAKE CHARLES
NEW ORLEANS
MORGAN CITY
GULF OF MEXICO
VICINITY MAP
N
LAKE PONTCHARTRAIN AND VICINITY
BAYOU DUPRE CONTROL STRUCTURE PERIODIC INSPECTION
LOCATION MAP
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
PLATE DUP-1

December 1982	Minor damage occurred to the service wharf on the protected side of the structure.
1 December 1983	The following items were noted during Periodic Inspection No. 3: (1) A segment of Range 17+00 (300 ft from the east bank) had scoured approximately 10 ft; (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 was noted; (4) Sector gates had corrosion within the tidal fluctuation area; (5) A hole 10 ft wide by 30 ft long by 5.5 ft 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 June 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 were not usable; (7) The 10- by 30- by 5-ft deep hole was apparently the remnants of a drainage ditch and had been closed by a small stone dike; (8) The L-shaped water stop and filler material had separated at the joint between the east gate monolith and the wing wall 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 at an accelerated rate as compared to the west I-wall.
18 March 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 destroyed by corrosion and removed by the contractor.

8 April 1987	The following items were noted during Periodic Inspection No. 5 (Phase 2): No new deficiencies were noted. Those deficiencies noted on 18 March had been corrected. A coal tar epoxy paint system was used instead of a vinyl paint system (original).
25 April 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 sheet pile due to differential settlement; (9) Between the two west side T-type floodwall monoliths, a ½-in. 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 in. on the east side and 1-½ in. 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 1-½ inches was observed between each of the four wing walls 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 (½ to 1-½ in.) 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 April 1993	The following items were noted during Periodic Inspection No. 7: (1) The east and west concrete sheet-pile walls were still settling; (2) Minor weathering effects were noted on the handrails; (3) The concrete sheet-pile

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 sheet pile and the T-wall on the east side; (5) Small spalls were noted in the surface of the gate bay monoliths and the channel walls on both the east and west sides; (6) Hairline cracks were noted on top of the gate bay 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 gate bay structure and handrail; and (8) Efflorescence was noted on the channel walls and on both west and east sides of the gate bay structures.

3 September 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/gate bay expansion joint material at the east and west side joints had deteriorated - the water stop was exposed; (3) The T-wall sheet-pile 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 sheet-pile 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 in. from the gate bay structure at the top of the wall - "L" shaped water stop barely spans the opening, and a depressed area in the backfill behind this joint was noted; (6) The corresponding openings in the three other wing walls 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.

Historical Repairs/Construction Work Bayou Dupre Structure.

Date	Description
April 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 sheet-pile I-walls were installed; and (4) Remaining 4 ft of earth fill was placed from the west I-wall to the shell closure across Bayou Dupre.
October 1975	Repairs were made to the north approach channel. Riprap 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.
April 1976	The Coast Guard had repaired navigational lights, lens, and batteries.
July 1977	Seven hundred linear feet of trench was dug for running electrical lines/ conduits to the control house and control panel boxes.
9 February 1981	Scour repairs were completed on both sides of the structure. The channel required 45,000 tons of riprap, 23,000 tons of Class “C” stone and 10,800 cu yd of shell.
2nd Quarter 1981	The Levee Board added fill and made general repairs to the tie-in levees.
Prior December 1983	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 June 1986	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.
March-April 1987	The following was performed: (1) Blasting and painting of sector gates was completed, 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) The timber fender system on both gates was replaced; (5) Repairs to the dolphins and their navigation lights on the east side of the structure were completed; (6) The tidal current warning system was repaired; (7) Staff gages were replaced; (8) The cathodic protection system was completely replaced, three rows of ship hull anodes were installed on each skin plate, new 60-in., 250-lb anodes were located within

PVC protection tubes, and the contractor drilled four 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 April 1993	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 sheet pile was repaired.

Other Features.

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. However, other drainage and flood control features that work in concert with the Corps levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section describes and presents 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.

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 29 August 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.

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 GIWW and 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 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 No. 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 has 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 13 and 14 show typical inlets and streets in St. Bernard. Photo 15 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 on page III-249 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 16, 17, and 18).The interior canals collect stormwater from streets and storm sewers and covey it to the pump stations, but they also are storage areas that work in conjunction with the pump stations.


Photos 13 and 14. Typical streets and inlets – St. Bernard.

Photo 15. Typical roadside ditch collector.

Photo 16. Interior Canal from Judge Perez Drive.

Photo 17. Interior Canal near Judge Perez Drive.

Photo 18. Forty Arpent Canal and at Pump Station No. 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 19). The calculated capacity of the interior canals and pump stations is 0.4 in./hr. 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 19. Onsite detention Basin on Judge Perez Drive.

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 17 is a map of St. Bernard Parish with the pump stations that were studied identified by red rectangles. 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 stations 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 eight 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).

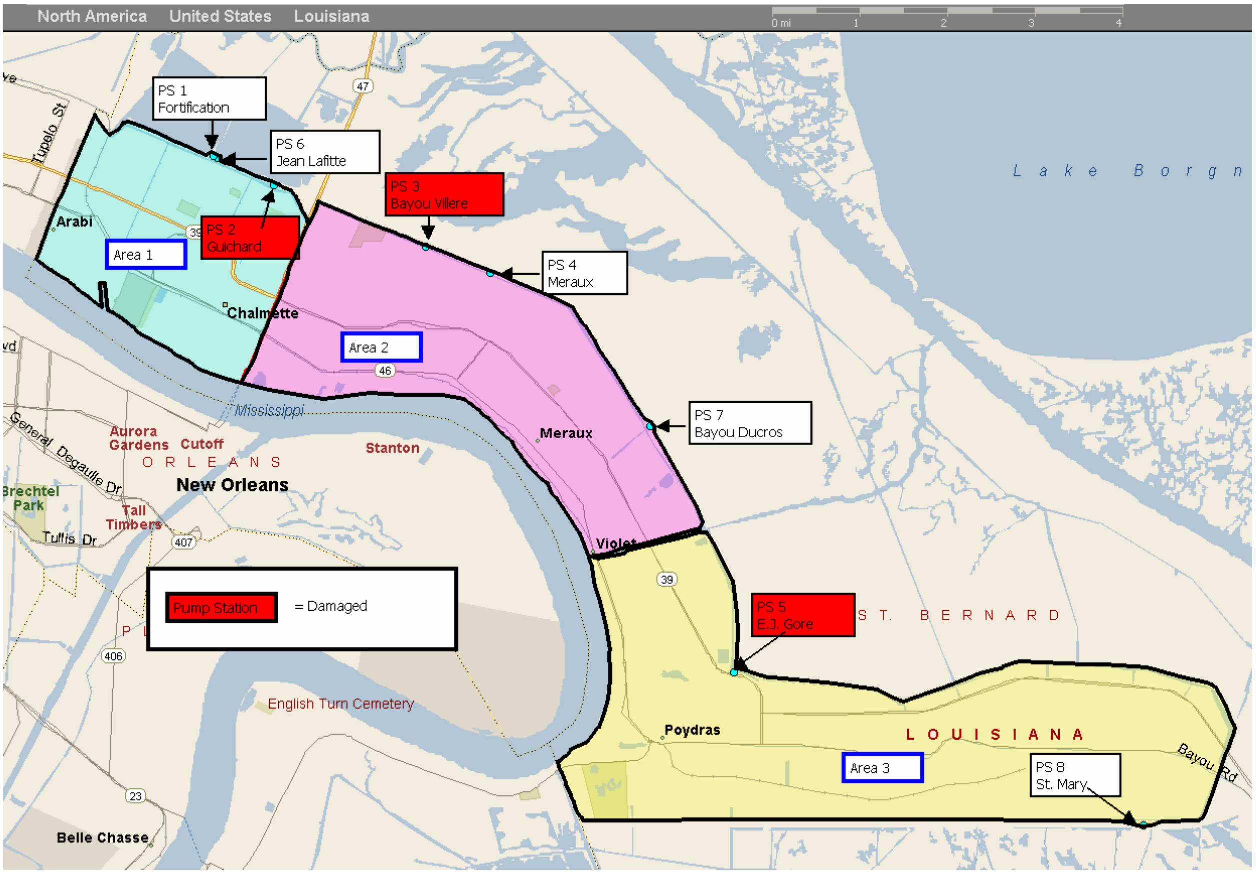


Figure 17. St. Bernard Parish pump station locations.

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 areas 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	1950s	Diesel	Horizontal
2**	267	1950s	Diesel	Horizontal
3	180	1950s	Diesel	Horizontal
4**	267	1950s	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	1950s	Diesel	Horizontal
2**	267	1950s	Diesel	Horizontal
3***	267	1950s	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	1980s	Diesel	Horizontal
2	111	1980s	Diesel	Horizontal
3	111	1980s	Diesel	Horizontal
4	111	1980s	Diesel	Horizontal
5	111	1980s	Diesel	Horizontal
6	111	1980s	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

Levees and Floodwalls.

MRL. MRL levees and floodwalls are addressed on page III-150, Orleans East Bank, MRL. There are some short reaches of floodwall near the IHNC and the Chalmette Battlefield that are part of the MRL.

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

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.

Pre-Katrina

The Jefferson Parish portion of the Lake Pontchartrain and Vicinity project is under construction. As of 29 August 2005, all of the levees had two 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-in 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 IPET 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 1,500 ft 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.

