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

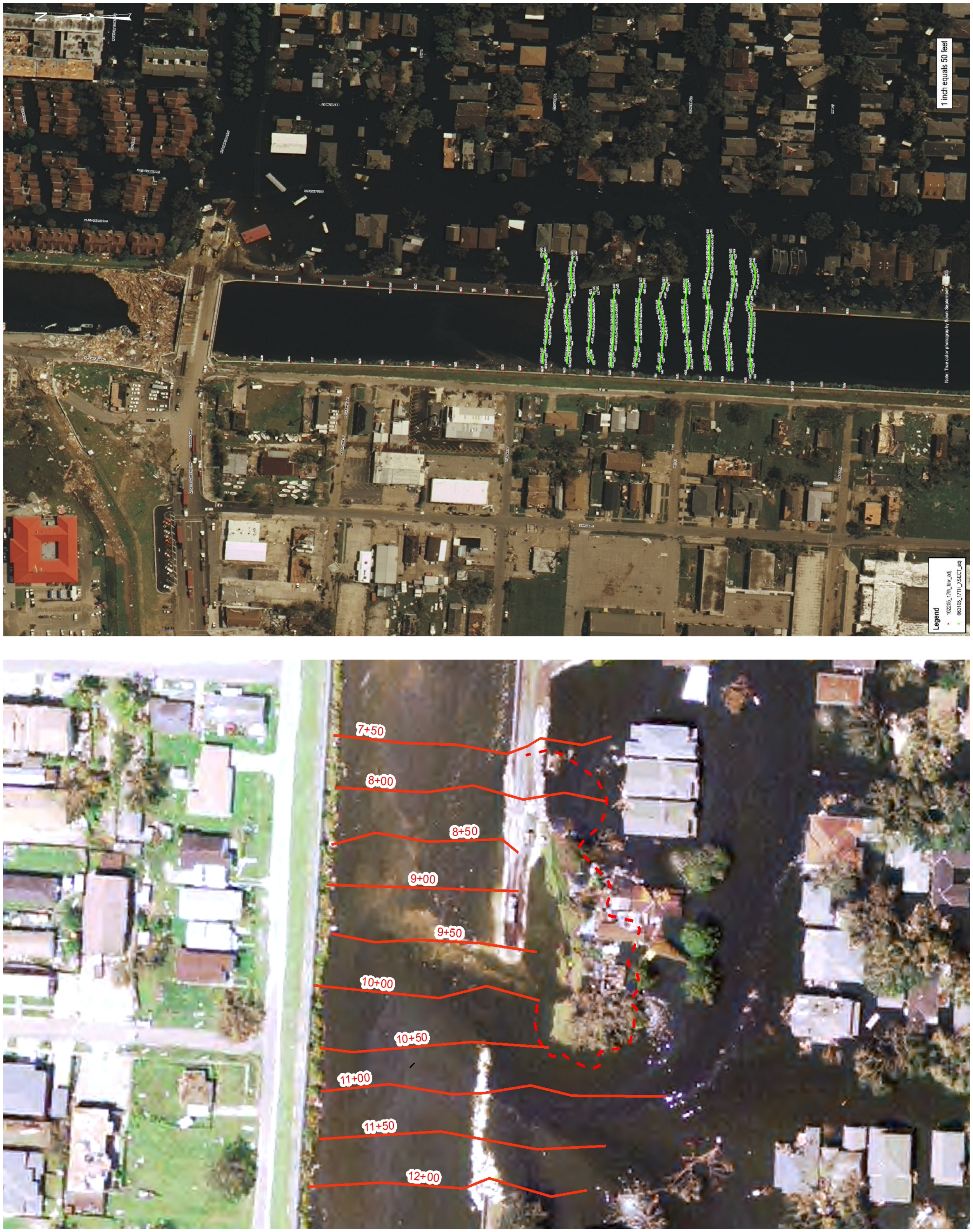


Figure VI-4. Location of multi-beam sonar survey cross-sections

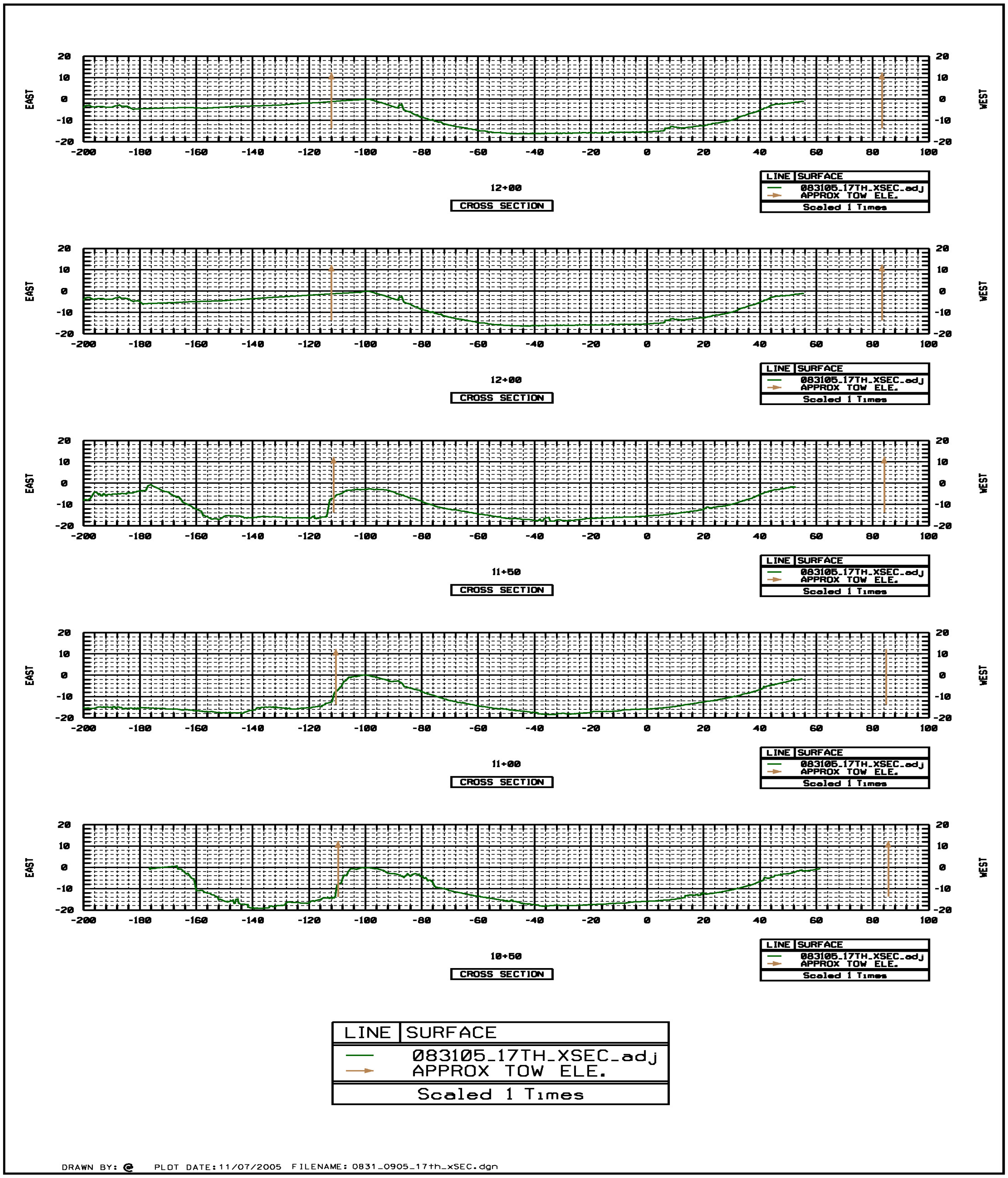


Figure VI-5a. Surface profiles at the breach

