Declaration No. II.

ENGINEERING FORENSIC STUDIES OF PERFORMANCE OF THE MAN-MADE FEATURES BORDERING THE INNER HARBOR NAVIGATION CANAL AT THE LOWER 9TH WARD DURING HURRICANE KATRINA

Report to

Katrina Canal Breaches Consolidated Litigation

[Civil Action Number: 05-4182 "K" (2)]

United States District Court

Eastern District of Louisiana

Pertains to MR-GO, Robinson

(No. 06-2268)

By

Dr. Robert Bea

Risk Assessment and Management Services

Moraga, California

July 11, 2008

Format edits: July 15, 2008

DECLARATION No II:

Robert G. Bea, under penalty of perjury, states as follows:

1. This Declaration is submitted on behalf of the Plaintiffs in *Robinson v. United States*. As detailed below, this Declaration constitutes the portion of my expert report concerning the performance during Hurricane Katrina of the man-made features bordering the Inner Harbor Navigation Canal (IHNC) at the Lower 9th Ward. In particular, this Declaration will address engineering forensic studies, providing insights into the reasons for breaches, failures, and overtopping that developed with respect to those features. I incorporate by reference here all previous expert reports I have prepared for the Plaintiffs in *Robinson v. United States*. If called to testify, I could and would testify competently as follows:

2. My Declaration is divided into three sections. Section I provides an overview of my qualifications to serve as an expert on the issues discussed herein. Section II provides a summary of engineering forensic analyses to determine the causes of the breaches, failures, and overtopping that developed with respect to the man-made features along the IHNC at the Lower 9th Ward during and following Hurricane Katrina. Section III summarizes the major conclusions drawn from this work. The following table of contents will serve as a guide to help reading this Declaration.

3. This Declaration supplements my previous declaration dated March 25, 2008 (Bea 2008a) that addressed engineering forensic studies that provided insights into the reasons for breaches, failures and overtopping that developed with respect to the man-made flood protection structures located at the Lower 9th Ward.

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I. QUALIFICATIONS

4. I have devoted the past two decades of my professional career to research associated with the catastrophic failure of engineered systems. This work has involved detailed studies and investigations of more than 600 major accidents, failures, and disasters associated with complex engineered systems. The research has focused primarily on interactions of engineering and organizational–institutional mechanics associated with catastrophic failures. A primary objective of this work has been development, validation, and application of advanced methods for Risk Assessment and Management (RAM) of complex engineered systems during their life-cycles (concept development through decommissioning). This work has been built on my more than five decades of experience with the design, construction, operation, maintenance, and decommissioning of major engineered systems around the world.

5. Of particular importance to my testimony is the work I have performed since August 29, 2005 to study developments associated with Hurricane Katrina and to investigate the failure of the hurricane flood protection system for the Greater New Orleans area (particularly Orleans and St. Bernard Parishes) associated with Hurricanes Katrina and Rita. During the past 35 months, I have spent more than 7,500 pro bono hours assisting in the forensic engineering studies of the breaches, failures and gaps in what the USACE has claimed to be a hurricane flood protection system that led to the catastrophic flooding of Greater New Orleans. In addition, I have performed studies of the USACE repairs and improvements to the existing hurricane flood protection system, their ongoing studies of the risks associated with future hurricane flood protection systems and the associations of these proposed protection systems with coastal environmental restoration.

(a) This work has involved multiple field studies (starting 30 September 2005) in Greater New Orleans, interviews of people with extensive knowledge of the history of the New Orleans flood protection system (how it was designed, constructed, operated, and maintained), and review of the primary investigations and studies that have documented this failure, including, but not limited to the following reports: (a) USACE Interagency Performance Evaluation Task Force, *Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System*, 2007; (b) D. Woolley and L. Shabman, *Decision-Making Chronology For the Lake Pontchartrain & Vicinity Hurricane Protection Project*, Report to Institute for Water Resources of the U.S. Army Corps of Engineers, June 2008; (c) Team Louisiana, *The Failure of the New Orleans Levee System During Hurricane Katrina*, Report to Louisiana Department of Transportation and Development, 2007; (d) American Society of Civil Engineers, *The New Orleans Hurricane Protection System: What Went Wrong and Why*, 2007; (e) National Academy of Engineering and National Research Council, Committee on New Orleans Regional Hurricane Protection Projects, Third Report, 2006; (f) Independent Levee Investigation Team, *Investigation of the Performance of the New Orleans Flood Protection Systems in Hurricane Katrina on 29 August 2005*, 2006; (g) National Institute of Standards and Technology, *Performance of Physical Structures in Hurricane Katrina and Hurricane Rita: A Reconnaissance Report*, 2006; (h) Federal Emergency Management Agency, *Hurricane Katrina in the Gulf Coast*, 2006; (i) Committee on Homeland Security and Governmental Affairs, *U.S. Senate, Hurricane Katrina A National Still Unprepared*, 2006; and (j) The Committee to Investigate the Preparation for and Response to Hurricane Katrina, U.S. House of Representatives, *A Failure of Initiative*, Feb. 2006.

(b) I was a co-leader of the Independent Levee Investigation Team (ILIT) that conducted a 10-month intensive forensic study of this failure. During this time, I made eight separate inspections of the MR-GO man-made features (from the land, water, and air). Since completion of the report (May 2006), I have made an additional six inspections of the MR-GO man-made features.

(c) I co-authored the report titled *Investigation of the Performance of the New Orleans Flood Protection Systems in Hurricane Katrina on 29 August 2005* (http://www.ce.berkeley.edu/~new_orleans/, November 2005 Preliminary Report, May 2006 Draft Final Report, July 2006 Final Report). I have authored and co-authored 13 refereed professional conference and journal papers that have documented this forensic work.

6. I am a professor in the Department of Civil and Environmental Engineering, and Associate Director of the Collaborative for Catastrophic Risk Management, University of California, Berkeley. I have held this position since January of 1989. My current responsibilities include teaching at the graduate and undergraduate levels, research, and university, professional and public service. Prior to that, for seven years I held the position of Vice President and Senior International Consultant at PMB - Bechtel's Ocean Engineering Division located in San Francisco, California. Attached hereto as Exhibit 1 is my curriculum vitae. This is attached to support my competency to attest to the matters stated herein.

7. I began my career in 1954 as an engineer-in-training for the U.S. Army Corps of Engineers. I was assigned to the Southern Florida Flood Control District and worked on engineering and construction of flood protection facilities (levees, drainage canals, pump stations) in the areas around Lake Okeechobee and the Everglades.

8. In 1960, I was employed by Shell Oil Company as a coastal - offshore civil engineer. During my career with Shell Oil Company, Shell Development Company and Royal Dutch Shell Company, I worked in exploration, drilling, production, refining, transportation, engineering, construction, operations, and research.

9. In 1965, I was Chief Offshore Civil Engineer and Manager of the Central Engineering Division for Shell Oil Company located in New Orleans. It was during that year my family lost our home in New Orleans East (Pines Village). This loss included all of our belongings due to flooding from breaches in the putative levees along the Inner Harbor Navigation Canal (IHNC) and the MR-GO – Gulf Intracoastal Waterway (GIWW) that developed during Hurricane Betsy in September 1965. At this time, I was leading engineering teams working on development of risk and reliability based criteria for design and requalification of offshore platforms and pipelines subjected to hurricanes in the vicinity of the Mississippi River Delta and elsewhere in the Gulf of Mexico.

10. In 1977, I was appointed vice president and chief engineer of an international consulting engineering and contracting company (Woodward-Clyde Ocean Services [now URS Corporation]) that provided coastal and offshore engineering services, including hurricane forecasting, development of reliability based design criteria, and engineering flood protection facilities for refineries and chemical processing plants along the Gulf coast.

11. In 1981, I founded the Ocean Engineering Services Division of PMB and became vice president and senior international consultant for PMB – Bechtel. The Ocean Engineering Services Division offered a wide variety of engineering services world-wide that included concept development, design, construction, maintenance, and decommissioning of marine systems. Of particular importance was work performed by this Division in arctic and sub-arctic

areas in development and testing of innovative systems to work in this very challenging environment. This work included development of risk and reliability based engineering design criteria for these systems.

12. I joined the faculty of the University of California Berkeley in 1989. Currently, I am responsible for one senior capstone design course (Engineering Systems) and two graduate courses (Margins of Quality for Engineered Systems, Human & Organizational Factors in the Reliability of Engineered Systems). During the past 2 decades, my research has been focused on development of technology to address human and organizational factors in the reliability of engineered systems. This research has involved detailed analyses of more than 600 major accidents and disasters involving engineered systems, including the flooding of the Greater New Orleans area during Hurricane Katrina.

13. I have published more than 450 technical papers in refereed journals and conferences and a similar number of technical papers, reports, and book chapters in non-refereed publications. This includes 13 refereed journal and conference papers that have addressed engineering and organizational causes of the major breaches that developed during Hurricane Katrina.

14. I have been recognized by the National Academy of Engineering, the International Energy Center, the Academy of Management, the American Society of Civil Engineers (ASCE), and American Society of Mechanical Engineers (ASME) for my pioneering contributions to development and application of advanced methods for risk assessment and management including human, organizational, and institutional factors.

15. In May 2007, I received the first University of California Berkeley Chancellor's award for research in the public interest for my work in development of understanding of the

causes of the failure of the flood protection system for Greater New Orleans and for my proposals for preventing and mitigating such failures in the future. In December 2007, I received a Proclamation from the New Orleans City Council in gratitude for "research and efforts in promotion flood protection and to aid in the recovery of New Orleans and Louisiana."

16. I am a registered professional civil, structural, and geotechnical engineer (retired) in Louisiana, Texas, Florida, California, Washington, Oregon, and Alaska.

17. This Declaration is an extension of and supplement to the Declarations I have submitted on April 16, 2006 and March 25, 2007 in behalf of the Plaintiffs in Robinson v. United States.

II. FORENSIC ENGINEERING OBSERVATIONS

Field Studies

18. I first arrived in New Orleans as a member of the National Science Foundation and University of California sponsored Independent Levee Investigation Team (ILIT) on 30 September 2005. Figure 1 depicts the locations of the major breaches and failures that I have studied (ILIT 2006). This Declaration will address analyses of the breaches that developed along Reach 1 at the Lower 9th Ward (identified as 6a and 6 in Figure 1). During the past nearly three years, I have made multiple field trips and performed investigations of the major breaches and failures that were responsible for flooding Greater New Orleans. These trips have included study and investigations of the repair and re-construction work that has been performed on these breaches and failures. I am not aware of any other civil engineer who has devoted more time to, or written more professional articles and papers concerning the performance of the man-made flood protection features along Reach 1 and Reach 2 of the MR-GO.

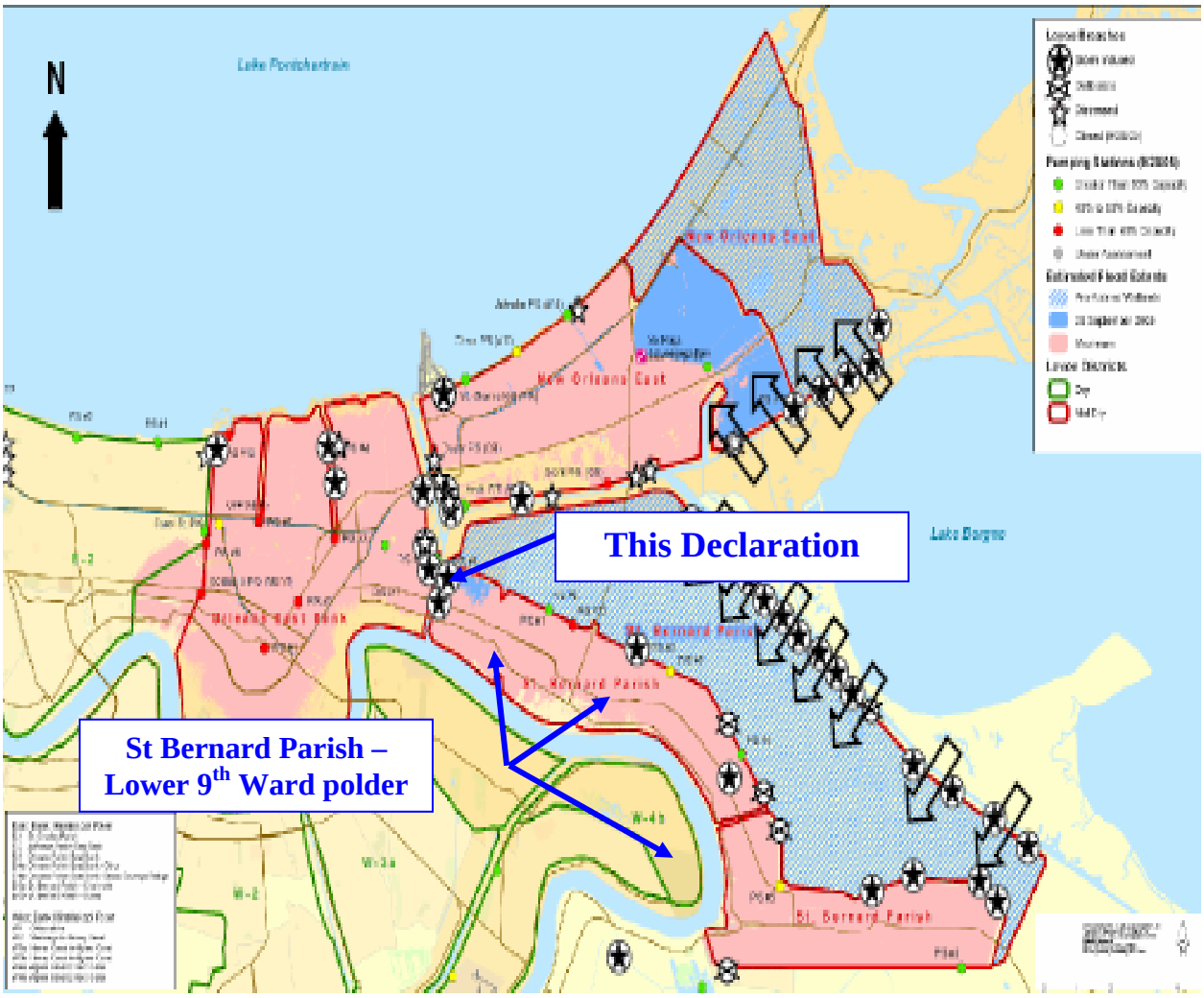


Figure 1: Summary of principal failures, breaches, and distress features with identification of the two breaches (6a, b) at the Lower 9th Ward adjacent to the IHNC (base map provided by USACE, ILIT 2006).

III. FORENSIC ENGINEERING ANALYSES

Field Observations

19. As the focused and amplified storm surge generated by Hurricane Katrina from Lake Borgne and the Gulf of Mexico pushed westward along the east-west trending channel of the MR-GO and into the 'T' formed with the IHNC, it rapidly raised the water levels in the

IHNC. As described by Dr. Paul Kemp (2007) and others (van Heerden et al 2007; Mashriqui et al 2006; Team Louisiana 2007), the MR-GO – ICWW 'funnel' and its 'T' intersection with the IHNC had multiple effects that were responsible for the early arrival and increased intensity of the Hurricane Katrina surge at this location (Figure 2).

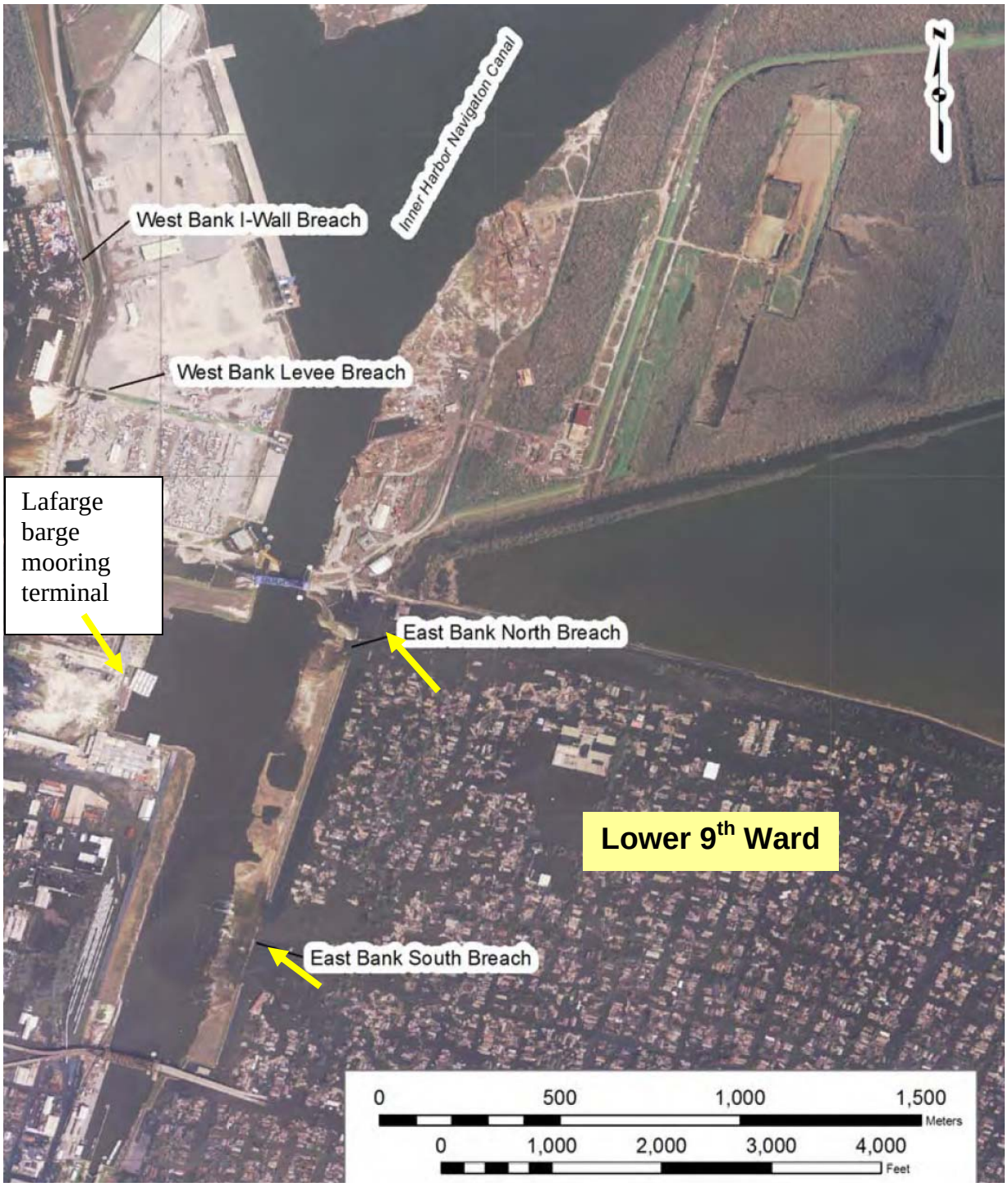


Figure 2: Breach locations at the Lower 9th Ward (USACE IPET 2007).

20. Figure 3 shows a hydrograph of measured water levels versus time in the IHNC channel. The peak surge recorded in this area during Hurricane Katrina was +14.5 feet at 9:00 am CDT (IHNC Lock). The floodwalls at the Lower 9th Ward would have begun overtopping at about 7:00 am CDT. It is important to recognize that development of breaching in the general vicinity of the Lower 9th Ward, the Port of New Orleans and elsewhere along the IHNC (e.g. the CSX railway sandbag gate closure blowout) would be expected to influence these recorded water elevations.

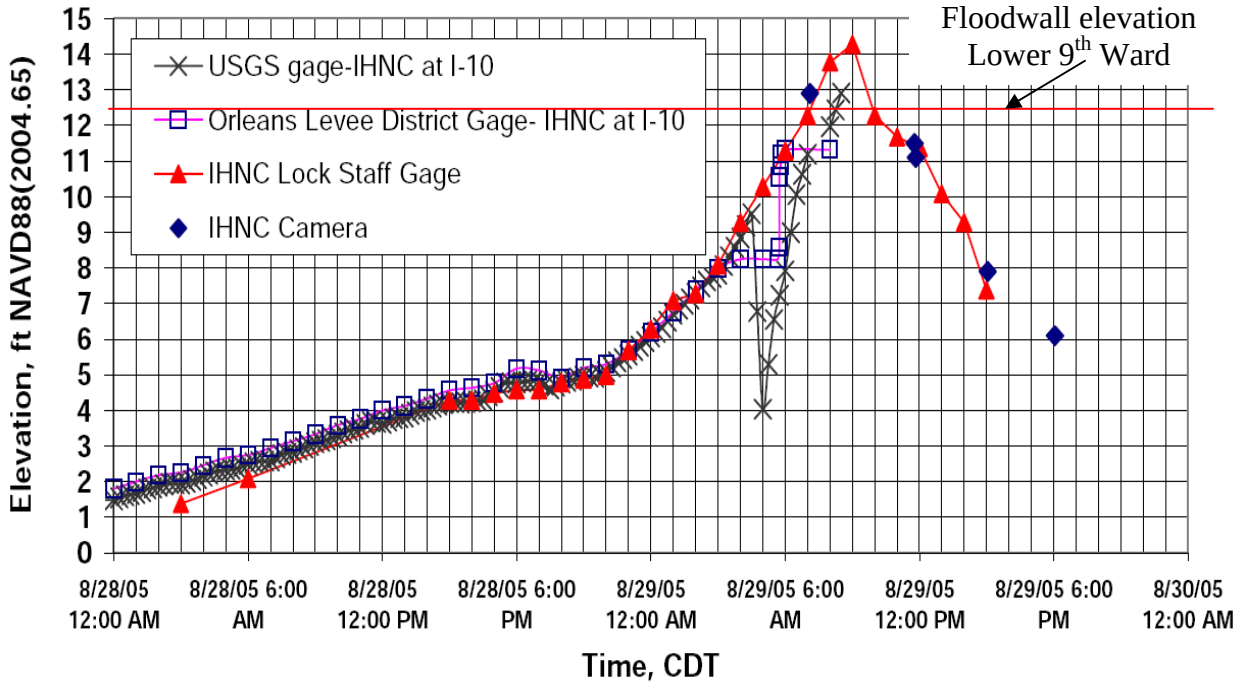


Figure 3: Hydrographs showing measured (and photographed) water levels at gage stations along the IHNC (USACE IPET 2007)

21. Figure 4 shows the high water marks on IHNC west bank from the IHNC lock on the south to Lake Pontchartrain on the north. The highest values (in excess of 15 feet) occur directly opposite to the MR-GO –IHNC "T" intersection. The IHNC lock also serves as a 'dam' to increase the water elevations in that area. The 'venting' effect of Lake Pontchartrain to lower the surge water levels is clearly evident.

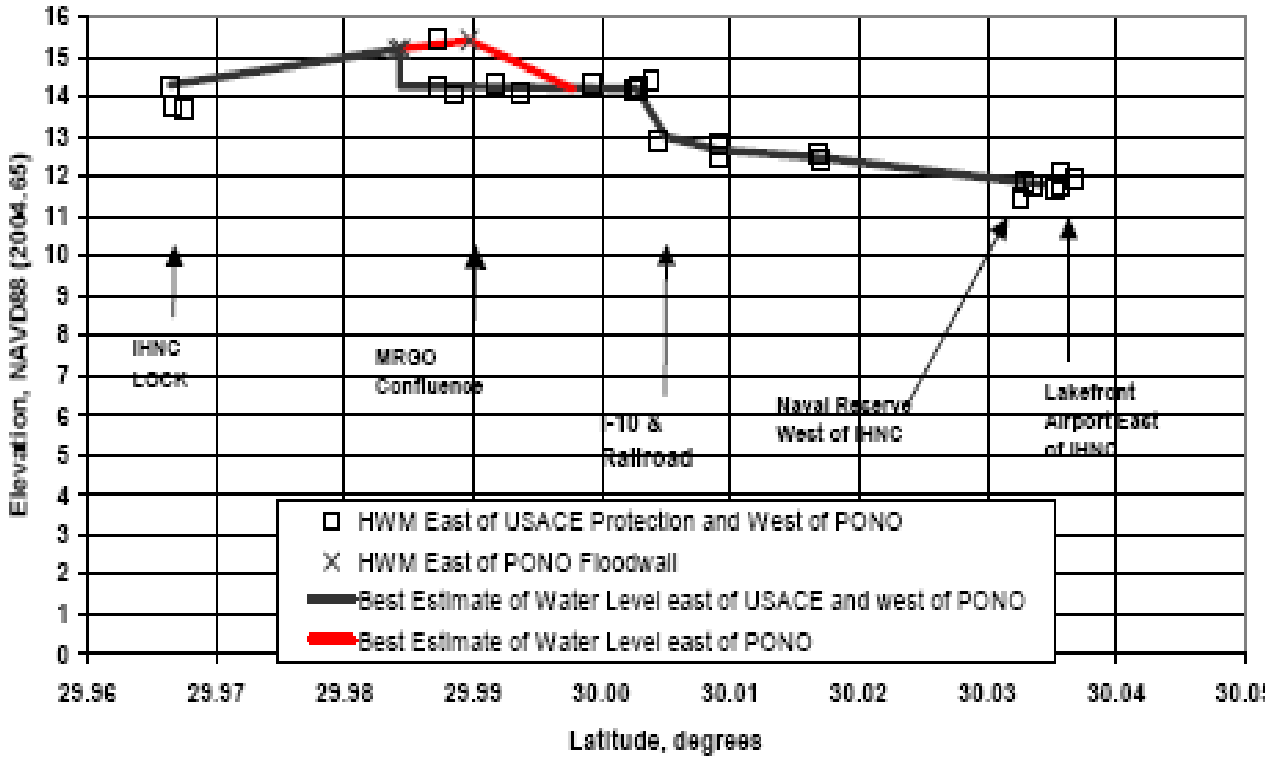


Figure 4: High water marks on IHNC west bank from IHNC lock on south the Lake Pontchatrain on north (USACE IPET 2007).

22. Figure 5 shows computed hydrographs at the IHNC lock for 5 different MR-GO conditions (Wit et al 2008). Table 1 summarizes the conditions represented in these Scenarios. Scenario 1 is for the Hurricane Katrina 'as was' conditions. The Scenario 2-series and Scenario 3 are for various elements associated with a hurricane 'Neutral' MR-GO. The term 'Vegetation' in Table 1 refers to the existing vegetation associated with the MR-GO environment at the time of Hurricane Katrina and with the environment for hurricane 'Neutral' MR-GO conditions. It is important to understand that the surge hydrographs in Figure 5 do not include breaching developed along any parts of Reach 1, Reach 2, or the IHNC (based on intact man-made flood protection structures). This is the reason why the peak surge shown in Figure 3 of +14.5 feet is less than the peak surge of +17.5 feet shown in Figure 5.

Table 1: Summary of physical and environmental conditions utilized in surge hydrograph analyses

Scenario	MRGO channel	GIWW channel	Reach 2 EBSBs	40 Arpent Levee	Vegetation
1	**Existing 8/2005**	**Existing 8/29/2005**	**Existing 8/29/2005**	**Existing 8/2005**	**Existing 8/2005**
2A	None	Pre-construction	None	LPV authorized & no flow past	Pre-construction
2B	None	Pre-construction	None	Existing 8/29/2005 & flow to Miss. River	Pre-construction
2C Neutral MR-GO	**Pre-construction**	**Pre-construction**	**Existing 8/2005**	**Existing 8/2005**	**Without MR-GO adverse effects**
2D	None	Pre-construction	Existing 9/9/1965	Existing 8/2005	Pre-construction
3	As authorized	As authorized	Existing 8/2005	Existing 8/2005	Pre-construction

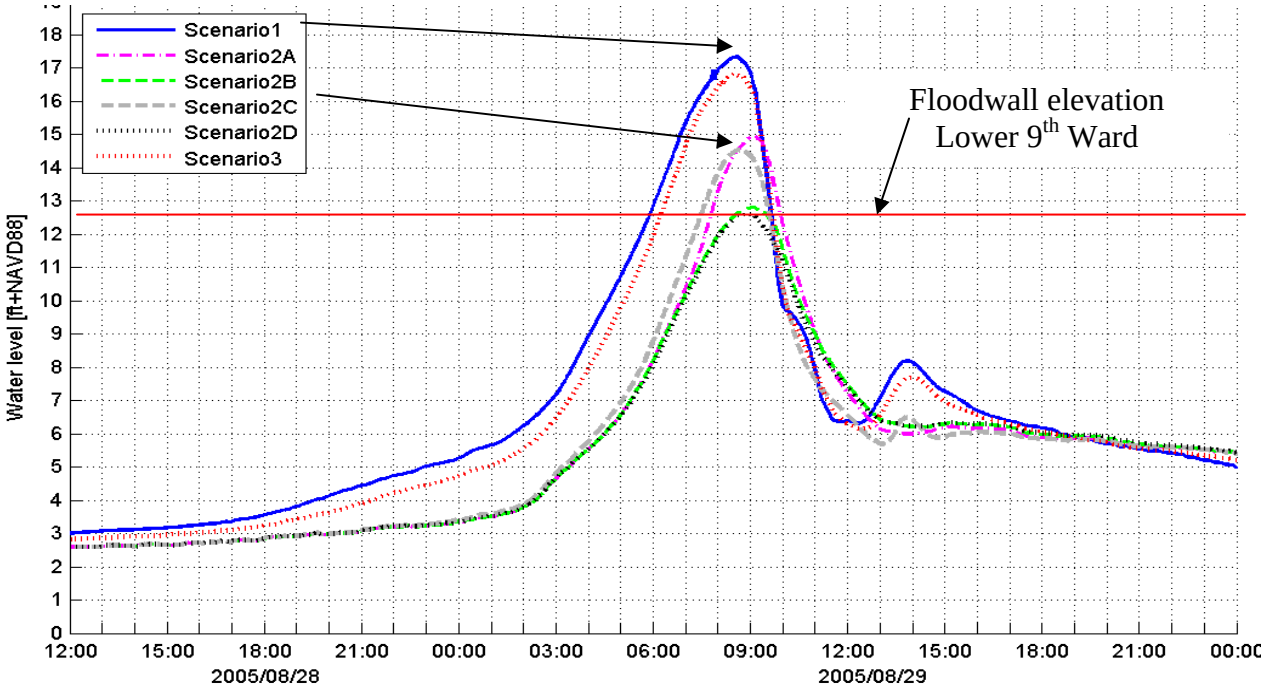


Figure 5: Computed hydrographs at the IHNC lock - Lower 9th Ward (Wit et al 2008).

23. The peak surge is approximately 17.5 feet for Scenario 1 and 14.5 feet for the 'Neutral' MR-GO Hurricane Katrina conditions (Scenario 2C). This peak occurs at approximately 8:30 am (CDT). The peak surge is approximately 15 feet, 13 feet, and 12.5 feet for Scenarios 2A, 2B, and 2D, respectively. The largest impacts on the peak surge and the

duration of the surge is attributable to the three 2-series Scenarios (no MR-GO channel or adverse effects on vegetation). The presence of the MR-GO channel, the associated man-made flood protection structures, and the adverse effects of the MR-GO on the surrounding natural environment had important exacerbating effects on the surge elevations and durations in the IHNC at the Lower 9th Ward.

24. There are important differences between the recorded hydrographs during Hurricane Katrina (Figure 3) and the computed hydrographs for Hurricane Katrina 'as was conditions' at the INHC lock – Lower 9th Ward between approximately 4:00 am and 9:00 am (CDT). Available evidence indicates these differences are due to the development of breaches and overtopping along the IHNC and GIWW (Gulf Intracoastal WaterWay) connecting the surge developed in the MR-GO and Lake Borgne. Based on the results summarized in Figure 4, overtopping of the floodwalls along the Lower 9th Ward would have started at approximately 6:00 am. The surge hydrograph in Figure 4 indicates overtopping would have started at approximately 7:00 am. The difference could be attributed to the initiation of breaching at the Lower 9th Ward. Overtopping of the floodwalls on the west bank of the IHNC in the vicinity of the Port of New Orleans would have started at approximately 7:00 am. For Scenario 2B, there would have been little or no surge overtopping at the Lower 9th Ward.

25. Hurricane Katrina surge waters produced two massive breaches on the east bank of the IHNC (at the western edge of the Lower 9th Ward) (Figures 6 and 7). The larger of these two breaches was the South Breach. This was a very long breach, nearly 900 feet in length, and the inrushing waters entered the adjacent Lower 9th Ward with great force.



Figure 6: South Breach at Lower 90th Ward (USACE IPET 2006). Note large excavation on IHNC side of floodwall. This excavation was associated with the EBIA site clearing operations.



Figure 7: North Breach at Lower 9th Ward (USACE IPET 2006. Note excavation on IHNC side of floodwall. This excavation was associated with the EBIA site clearing operations.

26. The majority of available evidence indicates the North Breach initiated before the wall was overtopped (about 5:00 am), with the breach fully developing between 6:00 and 7:00 am CDT on August 29, 2005 (USACE IPET 2007, ILIT 2006, Seed et al 2008, Sills et al 2008). Available evidence indicates the South Breach also initiated its development about 5:00 am CDT (overtopping started about 6:00 am CDT). The South Breach fully developed between 7:30 and 9:00 am CDT on the same day (USACE IPET 2007, ILIT 2006, Team Louisiana 2007). Eyewitness evidence and stopped clock data indicates that it took several hours for these breaches to fully develop (Figure 8, IPET 2006, ILIT 2006, Team Louisiana 2007).

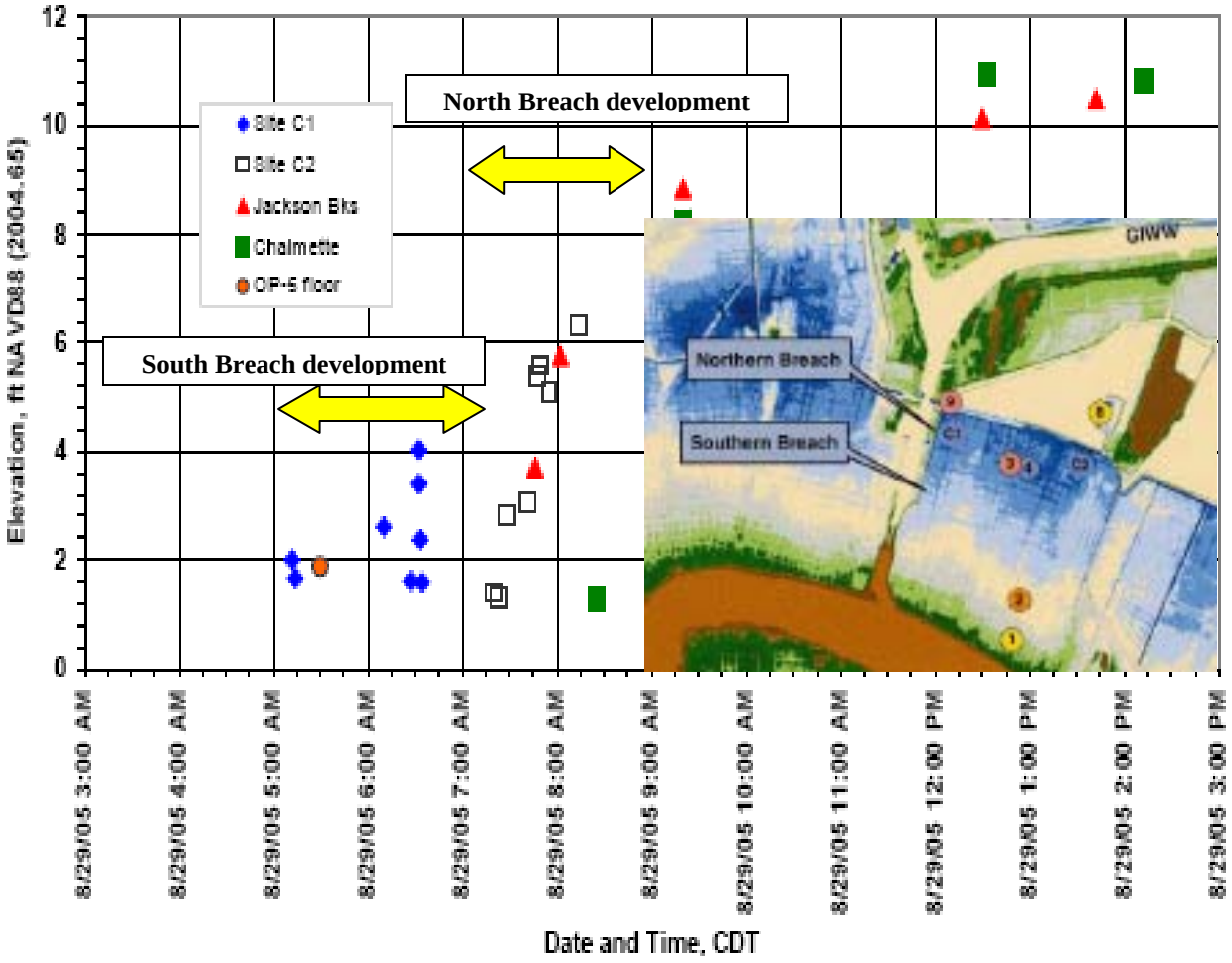


Figure 8: Stage interior hydrographs based on eyewitness accounts and stopped clock data in Lower 9th Ward and Chalmette (IPET 2007). Site 1 is Jackson Barracks (see inset map). Sites C1 and C2 are stopped clocks along the north border of the Lower 9th Ward (lowest elevation area). Site OP-5 is the pump station located on the north side of the Lower 9th Ward.

