DECLARATION OF ROBERT G. BEA

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 a summary of my review, and documentation of and responses (completed as of January 29, 2009) to the Defense's Expert Reports written by Mr. B. Ebersole, Dr. D. Resio, Dr. R. Mosher, and Dr. T. Wolff (cited Defense Expert Reports dated December 2008). My Declaration relates to the studies conducted to evaluate performance of the man-made flood protection structures along Reach 2 of the Mississippi River Gulf Outlet (MR-GO) and at the portion of the MR-GO Reach 1 adjacent to the Lower 9th Ward. This performance has been evaluated for two sets of Hurricane Katrina environmental conditions (surge, waves, currents): 1) the 'MR-GO As Was' conditions, and 2) the 'MR-GO Neutral' ("do no harm") conditions. I incorporate by reference here all previous Declarations and 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 four sections. Section I provides an overview of my analyses of the major issues raised in the cited Defense Expert Reports and discussed herein. Section II provides a summary of my responses to the cited Defense Experts analyses of the work contained in my previous Declarations and Expert Reports pertaining to analyses of the performance of the hurricane flood protection structures along Reach 2 of the MR-GO that developed during Hurricane Katrina (As Was and conditions; Scenarios 1 and 2C, respectively). Section III provides a summary of my responses to the cited Defense Expert's analyses of the work documented in my previous Declarations and Expert Reports pertaining

to analyses of the performance of the man-made hurricane flood protection structures along the portion of Reach 1 adjacent to the Lower 9th Ward during Hurricane Katrina (As Was and Neutral MR-GO conditions). Section IV summarizes the results of my reviews of the cited Defense Expert Reports. The following table of contents will serve as a guide to help reading this Declaration.

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3. This Declaration supplements my previous Expert Report, Declarations and Technical Reports dated July 14, 2008 documenting engineering forensic studies providing insights into the reasons for breaches, failures and overtopping that developed with respect to the man-made flood protection structures located along Reach 2 of the MR-GO and along the portion of the MR-GO Reach 1 at the Lower 9th Ward.

4. I received the cited Defense Expert Reports as electronic files during the afternoon of December 29, 2008. Requested supporting documentation for the Defense Expert Reports was received in partial form as electronic files during the afternoon of January 2, 2009. The requested supporting documentation was not complete. I have made multiple requests for the missing supporting documentation. These requests initially were denied by the Defense Counsel. I was only able to obtain parts of the requested supporting documentation on January 27, 2008 – too late to be able to review and integrated into my Expert Report. Important parts of the supporting documentation have been requested, but they not been provided by the Defense. Consequently, this review has not benefited from review of the supporting documentation. This Declaration summarizes the reviews and responses I have been able to develop and document during approximately 20 working days.

5. I have not been provided with the time required to respond to all of the important issues defined, developed and documented in the cited Defense Expert Reports. Nor have I had the opportunity to review the important supporting documentation (significant documents not provided as requested). I have endeavored to develop as complete as possible responses to the primary issues raised in the cited Defense Expert Reports that have potential major impacts on analyses of the causative factors that led to the observed performance of the man-made flood protection structures during Hurricane Katrina and the predicted

performance of these man-made hurricane protection structures during the MR-GO Neutral Hurricane Katrina conditions.

6. My analyses, evaluations, assessments and conclusions have been based on the background information and documentation cited in this Declaration. I reserve the right to modify my analyses, evaluations, assessments, and conclusions as more time is provided to enable me to develop and document these elements. Also, I reserve the right to modify my analyses, evaluations, assessments, and conclusions in the case that new or additional information becomes available in the future.

I. OVERVIEW

Summary of Defense & Plaintiffs Experts Conclusions

7. Table 1 summarizes my understanding of the major conclusions reached by the Defense and Plaintiffs Experts pertaining to the major breaches in the man-made flood protection structures along Reach 2 of the MR-GO and along the portion of Reach 1 MR-GO adjacent to the Lower 9th Ward. The purpose of this summary is to develop an overview of the primary technical elements resulting in the contrasting conclusions developed by the two groups of experts. This summary is further detailed in Appendix A.

8. My overall evaluation of the work contained in the Defense Expert Reports is that this work represents a substantial body of technical data, observations, analyses, assessments, and opinions intended to address some of the primary technical issues of concern in this litigation. The basic background cited in the Defense Expert Reports has been used in the studies and investigations documented in my previous Expert Reports, Declarations and Technical Reports. The December 2008 Defense Expert reports contain some new data, information, insights, and conclusions that have importance in these deliberations. Unfortunately, due to time and other resource limitations, I have not been able to fully analyze and consider these contributions.

9. There are many fundamental and important points of agreement between the Defense and Plaintiff Experts that background these analyses, conclusions, and expert opinions. There is much more agreement than disagreement. Based on the work documented in the cited Defense Expert Reports, it is my assessment that the primary differences in the Defense and Plaintiffs Expert's conclusions and opinions are focused in a few major issues of

critical importance. To develop clear understanding of what most likely happened and what most likely should have happened, it is important to strive to "sort the wheat from the chaff" and to attempt to see through the 'fog' created by the Expert deliberations about the very complex forensic engineering challenges involved in this litigation.

10. In the context of the man-made flood protection structures existing at the time of Hurricane Katrina, my conclusion is that the fundamental differences between the Defense Experts and Plaintiffs Experts assessments are focused on understanding of the most probable or likely <u>modes of performance</u> of the man-made flood protection structures during Hurricane Katrina (As Was, Neutral or Ideal).

11. The conclusions reached as documented in the Defense Experts Declarations and Technical Reports (as I understand them) is that the breaching of the Reach 2 EBSBs that developed during Hurricane Katrina was due primarily to <u>surge overtopping</u> and wave water flow induced erosion and scour that resulted in the final observed conditions of these flood protection structures.

12. The Defense Experts have further concluded that during the "Ideal MRGO" Hurricane Katrina conditions (Scenarios H5 and H6), the performance of the Reach 2 EBSBs would be similar to (same as) that observed during and following Hurricane Katrina. The flooding of the St Bernard Parish Polder <u>would have been the same</u> as (similar to) that experienced during Hurricane Katrina.

13. The conclusion documented in the Defense Experts Technical Reports (as I understand them) is that the breaching of the MR-GO Reach 1 man-made flood protection structures adjacent to the Lower 9th Ward that developed during Hurricane Katrina was due primarily to <u>surge water pressures</u> generated on the flood side of the flood protection

structures (including pressures generated in the ‘gap’ between the soils and the sheet piling on the water side), overtopping water erosion of the supporting soils on the land side (South Breach, failed after overtopping) and the reduced cross section of the land side supporting levee (North Breach, failed before overtopping).

14. The Defense Experts have further concluded that during the “Ideal MRGO” Hurricane Katrina conditions (Scenarios H5 and H6), that the performance of the Reach 1 flood protection structures at the Lower 9th Ward would be similar to (same as) that observed during and following Hurricane Katrina.

15. The conclusions reached as documented in the Plaintiffs Experts Declarations and Technical Reports is that the breaching of the Reach 2 EBSBs that developed during Hurricane Katrina (Scenario 1) was due to a combination of water side wave erosion initiated breaching exploited by surge water and wave flow through the EBSB crest breaches (crenellations) and on surge overtopping and wave water flow induced erosion and scour that resulted in the final observed conditions of these flood protection structures. The large breach that developed at the Bayou Bienvenue navigation – water control structure – EBSB interface was initiated by hydraulic conductivity effects under the structure – EBSB interface and the associated surge water pressures (lateral instability) and propagated to a very large breach by the inrushing waters.

16. The Plaintiffs Experts have further concluded that during the “Neutral MR-GO” (“do no harm” conditions) Hurricane Katrina conditions (Scenario 2C), the Reach 2 EBSBs would not suffer significant breaching. The EBSB interface breach at the Bayou Bienvenue navigation – water control structure would develop near the time of the peak surge. The EBSB interface breach at the Bayou Dupre navigaton – water control structure

would not develop. The flooding of the St Bernard Parish Polder would be significantly reduced compared with that experienced during Hurricane Katrina.

17. The conclusion documented in the Plaintiffs Experts Technical Reports is that the breaching of the MR-GO Reach 1 man-made flood protection structures adjacent to the Lower 9th Ward that developed during Hurricane Katrina was due primarily to surge water pressures generated on the flood side of the flood protection structures (including pressures generated in the 'gap' between the soils and the sheet piling on the water side), and hydraulic conductivity effects exacerbated by the USACE Lock Expansion Project EBIA site clearing poorly backfilled excavations (seepage effects and uplift pressures) accompanied by overtopping water erosion of the supporting soils on the land side (South Breach, failed after overtopping) and the reduced cross section of the land side supporting levee (North Breach, failed before overtopping).

18. The Plaintiffs Experts have further concluded that during the "Neutral MR-GO" ("do no harm") Hurricane Katrina conditions (Scenario 2C), that the performance of the Reach 1 flood protection structures at the Lower 9th Ward would be similar to (same as) that observed during and following Hurricane Katrina. Consequently, the flooding of the Lower 9th Ward portion of the St. Bernard – Lower 9th Ward Polder would be similar to that experienced during Hurricane Katrina.

Table 1 (a): Summary of primary conclusions reached by Plaintiffs and Defense Experts as they pertain to development of the breaches along Reach 2 of the MR-GO and along the portion of Reach 1 adjacent to the Lower 9th Ward during Hurricane Katrina and during Hurricane Katrina with Neutral MR-GO conditions.

Issue	Plaintiffs Experts	Defense Experts	Observations
Development of As Was MR-GO Reach 2 EBSB Major Breaches	Wave erosion of flood side face resulting in crest crenellation propagated by surge and wave overtopping erosion of the protected side face to develop breaches. Develop breaches early as surge waters rise admitting large volumes of water early into St Bernard Parish.	Surge and wave overtopping erosion of the protected side face propagating to the flood side face to develop breaches. Develop breaches after surge overtopping admitting large volumes of water later into St Bernard Parish.	Plaintiffs Experts performed qualitative and quantitative erosion analyses of flood side and protected side erosion. Defense Experts performed qualitative analyses of protected side erosion, quantitative analyses of flood and protected side water erosion velocities, and qualitative evaluations of erosion due to surge and wave overtopping.
Development of Neutral MR-GO Reach 2 EBSB Major Breaches	No significant breaching due to surge and wave action. Flooding limited to surge overtopping. Overtopping surge waters contained within the Reach 2 – 40 Arpent basin. Minor flooding due to rainfall.	Surge and wave overtopping erosion of the protected side face propagating to the flood side face to develop breaches. Flooding of Saint Bernard Parish same as for Hurricane Katrina.	Plaintiffs Experts performed qualitative and quantitative erosion analyses of flood side and protected side erosion. Defense Experts performed qualitative analyses of breach development, quantitative analyses of flood and protected side water erosion velocities, and qualitative analyses of erosion due to surge and wave overtopping.

Table 1 (b): Summary of primary conclusions reached by Plaintiffs and Defense Experts pertaining to development of major breaches along Reach 2 of the MR-GO and along the portion of Reach 1 adjacent to the Lower 9^{th} Ward during Hurricane Katrina and during Hurricane Katrina with Neutral MR-GO conditions.

Issue	Plaintiffs Experts	Defense Experts	Observations
Development of As Was MR-GO Reach 2 Structure – EBSB Interface Breaches (Bayous Dupre and Bienvenue)	Wave and surge overtopping erosion initiated breaching (Bayou Dupre) and surge pressure initiated seepage and uplift pressure instability leading to breaching (Bayou Bienvenue).	Attribute development of interface breaches due to overtopping flow erosion and scour.	Plaintiffs Experts performed qualitative and quantitative erosion and stability analyses of soils interfacing with structures. Analyses corroborated with qualitative analyses of photographic, video, and field survey evidence.
Development of Neutral MR-GO Reach 2 Structure – EBSB Interface Breaches (Bayous Dupre and Bienvenue)	Wave and surge overtopping erosion initiated breaching (Bayou Dupre) and surge pressure initiated seepage and uplift pressure instability leading to breaching (Bayou Bienvenue).	Attribute development interface breaches due to overtopping flow erosion and scour.	Plaintiffs Experts performed qualitative and quantitative erosion and stability analyses of soils interfacing with structures. Analyses corroborated with qualitative analyses of photographic, video, and field survey evidence.

Table 1 (c): Summary of primary conclusions reached by Plaintiffs and Defense Experts pertaining to development of major breaches along Reach 2 of the MR-GO and along the portion of Reach 1 adjacent to the Lower 9th Ward during Hurricane Katrina and during Hurricane Katrina with Neutral MR-GO conditions.

Issue	Plaintiffs Experts	Defense Experts	Observations
Development of As Was MR-GO Reach 1 Major Breaches (North and South) adjacent to Lower 9th Ward	Surge pressures induced lateral instability of floodwall – sheet pile – levee flood protection structure caused by foundation capacity degradations due to levee reduced cross section (North Breach) and uplift pressure effects. Pressure and uplift effects exacerbated by adjacent EBIA site clearing excavations.	Surge pressures induced lateral instability of floodwall – sheet pile – levee flood protection structure caused by foundation capacity degradation due to levee reduced cross section (North Breach) and overtopping erosion of the adjacent protected side foundation soils (South Breach).	Primary differences between Plaintiffs and Defense Experts analyses are focused in analyses of foundation seepage and uplift pressure effects and inclusion of the exacerbating effects of the adjacent EBIA site clearing excavations. Plaintiffs Experts analyses of the effects of the overtopping erosion did not indicate they were instrumental in initiation of the South Breach.
Development of Neutral MR-GO Reach 2 Major Breaches (North and South) adjacent to Lower 9th Ward	Surge pressures induced lateral instability of floodwall – sheet pile – levee flood protection structure caused by foundation capacity degradations due to due to levee reduced cross section (North Breach), seepage and uplift pressure effects. Pressure, seepage, and uplift effects exacerbated by adjacent EBIA site clearing excavations.	Surge pressures induced lateral instability of floodwall – sheet pile – levee flood protection structure caused by foundation capacity degradation due to levee reduced cross section (North Breach) and overtopping erosion of the adjacent protected side foundation soils (South Breach).	Primary differences between Plaintiffs and Defense Experts analyses are focused in analyses of foundation seepage and uplift pressure effects and inclusion of the exacerbating effects of the adjacent EBIA site clearing excavations. Plaintiffs Experts analyses of the effects of the overtopping erosion did not indicate they were instrumental in initiation of the South Breach.

Summary of Major Technical Concerns

19. Table 2 summarizes my understanding of the major technical elements of concern identified by the Defense Experts in their reviews of the analyses and investigations summarized in my Expert Report of July 2008. My Expert Report of July 2008 addressed development of the major breaches in the man-made flood protection structures along Reach 2 of the MR-GO and along the portion of Reach 1 MR-GO adjacent to the Lower 9th Ward.. The purpose of this summary is to help develop an overview of the primary technical elements of concern documented by the Defense Experts in their Expert Reports.

20. Because of the limitations in the available time and resources for this review cited at the beginning of this Declaration, this summary does not represent a complete and exhaustive summary of all of the important technical elements of concern raised by the Defense Experts. In the following parts of this Declaration, I will summarize my responses to the major technical elements of concern identified by the Defense Experts as defined in Table 2.

21. In the next part of this section, I will summarize my general observations concerning the major technical concerns and issues that have been documented in the cited Defense Expert Reports. In Part II of this Declaration, I address in detail the major concerns raised in the Defense Expert Reports regarding the work documented in my July 2008 Expert Report pertaining to the wave-induced erosion, scour, and breaching of the MR-GO Reach 2 man-made earthen flood protection structures (EBSBs). In Part III of this Declaration, I address in detail the major concerns raided in the Defense Expert Reports regarding the work documented in my July 2008 Expert report pertaining to the breaching that developed in the

hurricane flood protection structures adjacent to the MR-GO Reach 1 segment adjacent to the Lower 9th Ward.

Table 2: Summary of the major technical elements of concern

Major Technical Concerns	Description
Validity of the MR-GO Reach 2 EBSB flood side wave erosion - breaching analyses, conclusions, and opinions	• Quantitative analyses of flood side wave inducing erosion velocities. • Quantitative soil erosion characterizations. • Quantitative assessments of protective vegetation (grass, turf) erosion resistance. • Quantitative analyses of flood side wave induced soil erosion – breach development. • Analyses of post-hurricane photographic, video, LiDAR, and field inspection survey data combined with results from the quantitative analyses to characterize development of the major breaches.
Validity of the MR-GO Reach 1 Lower 9th Ward breach development analyses, conclusions, and opinions	• Characterizations of the hydraulic conductivities of buried marsh layers. • Quantitative analyses of foundation soil hydraulic conductivity (permeability, pressure) effects on stability of the flood protection structures. • Quantitative analyses of flood protection structure surge overtopping protected side soil erosion effects on stability of the flood protection structure at the South Breach. • Analyses of post-hurricane photographic, video, LiDAR, and field inspection survey data combined with results from the quantitative analyses to characterize development of development of the North and South Breaches.

General Observations

Forensic Engineering Limitations – Cognitive Biases

22. My overall evaluation of the work documented by the Plaintiffs and Defense Experts is that there are substantial merits in both sets of expert opinions and conclusions regarding the primary causative factors involved in development of the major breaches in the man-made flood protection structures along the MR-GO Reach 2 and along the MR-GO

Reach 1 adjacent to the Lower 9th Ward that happened during Hurricane Katrina. The analyses, assessments, conclusions, and opinions developed by the Plaintiffs and Defense Experts are based on a very extensive body of technical work. However, in my expert opinion, none of the expert analyses of the complex and inter-related causative factors are free from limitations in knowledge, technology, uncertainties, analytical methods, and very important 'cognitive biases.' Because of these complexities and uncertainties, it is not easy to see through the 'fog' created by the expert's analyses.

23. There are the many types of cognitive biases (e.g. hindsight, confirmation, correlations, wishful thinking, rational, knowledge, beliefs, recall, perceptions, social, organizational) involved in development of the expert analyses of data and information (Bea 2000, 2005, 2009). A very important category of these cognitive biases are those developed by the 'organizational' and 'social' elements. Frequently, these cognitive biases come from social, group, and organizational acceptability, conformance, and achievement incentives (Weick 1995). To be acceptable and recognized by the group or organization, a complex web of incentives and processes are developed to encourage individuals to conform to the social, cultural, organizational 'norms.' These form an intricate network of beliefs, values, and feelings. Often, the cognitive bias identified as 'Group Think' and 'Organizational Distortions' results from such pressures and incentives (Bella 2006, Dorner 1996, Weick 1995). It is not reasonable for any of the experts who are involved in this work to believe that their analyses, conclusions, and opinions are free from such cognitive biases. There is ample evidence in their work to indicate otherwise.

