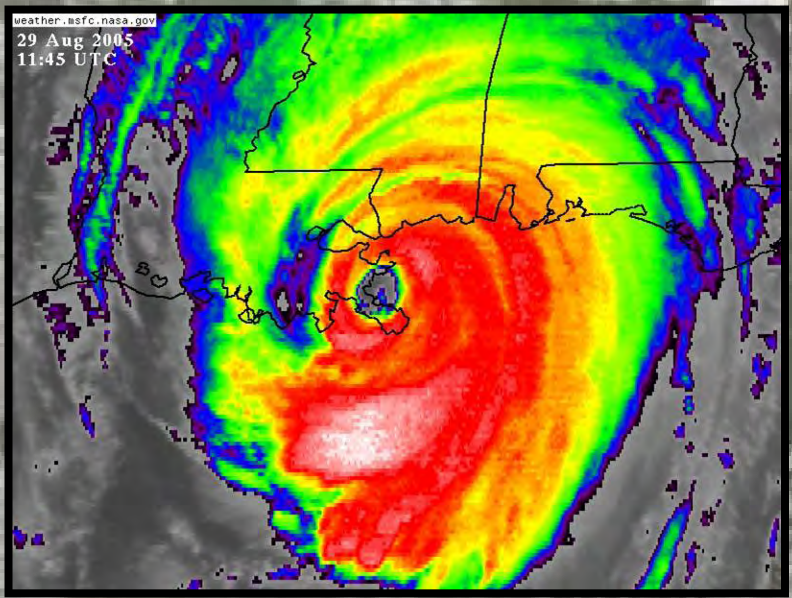
US Army Corps of Engineers

Performance Evaluation Status and Interim Results, Report 2 of a Series

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

by Interagency Performance Evaluation Task Force

10 March 2006



FINAL DRAFT

(Subject to Revision)

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Contents

IX. Appendices

I. Executive Summary

Report 2, Performance Evaluation Status and Interim Results, is the second in a series concerning the in-depth analysis of the New Orleans and Southeast Louisiana Hurricane Protection System being conducted by the Interagency Performance Evaluation Task Force (IPET). It provides a status report on the conduct of the scope of work outlined in Report 1, Performance Evaluation Plan and Interim Status, as well as preliminary results emerging from the analysis. The frequent professional interaction and review comments provided by the ASCE External Review Panel have been a substantial asset to the IPET in the conduct of the analysis and development of the results described in this report.

It is important to stress that this report provides a snapshot of a large multi-disciplinary analysis that is ongoing. Every effort has been made to properly qualify the level of development of the individual activities and the emerging results presented herein. The information is being provided at earliest possible time to allow broad exposure, external evaluation and feedback and application as appropriate. The work remaining is substantial and may result in some modifications and changes to the information presented, as well as substantial new results and findings. The information provided in this report should be considered a working draft and subject to revision prior to the completion and release of the IPET final report.

The key objective of the IPET is to understand the behavior of the New Orleans Hurricane Protection System in response to Hurricane Katrina and assist in the application of that knowledge to the reconstitution of a more resilient and capable system. As such the IPET analysis is geared toward determining why certain sections and structures breached, and using that understanding to both assess the integrity of the remaining portions of the system and to assist in designing more resilient protection measures. IPET is also conducting a risk and reliability assessment of the entire system to aid in understanding the levels of protection that will exist for the future. To do this the IPET Teams have been conducting an integrated set of analyses designed to provide a balanced assessment of the performance of all aspects of the physical system. The IPET is not addressing the issues of organizational and jurisdictional complexities that can impact the effectiveness of the physical system. These issues are being addressed in a separate activity.

This report is not intended as a final expression of the findings or conclusions of the United States Army Corps of Engineers, nor has it been adopted by the Corps as such. Rather, this is a preliminary report summarizing data and interim conclusions compiled to date. It is intended to provide information necessary for the future decisions that will be made regarding the status of the New Orleans Hurricane Protection System. As a preliminary report, this document and the information contained therein are subject to revisions and changes as additional information is obtained.

The architecture of this report is aligned with the five major questions that comprise the IPET mission. Those questions involve 1) The System: documenting the pre-Katrina characteristics of the hurricane protection system (HPS) components and contrasting them to the original design intent, 2) The Storm: understanding the surge and wave environment created by the storm and the forces incident on the levees and floodwalls, 3) The Performance: understanding the performance of the levees and floodwalls and assessing the residual capability of the reconstituted HPS, 4) The Consequences: understanding the resultant flooding (including the role of the pump stations) and the losses due to flooding from Katrina and assessing the extend of flooding and losses if no catastrophic breaching had occurred and 5) The Risk: determining the risk and reliability of the HPS prior to Katrina and after planned repairs and improvements. All of these efforts are underpinned by the development of an accurate geodetic reference datum to ensure that all geospatial aspects of the analyses and results are accurately related.

A number of major tasks are nearly complete, including the Geodetic Vertical and Water Level Datum and the Storm Surge and Wave Analysis. Others, such as the structural performance analysis, have completed a prototype of the final analysis for a component of the overall HPS, providing useful results but relevant to only that portion of the HPS. Still other tasks, such as the consequence analysis and the risk and reliability analysis present samples of their work and examples of the types of products that are being generated, but do not provide results at this time.

The Datum: Because of the complex and variable subsidence in Southeast Louisiana, establishing an accurate vertical reference for measurements has been a constant challenge. By accelerating an effort already underway by the Corps of Engineers and the NOAA National Geodetic Survey, a new Datum was established using GPS technology. Additional surveys were accomplished to accurately determine the elevation of all critical features and structures that comprise the hurricane protection system as well as perishable data such as high water marks resulting from Katrina. These efforts documented that many sections of the levees and floodwalls were substantially below their original design elevations, an effective loss in protection. For example, the structures associated with the Inner Harbor Navigation Canal were originally constructed to an elevation of 15 feet (relative to mean sea level) but are now just over 12 feet, a typical loss of approximately 2.7 feet in elevation over the lifetime of the project.

The System: A major effort has been ongoing to characterize the HPS components to include geotechnical information relevant to their design. This report presents a reach by reach description of the physical characteristics of the hurricane protection structures and their pre-Katrina condition. The condition information includes the actual elevations of the structures relevant to their original design elevations, indicating the impact of subsidence and settling. Contrast of the design and as built conditions are made in the performance analysis section of this report.

The Storm: The characterization of storm surge and waves has two components, a regional modeling effort to determine the time history of surge and wave conditions experienced by the entire HPS and a high resolution modeling effort to create more refined definition of water levels and conditions in the drainage and navigation canals. The regional surge and wave work is nearly completed and provides a clear picture of the hydrodynamic conditions that existed during Katrina. The results presented herein show highly variable wave and surge levels depending on location. Along the South Shore of Lake Pontchartrain, surge levels were slightly below the design levels but significant wave heights were higher than the design assumptions by around one foot. In the Gulf Intra-Coastal Waterway and along the Mississippi River Gulf Outlet, design water levels were exceeded by 1 to 5 feet, but significant wave heights were about equal to the design assumptions. However, wave periods, closely correlated to wave runup, were three times greater for Katrina than the design assumptions. The east facing levees in Plaquemines Parish experienced water levels approximately 6 feet greater than the design criteria, along with wave heights that exceeded design waves by up to 4 feet and wave periods much greater than the design assumptions.

The high resolution hydrodynamic analysis is developing detailed information on the interaction of the surge and waves and structures. For example, Boussinesq simulations at four specific levee transects along the Mississippi River Gulf Outlet (MRGO) provide time histories of combined wave and surge water levels, overtopping rates, and flow velocities along the back and front sides of the levees. The simulations predict continuous overtopping from 0630 to 0900 hr. Work ongoing in the drainage and navigation canals will determine the water and wave conditions in the canals prior to and at the time of the breaches and estimate the hydrographs of canal water flowing into the protected areas as an input to the interior drainage and flooding analysis. This will also provide the static and dynamic forces experienced by the levees and floodwalls.

The Performance: A key objective of the performance analysis is to assess the residual performance of the entire HPS. The strategy for accomplishing this assessment is to understand why breaching occurred in specific locations, augment that with an understanding of why breaching did not occur along reaches with similar characteristics and develop appropriate assessment methodologies to apply to the remainder of the system. This report provides a detailed analysis of the 17th Street Drainage Canal breach, including the most likely failure mechanism and a discussion of the site and structural conditions that led to that failure. Similar analyses are ongoing for the London Avenue, IHNC, GIWW/MRGO and Plaquemines breaches and will be presented in the IPET final report. The analysis of 17th Street breach provides a perspective of the information, analysis approaches and types of results that can be expected from the other analyses.

Our preliminary analysis identified the failure on the 17th Street canal to be initiated by a deflection of the floodwall that allowed application of full hydrostatic pressure vertically along the floodwall/sheet pile. This force coupled with relatively weak shear strengths in the clay layer under the inboard toe of the levee allowed the lateral translation of the levee from the floodwall back along a failure plane in the clay layer. The peat layer above the clay did not initiate the failure. This failure mechanism was not anticipated by the design criteria used. Our early evaluations indicate that in the absence of the observed failure mechanism, the floodwalls would likely have maintained a safety factor greater than one to the design elevation. Additional analysis is ongoing to develop a clear picture of the water environment in the canal at the time of the breach to better understand any role that wave action may have had in the initiation of the deflection of the wall. Lessons learned in this analysis are being used to shape the assessment of other I-wall sections around the HPS.

The Consequences: The development of interior drainage and flooding models has progressed and analysis is underway for all parishes. This report provides sample outputs from the process for Orleans Parish that demonstrate the information, methodologies and the types of results that will be available for the entire study area. The pump station performance analysis is in advanced stages. This report presents a general description of the pump stations for each parish and more in-depth information for the pump stations and their performance in St. Bernard Parish as an example of the types of results that will be available for all parishes. The analysis of losses is also ongoing but no prototype products are yet available to illustrate the expected final products. Some preliminary estimates of direct damages from inundation of structures and content have been assembled for primary areas of four parishes (Orleans, St. Bernard, Jefferson, and Plaquemines) based on availability of GIS grids topography and inundation. Compilation of data for more detailed estimates of commercial and public-sector damages is partially complete with some preliminary information becoming available for costs of repair and restoration of infrastructure such as distribution sub-grids for electrical services. Based on the existing data and its analysis to date, there is no evidence of significant impacts on fish or wildlife associated with levee breaching or the dewatering of the flooded areas of metropolitan New Orleans by pumping on fish, macroinvertebrate, or shellfish populations of Lake Pontchartrain, Mississippi Sound, or the offshore waters of the Northern Gulf of Mexico. However, wetlands within the flood protection system were impacted by high salinity associated with breached/overtopped levies within St. Bernard Parish.

Risk: A risk and reliability model has been developed to allow a system-wide assessment of the risk inherent in the HPS prior to Katrina and flowing planned repairs and upgrades to the HPS. The model is being applied to Orleans East Parish to demonstrate the types of information used, how the model is applied and the results of this type of modeling. The focus of the model is to be able to consider uncertainties in geotechnical conditions and information, structural conditions and performance, types and levels of forces created by storms and the character and path of the storms themselves. The information in this report should be considered as preliminary examples of this methodology, not results to be used for analysis or application. The nature of the risk products and their relationship to consequence analysis is discussed.

II. Introduction

Background

The Interagency Performance Evaluation Task Force, IPET, was initiated by the Chief of Engineers to determine the facts concerning the performance of the New Orleans hurricane protection system (HPS) in response to Hurricane Katrina. IPET has over 150 experts from 50 organizations conducting in-depth analyses that includes understanding the surge and wave levels resulting from the storm, determining the forces experienced by the HPS, understanding the design, as-built and as-maintained character of the HPS, determining the most likely causes and mechanisms for observed behavior (failure and success), characterizing the extent and consequences of flooding to include the influence of the pumping stations, and performing a risk and reliability assessment of the HPS.

> …"*to provide credible and objective scientific and engineering answers to fundamental questions about the performance of the hurricane protection and flood damage reduction system in the New Orleans metropolitan area.*"
> LTG Carl A. Strock, Chief of Engineers, 10 Oct 2005

Fundamentally, the IPET analysis will assist the Corps and other responsible agencies in understanding why various components of the hurricane protection system performed as they did during Katrina, providing input to all of the ongoing efforts to reconstitute the Hurricane Protection System. This includes support to the three main efforts to fully achieve the authorized levels of protection, repair of the areas seriously damaged by Hurricane Katrina, the design and construction efforts to restore the HPS to authorized elevations of protection (one third is estimated to be below authorized levels due to settling and subsidence) and the design and construction for the completion of the previously authorized hurricane protection system (not yet completed because of lack of funds). The goal is to be able to use these lessons learned to reconstitute a more resilient and capable HPS than that which existed prior to Katrina. The extensive information repository, analytical tools and analysis results will also provide a significant new body of knowledge and analytical capability from which the Corps can begin evaluation of alternative approaches to providing higher levels of protection in the future. It is also hoped that the findings of the IPET efforts, coupled with the insights and interpretations of the ASCE External Review Panel and the NRC Committee on New Orleans Regional Hurricane

Protection Projects will contribute to positive changes in engineering practice and water resources policy for the future.

During the conduct of the IPET studies, there has been continuous interaction with the Corps of Engineers entities in New Orleans responsible for the repair and reconstitution of hurricane protection in the New Orleans region. These organizations, Task Force Hope, Task Force Guardian and the New Orleans District, have representatives embedded in the IPET Teams and provide an effective two-way conduit for information and rapid transfer of results and lessons learned. It is imperative that the knowledge gained by the IPET and others be immediately made available to those responsible for repair and reconstruction.

IPET Report 1, Performance Evaluation Plan and Interim Status, published on 10 January, 2006, documented the IPET scope of work and analysis methods that resulted from significant interaction with the individual experts and the collective body of the External Review Panel. ASCE provided their formal review of IPET Report 1 in a letter report to the Chief of Engineers on 20 February 2006, available on the ASCE Web Site. The National Research Council Committee published their comments and review of the IPET activities and Report 1 in a letter report to the Assistant Secretary of the Army for Civil Works on 21 February, 2006, available on the National Academies of Engineering Web Site.

IPET Report 1, available on the IPET Web Site, *https://IPET.wes.army.mil*, also provided a status report of the analysis in the various task comprising the IPET plan with a limited number of example products, mostly related to the initial storm surge and wave modeling. It included significant background information concerning the organization of the IPET activities, the participants and their affiliations, information sources and management and the general approach for accomplishing the scopes of work. The primary reference information in Report 1 will not be duplicated in this report. Some common components will appear in Report 2 if they required update or expansion to provide complete documentation for this effort. This will mostly be in the form of Appendices that provide detail for the discussions in the main body of the report.

Objective and Scope

The objective of Report 2, Performance Evaluation and Interim Results, is to present a synopsis of analyses to date and present the results of those analyses. A secondary objective is to provide at least a full prototype of the analysis that will be achieved for all aspects of the effort to allow the ERP and NRC reviewers a greater opportunity to provide feedback and advice to enhance the ultimate impact and value of the IPET efforts.

This report is structured around the five major questions that comprise the IPET mission. It will for the first time present some significant results of analysis that will form the basis for the findings in the IPET Final Report, Report 3, scheduled for 1 June, 2006. These results will range from the relatively complete products of some aspects of the performance evaluation to prototypes of products for other tasks. The geodetic vertical and water level datum and the storm surge and wave condition analyses are examples of areas where the full scope of the work is nearly complete.

In other areas the analysis is nearly complete for a portion of the scope of work, for example the structural performance analysis of the 17th Street drainage canal breach. This represents a relatively complete picture of the extent and detail of the analyses being conducted for other components of the system and will be the basis for extension of the results to the evaluation of other areas of the HPS with similar characteristics or conditions. While the final report will contain some additional information concerning the 17th Street breach, the results presented in this report are considered validated and credible.

The information for other tasks, for example the risk and reliability analysis, will be prototypes for the final products that are under development. The intent for these areas is to document and describe how these products are being developed and what they will look when published in the final report. In the case of the consequence analysis and the risk and reliability analyses, Orleans East will be used to demonstrate and describe prototype products. The prototype products will be configured with actual data, however, the data and analysis may not be complete enough to make these products suitable for application. The report will be provided to the ASCE External Review Panel on 9-10 March, 2006 in Vicksburg, MS and to the NRC Committee on New Orleans Regional Hurricane Protection Projects on 20 March, 2006 in New Orleans, LA.

Approach

This report, expected to represent the general architecture for the IPET final report, will focus on the answers to the five fundamental questions posed in Report 1 as the primary focus of the IPET activities:

- **Hurricane Protection System:** What were the design criteria for the pre-Katrina hurricane protection system, and did the design, as-built construction, and maintained condition meet these criteria?

- **Storm:** What were the storm surges and waves used as the basis of design, and how do these compare to the storm surges and waves generated by Hurricane Katrina?

- **Performance:** How did the floodwalls, levees, pumping stations, and drainage canals, individually and acting as an integrated system, perform in response to Hurricane Katrina, and why?

- **Consequences:** What have been the societal-related consequences of the Katrina-related damage?

- **Risk:** Following the immediate repairs, what will be the quantifiable risk to New Orleans and vicinity from future hurricanes and tropical storms?

To answer these questions, there has been a considerable effort in developing the baseline information to support the specific analyses that they imply. A significant component of that effort has been the development of a data repository and data management capability to ensure the quality of the data used in the IPET analyses as well as making a comprehensive data and information source available for this and other applications concerning hurricane protection in the New Orleans area. This effort was driven by a data requirements matrix that defined the information critical to the successful completion of the planned scopes of work, the proposed sources of that information and the time schedule for when it was needed. An updated Data Requirements Matrix is provided in Appendix A. The IPET Data Repository was documented in Report 1 and will not be described herein with the exception of the update with regard to its status and general content provided in Appendix B. In addition, Appendix C updates the information concerning the IPET public web site, *https://IPET.wes.army.mil*, the principal mechanism to rapidly distribute IPET information and results to the public.

The first major section of the report deals with the development of a new Geodetic Vertical and Water Level Datum. This represents an acceleration of efforts that were ongoing between the Corps of Engineers and the NOAA National Geodetic Survey. This effort supports all other efforts by providing a modern and validated datum for referencing all measurements, the relative positions of all features and products of the analyses that are sensitive to geo-position. It was an essential because of the complex legacy of multiple reference frameworks and the very significant and variable subsidence that pervades South East Louisiana.

The second major section deals with description of the Hurricane Protection System (HPS). This section focuses on the character of the HPS starting with the definition of the Standard Project Hurricane (SPH), translation of the SPH into authorized levels of protection, design criteria and assumptions for the structures proposed to provide that protection, as-built character flowing construction and the maintained condition of the structures. This section includes a description of the geotechnical information available to and used for the design and construction. This is the first step in understanding and examining the performance of the entire HPS and its status just prior to Hurricane Katrina. To augment this information a chronology of the significant decisions and communications are presented in Appendix C. This report uses the 17th Street Drainage Canal as a first example of this chronology. The final report will include similar information for a broader segment of the HPS.

The third section deals with characterization of Hurricane Katrina. This involves regional and high resolution modeling of the surge and waves generated by the storm to understand the time history of the water levels and static and dynamic forces that impacted the HPS. The regional modeling provided a perspective of the surge and wave environments for all locations around the HPS. The high resolution hydrodynamic modeling was focused on creating a more accurate representation of these water levels and forces in the confined areas of the drainage canals, the Inner Harbor Navigation Canal (IHNC) and the Gulf Intra-coastal Water Way (GIWW). A time history of water conditions and the resultant forces are essential to conducting a credible performance analysis, allowing the level of forces appropriate to be used in evaluation of structure performance based on the established timing of events. The second component documents the establishment of a time line of events, essentially the timing of the breaching and overtopping of the HPS components and flooding of various drainage areas relevant to the timing of the storm. This is an essential input to both the high resolution hydrodynamics work and the structural performance analysis. The time line provides guidelines for when water in the canals would be lost to flooding, impacting water levels and subsequent forces in the canals. It also allows accurate determination of the time history and character of water levels and related forces to which structures were subjected at the time of overtopping or breaching.

The next section documents the structural performance of the HPS. The performance analysis is presented for the 17th Street Drainage Canal breach. While this analysis is still not final, the results to date, specifically the depiction of the failure mechanism for the event, are considered valid and the most likely cause of the breach. This description represents the approach and methods that are being used for understanding the breaching and overtopping events for other parts of the HPS. This section will also provide a status of work under way to analyze other components of the system. Finally, this section will address how the information concerning performance at specific locations is being used to address the assessment of the capacity of other similar reaches or structures within the HPS.

The Consequence section will document the status of efforts to model the flooding resulting from overtopping and breaching and the losses due to that flooding. The flooding analysis includes characterization of the pump station performance from the perspectives of evacuation of water during and after the storm and as a source of water fro flooding via backflow through idle pumping facilities. Prototype products from the pump station analysis and interior drainage analysis are provided as representative of the analysis being accomplished for the entire area of interest. The consequence analysis of losses for input to risk and reliability analysis will use Orleans East as a prototype for developing sample products. The consequence analysis is in the process of determining the likely extent of flooding and losses if there had been no catastrophic breaching of the HPS, the prototype analysis presented her is limited to the actual flooding and losses resulting from Katrina for one polder.

The final section deals with the risk and reliability analysis and will document the methodology developed and present an example application of the methodology for Orleans East. This example is not considered a validated result, simply a representative example of the types of risk products and information that will be available in the final report. This section is also intended to demonstrate the value of the risk approach as a means of evaluating the system-wide performance of the HPS.

III. Geodetic Vertical and Water Level Datum

Summary of Scope and Purpose

The primary focus of this task is to establish a consistent, vertical reference framework model to support IPET performance evaluation activities. This geodetic framework--currently (NAVD88-2004.65)--will allow long-term monitoring of absolute flood/hurricane protection elevations relative to the local water surface reference datum, e.g., local mean sea level, river low water reference planes, etc. Controlling elevations on floodwalls, levees, pump stations, and bridges through the SE Louisiana region were surveyed relative to this framework. The framework additionally provides a consistent reference system for numerical and physical model studies performed in the region. This task assessed the impact of potential reduced flood/hurricane protection resulting from elevation changes (i.e., net land subsidence and sea level rise) throughout the region. The IPET additionally evaluated and compared flood/hurricane structure protection elevations (and older reference datums) at the time of original design/construction with the current elevations (“pre-Katrina”). Quality control field checks on recent aerial and LIDAR mapping will also be performed.

All of this work was accomplished in the field using water level gages (existing and historical), static GPS observations, and conventional topographic surveying methods. Archival data from the New Orleans District, and NOAA (National Geodetic Survey (NGS), and Center for Operational Oceanographic Products and Services (CO-OPS)) were used in these assessments.

The information contained in this Interim Geodetic Vertical and Water Level Datum section shall be considered provisional and subject to correction. Some of the geodetic and topographic survey data used in this assessment has not yet been fully quality assured. Due to time constraints, geodetic and water level datum concepts, assumptions, and estimates have not been adequately reviewed by the interagency team members, nor has an independent external review been conducted. Analysis of geodetic satellite observations and water level datum records obtained during the period November 2005 through mid February 2006 is still in progress. These actions will be completed prior to issuance of the IPET final report.

Background: (Education on Datums)

General Background on Southeastern Louisiana Elevation Datums

Geodetic Datums are vertical datums referenced to local mean sea level from a select set of tide gages, at different locations. In the United States, several vertical adjustments were made between 1900 and 1929. Since 1929, only two official datums exist, with several adjustments made in areas such as Southern Louisiana, where some original and releveling adjustments have been made. These datums make up the National Geodetic Vertical Datum of 1929 (NGVD 29). It was originally called the Sea Level Datum of 1929 (SLD 29) until Congress approved the name change on May 10, 1973. In 1929, the United States Coast and Geodetic Survey (USC&GS) created the SLD 29 (NGVD 29) as the datum with which to adjust all vertical control to, in North America. The 1929 datum is defined by 26 Tide Stations, held fixed to Local Mean Sea Level; 21 tide stations in the United States; and 5 tide stations in Canada. There were several adjustments to the datum, but no change in the definition of the datum until 1991, when the National Oceanic and Atmospheric Administration's (NOAA), National Geodetic Survey (NGS) established the North American Vertical Datum of 1988 (NAVD 88). Adjustments on the datums are noted by the year in parentheses after the datum name, i.e. NGVD29 (19xx) where 19xx is the year the NGVD29 datum was readjusted.

Before defining this datum and understanding the difference between NGVD 29 and NAVD 88, some key definitions of important factors must be explained. For example, the term **Equipotential** is defined as an irregular surface, perpendicular to the force of gravity at every location. This means that a potential gravitational force is the same at all locations along one surface, producing an infinite number of equipotential surfaces surrounding earth; and each of these locations along the surface has its own distinct shape and isn't parallel. A **Geoid** is an equipotential surface which most closely fits local mean sea level. It has problems in that it has variations in its local mean sea level. For example, the local mean sea level in New Orleans is not the same as in Florida. Variations in earth's gravitational field have an impact on the shape of a geoid. Therefore, local mean sea level at one location is not necessarily on the same equipotential surface as the local mean sea level for another location. Due to this difference in local mean sea level and the requirement to hold the 26 tide stations fixed, the network was warped to allow the local mean sea level at tide stations to remain fixed; hence, NGVD 29 is not equipotential.

On the other hand, NAVD 88 is defined by a tidal bench mark at Father Point/Rimouske, an International Great Lakes Datum of 1985 (IGLD 85) water level station at the mouth of the St. Lawrence River, in Quebec, Canada. Its elevation is held fixed in a minimally constrained, least square adjustment, which isn't distorted by constraints of local mean sea level in different areas, as in NGVD 29. Both datums produce orthometric heights or elevations. An orthometric height of a point on earth's surface is the distance from the reference surface (geoid) to the point, measured along the plumb line, normal to the geoid.

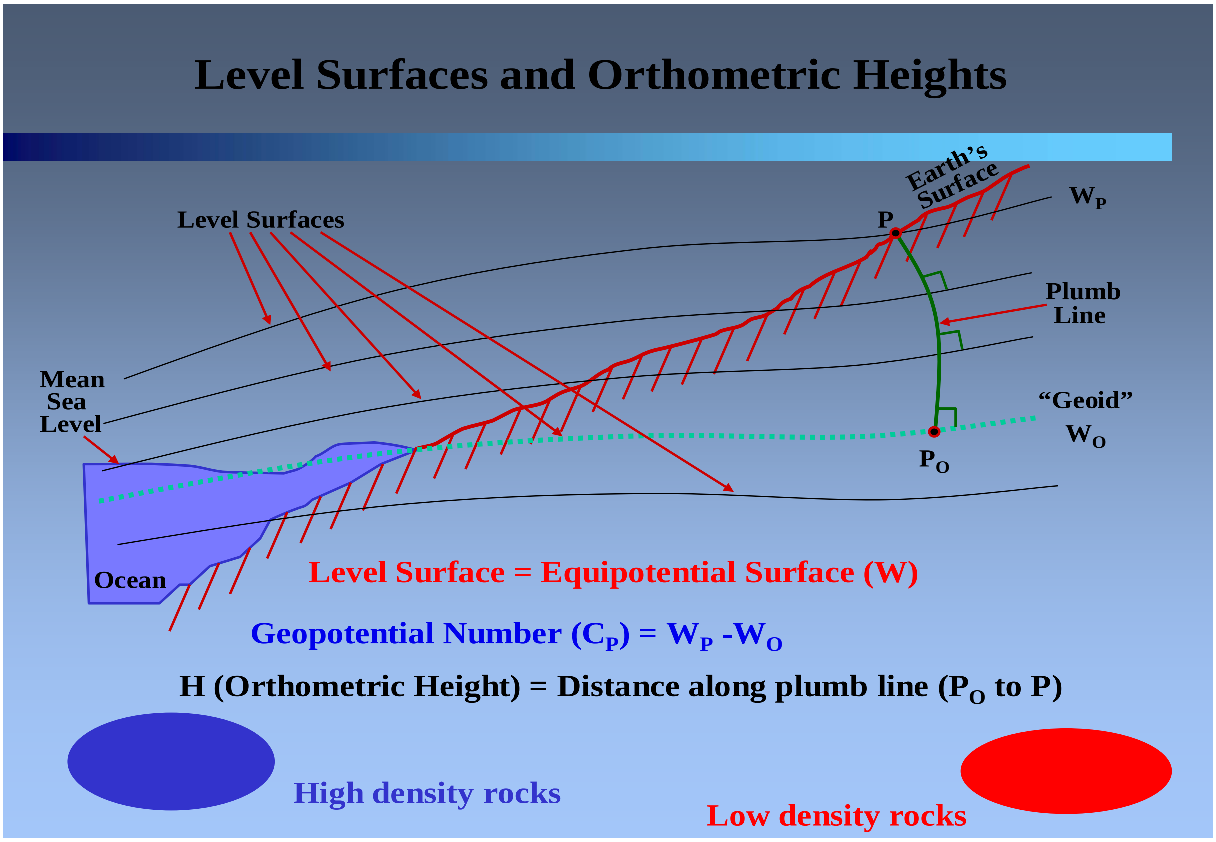


Figure III-1. Equipotential Surface and Orthometric Heights

Figure III-1 Key:

Level surfaces – in order to understand this term, imagine earth standing still; hence, the oceans standing still. There are no effects such as currents, tides, and winds, except for slight undulations created by gravity effects. Those slight undulations equal level surfaces.

Geoid – the level surface relating to today's mean sea level surface. This does not truly coincide with mean sea level because of the non-averaging effects of currents, tides, water temperatures, salinity, weather, solar/lunar cycle, etc. The geoid is a best-fit mean sea level surface.

Equipotential surfaces - add or subtract water and level surface changes, parallel to previous surface. This means creates an infinite number of possible level surfaces. Each equipotential surface has one distinct potential quantity along its surface.

Point on earth's surface - the level surface parallel to the geoid, achieved by adding or subtracting potential. Lines don't appear parallel; they are based on the gravity field and are affected by mass pluses and minuses.

Geopotential number - the numerical difference between two different equipotential surfaces. **W** = potential along a level surface. **CP** = geopotential number at a point.

Plumb line (over exaggerated in drawing) - a curved distance due to effects of direction of gravity, known as deflection of the vertical.

Orthometric height - exactly the distance along this curved plumb line between the geoid and point on the earth's surface. Close approximations can be made, but for accuracy, the gravity needs to be measured along this line, requiring a bored hole, which is impractical.

Illustration III-1. Excerpt from U.S. Army Corp of Engineers (Mean Gulf Level of 1899) Manual

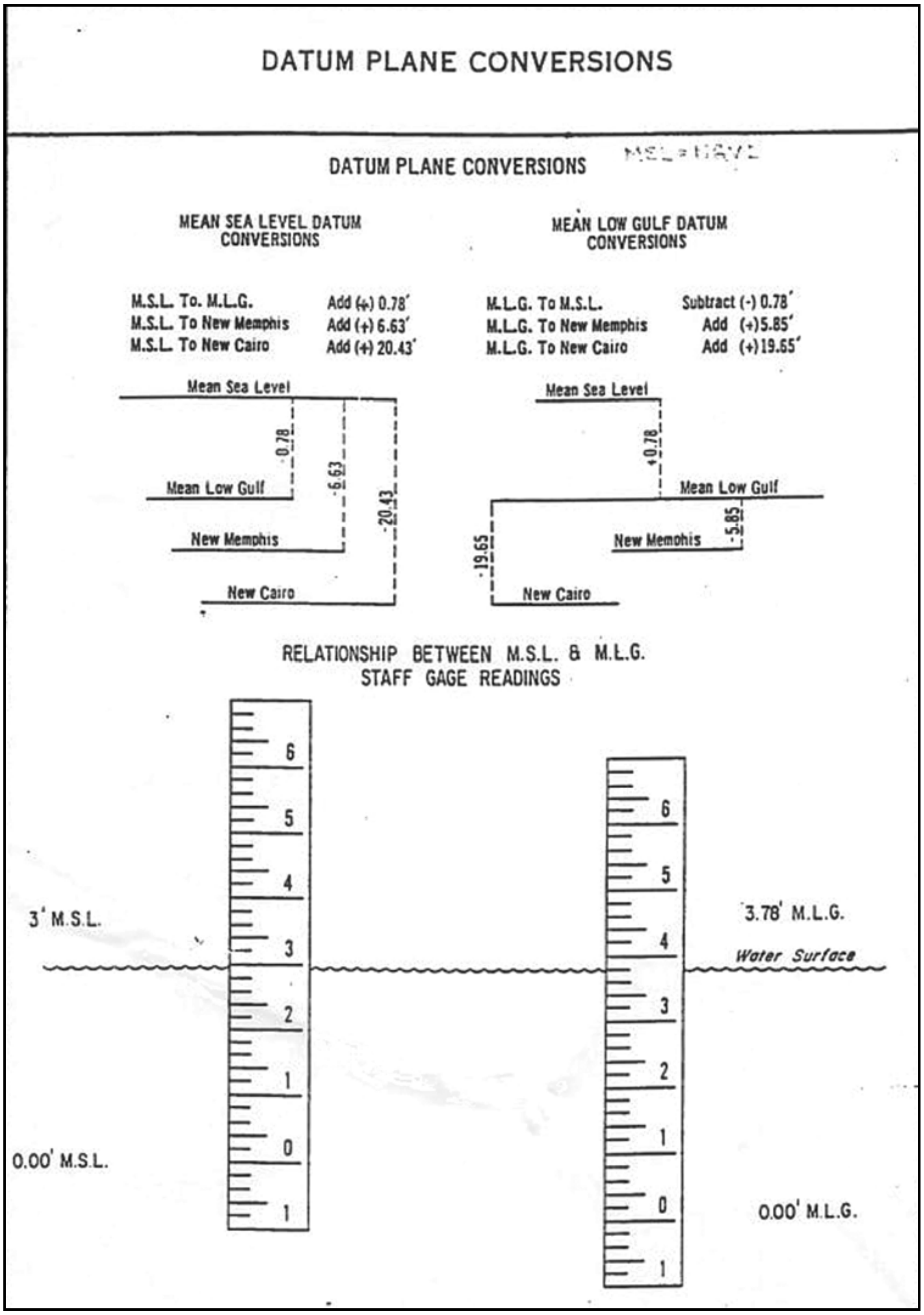
"In 1850, pursuant to an Act of Congress, the Secretary of War directed Mr. Charles Ellet Jr. to make a complete survey of the Ohio and Mississippi Rivers, with a view toward a master plan for flood prevention and navigation. In 1876, before the Mississippi River Commission was formed to coordinate all activities on the river, a survey of the Mississippi was begun in the vicinity of Cairo, Illinois, nicknamed Little Egypt. A temporary datum was adopted at 300 feet below a plane known as the Cairo City Datum of 1871. When the same survey was begun in the vicinity of Memphis in 1877, another temporary datum was adopted at 225 feet below the high water of June 23, 1858 at Memphis without any connection to the lower Delta Survey Datum of 1858. The first connection by precise levels between Memphis and Cairo was completed in 1880. The Mississippi River Commission established a tide gage at Biloxi, Mississippi. In 1882, a final value was adopted for Mean Gulf Level by the Mississippi River Commission based on the mean years of 1882, 1884, 1896, 1897 and 1898. In 1890, re-leveling was started at Fort Adams, Mississippi. The re-leveling ran south to Baton Rouge, Louisiana and north to Cairo, Illinois. In 1910 the level line from Memphis to Cairo was completed.

The U.S. Coast & Geodetic Survey (USC&GS) adopted the Mississippi River Commission value of Mean Gulf Level of 1899 and used it in the general adjustment of 1898, 1903 and 1907. The USC&GS later performed the General Adjustment of 1929, in reference to adjustments and datum relationships. The published elevations of the Mississippi River Commission for level lines between Biloxi and New Orleans and along the Mississippi River are mainly observed elevations based on one tide station, without orthometric corrections applied or corrected for closure. The relationship of Mississippi River Commission Vertical Datums with the Mean Sea Level Datum of 1929 will vary as a function of observational error and as the orthometric height varies. In 1944, the varying difference was noted between Mississippi River Commission Vertical Datum and USC&GS 1929 resulted in the tie-point method being established. However, the tie-point method seems to have faded from use. The Mississippi River Commission Vertical Datums have evolved into merely a number of indices that are transformed by algebraic addition. The true relations between the various Mississippi River Commission Vertical Datums and Mean Sea Level 1929 are now obscured by time and no longer used. The index relationships are as follows:"

Datum	Conversion to Mean Sea Level 1929
Ellet Datum of 1850	unknown
Delta Survey Datum of 1858	0.86
Old Memphis Datum of 1858	-8.13
Old Cairo Datum of 1871	-21.26
New Memphis Datum of 1880	-6.63
Mean Gulf Level Datum (preliminary) 1882	0.318
Mean Gulf Level Datum of 1899	0.00
New Cairo Datum of 1910	-20.434
Mean Low Gulf Level Datum of 1911	-0.78

Figure III-2. Datums and Conversions (all differences are in feet) Reference: Point of Beginning; Surveying Little Egypt by Milton Denny, PLS

Illustration III-2. Visual Chart of Datum Plane Conversions



Definitions:

The Cairo Datum (also referred to as New Cairo Datum) **-** based on a benchmark at a Corps of Engineers facility in Cairo, Illinois. Benchmark originally 20.434 feet above LMSL, so one had to always subtract 20.434 from each Cairo Datum number to equate it to LMSL.

Tidal Datums **-** used to establish local tidal phase averages as reference levels from which to reckon height or depth observations. To accurately compute, observations must be taken at a tide gage that has been collecting data for a period of over a 19 year National Tidal Datum Epoch. This time period allows inclusion of all variations in the path of the moon about the sun. Tidal datums are locally derived and should not be extended into areas which have differing hydrographic characteristics, without substantiating measurements. The most commonly used tidal datums are:

→ Mean Higher High Water (MHHW) **-** the average height of higher high waters at a tide gage, covering a 19-year period;

→ Mean High Water (MHW) **-** the average height of all high waters at a place, covering a 19-year period;

→ Mean Tide Level (MTL), a plane often confused with LMSL that lies close to LMSL. MTL is the midpoint plane exactly between the average of MHW and MLW at a tide station. The difference is MTL does not include all the tide levels (i.e. MHHW and MLLW) unless the tide at a particular location is diurnal;

→ Local Mean Sea Level (LMSL), commonly referred to as Mean Sea Level (MSL) **-** the average height of the surface of the sea at a tide station for all stages of the tide, covering a 19-year period which is usually determined from hourly height readings measured from a fixed and predetermined reference level; and

→ Mean Lower Low Water (MLLW) **-** the average height of the lower low waters at a tide gage over a 19-year period.

Subsidence and Louisiana Surface Levels

Subsidence is the lowering or sinking of earth's surface. In Louisiana, subsidence is occurring at a rate of up to one inch, every three years, in some areas; especially in Southern Louisiana. Until the October 2005 release (by NOAA's National Geodetic Survey) of 85 benchmarks located in southern Louisiana, which showed heights (elevations) accurate to between 2 and 5 centimeters, surveyors, engineers, and the U.S. Army Corp of Engineers in New Orleans used vertical heights that had not been calibrated nor checked for several years; hence, inaccurate. Some of the 85 stations, which are part of the NAVD 88 (2004.65) epoch, showed as much as a one foot subsidence, or change, since the original published heights, covering a 10-year period. The average rate of subsidence across the area was about 0.6 feet subsidence/change, over a 10 year period.

This indicates that heights (elevations) published in the 60's, 70's, 80's, and early 90's may have changed even more. Southern Louisiana is currently undergoing the largest loss of land in the nation, due to subsidence and erosion; especially in the New Orleans area.

NOAA's objective is to improve upon the current vertical reference system, the NAVD 88 (2004.65) epoch, which consistently evaluates previously constructed, and proposed flood control and hurricane protection structures in New Orleans and Southeast Louisiana.

During a recent conference, Coastal Zone '05, officials from NOAA announced the new elevations for Louisiana [NAVD 88 (2004.65)], to improve the accuracy of the state's survey benchmarks and insure their accuracy for longer periods than in the past. "Using new technology available, such as the Global Positioning System and NOAA's Continuously Operating Reference Stations, will allow us (NOAA) to provide accurate elevation reference points in an efficient and timely manner," said Richard Spinrad, Ph.D., Assistant Administrator for NOAA's National Ocean Service.

"These new heights are more considerably accurate than what we have been able to measure previously," said Charlie Challstrom, former director of NOAA's National Geodetic Survey. "There is much work to be done, including providing tools and educating users on how to utilize the new information for future projects." It is critical that users of elevation data apply it in accordance with new approaches being developed, and work with NOAA and the Louisiana State University's, Spatial Reference Center (LSRC) to improve the geospatial reference system in Louisiana. While there will be fewer specific benchmarks maintained, the overall accuracy of the heights will be maintained for longer periods.

NOAA does not predict the rates of subsidence, nor attempt to determine its causes. We supply data used by the U.S. Geological Survey, U.S. Army Corps of Engineers, FEMA, state agencies, academia, emergency planners, engineers, surveyors, environmental restoration efforts, and others, to determine those rates. Furthermore, NOAA plans to maintain and update the NAVD 88 (2004.65) network of stations.

General background on the Low Water Reference Plane (LWRP)

The LWRP is the statistical elevation profile of the river, based on gage readings for times when the river discharge was exceeded 97% of the time or record during that twenty year period of observation. We have two known "epochs" of Miss. River LWRP the 1974 and the "1993" that are active. The 1974 LWRP Mile 313.7 to 242.0 is based on [the] 97% discharge duration of Tarbert Landing (1954 - 1973) and corresponding stages; mile 242.0 to Head of Passes is based on the Mean of 40 years (1891-1930) at Regular (MRC) gages and adjusted from low water information obtained Sept. 1931 and Nov. 1933.

New Orleans District updated the LWRP in the early 1990's and may have used a different statistical construct.

Background and Information on the Mean Low Gulf (MLG), as it interfaces with low water reference plane, will be provided in the final report.

Data Collection and Processing for Tidal/Datum Relationships

Development of Phase 1 Survey Data Collection Network Design

In order to develop a relationship between the local mean sea level and the current geodetic vertical network across the project area, measurements had to be made between tidal stations and the geodetic vertical network. This data collection effort was referred to as the Phase 1 survey. The Phase 1 survey involved GPS static survey measurements of existing and historical NOAA and USACE water level and tidal stations measured relative to NAVD 88_2004_65 benchmarks. Because of time constraints, the idea to use existing and historical gage information was chosen over installing gages over greater New Orleans for a period of one year. Conventional leveling, using precise digital leveling instruments, was used to measure differences between a minimum of three tidal benchmarks at each tidal station location to check for consistency as required by NOAA CO-OPS.

Static Survey Phases

Three phases of GPS surveys were planned as the water receded and survey crews moved southeast and northeast along partially closed roads. Two phases were planned from a meeting in November 2005 and a third phase was added and then abandoned as the crews found tide gauge sites underwater and monuments destroyed. The Phases were called Phase 1A, Phase 1B, and Phase 1C.

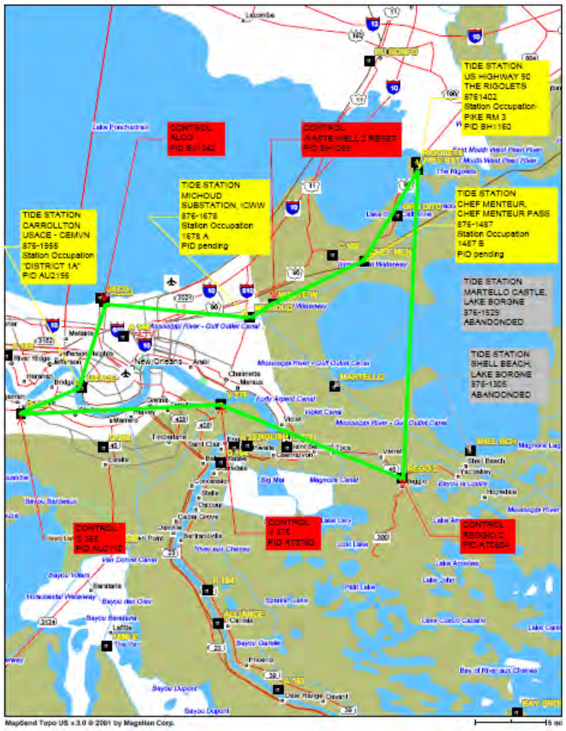


Figure III-3. Phase 1A Stations

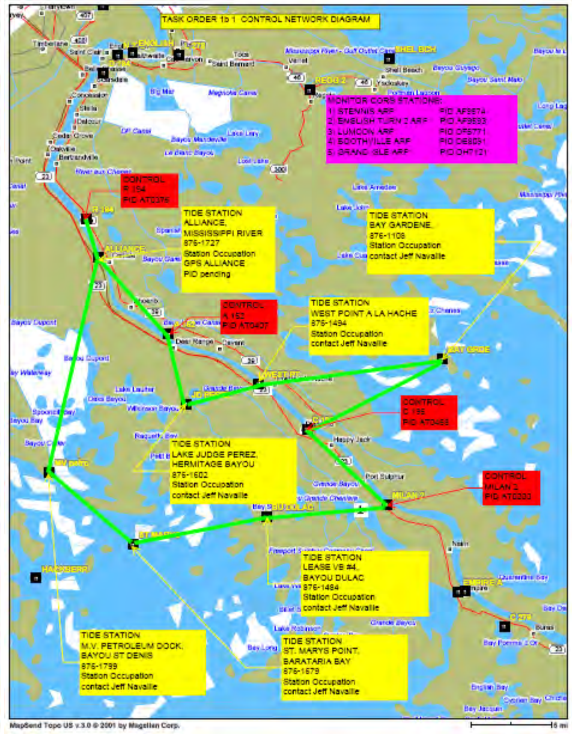


Figure III-4. Phase 1B Stations

The Phase 1A GPS static survey design unfolded as government personnel and contractors slowly reentered restricted areas just outside the City of New Orleans. Originally six tide stations were to be measured for the purpose of tying the tidal datums to the geodetic datum in Phase 1A. Three of the six tide stations were found to be totally destroyed during a reconnaissance survey to recover monuments and take photographs of these tide station sites. On 9 November 2005, team members visited the site of Tide Station 8761426 Greens Ditch, Lake St. Catherine to recover tidal bench marks. They reported the entire area had been graded with no sign of "All in the Family Camp" (a reference sited in the benchmark descriptions) or any of the tidal benchmarks. They reported no references existed to measure distances in order to recover the monuments. On 9 December 2005, personnel from 3001 visited Tide Station 8761529, Martello Castle, Lake Borgne. A photograph taken with a camera direction to the northeast shows the complete destruction of the castle. The three tidal benchmarks on this structure are considered destroyed. To the southeast of the castle on the marsh shoreline, four tidal benchmarks A, B, C and D were monumented in 1982; however, the personnel indicated that since 1982 those marks would now be 30 or 40 feet off the shoreline underwater. On 9 December 2005, personnel from 3001 visited Tide Station 8761305, Shell Beach, Lake Borgne. Three photographs taken showed the total destruction of the tide station that once recorded water level measurements from a large concrete quay built in World War II. One photograph to the west with the Fort Beauregard ruins in the background depicts the shoreline difference since 1982. The tidal benchmark in the foreground assumed to be 1305F or 1305G is bent and out in the water. This shoreline retreated at least 20 to 30 feet. One tide station at the New Orleans

District Office was added to the survey. NOAA named the station Carrollton. The USACE tide station was not measured to NOAA standards; however, useful information for this report will be tabulated including this tide station on the Mississippi River.

Phase 1B changed four or five times during the reconnaissance. Access to most of the tide stations in this area are by boat. The final GPS network for this static survey included NOAA Tide Stations: 8761602, Lake Judge Perez, Hermitage Bayou and 8761799, M.V. Petroleum Dock, Bayou St. Denis. Also included was a USACE tide station located at West Point, A La Hache on the Mississippi River. Seven benchmarks were recovered at 8761602, Lake Judge Perez, Hermitage Bayou (**Lake Judge Perez)**, one of which was damaged. This site was used in phase 1b1. The tide station USACE West Point, A La Hache, Mississippi River (**Pointe a la Hache)** was visited and incorporated into the Phase 1b1 network. At the tidal station 8761799, M.V. Petroleum Dock, Bayou St. Denis (**MV Petro**) 4 of the 5 monuments were recovered. Several tidal stations that were proposed to be included in the phase 1B survey were reported destroyed. Only two primary NOAA benchmarks at USACE Alliance, Mississippi River (Alliance) were not recovered. Three NGS vertical rod marks were recovered along the highway. Instead of using this site, the tidal station at **EMPIRE** was used since more monumentation called for on the description sheet were recovered at that site. It was also in close proximity to one of the 2004.65 marks, so only level work needed to be preformed here. The tidal station 8761679 St. Mary's, Barataria Bay (**St. Mary's Point**) was not used since no monuments were recovered at this site as it is now open water. The tidal station 8761108 Bay Gardene, Gulf of Mexico, was not used since insufficient monuments were recovered for it to be considered for use in the Phase 1b scheme. One of the monuments was found bent over, another was in about three feet of water, and another was believed to be under a pile of shell material. Not sure about the others. Pictures were taken to document the site.

The initial Phase 1C survey was removed due to time constraints, access to tidal benchmarks, and speculation, based on aerial photography, that the marks would not be found in useable condition. Another task order called Phase 1C was executed 28 February 2006 to identity, if possible with GPS, a 0.1-foot difference noticed in the Phase 1A measurements at Tide Station 8761927 U.S.C.G, New Canal relative to Tide stations 8761487, Chef Menteur and 8761402 , The Rigolets east of New Orleans. At a minimum, the task order will check the vertical control back to the primary tide gauge for Lake Pontchartrain and Lake Borgne, which is 8747437, Bay Waveland Yacht Club, Bay St. Louis, Mississippi.

The initial design of the GPS networks was based on the location and type of vehicle access to the tide stations. A National Geodetic Survey requirement for at least four NAVD 88_2004_65 geodetic marks surrounding the tide stations was carried out to ensure no recent benchmark settlements were placing unwanted bias into the GPS network measurements. A USACE civil engineer at the Engineer Research and Development Center, Alexandria, Virginia developed the preliminary GPS networks that could be field modified by a survey field coordinator from Jacksonville District on the ground in New Orleans. The network GPS diagrams were then sent to the National Geodetic Survey (NGS) for pre-approval into the National Spatial Reference System database of geodetic information. The networks were also checked to meet NGS GPS derived height specifications for data collection under the NGS two centimeter standard.

The Phase 1A GPS survey was exclusively land vehicle access after Martello Castle and Shell Beach tide stations were found destroyed. These two sites could only be accessed by water or air vehicles. The Phase 1B GPS survey network went through numerous changes as many sites were either destroyed or found under-water. A few USACE water level gauge sites on the Mississippi River were being added and removed as well as field conditions changed.

Contractor Data Collection and Processing Procedures

All of the data collection for this task was accomplished through a St. Louis District task order to 3001 Inc. who performed the data field data collection and processing.

The GPS data was collected using four Trimble 4000 SSE receivers, two Trimble 4000 SSI receivers, one Trimble 4700 receiver, six fixed-height tripods, six Trimble Compact L1/L2 antennas with ground plane and one Trimble microcentered L1/L2 antenna with ground plane. The differential leveling was performed with a Leica DNA 03 differential level.

GPS Data Collection and Processing. The static GPS network for this part of the project was designed to provide measurements from newly published NGS control points with NAVD88 2004.65 elevations to existing and historical tide stations. The GPS field procedures followed the NGS Bluebook specifications, as defined by NOAA 2005 - Guidelines for establishing GPS derived orthometric heights (standards: 2cm and 5cm) as well as the guidelines established in EM 1110-1-1003. The GPS network design was approved by the NGS Representative on the IPET project. The network was designed to include enough existing local control to establish elevations and positions on the temporary benchmarks which were surveyed as part of the network. The network was also tied into Continuously Operating Reference Stations (CORS). The datasheets for the CORS and the NGS monuments used can be found in the survey report supplied by 3001 Inc. (IPET-Survey Report.pdf) posted on the IPET Data Repository. The network was designed with multiple, simultaneous occupations of points in order to provide redundant vectors and loop closures.

The baselines were processed using Trimble Geomatic Office's baseline processing module, WAVE (*Weighted Ambiguity Vector Estimator*). Ionosphere-free fixed solutions were found to provide the best results. Preliminary blunder detections were undertaken using "Redundant Vectors" and Global Network Closures and any extremely large errors were eliminated.

The data are then processed using a minimally constrained geodetic control network to test the network internally, without external constraints, and produce a statistical summary. The statistics from this process are required to be within the tolerance outlined in the Geometric Geodetic Accuracy Standards and Specifications for using GPS Relative Positioning Techniques, published by the FGCC. These tolerances are represented as ellipsoids showing the margin of error value on a graph of the theoretical points, covariance values that indicate the degree of error of the vectors relative to the other vectors in the network, and a chi-squared test that compares the predicted variance determined through a least-squares analysis to the observed variance. The summary is evaluated to eliminate vectors that are outside of the error tolerances to be replaced with redundant vectors that are within the tolerances until all tolerances are met.

The quality of the existing horizontal controls is assessed before undertaking the constrained adjustment. Geodetic inverses between the control monuments were compared with the geodetic inverses derived from the minimally constrained least square adjustment results. This distance analysis is especially useful, since it provides a datum invariant means of comparison. Once the minimally constrained network satisfies the requirements of the above tests, control points in the network are selected with an optimum spatial relationship to fully constrain the network to known control points, and have their provided values entered as the position for those points and the network re-adjusted. The fully constrained positions are shown on the next two pages, and they are also in Appendix I and Appendix J. The same statistical tests are rerun on the adjusted network, as well as visually comparing adjusted values of control points to provided values of control points not used as constraints. Again, the summary is evaluated to identify vectors outside of the tolerances and constraining points reselected to obtain the best fit to the geoid where all vectors are within the prescribed tolerances.

The adjustment results show that the a posteriori variance factor of the network was close to 1.0, as should be desired, and passed the $\chi 2$ test. None of the residual components in the network were flagged for possible rejection under the τ-max test at the 0.05 level of significance. The relative confidence ellipses reveal that the horizontal positional accuracy between all directly connected pairs of stations in the network were better than (1:100,000) at the 95% level of confidence.

Leveling Procedures Used. Leveling to tidal marks in the marsh area were performed to second order, class II modified guidelines that were developed by USACE and NOAA NGS. These guidelines will be published in an appendix for the IPET final report. All leveling that was done on land, that could be driven to, followed the second order class I leveling procedures as described by the Specification and Standards of Accuracy established by the Federal Geodetic Control Subcommittee (FGCS).

USACE processing of GPS data and network adjustments

Preliminary processing of the GPS data collected for phase 1a and 1b was performed by ERDC-TEC and USACE SAJ using Trimble Geomatics Office and GRAFNAV software respectively. The preliminary results were used in the computation of the initial calculations for the local mean sea level values. Additional details to be provided in the final report.

NGS validating of Blue Booking/Publishing of phase 1 survey points

All of the GPS and leveling data will be processed and adjusted to NGS Blue Booking standards for publishing control to provide the final NAVD 88 2004.65 elevations for each tidal station observed in the phase 1 survey. This final processing is scheduled to be completed in late March 2006.

Processing of LMSL values & relationship between NAVD88 2004.65

Once the Phase 1 static surveys were performed, processed, and adjusted, the preliminary relationship between the current LMSL and the NAVD88 2004.65 datum adjustment at the various tide stations were computed by NOAA CO-OPS and USACE ERDC-TEC. The Blue Booking / Publishing of the GPS and level data in March 2006 will provide final values for publishing of the LMSL and NAVD88 2004.65 relationship. Methodology used by ERDC-TEC and NOAA CO-OPS will be explained in detail in the final report.

Data Analysis and Impacts

Evaluation of Designed and Constructed Elevations on Flood Control & Hurricane Protection Structures

Purpose. This Section reviews the various datums and elevations used in the design and construction of selected flood control and hurricane protection structures in the New Orleans area. An estimate is made of the originally constructed flood protection elevations relative to the local water surface and geodetic datums then used as construction references. Pre-Katrina flood protection elevations are estimated relative to the current local mean water surface and the latest geodetic reference scheme, based on topographic and geodetic surveys performed after the hurricane. Emphasis is placed on assessing elevations relative to the local mean water surface since hydraulic analyses and flood protection elevations were computed based on this surface. The focus is primarily on floodwall projects in Orleans Parish were surge elevations were near the design elevation of the structures.

Methodology. Originally constructed elevations were estimated based on a review of design memorandums and contract documents associated with a project. Archive geodetic control data was obtained from the US Coast & Geodetic Survey (USC&GS)—now the NOAA-National Geodetic Survey (NOAA NGS). Water level information was obtained from the NOAA/National Ocean Survey (NOS) Center for Operational Oceanographic Products and Services (CO-OPS). An evaluation of pre-Katrina (August 2005) elevations was based on post-Katrina geodetic and topographic surveys performed by New Orleans District, Task Force Guardian, and IPET survey crews.

Geodetic Datum and Tidal Epoch Elevations. As outlined in the Background, elevations throughout the IPET study area are referenced to a consistent geodetic datum—NAVD88 (2004.65). In order to relate this geodetic reference datum to the local water surface, long-term observations from water level gage data needs to be analyzed. The requirement to reference geodetic elevations to a water surface elevation is clearly outlined in Section II-5-4 (Water Surface Elevation Datums) of the Coastal Engineering Manual (EM 1110-2-1100):

> *Water level and its change with respect to time have to be measured relative to some specified elevation or datum in order to have a physical significance. In the fields of coastal engineering and oceanography this datum represents a critical design parameter because reported water levels provide an indication of minimum navigational depths or maximum surface elevations at which protective levees or berms are overtopped. It is therefore necessary that coastal datums represent some reference point which is universally understood and meaningful, both onshore and offshore. Ideally, two criteria should be expected of a datum: 1) that it provides local depth of water information, and 2) that it is fixed regardless of location such that elevations at different locations can be compared. These two criteria are not necessarily compatible.*

The two criteria expected of a datum are important concepts—especially the statement that they are "not necessarily compatible." This is exactly the case in the New Orleans area. The local depth of water information (e.g., MSL) cannot be simply correlated at different locations with a geodetic datum, such as NAVD88 (2004.65). Although geodetic reference datums are useful for providing consistent surveying, modeling, and subsidence analysis over a region, they do not provide a direct relationship to local water surface elevations that are the basis for flood protection elevations. Where this water surface is not constant (e.g., in tidal areas or rivers), a dense gage network is needed to model this water surface (MSL) relative to the geodetic reference datum (NAVD88 (2004.65)).

USACE EM 1110-2-1003 (Hydrographic Surveying) notes the importance of obtaining updated water level reference datums and tidal epochs for dredging navigation projects:

> *All USACE project reference datums, including those currently believed to be on MLLW, must be checked to ensure that they are properly referred to the latest tidal epoch, and that variations in secular sea level, local reference gage or benchmark subsidence/uplift, and other long-term physical phenomena are properly accounted for. In addition, projects should be reviewed to ensure that tidal phase and range characteristics are properly modeled and corrected during dredging, surveying, and other marine construction activity, and that specified project clearances above grade properly compensate for any tidal range variances. Depending on the age and technical adequacy of the existing MLLW reference (relative to NOS MLLW), significant differences could be encountered. Such differences may dictate changes in channels currently maintained. Future NOS tidal epoch revisions will also change*

the project reference planes. In many projects, existing NOS tidal records can be used ... tidal observations and/or comparisons will be necessary for projects in areas not monitored by NOS or in cases where no recent or reliable observations are available.

Other Corps of Engineers guidance documents emphasize the need to obtain accurate water surface profiles for use in design and construction. These include EM 1110-2-1416 (River Hydraulics), EM 1110-2-1607 (Tidal Hydraulics), EM 1110-2-1913 (Design & Construction of Levees), and EM 1110-2-1614 (Design of Coastal Revetments, Seawalls, and Bulkheads). The Hydraulic Engineering Center (HEC) Research Document No. 26 "Accuracy of Computed Water Surface Profiles" (1986) states in its Introduction that:

"*Water surface profiles are computed for a variety of technical uses ... flood insurance studies, flood hazard mitigation investigations, drainage crossing analysis, and other similar design needs. The accuracy of the resulting computed profiles has profound implications. In the case of flood insurance studies, the computed profile is the determining factor in the acceptability of parcels of land for development. For flood control projects, the water surface elevation is important in planning and design of project features and in determining the economic feasibility of proposed solutions ... the relationship between mapping accuracy and resultant computed profile accuracy is therefore of major interest to engineers responsible for providing cost-effective technical analysis.*"

In analyzing pre- and post-Katrina levee/floodwall elevations, geodetic elevations on either NAVD29 or NAVD88 (2004.65) are adjusted to the local water level datum (e.g., sea level) published by the NOAA Center for Operational Oceanographic Products and Services (CO-OPS). The latest time period (National Tidal Datum Epoch) available is the 19-year period 1983-2001, which was released by CO-OPS in 2003. Nearly all of the floodwalls in the study area were designed and constructed during the previous tidal epoch (1960-1978); however there is no indication in design memorandums or contract documents of this, or previous, tidal epoch. The difference between the 1960-1978 and 1983-2001 epochs at the New Canal gage in Lake Pontchartrain is 0.15 ft, as shown in Figure III-5 below. In general, the MSL epoch change in the region averages about 0.2 ft.

In a high subsidence area such as New Orleans, the apparent sea level increase is significant. This means that an average mean sea level computed over a 19-year period may not represent the latest sea level condition, and related flood protection levels. In high-subsidence areas, NOAA has adopted alternate procedures for computing accepted tidal datums using the last several years of sea level data rather than the 19- year tidal epoch—typically the latest 5-year epoch. Reference NOAA Special Publication NOS CO-OPS 1 (Tidal Datums and Their Applications) and NOAA Special Publication NOS CO-OPS 2 (Computational Techniques for Tidal Datums Handbook).

A 5-year tidal epoch (e.g., 2001-2005) has not yet been developed for this Interim Report. Therefore, references to Mean Sea Level relate to an older 1960-1978 or 1983-2001 epochs. Given the historic subsidence occurring in this area, any conversion from the NAVD88 (2004.65) geodetic datum to an older MSL epoch could be underestimated by 0.1 to 0.3 ft or more.

Typical Geodetic and Water Level Datums used in New Orleans Area Floodwall Construction. The following graphic illustrates the various geodetic and water level datums existent over the years on a 1931 benchmark near the 17th Street Canal on Lake Pontchartrain. This graphic is typical of benchmarks throughout this high subsidence region. It shows that significant elevation differences relative to MSL can result depending on which NGVD29/NAVD88 datum or adjustment is selected. This is especially critical in a high subsidence area where using an outdated or superseded datum to construct a flood protection structure can result in a lower elevation than that intended in the design. Likewise, hydrologic or hydraulic models using terrain data based on disparate datums can have adverse computational impacts.

Water level data is based on direct vertical control connections between Benchmark ALCO and a NOAA National Water Level Observation Network (NWLON) gage (USCG New Canal) located in the same area. Published water level data (and reference datums) for this gage is based on data obtained between October 1983 and September 1992, and adjusted by NOAA for subsequent epoch changes. In November 2005, NOAA reinstalled a gage at this site and data collected from that time will be used to evaluate later epoch references.

A similar evaluation can be made at other NWLON gage sites in the New Orleans area—both at historic sites and at newly established sites.

(Note that Benchmark ALCO was not directly referenced in contract plans for any floodwall construction on the 17th Street Outfall Canal).

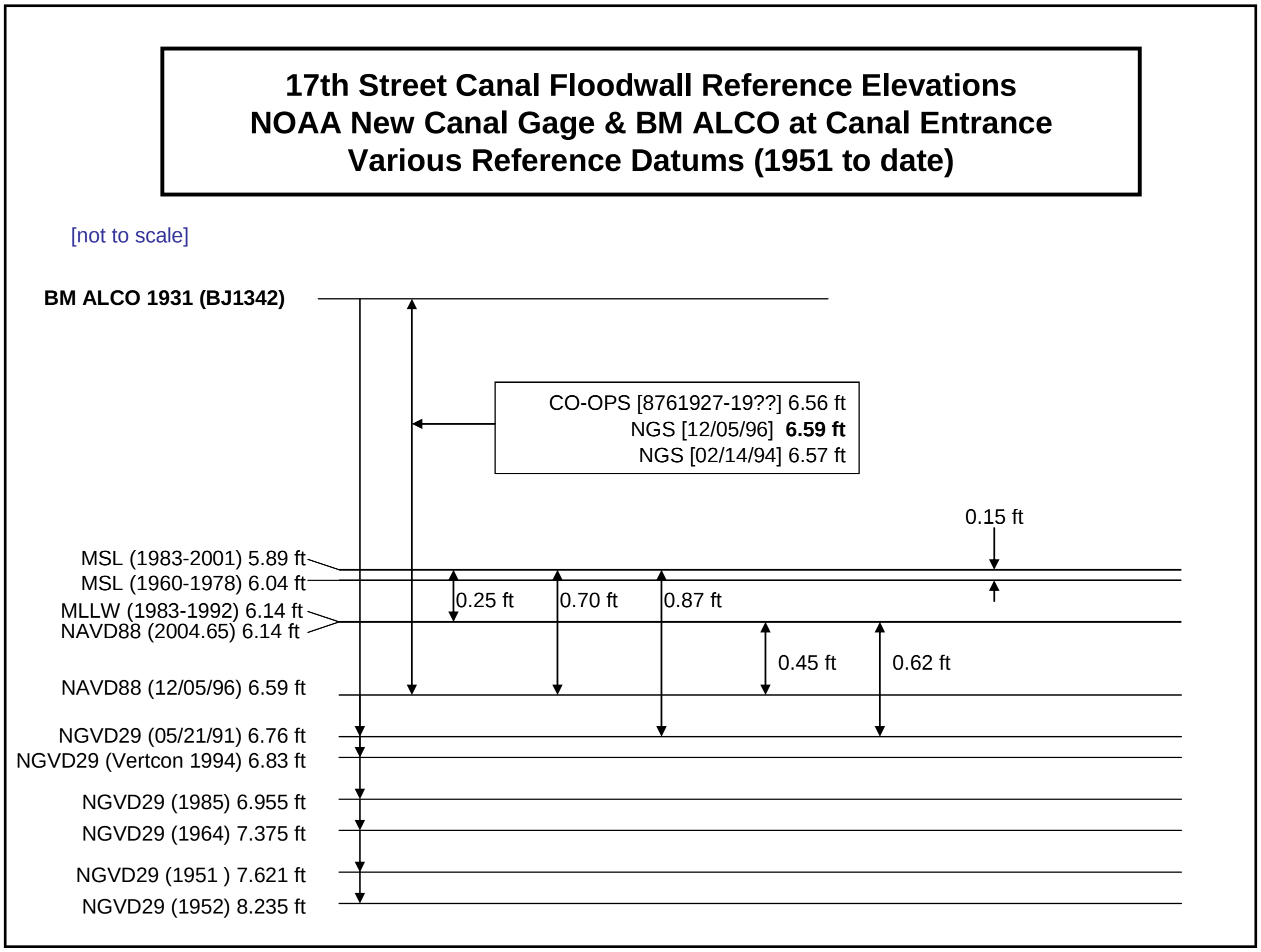


Figure III-5. Datum Relationships at Benchmark ALCO and NOAA New Canal Gage (1951 to date) (Source: NOAA CO-OPS and NOAA NGS (USC&GS))

The above figure does not show the original presumed convergence (or equivalency) of MSL and NGVD29, perhaps back in the early 1930s. Although NAVD29 (and previous adjustments) was originally based (or adjusted) to a "sea level" datum, it is not absolutely certain that NGVD29 and MSL converged at Lake Pontchartrain in the 1930s. (See Background discussion on sea level datums connected from Biloxi, MS).

Especially note that the 0.25 ft difference shown for the current NAVD88 (2004.65) datum to MSL is relative to an older, long-term tidal epoch (1983-2001). This 0.25 ft difference is used as a datum conversion for projects along this portion of Lake Pontchartrain. The datum conversion in the Inner Harbor Navigation Canal (IHNC) has not been fully determined by NOAA CO-OPS as of this Interim Report. It is estimated at 0.2 ft based on interpolations from the nearest NWLON gages. Updated conversions for the IHNC, GIWW, and MRGO will be contained in the Final Report. These updated conversion values will be based on a shorter-term, more recent epoch. At the New Canal gage, the conversion is likely to be larger than the current 0.25 ft value if there has been an "apparent sea level rise" since the 1983-2001 epoch.

1. Orleans Avenue Outfall Canal Construction Reference Datums. The following construction drawings and Design Memorandums were reviewed as part of this assessment:

- DACW29-93-C-0077: Orleans Avenue Canal—Flood Protection Improvement Project—Phase II-D (West Side: B/L Sta. 2+39.00 to Sta. 29+07.50)
- DACW29-97-C-0029: Orleans Avenue Outfall Canal—Parallel Protection-Phase II-A—East Side Floodwall (B/L Sta. 3+60.00 to Sta. 90+26.33)
- DACW29-95-B-0035: Orleans Avenue Outfall Canal—Parallel Protection-Phase II-C—West Side Floodwall (B/L Sta. 21+34.52 to Sta. 63+66.22)
- DACW29-99-C-0025: Filmore and Harrison Avenue Bridges—Phase I-C
- DACW29-00-B-0094: Robert E. Lee Boulevard Bridge—Phase I-B
- GDM No. 19—Orleans Avenue Outfall Canal (Volumes I, II, & III)—1988
- DM 01 Part III Hydrology and Hydraulic Analysis—Lake Pontchartrain & Vicinity-Lakeshore (Sep 1968)

Design Elevation Parameters. Parallel protection elevations are shown in GDM No. 19 and on various contract plans. GDM No. 19 (Vol I) notes that the SPH design stillwater surface elevation of Lake Pontchartrain at 11.5 ft NGVD. This base elevation was used in subsequent HEC-2 models to compute required floodwall elevation on each side of the canal and at the bridges. The design stillwater elevations in the canal at the Filmore Ave. Bridge is 12.10 ft NGVD, and 12.30 ft NGVD at the Harrison Avenue Bridge (DACW29-99-C-0025). The design canal stillwater elevation at the R.E. Lee Bridge was 11.90 ft NGVD (DACW29-00-B-0094). In these hydraulic analysis models, the stillwater elevation relative to NGVD (i.e., NGVD29) was generally assumed to be MSL. A standard freeboard (2 ft typical) and settlement (0.5 ft typical) was added to these stillwater heights to arrive at a design protection elevation referenced to NGVD. Typical flood protection elevations in the canal ranged from 14.0 to 14.9 ft. (DM 01 Part II noted a USACE recommendation for a 3-ft freeboard allowance vice 2 feet previously authorized—this recommendation was rejected).

Various contract plans indicate a “normal water surface” or “normal water level” elevation of 1.0 ft NGVD in the canal. The source of this apparent superelevation is not noted, nor is there any indication that this value was incorporated into the hydraulic analyses used in determining floodwall heights. (This is based on discussions with MVN personnel who ran these original hydraulic models). The 1.0 ft canal superelevation is believed to have been taken from pump station hydrograph records, or from gage records on Lake Pontchartrain or on the IHNC. Although a “NGVD” datum is noted, the year or adjustment epoch is not shown. The superelevation does roughly correlate with the approximate 0.9 ft amount that MSL elevation is above NGVD29 at

Benchmark ALCO—see Figure III-5. A typical section showing the normal canal water elevation is shown in the figure below, taken from DACW29-95-B-0035.

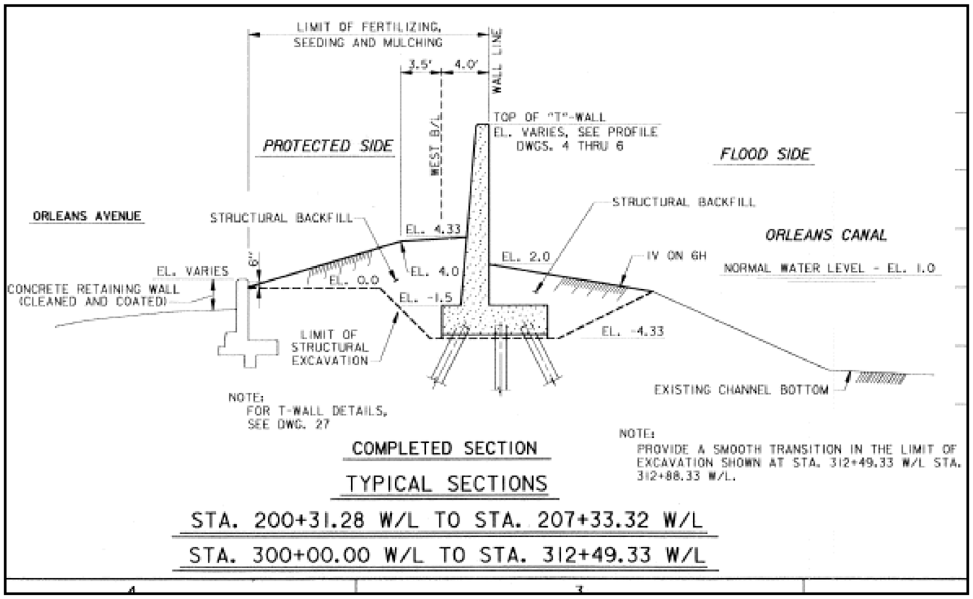


Figure III-6. "Normal Water Surface" Notation on Flood Side of Orleans Outfall Canal (Typical)

Reference Benchmark for Orleans Canal Floodwall Construction. Contract drawings indicate that Benchmark "CHRYSLER RM" was used as the vertical reference for design and construction associated with floodwalls constructed on the Orleans Avenue Outfall Canal. This mark was used for all the projects referenced above. This benchmark, originally set in 1931 by the USC&GS (now the National Geodetic Survey), is located in a concrete retaining wall at the intersection of Lakeshore Drive and the Orleans Outfall Canal.

No other benchmarks are noted in the construction plans. It is presumed all construction stake out during the period 1993 to 2000 was performed relative to this single benchmark.

Reference Datum of Benchmark "CHRYSLER RM." The Phase II-D Plans (DACW29-93-C-0077) note that PROJECT BM "CHRYSLER RM" is at elevation 7.11 ft "M.S.L." (Mean Sea Level) and on a "1983 Datum." The General Notes on the Phase II-D Plans indicate that "all elevations are expressed in feet and refer to National Geodetic Vertical Datum (N.G.V.D.). No datum date reference is indicated.

The Phase II-A Plans (DACW29-97-C-0029) and Phase I-C Plans (DACW29-99-C-0025) note in the "Tabulation of Bench Marks" that 'CHRYSLER RM" is at elevation "7.11 [ft] N.G.V.D. (1983 Epoch)." No reference to "NGVD29" or a subsequent adjustment is made.

The Phase I-B Plans (DACW29-00-B-0094) note CHRYSLER RM as 7.11 ft N.G.V.D. on the "1984 Epoch."

Thus, all construction documents are consistent in specifying a constant reference elevation and benchmark.

Historical Adjustments to CHRYSLER RM (1951 to date). The following table illustrates the various elevations associated with Benchmark CHRYSLER RM. Most of the changes are due to readjustments of level lines by the NOAA NGS (USC&GS), to account for subsidence in this area.

Table III-1
Successive Elevations on Benchmark CHRYSLER RM from 1951 to 2006

Elev, ft	Datum	Adjustment	Agency	Reference
8.533	NGVD29	19 Mar 52	USC&GS	
7.923	NGVD29	1951	USC&GS	L-13860
7.694	NGVD29	9 Apr 65	USC&GS	L-19622
7.108	NGVD29	1 Sep 82	USC&GS	L-19622/13860
7.231	NGVD29	30 Jan 86	USC&GS	L-24903
7.03	NGVD29	21 May 91	USC&GS	L-25283
6.83	NGVD88	14 Feb 94	USC&GS	BJ1349
6.85	NGVD88	Dec 96	USC&GS	BJ1349
6.42	NGVD88 (2004.65)	10 Feb 06	USC&GS	(unpublished/L-25517)
6.38	NGVD88 (2004.65)	11 Feb 06	USACE	IPET Survey Team
6.13 est	LMSL (1983-2001)	2005	NOAA CO-OPS	Provisional
TBD	LMSL (2001-2005)	(May 2006)	NOAA CO-OPS	

The “7.108” ft elevation from the 01 Sep 82 adjustment of CHRYSLER RM appears to be the source for the “7.11” ft elevation shown on the contract plans. Although more recent adjustments were available (1986 and 1991), the variance between these adjustments (± 0.1 ft) is not significant. It appears the “1983 Epoch” referenced in various contract documents may be referring to the horizontal adjustment datum, i.e., North American Datum of 1983 (NAD83). The above table clearly shows a subsidence trend in this area over a 50-year period, and the need to account for these relative elevation variations and trends. The 10 Feb 06 adjustment is based on unadjusted level data from 1994, as corrected to the epoch NAVD88 (2004.65). The 11 Feb 06 adjustment is based on a Third-Order differential level line run from Benchmark ALCO to Benchmark CHRYSLER RM, holding the NGS published NAVD88 (2004.65) elevation of ALCO fixed.

The Local Mean Sea Level difference based on the epoch (1983-2001) is provisional and is estimated based on provisional data from the NOAA New Canal gage (17th Street Canal). Local Mean Sea Level elevation differences for a later epoch (2001-2005) have not been computed as of this Interim Report date. They will be provided in the Final Report. It is estimated that the LMSL (2001-2005) difference from NAVD88 (2004.65) will be will be larger than that relative to the older LMSL epoch (1983-2001).

Local Mean Sea Level Relationships at the Orleans Avenue Outfall Canal. The elevation of Benchmark CHRYSLER RM can be related to the local mean sea level (LMSL) of Lake Pontchartrain using the relationships at the New Canal Gage (BM ALCO), which is slightly over a mile to the west of the Orleans Outfall Canal.

From Figure III-5 at the 17th Street Canal (New Canal Gage-Benchmark ALCO):

ALCO MSL (epoch 1983-2001)	5.89 ft (provisional)
ALCO NAVD88 (12/05/96)	6.59 ft
Difference:	(0.70 ft) [MSL — NAVD88]

CHRYSLER RM [NAVD88 (12/05/96)]	6.85 ft
Difference [MSL (epoch 1983-2001) — NAVD88]	-0.70 ft
LMSL at CHRYSLER RM (epoch 1983-2001)	6.15 ft

From the above, the estimated LMSL elevation of Benchmark CHRYSLER RM is 6.15 ft. This is based on the NOAA Tidal Epoch of 1983-2001.

For information, the LMSL elevation of CHRYSLER RM relative to the superseded 1960-1978 tidal epoch is estimated as:

CHRYSLER RM [NAVD88 (12/05/96)]	6.85 ft
Difference [MSL (epoch 1960-1978) — NAVD88]	-0.55 ft
LMSL at CHRYSLER RM (epoch 1960-1978)	6.30 ft

The elevation difference is attributable to the 0.15 ft change between the epochs. Since the 1983-2001 tidal epoch was not updated until ca 2003, mean sea level relationships during the time of construction would have had to been referenced to the 1960-1978 epoch. However, none of the contract documents makes mention of any particular tidal epoch.

Impact of Datum Variations on Constructed Floodwall Elevations. Given the nearly universal presumption that "NGVD" and "MSL" were equivalent "sea level" datums, and that floodwall design was computed relative to Lake Pontchartrain MSL, the actual constructed elevation on a typical floodwall in the London Avenue Outfall Canal is reduced by approximately:

Benchmark CHRYSLER RM	7.11 ft "NGVD" (Contract Plans-1982 adjustment)
Benchmark CHRYSLER RM	6.30 ft LMSL (1960-1978 epoch)
Difference:	0.81 ft

In effect, floodwalls designed relative to a MSL or LMSL datum would have been constructed about 0.8 ft lower when using the NGVD29 geodetic datum from a 1982 adjustment as a reference. Thus a floodwall designed to 14.0 ft

NGVD (i.e., MSL) would actually be constructed to 13.2 ft relative to LMSL (1960-1972 epoch), or 13.1 ft relative to the 1983-2001 LMSL epoch.

Assessment of Pre- and Post-Katrina Flood Protection Elevations (Orleans Avenue Outfall Canal). To evaluate pre-Katrina flood protection elevations, conventional topographic survey data taken just after the hurricane were obtained. Post-Katrina floodwall cap elevations were observed using conventional topographic surveying techniques—differential leveling and RTK methods. These elevations are also likely representative of pre-Katrina conditions in 2005. These surveys on the NAVD88 (2004.65) geodetic reference system can be adjusted to LMSL using the latest tidal datum epoch available (1983-2001)—e.g., topographic survey elevations observed on the NAVD88 (2004.65) geodetic datum were reduced by 0.25 ft to relate them to the estimated LMSL (1983-2001 epoch) elevation of Lake Pontchartrain. As noted above, this 0.25 ft conversion is provisional and does not necessarily reflect the current (2006) LMSL estimate in Lake Pontchartrain.

Designed and current floodwall elevations for selected sections of the Orleans Avenue Canal are listed in the following table. The average elevation was computed from representative shot points taken atop the floodwall along each reach. Variances in the floodwall cap elevation were as much as ± 0.5 ft along some reaches—probably due to uneven settlement.

Table III-2
Design and Current Floodwall Elevations in Selected Reaches (Orleans Avenue Outfall Canal)

Reach	No. of Shot Points	Design Elevation NGVD (MSL)	Average Elevation (2005-2006)	
			NAVD88 (2004.65)	LMSL (1983-2001)
WEST BANK RE Lee Blvd. to Filmore Ave.	15	N/A	13.2 ft	13.0 ft
WEST BANK Filmore Ave. to Harrison Ave.	20	14.0 ft (T-Wall)	13.4 ft	13.2 ft
WEST BANK Harrison Ave. to PS 7 / I-610	28	N/A	14.0 ft	13.8 ft
EAST BANK RE Lee Blvd. to Filmore Ave.	21	14.4 ft (I-Wall)	13.4 ft	13.2 ft
EAST BANK Filmore Ave. to Harrison Ave.	25	14.8 ft (I-Wall)	13.8 ft	13.6 ft
EAST BANK Harrison Ave. to PS 7 / I-610	19	14.9 ft (I-Wall)	13.9 ft	13.6 ft

Differences in floodwall cap elevations range between 0.8 ft and 1.3 ft.

2. London Avenue Outfall Canal Construction Reference Datums. The following construction drawings and Design Memorandums were reviewed as part of this assessment:

- DACW29-94-C-0079 (94-B-0047) As Built Mark Up—London Ave. Outfall Canal Parallel Protection— Mirabeau Ave.-to R.E. Lee Blvd (West Bank)—Mirabeau Ave. to Leon C. Simon Blvd. (East Bank)

- DACW29-02-C-0013 (01-B-0092) London Ave. Outfall Canal Parallel Protection—Floodproofing Mirabeau and Filmore Ave. Bridges
- DACW29-94-C-0003 (93-B-0080) As-Built London Ave. Outfall Canal Parallel Protection—Pump Station 3 to Mirabeau Ave. Floodwall
- DACW29-99-C-0005 (98-B-0060) As-Built London Ave. Outfall Canal Parallel Protection—Floodproofing Gentilly Blvd. Bridge
- DACW29-98-C-0082 (98-B-0065)As-Built London Ave. Outfall Canal Parallel Protection— Floodproofing Leon C. Simon Blvd. Bridge
- GDM 19A (Vol I and II) London Ave. Outfall Canal (1989)
- GDM 20 (Draft) London Ave. Canal Floodwalls and Levees—Orleans Levee District—Apr1986
- DM01 Part III Hydrology and Hydraulic Analysis—Lake Pontchartrain & Vicinity-Lakeshore (Sep 1968)

Design Elevation Parameters. Parallel protection elevations are shown in GDM No. 19A and on various contract plans. The design SPH stillwater surface elevation of Lake Pontchartrain is 11.5 ft NGVD. This base elevation was used in subsequent HEC-2 models to compute required floodwall elevation on each side of the canal and at the bridges. As in other Lake Pontchartrain projects, the "NGVD" elevation is assumed to be MSL or LMSL—e.g., "Lake Pontchartrain Normal Water Level = 0.0 ft MSL."

The design stillwater elevation in the London Avenue Outfall Canal was 11.85 ft "NGVD." The 14.4 ft NGVD floodwall design was derived by adding 2.0 ft freeboard and 0.5 ft settlement allowances to the 11.85 ft stillwater elevation. Again, the NGVD floodwall elevation was generally assumed to be equivalent to MSL.

Reference Benchmark used in Orleans Outfall Canal Parallel Floodwall Construction. Benchmark "P 153" was used as the vertical reference for design and construction associated with most of the floodwalls constructed on both banks the Orleans Avenue Outfall Canal. This benchmark, originally set in 1951 by the US Coast & Geodetic Survey (USC&GS—now the National Geodetic Survey), is destroyed. It was located on the Lakeshore Drive Bridge over the London Avenue Canal. The mark was destroyed ca 2002 when a new bridge was constructed. (2005/2006 post-Katrina construction and topographic surveys in the London Avenue Canal have been referenced to Benchmarks GRAHAM and GRAHAM RM, both of which were on the original USC&GS level line with P 153).

Benchmark P 153 was used for most of the floodwall projects listed above. No other benchmarks are noted in the construction plans except on the 1998 Leon Simon Bridge Floodproofing project (DACW29-98-C-0082) where Benchmark "AA 190" was listed in addition to "P 153." On the 1999 Gentilly Blvd. Bridge floodproofing project (DACW29-99-C-0005), a Benchmark "U 153" is referenced in addition to "P 153"—as shown in the figure below. Other than on these two projects, it is presumed all other floodwall construction stakeout was performed relative to the single benchmark "P 153."

Reference Datum of Benchmark "P 153." Contract DACW29-94-C-0079 is typical in referencing the elevation of Benchmark "P 153" relative to "N.G.V.D. (EPOCH 1964)." The elevation noted for the "1964 Epoch" is 11.270 ft. This elevation is actually based on a 9 April 1965 USC&GS readjustment of the NGVD29 network in this area. Bridge floodproofing projects in the late 1990s show both the 11.270 ft NGVD 1964 Epoch and a 10.39 ft elevation based on the 1991 epoch. The figure below shows dual NGVD29 reference datums (epochs) for "P 153."

REFERENCE BENCH MARK		
DESIGNATION	DESCRIPTION	ELEVATION
P 153	AT NEW ORLEANS, ABOUT 0.8 MILES ALONG LAKESHORE DR. FROM THE WEST SIDE OF TRAFFIC CIRCLE AT THE JUNCTION OF ELYSIAN FIELDS AVE., ABOUT 0.55 MILES NE ALONG LAKE TERRACE DR. FROM THE EAST END OF THE LAKESHORE DR. BRIDGE OVER BAYOU SAINT JOHN, THENCE 0.1 MILES EAST ALONG LAKESHORE DR. TO THE BRIDGE ACROSS LONDON AVE. CANAL. SET IN THE TOP OF THE EAST END OF PEDESTRIAN WALK ALONG THE SOUTH SIDE OF THE BRIDGE OVER THE EAST ABUTMENT OF THE BRIDGE, 5 FT. SOUTH OF THE SOUTH CURB OF THE DRIVE, 6 IN. WEST OF THE EAST END OF THE BRIDGE AND ABOUT 1 FT. ABOVE THE DRIVE.	11.270 N.G.V.D. (1964 EPOCH) 10.390 N.G.V.D. (1991 EPOCH)
U 153	IN NEW ORLEANS, AT 2251 NORTH BROAD AVENUE, 33.7 M (110.6 FT.) SOUTHEAST OF THE SOUTHEAST CORNER OF PUMP STATION 3 AT 2251 NORTH BROAD STREET, 9.7 M (31.8 FT.) SOUTHWEST OF THE NORTHEAST CORNER OF A RETAINING WALL, 6.8 M (22.3 FT.) WEST OF THE NEAR RAIL OF THE SOUTHERN RAILROAD, 5.6 M (18.4 FT.) NORTHEAST OF THE NORHTWEST CORNER OF A FENCE, AND THE MONUMENT PROJECTS 0.2 M (0.7 FT.) ABOVE THE GROUND SURFACE.	4.81 N.G.V.D. (1991 EPOCH)

Figure III-7. Reference Benchmarks (Gentilly Blvd. Bridge Floodproofing—DACW29-99-C-0005)

Historical Adjustments to P 153 (1951 to date). The following table (Table III-3) illustrates the various elevations associated with Benchmark P 153. Most of the changes are due to readjustments of level lines by the NOAA NGS (USC&GS), to account for subsidence in this area.

Table III-3 Successive Elevations on Benchmark P 153 from 1951 to 2006				
Elevation, ft	**Datum**	**Adjustment**	**Agency**	**Reference**
12.087	NGVD29	19 Mar 52	USC&GS	
11.476	NGVD29	1951	USC&GS	L-13860
11.270	NGVD29	9 Apr 65	USC&GS	L-19622
10.708	NGVD29	1 Sep 82	USC&GS	L-19622/13860
10.623	NGVD29	30 Jan 86	USC&GS	L-24903
10.39	NGVD29	21 May 91	USC&GS	L-25283
10.20	NGVD88	14 Feb 94	USC&GS	BJ1361
10.21	NGVD88	5 Dec 96	USC&GS	BJ1361
9.79	NGVD88 (2004.65)	10 Feb 06	USC&GS	(unpublished/L-25517)
9.54 est	LMSL (1983-2001)	2005	NOAA CO-OPS	provisional
TBD	LMSL (2001-2005)	(May 2006)	NOAA CO-OPS	

The 10 Feb 06 NAVD88 (2004.65) elevation shown for P 153 is not based on recent observations since the mark no longer exists. This is the computed elevation assuming no subsidence has occurred since 1994. The 09 Apr 65 NGVD29 elevation of 11.27 ft corresponds to that used for most of the London Avenue Canal floodwall construction during the early 1990s. This elevation is listed as “Epoch 1964.”

It is uncertain why the later readjustment elevations (i.e., 1982 and 1986) were not used for contracts issued after 1990. The 0.65 ft elevation change from 1965 to 1986 is significant. One of the As-Builts from a later contract that listed the 1991 elevation of P 153 (10.39 ft) appears to have held the 1965 elevation for construction stake out in setting the top of the floodwall, in lieu of the 1991 elevation—a 0.9 ft difference.

As in previous outfall canal projects in this area of Lake Pontchartrain, the above table clearly shows a subsidence trend in this area over a 50-year period, and the need to account for these relative elevation variations.

The Local Mean Sea Level difference based on the epoch (1983-2001) is provisional and is estimated based on provisional data from the NOAA New Canal gage (17th Street Canal). Local Mean Sea Level elevation differences for a later epoch (2001-2005) have not been computed as of this Interim Report date. They will be provided in the Final Report. It is estimated that the LMSL (2001-2005) difference from NAVD88 (2004.65) will be larger than that relative to the older LMSL epoch (1983-2001).

Local Mean Sea Level Relationships at the London Avenue Outfall Canal. The elevation of Benchmark P 153 can be related to the local mean sea level (LMSL) of Lake Pontchartrain using the relationships at the New Canal Gage (BM ALCO), which is about 2 ½ miles to the west of the London Outfall Canal.

From Figure III-5 at the 17th Street Canal (New Canal Gage-Benchmark ALCO):

ALCO MSL (epoch 1983-2001)	5.89 ft (provisional)
ALCO NAVD88 (12/05/96)	6.59 ft
Difference:	(0.70 ft) [MSL — NAVD88]

P 153 [NAVD88 (12/05/96)]	10.21 ft
Difference [MSL (epoch 1983-2001) — NAVD88]	-0.70 ft
LMSL at P 153 (epoch 1983-2001)	9.51 ft

From the above, the estimated LMSL elevation of Benchmark P 153 is 9.51 ft. This is based on the NOAA Tidal Epoch of 1983-2001 and is approximately representative of the MSL elevation at the time of construction.

The LMSL elevation of P 153 relative to the superseded 1960-1978 tidal epoch is computed as:

P 153 [NAVD88 (12/05/96)]	10.21 ft
Difference [MSL (epoch 1960-1978) — NAVD88]	-0.55 ft
LMSL at P 153 (epoch 1960-1978)	9.66 ft

The elevation difference is attributable to the 0.15 ft change between the epochs. Since the 1983-2001 tidal epoch was not updated until ca 2003, mean sea level relationships during the time of construction would have had to been referenced to the above 1960-1978 epoch.

Impact of Datum Variations on Constructed Floodwall Elevations. Given the nearly universal presumption that “NGVD” and “MSL” were equivalent datums, and that floodwall design was computed relative to MSL = 0.0 ft on Lake Pontchartrain, the actual constructed elevation on a typical floodwall in the London Avenue Outfall Canal is reduced by approximately:

Benchmark P 153	11.27 ft “NGVD” (Contract Plans)
Benchmark P 153	9.66 ft LMSL (1960-1978 epoch)
Difference:	1.61 ft

In effect, floodwall elevations designed relative to a LMSL datum would be constructed about 1.6 ft lower when using the 1965 adjustment of the NGVD29 geodetic datum as a reference. Thus a floodwall designed to 14.4 ft NGVD (i.e., MSL) would actually be constructed to 12.8 ft relative to LMSL (1960-1978 epoch), or 12.7 ft relative to the 1983-2001 tidal epoch.

Assessment of Pre- and Post-Katrina Flood Protection Elevations (London Avenue Outfall Canal). Designed and current floodwall elevations for selected sections of the London Avenue Canal are listed in the following table. Data were obtained and adjusted using identical procedures outlined for the

Orleans Avenue Outfall Canal evaluation. The average elevation was computed from representative shot points taken atop the floodwall along each reach. Variances in the floodwall cap elevation were typically ± 0.2 ft along some reaches.

Table III-4
Design and Current Floodwall Elevations in Selected Reaches (London Avenue Outfall Canal) New Orleans District/Task Force Guardian Post-Katrina Surveys Oct-Dec 2005

Reach	No. of Shot Points	Design Elevation NGVD (MSL)	Average Elevation (2005-2006)	
			NAVD88 (2004.65)	LMSL (1983-2001)
WEST BANK Leon Simon Ave. to RE Lee Blvd.	N/A	N/A	N/A	N/A
WEST BANK RE Lee Blvd. to Filmore Ave.	18	14.4 ft	13.0 ft	12.8 ft
WEST BANK Filmore Ave. to Mirabeau Ave.	23	14.4 ft	12.9 ft	12.7 ft
WEST BANK Mirabeau Ave. to Gentilly Ave.	27	14.4 ft	12.9 ft	12.7 ft
WEST BANK Gentilly Ave. to Pump Station 3	19	14.4 ft	12.9 ft	12.7 ft
EAST BANK Leon Simon Ave. to RE Lee Blvd.	8	14.4 ft	12.8 ft	12.6 ft
EAST BANK RE Lee Blvd. to Filmore Ave.	26	14.4 ft	12.9 ft	12.6 ft
EAST BANK Filmore Ave. to Mirabeau Ave.	17	14.4 ft	12.9 ft	12.6 ft
EAST BANK Mirabeau Ave. to Gentilly Ave.	24	14.4 ft	12.9 ft	12.7 ft
EAST BANK Gentilly Ave. to Pump Station 3	18	14.4 ft	13.1 ft	12.8 ft

NOTE: Topographic survey elevation data in this table derived from BM GRAHAM has not been verified.

During January 2006, Post-Katrina Overbank Surveys were taken north and south of the breach areas by 3001 Inc. These surveys were performed in support of IPET Team 5b physical modeling of the two breach sites on the canal. They also provide a quality assurance check on the above Task Force Guardian surveys performed shortly after Katrina. State plane coordinates are LA 1702 South and elevations are in feet NAVD88 (2004.65). The stationing is not the floodwall alignment.

Table III-5
Post-Katrina Floodwall Elevations Vicinity Breach Areas (London Avenue Outfall Canal) IPET Overbank Surveys January 2006 (3001, Inc.)

X	Y	Elev (ft)	Location	Datafile reference
North Breach — West Bank — South of RE Lee Blvd				
Vicinity of Burbank Drive (South of RE Lee)				
Sta. 15+50				
3680399.87	554667.93	13.041	Top Edge Conc Fldwal	17thLondon.dc
3680399.17	554667.96	13.107	Top Edge Conc Fldwal	17thLondon.dc
Sta. 16+00				
3680403.85	554618.86	13.013	Top Edge Conc Fldwal	17thLondon.dc
3680403.4	554618.87	13.013	Top Edge Conc Fldwal	17thLondon.dc
South Breach — East Bank — North of Mirabeau Avenue				
Vicinity of Wildair Drive (North of Mirabeau)				
Sta. 51+00				
3680710.06	551132.49	12.86	TPF * (West Bank)	Book# 060856
3680709.06	551132.43	12.86	TPF (West Bank)	Book# 060856
Sta 51+50				
3680712.27	551082.53	12.86	TPF (West Bank)	Book# 060856
3680711.27	551082.47	12.87	TPF (West Bank)	Book# 060856
3680837.01	551090.56	12.87	TPF (East Bank)	Book# 060856
Sta. 52+00				
3680841.23	551040.73	12.87	TPF (East Bank)	Book# 060856
3680717.48	551032.76	12.88	TPF (West Bank)	Book# 060856
3680716.49	551032.7	12.89	TPF (West Bank)	Book# 060856
Vicinity of Mirabeau Avenue Bridge				
Sta. 58+00				
3680889.96	550392.53	12.72	TPF (East Bank)	Book# 060856
3580888.97	550392.46	12.72	TPF (East Bank)	Book# 060856
Sta. 59+00				
3680895.17	550342.76	12.77	TPF (East Bank)	Book# 060856
3680894.17	550342.69	12.77	TPF (East Bank)	Book# 060856
3680763.44	550334.28	12.87	TPF (West Bank)	Book# 060856
3680762.44	550334.21	12.87	TPF (West Bank)	Book# 060856
Sta. 59+50				
3680898.39	550292.86	12.77	TPF (East Bank)	Book# 060856
3680897.39	550292.79	12.77	TPF (East Bank)	Book# 060856

*TPF – top of concrete floodwall.
Note: duplicate shots are at the flood side and protected side of the floodwall concrete cap.

Comparison between the Oct-Dec 2005 MVN/Task Force Guardian surveys and the 2006 IPET surveys indicates a NAVD88 (2004.65) elevation agreement to within ± 0.1 ft. In general, current floodwall cap elevations are running about 1.7 ft below the original design elevation. This is consistent with the 1.6 ft estimated reduction computed in the preceding paragraph.

Floodwall elevations near the Mirabeau Avenue breach area were running between 12.5 and 12.6 ft LMSL (1983-2001). This assumes no abnormal undulation in the breach site—a reasonable assumption given the fairly uniform elevations in the existing (unbreached) floodwalls. Updated sea level epochs may reduce this relative elevation even further. A more detailed analysis of pre- and post-Katrina elevations on floodwalls adjacent to the North Breach (R.E. Lee Blvd) and South Breach (Mirabeau Ave) will be included in the Final Report.

3. 17th Street Outfall Canal Construction Reference Datums. The following construction drawings and Design Memorandums were reviewed as part of this assessment:

- Contract 92-1 Board of Levee Commissioners of East Jefferson Levee District -17th Street Canal West Side Levee Improvements
- Orleans levee District (OLD) Contract 02043-0489 As Built—17th Street Canal Phase IB—Hammond Hwy to Southern RR 1990
- DACW29-93-B-0025 Excavation and Flood Protection 17th St Canal—Capping of Floodwalls—East Side Levee Improvements
- DACW29-95-C-0093 (95-B-0095) As Built Markup—17th St Outfall Canal-Metairie Relief—Floodproofing Veterans Blvd Bridges
- GDM 20 Vol I & II-17th St Outfall Canal (Metairie Relief) Orleans Parish & Jefferson Parish 1990
- DM01 Part III Hydrology and Hydraulic Analysis—Lake Pontchartrain & Vicinity-Lakeshore (Sep 1968)

Design Elevation Parameters for 17th Street Canal
EAST SIDE LEVEE IMPROVEMENTS—FLOODWALL CAPPING (DACW29-93-B-0025)

Floodwall cap elevations:

Southern Railway Sta 126+02 to I-10 Bridge Sta 97+52	elev 15.0 ft NGVD
I-10 Bridge Sta 94+17 to Vet Hwy Sta 81+52	elev 14.5 ft
Vet Hwy Sta 80+00 to Hammond Hwy Sta 8+49	elev 14.0 ft
Hammond Hwy Sta 7+03 to Sta 0+00	elev 14.0 ft

Plans state normal water surface 1.5 to 2.0 ft NGVD (source of hydrograph not noted in plans)
Contract plan elevations are referenced to “USCE MONUMENT 14” elevation 8.77 ft NGVD

WEST SIDE LEVEE IMPROVEMENTS (Contract 92-1—1992) As-Builts

Top of Required Floodwall Elevations:

Lakefront Levee (Sta 549+78) to Vet Hwy (Sta 625+02)	elev 14.0 ft
Vet Hwy (Sta 626+25) to I-10 Bridge (Sta 638+84)	elev 14.5 ft
I-10 Bridge (Sta 642+23) to South. Railway Bridge (Sta 669+17)	elev 15.0 ft

Normal water surface elevation 1.5 ft to 2.0 ft
Reference construction benchmark: USCE Monument 14--elev: 8.77 NGVD (no epoch noted)

VETERANS BLVD BRIDGE FLOODPROOFING (DACW29-95-C-0093)

Still water level	12.5 NGVD
Wave action	14.5 NGVD
Design water level	12.5 ft @ 6,650 cfs @ 300 yr
Normal water level	1.5 to 2.0 ft NGVD @ 0 cfs (no hydrograph shown in plans—specifications not available)

Project Reference Benchmark: "**T-193**" elev 9.741 (NGVD 1972 epoch) on bridge abutment (last recovered 1994)

Phase I-B HAMMOND HWY TO SOUTHERN RAILWAY (OLD Contract 02043-0489 —1990):

Contract plans note that elevations are referred to MSL.
"Normal Water Surface" elevation ranges from 1.0 to 2.0 ft ... apparently either based on a pump station gage hydrograph or perhaps from a gage at Lake Pontchartrain (not indicated in the Plans). Section views indicate the normal water surface elevation is 1.0 ft (typical).
Floodwall sheet pile top elevations vary: 13.5, 14.0, & 14.5 ft

GDM 20 (1990)

Elevations referenced to NGVD (no epoch date noted).
Hydraulic & Structural design criteria:
  Lake Pontchartrain stillwater elevation 11.5 ft @ 300 year SPH
  Wind tide level (17th St Canal) 11.50 to 12.50 ft
  East Bank floodwall elevations: 14.00 to 15.00 ft
  West Bank floodwall elevations: 16.50 to 15.00 ft

Reference Benchmark used in 17th Street Canal Parallel Floodwall Protection. Benchmark "USACE MONUMENT 14" was apparently used as the vertical reference for nearly all the floodwall design and construction on the 17th Street Outfall Canal. The exception is the Veterans Blvd Bridge floodproofing project (DACW29-95-C-0093) in which a benchmark "T 193" in indicated on the contract plans. The origin of benchmark MONUMENT 14 could not be determined from New Orleans District records. The source survey data for the elevation shown on the contract drawings (8.77 ft NGVD) could not be found. The mark was never incorporated into the USC&GS (now the NOAA National Geodetic Survey) database.

No other benchmarks are noted in the construction plans reviewed above. It is presumed all construction stakeout for the East Bank (Orleans Parish) and West Bank (Jefferson Parish) floodwalls was performed relative to a single benchmark—MONUMENT 14.

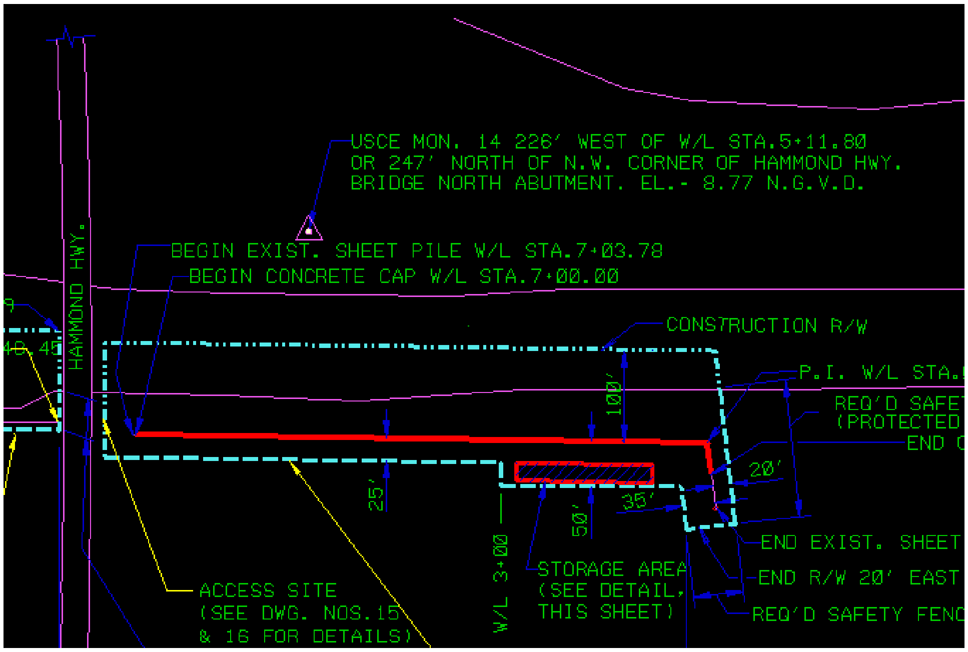


Figure III-8. 17th Street Canal Reference Benchmark USACE MONUMENT 14 near Hammond Hwy

Derived Elevations of Benchmark MONUMENT 14. Post-Katrina surveys to MONUMENT 14 indicated its elevation was suspect—not only currently but also most likely at the time of initial floodwall construction.

A differential level line run in November 2005 from primary Benchmark ALCO to MONUMENT 14 yielded an elevation of 7.06 ft NAVD88 (2004.65) on MONUMENT 14. Comparing equivalent reference datums and adjustment epochs:

MONUMENT 14	7.06 ft NAVD88 (2004.65)
Difference (NGVD29-NAVD88 (2004-65))	+0.62 ft [from Figure III-5]
MONUMENT 14 (most probable elevation)	7.68 ft NGVD29 (05/21/91)

Thus, the most probable elevation in 1991 is 7.68 ft (assuming no significant subsidence to date). The difference in elevation due to datum uncertainty is estimated as:

MONUMENT 14 (Construction Plans)	8.77 ft NGVD (unknown adjustment epoch)
MONUMENT 14 (most probable elevation)	7.68 ft (05/21/91)
Difference	1.09 ft (due to datum readjustment)

It is not likely a datum readjustment accounted for the large 1.09 ft difference.

Given “NGVD” was generally assumed to equal “MSL” on design and construction documents, the LMSL (1983-2001) elevation of MONUMENT 14 is estimated as:

MONUMENT 14	7.06 ft NAVD88 (2004.65)
Difference (MSL-NAVD88 (2004-65))	-0.25 ft [from Figure III-5]
MONUMENT 14	6.81 ft LMSL (1983-2001)

Then,

MONUMENT 14 (Construction Plans)	8.77 ft NGVD ≈ MSL
MONUMENT 14	6.81 ft LMSL (1983-2001)
Difference	1.96 ft

This 1.96 ft elevation disparity at Benchmark MONUMENT 14 may be due to a number of factors:

- The origin of the 8.77 ft elevation shown on the plans is unknown. There are no records available indicating how this elevation was set.
- It is uncertain what date the elevation was established, or on what vertical datum/adjustment it was referred to.
- Assumption that NGVD = MSL.
- Subsidence may have occurred since the elevation was established.
- Mark had incorrect elevation in 1990 (this is believed to be the likely problem based on recollections by MVN personnel).

The above assumptions can be roughly confirmed using pre-Katrina LIDAR topography (2000) and/or post-Katrina conventional topographic surveys in 2006 and 2006—see assessment following.

Assessment of Pre- and Post-Katrina Flood Protection Elevations (17th Street Outfall Canal). Design and current floodwall elevations for selected sections of the 17th Street Canal are listed in the following table, based on post-Katrina topographic surveys performed by MVN/Task Force Guardian and IPET Team 6. Data were obtained and adjusted using identical procedures outlined for the previous Orleans and London Canal evaluations. The average elevation was computed from representative shot points taken atop the floodwall along each reach. Variances in the floodwall cap elevation were typically less than ± 0.2 ft along some reaches.

Table III-6
Design and Current Floodwall Elevations in Selected Reaches (17th Street Outfall Canal) New Orleans District/Task Force Guardian Post-Katrina Surveys Oct-Dec 2005

Reach	No. of Shot Points	Design Elevation NGVD (MSL)	Average Elevation (2005-2006)	
			NAVD88 (2004.65)	LMSL (1983-2001)
WEST BANK Lakefront Levee to Veterans Hwy	58	14.0 ft	12.7 ft	12.4 ft
WEST BANK Veterans Hwy to I-10 Bridge	23	14.5 ft	13.4 ft	13.1 ft
WEST BANK I-10 Bridge to Southern RR	16	15.0 ft	13.4 ft	13.1 ft
EAST BANK Hammond Hwy to Veterans Hwy	26	14.0 ft	12.4 ft	12.1 ft
EAST BANK Veterans Hwy to I-10 Bridge	37	14.5 ft	13.5 ft	13.2 ft
EAST BANK I-10 Bridge to Southern RR	18	15.0 ft	13.6 ft	13.3 ft

During January 2006, Post-Katrina Overbank Surveys were taken north and south of the breach areas by 3001 Inc. These surveys were performed in support of IPET physical models of the breach sites. They also provide a quality assurance check on Task Force Guardian surveys performed after Katrina. State plane coordinates are LA 1702 South and elevations are in feet NAVD88 (2004.65). The stationing is not the floodwall alignment.

Table III-7 Post-Katrina Floodwall Elevations Vicinity East Bank Breach Area (17th Street Outfall Canal) IPET Overbank Surveys January 2006 (3001, Inc.)				
X	**Y**	**Elev (ft)**	**Location**	**Datafile reference**
South of Hammond Hwy (Vicinity Hay Place)				
Sta. 4+50				
3664412.64	554305.82	12.373	Top Conc Fldwall	17thLondon.dc
3664413.38	554305.78	12.376	Top Conc Fldwall	17thLondon.dc
Sta. 5+00				
3664409.22	554256.33	12.418	Top Conc Fldwall	17thLondon.dc
3664409.99	554256.3	12.425	Top Conc Fldwall	17thLondon.dc
Sta. 5+50				
3664406.5	554205.41	12.329	Top Conc Fldwall	17thLondon.dc
3664405.82	554205.56	12.318	Top Conc Fldwall	17thLondon.dc
South of Hammond Hwy (Vicinity 40th Street)				
Sta. 14+00				
3664348.77	553357.14	12.409	Top Conc Fldwall	17thLondon.dc
3664348.05	553357.13	12.36	Top Conc Fldwall	17thLondon.dc
Sta. 14+50				
3664345.33	553307.32	12.389	Top Conc Fldwall	17thLondon.dc
3664344.67	553307.28	12.414	Top Conc Fldwall	17thLondon.dc
Sta. 15+00				
3664341.03	553257.2	12.461	Top Conc Fldwall	17thLondon.dc
3664341.86	553257.23	12.475	Top Conc Fldwall	17thLondon.dc
Note: duplicate shots are at the flood side and protected side of the floodwall concrete cap.				