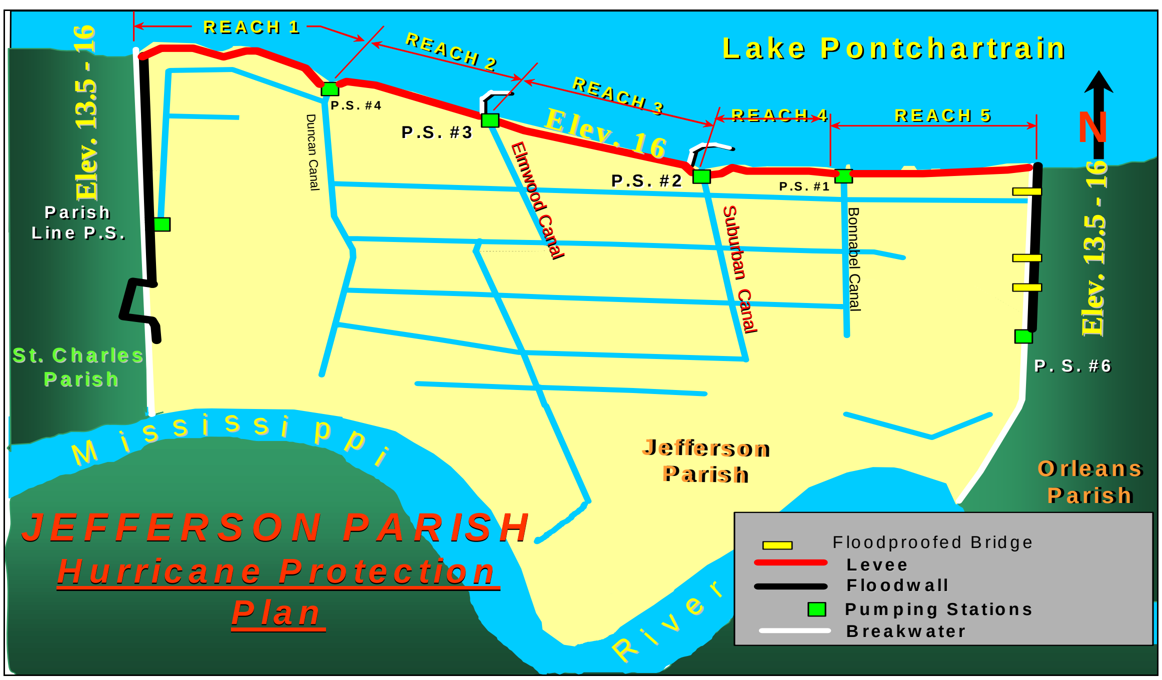


Figure 18. Jefferson Parish hurricane protection plan.

Design Criteria and Assumptions - Functional Design Criteria and Revisions/Deviations

Hydrology and Hydraulics.

For Jefferson East Bank, the design hurricane characteristics utilized in the design memoranda are shown in Table 16; 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 16
Design Hurricane Characteristics

Location	Track	CPI, in.	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] Wind speeds represent a 5-minute average 30 ft above ground level.

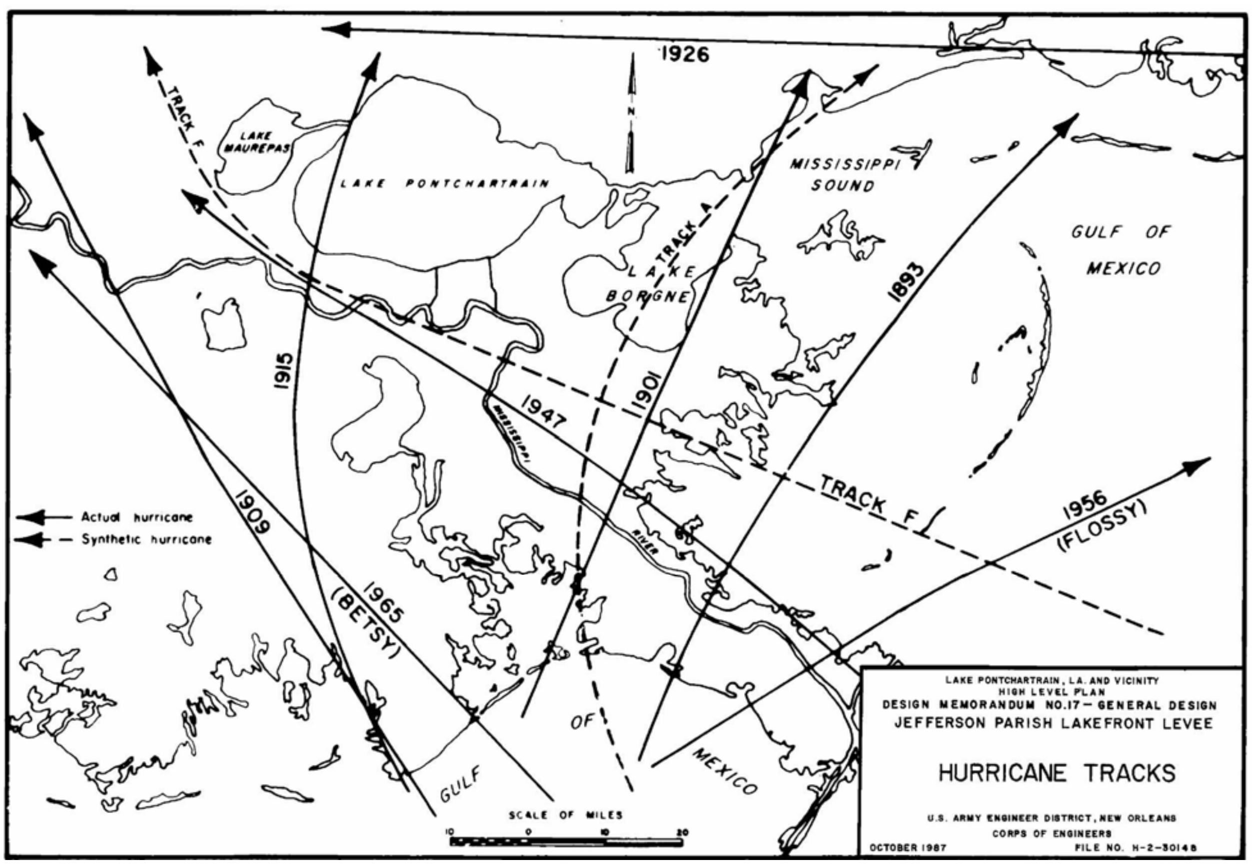


Figure 19. Hurricane tracks, Jefferson Parish Protection System

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.

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.

Summary. Table 17 contains maximum surge or wind tide level, wave, and design elevation information.

Table 17
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, November 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, July 1987				11.5 NGVD	-	3.0	14.5 NGVD and greater[1]
Return Levee, Sta. 173+04.7 to 130+70	DM17A, July 1987				11.5 NGVD	-	3.0	14.5 NGVD
Return Levee, Sta. 130+70 to 65+20.4	DM17A, July 1987				11.0 NGVD	-	3.0	14.0 NGVD

[1] Transitions to Lakefront design elevation.

Geotechnical.

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 1950s 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 elevation 18.0 ft (net 16.0 ft) for Reach A and, elevation 16.0 ft (net 16.0 ft) 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.

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 ft, but range from -45 to -100 ft.

A surficial marsh veneer, 5 to 15 ft 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.

Foundation Conditions. The upper 20 ft 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 to 30 ft 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 elevation -50 ft. These are the dominant features in the design of the foundation works. The foundation conditions are the same virtually throughout the Jefferson Lakefront Project. Reach A possesses a better shear strength with C and B progressively worse.

Field Exploration.

a. A total of 38 5-in.-diameter undisturbed and 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 ft 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.

Seepage. The analyses for required penetration for seepage cutoff 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.

Pile Foundation. Ultimate compression and tension pile capacities versus tip elevations were developed for 12-, 14- and 16-in.-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 18 and 19. The estimated tip elevations are based on the FSs presented in Table 20.

Table 18
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 19
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 20 Recommended Factor of Safety	
With Pile Load Test	**Without 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, 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.

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., ed. 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 FS 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 pounds per inch 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, 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 using the design Q strengths with appropriate hydraulic loading and were based on a minimum FS of 1.3.

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. FSs of 1.5 for static water and 1.25 for static water plus dynamic wave force were applied to design shear strengths. 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.

T-Walls. *Deep Seated Stability Analysis.* A conventional stability analysis utilizing a 1.30 FS 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.

Erosion Protection. Due to the short duration of 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.

Review Comments. No comments to change design criteria.

Jefferson Parish, St. Charles Parish Return Levee. This project is 3.4 miles in length and includes all but 225 ft of T-wall on piling under Interstate 10. A levee/I-wall was used.

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 sub-surface deltaic, lacustrine, and marine deposits. Subsurface elevations at the top of Pleistocene average -65 ft, but range from - 45 to approximately -105 ft.

A surficial marsh veneer, 5 to 15 ft thick throughout the project, represents the last stage of sedimentation in the area. Marsh type sediments are a result of annual Mississippi River over-bank 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.

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.

Field Investigation. A total of 12 general type and 14 undisturbed soil borings were taken and tested by the Corps along the alignment of the existing levee/I-wall. The general type soil borings, 1-G through 11-G and 5-GA, extend to an approximate elevation of -100 ft NGVD and the 14 undisturbed soil borings, 1-U through 14-U, extend to an elevation between -80 and -100 ft NGVD.