27. Available information and experience indicates that it can take several hours for floodwall or levee breaches – failures - to fully develop. Development of a breach is most often a progressive process involving multiple interactive modes of failure (Figure 9) and does not develop instantaneously. For example, if the process is initiated by seepage – hydraulic effects developed under the floodwall and supporting levee, the seepage will take time (several hours or more) to develop significant water conduits and weaken or remove soil. At the same time, in response to these effects, there can be associated movements of the supporting levee. These movements can act to open the vertical joints in a concrete floodwall (water stops). If the supporting levee – soils differential horizontal motions are great enough these can result in separation of the steel sheet piling interlocks. Water flowing through the water stops and sheet piling interlocks can act to erode the supporting soil levee. Flooding can be initiated with a combination of water coming under the levee and through the opened water stops. This process was captured by a video camera during development of the breach at the 17th Street Canal (Bea 2008, 2007; Bea and Cobos-Roa 2007). This breach took approximately 3 hours to develop. Similar processes have been photographed during development of breaches in soil levees in other areas initiated by seepage and under-base hydraulic effects (de Vriend and Barends 2007).

28. There can be similar developments when the progressive failure process is initiated by or accompanied by overtopping (Figure 9). Overtopping a levee or floodwall can lead to erosion on the back or protected side. If the protected side of the floodwall is not provided with erosion protection (as was the case for all of the flood protection I-walls in the Greater New Orleans Area – including those at the Lower 9th Ward), erosion can act to remove supporting soils. Many such features were observed along the flood protection structures bordering the IHNC. The erosion takes significant time (hours) to develop in depth and width

(Briaud et al 2007, 2008, IPET 2007, ILIT 2006). The time to develop significant erosion will be function of the impact of the water on the soil (water vertical velocity and pressure effects developed in the soils) and the soil erosion resistance characteristics (well compacted cohesive soils are highly erosion resistant; loosely compacted sandy soils are highly erosive). If the floodwall is progressively tipping under the water forces and levee – supporting soil movements, there will be a progressive widening in the erosion or scour trench. If the erosion process removes enough of the supporting soils, the floodwall and supporting sheet piling subjected to lateral forces from the water can tip over. The overtopping mechanisms can interact with the previously discussed mechanisms that are not dependent on overtopping. All of these developments take time to evolve and interact and generally take several or more hours to develop fully into a breach.

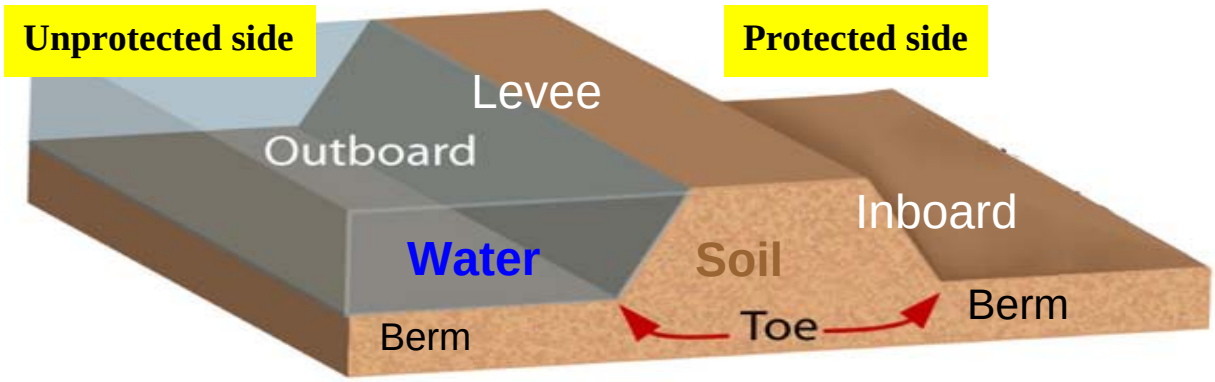
29. Available information indicates that all of the foregoing mechanisms were operative in development of both breaches at the eastern side of the Lower 9th Ward. In addition, another mechanism – contributor – was present: the Ingram barge (ING 4727) found inside the South Breach at the Lower 9th Ward. Although this mechanism is not a primary subject of this Declaration, the work summarized herein, and the forensic investigations of the USACE (IPET 2007), ILIT (2006), and Team Louisiana (2007) indicate that the ING 4727 barge was a victim, not a cause of the South Breach. This evidence indicates the barge arrived in the last stages of development of the South Breach.

30. Available information indicates that the barge most probably broke away from its moorings at the Lafarge Terminal (on the east side of the IHNC opposite the north end of the Lower 9th Ward floodwall and immediately south of the Florida Avenue Bridge, (Figure 2) at approximately 9:00 to 10:00 am (CDT). The empty barge was tied broadside outside another

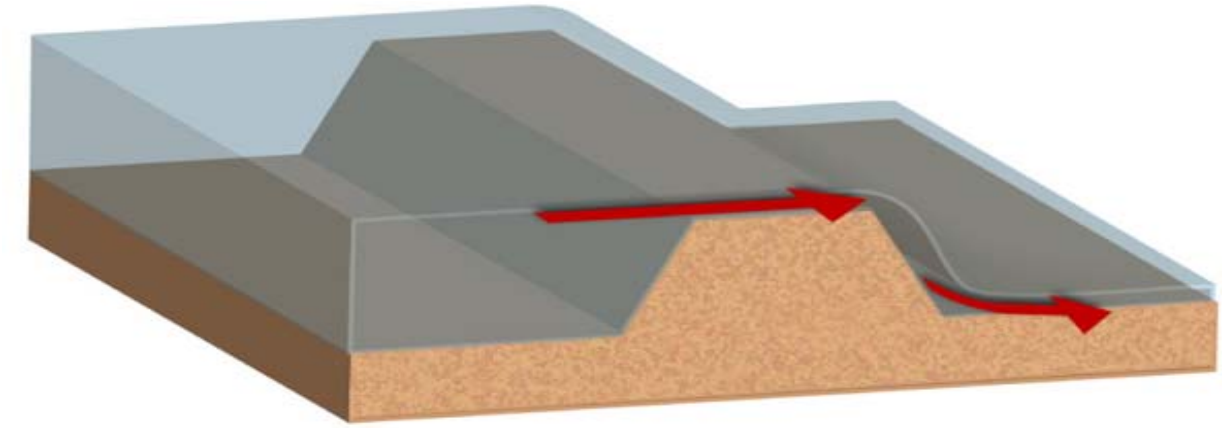
loaded barge tied to the wharf. Five other barges were moored along side each other immediately to the north. The geometry and ballast condition of the barge was such that forces acting on it would be dominated by wind forces (IPET 2007). At this time, the winds were forceful enough and from a direction that would carry the barge into the IHNC – away from the Lafarge Terminal (IPET 2007). The wind and current forces in the IHNC were then sufficient to drive the barge south toward the IHNC lock (de Wit et al 2008, IPET 2007) - arriving in the vicinity of the South Breach between approximately 9:30 am to 10:30 am (CDT) – well after the South Breach had completed its development. These investigations indicate the barge was drawn into the south end of the South Breach by a significant flow of water through and over the south end of this breach. Available evidence indicates the barge impacted the south end of the South Breach, causing local damage to the failing concrete floodwall and driven by the wind and forces from the inrushing surge flood waters the barge scraped over the top of the floodwall and was swept to a position against the first row of homes facing Jourdan Avenue (runs parallel to the floodwall).

31. This discussion serves to highlight that there can be and generally will be different mechanisms active during development of a breach or failure of a flood protection structure (Figure 9). These different mechanisms (e.g. overtopping erosion, under-base seepage – soil strength reduction, hydraulic uplift, water stop and sheet pile interlock failure, interactions with floating objects and debris) can act singly or more commonly in combination during development of a breach or failure. Given that it is rare that sufficient valid measured data or photographic evidence is available to enable accurate assessment of the interactions and developments of these potential interacting mechanisms, development of the mechanisms and their interactions must therefore be deduced from a variety of observational and analytical sources. The greater number and wider variety of reliable sources (clues) – then the deductions

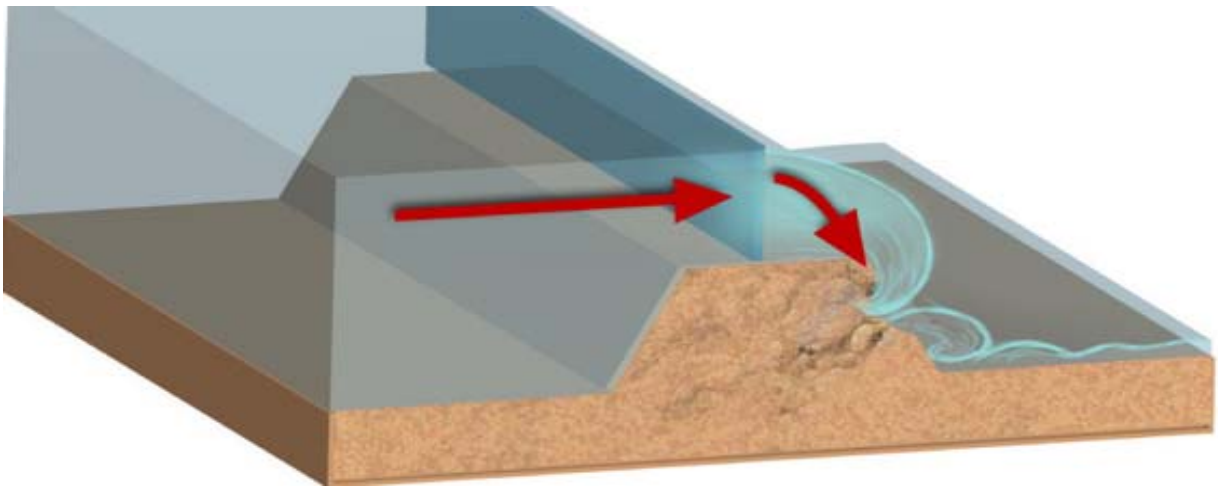
can be more probable or accurate. However, this is only true if a wide variety of potential biases (e.g. institutional, hindsight, confirmational, small data sets, correlation, reductive, belief) are not allowed to influence the deductions – i.e., they are neutralized (Bea 2006a, 2006b). In the work summarized in this Declaration and the supporting Technical Report (Bea and Cobos-Roa 2008), significant efforts have been devoted to gathering information from as wide a variety of reliable sources as possible and to neutralizing potential biases in developing the conclusions documented in this Declaration.



Levee geometry

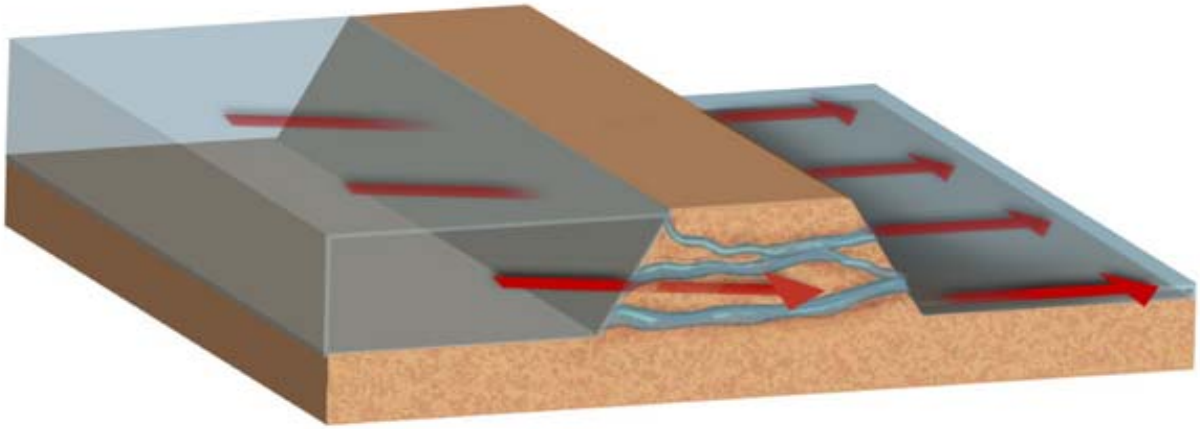


Water over the levee - overtopping

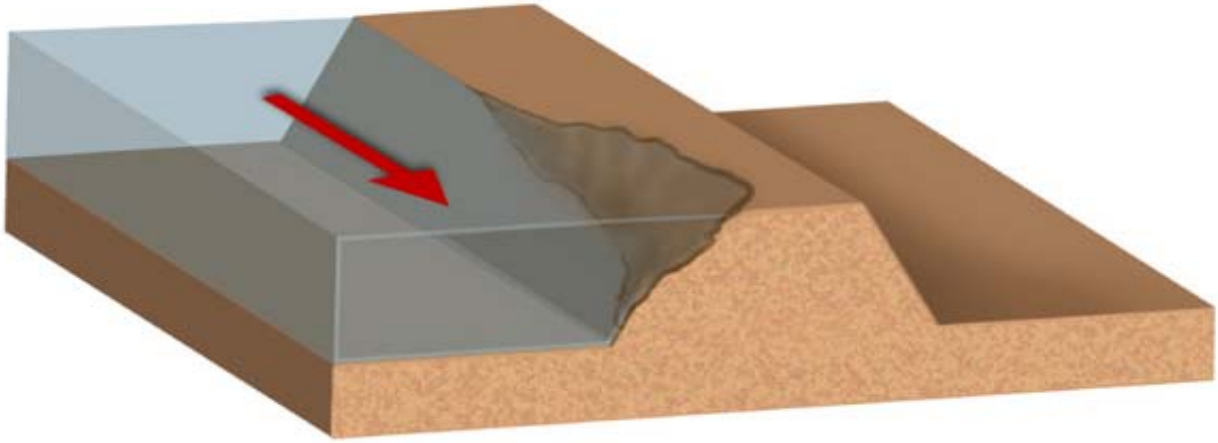


Erosion by water after overtopping

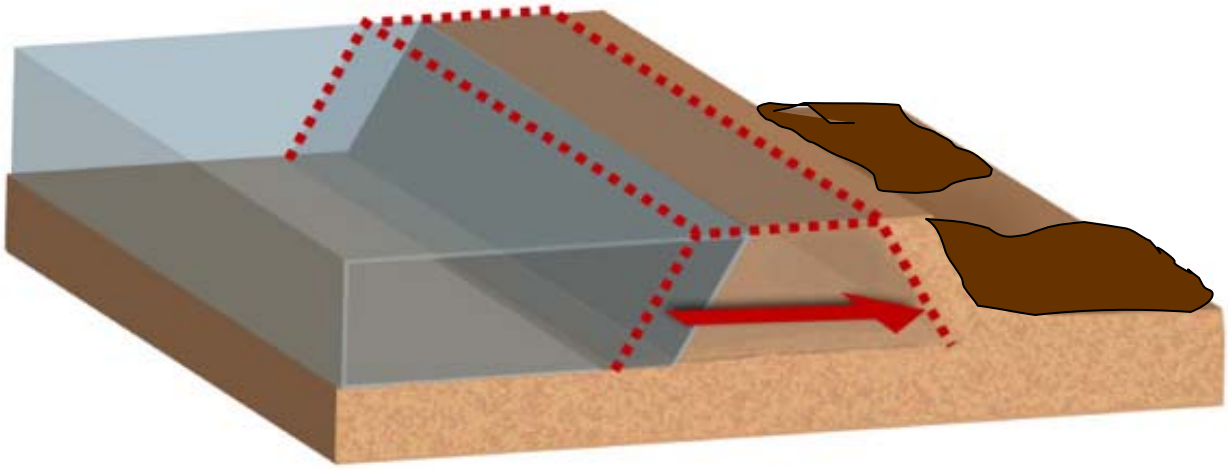
Figure 9a: Levee failure modes (Deretsky, National Science Foundation 2008).



Erosion due to water coming through the levee

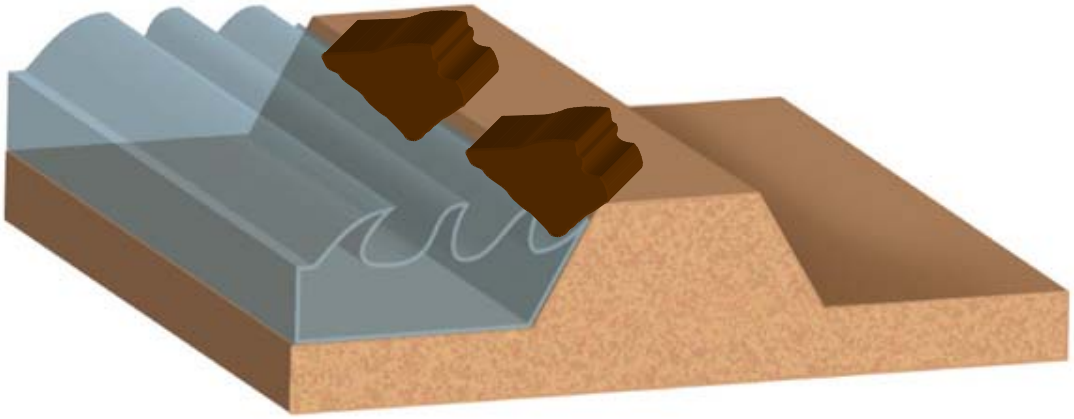


Current erosion on outboard levee and berm surfaces

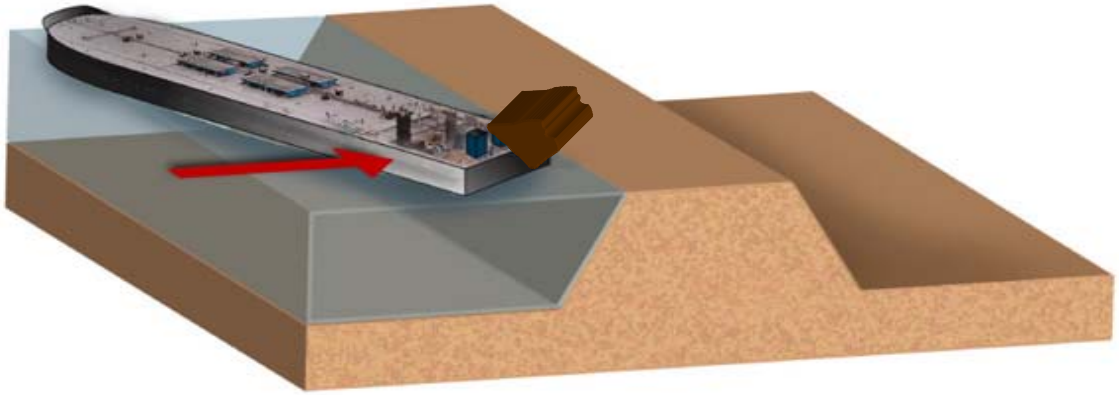


Lateral instability of the levee due to water force

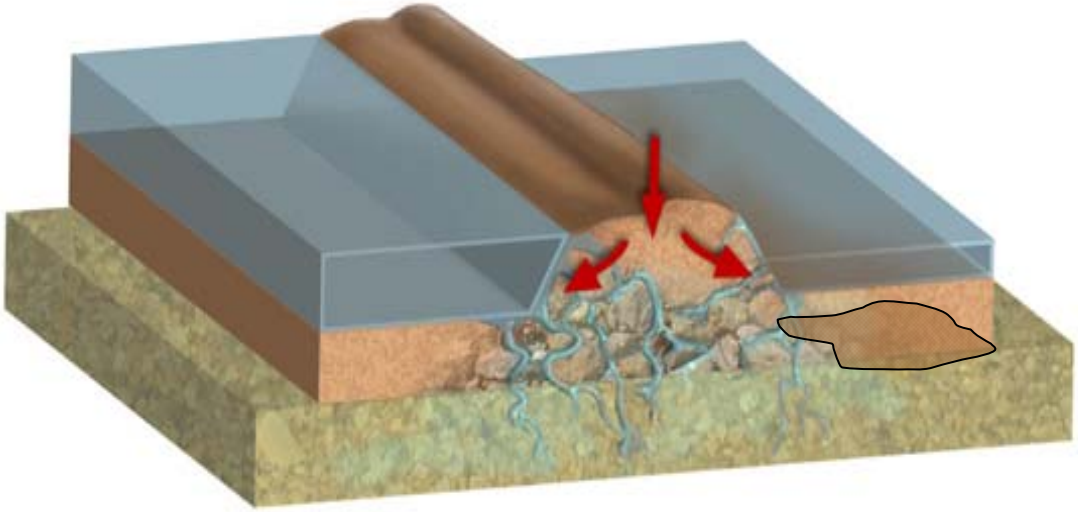
Figure 9b: Levee failure modes (Deretsky, National Science Foundation 2008).



Levee erosion due to wave impacts

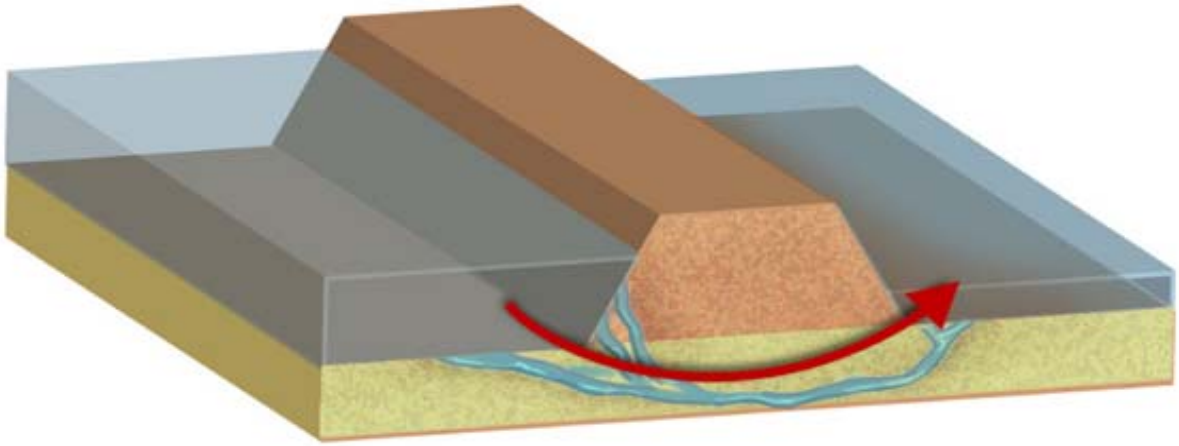


Increased lateral forces and local damage due to mpact from floating objects and debris

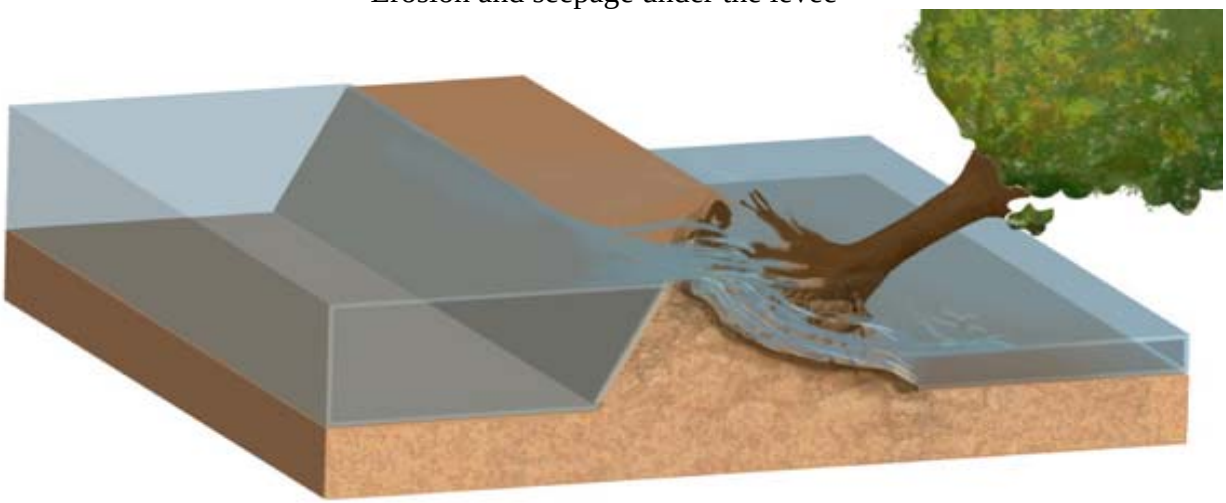


Soil liquefaction due to repeated cyclic forces from waves and currents

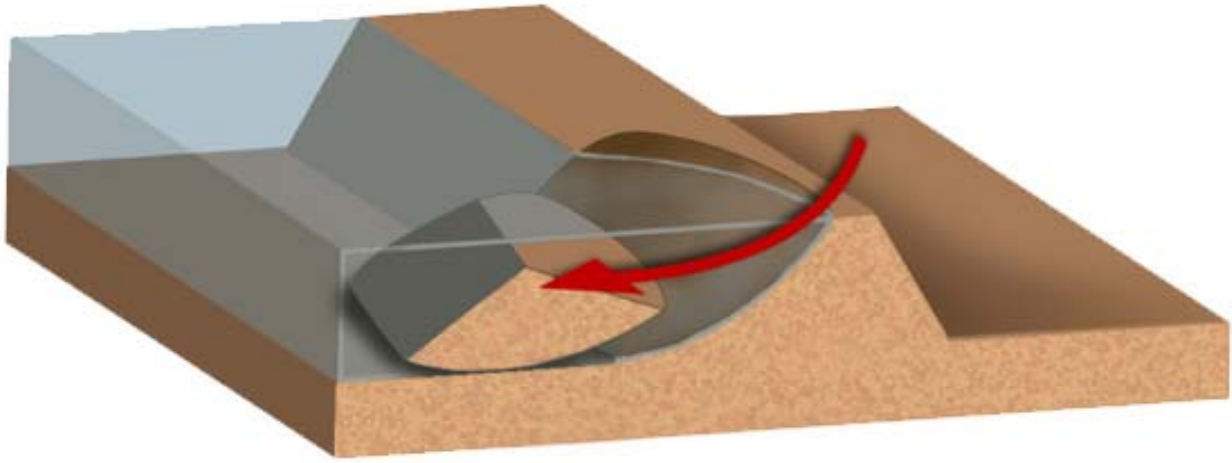
Figure 9c: Levee failure modes (Deretsky, National Science Foundation 2008).



Erosion and seepage under the levee



Damage by trees pushed over by water and wind



Lateral instability of sections of the levee due to effects of water on soils

Figure 9d: Levee failure modes (Deretsky, National Science Foundation 2008).

32. Photographic evidence and post failure investigations indicate the North Breach was a short-narrow movement that apparently started under the landside toe and progressed towards the waterside. All of this happened before this section of the wall was overtopped. The concrete I-wall failed and the sheet pile was elongated landward. The movement and resting place of the sheet pile indicates that the supporting levee and foundation materials were washed away beneath the sheet pile and the water force pushed away the steel sheet piles and twisted them until a section of the sheet piles rotated 90 degrees – against the rising surge waters in the IHNC. Figure 10a shows the failed levee - floodwall at the North Breach.



Figure 10a: North Breach. Note the sheet pile – concrete I-wall connection after rotating (ILIT 2006)

33. Field observations indicate the South Breach was a translational-relatively deep stability failure. It is longer than the North Breach, with approximately 900 feet of failed embankment, whereas the North Breach was approximately 250 feet long. Evidence from available reports and photographs shows that the I-wall lost lateral support when a scour trench opened on the landside crown and subsequently a gap opened on the waterside allowing water pressures to load the sheet pile and trigger a non-circular failure. Figure 10b shows the post-failure configuration of the South Breach.



Figure 10b: South Breach at Lower 9th Ward (ILIT 2006). Note the sheet pile displaced landward without flipping as at the North Breach.

Background

34. The levees protecting the Lower Ninth Ward were relatively small earthen embankments with slopes inclined 3 feet horizontal to 1 foot vertical (USACE DM03, 1966). Design cross sections from the 1966 Design Memorandum show that in the proximity of the landside toe of the levee a drainage channel known as the Jourdan Avenue Canal existed and was bottomed at elevation -10 feet to –12 feet NAVD88 (Figure 11). Toe elevations along the landside (protected side) were approximately –4 feet NAVD88. The landside canal was filled sometime in the 1990's with unknown materials (ILIT, 2006).

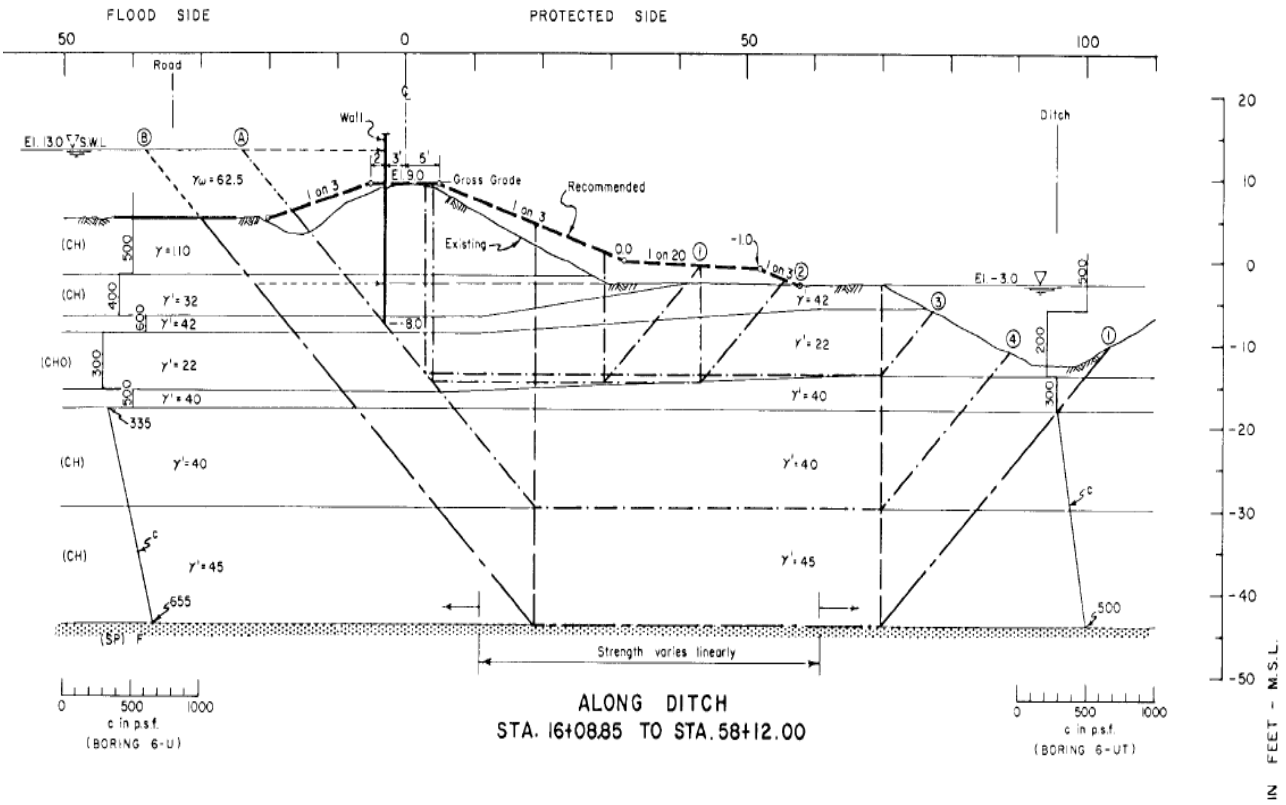


Figure 11: Analysis cross-section for the levee stretch south of Florida Avenue (USACE DM 03 1966).

35. Based on the design drawings (Figure 12) the design sheet piles were topped at elevation +14.8 feet NAVD88, and tipped at elevation –8 feet. Elevation survey data provided by IPET (2007) indicate that the top of the sheet pile supported I-wall was located at elevation

+12 feet (NAVD88), and the sheet piles tipped at -10.5 feet. This indicates a 2.5 feet elevation difference between the design and pre-Katrina conditions.

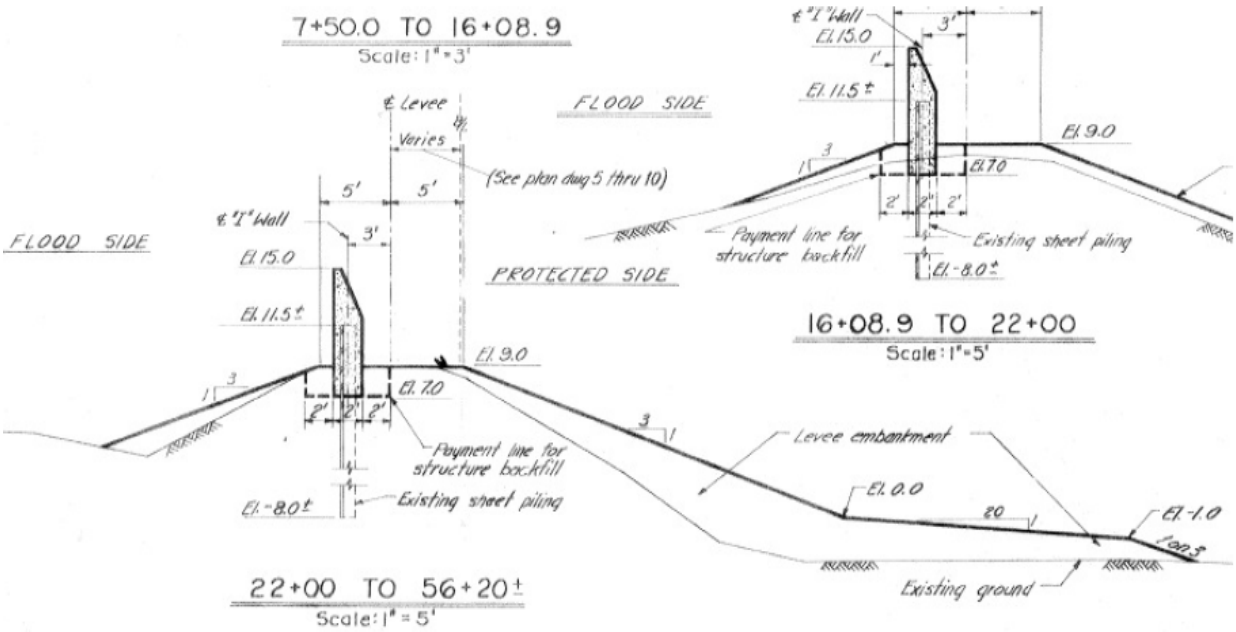


Figure 12: I-wall as-built drawings for Lower 9th Ward south of Florida Avenue (USACE DM 03 1966).