Based on provisional observations, current floodwall cap elevations appear to be running about 1.5 to 2 ft below the original design elevation. This is somewhat consistent with the 1.96 ft estimated reduction computed in the preceding paragraph.

Floodwall elevations near the Hammond Highway breach area were running between 12.1 and 12.2 ft LMSL (1983-2001) based on the IPET surveys and slightly lower (11.9 ft to 12.1 ft) using MVN survey data closer to the breach site. (Shots on floodwalls on each side of the breach were actually down to about elevation 11.7 ft; however it is not clear if the walls were deformed/deflected at these points). Updated sea level epochs may reduce these relative elevations even further.

The approximately 2-ft difference indicated in the above table correlates with the elevation projections made in the previous paragraphs. The above can be illustrated in the following graphic.

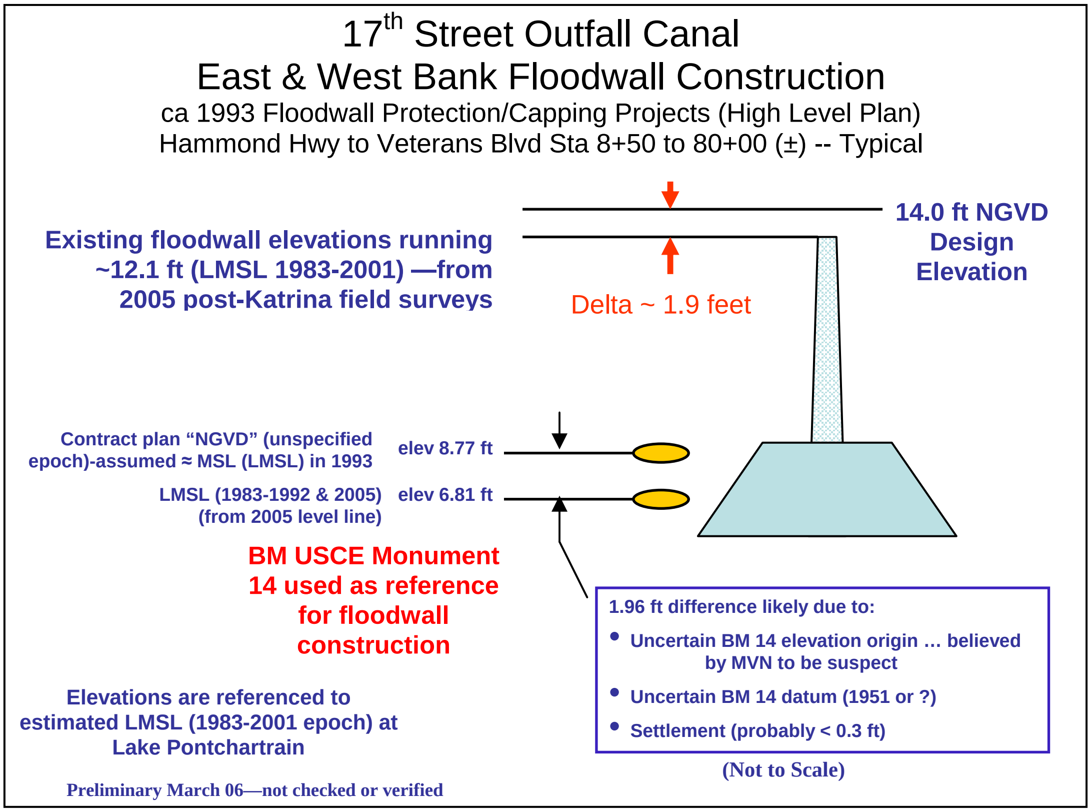


Figure III-9. Design vs. Current Floodwall Elevation—East Bank 17th St Outfall Canal

Pre-Katrina LIDAR Elevations on the 17th Street Floodwall Caps (East Side Breach Site). (This information has not yet been obtained for this Interim Report since it has not yet been adjusted to NAVD88 (2004.65). This LIDAR data will be compared with post-Katrina topographic survey data taken adjacent to the breach site. This analysis will be included in the Final Report.)

4. Inner Harbor Navigation Canal (IHNC) Construction Reference Datums. The following as built construction drawing was reviewed as part of this assessment:

- DACW29-70-B-0088 As Built Mark Up-IHNC Inner Harbor Navigation Canal East Levee—IHNC Lock to Florida Ave Levee & Floodwall Capping

Other floodwalls along the IHNC east or west bank were not evaluated in this assessment since the above area covers the critical breach site at the Lower 9th Ward.

Design Elevation Parameters for East Levee Floodwall Capping (1969). IHNC Lock to Florida Ave Sta. 0+00 to 56+20. Reference benchmark used for construction: "BM 1" or same mark as USC&GS "M-152"

- Elevation 21.811 ft MSL (1969 contract plans)

- (Located on IHNC East Lockwall—intact 2006)

- 2005 Post-Katrina GPS connection (MVN 10 Nov 05): Elev 20.34 ft NAVD88 (2004.65)

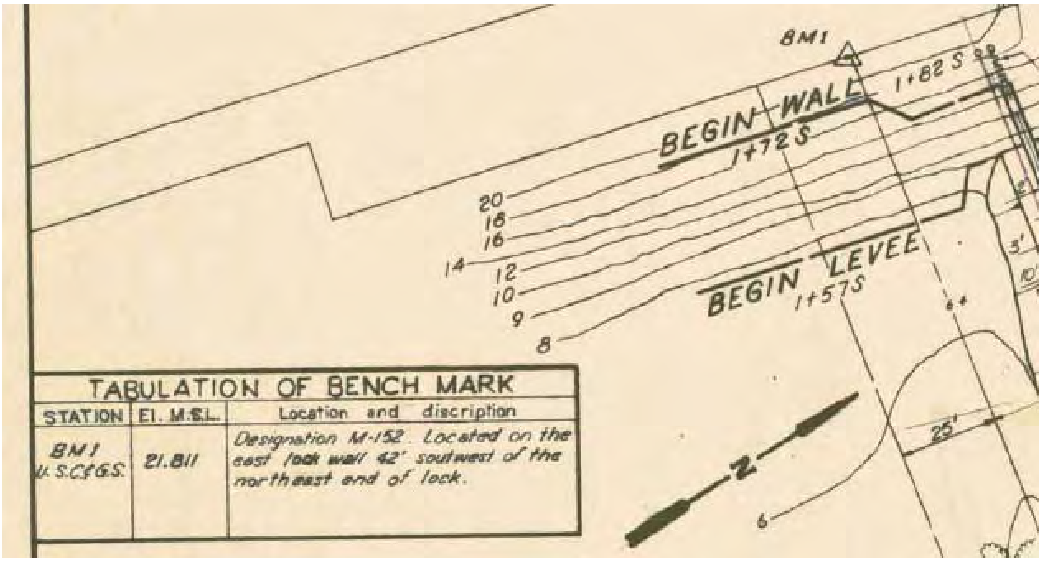


Figure III-10. Location and Description for “BM 1” (M 152 USC&GS) at IHNC Lock

- I-Walls constructed to 15.0 ft MSL—per As-Built Plans

- (No DM/GDM could be found noting design & freeboard parameters)

Historical Adjustments to Reference Benchmark M 152 (1951 to date). The following table (Table III-8) illustrates the various elevations associated with Benchmark M 152. Most of the changes are due to readjustments of level lines by the NOAA NGS (USC&GS), to account for subsidence in this area.

Table III-8
Successive Elevations on Benchmark M 152 from 1951 to 2005

Elev, ft	Datum	Adjustment	Agency	Reference
22.090	NGVD29	1951	USC&GS	L-13860
22.697	NGVD29	19 Mar 52	USC&GS	
21.070	NGVD29	1951/1 Sep 82	USC&GS	L-13860
21.811	NGVD29	1963/9 Apr 65	USC&GS	L-19622
21.811	MSL	1969 Contract Plans	MVN	DACW29-70-B-0088
21.071	NGVD29	1963/1 Sep 82	USC&GS	L-19622
21.070	NGVD29	1982	USC&GS	L-19622
21.148	NGVD29	1985/30 Jan 86	USC&GS	L-24903
20.96	NGVD29	21 Jun 91	USC&GS	L-25283/AU0668
20.963	NGVD29	1995	USC&GS	L-25517
20.76	NAVD88	14 Feb 94	USC&GS	AU0668
20.81	NAVD88	Dec 1996	USC&GS	AU0668
20.34	NAVD88 (2004.65)	10 Nov 05	USACE	MVN
TBD	LMSL (1983-2001)	(May 2006)	NOAA CO-OPS	
TBD	LMSL (2001-2005)	(May 2006)	NOAA CO-OPS	

From the above table it is apparent that the then (1969) most current elevation (21.811 ft) of M 152 was used in the contract plans, irrespective of the fact that the NGVD29 elevation was given as MSL.

The difference between MSL and NGVD29 at this location on the IHNC during the 1963-1969 period has not been determined. It is uncertain that older gage data would be able to quantify this difference to any level of confidence.

Local Mean Sea Level elevation differences in the IHNC have not been computed as of this Interim Report date. They will be provided in the Final Report. It is estimated that the LMSL (1983-2001) difference from NAVD88 (2004.65) will be around 0.2 ft ± 0.1 ft. The difference may be slightly larger for a later LMSL epoch (2001-2005).

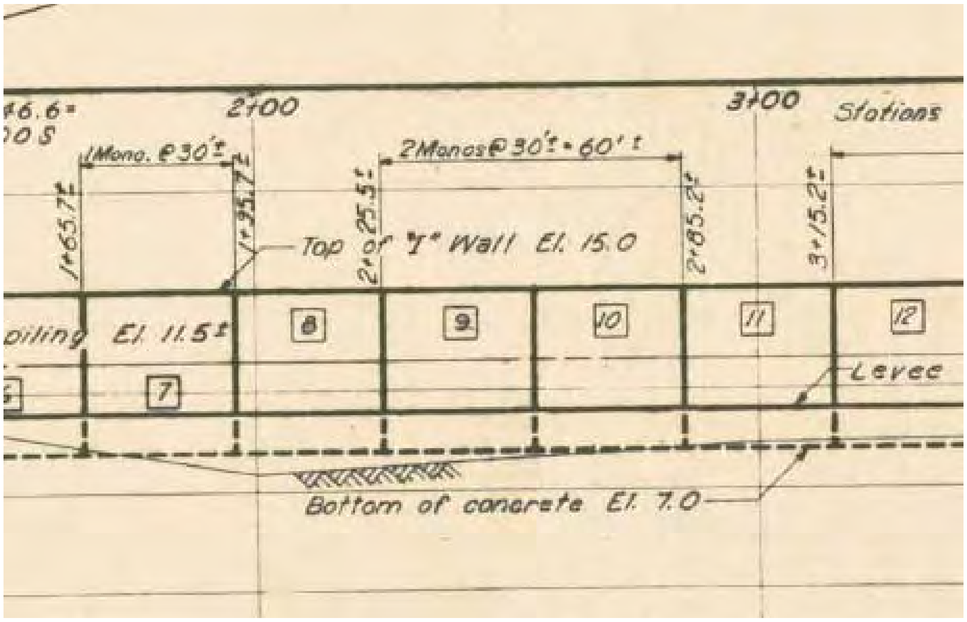


Figure III-11. East Side I-Wall Design Elevation 15.0 ft (Sta. 2+00 Typical)

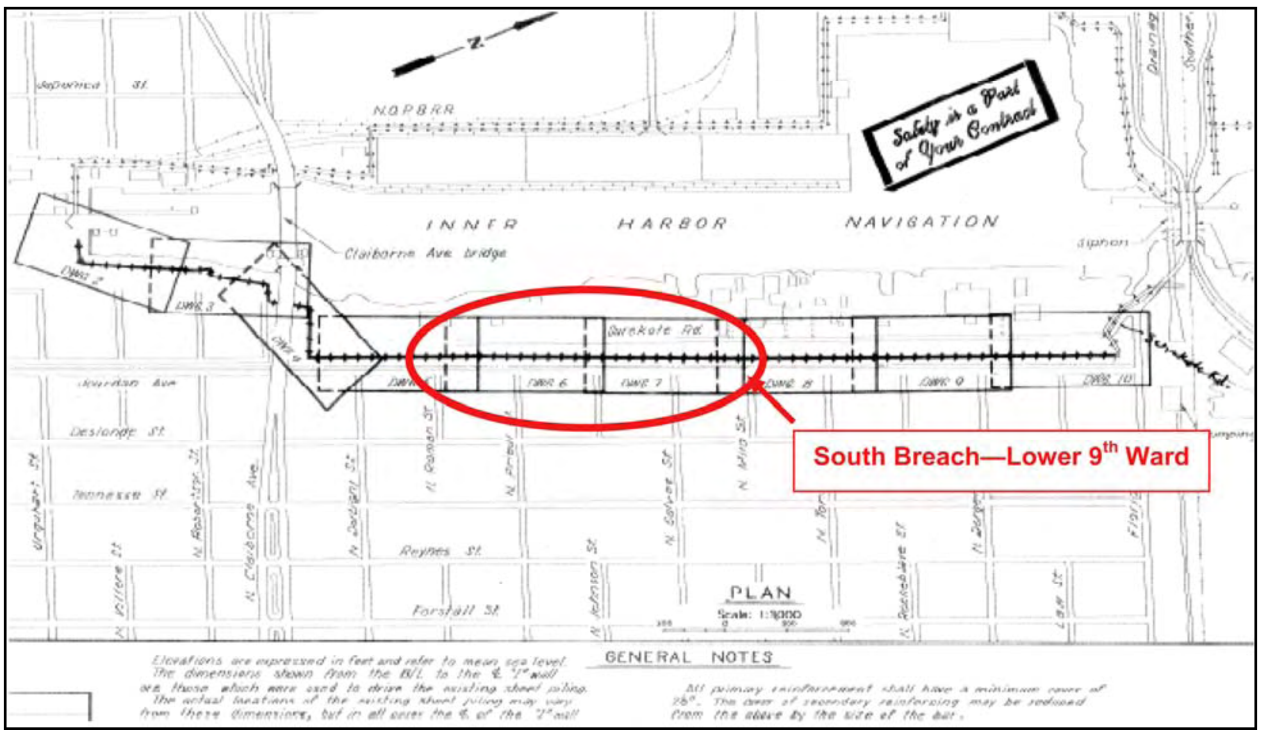


Figure III-12. IHNC East Side Floodwall Capping—IHNC Lock North to Florida Avenue (Lower 9th Ward Breach at approximately Sta. 2+00)

Assessment of Pre- and Post-Katrina Flood Protection Elevations (IHNC East Bank Floodwall between Claiborne and Florida Avenues). New Orleans District survey crews ran levels/RTK surveys to various points along the IHNC, as shown in the drawing below.

Figure III-13. Selected Post-Katrina Elevations on IHNC Floodwalls

12.35 ft - 50' south of floodgate w-27/w-28 off France road.
12.62 ft - 150' north of same floodgate off France road.
12.61 ft - near pumping station on Florida Ave
12.76 ft - 300' south of pump station Florida Ave.

During January 2006, Post-Katrina Overbank Surveys were taken north and south of the breach area by 3001 Inc. These surveys were performed in support of IPET physical modeling. They also provide a quality assurance check on MVN Task Force Guardian surveys performed after Katrina. State plane coordinates are LA 1702 South and elevations are in feet NAVD88 (2004.65). The stationing is not the floodwall alignment.

Table III-9 Post-Katrina Floodwall Elevations in Selected Reaches (East Bank IHNC) IPET Surveys Overbank Surveys January 2006 (3001, Inc.)				
X	**Y**	**Elev, ft**	**Location**	**Datafile reference**
RTK shots atop East Bank floodwall vicinity Florida Avenue Bridge:				
Sta. 0+00				
3696362.82	540601.98	12.616	Top Edge Conc Fldwal	IHNCEAST.dc
3696363.6	540602.19	12.638	Top Edge Conc Fldwal	IHNCEAST.dc
Sta. 0+50				
3696375.81	540546.84	12.561	Top Edge Conc Fldwal	IHNCEAST.dc
3696374.68	540547.01	12.589	Top Edge Conc Fldwal	IHNCEAST.dc
RTK shots atop floodwall vicinity Claiborne Avenue Bridge:				
Sta. 41+65				
3695275.99	536566.87	13.402	Top Edge Conc Fldwal	IHNCEAST.dc
3695275.76	536566.93	13.399	Top Edge Conc Fldwal	IHNCEAST.dc
Sta. 44+00				
3695089.7	536384.8	13.271	Top Edge Conc Fldwal	IHNCEAST.dc
3695089.47	536384.94	13.333	Top Edge Conc Fldwal	IHNCEAST.dc
Sta. 44+50				
3695069.01	536338.05	13.323	Top Edge Conc Fldwal	IHNCEAST.dc
3695069.34	536337.93	13.296	Top Edge Conc Fldwal	IHNCEAST.dc
Note: Duplicate shots are at the flood side and protected side of the floodwall concrete cap.				

From the above tables, elevations along the East Bank floodwall north of the breach area were running around 12.6 ft to 12.7 ft NAVD88 (2004.65). South of the breach area the elevations range from 12.7 ft to 13.4 ft near the Claiborne Avenue Bridge.

Assuming a 0.2 ft difference between LMSL and NAVD88 (2004.65)—[this value has not been quantified at the time of this Interim Report]—then the post-Katrina floodwall elevation relative to LMSL is approximately 12.5 ft. This 12.5 ft LMSL elevation would also be representative of the 2005 pre-Katrina floodwall elevation in this reach.

5. Stillwater and Normal Water Surface Elevations in Design Documents. Various design memorandums (DM) were reviewed to assess the reference datums used in determining hurricane design elevations. These included:

- DM 01 Part 1 Hydrology and Hydraulic Analysis--Lake Pontchartrain & Vicinity--Chalmette (Aug 1966)
- DM 01 Part 2 Hydrology and Hydraulic Analysis--Lake Pontchartrain & Vicinity--Barrier (Aug 1967)
- DM 01 Part 3 Hydrology and Hydraulic Analysis--Lakeshore (Sep 1968)
- DM 13 Vol I GDM Orleans Parish Lakefront Levee West of IHNC (Nov 1984)

Lake Pontchartrain and Vicinity Projects. DM 01 Part 2 (1967) states the average high tide of Lake Pontchartrain at 1.4 ft. This level is used as a base (or initial) elevation for subsequent storm surge modeling. The design memorandum notes all elevations are referred to “Mean Sea level.”

DM 01 Part 3 (1968) and DM 13 Vol 1 (1984) later noted the average high tide in Lake Pontchartrain at 0.7 ft. This was adjusted down 0.7 ft from the 1.4 ft average high tide cited in the 1967 Barrier Plan (DM 01 Part 2). This was based on a USC&GS releveling and gage adjustment.

TABLE 1
CRITICAL AVERAGE LAKE ELEVATIONS
DESIGN HURRICANE

Track	Starting lake stage feet	Contributions			Final lake stage feet
		Rainfall feet	Runoff feet	Overflow feet	
A	0.7	0.6	0.1	0.2	1.6
C	0.7	0.7	0.1	0.7	2.2
F	0.7	0.7	0.1	0.6	2.1

Figure III-14. Average Lake Pontchartrain stages (DM 01 Part 3—1968)

Other design memorandums note the “normal water level” of Lake Pontchartrain at 0.0 ft MSL (Appendix B of GDM 20 (Draft) London Ave. Canal Floodwalls and Levees—Orleans Levee District—(Apri1986).

(Note that the Design or Hurricane Tide is the maximum stillwater surface elevation experienced at the location during the passage of a hurricane. This Design Tide uses the initial normal (predicted) tide as a base reference, or alternately the high tide. EM 1110-2-1913 notes freeboard was, in the past, used to account for hydraulic, geotechnical, construction, operation, and maintenance uncertainties. Currently a risk-based analysis is used to set the final levee grade to account for settlement, shrinkage, cracking, geologic subsidence, and construction tolerances.)

DM 01 Part 1—Chalmette (1966) indicates a “normal predicted tide” of 1.60 ft (MLW) and a (-) 0.60 ft correction from MLW to MSL. This implies a normal predicted tide of 1.0 ft MSL at the Chalmette area. Resultant observed and computed hurricane surge heights are relative to MSL. A plate depicting typical tidal cycles in Lake Borgne and Lake Pontchartrain indicates MSL elevations average +1.0 ft above 0.0 MSL in both areas. DM 01 states the average tidal ranges in Lake Borgne and Lake Pontchartrain are +1.0 ft and 0.5 ft respectively, and the average elevation of the lakes “differs very little.” The elevation of Lake Borgne is given at 0.9 ft and Lake Pontchartrain 1.0 ft. The source of these elevations (i.e., gage and/or leveling datum) is not readily apparent in the design memorandum. Given all elevations in the design memorandum refer to MSL it is presumed that these 0.9 and 1.0 ft “normal water surface” superelevations also refer to MSL. If these elevations are based on gages referenced to a "NGVD" datum, this is not apparent from the limited records viewed.

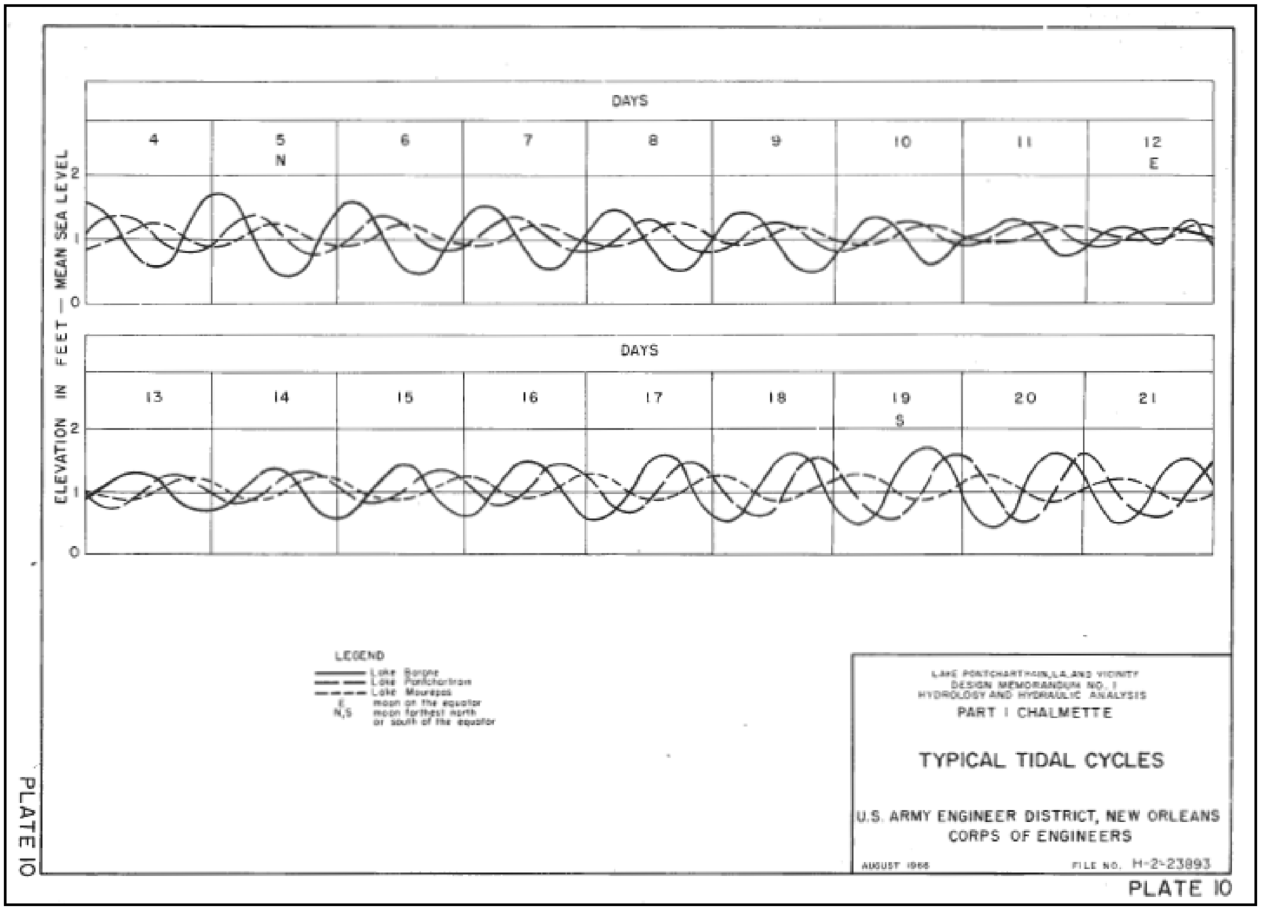


Figure III-15. +1.0 ft superelevation on Lake Borgne and Lake Pontchartrain (DM 01 Part 1)

The following plate from DM 01 Part 3 depicting wind tide profiles indicates the Mean Lake Level of Lake Pontchartrain as +1.0 ft.

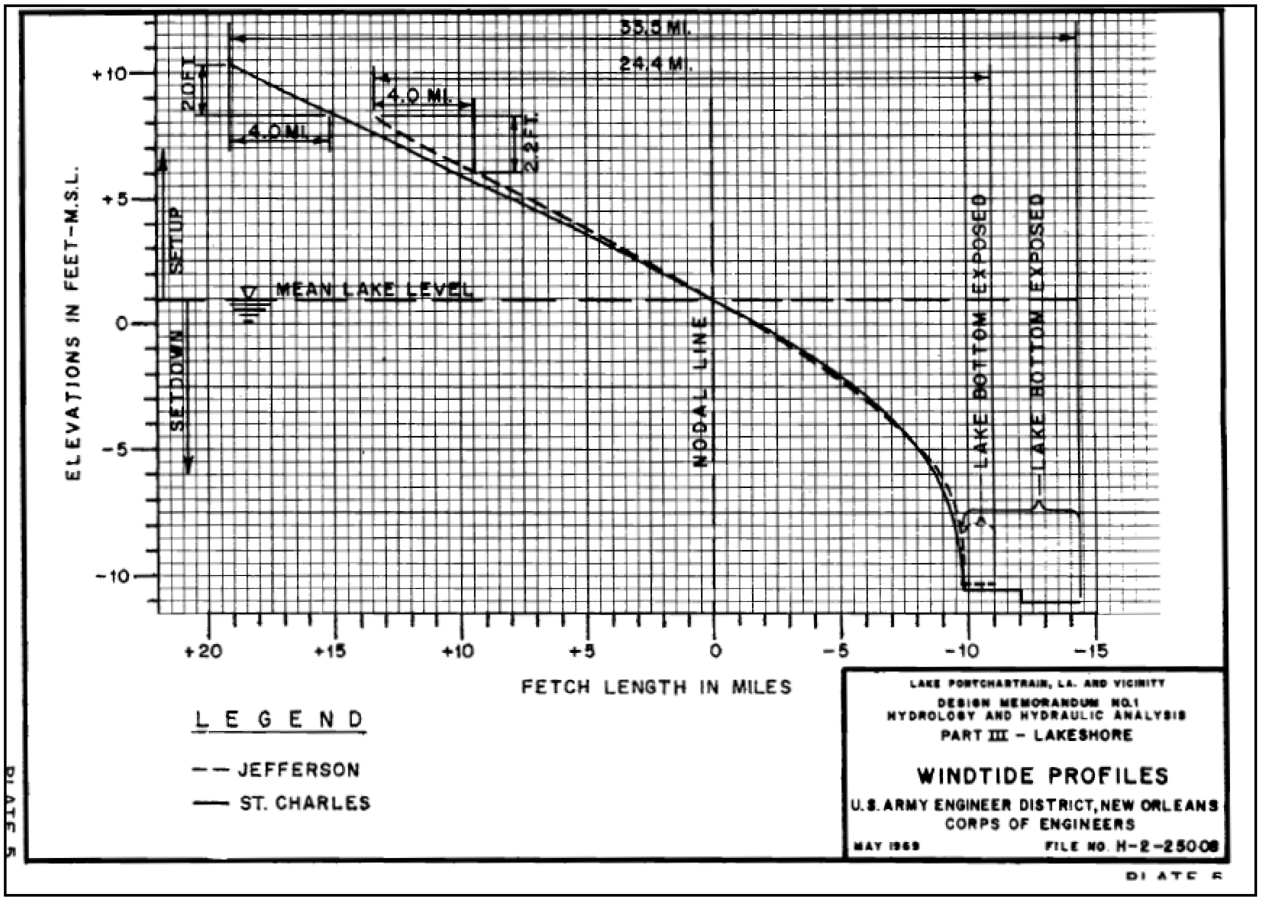


Figure III-16. +1.0 ft “Mean Lake Level” relative to MSL for Lake Pontchartrain (DM 01 Part 3)

Design and construction document depict Normal Water Levels in the outfall canals ranging between +1.0 ft to +2.0 ft. The source and reference datum for these estimates is not clear from the documents.

New Orleans to Venice Projects. Referenced design memorandums:

- DM 01 GDM Supp 04—New Orleans to Venice--Reach B2--Fort Jackson to Venice (Aug 1972)

- DM 01 GDM Supp 06--West Bank Mississippi River Levee--City Price to Venice (Mar 1987)

Stillwater elevations and hurricane design elevations on the New Orleans to Venice projects generally refer to the MSL datum (DM 01 GDM Supp 04--1972). Tides along the coast are noted having a mean range of 1 foot. Both headwater flooding and tidal effects are compensated for in computing surge elevations in the Mississippi River north of Venice. Page A-16 of DM 01 GDM Supp 04 states the Predicted “Mean Normal Tide” in the project area varies from 0.4 ft to 1.0 ft MSL. It is unclear if this Mean Tide is equivalent to Mean Tide Level or how it relates to Mean Sea Level. The design hurricane surge height for the project area is given as 11.5 ft MSL.

DM 01 GDM Supp 06 (1987) noted that surge studies performed after Hurricane Betsy in 1965 were in error by as much as 1 foot due to readjustments to the NGVD level network in this area. This resulted in hurricane stages being 1 foot too high.

> (4) Subsequent to completion of the NESCO study, it became apparent based on a new level network that bench mark elevations in the study area were actually as much as 1 foot lower with respect to national geodetic vertical datum than their recorded elevations. Therefore, all stages experienced during Betsy and used in the NESCO study were recorded too high with respect to national geodetic vertical datum. The undisturbed river profile and the Betsy surge crest profile used and computed in the NESCO study are shown on figure 6, plate B-10. However, the maximum stage shown at West Pointe-a-la-Hache, mile 49 AHP, was 14.4 feet rather than 15.2 feet, and the mean stage at the Carrollton gage, mile 103 AHP, prior to Betsy, was 2.0 feet rather than 2.7 feet. The 2.0-foot stage at Carrollton is the mean tide level on the day before Betsy struck the Louisiana coast. Corrected profiles are shown on plate B-7.

Figure III-17. NGVD29 network adjustment impact (Appendix B--DM 01 GDM Supp 06 (1987))

Mississippi River Gulf Outlet Projects. Referenced design memorandums:

- DM 01 A--MRGO Channels Mile 63.77 to 68.85 (Jul 1957)
- DM 01 B--MRGO Channels Mile 39.01 to 63.77 (May 1959)
- DM 01 C--MRGO Channels Mile 0 to 36.43 (Bayou La Loutre) Mile 0.0 to (-) 9.75 (38 ft Contour) (Nov 1959)
- DM 02 GDM Supp 03-Bayou La Loutre Reservation (Feb 1968)
- DM 01 GDM--Michoud Canal (Jul 1973)

All documents refer MRGO channel elevations to Mean Low Gulf (MLG), which is 0.78 feet below MSL. This reference is standard for dredging and navigation projects in this region—see the Background to this Report.

Records from a water level recording gage on the GIWW at Paris Road indicated average yearly high and low water stages significantly above that expected for an area subject to direct tidal flow, as shown in the figure below. The reason for this anomaly in unclear.

TABLE A-7
Average Annual High & Low Water Stages
Gulf Intracoastal Waterway at Paris Road Bridge

Year	Mean High Water (m.s.l.)	Mean Low Water (m.s.l.)
1959	Insufficient records	
1960	Insufficient records	
1961	2.58	1.66
1962	2.37	1.30
1963	2.27	1.27
1964	2.51	1.34
1965	2.77	1.37
1966	2.83	1.46
1967	Insufficient records	
1968	2.86	1.54
1969	3.30	1.87
1970	3.30	1.91

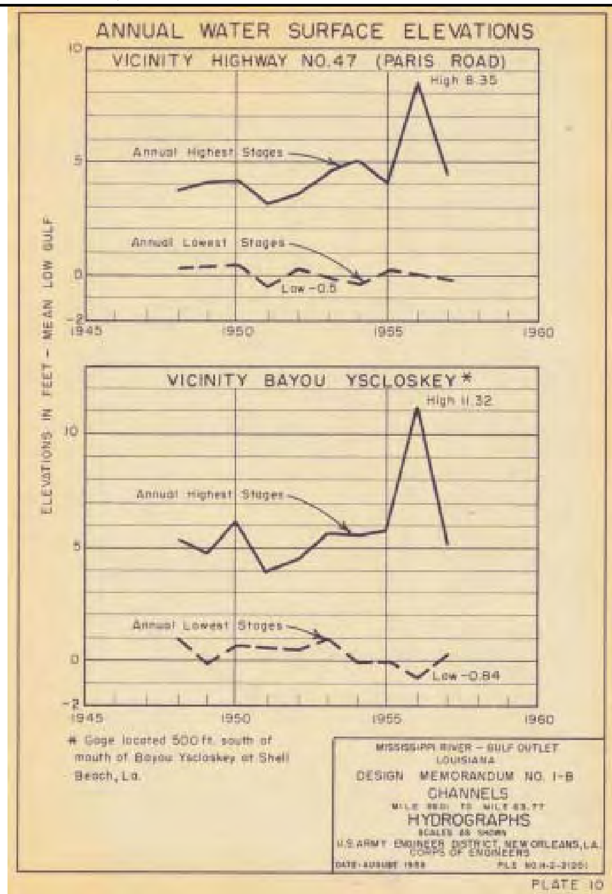


Figure III-18. GIWW water level stages at Paris Road (DM 01 GDM)

DM 02 GDM Supp 03-Bayou La Loutre Reservation (Feb 1968) notes the Average Water Surface for this section of the MRGO at 0.75 ft MSL. The maximum expected hurricane surge (SPH) is 15.0 ft MSL.

Preliminary Findings

Maps of datum/adjustment differences (project area with values)

The following figures show the relationship between NGVD 29(1991) and NAVD88 2004.65 elevations differences, the NGVD 29(1991) and NAVD 88(1994/1996) elevation differences, and the NAVD88(1994/1996) and NAVD88 2004.65 elevation differences at selected control monuments.

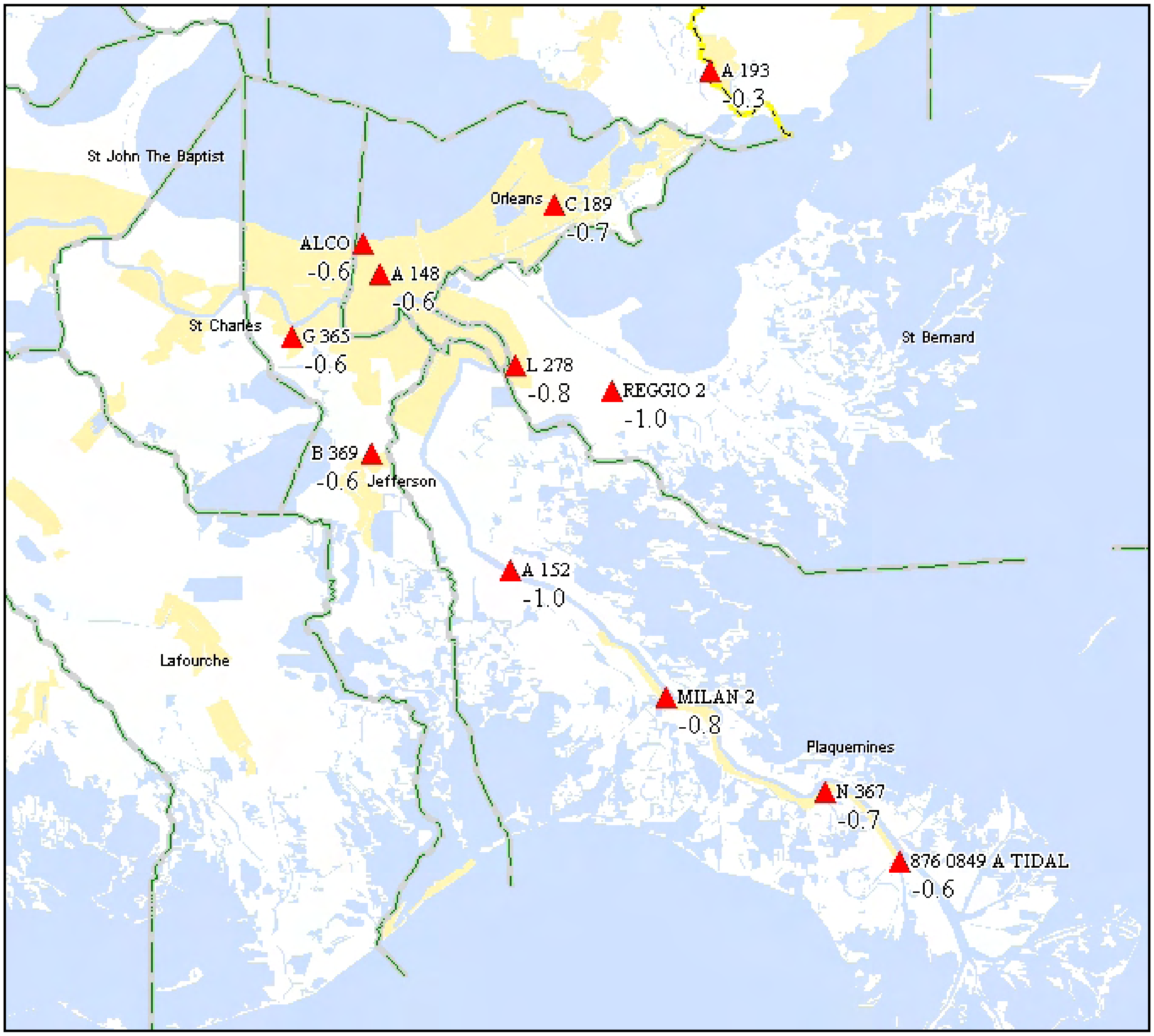


Figure III-19. Elevation Difference between NGVD29(1991) and NAVD88(2004.65) at select control monuments (values in feet)

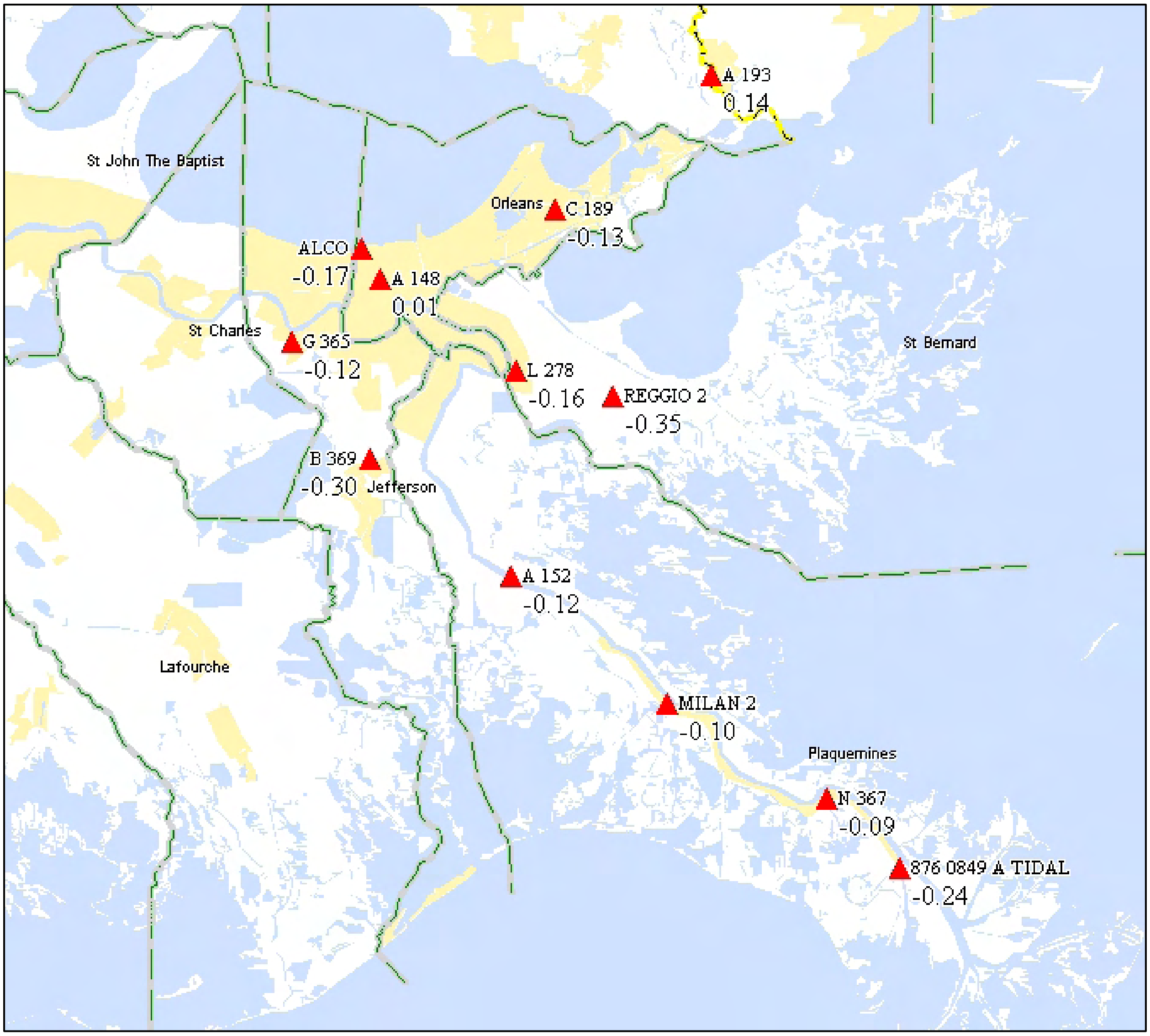


Figure III-20. Elevation Difference between NGVD29 (1991) and NAVD88(1994/1996) at select control monuments (values in feet)

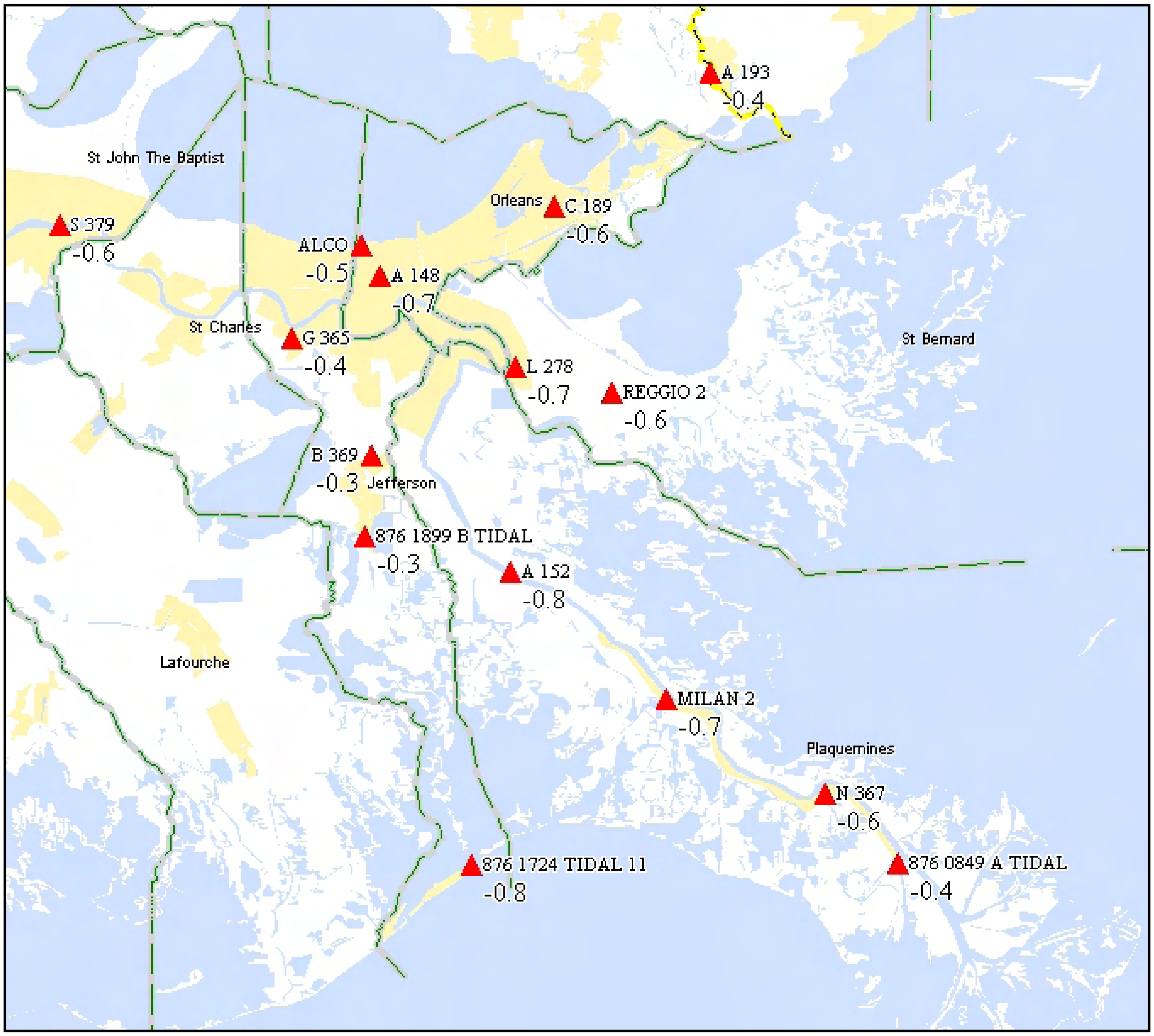


Figure III-21. Elevation Difference between NAVD88(1994/1996) and NAVD88(2004.65) at select control monuments (values in feet)

Preliminary relationships between the LMSL and NAVD88 2004.65

The preliminary results show the LMSL is almost a constant level surface above NAVD 88_2004.65 (the current geodetic datum). Two anomalies were noticed in the preliminary results. The LMSL above the current geodetic datum 8761602 Lake Judge Perez, Hermitage Bayou was 0.1 feet higher than the other stations. This is because the station is located far into the bayou above Barataria Bay. The range of tide will decrease significantly here (0.42 feet) and the presence of the land will force the water to slightly rise. This raises the LMSL. The LMSL at 8761678 Michoud Substation, Intercoastal Waterway is a half-foot lower than the other water level stations. The contractor went back into the field 21 and 22 February 2006 to measure from a different monument (WES 19 1978) here. The results were almost identical; the LMSL is 0.4 feet below the current geodetic datum and 0.5 feet lower than the LMSL at the other water level stations. This maybe either a hydraulic event from the canals and locks, or this is due to a large intake of water at the Michoud Substation.

The following figures show the preliminary relationship between the LMSL and the NAVD88 2004.65 datum.

Figure III-22. Map of Tide Stations and Values from NOAA CO-OPS Showing the height of the LMSL above NAVD88 2004.65 values (all values in feet)

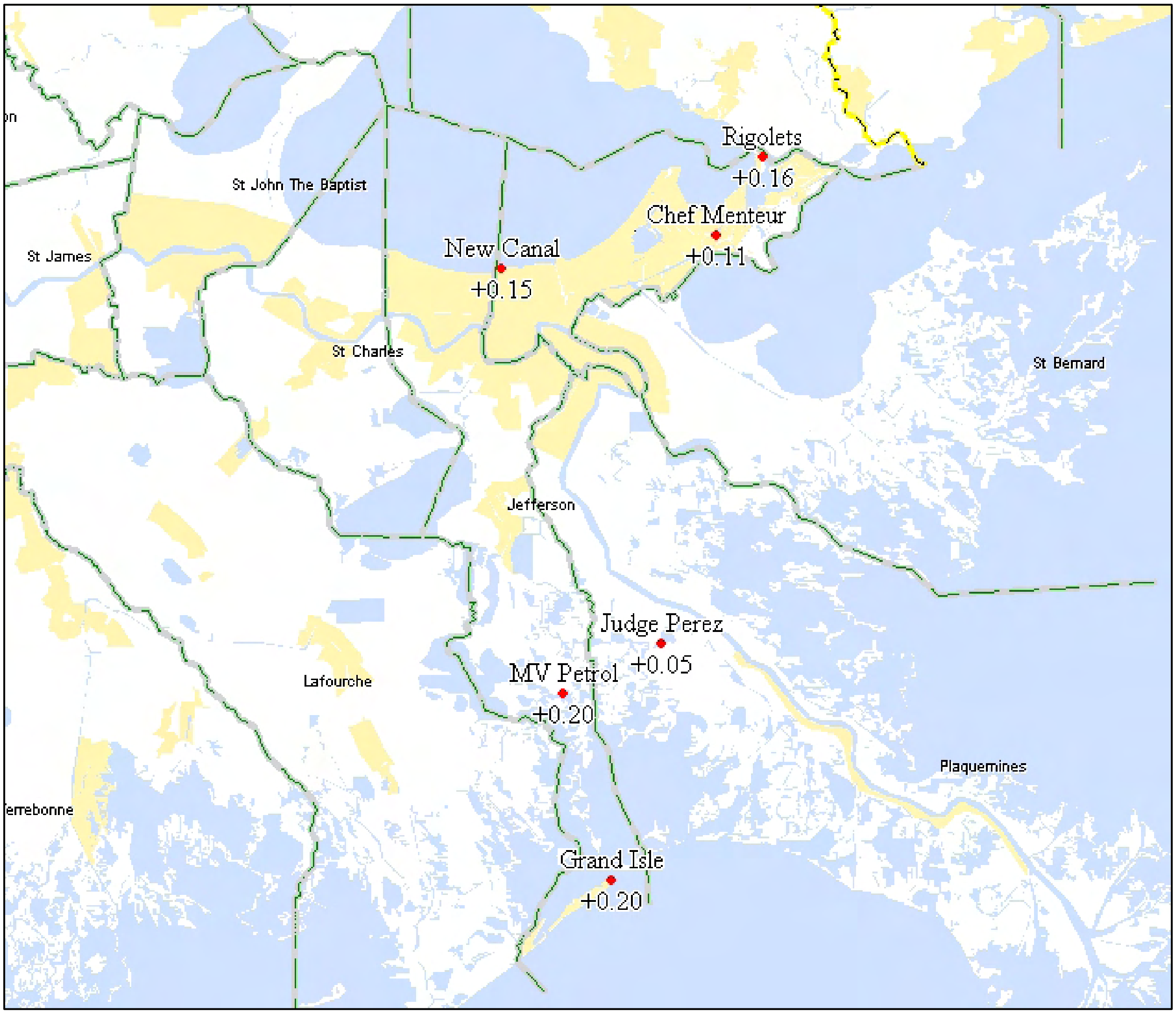


Figure III-23. Map of Tide Stations and Values from ERDC-TEC Showing the height of the LMSL above NAVD88 2004.65 values (all values in feet)

Example Datum shifts & Local Mean Sea Level relationship to the datum over time

The following figure shows the changes in the elevation values at Benchmark ALCO 1931 from 1952 until present including an elevation of LMSL in 2005. The changes in elevation are due to various adjustments on the datums and a datum shift (between NGVD29 and NAVD88).

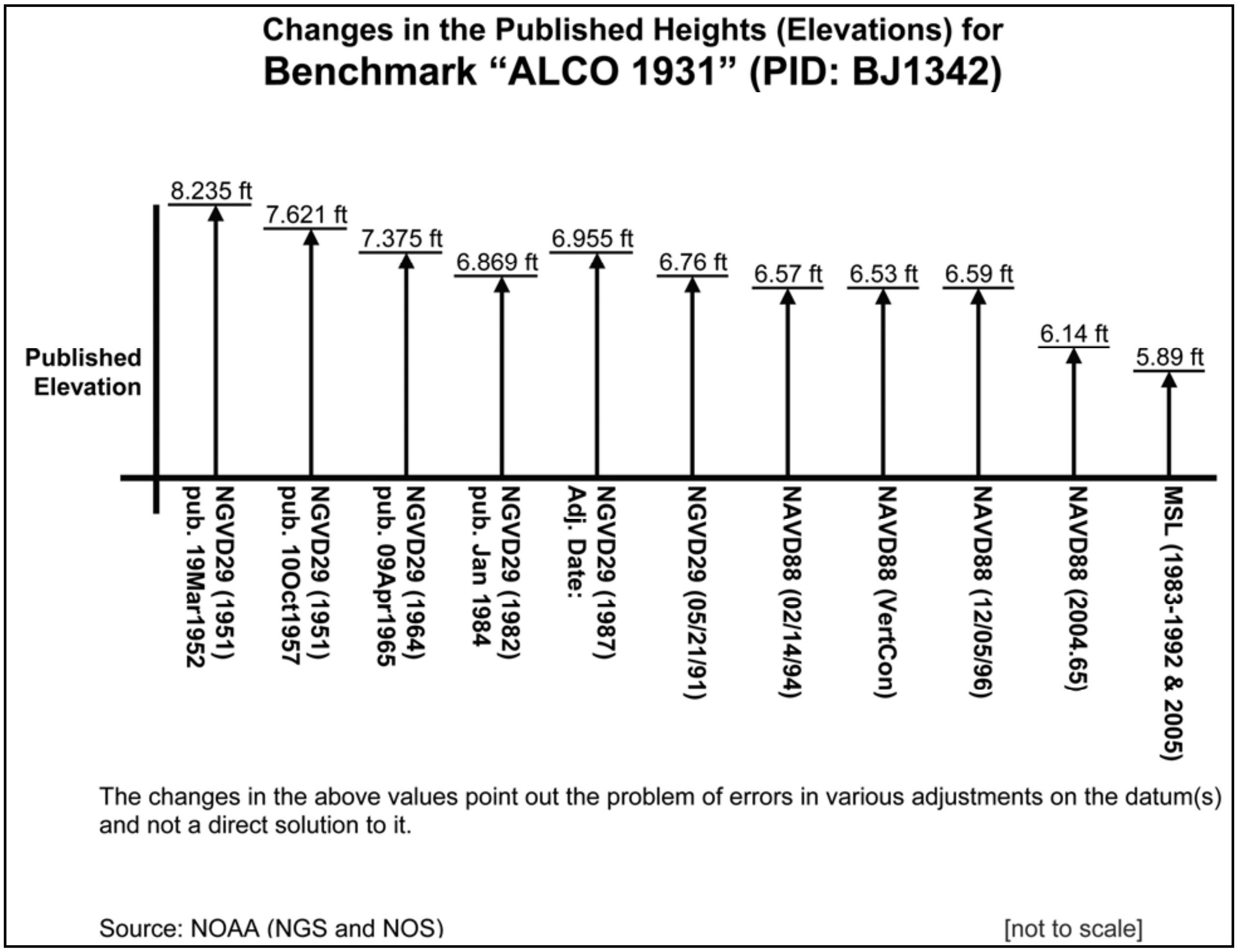


Figure III-24. Elevation changes at Benchmark "ALCO 1931" since 1951

Preliminary Methodology (Procedures) for conversion of previous vertical datum/adjustments to NAVD88 2004.65

The methodology used to shift historical survey data to NAVD88 (2004.65) will vary dependent upon many factors such as time, funds, accuracy requirements, etc. Generally there are four methods to determine the datum/epoch shift.

1. Field Measurements w/ Known Historical Elevation: This method will yield the most accurate values based on the historical reference marks. The reference marks will need to be recovered and occupied/surveyed using the guidelines in NGS 58. The difference between the elevation used for the original survey and the elevation established from the new network will directly tie in the old work to the latest control. This will not account for any differential subsidence that occurred between the reference mark and the survey positions.

2. Field Measurements w/o Known Historical Elevation: When the reference benchmark is not recorded and unknown, some assumptions will be required such as what mark was used and what its elevation was. Again follow the procedures in NGS 58 to establish new elevations on the reference mark. The historical elevation will have to be assumed based on what was available at the time of design. The difference between the assumed historical elevation and the newly established elevation will be used to shift the survey to the new datum/epoch.

3. Common Published Marks in Survey Area: When time and money are constraints, the closest marks with published elevations in both datum/epochs can be used to determine an average shift for the area. This method contains many assumptions and therefore is the least accurate but may be of some use for projects that don't require accuracy.

4. CORPSCON: This method does not account for subsidence or the change in elevation from epoch to epoch. CORPSCON model was also tied to the published elevations at the time the model was created which contained errors associated with the already deteriorating elevation accuracies. This method should not be used for anything other that a pure datum shift keeping in mind that subsidence is not accounted for.

Summary of Findings and Recommendations

This section summarizes tentative recommendations based on findings and lessons learned from this portion of the study. These draft recommendations are subject to additional review and consideration by IPET Geodetic Vertical and Water Level Datum Team interagency members and other external reviewers.

Dual Elevations on Flood Control and Hurricane Protection Structures

Finding: Design and construction documents referenced both geodetic datums (e.g., NGVD29) and water level datums (e.g., MSL) without defining the geographical relationships, numerical differences, observation epochs, or other significant metadata associated with these datums. In most cases, NGVD29 was incorrectly assumed as an equal elevation to MSL.

Recommendation: Planning, design, construction, and operation & maintenance inspection documents containing elevation data on flood control structures should show both geodetic and water surface referenced elevations. The relative water surface reference datum (i.e., LMSL) should be used as the baseline for hydraulic modeling and related levee height design computations. The terrestrial geodetic datum should be used for construction stake out and subsequent periodic subsidence modeling. The base gage defining a water level datum must be clearly defined, along with applicable tidal or river stage epochs, and conversion parameters to relate water level datums to the local geodetic datum.

Geospatial Data Source Feature or Metadata Records

Finding: Design and construction documents seldom identified the source of hydrographic, topographic, or construction survey records, including water level gage records.

Recommendation: Planning, design, and construction documents containing survey information should contain detailed source (i.e., metadata) information on geospatial coordinates or terrain models included in those documents. This would include the location and repository for the original source data, field book numbers, monument descriptions, etc. Geospatial metadata incorporated in documents shall have sufficient detail such that there is no uncertainty (currently or in the future) as to the location of the original data, its origin, and other temporal relationships.

Epoch Designations of Published Topographic Elevations

Finding: Design and construction documents seldom identified the epoch associated with a particular datum. This is especially critical in a high subsidence area where apparent sea level rise (i.e., combined sea level rise with subsidence) can have significant changes over a relatively short period.

Recommendation: Reported elevations of surface topography, subsurface bathymetry, and/or constructed structures in high subsidence areas should contain feature (metadata) information on the source datum and applicable adjustment epoch date. This applies to both geodetic elevations (e.g., 12.345 ft NAVD88 (2004.65)) and water level based elevations (e.g., (-) 5.25 ft LMSL (2001-2005) or 35.0 ft MLLW (1983-2001) or 12.3 LWRP (1974)). Hard copy or CADD data files should place this metadata information in the General Notes on the first sheet of a series, with appropriate references on subsequent sheets that depict topographic information.

Future Updates to NAVD88 in New Orleans Region

Finding: Geodetic elevations are extremely time-dependent in this region and must be periodically adjusted to account for apparent sea level changes.

Recommendation: The current (2004.65) adjustment to the “time-dependent” (VTDP) NAVD88 network for the Southeast Louisiana area should be periodically reviewed for subsidence relative to the nationwide spatial reference system. This review should be performed annually by the NOAA National Geodetic Survey (NGS) using CORS observations and other applicable geodetic sources. When periodic reviews by NGS indicates average elevation changes in the VTDP network exceed 0.05 ft, then actions should be taken to revise and update the time-stamped NAVD88 VTDP network for this region. This update should be performed at least every 5 years regardless of elevation changes. NGS must closely coordinate subsequent updates with the Corps of Engineers and other federal, state, parish, levee board, and other local agencies to ensure that engineers and others responsible for the planning, design, and construction of flood control structures are made aware of the revised adjustments. These subsequent adjustments must also be closely coordinated within NOAA to ensure CO-OPS water level datum references are appropriately revised to reflect any geodetic datum revisions.

Additional Co-located CORS and NWLON Sites for Subsidence Monitoring

Finding: There is an insufficient density of subsidence and water level monitoring points to adequately evaluate current flood protection elevation elevations of control structures.

Recommendation: NOAA should establish subsidence and water level monitoring instrumentation at the following sites in Southeast Louisiana by NOAA. These sites will be used to monitor future land subsidence and reference water level datums, as required to assess and update protection elevations of flood control structures throughout the region. Each site should contain complete NOAA quality CORS GPS and NWLON gage instrumentation.

1. Lake Pontchartrain (USCG Station--17th Street Canal—NOAA New Canal gage site)

2. Lake Pontchartrain (East end—The Rigolets or Chef Menteur area—NOAA gage sites)

3. IHNC (Corps of Engineers Lock—existing gage site)

4. GIWW-MRGO (Michoud Substation area—NOAA gage site)

5. Lake Borgne (New Shell Beach area)

6. Venice, LA (New Orleans District Project Office)

7. Mississippi River (Carrollton gage site-New Orleans District Office)

New Orleans District Water Level Gages

Finding: There is an insufficient density of subsidence and water level monitoring points to adequately evaluate current flood protection elevation elevations of control structures.

Recommendation: To provide additional surface modeling coverage, New Orleans District gages (and those maintained by the USGS, NWS, levee boards, and others) should be connected and referenced to NAVD88 (2004.65), or the latest geodetic datum published by NGS. New Orleans District should make modifications to District-owned gages to meet NOAA NWLON specifications and include these gages in the NWLON.

Local Mean Sea Level Epoch Updates and Relationships

Finding: 19 year updates to LMSL computations is too long an interval in this high subsidence area.

Recommendation: LMSL epochs should be periodically updated by NOAA CO-OPS in order to monitor subsidence and/or apparent sea level rise at NWLON gage sites. Five-year tidal epochs should be computed and reevaluated yearly, and apparent sea level rise estimated for NWLON gages in the area. CO-OPS should perform these periodic evaluations in close coordination with New Orleans District hydraulic engineers (CEMVN-EH). The New Orleans District should reassess gage datums on non-NWLON gages on an annual basis, in close coordination with CO-OPS reevaluations and updates. NOAA CO-OPS should develop and publish an operating manual specific to the process of maintaining water level datums in this Southeast Louisiana region.

Mean Sea Level and Local Mean Sea Level

Finding: These two terms should not be used interchangeably.

Recommendation: When referring to the mean water surface at or near a specific flood control project, LMSL should be used. A LMSL derived elevation should clearly identify the water level reference gage location and the time series (epoch) over which the mean surface elevation was computed.

Coordination of Topographic Survey Data Collection, Processing, and Management

Finding: A variety of topographic survey data is produced by various elements within and outside the New Orleans District, primarily by contracted surveying and mapping firms. Given this dispersion, locating datasets is a difficult process.

Recommendation: The New Orleans District should develop a comprehensive GIS system to maintain hydrographic, topographic, and geodetic data collected by various engineering, construction, and operations entities within and/or external to the District. Data formats should be standardized based on existing Corps guidance—e.g., CADD/GIS Technology Center, EM 1110-1-1005 (Topographic Surveying), etc.

Vertical Control Monumentation Requirements and Stakeout Procedures on Flood Control Construction Projects

Finding: Most construction contract documents reference only one benchmark for controlling construction.

Recommendation: A minimum of three (3) permanent benchmarks should be identified on design and construction drawings for all flood control projects. These marks should be established during the planning and design phase. The marks shall be situated at each end of the project. They shall be established relative to existing NAVD88 (20XX.XX) control established by the NGS, using either conventional differential leveling and/or the latest NGS-approved differential GPS network observations. Prior to and during actual construction stake out, these primary reference marks should be verified externally and internally. Field records of these survey verifications shall be permanently archived.

LIDAR and Photogrammetric Mapping Calibration and Testing

Finding: Various LIDAR mapping projects covering the region were not independently ground truthed for absolute accuracy.

Recommendation: Contracts for aerial mapping services shall contain quality assurance provisions for calibrating, ground truthing, and testing delivered mapping products. These methods should follow long-established testing methods outlined in standards such as USACE EM 1110-1-1000 (Photogrammetric Mapping), FGDC, ASPRS, and FEMA.

USACE Policy and Manual on Maintaining Geodetic and Water Level Datums in High Subsidence Areas

Recommendation: USACE ERDC should develop an Engineering Manual (or an addendum/update to the Coastal Engineering Manual) providing theory, guidance, & procedures on maintaining reliable reference datums in high-subsidence areas, including distinguishing engineering applications between water level and geodetic datums. Alternatively, this guidance may be implemented by a policy document (Engineering Regulation).

Differential GPS Survey Standards for Establishing Construction Control

Recommendation: NGS procedures shall be used for establishing supplemental orthometric elevations using GPS. NGS shall develop and promulgate specific operating procedures applicable to this high-subsidence area. These procedures should include methods of determining orthometric elevations relative to local VTDP benchmarks as well as methods for direct establishment of orthometric elevations from CORS stations. Both geodetic accuracy and construction accuracy methods should be covered. Required accuracies are outlined in EM 1110-1-1005.

Supplemental Field Survey Support to Other IPET Teams

This section summarizes topographic survey support performed by the IPET Survey Team in support of modeling requirements needed by other IPET study teams. Approximately 75% of Team 6's field survey work involved support to other IPET Teams. These surveys were performed concurrent with the primary geodetic control surveys connecting NOAA NWLON gages. Field survey operations began in early December 2005 and are still in progress as of the end of February 2006. Surveys were performed throughout the entire study area: Orleans, St. Bernard, Plaquemines, St. Charles, and Jefferson Parishes.

Field survey operations were performed by 3001 Inc., a Louisiana based surveying company. This firm was under an Indefinite Delivery Contract to St. Louis District. St. Louis District awarded a labor-hour type task order to 3001 Inc. on 5 December 2005. IPET Team members Bill Bergen (HQUSACE) and Jeff Navaille (Jacksonville District) arrived in New Orleans on 4 December 2005 and began working out of the New Orleans District Office. Initial efforts involved controlling pump stations, high water mark (HWM) locations, and NOAA NWLON tidal gage sites, which included setting benchmarks for subsequent GPS connections to the NGS NAVD88 (2004.65) reference network. The first 3001 Inc. survey crew arrived in New Orleans on 11 December 2005 and began static GPS surveys for benchmarks at pump stations and priority HWM sites. Three 3001 Inc. survey crews were fully operating by 14 December 2005 and continued working on the various tasks outlined below through 23 December 2005. Survey operations resumed on 3 January 2006 and are continuing at this date.

The following list summarizes various field survey projects performed from 5 December 2005 through February 2006. The supported IPET model is shown in parenthesis.

- High Water Mark Surveys: Leveling to approximately 50 HWM points plus 2,000 ft of levee profile surveys along a five (5) mile levee in St. Bernard Parish (IPET Numerical Storm Surge Models)

- High Water Mark Surveys: Interior Orleans Parish—levels to various residential locations (IPET Numerical Storm Surge Models)

- High Water Mark Surveys: Plaquemines Parish—levels to various locations (IPET Numerical Storm Surge Models)

- Surge Elevation Surveys: Orleans Marina & Lakefront Airport—levels to time-stamped Katrina storm surge points (IPET Numerical Storm Surge Models)

- Bridge Surveys: Low-chord elevation and obstruction surveys (IPET Numerical Storm Surge Models)

  Orleans Outfall Canal: 4 auto bridges
  London Ave Canal: 1 RR bridge and 6 auto bridges

IHNC: 3 RR bridges
17th St Canal: 5 auto bridges

- Pump Station Control Surveys: Approximately 69 pump station first floor elevations throughout Orleans, Jefferson, St. Bernard, and Plaquemines parishes (IPET Pump Station Performance Assessment)

- Pump Station Control Surveys: 5 pump station first floor elevations in St. Charles Parish (IPET Pump Station Performance Assessment)

- Lake Pontchartrain Water Level Gage GPS Surveys: Tie in reference marks on eight (8) USGS, NWS, and levee board gages in the vicinity of Lake Pontchartrain and the IHNC(IPET Numerical Storm Surge Models)

- IHNC West Bank Levee Profile Surveys: SeaLand/Maersk Private Levee (IPET Numerical Storm Surge Models)

- IHNC West Bank Breach Area Topographic Surveys: Florida Ave to I-10 Bridge (IPET Interior Drainage Modeling)

- Ground Truthing/Calibration Surveys of Low-Altitude 2000/2005 LIDAR DEMs: (IPET Data Management)

- Ground Truthing/Calibration of High-Altitude JALBTCX 2005 LIDAR: North shore of Lake Pontchartrain (JALBTCX & IPET Data Management)

- Ground Truthing/Calibration of High-Altitude FEMA/LSU LIDAR: Selected side shot calibration points throughout region (IPET Data Mamagement)

- Hydrographic and Topographic Canal Cross-Sections: Selected sites in Jefferson & Orleans Parishes (IPET Interior Drainage Model)

- Levee/Floodwall Overbank Cross-Sections: London Avenue, 17th Street, & IHNC Breach Sites: (IPET Physical Model of Breaches & IPET Floodwall Performance Analysis)

- Interior Drainage Topographic Sections: Approximately 85 cross-sections at selected locations throughout St. Bernard Parish (IPET Interior Drainage Support)

- Invert Elevations: London & Orleans Outfall Canal pump stations (IPET Numerical Storm Surge Models)

- TBM Descriptions: Stable and recoverable marks to be documented and described in accordance with New Orleans District procedures (MVN/Task Force Guardian)

- Orleans Outfall Canal BM ALCO to CHRYSLER Level Run (IPET Survey Team)

- IHNC Hydrographic Multibeam Survey: Seabrook Bridge to GIWW and GIWW to Mississippi River (IPET Storm Surge/Wave Hydrodynamics)

- High Water Mark Surveys: Orleans Parish vicinity Ninth Ward—levels to various locations (IPET Numerical Storm Surge Models)

Field Survey Procedures and Specifications

All field surveys for supplemental topographic work were performed following established Corps of Engineers and NOAA standards and specifications.

Static GPS surveys were performed to set permanent or temporary benchmarks throughout the five-Parish area. Supplemental topographic surveys were performed from these benchmarks to HWMs, pump stations, floodwalls, etc. Over 100 benchmarks have been established to date.

These static GPS surveys were rigorously connected to the NGS approved NAVD88 (2004.65) network. Procedural GPS survey methods followed (and actually exceeded) the guidelines in the following NOAA publications:

- **NOAA 1997.** NOAA Technical Memorandum NOS NGS-58, Zilkoski, D.B., D'Onofrio, J. D., and Frankes, S. J. (Nov 1997) “Guidelines for Establishing GPS-Derived Ellipsoid Heights (Standards: 2 cm and 5 cm),” Version 4.1.3. Silver Spring, Maryland.

- **NOAA 2005.** “Guidelines for Establishing GPS Derived Orthometric Heights (Standards: 2 cm and 5 cm)” version 1.4, National Geodetic Survey (2005 DRAFT)

Procedural specifications applicable to topographic engineering and construction surveys included:

- **EM 1110-1-1003** NAVSTAR Global Positioning System Surveying

- **EM 1110-1-1005** Control and Topographic Surveying (1 January 2006 Draft)

The above guidance documents also contain the accuracy standards required for hydraulic modeling type surveys involved on these projects. In general, required vertical accuracy tolerances were ± 0.1 foot. Horizontal accuracy varied depending on the nature of the survey—e.g., HWM horizontal locations are not as critical as floodwall cap locations.

Topographic surveys were performed using all of the following methods and equipment:

- Conventional differential leveling (spirit/compensator/digital levels)
- Electronic total stations

- Static Differential GPS surveys
- GPS real time kinematic (RTK) methods

Field survey data was collected in a standard bound survey book and/or on an electronic data collector attached to or part of a total station or RTK survey system. Digital images were taken for HWM and pump station first floor elevation shots.

Figure III-25. (Left) Static GPS survey to establish elevation on a benchmark outside a St. Bernard Parish pump station. (Right) Leveling first floor elevation inside Jefferson Parish Pump Station No. 3.

All of the above manuals were cited in the St. Louis District task order specifications.

Hydrographic surveys, including multibeam surveys, were performed following the guidance for Special Surveys (i.e., non-navigation/dredging surveys) in: **EM 1110-2-1003** Hydrographic Surveying

Figure III-26. (Left) IHNC Almonaster Bridge—low chord elevation 3.51 ft NAVD88 (2004.65). (Right) Leveling to USGS recording gage and Orleans Levee District staff gage on I-10 bridge over IHNC

Data Processing and Submittal

The contractor processed and reduced all survey data to a submittal format consistent with EM 1110-1-1005 and the New Orleans District. GPS baselines were reduced and networks adjusted using standard COTS software packages — e.g., Trimble Geomatics Office. Data submittals were posted on an ERDC ftp site for transfer to the requesting IPET Team.

All data submittals contain supplemental metadata records that are compliant with the Federal Geographic Data Committee Standard "Content Standard for Digital Geospatial Metadata", FGDC-STD-001-1998.

Quality Control and Quality Assurance Procedures

The survey contractor (3001 Inc.) was responsible for performing quality control over all work performed, in accordance with the Quality Control Plan submitted on award of the basic Indefinite Delivery Contract. Many of the specifications listed above provide forms of quality control by requiring specific observing schemes, redundant observations, connection checks between control points, closed loop level lines, periodic RTK calibration checks, level peg tests, etc. The contractor was expected to perform additional quality control checks during data processing and prior to submittal.

Quality assurance checks were performed by both the contractor and government (IPET Survey Team). GPS observations establishing supplemental vertical control points were checked by running independent solutions from NOAA CORS stations distant from the NAVD88 (2004.65) project network. This afforded a blunder check on all points. The government performed spot checks on data submittals, including reality checks by modelers receiving the data.

A few isolated survey data errors or blunders were found by both the contractor and government, indicating a quality control/assurance process was in place.

Quality assurance is still in progress as of this Interim Report.

IV. The Hurricane Protection System

Executive Summary

This part of the report is an initial attempt to provide a comprehensive characterization of the Hurricane Protection System and the design assumptions used in its development. The first part provides a general description of the HPS including the distribution and character of the structures and features that comprise the system. This information is provided on a Parish by Parish and reach by reach basis as available. The degree and types of damage suffered from Katrina is also documented along with the repair strategies currently underway. This information is to serve as a system wide description for the overall IPET analysis.

A discussion is presented on the Standard Project Hurricane used for the design of the HPS. This includes how it was defined, assumptions made in determining the design elevations necessary to protect against the SPH, and the factors considered in arriving at these design criteria. This includes a reach by reach documentation of the surge and wave factors to include wave runup and freeboard, resulting in the design elevations. A detailed documentation is provide on the history of the 17th Street Outfall Canal component of the HPS. This provides a chronology of the key documents and communications that led to the system in place prior to Katrina. It serves as an example of the types and level of information that will be included for the remainer of the HPS in the IPET final report.

The scope of this part of the report as presented here represents only a portion of the information intended for the final report. In the IPET final report there will be additional information concerning the specific structural design assumptions, as-built and condition information to provide a more complete picture of the HPS. Some of this type of information is presented in the Part VI, Performance, of this report to include the geological information and geotechnical data available for the design and construction activities. This information currently focuses on the 17th Street Canal in support of the performance analysis of the breach site used as an example of the analyses to be provided for the major structures of the HPS.

Design Criteria for the System

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 Canal and Inner Harbor Navigational Canal (IHNC). Figure IV-1. is an index map showing the individual polders within the Lake Pontchartrain, LA and Vicinity HPP.

Plaquemines Parish Basin includes long, narrow strips of protected land on both sides of the Mississippi River between New Orleans and the Gulf of Mexico. The Mississippi River Levees (MRL) protect the Parish from floods coming down the river. Protection from hurricane induced tidal surges is achieved by the **New Orleans to Venice (NOV) HPP.** The NOV HPP is a system of levees on the gulf side of the protected lands and additional berms and floodwall on top of the MRL along the river. The NOV extends from Phoenix, LA to Venice, LA. A HPP map is not available for NOV however.

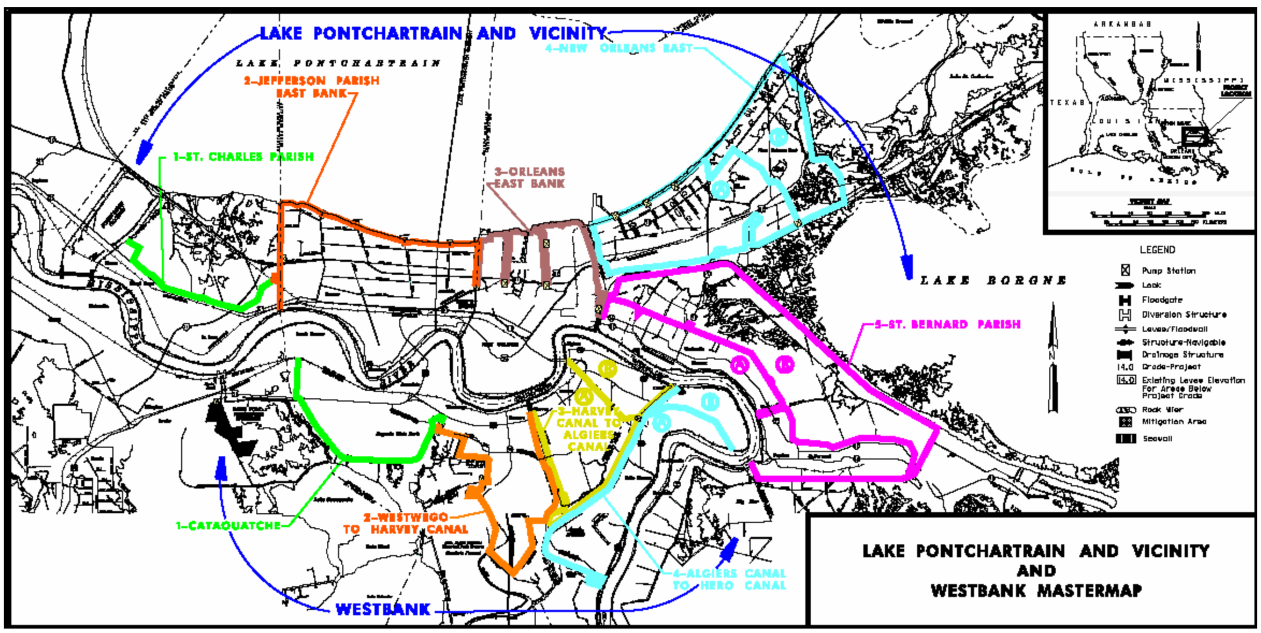


Figure IV-1. Index Map to Lake Pontchartrain, LA and Vicinity Hurricane Protection Project

Figure IV-2. Extent of NOV Hurricane Protection in Plaquemines Parish. The NOV consists of five distinct reaches; Reach C, Reach St. Jude to City Price, Reach A, Reach B-1 and Reach B-2.

Orleans East Bank – HPP Features. This portion of the project that protects the city of New Orleans was designed to protect 28,300 acres of urban and industrial lands. The levee portion is constructed with a 10-foot crown width with side slopes of 1 on 3. Along Lake Pontchartrain Lakefront the top elevation of the earthen levees range between elevation +13 and +18 ft National Geodetic Vertical Datum (NGVD). Floodwalls were designed to provide lines of protection on the east side of the 17th Street Canal, both sides of Orleans Avenue Canal and London Avenue Canal, and the west side of the IHNC. Floodwalls consist of reinforced concrete T-wall floodwalls and reinforced concrete I-wall floodwalls constructed on the top of sheet-pile, and sheet piling without a concrete section. Top elevations of the floodwalls vary between elevation +13 and +15 ft.

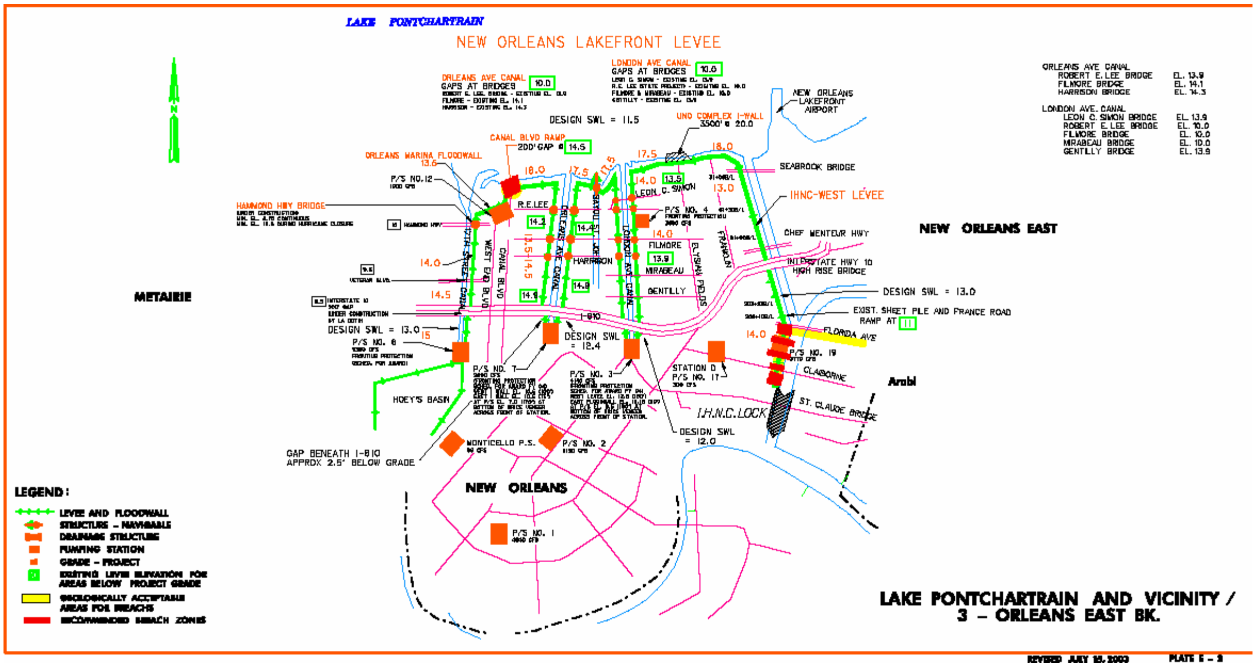


Figure IV-3. HPP features – New Orleans East Bank

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

IHNC Canal (West Bank). The Inner Harbor Navigation Canal is located in the east portion of Orleans Parish and is described in the IHNC section of this report.

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

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.

Orleans Avenue Canal. The Orleans Avenue Canal extends about 2.4 miles from Pumping Station No.7 in the vicinity of I-610 to its mouth at Lake Pontchartrain.

Table IV-1 New Orleans East Bank Hurricane Protection System	
19.2 miles	levee and floodwall
13	pump stations
15	roadway floodgates

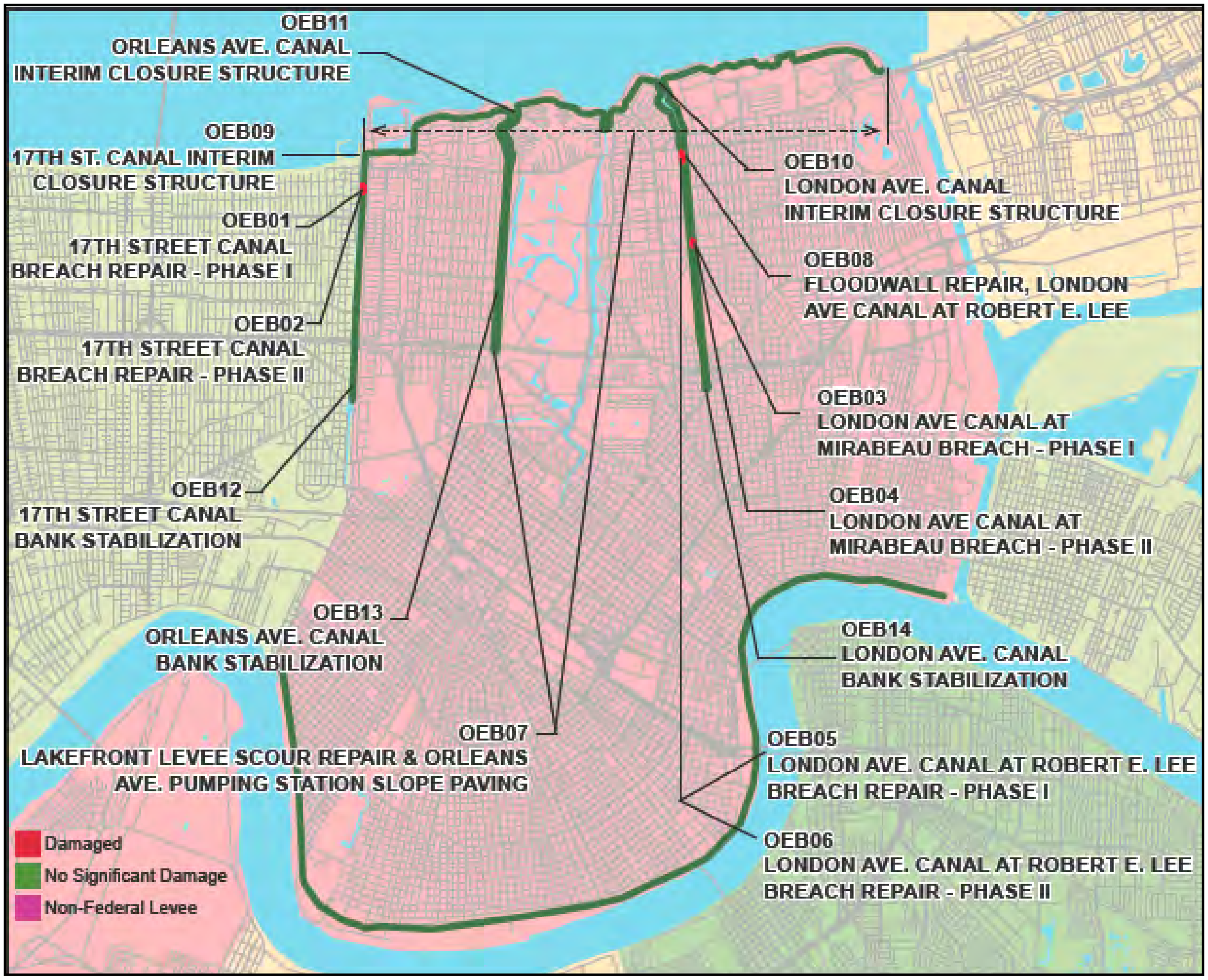


Figure IV-4. Damages and Repair Contracts – New Orleans East Bank

Primary damages to the flood protection in the Orleans East Bank basin consists of a 455- ft breach in the east side I-wall along 17th St. Canal, breaches on both the east side (425 ft) and west side (720 ft) I-wall along London Ave. Canal, breaches along the west side of IHNC floodwall and damages to all fifteen pumping stations.

New Orleans East Basin. The hurricane protection system for the New Orleans East (NOE) Basin was designed as part of the Lake Pontchartrain, LA and Vicinity Hurricane Protection Project. The NOE portion of the project protects 45,000 acres of urban, industrial, commercial, and industrial lands.

Figure IV-5. illustrates the boundaries and basic flood protection components within the NOE Basin. The levee is constructed with a 10-ft crown width with side slopes of 1 on 3. The height of the levee varies from 13 to 19 ft. There are floodwall segments along the line of protection that consists of sheet-pile walls or concrete I-walls constructed on top of sheet-pile. The line of protection was designed to provide protection from the Standard Project Hurricane (category 3 hurricane).

Figure IV-5. 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 Southpoint), the eastern boundary of the Bayou Savage National Wildlife Preserve, and the north side of the Gulf Intracoastal Waterway (GIWW) (between the IHNC and eastern edge of the Bayou Savage National Wildlife Preserve). The main components are described in the next section moving clockwise through the basin, beginning at the Lakefront Airport and ending at the western end of the GIWW.

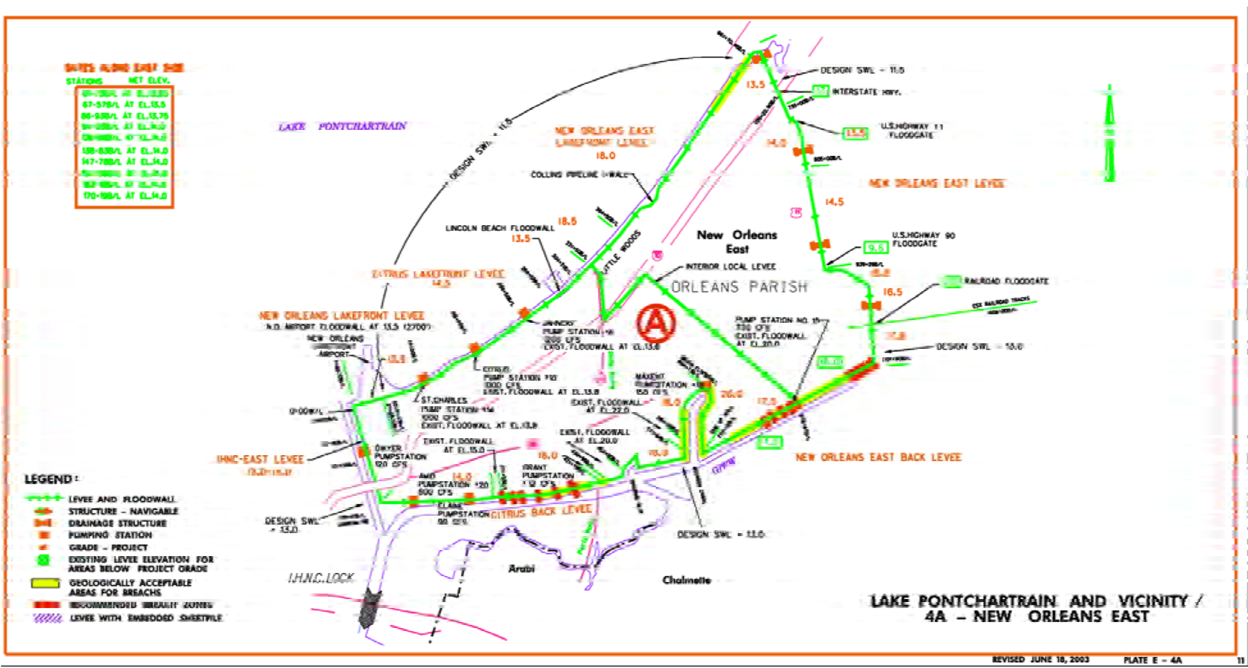


Figure IV-5. NOE Basin general components and top of levee/floodwall as-built elevations (feet) (source USACE, New Orleans District (Wayne Naquin)

Hurricane Protection Features New Orleans East Basin, Orleans Parish.

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

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

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

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

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

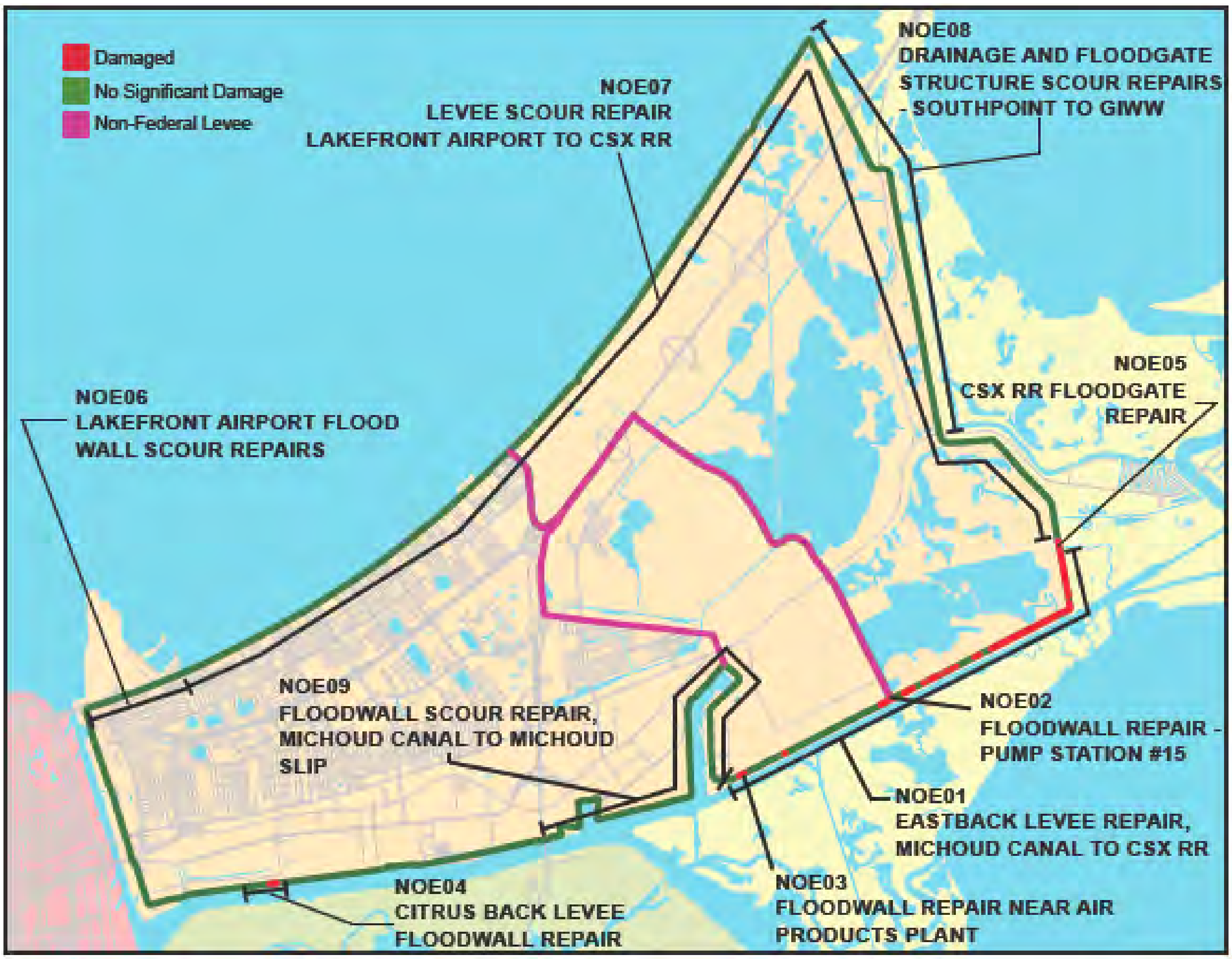


Figure IV-6. Hurricane Protection Features - New Orleans East Basin

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

<u>West and East Sides, IHNC, Orleans Parish.</u> 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

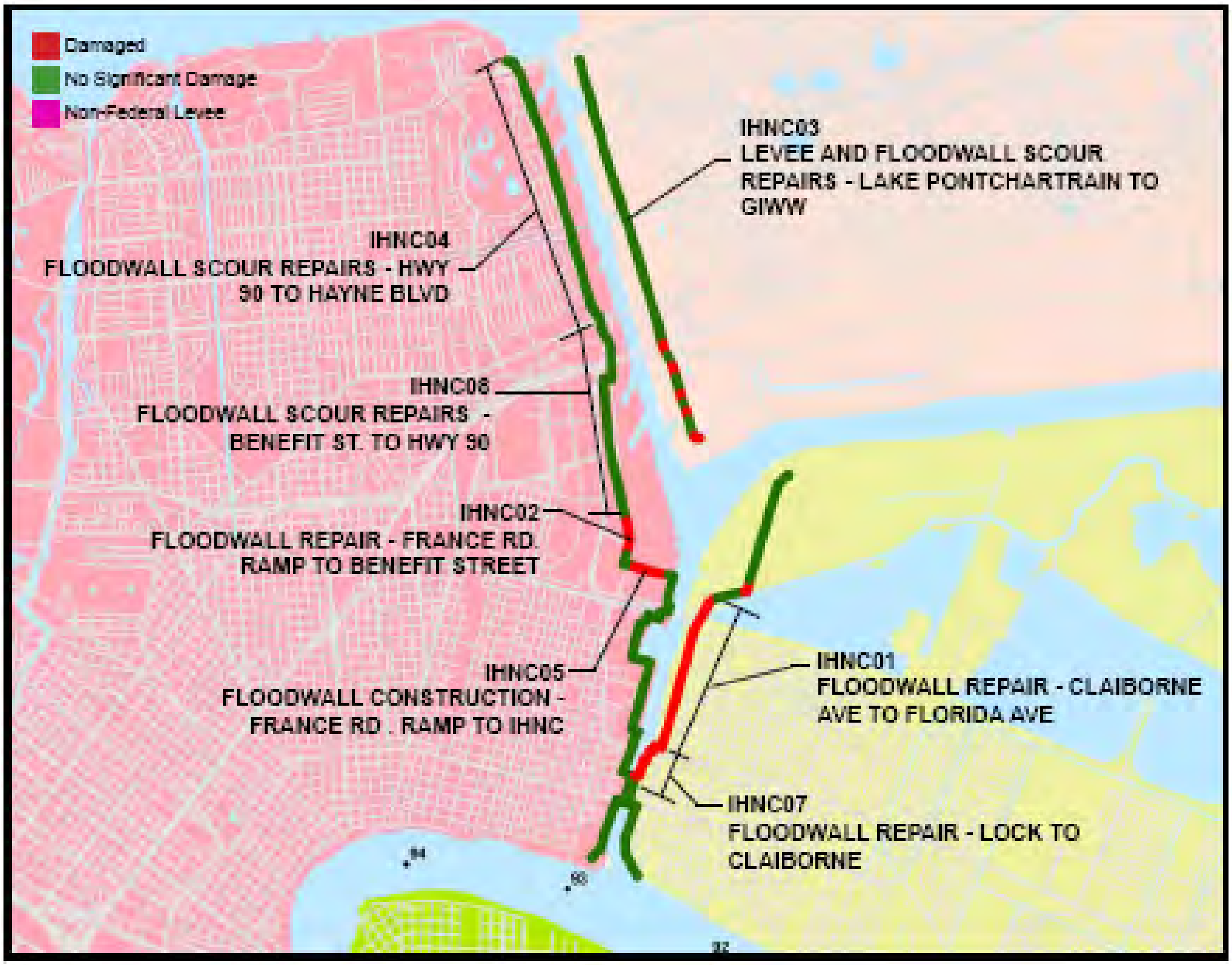


Figure IV-7. Hurricane Protection Features – IHNC

IHNC Damages. Overtopping of the hurricane protection by Hurricane Katrina was evident along nearly all portions of the canal. There were four breaches in the protection system, two on the east side and two on the west side.

The east side breaches are both located in the lower 9th ward neighborhood and the west side breaches are both in the vicinity of France Road and Benefit Street.

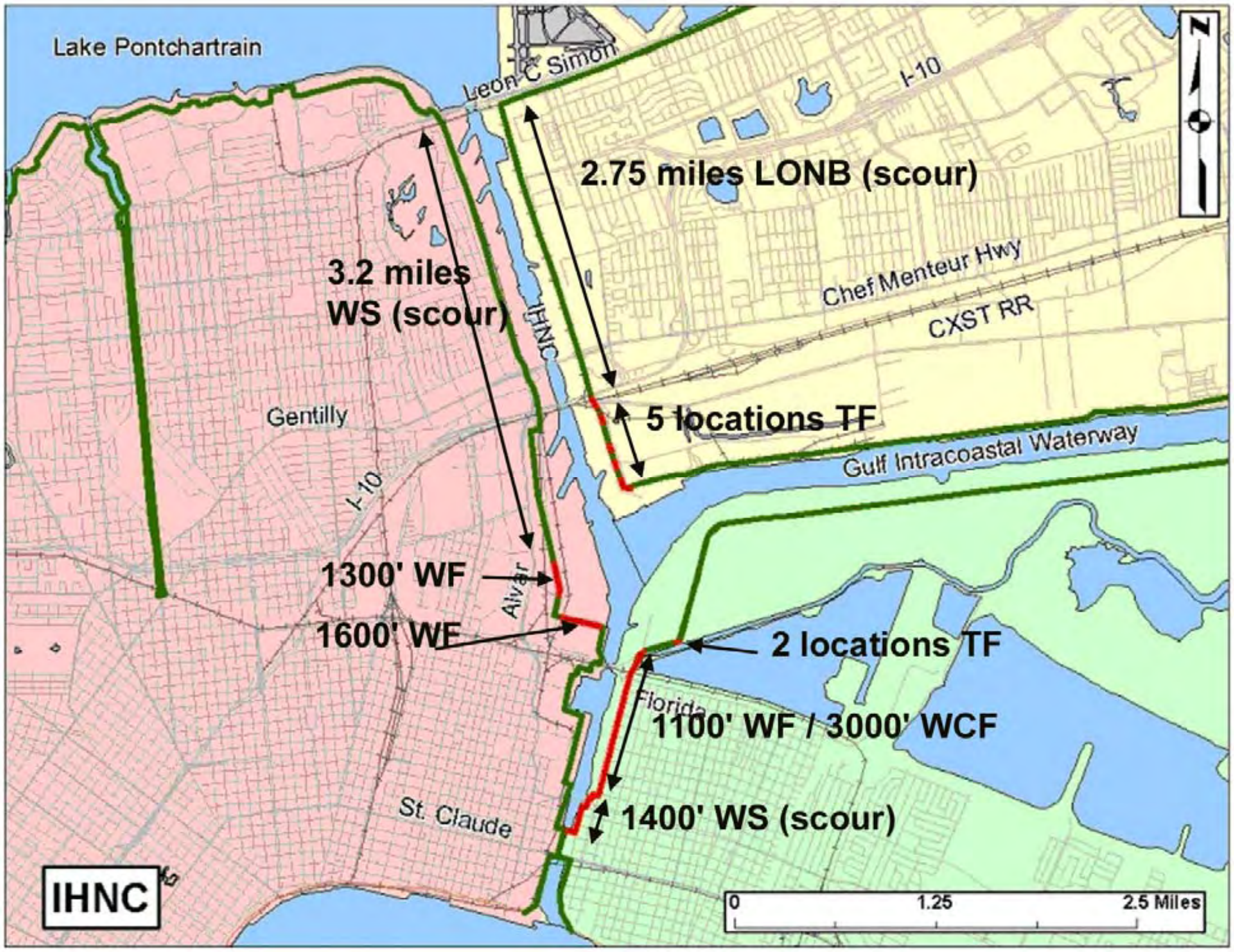


Figure IV-8. Damaged areas along the IHNC

Table IV-3 Hurricane Protection System for IHNC Hurricane Protection System	
12.3 miles	Levee and floodwall

St. Bernard Parish Basin. The St. Bernard Basin hurricane protection system includes the levee/floodwall extending from the Inner Harbor Navigation Channel (IHNC) easterly, along the Gulf Intracoastal Waterway (GIWW), to the Bayou Bienvenue Control Structure, continuing along the Mississippi River Gulf Outlet (MRGO) southeasterly, then turns generally to the west, where it ties into the Mississippi River Levee at Caernarvon, as shown on the map below. A portion of the hurricane protection system in this area also provides hurricane protection to the Lower 9th Ward area in Orleans Parish.

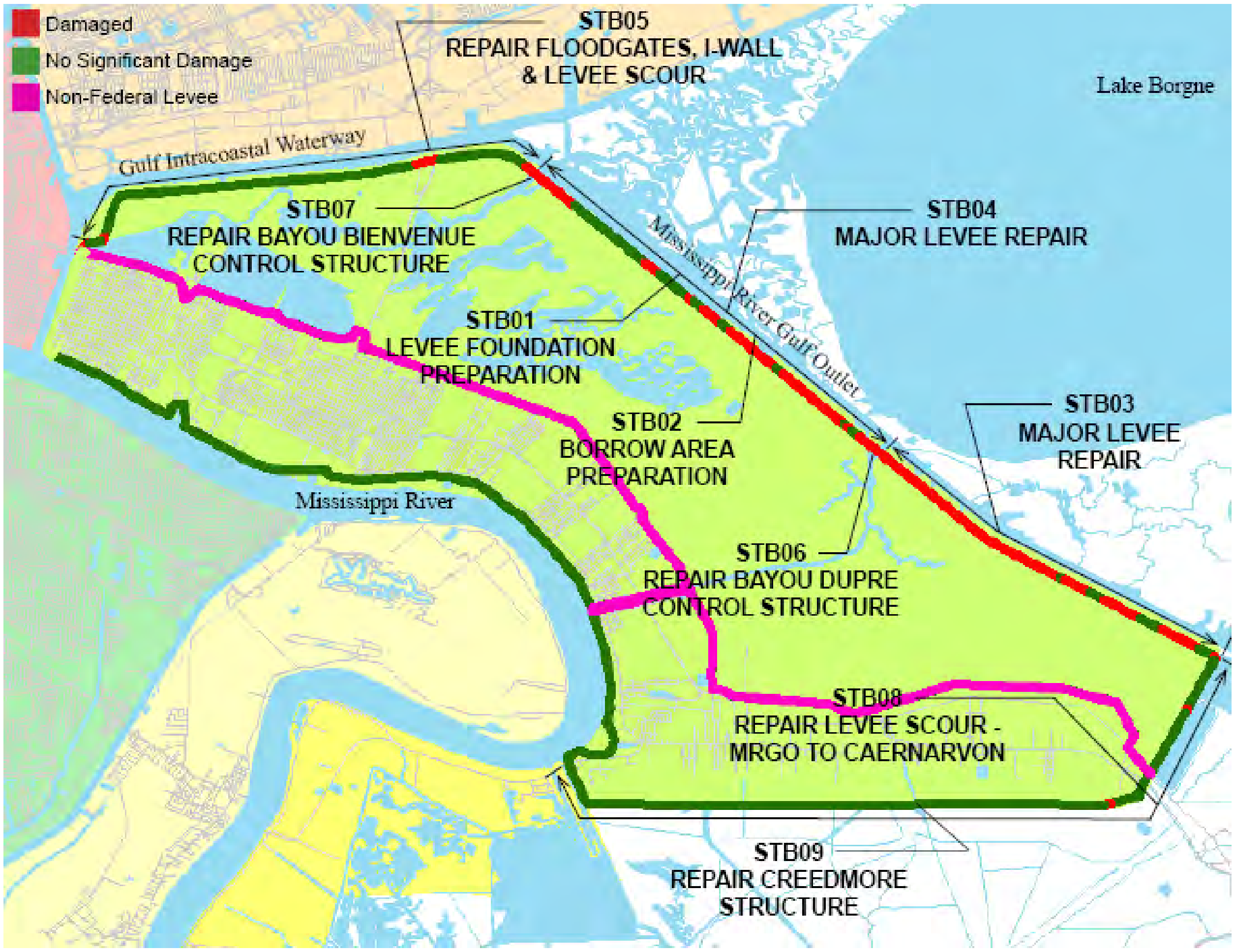


Figure IV-9. Hurricane Protection Project Features – St. Bernard

Table IV-4 **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

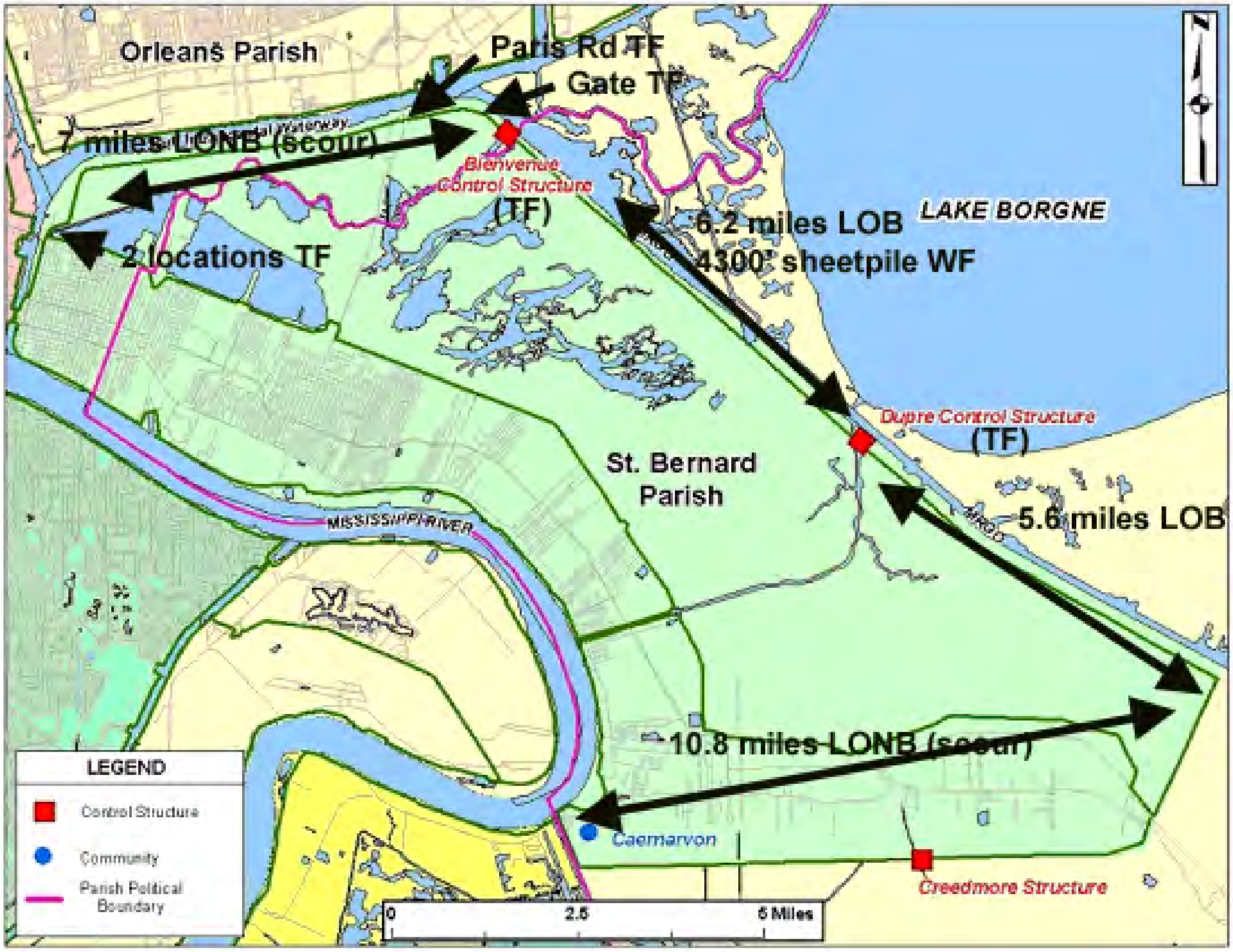


Figure IV-10.

