US Army Corps of Engineers

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

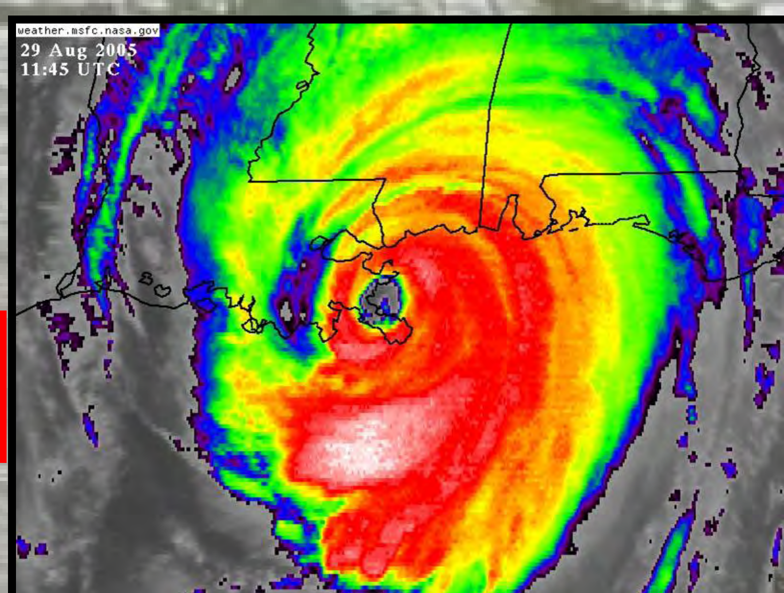
Draft Final Report of the Interagency Performance Evaluation Task Force

Volume V – The Performance — Levees and Floodwalls

1 June 2006

FINAL DRAFT

(Subject to Revision)



Volume V
The Performance — Levees and Floodwalls

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

Contents

Executive Summary

This report summarizes a comprehensive investigation of the performance of floodwalls and levees in New Orleans during Hurricane Katrina. In all, about 50 locations were studied, where breaches in the hurricane protection system occurred due to overtopping and erosion, or due to structural instability.

The majority of the breaches resulted from erosion following overtopping of floodwalls or levees. Overtopping and erosion led to failure of I-walls when water cascading over the tops of the walls scoured and eroded the soil on the protected side of the wall, eventually eroding away so much of the soil supporting the wall that the wall became unstable. Overtopping and erosion led to failure of levees when the soils of which the levees were constructed had insufficient resistance to erosion to withstand the velocity of the water flowing down the protected side of the levee embankment, eventually cutting through the levee crest, washing out the levee.

Most alarming were the failures of I-walls that occurred before overtopping, as a result of foundation instability. Breaches resulting from failures of this type occurred at the 17th Street Canal and the Inner Harbor Navigation Canal (IHNC), where the failure occurred in weak foundation clay, and at the London Avenue Canal, where the instability resulted from intense seepage and high uplift water pressures in the sand foundation. A common element in these I-wall failures was development of a gap between the wall and the soil on the canal side of the wall. Water entered these gaps, increasing the water loads on the walls. Limit equilibrium stability analyses indicate that the factors of safety against instability dropped by about 25% when the gaps formed and water flowed into them. Limit equilibrium stability analyses, centrifuge model tests, and finite element soil-structure interaction analyses all showed that gap formation played a key role in the instability of the walls. At the London Avenue I-wall, with sand beneath the levee and I-wall, the opening of the gap allowed water to flow down the back of the I-wall, introducing high water pressures into the sand, resulting in high uplift water pressures, increased hydraulic gradients, and greatly increased likelihood of subsurface erosion and piping on the protected side of the wall.

The clay in the foundations of the 17th Street Canal I-wall and the IHNC I-wall was found to be normally consolidated, with undrained shear strength that was lower beneath the levee slopes and beyond the toe than beneath the levee crest, where the clay had been compressed under higher pressure. This variation of the undrained strength of the clay with pressure was found to be an important aspect of the foundation soil behavior, and a key factor in evaluation of stability.

The ability of levees to withstand overtopping without suffering extensive erosion varied significantly throughout the New Orleans area. The areas where the levees were made of clay performed well in spite of the fact that they were overtopped. In other areas the levees were completely washed away after being overtopped. The difference in performance was found to depend on the type of material that was used to construct the levees. Rolled clay levees withstood overtopping the best, and levees constructed of sand and silt by hydraulic filling suffered far more erosion from overtopping.

The investigations described in this report provide a basis for more reliable designs for floodwalls and levees, and the lessons learned from these studies have been incorporated in the design work of the Task Force Guardian team. The findings are also useful in assessing the current conditions and stability of the unfailed sections of the levees and floodwalls.

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Floodwall and Levee Performance Analysis

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, and resulted in high surge levels in the Inner Harbor Navigations Canal, the Mississippi River Gulf Outlet, the Gulf Intracoastal Waterway, and the Mississippi River.

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 volume. The focus of the effort was to assess the performance of floodwalls and levees throughout the system, to investigate the most likely causes of the damage and failure of the levees and floodwalls in the system, to compare the damaged components with similar sections or reaches where the performance was satisfactory, and to understand the mechanisms that led to the breaches in order to assess likely future performance of the un-breached reaches of the flood-protection system, and to provide a basis for design of improvements capable of providing protection against future storms of even greater destructive power than Katrina.

The performance of levees and floodwalls varied significantly throughout the New Orleans area. The investigation described here shows that the two main causes of breaches in the floodwall and levee system were erosion due to overtopping and instability due to soil foundation failure. The investigation has looked at the most likely causes of the damage and failure of the levees and floodwalls in the system and compares them with similar sections or reaches where the performance was satisfactory. It is important to understand in detail the most likely mechanisms that led to the breaches along the reaches in order evaluate the likely future performance of the un-breached reaches of the flood-protection system.

The investigation described here involved: (1) comprehensive assessment of the background information and examination of the entire levee system to identify reaches that performed satisfactorily and those that suffered damage; (2) characterization of the damaged reaches based on the breach mechanism, the surge height, and the wave action; (3) detailed analyses to ensure that site conditions and breach mechanisms are well understood; (4) use of this information to evaluate future performance of the flood-protection system.

Breaches due to instability of floodwalls occurred at one location on the 17th Street Canal, at two locations on the London Avenue Canal, and at one location on the Inner Harbor Navigation Canal (IHNC). Breaches due to erosion as a result of overtopping occurred at three locations on the Inner Harbor Navigation Canal and at many locations on the Mississippi River Gulf Outlet, the Gulf Intracoastal Waterway, and the Mississippi River levees. Figure 1 shows in red the extent of the damage to the levees and floodwalls. Of the 284 miles of levees and floodwalls, 169 miles were damaged. Figure 2 shows the locations of the major breaches in the flood control system.

The performance of levees varied significantly. In some areas, the levees performed well in spite of the fact that they were overtopped. In other areas the levees were completely washed away after being overtopped. The keys to the ability of some levee reaches to withstand overtopping without erosion were: (1) the type of material of which the levees were constructed, and (2) the severity of the surge and wave action to which the levees were subjected.

This report describes the limit equilibrium stability analyses, the centrifuge model studies and the finite element soil-structure interaction studies used to evaluate floodwall and levee performance at the 17th Street Canal, the London Avenue Canal, the IHNC, and the Orleans Canal, followed by the studies performed to assess performance at breaches caused by erosion.

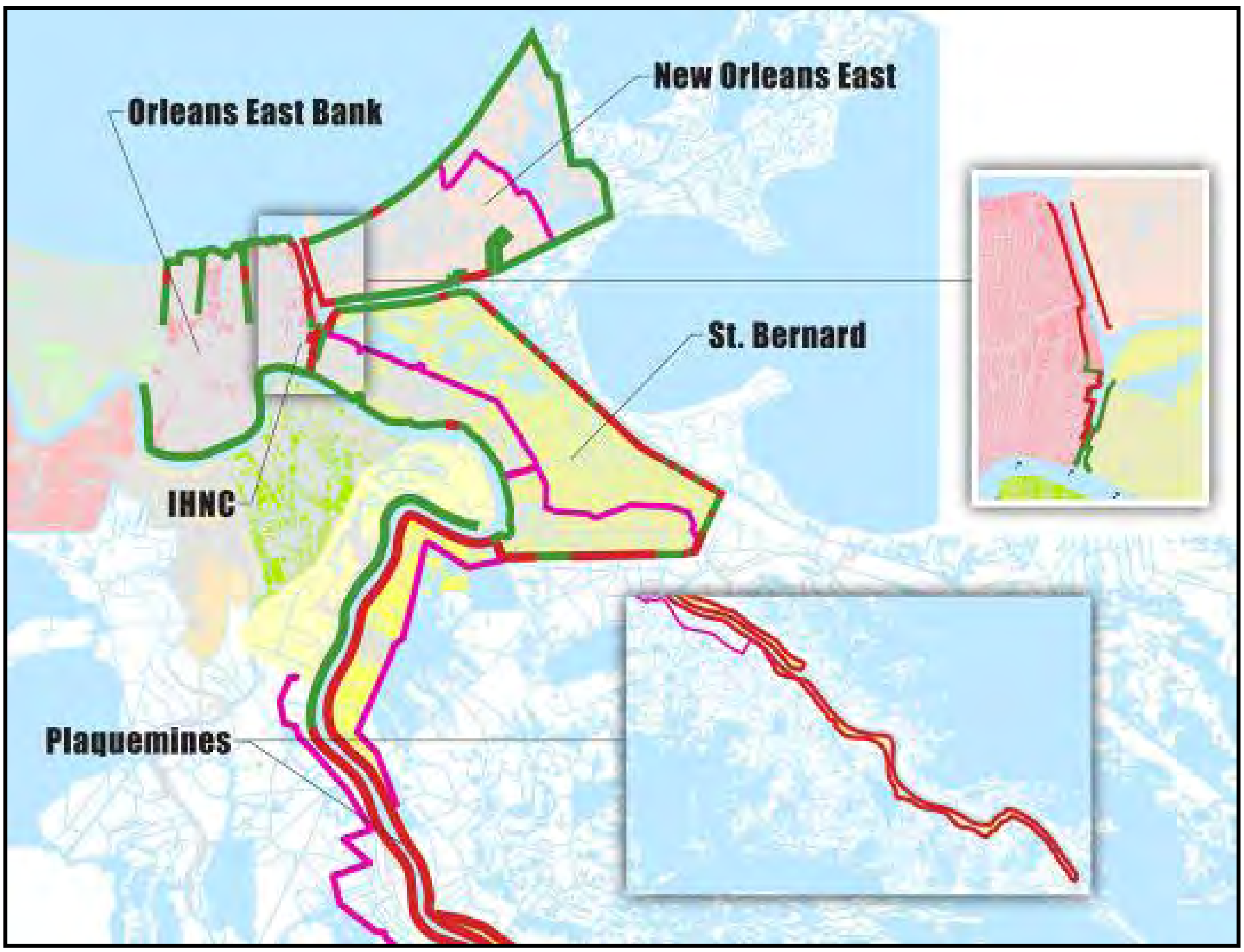

Figure 1. Damage to the New Orleans Hurricane Protection System

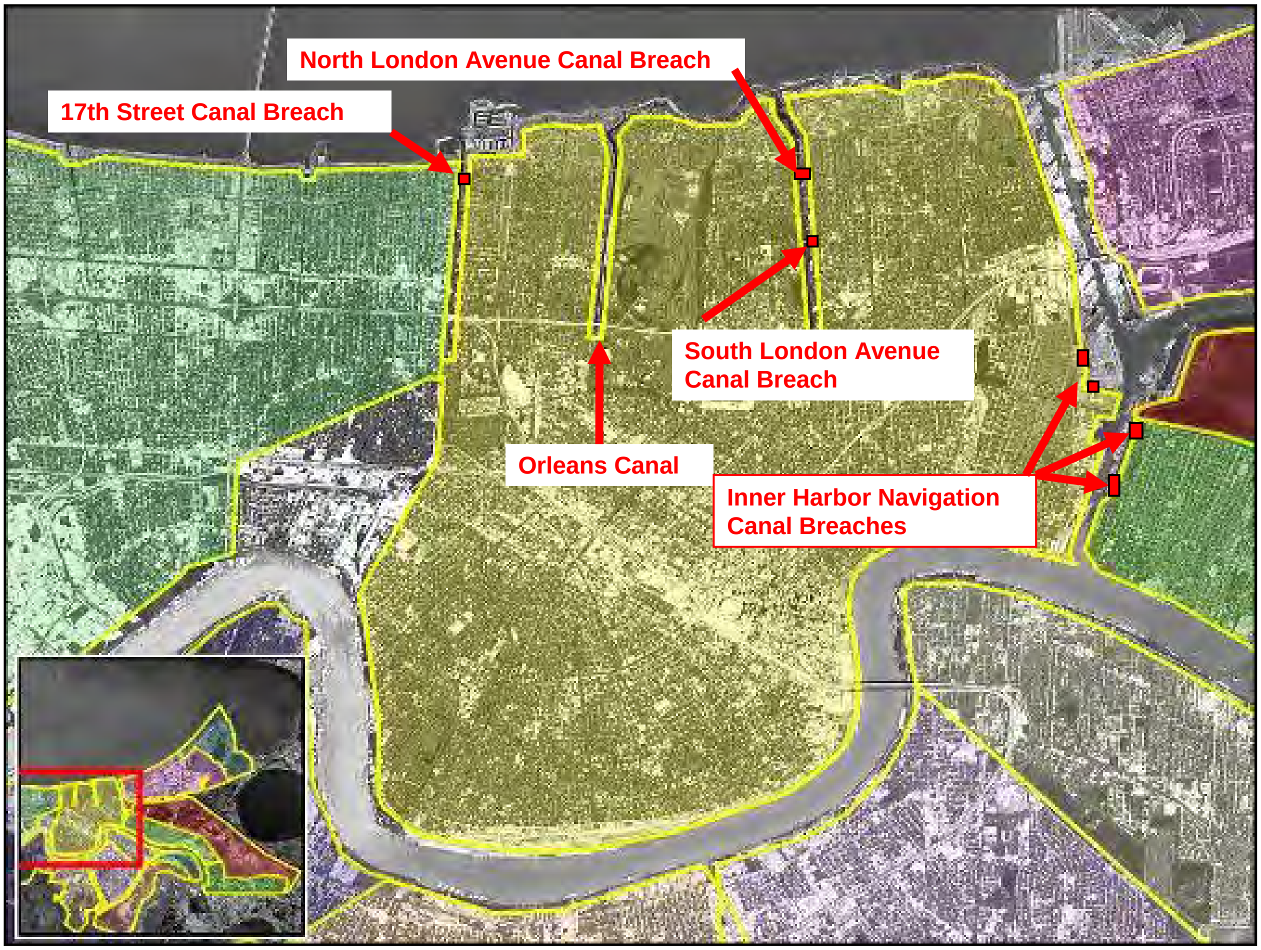

Figure 2. Location of Breaches in Orleans Parish, East Bank

17th Street Canal Breach

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 3 shows an aerial photo showing an approximately 450-ft breach in the floodwall along the east side of the 17th Street Outfall Canal south of the old Hammond Road Bridge. Figure 4 shows that a section of levee has moved more than 40 feet inward to the land side. It appears that the remaining levee section making up the breach was washed away by the water flowing through the breach. 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.

Figure 3. Aerial Photograph of the 17th Street Canal Breach Looking South From the Old Hammond Road Bridge

Figure 4. Aerial Photograph of the 17th Street Canal Breach Showing I-Wall and Embankment Translation

A transverse multi-beam sonar survey of the surface of the canal bottom and breach was conducted before construction of an emergency closure across the breach, at the cross section shown in Figure 5. Topographic cross sections developed from the results of this survey are shown in Figure 6a and 6b. The survey showed that the crest of the levee on the canal side remained essentially in place after the breach, as shown by the ground surface profile at Station 11+50, in Figure 7. It can be seen that the levee and floodwall moved about 40 ft laterally during the failure, and was tilted toward the landside at about 45 degrees after the failure.

Figure 5. Location of Multi-Beam Sonar Survey Cross-Sections at the 17th Street Canal Breach

After the emergency closure was complete and the water levels were drawn down, large blocks of the marsh were found strewn in neighborhoods surrounding the breach, as shown in Figure 8. A close examination of the marsh blocks reveals that an approximately one-ft-thick clay layer is attached to the bottom of the marsh block, Figure 9. In order to inspect the failure plane or zone, a backhoe trench was excavated to expose a vertical surface through the slide block. A photograph of the side of this trench is shown in the upper left corner of Figure 10, and a closer view is shown in Figure 11. The photograph in Figure 11 shows a portion of the clay layer, which was initially located beneath the marsh layer, was displaced upward and over a portion of the marsh layer by the lateral displacement that occurred during the failure. The shearing mechanism that resulted in this condition is shown in Figure 12. This shows that the failure plane of the slide block was within the clay under the levee, and that the failure plane came upward through the marsh layer further landward.

A description of the geology and soil stratification in the area are discussed in the following section.