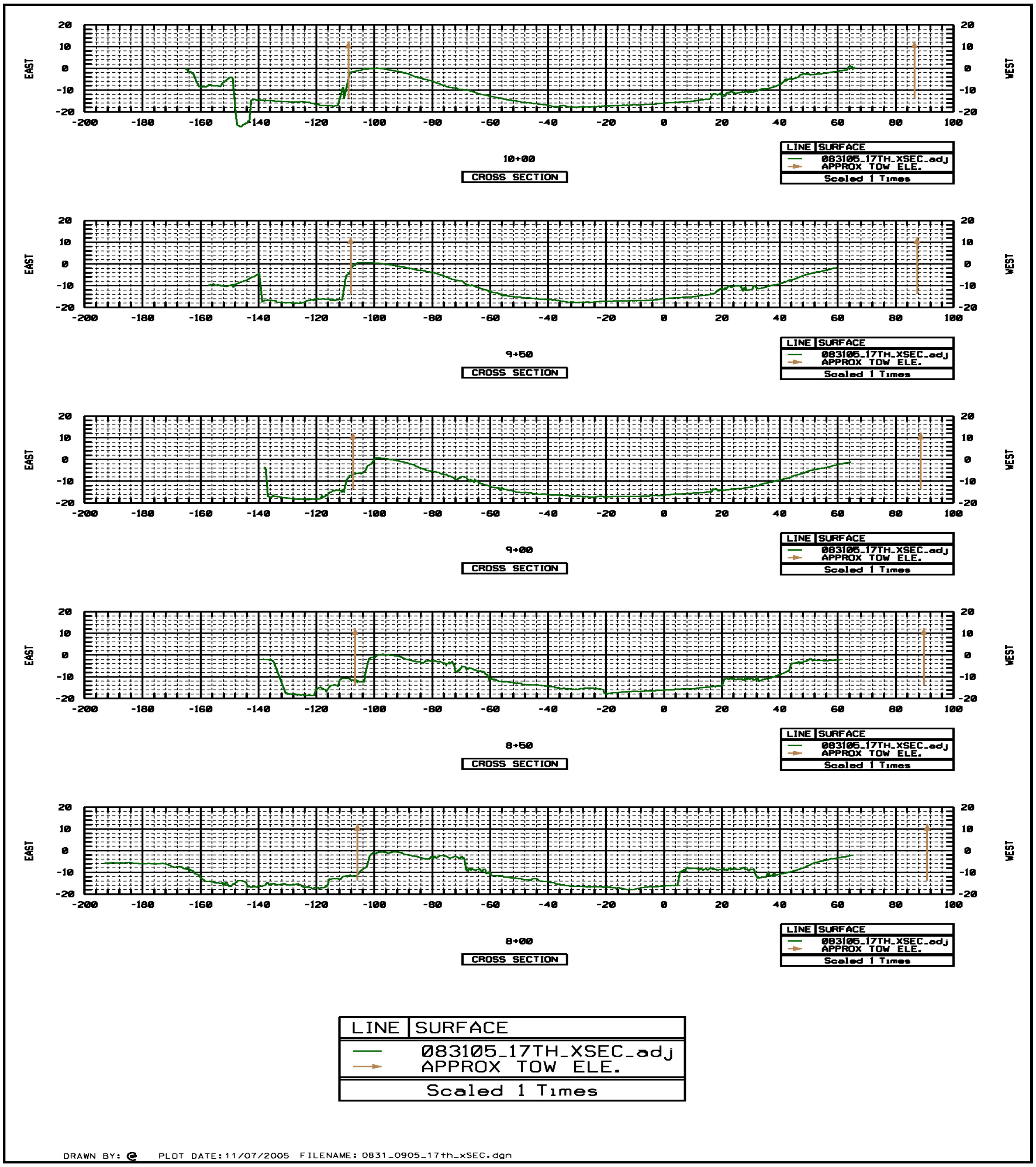
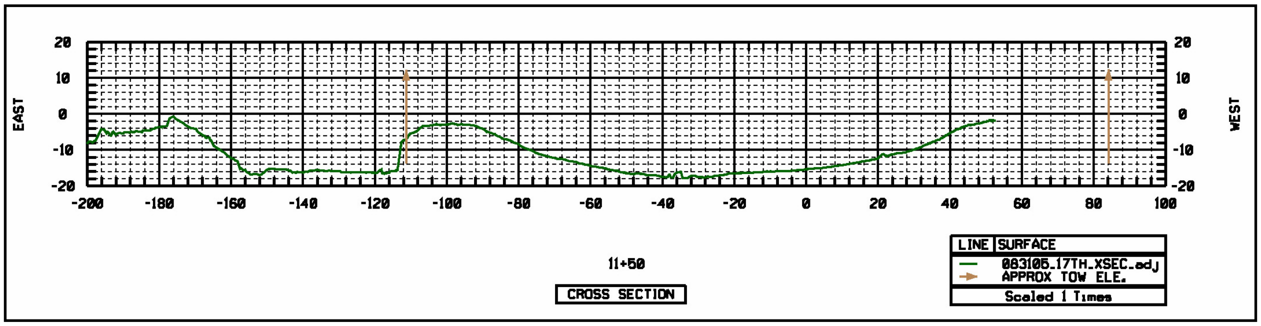


Figure VI-5b. Surface profiles at the breach



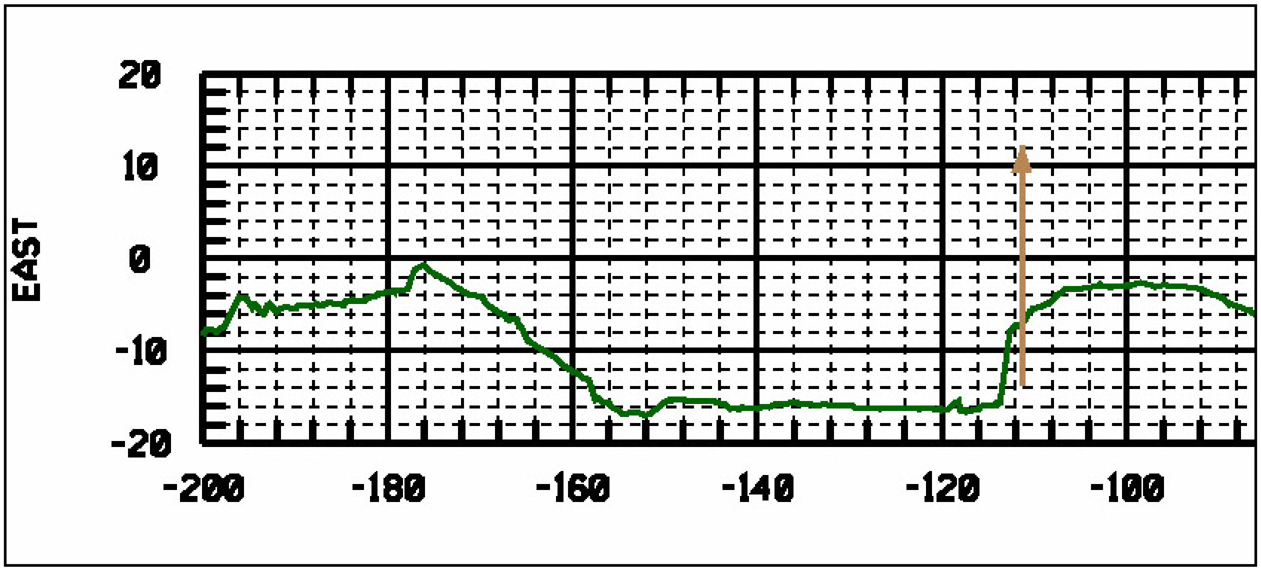


Figure VI-6. Profile for Station 11+50

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).

