Declaration No. I.

ENGINEERING FORENSIC STUDIES OF PERFORMANCE OF THE MAN-MADE FEATURES BORDERING THE REACH 2 OF THE MISSISSIPPI RIVER-GULF OUTLET (MR-GO) 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

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DECLARATION NO. I

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 along the Mississippi River- Gulf Outlet (MR-GO) St. Bernard Parish Reach 2 frontage. 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. Sections II and III provide a summary of forensic engineering observations and analyses to determine the causes of the breaches, failures, and overtopping that developed with respect to the man-made features along the MR-GO St. Bernard Parish Reach 2 frontage during and following Hurricane Katrina. Section IV summarizes the major conclusions drawn from this work. Section V provides a list of the references cited in this Declaration. The following table of contents will serve as a guide to help reading this Declaration.

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I. QUALIFICATIONS

3. 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.

4. 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 seventeen professional conference and journal papers (4 in final submission) that have documented this forensic work.

5. I am a professor in the Department of Civil and Environmental Engineering, and associate director of the Center 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.

6. 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.

7. 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.

8. 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.

9. In 1977, I was appointed vice president and chief engineer of an international consulting engineering and contracting company (Woodward-Clyde Consultants Ocean Services Division [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.

10. 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.

11. 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). I am associate director of the Center for Catastrophic Risk Management. 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.

12. I have published approximately 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.

13. 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 of advanced methods for risk assessment and management including human, organizational, and institutional factors.

14. 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."

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

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

II. FORENSIC ENGINEERING OBSERVATIONS

DEFINITION OF TERMS

17. As a preliminary matter, I will define terms that are used in my Declaration.

(a) The MR-GO is a man-made navigational waterway approximately 76 miles long. It runs generally on a northwest direction from Breton Sound in the Gulf of Mexico through the parishes of St. Bernard and Plaquemines to New Orleans. The MR-GO joins the

Gulf Intracoastal Waterway (GIWW) near the Paris Road Bridge and runs toward the west to the Inner Harbor Navigational Canal (IHNC, also known as the Industrial Canal). This Declaration will address the performance of man-made flood protection features constructed along one section (Reach 2) of this navigational waterway (Figure 1):

Reach 1: the east-west oriented section running between the IHNC and the confluence of the GIWW and the MR-GO near Paris Road Bridge including the north-south section of the IHNC between the navigation locks and the Port of New Orleans at the "T" intersection with the MR-GO – GIWW.

Reach 2: the southeast-northwest section extending from just east of the Paris Road bridge down through Breton Sound into the Gulf of Mexico including the two navigation structures located at Bayou Dupre and Bayou Bienvenue.

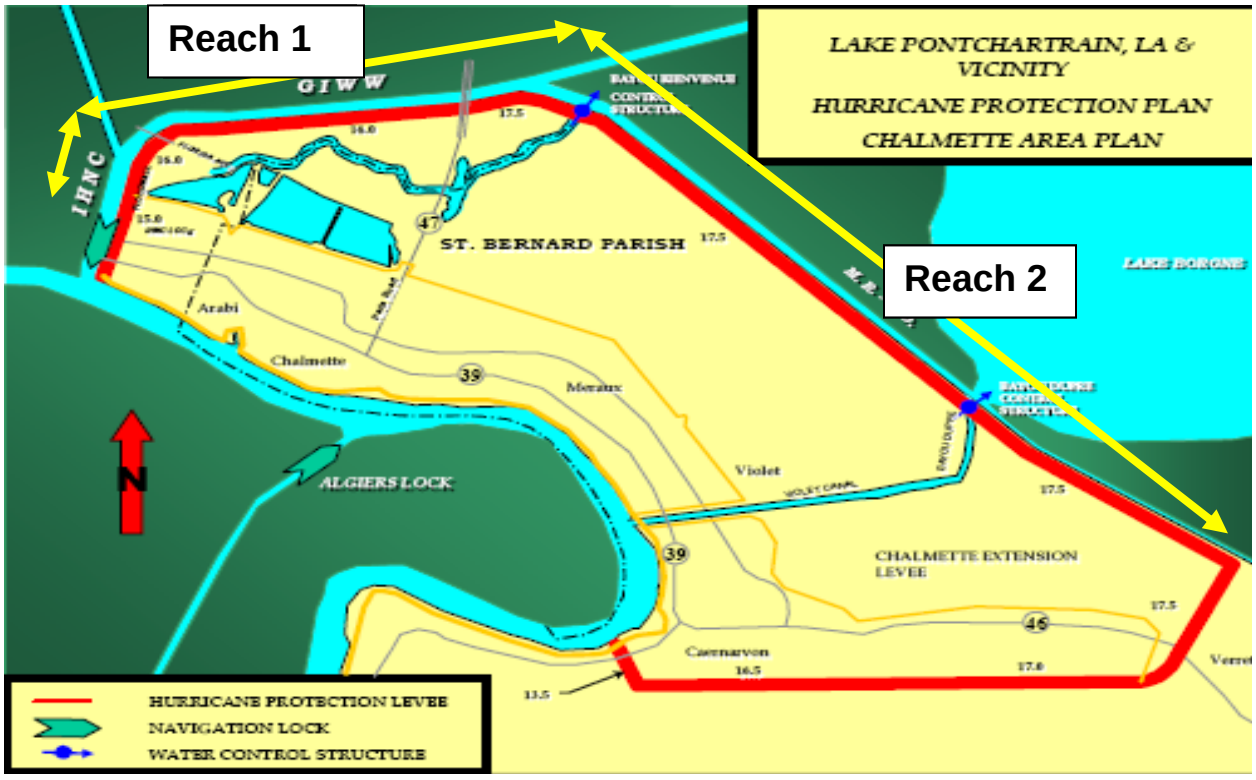


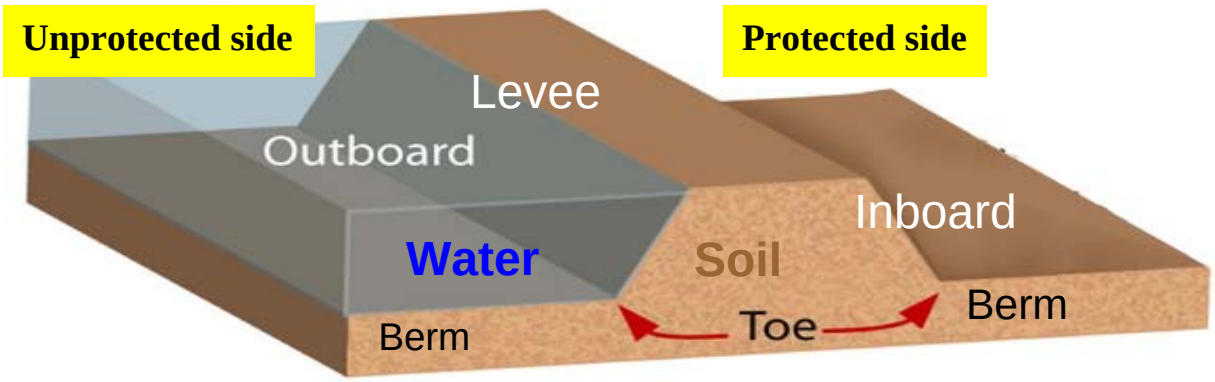
Figure 1: MR-GO Reach 1 and Reach 2 (USACE IPET 2007)

(b) The term “Levee(s)” refers to a Hurricane flood protection structure(s) whose purpose is to protect human population living within urban areas and which are designed and constructed in accordance with pertinent USACE manuals, guidelines, standards, criteria, and policies and/or conform to generally-accepted engineering standards for such man-made features. The “protected side” (back side) of the levee is intended to provide protection from Hurricane flood waters. The “unprotected side” (front side) of the levee is exposed to Hurricane flood waters. As described below, among other things, these man-made features, in order to qualify as “Levees,” must be designed to withstand the specified level of storm surge, waves, and currents that they are intended to contain, and must not be expected to erode or wash away during a Hurricane for which they are intended to serve as flood protection for humans and their property. Towards this end, and as set forth in the USACE publications *Coastal Engineering Manual* (USACE Pub. EM 1110-2-1100) and the *Design and Construction of Levees* (USACE Pub. EM 1110-2-1913), to qualify as “Levees” these man-made structures must be either (a) composed mostly of compacted clay-like cohesive soils, or (b) if constructed of more loosely compacted soils, they must be capped or covered with more dense materials that will protect against the underlying soils being washed away by wave action, storm surge, elevated tides and other water related phenomena associated with Hurricanes (Figure 2). All important potential modes of failure and their interactions must be properly addressed during design, construction, operation, and maintenance for a man-made flood protection structure to be called a Levee (USACE 2000, USACE 1999). As noted below, in my opinion there were no “Levees” along virtually the entire MR-GO St. Bernard Parish Reach 2 frontage before Hurricane Katrina. Instead, this frontage had no effective Hurricane protection since the USACE decided after 1965 to construct man-made

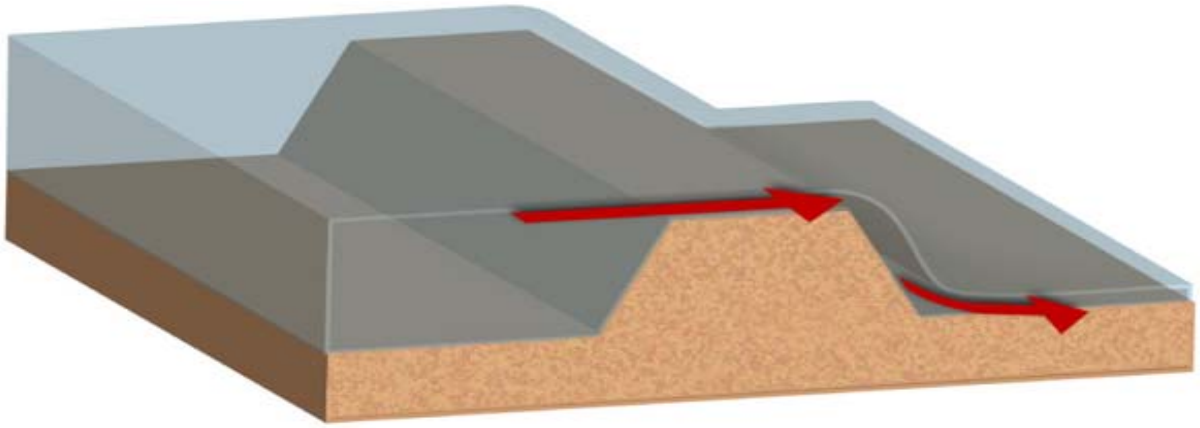
features that were nothing more than miles of embankments comprised of dredged spoil that was not suitable for Levees.

(c) The term "Earthen Berm / Spoil Bank" ("EBSB") refers to an embankment comprised of materials composed of sandy, silty, soil containing organic matter, clays and shells and/or uncompacted hydraulic fill not suitable for constructing hurricane flood protection structures in urban areas as specified in pertinent USACE manuals, guidelines, standards, criteria and policies and/or generally-accepted engineering standards for such man-made features. As noted below, there were only EBSBs along virtually the entire MR-GO St. Bernard Parish Reach 2 frontage before Hurricane Katrina.

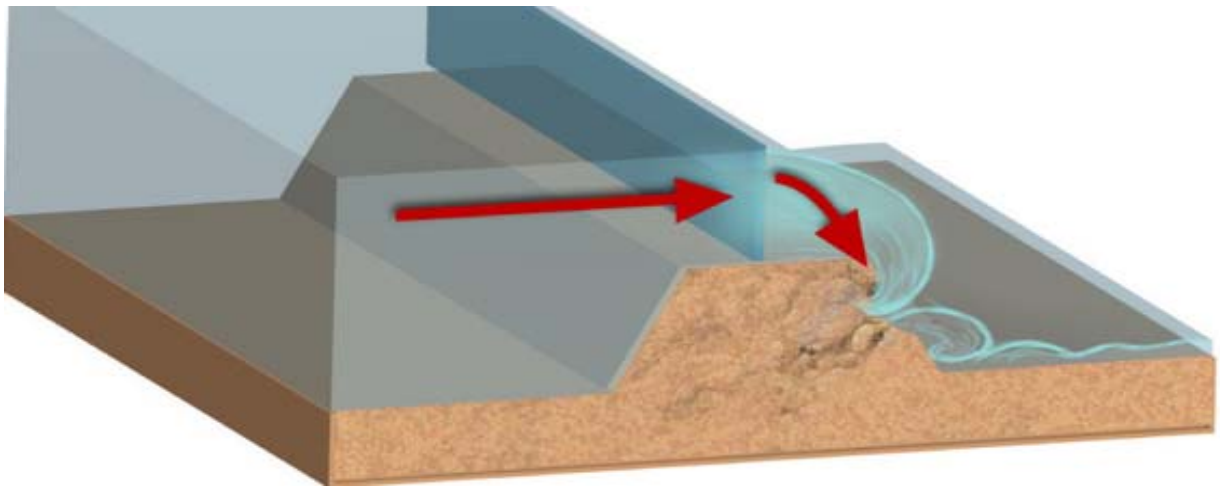
18. As a further preliminary matter, some of the documents, charts, photos, and other exhibits referenced in my Declaration refer variously to "levee(s)," "levee sections," "levee breaches," and so forth. The use of these terms is not intended to suggest or agree that the referenced man-made flood protection features along Reach 2 and portions of Reach 1 of the MR-GO before Hurricane Katrina were in fact "Levees" as I have defined them above. In fact, as detailed below, it is my opinion that the vast stretches of Reach 2 and portions of Reach 1 of the MR-GO and GIWW before Katrina for the most part did not have hurricane flood protection "Levees" as I have defined them, but instead these man-made features were "EBSBs" as I have defined them above and consequently they were not properly designed or constructed to protect human population living within urban areas.



Levee geometry

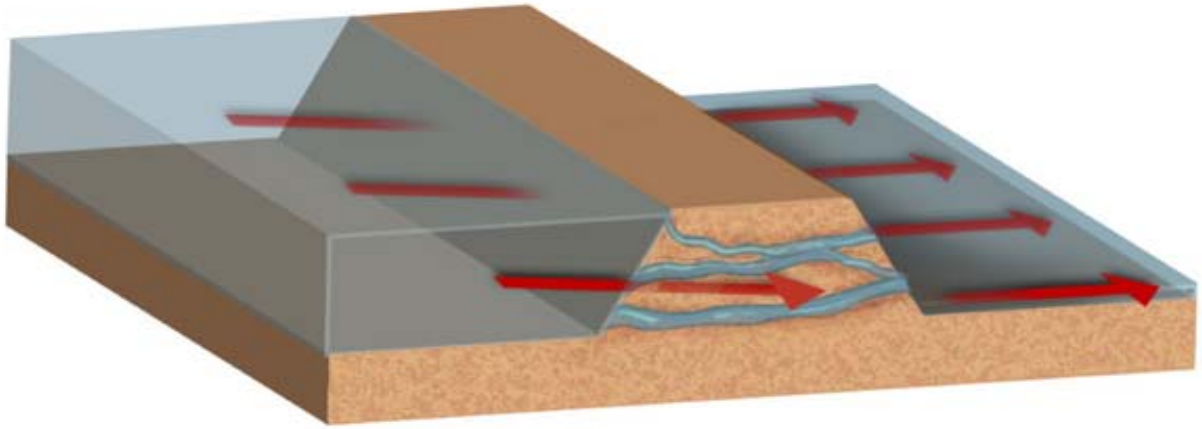


Water over the levee - overtopping

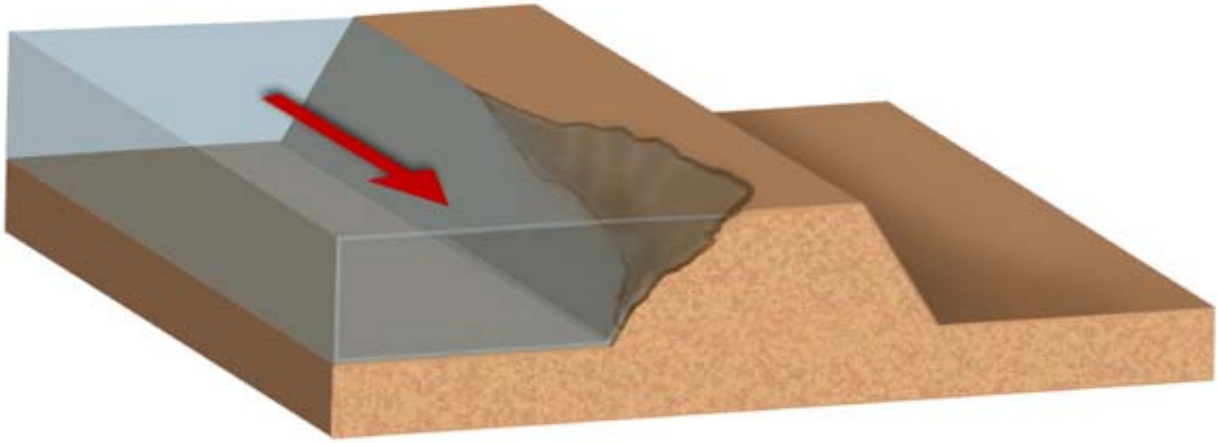


Erosion by water after overtopping

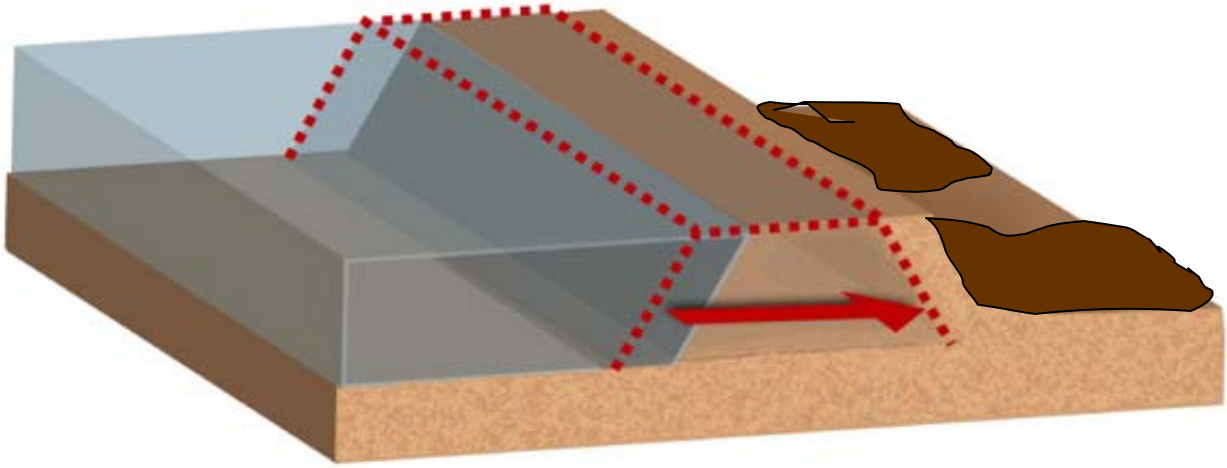
Figure 2a: Levee failure modes (Deretsky, National Science Foundation 2008).



Erosion due to water coming through the levee

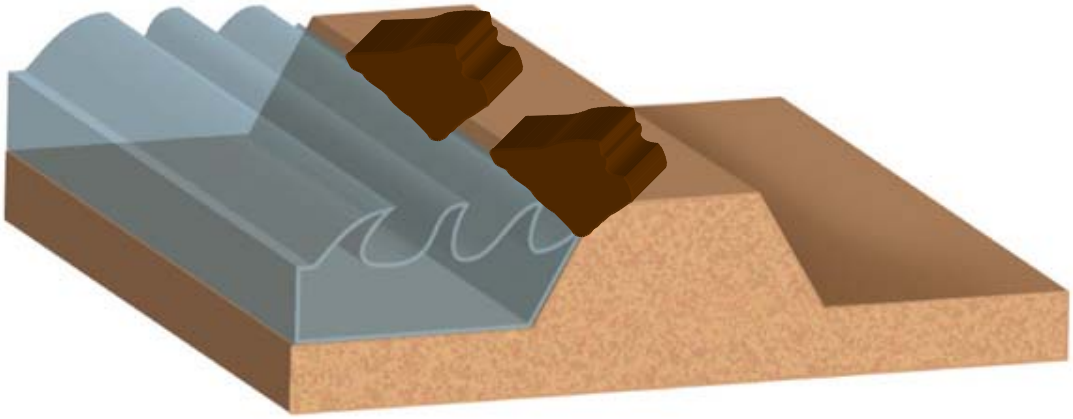


Cu**rrent erosion on outboard levee and berm surfaces**

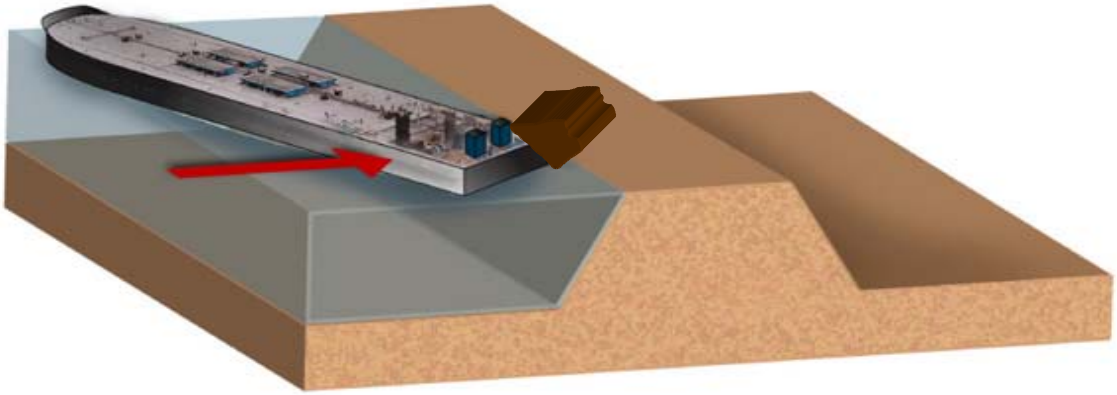


Lateral instability of the levee due to water force

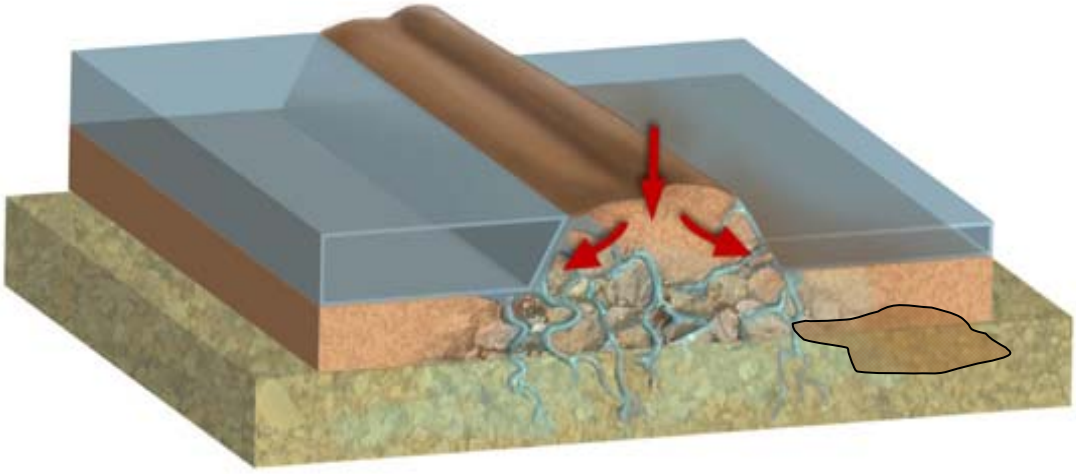
Figure 2b: Levee failure modes (Deretsky, National Science Foundation 2008).



Levee erosion due to wave impacts

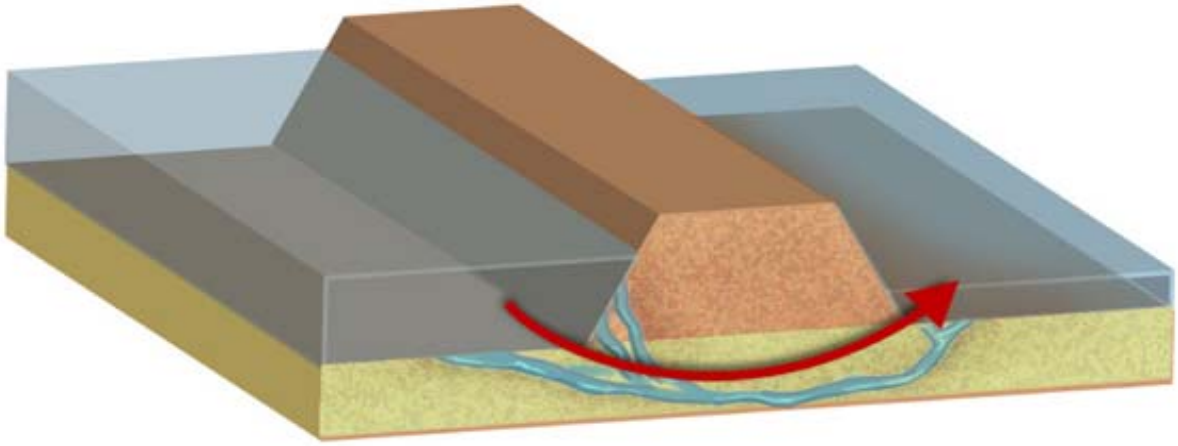


Increased lateral forces and local damage due to mpact from floating objects and debris

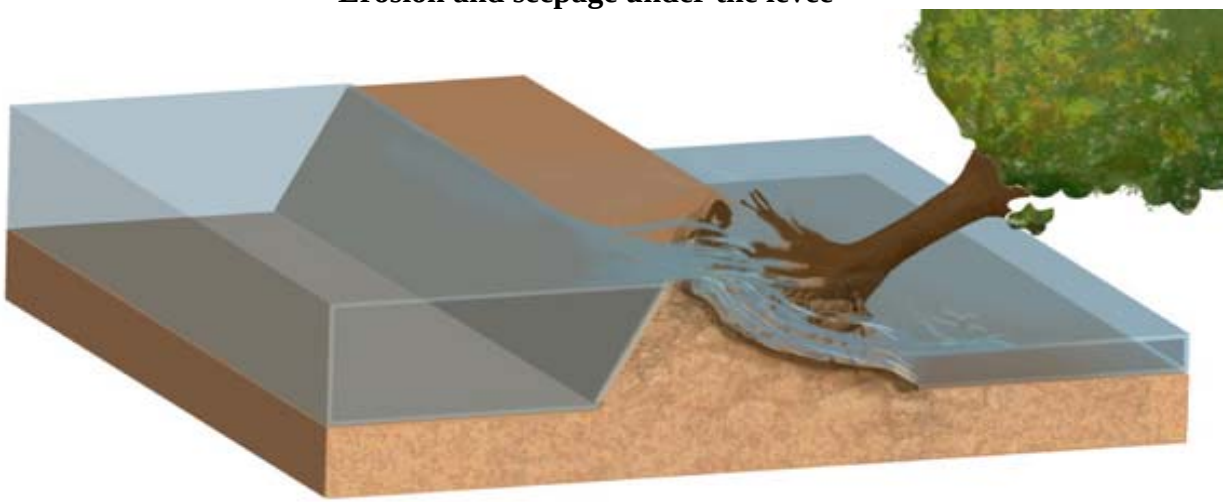


Soil liquefaction due to repeated cyclic forces from waves and currents

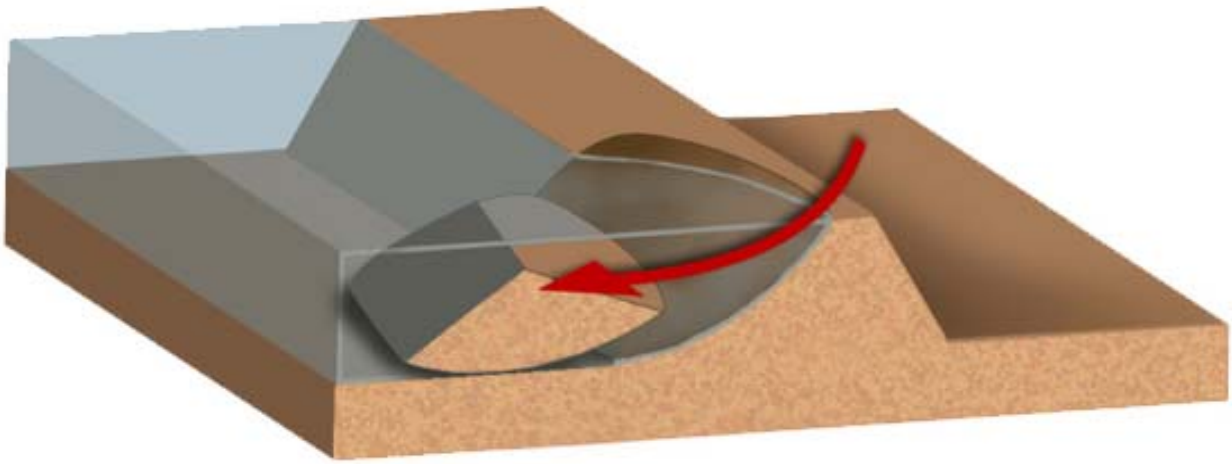
Figure 2c: Levee failure modes (Deretsky, National Science Foundation 2008).



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 2d: Levee failure modes (Deretsky, National Science Foundation 2008).

19. 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 3 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 the MR-GO Reach 2 (identified as 2a and 2b in Figure 3). During the past 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 features along Reach 2 of the MR-GO.

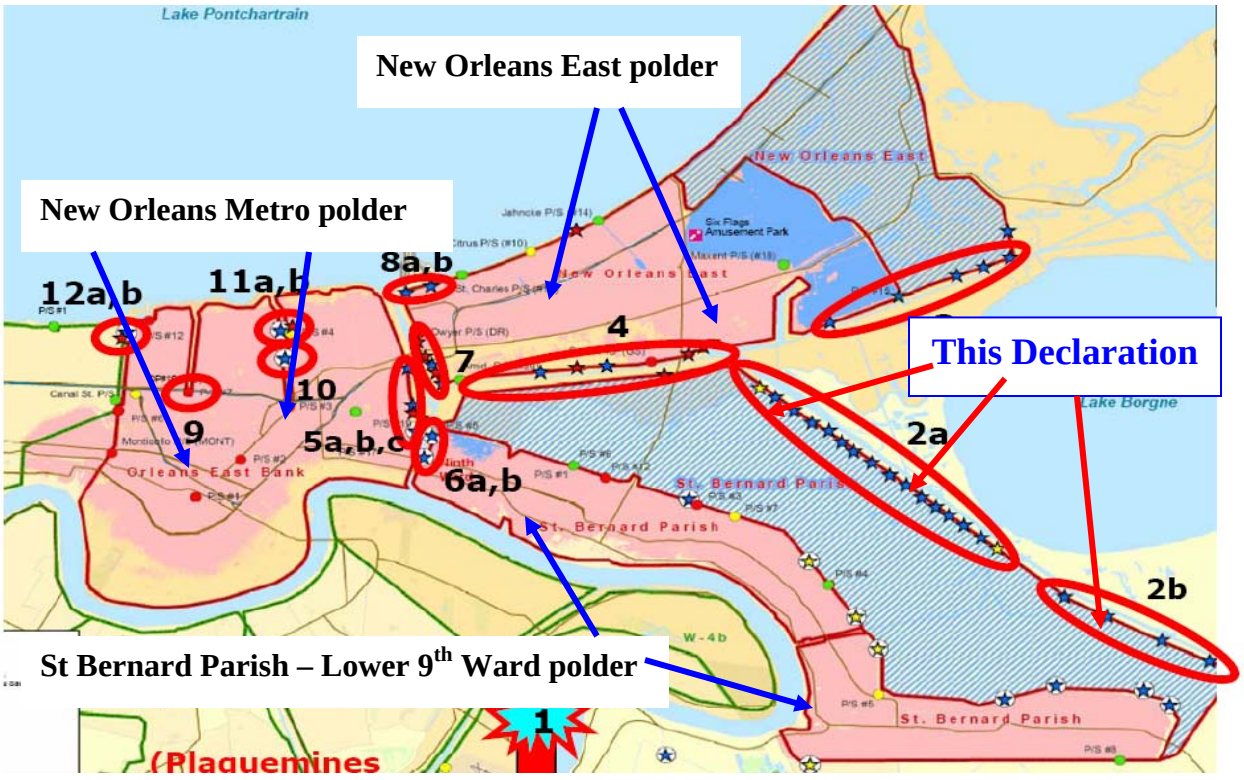


Figure 3: Summary of principal failures, breaches, and distress features and locations of 3 primary polders in the Greater New Orleans area (ILIT 2006).

III. FORENSIC ENGINEERING ANALYSES

MR-GO Reach 2 EBSBs

Field Observations

20. Field investigations performed shortly after the passage of Hurricanes Katrina and Rita showed that the EBSBs fronting Reach 2 of the MR-GO and Lake Borgne and their junctures to the navigation structures at Bayou Dupre and Bayou Bienvenue were catastrophically eroded and breached in much the same way as a sand castle built on an ocean beach is dissolved by ocean waves as the tide comes in (Figures 4 – 6) (IPET 2007, Team Louisiana 2007, ILIT 2006). In addition, two large breaches occurred at the west end of this basin fronting the IHNC (Lower 9th Ward). These Reach 1 Extension breaches are discussed in Declaration No. 2. The result was that the St Bernard – Lower 9th Ward basin flooded extremely rapidly and the in-rushing storm surge drove the waters to an elevation of approximately + 12 feet above mean sea level in this basin. As a result, even homes and businesses located on ground well above sea level were inundated. Sites on lower ground were inundated to greater depths (M. Kok et al, Polder Flood Simulations for Greater New Orleans: Hurricane Katrina August 2005, 2007).



Figure 4: Severely eroded and breached levee section along the northwest frontage of the St. Bernard Parish fronting the MR-GO channel (ILIT 2006). This section of EBSB was analyzed to determine development of wave and surge breaching.



Figure 5: MR-GO levee east of Bayou Dupre (MR-GO is at bottom of photograph) showing breaching initiating with wave bench formation at EBSB mid-height followed by wave overtopping and backside erosion (Team Louisiana 2007).



Figure 6: MR-GO levee between Bayou Dupre and Bayou Bienvenue showing development of EBSB wave bench formation and crown wave breaching leading to backside erosion (USACE IPET 2007). Note the grass scoured by waves at EBSB mid-height – indicating the elevation at which EBSB erosion and scour was initiated.

21. Figures 7 through 13 show the massive breaches that eroded at the contacts between the concrete navigation lock structures at Bayou Bienvenue and Bayou Dupre and the adjacent EBSBs. Deep scour holes were found both upstream and downstream of these breaches.



Figure 7: Breach at south side of Bayou Bienvenue at tie-in of navigation structure with EBSB (EPA 2005). Note shell fill scattered to east and west on the south side banks.

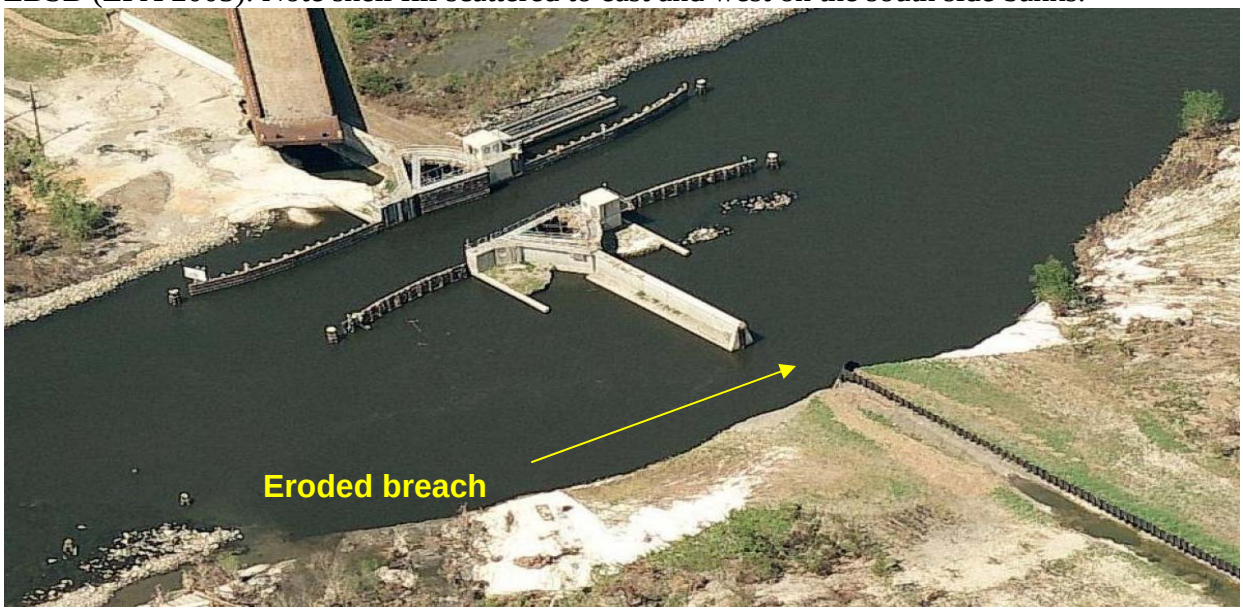


Figure 8: Large eroded breach at the contact between the south end of the concrete navigation lock structure at Bayou Bienvenue and the adjacent EBSB section (ILIT 2006).



Figure 9: Large eroded breach at the contact between the south end of the concrete navigation lock structure at Bayou Bienvenue and the adjacent EBSB section (ILIT 2006). Bent and exposed sheet piling used to compensate for levee settlements.



Figure 10: Large eroded breach at the contact between the north end of the concrete navigation lock structure at Bayou Dupre and the adjacent EBSB (EPA 2005).



Figure 11: Large eroded breach at the contact between the north end of the concrete navigation lock structure at Bayou Dupre and the adjacent EBSB section (ILIT 2006). Center concrete sheet pile panels failed during backflow and were found in the outboard scour hole. Note the very different appearance compared with that in Figure 11.



Figure 12: Large eroded breach at the contact between the north end of the concrete navigation lock structure at Bayou Dupre and the adjacent EBSB section (ILIT 2006) showing dredged spoil used in adjacent EBSB.



Figure 13: Photograph looking at the inboard side from the breach showing eroded shell and sand detritus deposited from the EBSB – Bayou Dupre navigation structure breach (ILIT 2006).

22. It is of particular importance to note that the massive erosion features at the south end of the Bayou Bienvenue navigation lock structure (Figure 8) and at the north end of the Bayou Dupre navigation lock structure (Figure 10) were directly correlated with the courses of the original Bayous (Figure 14). Available information on the soils and light weight shell fill used to fill these bayous and the fill used at the abutments (Figures 15 and 16) indicates high potentials for significant under-base seepage and erosion, and excess pore water pressures acting to reduce soil strength in and under the affected sections of the navigation structures (abutments). These effects would have exacerbated the erosive and scour effects in the area of the breaches.

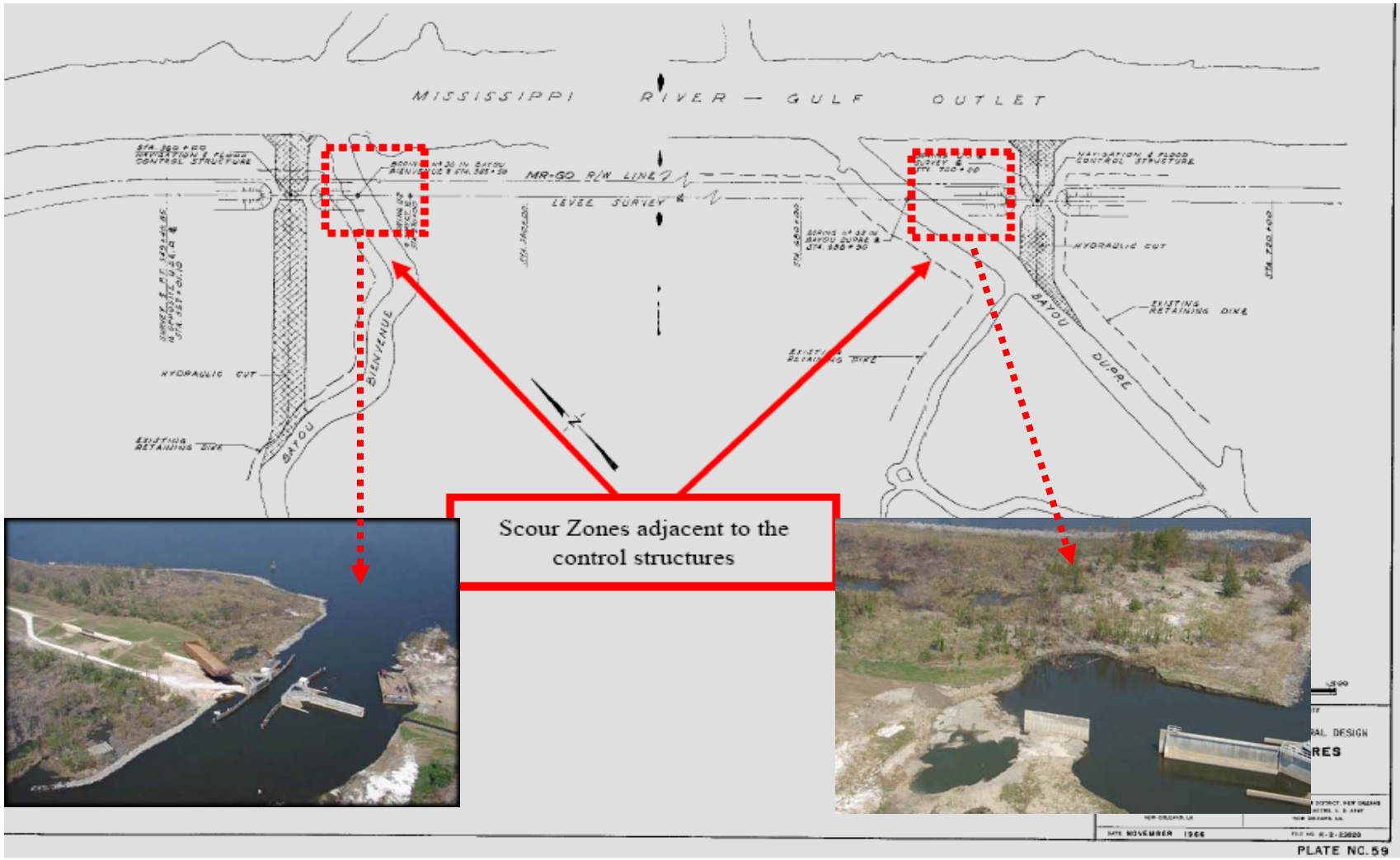


Figure 14: Correlation of areas of severe scour and erosion damage to the abutments of the two navigation structures with alignment of the abandoned original channels (ILIT 2006).

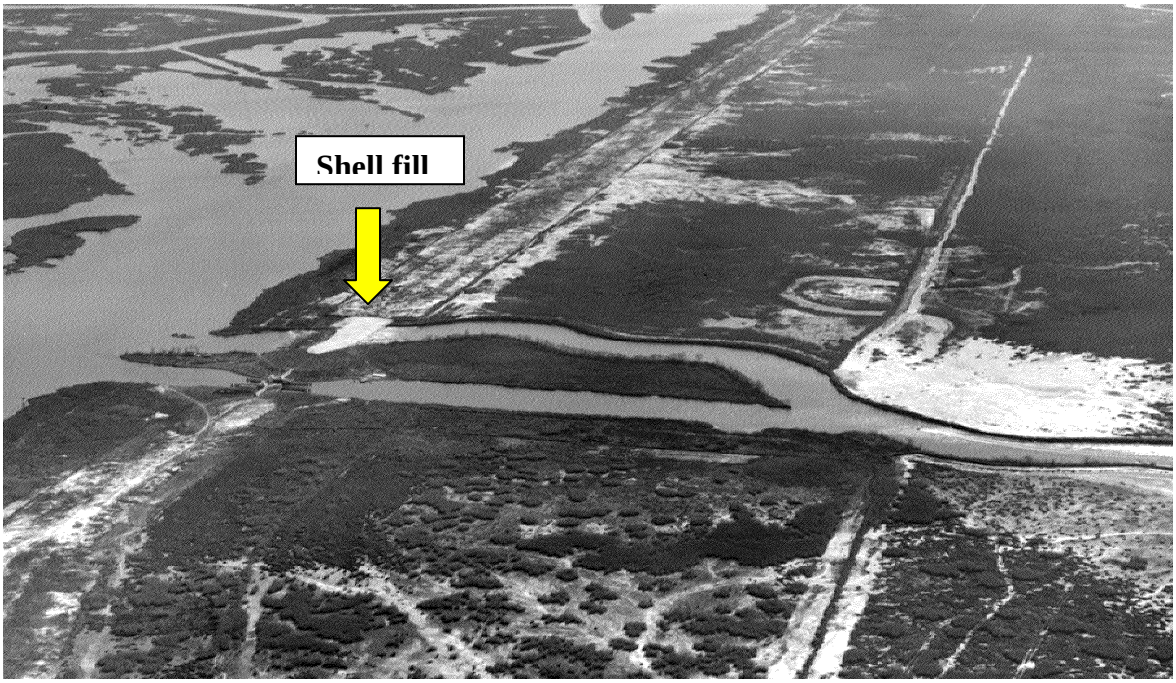


Figure 15: Photograph of MR-GO (to left looking south) during construction of navigation structure at Bayou Bienvenue (USACE photograph from the New Orleans Public Library). Note shell fill being placed in the previous course of the Bayou. Note also EBSBs being placed along current alignment.

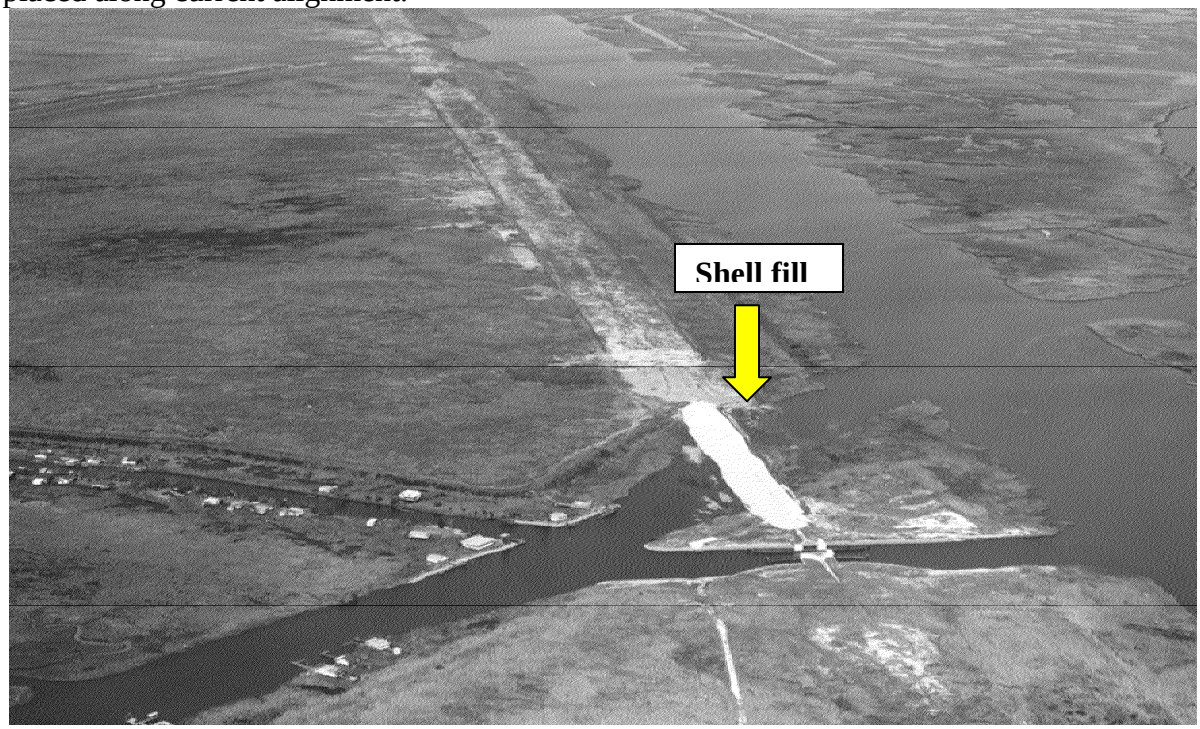


Figure 16: Photograph of MR-GO (to right looking north) during construction of navigation structure at Bayou Dupre (USACE photograph from the New Orleans Public Library). Note shell fill being placed in the previous course of the Bayou. Note also EBSBs being placed along current alignment.

23. This EBSB frontage along Reach 2 of the MR-GO is one of only two locations where the man-made features adjacent to the three main basins of New Orleans are exposed directly to storm waves crossing a large body of Gulf of Mexico waters (Lake Borgne) without the protection of significant swamps, marshes, and wetlands on their outboard sides. These natural protective buffers serve to reduce the energy in the storm waves and currents, reducing their height and velocity, and thus their erosive potential (Freudenberg et al 2007; National Research Council 2006, 2007; USACE 2006; Kahan et al 2006; Bea 2008). The catastrophic erosion of the EBSBs is not surprising since this section that was so exposed to severe (unprotected) storm waves was only about 14 feet high, was not armored to resist erosion and scour from surge, waves, and currents, and large portions was mostly comprised of highly erodible sands, silts, clays and lightweight shell sand fill underlain with highly organic layers and sand deposits (Figures 17 – 19).



Figure 17: Photograph of MR-GO (to left looking south) during construction of foundations for future EBSB flood protection structures (USACE photograph from the New Orleans Public Library).

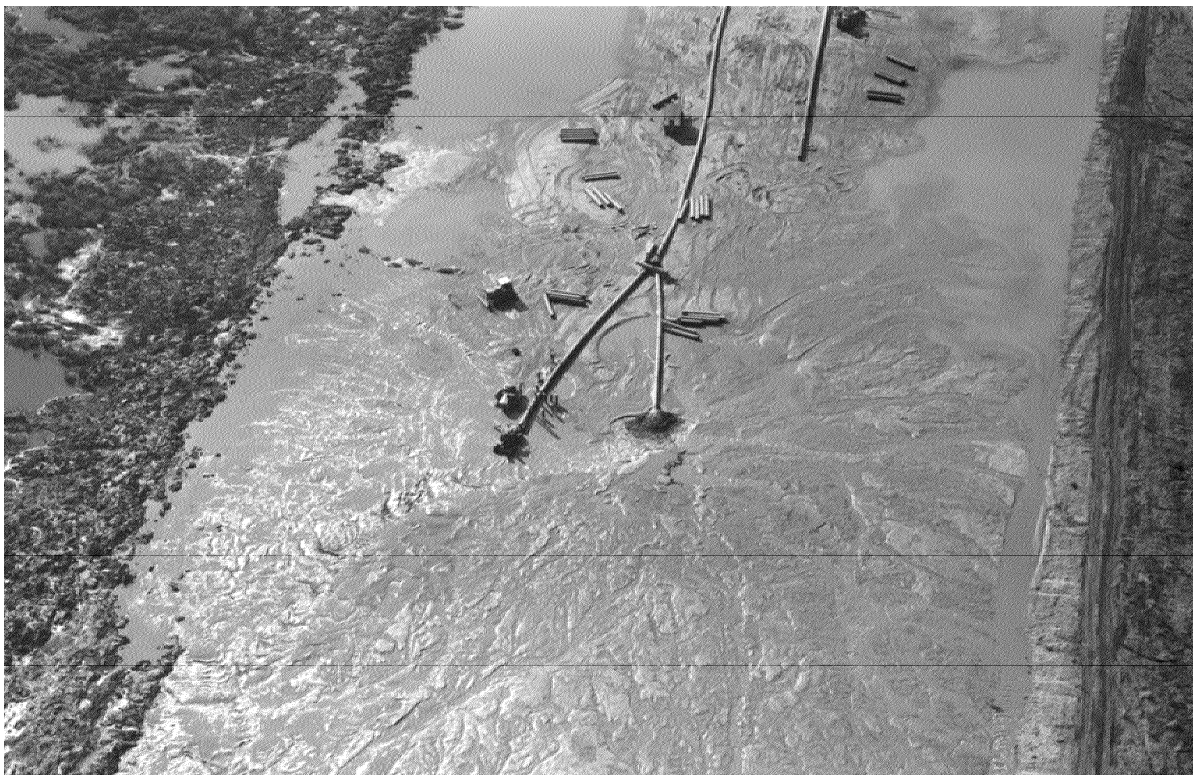


Figure 18: Photograph of dredged spoil being deposited on top of marsh deposits forming the foundations for the future EBSB flood protection structures (USACE photograph from the New Orleans Public Library).



Figure 19: Photograph of MR-GO (to right looking north) during construction of foundations for future EBSB flood protection structures (USACE photograph from the New Orleans Public Library).

Breaching Mechanisms

24. Multiple mechanisms for erosion, scour and breaching that occurred along the Reach 2 frontage section have been addressed during this study. This study has considered results from the studies performed by the USACE IPET (2007), the ILIT (2006), and Team Louisiana (2007). When multiple modes of failure are involved and interacting in different ways at different times and places, the uncertainties associated with 'looking back' at failures are significant. It is not easy to develop a 'correct' diagnosis. Most important in the previous analyses were how questions concerning the failures and breaching were posed to the analysts and investigators. For example, in the USACE IPET analyses, it was assumed at the outset of this work that failure and breaching occurred AFTER overtopping by the Katrina surge. This led to the assessment that it was backside erosion and scour that led to the breaching. This assumption currently persists (Sills et al 2008). In this study, no such assumptions were made. All plausible failure modes were addressed (e.g. Figure 2) with particular attention given to wave attack on the unprotected sides of the EBSBs and underbase seepage through the buried marsh layers and highly permeable shell fills used at the navigation structure tie-ins. This strategy was used to 'un-bias' the results from the failure and breaching analyses. All available sources of data and information were used to corroborate how the breaching and failures most probably developed.

25. As shown in Figure 20, high energy waves can scallop and erode the outboard face of an EBSB. They can also rush up the face toward the crest, and can erode "notches" in the crest from the front side. Subsequent waves can then pass through these notches, especially as the storm surge continues to rise, and the flow can widen the notches and also erode the back face levee slope (crown breaching). This exploitation and widening of crest

notches is called crenellation, after the crenellation (notched shape) that often topped castle walls (Vriend, Barends 2006; Koelewijn, Sellmeijer 2006; Hughes 2007; Ebersole 2008). This broadside attack on the EBSBs would be exacerbated by the very strong long-shore and cross-shore currents that were developed in this area. Figures 4 through 6 show clear evidence of this action along many sections of the MR-GO Reach 2 EBSBs.



Figure 20: Wave erosion on the front berm and face combined with through flow and under base seepage and erosion of the lower back face (ILIT 2006)

26. The overtopping back-side erosion mode of performance (Figure 21) was studied extensively during the IPET (2007) and ILIT (2006) investigations, and most lately by Ebersole (2008). This work involved extensive field, laboratory, and analytical studies (Wibowo et al 2007, Briaud et al 2007, 2008). In the ILIT study, soil samples were taken from levees that did not fail (sustained overtopping without breaching) and those that did fail (breached). In general, this work on sheet-flow and back-slope erosion was well corroborated with results from the extensive field and analytical studies performed by the USACE IPET (2007), Team Louisiana (2007), and ILIT (2006). The studies performed by IPET involved in situ erosion tests performed at a large number of locations in the Greater New Orleans area

(Wibowo et al 2007). General observations of levee performance were correlated with results from these studies. However, no analyses were developed to directly connect results from the in situ field tests and analyses of performance of the EBSBs due to wave action and surge overtopping.

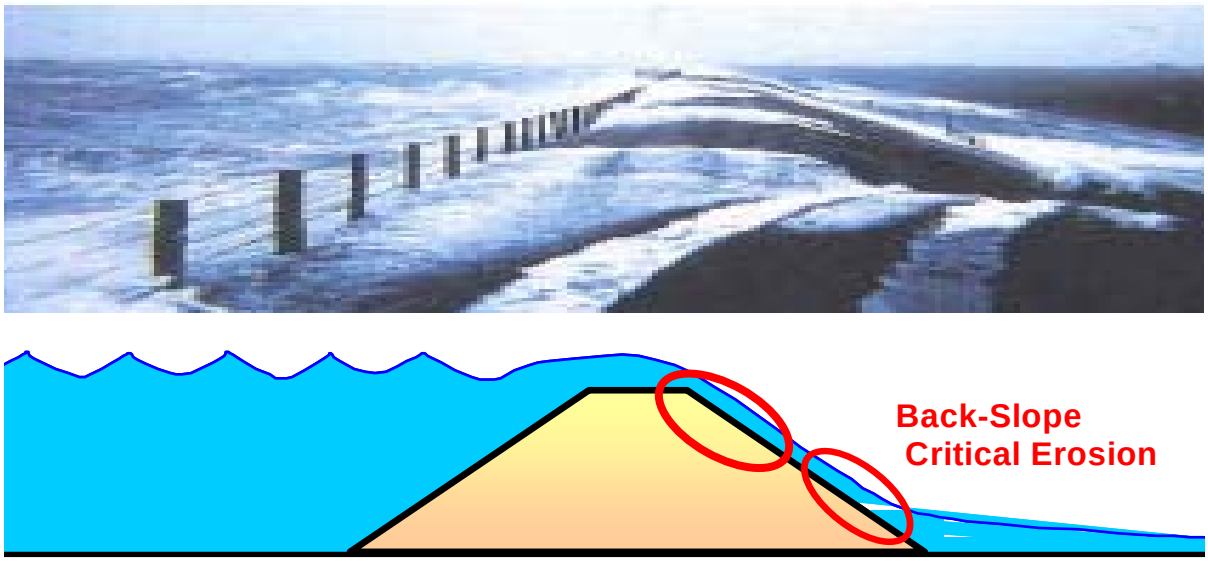


Figure 21: Sheet flow overtopping erosion of the lower back slope face with critical erosion at toe of the levee on the backside (ILIT 2006)

27. USACE observations from the earliest days following Katrina concluded and declared publicly that the MR-GO Reach 2 EBSBs had failed due to “massive overtopping” (Times Picayune, September 1 2005). Subsequently, the USACE IPET investigations confirmed these observations and based its analyses on the scenario that the purported Reach 2 ‘levees’ and areas adjacent to the two navigation structures breached as a result of scour and erosion induced by overtopping (IPET 2007, Sills et al 2008). It is obvious that the Reach 2 EBSBs were overtopped during Hurricane Katrina. What is not so obvious, however, is whether the overtopping protected side breaching was preceded by wave attack breaching – crenellation – on the unprotected side. Results from my and other investigations (ILIT 2007, Seed et al 2007a, 2008a , Team Louisiana 2007) indicates that in many cases – if

not the majority of cases – it is very likely that the backside overtopping erosion was preceded by front side wave attack erosion. The front side erosion transitioned into backside erosion after overtopping, and development of the breaching resulted in erasing much of the evidence of the preceding front side breaching.

28. It also is obvious that due to decisions that were made regarding the reference vertical elevation datum for construction of the supposed levees, they were not built to their authorized elevations (USACE IPET 2007, Woolley, Shabman, *Hurricane Protection Decision Chronology Report* 2008). The decision to use an inaccurate and obsolete vertical datum combined with the incomplete construction of the levees authorized for construction in 1965 as part of the Lake Pontchartrain and Vicinity Hurricane Protection Project resulted in important decreases in the EBSB crown elevations and elevations of the navigation - lock structures (Bea and Storesund 2008). These decreases in the EBSB and navigation structure elevations and cross sections had very important effects that exacerbated the overtopping erosion and breaching effects (GEI Consultants 2007, USACE IPET 2007, Team Louisiana 2007, ILIT 2006, Bea and Storesund 2008).

EBSB Breaching Study Location

29. Figures 22 and 23 show the EBSB site located south of Bayou Bienvenue that was analyzed in detail to determine how breaching would have developed during Hurricane Katrina (station 497+00). This site was chosen because photographic evidence and pre and post Katrina elevation surveys showed that the area was severely eroded during Hurricane Katrina (Figures 24 and 25).

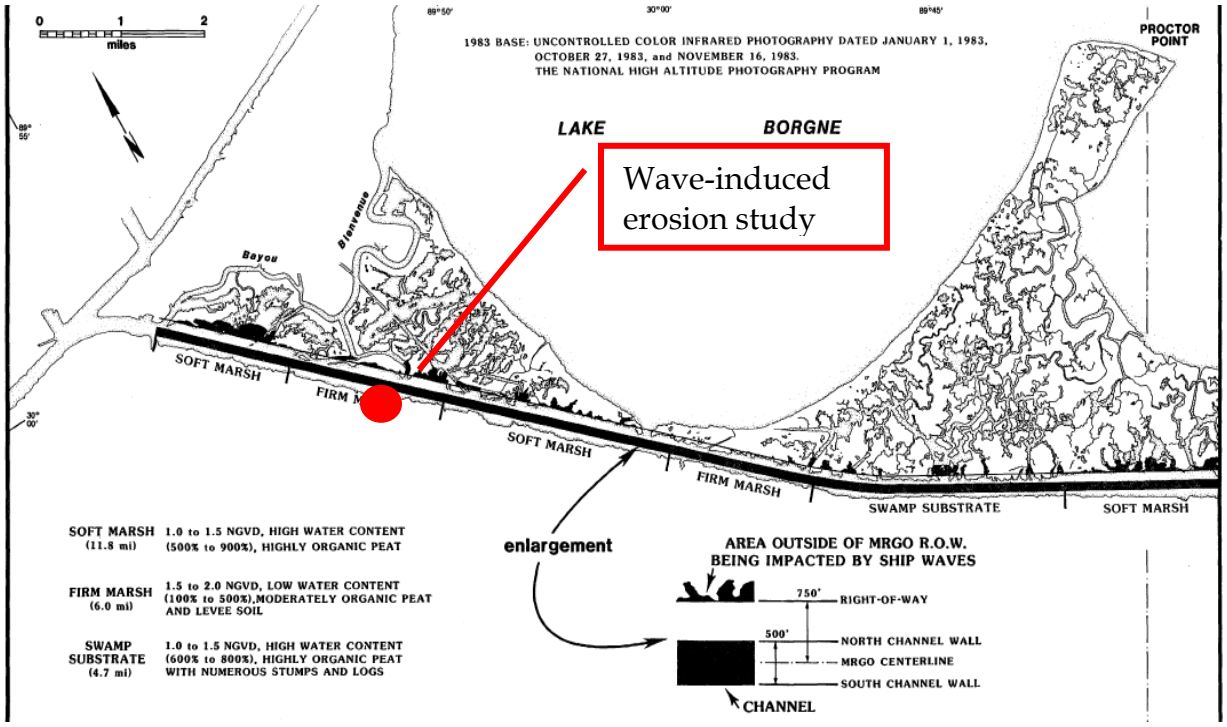


Figure 22: Site chosen for wave induced erosion – scour study (Bea and Storesund 2008).

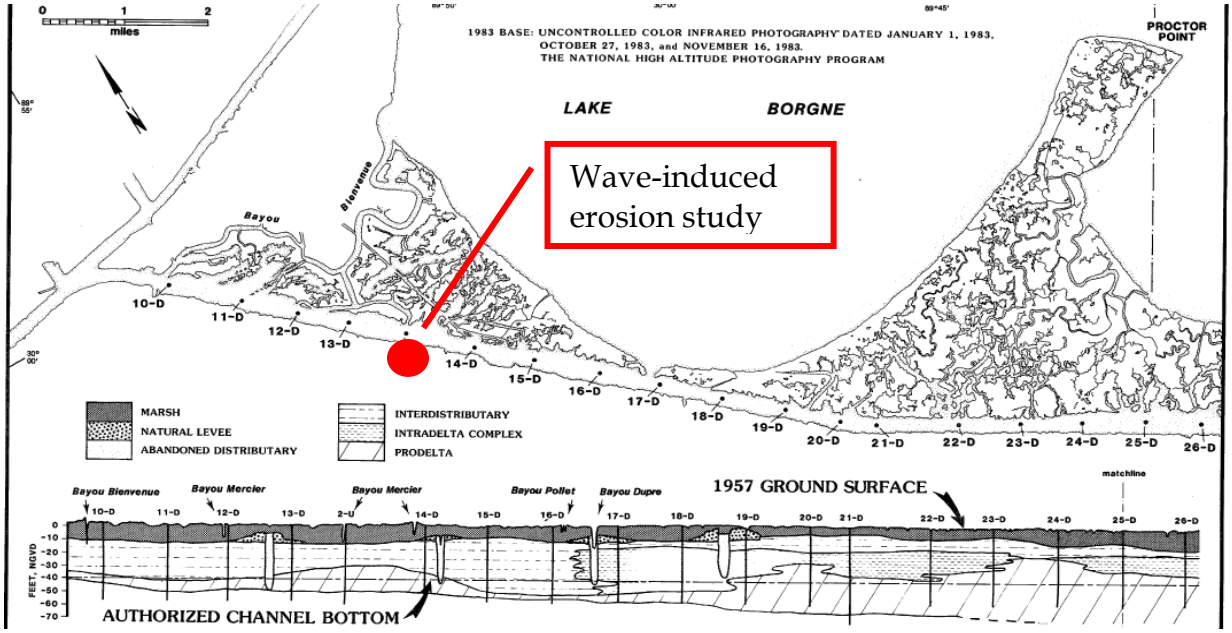


Figure 23: A generalized cross section as presented in CEI (1984) and originally created by Kolb and Van Lobik (Bea and Storesund 2008). Note entire area is covered with "Marsh" deposits from 10 feet to 15 feet thick. These deposits were buried by the MR-GO dredged spoil banks and subsequently by the Reach 2 EBSBs.