24. Based on my forensic engineering experience and research (and 56 years of experience the other areas of engineering), it is clear that the organizational and social

cognitive biases have been very influential in development of the analyses and conclusions documented by the cited Defense Experts (Ebersole, Resio, Mosher, Wolff). All of these very experienced, knowledgeable, reputable, and recognized experts have very strong 'ties' to the USACE. All currently are or formerly were employees of the USACE. All have strong family – social and professional practice 'ties' to the USACE. The professional career activities, rewards and incentives of all of the cited Defense Experts are heavily dependent on 'approval' and 'acceptance' by the USACE. During development of the Plaintiffs Experts 'team', these issues exerted significant influences on the experts that would or would not work with the Plaintiffs experts. Many were afraid of the 'wrath of the Corps' because their research, professional practice, and professional recognition would be harmed. It is not reasonable to expect that important and influential organizational – social biases (cognitive filters) would not be present in the Defense Expert's analyses, observations, and conclusions. <u>Who we are can't be separated from our social and cultural background.</u>

25. A recent editorial article in *Newsweek* (January 12, 2009, p 17) addressed challenges associated with some of the key cognitive biases associated with the work of 'experts' (scientists and engineers):

> *"Proponents of a particular viewpoint, especially if their reputation is based on the accuracy of that viewpoint, cling to it like a ship-wrecked man to flotsam. Studies that undermine that position, they say, are fatally flawed. In truth, no study is perfect, so it would crazy to chuck an elegant, well-supported theory because one new finding under-cut it. But it's fascinating how scientists (engineers) with an intellectual stake in a particular side of a debate tend to see flaws in studies that undercut their dearly held views, and to interpret and even ignore "facts" to fit their views. No wonder the*

*historian Thomas Khun concluded almost 50 years ago that a scientific paradigm topples only when the last of its powerful adherents dies.” (***On Science** by Sharon Begley, “On Second Thought …”*)*

26. In the work documented in my Expert Reports and Declarations I have attempted to neutralize the wide range of biases that can influence my analyses, deductions, and conclusions (diagnoses). Given that I started my career with the USACE (1954) and was assigned to help “drain the Everglades”, that my father was a career USACE officer – employee (1934-1977), that my family lost all of our belongings and home as a result of the failures of the flood protection system in New Orleans East during Hurricane Betsy (1965), that I have not had any relationships, consulting work, professional or research associated with the USACE since I left the USACE (1959), and that I am a Plaintiffs Expert (2006-2009), I must recognize my potential cognitive biases. These biases must not be allowed to deter me from ‘telling the whole truth and nothing but the truth.’ The primary method I have used to help me neutralize these cognitive biases is one I have termed ‘triangulation’ (reference to the process used by surveyors and navigators to determine geographic locations). This method is based on using data, information, knowledge, analyses and conclusions (diagnoses) from as many reliable and credible sources as possible (Bea 2009). In my triangulation I have attempted to use at least 3 ‘independent’ (not involving the same important potential biases) sources of diagnoses to help formulate and corroborate my deductions and conclusions (e.g. results from the USACE IPET, National Institute of Standards and Technology, and Congressional and White House investigations and studies of failures of the New Orleans flood protection system during Hurricane Katrina). Given the independent sources of data, information knowledge, analyses and conclusions, if I was not

able to triangulate my deductions and conclusions, then I examined the potential deficiencies that could pervade my and the other deductions and conclusions. If those deductions and conclusions were reasonable and justifiable, then I revised my deductions and conclusions and repeated the diagnostic process until I had achieved 'reasonable' agreement. I changed my diagnoses and conclusions given improved knowledge and confirmation of this knowledge by other experts.

27. My review of the analyses, observations, and conclusions documented in the cited Expert Reports does not indicate to me that they have made any similar efforts to recognize or neutralize the potential 'biases' implicitly and explicitly integrated into their expert opinions. In many cases, immature and faulty judgments and opinions have been developed based on incomplete, inaccurate, and superficial understanding of what has been documented in my Expert Reports. This is not accidental. It is intentional.

Forensic Engineering Limitations – Data & Analytical Models

28. There are important and significant limitations in measured data, observations, recordings, and available information developed before, during and following Hurricane Katrina. The multiple 'settings' in which these failures developed are extremely complex. There are important limitations associated with instrumentation, data gathering, processing, and interpretation. In many cases, there is no definitive and objective data available; thus, inference and deductive processes must be used to help 'fill in the gaps' in knowledge.

29. There are similar limitations in the state-of-practice (SOP) and state-of-art (SOA) qualitative and quantitative analytical models used by the Defense and Plaintiffs Experts. Complexities in the actual physical environments, and understanding and analyzing the complex physics and mechanics associated these very complex environments results in

significant uncertainties associated with results from the expert analyses (these points are well summarized in the December 2008 Defense Expert Report written by Dr. Wolff). <u>I have found many instances in which the Defense Experts define 'flaws' and 'limitations' in the work, conclusions and opinions developed by the Plaintiffs Experts that are also clearly present in the work, conclusions and opinions developed by the Defense Experts. Yet, the Defense Experts have not recognized nor attempted to ameliorate these same 'flaws' and 'limitations' in the work documented in their December 2008 Expert Reports.</u> Given the nature of the deliberations and the stage of development of the SOP and SOA of forensic engineering, differences resulting from the Defense Experts and Plaintiffs Experts analyses, interpretations, conclusions, and opinions should be expected (Hale et al 1997, Dorner 1996).

30. An issue of critical importance in 'forensic engineering' regards 'analytical models - how these models are assembled and developed. Traditionally, engineers are trained to use 'decomposition' oriented analytical models. The theme of these models is if one wants to understand something (a 'system'), then divide the system into small parts (components), develop an understanding of the parts, and then assemble components to develop an understanding of the system. Recognition of flaws embedded in the traditional engineering 'decomposition' approach has led to development of 'system analytical approaches.' The theme of system analytical approaches is development of understanding of system 'parts and pieces' must be preceded by development of a comprehensive understanding of the performance of the system and how the parts and pieces of the system are inter-related and interact to develop the performance of the system; this is termed 'system synthesis analysis,' Once the system synthesis analysis has been completed (understanding the 'forest'), then traditional decomposition analytical approaches can be employed to enhance understanding of

the components that comprise the system: synthesis before decomposition (understand the forest before attempting to understand the trees, branches, leaves, and roots).

31. One should not expect to make proper ‘sense’ of the performance and behavior of complex systems by choosing a few individual components that comprise the system, analyzing the performance of these components, and then deducing the performance of the system based on the understanding the performance of ’selected’ individual components. The result of this approach is that the wrong individual components are selected, incompletely understood, and then assembled incorrectly. The insights developed and the conclusions reached as a result of such an approach are deeply flawed. Subtle interactions and ‘boundary conditions’ associated with the system components will contribute to development of incorrect deductions and analyses when ‘decomposition’ selected components’ approaches are used; this is an intentional process of selecting the components and facts that support a preconceived ‘picture’ of how a system performs.

32. In development of my analyses, observations, conclusions, and opinions, in all cases, I have attempted to understand a particular ‘system’ (e.g. flood protection structure at the Lower 9th Ward) before I tried to understand the elements comprising the: synthesis before decomposition. I have not observed a similar process in much of the work documented by the Defense Experts. Frequently, analyses have been prematurely focused on a few selected specific details (premature decomposition) and conclusions drawn from these detail-focused analyses. The flawed details are then assembled in an attempt to understand the performance of the system. Important interactions and inter-relationships between the elements and components involved in the system performance are not captured in this ‘reverse forensic engineering’ process: <u>start with the answer you think is right, and then work backward to</u>

‘prove’ the answer. In forensic engineering, this is a deadly approach that can and will only produce dense ‘fog’ that obscures attempts to see the truth.

Forensic Engineering Limitations – Experts & Expert Teams

33. An important part of forensic engineering regards definition of the term ‘expert.’ The most important parts of expertise are technical knowledge, skills, applications and experience that pertain to the matter at hand (Bea 2000, 2005, 2009). Different expert engineers have different ‘areas’ and ‘depths’ of ‘expertise.’ The areas of expert engineering include research (basic), development (applied research), design, construction, operations, maintenance, decommissioning, management, and forensic engineering. The term ‘depths’ refers to the extent of development of applicable knowledge, skills, applications and experience that pertain to the matter at hand. In this case, one of the major areas of importance is expertise in ‘forensic engineering’. Vick observes (2002):

> *“Substantive expertise can be divided into two components. One is how much a person knows – the size of one’s domain-knowledge database. The other is how this knowledge is accessed – the search algorithm used to summon it for the problems at hand. Experts plainly possess more domain knowledge than novices, but they access it differently too.*

I have summarized the attributes of ‘experts’ as defined and described by Vick (2002) and Klein (1999) in Table 3.

Table 3: Summary of characteristics of forensic engineering 'experts'

Vick (2002) defines experts as those people who: • Are quicker and more accurate at 'sensemaking' (forward and backward reasoning), • Have better self-knowledge (self-monitoring), • Anticipate (know what to expect, think and plan ahead), • See the problem at deeper levels (multiple interactive mental models, search for subtle clues), • Develop penetrating insights (access to greater repertoire of problem representations), • Recognize expertise is domain-specific and limited (understand their limitations)
Klein (199) defines experts as those people who comprehend and understand: • Patterns (deep knowledge and experience based mental models, intuition), • Anomalies (clues and evidence that does not match the mental models), • Situations (active and inactive physical and social environments), • Workings (how things have happened and can happen), • Opportunities (high leverage points in developments enabling major changes), • Improvisation (thinking out of the box in innovative and creative ways), • Past and future events (students of history, deep insight, simulations of what did happen and could happen), • Small differences (subtle clues indicating plausibility and veracity), • Own limitations (know the borders of their knowledge, experience, and capacities), and • Thinking about thinking (reflective, backward reasoning).

The expert opinions of <u>all</u> of the 'experts' involved in these deliberations should be evaluated based on these proven criteria.

34. Another one of the important elements in forensic engineering investigations is 'requisite variety' represented by and in the expertise of the investigation team (Taleb 2007; Hale et al 1997; Dorner 1996; Klein 1998; Center for Chemical Process Safety 1994; ASCE 1989). The variety (knowledge, skills, experience) represented in the forensic engineering team must be able to match the variety of factors and elements represented in the failures being investigated. This variety encourages deliberations based on different viewpoints with

an objective of finding what is right, not who is right. This is a form of 'triangulation' to improve the chances that correct diagnoses are performed and that correct conclusions and opinions are developed from the diagnoses. Guidelines have been developed to help determine how to comprise effective forensic engineering teams and how such teams should conduct their work (ASCE 1989). NASA has developed similar guidelines for investigations of failures associated with space flight operations and these were applied during the Columbia Accident Investigation (CAIB 2003; Bea 2009). Other organizations concerned with public safety (e.g. US National Transportation Safety Board, US Chemical Safety Board, UK Health and Safety Executive) have developed similar guidelines for conduct of forensic engineering investigations.

35. During the past two decades, I have served as a principal in investigation of approximately 30 major failures of engineered systems including the Piper Alpha (North Sea) and Goodwyn (Northwest Shelf of Australia) offshore platforms, the tankers Exxon Valdez and Braer, the NASA Columbia space shuttle, and the Longford (Exxon Australia) refinery explosions. I was a principal investigator in the National Science Foundation sponsored Independent Levee Investigation Team (ILIT) investigation of the failures of the flood protection system for the Greater New Orleans Area that developed during Hurricane Katrina (primary responsibilities for the forensic engineering risk analysis, human, organizational, and institutional aspects). During the past 9 months, I have served as a Principal Investigator in gathering data, performing field investigations, and performing analyses of the breaches that developed in the Mississippi River and its tributary levees during the July 2008 flooding of the mid-West States of Iowa, Missouri, and Illinois (Storesund et al 2008). This investigation also is sponsored by the National Science Foundation.

36. Before 1988, I was involved as a principal investigator in approximately 20 major failures including the U.S. Air Force Texas Tower offshore radar platform (located offshore the coast of New York; failed 1961), failure of the mobile offshore drilling unit Ranger I (1977), and Shell Oil Company's South Pass Block 70 platforms (located in the Mississippi River Delta; failed during Hurricane Camille in 1969).

37. This work and the associated research involved study and analysis of more than 600 failures of engineered systems (Bea 2000, 2009). This work has resulted in a large number of peer reviewed publications including the text titled *Human and Organization Factors: Risk Assessment and Management of Engineered Systems* (2009). See my Vita (2009) for a listing of my many other journal publications that relate to my 'forensic engineering' work. I am a member of the American Society of Civil Engineers (ASCE) Technical Council on Forensic Engineering. At the present time, the Technical Council on Forensic Engineering is revising the ASCE guidelines titled *Guidelines for Failure Investigation* (1989). This guideline and the similar guidelines cited should be required reading for the experts engaged in these deliberations. My background and experience of more than 55 years qualifies me to perform the forensic engineering work documented in the Declarations, Technical Reports, and technical publications I have written.

38. While the cited Defense Experts clearly have significant breadth of expertise and depth of this expertise (they are recognized and respected as engineering experts), <u>I have not detected significant expertise in 'forensic engineering.</u>' Adequate expertise in research (basic and applied), management, and design engineering does not equate to adequate expertise in 'forensic engineering' (ASCE 1987, CAIB 2003, Perrow 1999).

Forensic Engineering Analytical Models – Validity and Validation

39. The Defense Experts have expressed repeatedly important concerns about the validity of forensic engineering analytical methods and processes used during the analyses documented in my July 2008 Expert Report. It is important to recognize that these same concerns are applicable to the engineering analytical methods and processes used during the analyses documented in the December 2008 Defense Expert Reports. The Defense Expert Reports are replete of evidence that these validity concerns have been adequately addressed. In development of a discussion regarding validity of engineering analytical models and processes, it is important to define some key terms:

- Valid - being supported by objective truth or generally accepted authority, based on flawless reasoning and solid ground, well grounded, sound, having a conclusion correctly derived from premises, cogent, convincing.
- Reliable - suitable or fit to be relied upon, trustworthy, worthy of full confidence, dependable. A reliable method is one that yields valid and consistent results upon repeated use. A reliable method is suitable for its intended purposes.

40. Campbell and Stanley (1963) have addressed the approaches that can be used to establish the validity of engineering analytical methods and processes. There are two approaches: 1) external, and 2) internal.

- External validity is the extent to which the method (approach) is generalizable or transferable. A method's generalizability is the degree the results of its application to a sample population can be attributed to the larger population. A method's transferability is the degree the method's results in one application can be applied in another similar application.

- Internal validity is the basic minimum without which the method is uninterpretable. Internal validity of a method addresses the rigor with which a method is conducted - how it is designed, the care taken to conduct measurements, and decisions concerning what was and wasn't measured. There are four different types of internal validity: 1) face, 2) content, 3) criterion-related, and 4) construct.

41. Face validity is the degree to which a method appears to be appropriate for doing what it intends to do. Face validity is based on justifications provided by the state-of-art and state-of-practice knowledge and experience. Content validity addresses the degree to which the method addresses the problem (issue) it is intended to address. Criterion validity addresses the degree to which the method allows for assessment of an issue or problem beyond the testing situation; the generalizability of the method. Criterion validity may be concurrent or predictive; the evaluation may be either be intended to assess a criterion independently evaluated at the same time (concurrent), or to predict achieving a criterion in the future (predictive). Construct validity addresses the degree to which the results of the method can be accounted for by the explanatory constructs of a sound theory. A method's construct validity can be assessed by specifying the theoretical relationships between the concepts and then examining the empirical relationships between the measures of the concepts, and then interpreting how the observed evidence clarifies the concepts being addressed. Construct validity is demonstrated when measures that are theoretically predicted to be highly interrelated are shown in practice to be highly interrelated.

42. Kardon, Bea, and Williamson (2006) applied these concepts to evaluation of the validity of methods used originally by the USACE to determine the lateral stability of the flood protection structure at the 17th Street Canal breach. This study concluded:

"Important failure modes in the 17th Street canal levee-floodwall system components were not recognized. The combination of methods used to perform the design analyses were neither valid nor reliable. When the system was tested, it failed."

43. Other validation and validity considerations associated with forensic engineering analytical models include (*'Daubert Tests'*) (Mahle 2008, Strickland 2008):

- Testing – has the theory or methodology been tested or can it be, in accordance with the Scientific Method?
- Rate of Error – What is the known or potential rate of error in an expert's methodology?
- Peer Review – Has the theory or technique been published in peer reviewed journals?
- General Acceptance – Has the theory or methodology gained general acceptance in the relevant scientific - engineering community?

44. Peer Review is a particularly important part of the validation process. I have authored or co-authored 21 refereed journal and conference papers that pertain to the failures of the Greater New Orleans flood protection structures during Hurricane Katrina. These publications are cited in my Vita (2009) and herein. Sixteen of these publications pertain to analyses of the performance characteristics of the flood protection structures and 5 pertain to analyses of the risk management, human, organizational, and institutional aspects that were an integral part of development of the failures. Five of these publications pertain specifically to the wave initiated breaching of the Reach 2 EBSBs and 5 of these publications pertain specifically to the failure of the flood protection system at the Lower 9th Ward.

45. I have not been able to find or find reference to a single peer reviewed journal publication authored by Dr. Mosher, Dr. Wolff, Dr. Resio, and Mr. Ebersole that pertain to the performance characteristics of the Greater New Orleans *flood protection structures* during

Hurricane Katrina. Based on the information cited in the December 2008 Expert Reports, Dr. Resio and Mr. Ebersole have published 4 papers (total) that pertain to the environmental characteristics (winds, waves, currents, surge) associated with Hurricane Katrina. Thus, the background information presented in the cited Expert Reports would fail to pass this part of the *Daubert Tests* as they are related to performance of the flood protection structures during Hurricane Katrina.

46. During the forensic engineering work documented in my July 2008 Expert Report and in this Expert Report, the foregoing processes have been applied to establish the validity and reliability of the analytical models (both quantitative and qualitative). The next parts of this Declaration will further detail validations of the wave induced erosion analytical models and lateral stability – hydraulic conductivity analytical models employed in these studies.

Development of Failures: Time and Multi-Modal Processes

47. Two general sets of comments developed in the Defense Expert Reports apply to 'multi-modal time dependent failure development processes' and to 'uncertainties' associated with analyses of the failure development processes. My experience associated with forensic engineering studies of failures of engineered structures and systems clearly indicates that in the vast majority cases, many 'modes' of failure are involved during development of a failure. A failure can be initiated by a certain mode of failure (e.g. failure of an element or component that comprises the engineered system). As the failure continues to develop and propagate within the system, other modes of failure can be activated and involved. By the conclusion of the failure, many interacting modes of failure can be involved. This sequence of interacting failure modes can be very difficult to properly understand after the failure has

occurred. I have expressed this difficulty with the observation that "you can not tell which way a train went by looking at the tracks." That observation is intended to point out that other 'clues' and evidence must be gathered and properly analyzed to determine which way the train went. This is a diagnostic process focused on <u>'connecting the proper dots properly'</u> to develop plausible and probable deductions of 'causes and effects' (Hale et al 1997). <u>Developing a reasonable understanding of the failure development process is not so much about gathering data and information as it is about properly analyzing and understanding the data and information.</u>

48. Another part of this challenge is associated with identifying the multiple modes of failure that can be active or activated during development of a failure. Rarely is only one mode of failure present in development of failure of a complex system. I have likened this process to a 'photo finish horse race without a camera.' There are many horses (failure modes) involved during the race (development of the failure), and at the end of the race (completed failure), there can be one or more horses that win the race. The forensic engineer does not have a camera that shows which horses were involved nor which horse or horses 'won' the failure race. As a result, generally it is not possible to develop 'unambiguous' – deterministic – characterizations of failures of complex engineered systems, particularly when they are embedded in similarly complex natural environmental systems and settings. All of the potentially active modes of failure must be recognized and analyzed to develop a realistic understanding of how a system failure develops. In addition, their potential interactions also must be recognized and analyzed. Plausibility – and coherent diagnosis processes should dominate this process.

49. The time dependent characteristics of development of failures of engineered systems (e.g. breaches in flood protection structures) also must be understood. The last phase in development of failure of a complex engineered system can be very abrupt and happen very quickly. Generally, it is this last phase in development of the system failure that is most evident after the failure. However, analyses of the failure of many types of engineered systems – including coastal, offshore, and ocean engineered systems – clearly shows that generally the failure sequence starts with a long 'incubation period' followed by a long 'propagation period' that culminates in the abrupt final phase in evolution of the failure (Bea 2000, 2009).

50. Analyses of time characteristics of the breach development at the 17th Street Canal during Hurricane Katrina clearly showed that this breach took several hours to develop. My forensic engineering analyses of this failure (Bea 2008, Bea and O'Reilly 2009) show that it was (most probably) initiated by an overblown oak tree located at the levee toe at south end of the breach (about 6:00 am CDT). The failure sequence was culminated 4 to 5 hours later (about 1:00 to 11:00 am CDT) with the majority of the breached section of this flood protection structure laterally displaced some 60 feet toward the protected side. The evidence clearly shows that as the levee and foundation soils began to displace laterally and differentially, the concrete flood wall panels tilted and separated at the vertical joints and flood water entered in large volumes through these opened joints. At this same time, hydraulic seepage and uplift pressure effects developed in the foundation soils under the levee thereby reducing its lateral resistance and in the process developing large lateral displacements of the flood protection structure (concrete flood wall, supporting sheet piling, levee, and foundation

soils). Forensic engineering evidence and analysis indicates this was a time-dependent, multi-mode breach development process that took several hours to fully develop.

Forensic Engineering Uncertainties – The End of Certainty

51. Topics associated with uncertainties associated with results from analytical models used to assess development of the MR-GO Reach 2 and Reach 1 breaches have repeatedly and appropriately been raised by the Defense Experts. My Expert Reports, Declarations and Technical Reports have included explicit analyses of some of the important uncertainties associated with some of the analyses. Based on my previous 4 decades of work regarding the reliability characteristics of engineered systems, I have organized these uncertainties into four general categories (Bea 2000, 2005, 2009): 1) Type I – inherent or natural variabilities (information insensitive), 2) Type II – model, parametric, state (information sensitive), 3) Type III – human and organizational performance, and 4) Type IV – information development and utilization.