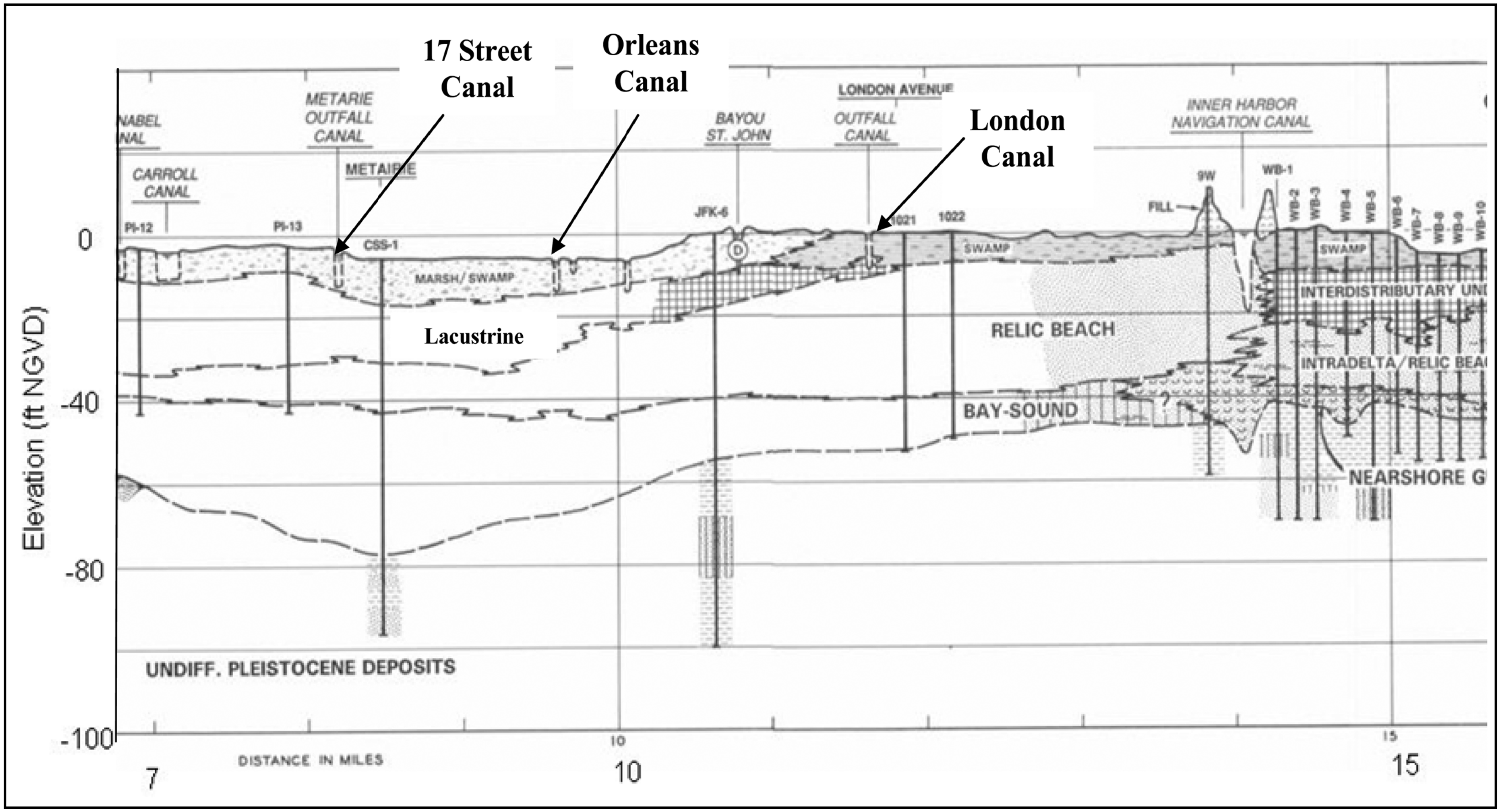


Figure VI-10. Geological cross-section extending west to east direction across western Jeffereson Parish and into eastern Orleans Parish. Section runs from near the 17th Street Canal to the Inner Harbor Navigation Canal. Major outfall canals in Orleans Parish are noted on the section. Cross-section shows 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*.