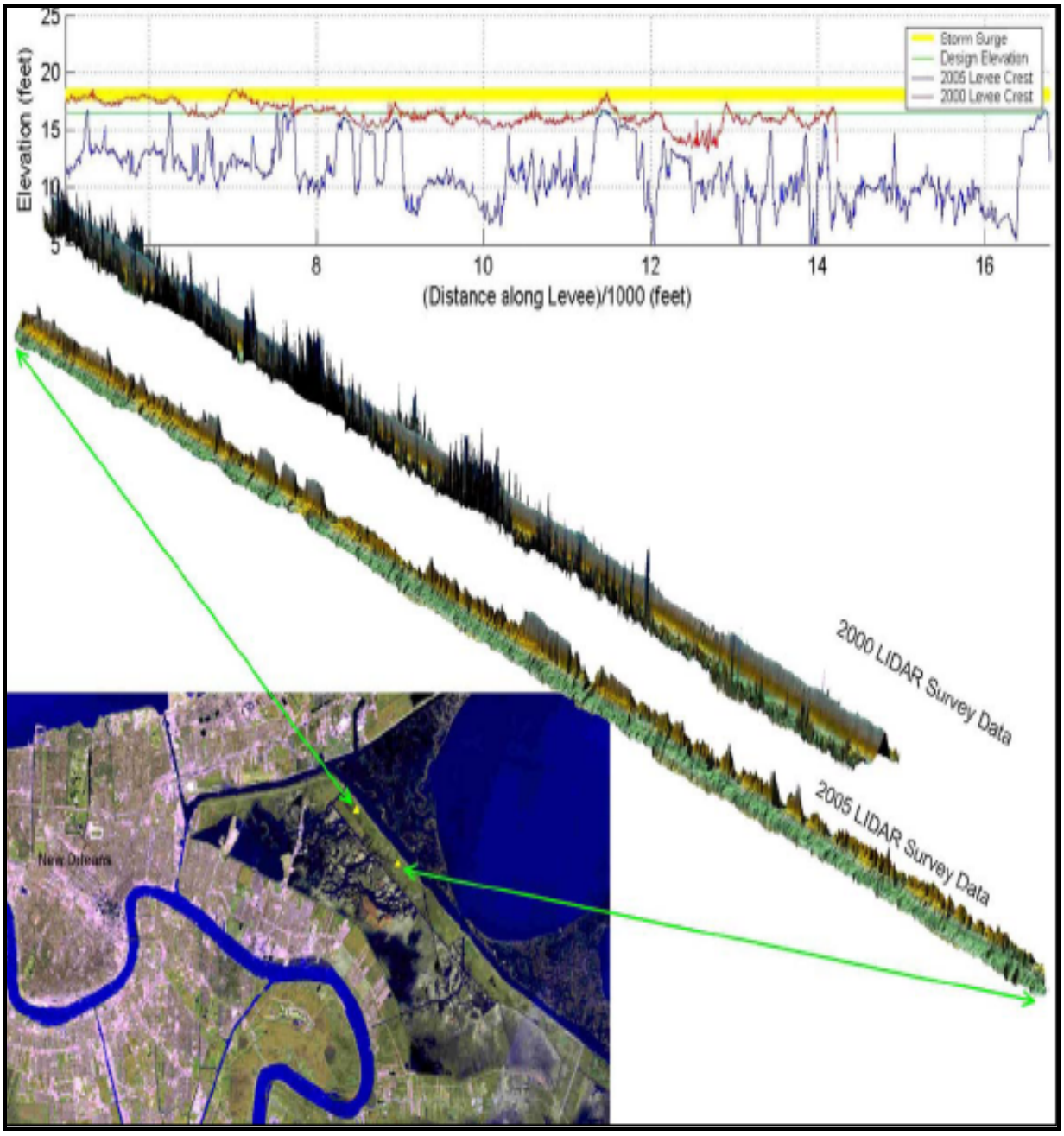


Figure 24: Pre and post Katrina levee crest elevation surveys (ILIT 2007).

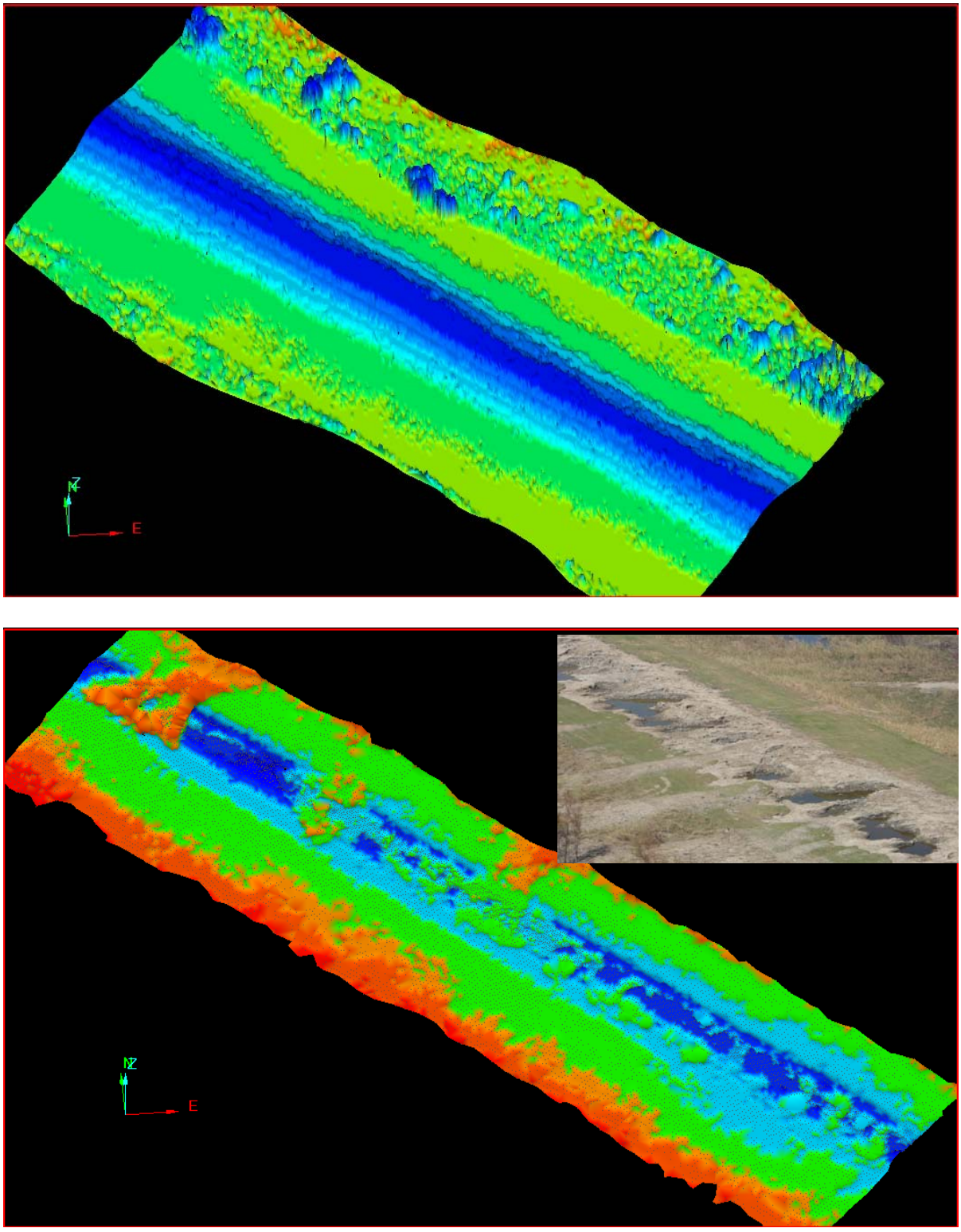


Figure 25: Aerial LiDAR surveys captured the erosion features of the MR-GO levees following Hurricane Katrina (Bea and Storesund 2008). Elevations are colored from red (low) to blue (high). Above is an aerial oblique photograph showing erosion and scour as depicted in the Aerial LiDAR survey.

30. Figure 26 presents a historic timeline of the levee crest elevation at Station 497+00 from 1966 to 2007. At the time Hurricane Katrina hit the MR-GO area, the levee crest elevation had settled approximately 1.5 feet below the target crest elevation of +17.5 feet (NGVD). Also included on this plot are approximations of storm surge elevations (high water marks) near Station 497+00 from observations following Hurricanes Betsy (USACE, 1965) and Camille (USACE, 1970).

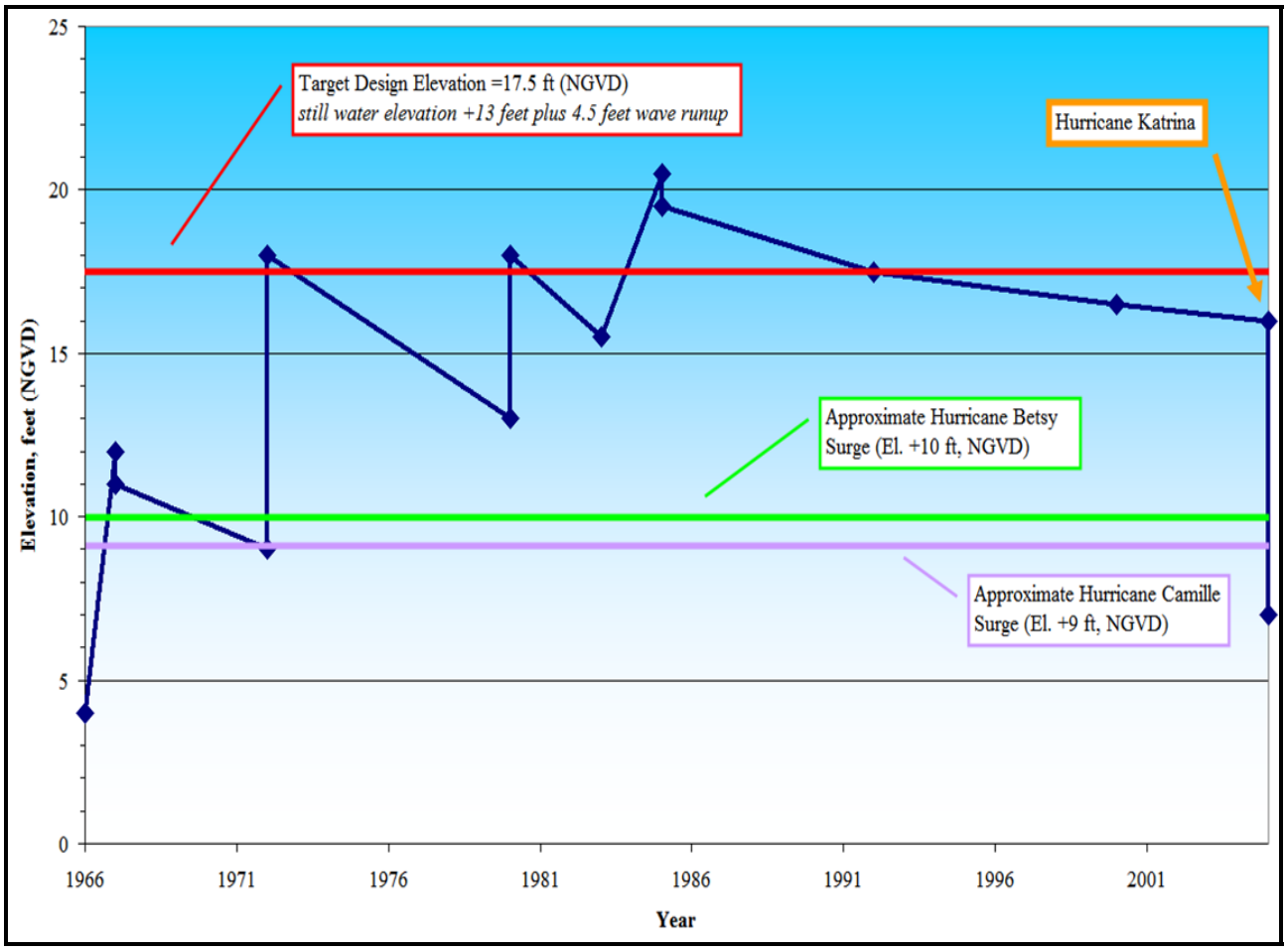


Figure 26**:** Plot of levee crest elevation from 1966 to 2007, when Hurricane Katrina hit New Orleans (Bea and Storesund 2008).

EBSB Study Location Soil Erosion & Scour Characteristics

31. The ILIT team undertook a program of soil sampling and testing at various locations along the Reach 2 affected EBSBs (Figure 27). These samples were transported to a laboratory at Texas A&M University and tested to determine soil properties and characteristics as well as their erosion – scour resistance (ILIT 2006, Briaud et al 2007). Previously, as a part of a long-term research program at Texas A&M University concerned with erosion around bridge piers and scour of earth levees, a specialized testing apparatus had been developed and validated to simulate sheet-flow erosion characteristics on samples of soil (Briaud et al 2007, 2008). This testing apparatus was used to determine the conditions (compaction, salinity, water velocity) at which the soil samples would erode and the rates at which the samples would erode.

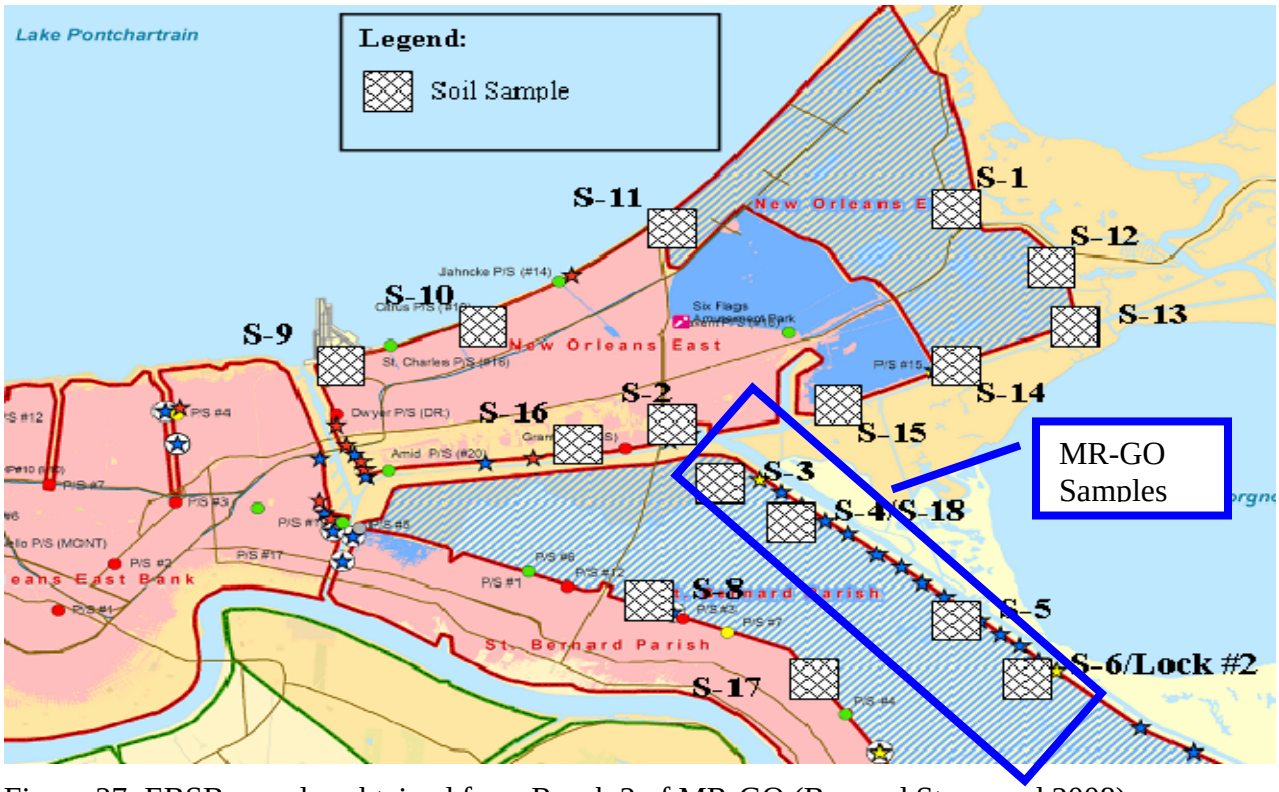


Figure 27: EBSB samples obtained from Reach 2 of MR-GO (Bea and Storesund 2008).

32. Results from the laboratory sheet-flow erosion testing on the samples retrieved from the Reach 2 of MR-GO are summarized in Figure 28. The soils obtained from Reach 2 (samples #4, #5, #6), the areas that suffered significant breaching during Hurricane Katrina showed "very high erodibility" (ILIT 2006, Briaud et al 2007, 2008). These soils were characterized with very high sand, silt, and shell content and little compaction.

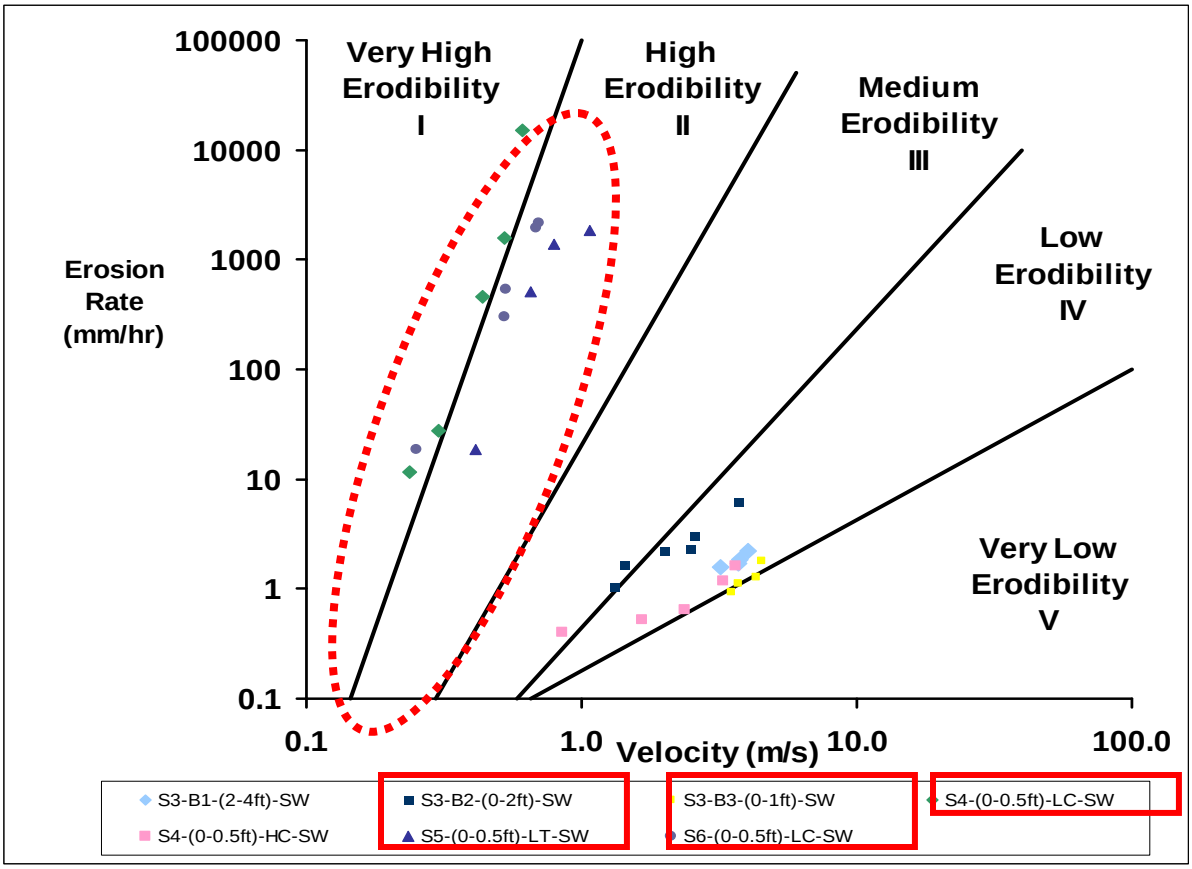


Figure 28: Samples (red boxes) obtained and tested from the MR-GO were found to have high to very high erodibility based on results from laboratory testing performed by ILIT (2006) (Bea and Storesund 2008).

33. Figure 29 summarizes data from the laboratory testing that was utilized to evaluate the wave erosion – scour characteristics at the study location. Figure 30 is a picture of the soils sampled from this location.

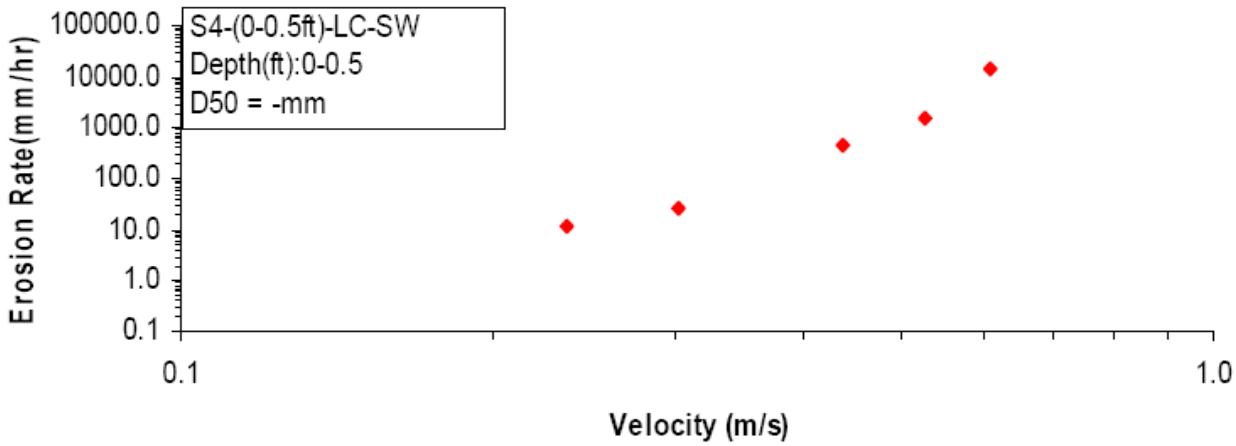


Figure 29: Erodibility test results on bulk soil samples from the MR-GO levees (ILIT 2006).



Figure 30: Close up photograph of soils on shoulder of breach on MR-GO EBSB at station 497+00 showing dredge spoil materials used in construction of the levee (Bea 2005). Bulk samples were taken of the soils from this breach shoulder and tested in the laboratory to determine the EBSB scour – erosion characteristics used in this study (Bea 2005).

34. Erodibility in situ tests and evaluations were performed by IPET following Hurricanes Katrina and Rita (Figure 31). The results of their study are presented as an appendix in Volume V of the IPET Report series. Four samples were collected from the

same general vicinity as examined in this study. Only one of the four test locations was on an existing levee (M3). Two of the test locations were reconstructed levee sections that had undergone some degree of compaction. One test location (M2-2) was at a local borrow pit which was in an uncompacted state (hydraulic fill materials from the MR-GO channel excavations). These tests found that the (pre and post) materials used to construct the MR-GO Levee ranged from resistant to erosion to very erodible. The local borrow pit (site M2-2 consisting of uncompacted hydraulic fill used to construct the original MR-GO Levee) was characterized as erodible and very erodible. The existing levee location (M3) had test results that were very erodible and resistant.

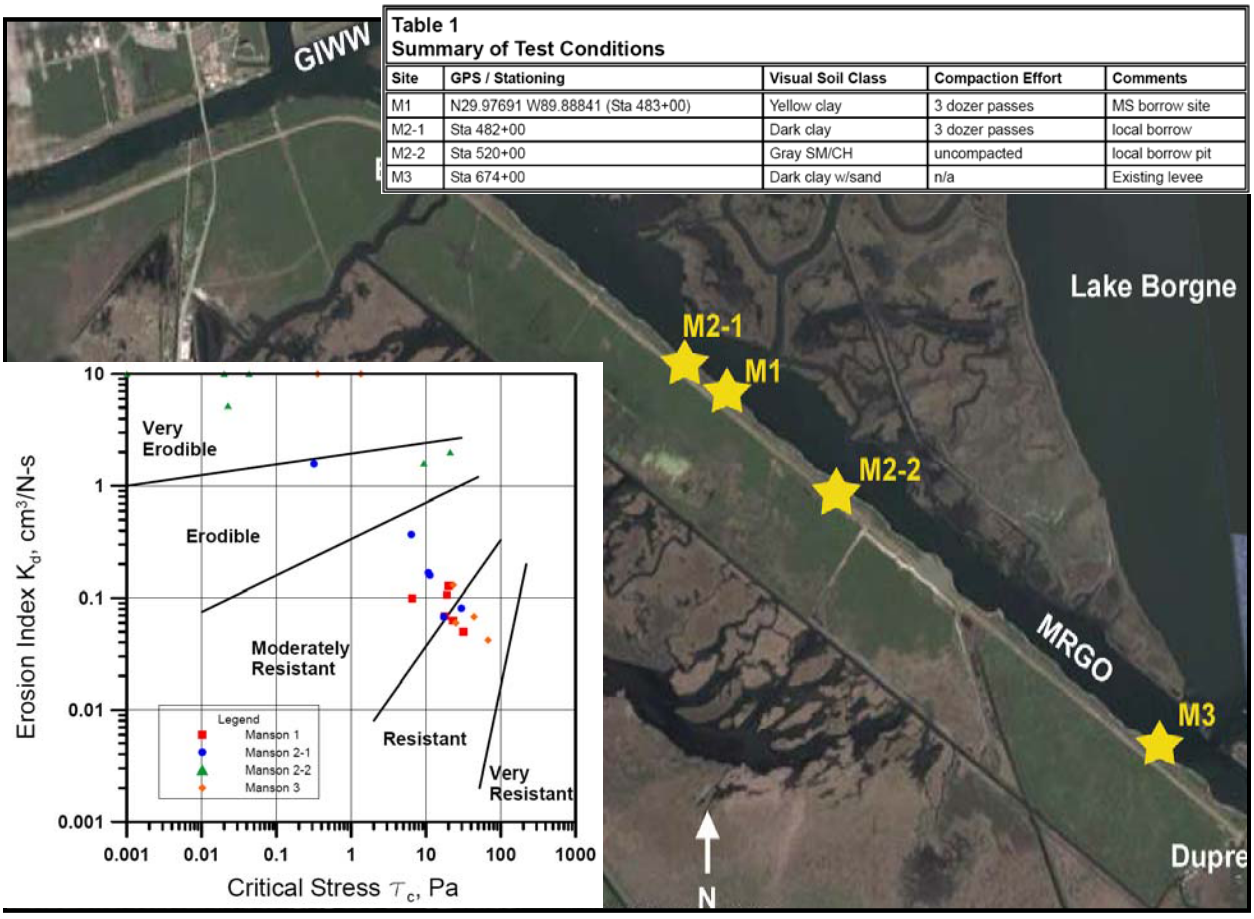


Table 1
Summary of Test Conditions

Site	GPS / Stationing	Visual Soil Class	Compaction Effort	Comments
M1	N29.97691 W89.88841 (Sta 483+00)	Yellow clay	3 dozer passes	MS borrow site
M2-1	Sta 482+00	Dark clay	3 dozer passes	local borrow
M2-2	Sta 520+00	Gray SM/CH	uncompacted	local borrow pit
M3	Sta 674+00	Dark clay w/sand	n/a	Existing levee

Figure 31: Results from erodibility evaluations performed by IPET following Hurricane Katrina found samples along the MR-GO to be very erodible and erodible (USACE IPET 2007e).

EBSB Profile

35. The EBSB profile analyzed to characterize performance – erosion – breaching characteristics during Hurricane Katrina was one at Station 497+00 as it was prior to Hurricane Katrina (Figure 32). The design profile for the Reach 2 man-made hurricane flood protection alignment was also studied to enable evaluations of the effects of changes in the profile on performance of the EBSBs.

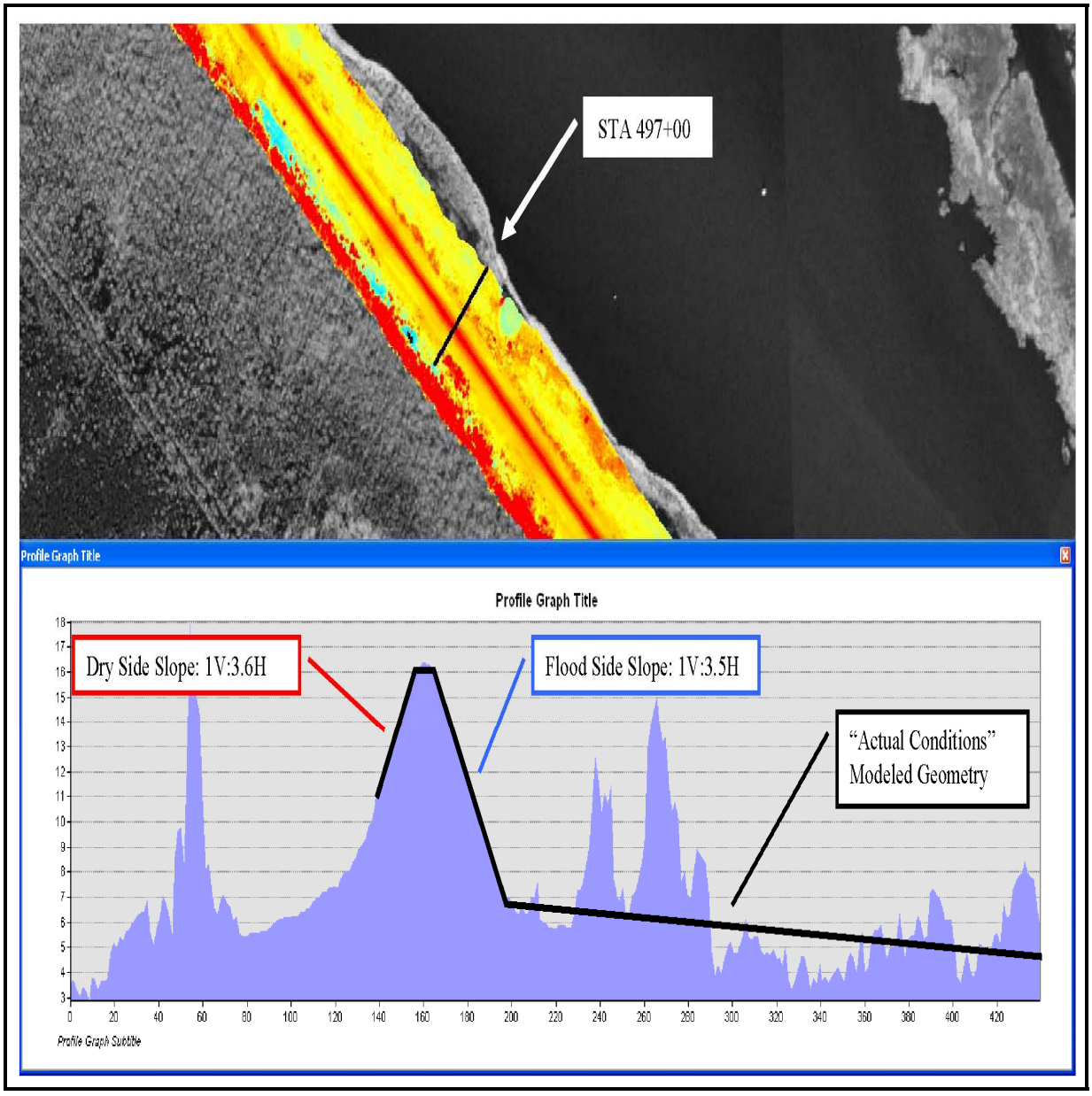


Figure 32: Study location levee profile prior to Hurricane Katrina (Bea and Storesund 2008).

Hurricane Katrina Hydrodynamic Conditions

36. Table 1 summarizes the principal conditions and characteristics that were integrated into the Hurricane Katrina surge, current, and wave analytical models developed by Wit et al 2008, and Gautier et al 2008 to serve as input for analyses of the performance characteristics of the MR-GO Reach 2 man-made hurricane flood protection structures. 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 'Neutral' MR-GO conditions.

Table 1: Summary of simulation Scenarios for characterization and description of MR-GO Reach 1 and Reach 2 Hurricane Katrina surge, current, and wave characteristics.

Scenario	MR-GO channel	GIWW channel	Reach 2 EBSBs	40 Arpent Levee	Vegetation
1	**Existing 8/29/2005**	**Existing 8/29/2005**	**Existing 8/29/2005**	**Existing 8/29/2005**	**Existing 8/29/2005**
2A	None	Pre-construction	None	LPV authorized & no flow past	Pre-construction
2B	None	Pre-construction	None	Existing 8/2005 & flow to Miss. River	Pre-construction
2C 'Neutral' MR-GO	**Pre-construction**	**Pre - construction**	**Existing 8/29/2005**	**Existing 8/29/2005**	**Pre-construction**
2D	None	Pre-construction	Existing 9/9/1965	Existing 8/29/2005	Pre-construction
3	As authorized	As authorized	Existing 8/29/2005	Existing 8/29/2005	Pre-construction

37. These ‘Neutral’ MR-GO Hurricane Katrina Conditions were analyzed to help determine how the design, construction, operation, and maintenance of the MR-GO affected the performance of the man-made flood protection structures during Hurricane Katrina (Bea 2008). The ‘base case’ for the ‘Neutral’ MR-GO Hurricane Katrina Conditions is identified in Table 2 as Scenario 2C. The other Scenario 2 and Scenario 3 analyses are intended to provide insights into the affects of the MR-GO design, construction, operation, and maintenance parameters important to the performance characteristics of the man-made structures associated with Reach 1 and Reach 2.

38. The ‘Neutral’ MR-GO Hurricane Katrina Conditions assessment is based on the contention that the Congressionally authorized MR-GO navigation project should ‘do no harm’ to the environment and to the man-made hurricane flood protection structures that defend this region from hurricane flooding. Thus, if there would be negative effects of the MR-GO that would have substantial deleterious effects on these elements, then it was incumbent that the USACE properly address and mitigate these negative effects. Such negative effects as destruction and degradation of the natural hurricane flood protection features (barrier beaches, land bridges, marshes, wetlands, swamps, salinity increases resulting in degradation of vegetation and habitats) and those associated with constructed works (e.g. protective berms for flood protection levees and other man-made structures, channel water flow increasing surge elevations, channel effects increasing hurricane waves and currents and channel induced erosion of both natural and man-made flood protection elements) have been addressed in development of the ‘Neutral’ MR-GO Hurricane Katrina Conditions addressed in this Declaration.

39. The negative effects of the MR-GO could have and should have been addressed by mitigating elements applied directly to the MR-GO channel itself and to the adjacent man-made flood protection structures. It was incumbent on the USACE to address these potential environmental effects in the MR-GO environmental impact statement (USACE 1976). The scientific and engineering technology, although not as well developed as today, did exist to properly address these effects (see Appendix B in Declaration by Robert Glenn Bea dated September 17, 2007; USACE 1975; Bretschneider and Collins 1966; Woolley and Shabman 2008; Colten 2006; ILIT 2006; Team Louisiana 2007; Coastal Environments 1984; Arnold 2007, 2008). In fact, the history of the development of the MR-GO project (*MR-GO Timeline 1722 A.D. – Present*, Katrina Consolidated Litigation Committee, 2008) clearly shows that the USACE was aware of and in fact did consider many of the critical negative impacts. It is also clear, that preoccupations with continued development of navigable waterways, budgets, and schedules led to decisions not to mitigate these important impacts. Today, MR-GO channel mitigations are being developed and evaluated by the USACE as part of the *Mississippi River Gulf Outlet Deep-Draft De-Authorization Report* to Congress (USACE 2008a), the *Louisiana Coastal Protection and Restoration Project* (LCPRA, USACE 2008b), and the MR-GO Reach 2 man-made flood protection structures mitigations included in the *Hurricane and Storm Damage Reduction System Design Guidelines* (USACE 2008c). Potential channel mitigating measures include placing barriers or gates at the Gulf of Mexico – MR-GO entrance, at the intersection of the MR-GO with the GIWW, and at the IHNC (Seabrook), restoring natural protective features (barrier beaches, wetlands, marshes, swamps), providing armoring of channel banks, and provision of foreshore protection to protect the EBSB and other similar levee alignments.

Potential man-made flood protection structure mitigating measures include provision of protected and flood side armor for levees and embankments, wave berms, foreshore protection, EBSB and levee profiles that reduce surge – current – and wave effects, use of proper materials and construction methods, use of deeper sheet piling and other means to cut-off subterranean seepage paths, and construction and maintenance procedures to maintain levees and EBSBs to adequate elevations. The effects of these mitigations have been included in characterization of the 'Neutral' MR-GO Hurricane Katrina Conditions.

40. The primary locations identified for outputs from the analytical models utilized by Wit et al (2008) and Gautier et al (2008) are shown in Figure 33. Based on the analytical results provided by Wit et al (2008), the surge hydrographs associated with the MR-GO Reach 2 locations are summarized in Figures 34 and 35. These surge hydrographs are based on retaining the man-made flood protection structures in their pre-Katrina conditions – no breaching – only overtopping.

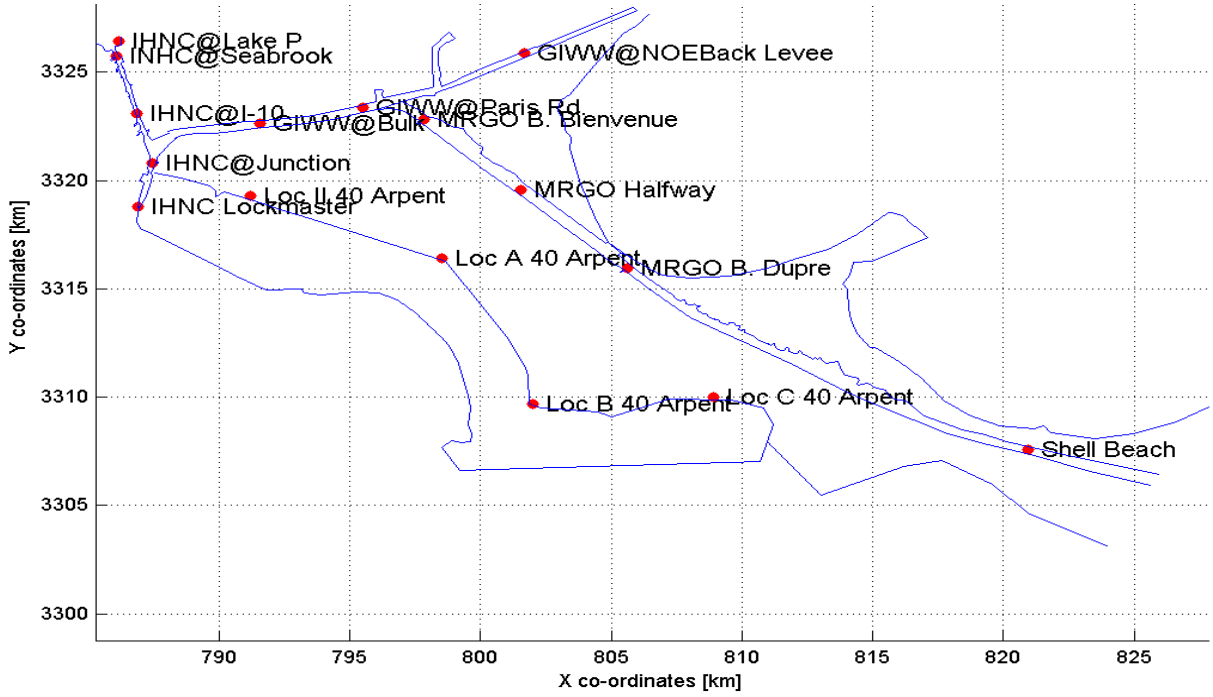
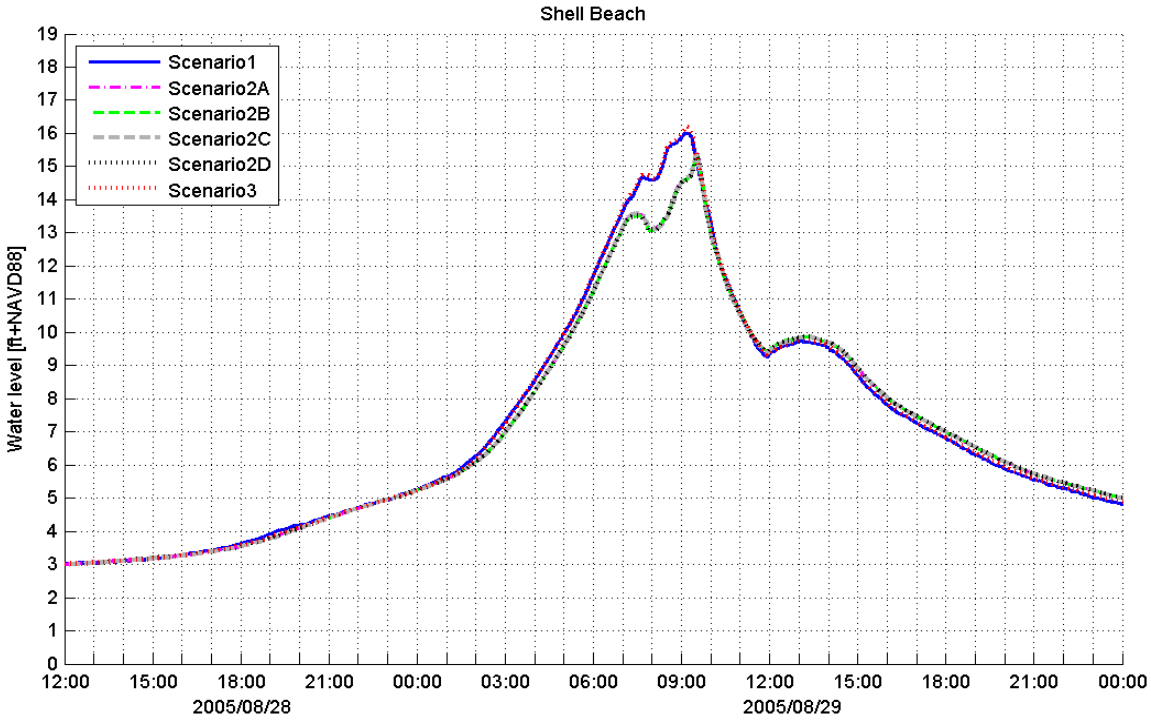


Figure 33: Primary locations for Hurricane Katrina simulations outputs (Wit, et al 2008).



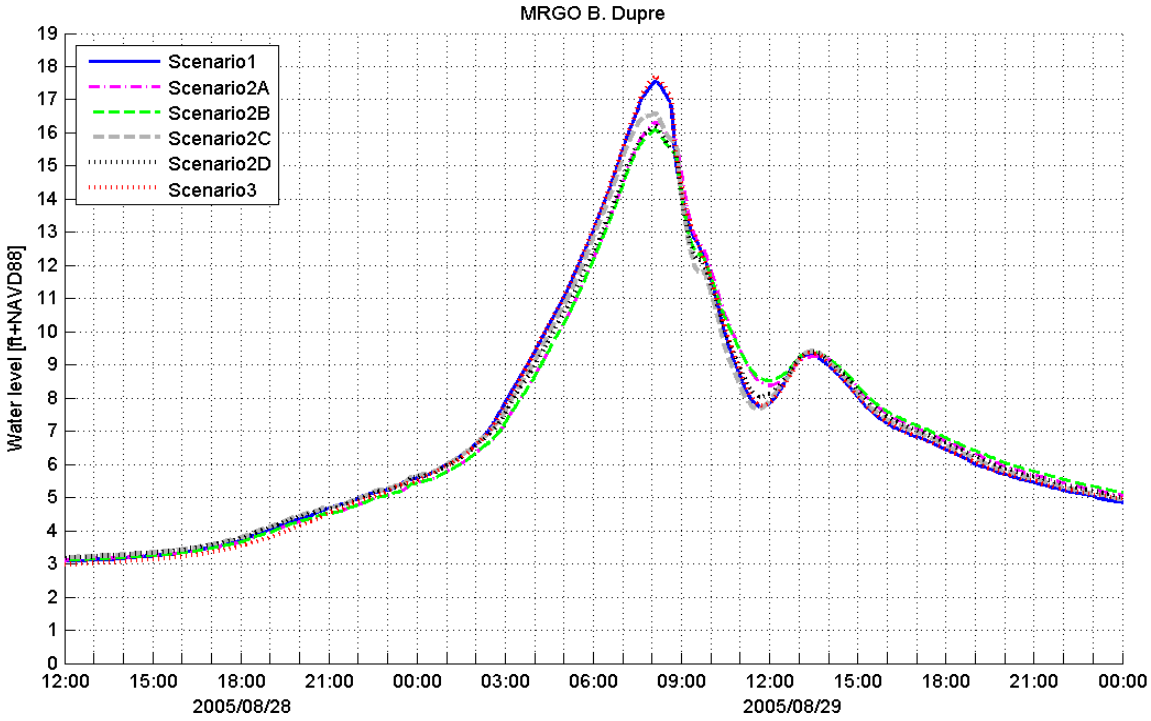


Figure 34: Surge hydrographs for Shell Beach and Bayou Dupre (Wit, et al 2008).

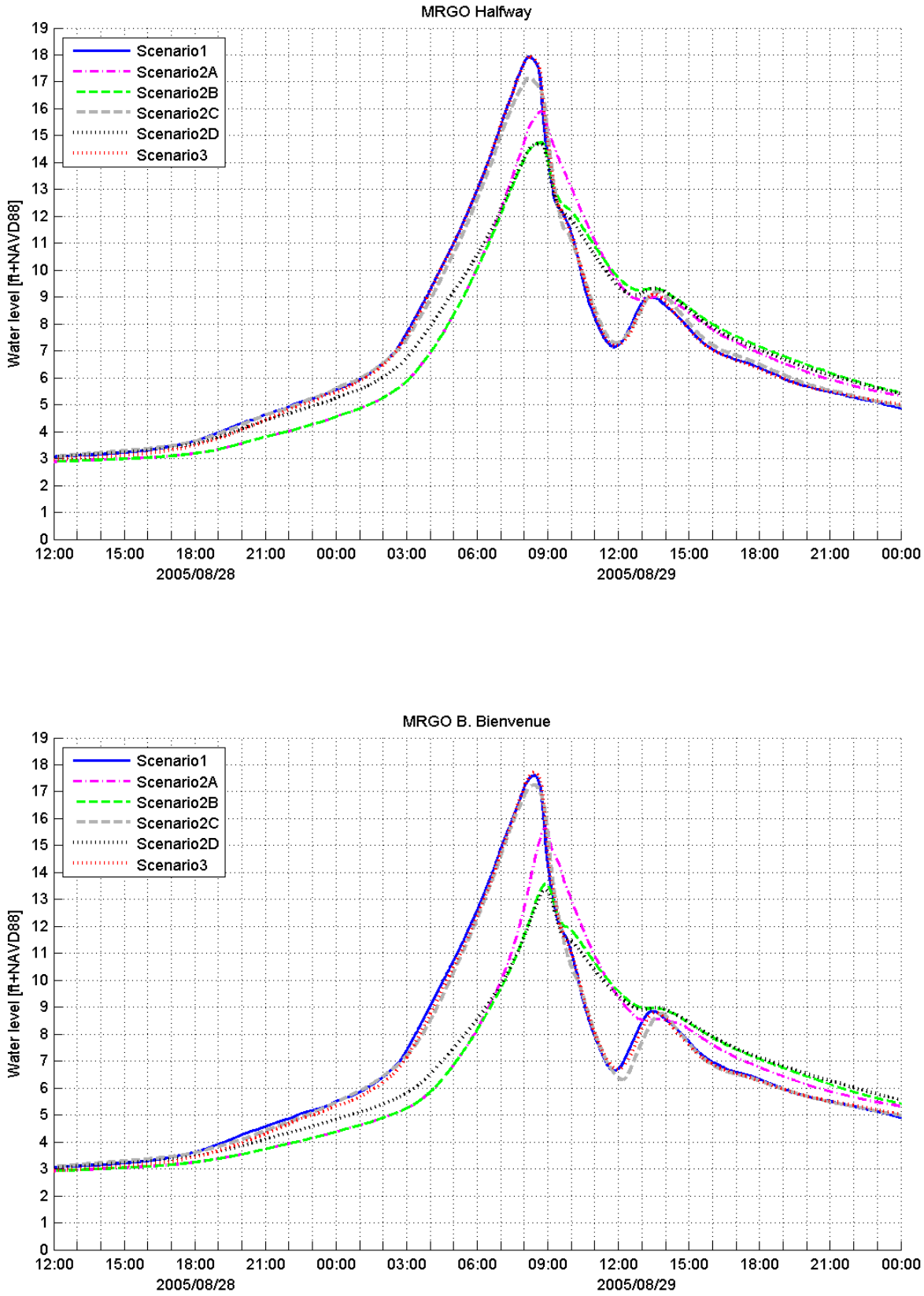


Figure 35: Surge hydrographs for MR-GO Halfway and Bayou Bienvenue (Wit, et al 2008).

41. As would be expected, depending upon location and the associated geographic – bathymetric – environmental conditions, the presence of the MR-GO has very different effects upon the Hurricane Katrina surge conditions. Comparing results between the Hurricane Katrina 'as was' surge hydrographs and the 'Neutral' MR-GO Hurricane Katrina hydrographs, in the relatively open water areas (e.g. Shell Beach, Bayou Dupre), the MR-GO channel has little influence on the surge characteristics. The Hurricane Katrina surge hydrographs that vary different parameters that affect the surge characteristics help identify how the Reach 2 EBSB alignment has significant effects on the surge characteristics along Reach 2. The EBSB alignment acts like a 'barrier' to the surge waters – and the surge waters pile up against this barrier (Scenario 2B). This 'piling up' effect was addressed by Bretschneider and Collins in their work on the 'funnel' effects for the USACE (1966). If this barrier is removed, then the surge waters have a 'place to go' – an enlarged area for volumetric, absorption, and frictional dissipation. Comparing the Hurricane Katrina peak surge at the EBSB alignment (about + 18 feet) with that at the 40 Arpent alignment without the EBSBs (Scenario 2D), the surge is dramatically reduced (about +13 to +14 feet). For the interior Reach 1 locations, the effects of the 'funnel' are further amplified by the presence of the MR-GO channel – the channel provides a conduit for earlier delivery and longer duration of the surge waters and for amplification of the surge due to the increased volume of water that can be delivered to these areas by the channel itself.

42. Water current discharges and velocities associated with Hurricane Katrina and the MR-GO conditions also were evaluated for the areas of interest. These current characteristics also were very dependent on bathymetric, geographic, topographic, and vegetation (roughness) characteristics. Figure 36 shows the spatial current velocities for

Scenario 1 Hurricane Katrina ‘as was’ conditions near the time of the peak surge (August 29, 2008, 8:00am CDT referenced to Bayou Bienvenue location).

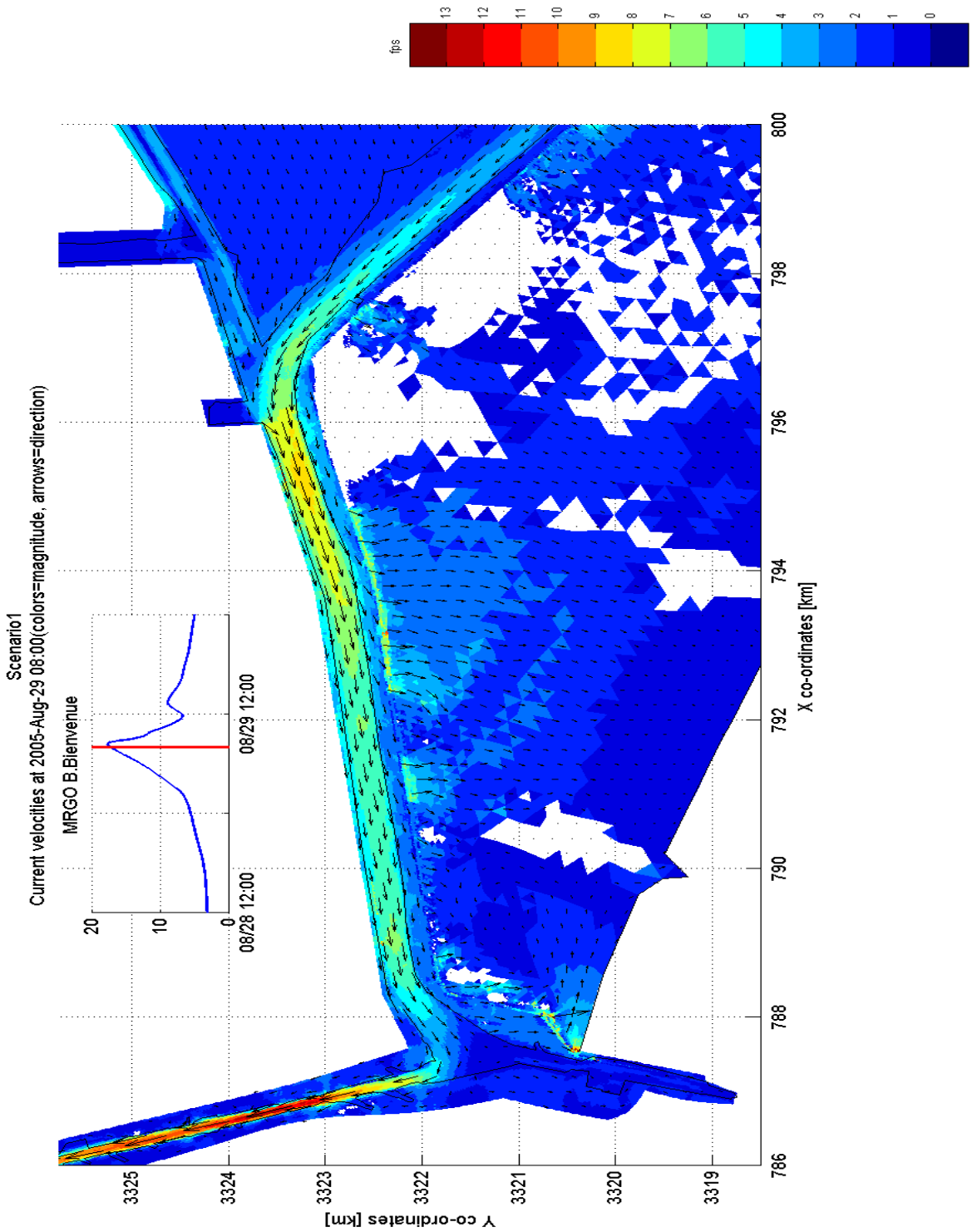


Figure 36: Spatial current velocities (magnitude with cooler scale, direction with arrows for Scenario 1 at 8:00 am CDT (Wit, et al 2008).

43. Because of the potential importance of these currents to performance of the EBSBs bordering Reach 2 of the MR-GO, special current output locations were identified as shown in Figure 37 (Wit, et al 2008). Location 3 is of particular interest because this is the location used to perform detailed analyses of the wave and overtopping erosion and breaching characteristics of the EBSBs. Figure 38 shows the current velocities at Locations 1 - 3 (black line at toes of EBSBs and dashed green line - 'a' - at edge of MR-GO channel) from 4:00 am to 9:00 am CDT. The current velocities at the toes of the EBSBs are generally less than about one foot per second and have flow angles that parallel the face of the EBSB and MR-GO channel elevation.

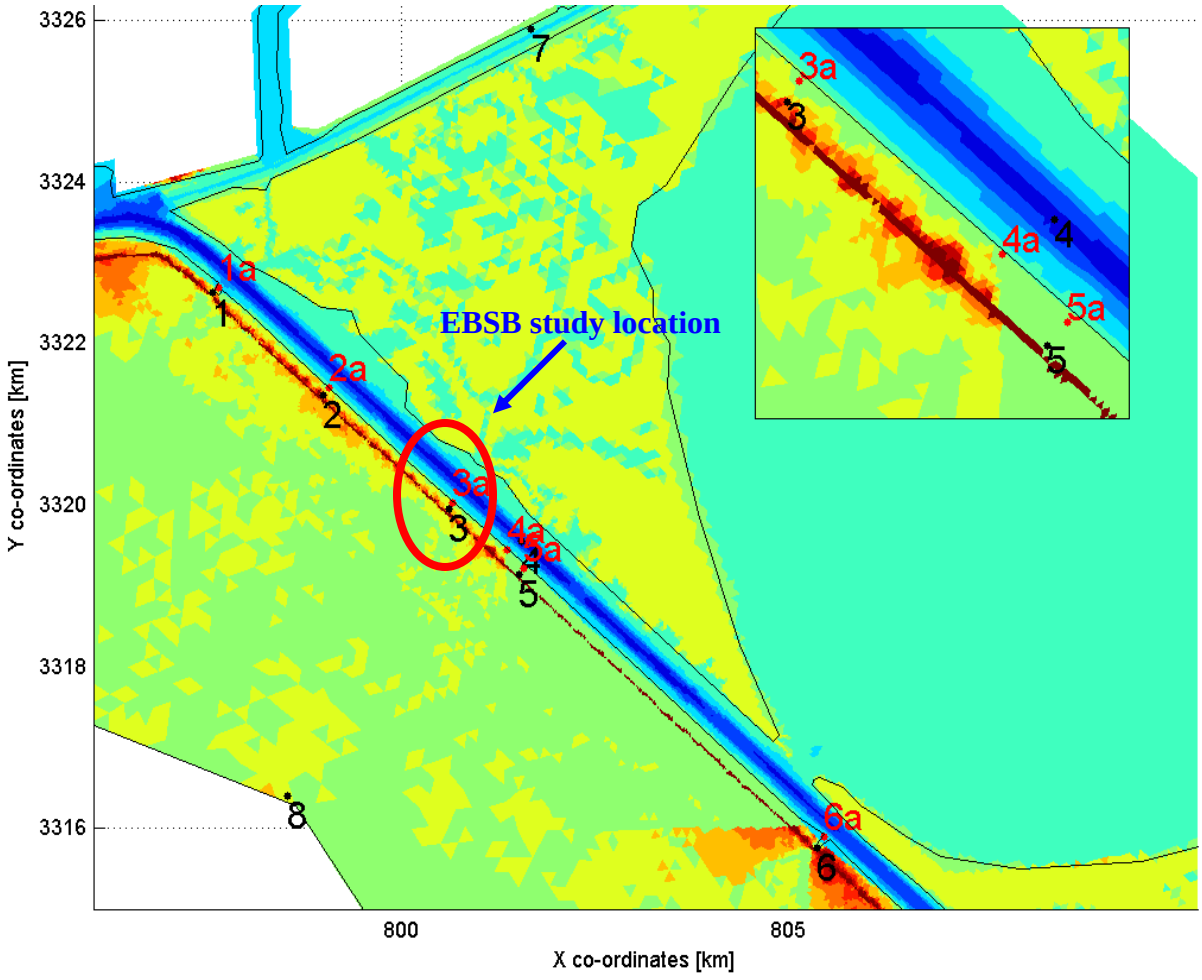


Figure 37: Output locations for MR-GO Reach 2 (Wit, et al 2008, Gautier, et al 2008).

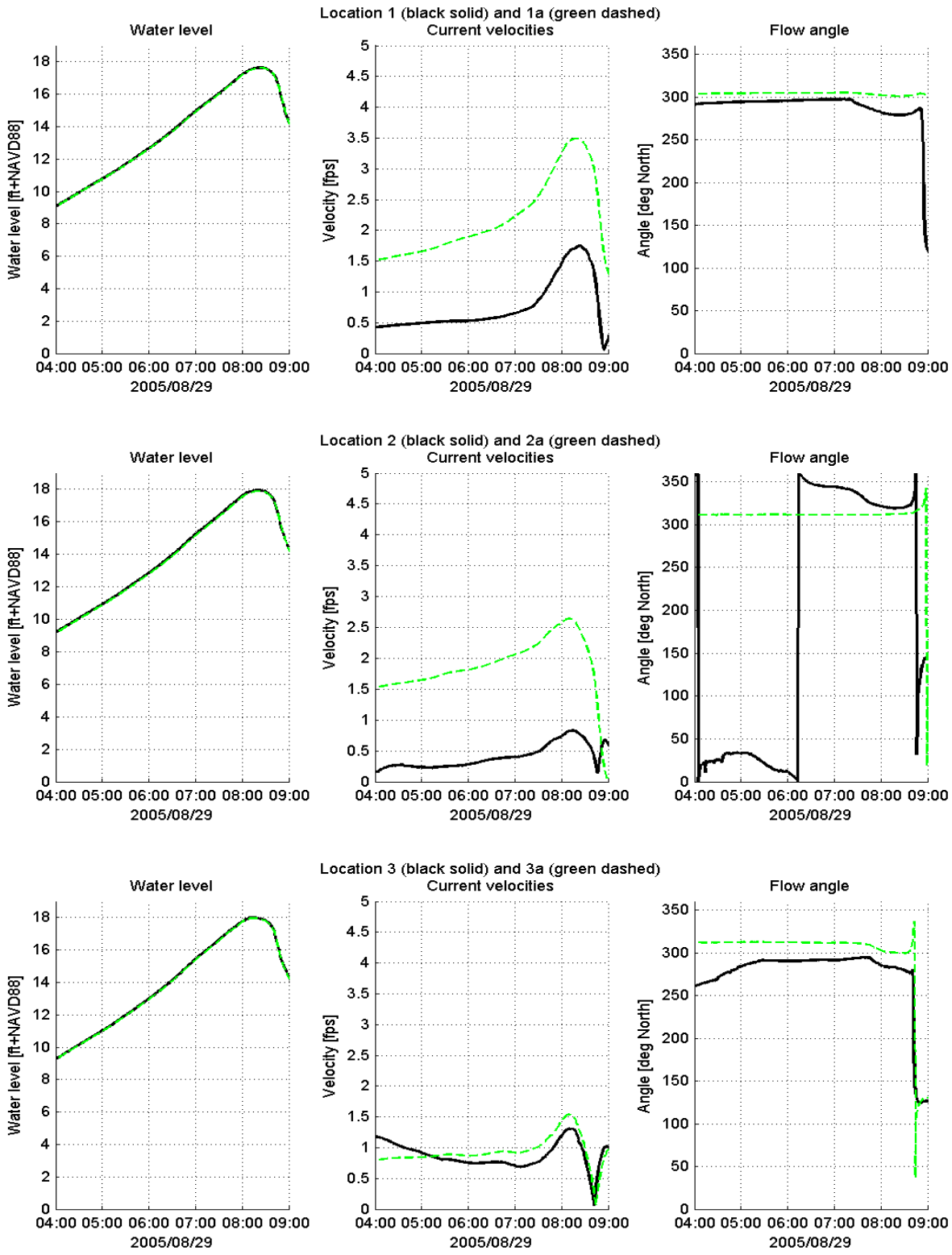


Figure 38: Results for Scenario 1 for locations 1, 2, and 3 (Figure 37) from 4:00 am to 9:00 am CDT. Note maximum surge elevation of 18 feet (NAVD 88) just after 8:00 am CDT (Wit et al 2008).

44. The third Hurricane Katrina hydrodynamic characteristic addressed by Wit, et al (2008) and Gautier, et al (2008) were waves (heights, periods, directions). Because of the sensitivity of the waves to local effects (bathymetry, topography, vegetation), a series of computational grids were used to evaluate wave characteristics in the Lake Borgne region (Figure 39). Details of the sub-grids for the areas of the GIWW and the MR-GO Reach 2 are shown in Figure 40. These sub-grids are needed because of the changing scales of detail that must be addressed as one simulates regional to area to location specific hydrodynamic conditions.

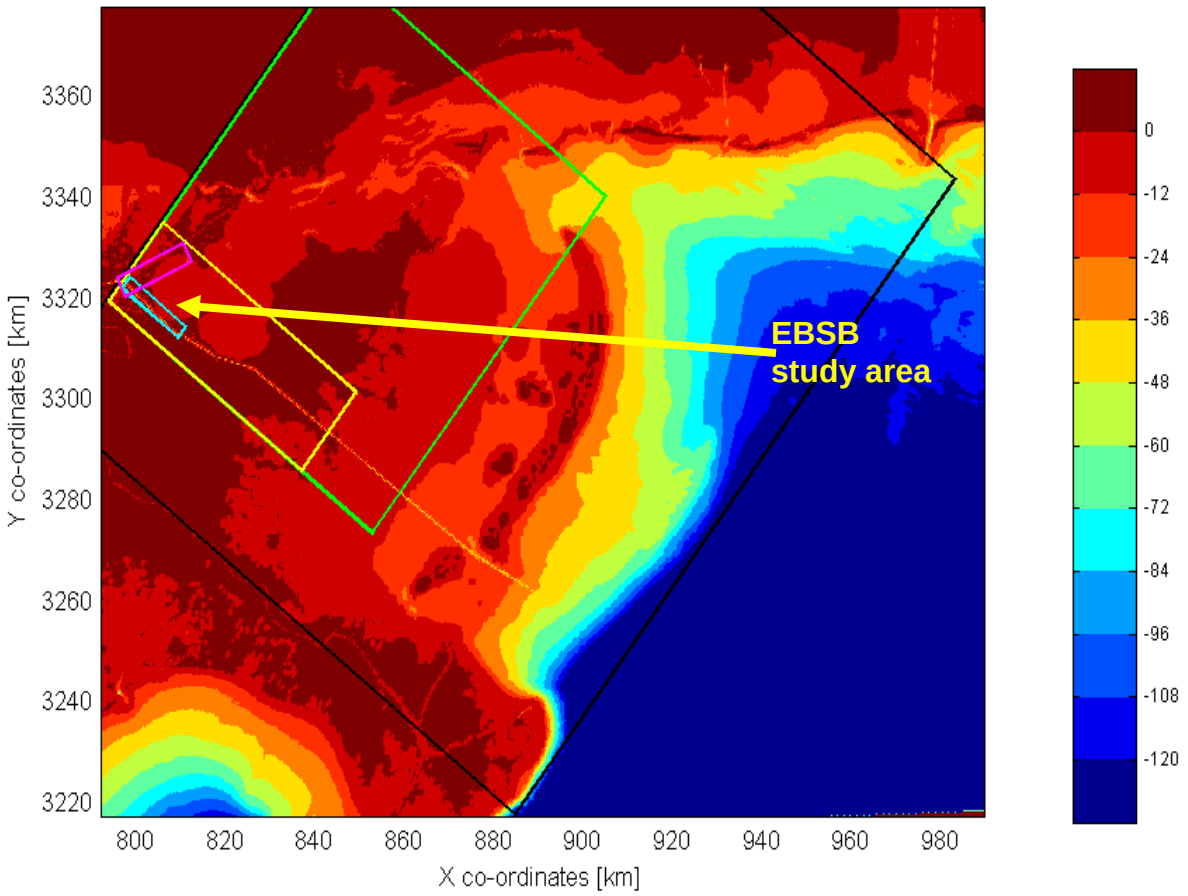


Figure 39: Position of computational grids and bathymetry (feet relative to NAVD 88). Black border A-grid, Green border C-grid, Yellow border M-grid, Cyan border F-grid, Pink border G-grid).

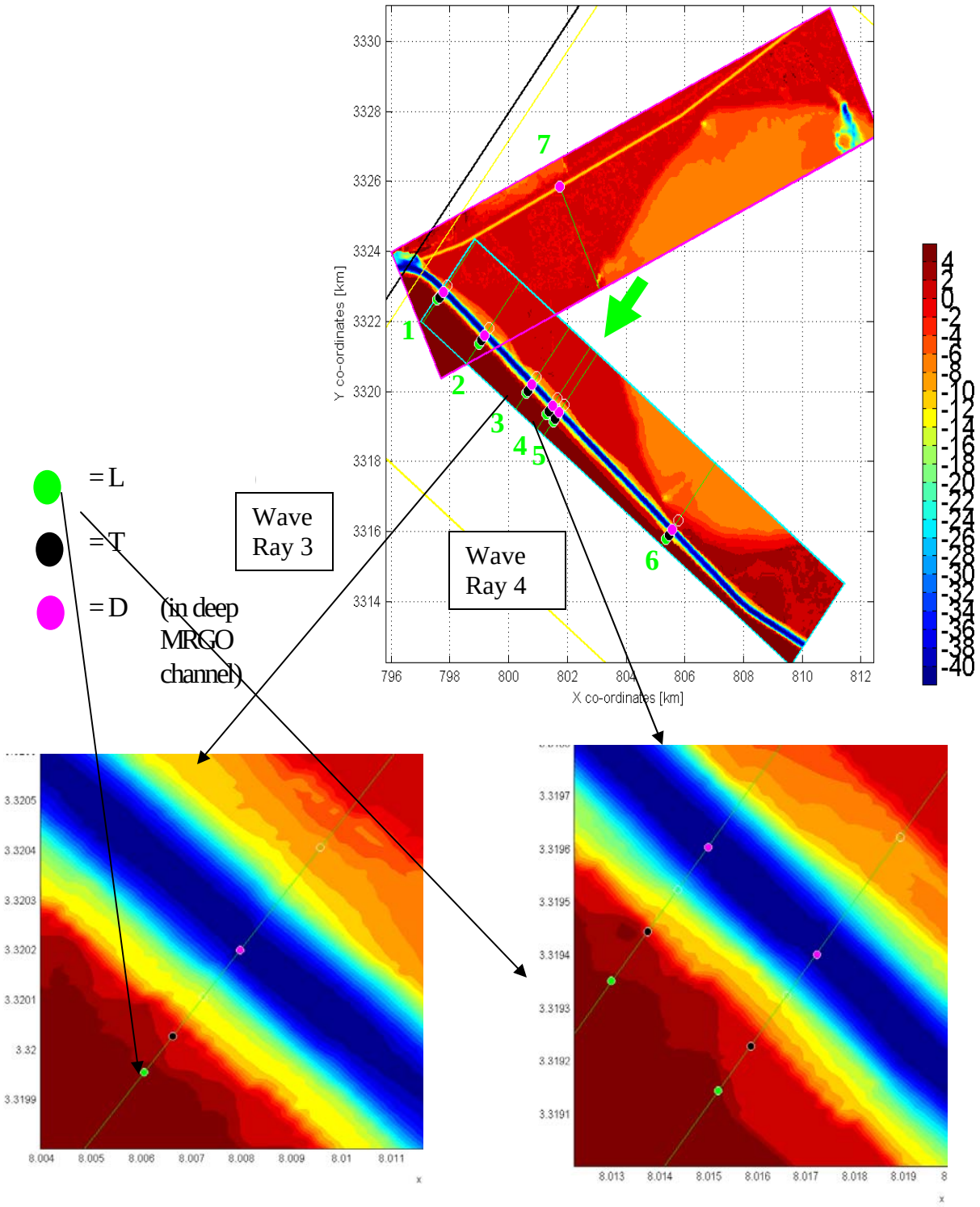


Figure 40: Bathymetry (feet NAVD 88) for F- and G-grids and Wave Rays 3 and 4 at MR-GO channel and EBSB study location (Gautier, et al 2008).