52. Examples of Type I uncertainties are the strength characteristics of soils and the heights of waves (at given times and places). Examples of Type II uncertainties result from ‘model limitations’ (frequently expressed as ‘assumptions’) such as those associated with 2-dimensional analytical models when applied to 3-dimensional processes and those developing from ‘static’ (not time varying) analyses when applied to ‘dynamic’ time-dependent processes. Examples of Type III uncertainties are those associated with human and organization ‘errors’ (performance malfunctions) such as mistakes, cognitive malfunctions, breakdowns in communications, and intentional and unintentional violations. Examples of Type IV uncertainties include malfunctions in access, development and use of knowledge (unknown knowables) and lack of information (unknown unknowables) (Taleb 2007). My

evaluations of the Hurricane Katrina breach analysis – engineering forensics - work documented in the Plaintiffs and Defense Expert Reports has identified the presence of all four of these types of uncertainties.

53. My work has addressed Type I and Type II uncertainties using several different approaches. One approach has been to perform analyses based on a plausible range of quantifications of important input parameters (e.g wave heights, soil permeabilities) – a parametric variable approach. Another approach I have used is to compare results from our analyses with those from other appropriate (validated, calibrated, reliable) analytical models and with available laboratory and field 'experimental' data – a model validation approach. A third approach I have used is to gather data and information from other qualified 'expert' analyses to help validate and corroborate interpretations of results from the analytical models – a forensic engineering analytical model 'triangulation' approach.

54. Due to limitations in available resources, I have not performed formal analyses of the Type I and Type II uncertainties associated with all of the analytical models used to help develop evaluations of development of breaches in the man-made flood protection structures during Hurricane Katrina. In the case of the breaches that developed at the Lower 9th Ward and at the 17th Street Canal, I have performed formal reliability based evaluations of the Type I uncertainties associated with the applied analytical models. For example, the uncertainties associated with computed lateral stability Factors of Safety (ratio of system 'capacity' to 'design loading') have been explicitly analyzed; the Factors of Safety which have been reported are the Mean (average, 'best estimate') Factors of Safety. The uncertainties associated with this Mean Factor of Safety have been reported to help develop an understanding of the uncertainties associated with results from the analytical models; the

computed Coefficients of Variation (ratio of Standard Deviation to Mean value) of the computed Factors of Safety are in the range of 30% to 40%. These results are in general agreement with those developed by Wolff (1996, 1994) and documented in the USACE Engineer Technical Letter titled “Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies” (USACE 1999). These are also in agreement with those developed by other investigators (e.g. Wu et al 1989, Christian 2004).

55. I have devoted significant time during my career to performing formal analyses of uncertainties associated with analytical models used in coastal, offshore, and ocean engineering (e.g. Bea 1990; work associated with reliability analyses of ocean structure systems; consult the list of published references incorporated with my Vita). Consideration of Type I uncertainties associated with the quantitative analytical models used in the breach development studies I have performed indicates the total uncertainties associated with predictions and evaluations of the development of the major breaches are very large - comparable with those associated with fatigue analyses of coastal, offshore and ocean structures. The magnitude of the Type I uncertainties can be expressed quantitatively with the Coefficient of Variation (ratio of the Standard Deviation to the Mean value).

56. I have estimated that the Type I uncertainties associated with results from the analytical models employed to help evaluate development of the breaches (e.g. timing, participating modes of failure) are in the range of V = 30% to 40%. For further details about development of this evaluation, refer to my Declaration and Technical Report associated with analyses of the breach of the 17th Street Canal (Bea 2008, Bea and Cobos-Roa 2008b) flood protection structure.

57. Type II uncertainties can be expressed with a 'Model Bias' (Bea 2000, 2005, 2009). This Model Bias is defined as the ratio of the true or actual value of a system performance characteristic (e.g. load resistance, displacement at given loading) in prototype conditions ('in the field') to the predicted value (nominal) using a specific analytical model and input parameters. For analysis of results from a particular 'test' (performed in the laboratory or field), an analytical model is used to predict the 'output' (or outputs) from that test. Measured 'input parameters' and 'responses' (outputs from information from the experiments) are used in determination of the Bias. A Model Bias equal to unity indicates that the analytical model (including the input parameters) is able to predict what actually happened (as reflected in the outcomes from the experiments). Under repeated 'tests', the uncertainties associated with the analytical model can be determined and this reliability expressed with the Mean Bias and Coefficient of Variation of the Bias.

58. Based on my previous experience with similar complex models (e.g. those used to predict fatigue cracking and displacements of arctic structure foundations and wave and current forces acting on coastal and offshore structures, consult the references cited in my Vita for additional detail), I have estimated that the Type II uncertainties are in the range of 40% to 60% (Coefficient of Variation) (Bea 2000).

59. Based on this reasoning and analysis, the resultant Type I and Type II uncertainties associated with the breach development analytical models would be in the range of 50% to greater than 70% (Coefficient of Variation). I analyzed the total uncertainties associated with the analyses of wave-induced erosion of the EBSBs and obtained a total uncertainty of 64%. Similar ranges of uncertainties have been developed by other investigators concerned with evaluations of uncertainties associated with performance of

ocean structures in a wide range of environmental conditions (storms, earthquakes, ice movements). For example, uncertainties associated with results from fatigue analyses of coastal and offshore structures have been found to be greater than 100% (Coefficient of Variation).

60. I interpret these results and insights as follows. Given that I have successfully 'un-biased' the results from a particular analytical model (i.e. the results represents Means or 'Best Estimate' results), I could estimate the likelihood that the true result for a particular 'test' is greater than or less than the Mean value. Given that it is reasonable to assume that the potential results from the analytical model are Lognormally Distributed (results are well characterized with a Normal Distribution of the Logarithms of the variate; Lognormal Distribution chosen because the resultant variable is the product of a series of random variables), and given I have computed that the 'best estimate' (mean) wave EBSB erosion distance at a given time and location is 100 feet and that the Type I and Type II uncertainties in this analysis have a Coefficient of Variation of 64%, then the 70th percentile and 30th percentile values would be approximately 160 and 30 feet, respectively. The 'true value' would have a probability of 70% of being equal to or less than 160 feet and a 50% chance of being equal to or less than 80 feet.

61. This discussion is intended to highlight difficulties associated with 'deterministic' (did or did not, would or would not, is or is not) interpretations of results from analyses (inductive, deductive, qualitative, quantitative) of development of the breaches in the man-made flood protection structures during Hurricane Katrina. Instead, interpretations that incorporate adequate understanding of the uncertainties that are associated with the results from the <u>ALL</u> of the analyses should be used – this represents "The End of Certainty"

(Prigogine 1997). In my Declarations and Technical Reports, I have attempted to focus my linguistic terms (e.g. “would or did develop”) on those results that my assessments indicate as ‘most probable’ – possessing the highest likelihoods of occurrence. The Defense Experts should be encouraged to develop assessments of the Type I and Type II uncertainties associated with their analytical models and to state their conclusions in terms of the resulting uncertainties; thus far in their work, there has been no evidence of characterizations of the uncertainties associated with results from their analyses.

II. PERFORMANCE OF THE MR-GO REACH 2 MAN-MADE FLOOD PROTECTION STRUCTURES DURING HURRICANE KATRINA

62. This section addresses the major concerns documented in the cited Expert Reports concerning the analytical models applied in our studies of wave-induced breaching of the MR-GO EBSBs. My responses to these concerns are further detailed in Appendix B. In this section, I also will address major conclusions documented in the cited Expert Reports that pertain to the breaches that developed adjacent to the navigation – water control structures located at Bayou Bievenue and Bayou Dupre.

Analyses of Wave-Induced Erosion of EBSBs

63. The wave-induced erosion analytical model applied in our studies of performance of the Reach 2 EBSBs was used to develop quantitative evaluations of the magnitude of erosion of the flood side faces of the EBSB Study Location – specifically the horizontal distance over which significant erosion of the flood side face of the EBSB was developed by wave-induced scour during Hurricane Katrina.

64. Numerical modeling techniques were used to simulate wave attack, using the LS-DYNA software package. Aerial LiDAR data from before and after Hurricane Katrina was the basis for the geometric configuration (topography) before and after the hurricane. Samples gathered from the faces of the EBSB at the defined Study Location (MR-GO Station 497+00) were tested in the laboratory Erosion Function Apparatus (EFA). The EFA test results were used to characterize the erodibility of EBSB materials at the Study Location (Briaud 2008, ILIT). Interaction wave shear velocities (parallel to the EBSB face) were evaluated for a time history of storm surge elevations and wave characteristics (Significant

Wave Heights, Peak Spectral Periods). These interaction shear velocities were then used to calculate the magnitude of lateral erosion.

65. Calculation of the wave induced lateral erosion of the EBSB Study Location was evaluated based on a three-step process:

(1) velocity profiles were generated using the LS-DYNA computer program (assuming actual geometric configuration) 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 based on the durations and magnitudes of the waves; 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 estimate the magnitude of lateral erosion.

66. Wave-induced cumulative erosion of EBSB materials (E) 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 determined for very small time intervals (0.01 seconds) calculated at the defined storm surge elevations. The erodibility at the time step ($\dot{e}(v)$) was multiplied by the value of the time step to calculate the lateral extent of erosion (EΔt) and these incremental values were then summed to obtain the lateral erosion: $\mathrm{E} = \sum \dot{e}(v(t))\Delta t$ (Figure 1). A plot was then generated (e.g. Figure 2) displaying the calculated lateral erosion as a function of time as a function of a range of parameters to characterize the EBSB Wave Erosion Study Location (geometry, grass cover, erodibility, and hydrodynamics).

67. To review, the flood-side wave-induced erosion method consists of the following steps:

(1) Define geometric configuration (topography of EBSB);

(2) Define hydrodynamics (significant wave height, peak spectral wave period, wave direction, storm surge) during the hurricane;

(3) Characterize the EBSB surface (grass cover);

(4) Characterize the EBSB erodibility;

(5) Analyze rush-up and rush-down wave-induced shear velocities on the EBSB flood side face using the computer program LS-DYNA;

(6) Calculate the cumulative lateral erosion (summation of erosion computed from small time steps of velocities at a given surge elevation for a given sea state.

68. The following factors were addressed during development of validations of this analytical model:

(a) Testing – has the analytical method been tested or can it be, in accordance with the Scientific Method?

(b) Rate of Error – what is the known about the potential rate of error in the methodology?

(c) Peer Review – have the analytical method and applications been published in peer reviewed journals or conference proceedings?

(d) General Acceptance – has the analytical method gained 'general acceptance' in the relevant scientific and engineering community?

69. An assessment of these factors as they relate to the flood-side wave-induced erosion analytical method is summarized in Table 4. The flood-side wave-induced erosion

method steps are shown in the left-hand column, and the assessment factors are represented in the subsequent columns. The references cited in this Table are included in the list of References at the end of this Declaration.

70. The flood-side wave-induced erosion method consists of the aggregation of multiple scientific and engineering State of the Practice and State of the Art methods that have a proven track records, are well-published, and are well accepted within the engineering community. The identification of non-overtopping, wave-induced erosion and resulting breaching of coastal flood protection dikes is well documented in the literature. The development of flood-side wave-induced erosion method was not intended to argue the validity of non-overtopping wave-induced erosion (as mentioned earlier, this failure mode has already been identified and accepted within the engineering community), rather to provide a means by which to quantify the magnitude of erosion experienced, using an aggregation of accepted scientific and engineering methods. No new hypotheses are associated with the development or execution of the flood-side wave-induced erosion method. The aggregation of individual accepted scientific and engineering methods into the flood-side wave-induced erosion method can be and has been tested in accordance with the Scientific Method.

71. The rate of error, or ‘level of confidence’ has been identified in the documentation of the flood-side wave-induced erosion method. The controlling aspect of error associated with the method is the high degree of uncertainty in the input parameters. This subject has been addressed earlier in this Declaration.

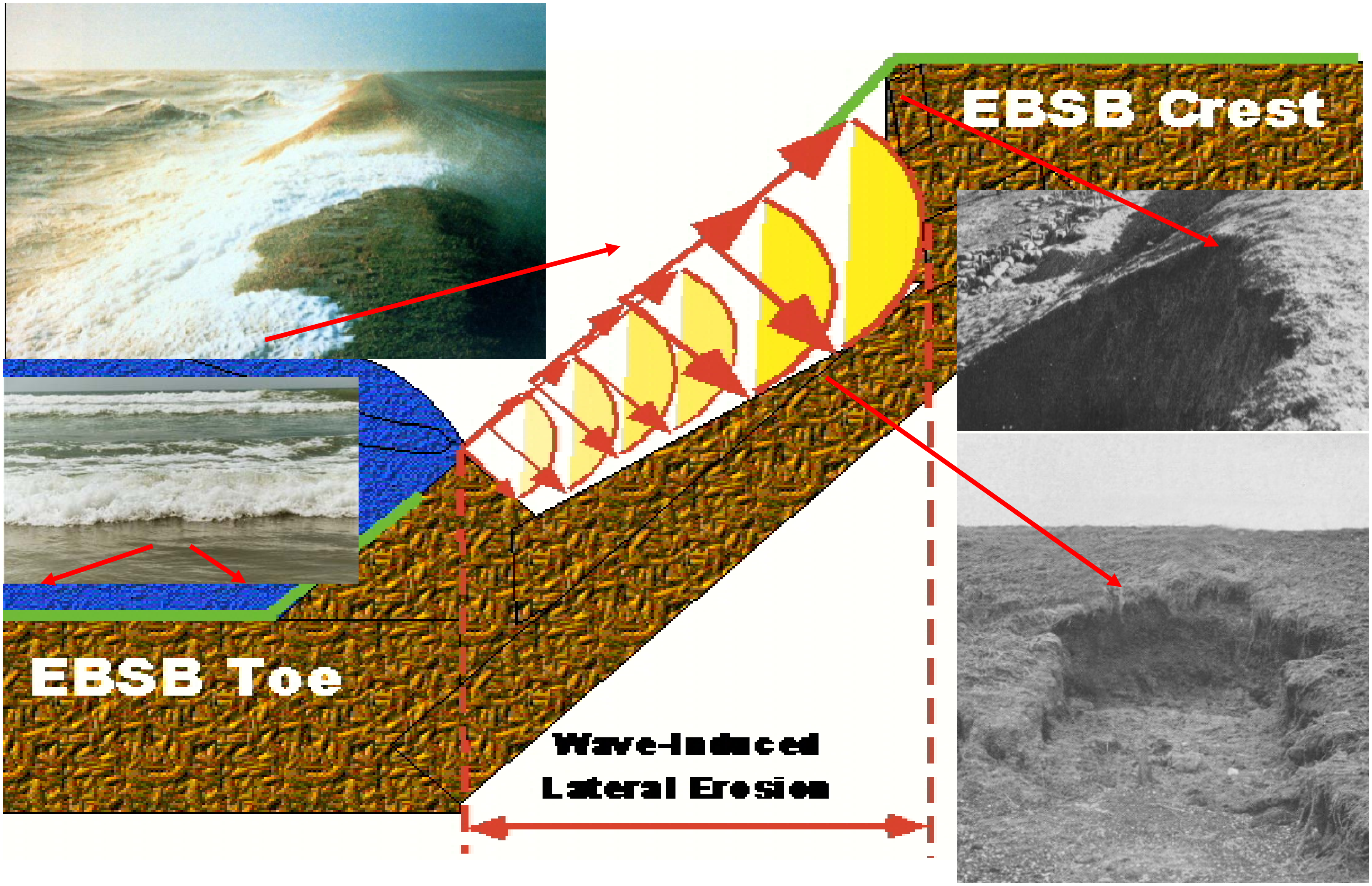


Figure 1: Illustration of analytical model employed to determine the lateral extent of wave-induced erosion of waterside face of earthen flood protection structures.

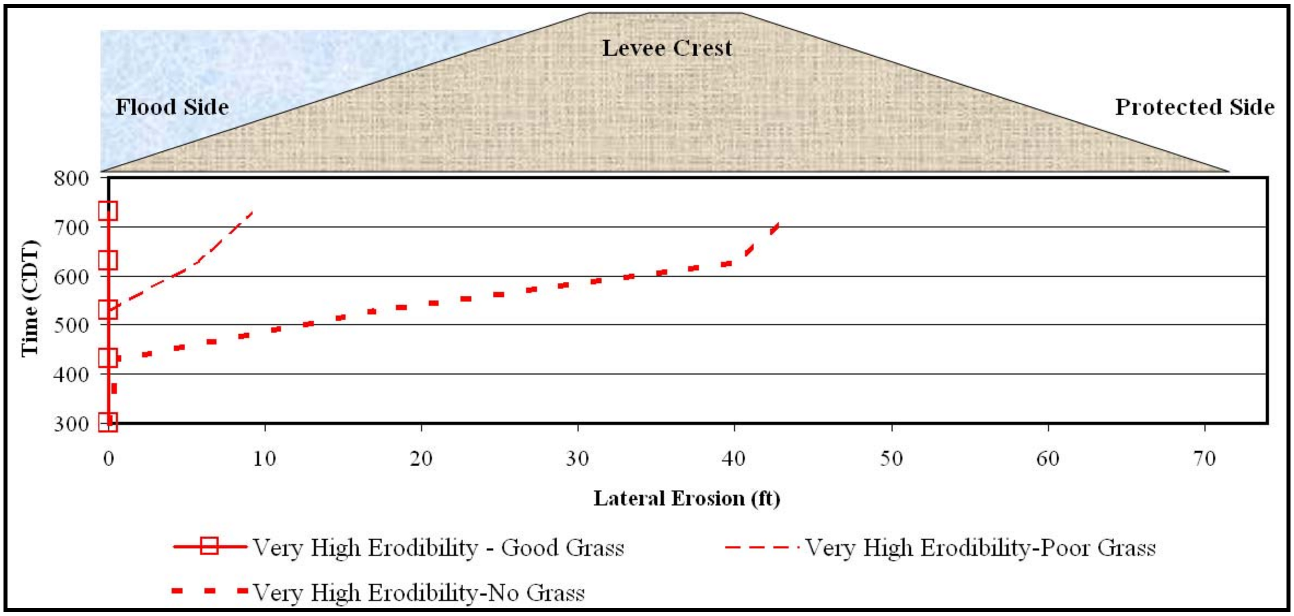


Figure 2: A plot of lateral erosion induced from wave-attack on the MR-GO EBSB for the Scenario 2C hydrodynamic conditions with very high erodibility materials and different levels of grass armor (good grass turf, poor grass turf, and no grass turf).

72. Multiple flood-side wave-induced erosion method publications have been prepared and submitted to peer-reviewed technical journals. The preparation and submission of these publications occurred immediately following completion of the analyses and it is anticipated that these technical articles will be accepted and published during 2009 (see References for citations of these publications).

73. The development and utilization of the flood-side wave-induced erosion method was a direct result of active research and development activities engaged in by the author assisted by graduate students at the University of California Berkeley. The developers physically inspected the site, collected and reviewed the results of soil characterization samples, reviewed the hydrodynamic characteristics, and reviewed LiDAR topographic information at the site. No extrapolation was used for this method. All values were interpolations based upon accepted scientific and technical methods. A detailed discussion and corresponding technical engineering analyses of alternative explanations have been

presented outlining the flood-side wave-induced erosion method and its results. The analyses, application of the flood-side wave-induced erosion method, and results are made in accordance with generally accepted professional scientific and engineering principles and practices.

74. The following part of this section and Technical Report II further details validations of the procedures, processes, and parametric characterizations used in our analyses of the MR-GO Reach 2 EBSB flood side wave erosion and breach development. These parts address the following primary quantitative wave erosion and breach development analysis components that comprise the "EBSB Wave Erosion Model":

1. Wave inducing erosion velocities,
2. Soil erosion characterizations,
3. Protective vegetation (grass, turf) erosion resistance characterizations, and
4. Flood side wave induced soil erosion – breach development.