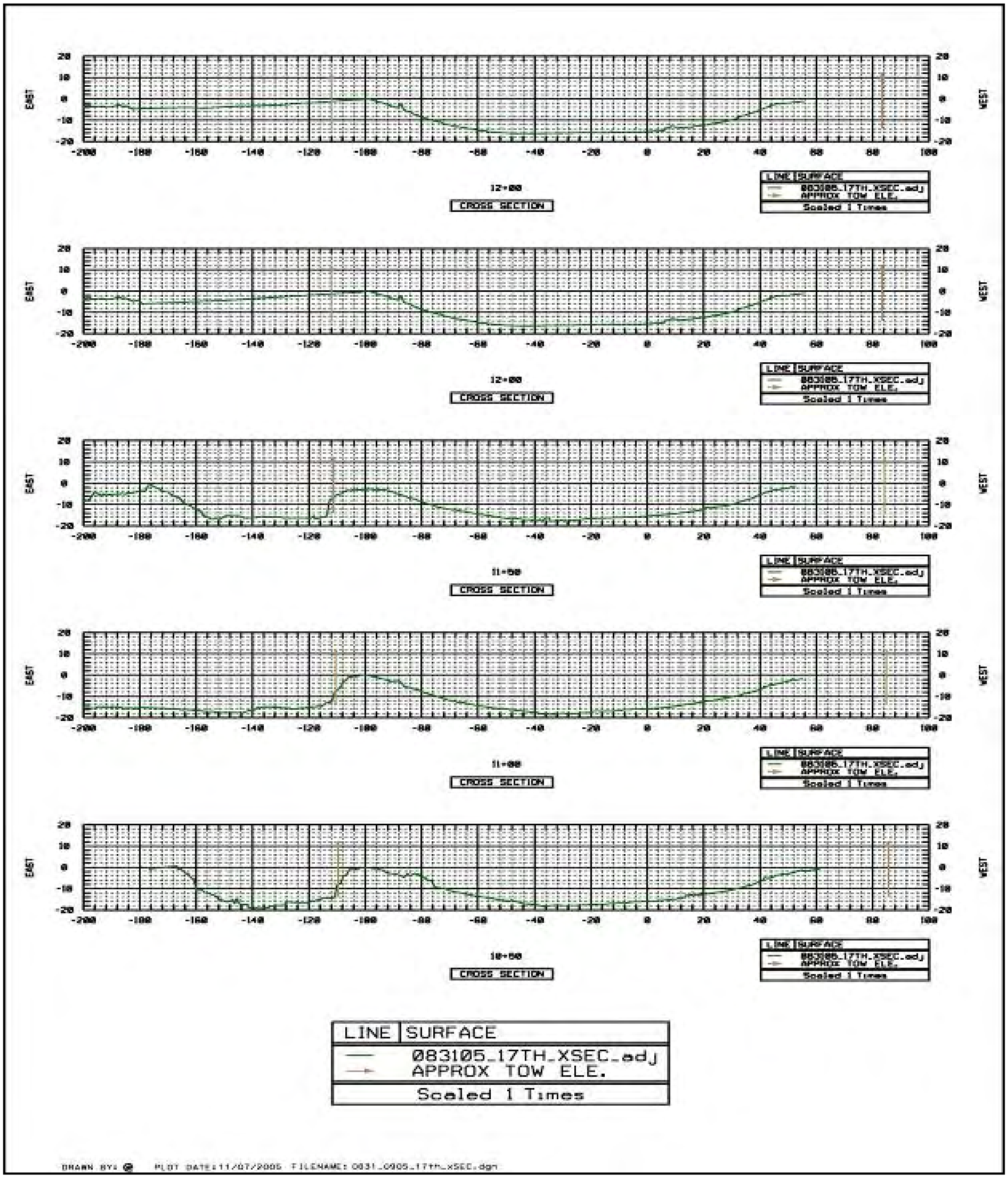

Figure 6a. Surface Profiles at the 17th Street Canal Breach

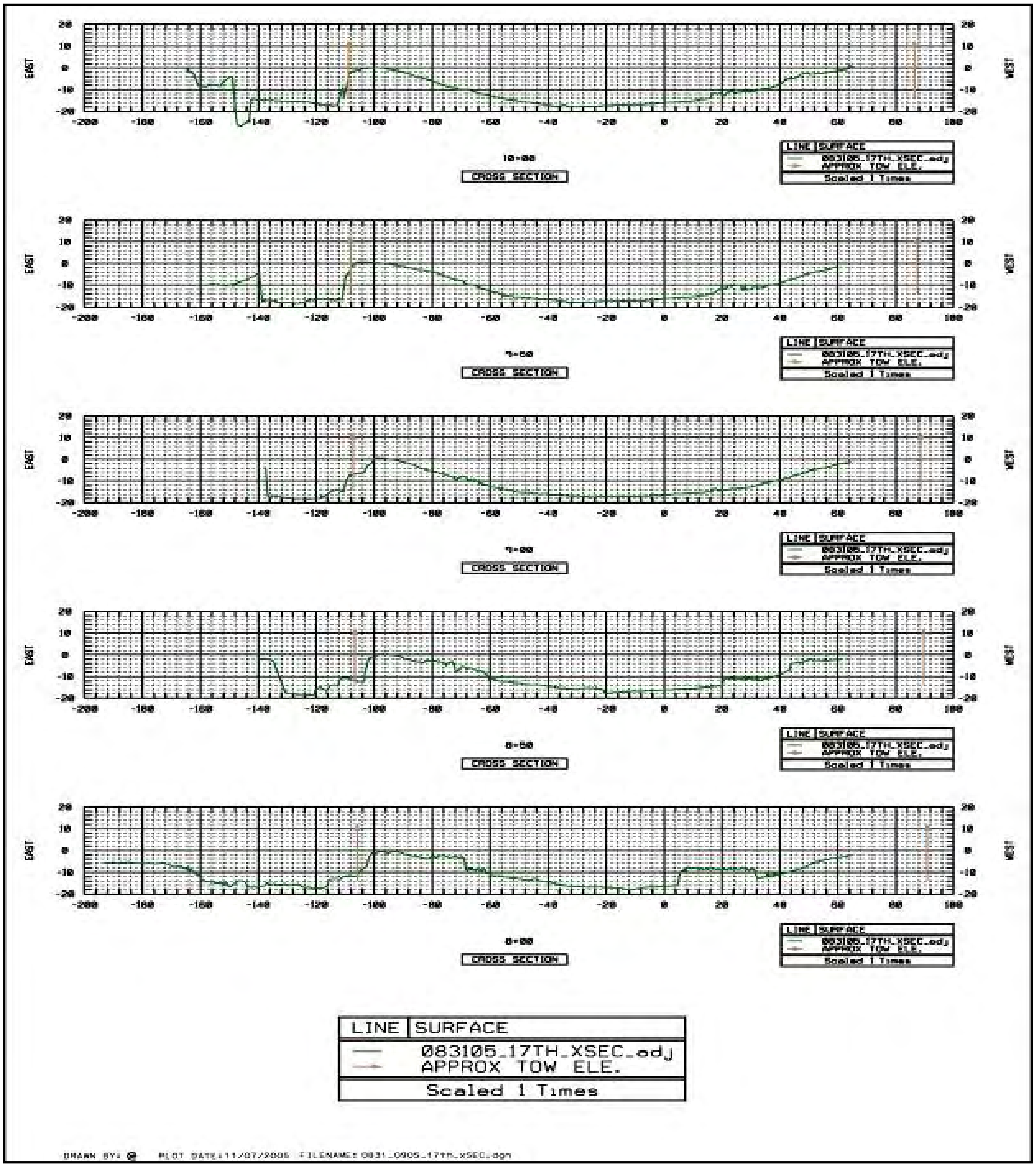

Figure 6b. Surface Profiles at the 17th Street Canal Breach

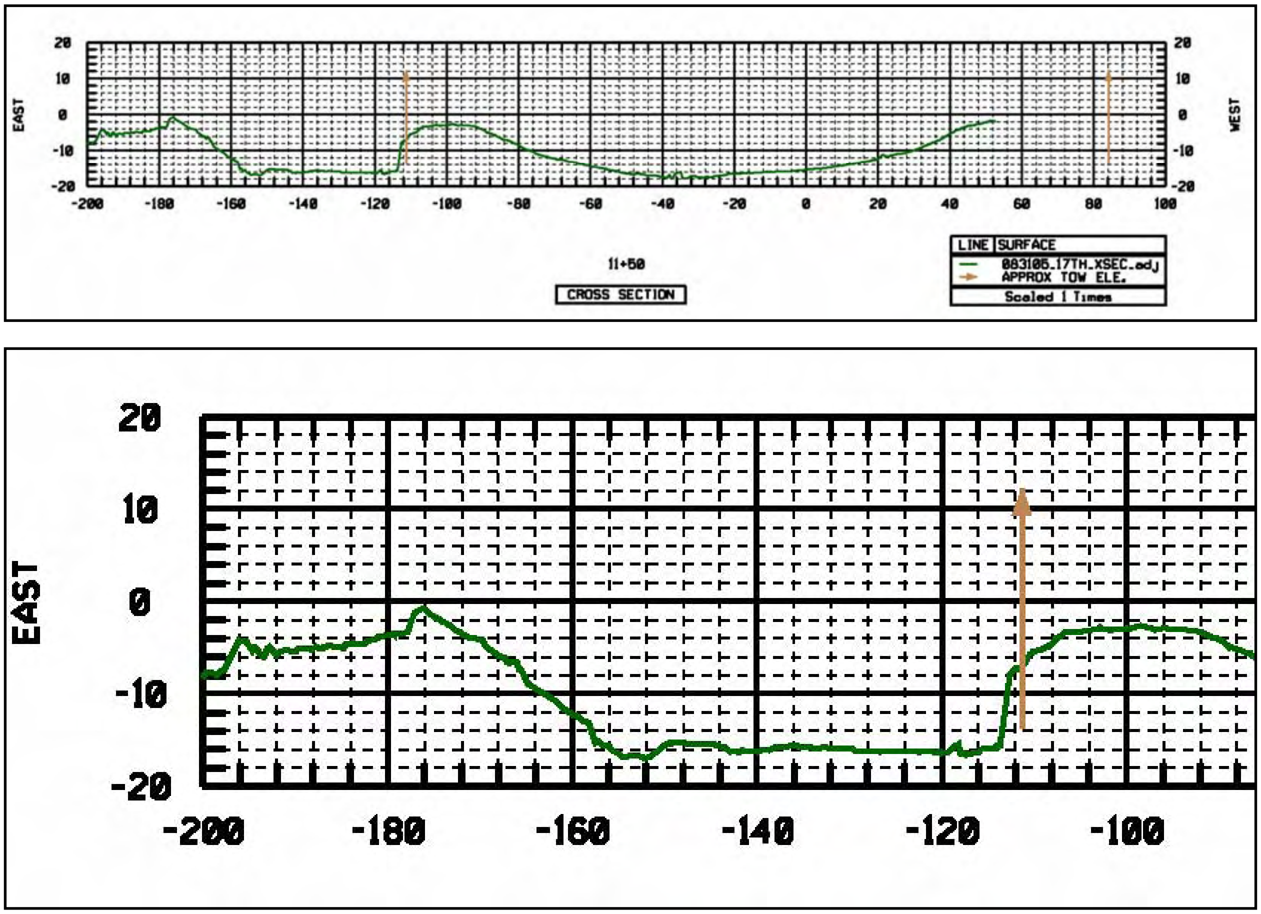

Figure 7. Profile for Station 11+50 Through the 17th Street Canal Breach

Figure 8. Marsh Blocks From the Levee Embankment at the 17th Street Canal Breach

Figure 9. Clay Attached to Marsh Blocks at the 17th Street Canal Breach

Figure 10. Photographs Taken at the 17th Street Canal Breach After the Failure

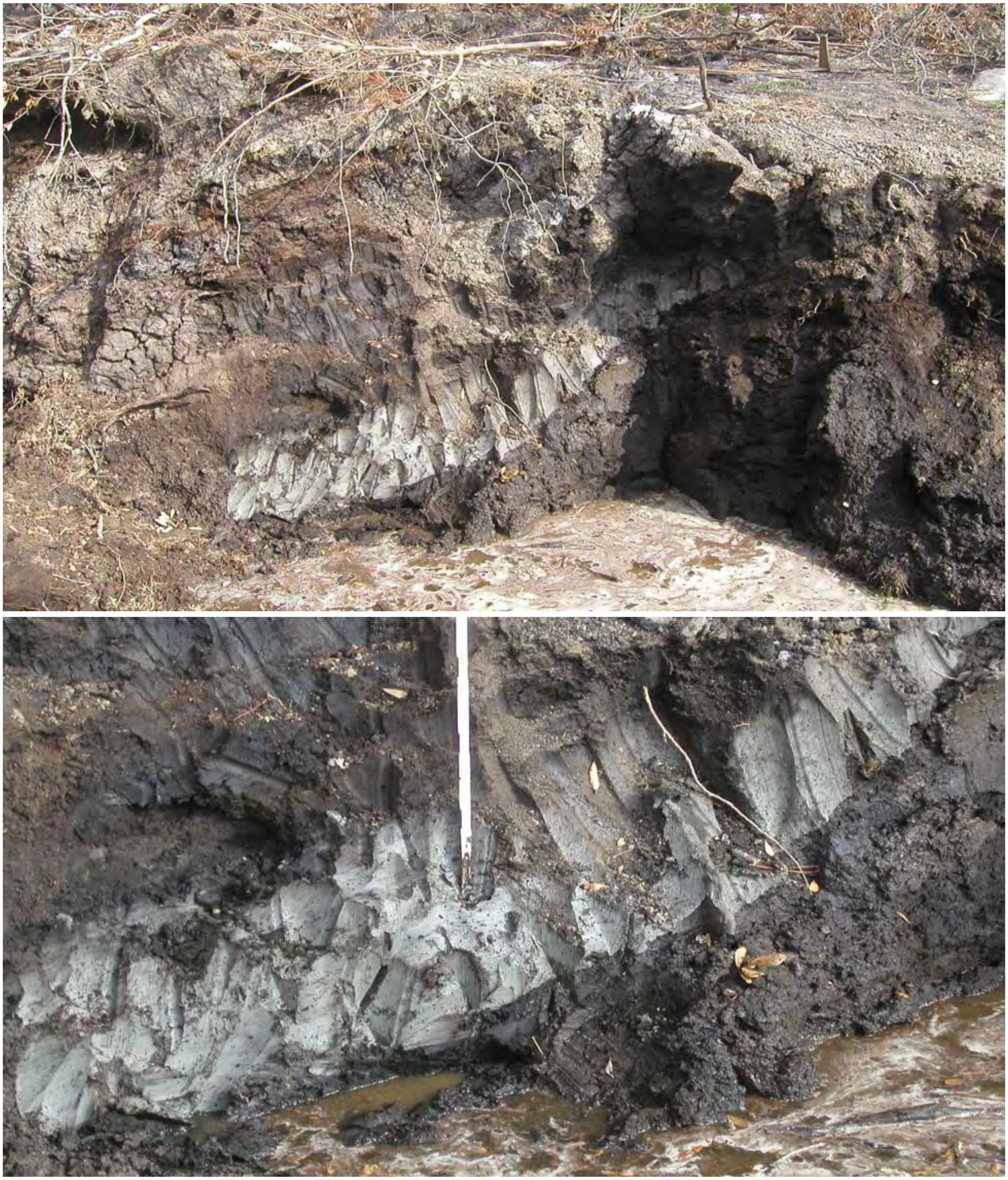

Figure 11. Exposed Failure Plane at the 17th Street Canal Breach, Showing Clay That Was Initially Below the Marsh Layer Was Displaced Above Some of the Marsh Material During the Failure. The marsh material above and below the clay is the same layer, confirmed by age dating. The clay was moved into this position during the failure, as shown in Figure 12

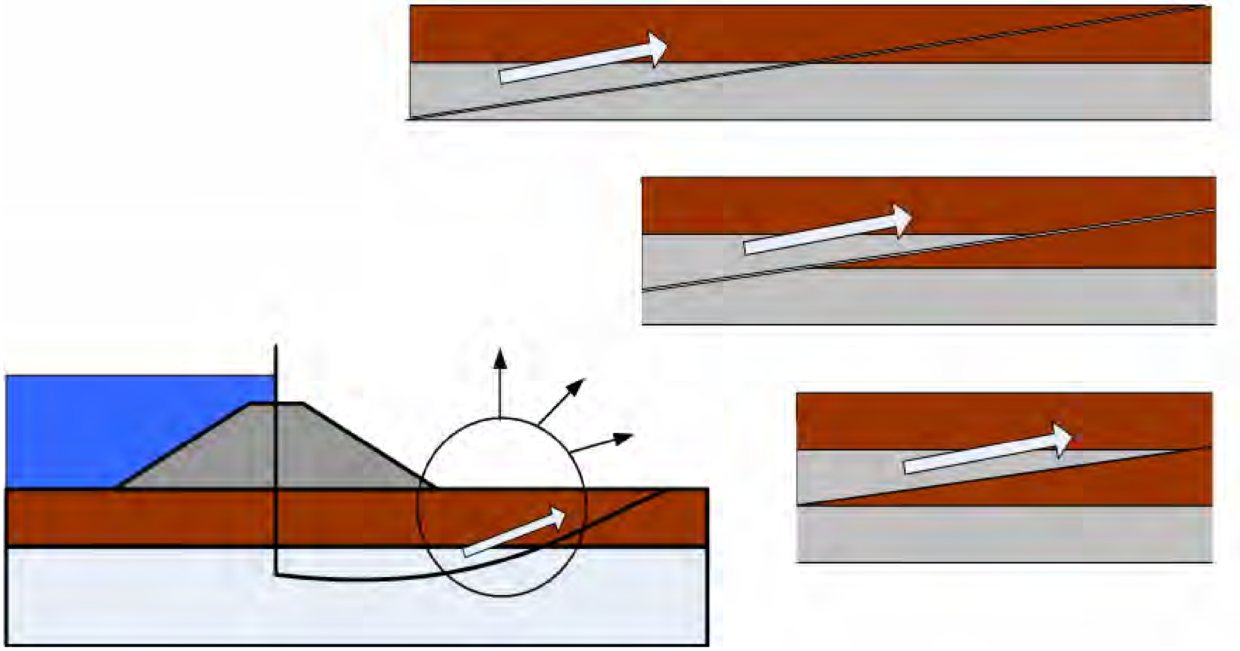

Figure 12. A Portion of the Clay Layer, Initially Located Beneath the Marsh Layer, Was Displaced Upward and Over a Portion of the Marsh Layer as it was Displaced Laterally by the Failure. A photograph of the displaced clay, with marsh material above it and below it, can be seen in Figure 11

Soil conditions and soil properties

The soil conditions in the area of the New Orleans outfall canals has been determined through evaluation of existing and recently drilled engineering borings, earlier geologic mapping studies of the area (Dunbar et al. 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), and new studies performed since August 2005.

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 Pleistocene 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 as indicated in the cross section shown in Figure 13. Overlying this shallow water sequence are approximately 10 to 20 ft of marsh and swamp deposits that correspond to the late stages of deltaic sedimentation as these deltaic deposits became subaerial. A buried barrier beach ridge extends in a southwest to northeast direction in the subsurface, along the southern shore of Lake Pontchartrain, as shown by the geologic map in Figure 14. 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 14, the site of the levee breach at the 17th Street Canal is located on the northern side of the beach ridge where the sand ridge is thinner and there is a layer of clay between the sand and the marsh layer, while both of the London Canal breaches are located over the thickest part of the barrier beach ridge complex, where the sand deposit lies directly beneath the marsh layer, as shown in Figure 13.

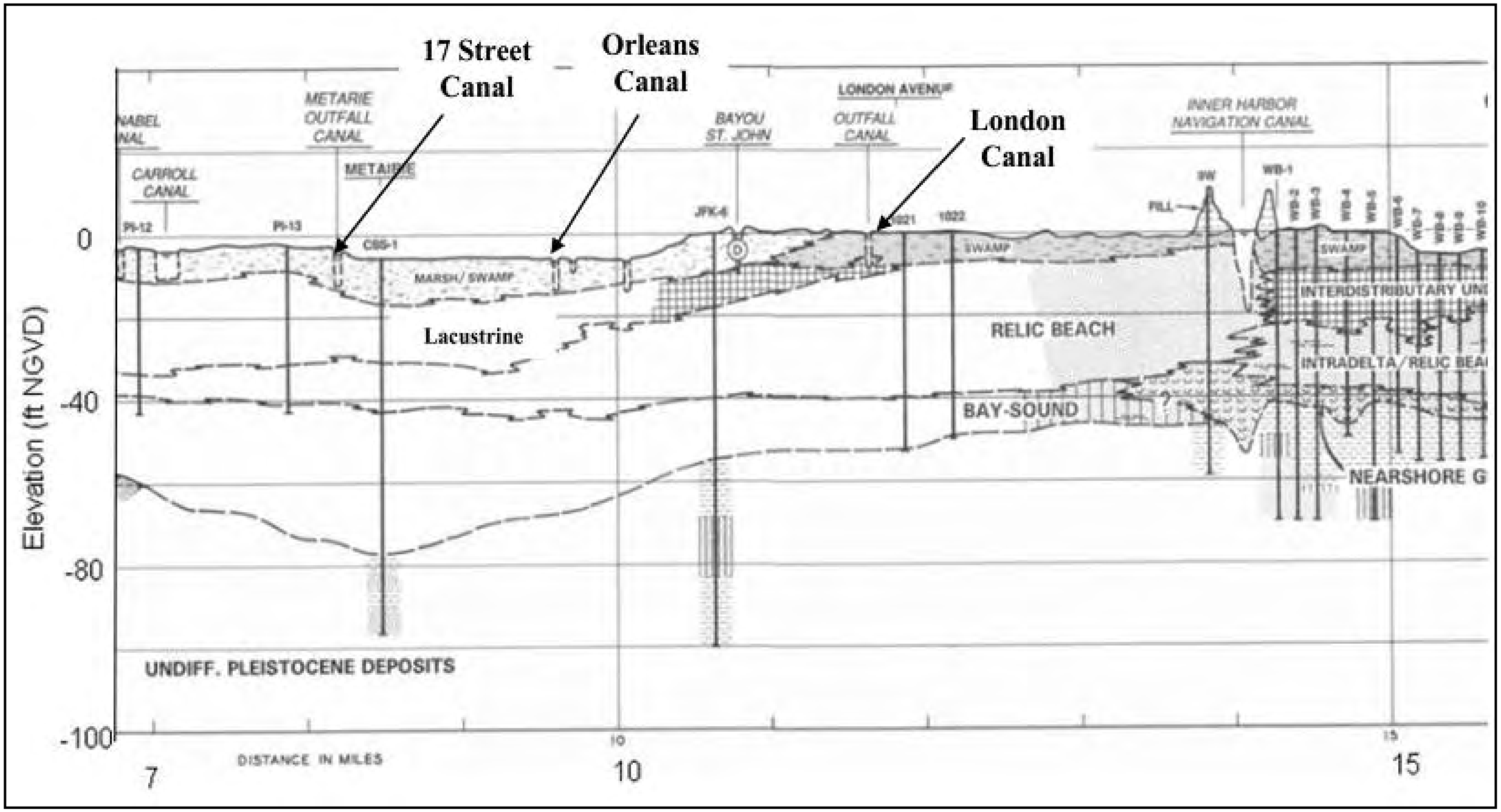

Figure 13. Geological Cross Section Extending West to East Direction Across eastern Jefferson Parish and Into Western Orleans Parish. Section runs from near the 17th Street Canal to the Inner Harbor Navigation Canal. Major outfall canals in Orleans Parish are noted on the section. Cross section shows 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 estuarine), 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*.

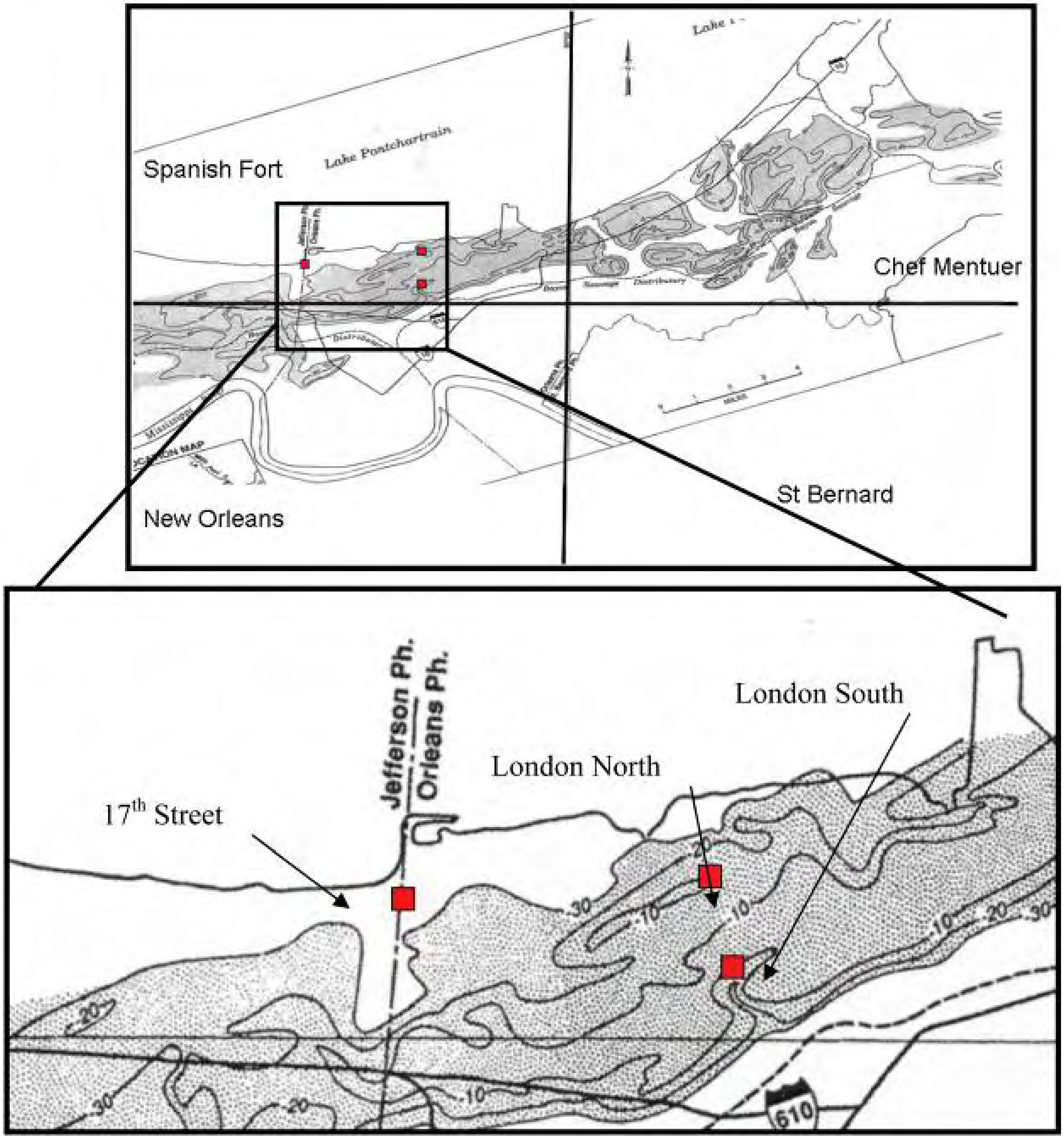

Figure 14. Generalized Contour Map Showing the Pine Island Beach, Contour Values Are in Ft MSL (Saucier, 1994). Upper figure shows general trend of the contours of the top of beach ridge in the New Orleans area, lower figure shows detailed view at the canals. London canal levee failures are located along the axis of the beach. The 17th Street Canal levee break is 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

Shear Strength Assessment for Analysis of the 17th Street Canal Breach

A considerable number of borings had been made 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. Figure 15 shows the locations of the available borings and Cone Penetration Test probes. Several hundred unconfined compression tests and unconsolidated-undrained (UU) tests have been conducted on the soils at the site. A summary of these is presented in the Appendix “Data Report on 17th Street Canal breach.”

A detailed representation of the soil stratification along the centerline of the levee in the breach area is shown in Figure 16. Figure 17 shows the cross section for Station 10+00, from the west bank to the east bank where the breach occurred. The subsurface in the breach area was divided into six soil types over the depth of the investigation, as shown in Table 1.