45. Of particular importance in evaluation and analyses of performance of the Reach 2 EBSBs and at the EBSB study location are wave characteristics along Rays 3 and 4, and at the positions located at the toes of the EBSBs indicated as 'L' (on levee, elevation approximately +5 to +6 feet (NAVD 88) (Figure 40).

46. Figure 41 shows the significant wave heights (Hs, average of the highest one third wave heights – reflecting the energy content of the sea state) in the MR-GO Reach 2 area during the peak conditions generated by Hurricane Katrina – Scenario 1 as was conditions. The waves are approaching the EBSBs essentially broadside with their crests aligned with the crest of the EBSBs – the most damaging approach direction.

47. Figure 42 shows propagation of the waves along Rays 3 and 4 during the most severe conditions (8:00 am CDT – close to the same time as the peak surge). The effects of bathymetry, topography, and roughness are evident. The waves decrease in height as they cross the area bordering Lake Borgne. When the waves encounter the deeper water of the MR-GO channel, their heights increase. When the waves cross over the other side of the MR-GO channel, their heights rapidly decrease – primarily due to shallow water wave shoaling and breaking. For the Hurricane Katrina peak conditions, the maximum significant wave heights at the toes of the EBSBs are approximately 6 feet. Note that these wave heights are very dependent on the water depth defined as the EBSB toe (L). For wave Ray 3, this position is close to the edge of the channel with the water depth close to mean sea level elevation. For wave Ray 4, this position is at the toe of the EBSB. These waves have 'peak periods' (Tp, the 'period' is the time between two successive wave crests, the 'peak period' is the period associated with the maximum energy in the sea state) of approximately 5.6 seconds.

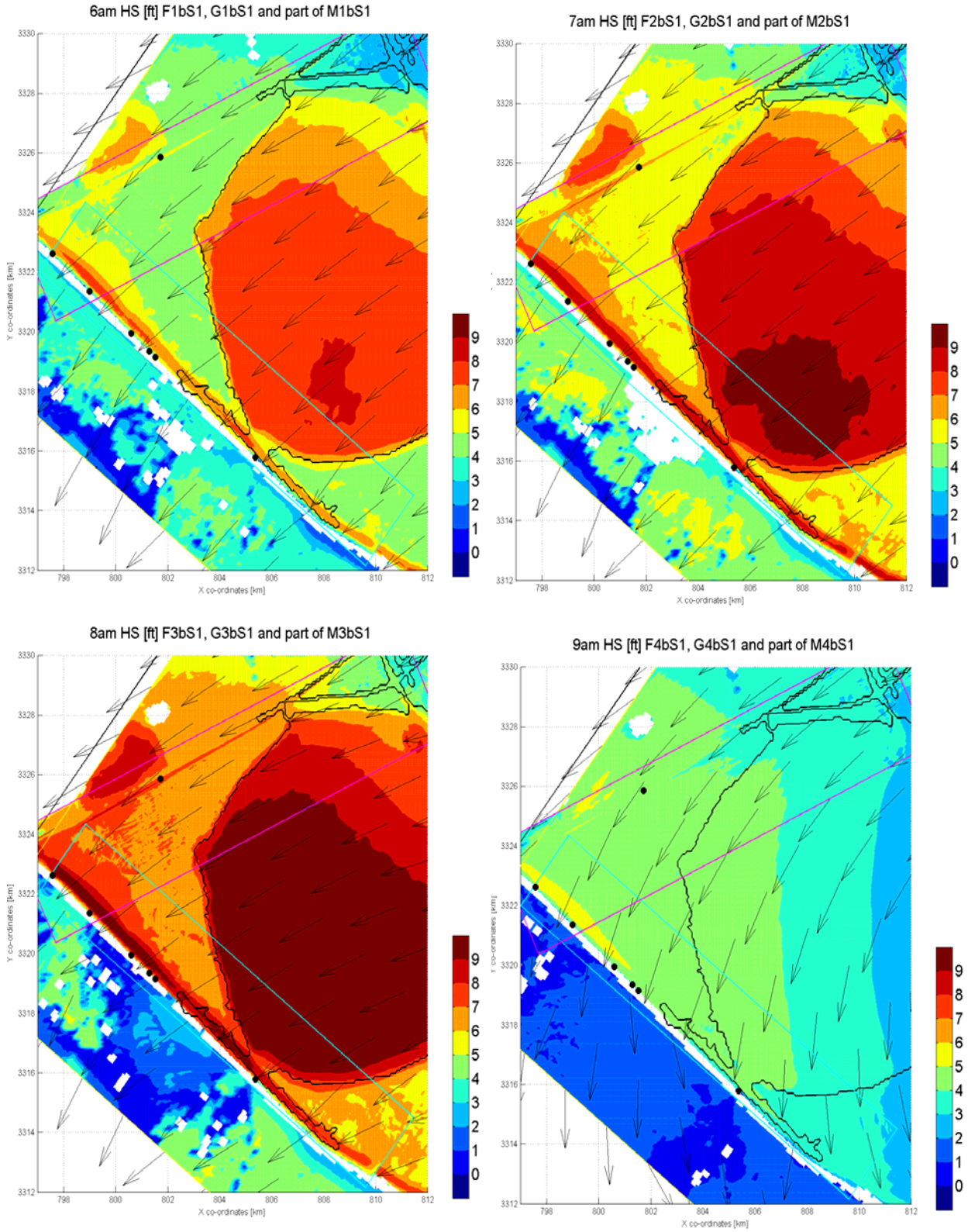


Figure 41: Significant wave heights (Hs, feet) and wave directions during period 6:00 am to 9:00 am CDT in the MR-GO Reach 2 area (Gautier, et al 2008).

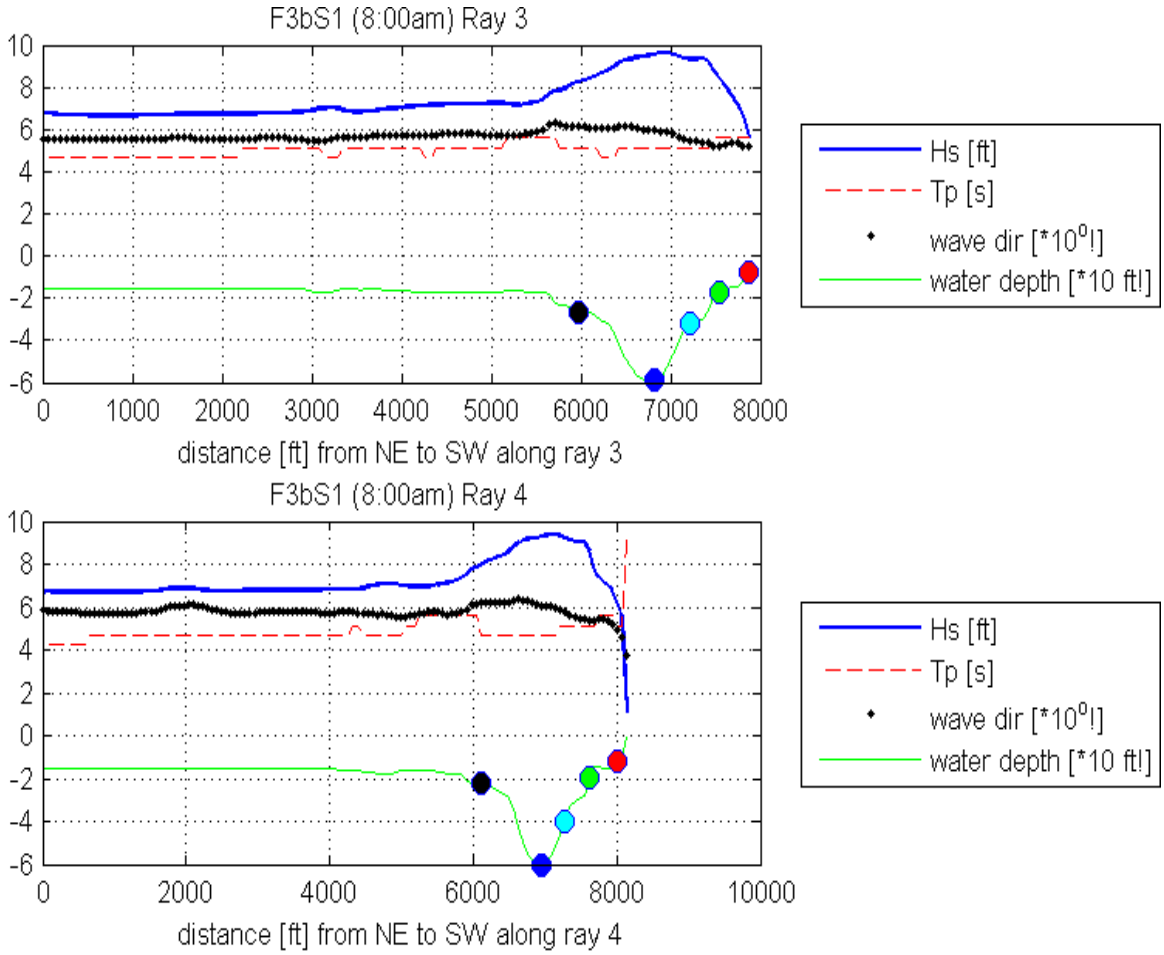


Figure 42: Hurricane Katrina 'as was' (Scenario 1) wave parameters on Rays 3 and 4 at 8:00 am CDT (Gautier, et al 2008).

48. Figure 43 summarizes the Hurricane Katrina 'as was' Scenario 1 significant wave heights for Location 3 (EBSB study site, Ray 3). Also shown are the surge elevations provided by Wit, et al (2008) and computed shallow water 'breaking' wave heights based on the USACE STWAVE breaking wave height model (Smith 2008, IPET 2007) and an earlier breaking wave height model developed by Bretschneider (1954) based on measured wave heights in Lake Okeechobee, Florida during storm conditions (very comparable with Lake Borgne conditions). As noted earlier, the peak wave heights are closely related in time to the

peak surge elevations. Further, the maximum wave heights at the toe of the EBSB – Levees are governed by the water depths at this point and shallow water wave shoaling and breaking processes. The results provided by Wit, et al (2008) and Gautier, et al (2008) corroborate these observations.

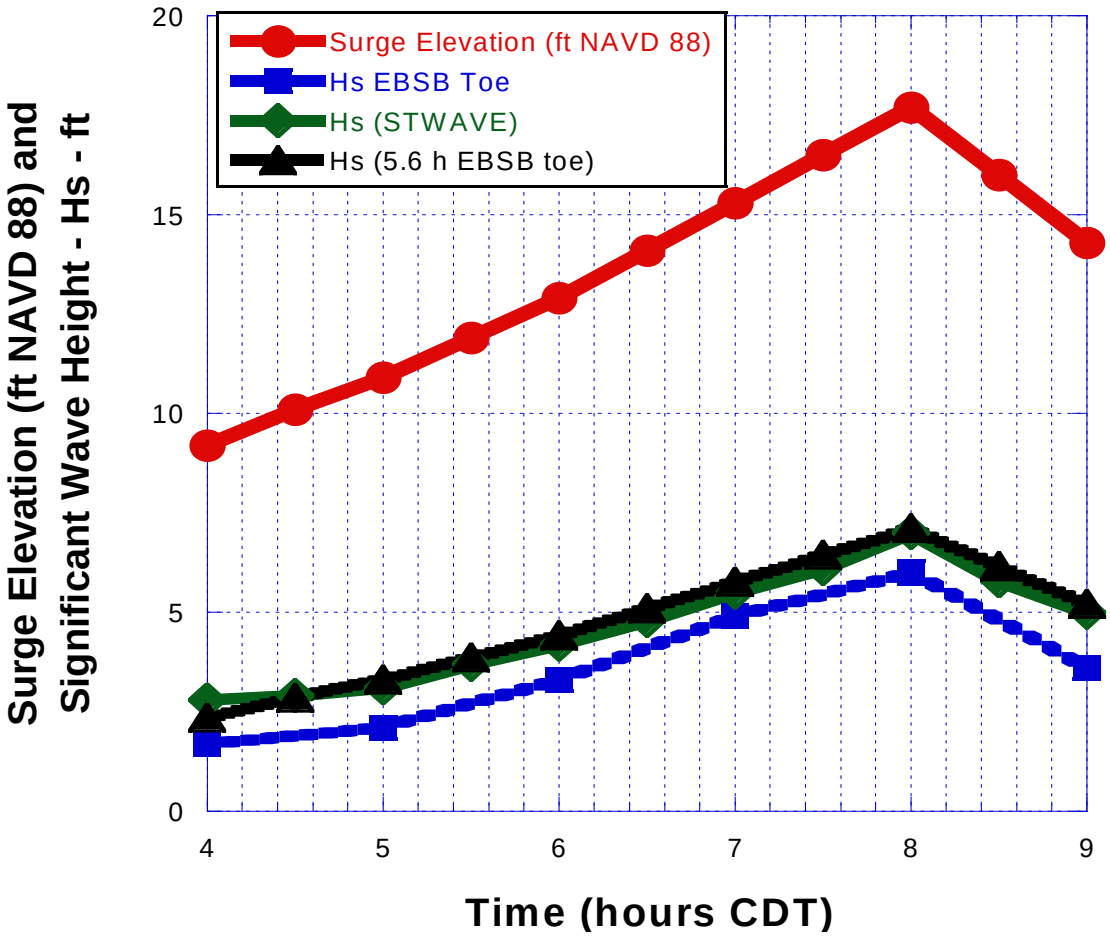


Figure 43: Hurricane Katrina 'as was' (Scenario 1) surge elevation and significant wave height conditions at MR-GO Reach 2 Location 3 – Ray 3 (EBSB breaching study site) based on analyses performed by Wit, et al (2008) and Gautier, et al (2008). Local shallow water wave height breaking 'limits' shown are based on USACE STWAVE model (2006, Smith 2007) and Bretschneidner model (1954).

49. Figure 44 summarizes Hurricane Katrina 'Neutral' MR-GO (Scenario 2C) wave height characteristics along Rays 3 and 4. Figure 45 shows a summary of the Hurricane Katrina 'Neutral' MR-GO (Scenario 2Ci) surge elevation and significant wave height conditions at the MR-GO Reach 2 Location 3 – Ray 3 EBSB breaching study site (location

L) and adjacent locations (Ray 4 EBSB location L) based on analyses performed by Wit et al (2008) and Gautier et al (2008). The significant wave heights are reduced by the 'Neutral' MR-GO channel (area occupied with native vegetation) and the vegetation on the Lake Borgne side of the EBSB Reach 2 alignment. The peak condition maximum significant wave heights at the toes of the EBSBs have been reduced to in the range of 4.0 to 4.6 feet (Figure 41).

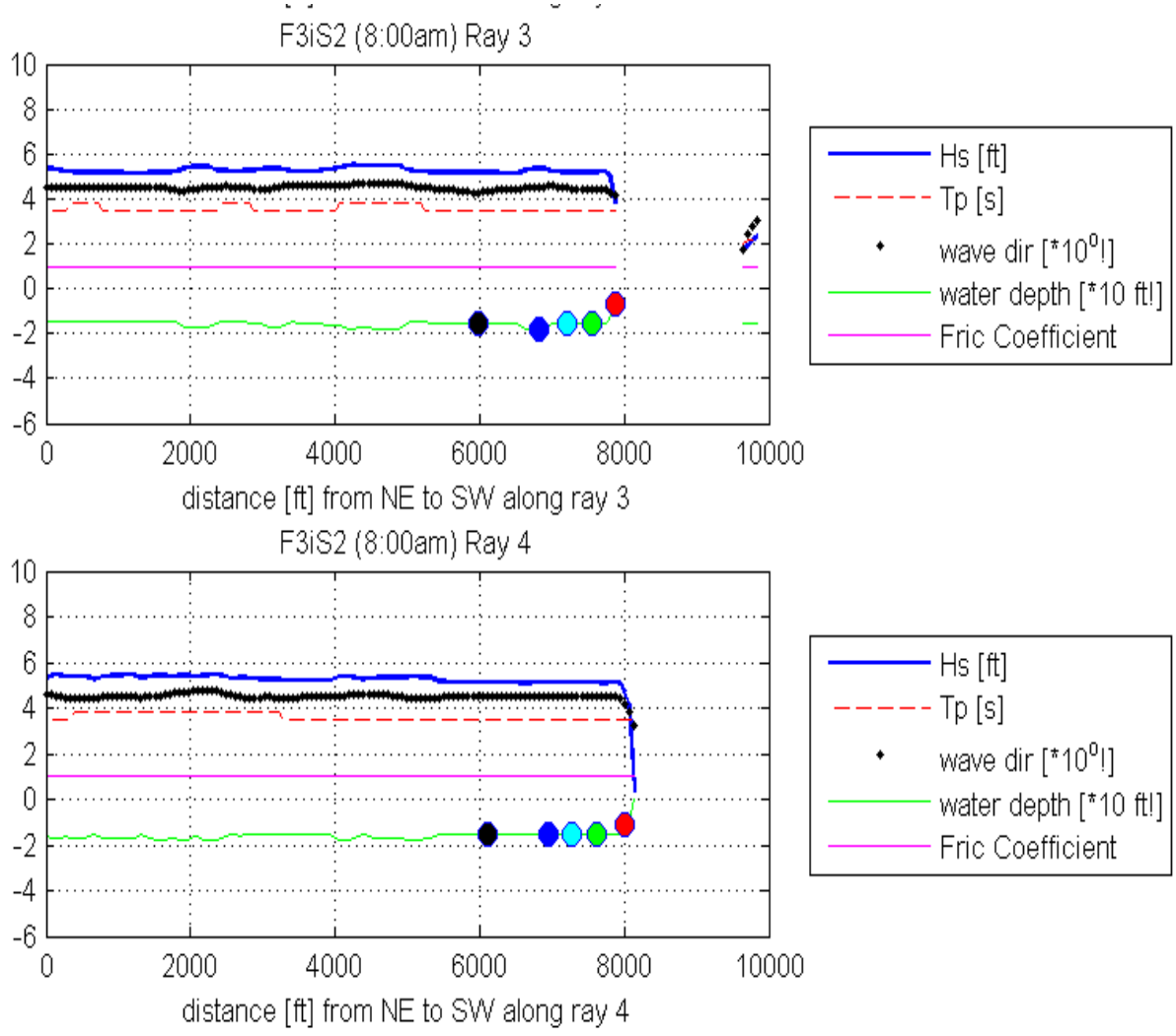


Figure 44: Hurricane Katrina 'Neutral' MR-GO conditions (Scenario 2c) wave parameters on Rays 3 and 4 at 8:00 am CDT (Gautier, et al 2008).

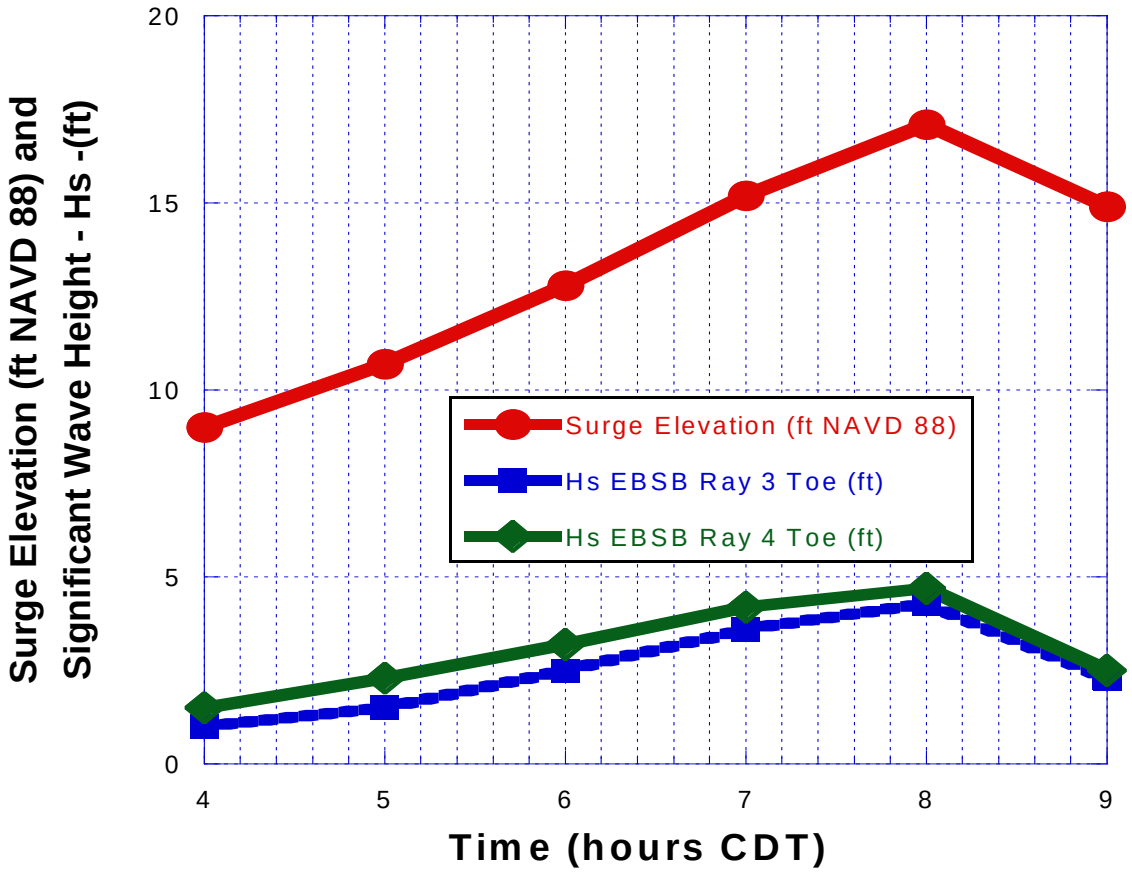


Figure 45: Hurricane Katrina 'Neutral' MR-GO conditions (Scenario 2Ci) surge elevations and significant wave heights at MR-GO Reach 2 Location 3 – Ray 3 (EBSB breaching study site) and Location 4 – Ray 4 (south of EBSB study location) based on analyses performed by Wit, et al (2008) and Gautier, et al (2008).

LOCAL VEGETATION EFFECTS ON HURRICANE KATRINA WAVE CONDITIONS

50. The results developed by Wit, et al (2008) and Gautier, et al (2008) have addressed the effects of the vegetation that existed in the Lake Borgne region at the time of Hurricane Katrina – the MR-GO 'as was' Scenario 1 conditions. Also, they have addressed the effects of the vegetation that would have existed in Lake Borgne region at the time of Hurricane Katrina – the 'Neutral' MR-GO conditions. However, because of 'scale' constraints in these simulations, the established vegetation existing between the MR-GO channel alignment and the toes of the hurricane flood protection EBSB 'berms' at the time of occurrence of Hurricane Katrina (Scenario 1), nor the established vegetation that would have

existed between the MR-GO channel alignment and the toes of the hurricane flood protection EBSB ‘berms’ were included in the hurricane condition simulations. Figures 46 - 49 are photographs of the vegetation that existed following Hurricane Katrina along portions of this zone. Analysis of the available 2000 LiDAR survey data gathered along this alignment indicates an average elevation of this vegetation of +10 feet NAVD 88 (Morris 2008). The width of the zone varied between an average of 200 to 300 feet with a minimum of zero feet – the banks of the MR-GO channel were at the toes of the EBSB berms. Often these maximum MR-GO channel encroachments into the EBSB alignment were defended with rock riprap (Figure 49).

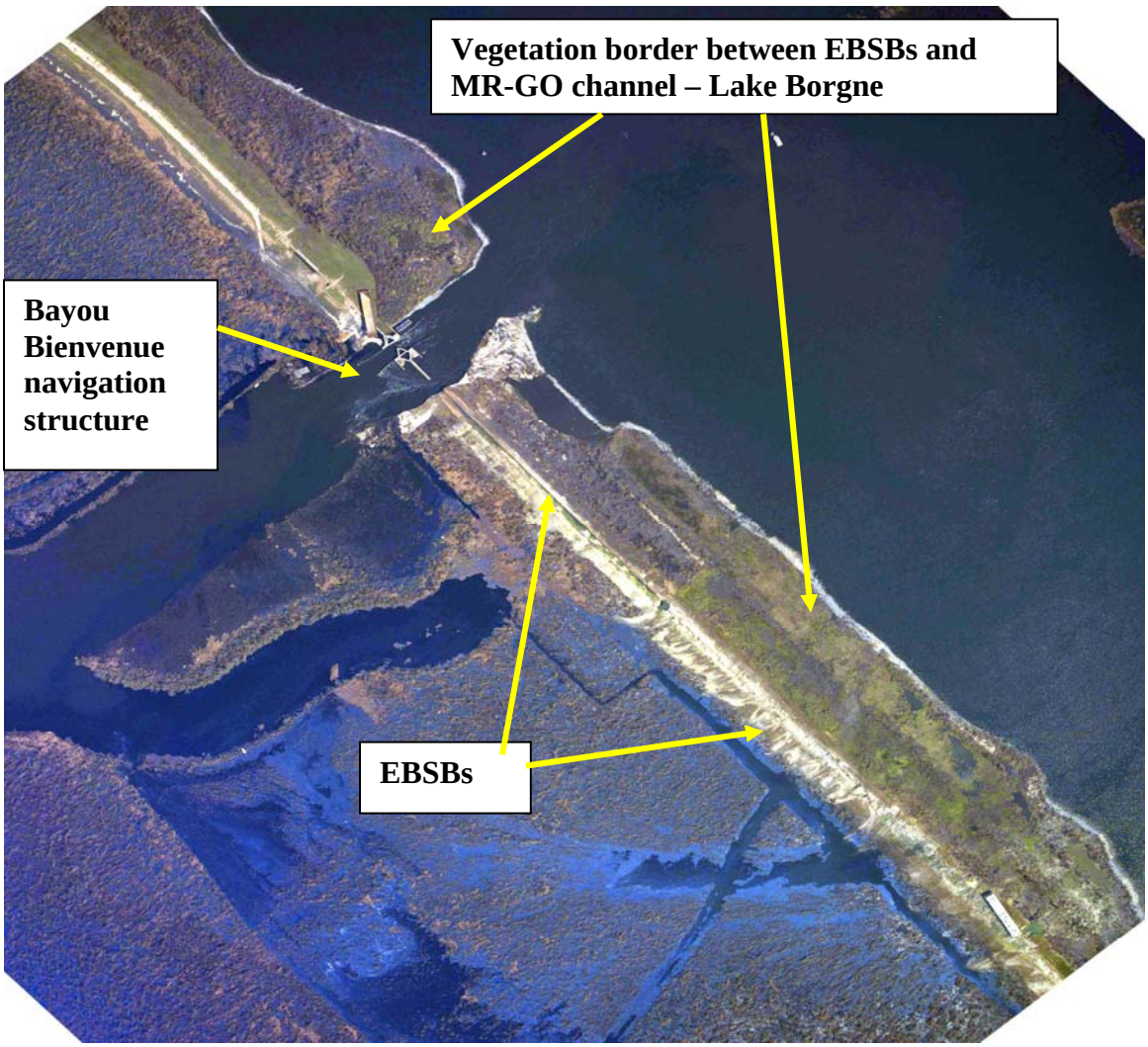


Figure 46: Aerial photograph showing vegetation border on Lake Borgne side of EBSBs in vicinity of Bayou Bienvenue following Hurricane Katrina (NOAA 2005).



Figure 47: Aerial photograph showing vegetation border on Lake Borgne side of EBSBs in vicinity of Bayou Bienvenue following Hurricane Katrina (ILIT 2006).



Figure 48: Photograph showing vegetation border on Lake Borgne side of EBSBs in vicinity of Bayou Dupre following Hurricane Katrina (Bea 2006).

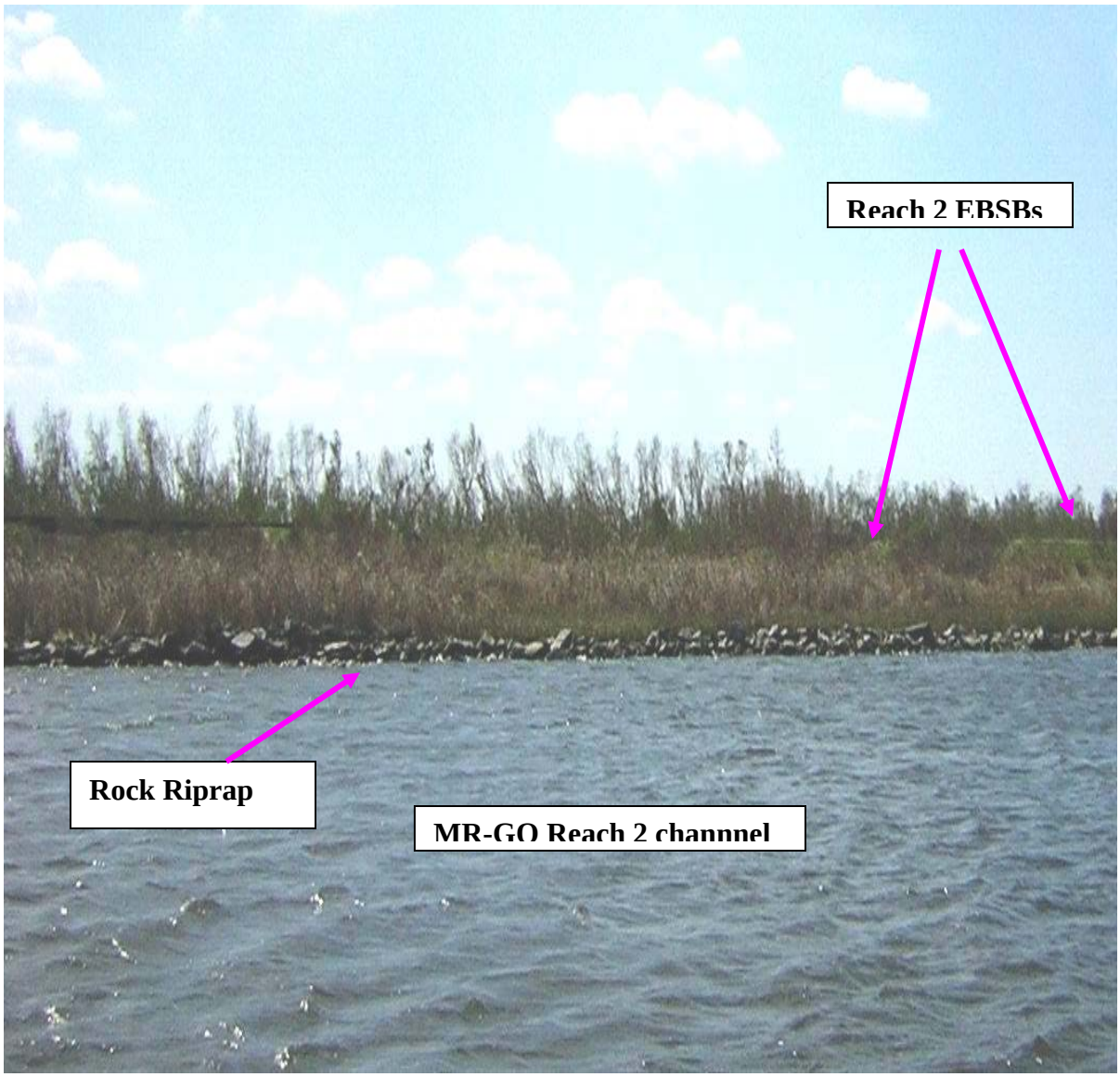


Figure 49: Photograph of the vegetation and rock riprap between the toes of the EBSBs and the MR-GO channel mid way between Bayou Dupre and Bayou Bienvenue (looking toward the Reach 2 EBSBs from the MR-GO channel) (Storesund 2005).

51. In many cases there were good correlations between the presence of this defensive line of vegetation and the lack of any significant erosion or breaching of the EBSBs during Hurricane Katrina. The processed post-Hurricane Katrina LIDAR information summarized in Figure 52 is an example of one such case along the MR-GO Reach 2 EBSB alignment mid-way between Bayou Bienvenue and Bayou Dupre (Morris 2008). This section

of the EBSBs was heavily overtopped during Hurricane Katrina. As shown by the post Hurricane Katrina cross section of the EBSBs and the defensive line of vegetation (inset cross section of EBSB Figure 50), there was no significant damage to the elevation or slopes of the EBSBs. Figure 51 shows the comparable 'reverse' correlation data (Morris 2008). In this case there was no protective vegetation, and the EBSB was virtually destroyed (Figure 51 inset cross section of EBSB). These two sections of the EBSB were adjacent to each other and had comparable profiles and crown elevations. It is reasonable to expect that the Hurricane Katrina surge, wave, and current characteristics were very comparable and that the soils comprising the two sections of the EBSBs were comparable (dredged spoil), and that the protective surface vegetation (grass, turf) was comparable. However, the performance of these two sections of the EBSBs was dramatically different.

52. The extensive field investigations performed as part of this study clearly showed that when there were combinations of good defensive vegetation in the forms of outboard vegetation and grass or turf covering of the levees and the levees were composed of compacted cohesive soils, the performance of the levees was excellent even though they had been severely overtopped with the surge from Hurricane Katrina (ILIT 2006, Seed et al 2007 a, 2008a). Figure 52 shows a photograph of the levees that defended the eastern flank of the New Orleans East polder. As indicated by the debris on the protected side, this levee had been heavily overtopped. However, due to the combination of wave side vegetation, grass on the levee slope, and a levee built of compacted cohesive soils, this levee survived without any significant damage.

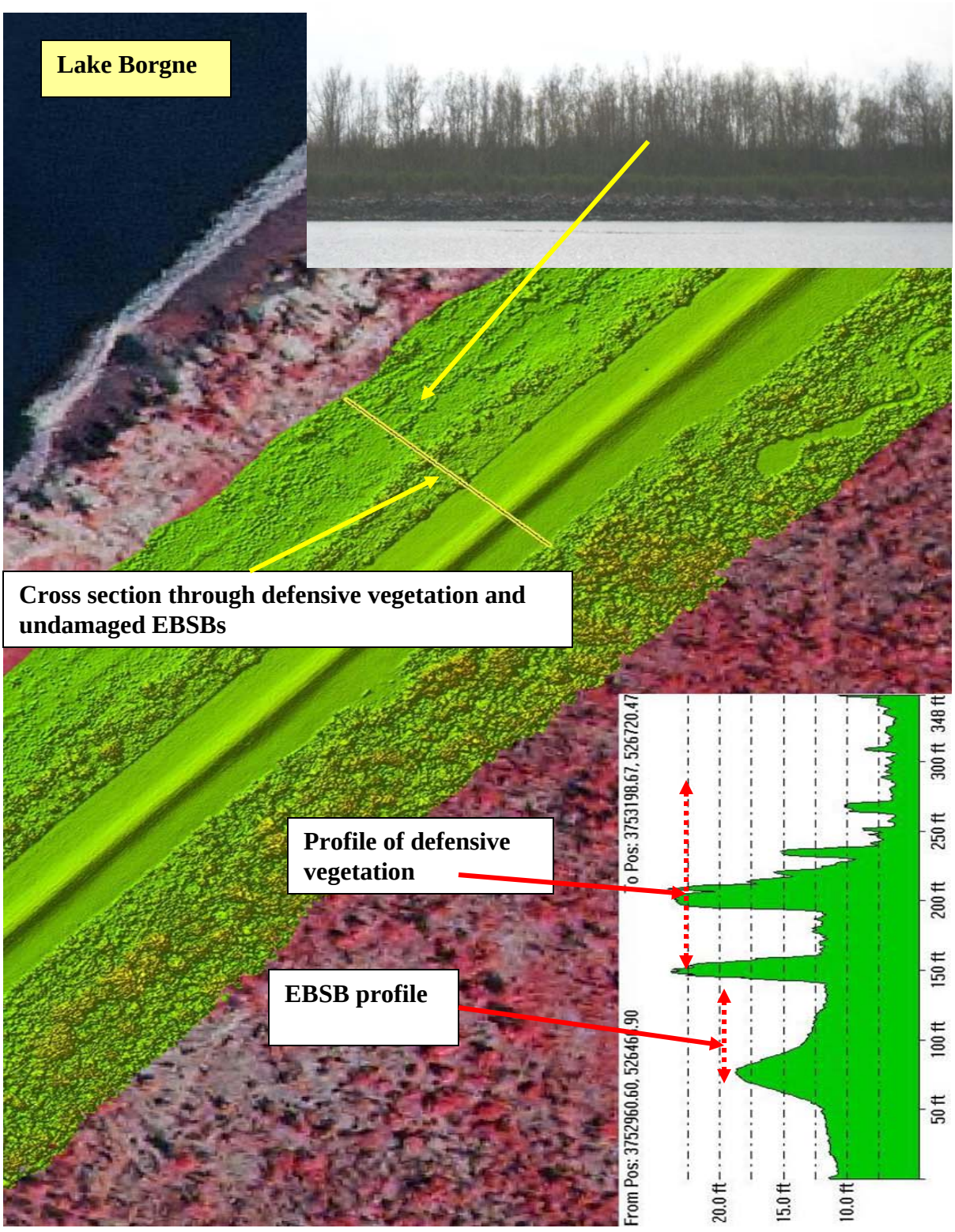


Figure 50: Processed LiDAR post-Hurricane Katrina survey data showing the substantial defensive line of vegetation outboard of the MR-GO Reach 2 EBSBs and the undamaged profile of the EBSBs in this section of the alignment (Morris 2008).

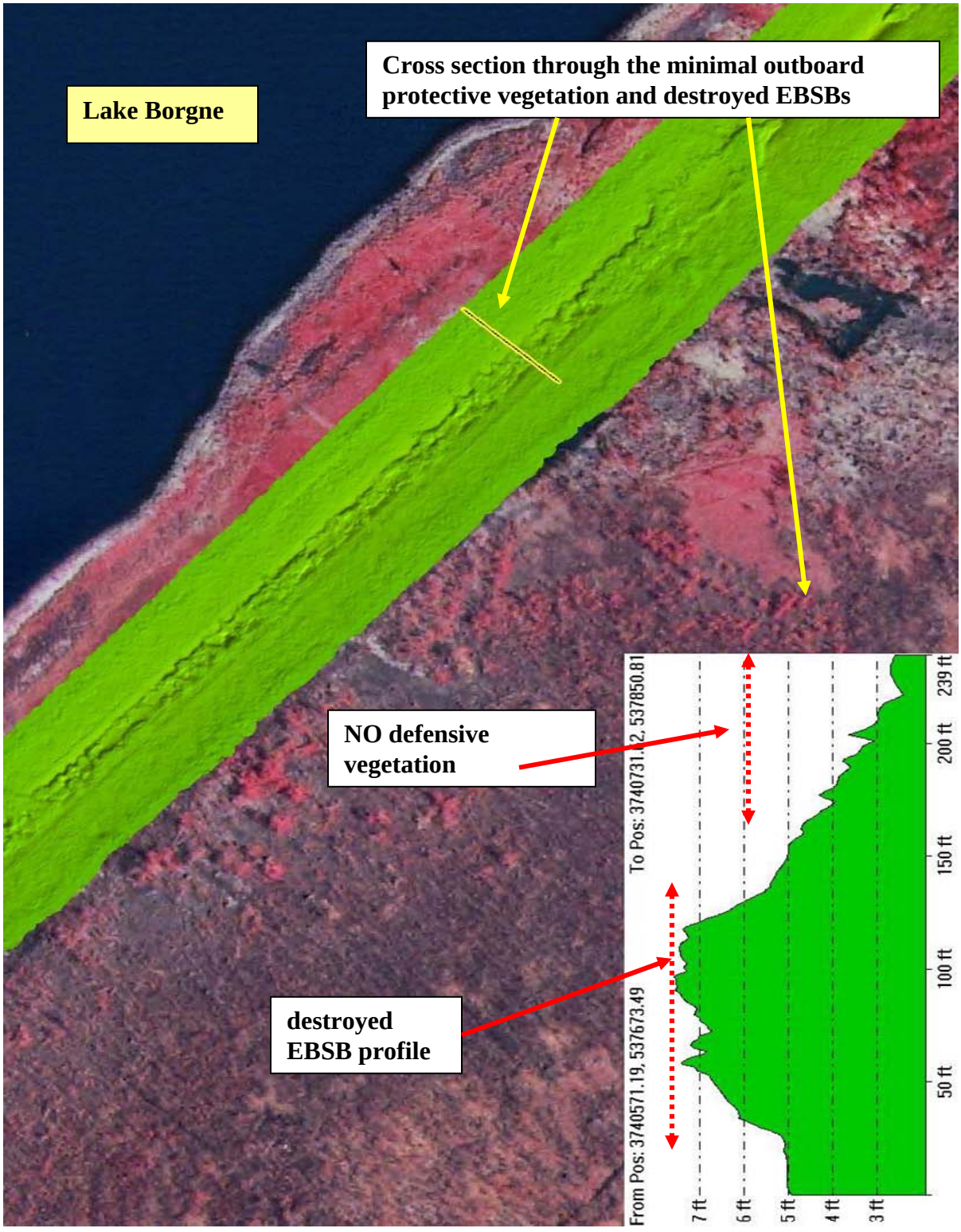


Figure 51: Processed LiDAR post-Hurricane Katrina survey data showing NO defensive line of vegetation outboard of the MR-GO Reach 2 EBSBs and the breached profile of the EBSBs in this section of the alignment (Morris 2008).



Figure 52: Photograph of heavily overtopped levee defending the east side of the New Orleans East polder taken following Hurricane Katrina (Bea 2005). Note the heavy wave side vegetation and densely grassed levee crown and slopes. This levee survived without damage.

53. The importance of the vegetation zone between the EBSB and the MR-GO channel regards its role in providing natural defenses or barriers against incoming hurricane waves and currents. The effects of such natural defenses – vegetation – have been studied extensively for a wide variety of environmental settings. These studies have involved both development of analytical models, gathering field and laboratory experimental data, and validation / calibration of the analytical models with the experimental data. These studies have shown that natural vegetation defenses (exposed and submerged) can have dramatic effects on waves and currents. An excellent review and summary of a large portion of this body of technology has been provided recently by URS in a study performed for the USACE (URS Corporation 2007) and by Tschirky (2000). Other landmark studies and reports

contributing to this body of technology include that provided by Cramfield (1977, 1983); Reid (1976); Dean and Dalrymple (1984); Dalrymple et al (1984); Knutson (1982, 1988); Freeman et al (2000); Massel et al (1999); Asano et al (1992); NRC (2007); Kahan et al (2006); and Kobayashi et al (1993).

54. The fundamental conclusions from this body of knowledge is that vegetation can have important effects on waves and currents that depend on the characteristics of the hydrodynamic flow regimes (wave heights and periods, current velocities, water depth), vegetation density, vegetation diameters and heights relative to the water depths, and vegetation shapes and stiffness (Freeman et al, 2000; Tschirky, 2000). Wave and current damping increase with the density, stiffness, and diameters of the vegetation and vice versa, and exposure of the vegetation above the water. Wave and current damping decrease with the intensity of the flow regime (laminar to turbulent), increase in the flexibility of the vegetation, and increasing submergence of the vegetation.

55. Guidelines to address decay of wind waves (waves developed locally within the wind field area – similar to the dominant wave components in Lake Borgne Hurricane Katrina conditions) propagating over vegetated areas have been provided by Cramfield in work done for the USACE (1977, 1983). The guidelines summarized in Figure 53 (Cramfield 1977) indicate that wave decay friction factors associated with vegetation can be up to 100 times the friction factor associated with unvegetated shallow water. Friction factors for various vegetation types are given as a function of water depth including a) thick stands of marsh grass, b) dense grass, brush or bushy willows and c) scattered trees, and dense stands of trees. Based on a water depth of 10 feet, the friction factor for marsh grass is 0.2, for dense grass and brush is 0.48, and for dense stands of trees is 0.90. Given that the IPET wave

analyses were based on a normal bottom friction factor of 0.02, the values summarized in Table 4 appear to be unrealistically conservative for normal coastal vegetation conditions.

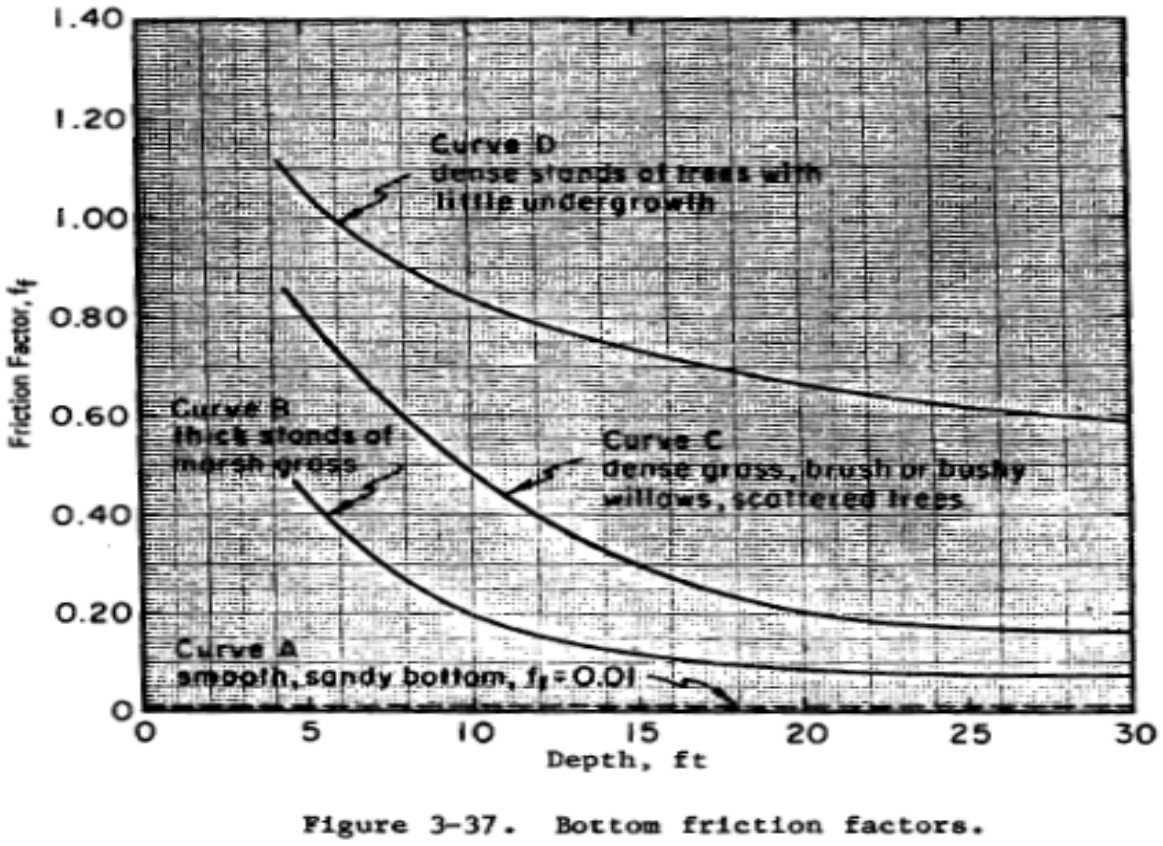


Figure 53: Guidelines developed by Cramfield for USACE to address decay of wind waves propagating over vegetated areas. This graph was published in the USACE Shore Protection Manual (1988).

56. Dean (1978), Dalrymple et al (1984), Dean and Dalrymple (1984), Massel et al 1999, and Knutson, et al (1982, 1988) developed analytical models to characterize shallow water wave dissipation developed by propagation through and over vegetated areas. These analytical models have been validated with laboratory and field experimental results (e.g. Tschirky 2000, Knutson 1988). These analytical models characterize wave decay as functions of a vegetation drag coefficient (recognize shape, roughness, flexibility, flow regime – intensity - turbulence), the average diameter, spacing and height of the vegetation, the incident wave height and the propagation distance. Figures 54 and 55 summarize results

from these analytical models for two characterizations of coastal vegetation identified as “Light” and “Dense”.

57. The vegetation drag-flow characteristic (Cd or cd*cp) for these two designations were based on the experimental work performed and analyzed by Freeman et al (2000), Knutson (1988), Tschirky (2000), Jarvela (2004), and Wu et al (1999). The vegetation physical characteristics for these two designations were based on the report published by Penland et al (2008) (also Penland/FitzGerald et al 2008) and on field vegetation survey data provided by Dr. Gary Shafer (Southeaster Louisiana University, 2008). A vegetation drag coefficient of Cd = 0.5 was used for these conditions. Incident significant wave heights of 2 and 8 feet with periods of 3 and 5 seconds in water depths of 5 and 15 feet were used to evaluate the wave decay characteristics. The’ Light’ vegetation was described as having average diameters of 0.1 foot, clear spacings of 1.0 foot, and heights of 5 feet. The ‘Dense’ vegetation was described as having average diameters of 1.0 foot, clear spacings of 2.0 feet, and heights equal to the water depths of 5 and 15 feet. Both the Dean and Dalrymple (1984) and Knutson (1988) analytical models were used to calculate wave height dissipation.

58. Results summarized in Figures 54 and 55 show that the significant wave heights are reduced to less than 30 percent of the incident heights in a travel distance of 200 feet. ‘Light’ and ‘Dense’ vegetation characteristics produce very significant wave height reductions. These wave height reductions are much higher than those based on ‘traditional’ wave decay analytical models utilizing traditional bottom friction coefficients. This wave decay is in agreement with measurements made in the field under comparable conditions (Massel et al 1999; Kobayashi et al 1993; Tschirky 2000; Moller et al 1999; Knutson 1988).

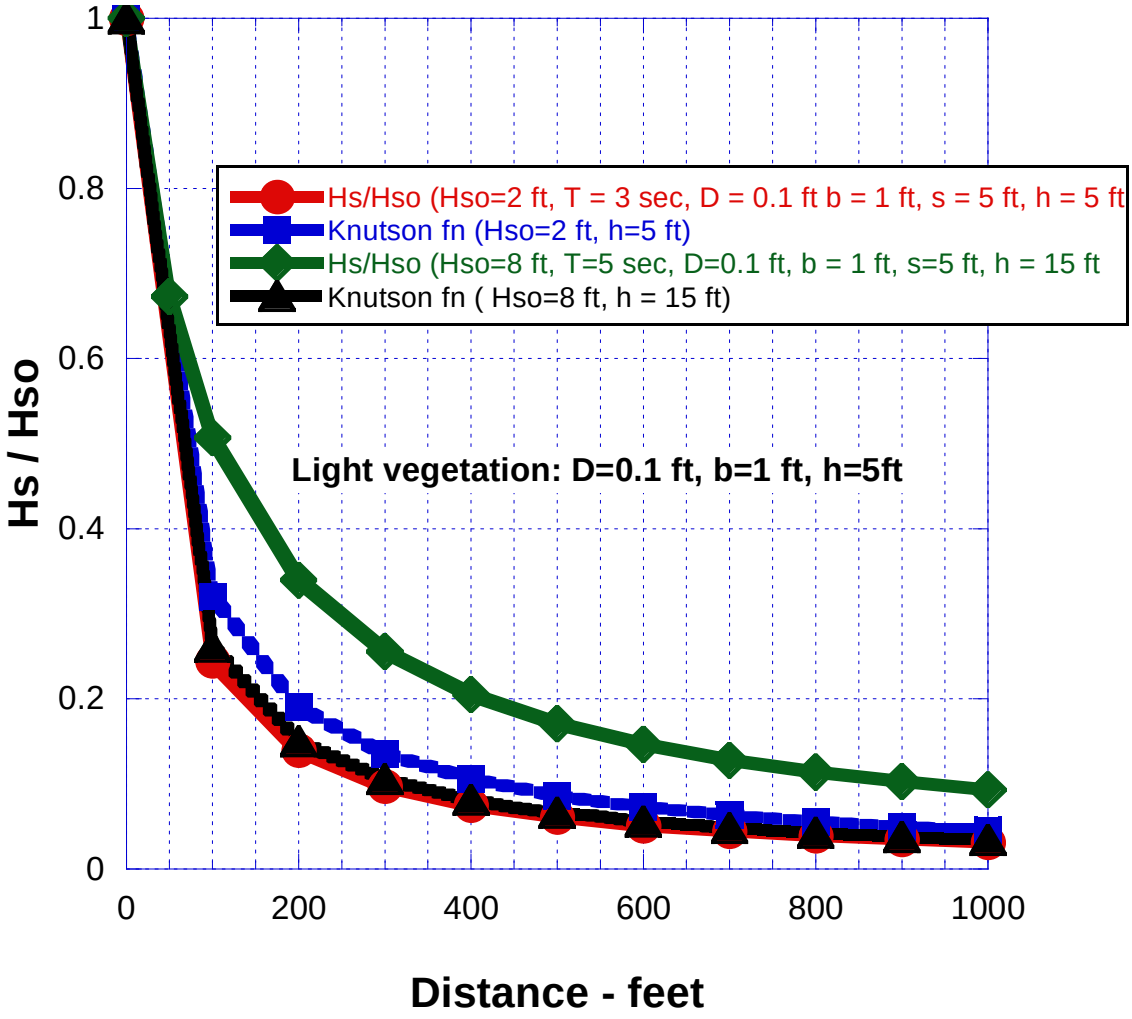


Figure 54: Decay of incident significant wave height (Hso) as function of propagation distance through 'Light' vegetation.

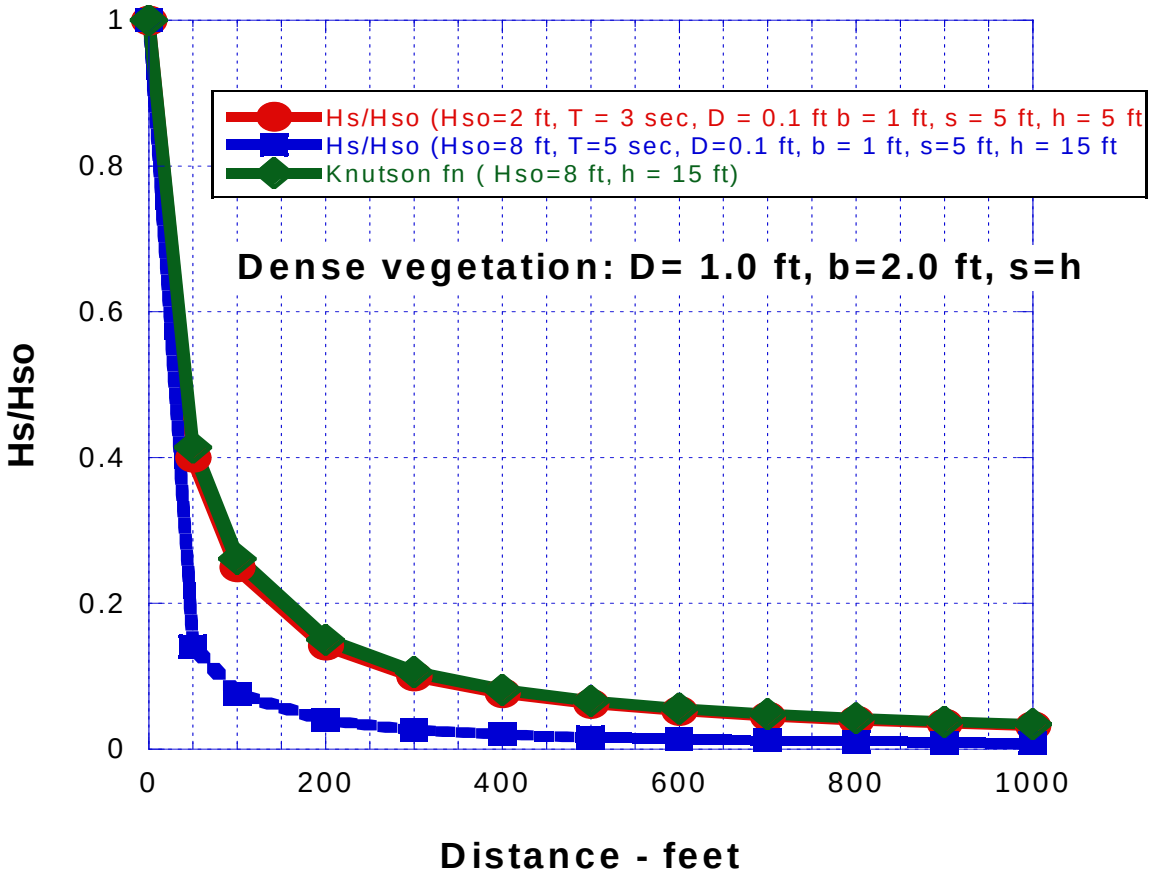


Figure 55: Decay of incident significant wave height (Hso) as function of propagation distance through 'Dense' vegetation.

59. Guidelines to address vegetation effects on decay of wind waves also have been addressed by Delft University of Technology, SVASEK Hydraulics, and the Cambridge Coastal Research Unit, University of Cambridge, UK (Gautier, et al 2008). The formulation expresses the vegetation effects on waves as a 'Collins friction coefficient' – Cf_{veg} (Collins (1972):

$$Cf_{veg} = Cw * d * n * h_{veg}$$

Where Cw is a measure of the vegetation stem rigidity (0 to 1), d is the average plant diameter in meters, n is the average number of plants per square meter, and h_{veg} is the stem height of the vegetation. Based on the description of the types of vegetation that existed on

the east side (Lake Borgne side) of the authorized MR-GO channel in the 1950s (time before MR-GO adverse effects on vegetation) provided by Penland et al (2008) and Penland/FitzGerald et al (2008) (summarized in Figure 56), three types of vegetation were characterized: 1) Brackish Marsh, 2) Fresh (water) Marsh, and 3) Swamp and Trees. The Brackish Marsh was characterized as composed of oyster grass, cord grass, salt grass, and salt meadow. The Fresh Marsh was characterized as composed of reeds, maiden cane, spike rush, coastal shrubs and button brush. The Swamp and Trees were characterized as composed of cypress, willow, sugarberry, cedar elm, and hackberry trees. These characterizations of the native vegetation and habitats to the east of the MR-GO channel were corroborated with photographs taken at the time of construction of the MR-GO (Figures 57 – 59).

Based on the on field vegetation survey data provided by Dr. Gary Shafer (Southeastern Louisiana University, 2008), the following characteristics were used as input to determine the Collins Friction Coefficient for the Brackish and Fresh Marsh: Cw = 0.5 (measure of the vegetation stem rigidity), d = 0.01 and 0.02 meters (average plant diameter), n = 49 per square meter (average number of plants), and h_{veg} = 2 meters (stem height of the vegetation). These parameters resulted in Collins Friction Coefficients of 0.49 and 0.98. For the Swamp and Trees the following coefficients were used: Cw = 0.9 (measure of the vegetation stem rigidity), d = 0.2 meters (average plant diameter), n = 0.9 per square meter (average number of plants), and h_{veg} = 12 meters (stem height of the vegetation). These parameters resulted in Collins Friction Coefficient of 2.0. These coefficients are in good general agreement with those described earlier. These coefficients were used as background to define the parameters used in the ‘‘Neutral’’ MR-GO Hurricane Katrina wave simulations (Gautier, et al 2008).

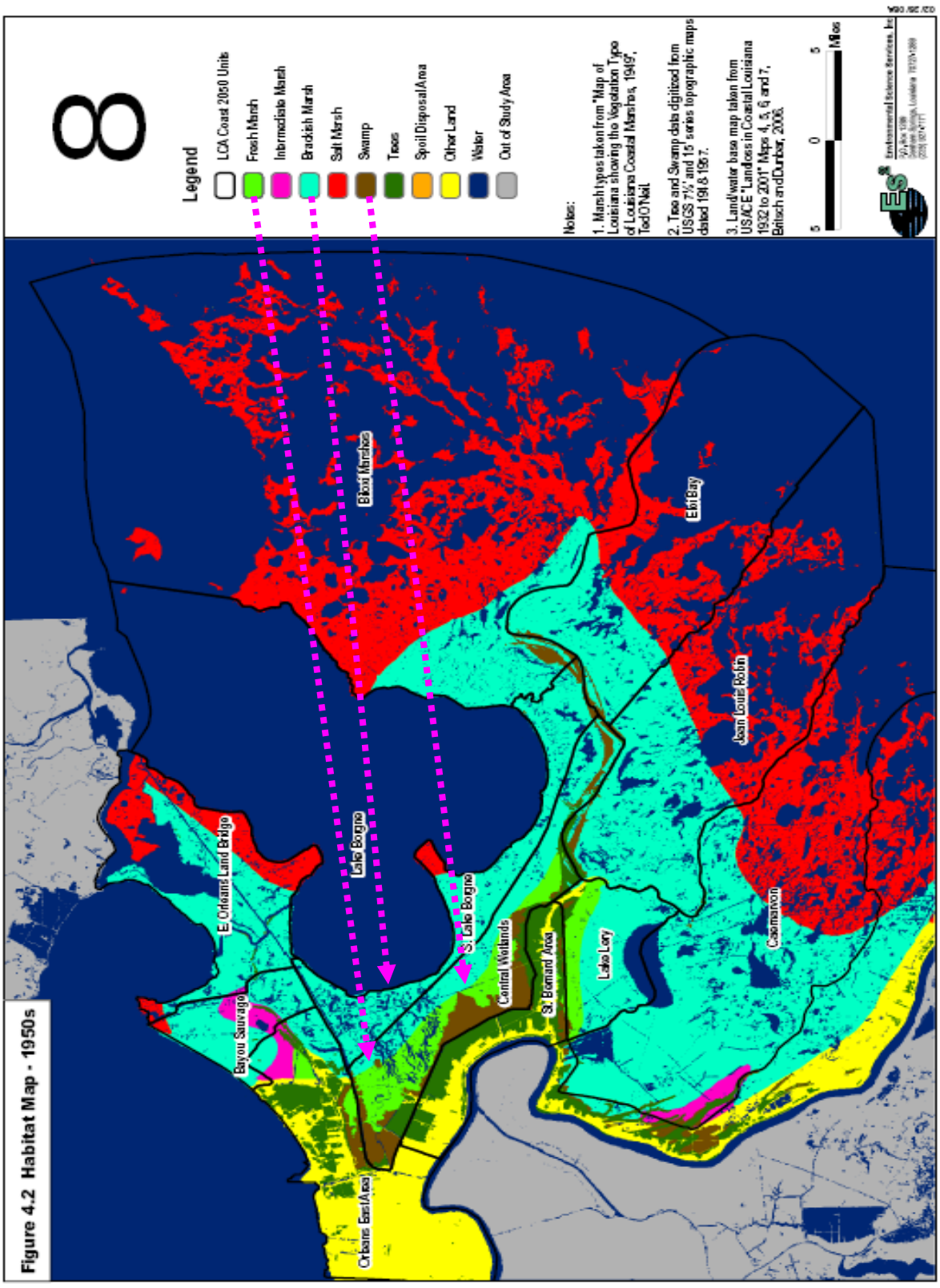


Figure 56: Habitat vegetation map – 1950s (Penland et al 2008, Penland/FitzGerald et al 2008).



Figure 57: Photograph of dredge under contract to USACE cutting access channel for the future MR-GO Reach 2 channel (USACE photograph from the New Orleans Public Library). Note spoil area being developed to left of photograph.



Figure 58: Photograph of dredge under contract to USACE cutting access channel for the future MR-GO Reach 2 channel (USACE photograph from the New Orleans Public Library). Note dredge spoil being deposited between the dikes that parallel the future MR-GO channel.



Figure 59: Photograph of dredge under contract to USACE channel for the future MR-GO - GIWW Reach 1 channel (USACE photograph from the New Orleans Public Library). Note the dense cypress tree forest in background. Forest logged, stumps removed in foreground.

EFFECTS OF GRASS COVER ON PERFORMANCE OF EBSBS

60. There is another 'local' vegetation effect of importance in evaluating the performance of the EBSBs – the grass (turf, sod) cover on surfaces of the EBSBs. As shown in Figure 60 (Verheij (1997), depending on the 'density' of the grass (above and below ground) and on the substrate in which the roots are embedded, the grass cover can have very important effects on the ability of flowing water to breach the grass and substrate and then be able to directly attack the underlying soils (Hughes 2007; Briaud et al 2008; ILIT 2006; IPET 2007; Team Louisiana 2007; FHWA 2002; Bezuijen and den Adel 2006; Vriend and Barends 2006; Seijffert 2007; Verheij et al 1997; D'Eliso 2007; Hewlett et al 1980; Whitehead et al 1976; Meers and Sigffert 1994; Akker et al 2007).

61. In the case of the EBSBs at the time of Hurricane Katrina, observations of the grass cover immediately following Hurricane Katrina indicated it varied from very dense to very sparse (Team Louisiana 2007, ILIT 2006). Sparse grass cover and an erodible substrate (e.g. sand, silt) have little effect on the erosion from either waves or currents. Dense grass cover and an erosive resistant substrate can have very significant effects in preventing or delaying erosion from both wave and current attack. The analyses summarized herein have been based on a moderate natural cover of grass on a sandy – silty substrate with some cohesive binder – consistent with 'Neutral' MR-GO conditions (with as-constructed EBSBs built with dredge spoil). Analytical sensitivity studies are summarized herein to illustrate the potential effects of plausible variations in this important element in analyzing breaching characteristics of the EBSBs for the Hurricane Katrina hydrodynamic conditions.

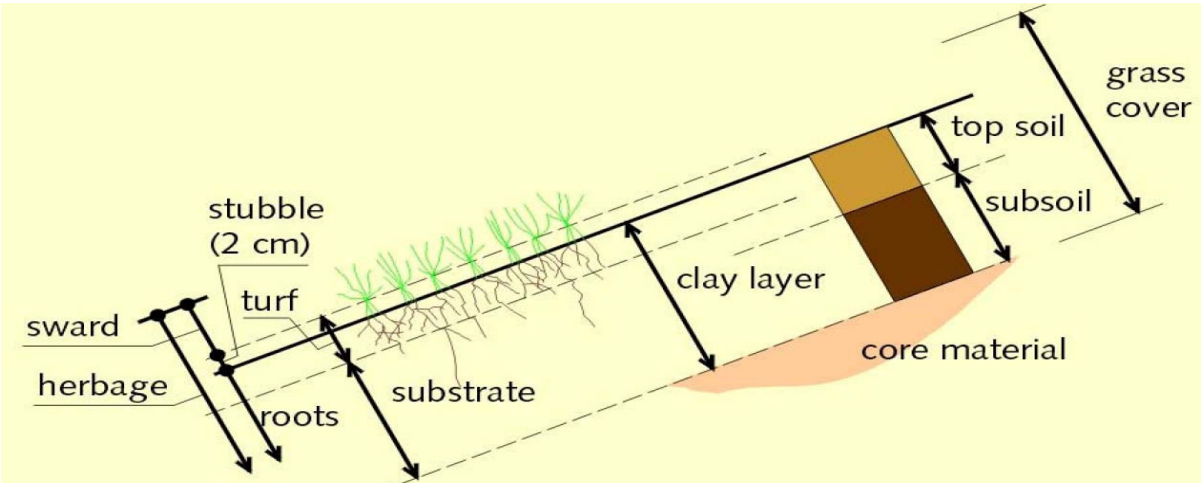


Figure 60: Structure and components of a grass cover (Verheij et al 1997).