36. Figure 13 shows the elevation difference between landside toe elevations from LIDAR surveys performed in 2000, showing an elevation difference of approximately 4 feet between the North Breach and the South Breach. As shown in Figure 14, There were substantial differences between the design levee profiles and those actually present.

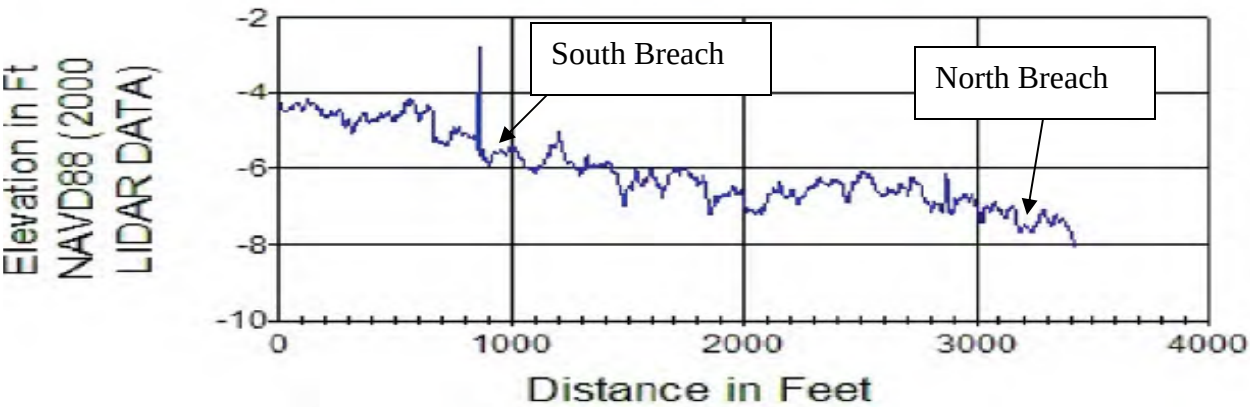


Figure 13: LiDAR survey along landside toe (USACE IPET 2007).

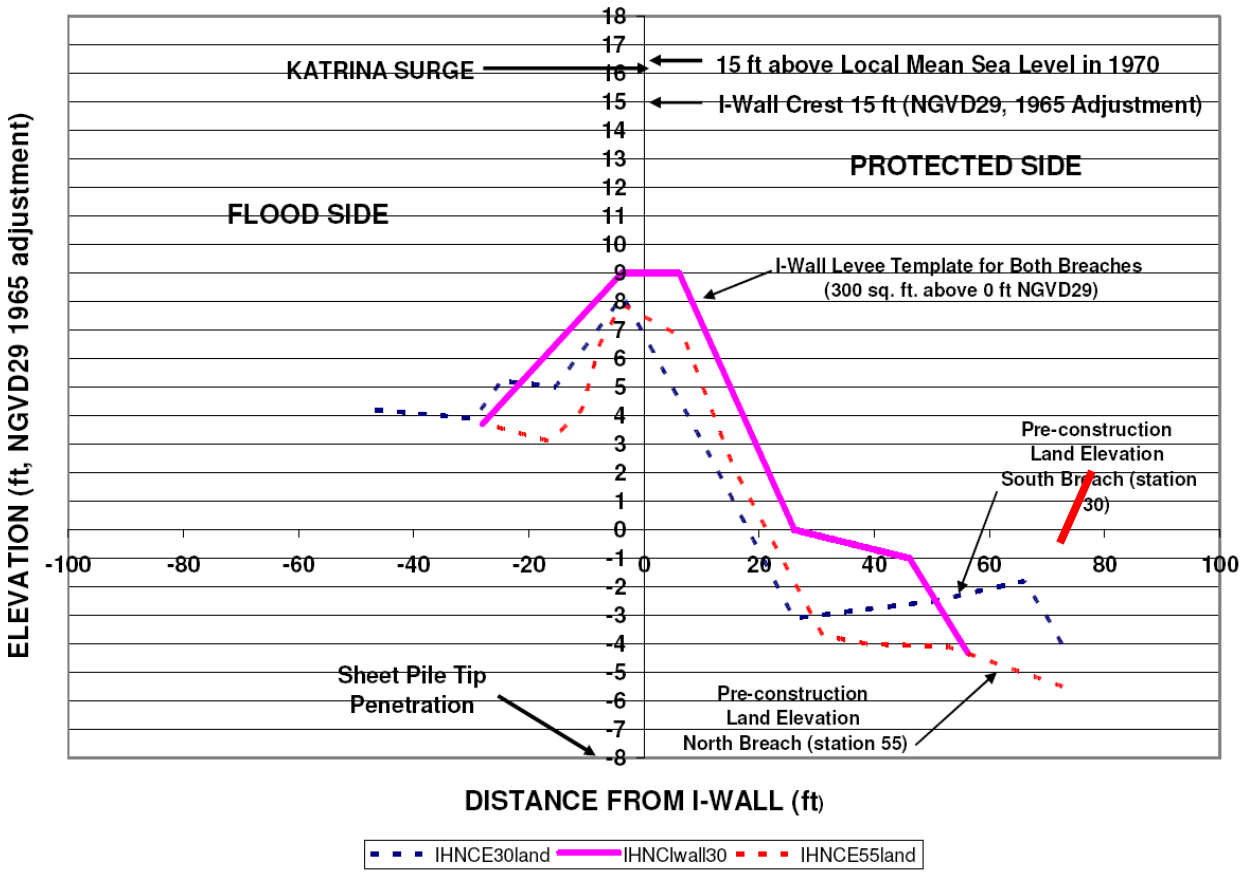


Figure 14: Comparison of the as-built I-wall levee profile with pre-construction surveys at south and North Breaches (Team Louisiana, 2007)

37. The levee embankments are underlain by a thick irregular Marsh (Peat) deposit (Figures 15 - 16). Field evidence and analysis results from the ILIT investigation (ILIT 2006) indicated this layer had important influences on the development, shape and extents of both the North and South Breaches.

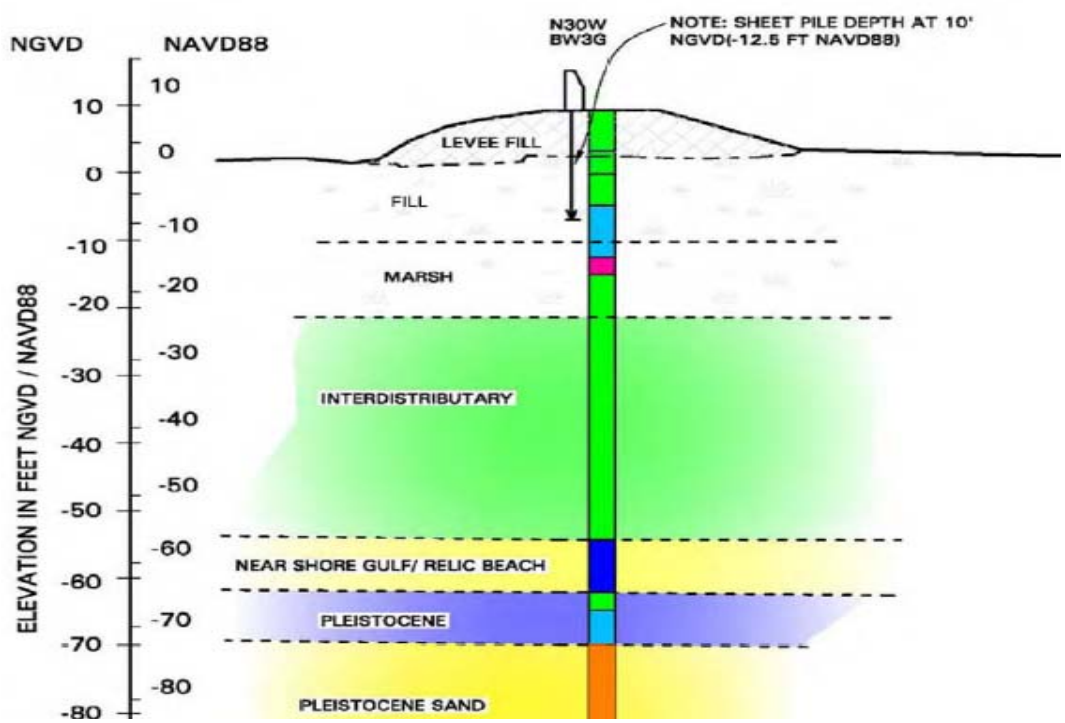


Figure 15a: North Breach geologic cross section (IPET 2006).

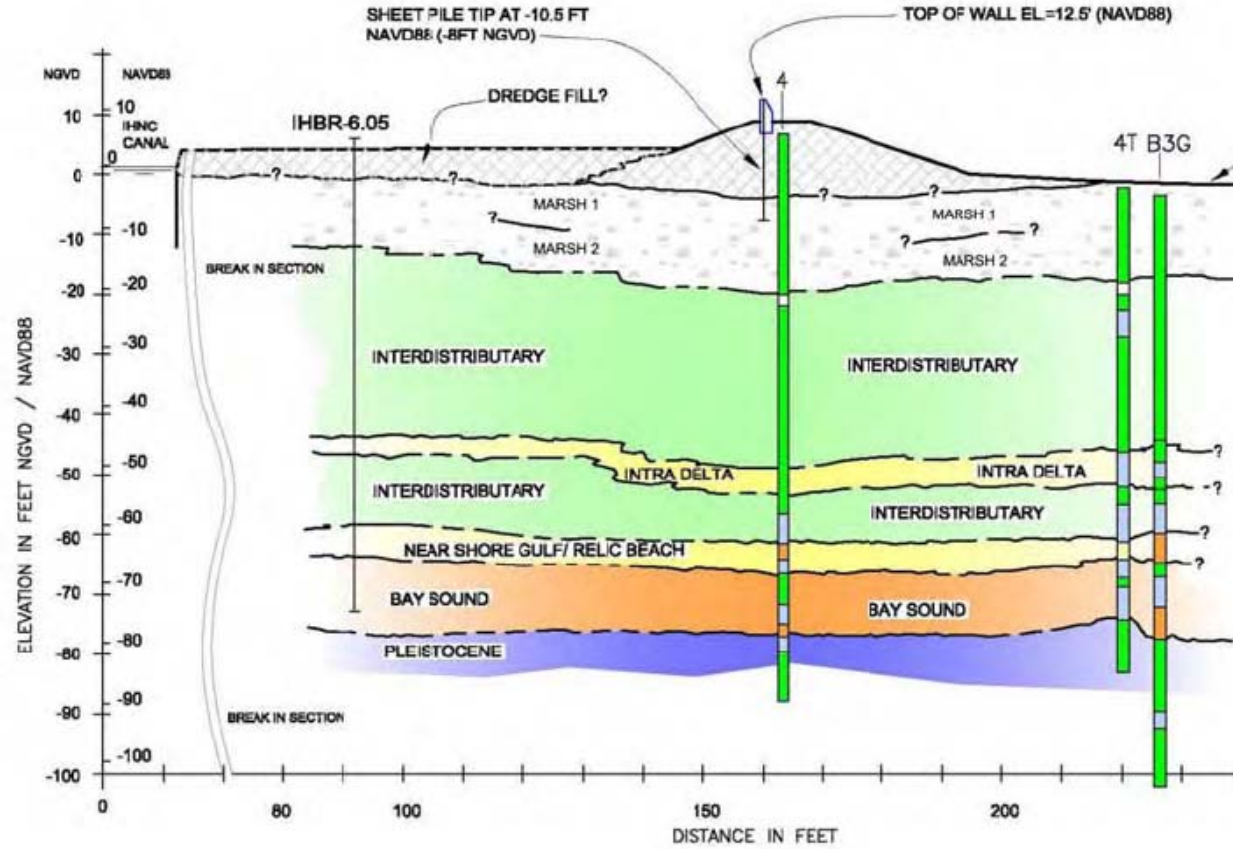


Figure 15b: South Breach geologic cross section (IPET 2006).

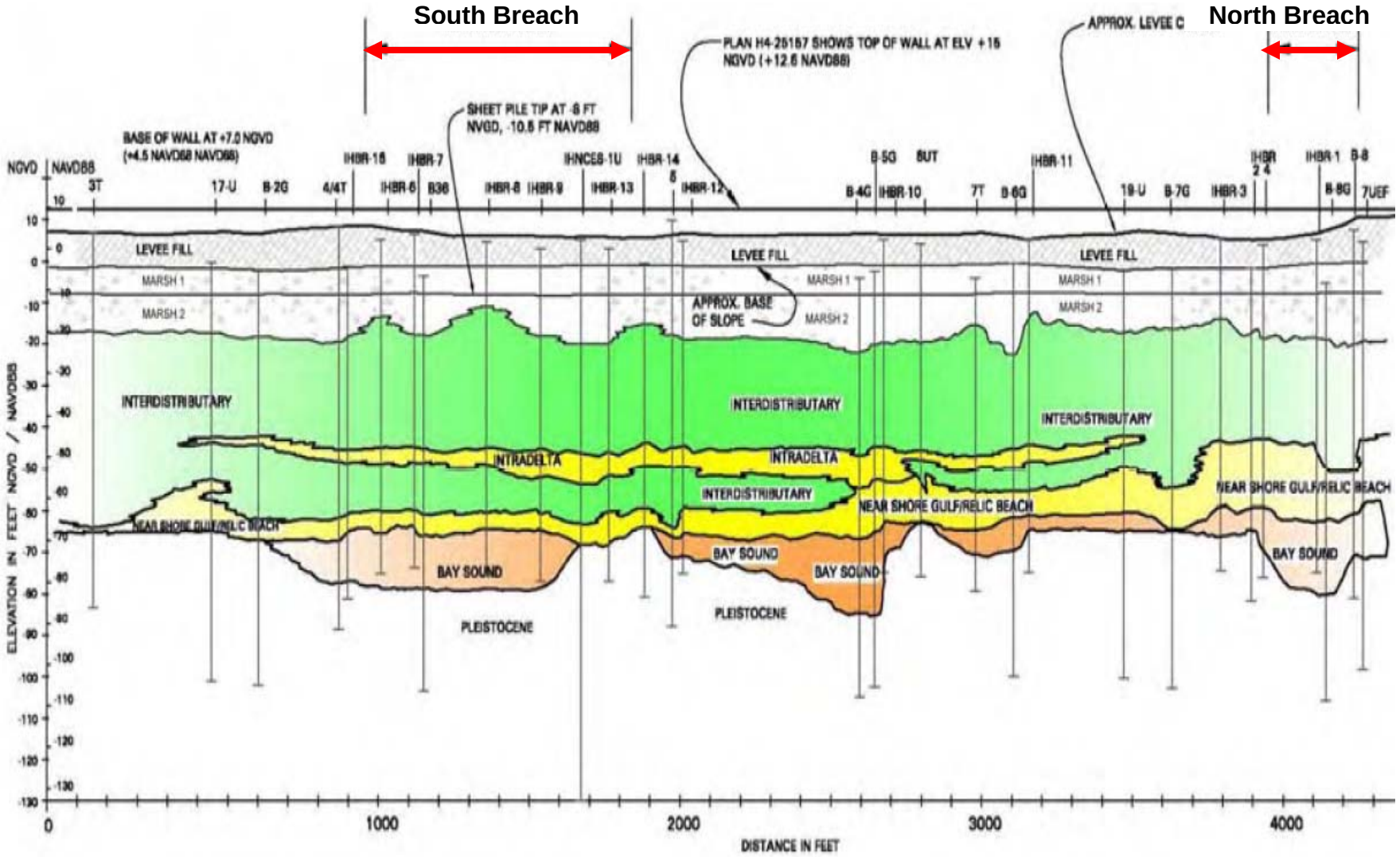


Figure 16: East bank of IHNC geologic section through South Breach and North Breach sites (USACE IPET 2007)

38. During the 1990s, work was commissioned by the USACE to expand the IHNC locks to relieve shipping congestion and allow further expansion of the Port of New Orleans. Congress authorized the study for the lock enlargement / replacement in 1956. The proposed new locks would require a bypass channel that would be constructed at the East Bank Industrial Area (EBIA). The EBIA was located in the Lower 9th Ward, west of the floodwall and east of the IHNC, between Florida Avenue and Claiborne Avenue (Figure 17).



Figure 17: IHNC and EBIA (left side) (USACE photograph)

39. Available documentation indicates that the USACE purchased the EBIA site from the Port of New Orleans in order to proceed with the bypass channel construction (WGI, "Project Work Plan, Project Site Development and Remedial Action of East Bank Industrial Area, Inner Harbor Navigation Canal Lock Replacement Project," report to USACE, New Orleans District, 2000). WGI began working at the job site in January 2001.

40. The EBIA consists of 32 acres of former industrial sites previously leased from the Port of New Orleans to private owners. The EBIA consisted of six facilities named for their

former occupants (Figure 18). These included (starting at the south) the International Tank Terminal, Saucer Marine, Mayer Yacht – Distributors Oil, Indian Towing, McDonough Marine, and Boland Marine. Available information indicates that the two Lower 9th Ward breach sites were located entirely in the Saucer Marine Service property (South Breach site, used primarily for ship building operations) and Boland Marine property (North Breach site, used primarily for ship repairs, storage, office space, painting operations, and steel fabrication).

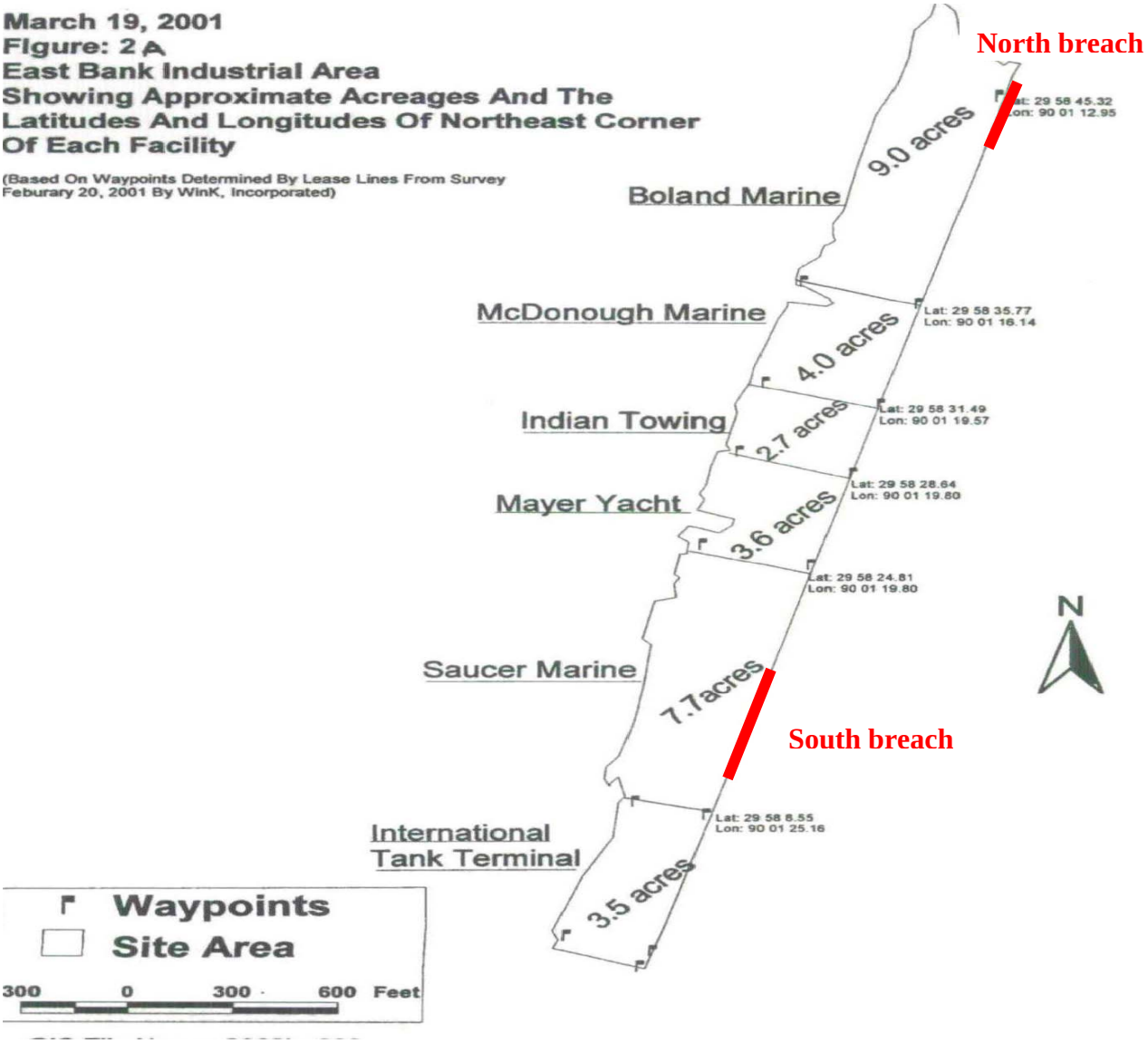


Figure 18: East Bank Industrial Area (EBIA) (WGI 2005). Site cleared to prepare for USACE Lock Expansion Project. The Boland Marine parcel is opposite the North Breach. The Saucer Marine parcel is opposite the South Breach.

41. At one time, underground storage tanks were located at both the Boland property and the Saucer property. These tanks were reported to have been removed. The State of Louisiana listed Boland Marine as an unspecified hazardous waste generator. In 1991, an anonymous employee contacted the Louisiana Department of Environmental Quality and reported that the company had buried drums containing hazardous waste onsite.

42. The EBIA had a long history spanning more than 4 decades of industrial use by marine service and petroleum distribution companies. Purportedly, tenants contaminated the property as a result of their industrial operations. Also of particular significance at the EBIA was the active placement of construction materials (concrete rubble) and solid wastes (barges, metal turnings, concrete blocks) to protect the properties from the effects of ship wakes. These two activities had two primary impacts derived from the existence of hazardous constituents that could negatively affect: 1) the environment at the time of construction, and 2) construction of the bypass channel.

43. To ameliorate these potential negative effects, the USACE New Orleans District contracted with WGI to characterize and remediate the EBIA's environmental contamination and identify and remove construction materials likely to interfere with bypass channel construction. Figures 19a and 19b summarize the site clearing activities at the north Boland site (adjacent to the North Breach) and south Saucier Marine site (adjacent to the South Breach). Note the very large number of surface and subsurface facilities that had to be removed. Even more facilities were discovered underground and removed during the site clearing operations (WGI 2005).

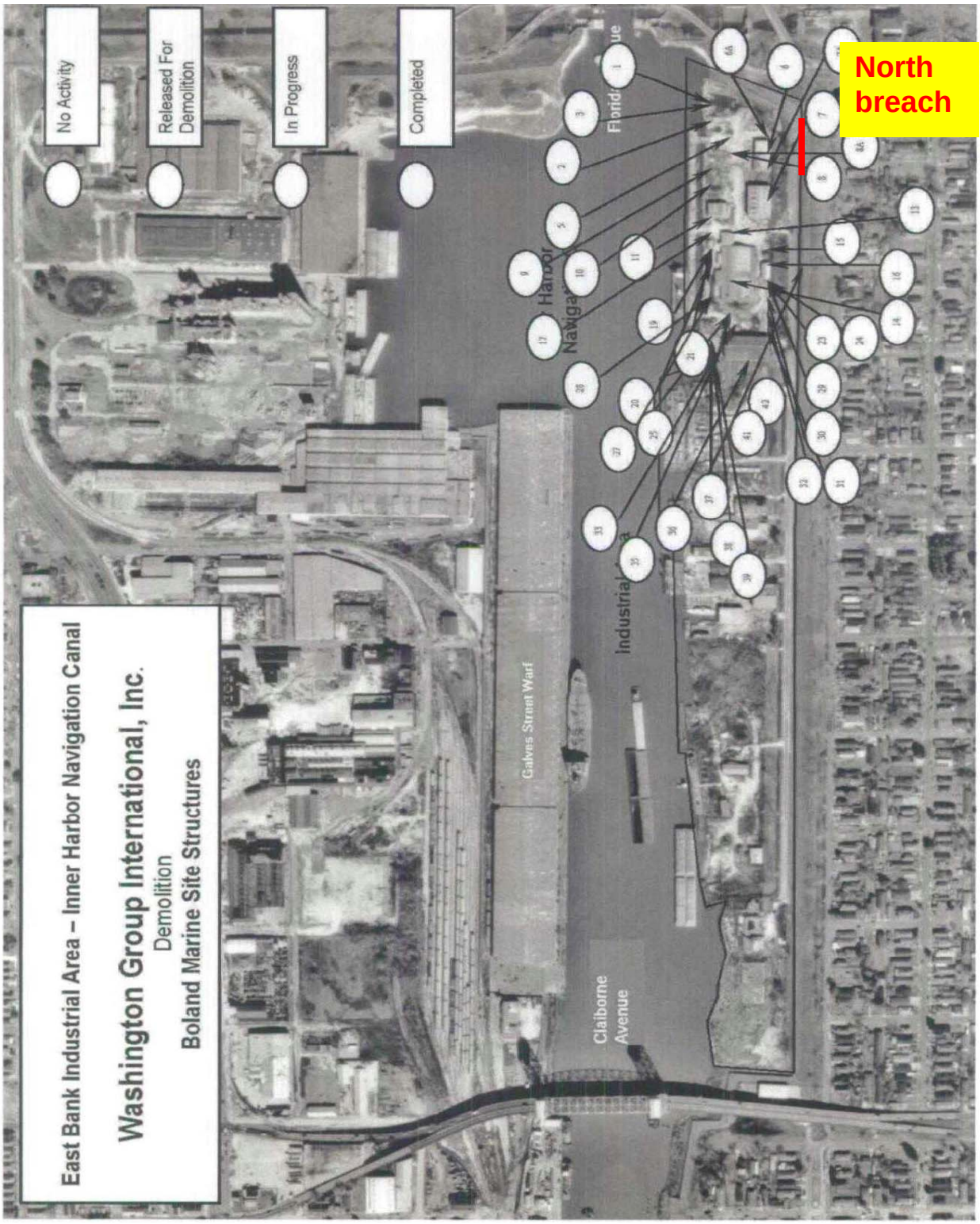


Figure 19a: Boland Marine site structure removals and excavations at North end of EBIA – adjacent to North Breach (WGI 2005).

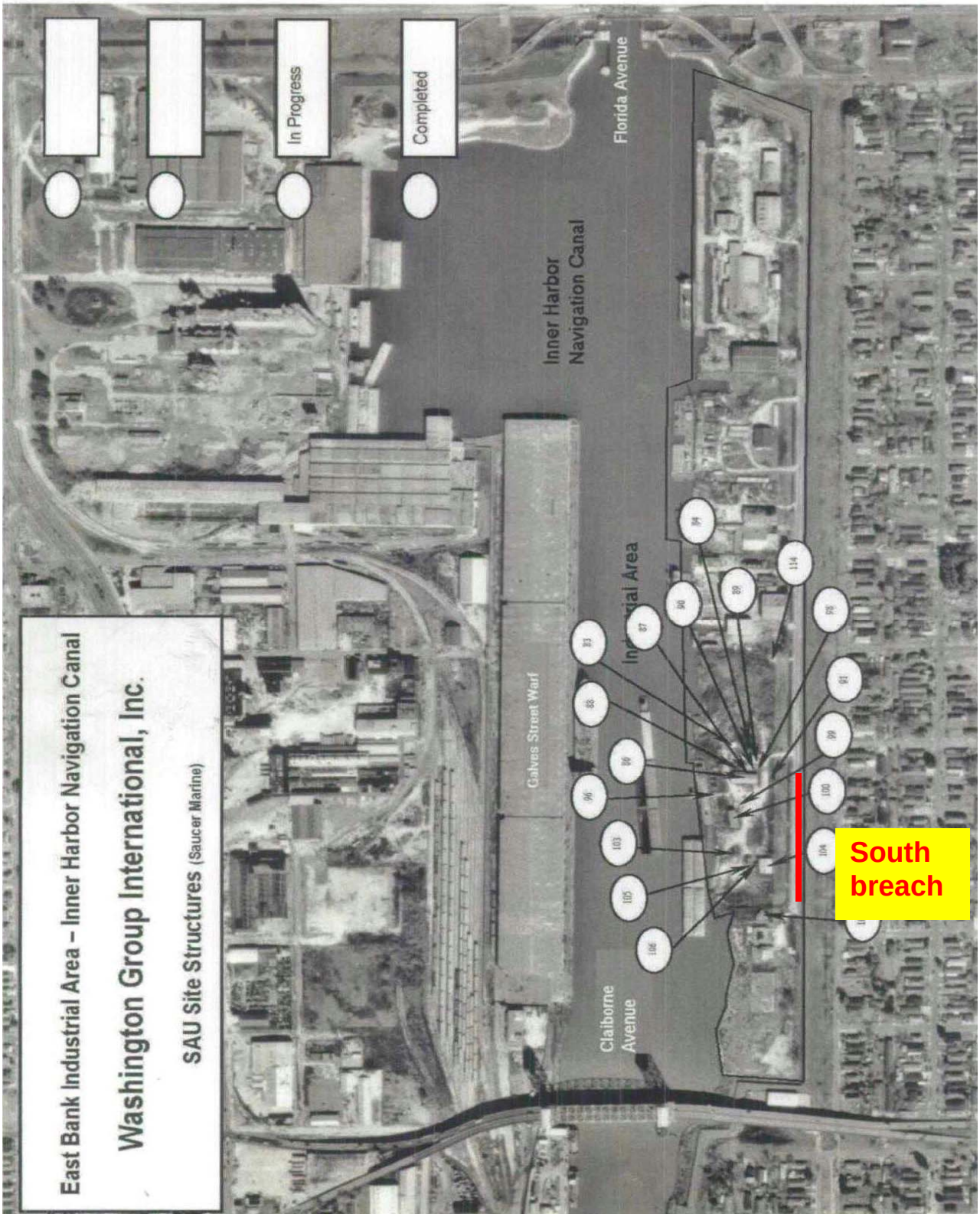


Figure 19b: Saucier site structure removals and excavations at South end of EBIA adjacent to South Breach (WGI 2005).

44. For remediation, the specifications required the canal bank be excavated to a depth four feet below the existing grade and 15 feet from the edge of the water onto land. When possible, the work was done during low tide to facilitate visual observation of the work. When work was completed, the landside of these excavations became the 'new' shoreline. In one case, a very extensive borrow pit was excavated midway along the EBIA to provide backfill materials for some of the excavations (Figure 20). The east side of this borrow pit was about 30 feet from the floodwall. Because of concerns for seepage under the floodwall, clay was placed on the eastern wall of the borrow pit (Figure 21). After removal of the borrow materials, the pit was flooded with water from the IHNC bringing the IHNC waters to the toe of the levee (Figure 22).



222110904-05 Clay wedge placed in Area 1 excavation to rebuild small section of original levee removed dur remediation

Figure 20: Excavations for soil borrow pit adjacent to flood wall (WGI 2005).



Figure 21: Excavations for soil borrow pit adjacent to flood wall (WGI 2005). Excavation was subsequently flooded with water from IHNC.



Figure 22: Soil borrow pit breached bringing IHNC waters to within 100 feet of the floodwall (WGI 2005).

45. It is reported by WGI ("Technical Completion Report," 2005) that in the majority of cases the canal bank was excavated as required to a depth of four feet, additional observations of contamination prompted excavations to depths exceeding nine feet. The records provided by WGI indicate that a total of 12,259 tons of contaminated soils were removed from the Boland site and 35,636 tons of contaminated soils were removed from the Saucer site.

46. In addition to removal of canal-side soils that had previously provided stability and protection to the flood protection structure, numerous underground storage tanks and debris were removed. At the Saucer site, large underground storage tanks had to be removed. In addition, a buried railroad tank car required construction of a cofferdam so that the tank car could be filled with air, floated free, lifted and transported from the site (Figures 23 - 25). At the northern Boland site, a wharf extended along most of the IHNC. Following removal of the wharf materials, the piles comprising the foundation were pulled and transported from the site. Excavations to remove contaminated soils were made immediately adjacent to the floodwall at the Boland Marine Site (Figures 26 - 27).

47. All of these excavations encountered severe water intrusion difficulties. Constant attention had to be paid to dewatering the excavations. Water intrusion was most severe at depths below the ground in the range of –15 feet to as deep as –30 feet. The greatest water intrusion was associated with excavations that encountered significant layers of buried organic materials. The experience with seepage and water intrusion associated with the deep EBIA excavations and those associated with another similar site to the north clearly indicate the pervasive nature of this problem. This experience clearly refutes the IPET conclusions (Volume V, V-127, IPET 2007): *"While these values of permeability would indicate seepage, IPET has found no justification for*

these assumptions and no physical evidence of seepage being a systemic performance issue at this site."



Figure 23: Cofferdam at Boland Marine site dug to depth exceeding 25 feet (WGI 2005).



Figure 24: Cofferdam at Boland Marine site dug to depth exceeding 20 feet. Water entry was associated with penetration of organic layers encountered at – 16 feet (WGI 2005).



104040402-10 BOL – Lifting Tanker From Inside The Cofferdam – Stewart Construction

Figure 25: Removing buried railroad tank car inside coffer dam at Boland Marine site (WGI 2005).



Figure 26: Excavations to remove contaminated soils at Boland Marine site immediately adjacent to floodwall (WGI 2005).



Figure 27: Excavations to remove contaminated soils at Boland Marine site immediately adjacent to floodwall (WGI 2005).

48. Fill used to backfill the excavations ranged from imported sand (Figures 28 and 29) to ‘re-worked’ fill materials from the EBIA.



Figure 28: Backfilling excavations with imported sand at Boland Marine site (WGI 2005).



Figure 29: Backfilling excavations with imported sand at Boland Marine site (WGI 2005).

49. The excavations consistently encountered a "buried swamp" at an elevation between approximately –15 feet and –25 feet (MSL) (Figures 30 - 32).



Figure 30: Buried cypress tree stumps encountered in borrow pit at McDonough Marine (WGI 2005). Note proximity of floodwall in background.



Figure 31: Cypress swamp vegetation encountered in borrow pit (WGI 2005).



Figure 32: Cypress swamp tree trunk removed from borrow pit (WGI 2005).

50. The effects of these site remediation activities clearly extended well below the specified soil excavation levels cited earlier. Removal of barges, piles, underground storage tanks, and underground utilities (e.g. sewer, water, gas lines) could easily have reached to and substantially exceeded elevations of minus 10 feet to 25 feet; thus intersecting the underlying marsh deposits (Figures 20 - 27) and providing. When backfilled with porous – pervious – materials, hydraulic connections would be developed between the backfilled excavations and the buried pervious – permeable marsh and swamp layers. This potential was particularly evident in the case of the sewer lift station and sewer line removed at the Saucier Marine site (Figures 33 - 35). The backfilled sewer lift station excavation was within approximately 100 feet of the floodwall. The sewer line crossed the floodwall at right angles and was located near the center of the South Breach.



Figure 33: Sewer lift station removal cofferdam located outside of floodwall at the Saucier Marine site (WGI 2005). Note floodwall in background. An underground sewer main crosses under the floodwall at right angles and was subsequently removed.