Table 1
Major Soil Groups at the 17th Street Outfall Canal Breach Site

Layer	Approximate Elevation of Top of Layer, ft (NAVD88)	Approximate Elevation of Bottom of Layer, ft (NAVD88)	Soil Type	Consistency
Embankment	5	-11.5	Clayey (CL’s and CH)	Stiff
Marsh	-11.5	-16.5	Organic/Peat	Very Soft
Lacustrine	-16.5	-36.5	Clays (CH)	Very Soft
Beach Sand	-36.5	-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

Figure 15. Boring and CPT Location Map – 17th Street Canal Breach

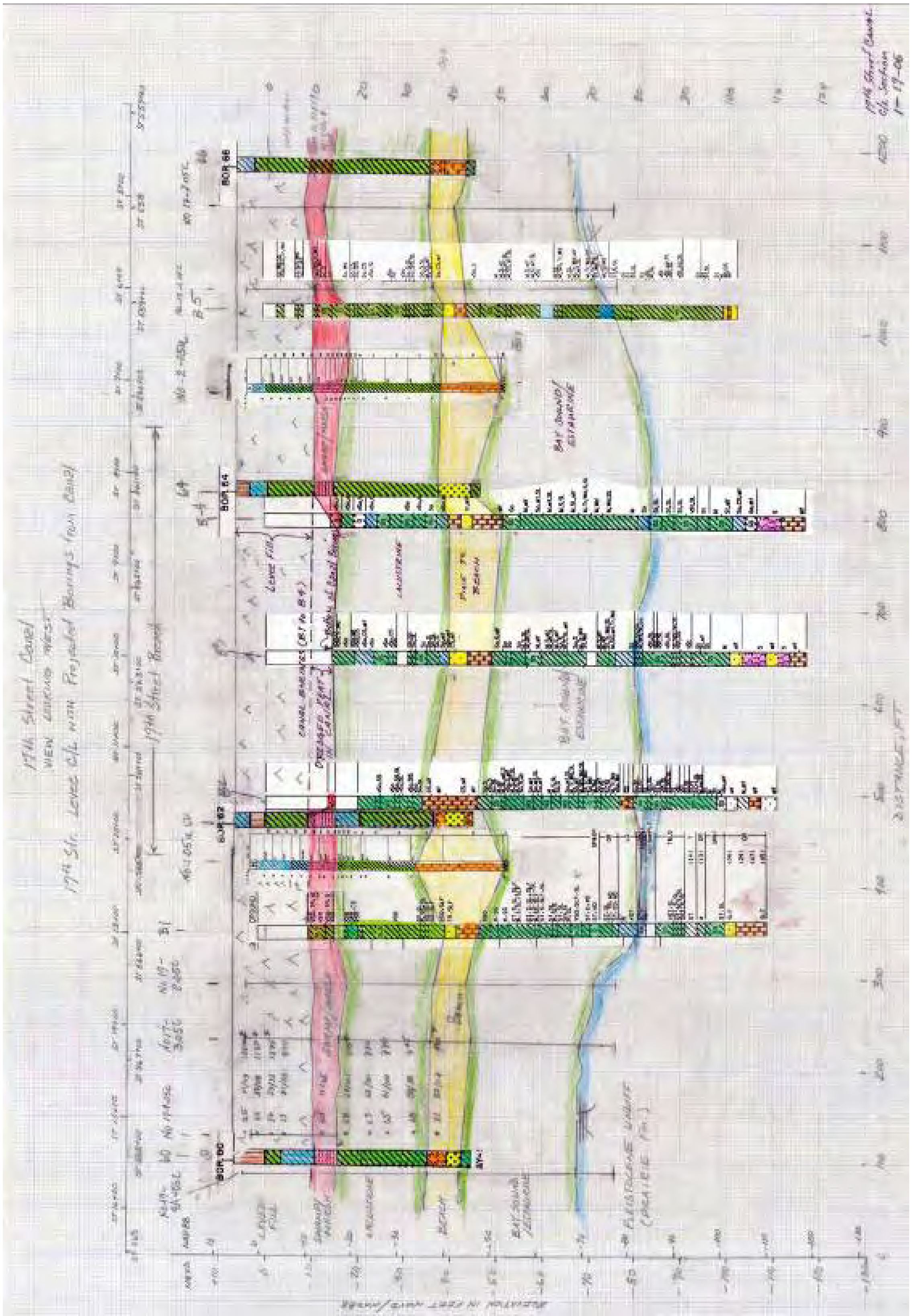

Figure 16. Centerline (CL) Longitudinal Section – 17th Street Canal Breach

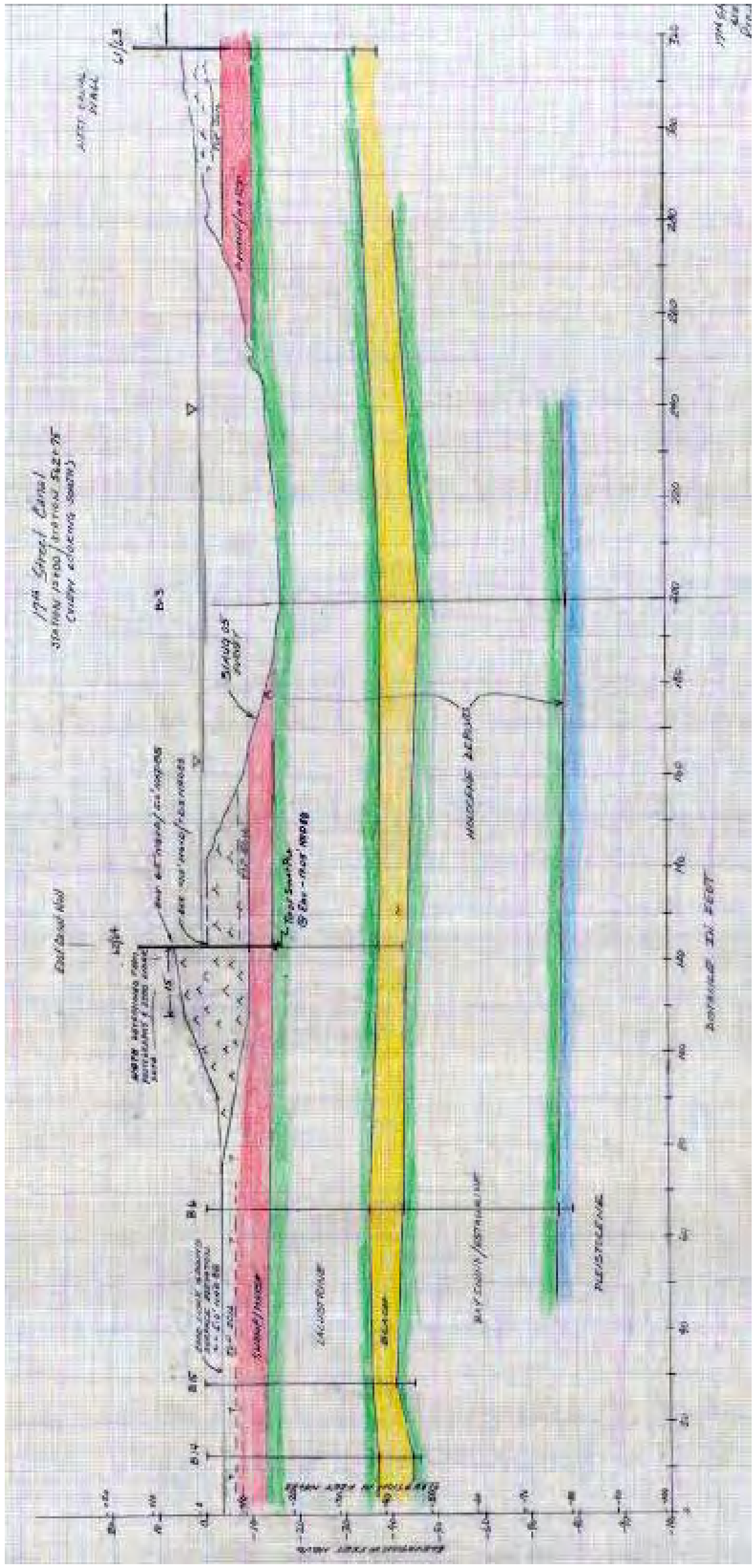

Figure 17. Prefailure Cross Section 17th Stree Canal Breach (New Stationing Sta 10+00) / (GDM Stationing Sta. 562+75)

The data available from previous and new studies in the 17th Street Canal area were used 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 levee fill is compacted CL or CH material, with an average liquid limit of about 45%. Beneath the fill is a layer of marsh 5 ft to 10 ft thick. The marsh 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 marsh is about 80 pcf. Beneath the marsh is a lacustrine clay 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 marsh.

The measured shear strengths of the levee fill scatter very widely, from about 120 psf to more than 5,000 psf, and cannot be interpreted without applying judgment. The values used are based on the combined judgment of the IPET team to make the most reasonable interpretation of the scattered data. Placing the greatest emphasis on data from UU tests on 5-in.-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 marsh (or peat) deposit is stronger beneath the levee crest where it was consolidated under the weight of the levee, and weaker at the toe of the levee and beyond, where it less compressed. The measured shear strengths of the marsh 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 at all locations that was used in the design analyses.

The clay (which has been found to be 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. 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 least at the levee toe and beyond where the pressure is lowest, and varying with depth, increasing at a rate of 11 psf per foot at all locations.

The design analyses used undrained strengths for the levee fill, the marsh, 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 2:

Table 2
Comparison of Strengths of the Levee and Marsh 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
Marsh	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 marsh used in the 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) is essentially the same in the IPET 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 the design are considerably higher than the strengths from the IPET strength model.

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 marsh 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, “17th Street Shear Strength Final Draft.”

Limit Equilibrium Stability Assessment

Limit equilibrium analyses were used to examine stability of the levee and I-wall. The results of these analyses are interpreted in terms of factors of safety and probabilities of failure.

Stability analyses were performed for three cross sections within the breach area 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[1]. The SLIDE analysis results were checked using UTEXAS4[2].

It was found that:

- The calculated factors of safety decreased as the elevation of the water level on the canal side of the wall increased, and

- Smaller factors of safety were calculated when it was assumed that a gap existed between the wall and the soil on the canal side of the wall, with hydrostatic water pressures acting within this gap, increasing the load on the wall.

It seems likely that, as the water level in the canal rose, the I-wall deflected towards the land side, causing it to pull away from the levee fill. When the resulting gap between the fill and the wall filled with water, the increased hydrostatic pressure became a significant factor in decreasing the wall stability.

The results of the analyses are 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 condition with a gap behind the wall vary from 9.8 ft to 10.6 ft, as compared with a water level of 7 ft to 8 ft at the time failure began based on an eyewitness report. It appears that wave effects might raise the effective water level by 1 to 2 ft, to as much as 10 ft. In addition, it was found that using non-circular slip surfaces reduced the calculated factors of safety by about 5% as compared with those calculated using circular slip surfaces. Thus, considering the influence of waves increasing the effective average water level, and non-circular slip surfaces resulting in lower factors of safety, it is apparent that the results of the stability analyses are in close agreement with the observed performance.

The calculated factors of safety are about 25% lower when it is assumed that a separation or gap develops between the wall and the levee fill on the canal side of the wall. The results calculated assuming that a gap formed and full hydrostatic water pressure acted in the gap, are consistent with field observations, indicating that it is highly likely that a gap did form in the areas where the wall failed. It seems likely that when a gap 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 have confirmed this mode of behavior.

The New Orleans District Method of Planes[3] used for the design analyses is a conservative method of slope stability analysis. All other things being equal, the factor of safety calculated

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[1] Available from Rocscience Inc., 31 Balsam Avenue, Toronto, Ontario, Canada M4E 3B5
[2] Available from Shinoak Software, 3406 Shinoak Drive, Austin, TX 78731
[3] A study of the Method of Planes, undertaken by IPET at the request of the New Orleans District Task Force Guardian, indicates that the Method of Planes gives lower factors of safety than more accurate methods of analysis, such as Spencer's method. The magnitude of the difference between the two varies from case to case.

using the Method of Planes was about 10% lower than the factor of safety calculated using Spencer's method[4], 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 gap 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 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; and it thus appears that the margin of safety was small in areas that did not fail. It is possible that gaps did not form in those areas, and the wall was, therefore, less severely loaded.

Estimates of probability of failure for a water level of 7.0 ft NAVD88 are about 12% in the breach area, and 1% in adjacent areas with clay strengths 20% higher. For a water level of 10.0 ft, the estimated probability of failure is 58% in the breach area and 16% in adjacent areas.

A more complete description of the stability analyses and results is contained in the Appendix, "17th Street Stability Final Draft."

Centrifuge Modeling Results for the 17th Street Canal Breach

The physical (centrifuge) models of the levees on 17th Street, London Avenue, and Orleans canals have provided detailed insights into the mechanisms that led to the breach. A more complete description of the centrifuge modeling effort is contained in the Appendix, "IPET Centrifuge Model Test Report." The centrifuge modeling has contributed to the overall understanding of the performance of the outfall canal levees.

The centrifuge model results revealed that the gap formation had a major contribution to all of the beaches on the outfall canals. In all of the scale models where the toe of the sheet pile wall terminated in the clay layer, like the 17th Street Canal breach location, a translational failure occurred through the clay when the gap opened and filled with water. This is clearly seen in the instruments recording movement of the wall and in the video imagery, Figure 18.

The sliding surface developed near the top of the clay layer and progressed landwards until it was outside the levee, then turned upward to exit through the marsh layer. This mechanism is similar to the observations in the field from 17th Street. The movement of the model wall was arrested before larger displacements took place to capture the final state of the 17th Street model (Model 1), Figure 19. Despite the fairly significant lateral movement, there is minimal heave of the swampy marsh layer on the landward side at the stage shown in Figure 19.

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[4] Spencer, E. (1967) "A Method of Analysis of the Stability of Embankments Assuming Parallel Inter-Slice Forces," *Geotechnique*, Institution of Civil Engineers, Great Britain, Vol. 17, No. 1, March, pp. 11-26.

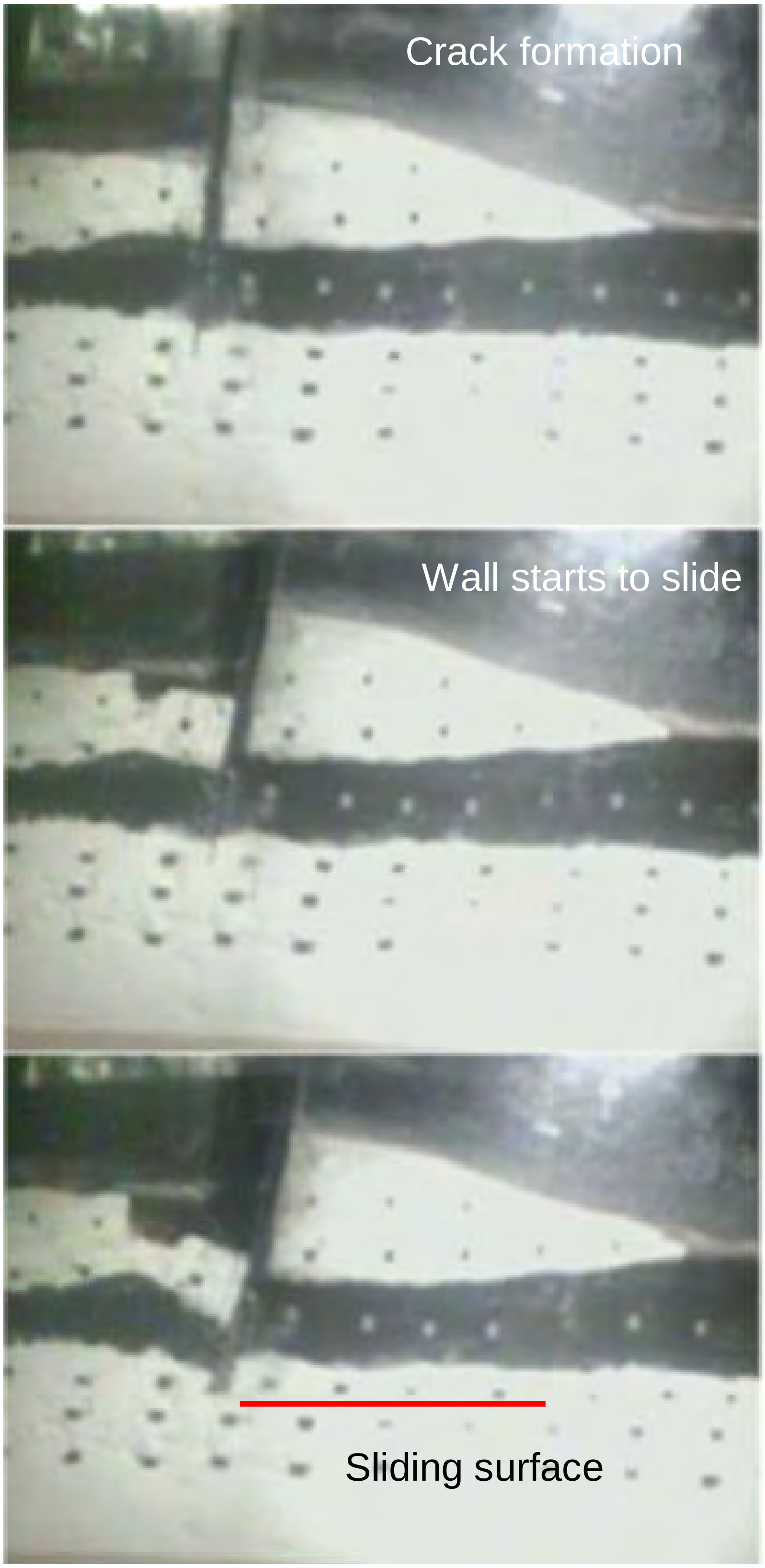

Figure 18. Sliding Surface Forms at Top of Clay Layer in 17th Street Canal Model 1

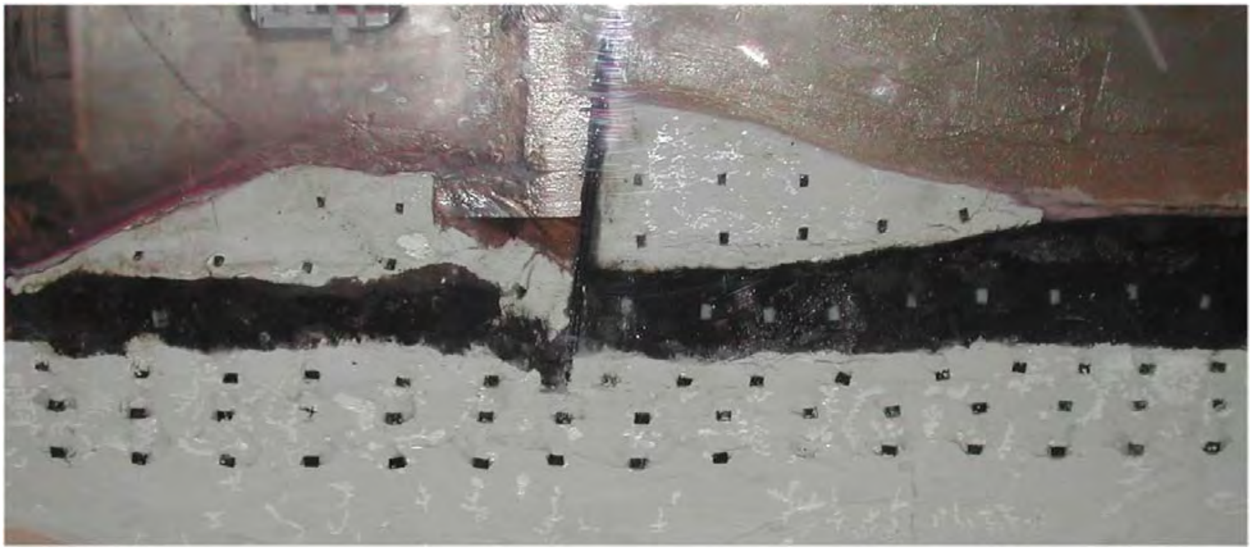

Figure 19. Final State of 17th Street Model 1 Showing Lateral Translation on Sliding Surface Starting at the Toe of the Wall and Progressing Landward

All of the clay foundation models failed with a similar mechanism under the flood loading, commencing with a small rotation and formation of a gap behind the wall, followed by further rotation and large translation associated with a shear plane that formed from the toe of the sheet pile wall and progressed landwards through the upper part of the clay layer. This was consistent with the field observations.

Other observations relevant to the site conditions on the 17th Street Canal were also noted from the centrifuge model tests. The very soft, normally consolidated clay beneath the levee experienced significant settlements as the weight of the levee was increased to its full value. The marsh layer also tended to be compressed below the levee, but predominantly through elastic compression.

In the 17th Street Canal models, the formation of the gap behind the wall was followed by the immediate development of a slip movement at the toe of the wall, and an increasing rate of landward movement. As the initial slip surface extended toward the toe of the levee, the weaker clay farther from the centerline of the levee was less able to resist the driving forces acting on the levee block.

A more complete description of the centrifuge modeling effort is contained in the Appendix, “IPET Centrifuge Model Test Report.”

Finite Element Soil-Structure Interaction Results for the 17th Street Canal Breach

Finite element soil-structure interaction analyses were conducted to provide a third approach to development of a complete understanding of the 17th Street Canal breach mechanism. A two-dimensional (2-D) cross section at Station 10+00 of the east side of the 17th Street Canal at the breach location was modeled, as shown in Figures 20 and 21. A detailed description of the nonlinear finite-element soil-structure interaction analyses and results is contained in the Appendix, "17th Street SSI Station 10+00."

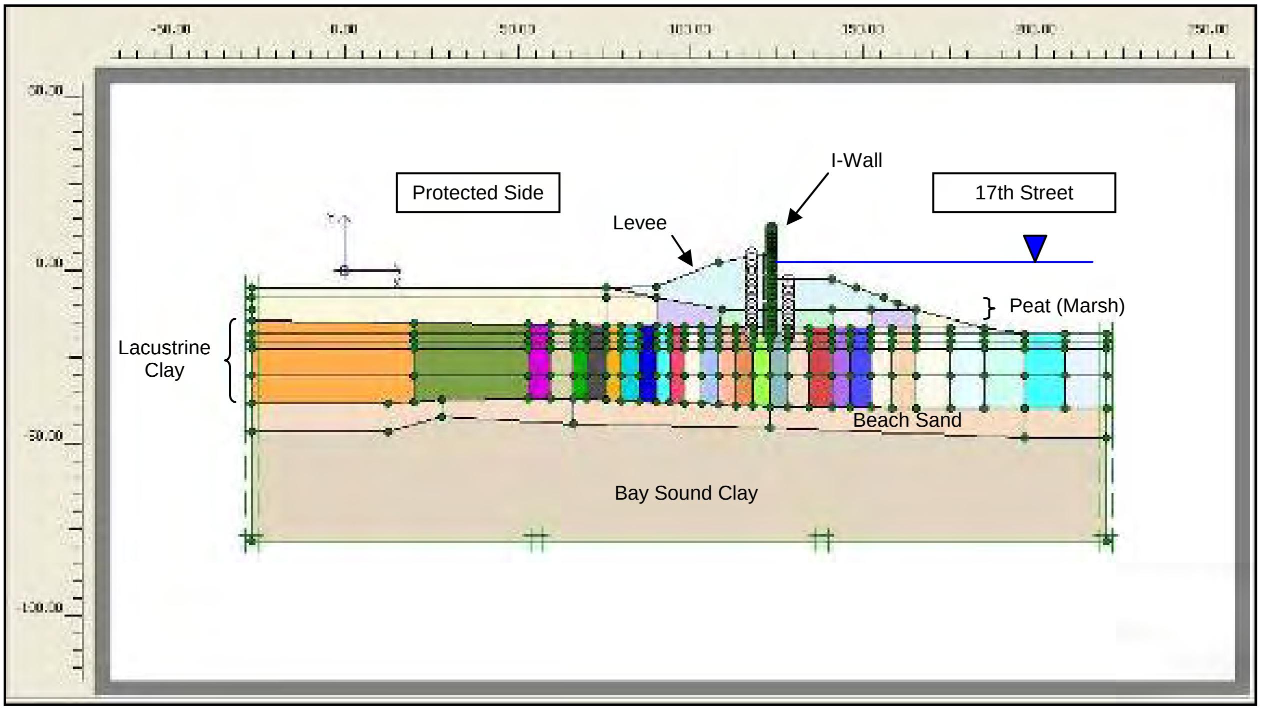

Figure 20. Two-Dimensional Cross-Section Model Used in Soil-Structure Interaction Analysis of Station 10+00 on the 17th Street Canal East Side Breach

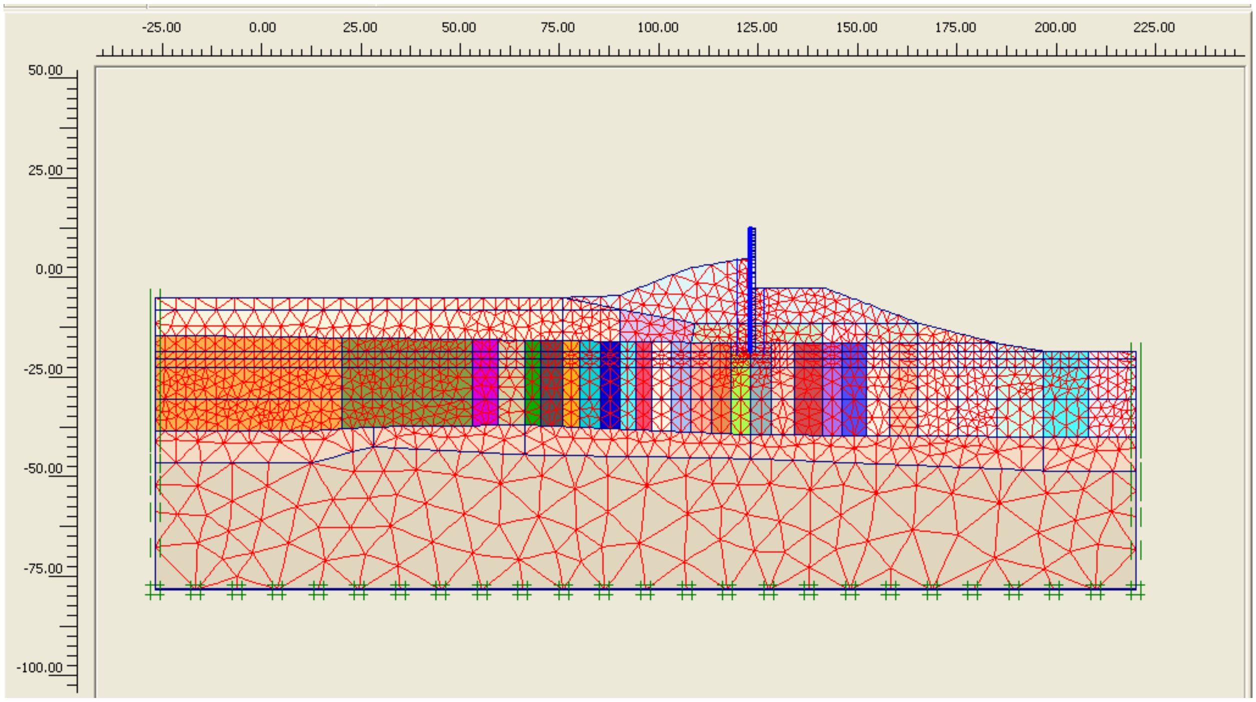

Figure 21. Finite-Element Mesh Used in Soil-Structure Interaction Analysis of Station 10+00 on the 17th Street Canal East Side Breach

The finite element analyses show that gap formed as the water rose on the canal side of the wall. The criterion for gap formation was earth pressure against the wall less than the hydrostatic water pressure at that depth. When that condition was reached, the finite element mesh was adjusted to separate the soil elements from the wall elements with a gap. The gap began to open when the water in the canal rose to elevation 6.5 ft. Eventually, the gap extended to the tip of the sheet pile, which was at the lacustrine clay-marsh interface. Factors of safety were computed using the strength reduction method. The strength reduction method involves performing a series of finite element analyses using values of the strength parameters c and ϕ (or c' and ϕ') that are reduced by dividing them by assumed values of factor of safety. The correct factor of safety, as determined by this method, is the smallest value that results in unstable conditions in the analysis. When the gap developed and filled with water the factor of safety decreased suddenly by about 25%, from 1.45 to 1.16, as shown in Figure 22.