62. In consideration of grass 'lift-off' times (time from initiation of wave or current attack to exposure of the grass substrate), two types of flow conditions were considered: 1) quasi-steady flows parallel to the grass cover (e.g. levee overtopping – backside conditions), and 2) highly unsteady turbulent flows normal to and parallel to the grass cover (e.g. levee front side attack by wave impacts – up-rush and down-rush dominated

flows) (Hughes 2007; IPET 2007; ILIT 2006; Ebersole 2008; Seijffert 1998; Briaud et al 2001, 2007, 2008; Verheij et al 1997; Hewlett et al 1987; Whitehead et al 1976; USDA 1966; Federal Highway Administration 2006; Akkerman et al 2007; Pullen et al 2007). Interactions between the fluid flow environment and the grass – substrate environment must be considered in developing assessments of the 'lift-off' times or fluid flow 'durability' of a grass cover (Figure 61).

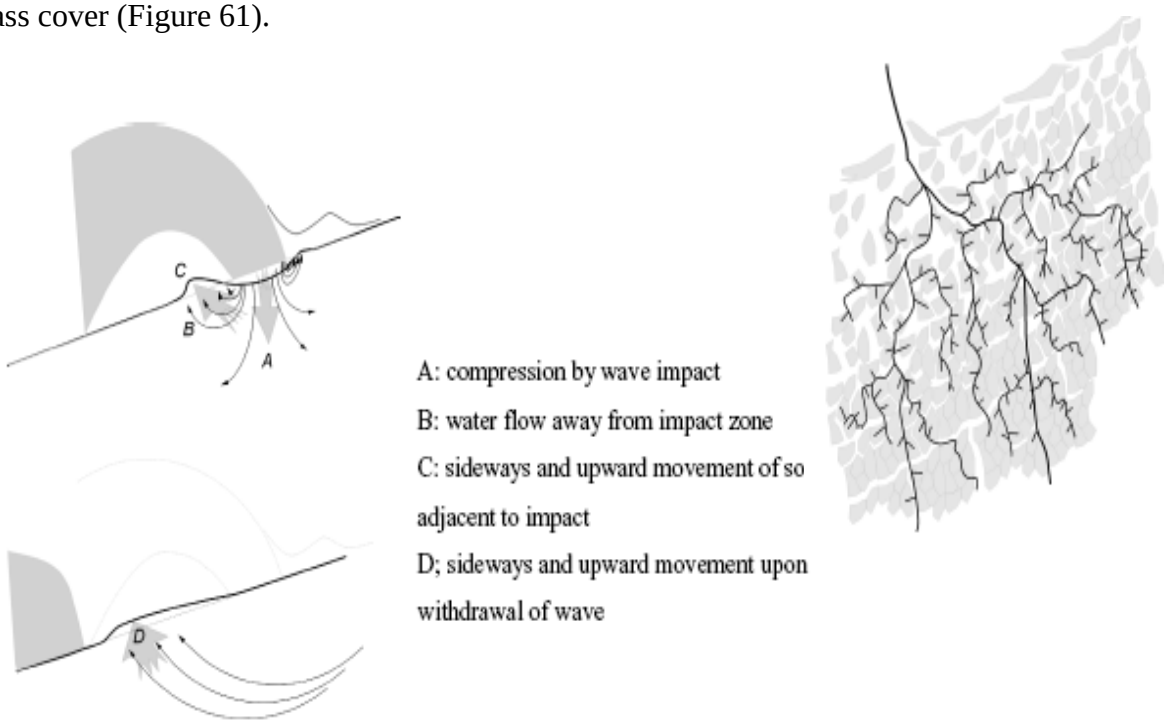


Figure 61: Water flow through the soil and movement under wave attack as affected by the structure of the grass and substrate (Verheij et al 1997).

63. Large scale tests were performed by Veheij et al (1995) to evaluate the erodibility of grass covered slopes subject to wave attack (Figures 62 and 63).

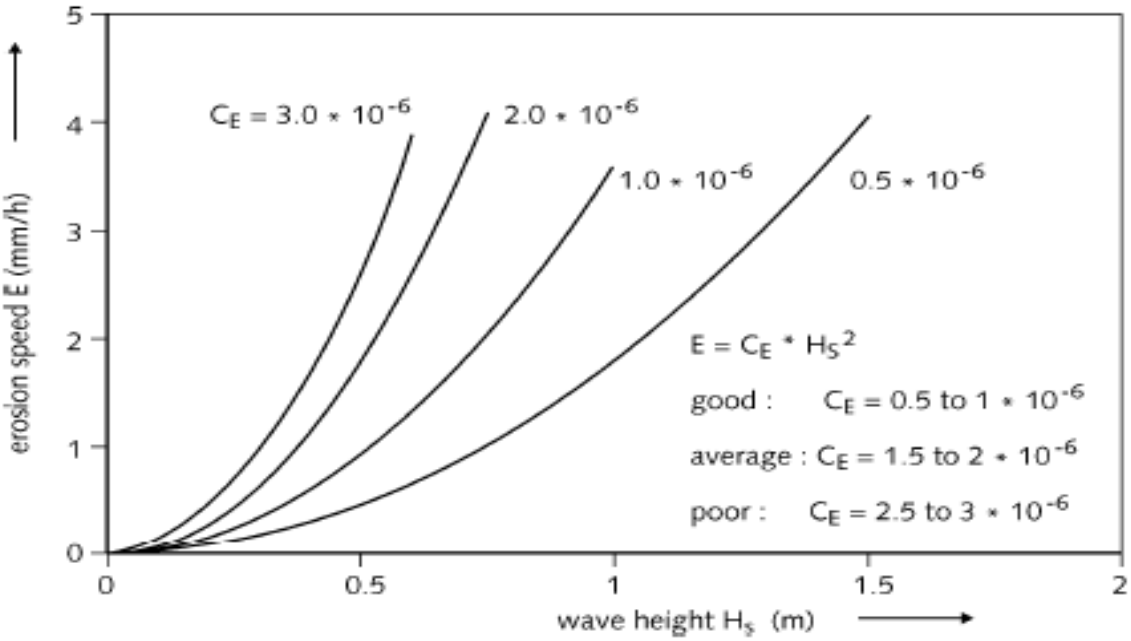


Figure 62: Large scale wave tank tests on grass (Delft Hydraulics Laboratory).



Figure 63: Erosion – scour area after 11 hours of wave attack (Delft Hydraulics Laboratory).

An important product of this extensive testing program was characterization of the erosion speed of a levee or dyke slope as a function of the 'density' of the grass cover – for clay substrate. The ranges of erosion speed as a function of good, average, and poor grass cover densities for a clayey substrate are shown in Figure 64.



Erosion-resistant grassland	Expected values for c_E
Good	0.5 to $1.5.10^{-6}$ $m^{-1}s^{-1}$
Moderate	1.5 to $2.5.10^{-6}$ $m^{-1}s^{-1}$
Poor	2.5 to $3.5.10^{-6}$ $m^{-1}s^{-1}$

Figure 64: Rate of erosion (E) as a function of the significant wave height (Hs) and the density of the grass cover – clay substrate (Veheij et al 1997).

This work resulted in development of an analytical expression for the rate of erosion (E_{grass}) as a function of the significant wave height (H_S) and a grass- clayey substrate erosion coefficient (C_E):

$$E_{grass} = C_E * H_S^2$$

The significant wave height in this expression is that in the deepwater portion of the laboratory wave tank that was used to perform the experiments. The significant wave height at the beginning of the wave impact zone was 30 percent of the deepwater significant wave height (Figure 65) (Seijffert and Verheij, 1998). In application of these results it is very important to reference and use the proper significant wave height – the significant wave height at the wave impact elevations of the EBSBs.

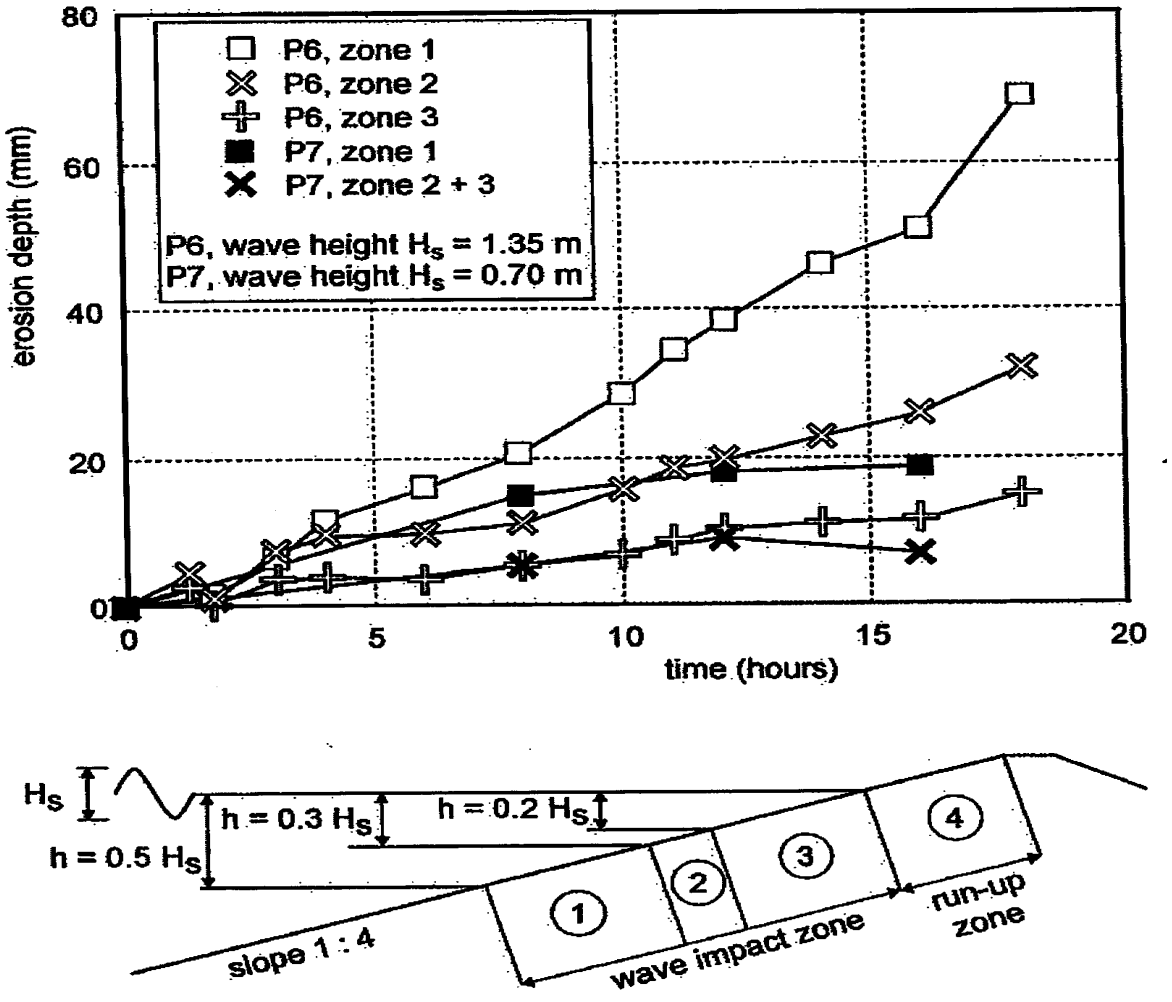


Figure 65: Delft Laboratory wave tank experimental set-up to measure grass erosion as a function of wave attack (Seijffert and Verheij 1998).

64. These results were used by Verheij et al to evaluate the lift-off times for various grass covers on a clayey substrate having a thickness (d) of 50 mm (about 2 inches) (Figure 66). They used a factor of safety (γ) of 2.0 to develop these results. For example, given a significant wave height of 1.0 meter at the wave impact zone, the poor and moderate grass covers were able to 'survive' – not lift off – for a period of 4 to 7 hours.

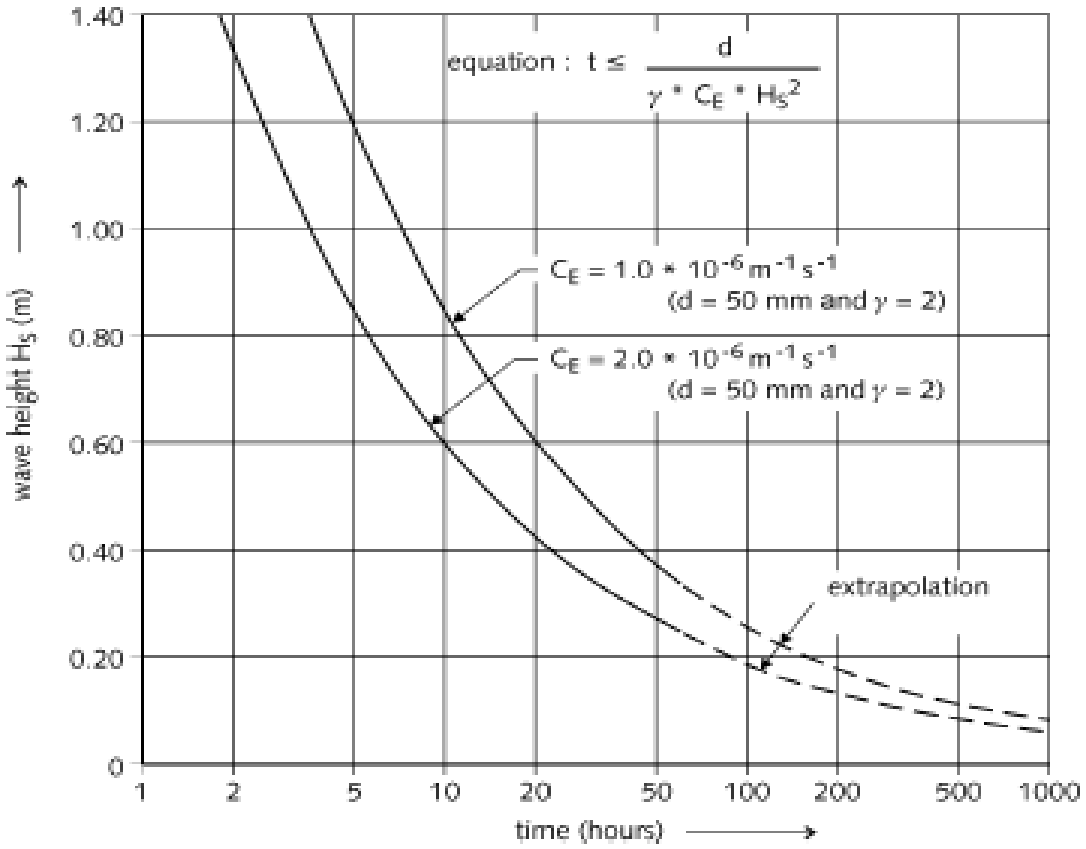


Figure 66: Grass 'lift-off' time (maximum permissible duration of wave attack) as a function of significant wave height (Verheij et al 1997) for moderate density grass cover on clayey substrate thickness of 50 mm (2 inches) and a 'safety coefficient' of 2.0.

65. Grass lift-off under quasi-steady flow overtopping type conditions on the protected sides of sea dykes and levees also has been studied extensively (Hewlett, et al 1987; Hughes 2006; Akkerman et al 2007). Of particular interest in this investigation were a series of field erosion tests performed at a Dutch Gronigen Sea Dyke (Akkerman et al 2007) (Figure 67). These tests involved both intact and 'defective' (gaps) grass slopes. The grass

slopes had different degrees of maturity and coverage but only included grass with a cohesive – clay substrate.



Figure 67: Wave overtopping erosion tests at Gronigen sea dyke (Akkerman et al 2007).

The Gronigen Sea Dyke tests on grass cover subjected to quasi-steady flows developed results in general agreement with laboratory and field test data analyzed by Hewlett et al (1987). The results of this work are summarized in Figure 68.

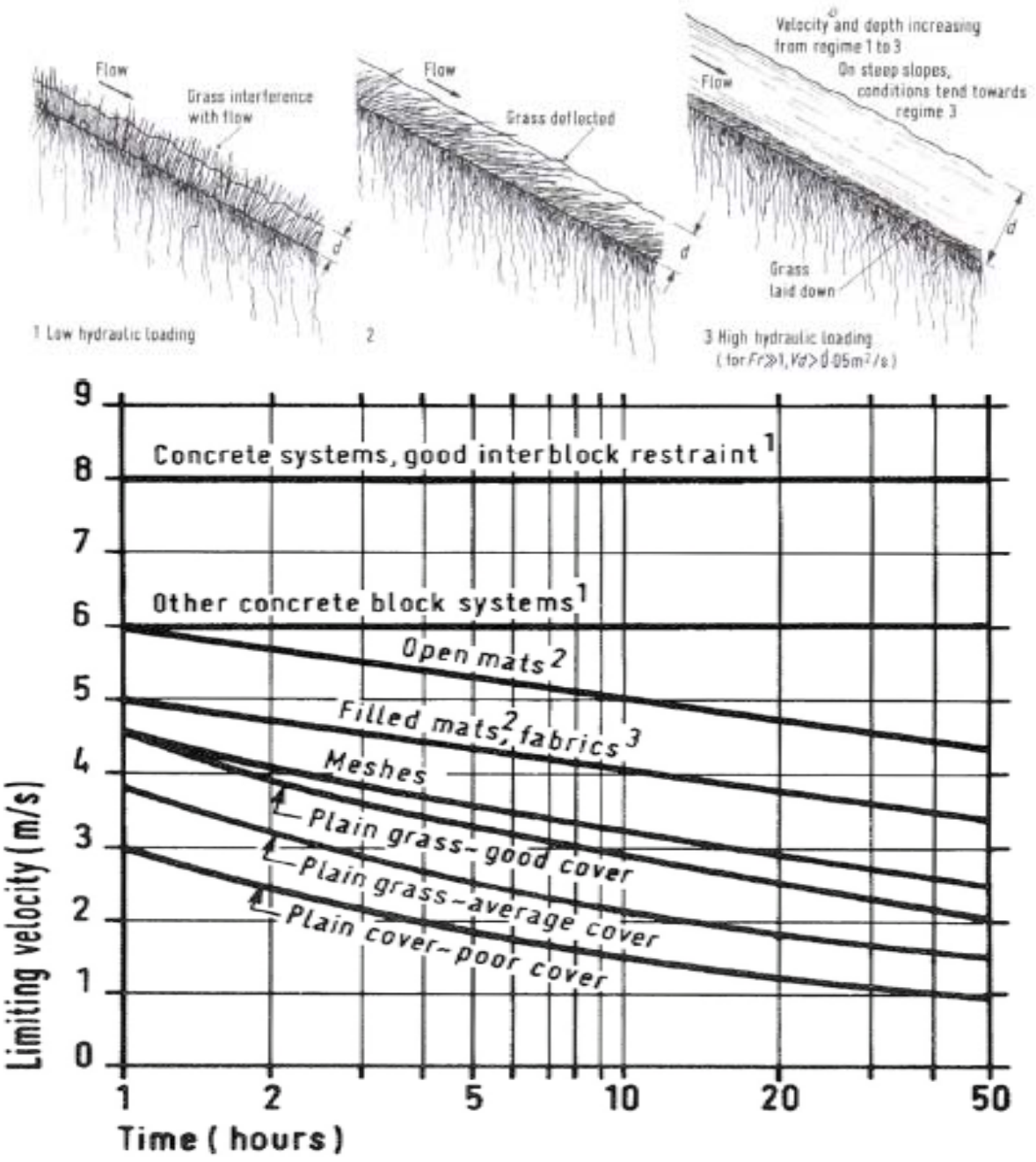


Figure 68: Summary of test data analyses by Hewlett et al (1987) for different types of grass cover including reinforced grass, mats and concrete block systems.

66. The references cited previously in this section were used to develop guidelines to evaluate expected grass 'lift-off' times for quasi-steady flow and highly unsteady flows for the EBSB substrate conditions (clayey sands and silts, sands and silts characteristic of dredge spoil). These guidelines are summarized in Figures 69 and 70. The guidelines for the highly unsteady flows – wave impacts and run-up / run-down were derived using the guidelines provided by Verheij et al (1997) and corroborated with information provided by Hewlett et al 1987). The reference significant wave heights are those at the wave impact zone.

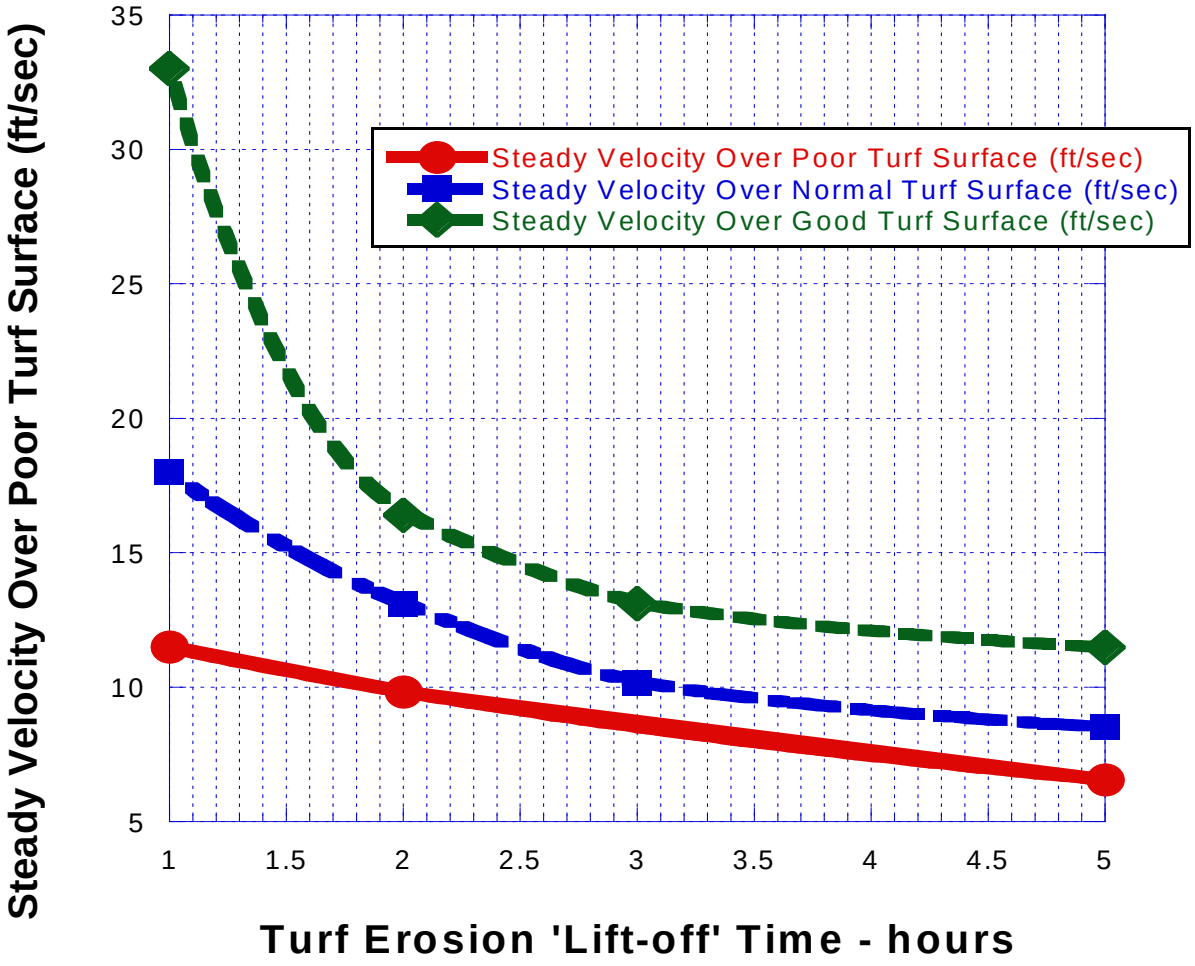


Figure 69: Quasi-steady flow grass lift-off times for EBSB erosion analyses.

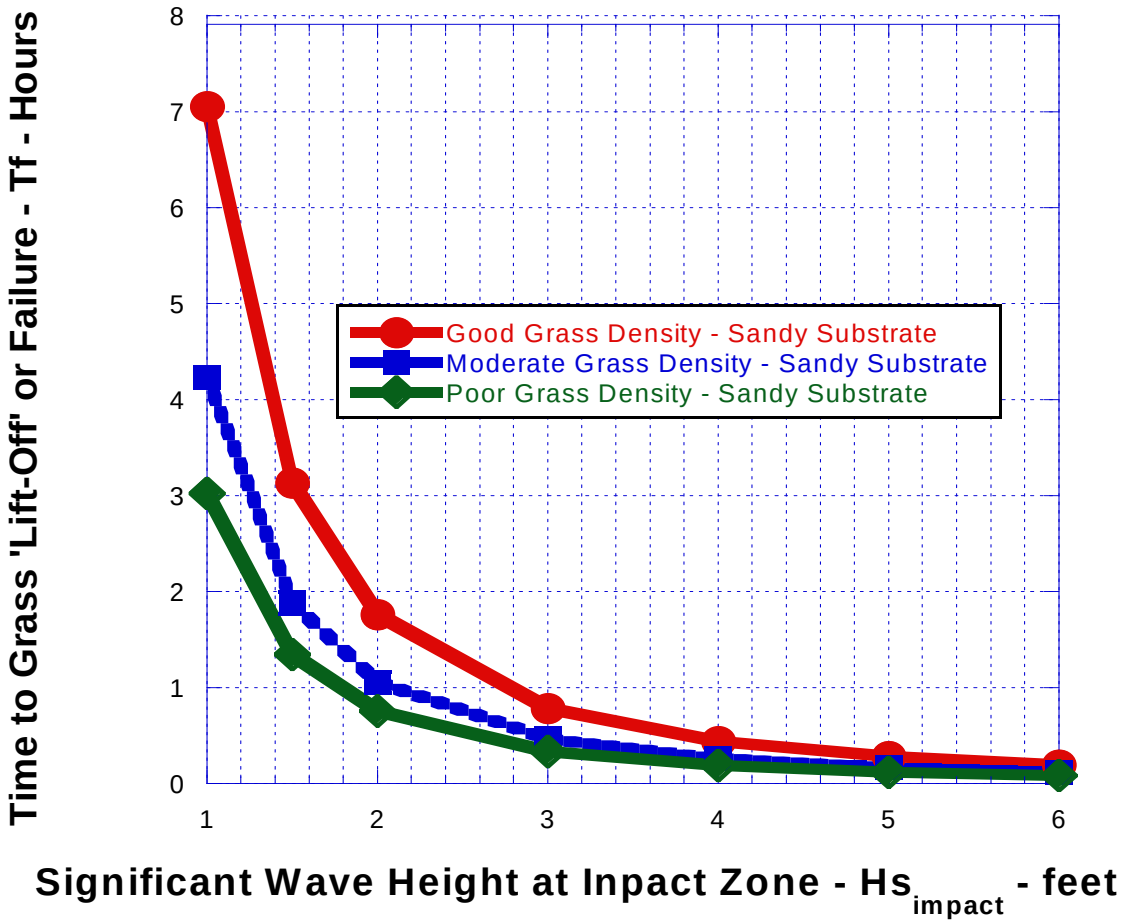


Figure 70: Highly unsteady flow grass lift-off times for EBSB erosion analyses.

67. In the case of either steady or unsteady flows in which the intensity of the steady or unsteady flow change as a function of time (e.g. waves with rising water level due to hurricane surge or overtopping back-side flows that change with the surge height above the EBSB crown), it is necessary to develop a an analytical model to characterize 'damage accumulation'. In this study, a linear "Miner's fatigue damage accumulation" model has been used (Lematire and Desmorat 2005). In this damage accumulation model the damage at which 'failure' (grass lift-off) develops is when the total damage (D) reaches unity (D = 1.0). The total damage is computed as the summation of the time the grass is exposed to each of

'n' sea states (t_{gi} - time grass exposed to given 'ith' sea state) divided by the time to grass lift-off in that sea state (Tf_{gi}):

$$D = \sum_{i=1}^{n} \frac{t_{gi}}{Tf_{gi}} \leq 1.0 \text{ (D = 1 = 'failure')}$$

Miner's fatigue damage accumulation rule has proven to be robust in application to determining the damaging effects of repeated cyclic straining of a wide variety of man-made (e.g. steel, concrete, plastics) and natural materials (e.g timber, soil, rock) (Lematire and Desmorat 2005, Suresh 1998).

PERFORMANCE OF THE REACH 2 MAN-MADE FLOOD PROTECTION STRUCTURES FOR MR-GO HURRICANE KATRINA CONDITIONS

68. Figure 71 summarizes the various effects that hurricane waves transiting from the open waters of Lake Borgne 'feel' (or are affected by) as they cross the Brackish Marsh, the MR-GO channel, the west channel bank, and finally the berm in front of the EBSBs. Four conditions are identified for the waves arriving at the toes of the EBSBs: 1) Condition 1 - no effects on Lake Borgne waves, 2) Condition 2 - decay due to Brackish Marsh and breaking and shoaling on channel bank and berm, 3) Condition 3 - Condition 2 and no MR-GO channel effects, and 4) Condition 4 - Condition 3 with bank vegetation effects. Condition 4 was used in the work summarized here to evaluate the performance of the EBSBs – the 'Neutral' MR-GO Hurricane Katrina Conditions (Bea and Storesund 2008a). It is very important to characterize the geographic, bathymetric, vegetation, and wave dissipation characteristics associated with each of the 'zones' identified in Figure 71.

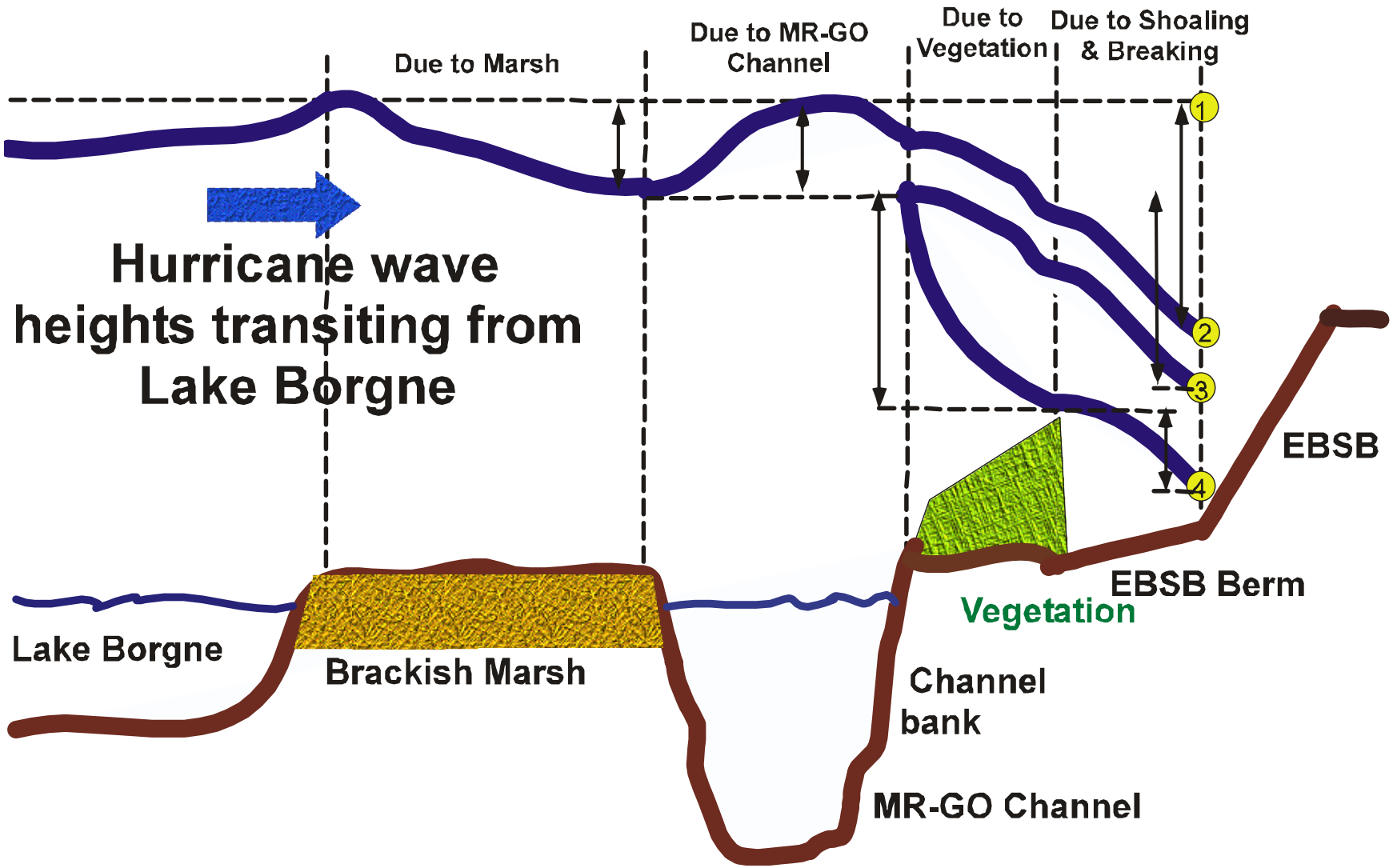


Figure 71: Transit effects as hurricane waves propagate from Lake Borgne to the toes of the EBSBs (based on simulation results provided by Gautier, et al 2008).

69. It was noted earlier in this Declaration, that the 'Neutral' MR-GO Hurricane Katrina Conditions hydrodynamic simulations did not incorporate local effects associated with the natural wave vegetation buffer zone between the authorized channel of the MR-GO and the toes of the EBSBs and navigation structures. The previous section has summarized results from validated analytical models that indicate that a well developed vegetation buffer zone (100 – 200 feet wide, 10 to 15 feet tall, moderate density) – consistent with 'Neutral' MR-GO Conditions - would reduce the incident waves to approximately 50 percent of the incident shallow water wave heights (water depths consistent the surge that accompanied Hurricane Katrina). The incident wave significant heights for the various locations along the EBSB alignment for the peak surge conditions ranged from 4.3 feet to 4.5 feet (upper bound 4.7 to 5.1 feet, lower bound 2.3 to 3.4 feet) – depending on the water depth and local bathymetry associated with the position of the base or toe of the man-made hurricane flood protection and navigation structures (Gautier et al 2008). The peak wave periods associated with these conditions ranged from about 3 to 4 seconds. These waves approached the Reach 2 alignment essentially 'broadside' with their crests about parallel to the Reach 2 alignment. Given that these wave heights would be further reduced by the local vegetation buffer zone, the maximum significant wave heights would be in the range of 2 to 3 feet (best estimate). The surge elevations would be unaffected. The currents at the toes of the EBSBs would be negligible.

LS-DYNA Computer Program

70. The computer program LS-DYNA (Livermore Software Technology Corporation. 2007) (Figure 72) was used to analyze the hydrodynamic conditions – levee profile – soil erosion and scour characteristics at the EBSB erosion – scour – breaching study site (Bea and Storesund

2008a). LS-DYNA is a general purpose finite element computer program for analyzing large static and dynamic deformation responses of structures and of structures coupled to fluids. The main solution methodology is based on explicit time integration. LS-DYNA's capabilities include non-linear dynamics, fluid analysis, and smooth particle hydrodynamics. The contact algorithms include flexible body to rigid body contact, eroding contact, and CAD surfaces. LS-DYNA contains approximately one-hundred constitutive models and ten equations of state to cover a wide range of material behavior. In addition to the standard constitutive models, user-defined models also are available.

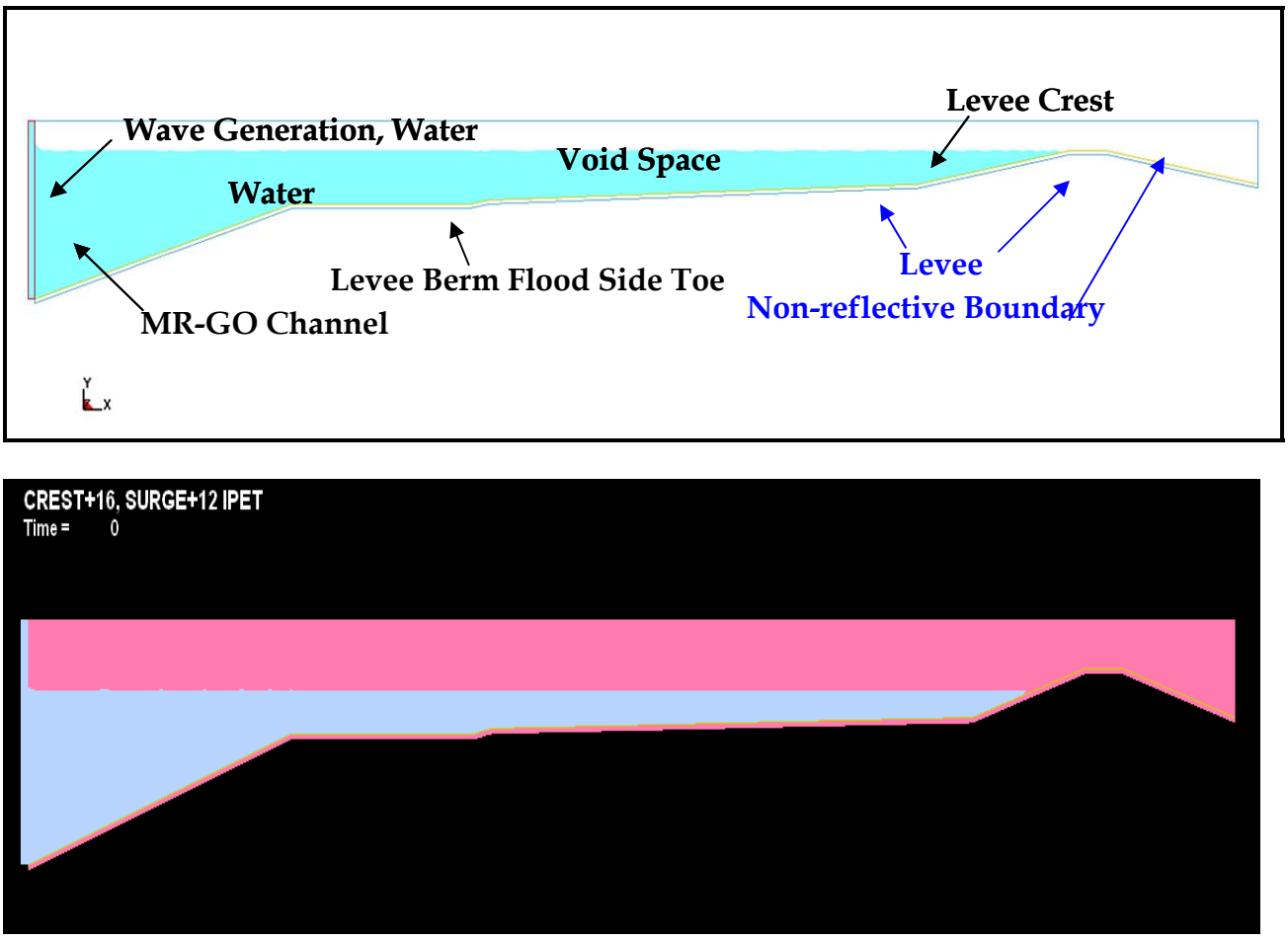


Figure 72: Overview of the LS-DYNA finite element analysis model used for the numerical simulations (Bea and Storesund 2008).

71. Interaction shear velocities between the storm surge and the earthen levees were evaluated for a selection of storm surge elevations. Simulations were performed using the design geometric levee configuration and the actual levee configuration (as determined based on the available Aerial LiDAR data from 2000). For each storm surge elevation a numerical simulation was conducted for 150 seconds using the Hurricane Katrina hydrodynamic characteristics (surge, currents, waves) defined by Wit et al (2008) and Gautier et al (2008) summarized earlier in this Declaration. Additional Hurricane Katrina hydrodynamic characteristics were simulated in parametric analyses as defined by IPET (2007), Team Louisiana (2007), and LSU – Technical University at Delft (van Heerden 2007). This simulation time was selected in order to optimize computing time, while achieving 'steady state' wave impact conditions (interaction shear stresses and velocities) in the mode elevation Wave characteristics were validated with results from empirical models (e.g. USACE CEM 2007). Interaction shear velocities were determined by tracking the velocity profiles at selected nodes (Figure 73).

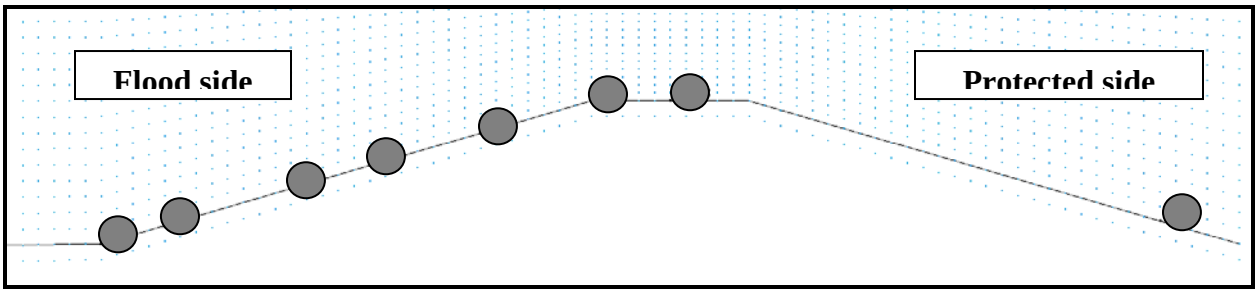


Figure 73: An overview of 'monitoring' nodes used to calculate shear velocities for the actual EBSB configuration.

Hydrodynamic Velocities and Cumulative Erosion Results

72. The fluid – soil EBSB interaction shear velocities calculated by the LS-DYNA numerical simulation were compared against the erosion/scour soil limits (capacity) as estimated based on the Erodibility Function Apparatus and EBSB soil composition. Soil samples collected

from the ESBSs south of Bayou Bienvenue and north of Bayou Dupre were found to be generally 'very high' to 'high' erodibility materials (hydraulic fill derived from the MR-GO navigation channel).

73. The analyses were oriented to determining the likelihood of EBSB breaching primarily due to wave impact prior to EBSB overtopping. As a result, the analysis time frame was limited to the time before the storm surge overtopped the EBSB. Once overtopping occurred, the highly erodible materials would have been extremely susceptible to erosion due to the high velocities and low material erosion resistance.

74. A discussion of the simulated shear velocities for the evaluated Scenario 1 (Actual MR-GO) and Scenario 2C ('Neutral' MR-GO) cases is presented in the following sections at each of the simulation time steps (0430 CDT, 0530 CDT, 0630 CDT, 0730 CDT, and 0830 CDT).

Analysis for Scenario 1 (0430 CDT)

Scenario 1 at 0430 CDT (Figure 74) consisted of an EBSB crest elevation of +16 feet (actual geometry) and hydrodynamic conditions consisting of a storm surge elevation of +10.1 feet, a wave period of 4.0 seconds and a significant wave height of 2.0 feet. There was no overtopping.

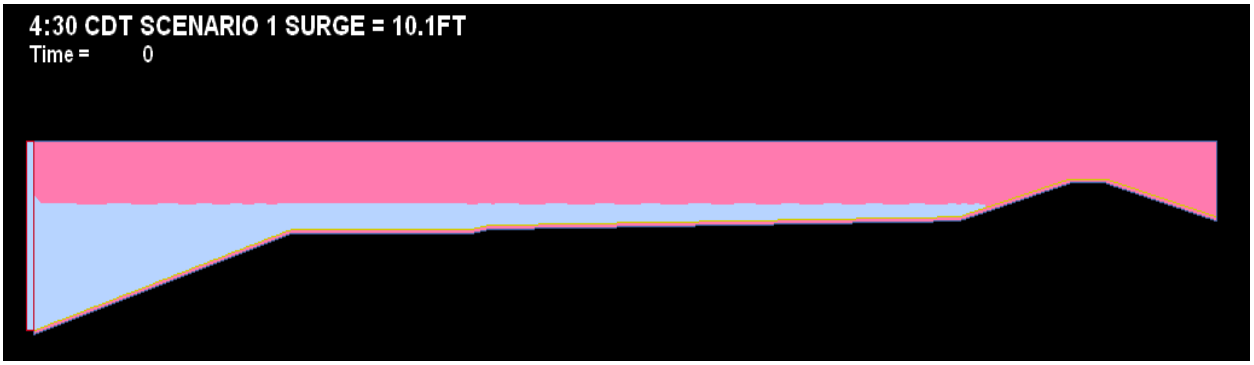


Figure 74: Initial conditions for Scenario 1 at 0430 CDT.

At Elevation +10.1 feet (all elevations are referenced to NAVD 88), the storm surge reached the flood side toe of the EBSB. A plot of the shear velocities is presented in Figure 75. The peak shear velocity at the storm surge elevation was determined to be 10 feet per second.

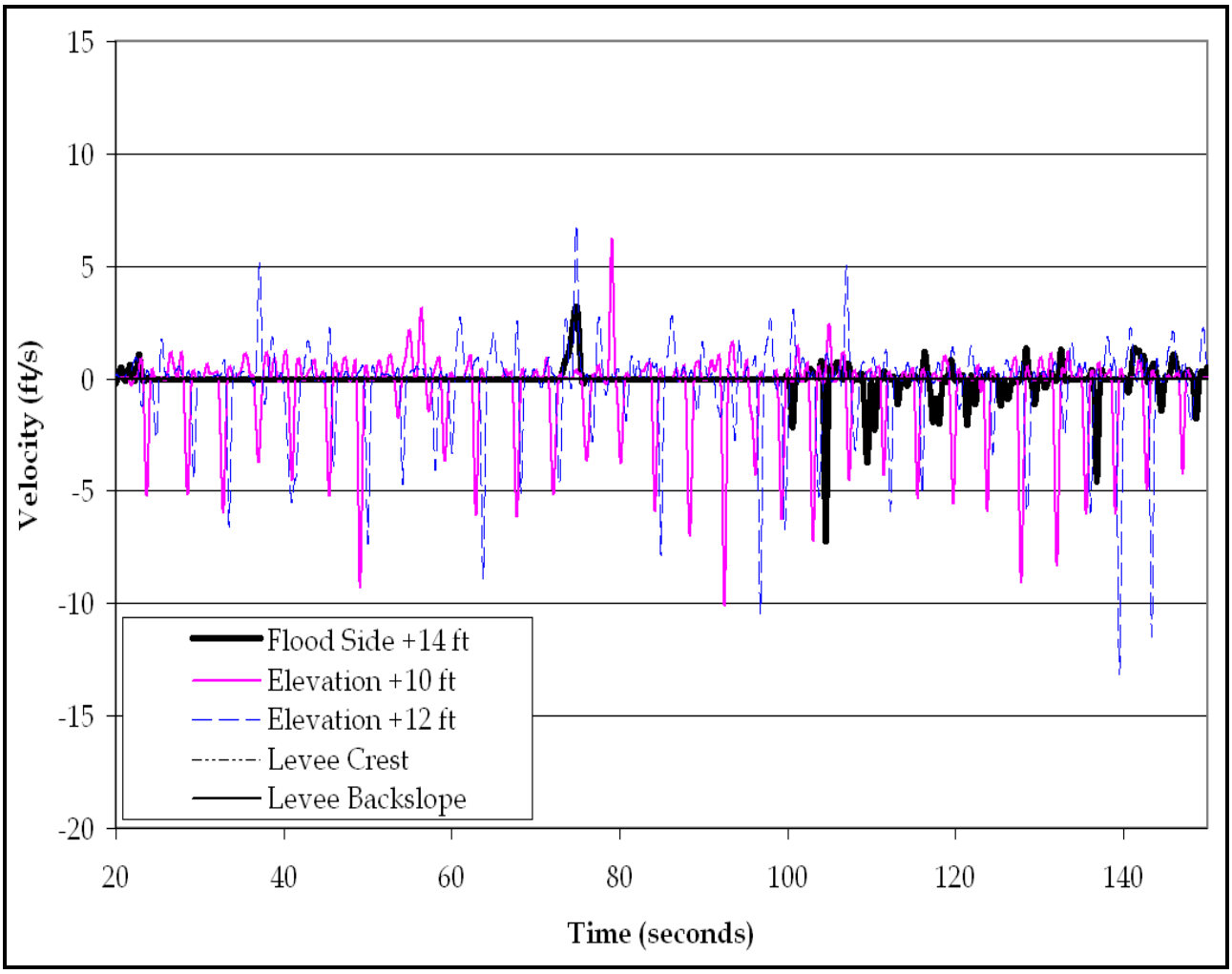


Figure 75**:** A plot of the velocity time history on the flood side face of the EBSB with the storm surge at +10.1 feet.

Figure 76 provides a graphical representation of water movement over time. With the hurricane storm surge at Elevation +10.1 feet, the actual geometric configuration of the EBSB induces a tendency for the waves to exhibit a primarily 'run-up' and 'rundown' behavior.

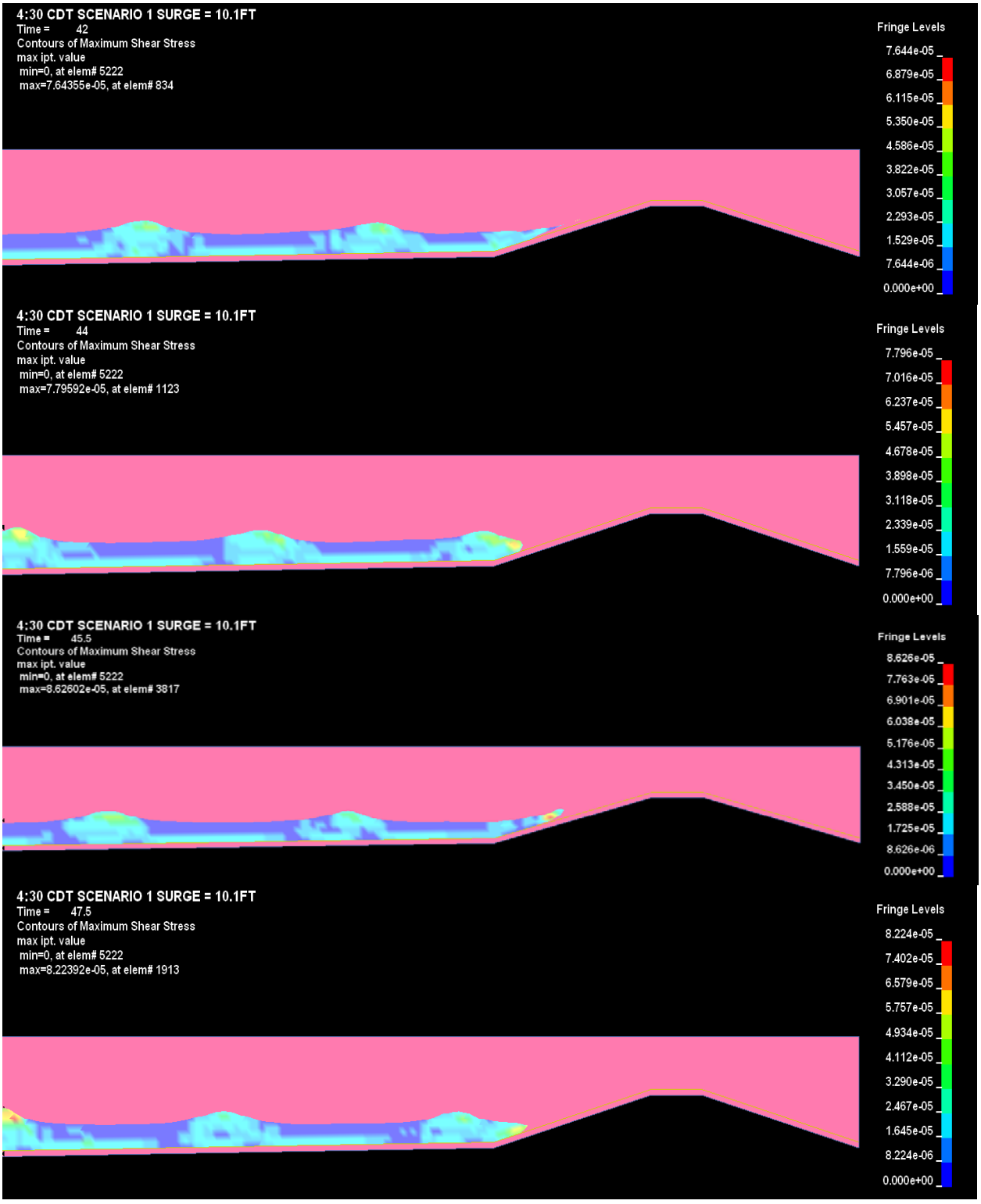


Figure 76**:** With the hurricane storm surge at Elevation +10.1 feet, the geometric configuration of the EBSB induces a tendency for the waves to exhibit a primarily 'run-up' and 'rundown' behavior.

Analysis for Scenario 1 (0530 CDT)

75. Scenario 1 at 0530 CDT (Figure 77) consisted of an EBSB crest elevation of +16 feet and hydrodynamic conditions consisting of a storm surge elevation of +12.0 feet, a wave period of 4.5 seconds, and a significant wave height of 2.7 feet. There was no overtopping.

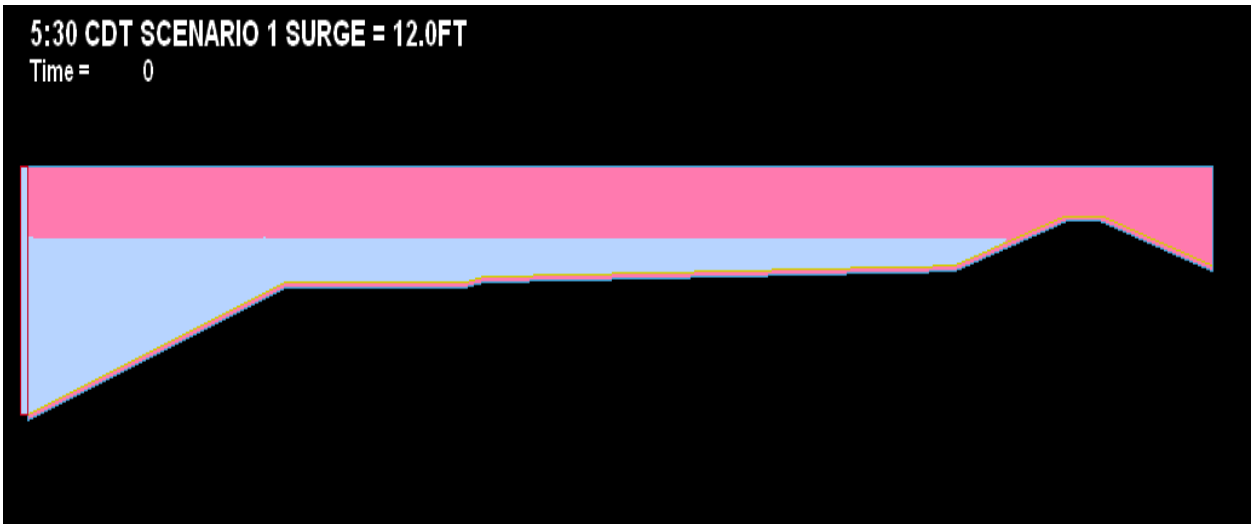


Figure 77: Initial conditions for Scenario 1 at 0530 CDT.

76. At Elevation +12.0 feet, the storm surge had reached the main EBSB flood side slope. This slope was vegetated at the time of the hurricane, so some erosion resistance was provided by the grassy cover. A plot of shear velocities is presented in Figure 78. Based on the velocity profile, the majority of the velocities significantly exceed the critical erosion values, so this implies that erosion and scour would occur as a result of wave impact at this elevation. The peak velocity was determined to be 12 feet per second.

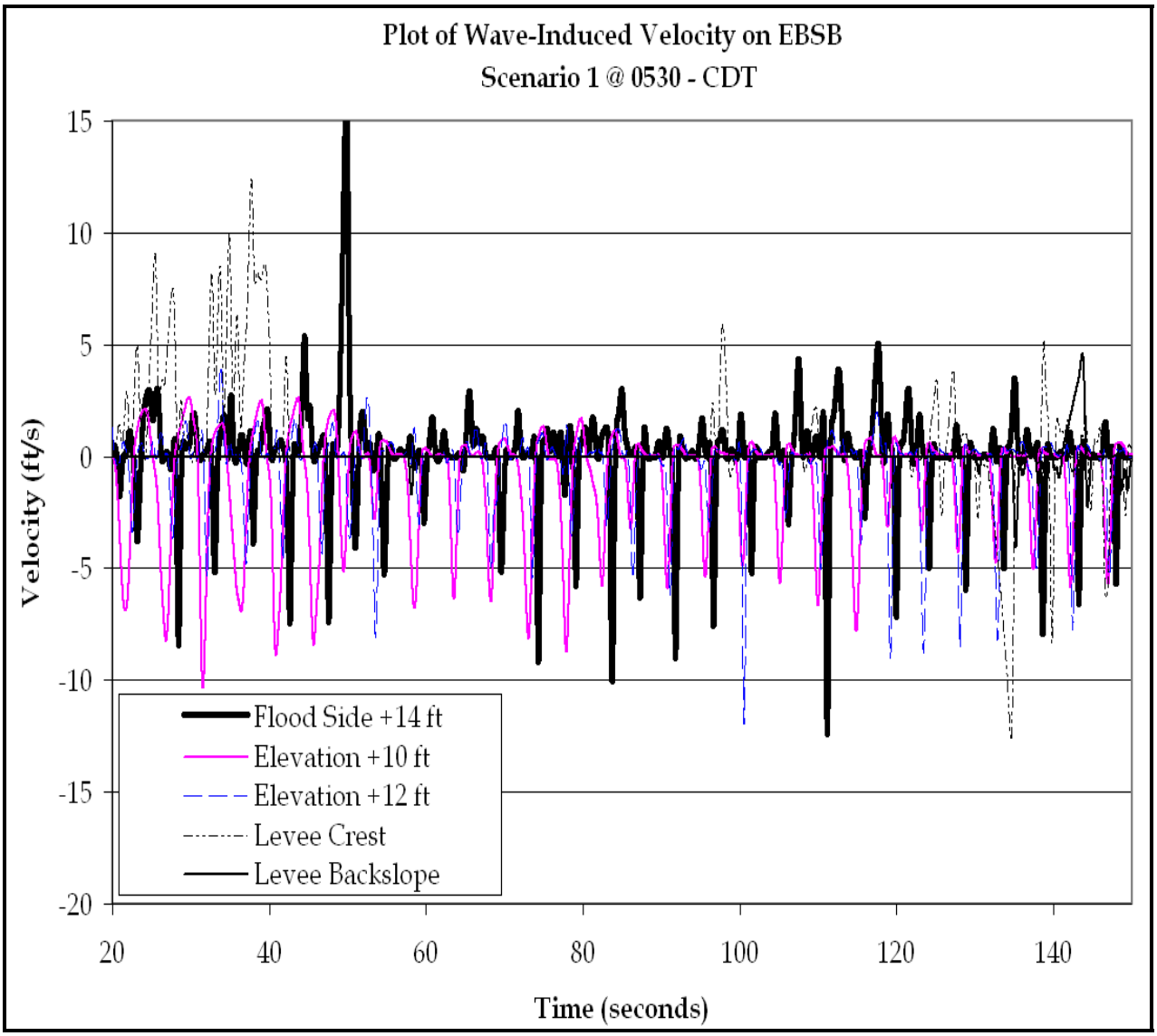


Figure 78: Time history plot of shear velocities for the storm surge at Elevation +12 feet.

77. Figure 79 provides a graphical representation of water movement over time. With the hurricane storm surge at Elevation +12.0 feet, the geometric configuration of the EBSB induces a tendency for the waves to exhibit a primarily 'run-up' and 'rundown' behavior, with moderate to major wave impacts as well as wave-induced overtopping.

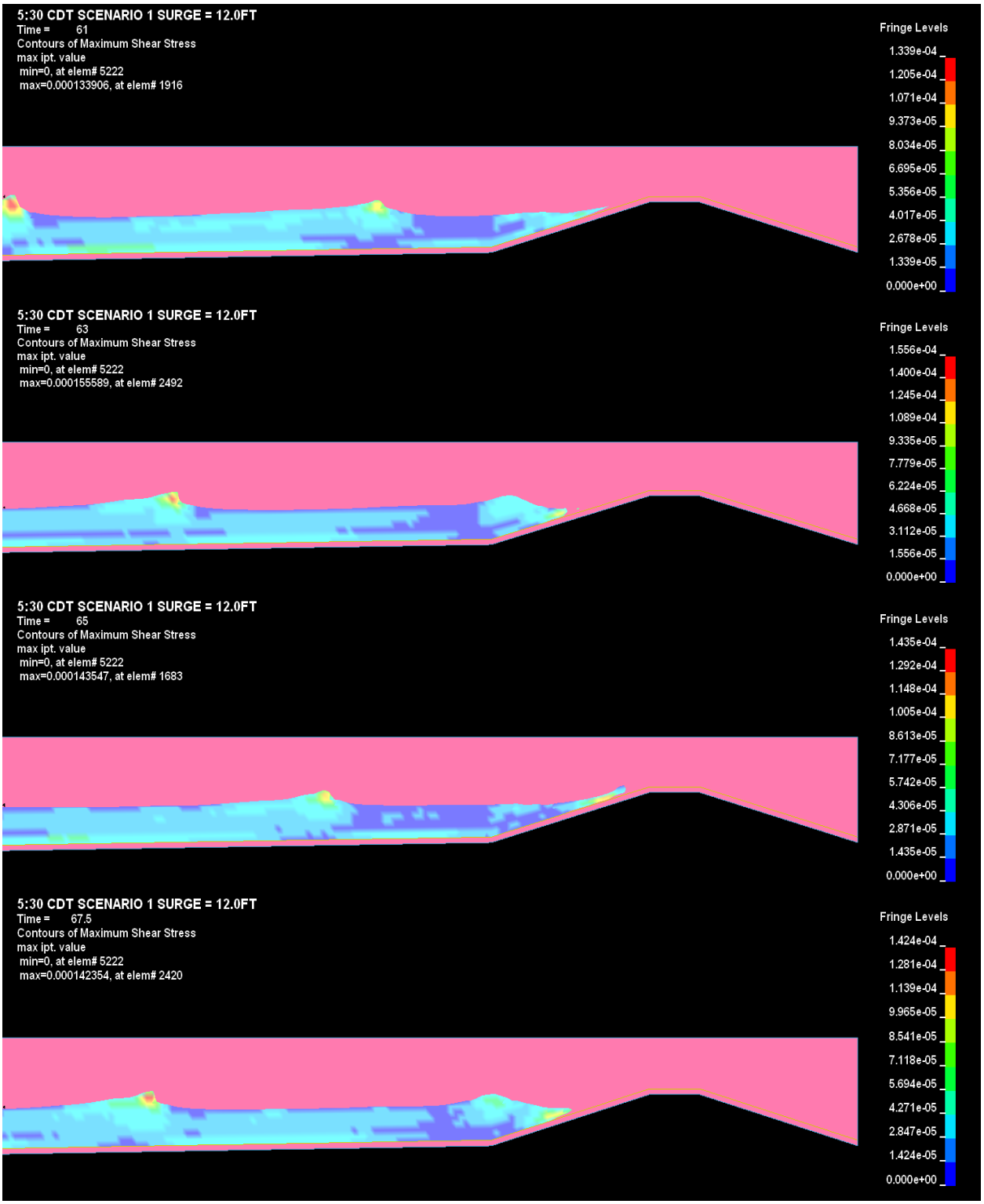


Figure 79**:** With the hurricane storm surge at Elevation +12.0 feet, the actual geometric configuration of the EBSB induces a tendency for the waves to exhibit a primarily 'run-up' and 'rundown' behavior with moderate to major wave impacts.

Analysis for Scenario 1 (0630 CDT)

78. Scenario 1 at 0630 CDT (Figure 80) consisted of an EBSB crest elevation of +16 feet (actual geometry) and hydrodynamic conditions consisting of a storm surge elevation of +14.2 feet, a wave period of 4.9 seconds, and a significant wave height of 4.1 feet. There was limited overtopping.

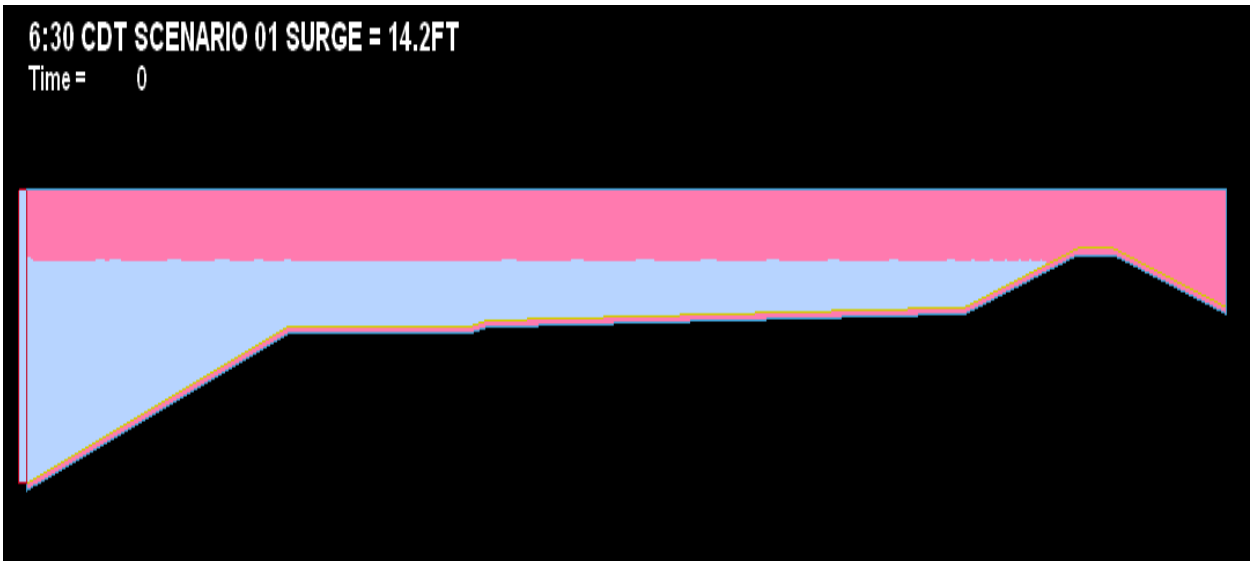


Figure 80**:** Initial conditions for Scenario 1 at 0630 CDT.

79. At Elevation +14.2 feet, the storm surge was nearing the crest of the EBSB and limited (wave-induced) overtopping was occurring. A plot of shear velocities is presented in Figure 81. The peak velocity was determined to be 15 feet per second.