Figure 34: Sewer lift station being removed. Note water charging developing in the bottom of the excavation indicating active hydraulic connectivity and conductivity (WGI 2005).



Figure 35: Sewer lift station excavation being backfilled. After backfilling, the cofferdam sheet piles were pulled (WGI 2005).

51. WGI completed its work, removed its equipment, and demobilized from the job site in May 2005 (WGI 2005).

52. The WGI experiences associated with the EBIA site clearing excavations (seepage, high permeability organic layers, soil heaving) were corroborated during a construction project conducted to the north of the Lower 9th Ward at Dwyer Road (Rosenberg 2008). Excavations conducted at this location (2000-2003) encountered similar high permeability organic layers at comparable depths encountered by WGI at the EBIA. These high permeability organic layers were associated with significant and persistent water seepage that had not been anticipated by the USACE and their consultants at the time that the background engineering work was done for the Dwyer Road construction. The contractor repeatedly encountered severe

seepage and soil heaving problems that threatened the stability of the earthworks and that had to be solved 'on the job.'

53. The information that is available clearly indicates a high degree of correlation of the site remediation work conducted by WGI at the EBIA sites and the breach locations in the flood protection structure adjacent to the Lower 9th Ward. This remediation work (removal of soils, formation of cavities, vibrations) very probably had important effects on the soils relied upon to provide stability and protection for the flood protection structure. Removal and disturbance of the surface soils overlying the permeable subsoils (Marsh and Swamp layers) would have facilitated the under seepage.

54. Information regarding waterside excavations indicate that work near the area of the breaches placed permeable material down to elevations of approximately –25 feet, approximately 15 feet deeper than the sheet pile tips; thus producing significantly shorter paths for hydraulic pressures to reach beneath the embankment soils and levee toe, starting material saturation and softening potentially faster, as opposed to the subsoils being hydraulically charged from more than two hundred feet away (e.g. charged from the IHNC canal). Figures 36a through 36d show an evolution of the excavation along the IHNC waterside, and how alarmingly close the IHNC canal was moved to the floodwall early in 2005. Figures 36a through 36d shows multiple excavations located from 10 to 100 feet from the I-walls.



Figure 36a: East bank of IHNC as of October 18, 1996 (Bea and Cobos-Roa 2008).

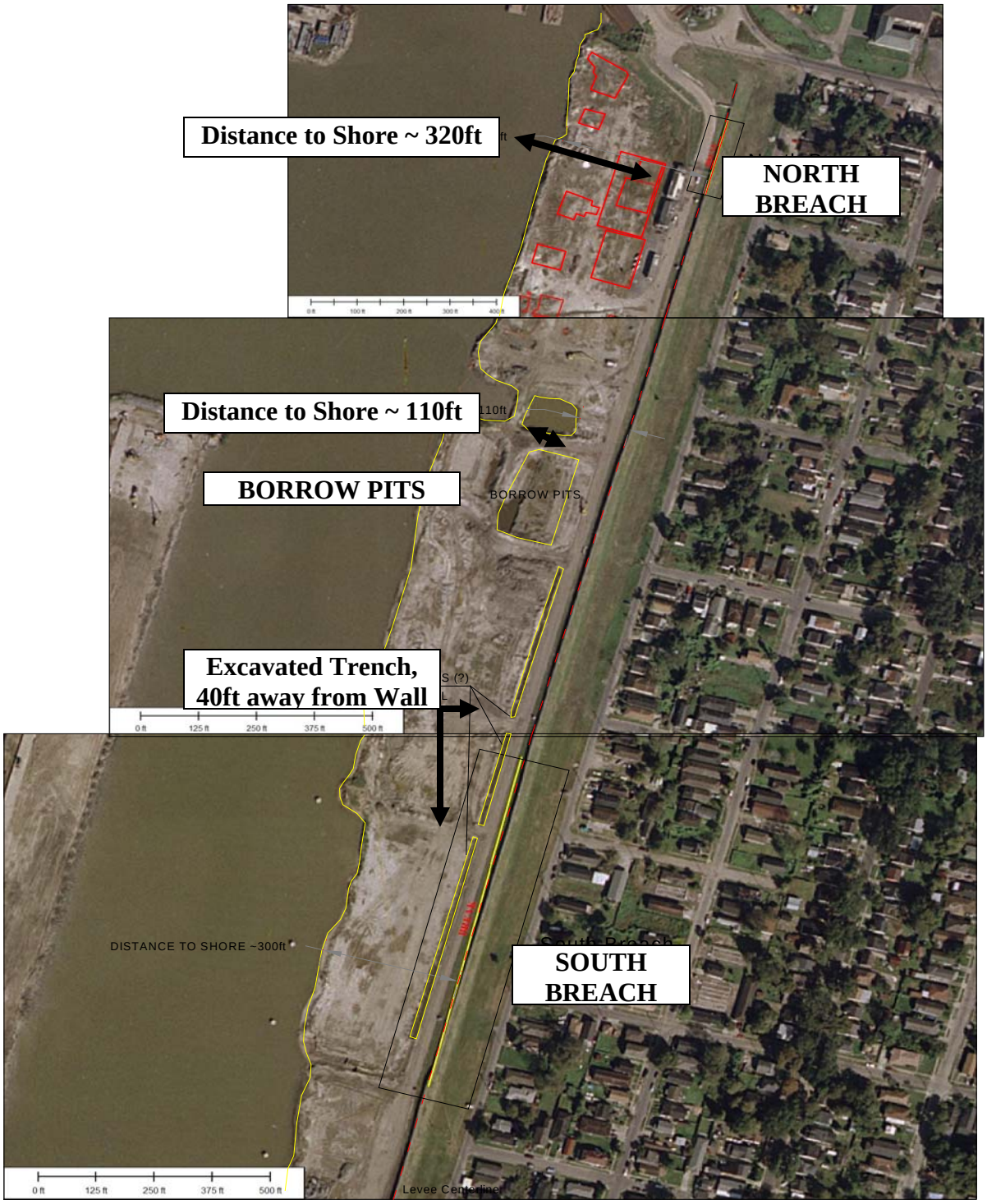


Figure 36b: East bank of IHNC as of November 13, 2002 (Bea and Cobos-Roa 2008).

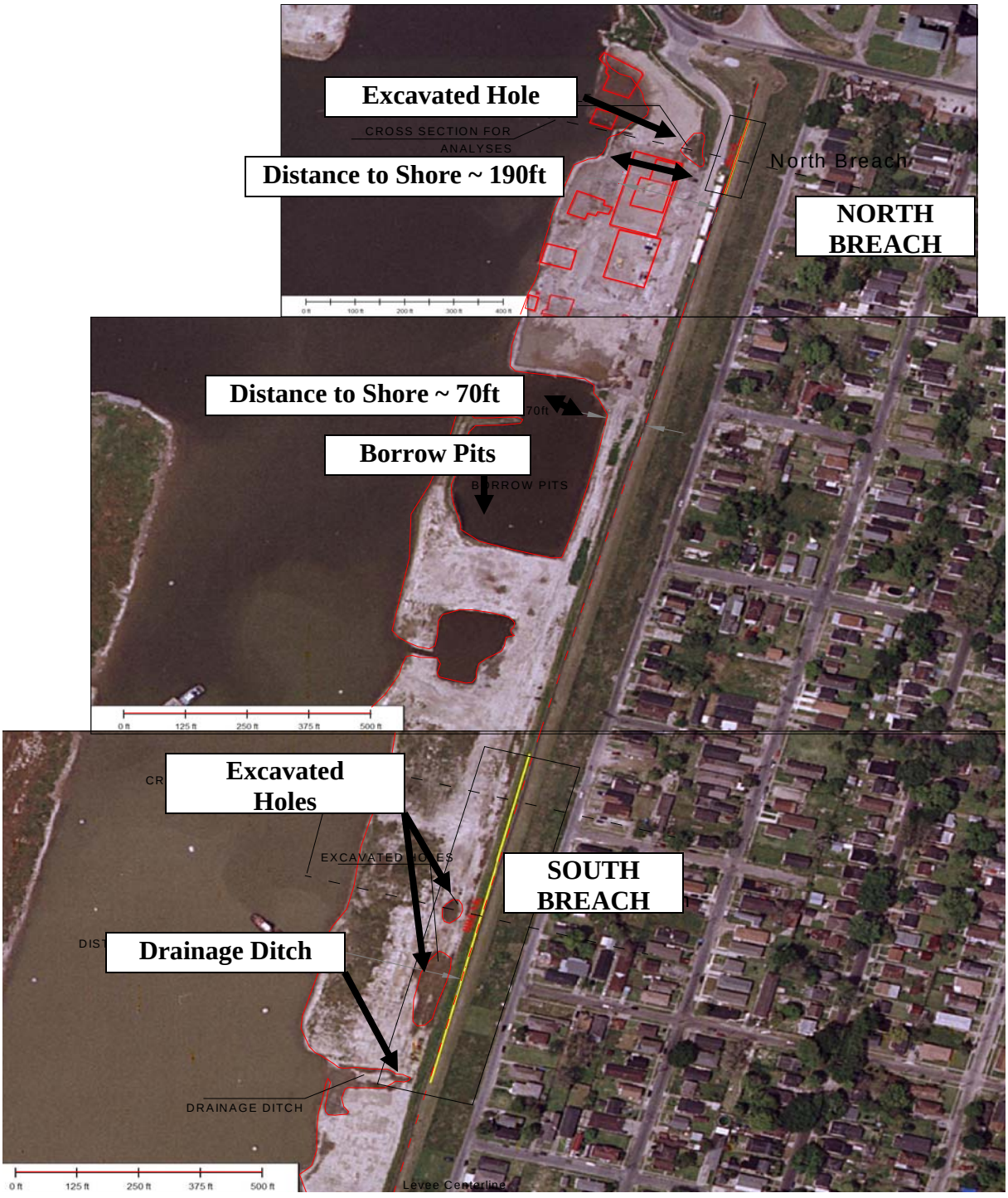


Figure 36c: East bank of IHNC as of April 13, 2005 (Bea and Cobos-Roa 2008).

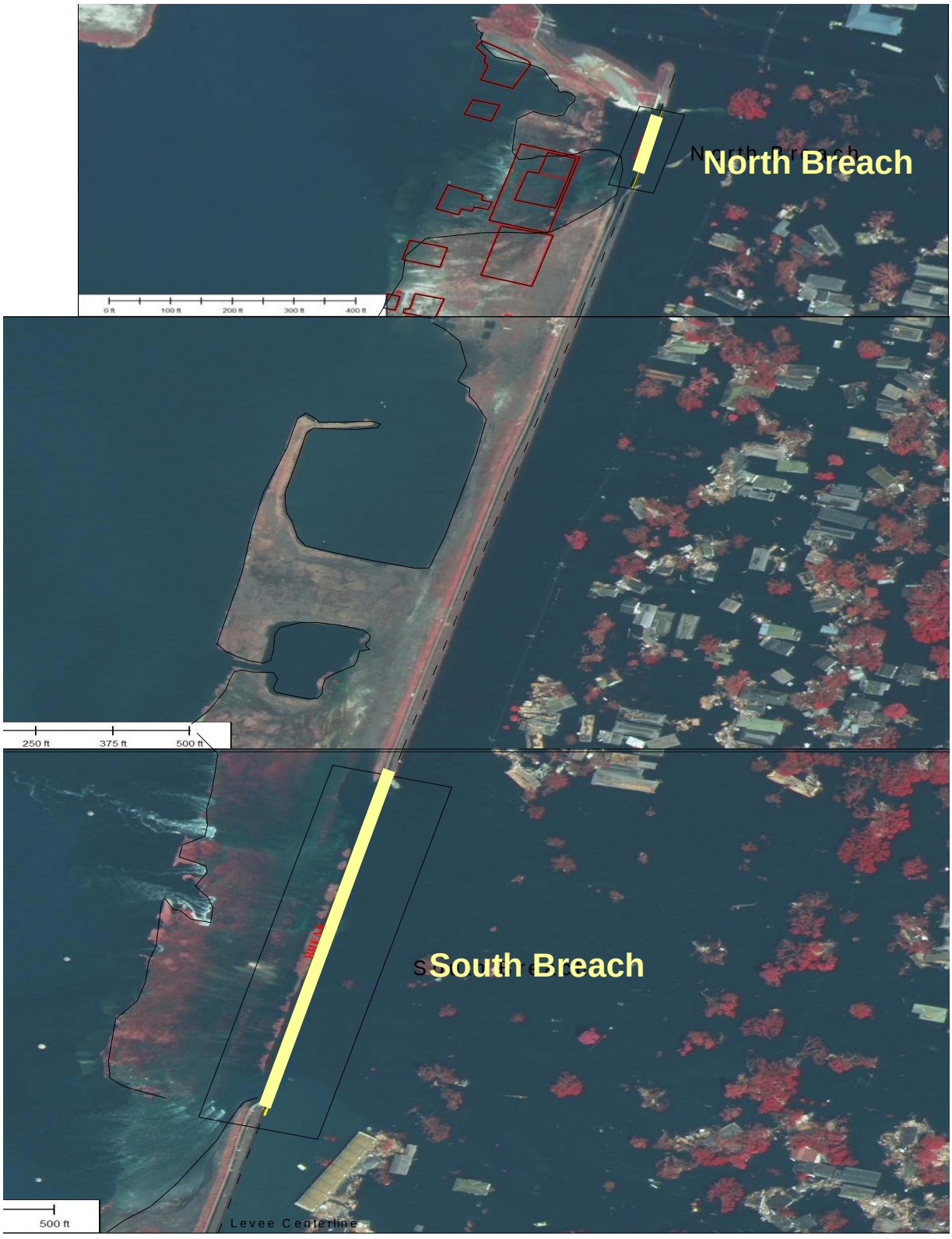


Figure 36d: East bank of IHNC as of September 2, 2005 (Bea and Cobos-Roa 2008).

55. The USACE has specific guidelines for work near or within federally constructed flood control projects (USACE 2005, Bea and Storesund 2008). These guidelines (USACE *Guidance for Work Proposed Near or Within a Federally Constructed Flood Control Projects,* 2005) stipulate (paragraphs 1. and 2., page General -1): *"The critical area is generally considered the area from 300 feet riverward to 500 feet landward of a flood control project centerline. In some instances the critical area is extended beyond 500 feet if any impact on the flood control project can be considered." "The Corps of Engineers provides engineering review to ensure that any work within or near the flood control unit does not reduce the level of protection and to assure the continued integrity of the flood control system."* Further, these guidelines stipulate (paragraphs 1 and 4.2., page Excavation and Backfill –1 and 2): *"Excavation and backfill in the critical area of a flood control project could have a direct impact on the stability of the flood control project. Improper excavation in the levee embankment or in the critical area can create unstable slopes and slope failures, which damage the levee embankment and weaken the flood protection structure. Additional damages to the levee embankment or flood protection structure can also be caused by underseepage conditions associated with piping and heave. This removal of material from the flood control structure foundation (piping) can ultimately produce collapse of a levee embankment or floodwall. All excavation and backfill activities within the critical area are to be reviewed and approved by the COE and the local sponsor before construction begins." "Care shall be taken to replace the natural foundation stratification and embankment zoning. <u>In lieu of selective stockpiling and zoned backfill, impervious materials from an approved borrow may be used</u>."* (underline added for emphasis). Thus the USACE had specific guidelines regarding excavations and backfill intended to prevent what developed during the EBIA navigation lock expansion site clearing operations.

56. The report by Bea and Storesund (2008) provides a detailed discussion of applicable USACE guidelines regarding excavations, backfilling and seepage – hydraulic connectivity considerations associated with adjacent flood protection structures. This detailed literature search and review did not disclose any specific procedures or processes implemented by the New Orleans District of the USACE to address potentially damaging effects of the excavations to remove underground facilities in the immediate vicinity of the flood protection structure protecting the Lower 9th Ward. It is evident that the primary concerns were focused on the IHNC lock expansion project motivated removal of potential above and below ground obstructions and on amelioration of the heavily polluted soils underlying the EBIA sites. It is also evident that there were no significant investigations or analyses regarding the potential damaging effects of the EBIA excavations and backfilling on the integrity of the man-made flood protection structures immediately adjacent to these excavations.

57. After a detailed review of all available borings and cone penetrometer test (CPT) probes (31 total), the locations of soil explorations were geo-referenced using a GIS interface overlaying aerial photographs of the area (Bea and Cobos-Roa 2008). From the pre- and post-Katrina LiDAR surveys and the USACE DM03 ground elevations, a longitudinal profile was drawn, locating the borings and CPTs. To include the explorations into this report, the 1966 DM03 borings had to be "measured" from the image versions of this report, as well as the laboratory testing results. The 2005 IPET borings are available in the IPET website (www.ipet.wes.army.mil).

58. Based on the available soil boring and CPT data and geologic maps of the area, the stratigraphic interpretation clearly shows that the embankment fills that compose the levees are mainly composed by clays (CL, CH), and an irregular Marsh (peat,swamp) layer underlies

the levee. The Marsh layer appears to be continuous along the entire IHNC east bank and it is thickest near the area surrounding the South Breach. Horizontal lines representing approximate elevations of I-wall crest, sheet pile tip, levee crest and levee toe are included in the longitudinal profile. From the longitudinal profile, two cross sections (perpendicular to the longitudinal alignment) were created. The first one is located in the area along the South Breach, and the second one in the North Breach. The borings in the vicinity of the section were located at their appropriate elevations and another interpretation (more detailed) was performed at the particular section. These sections were chosen for the seepage and stability analyses.

North Breach Seepage & Stability Analyses

59. Based on available soil boring and CPT data, historic aerial photographs, and previous geologic studies performed in this are, Figures 37 - 39 shows the interpreted cross sections through the North Breach. The North Breach is located between stations 55+00 and 58+00 of the original DM03 levee stationing. The first thing to note in this section is the elevation of the landside toe, which is located between elevations –7 feet and –8 feet; being approximately 10 feet lower than the waterside ground elevation.

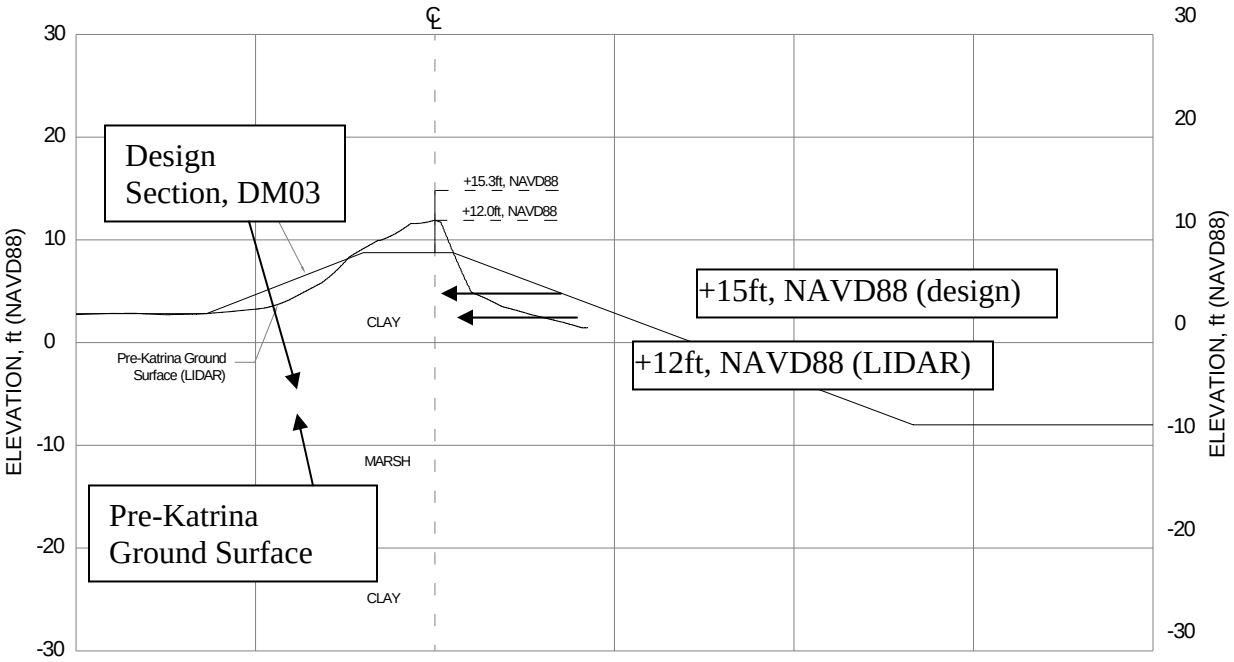


Figure 37: North section comparing design geometry from DM03 and pre-Katrina section from LIDAR survey performed in 2000 (Bea and Cobos-Roa 2008).

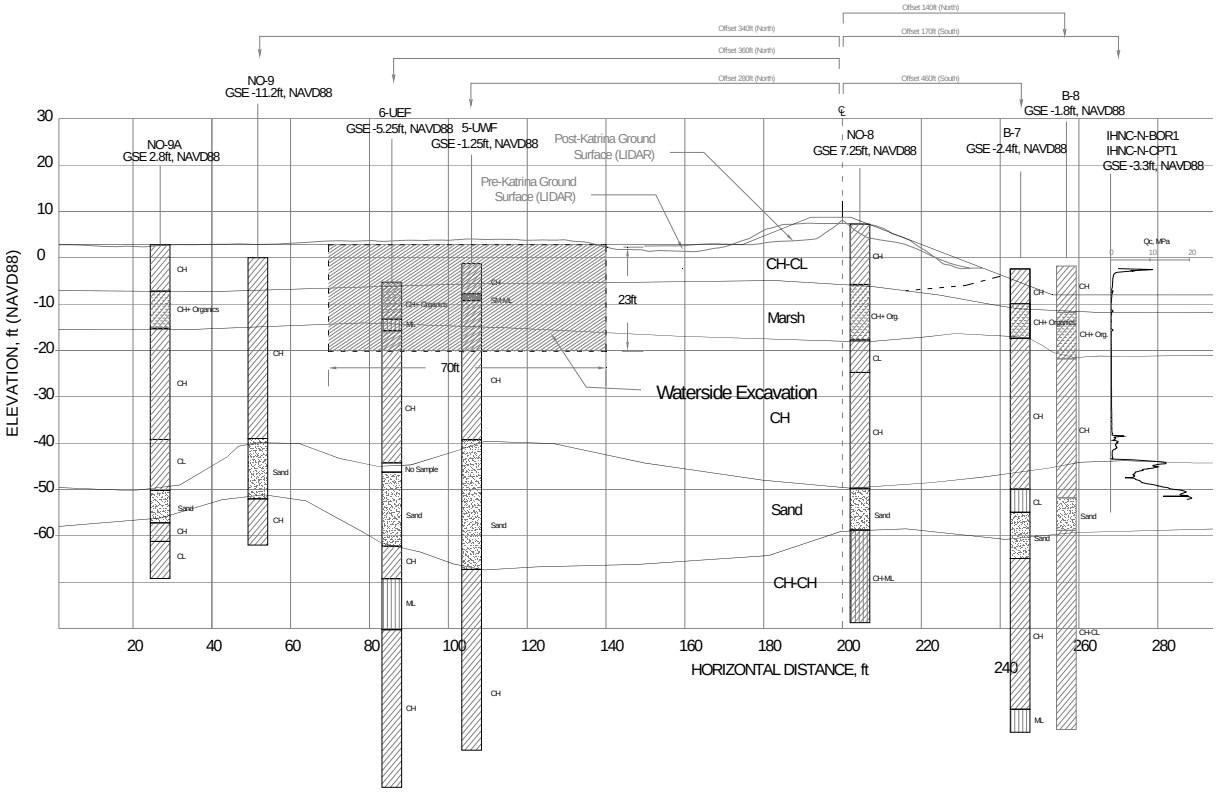


Figure 38. Cross section through North Breach (Bea and Cobos-Roa 2008).

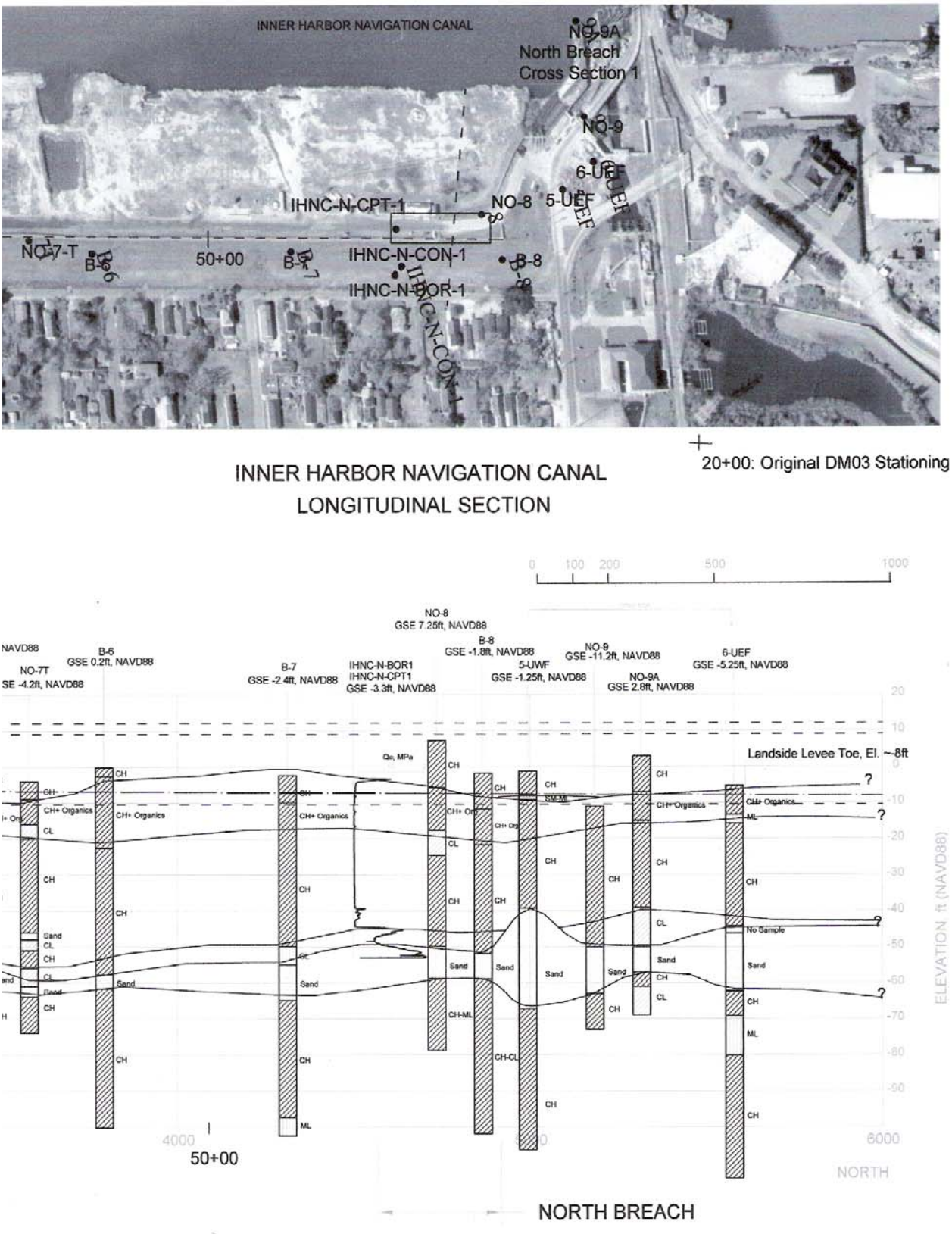


Figure 39: Longitudinal Profile along IHNC, north breach (Bea and Cobos-Roa 2008).

60. The stratigraphic characterization shown in Figure 39 was based on borings NO-8 (1966), 6UEF and 5UWF (1971), B-7, B-8 IHNC-N-CPT-1 and IHNC-N-BOR-1 (2006). The levee embankment is composed mainly by compacted clays, underlain by a Marsh deposit, described as organic matter, organic clay, wood, roots and saturated-very low density material. A thin clay blanket seems to be located at the toe of the levee and towards the waterside surface. Given the poor descriptions presented in the borings from DM03 and the IPET (2006) report, samples containing organic materials with high water contents, low density and low strengths were classified as Marsh samples.

61. Underlying the marsh deposit, a thick "fat" clay deposit occurs from elevation -20 to approximately –50 feet, located atop a sand deposit. The sands are in turn underlain by another clay and silt deposit, probably associated with Pleistocene age deposits.

62. Based on the aerial photography (Figures 33 – 36), the excavated hole on the IHNC side of the North Breach was located approximately 60 feet away from the centerline of the levee - floodwall. Based on available photographic evidence, the hole was determined to be approximately 23 feet deep and 70 feet wide. The walls of the excavation were assumed vertical since the wall slopes do not have any significant influence on the analytical results. Based on the available photographic documentation, the material used to backfill these holes was characterized as coarse-grained materials (sand). The sheet pile extends from elevation +12.0 feet down to elevation -10.5 feet (NAVD 88). It is evident that the sheet pile penetration is not sufficient to completely cut the Marsh layer.

63. The software SEEP\W, part of the package GEOSTUDIO 2004, was used for analyses of seepage and hydraulic effects. The program uses a two-dimensional finite element code to route the boundary conditions imposed as hydraulic heads on the boundaries of the

problem through a porous media, calculating heads and pressures along the nodes of the finite element mesh. The software requires the generation of permeability functions that include the decrease of permeability depending upon increases in negative pore pressures (suction).

64. Figure 40 shows a typical mesh used in the SEEP\W finite element analyses. The measured Katrina surge hydrograph in Figure 3 was used in these analyses. Table 1 summarizes the hydraulic properties of the soil layers, including conductivity at saturation and anisotropy ratio (ratio of vertical to horizontal permeability, ky / kx). Because of the lack of reliable laboratory tests or field pumping tests performed by the USACE on the buried marsh – swamp layers that underlie the Lower 9th Ward, a plausible range of hydraulic conductivities – intrinsic permeabilities were analyzed. See Appendix A and the supporting Technical Report by Bea and Cobos-Roa (2008) for a discussion of the background for the ranges of permeability used in these analyses. The last row in Table 2 gives the permeability value used for gap formed between the levee and sheet-pile. This element was not modeled in the IPET (2007) seepage computations.

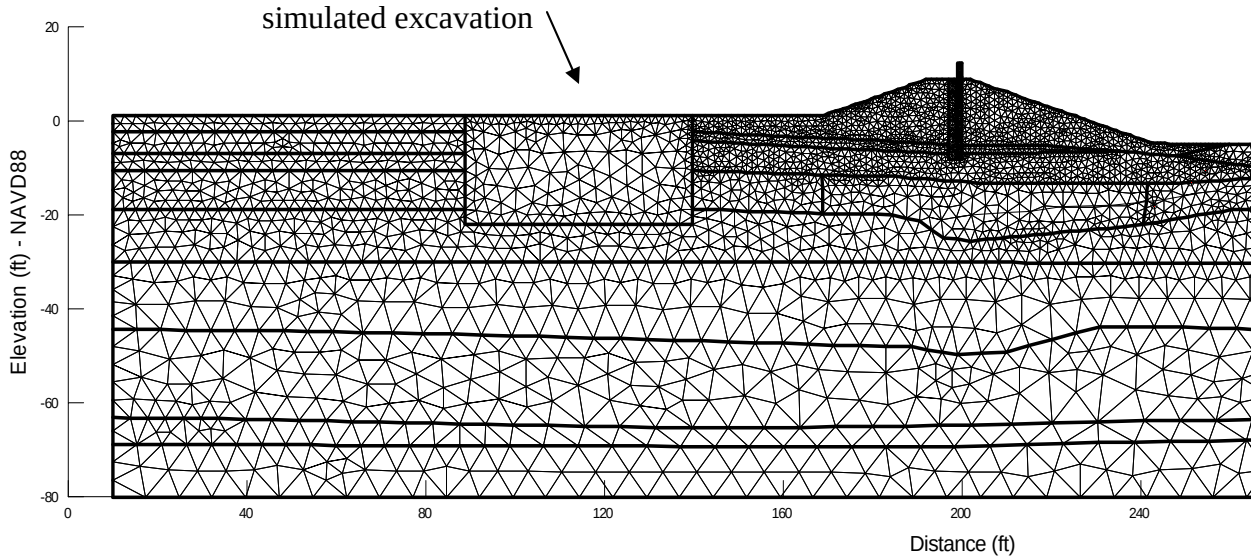


Figure 40: Typical finite element mesh for seepage analyses (Bea and Cobos-Roa 2008).

Table 2: Free-field material permeabilities used in seepage analyses.

Material	**K at saturation (cm/s)**	**Anisotropy Ratio (ky/kx)**
Clays	10^{-6}	0.3
Sands	10^{-2}	0.25
Marsh (upper bound)	10^{-2}	0.10
Marsh (lower bound)	10^{-6}	0.10
Gap	100	1.0
Waterside Excavation Fills	10^{-1}	1.0

65. Based on results from transient seepage analyses, Figure 41 shows the evolution of pore water pressure at three different locations at the toe of the levee. After 6:00 am (CDT) on August 29, 2005, the calculated pore pressures are almost equal to the values of overburden stress acting on the surficial clay layer. This would lead to the initiation of uplift deformation (or failure) near the toe of the levee (Figure 9d).

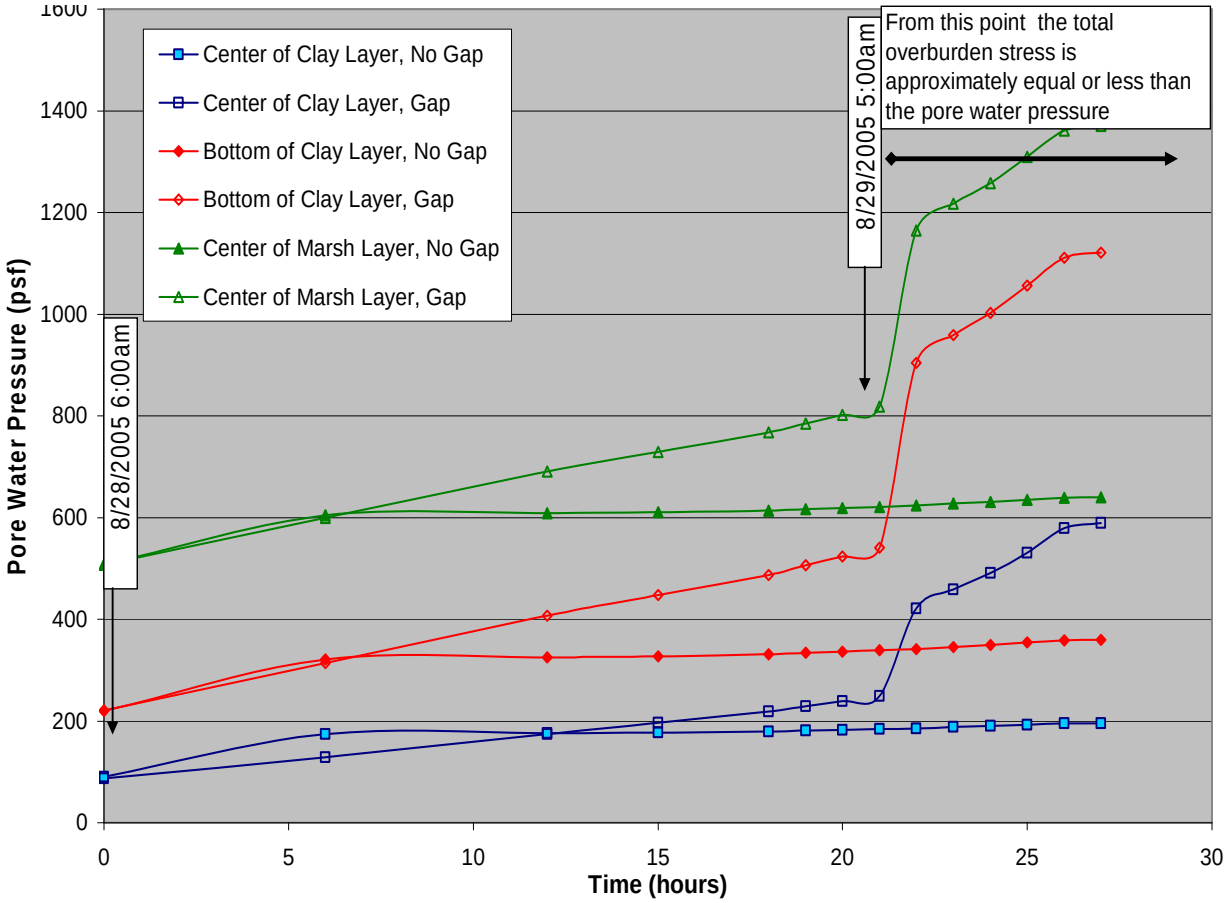


Figure 41. Transient seepage analyses results. Pore pressure versus time on three points near the toe of the levee (Bea and Cobos-Roa 2008).