<u>Plaquemines Parish Basin.</u> Altogether the Plaquemines Parish MRL and NOV systems include 162 miles of levee and 7 miles of floodwall. There are 19 non-federal pump stations for interior drainage. The levees are crossed by numerous pipelines, constructed in various manners. Some crossings bridge the levee without touching the embankment; some are constructed on top of the line of protection; and some pass through the line of protection with measures to prevent seepage. There is also a wicket gate closure on the back levee at Empire, where a shipping canal connects the Mississippi River to the Gulf of Mexico.

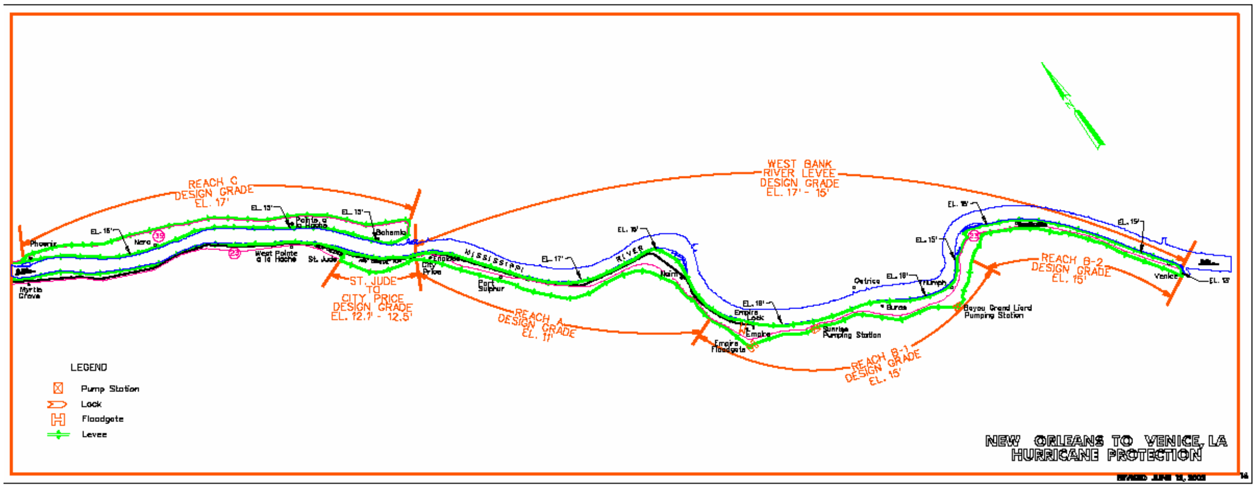


Figure IV-11. Hurricane Protection Project Features

Table IV-5 Summary Plaquemines Basin Hurricane Protection Features	
Mississippi River levee and floodwall	109 miles (34 miles part of NOV)
Floodwalls	6.4 miles
Hurricane Protection back levee	53 miles
Road Closure Structures	?
Numerous pipeline crossings	
Pump stations	19
Marine floodgate Empire	1

Design Criteria and Assumptions

Standard Project Hurricane

The Standard Project Hurricane (SPH) model is one of two approaches the Corps of Engineers (USACE) presently uses to model tropical storm wind fields. The second approach is Probable Maximum Hurricane (PMH). The first SPH was approved by USACE in a design study for Lake Okeechobee, Florida (U.S. Weather Bureau, Mar 1954).

Guidance on the selection of site-specific storm meteorological parameters was initially given in National Hurricane Research Project Report No. 33 (U.S. Weather Bureau, Nov 1959). The Weather Bureau and USACE jointly derived the specifications, criteria, procedures, and methods contained in this report. The goal of the guidance was to provide generalized hurricane specifications consistent geographically and meteorologically for use in establishing hurricane design criteria for hurricane protection works.

Report No. 33 defines the Standard Project Hurricane (SPH) as “the most severe storm that is considered reasonably characteristic of a region.” The SPH index is based on an analysis of past hurricanes of record. Hurricane characteristics are correlated with intensity criterion, location, and other features.

The specifications for SPH were reviewed several times after 1959, and the Weather Bureau issued updates. After Hurricane Betsy in 1965, the Weather Bureau revised the wind field parameters, but did not change the other characteristics of the SPH (U.S. Weather Bureau, Aug 1965, Nov 1965, Feb 1966).

In 1979, a new report, NOAA Technical Report NWS 23, was published containing revised criteria for the SPH.

Probable Maximum Hurricane

The Probable Maximum Hurricane (PMH) is defined as a hypothetical steady state hurricane having a combination of values of meteorological parameters that will give the highest sustained wind speed that can probably occur at a specified coastal location.

The first PMH studies were requested by the Corps of Engineers for the Narragansett Bay and New Orleans regions. The central pressures were determined as a ratio to the central pressure for the SPH. The remaining factors for the PMH were essentially the same as for the SPH. An unpublished PMH study by the U.S. Weather Bureau in the 1960s generalized criteria for PMH along the East and Gulf coasts. The central pressure and peripheral pressure for the PHM differed from that of the SPH; values of the other parameters remained unchanged even though the list of hurricanes of record was updated.

Design Hurricane, Lake Pontchartrain, LA and Vicinity

SPH values for Lake Pontchartrain, Louisiana, and Vicinity, as presented in Lake Pontchartrain, Lousiana and Vicinity, Design Memorandum No. 1 Hydrology and Hydraulic Analysis, Parts I through IV, are shown in Table IV-6.

Table IV-6
SPH meteorological parameters

Central Pressure Index	27.6 inches
Radius to Maximum Winds	30 nautical miles
Forward Speed	Varied by location, 5, 6, or 11 knots
Calculated Wind Speed, V	100 miles per hour
SPH frequency	0.01 percent storm in Zone B

According to Design Memorandum No. 1, the standard project hurricane parameters were selected for the design hurricane due to the urban nature of the project area. The rationale presented is that a hurricane of a lesser intensity, which would indicate a lower levee grade and an increased frequency of occurrence, would expose the protected areas to “hazards to life and property that would be disastrous in event of a standard project hurricane." The rationale for selection of the SPH as the design hurricane is being further investigated.

For the Lake Pontchartrain, LA, and Vicinity project, the hurricane surge height is defined as the elevation of the stillwater level at a given point resulting from hurricane surge action. It is the sum of tide, pressure setup, set up due to winds over the continental shelf, and buildup. Where appropriate, the wind tide level was used in lieu of the stillwater level.

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

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

where

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

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

This procedure was developed for an area along the Mississippi gulf coast where reliable data was available for several hurricanes to validate the methodology. Two historical storms, the September 1915 and September 1947 hurricanes, were used to establish and verify procedure. Isovel patterns, central pressure index, radius to maximum winds, forward speed, and maximum windspeed[1] parameters were available for these two storms.

The computed maximum surge height was compared to the observed high water marks from these storms. 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.

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

The procedure was then applied to the Louisiana coast. In addition to the aforementioned hurricanes, a third hurricane, 1956, was used to verify the process. The surge adjustment factor was adjusted. Table IV-7 shows the surge computations and the comparison with observed high water marks from the three hurricanes.

Table IV-7
Verification of Hurricane Surge Heights

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

Table IV-8
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 Elevation, Ft	Runup Height Ft	Freeboard, Ft	Design Elevation Protective Structure, ft
Citrus back levee, west of Paris Road	Dm1, part 1, Aug 1966	-	-	-	13.0 MSL	-	1.0	14.0 MSL
Citrus back levee, east of Paris Road	DM2, Aug 1967	13.1	4.7	5.4	13.0 NGVD	5.0**	-	18.0 NGVD
New Orleans East back levee	DM1, Part 1, Aug 1966	13.1	4.7	5.4	13.0 MSL	4.5	-	17.5 MSL
Chalmette Loop IHNC to Paris Road	DM1, Part 1, Aug 1966	-	-	-	13.0 MSL	-	1.0	14.0 MSL
Chalmette Loop Paris Road to Bayou Lawler	DM1, Part 1, Aug 1966	16.3	7.0	6.4	13.0-12.5 MSL	4.7	-	17.5 MSL
Chalmette Loop Bayou Lawler to Violet	DM1, Part 1, Aug 1966	9.7	4.6	5.2	12.5-13.0 MSL	4.3	-	17.5 MSL
New Orleans East South Point to Highway 90	DM16, Sep 1987	-	-	-	11.5-12.2 NGVD	-	2.0	13.5-14.5 NGVD
Chalmette Extension MRGO	DM1, Part 4, Oct 1967	16.3	6.6	6.2	12.5 MSL	4.6	-	17.5 MSL
Chalmette Extension Verret	DM1, Part 4, Oct 1967	10.1	4.4	5.1	12.2 MSL	4.8	-	17.5-16.5 MSL

Table IV-8
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 Elevation, Ft	Runup Height Ft	Freeboard, Ft	Design Elevation Protective Structure, ft
Chalmette Extension, Toca	DM1, Part 4, Oct 1967	9.7	4.5	5.1	11.8 MSL	4.4	-	16.5 MSL
IHNC, Seabrook to Railroad	DM2 sup8, Feb 1968				11.4 – 12.9 MSL	0	1.0	13.0 – 14.0 MSL
IHNC, Railroad to Mississippi River	DM2 sup8, Feb 1968				12.9 – 13.0 MSL	0	1.0	14.0 MSL
New Orleans East Sta 1030+00 to GIWW	DM16, Sep 1987	13.1	4.7	5.4	13.1 NGVD	4.5	-	17.5 NGVD
New Orleans East Lakefront, Citrus to South Point	DM15, Apr 1985	24.4	7.8	7.3	11.5 NGVD	6.5-7.0	-	18.0-18.5 NGVD
Citrus Lakefront, 28+31 – 64+00*	DM14, Jul 1984	-	-	-	11.5 NGVD	-	3.0	14.5 NGVD
Citrus Lakefront, 64+00 to 331+5	DM14, Jul 1984	24.4	7.8	7.3	11.5 NGVD	3.0**	-	14.5 NGVD
Orleans Parish Lakefront Levee	DM13, Nov 1984	4.6 - 24.4	1.33 – 7.8	7.3	11.5 – 12.9 NGVD	3.5 – 8.5		17.5 – 20.0 NGVD
Orleans Parish Lakefront Seabrook Floodwall	DM13, Nov 1984	NA	4.1	7.3	11.5 NGVD	3.0	-	15.0 NGVD
Jefferson, St Charles Parish Return Levee	DM17A, Jul 1987				10.5-11.5 NGVD		3.0	13.5-14.5 NGVD
Jefferson Parish Lakefront	DM17, Nov 1987	24.6	7.9	7.2	11.5 NGVD	-	-	16.0 NGVD
Orleans Marina Floodwall and New Basin Canal Gate	DM22, Apr 1993	-	-	-	11.5 NGVD	-	2.0	13.5 NGVD
Bayou St John Closure	DM22, Apr 1993	-	7.8	7.3	11.5 NGVD	6.5	-	18.5 NGVD
Bayou St John Structure	DM22, Apr 1993	-	2.1	7.3	11.5 NGVD	5.0	-	16.5 NGVD
Pontchartrain Beach Levee and Floodwall	DM22, Apr 1993	-	6.1	7.3	11.5 NGVD	8.5	-	20.0 NGVD
Lincoln Beach and New Orleans Airport Floodwalls	DM22, Apr 1993	-	-	-	11.5 NGVD	-	2.0	13.5 NGVD

Table IV-8 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 Elevation, Ft**	**Runup Height Ft**	**Freeboard, Ft**	**Design Elevation Protective Structure, ft**
St Charles, Citrus, and Jahncke PS Floodwalls	DM22, Apr 1993	-	-	-	11.5 NGVD	-	3.0	14.5 NGVD****
London Ave Outfall Canal	DM19a, Jan 1989	-	-	-	11.5 NGVD at Lake Pontchartrain	-	2.0	13.5 – 19.1 NGVD1
Orleans Ave Outfall Canal	DM19, Aug 1988				11.5 NGVD at Lake Pontchartrain		2.0	13.5 – 14.4 NGVD
17th St Outfall Canal	DM20, Mar 1990	-	-	-	11.5 NGVD at Lake Pontchartrain	-	2.0	14.0 – 16.0 NGVD

* at New Orleans Lakefront Airport, assume no waves
* includes 0.5 ft for change in levee footprint
**foreshore protection reduces wave runup
**** Existing floodwalls did not have 3 ft freeboard, but recommendation was not to raise them until elevation < 13 ft NGVD
1 Height of floodwall depends on flood proofing and pumping capacity
2 Waves determined with breakwater with crest elevation of 5.6 NGVD

Computed surge heights for Hurricane Betsy using the same Z factors averaged about 2.2 feet higher than observed surge heights. This was attributed to the effect of the high forward speed of Hurricane Betsy. A fast moving hurricane does not allow enough time for the surge heights to approach the steady state of water superelevation. The DM stated that the Z factors derived from the slow moving hurricanes should be used for design purposes.

In portions of the project area, such as along the GIWW and IHNC, the maximum surge height plus one foot of freeboard was used as the protective structures design elevation. It was believed that structures in these areas were not exposed to wave runup.

In some areas, wave runup on a protective structure was considered. Wave runup was considered to be the ultimate height to which water in a wave ascends on the slope of a structure. The condition occurs when the surge height is at a maximum. For the Lake Pontchartrain, LA and Vicinity project, the wave runup was calculated by the interpolation of model study data developed by Saville, which relates relative runup, wave steepness, relative depth, and structure slope.

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. 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 on each berm to determine the required levee elevation.

Wave data, runup elevations, and required elevations of protective structures for St. Bernard protection structures are shown in Table IV-8. All calculations were made using the MSL datum.

Chalmette Extension. Maximum wind tide levels were computed using the same equation; surge reduction factors were developed. Consideration was next given to the limit of overland surge penetration, which is dependent on the height of the surge and the duration of high stages at the coast. A study of available observed high water marks at the coastline and inland was made. A consistent, simple relation between the maximum surge height and the distance inland from the coast was developed. The relationship appeared to be independent of forward speed, windspeed, or direction. The data indicated that the weighted mean decrease in surge height inland is at a rate of 1 ft per 2.75 miles. The location of maximum surge height was determined. The computed wind tide elevation at this location was reduced at the rate of 1 ft per 2.75 miles to the levee location. Table IV-9 shows the maximum surge heights and surge reduction factors for the Chalmette Extension.

Table IV-9
Surge Reduction Factors

Location	Surge Reduction Factor, Z	Wind Tide elevation, surge reference line, FT MSL	Wind Tide elevation, levee location, FT MSL
MRGO	0.30	12.5	12.5
Verret	0.48	15.1	12.2
Toca	0.52	15.8	11.8

Wave runup was computed using the same methodology as the Chalmette Loop. Wave data, runup elevations, and required elevations of protective structures are shown in Table IV-8. All calculations were made using the MSL datum.

Citrus Back Levee. The methodology used for the Chalmette Loop was used for the Citrus back levee for the computation of surge and wave runup. Along the GIWW west of Paris Road, it was assumed the structures in this area would not be exposed to wave runup; the maximum surge height plus one foot of freeboard was used as the protective structures design elevation. East of Paris Road, wave runup was incorporated into the design elevation. An additional 0.5 ft was added to the design elevation for the area east of Paris Road because of the adoption, based on soil studies and comparative cost estimates, of a levee cross section configuration different than used for the Chalmette Loop. Wave data, runup elevations, and required elevations of protective structures are shown in Table IV-8. All calculations were made using the MSL datum.

Lake Pontchartrain Lakefront. The Lake Pontchartrain Lakefront consists of New Orleans East, Citrus, New Orleans and Jefferson Parish protection systems. For these protection systems, the still water level and protective structure heights in DM1 assumed the barrier plan was in place. When the decision was made to eliminate the barrier plan, the design heights were recomputed.

In Lake Pontchartrain, the still water level is the sum of the surge, 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). The method used to compute the water level from surge started with a surge hydrograph at Long Point in Lake Borgne developed using a method developed by R.O. Reid, modified so that the peak of the hydrograph coincided with the maximum surge elevation computed using the general wind tide equation in DM1. 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.

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.

The resultant hydrographs for the SPH are shown in Figure IV-12.

During hurricanes, strong winds blow over Lake Pontchartrain, driving large quantities of water toward the leeward shore. It is necessary to compute the wind tide level for the lake. The lake was divided into parallel segmental regions and setup and setdown were computed within these regions from the windward shoreline to the leeward shoreline using the average windspeeds from the isovel patterns and depths from hyrographic charts. Wind setup was computed using the following equation

$$\Delta S_i = d_t \left[\sqrt{\frac{2kU^2 \Delta x}{g(d_T)^2} + 1} - 1 \right]$$

$$d_T = d_{i\,i=1}^{\,i=M-1+\sum \Delta S_i}$$

$$S = \sum_{i=1}^{i=M} \Delta S$$

where

$S =$ setup or setdown, in feet measured above or below the mean water level of the surge of the lake

Dt = average depth of fetch, in feet below mean water level
U = windspeed in miles per hour over fetch
F = fetch length in miles
N = planform factor, generally equal to unity

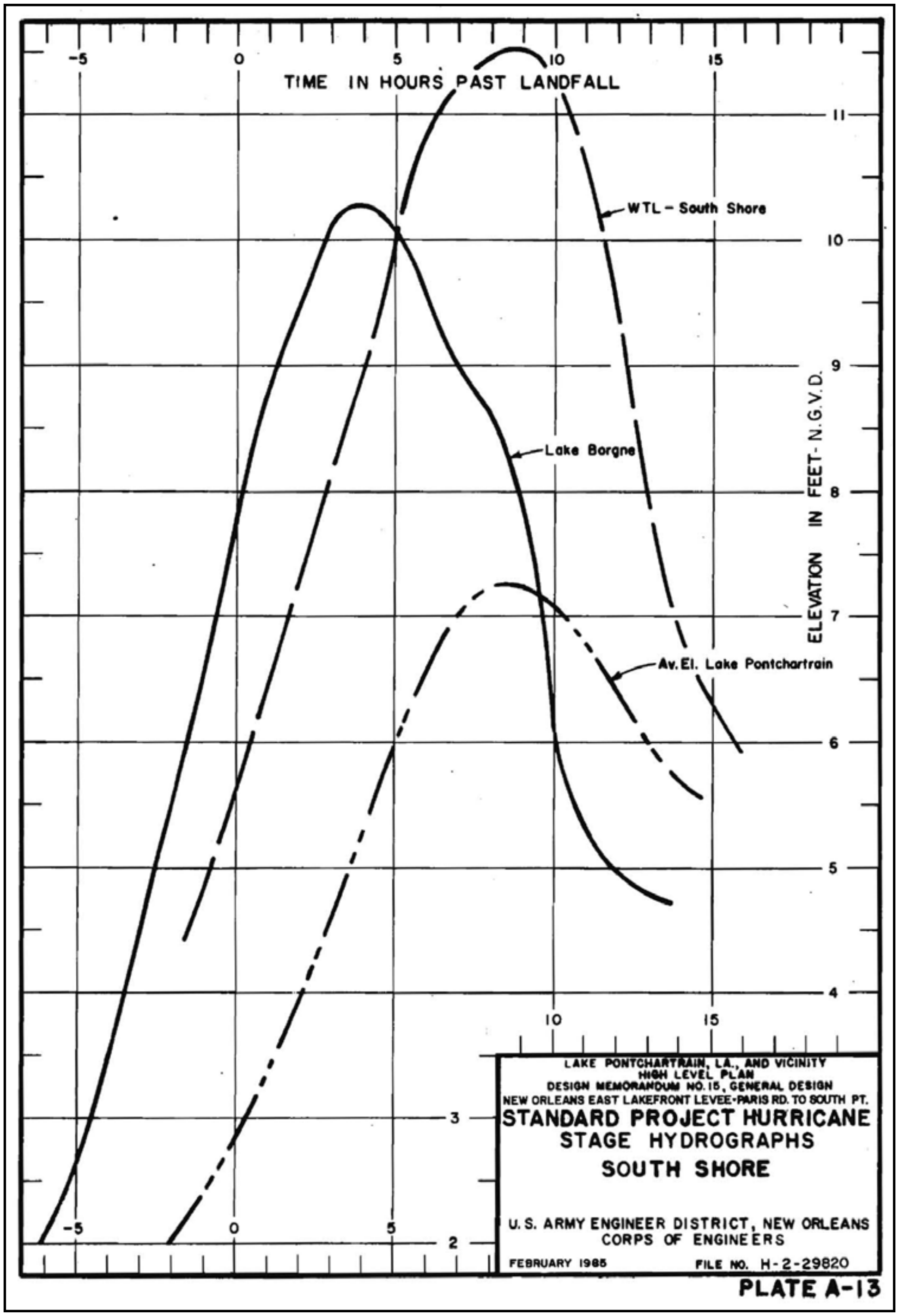


Figure IV-12.

Water surface contours were developed for the lake; tilt and wind tide levels were determined from the contours. Computed stages from the 1915 hurricane compared favorably with observed high water marks.

For the early DMs, wave heights and periods were developed using CERC Technical Report 4. Wave runup was calculated by the interpolation of model study data developed by Saville, which relates relative runup, wave steepness, relative depth, and structure slope. In the second endorsement to DM14, an evaluation was made of the runup height using the methodology contained in the Shore Protection Manual (SPM), 1977, and the SPM 1984. The computed runup varied from 2.7, for the method contained in SPM 1977, to 3.3, for the method contained in SPM 1984. It was concluded that the modest difference in wave runup from the value computed using the method in Technical Report 4 was not significant, and consistency in design was recommended.

For Jefferson Parish lakefront, the wave heights were developed based on waves breaking on the berms.

Wave data, runup elevations, and required elevations of protective structures are shown in Table IV-8. All calculations were made using the NGVD datum; it was assumed that MSL and NGVD datums were the same.

Jefferson-St Charles Parish Return Levee. For the Jefferson- St Charles Parish return levee, the method to compute wind tide elevation used for the lakefront levees was used. During the time of maximum wind tide, the winds are parallel or leeward to the levee; therefore, wave runup is not a factor. Three feet of freeboard was added to the maximum wind tide elevation. Design elevations are shown on Table IV-8. All calculations were made using the NGVD datum; it was assumed that MSL and NGVD datums were the same.

London Ave Outfall Canal. Backwater calculations were performed with a starting water surface elevation of 11.5 NGVD and varying outflow from Pump Station 3 and Pump Station 4. Several backwater calculations were made to represent conditions with bridges as of 1987, all bridges raised, two bridges floodproofed, and all bridges floodproofed. The computed water surface elevation at Pump Station 3 varied from 11.85 NGVD to 17.10 NGVD. The lowest elevation, 11.85 ft NGVD, represented all bridged floodproofed with a pump capacity of 3,475 cfs. This pump capacity was determined from information provided in the January 1986 report, “Hydraulic Study of London Ave Outfall Canal.” Pump Station 3, with a capacity of 4,300 cfs, was assumed to be ineffective. Pump Station 4, with a capacity of 3,980 cfs, was assumed to have an operational capacity of 2,475 cfs. The plan by the Sewage and Water Board to add 1,000 cfs capacity to Pump Station 4 was included in this analysis.

The highest elevation, 17.10 ft NGVD, assumed that all bridges were floodproofed, the pumping capacity of Pump Station 3 and 4 was 8,280 cfs, and 1,000 cfs would be added to Pump Station 4 in the future.

Design elevations, from backwater calculations, with 2.0 feet of freeboard, are shown on Table IV-8. All calculations were made using the NGVD datum; it was assumed that MSL and NGVD datums were the same.

Orleans Ave Outfall Canal. Backwater calculations were performed with a starting Lake Pontchartrain water surface elevation of 11.5 NGVD and varying outflow from the pumping station that put water into the canal. Several backwater calculations were made to represent conditions with bridges as of 1984, all bridges raised, and various combinations of floodproofing. The computed water surface elevation at Pumping Station 7 varied from 11.71 NGVD to 12.40 NGVD. The lowest elevation, 11.71 ft NGVD, represented all bridged raised with a pump capacity of 3,250 cfs. The highest elevation, 12.40 ft NGVD, assumed bridges floodproofed and a pump capacity of 4,450 cfs.

Design elevations, from backwater calculations, with 2.0 feet of freeboard, are shown on Table IV-8. All calculations were made using the NGVD datum; it was assumed that MSL and NGVD datums were the same.

17th Street Outfall Canal. Backwater calculations were performed with a starting Lake Pontchartrain water surface elevation of 11.5 NGVD and varying outflow from the pumping stations that put water into the canal. Several backwater calculations were made to represent conditions with bridges as of 1990, all bridges raised, and various combinations of raising, flood proofing, or existing conditions. The computed water surface elevation at the railroad bridge varied from 11.71 NGVD to 13.92 NGVD. The lowest elevation, 11.71 ft NGVD, represented all bridged raised with a pump capacity of 6,650 cfs. The highest elevation, 13.92 ft NGVD, assumed existing bridges and a pump capacity of 9,630 cfs.

Design elevations, from backwater calculations, with 2.0 feet of freeboard, are shown on Table IV-8. All calculations were made using the NGVD datum; it was assumed that MSL and NGVD datums were the same.

17th Street Design History

The chronology of the hurricane protection system features at the 17th Street Canal was prepared to meet the following objectives:

- To prepare a chronologic history comprehensive in nature to ensure the IPET is aware of all activities prior to Katrina that have value in accomplishing the IPET scope of work;
- To produce a report that includes descriptions of the various types of activities of value to the IPET and listings of documents that provide pertinent information.

The chronology serves, more or less, as an annotated bibliography of the most critical documents of the thousands of documents made available to the research team. The chronology is arranged with the most recent entries listed first. An analysis of the paper trail reveals that four events serve as major turning points in the evolution of the project:

- The Report of the Chief of Engineers, dated March 4, 1964, recommends the barrier plan that serves as the basis for the feasibility report on the hurricane protection project and the subsequent project authorization in the 1965 Flood Control Act.

- The U.S. District Court Injunction of December 1977 (modified March 1978) enjoins the Corps from constructing the barrier complexes.

- The Reevaluation Study, dated July 1984, which serves as the basis for the feasibility report of the hurricane protection plan and becomes the vehicle for authorization of the high-level plan.

- General Design Memorandum No. 20, 17th Street Outfall Canal, dated March 1990, which examines two alternative plans for providing high-level protection: fronting protection (butterfly gates at canal entrances) and parallel protection (floodwalls and flood proofing of bridges).

The chronology is arranged with the most recent entries listed first. The parenthetical information following each entry represents one of the four locations from which the documents were obtained: (1) the IPET public website, (2) the New Orleans District ProjectWise Server, (with control numbers A followed by 7-digit number), (3) the New Orleans District geotechnical map files, and (4) compact disks prepared in response to the U.S. Senate Committee on Homeland Security and Governmental Affairs.

- *Agenda, Contract 02-C-0016, 17th St. Outfall Canal, Metairie Relief, Hammond Hwy. Complex Progress Meeting*, dated **May 18, 2005**. The purpose of this meeting was to review job progress (89% complete through April 30, 2005) of completed phases of work, current work underway, and scheduled work. The purpose of the meeting was also to review outstanding submittals, modifications, and corrective actions. (A0000150)

- *Agenda, Contract 02-C-0016, 17th St. Outfall Canal, Metairie Relief, Hammond Hwy. Complex Progress Meeting*, dated **April 20, 2005**. The purpose of this meeting was to review job progress (87% complete through March 31, 2005) of completed phases of work, current work underway, and scheduled work. The purpose of the meeting was also to review outstanding submittals, modifications, and corrective actions. (A0000160)

- *Agenda, Contract 02-C-0016, 17th St. Outfall Canal, Metairie Relief, Hammond Hwy. Complex Progress Meeting*, dated **March 16, 2005**. The purpose of this meeting was to review job progress (86% complete through March 1, 2005) of completed phases of work, current work underway, and scheduled work. The purpose of the meeting was also to review outstanding submittals, modifications, and corrective actions. (A0000159)

- *Annual Inspection of Completed Works Program, 2004 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District.* Memorandum dated **December 20, 2004**. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected October

15, 2004 and received an ACCEPTABLE rating. (Senate CD 15 – 15 Nov 05, disk 2 of 2)

- *Data pertaining to the Louisiana Hurricane Protection Study*, dated **March/April 2004**. The documents posit several proposed feasibility study alternatives to upgrade the hurricane protection project to accommodate a Category 4 or Category 5 storm. Alternatives include among others: raising all existing levees and building structures at outfall canal entrances; raising existing levees, with the exception of those along the IHNC and GIWW and placing a structure at the confluence of the GIWW and MRGO and a second structure at Seabrook; and structures at the Chef and Rigolets passes. (A0002025, A0002027, A0002028, A0002029, A0002030)

- *Annual Inspection of Completed Works Program, 2003 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District*. Memorandum dated **2003**. The hurricane protection levees and floodwalls within the East Jefferson Levee District were inspected September 19, 2003, and were assigned an ACCEPTABLE rating. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected June 4, 2003 and received an ACCEPTABLE rating. (Senate CD 15 – 15 Nov 05, disk 2 of 2)

- *Transmittal No. 56*, dated **June 12, 2002**. Document indicates that ED-FS has reviewed the H-pile compression load test at Hammond Highway at the 17th Street Canal. The H-pile test pile was driven to elevation -78.5, or 2.5 feet deeper than the tip elevation of -76.0 shown on the plans. ED-FS recommends a pile tip elevation of -76.0 which will result in a F.S.> than 2.0. (A0000152)

- *Annual Inspection of Completed Works Program, 2002 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District*. Memorandum dated **2002**. The hurricane protection levees and floodwalls within the East Jefferson Levee District were inspected November 22, 2002. They were found to be "exceptionally well maintained," and were assigned an ACCEPTABLE rating. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected May 31, 2002 and received an ACCEPTABLE rating. (Senate CD 15 – 15 Nov 05, disk 2 of 2)

- *Drawings, Test Pile Frame, 17th Street Outfall Canal, Hammond Highway Complex*, dated **April 10, 2002.** (MVN Geotech Map Files)

- *Annual Inspection of Completed Works Program, 2001 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District*. Memorandum dated **2001**. The hurricane protection levees and floodwalls within the East Jefferson Levee District were inspected October 12, 2001. They were found to be "exceptionally well maintained," and were assigned an OUTSTANDING rating. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected May 18, 2001 and received an OUTSTANDING rating. (Senate CD 15 – 15 Nov 05, disk 2 of 2)

- *Annual Inspection of Completed Works Program, 2000 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District*. Memorandum dated **December 12, 2000**. The hurricane protection levees and floodwalls within the East Jefferson Levee District were inspected

October 13, 2000. They were found to be “exceptionally well maintained,” and were assigned an OUTSTANDING rating. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected June 2, 2000 and received an OUTSTANDING rating. (Senate CD 15 – 15 Nov 05, disk 2 of 2)

- *Correspondence regarding directional boring under the Inner Harbor Canal, London Canal, and the 17th Street Canal*, dated **May 2000**. This is a series of correspondence between the Corps of Engineers, the Gilbert Southern Corporation, and Bay Equipment Company concerning the guidelines and safety factors of the referenced subject material. File contains drawings depicting the fiber optic cable route at the outfall canals. (A0001813) Supporting information can also be found in A0003693 and A0003694.

- *Annual Inspection of Completed Works Program, 1999 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District*. Memorandum dated **December 16, 1999**. The hurricane protection levees and floodwalls within the East Jefferson Levee District were inspected October 8, 1999. They were found to be “exceptionally well maintained,” and were assigned an OUTSTANDING rating. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected May 21, 1999 and received an OUTSTANDING rating. (Senate CD 15 – 15 Nov 05, disk 2 of 2)

- *Annual Inspection of Completed Works Program, 1998 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District*. Memorandum dated **December 15, 1998**. The hurricane protection levees and floodwalls within the East Jefferson Levee District were inspected October 9, 1998. They were found to be “exceptionally well maintained,” and were assigned an OUTSTANDING rating. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected May 29, 1998 and received an OUTSTANDING rating. (Senate CD 15 – 15 Nov 05, disk 2 of 2)

- *Correspondence regarding Sediment Sampling, Lake Pontchartrain and Vicinity, Hurricane Protection Plan, (HLP), Fronting Protection for Pumping Station Nos. 3, 4, 6, and 7 at London, 17th Street, and Orleans Avenue Outfall Canals*, dated **1998**. This file contains a series of correspondence relating to the subject matter and includes maps and drawings of sediment sample locations. (A001811)

- *Lake Pontchartrain, LA, and Vicinity Hurricane Protection, High Level Plan, Orleans Parish – Jefferson Parish, Fronting Protection for Pumping Station No. 6 at the 17th Street Outfall Canal*. Construction drawings, DACW-29-99-C-0018, (98-B-0012) dated **1997**. (IPET)

- *Plans for Lake Pontchartrain, Louisiana and Vicinity, Hurricane Protection, High Level Plan, Orleans Parish – Jefferson Parish, Fronting Protection for Pumping Station No. 6 at 17th Street Outfall Canal*, plan drawings dated **1997.** File also contains supporting documentation for contract DACW29-99-0018. (Senate CD 15, 15 Nov 05, disk 1 of 2)

- *General Surveys, 17th Street Canal*, **1997**. This collection of documents contains survey data, field survey books and cross-section computations spanning the years 1979 through 1997. Sheets include: canal cross sections; cross section data by Walker and Avery, Inc. (119 sheets); field notes and traverse computations by ED-SS; and field notes by Modjeski and Masters. (A0001001)

- *Annual Inspection of Completed Works Program, 1997 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District*. Memorandum dated **December 24, 1997**. The hurricane protection levees and floodwalls within the East Jefferson Levee District were inspected September 19, 1997, and were assigned an OUTSTANDING rating. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected May 29, 1997 and received an OUTSTANDING rating. (Senate CD 09 Dec 05)

- *Supplemental Agreement between the United States of America, the Orleans Levee District, the East Jefferson Levee District, and the Sewerage and Water Board of New Orleans*. Signed agreement dated **February 18, 1997**. (Senate CD 16 – 24 Oct 05)

- *Orleans Marina Permit* dated **January 13, 1997**. This is a series of correspondence regarding a request from the Sewerage and Water Board for a permit to jack pipe under the levee and storage monolith at the Orleans Marina. (A0001822)

- *Annual Inspection of Completed Works Program, 1996 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District*. Memorandum dated **December 13, 1996**. The hurricane protection levees and floodwalls within the East Jefferson Levee District were inspected September 20, 1996, and were assigned an OUTSTANDING rating. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected May 31, 1996 and received an OUTSTANDING rating. (Senate CD 09 Dec 05)

- *Design Memorandum No. 20, General Design Supplement No. 1, Orleans Parish/Jefferson Parish, 17th Street Outfall Canal, Lake Pontchartrain, LA, and Vicinity Hurricane Protection Project, High Level Plan,* **January 15, 1996**. This supplement posits a historical, design, and engineering analysis for improvements to the fronting protection at pumping station no. 6, in an effort to propose improvements that will allow the station to meet design heights for the standard project hurricane. Document includes analysis of hydrology, hydraulics, geology, foundation investigations, and design. (IPET)

- *Annual Inspection of Completed Works Program, 1995 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District*. Memorandum dated **December 12, 1995**. The hurricane protection levees and floodwalls within the East Jefferson Levee District were inspected September 22, 1995, and were assigned an OUTSTANDING rating. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected May, 1995 and received an OUTSTANDING rating. (Senate CD 09 Dec 05)

- *Lake* Pontchartrain*, LA, and Vicinity Hurricane Protection, High Level Plan, 17th Street Outfall Canal, Orleans Parish – Jefferson Parish, Veterans BLVD Bridges*. As built drawings, DACW-29-C-0093, (95-B-0095) dated **June 1995**. (IPET, Senate CD 13 – 15 Nov 05)

- *17th St. Outfall Canal, History of Surveys Used for Constructing Floodwalls and Canal Dredging*, dated **February 8, 1995**. This document analyses the surveys and concludes that the floodwalls on both sides of the canal were constructed approximately 5.5 inches lower than the elevations indicated on the plans and specifications. Also, the I-walls were supposed to have been constructed with 6 inches of allowable settlement; instead they were constructed with only an 0.5-inch overbuild. (A0001034)

- *17th Street Canal, East Side, Pittman Construction (DCAW29-93-C-0081), Concrete Compression Test Specimen Data*, dated **1995**. This collection contains 180-pages of test specimen data sheets ranging in dates from 1993 through 1995. (A0001112)

- *Annual Inspection of Completed Works Program, 1994 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District*. Memorandum dated **December 19, 1994**. The hurricane protection levees and floodwalls within the East Jefferson Levee District were inspected October 4, 1994, and were assigned an OUTSTANDING rating. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected June, 1994 and received an OUTSTANDING rating. (Senate CD 09 Dec 05)

- *Memorandum for File by Charlie Rome (CELMN-ED-G*), dated **November 21, 1994**, regarding 17th Street Canal Floodwall, Orleans Parish, Vets to Lake, Field Trip Report. This document is an account of the trip on November 8, 1994, to evaluate the extent of damage to an unidentified monolith after the contractor had removed defective concrete. The inspectors indicate that the monolith could be repaired by patching, and make several recommendations on how to complete the repairs. Photos of the defective monolith accompany the trip report. (A0001318)

- *Annual Inspection of Completed Works Program, 1993 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District*. Memorandum dated **December 22, 1993**. The hurricane protection levees and floodwalls within the East Jefferson Levee District were inspected October 22, 1993, and were assigned an OUTSTANDING rating. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected May, 1993 and received an OUTSTANDING rating. (Senate CD 09 Dec 05)

- *17th Street Canal Floodwall, Pittman Construction, (DCAW29-93-C-0081), Expansion Joint Filler Submittal*, dated **August 4, 1993**. In the supporting documentation, dated August 10, 1993 and contained within, the New Orleans District offers no objection to the recommended expansion joint filler provided that it meets all requirements of ASTM D 1752-84, including the .25-inch maximum for Extrusion. The document notes that the Recovery, reported as Compression Set, needs to be determined after 10 minutes or if the initial test fails, 1 hour and not the 24 hours reported. Documents also recommend that it be verified that the compression test results are in psi. (A0001075)

- *17th Street Canal Floodwall, Pittman Construction, (DCAW29-93-C-0081), Expansion Joint Filler Submittal,* dated **August 4, 1993**. The supporting documentation includes a letter from Louisiana Industries to Pittman Construction, dated July 13, 1993, concerning the 17th St. Canal (DACW2993B0025). The letter certifies that the mix design will meet or exceed the indicated design strength at a designated age when tested in accordance with the applicable ASTM Standards. (A0001073)

- *Contract Award Information, Contract No. DACW29-93-C-0081*. Contract, dated **June 28, 1993**, for the Lake Pontchartrain, LA and Vicinity, Hurricane Protection Project, High Level Plan, 17th Street Outfall Canal, Flood Protection Improvement Project, Capping of Floodwall, East Side Improvements, Orleans Parish, LA, is awarded to Pittman Construction. Supporting contract documentation includes the court decision that settled the dispute between Pittman Construction and the Corps. The court decision posits a narrative history of the dispute. (Senate CD 15 – 15 Nov 05, disk 2 of 2)

- *Annual Inspection of Completed Works Program, 1992 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District*. Memorandum dated **December 14, 1992**. The hurricane protection levees and floodwalls within the East Jefferson Levee District were inspected September 24, 1992, and were assigned an OUTSTANDING rating. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected May, 1992 and received an OUTSTANDING rating. (Senate CD 09 Dec 05)

- *Lake Pontchartrain, LA, and Vicinity Hurricane Protection, High Level Plan, 17th Street Canal, Excavation and Floodwall Protection, Capping of Floodwalls*. Construction drawings, including supplemental drawing from Modeski and Masters dated **November 1992**. (IPET, Senate CD 13 – 24 Oct 05)

- *Excavation and Flood Protection – 17th Street Canal, Capping of Floodwalls, East Side Levee Improvements*. As-built drawings (DACW29-93-0025) dated November 1992. (Senate CD 13 – 15 Nov 05)

- *17th Street Canal, West Side Levee Improvements* (Contract 92-1). Construction drawings from the Board of Levee Commissioners of the East Jefferson Levee District dated **March 1992**. Includes cross-section of levee and floodwall improvements and dredging cross-sections. (IPET)

- *Letter from the Sewerage and Water Board of New Orleans to Ron Ventola, Chief of Regulatory Function, New Orleans District*, dated **May 28, 1992**, regarding Permit No. LMNOD (17th Street Canal) 2, dated June 13, 1984. The intent of this letter is to seek an extension to the dredging permit issued by the Corps in 1984, but it also serves as a basic history of the three-phase dredging process carried out by the Sewerage and Water Board in the 17th Street Canal between June 1984 and May 1992. (CEMVN-OD)

- *Annual Inspection of Completed Works Program, 1991 Annual Inspection for Maintenance of Completed MR&T Flood Control Works in the New Orleans District*. Memorandum dated **December 10, 1991**. The hurricane protection levees and floodwalls within the East Jefferson Levee District were inspected October 29, 1991, and were assigned an OUTSTANDING rating. The hurricane protection levees and floodwalls within the Orleans Levee District were inspected June, 1991 and received an OUTSTANDING rating. (Senate CD 09 Dec 05)

- *Permit Review Sheet: 17th St. Canal 2, Req. by Boh Bros. to deposit dredged material from SW&B project to dredge 17th St. Canal*, dated **August 31, 1990**. This is a series of correspondence regarding the request by Boh Bros. to deposit the dredge material at the Bucktown Marina site. The New Orleans district offers no objections to the request provided that the material is not placed in the areas of new levee section, including any berms. (A0000110)

- *Lake Pontchartrain, LA, and Vicinity, Lake Pontchartrain High Level Plan, Design Memorandum No. 20, General Design, 17th Street Outfall Canal,* **March 1990.** The DM examines two alternative plans for providing "high level" standard project hurricane protection: fronting protection (butterfly gates at canal entrances) and parallel protection (floodwalls and flood proofing of bridges), with the parallel protection plan representing the recommended plan. DM includes discussion of the project plan, hydrology, hydraulics, geology, foundation investigation and design, and structural designs, and is complete with plates and diagrams. Includes pre-construction plan drawings. (IPET, Senate CD 13 – 24 Oct 05)

- *Excavation and Flood Protection – 17th Street Canal, Phase IB, Hammond Highway* to *Southern Railway* (Contract 2043-0489). As built drawings (DACW-29-93-B-0025) from the Board of Levee Commissioners of the Orleans Levee District dated **February 7, 1990**. (IPET)

- *Letter from Frederick M. Chatry, Chief, Engineering Division, New Orleans District, to Modjeski and Masters, Consulting Engineers*, dated **October 20, 1989**, concerning the 17th Street Canal Parallel Flood Protection, Phase 1B, Hammond Highway to Southern Railway, OLB Project No. 2043-0207. In this letter, the Corps posits two additional revisions to the final plans and specifications submitted by Modjeski and Masters on October, 10, 1989 reducing the requirement for each layer to be compacted to a least 90 percent of the maximum dry density of optimum water content, rather than the proposed 95 percent (ASTM D698); and a revision in the sheet pile tip elevations to a higher elevation as imposed by LMVD, which will result in lower overall cost for the project. Letter indicates that once these revisions are incorporated into the plans and specifications, the Corps will have no objection to Modjeski and Masters proceeding with the proposed work. (A0000100)

- *Letter from Modjeski and Masters, Consulting Engineers, to Frederick M. Chatry, Chief, Engineering Division, New Orleans District*, dated **October 10, 1989**, concerning the 17th Street Canal Parallel Flood Protection, Phase 1B, Hammond Highway to Southern Railway, OLB Project No. 2043-0207. This letter posits the changes to the plans and specifications made in response to Corps letter of August 22, 1989, with a detailed description of the embankment construction process to address specific concerns toward the maximum density of the embankment material**.** (A0000099)

- **-***Excavation and Flood Protection, 17th Street Canal, Phase 1B, Hammond Highway to Southern Railway*. "Preliminary" specifications for contract 2043-**____**, dated **October 10, 1989**, by the Board of Commissioners of the Orleans Levee District. Document includes preliminary specifications for general specifications, demolition, dredging and levee construction, and steel sheet piling. (A0000095)

- *-Letter from Frederick M. Chatry, Chief, Engineering Division, New Orleans District, to Modjeski and Masters, Consulting Engineers*, dated **August 22, 1989**, concerning the 17th Street Canal Parallel Flood Protection , Phase 1B, Hammond Highway to Southern Railway, OLB Project No. 2043-0207. In this letter, the Corps indicates that it has reviewed plans, specifications, and design calculations submitted by Modjeski and Masters on July 10, 1989, and posits four primary revisions to include: degrading the existing levee crown elevation at station 570+00 to elevation 5.5 as shown in the design analyses; correcting the new I-wall B/L offset at station 657+00 to 200 feet; and answering specific questions pertaining to the maximum density of the embankment material. Letter also acknowledges Corps concurrence to a request to delete the riprap specified for the east side levee between Hammond Highway bridge and station 615+00. (A0000088). See, also memorandum from Rodney P. Picciola, Chief, Foundations and Materials Branch, dated July 28, 1989. (A0000089)

- *Memorandum from Fred H. Bayley III, Chief, Engineering Division, Lower Mississippi Valley Division, to the Commander, New Orleans District, Regarding Sheet Pile Wall Design Criteria*, dated **July 24, 1989**. This memorandum summarizes the guidance for determining sheet pile wall penetrations, deflections, and other topics, and it references the sources detailing new I-wall design criteria for determining the penetration of sheet pile floodwalls founded in soft clays; estimating sheet pile deflections and design of I-walls to withstand these deflections; and sheet pile finite element-based design procedures for sheet pile walls**.** (A0000097, A0000101)

- *Letter from Modjeski and Masters, Consulting Engineers, to Frederick M. Chatry, Chief, Engineering Division, New Orleans District*, dated **July 10, 1989**, concerning the 17th Street Canal Parallel Flood Protection, Phase 1B, Hammond Highway to Southern Railway, OLB Project No. 2043-0207. This letter contains plans and design calculations submitted Modjeski and Masters, Consulting Engineers. The document posits revised slope stability and sheet pile design calculations that address comments made by the Corps by letter of April 25, 1989. A brief description of the revisions made to the cross-sections is given for each of the eight reaches. Also given for each of the reaches is a listing of new submittals, stating which comments from the Corps were addressed. (A0000090, A0000091, A0000092)

- *17th Street Canal Drawings*, dated **June 16, 1989**. Drawings depicting shear soil strength, stability, and sheet pile analyses for reaches 1 through 8. (A0000094)

- *Letter from Frederick M. Chatry, Chief, Engineering Division, New Orleans District, to Modjeski and Masters, Consulting Engineers*, dated **April 25, 1989**, concerning the 17th Street Canal Parallel Flood Protection , Phase 1B, Hammond Highway to Southern Railway, OLB Project No. 2043-0207. The letter posits revisions of the landside slope stability analysis furnished by Modjeski and Masters in letter dated April 10, 1989, and offers seven comments from the MVN Foundations and Materials Branch for consideration pertaining to soil shear strength, landside and canal side stability, and I-wall stability at various reaches. (A0000083, A0000084)

- *Letter from Modjeski and Masters, Consulting Engineers, to Frederick M. Chatry, Chief, Engineering Division, New Orleans District,* dated **April 10, 1989**, concerning the 17th Street Canal Parallel Flood Protection , Phase 1B, Hammond Highway to Southern Railway, OLB Project No. 2043-0207. This letter addresses comments posited by the Corps in a letter, dated October 21, 1989, with regard to preliminary plans submitted by Modjeski and Masters. Letter indicates that Modjeski and Masters' review of those comments reveal that the slope stability calculations for the first six reaches of the project do not properly reflect the actual factors of safety. The letter goes on to state that in order to achieve the required factors of safety using the cross-sections proposed by Eustis Engineers, a great deal of earthwork would be required on the landside of the levee. Because of proximity of development on the landside of the levee, Masters and Modjeski developed new levee cross sections that required no work on the landside slope, and provides descriptions of the revisions of cross sections, slope stability, and sheet pile analyses for reaches 1 thru 8. (A0000085, A0000086)

- *Letter from Eustis Engineering, Geotechnical Engineers, to Modjeski and Masters, Consulting Engineers,* dated **August 31, 1988**, concerning the geotechnical analyses of the Metairie Relief Canal (17th Street Canal) OLB Project No. 2043-0222. This report contains the results of revised cantilever floodwall analyses and revised slope stability analyses for the proposed modifications along the Orleans side of the canal between stations 553+70 and 670+00. (A0000105)

- *Letter from Frederick M. Chatry, Chief, Engineering Division, New Orleans District, to* Modjeski *and Masters, Consulting Engineers,* dated **January 4, 1988**, concerning the 17th Street Canal Parallel Flood Protection , Phase 1B, Hammond Highway to Southern Railway, OLB Project No. 2043-0207. This letter serves as the first review of Modjeski and Masters' in-progress plans and specifications for the project, and offers several comments pertaining to sheet pile tip penetration and floodwall stability between stations 636+00 and 638+31; 625+00 and 635+00; 614+00 and 615+00; and 589+00 and 590+00. Letter also addresses the issue of dredging on the Orleans Parish side of the canal and describes requirements necessary to detect scour/erosion and prevent levee failure. Requirements include adding control lines to drawings; cross-section surveys the existing levee and canal bank; initial cross-section surveys of the levee and dredged canal immediately after construction; and annual cross-sectional surveys to be provided to the Corps thereafter. Several enclosures accompany this document. (A0000109)

- *17th St. Canal, I-Wall Criteria*. This handwritten document, dated **August 16, 1988**, appears to be an agenda or notes from a meeting of New Orleans district personnel and representatives from Modeski and Masters and Eustis Engineering. Topics include I-wall stability (Q&S cases), and stress loading conditions for maximum tip penetration. Last topic indicates, "Never run S-CASE F.S. = 1.0; never run deflections on S-CASE." (A0000107)

- *Excavation and Flood Protection of the 17th Street Canal, Phase III: Lake Pontchartrain to Hammond Highway Bridge* (Contract 4117). Sewerage and Water Board of New Orleans specifications for phase III of the project, dated **April 1988**. (Senate CD 13 – 15 Nov 05)

- *17th Street Canal, (Contract 4117)*. These drawings from the Sewerage and Water Board of New Orleans are marked "Final Check Set, **August 12, 1987**," Drawings include typical sections, plans and profiles, canal contours, sheet pile wall details, cross sections, and pedestrian bridge. (MVN Geotech Map Files)

- *Supplemental Agreement between the United States of America and the Jefferson Levee District for Local Cooperation at Lake Pontchartrain and Vicinity High Level Plan*. Signed agreement, dated **January 16, 1987**. (Senate CD 16 – 24 Oct 05)

- *Supplemental Agreement between the United States of America and the Orleans Levee District for Local Cooperation at Lake Pontchartrain and Vicinity High Level Plan*. Signed agreement dated **June 21, 1985**. (Senate CD 16 – 24 Oct 05)

- *Interim Agreement between the United States of America and the Orleans Levee District for Local Cooperation at Lake Pontchartrain and Vicinity High Level Plan*. Signed agreement dated **February 20, 1985**. (Senate CD 16 – 24 Oct 05)

- *Memorandum from Maj. Gen. John F. Wall, Director of Civil Works, U.S. Army Corps of Engineers, to the Commander, Lower Mississippi Valley Division*, dated **February 7, 1985**, regarding the Lake Pontchartrain, Louisiana, and Vicinity Hurricane Protection Project. In this memorandum, the director of civil works indicates that he reviewed the revised Post Authorization Change (PAC) Notification Report, the July 1984 Reevaluation Report and the final supplement to the Environmental Impact Statement, and approves the PAC. (Senate CD 16 – 24 Oct 05)

- *Metairie Relief Canal As Built Cross Sections, Phase I, Sewerage and Water Board Contract No. 4053*. These drawings are after-dredge sections for stations 643-671, dated **December 1984**. (MVN Geotech Map Files)

- *17th Street Outfall Canal Hydraulic Grade Lines, Phase I, Contract 4053*. Sewerage and Water Board of New Orleans drawings dated **August 30, 1984**. Drawings also include cross sections. (MVN Geotech Map Files)

- Lake *Pontchartrain, LA, and Vicinity Hurricane Protection Project, Reevaluation Study*, **July 1984**. This study is conducted in response to a 1977 Federal injunction that halted portions of the project approved by the Flood Control Act of 1965, specifically the floodgate barrier components of the plan. The study examines the continued feasibility of the barrier plan and examines the feasibility of providing hurricane protection solely by the means of raising and strengthening levees or floodwalls (high level plans). The study concludes that a high level plan represents the most feasible plan of protection. The plan would provide for 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; a new levee on the east bank of St. Charles Parish north of US Highway 61. The treatment of the outfall canals at the lakefront remained unresolved, with five potential solutions, ranging from higher and stronger levees to floodgates and auxiliary pumping stations at the canal openings, discussed. Volume II of the study contains all technical and engineering data used to support information in the reevaluation study, including hydrology and hydraulics, foundation design and geology, engineering alternatives. This reevaluation study serves as the basis for the feasibility report of the hurricane protection project and becomes the vehicle which leads to authorization of the high-level plan. (IPET)

- *Department of the Army Permit, Permit No. LMNOD-SP (17th Street Canal) 2*, dated **June 13, 1984**. Permit to allow Sewerage and Water Board of New Orleans to dredge, enlarge and maintain an area and install and maintain flood walls and mooring structures in the 17th Street Canal (Metairie Relief Canal) from Pumping Station No. 6 to a point about 400 feet north of the Bucktown Pedestrian Bridge, subject to the conditions listed in the permit. Complete with 11 sheets. (CEMVN-OD)

- *Chronology of the 17th Street Canal Permit Application by Sewerage and Water Board of New Orleans*, dated **June 13, 1984**. This handwritten chronology details, extensively, the permit application process of the Sewerage and Water Board from its first submission of an application to dredge in the 17th Street Canal on July 15, 1974 through the final permit issuance on June 13, 1984. (CEMVN-OD)

- *Letter from Frederick M. Chatry, Chief, Engineering Division, New Orleans District, to the New Orleans Sewerage and Water Board*, dated **January 31, 1984**, regarding the Eustis Engineering report on the 17th Street Outfall Canal Test Section, forwarded by Modjeski and Masters on January 17, 1984. In this letter, the Corps concurs with Eustis' conclusions that a "layer of contaminated sand acts as a seal in preventing the water in the canal from influencing the hydrostatic head at and beyond the levee toe," and "Upon completion of the proposed dredging to design grade in the canal, sedimentation will probably deposit on the bottom…further sealing off the water pressure in the canal from the surrounding ground water." (A0000087)

- *Seventeen Street Canal Drainage Basin Study***, January 1983**. This study, prepared under the direction of the Sewerage and Water Board of New Orleans and the Jefferson Parish Council, provides the first in-depth study of the 17th Street Canal drainage basin that comprises 7,860 acres in Orleans Parish and 2,550 acres of Jefferson Parish. (IPET)

- *Report to the Secretary of the Army by the U.S. General Accounting Office: Improved planning needed by the Corps of Engineers to resolve environmental, technical, and financial issues on the Lake Pontchartrain Hurricane Protection Project*, dated **August 17th, 1982**. This documents, which is critical of the Corps' planning effort with regard to the project, posits a general history of the hurricane protection project from its authorization 1965 through 1982. The treatment of the outfall canals is of great significance in this report. The document indicates that discussions between the corps and local sponsors about the alteration of the drainage canals were not conclusive, owing largely to the sponsors lack of financial capability. The report notes that the Orleans Levee District "believed that the Corps' standards may be too high for what is really needed for adequate protection and for what is affordable by local sponsors." (A0001840)

- *Lake Pontchartrain, Louisiana and Vicinity, Hurricane Protection Project, Combined Phase I Type General Design Memorandum and Revised Environmental Impact Statement, Plan of Study*, dated **September 1981**. This plan of study was initiated in response to the court injunctions against the barrier complexes. The plan recommends the pursuit of a fast-track study effort and recommends a firm decision concerning the future study direction by mid-December, 1981. (Senate CD 13 – 15 Nov 05).

- *Modification of U.S. District Court Injunction*, **March 1978**. The court modified its order of December 1977 and lifted the injunction against all features of the authorized project other than the construction of the barrier complexes. The Corps determines the revised Environmental Impact Statement will need additional study and will not be complete until November 1985. As a result, in December 1981, the Corps directs future study efforts on toward the "high-level plan" that manifests itself in the July 1984 Reevaluation Study. (Contained within A0001840)

- *U.S. District Court Injunction*, **December 1977**. The Corps was enjoined by the court from constructing the barrier complexes, the New Orleans East levee system, and the Chalmette Area plan of the Lake Pontchartrain Hurricane Protection Project, authorized in 1965, pending the revision and acceptance of the Environmental Impact Statement. (Contained within A0001840)

- *Record of Public Meeting, Lake Pontchartrain, Louisiana, and Vicinity, Hurricane Protection Project*, dated **June 1975**. This document is a transcript of the public meeting held at the University of New Orleans on February 22, 1975. (Senate CD 13 – 24 Oct 05)

- *Final Environmental Impact Statement: Lake Pontchartrain, Louisiana, and Vicinity Hurricane Protection Project*, dated **August 1974**. This study describes the protective features and identifies the environmental effects of the hurricane protection project described in House Document 231, 89th Congress, 1st session (barrier plan) and approved by the 1965 Flood Control Act. (Senate CD 13 – 24 Oct 05)

- *17th St. Canal Boring Locations*, dated **1973**. This log of boring samples from 1971 through 1973 is accompanied by transmittals of the results of soil tests. (A0000393)

- *The Board of Levee Commissioners of the Orleans Levee District, Emergency Operations Plan*, dated **1972**. The document details responsibilities of the board under the emergency operations plan in terms of preparations and surveillance; high tide emergencies; and hurricane emergencies. (A0001839)

- *Hurricane Study, History of Hurricane Occurrences along Coastal Louisiana*, dated **August 1972**. This document, prepared by the New Orleans District, posits historical research, a summary of hurricane occurrences, descriptions of hurricanes and hurricane tracks dating back to the 19th century. (Senate CD 13 – 24 Oct 05)

- *Orleans Parish Lakefront Levee West of IHNC: Outfall Canals.* Drawings, dated **1970**, depicting outfall canal cross-sections, piezometer ranges, and log borings for the 17th Street, London Avenue, and Orleans canals. (A0002038)

- *Lake Pontchartrain Louisiana and Vicinity, Design Memorandum No. 1, Hydrology and Hydraulic Analysis, Part II – Lakefront*, dated **September 1968**. This document covers the hydraulic design of the lakeshore protection under the authorized project. (Senate CD 13 –15 Nov 05)

- *Lake Pontchartrain Louisiana and Vicinity, Design Memorandum No. 1, Hydrology and Hydraulic Analysis, Part II – Barrier*, dated **August 1967**. This design memorandum includes the description and analyses of essential data, assumptions, and criteria used for studies which provide the basis for determining design surge heights, run-up, overtopping and frequencies for the Lake Pontchartrain Barrier. It also includes the average lake levels for the design hurricane on different tracks. (Senate CD 13 – 15 Nov 05)

- ***House Document No. 231, 89th Congress, 1st session.*** The report of the Chief of Engineers, **March 4, 1964,** transmitted to Congress the report of the Board of Engineers for Rivers and Harbors, accompanied by the reports of the district and division engineers and the concurring reports of the Mississippi River Commission for those areas under its jurisdiction. The report posits a recommendation for what came to be known as the "barrier plan": "For protection from hurricane flood levels...the most suitable plan would consist of a barrier extending generally along US Highway 90...together with floodgates and a navigation lock in the Rigolets, and flood and navigation gates in Chef Menteur Pass; construction of a new lakeside levee in St. Charles Parish...; extension upward of the existing riprap slope protection along the Jefferson Parish levee; enlargement of the levee landward of the seawall along the 4.1 mile lakefront, and construction of a concrete-capped sheet pile wall along the levee west of the Inner Harbor Canal..." The report serves as the basis for the feasibility report on the hurricane protection project and subsequent project authorization in the Flood Control Act of 1965, also known as PL 298, 89th Congress, 1st Session. (IPET)

- *Effects on Lake Pontchartrain, LA., of Hurricane Surge Control Structures and Mississippi River Gulf Outlet Channel, Technical Report No. 2-636*, dated **November 1963**. This model study conducted by the Waterways Experiment Station from January 1960 through June 1961 analyzes the effects of gated structures under the proposed barrier system for hurricane protection on the salinity and hydraulic regimen of Lake Pontchartrain and its connecting waterways and lakes. (Senate CD 13 – 15 Nov 05)

- *Interim Survey Report, Hurricane Study, Lake Pontchartrain, Louisiana, and Vicinity*, dated **November 21, 1962**. This interim report posits the recommended plan for the Lake Pontchartrain basin. The recommended plan includes a barrier at the west end of the lake to exclude hurricane storm surges and the construction and enlargement of protective works fronting developed or potentially developable areas. (IPET)

- *Letter from Acting New Orleans District Engineer to the Board of Commissioners, Pontchartrain Levee District,* dated **September 5, 1962** regarding the 17th Street Canal Levees. The letter informs the board of commissioners that the 17th Street Canal Levee, Lake Pontchartrain Protection Levee, Station minus 3+62 lakeward of the Lake shore Hammond Highway to Station 118+12 at the Southern Railroad has been completed by the federal government under the 1928 Flood Control Act, as amended. (Senate CD 13 – 15 Nov 05)

- *Letter from the Board of Levee Commissioners of the Orleans Levee District to the District Engineer, New Orleans District*, dated **March 1, 1962**, concerning the board's view of hurricane protection along the south shore of Lake Pontchartrain. In this letter the board indicates that since the time of the 1950 study by Bedell & Nelson in 1950, the Orleans Levee Board had done considerable work along the seawall in the Lakeshore Parkway. In light of this, the Orleans Levee Board suggests that the breakwater recommended in the 1950 report is unnecessary and undesirable from an esthetic point of view. (Letter contained within ***House Document No. 231, 89th Congress, 1st session,*** dated March 4, 1964).

- -A *Detailed Report on Hurricane Study Area #1, Lake Pontchartrain and Vicinity, Louisiana*, report by the Department of the Interior, dated **March 1962**. This report analyzes the environmental effects of barrier structures and high level plans on the hydrological regime of Lake Pontchartrain. (Senate CD 13 – 15 Nov 05)

- -*Levee Work, F.Y. 1957, Item C – 17th Street Canal Levee Enlargement, Lake Pontchartrain Protection Levee, Plan Profile and Borings* dated, **January, 1957**. Corps of Engineers drawings depicting boring and section data from west canal levee opposite current-day breach location. Dates of levee embankment borings are noted as Nov. 8-12 & 15, 1948; borrow area borings, Nos. 1-10, January 21, 1957. (MVN Geotech Map Files)

- -*Geological Investigation of the New Orleans Harbor Area, TM No. 3-391*, dated **June 1954**. This study, produced by the Waters Experiment Station, is based on boring logs collected in the late fall and winter of 1949-1950. A list of the borings is contained in Appendix C. (Senate CD 13 – 24 Oct 05)

- -{Unknown Document Title}, by Bedell & Nelson, dated **October 1950**. The Orleans Levee Board and the Corps conducted a study of the lakefront to protect New Orleans from Lake Pontchartrain storm surges. The report by Bedell & Nelson, prepared for the board and shared with the Corps, recommended the installation of a breakwater from the New Basin Canal to the Industrial Canal along the south shore of Lake Pontchartrain to prevent overtopping of the seawall by wave action caused by hurricane winds. (See *Letter from the Board of Levee Commissioners of the Orleans Levee District to the District Engineer, New Orleans District*, dated March 1, 1962).

- -*Review Report: Lake Pontchartrain, La., From the Orleans-Jefferson Parish Line Westward and Northward to the Vicinity of Frenie, La*. New Orleans District document dated **April 15, 1948**. This review report was prepared in the aftermath of the hurricane of September 19, 1947, and recommends modification of the adopted project (Flood Control Act of 1946) to provide for increased protection against storm surge and waves from Lake Pontchartrain, by landside enlargement of the existing embankment along the lake, with suitable wave erosion protection, and the enlargement of return levees along the Orleans and St. Charles Parish lines. Document includes wind velocity records, hydrographs of Sept-Oct of 1947 and March 1948, rainfall frequencies; boring data, and levee profiles and typical cross-sections. (A00001300)

Finding Aid

MVN Records

Katrina Chronologic History Data Collection
-Lake Pontchartrain LA and Vicinity
-17th St. Canal
-ED-F Geotech

A0000083	A0000084	A0000085	A0000086
A0000087	A0000088	A0000089	A0000090
A0000091	A0000092	A0000094	A0000095
A0000097	A0000099	A0000100	A0000101
A0000105	A0000107	A0000109	A0000110
A0000150	A0000152	A0000159	A0000160
A0000393	A0001001	A0001073	A0001075
A0001112	A0001300	A0001318	

-ED-T Structures
A0001034
A0002065

-Lakefront Adjoining Orleans, London, 17th, IHNC Canals
-ED-F Geotech

A0001811	A0001813	A0001822	A0002025
A0002027	A0002028	A0002029	A0002030
A0002038	A0003693	A0003694	

-ED-T Structures
A0001839 A0001840

Senate CDs

CD 13 – 24 Oct 05
-01 General Documents
-02 Record of Public Meeting Hurricane Protect Plan (June 1975)
-06 Environmental Statement Final Hurricane Protection Project (Aug 1974)
-07 History of Hurricane Occurrences along Coastal LA (Aug 1972)
-10 TM 3-391 Geological invest of NO Harbor Area (June 1954)

-02 Lake Pontchartrain, Louisiana, and Vicinity
-02 17th Street Outfall Canal (Orleans Parish and Jefferson Parish)
-01 Plans and Specifications
-ED-T Pre Constr Plans
-M&M Supplemental DGNS
-Modeski & Masters Drawings

CD 13 – 15 Nov 05
-02 Lake Pontchartrain and Vicinity, Louisiana (Orleans Parish)
-As Built Drawings
-DACW29-93-0025 (Nov 1992)
-DACW29-95-C-0093 (June 1995)

-Pre-Construction Reports
-Detailed Report Hurricane Study Area No 1 (March 1962)
-DM1 Hydrology and Hydraulic Analysis Part II Barrier (Aug 1967)
-DM1 Hydrology and Hydraulic Analysis Part III Lakeshore (Sept 1968)
-DM Env Impact Statement Phase I REVISED (Sept 1981)
-Excav and Flood Prot of the 17th St. Canal Phase III, Contract 4117 (April 1988)
-TR 2-636 Effects on Lake Pont of Hurr Surge Cont Struc (Nov. 1963)

CD 15 – 15 Nov 05 (disk 1 of 2)
-Contracts
-DACW29-99-C-0018
-DACW29-99-C-0018 Drawings
-DACW29-99-C-0018 MiscDocNo1
-DACW29-99-C-0018 MiscDocNo2
-DACW29-99-C-0018 MiscDocNo3
-DACW29-99-C-0018 MiscDocNo4
-DACW29-99-C-0018 MiscDocNo5

CD 15 – 15 Nov 05 (disk 2 of 2)
-Annual Inspection Maintenance Completed MR&T Flood Contrl Work
-Ann Inspection Maint Completed Flood Contrl Works (2001)
-Ann Inspection Maint Completed Flood Contrl Works (2002)
-Ann Inspection Maint Completed Flood Contrl Works (2003)
-Ann Inspection Maint Completed Flood Contrl Works (2004)
-Ann Inspection Maint Completed MR&T Flood Contrl Works (1998)
-Ann Inspection Maint Completed MR&T Flood Contrl Works (1999)
-Ann Inspection Maint Completed MR&T Flood Contrl Works

(2000)

-Contracts
-DACW29-93-C-0081
-Inspector Quality Assurance Report (QAR)
-<u>17th Street Outfall Canal</u>
-DACW-29-02-C-0016 (Gulf Group Inc.)
-DACW-29-95-C-0093 (Johnson Bros. Corp of LA)

CD 16 – 24 Oct 05
-<u>01 Lake Pontchartrain and Vicinity, LA Hurricane Protection</u>
-Lake Pontchartrain and Vicinity 6
-Lake Pontchartrain and Vicinity 7
-Lake Pontchartrain and Vicinity 8
-Lake Pontchartrain and Vicinity 16 JAN 1987
-Lake Pontchartrain and Vicinity 18 FEB 1997
-LakePont-PAC-approval-7Feb85

CD 09 Dec 05
-HSGAC
(Q1) LTR LK Pontch 17th St canal, HamHwy to South RR, complete 09.05.62
(Q7) -insp91
-insp92
-insp93
-insp94
-insp95
-insp96
-insp97

Status of Remaining Efforts

NWS Technical Report No. 23, published in 1979, is the last update of SPH meteorological parameters, based on hurricane data through 1975. The New Orleans District has requested the National Climatic Data Center (NCDC) to update basic SPH meteorological parameters for Zone B along the central U.S. Gulf Coast. Three parameters will be updated; central pressure index (CPI), pressure gradient, and adjustment for filling over land. Additional work related to the determination of the SPH indices will be determined after the parameters are updated. New SPH windfields will be generated.

The CPI will be updated using data from 1900 through 2005. The updated CPI will be compared with values determined in previous technical reports, and changes in the CPI over the last 30 years will be identified. NCDC will determine the frequency, cumulative percent of occurrence, and occurrences per 100 years of the updated CPI in Zone B.

One of the most important indices used for determining SPH is the pressure gradient, defined as the difference between the hurricane central pressure and peripheral pressure. NCDC will review hurricane data from 1976 through 2005 to determine if any changes have occurred in the mean peripheral pressure since 1975.

As landfalling hurricanes move from open water onto rougher land surfaces, they weaken, and their central pressure fills (weakens). The factors for reducing hurricane wind speeds over land are dependent on the time that the storm center remains over land, the size of the land, and the roughness lengths present over the land mass. Using observations of landfalling hurricanes in Zone B since 1975, NCDC will update the adjustment factors for filling, determine the average rate of filling for hurricanes in Zone B, and compare the updated adjustment for filling over land with earlier results.

New Orleans District will also conduct modeling to evaluate how methodologies used to determine still water levels and waves can affect design elevations and to determine if changes in SPH meteorological parameters, landscape, and critical track can affect design elevations. IPET will assist in the coordination and technical review of this effort.

V. The Storm

Executive Summary

The following information is presented in this chapter: regional hydrodynamic conditions created by Hurricane Katrina (waves and water levels), local high-resolution hydrodynamic conditions at the levees and floodwalls, as well as, hydrostatic and hydrodynamic forces and loadings that the levees and floodwalls were subjected to during the storm. Of particular interest is the temporal variation of wave and water level conditions, and loadings. Maximum conditions are also of great interest as is the timing and phasing of different types of loadings and forces.

A combination of numerical model results and measured data were used to make the assessment of wave and water level conditions along the entire periphery of the hurricane protection system. The WAM and STWAVE wave models, and the ADCIRC storm surge model, were used to characterize the regional wave and storm surge climate produced by Hurricane Katrina. Models were forced with high-accuracy surface wind and pressure fields, and computations were made on high-performance supercomputers. This report reflects progress to date, and represents a point in time that is 60% through the study process.

Observed peak water levels along the south shore of Lake Pontchartrain were 10.7 to 11.7 ft, which were less than or right at the design peak water levels of 11.8 ft. In the Inner Harbor Navigation Canal (IHNC), north of the intersection of IHNC with the Gulf Intracoastal Waterway (GIWW)/Mississippi River Gulf Outlet (MRGO), there is a large gradient in peak water level, from 15.2 ft just south of the intersection to 11.7 ft at the IHNC entrance to Lake Pontchartrain. In this reach of canal, peak water levels were slightly less than, right at, or above the design levels depending on location. Between this intersection and the IHNC Lock to the south, peak water levels exceeded the design level of 13.2 ft by 1 to 2 ft. Along the east-west oriented GIWW/MRGO channel section, peak water levels exceeded the design value of 13.2 ft by 1 to 5 ft. Along the MRGO adjacent to the St. Bernard Parish hurricane protection levee, peak water levels were over 18 ft, which exceeds the design levels by 5 to 6 ft. Along east-facing hurricane protection levees in south Plaquemines Parish, peak water levels reached 20 ft and they exceeded design levels by as much as 6 ft. All elevations cited are referenced to NAVD 88 2004.65 datum.

Peak significant wave height along the south shore of Lake Pontchartrain reached at least 9.4 ft, exceeding design values by about 1.0 to 1.5 ft. Estimated wave periods were about equal to design values. Along the levees adjacent to Lake Borgne, estimated significant wave heights were less than design values but wave periods exceeded the design wave periods by a factor of 3. Since both wave height and wave period influence the potential for wave run-up and overtopping, the design wave height and period values should be re-examined. In south Plaquemines Parish, design wave height conditions were exceeded by 2 to 4 ft and design wave periods were exceeded by a factor of two to three. Design wave conditions should also be re-examined for these levee systems.

An analysis was performed to examine the influence of the MRGO channel on storm surge propagation into the New Orleans vicinity. The section of waterway where the GIWW and MRGO occupy the same channel allows Lake Pontchartrain and Lake Borgne to be hydraulically connected to each other via the IHNC. Storm surge experienced in the IHNC and the GIWW/MRGO section of waterway is dictated by storm surge conditions in both Lakes due to this hydraulic connection. The long northwest/southeast-oriented section of the MRGO channel to the east of Paris Road Bridge, which seems to be the one that has raised the most concern, only influences the storm surge in the IHNC and GIWW/MRGO canals by a few tenths of a foot for high storm surge events (storms like Hurricanes Betsy and Katrina). It has a more important role for low surges, less than 4 ft in amplitude, but still only creates changes of less than 0.6 ft in some cases and less than 0.3 ft in most cases. The MRGO role in propagation of low amplitude astronomical tide and influx of higher saline water into Lake Pontchartrain has been established; the low-amplitude tide propagates primarily through channels, of which the MRGO is one. However, during high storm surge conditions, when the wetlands become inundated, this reach of the MRGO becomes much less important in storm surge propagation into the IHNC and GIWW/MRGO section. A more detailed analysis is provided in the form of a white paper on the subject.

Detailed analysis of waves, water levels and flow in the 17th street canal was completed. Analysis included surge and detailed wave numerical modeling as well as analytical modeling of flow in and near the breach. Observations from local residents indicate that the breach was initiated before 0630 on August 29. Unconfirmed measured water levels at the pump station appear to confirm this time of breach. Water levels at this time appear to be in the range of 6 – 8 ft and waves were roughly 1 to 2 ft. The predictions indicate that the hydrodynamic loads were primarily hydrostatic during this time period. The results of the surge modeling suggest that the without-breach currents in the canal were negligible. However, the currents were substantial in the neighborhood of the breach. The peak breach discharge occurred at approximately 0900 on August 25, 2005 at slightly greater than 40,000 cfs. The minimum sill elevation also occurred at 0900 and was approximately -12.1 feet. These predictions are preliminary and do not include some effects such as damping of the waves from the bridges and debris at the bridges. Analytical analysis of the barge in the IHNC indicate that the barge impact is a potential contributor to failure of a floodwall and variable draft and details of the collision are primary variables in this determination. Boussinesq simulations of wave and surge on and near the MRGO levees indicate a peak average flow depth over the levee crest of approximately 1.5 ft and an average velocity of 6.5 ft/s. On the backface of the levee, the gravity driven downrush velocities occur at maximum overtopping, with wave-averaged values near 10 ft/s, and instantaneous velocities reaching 15 ft/s. Simulations suggest that average backface velocities exceeded 10 ft/s continuously for 1 hour (0730-0830), and 5 ft/s for two hours (0700-0900). From 0630-0900, the simulations predict continuous overtopping. Construction of the physical model of the outer portion of the 17th Street Canal is complete and the model is presently being setup for initial runs.

This chapter references a Wave and Storm Surge Analysis Technical Appendix. This appendix will be released with the final report.

Hurricane Katrina Description and History

The approximate storm track for Hurricane Katrina is shown in Figure V-1. The position of the storm center is shown in blue “X’s”, at particular days/times in late August 2005. All times are referenced to UTC.

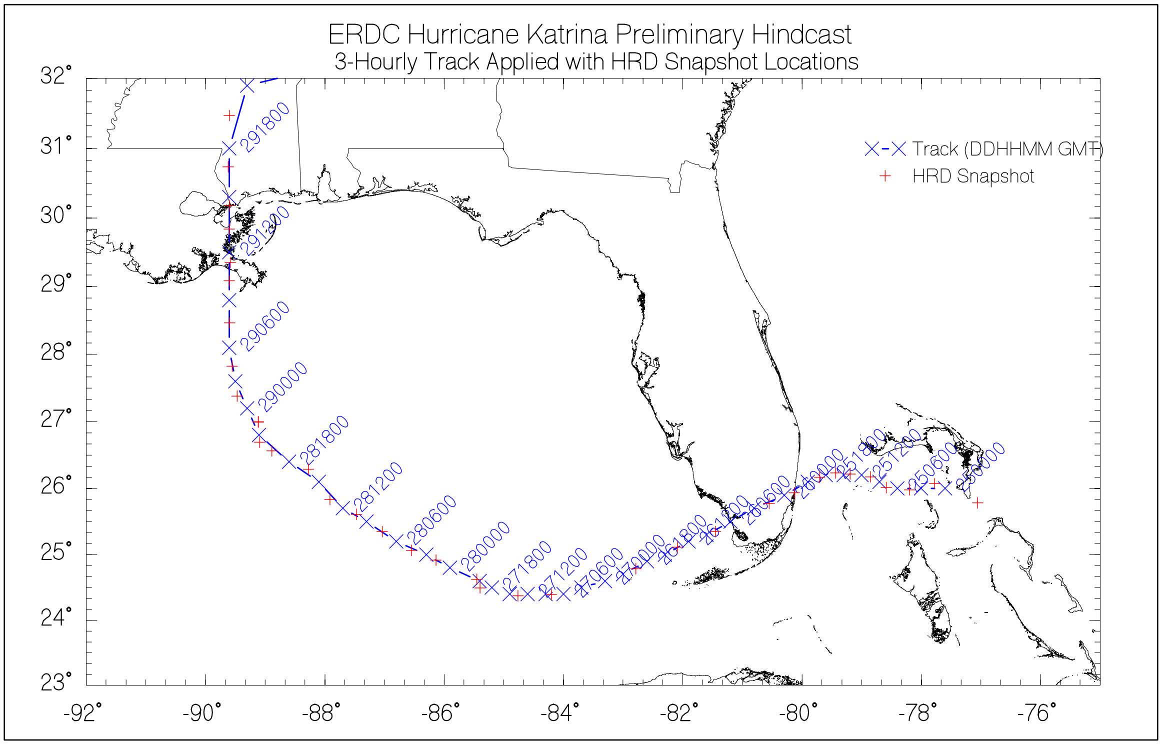


Figure V-1. Hurricane Katrina track

Table V-1 shows the latitude/longitude coordinates for the storm center, the minimum central pressure in the eye of the storm, and the maximum sustained surface wind speed for select times shown in Figure V-1, between 1800 UTC on August 27 and 1800 UTC on August 29. Information in Table V-1 was extracted from the Tropical Cyclone Report for Hurricane Katrina provided by the NOAA

Table V-1
Hurricane Katrina Characteristics

Date/Time (UTC)	Latitude (deg)	Longitude (deg)	Central Pressure (mb)	Maximum Wind Speed (knots)
Aug 27 1800	24.5	85.3	948	100
Aug 28 0000	24.8	85.9	941	100
Aug 28 0600	25.2	86.7	930	125
Aug 28 1200	25.7	87.7	909	145
Aug 28 1800	26.3	88.6	902	150
Aug 29 0000	27.2	89.2	905	140
Aug 29 0600	28.2	89.6	903	125
Aug 29 1200	29.5	89.6	923	110
Aug 29 1800	31.1	89.6	948	80

National Hurricane Center (Knabb, Rhome, and Brown 2005). The information provides a summary of key hurricane characteristics during the time the storm was at its greatest intensity.

Once Katrina emerged in the Gulf of Mexico after passing over the Florida peninsula, it strengthened quickly and by 0600 UTC on August 26 it had again reached hurricane strength. The storm intensified, and early on August 27 Katrina became a Category 3 storm. The Saffir-Simpson hurricane categories are based on maximum sustained surface wind speed. During that day, August 27, the storm tracked primarily westward. At about 0000 on August 28 the storm turned toward the northwest and experienced rapid intensification; it evolved from a Category 3 intensity storm to a Category 5 storm in about 12 hours. Katrina attained its peak intensity at around 1800 UTC on August 28; the maximum sustained surface wind speed reached 150 knots. At this point, the storm was centered approximately 170 miles south-southeast of the Mississippi River mouth headed to the northwest. At about 0000 on August 29, the storm turned to the north; and as it tracked northward it began to diminish in intensity. By the time it made first landfall near Buras, LA, at 1110 UTC, the maximum sustained wind speed had decreased to 110 knots (upper Category 3 strength). Katrina was a very large storm, in terms of its spatial extent, during its migration through the Gulf, and it remained a very large storm even as it weakened prior to and after first landfall. At approximately 1445 UTC on August 29, the storm crossed the Mississippi Gulf coast near the Mississippi/Louisiana border. The maximum sustained wind speed at final landfall was estimated to be 105 knots. Katrina continued to weaken, and was at Category 1 strength by 1800 August 29. Knabb, Rhome, and Brown (2005) provide a much more detailed description of the storm and its characteristics throughout its history.

Time Line of Performance Events

Hurricane protection system timeline

General. The following is a preliminary hurricane protection system time line summary based on qualitative results of water levels and eyewitness accounts. The primary purpose of these efforts is to aid in the development of a probable timeline for the performance of the hurricane protection system. The timeline will be used as another way to assess the system performance and compare numerical and physical model results with field observations. To date, over 200 high-water marks have been identified and surveyed. With respect to the eyewitness accounts, over 600 people have been contacted and over 175 interviews have been conducted with people who observed flooding induced by Hurricane Katrina. Other means of establishing the timing of events have included documentation of stopped clocks in houses, and the collection of videos and still photos. Attempts have been made to get data from security cameras, but these efforts have produced limited results to date. A USACE news release requesting relocated residents of the greater New Orleans area who stayed during Hurricane Katrina and personally witnessed flooding due to levee overtopping or floodwall breaching before relocating to provide information, photos, and any other related data to IPET was published on 16 February 2006 (Appendix G). This was a nationwide news release with a focus on the gulf south region. In addition to the development of the high-water marks and interviews, considerable effort has been expended in establishing the hydrologic connectivity of this extremely complex system. High-water mark collection is nearly complete, however, additional efforts are required to complete the eyewitness activities and develop a final timeline for the hurricane protection system.

For this preliminary timeline summary, nine sub-areas have been identified. The general locations of these areas are shown in Figures V-2 and V-3. These include: (1) 17th Street; (2) London West; (3) London East; (4) South Gentilly/ West Industrial Canal /Upper Ninth Ward; (5) Bartholomew Golf Course; (6) New Orleans East; (7) Lower Ninth Ward and St. Bernard Parish; (8) New Orleans Downtown; and (9) South East Metairie. Although this summary reflects the results of over 175 interviews, it must still be considered preliminary as data are still being collected at this time, and the complete hydrologic picture has yet to be finalized.

1. **17th Street.** Although this area has been covered extensively, the number of people identified as having remained in the area during Hurricane Katrina is fairly small. However, there is some degree of confidence in the results in this area, owing to the credibility and details of the eyewitnesses' accounts. The general consensus is that the initial breach may have occurred early on the morning of Monday, August 29th. While there is the expected wide range of eyewitness times throughout this area, two reliable accounts state that the initial breach was first observed around daybreak (about 0630). One account is from a man in the high-rise building just north of the breach who had a telescope trained on the floodwall area. He reported that just as dawn broke, he saw one section of

Figure V-2. Location of eyewitness sub-areas west of the IHNC

Figure V-3. Location of eyewitness sub-areas east of IHNC the wall was breached (leaning over). Sometime later when he looked the breach had fully developed. Another eyewitness, viewing from directly across the canal from the west wall observed a single section (panel) leaning over at about day-break. He left and came back about 2 to 3 hours later and observed that there were a number of sections all the way down or gone, suggesting full development of the breach. Other eyewitness accounts in the area generally report seeing the first signs of major flooding in the 0900 to 1000 timeframe, with two accounts near the breach describing rapid flooding between 0900 and 0930. The stopped clock data in the vicinity of the breach also support the 0900 to 0930 timeframe. The eyewitness accounts also generally indicate that there was no significant flooding in the area before 0900, which suggest a possible catastrophic type breaching at that time.

Figure V-4 shows a stage hydrograph developed from Pump Station #6 records on 17th Street. As shown in Figure V-4, the stage on the 29th increases until about 0400 where it flattens out and then the stage drops slightly at about 0630, which would correspond with the eyewitness accounts of the first panel breaching. A dramatic drop in stage occurs around 0930, which corresponds with the eyewitness account of the complete development of the breach. Although the stage changes do correspond with the observed eyewitness accounts, further study is needed to insure that these changes in the stage hydrograph don't reflect the passing of the storm surge, pump operations, gage malfunctions, or other factors. There are also questions about pump Station #6 data that must be addressed before its reliability can be accepted.

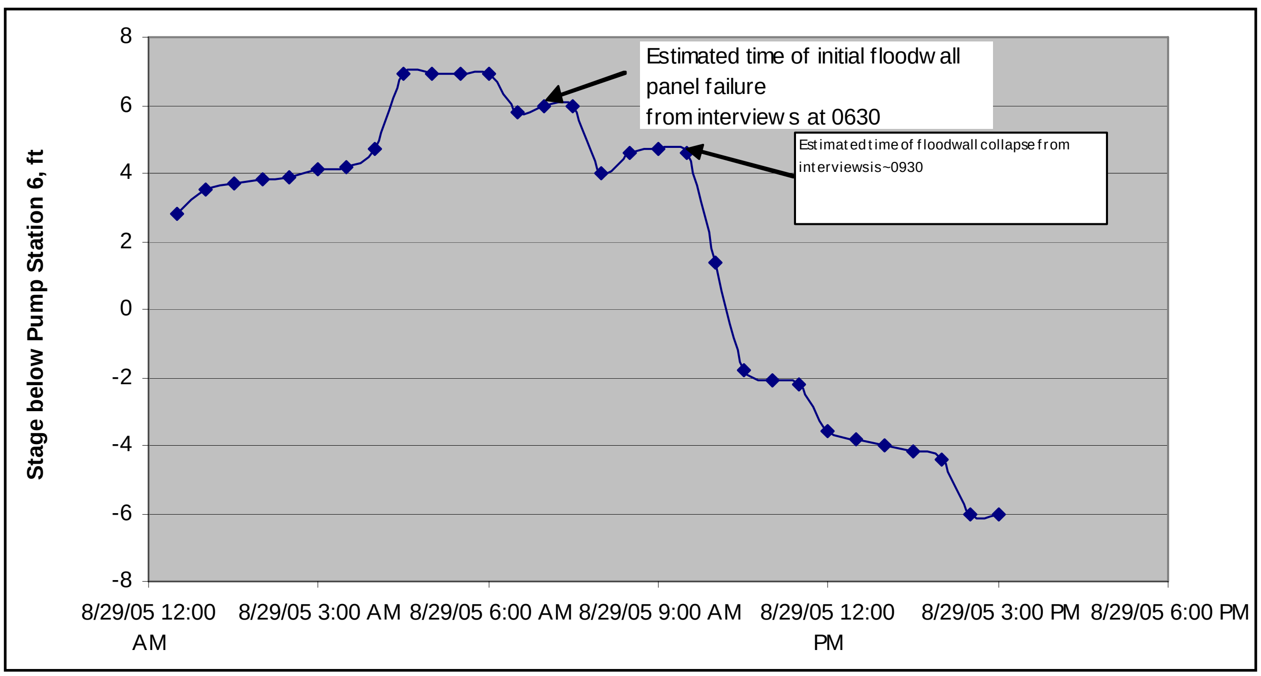


Figure V-4. Stage Hydrograph from Pump Station on 17th Street Canal. This data is being used to supplement eyewitness and clock efforts to determine timing of events, and is not intended to represent absolute elevations

Based on the above data, it appears that the initial failure of the floodwall (single panel) occurred early on the morning of the 29th at least by about 0630, and was probably fully developed (possibly catastrophically) by about 0900 to 0930.

If the initial breach occurred around 0600-0700 in the morning, then according to stage hydrograph data based on digital pictures and eyewitness accounts (Figure V-5), the stage in the canal would only have been at about elevation 6.8 to 7.8 ft NAVD88 (2004.65), which would be well below the top of the wall. According to post-Katrina surveys, the top of the 17th Street floodwall is about 12.4 feet NAVD88 (2004.65) at the breach. The estimated stage at the Lake Pontchartrain end of the 17th Street Canal at 0930 was about 11 ft NAVD88 (2004.65) (Figure V-5).

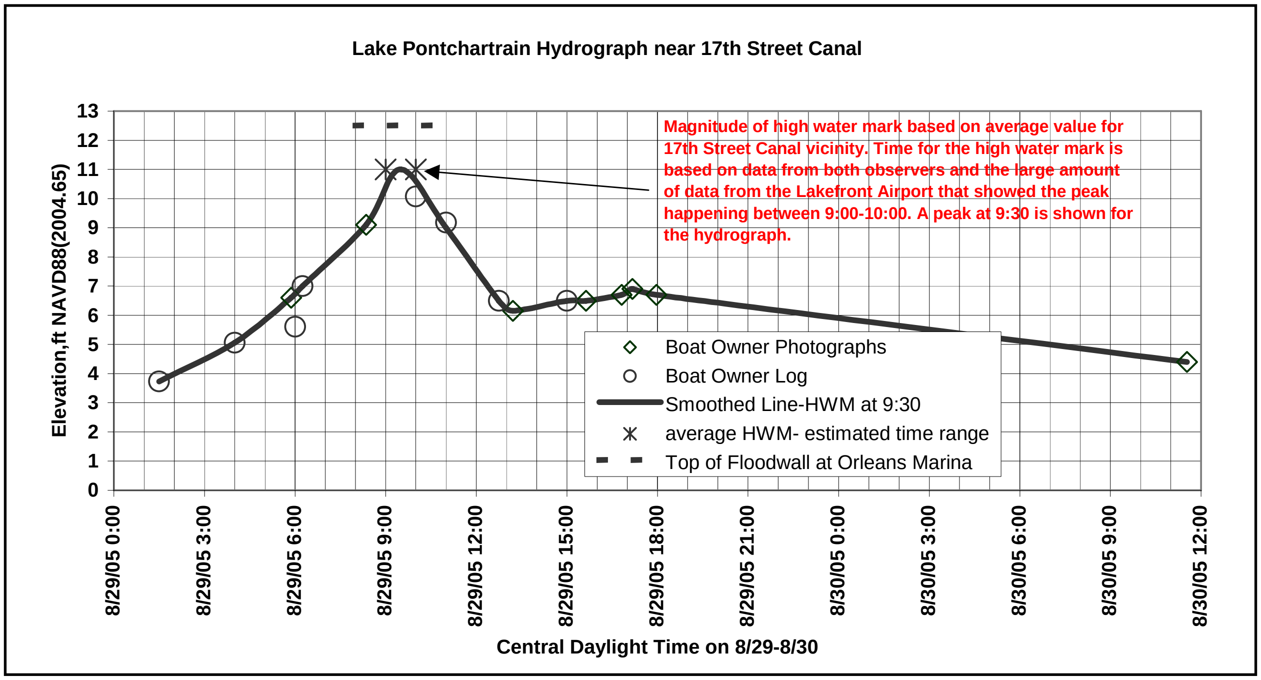


Figure V-5. Stage Hydrograph for 17th Street Canal and Vicinity based on digital photographs and eyewitness account

2. **London West.** Similar to the 17th Street area, there is a scarcity of eyewitness accounts in the London West area. Consequently, there is still some uncertainty with respect to the time of the breach on west side of the London Canal. There are only a few eyewitness accounts in the northern part of the area near the breach. Three of these accounts place the flooding time between 0900 and 1000 on the 29th, and one, which is felt to be a very reliable witness (although he is south of Mirabeau St.) places it between 0700 and 0800. Stopped clock data in the vicinity of the breach is very consistent, with the majority of the times being between 0730 and 0830. Further south (between Mirabeau and I-610), there are more eyewitness accounts. Based on these accounts, it appears that the water began to enter this southern area in the early to mid afternoon period. However, an eyewitness account also reported water flowing south to north over Gentilly Ridge into this area at about 1000.

Based on the accounts at this time, the best current estimate for the time of the breach is sometime before 0900 on the 29th. Additional effort is being expended in the area near the breach to determine if this time estimate can be refined further, and to determine how and when flood waters entered the southern end of the area.

3. **London East.** A large number of eyewitness accounts were conducted in the London East area. Although there is the usual time spread between the data, there seems to be fairly consistent grouping of times between 0700 and 0900, with quite a few reliable accounts between 0700 and 0800 near the east breach. Based on these data, the time of the breach at London East appears to be between 0700 and 0800 on the 29th. It should also be noted that there is video and photography evidence that shows water flowing over Gentilly Ridge into this area from south to north at about 1230 on the 29th.

Assuming the breach occurred between 0700 and 0800, then the corresponding elevation in the canal would have been about 7.8 to 8.8 ft NAVD88 (2004.65), according to the stage hydrograph (Figure V-5). The elevation of the floodwall in this vicinity is about 12.9 ft NAVD88 (2004.65).

4. **South Gentilly Ridge/West Industrial Canal/Upper Ninth Ward.** There are three breach locations on the west side of the Industrial Canal. These include the breach near I-10 through the railroad line, and the breach in the floodwall and earth levee near Pump Station # 19. The elevation of the floodwall along the west side of the canal is about 13 ft while the earth levee is about 10.7 ft.

There are numerous eyewitness accounts in this area that are remarkably consistent. Most recall seeing the first signs of rushing water between 0600 and 0700 on the 29th. Based on these accounts it appears that flood waters may have been coming from the Industrial Canal some time before 0600. Flow over the floodwalls and from the breach or breaches would quickly enter the east-west Florida Canal, thereby providing a possible explanation of the early flooding times as far east as Pump Station #3. The north-south Peoples Canal also provides a direct conduit of water to the northern areas, both north and south of Gentilly Ridge.

The gage records at the Lock and at I-10 (Figure V-6) provide insight into the timing and manner of the breach(s). According to the USGS gage at I-10, there is a dramatic drop in stage of about 5 feet at about 0430 that morning, while the Orleans Levee District Gage flattens out during this same period. Following this period, the stages at both gages continue to rise. While these data should not be viewed as absolute (particularly the 5 foot drop in stage) it does appear that something may have occurred to impact the gage in the 0400 to 0500 timeframe. Thus the preliminary analysis of the gage data may support the eyewitness accounts of early overtopping/breach(s) along the west side of the Industrial Canal. The reliability of this data must be examined closer to ensure that these changes were not due to mechanical problems with the gages. Another complicating factor is that the two large breaches on the east side of the Industrial Canal may have contributed to the stage reduction at the I-10 gages, although

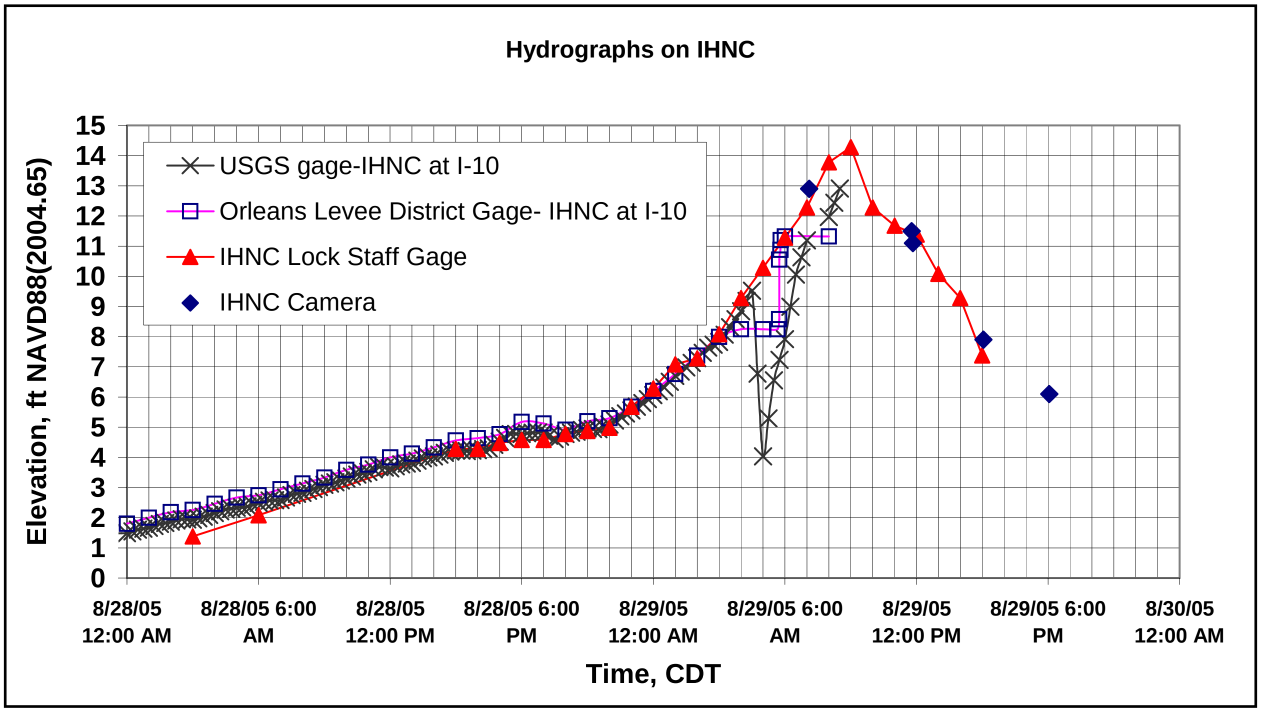


Figure V-6. Stage Hydrographs for IHNC at I-10, and IHNC at Lock

preliminary analyses suggest that they occurred later than the 0400-0500 time-frame. Therefore, while the gage data does provide useful insight, additional hydrologic analysis is needed in order to confirm its reliability.

While the data suggest that this area was inundated very early (between 0600 and 0700, and possibly earlier in some places), the timing of the individual breaches and their timing relative to overtopping is undetermined at this time. Detailed hydrologic analysis of this area is continuing to determine if this can be established with certainty.

5. **Bartholomew Golf Course**. This area is bounded on the north by Lake Pontchartrain, on the east by the Industrial Canal Floodwall, on the south by Gentilly Ridge, and on the west by the railroad grade and Peoples Canal. There are four ways for water to enter this area: (1) overtopping of the hurricane protection levee along Lake Pontchartrain; (2) overtopping of the Industrial Canal floodwall; (3) flow over Gentilly ridge from the south; and (4) from Peoples Canal through the railroad grade. There is considerable uncertainty among the eyewitness accounts in this area, with the majority of the times ranging from mid-morning to late afternoon. There are several reliable accounts that observed the floodwall overtopping early in the morning on the 29th. Both these eyewitnesses noted that this was more wave splashing rather than complete overtopping. They also noted that while this did put water in the street and up to their houses that it ran off quickly, and that the major flooding did not occur till later in the day. Numerous eyewitnesses reported water spewing up through the storms drains consistent with backflow from Peoples Canal. There were also accounts of water coming under the Chef Menteur (Gentilly) overpass into the area from the south. More analyses are needed in this area to narrow this uncertainty.

6. **New Orleans East.** The New Orleans East area is bounded on the south by the Intracoastal Waterway, on the west by the IHNC, and on the north and east by Lake Pontchartrain and marsh lands. Significant levee overtopping and breaches occurred all along the Intracoastal Waterway. There were also a few breaches along the floodwall on the IHNC near I-10, as well as overtopping of the floodwall near the Lakefront Airport. Overtopping also occurred along the levee at Lake Pontchartrain, but to a much lesser degree than on the Intracoastal Waterway. Therefore, the New Orleans East area received flood waters from all directions.

Approximately 25 eyewitness interviews have been conducted in the New Orleans East area. Stopped clock data have also been gathered in this area, as well as video footage of the levee overtopping at the Michoud power plant. Based on these data, it appears that water began overtopping the Intracoastal Waterway levee about 0600 on the 29th, and according to several eyewitnesses, this overtopping continued for about 5 hours. Although there are a number of sources of water for this area, the eyewitness accounts report that the majority of the water came from the south (Intracoastal Waterway). Further hydrological analysis is needed to confirm this. Eyewitness accounts and clock data indicate significant flooding occurred in the area south of Dwyer Road and west of Crowder Road between 0600 and 0800. Video footage shows overtopping of the levee near the Michoud power plant. There is also evidence of from 2 to 5 feet of flow overtopping the railroad grade just south of Chef Menteur from south to north. North of Dwyer Road, the flooding times are a little later, in the 0800 to 1000 timeframe. Several eyewitness accounts just south of the Lake Pontchartrain levee reported flood waters arriving in the 0800 to 0900 timeframe from the south. Farther east of Crowder Road, the times are generally in the late morning to early afternoon. Further analysis is needed to determine if these time differences are due to travel times, topography, or other hydrological factors. Additional efforts are planned for the East New Orleans area to refine these time estimates.

7. **Lower Ninth Ward and St. Bernard Parish.** This area is bounded on the south by the Mississippi River, on the west by the IHNC, and on the north and east by the Intracoatal Waterway and MRGO. The primary sources of flooding for this area are the overtopping and two breaches along the IHNC, and the overtopping and numerous breaches along the Intracoastal Waterway and MRGO. Data in this area includes eyewitness accounts, stopped clocks, and video footage.

To date, there have only been a limited number of interviews in the Lower 9th Ward, primarily due to the fact people have only recently been allowed back in to this area. However, the eyewitness accounts are fairly consistent in this area. Based on these data, the floodwaters appear to have entered the Lower 9th Ward from the IHNC in the 0730 timeframe. Reports at the Jackson Barracks, about 1.5 miles due east of the breaches, indicate a rush of water arriving from the west shortly before 0800.

The stage in the IHNC Lock during the 0730 timeframe was about 13 feet as shown in Figure V-6. Further hydrologic analysis is needed to establish the over-topping and breaching relationships in this area.

To date, no eyewitness accounts of the overtopping or breaching along the Intracoastal Waterway or MRGO have been recorded. Hydrologic analysis is continuing in an effort to establish this timing and the time lag before waters began to enter the Chalmette area. Eyewitness accounts, stopped clock data, and video footage suggest that the floodwaters first entered the areas east of Paris Road (Chalmette) from the northeast (Intracoastal Waterway and MRGO) in the 0800 to 0830 timeframe. Video footage in the Corinne Estates Subdivision in Chalmette provides a good documentation of this flooding. The video also shows large clumps of marsh grass moving in a northeast to southwest direction, clearly indicating flows from the Intracoastal Waterway and MRGO area. These marsh grasses are a common feature on houses and other structures through this entire area, but are rarely, if ever seen west of about Paris Road. Additional analysis is required to refine the time estimates in the Chalmette area, and the area farther east in St. Bernard Parish.

8. **New Orleans Downtown.** At present, only a few interviews have been conducted in this area. Based on these limited interviews, it appears that water started to appear in this area sometime on Monday evening through Tuesday morning. Additional effort will be required to further refine these estimates.

9. **South East Metarie.** A few contacts have been made in this area, but no eyewitness interviews have been conducted yet. Additional efforts are planned for this area.

Regional Hydrodynamics

Summary of Work Accomplished

<u>Development of Wind and Atmospheric Pressure Input</u>

Accurate modeling of waves and storm surge is highly dependent on the accuracy of the wind input to the models. Wind speed is a very important factor influencing the regional wave and storm surge climate, in addition to topographic features which influence wave and surge development and propagation. Surface wind shear stress, the primary forcing to both types of models, dictates the level and frequency of wave energy and storm surge amplitude. Shear stress is non-linearly related to wind speed (a quadratic or cubic dependency) so having accurate winds is crucial. Errors in the input winds are amplified in a non-linear manner. The quality of wave and surge model results is only as good as the meteorological input to the models, particularly wind speed.

Wave and surge models require wind and pressure fields for the entire modeling domain, which for this study included the entire Gulf of Mexico. The work to characterize regional wave and water level conditions was required by several other study tasks, early in the study process. Therefore a spiral development approach was adopted to produce results quickly and then refine the results once other tasks had the information they needed to proceed. The need to produce results quickly dictated the approach that was taken early on.

For the storm surge modeling reflected in this report, wind and atmospheric pressure fields were generated using a Planetary Boundary Layer (PBL) model (Thompson and Cardone 1996). Coupled ADCIRC-PBL models were already in place as a result of prior work done for the U.S. Army Engineer District, New Orleans, so it was utilized while work on the "final" wind and pressure fields was underway. The PBL model employs a moving nested-grid approach (five levels or nests with increasingly higher resolution nearest the storm center) to compute spatially-varying wind and pressure fields as a function of time. For input, the PBL model requires information about the storm position (track), the maximum sustained surface wind speed and central pressure (the type of information shown in Table V-1). Input data for the PBL model were obtained from NOAA. Radius-to-maximum-wind values are computed internally within the five-level model using the method presented in Jelesnianski and Taylor (1973). Radii-to-maximum-winds, which influence spatial variation of the wind field are calculated as a function of central pressure and maximum sustained wind speed. For the final storm surge modeling, wind and pressure fields will be developed using the more rigorous approach outlined below.

For the Gulf-scale and regional-scale wave modeling reflected in this report, preliminary wind fields produced by Oceanweather, Inc. (OWI) were used, which include H*Wind snapshots developed by the NOAA Hurricane Research Division (HRD). An approach which utilized the H*Wind snapshots was taken because the method to link these wind inputs to Gulf-of-Mexico-scale and region-scale wave modeling had been previously developed as part of a National Ocean Partnership Program project, the linkage was readily adaptable for use in this investigation. This methodology for generating surface winds will be adopted to provide input to all final storm surge and wave modeling. The H*Wind snapshots integrated into the preliminary wind fields were primarily based on those created in real-time as part of forecast operations, with some limited re-analysis. The final winds will benefit from a much greater reanalysis effort; which according to HRD staff, is the most intensive analysis of hurricane surface winds that has ever been undertaken by that office.

H*Wind snapshots for the inner core of the hurricane are constructed using a method developed at HRD called the HRD Surface Wind Field Analysis System (Powell et al. 1998, *http://cat5.nhc.noaa.gov/Hwind/*) which utilizes measured meteorological data from a number of different types of sensors and data acquisition processes. All wind measurements are transformed to a standard 10-m elevation, averaging period (1-minute sustained wind speed) and exposure (marine or land). The data are scrutinized for quality. The product of this man-machine mix is a wind streamline and isotach contour plot that is fixed (storm centered) in space and time (see Figure V-7 which is the preliminary snapshot for 1030 UTC on August 29 just prior to landfall). There are 36 unique H*Wind analysis snapshots that comprise the duration of this storm. Snapshots were computed for each of the times denoted with small red crosses (+) in Figure V-1. They represent the best wind estimate for the target domain on which the snapshot is placed. The development of the full domain winds requires two procedures. First, snapshot H*Wind fields are repositioned to the storm track, and then a moving center interpolation algorithm is applied to preserve the characteristics of the tropical storm wind core in space and time.

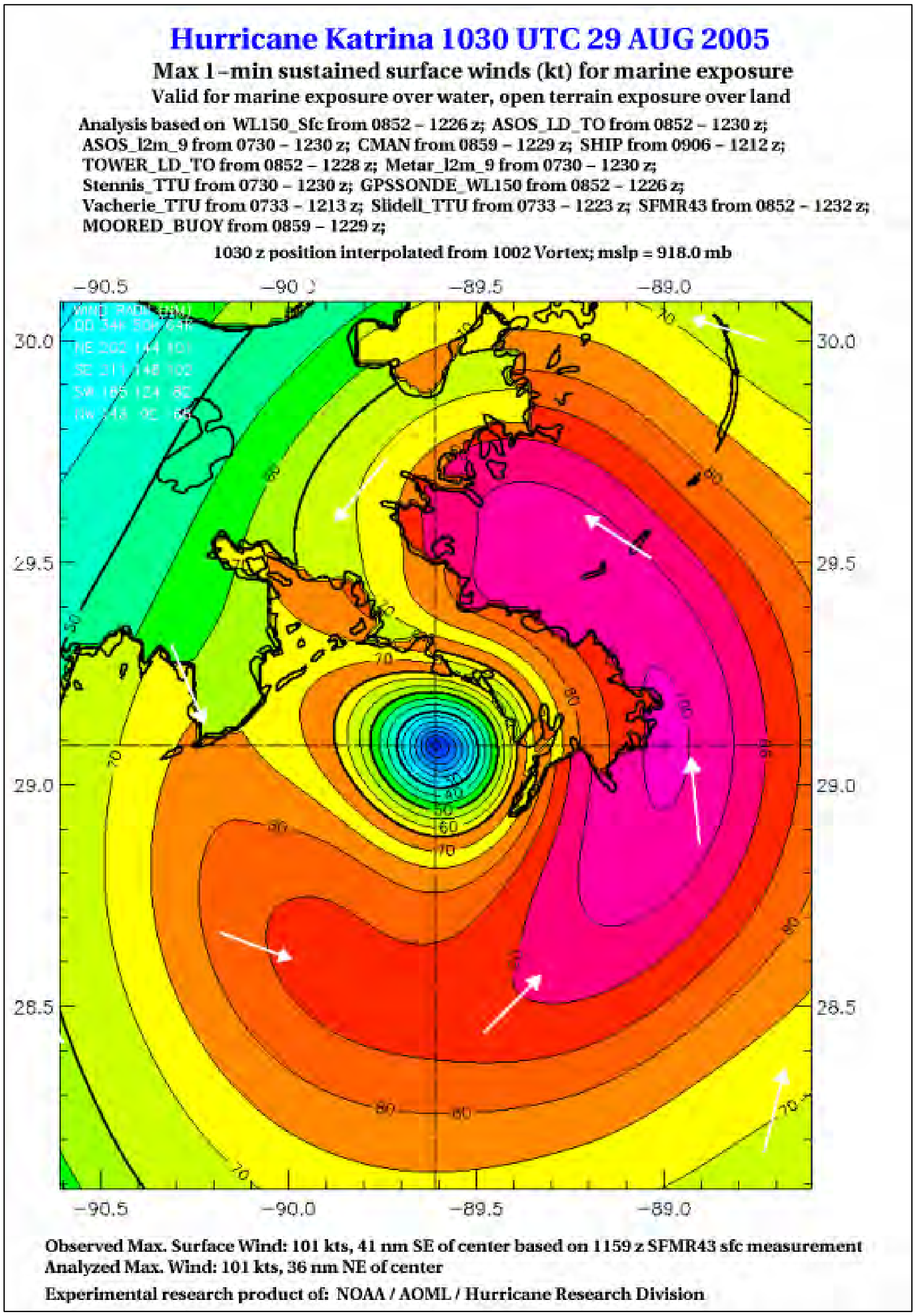


Figure V-7. Preliminary H*Wind snapshot for 1030 UTC on August 29, just prior to landfall (Wind speeds are color contoured in knots, representing 1-minute sustained surface wind speeds. Note this wind field includes both marine and land exposures identified by the abrupt change in color contours over the land)

The wave and surge modeling activities require complete wind field specification for the entire target domain; the H*Wind technique is only used to define wind conditions in the core of the storm. Accomplishing this task requires background estimates which are derived from the NOAA National Centers for Environmental Prediction/National Center for Atmospheric Research (NCEP/NCAR) Reanalysis Project (Kalany et al. 1996). The NCEP/NCAR winds are rigorously analyzed and rely on data assimilation methods using data not originally used in the NCEP operational forecast. A final step is to inject local marine data (adjusted to a consistent 10-m elevation and adjusted for neutral stability. This procedure uses an Interactive Objective Kinematic Analysis (IOKA) System (Cox et al. 1995) developed by Oceanweather, Inc. (OWI). Oceanweather produced the final wind and pressure fields.