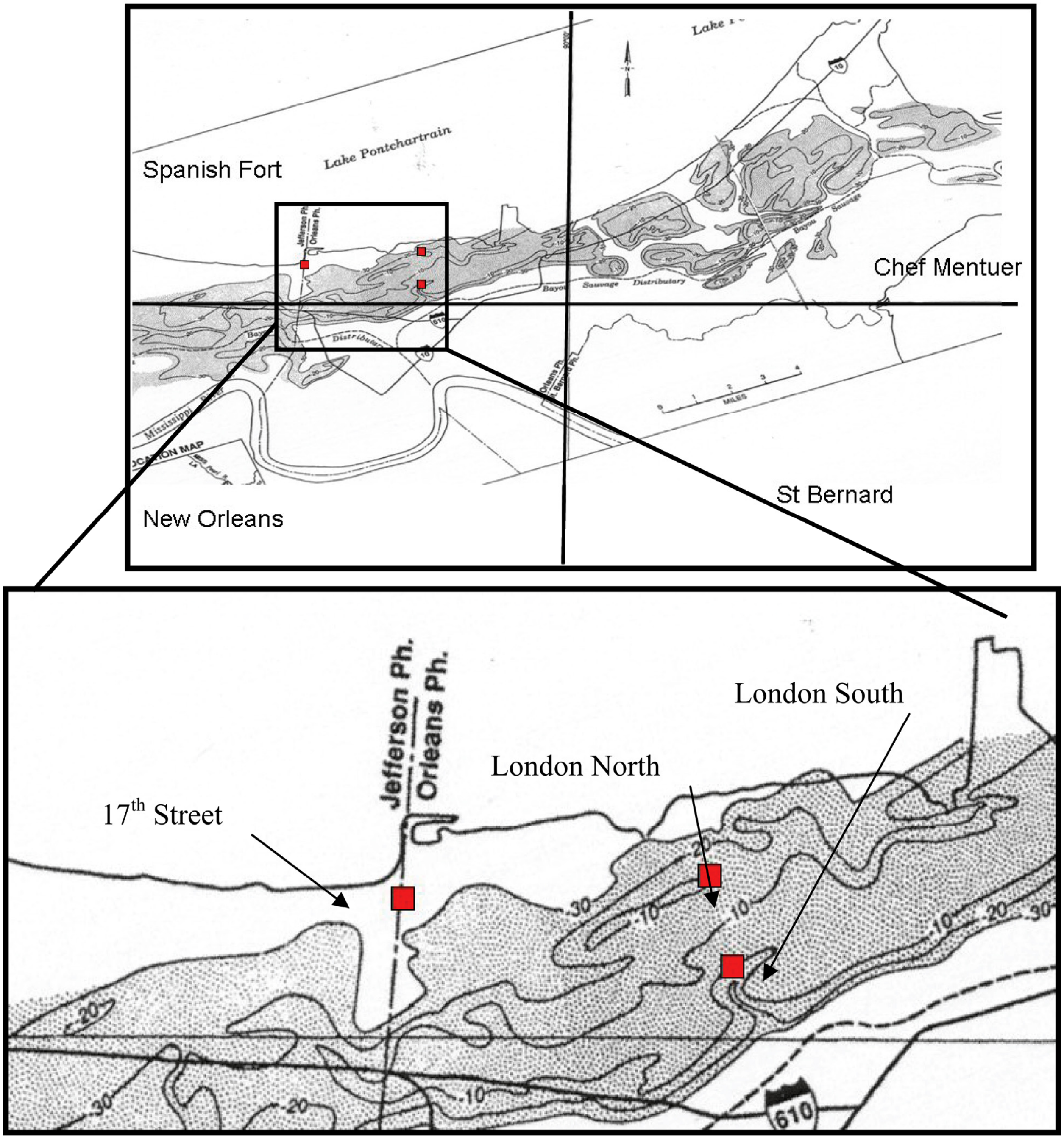


Figure VI-11. Generalized contour map showing the Pine Island Beach, contour values are in ft MSL (Saucier, 1994). Upper figure shows general trend of beach ridge in the New Orleans area, lower figure shows detailed view at the canals. London canal levee failures are located along axis of the beach. 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.

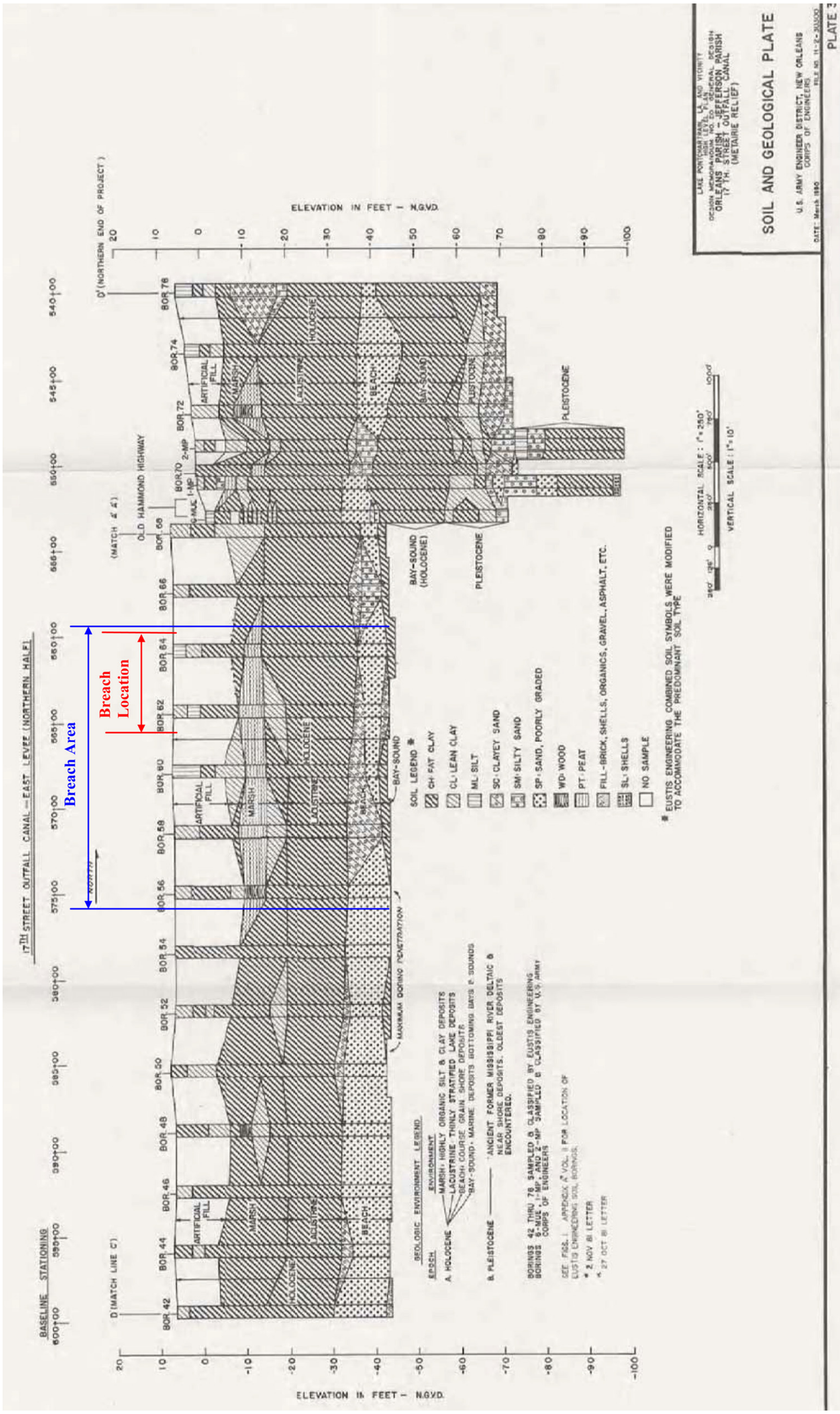


Figure VI-12. Geological Profile showing Breach Area (East Levee)