66. As noted, these seepage – pore pressure – uplift pressure analyses employed a very wide range in hydraulic conductivity of the buried marsh – swamp layers – 10^{-2} cm/s to 10^{-6} cm/s – 4 orders of magnitude. As detailed in the supporting Technical Report by Bea and Cobos-Roa (2008), for this entire range of hydraulic conductivity, the transient water pressures developed at the time of breach development were able to attain within 70% to 80% of the steady state conditions. The results are relatively insensitive to a plausible range in hydraulic conductivity.

67. The SEEP\W analyses of seepage – hydraulic effects are based on two-dimensional finite element analyses. Exploratory studies of hydraulic effects associated with

excavations near levee toes comparing results from two-dimensional and three-dimensional analyses have been conducted (Money 2006). These studies addressed excavations near levee toes that are very comparable with those addressed during this investigation. This study shows that the two-dimensional analyses substantially underestimate the hydraulic effects compared with the three-dimensional analyses – in the range of 50% to 150% underestimates. Thus, the seepage – hydraulic pressure results associated with the EBIA excavations documented in this Declaration are expected to result in in underestimates of the hydraulic effects (pressures, gradients) on stability of the flood protection structures.

68. Figure 42 shows the evolution of hydraulic gradients at the same three points. An arrow points to the approximate location of 5:00 am on August 29, 2005. The data shows a rapid increase in hydraulic gradients after this time. Gradients in excess of unity indicate a high probability of 'blow-out' conditions that can lead to undermining and failure of the levee – floodwall system. These high flow gradients would lead to softening of materials at the toe of the levee that could progress into a slope failure. Such localized water flows together with associated movements of the supporting levee and floodwall – opening the vertical water stops – would explain early entry of water into this part of the Lower 9th Ward (Figure 18).

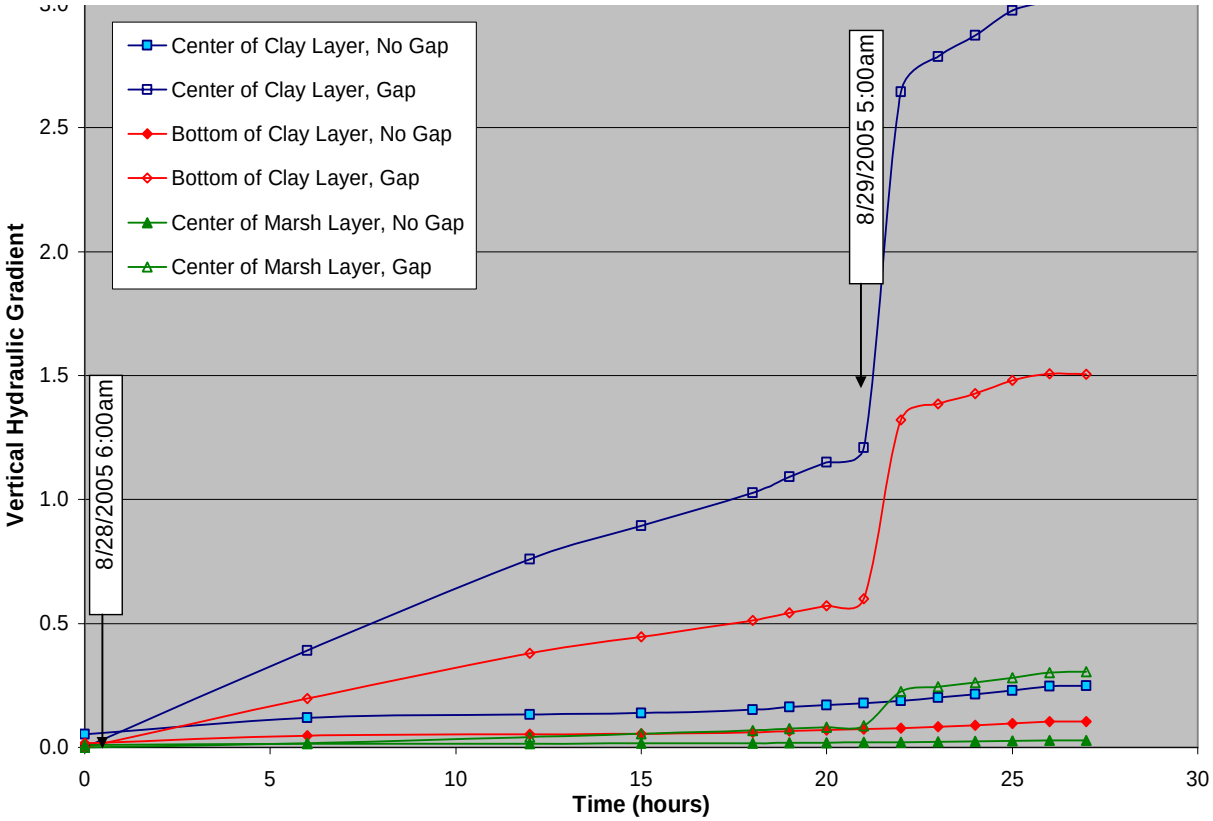


Figure 42: Hydraulic gradients versus time at three points near the toe of the levee at the North Breach. Marsh kx = 10^{-3} cm/s (Bea and Cobos-Roa 2008).

69. As shown in Figure 43, a significant head loss is experienced by the surficial clay on the landside of the North Breach; evidenced by the closely spaced head contours near the toe. This translates to the high gradients shown in Figure 44, where gradients in excess of unity are computed between the landside toe and the drainage canal structure, which acts as a more draining material.

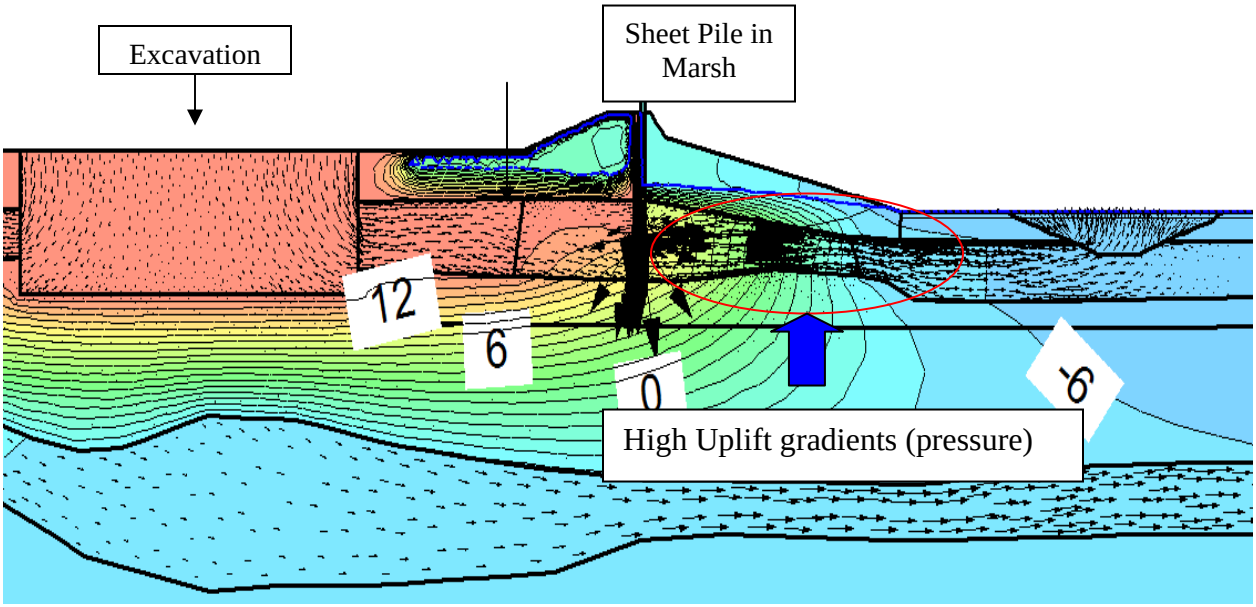


Figure 43: Transient model result, Total Heads (feet) (Bea and Cobos-Roa 2008). Time: between 7:00 and 8:00 am (CDT) on August 29, 2005. Note the communication of water from the excavation through and under the levee and sheet piling and exiting at the toe of the levee where the upward water pressures exceeds the downward soil pressures. These upward pressures act to reduce the effective weight of the soils on the protected side – thus reducing the lateral stability of the soil – levee – floodwall system.

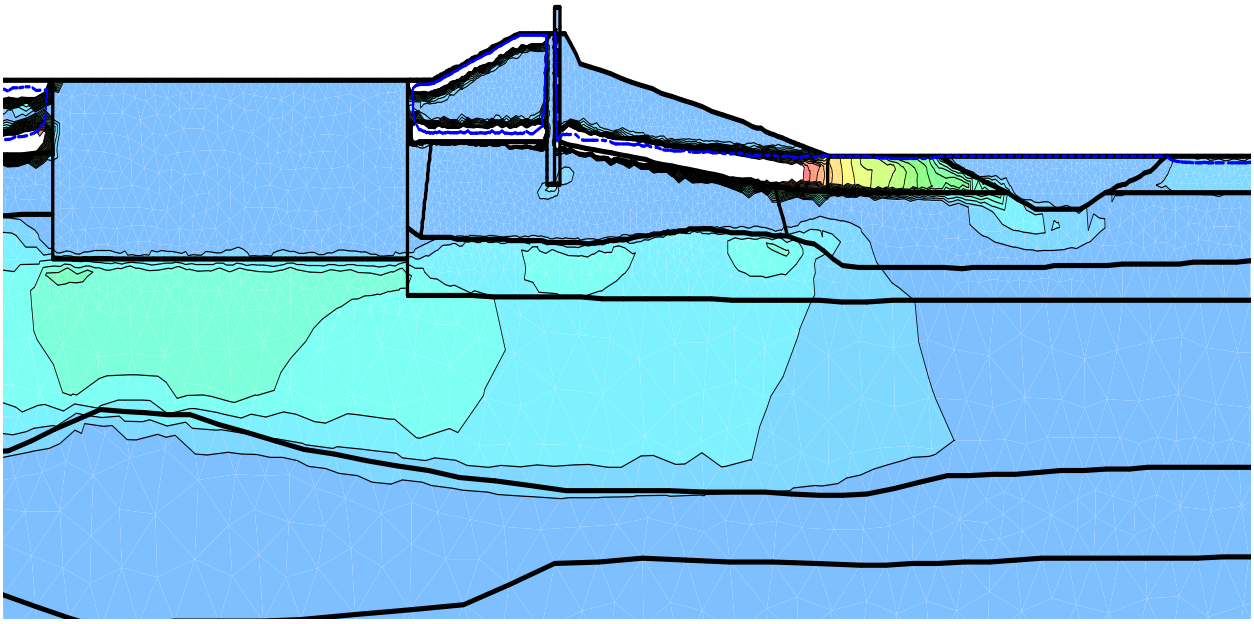


Figure 44: Transient model result, Hydraulic gradients (Bea and Cobos-Roa 2008). Time: between 7:00 and 8:00 am on August 29, 2005. Gradient through the clay layer above the marsh deposit are greater than 1.5 indicating a hydraulic blow-out at the toe of the levee.

70. The effects of different positions and backfill in the EBIA excavations were analyzed by modeling the permeable backfill 60 feet away from the waterside toe and at the waterside toe. A clay-filled excavation was also modeled in order to capture the effect of the placement of low permeability material on the excavations. Figures 45 and 46 show calculated pore water pressures for different excavation configurations and presence of gap on the center of the clay layer and center of the marsh layer, respectively.

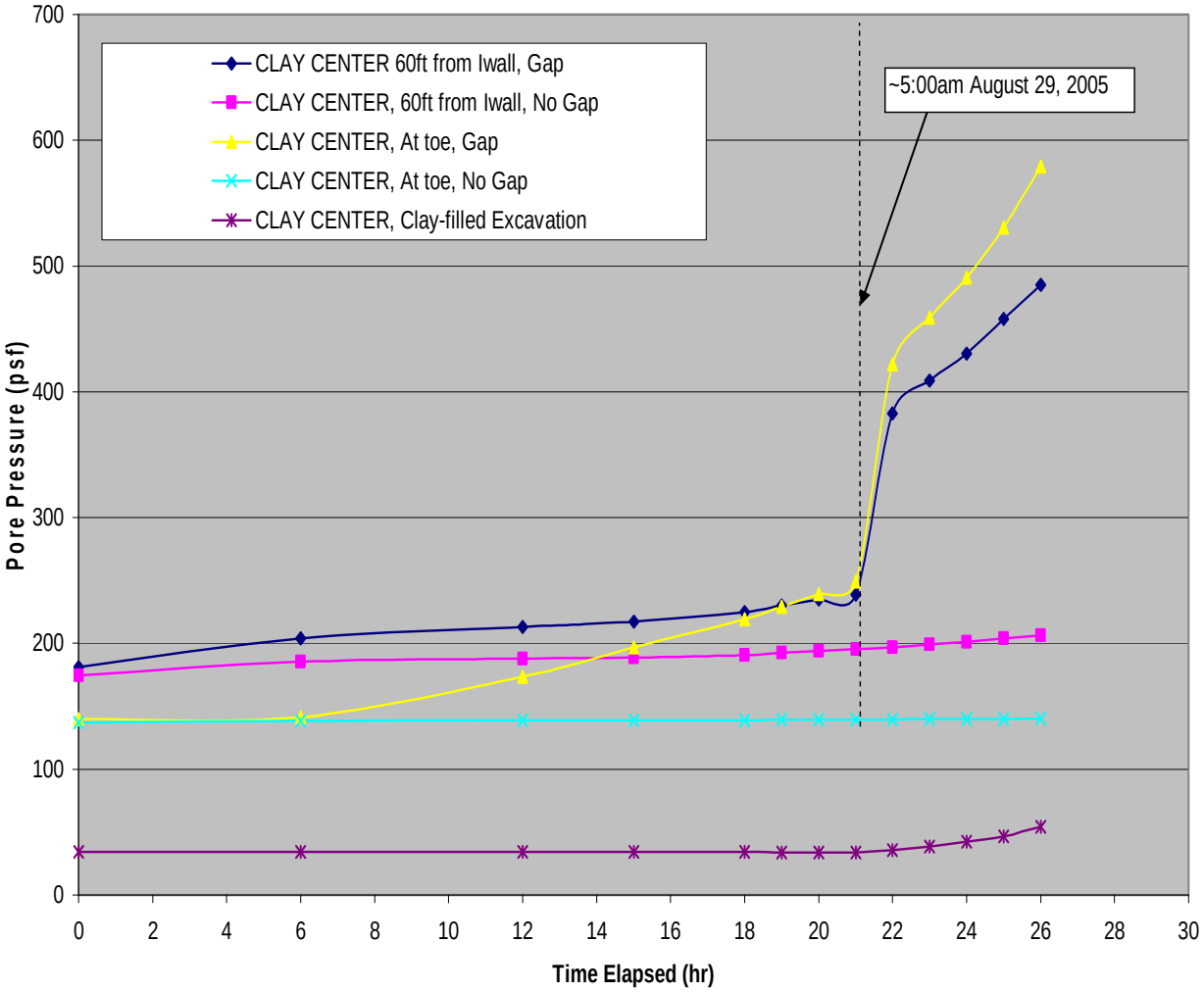


Figure 45: Transient model results (Bea and Cobos-Roa 2008). Pore pressures along center of clay layer. Marsh kx = 10^{-3} cm/s.

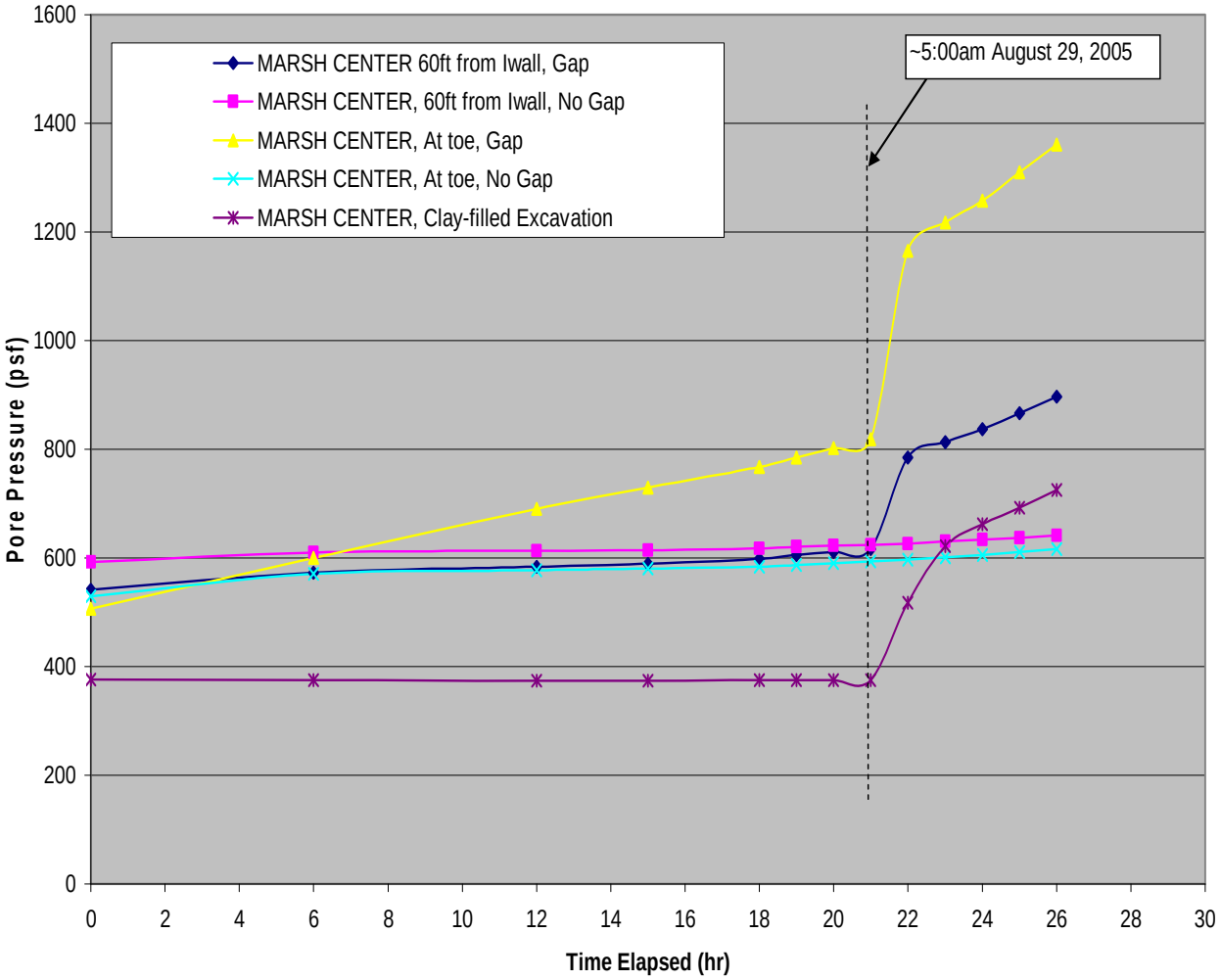


Figure 46: Transient model results (Bea and Cobos-Roa 2008). Pore pressures along center of Marsh layer. Marsh kx = 10^{-3} cm/s.

71. A factor of safety to hydraulic uplift was calculated as the relation between the effective stress acting at the bottom of the blanket and the pore pressure at the same point (Figure 47). For the case of no-gap and clay filled excavation, the computed factors of safety against uplift are well above two, whereas for the cases considering a gap and excavation filled with permeable material, the factors of safety range from 1.2 to 0.8 after about 5:00 am (CDT) on August 29, 2005. These low factors of safety clearly indicate a very high potential for 'blowout' and uplift failure.

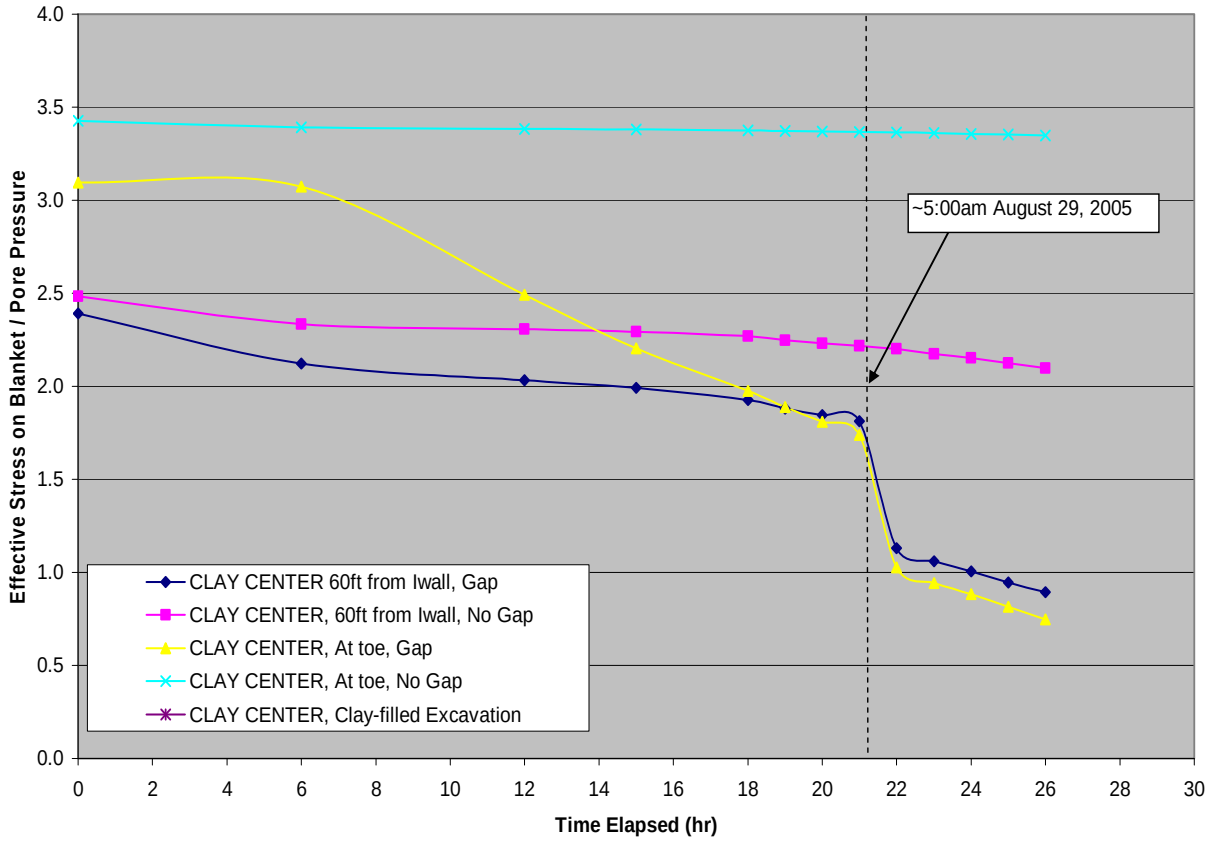


Figure 47: Factor of Safety against hydraulic uplift, South breach (Bea and Cobos-Roa 2008). Marsh kx = 10^{-3} cm/s. The factor of safety drops below unity at about 5 am (CDT) August 29, 2005 indicating the onset of levee – floodwall failure.

72. The next step in the study of the North Breach was to perform slope stability analyses that incorporate the hydraulic response of the levee – floodwall system. The software SLOPE\W was employed in these analyses. The pore pressures generated by the SEEP\W finite element analyses were used by SLOPE\W when calculating lateral stability factors of safety. The software uses Limit Equilibrium Analysis to calculate factors of safety by a number of analytical methods. The results included here use the Spencer method.

73. To evaluate the effects of formation of a waterside gap between the sheet piling and the supporting soil levee, factors of safety were calculated for the cases where no gap was developed (therefore a failure surface extending to the waterside slope was used), and a second

set of analyses were performed assuming that a gap had formed throughout the length of the sheet pile. Because of the uncertainties associated with the permeability characteristics of the Marsh deposits that underlie this area, the lateral stability analyses were performed for horizontal permeability in the range of $1x10^{-3}$ cm/s to $1x10^{-6}$ cm/s. Consequently, the pore pressure field generated from the seepage analyses for each of the permeability and storm surge elevation values were used to calculate the factor of safety at different water elevations.

74. Based on the soil boring and testing information cited earlier and detailed by Bea and Cobos-Roa (2008), median values of the soil properties for the different stratagraphic layers were determined for unit weight, undrained shear strengths and friction angles. Table 3 summarizes the material properties used in the slope stability analyses. Figure 48 shows the input geometry used for the stability analyses of the North Breach.

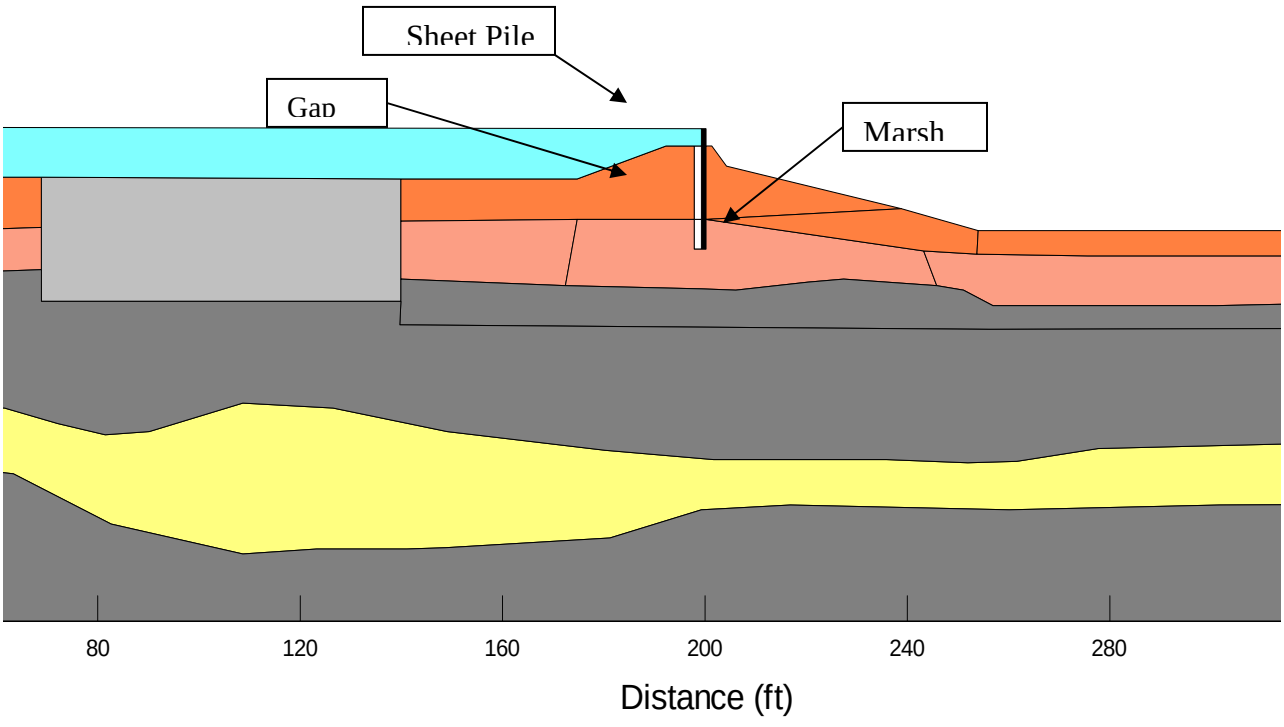


Figure 48: Cross section used for slope stability analyses for the North Breach (Bea and Cobos-Roa 2008).

Table 3: Median soil properties identified for soil layers used in stability analyses for the North Breach

Material	Shear Strength Parameter			
	Su (psf)	ϕ (deg)	Su/p'	γ (pcf)
Clays (Fill)	555			105
Marsh (under levee)	442			86
Marsh (free field)	250			86
Sands		30		110
Overconsolidated "Crust"	300			100
Deep Clays (CH)			0.26	110

75. Figure 49 shows results of the lateral stability factor of safety (ratio of 'driving' force to 'resisting' force) for the North Breach. These factors of safety are based on the use of mean (average, expected) values for all of the material parameters. For the case where the gap has formed and the excavation is either 60 feet from the levee – floodwall or at the toe of the levee, the 'best estimate' (Mean value) of the factor of safety becomes less than unity at a surge elevation of about + 9 feet (NGVD 88) – or about 6:00 am (CDT) on August 29, 2005.

76. These results indicate that the North Breach developed well before overtopping of the floodwall occurred and that this failure was clearly linked to the adjacent EBIA site clearing excavations and backfilling operations and their influences on underseepage and stability of the floodwall sections adjacent to the Lower 9th Ward.

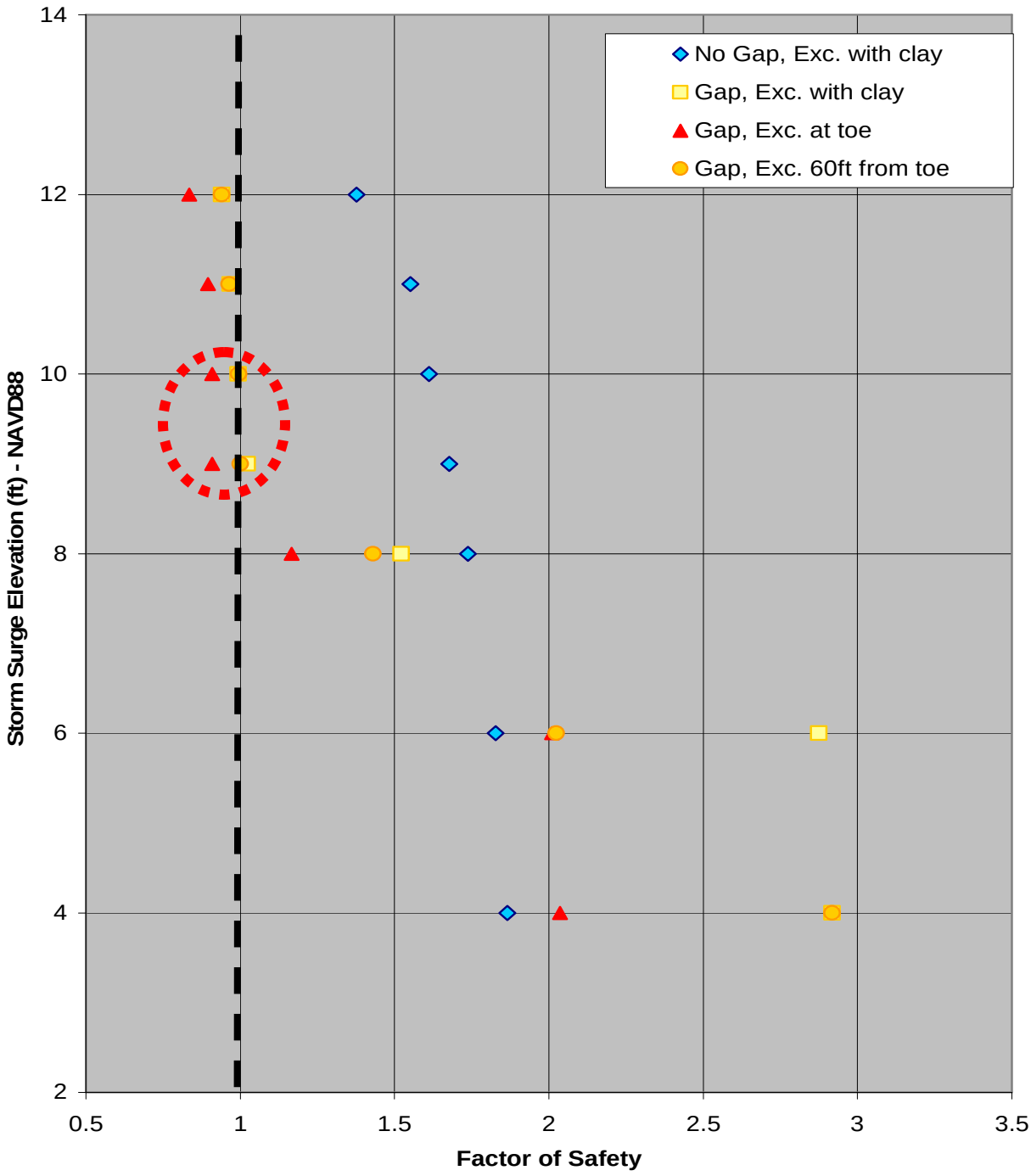


Figure 49: Factor of safety (mean values) versus storm surge elevation for the North breach (Bea and Cobos-Roa 2008).

77. Figure 50 shows the critical shear surface for a storm surge of 12 feet, approximately at 7:00 am (CDT) on August 29, 2005. The mean factor of safety is approximately 0.8 at this time and the failure surface has been completely developed.

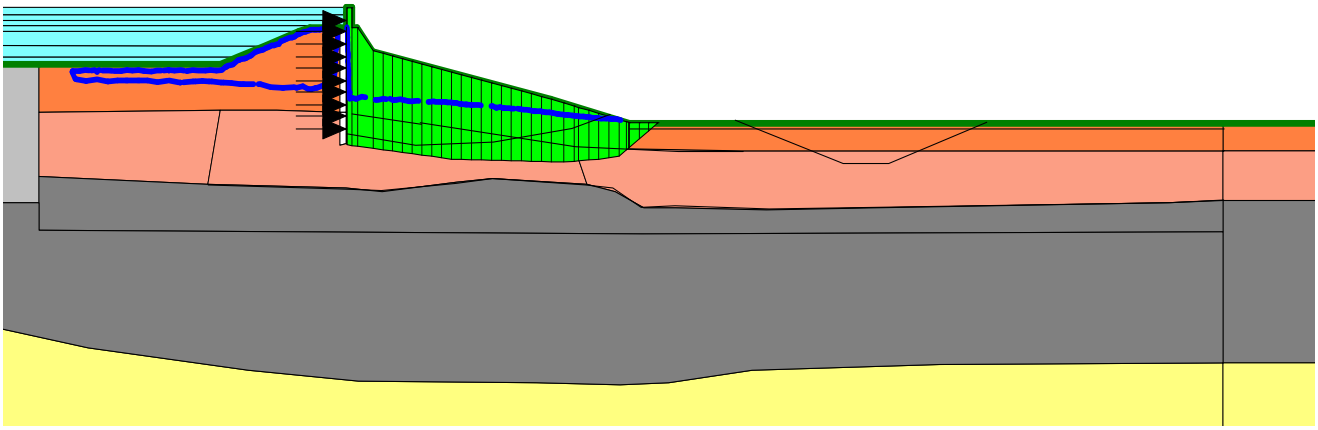


Figure 50: Critical shear surface for North breach, storm surge 12 feet. Factor of Safety = 0.80 (Bea and Cobos-Roa 2008).