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 of 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 ft governed the required penetration.

b. T-Wall. A steel sheet-pile cutoff 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 of 3.0 for soft clays was determined for the T-wall sections. The required penetration for seepage cutoff is elevation -12.0 ft. The steel sheet-pile construction tip elevation is -18.75 ft, since the existing 20-ft long steel sheet piling in the existing levee will be used.

Pile Foundation. Ultimate compression and tension pile capacities versus tip elevations were developed for 12- and 14-in. square prestressed concrete piles. Overburden stress in the soft clay material was limited to approximately 1,000 lb/sq ft 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 FS presented in Table 21.

Table 21
Recommended Factors of Safety for Pile Capacity Curves

	With Pile Load Test	Without 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, 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.

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 FS of 1.3.

b. For the I-Walls. A conventional stability analysis utilizing a 1.30 FS 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.

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 FS of 1.5 was applied to the design shear strengths as follows: 0 developed = arctan (tan 0 available/FS) and cohesion value/FS. 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. The S-Case governed the required penetration.

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 FS of 1.3.

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 floodside 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 in. of riprap or gabions on a 6-in. -thick shell bedding.

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.

Structural.

Jefferson Parish Lakefront Levee – Reference 45. Images of map and plan view shown on following pages. As constructed the Jefferson Parish Lakefront Levee Hurricane Protection System consists primarily of geotextile-reinforced earthen levee. Structures included 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 No. 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 No. 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 No. 1 (Bonnabel) and No. 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.

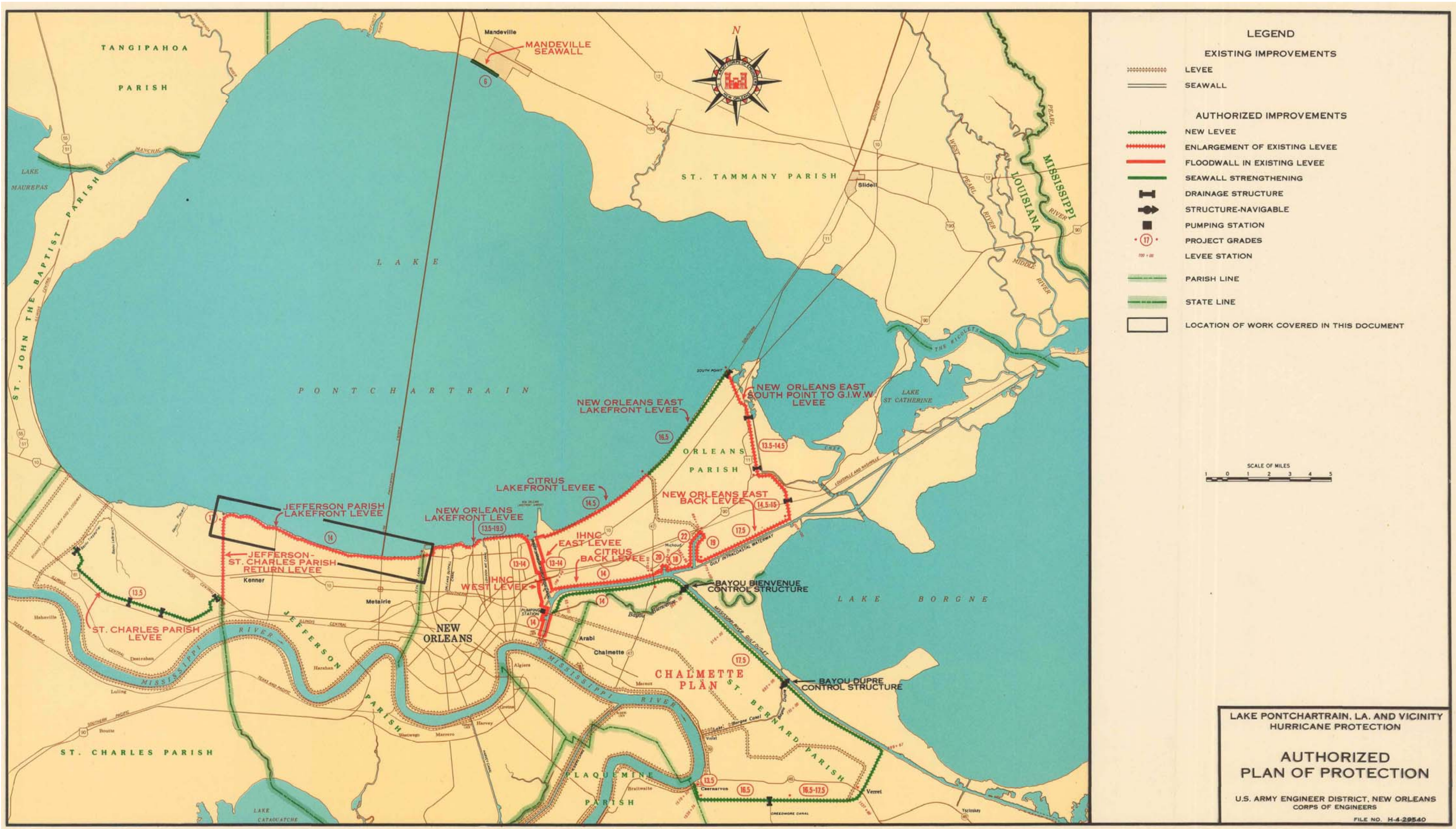
A supplemental design memorandum was proposed to cover hurricane protection at pumping Stations No. 2 (Suburban) and No. 3 (Elmwood). However, fronting protection constructed at both stations is similar to that of PS No. 1 and PS No. 4. In addition, pile-founded concrete and sheet-pile breakwaters are 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 (ft NGVD)*
Wind Tide Level (Lake Pontchartrain)	11.5
Landside of Floodwall	0.0



LEGEND
EXISTING IMPROVEMENTS
LEVEE
SEAWALL
AUTHORIZED IMPROVEMENTS
NEW LEVEE
ENLARGEMENT OF EXISTING LEVEE
FLOODWALL IN EXISTING LEVEE
SEAWALL STRENGTHENING
DRAINAGE STRUCTURE
STRUCTURE-NAVIGABLE
PUMPING STATION
PROJECT GRADES
LEVEE STATION
PARISH LINE
STATE LINE
LOCATION OF WORK COVERED IN THIS DOCUMENT
SCALE OF MILES
LAKE PONTCHARTRAIN, LA. AND VICINITY
HURRICANE PROTECTION
AUTHORIZED PLAN OF PROTECTION
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
FILE NO. H-4-29540
MANDEVILLE SEAWALL
LAKE PONTCHARTRAIN
TANGIPAHOA PARISH
ST. TAMMANY PARISH
ST. JOHN THE BAPTIST PARISH
LAKE MAUREPAS
NEW ORLEANS EAST LAKEFRONT LEVEE
NEW ORLEANS EAST SOUTH POINT TO G.I.W.W. LEVEE
LAKE ST. CATHERINE
ORLEANS PARISH
CITRUS LAKEFRONT LEVEE
NEW ORLEANS EAST BACK LEVEE
JEFFERSON PARISH LAKEFRONT LEVEE
NEW ORLEANS LAKEFRONT LEVEE
IHNC EAST LEVEE
CITRUS BACK LEVEE
JEFFERSON-ST. CHARLES PARISH RETURN LEVEE
IHNC WEST LEVEE
BAYOU BIENVENUE CONTROL STRUCTURE
LAKE BORGNE
ST. CHARLES PARISH LEVEE
NEW ORLEANS
JEFFERSON PARISH
CHALMETTE PLAN
BAYOU DUPRE CONTROL STRUCTURE
ST. BERNARD PARISH
ST. CHARLES PARISH
PLAQUEMINE PARISH
MISSISSIPPI RIVER
LOUISIANA
MISSISSIPPI