Table 4: Evaluation of Analytical Method Validity Assessment Factors

Flood-side Wave-Induced Erosion Method		Validity Assessment Factors						
					General Acceptance			
Step No.	Description	Testing	Rate of Error	Peer Review	Direct Research	Unjustifiable Extrapolation	Alternative Explanations	Standard of Care
1	Define geometric configuration (topography of EBSB)	Extensive testing and validation.	Industry established rates of error exist.	See List A in References for a sample of peer-reviewed articles.	Utilization of Aerial LiDAR for topographic evaluations is well accepted within the scientific and technical communities.			
2	Define hydrodynamics (significant wave height, wave period, wave direction, storm surge) during the hurricane	Extensive testing and validation.	Industry established rates of error exist.	See List B in References for a sample of peer-reviewed articles.	Utilization of the ADCIRC, SWAN, and FINEL models to generate hydrodynamic characteristics is well accepted within the scientific and technical communities.			
3	Characterize the EBSB armoring (grass cover)	Extensive testing and validation.	Industry established rates of error exist.	See List C in References for a sample of peer-reviewed articles.	Utilization of correlations between grass cover types and erosion resistance is well accepted within the scientific and technical communities.			
4	Characterize the EBSB erodibility	Extensive testing and validation.	Industry established rates of error exist.	See List D in References for a sample of peer-reviewed articles.	Utilization of erodibility relations for different soil types (silts, clays, and sands) is well accepted within the scientific and technical communities.			
5	Analyze rush-up and rush-down wave-induced shear velocities on the EBSB flood-side face using the computer program LSDYNA	Extensive testing and validation of rush-up and rush-down velocities.	Industry established rates of error exist.	See List E in References for a sample of peer-reviewed articles.	Utilization of formulas to estimate rush-up and rush-down velocities is well accepted within the scientific and technical communities. Additionally, the computer program LS-DYNA is accepted as an appropriate tool for the detailed technical analysis of fluid-structure interactions.			
6	Utilize the calculated shear velocities, flood-side armoring, and erodibility characteristics to calculate lateral erosion a) velocity profiles were generated using LSDYNA (assuming actual geometric configuration) at discrete time intervals; b) 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 c) estimated soil erodibility characteristics were used, in combination with the estimated wave-induced flood face velocities, to estimate the magnitude of lateral erosion.	Testing of the method is possible provided the appropriate model input values are provided. Very few case studies exist that document all required input values. Two validation runs have been conducted using the method: post-Hurricane Gustav, results from Hughes (2008) and Stanczak (2008). These validation analyses provide strong support that the method is accurate and reliable.	Rates of error can be calculated and are a function of the errors associated with the model input. Techniques such as First Order Second Moment or Monte Carlo Simulation techniques can be used to describe the aggregate model error.	Multiple flood-side wave-induced erosion method publications have been prepared and submitted to peer reviewed technical journals. The preparation and submission of these publications occurred immediately following completion of the analyses and it is anticipated that these technical articles will be accepted and published by late 2009.	The development and utilization of the flood-side wave-induced erosion method was a direct result of active research and development activities engaged in by the authors. The authors physically inspected the site, collected and reviewed the results of soil characterization samples, reviewed the hydrodynamic characteristics, and reviewed LIDAR topographic information at the site.	No extrapolation was used for this method. All values were interpolations based upon accepted scientific and technical methods.	A detailed discussion and corresponding technical engineering analyses of alternative explanations have been presented in the same document outlining the flood-side wave-induced erosion method and its results.	The analyses, application of the flood-side wave-induced erosion method, and results are made in accordance with generally accepted professional scientific and engineering principles and practices.

75. My July 2008 Expert Report, Declaration (I) and Technical Report (I) that addressed the wave and overtopping induced erosion of the MR-GO Reach 2 EBSBs included two validation case studies. The first was focused on the "EBSB Study Location" (Station 497+00) south of Bayou Bievenue and another nearby location north of Bayou Bienvenue. The EBSB Study Location was breached during Hurricane Katrina and the nearby location north of Bayou Bienvenue did not breach during Hurricane Katrina. Both locations were subjected to similar environmental conditions. However, the man-made earthen structure at the EBSB Study Location was comprised of uncompacted (received no systematic mechanical compaction) dredge spoil (from the construction and maintenance of the adjacent MR-GO). The man-made earthen structure at the nearby location north of Bayou Bienvenue was constructed with cohesive soils transported to the location and compacted following placement. The wave induced breach development analysis process was applied to analyses of the performance of the man-made earthen structures at these two locations. A range of plausible environmental 'loading' conditions (surge, waves, currents) and earthen structure conditions (cross section geometry, grass-turf protection, soil erodibility) were addressed using the analytical process (July 2008 Expert Report by Bea). Results from the analytical process were in conformance with the investigator's interpretations of the available data and information gathered before and after Hurricane Katrina (e.g. photographic, video, LiDAR surveys, bathymetric surveys, field inspections); the EBSB study location breached and the location north of Bayou Bienvenue did not breach.

76. Relative to these EBSB Wave Erosion Model validation studies, it is important to note that I did not assume that the elevations and properties of the EBSBs were constant along Reach 2 (December 2008 Expert Report by Dr. Resio). The EBSB Study Location was the focus of my quantitative analyses of wave induced and surge and wave induced development of the

observed breaching <u>at this location</u> (Bea, Cobos-Roa, and Storesund 2008, Bea and Storesund 2008, Storesund, Bea and Huang 2008). We termed this development as EBSB wave induced 'crenellation.' We used available data (LiDAR surveys, photographs, videos, field observations, soil borings) and results from other available sources (e.g. IPET, ILIT, and Team Louisiana reports) to 'extrapolate' these results to the other portions of the Reach 2 EBSBs. Performance of the sheet pile repaired sections of the EBSBs was not addressed in detail (no indication of breaching during Hurricane Katrina; overtopped, failed section in sheet pile interlocks during outflow of flood waters following passage of Hurricane Katrina). Performance of the interfaces between the EBSBs and the navigation – flood control structures at Bayous Dupre and Bievenue were addressed and it was concluded that the breaches had developed close to the time of the occurrence of the peak surge. These analyses and conclusions are documented in my July 2008 Declaration (Part 1) and the associated Technical Reports. These analyses and conclusions also have been included in two peer reviewed - refereed journal papers which have been reviewed and are currently in publication (Bea and Storesund 2008; Storesund, Bea, and Huang 2008). The timing and extent of development of the Reach 2 man-made flood protection structure breaches were then used for the subsequent analyses of the flooding of the St-Bernard – Lower 9th Ward Polder.

77. During development of the analytical model employed to evaluate the quantitative effects of flood side wave erosion of the man-made earthen structures, the general issues identified in Table 2 were addressed. Each step in the quantitative analytical process was validated to the extent possible (e.g. verifying the wave forms and erosion velocities, the soil erosion characteristics, and the surge-wave-EBSB 'interactions' during the time history leading up to surge overtopping). For example, we used published (and peer reviewed) methods to

validate the wave up-rush and down-rush velocities including those published in the USACE Coastal Engineering Manual (2006) and those published as part of the USACE sponsored IPET study – specifically the COLWAVE analysis results. These additional validations are documented in my July 2008 Declaration and Technical Report. As pointed out in the Defense Expert Report by Dr. Wolff, it is of critical importance to validate all aspects of numerical analytical models to help assure the reliability and reality of results from such models. A mantra of an experience engineer is: "trust no analytical model until it has been properly calibrated and validated." The critiques provided by Mr. Ebersole and Dr. Resio also have addressed many of the important points that must be addressed in development and validation of numerical analytical models used to simulate results from complex engineered systems (e.g. the MR-GO EBSBs) embedded in an even more complex natural environment (e.g. Hurricane Katrina).

78. This section and Technical Report 2, which accompanies this Declaration, summarizes additional studies performed during development of the EBSB Wave Erosion Model. These additional studies have not been documented in my previous Expert Reports, Declarations and Technical Reports. For brevity, this analytical model will be termed herein as the "EBSB Wave Erosion Model.' The fundamental purpose of the studies have been to validate the EBSB Wave Erosion Model through comparisons with a field case history (Hurricane Gustav, Technical Report III), data from laboratory experiments (Hughes 2007, D'Elsio2007, 2006a – 2006c), and two validated wave erosion analytical models (Kriebel-Dean 1993 and USACE CEM Chapter 3, 2006, Allsop et al 2007, Stanczak, Oumerachi, and Kortenahus, 2006a, 2006b, 2007a – 2007c, and Starzack 2008, Vrijling 1987).

79. It is important to understand that the fundamental goal of the EBSB Wave Erosion Model is to provide a means to perform quantitative analyses (numerical simulations) of

the rate and magnitude (vertical and horizontal extents) of erosion of the flood side face of the EBSBs as a result of wave action. Other analytical models were used in our work to characterize the development of the breaches due to 'overtopping' of the EBSBs by the hurricane surge and waves. It is not a fundamental goal nor an objective to analyze 'details' (e.g. the shape of erosion – deposition features) of the wave induced erosion.

80. <u>My review has not disclosed that the cited Defense Expert Reports contain results from any quantitative analyses of wave induced erosion of the flood faces or protected faces of the EBSBs</u>. Inductive and subjective reasoning has been used by the Defense Experts to combine results from observations, analyses of environmental conditions (e.g. surge and wave overtopping velocities), photographs and surveys (e.g. LiDAR, soil borings, results from soil erosion and scour field and laboratory tests) to deduce that there was no significant wave side erosion that could have developed early breaching of the crests (crowns) of the EBSBs. After this first phase in the breaching process, the rising surge waters were able reach and concentrate flow through these breaches, and the surge waters and waves rushing through them were able to rapidly open – widen – and deepen these breaches. Consequently, Hurricane Katrina flood waters were able to enter the St Bernard – Lower 9th Ward Polder in very large volumes early the morning of August 29, 2005 (e.g. 8:30 to 9:00 am CDT).

81. <u>The Defense Expert's inductive and subjective reasoning has been focused and premised on a single mode involved in development of the major breaches in the EBSBs – surge overtopping flows (erosion from the protected side to the water side). Other modes of development have been excluded (e.g. wave induced erosion, seepage exacerbated erosion).</u> It has been assumed that the EBSBs did not experience any significant damage prior to overtopping

by Hurricane Katrina's surge. Even when the Defense Expert's analyses indicated that surge overtopping would not justify such conclusions, they did not alter their conclusions:

> *"Figure 52 indicates a lack of correlation between water height (above levee crown elevation) and erosion depth…"* (page 53 December 2008 Expert Report by Dr. Mosher).

82. In our analyses, the issues associated with wave induced breaching of the Reach 2 EBSBs are of critical importance to develop a coherent and logical explanation of the observed flooding associated with Hurricane Katrina and in determining whether or not the Neutral MR-GO conditions would result in flooding similar to or very different than the observed flooding. My review has not disclosed that the Defense Experts have applied any similar 'coherent' analyses that are able to explain the early development of the breaches (before the time of arrival of the peak surge) and the subsequent flooding reported in St. Bernard Parish (IPET 2007, Team Louisiana 2007). Given the background and analyses documented in my July 2008 Expert Report, clearly the following conclusion reached by Dr. Mosher is not justified or defensible:

> *"There is no evidence to support that wave-induced flood side erosion was a failure mechanism during the hurricane"* (page 7, December 2008 Expert Report by Dr. Mosher).

83. The issue of the modes of development of the major EBSB breaches is of critical importance in explaining the timing of the observed early and rapid flooding in the St Bernard Parish portion of this Polder. If the flooding came solely from breaches that developed after the peak surge arrived at the Reach 2 EBSBs (e.g 8:00 to 9:00 am CDT), then it has not been explained by the Defense Experts how the surge and waves were able to develop the breaches to the extent that the significant early flooding in St Bernard Parish was explained. The unprotected side wave breaching of the crests of the EBSBs followed by exploitation of these breaches by the

rising hurricane flood waters does lead to a plausible explanation of the observed flooding and its timing.

84. As documented previously in my Expert Report, Declarations and Technical Reports, the EBSB Wave Erosion Model consists of three primary parts:

1. velocity profiles are generated using the computer program LS-DYNA (based on prescribed geometric configurations) at discrete time intervals;
2. resistance to wave-induced erosion as a result of grass cover (turf) on the flood face of the EBSB is determined and no wave-induced erosion is applied until the grass cover protection has failed; and
3. laboratory based testing soil erodibility characteristics are used, in combination with the estimated wave-induced flood face velocities, to estimate the magnitude of lateral erosion.

The details of each of these steps are documented in my July 2008 Declarations and Technical Reports.

85. The EBSB Wave Erosion Model is founded similar analytical models and procedures used to evaluate 'fatigue' effects induced in coastal, offshore, and ocean structures (Bea 1990, 2001, Bouckovalas, Marr and Christian 1985) and a wide variety of other types of engineered structures subjected to the effects of repeated – cyclic 'loadings' (pressures, strains, stresses) (e.g. Barsom and Rolfe 1977,Wirsching 1984, Almar-Naess 1985; Xu et al 1999). Such fatigue analyses also have been applied to a wide variety of geotechnical engineered 'earthen structures' such a dams, pile and mat foundations, and earth slopes subjected to cyclic 'loadings' induced by environmental conditions acting on the structures (e.g. waves, currents, earthquakes, wind, ice floes and bergs). Both 'low-cycle' (e.g. 1 to 100 cycles) and 'high cycle' (e.g. 100 to 1,000,000 cycles) fatigue effects are addressed by these analyses. In addition, the effects of both

‘regular’ (periodic) and ‘irregular’ (non-periodic) loading effects are addressed by such analyses (e.g. Miles 1954, Dowling 1972, Wetzel 1977, Wirsching 1984, Wirsching and Light 1980, Lutes et al 1984, Berge 1985). The primary objective of fatigue analyses is to determine if repeated cyclic loadings will produce undesirable degrading effects in the performance characteristics (e.g. strength, capacity, stiffness) of elements, components, and systems that comprise engineered structures and to provide adequate safeguards (e.g. reduce cyclic ‘loadings’, increase cyclic loading ‘capacities’) to prevent and mitigate degrading effects. Inspection, Maintenance, and Repair (IMR) methods are applied to address unanticipated cyclic ‘loading’ fatigue degradations (e.g. Reliability Centered Maintenance; Jones 1995; Bea 1994a, 1994b).

86. The fatigue analysis of coastal, offshore, and ocean structures has been founded on five basic components (Bea 1990, 1992):

1) Characterization of the life-cycle (short term and long term) cyclic ‘loading’ conditions (e.g. from waves and currents);
2) Determination of the cyclic ‘loadings’ (repeated straining) imposed on or induced in the structure elements, components, and system;
3) Evaluation of the cyclic strains (stresses, displacements) developed in the structure elements, components, and system;
4) Determination of degradations in strength (load resistance) and stiffness (displacements, deformations) caused by the cyclic ‘loadings’ (fatigue damage); and
5) Evaluation of the acceptability of the performance characteristics of the structure elements, components, and system.

If the performance characteristics of the engineered elements, components, and system are deemed to be ‘unacceptable’, then the elements, components, and system can be re-configured so that the fatigue effects are 'acceptable'. IMR (Inspection, Maintenance, Repair) programs are employed to help assure that the acceptable fatigue performance characteristics are maintained through the life of the structure system (Bea 2000).

87. One of the important technical points raised in the Defense Expert Reports regards the time-magnitude characteristics of the environmental 'loadings' ('demands') used in the EBSB Wave Erosion Model. The EBSB Wave Erosion Model is based on time simulations or 'snapshots' of 'regular' waves that are associated with the development of Hurricane Katrina. As pointed out by the Defense Experts (e.g. Resio, Ebersole), the wave – time histories are irregular because they represent complex combinations of different amplitudes and frequencies of wind wave 'components.' In the EBSB Wave Erosion Model, we have simulated these irregular seas with a regular sequence of waves whose heights are based on the Significant Wave Height associated with a particular 'sea state' (time, location) in the storm history. The Significant Wave Height is the average height of the highest one-third waves at a given location for a given period of time. The Significant Wave Height is the one that is used to characterize the 'energy' (potential, kinetic) that exists in a particular sea state. The maximum wave height (and lower order wave heights) are functions of the duration of the sea state and the wave amplitudes and frequencies represented in that sea state. The EBSB Wave Erosion Model has been based on regular wave simulations whose wave periods (time between occurrence of two wave crests) are those associated with the Peak Spectral Wave Period. The Peak Spectral Wave Period is the wave period associated the peak of the sea state energy.

88. Important issues associated with irregular sea states and regular wave simulations used in fatigue analyses of coastal, offshore, and ocean structures have been addressed and are incorporated in this technology – principally in the form of "Rainflow Corrections" (Almar-

Naess 1985, Wirsching 1983, Lutes et al 1984, Berge 1985). In fatigue analyses of coastal, ocean, and offshore structures it is not possible to perform numerical simulations of all possible important characterizations of three-dimensional irregular sea states. The impossibility becomes even greater when there are many sea states that must be simulated (through the fatigue exposure period considered) and the structures on which the sea states act involve non-linearities (stress and strain are not proportional). Corrections to results from 'regular wave' simulations to recognize the potential effects associated with 'real irregular wave' conditions have been developed in the form of "Rainflow Corrections" (e.g. Wirsching 1984, Lutes et al 1984, Xu et al 1999, Berge 1985). For structures subjected to shallow water wave conditions and forces, the corrections to results from regular wave simulations of fatigue damage (low and high cycle) based on the sea state Significant Wave Heights and Peak Periods (stress and strain <u>ranges</u>) have been found to be in the range of 0.8 to 0.9 – <u>no major corrections are required to the regular Significant Wave Height and Peak Period based analyses of the accumulated strain ranges that lead to fatigue damage (Type II Model Bias is close to unity).</u> This is because the fatigue process is dependent on the accumulation of 'damage' developed by both large strain ranges and small strain ranges. The fatigue cumulative damage analysis process uses 'linear accumulation' of damage – a summation of the damage contributed by each of the cyclic strain ranges. This is the same method used in the EBSB Wave Erosion Model.

89. The fatigue damage accumulation background has been used in the EBSB Wave Erosion Model. Based on the background developed in fatigue analyses of coastal, offshore, and ocean structures (and other similar structures subjected to repeated loadings), no corrections have been applied to the results from the 'regular Significant Wave Height and Peak Period wave based' EBSB Wave Erosion Model damage accumulation process to account for irregular wave

loading histories. The Defense Experts are encouraged to become familiar with this background to help assuage their concerns about the regular wave analysis processes that have been used in the EBSB Wave Erosion 'damage accumulation' analyses (erosion distance – Figure 1).

90. The scope of the validations of the EBSB Wave Erosion Model documented in the Technical Report that accompanies this Declaration include comparisons of velocity profiles predicted with LS-DYNA with velocity profiles measured during laboratory wave flume testing on a model of a proposed configuration of a proposed MR-GO earthen flood protection structure (armored with an articulated concrete mattress), comparisons of lateral erosion as predicted by the wave-induced erosion method with that measured during a large scale laboratory wave flume test performed on a model of an earthen flood protection structure, comparisons of lateral erosion as predicted by the wave-induced erosion method and that observed at the EBSB Study Location following Hurricane Gustav, and comparisons of the predicted lateral erosion via the wave-induced erosion method and closed form analytic solutions (Kriebel and Dean, 1993; USACE Coastal Engineering Manual 2006; Stanczak, 2008; and Allsop et al 2007). These other analytical methods have been validated with field observations and laboratory experimental data.

91. The scope of this part of the validation work associated with comparisons with results from other similar analytical models consisted of developing three simulation case studies and conducting analytical calculations of estimated lateral erosion based on Kriebel and Dean (1993) and Stanczak (2008). The three validation case studies are lateral erosion at MR-GO Station 497+00 during Hurricane Gustav, velocity comparison based on laboratory experimental results provided by Hughes (2008), and lateral soil erosion magnitude and duration validation based on laboratory experimental results provided by Geisenhainer and Kortenhaus (2006).

92. For the validation numerical modeling analysis to simulate wave attack on the MR-GO man-made earthen flood protection structures during the Hurricane Gustav (2008), the analysis is performed using the LS-DYNA software package from Livermore Software Technology Corporation. Hydrodynamic inputs (storm surge, wave periods, and significant wave heights) and MR-GO grass cover characteristics were provided by Bea (2008c) and van Heerden and Kemp (Technical Report III, 2008) as input into these numerical analyses.