Figure 22. Factor of Safety Versus Canal Water Elevation Computed in the Soil-Structure Interaction Analysis of Station 10+00 on the 17th Street Canal East Side Breach

The development of a gap, which immediately filled with water, resulted in a marked increase in calculated displacements, as shown in Figure 23. As the water in the canal rises from El 6.5 to El 9.0, the maximum lateral deformation increases from 1.6 ft, the condition shown in Figure 24, to 5.3 ft, the conditions shown in Figure 25; and the factor of safety decreases from 1.16 to 0.98. Figure 26 shows that the contours of maximum shear strains form a shear surface that compare well with the critical failure surface found in the limit equilibrium stability analysis, Figure 27.

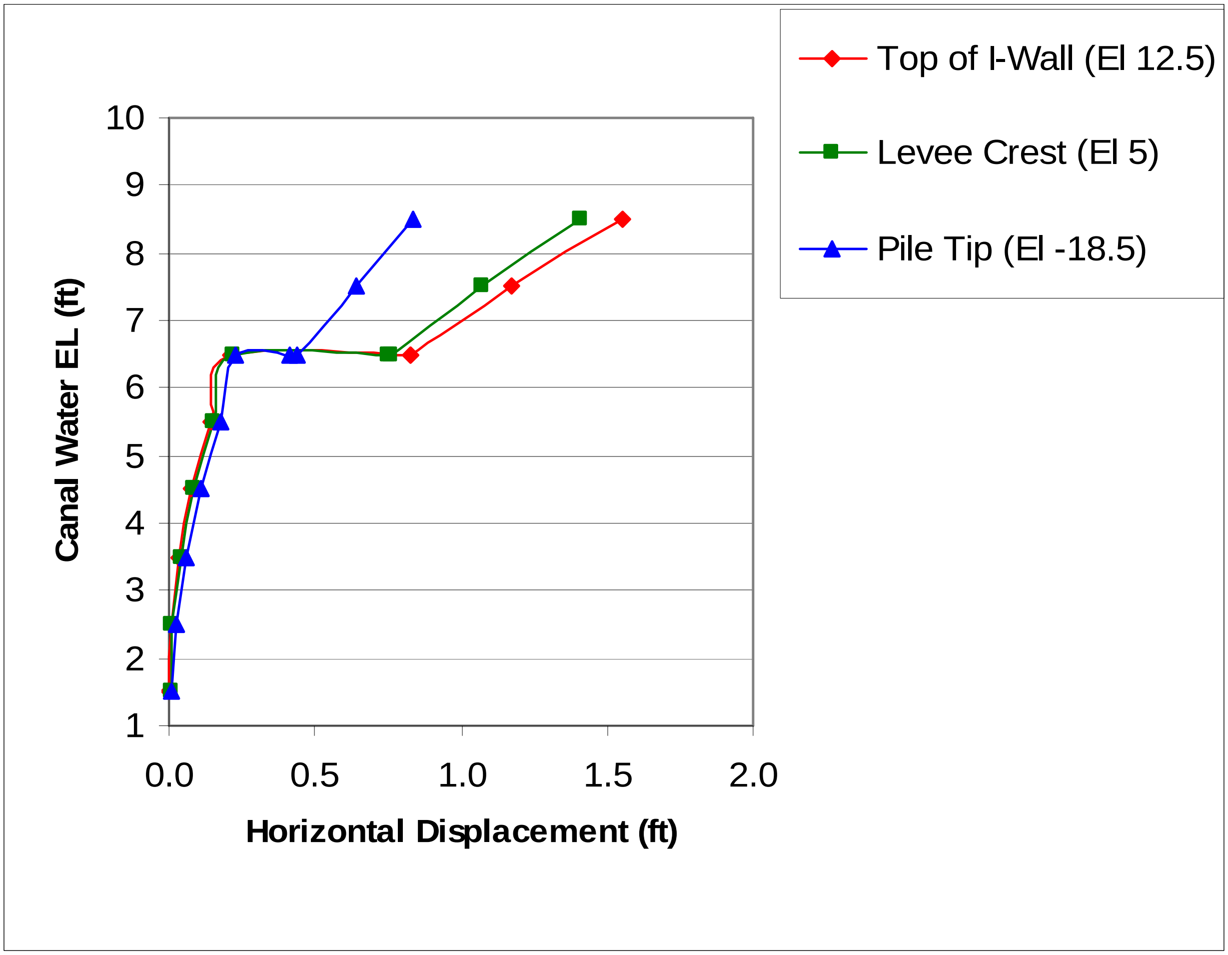

Figure 23. Horizontal Sheet Pile Deflections Versus Canal Water Elevation Computed in the Soil-Structure Interaction Analysis of Station 10+00 on the 17th Street Canal East Side Breach

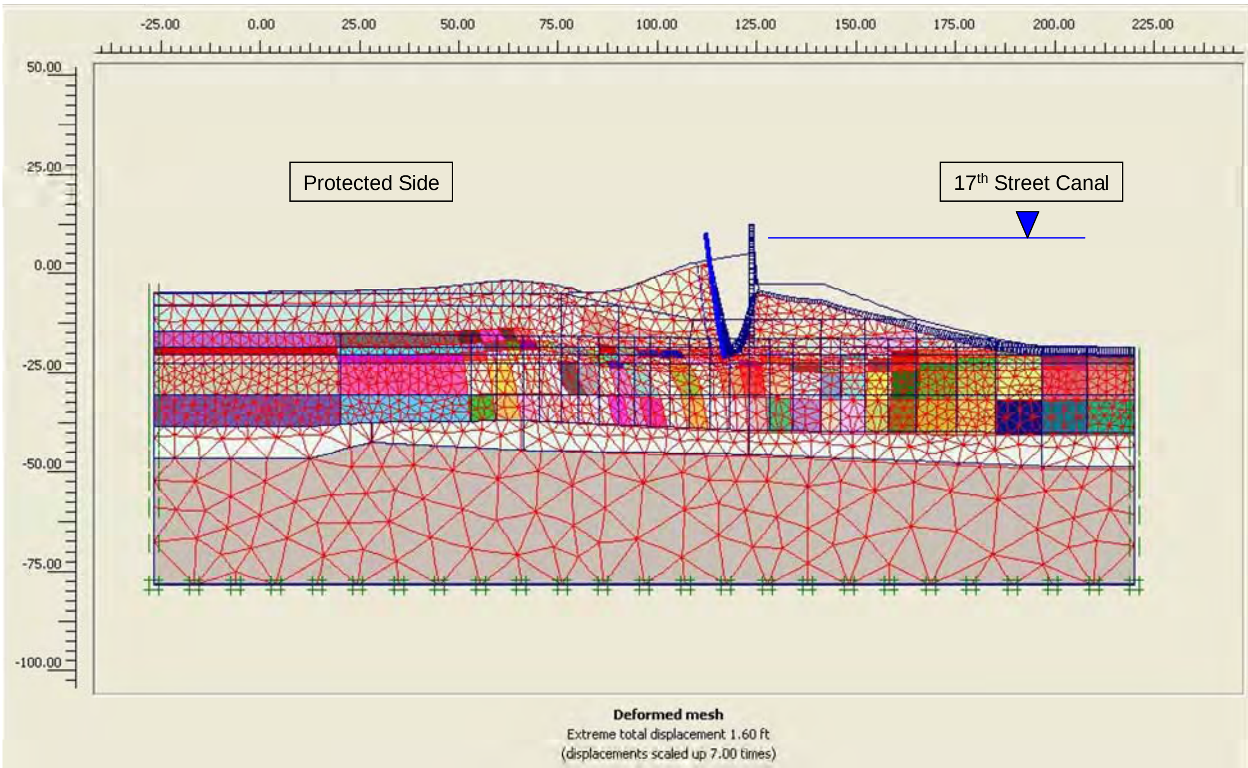

Figure 24. Deformed Mesh for Canal Elevation 6.5 ft and Gap to Elevation -18.5 ft Computed in the Soil-Structure Interaction Analysis of Station 10+00 on the 17th Street Canal East Side Breach (Note: Canal elevation not to scale in figure)

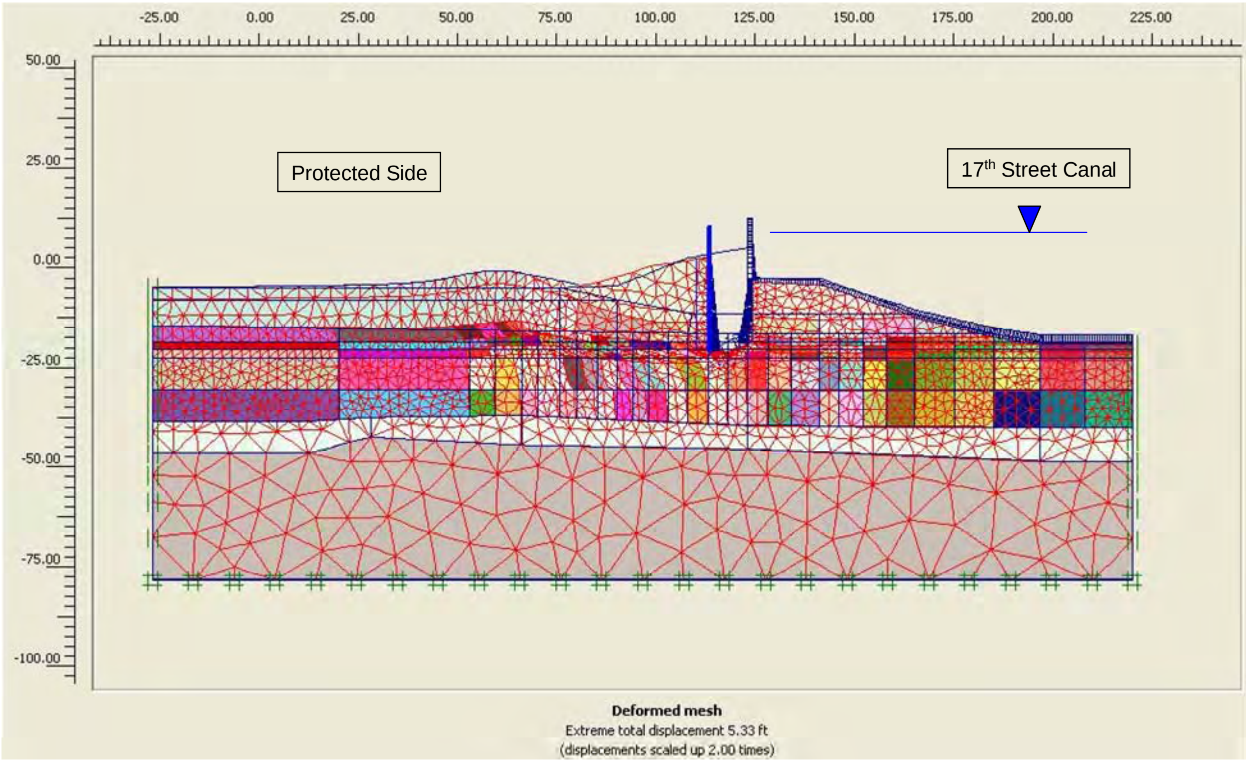

Figure 25. Deformed Mesh for Canal Elevation 9.0 ft and Crack to Elevation -18.5 ft Computed in the Soil-Structure Interaction Analysis of Station 10+00 on the 17th Street Canal East Side Breach (Note: Canal Elevation not to scale in figure)

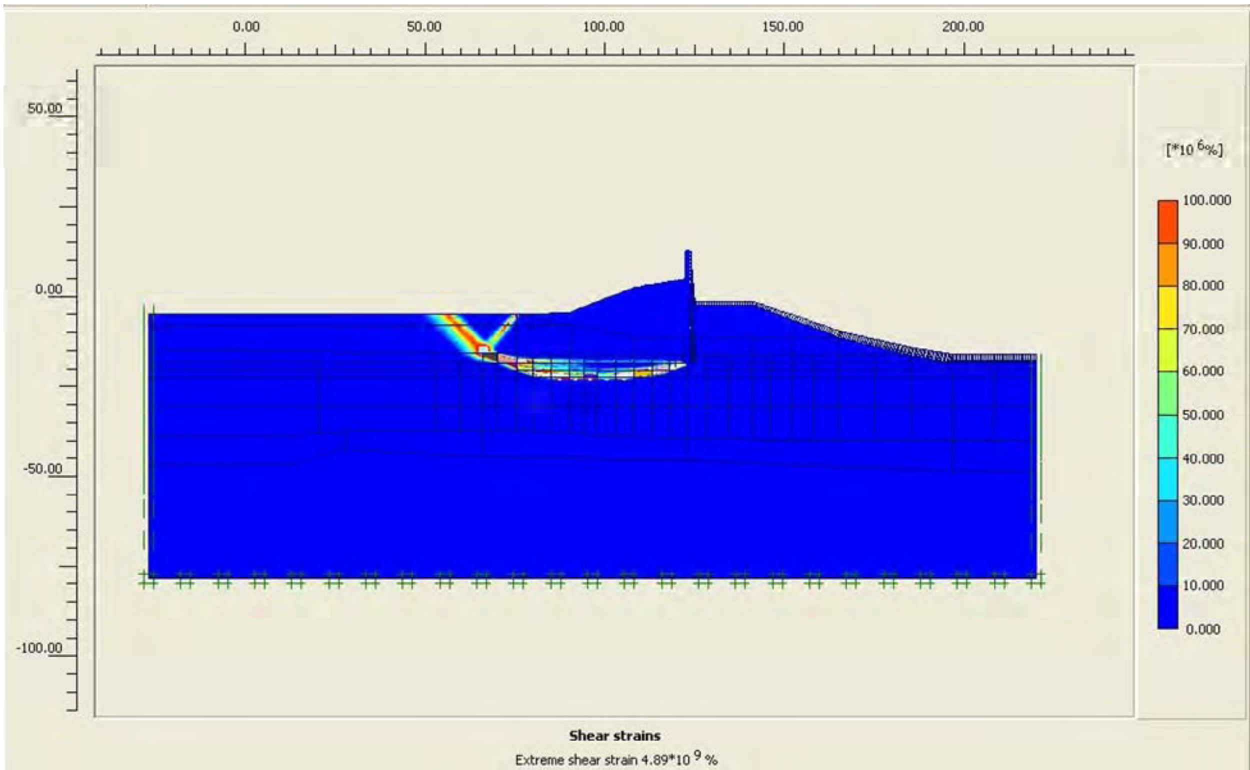

Figure 26. Reaches of Large Shear Strains from Strength Reduction for Canal Elevation 9.0 ft and Gap to Elevation -18.5 ft Computed in the Soil-Structure Interaction Analysis of Station 10+00 on the 17th Street Canal East Side Breach

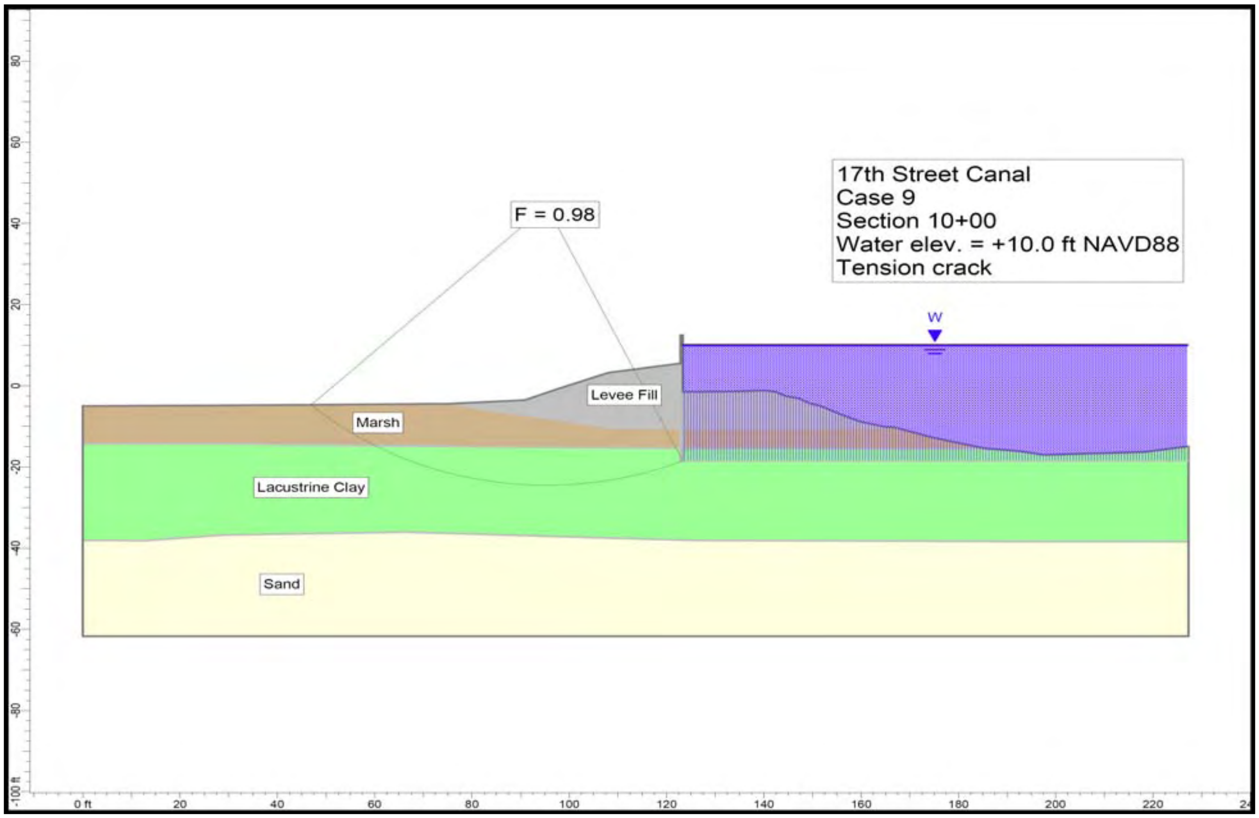

Figure 27. Critical Circle for 17th Street Canal Station 10+00 – Water Elevation 10 ft, With a Water-Filled Gap Behind the Wall

A more complete description of the finite element soil-structure interaction is contained in the Appendix, "17th Street SSI Station 10+00."

Summary of 17th Street Canal Breach Assessment

Eye-witness reports indicate that the breach began to develop about 6:00 AM on Monday, 29 August 2005, and was fully developed before 9:00 AM. Field evidence, analyses, and physical model tests show that the breach was due to instability caused by shear failure within the clay in the foundation beneath the levee and the I-wall, with a rupture surface that extended laterally beneath the levee, and exited upward through the marsh layer. A key factor in the failure was the 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 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 and lateral variation of shear strength beneath the levee must be considered for other locations throughout the system when geologic conditions are similar to those at the 17th Street Canal.

Assessment of London Avenue Canal Breaches

Two I-wall failures resulting in breaches occurred at the London Avenue Canal during Hurricane Katrina, one on the east side of the canal at Mirabeau Avenue (the south breach), and the other on the west side of the canal at Robert E. Lee Boulevard (the north breach). At both locations, the levees and I-walls were founded on a layer of marsh material overlying sand. In addition, the I-wall on the east side, across the canal from the north breach, moved and tilted, but did not breach.

The south breach, shown in Figure 28, occurred about 6:00 AM to 7:00 AM on August 29th, when the water level in the canal was 7.1 ft to 8.2 ft NAVD88. The breach was narrower than the breach at the 17th Street and the London Avenue north breach. A deep scour hole formed due to the inrush of water, and a large amount of eroded sand was deposited in the neighborhood inland of the breach. It appears that the breach was quite narrow when it formed, and subsequently widened to about 60 ft as wall panels adjacent to the initial breach were undermined by scour, and tilted into the scour hole.

Figure 28. South Breach, London Avenue Canal

The north breach, shown in Figure 29, occurred about 7:00 AM to 8:00 AM on August 29th, about an hour after the south breach, when the canal water level was 8.2 ft to 9.5 ft NAVD88. The breach was about 410 ft wide, approximately the same width as the breach at 17th Street. As the breach occurred, the ground surface on the protected side of the levee heaved upward, taking with it the playhouse shown in Figure 30. The upward movement of the playhouse can be discerned by comparing the before and after photos in Figure 30. The I-wall opposite the north breach (on the east side of the canal) moved and tilted significantly, presumably at about the same time as the breach occurred on the west side, but the east I-wall did not breach; this tilted but intact I-wall is shown in Figure 31. A line of sinkholes was observed at the inland side of the distressed east I-wall, and a sand boil at the inboard embankment toe indicates that erosive seepage and piping had occurred beneath the levee.

Figure 29. North Breach, London Avenue Canal

Figure 30. Playhouse at the London Avenue Canal North Breach Before the Failure (top photo) and After (bottom photo)

Figure 31. Tilted I-Wall Opposite the North Breach on the London Avenue Canal

At both the south and north breach locations, it seems likely that underseepage and internal erosion caused or contributed to the failures.

It is not possible to establish the cause of the south breach with certainty based on the field observations made after the failure. The failed sections of the I-wall have not yet been found, and large volumes of sand were moved by the inflow of water through the breach, covering the landscape. The failure might have resulted from underseepage erosion and piping, or from sliding instability aggravated by under-seepage and uplift pressures. Analyses have been performed to examine both of these possibilities.

As shown in Figure 29, a long section of the floodwall at the north breach was displaced laterally, and it seems likely that sliding instability was likely the primary mode of failure, with seepage and high pore pressures in the sand as a significant contributing factor. It seems likely that the failure and breach were the result of insufficient passive resistance to counteract the water pressure forces to which the wall was subjected. The passive resistance was likely reduced by the effects of water seeping through the foundation soils beneath the levee and the marsh layer inland, inducing uplift pressures and reducing shear strengths. Analyses have been performed to examine the likelihood of erosion and piping, and instability due to uplift and reduced shear resistance.