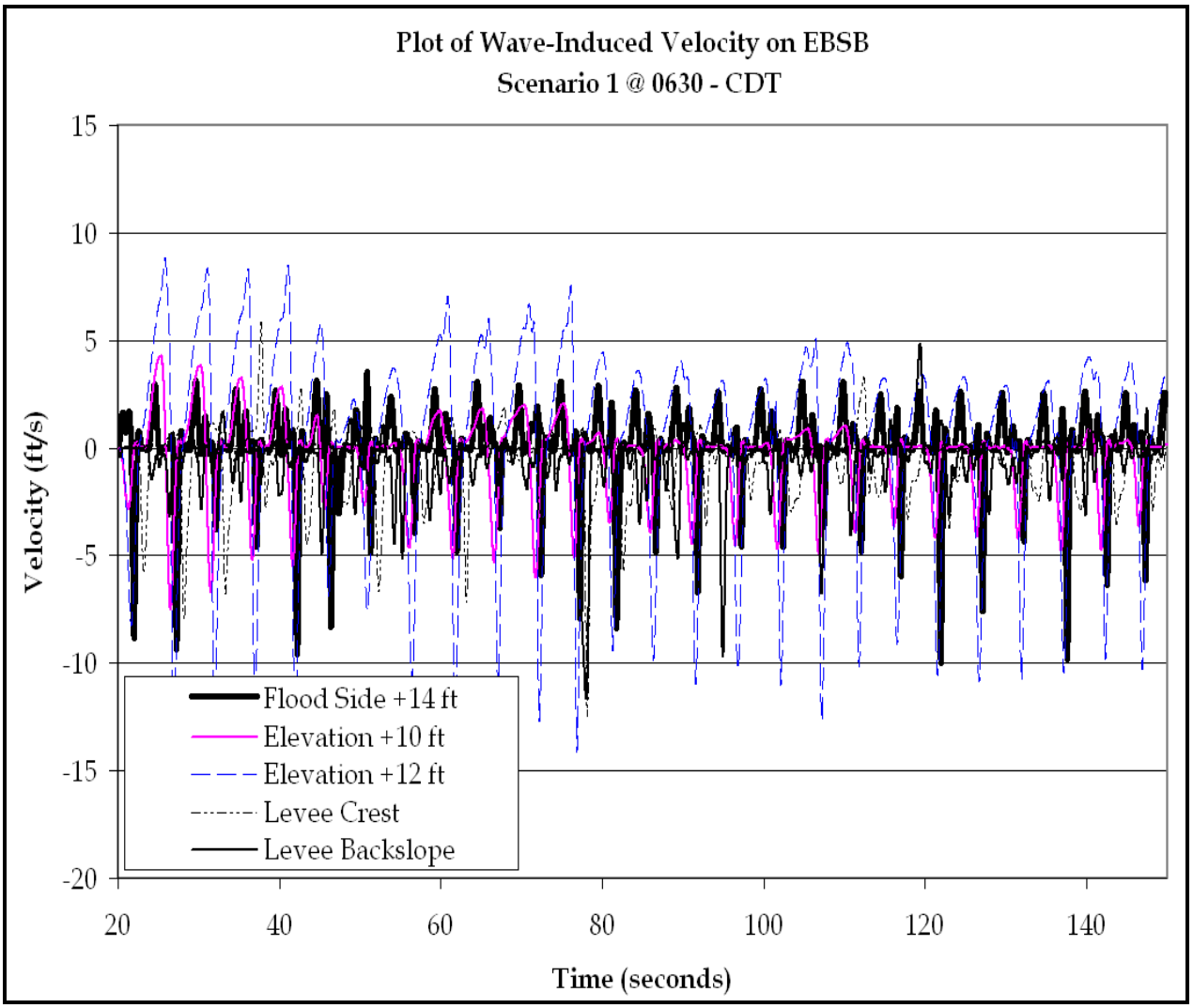


Figure 81: Time history plot of shear velocities for the storm surge at Elevation +14.2 ft.

80. Figure 82 provides a graphical representation of water movement over time. With the hurricane storm surge at Elevation +14.2 feet, the geometric configuration of the EBSB appears to not promote direct wave impacts, but rather induces a tendency for the waves to 'spill over' the top of the EBSB, contributing significantly to run-up and run-down erosion with overtopping.

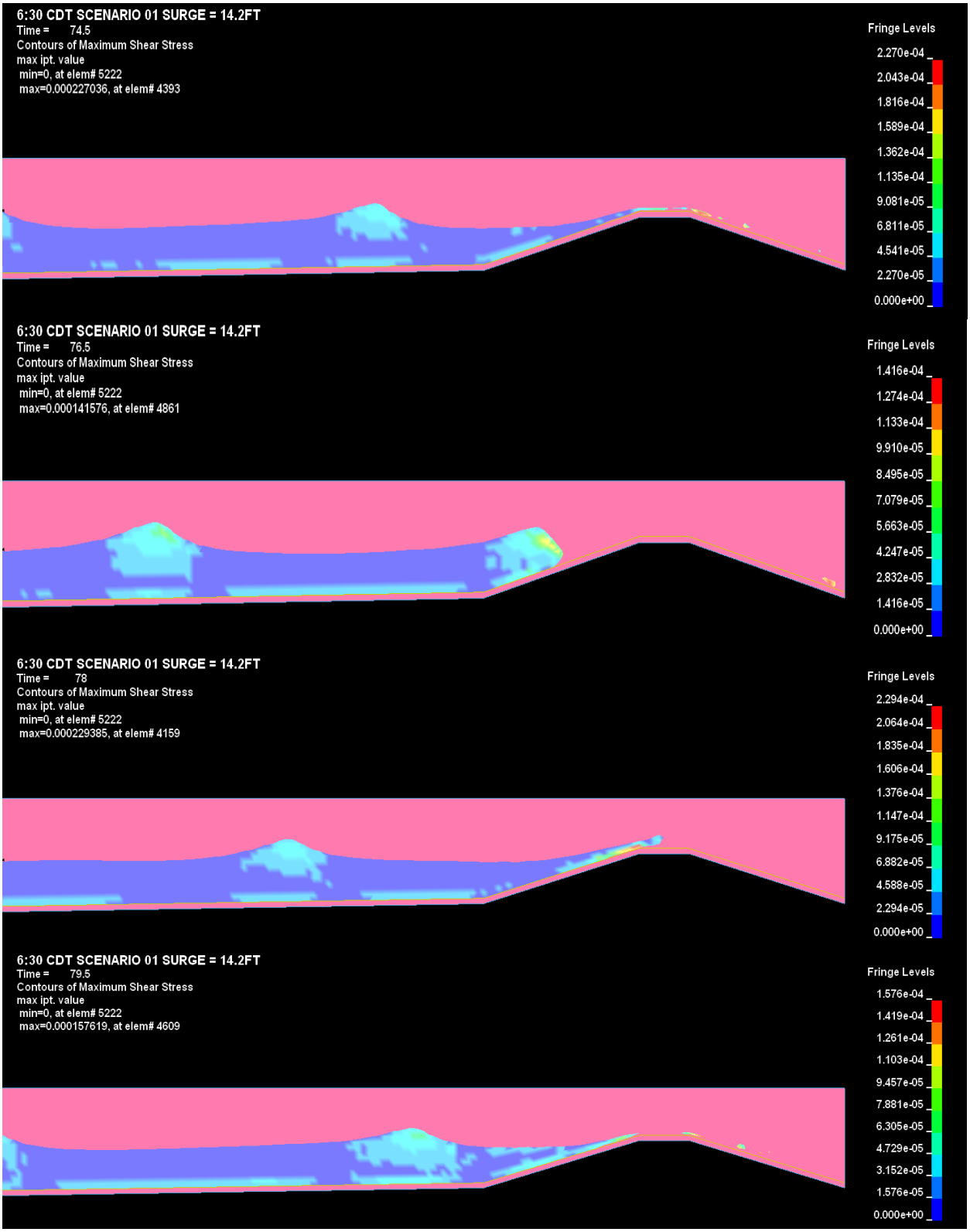


Figure 82**:** With the hurricane storm surge at Elevation +14.2 feet, waves induce predominantly run-up and run-down erosion with some overtopping.

Analysis for Scenario 1 (0730 CDT)

81. Scenario 1 at 0730 CDT (Figure 83) consisted of a crest elevation of +16 feet (actual geometry) and hydrodynamic conditions consisting of a storm surge elevation of +16.6 feet, a wave period of 5.4 seconds, and a significant wave height of 5.5 feet. There was active overtopping and backside flow.

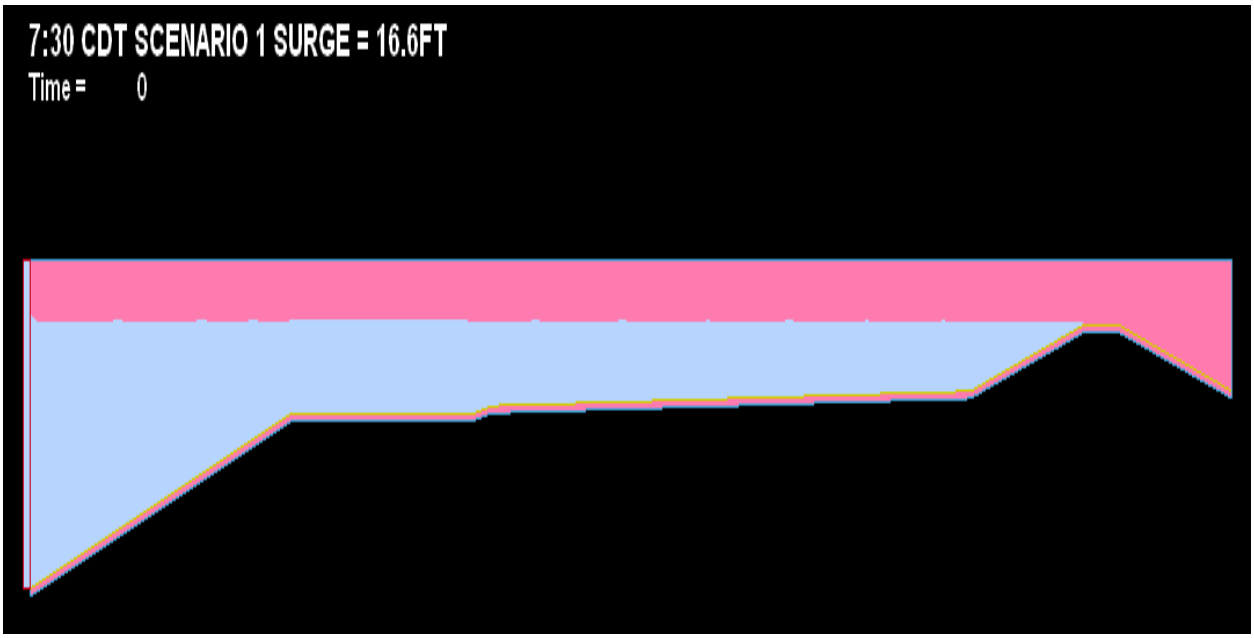


Figure 83: Initial conditions for Scenario 1 at 0730 CDT.

82. At Elevation +16.6 feet, the storm surge had reached the crest of the EBSB. A plot of the shear velocities is presented in Figure 84. The peak shear velocity was determined to be 14 feet per second. At this storm surge elevation and wave action results in active overtopping of the EBSB.

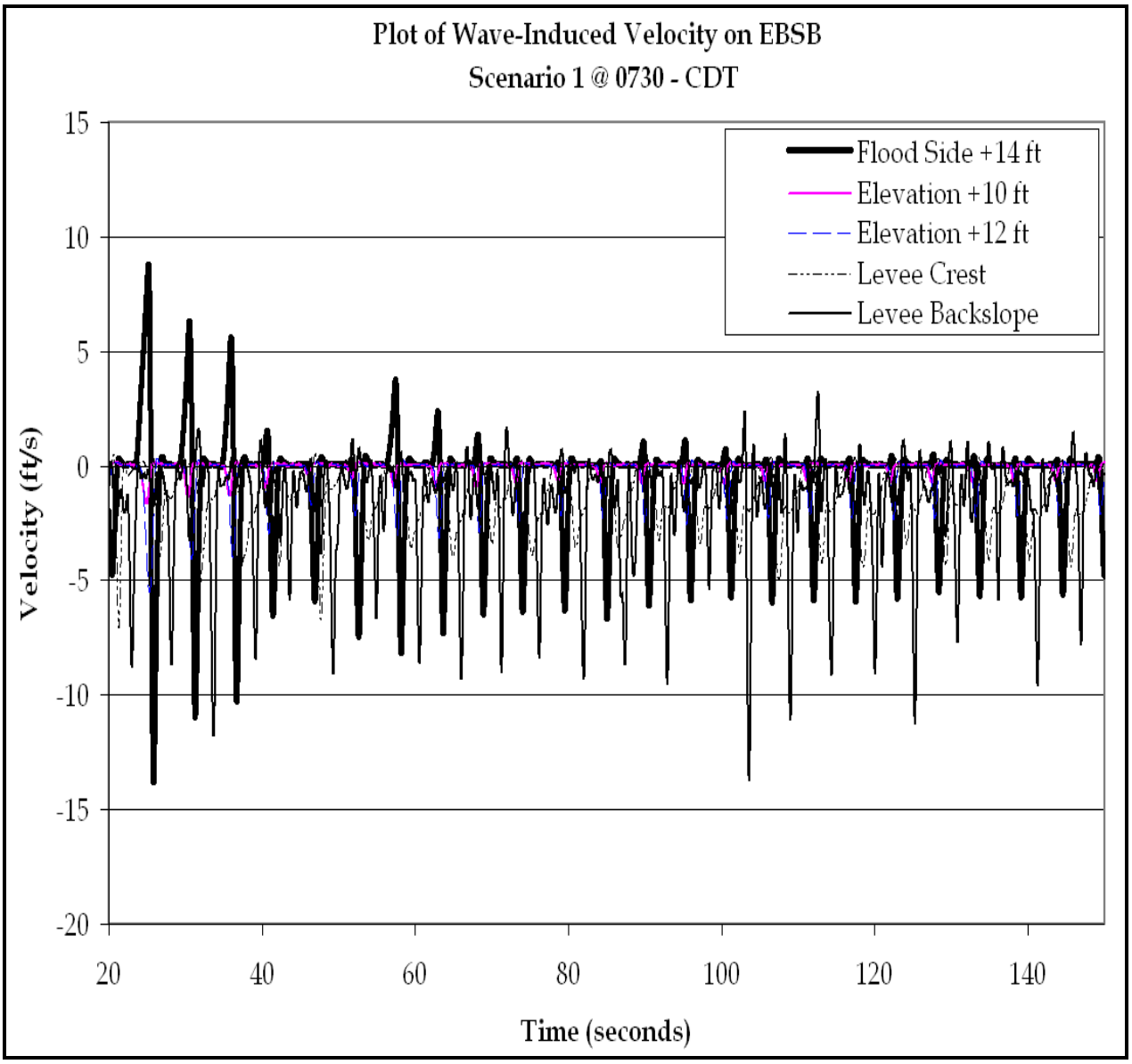


Figure 84: Time history plot of shear velocities for the storm surge at Elevation +16.6 feet.

83. Figure 85 provides a graphical representation of water movement over time. With the hurricane storm surge at Elevation +16.6 feet, the geometric configuration of the EBSB induces a tendency for the waves to exhibit a primarily overtopping behavior, with moderate wave impacts.

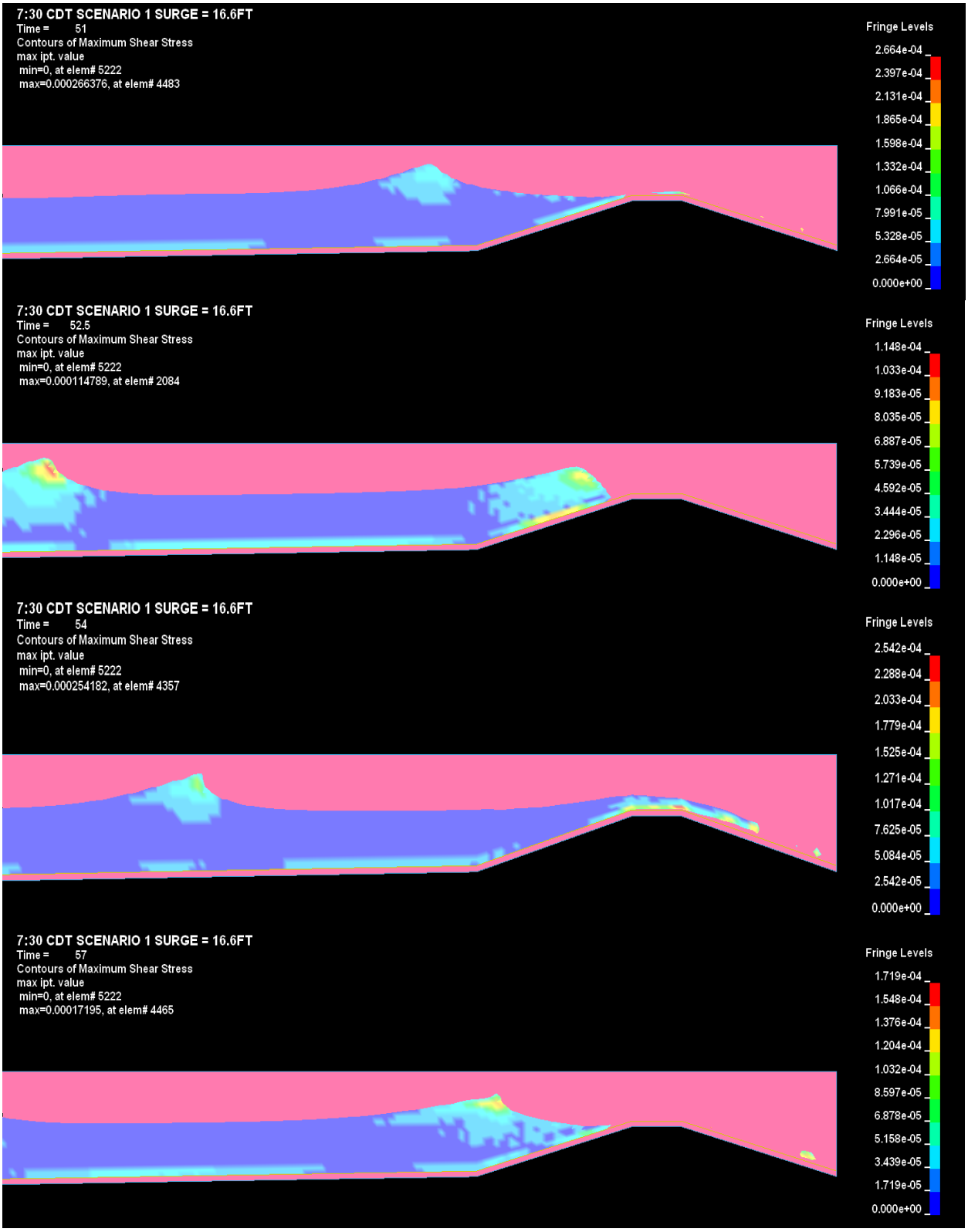


Figure 85**:** With the hurricane storm surge at Elevation +16.6 ft, the waves to exhibit a primarily overtopping condition with negligible wave impacts.

Analysis for Scenario 1 (0830 CDT)

84. Scenario 1 at 0830 CDT (Figure 86) consisted of a crest elevation of +16 feet (actual geometry) and hydrodynamics consisting of a storm surge elevation of +16.1 feet, a wave period of 5.2 seconds, and a significant wave height of 4.8 feet. There was wave-induced overtopping.

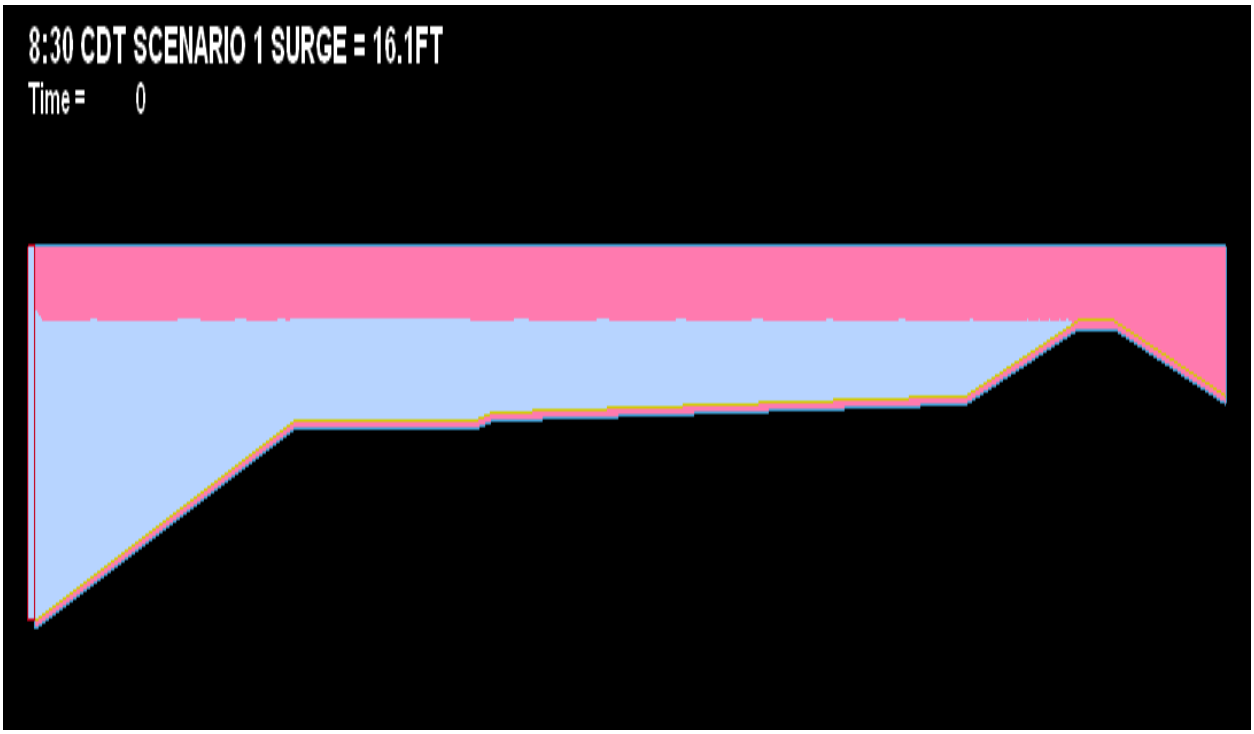


Figure 86: Initial conditions for Scenario 1 at 0830 CDT.

85. A plot of shear velocities is presented in Figure 87. The peak velocity was determined to be 14 feet per second.

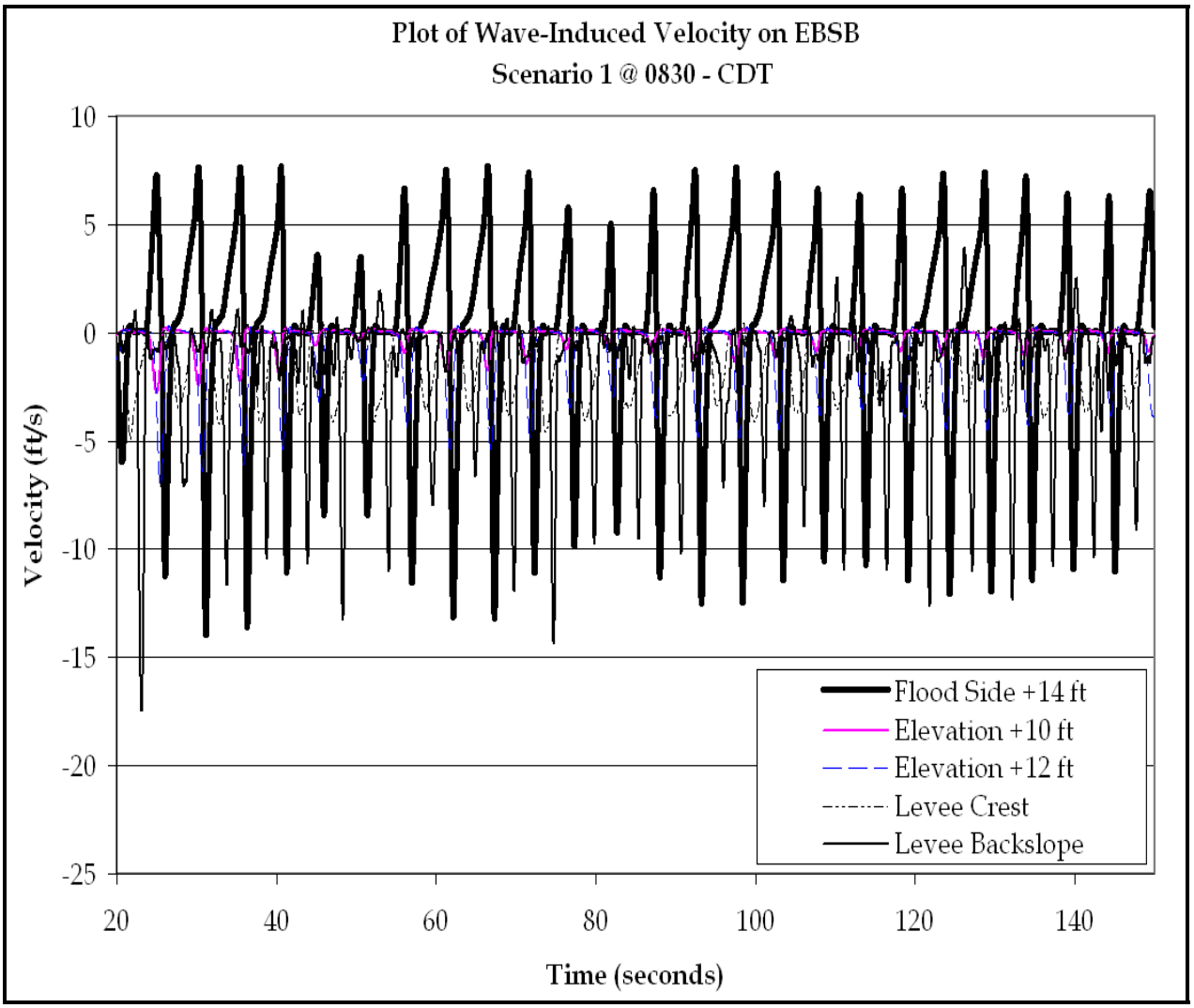


Figure 87: Time history plot of shear velocities for the storm surge at Elevation +16.1 feet.

86. Figure 88 provides a graphical representation of water movement over time. With the hurricane storm surge at Elevation +16.1 feet, the design geometric configuration of the EBSB induces a tendency for the waves to exhibit a primarily overtopping behavior, with minor wave impacts.

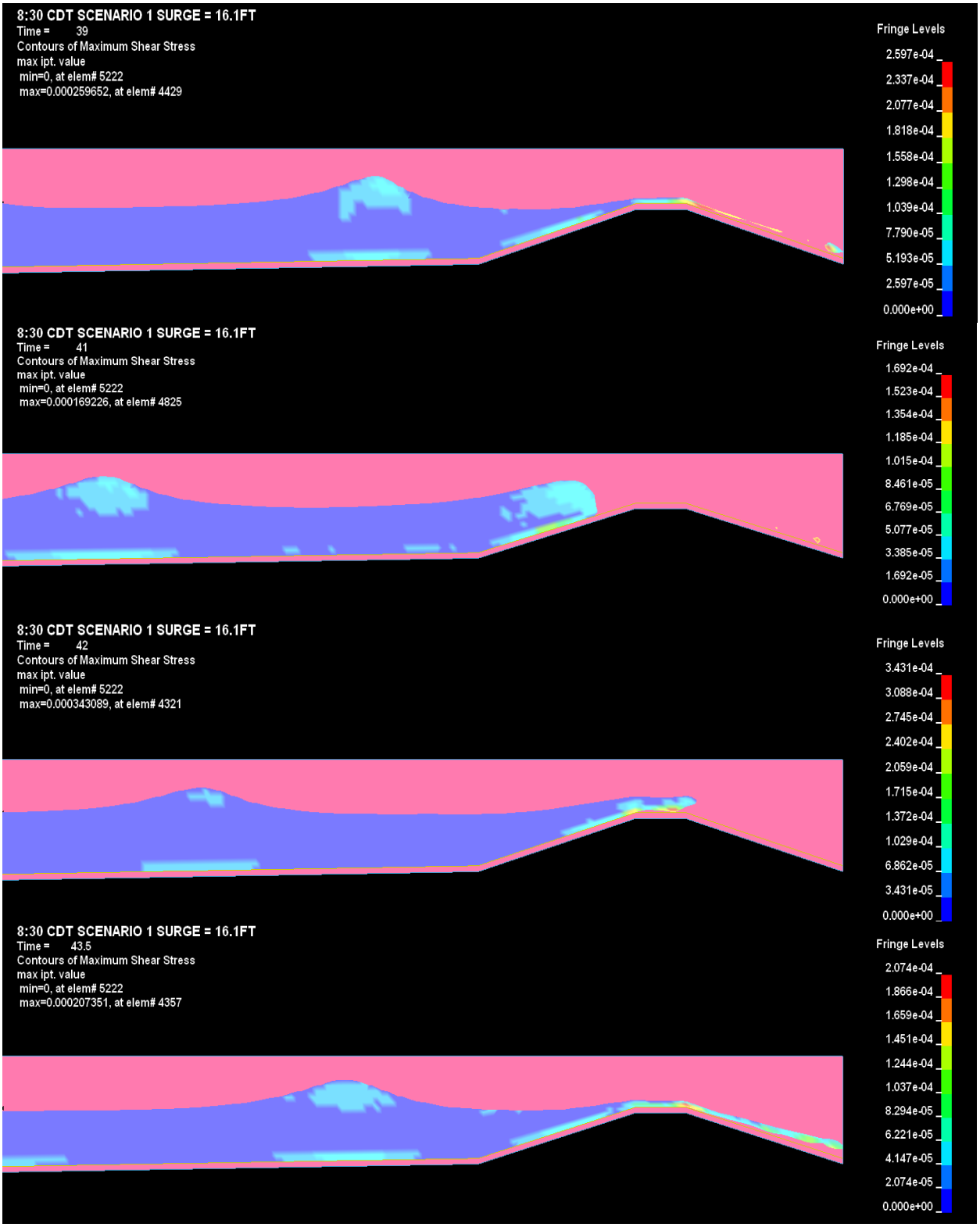


Figure 88**:** With the hurricane storm surge at Elevation +16.1 ft, the design geometric configuration of the EBSB induces a tendency for the waves to exhibit a primarily an overtopping behavior with minor wave impacts.

Analysis for Scenario 2C – 'Neutral' MR-GO Hurricane Katrina Conditions

87. The analyses for Scenario 2C – the 'Neutral' MR-GO Hurricane conditions were performed in the same manner as previously described for Scenario 1. The details of the Scenario 2C analyses will not be provided here. The details can be reviewed in the Technical Report by Bea and Storesund (2008a). The time histories of water velocities over the faces of the EBSB and water movements are provided in Figures 89 and 90 for the hydrodynamic conditions that provided the maximum velocities over the faces of the EBSB. The maximum velocities acting on the faces of the EBSBs developed at 0630 CDT for hydrodynamic conditions consisting of a storm surge elevation of +14.0 feet, a wave period of 3.4 seconds, and a significant wave height of 1.5 feet. There was no active overtopping.

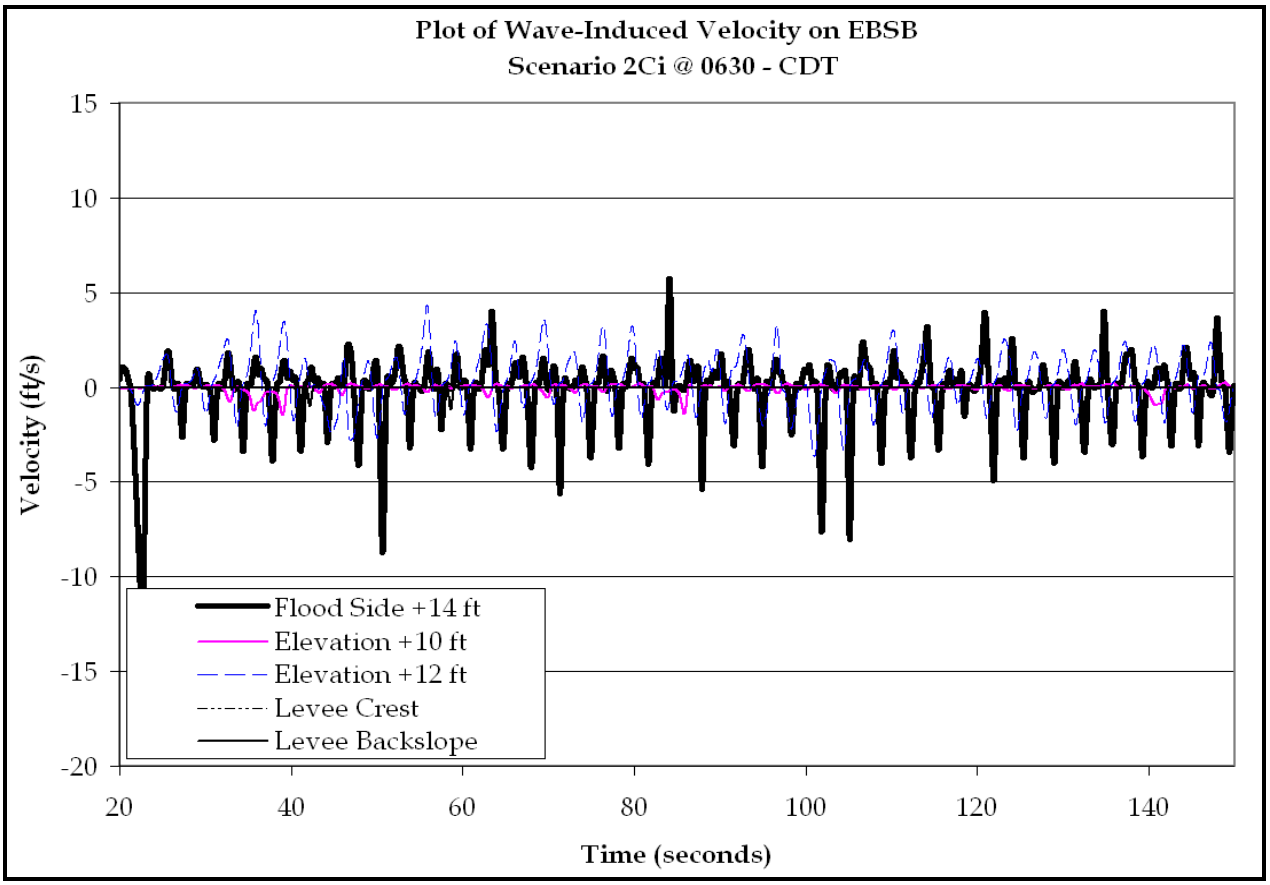


Figure 89**:** Time history plot of shear velocities for the storm surge at Elevation +14.0 feet.

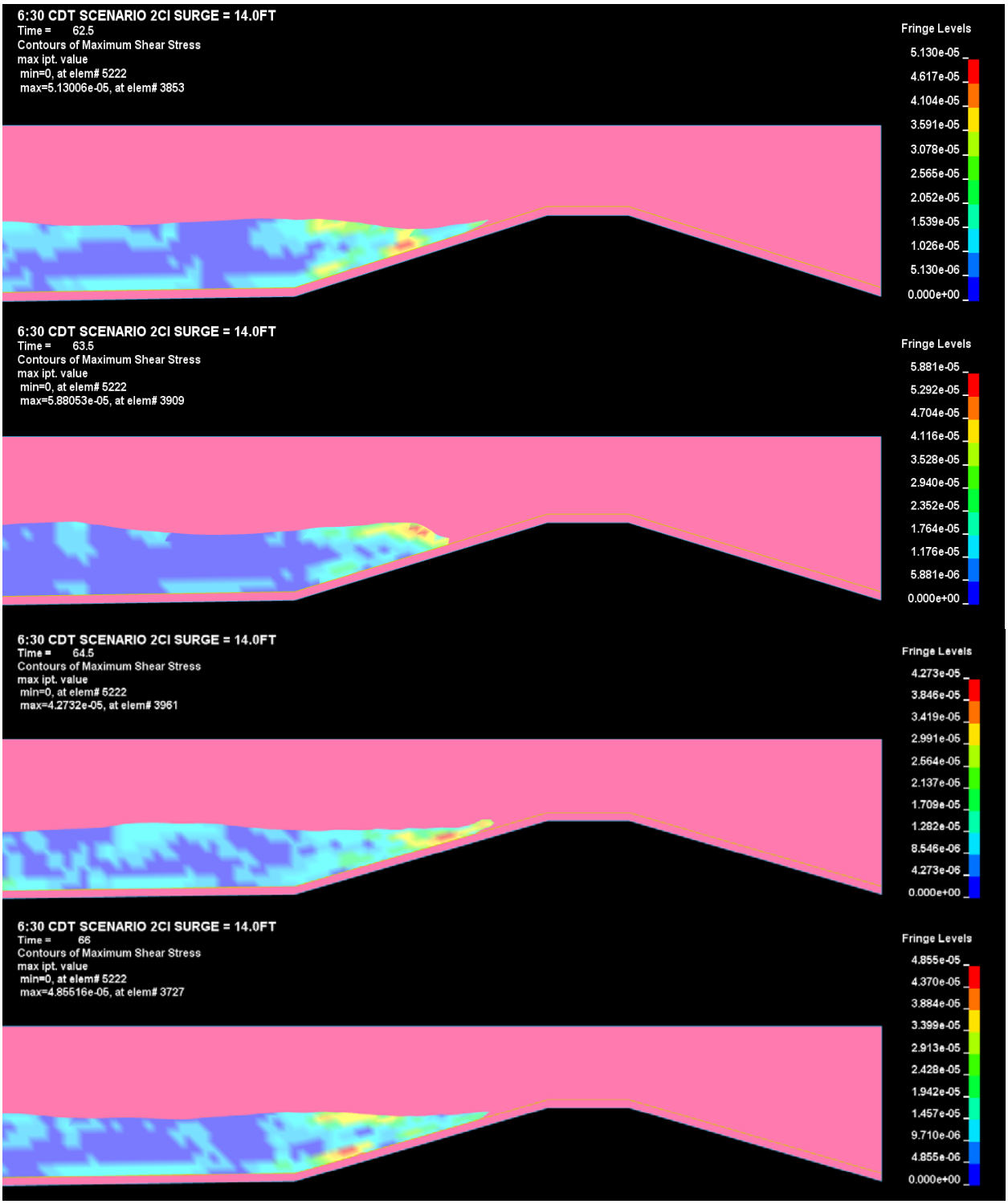


Figure 90**:** With the hurricane storm surge at Elevation +14.0 feet, the waves exhibit a plunging behavior with minor to moderate wave impacts and wave-induced overtopping.

Evaluation of EBSB Breach due to Wave Impacts

88. Lateral EBSB erosion was evaluated based on a three-step process: (1) velocity profiles were generated using LS-DYNA at discrete time intervals; (2) resistance to wave-induced erosion as a result of grass cover (turf) on the flood face of the EBSB was estimated and no wave-induced erosion was applied until the grass cover protection failed; and (3) estimated soil erodibility characteristics were used, in combination with the estimated wave-induced flood face velocities, to evaluate the magnitude of lateral erosion.

89. The analytical model used to describe the lateral erosion analyses is presented in Figure 91. Once the grass cover protection was compromised (failed at Damage Ratio of D = 1.0), the lateral erosion as determined by the LS-DYNA velocity results coupled with erodibility characteristics of the EBSB results in an erosion notch that migrates laterally towards the protected side of the EBSB. As shown in Figures 4 - 6, at this stage, the 'scarp' or head of the erosion feature is pointed toward the direction of surge and wave attack.

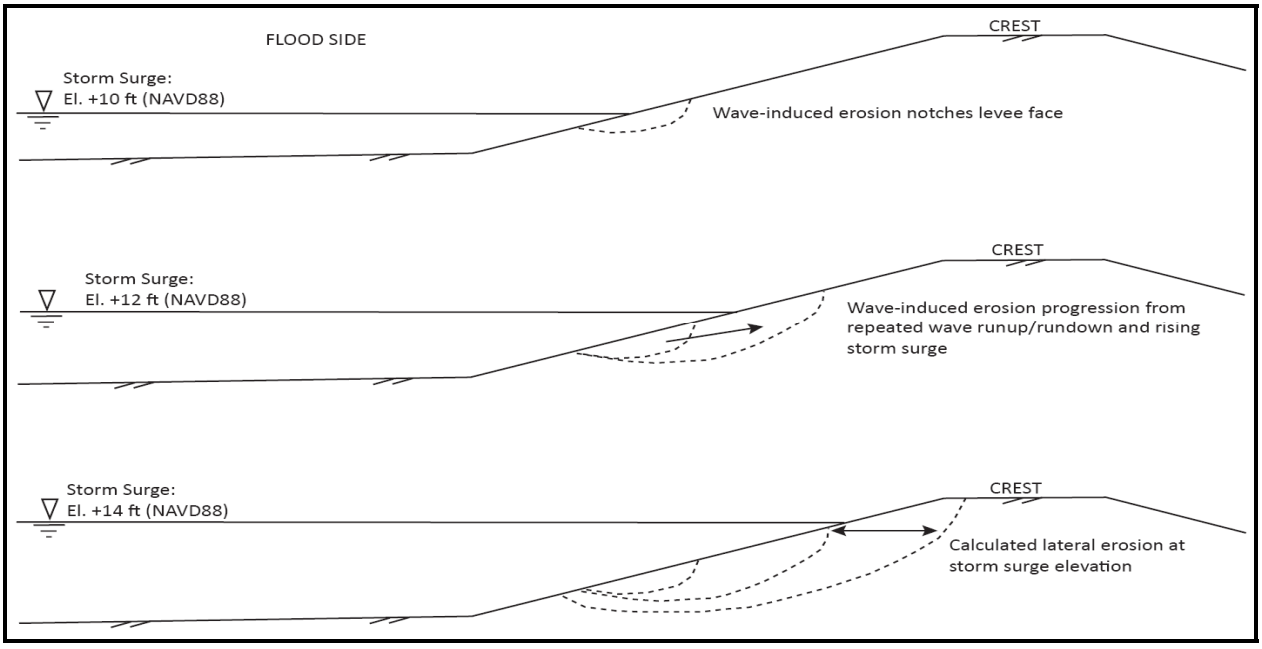


Figure 91: Development of wave-induced erosion on the flood face of the EBSB as a result of wave run up and rundown velocities on very high and highly erodible EBSB materials.

90. Figure 92 summarizes the grass lift-off times for good and poor grass density for Scenarios 1 and 2C. For Scenario 1 hydrodynamics, good grass density on sandy substrate was compromised by 0530 CDT and poor grass density on sandy substrate was compromised by 0500 CDT. For Scenario 2C hydrodynamics, good grass density on sandy substrate was compromised by 0800 CDT and poor grass density on sandy substrate was compromised by 0645 CDT. For the Scenario 1 wave conditions, the grass lift-off developed in 1 to 2 hours of wave exposure. For the Scenario 2C wave conditions, the grass lift-off developed in 3 to 4 hours of wave exposure. As noted previously, no EBSB erosion was calculated until turf lift-off developed.

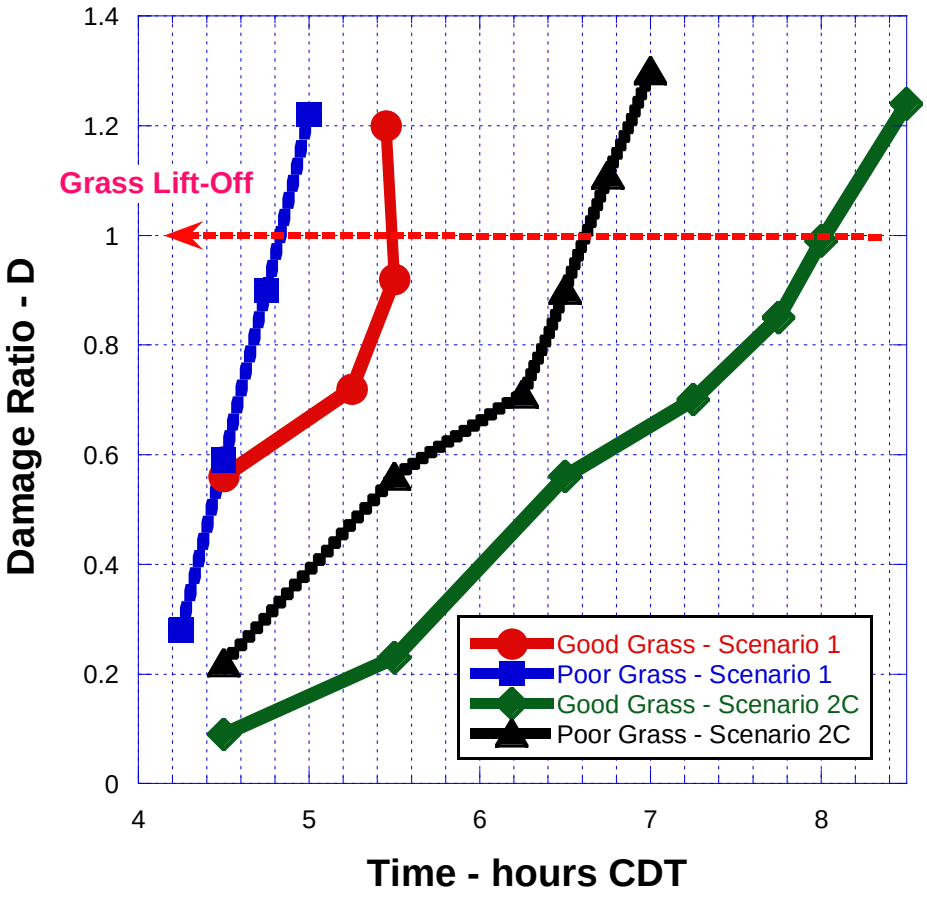


Figure 92: Grass lift-off times for Scenario 1 and Scenario 2C hydrodynamic conditions.

Wave-Induced Erosion Analyses

91. Wave-induced cumulative erosion of EBSB materials was calculated using an integrated time-step function based on the erodibility of the EBSB material at the time step being evaluated (where the erodibility of the time step is determined by the time-average shear velocities calculated at the defined storm surge elevations). The erodibility at the time step ($\dot{e}(v)$) is then multiplied by the value of the time step to calculate the extent of erosion (E) as follows:

$$E = \sum \dot{e}(v(t))\Delta t$$

92. Figures 93 through 95 show plots of computed lateral erosion induced from wave-attack on the MR-GO EBSB during Scenario 1 hydrodynamic conditions. These analyses include EBSBs constructed from 'very high erodibility,' 'high erodibility,' and 'low erodibility' materials. Each plot shows the erosion protection afforded based on 'good,' 'poor,' and 'no' grass cover.

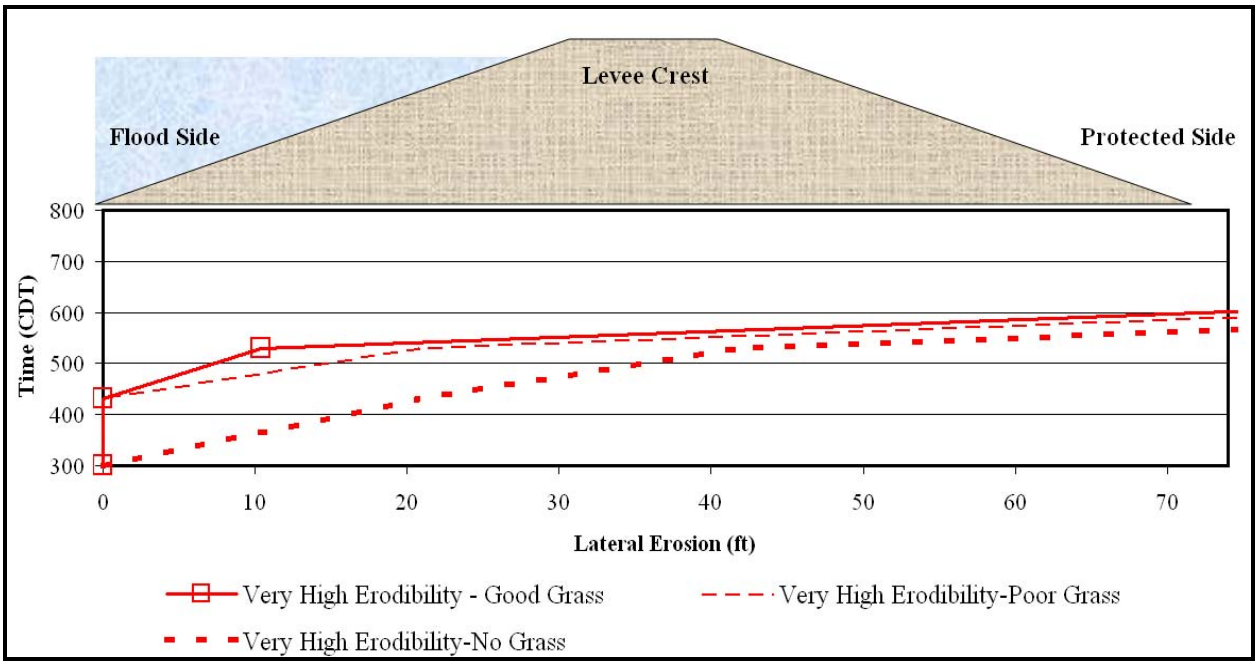


Figure 93: Lateral erosion induced from wave-attack on the MR-GO EBSB for the Scenario 1 hydrodynamic conditions with very high erodibility materials and different levels of grass armor (good grass turf, poor grass turf, and no grass turf).

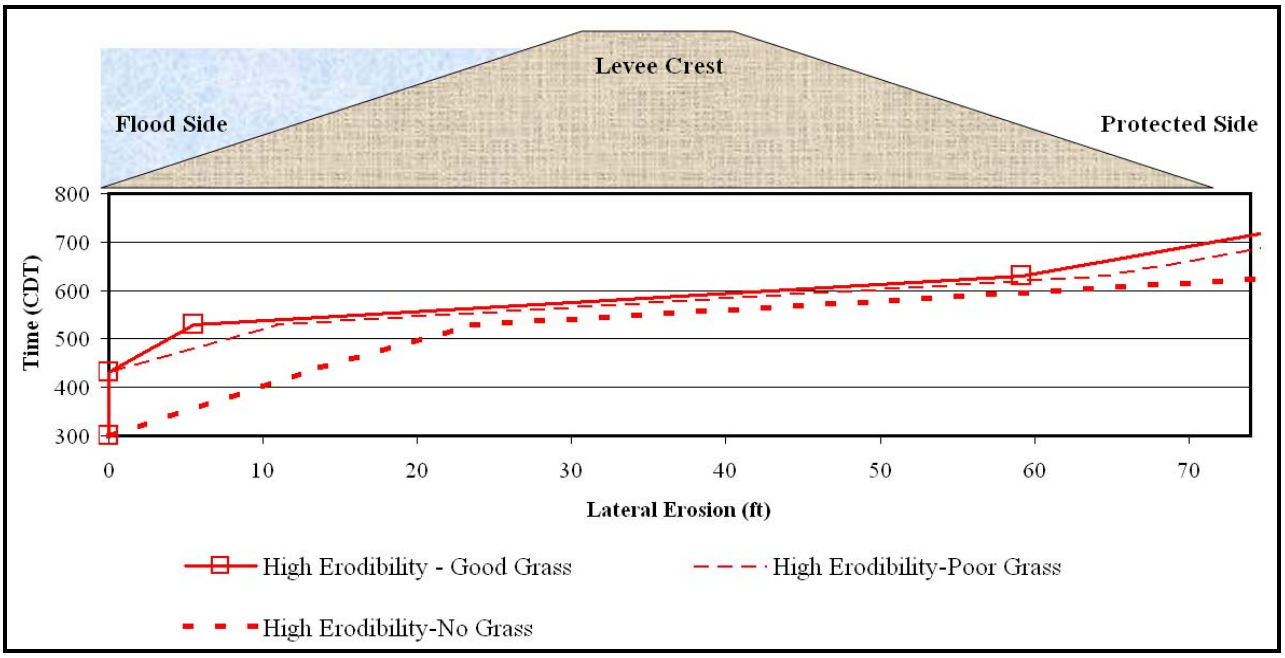


Figure 94: Lateral erosion induced from wave-attack on the MR-GO EBSB for the Scenario 1 hydrodynamic conditions with high erodibility materials and different levels of grass armor (good grass turf, poor grass turf, and no grass turf).

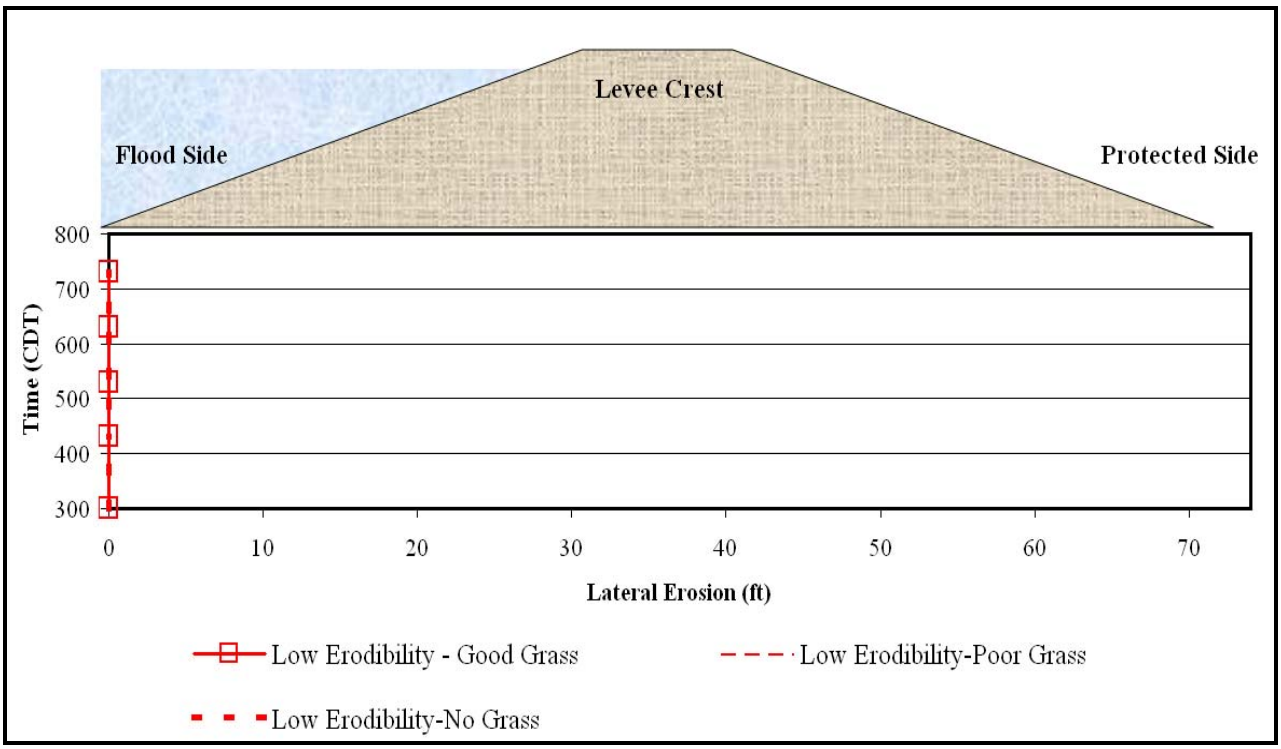


Figure 95: Lateral erosion induced from wave-attack on the MR-GO EBSB for the Scenario 1 hydrodynamic conditions with low erodibility materials and different levels of grass armor (good grass turf, poor grass turf, and no grass turf).

93. Figures 96 through 98 show plots of lateral erosion induced from wave-attack on the MR-GO EBSB during Scenario 2C hydrodynamic conditions and an EBSB constructed from 'very high erodibility,' 'high erodibility,' and 'low erodibility' materials, respectively. Each plot shows the erosion protection afforded based on 'good,' 'poor,' and 'no' grass cover.

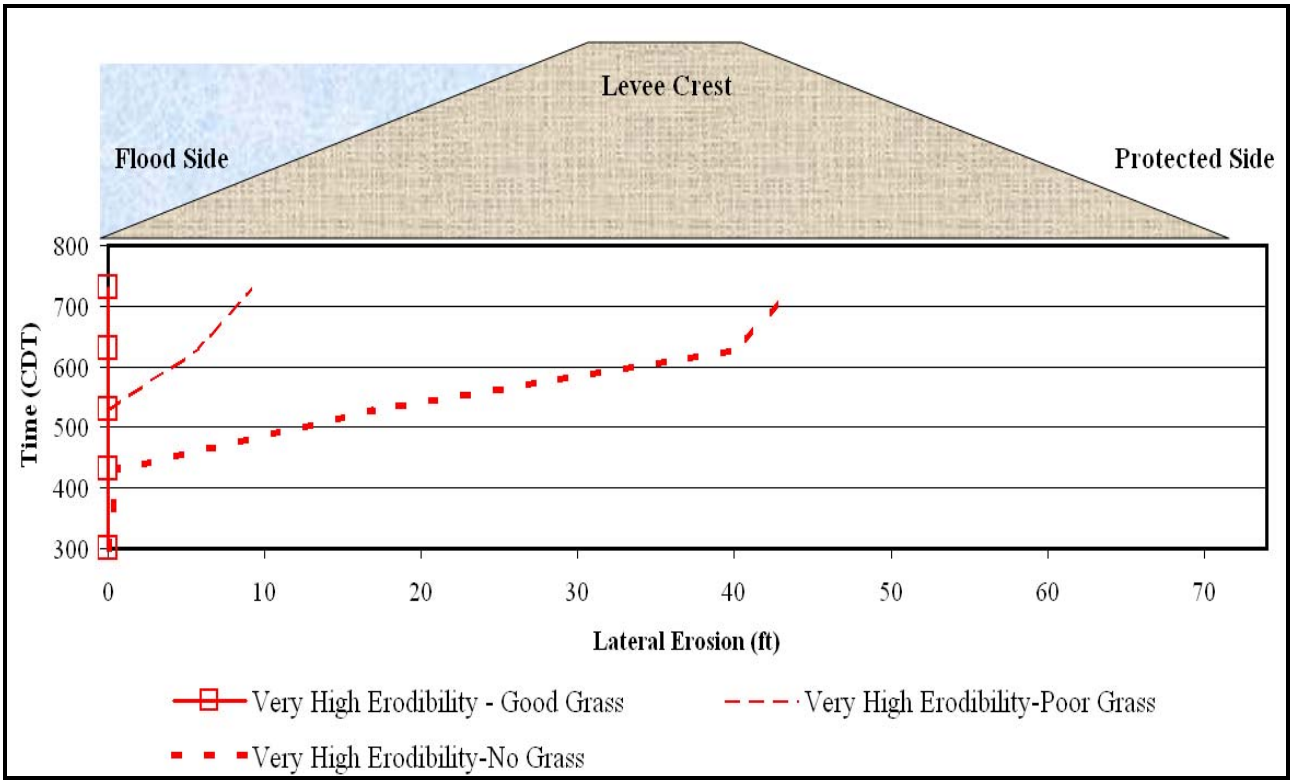


Figure 96: Lateral erosion induced from wave-attack on the MR-GO EBSB for the Scenario 2C hydrodynamic conditions with very high erodibility materials and different levels of grass armor (good grass turf, poor grass turf, and no grass turf).

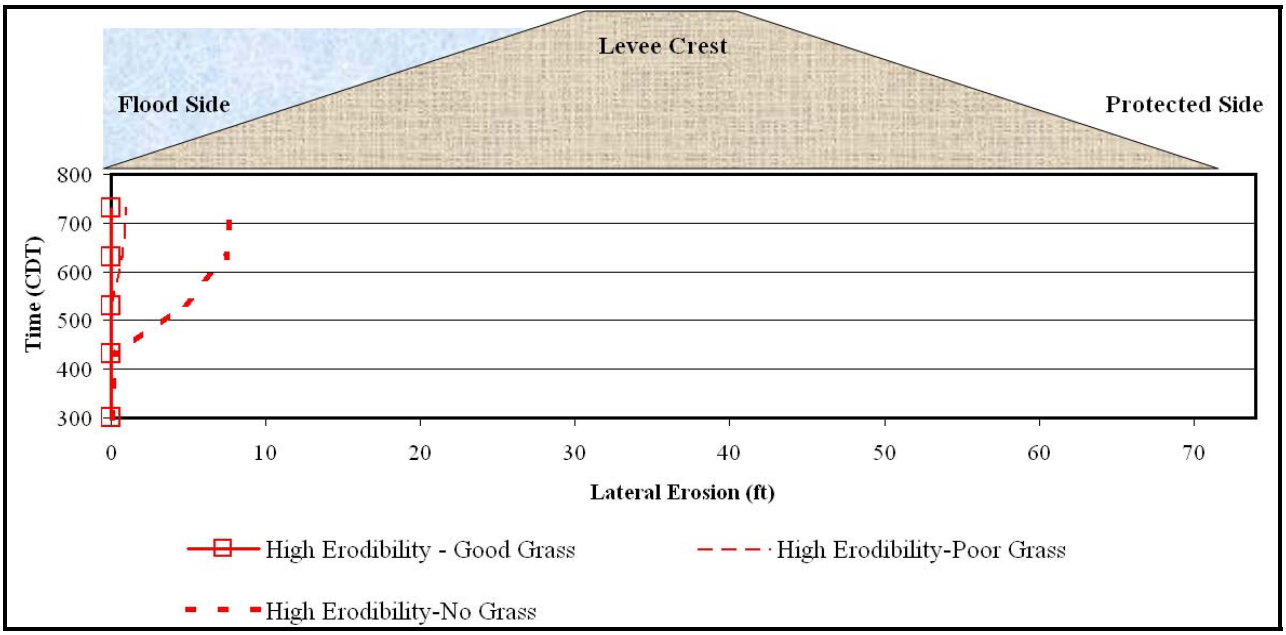


Figure 97: A plot of lateral erosion induced from wave-attack on the MR-GO EBSB for the Scenario 2C hydrodynamic conditions with high erodibility materials and different levels of grass armor (good grass turf, poor grass turf, and no grass turf).

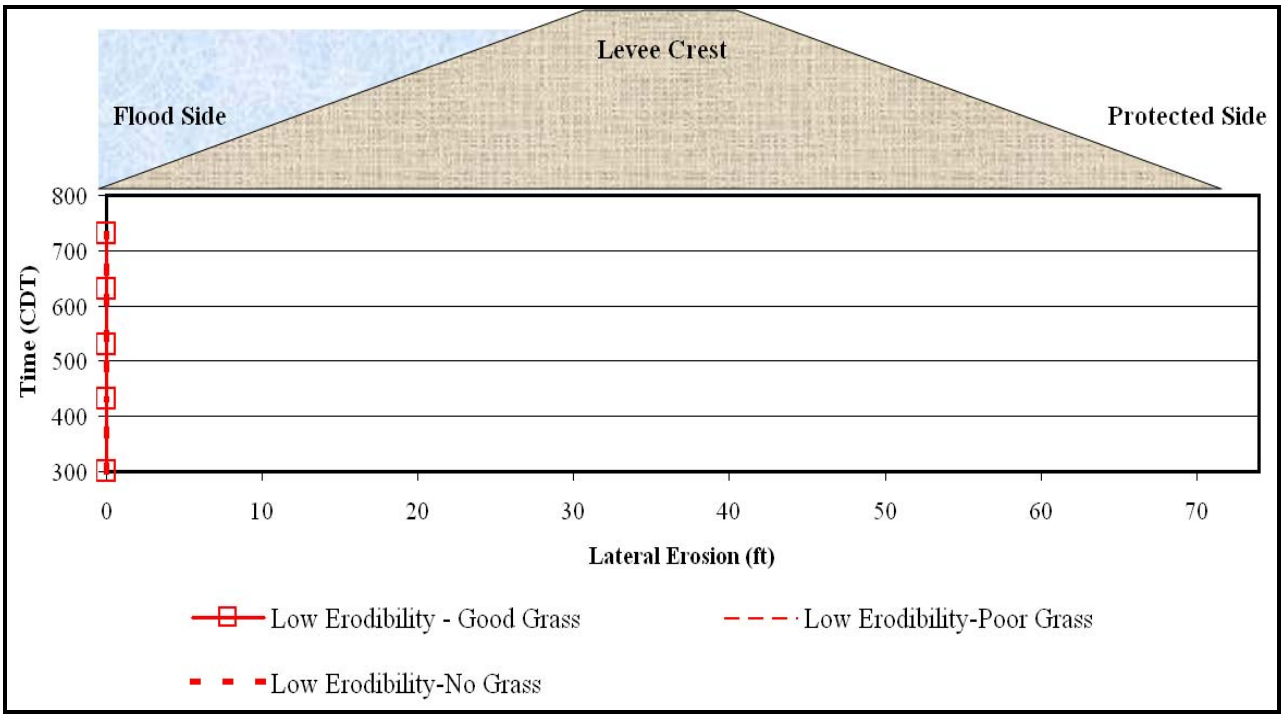


Figure 98: A plot of lateral erosion induced from wave-attack on the MR-GO EBSB for the Scenario 2C hydrodynamic conditions with low erodibility materials and different levels of grass armor (good grass turf, poor grass turf, and no grass turf).

94. Based on results from these analyses, it is concluded that for Scenario 1 – Hurricane Katrina ‘as was’ – hydrodynamic conditions, it was very likely for the EBSB to have been breached through the crest solely as a result of wave-induced erosion of the very high and high erodible materials used to construct the EBSB (no significant erosion would be anticipated had low erodibility materials been used to construct the EBSB).

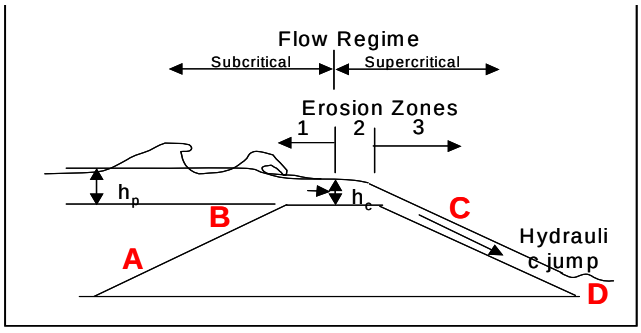
95. For the ‘Neutral’ MR-GO Hurricane Katrina Conditions - Scenario 2C hydrodynamic conditions - wave-induced breaching of the EBSB is not likely. Only if the EBSBs were constructed from highly erodible materials and had no protective grass cover at the time of the hurricane is there likelihood that the wave induced erosion would have reached the crest of the EBSBs. The assumption of no EBSB grass cover is not consistent with the ‘Neutral’ MR-GO conditions.

96. It is also quite apparent that an EBSB constructed of ‘good’ materials with low erodibility would have performed significantly better than one constructed of very high erodibility materials. Subjected to the same wave impacts, the EBSB constructed with ‘good’ materials (cohesive and compacted) would have experienced insignificant erosion. EBSB armoring in the form of ‘good’ (dense, good root structure development) grass or turf also provides an important element of protection to help assure acceptable performance in hurricane conditions (Hughes 2007).

EBSB Overtopping-Induced Erosion

97. Overtopping of levees, earthen dams and coastal dikes has been the subject of extensive research and engineering (Pullen et al 2007; Schuttrumpf and Oumeraci 2005; Meer and Janssen 1994; D'Eliso 2007; Schuttrumpf 2007; Meer 2002; Meer and Breteler 1990; Hughes 2008; USACE Coastal Engineering Manual 2006; USACE 2007). This condition was studied extensively during the IPET investigation (IPET 2007; Hughes 2007; Ebersole 2008). Much excellent technical engineering and scientific work on hydrodynamic characteristics and effects was done as part of this investigation.

98. The preceding part of this Declaration has addressed the "sub-critical" hydraulic flow regime identified in Figures 99 and 100 – the unprotected side attack of the surge and waves on the EBSBs – identified as 'Erosion Zone 1.' Erosion Zones 2 and 3 also must be addressed to develop a reasonably complete understanding of how 'surface hydraulic' attack can result in development of breaches in the man-made flood protection structures – EBSBs. The analytical approach used in this study is outlined in Figure 101.