78. These analyses are similar to those developed by IPET (2007) – with two categories of important differences. IPET performed analyses of the hydraulic conductivity effects as they pertained to seepage and the potential for levee blow-out (Vol. V, Appendix 17). IPET also performed stability analyses of the soil – levee – floodwall system (Vol. V, Appendix 11). IPET did not perform analyses in which the two aspects were integrated – combined hydraulic pressure and stability considerations. As has now been found at several other breach locations, the hydraulic pressures in the buried subsurface marsh – swamp layers can generate significant uplift forces that can contribute to destabilization of the soil – levee – sheet pile – floodwall system (Bea and Cobos-Roa 2008, Bea 2008, Seed et al 2008b). In addition, the IPET analyses did not include the effects of the EBIA excavations nor the important subsurface soil-water hydraulic effects associated with the gap formed between the supporting sheet piling and the soils on the IHNC side during the surge rise. It is for these reasons that the IPET stability analyses were not able to accurately determine the water levels at which the breaches fully developed – factors of safety greater than unity (reference Vol. V, Appendix 11, V-11-12, IPET 2007) were computed for water levels (times) at which observational data gathered by IPET

indicated the breaches had developed and were admitting floodwaters to the Lower 9th Ward (Figure 18).

South Breach Seepage & Stability Analyses

79. The south breach extends from station 24+00 to 33+00 of the original DM03 (USACE, 1966). Figures 51 – 53 show the interpreted cross sections through this breach site. Figure 3 shows a thick Marsh layer, bottomed in an irregular fashion around elevation –25 feet (NAVD88). Like the north section, a thin clay layer covers the surface and is apparently related to embankment fill. The top of the Marsh deposit is located at about elevation –5 feet, but a thin clay layer appears to cut the marsh layer in half.

80. The sheet pile extends from +12.0 feet to -10.5 feet (NAVD88), and again, the sheet piling fail to cut through the Marsh deposit. The cross section in Figure 53 shows another waterside excavation; this time narrower than the one found in the North section, but with the same depth to the bottom and backfill material.

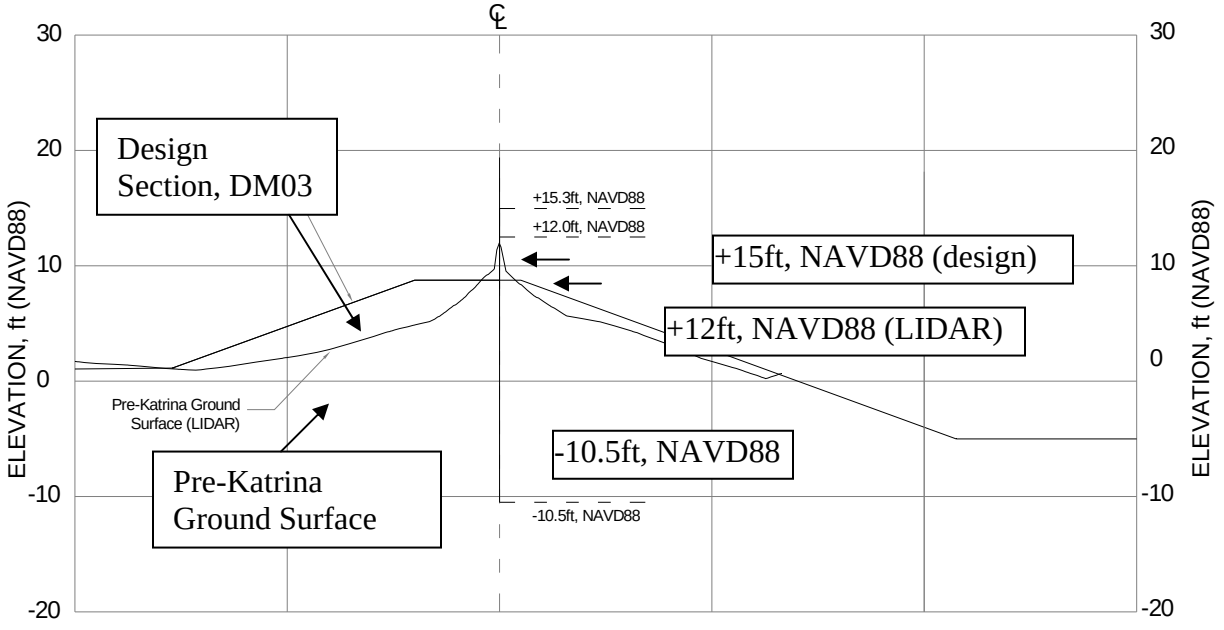


Figure 51: South section comparing design geometry from DM03 and pre-Katrina section from LIDAR survey performed in 2000 (Bea and Cobos-Roa 2008).

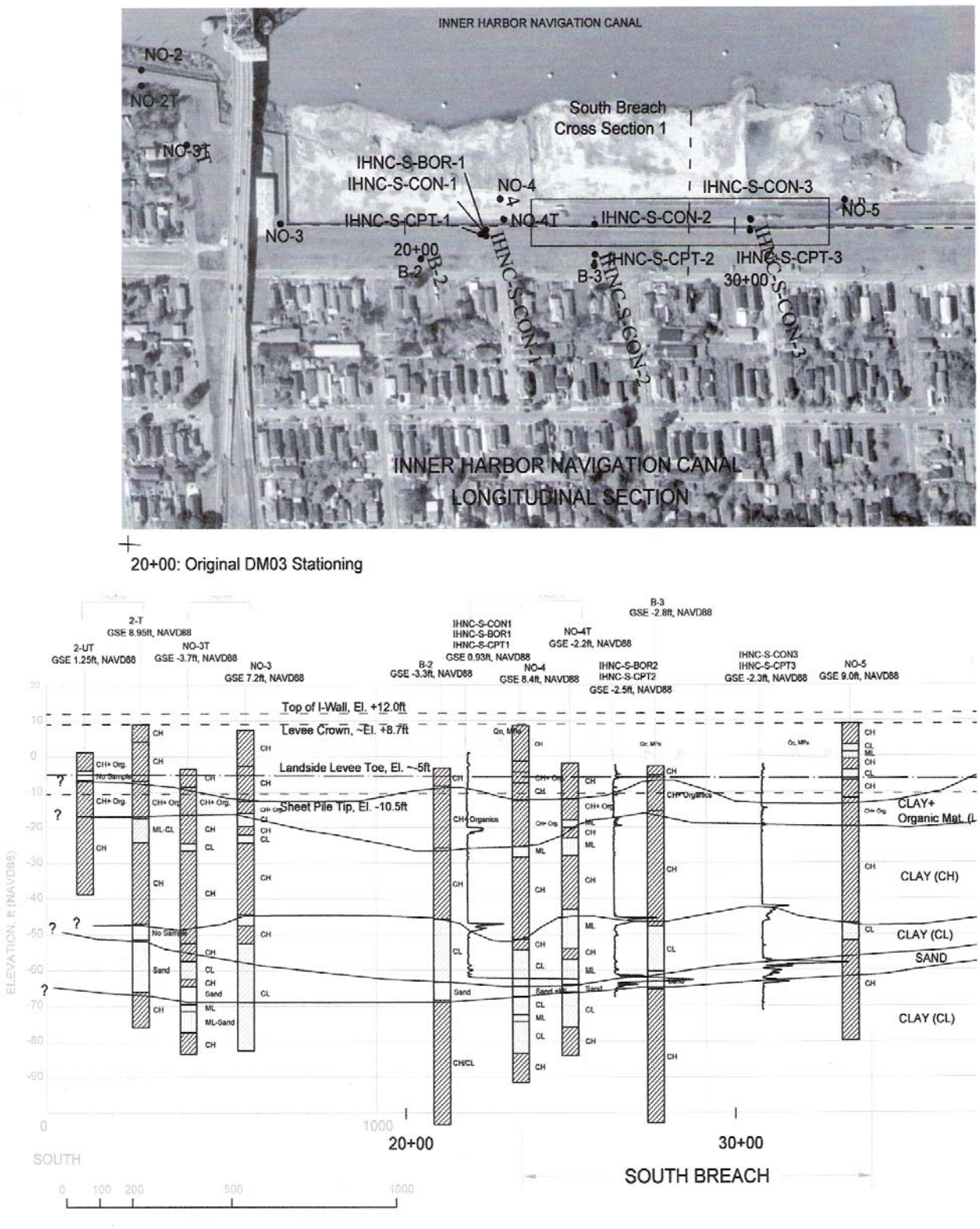


Figure 52: Longitudinal Profile along IHNC, south breach (Bea and Cobos-Roa 2008).

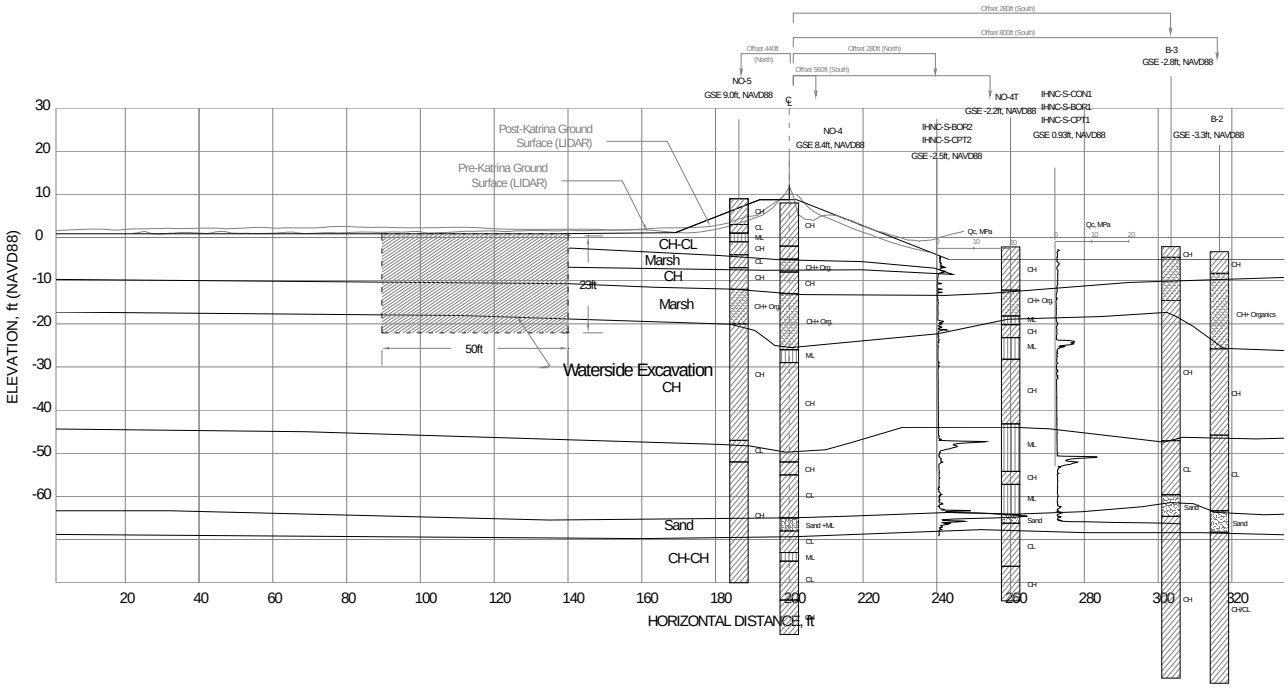


Figure 53. Cross section through South breach (Bea and Cobos-Roa 2008).

81. Transient seepage analyses were performed for the rising water level in the IHNC according to the measured surge elevation hydrographs in this area (Figure 2). The surge hydrograph was input into the seepage models which generated analytical results beginning at a water elevation +2 feet.

82. To account for the effects of increased stresses beneath the levee on Marsh permeability, the Marsh was divided into “free field marsh”, and “marsh beneath embankment”. The latter case was always 10 times lower than the free field case.

83. The results of the seepage analyses (Figure 54 and 55) show that after 33 hours of surge development (August 28, 2005 12:00 am to August 29, 2005, 9:00 am CDT), high pore pressures and hydraulic gradients developed along the Marsh deposit for the entire range of permeabilities utilized in the analyses. There was less than a 10% difference in the pore water pressures caused by the range in permeabilities.

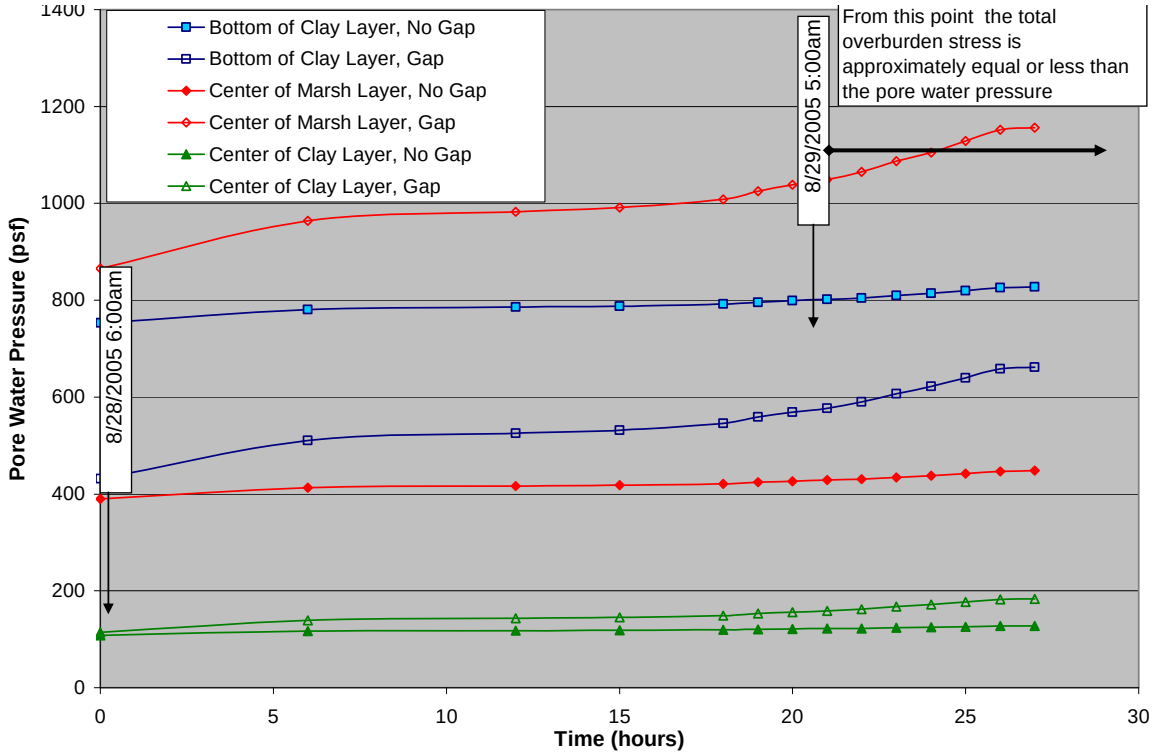


Figure 54: Pore pressure versus time at three points near the toe of the levee for Marsh permeability of 10^{-3} cm/s (Bea and Cobos-Roa 2008).

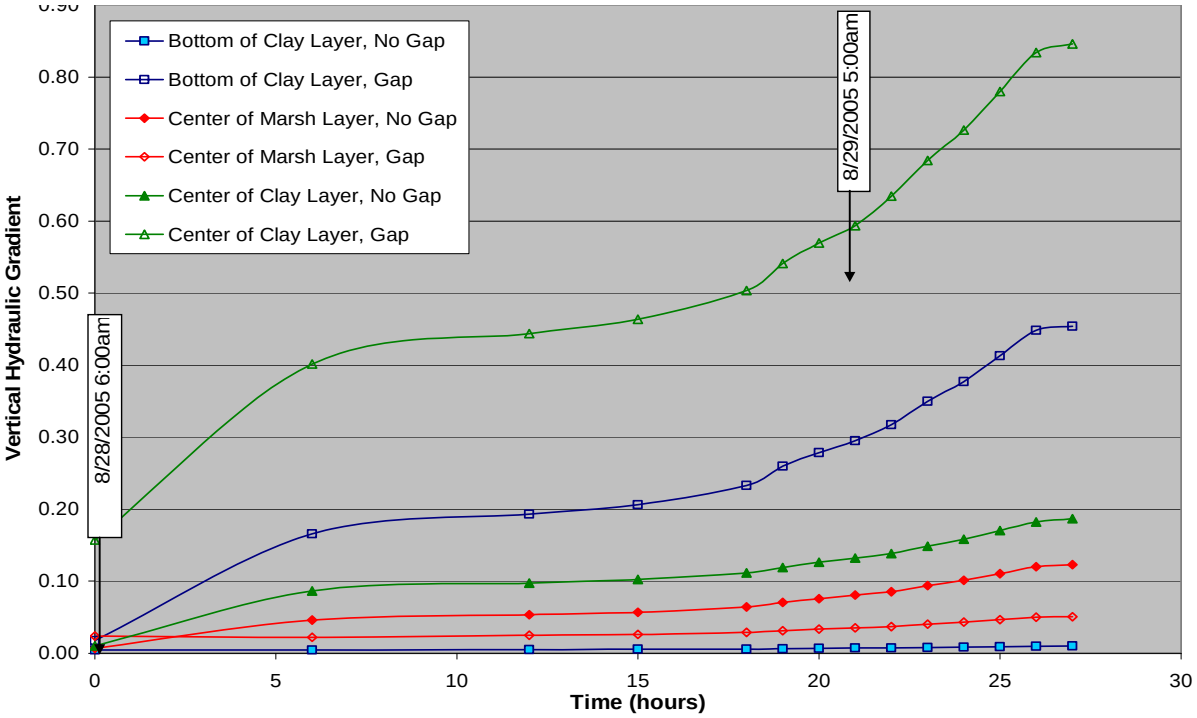


Figure 55: Vertical hydraulic gradients versus time on three points near the toe of the levee for Marsh permeability of 10^{-3} cm/s (Bea and Cobos-Roa 2008).

84. Results from the transient seepage analyses show that the differences in pore pressure are important when the gap is introduced into the model. This is because the full head of the storm surge is applied along the back of the sheet pile and to the foundation soils. As mentioned before, the South Breach section shows a clay layer just beneath the sheet pile tip, therefore the differences between gap and no gap are not as pronounced as the North Breach model shows, which has the sheet pile bottomed in the middle of the Marsh deposit. When the gap was not considered, the degree of saturation and advance of the wetting front was completely different from the results shown here.

85. Figures 56 and 57 summarize the analytical results of the transient seepage analyses through the levee – floodwall cross section in terms of total heads (feet of water) and hydraulic gradients at 7:00 am on August 29, 2005. It can be seen to the left of the sheet pile, that the IHNC side portion of the levee is not fully saturated, but to the protected side, the phreatic (water table) surface has risen enough to exit on the levee slope.

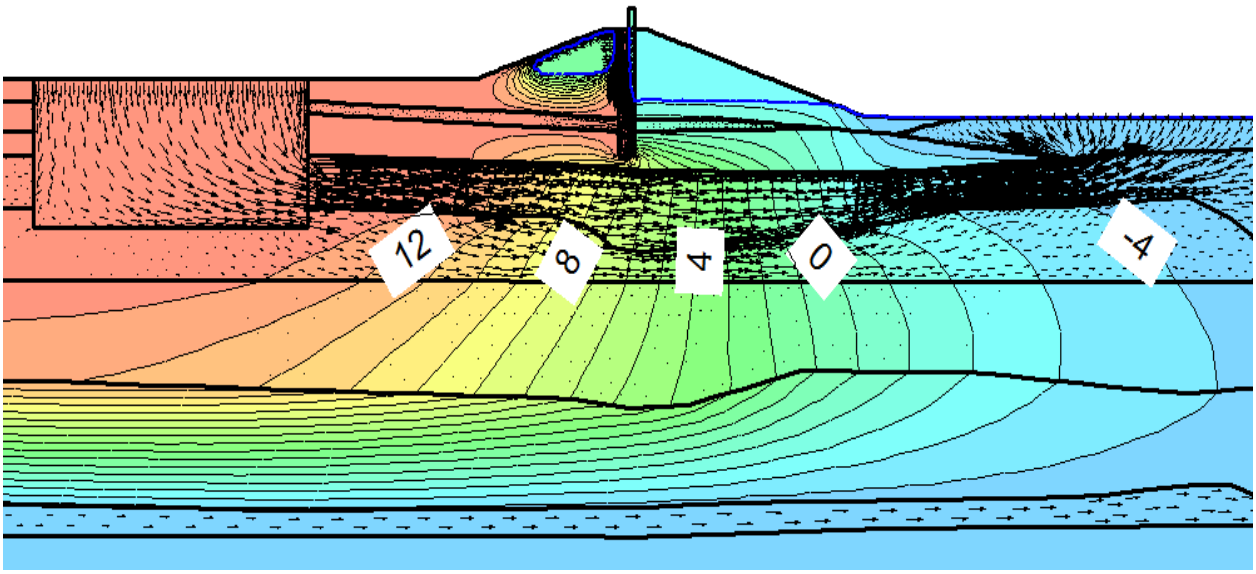


Figure 56: Transient seepage analysis results (Bea and Cobos-Roa 2008). Total Heads (feet). Time: between 7:00 and 8:00 am on August 29, 2005. The blue line represents the calculated phreatic surface. Note that there is virtually no head loss on the levee until the sheet pile is reached (red zone extending from the waterside towards the right).

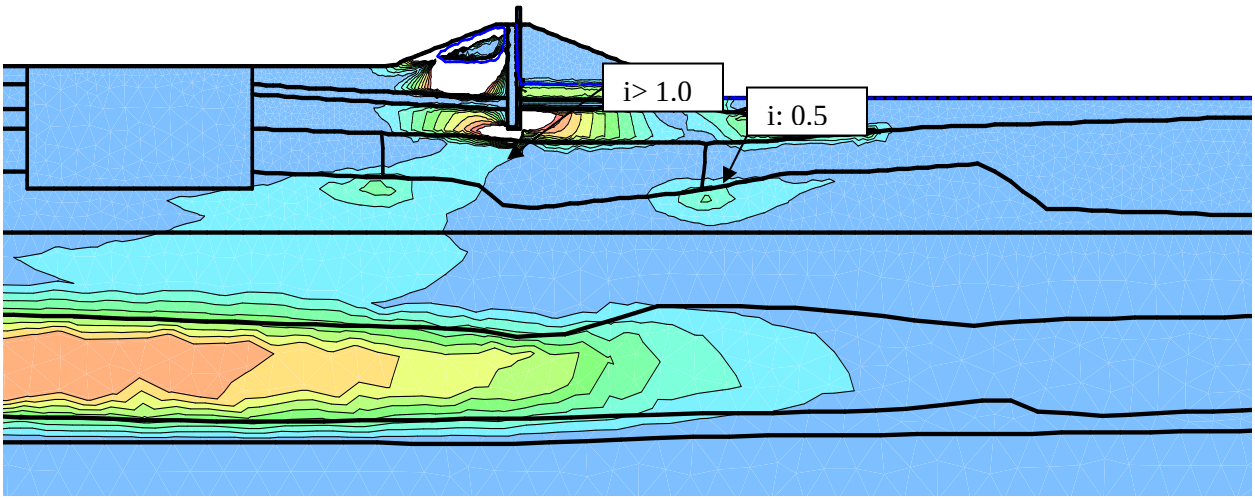


Figure 57: Hydraulic gradients from transient seepage analysis (Bea and Cobos-Roa 2008). Time: between 7:00 and 8:00 am on August 29, 2005. Maximum Gradient through the clay layer above the marsh deposit is approximately 0.5 (hydraulic gradient contour interval is 0.1).

86. From the plot in Figure 57, it can be seen that the model predicts hydraulic gradients of approximately 0.5 at the clay layer near the toe of the levee. In the vicinity of the sheet pile tips, the hydraulic gradients exceed unity. Values higher than 0.5 are considered unacceptable by the current practice. As noted earlier, these results are anticipated to be conservative – i.e. they underestimate the hydraulic – seepage effects associated with the EBIA excavations.

87. The effects of the EBIA excavations were analyzed by modeling the permeable backfill 60 feet away from the levee waterside toe and at the waterside toe. A clay-filled excavation was also modeled in order to capture the effect of the placement of low permeability material on the excavations. A factor of safety to hydraulic uplift was defined as the ratio of the effective stress acting at the bottom of the levee fill and the pore pressure at the same point (Figure 58). For the case of no-gap and clay filled excavation, the computed factors of safety against uplift are well above two, whereas for the cases considering a gap and excavation filled with permeable material, the factors of safety range from 1.2 to 0.9 after 5:00 am on August 29, 2005.

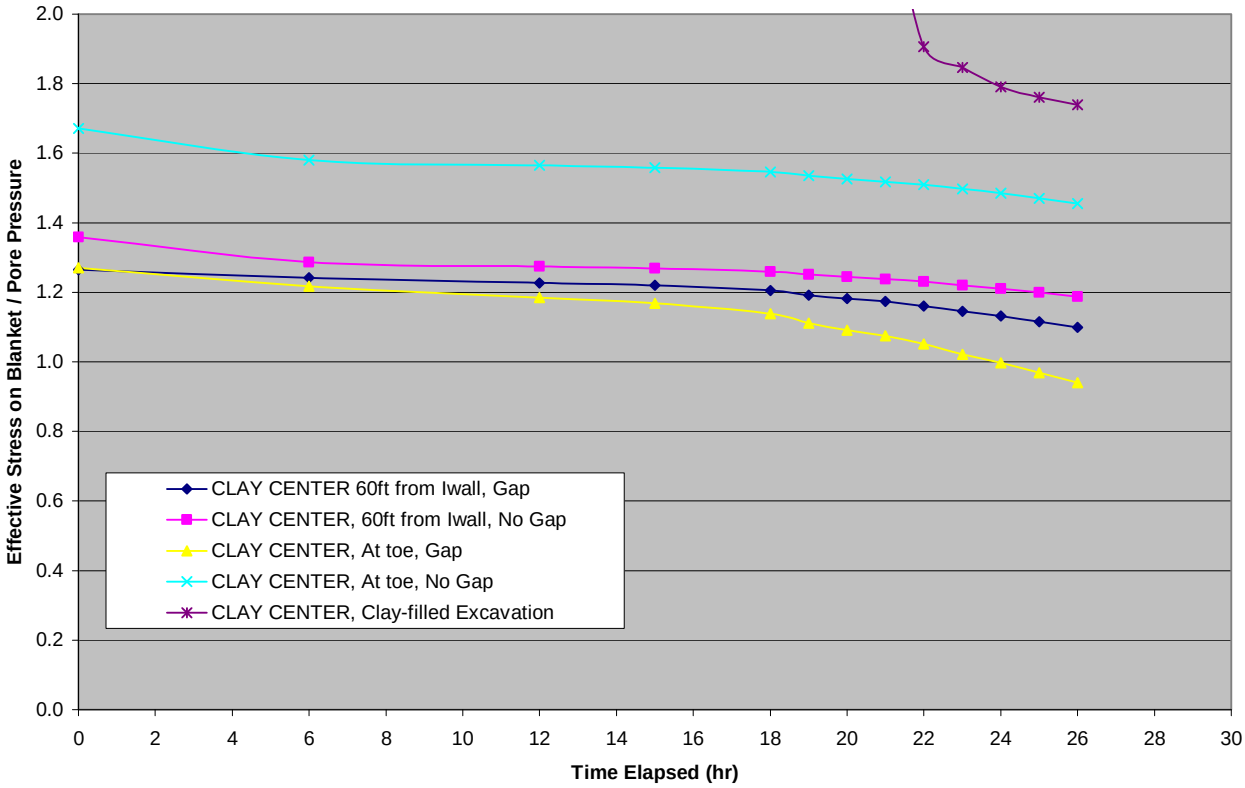


Figure 58: Factor of Safety against hydraulic uplift, South breach. Marsh kx = 10^{-3} cm/s (Bea and Cobos-Roa 2008).

88. The next step in the study of the South Breach was to perform slope stability analyses that incorporate the hydraulic response of the levee – floodwall system. The software SLOPE\W was employed in these analyses. The pore pressures generated by the SEEP\W finite element analyses were used by SLOPE\W when calculating lateral stability factors of safety. The software uses Limit Equilibrium Analysis to calculate factors of safety by a number of analytical methods. The results included here use the Spencer method.

89. To evaluate the effects of formation of a waterside gap between the sheet piling and the supporting soil levee, factors of safety were calculated for the cases where no gap was developed (therefore a failure surface extending to the waterside slope was used), and a second set of analyses were performed assuming that a gap had formed throughout the length of the

sheet pile. Because of the uncertainties associated with the permeability characteristics of the Marsh deposits that underlie this area, the lateral stability analyses were performed for horizontal permeability in the range of $1x10^{-3}$ cm/s to $1x10^{-6}$ cm/s. Consequently, the pore pressure field generated from the seepage analyses for each of the permeability and storm surge elevation values were used to calculate the factor of safety at different water elevations.

90. Based on the soil boring and testing information cited earlier and detailed by Bea and Cobos-Roa (2008), median values of the soil properties for the different stratagraphic layers were determined for unit weight, undrained shear strengths and friction angles. Table 4 summarizes the material properties used in the slope stability analyses. Figure 59 shows the input geometry used for the stability analyses of the North Breach.

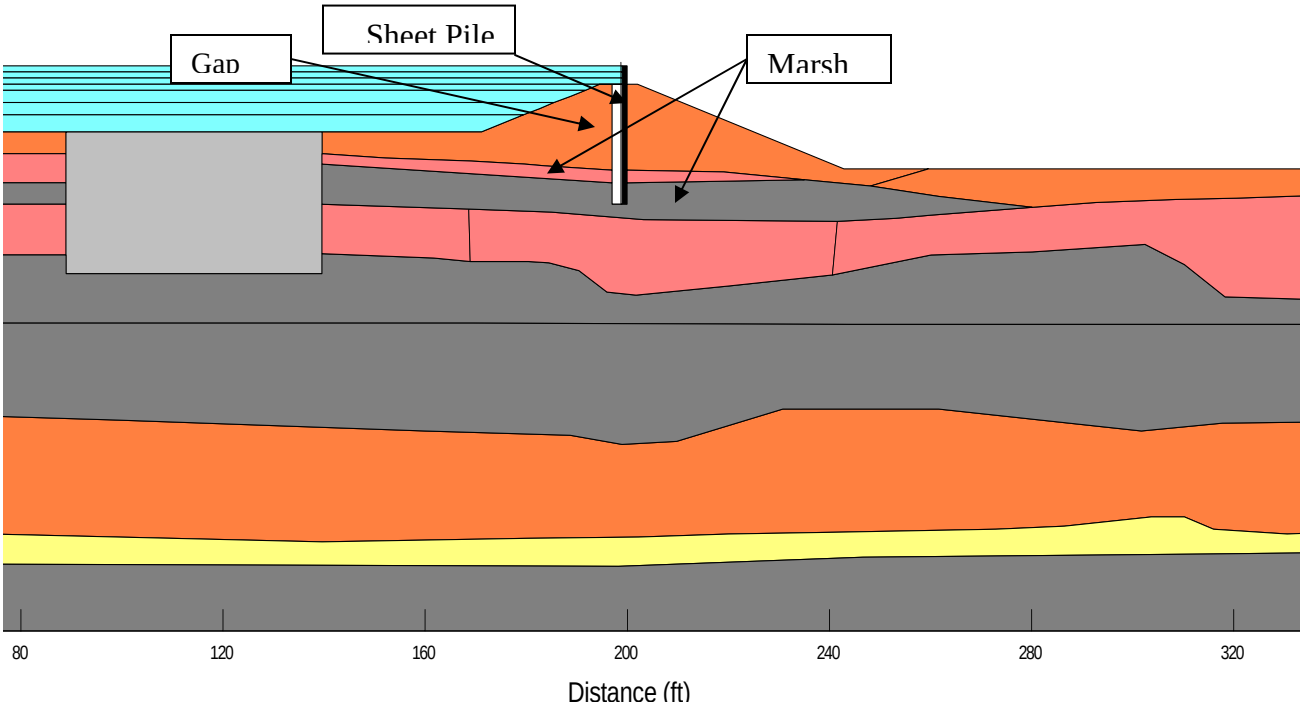


Figure 59: Analysis cross-section for South breach (Bea and Cobos-Roa 2008).

Table 4: Material properties used for the slope stability models, South Breach.

Material	Shear Strength Parameter			
	Su (psf)	ϕ (deg)	Su/p'	γ (pcf)
Clays (Fill)	342			104
Marsh (under levee)	400			86
Marsh (free field)	250			86
Sands		30		110
Overconsolidated "Crust"	300			104
Deep Clays (CH)			0.26	104

91. Figure 60 shows the calculated factors of safety (ratio of lateral 'driving forces' to 'resisting forces') based on mean (expected, average) values for all of the parameters for the South Breach for the cases of no gap formed, and the gap fully formed. Results from the analyses show that factors of safety drop below unity (probability of failure of 50%) for all cases between elevations +12 feet and +14 feet between 7:00 am and 9:00 am on August 29, 2005. These analytical results are in agreement with available information on the response of this levee - floodwall during hurricane Katrina (IPET 2007, ILIT 2006, Team Louisiana 2007).

92. These results indicate that the sequence of events leading to complete development of the South Breach developed after the similar sequence of events at the North Breach. Given the elevation of the floodwall, these analytical results indicate overtopping would have been occurring during the development of this breach. Forensic engineering observations clearly indicated the erosion 'trench' adjacent to the floodwall on the protected side (ILIT 2006;

IPET 2007; Team Louisiana 2007). Initiation of the South Breach was clearly linked to the EBIA site clearing excavations adjacent to the floodwall.