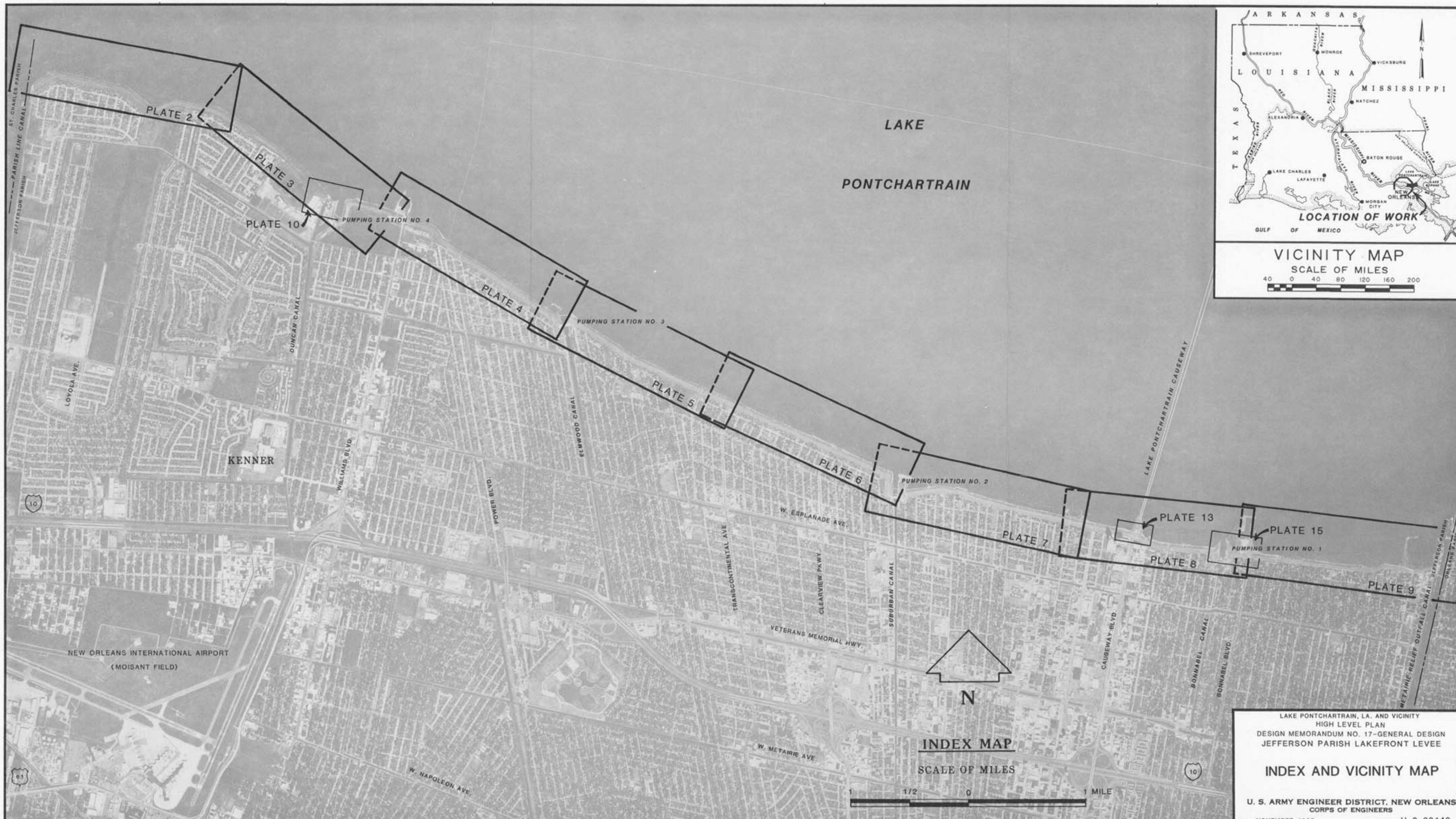

LAKE
PONTCHARTRAIN
PLATE 2
PLATE 3
PLATE 10
PUMPING STATION NO. 4
PLATE 4
PUMPING STATION NO. 3
PLATE 5
PLATE 6
PUMPING STATION NO. 2
PLATE 7
PLATE 13
PLATE 15
PUMPING STATION NO. 1
PLATE 8
PLATE 9
KENNER
NEW ORLEANS INTERNATIONAL AIRPORT
(MOISANT FIELD)
LOYOLA AVE.
DUNCAN CANAL
WILLIAMS BLVD.
POWER BLVD.
ELMWOOD CANAL
TRANSCONTINENTAL AVE.
CLEARVIEW PKWY.
SUBURBAN CANAL
CAUSEWAY BLVD.
BONNABEL CANAL
BONNABEL BLVD.
METAIRIE RELIEF OUTFALL CANAL
LAKE PONTCHARTRAIN CAUSEWAY
W. ESPLANADE AVE.
VETERANS MEMORIAL HWY.
W. METAIRIE AVE.
W. NAPOLEON AVE.
ST. CHARLES PARISH
JEFFERSON PARISH
PARISH LINE CANAL
ORLEANS PARISH
N
INDEX MAP
SCALE OF MILES
1 1/2 0 1 MILE
VICINITY MAP
SCALE OF MILES
40 0 40 80 120 160 200
ARKANSAS
LOUISIANA
MISSISSIPPI
TEXAS
SHREVEPORT
MONROE
VICKSBURG
NATCHEZ
ALEXANDRIA
BATON ROUGE
LAKE CHARLES
LAFAYETTE
MORGAN CITY
NEW ORLEANS
LOCATION OF WORK
GULF OF MEXICO
LAKE PONTCHARTRAIN, LA. AND VICINITY
HIGH LEVEL PLAN
DESIGN MEMORANDUM NO. 17-GENERAL DESIGN
JEFFERSON PARISH LAKEFRONT LEVEE
INDEX AND VICINITY MAP
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
DATE NOVEMBER 1987
FILE NO. H-2-30148

PLATE 1

Floodwall Gross Grades

	Elevations (ft 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 1 and 4)

Unit Weights

Item	*lb/cu ft*
Water	64.0
Concrete	150.0
Steel	490

Design Loads

Earth Pressure (lateral)	
Water loads	
Wind Loads	50 lb/sq ft

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	
f'_c	3,000 psi
fy (Grade 40)	40,000 psi
Maximum flexural reinforcement ratio	0.25 x balance ratio
Minimum flexural reinforcement ratio	200/fy
f'_c (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. FSs 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

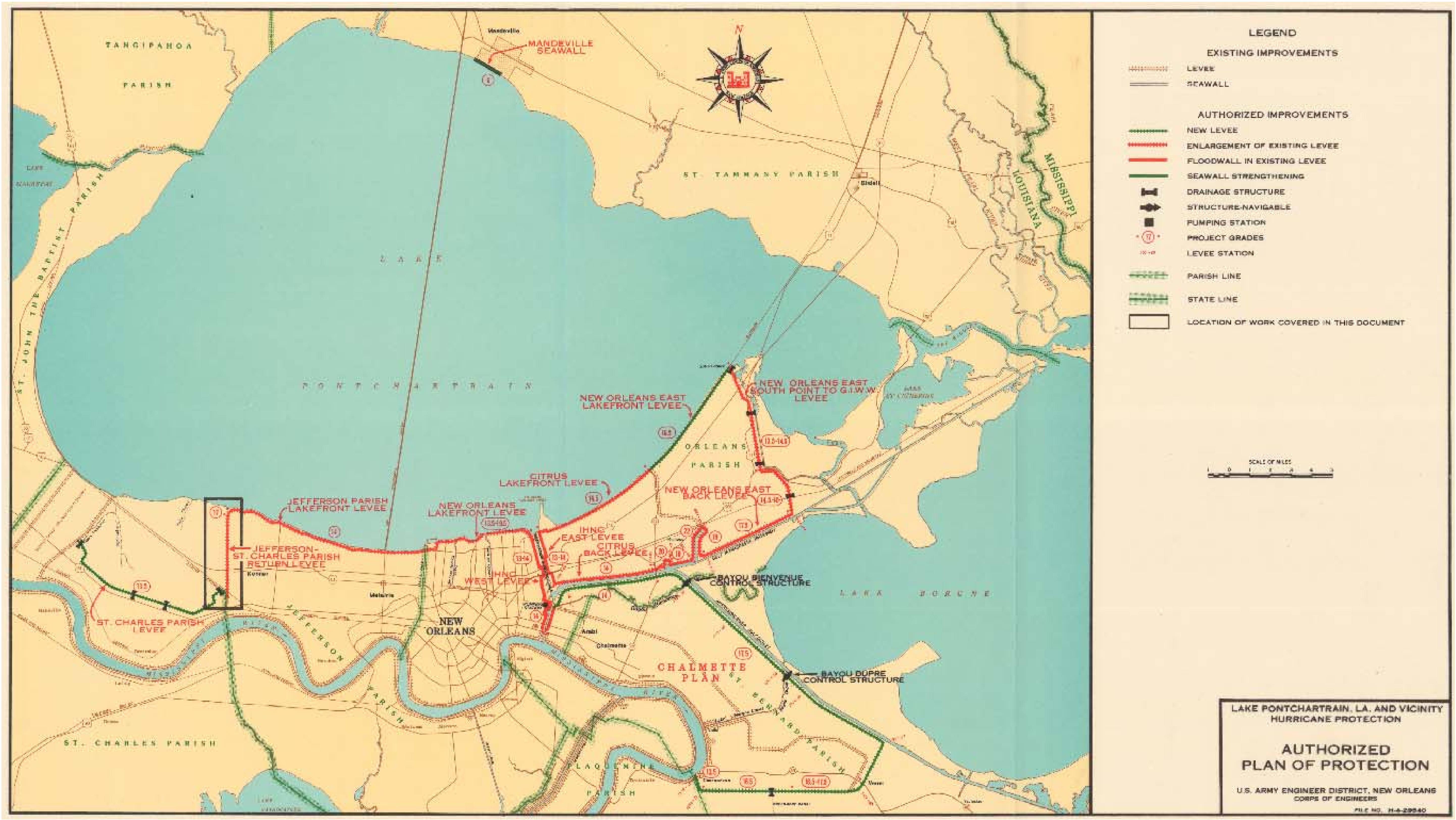
Case I	Static water pressure, no wind, impervious sheet-pile cutoff, no dynamic wave force
Case II	Static water pressure, no wind, pervious sheet-pile cutoff, no dynamic wave force
Case III	Stillwater pressure to elevation 11.5 ft, dynamic wave force, impervious sheet-pile cutoff (75 percent forces used)
Case IV	Stillwater pressure to elevation 11.5 ft, dynamic wave force, pervious sheet-pile cutoff (75 percent forces used)
Case V	No water, no wind
Case VI	No water, wind from protected side (75 percent forces used)
Case VII	No water, wind from floodside (75 percent forces used)

Gates.

Loading Cases

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

Jefferson Parish/St. Charles Parish Return Levee –Reference 46. Map of area shown on following page. 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.