Generation of the surface pressure fields follows a slightly different approach using the TC96 model (Thompson and Cardone 1996). This model (TC96) was initially developed over thirty years ago. The model solves, by numerical integration, the vertically averaged equations of motion that govern a boundary layer subject to horizontal and vertical shear stresses. Upgrades and modifications of the TC96 have been made over the development cycle (Cox and Cardone 2000). The pressure fields generated for the Katrina study are built from parameters that are derived from data in meteorological records and the ambient pressure field. The symmetric part of the pressure field is described in terms of an exponential pressure profile from Holland (1980). The pressure field snapshots, aligned to the storm track, are spatially and temporally interpolated in a similar fashion as done for the winds and placed on the identical fixed latitude/longitude grid. No synoptic-scale inputs were considered in this application.

All wind and pressure fields produced by Oceanweather, Inc. (*http://www.oceanweather.com*) were created for two domains, a Gulf-of-Mexico-scale domain (called the basin-scale domain) and a Louisiana/Mississippi regional domain. Specifics of the wind and pressure field domains are provided in Table V-2. Winds and pressures are more highly resolved at the regional scale that at the basin scale. Wind and pressure fields were defined every 15 minutes. Surface winds from OWI represent 30-min average wind speeds. A few results of the wind analysis are presented below. More detail about the process used to generate the wind and pressure fields and the quality of results are contained in the Wave and Storm Surge Analysis Technical Appendix.

Table V-2
Wind and Pressure Field and Offshore Wave Model Domain Characterization

Domain	Longitude (deg)		Latitude (deg)		Res. (deg)	Duration (yr/mon/day/hr)	Wind Input Interval (sec)
	West	East	South	North			
Basin	98 W	80 W	18 N	30.8 N	0.1	2005082500 – 2005083100	900 (30-min avg winds)
Region	91 W	88 W	28.5 N	30.8 N	0.00833	2005082906 - 2005082918	900 (30-min avg winds)

Wind Conditions During Katrina

Figure V-7 shows the sustained surface wind field just prior to landfall. The white vectors in the figure indicate the general wind direction and they reflect the counterclockwise rotation of the wind fields about the storm center. Peak wind speeds are seen to the right of the storm center, which is typical for hurricanes. Maximum surface wind speeds exceed 100 knots. At landfall, along the entire southeastern Louisiana coast, east of the MS River, surface winds are at hurricane force (64 knots) or greater.

Considerable effort is being expended to maximize use of measured meteorological data in the process to create H*Wind snapshots as well as the IOKA process to develop the basin and regional-scale wind fields, because of the critical nature of winds in the wave and storm surge modeling. In many locations, model results are the only source of information for quantifying the wave and water level conditions along the periphery of the hurricane protection system. So it is very important to understand and quantify the accuracy of model input and model-generated results. Comparison of model results to measurements is a very high priority in all facets of the IPET wave and water level analysis.

Figures V-8 and V-9 show comparisons between measured wind speed and direction with the preliminary wind product produced by OWI for two locations, at Southwest Pass to the Mississippi River (Figure V-8) and at the NOAA National Data Buoy Center Buoy 42007 (Figure V-9). Both of these locations (see the map in Figure V-10) are in positions that were east of the storm's path.

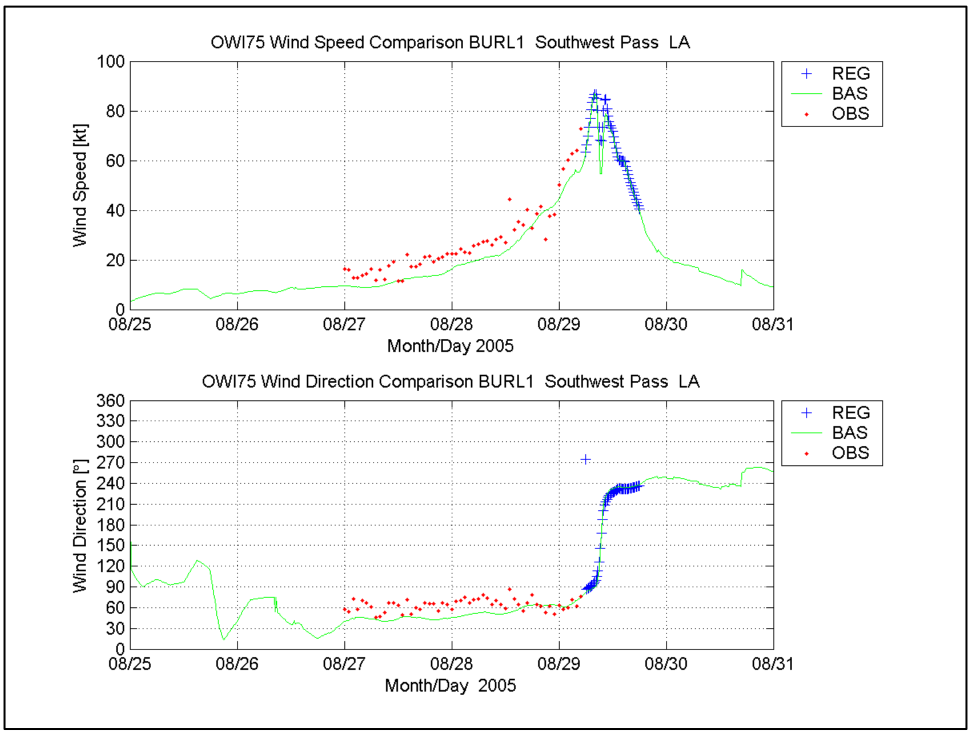


Figure V-8. Comparison of wind speed (upper panel) and direction (bottom panel) at Southwest Pass, LA

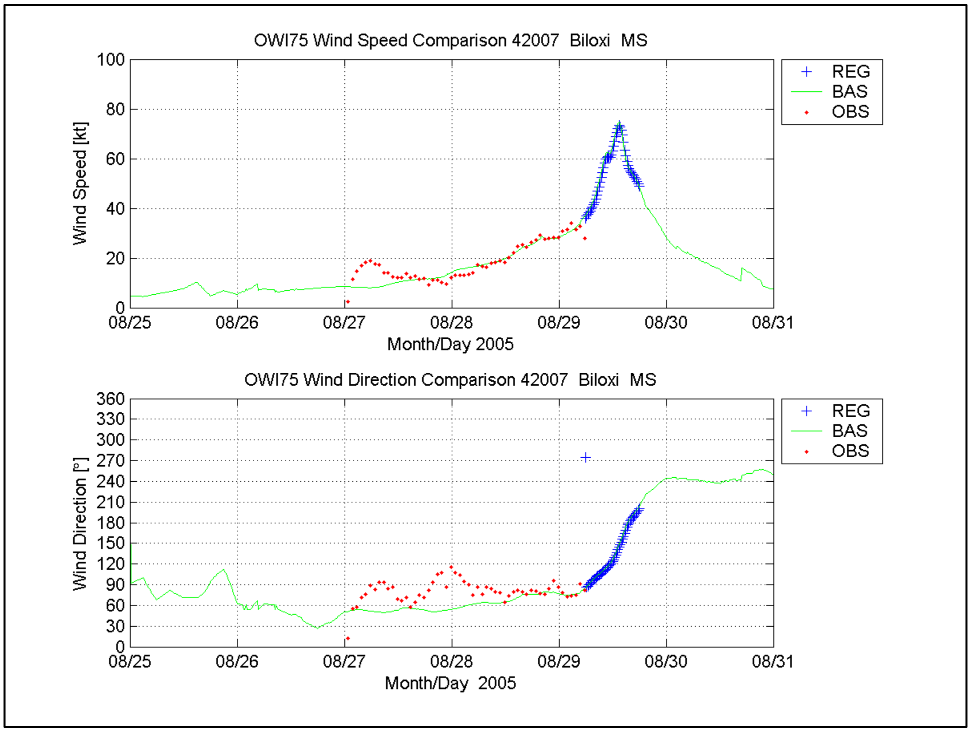


Figure V-9. Comparison of wind speed (upper panel) and direction (bottom panel) at NOAA NDBC Buoy 42007

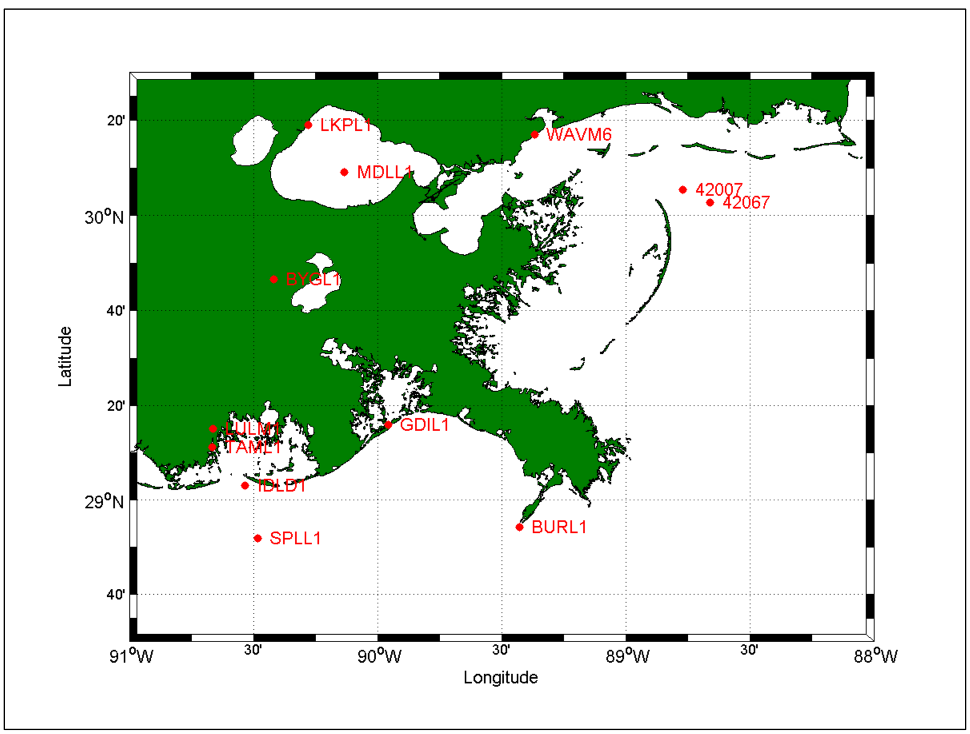


Figure V-10. Wind measurement sites within the regional domain

Figure V-11 shows a comparison for Grand Isle, LA, which was to the west of the storm path. Computed basin-scale winds are indicated by the green line, regional-scale computed winds are shown with blue crosses, and measured winds are indicated with red dots. Note that regional winds were developed for a shorter period of time that encompasses the peak of the storm.

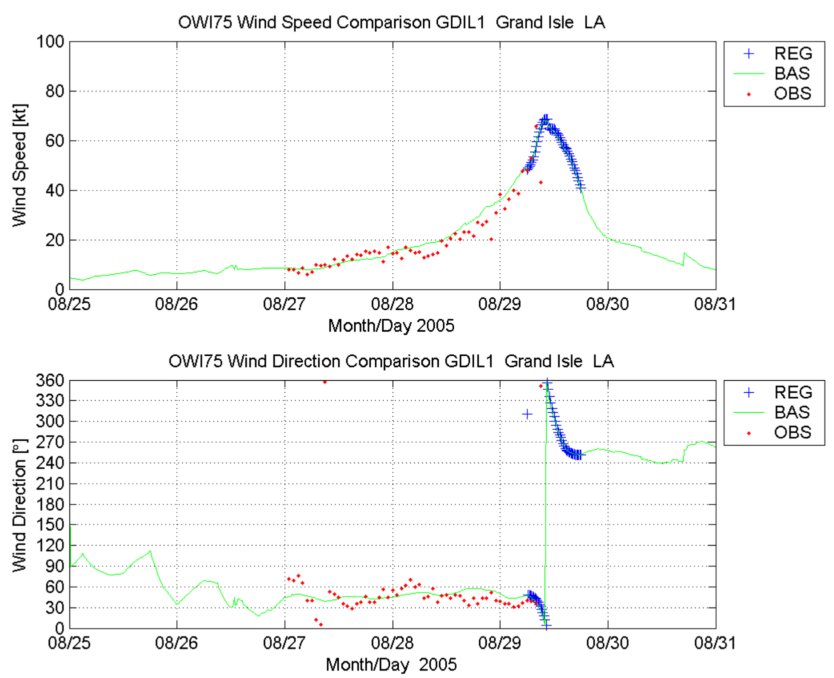


Figure V-11. Comparison of wind speed (upper panel) and direction (bottom panel) at Grand Isle, LA

The H*Wind/IOKA winds show that for at least four to five days prior to landfall, winds were steadily out of the east and northeast and gradually increasing in speed. This trend is confirmed by the measurements. Persistent winds blowing from east to west are notable in that for several days prior to landfall, these winds were acting to push water from east to west along the Mississippi/Alabama continental shelf toward the Mississippi River delta and southeastern Louisiana. This regional-scale movement of water began to build the storm surge in southeastern Louisiana and flood low-lying wetlands well in advance of the storm's arrival. The figures also provide an indication of the accuracy of the windfield products that are being created for use in the wave and storm surge modeling. Overall trends are captured well and magnitudes are reasonably accurate. The greatest errors are in wind direction. Errors are smallest during the day prior to landfall, when wind speeds rapidly increase in magnitude. Additional comparisons are provided in the Wave and Storm Surge Analysis Technical Appendix.

Note that each of the wind measurement sensors near the path of the storm failed prior to the peak of the storm. This was a recurring theme, for wind sensors and water level sensors; failure of instrumentation to function or survive and capture conditions just prior to, during, and after the storm peak, i.e., the crucial part of the storm. There is great need for instruments that can measure surface wind conditions (and water level) reliably during the peaks of severe hurricanes.

Regional Waves Approach

Wave modeling was done to characterize wave conditions just seaward of the hurricane protection system, throughout the entire study region. With one exception (at essentially a single point in Lake Pontchartrain just north of the 17th Street Canal), no shallow-water wave measurements were available that captured wave conditions during the storm just seaward of the levees and floodwalls. Wave measurements were available at a few offshore sites, some of which survived the peak of the storm; but these sites are too far away and in much deeper water, and they can not be used to characterize conditions adjacent to the hurricane protection system. The paucity of nearshore wave data highlights the need for shallow-water wave measurements that are routinely collected for storms and made in ways that can withstand, survive, and record during severe hurricane conditions, and capture the peak conditions. In light of the limited amount of nearshore wave measurements, wave modeling was employed to provide the required information, at the resolution needed, for the very large study area.

Wave modeling was done using a nested approach, with three levels of nesting: 1) basin-scale modeling for the entire Gulf of Mexico; 2) regional-scale modeling at higher resolution for a much smaller domain that encompassed southeastern Louisiana and part of the Mississippi coast, with more resolved wind field input, and 3) nearshore, shallow-water, local-scale modeling which was done at very high 200-m resolution. At each successive nest level, additional resolution was employed to maximize accuracy (resolution is directly related to accuracy) and to treat the important physical processes such as depth effects as accurately as was computationally feasible. Wave boundary conditions for modeling done in each successively refined domain are derived from modeling done at the next coarser domain. The effects of storm surge on water depth were only addressed in the nearshore, shallow-water wave modeling.

The key output product from the most refined nearshore wave modeling work is information to characterize the temporal variation of significant wave height, peak spectral wave period, mean wave direction computed using the full energy spectrum, along the entire periphery of the hurricane protection system that was considered in this study. Maximum wave conditions are also of great interest, and local maxima were compared to the design wave conditions and to the limited set of wave measurements that was available (comparisons to design wave conditions are presented later). Frequency-direction energy spectra were computed at locations where the high-resolution hydrodynamic analysis was done, which required the energy spectra.

Every effort was made to compare model predictions with measured wave data, to assess model accuracy and provide a level of confidence in model-derived results. These comparisons also help assess uncertainty in model predictions. Comparisons were made using measurements from several sources:

1) two small buoys (nearly co-located) that were deployed in Lake Pontchartrain just prior to the storm by the U.S. Army Corps of Engineers, New Orleans District, functioned during the storm, and were recovered after the storm, 2) a number of large NOAA NDBC buoys that are located in deeper water (see Figure V-12 for buoy locations), and 3) satellite-mounted altimeter.

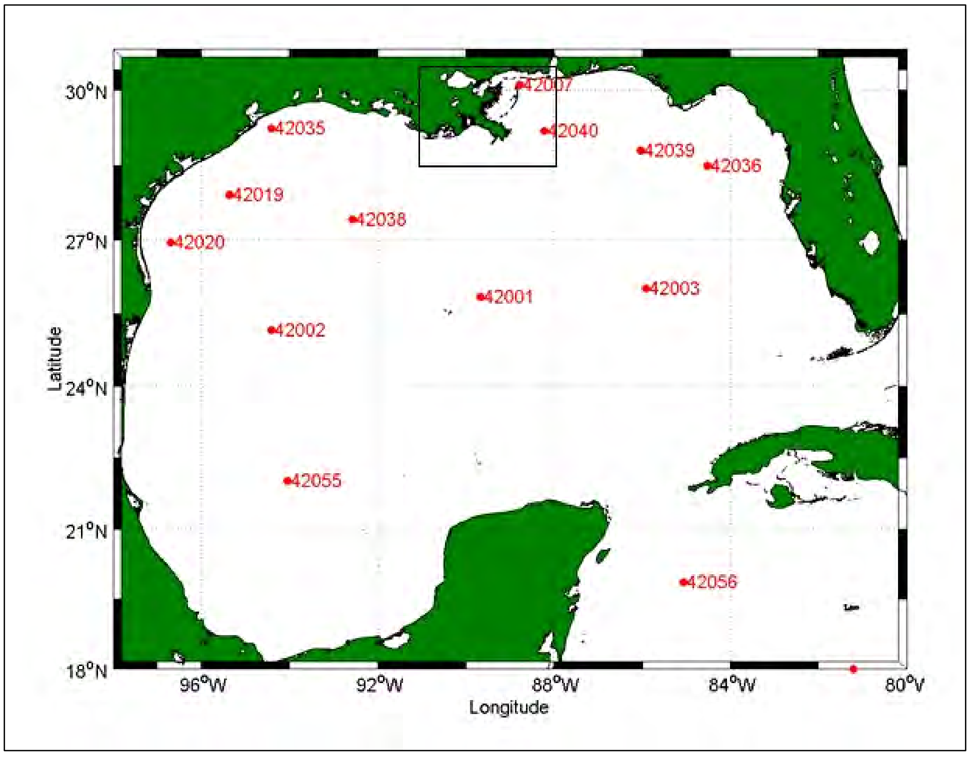


Figure V-12. Offshore wave modeling domains and location of NOAA NDBC buoys

Offshore Waves

Offshore wave-modeling was done using two models, WAM Cycle 4.5 (Komen et al. 1994) and WAVEWATCH III (Tolman 1998, 1999). The WAM model was selected to generate wave conditions for the "production" modeling, since it has been used during the past decade or so by the Corps of Engineers for its detailed wave generation modeling (particularly for hurricanes). The WAM model was applied for basin- and regional-scale domains, the same ones defined in Table V-2. Both domains correspond to those employed in development of wind and pressure fields. Figure V-12 shows the basin-scale domain (entire Gulf) and the regional domain (the black box in the figure that encompasses the Louisiana/Mississippi coastal region). The exact regional domain and the local bathymetry in this area are shown in more detail in Figure V-13.

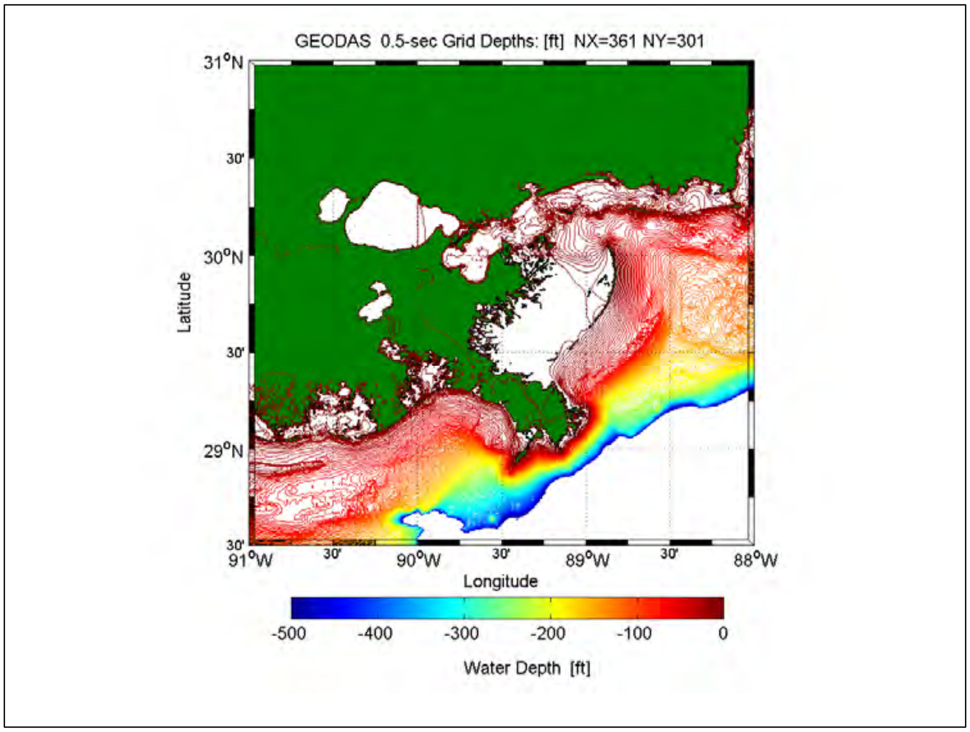


Figure V-13. Regional wave modeling domain and bathymetry

The WAVEWATCH III model was also applied; it is another commonly used model for ocean-scale wave generation and it is the standard model used by NOAA. Wind input for all wave modeling was done using the wind fields described previously based on the H*Wind/IOKA process. The model-to-model comparisons also shed light on uncertainty inherent in the model results.

For the model-to-model comparisons, done for Katrina only, WAM produced slightly better results than WAVEWATCH III at all NDBC buoy locations, particularly in the vicinity of NDBC buoy 42007. Many more details regarding the model-to-model comparisons, using a wide range of statistical error measures, are provided in the Wave and Storm Surge Analysis Technical Appendix.

Figure V-14 illustrates the complexities of the wave field generated by Hurricane Katrina. The figure shows the maximum significant wave height computed at each point in the regional modeling domain, at any time during the simulation. The regional-scale simulation is 12 hr in duration, starting on 29 August 0600 UTC and ending at 29 August 1800 UTC. The overall maximum significant wave height occurs at 89.1417W 28.966N with a value of approximately 53 ft. These wave conditions are extreme. It is important to note that while Katrina was a Category 5 storm prior to landfall, it generated wave conditions that are characteristic of a storm at that intensity. Those large waves propagated outwards from the storm and impacted coastal Louisiana and Mississippi.

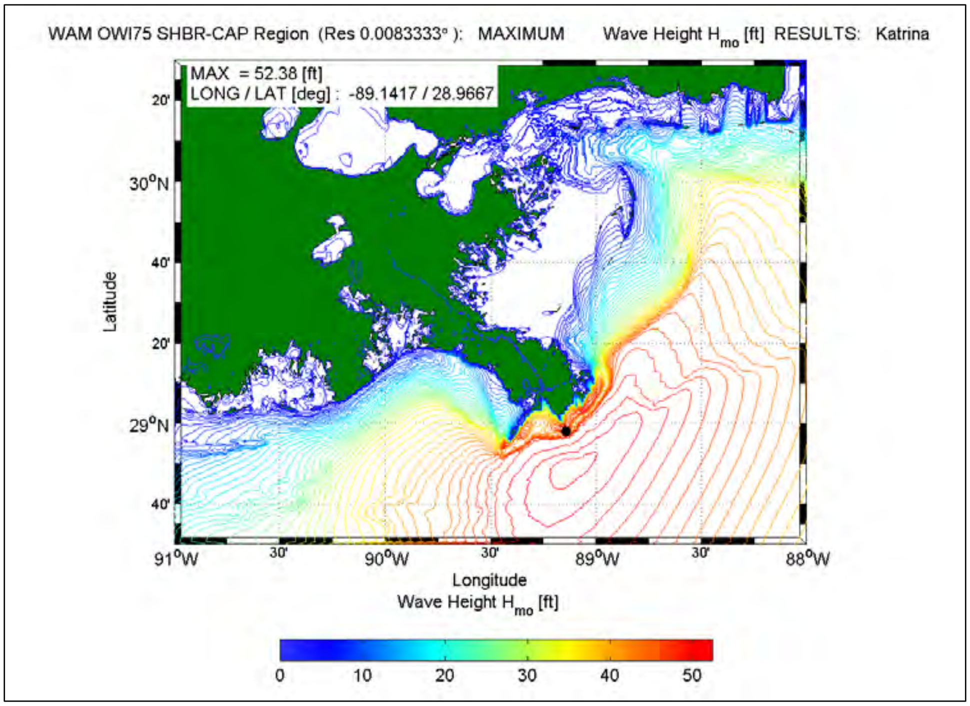


Figure V-14. Color contour of the maximum wave height conditions in the Region domain for the simulation period 2005082906 through 2005082918 UTC

Shallow water effects of shoaling and more importantly refraction focus the offshore energy towards the Mississippi River delta. When waves break due to their arrival in shallow water, wave energy decreases. In areas dominated by depth-induced breaking, significant wave heights are generally on the order of 60% of the local water depth. For example, a sea state in which the significant wave height is about 40 ft would begin to experience considerable depth-limiting breaking in about 65 feet of water. This tendency is evident in the dramatic decrease in wave height along the Mississippi River Delta. It is also apparent along the southeastern Louisiana barrier island chain where considerable energy dissipation takes place well seaward of the barrier islands due to depth-induced breaking. The pattern of wave height maxima follow the bathymetry pattern closely (compare Figures V-13 and V-14), an indication of depth limited breaking effects. Offshore, deeper-water wave conditions along the southeastern Louisiana coast are computed to be 35 ft in the northern areas, increasing to approximately 50 ft adjacent to the Mississippi River delta.

The WAM simulation assumes constant water depths, i.e., no changes due to storm surge. Therefore WAM results landward of the barrier islands indicated in Figure V-14 will be lower compared to expected results when storm surge effects on water depth (increases) are considered. The nearshore wave modeling considers this effect.

The maximum mean wave period results for the regional WAM simulation are shown in Figure V-15. This figure illustrates the diverging wave climate east and west of Hurricane Katrina's path. To the west, the mean wave period is

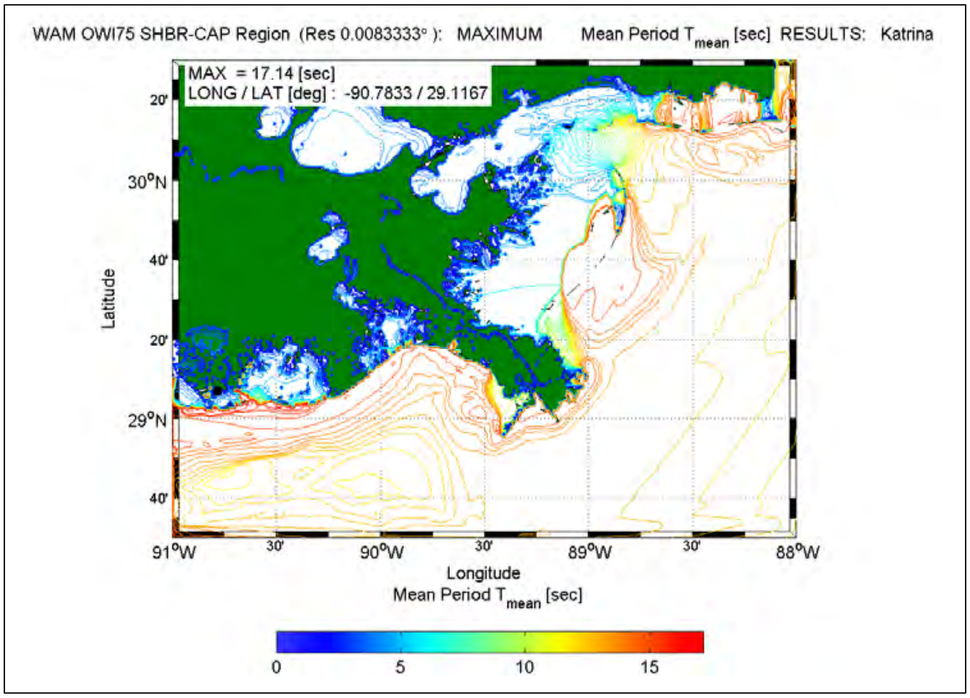


Figure V-15. Color contour of the maximum mean wave period conditions in the region domain for the simulation period 2005082906 through 2005082918 UTC

dominated by swells, having periods ranging from 12 to more than 15 sec; whereas, in the front right hand quadrant of Katrina, local wind seas abound with limited, yet distinct long period swell lobes. Long-period swells are present, but considerable energy is also present at higher frequencies. Shadow zones in wave period appear (lower T_{mean} values) also are evident in the lee of capes or islands. Also evident are zones of large mean period values landward of island gaps (around Horn and Dauphin Islands along the Mississippi coast) in the eastern portion of the Mississippi Sound.

Comparisons of wave model results with measurements are an important facet of the work. A few of those comparisons are presented below. A much more detailed description of the offshore wave modeling work, additional model-to-measurement comparisons, and much more information on the model-to-model comparisons are presented in the Wave and Storm Surge Analysis Technical Appendix.

Comparisons of WAM results to measurements made at NOAA NDBC Buoys 42040 and 42007 are shown here. Of all the buoys for which data are available, these two are in locations that best reflect the wave climate that southeastern Louisiana was subjected to during the storm. Buoy locations are shown in Figure V-12. Comparisons for Buoy 42040 are shown in Figure V-16 and comparisons for Buoy 42007 are shown in Figure V-17. Each figure shows a comparison for energy-based significant wave height, peak and mean spectral wave periods, mean wave direction, wind speed, and wind direction.

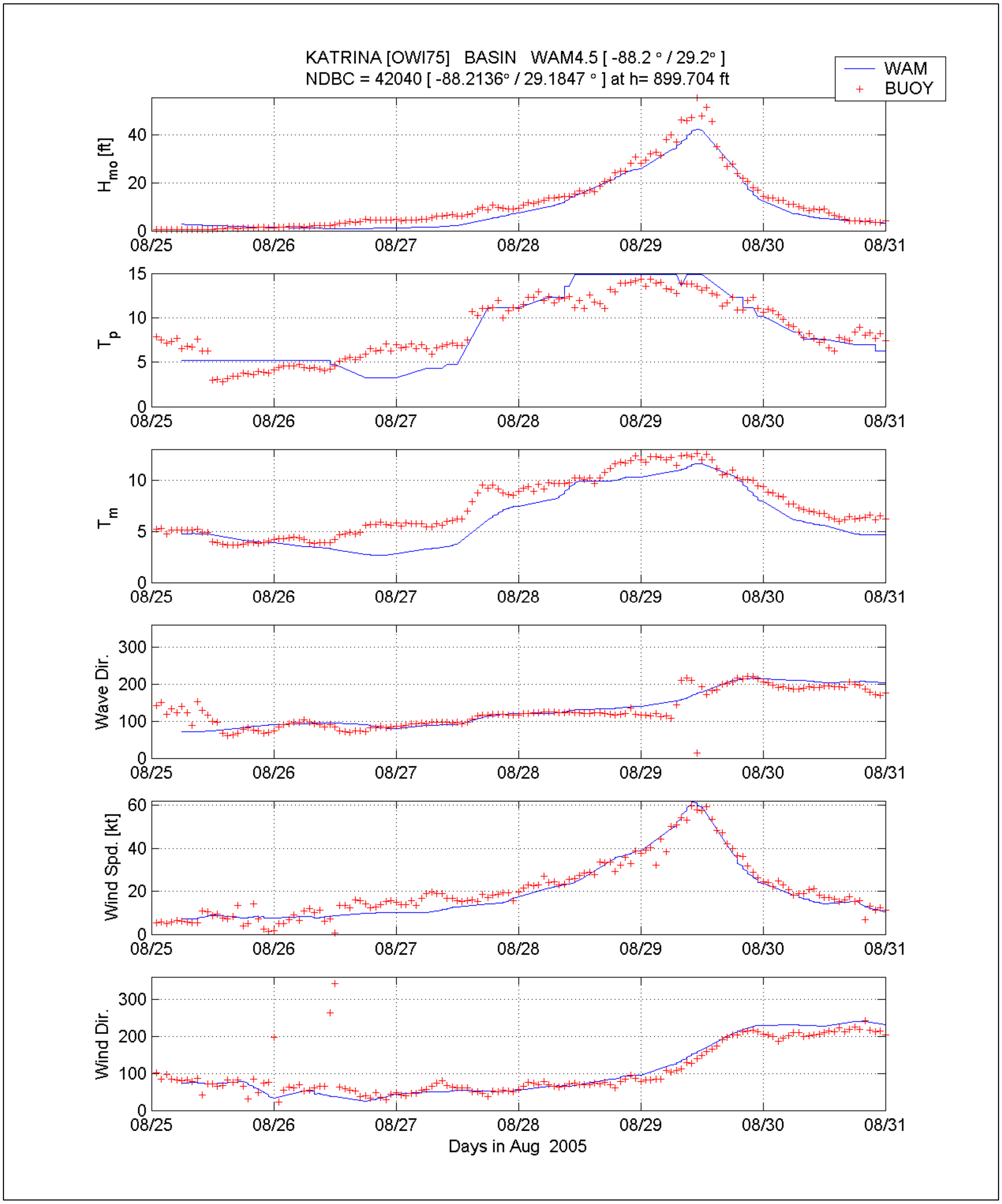


Figure V-16. Comparison of WAM Cycle 4.5 basin-scale (blue line) to the measurements at NDBC 42040

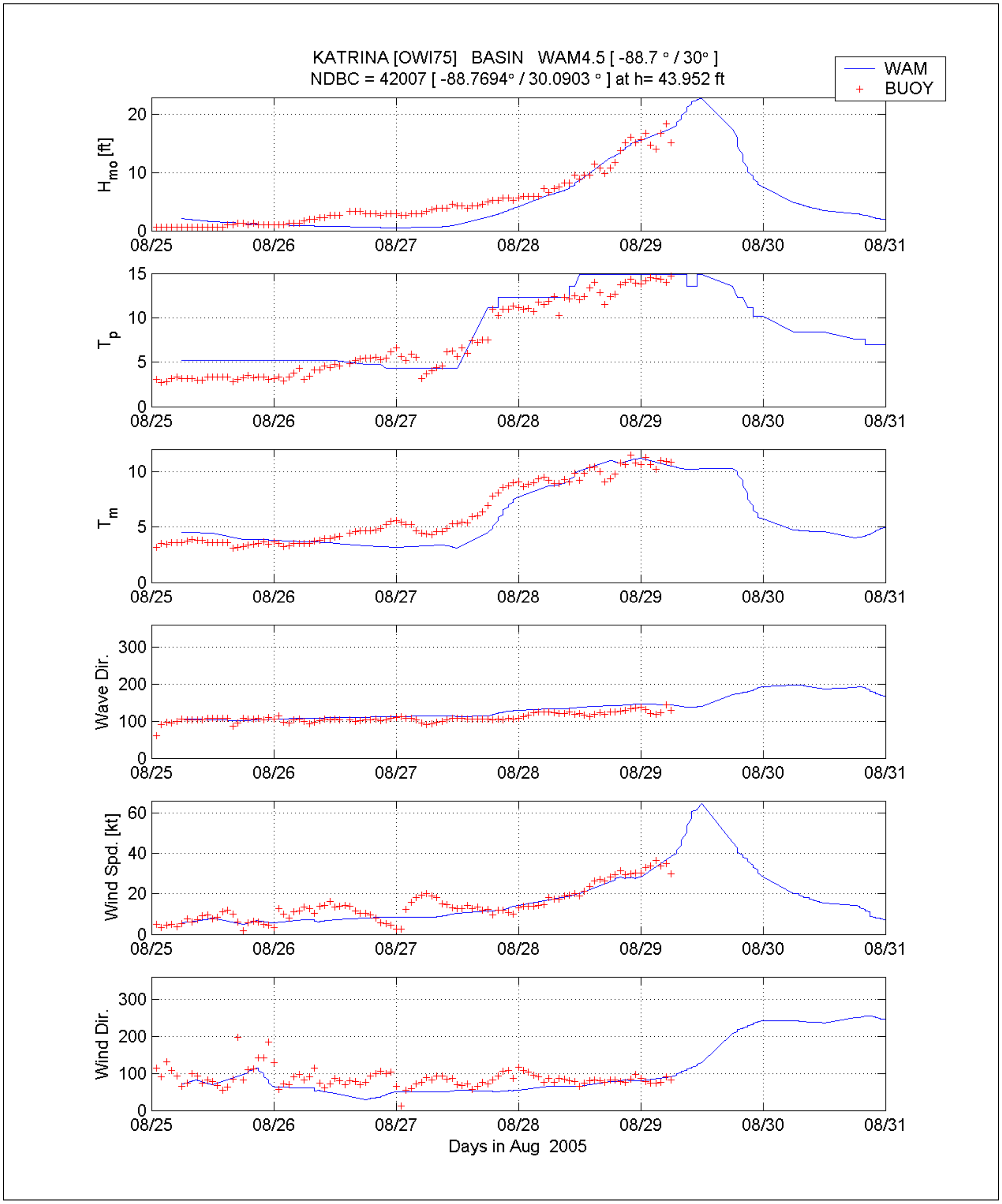


Figure V-17. Comparison of WAM Cycle 4.5 basin-scale (blue line) to the measurements at NDBC 42007

The maximum height measured at 42040 is approximately 55 ft, which is believed to be the largest significant wave height ever recorded by an NDBC buoy. Measured peak wave periods are between 13 and 15 sec near the storm peak. The maximum computed significant wave height is about 42 ft, and the computed peak periods at this time are 15 sec. At this point, it is unclear how the final winds will influence these results, but there are indications that the preliminary wind fields are low during some of the most intense stages of the storm. Computed wave directions agree reasonably well with measured wave directions.

The final comparison is made to the most landward buoy, located in the shallowest water depth in the NDBC Gulf of Mexico array. Buoy 42007 is located just west of the northern tip of the Chandeleur Island chain in a water depth of 44 ft. It is unfortunate though that this buoy did not survive Katrina and as evidenced by the wave record; it failed well before the storm peak. During the growth stage of the storm, measurements indicate a methodical, slowly increasing wave height that is dominated by wind-seas (characterized by short periods on the order of 5 sec) until 27 August 1800 UTC where there is a dramatic shift in T_p, an indication of the early arriving swell energy that reaches southeastern Louisiana well before (2 days) arrival of the intense core of the storm. The down-shifting in frequency (or increasing T_p) continues, with the increase in wave energy until failure of the buoy. Approaching the time of failure, there is only a modest change in the vector mean wave direction, changing by at most 30 deg. This should not be surprising because to the south, west, and north there is considerable sheltering due to the influence of land features. Thus there is a very small window available to receive wave energy at this location. Prior to 28 August, wave heights are under-predicted. After 28 August, model results agree reasonably well with measurements. The maximum computed significant wave height at this location is approximately 23 ft, with peak wave periods of 15 sec. Computed wave directions agree well with measured directions. It is clear that the hurricane has spawned energetic long-period swells which propagate into the region.

The primary purpose of the offshore wave modeling task is to provide boundary condition information to the nearshore wave modeling effort (all the nearshore domains). An example of the directional wave spectrum provided as a boundary condition to the nearshore wave modeling is shown in Figure V-18. The spectrum reflects the directional distribution of the incident wave energy as a function of wave frequency (frequency is inversely related to wave period). In Figure V-18, the red vector indicates a mean wave direction, here showing waves approaching from the southeast. The colored area indicates the spectral region encompassing all wave frequencies and directions that are present in the sea state at this location. The red colors indicate the frequency-direction characteristics that contain the highest energy levels (the integrated energy-based significant wave height is almost 13 ft).

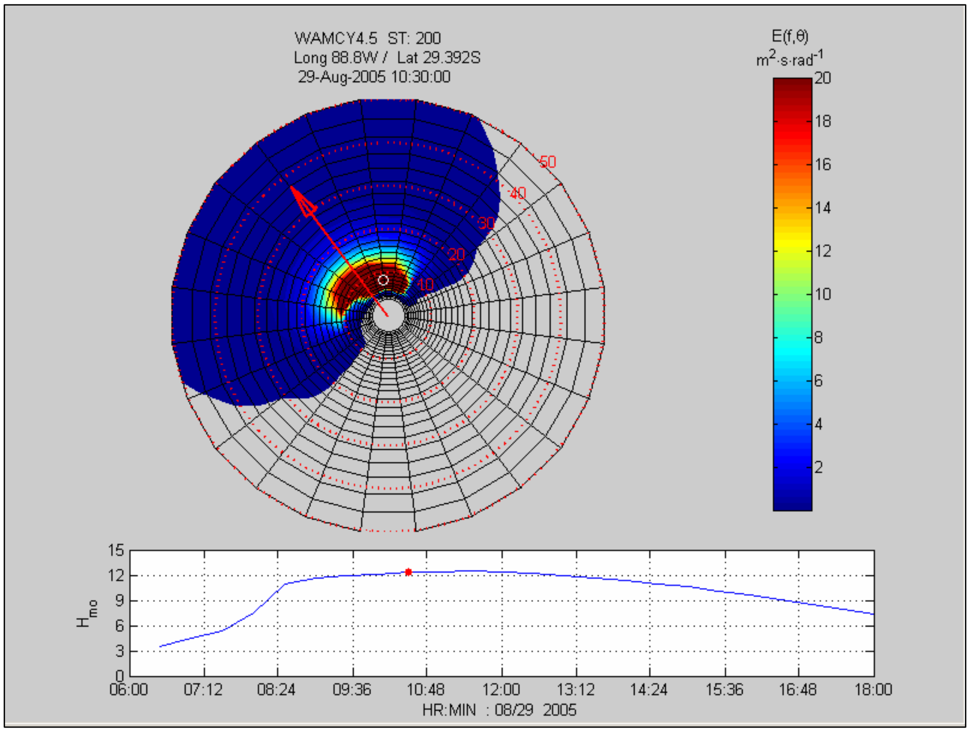


Figure V-18. Example of the directional wave spectra color contoured in the upper panel and the significant wave height trace in the lower panel (note units are in CGS system)

Nearshore Waves

The STWAVE model (Smith, Sherlock, and Resio 2001) was adopted for the nearshore wave transformation modeling; it is the standard model used by the Corps of Engineers to simulate nearshore wave transformation. All "production" runs and results presented in this report were made with STWAVE.

STWAVE was applied on three grids for the southern Louisiana area (Lake Pontchartrain, Louisiana Southeast, and Louisiana South). The input for each grid includes the bathymetry (interpolated from the storm surge model bathymetry), surge fields (interpolated from storm surge model output), and wind (the preliminary OWI/H*Wind wind fields). For the Pontchartrain and Louisiana South grids, the wind applied in STWAVE is constant over the entire domain and is taken from approximately the center of each grid. Spatially variable winds were simulated on the Louisiana Southeast grid, and STWAVE was run at 30-min intervals from 0630 to 1800 UTM on 29 August 2005 for the Southeast and South domains (matching the regional wave simulation that supplied input boundary conditions) and at 30-min intervals from 0000 on 29 August 2005 to 1200 on 30 August 2005 for the Pontchartrain domain.

A few modeling results are presented below for two of the three model domains, Lake Pontchartrain and Louisiana Southeast, where the greatest wave action occurred along the hurricane protection system. The Wave and Storm Surge Analysis Technical Appendix describes the nearshore wave modeling work in more detail, and it contains more results including those for the Louisiana South domain.

Lake Pontchartrain Grid. The first grid covers Lake Pontchartrain at a resolution of 656 ft (200 m) (north-south) by 656 ft (200 m) (east-west). The domain is approximately 15.5 by 24.9 miles (25 by 40 km). Lake Pontchartrain is run with the full-plane STWAVE to include generation and transformation along the entire lake shoreline. The full-plane version of the model considers wave growth, propagation, and transformation for the complete 360-degree plane. The grid parameters are given in Table V-3. Figure V-19 shows the bathymetry for the Lake Pontchartrain Grid relative to NGVD 29. Brown areas in the bathymetry plots indicate land areas at 0 ft relative to the datum.

Lake Pontchartrain Results. The peak wave conditions on the south shore of Lake Pontchartrain occur at approximately 1400 UTC on 29 August 2005 (9:00 a.m. CDT). The wind at this time is 59.5 knots (30.6 m/sec) approximately from the north. Figure V-20 shows the maximum significant wave height for the entire simulation period for each grid cell within the domain. The wave direction that corresponds to the time of maximum wave height is also shown. The maximum wave height is 9.5 ft with a peak wave period of approximately 7 sec. The maximum wave heights range from 8.5 to 9.5 ft on the New Orleans vicinity lakefront and the associated peak periods are approximately 7 sec.

Table V-3
STWAVE Grid Specifications

Grid	State Plane	X origin ft	Y origin ft	Δx ft	Δy ft	Orient Deg	X cells	Y cells
Lake Pontchartrain	LA South	3563779.5	690485.6	656	656	270	208	337
Louisiana Southeast	LA Offshore	4294586.6	1639491.5	656	656	141	683	744
Louisiana South	LA Offshore	3997126.0	1264895.0	656	656	108	664	839

At the entrance to the 17th Street Canal, the maximum significant wave height was computed to be 8.7 ft; and the peak period at that time was 6.7 sec. At the time of maximum wave conditions, waves were approaching from directions just to the west of north. At the entrances to Orleans Avenue and London Avenue Canals, peak significant wave heights and corresponding peak periods were 8.8 ft and 6.7 sec peak period, and 9.1 ft and 6.7 sec, respectively. Peak waves approached from just west of north at both sites. The maximum computed wave heights along Orleans East (east of IHNC) were 8.8 ft and corresponding peak periods were 6.7 seconds. The peak waves approached from the northwest.

Three small wave buoys were deployed by the U.S. Army Engineers, New Orleans District, in Lake Pontchartrain on 27 August 2005 to capture wave conditions during the storm. Two of those gauges were recovered and provide valuable comparison data. The deployment locations were 30 deg 2.053’ North, 90 deg 7.358’ West for Gauge 22 and 30 deg 1.989’ North, 90 deg 7.932’ West

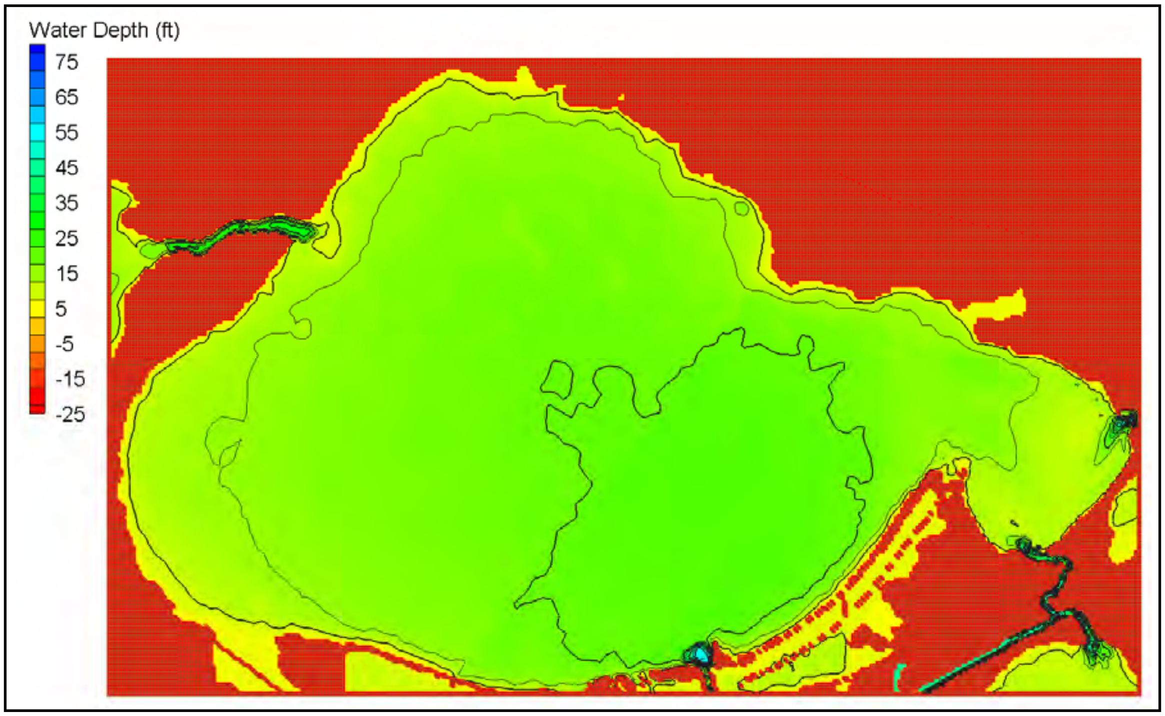


Figure V-19. Lake Pontchartrain bathymetry grid (depths in feet, NGVD 29)

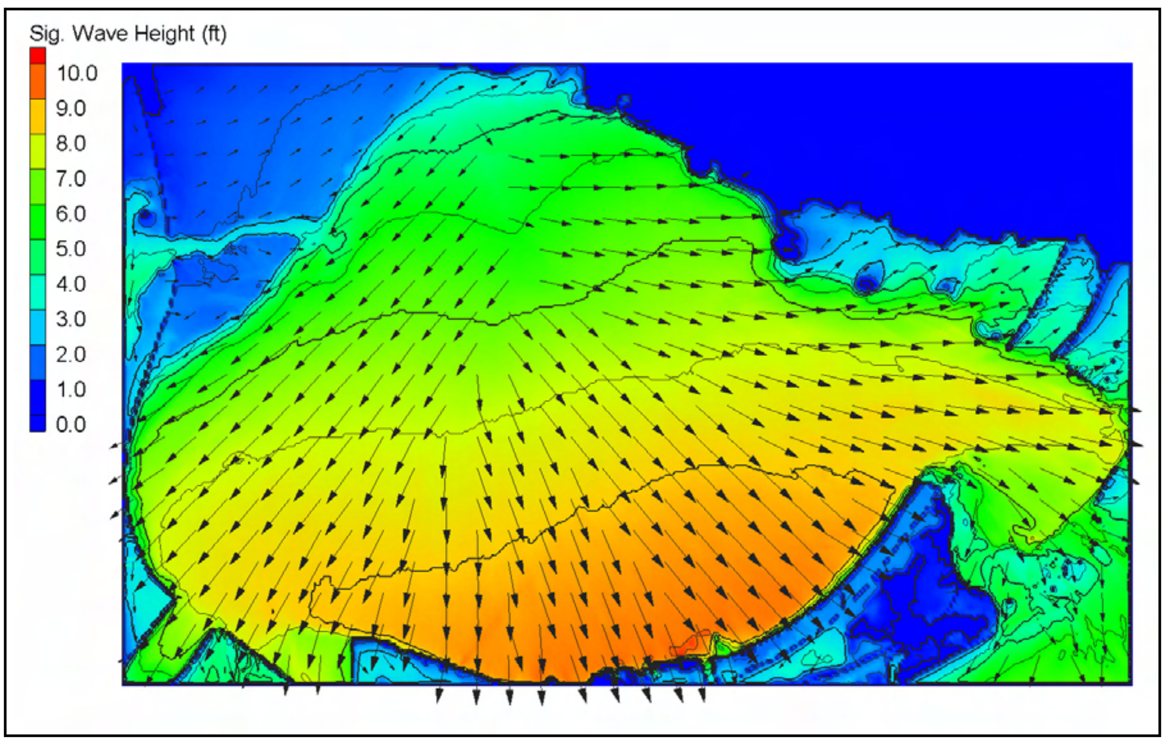


Figure V-20. Lake Pontchartrain maximum modeled significant wave height and corresponding mean direction for 0000 UTC on 29 August to 1200 UTC on 30 August 2005 (wave heights in feet) for Gauge 23. Gauge 22 was directly north of the 17th Street Canal entrance and Gauge 23 was west of Gauge 22. Both gauges were in approximately 13-ft (4-m) water depth. The sampling records were a relatively short 8.5 min, so there is a lot scatter in the data. Also, at the peak of the storm, the wave heights drop from approximately 8 of 9 ft to 5 ft. The developer of the bouys has examined the data and concurs with our assessment that the data appear to be inaccurate near the peak; the buoy appears to have tilted to an extreme value under the action of the most extreme winds near the peak.

Figures V-21 and V-22 show comparisons of significant wave height and peak spectral wave period for the buoy locations, respectively. The symbols without lines are the 8.5-min measured wave parameters; the blue lines are the measurements with the spectra averaged over 3 records (25.5 min), and the red lines are the modeled parameters (30-min average). The STWAVE results are essentially the same for the two gauge sites. The modeled wave heights are approximately 1 to 2 ft lower than the measurements in the building part of the storm (0630-1200 UTC) and very similar to the measurements in the waning part of the storm (1500-1800 UTC). The measurements at the peak are not reliable. The modeled peak periods are consistent with the measurements, from 0.0 to about 0.5 sec low in the building stage and just prior to the peak,, and 0.5 to 1.5 sec low in the waning stages of the storm.

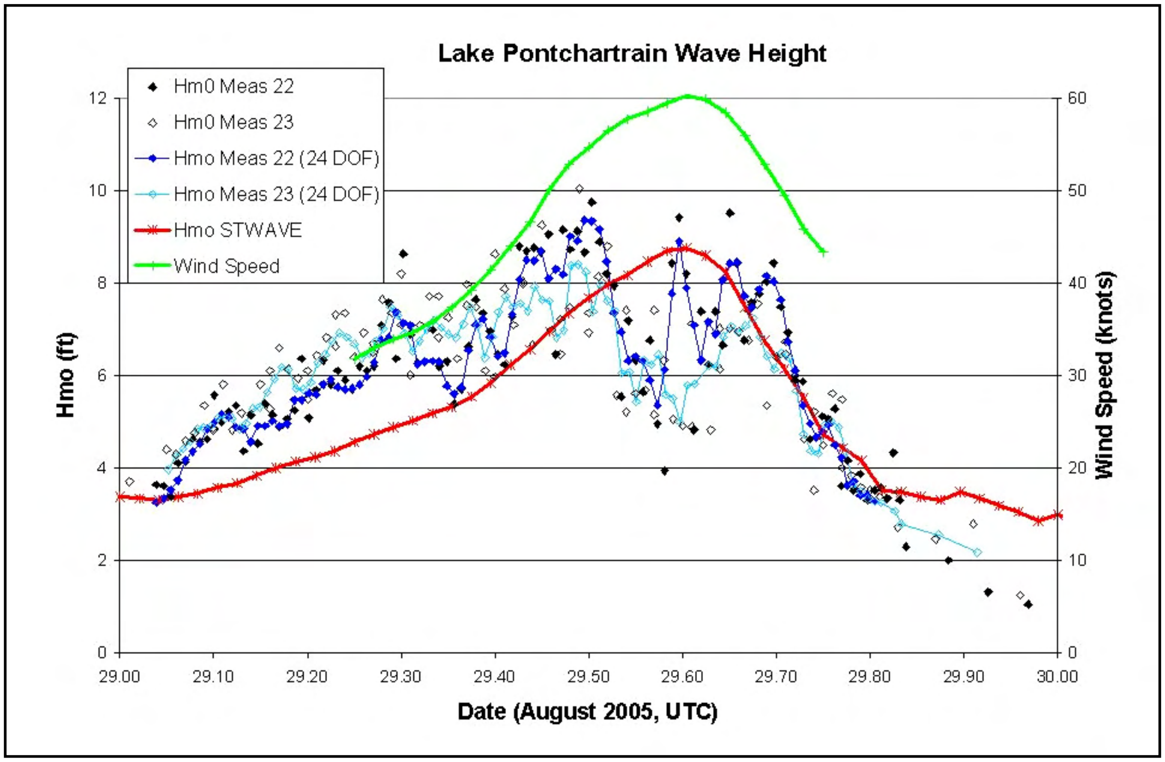


Figure V-21. Lake Pontchartrain measured and modeled significant wave height

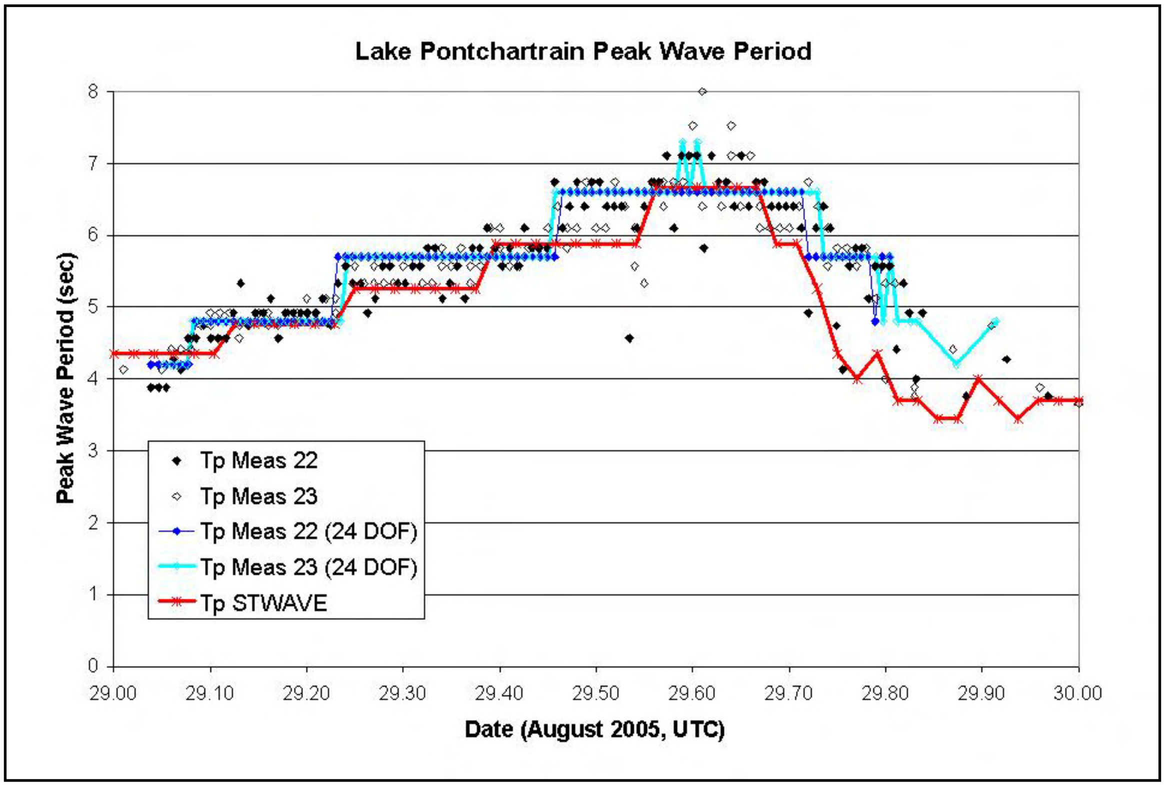


Figure V-22. Lake Pontchartrain measured and modeled peak wave period

STWAVE is a steady-state wave model, which means that the waves reach equilibrium with the local forcing conditions (wind, surge, and boundary waves). Thus, the STWAVE modeling assumes that the winds and surge vary slowly enough for the waves to reach quasi steady state. For Hurricane Katrina, the winds are time varying and the grid domains are relatively large, so the time-dependent SWAN model (Booij, Ris, and Holthuijsen 1999; Booij et al. 2004) was used to evaluate the importance of time variation. Lake Pontchartrain was chosen for this test because the waves are all locally generated and time dependence is expected to have the greatest impact there. Measured data in Lake Pontchartrain (the only available data) enable comparisons between model results obtained using the steady/unsteady approximations and measurements, and assessment of model-to-model differences in light of model-to-measurement differences.

To test the importance of time dependence, SWAN was run in both steady-state and time-dependent modes for 29 August 2005 from 0630 to 1800 UTC. The comparison was made using 1-min time steps for the time-dependent run and forcing the steady-state run to an accuracy of 99 percent with a maximum of 15 iterations (this is a more stringent iteration parameter selection than the default values). All other SWAN model defaults were used. The time-dependent simulation requires about 2.5 hours of simulation time to ramp up (0630-0900), but following this time, the differences in wave height along the southern New Orleans lakeshore are less than 2 percent (average difference is 0.2 percent), with the steady-state simulation giving slightly higher wave heights. The average directional difference is less than 3 deg and the periods are essentially the same. Based on these results, time dependence is not a concern in the hurricane simulations in the nearshore domains, and steady-state simulations will be used for the 95% solution (final results). Run times are significantly reduced for steady-state compared to time-dependent simulations.

STWAVE wave heights are an average of 2 percent higher than SWAN results. STWAVE wave heights are higher at the peak of the storm and lower height on the building and waning legs of the storm, compared to SWAN results. The computed peak significant wave height using SWAN was 7.7 ft, about 1 ft less than the peak value computed using STWAVE (8.7 ft). The measurements are not reliable at the peak of the storm, when the wave heights are most critical. Just prior to the point in time the measurements appeared to become suspect (decreasing heights despite increasing winds), the maximum wave heights measured at the two buoy locations were 8.4 and 9.4 ft. SWAN results are closer to the measurements on the building portion of the storm and STWAVE results are closer on the waning portion of the storm.

STWAVE peak periods are 9 percent longer than the SWAN peak periods on average. STWAVE shows better agreement with the wave period measurements, but both models are generally are within 1 sec of each other. The maximum peak period computed with SWAN was 5.7 sec, about 1 sec less than the maximum computed with STWAVE (6.7 sec). The measurements suggest maximum peak periods of 6.7 to 7.3 sec.

In general, overall, STWAVE produced slightly better results. SWAN predicted a broader wave event, i.e. wave height and period results more slowly varying with time, than did STWAVE. Figures showing results from these comparisons are provided in the Wave and Storm Surge Analysis Technical Appendix.

Louisiana Southeast Grid. The second grid covers the coastal area southeast and south of New Orleans at a resolution of 656 ft (200 m). The domain for the southeast grid is approximately 84.9 by 92.4 miles (136.6 by 148.8 km) and extends from Mississippi Sound in the northeast to the Mississippi River in the southwest. The southeast grid was run with the half-plane version of STWAVE for computational efficiency. The grid parameters are given in Table V-3. Figure V-23 shows bathymetry for the southeast grid.

Louisiana Southeast Results. The peak wave conditions on the southeast grid occur between approximately 1100 and 1200 UTC on 29 August 2005. The highest waves along the Mississippi River levees occur around 1100 UTC (6:00 a.m. CDT) and along the Lake Borgne shoreline around 1200 UTC (7:00 a.m. CDT). Figure V-24 shows the maximum significant wave height and corresponding mean wave direction for the entire simulation period for each grid cell within the domain. Figure V-25 shows the peak wave period field that corresponds to the time of maximum significant wave height.

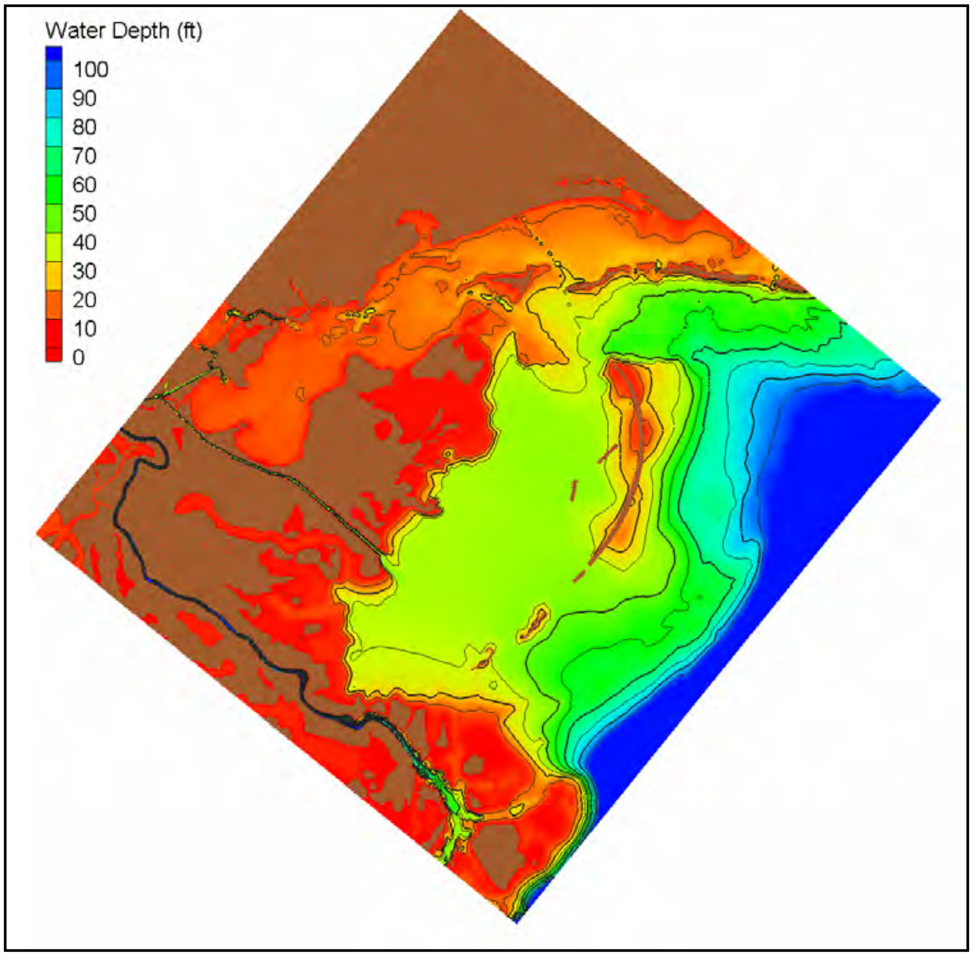


Figure V-23. Louisiana Southeast bathymetry grid (depths in feet, NGVD 29)

The maximum significant wave heights range from 6 to 10 ft along the levee system and the associated peak periods are 7-16 sec. The longer wave periods originate from wave energy traveling between the islands from the Gulf of Mexico. Larger wave heights occur in lower Plaquemines Parish (7-10 ft) and smaller heights in upper Plaquemines and St. Bernard Parishes (5-6 ft). The peak periods are relatively large (up to 16 sec) because of wave penetration through gaps between the barrier islands.

Along the back levee of Orleans Parish, adjacent to the GIWW, maximum computed significant wave heights and peak periods were 5.2 ft and 16.3 sec, respectively. Peak waves approached from the southeast. Along the St. Bernard Parish hurricane protection levee adjacent to the MRGO, with an eastern exposure, peak wave heights and periods were approximately 4.9 to 5.2 ft and 16.3 sec. At the time of peak wave conditions, waves approached from the southeast, rather obliquely, relative to the levee system. Along the portion of the St. Bernard Parish hurricane protection levee with a southern exposure, peak wave heights were less, about 2.3 ft and peak periods were quite long, 18.0 sec. Here, waves approached from the south at their peak conditions.

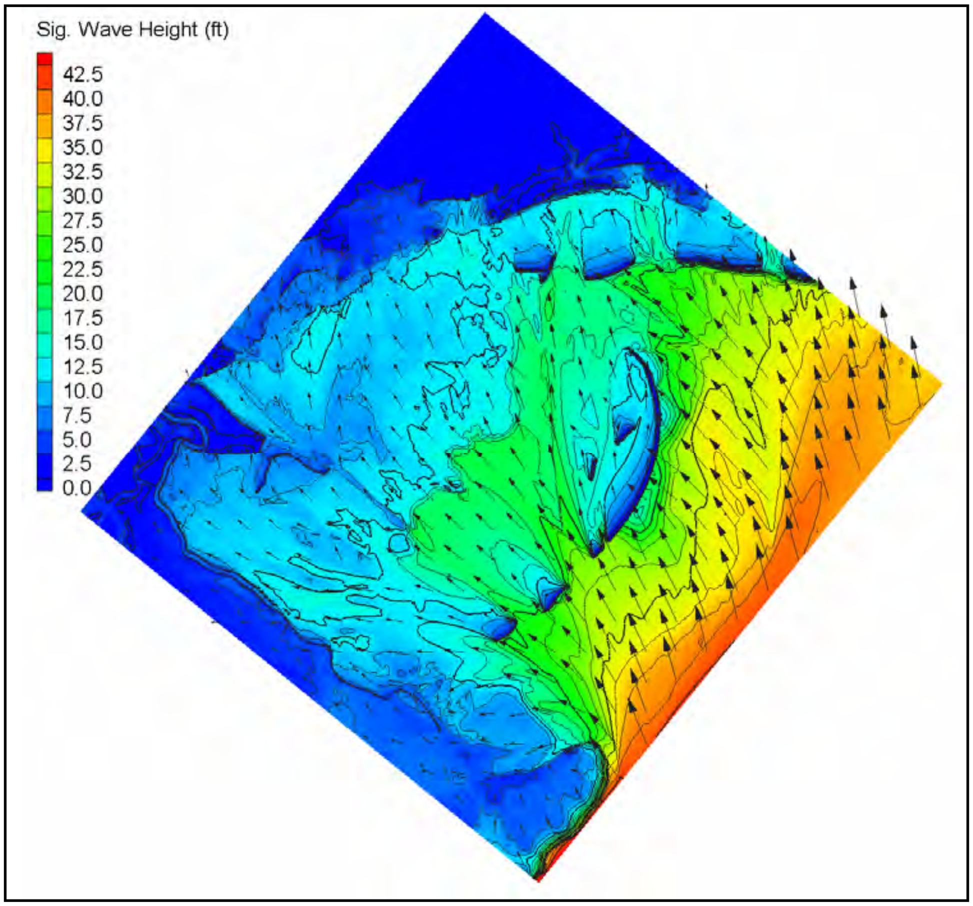


Figure V-24. Maximum significant wave heights, and corresponding mean wave directions, for the Louisiana Southeast domain, for time period 0630 to 1800 UTC on 29 August 2005 (wave heights in feet)

In southern Plaquemines Parish, along east-facing levees of the hurricane protection system on the east side of the Mississippi River, maximum significant wave heights ranged from approximately 7.4 to 9.4 ft, and the associated peak periods were 13.5 sec. The longer wave periods originate from wave energy traveling between the barrier islands from the Gulf of Mexico. In the southern-most portion of Plaquemines Parish, south of Tropical Bend maximum wave heights were 7.2 to 8.0 ft, with periods ranging from 13.5 to 14.9 sec.

Information pertaining to nearshore wave modeling for the Louisiana South domain and information for the levees on the west side of the Mississippi River, is contained in the Wave and Storm Surge Analysis Technical Appendix. Later in this chapter, additional information is presented that compares current best estimates of peak wave conditions with those used in the design of the hurricane protection projects.

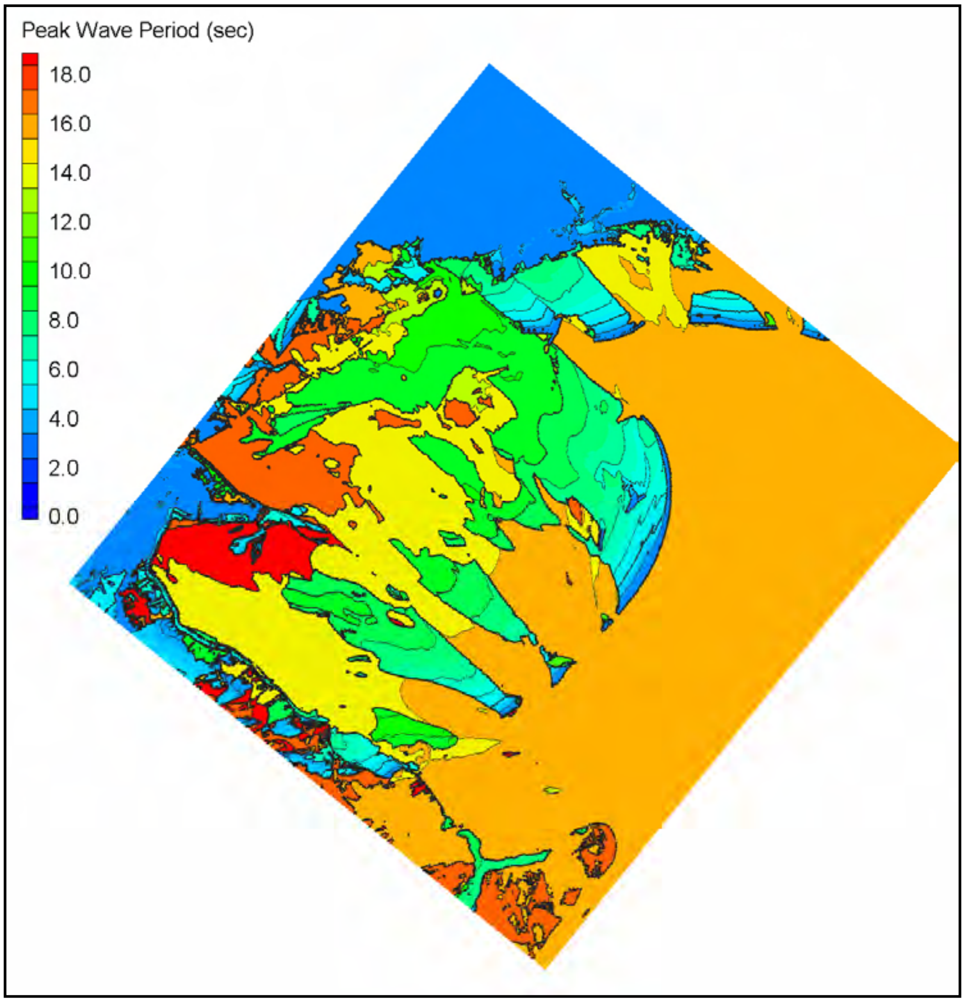


Figure V-25. Southeast Louisiana modeled peak wave period corresponding to the maximum wave height for 0630 to 1800 UTC on 29 August 2005 (periods in sec)

Regional Water Levels Approach

A combination of measurements and numerical modeling using the ADCIRC model was used to develop information with which to characterize the temporal variation of water level and local water level maxima associated with Hurricane Katrina. Development of the ADCIRC model of southeastern coastal Louisiana (Westerink et al. 2005, Feyen et al. 2005) has been underway for several years. The term water level as used in this chapter describes the more slowly varying water surface (variations that occur on the time scales typically associated with the astronomical tide or storm surge, changes of tenths or whole feet per hour). Contributors to water level that are considered in the current modeling are tide, wind and atmospheric-pressure drive storm surge, and water discharge within the Mississippi River. Precipitation and other water inflows are not included.

Variations on these time scales are contrasted with the much more rapidly varying water surface associated with shorter-period wind wave action (oscillatory motions in which the water surface can vary on the order of feet at times scales of up to tens of seconds. Wind waves were discussed in the previous sections.

Measured water level hydrographs are the most reliable source of data for capturing both the temporal variation and the maximum. Water level fluctuations were measured during the build-up stage of the storm at a number of sites throughout the study region; however, few operated throughout the storm. Most failed prior to the peak. Consequently, while there is little measured data that captures both the temporal variation of water level prior to, during, and after the peak conditions and the maximum condition. In a few cases, photographs and other visual images were utilized to provide information about the temporal variation of water level.

An extensive post-storm effort was undertaken to identify and survey high water marks following passage of the storm. While certain high water marks capture the peak water levels well, they contain no information about the temporal variation of water level. High water marks also have their own inherent issues of quality, uncertainty whether they in fact do reflect a peak condition, and whether or not water surface motions due to short wind waves or other factors are reflected in a high water mark.

Water level measurements are able to provide temporal variation and maxima information at only a subset of the locations of interest. Many of the high water marks are of questionable quality. Storm surge modeling was used to complement quality water level measurements were they existed and provide water level information in the many locations were measured data were not available or were of questionable quality. Hydrograph data and the highest quality high water marks also are used to evaluate the accuracy of the storm surge model. As is the case for the wave modeling, model-to-measurement comparisons provide valuable information for quantifying the uncertainty in model predictions.

Hydrograph and High Water Mark Analysis

High Water Marks. The passage of hurricanes results in short-period wind waves on top of the much longer-period storm surge that creates significant entrainment of various types of debris including vegetation, seeds, dirt, man-made trash, and dislodged building material. Depending on local conditions, the entrained debris will deposit on or adhere to some surfaces once the peak stage has been reached and the stages begin to fall. The deposited debris leaves what is referred to as a high water mark (HWM) and the mark is used to quantify the magnitude of peak storm surge. The highest quality marks for estimating storm surge are those that have little or no wave effect (i.e., no influence of wave crests or wave run-up). Some HWMs are collected where significant wave effects are present but that effect is noted.

The HWM data were collected during September through November, 2005. Various organizations participated in the collection of the data including the U.S. Army Corps of Engineers' (USACE) U.S. Army Engineer Research and Development Center (ERDC), the U.S. Army Engineer District, New Orleans (CEMVN), Louisiana State University (LSU), the U.S. Geological Survey (USGS), Levee Districts in the New Orleans area, and the Federal Emergency Management Agency (FEMA).

The HWM and hydrograph data presented are mostly referenced to the latest epoch of NAVD88, 2004.65. Most of the data have been converted to this datum, but a few have not. The datum issue is a significant one. In the vicinity of the Inner Harbor Navigation Channel (IHNC) westward to the vicinity of the 17th Street Canal, the benchmarks complying with 2004.65 result in elevations that are about 0.5 to 0.6 ft lower than elevations derived from benchmarks based on the previous NAVD 88 epoch.

The high water mark data presented herein are rated as excellent, good, and fair/poor, depending on the degree to which the mark is a reliable indicator of the water level, absent wave crest effects or wave run-up effects. Marks rated excellent were those acquired in the interior of buildings, where short wave effects were considered to be absent or minimal. Good marks were typically associated with exterior marks that were consistent with excellent marks measured nearby, or where by the nature of the physical setting for the mark, little to no influence of wind wave action was expected. Excellent water marks were primarily used to characterize local water level maxima, unless no excellent marks were available in an area of interest. In that case marks rated as good were used. Use of fair or poor marks to estimate maximum water level was avoided if at all possible. Both excellent and good marks were used in the comparison with ADCIRC model results.

Along the south Lake Pontchartrain shoreline, at the entrance to the 17th Street Canal, thirteen high water marks rated as "excellent" marks were averaged, and the resultant high water was computed to be 10.8 ft NAVD 88 (2004.65). At the entrance to Orleans Avenue Canal, a single high water mark was available, which was not of high quality. Its value was 10.8 ft NAVD88 (2004.65), the same as the value at the entrance to 17th Street Canal, and similar to the value from London Avenue Canal, so it is considered to be a reliable mark. There were two marks rated as "good" collected at the entrance to London Avenue Canal, and a number of other marks rated to be of lesser quality. The average of the two "good" marks was 10.7 ft NAVD88 (2004.65). Several other marks in the area showed elevations similar to this elevation, so the average of the two marks was considered to be reliable. At the entrance to the Inner Harbor Navigation Canal (IHNC) there were five marks rated "excellent", three to the west side of the entrance and two to the east side at Lakefront Airport. The average of all the five excellent marks was 11.7 ft NAVD88 (2004.65).

Measured high water marks varied considerably within the following series of canals/channels in Orleans and St. Bernard Parishes: the north-south running IHNC that extends from its Lake Pontchartrain entrance to the lock connecting the IHNC to the Mississippi River (the IHNC Lock), and the east-west running canal which serves as the combined Gulf Intracoastal Waterway (GIWW) and Mississippi River Gulf Outlet (MRGO).

At the Lake Pontchartrain entrance to the IHNC, the peak water level was 11.7 ft NAVD88 (2004.65). To the south of the entrance, in the IHNC, an excellent mark north of the Danzinger Bridge indicated 12.4 ft NAVD88 (2004.65), and an average of two excellent marks immediately adjacent to the Bridge on its north side indicated a peak water level of 12.7 ft NAVD88 (2004.65). Further to the south, just to the south of the confluence of IHNC with GIWW/MRGO, two excellent high water marks indicated 15.2 ft NAVD88 (2004.65). At the end of the IHNC, at the IHNC Lock, the maximum from a gage record was 14.3 ft NAVD88 (2004.65) and there were two excellent high water marks nearby that averaged 13.8 ft NAVD88 (2004.65).

In the 6-mile long GIWW/MRGO channel, an excellent high water mark indicated 16.3 ft NAVD88 at the Paris Road Bridge (not yet referenced to the recent NAVD88 epoch). At the point where the GIWW and MRGO diverge (adjacent to Lake Borgne), at Bayou Beinvenue flood control structure, an excellent mark indicated peak water level of 18.4 ft NAVD88. At the influence of the GIWW/MRGO with the IHNC the peak water level was 15.2 ft NAVD88 (2004.65). The gradient in peak water level (increasing level from Lake Pontchartrain to Lake Borgne) reflects the hydraulic connectivity between Lake Borgne and Lake Pontchartrain via these channels.

The Wave and Storm Surge Analysis Technical Appendix contains much more information concerning the high water marks, including a series of images that show locations where the marks were left. Placement of the marks on images was useful for understanding the setting in which the mark was left and potential for short wave influence. Figure V-26 is an example of a photograph with placed HWMs, for the entrance to the 17th St Canal. The high water mark data are also available in a series of spreadsheets that contain pertinent information regarding each HWM as well as a quality assessment made by the IPET hydrograph and high water mark analysis team. The spreadsheets also indicate the datum for each mark. High water mark data are available for the Louisiana and Mississippi coasts.

Analysis and presentation of high water marks presented in this section focuses on those marks that reflect water level conditions along the outer periphery of the hurricane protection system, for use in analyses of the regional water level conditions. There are many high water marks in the interior areas that were flooded. These data are included in the spreadsheets.

Additional information is also provided later in this chapter that compares estimates of water level maxima to the maximum water level conditions considered in the design of the hurricane protection projects throughout the study region. The results presented there reflect our present best estimates of water level maxima using HWMs where excellent marks exist, maxima from measured hydrographs, or maxima determined form model results in the many locations where no measured data are available.

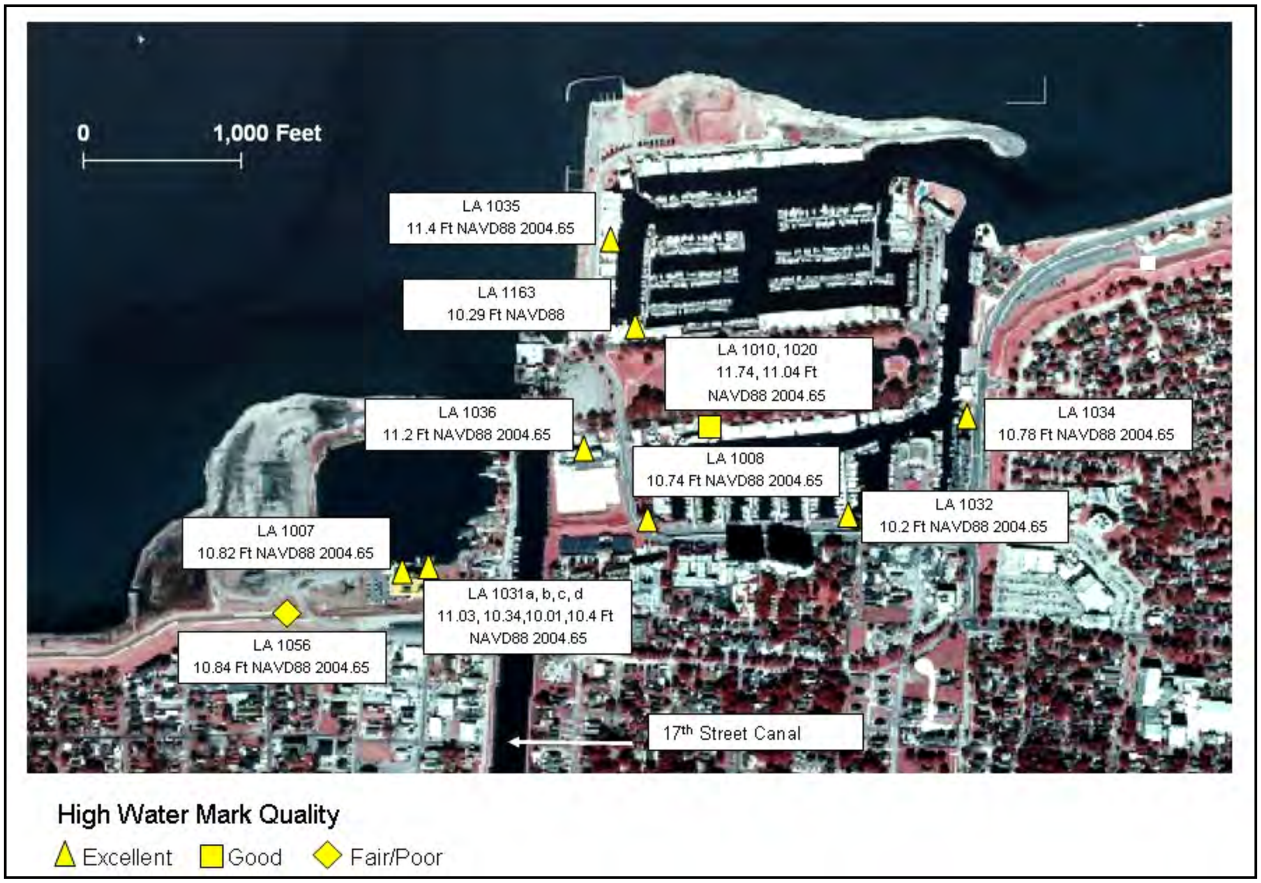


Figure V-26. High water marks on the west side of the entrance to the 17th Street Canal

Hydrographs. The hydrograph data come from various sources including gage data, staff readings, and surveys of water level position relative to physically identifiable objects that were captured in time tagged digital pictures. Data from the following sources are reflected in this report:

a. USGS gages.

b. USACE gages (acquired by the New Orleans District, CEMVN).

c. NOAA NWS gages

d. Levee District gages.

e. Staff gage at the IHNC Lock.

f. Digital photographs taken by an individual at the Municipal Yacht Club.

g. Digital photographs taken by various individuals at the Lakefront Airport.

Gage data acquired in the IHNC are shown in Figure V-27 from the USGS gage at Interstate 10 (I-10), the Orleans Levee District gage at I-10, and the staff gage at IHNC Lock which was read by CEMVN lock personnel throughout the storm. The staff gage was set by lock personnel just prior to the storm without being surveyed to an established datum. Subsequent to the storm, the 15.0-ft

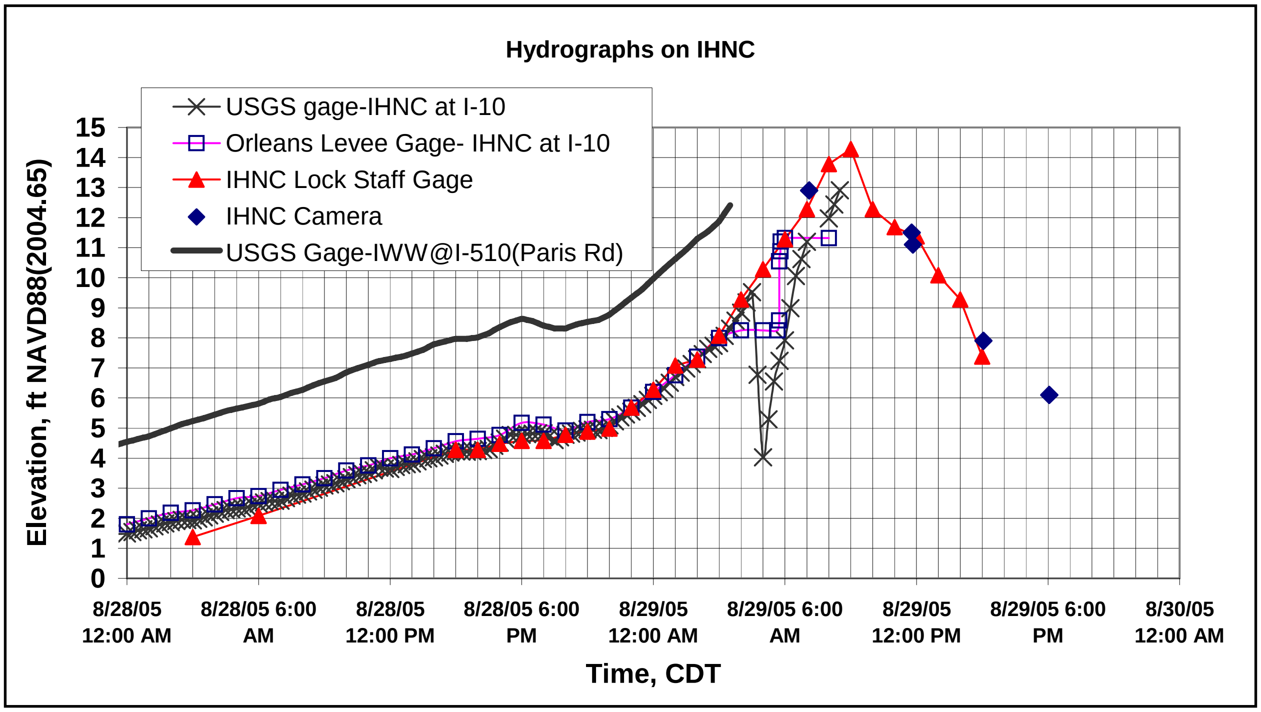


Figure V-27. Hydrographs on the Inner Harbor Navigation Canal (IHNC)

mark on the staff gage was surveyed and found to be at an elevation of 14.28 ft NAVD88 (2004.65). The staff gage readings in the IHNC log were corrected by the 0.72 ft difference and are plotted in the figure. The USGS and the Orleans Levee District gages are located near the railroad floodwall opening just south of I-10. The gate through the floodwall was damaged and sand bags were used to close the opening prior to the storm, based on conversation with a representative of the Orleans Levee District. Based on data from the USGS gage, the sand bags and/or one or both of the breaches on the west side of the IHNC appeared to have failed at approximately 0430 CDT (0930 UTC) on Monday, 29 August. The Orleans Levee District gage, while not showing the large drop, also shows a significant change in water level. The Paris Road gage on the Gulf Intracoastal Waterway/Mississippi River Gulf Outlet (GIWW/MRGO) is about 6 miles east of the intersection of the GIWW/MRGO and the IHNC. The USGS gage at Paris Road requires a datum adjustment to NAVD88 (2004.65), that has not been made yet.

Gages along Lake Pontchartrain were separated into those located west of 90 degrees longitude and those located east of 90 degrees. Figure V-28 shows measured data from gages east of 90 degrees longitude and include USGS Bayou Rigolets near Slidell, USGS Rigolets at Highway 90 near Slidell, and USGS Little Irish Bayou at Highway 11 near Slidell. Only gages providing data throughout a significant portion of the storm are plotted. Since the three curves are similar and Little Irish Bayou survived more of the storm, Little Irish Bayou gage is being surveyed to reference those data to the NAVD88 2004.65 datum.

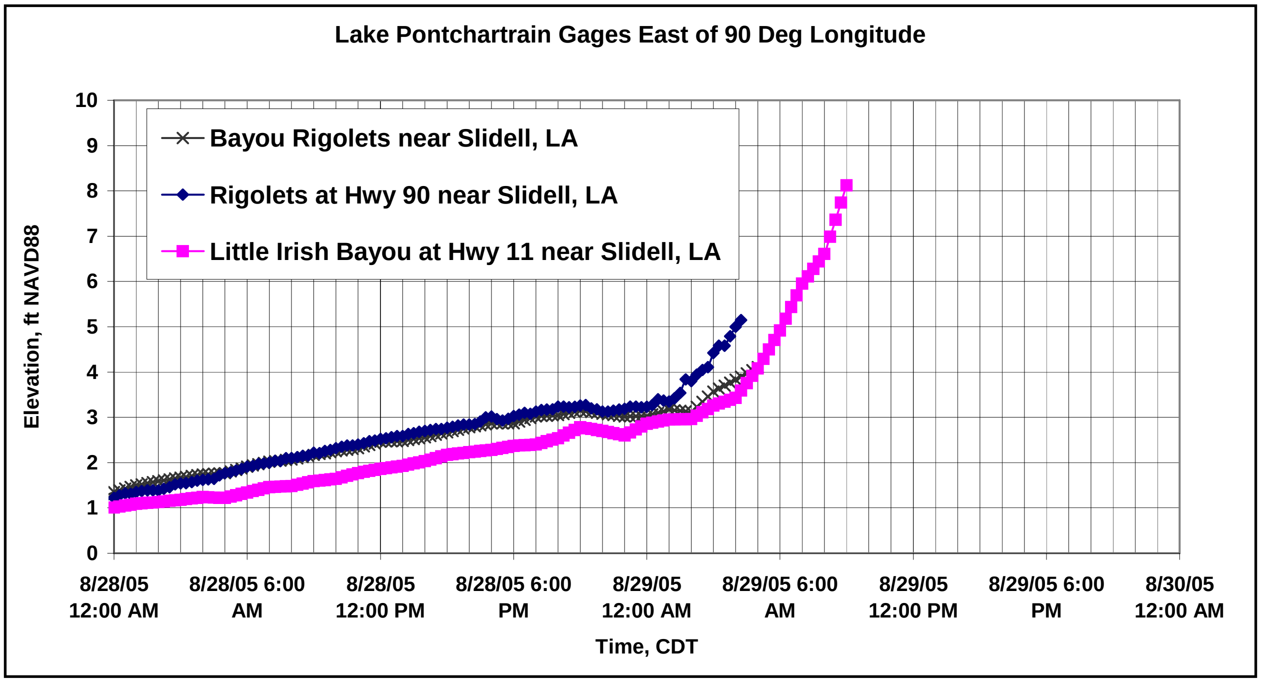


Figure V-28. Hydrographs on Lake Pontchartrain, gages located east of 90 degrees longitude

Figure V-29 shows measured data for gages west of 90 degrees longitude and include the National Weather Service (NWS) gage in Lake Pontchartrain on the Causeway designated Midlake, USGS gage Pass Manchac at Turtle Cove near Pontchatoula, and Orleans Levee District gage at Southshore Marina. After the passage of Katrina, the USGS installed a temporary gage about ¼ mile north of the NWS gage at Midlake on the Causeway. That gage became operational at 4:00 PM on September 2. All four gages are being surveyed to establish the data records relative to NAVD88 2004.65 datum.

Figures V-30 and V-31 are hydrographs acquired by NOAA National Ocean Service (NOS) at stations 8760922 at Southwest Pass, Louisiana and station 8761724 at Grand Isle/East Point, Louisiana. The instruments at these stations are among the few that functioned throughout Katrina's passage and recorded peak water levels. The Grand Isle station recorded a peak water level of 5.70 ft above mean lower-low water (MLLW) at 09:06 UTC on 29 August 2005. The Southwest Pass station recorded a peak water level of 7.61 ft above MLLW at 09:30 UTC on 29 August 2005.

One individual stayed at the Municipal Yacht Harbor (MYH) on his boat, the 53-ft *Manana*, during Katrina. The MYH is located immediately east of the entrance to the 17th Street Canal on Lake Pontchartrain (the largest, northern-most harbor shown in Figure V-26). He moored his 53-ft boat, a trawler-type steel-hull vessel that was built in 1946 and last retrofitted in 1995, with multiple 2-in diameter hawsers. The digital photographs taken on 29 August by that individual were tagged with time that was believed to be one hour behind Central Daylight Time. The LSU personnel examined the camera and confirmed that the camera file times were one hour behind CDT.

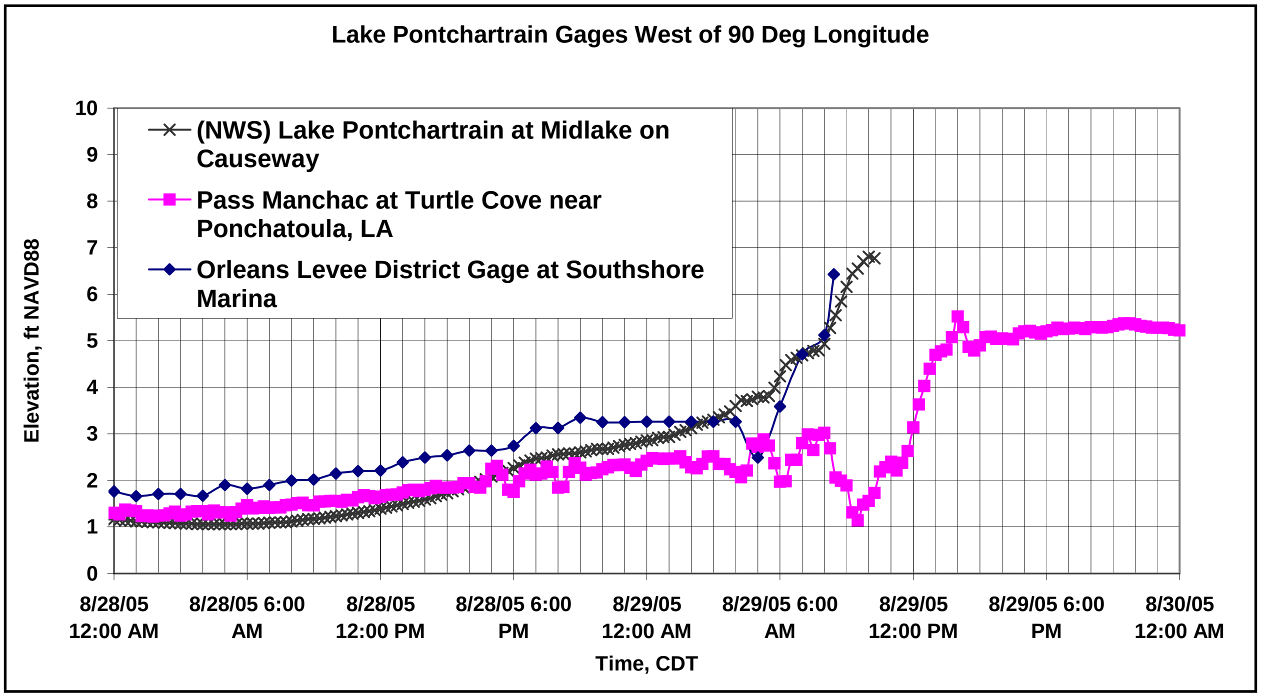


Figure V-29. Hydrographs on Lake Pontchartrain, gages located west of 90 degrees longitude

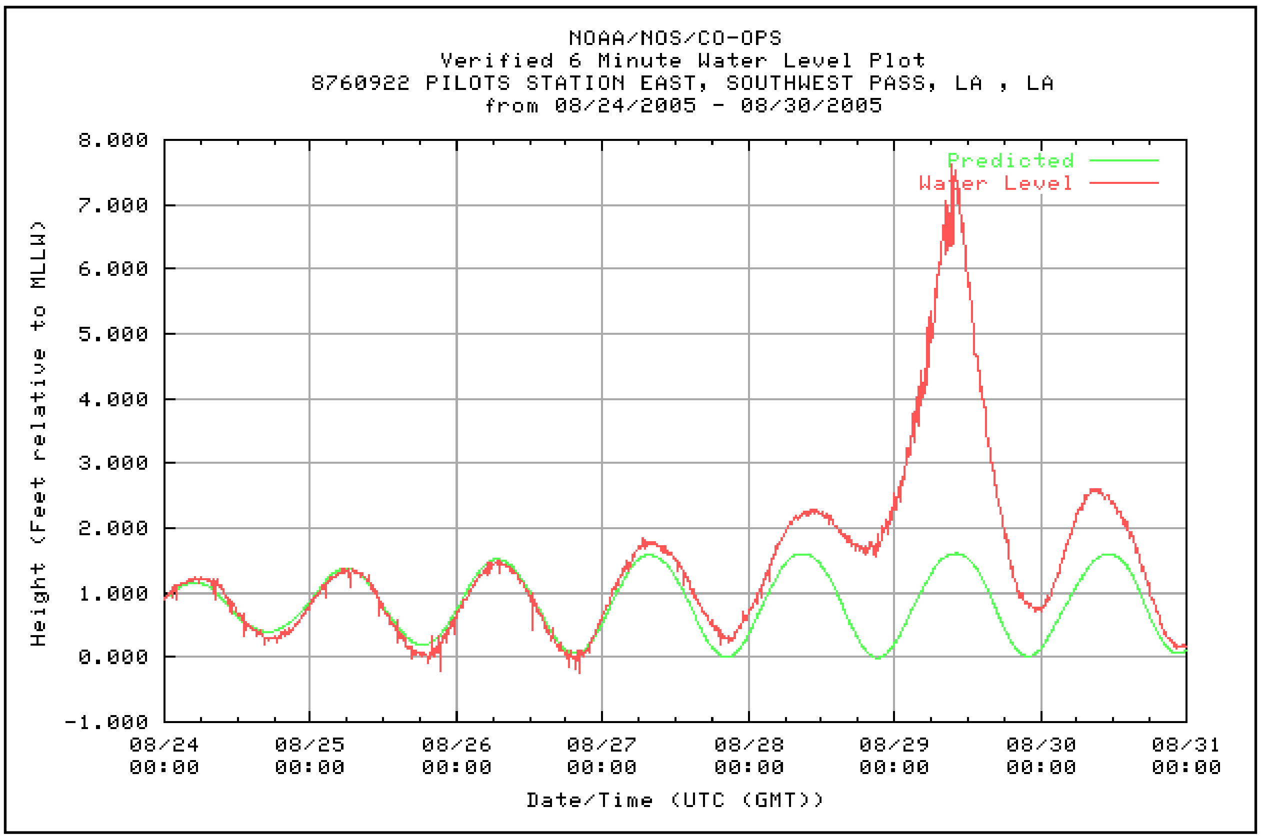


Figure V-30. Hydrograph for NOAA National Ocean Service station at Southwest Pass, Louisiana

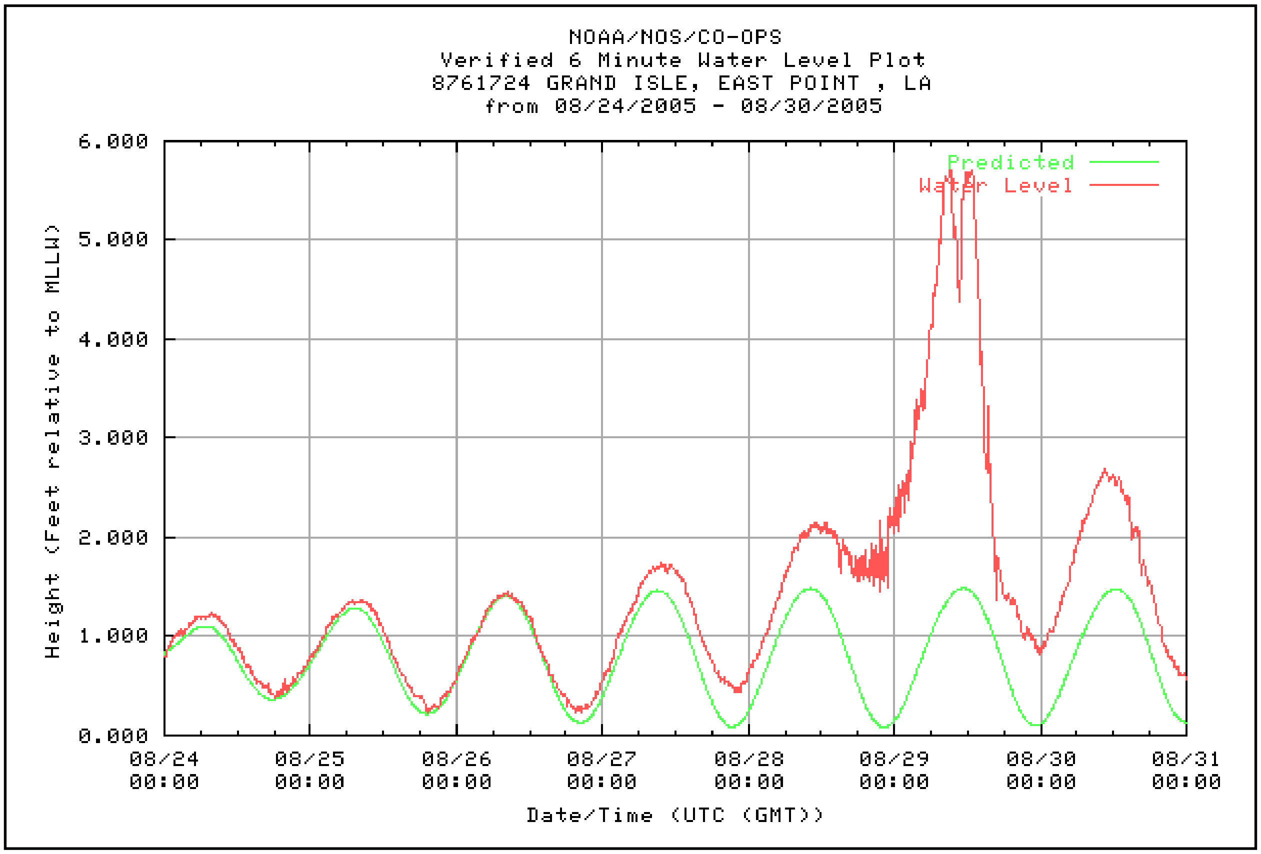


Figure V-31. Hydrograph for NOAA National Ocean Service station at Grand Isle, Louisiana.

A log of visual observations was maintained by another individual on 29 August who remained on his boat at the Orleans Marina near the 17th Street Canal entrance and just south of the Municipal Yacht Harbor. That individual was interviewed and points recorded in his log were surveyed.

The various time-tagged data points in the MYH and the Orleans Marina were surveyed and are plotted in Figure V-32, with their respective times. The survey was conducted using 2004.65 benchmarks. Also shown is the average high water mark elevation computed from high water marks acquired in the vicinity of the entrance to the 17th Street Canal, 10.8 ft NAVD88 2004.65 datum. All marks used are considered to be excellent high water marks, i.e., acquired within the interiors of buildings. The timing of the high water mark is somewhere between 9:00 and 10:00 CDT (1400 and 1500 UTC). A time of 9:30 CDT (1430 UTC) is used on the plot to indicate the time of peak water level until a better estimate is determined.

Digital photographs were taken by members of the Orleans Parish Levee District at the Lakefront Airport on 29 August and the water level location in each of the photographs was surveyed. These data are plotted in Figure V-33. Also shown is an average high water mark elevation, of 11.7 ft NAVD 88 2004.65, computed from five excellent high water marks acquired at the Lake Pontchartrain entrance to the IHNC (both east and west sides).

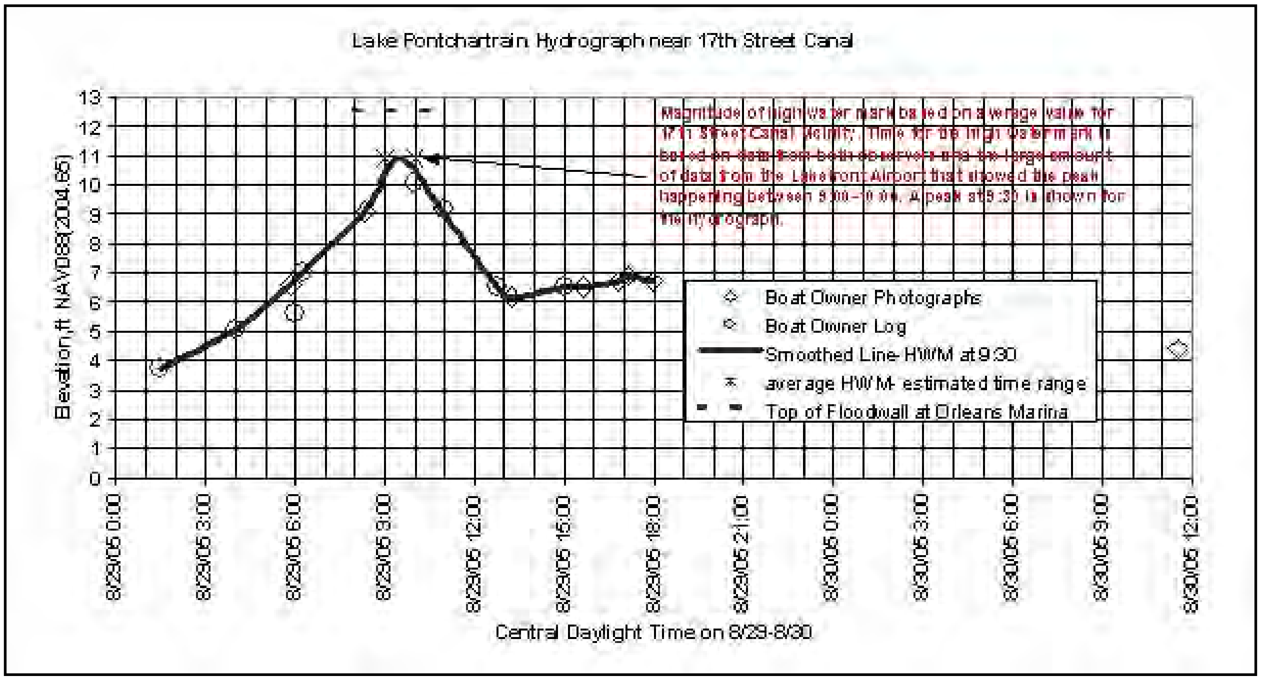


Figure V-32. Reconstructed hydrograph at the entrance to the 17th Street Canal

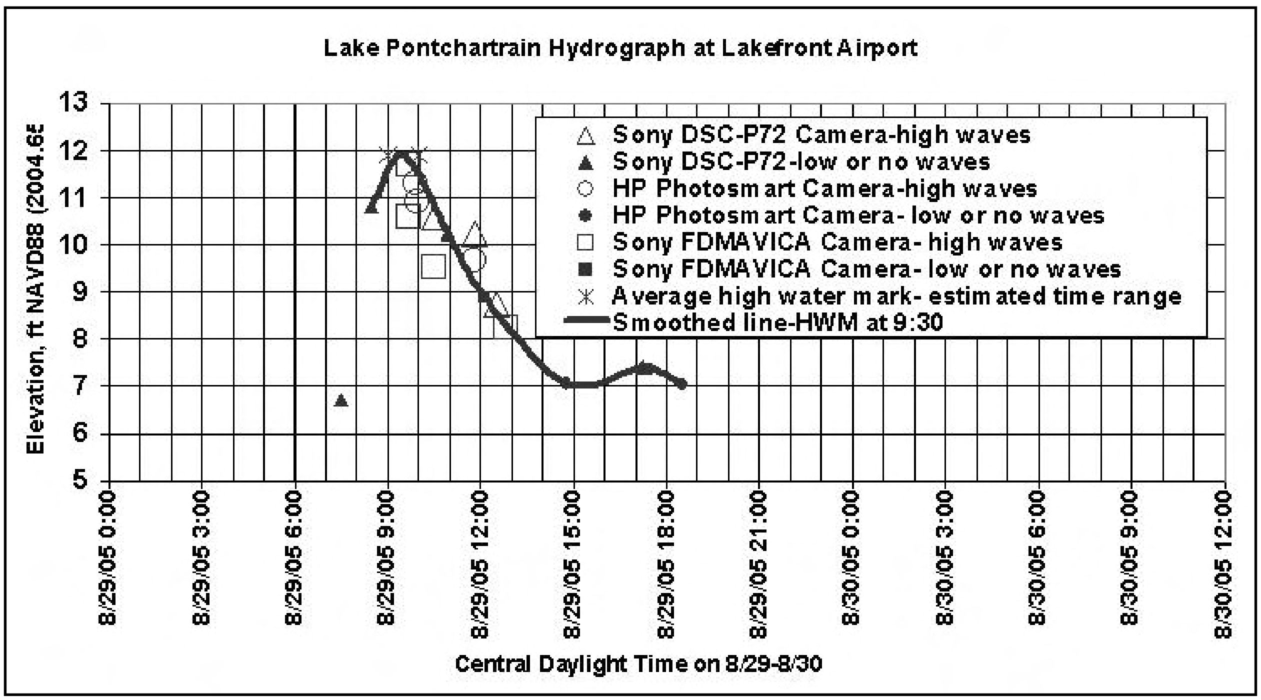


Figure V-33. Reconstructed hydrograph at the Lakefront Airport, entrance to the IHNC

Storm Surge Modeling

In this brief summary, we describe application of the ADCIRC hydrodynamic model to hindcast the storm surge development and propagation during Hurricane Katrina. Over the past decade, extensive storm surge model development, application, and validation efforts have been made in Southern Louisiana. This work has improved storm surge modeling capabilities within a physics-based framework that correctly accounts for and simulates the forcing and response processes (Westerink et al. 2005, Feyen et al. 2005). These efforts have taken advantage of the evolution of unstructured grid computational algorithms as well as massively parallel software and hardware.