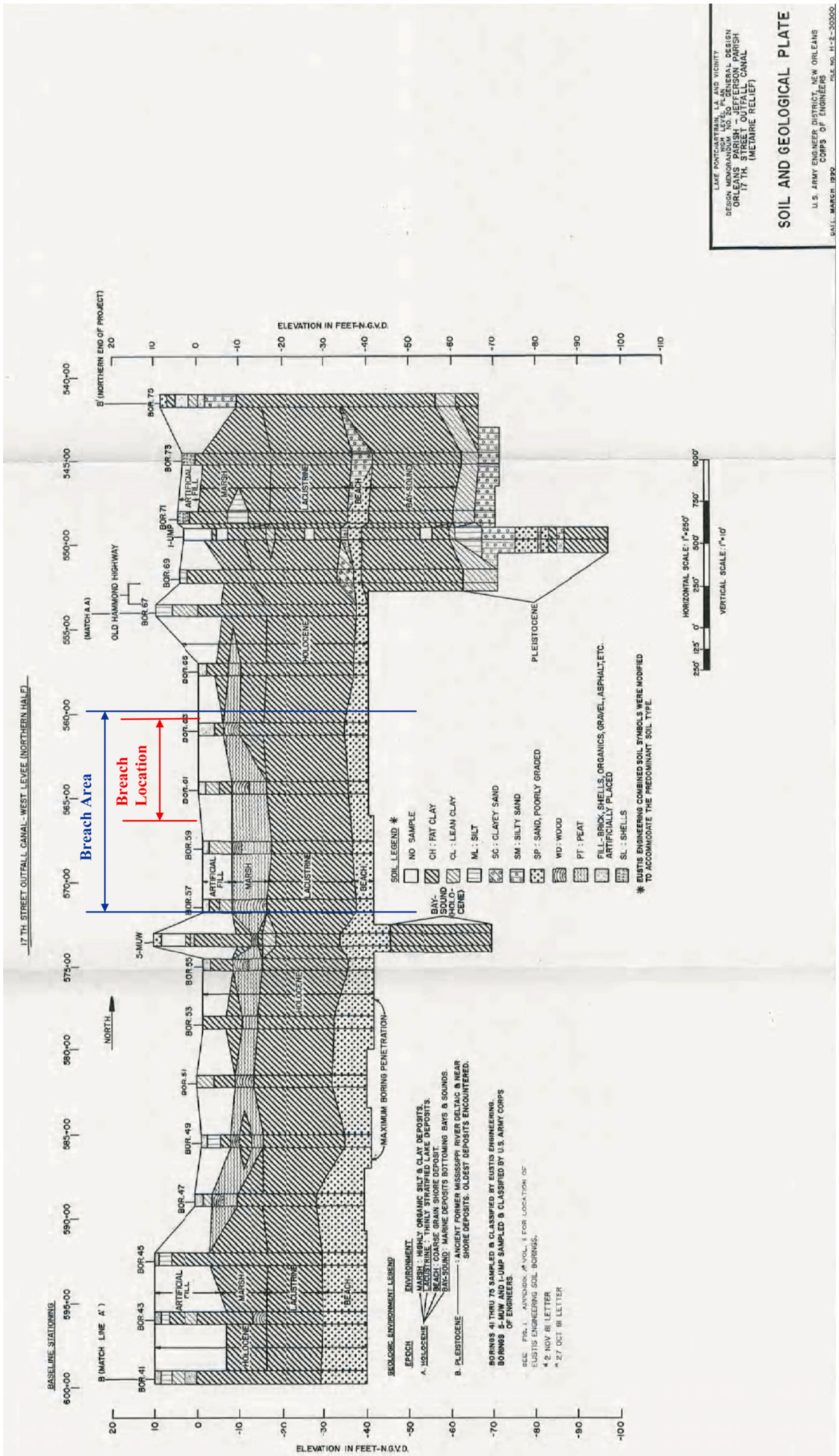


Figure VI-13. Geological Profile showing Breach Area (West Levee)

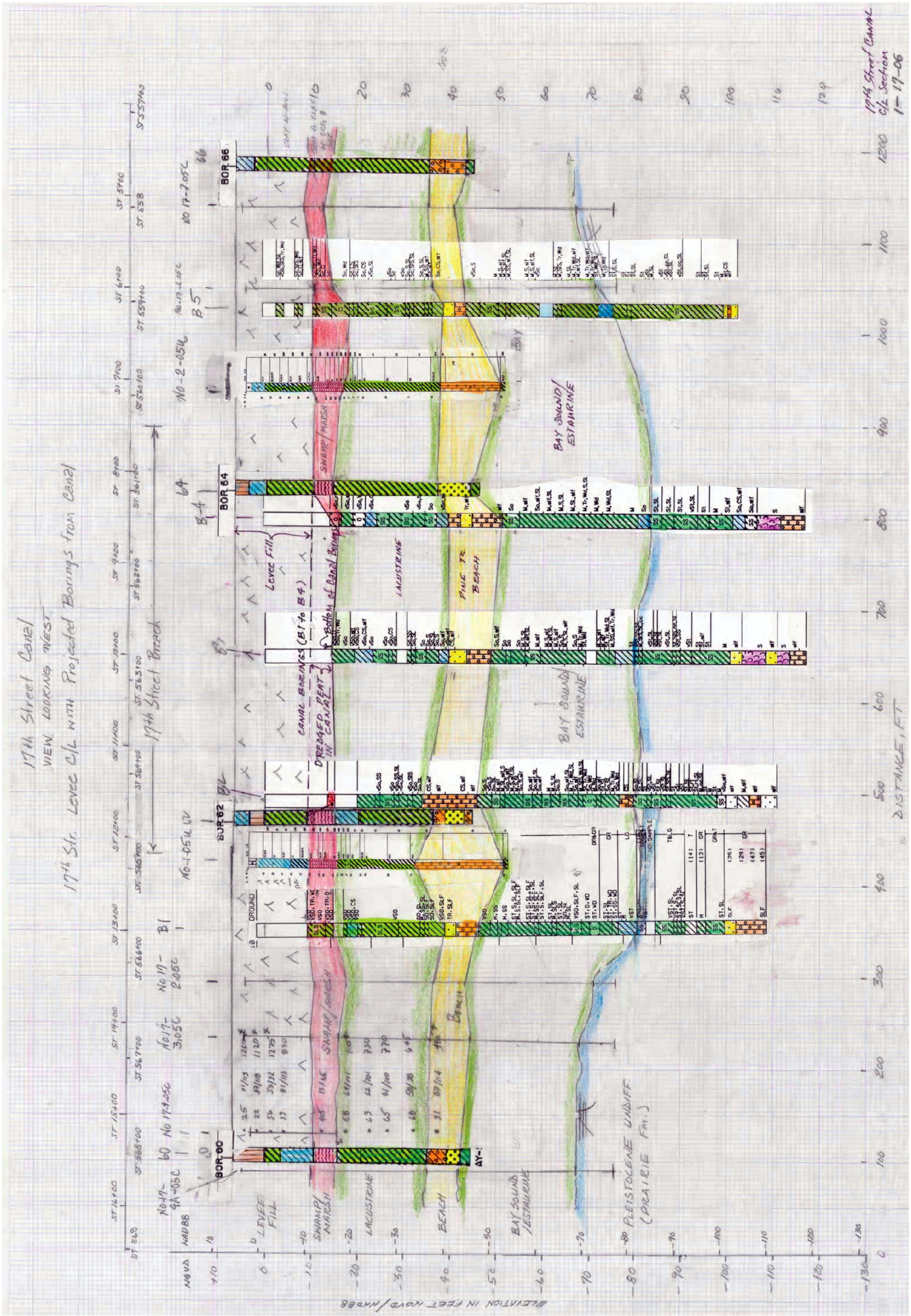


Figure VI-14. Centerline (CL) Cross-section of Breach

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.