LEGEND
EXISTING IMPROVEMENTS
LEVEE
SEAWALL
AUTHORIZED IMPROVEMENTS
NEW LEVEE
ENLARGEMENT OF EXISTING LEVEE
FLOODWALL IN EXISTING LEVEE
SEAWALL STRENGTHENING
DRAINAGE STRUCTURE
STRUCTURE-NAVIGABLE
PUMPING STATION
PROJECT GRADES
LEVEE STATION
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HURRICANE PROTECTION
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ST. JOHN THE BAPTIST PARISH
ST. TAMMANY PARISH
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MANDEVILLE SEAWALL
NEW ORLEANS EAST LAKEFRONT LEVEE
NEW ORLEANS EAST SOUTH POINT TO G.I.W.W. LEVEE
ORLEANS PARISH
CITRUS LAKEFRONT LEVEE
NEW ORLEANS EAST BACK LEVEE
JEFFERSON PARISH LAKEFRONT LEVEE
NEW ORLEANS LAKEFRONT LEVEE
JEFFERSON-ST. CHARLES PARISH RETURN LEVEE
IHNC EAST LEVEE
CITRUS BACK LEVEE
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NEW ORLEANS
JEFFERSON PARISH
CHALMETTE PLAN
ST. BERNARD PARISH
BAYOU DUPRE CONTROL STRUCTURE
ST. CHARLES PARISH
PLAQUEMINES PARISH
LOUISIANA
MISSISSIPPI

Structural Design

Design Criteria

Water Elevations

Water Elevations	*Elevations (ft 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 (ft 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/cu ft*
Water	64.0
Concrete	150.0
Steel	490

Design Loads

Earth pressure (lateral)	
Water loads	
Wind loads	50 lb/sq ft

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	
f'_c	3,000 psi
fy (Grade 40)	40,000 psi
Maximum flexural reinforcement ratio	0.25 x balance ratio
Minimum flexural reinforcement ratio	200/fy
f'_c (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 elevation 11.5 ft NGVD

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 FS of 1.5 with the S-Case governing. No mention was made of a FS 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 percent forces used)

Case V No water, wind from floodside (75 percent forces used)

Along Lake Pontchartrain

Case VI Still-water pressure to elevation 11.5 ft NGVD, dynamic wave force, impervious sheet-pile cutoff (75 percent forces used)

Case VII Still-water pressure to elevation 11.5 ft NGVD, dynamic wave force, pervious sheet-pile cutoff (75 percent forces used).

Sources of Construction Materials.

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, listed by DM.

Jefferson Parish	**DM**
St. Charles Return Levee	
Vicinity of the Airport	PZ-22 (East-West)
	Frodingham 1B (North-South)
Under I-10	Frodingham 1B
Vicinity of Vintage Drive	PZ-22*
Jefferson Lakefront Levee	
Recurve Wall	PZ-22*
PS No. 4 (Duncan) Tie-In	**
Williams Boulevard Roller Gate Tie-In	**
Causeway Blvd.	PZ-22 & Concrete Tie-Back System
PS No. 1 (Bonnabel) Tie-In	BZ-7 & PZ-35*
Bonnabel Boat Launch Swing Gate Tie-In	**
17th Street Canal	
PS No. 6 to Hammond Hwy	Hoesch 12
* As-advertised – Not confirmed as-built. ** Information not located at the time of publication.	

Levee Material.

Sources of Borrow Materials. The planned source of borrow for this project is to haul clay fill from the Bonnet Carre spillway.

Source of Borrow Materials. No borrow is required since the placement of the T-wall required degrading of the existing levee.

As-built Conditions

Changes Between Design and Construction (i.e., cross sections, alignment, sheet-pile tip elevation, levee crest elevation).

DACW29-99-C-0046. Lake Pontchartrain, LA, and Vicinity, High Level 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 ft west to provide adequate outflow channel width. Also, while driving sheet piles for the breakwater they encountered an obstruction, and some of the sheet piles were cut off.

DACW29-98-C-0003. Lake Pontchartrain, LA, 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, LA.

Reviewed modification log report; no applicable modifications or changes were found.

DACW29-98-C-0012. Lake Pontchartrain, LA, and Vicinity, Jefferson Parish Lakefront Levee, Jefferson Parish, Louisiana, Landside Runoff Control, Reach 1.

Reviewed modification documents; no applicable modifications or changes were found.

DACW29-98-C-0031. Lake Pontchartrain, LA, 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, LA.

Reviewed modification documents; no applicable modifications or changes were found.

DACW29-98-C-0068. Lake Pontchartrain, LA, and Vicinity, Jefferson Parish Lakefront Levee, Landside Runoff Control, Reach 5, B/L Station 422+00 to B/L Station 509+75, Jefferson Parish, LA.

Reviewed modification log report; no applicable modifications or changes were found.

Inspection During Original Construction, QA/QC, State What Records Are Available.

See pages III-134 through III-135, Orleans East Bank, for description of how records are kept.

DACW29-98-C-0012. Lake Pontchartrain, LA, and Vicinity, Jefferson Parish Lakefront Levee, Jefferson Parish, Louisiana, Landside Runoff Control, Reach 1.

Attached are in-place density tests, pile test reports, and daily material quantities.

DACW29-98-C-0031. Lake Pontchartrain, LA, 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, LA.

No QA/QC reports were found.

DACW29-98-C-0068. Lake Pontchartrain, LA, and Vicinity, Jefferson Parish Lakefront Levee, Landside Runoff Control, Reach 5, B/L Station 422+00 to B/L Station 509+75, Jefferson Parish, LA.

Attached are concrete field tests, stored material lists, percent complete list, and preparatory inspection reports.

DACW29-02-C-0016. – Lake Pontchartrain Bridge at Hammond Highway, Jefferson and Orleans Parish.

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.

DACW29-95-C-0103 – Westwego to Harvey Canal, Estelle Pumping Station – Louisiana Power and Light Powerlines, 1st Lift, Jefferson Parish, LA.

Attached are moisture analysis records and percent complete lists.

Inspection and Maintenance of Original Construction

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 provided 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.

Periodic inspections.

There are no structures which fall under the Periodic Inspection Program in the Jefferson East Bank area.

Other Features

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. However, other drainage and flood control features that work in concert with the Corps levees and floodwalls are also an integral part of the overall drainage and flood damage reduction system. This section describes and presents 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.

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 29 August 2005.

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

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 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. Floodwater 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 to 25 ft. 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 on page III-278 of this volume.

Most of the local drainage is collected by underground storm drains and ditches in this urbanized basin. Photos 20, 21, and 22 show typical inlets and streets. Photos 23 and 24 show storm sewer pipe outfalls into the canals.

Photos 20, 21, and 22. Typical streets and inlet – Jefferson East Bank.

Photo 22. Typical streets and inlet – Jefferson East Bank.

Photos 23 and 24. Storm sewer outfalls into canals.

The interior canals are open and are grass-lined, concrete-lined, or both (Photos 25 and 26). The interior canals not only collect stormwater from streets and storm sewers and covey it to the pump stations, but they also are storage areas that work in conjunction with the pump stations.

Photo 25. N. Cumberland interior pump at Interior Canal No. 4, W. Napoleon Avenue.

Photo 26. Interior Canal No. 3 from Bissonet Drive near Veterans Boulevard.

Photo 27. Elmwood Canal (interior) from Kawanee Avenue.

Design Criteria. The current design criterion for Jefferson East Bank is the 10 percent 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 in./hr. It will increase to 0.7 in./hr 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 offset downstream impacts.

Where local drainage is considered to need improvement, Jefferson Parish is working to improve the drainage. In some cases, Jefferson Parish and the Corps 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 four canals and increasing pumping capacity at Elmwood Pump Station No. 3 and the Suburban Pump Station No. 2. Prior to Hurricane Katrina, the pump stations were under construction, the Suburban Canal and Canal No. 3 were complete, Elmwood Canal was nearly complete, and Soniat Canal was partially complete.