TF01 Computational Model. The model domain/grid used in our Katrina simulation is based on an extension of the S08 model (Westerink et al. 2005, Feyen et al. 2005). The S08 model incorporates the western North Atlantic Ocean, the Gulf of Mexico and Caribbean Sea to allow for full dynamic coupling between oceans, continental shelves, and the coastal floodplain without necessitating that these complicated couplings be defined in the boundary conditions (Blain et al. 1994).

The S08 domain/grid has been extensively applied and validated in a number of hindcast studies. These hindcasts included air-sea interaction and forcing as well as tides. Wave-current interaction was not taken into account.

For the Katrina hindcast, the S08 model/domain was extended by adding resolution along the north shore of Lake Pontchartrain as well as the inlets and coastal floodplain (up to the 60-ft contour) along the Mississippi and Alabama coasts. The resulting TF01 model, shown in Figures V-34 and V-35, allows for a better representation of the flooding event as Katrina made its second landfall.

The bathymetric/topographic elevation data were interpolated to the computational mesh by moving progressively from the coarsest to finest areas of the domain. Deep water bathymetric depths were first interpolated from a $5^{o} \times 5^{o}$ regular grid based on the ETOPO5 values. Subsequently values were obtained from the NOAA NOS depth sounding database and USACE CEMVN and USGS topographic survey values using an element-based gathering/averaging procedure instead of a direct interpolation procedure. The gathering/averaging procedure searches for all available sounding/topographic survey values within the cluster of elements connected to one specific node, averages these values and assigns the average value as the depth/topographic elevation to that node. This gathering/averaging procedure essentially implements grid scale filtering to the bathymetric/topographic data and ensures that bathymetry/topography is consistent with the scale of the grid. Bathymetry/topography was hand-checked; in regions with missing or incorrect data, supplemental data from the CEMVN, USGS or NOS bathymetric/ topographic charts was applied.

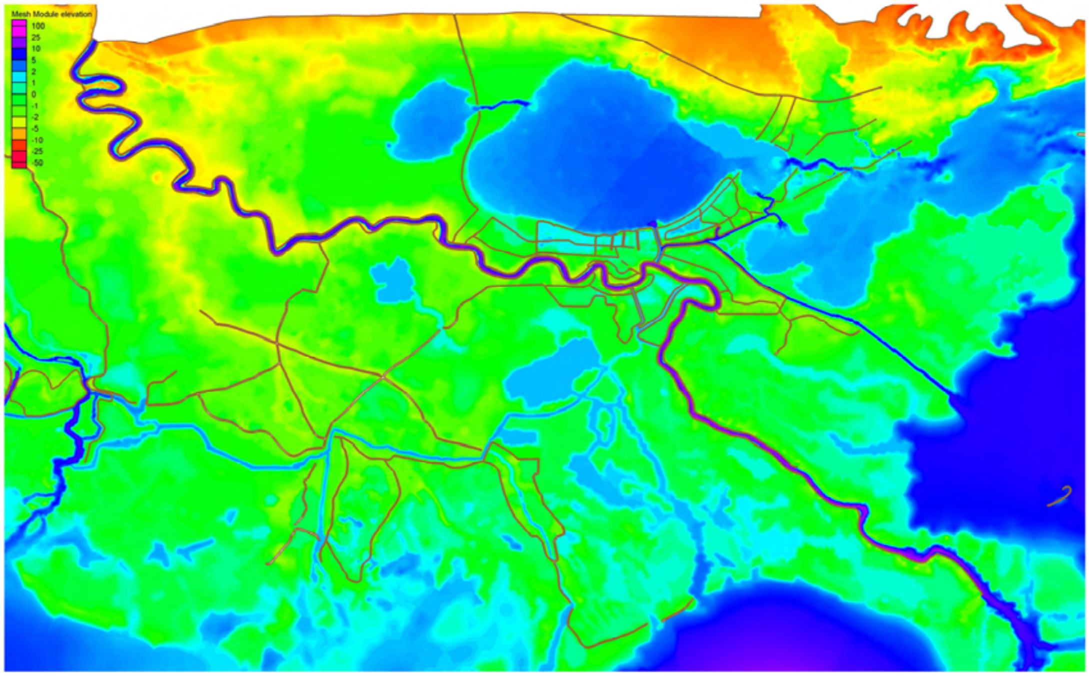


Figure V-34. Bathymetry/topography used in the ADCIRC storm surge model (TF01 grid)

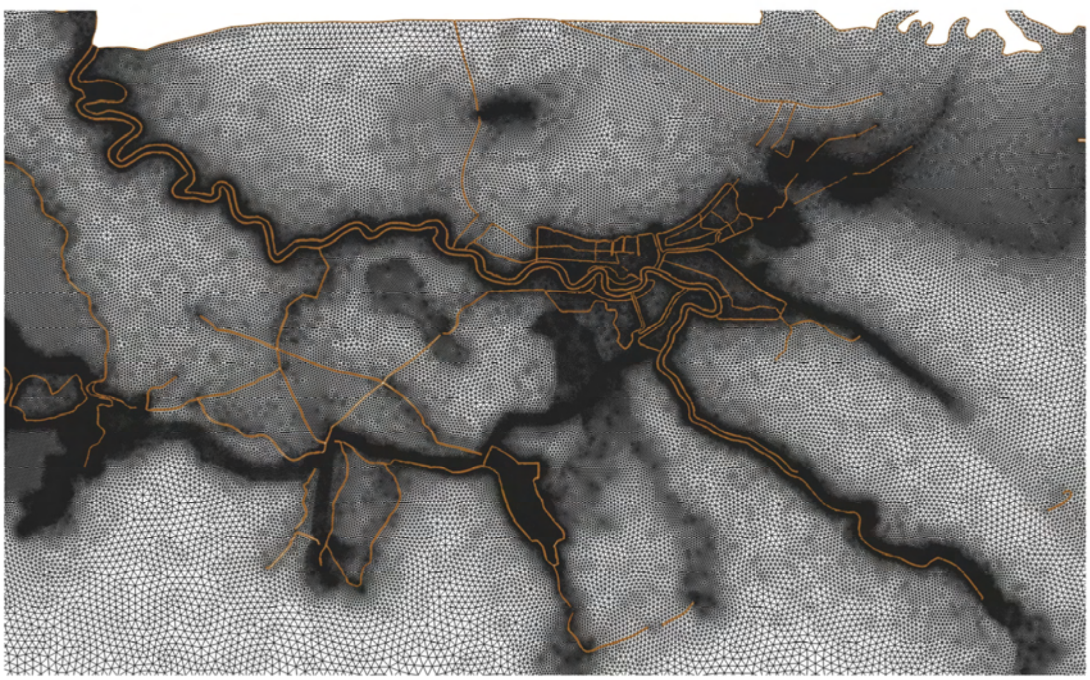


Figure V-35. Grid resolution used in the ADCIRC storm surge model (TF01 grid)

In order to provide a continuous elevation field from offshore on to land and reference these depths to the model's datum, it is necessary to account for the original datums for the bathymetric and topographic data. First, the desired datum for the ADCIRC model is mean sea level offshore. Second, bathymetric data are provided relative to the tidal mean lower low water datum. Examination of NOAA benchmarks in the Southern Louisiana region shows that on average mean sea level is approximately 0.6 ft above mean lower low water datum (MLLW). The topographic data were provided relative to NGVD 29. When the grid was constructed, in light of datum uncertainties, 0.0 ft NGVD 29 was assumed to be approximately 0.0 MLLW, i.e. NGVD 29 was assumed to be about 0.6 ft below mean sea level. The initial water height is raised 0.6 ft so that the modeled mean sea level matches on average the mean sea level relative to bathymetry and topography (the currently used ADCIRC model datum is NGVD29). Note that these adjustments are provided across the domain and will correct the original data to mean sea level on an average regional basis. Recent information acquired during the IPET study suggests that in the New Orleans vicinity NAVD88 2004.65 is about 0.2 to 0.25 ft below local mean sea level (LMSL for the 1983 to 2001 tidal epoch). Therefore, to convert ADICRC water level results to NAVD88 2004.65, 0.4 ft are subtracted from the model results. Recent information from the IPET datum work suggests that NGVD 29 (1991) is 0.88 ft below local mean sea level for the 1983 to 2001 tidal epoch, along the south shore of Lake Pontchartrain. Bathymetric and topographic data used to construct the ADCIRC model are being re-examined and converted to NAVD88 2004.65.

Storm Forcing and Other Details. Astronomical tides are forced in the simulation reflected in this report; wind waves are not. Work to couple the wave and storm surge models is ongoing. The Mississippi and Atchafalaya rivers are forced with steady flows of 22000 ft^3/s and 67000 ft^3/s respectively.

Steric effects due to the thermal expansion of surface ocean water during late summer are pronounced in the Gulf of Mexico. This expansion is approximately captured by the long term solar annual and semiannual (*Sa* and *Ssa*) harmonic constituents. Examination of the harmonic constants computed by NOAA for stations across Southern Louisiana shows that the amplitude of the *Sa* and *Ssa* constituents is on average just over 0.61 ft. It is assumed that the hurricanes generally take place during the times when the expansion is at its largest in the late summer. Therefore, the initial water surface was raised an additional amount, a steric adjustment of 0.61 ft.

Marine wind and atmospheric pressure fields were generated using the 5 level version of the Planetary Boundary Layer (PBL) model (Thompson and Cardone 1996). The model was run with 1.5-hourly input minimum atmospheric pressure in the storm eye, maximum wind speed and eye location interpolated from available preliminary NOAA two- to three-hourly values. The input for the PBL simulation is given in the Wave and Storm Surge Analysis Technical Appendix. The PBL model output consists of 30-minute averaged wind and pressure fields available every 15 minutes (necessary to avoid substantial aliasing in ADCIRC's Eulerian wind and pressure field interpolation algorithm). Since the air-sea drag laws have been developed assuming 10-minute averaged winds, a conversion to 10-minute averaged winds was implemented by multiplying the PBL 30-minute winds by a gust factor of 1.04.

Viscous hydrodynamic parameters are specified globally constant for bottom friction and lateral viscosity using standard physically relevant values as applied in S08 simulations. We emphasize that no tuning or optimization was performed with respect to the selected values and that with the exception of the domain/grid, all model parameters were defined as in previous hindcasts.

Description of Hurricane Katrina Storm Surge. It is noted that the center of the storm tracked largely east of the city of New Orleans (about 28 miles due east at its closest point). However the storm was in the vicinity of critical features in the vicinity of New Orleans, the storm center being as close as 10 miles due east of the St. Bernard Parish/Chalmette hurricane protection levee which runs along the Mississippi River – Gulf Outlet (MRGO) and as close as 20 miles due east of the confluence of the Gulf Intracoastal Waterway (GIWW) and the MRGO. The influence of the MRGO on storm surge that reaches the metropolitan New Orleans area has been the subject of considerable debate. That issue is addressed later in this chapter and in greater detail in the Wave and Storm Surge Analysis Technical Appendix.

Prior to landfall, the counterclockwise rotating winds of Hurricane Katrina began to push water from east to west. This pattern existed several days prior to landfall. This water began to first inundate the wetlands with several feet of water and then pile up water against the east- and northeast-facing levee systems throughout the southeast Louisiana region. As the storm made landfall in southern Louisiana and continued in a north-northeast direction, the buildup in surge along the levee systems increased until the storm center passed, and then the surge began to decrease. The greatest buildup of water occurred about halfway down that portion of the MS River and “back” levee system in Plaquemines Parish, which is located southeast of New Orleans. A slightly smaller buildup in storm surge occurred in Lake Borgne as water piled up against the eastern-facing hurricane protection levees along St. Bernard and Orleans Parishes.

In addition to the local buildup of water against the levees, these local surges propagate away from their region of initial generation. The surge generated against the river and back levees of Plaquemines Parish propagated up the Mississippi River as well as across Breton and Chandeleur Sounds. The latter surge interacts with the wind fields and propagates to the north-northeast paralleling the path of the storm center as it advanced. As the storm pushed this surge to the north-northeast, piling the water up against the Mississippi Gulf coast and combining with more locally generated surge, water levels reached their highest values along the Mississippi coast to the east of the location at second landfall. This local maximum storm surge region to the right of the storm track is typical of land-falling hurricanes.

Figure V-36 shows color-shaded contours of the maximum water level computed for the storm at each grid node, in feet NGVD29, for the entire Louisiana and Mississippi coastal region computed with the ADCIRC model. Figure V-37 shows contours for the metropolitan New Orleans vicinity. Peak water levels in

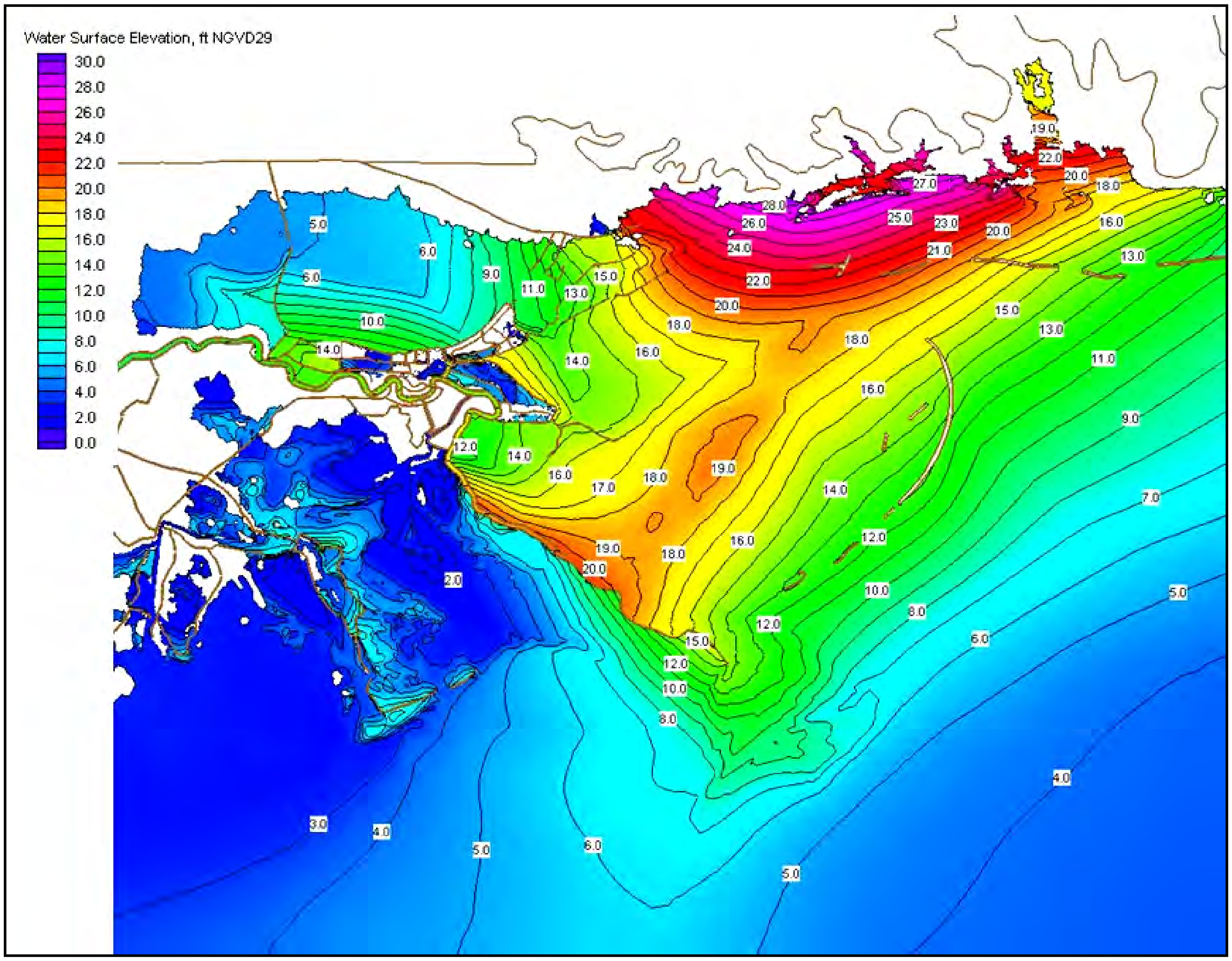


Figure V-36. Maximum computed storm surge using the ADCIRC model, Mississippi to Louisiana region (water levels in feet, NGVD 29)

southeastern Louisiana were computed to be about 20 to 21 ft (dark orange contours), NGVD29, along the east-facing Mississippi River and back levees that protect communities along the river. At the levees facing Lake Borgne along the MRGO, maximum computed water levels where 17 to 18 ft (light orange contours). Along the south shore of Lake Pontchartrain, maximum levels were computed to be between 9 and 13 ft (green contours). Along the coast of Mississippi, maximum water levels were computed to be 27 to 28 ft (pink contours).

Note the pattern of water level gradient within the GIWW/MRGO and the IHNC. The pattern is similar to that reflected in the high water marks.

Figures V-38 through V-40 show computed time series of water surface elevation, in feet NGVD29, at twelve locations throughout the metropolitan New Orleans area. Figure V-38 shows locations along the south shore of Lake Pontchartrain. The computed time of arrival of the peak surge is about 13:59 UTC on August 29, 2005 (or about 9:00 a.m. local time, CDT). The simulated time of arrival for the peak surge is slightly ahead of the observed time of

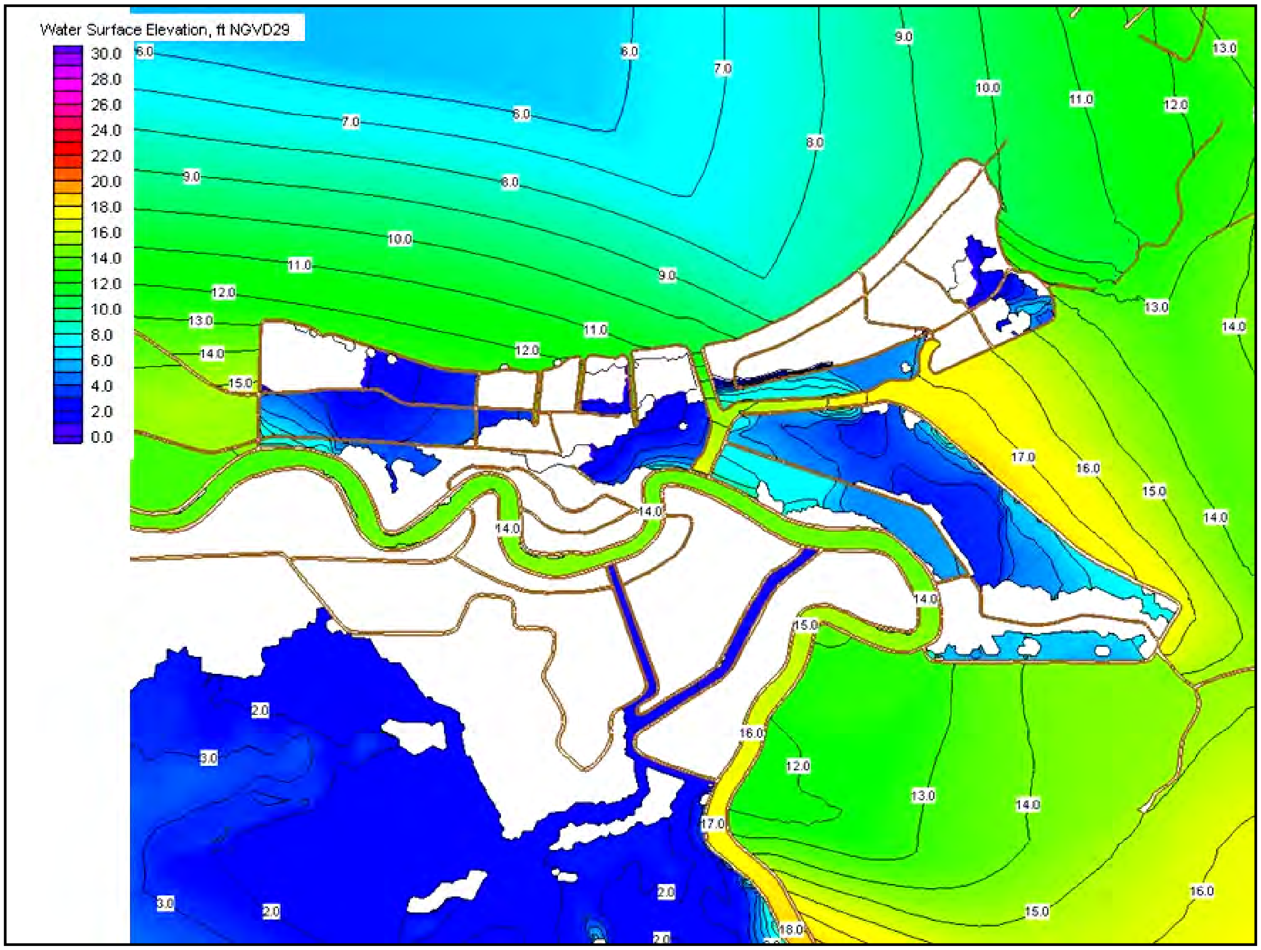


Figure V-37. Maximum computed storm surge using the ADCIRC model, metropolitan New Orleans vicinity (water levels in feet, NGVD 29)

arrival, which is estimated to have occurred sometime between 9:00 a.m. and 10:00 a.m. CDT. Figure V-39 shows the same information for locations along the MRGO and GIWW/MRGO. Model results indicate that the peak of the storm surge wave took approximately 50 min to propagate from the southeastern corner of the levee along the MRGO in St. Bernard Parish to the junction of the IHNC and MRGO, as the storm tracked to the north-northeast. The computed time of arrival of the peak surge at the IHNC Lock is about 13:35 UTC (8:35 a.m. CDT). The observed hydrograph at the Lock shows arrival of the peak surge at about 9:00 a.m. CDT, or slightly later. However, the timing of the peak at the IHNC Lock may be influenced by the breach on the IHNC into the Lower 9th Ward.

Hydrograph data at the IHNC Lock and the reconstructed hydrographs at the entrance to 17th Street Canal and at Lakefront Airport suggest that the time of peak surge arrival predicted by the storm surge model is about 30 min early. This may change once the final winds are incorporated into the modeling.

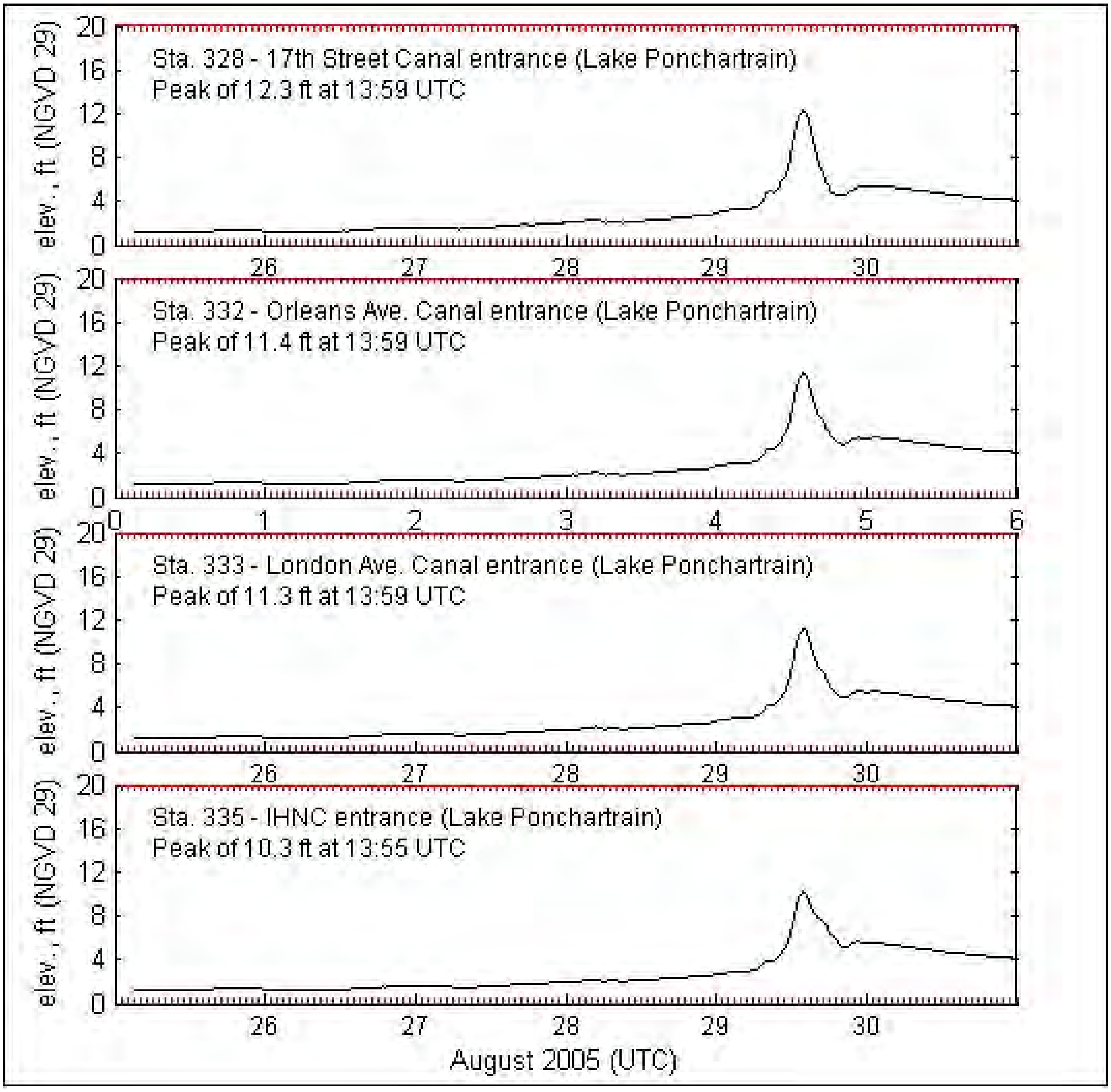


Figure V-38. Change in water surface elevation, with time, for locations along the south shore of Lake Pontchartrain

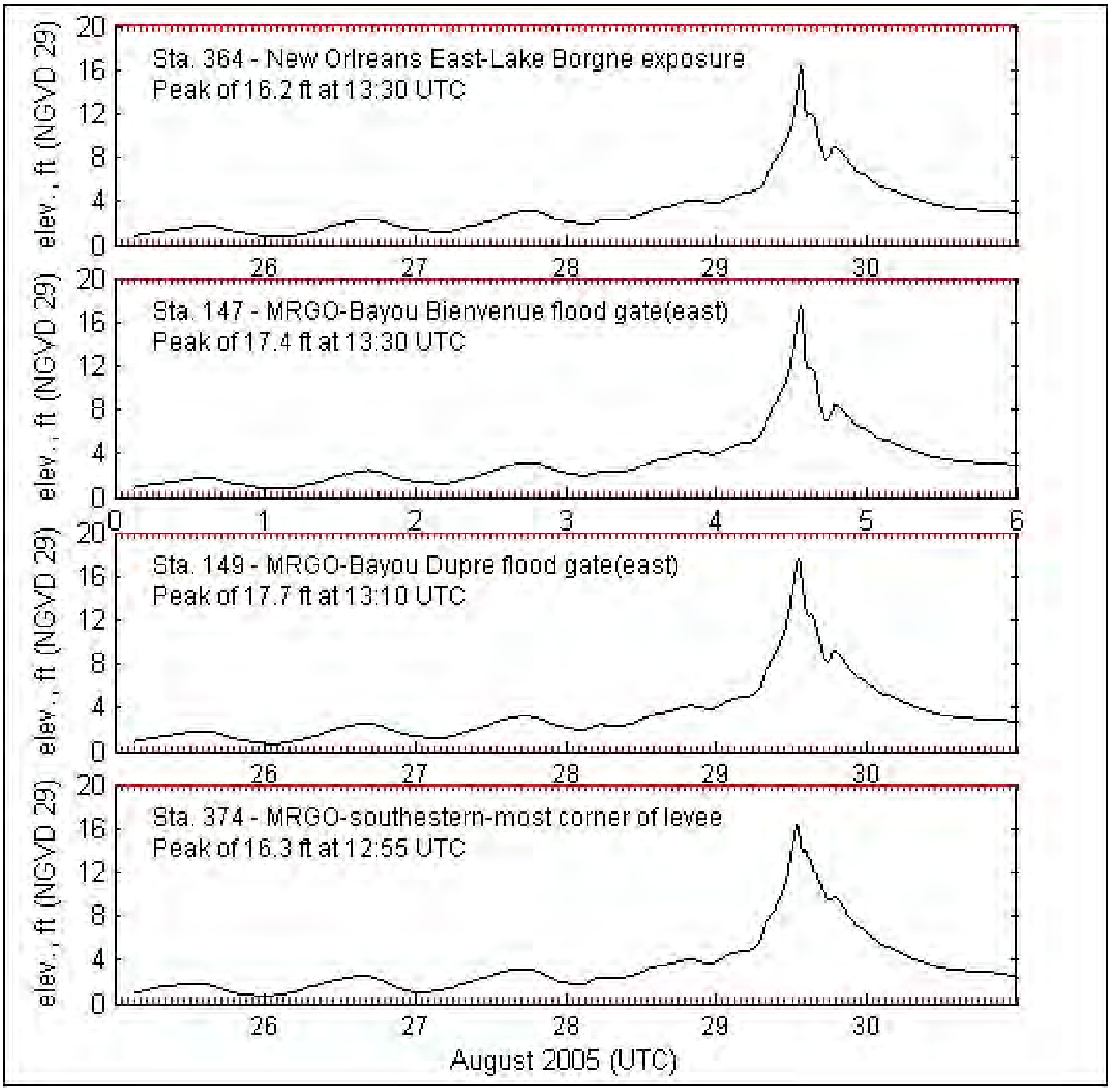


Figure V-39. Change in water surface elevation, with time, for locations in the GIWW and MRGO with exposure to Lake Borgne

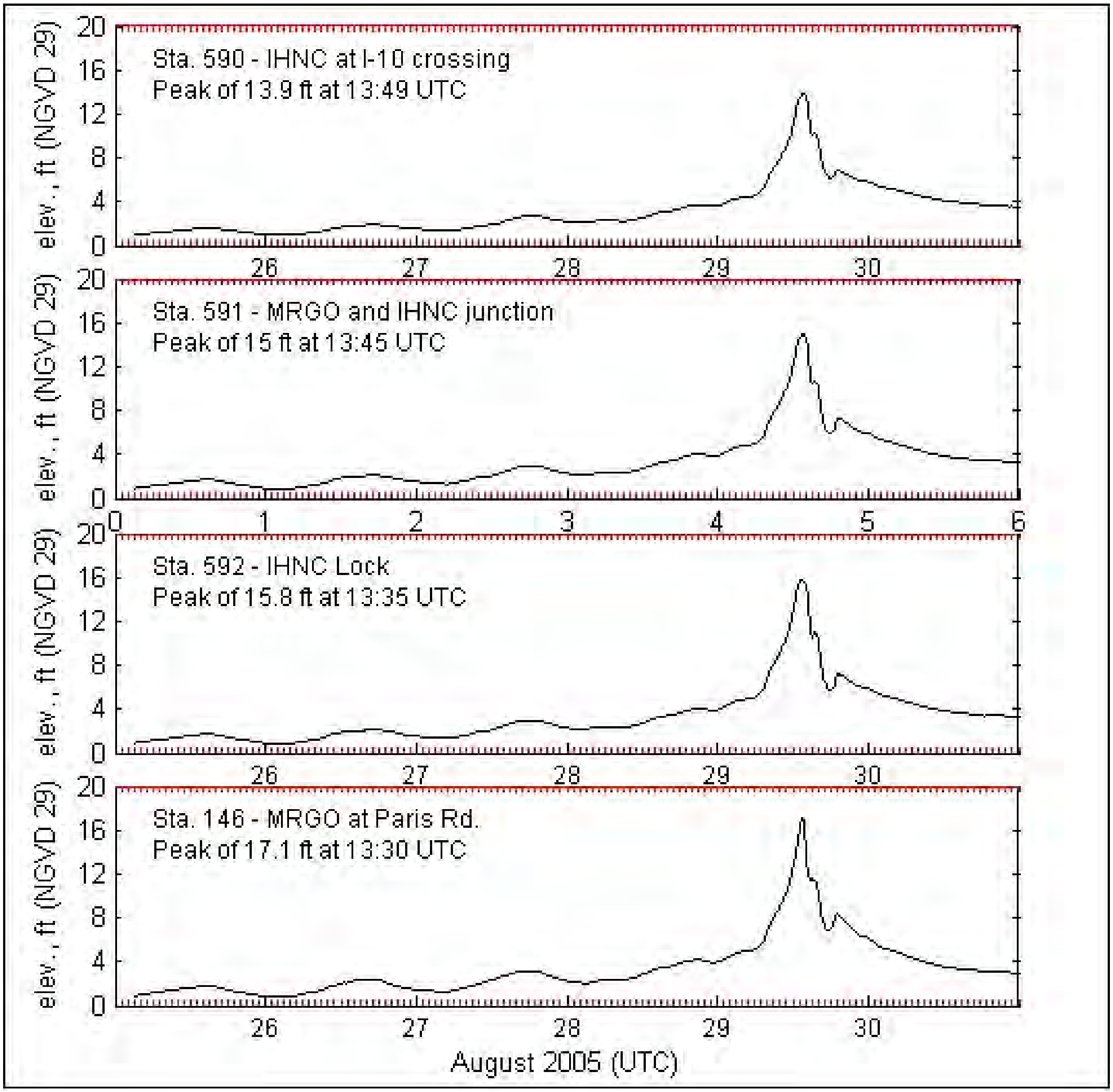


Figure V-40. Change in water surface elevation, with time, for locations along the GIWW/MRGO and IHNC

Model-generated maximum water levels are compared to measured high water marks and to design water levels in the following section. More documentation describing the storm surge modeling, a more detailed description of the storm surge propagation through the region, and additional model-to-measurement comparisons are provided in the Wave and Storm Surge Analysis Technical Appendix.

Comparison of Katrina Wave and Water Level Maxima with Design Values

Peak wave and water level conditions experienced during Hurricane Katrina are compared to values used in the design of the hurricane protection system. In the series of figures that follow, design values are shown in yellow boxes with the label “D”; computed, model-derived values are shown in blue with the label “C”; and where measurements are available, measured values are shown in green boxes with the label “M”. Design values were taken from the original Design Memoranda, which generally cited significant wave height and period. The Design Memoranda do not specify whether a peak or a mean period was used. At the time the projects were designed, this distinction between different measures of wave period was probably not made. Computed wave maxima were estimated using STWAVE model results (significant wave height and peak wave period) and computed water level maxima were estimated using ADCIRC results (maximum water surface elevation).

Peak measured wave conditions were only available at the entrance to the 17th Street Canal; however, the measurements are of questionable accuracy at the peak of the storm. The maximum measured wave height and period values that are used are those measured just prior to the point at which the data appear to become suspect, from both wave buoys. For water level conditions, at sites where hydrographs captured the peak water level, that value is presented. Where high water marks rated “excellent” are available, those values are shown. If no excellent marks are available in an area of interest, then marks rated “good” or the best available quality of mark, were used.

All water levels are converted to a common datum NAVD88 (2004.65) for the purposes of this comparison using datum conversions based on current IPET datum analysis results. To convert from the ADCIRC model datum to NAVD88 2004.65, 0.4 ft are subtracted from model results. The Design Memoranda cite design water levels relative to a number of difference reference frames, mean sea level, MSL, National Geodetic Vertical Datum (without reference to any specific epoch), and to still water level, SWL. The earliest design documents cited SWL and MSL; later design documents cited NGVD. It appears that the intent of the designers has always been to relate design water levels to mean sea level, and this intent has been confirmed with CEMVN staff so that assumption is used. To convert design water levels to NAVD88 (2004.65) datum, 0.25 ft are added to the design water level values along the south shore of Lake Pontchartrain (correction derived from the New Canal datum analysis), and 0.2 ft are added to values in the vicinity of IHNC/GIWW/MRGO canals (an average of corrections derived from the New Canal, 0.25 ft, and Chef Menteur, 0.15 ft, datum analyses). For southern Plaquemines Parish, the same 0.2 ft correction was applied until more definitive information becomes available.

Wave Maxima. Figure V-41 shows wave maxima for the south shore of Lake Pontchartrain in Jefferson and Orleans Parishes. Significant wave heights measured and computed for Katrina exceeded design wave heights by 0.9 to 1.6 ft. Peak wave periods during Katrina were about equal to the design values.

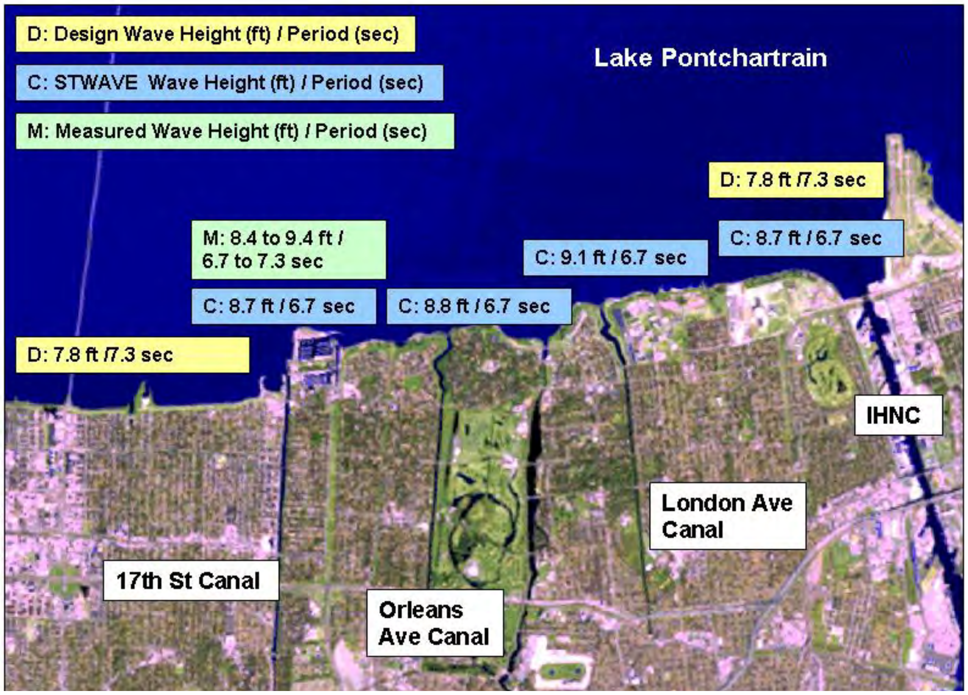


Figure V-41. Wave maxima along the south shore of Lake Pontchartrain hurricane protection system

Figure V-42 shows wave maxima for the eastern portion of Orleans Parish. On Lake Pontchartrain, significant wave heights computed for Katrina exceeded design wave heights by 1 ft; peak wave periods were 0.6 sec less that the design values. On the east-facing side of the Parish, significant wave heights computed for Katrina exceeded the design value by 0.8 ft; and wave period exceed the design value by 1.3 sec. On the back levee of Orleans Parish, along the GIWW, with exposure to Lake Borgne, maximum significant wave height computed for Katrina only exceeded the design value by 0.3 ft, but the peak wave period exceeds the design value by about a factor of 3. The design wave periods are more typical of those for wind seas. Wave model simulations show that during Katrina, the eastern-facing levee systems were subjected to longer-period energy propagating from the Gulf past the barrier islands. Re-examination of the design wave conditions along the eastern-facing levees at this location is recommended, in light of the large differences between design periods and the wave periods generated by Hurricane Katrina. Both wave heights and wave periods define the potential for wave run-up and overtopping.

Figure V-42. Wave maxima along eastern Orleans Parish hurricane protection system

Figure V-43 shows wave maxima for the easternmost portion of St. Bernard Parish. Along the MRGO, significant wave heights computed for Katrina were less than the design wave heights by 1.7 to 1.8 ft. However, peak wave periods computed for Katrina were nearly two to three times greater than the design values. On the south-facing portion of the hurricane protection levee, significant wave heights computed for Katrina were less than design values by about 2.2 to 2.3 ft; wave periods exceed design values by a factor of about three. Design wave conditions at these locations should be re-examined as well. Lower wave heights will reduce run-up; higher wave periods will increase wave run-up.

Figure V-44 shows wave maxima for areas of Plaquemines Parish. Along the levees east of the Mississippi River with exposure to waves approaching from the east, significant wave heights computed for Katrina exceeded design wave heights by amounts ranging from 2 to 4 ft. Peak wave periods computed for Katrina were much greater than the design periods, two to three times greater. On the west-facing levees on the west side of the Mississippi River, in some locations, significant wave heights computed for Katrina exceeded the design values and in some locations computed wave heights were less than design values. In all cases the computed wave periods exceeded the design wave periods. Design wave conditions should be re-examined along the west-facing levees.

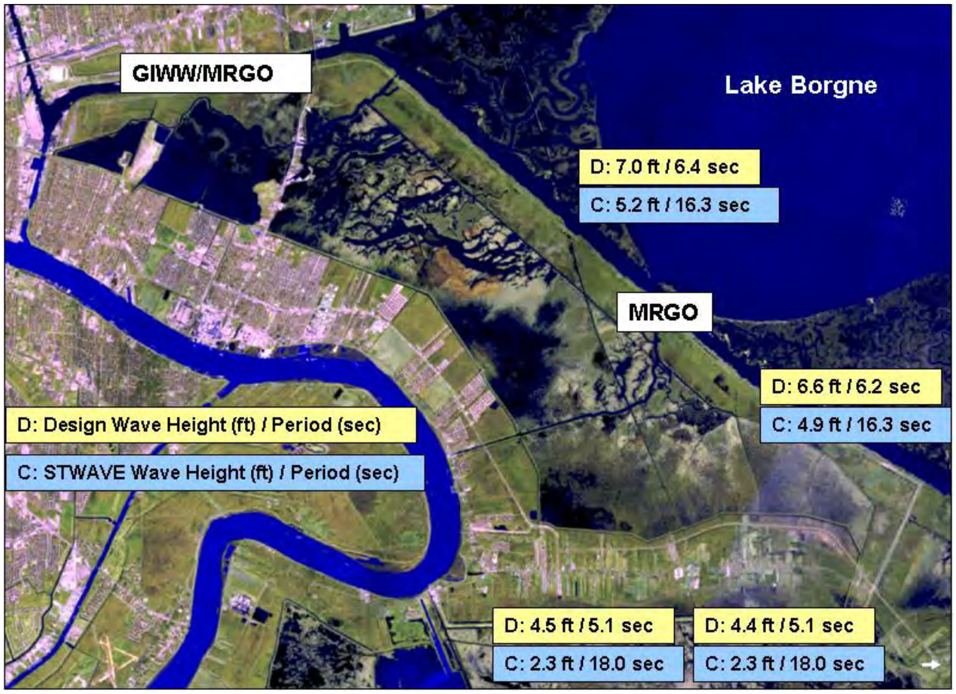


Figure V-43. Wave maxima along hurricane protection system of St. Bernard Parish

Figure V-44. Wave maxima along hurricane protection levees in Plaquemines Parish

Water Level Maxima. Figure V-45 shows water level maxima for the south shore of Lake Pontchartrain in Jefferson and Orleans Parishes. Peak water levels during Katrina at the entrances to 17th Street (10.8 ft NAVD 88, 2004.65), Orleans Avenue (10.8 ft NAVD 88, 2004.65) and London Avenue Canals (10.7 ft NAVD88 2004.65), were about 1 ft less than design values. The peak values were all based on high water marks. The design water level is 11.8 ft NAVD88 (2004.65) throughout the region.

Figure V-46 shows water level maxima for eastern Orleans Parish. On the Lake Pontchartrain side, the design water level is 11.8 ft NAVD88 2004.65 and the measured peak water level at the entrance to the IHNC was 11.7 ft, NAVD88 2004.65. At this location the peak water levels were right at the design levels. On the back levee, adjacent to the GIWW, with exposure to Lake Borgne, and along the GIWW/MRGO, design water levels range from 13.0 to 13.2 ft NAVD88 2004.65. High water mark data suggest that the design water levels were exceeded along these canals, by amounts ranging from 1 to approximately 5 feet. Within the IHNC, north of its junction with the GIWW/MRGO, design water levels range from 11.8 to 13.1 ft NAVD88 2004.65. High water marks suggest that design water levels in this section of channel were right at design levels or slightly below.

Figure V-47 shows water level maxima for eastern St. Bernard Parish. As stated above, the design water levels along the GIWW/MRGO were exceeded by amounts ranging from 1 to 5 feet. In the IHNC, south of its junction with the GIWW/MRGO, design water levels are 13.2 ft NAVD88 2004.65 and an excellent high water mark indicated a peak water level of 15.2 ft. The hydrograph from the IHNC Lock indicates the peak reached 14.3 ft NAVD 88 2004.65. Within the IHNC, south of its junction with the GIWW/MRGO, peak water levels during Katrina exceeded deign values by 1 to 2 feet. Along the MRGO, the design water level varies from 13.2 ft to 12.7 ft NAVD88 2004.65. High water marks indicate that design water levels were exceeded along the MRGO hurricane protection levee by amounts ranging from 3 to 5.5 ft.

Figure V-48 shows water level maxima for southern Plaquemines Parish. For the east-facing levees and flood walls, design water levels ranged from 12.8 to 14.2 ft NAVD88 2004.65. Not all high water mark data have been processed for this region; but based on model results, peak water levels during Katrina exceeded the design values south of Phoenix by as much as 6 ft. At the southernmost end, near Venice, computed Katrina peak water levels were right at design levels. On the levees facing west on the west side of the Mississippi River, again based solely on model results, peak water levels during Katrina exceeded the design values in some areas, by amounts up to approximately 1 ft; but in other areas, the peak values were less than the design values by about the same amount.

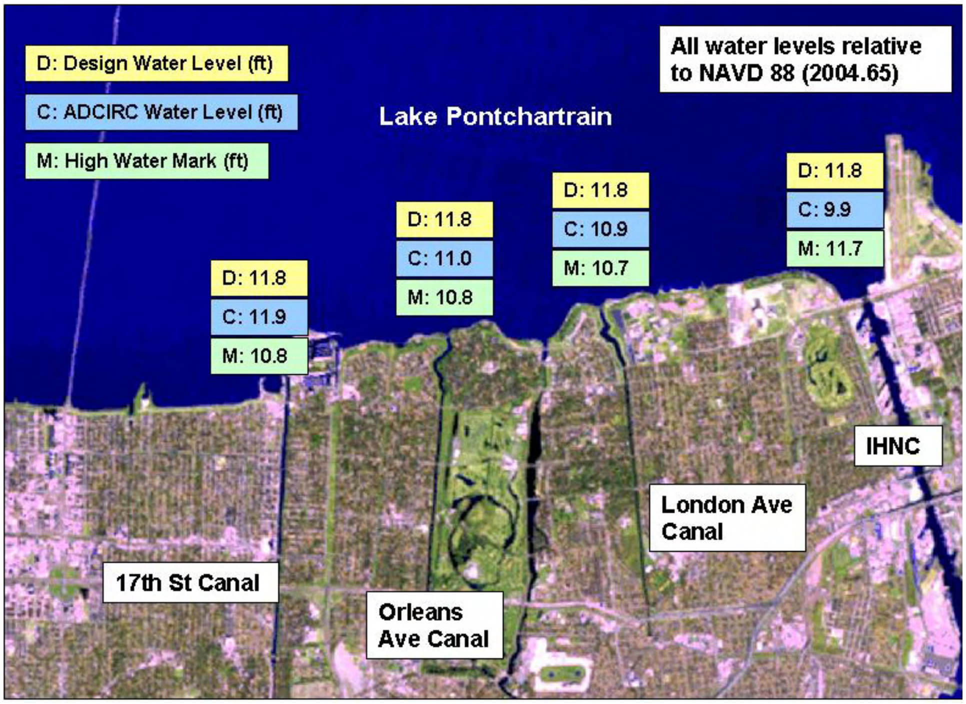


Figure V-45. Water level maxima along the south shore of Lake Pontchartrain hurricane protection system

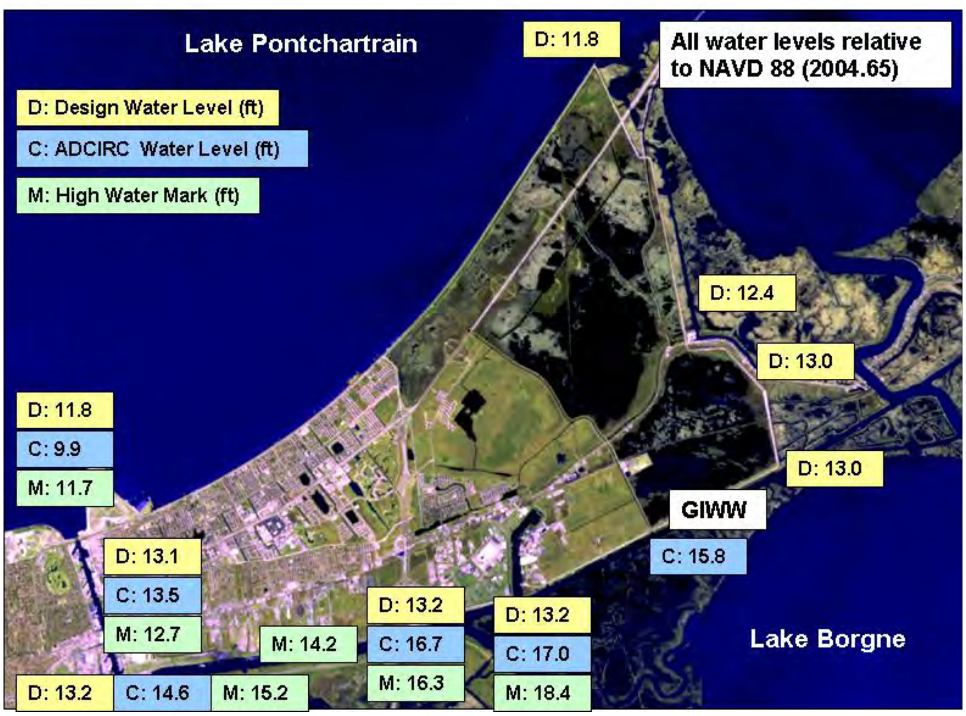


Figure V-46. Water level maxima for eastern Orleans Parish hurricane protection system

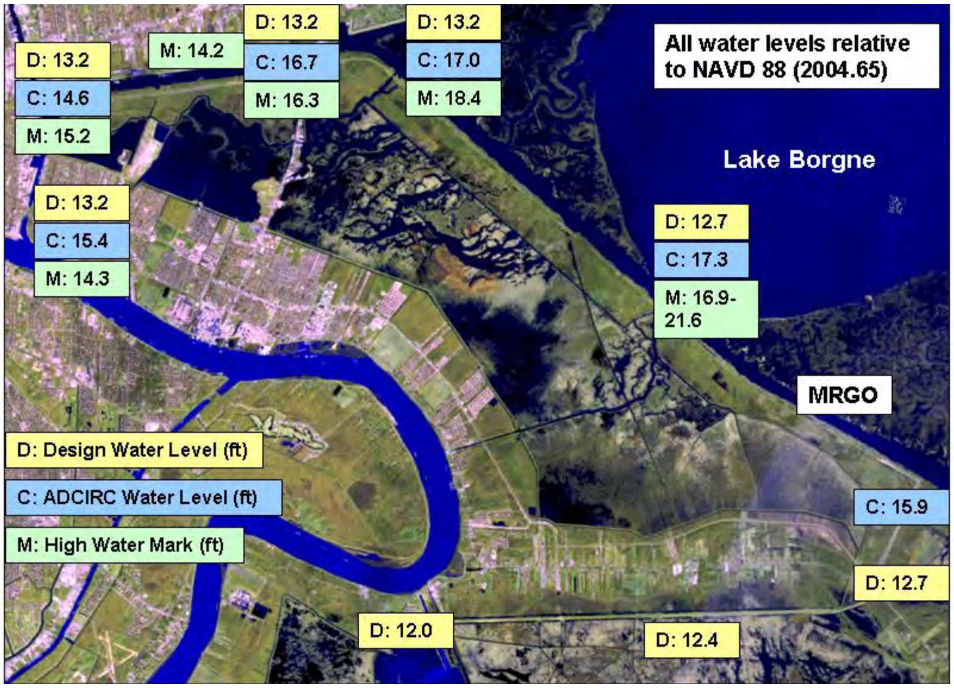


Figure V-47. Water level maxima for eastern St. Bernard Parish hurricane protection system

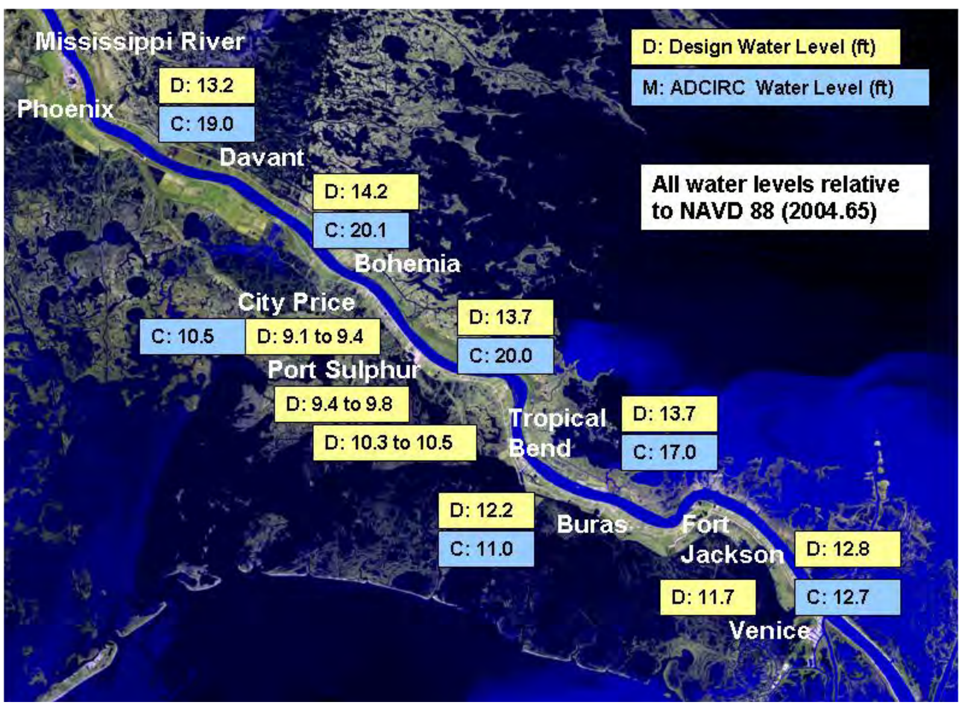


Figure V-48. Water level maxima for Plaquemines Parish hurricane protection system

Influence of the MRGO on Storm Surge in the New Orleans Vicinity

The Mississippi River Gulf Outlet (MRGO) role in propagation of low amplitude astronomical tide and influx of higher saline water into Lake Pontchartrain has been established. Concerns have been raised regarding the role of the MRGO on storm surge propagation into the metropolitan New Orleans vicinity.

From the perspective of long wave propagation, of which the tide and storm surge are examples, the critical section of the MRGO is Reach 1, the section of waterway where the GIWW and MRGO occupy the same channel (see Figure V-49). It is through this channel that Lake Pontchartrain and Lake Borgne are hydraulically connected to one another via the IHNC. The two Lakes are also connected to each other via the Rigolets and Chef Menteur Pass; the IHNC is the smallest of the three connections. Reach 1 existed as the GIWW prior to the construction of the MRGO, although the maintained depth was lower. As a result of this hydraulic connection, the storm surge experienced within the IHNC and Reach 1 (GIWW/MRGO) is a function of storm surge in both Lakes; a water level gradient is established within the IHNC and Reach 1 that is dictated by the surge levels in both Lakes. This is true for both low and high storm surge conditions.

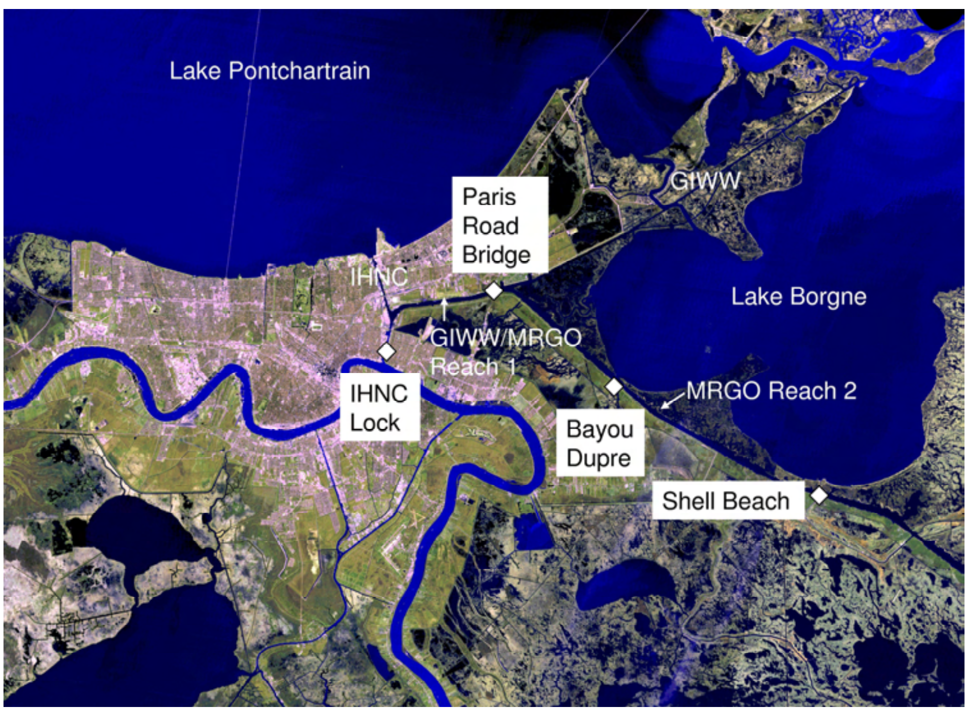


Figure V-49. Location of the MRGO (Reaches 1 and 2)

To prevent storm surge in Lake Borgne from influencing water levels experienced in the IHNC or GIWW/MRGO sections of waterway, flow through the Reach 1 channel must be dramatically reduced or eliminated, either by a permanent closure or some type of structure that temporarily serves to eliminate this hydraulic connectivity. The presence of an open channel is the key factor. If the hydraulic connectivity between Lake Pontchartrain and Lake Borgne is eliminated at a point within Reach 1, tide or surge to the west of this point will become primarily influenced by conditions at the IHNC entrance to Lake Pontchartrain; and tide or storm surge to the east of this point will become primarily influenced by conditions in Lake Borgne.

Most concern seems to be focused on MRGO/Reach 2 that runs from the GIWW/MRGO confluence, just east of the Paris Road Bridge, to the southeast (see Figure V-49). Three previous studies have been performed to examine the influence of MRGO/Reach 2 on storm surge in New Orleans and vicinity (two initiated by the U.S. Army Corps of Engineers and one commissioned by the Louisiana Department of Natural Resources), in addition to work performed to examine this issue as part of the IPET study. The IPET work to examine the influence of the MRGO/Reach 2 was done with the current version of the ADCIRC model, as reflected in this report. All studies have reached the same conclusion. The change in storm surge induced by MRGO/Reach 2 (computed as a percentage of the peak surge magnitude) is greatest when the amplitude of the storm surge is low, on the order of four feet or less. In these situations, changes induced by the MRGO in the metropolitan New Orleans area are rather small in terms of absolute water surface elevation changes, 0.6 ft or less in all cases and less than 0.3 ft in most cases, but this amount can be as much as 25% of the peak surge amplitude when the amplitude is low. When the long wave amplitude is very low, the surge is more limited to propagation via the channels, and the MRGO has its greatest influence. Once the surge amplitude increases to the point where the wetlands become inundated, this section of the MRGO plays a diminishing role in influencing the amplitude of storm surge that reaches the IHNC. For storm surges of a magnitude produced by Hurricanes Betsy and Katrina which overwhelmed the wetland system, both more than 7 ft peak surge and Katrina near 18 ft in Lake Borgne, the influence of MRGO/Reach 2 on storm surge propagation is quite small, just a few tenths of a foot at most in the IHNC and GIWW/MRGO in terms of absolute water surface elevation changes. These small changes represent only a few percent of the surge amplitude. When the expansive wetland is inundated, the storm surge propagates primarily through the water column over this much larger flooded area, and the channels become a much smaller contributor to water conveyance.

The hurricane protection levees along the south side of Orleans Parish and the eastern side of St. Bernard Parish along the MRGO, which together are referred to as a “funnel”, can locally collect and focus storm surge in this vicinity depending on wind speed and direction. This localized focusing effect can lead to a small local increase in surge amplitude. Strong winds from the east tend to maximize the local funneling effect.

Additional detail concerning the work to examine the influence of the MRGO on storm surge, and a more detailed explanation of why the effect is so small at high storm surge levels, is included in Appendix E titled “Note on the Influence of the Mississippi River Gulf Outlet on Hurricane Induced Storm Surge in New Orleans and Vicinity.”

Status of Remaining Efforts

Remaining work includes incorporating the final wind and pressure fields produced by NOAA Hurricane Research Division and Oceanweather, Inc. into all wave and storm surge modeling. The ADCIRC model set-up will be modified to incorporate recent topographic survey data and recent datum information as well as grid mesh refinements. Coupling between storm surge and wave models will be completed and applied for the storm (WAM, STWAVE and ADCIRC coupling). An STWAVE domain for the Mississippi coast will be set up and applied. Spatially variable wind fields will be integrated into the STWAVE modeling for Lake Pontchartrain and Louisiana South domains (this has been done for Louisiana Southeast). Datum adjustments will be made to high water mark and hydrograph data that have not been corrected yet. Exhaustive model-to-measurement comparisons and model skill assessment will continue. Sensitivity tests will be done for both wave and surge models to examine the role of pre- and post-storm wetland roughness on computed waves and water levels. Sensitivity tests will be done to examine influence of a degraded eastern barrier island chain on wave and storm surge conditions. Other sensitivity runs will be done to examine the role of model parameters and uncertainty in model input on wave and storm surge results. The final report will be prepared and data sets will be prepared for public release.

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High Resolution Hydrodynamics

Summary of Accomplishments

The present report is an extension of Report 1 and does not include discussion of the goals and objectives of this task.

As discussed in Report 1, the task Estimation of Forces on Levees is focused on providing high resolution time histories of water levels, waves and related forces on levees and floodwalls in the New Orleans area, along with an analysis. Report 1 contained descriptions of the types of models and methods that will be used in these analyses and the reasons for their application to this problem. As required, additional supplemental technical information will be presented in Appendices in the present report to build upon the technical content contained in Report 1.

Initial timelines indicated that we would provide information for all of New Orleans canals and the large flood-protection levees in St. Bernard and Plaquemines Parish in this report; however, sufficient bathymetric and topographic information to allow accurate high resolution computations of the type undertaken here was available only for the 17th Street Canal in time for model runs required for this report. In order to avoid undo speculative results, this report will only examine conditions in these latter areas.

It should be noted that delays in the availability of bathymetric and topographic information required for construction of the 17th Street physical model have also delayed that model somewhat; however, it is hoped that an aggressive testing schedule will allow us to still meet our goal of completing initial testing for waves passing through the entrance to the canal and under the flood-proof bridge near the site of the levee/floodwall failure by mid-March.

Analyses of Water Levels

In areas exposed to the open Gulf, massive quantities of water were driven against miles of coastal levees. Since the appropriate levee heights were modeled in the large-scale ADCIRC and STWAVE runs performed within the Surge and Wave Model Group, the effects of levee overtopping are implicitly included in the boundary conditions provided for the high resolution calculations undertaken here. Levee breaching was not represented in the Surge and Wave Model Group’s calculations; however, these effects should be quite small in the St. Bernard and Plaquemines areas.

In contrast to the situation along the open Gulf, water levels within canals can depend strongly on the time of breaching and size of the breaches relative to the canal cross section. As a baseline study, a series of ADCIRC model tests were performed to examine the variation of water surface elevation (WSE) and current speeds within the 17th Street Canal for the case of no breaching. In idealized tests with no wind forcing on water within the canal, the WSE time series throughout varied little (less than 3 cm) from the input forcing hydrograph at the Lake Pontchartrain boundary for simulated conditions during Katrina. This shows that water levels within these canals will tend to be approximately equal to the level at the boundary, plus the effect of wind set-up along the canal. During these tests, steady currents were quite small (less than 0.1 m/sec) with some seiching, possibly due to numerical effects, producing velocities in the range of 0.35 m/sec.

Detailed Time History of Water Levels, Waves, and Related Forces

St. Bernard and Plaquemines Parish. Boussinesq simulations at four specific levee transects along the Mississippi River Gulf Outlet (MRGO) provide time histories of combined wave and surge water levels, overtopping rates, and flow velocities along the back and front sides of the levees. The northernmost transect is a few miles south of the intersection of MRGO and the Intercoastal Waterway, while the southernmost transect is near the Bayou Dupre Control Structure. Simulations cover the time from 0100 to 1100 CDT on August 29th. The largest waves and surge occur at roughly the same time (0700-0800). Maximum surge values were near 18 feet along MRGO, while maximum wave heights were 2-3 ft. The levees at the four transects experience similar conditions. Wave spectra were taken from STWAVE simulations (Surge and Wave Model Group) at locations inside the MRGO, and thus predicted wave heights were relatively low due to dissipative propagation over the marshes of Lake Bourne. At peak wave height, the predicted wave-induced increase in the mean water level (setup) at the levee toe was 1-1.5 ft.

Maximum overtopping rates occur at 0800, with wave-averaged values near 10 ft^3/s per ft of levee length. This corresponds to an average flow depth over the levee crest of approximately 1.5 ft and an average velocity of 6.5 ft/s. On the backface of the levee, the gravity driven downrush velocities occur at maximum overtopping, with wave-averaged values near 10 ft/s, and instantaneous velocities reaching 15 ft/s. Simulations suggest that average backface velocities exceeded 10 ft/s continuously for 1 hour (0730-0830), and 5 ft/s for two hours (0700-0900). From 0630-0900, the simulations predict continuous overtopping. For approximately one hour before and one after this time period, predicted over-topping was intermittent and due to only wave overwash. During these times, the predicted uprush and downrush velocities along the front face of the levees are maximum. These velocities are related to the swash oscillations, with maximum runup velocities near 10 ft/s, and downrush velocities of 5 ft/s. These values are peak values, with time and depth-averaged values of horizontal velocity on the front face very small during periods of non-continuous overtopping. The vertical profile of the time-averaged velocities (undertow) will be investigated further if needed.

17th Street Canal. As noted above information on the timing of breaching and the size of the breach are extremely important to the estimation of water levels within a canal. The following provides an analysis of the nature of this interrelation. As shown, for sufficient breach size, it is possible for water levels at the breach to remain constant or even become lower while water levels at the entrance continue to rise. In this context, observed water levels and eye-witness accounts become a vital part of the methodology for estimating water levels during the storm.

Figure V-50 shows valuable information collected by Data Collection and Management Group**,** along with the time series of ADCIRC water levels at a point near the entrance of the canal. The data cover the period during and immediately after passage of Hurricane Katrina. In Figure V-50, the solid line denotes the “best fit” to observed and photographed water levels throughout Katrina. Open circles, open triangles, and x’s denote the sources of data used in this compilation. The dashed line and black dots show the ADCIRC results; and the red dots show reports of water levels observed by the pump operator at the south end of this canal. Also shown in this figure is an estimate of the water level shortly after 1100 CDT on the same day, obtained from a frame of an amateur video taken from a nearby high-rise building near the shore of Lake Pontchartrain. Figure V-51 shows the video frame. The top of the levee on the west side of the canal, inside the canal, is estimated to be at elevation +3 ft NAVD88 2004.65. The top of the wall is at approximately +12.5 feet NAVD88 2004.65. The estimated water level from this photo is approximately +2 ft, ±2 feet, NAVD88 2004.65. The estimated water level from this photo is approximately +1 ft, ± 2 feet, NAVD88 2004.65.

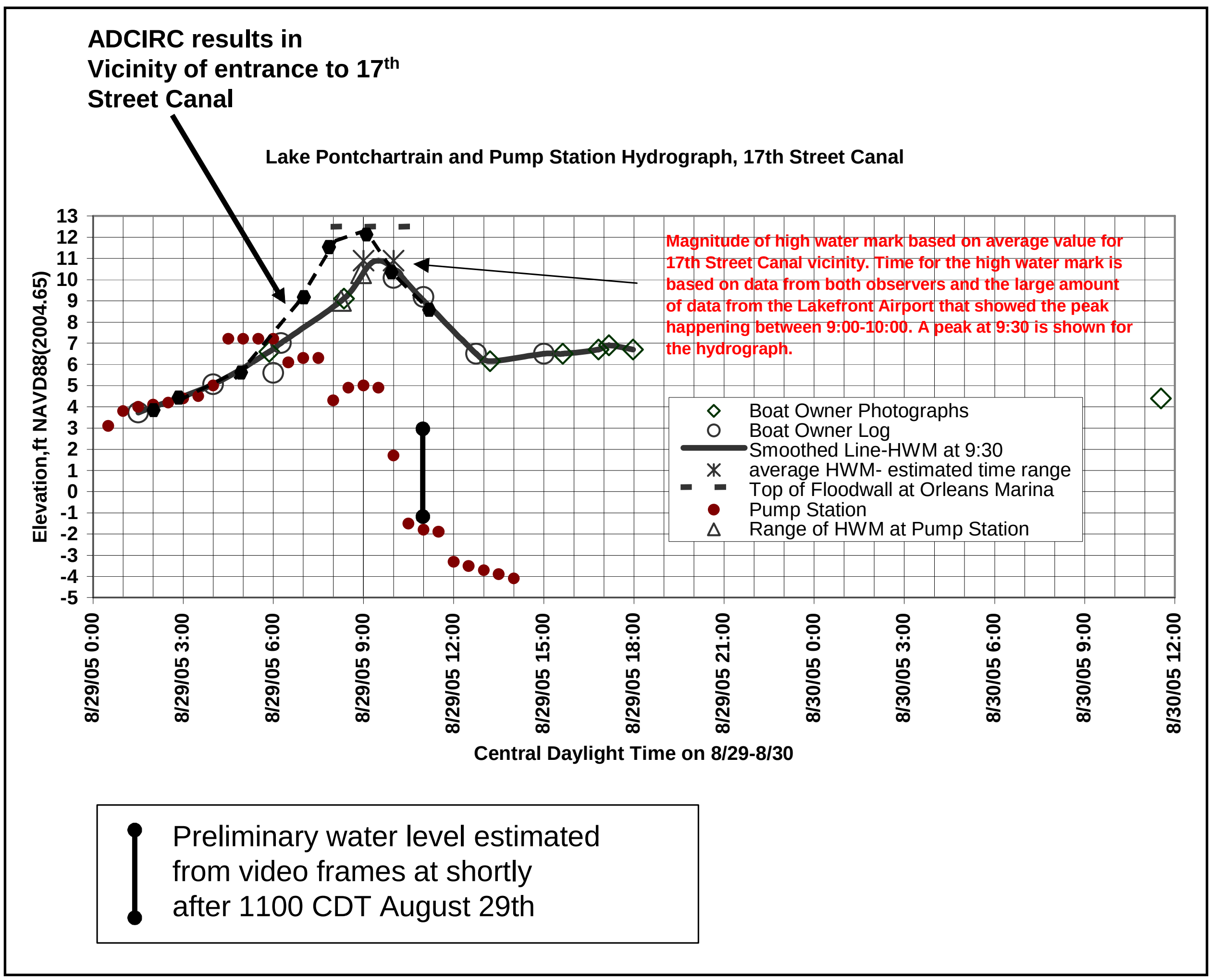

Figure V-50. Observed and estimated water levels inside and in the entrance to the 17th Street Canal during Hurricane Katrina

Figure V-51. Frame from a video of the breach in the 17th Street Canal shortly after 11:00 on August 29th

At this point, we can say with some certainty, as confirmed by at least two independent observers, that the floodwall had already failed by daybreak on the morning of Katrina. Examination of the water levels in Figure V-50 suggests that the water level at the time of failure was in the range of +6 to +7 feet (NAVD88 2004.65). Subsequent analyses and discussions with the pump operator who made the observations at the south end of the canal are in progress and once these are complete, we will be able to provide appropriate results, including estimates of uncertainties, for the critical period near the peak of Katrina.

As can be seen from the above discussion, there is some uncertainty in the water levels that should be used in analyses of wave conditions within this canal. It is also important to recognize that results from the physical model should provide valuable information for subsequent model runs within this canal. However, in spite of these potential complications, we believe that it is possible to provide reasonable first estimates of wave conditions during the storm. Figure V-52 shows estimated wave heights at the site of the breach based on two different sets of assumptions. The line labeled “wave height 1” includes an estimated decay due to the bridge and debris on the north side of the bridge; whereas, the line

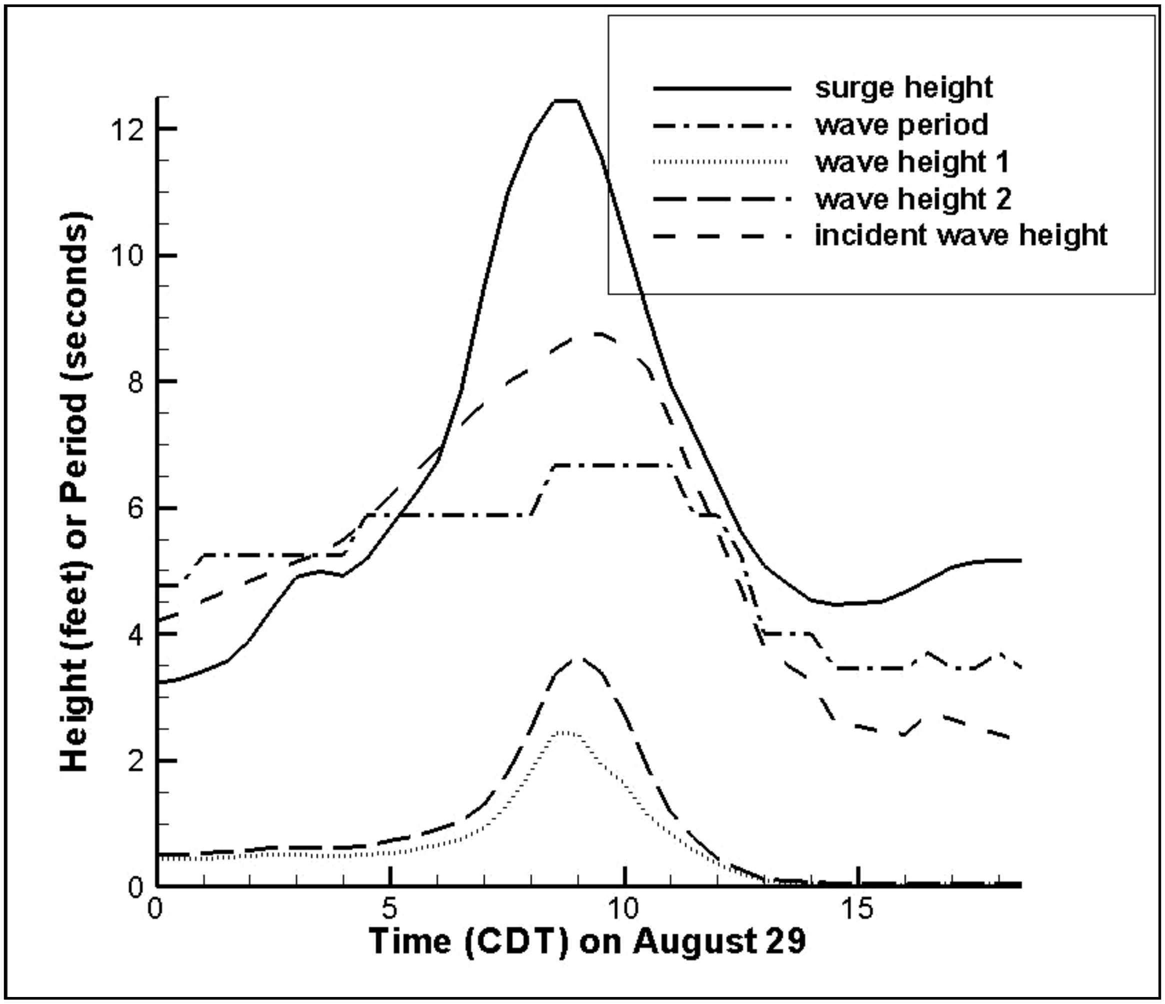

Figure V-52. Time series of estimated water level and wave conditions at the site of the 17th Street canal breach, under the assumption that water levels at the breach are equal to those at the entrance

labeled “wave height 2” neglects this decay. Water levels throughout the storm are set to the water levels shown in Figure V-52. Wave periods are essentially the same for the entrance of the canal as reported by the Surge and Wave Model Group.

Boussinesq simulations indicate that wave heights in the canal at the time of breach (~0600 CDT) were less than 1 ft. These simulations do not yet include any dissipation or reflection due to debris or the bridge, and also do not include wave growth due to wind forcing. These simulations do capture the complex, 3D bathymetry-driven wave transformation at the canal entrance. The small predicted wave height in the early morning leads to pressure predictions that are dominantly hydrostatic, with wave-related bottom pressure oscillations of 21 – 25 psf in amplitude with period of 5 - 8 seconds, or a wavelength of 110 - 210 ft in 26 ft of water. Hydrostatic bottom pressures at this time were approximately 1600 psf. Simulations at later times, when the wind and wave direction was better aligned with the canal orientation (roughly 1200 CDT), predict larger wave heights in the canal, approaching 3 ft. Preliminary runs also indicate the possible existence of a complex 3D wave field inside the canal, with certain sections of the canal experiencing cross-channel oscillations. Physical modeling is necessary to investigate the existence of such modes.

The dynamic forces and moments acting on the flood walls due to waves could be significant to wall stability. For purposes of illustration here, we consider the reasonably representative case of a mean water level of 5 ft against the floodwall and a wave height of 2 ft propagating along the wall. The static hydraulic force and moment about the base per unit wall length for this scenario are 800 lb/ft and 1333 ft-lb/ft, respectively. Applying linear wave theory for this example, the percentage fluctuating force and moment contributions relative to static values at the wave crest and trough are shown in Table V-4 below.

Table V-4
Percentage Change From Hydrostatic Forces and Moments on a Floodwall
With a Mean Water Depth of 5 feet and a 2 foot Wave Height

Percentage Change in	Under Crest	Under Trough
Force	+ 44 %	-_36 %
Moment	+73 %	- 49 %

The results of the simple calculation in Table V-4 illustrate that waves can play a potentially significant role in the integrity of a flood wall. Additionally, the effect of fluctuating forces and moments may be relevant to foundation stability. Finally, the fluctuating forces and moments would propagate along the flood wall, thereby causing shear forces between the adjacent wall panels. In summary, the role of fluctuating loads on the flood walls may be significant and should be considered in this evaluation. Although the simple example here has considered only a single linear wave, the final results will evaluate the forces and moments associated with irregular and nonlinear waves.

Barge Motions and Forces in the Inner Harbor Navigation Canal

A limited description of the work conducted on this issue was presented in Report 1. The complete treatment and summary is presented in the following paragraphs.

This analysis relates to the motions of and potential collision forces due to a free floating barge under the action of wind forces. The issue addressed is whether the barge that floated through the east floodwall of the IHNC Canal could have contributed to its failure through impact.

The equations governing the effective wind speed acting on a barge present in the wind boundary layer are examined and an effective wind speed defined for drag force calculations. Static wind forces and moments acting on a lightly loaded barge and then transferred to the east IHNC floodwall due to a wind speed of 100 miles per hour have been examined and found to represent a reasonably small fraction of the hydrostatic forces and moments exerted directly on the floodwall. These forces and moments have been expressed as averages per unit length on the floodwall although the barge related forces were likely transferred as a concentrated loading rather than uniformly.

The equation of motion of a freely floating barge has been developed and cast in non-dimensional form for easy application. The equations include development of the terminal velocity of the barge. The equation is solved for the non-dimensional velocity and displacement.

It is found that the terminal velocity of the barge is achieved rather quickly for the wind speed examined (100 miles per hour) and that for barge conditions in the INHC the momentum and energy impact on the east flood wall depend primarily on the draft of the barge during the event. Simplified equations have been presented for terminal momentum and energy for use by others in evaluating whether the barge was a contributor to the failure of the INHC flood wall in the Lower Ninth Ward area. The forces depend on the details of the collision including the time over which the momentum is transferred from the barge to the floodwall and the orientation of the barge relative to the wall during impact.

Hydraulics of 17th Street Canal Including Breach Characteristics

The availability of data relating to the hydraulics and breach characteristics in the 17th Street Canal provide a unique opportunity to evaluate the contribution of this breach to the flooding during Hurricane Katrina.

The water level time history in Lake Pontchartrain was established through interviews and collection of other perishable information by the Data Collection and Management Group. Additionally, the pump operator at the south end of the 17th Street Canal recorded visual observations of the water level on a staff at this location. These results combined with limited eyewitness accounts of the timing of breach width characteristics provide the basis for the preliminary hydraulic analysis. The main results of that analysis are reviewed in the following paragraphs.

The initial breach appeared to have occurred at approximately 0600 (CDT) on August 29, 2005 and was later observed to be wider at 0900 on the same day. Standard steady state hydraulic calculations were carried out to estimate the time history of discharges into the canal from Lake Pontchartrain and through the breach. With these estimates available, the consideration was made that the flow through the breach was critical which allowed the breach sill elevation to be estimated.

The peak breach discharge occurred at approximately 0900 on August 25, 2005 at slightly greater than 40,000 cfs. The minimum sill elevation also occurred at 0900 and was approximately -12.1 feet. The next phase of this analysis will reduce uncertainties in the observational data to the degree possible and will evaluate the reasonableness of the calculations. It is noted that the

ADCIRC numerical model is also being applied to evaluate the hydraulics in this canal.

Physical Model

The 14,000 sq ft, 1:50 scale, physical model of the 17th St Outfall Canal has been constructed as of this report date and is being readied for testing. Construction was performed in 6 weeks for a model area that would typically require 4 months to construct. A physical model at this scale is a useful tool in providing objective results for wave conditions in the canal during the storm. The physical model includes reproduction of over one mile along the lakefront, the Hammond Highway Bridge, and a portion of the canal 1200 ft beyond the breach zone. Figure V-53 shows the model during the final stages of construction. Data collection will now be initiated with wave and water level conditions determined from numerical models conducted by the Surge and Wave Model Group. Wave data from the physical model will aid in the calibration of numerical wave models for wave transmission and these models will provide detailed response of the entire canal to short and long wave energy. Tests will proceed from the present to April 15. Appendix E discusses the physical model work in greater detail.

Figure V-53. Physical model during construction; left photo showing overall view, and right view looking south, down the 17th St Canal

Interim Results

ADCIRC Model Tests

A series of ADCIRC (Luettich and Westerink, 2004) model tests were performed to examine the variation of water surface elevation (WSE) within the 17th Street and London Avenue Canals. In addition, a series of sensitivity tests were performed to investigate the effect of boundary condition specification and bottom friction on predicted WSE's and current speeds. The grid domains used for these tests are shown in Figure V-54.

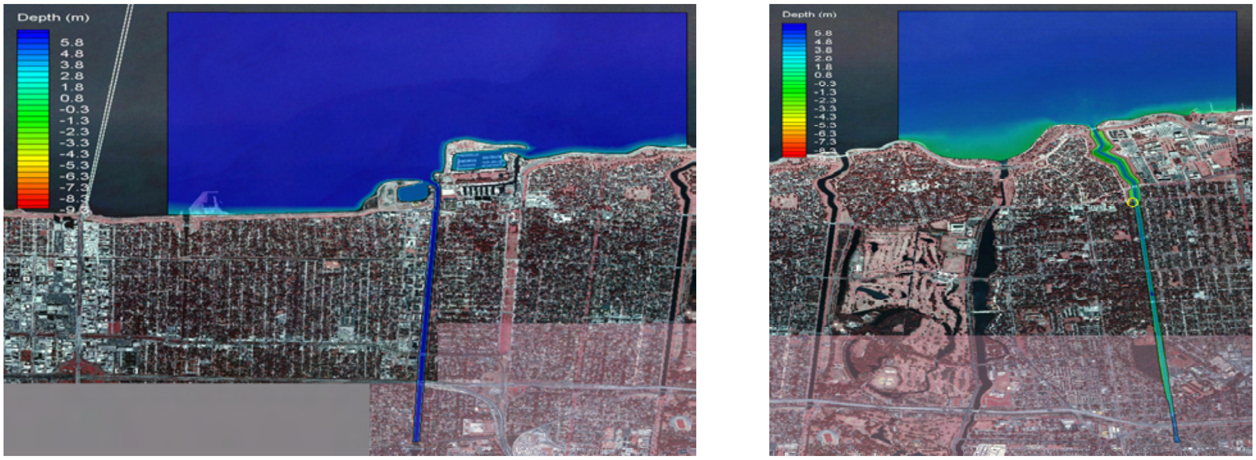

Figure V-54. 17th Street (left) and London Avenue (right) Canals grid domains

Simulations to date have been performed using Lake Pontchartrain WSE boundary forcing only (provided by regional surge and wave modeling efforts). Therefore, all results presented herein do not include additional water level and velocity contributions from locally-generated wave and wind effects. Furthermore, all simulations to date were performed without allowing the canals to breach.

In both the 17th Street and London Avenue Canals, maximum velocity magnitudes during the storm, in the absence of a breach, were small, on the order of 0.35 m/s. A long-period (on the order of one hour) oscillation in the velocity field was simulated in both canals during rising surge.

The WSE time series throughout both canals varied little from the input forcing hydrograph at the Lake boundary (Figure V-55). At the storm peak, water level inside the canal was less than 3 cm different from that in the Lake. No long-period oscillation in water level was observed in the simulated results.

Lateral Boundary Condition. The effect of the specification of lateral boundary conditions is shown in Figure V-56. The lakeward boundary condition is a time series of WSE from the Katrina ADCIRC output provided by Surge and Wave Model Group. The lateral boundaries are specified as combinations of radiation and slip wall (zero-gradient). “West Rad” corresponds to a radiation boundary condition on the west boundary and a slip wall on the east. The “Both Rad” and “Both Wall” are what they state. It is seen in Figure V-56 that WSE variations at the breach are essentially the same regardless of the boundary condition specified, with the exception that the case without radiation boundaries (“Both Wall”), which traps a reflected wave.

The time variation of the WSE as a function of position within the 17th St. Canal is shown in Figure V-57, in which the WSE at the Breach location, Mid-Canal and at the pump station are nearly identical.

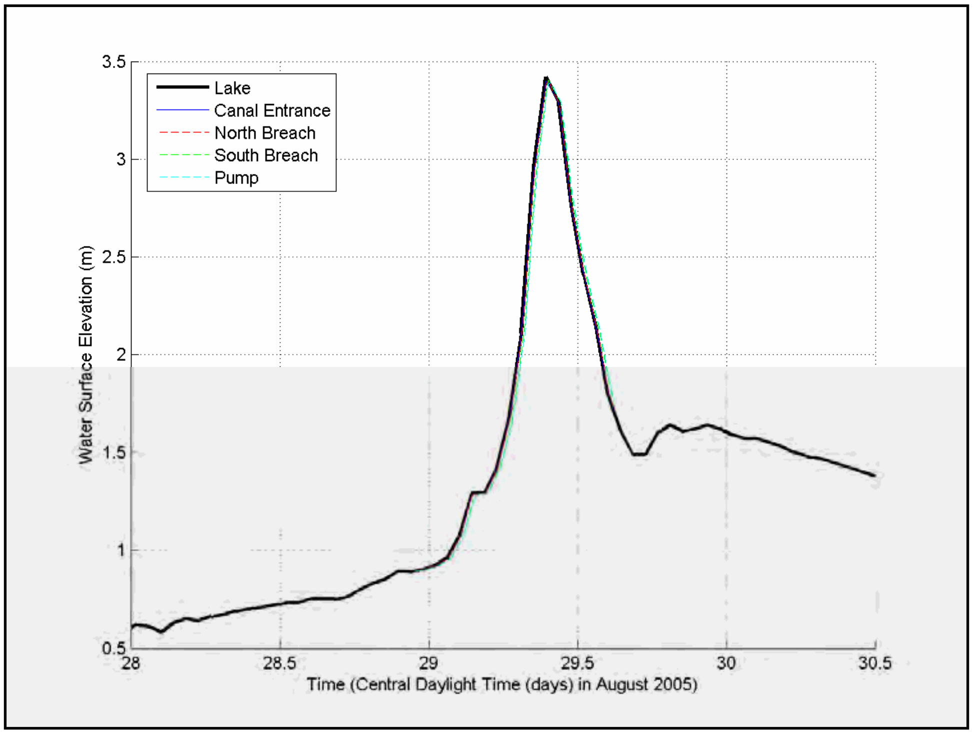

Figure V-55. London Avenue Canal water surface elevation timeseries compared with input Lake forcing timeseries

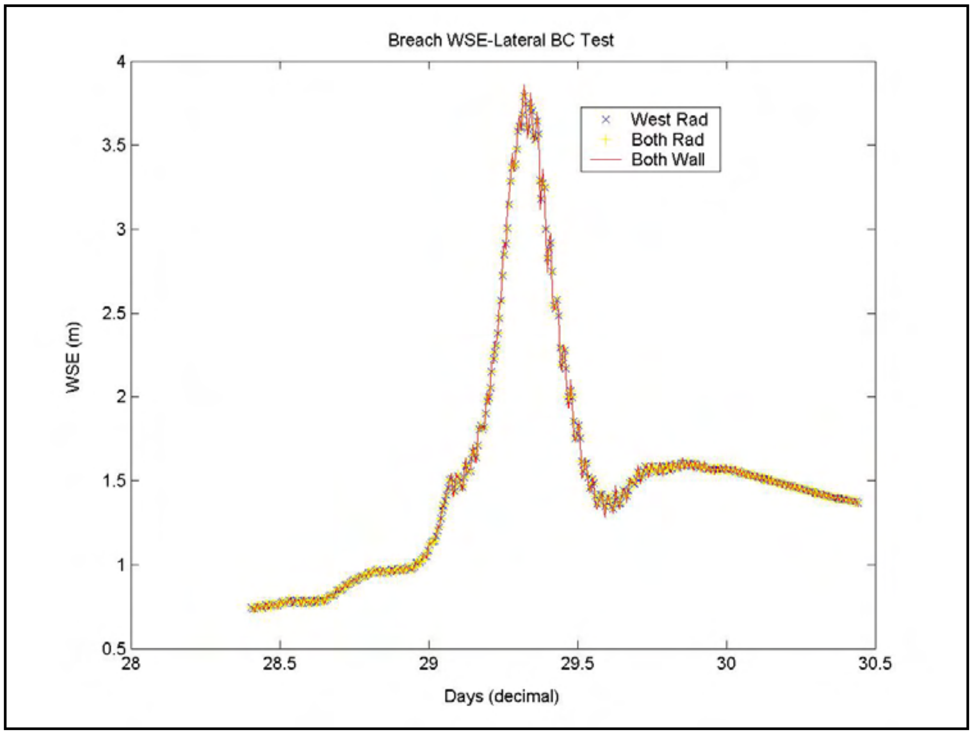

Figure V-56. Lateral boundary condition tests

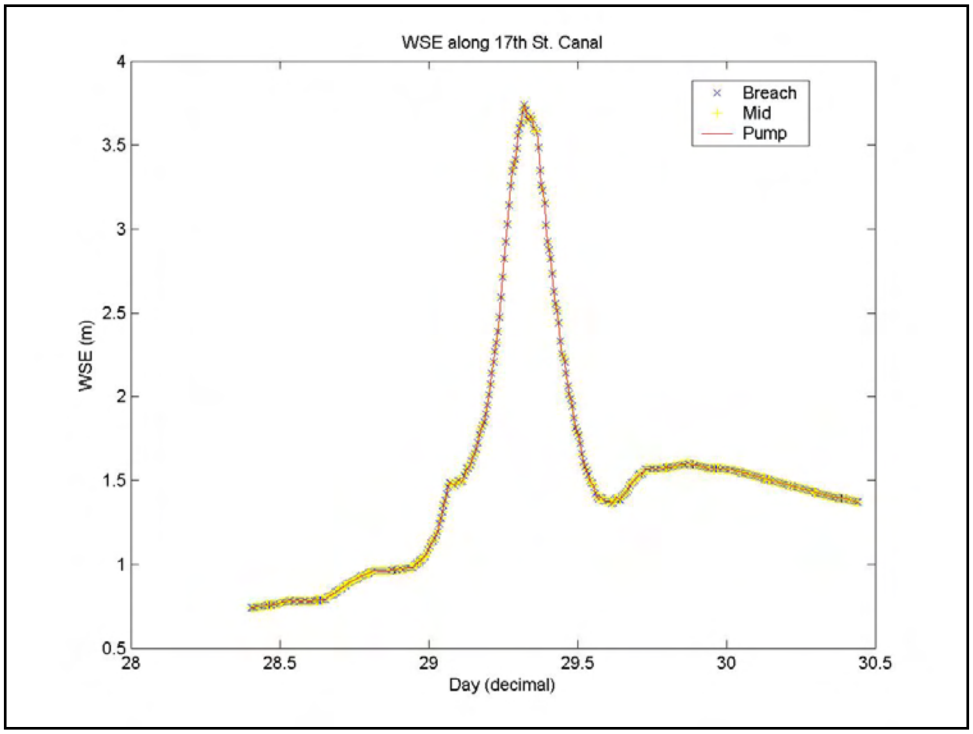

Figure V-57. Along channel water surface elevation variation. “Both Rad” lakeward boundary

Canal Side-Wall Boundary Condition. A series of tests were conducted to investigate the canal side-wall boundary condition using the London Avenue ADCIRC mesh. Two boundary conditions were tested: 1) a slip condition, representing an idealized flow at the canal walls and 2) a no-slip condition, representing the effects of viscosity on the flow at the canal walls. Figure V-58 gives snapshots of the velocity fields with the slip and no-slip boundary.

The impacts of this boundary condition are evident in the velocity magnitude patterns, where velocity magnitude drops to zero at the canal walls using the no-slip boundary condition. In contrast, the velocity magnitude across the canal is more uniform when a slip boundary condition is used. Peak velocity magnitude occurs at the canal entrance during rising surge for both the slip and no-slip scenarios and is 0.35 m/s and 0.25 m/s, respectively. While the percent difference is large, 30%, the velocity magnitudes in both scenarios are small.

While there are some differences in the velocity fields between the slip and no-slip cases, differences in water level within the canal are imperceptible. Furthermore, these differences in water level are well within the uncertainty of the water level hydrograph input and numerical model error.

Bottom Friction. To determine the relative impact of friction on the velocity fields and water levels within the canals, sensitivity tests were conducted using the London Avenue Canal ADCIRC mesh. Bottom friction was defined throughout the model domain using a quadratic friction law, with the dimensionless friction factor, C_f, held constant. Two values of the dimensionless friction factor

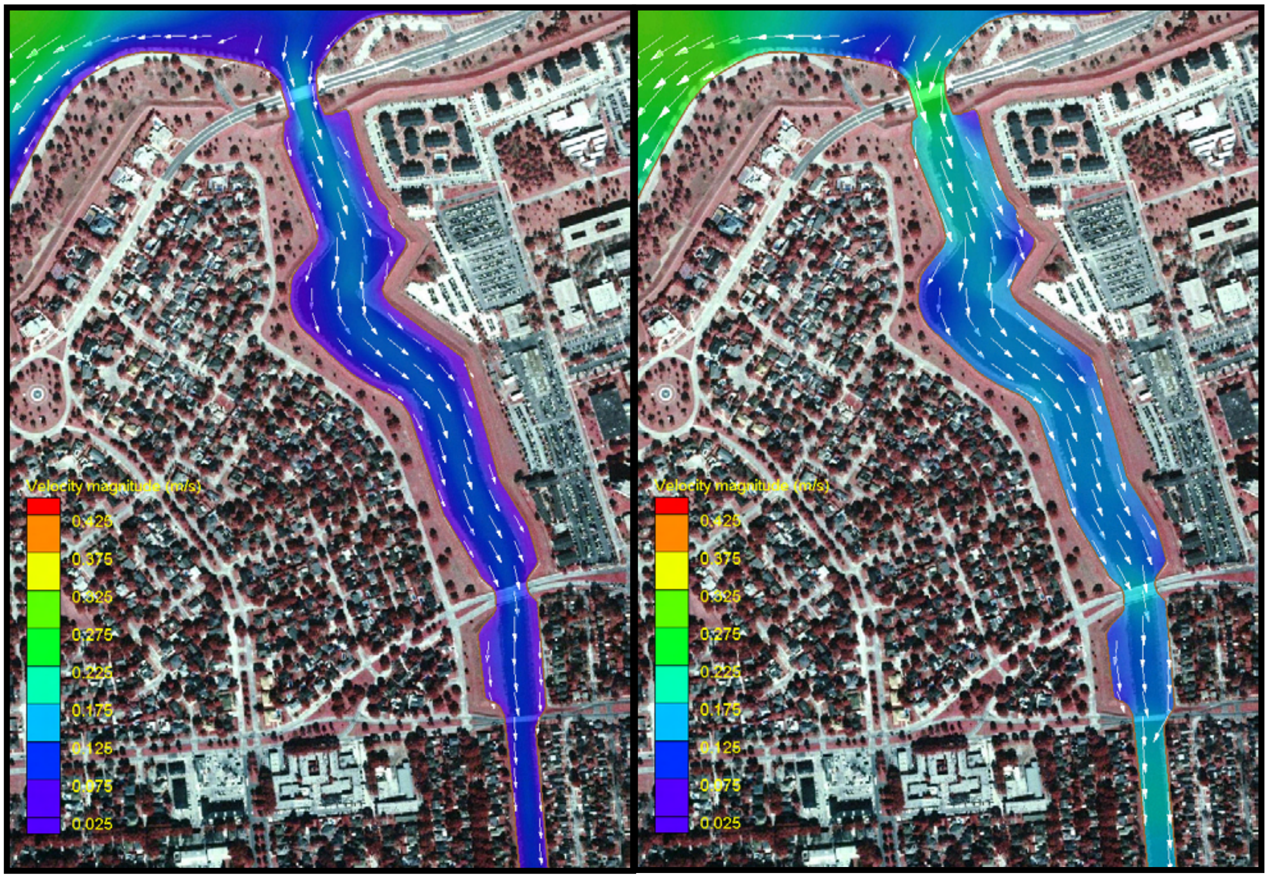

Figure V-58. Snapshot of ADCIRC velocity fields in London Ave Canal during rising surge for no-slip (left) and slip (right) boundary conditions

were assessed: 0.003, representing a smoother bottom, and 0.005, representing a rougher bottom. These values were selected to represent a reasonable range within the canals and follow the recommended values presented in Chow (1959). As with the side-wall boundary condition investigations, bottom friction impacts to water levels within the canal were imperceptible. In addition, the differences in velocity fields were small, with the largest differences occurring at the canal entrance during rising surge. Here, the largest difference was 0.01 m/s, or 3%.

Boussinesq Modeling

Basic Boussinesq Model Information: COULWAVE. COULWAVE (Cornell University Long and Intermediate Wave model) was developed by Patrick Lynett (Texas A&M) and Phil Liu (Cornell) at Cornell during the late 90's. The target applications of the model are nearshore wind wave prediction, landslide-generated waves, and tsunamis, with a particular focus on capturing the movement of the shoreline, i.e. runup, overtopping, and inundation.

COULWAVE has the capability of solving a number of wave propagation equations; however the applications for this project use the Boussinesq-type equations. To derive the Boussinesq-type model, one starts with the primitive equations of fluid motion, the Navier-Stokes equations, which govern the conservation of momentum and mass. The fundamental assumption of the Boussinesq is that the wavelength to water depth ratio is large; thus the model is not applicable for deep water waves. This fundamental assumption yields additional physical limitations, such as the vertical variation of the flow must be small, and turbulence must be parameterized – physics such as wave overturning and overtopping of vertically-walled structures are, theoretically speaking, beyond the application bounds of the model. Applications for which COULWAVE has proven very accurate include wave evolution from intermediate depths to the shoreline, including turbulence dissipation from wave breaking and bottom friction.

Additional Details on Wave Simulation near and inside the 17th Street Canal. These two-horizontal-domain simulations use the ADCIRC grid in the vicinity of the canal. The ADCIRC grid is down-interpolated using an inverse distance weighted algorithm with care taken to eliminate coarse grid artifacts such as stepped bathymetry profiles. The total Boussinesq numerical grid is 1.8 mi^2, using a 4.9-ft grid step in both horizontal directions. The incident wave spectra are provided from STWAVE runs and water levels are provided from ADCIRC.

The first simulation recreates conditions near the canal at 0600 on August 29th; a time near the initiation of the breach. Waves approach the canal from the northeast with a significant wave height of 6.6 ft. The surge at this time was roughly 6.6 ft. Figure V-59 shows a snapshot in time of the wave field near the canal entrance. This simulation suggests that the marina just to the northeast of the canal entrance acts as an effective obstacle to wave energy approaching the canal. Wave heights in the canal are near 0.82 ft. Figure V-60 gives the canal-length profile of wave height, mean wave period, and mean bottom pressure oscillation (amplitude of the dynamic bottom pressure). Time series of free surface and bottom pressure are written to derive this data, and 15 minute segments are analyzed, taken 45 minutes after the start of the simulation. Generally, wave properties are constant through the canal, with slightly larger values at the northern segments south of the bridge. Note that this simulation likely underestimates the dissipation/reflection of wave energy by the marina and the residential area to the east of the canal, as the utilized elevation map characterizes this area as flat, and neglects the widespread infrastructure.

A second simulation was run using a wave spectra approaching the canal from a nearly normal direction, relative to the canal orientation. This situation corresponds to a time near 1200, with a wave height of 5.3 ft and surge of 6.6 ft. A snapshot of this simulation is shown in Figure V-61. Due to a more direct approach into the canal, wave heights in the canal are a much larger fraction of the incident wave, approaching 3.3 ft. This simulation also suggests the possibility of cross-channel modes, which can be inferred from the braided wave pattern in Figure V-61.