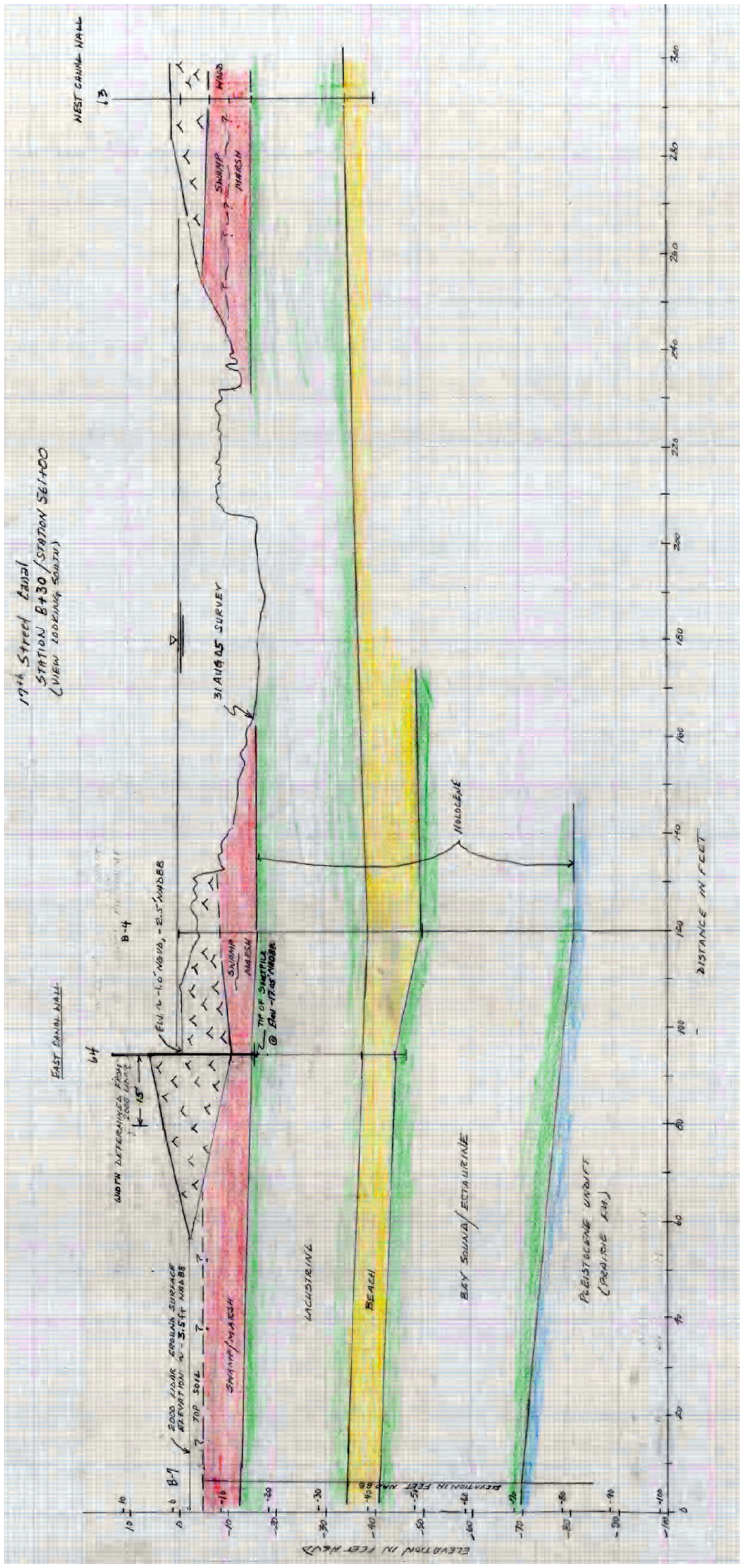


Figure VI-16. Prefailure Cross-Section at Sta 8+30 (New Stationing)/ Sta. 561+00 (GDM Stationing)

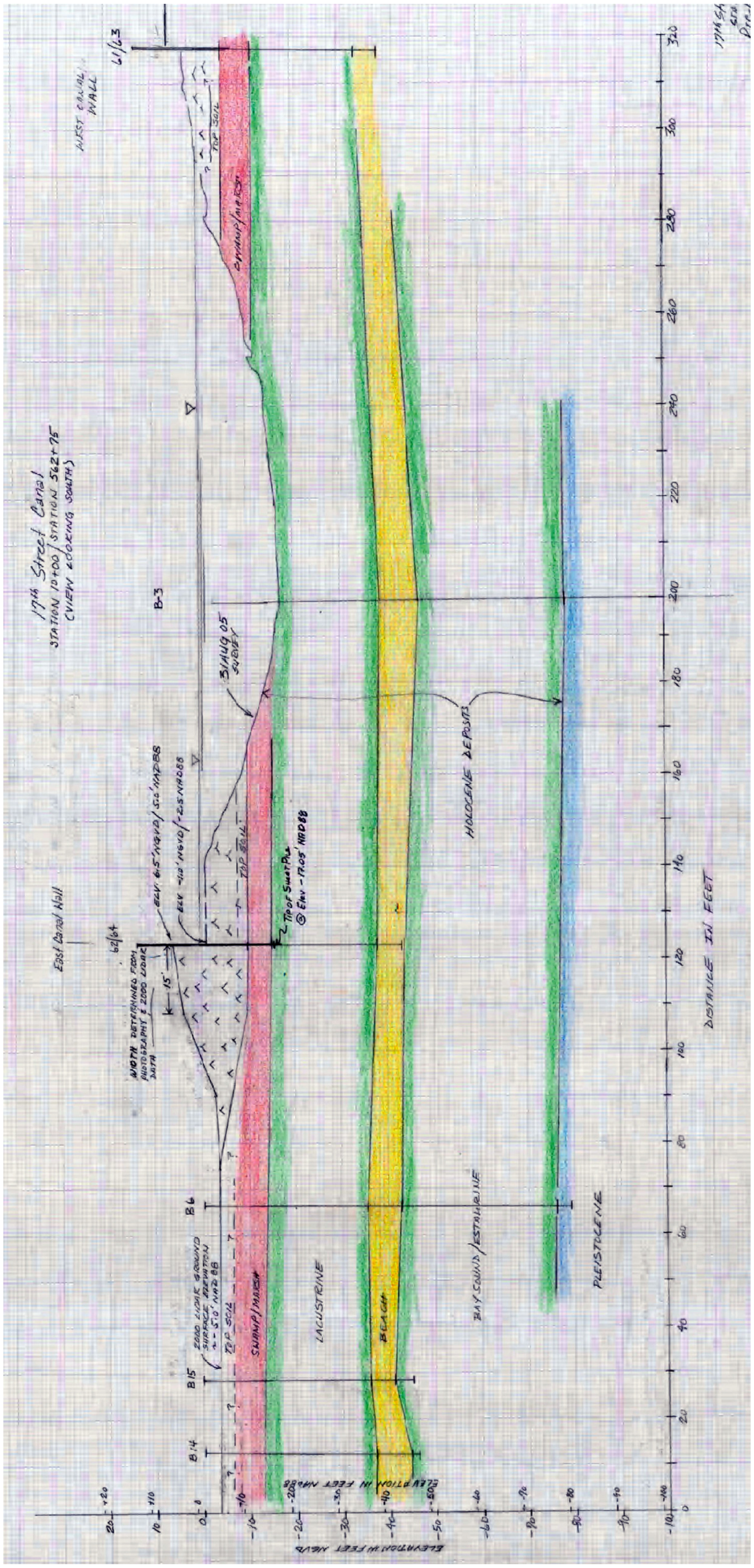


Figure VI-17. Prefailure Cross-Section at Sta 10+00 (New Stationing)/ Sta. 562+75 (GDM Stationing)