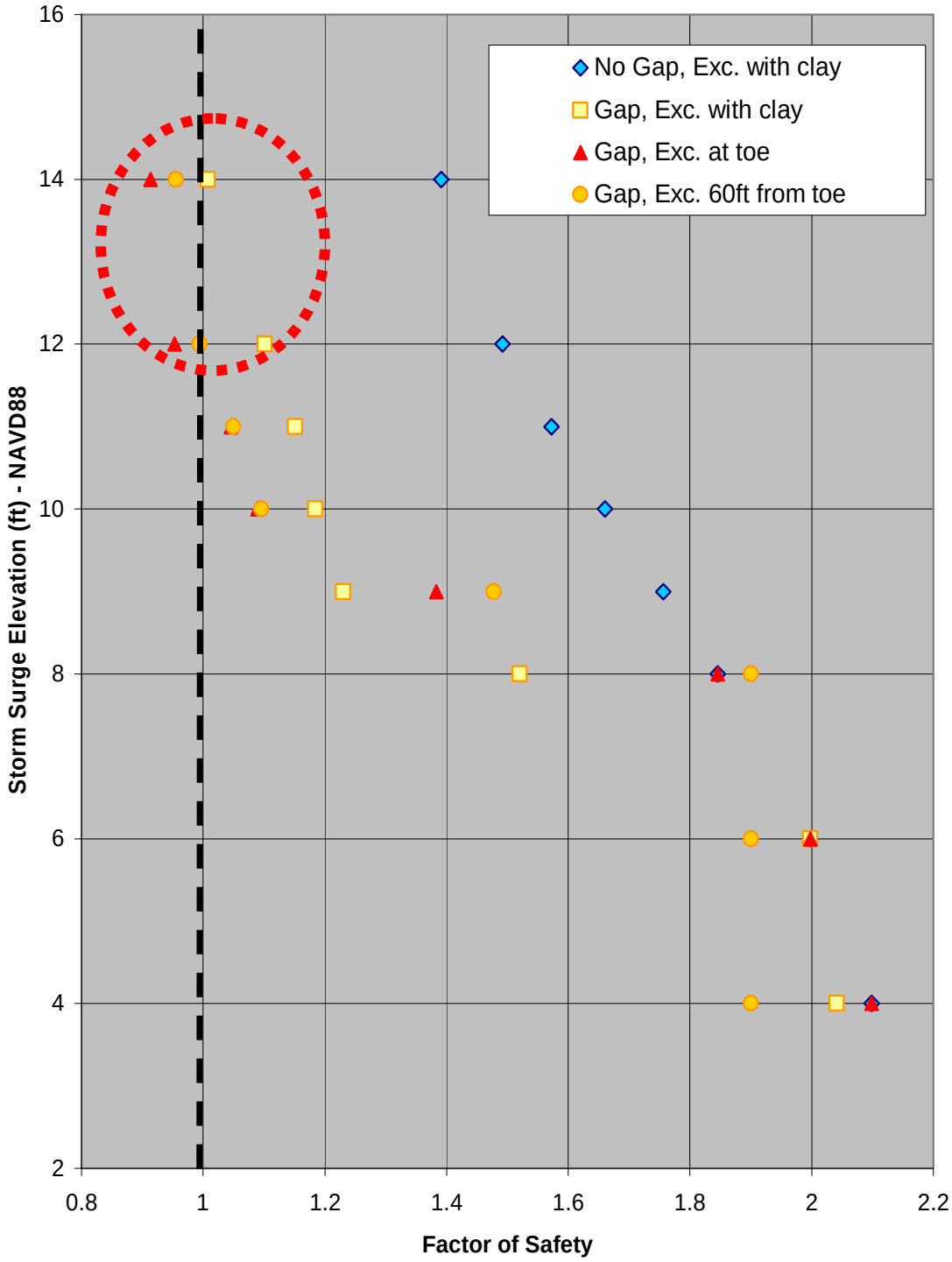


Figure 60: Factors of Safety (based on mean values of input parameters) versus storm surge elevation for the South Breach for Marsh permeability of kx = 10^{-3} cm/s (Bea and Cobos-Roa 2008).

93. Critical failure surfaces were determined to be non-circular surfaces starting at the tip of the sheet pile and progressing towards the landside toe along the Marsh deposit. Figure 61 shows the critical surface using Spencer's method for a storm surge of 14 feet when the Factor of Safety for lateral stability was 0.94 – at a 'failure' condition.

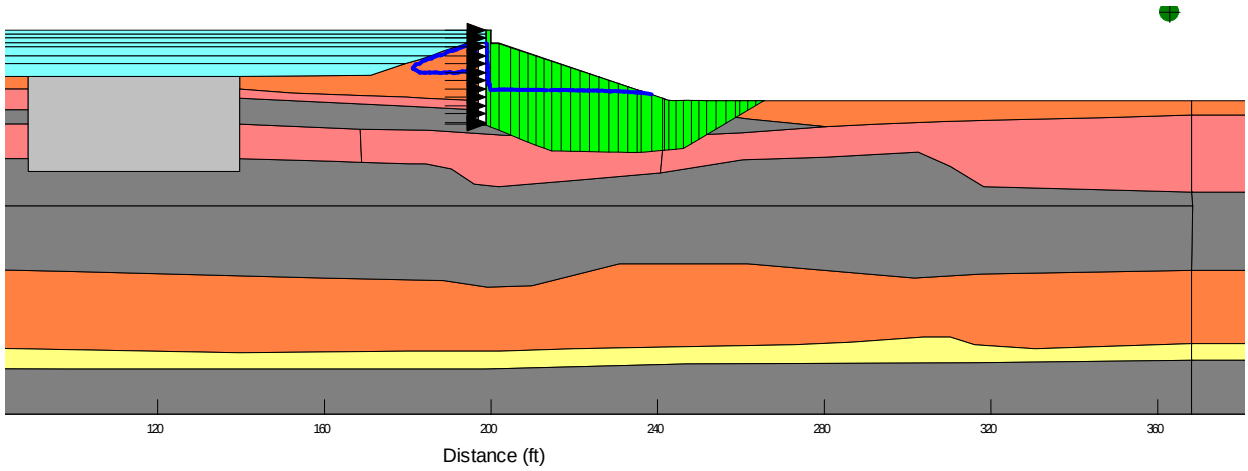


Figure 61: Critical shear surface for South Breach with storm surge at 14 feet and factor of safety = 0.94 (Bea and Cobos-Roa 2008).

94. Figures 62 - 64 shows photographs of the South Breach taken shortly after its development. The photographs shows a displaced segment of the protected side levee very similar in shape to the green failure wedge indicated in Figure 61 – indicating large lateral movements of the supporting levee on the protected side. Also, note the floodwall with the extensive overtopping erosion trench either side of the breach section did not participate in the failure – development of the South Breach. In Figure 64 near the center of the breach, note the exposed buried utility pipelines that cross at right angles to the floodwall. See the supporting Technical Report by Bea and Storesund (2008) for more background on these utility crossings. The floodwall between the North and South Breaches was severely 'distorted', indicating this stretch of the flood protection structure was near the point of initiating the progressive failure –

breaching process observed at the North and South Breach sites (ILIT 2006, Team Louisiana 2007, IPET 2007, Seed et al 2008b).



Figure 62: Photograph of South Breach looking north showing the pushed-back levee section on the protected side of the floodwall (EPA 2005).



Figure 63: Photograph of South Breach looking south showing the pushed-back levee section on the protected side of the floodwall (IPET 2007). Note the deep trench eroded on the back side of the floodwall by the overtopping floodwaters.

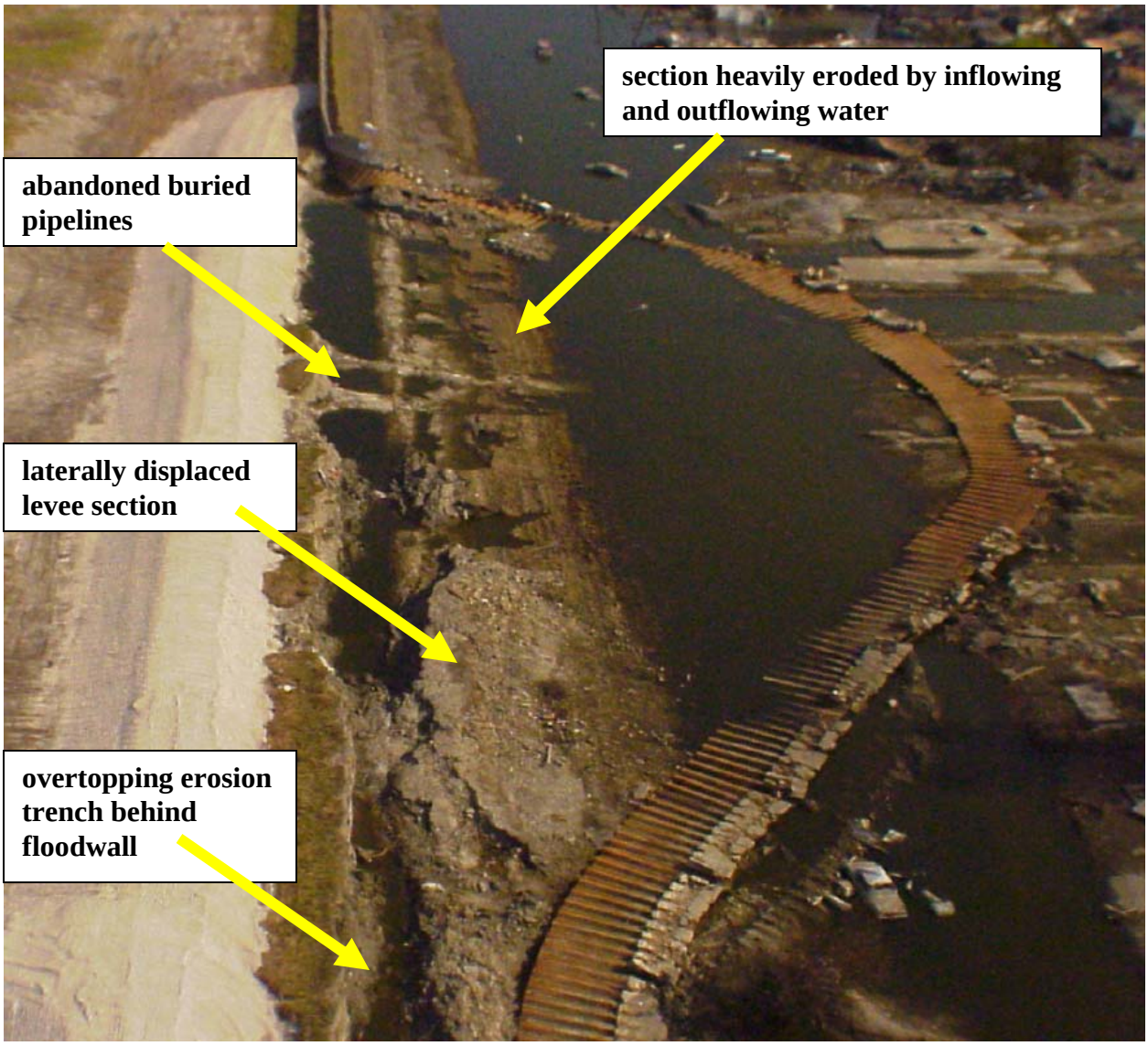


Figure 64: Aerial view of South Breach showing displaced levee section, overtopping erosion trench, and utility line crossing near the center of the breach (USACE 2006).

95. An alternative mode of failure investigated in assessing development of the South Breach was overtopping, trench erosion, and lateral toppling of the I-wall. The field investigations made by the ILIT, IPET, and Team Louisiana teams identified the deep eroded trench along the inboard side of the flood wall (Figure 63, Figures 65-67). The trench depth varied along this alignment and was generally in the range of 4 to 7 feet (Figure 65).



Figure 65: Eroded trench along inboard side of the floodwall at the south end of the South Breach at the Lower 9th Ward (ILIT 2006).


Figure 66: Eroded trench along inboard side of the floodwall at the center of the South Breach at the Lower 9th Ward (Raffman 2008)



Figure 67: Eroded trench along inboard side of the floodwall at the center of the South Breach at the Lower 9th Ward (Raffman 2008).

96. Given that the floodwall was overtopped about 7:00 am CDT (Figure 2) and that the stability analyses indicated that the water level would need to reach +12 to +14 feet NAVD 88 before a lateral stability failure developed (about 8:00 to 9:00 am CDT), there would have been 1 to 2 hours of overtopping flow. This overtopping flow would have a depth over the top of the wall of 1 to 2 feet (Figure 68). The height from the top of the flood wall to the top of the soil on the protected side would 7.5 to 8.5 feet. Based on the results summarized in Figure 69, the plunging jet velocity at impact on the soil would be 22 to 26 feet per second (IPET 2007). This is a very high impact velocity and would exert high erosive and scour stresses on the surface soils supporting the floodwall.

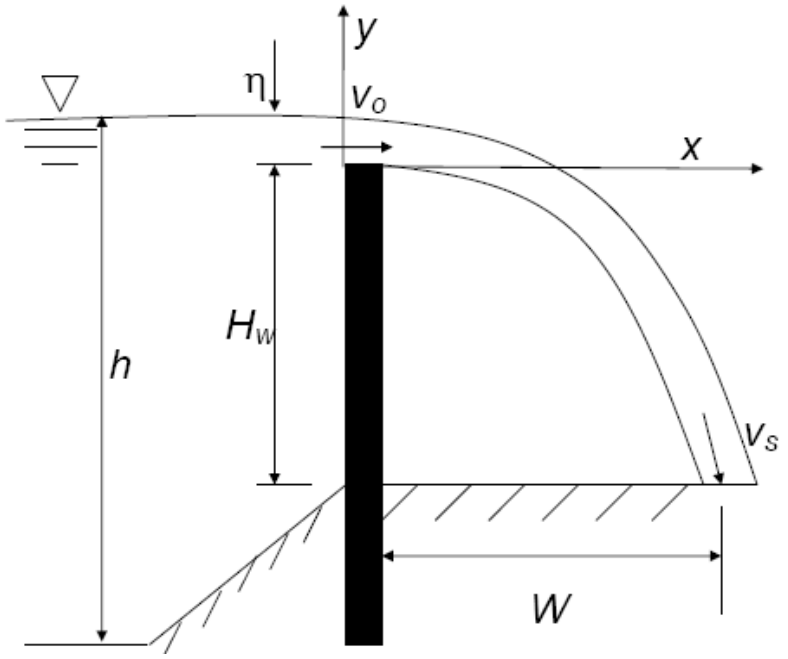


Figure 68: Definition sketch for analytical model to evaluate overflow of floodwall (IPET 2007).

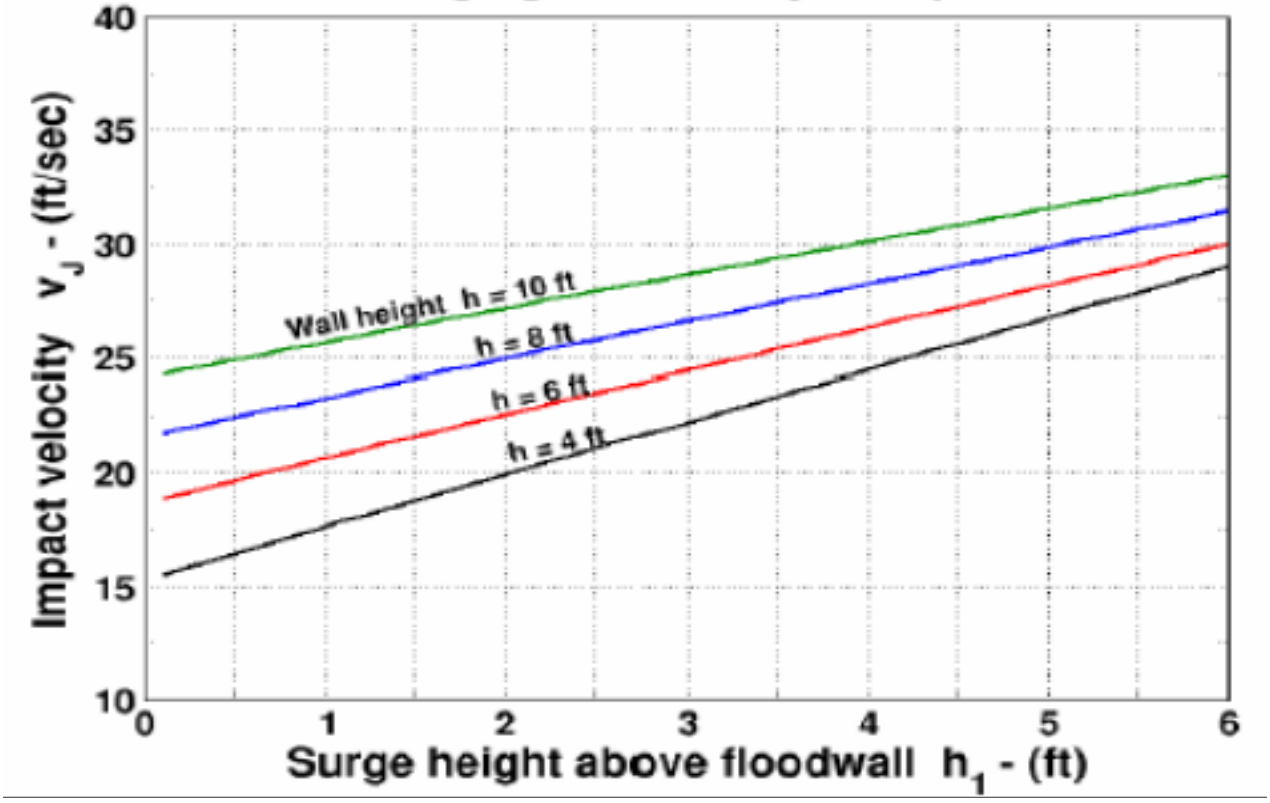
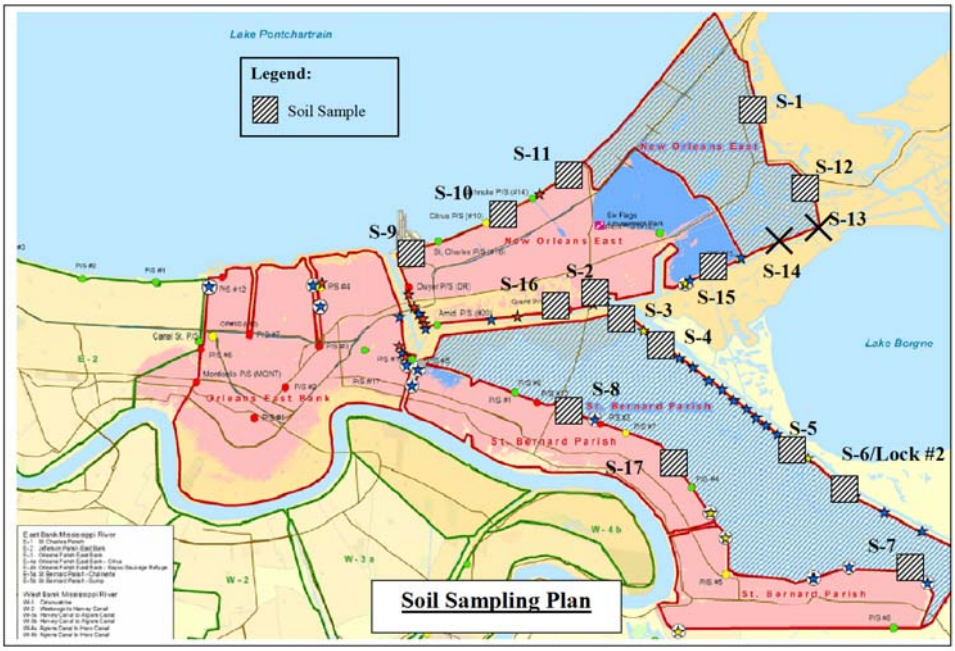


Figure 69: Plunging jet velocity at impact with ground as function of surge height above top of the floodwall (Figure 68) (Hughes 2006).

97. Three different methods were used to estimate the depth of the erosion trench that could develop. The first method was developed by Briaud et al (2001, 2008). The second method was based on the guidelines issued by the Federal Highway Administration for use in hydraulic design of culverts and channels (FHWA 2006). The third method was developed by Hanson, et al (1991, 2001, 2003).

98. The surface soils at this location were characterized by the IPET studies as clay with sand and silt seams (IPET 2007). Briaud et al (2008) performed laboratory erosion tests on levee surface soils in the Greater New Orleans area as part of the ILIT investigation (Figure 70) (ILIT 2006, Briaud et al 2007, 2008). The surface soil samples gathered from sites having characteristics similar to those at the Lower 9th Ward were generally indicated to have "Medium Erodibility – III" characteristics. Wibowo et al (2007) performed in situ 'jet tests' on levee soils in the Greater New Orleans area as part of the IPET investigation (IPET 2007). The results of the erosion 'Jet Index' tests for the New Orleans East levee sites (Michoud Canal and Slip sites) are summarized in Figure 71. The soil boring and testing data on the surface soils at these locations indicates they are similar to those at the Lower 9th Ward (IPET 2007).



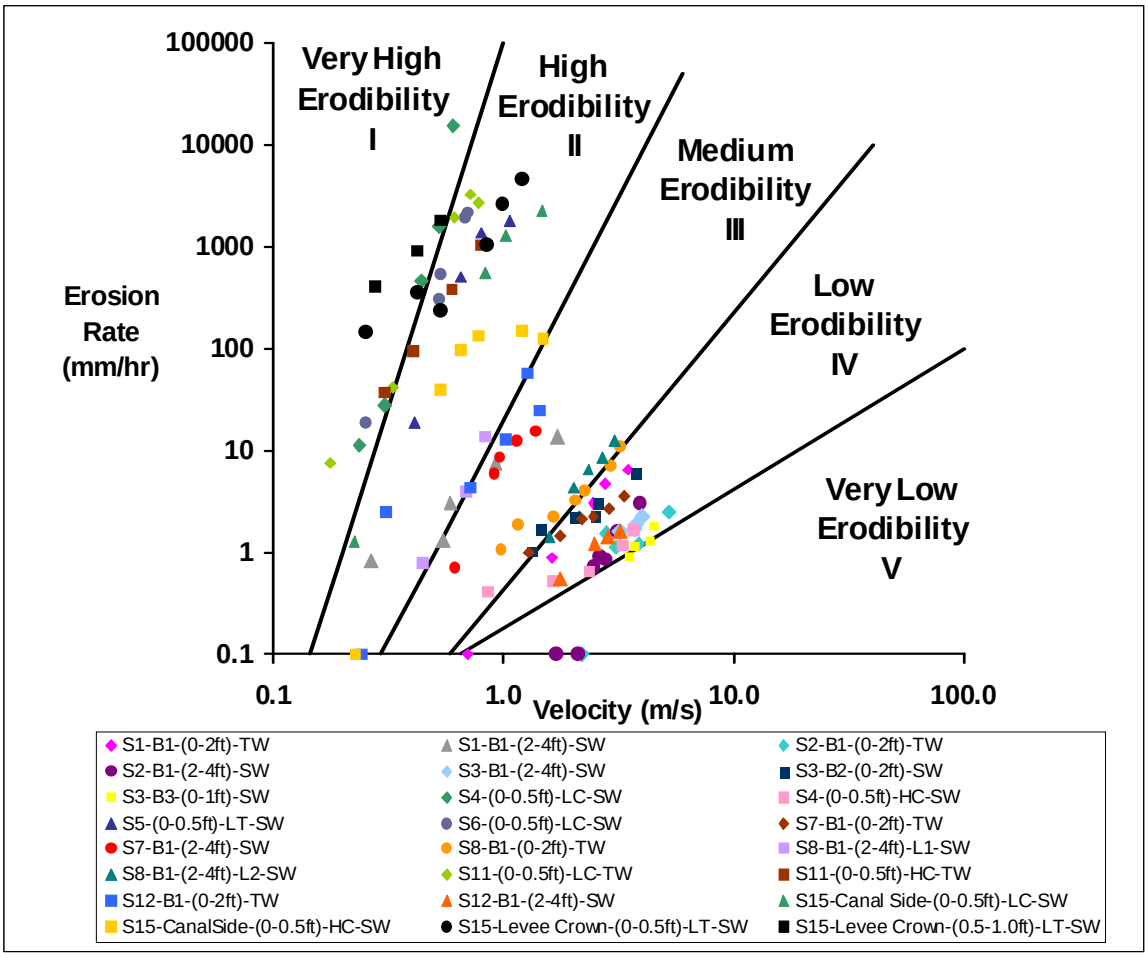


Figure 70: Sampling locations and erosion test results (ILIT 2006, Briaud et al 2008).

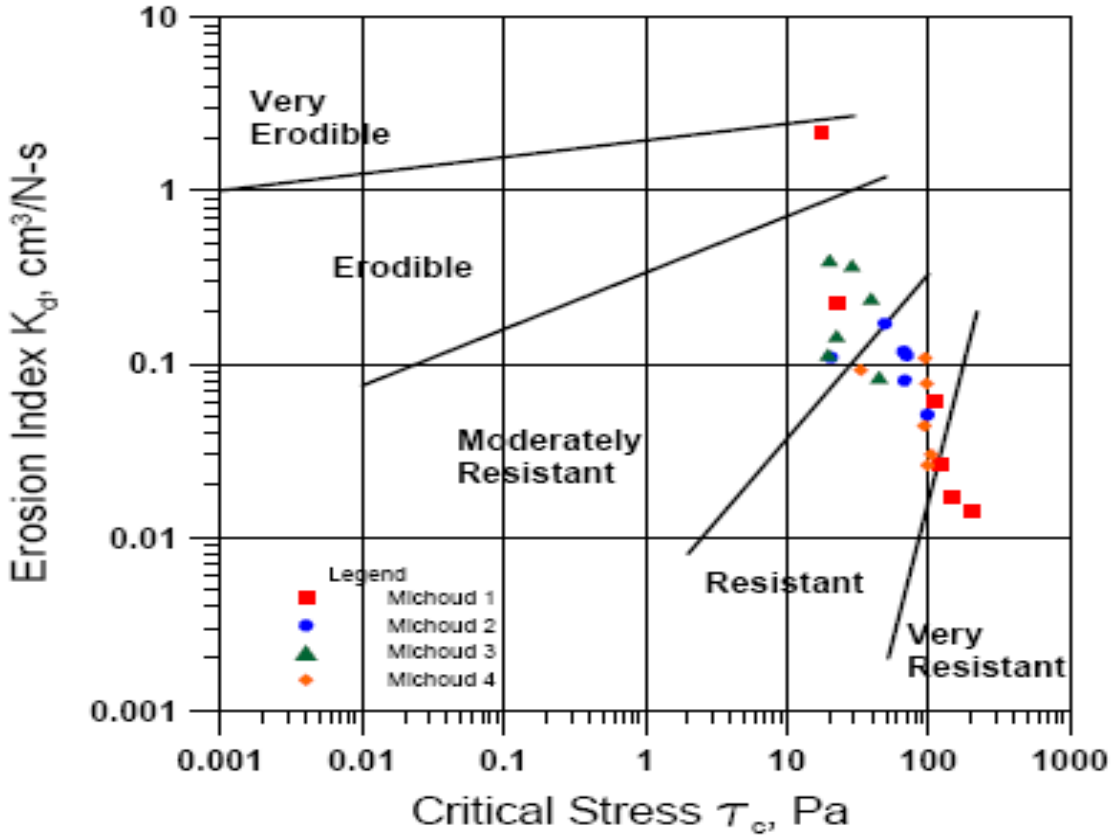


Figure 71: 'Jet Index' erosion test results for New Orleans Est levee sites (Michoud Canal and Slip sites) (IPET 2007).

99. For an overtopping flow period of 1 to 2 hours, application of the three methods to the soil and overtopping flow conditions at the Lower 9th Ward resulted in erosion depths of: Method 1 (Briaud et al) 6 to 8 feet, Method 2 (FHWA) 6 to 9 feet, and Method 3 (Hanson et al) 4 to 8 feet. The three methods indicated very comparable erosion depths. These 'hindcast' erosion depths are in good agreement with those observed in the vicinity of the South Breach (Figures 63 – 67). They indicate that overtopping flow conditions must have existed for 1 to 2 hours at the Lower 9th Ward during hurricane Katrina.

100. Finite element analyses (PLAXIS) were performed to evaluate the lateral stability of the floodwall for the erosion trench lateral failure mode (Figure 72) (Seed et al 2008). Erosion trench depths in the range of 6.5 to 8.0 feet were evaluated. The results showed that an erosion trench with a depth of 8 feet at the inboard side would be required to cause excessive leaning of

the floodwall for a surge elevation of +14 feet NAVD 88. However, the results showed that for the floodwall excessive leaning condition, it still had a factor of safety against instability of 1.1 (Seed et al 2008).

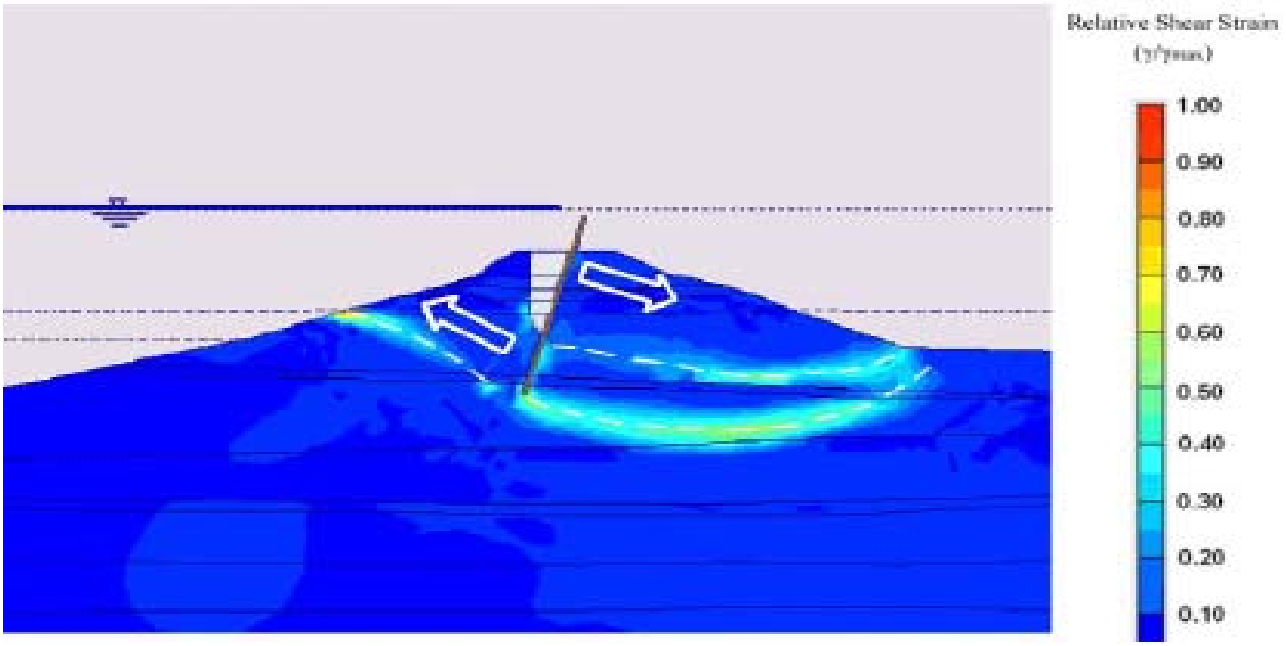


Figure 72: Finite element analysis results showing the stable 'cradling' of a partially toppled I-wall and supporting sheet piling at the South Breach for a water level of +14 feet NAVD 88 (factor of safety against instability = 1.2) (Seed et al 2008).

101. These finite element analysis results did not include the under seepage-induced pore water pressures and hydrostatic uplift pressures that have been summarized herein and detailed by Bea and Cobos-Roa (2008). The comparable finite element results for these conditions are shown in Figure 73. These results are for a surge water level at elevation +13.5 feet NAVD 88 and a horizontal hydraulic conductivity of Kh = 10^{-5} cm/s. Finite element analyses for a higher horizontal hydraulic conductivity of Kh = 10^{-4} cm/s indicated the floodwall – sheet pile – levee system would develop a factor of safety of unity (FS = 1) at a water elevation of +13 feet NAVD 88 (Figure 74). Thus, the results indicate that the lateral instability of the entire floodwall – supporting soil levee system was the most likely mode of failure. This mode of failure was due primarily to the surge hydrostatic pressures (including those developed below the

soil on the unprotected side – the water in the gap between the sheet piling and soils) and the under seepage-induced pore water pressures and hydrostatic uplift pressures.

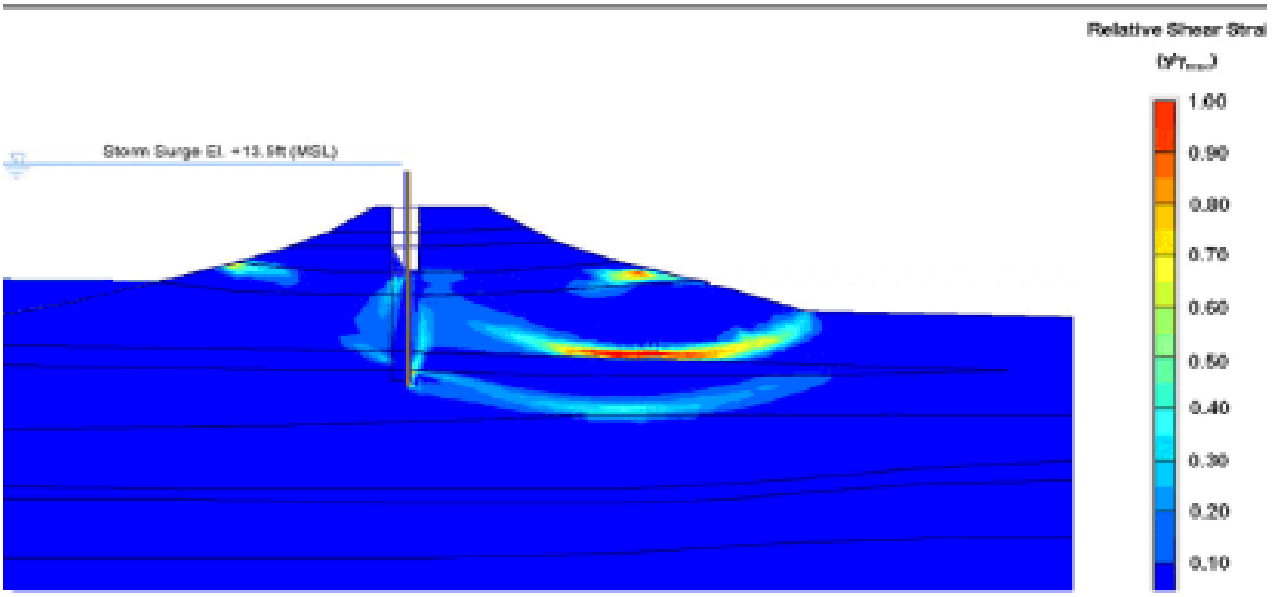


Figure 73: Finite element analysis results showing the two critical potential failure surfaces for the South Breach with the surge water level at elevation +13.5 feet NAVD 88 for pore water and under seepage conditions based on a hydraulic conductivity Kh = 10^{-5} cm/s (Seed et al 2008).

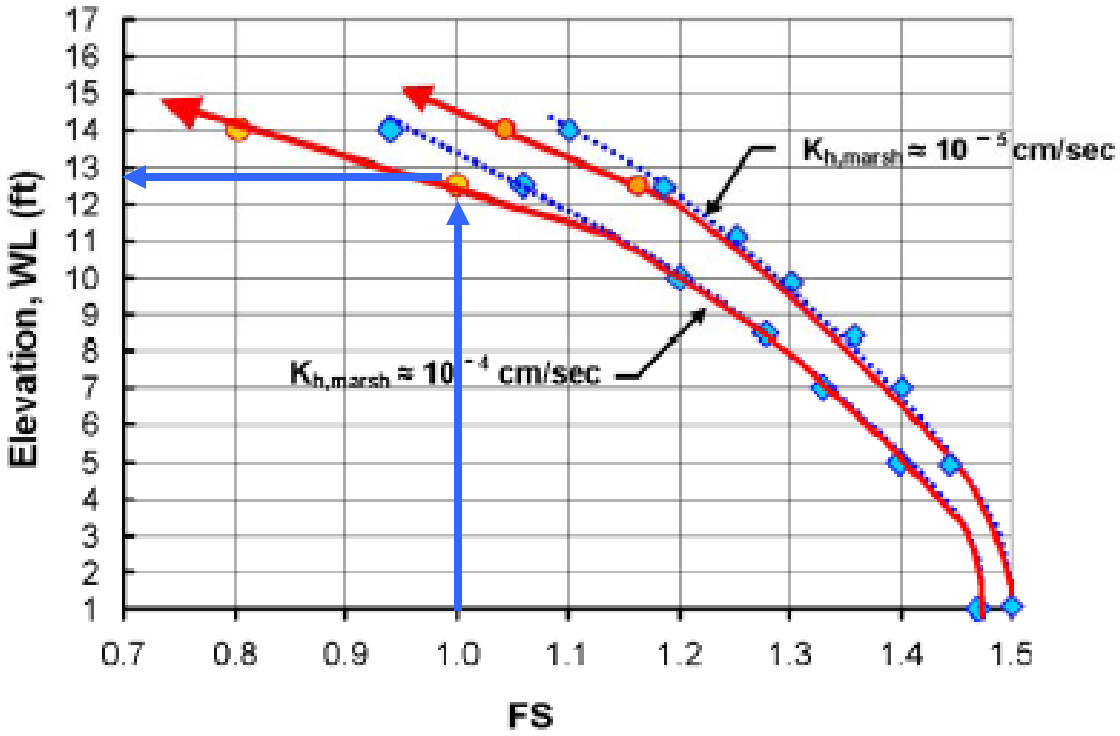


Figure 74: The Factor of Safety (FS) for lateral instability as a function of surge water elevations at the South Breach (Seed et al 2008). The Factor of Safety of unity is developed at a water elevation of +13 feet NAVD 88.