Additional Details on Wave Simulation along MRGO Levees. Wave impact on levees along MRGO are simulated at four specific transects, as shown in Figure V-62. The levee profiles are taken from the “Lake Pontchartrain, LA and Vicinity Design Memorandum No. 3”, dated November 1966. Incident wave

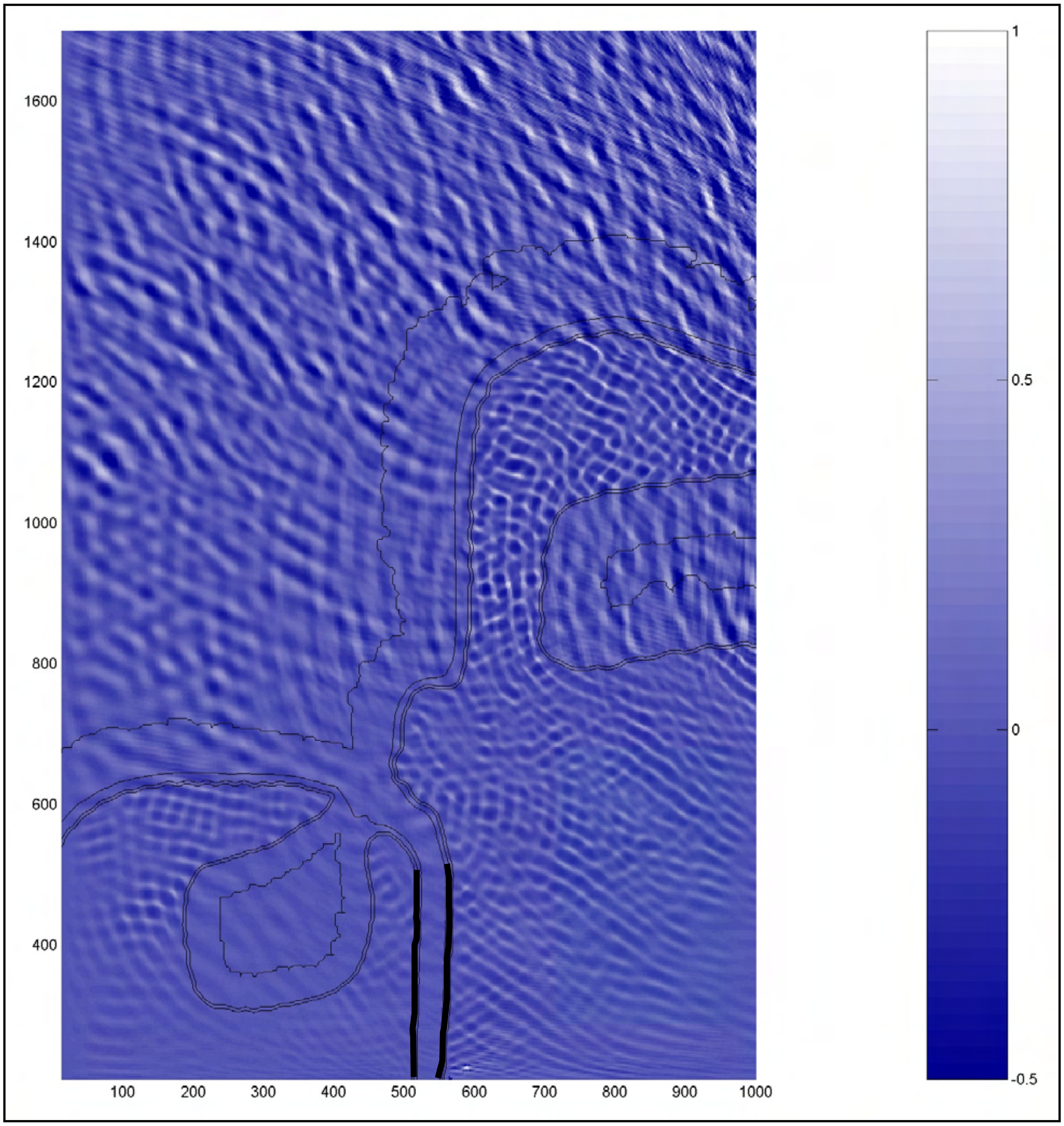

Figure V-59. Snapshot of Boussinesq simulation corresponding to a local time of 0600

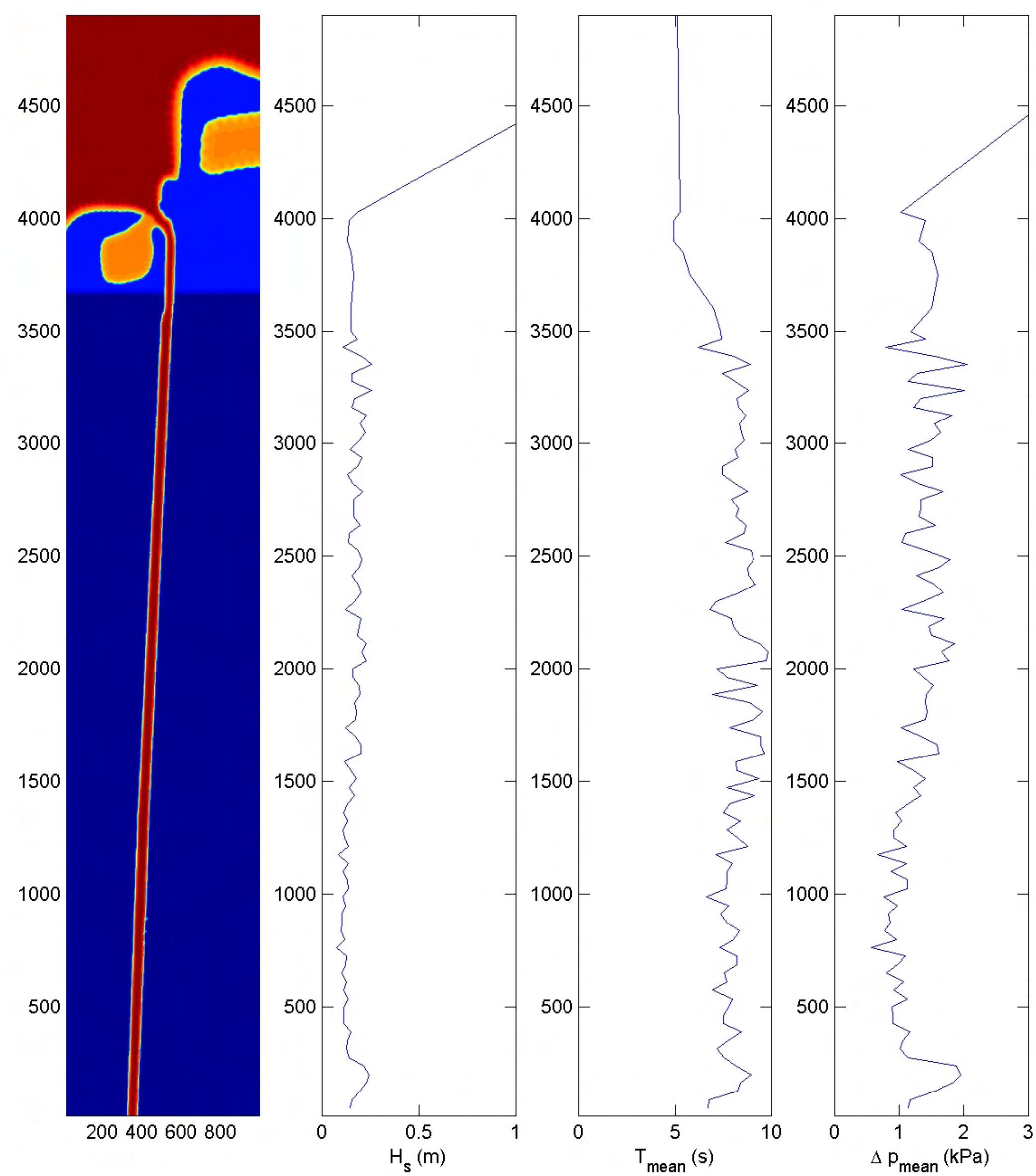

Figure V-60. Canal length profiles for the 0600 simulation

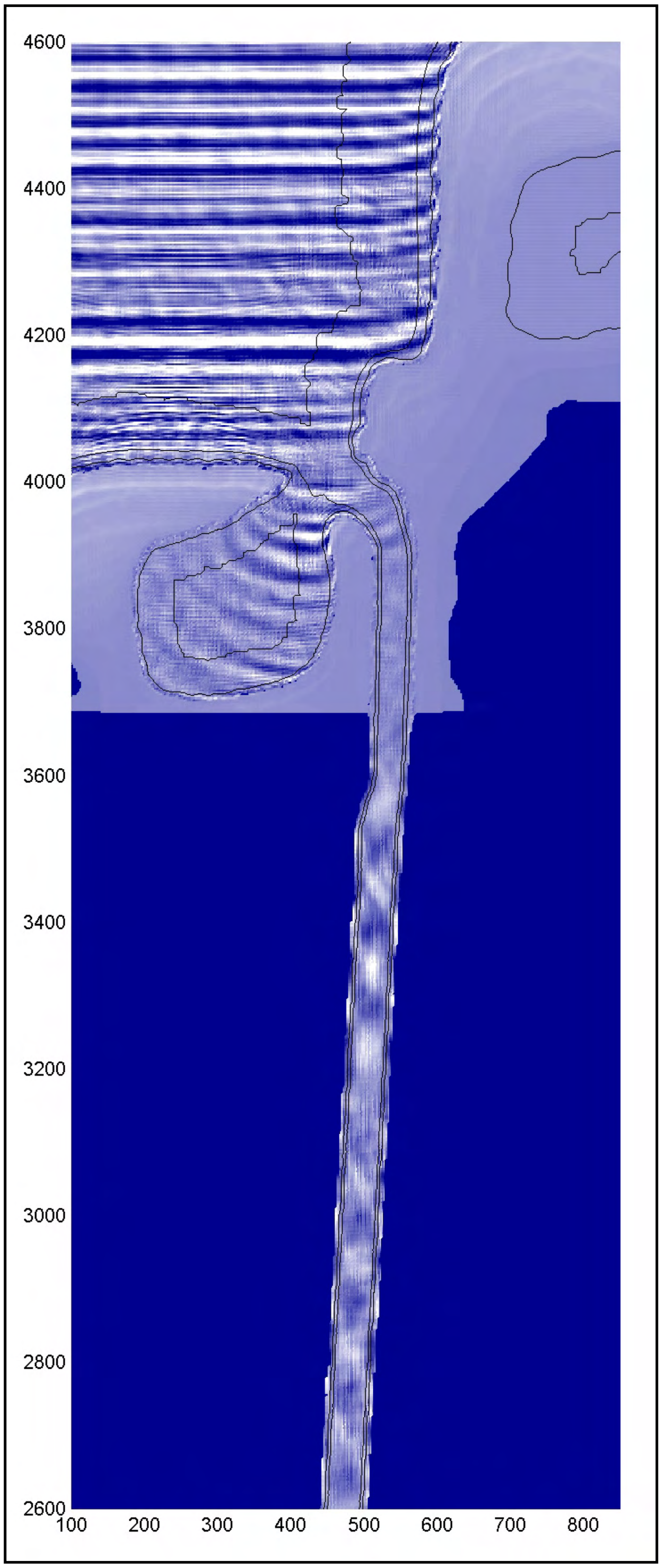

Figure V-61. Snapshot of free surface elevation for the normal incidence wave spectra

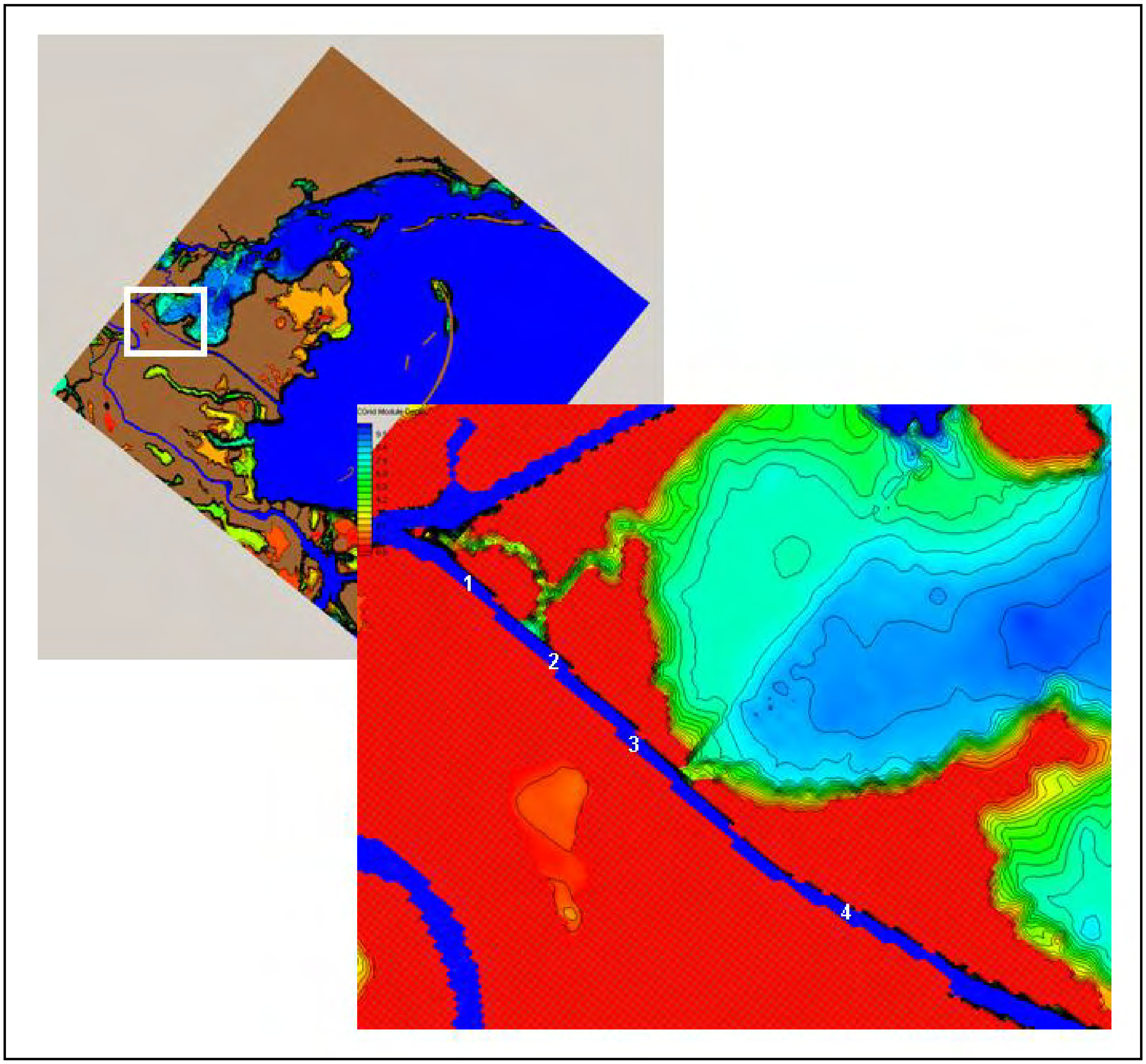

Figure V-62. Location of MRGO transects for simulation

conditions are provided by STWAVE and surge levels by ADCIRC. For each transect, wave spectra and surge levels are specified at 30 minute intervals, from 0600 to 1800 UTC (0100 – 1300 CDT). At each time interval, a simulation is run. An example snapshot from a simulation is given in Figure V-63. These simulations use a 1.64-ft grid, and are run for 30 minutes, with the last 15 minutes of output analyzed.

The time series output of each simulation is distilled into maximum and mean values of frontface runup, frontface velocities, overtopping flux, and backface velocities. Plots of these values for each station are given as Figures V-64 to V-67.

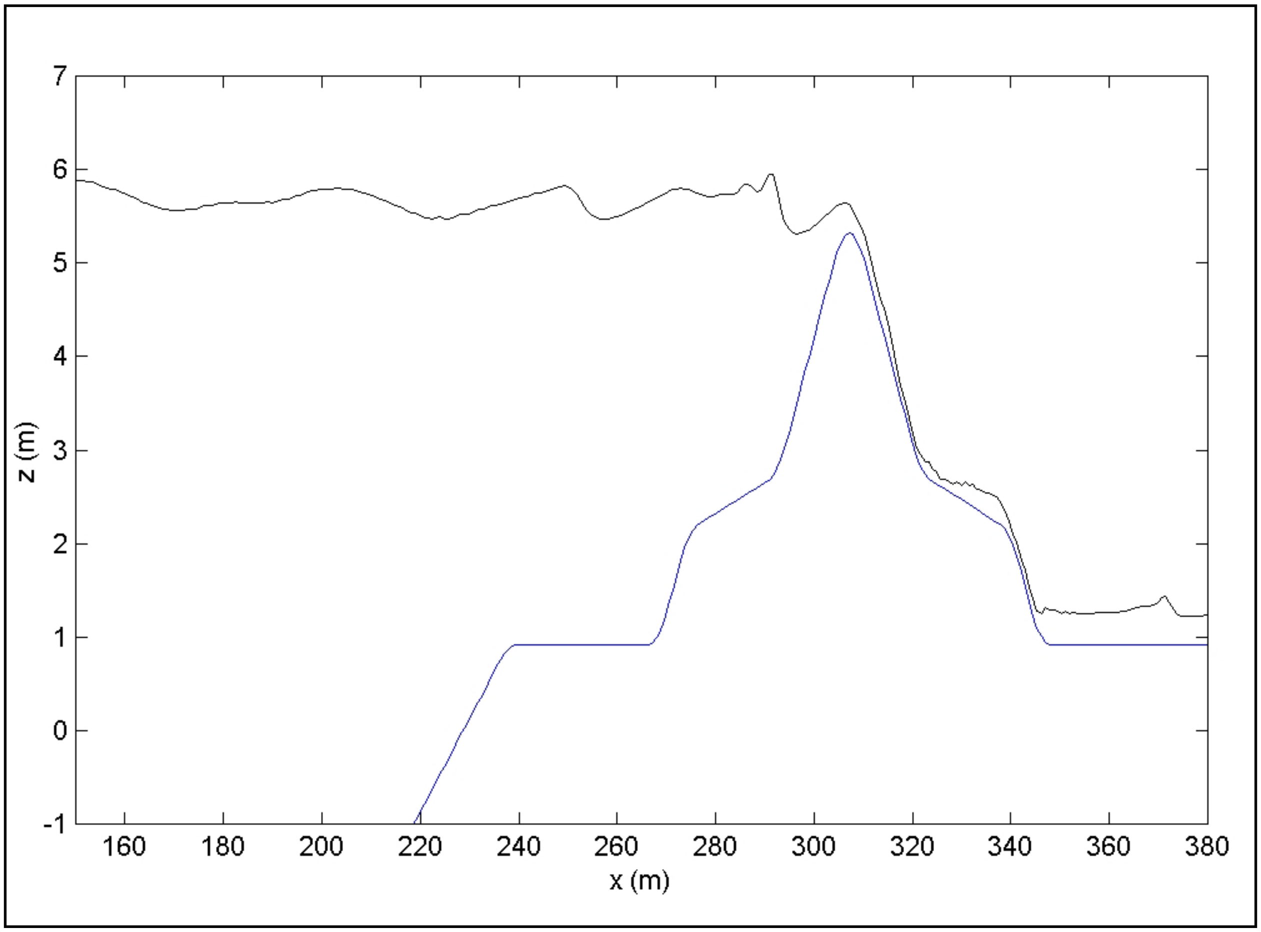

Figure V-63. Simulation snapshot from MRGO station 540 (location #2) at time 1200 UTC (0700 CDT)

Hydraulics of the I7th Canal Breach During Katrina Flooding

Introduction. This develops and provides a preliminary application of an engineering methodology for the analysis of the hydraulics in the 17th Street Canal. The analysis applies the time histories of the water levels at the two ends of the Canal and the geometric characteristics of the canal to estimate the flows through the breach at the 17th Street Canal as a function of time. Based on these results and eye witness accounts of the times of initial failure and later widening of the breach through the levee, approximate discharges through the breach and dimensions of the breach as a function of time are developed. The discharges through the breach will be used in conjunction with other information relating to flooding to improve understanding of the several sources contributing to and the timing of flooding.

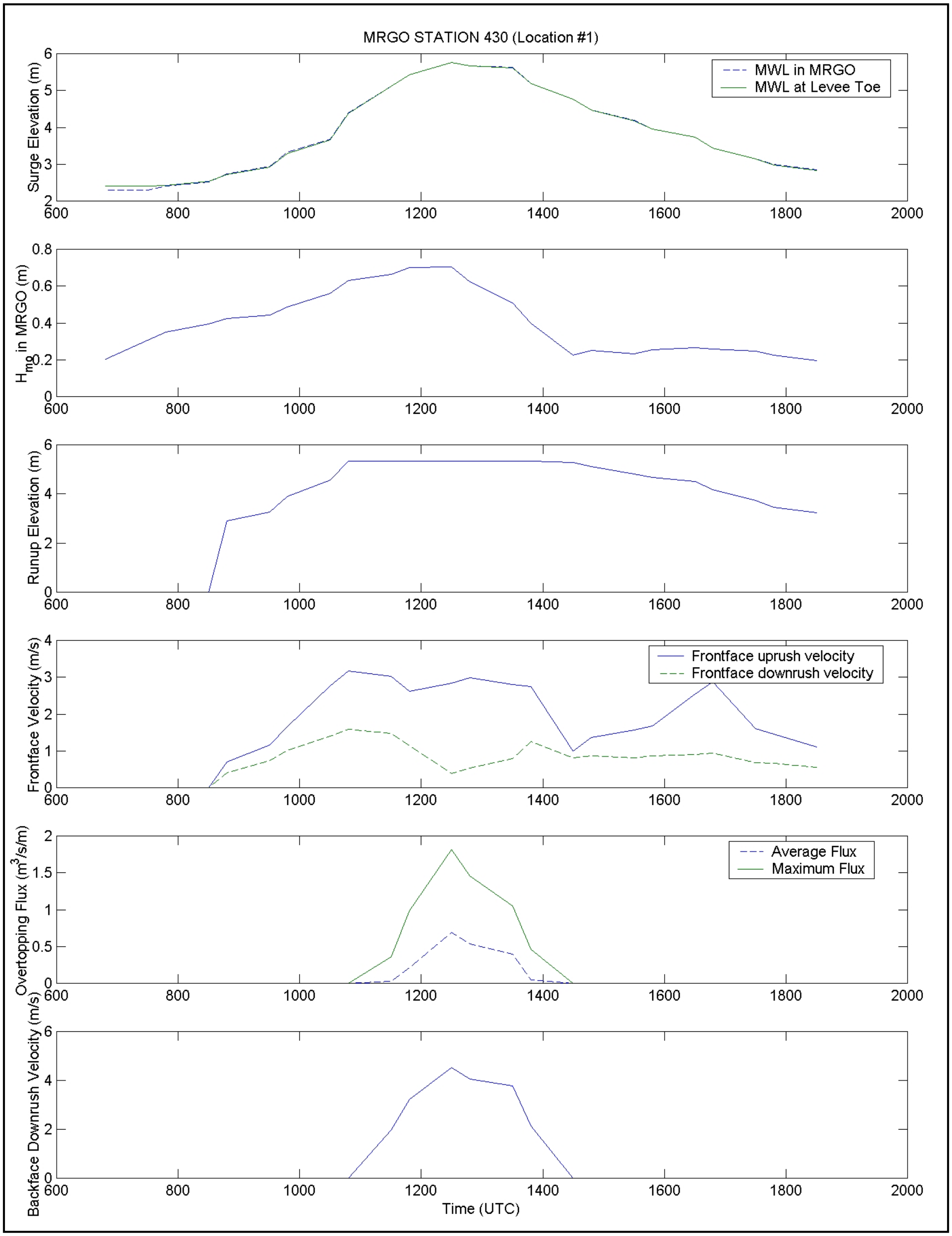

Figure V-64. Simulation summary for MRGO station 430

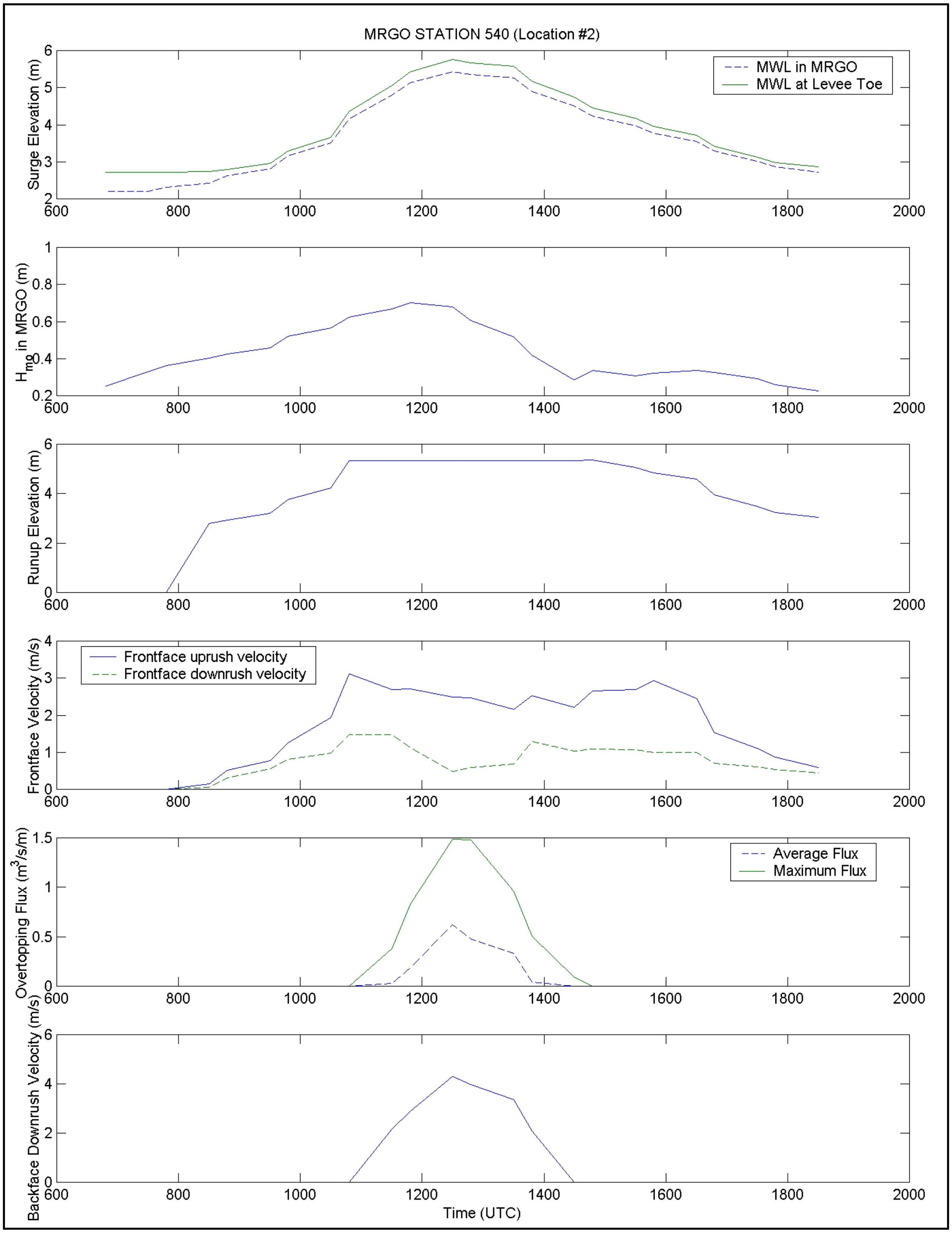

Figure V-65. Simulation summary for MRGO station 540

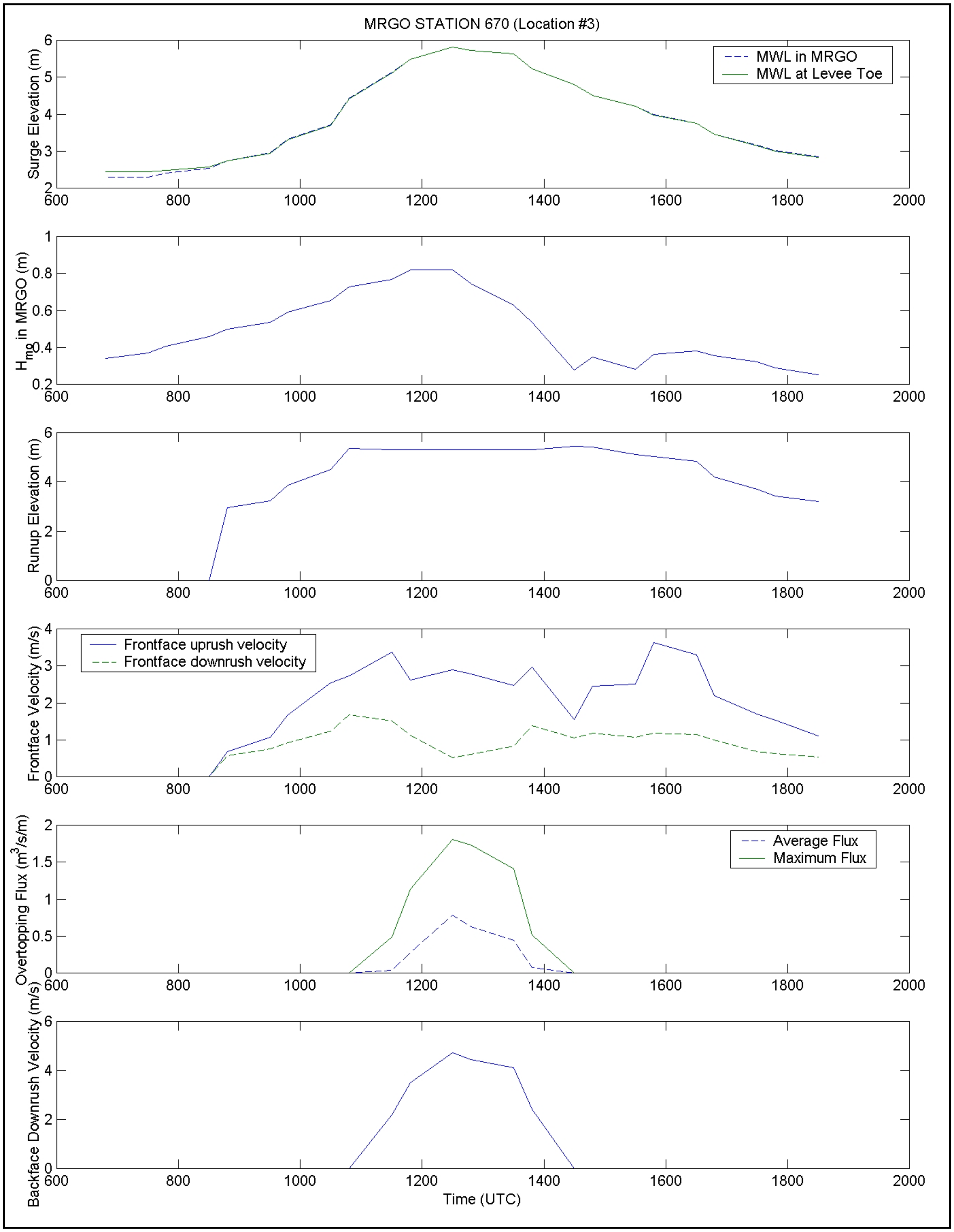

Figure V-66. Simulation summary for MRGO station 670

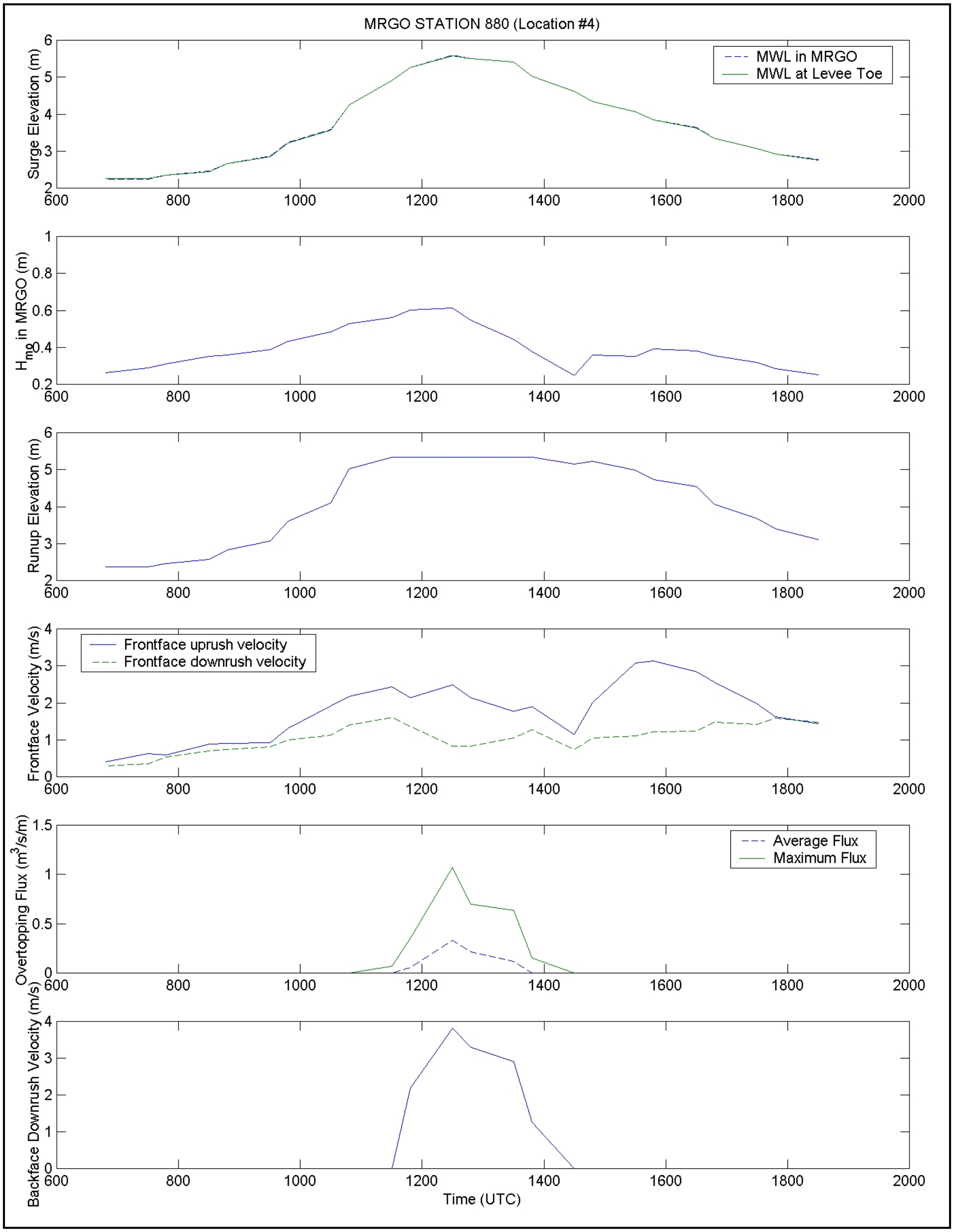

Figure V-67. Simulation summary for MRGO station 880

Available Information. The Data Collection and Management Group has developed the time history of water level in Lake Pontchartrain, η_o in the vicinity of the 17th Street Canal. Additionally, the pump operator at the south end of the 17th Street Canal conducted observations of water level, η_3, on a graduated staff every one-half hour during Katrina. Both of these water level time histories are presented in Figure V-68.

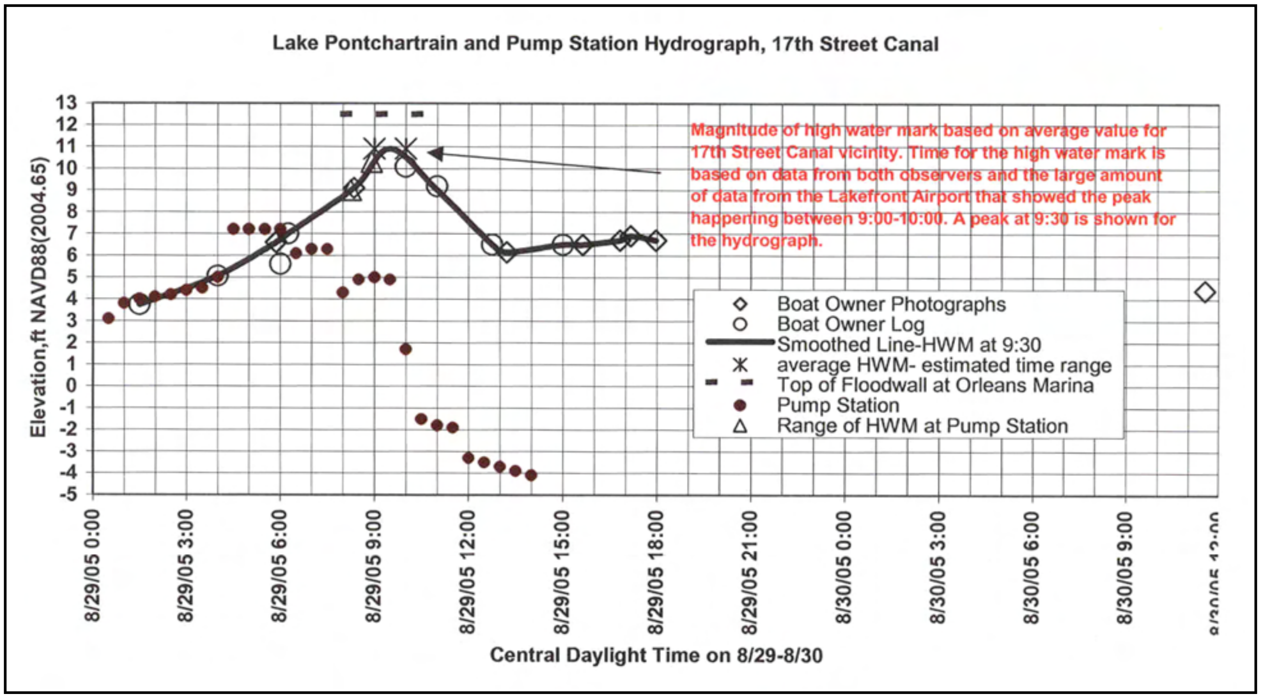

Figure V-68. Water level time histories in Lake Pontchartrain and at the south end of the 17th street canal

Although there is presently some uncertainty of the staff datum used by the pump operator and the validity of the associated elevations, they are the best information available of the water levels at the south end of the 17th Street Canal. Additional efforts will be made to evaluate these elevations.

Figures V-69 and V-70 present an idealized planview and cross-section of the 17th Street Canal, respectively.

Methodology. The equation relating the water level in Lake Pontchartrain, η_o, and the water level immediately inside the canal south of the bridge, η_1, can be expressed as

$$\eta_o = \eta_1 + \frac{Q_1^2(1+K_{en}+K_{BR})}{2gW^2(h+\eta_1)^2} \qquad \text{(V-1)}$$

in which K_{en} is the entrance loss coefficient and K_{BR} is the bridge loss coefficient.

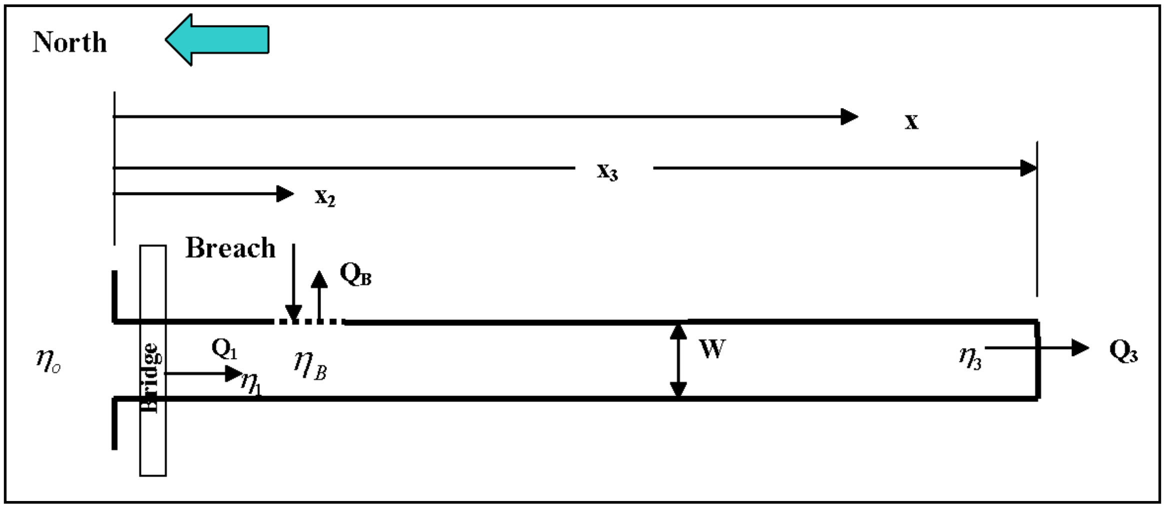

Figure V-69. Idealized planview of 17th street canal

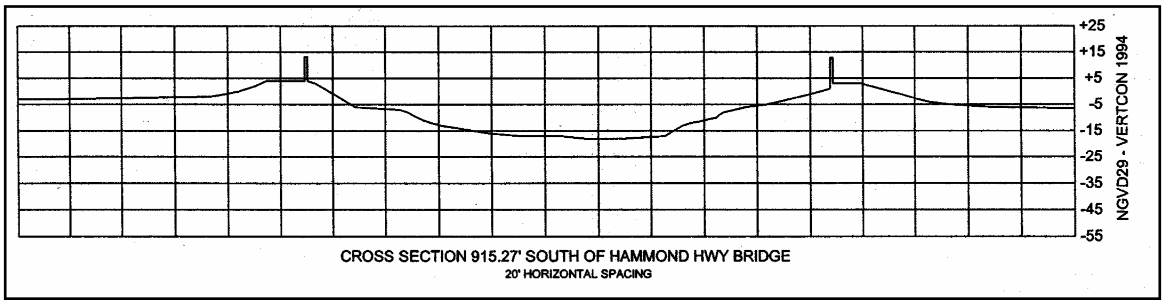

Figure V-70. Typical cross-section of the 17th street canal.

The equation relating conditions at Location 1 to those at the breach is

$$\eta_1 + \frac{Q_1^2}{2gW^2(h+\eta_1)^2} = \eta_B + \frac{fx_2Q_1^2}{8gW^2(h+\eta_{1,B})^3} \qquad \text{(V-2)}$$

in which f is the Darcy-Weisbach friction coefficient and $(h+\eta_{1,B})$ represents the total effective canal depth between Location 1 and the breach.

Finally, the equation relating conditions at the breach to those at the south end of the canal is

$$\eta_B = \eta_3 + \frac{Q_3^2}{2gW^2}\left(\frac{1}{(h+\eta_3)^2} - \frac{f(x_3-x_2)}{4(h+\eta_{2,3})^3}\right) \qquad \text{(V-3)}$$

Equations 1, 2 and 3 provide relationships for the three unknowns, η_1, η_B and Q_1 and can be solved directly for these three variables. With Q_1 known, the total flow through the breach can be determined as $Q_B = Q_1 - Q_3$. Of course, Q_3

is negative and will contribute to the flow through the breach during periods of pumping into the canal.

Breach Characteristics. With the discharge through the breach established, it is possible to estimate characteristics of the breach geometry and, to some extent, the reliability of the water level observations at the south end of the 17th Street Canal.

First, assuming that the breach is rectangular and that critical flow exists through the breach with unit discharge, $q_B = Q_B / W_B$ where W_B is the breach width, the depth on the breach sill, h_B is

$$h_B = \left[\frac{q_B^2}{g} \right]^{1/3}$$

and the elevation of the sill, z_S is

$$z_S = \eta_B - \frac{3}{2} h_B$$

Results. The above equations were applied to calculate the flows and breach characteristics for the following conditions and values of variables:

$f = 0.08$, $W = 200\,ft.$, h = 10 ft, x_2 = 2,200 ft, x_3 = 12,200 feet, W_B = 200 ft up to time 0900 and = 450 ft after 0900[1]. The pump discharge, Q_3 = -5,000 cfs up to time 0900 and = 0 cfs after 0900[2].

Figure V-71 presents Q_B, the flow through the breach and Figure V-72 presents the sill depth under the consideration that the flow is critical through the breach.

Consideration of Wind-Induced Barge Motions and Forces in the Inner Harbor Industrial Canal

Introduction. This addresses the issue of whether the barge that traversed from the Industrial Navigation Harbor Canal (INHC) through the flood wall to the Lower Ninth Ward could have been a cause of the levee failure in this area or whether the barge was simply transported through the levee subsequent to its failure. The Task 5 responsibility is to establish the associated forces relative to this issue.

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[1] The timing of breach width increase is based on one eyewitness account that one section of floodwall was breached by 0630 and that a greater width of wall had been lost by 0930. The final breach width is approximately 450 feet.

[2] The time history of pump operations will be validated in the final version of this analysis.

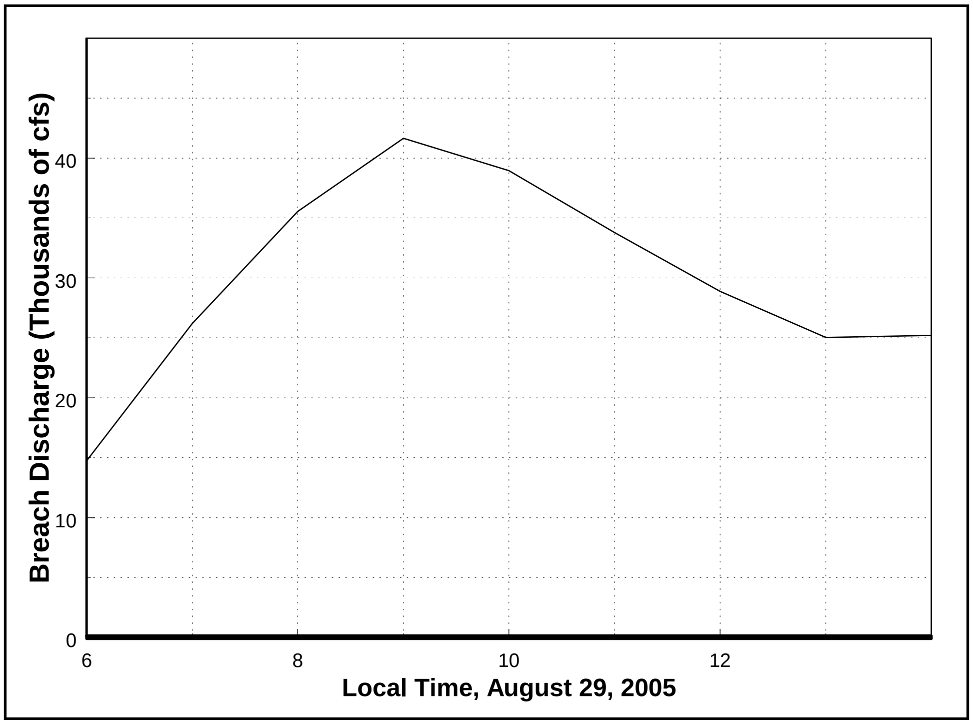

Figure V-71. Estimated breach discharge as a function of time

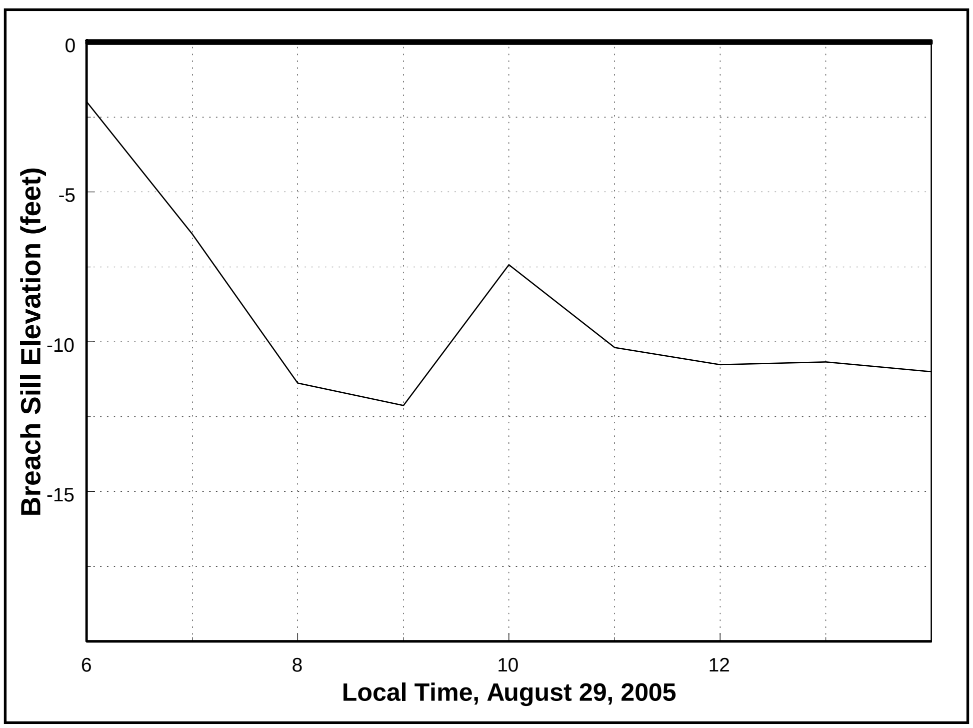

Figure V-72. Estimated time history of breach sill elevation

This brief report examines the wind forces exerted on the barge and the associated velocity, momentum and energy of the barge as it traverses a path across or diagonally along the canal to the location of levee failure. This analysis considers the situation prior to levee failure and no water current forces are considered. Following development of the velocity and trajectory equations, examples are presented to illustrate application of the methodology.

This report is organized as follows. “Barge Characteristics” describes, to the extent possible, the characteristics of the barge that was located outside the INHC after the levee failed. The following section estimates the winds and wind forces on a barge immersed within the wind boundary layer. These wind forces on a static barge are compared with the static hydrodynamic forces which existed immediately prior to levee overtopping. The next section examines the dynamics of the barge for various drafts and provides a basis for quantifying the barge trajectory and momentum and energy upon impact with the east floodwall. Examples illustrating application of the methodology developed are presented in the next section. Recommendations and the summary and conclusions are presented in the final section.

The main focus of this report is to provide a method for quantifying the barge characteristics relative to its possible role in failure of the IHNC east flood wall. The detailed calculations employing this methodology will require improved estimates of the barge and other characteristics required by the methodology.

Figure V-73 shows a plan view of the barge in the INHC and the winds that were directed on the barge.

Barge Characteristics. During the site visit on December 22, 2005, the dimensions of the barge identified as “**ING 4727**” were estimated as:

Hull Depth = 12 feet

Superstructure Height Including Covers for Contents = 11 feet

Barge Length = 200 feet

Barge Width = 35 feet

Figure V-74 presents these barge dimensions.

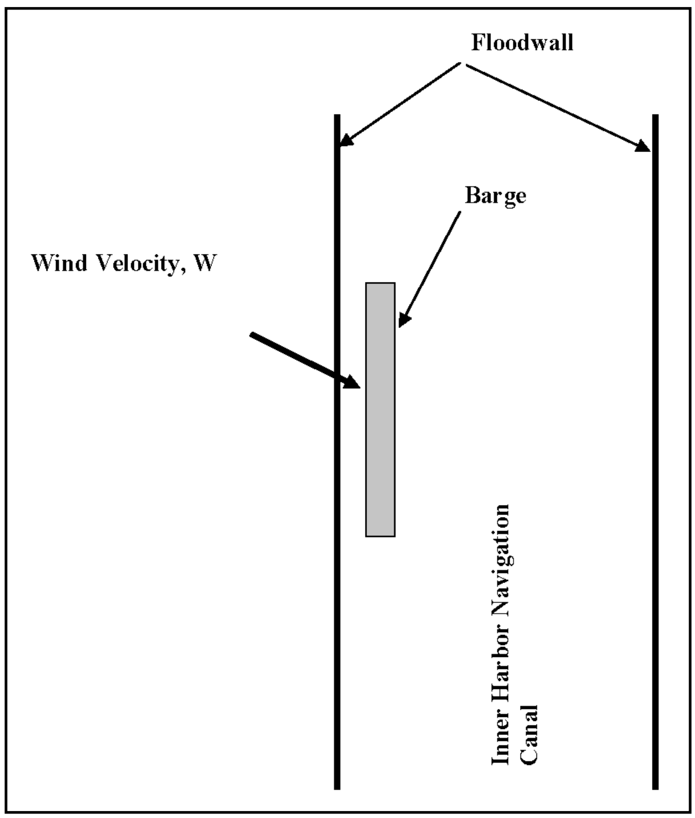

Figure V-73. Definition Sketch of Inner Harbor Navigation Canal and Wind Blowing on the Barge

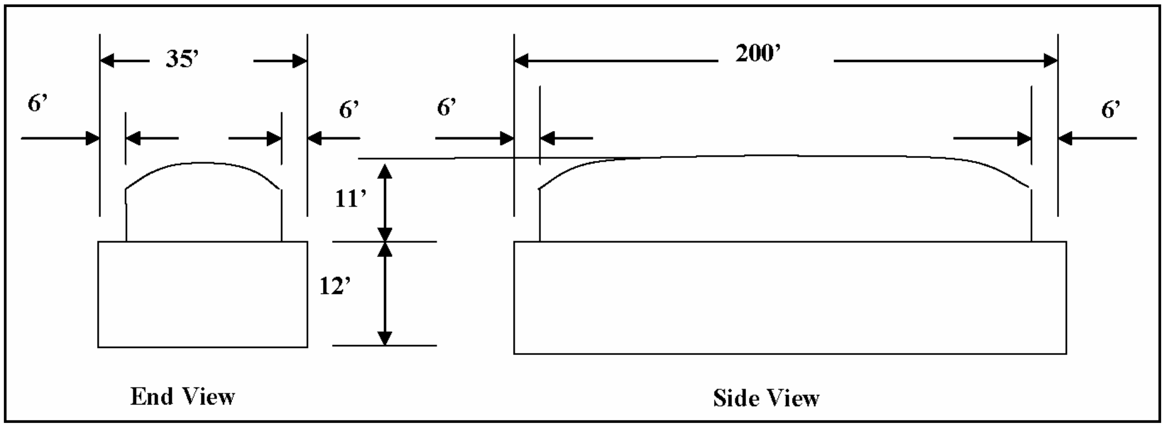

Figure V-74. Estimated Dimensions of Barge Observed on Site Visit to Lower Ninth Ward

Wind Loading and Comparison With Hydraulic Forces on East Flood Wall.

Wind Profile and Effective Wind Speed, W_{eff}. The relevant wind speed is that which is exerted on the barge. For a drag force relationship, this is the root-mean square of the wind speed over the vertical dimension of the above water portion of the barge. For purposes here, the following simple relationship for the vertical distribution of wind speed is considered

$$W(z) = W(30)\left(\frac{z}{30}\right)^{1/7} \tag{V-4}$$

in which z is the elevation above the water surface in feet and $W(30)$ is the reference wind speed at 30 feet above the water surface. The draft of the barge will be denoted as d. Thus the vertical dimension of the barge exposed to the wind is $(23-d)$ feet. The effective wind speed, W_{eff} for drag force computations is therefore

$$W_{eff} = \sqrt{\frac{\int_0^{23-d} W^2(z)\ell(z)dz}{\int_0^{23-d} \ell(z)dz}} \tag{V-5}$$

in which $\ell(z)$ is the length of a barge element at elevation z and $23-d$ is the height of the barge above the water level. Although the length of a barge element does vary slightly with elevation as shown in the previous section, this variation is reasonably small and for purposes here we will consider that $\ell(z)$ is uniform over the height, $23-d$. This results in the effective velocity, W_{eff}

$$W_{eff} = 0.882\left(\frac{23-d}{30}\right)^{1/7} W(30) \qquad \text{(V-6)}$$

Wind Drag Forces on Barge. The drag force, $F_{D,a}$ exerted by the wind on the barge are given by

$$F_{D,a} = \frac{\rho_a C_{D,a} A_a W_{eff}^2}{2} \qquad \text{(V-7)}$$

in which ρ_a is the mass density of air, $C_{D,a}$ is the so-called "drag coefficient" of the barge to winds and A_a is the "projected area" of the barge perpendicular to the wind velocity vector.

For purposes of examples presented in this report, we will consider the wind to be directed broadside to the barge, a wind mass density, ρ_a = 0.002 slugs/ft^3 and a barge length = 200 feet. Thus, the relevant area in Equation V-7 is

$$A_a = 200(23-d) \qquad \text{(V-8)}$$

Static Hydraulic Forces and Moments on Flood Wall Immediately Before Overtopping

Figure V-75 depicts a typical section of the flood wall at an imminent overtopping condition.

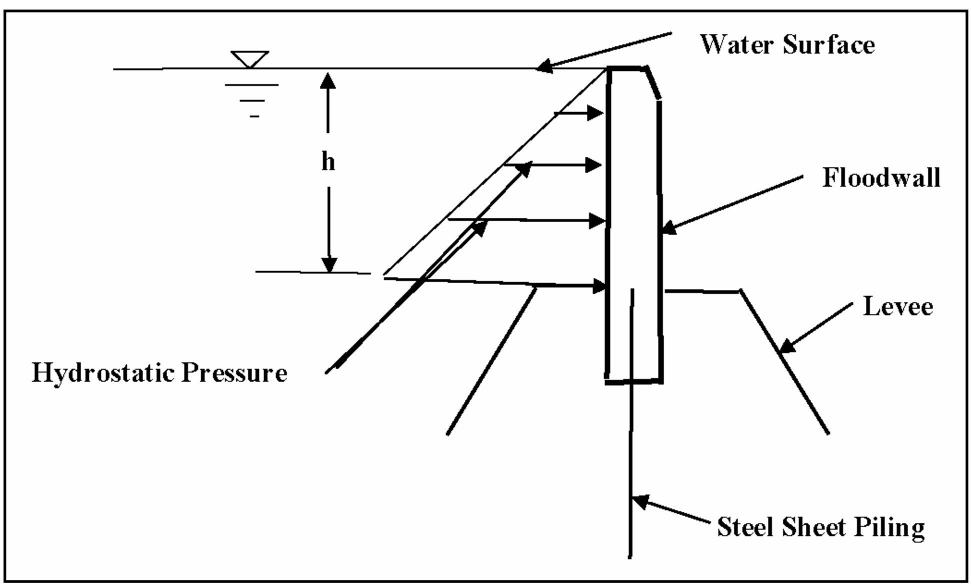

Figure V-75. Definition Sketch for East Floodwall at Imminent Overtopping Condition

The hydrostatic force, F_{HS} on the floodwall per unit floodwall length for the imminent overtopping condition shown in Figure V-75 is

$$F_{HS} = \rho_w g \frac{h^2}{2} \tag{V-9}$$

in which ρ_w is the mass density of water taken here as 1.94 slugs/ft^3 and g is the acceleration of gravity.

The hydrostatic moment, M_{HS} about the base of the floodwall per unit length of flood wall is given by

$$M_{HS} = \rho_w g \frac{h^3}{6} \tag{V-10}$$

Comparison of Hydrostatic Forces and Moments With Static Wind forces and Moments

To calculate wind forces, we need to select a reference wind speed, $W(30)$ as shown in Equation V-4. For most of the examples presented in this report, a reference wind speed of 100 miles per hour (146.7 ft/sec) and a wind drag coefficient, $C_{D,a}$ = 0.5 have been selected for illustration purposes. To illustrate the maximum wind force, a lightly loaded barge condition is selected with a barge draft, d = 4 feet. Applying Equation V-6, the reference wind speed, W_{eff} =121.2 ft/sec. The wind drag force per unit barge length f_{HS} , is then

$$f_{D,a} = \frac{\rho_a C_{D,a}(23-d)W_{eff}^2}{2} = 139.5 \text{ pounds/foot} \tag{V-11}$$

This value is compared to the hydrostatic force per unit length of 1,999 pounds/foot based on a floodwall height = 8 feet. Thus, the static wind force is equal to approximately 7% of the hydrostatic force. However this result is based on a uniform transfer of the wind load on the barge to the floodwall. If this transfer is concentrated, the local wind related loads acting on the floodwall per unit length could be much greater than those calculated above.

The wind related moments about the bottom of the floodwall are considered to result from application of the wind related forces at the mid-elevation of the barge draft, i.e., 2 feet below the crest of the floodwall. In this case, the moment due to the wind is 837 foot pounds per foot compared to the hydrostatic moment of 5,331 foot pounds per foot or the wind moment is approximately 16% of the hydrostatic moment. However, the same comment applies to moments as was presented for forces regarding the consideration that the wind forces were applied uniformly along the wall.

The following section examines the dynamics of the floating barge.

Barge Dynamics Under the Action of Wind Forces

Equation of Motion and Solution. The equation of motion of the barge is:

$$m_T \frac{dV}{dt} = K_1 W_{eff}^2 - K_2 V^2 \tag{V-12}$$

in which m_T is the total effective mass of the floating barge and is the sum of the physical mass and the added mass, V is the barge velocity, t is time after the barge starts to float free, W_{eff} is the effective wind speed acting on the barge as described earlier. The factor, K_1 has been defined earlier as

$$K_1 = \frac{\rho_a C_{D,a} A_a}{2} \tag{V-13}$$

The factor K_2 is defined as

$$K_2 = \frac{\rho_w C_{D,w} A_w}{2} \tag{V-14}$$

in which ρ_w has been defined as the mass density of water, $C_{D,w}$ is the so-called "drag coefficient" of the barge to the water and A_w is the "projected area" of the barge perpendicular to the water velocity vector. In subsequent calculations, the following values of drag coefficients will be applied: $C_{D,a}$ = $C_{D,w}$ = 0.5. The dimensions of both K_1 and K_2 are "force/velocity squared." The complete barge dimensions were presented in section above titled "Consideration of Wind-Induced Barge Motions and Forces in the Inner Harbor Industrial Canal" (see Figure V-74).

Estimation of K_1 and K_2 Factors and Steady State Velocities. From Equation V-10, it is seen that the steady state (or terminal) velocity of the barge, $V(\infty)$ is given by

$$V(\infty) = \sqrt{\frac{K_1}{K_2}}\, W_{eff} \tag{V-15}$$

The values of K_1 and K_2 will be estimated for the case of the barge fully loaded and loaded very lightly. The barge is considered broadside to the wind. The results of these estimates are presented in Table V-5. The values of the dimensionless terminal barge velocity, $V(\infty)/W_{eff}$ are also presented in

Table V-5. Note that the length of the barge acted upon by winds has been taken as 188 feet.

Table V-5 **Estimation of K_1 and K_2 for Two Cases**				
Case	**Description**	K_1 **(Pounds-sec²/ft²)**	K_2 **(Pounds-sec²/ft²)**	$V(\infty)/W_{eff}$
1	Fully Loaded, Draft d = 9 feet	1.32	873	0.039
2	Lightly Loaded, Draft d = 4 feet	1.79	388	0.068

<u>Non-Dimensionalization and Solutions of the Equation of Motion</u>

It is useful to cast the equation of motion in non-dimensional form as:

$$\frac{m_T}{K_1 W_{eff}^2}\frac{dV}{dt} = 1 - \frac{K_2}{K_1}\frac{V^2}{W_{eff}^2} \qquad \text{(V-16)}$$

from which the solution can be shown to be:

$$V(t) = V(\infty)\tanh\left(\sqrt{\frac{K_1 K_2}{m_T}}W_{eff}\,t\right) \qquad \text{(V-17)}$$

The non-dimensionalizing time, t_* ,is defined as

$$t_* = \frac{m_T}{\sqrt{K_1 K_2}W_{eff}} \qquad \text{(V-18)}$$

and is the time at which the barge velocity is 76.2% of its terminal velocity. Choosing the non-dimensionalizing velocity as the terminal velocity, $V(\infty)$, and denoting non-dimensional quantities by primes (e.g., $t' = t/t_*$, the solution for the non-dimensional velocity, $V'(t')$ is

$$V'(t') = \tanh(t') \qquad \text{(V-19)}$$

The non-dimensional barge displacement, $x'(t') = x(t)/x_*$, can be shown to be

$$x'(t') = \ln[\cosh(t')] \quad \text{(V-20)}$$

$$\text{where } x_* = \frac{m_T}{K_2} \quad \text{(V-21)}$$

The advantages of the non-dimensional solutions presented is that they depend on only one variable, t'.

Figure V-76 presents the non-dimensional solutions for the range $0 < t' < 5$ which will be shown to provide adequate information to analyze the case of the barge motions and forces in the INHC canal.

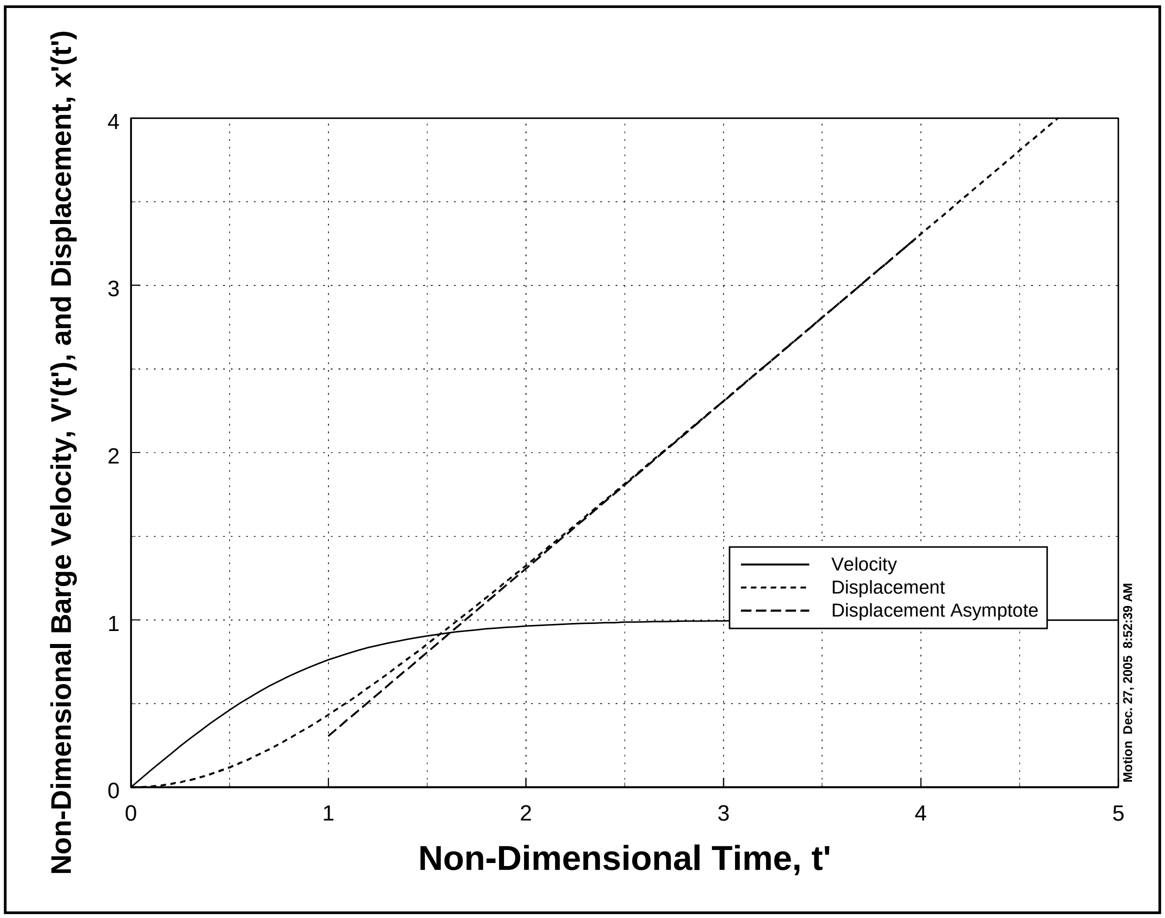

Figure V-76. Non-Dimensional Barge Velocity and Displacement

The non-dimensional relationships are plotted in a different manner in Figure V-77 which has advantages for our particular applications. Figure V-77 presents the non-dimensional barge velocity, $V'(t')$ as a function of the non-dimensional barge displacement, $x'(t')$. In applications, the quantity x is the path of the barge from its starting point to its ending point where it would impact the east flood wall of the INHC canal. This quantity is based on barge and other conditions and is the non-dimensional distance, x'. Entering Figure V-77 with

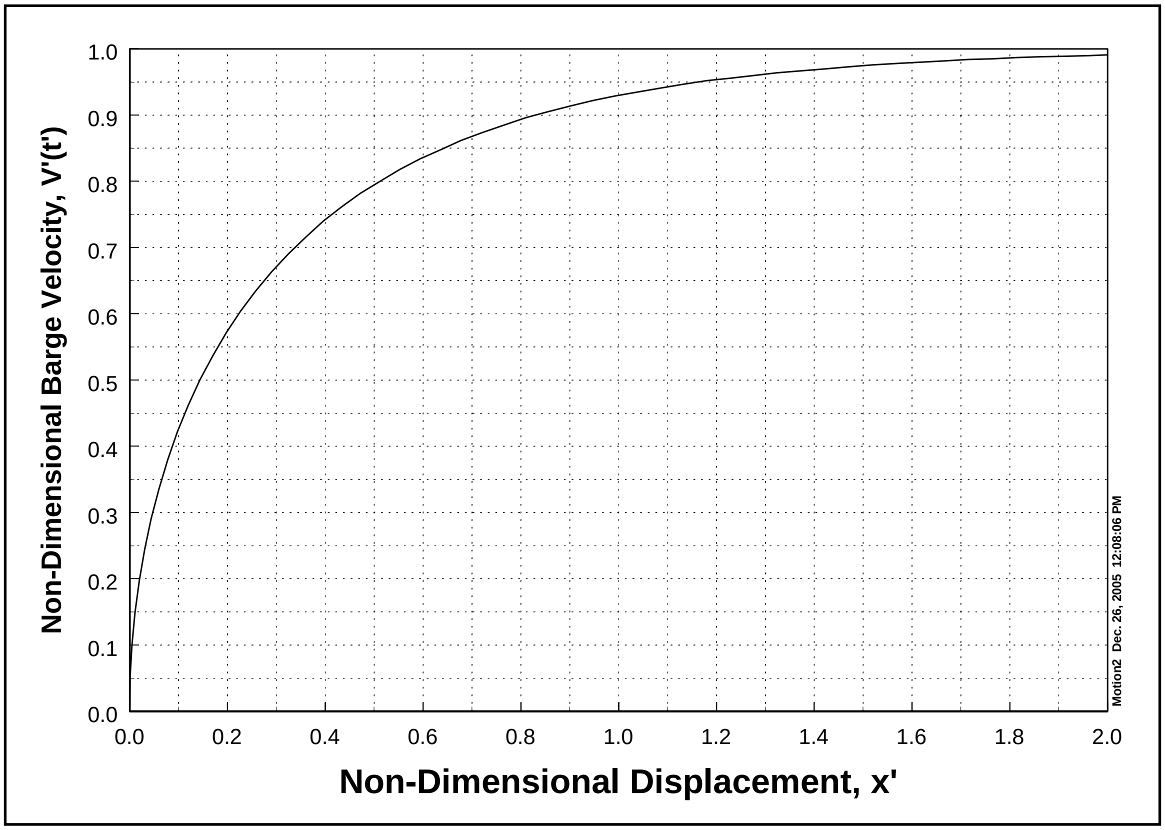

Figure V-77. Relationship Between Non-dimensional Barge Velocity, V'(t') and one-dimensional Displacement, x'(t')

this x' quantity on the abscissa, the non-dimensional velocity, V' is determined. The dimensional velocity, V is then quantified. Finally the momentum and energy of the barge upon impact are determined as:

$$\text{Momentum} = m_T V \tag{V-22}$$

$$\text{Energy} = \frac{m_T V^2}{2} \tag{V-23}$$

The barge displacement, x, should increase linearly with time after the barge has reached its terminal velocity, $V(\infty)$ and this appears to be the case from Figure V-76 but is not so apparent from Equation V-20. However, from Equation V-18, for large t',

$$x'(t') = t' - \ell n(2) \tag{V-24}$$

which is plotted as the asymptote in Figure V-76. Expressing Equation V-24 in dimensional form, this equation becomes

$$x(t) = V(\infty)t - \frac{m_T}{K_2}\ell n(2) \quad \text{(V-25)}$$

which demonstrates the expected linearity of the relationship for large time. The second term on the right hand side of the above equation accounts for the acceleration phase of the barge response, as can be appreciated by the role of the total mass, m_T , such that a larger mass tends to prolong the acceleration phase and thus reduce the displacement at any particular time.

The procedure for calculating barge motion characteristics will be illustrated in the following section of this report.

Examples Illustrating Application of the Methodology

Consistent with the results in Table V-5, two cases are considered: Case 1 in which the barge is fully loaded with a draft of 9 feet and Case 2 for which the barge draft is 4 feet. It is noted that the examples presented here are for illustrative purposes of the methodology. After the detailed characteristics of the barge are more fully established, the motion and force characteristics can be more fully quantified.

Case 1. Barge Fully Loaded. For Case 1, the total mass, m_T is the sum of the physical mass, m_P and the added mass, m_A. The physical mass is equal to the mass of the displaced water or 122,220 slugs. Assuming an added mass coefficient of 0.2, the total mass, m_T = 144,664 slugs.

For a barge exposure above water of 14 feet (d =9 feet), based on Equation V-6, the reference wind velocity, W_{eff} is 0.791 x $W(30)$. Considering, as an example, $W(30)$ = 100 mph = 146.7 ft/sec, W_{eff} = 116.0 ft/sec. The K_1 and K_2 values are 1.32 pound-sec^2/ft^2 and 873 pound-sec^2/ft^2, respectively as given in Table V-5. The non-dimensionalizing quantities are t_* = 36.7 sec, $V(\infty)$, the barge terminal velocity = 4.52 ft/sec, and $x_* =$ 165.7 ft.

The distance across the IHNC from the western floodwall to the eastern floodwall is approximately 1,100 feet. Considering that this is the trajectory of the barge, the translation distance is 1,082.5 feet (the width of IHNC minus one-half the barge width). Thus the value of x' is 6.53. Referring to Figure V-77, it is clear that the barge would have achieved its terminal velocity, $V(\infty)$ of 4.52 ft/sec. Thus the momentum and energy upon impacting the wall are:

Impact Momentum = 653,900 pound sec.

Impact Energy = 1.48 million foot pounds.

This example is provided as an illustration of the application/interpretation of the impact momentum. Consider this momentum to be transferred in, say 10 seconds allowing for barge deformation. If the form of the transfer is triangular, that is the force starts at zero, rises to twice the average value, then decreases to zero force in 10 seconds, then the maximum force acting on the flood wall would be 130,780 pounds. This is compared to the hydrostatic force of 399,000 pounds over the barge length of 200 feet. Thus, for this impact time of 10 seconds, the maximum impact force is 33% of the hydrostatic force. It is cautioned that: (1) The actual impact time would require a careful analysis of the barge and floodwall deformation characteristics and consideration of various barge orientations upon impact. Shorter impact times will result in greater maximum impact forces, and (2) The impact forces may be localized thus resulting in greater impact forces per unit length of the floodwall.

Case 2. Barge Lightly Loaded. The draft for this case is 4 feet as shown in Table V-5. As for Case 1, the total mass, m_T is the sum of the physical mass, m_P and the added mass, m_A. The physical mass is equal to the mass of the displaced water or 54,320 slugs. Again assuming an added mass coefficient of 0.2, the total mass, m_T = 65,184 slugs.

For a barge exposure above water of 19 feet (d =4 feet), based on Equation V-6, the reference wind velocity, W_{eff} is 0.826 x $W(30)$. Considering $W(30)$ = 100 mph = 146.7 ft/sec, W_{eff} = 121.2 ft/sec. Considering $C_{D,a} = C_{D,w}$ = 0.5, the K_1 and K_2 values are 1.79 pound-sec^2/ft^2 and 388 pound-sec^2/ft^2, respectively as given in Table V-5. The non-dimensionalizing quantities are t_* = 20.4 sec, $V(\infty)$, the barge terminal velocity = 8.24 ft/sec, and $x_* =$ 168.0 ft.

Considering the same barge trajectory as for Case 1, the value of x' is 6.44. As for Case 1, referring to Figure V-77 it is clear that the barge would have achieved its terminal velocity, $V(\infty)$ of 8.24 ft/sec. Thus the momentum and energy upon impacting the wall are:

Impact Momentum = 537,120 pound sec.

Impact Energy = 2.21 million foot pounds.

General Case of Arbitrary Draft

It has been demonstrated that for a reference wind speed of 100 miles per hour, the barge will reach its terminal velocity regardless of the draft and with a minimum distance of the IHNC width translation distance (minus one-half the barge width). Thus, it is possible to develop the following simple equations for impact momentum and energy for the barge of interest.

Impact Momentum. For the barge of interest and considering that the barge had reached its terminal velocity at impact, the equation for the terminal momentum can be written as

$$\text{Terminal Momentum} = 275.2\sqrt{d}\,(23-d)^{9/14}W(30) \text{ (in pound sec)}$$

Note that consistent units must be used in these equations. Thus $W(30)$ is in ft/sec.

Impact Energy. For the same considerations as above for terminal momentum, the terminal energy can be shown to be

$$\text{Terminal Energy} = 2.32(23-d)^{9/7}(W(30))^2 \text{ (in foot pounds)}$$

Plots of the impact momentum and impact energy are presented in Figure V-78.

Figure V-78 presents non-dimensional plots of terminal momentum and energy versus barge draft. For purposes here, the non-dimensional terminal momentum and velocity have been defined as the ratio of these quantities to the values for a 9 foot barge draft and for a wind speed, $W(30)$ = 144.67 ft/sec (100 miles per hour).

Thus the terminal momentum for any draft and wind speed is determined by multiplying the value for 9 feet (653,900 pound sec) by the appropriate value in Figure V-78 and the ratio of the wind speed of interest, $W(30)$ to 146.7 (all in feet/sec).

Similarly, the terminal energy is determined by multiplying the terminal energy for a draft of 9 feet (1.48 million foot pounds) by the appropriate value in Figure V-78 and the ratio of the square of the wind speed of interest , i.e., $W^2(30)$ to $(146.7)^2$ where all wind speeds are in ft/sec.

Interim Results Summary

The equations governing the effective wind speed acting on a barge present in the wind boundary layer have been examined and an effective wind speed defined for drag force calculations. Static wind forces and moments acting on a lightly loaded barge and then transferred to the east IHNC floodwall due to a wind speed of 100 miles per hour have been examined and found to represent a reasonably small fraction of the hydrostatic forces and moments exerted directly on the floodwall. These forces and moments have been expressed as averages per unit length on the floodwall although the barge related forces were likely transferred in a concentrated manner rather than in a uniform manner.

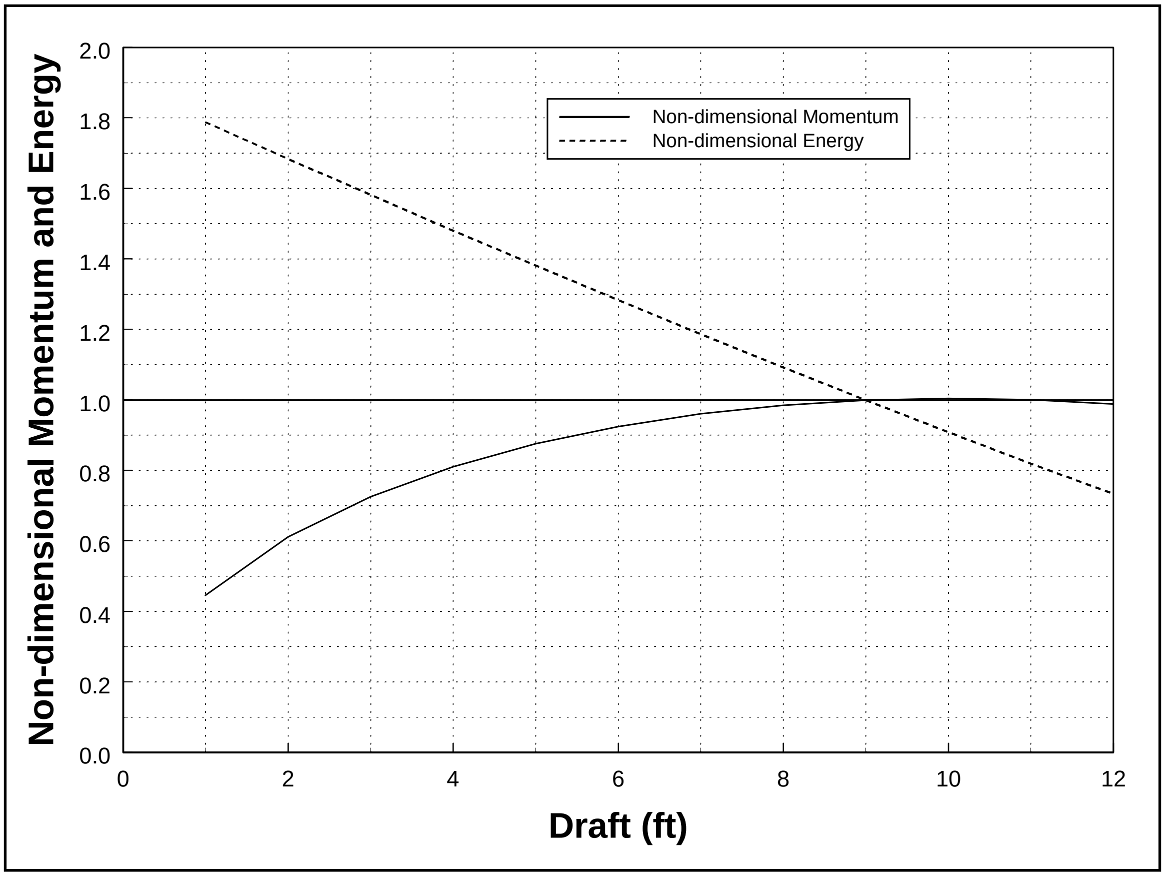

Figure V-78. Non-dimensional Barge Terminal Momentum and Energy vs Barge Draft

The equation of motion of a freely floating barge has been developed and cast in non-dimensional form for easy application. The equations include development of the terminal velocity of the barge. The equation is solved for the non-dimensional velocity and displacement.

It is found that the terminal velocity of the barge is achieved rather quickly for the wind speed examined (100 miles per hour) and that for barge conditions in the INHC the momentum and energy impact on the east flood wall depend primarily on the draft of the barge during the event. Simplified equations have been presented for terminal momentum and energy for use by others in evaluating whether the barge was a contributor to the failure of the INHC flood wall in the Lower Ninth Ward area.

Physical Model

Since the last report, the 14,000 sq ft, 1:50 scale model of the 17th St Canal has been constructed, with completion of concrete placement and molding as of 25 Feb 06. The model will be painted; gauges located, and wave generator calibration begun the following week. Data collection will begin 9 March 06. Figure V-79 shows the layout of the region modeled in the 150-ft-wide by 190-ft-long test basin.

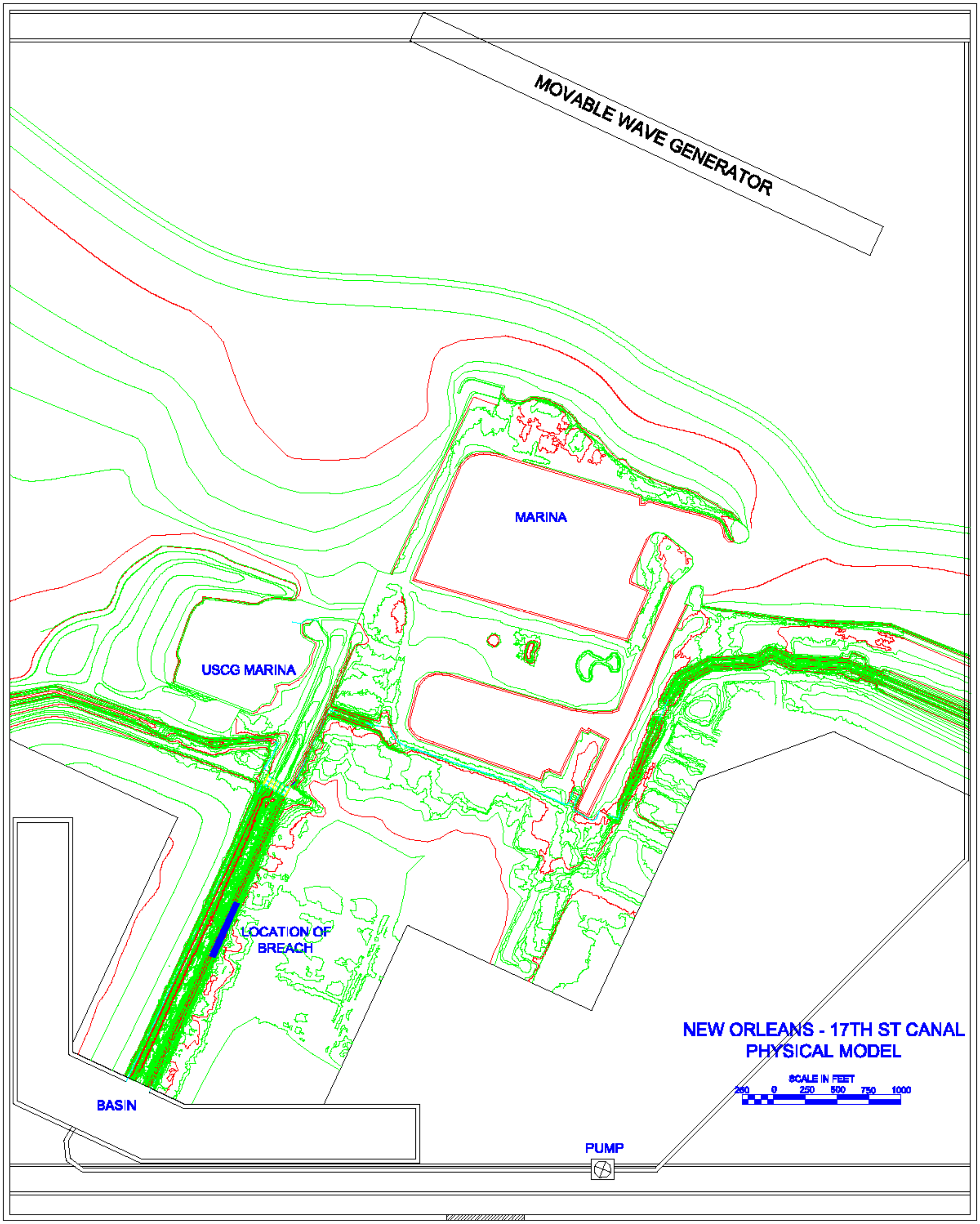

Figure V-79. Layout of 17th St Canal physical model

The physical model includes one-half mile of the 17th St Outfall Canal from the Hammond Highway Bridge to the breach area and 1,200 feet beyond the breach in accurate detail. The remainder of the surface area of the canal is included as a basin region to provide storage area for wave setup and to provide an input region for flow to simulate the pumping system flow.

Input for reproduction of waves and water level was received from Surge and Wave Model Group and Estimation of Forces on Levee Group. Figure V-80 shows the surge height, wave height, period and direction as the storm progressed through time near the 17th Street Canal. Wave information was calculated for four locations evenly spaced across the one mile of lakefront that the physical model reproduces. As can be noted in Figure V-80, the wave data for the four locations plot nearly on top of one another, indicating uniformity in wave height and direction for the 17th St region of lakefront.

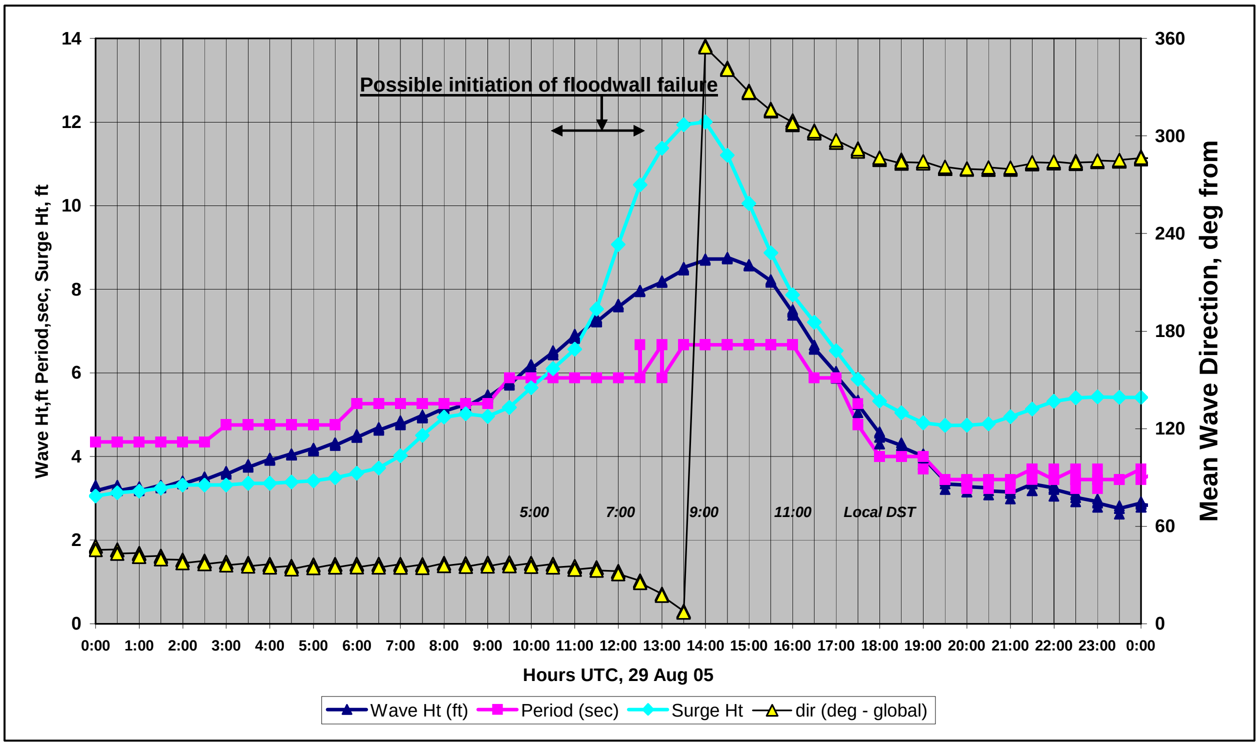

Figure V-80. Surge height and wave information at 17th Street Canal, 14-ft contour, in Lake Pontchartrain

Status of Efforts Remaining

Although it has been demonstrated that the barge terminal momentum and energy could have been considerable and thus possible contributors to the levee failure at the Lower Ninth Ward, this is not evidence that the barge <u>did</u> contribute to the failure. Thus it is recommended that other types of forensic evidence be sought including indications of whether evidence of substantial impact with the flood walls is present on the barge and as much as possible about the mooring arrangement and conditions of the mooring lines after levee failure. Other types of forensic evidence may also be available.

ADCIRC. Subsequent test simulations are planned to incorporate inflow at the pump station, flow into the canals generated by wave setup, flow out of the canals due to breaching, and the effects of bridges. The testing of bridge piers and pump station flows is underway. The conceptual development of representing breaches is underway.

Wave Modeling. Detailed time histories of wave impact on levees will be performed near the failures on the Intracoastal Waterway and the Industrial Canal. The method of simulation here will be comparable to that already performed along MRGO locations; 1HD transects will be examined. As these locations contain vertical T-walls, when the Boussinesq simulations predict strong overtopping, the simulations will be checked with a Boussinesq-RANS (Reynolds-Averaged Navier-Stokes, 2D-vertical) hybrid model, where the T-wall will be located in the RANS domain. This approach permits physically reasonable representation of the overtopping and associated forces during interaction with a vertically-walled structure.

Similar to the analysis presented for the 17th Street Canal, wave simulations will be undertaken for the entire lengths of the London Avenue Canal, Orleans Avenue Canal, and the Industrial Canal, including wave generation effects estimated via STWAVE. These simulations will be run for selected times, and the wave heights and related dynamic pressures/forces will be examined.

Status of Remaining effort for Analytical Analysis of Levee System. Analytical modeling of flow over levees, through breaches, flow in canals, runup and overtopping of levees, rubble armor stability and damage, and forces on levees and floodwalls has been conducted for some of the major features of the levee system. These modeling techniques have been discussed in preliminary reports. Detailed analysis of flow in the 17th street canal has been conducted. However, because bathymetric and topographic data have only just been received, this task has progressed at a slow rate. Bathymetric and topographic data now exist to allow detailed analysis of flow near and within the 17th Street Canal, London Canal and lakefront areas. Data are still being processed but should be ready soon for analyzing flow in the IHNC, MRGO, and lower Mississippi River areas (Plaquemines Parish). Assuming that these data are supplied in the next two weeks, the bulk of this analysis will be completed for the next 90% report

Plans for Additional Breach Flow Analysis. The final analysis of the hydraulics in the 17th Street Canal and the breach flow and geometric characteristics will be refined through: (1) Evaluation and, if necessary, modification of the time history of the water levels at the south end of the 17th Street Canal, (2) Consideration of the effective canal width as a function of water level in the canal, (3) Inclusion of the hydraulics of the bridge as appropriate (The lower member of the bridge is at an elevation of approximately +6 feet, although this elevation requires verification), and (4) Evaluating whether the inertia terms require consideration in the analysis (A preliminary assessment indicates that they are relatively small).

Planned Efforts to Investigate Breaching in London Avenue and IHNC Canals. The London Avenue and IHNC Canal breaches are considerably more complicated than that in the 17th Street Canal. Both of these canals experienced multiple breaches and considerably less data exist to support the breach analysis/ interpretation in these canals. Thus the "piecing together" of the limited information to form coherent scenarios of the timing and sequence of the various breaches will be quite difficult and will necessarily encompass greater uncertainty. It is possible that more useful information will emerge, although in view of the past thorough efforts of Task 1, it is doubtful that this will add significantly to the presently available information.

The data available for the two additional breached canals include the water level time histories in Lake Pontchartrain, eye witness accounts and limited photographic information. Most of this information was collected by Task 1. This information includes some accounts of when flooding was first observed at particular locations and the rates of water level rise at locations.

The time dependence of breaching adds complications to the analysis/ interpretation. If the breaching mechanism of canals depended only on the <u>instantaneous</u> loading, it could be argued that multiple breaches could only occur if the more distant breaches from Lake Pontchartrain occurred first because breaches at other locations would reduce the water levels in those portions of the canal more distant from Lake Pontchartrain, thereby reducing breaching potential. Because the mechanisms of breaching are time dependent, the above logic does not strictly apply; however, breaching would lower water levels at more distant locations from Lake Pontchartrain. Thus, combined with geotechnical and flooding analysis, considerations of this type may be useful in establishing breaching characteristics.

In summary, the investigation of hydraulics and breaching timing in the London Avenue and IHNC canals will be complicated by the limited data and may result in several equally plausible scenarios. However, the effort will combine all available data and will be coordinated closely with the geotechnical and flooding investigations, thus providing a basis for identifying the most probable scenarios that are consistent with all sources of reliable information.

Physical Model. Wave height and velocity data will be collected in the canal region and at various locations approaching the canal. Twenty locations can be measured simultaneously for wave height using capacitance-type wave gauges. Velocity will be measured with acoustic Doppler velocometers. Referring to Figure V-80 above, data will be collected at hours 10, 11, 12 (rising surge level), at hour 14 (peak surge level) and possibly at hours 15 and 16 (falling surge level). Focus will be on times of rising and peak surge levels, as failure of the floodwall likely occurred during this time frame. For each time, repeat tests will be run, the water level will be varied around the numerical surge prediction, and wave height varied around the numerical wave model prediction to produce a suite of wave height values.

Figure V-81 shows an example of the wave spectra providing wave input for the physical model's wave generator. In the first phase of data collection a uni-directional wave generator will be operated, with a directional wave generator coming available later, if required. Also a debris field will be created to determine wave height sensitivity to waves transmitting past the Hammond St. Bridge where photos did indicate a debris field against the bridge after the storm (see Figure V-82).

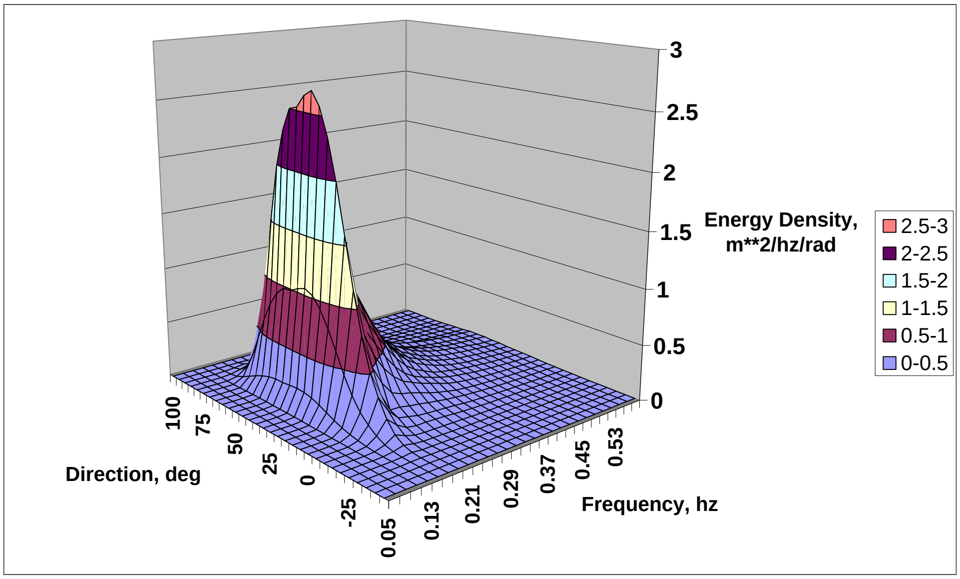

Figure V-81. Wave spectra example at hr 1000 UTC, 29 August 2006 (6.2-ft, 5.9 sec, 37-deg wave)

Figure V-83 shows the Hammond St Bridge profile and its position relative to the highest surge level. This indicates that the bridge had a blocking effect as the surge level rose above the 6-ft level. The testing will provide the effects of the bridge on wave transmission toward the breach region.

Figure V-82.   Debris field against Hammond St. Bridge

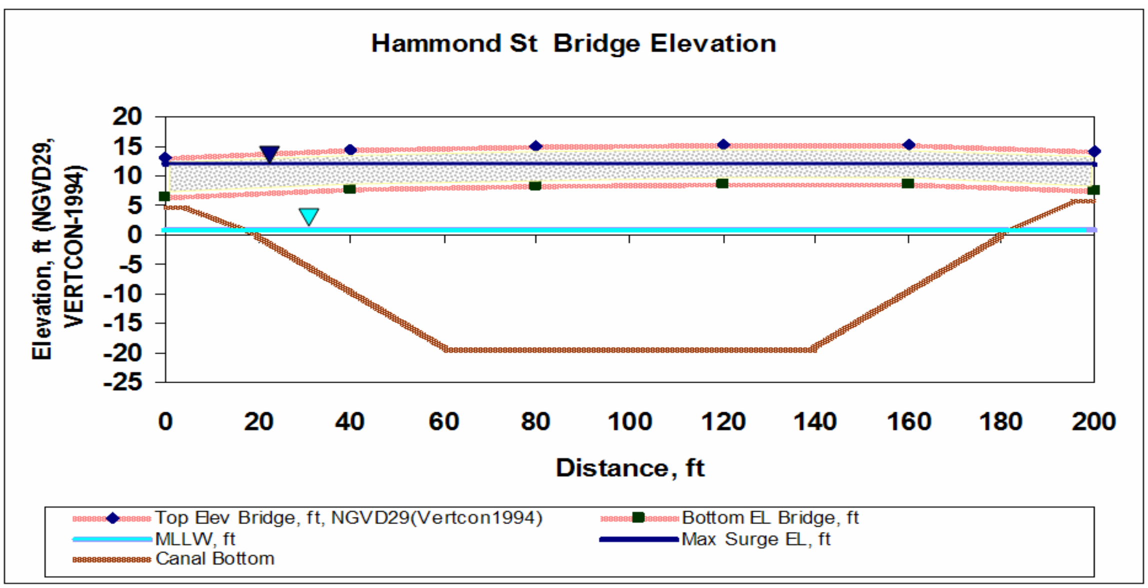

Figure V-83.   Location of maximum surge level relative to Hammond St. Bridge

VI. The Performance

Executive Summary

In this interim report, IPET is presenting a detailed assessment of the 17th Street Canal breach, and comparison with adjacent areas that did not fail. This investigation is an important step in IPET's system-wide investigation of the floodwall and levee performance, and illustrates the methods that will be applied throughout the system.

The initial data collection has been completed for 17th Street Canal, London Avenue Canal, Inner Harbor Navigation Canal, New Orleans East, Mississippi River Gulf Outlet, St. Bernard Parish, and Plaquemines Parish. The assessment of data and the investigation into the causes of the damage to floodwalls and levees are proceeding sequentially.

The investigation of the 17th Street Canal breach has revealed that it initiated about dawn on Monday, 29 August 2005, and was fully developed before 0900 CDT in the morning of the same day. Field evidence, analyses, and physical model tests show that the breach was due to instability caused by shear failure within the clay at the tip of the sheet pile, extending laterally beneath the levee, and exiting through the peat. It seems highly likely that a key factor in the failure was formation of a gap between the wall and the levee fill on the canal side of the wall, allowing water pressure to act on the wall below the surface of the levee. Another important factor was the low shear strength of the foundation clay, particularly beneath the outer parts of the levee and beyond the toe of the levee.

These two important factors in the mechanism of failure have significant system-wide implications because gap-formation mechanism and lateral variation of shear strength beneath the levee must be considered for other I-wall sections.

The damage assessment of the hurricane protection system for the New Orleans East basin reveals that the damage was due to overtopping and the accompanying erosion that occurred with the overtopping. No evidence of foundation failure mechanisms in the levees was found. Breaches of the levees were due to erosion. Damage to floodwalls was due to loss of soil support on the land side due to erosion.

In its final report, IPET will use pre-Katrina and post-Katrina LIDAR surveys to determine depth and surface area of erosion in order to categorize the

severity of the erosion and compare this with storm surge height, wave height, and their duration, along with levee surface soil type and elevation of the levee crest. This should provide an indication of why certain reaches had greater damage than other reaches.

It is important to stress that this report provides a snapshot of an ongoing effort. The information is being provided at the earliest possible time to allow broad exposure, external evaluation, and feedback and application, as appropriate. The work remaining is substantial and may result in some modifications and changes to the information presented, as well as substantial new results and findings. The information provided in this report should be considered a working draft and subject to revision prior to the completion and release of the IPET final report.

Floodwall and Levee Performance Analysis

Information regarding the performance of the floodwalls and levees making up the hurricane protective system for the New Orleans area, including St. Bernard Parish and Plaquemines Parish, during Hurricane Katrina is presented in this chapter. The focus of the effort is to assess the performance of floodwalls and levees throughout the system, investigate the most likely causes of the damage and failure of the levees and floodwalls in the system, compare the damaged components with similar sections or reaches where the performance was satisfactory, and understand the mechanisms that led to the breaches along a reaches in order to assess the potential performance of the similar un-breached reaches of the protective system.

The approach is to conduct a comprehensive assessment of the background information, examine the entire levee system to identify areas or reaches that have performed satisfactory and those that have suffered damage, characterize damage areas or reaches based on the type of damage, the surge height and the wave action, and analyze select breaches separately in detail to ensure that no important site conditions or breach mechanisms are overlooked and use this information in evaluating the system's performance.

The performance of the floodwalls and levees effort is not complete, and only interim results will be presented in this chapter. The assessment of the 17th Street Canal breach is presented to illustrate how IPET is conducting the detail investigation of the breaches and how the results are being applied to the evaluation of the rest of the system. A summary of the damage survey for the New Orleans East federal levee system is presented as an example of how the system performance information is being collected to form the basis for the system assessment.

This chapter will only summarize results obtained to date. The summary will only broadly cover the data that has been collected and evaluated and the approaches taken to produce the results presented here. Detailed descriptions of these efforts are documented in a series of reports that will be found in

Appendix K, a document which serves to provide technical support to the information presented in this chapter.

Outfall Canals

Summary of Work Accomplished

The initial data collection has been completed for 17th Street Canal, London Avenue Canal, Inner Harbor Navigation Canal, New Orleans East, Mississippi River Gulf Outlet, St. Bernard Parish, and Plaquemines Parish. The assessment of data and the investigation into the causes of the damage to floodwalls and levees are proceeding sequentially. The breaches at 17th Street Canal, London Avenue Canal, and Inner Harbor Navigation Canal are being investigated in detail in the order they are listed. Preliminary results for the 17th Street Canal are presented in this interim report. The breaches at 17th Street and London Avenue canals are being compared to Orleans Canal, which is located between the two canals, but did not seem to suffer any significant damage. It is important to understand why the I-wall sections at 17th Street and London Avenue canals failed, and Orleans Canal I-walls sections did not fail. It is important because of its implications for the performance of the I-wall sections throughout the hurricane protection system. The results reported here is IPET's initial assessment, and more work is underway to better understand the cause of the breach. The soil-structure interaction analysis and centrifuge tests are underway, and they may provide some additional information on the cause of the breach. The IPET team is continuing to investigate other possible factors that may have influenced the performance, such as wind and wave loading, seepage effects, and a loss of support due to damage to the levees from tree uprooting during the storm.

The assessment of the damage to the floodwalls and levees in New Orleans East, St. Bernard Parish, and Plaquemines Parish is proceeding. The initial assessment of damage of the New Orleans East basin is presented in this interim report.

Interim Results - Assessment of 17th Street Canal Breach

On Monday, August 29, 2005, Hurricane Katrina struck the U.S. Gulf Coast. The effects of the storm were being felt in the New Orleans area during the early morning hours. The storm produced a massive surge of water on the coastal regions that overtopped and eroded away levees and floodwalls along the lower Mississippi River in Plaquemines Parish, along the eastern side of St. Bernard Parish, along the eastern side of New Orleans East, and in locations along the Gulf Intracoastal Waterway and the Inner Harbor Navigation Canal. Surge water elevated the level of Lake Pontchartrain, and shifting storm winds forced the lake water against the levees and floodwalls along its southern shores and New Orleans outfall canals. Although most of the protection structures along Lake Pontchartrain were not overtopped, hydraulic forces caused breaches of flood-walls along 17th Street Canal and the London Avenue Canal.

Observations made at the breach at the 17th Street Canal show that the most likely cause of breach is due to a soil foundation failure. Figure VI-1 is an aerial photo showing an approximately 450-foot breach in the floodwall along the east side of the 17th Street Outfall Canal south of the old Hammond Road bridge. Figure VI-2 shows that a section of levee has moved more than 40 feet inward to the land side. It appears the remaining levee section making up the breach was washed away by the water flowing through the breach. In the photograph in Figure VI-3, the top of the I-wall section of the floodwall in the breach can be seen adjacent to the levee section that moved into the land side.

Before the construction of the emergency closure of the breach, a transverse multi-beam sonar survey of the surface of the canal bottom and breach was conducted, Figure VI-4. The survey revealed that the crest of the levee on the canal side was still present after the breach, Figure VI-5. Figure VI-6 shows a close-up of the profile at Station 11+50. It shows that the breach started at or near the floodwall and moved laterally under the land side portion of the levee at or near the elevation of the tip of the sheet pile.

Figure VI-1. Aerial photograph of the 17th Street Canal breach looking south from Old Hammond Road bridge

Figure VI-2. Aerial photograph of the 17th Street Canal breach looking north towards Old Hammond Road bridge

After the emergency closure was complete and the water levels were drawn down, large blocks of peat were found strewn in neighborhoods surrounding the breach, Figure VI-7. A close examination of the peat blocks reveals that an approximately one-foot-thick clay layer is attached to the bottom of the peat, Figure VI-8. In order to inspect the failure plane or zone, a backhoe was brought in to expose a vertical surface through the slide block that translated to the land side. The excavation uncovered a thin layer of clay, approximately one foot thick, protruding up through the peat at an angle between 20 to 30 degrees from horizontal, Figure VI-9. Samples of the peat above and below the clay layer were taken for carbon dating to assure that the peat was from the same deposit. This clay layer protruding through peat would only occur if the slide block with the clay attached to the bottom of the peat layer rode up over the intact peat during the deformation of the levee causing the breach. This implies that the failure plane of the slide block occurred through a clay layer below the peat. In order to understand how the failure mechanism may have occurred, the geology and soil stratification for the area were investigated.