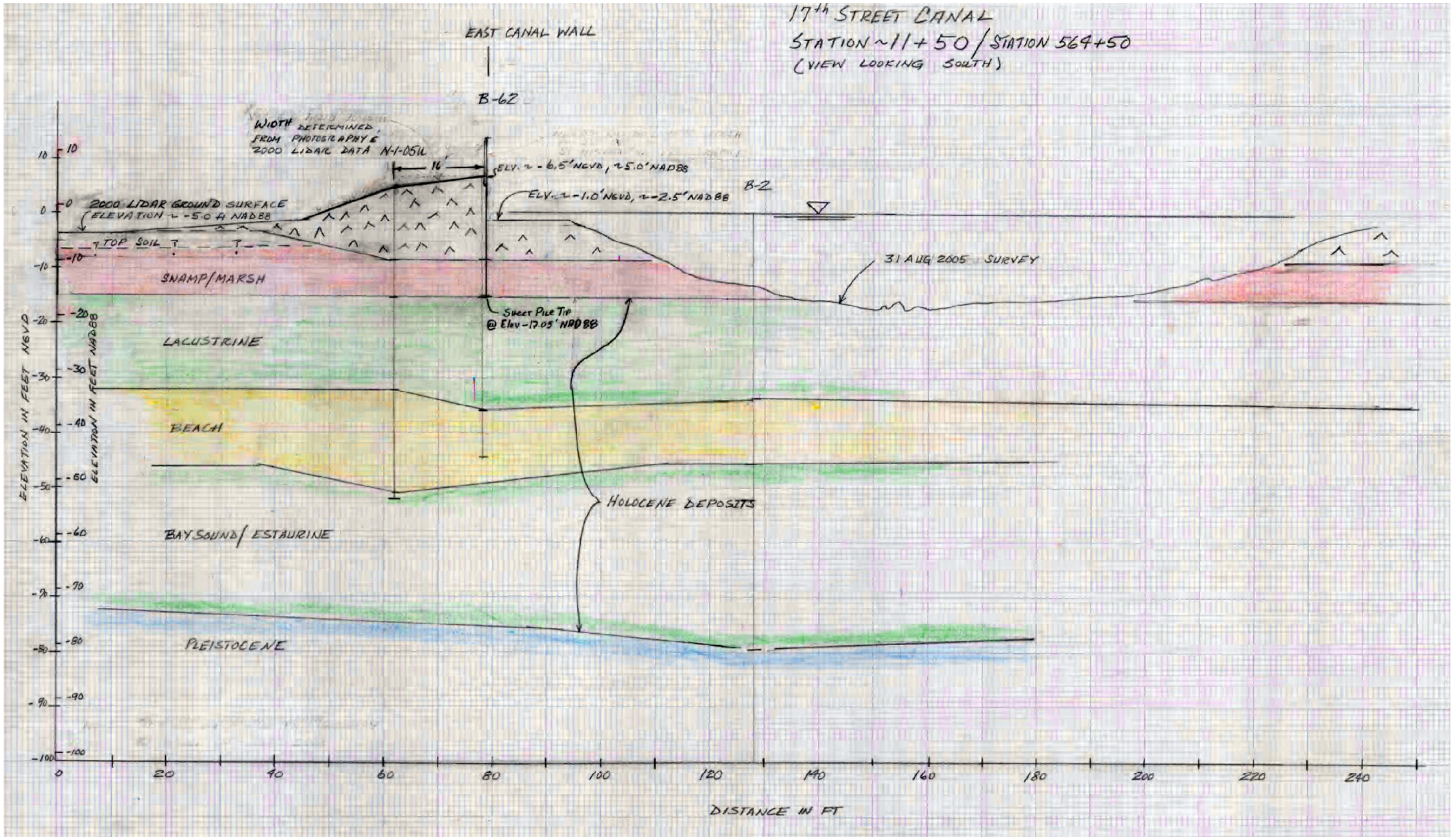


Figure VI-18. Prefailure Cross-Section at Sta 11+50 (New Stationing)/ Sta. 564+50 (GDM Stationing) (Note: Most scour occurred near this station)

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.

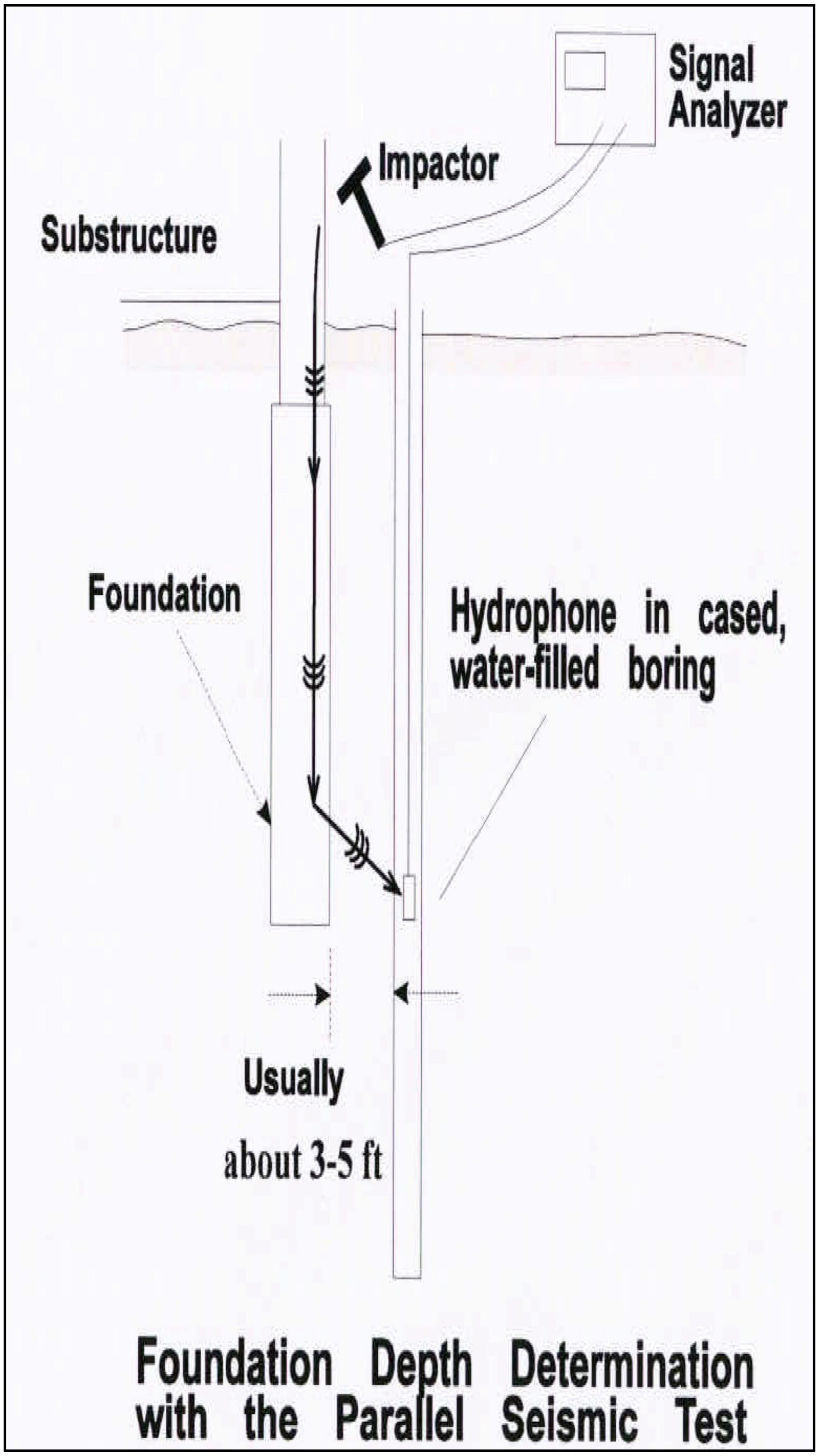


Figure VI-19. Parallel Seismic test setup

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 include~~s~~: 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.