93. Velocity profiles were generated based on results from the laboratory study reported by Hughes (2008), using the LS-DYNA software program. The topography and hydrodynamics used in the experiments reported by Hughes (2008) were used as inputs to the numeric analyses. No lateral erosion measurements were made in the experimental work reported by Hughes (2008). Thus, the validation case study will be limited to overtopping velocity profile comparisons.

94. The third validation case study was developed to confirm the wave-induced erosion rate (distance and time to breaching) determined using the EBSB Wave Erosion Model. Test results from laboratory experiments performed by Geisenheiner and Kortenhaus (2006), as reported by D'Eliso (2008) were used to validate the EBSB Wave Erosion Model.

95. Solutions for wave induced erosion based on the analytical models developed by Kriebel and Dean (1993) and Stanczak (2008) and Allsop et al (2007) were derived (in Microsoft Excel) for the erosion analyses for Scenario 1 hydrodynamics (Katrina as was conditions), Scenario 2C hydrodynamics (Katrina Neutral MR-GO conditions), the Hurricane Gustav case study, and the Geisenheiner and Kortenhaus case study. These wave-induced erosion analytic solutions do not address all the parameters accounted for in the LS-DYNA erosion evaluation, but because these analytical solutions have been validated with field observations and laboratory

experimental data they will be useful to validate the general magnitudes (and times) of estimated breaching.

96. The Technical Report (II) that accompanies this Declaration summarizes the details of the validation simulations described in the foregoing paragraphs. The following is a summary of results from these validation simulations.

(a) The EBSB Wave Erosion Model of the MR-GO EBSB Study Location erosion development during Hurricane Gustav showed that there would be no significant erosion (grass cover or soils). The observations performed by van Heerden and Kemp (Technical Report III) following Hurricane Gustav showed that there was no significant erosion at this location. Thus, results from the EBSB Wave Erosion Model are in agreement with the observations.

(b) The EBSB Wave Erosion Model analyses of the laboratory experimental data reported by Hughes (2008) show good agreement between predicted and measured water velocities at crown and backside of the model levee used in these experiments.

(c) The EBSB Wave Erosion Model analyses of the laboratory experimental data reported by Gesenhainer and Kortenhaus (2006) match closely the measured breaching time (and distance) of the model sand dike.

(d) Analyses of the EBSB Study Location predicted breaching times during Hurricane Katrina conditions (Scenario 1) based on the EBSB Wave Erosion Model (1.5 hours) were in excellent agreement with those based on the analytical models provided by Kriebel and Dean (2.4 hours) and Starczak and Allsop et al (2.0 hours).

(d) Analyses of the EBSB study location predicted breaching times during Hurricane Katrina MR-GO Neutral conditions (Scenario 2C) based on the EBSB Wave Erosion Model (no breaching or significant erosion) were in good agreement with those based on the analytical models provided by Starczak and Allsop (no breaching or significant erosion). Given that consideration is given to the turf / grass cover lift-off time, the analytical model provided by Kriebel and Dean also predict no significant erosion.

(e) Analyses of the EBSB study location erosion during Hurricane Gustav conditions based on the analytical models provided by Starczak (2008) and Allsop et al (2007) were in excellent agreement with those based on the EBSB Wave Erosion Model analyses (no significant erosion or turf – grass lift off) and with the observations of the EBSB study location following Hurricane Gustav (Technical Report III).

(f) Analyses of the laboratory experimental dike wave breaching experiments performed by Geisenhainer and Kortenhaus (2006) based on the analytical models provided by Starczak and Allsop et al (2007) were in excellent agreement with those based on the EBSB Wave Erosion Model analyses (0.4 and 0.5 hours, respectively compared with an experimental result of 0.5 hours).

(g) <u>The primary conclusion reached as a result of these and validation analyses summarized in my earlier Declarations and Technical Reports is that the EBSB Wave Erosion analyses are able to reliably replicate results from laboratory experiments, field observations, and results from other analytical models which have been validated with laboratory and field experimental data.</u>

Breaches at Bayou Bienvenue and Bayou Dupre

97. The cited Defense Expert Reports have not addressed extensively or in detail the large breaches that developed at the interfaces of the navigation – water control structures at Bayou Bienvenue and Bayou Dupre with the adjacent EBSBs. The fundamental conclusions

reached in the cited Defense Expert Reports is that the breaches developed due to overtopping by Hurricane Katrina's surge and waves. Further, they conclude that under the "Ideal MR-GO" Hurricane Katrina conditions these breaches would develop at similar times and at similar ways. The Defense Expert Reports have not documented any quantitative analyses that justify these conclusions. They rely solely on analyses of data and information gathered following Hurricane Katrina.

98. My cited Declarations and Expert Reports detail extensive analyses of development of these important breaches in the MR-GO Reach 2 flood protection structures. These analyses included quantitative analyses of seepage and hydraulic uplift effects, lateral stability and erosion and scour that could be used to characterize the most probable modes of failure involved during development of these breaches. In addition, extensive use was made of field observations, photographs, videos, and survey data that was gathered before and after Hurricane Katrina.

99. This work indicated that the breach that developed at the south end of the interface of the Bayou Bienvenue navigation – water control structure with the adjacent EBSB was due to seepage induced lateral instability. Initiation of development of the breach occurred near the time of the peak surge generated by Hurricane Katrina. The seepage induced lateral instability mode of failure was associated with the inability of the sheet piling to shut off seepage and hydraulic pressures from the flood side that propagated through a 'window' in the shell fill that had been used in construction of the interface with the adjacent EBSB. The shell fill was used to minimize overburden loadings imposed on the weak soils that comprised the original Bayou Bienvenue channel.

100. The north side EBSB interface with the Bayou Bievenue navigation – water control structure did not breach. Analysis of this contrasting 'non-failure' indicated that even though this interface was subject to identical or very similar conditions during Hurricane Katrina, the EBSB itself was constructed using cohesive compacted soils and there was no 'window' between the bottoms of the sheet piling and the underlying fill for the seepage and hydraulic conductivity effects to develop. Even though this interface also was 'attacked' during grounding of a barge that was found on top of this interface following Hurricane Katrina, the interface experienced only minor surface erosion.

101. Analyses of the breach that developed at the interface of the Bayou Dupre navigation and water control structure with the adjacent EBSB (north side) showed that (most probably) the breach developed due to surge and wave overtopping erosion. Analyses of seepage and lateral instability indicated that the interface should not fail during Hurricane Katrina. My analyses of pre and post Katrina survey data, photographs, videos, and field inspections corroborated these analyses. This included the erosional features found adjacent to the south interface of the Bayou Dupre structure.

102. Analyses of the north side breach that developed at the Bayou Dupre structure as documented in the cited Defense Expert Reports is summarized by the following quotation: (December 2008 Expert Report by Dr. Mosher):

> *"Along Reach 2, the most severe scour and erosion occurring at transitions was at the two control structures (Bayou Bienvenue and Bayou Dupre). Figures 50 and 51 shows the scour and erosion (at) Bayou Bienvenue and Bayou Dupre, respeictively. The scour and erosion at the transition between the walls of (the) control structures and the levees was so severe that the walls collapse (d) allowing large hole (s) to develop."*

103. Figure 3 is a copy of the figure referenced in this quotation. Figure 3 shows the breach that purportedly developed during Hurricane Katrina at the north end of the Bayou Dupre structure's interface with the adjacent EBSB. The referenced photograph clearly shows the missing concrete sheet piling section and the very large scour holes that are referred to in this quotation. No other analyses, data, or information are cited to justify the observations and conclusions that were reached about development of this large breach in the flood protection structure. The conclusion is that these features developed during Hurricane Katrina.

Figure 51. Bayou Dupre Control Structure

Figure 3: Referenced figure from Defense Expert Report by Mosher (2008). Note missing concrete sheet pile panels at end of the Bayou Dupre navigation and water control structure.

104. Figure 4 is a figure from my July 2008 Declaration (I) and Technical Report (II) that shows the breach at the north side of the Bayou Dupre structure. Another photograph from the series taken the morning of August 30th is shown in Figure 5. This photograph also shows

that the sheet pile wall is still attached to the abutment of the navigation – water control structure.

105. I spent many hours searching through photographic, survey, and field observations evidence that would provide reliable information about development of this breach. I located a series of aerial photographs that were taken during the morning following the passage of Hurricane Katrina (by EPA representatives on August 30, 2005). The primary reason for this search was because of concerns about effects on the post-Katrina evidence that were developed during the passage of Hurricane Rita (September 23, 2005). Hurricane Rita reportedly generated surges (storm tides) having peak elevations in the range of +5 to +7 feet (NAVD88) (Knabb et al 2006). Comparisons of photographic, video and survey evidence developed following Hurricane Katrina and before Hurricane Rita clearly showed that the surge (storm tides) and waves developed important effects on the features and breaches that were developed during Hurricane Katrina. In addition, the tidal and 'residual' storm tides caused by Hurricane Katrina had important effects in development and changing the features developed during the 'onslaught' of Hurricane Katrina.

106. The photographs taken the morning following Hurricane Katrina clearly do not show the same 'features' as those found during the time following Hurricane Rita (e.g. Figure 3). The erosional feature at the concrete sheet pile – EBSB interface is much narrower. The breach has not developed fully at the interface with the adjacent EBSB. The concrete panel has not separated from the adjacent sections – it is intact. The strong flood and storm tide flow is evident.

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 4: Referenced figure from Expert Report Declaration I by Bea (2008).

Figure 5: Photograph of Bayou Dupre navigation – water control structure taken the morning following Hurricane Katrina (EPA photograph). Photograph shows that the concrete sheet piling adjacent to the abutment of the structure are intact.

107. Figure 6 is a frame from a video taken during an aerial survey that included the Bayou Dupre structure – EBSB interface. This video was taken during the survey flight made on September 14, 2005 – before the arrival of Hurricane Rita on September 23rd. This and other similar videos were taken during aerial LiDAR surveys performed for the USACE following Hurricane Katrina (and before Hurricane Rita). Comparisons of Figures 3, 4 and 5 indicate that the breach features developed and expanded significantly during the interval between Hurricane Katrina and Hurricane Rita; likely due to the storm and tidal flows that followed the passage of Hurricane Katrina and accompanied Hurricane Rita. The concrete panel is missing in the video frame (September 14) and present in the photograph taken the morning following the passage of Hurricane Katrina.

108. During this investigation, contacts were made with the contractor who performed the repairs of this breach (Ehrensing 2006). The contractor reported that the concrete panel/s were found on the bottom of the scour hole on the flood (east) side of the breach facing the MR-GO channel. The concrete panel did not fail during the passage of Hurricane Katrina. Further the concrete panel did not fail during the Hurricane surge overtopping (it would fallen toward the protected side). The concrete panel failed after Hurricane Katrina and before Hurricane Rita. The concrete panel most likely failed during storm tide outflows after the passage of Hurricane Katrina (Figure 7).

Figure 6: Aerial video frame from survey performed for USACE on September 14, 2005.


Figure 7: Aerial photograph of Bayou Dupre north side breach developing during ebb tidal flow following Hurricane Katrina (USACE photograph). Note breach development at EBSB – concrete sheet pile interface.

109. This comparison of results from two sets of forensic engineering analyses of the same breach (Plaintiffs and Defense) illustrates how it is easy to draw the wrong conclusions through analyses of flawed observational data; to not perform other analyses that challenge and corroborate the observations based analyses; and to jump to conclusions concerning the causative conditions and factors. The wrong 'dots' (clues) are connected in the wrong ways and the wrong conclusions are drawn from the analyses - "seeing is not believing - believing is seeing" (results from confirmation, organization, and social biases). This same theme also is evident in the Defense Expert's analyses of the breaches that developed at the Lower 9th Ward.

Effects of MR-GO Channel Widening on EBSB Elevations

110. An important issue not addressed in any of the cited Defense Expert Reports is that of subsidence – settlement – of the earthen man-made flood protection structures (EBSBs, Levees) caused by the widened channel of the MR-GO and its effects on the development of breaches along Reach 2. This issues is addressed in my July 2008 Expert Report and Declaration (I).

111. Figure 8 is taken from the July 2008 Plaintiffs Expert Report by Fizgerald, et al. It is an aerial photograph that shows the locations of two geologic cross sections. The B-B' cross section is perpendicular to the MR-GO channel in the area close to our EBSB wave breaching study location. The geologic cross section is shown in Figure 9. This cross section shows the vertical sequence of soil deposits above the top of the Pleistocene contact (former low standing level). The soils identified are a) Marsh, b) Interadelta, delta front and distributary mouth bar and c) Nearshore gulf. The Marsh deposits are described as "highly organic clayey peats and peats." The authors of this Expert Report note (Fitzgerald et al 2008):

"The marsh deposits constitute 90% of the land surface within the MR-GO region with an average thickness of 10 ft (USACE, 1958). They are subject to rapid compaction under load (USACE, 1958). Marsh deposits represent approximately 25% of the material that was excavated to create the MR-GO channel. The high water and organic content of the marsh deposits create unstable channel banks when exposed to waves."

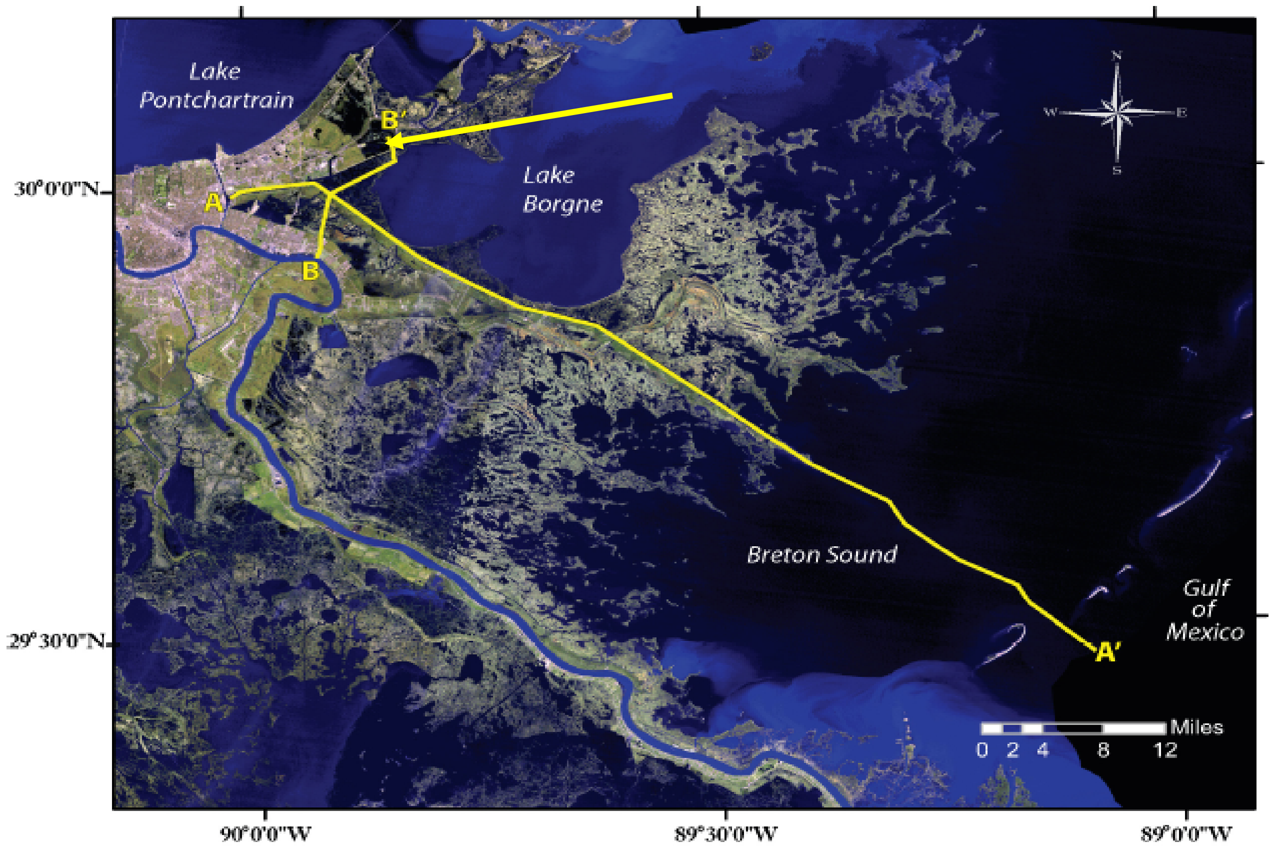


Figure 8: Aerial photograph identifying geologic cross sections A-A' and B-B' (Fitzgerald et al 2008).

112. Fitzgerald et al continue:

"Interdistributary deposits are present as clay wedges between major distributaries (USACE, 1958). Clay sequences are interrupted by silty or sandy units that were deposited by small distributaries (USACE, 1958). ….These deposits represent approximately 50-70% of all material that was excavated during MR-GO channel

construction. According to the U.S. Army Corps of Engineers remarks in the 1958 reort titled Geological Investigation of the Mississippi River Gulf Outlet Channel, …..it is possible that the poorly consolidated, high –water-content interdistributary clays will tend to flow laterally into [the MR-GO channel] excavation particularly under the extra weight of a spoil bank (Table 6.1, USACE, 1958."

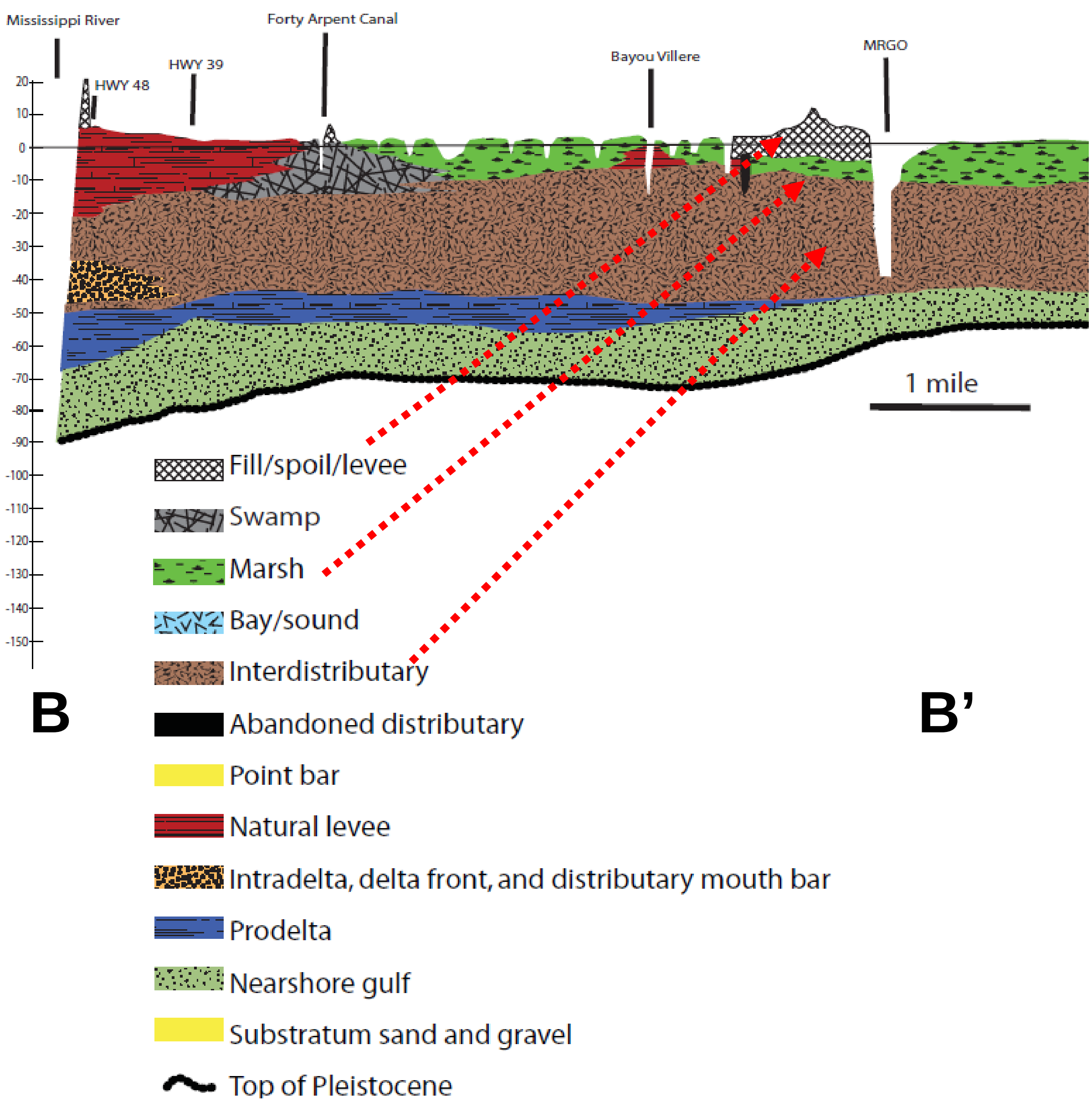


Figure 9: Geologic cross section (B-B') perpendicular to MR-GO channel showing soil deposits in the Lake Borgne region (adapted from Fitzgerald et al 2008).