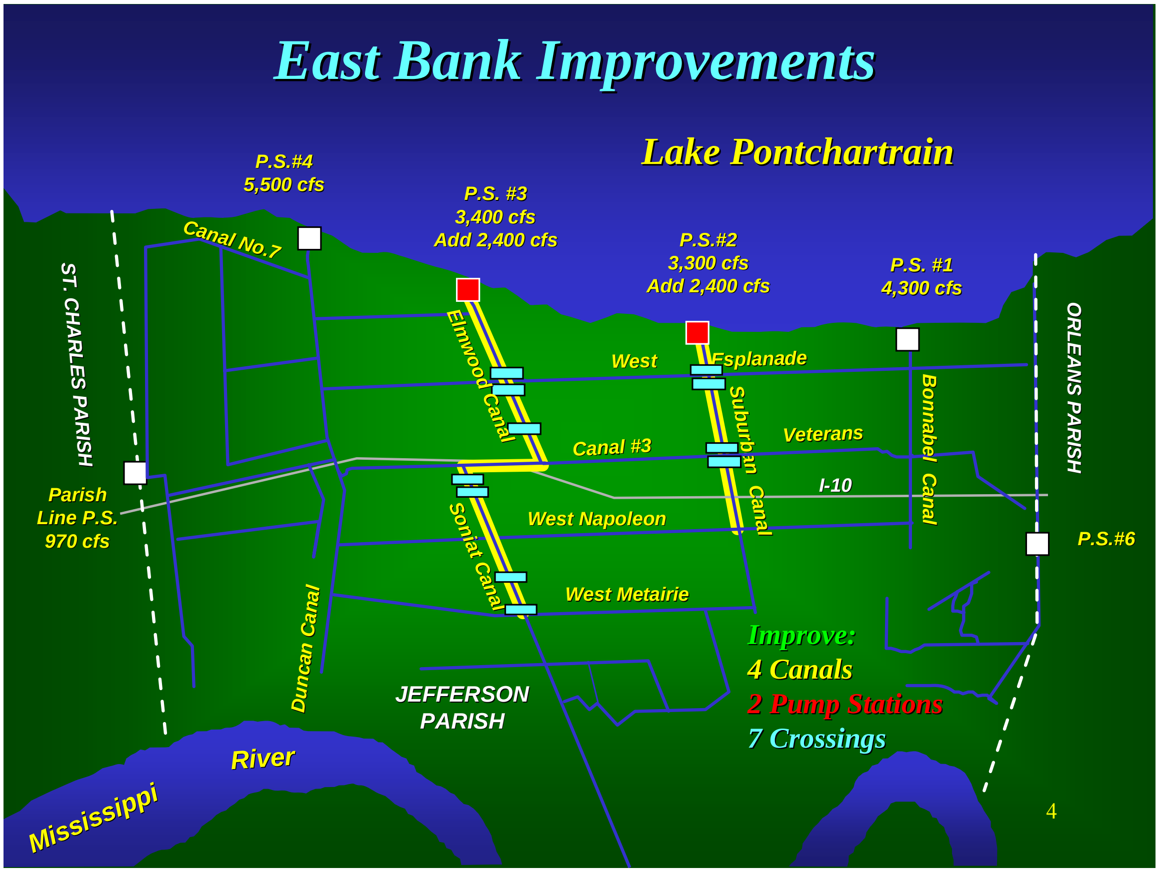

Figure 20. SELA Urban Flood Control Projects in Jefferson East Bank.

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 bullets. 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 rainwater to flow in different directions depending on the rainfall patterns and available capacities at the pump stations. The West Bank is subdivided into subbasins that, for smaller rainfall events, operate independently. However, overbank flow does occur between adjacent subbasins for a 10-year event. This report examined six 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).

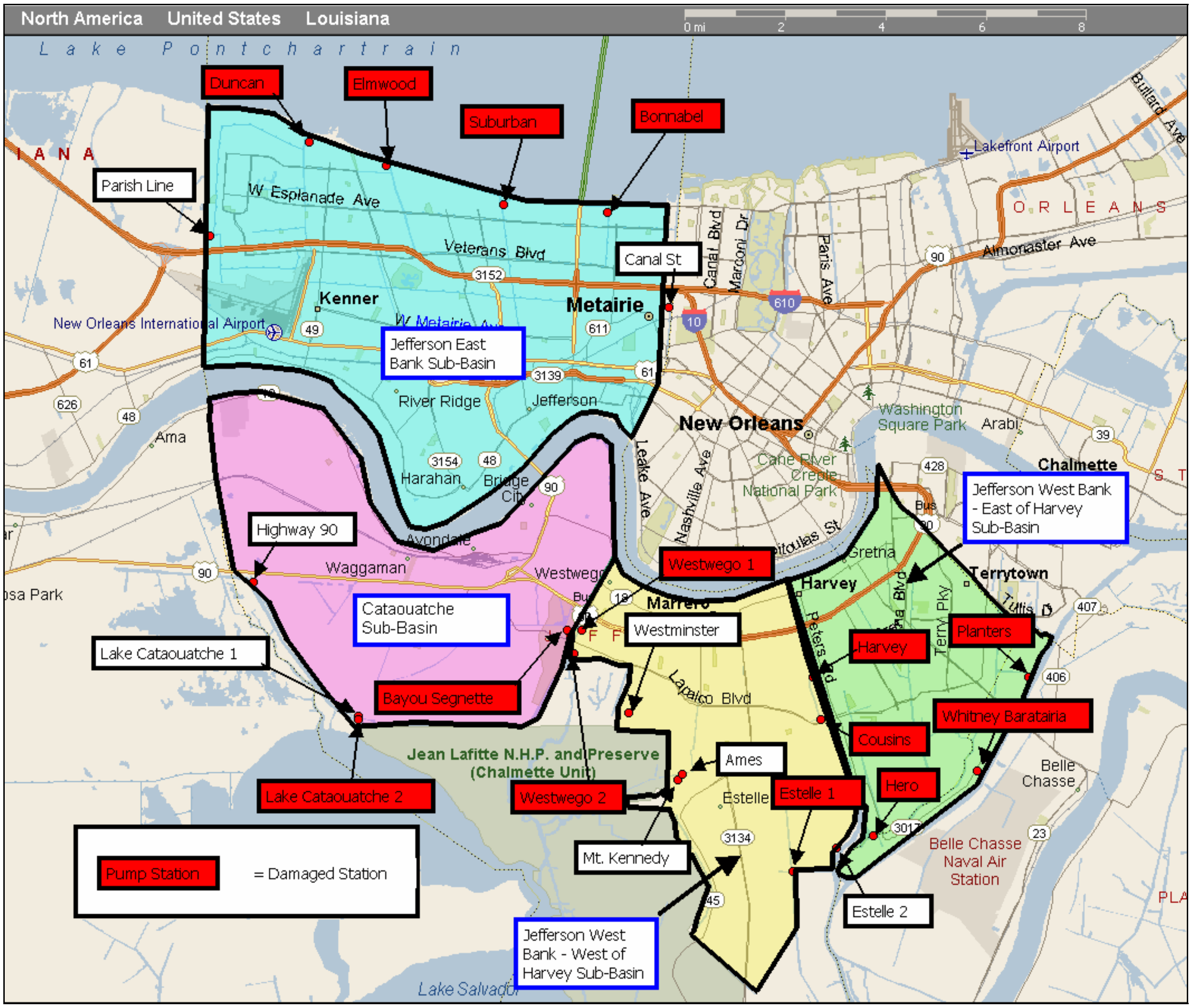

Figure 21. Jefferson Parish pump station locations.

Table 22 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; missing information was left blank or highlighted.

Table 22
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: 3,750

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	1,050	1986	Diesel	Horizontal
4	1,050	1986	Diesel	Horizontal
5	1,050	1986	Diesel	Horizontal

Suburban

Intake location: .... Suburban
Discharge location: .... Lake Pontchartrain
Nominal capacity: .... 5,155 cfs

Pump	Capacity (cfs)	Year (Installed)	Driver Electric /Diesel	Pump Configuration
1	1,050	1983	Diesel	Horizontal
2	1,050	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	1,050	2005	Diesel	Horizontal
8	1,050	2005	Diesel	Horizontal

Elmwood

Intake location: ....Elmwood Canal
Discharge location: ....Lake Pontchartrain
Nominal capacity: ....5,912 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	1,250	2004	Diesel	Horizontal
10	1,250	2004	Diesel	Horizontal

Duncan

Intake location: ....Duncan Canal
Discharge location: ....Lake Pontchartrain
Nominal capacity: ....4,800 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	1,050	1986	Diesel	Horizontal
4	1,050	1986	Diesel	Horizontal
5	1,050	1986	Diesel	Horizontal
6	1,050	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

Levees and Floodwalls.

MRL. MRL levees and floodwalls are addressed on page III-150, Orleans East Bank, MRL. There are no floodwalls that are part of the MRL Project.

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

Introduction

Hurricane Protection Features St. Charles Parish East Bank

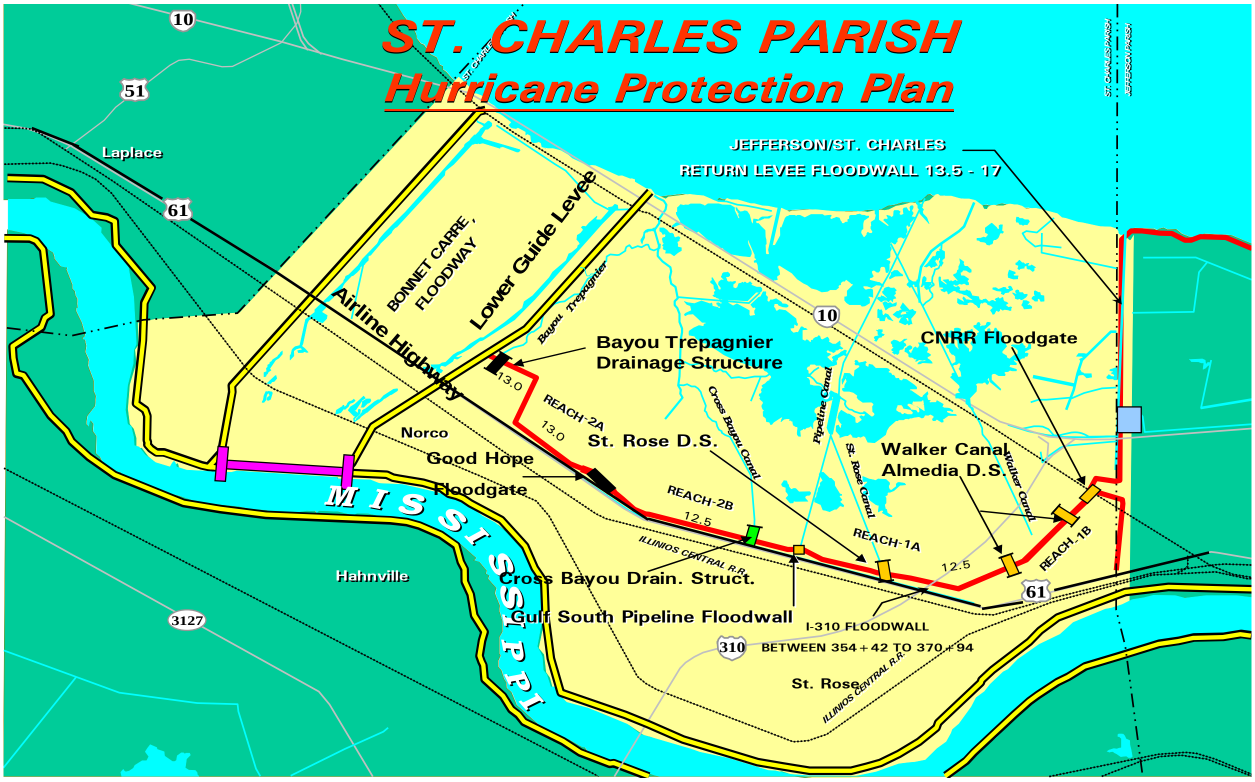

Figure 22. St. Charles Parish Hurricane Protection Plan.