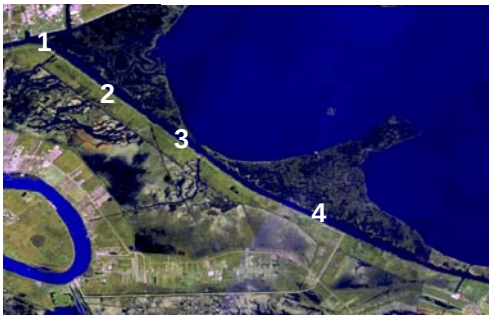


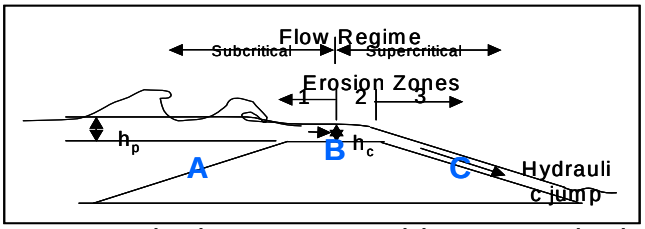
- **C** - Supercritical flow – pulsating w each wave
- **D** - Hydraulic jump and high turbulence
- violent and turbulent hydrodynamic conditions on back side
- much higher velocities on back side than on front side

- **A** - Wave orbital motions on front face in deeper water
- **B** - Zone of wave breaking and energy dissipation in shallow water



Figure 99: EBSB overtopping and overflow due to surge and wave action (Ebersole 2008).





- mean U is time averaged bottom velocity
- max U_c is average of max bottom velocities under each wave crest

Site	SWL above crest (ft)	wave height (ft)	wave period (sec)	front side (A) mean U (ft/sec)	front side (A) max Uc (ft/sec)	crest (B) mean U (ft/sec)	crest (B) max Uc (ft/sec)	back side (C) mean U (ft/sec)	back side (C) max Uc (ft/sec)
1	-1	2-5	14	0	5	3	10	4	9
2	+2	2-6	14	2	6	5	11	8	15
3	+4	2-5	14	7	11	8	12	14	20
4	-2	2-5	14	0	5	1	7	2	7

Figure 100: Velocity magnitudes at the MR-GO EBSBs (Ebersole 2008). Note the four locations (numbered 1-4) on the inset at the top left side. Location 2 is very close to the location used to perform detailed fluid dynamics –soil interactions of EBSB performance characteristics documented by Bea (2008a) and Bea and Storesund (2008a).

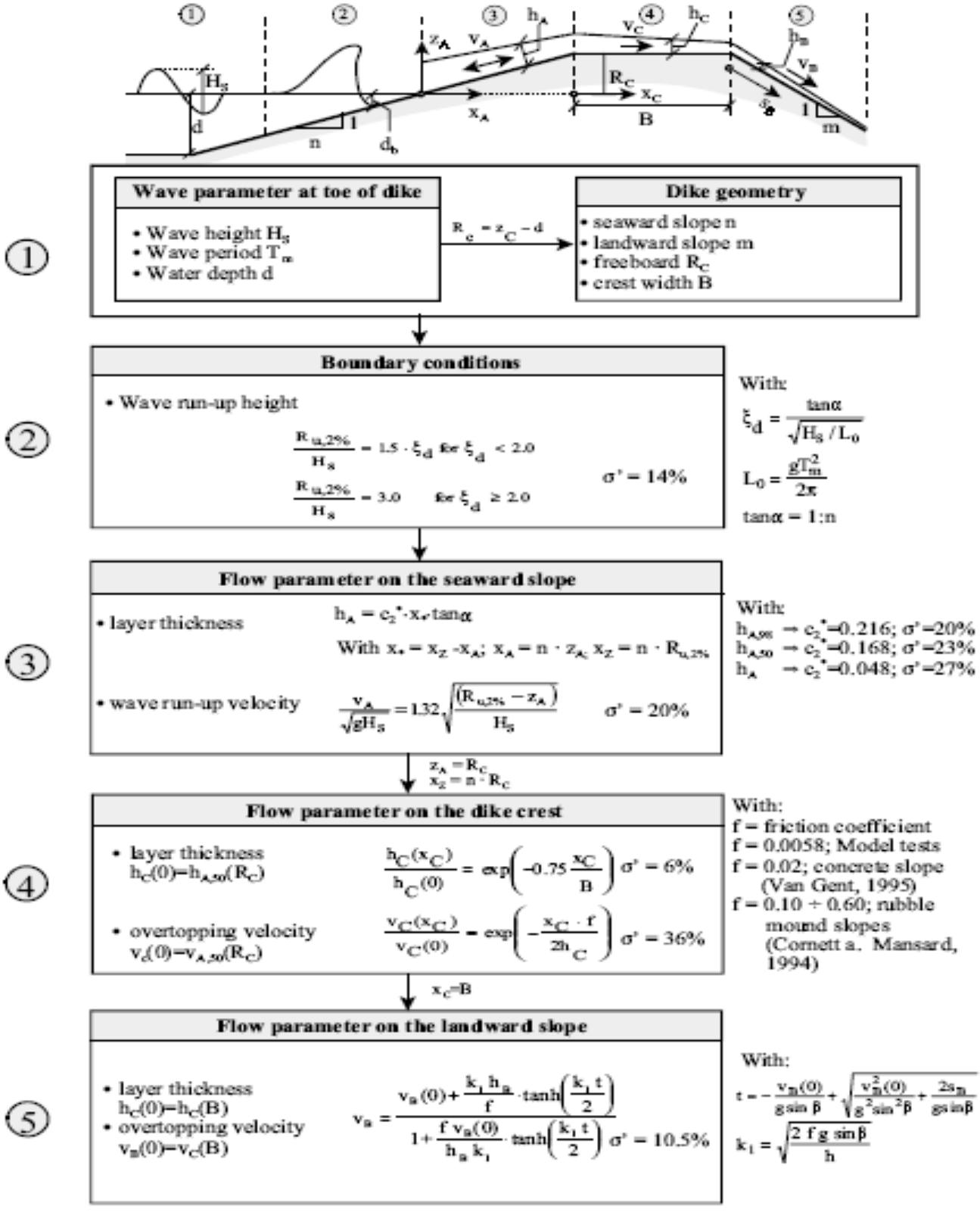


Figure 101: Prediction of overtopping flow characteristics (Schuttrumpf 2007).

99. As shown in Figure 100, the water velocities developed at the crest and backside of the EBSBs can be higher than those acting on the front sides. For the peak surge conditions, a primary determinant in these velocity characteristics and the associated erosion and breaching will be the elevation of the surge (hs) relative to the elevation of the crest of the EBSB (hc). The waves arriving at this point will have been affected by a wide variety of processes (e.g. diffraction, refraction, shoaling, friction, vegetation interactions, breaking, current interactions) (Pullen et al 2007; Goda 1985; D'Eliso 2007; Schuttrumpf 2007).

100. The wave characteristics at the crown of the EBSBs will be affected in a significant way by local elements (e.g. geometry of the MR-GO channel, channel bank, front side berm and levee face). The IPET investigation addressed some of these local effects to develop insights into the three dimensional characteristics of the waves arriving at the unprotected side face of the EBSBs (e.g Figure 102) and into the two dimensional affects of the waves as they arrived at the crown of the EBSBs (e.g Figures 103 and 104). These high-resolution simulations of hydrodynamic interactions utilized the COULWAVE (Cornell University Long and Intermediate Wave Model) based Boussinesq-type model (IPET 2007). Due to the wide range in geometric scales and details that are needed to properly evaluate the hydrodynamic processes and effects, multiple models capable of addressing the different scales and details must be properly employed and integrated. The IPET investigation of hydrodynamic characteristics and effects devoted significant efforts to assure that these different scales and details were properly addressed and integrated (IPET 2007). The investigation on which this Declaration is based repeated the same processes to properly address the different scales and required levels of detail.

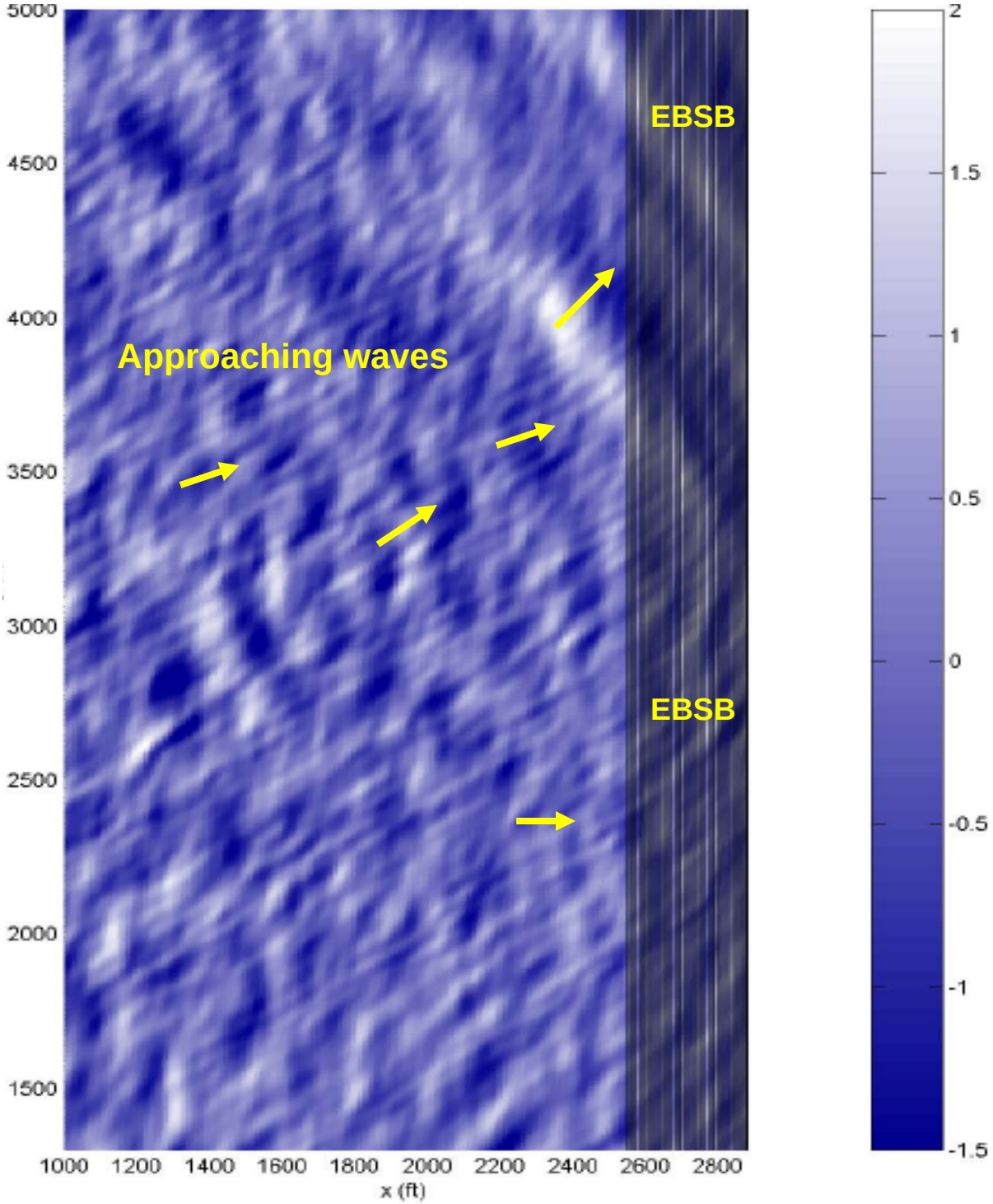


Figure 102: Plan view of 3-dimensional multi-directional sea state simulation at Point 2 IPET study location at 7:30 am CDT (dimensions shown are in feet, IPET 2007). The incident significant wave height is 3.2 feet and the surge elevation is 17.4 feet. The EBSB front face is shown as the dense black contours on the right. The short crested waves are arriving at various angles – almost broadside to the EBSB.

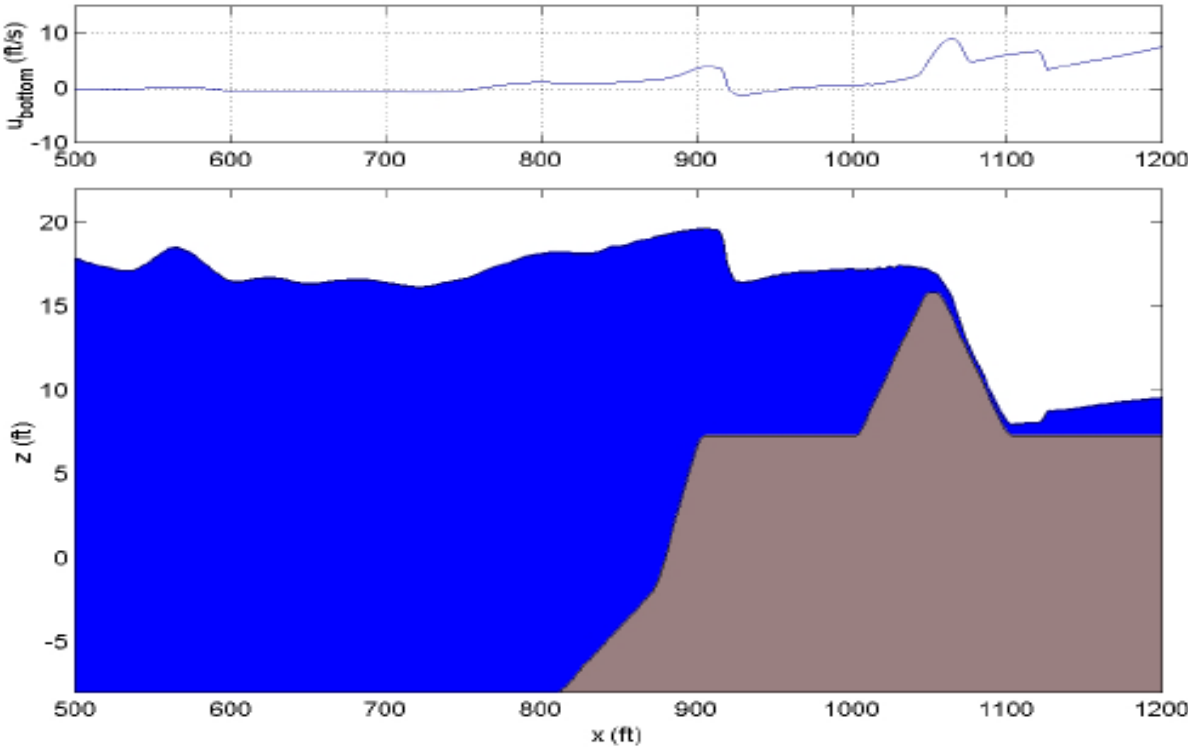


Figure 103: COLWAVE simulation for MR-GO Point 2 at 7:30 am CDT (IPET 2007). Incident significant wave height is 3.2 feet and surge elevation is +17.4 feet. EBSB crest elevation is 15.8 feet. Top plot shows the spatial profile of the near bottom velocity.

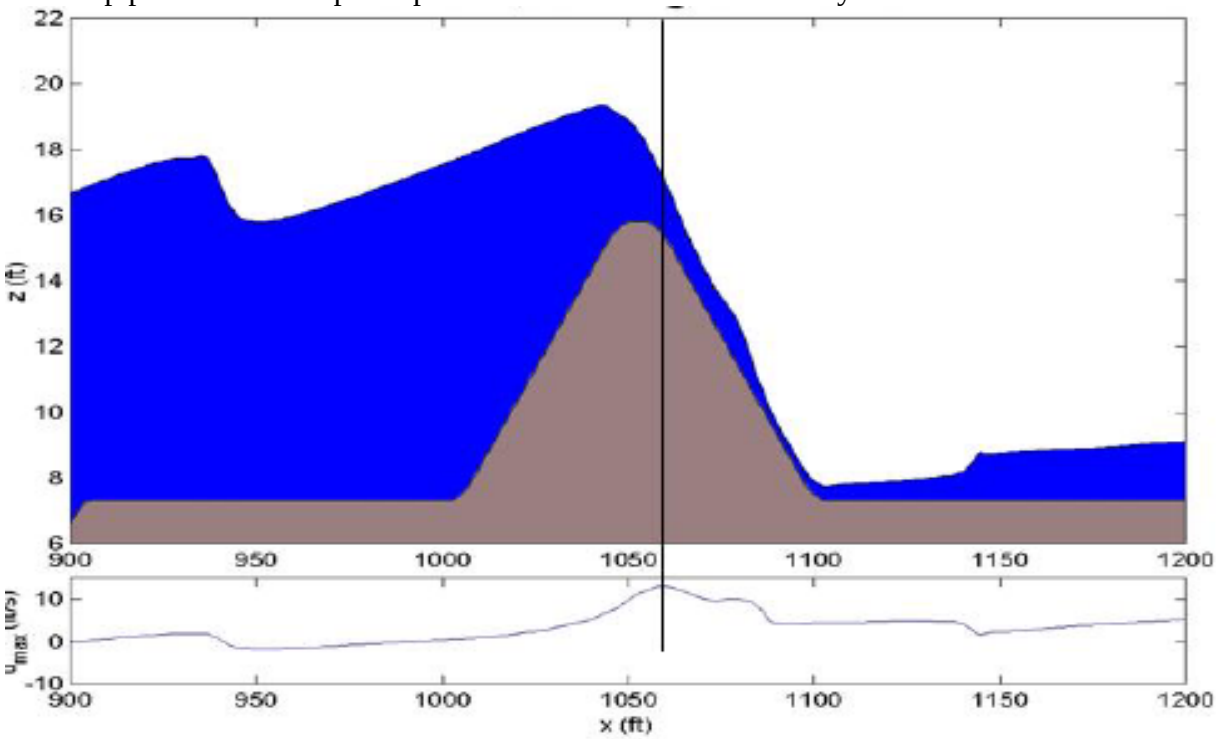


Figure 104: COLWAVE simulation for MR-GO Point 2 at 7:30 am CDT (IPET 2007). Incident significant wave height is 3.2 feet and surge elevation is +17.4 feet. EBSB crest elevation is 15.8 feet. Bottom plot shows the spatial profile of the near bottom velocity.

101. The results summarized in Figures 103 and 104 have particular importance. These results are for the conditions at the peak surge period of Hurricane Katrina along the MR-GO Reach 2. The incident significant wave height is 3.2 feet and the surge elevation is + 17.4 feet. These waves had unusually long peak periods – 13.5 seconds. Given the bathymetry of the Lake Borgne region, the bottom conditions, and the local wind generated wave conditions, these unusually long peak wave periods do not appear to be plausible. The EBSB crest elevation is 15.8 feet. The simulations show EBSB crown and protected side (Supercritical erosion zones 2 and 3 – Figures 99 – 100) near-bottom water velocities for these two time 'snapshots' in the range 5 to approximately 12 feet per second – in good agreement with the velocities developed by the LS-DYNA model.

102. The IPET investigation of the hydrodynamic characteristics and effects addressed the wave and surge overtopping characteristics associated with the EBSBs in their before and after Hurricane conditions (Figure 105). Because the crown elevations of the EBSBs have a very important influence on the overtopping rates and flow conditions at the EBSB crowns and backside slopes, the overtopping rates and flows were computed for the two bounding conditions – the EBSB crown elevations before and after Hurricane Katrina. These overtopping rates were then compared with the crown elevations of the EBSBs following Hurricane Katrina. The results generally showed good agreement between the overtopping rates and the areas that had been most severely eroded (Figure 106). This led to the general conclusion that overtopping had been primarily responsible for breaching the EBSBs. Results of the study provided by Ebersole (2008) showed many instances of front-side damage to the EBSBs caused by wave action.

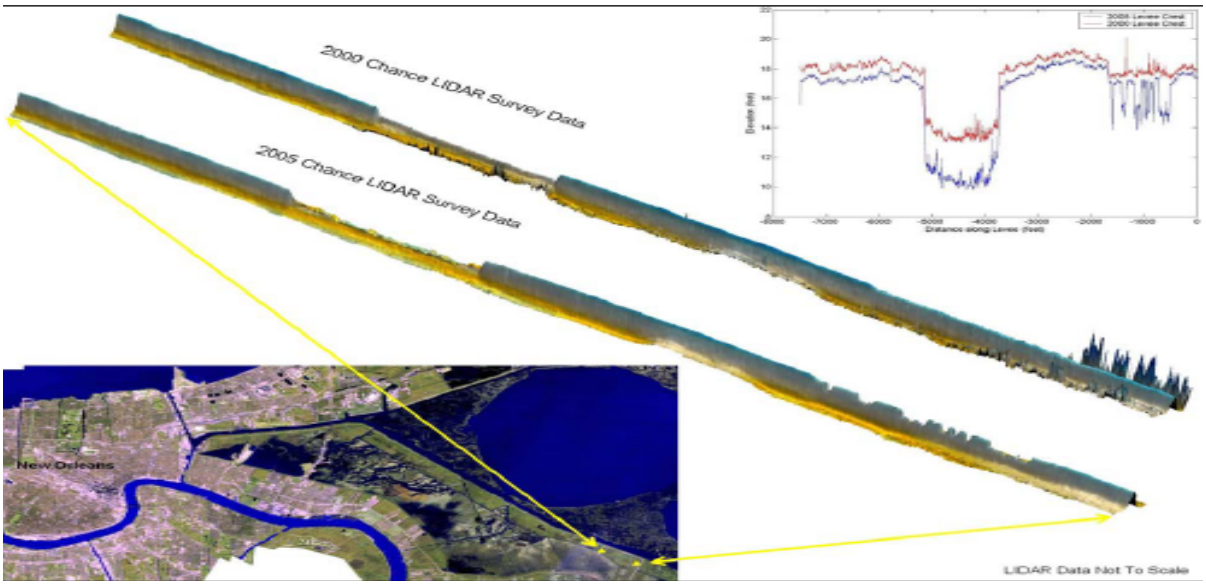


Figure 105: Topography of the EBSB crowns from pre- and post Katrina LIDAR surveys (IPET 2007).

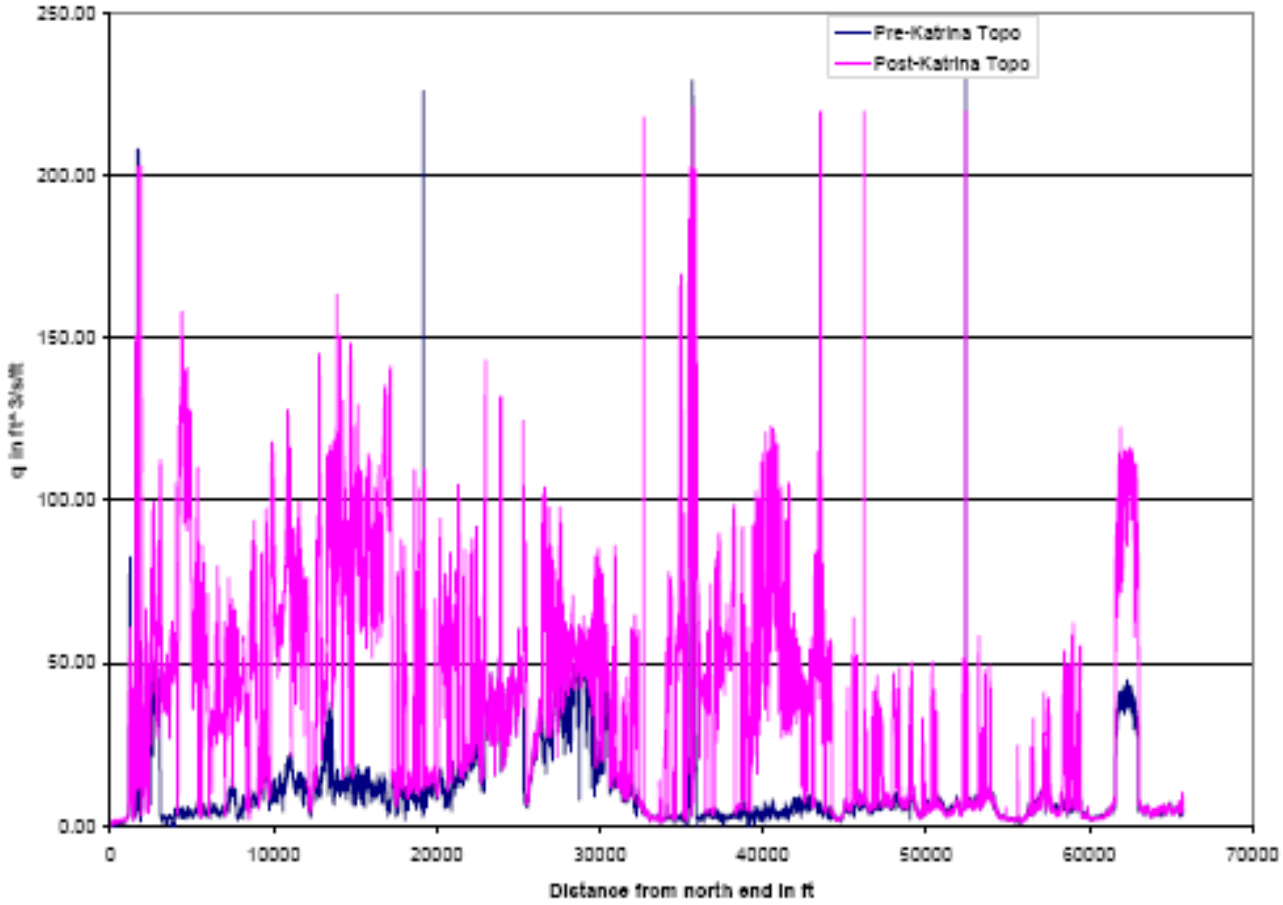


Figure 106: Overtopping rates for pre-Katrina and post-Katrina LIDAR survey crest heights along centerline of MR-GO levee corresponding to maximum surge elevation at approximately 7:30 am CDT on August 29, 2008 (IPET 2007).

103. Results from the investigation summarized here, in the Declaration by Bea (2008a) and the Technical Report by Bea and Storesund (2008a), demonstrate that during Hurricane Katrina wave side attack would have initiated breaching at many Reach 2 locations – Stage 1 in the breaching process. Once this front side to back side erosion process was started, significant surge and wave overtopping backside flow would continue and intensify the erosion process – Stage 2 - and result with final breaching of the EBSBs. This work indicated that, initially, the backside erosion process would be focused in the area on the backside of the EBSBs near their crowns or crests and would progress toward both the unprotected and protected sides. These fluid dynamics – soil erosion analyses showed erosion profiles that transitioned relatively seamlessly from front side to back side erosion. In this process, the 'evidence' for front side oriented erosion scarps was erased and replaced with backside oriented erosion scarps. In some instances, observations indicated that breaches that had developed during Hurricane Katrina were 're-activated' by water backflow through the breaches after the hurricane had passed. This process erased both front and back side erosion scarps. In these cases, it was not possible to determine how the erosion process had initiated – front side or back side. Flooding and outflows that accompanied Hurricane Rita that affected this region 4 weeks after Hurricane Katrina further obscured many erosion processes – particularly at the breaches associated with the EBSB wing wall tie in connections with the navigation structures at Bayou Dupre and Bayou Bienvenue.

104. Field experiments have been performed by Hanson et al (2001, 2003) and Temple and Hanson (2005) to evaluate erosion processes during embankment (earthen dam) overtopping. The erosion migration process proceeds through a series of progressive 'steps' that depend on how the overtopping process is initiated. If it is initiated by general overtopping of the slope (Figure 107), the process is initiated with down slope propagating erosion that then transitions

into erosion that propagates upslope toward the reservoir. However, if the process is initiated with a 'head-cut', the erosion propagates primarily upslope toward the reservoir (Figure 108). The measured vertical profiles of the head-cut erosion features (Figure 109), indicate that the process moves rapidly toward the reservoir and the rate of propagation is a function of the characteristics of the soils that comprise the embankment and the overflow water shear stress exerted on the soils (Temple and Hanson 2005).



Figure 107: Erosion processes during overtopping test: (a) small overfalls, (b) consolidation of overfalls, (c) headcut at downstream crest, (d) headcut at upstream crest, (e) flow through breach (Hanson et al 2003).



Figure 108: Embankment overtopping test with head cut notch to conduct flow and initiate breaching (Hanson et al 2001). The embankments had 1:3 slopes and were comprised of clayey (5 to 25%) sandy – silty (75 to 95%) soils.

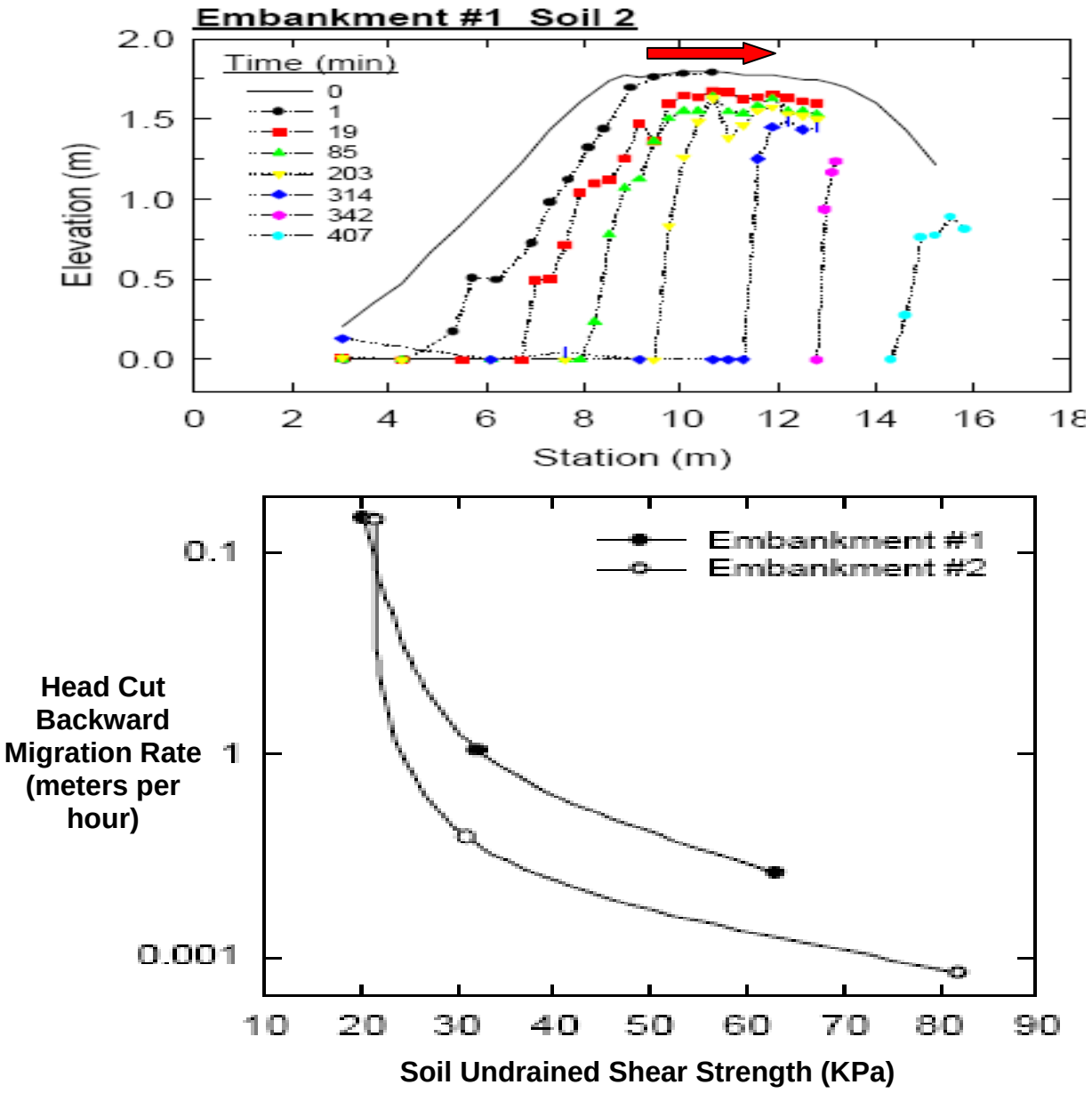


Figure 109: Centerline profile showing up-stream head cut soil profile migration as function of time of overflow (upper) and head cut backward migration rate (lower) as a function of the soil unconfined compressive strength (qu, 1 Kpa = 21 psf) (Hanson et al 2001).

105. For the range of EBSB soil surface strengths (240 to 350 pounds per square foot) (IPET 2007, ILIT 2006), the results developed by Hanson et al (2001, 2003, 2005) and Hunt et al (2005) indicate head cut regression rates in the range of 1 or more meters per minute or 200+ feet

per hour! It will be shown later in this Declaration that these results agree very well with EBSB erosion results based on several other approaches.

106. The IPET hydrodynamic characteristics and effects investigation and studies addressed the range of backside slope velocities that should be addressed for overtopping conditions (Figure 110) (IPET 2007). The range of backside slope velocities for the pre-Katrina EBSB backside slopes (1:3 to 1:4 – ratio of vertical to horizontal dimension) for surge heights above the EBSB crown of 1 to 2 feet are 5 to 18 feet per second (depending on the slope roughness expressed with the range of Manning's friction coefficients 0.025 and 0.10). These backside slope velocities do not include the additional velocities associated with wave 'pulses' that pass over the EBSB crests – crowns and propagate down the backside of the EBSBs. Hughes (2007) evaluated representative wave overtopping flow parameters applicable to these analyses (Figure 111). For the Hurricane Katrina conditions, wave pulse velocities in the range of 13 to 15 feet per second are indicated. These velocities are in excellent agreement with those determined using the LS-DYNA simulations (Figures 84 and 87) for the surge elevations close to the elevation of the crown of the EBSB.

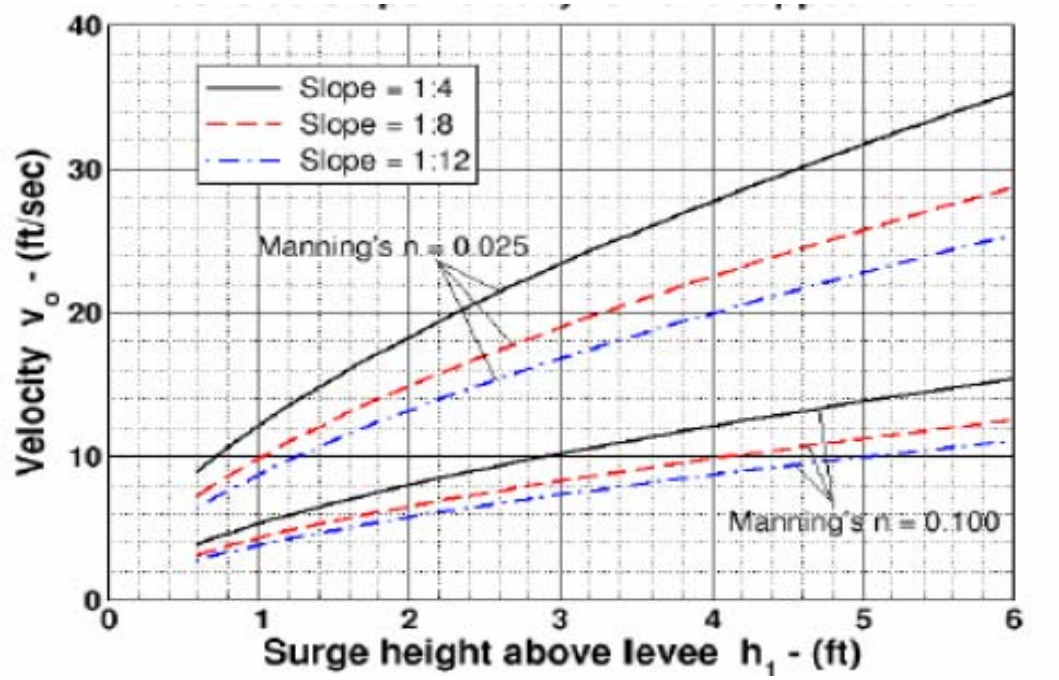


Figure 110: Overtopping flow velocity as a function of surge height (Hughes 2006, IPET 2007).

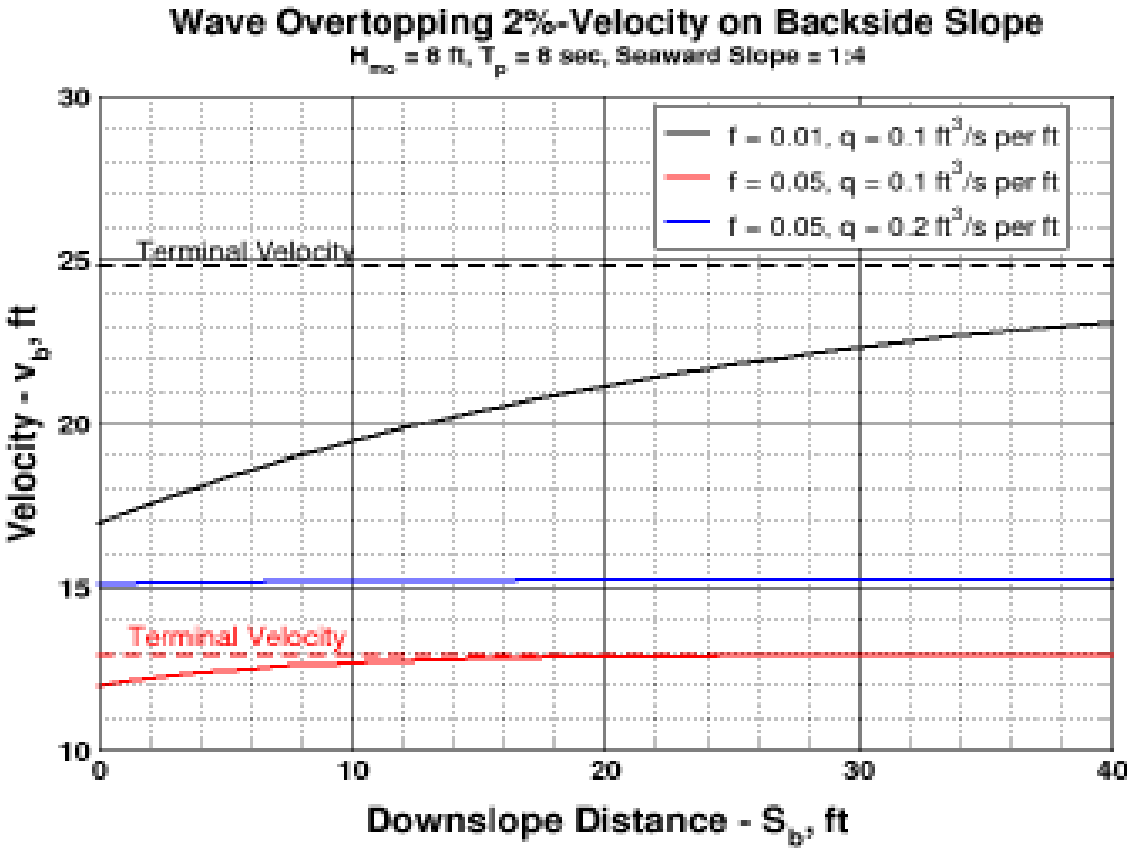


Figure 111: Peak velocities on levee protected-side slope exceeded by 2% of the waves (Hughes 2007).

107. The analytical models documented in the IPET hydrodynamic characteristics and effects investigation and studies (IPET 2007, Hughes 2007) and those developed by Schuttrump (2007), Pullen et al 2007), D'Eliso (2007) and Hewlett et al (1987) were used to develop the EBSB surge and wave overtopping velocity assessments summarized in Figure 112. The velocities are shown as a function of the water surface height over the EBSB crest (surge and wave elevation minus the EBSB crown elevation). Given EBSB crown elevations in the range of +14 to +17 feet (NAVD 88) and peak surge elevations of +18 feet (NAVD 88), then one could expect backside water velocities in the range of 5 to 17 feet per second. These results are consistent with those developed by the IPET studies. Terminal (maximum) overtopping flow velocity values also were evaluated based on the analytical model provided by Hewlett et al (1987) (Figure 113). Results were determined for a range of Manning's coefficients and these are summarized in Figure 114.

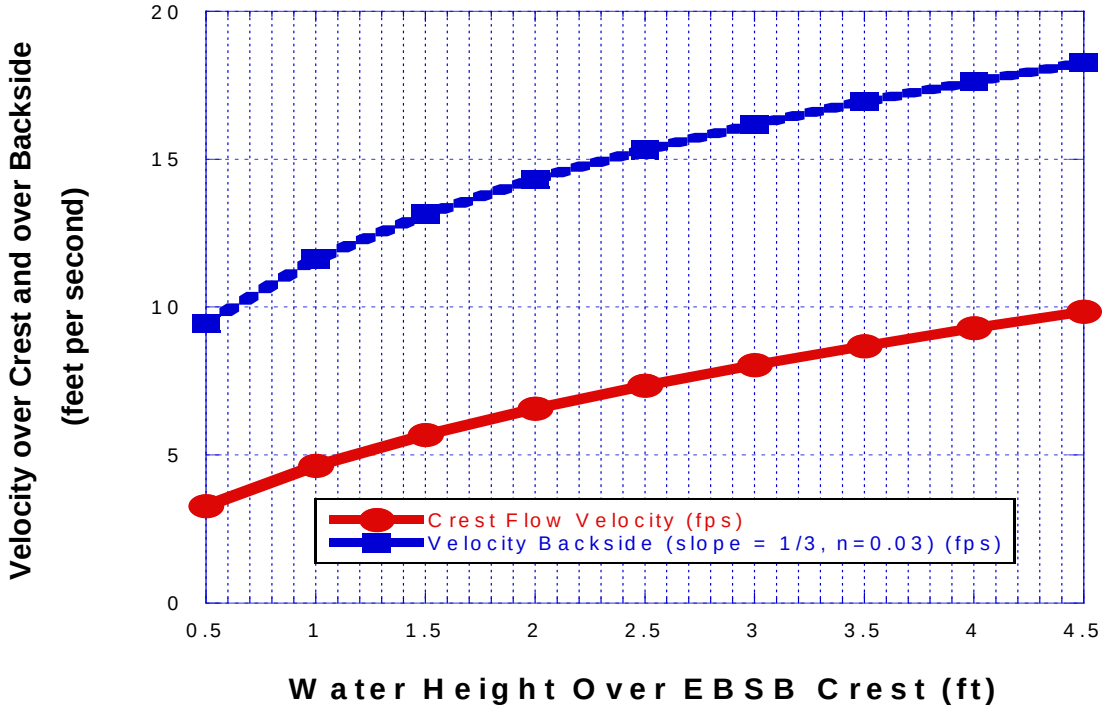


Figure 112: Crest and backside slope surge overtopping velocities as a function of the surge height over EBSB crest.

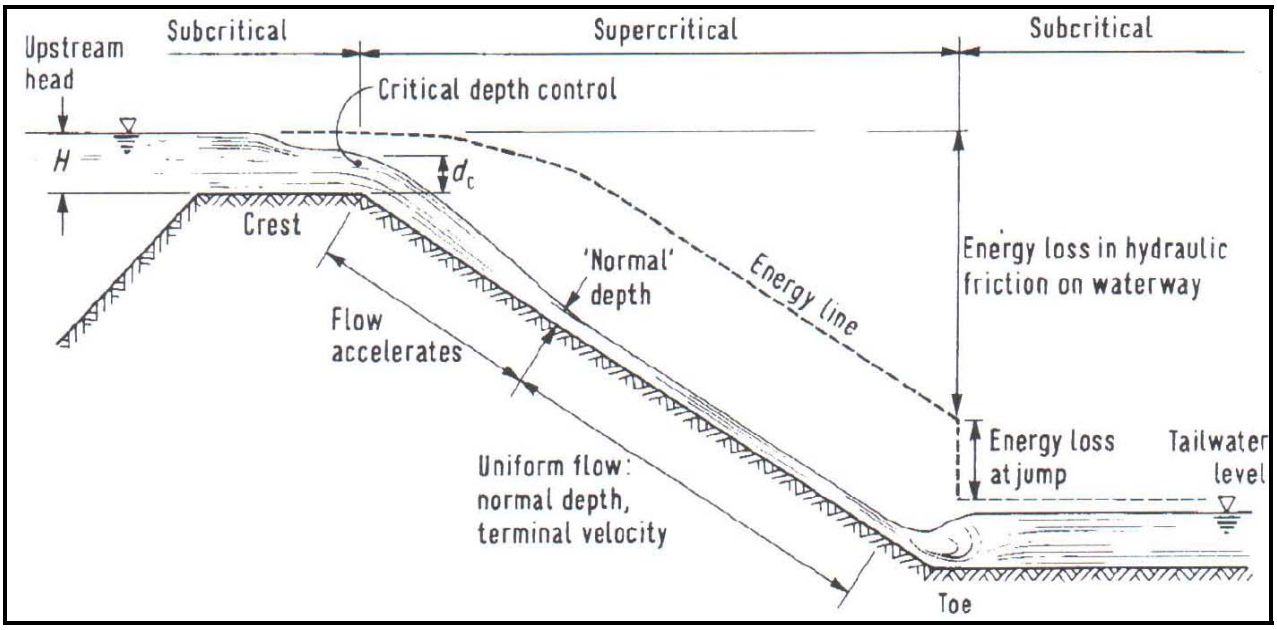


Figure 113: Graphic depiction of flow regimes during overtopping conditions (from Hewlett et al, 1987).

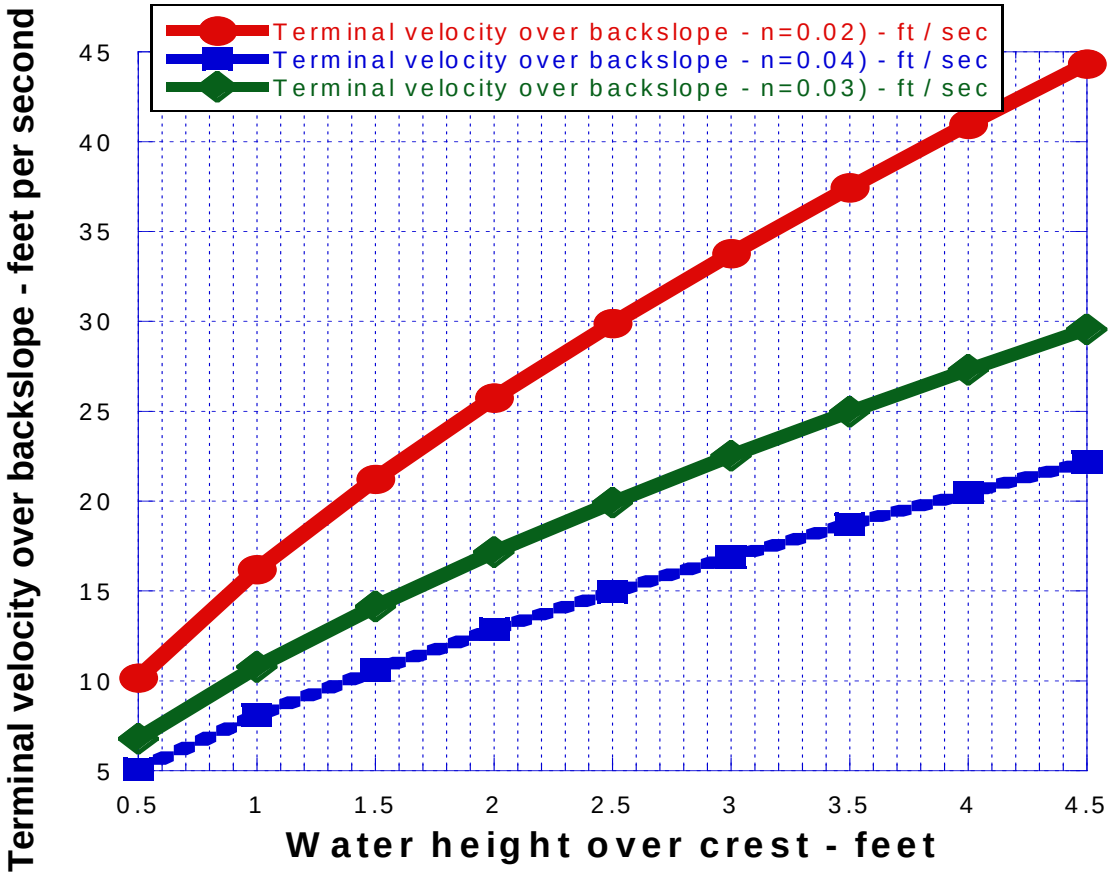


Figure 114: EBSB backside slope overtopping velocities as a function of the water height over EBSB crest.

108. The next step in these analyses was to evaluate damage done to the EBSB protective grass cover (good and poor grass cover on sandy substrate). The results are summarized in Figure 115. For Scenario 1 Hurricane Katrina wave and surge conditions, the results show that the EBSB backside grass cover would be breached (lift-off) only for the poor grass cover at about 7:45 am CDT. The backside good grass cover would not be breached. For Scenario 2C 'Neutral' MR-GO conditions, the results show that the EBSB backside good or poor grass cover would not be breached.

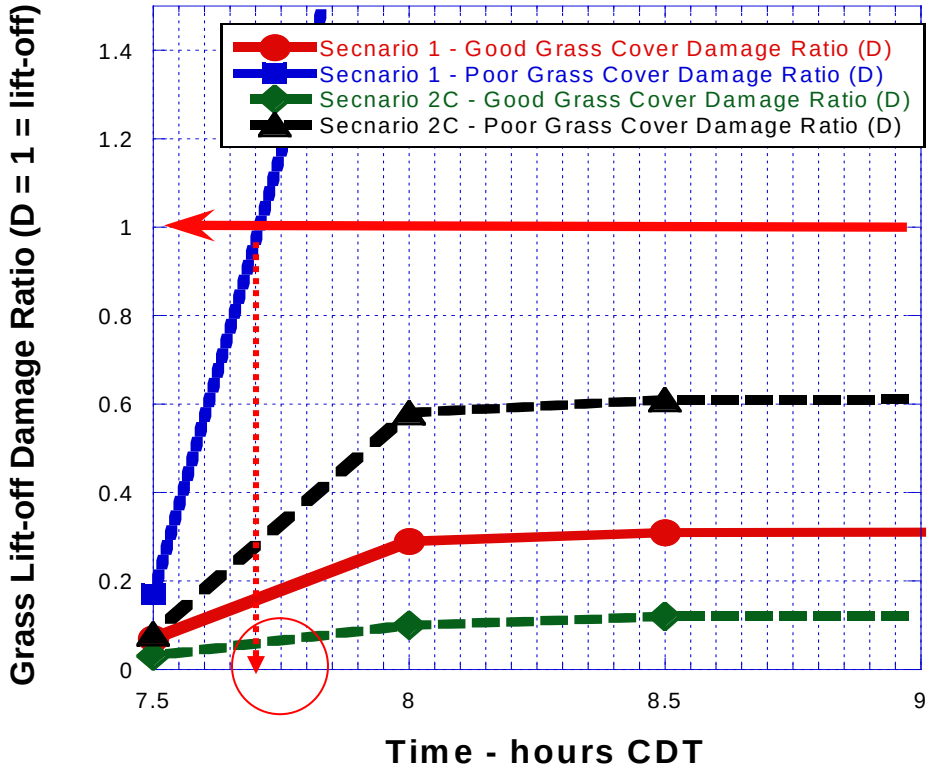


Figure 115: Total grass damage due to backside flow as function of Hurricane Katrina surge and wave conditions at the times shown.

109. These analyses indicate that the only evaluated condition that would result in compromise of the grass cover protection and initiation of active overtopping scour was the Scenario 1 'poor grass' cover case. Based on a Hurricane Katrina overtopping duration of 45 minutes (0745 CDT to 0830 CDT), the EBSB section would be expected to incur approximately

100 feet of erosion from the unprotected to the protected side for Very High Erodibility materials, approximately 90 feet for High Erodibility materials, and no erosion for Low Erodibility materials.

Parametric Wave Attack Erosion Evaluations

110. Parametric analyses of EBSB front side wave attack erosion were conducted to determine the influences of variations in important parameters on the EBSB performance characteristics during Hurricane Katrina. The evaluations considered a number of hydrodynamic conditions, impact of hurricane storm surge currents running parallel to the shoreline/EBSBs, and design configuration (EBSB side slopes of 1 on 5, crest elevation +17.5 feet) vs. actual configuration (EBSB side slopes of 1 on 3.5, crest elevation +16.0 feet) of the EBSBs. Table 2 summarizes Hurricane Katrina hydrodynamic conditions based on results provided by IPET (2007), LSU (2007), and Kok et al (2007). These different hydrodynamic conditions were used in parametric studies of EBSB wave breaching. These parametric studies also addressed the actual and design EBSB cross section geometries. The storm surge elevations varied from Elevation +8 ft to Elevation +17.5 ft. The significant wave heights varied from 1.1 feet to 7.9 feet with wave periods between 2 seconds and 13.5 seconds.

Table 2: Summary of IPET (2007), LSU (2007), and Kok et al (2007) Hurricane Katrina Hydrodynamic Conditions

Hydro-dynamics	EBSB Geometry	Storm Surge Elevation (ft +NAVD 88)	Significant Wave Height (H_{mo}) - feet	Wave Period - sec
IPET	Actual	8.0	1.1	3.5
IPET	Actual	12.0	1.6	4.4
IPET	Actual	17.5	2.6	13.5
LSU	Actual	8.0	1.3	2.0
LSU	Actual	12.0	2.0	2.0
LSU	Actual	17.5	3.2	6.0
IPET	Design	8 .0	1.1	3.5
IPET	Design	12 .0	1.6	4.4
IPET	Design	17.5	2.6	13.5
LSU	Design	8.0	1.3	2.0
LSU	Design	12.0	2.0	2.0
LSU	Design	17.5	3.2	6.0
Kok	Actual	10.2	3.9	3.3
Kok	Actual	11.1	4.5	3.5
Kok	Actual	12.0	5.2	3.8
Kok	Actual	12.8	5.9	4.0
Kok	Actual	13.8	6.6	4.2
Kok	Actual	14.8	7.3	4.5
Kok	Actual	16.0	7.9	4.7
Kok	Actual	17.1	7.6	4.9

111. Figures 116 and 117 show plots of lateral erosion induced from wave-attack on the actual and design geometric EBSB configurations, respectively. These analyses do not include delays in breaching due to the EBSB grass cover; these results are for no grass cover. Based on these simulations, and the EBSB geometric configuration and materials, the EBSB crest is breached by wave side attack around 5:00 am CDT. After 5:30 am CDT, the magnitude of lateral erosion dramatically reduces at the outboard face because the outboard face is fully submerged by the storm surge (which occurs around 6:00 am CDT). At this stage, the erosion has shifted to the backside face.

112. A comparison of the results from the parametric analyses and results from the 'base cases' (Scenario 1 and Scenario 2C) analysis is presented in Figure 116. All three of the parametric Hurricane Katrina hydrodynamic conditions indicate that the EBSB would have been breached as a result of front side wave-induced erosion.

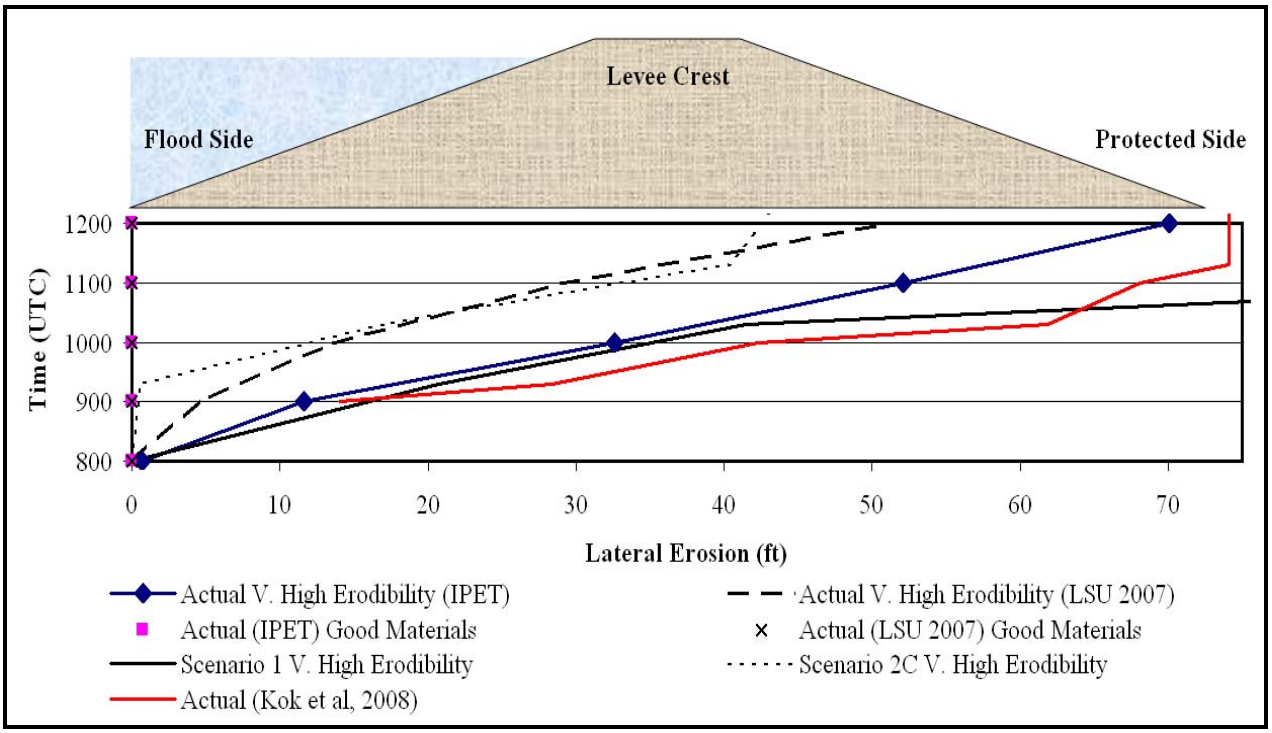


Figure 116: Comparison of lateral erosion for the actual EBSB configuration. Local CDT time is UTC time minus 5 hours.

113. Comparison of the parametric study results for the actual and design EBSB configurations (Figure 117) show comparable lateral erosion magnitudes with the design profile sustaining slightly greater front side wave attack erosion. This is due to the higher crest elevation of the design profile (+17.5 feet) and the resulting longer exposure of the front side to wave attack before overtopping.

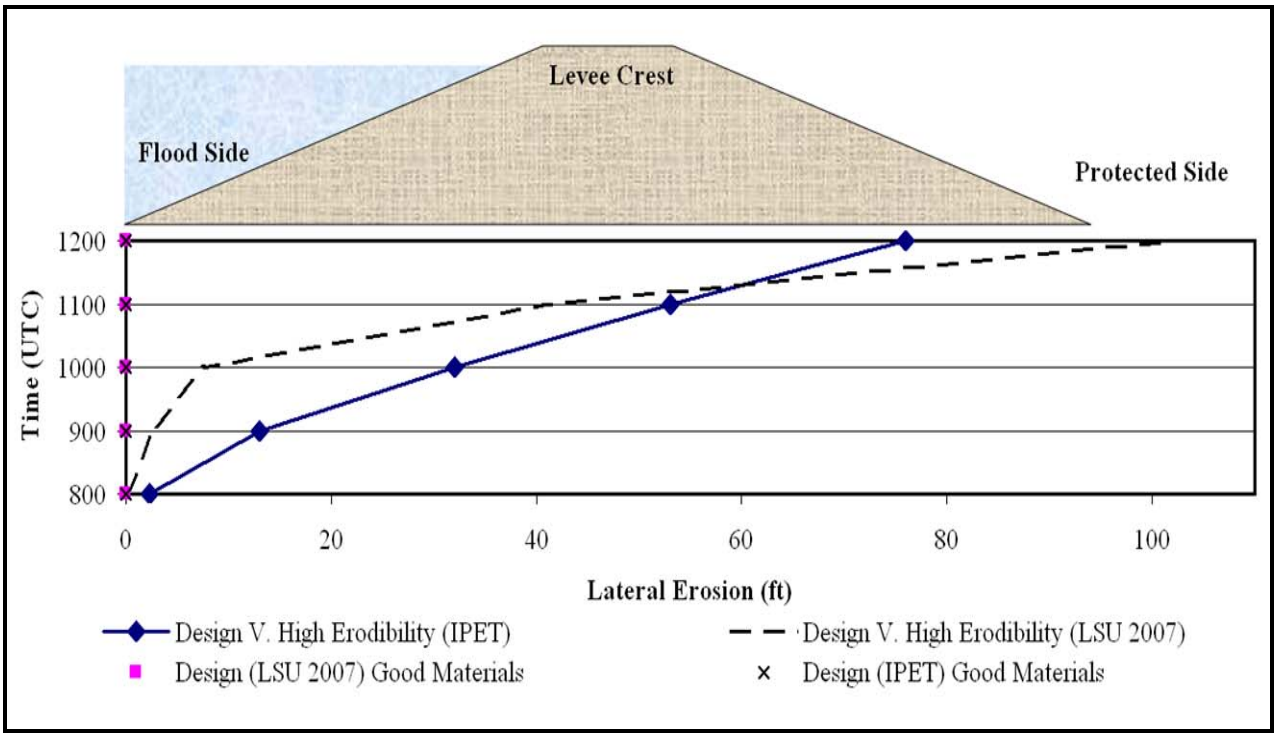


Figure 117: Comparison of lateral erosion for the design EBSB configuration. Local CDT time is UTC time minus 5 hours.

114. All of the parametric analyses confirm that levees constructed of 'good' materials (cohesive, compacted) with low erodibility would have performed significantly better than one constructed of high erodibility materials. Subjected to the same front side wave attacks, the levee with 'good' materials would have experienced insignificant front side erosion.

115. Erosion analyses were also performed accounting for current velocities parallel to the EBSB flood-face. These current velocities were generated as a result of water flow through the MR-GO and GIWW towards the IHNC and Lake Pontchartrain. The magnitude of the current velocity was on the order of 0.5 to 1 feet per second at the face of the EBSBs (Wit et al 2008). Figure 118 shows the result of inclusion of the current velocities. There was no significant impact of this velocity component.

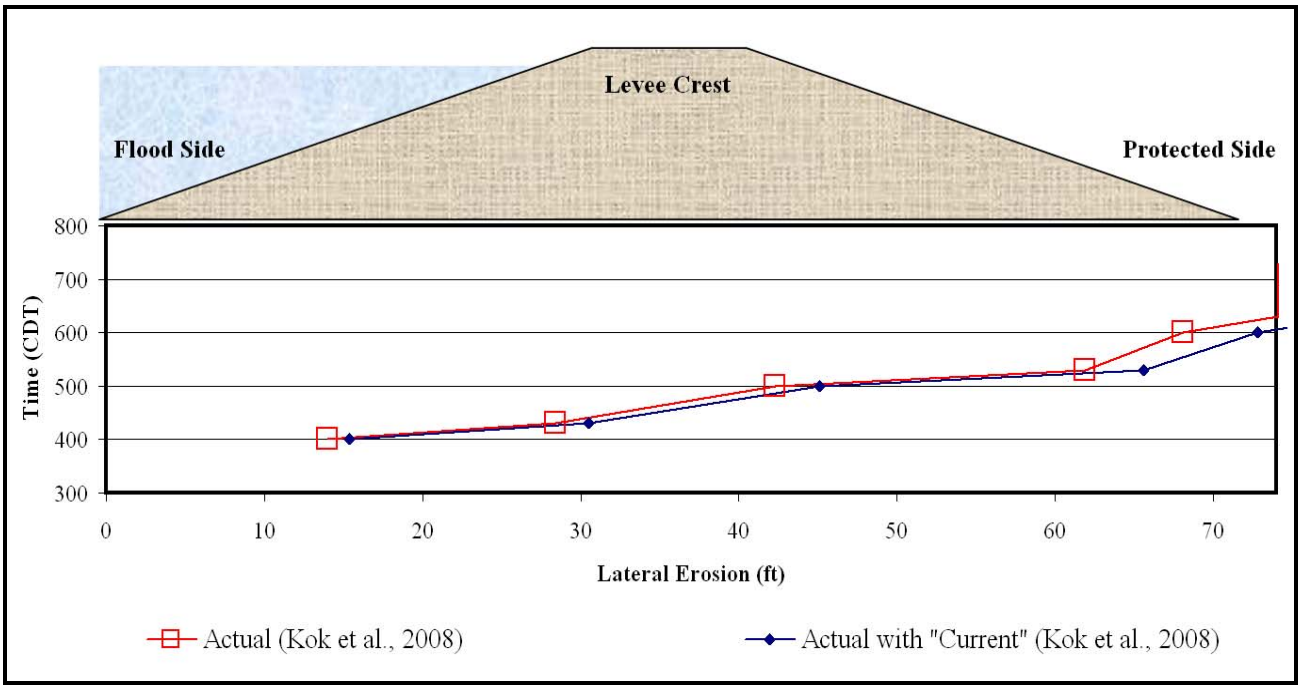


Figure 118: Comparison of lateral erosion for the actual EBSB configuration with and without storm surge currents for the Hurricane Katrina hydrodynamic conditions (Kok et al 2008; Wit et al 2008; Gautier et al 2008). The storm surge currents result in a negligible increase in the magnitude of lateral erosion.

116. Analyses were performed to evaluate wave-induced erosion on a properly designed and constructed Levee with the same geometric configuration and hydrodynamic loading as the EBSB study location (Bea and Storesund 2008). The erodibility characteristics of a low erodibility levee material were selected from the MR-GO Levees north of Bayou Bienvenue. The results showed that for all of the surge – wave – erosion analyses for the properly designed and constructed levee there is insignificant erosion for the Hurricane Katrina 'as was' conditions. These analytical results were in excellent agreement with the observations of man-made structures which had failed and which had not failed during Hurricane Katrina (ILIT 2006).

117. Based on these analyses, one can conclude that a levee constructed of 'good' materials with low erodibility would have performed significantly better than one constructed of

high erodibility materials. For all three sets of Hurricane Katrina hydrodynamic conditions, the levee constructed with 'good' compacted materials experienced insignificant erosion. The analyses show that the levee would have performed very well during overtopping conditions as well (Bea and Storesund 2008a). Results from the analyses are corroborated with observations of the performance of this section of the levee following Hurricane Katrina. The region (north of the Bayou Bienvenue Control Structure) where the soil samples used for the laboratory erosion testing did not experience any significant wave-induced erosion during Hurricane Katrina.

IPET and Ebersole EBSB Breaching Studies

118. IPET (2007) identified a four-stage progressive erosion development profile (Figure 119) for overtopping-induced erosion. This failure mechanism results in a 'ridge' on the flood side of the levee. Levees that experienced wave-induced erosion would have an intact protected side and scoured flood side.