113. Fitzgerald et al continue (Figure 10):

"Conceptual model for lateral displacement of the interdistributary deposits into the MR-GO channel excavation due to overburden loading and the high-angle of channel walls as built by the USACE. This phenomenon was predicted prior to construction by USACE (1958) and resulted in the need for continual channel maintenance dredging as noted in USACE (1976). The migration of sediments into the channel from the underlying interdistributary deposits results in subsidence of the land surface and increases rates of landloss along the channel. Waves produced by wind and the passage of large oceangoing vessels accelerate the rate of bank failure and landloss within the marsh deposits."

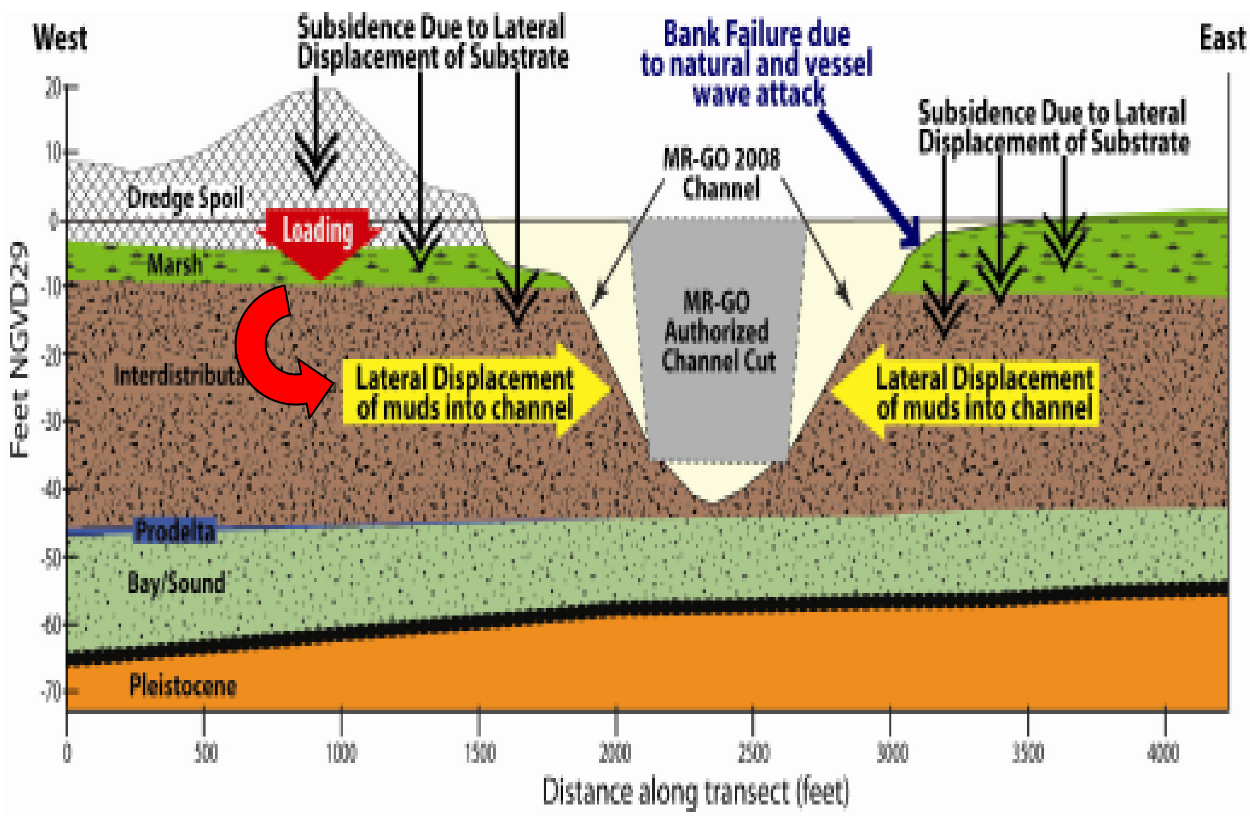


Figure 10: Model for lateral displacement of interdistributary deposits into the MR-GO channel excavation due to overburden loading (after Fitzgerald et al 2008).

114. Figure 11 is taken from the July 2008 Plaintiffs Expert Report by Morris. Morris observed:

> *"Figure 7-3 depicts a portion of the Earthen Berm Spoil Bank (EBSB) along Reach 2 of the MrGO where the canal [channel] bank was allowed to erode to the point that the water's edge was within 200 feet of the toe of the EBSB. This area is approximately 1300 feet long. The elevation of the EBSB in this area was approximately 13 feet just prior to Katrina, or more [than] 4 feet below the design elevation."*

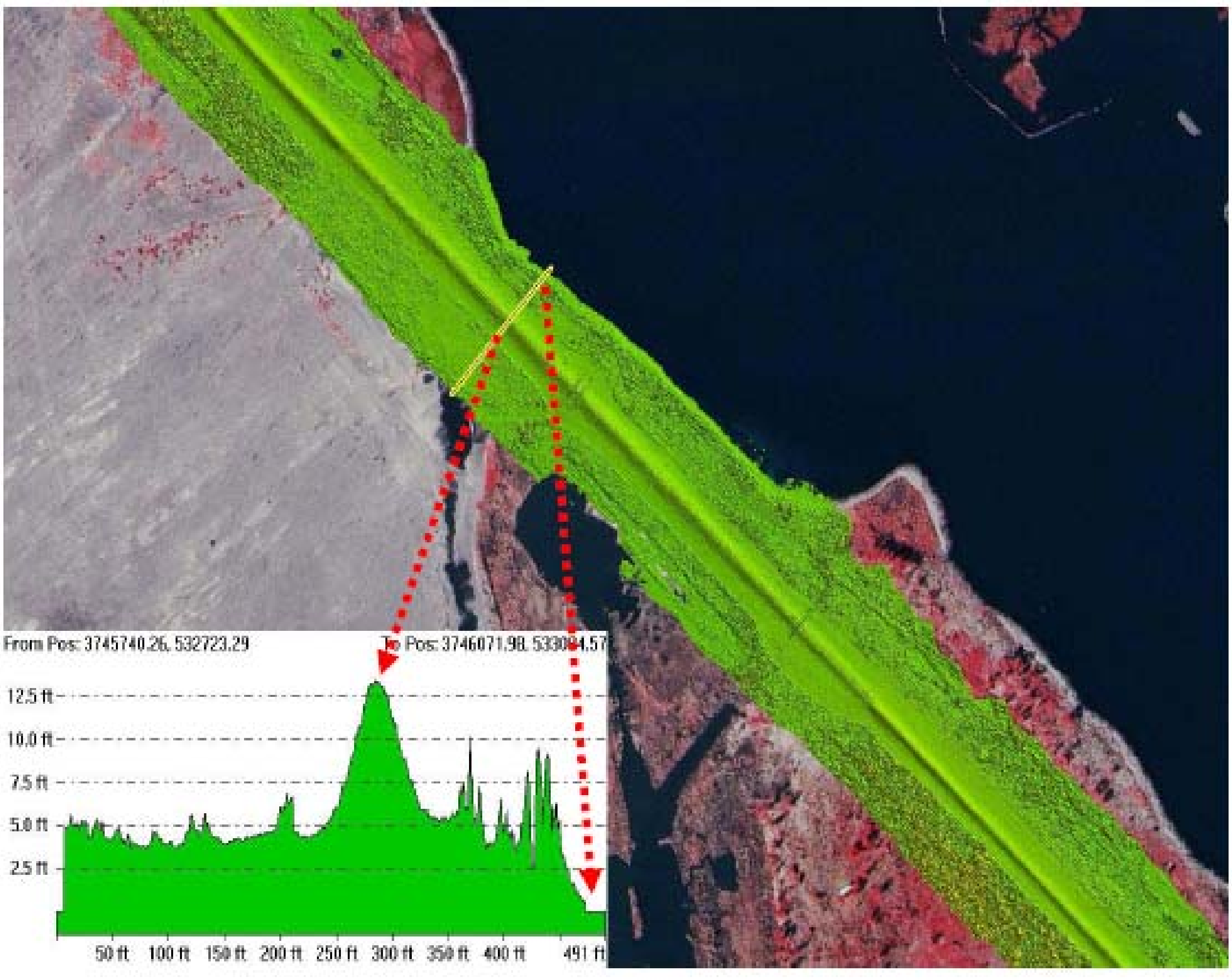


Figure 11: Processed LiDAR survey data showing EBSB cross section relative to the expanded channel of the MR-GO (Morris 2008).

115. Figure 12 is taken from my July 2008 Plaintiffs Expert Report. In this report I observe:

"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."

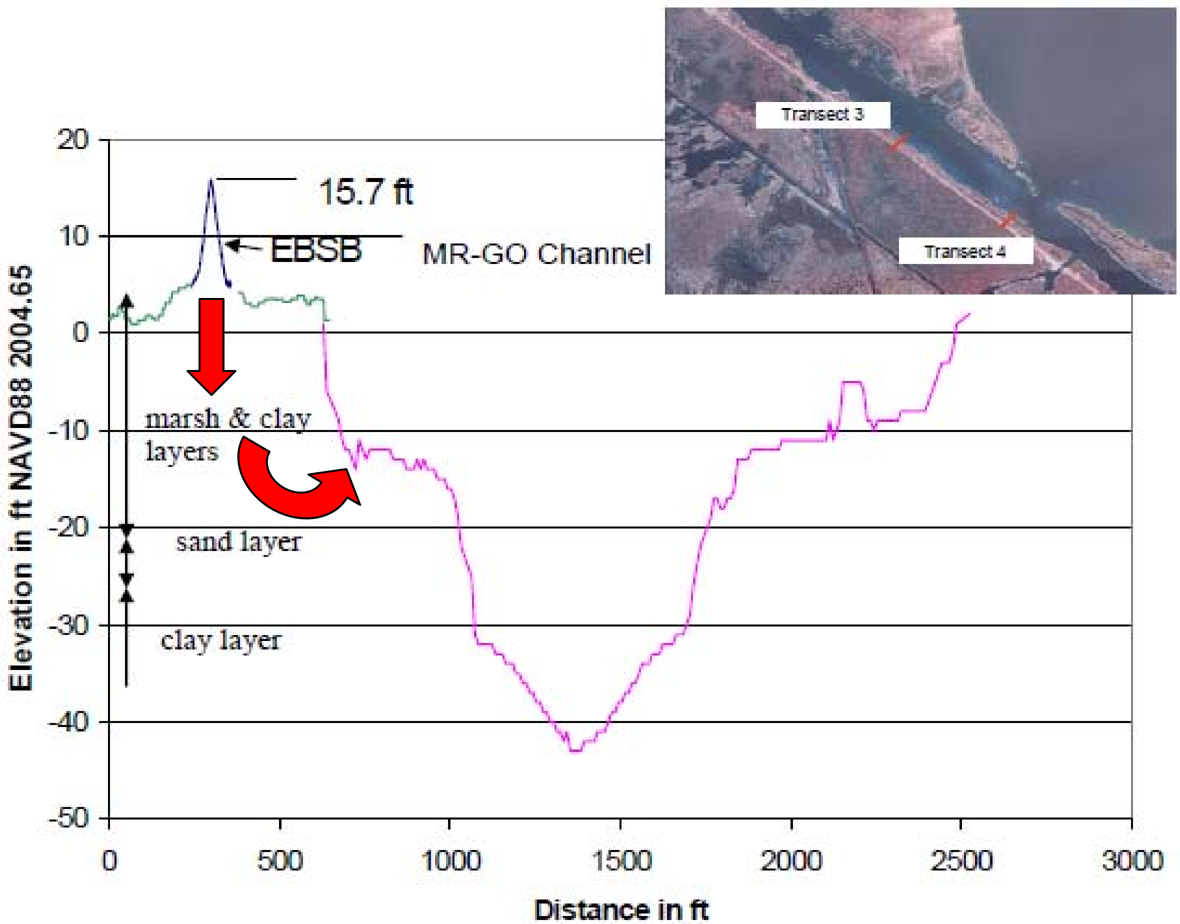


Figure 12: Cross section at EBSB (after Morris 2007).

116. 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 <u>clays will tend to flow laterally into an excavation</u> particularly under the extra weight of a spoil bank.*" (underline added for emphasis).

117. 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).

<u>During my investigations, no evidence could be obtained indicating that the USACE either understood or took proper corrective action to mitigate these dangers to the EBSBs.</u>

118. In my July 2008 Expert Report, I analyzed the potential for settlements of the EBSBs associated with the proximity of the enlarged channel of the MR-GO. Several different analyses showed that such settlements should be expected and they could be expected to lead to significant decreases in the protective elevations of the earthen protective structures adjacent to Reach 2 sections of the MR-GO. Further, based on results from recent field observations and surveys performed by Kemp et atl (2007) of the Reach 2 earthen protective structures, I concluded that since the post-Katrina repairs to the EBSBs were completed, the 'lateral creep' or 'clay squeezing' from under the EBSBs had been rejuvenated and accelerated due to the

increased overburden pressures from the heightened EBSBs. All of these analyses and observations are documented in my July 2008 Expert Report and Declaration (I).

119. Figure 13 shows an post-Hurricane Katrina aerial photograph together with 2000 and 2005 LiDAR survey based crest elevations of the earthen flood protection structures in the area shown in Figures 11 and 12. This is the stretch of the EBSBs that had been repaired with sheet piling before Hurricane Katrina. The protective elevations along this stretch were in the range of +14 to +12 feet (NAVD88). As shown in Figure 13, due primarily to surge overtopping of the sheet piling, this area had been significantly eroded during Hurricane Katrina – to elevations as low as +2 feet (NAVD88).

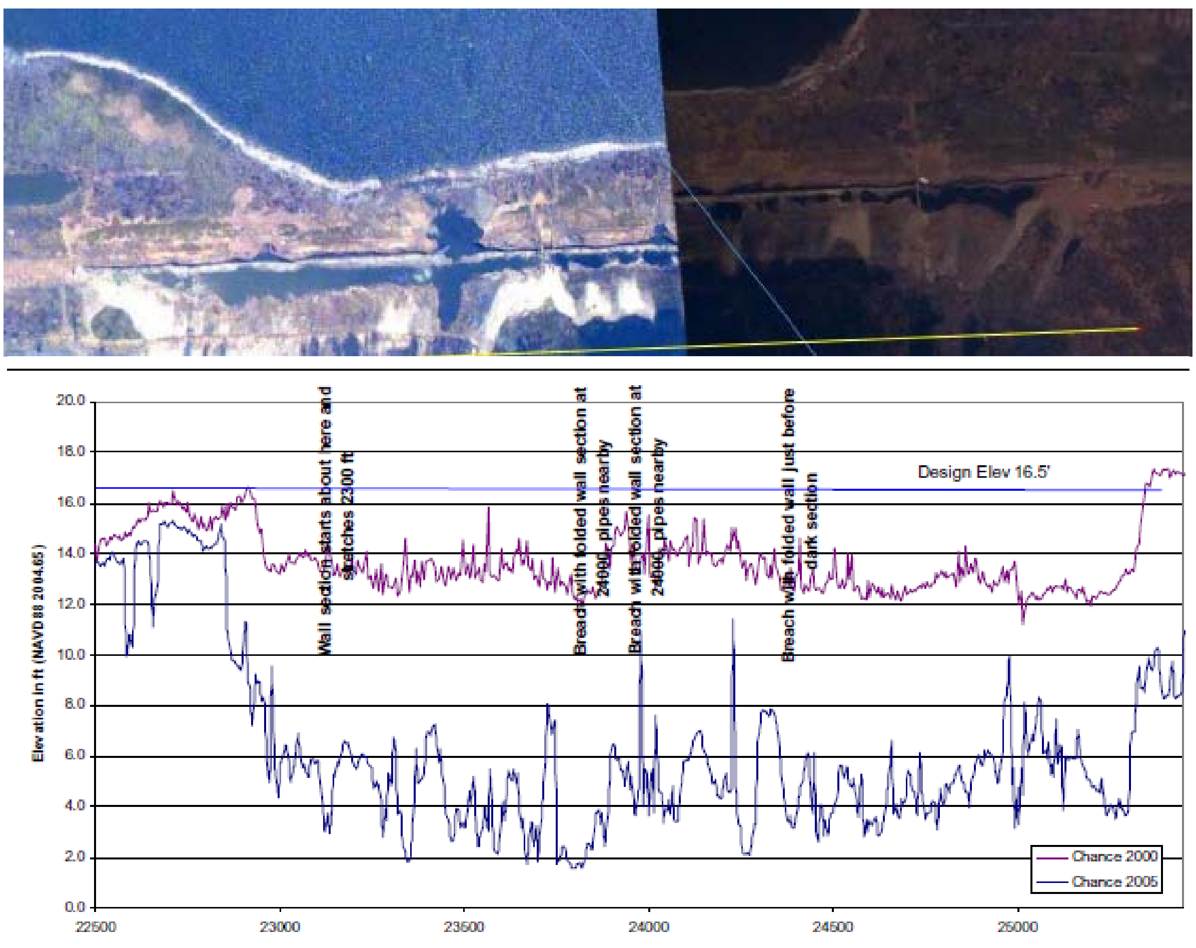


Figure 13: Pre and Post Katrina elevations of earthen flood protection structures in area repaired with sheet piling (after USACE 2006).

120. Figure 14 (from Defense Expert Report by Resio) shows comparisons of the pre and post Hurricane Katrina elevations of the Reach 2 earthen protective structures together with Hurricane Katrina's peak surge elevations based on results from analyses and observations. The sheet pile repaired areas are identified in Figure 14. Dr. Resio concludes that the comparisons clearly indicate that the erosion and breaching that developed during Hurricane Katrina were focused on the portions of the earthen protective structures that had the lower crest elevations. Further, Dr. Resio concludes that the erosion and breaching was due primarily to surge overtopping erosion and breaching of the areas having low protective elevations. Dr. Resio does not address the many other factors involved in determining the performance of these earthen protective structures during Hurricane Katrina (e.g. differences in wave action, soil characteristics, vegetation characteristics). None of the analyses documented in the cited Defense Expert Reports addresses the potential connections between the proximity of the MR-GO channel and the attendant subsidence of the earthen protective structures in determining their performance during Hurricane Katrina. This is an important omission in the Defense Expert analyses of the performance of the Reach 2 EBSBs.