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 (U.S. Highway. 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 23 **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

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 3 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 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 6 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 floodgates 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 IPET analysis and based on a reanalysis of design storm calculations. Additional contracts may be required as a result of this analysis.

Design Criteria and Assumptions - Functional design criteria.

Hydrology and Hydraulics.

For St. Charles East Bank, the design hurricane characteristics utilized in the design memoranda are shown in Table 24; 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 24
Design Hurricane Characteristics

Location	Track	CPI in.	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] Wind speeds represent a 5-minute average 30 ft above ground level.

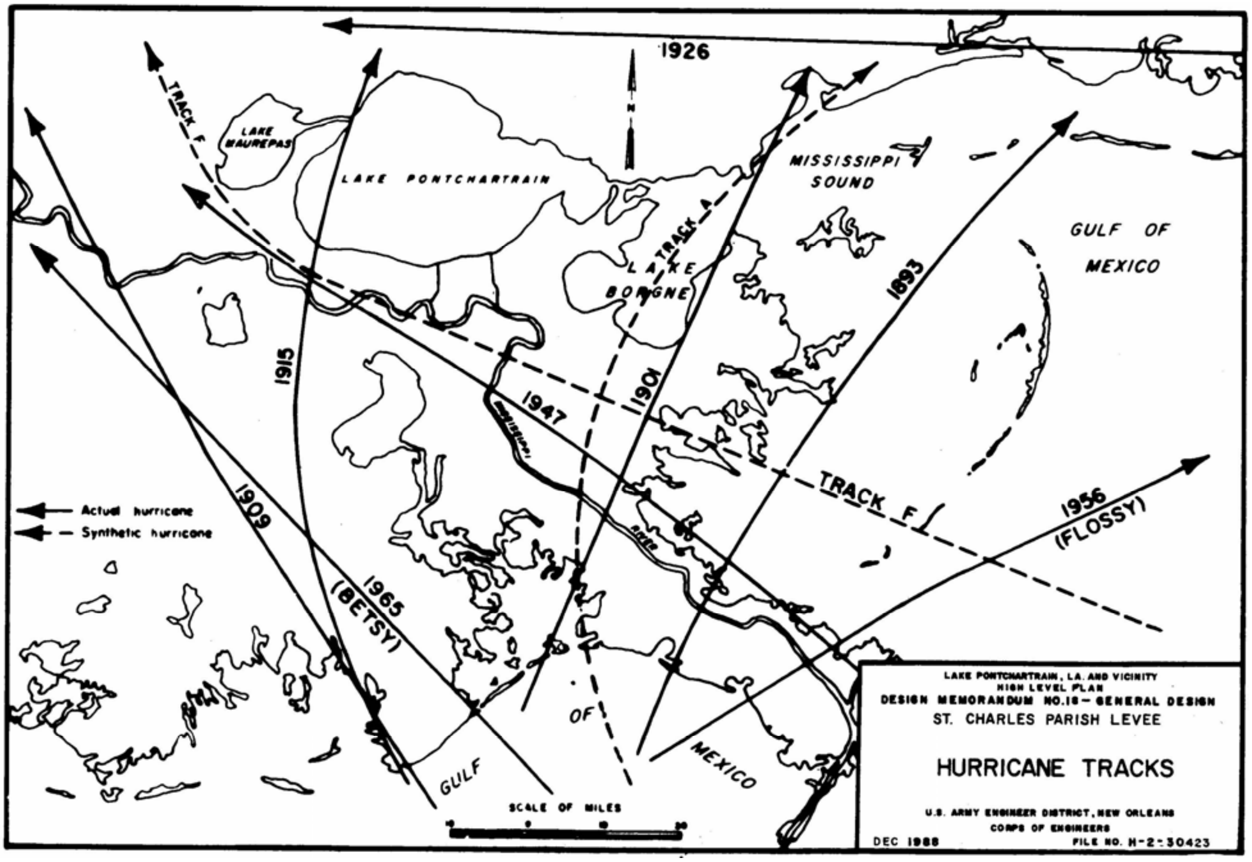

Figure 23. Hurricane tracks, St. Charles Parish Levee.

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 25 shows the wind tide level at the surge reference line and at the levee location.

Table 25
Wind Tide Levels

Location	Wind Tide Level, 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

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 floodside of the levee and raising the levee elevation 1 ft.

Summary. Table 26 contains maximum surge or wind tide level, wave, and design elevation information.

Table 26
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

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 were developed using the Corps 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.

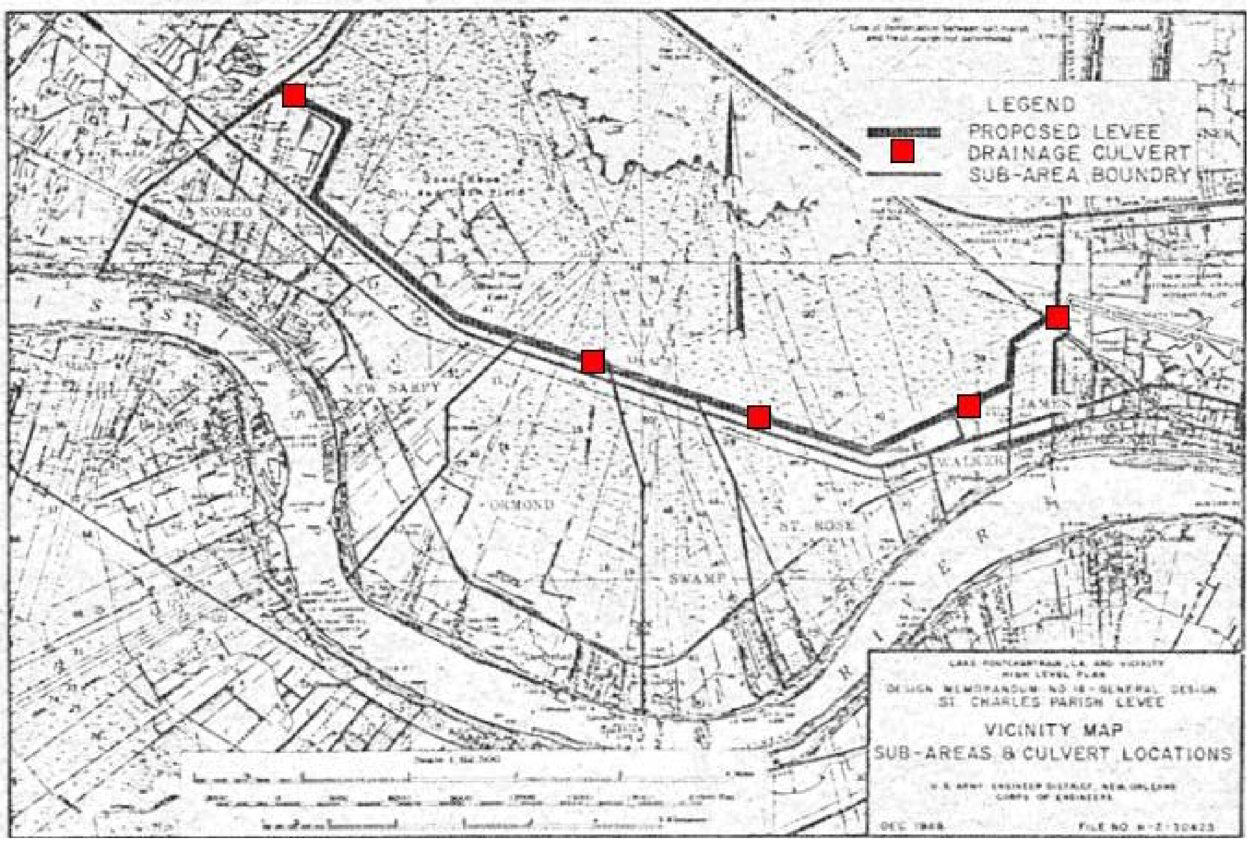

Figure 24. Culvert locations.

A submerged outlet condition was assumed. Flow through the culvert was computed by use of the equation

$$Q = CA(2gh)^{0.5}$$

where

Q = Discharge, cubic feet per second

C = coefficient of discharge, 0.80

A = clear structure area, square feet

g = acceleration due to gravity

h = difference in inside and outside water levels, feet

The pertinent design data are shown on Table 27.

Table 27
Design Data — Gated Culverts

Subdrainage Area Characteristics							Culvert Design Data					
Subdrainage Area	**Area sq mi**	**SCS CN**	**% Area Imper-vious**	**Flow Length ft**	**Tc hours**	**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

Geotechnical.

St. Charles Parish Frontline Levee. St. Charles Parish Frontline 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.

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 ft between the natural levee ridge of the present Mississippi River and the marshes adjacent to Lake Pontchartrain. Minimum elevations of mean sea level or slightly below are found in the marsh area adjacent to Lake Pontchartrain.