Seepage and Stability Analysis of the London Avenue Canal Breaches

Seepage and stability analyses were performed to investigate if the erosion and piping and/or sliding instability caused the foundation failure at these breach locations. Finite element analyses of seepage beneath the I-wall were performed to determine the pore pressures in the sand beneath the marsh layer. The characteristics of the cross sections for the south and north breach are shown in Figure 32 and Figure 33. The relevant materials are the sand at the base of the section, the overlying marsh layer, and the clayey levee fill. Thorough analyses of transient and steady seepage indicated that (a) steady seepage through the sand was established quickly, and (b) the pore pressures within the sand and the uplift pressures on the base of the marsh layer are not affected by the permeability values assigned to the marsh layer and the levee fill, provided that those materials are at least two orders of magnitude less permeable than the sand. The values of permeability of the marsh layer and the levee fill used in the seepage analyses were selected in accordance with these findings, and were considered to be reasonable estimates of the permeabilities of these materials. A more complete description of the seepage analysis is contained in the Appendix, "Analysis of London Avenue Canal I-wall Breaches."

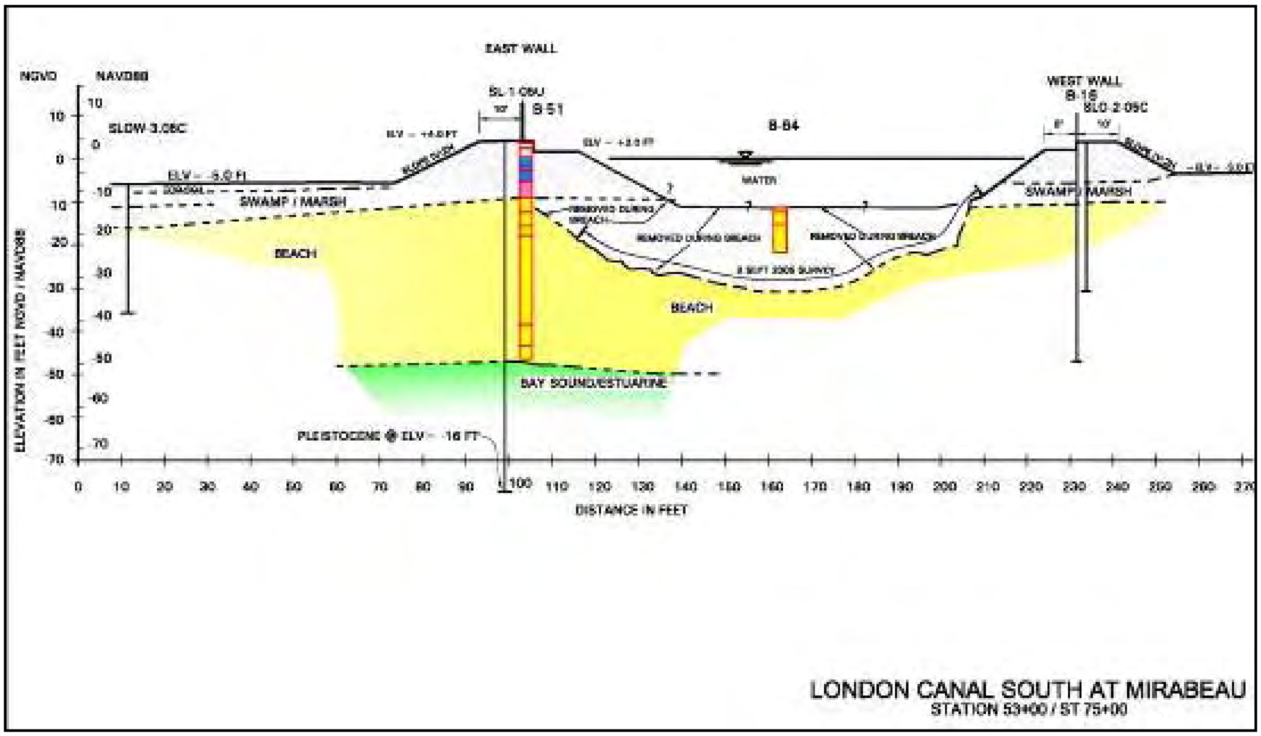

Figure 32. London Avenue Canal South Breach Cross Section

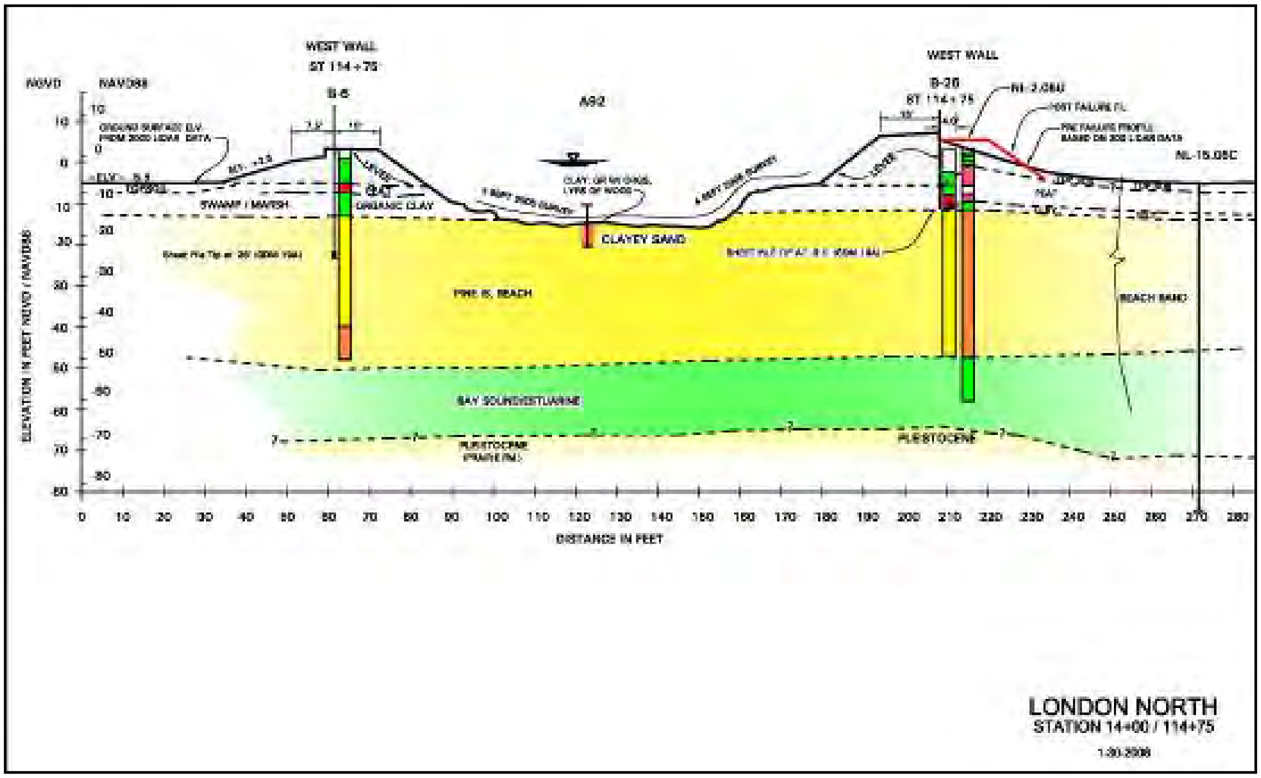

Figure 33. London Avenue Canal North Breach Cross Section

In conditions like those at London Avenue, with the I-walls penetrating down to the cohesionless sand layer in the foundation, a crack or gap extending to the tip of the sheetpile is not possible, because the sand is incapable of sustaining a crack. These cases were analyzed using what was termed a "half-cracked" condition, with the crack extending only to the top of the sand. Below the top of the sand the sheetpile is loaded by water pressures that are lower than hydrostatic, and active earth pressures.

The analyses described in the Appendix, "Analysis of London Avenue Canal I-wall Breaches," indicate a strong likelihood that high uplift pressure on the base of the levee and the marsh layer was a key factor in the failures at both the south and the north breaches. At both locations, these high uplift pressures probably resulted in the development of a rupture through the marsh layer, and hydraulic gradients large enough to cause erosion of the sand upward through the rupture.

At the south breach area, this erosion may have been the principal mode of failure, with gross instability occurring after considerable volumes of sand, marsh, and levee fill had been removed by erosion and piping. Without alteration of the south breach cross section by erosion and piping, the calculated factors of safety with respect to instability are greater than 1.0, indicating that alteration of the profile by erosion and piping probably played an essential role in the failure at this location where the sand was dense, and the sand friction angle would have been high. The conclusion that the failure probably started in a small zone of intense seepage is consistent with the narrow breach that eventually developed.

At the north breach area, the probability of erosion and piping is slightly less than at the south breach, although still very high. The probability of instability at the north breach is higher than at the south breach, due to the fact that the sand at the north breach was loose, and would have had a low friction angle. High uplift pressures likely resulted in a rupture through the marsh layer and the underlying thin layer of lacustrine clay. At this location, however, the high pore pressures within the sand would have been sufficient to cause instability without significant alteration of the cross section by erosion. The failure at the north affected a much wider zone than the failure at the south, indicating that intense localized erosion and piping probably did not play a key role in the failure at the north breach. It appears that high uplift pressures and lower friction angle of the less-dense sand were key elements in the failure at the north breach.

Centrifuge Modeling Results for the London Avenue Canal Breaches

In the London Avenue Canal model tests, where the toe of the sheet pile wall penetrated into the sand and was restrained from lateral movement by the sand, the opening of the gap on the canal side of the floodwall was followed by rotation of the top of the wall landward. With the opening of the gap, the model tests for the London south breach and the London north breach both showed gross movements indicative of wall failure, as may be seen in Figure 34. Rotation of the wall was accompanied by a translational sliding of the landward part of the levee and marsh layer, on top of the underlying sand. Rotation of the wall is linked to reduction in effective stress on the sand layer beneath the landward side of the levee. Once the gap was fully opened, pore pressure transducers in the foundation sand layer showed that the pore pressure in the sand rose very significantly. The increasing water pressure in the sand layer reduced the effective stress at the base of the marsh layer. The reduction in vertical effective stress has two effects: the first is to increase the likelihood of uplift of the swampy marsh (as the increasing water pressure in the foundation balances or exceeds the weight of the levee and marsh layer above) and the second is to reduce the stiffness of the sand surrounding the toe of the sheet pile wall, reducing the passive resistance that the sand provides.

Figure 34 Superposition of Video Images of the Rotational Failure of the London South Model Wall, Model 1

The high pore water pressures under the toe of the levee on the landward side and under the swampy marsh led to water being ejected from the ground at the toe of the levee. This may be deduced from the piezometric heads near the toe of the levee, and can be seen clearly in the video images of the landward toe. Figure 35 shows a view of the levee and floodwall as seen from the ‘backyard’ of houses behind the levee, showing ‘black’ water emerging from the toe of the levee as the flood wall rotates landward. This supports the earlier assessment based on seepage analyses that a rupture through the marsh layer overlying the sand at the south breach led to a failure initiated by erosion and piping.

Figure 35. Water Emerging From Toe of Levee as Failure Progresses, London South Model 2

A more complete description of the centrifuge modeling effort is contained in the Appendix, "IPET Centrifuge Model Test Report."

Finite Element Soil-Structure Interaction Results for London Avenue Canal Breaches

Finite element soil-structure interaction analyses were conducted to provide a third approach to development of a complete understanding of the London Avenue Canal breach mechanisms. A two-dimensional cross section through the north breach is shown in Figures 36 and 37. The older sheetpile wall shown in the figures, which was left in place when the newer one was constructed, does not influence the results of the analysis in any significant way. A detailed description of the nonlinear finite-element soil-structure interaction analyses is contained in Appendix "London Avenue SSI.".

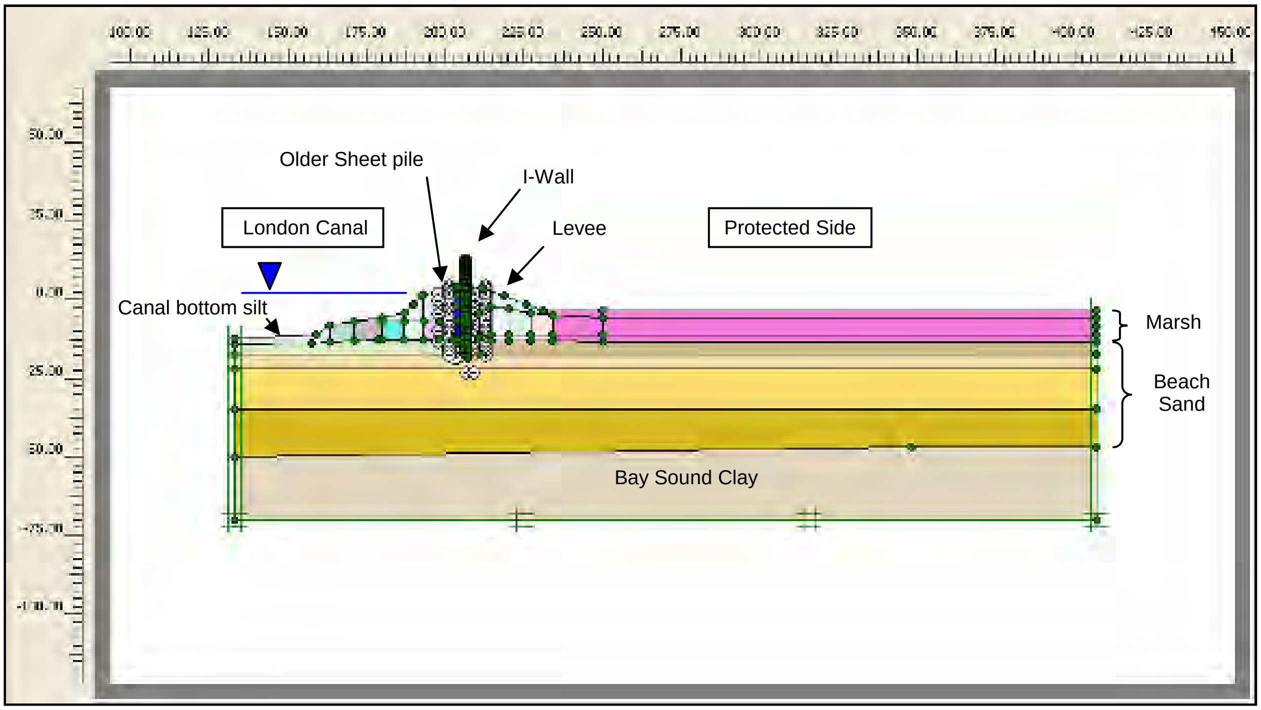

Figure 36. Two-Dimensional Cross Section Model Used in the SSI Analysis of the London Avenue Canal North Breach

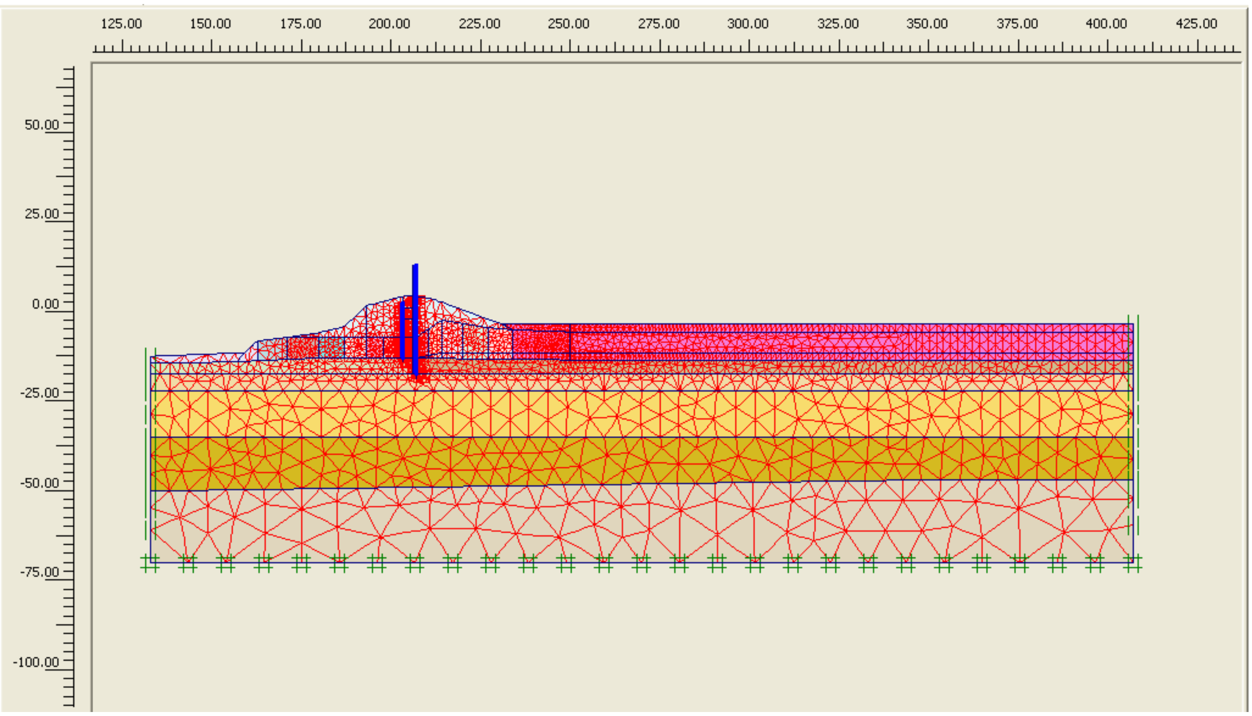

Figure 37. Finite Element Mesh Used in the SSI Analysis of the London Avenue Canal North Breach

Like the limit equilibrium stability analysis and centrifuge tests, the finite element analyses showed that the formation of a gap down to the sand layer leads quickly to failure. Some of the details regarding formation of the gap were found to be dependent on the stiffness assigned to the levee fill and the marsh layer. The lower and upper limits of the possible stiffness values for the levee fill and the marsh layer were used to investigate the effect of stiffness on the formation of the gap. Figure 38 shows how the gap increased in depth as the canal water elevation increased. The gap began to form once the canal water elevation reached the crest of the levee, and progressed down the wall as the canal water elevation continued to rise. The rate of progression of the gap down the wall was found to depend on the soil stiffness, with the stiffer soil allowing the canal water elevation to reach a higher level before the gap extends to the top of the sand layer.

Once the gap extended full depth, the water pressure in the sand rose rapidly, and the elevated water pressure in the sand layer reduced the vertical effective stress at the base of the marsh layer, Figure 39. The reduction in vertical effective stress has two effects: the first is to increase the uplift pressure on the bottom of the marsh layer, as shown in Figure 39; and the second is to reduce the stiffness of the sand surrounding the tip of the sheet pile wall, reducing the passive resistance provided by the sand.

The finite element soil-structure interaction analyses are consistent with the behavior observed in the centrifuge tests, where the opening of the gap on the canal side of the floodwall was followed by rotation of the wall landward, as shown in Figure 40. Figure 41 shows the horizontal displacement of the top and tip of the wall as the canal water rises. It shows that the tip did not translate horizontally; instead, the wall rotated about the tip. Figure 41 also shows the significant increase in rotation of the wall once the crack reached the sand layer.

The factor of safety decreased substantially when the crack opened fully, as may be seen in Figures 41 and 42. When the crack (or gap) opened, the factor of safety decreased with no change in water level, as shown by the horizontal line segments in Figures 41 and 42. For the low-stiffness soil, the gap opened fully at a canal water elevation of 6.0 ft, and the factor of safety decreased from 2.5 to 1.8. For the stiff soil, the crack opened fully at a canal water elevation of 8.0 ft, and the factor of safety decreased immediately from 2.1 to below 1.0. If the canal bottom was covered with silt, and the sand layer was therefore not connected to the canal, the opening of the gap would be the means for connecting the sand layer to the canal.

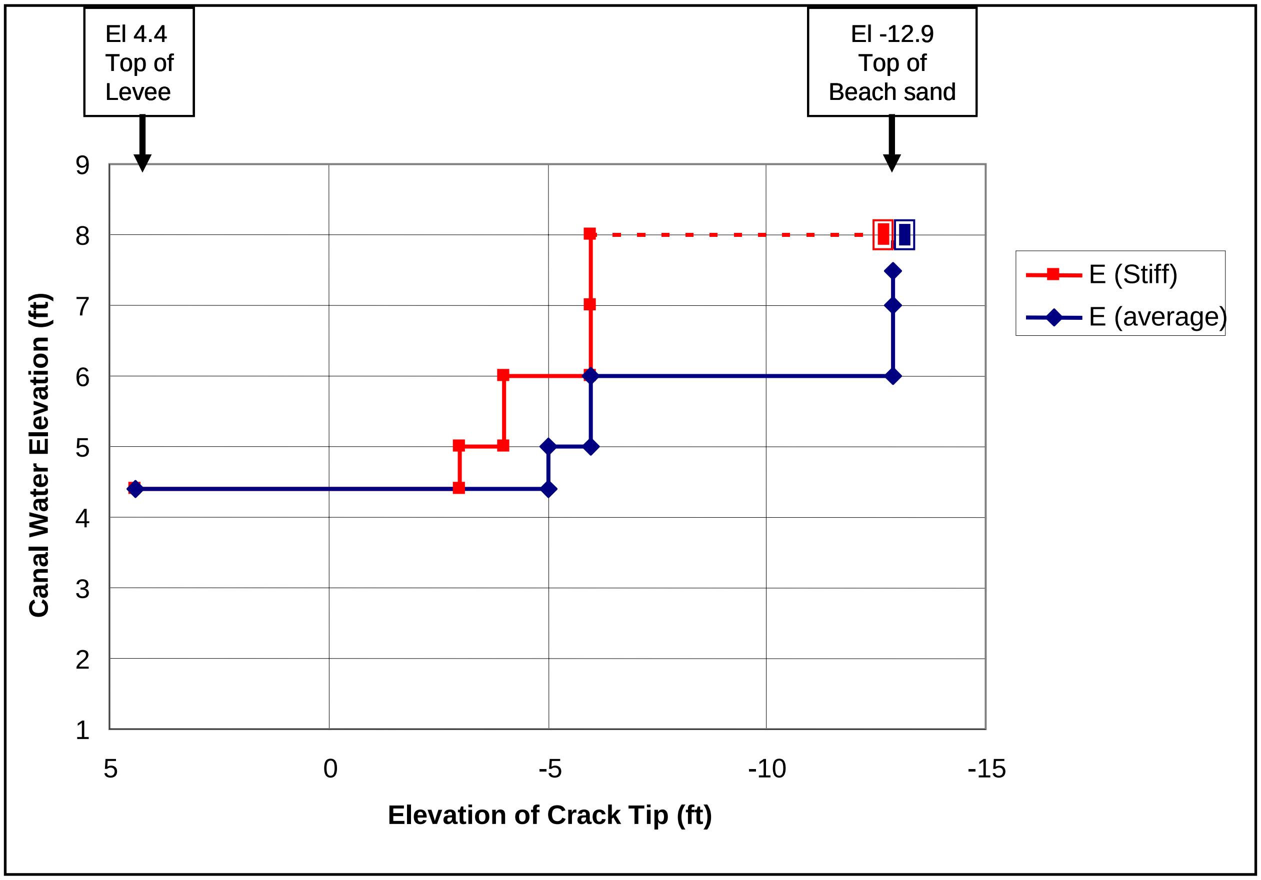

Figure 38. Elevation of Gap Tip Versus Canal Water Elevation

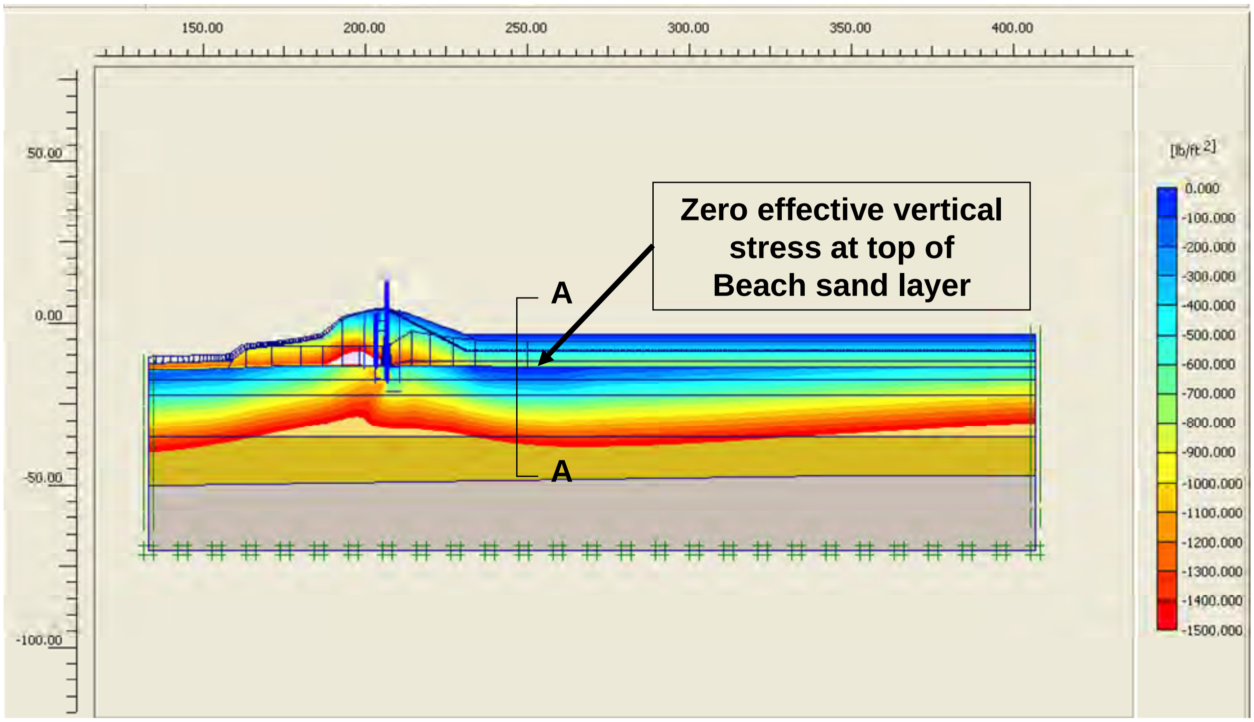

Figure 39. Effective Vertical Stress in the Beach Sand for the Analysis Using Average Stiffness Values with Canal Elevation 6.0 ft and Gap to Elevation -12.9 for the London Avenue North Breach