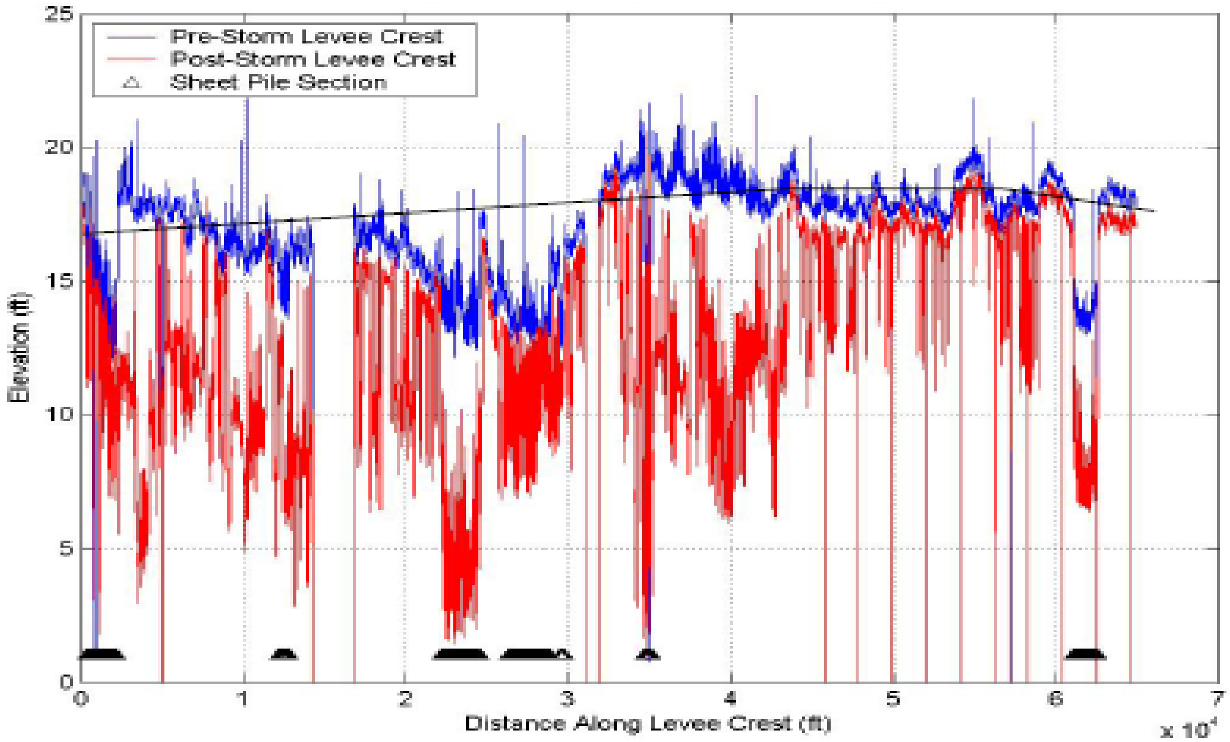


Figure 14: Comparisons of pre and post Katrina LiDAR based crest elevations of Reach 2 man-made earthen flood protection structures with Hurricane Katrina peak surge elevations determined from analyses and observations (Resio, 2008).

Evaluation of MR-GO EBSB (Reach 2) Breaching Mechanisms

121. I have performed an evaluation of breaching mechanisms associated with the MR-GO EBSB (Reach 2) during Hurricane Katrina was performed to characterize the relevant breach mechanisms. The MR-GO EBSBs have a length of over 64,000 feet, with significant variability in materials and geometry over this length. The performance of the EBSBs is dictated by the ability of the EBSB to withstand the imposed storm surge and wind waves. There are a number of factors (Figure 15) that impact the ability of the EBSB to resist storm surge and waves (Capacity); these include trees, shrubs, grasses in the Riparian/Wetland zone that 'break up' incident waves in front of the EBSB; density of the grass armoring (turf and substrate); the

geometry of the EBSB (side slope angles, freeboard/crest elevation, crest width); and EBSB materials (soil type, soil stratigraphy, soil density, and soil erodibility).

122. Similarly, there are a number of cumulative factors that impose "Demands" on the EBSB, these factors include (Figure 15): bathymetry (directly influences the wind-generated wave characteristics incident to the EBSB and the proximity of deep water to the EBSB location); topography (impacts wave characteristics as the storm surge rises because it controls water depth, and topography provides the means for vegetation establishment in front of the EBSB); storm surge level (the higher the storm surge, the higher the EBSB needs to be to prevent overtopping and higher storm surge levels allow for the generation of larger wind waves because the water depth increases); and wind waves (the direction, period, and height of the wind waves dictate the rate and magnitude of both up-rush and down-rush velocities <u>and</u> wave-induced overtopping of the EBSB).

123. The EBSB Demands and Capacities were not the same over the entire EBSB Reach 2 alignment. In order to fully analyze the EBSB performance, addressing each of the identified Capacity and Demand factors along the entire Reach 2 alignment must be addressed to appropriately analyze EBSB performance during Hurricane Katrina.

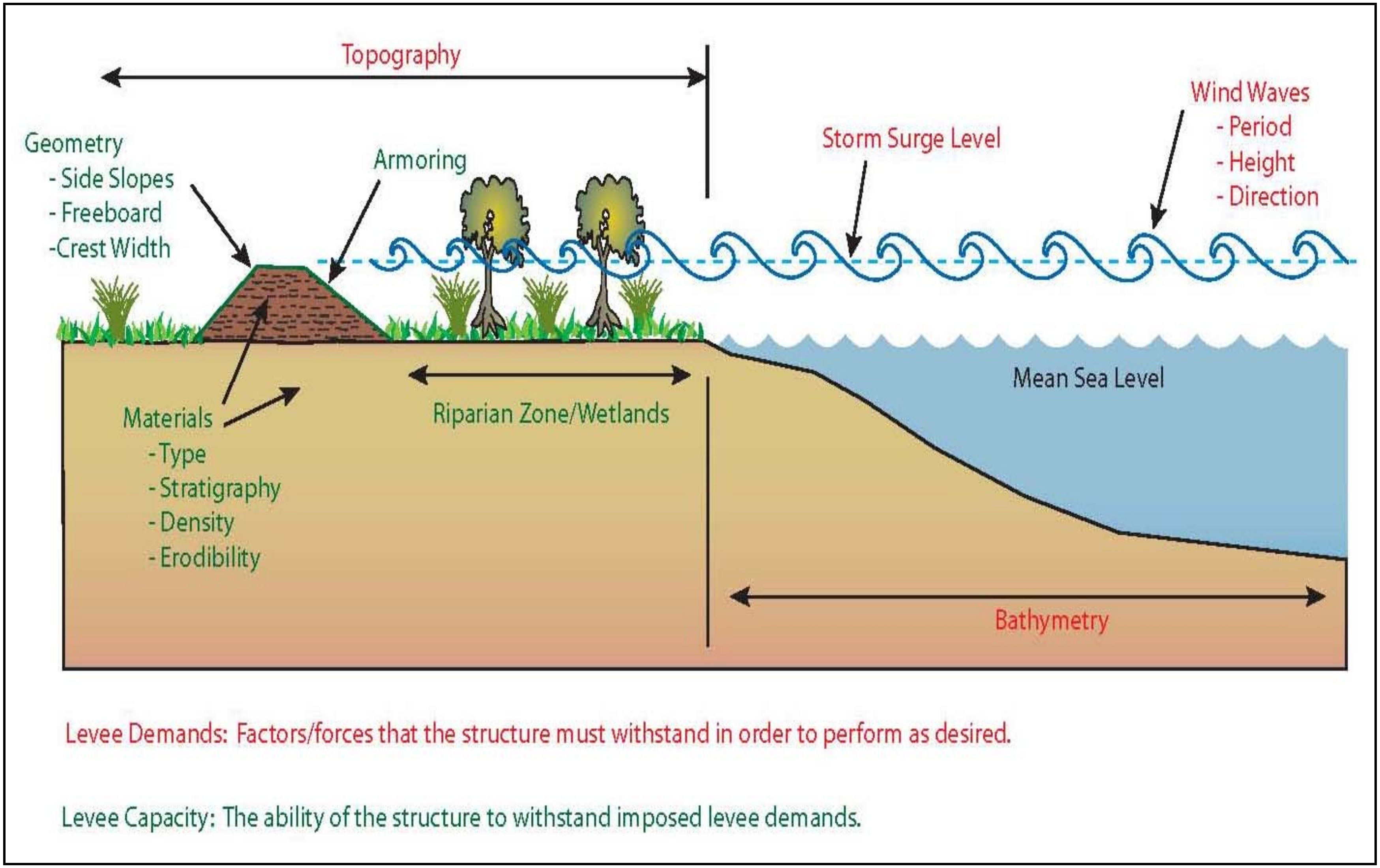


Figure 15: Summary of EBSB Performance Parameters.

124. Due to the very large spatial scale of this problem, and severe lack of information, two analysis phases were employed to characterize breaching mechanisms over the entire Reach 2 alignment:

- Phase I – Analysis of wave initiated – overtopping exploited breaching (flood side to protected side) and surge overtopping initiated breaching (protected side to flood side) at the EBSB Wave Breaching Analysis Location (Station 497+00) and at other Reach 2 locations to identify key parameters involved in development of the breaches (or non breaching) during Hurricane Katrina. This background was then extrapolated to other Reach 2 locations based on available photographic images, visual observations in the field, and review of available soil boring – laboratory and field testing data and Aerial LiDAR data. This approach does not fully address all of the identified Demand and Capacity parameters. In addition, there are 'data noise' challenges in that all of the required data is not available or there are distortions introduced into the photographic – video observations based on angles photographs were taken, lighting (time of day), etc. and resolution (point spacing) of the Aerial LiDAR data being too sparse to adequately characterize the features. Additionally, our in-the-field observations were limited to a few select locations due to access and time restrictions following Hurricane Katrina, so we were unable to visually observe the entire Reach 2 segment of the MR-GO EBSBs.
- Phase II – A detailed analysis that includes information from Phase I and all the identified Demand components (bathymetry, topography, storm surge level, wind wave characteristics) and Capacity components (riparian/wetland zone vegetation, armor characteristics, geometry, and materials) for the entire Reach 2 alignment. Such analyses were performed at

the Wave Breaching Study Location (Station 497+00) and at a location north of Bayou Bienvenue (documented in Bea July 2008 Expert Report).

125. Information used as part of our Phase I analyses include: available aerial photography (GE, 2005, NOAA, 2005), aerial video (USACE, 2005) and Aerial LiDAR data (IPET, 2005). These analyses are documented in my July 2008 Expert Report. Three primary breach types were identified: 1) Surge overtopping initiated breaching (protected side to flood side – 'Overtopping'); 2) Wave initiated – overtopping exploited breaching (flood side to protected side – 'Wave and Overtopping'), and Sheet Pile Failure (breaching and overtopping). There were also transition failure (erosion/scour around the concrete sheet pile/EBSB interface at the Bayou Bienvenue and Bayou Dupre control structures, as well as pipe line crossings), however, these were considered 'overtopping' breaches in our cursory evaluation.

126. Our Phase II analysis at MR-GO Station 497+00 included direct consideration of all the Demand parameters (bathymetry, topography, storm surge level, wind wave characteristics) as well as all the Capacity parameters (/wetland zone vegetation, armor characteristics, geometry, and materials). We performed similar analyses at a location north of Bayou Bienvenue and at the location north of the Bayou Dupre navigation – water control structure interface with the EBSB. These analyses are documented in my July 2008 Expert Report. We have also completed Phase 2 analyses at a series of locations along Reach 2 of the MR-GO to develop the additional details required to develop a reliable assessment of the breaching mechanisms and mechanics. This work is documented in Part III of this Expert Report – Technical Report IV, Phase 2 Analyses of Reach 2 EBSB Breach Development.

Breach Development Mechanisms

127. Because the sequence of actual events that developed and occurred during Hurricane Katrina was not observed first hand, classification by remnant features is necessary. Several breach development sequence guides were used in development of our classification of breach development mechanisms and stages: IPET (2007) and D'Elsio (2006a-c. 2007) for surge overtopping-induced erosion (Figures 16 and 17), and Bruun (1985), Visser (1998), Carlson and Sayre 1961, Stanczak (2008, and Stanczak et al (2006a-6, 2007a-c) for wave-induced erosion (Figures 18 and 19). Our breach classification includes the following (Technical Report II):

- Surge overtopping initiated breaching (protected side to flood side development)
- Wave initiated overtopping exploited breaching (flood side to protected side development)

Our non- breach erosion classification includes the following:

- Wave erosion of flood side
- Surge overtopping erosion of protected side
- Wave head cutting (Crenellation), and
- Sheet pile overtopping erosion.

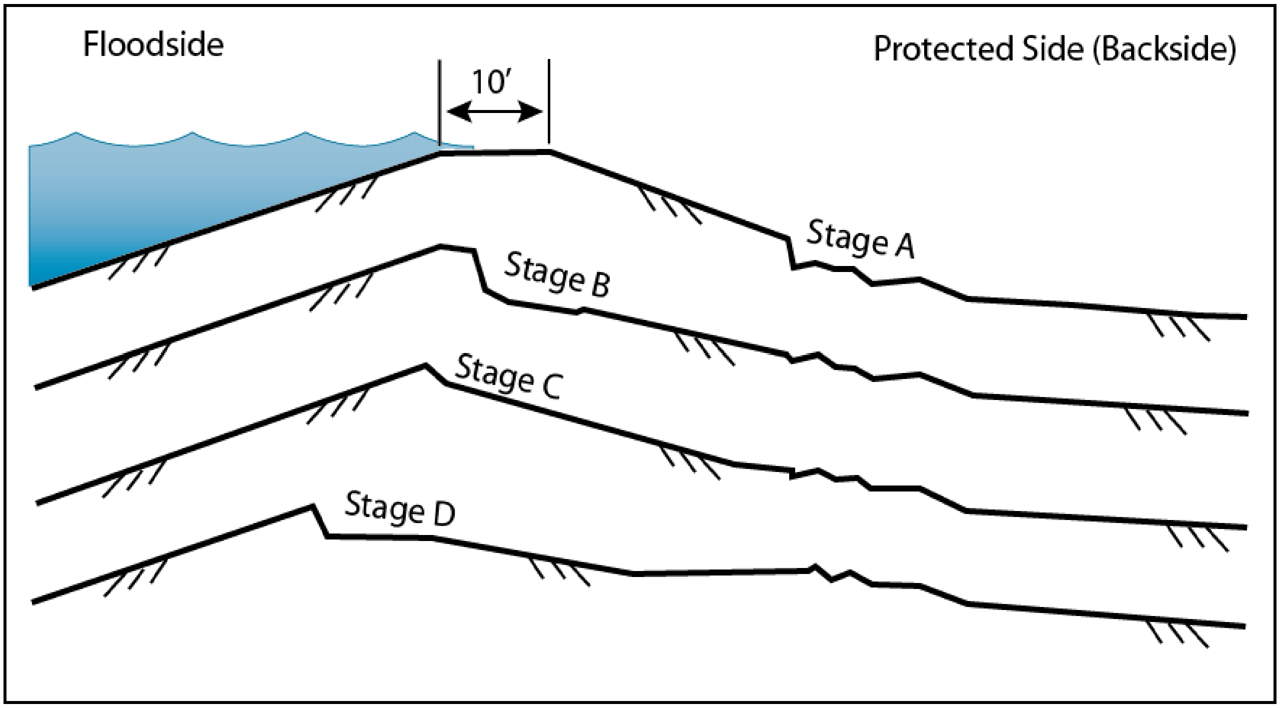


Figure 16: Progressive erosion from protected side to the flood side based on Overtopping Erosion only (from Mosher, 2008; IPET 2007).

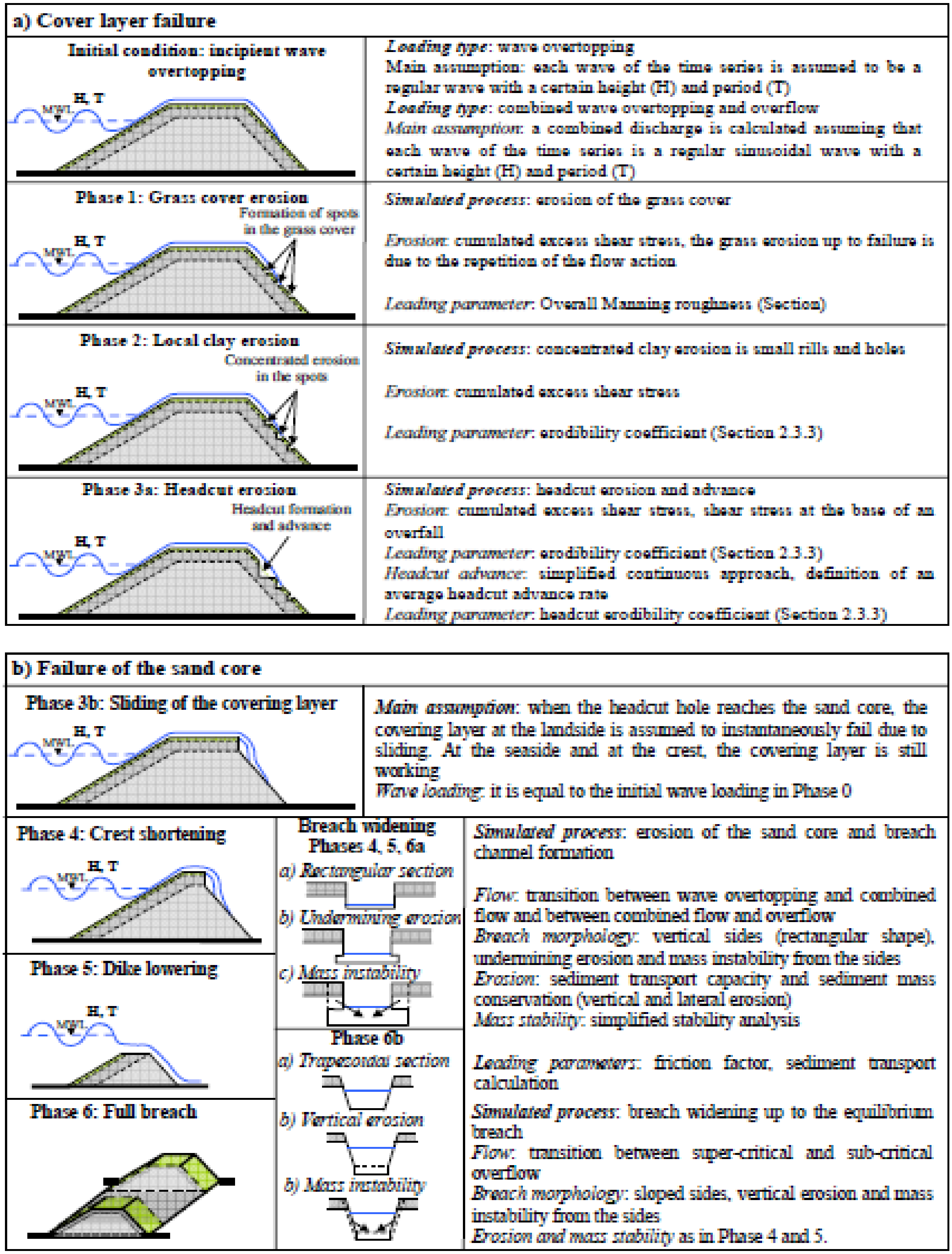


Figure 17: Overtopping erosion breach development mechanics (D'Elsio 2006a-c, 2008).

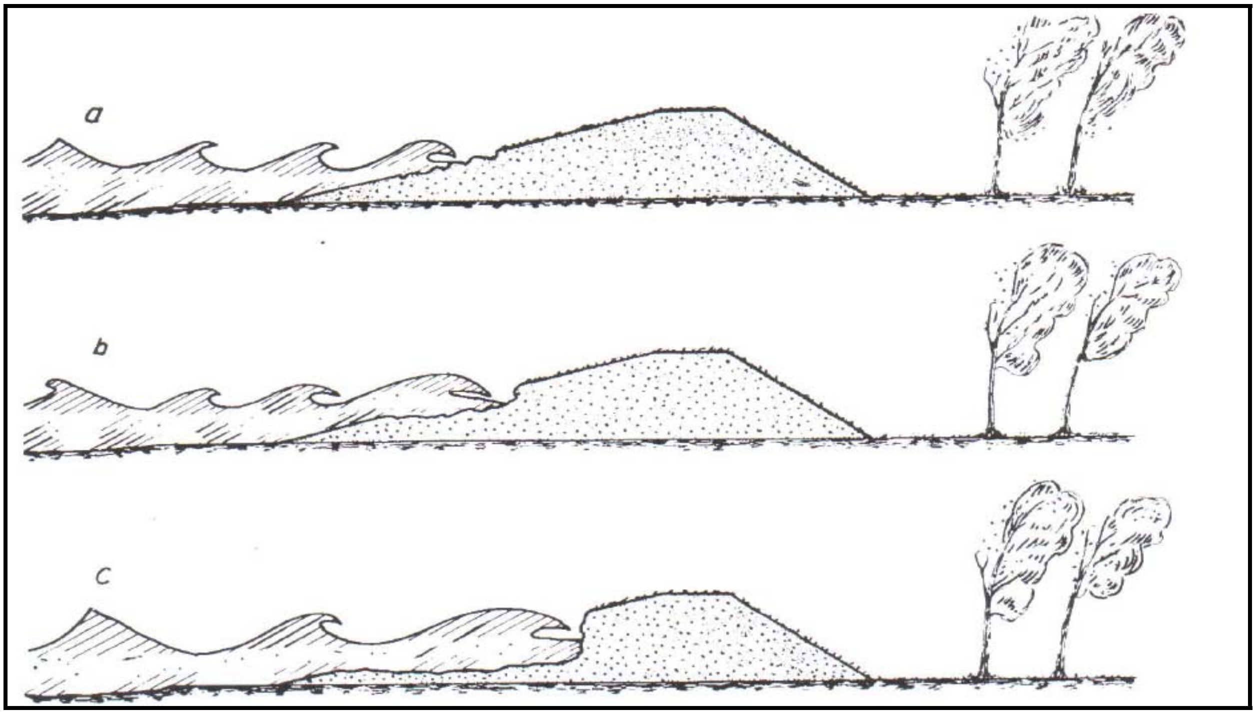


Figure 18: Wave-induced erosion initiates from the flood side and moves to the protected side (Bruun, 1985).