Figure VI-3. Aerial photograph of the 17th Street Canal breach showing I-wall and embankment translation

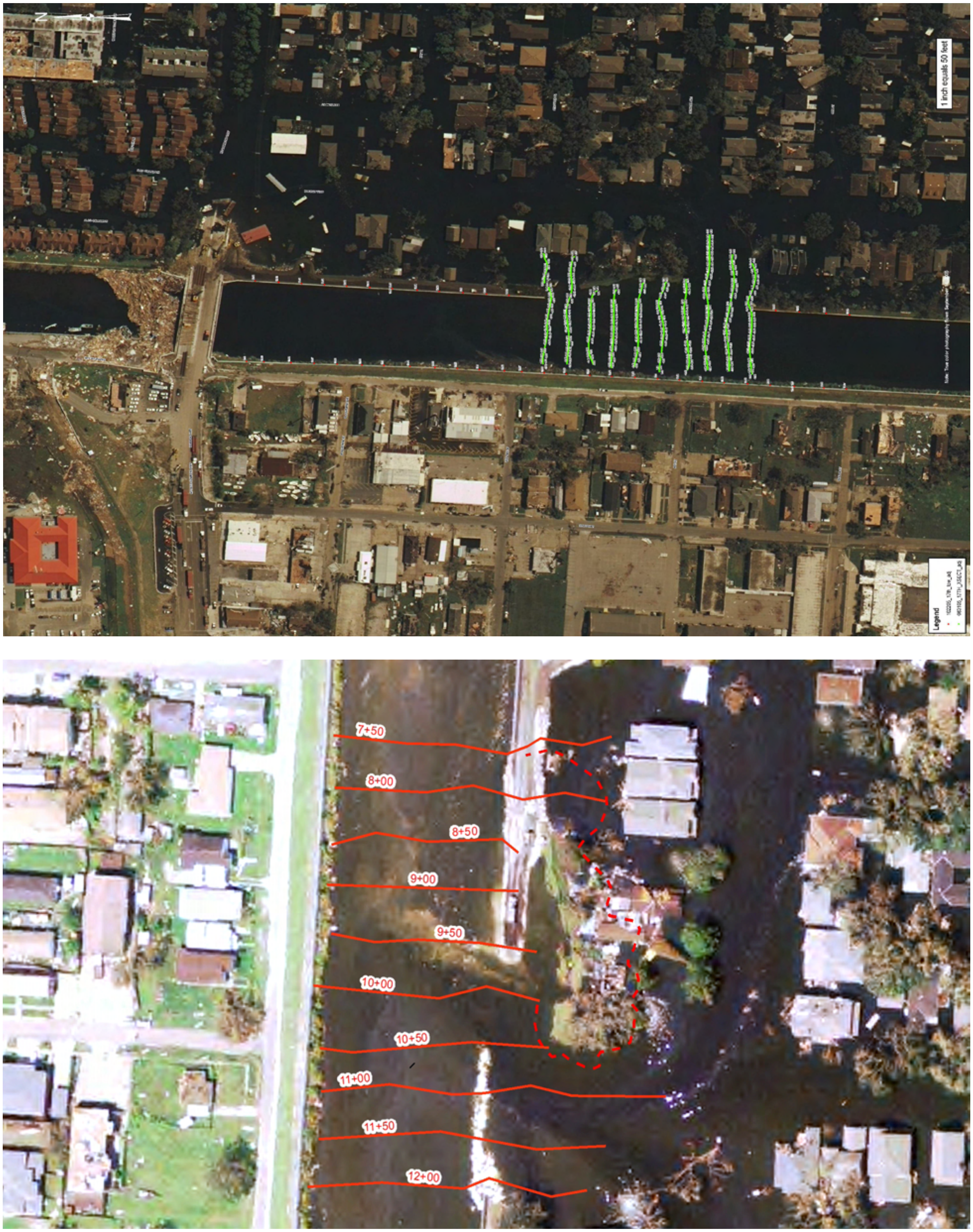

Figure VI-4. Location of multi-beam sonar survey cross-sections

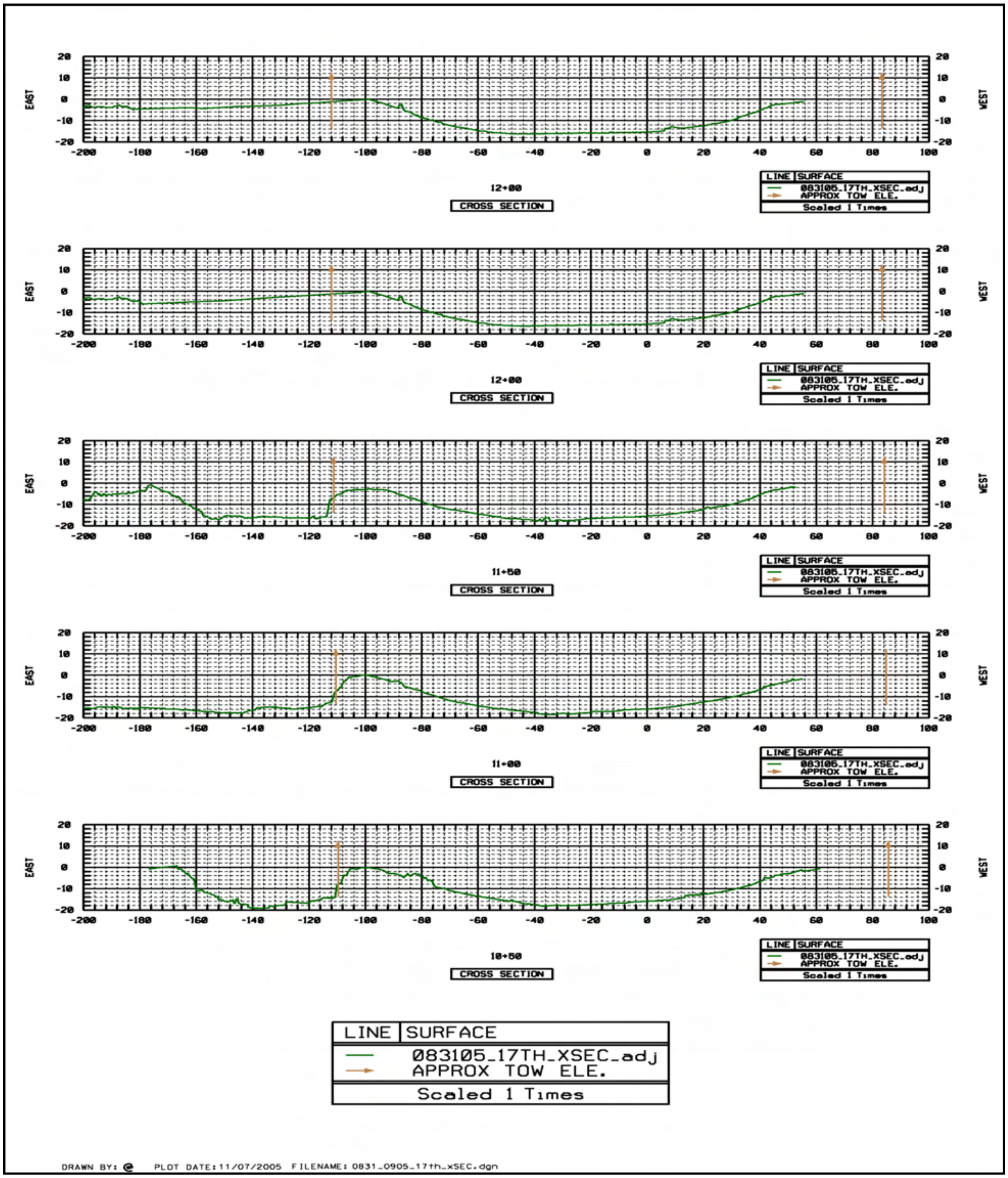

Figure VI-5a. Surface profiles at the breach

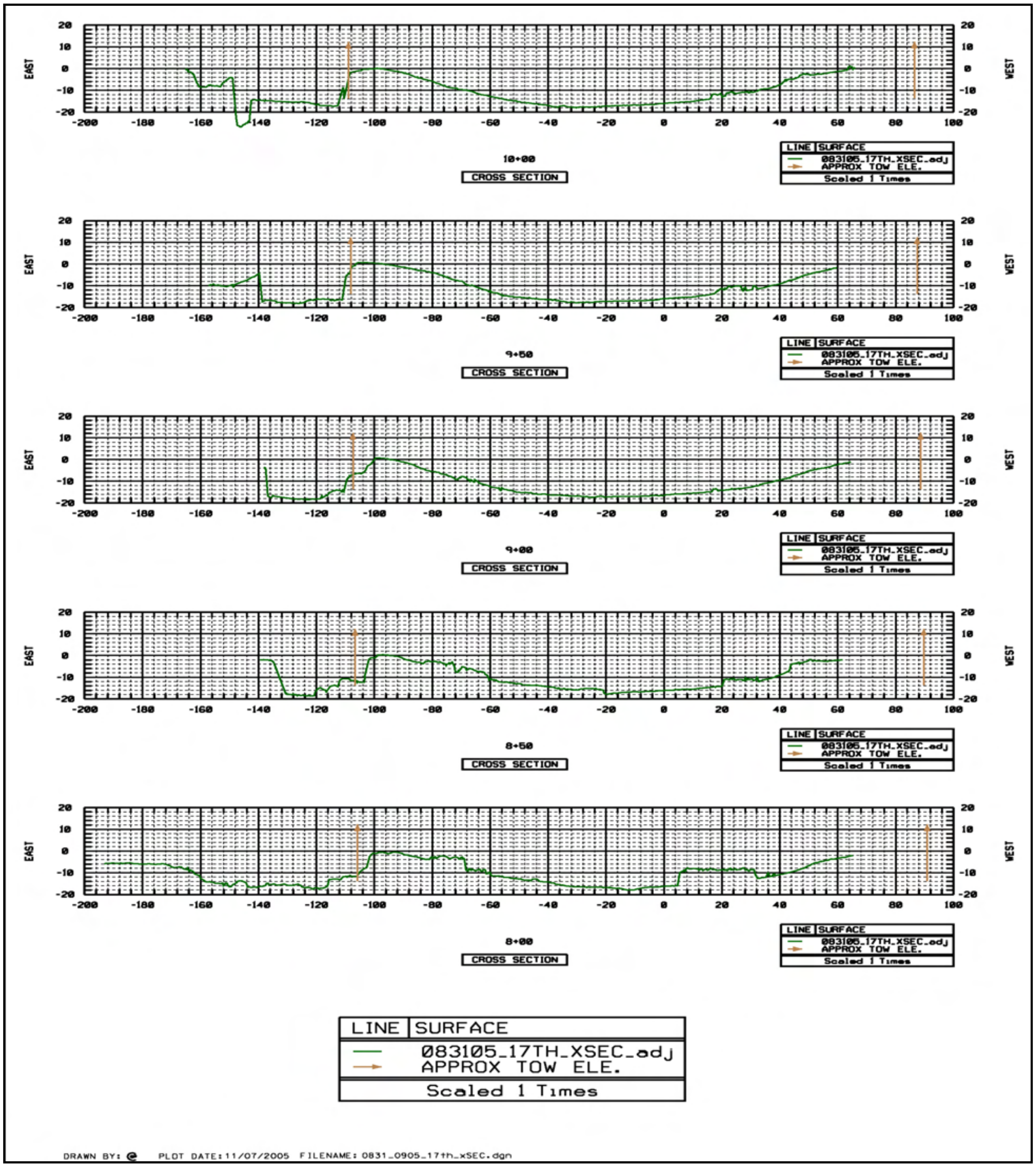

Figure VI-5b. Surface profiles at the breach

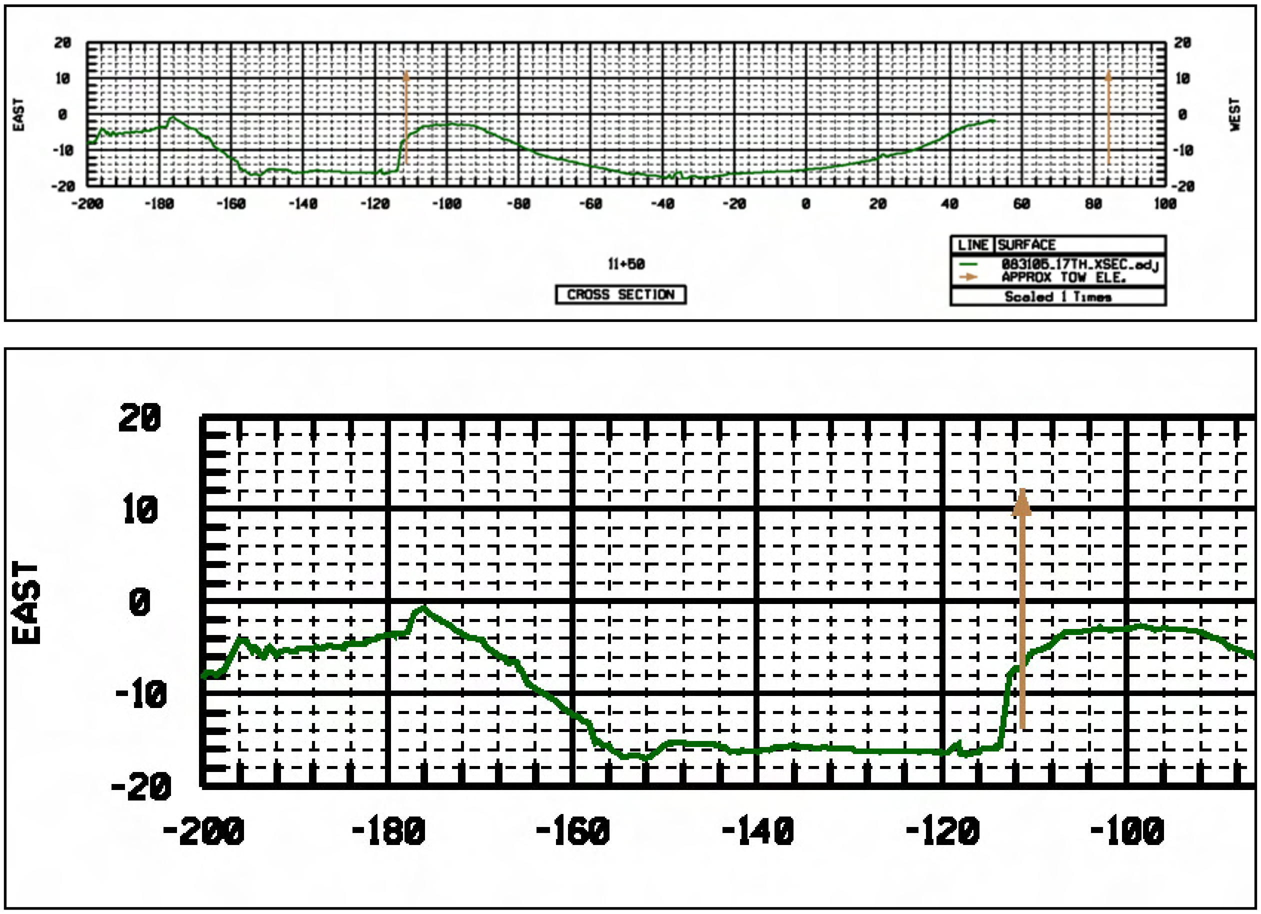

Figure VI-6. Profile for Station 11+50

Figure VI-7. Peat blocks from the levee embankment

Figure VI-8. Clay attached to peat blocks

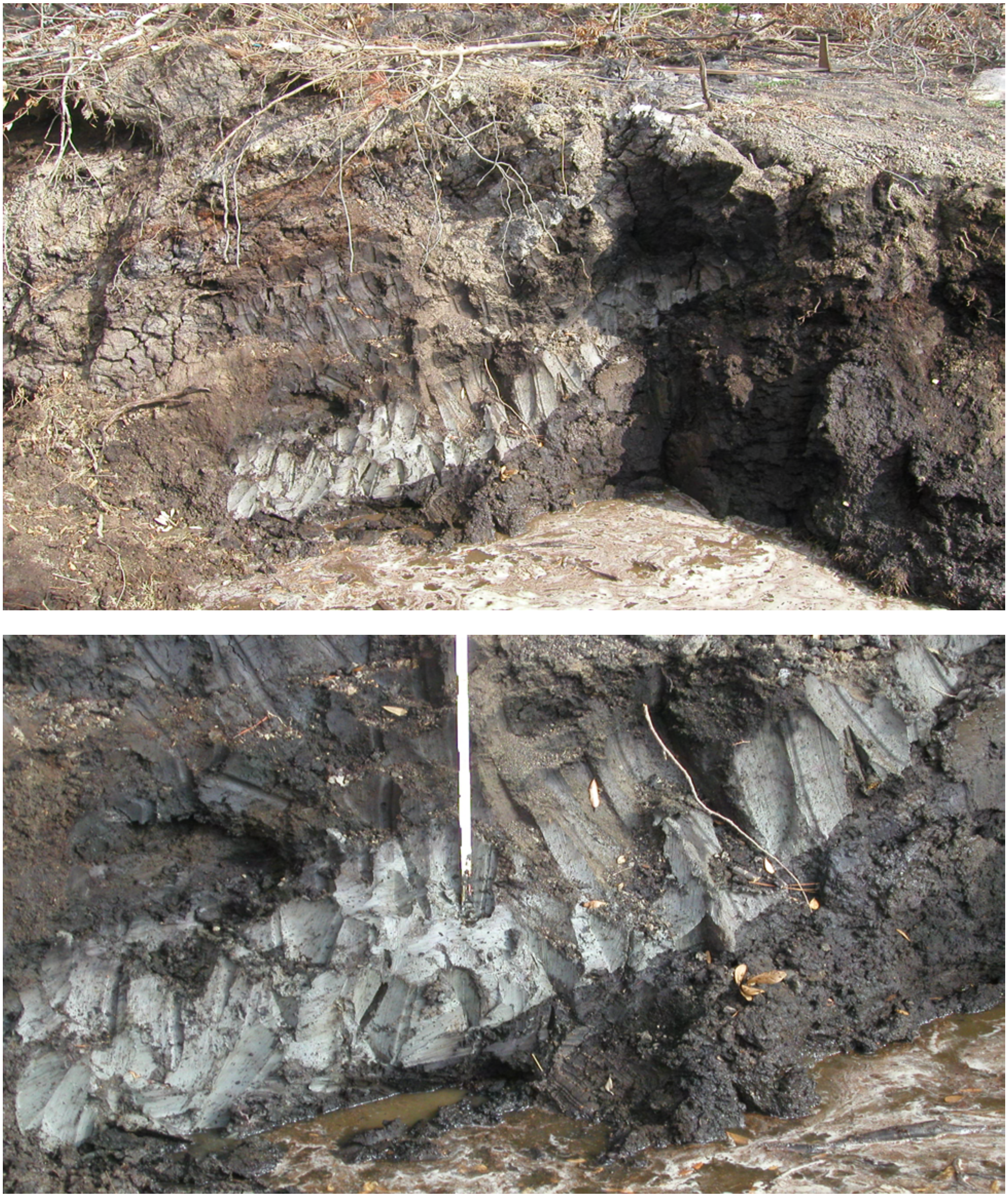

Figure VI-9. Exposed failure plane

Geology of the Area

The geology of the New Orleans area outfall canals has been determined from data collection activities at each of the breach sites by an IPET study team, from an evaluation of existing and recently drilled engineering borings at each failure area, and earlier geologic mapping studies of this area (Dunbar and others, 1994 and 1995; Dunbar, Torrey, and Wakeley, 1999; Kolb, Smith, and Silva, 1975; Kolb, 1962; Kolb and Van Lopik, 1958; and Saucier, 1963 and 1994). Geologic mapping of the surface and subsurface in the vicinity of the canal failures identifies distinct depositional environments, related to Holocene (less than 10,000 years old) sea level rise and deposition of sediment by Mississippi River distributary channels during this period. Overlying the Pliestocene surface beneath the 17th Street Canal are approximately 50 to 60 ft of shallow water, fine-grained sediments consisting of bay sound or estuarine, beach, and lacustrine deposits (Figure VI-10). Overlying this shallow water sequence are approximately 10 to 20 ft of marsh and swamp deposits that correspond to the latter stages of deltaic sedimentation as these deltaic deposits became subaerial. A buried barrier beach ridge extends in a general southwest to northeast direction in the subsurface along the southern shore of Lake Pontchartrain (Figure VI-11). A stable sea level 10 to 15 ft lower than current levels permitted sandy sediments from the Pearl River to the east to be concentrated by longshore drift, and formed a sandy spit or barrier beach complex in the New Orleans area (Saucier, 1963, 1994). As shown by Figure VI-11, the site of the levee breach at the 17th Street Canal is located on the protected or landward side of the beach ridge, while both of the London Canal breaches are located over the thickest part or axis of this barrier beach ridge complex. Foundation soils beneath the levee breaches are impacted by their proximity to the buried beach complex (Figure VI-10). Soils beneath the 17th Street area are finer-grained and much thicker in comparison to those beneath the London Canal. A complete discussion of soil types, associated engineering properties, and corresponding environments of deposition is presented in the Performance Appendix under Appendix A. Other sources of information for relationships between deltaic depositional environments, soil types, soil properties, and engineering data are presented in Kolb (1962), Montgomery (1974), or Saucier (1994).

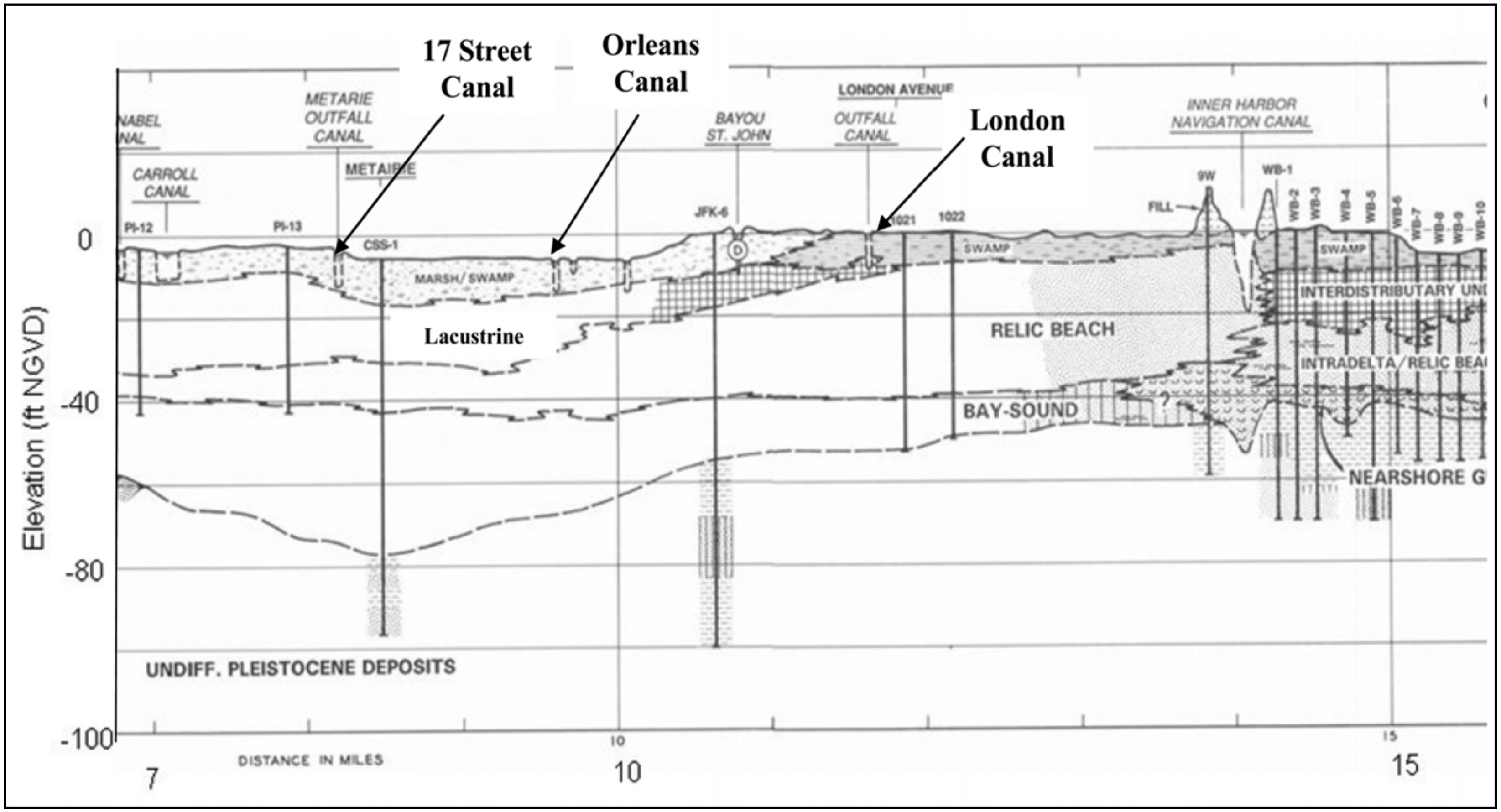

Figure VI-10. Geological cross-section extending west to east direction across western Jeffereson Parish and into eastern Orleans Parish. Section runs from near the 17th Street Canal to the Inner Harbor Navigation Canal. Major outfall canals in Orleans Parish are noted on the section. Cross-section shows the different environments of deposition in the subsurface overlying the Pleistocene (10,000 to 2 million years old) surface. Holocene (less than 10,000 years old) shallow water fill composed of between 50 to 60 feet of bay-sound (and/or estaurine), relic beach (i.e., buried Pine Island Barrier Beach complex) and lacustrine or interdistributary deposits. Shallow water environments are overlain by 10 to 20 ft of marsh and swamp deposits. Detailed explanation of environments with discussion of lithogology and engineering properties is presented in Appendix A. Cross-section modified from east half of section C –C', Spanish Fort Quadrangle (Dunbar and others, 1994). Maps and cross-sections from the New Orleans area are available at *lmvmapping.erdc.usace.army.mil*.

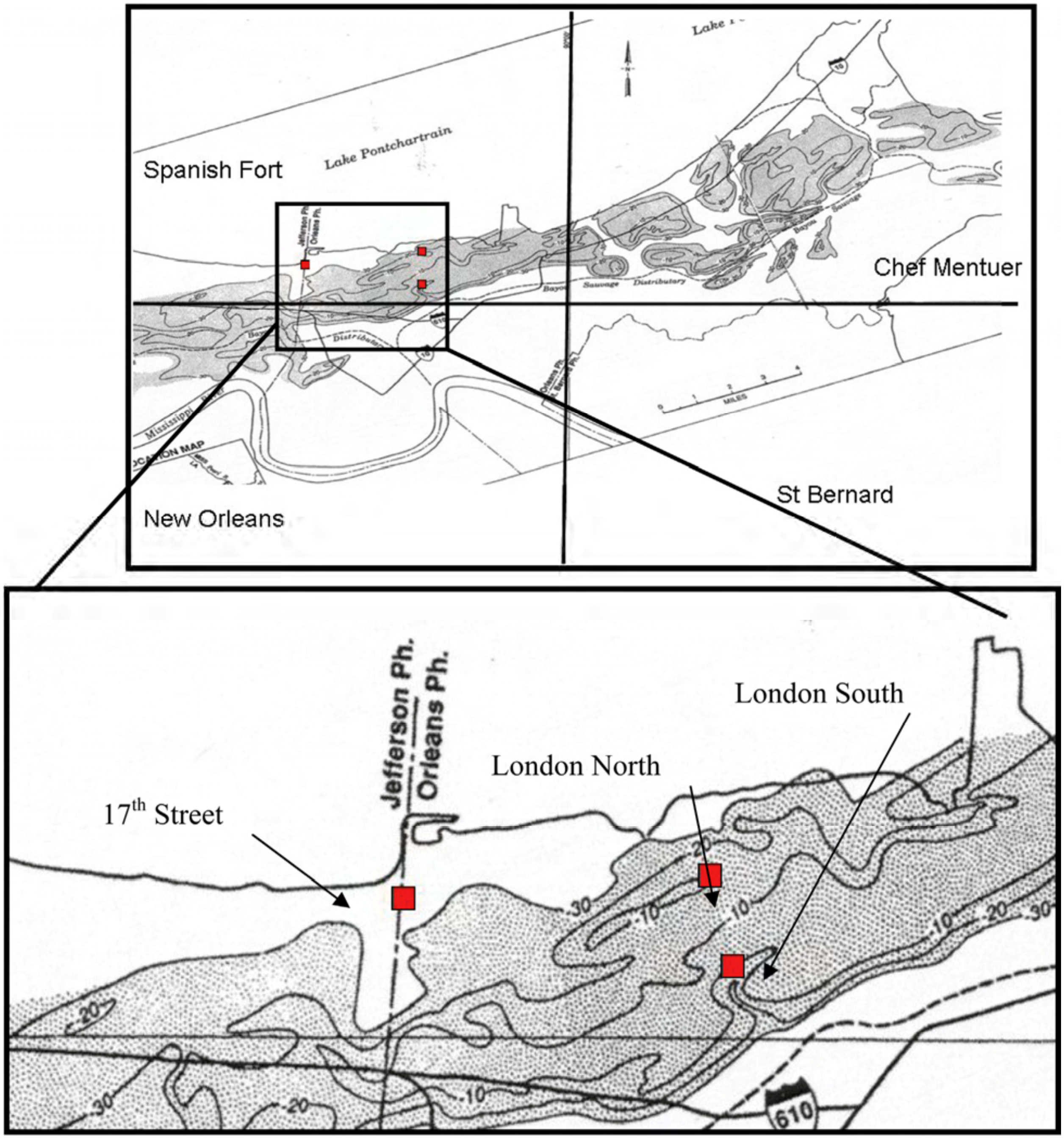

Figure VI-11. Generalized contour map showing the Pine Island Beach, contour values are in ft MSL (Saucier, 1994). Upper figure shows general trend of beach ridge in the New Orleans area, lower figure shows detailed view at the canals. London canal levee failures are located along axis of the beach. The 17th Street Canal levee break located on the protected or back barrier side of the beach ridge and consequently is dominated by fine-grained deposits corresponding to low energy depositional type settings. Extent of beach ridge shown extends across the Spanish Fort, Chef Mentuer, and New Orleans 15-min. USGS topographic quadrangles

Soil Stratification

A significant amount of information was obtained from General Design Memorandum No. 20 – 17th Street Outfall Canal – Volume 1 (GDM No. 20) in the development of pre-Katrina cross sections. Figures VI-12 and VI-13 show longitudinal profiles of the east and west bank levees of the northern half of the 17th Street Outfall canal, respectively. These figures, obtained from GDM No. 20, show boring locations and the soil types obtained during the explorations for the project upgrade. Noted on the figures is the location of breach site situated on the east bank of the canal between Stations 560+50 and 564+50. A more detailed representation of the soil stratification along the centerline in the breach area is shown in Figure VI-14. This profile was constructed using additional soil data acquired during the post-Katrina soil exploration conducted during September through October 2006. A plan view showing the locations of both old and new borings is shown in Figure VI-15. The new borings were needed because only the two old borings, B62 and B64 (reported in GDM No. 20), were in the immediate vicinity of the breach. Additionally, data from cone penetration testing, from the new exploration program, were used to supplement soil data from the old and new borings and refine the stratigraphy in the breach area. The information presented on Figure VI-15 yielded the following interpretation of the subsurface stratigraphy in the breach area. The subsurface in the breach area was simplified into six basic groups of soil types over the depth of the investigation shown in Table VI-1.

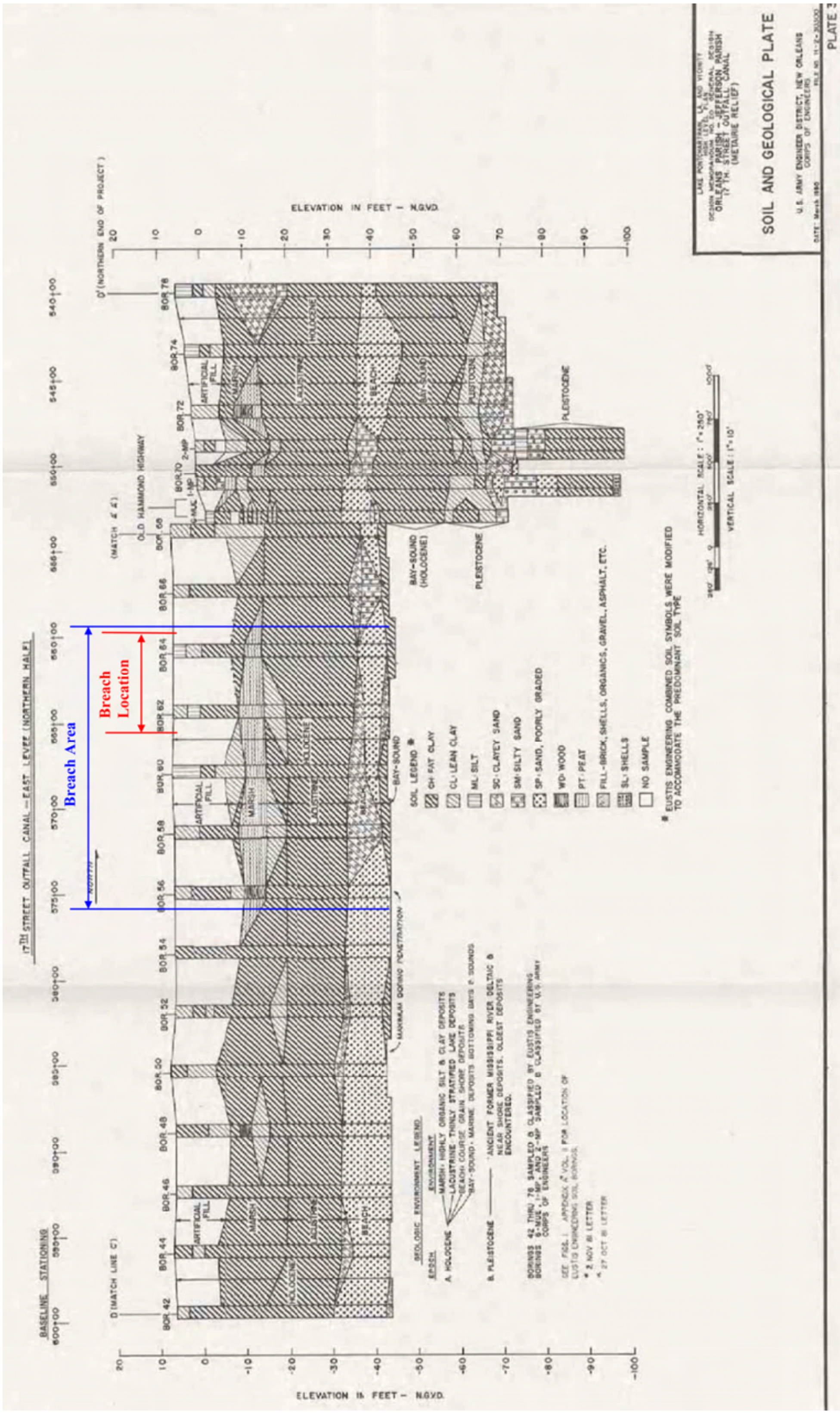

Figure VI-12. Geological Profile showing Breach Area (East Levee)

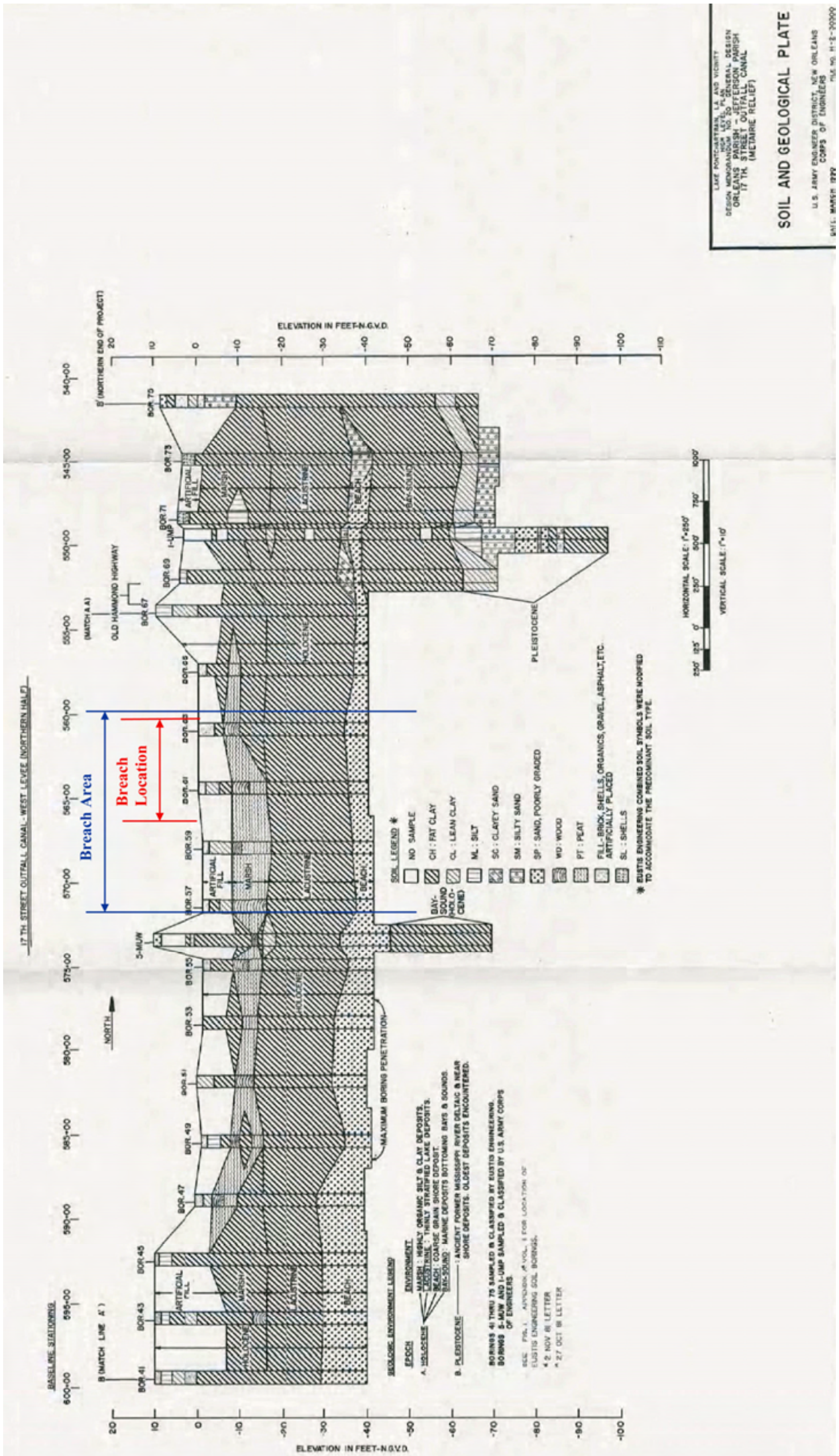

Figure VI-13. Geological Profile showing Breach Area (West Levee)

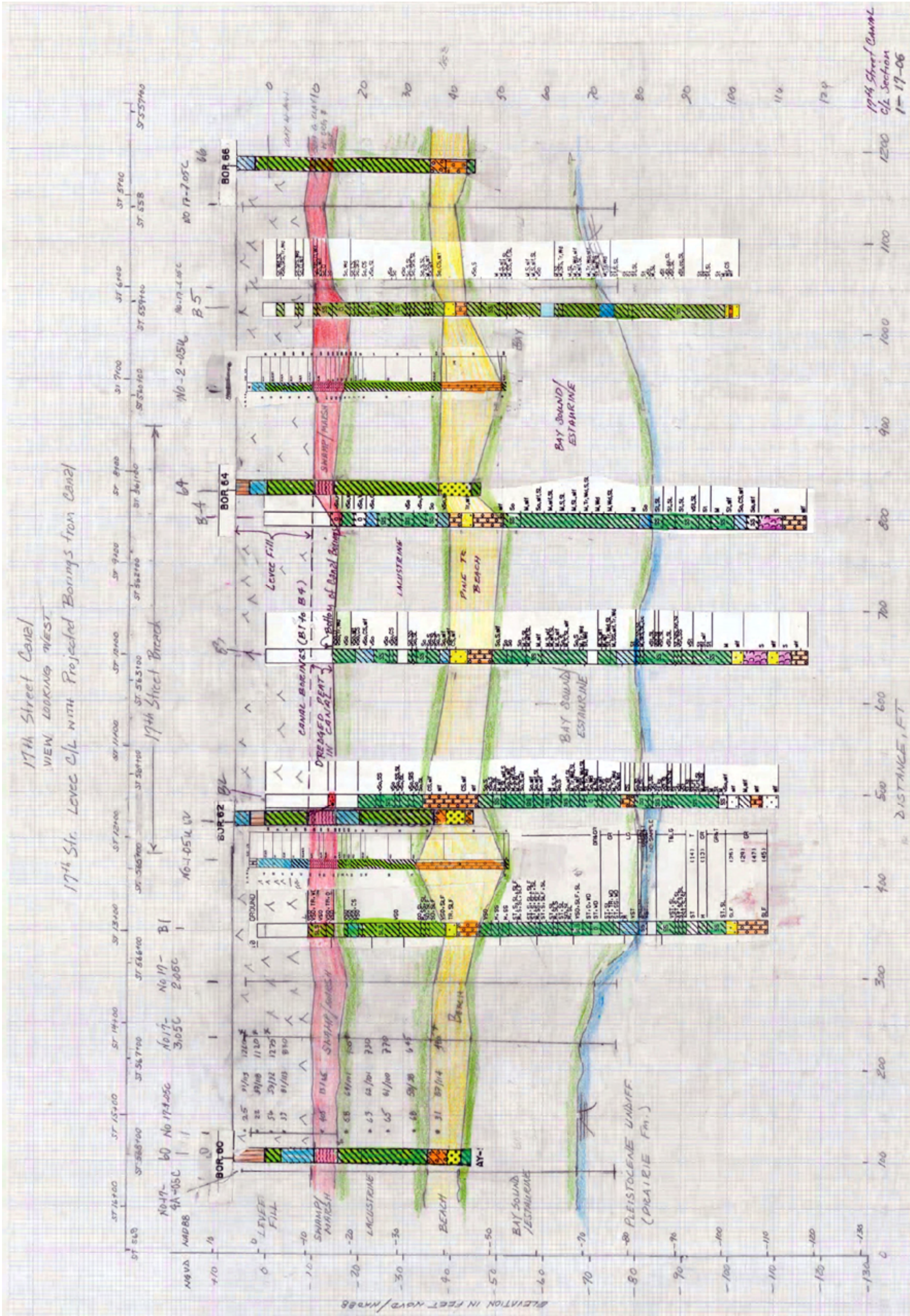

Figure VI-14. Centerline (CL) Cross-section of Breach

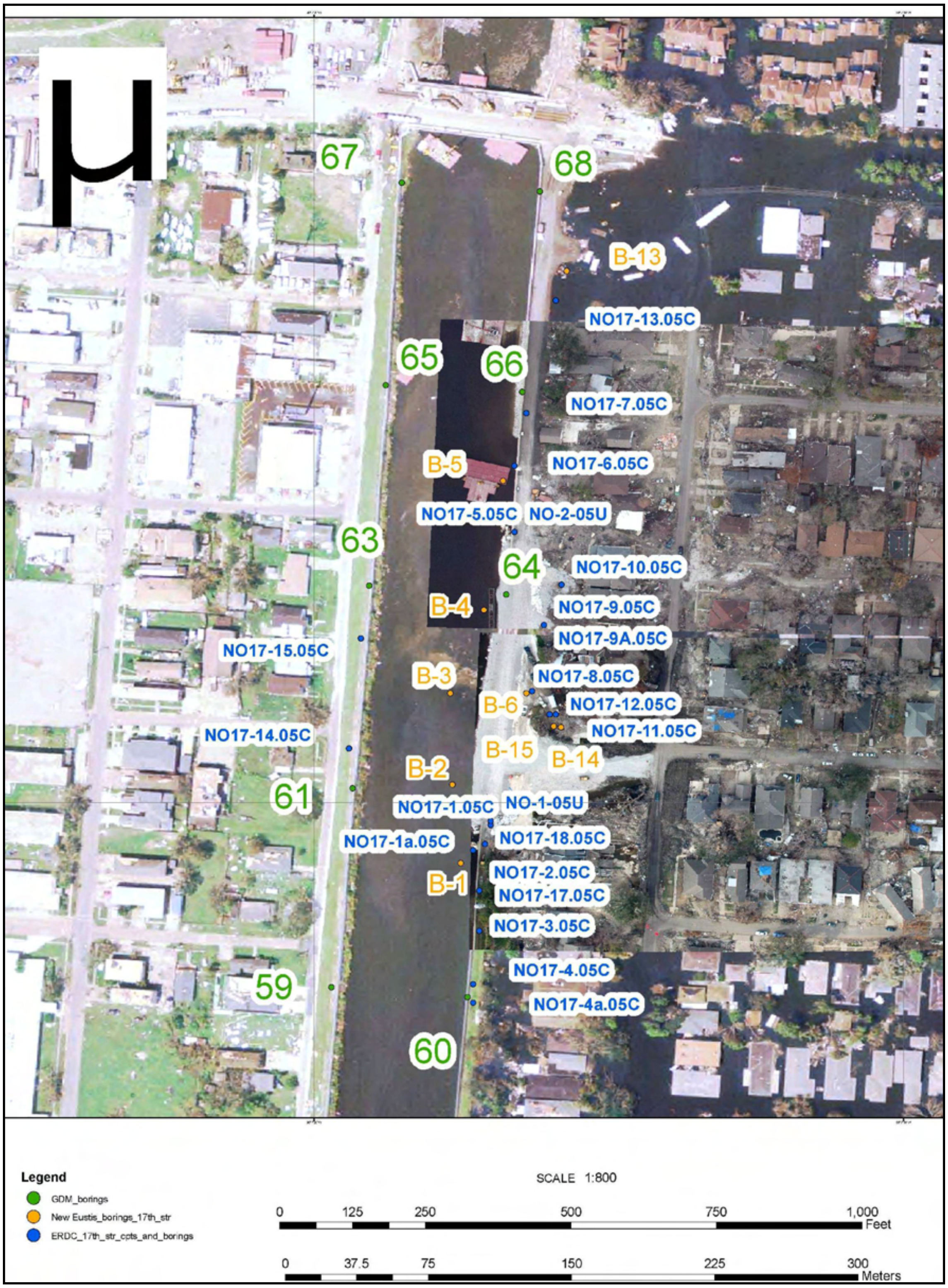

Figure VI-15. Boring and CPT Location Map

Table VI-1
Major Soil Groups at the 17th Street Outfall Canal Breach Site

Layer	Approximate Elevation of Top of Layer, ft (NGVD)	Approximate Elevation of Bottom of Layer (NGVD)	Soil Type	Consistency
Embankment	6.5	-10	Clayey (CL's and CH)	Stiff
Marsh	-10	-15	Organic/Peat	Very Soft
Lacustrine	-15	-35	Clays (CH)	Very Soft
Beach Sand	-35	-45	Sand	
Bay Sound/Estuarine	-45	-75	Clayey (CH)	Stiff to V. Stiff
Pleistocene (Undifferentiated) Prairie Formation	-75		Clays – Generally CH with some sand	Stiff

Three representative transverse cross sections through the levee breach site were prepared from the data at hand. These three sections were developed from Station 8+30, Station 10+00, and Station 11+50. Station 8+30 is the most northerly station of the three. These cross sections were prepared with the intent that they represent the conditions that existed immediately before the arrival of Katrina. Data from a pre-Katrina airborne LIDAR (Light Detection and Ranging) survey on the New Orleans Levee System that was conducted during the year 2000 were used to improve the surface topography in the breach area from that presented in the GDM No. 20 and the design documents. The LIDAR data is the best data available for establishing the cross sections before Katrina, because accurate ground survey data were not available during the preparation of this report. Unfortunately, the LIDAR system cannot penetrate through water, so it was not possible to use this technology to acquire the ground topography in the canal. A hydrographic survey was obtained immediately after Katrina, on August 31, 2006, to obtain the surface elevations of the canal between the floodwalls on the east and west banks.

The three representative cross sections for Station 8+30, Station 10+00, and Station 11+50 are shown in Figures VI-16, VI-17, and VI-18, respectively. Three sections were prepared because the levee dimensions are variable in the breach area on the east bank. Each cross section shows the conditions across the entire canal from the west bank to the east bank where the breach site is located. A degree of interpretation was necessary, particularly pertaining to the east bank protected side, to complete the cross sections because of the lack of soil boring data in this area. Thus, the marsh/peat layer was interpreted to be thinner under the centerline of the levee than at the toe due to consolidation from the surcharge caused by the weight of the levee. Also, an interpretation was made to include a 2- to 3-ft layer of topsoil over the top of the peat in this area.

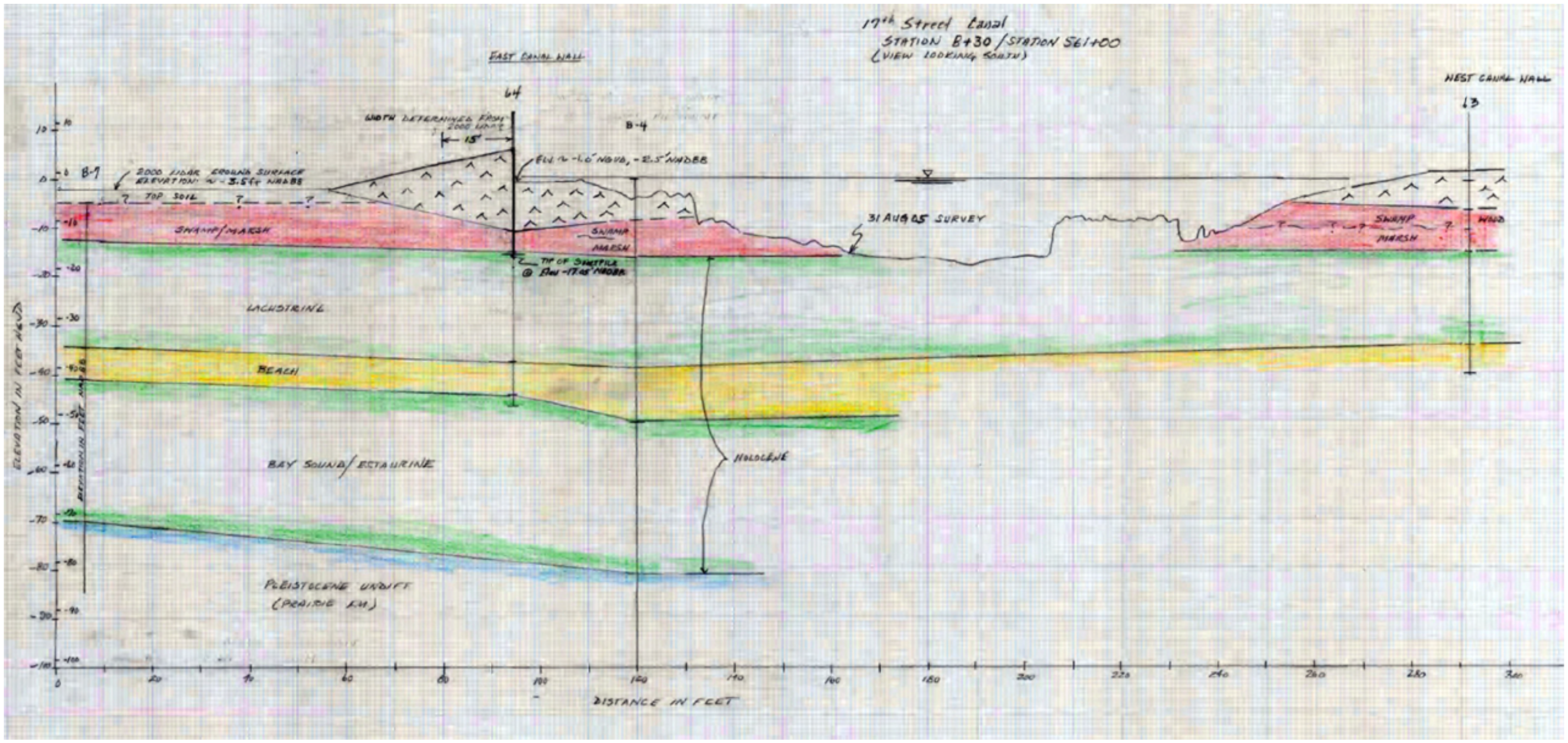

Figure VI-16. Prefailure Cross-Section at Sta 8+30 (New Stationing)/ Sta. 561+00 (GDM Stationing)

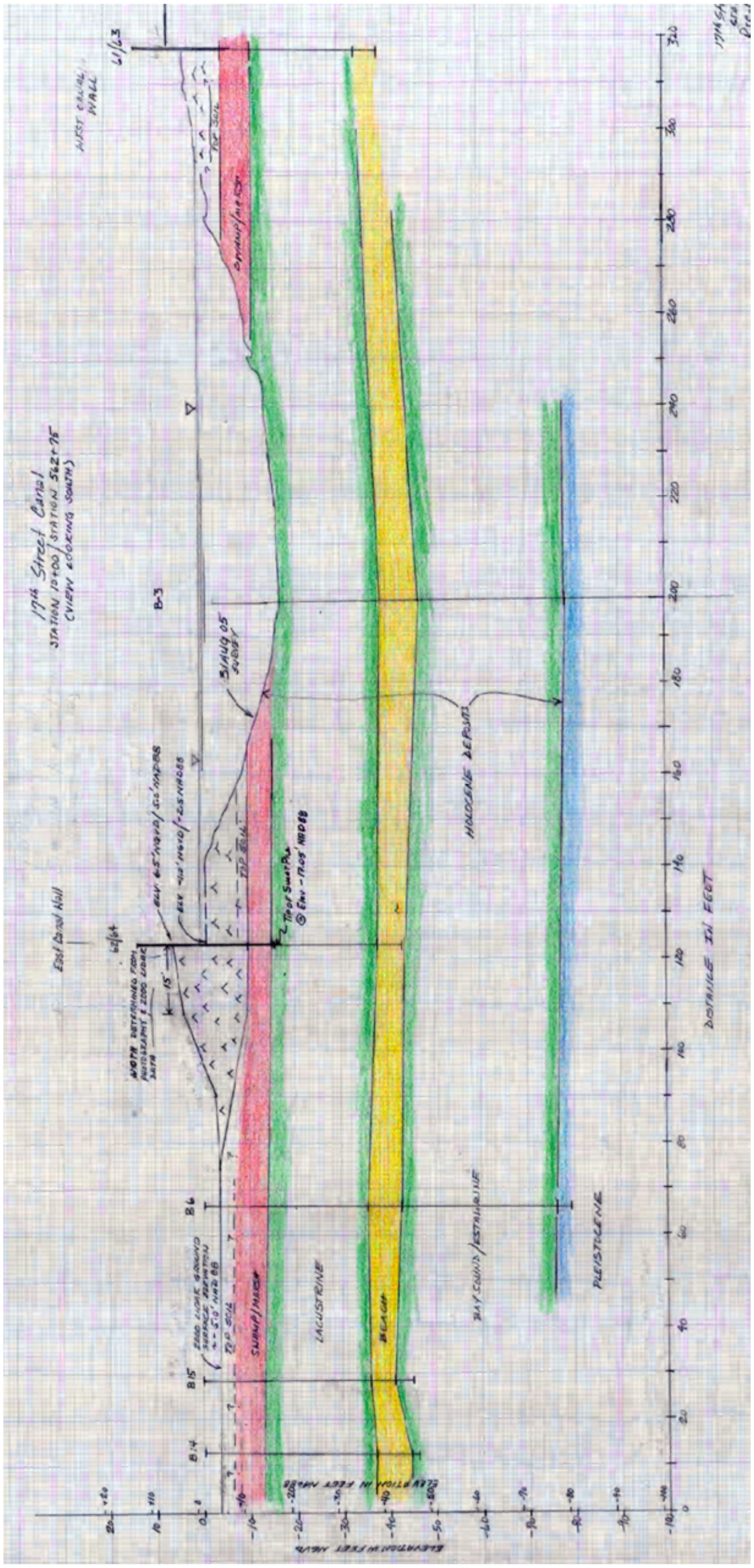

Figure VI-17. Prefailure Cross-Section at Sta 10+00 (New Stationing)/ Sta. 562+75 (GDM Stationing)

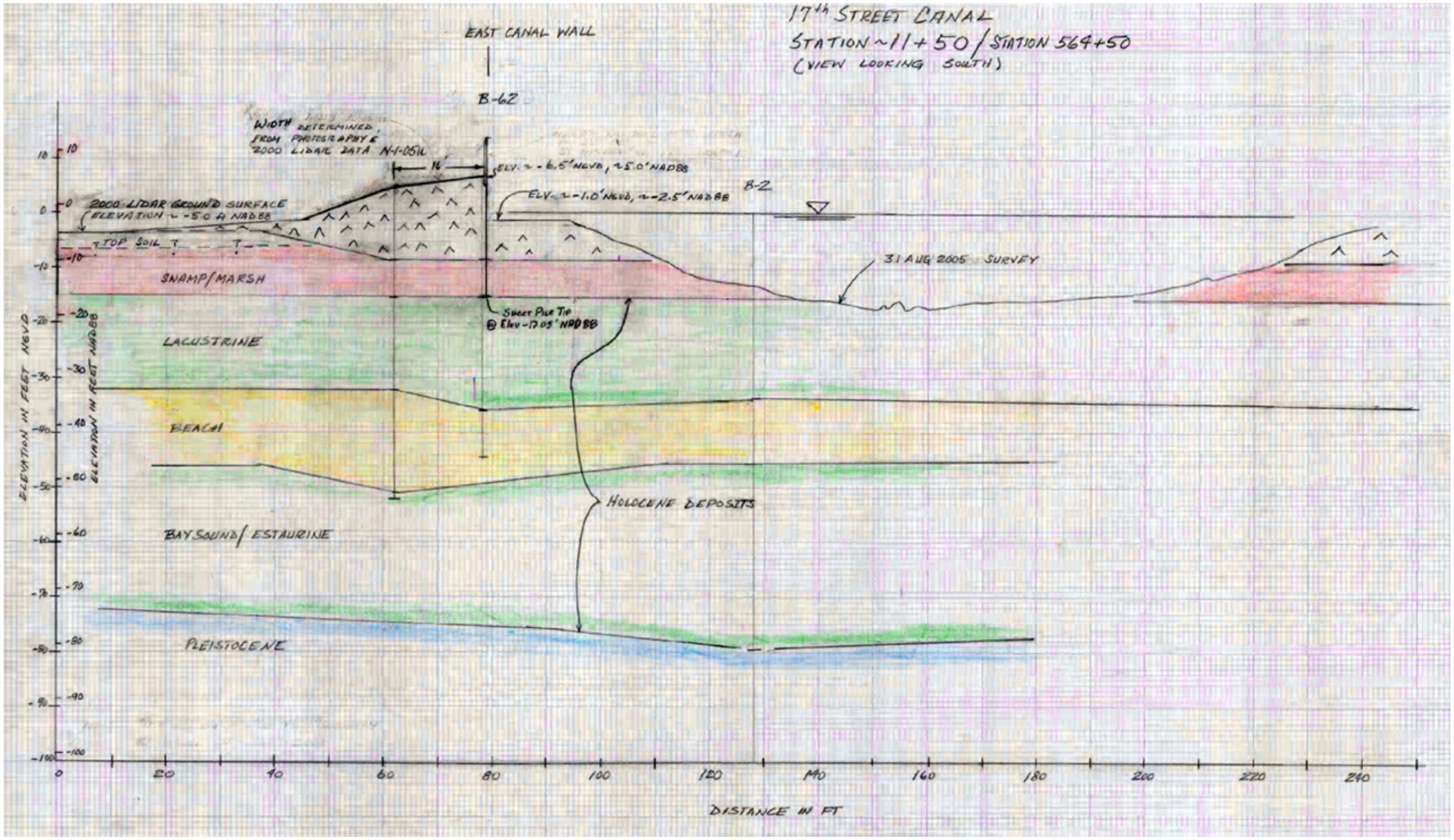

Figure VI-18. Prefailure Cross-Section at Sta 11+50 (New Stationing)/ Sta. 564+50 (GDM Stationing) (Note: Most scour occurred near this station)

I-Wall Section

Because of some discrepancies in the design documentation, the embedment length of the sheet pile wall came into question. Sheet piling from the breach area was covered up during the emergency closure so their length could not be measured. A nondestructive testing investigation was conducted using the Parallel Seismic (PS) method to try to determine the lengths of sheet piles below the concrete section of the I-walls. The PS method involves impacting the exposed portion of the foundation or substructure attached to the foundation or a location which when impacted couples sufficient energy to the pile to generate a sound or stress wave which travels down the foundation, Figure VI-19. The wave energy is tracked by a hydrophone receiver suspended in a water-filled, cased and sometimes grouted borehole drilled typically within 3-5 feet of the foundation edge. The PS tests typically involve lowering the hydrophone to the bottom of the boreholes, impacting the exposed portion of the foundation structure, and recording the hydrophone responses. Then the hydrophone receiver is raised to the next test elevation. This test sequence is repeated until the top of the casing or the top of the water level in the casing is reached. The pile depth is determined by plotting the hydrophone response from all depths on a single plot. For soils of constant velocity surrounding the piles, a break in the slope of the line occurs below the bottom of the piles indicating the pile depth. For soils with varying velocities, a break often cannot be identified from the slope of the lines, but the bottom of the piles can be identified by observing the traces of the hydrophones' plot to identify changes in the response, such as a reduction in signal amplitude, change in signal frequency, or diffraction/reflection of the tube wave energy from the foundation bottom.

The PS method investigation was performed on 27-28 October 2005. The three levee locations were tested at 17th Street Canal near the breach area. These initial measurements indicated sheet pile lengths of approximately 15 feet below the crest of the levee. This length is 7 feet shorter than the final Plans and Specifications called for. To clear up this discrepancy, sheet piles were recovered north and south adjacent to the breach area on 12-13 December 2005. The lengths of the sheet piles recovered were at the length, approximately 23.5 feet (22 feet below the crest of the levee), which was specified in the Plans and Specifications for the construction.

This raised the question of why did the PS method and a similar method, Seismic Cone Penetrometer Tool (SCPT) used by the Louisiana State Investigation Team, incorrectly indicate the sheet pile length. A review found that the error was not due to problems with the actual test method, both rather were due to misinterpretation of the data. The primary problems involved in the interpretation of the data were:

1. The apparent ground and tube vibrations showed slower velocity and a weaker signal at the incorrectly predicted 7-foot short sheet pile depths. This may be due to strong energy emitting from the concrete walls in the ground or the change in soil velocity at the interface between the levee material and the saturated peat layer, which was approximately at the depth of the incorrectly predicted sheet pile depths.

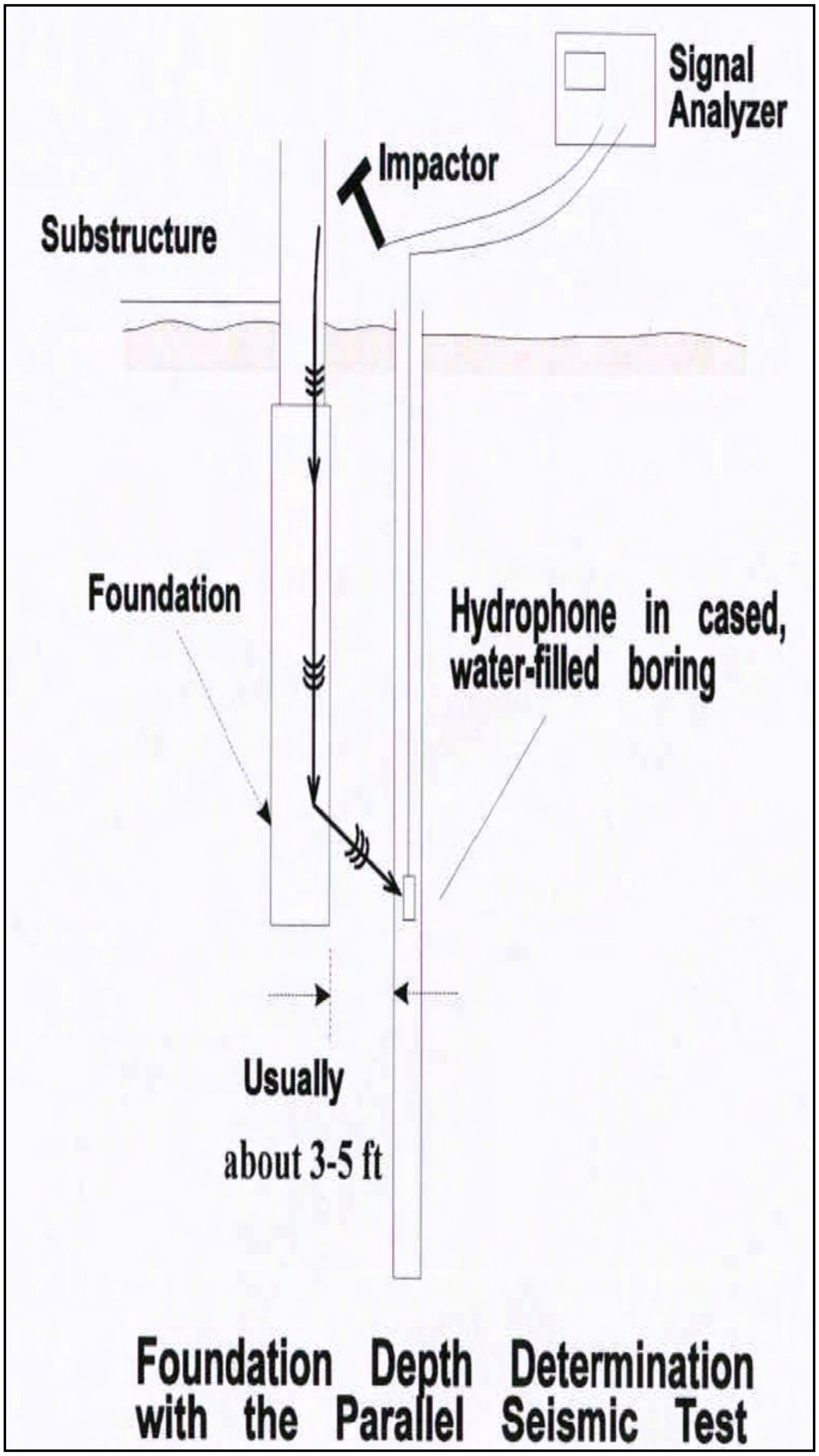

Figure VI-19. Parallel Seismic test setup

2. Lack of experience of interpretation of the hydrophone response of the sheet pile walls diffraction events due to spreading out of the energy for a wall shaped foundation.

3. Lack of data available to clearly identify the weak diffraction of the wave energy emitting from the sheet pile tips because the borehole casings extended only a few feet below the actual sheet pile tip depth.

An additional nondestructive investigation using both the PS method and SCPT was conducted on 21-22 December 2005 at the south end of the 17th Street Canal breach and at the Inner Harbor Navigational Canal (IHNC). The re-tests at the south end of the 17th Street Canal breach resulted in a clearer identification of the sheetpile tips in the hydrophone response because of a casing that was deeper and closer to the sheet piling and a stronger signal produced by varying the impact locations. The re-test at IHNC was less successful. No clear diffraction arrival events at the sheet pile tips were found. The lack of the diffraction arrival events may be due to apparent lack of tight contact between the sheet piles and the surrounding soil. A clear separation of the soil and wall was still evident at the ground surface. For more details on the sheet pile depths and the concrete and sheep pile material tests, see Appendix K.

Assessment of Soil Properties and Shear Strengths

A considerable number of borings were drilled in the breach area and in neighboring areas before the failure. Additional borings have been drilled, cone penetration tests have been performed, and test pits have been excavated since the failure. Several hundred UC tests and UU tests have been conducted on the soils at the site. A summary of these are presented in Appendix K, K-1.

Shear Strengths of Levee and Foundation

The data available from previous and new studies in the 17th Street Canal area were analyzed to develop a shear strength model, called here the "IPET" strength model, for use in analyzing the stability of the I-wall in the breach and adjacent areas. The shear strength evaluation focused on (1) the levee fill, (2) the peat (or marsh) layer beneath the levee, and (3) the clay (or lacustrine) layer beneath the peat.

The levee fill is compacted CL or CH material, with an average Liquid Limit of about 45. Beneath the fill is a layer of peat or "marsh" 5 ft to 10 ft thick. The peat is composed of organic material from the cypress swamp that occupied the area, together with silt and clay deposited in the marsh. The average moist unit weight of the peat is about 80 pcf. Beneath the peat is a clay or "lacustrine" layer, with an average Liquid Limit of about 95%. The clay is normally consolidated throughout its depth, having been covered and kept wet by the overlying layer of peat.

Sources of Information on Shear Strengths

A considerable number of borings were drilled in the breach area and in neighboring areas before the failure. Additional borings have been drilled, cone penetration tests have been performed, and test pits have been excavated since the failure. The IPET strength model derived from the results of these tests was developed to characterize as accurately as possible the undrained shear strengths of the levee fill, the peat, and the clay.

The IPET Shear Strength Model

The measured shear strengths of the levee fill scatter very widely, from about 120 psf to more than 5,000 psf. Placing greatest emphasis on data from UU tests on 5-inch-diameter samples, which appear to be the best-quality data available, s_u = 900 psf is a reasonable value to represent the levee fill. This strength can be compared to a value of 500 psf for the levee fill used in the design analyses. The peat (or marsh) deposit is stronger beneath the levee crest where it had been consolidated under the weight of the levee, and weaker at the toe of the levee and beyond, where it has not been compressed. The measured shear strengths of the peat scatter very widely, from about 50 psf to about 920 psf. Values of s_u = 400 psf beneath the levee crest, and s_u = 300 psf beneath the levee toe appear to be representative of the measured values. These strengths can be compared to a value of 280 psf used in the design analyses.

The clay (which is the most important material with respect to stability of the I-wall and levee) is normally consolidated. Its undrained shear strength increases with depth at a rate of 11 psf per foot of depth. This rate of increase of strength with depth corresponds to a value of $s_u /p' = 0.24$. There is very little scatter in the results of the CPTU tests, and these values provide a good basis for establishing undrained strength profiles in the clay. The undrained strength at the top of the clay is equal to 0.24 times the effective overburden pressure at the top of the clay, and the undrained strength increases with depth in the clay at a rate of 11 psf per foot. With this model, the undrained shear strength of the clay varies with lateral position, being greatest beneath the levee crest where the effective overburden pressure is greatest, and varying with depth, increasing at a rate of 11 psf per foot at all locations.

Comparison of IPET Strengths with Strengths Used in Design

The design analyses used undrained strengths for the levee fill, the peat, and the clay, and a drained friction angle to characterize the strength of the sand layer beneath the clay, as does the strength model described above. Thus, the strengths are directly comparable. Strengths from the IPET strength model are compared to the design strengths in Table VI-2:

Table VI-2
Comparison of Strengths of the Levee and Peat Used in the Design with the IPET Strength Model

Material	Strength Uses for Design	Strength Model Based on all Data Available in February 2006
Levee fill	s_u = 500 psf, ϕ = 0	s_u = 900 psf, ϕ = 0
Peat	s_u = 280 psf, ϕ = 0	s_u = 400 psf, ϕ = 0 beneath levee crest s_u = 300 psf, ϕ = 0 beneath levee toe

It can be seen that the strengths for the levee fill and the peat used in design are consistently lower that those for the IPET strength model, which were estimated using all of the data available in February 2006.

The values of strength for the clay vary with depth and laterally, as discussed above. The rate of increase of strength with depth (11 psf per foot in the IPET strength model) are essentially the same in the strength model as for the design strengths. Beneath the levee crest, the design strengths are very close to the IPET strength model. At the toe of the levee, however, the strengths used in design are considerably higher than the strengths from the IPET strength model.

Comparison of Strengths within the Breach Area with Strengths Elsewhere

Field observations and preliminary analyses show that the most important shear strength is the undrained strength of the clay. Critical slip surfaces intersect only small sections within the peat and the levee fill, and do not intersect the sand layer beneath the clay at all. Therefore the strengths of these materials have small influence on stability, and minor variations in these strengths from section to section would not control the location of the failure. For this reason, the comparison of strengths in the breach area with strengths elsewhere has been focused on the undrained strength of the clay.

Although the data is sparse, it is fairly consistent, and it appears that the clay strengths in the areas north and south of the breach are higher than those in the breach. Based on data available for comparison, the undrained strengths of the clay in the areas adjacent to the breach are 20% to 30% higher than those in the breach area. Strength differences of this magnitude are significant. They indicate that the reason the failure occurred where it did is very likely that the clay strengths in that area were lower than in adjacent areas to the north and south.

A more complete description of the IPET strength model and the tests that support it is contained in Appendix K1.

Future Soil Data Gathering

The soil properties (shear strengths, consolidations, moisture contents, grain size analysis, etc) obtained from the General Design Memorandum (GDM) has been complied for the entire 17th St. Canal, Orleans Canal, and London Canal, and Inner Harbor Navigation Canal (IHNC). In addition, the data from soil borings under the direction of the New Orleans District performed after Hurricane Katrina at the 17th St. Canal, London Ave. Canal, and IHNC was obtained. Laboratory testing of samples from these borings is complete and includes unconfined compression tests, Q tests (unconsolidated – undrained triaxial tests), one-point Q tests (unconsolidated – undrained triaxial test on one sample at existing confining pressure), Atterberg Limits, moisture contents, and grain-size analysis.

In September and October 2005, the IPET Team performed soil borings and cone penetrometer tests at the breach areas on 17th St. Canal, London Ave. Canal, and IHNC. Standard laboratory testing of samples taken by the IPET Team are almost complete and includes: unconfined compression tests, Q tests, Atterberg Limits, moisture contents, organic contents, R-bar tests (consolidated-undrained triaxial tests with pore pressure measurements), consolidation tests, and grain size analysis. The intent of the laboratory testing of these samples is to verify the data obtained from the GDM and the post-Katrina borings by the New Orleans District. Direct simple shear are planned to obtain additional strength data for the clay layers. Field vane shear tests and additional cone penetrometer tests are also planned to obtain additional strength data in the breach area at the levee centerline and at the levee toe.

Limit Equilibrium Analyses of 17th Street Canal Breach

Limit equilibrium analyses are used to examine stability of the levees and I-wall section of the floodwall, and to examine possible mechanisms of failure at each breach site. The results of these analyses are interpreted in terms of factors of safety and probabilities of failure. This interim report will examine what the factors of safety are for the 17th Street Canal levee and I-wall section based on the IPET shear strength model described in earlier sections of this report, and how the factors of safety vary with water level in the canal. The results reported here is IPET's initial assessment, and more work is underway to better understand the cause of the breach.

Stability Analyses

Stability analyses were performed for three cross sections within the breach area (Stations 8+30, 10+00, and 11+50) using the IPET shear strength model. The results of these analyses were compared with the results of the analyses on which the design of the I-wall was based, and additional analyses were performed for the design cross-section geometry and shear strengths, using Spencer's method and the computer program, SLIDE.

It was found that

- The calculated factors of safety decreased as the elevation of the assumed water level increased, and
- Smaller factors of safety were calculated when it was assumed that a gap (or crack) existed between the wall and the soil on the canal side of the wall,

and that hydrostatic water pressures acted within this crack, increasing the load on the wall.

It seems likely that such a crack, or separation, between the wall and the levee fill formed as the water level rose, causing the wall to deflect away from the canal, and that this was a significant factor in the failure.

The results of the analyses are reasonably consistent with the performance of the I-wall in the breach area. Calculated water levels for factors of safety equal to 1.0 for the cracked condition vary from 11.3 ft to 12.1 ft NGVD, as compared with a water level of 7.5 ft to 9.5 ft at the time failure began based on an eye-witness report. It appears that wave effects might raise the effective water level by 1 to 2 feet, to as much as 11.5 ft. This would reduce the difference between calculated and observed water levels to cause failure to one to two feet. This may indicate that the IPET shear strengths are a little higher than the actual shear strengths.

The difference between calculated and observed water levels causing failure could also be due to the fact that, so far, the stability analyses have only considered circular slip surfaces. Further analyses will be performed using noncircular slip surfaces. While the critical noncircular slip surfaces are assured to have lower factors of safety than the critical circular slip surfaces, it remains to be seen whether the difference is significant or not. Even without this refinement of the analyses, it can be concluded that the IPET strength model is a reasonable representation of the actual conditions in the 17th Street Canal breach area, and that the stability analysis mechanism described here is consistent with the field observations.

The calculated factors of safety are about 25% lower when it is assumed that a crack develops between the wall and the levee fill on the canal side of the wall. The results calculated assuming that a crack formed and full hydrostatic water pressure acted in the crack, are consistent with field observations, indicating that it is highly likely that a crack did form in the areas where the wall failed. It seems likely that when a crack formed and the portion of the wall below the levee crest was loaded by water pressures, the factor of safety would have dropped quickly by about 25%. Soil structure interaction analyses and centrifuge model tests will likely provide further understanding of crack formation and its relation to wall stability.

The New Orleans District Method of Planes used for the design analyses is a conservative method of slope stability analysis. All other things being equal, the factor of safety calculated using the Method of Planes was about 10% lower than the factor of safety calculated using Spencer's method, which satisfies all conditions of equilibrium.

The factors of safety calculated in the design analyses were higher than the factors of safety calculated for the conditions that are believed to best represent the actual shear strengths, geometrical conditions, and loading at the time of failure. The principal differences between the design analyses and the conditions described in this report relate to (1) the assumption that a crack formed between the wall and the levee soil on the canal side of the wall, and (2) the fact that the design analyses used the same strength for the clay and the peat beneath the levee slopes, and for the area beyond the levee toe, as for the zone beneath the crest of the levee. The IPET strength model has lower strengths beneath the levee slopes and beyond the toe.

Factors of safety for areas adjacent to the breach, where clay strengths are higher, were about 15% higher than those calculated for the breach area. These differences in calculated factor of safety are not large; thus appears that the margin of safety was small in areas that did not fail. It is possible that areas adjacent to the breach remained stable primarily because cracks did not form in those areas, and the wall was therefore less severely loaded.

Estimates of probability of failure for a water level of 8.5 ft NGVD are about 30% in the breach area, and 10% to 15% in the areas north and south of the breach. For a water level of 11.5 ft, the estimated probability of failure is about 50% in the breach area and 30% to 40% north and south of the breach. If stability analyses considering noncircular slip surfaces result in appreciably lower factors of safety, the corresponding probabilities of failure will be higher.

A more complete description of the stability analyses and results is contained in Appendix K1.

Drainage Canals – Physical Centrifuge Modeling

Scale modeling using large geotechnical centrifuges at RPI and at ERDC has commenced with trial models of London Avenue and 17th Street Canal levees based on the available site characterization and performance analyses. The experiment plan has been developed in close collaboration with numerical work being performed as part of the Floodwall and Levee Performance Analysis effort, to ensure that the models can meet their primary objective of providing qualitative insight and independent validation of the numerical analyses. Bulk samples of peat from the field have been taken for direct use in the models. A kaolin clay and fine sand have been used to replicate the clay and sand layers in the field. In common with standard geotechnical centrifuge model practice, the models are designed to be geometrically similar, reduced scale models with all significant engineering parameters (dimensions, permeability, density, strength and stiffness) correctly reproduced. Custom-built chambers have been constructed to contain the models with windows to facilitate video imagery of the onset of failure in the levee and foundations. The first trial models have been completed. The results are encouraging, showing that failure mechanisms consistent with the field observations can be realistically reproduced. Instrumental data from the model tests, particularly of the development of pore water pressure in the soil layers beneath the levee, are being examined and compared with numerical analyses. A full series of model tests will be carried out during March and April, using both centrifuge facilities as appropriate.

The design of the scale models has benefited from the extensive data collection and analysis in the field and from the site investigation and characterization activity under the levee performance analysis task. Collaboration with all members of the floodwall and levee performance analysis group and subsequent exchange of cross-sections, long sections, and soil properties have ensured that for each of the drainage canal sections investigated, the scale model design has proceeded with the best available information.

17th Street Canal Levee Model

The cross section here consists broadly of a clay levee on a foundation of peat and lacustrine clay. The selection of materials for the trial model comprised of speswhite kaolin clay for the levee and lacustrine clay stratum, and natural peat for the peat layer. The sheet pile wall was modeled using an aluminum plate. A cross section through the trial model is shown in Figure VI-20.

The clay levee in the trial model had strength after consolidation of 500 psf (based on the original design values). For kaolin clay, this is equivalent to a saturated density of around 110 pcf. Future models will use an increased strength of 900 pcf (kaolin saturated density of 113 pcf), based on the latest assessment of all information. The geometry of the clay levee was based on information available from design documents, as-built documents, LIDAR surveys, and field reconnaissance. The peat layer was formed from the natural peat samples taken from the field. The steel sheet pile wall was modeled for the 17th Street model using a solid steel plate of thickness 0.125 in., such that the bending stiffness of the wall is a correct representation of the sheet pile wall in the field (based on the PMA-22 section), as discussed above.

The underlying clay layer has strength after consolidation, increasing from 280 psf to 390 psf at the base (an increase of 11 psf per foot depth). Constructed using reconstituted kaolin clay, the saturated density of the clay will again be around 110 pcf.

Pore pressure transducers are located on the interface between the peat and the clay stratum and within the clay layer and the clay levee. Once steady state conditions are established at the start of the model, the precise rate of rise of the flood in the canal is immaterial, as the performance of the foundation and levee will be undrained.

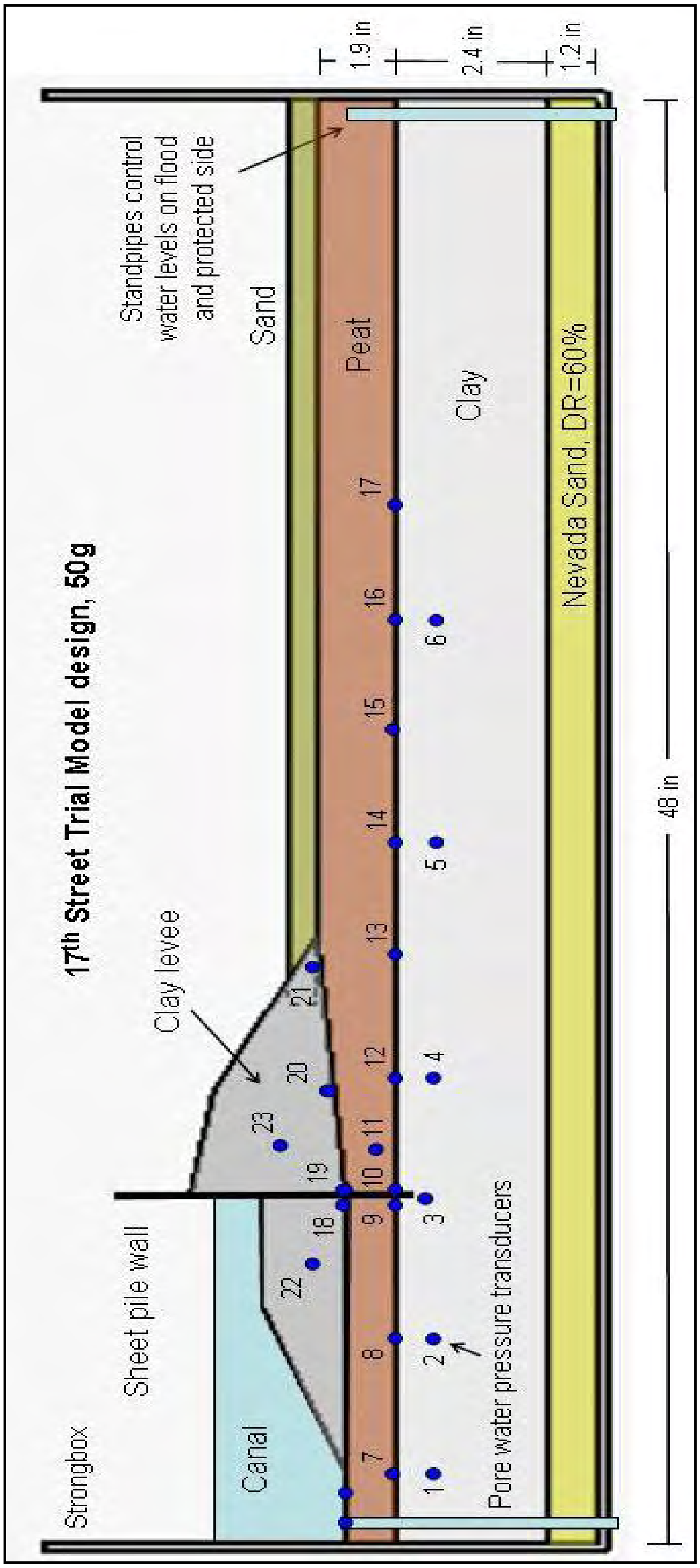

Figure VI-20. Diagram of 17th Street Trial Model Design (model units)

Interim Results

The results from the trial models have been encouraging. The model making process has been tested through the construction of the two trial models, one of which involved a sand bed beneath the peat and one of which involved a clay layer. Techniques for placing the sand and peat and for consolidating the clay have proved satisfactory and resulted in a layered model with densities and strengths close to the target density/strength profile based on the current available information. The approach, developed during the workshops, towards the sequence and method of construction of the levee and sheet pile wall has also proved successful. The hydraulic system to control water levels in the ground and the canal has permitted steady state conditions to be developed prior to the flood stage, and then for the water in the canal to be raised progressively until large-scale movements of the levee and flood wall were initiated, as may be seen after the trial model test. The 17th Street Canal was the second trail and has also provided good results, confirming the model process and design. Figure VI-21 shows the movement of the levee landward after the model test was completed and the water had been drained from the canal side (left).

In this case, as the water rose in the canal the wall again started to lean over, which resulted in a sliding failure in the clay layer immediately below the peat. Data from both the trial models are being assessed in detail prior to the initiation of the main model test phase, planned to commence at ERDC in March.

Floodwall and Levee Performance System Wide Assesement

Observations indicate that water overtopping the floodwalls led to extensive scour and erosion in some locations, which may ultimately have resulted in breaches in the flood protection system. The performance of levees varied significantly throughout the New Orleans area. In some areas the levees performed well in spite of the fact that they were overtopped. While in other areas the levees were completely washed away after being overtopped. Several possible factors could explain the differences in performance. One would be the type of material that was used to construct the levees. Another could be the direct wave action on the levees. The degree of dependence of overtopping versus wave action on the scour and erosion of the levees is yet to be determined and will be addressed in the high resolution analysis if the hydrodynamic environment experienced by the structures in the confined canals and channels. This task will examine the type of material used in construction of the levee versus the surge height and wave height to investigate their interdependence.

Figure VI-21. Sliding movement of the levee landward (to the right) observed at the completion of the 17th Street trial model test

Another common problem observed throughout the flood protection system was the scour and washout found at the transition between structural features and earthen levees. In many cases, the structural features were at a higher elevation than the connecting earthen levee, resulting in scour and washout of the levee at the end of the structural feature. At these sites, it appears the dissimilar geometry concentrates the flow of water at the intersection of the levee with the structural feature, causing turbulence that resulted in the erosion of the weaker levee soil. This task will examine the transitions to investigate their performance during Hurricane Katrina, highlighting both satisfactory and unsatisfactory performance of these transitions.

Penetrations through the flood protection systems required in order to permit through passage of trains and other surface transit produced additional transitions between dissimilar sections. Gate closures are provided at these locations in order to prevent flood waters from flowing into the protected area. This task will examine these gate closures to assess whether they were closed prior to the storm surge and to evaluate their performance during the storm surge.

The following section is initial damage assessment for the New Orleans East Basin hurricane protection system. This is presented here as an illustration of IPET's initial system-wide investigation of the floodwall and levee performance.

General Description of the New Orleans East Basin Hurricane Protection System

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

NOE Basin Components

Figure VI-22 illustrates the boundaries and basic flood protection components within the NOE Basin. This drawing 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 Southpoint), the eastern boundary of the Bayou Savage National Wildlife Preserve, and the north side of the Gulf Intracoastal Waterway (GIWW) (between the IHNC and eastern edge of the Bayou Savage National Wildlife Preserve). The main components are described in the next section moving clockwise through the basin, beginning at the Lakefront Airport and ending at the western end of the GIWW.

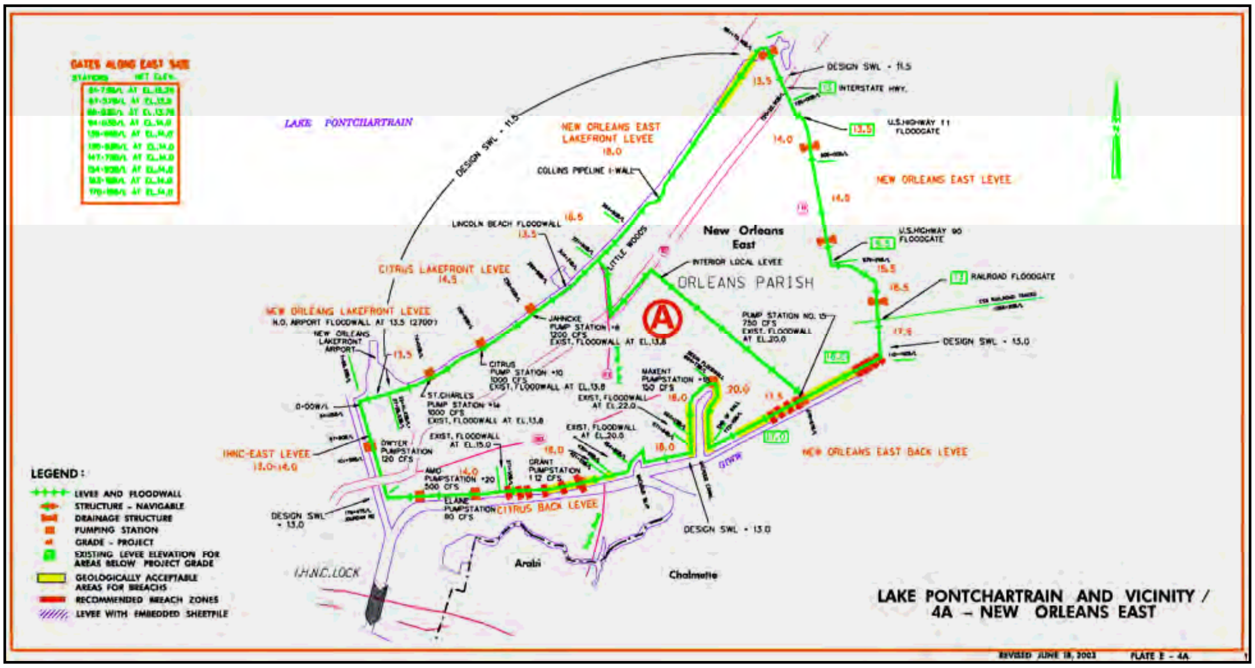

Figure VI-22. NOE Basin general components and top of levee/floodwall as-built elevations (feet) (source USACE, New Orleans District (Wayne Naquin)

Hurricane Protection Features

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

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

GIWW - The basin includes the Citrus Back Levee and New Orleans East Back Levee which consisting of approximately17.5 miles of earthen levees and concrete floodwalls along the northern edge of the GIWW.

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

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

Table VI-3 Summary of NOE Basin Hurricane Protection Features	
Exterior Levee and Floodwall (I-wall)	39 miles
Drainage Structures	4
Pump Stations	8
Highway Closure Structures	2
Railroad Closure Structure	1

IPET Investigation of Hurricane Protection Project Performance

Levee/Floodwall Damage Categories

Figure VI-23 illustrates the spatial distribution of levee and floodwall performance along the basin boundaries. This study is not concerned with the inner levees that are not federally owned.

Summary of Damages from Hurricane Katrina

Significant damages occurred mainly along the IHNC, southern end of the NOE Levee, NOE Back Levee, and the Citrus Back Levee. The IHNC will be discussed in another report. Levee and floodwall damages have been documented by the Task Force Gaurdian in their Project Information Reports (2005) and Damage Survey Report (2005) for NOE Basin. The TFG describes the major damages as follows:

- 12,750 ft of levee breach in the NOE Back Levee between Michoud Canal along the GIWW up to the CSX railroad crossing along the NOE Levee.
- Floodwall breaches at Pump Station 15 (800 feet) near the Maxent Levee and at the Air Products Hydrogen Plant near the Michoud Canal (300 feet).
- Floodgate, floodwall, and adjacent levee damage at the CSX railroad.
- 2000 feet of floodwall damage in the Citrus Back Levee along the GIWW between the IHNC and Paris Road.
- Levee and floodwall scour along the lakefront and NOE levees.
- Damage to all eight pump stations.
- Note: Overtopping was generally associated with varying degrees of scour (surface erosion), generally on the levee landside.

Legend
LONB = Overtopped levees, no breaching
WS = Overtopped floodwalls, no breaching (stable)
LOB = Overtopped levees, breaching
TF = Transition failure (floodwall to levee transition)
WF = Overtopped floodwalls, breached (failure)
WCF = Overtopped floodwalls, no breaching but came close

Figure VI-23. Generalization of levee and floodwall failures in the NOE basin

Table VI-4 provides the gross estimated linear feet of missing levee, damaged levee, and damaged floodwall.

Table VI-4
NOE Basin - Gross Linear Estimates of Damaged Features (Damage Survey Report, TFG 2005)

Total length of levee w/o cross section	2,900 ft.
Total length of levee w/reduced cross section	3,800 ft.
Total length of damaged flood wall	24,600 ft
Total	31,300 ft.

Nine separate construction projects have been identified by Project Information Report (TFG 2005) to repair the damaged areas and restore flood protection to pre-hurricane Katrina conditions. These projects represent an estimated $52.4 M (not including pump stations) in construction costs. Figure VI-24 shows the linear extent of each repair contract. Table VI-5 describes the damage as light, moderate or heavy, in addition to the repair method.

Figure VI-24. NOE - Project Summary Map of repair contracts, Project Information Report (TFG 2005)

Table VI-5
NOE Damage Synopsis

Citrus Lakefront Levee and Floodwall			
Lakefront Airport Floodwall (Capped I-wall)	Moderate scour	the land side of the floodwall	Excavate the scour area, place flowable fill and compacted material, place bedding material and 6"-7" slope pavement
Star & Strips Bvld Floodwall	None noted		
Jancke Pumping Station Floodwall	Light Scour	the land side of the floodwall	Excavate the scour area, place flowable fill and compacted material, place bedding material and 6"-7" slope pavement
Lincoln Beach Floodwall	Light Scour	the land side of the floodwall	Excavate the scour area, place flowable fill and compacted material, place bedding material and 6"-7" slope pavement
New Orleans East Lakefront Levee			
Collins Pipeline	None noted		
South Point to GIWW Levee			
Drainage structure, N19 (400+/- lf south of South point)	Moderate scour	the lake side of levee	Excavate the scour area, place compacted material, place bedding material and gabions
Other Drainage structures	Light Scour	the lake side of levee	Excavate the scour area, place compacted material, place bedding material and gabions
Pumping Stations	None noted		
CSX Railroad gate	Heavy Scour	the land side of the floodwall	Raising the flood protection from (NAVD29) 13.5 to '88 datum Elevation 20
New Orleans Back Levee			
OP Pump Station 15	Rotation & Failure of Iwall Tie-In Walls to frontage Twalls	10'-12' Scour holes on both FS & PS of wall	Replace uncapped I-wall w/ pile founded T-walls, Raise protection from (29 datum) 17 to (88 datum) 23.
I-wall West of OPPS 15	Moderate scour	Both FS & PS	Excavate the scour area, place compacted material and graded stone
East Michoud Canal (Air Products Breach)	Rotation & Failure of Iwall Tie-In Walls to levee	10'-20' Scour holes on both FS & PS of wall; 300 lf long	Replace uncapped I-wall w/ new levee section and uncapped Iwall; Raise protection from (29 datum) 17 to (88 datum) 21.
Michoud Slip to Michoud Canal Floodwalls	Light to moderate scour	PS of floodwall	Excavate the scour area, place flowable fill and compacted material, place bedding material and 6"-7" slope pavement
Citrus Lakefront Levee and Floodwall			
IHNC to Paris Road	Light Scour	the land side of the floodwall	Excavate the scour area, place flowable fill and compacted material, place bedding material and 6"-7" slope pavement
Citrus Floodwall at Bulk Loading Facility	Rotation & Failure of I-wall	6'-10' Scour holes on both FS & PS of wall	Replace I-wall w/ new L-type wall Raise protection from (29 datum) current 13.5 to (88 datum) 15 (as built elevation)

VII. The Consequences

Executive Summary

In the IPET Final Report, this chapter will answer the questions pertaining to "The Consequences" as presented in IPET Report 1. The interim results on the societal-related consequences of the Katrina-related damage, the relationship of local consequences to the performance of individual components of the flood protection system, the consequences had the system not suffered catastrophic failure, and the consequences of Katrina that extend beyond New Orleans and vicinity are presented.

An important component in assessing interior drainage and related consequences are the pump stations. The pumping stations designed and constructed in the greater New Orleans area are not part of the flood and hurricane protection system. Their purpose is to evacuate accumulated precipitation occurring during storms since much of the area is below the level of Lake Pontchartrain, sea level, and the Mississippi River. Many pumps, particularly the larger ones, have horizontal shafts with the impeller located above the normal water surface of the suction side. Maximum water levels occurring during the Katrina storm exceeded the design discharge side water levels causing some pumps, particularly those in Jefferson Parish, to experience reverse flow during the time they were not operated. All of the pump stations were designed to be operated by personnel at the station. None can operate without operators present (i.e., none are remote operated or use automatic controls). None of the pump houses were designed to protect themselves from local flooding as happened during Hurricane Katrina. The pump station evaluations in this interim report include the condition assessment of components and record of pumping station performance during Katrina. The interim information to date for each parish and for each station is reported in the pump station technical and detailed report appendices.

To answer the questions regarding how the hurricane protection system would perform under various conditions, the interior drainage analysis focuses on the filling and unwatering of the separate areas protected by levees and pump stations. Interior drainage models will be developed for St. Charles, Jefferson, Orleans, St. Bernard and Placquemines Parishes that simulate water levels for what actually happened during Hurricane Katrina and what would have happened had all the hurricane protection facilities remained intact, functioned as designed, and operated as planned. Interior modeling data will be used in the Consequence and Risk and Reliability analyses to assess, measure, and report risks for various

scenarios to help the public and officials make decisions. These models will also be useful to examine the degree of flooding that would result from any future scenarios.

Flooding and the level of destruction initiated by Hurricane Katrina are unprecedented from a natural disaster in U.S. history. The consequences from this event are both widespread and long-lasting. They can be described in economic, human health and safety, social and cultural, and environmental terms. The assessment of consequences has several purposes integral to understanding the dimensions of the event that happened. For instance, the economic impacts of the event went far beyond the direct impacts on the residents and businesses in New Orleans. Additionally, consequences are one of the dimensions of risk necessary to understand the level of safety provided by the hurricane protection system. Or, conversely, assessing consequences is part of estimating the residual risk borne by those who lived in the New Orleans area pre-Katrina and those who will live there after the protective system is restored. Therefore, the assessment of consequences will go beyond the grim accounting of destruction in people's lives, property, and the social fabric of New Orleans that actually happened. To provide a complete understanding of risk, consequences must be assessed under some "what if" scenarios. These losses, in turn, provide input for the IPET Risk and Reliability Assessment Team. Consequence assessment, in this mode, requires predictive approaches and frameworks. These approaches will be described and documented in detail in an Appendix to the final report.

Pumping Station Performance

Summary of Work Accomplished

To date this effort has obtained available documents through the contracted Architecture-Engineering firm (CH2M Hill), Task Force Guardian, Task Force Hope, and the USACE New Orleans District. The team has obtained documents and information from each of the parishes and responsible entities for pump station operations. The information includes configuration, capacity and location of each of the pump stations, photos of stations and components, pump performance curves, records of operation, fuel and/or power sources, backflow prevention devices, valves and gates for operations and the like. At this time approximately 90% of the work for St. Bernard Parish has been completed and submitted as a Technical Appendix (Appendix I, Pump Station Technical and Detailed Report) for this report. This appendix serves as the example of the complete pump station analysis that can be expected for all of the study areas for the final report.

Interim Results

St. Bernard Pump Stations

There are eight pump stations in St. Bernard Parish. All pumps are powered by diesel engines which are mechanically connected to the pumps. Five stations (representing 80% of total capacity) have operating floors approximately 12 feet above the natural ground surface which substantially reduced storm-induced damage. Three stations (#2, #3 and #5) were flooded to a depth of six to eight feet above the operating floor which destroyed the diesel engines, vacuum pumps, and many accessories. Until this equipment can be replaced, the stations cannot be operated. The metal framed & sided buildings suffered considerable damage while the structures built of concrete and brick experienced little damage. The three flooded stations accounted for 90% of the total estimated damage of $10.7 million.

The eight pump stations in St. Bernard Parish have a total discharge capacity of 6,960 cfs (cubic feet per second) to evacuate accumulated precipitation in a drainage area of 17,620 acres. If all pumps were to operate at rated capacity, they could keep up with a steady precipitation rate of 0.39 inches per hour. All stations use pumps directly connected to diesel engines.

Each pump station was visited to obtain operating logs of individual pump units. As can be seen in the performance chart (the white portion of each bar), a significant amount of operating information was lost or not available.
Figure VII-1 shows the daily operational status of the percentage of 28 main pumps from August 28th, through September 15th when continued pumping was no longer required. Although only three of the eight stations suffered substantial damage, these three accounted for nearly half (13) of the pump units.

Jefferson Parish Pump Stations

No Jefferson Parish pump station was flooded during Katrina and, as a result, none experienced significant damage. Primarily, the damage was to roofs, gutters, skylights, gutters, etc. A total estimated damage for all stations was $760,000. For their safety, the station operators were ordered to leave their stations prior to the arrival of Katrina. During this time when operators were absent, the pumps were not operated. The surface water level in Lake Pontchartrain exceeded the design level of the stations discharging to the lake thereby allowing reverse flow to occur.

Jefferson Parish has six pump stations on the East bank and twenty on the West bank. The East bank stations have a total capacity of 20,835 cfs to drain an area of 29,300 acres. If all pumps were to operate at rated capacity on the East bank, they could keep up with a steady precipitation rate of 0.70 inches per hour. The West bank stations have a total capacity of 23,354 cfs to drain an area of 44,200 acres. If all pumps were to operate at rated capacity, they could keep up with a steady precipitation rate of 0.52 inches per hour. All stations use pumps directly connected to diesel engines.

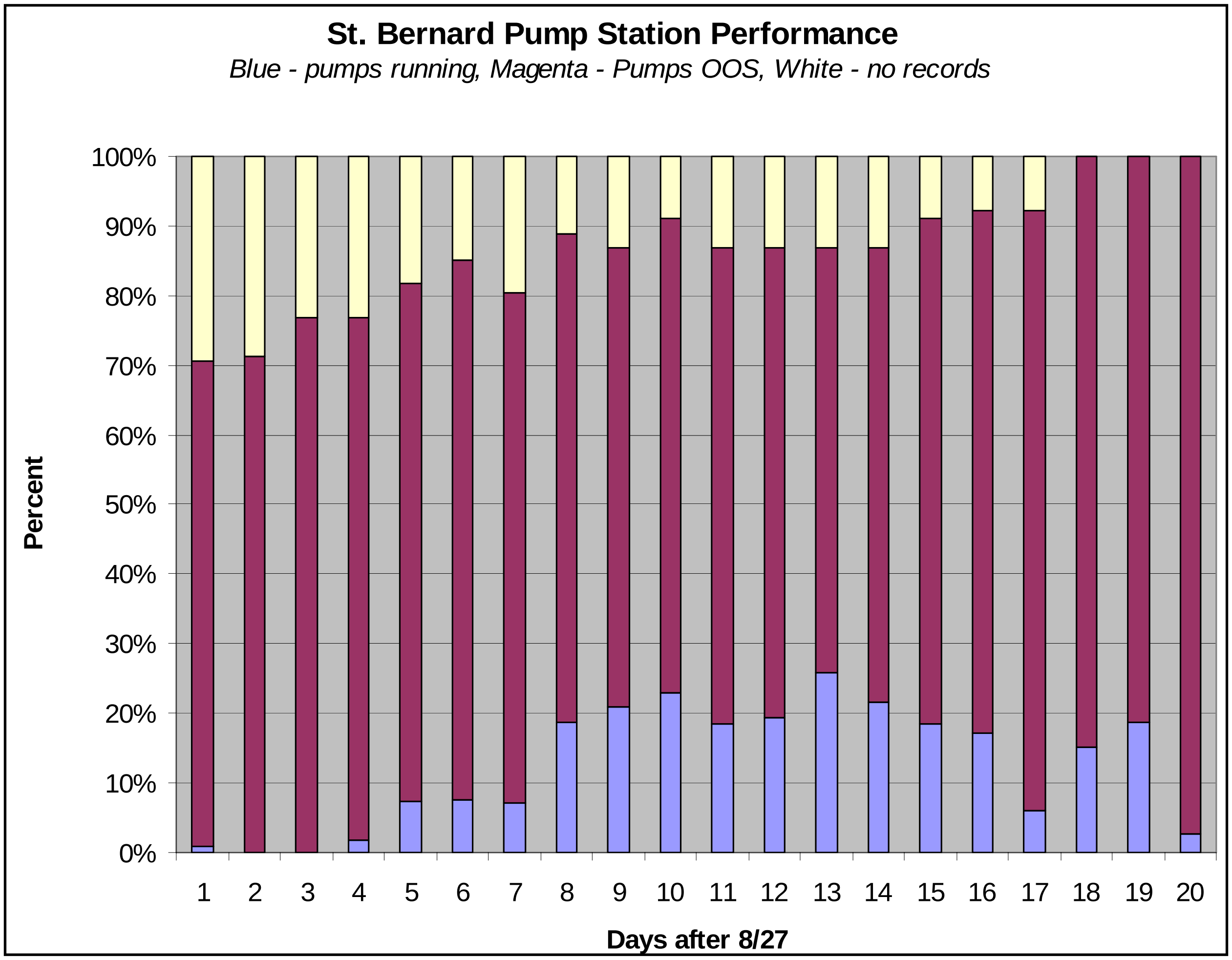

Figure VII-1. St. Bernard pump station performance

Plaquemines Parish Pump Stations

Nine stations suffered significant damage – principally from flooding. Total cost to restore the stations to pre-storm condition was estimated to be $8 million. One station (Belair) was so damaged, that a new pump house, including the foundation slab, is required. At stations where the diesel engines were destroyed due to flooding, new engines will be installed on an elevated platform or support which is now typical of newer construction. A hydraulic drive system will transmit power from the engine to the pump shaft.

There are 19 pump stations in Plaquemines Parish including two which are privately owned. These stations have a total discharge capacity of 13,680 cfs to evacuate accumulated precipitation in a drainage area of 55,000 acres. If all pumps were to operate at rated capacity, they could keep up with a steady precipitation rate of 0.25 inches per hour. All stations use pumps directly connected to diesel engines.

Orleans Parish Pump Stations

The pump stations and generating station in Orleans' metropolitan area suffered significant damage – principally due to flooding of the electrical motors, generators and switchgear. Pump stations in the Orleans east area also were significantly damaged from flooding. Many of the pump bearings were damaged from operating with dirty water. Some of the diesel engines were also destroyed. Neither of the west bank stations experienced any flooding. The contractor hired by the Corps estimated a total damage of more than $ 39 million in their damage survey report. When New Orleans District's Corps of Engineers completes their *Project Information Report*, the damage estimate is expected to be substantially less.

Orleans Parish has 22 pump stations on the East bank and two on the West bank. Twelve of the East bank stations are located in the metropolitan area as well with the remaining ten located east of the Inner Harbor Navigation channel (Orleans east). All stations in the metropolitan area have pumps which are electrically driven – most by direct-drive 25 Hz motors. A central diesel-electric generating station provides 25 Hz electricity for these stations. All stations in Orleans east and the two on the west bank have pumps which are diesel driven.

Status of Remaining Efforts

The remaining Parishes will be brought to the 90% level similarly to the work accomplished for St. Bernard Parish for the final report.

Interior Drainage Analysis

Summary

To help answer the questions regarding how the hurricane protection system would perform under various conditions, the interior drainage analysis focuses on the filling and unwatering of the separate areas protected by levees and pump stations. Interior drainage models are being developed for St. Charles, Jefferson, Orleans, St. Bernard and Placquemines Parishes that simulate water levels for what actually happened during Hurricane Katrina and what would have happened had all the hurricane protection facilities remained intact, functioned as designed, and operated as planned.

Other IPET task teams are providing data needed to estimate the flow into and out of the modeled parishes. Data provided includes storm surge and wave heights, levee breach geometry, and storm water pump station operation. Since these data are needed at many locations for the duration of the event itself, it is anticipated some of the data will be difficult to obtain due to the extent and severity of the hurricane and the resulting flooding.

Two scenarios are being modeled in this effort:

Pre-Katrina or As-Designed Scenario: Using the interior drainage models, simulate what would have happened during the Katrina event had all hurricane protection facilities remained intact, functioned as designed, and operated as planned. No levees will be failed for this scenario even where overtopping occurs. All water will be removed by the pumping stations.

Katrina or Actual Performance Scenario: Using the interior drainage models, simulate what happened during the Katrina with the hurricane protection facilities performing as actually occurred. All water will exit flooded areas through original breaches, man-made breaches, temporary pump stations and operating pumping stations.

Results from these scenarios will be used to provide input to answer the following specific questions that relate to the overall mission questions for IPET.

- How did the floodwalls, levees and drainage canals, acting as an integral system, perform during and after Hurricane Katrina?

- How did the pumping stations, canal gates and road closures, acting as an integral system, operate in preventing and evacuating the flooding due to Hurricane Katrina?

Interior modeling data will be used in the Consequence and Risk and Reliability analyses to assess, measure, and report risks for various scenarios to help the public and officials make decisions.

Background

Interior drainage/flooding models are not necessary to estimate water elevations in an interior leveed area for a catastrophic condition such as Hurricane Katrina where water levels rise rapidly until they reach the level of Lake Pontchartrain, the IHNC, or Lake Borgne. However, interior drainage/flooding models are essential for estimating the peak water elevation and extent of possible flooding, if any, when the hurricane protection system performs satisfactorily or without catastrophic failure. The models can also be used to estimate the time needed to unwater an area once it is flooded.

Many people will want to know the level of risk to which they are subjected on or before June 1, 2006 when the pre-Katrina level of protection will be achieved at the levee breach locations. The interior drainage/flooding models will be used to examine the resultant flooding for Hurricane Katrina rainfall, storm surge, wave heights, and pump station operations given the observed flood protection system performance and for the situation of no catastrophic structural failures. As such, the models will determine estimated peak water elevations and areas inundated within the protected areas for these two situations. These models will also be useful to examine the degree of flooding that would result from other storm or structural and pumping station performance scenarios.

The modeling for Katrina is being accomplished by four teams. Table VII-1 shows the modeling responsibilities.

Each area has been assigned a priority. The priority is based on flooding experienced during the Katrina event. Development is progressing on all models with the 2 and 3 priority areas not as far along as the priority 1 areas. Table VII-1 lists HEC-RAS and HEC-HMS. These refer to tools developed by the Corps of Engineers, Hydrologic Engineering Center. HEC-RAS refers to the River Analysis System software package. HEC-RAS is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. HEC-HMS refers to the Hydrologic Modeling System software package. HEC-HMS is designed to simulate the precipitation-runoff processes. For this study, it is used to transform the precipitation, observed during Katrina, into runoff. This runoff is input to the HEC-RAS model and routed to the pump stations.

Table VII-1
Modeling Responsibilities

Leveed Area	Priority	Team	
		RAS	HMS
Jefferson East Bank	1	CTE	CTE
Jefferson West Bank	2	CTE	CTE
Orleans East Bank	1	MVK	MVK
New Orleans East	1	MVN	MVN
Orleans West Bank	2	MVN	MVN
St. Bernard	1	MVN	HEC
St. Charles East Bank	3	MVN	HEC
Plaquemines	1	HEC	HEC

CTE – CTE Consultants, Chicago, IL
MVK – Corps of Engineers, Vicksburg District
MVN – Corps of Engineers, New Orleans District
HEC – Corps of Engineers, Hydrologic Engineering Center, Davis, CA

Summary of Accomplished Work

The sequence of work for the interior drainage/flooding analysis is:

Develop HEC-RAS models using existing models, if available. Otherwise, construct new RAS models using current LIDAR data.

Develop HEC-HMS models using existing models, if available. Otherwise, construct new HMS models.

Conduct a sensitivity evaluation of critical model parameters.

Compute the Actual Performance Scenario (Task 3) results using Katrina data. Adjust model parameters, as appropriate.

Compute the As-Designed Scenario results.

The team is currently completing steps 1 and 2, developing the HEC-RAS and HEC-HMS models. The status of each leveed area is shown in Table VII-2.

Table VII-2

Leveed Area	Priority	Est. % Complete	
		RAS	HMS
Jefferson East Bank	1	60	75
Jefferson West Bank	2	50	75
Orleans East Bank	1	70	50
New Orleans East	1	50	35
Orleans West Bank	2	15	15
St. Bernard	1	75	33
St. Charles East Bank	3	15	15
Plaquemines	1	70	70

Interim Results

The interior drainage/flooding analysis require both field and analytical data from several tasks on the IPET team. This includes high water elevations, flooding and unwatering time sequence, levee and flood wall geometries, storm surge and wave heights, and stormwater pump station performance information. Consequently, interim results will not be available and steps 4 and 5 will be accomplished near the end of the IPET effort. However, a summary description of how the models are being developed and generic samples of input, parameters, and output is provided in the description.

RAS Interior Modeling

Method. The Corps of Engineers' Hydrologic Engineering Center's River Analysis System (HEC-RAS) will be used for this study. RAS models will be developed for each area and will be operated independently in accordance with the current drainage patterns within the greater New Orleans area. Each parish maintains their own drainage system and the models will reflect this operation. Figure VII-2 shows the model locations. Table VII-3 lists their names.

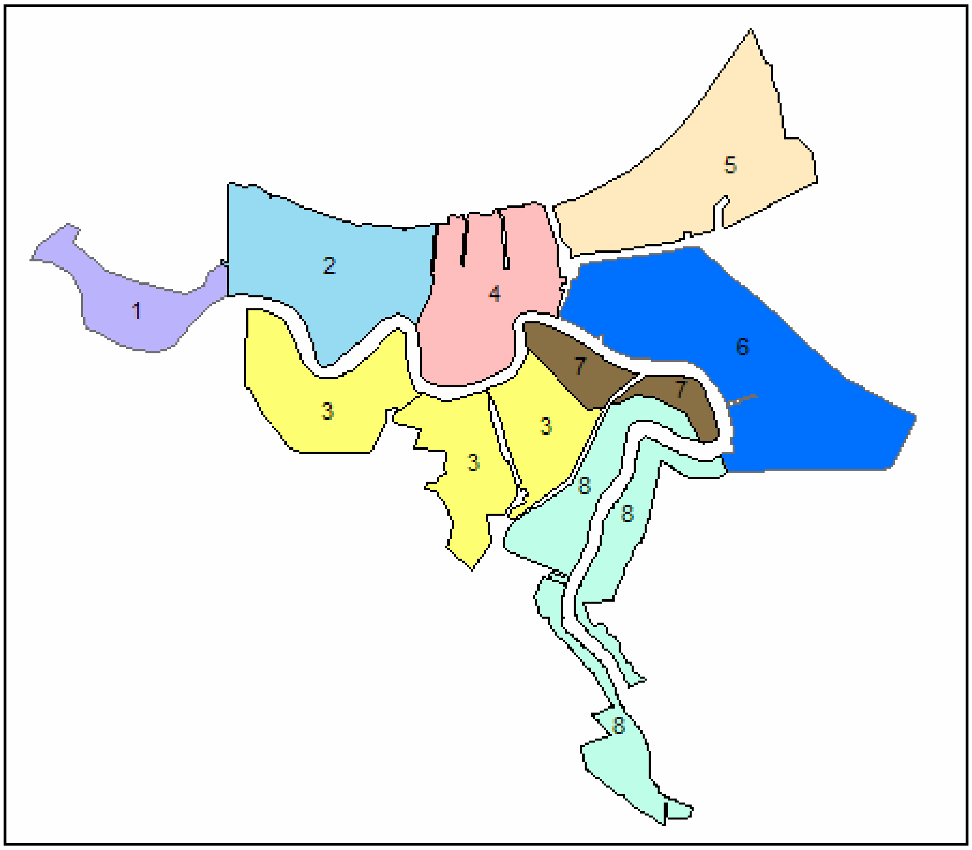

Figure VII-2. Model areas

Table VII-3 Model Areas	
Number	**Name**
1	St. Charles Parish
2	East Bank Jefferson Parish
3	West Bank Jefferson Parish (3 separate models make up this area)
4	Orleans Parish
5	New Orleans East (Part of Orleans Parish)
6	St. Bernard Parish (Includes Lower 9th Ward)
7	Algiers (Part of Orleans Parish)
8	Plaquemines Parish

The West Bank Jefferson Parish is actually broken into 3 separate models, thus the three divisions. The Algiers area is being developed as one model. The Plaquemines Parish model actually extends down river to Venice. The entire extend is not shown on this figure. HEC-RAS models existed for a few locations within the modeling area prior to Katrina. These models were adopted and updated to reflect conditions at the time of Katrina.

Terrain. All HEC-RAS models are based on 5 Meter Lidar data set. The datum of the LIDAR is NAVD88. The vertical accuracy for this data is

+/- 0.7 feet. The horizontal projection is Louisiana State Plane South 1983 feet. All models have been georeferenced to this projection. The basin boundaries for the HMS models are in the same projection.

RAS Storage Areas. The area between drainage canals were modeled as storage basins. These basins are delineated based on geographic features within each leveed area. Features used as divisions between storage areas are levees, railroads, roads, elevated areas, etc. The storage area elevation-volume data and the connection between the storage areas in the RAS models were developed using the Hydrologic Engineering Center's HEC-GeoRAS software. GeoRAS is an ArcMap extension. It provides the tools to draw a polygon representing the storage basin shape and then extract the volume-elevation data from the 5 meter grid. Additionally, it also provides the tools to draw a line which represents the connection between adjacent storage basins and then extract a profile of the connection that represents the elevations from the 5 meter grid. Each RAS model includes storage basins and storage area connections (flow diversions). Figure VII-3 is a sample which shows storage basin outlines for Orleans Parish.

Water can flow between storage areas through storage area connections. These connections are modeled hydraulically using either a weir equation or a linear routing method to transfer flow between the storage areas. Flow can go in either direction, and submergence on the weir is accounted for. Both the weir coefficients and the linear routing coefficients are used as calibration parameters to slow down or increase the spread of the water through the system.

Geometric Data. Cross section data is used to represent the canals and the enclosed storm drains. Terrain information for describing the cross sections has come from a range of sources. General terrain for the open canals is a combination of the terrain model and surveyed cross section data. In general, the terrain model does not provide enough detail to hydraulically describe the canals. Additionally, the terrain model does not include any elevation data below the water surface. Surveyed cross section data has come from previous studies as well as newly surveyed cross sections. At the time of writing this preliminary report, we had not received all of the detailed surveys needed for all of the models. Therefore, the incorporation of detailed surveys for canals is an ongoing process.

Storm Drain System. Storm drains are modeled in HEC-RAS as normal cross sections with lids to represent a pressurized pipe. HEC-RAS has a feature called the Priessmann Slot option that allows the open channel flow equations to mimic pressure flow equations for an enclosed cross section. The Priessmann slot option puts a small slot in the lid of the cross section to allow the water surface to rise to the hydraulic gradeline within the pipe. This slot is extremely small and the wetted perimeter of the slot is not included in the conveyance calculations for the storm drain. The width of the slot is calculated in order to get the open channel flow wave celerity to be equal to the pressure wave celerity. This capability allows the HEC-RAS model to handle both open channel flow and pressure flow within a storm drain using the same set of equations.