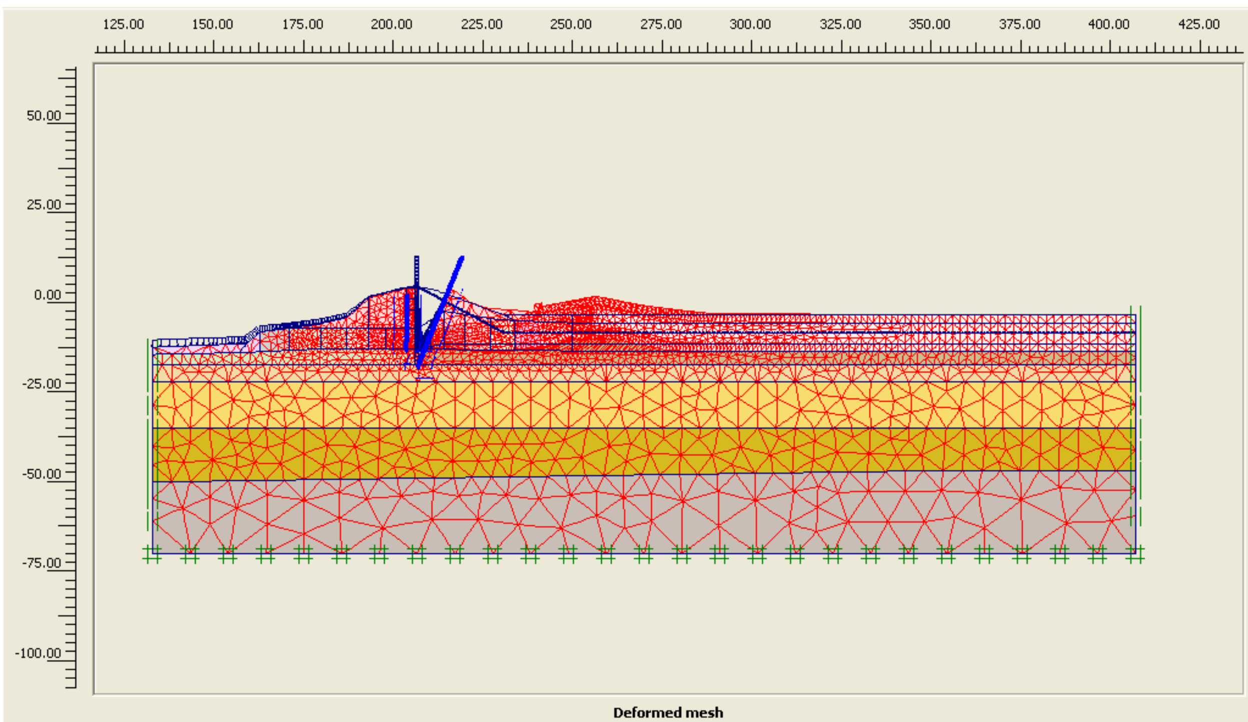

Figure 40. Ultimate Mechanism from Strength Reduction Analysis Using Average Stiffness Values with Canal Elevation 6.0 ft and Gap to Elevation -12.9 ft, London Avenue North Breach (displacements are exaggerated)

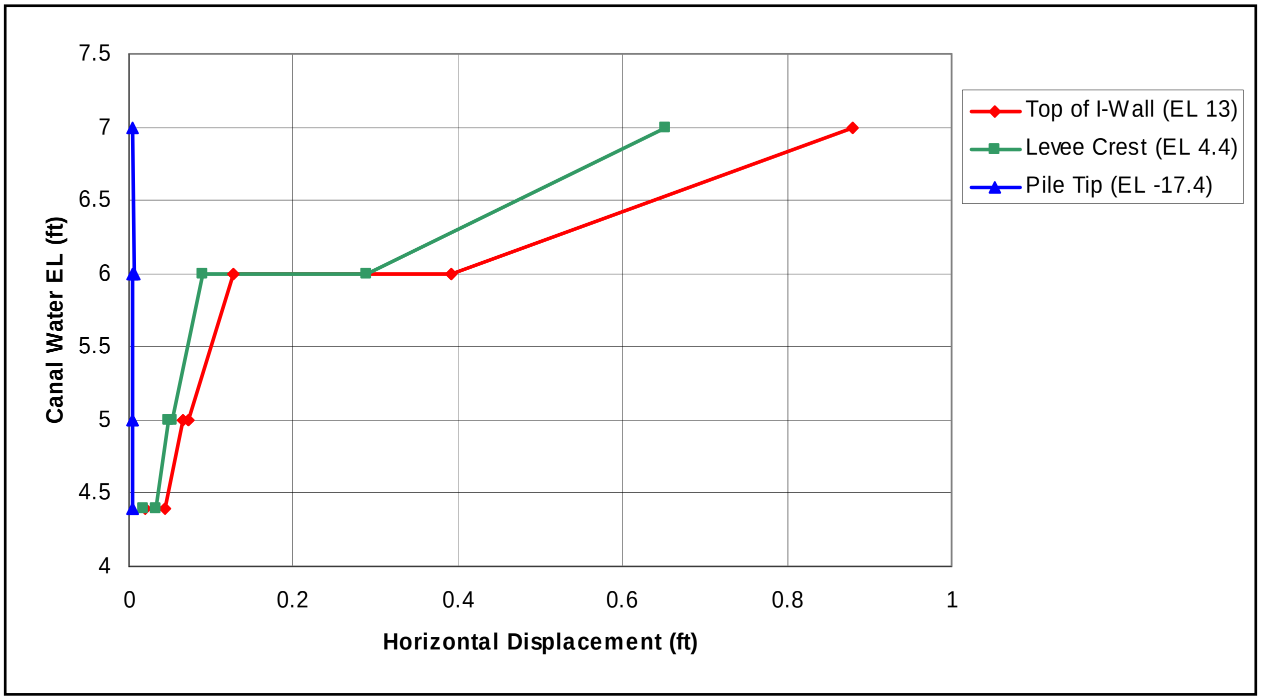

Figure 41. Horizontal Sheet Pile Deformations versus Canal Water Elevation Computed Using Average Stiffness Values for the London Avenue Canal North Breach

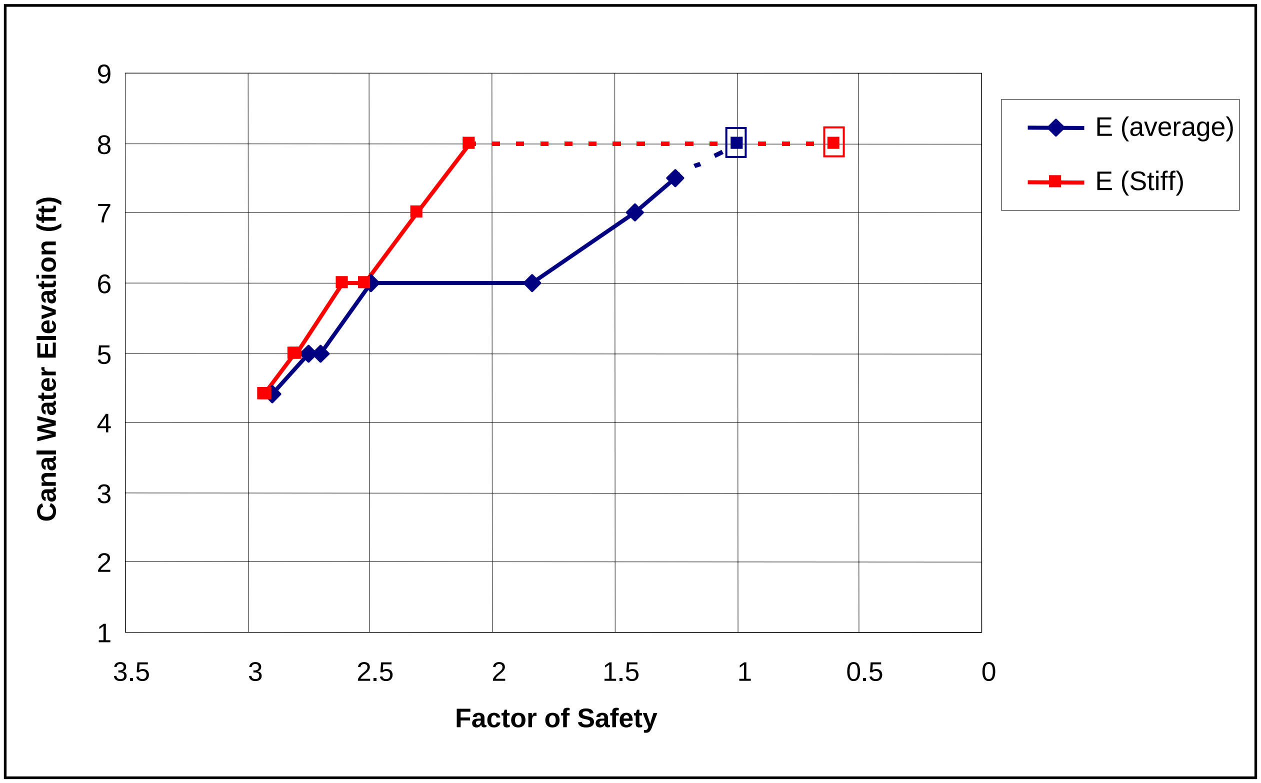

Figure 42. Factor of Safety Versus Canal Water Elevation Calculated Using the Strength Reduction Method for the London Avenue Canal North Breach

Summary of Assessment of the London Avenue Canal Breaches

The breach on the London Avenue Canal near Mirabeau Avenue (the south breach) occurred at 6:00 AM to 7:00 AM on Monday, 29 August 2005. The breach on the London Avenue Canal near Robert E. Lee Boulevard (the north breach) occurred by 7:30 AM. Field evidence, analyses, and physical model tests show that the breaches were due to the effects of high water pressures within the sand layer beneath the levee and I-wall, and high water loads on the walls. The London Avenue Canal breaches had a key factor in common with the 17th Street Canal breach – formation of a gap between the wall and the levee fill on the canal side of the wall. At both the 17th Street Canal and the London Avenue Canal, formation of a gap allowed high water pressures to act on the wall below the surface of the levee, severely loading the wall. At the London Avenue Canal, an additional effect of the gap was that water flowed down through the gap into the underlying sand. High water pressures in the sand uplifted the marsh layer on the landside of the levee, resulting in concentrated flow and erosion, removing material and reducing support for the floodwall.

Analyses of the south breach showed that erosion is most likely the principal mode of failure, with sliding instability occurring after significant volumes of sand and marsh had been removed by erosion and piping. Without alteration of the south breach cross section by erosion and piping on the landside of the levee, the calculated factors of safety with respect to sliding instability are greater than 1.0, indicating that alteration of the cross section by erosion and piping probably played an essential role in the failure at this location.

Field observations at the north breach indicate that the canal-side levee crest remained intact after the breach, and a playhouse on the property adjacent to the breach was heaved upward as the ground beneath it heaved upward during the failure. The analyses described in this report show that conditions for erosion and piping were present at the north breach, but the more likely cause of the failure was sliding instability. High uplift pressures likely resulted in a rupture through the marsh layer and the underlying thin layer of clay. At this location, however, the high pore pressures within the sand would reduce passive resistance sufficiently to result in sliding instability without significant alteration of the cross section.

It seems reasonable to assume that the wall on the opposite side of the canal from the north breach, which moved and tilted, must have been close to failure, but this location has not been analyzed in detail.

Assessment of Inner Harbor Navigation Canal Breaches

Four breaches occurred on the IHNC (Inner Harbor Navigation Canal) during Hurricane Katrina on the morning of August 29th. Two of the breaches occurred on the east bank between the Florida Avenue Bridge and the North Claiborne Avenue Bridge adjacent to the 9th Ward, and two on the west bank, just north of the intersection of France Road and Florida Avenue. The locations of theses breaches are shown in Figure 43. Three of the breaches involved failures of floodwalls on levees, and one involved failure of a levee due to overtopping erosion.

All of the IHNC floodwalls and levees were overtopped on August 29th. The peak storm surge elevation in the IHNC was 14.2 ft at 9:00 AM, as can be seen in Figure 44. This peak water level is about 1.7 ft above the tops of the floodwalls and levees. The reaches where the floodwalls and levees did not collapse have, therefore, survived water loading considerably higher than the design loading.

Initial observations after the hurricane revealed that overtopping had eroded at least one section of levee (without floodwall) along the west bank, and had eroded the soil adjacent to the wall at three other locations along the east and west bank. It appeared that water flowing over the floodwall scoured and eroded the levee on the protected side of the I-wall, exposing the supporting sheet piles and reducing the passive resistance, as can be seen in Figure 45. The erosion appeared to be so severe at the breach locations that the sheet piles may have lost all foundation support, resulting in failures of the type shown in Figure 46. Perhaps the best evidence of this scour can be seen along the un-breached reaches of the east bank I-walls, where U-shaped scour trenches were be found adjacent to the I-walls. As the scour increased, the I-wall may have moved laterally and leaned toward the protected side, causing the scour trench to grow as the water cascaded farther down the slope until sufficient soil resistance was lost and complete failure occurred.

Although it is clear that the walls were overtopped, and that their stability was compromised by the erosion that occurred, it is also clear that one of the east side breaches occurred before the wall was overtopped. Eyewitness reports indicate that the water level in the 9th Ward near Florida Avenue was rising as early as 5:00 AM, when the water level in the IHNC was still below the top of the floodwall, as shown by the hydrograph in Figure 47. Stability analyses indicate that foundation instability would occur before overtopping at the north breach on the east side of the IHNC. This breach location is thus the likely source of the early flooding in the Lower 9th Ward. Stability analyses indicate that the other three breach locations would not have failed before they were overtopped.

The soil immediately beneath the levees and floodwalls at all four breach locations included marsh, beneath which was clay, and beneath the clay, sand. Through most of their lengths, the critical circles passed through the marsh and clay. The critical circles did not extend to the sand layer beneath the clay.

Formation of a gap on the canal side of the wall, allowing hydrostatic water pressure acting through the full depth of the gap, causes a very significant reduction in the value of the calculated factor of safety. Evidence that a gap did form behind the wall near the breaches can be seen in Figures 48 and 49.

Stability analyses of the north breach on the east side resulted in a computed factor of safety equal to 1.0, with a gap on the canal side of the wall and water in the IHNC at elevation 11.2 ft. This is about 1.0 ft higher than the average IHNC water level at the time flood water was observed in the Lower 9th Ward. Considering that the effective water level could have been one foot higher due to wave effects, this result is consistent with the observed IHNC water level when flood water was first reported in the Lower 9th Ward. It thus appears that the north breach occurred before overtopping, and that this breach was the source of the first influx of water into the 9th Ward.

Figure 43. Four Breach Locations on the Inner Harbor Navigation Canal

Stability analyses of the south breach on the east side, and the north breach on the west side, resulted in computed factors of safety larger than 1.0 with the water level at the top of the wall and a crack behind the wall, indicating that the walls at those locations would have remained stable if none of the soil supporting the wall had been removed by erosion. Stability analysis of the south breach on the west side, where there was no I-wall, showed that the factor of safety there was also high, and the breach was due to overtopping erosion.

The lower computed factor of safety at the north breach on the east side is attributable to the fact that the ground elevation on the protected side is lower at that location and there was less soil on the protected side of the wall that was able to provide support for the wall.

The IPET strength model used for the north breach on the east bank, which is based on all of the data available in May 2006, agrees fairly closely with the design strengths reported in the General Design Memorandum (GDM) No. 3 under the center of the levee. Both the GDM and the IPET strength model assign lower strengths beneath the embankment toe and beyond than beneath the crest of the embankment, but the GDM strengths at this location are higher than the IPET strengths. The GDM strengths are, thus, reasonably consistent with the currently available data.

The design analyses were performed using the Method of Planes, without a crack between the wall and the levee fill on the canal side of the wall. For the canal water level at 10.5 ft (the design water level), the factor of safety computed using the Method of Planes was 1.25. The minimum factor of safety calculated for the same conditions using Spencer's method was 1.45, indicating that the Method of Planes is conservative by about 14% in this case.

In summary, the foundation failure at the north breach on the east side of the IHNC was a result of differences between the actual conditions and assumptions used as the basis for the design. Those differences are (1) the ground surface beyond the toe of the levee at the north breach location was lower than the landside ground surface in the design cross section, and (2) the design analyses did not consider the possibility of a gap forming behind the wall, allowing water to run into the gap and increase the load on the wall. The other three breaches on the IHNC were due to overtopping and erosion.

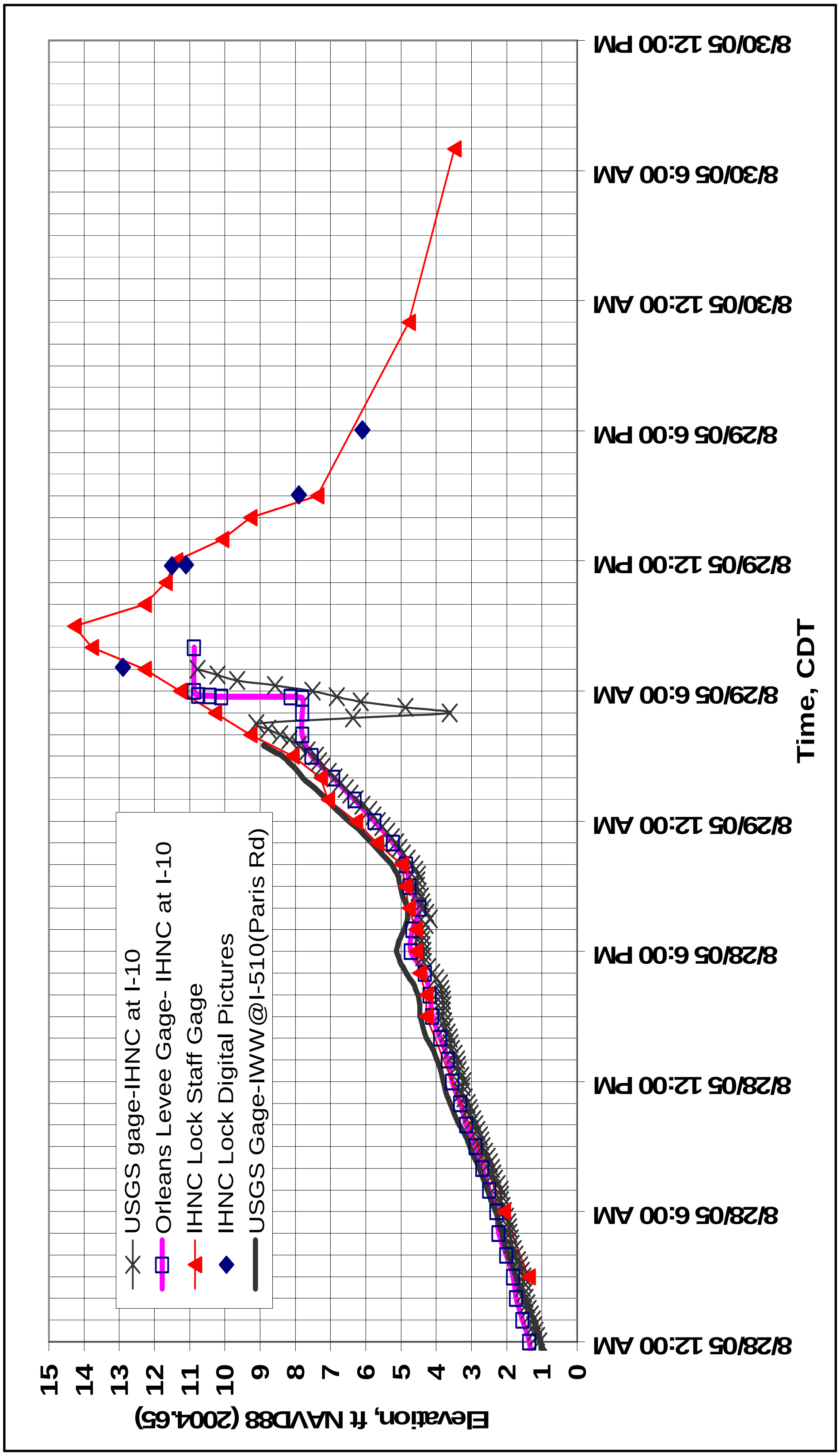

Figure 44. Katrina Hydrograph for the Inner Harbor Navigation Canal

Figure 45. Scour and Erosion on the Protected Side of the IHNC Adjacent to the 9th Ward in the Vicinity of the South Breach

Figure 46. Scour and Erosion Leading to the Failure of the I-Wall on the IHNC Adjacent the South Breach (9th Ward)

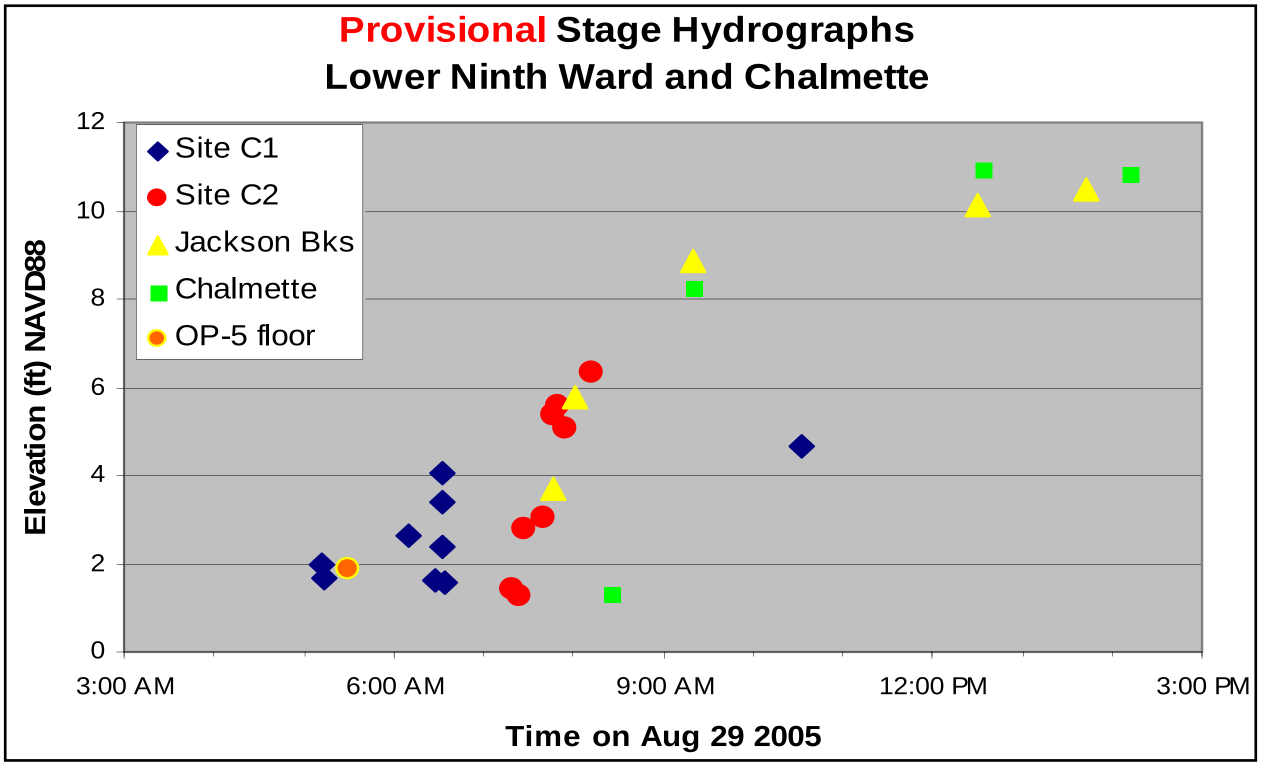

Figure 47. Hydrograph for the 9th Ward Inundation

Figure 48. IHNC East Bank – South Breach – Wall Movement

Figure 49. IHNC East Bank – North Breach – Wall Movement

Assessment of Orleans Canal and Michoud Canal I-walls

The I-walls at the Orleans Canal and Michoud Canal did not fail, even though they were severely loaded during Hurricane Katrina. The purpose of studying them was to be able to make detailed comparisons between their successful performance and that of the I-walls at 17th Street Canal, London Avenue Canal, and Inner Harbor Navigation Canal (IHNC), which breached. It is important to determine if the analysis and physical modeling methods that indicated unstable conditions for the 17th Street Canal, London Avenue Canal, and IHNC I-walls, which did fail, would indicate stable conditions for the Orleans Canal and Michoud Canal I-walls, which did not fail. These assessments of stable I-walls provide insight into the performance of other I-walls in the hurricane protection system.