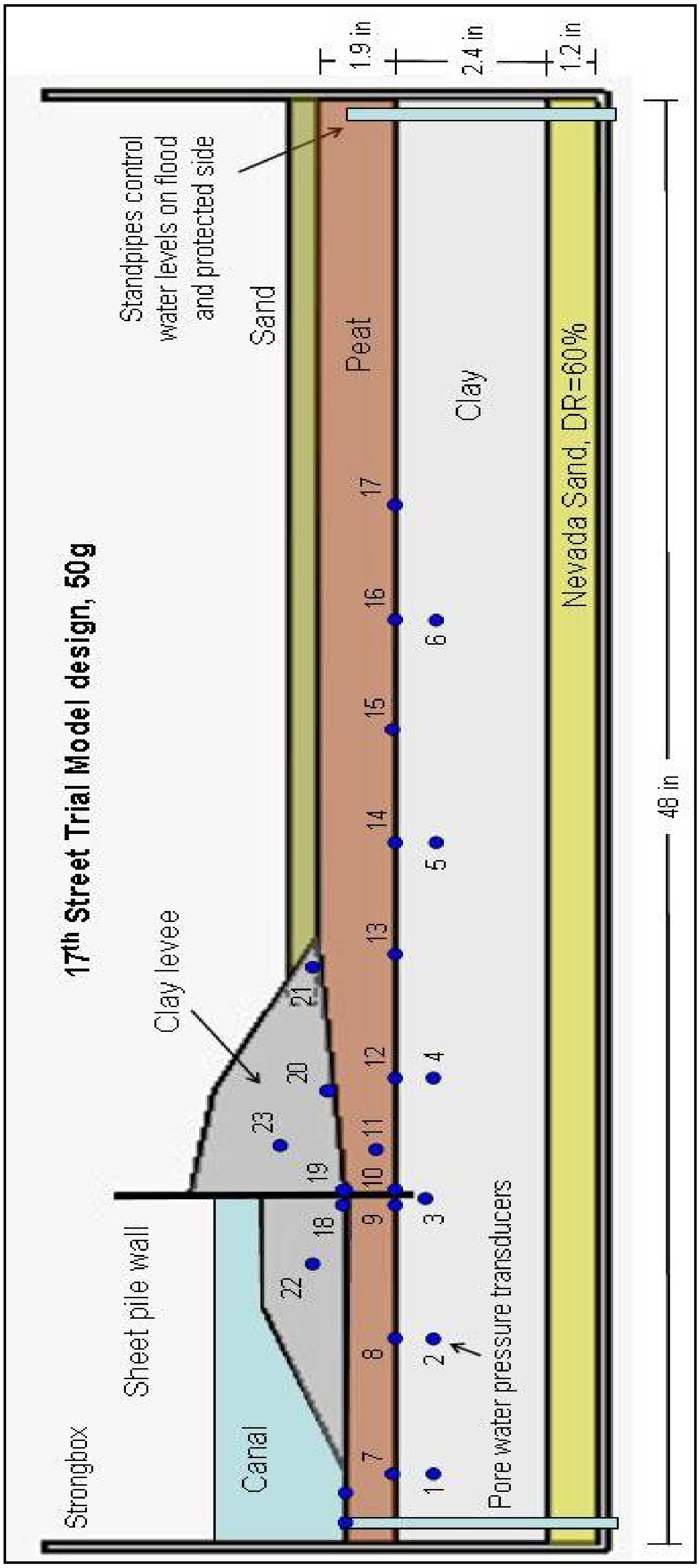


Figure VI-20. Diagram of 17th Street Trial Model Design (model units)

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 Assesment

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.

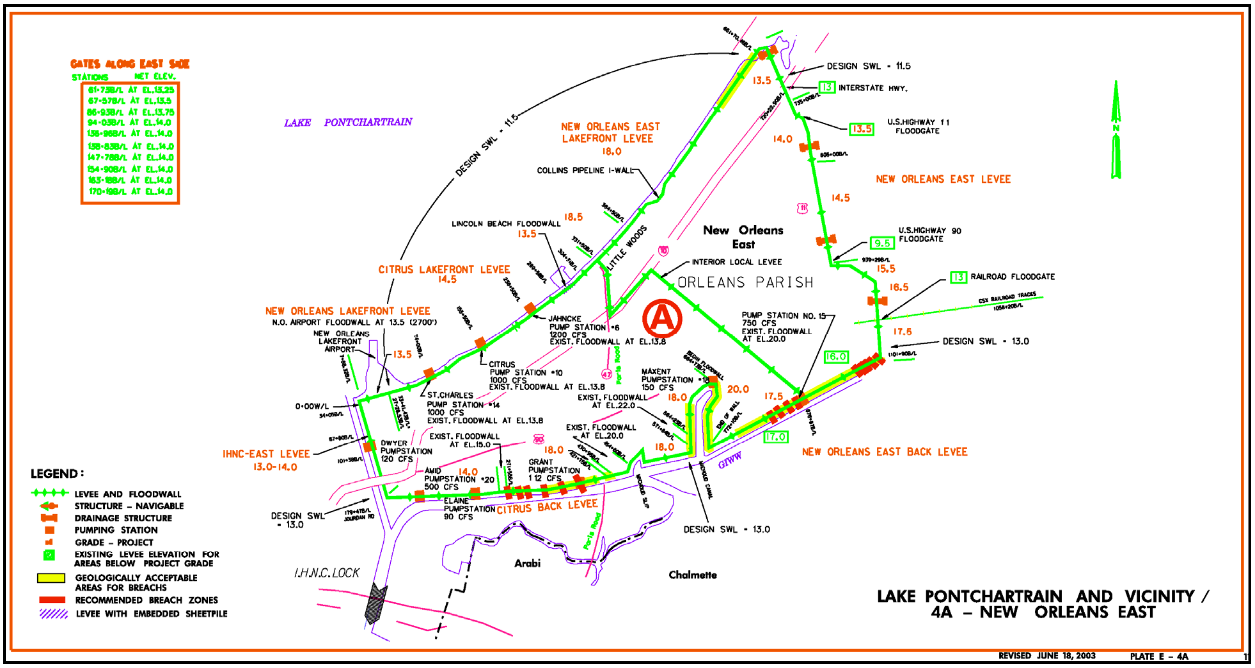


Figure VI-22. NOE Basin general components and top of levee/floodwall as-built elevations (feet) (source USACE, New Orleans District (Wayne Naquin)

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)