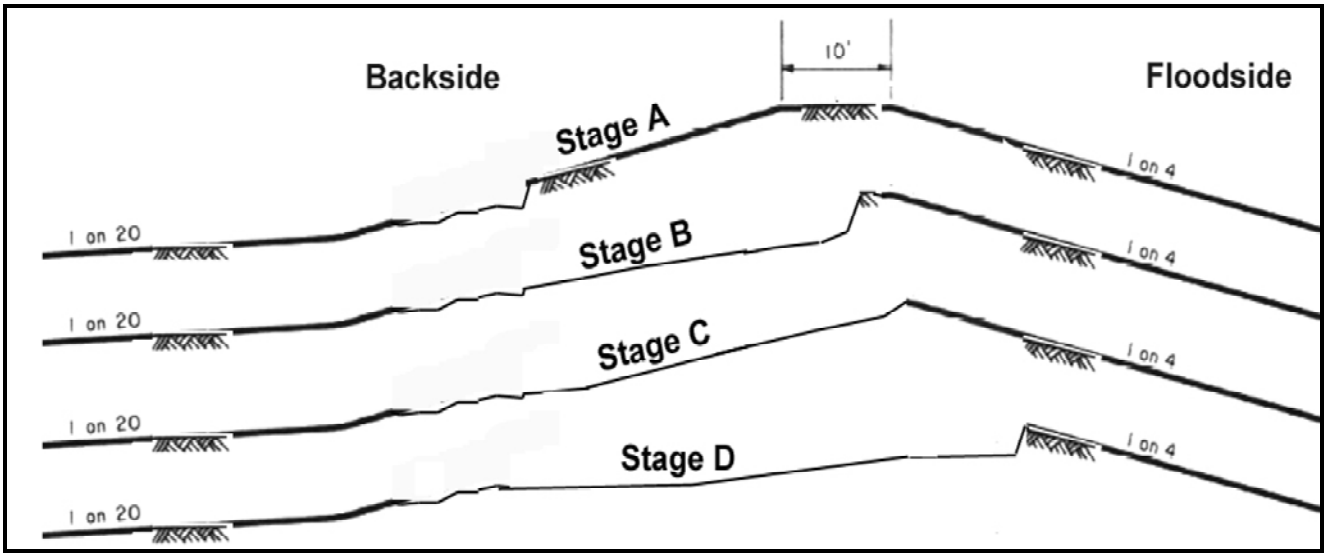


Figure 119: IPET (2007) provided an overtopping erosion development profile. Note the remains of a 'ridge' on the flood side of the levee following Stage D of overtopping induced erosion. This ridge is not present when erosion is initiated on the flood side of the levee.

119. Ebersole (2008) identified locations where flood side notching occurred (e.g. Figures 120-122). These and other photographs confirm wave induced erosion did occur on the MR-GO Levees (Ebersole 2008). The extent and prevalence of wave-induced erosion is still debatable since in many cases the indicative physical evidence has been 'washed' away by subsequent overtopping erosion. Evidence developed during this investigation clearly indicates wave-induced erosion is very likely for the Hurricane Katrina conditions and for EBSB sections that were poorly protected by grass and other natural vegetation defenses.

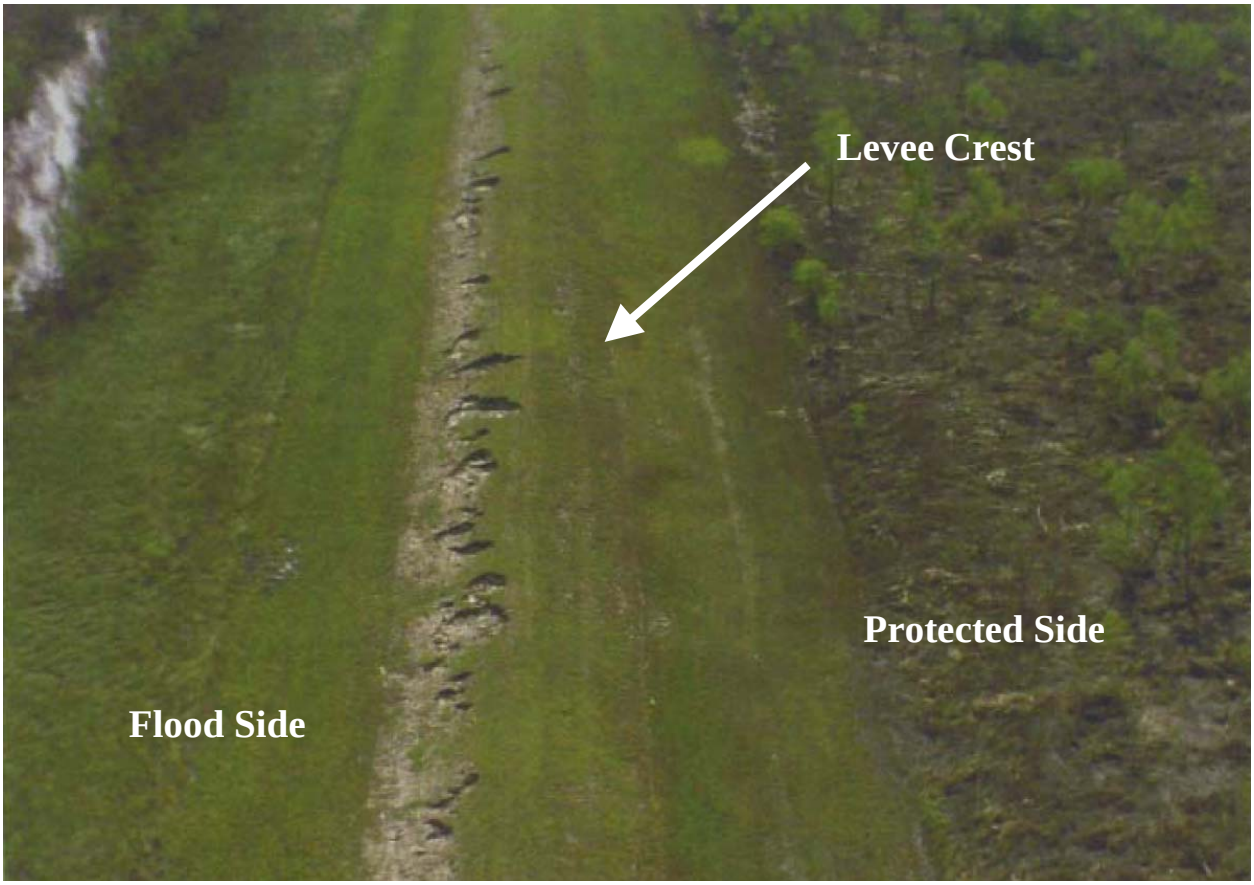


Figure 120: Ebersole (2008) identified locations where wave-induced erosion was observed with no overtopping-induced erosion. Note grass removed by wave action on flood side.

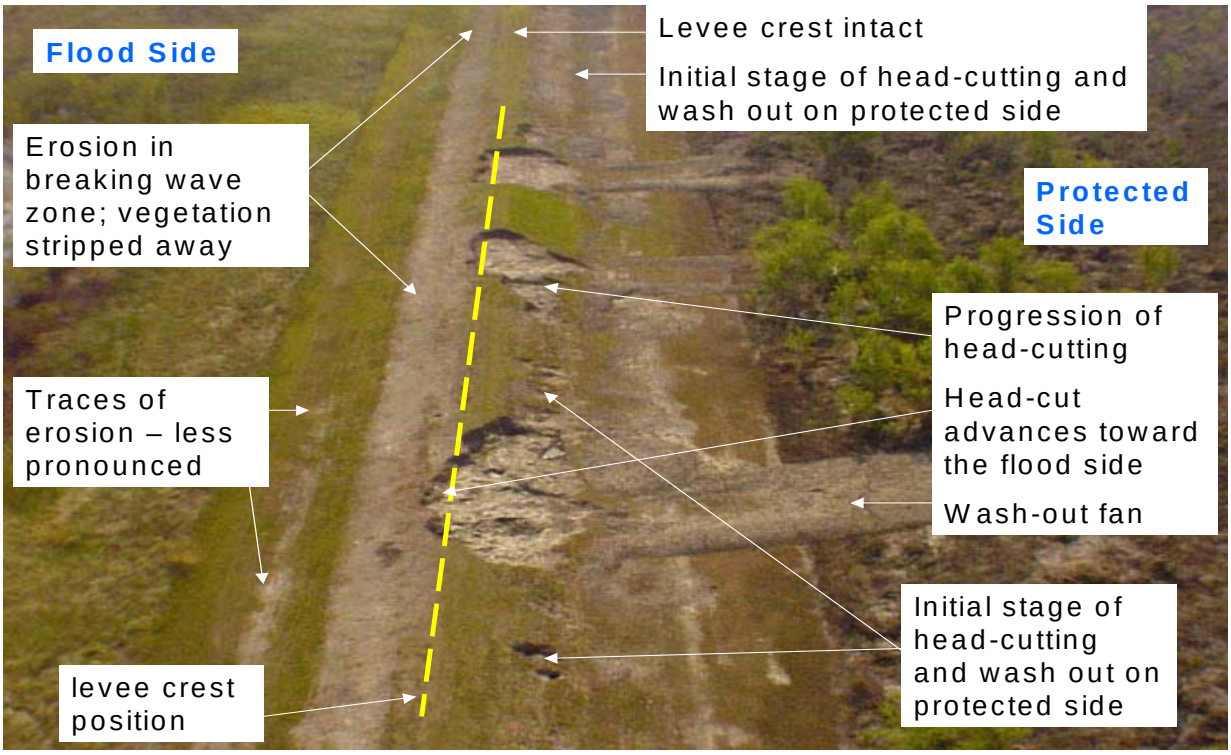


Figure 121: Ebersole (2008) identified locations where wave-induced erosion was observed with no overtopping-induced erosion.

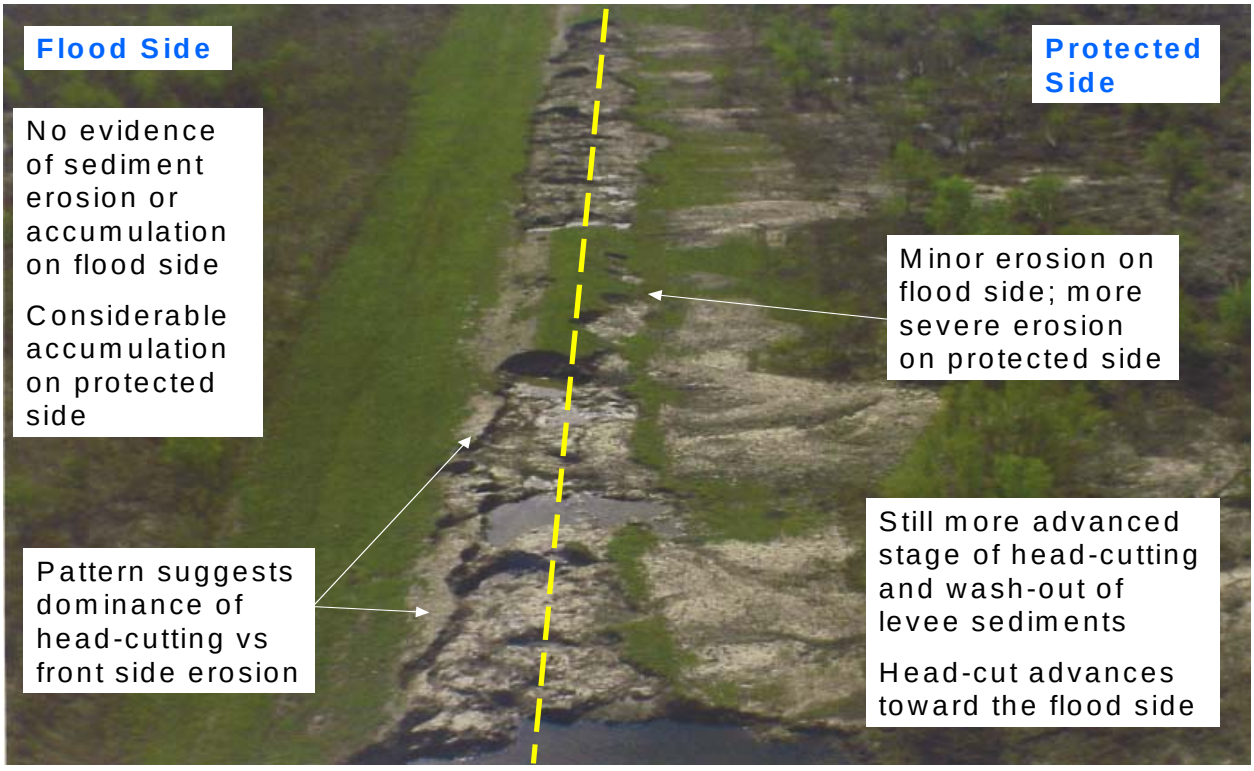


Figure 122: Ebersole (2008) identified locations where wave-induced erosion was observed with no overtopping-induced erosion.

120. Results from this study were compared with results presented by Ebersole (2008). As seen in Figure 123, there was evidence that wave-induced notching occurred without overtopping induced levee protected side erosion. Figure 123 clearly shows that even with overtopping conditions, the only 'damage' incurred to the levee at this (location not specified) was on the outboard face. The erosion profile also resembles the erosion profile shown in this Declaration.

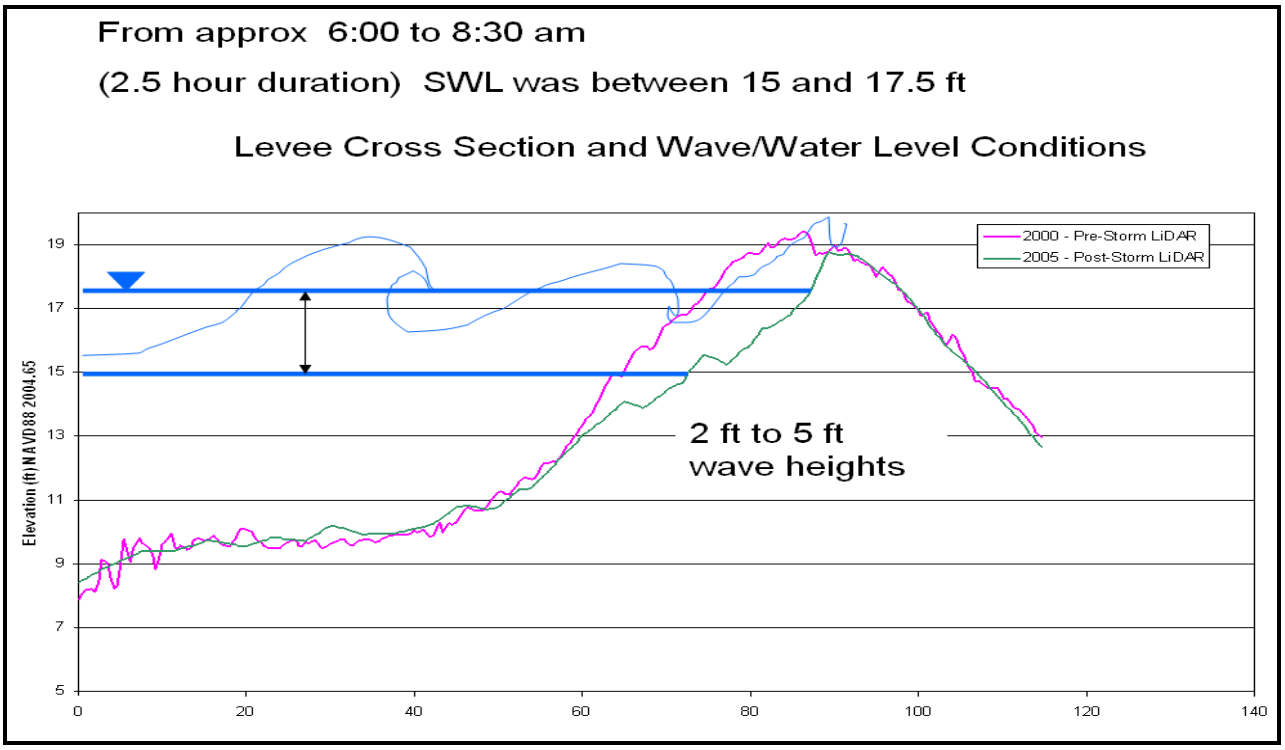


Figure 123: Pre and Post Aerial LiDAR are highlighted to show wave-induced notching of the levee (Ebersole 2008).

Conclusions and Observations from Analytical Results

121. Based on these analyses, it is concluded that for the Hurricane Katrina hydrodynamic conditions the EBSB study section breached through the crest solely as a result of wave-induced erosion acting on the front side of the EBSB. Wave induced breaching occurred before overtopping by Hurricane Katrina's storm surge. By 6:00 to 7:00 am CDT the storm surge was above the levee crest resulting in active overtopping of the EBSB and further developing the breaches developed by wave side attack. In this process, the overtopping back side erosion would erase the wave side attack features, thus giving the false impression that the breach had developed solely because of overtopping.

Assessment of MR-GO Reach 2 EBSB Breaching

122. Locations along the MR-GO Reach 2 EBSB locations where scour and erosion occurred outboard of the EBSB centerline were identified as probable wave-induced erosion breaching sites. Examination of the 2005 aerial LiDAR surveys was conducted to identify these areas. Based on this review, probable wave-induced and overtopping exploited erosion sites were identified (Figure 124). Characteristics used to identify wave-induced erosion sites included regions with significant erosion but lack of a flood side toe ridge which would occur as a result of overtopping induced erosion creating a failure zone that propagates from the protected side of the EBSB to towards the flood side toe, areas with visible flood side notching, and zones with wider erosion extents on the flood side as compared with the protected side.

123. Study of pre- and post- Hurricane Katrina LiDAR surveys that delineated breach features (Morris 2008, Ebersole 2008) coupled with results from analyses of the performance of the man-hade hurricane flood protection structures during hurricane Katrina indicate that approximately 35 percent of the EBSBs along the Reach 2 alignment were breached due to

attack by waves, 47 percent were breached by a combination of wave attack and surge overtopping – backside erosion (including sheet pile repair breaches), and 18 percent were overtopped but did not breach (sheet pile repair sections, Dupre and Bienvenue navigation structures, some segments of the EBSBs) (Figure 124). Analyses of the breaching that developed at the Bayou Dupre and Bayou Bienvenue navigation structure wing walls were determined to have been caused by overtopping erosion (section north of Dupre structure wing wall to EBSB connection) and by the combination of seepage, uplift and surge lateral forces (section south of Bienvenue structure wing wall to EBSB connection) (Bea 2008a, Bea and Storesund 2008a, Bea and Cobos-Roa 2008a).

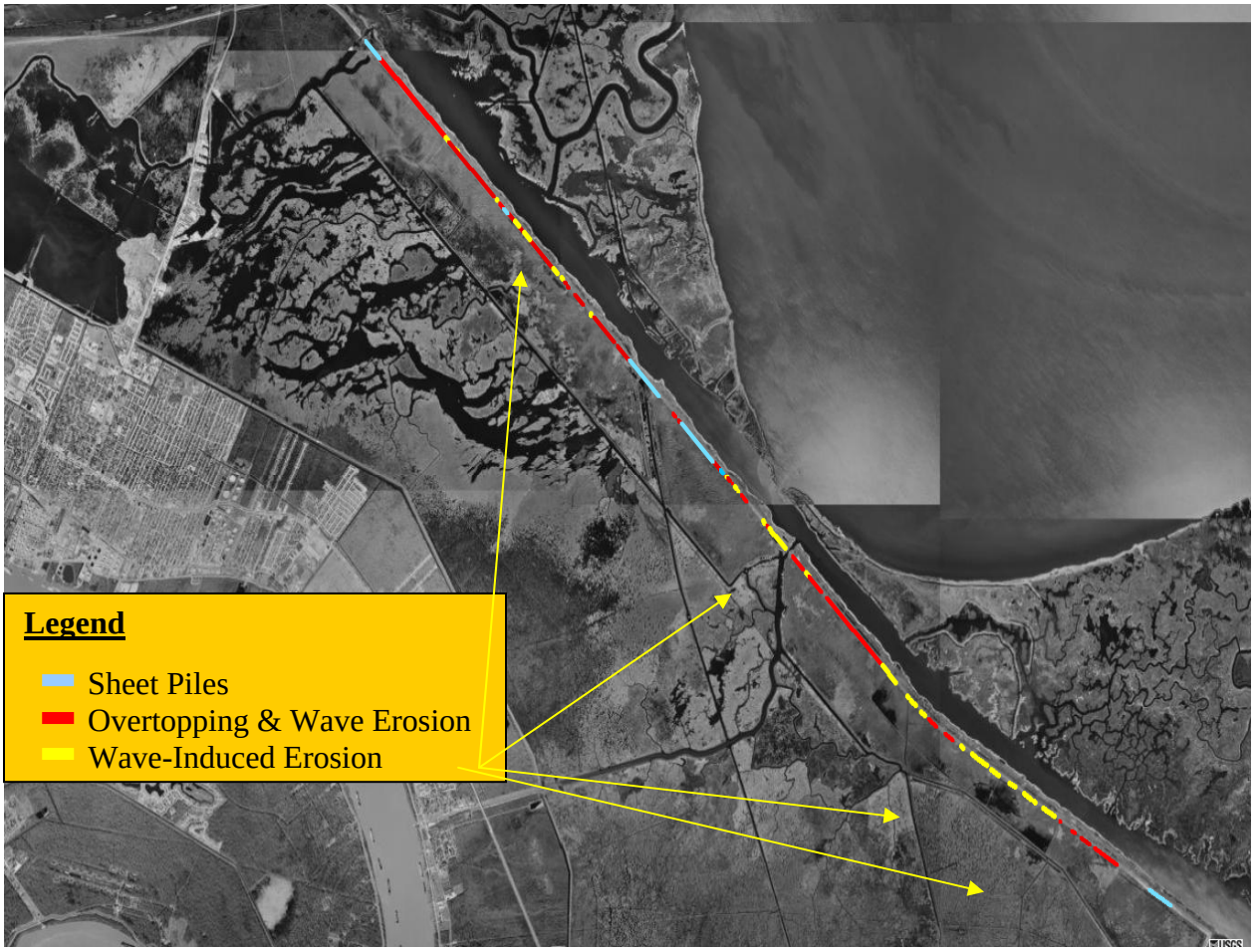


Figure 124: Identification of wave-induced breach sites based on visual examination of 2005 aerial LiDAR data (Bea 2008a, Bea and Storesund 2008a).

BAYOU BIENVENUE NAVIGATION STRUCTURE BREACH

124. The navigation structure at Bayou Bienvenue (Station 360+00) ties into the EBSBs to the north and south with concrete I-walls with top elevation of approximately +17.5 feet (Figure 125).

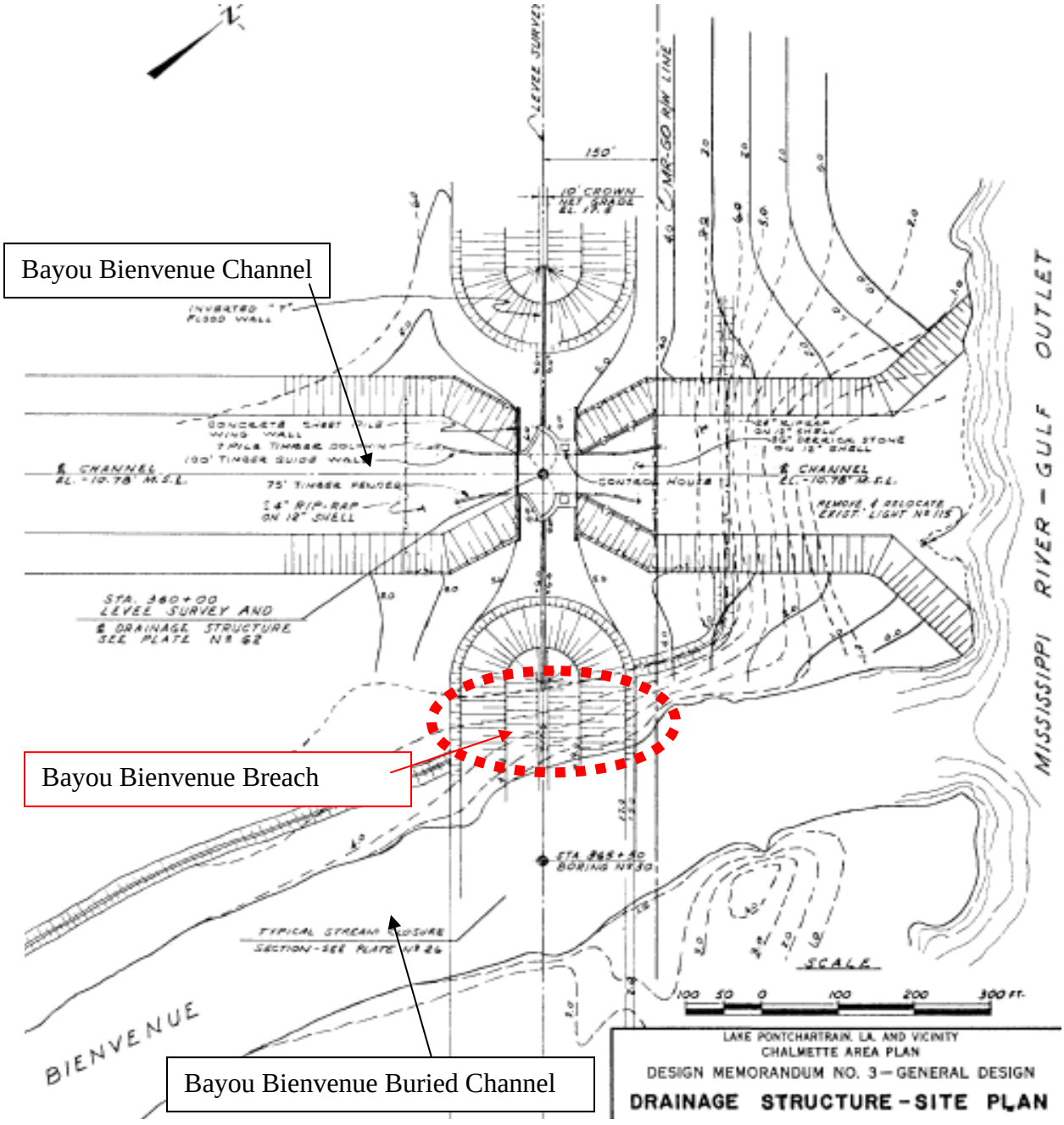


Figure 125: Bayou Bienvenue Control Structure. Source: Plate 60, DM03 – Chalmette Area Plan, 1966 (Bea and Cobos-Roa 2008a).

125. The T-wall connecting the navigation structure locks and the EBSB is founded on timber piles that are 88 feet long. The piles next to the T-wall are steel sheet piles driven down to elevation -5.2 feet (NAVD88). As shown in Figure 126, the bottom of the sheet pile (elevation -5.2 feet) did not extend through the shell fill placed over the original bayou channel. This left a "window" of permeable material of about 5 feet in thickness for under seepage to develop.

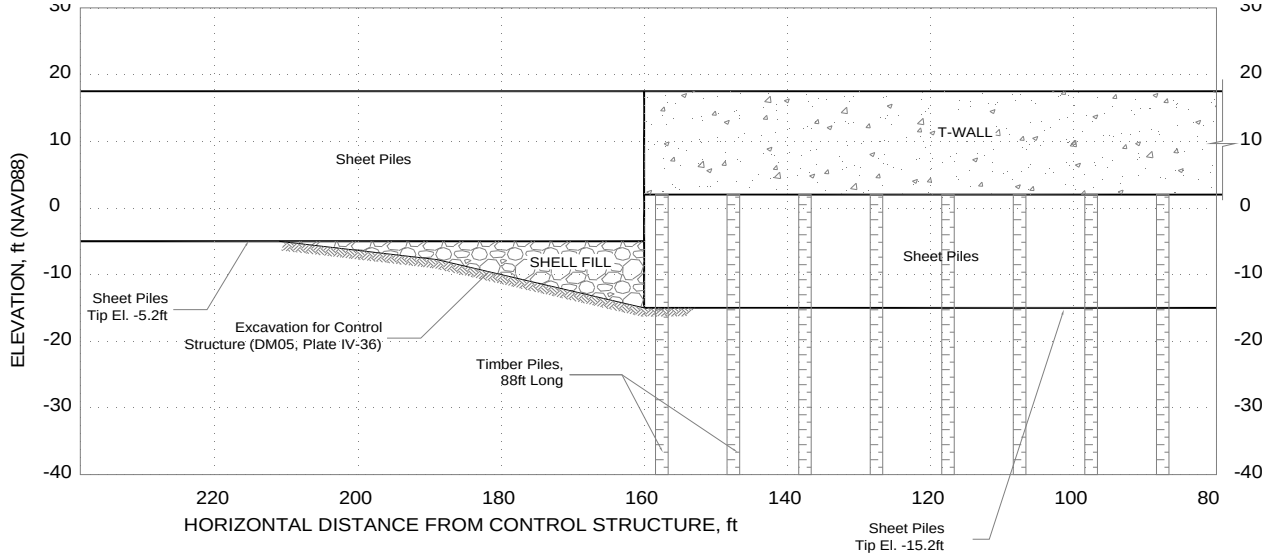


Figure 126: Superimposed sections showing gap between shell materials and bottom of sheet pile (Bea and Cobos-Roa 2008).

126. The stratigraphic interpretation for the Bayou Bienvenue breach location was based on borings G-29, G-30 and G-31 (1966), 1-CHBB and 2-CHBB (1987), B-1U through B-5 (2005), and U-2 (1966). The levee embankment is composed mainly by semi-compacted clays, underlain by a Marsh deposit, described as organic matter, organic clay, wood, roots and saturated-very low density material. To minimize weight imposed on the soft subsoils, shell materials were placed to fill the excavations for the navigation structures. Based on the available boring data, samples that contained organic materials with high water contents, low density and low strengths were classified as Marsh samples. Figure 127 shows the longitudinal profile created for the Bayou Bienvenue area. Underlying the marsh deposit, a thick "fat" clay deposit

occurs from elevation –30 feet to approximately –40 feet, located atop a sand deposit. The sands are in turn underlain by another clay and silt deposit, probably associated with Pleistocene age deposits.

127. The software SEEP\W, part of the package GEOSTUDIO 2004 was used for seepage modeling. The program uses a 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). Figure 128 shows the finite element analysis mesh used in SEEP\W and Table 3 summarizes the hydraulic properties of the soil layers, including conductivity at saturation and anisotropy ratio (ratio of vertical to horizontal permeability, ky / kx).

128. Transient analyses were performed for the rising surge water level on the flood side. The surge hydrograph was input to the seepage models, which generated calculation steps beginning at a water elevation +2 feet, believed to have been established before the storm surge occurred, thus having developed a steady state regime prior to the water rise.

129. 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. The values shown in Table 3 correspond to the free field case.

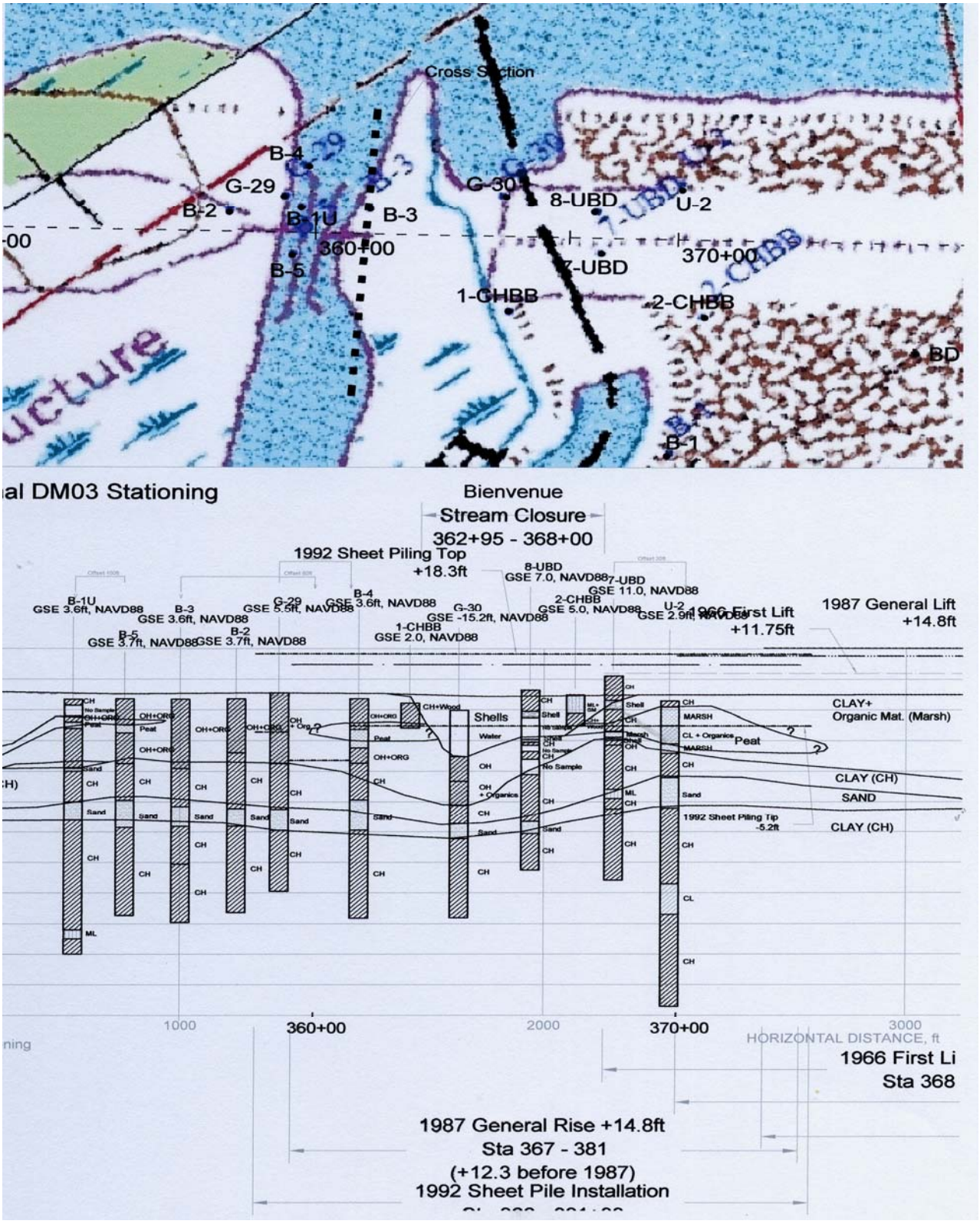


Figure 127: Geologic interpretation near the breach area (Bea and Cobos-Roa 2008).

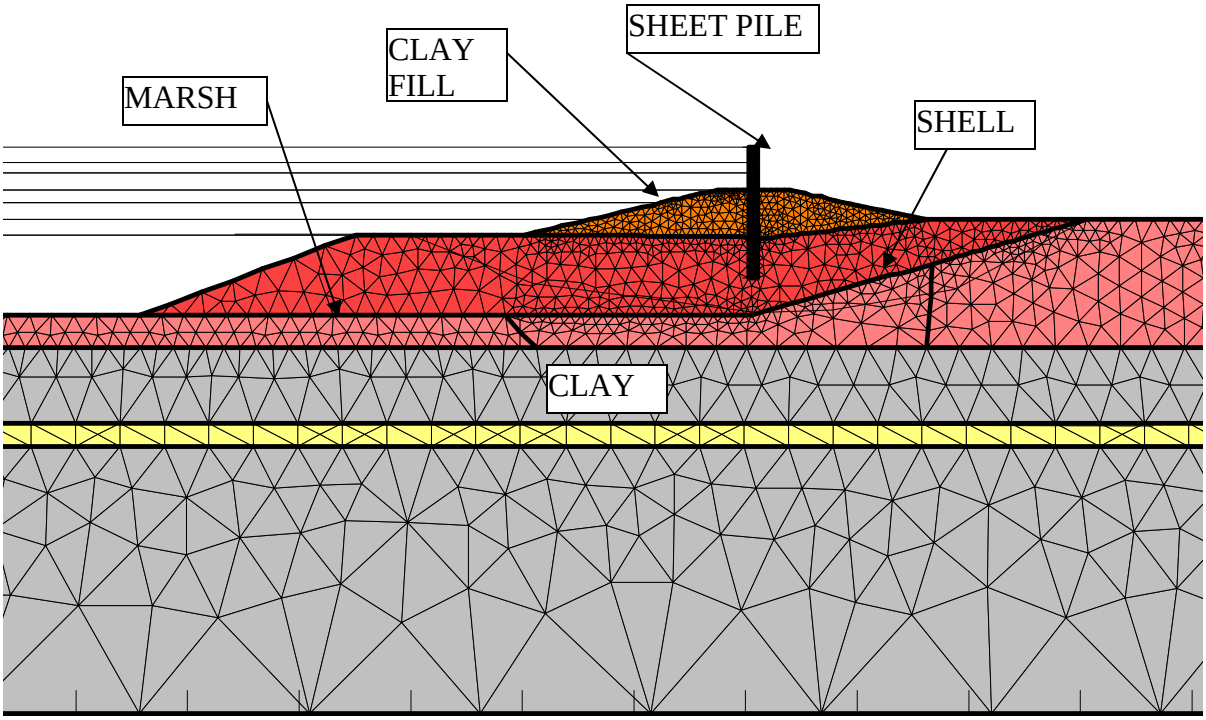


Figure 128: Cross Section and Finite Element Mesh used for the seepage analyses at Bayou Bienvenue (Bea and Cobos-Roa 2008).

Table 3: Material properties (free field) 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^{-3}	0.10
Marsh (lower bound)	10^{-6}	0.10
Shell Fills	10^{-2}	1.0

130. Results of the analyses show that after 33 hours of surge (August 28, 2005 0:00 am to August 29, 2005, 9:00 am CDT, high pore pressures near a steady state condition developed in the Marsh and Shell deposits (Figure 129). The transient models start from the low pre-storm generated steady state at 2 feet, generated by August 28, 2005 6:00 am hours, and in 16 time increments, elevation +18.0 feet (NAVD88) is reached on August 29, 2005 8:30 am

(CDT). Figures 129 and 130 show the calculated phreatic (water table) surface, total water heads and velocity vectors on August 29, 2005 8:30 am (CDT). It can be seen that the levee is not fully saturated, but the water level exits at the toe of the fill materials. The large arrows indicate very high flow velocities concentrated at the toe of the levee triggering blow-out of the shell materials.

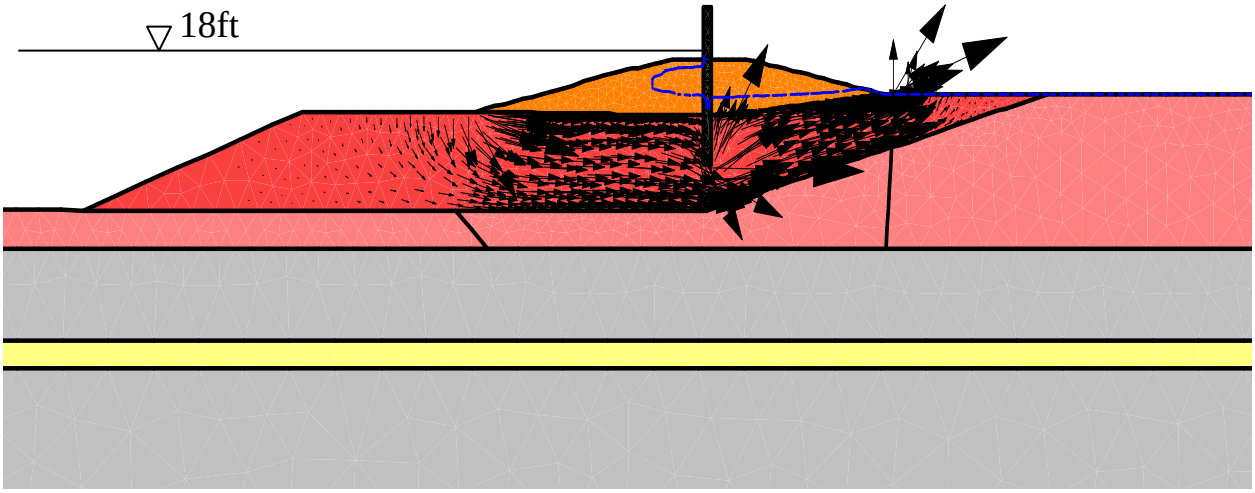


Figure 129: Results from the transient seepage analyses (August 29, 2005 8:30 am CDT) (Bea and Cobos-Roa 2008).

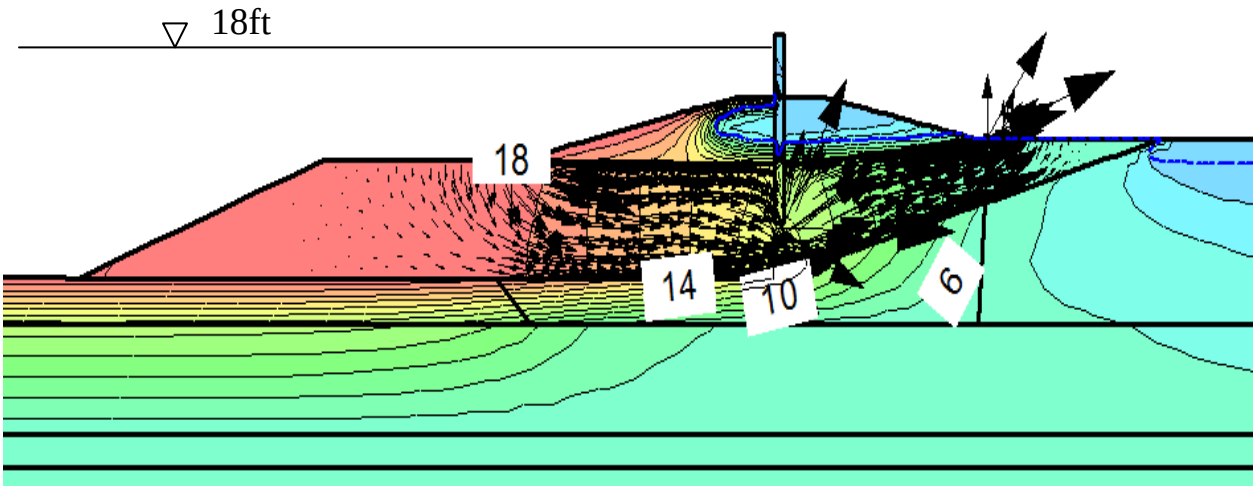


Figure 130: Total water heads on August 29, 2005 8:30 am CDT (Bea and Cobos-Roa 2008).

131. Slope stability analyses were performed for the Bayou Bienvenue breach that incorporated the hydraulic response of the levee. The software SLOPE\W was employed for this task. The pore pressures generated by the SEEP\W finite element analyses are used by SLOPE\W when calculating Factors of Safety. The software uses Limit Equilibrium Analysis to calculate Factors of Safety by a number of methods. The critical failure surface search resulted in shallow non-circular surfaces, passing through shell layers for Bayou Bienvenue.

132. Soil properties determined for unit weight, undrained shear strengths and friction angles were assigned to the different layers according to the stratigraphic interpretation discussed earlier and detailed in the technical report that supports this Declaration (Bea and Cabos-Roa 2008) (Table 4). The available borings are 'toe' borings; consequently, no soil shear strengths under the levee embankment are available. Based on recent reports (IPET, 2006; ILIT, 2006), the difference between shear strengths between the two conditions ranges between 30% and 60%. A 40% increase was applied to the Marsh beneath the levee embankment. Figure 131 shows the input geometry used for the stability analyses of the breach.

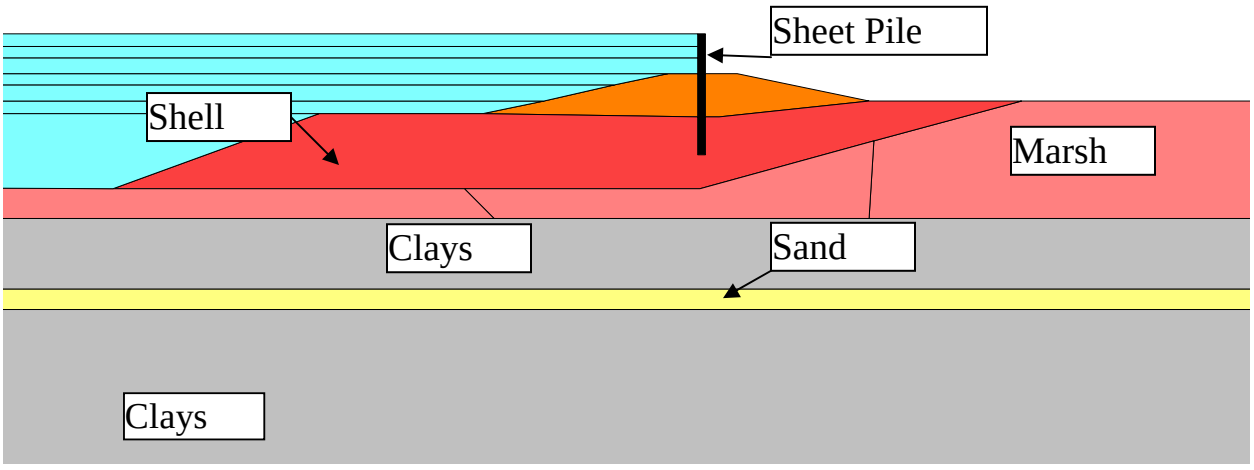


Figure 131: Analysis cross section for Bayou Bienvenue (Bea and Cobos-Roa 2008).

Table 4: Material properties used for analysis of lateral stability at Bayou Bienvenue.

Material	Shear Strength Parameter			
	Su (psf)	ϕ (deg)	Su/p'	γ (pcf)
Clays (CL, CH)	110			105
Shell (anthropogenic)[1]		30		105
Marsh (Peat)	142 (toe)			100
Grey Clay (CH)			0.26	111
Sand		30		120
Clays (CH)			0.26	120

133. Figure 132 shows the "mean (average) values" of results for factor of safety for the Bayou Bienvenue wing-wall tie-in section to the EBSB. Mean (average) values are used for all of the parameters used in the analyses. Results from the analyses show that this section reaches a Factor of Safety of unity (FS = 1.0, probability of failure Pf = 50 percent) when the water reaches the sheet pile top (elevation +18 feet). These results indicate that the entire section including the sheet piles most likely did not fail until overtopping had occurred. This analytical result is corroborated by the observed condition of the sheet piles at the breach (bent inward toward the protected side).

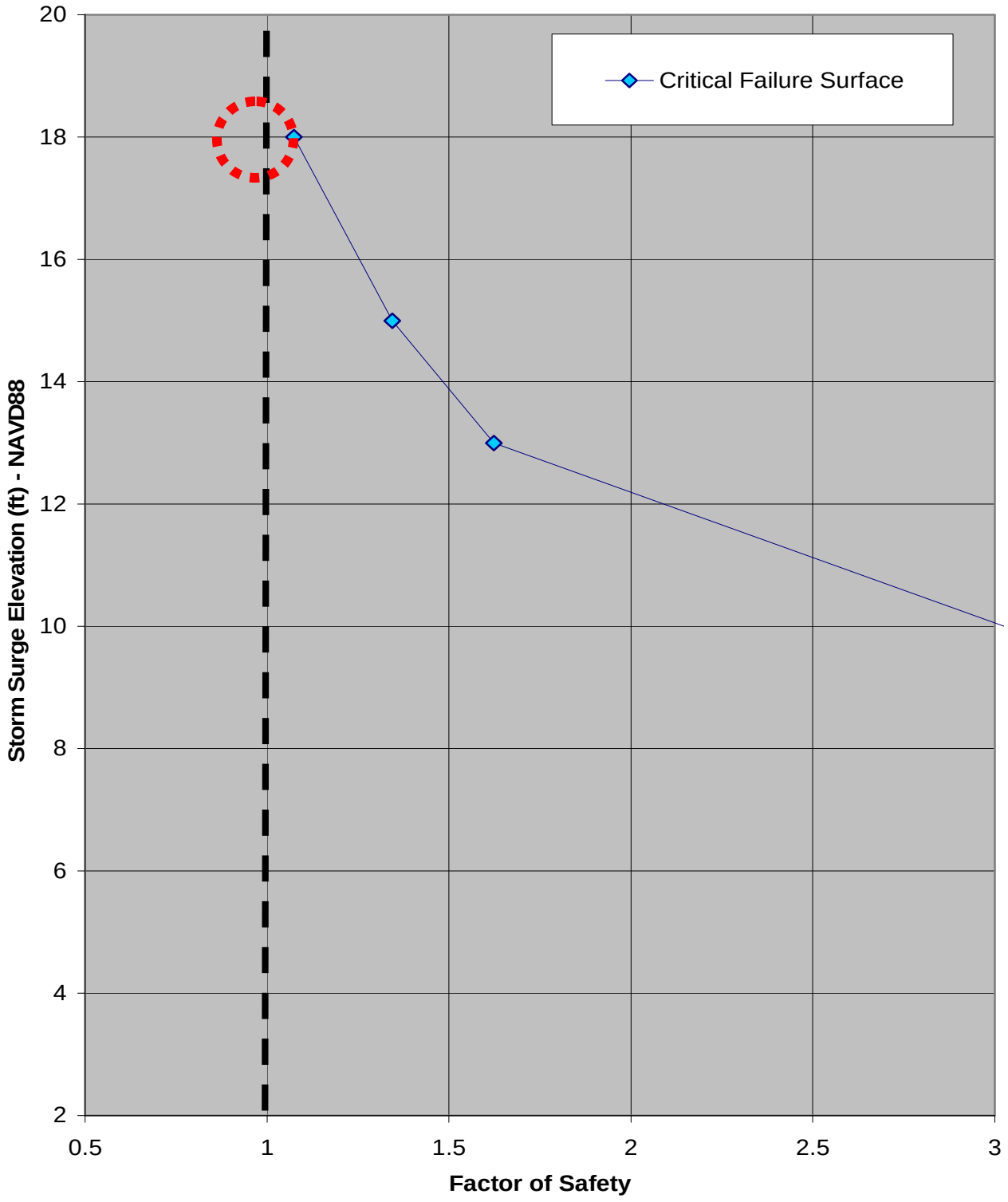


Figure 132: Factor of Safety versus storm surge elevation for Bayou Bienvenue breach (Bea and Cobos-Roa 2008).

BAYOU DUPRE NAVIGATION STRUCTURE BREACH

134. The control structure at Bayou Dupre (Figure 133) keys into the adjacent EBSBs with concrete sheet piles topped at elevation 17.5 feet. On the North side, the EBSB which the control structure ties into is approximately 3 feet taller than the top of the concrete structure.

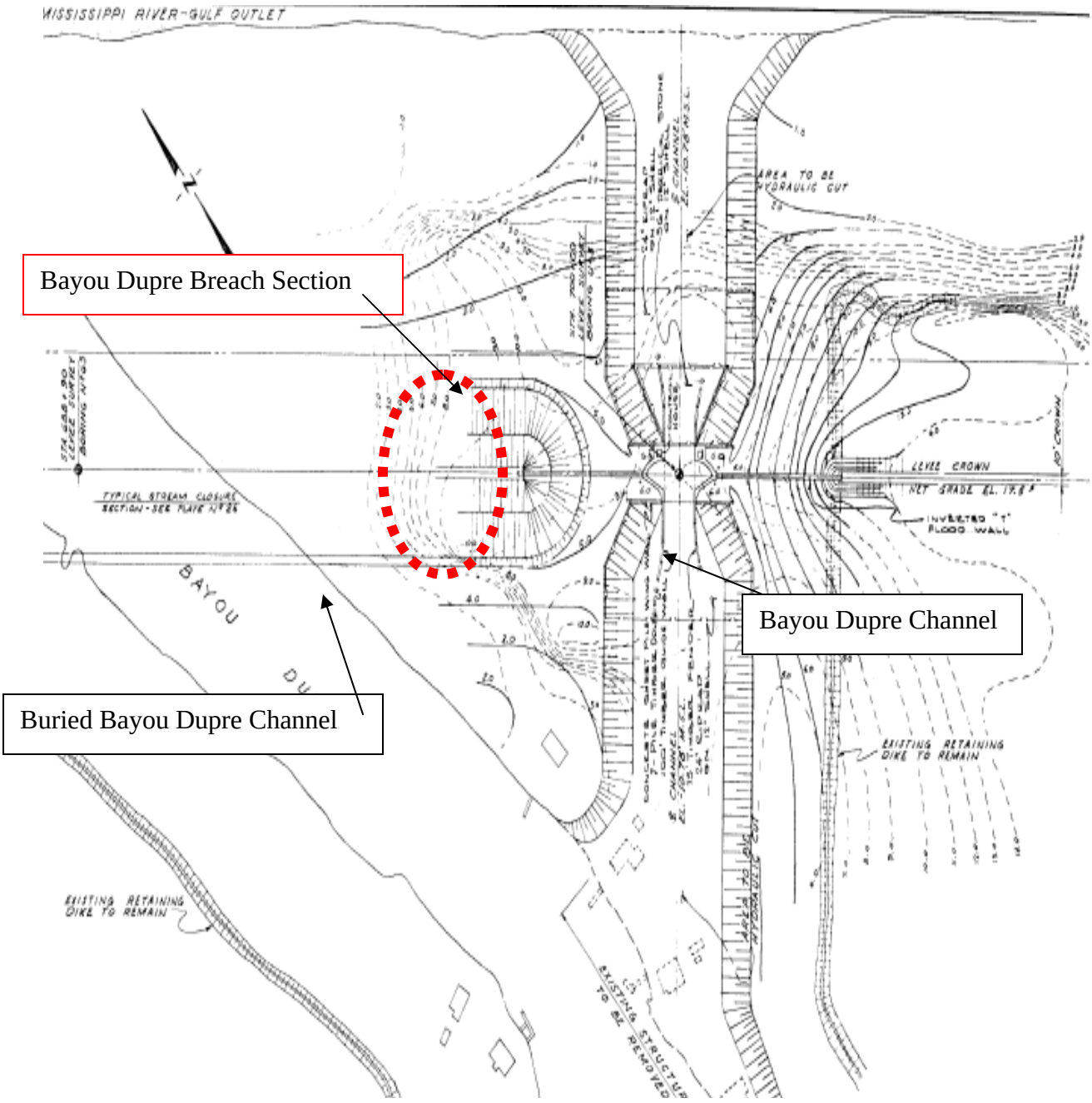


Figure 133: Bayou Dupre Control Structure. Source: Plate 61, DM03 – Chalmette Area Plan, 1966 (Bea and Cobos-Roa 2008).

The main breach on this site took place just north of the control structure; at the interface between the concrete structure and the connecting earthen levee (Figure 134).

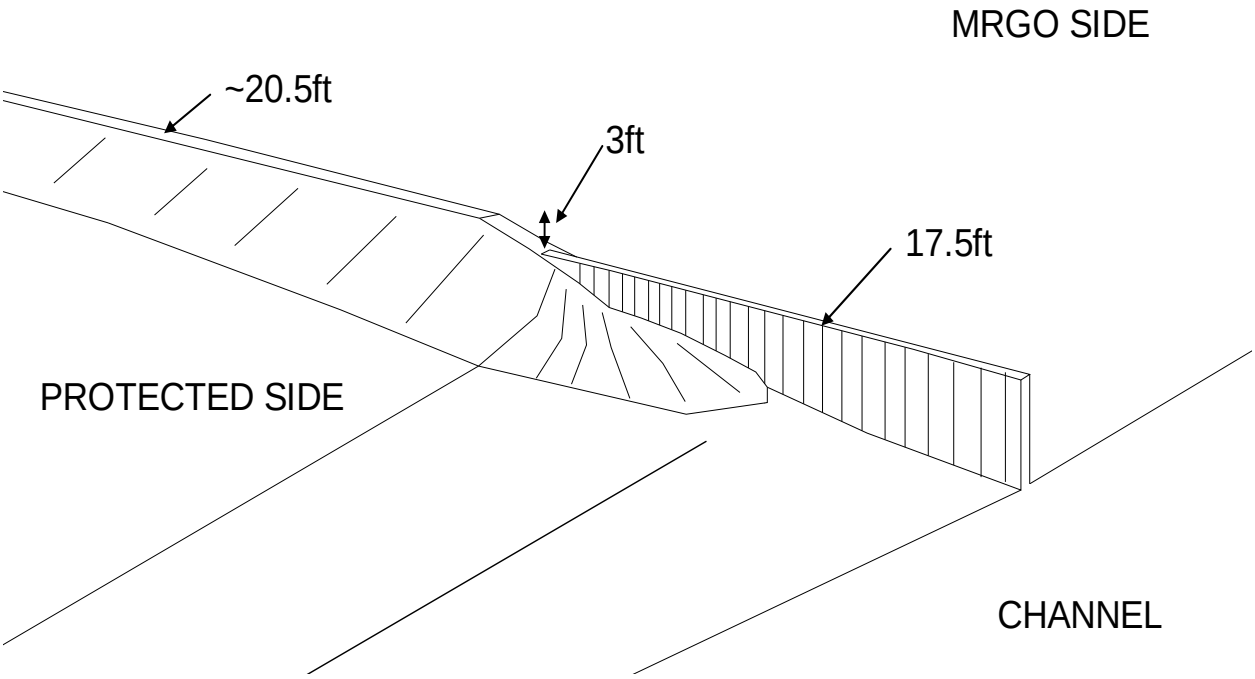


Figure 134: Three dimensional view of Bayou Dupre at the breach site. Note the 3foot difference between the concrete piles and the earthen embankment (Bea and Cobos-Roa 2008).

135. The stratigraphic interpretation was based on borings G-63, G-64 (1966), 14-CHBB, 17-CHBB, 18-CHBB, 15-CHBB, 16-CHBB (1987), 15-UBD (1984), D-1 through D-4 (1967), and MR-GO 27 (2005). According to the design documents, the levee embankment is composed mainly by semi-compacted clays. However, photographic evidence of the soils present in the shoulders of the breach clearly indicate that the EBSB was in fact composed of shells and mixed finer materials (Figure 135). The embankment is underlain by a Marsh deposit, described as organic matter, organic clay, wood, roots and saturated-very low density material.



Figure 135: Shoulder of Bayou Dupre breach showing coarse grained soils and shells (Bea 2005).

136. Given the poor descriptions presented in the borings from DM03 and the IPET (2006) report, samples that contained organic materials with high water contents, low density and low strengths were classified as Marsh samples. Figure 136 shows the longitudinal profile created for the Bayou Dupre breach section.

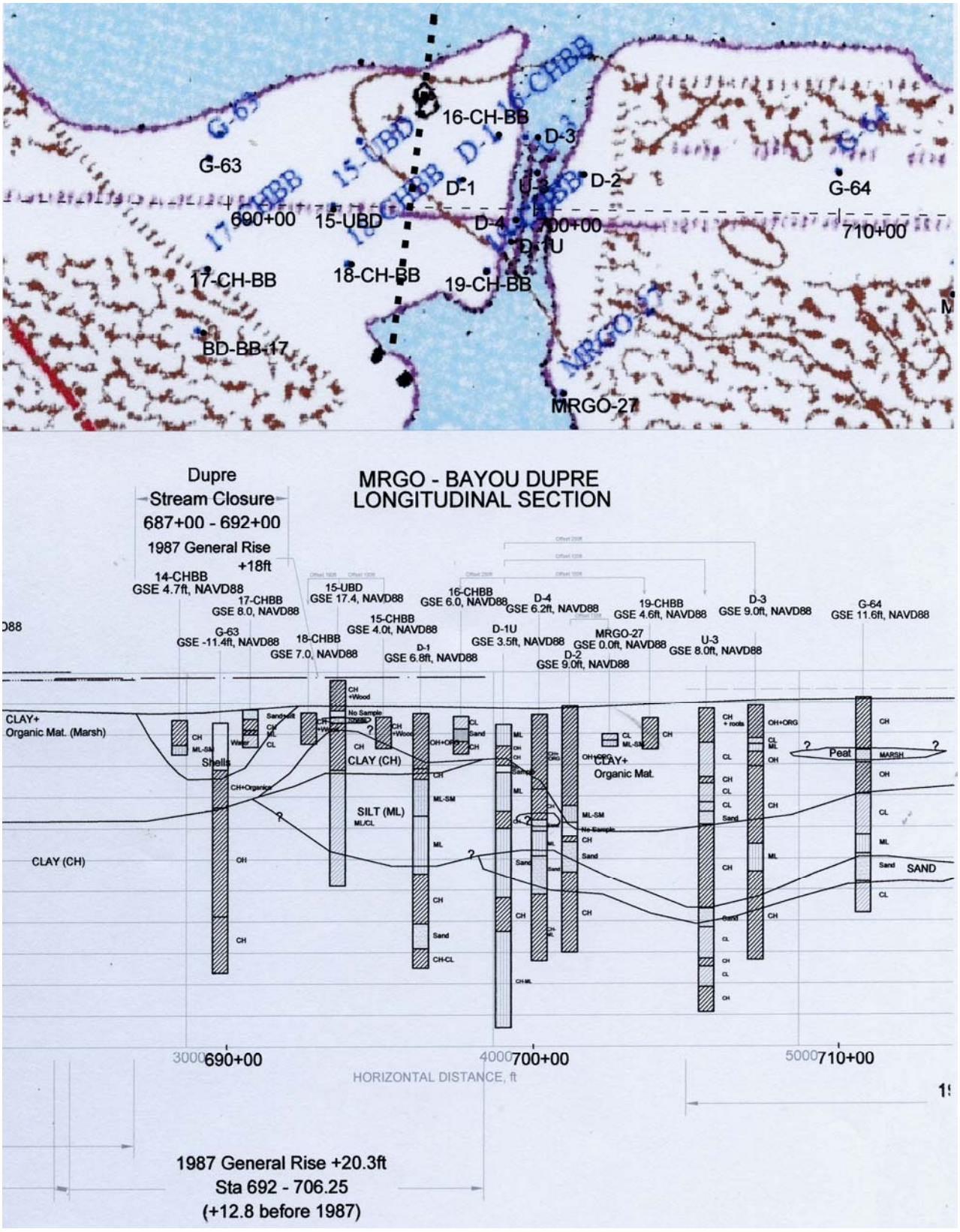


Figure 136: Stratigraphic profile for Bayou Dupre breach section (Bea and Cobos-Roa 2008).

137. Based on the material properties summarized in Table 4, baseline steady state seepage analyses were performed using a water elevation of +18.0 feet (NAVD88). These elevations were taken as reference of maximum development of pore pressures. Figure 137 shows the cross section and mesh used for the analyses in this section.

138. From Figures 138 and 139 it can be observed that the flow regime is dominated by the presence of the permeable shell materials, inside and below the embankment. Marsh has a smaller influence on the hydraulic behavior of the system. The black arrows represent velocity vectors of subsurface flow, with the largest arrow with a value of over 20 feet per day. The results summarized in Figure 138 are based on a Marsh deposit horizontal permeability of 10^{-6} cm/s. The results summarized in Figure 139 are based on a horizontal permeability of 10^{-3} for the same layer.

139. It is clearly seen that when the Marsh is treated as a clay (Figure 138), there is a substantial head loss before the centerline of the levee is reached; whereas the head loss is smaller and more uniform when the marsh is assigned a higher permeability, therefore making heads and pore pressures larger in this stratum. The concentrated black arrows run inside the shell deposits.

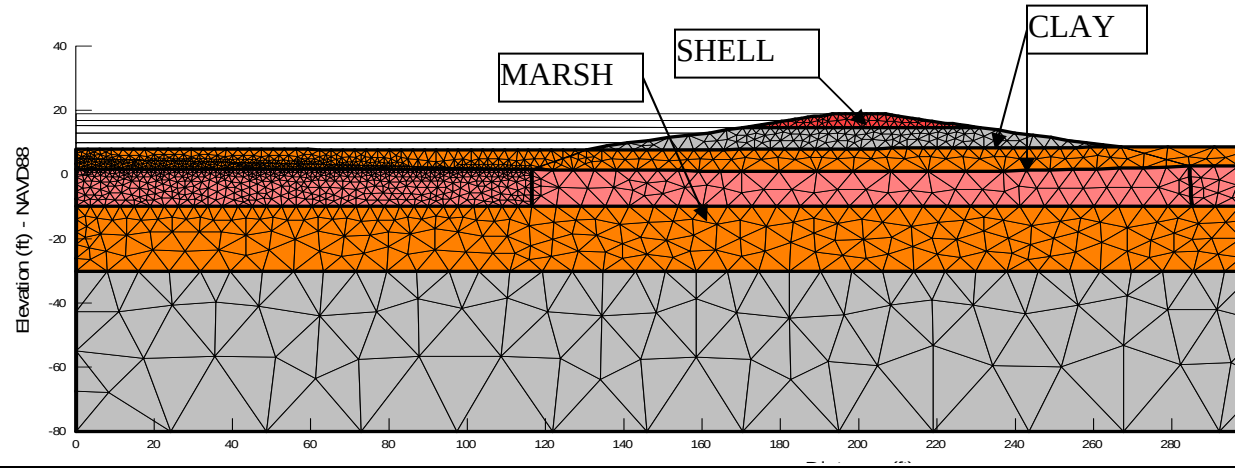


Figure 137: Cross section and mesh used for the analyses at Bayou Dupre (Bea and Cobos-Roa 2008).

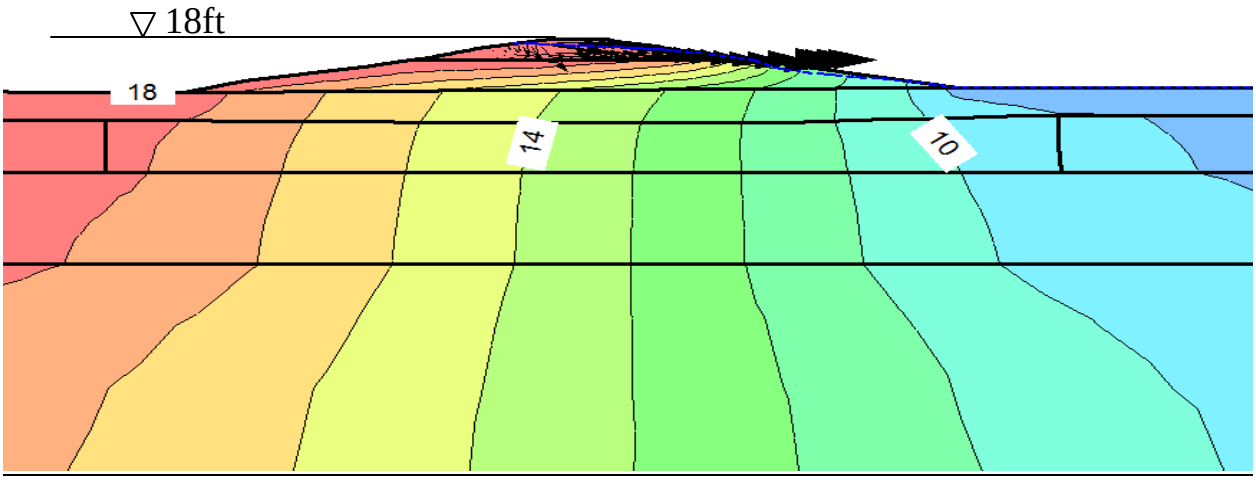


Figure 138: Steady State Total Heads at Bayou Dupre (Bea and Cobos-Roa 2008). Storm surge at +18 feet. Marsh k: 10^{-6} cm/s.