Analysis of the Performance of the Orleans Canal I-walls

At the Orleans Canal south area (Station 8+61), the marsh layer beneath the levee is underlain by sand, as shown in Figure 50. At the Orleans Canal north area (Station 64+27), the marsh layer beneath the levee is underlain by clay, as shown in Figure 51. The geologic conditions at these two locations on the Orleans canal are thus directly comparable to the locations at the 17th Street Canal and the London Avenue Canal where breaches occurred.

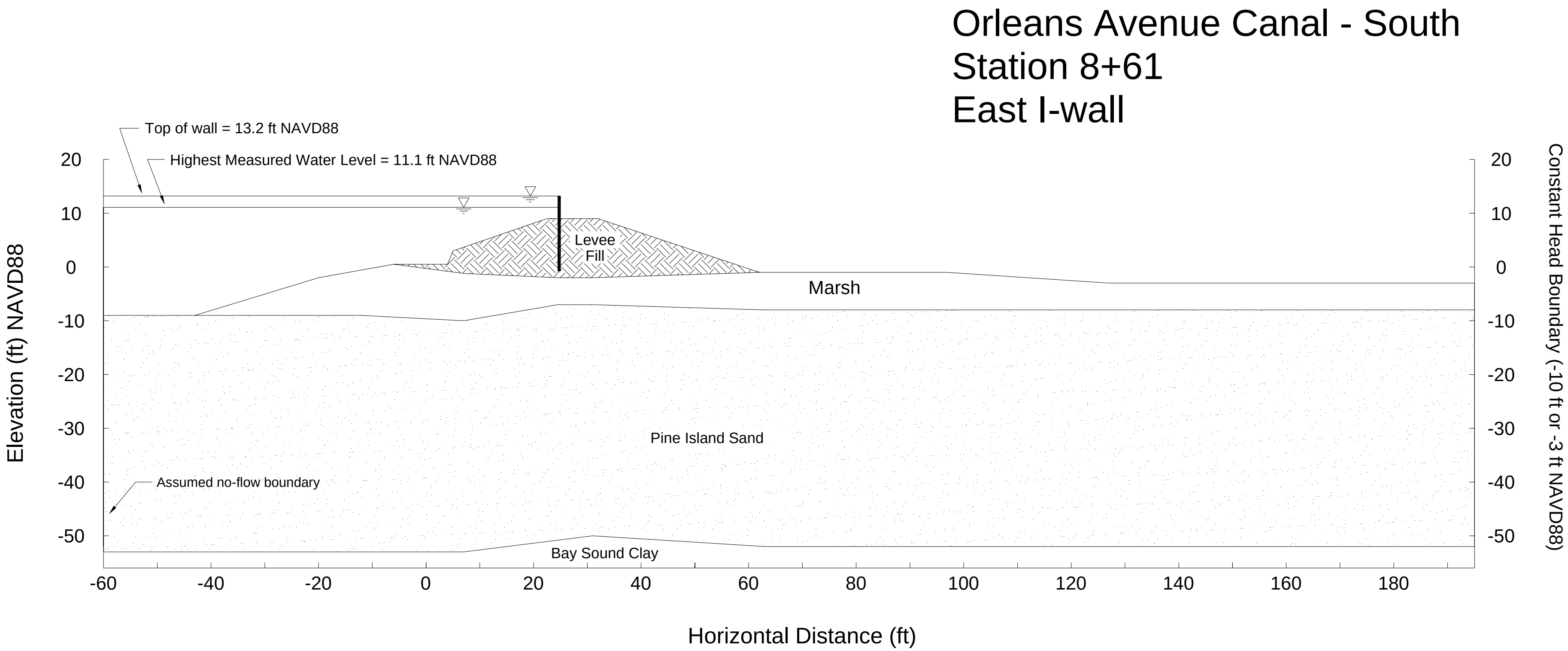

Figure 50. Schematic Cross Section at Orleans South, With Seepage Boundary Conditions

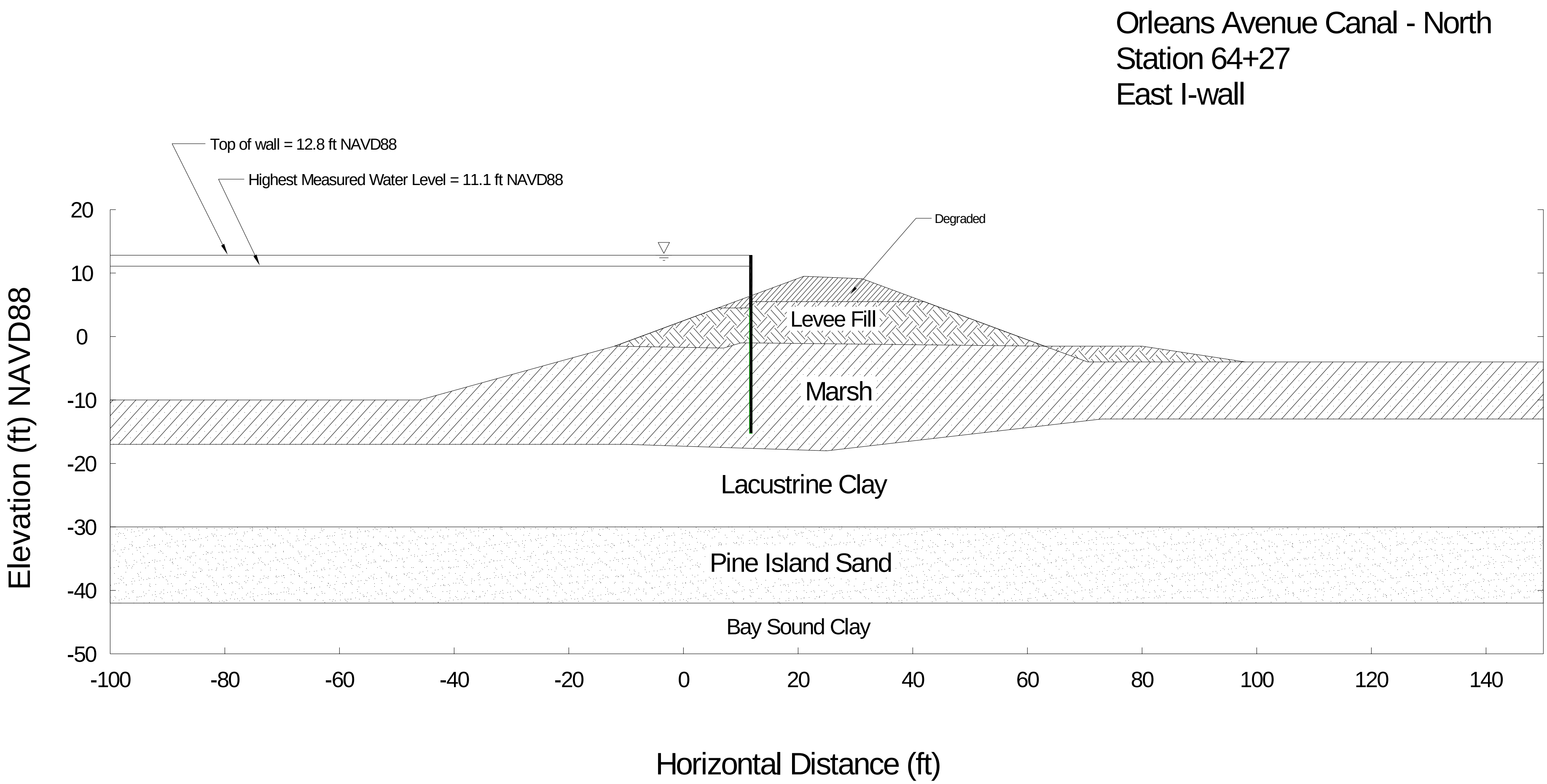

Figure 51. Schematic Cross Section of Orleans North

For purposes of evaluating stability of the walls, it was assumed that a gap would form as it had at the other I-wall locations. There are two possible conditions regarding formation of a gap behind the wall. The first involves a gap extending to the bottom of the wall, as considered at the 17th Street Canal. The second involves continuation of this gap below the bottom of the wall, by hydraulic fracturing of the levee fill and marsh material below the wall. Hydraulic fracturing is possible in any location where (1) water pressures exceed the total stress on a potential fracture plane, as would be the case for a vertical plane extending below the bottom of the wall, and (2) the soil has sufficient strength for the gap to remain open, supported by the water pressure. For the conditions at Orleans south, the levee fill and marsh are strong enough to maintain a water-filled gap extending down to the marsh-sand interface. Water would fill this gap, loading the wall and the fracture plane below the wall, and introducing the canal head at the top of the sand.

Formation of a hydraulic fracture below the bottom of the wall would result in very severe loading on the wall for two reasons: (1) the gap would allow water to flow directly into the sand layer, increasing pore pressures and uplift pressures, and (2) the gap beneath the wall would extend the vertical face on which water pressures would act, thereby greatly increasing the water load acting on the plane of the wall. Although formation of a deeper gap by hydraulic fracturing has not been confirmed by field observation, it does appear to be feasible in soils as strong as the levee fill and marsh at Orleans south. Although occurrence of hydraulic fracturing is a scenario not often encountered, it is believed to be a condition that should be evaluated in this study of the performance of the Orleans south I-wall.

The same type of seepage analysis and interpretation of results that showed high hydraulic gradients, factors of safety less than 1.0, and probability of erosion greater than 99% at the London Avenue south breach, showed moderate hydraulic gradients, factors of safety larger than 1.0, and probabilities of erosion of 3% and 28% for the highest water level experienced on the Orleans Canal, with an assumed gap extending below the tip of the wall to the top of the sand layer. The fact that no signs of erosion due to underseepage were observed in the Orleans south area after the flood neither confirms nor refutes these calculated probabilities. The analyses indicate a possibility of erosion, but this would not necessarily result in failure of the I-wall. Erosion, if it did occur beneath the marsh layer, might not result in visible manifestations at the ground surface, and might not move a sufficient quantity of sand to alter the cross section significantly.

The same type of stability analyses and interpretation of results that showed factors of safety less than 1.0, and probabilities of instability varying from 70% to 97% at the London Avenue north breach, showed factors of safety varying from 1.9 to 2.7 for the highest water level observed at the Orleans Canal, and probabilities of instability lower than one in one million.

The same type of stability analyses and interpretation of results that showed factors of safety from 1.0 to 1.2 for a range of water levels, and probabilities of instability from 12% to 60% for the 17th Street Canal breach, showed factors of safety from 1.5 to 1.6 for the highest water level observed at the Orleans Canal, and probabilities of instability from 1% to 3%.

These results show that the methods of stability analysis applied to conditions with clay beneath the levee and marsh layer are capable of modeling instability where it occurs, and stable conditions where they occur.

A more complete description of the stability analyses and results is contained in the Appendix, “Analysis of the Performance of Orleans Canal I-Walls.”

Centrifuge Modeling Results for the Orleans Canal I-wall

Similar to the cases for slope stability and seepage analyses, scale models of the Orleans Canal sections were tested in the geotechnical centrifuge to confirm that a failure would not occur for these sections. The cross section chosen to represent the southern portion of the Orleans Avenue levees is similar to the cross section at London Avenue, except that the levee is wider and higher, and the penetration of the floodwall is less (the toe of the wall is at the base of the levee/top of the swampy marsh layer).

As with the other levee sections, the instruments beneath the model levee responded to the rising water in the canal by increased water pressures in the sand, reducing nearly linearly from a maximum near the canal to a minimum to the landward side of the levee, as shown in Figure 52. At the water level rose in Orleans Avenue canal, the floodwall did not show any movement to indicate the opening of a gap. A cross section through the centrifuge model at the maximum water level reached during Hurricane Katrina is shown in Figure 53.

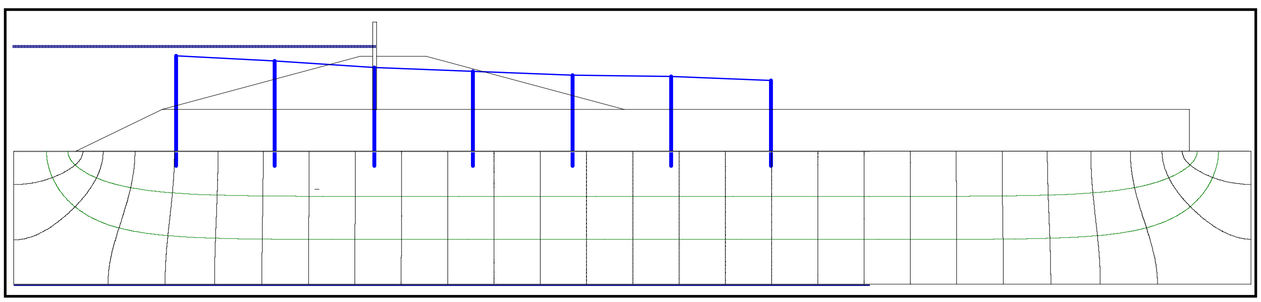

Figure 52. Seepage Flow Net for Orleans Avenue South with Piezometric Levels Under the Swampy Marsh at Katrina Flood Level

Figure 53. Orleans South Model at Katrina Flood Level

The piezometric levels below the swampy marsh in the Orleans South model showed a declining level from the canal side to the protected side of the levee, as may be seen in Figure 54. The additional height and width of the Orleans South levee, seen superimposed in the figure for both Orleans South and London South, significantly increase its capacity to resist the flood levels, and no evidence was seen of a gap opening through the swampy marsh to provide a hydraulic connection to the sand below. The London Avenue Canal south breach piezometric levels and cross section, shown in Figure 54 for comparison, show a more critical condition in terms of uplift under the landward side of the levee. Both models, however, show high piezometric levels near the toe of the levee on the landward side, sufficient to lead to uplift of the marsh layer.

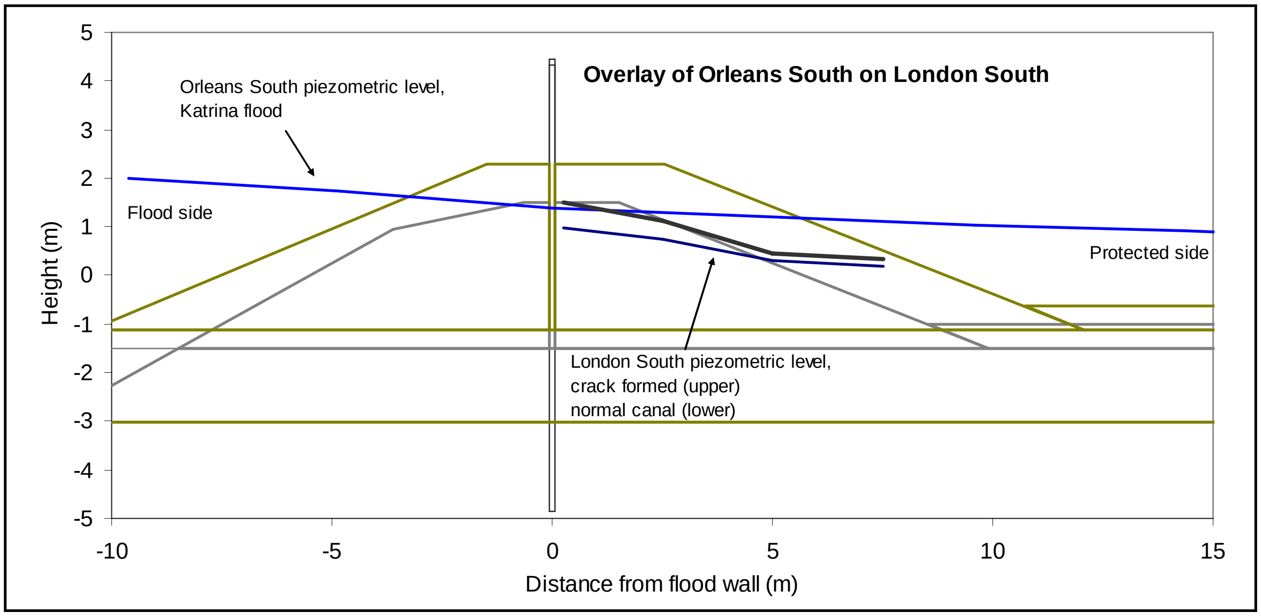

Figure 54. Comparison of Piezometric Levels in Orleans South (Katrina Flood) and London South, with the Base of the Marsh Layer Assumed to be at a Common Elevation

As the water level was raised above the Katrina flood level in the Orleans south model, a gap was seen to open to the toe of the floodwall, associated with small movements of the flood wall landward, as shown in Figure 55. Unlike the other models, this gap did not develop into a full failure condition; and no unstable movement of the floodwall was observed, despite the water level reaching the top of the floodwall. Translational movements were observed to be controlled by compression of the swampy marsh layer. Relative movement between the top of the sand and the swampy marsh was apparent, in a similar manner, to the uplift mechanism in the London Avenue models.

Figure 55. Small Sliding Movement of Marsh as Gap Forms, Orleans South Model

Comparison between the geometries of Orleans and London Avenue shows that the vertical effective stress in the sand foundation below the levee was considerably higher under the Orleans levee than under the London Avenue levees. Figure 56 shows the vertical effective stress on the underside of the marsh layer measured in the Orleans South and, for comparison, the London South models. The tendency of the floodwall to rotate is reduced by the height and weight of the landward section of the Orleans levee.

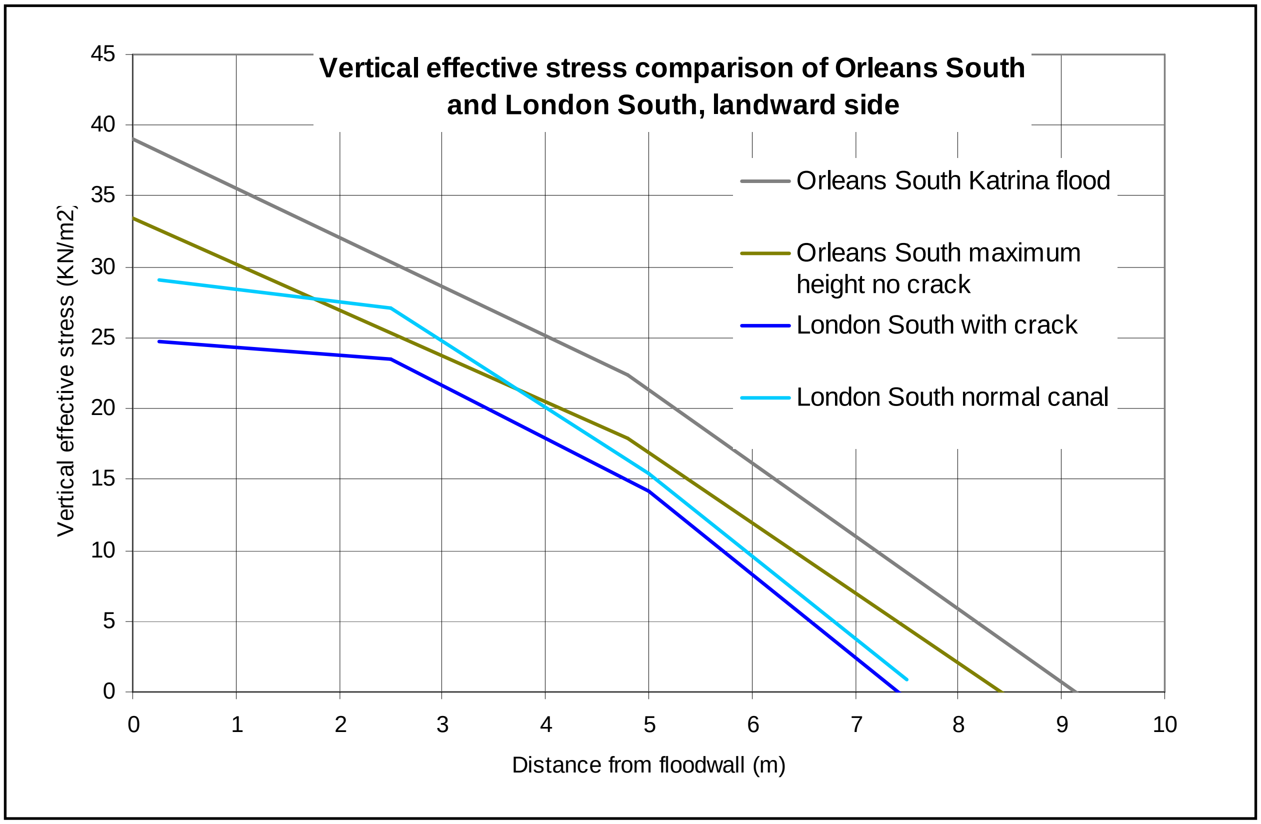

Figure 56. Vertical Effective Stress Comparison Between Orleans South and London South

A more complete description of the centrifuge modeling effort is contained in the Appendix, "IPET Centrifuge Model Test Report."

Analysis of the Performance of the Michoud Canal I-walls

Although no breach occurred in the Michoud Canal I-walls, they are being rehabilitated with additional relief wells and stability berms because of concern for their possibly marginal stability. An evaluation of the performance of these I-walls is in progress. It will be added to Volume V when it has been completed. An aerial view of the Michoud Canal is shown in Figure 57.