Figure VII-3. Storage areas for Orleans Parish

The dimensions and slopes of the main storm drains have been obtained from drainage system maps, as well as previous models of the storm drainage system. Sub collector storm drains were added by putting in lateral structures in HEC-RAS along the main reaches. The lateral structures were used to put in culverts that represent the secondary storm sewer pipes that flow into the main storm sewer pips. The culverts were directly connected to the surface by connecting them to the storage areas that are being used to represent the surface terrain. With this setup, any water that goes into a surface storage area can then get into the storm drains through the culverts that are connected to them. Additionally, water that backs up within any stormdrain can also flow out into the surface areas through these culverts.

Pump Plants. Pump station data is being collected for all of the pump plants in the 5 parishes. HEC-RAS uses an inline structure to represent the pump house within a canal (for example, the 17th street canal pumping station). A series of pumps are then added to pump water from the interior sump area to the exterior canal system on the other side of the structure. HEC-RAS has the capability to model pumps of different sizes, capacities, and different on and off elevations that represent the normal operations of the pumps. Additionally HEC-RAS has the ability to enter pump override rules. These rules are being used to mimic the stopping and starting of pumps due to power failures that occurred during the event.

Boundary and Initial Conditions. Initial conditions were modeled by putting a base flow in all of the storm sewers and canals. HEC-RAS then computes a backwater profile to get the initial water surface. Just upstream of the pump station, the water surface is actually much lower in the sump area than it is on the open canal side. To accommodate this, HEC-RAS has an option where you can input a water surface to be used in the backwater computations. Initially, all of the storage areas are dry. This is simulated by setting the starting water surface elevation to the minimum elevation in each of the storage areas.

Results from the ADCIRC model are used as the exterior boundary conditions to the HEC-RAS models. Stage-hydrographs are applied directly to the canals that are open to Lake Pontchartrain and the Mississippi river. To apply the ADCIRC results in areas that were not modeled as canals (for example, the levees along lake Pontchartrain and the back levees for New Orleans East and St. Bernard Parish), it was necessary to put in model reaches with cross sections representing the lake areas. Stage hydrographs from ADCIRC were applied to each of these model reaches. Figure VII-4 depicts an ADCIRC computed stage-hydrograph used as a boundary condition for the 17trh Street Canal at Lake Pontchartrain. Each reach is connected to the interior area by using the lateral structure option in HEC-RAS. These lateral structures represent the levees that separate the interior areas from the unprotected exterior areas. The lateral structure option in HEC-RAS allows the model to calculate overtopping flows, as well as any levee breaches that occurred along these levees.

Levee Failures. As mentioned previously, levees and levee breaches are modeled as lateral structures along the canals and lake areas. The top of the levee is the top of the lateral structure. Flow over this structure is modeled with the weir equation. A levee breach can be added to any lateral structure. The breach outflow hydrographs are directly connected to a storage area that represents the land surface inside the levee in that area. The breach can be triggered based on a water surface elevation, time, or elevation and duration above an elevation. HEC-RAS requires the modeler to enter the maximum breach size and duration of the breach development. Breach information has been collected by another IPET team. This information is being used to estimate the breach parameters for within HEC-RAS. Figure VII-5 shows a sample result for a levee breach with the corresponding flow through the breach and the resultant stage in the flooded area.

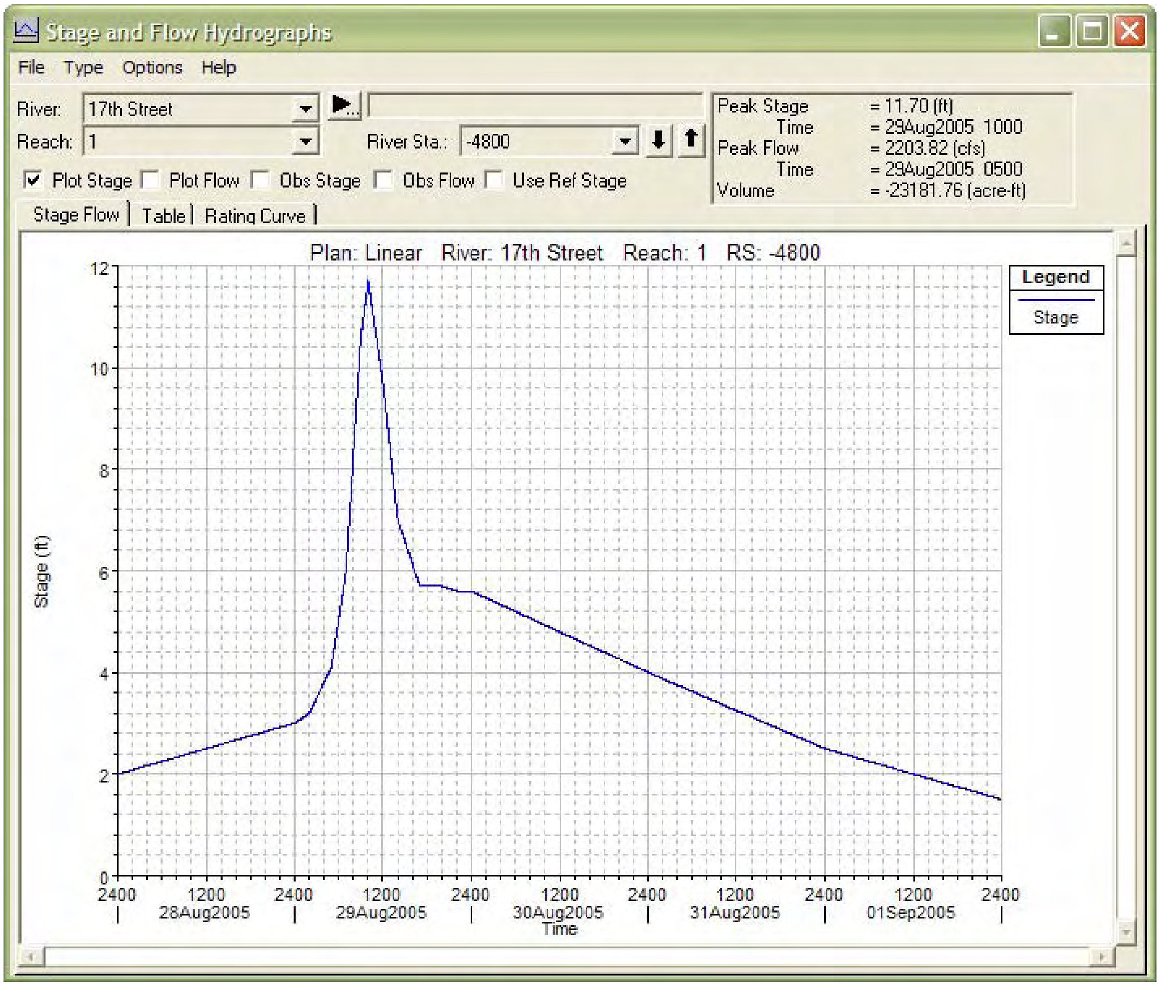

Figure VII-4. ADCIRC stage-hydrograph 17th Street Canal at Lake Pontchartrain

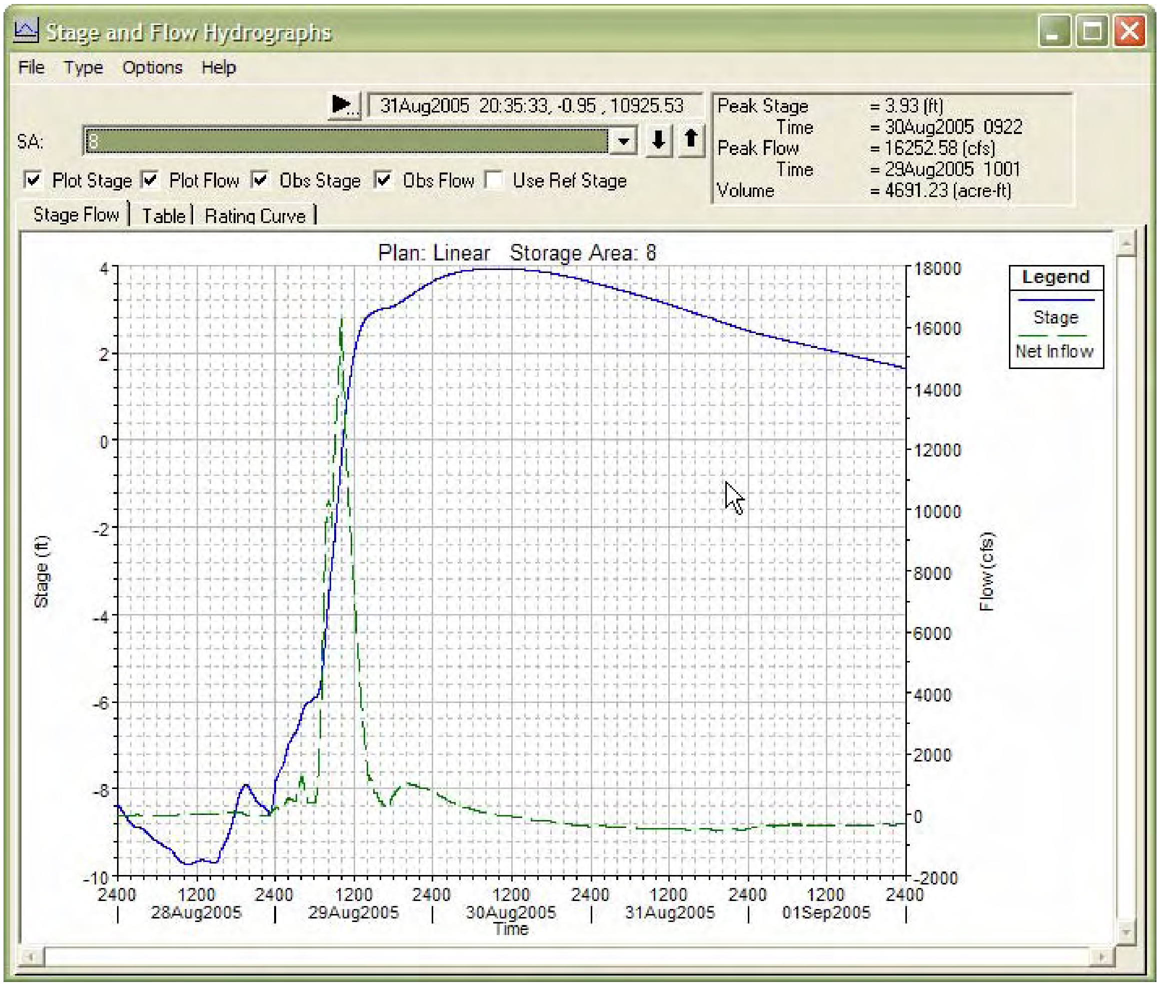

Figure VII-5. Levee breach hydrographs

Sample Results. Figure VII-6 depicts a sample result of the floodplain in Orleans Parish. This type of visual result is available for multiple time steps through the entire simulation. In this sample, the red locations show the actual failure locations during the Katrina event. The depth of water is indicated by the shade of blue with dark blue representing deeper water. Final products will have a legend which details information on the plots such as water depth. Additionally, animations of the flood progression can be produced.

HMS Interior Modeling

Method. The Corps of Engineers' Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) and will be used for this study.

Basin Models. The HMS models have been built to correspond directly to the RAS models. The HMS basin boundaries are a reflection of the RAS storage area boundaries. Applying this method allows the HMS model to transform the Katrina precipitation into runoff. The computed hydrograph is input to RAS as

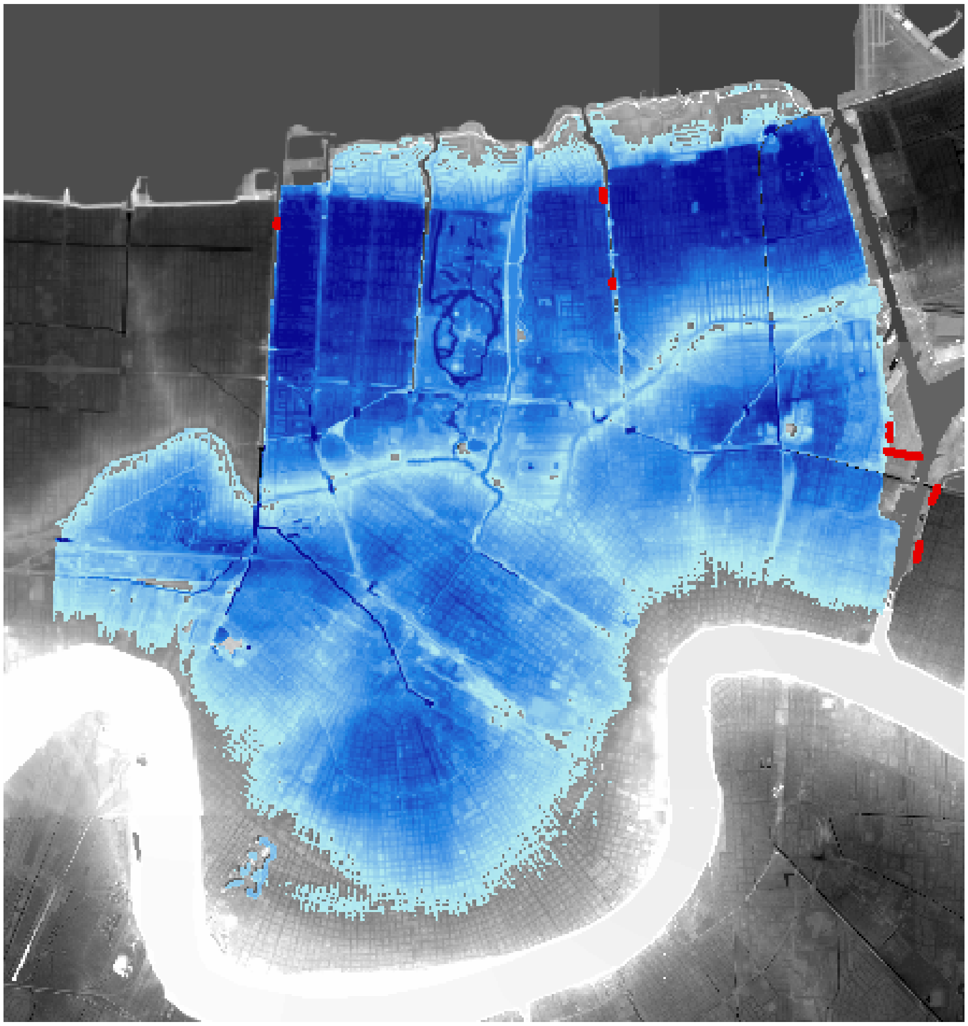

Figure VII-6. Sample result for Orleans parish

Inflow to a Storage Area. Figure VII-7 depicts an HMS basin model setup for the upper Plaquemines Parish area.

Rainfall. Radar rainfall data, referred to as Multisensor Precipitation Estimator (MPE), was used as a boundary condition in the hydrologic models to determine runoff hydrographs produced by the Hurricane Katrina event. MPE data from the Lower Mississippi River Forecast Center (LMRFC) was downloaded from the following website: *http://dipper.nws.noaa.gov/hdsb/data/nexrad/lmrfc_mpe.php*. Raw radar data is adjusted using rain gage measurements and possibly satellite data to produce the MPE product. Figure VII-8 shows the amount of precipitation estimated by the MPE product from August 29, 0600 – 0700.

The radar rainfall data was imported into a GIS where a precipitation hyetograph was computed for each subbasin in the different basin models. The individual hyetographs were imported into a DSS file where they were read by HEC-HMS.

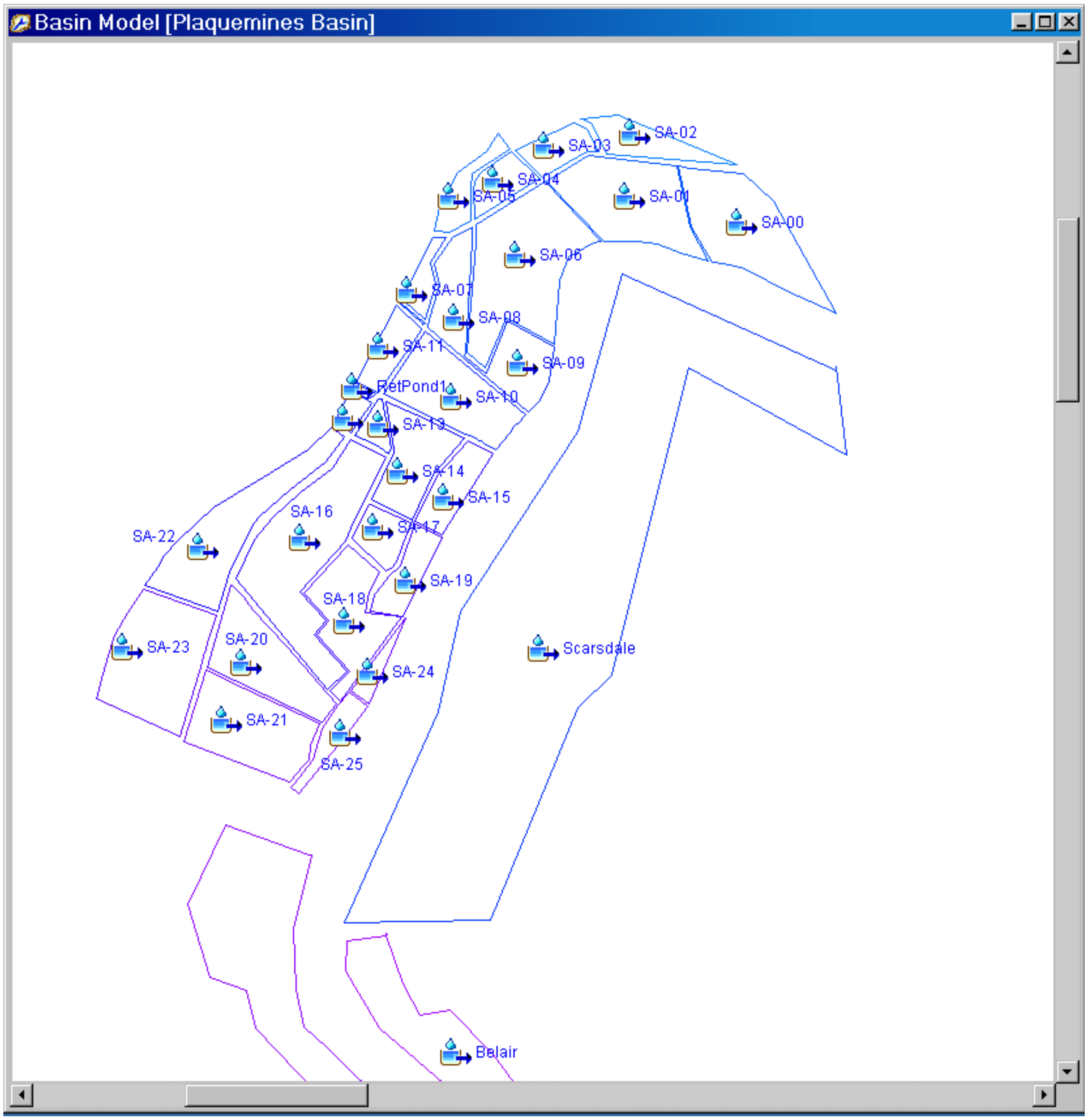

Figure VII-7. Sample HMS Basin model

Land Use and Soil Data. Land use and soil data were used to estimate SCS curve numbers. Land use data was obtained from the New Orleans District (MVN). The land use data was a raster coverage of 24 different land use types (Table VII-4). Soil data, contained in the Soil Survey Geographic (SSURGO) Database, was downloaded from the following National Resources Conservation Service (NRCS) website: *http://www.ncgc.nrcs.usda.gov/products/datasets/ssurgo/*. SSURGO is a digital copy of the original county soil survey maps and provides the most detailed soil maps from the NRCS.

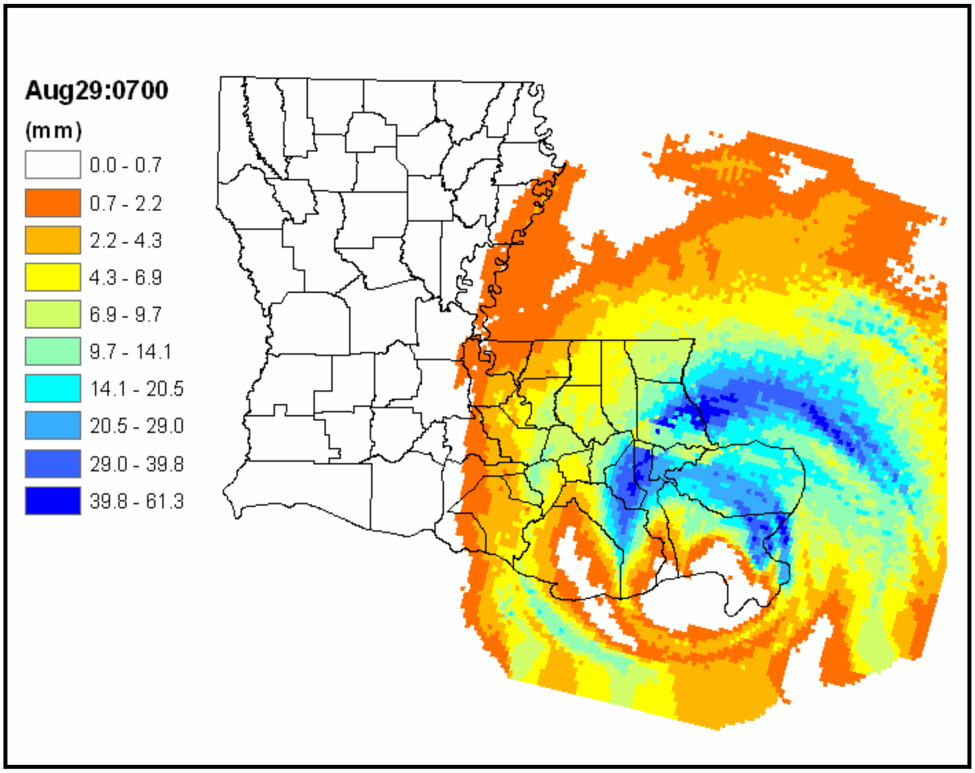

Figure VII-8. Katrina Precipitation

Table VII-4
Curve Numbers

	LAND USE	A	B	C	D
1	Fresh Marsh	39	61	74	80
2	Intermediate Marsh	39	61	74	80
3	Brackish Marsh	39	61	74	80
4	Saline Marsh	39	61	74	80
5	Wetland Forest-Deciduous	43	65	76	82
6	Wetland Forest- Evergreen	49	69	79	84
7	Wetland Forest- Mixed	39	61	74	80
8	Upland Forest- Deciduous	32	58	72	79
9	Upland Forest- Evergreen	43	65	76	82
10	Upland Forest- Mixed	39	61	74	80
11	Dense Pine Thicket	32	58	72	79
12	Wetland Scrub/shrub - deciduous	30	48	65	73
13	Wetland Scrub/Shrub - evergreen	35	56	70	77
14	Wetland Scrub/Shrub - Mixed	30	55	68	75
15	Upland Scrub/Shrub - Deciduous	30	48	65	73
16	Upland Scrub/Shrub - Evergreen	35	56	70	77
17	Upland Scrub/Shrub - Mixed	30	55	68	75
18	Agriculture-Cropland-Grassland	49	69	79	84
19	Vegetated Urban	49	69	79	84
20	Non-Vegetated Urban	71	80	87	91
21	Upland Barren	77	86	91	94
22	Wetland Barren	68	79	86	89
23	Wetland Complex	85	85	85	85
24	Water	100	100	100	100

Loss Rates. Loss rates were computed by determining the amount of precipitation intercepted by the canopy and depressions on the land surface and the amount of precipitation that infiltrated into the soil. Precipitation that is not lost to interception or infiltration is called “excess precipitation” and becomes direct runoff. The Soil Conservation Service (SCS) Curve Number (CN) method was used to model interception and infiltration. The SCS CN method estimates precipitation loss and excess as a function of cumulative precipitation, soil cover, land use, and antecedent moisture. This method uses a single parameter, a curve number, to estimate the amount of precipitation excess\loss from a storm event. Studies have been carried out to determine appropriate curve number values for combinations of landuse type and condition, soil type, and the moisture state of the watershed.

Table VII-4 was used to estimate a curve number value for each combination of land use and soil type in the study area. This table was supplied by the MVN office. Each soil type in the SSURGO Database was assigned to one of the four hydrologic soil groups (A, B, C, or D). The percent impervious cover is already included in the curve number value in Table VII-4. More information about the background and use in the SCS curve number method can be found in Soil Conservation Service (1971, 1986).

Transform. Excess precipitation was transformed to a runoff hydrograph using the SCS unit hydrograph method. The SCS developed a dimensionless unit hydrograph after analyzing unit hydrographs from a number of small, gaged watersheds. The dimensionless unit hydrograph is used to develop a unit hydrograph given drainage area and lag time. A detailed description of the SCS dimensionless unit hydrograph can be found in SCS *Technical Report 55* (1986) and the *National Engineering Handbook* (1971).

Drainage area was computed using GIS and input into HEC-HMS. Lag time was computed by using an estimate of travel time for the longest flow path.

Sample Results. Figure VII-9 depicts results for an HMS subbasin. The upper graph shows precipitation and the lower graph shows the runoff from the subbasin. This runoff hydrograph will be entered in the HEC-RAS model.

Status of Remaining Effort

- Step 3 – Sensitivity analysis of critical parameters. For the hydraulic modeling, factors that will be tested include breach opening widths, breach times, weir coefficients and roughness values. In the hydrologic modeling, the sensitivity of lag times and soil parameters will be tested.
- Steps 4 & 5 – Run the two scenarios.
- Determine if sediment and debris impact drainage efficiency.
- Complete the technical appendix to the final report.

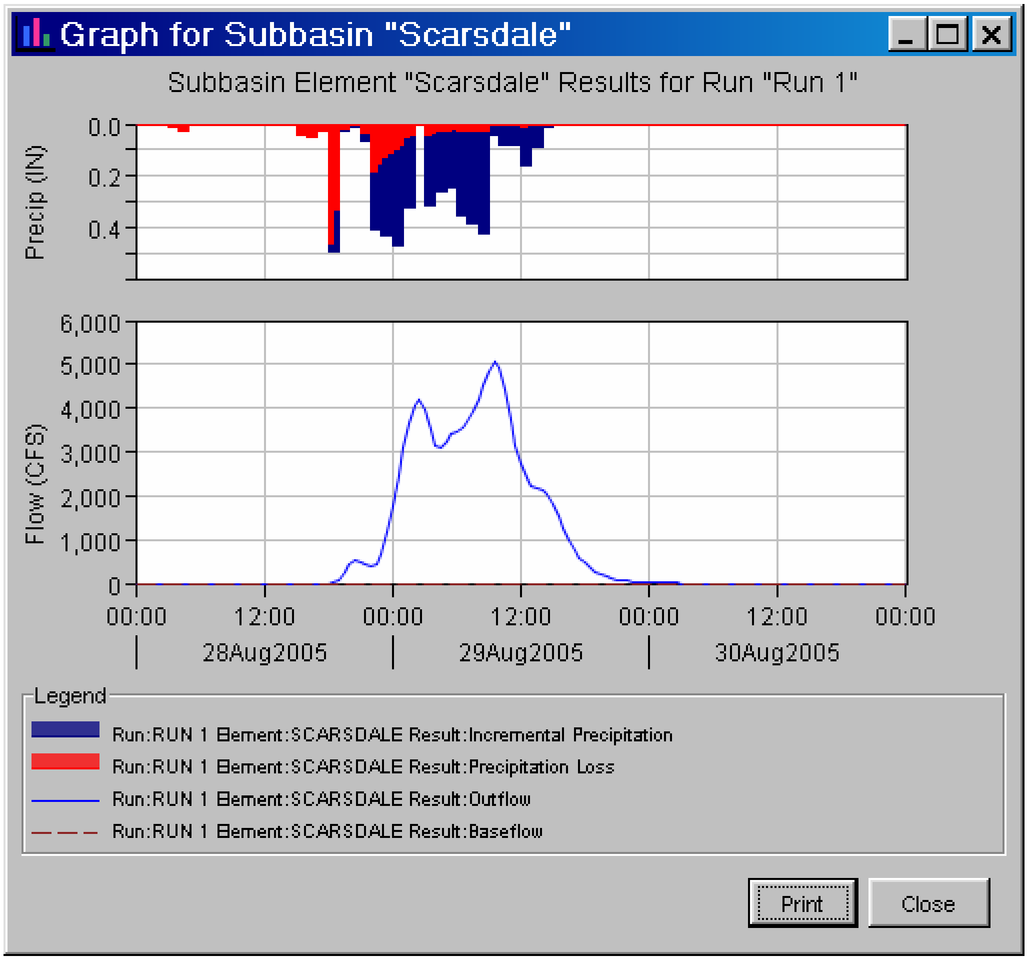

Figure VII-9. HMS Subbasin results

Final Report

The following output will be produced for both scenarios, as applicable.

Modeling Output

- Effective rainfall and runoff volume and distribution
- Flow hydrographs at breaches, overtopped areas, pump stations, and other entry and exit points
- Interior leveed area stage and volume hydrographs
- Filling and unwatering timeline

Visual Displays

- Cross-sections, alignments and storage basins
- Hydrologic basin delineations

- Radar and hyetographs
- Breach locations, volumes, and flow hydrographs
- Overtopping locations, volumes, and flow hydrographs
- Pump station locations, volumes, and flow hydrographs
- Computed and observed stage and volume hydrographs for interior leveed areas
- Inundation area maps
- Time lapse animation of water flowing into and out of interior leveed areas

Tables

- Event timeline
- Summary of volume of flood water from each source

Losses Analysis

Flooding and the level of destruction initiated by Hurricane Katrina are unprecedented from a natural disaster in U.S. history. The consequences from this event are both widespread and long-lasting. They can be described in economic, human health and safety, social and cultural, and environmental terms. The assessment of consequences has several purposes integral to understanding the dimensions of the event that happened. For instance, the economic impacts of the event went far beyond the direct impacts on the residents and businesses in New Orleans. Additionally, consequences are one of the dimensions of risk necessary to understand the level of safety provided by the hurricane protection system. Or, conversely, assessing consequences is part of estimating the residual risk borne by those who lived in the New Orleans area pre-Katrina and those who will live there after the protective system is restored. Therefore, the assessment of consequences will go beyond the grim accounting of destruction in people's lives, property, and the social fabric of New Orleans that actually happened.

To provide a complete understanding of risk, consequences must be assessed under some "what if" scenarios. These losses, in turn, provide input for the IPET Risk and Reliability Assessment Team. Consequence assessment, in this mode, requires predictive approaches and frameworks. These approaches will be described and documented in detail in an Appendix to the final report.

Table VII-6 explains overall logic of the analysis. The events we will analyze include:

- Katrina with the actual system performance along with a suite of consequences: economic (Econ-0), Social-Cultural (Soc-0), Human Health (Hum-0) and Environmental (Env-0) based on pre-Katrina New

Orleans (Orleans, Plaquemines, St. Bernard, St. Charles and Jefferson Parishes).

Table VII-6

Event	System Performance	Consequences	Conditions Pre-Katrina New Orleans	 Post-Katrina New Orleans 1	 Post-Katrina New Orleans 2
Katrina	Actual	Economic	Econ-0	NA	NA
		Social-Cultural	Soc-0	NA	NA
		Human Health	Hum-0	NA	NA
		Environmental	Env-0	NA	NA
Katrina	System works as planned	Economic	Econ-1	NA	NA
		Social-Cultural	Soc-1	NA	NA
		Human Health	Hum-1	NA	NA
		Environmental	Env-1	NA	NA
Other	Probabilistic	Economic	NA	Econ-2	Econ-3
		Social-Cultural	NA	Soc-2	Soc-3
		Human Health	NA	Hum-2	Hum-3
		Environmental	NA	Env-2	NA

- Katrina, assuming that the floodwalls worked as planned with the same suite of consequences based on pre-Katrina New Orleans.
- Other probabilistic scenarios about the system (provided by the Risk and Reliability Team) based on Post Katrina New Orleans as of June 1, 2006 Post-Katrina New Orleans 1 is one such scenario. Economic consequences (Econ-2) will be limited to the direct economic impacts to the City of New Orleans). Human Health (Hum-2) will be limited to loss of life supplemented by qualitative analysis. Social consequences (Soc-2) will be discussed in qualitative terms. The analysis of environmental consequences (Env-2) will be symmetrical throughout the scenarios. The same approach does not apply to Post Katrina New Orleans 2, another scenario supplied by the Risk and Reliability Team.

Thus far, most of the work has been on conceptualizing the problem, writing contracts, coordinating with the other teams and collecting data. A sample of results is available for direct economic impacts and for social cultural; none are yet available for human health and safety.

The environmental impacts team, which has been functioning the longest, reported preliminary findings. Based on the existing data and its analysis to date, there is no evidence of significant impacts on fish or wildlife associated with levee failure or the dewatering of the flooded area by pumping on fish, macro-invertebrate, or shellfish populations of Lake Pontchartrain, Mississippi Sound, or the offshore waters of the Northern Gulf of Mexico. This conclusion is tentative and may need to be revised as additional evidence is accumulated. There is presently only enough information to suggest that benthic assemblages in the immediate proximity of active pumps were impacted by levee failure and dewatering. Information from a follow up study should clarify this issue. Impacts further afield (Lake Borgne and Lake Pontchartrain) will have to await the analyses from EPA's National Coastal Assessment program and from ERDC. However, wetlands within the flood protection system were impacted by high salinity associated with breached/overtopped levies within St. Bernard Parish. The degree of ecological impact is yet to be determined.

The sections below describe the efforts underway, provide some initial quantitative and qualitative results thus far, and outline the status of remaining efforts.

Summary of Work Accomplished

As generally defined for the IPET mission, the economic consequence analysis is being developed to investigate various scenarios associated with hurricane Katrina and the possible future occurrence of similar or more severe storms. Specific to occurrence of Katrina, two scenarios involve the assessment of flooding and inundation with subsequent physical and economic consequence for storm conditions as they transpired on 29 August, 2005. One scenario examines economic consequences due to physical levee or floodwall failure as it actually happened. Another scenario examines how consequences would have differed assuming performance of the levee and floodwall system commensurate with its intended level of protection. Additional scenarios involve assessment and evaluation of what will be at risk as of June 1st at the beginning of the 2006 hurricane season in relation to varying sets of conditions for possible future storms and potential for levee or floodwall failure in different reaches of the levee\floodwall system.

Requirements for consequence analysis involve estimation of direct, indirect, and induced economic impacts of storm effects with regard to flooding and inundation and related costs or damages associated with varying scenarios. Isolating the flooding and inundation related costs will require estimation of wind-driven damages so that the marginal economic costs or value of the levee\floodwall system can be determined.

To date, an extensive review of economic models available to assess indirect and induced market impacts produced a consensus agreement that none are available that is readily applicable to the circumstances of Katrina. This is due to the significant change or transformation of regional economic relationships. These changes have occurred due to widespread catastrophic loss or disruption of business sector activities and community\public activities combined with extensive displacement of households and workforce. It is also due to remaining uncertainties of residents, planners, officials, and investors on issues such as re-evaluation of risks, the pace and priorities of recovery, and numerous other issues fundamental to understanding and predicting market outcomes. Tentatively (and within the constraints of IPET), it has been determined that the approach for evaluating indirect impacts can be facilitated using a simplified economic base model approach possibly combined with adaptation of an economic impact model designed to address in-migration and out-migration or loss of workforce and physical capital assets. To be comprehensive, this will be done using a consistent multi-regional national modeling framework. Where practical, all modeling work will be calibrated using the detailed data to be provided by the Louisiana Department of Labor (described further below).

Interim Results

Some preliminary estimates of direct damages from inundation of structures and content have been assembled for primary areas of four parishes (Orleans, St. Bernard, Jefferson, and Plaquemines) based on availability of GIS grids topography and inundation. Presently available grids include areas of Central New Orleans, Mid-City, Old Metarie, New Orleans East, Lower 9th Ward and Plaquemines Parish communities. Figure VII-10 shows the spatial distribution for the preliminary estimates of residential damage in thousands of dollars by census block. These estimates will be cross-checked with other source data that is currently being processed. Updates will be made when more reliable estimates warrant changes.

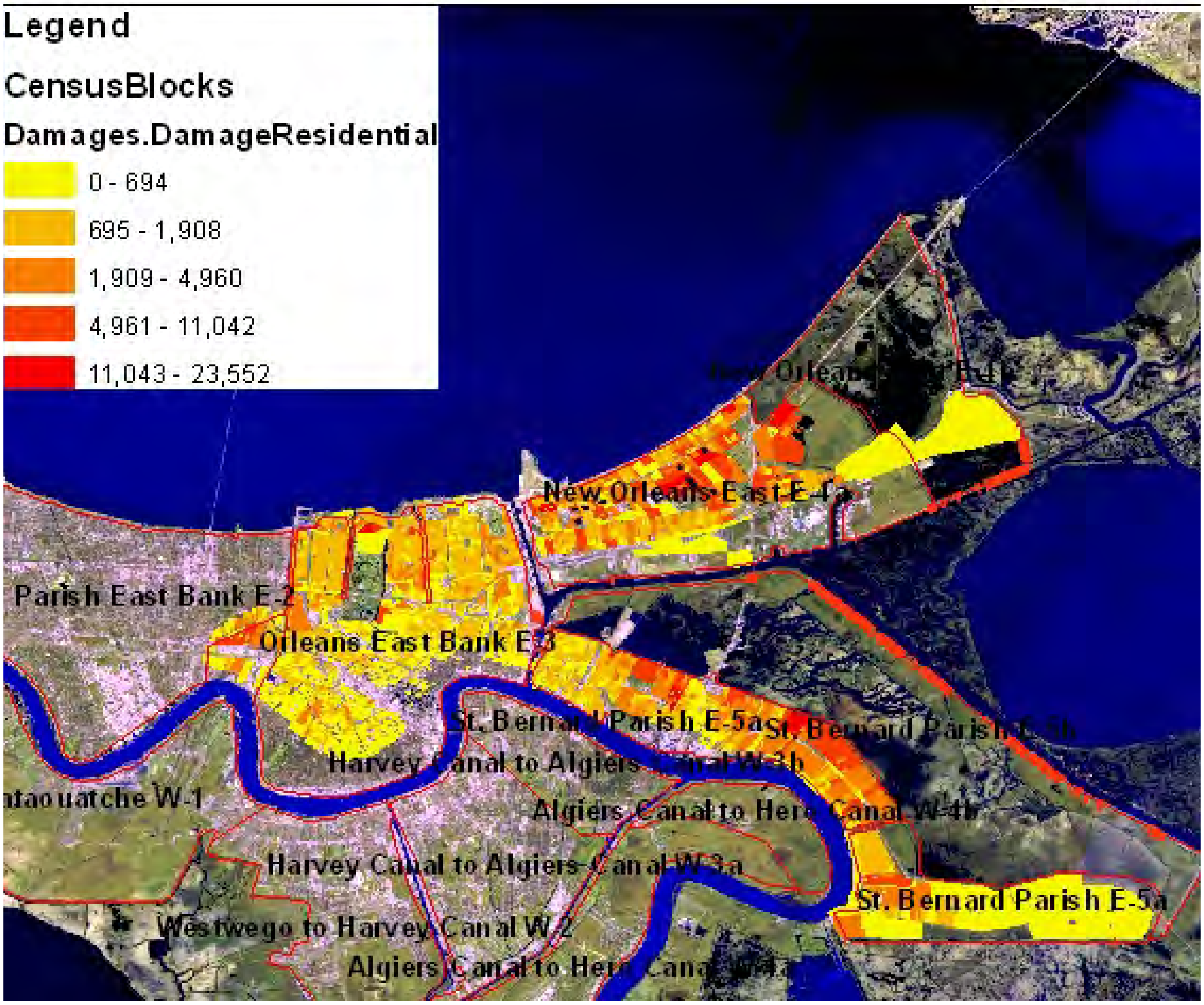

Figure VII-10. Preliminary Estimate of Residential Losses by Census Block

To date, flood damage estimates have been compiled according to aggregate categories of residential structures, commercial structures, and vehicles. Table VII-7 shows the preliminary estimate of total damage in thousands of dollars by parish and zip code for the areas shown in Figure VII-10. Because these values are preliminary and will be revised, a discussion of these figures at this stage would be pre-mature.

Table VII-7
Preliminary Estimate of Number and Damage (in millions of dollars) by Major Categories by Parish and Zip Code

Parish	Zip Code	Residential Damage	Number Housing Units	Non-residential Damage	Number Non-residential	Vehicle Damage	Number Vehicles
Jefferson	70001	$84,872	1,178	$7,757	54	$22,045	1,855
Jefferson	70005	$130,628	1,746	$7,432	190	$73,803	6,152
Jefferson	70121	$30,142	1,105	$7,586	107	$22,113	2,424
Parish Total		**$245,642**	**4,029**	**$22,775**	**351**	**$117,961**	**10,431**
Orleans	70112	$34,469	612	$296,314	644	$57,271	4,297
Orleans	70113	$68,653	2,074	$62,532	431	$99,376	9,272
Orleans	70115	$234,050	4,651	$47,038	772	$249,265	19,451
Orleans	70116	$119,777	3,177	$20,631	354	$100,223	8,007
Orleans	70117	$958,703	13,632	$61,583	680	$326,898	22,694
Orleans	70118	$384,094	7,227	$61,230	625	$259,985	19,302
Orleans	70119	$728,308	13,819	$197,688	1,612	$478,061	31,644
Orleans	70122	$1,349,469	16,408	$93,716	976	$487,073	32,769
Orleans	70124	$1,006,723	10,609	$56,183	504	$267,745	18,059
Orleans	70125	$419,207	6,556	$98,485	900	$313,229	20,202
Orleans	70126	$1,147,323	12,041	$226,037	1,092	$443,987	29,371
Orleans	70127	$858,781	8,001	$319,068	794	$305,107	19,954
Orleans	70128	$778,302	6,541	$113,513	480	$220,146	14,292
Orleans	70129	$293,981	2,613	$129,478	341	$101,267	6,545
Orleans	70130	$35	2	$412	134	$313	160
Parish Total		**$8,381,874**	**107,963**	**$1,783,909**	**10,339**	**$3,709,945**	**256,019**
Plaquemines	70041	$47,894	1,374	$11,074	61	$29,873	2,413
Plaquemines	70083	$6,510	233	$2,093	18	$7,105	675
Plaquemines	70091	$1,137	66	$1,141	2	$1,036	79
Parish Total		**$55,541**	**1,673**	**$14,308**	**81**	**$38,015**	**3,167**
St. Bernard	70032	$272,904	3,629	$67,242	300	$96,294	6,251
St. Bernard	70040	$7,014	109	$139	3	$1,757	126
St. Bernard	70043	$1,076,986	11,227	$139,754	762	$240,248	16,439
St. Bernard	70075	$421,454	3,274	$21,203	99	$69,937	4,565
St. Bernard	70085	$110,995	2,107	$12,338	50	$36,660	2,469
St. Bernard	70092	$378,715	3,983	$15,613	112	$70,376	4,652
Parish Total		**$2,268,068**	**24,329**	**$256,291**	**1,326**	**$515,273**	**34,502**
Grand Total		**$10,951,124**	**137,994**	**$2,077,283**	**12,097**	**$4,381,193**	**304,119**

Status

Compilation of data for more detailed estimates of commercial and public-sector damages is partially complete with some preliminary information becoming available for costs of repair and restoration of infrastructure such as distribution sub-grids for electrical services. Efforts are also in progress to facilitate the sharing of business sector data by the Louisiana Department of Labor (LDL). The LDL compiles data at the individual business level concerning physical location of business, type or classification of business, wages, and employment that is not disclosed publicly (due to requirements for confidentiality). This information is crucial to developing viable estimates of business sector inventory and estimation of direct economic impacts of Katrina within the schedules mandated for IPET. For this reason, the IPET has arranged an acceptable use of pre-Katrina LDL data that is consistent with the confidentiality restrictions (i.e., calendar year 2001 through the second Calendar year quarter of 2005) and post-Katrina, third and (time permitting)fourth quarter employment and wage data.

The USACE District Office in New Orleans has worked extensively to develop and apply GIS systems that enable economic analysis at the Census Tract and Block level and continues to integrate imagery and topographic data to refine economic analyses. This includes efforts to link the GIS product to hydraulics and hydrology modeling and analysis supported by other Tasks under IPET. It is anticipated that this methodology for tabulation of economic impacts of Katrina will be in place by late April and will facilitate scenario analysis conducted by the risk analysis team, task 10 of the IPET.

Activities to Complete

In addition to the completion and cross-checking of the activities already described, work will soon begin to investigate and compile data on the costs of Katrina to waterborne navigation. This includes estimation of damages to the waterway system and costs due to service loss caused by obstruction(s) and requirements for post storm dredging. Further, assessment of impacts on connecting inland waterway and deep-draft vessel services and port facilities will be carried out. It is anticipated these efforts will be largely completed by late March to early April.

Employment and wage data provided by the LDL will require tabulation of aggregates by location and business activity. As the 3rd and 4th quarter 2005 data becomes available, job loss and displacement statistics can be compiled. In addition, a review of the historical LDL data will be carried out to assist in the calibration of impact models and possible evaluation of trends or expected changes in economic relationships that will be analyzed via the Risk and Reliability Team.

Social, Cultural and Historic Consequences

The Social Problem and Objectives

The important consequences go beyond the event’s physical damage. New Orleans is a unique community and this community has and will continue to experience unprecedented social and cultural change with regional and national implications. Social Consequences of the event, the impacts, the aftermath and the rebuilding is and will continue to be a key factor in decision making about future hurricane protection.

The objective is to address Social Cultural and Historic Consequences of Hurricane Katrina and the Levee Performance in New Orleans as follows:

- Quantify key parameters reflecting the social conditions in neighborhoods prior to the event; Include those parameters reflecting vulnerable populations (age, gender, income, disabilities, ethnic minorities, vulnerable groups)
- Quantify impact and consequences of the event on those neighborhoods, communities, and parishes
- Identify key institutions and changes in the functioning of those institutions as a consequence of the event
- Identify the cultural historic consequences of the event by social areas of the study region (neighborhood/significant sites)

Summary of Work Accomplished

A group of academic and other applied social scientists have been assembled to assist in developing a work plan. The group met in New Orleans for this purpose in January 2005, and a comprehensive work plan was finalized in early February. Working as a team, tasks outlined in the work plan have been delegated based on areas of team expertise. The plan adopts a conceptual framework for understanding the impacts of the event that is based on methods found in the current disaster research literature. Much of the current efforts are focused on compiling the relevant data sets. In addition, a number of other scholars and agencies are examining the Katrina event, and the team is reviewing methods, data and analysis that is being used in these efforts. Some preliminary information has assisted the team in directing its work.

Interim Results

Katrina was a unique event both in term of scale, response and the uncertainties of recovery. Some of the Parishes impacted by the Katrina event were in population decline between the U.S. Bureau of Census count in 2000 and Pre-Katrina 2005. Orleans Parish had a 5 percent decline in less than 5 years prior to Katrina while Plaquemines is estimated to have 2 percent decline over the same period. The Katrina event had the greatest impact on the Parishes in terms of the number of people. Orleans Parish had a pre-Katrina population of 485 thousand residents and a January 2006 estimated population of 171 thousand. Estimates indicate since the immediate impact (October 2005) 80 thousand residents have returned in interim recovery period (January 2006).

Researchers at Brown University indicated that the population of damaged areas was nearly half African American, living in rental housing and disproportionately below the poverty line. In the City of New Orleans, 75 percent of those living in impacted areas were African American, 29 percent poor and 52 percent renters. An estimated 5.7 percent of those residing in the New Orleans damaged area were over 65 years of age with one or more disabilities. In examining the mortality data made available by the State of Louisiana, the average age of those losing their life as result of the event was over 65 years old. These initial results indicate that the event hit areas that were highly vulnerable.

On an institutional level, the Brookings Institution indicates that only 32 percent of the New Orleans hospitals are open. Only 15 percent of the schools have reopened with over 9,000 students enrolled. Universities in the city are open, but have not fully regained pre-Katrina functionality. Electricity has been restored to about 95 percent of former customers, but only between 30-35 percent of the customers have either not returned or are unable to reconnect to the system. Based on this interim information, the institutions serving the city remain overwhelmed.

Status of Remaining Efforts

The team is evaluating data sources and developing more refined information of the social characteristics in the Parishes and neighborhoods. Data from the New Orleans Planning Department are being generated. Data on institutions will also be collected from a variety of existing sources based on the service areas of the institutions. Qualitative data on historic and significant cultural areas of the metropolitan region is being collected from community leaders and those experts in the cultural history of the areas. Data on institutions will also be collected from a variety of existing sources based on the service areas of the institutions. Field observations will begin in early March 2006, to collect indicators of residential and business reoccupation. All data will be organized within a Geographic Information System format.

The final analysis will be completed by June of 2006 with key parameters and summaries being made available in April 2006. The final report will be organized on a neighborhood by neighbor basis, providing both pre- and post-conditions with a discussion of social conditions in the one to five year time frame.

Human Health & Safety Consequences

Purpose

The human health and safety consequences assessment partially addresses the following IPET question included in the December 6, 2005 ASCE comments on the IPET detailed scope of work:

What were the societal-related consequences of the flooding and hurricane damage, and what are the future societal-related risks that will be faced in New Orleans following reconstruction?

To answer this question for human health and safety consequences, this subtask is proceeding on two somewhat independent tracks.

The first track is characterizing potential human health and safety impacts on New Orleans residents resulting from the actual Katrina event. This effort is largely descriptive and involves no original quantification of impacts. Rather, the effort seeks to identify and describe the most important actual and potential human health and safety impacts of Katrina, including susceptible population subgroups, timing of impacts (onset, duration), possible effects on others, and other relevant information. Hard data on fatalities is being gathered from original sources; however, it is not anticipated that quantitative estimates of morbidity cases will be available from external sources and no attempt will be made to estimate them by the study team. Rather, the study will identify external studies that are attempting to quantitatively estimate morbidity, and pull hard data from those studies to the extent that they are available and releasable.

The second track, which addresses the primary objective of this subtask, is to estimate potential flood-related mortality risks in greater New Orleans under different post Katrina risk scenarios (i.e., event-performance-flooding scenarios). This includes a scenario where the flood/hurricane system performed as designed as well as others developed by the Risk and Reliability Analysis Team.

The LIFESim dam/levee failure life loss model has been selected for modeling flood-related mortality. The main advantages of this model are 1) it can be run in uncertainty (probabilistic) mode that best facilitates mortality assessment due to flooding resulting from levee breaches and overtopping, 2) it includes a sophisticated warning and evacuation (including mobilization and transportation) module, and 3) it links to readily available data sources, including USGS topographic data (as adjusted based on IPET LIDAR controls), census tiger data on population and road network, and building data from HAZUS-MH database.

Summary of Work Accomplished

A conceptual framework for adapting and calibrating the LIFESim model to the New Orleans context has been developed. This calls for adding a rescue module to the model that draws on the actual experience in the Katrina event that will be used to account for escape/rescue of the population at risk (those who do evacuation prior to the event). The framework also identifies the need for flood routing input data that reflects not just maximum flood water elevations for Katrina flooding and post-Katrina risk scenarios, but also rates of inundation, and possibly velocities.

- Links with the social/cultural subtask have been developed for sharing of data on the Katrina rescue profile and pre- and post-Katrina demographic data.
- Links have been developed with the Risk and Reliability and Interior Drainage Team efforts to ensure flood routing data is developed in the required form.
- Publicly available data on flood-related fatalities, by parish, has been developed from numerous sources, some of which includes relevant

demographic data (e.g., age). Links have been established with the LSU Hurricane Center in an effort to improve the fatalities dataset with information developed by the Center.

- GIS data layers for pre-Katrina road network and buildings have been developed.
- Several external studies that are attempting to quantify morbidity impacts of Hurricane Katrina have been identified and preliminarily characterized.

Interim Results

At this time, no interim results have been generated.

Status of Remaining Efforts

- Adaptation of the LIFESim model to the New Orleans context is proceeding.
- Calibration of the LIFESim model awaits the availability of Katrina flood routing data in the appropriate hydrograph (time-dependent) format, as well as a more complete dataset on Katrina-related fatalities.
- Development of GIS data layers for post-Katrina demographics is nearing completion.
- Application of the LIFESim to estimate fatalities for post-Katrina risk scenarios awaits completion of adaptation and calibration of the GIS model, development of all relevant post-Katrina GIS data layers, and data on the provision of flood hydrographs for representative event cases for the risk and reliability analysis.

Environmental Subtask

Summary of Work Accomplished

The environmental subtask of the consequences effort examines the direct, intermediate and long-term environmental consequences (ecological resources degraded and environmental benefits lost) stemming from events associated with Hurricane Katrina; in particular, the local impacts from flooding within the hurricane protection system on ecological resources, including species of ecological and economic value, and pest species that might threaten human comfort, health and safety. It also examines impacts to the integrity of habitat and communities —the ecosystems—supporting resource and pest species. This was achieved primarily through assembly and analysis of data collected by other responsible federal and state agencies, universities, and other reputable organizations. Additional original data were collected on site and analyzed by personnel at the Engineer Research and Development Center of USACE at Vicksburg, MS. Based on subtask results, a forecast is to be made of the environmental consequences resulting from the same flooding, breaching , pumping and other Katrina-related events given the environmental conditions that are expected as of June 1, 2006.

Once the subtask problem was clearly defined, a concept model of the potentially impacted ecosystems and ecological resources were developed (Figure VII-11). The geographical boundaries of the potentially impacted ecosystems were defined at two primary levels: the "inner ecosystem" inside the flood-protection system in Orleans, Jefferson, Saint Bernard, Saint Charles and Plaquemines parishes and the "outer ecosystem" outside the flood-protection system and within reach of flood effects and flood-water management.

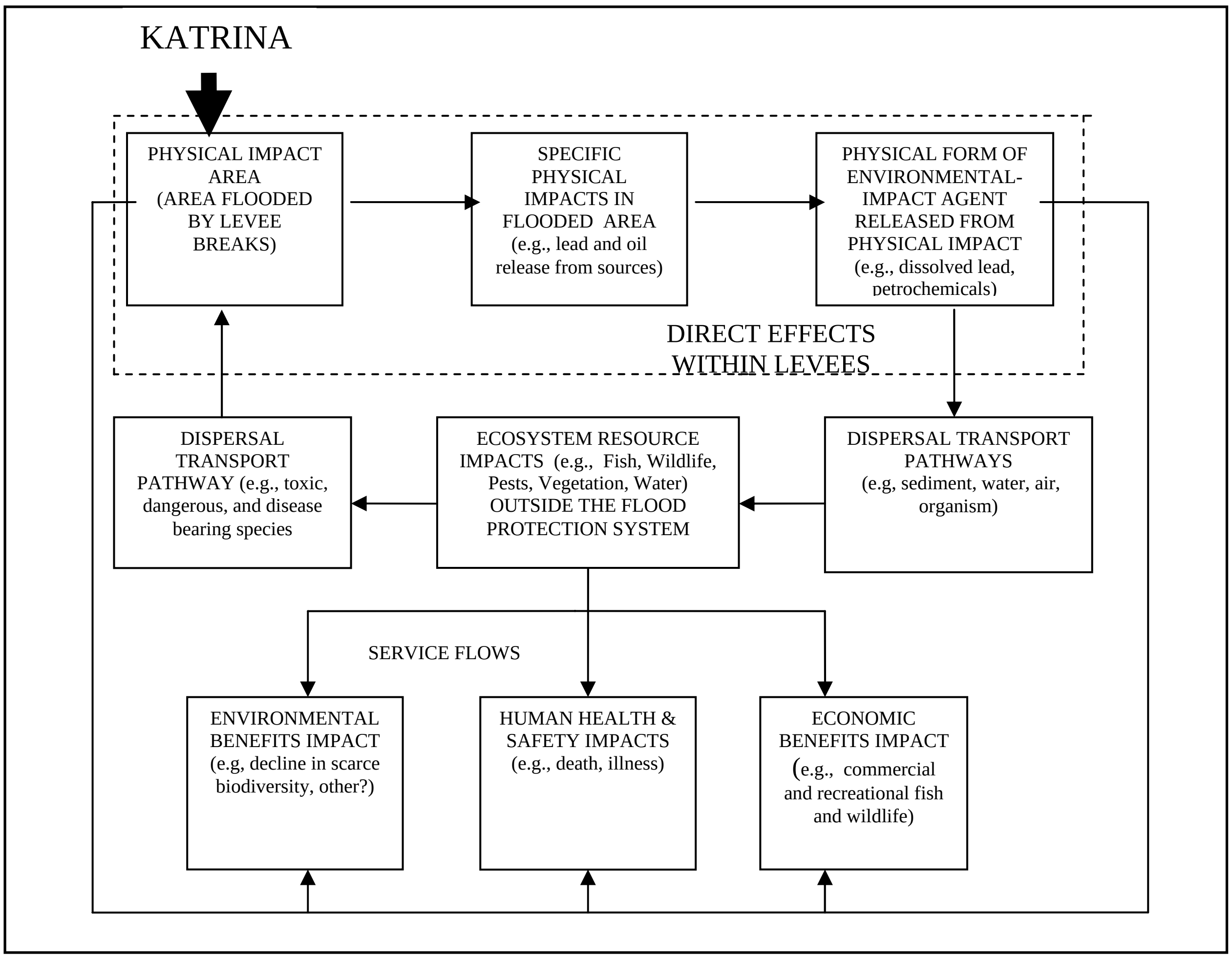

Figure VII-11. Conceptual model of the ecosystem area directly impacted by flooding and the area externally impacted outside the flood-protection system by indirect impacts

Because ecosystems are hierarchically organized, the inner and outer ecosystem includes a mix of smaller ecosystems defined by the biotic communities and their associated habitats. Marine, brackish and freshwater aquatic ecosystems and wetlands, as well as uplands are found in both inner and outer ecosystems. Critically important in determining the dimensions of the outer ecosystem and impacts from flooding and flood management, was identification of all hydrologic and other connection among ecosystems, including, primarily, levee overtopping and breaching, and flood water pumping. Aerial and overland connectivity was also considered to the extent it was related to flooding and flood-water management.

The inner ecosystem included all ecosystems within the flood-protection system, which included 13 sub-basins; i.e., polders (Figure VII-11). Within the flood protection system, the analysis concentrated on the direct effects of water level, water quality, and residual quality of soils after flood recession. It included the impact of salt-water intrusion by water pumping, levee overtopping, and levee breaching on wetlands and native species dependent on waters of lower salinity. From the perspective of total area, the most significant inner ecosystem wetlands existed in the Saint Bernard E-5b polder (Violet Marsh). Another important area, **Bayou Sauvage National Wildlife Refuge, is situated nearby; most of it in Polder E-3 (Orleans East Bank).** The subtask also addressed the impacts of possible insect, reptile, rodent and other pest species as vectors of threat to the comfort, health and safety of people within the area protected by system levees (information provided to the health and safety subtask leader).

Definition of the outer ecosystem focused on the possible transport of chemical and biological contaminants from within the flood-protection system and possible uptake in resource species including shrimp, oysters, fin-fishes, state threatened and endangered species, and supporting habitats. Areas that were breached and topped by storm surge were identified by maps and aerial photography, and by model results obtained from other IPET team efforts. The possible contaminants include toxic metals, organic synthetics, and indicators of fecal contamination considered a health threat by the Environmental Protection Agency at levels over posted standards of acceptability. Possible physical impacts were also considered. Within the outer ecosystems, lakes Pontchartrain and Borgne were separated from the more remote near-shore gulf and delta regions based on relative probability of impact from flood-protection system failure. Lastly, we considered the possible transport of pest species out of the inner ecosystem, including, especially, the invasive Formosa termite moved with debris from flooded areas.

While some of the lost environmental benefits considered here were confined to the region of direct impact in the inner and outer ecosystems, indirect loss of national benefits, primarily in the form of federal endangered and threatened species and their support ecosystem were also considered.

For each case, and to the extent data were available, pre-Katrina background information is provided to contrast with post-Katrina conditions occurring as if the flood-protection system performed as planned and occurring as a consequence of the flood protection system failures. These post-Katrina effects from wind, contaminants transport from other areas, and other general storm damage were sorted from the specific post-Katrina impacts of flooding and flood management in the flood protection system for the Greater New Orleans area.

Hydrologic pathways included the pumping of possibly contaminated flood waters into Lake Pontchartrain and other ecosystems adjacent to and within the flood protection system. Contaminants could possibly include metals, synthetic organics, nutrients, and pathogens indicating human fecal or other possible vectors of human health threat. Contamination is determined by concentrations greater than acceptable standards. Pumped flood waters or waters moving over top or through breaches in levees also may have been altered in other ways that could be stressful to communities in receiving ecosystems, such as different concentrations of salt and oxygen.

Within the flood protection system our concern for natural ecosystems was most focused on Violet Marsh, which occupies nearly all of Polder E5a in Saint Bernard Parish and one of the most important wetlands between the city of New Orleans and the Gulf. Changes in its composition were not only of environmental concern, but could influence future flood protection system performance. Of particular interest in that regard were possible effects of salinity on the vertical structure; i.e., the amount of forested wetland verses lower lying herbaceous wetland (e.g., grasses, rushes, sedges). The Bayou Sauvage National Wildlife Refuge nearby was also a concern.

Other environmental impacts may have also occurred through aerial and human-transport vectors. This included dust blown from the flooded impact site once the flood water receded and flying pest species, such as mosquitoes. In addition, some pests may have dispersed by land. These included the noxious invasive species, the Formosa termite, which could have been dispersed with debris among the flooded ecosystems and to outside terrestrial ecosystems. In addition, rodent vectors of disease and property damage move by land ahead of the rising waters, and may have colonized unflooded areas within the flood protection system.

In addition to undesirable connection, the management of flooding could conceivably cause physical barriers to natural movements of desirable species. This could occur from chemical alteration of connecting waters, for example, or temporary or permanent changes in levees or other physical structure.

In keeping with the boundaries of ecosystems defined in the concept model, the results are reported by ecological resource categories and pest species for near-shore gulf and delta ecosystems most remote from impacts, the exterior ecosystems most likely to be impacted outside the flood-protection system, which are lakes Pontchartrain and Borgne, and the interior ecosystems within the flood-protection system.

Interim Results

Outer Ecosystems

Near-shore Gulf and Delta Ecosystems

Fisheries. Fall of 2005 trawl surveys found no indication of reductions in offshore fish or shrimp populations or fish kills (SEAMAP program, *www.gsmf.org*). Details will be provided in final report.

Immediately following Hurricane Katrina, NOAA, EPA, USGS and Dauphin Island Marine Lab assessed potential contamination levels present in inshore and offshore water, sediment and fish and shellfish tissues. Bacterial contamination (E. coli, Enterococcus, or Vibrio cholera and other V. spp.) in water and sediments from Mississippi Sound and offshore areas did not exceed EPA standards for recreational waters (Peterson et al. 2005a and 2005b). Of other Vibrio species that were encountered, concentrations were not beyond those expected under normal (pre-Katrina) conditions.

Persistent organic compounds (PCB's and DDT's) and polycyclic aromatic compounds (PAC's) in fish tissues did not exceed FDA standards for consumption (Krahn et al. (2005a and 2005b).

Elevated bacterial concentrations in mussels consistent with a storm runoff event have been reported by NOAA's Status and Trends – Mussel Watch program immediately after Hurricane Katrina (*http://ccma.nos.noaa.gov/cit/katrina/prelim.html*). Subsequent sampling by the EPA and the Mississippi Department of Environmental Quality (EPA-DEQ 2005) found few instances of elevated bacterial concentrations or priority pollutants in Mississippi waters and the States of Mississippi and Louisiana and the Food and Drug Administration have all issued news releases indicating that seafood, including oysters are now safe to eat (*www.fda.gov/bbs/topics/NEWS/2005/NEW01271.html*). However, quantitative data has yet to be found.

Wildlife. The information so far discovered to evaluate potential impacts to wildlife populations is scarce and of limited utility. At least eight national wildlife refuges were closed as a consequence of the storm *http://www.fws.gov/southeast/news/2005/r05-098.html* but little information has been provided about wildlife impacts. Most areas of the refuges that were previously open to the public have reopened *http://www.fws.gov/southeastlouisiana/Katrina.htm*. The loss of sea turtle nesting sites along the Alabama coast, Alabama beach mouse dune habitat, and red cockaded woodpecker habitat in Noxubee National Wildlife Refuge (New Release Sept 9, 2005, *www.fws.gov/southeast*). These locations are remote from the flood-protection system and not likely to have been impacted by levee failure in any measurable way

Several threatened and endangered species have been observed in nearshore gulf and delta ecosystems. These include the West Indian Manatee Trichechus manatus, Atlantic Ridley Lepidochelys kempiisea (a sea turtle), piping plover Charadrius melodus, brown pelican (Pelecanus occidentalis), bald eagle (Haliaeetus leucocephalus), gulf sturgeon Acipenser oxyrinchus desotoi, and Louisian Quillwort Isoetes louisianensis. Little information on status of these species has been reported as yet.

Lakes Pontchartrain and Borgne

Fisheries. Information on fish and benthic invertebrate populations before Hurricane Katrina is available through the EPA's EMAP program (*http://www.epa.gov/emap/index.html*). Data from 2005 or 2006 are being analyzed (will be assembled in a Table for pre-Katrina assessments). Other information on the fish assemblages of Lake Pontchartrain, including a recent assessment of fish-habitat relationships, is summarized in O'Connell et al (2004) and in the University of New Orleans Vertebrate Museum's database of fish collections in Lake Pontchartrain (*http://www.nekton.uno.edu/about.htm*).

Both the Mississippi Department of Marine Resources (DMR) and the Louisiana Department of Wildlife and Fisheries report significant physical damage to oyster beds due to scouring, sedimentation, and debris deposition (*http://www.dmr.state.ms.us* and personal communication, Marti Bourgeois, LA DWF). However, quantitative data before and after the Hurricane have yet to be discovered.

Immediately after Hurricane Katrina, Louisiana Department of Environment Quality (LADEQ) (2005) found high bacterial counts in the water on the northern shore of Lake Pontchartrain; however, concentrations of organic contaminants were generally below water quality standards. In addition post-storm water quality assessments revealed significant low dissolved oxygen conditions and fish kills along the northern shore of the lake, but attributed these results to the storms and not to pumping of the floodwaters from the flood-protection system (*http://www.deq.louisiana.gov/portal/portals/0/news/ pdf/Post-KatrinaWaterQualityAssessment9-20-05.ppt*). LA DEQ (2005) also anecdotally noted there were "numerous bait fish and mullet" and live crabs in the lake following the Hurricane.

Additional post-storm benthic data is available from a one-time sampling of Lake Borgne conducted in late November 2005 by ERDC (Ray 2006, unpublished). The sampling effort collected sediment grain size, infaunal, and water quality data to assess foraging habitat of the endangered Gulf Sturgeon (Acipenser oxyrinchus desotoi). This information is presently being analyzed.

Many of the threatened and endangered species that occur in the near-shore gulf and delta region also occur in lakes Pontchartrain and Borgne. Of these, the gulf sturgeon population has been most closely monitored in the area. At the time of the storms most sturgeon were in their summer resting areas well away from the New Orleans in the Pearl and Bogue Chito Rivers. Of 40 fish carrying telemetry tags, none have been located since the storm. The other threatened and endangered species occur incidentally and seasonally, and there is no reported knowledge of their occurrence since Katrina in the region under study. The endangered Gulf Sturgeon may have been significantly impacted by the hurricanes, but probably not by the levee failure and dewatering process.

Development of a contaminants movement model out of the inner ecosystem to Lake Pontchartrain is nearly complete. Early evaluations of arsenic transport indicate that contaminants are quickly dispersed and diluted in Lake Pontchartrain to concentrations below EPA standards. Other contaminants, including indicators of human pathogens have yet to be analyzed with the model.

Inner Ecosystems

Contaminants. There are no important fish populations in inner ecosystem waters. However, there are important wetlands, which were sampled by ERDC and important associated wildlife populations. The central repository for Katrina-related water and sediment quality data is EPA's STORET Katrina Central Warehouse. Because of the availability of the data, only fecal coliform data are being supplied to the modeling team to evaluate pathogen transport out of the system into the surrounding ecosystem environment. Fecal coliform measurements in the flood water of New Orleans routinely exceeded water quality standards based on data obtained by EPA and LADEQ. Some metal concentrations were above standards set by EPA in the flood waters but do not differ substantially from typical storm-water runoff. Because of the volume of water pumped, however, these data raised some concerns about potential environmental impacts and public health threats resulting from the failure of the levees and were responsible in part for applying a contaminants transport and fate model to assess impacts outside the flood-protection system.

Sampling conducted by ERDC in Violet Marsh was designed to compliment and extend this fecal coliform data set. The data are providing a means to quantitatively evaluate the distribution of treated and untreated sewage in flood waters of St. Bernard Parish that were pumped into Violet Marsh from Polder E-5a. The flooded sewage treatment plant off Florida Avenue (at Dubreuil Street) and the oil spill from the Murphy Oil Company on Paris Road were selected as potential environmental contaminant sources (both in Polder E-5a of Saint Bernard Parish). Samples were then taken on transects some distance from these pumps into Violet marsh. Sediments from the top of each core were sampled to derive information on recently deposited material while deeper sediments were sampled to assess materials deposited prior to the storms. These have yet to be fully analyzed for fecal coliforms and ratio of coprostanol to cholesterol. Unlike water samples, sediment quality standards for microbes do not exist. Metal and organic contaminants are also being similarly analyzed.

Salinity changes and preliminary data on benthic invertebrates in Violet Marsh indicates that species composition of benthic assemblages in the immediate vicinity of active pumps suggest a history of recent disturbance consistent with higher salinities and other storm impacts probably associated with levee breaching along MRGO.

Wildlife. While no data on wildlife use has been described, undoubtedly Violet Marsh in Saint Bernard Parish gets similar wildlife use as a nearby refuge. Located within the City of New Orleans, Bayou Sauvage National Wildlife Refuge is the nations largest wildlife refuge (23,000 acres) and includes both fresh and brackish marshes. It includes "an enormous wading bird rookery from May until July… and tens of thousands of waterfowl winter in its…marshes" *http://www.fws.gov/bayousauvage/*. The Hurricane occurred during the interval between the seasons of greatest waterbird use. Damage, if any, to the freshwater wetlands has yet to be reported.

The condition of bald cypress is especially relevant to use as bird nesting areas (rookeries). Because it requires freshwater for survival, elevated salinities associated with levee failure may have stressed the cypress and perhaps killed trees. That condition is being monitored with the onset of the spring growing season along with impacts on marsh vegetation (mostly grasses, sedges, rushes).

No state or federal threatened and endangered species have been identified in Violet Marsh. However, bald eagle and brown pelicans occur in the Bayou Sauvage National Wildlife Refuge. Although other species might conceivably pass through the area, the probability of impact on the extant populations of all threatened and endangered species is likely to be small.

Pest Species. There has been considerable concern that the Formosa termite, Coptotermes formosanus, an invasive and destructive species, might be introduced to uninfected areas by rafting of the colonies on floodwaters or inadvertently with debris disposal. While much of the debris seems to have been retained in land fills within the flood-protection system, analysis of possible disposal outside the system and pre-Katrina range of the termite is underway. Other potential pest impacts are being sought out in useful data, but little has been found to date.

Status of Remaining Efforts

Analyses of sediment and water samples obtained by ERDC for ecosystems receiving flood waters are in the process of being completed. The contaminants model is being refined and used to simulate movements of metal, organic, and pathogen contaminants out of the inner ecosystem to Lake Pontchartrain. Additional unpublished data on wetland and wildlife condition and fisheries condition are being sought from responsible agencies. Assessment of existing ecosystem resource condition and impact by a June 1 hurricane event will follow upon compilation of all data.

Tables and Figures will be completed as all data becomes available. A preliminary list of Tables and Figures is listed below:

Tables.

1. Regional marine fish and sediment contamination results, pre- and post-Katrina
2. Wetland and barrier island area apparently lost after the storms—pre- and post- Katrina.
3. Areas and volumes of water in flood zone at observed flood peak in each Parish and in total, a performing flood system, Lake Pontchartrain, and Violet Marsh.
4. Pre- and post-Katrina live cypress and herbaceous plant cover and water salinity in violet marsh.
5. Differences in benthic invertebrates at pumped and un-pumped outfalls at violet marsh.

6. Lake Pontchartrain summary of pre- and post-Katrina fish, shellfish, water, and sediment pathogen and metal and organic contaminant concentrations.
7. Parish flood water summary of pathogen, metal, and organic contaminants.
8. Threatened and Endangered Species status summary for recent years including 2006.
9. Summary of likely impacts to ecosystems as of June 1, 2006 of an event like hurricane-Katrina.

Figures.

1. Conceptual model of the ecosystem.
2. Outer ecosystem potentially impacted defined by fisheries, contamination, and wetland data.
3. Inner ecosystem defined by the protection system structure around five parishes and the surrounding area.
4. Diagram showing ecosystem pathways of potential impacts through contamination of resources.
5. Contaminants model structure and 5 figures showing outer ecosystem distribution of pumped contaminants.
6. Debris dispersal to landfills, incinerators.

VIII. The Risk

Executive Summary

The mission of the IPET risk and reliability analysis is to examine the risks to life and property posed by the New Orleans hurricane protection system that was in place prior to Katrina and by the system as it is expected to exist at the start of the next hurricane season (1 June 2006). The risk analysis will consider the expected performance of the various elements of the system and the consequences associated with that performance. All engineered systems impose risks that result from humans using technology to create conditions or activities that are not produced by nature. For instance, the hurricane protection system in New Orleans has been designed to control interior flooding within New Orleans and protection to the city from storm induced surges and waves. The hurricane protection system (HPS) project is designed to perform this function without imposing unacceptable risks to public safety, property and welfare.

The risk analysis covers four states that represent the condition of the New Orleans hurricane protection system.

- The system as it existed before the arrival of Hurricane Katrina. Knowledge gained from IPET studies will be considered in the analysis.

- After Hurricane Katrina with repairs that have been completed prior to the 2006 hurricane season. Some projects may be ongoing after 1 June 2006.

- After Hurricane Katrina with all repair and improvement projects complete, but prior to longer-term increases in the authorized level of protection.

- The system as authorized before the arrival of Hurricane Katrina. All authorized components of the HPS are constructed and knowledge gained from IPET studies will be considered in the analysis.

The difference in relative risks among the three states will be a unified measure for fully evaluating the performance of the integrated system before Hurricane Katrina, after Hurricane Katrina, and during the interim recovery period.

Two groups of questions concerning the performance of the hurricane protection system (HPS) are addressed by the risk and reliability analyses:

Pre-Katrina: The system as it existed before the arrival of Hurricane Katrina. This state is the baseline for estimating risk, and includes the following:

1. What was the reliability of the hurricane protection system to prevent flooding of protected areas of the HPS that was in existence before the arrival of Katrina, for the standard project hurricane? Note that some components of the authorized projects had not been constructed prior to Katrina.

2. What was the reliability of the hurricane protection system to prevent flooding of protected areas with all of the authorization projects completed, for the standard project hurricane?

3. What is the estimated annual rate of occurrence of system failure due to hurricane events?

4. What are the probability distributions and annual rates of consequences that would result from failure of the hurricane protection system as defined in terms of life loss and economic impact?

5. What is the uncertainty in these estimates?

The pre-Katrina analysis does not attempt to recreate the design intent or knowledge that the designers used to determine the configuration of the HPS. Engineering parameters, foundation conditions and operational information gained by IPET through exploration and testing since the hurricane are used. This allows for an assessment of the actual risks that existed pre-Katrina. An additional analysis was conducted on the authorized HPS that includes all features in the original design that were not completed prior to Katrina.

Post-Katrina: After Hurricane Katrina with repairs made prior to the 2006 hurricane season, and during the interim recovery period after the hurricane protection system has been strengthened and improved, but prior to longer-term increases in the authorized level of protection. This group includes:

1. What is the reliability of the HPS to prevent flooding of protected areas for the authorized standard project hurricane with the system repairs and improvements in place as of June 1, 2006?

2. What is the frequency of flooding due to the range of expected hurricane events with the system repairs and improvements in place as of June 1, 2006?

3. What are the probability distributions and annual rates of consequences that would result from failure of the hurricane protection system as defined in terms of life loss and economic impact?

4. What is the uncertainty in these estimates?

The condition of the system has been degraded by the effects of hurricane Katrina. Flood walls and levees may have been overtopped, damaged by impacts from debris, saturated, submerged and/or breached. Permanent repairs on these elements have been accomplished since the hurricane that may have different material strength parameters than the original feature. This difference in strengths is considered in the analyses of component reliability. The pumping system was also damaged and shut down or submerged. The post Katrina reliability of the levees, flood walls and pumping stations will be considered in the risk assessment. The reliability of the various elements of the protection system will be determined using analytical and expert elicitation methods.

The term reliability is intended to mean the conditional probability of a component or system performing intended function. This result can also be used to determine the conditional probability of failure. System failure refers to the failure of the HPS to provide protection from flooding in one or more protected areas and can also be thought of as the occurrence of flood inundation. The effectiveness of the protection system is also dependent upon how well the operational elements of the system performed. Elements such as road closure structures, gate operations and pumping plants, etc. that requires human operation and proper installation during a flood fight can dramatically impact flood levels. The lessons learned concerning the performance of these elements during Katrina will be considered in the analysis.

The changed demographics of the local areas protected by the system will be considered when determining the consequences. In some areas, many homes and much of the infrastructure were destroyed by the hurricane and some may not be rebuilt. Therefore the pre-Katrina populations and property values will be impacted and must be considered in the post-Katrina analysis.

Risk is generally calculated by combining the probability of system failure with the consequences associated with that failure. For New Orleans, the post Katrina risks will be lower primarily due to reduced population at risk and lower economic activity. Consequences in terms of loss of life, however, are greatly dependent upon warning time and the effectiveness of the implementation of the evacuation plans. While recommendations may be made concerning evacuation planning, the effectiveness of plan implementation is beyond the control of USACE. In order to better compare the adequacy of pre and post Katrina HPS, probability of failure and inundation mapping will be used as the primary metric by which to measure the effectiveness of repairs and improvements. Coordination is ongoing with the consequence team to determine the manner by which loss of life calculations will be made.

Summary of Work Accomplished

Risk Model

Work accomplished to date has focused on development and testing of a spreadsheet template to be used for all polders and the associated mathematical relationships required to incorporate hurricane, reliability and consequence inputs. The risk model for the New Orleans East polder has been completed and will be used as the template for the remaining polders. All polder models are based upon the same basic event tree with some alterations to consider any unique features of the polder.

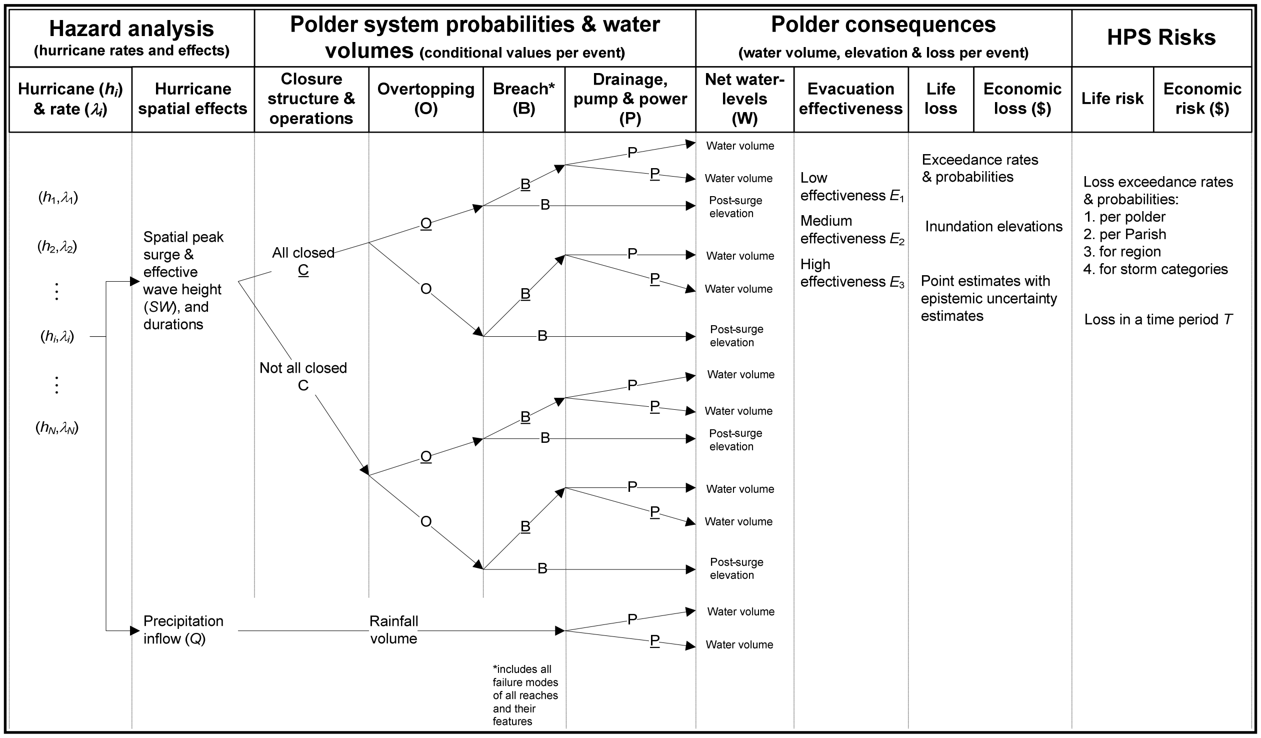

Figure VIII-1. Risk Model Event Tree. Underlined events (i.e., <u>*C*</u>, <u>*P*</u>, <u>*O*</u>, and <u>*B*</u>) are the complements of the respective events (i.e., *C*, *P*, *O*, and *B*).

System/Polder definitions

The physical characteristics of each polder must be accurately modeled in the risk model. The polder perimeter is separated into reaches that have similar levee cross-sections, foundation conditions and engineering parameters. The interior of each polder has been separated into sub-polders that are defined by the interior drainage and pumping systems.

The field work required to develop the system model for the New Orleans East polder has been completed and much work has been done on the other polders. A standard format for the collection of polder data was used to assemble the physical characteristic of the polder. Design Memorandums (DM) for each project were used to develop maps of each polder showing the location of critical features, stationing and elevations. Field trips were made to verify these maps and to note any changes made since construction. Team members traveled the entire perimeter of the polder and documented all critical features by taking GPS coordinates, photographing areas of interest and noting deviations from DM maps. A sample map for the New Orleans East polder is shown in the appendix.

The system consists of polders, sub-polders and reaches. The definition of these polders, sub-polders and reaches are based on the following considerations:

- Local jurisdiction,
- Floodwall type and cross section,
- Levee type and cross section,
- Engineering parameters defining structural performance,
- Soil strength parameters,
- Foundations parameters, and
- Surge and wave levels.

A sample of the spreadsheet model for a specific reach of the New Orleans East polder that has been developed based on the event tree is shown in Figure VIII-2. The figure shows the case of overtopping (OT) for a reach using several storms. Note inflow volumes are calculated for each hurricane run.

The primary goal of the risk model is to determine the volume of water entering each polder due to surge and wave overtopping, breaches and precipitation for each hurricane event. The perimeter of each polder is segmented into reaches that have similar characteristics as defined in Section 2.2. below. Since polders are made up of sub-polders based on interior drainage and pumping systems, the model must take into account the interflow between sub-polders. Inflow volume calculations are made for each reach and sub-polder and then aggregated to determine the total volume of water in each polder due to the hurricane event. Volumes will be post-processed with the topography of the interior of each polder to determine water elevations within the polder, and frequencies associated with each elevation. Water elevations within polders are determined using stage-storage relationships provided by the interior drainage modeling done by other IPET teams. A typical stage-storage curve is shown in Figure VIII-3.

Reach Number	1
Reach start-end stations	To be provided
Reach coordinates	To be provided
Equal allocation to Sub-Polder(s)	1
Reach length (ft)	2000
Reach elevation (ft)	16
Mean (Weir Coeff.)[1]	3
COV (Weir Coeff.)	0.2

[1]Use 3.0 for floodwalls, 2.6 for levees, and 2.0 for gates

Hurricane Runs			1								
Run	Rate (R)		Surge+Waves		Duration		OT Length		OT Probability	OT Volume (Weir Eq)	
i	Mean	StD*	Hs		T		L		P(OT)	V|OT	
			Mean	StD*	Mean	StD*	Mean	StD*		Mean	StD
ID	event/yr	event/yr	ft	ft	sec	sec	ft	ft		ft^3	ft^3
1	5.00E-04	0.00E+00	25	0	5400	0	2000	0	1.00E+00	8.748E+08	1.750E+08
2	5.00E-04	0.00E+00	25	0	5400	0	2000	0	1.00E+00	8.748E+08	1.750E+08
3	7.50E-04	0.00E+00	24	0	5400	0	2000	0	1.00E+00	7.331E+08	1.466E+08
4	1.00E-03	0.00E+00	23	0	5400	0	2000	0	1.00E+00	6.001E+08	1.200E+08
5	1.00E-03	0.00E+00	22	0	5400	0	2000	0	1.00E+00	4.762E+08	9.524E+07
6	1.50E-03	0.00E+00	21	0	5400	0	2000	0	1.00E+00	3.622E+08	7.245E+07
7	2.00E-03	0.00E+00	20	0	5400	0	2000	0	1.00E+00	2.592E+08	5.184E+07
8	2.00E-03	0.00E+00	19	0	5400	0	2000	0	1.00E+00	1.684E+08	3.367E+07
9	2.00E-03	0.00E+00	18	0	5400	0	2000	0	1.00E+00	9.164E+07	1.833E+07
10	2.00E-03	0.00E+00	17	0	5400	0	2000	0	1.00E+00	3.240E+07	6.480E+06
11	3.50E-03	0.00E+00	16	0	5400	0	0	0	0.00E+00	0.000E+00	0.000E+00
12	5.00E-03	0.00E+00	15	0	4320	0	0	0	0.00E+00	0.000E+00	0.000E+00
13	5.00E-03	0.00E+00	14	0	3600	0	0	0	0.00E+00	0.000E+00	0.000E+00
14	5.00E-03	0.00E+00	13	0	3600	0	0	0	0.00E+00	0.000E+00	0.000E+00
15	5.00E-03	0.00E+00	12	0	3600	0	0	0	0.00E+00	0.000E+00	0.000E+00
16	5.00E-03	0.00E+00	11	0	3600	0	0	0	0.00E+00	0.000E+00	0.000E+00
17	5.00E-03	0.00E+00	10	0	3600	0	0	0	0.00E+00	0.000E+00	0.000E+00

* Reserved for future epistemic uncertainty analysis

Figure VIII-2. New Orleans East Polder Model

Figure VIII-3. Stage-Storage Curve for Citrus

The spreadsheet includes tabs for various braches of the tree provided in Figure VIII-1. The results from all the simulated hurricanes are used to evaluate Eq. 8-1 with results illustrated in Figures VIII-4 and VIII-5.

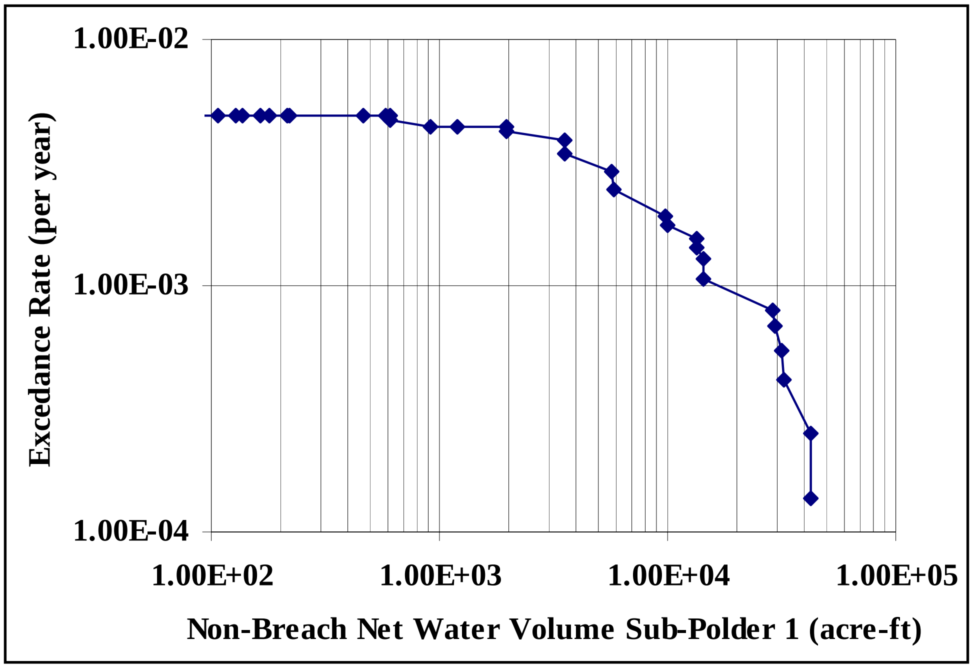

Figure VIII-4. Overtopping Risk Profile for Sub-Polder 1

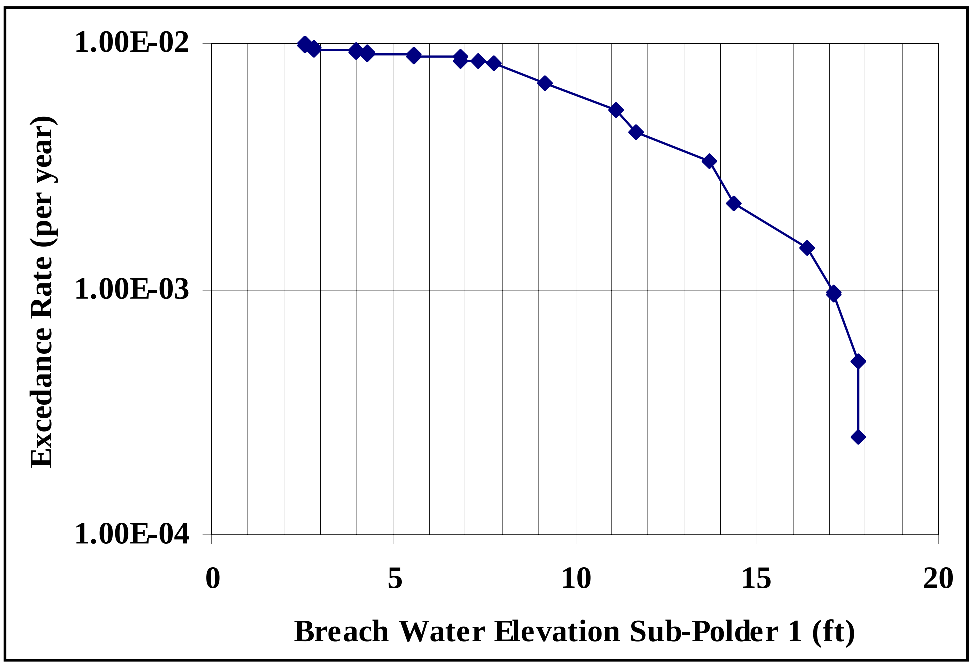

Figure VIII-5. Breach Risk Profile for Sub-Polder 1

Polder Geotechnical Subsurface Information

Geotechnical subsurface information has been collected from numerous borings and undisturbed samples found in the GDMs and from other USACE subsurface investigations. Polder maps will show the boring locations and data is presented in the forms of strip logs for each borings, profile cross sections under each levee section and laboratory test results for the continuous undisturbed samples. The laboratory test results include unconfined compression, triaxial (Q, R, and S) testing, Atterberg Limits and consolidation. This information has been carefully extracted and processed and incorporated into the reliability modeling. For each polder the subsurface geotechnical information will be interpreted from the test data to estimate statistical parameters and distributions and the spatial variability of the foundation materials. Appendix B includes a description of the geotechnical information gathered for the New Orleans East polder and similar information will be collected for the remaining polders.

Hurricane Hazard Modeling

Hurricane hazard is quantified using a joint-probability approach. This approach requires three main components: a hurricane parameterization scheme, a hurricane recurrence model, and a system load model. The standard hurricane parameterization at landfall in terms of landfall location, track direction, speed, central pressure drop, radius to maximum winds, and Holland’s B shape parameter is used. In addition, the variation of mean track with landfall direction and the pre-landfall variation of central pressure and storm speed is also considered. After landfall, the central pressure is filled using the standard exponential model.

The recurrence rate for different hurricane parameters at and before landfall is obtained through statistical analysis of FEMA’s HURDAT data set and various published results. While the model resembles others proposed in the literature, the specific form of distributions, dependencies and parameter values have been adjusted to best fit the historical record.

The loads of interest are surge levels and wave characteristics along the hurricane protection system and the rain rates inside the polders. The surge levels are the most critical loads. The main tool to estimate surges is the numerical code ADCIRC. Significant effort has been devoted to devise a computationally efficient scheme to accurately evaluate surges for about 40,000 different hurricane scenarios. The strategy makes combined use of preliminary runs with a very coarse spatial grid to determine the nature of the dependence of the surge on different parameters and the presence or lack of interaction among different parameters. Taking advantage of these characteristics, the number of needed ADCIRC runs has been reduced to about 1000. Not all 1000 runs are made with a high-resolution (HR) grid. The bulk of the runs use a mid-resolution (MR) grid. The MR results are then calibrated using about 40 HR runs.

The combinations of the parameter values listed in Table VIII-1 are considered in the ADCIRC modeling and the process used to select the parameter sets is described in Appendix J.

Table VIII-1 **Mid-Resolution Runs**		
Parameter	**Factorial 1**	**Factorial 2**
ΔP (mb)	41, 80, 115	80
V (km/h)	8, 21, 36	21
$X\cos(\theta)$ (km)	-130, -90, -50, -10, 30, 70, 110	-130, -90, -50, -10, 30, 70, 110
θ	-60, -30, 0, 30, 60	-60, -30, 0, 30, 60
$R_{\max}$	10%, 50%, 90% quantile from Eq. J-20	10%, 50%, 90% quantile from Eq. J-20
B	50% quantile from Eq. J-21	5%, 95% quantiles from Eq. J-21
$\Delta P_R(t)$	$\Delta P_{R,0.5}(t)$ from Figure J-21	$\Delta P_{R,0.5}(t)$ from Figure J-21
$V_R(t)$	$V_{R,0.5}(t)$ from Figure J-22	$V_{R,0.5}(t)$ from Figure J-22
$R_{\max}(t)$	From Eq. J-23	From Eq. J-23
$B(t)$	From Eq. J-24	From Eq. J-24
α	$0.035 + 0.0005\,\Delta P$	$0.035 + 0.0005\,\Delta P$
No. of runs	**945**	**210**
Total runs	**1155**	

For the HR runs, the subset of 36 hurricanes in Table VIII-2 is retained. In general, the levels in Table VIII-2 have been chosen to maximize the accuracy of calibration of the MR results.

For the waves, a parameterization scheme based on previous analyses is used. While the details of this approach are still being developed, it is expected that the wave contribution to the water level and other wave characteristics like HS and T will be related directly to the surge values.

Rainfall inside the polders is estimated based on statistics from NASA's Tropical Rainfall Measuring Mission (TRMM) experiment. Since rainfall is not a primary input variable for the performance of the hurricane protection system, the model needs not be very sophisticated. The variation of the symmetric component of rainfall with distance and central pressure is accounted for, but the effect of asymmetry due to storm motion and shear and the assessment of uncertainty is simplified.

Uncertainty on the elements that constitute the hazard model (recurrence rate, parameter distribution, loads) is assessed considering the limitations of the data to which the models are fitted, uncertainty on the future hurricane climate in the Gulf of Mexico, and errors in the load models.

Table VIII-2
High-resolution runs

Parameter	High-resolution model runs
ΔP (mb)	80, 115
V (km/h)	21
$X\cos(\theta)$ (km)	-90, -10, 70
θ	-60, 0, 60
R_{max}	10%, 90% quantiles from Eq. J-20
B	50% quantile from Eq. J-21
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$\Delta P_R(t)$	$\Delta P_{R,0.5}(t)$ from Figure J-21
$V_R(t)$	$V_{R,0.5}(t)$ from Figure J-22
$R_{\max}(t)$	from $R_{\max}$, $\Delta P(t)$ and $Lat(t)$; see Eq. J-23
$B(t)$	from B, $R_{\max}$ and $Lat(t)$; see Eq. J-24
α	$0.035 + 0.0005\,\Delta P$
No. of cases	**36**

For the 36 cases in Table VIII-2, the water levels *H* and the wave characteristics *W* are directly extracted from the HR runs. For the remainder of the cases run only with the MR grid, corrections must be made to reflect the bias of that coarser discretization. The bias is site-specific, as it depends on the local geometry of the coast, the topography, and the different local land coverage of the MR and HR grids. The correction further depends on the hurricane parameters. For example, the correction at a given location generally depends on landfall position, direction, and possibly storm intensity. The approach used to calibrate the MR runs is included in the appendices.

Reliability Modeling

System Reliability Model. The reliability of the hurricane protection system (HPS) under potential water surge and wave loadings is quantified using structural and geotechnical reliability models integrated within a larger systems description of each polder. We use standard reliability models that combine uncertainties in structural material properties, geotechnical engineering properties, subsurface soil profile conditions, and engineering performance models of levees, floodwalls, and transition points. Uncertainties due to spatial and temporal variation (aleatory uncertainty) and due to limited knowledge (epistemic uncertainty) are tracked separately in the analysis, to provide a best estimate of frequency of failures along with a measure of the uncertainty in that frequency.

To date, the reliability model has been developed for the Orleans East (NOE) polder as a means of exercising the approach. The perimeter protection system comprises levees, flood walls, levees with floodwalls on top, and various points of transition or localized facilities such as pumping stations, drainage works, pipes penetrating the HPS, or gates. This perimeter has been divided into reaches that are deemed to be homogeneous in three aspects: structural cross-section, elevation, and geotechnical cross-section. Approximately 20 such reaches have been identified for NOE.

Geometric and engineering properties have been identified for each reach of NOE and summarized in flat-file data tables. Structural cross-sections were initially identified by review of as-build drawings, aerial photographs, and GIS overlays; and were confirmed by on-the-ground reconnaissance by Team 10 members. Elevations were initially assessed in the same reconnaissance, and were later supplemented by LIDAR data and field surveys provided to the Team. Geotechnical cross-sections and corresponding soil engineering properties were derived from the original Design Memoranda for the respective project areas of the polder, supplemented by site characterization data collected post-Katrina at levee flood wall failure sites (cone penetrometer and laboratory measurements).

Reliability assessments are performed for individual reaches of the HPS for given water levels and loadings. This results in fragility curves for each reach by mode of failure. For each reach and mode of failure, the fragility curve gives the conditional frequency at which a failure state is exceeded. As a first step, engineering performance models and calculations have been adapted from original Design Memoranda. Engineering parameter and model uncertainties are propagated through those calculations to obtain approximate fragility curves as a function of water height on the HPS. These results will later be calibrated against the ongoing work the by the performance analysis team, which is applying more sophisticated analysis techniques to similar structural and geotechnical profiles in the vicinity of failures. Failure modes identified by the performance analysis Team will be incorporated into the reliability analyses as those results become available.

Systems risk model. The reliability assessments for individual reaches of the polder perimeter (and possibly of interior levees or walls) are combined in a systems model which brings together the uncertainties in hurricane hazard and HPS fragility to calculate frequencies of volume and duration of flooding within the polder. The systems risk model, embedded in a software application, is structured around an event-tree description of the occurrence of hurricane events, corresponding water and wave heights, and resulting response of the HPS. This model separately tracks aleatory and epistemic uncertainties from both the hurricane hazard and the structural and geotechnical response, producing a best estimate of frequency and duration of flooding, along with measures of uncertainty in those frequencies.

Events Studied. The events of interest that have been selected to predict component performance are overtopping (O), breach (B), and pumping (U). Shown below are the branch segments analyzed. Where an event is underlined, the event is the complement of the event (for example: $\underline{O}$ indicates a non-overtopping event). The branch segments from the event tree are:

$\underline{O}$, $\underline{B}$, U	O, $\underline{B}$, U
$\underline{O}$, $\underline{B}$, $\underline{U}$	O, $\underline{B}$, $\underline{U}$
$\underline{O}$, B	O, B

The probability of failure for the levees and floodwalls when subjected to combinations of overtopping and breaching (O, B; O, B; O, B) are evaluated separately from the performance of the pumping stations.

Failure of a component has been defined as an event that allows flood waters to enter the polder beyond that expected without failure. Only a complete breach of a levee or floodwall is considered; partial breaching is not included. The expression for determining the probability of failure has been included where known in order to identify the information required. All probabilities are conditional upon the flood elevation (and associated hazards, such as wave forces, where applicable).

Component Hazards. The following hazards are considered as component loads in the risk analysis:

- Flood elevation - storm surge plus wave setup
- Breaking waves
- Flood flow rate and duration for scour and erosion

Polder Components. The reliability examines the performance of the following components of the HPS system in the risk analysis:

- I-wall with sheetpile embedded in levees
- T-wall on levee
- Transitions and closures
- Levees

Structures or components not included. The following structures in the HPS system were not independently evaluated for their failure modes. Both structures can be addressed with the failure modes developed for I-walls.

- Concrete apron with some I-walls (treated as an I-wall with improved erosion resistance)
- Sheetpile with a 3 to 4 ft concrete cap (treated as an I-wall)

Failure Modes and Factors Contributing to Failure Modes Not Included. Some potential factors that may contribute to failure of a component have not been considered. These factors were screened prior to elimination from the risk analysis and it was determined that there was either little evidence that they occurred during Katrina, or they would have minimal potential for failure. Some of these factors may, however, be considered in future refinements of the risk model.

Settlement of levees and floodwalls over time. The time-varying nature of levee crest elevations are not considered. For these analyses, the crest elevations at the time of Hurricane Katrina will be used. The crest elevations were established from LIDAR surveys or surveys.

Piping soil failures under levees. In surveys conducted after Hurricane Katrina, piping was only observed in the canals where there were sand beds under the levee. Boils were sometimes found on the protected side of the levee, but the levee did not fail at these locations. The available geotechnical data used for levee designs, and that obtained under IPET, is not sufficiently detailed to determine localized weaknesses in the soil (i.e. local sand pockets) that may exist along the levees.

Maintenance of levees and floodwalls. The effects of maintenance on the HPS capacity over time are not included. Improper maintenance or neglect can lead to reduced capacity of the levees in particular; gates and other moving components also require maintenance. Trees, landscaping, and pools were observed on the protected (landside) embankments after Hurricane Katrina, indicating a lack of enforcement and maintenance of the levees. However, there is insufficient information about maintenance activities, or lack thereof, to include this factor in the risk analysis.

Barge or tree impact. Impact by a barge or floating tree, or other large object, on the floodwalls or levees are a possibility during a hurricane. However, during Hurricane Katrina there was no clear evidence of a component failure due to impact from a barge or tree. The barge found inside the New Orleans East polder near the ninth ward was reported to have floated over the levees and floodwalls during overtopping, or after the levee breach. Such an impact may cause local damage, but the flooding due to a single breach of a floodwall is not considered in these analyses. Flooding from a single breach caused by an impact during overtopping and breaching over miles of the HPS system is too small of an event to consider within the uncertainty that exists for the system analysis.

Blast events. Several statements raised the issue of some component failures being caused by blast events. Review of photographs and witness accounts and inspection of the HPS after Hurricane Katrina by multiple independent groups has not found any evidence of blast events. However, the failures of the levees and concrete floodwalls that did occur were sometimes so sudden that they may have sounded or appeared to be ‘explosive’ with the immense force that swept cars and homes along with the incoming surge waters.

Consequences Modeling

The primary output of the risk and reliability modeling of Team 10 will be an estimate of the probability of life loss and physical damage relating to the performance of the hurricane protection system in southeastern Louisiana. The three scenario cases which are being considered: 1) the pre-Katrina (August 28, 2005) risk, 2) the actual Katrina experience, and 3) the risk associated with conditions as of June 1, 2006. A probabilistic estimate of losses (life and property) will be provided.

IPET is working in close collaboration with the Consequence Team (Consequences) to ascertain appropriate relationships of inundation, impact and life and property loss. The consequence team is considering consequences in four areas: 1) economic consequences, including direct damage and indirect losses, at local, regional and national level; 2) environmental consequences; 3) social, cultural and historical consequences, and; 4) life safety and health consequences.

As of mid-February, the work of the consequence team has been initiated, but limited data has been collected and no firm inputs are available to the modeling effort of the Risk and Reliability Team. Liaison with the Consequence Team has contributed to the refinement of the flood life loss model (lifesim) and have established contact with the Louisiana State University Hurricane Center and Team Louisiana which have been tasked with the State of Louisiana to carry out forensic evaluation of the Katrina event.

Issues of interface between team activities remain a major concern. Attempts are underway to clarify the necessary input to the consequence team modeling of consequences in the categories mentioned above. It had earlier been assumed that a maximum flood elevation in each sub-folder would provide sufficient characterization of the event to generate consequence estimates. In further discussion with subgroups of the Consequence Team, it is evident that for the case of life loss several factors are considered of critical importance including rate of inundation, duration of inundation, and velocity of flow. These factors relate to the feasibility of evacuation and rescue to prevent life loss. For physical damage, it is also possible that these characteristics will be desirable for the refinement of loss estimates. Social and demographic data is also required for the life loss estimation. This data is currently being collected by other IPET Teams but has not been analyzed to develop useful relationships for the risk model. Detailed analysis of fatality data is still required to relate socio-economic demographic information to specific risk factors for fatality. The application of the flood life loss model (lifesim) requires more detailed consideration of both evacuation and rescue procedures. The work of other IPET teams has primarily been dedicated to documentation and forensic analysis of the Katrina event. This analysis is developing risk and reliability models which will be calibrated by earlier events including Katrina, but will be useful in evaluating potential variation in design, management and other risk-related factors for future events and future modification of the hurricane protection system. The establishment of valid general relationships between measurable event impacts and measurable event consequences is critical to the completion of the risk model. Currently, the consequence team has committed to focusing its attention on two specific quantitative characterizations of consequences: 1) life loss (rather than injury, health status, mental health, etc.) and, 2) the dollar value of direct physical damage to buildings and infrastructure (rather than indirect costs such as business interruption, loss of revenue, etc.). These simplifications are necessary because of difficulties in data collection and because of time limitations imposed on the preparation of the IPET report. It should be borne in mind, that these are only representative consequences and not comprehensive. The full social, economic and culture impact of the event will be considerably greater than that represented than the two selected factors.

Liaison with Louisiana State University Hurricane Center. Liaison with the Louisiana State University Hurricane Center has provided valuable input to the understanding of Katrina consequences. The Hurricane Center at LSU has been deeply involved in assessment of previous hurricane losses and modeling of expected losses due to future hurricanes for a number of years. Of specific relevance to the consequences evaluation, the LSU hurricane center is now working with the Louisiana State Coroner's Office to analyze fatality data on the roughly 1200 confirmed fatalities (bodies recovered). Of these, approximately 700 have been identified, and circumstances and location of death have been established. LSU is currently carrying out detailed studies of fatality circumstances and has developed a GIS for the location of victims recovered and their home addresses. This material is not currently available to IPET because of privacy concerns and further negotiation will be necessary to obtain data relevant to the IPET consequences study. The LSU Hurricane Center has collaborated with the FEMA mitigation assessment team which has carried out an analysis of building damage in the affected area and this data will be available from FEMA. The work is carried out under a FEMA contract with URS. The LSU Hurricane Center includes LSU faculty members with experience and expertise in a range of relevant areas: evacuation, experts in transportation, planning and traffic management have been directly involved in the development of state evacuation policy and have played a major role in the successful evacuation of over 1 million people from New Orleans. Members of the Sociology Faculty have worked on the analysis of behavioral aspects of warning and evacuation response in various neighborhoods and populations of New Orleans. Regional economists from LSU have developed input-output modeling for the region which will provide perspective on indirect losses at the regional level. The Hurricane Center also participated in the PAM exercise organized by FEMA in advance of Katrina and documentation of the PAM exercise should provide a useful input for the consequence calculation. The FEMA contractor for the PAM exercise was Innovative Emergency Management of Louisiana.

The Hurricane Center has developed its own models for the impact of hurricanes in the New Orleans region. It has calibrated ADCIRC for Betsy (1965) experience and it provided model results of Katrina impact to the Louisiana Department of Emergency Preparedness and the Times-Picayune in advance of Katrina landfall (these model results did not include breaching of the levee and floodwall system). Data sources identified by the LSU Hurricane Center have been communicated to the consequence team for follow-up. Risk Team liaison members met with the Life Safety and Health subgroup of the consequence team on February 22nd to clarify needed inputs for the consequence team and expected outputs from the consequence team which will contribute to the risk modeling effort. The clarification of required inputs and expected outputs of the consequence team represents a major step forward. It is now necessary to communicate those input needs to other relevant IPET teams and to incorporate those expected outputs into the risk model.

Risk Communication

A preliminary plan for communicating the results of the risk analyses to the USACE leadership and the public has been developed. This plan is not a part of this document but will be available for the ERP review meeting. The intent of this plan is to provide guidance concerning the types of questions that USACE can be expected to be asked from many different sources. The questions will encompass areas that are outside the IPET scope and beyond the responsibilities of USACE.

Status of Remaining Efforts

System Definition

Resources have been added that will assist in completing the field and geotechnical work required to define the remaining polders. Two teams will be used to supplement the field work already started on the polders using the completed New Orleans East polder as a template. This work is expected to be completed by the end of March.

Risk Model

The New Orleans East model will be refined using the experience gained during initial testing. Development of the risk models for the remaining polders will also use the New Orleans East polder as a template. These models should be developed rapidly once the system definitions are complete. It is expected that model testing and revisions will be complete by the end of March and that production runs will begin at that time.

Hurricane Modeling

The surge models using the ADCIRC MR have been started and initial runs will be used in testing of the risk model. The MR surge runs are expected to be complete by the end of March. Wave modeling continues to lag behind the surge modeling and additional assistance from ERDC CHL has been requested. This effort must be completed prior to initiating the risk model production runs.

Reliability Modeling

Reliability models have been developed for the New Orleans East polder that will be used as templates for the remaining polders. Changes in loading and material parameters will be made in the models to account for local conditions. Reliability model will follow closely polder development and is expected to be completed by early April.

Consequences

As of mid-February, the work of the consequence team has been initiated, but limited data has been collected and no firm inputs are available for the risk model. Team members providing liaison with the Consequence Team have contributed to the refinement of the flood life loss model (lifesim) and have established contact with the Louisiana State University Hurricane Center and Team Louisiana which have been tasked with the State of Louisiana to carry out forensic evaluation of the Katrina event.

Issues of interface between team activities remain a major concern. Attempts are underway to clarify the input required from other IPET teams for the Consequence Team to use in modeling of consequences. This is not expected to delay work since the primary risk model results will be in terms of the extent of inundation and the probability of system and component failure. Neither of these parameters requires input of consequences. The final determination of expected loss of life and economic losses however will require consequence input.