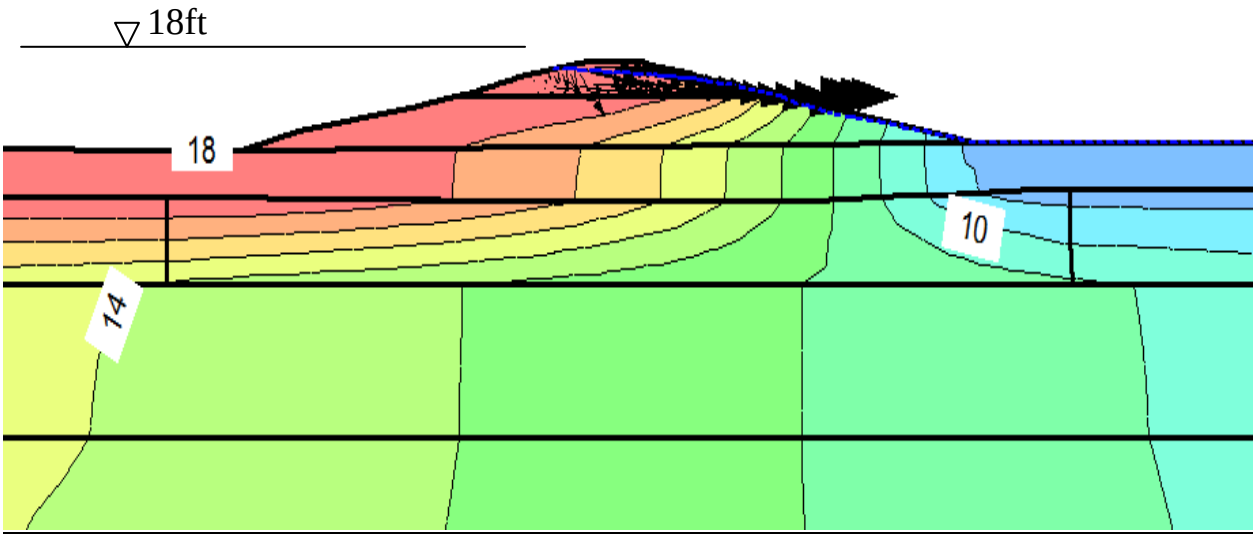


Figure 139: Steady State Total Heads at Bayou Dupre (Bea and Cobos-Roa 2008). Storm surge at +18 feet. Marsh k: 10^{-3} cm/s.

140. Figure 140 shows a plan view of the seepage analysis results, showing the top of levee composed of shell materials, and the transition to the sheet pile. Note the concentrated flow around the sheet pile, which is sufficient to start erosion and scour around the structure.

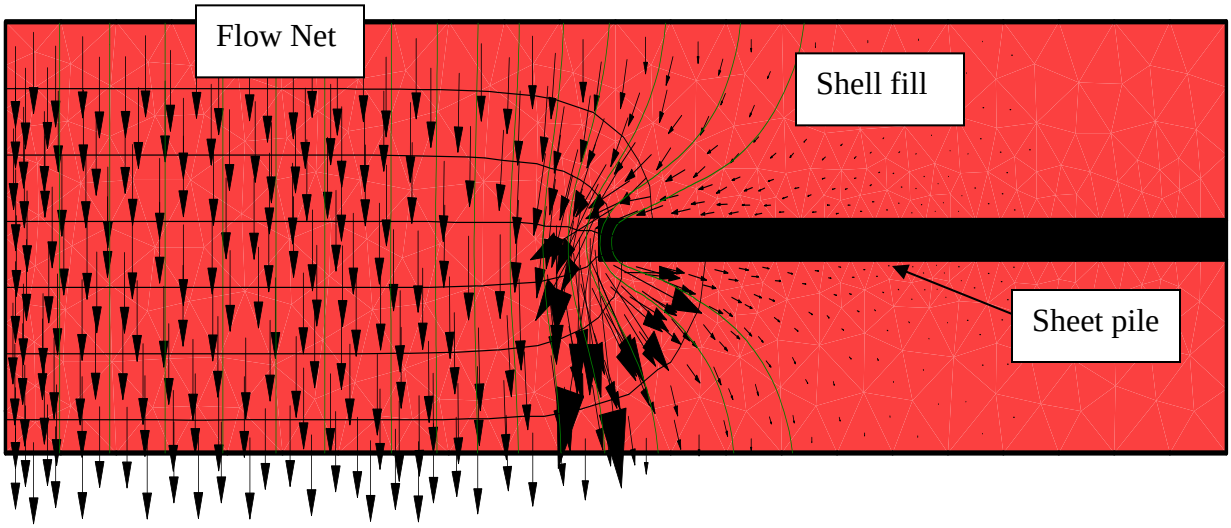


Figure 140: Plan view showing effects of overtopping in the transition (Bea and Cobos-Roa 2008).

141. Figures 141 and 142 show the transient model results on August 29, 2005 at 8:30 am (CDT), in terms of total heads and hydraulic gradients for the case of Marsh hydraulic conductivity of 10^{-3} cm/s. The hydraulic heads and gradients are sufficient to initiate significant scour and blow-outs leading to breaching.

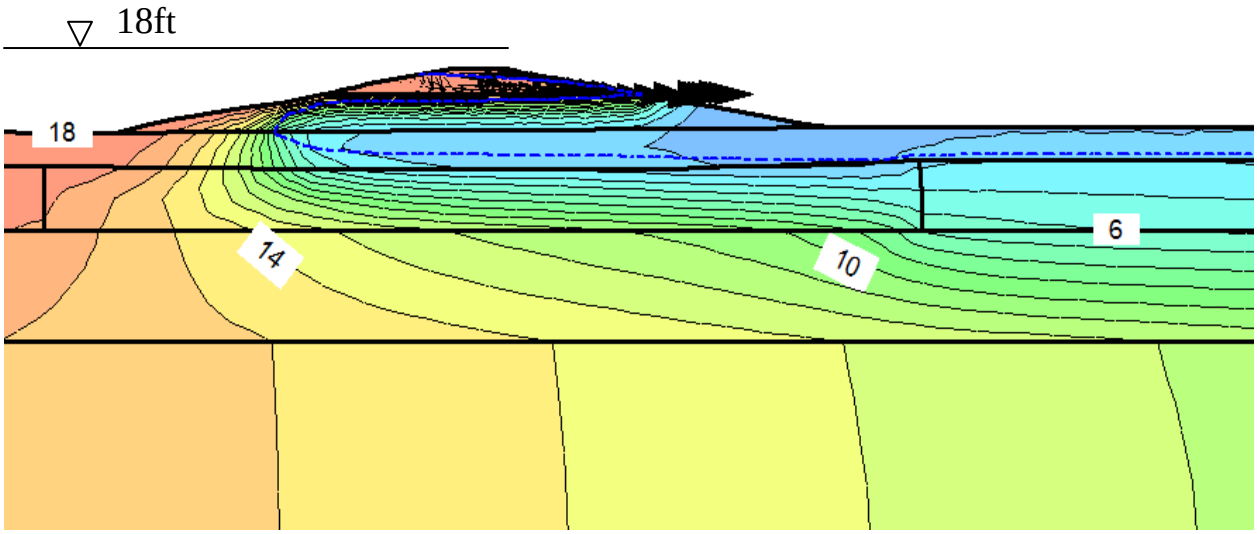


Figure 141: Transient model results (Bea and Cobos-Roa 2008). Storm surge at +18 feet. Total Heads on August 29, 2005 at 8:30 am (CDT). Marsh k: 10^{-3} cm/s.

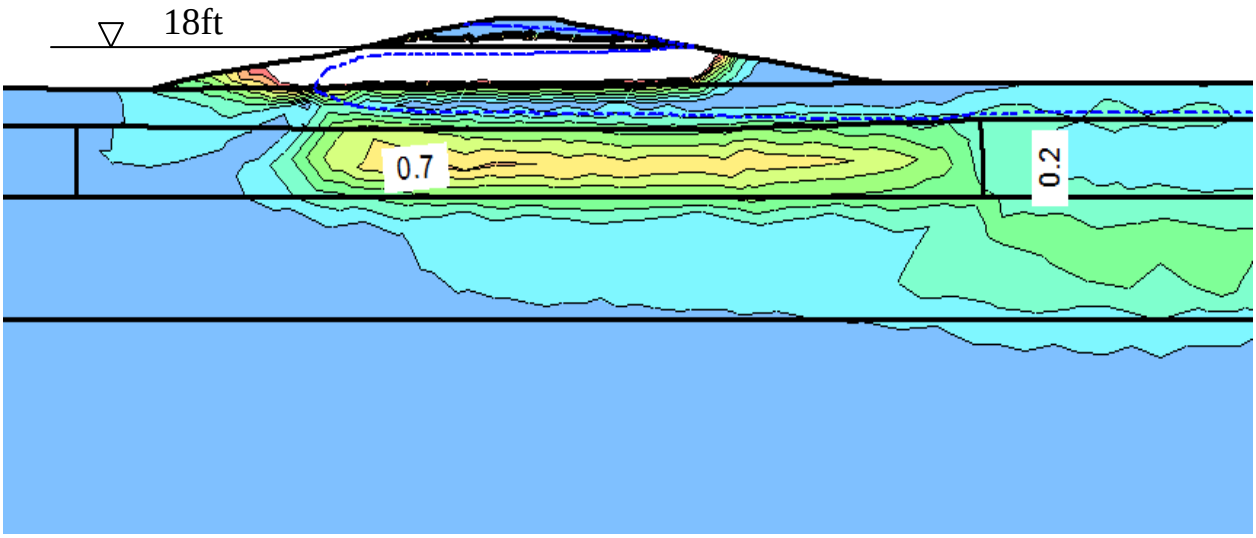


Figure 142: Transient model results (Bea and Cobos-Roa 2008). Hydraulic gradients on August 29, 2005 at 8:30 am (CDT). Marsh k: 10^{-3} cm/s.

142. To perform the lateral stability analyses, the soil properties for unit weight, undrained shear strengths and friction angles were assigned to the different layers according to the stratigraphic interpretation discussed earlier. As before, the available borings are "toe" borings, therefore no shear strengths under the levee embankment are available. Based on recent reports (IPET, 2006; ILIT, 2006), the difference between shear strengths between the two conditions ranges between 30% and 60%. A 40% increase was applied to the Marsh beneath the levee embankment.

143. Table 5 shows the material properties used in the slope stability calculations. Figure 143 shows the input geometry used for the stability analyses of Bayou Dupre.

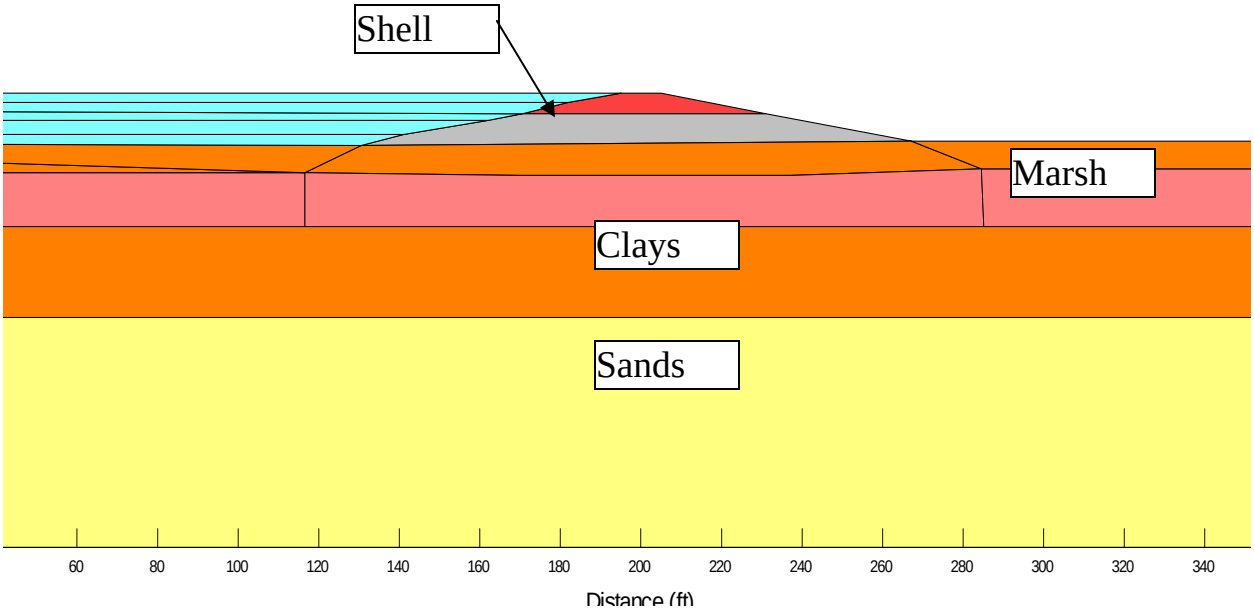


Figure 143: Analysis cross section for Bayou Dupre breach section (Bea and Cobos-Roa 2008).

Table 5: Material properties used for the slope stability models, Bayou Dupre.

Material	Shear Strength Parameter			
	Su (psf)	ϕ (deg)	Su/p'	γ (pcf)
Shell Fill		30		105
Clays (CL, CH)	240			115
Shell Fill[1]		30		105
Marsh (toe)	250			100
Grey Clay (CH)			0.26	117

144. Figure 144 shows the 'mean value' results of factor of safety for Bayou Dupre. For a storm surge of +18 feet the mean Factor of Safety is about FS = 1.5 (probability of failure, Pf = 16 percent), indicating that there is a high likelihood that this breach did not develop as a result of lateral instability, but rather as a result of seepage and scour through the EBSB – navigation structure concrete sheet piling interface. This analytical result is in conformance with observations of the breach at this location.

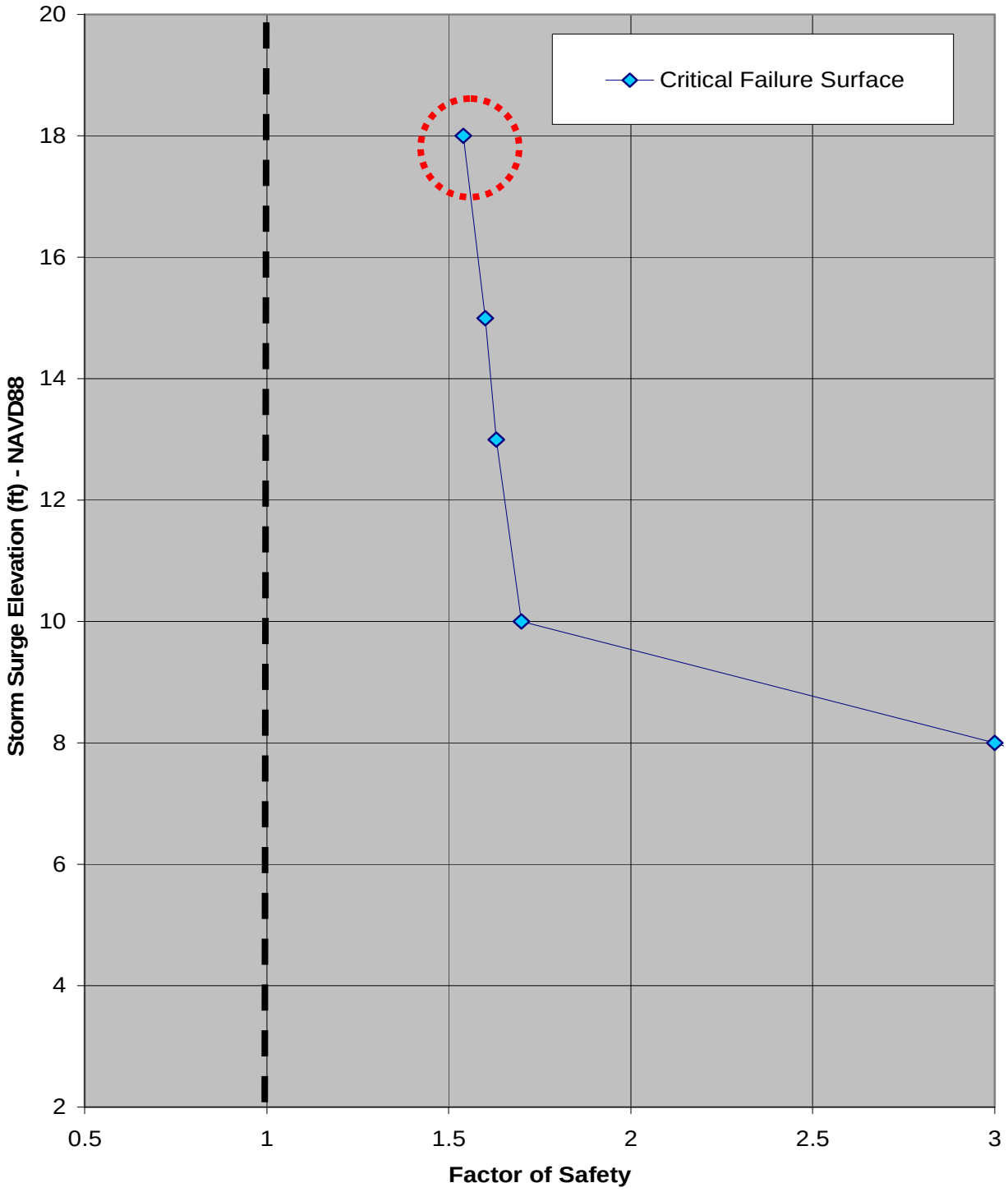


Figure 144: Computed Factors of Safety for Bayou Dupre breach section (Bea and Cobos-Roa 2008).

EBSB INSTABILITY EXACERBATED BY MR-GO CHANNEL DREDGING AND WIDENING

145. One of the modes of EBSB performance evaluated as part of this investigation was the potential for EBSB creep and settlements and associated soil lateral 'squeezing' exacerbated by the dredging and widening of the MR-GO channel. Analyses of the aerial LiDAR data gathered before and after Hurricane Katrina (Morris 2008) indicated that in many cases, sections of the EBSBs that had unusually low crown elevations were also located very close to the banks of the MR-GO channel (Figure 145). In the case of the location shown in Figure 145, the crown of the EBSB segment had sunk to an elevation of +12.5 feet NGVD 88. The banks of the MR-GO had come to within 150 feet of the toe of the EBSB segment.

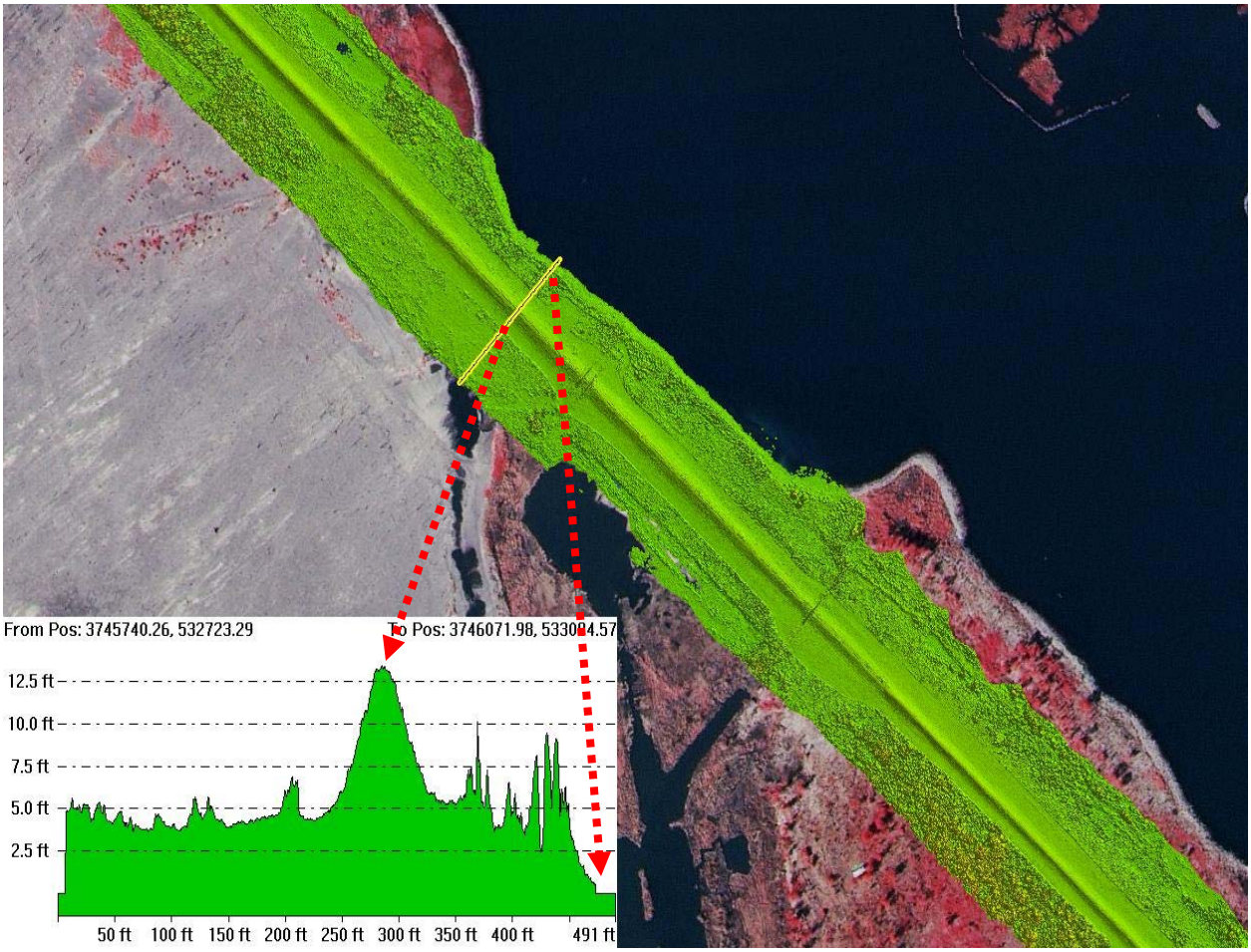


Figure 145: Processed LiDAR data showing EBSB cross section relative to the expanded channel of the MR-GO (Morris 2008).

146. As early as 1958, the USACE was concerned with the possibility of soil 'squeezing' into the adjacent MR-GO channel under the weight of the EBSBs (USACE 1958): "*As much as 75 percent of the material forming the bottom and sides of a channel crossing Chandeleur Sound between major distributary trends, particularly north of the North Islands, see plates 1 and 2), might consist of interdistributary and/or prodelta clays. From the standpoint of side slopes this may appear desirable, but it is possible that the poorly consolidated, high-water content interdistributary clays will tend to flow laterally into an excavation particularly under the extra weight of a spoil bank.*" (underline added for emphasis). This concern again surfaced in 1981: "*(h) Within 10 years the MR-GO bank will have eroded past the MR-GO R/W line (over 200 feet) and will threaten the stability of the hurricane levee.*" (underline added for emphasis (USACE 1981). During this investigation, no evidence could be obtained that would indicate that the USACE either understood or took proper corrective action to mitigate these dangers to the EBSBs.

147. Creep is a time dependent deformation of soils that is often referred to as 'secondary consolidation'. Consolidation – or squeezing of water from the soil – also results in a time dependent deformation of the soil. Creep does not require squeezing of water from the soil. Creep results from deformations of the soil structure that are dependent on the intergranular stresses and the nature of the soil particle contacts (e.g. electro-chemical for cohesive soils). For low stresses exerted on cohesive soils (less than about 50% of the soil shear strength), there are no significant creep deformations. For moderate stresses (about 50% to 75% of the soil shear strength), there can be significant creep deformations. For high stresses (greater than about 75% of the soil shear strength), there can be very large creep deformations that can become 'unbounded' and result in a stability failure.

148. Creep was studied as part of the USACE's Atchafalaya levee test section performed in the mid-1960s (Figure 146; Kaufan and Weaver 1967; USACE 1968). This levee test section was extensively instrumented and analyzed (Foott and Ladd 1977; Edgers 1973). The levee test section showed 'lateral squeezing' – displacement of the underlying soft clay soils that was concentrated in the underlying peat and soft organic clays (Figure 147). Lateral displacements after 75 months approached 3 feet and were accompanied by similar decreases in the elevation of the levee crown. The lateral displacements extended more than 300 feet from the levee crown.

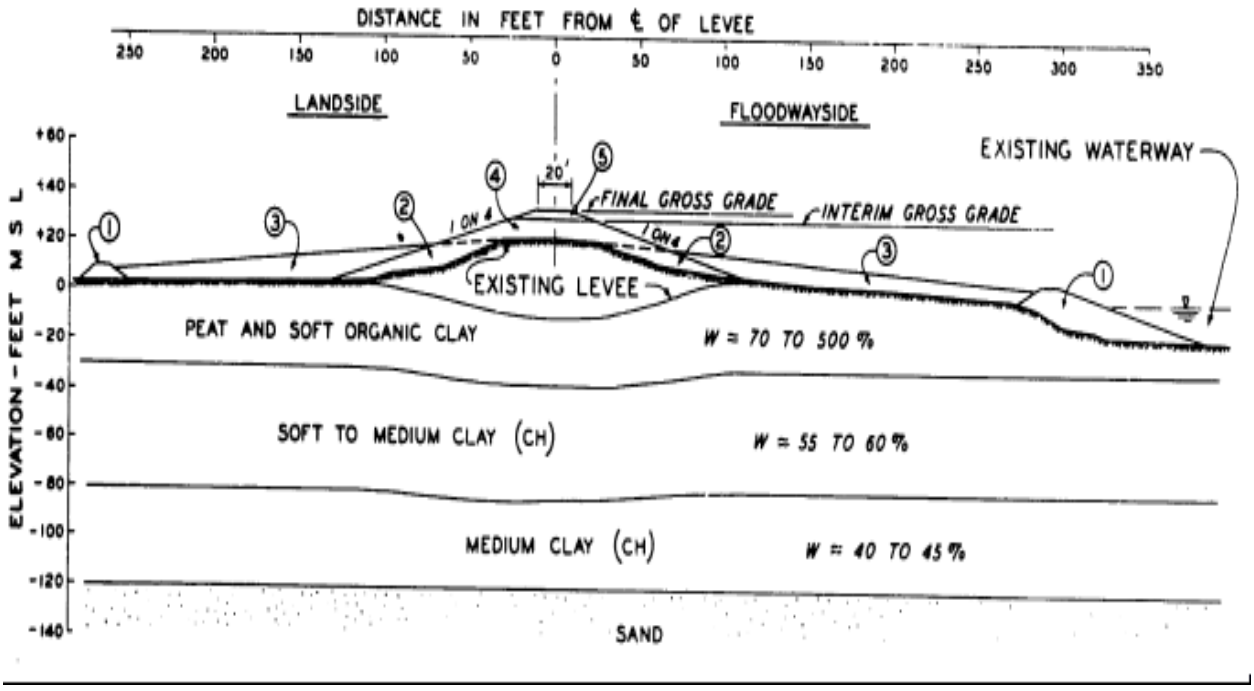


Figure 146: Atchafalaya Levee Test Section (Foot and Ladd 1977; USACE 1968).

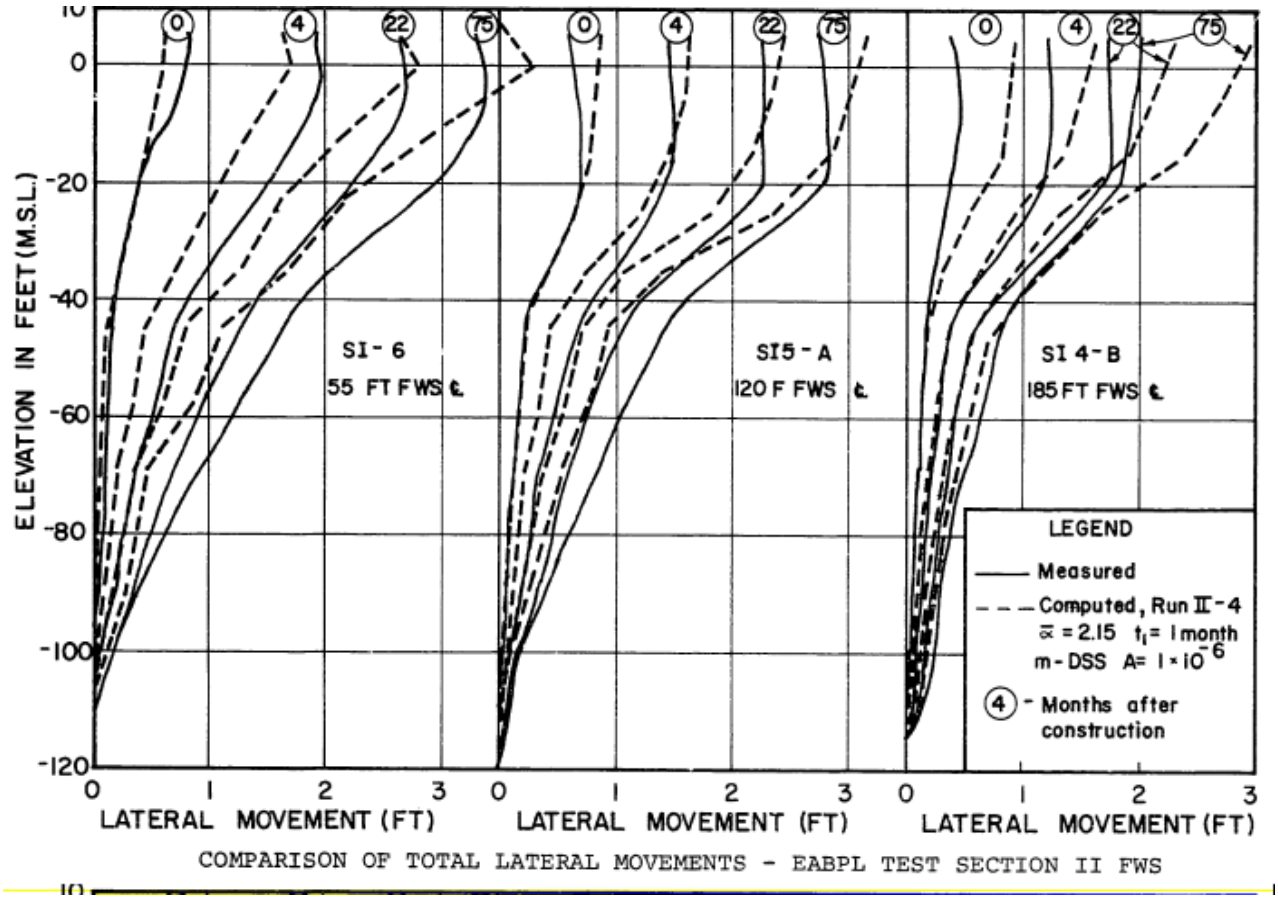


Figure 147: Atchafalaya Levee Test Section Soil Lateral Displacements from End of Construction (0 time) to 75 Months After Construction (Edgers 1973).

149. The analytical work performed by Edgers concluded: "*The lateral creep movements below heavily loaded areas cause large vertical settlements at these locations and a resultant heave of less heavily loaded areas such as beyond the berms of the Test Sections. The large berms of Test Section III result in reduced lateral movements close to the levee centerline and also reduced crest settlement caused by undrained creep*" (Edgers 1973). These results were confirmed by the later analyses performed by Redman and Poulos (1984).

150. This type of levee performance consideration has been encountered previously in connection with construction of embankments in Florida by the USACE (Schmertmann 1966; 2008). The embankments were constructed on top of a plastic clay (Marl). The overburden

stresses exerted by the embankments squeezed the Marl out from under the embankment and into the adjacent channel (Figure 148). The channel bottom grade could not be maintained. The tops of the embankments subsided and 'cones of soil heave' developed in the channel. It is noteworthy that this experience also involved a permeable layer that was under the mobile plastic soils – marl. Water pressures developed in this underlying permeable layer acted to exacerbate the soil heaving in the channel. A similar type of sand layer is located in the area of the MR-GO Reach 2 EBSBs, and when subjected to elevated water levels in the MR-GO could be expected to affect soil heaving in a similar manner.

151. Concerns for EBSB creep related displacements were reinforced when it was observed (March 2006) that 'islands' of soil were developing in the borrow pits adjacent to the recently re-constructed EBSBs along sections of the MR-GO Reach 2 (Figure 149). These islands of soil were not initially present after the EBSB re-construction was completed and the borrow pits allowed to fill with water (Esteves 2007, Figure 150). They were particularly evident where the edges of the borrow pits were close to the toes of the levee berms (Figure 151). The banks of the borrow pits were within 150 feet of the toe of the EBSB. Observations indicated that soil was filling the borrow pits from two sources: from the surface of the EBSBs (erosion) and from under the EBSBs (soil squeezing from under the EBSBs).

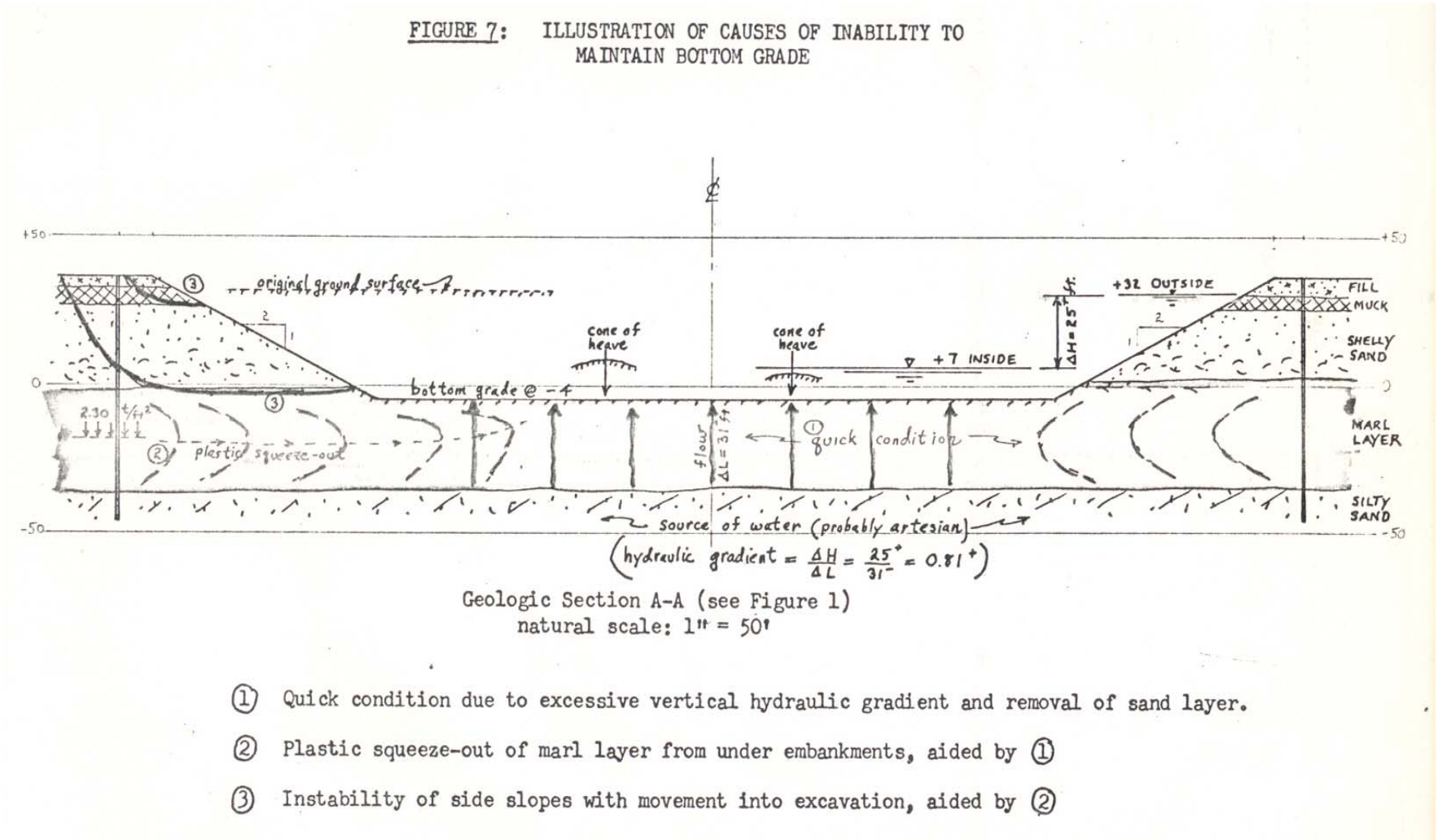


Figure 148. Transect of USACE embankment and channel in Florida showing causes of instability to maintain the channel by squeezing the plastic clay – marl – soils from under the constructed embankments (Schmertmann 1966; 2008).



Figure 149: Islands of soil growing in post-Karina soil borrow pits excavated on protected side of MR-GO Reach 2 EBSBs (Bea 2007).



Figure 150: Photograph of post-Hurricane Katrina EBSB repair borrow pits (Storesund 2006).



Figure 151: Picture of Southern section of MR-GO Reach 2 EBSBs with post-Karina soil borrow pits excavated on protected side of EBSBs. Bank of borrow pit is approximately 100 to 200 feet from toe of EBSB (Bea 2007).

152. Recently, Kemp has performed bathymetric surveys of these borrow pits (Kemp 2008). The surveys showed several dramatic instances of soil heave in the bottom of the borrow pits (Figures 152 and 153). The soil heave proceeded to the point where 'islands' were formed in the middle of the borrow pit (Figure 154) – similar to 'mud lumps' found in parts of the Mississippi River Delta (Morgan 1952; Coleman and Gagliano 1964) – or mud squeezing up between a person's toes and around their feet as they walk through soft mud. These features were not present when the levee re-construction was completed (Esteves 2007). These features were not connected with soil erosion and deposition associated with the surfaces of the EBSBs.

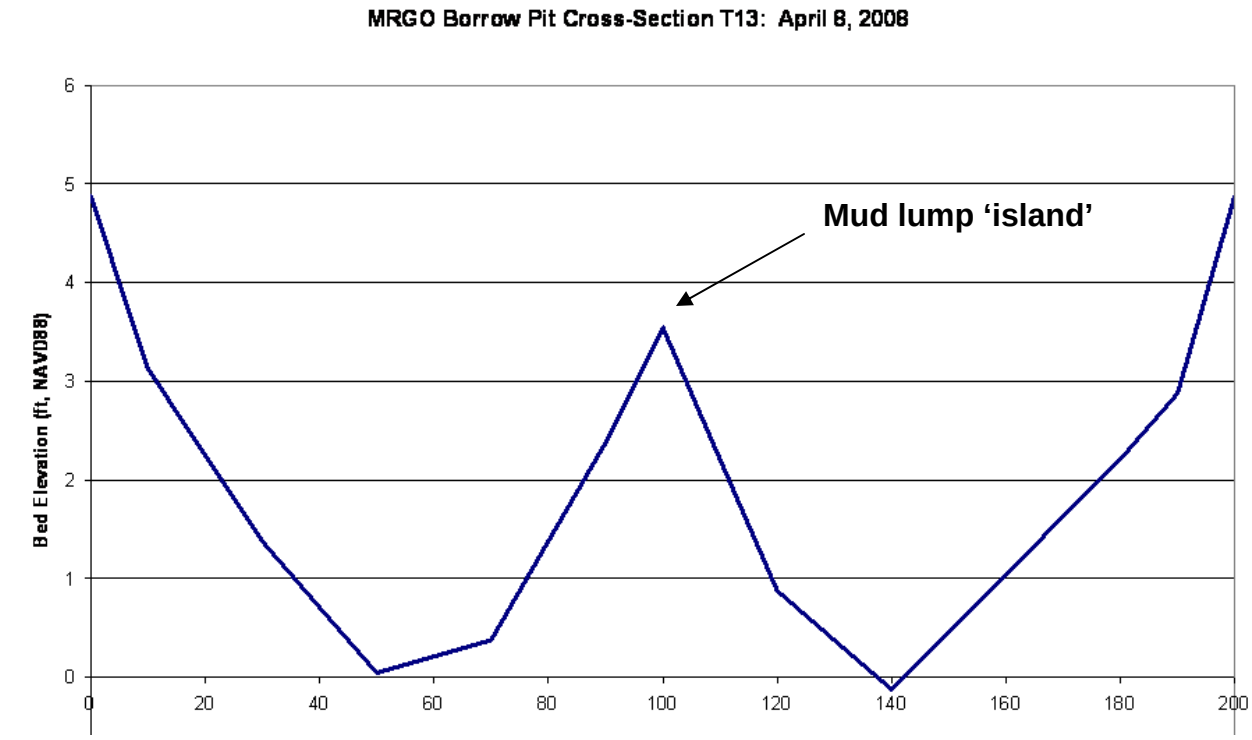


Figure 152: Traverse through the re-constructed EBSBs protected side borrow pits (Kemp 2008).

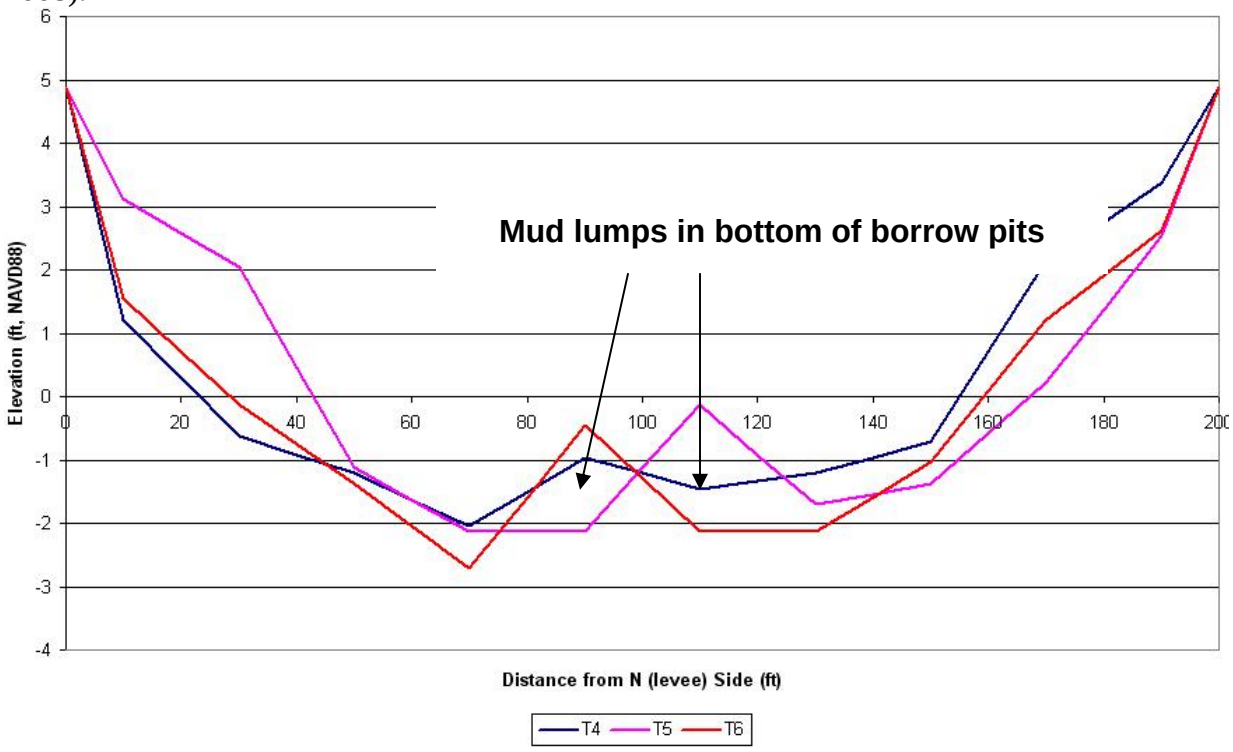


Figure 153: Traverses through the re-constructed EBSBs protected side borrow pits (Kemp 2008).



Figure 154: Photograph of mud lump 'island' being formed in borrow pit adjacent to re-constructed EBSBs (Kemp 2008).

153. This concern was raised again when cross sections were produced that showed the MR-GO channel adjacent to the EBSBs before Hurricane Katrina (Figures 155 and 156) (Morris 2007). The channel dredging and erosion of the banks of the MR-GO had widened to the point where there would be concerns for both the EBSB stability and exacerbation of the creep related displacements. The 'drop off' into the MR-GO channel was within 200 feet to 250 feet of the toes of the EBSBs.

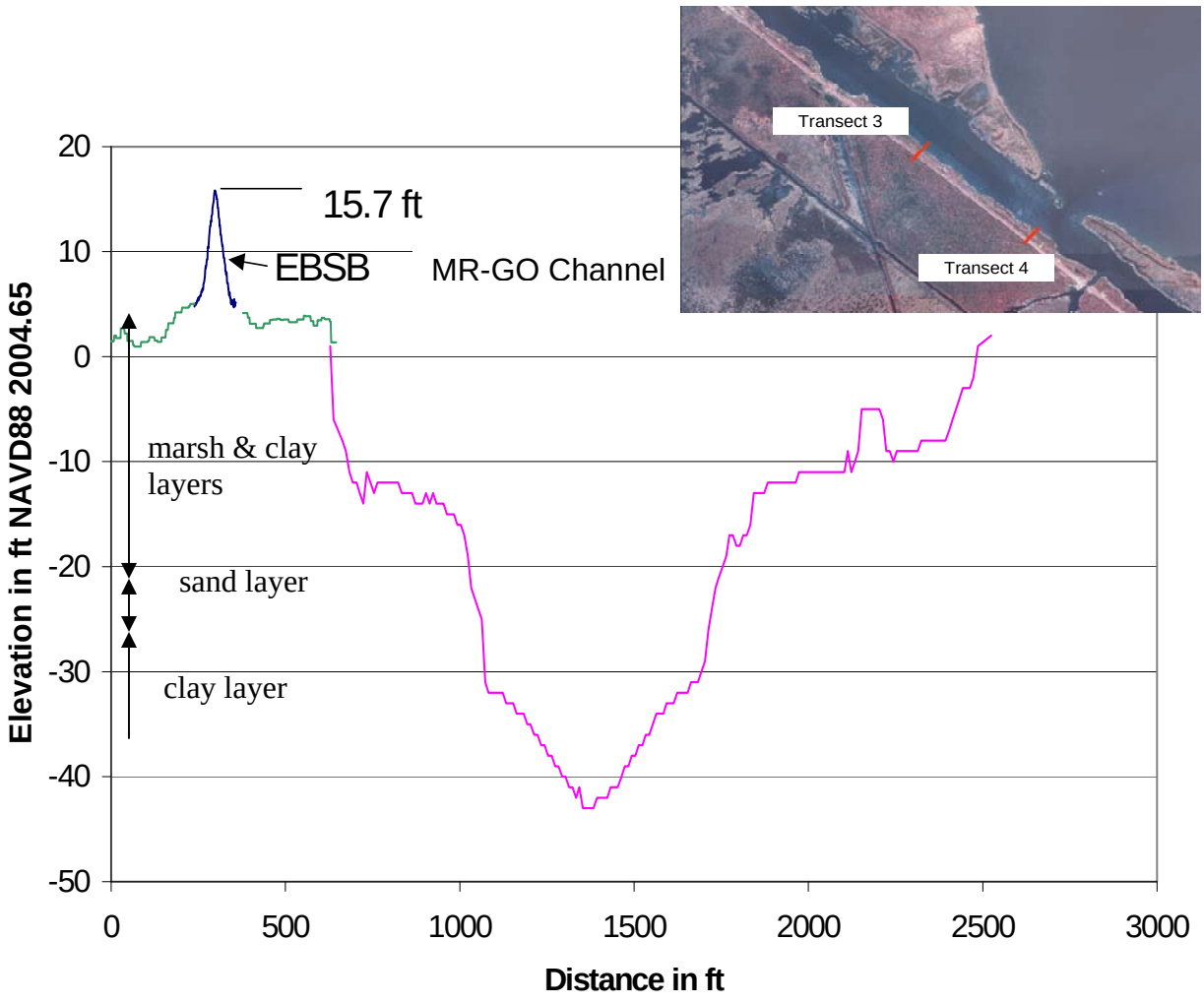


Figure 155: Cross section at EBSB breaching study location mid way between Bayous Dupre and Bienvenue (after Morris 2007).

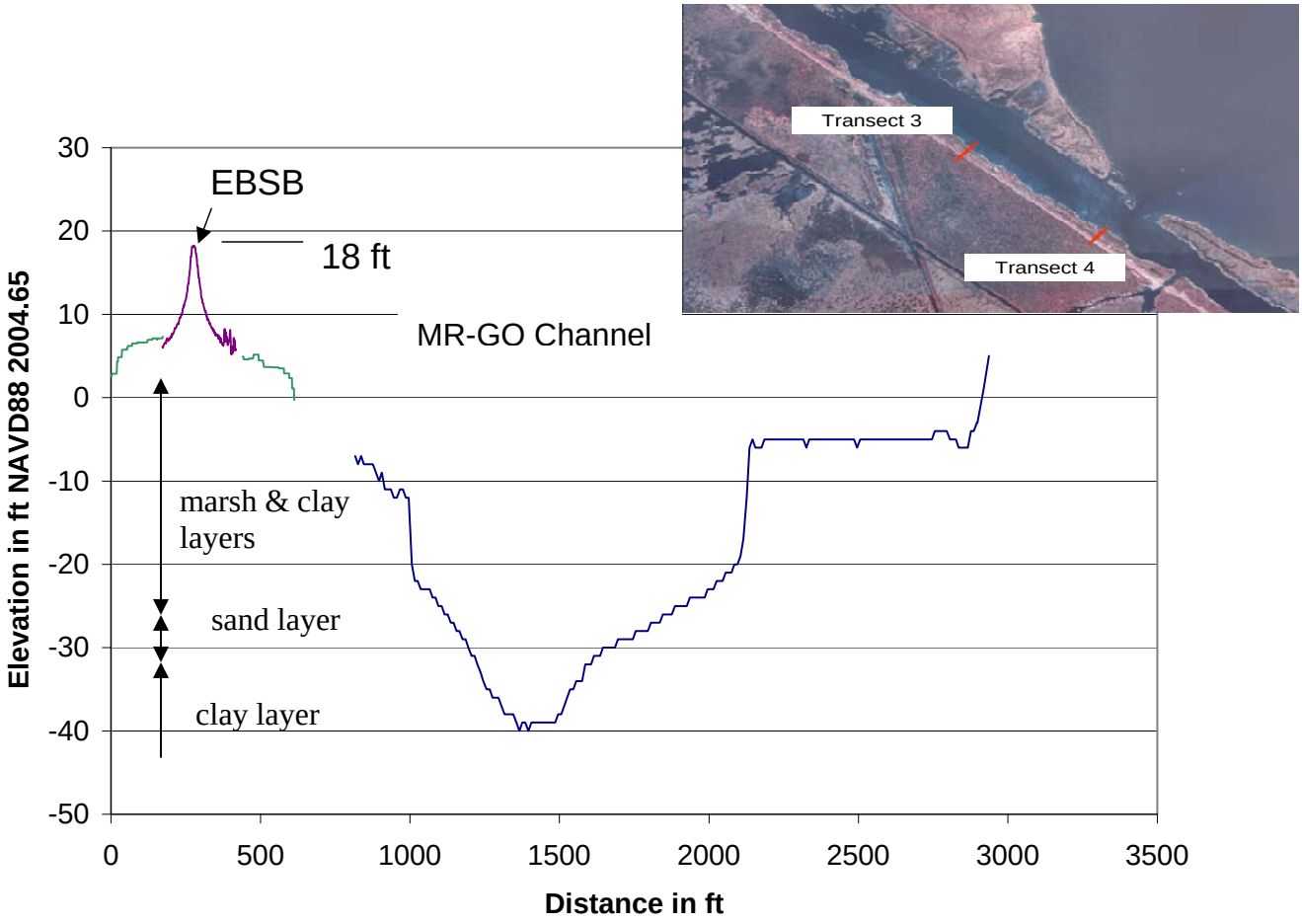


Figure 156: Cross section at EBSB south of Bayou Dupre (after Morris 2007).

154. Several different analytical approaches have been developed and applied to address this type of embankment and soil behavior (Silversti 1983; Poulos et al 1976; Edgers 1973; Redman and Poulos 1984). The analytical model developed by Silverstri was specifically for embankments founded on soft soils of limited thickness (Figure 157). Silverstri used this analytical model to evaluate soil squeezing behavior under 10 soil embankments (including the USACE Atchafalaya test levee cited earlier). These test embankments were constructed so as to develop long-term incipient failure (continuing horizontal and vertical settlements). Agreement between the Silverstri analytical model and the performance of these embankments was excellent. In addition, the Silverstri analytical model produced results that were very comparable with those from conventional lateral

stability analysis when these stability analyses took account of the soil creep strength and deformation characteristics (Silverstri 1983).

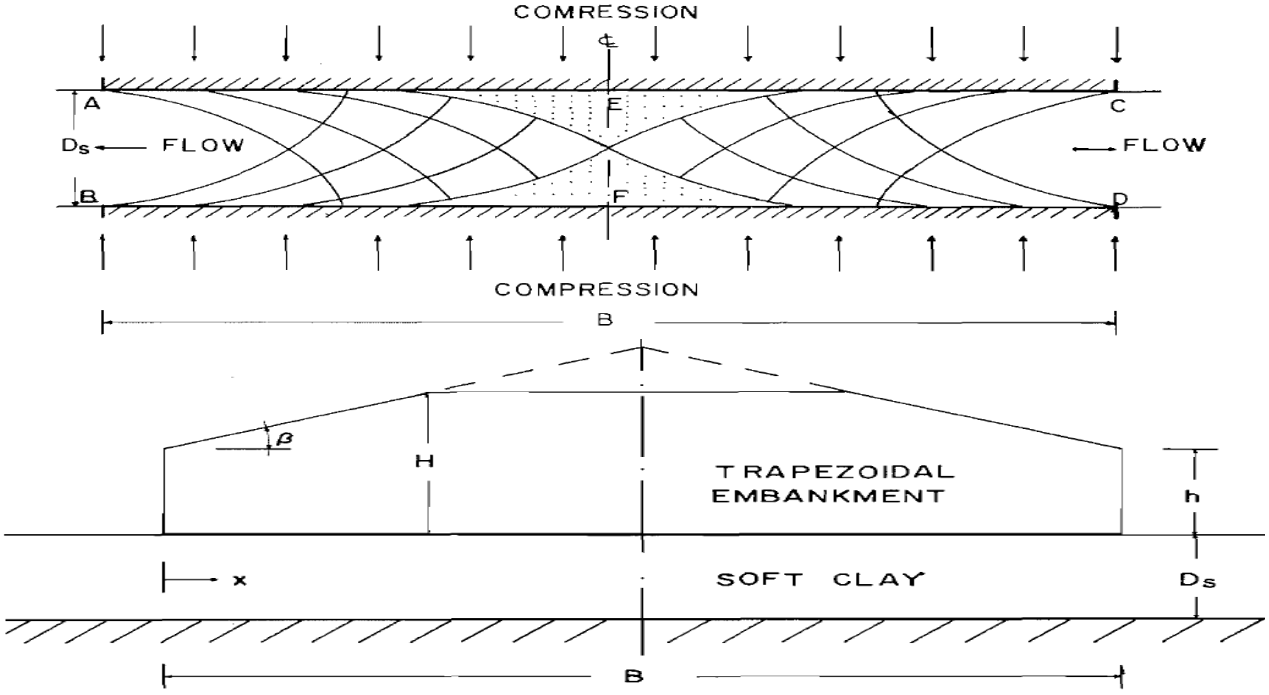


Figure 157: Analytical model for trapezoidal embankment on a soft clay (Silverstri 1983).

155. Silverstri's analytical models were applied to several of the cross sections of the EBSBs along Reach 2 of the MR-GO. The soil shear properties used in the stability analyses performed for the breaches at Bayous Bienvenue and Dupre were used in these analyses. Adjustments were made to the laboratory shear strengths to account for the long-term duration of application of the EBSBs loadings (Figure 158; Shehan et al 1996; Edgers 1977; Silvestri 1983). As summarized in Figure 159, Factors of Safety (FS) for squeezing of the cohesive soils under the EBSBs ranged from FS = 0.8 to FS = 1.1, indicating that lateral squeezing – creep of the underlying clays should be taking place along significant sections of the EBSBs. This lateral squeezing would be accompanied by significant decreases in the elevations of the crowns of the EBSBs.

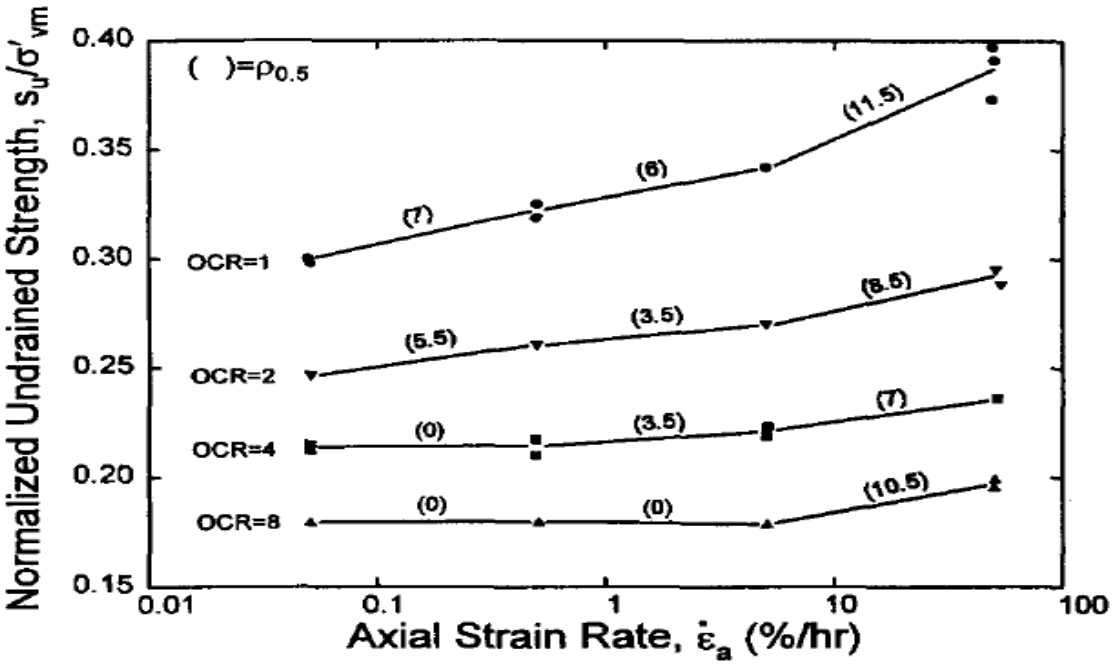


Figure 158: Reduction in effective undrained soil shear strength as a function of long-duration applied stresses (Edgers 1973; Sheahan et al 1996).

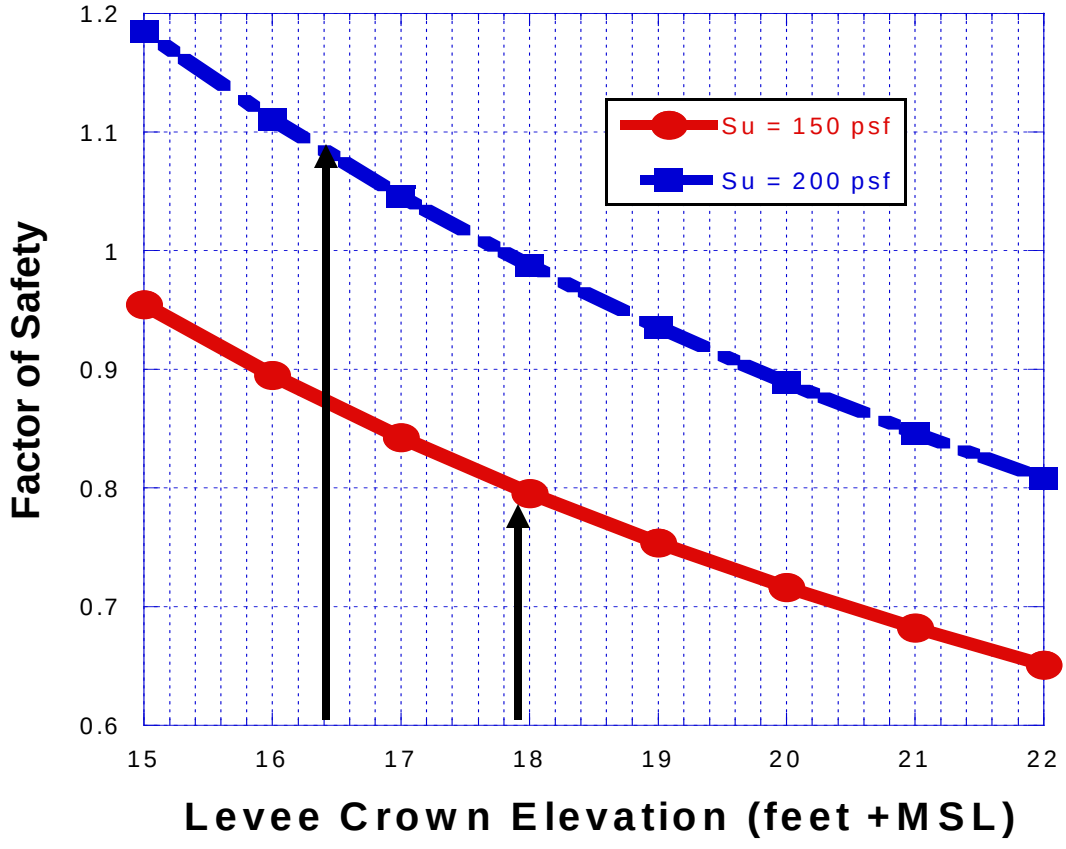


Figure 159: EBSB Factor of Safety against lateral squeezing – creep

156. Similar conclusions were reached by Schmertmann (2008): "*In your case, according to my rough estimates, the p/su = 7 (ratio of overburden pressure to effective soil shear strength) under the levee and also between its toe and the excavations…indicates it (underlying soft clay) should squeeze out.*" These analytical evaluations confirmed the field observations and levee test results cited earlier.

157. Stability analyses also were performed during this investigation using GEOSTUDIO software SLOPE\W (Spencer's method) to evaluate stability of a 'typical' EBSB under long-term creep conditions with the channel of the MR-GO located 250 feet away from the toe of the EBSB (Figure 160). The results (Figure 161) showed that the EBSB would be in a condition of 'Neutral' stability' – factors of safety close to unity. Lateral stability Factors of Safety ranged from FS = 1.0 to 1.2. These results confirmed those developed using the analytical model developed by Silversteri (1983). These results are confirmed by the instrumentation and analytical results associated with the USACE Atchafalaya levee Test Section 3 (Edgers 1973; Redman and Poulos 1984).

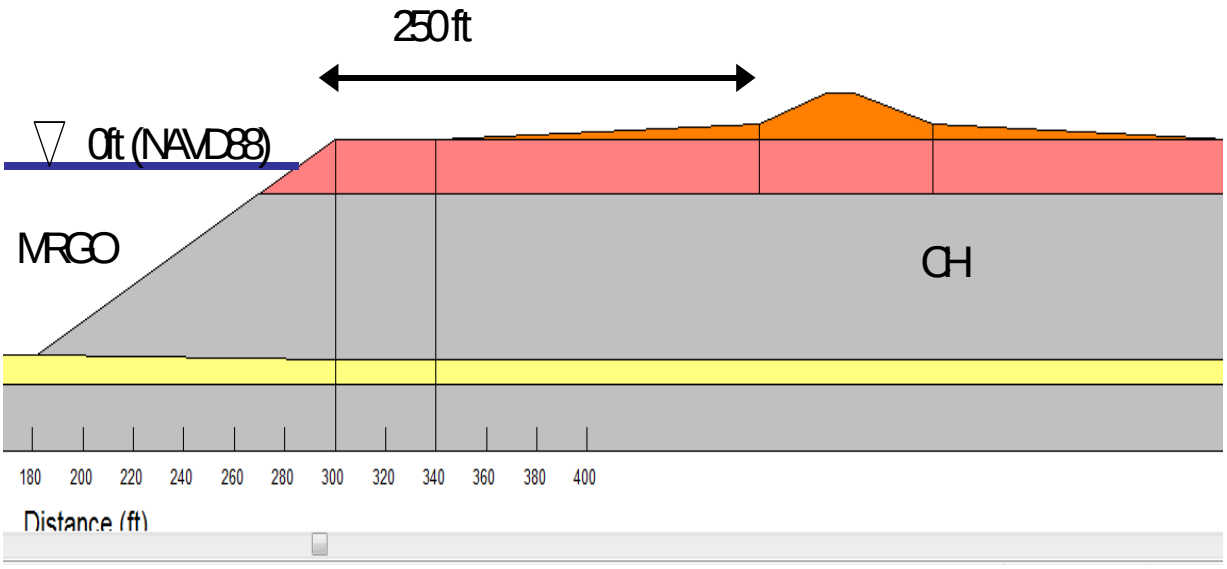


Figure 160: EBSB creep lateral stability analytical model.

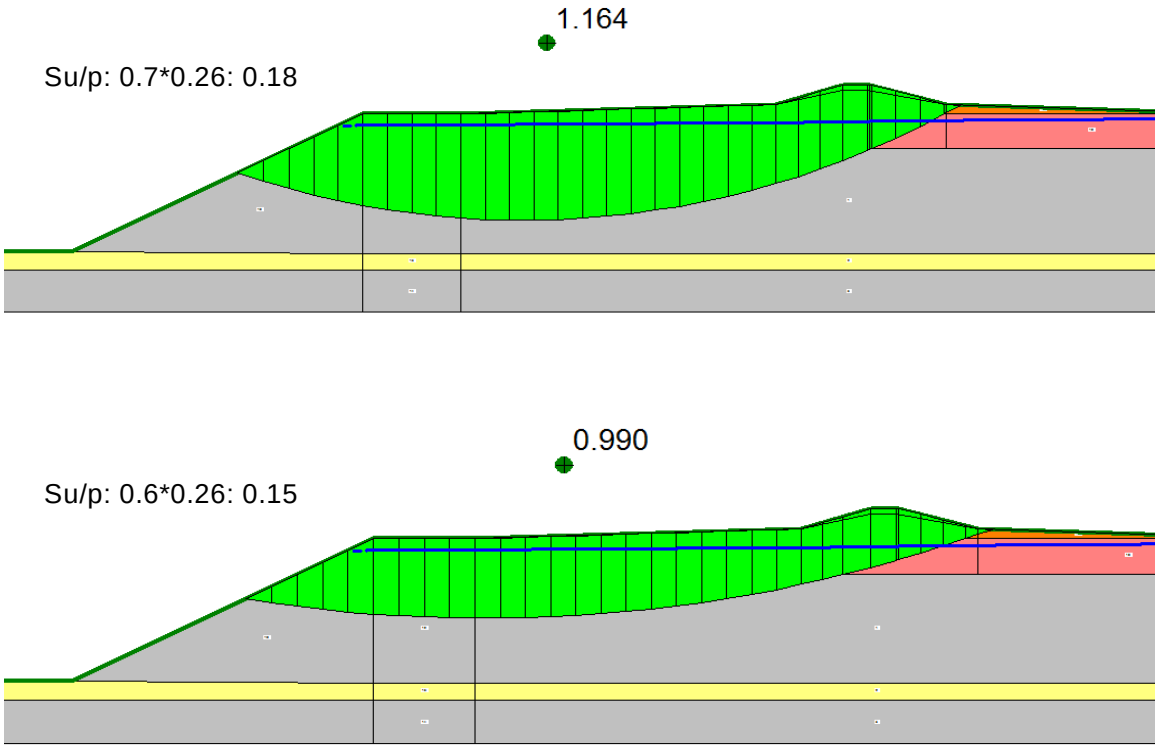


Figure 161: Lateral stability factors of safety (1.2 to 1.0) determined from GeoStudio analytical model using Spencers method.

158. The USACE has specific guidelines for ‘excavations’ near or within federally constructed flood control projects (USACE 2005, Bea and Storesund 2008b). These guidelines stipulate: “*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.*” During this study, no such analyses by the USACE could be located that addressed the effects of dredging and erosion of the MR-GO channel banks would have on the adjacent EBSBs stability and protective elevations.

159. It is clear that in some cases the MR-GO channel banks were allowed by the USACE to encroach into the ‘critical areas’ adjacent to the EBSBs. These encroachments endangered the elevations and stability of the EBSBs. The observations and analyses indicate

that lateral squeezing of the underlying marsh and clay layers into the adjacent MR-GO channel resulted in significant decreases in the crown elevations of the EBSBs. No documentation of precautions taken or analyses performed by the USACE to assure that the integrity and stability of the EBSBs was not endangered could be found during this study. Similar observations apply to the recent construction of soil borrow pits by the USACE adjacent to the protected sides of the MR-GO Reach 2 EBSBs.

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

Executed on July 11, 2008 in Moraga, California.



Robert Bea, Ph.D, PE

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