Figure 57. Aerial Photograph of Michoud Canal

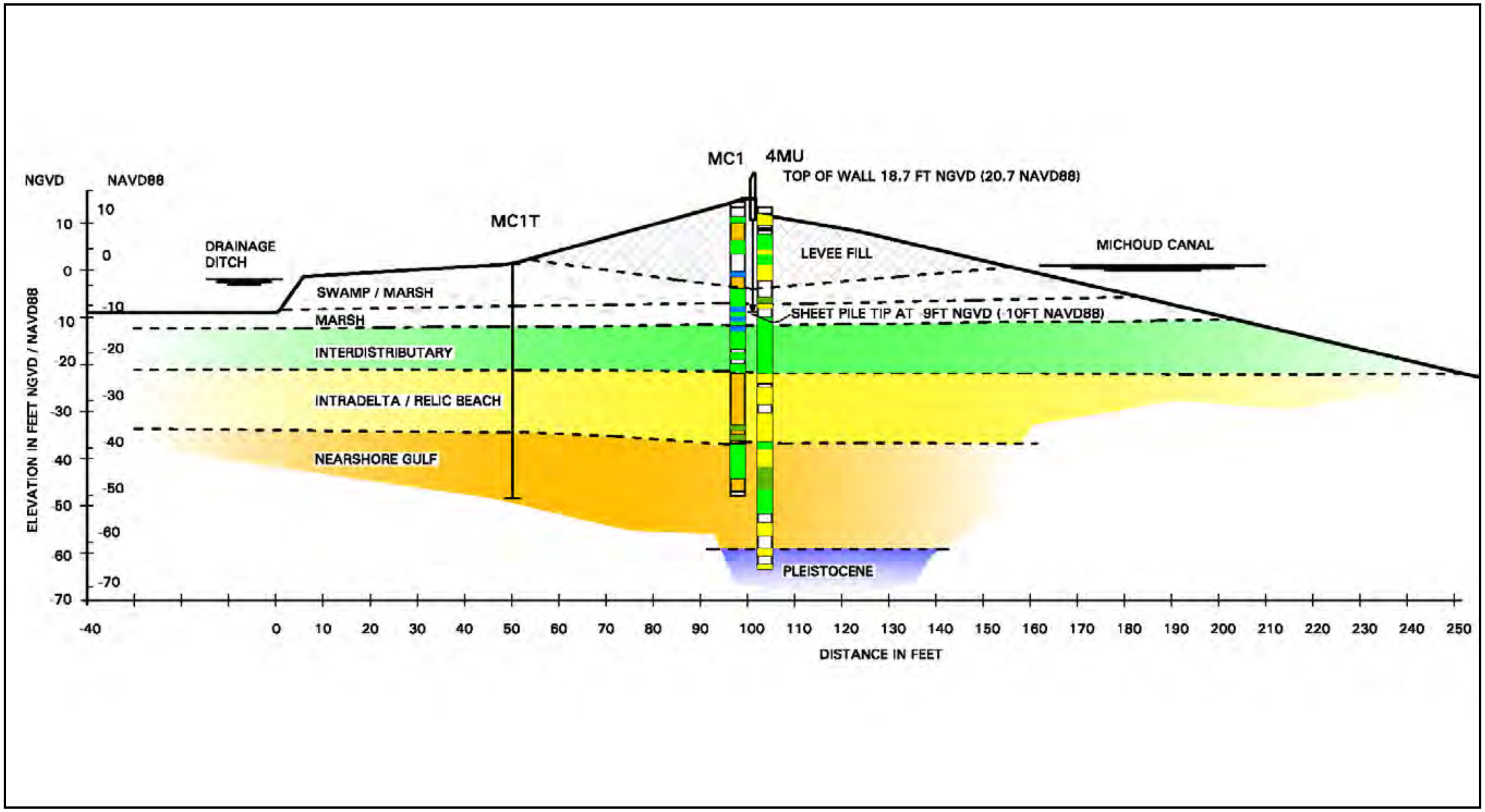

Figure 58. Geologic Cross Section of Sta. 4+00 on the Michoud Canal

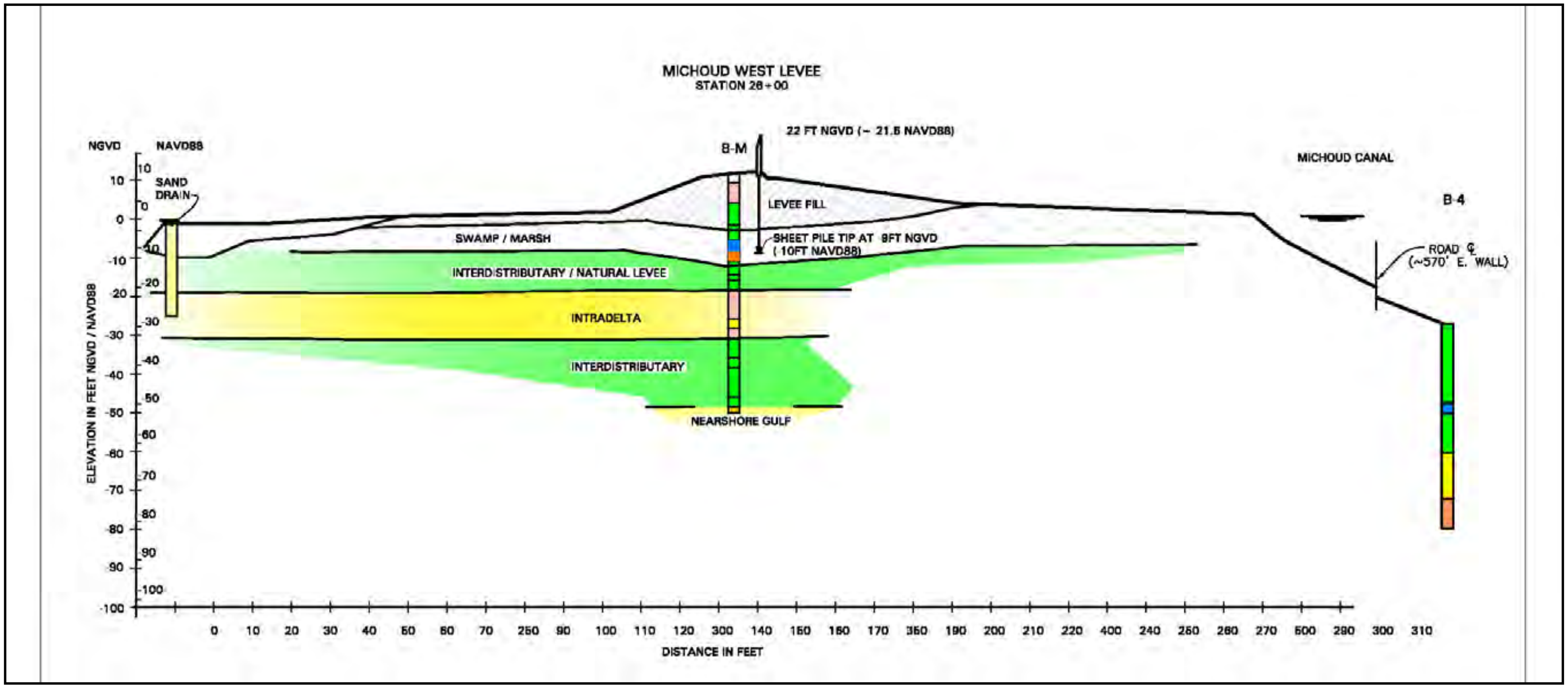

Figure 59. Geologic Cross Section of Sta. 26+00 on the Michoud Canal

Levee Erosion and Scour from Overtopping

Water overtopping the levees led to extensive scour and erosion in some locations, which ultimately 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. In other areas the levees were severely eroded and completely washed away after being overtopped. Levee performance during Hurricane Katrina has highlighted the importance of the resistance of levees to overtopping, as an important factor determining the resilience of the flood protection system.

Lengthy reaches (miles) of earthen levees and capped levees were overtopped. Some reaches showed signs of initial erosion, others showed signs of progressive erosion, and other reaches contained significant breaching. Similar to levees, lengthy reaches of floodwall were overtopped and were left in various stages of damage ranging from minor scour at the wall base to breaches where complete floodwall sections were flattened.

In the New Orleans East, Lakeshore, and St. Bernard Parish basins, approximately 50 miles of earthen levees overtopped but did not breach; approximately 20 miles of earthen levees overtopped and contained significant breaches; approximately 7 miles of floodwalls overtopped but did not breach; and approximately 2 miles of floodwalls overtopped and were breached. The majority of levees and floodwalls were damaged by overtopping, but did not breach.

In Plaquemines Parish, the combined length of the Mississippi River mainline levee and the back levee is about 162 miles. The length of I-walls and cantilever sheetpile walls is about 7 miles. All of the levees in Plaquemines Parish sustained damage due to overtopping, and there was considerable crown and slope scour in all sections. The mainline levee riverside slope pavement sustained damage from the hundreds of ships and barges that crashed into it. There were also several severe breaches, coinciding with pipeline crossings and with floodwalls. Five of the 7 miles of floodwall were damaged beyond repair. Major breaches occurred at sheet pile wing walls at two pump stations in the back levee. A major breach occurred at the Shell pipeline crossing near Nairn, and the West Pointe a la Hache pipeline crossing was severely damaged.

Several stages of erosion and scour progression were noted along numerous levee/floodwall reaches. Although conditions have been altered in many locations by construction of repairs, it is possible to infer useful information regarding soil erodibility from observations of performance during overtopping.

Failure Patterns

Very little evidence of front side (flood side) erosion was noted in the post-Katrina forensic inspections. Backside (landside) erosion patterns were observed along the breached and unbreached levee and floodwall in Orleans, St. Bernard, and Plaquemines Parishes. The following overtopping and breaching damage patterns were observed:

a. Earthen levee backside erosion caused by: (1) wave overtopping when the surge level was below the levee crest elevation, and (2) continuous water overtopping when the surge level exceeded the levee crest elevation. Progressive erosion of unprotected soil on the protected side (backside) likely contributed to levee breaching.

b. Damage to the earthen levee on the backside of floodwalls caused by wave and/or water overtopping, impacting the unprotected soil. Loss of lateral soil support and progressive erosion likely contributed to wall and levee breaching.

c. Damage to transitions between earthen levees and structures such as flood gates and floodwalls. Erosion of earthen levee material and scour at the transitions was observed, and localized overtopping was most likely due to levee/wall elevation differentials.

The following pictures and descriptions show examples of the damage patterns, and available additional information such as soil borings and pre-Katrina elevations are included to provide possible explanations for scouring erosion. Figure 60 shows the observed failure progression.

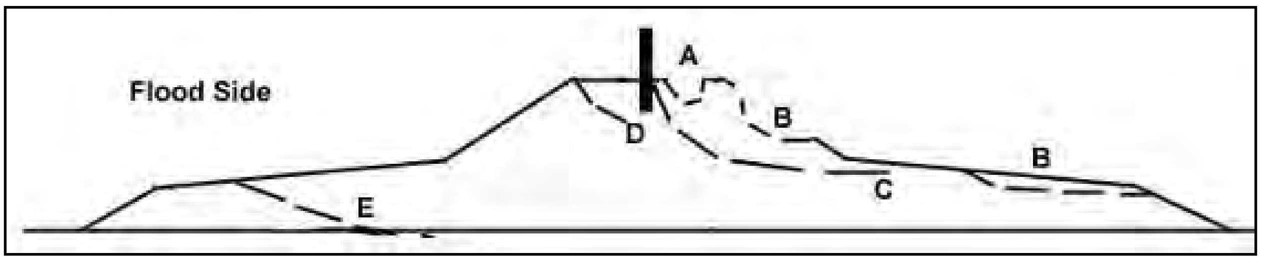

Figure 60. Erosion Progression Patterns for Earthen Levees, Floodwalls, and Exposed Sheet Pile

Scour pattern “A” indicates scour located on the protected side levee slope (or located immediately adjacent to the floodwall or sheet pile protected side); “B” indicates erosion on the protected slopes, including stabilizing transition slopes; “C” indicates erosion progressing to the levee crown adjacent to the floodwall or sheet pile protected side; “D” indicates scour on both the flood side and the protected side of the levee or floodwall; and “E” indicates that the original levee footprint has been significantly altered due to erosion, and the original foundation base may have scour holes or washouts.

Levees

The performance of levees varied significantly throughout the New Orleans area. The areas where the levees were made of clay performed well in spite of the fact that they were overtopped. In other areas the levees were completely washed away after being overtopped. Several factors appear to explain this difference of performance. One is the type of material that was used to construct the levees. The clay levees appeared to have withstood the storm the best. For instance, the levee at the Entergy power plant in the New Orleans East area, shown in Figure 61, suffered only minor erosion due to overtopping. This overtopping can be seen in the widely circulated picture taken by Entergy personnel during the storm shown in Figure 62. Preliminary results of cone penetrometer tests taken on the levee at the Entergy power plant indicate that the levee is constructed of clay (CH).

Figure 61. Overtopped Levee Under the Paris Road Bridge Adjacent to the Entergy Power Plant in the New Orleans East, After the Storm

Figure 62. Overtopping of the Levee Under the Paris Road Bridge Adjacent to the Entergy Power Plant in the New Orleans East During Hurricane Katrina

Levees with higher silt and sand content in the embankment material or zoned embankments with sand zones scoured worst after erosion began, and in some cases completely washed away. In general, levees that were subjected to overtopping with little wave action appeared to have survived better than levees that were subjected to overtopping and significant wave action. The levees along the Mississippi River Gulf Outlet on the northeast side of St. Bernard Parish and the New Orleans East back levee, which fronts Lake Borgne, had numerous breaches and were washed out over considerable lengths. These levees were constructed using hydraulic fill that contains significant silt and sand, and they were subjected to large waves and significant depths of overtopping.

An example of the erosion of a hydraulically filled levee is the New Orleans East back levee, shown in Figure 63. Figure 64 shows the location of the breaches along the levee caused by erosion, and Figure 65 shows the section of the levee that was hydraulically filled. It can be noted that the breaches are located in the section that was constructed of hydraulic fill. However, this is not a perfect correlation and other factors need to be considered. Figure 66 shows locations where the surge and wave hydrographs have been determined from calculations and high water marks. Figure 67 shows a plot of pre-Katrina levee elevations and the surge height plus the peak wave height at these locations. It can be seen that the greatest erosion of the hydraulically filled levees occurs when the surge height plus the peak wave height had the greatest level above the crest of the levee. Examination of this information has shown that the peak wave height is the most important component of the surge. The figure also shows that levees constructed of rolled fill had equal to or greater surge plus the peak wave height over their crest, yet did not breach. Therefore, it is concluded that the combination of hydraulically filled levees and high surge and wave action leads to breaches by erosion.

Figure 63. General Map of NOE Basin, Major Levee Segments are Lakefront Levees, NOE East Levee (South Point to GIWW), NOE Back Levee, Citrus Back Levee, and IHNC Levees

Figure 64. NOE Basin Post-Katrina Breaches

Figure 65. NOE Levee Construction Materials (USACE DM NOE Back Levee, Citrus Back Levee, South Point to GIWW Levee, Lakefront Levees, and IHNC Levees) 1971, 1969, etc.)

Figure 66. Selected Model Data Points for Surge Analysis

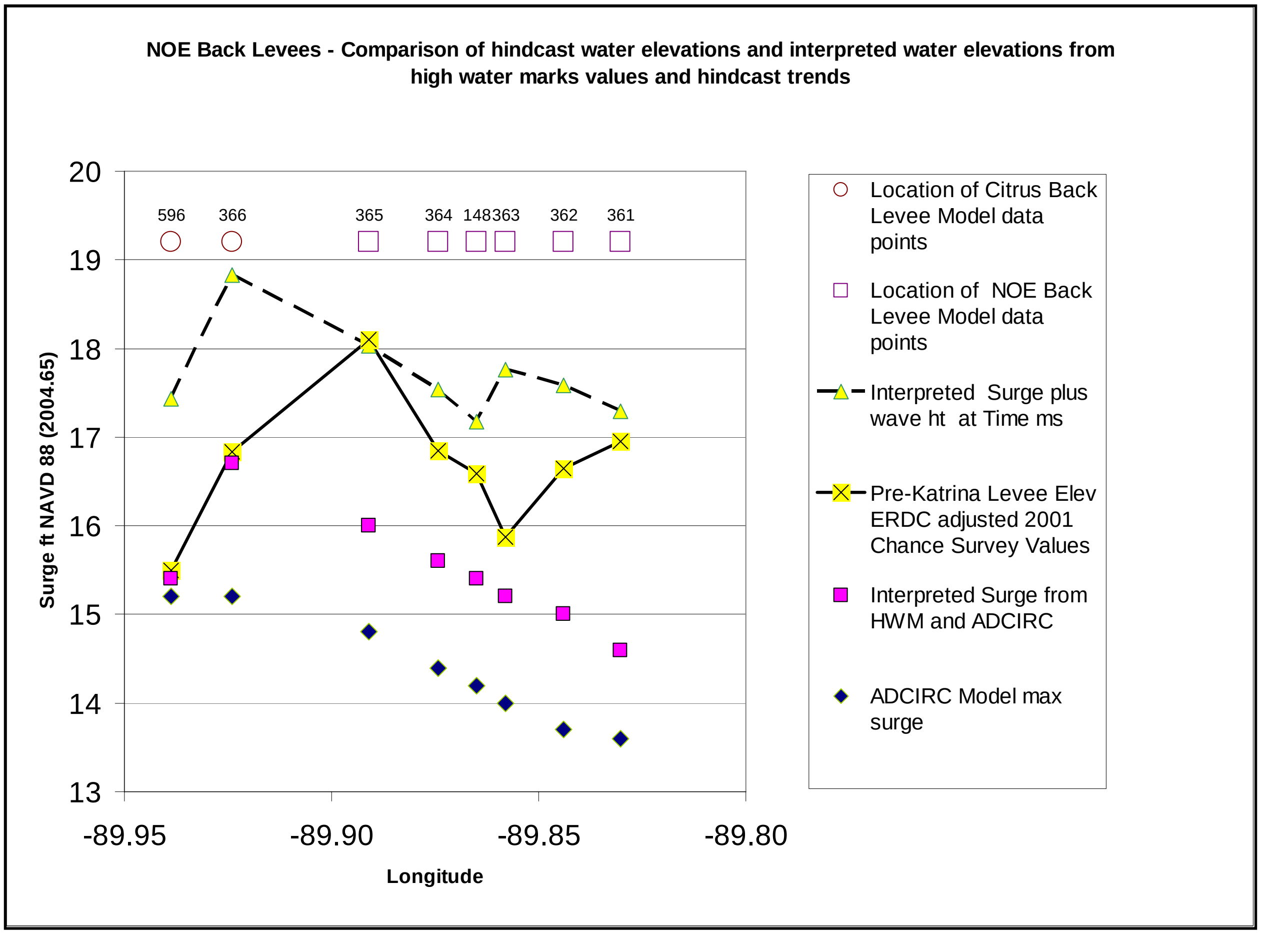

Figure 67. Data from Table 2 Plotted With Levee Elevations Derived From Pre-Katrina LIDAR for Citrus Back Levee and NOE Back Levee

Floodwalls

Water overtopping the floodwalls led to extensive scour and erosion in some locations, which ultimately resulted in breaches. This was most dramatic along the Inner Harbor Navigation Canal (IHNC) adjacent to the Lower 9th Ward where two I-wall breaches were caused by overtopping erosion. It appeared that water flowing over the floodwall scoured and eroded the levee on the protected side of the I-wall, exposing the supporting sheet piles and reducing the passive resistance. The water flowing over the top of the walls may have scoured significant amounts of soil from the levee adjacent to the wall. Evidence of this scour can be seen along the unbreached reaches of the I-walls on the IHNC where U-shaped scour trenches could be found adjacent to the I-walls, as can be seen in Figure 45. As the scour increased, the I-wall may have moved laterally and leaned to the protected side, causing the scour trench to grow as the water cascaded farther down the slope until sufficient soil resistance was lost and the wall failed, as shown in Figure 46. I-walls along the Mississippi River Gulf Outlet and the Mississippi River in Plaquemines Parish suffered similar damage due to overtopping where the greater the scour, the greater the lateral translation and tilting of walls.

It appears that the I-walls were not designed to withstand overtopping. The survivability of this type of wall could be significantly improved by providing erosion protection such as grouted rip-rap or concrete erosion mats running from the base of the wall down the face of the levee on the protected side.

While overtopping of the I-walls led to significant scour and damage in many cases, overtopping of T-walls did not lead to extensive scour and erosion. In general, T-walls did not suffer severe scour on the protected side, likely because the base of the inverted T-wall section extends out on the protected side, preventing scour adjacent to the T-wall stem.

Transitions

A 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 higher elevations than the adjacent earthen levee, resulting in scour and washout of the levee at the end of the structural feature. At these locations, the dissimilar geometry concentrates the flow of water at the intersection of the levee with the structure, causing high flow velocities and turbulence that resulted in the erosion of the levee soil. The performance at transitions could be improved by fully embedding the structural walls within the levee fill, and using the levee to transition the difference in elevation from the structure to the main section of the levee. In a few cases, observations indicated that this type of transition performed successfully. The embedded area and the transition to the main section of the levee should have erosion protection such as grouted rip-rap or concrete erosion mats.

In some cases, the structures were lower than the connecting earthen levees. At these sites, the flow of the water is channeled over the structural feature, causing erosion of soil on the protected side of the structure. The performance in these cases can be improved by providing erosion protection on the protected side of the structures and along the transition section.

Floodwall and Levee Performance Findings and Lessons Learned

Findings

The majority of approximately 50 levee and floodwall breaches resulted from overtopping and subsequent erosion, or from erosion-induced instability of I-walls.

A single design cross section was used for long sections of the I-walls, and was applied in areas where the protected side ground elevation was lower than considered in the design analysis.

Four major floodwall breaches, three in the outfall canals and one on the IHNC, resulted from shear failure or erosion and piping through the foundation soils at water elevations below the original design elevations. The foundation-induced breaches had the common element that a gap opened between the levee and floodwall, on the canal side of the wall, as the water rose against the wall. Water entering these gaps imposed increased loads on the walls.

Where the foundation soil was permeable sand, water flowing down through the gaps increased the water pressure in the sand, reduced the capacity of the foundation to resist load, and increased the likelihood of erosion and piping.

In sections where the foundation soils were clay, the shear strength of the clay was smaller beneath the levee slopes and beyond the toe than beneath the crest where the clay was compressed under higher pressures, and was therefore stronger.

No levee breaches occurred without overtopping. The degree of erosion and breaching of overtopped levees was directly related to the character of the in-place levee materials and the severity of the surge and wave action. Hydraulically filled levees with higher silt and sand content in the embankment material that were subjected to high overtopping surge and wave action suffered the most severe damage. Rolled clay levees performed well, even when overtopped.

I-walls throughout the hurricane protection system (HPS) that were subjected to overtopping suffered extensive erosion and scour of the foundation of the wall on the protected side. The only exceptions were walls that had paved surfaces adjacent to the walls on the protected side.

Significant scour and erosion occurred at many transitions between concrete structures and earthen levees.

While overtopping of the I-walls led to significant scour and damage in many cases, overtopping of T-walls did not lead to extensive scour and erosion, because the base of the inverted T-wall sections extended over the protected side.

T-walls performed well during Katrina. Because of their pile foundations, they are better able to transfer high lateral water loads into stronger underlying foundation materials.

Lessons Learned

Floodwall design criteria should consider a broad range of potential failure modes.

I-walls should be designed to be stable with a gap between the wall and the levee on the water side of the wall, with hydrostatic pressure acting through the depth of the gap.

Both horizontal and vertical variation of the strengths of clay foundation soils with overburden pressure should be considered in evaluations of levees and I-walls.

Water pressures in sand foundations beneath levees and I-walls should be evaluated by means of seepage analyses that reflect very conservative assumptions regarding hydraulic boundary conditions, and seepage control measures should be included in design as needed to reduce the potential for erosion and piping.

Rolled clay fill embankments are generally able to withstand overtopping without erosion for many hours, and should be used to construct levees wherever possible. Armoring can augment existing levee materials to provide improved erosion resilience.

Improved resistance against erosion at transitions between earthen levees and structures can be achieved by embedding the structural walls within the levee fill, and protecting the transition by armoring.

Design methods should be updated periodically to include the review of recent research and case histories.