Foundation Conditions. The subsurface consists of Recent deposits varying in thickness from about 50 ft between Stations 25+00 and 130+00 to over 100 ft 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 to 20 ft 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 ft between Stations 30+00 and 130+00 to at least 60 ft 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.

Field Exploration. Undisturbed borings 5 in. in diameter extending to an approximate elevation -80.0 ft NGVD were made at four locations along the levee baseline (Stations 5+00, 105+00, 205+00, and 296+50). General-type core borings, 1⅞-in.-ID, extending to an approximate elevation -60.0 ft NGVD 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, 1⅞-in.-ID, extending to an approximate elevation -70.0 ft NGVD were made in the bottom of Lake Pontchartrain in the recommended borrow area opposite the levee alignment.

Seepage. Not addressed.

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-in.-square concrete piles. Design data were determined for the Q and S shear strengths. In compression, a FS of 1.75 was applied to the shear strengths and a conjugate stress ratio K_0 = 1.0 was used in the S-Case for determining the normal pressure on the pile surface. In tension, a FS of 2.0 was applied to the shear strengths and a conjugate stress ratio K_a = 0.70 was used in the S-Case. Further, pile design loads versus tip elevations are presented for 16-in.-square concrete piles for the S-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 floodside and to elevation -1.5 ft NGVD on the protected side was determined using the design Q shear strengths.

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 (elevation 10.0 ft from Stations 0+00 to 140+00 and elevation 10.5 ft 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 FS 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 elevation 0.0 ft and to resist assumed failure towards the floodside 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 FS of 1.3 with the water at elevation -6.0 ft on the floodside. However, the ultimate stream closure configuration was designed for the most critical design hurricane condition, i.e., water at elevation -6.0 ft on the lake side and the prevention of assumed failure towards the lake side.

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 S shear test results, i.e., C = 0, = ϕ_a = 23°. A FS of 1.25 was applied to the friction angle as follows:

$$\phi_d = \tan^{-1}\left(\frac{\tan\phi \text{ available}}{\text{FS}}\right)$$

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 floodside 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.

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.

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 ft of riprap placed on 0.75 ft of shell extending from elevation 6.5 to 2.8 ft NGVD along the lake side slope of the levee. In addition to the levee slope protection, erosion protection will also be provided on the floodside slopes of stream closures and will extend from elevation 0.0 ft NGVD to the bottom of the streams. Further, 2 ft of riprap on 1 ft of shell will be placed 20 ft on each side of the floodwall and will extend from elevation 8.0 ft NGVD at the earth levee to elevation -6.0 ft NGVD at the drainage structure.

Review Comments. None provided with Reference No. 94.

<u>St. Charles Parish North of Airline Highway (Reference No. 48).</u> 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.

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 range from approximately +10 to +15 ft NGVD along the natural levee of the Mississippi River to elevation 0 ft NGVD in the backswamp and marsh areas.

Foundation Conditions. Engineering properties of the sediment beneath the project vary greatly. Generally, the subsurface consists of Holocene deposits ranging in depth from 55 to 80 ft and underlain by Pleistocene deposits. Specifically, from Station 0+00 to Station 27+00, the Holocene is between 55 and 80 ft 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 content and high water content. 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 ft thick and from Station 240+00 to Station 505+00, the sequence is between 30 and 40 ft 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 range in thickness between 5 and 20 ft. The bottom of the Holocene sequence is formed by Bay-sound deposits which range in thickness from 5 to 20 ft 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.

Field Exploration.

a. A total of eleven 5-in.-diameter undisturbed and 46 general type soil borings were taken and tested by the Corps 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 elevation -60 and -70 ft NGVD; and 11 undisturbed soil borings, 1-SCU thru 11-SCU, extend to an approximate elevation of -80 ft NGVD.

b. Twenty-eight 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.

Seepage.

a. **Seepage Blanket.** A seepage blanket over the landfills is required. A minimum 3-ft-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 value 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 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. 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.

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-in.-square prestressed concrete piles and for HP 12×53 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 28 and 29.

Table 28
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 29
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 FS of 2.0 in both compression and tension since pile load tests will be performed. For piles with negative skin friction, the following equation should be used:

$$Q(All) \quad \frac{\underline{Quit}}{- FS - NEG\ Skin\ Friction}$$

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.

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 FS of 1.3 was based on the <u>larger value</u> of the following two analyses:

(1) From the 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 pounds per inch at 5 percent strain and less than 40 percent of ultimate FS = factor of safety.

b. Once the critical wedges were determined by the 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 1,600 lb/in., a two-layer system was used with two-thirds of the required tensile strength in the bottom layer and one-third in the upper layer with a minimum of 3 ft 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 past the anchorage embedment requirement to attain a FS of 1.3 of the levee berm in certain cases.

For the pipeline crossings, the levee was designed by the Method of Planes for a minimum FS of 1.3 without the geotextile reinforcement, and the reinforcement was used to attain a FS 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 Method of Planes using the design Q strengths with appropriate hydraulic loading and was designed for a minimum FS of 1.3.

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 short-term Q-Case 1.5 for static water, 1.0 for static water plus 2 ft of freeboard, and long-term S-Case 1.2 for static water, were applied to the design shear strength as follows: φ developed = arctan (tan φ available/FS) and cohesion/FS. 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 was 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.

T-Walls. A conventional stability analysis utilizing a FS of 1.30 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.

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.

Review Comments. LMVD Comments (First endorsement) to limit the strain in geotextile to 5 percent instead of 7 to 8 percent were concurred with by the New Orleans District.

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, the levee 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

Basic data

Water elevations	*Elevations (ft 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

Grades

Floodwall Gross Grade	*Elevations (ft NGVD)*
I-wall	12.00 to 13.50
T-wall	12.00 to 13.00

Unit Weights

	lb/cu ft
Water	64.00
Concrete	150.00
Steel	490.00
Riprap	132.00
Saturated sand	122.00
Saturated clay	110.00
Saturated shell	117.00

Uniform Live Loads

Item/Description	*lb/sq ft*
Floors for vertical lift gate machinery	100

Design Loads

Wind loads	50 lb/sq ft

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-day 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

f'_c	3,000 psi
fy (Grade 40)	40,000 psi
Maximum flexural reinforcement ratio	0.25 x balance ratio
Minimum flexural reinforcement ratio	200/fy
f'_c (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 FS of = 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 percent forces used)
Case II	Static-water pressure to swl, no wind, pervious sheet-pile cutoff, no dynamic wave force (100 percent forces used)
Case III	Static-water pressure to swl, no wind, impervious sheet-pile cutoff, no dynamic wave force (100 percent forces used)
Case IV	Static-water pressure with water level 2 ft above swl, no wind, impervious sheet-pile cutoff, no dynamic wave force (75 percent forces used)
Case V	Static-water pressure with water level 2 ft above swl, no wind, pervious sheet-pile cutoff, no dynamic wave force (75 percent forces used)
Case VI	No water, wind from floodside (75 percent 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 U.S. Highway 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 ft 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. FSs 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 3:1 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 FS = 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 percent 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 percent forces used)
Case III	Static-water pressure to swl, no wind, impervious sheet-pile cutoff, no dynamic wave force (100 percent forces used)
Case IV	Static-water pressure with water level 2 ft above swl, no wind, impervious sheet-pile cutoff, no dynamic wave force (75 percent forces used)
Case V	Static-water pressure with water level 2 ft above swl, no wind, pervious sheet-pile cutoff, no dynamic wave force (75 percent forces used)
Case VI	No water, wind from land side (75 percent forces used)
Case VII	No water, wind from canal side (75 percent 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 FS = 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 percent forces used)

Case II Gate closed, no wind, static-water pressure to swl, no wind, pervious sheet-pile cutoff, no dynamic wave force (100 percent forces used)

Case III Gate closed, static-water pressure to with water level 2 ft above swl, no wind, impervious sheet-pile cutoff, no dynamic wave force (75 percent forces used)

Case IV Gate closed, static-water pressure with water level 2 ft above swl, no wind, pervious sheet-pile cutoff, no dynamic wave force (75 percent 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 floodside 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 floodside edge of base slab

Case X Gate open, wind from floodside, truck or train on protected side edge of base slab.

Sources of Construction Materials.

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	DM
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.	

Levee Material.

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.

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. There are ten soil reaches along the length of the alignment, and 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.

As-built Conditions

Changes Between Design and Construction (i.e., cross sections, alignment, sheet-pile tip elevation, levee crest elevation).

DACW29-98-C-0064. Lake Pontchartrain, LA, and Vicinity, High Level Plan, St. Charles Parish North of Airline Highway, Floodwall at I-310 Interchange, St. Charles Parish, LA.

Modification Nos. A00008 and A00009 were issued to allow excavation to remove pile driving obstructions, backfill with sand, rebuild levee, and drive sheet piles.

Inspection During Original Construction, QA/QC, State What Records Are Available.

See pages III-134 through III-135, Orleans East Bank, for description of how records are kept.

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.

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.

Periodic Inspections.

There are no structures under the Periodic Inspection Program in this polder.

Other Features

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. However, other drainage and flood control features that work in concert with the Corps levees are also an integral part of the overall drainage and flood damage reduction system. This section briefly describes and presents 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.

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 29 August 2005.

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

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 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.

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.

Levees and Floodwalls.

MRL **-** MRL levees and floodwalls are addressed on page III-150, Orleans East Bank, MRL. There are no floodwalls that are part of the MRL Project in this reach.

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.