102. This conclusion is corroborated by observations of the performance of comparable I-walls at other locations in the Greater New Orleans Area (ILIT 2006). Figures 75 and 76 are photographs of examples at sections of the Citrus Back Levee floodwall near the Bulk Loading Facility on the GIWW. Approximately 2,000 feet of this wall was displaced and severely tilted toward the protected side (Figure 77). Figure 75 shows a very deeply eroded trench on the inboard of the floodwall at one of the deepest sections observed behind these types of walls in the Greater New Orleans area. The wall leans inboard slightly and the lateral wall deflection resulted in heaving up of the soil against the sheet piles at the base of the floodwall which stabilized the wall in the partially displaced position. Figure 76 shows a second long reach of this Citrus Back Levee floodwall this time viewed from the unprotected side where the floodwall was pushed laterally to a position cradled by the far side of the trench behind the flood wall. The wall remained stable in that position.

103. Figure 78 shows the pre- and post-Hurricane Katrina I-wall conditions at this location. The top of the floodwall was located at elevation +15 feet and the sheet piling extended to –12.5 feet NAVD 88. Figure 79 shows an elevation survey of the overtopping scour trenches in front of and behind the I-wall. The scour trench was approximately 11 feet deep with the bottom of the trench at elevation –3 feet NAVD 88.

104. A soil boring near the center of this section of the wall (Figure 80) shows that this I-wall – levee section was supported by strong lean and organic clays. The marsh – swamp layers that underlie much of the Greater New Orleans were not present above an elevation of –60 feet NAVD 88. Thus, the natural conduits (permeable marsh and swamp deposits) for the under

seepage pore water pressures and the associated hydraulic uplift forces were not present and could not contribute the instability of the floodwall – sheet pile – levee floor protection system.



Figure 75: Significant lateral deflection of the Citrus Back Levee floodwall, seen from the inboard (protected) side (ILIT 2006). Note the heave adjacent to the displaced sheet piles and wall



Figure 76: Deflection and tilting of another section of the Citrus Back Levee Floodwall, this time viewed from the outboard side. Note the gap between the outboard levee toe section and the sheetpile curtain (ILIT 2006).



Figure 77: Aerial photograph of severely damaged Citrus Back Levee flood wall at the Bulk Loading Terminal Facility (IPET 2007).

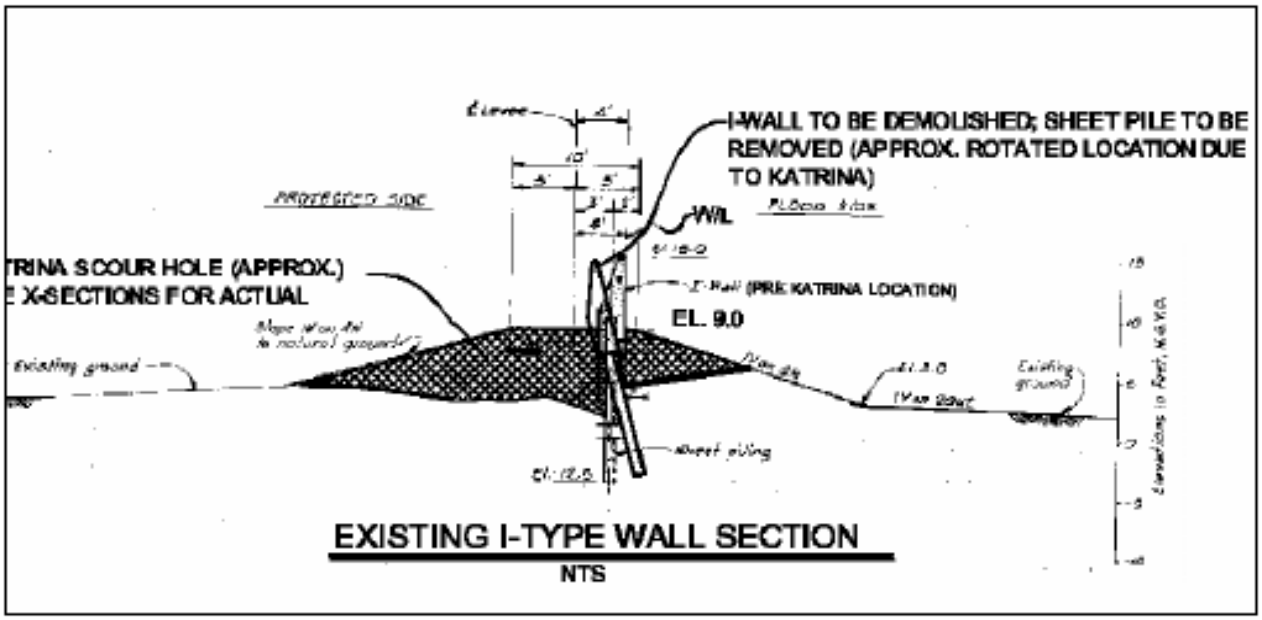


Figure 78: Pre- and post-Hurricane Katrina conditions of severely damaged Citrus Back Levee flood wall at the Bulk Loading Terminal Facility (IPET 2007).

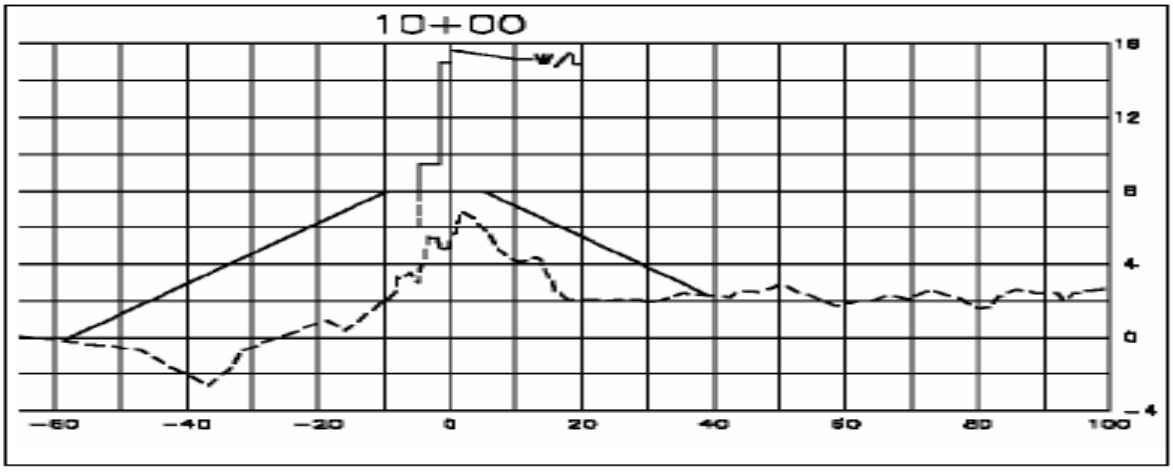


Figure 79: Overtopping scour pattern in front of and behind severely damaged Citrus Back Levee flood wall at the Bulk Loading Terminal Facility (IPET 2007). Scour trench has maximum depth of 11 feet with bottom of trench at –3 feet NAVD 88. The sheet piling penetrate to –12.5 feet NAVD 88.

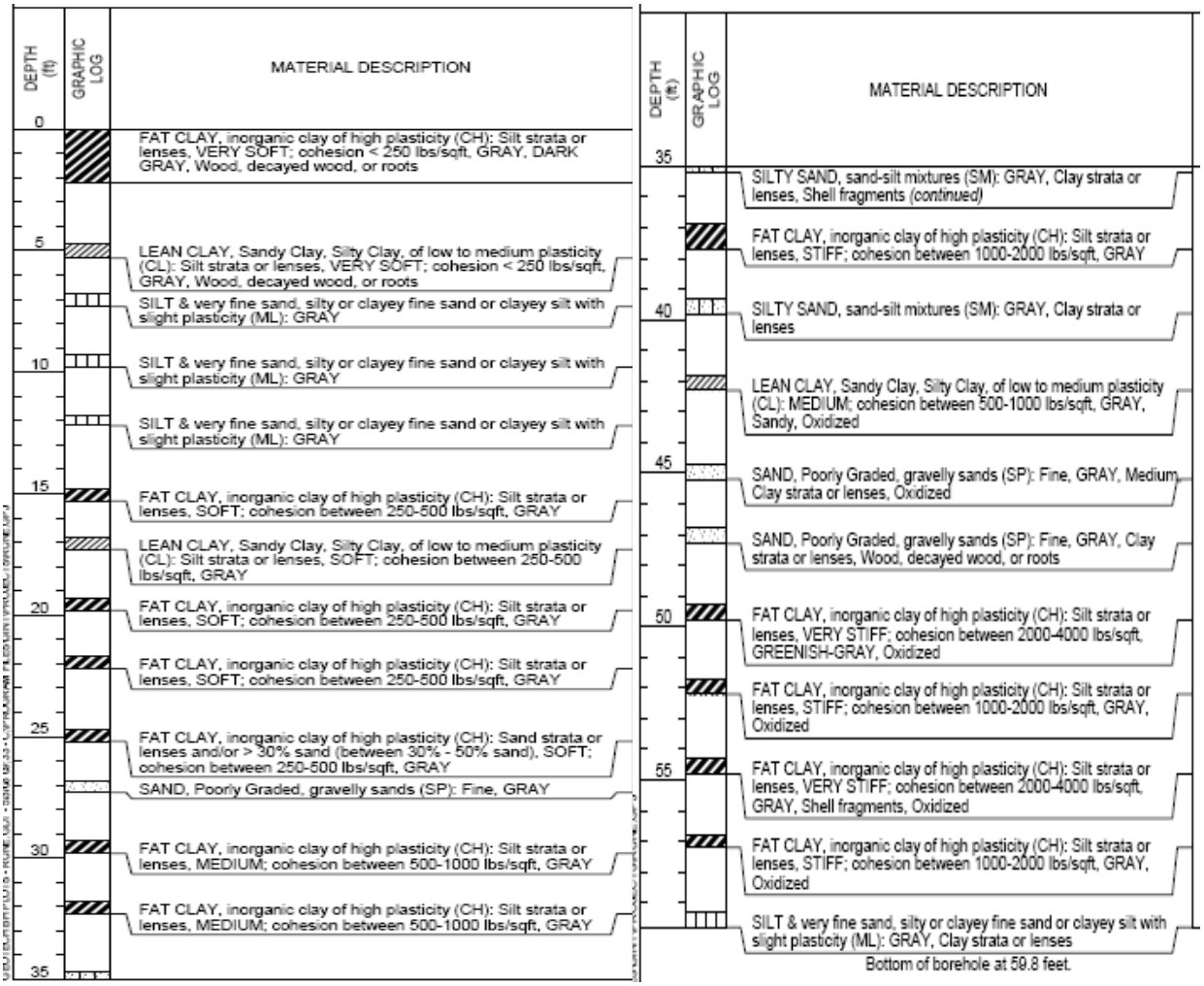


Figure 80: Nearby soil boring log shows top layers of clay interbedded with silt and sand seams to elevation –60 feet NAVD 88 (ILIT 2007).

Summary of Analytical Results

105. Marsh permeability plays a key role in the hydraulic behavior and instability of the breaches along the East levee of the IHNC-Lower Ninth Ward. Because of the inherent uncertainties associated with the Marsh permeability a wide range of permeabilities (10^{-3} to 10^{-6} cm/s) were used to characterize the Marsh deposit and compute the response of the levee and underlying materials.

106. The South and North breaches were modeled using two-dimensional finite element and limit equilibrium analyses and incorporated two main differences with published analyses (ILIT, IPET). The first was the presence of excavations on the EBIA side of the levee - floodwall. Previous analyses did not include the effect of the very deep excavations that took place along the EBIA during the 1990s. The influences of the EBIA excavations on the levee response to the storm surge is that it brings the IHNC effectively to the waterside toe of the levee. The 100 feet to 200 feet between the levee and the shore of the canal were effectively reduced to a couple of tens of feet, meaning that the full hydraulic load of the surge in IHNC will be directly applied below the toe of the levee. The second was generation of pore pressure and associated seepage through the waterside gap formed between the sheet pile and the embankment as the surge rose in the IHNC.

107. The seepage analyses show that pore pressures develop quickly and near-steady state conditions are reached within less than 30 hours of the initiation of the surge starting on August 28, 2005. Calculated pore pressures and hydraulic gradients are within 20 % to 30% of those computed for a hypothetical steady state at the water elevation at time of failure (between +9 feet and 10 feet for the North Breach, and between +12 feet and +14 feet for the South Breach). Results from current studies indicates that the two-dimensional seepage analyses will

produce conservative results compared with the three-dimensional conditions – the two-dimensional analyses will produce underestimates of the hydraulic effects.

108. The different relative permeabilities considered between Marsh and surrounding soils yield differences of pore pressure in the weaker soil layers that control lateral stability (e.g. Marsh layers). Higher permeabilities in the Marsh layers give slightly larger pore pressures, and consequently reduce the effective stresses that control soil shear strengths. Another factor influenced by increased pore pressures is the potential for "blowout" failures. This type of failure can be produced by uplift pressures that are equal or larger than the in-situ overburden stresses; the case was evidenced more critically at the North breach, at the thin clay layer located on the landside toe of the levee.

109. Sudden water loads on embankments impose pore pressures inside the levee and its foundation soils, generating excess pressures along fine-grained blankets and deposits with relatively low permeability. A slope stability analysis using an assumed phreatic (water table) surface (even if it were drawn correctly) would underestimate the pore pressures acting on levee and underlying soils. It is clear that seepage analyses (steady state or transient) are needed to enable more realistic characterizations of the development of pore pressures and consequently slope stability.

110. The analytical results developed as a result of this study differ from the ILIT (2006) and IPET (2007) results. The main difference is that in the studies reported here the transient seepage model considered the gap being opened and transporting water heads directly to the back of the sheet-pile, thus applying "instant" pressures at the tip of the sheet-piles. This is why more than 80% of the steady state pressures are developed within the system, compared with the 60 to 70% calculated by IPET in their seepage analyses. Seepage modeling performed

by ILIT did not consider the formation of the gap in the subsurface soil seepage and pressure calculations (it was only considered as a lateral force for stability calculations).

111. The breaches along the East side of the IHNC-Lower Ninth Ward during Hurricane Katrina were influenced in major ways by the EBIA excavations performed by WGI for the USACE IHNC navigation Lock Expansion Project. These excavations resulted in facilitating significant increases in the hydraulic conductivity and porewater pressure development paths. This led to much quicker and greater increases in seepage pressures and hydraulic gradients and led to decreases in the lateral resistance of the levee – floodwall sections.

112. Due to the large influences of hydraulic conductivity and lower lateral stability of the floodwall – levee section, the North Breach developed well before overtopping. The results summarized herein and detailed by Bea and Cobos-Roa (2008) indicate that the North Breach initiated its development at approximately 5:00 am CDT. These results also indicate that the North Breach completed its development with the surge elevation at +9 to +10 feet NAVD 88 at 6:00 am to 7:00 am CDT. The peak surge of + 14.5 feet NAVD 88 during Hurricane Katrina developed at 9:00 am CDT.

113. The South Breach developed after overtopping. The results summarized herein and detailed by Bea and Cobos-Roa (2008) indicate that the South Breach initiated its development at approximately 5:00 am CDT. These results also indicate that the South Breach completed its development with the surge elevation at +13 to +14 feet NAVD 88 at 7:30 am to 8:00 am CDT.

I declare under the penalty of perjury under the laws of the United States of American that the foregoing is true and correct.

Executed on July 11, 2008 in Moraga, California.



Robert Bea, Ph.D, PE

IV. APPENDIX A MARSH – SWAMP DEPOSITS PERMEABILITY – HYDRAULIC CONDUCTIVITY

1. During performance of the ILIT soil borings at the sites of the North Breach and South Breach at the Lower 9th Ward the very high hydraulic connectivity of the marsh and swamp layers that underlie this entire area was made apparent (Figure A-1). After completing one boring at the location, the soil boring rig was moved 200 feet away and a second boring was completed. Because of the variable soil characteristics, it was decided to drill a third soil boring midway between the first two borings. When the drilling unit encountered the buried marsh and swamp layers, water immediately began spouting from the two adjacent soil boring holes. Examination of the soil samples showed that the organic layers were very 'layered' – as one would expect from the accumulation of organic matter in the bottom of a marsh or swamp. The deposits were very anisotropic – the horizontal permeability was much greater than the vertical permeability. This observation showed that conventional sampling and laboratory testing methods (e.g. permeability determined from consolidation tests on samples from vertical cores) could not be relied upon to give realistic data regarding the horizontal permeabilities of these deposits.

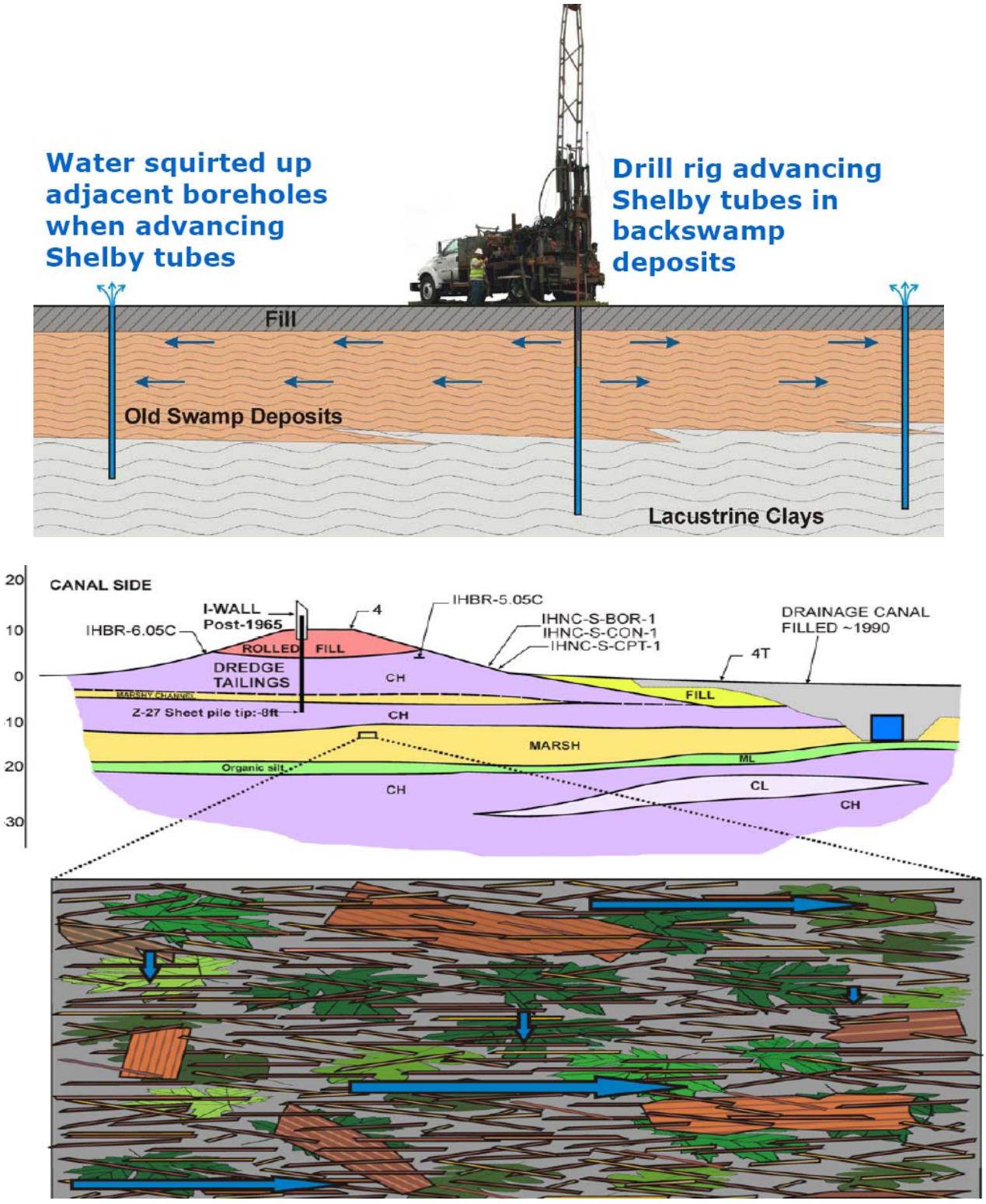


Figure A-1: Soil boring operations at the Lower 9th Ward demonstrated very high hydraulic conductivity of the buried Marsh – Swamp deposits (ILIT 2006, Rogers 2006).

2. Marsh (swamp, wetland, peat, humus) deposits are highly compressible materials that exhibit rapid decrease of hydraulic conductivity (permeability, K) as stresses increase and void spaces decrease (Mesri, 2007). The factors influencing the behavior of such soils depend not only on stresses, but on depositional mechanics, aging and decomposition, amount, type and orientation of fibers and saturation.

3. Hogan et al, (2006) shows field tests performed on a peat-bog fen deposit. Using pumping and slug tests the investigators were able to determine widely scattered permeability values, ranging from 10^{-3} to 10^{-4} centimeters per second (cm/s), but with values as high as 10^{-2} cm/s. The results of the test program showed that permeability decreases with depth. An important result from Hogan (2006) is that by using a load test, the time of dissipation of excess piezometric head occurred in just 2.5 hours, indicating high permeability values.

4. According to Egglesmann (1975), Phragmites peats have the highest permeability values, and these will decrease with time of drainage to a nearly constant value. His statistical investigation of more than 2000 permeability measurements on different peats, concluding that when large water discharges flow through peat, the macropores spaces are transformed into smaller spaces, leading to decreases in hydraulic conductivity and soil deformation (consolidation).

5. Beckwith et al (2003) have developed a new mechanism for extraction of "undisturbed" peat samples for permeability testing. Strong emphasis is made on the need to incorporate several samples at different depths inside a single deposit to capture the anisotropy (ratio of vertical to horizontal permeability, Kv/Kh) and variation of permeability with depth (and consequently stresses). A summary of the results developed by Beckwith et al (2003) is given in Figure A-2. The plots show the permeability (K, cm/s, $\log_{10} K = -1$ and K = 0.1 cm/s),

the ratio of the horizontal to vertical permeability (Kh/Kv, $\log_{10}$ Kh/Kv = 1 and Kh/Kv = 10). The results show very high horizontal permeabilities (Kh) close to the ground surface (dark circles, Kh = 0.1 cm/s) and these values decrease steadly with depth to values of about Kh = 10^{-4} cm/s. The anisotropy reflected in the ratio of Kv/Kh ranges from 1 to more than 1000. It is obvious that results from standard consolidation testing of conventional samples in which the primary drainage paths are vertical can not be relied upon to give valid indications of horizontal permeability in most Marsh deposits.

6. Laboratory methods tend to overestimate values of hydraulic conductivity (Ronkanen et al, 2005), compared with in-situ measurements. The large variation on the permeability values imposes the need for in-situ testing coupled with laboratory testing in order to assess a more realistic value of permeability for the Marsh near the breach site, and analyze the controlling aspects of the hydraulic behavior of this member.

7. Mesri et al (2007) indicate that initial permeability for Marsh deposits can be equal to 1000 times the initial permeability value for soft clay or silt deposits when subject to a fill load of about 10 feet.

8. Permeability tests such as piezometer slug test and cube methods have suggested by some of the above mentioned authors; however, there is no absolute value of the K values used in this study in order to generate a “best estimate” value for this permeability.

9. Future work on the breach site (and non-failure sites) should be carried out to assess the permeability of this complex deposit. Surridge et al (2005) and Clymo (2004) evaluated and compared results from laboratory and field tests, acknowledging the pronounced uncertainties and the fact that different organic deposits (swamp, marsh, peat, humus) have very different hydraulic properties, and at the moment, a generalization is not possible.

10. Table A-1 summarizes the permeability values computed and/or tested by field and laboratory methods found in the researched publications.

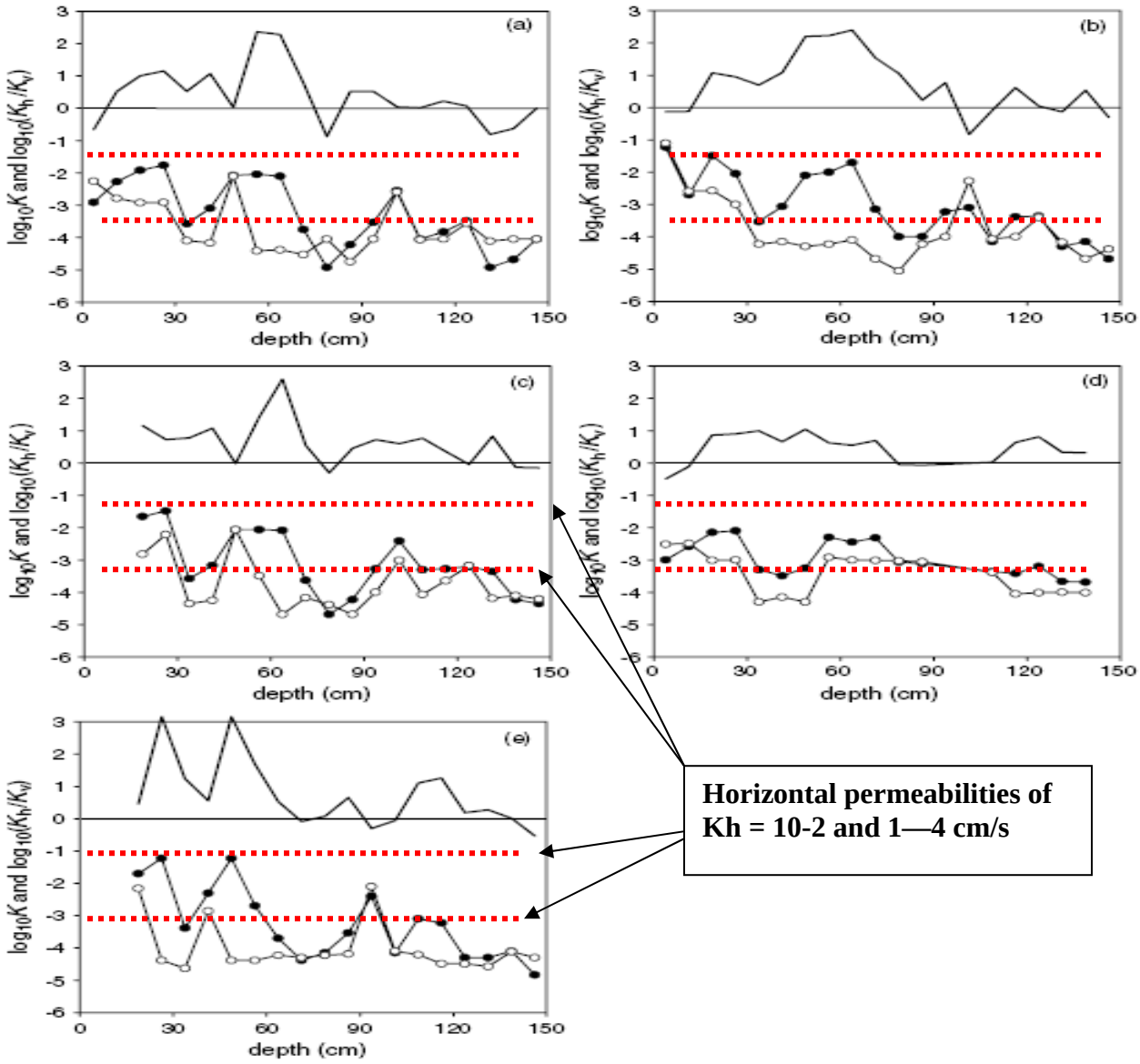


Figure A-2: Summary of permeability tests performed on Marsh deposits (Beckwith et al 2002). Red dotted lines indicate horizontal permeabilities of 10^{-2} and 10^{-4} cm/s.

Table A-1. Summary of permeability values.

Author	K-range (cm/s)		Observations
	Minimum	Maximum	
Hogan et al (2006)	10^{-4}	10^{-2}	K Varies with depth and peat type
Egglesmann (1975)	10^{-4}	$8x10^{-3}$	Highly dependant on drainage time
Beckwith et al (2003)	$2x10^{-3}$	$2x10^{-2}$	Calculated high anisotropy ratios and variations with depth
Clymo (2004)	10^{-6}	$2x10^{-5}$	Used falling head piezometers
Ronkanen et al (2005)	10^{-5}	0.30	Vertical conductivity was usually higher than horizontal

11. When selecting high permeability values for the Marsh deposit (e.g. 10^{-3} cm/s), this unit acts as a draining layer, conducting pore water pressures much faster beneath the levee. The effect of these values does not significantly increase the calculated hydraulic gradients on the levee toe, but it allows the potential for underseepage and water exiting at the levee toe.

12. If the Marsh is considered to have low permeabilities (10^{-5} cm/s and lower), the system now behaves as a blanket as described by the USACE EM 1110-2-1913 (2000). The principle of the blanket theory is that pore water pressures underneath an embankment carried by an underlying aquifer will impose pressures in excess of hydrostatic conditions that can cause uplift and potential for blowout failures and sand boils.

13. Modeling the Marsh deposit as a “draining” layer or as a “blanket” has different impacts on the performance of the system. Both can be negative. First, if the Marsh is considered a permeable layer, pore water pressures will develop faster beneath the levee toe and can accelerate the material softening and/or produce internal erosion.

14. Second, when modeled as a low permeability material, the deposit generates excess pore water pressures by being recharged from below by the aquifer, and might experience uplift pressures that can lead to failure. For the two breach sites at IHNC, this turned out not be

the critical condition, since the marsh is underlain by a thick clay deposit which would also become part of the blanket if the Marsh were to be modeled as a fine grained material, thus the calculated hydraulic gradients would be small.

15. The models incorporated all these conditions by performing transient seepage analyses for a range of permeabilities. The results show the that pore water pressures rise inside the Marsh deposit to values close to those of a steady-state condition (more than 80%).

16. Analyzing the seepage analyses performed by IPET, on Appendix 17 of their Final Report, the best estimate for the permeability of the Marsh deposit is 10^{-4} cm/s. According to the plot on Figure V-17-15 (Figure A-3), this permeability occurs when the soils are subject to overburden effective stresses from 0 to 500psf. This is a likely range for the shallow layer located at the toe of the levee, but the deeper Marsh underneath the levee is probably subject to effective stresses in the order of 0 to 1000 pounds per square foot (psf), with permeability of 10^{-5} to 10^{-7} cm/s.

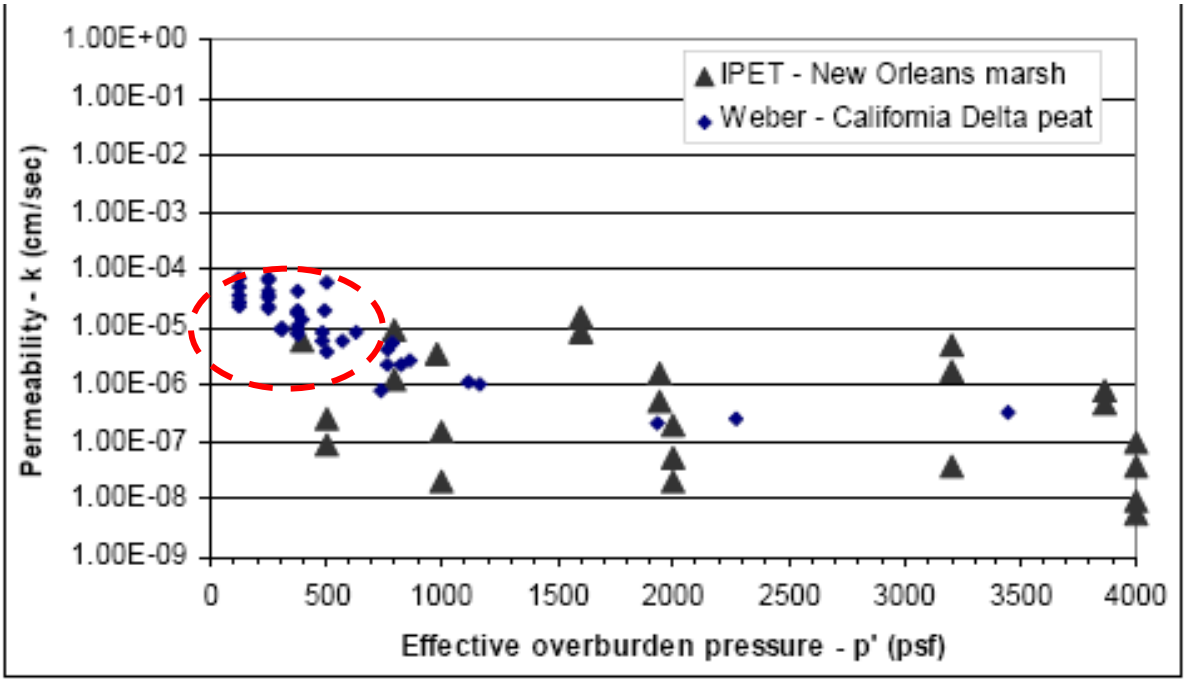


Figure A-3: Hydraulic conductivity values for Peat. The range of effective stresses for the shallow (toe) peats ranges from 0 to 1000psf (red circle). Source: USACE IPET.

17. This graph shows a trend that stabilizes somewhere between 10^{-6} and 10^{-7} cm/s as effective stresses increase; but the values reported by Weber (1969) in the California Delta peat seem higher with low stress levels; this might be due to the fact that Weber performed field tests (piezometers) and this type of test captures the "macro" effects of large particles and irregularities of the layer, such as vegetal matter, wood and roots. The laboratory consolidation testing performed by IPET was on smaller scale samples which are not able to account for these heterogeneities; and -obviously-, the "macro" permeability of the deposit is what controls the hydraulic behavior of the member.

18. Recently, as part of an assessment of levee vulnerability in the Sacramento Delta area, URS (2007) have performed a review of available data and information on the permeabilities of Marsh / Peat layers that underlie the levees in this region. This evaluation included the work cited by the USACE (Weber 1969). This work resulted in the following conclusions (URS 2007, p. 44): "*Consistent with its depositional history, the team believes that the horizontal permeability (kh) of peat/organic soils will generally be higher than the vertical permeability (kv), especially if the peat is in a 'free-field' condition , away from the consolidating loads of a constructed levee. Therefore, for the initial evaluations, the Levee Vulnerability Team recommended using an anisotropy (kh/kv) of 10 with a kh of* $1x10^{-4}$ *and a kv of* $1x10^{-5}$ *cm/s, for 'free-field peat/organic soil." "As expected, due to the consolidating impact of the weight of a constructed levee, peat/organic materials lying beneath the levee showed lower permeability than the free field peat. Therefore, for the initial analyses, the levee* vulnerability *evaluation team recommended using horizontal and vertical permeability values of*

$1x10^{-5}$ and $1x10^{-6}$ cm/s respectively. This is one order of magnitude lower than the permeability of the free field peat/organic soils."

19. Based on available information on the buried Marsh deposits under the Lower 9th Ward levees and floodwalls, it was not possible to define a single value for the intrinsic hydraulic conductivity – permeability. For this reason, the effects of a wide range of plausible permeabilities on seepage and hydraulic effects and horizontal stability of the levee – floodwall were evaluated in performing the floodwall – sheet piling – levee flood protection system.

V. REFERENCES

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