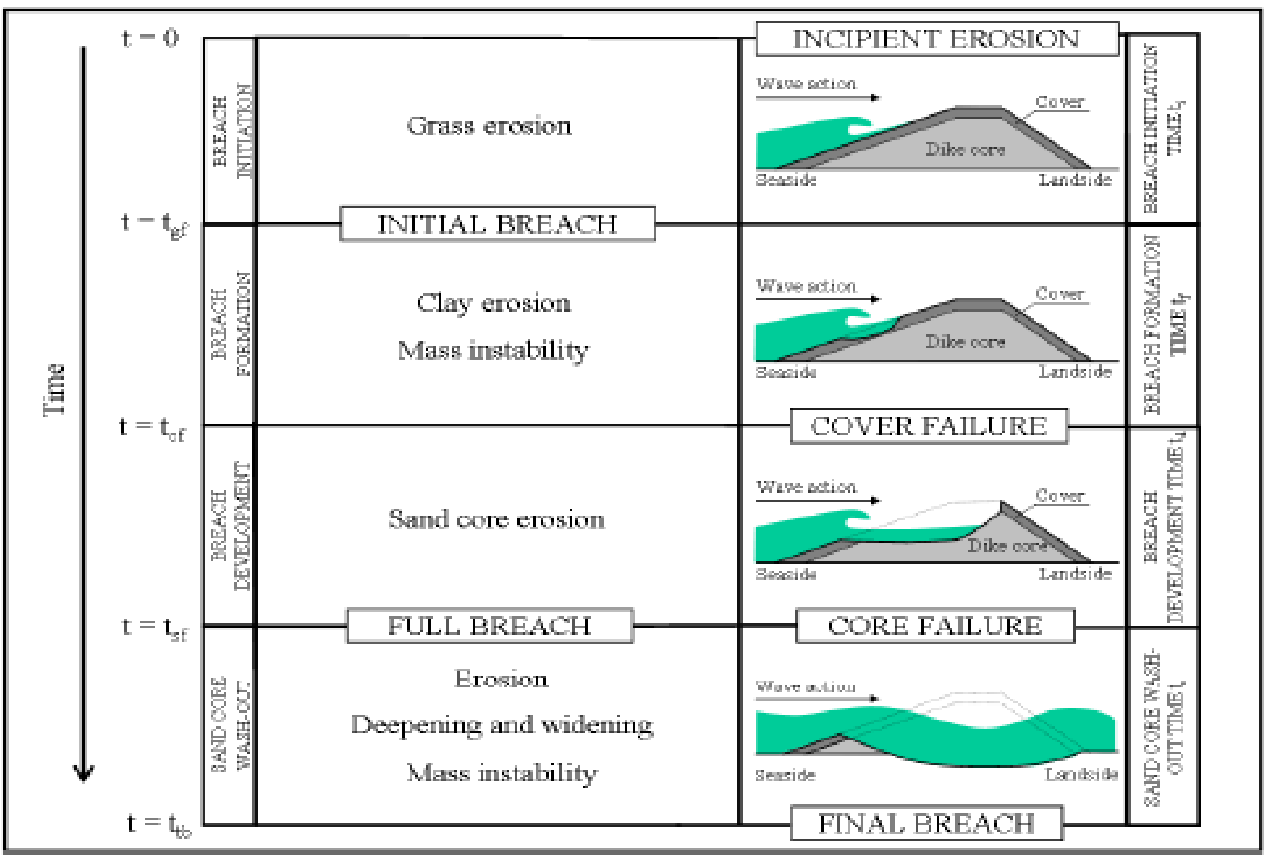


Figure 19: Wave-induced erosion breach development (Stanczak 2006a-b, 2007b, 2008).

128. *Protected Side to Flood Side* - Surge Overtopping Initiated Breaching – For surge overtopping events (Figure 20) no significant erosion occurs until the water exceeds the crest elevation (either from the mean storm surge elevation or pulses of water from wave-induced overtopping). Once exceeded, the water cascades down the backside and as the water cascades down the backside, the water velocity increases (until it reaches a terminal velocity). The high velocity rates result in high shear stresses and these shear stresses result in the active erosion. As the erosion progresses, a 'headcut' forms that traverses from the 'protected' side toward the 'flood side.' As severity of the headcut increases, the erosion moves from Stage A to Stage B, to Stage C, and finally to Stage D. The signature of this type of erosion is the lack of soils on the protected side and 'intact' sections on the flood side. The erodibility of the material and duration of overtopping impact the amount of erosion that occurs.

129. *Flood Side to Protected Side* - Wave Initiated Overtopping Exploited Breaching – For wave loading conditions (Figures 21 and 22), wave impact and run-up and run-down flow stresses are imposed on the front face as the waves either wash up and down (or waves crash into the face). The larger the waves, the more wave energy is transferred to the flood face. As the storm surge rises, the wave intensity increases and the amount of erosion increases. A 'reverse' headcut moves from the 'flood side' towards the 'protected' side. When the storm surge rises and nears the crest, wave pulses rushing across the crest induce vertical erosion. As the crest is breached (crenellation), active overtopping begins and another headcut forms, this time towards the 'flood side.' This type of erosion results in the lack of soils on the protected side as well as lack of soil sections on the flood side. Eroded soil is found on the 'protected' side because the inrush of water (once the crest is breached) flushes/transports flood side sediments to the

protected side. The erodibility of the material, the intensity of the wave action, degree of crest overtopping, and duration of waves/overtopping impact the amount of erosion that occurs.

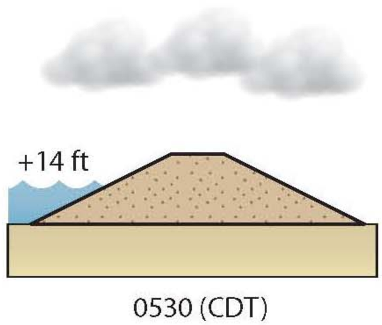
130. Wave Erosion of Flood Side – Similar to 'wave initiated breaching' but the EBSB crest is not breached (Figure 23). As a result, the erosion feature is only found outboard of the EBSB crest. The erodibility of the material, the intensity of the wave action, the 'density' of vegetative cover and duration of waves/overtopping impact the amount of erosion that occurs.

131. Surge Overtopping Erosion of Protected Side – Similar to 'surge overtopping initiated breaching' but the EBSB crest is not breached (Figure 24). As a result, the erosion feature is only found inboard of the EBSB crest. The erodibility of the material, the intensity of the wave action, the 'density' of vegetative cover and duration and intensity of surge overtopping flow impact the amount of erosion that occurs.

132. Wave Head Cutting (Crenellation) – Similar to 'wave initiated breaching' but the EBSB crest is breached but not exploited into a breach by the overtopping flow (Figure 25). As a result, the erosion feature is only found outboard of the EBSB crest. The erodibility of the material, the intensity of the wave action, the 'density' of vegetative cover and duration of waves/overtopping impact the amount of erosion that occurs.

133. Sheet Pile Overtopping Erosion – Sheet piles were installed at locations along the MR-GO where excessive settlement of the EBSBs resulted in the inability to achieve specified crest elevations and/or there were slope failures. The sheet piles were overtopped with overtopping plunging erosion. Erosion based on sheet pile overtopping was classified based on the presence of sheet piles and observed scour (Figure 26).

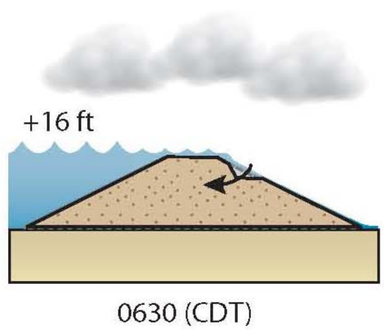
DESCRIPTION



Wave Height
- 3 ft
Wave Period
- 4.5 s

At 5:30 am (CDT) the storm surge was near Elevation +14 ft. The EBSB materials and armor are sufficient to resist the wave-induced up-rush and down-rush velocities. No erosion occurs.

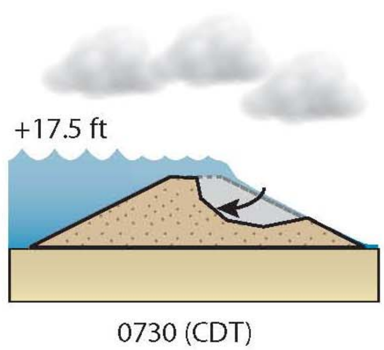
Stage A



Wave Height
- 4 ft
Wave Period
- 5 s

At 6:30 am (CDT) the storm surge was actively rising and was near Elevation +16 ft. Some wave-induced overtopping occurs as incoming wave crests impact the crest and spill over to the 'protected' side. The high overtopping velocities initiate a head cut erosion feature (Stage A) that migrates from the 'protected' side to the 'flood' side.

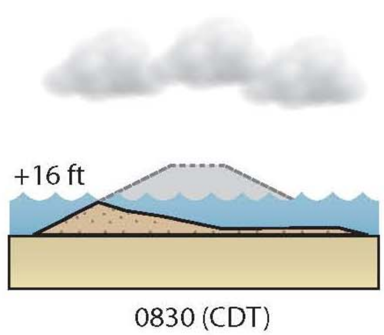
Stage B



Wave Height
- 5.5 ft
Wave Period
- 5.5 s

At 7:30 am (CDT) the storm surge peaks near Elevation +17.5 ft. The storm surge level is above the EBSB crest and active overtopping is underway. Additional overtopping-induced erosion continues to occur (Stage B).

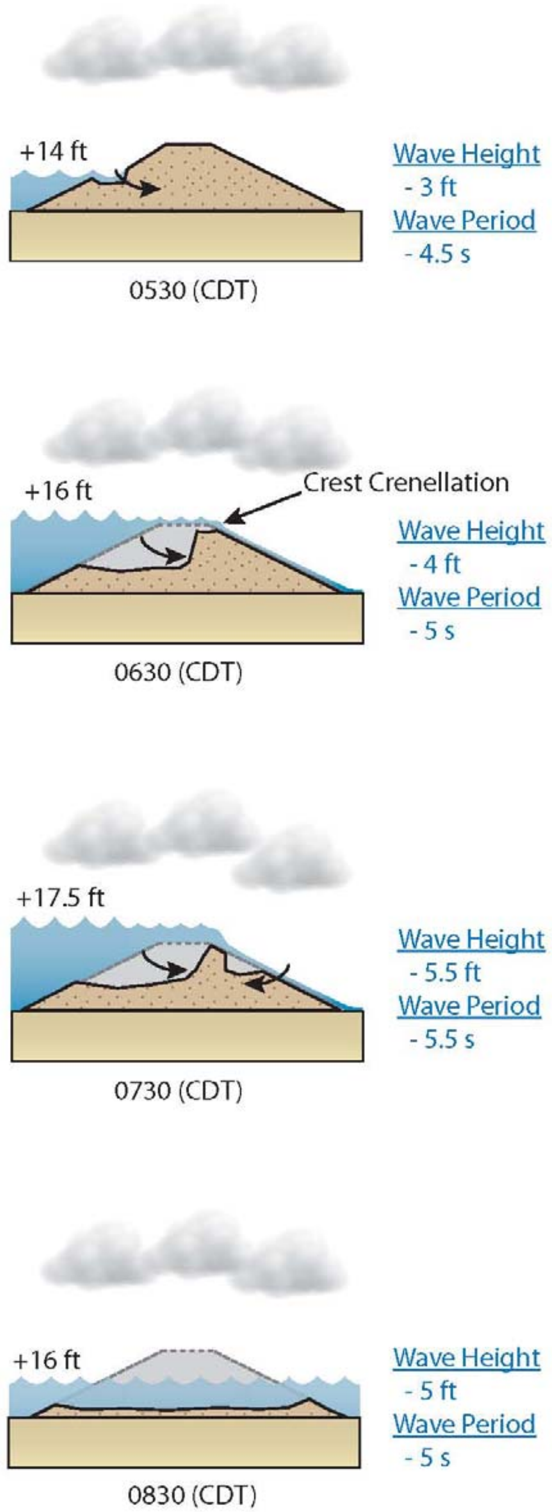
Stages C - D



Wave Height
- 5 ft
Wave Period
- 5 s

At 8:30 am (CDT) the storm surge, now receeding, was near Elevation +16 ft. The EBSB has been completely breached and floodwaters are allowed to freely wash in and out of the 'protected' area. Additional erosion occurs as floodwaters drain from the 'protected' area back out to sea (Stage D).

Figure 20: Sequence of events for overtopping-induced breaching.



DESCRIPTION

At 5:30 am (CDT) the storm surge was near Elevation +14 ft. The depth of the storm surge at the EBSB flood side face was sufficient to allow wave water to rush up and down the face. EBSB regions with poor grass cover and erodible (soft clays, silts, sands) begin to erode.

At 6:30 am (CDT) the storm surge was actively rising and was near Elevation +16 ft. Significant lateral wave-induced erosion of the erodible materials has occurred and active overtopping begins. Very little 'protection' is offered by the EBSB crest as most of it has been eroded through by wave action (Crenellation).

At 7:30 am (CDT) the storm surge peaks near Elevation +17.5 ft. Erosion occurs on BOTH the 'flood side' and the 'protected side.' Eroded sediments are quickly washed through the breach area and deposited on the 'protected side' of the EBSB as the flood waters rush into the 'protected area.'

At 8:30 am (CDT) the storm surge, now receeding, was near Elevation +16 ft. The EBSB has been completely breached and floodwaters are allowed to freely wash in and out of the 'protected' area. Additional erosion occurs as floodwaters drain from the 'protected' area back out to sea.

Figure 21: Sequence of events for wave and overtopping-induced breaching.

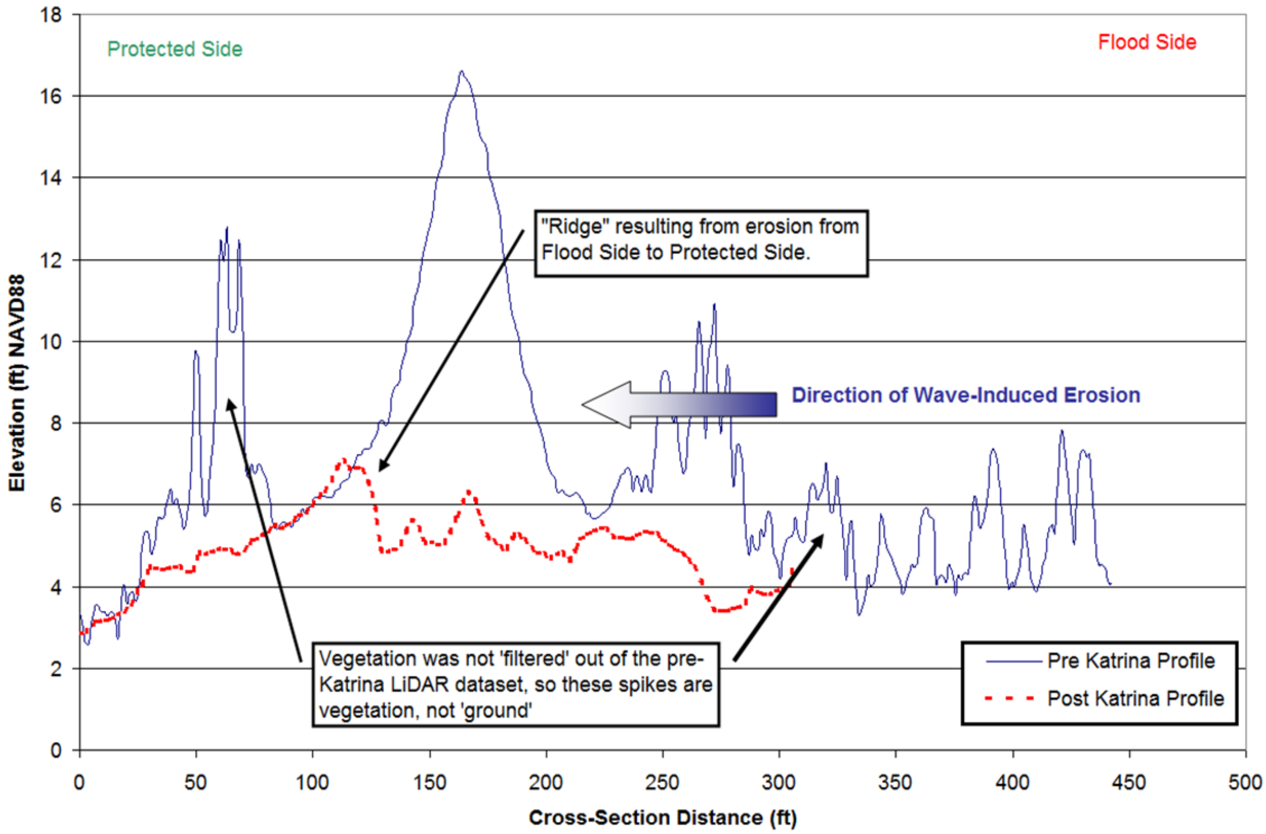


Figure 22: Pre and post-Hurricane Katrina comparisons of EBSB Wave Breaching Study Location (Station 497+00) cross-sections. Note distinctive shape of the post-hurricane EBSB cross-section.

Figure 23: Wave erosion of floodside of EBSBs. Note wave eroded 'mowed strip' on flood side.


Figure 24: Overtopping erosion of protected side of EBSBs.


Figure 25: Wave head cutting – crenellation - of the crest of the EBSBs.


Figure 26: Sheet piling overtopping erosion of EBSBs.

Phase I Analyses

134. Our Phase I breach mechanism study identified all of the foregoing multiple breach mechanisms at play. These analyses (Figure 27) indicated the following Reach 2 EBSB breach mechanisms, by percentages of the total length of EBSB breached:

- <u>Surge overtopping initiated breaching (protected side to flood side) – 55%</u>
- <u>Wave initiated overtopping exploited breaching (flood side to protected side) – 45%</u>

<u>Contrary to the assertions made by the Defense Experts in their December 2008 Expert Reports, I did not attribute development of all of the major breaches in the Reach 2 EBSBs to wave initiated overtopping exploited breaching. This is another in the long list of the ‘incorrect</u>

attributions' contained in the cited December 2008 Expert Reports – an immature 'rush to judgement'.

135. Breaches also developed at the sheet piling overtopped locations and at the Reach 2 EBSB interfaces with navigation – water control structures at Bayou Dupre and Bayou Bienvenue. These breaches are not included in the foregoing percentages.

136. Contrary to the opinions expressed by the Defense Experts, I did not extrapolate the results from the EBSB Study Location to the remainder of the Reach 2 EBSBs based solely on results from the EBSB Wave Erosion model. Many other 'Demand' (loading) and 'Capacity' (resistance, ability to perform intended functions given the Demands) factors were considered to develop an assessment of how and when the major breaches developed during Hurricane Katrina. In addition, I did not conclude that all of the breaching that developed along Reach 2 was due to water side wave erosion.

137. To demonstrate the breach development modes, example locations have been identified in Figure 12 along Reach 2 that show each of these breach mechanisms (Locations A – F). In Figures 28 – 33 a photograph shows the post-Katrina condition, a three-part illustration depicts the erosion sequence (at 5 am, 7 am, and 9 am, CDT, during Hurricane Katrina) with the rising, peak, and falling storm surge, and a cross-section of the before (2000) and post (2005) Katrina profiles (based on LiDAR surveys) that confirm the resulting shape depicted in the three-series illustrations.

138. Also shown are locations that were overtopped, but suffered no scour or erosion, represented by locations A and F were (No Breach). Location B demonstrates features from wave and/or overtopping breaching mechanism (near Station 497+00). Location C shows a sheet pile breach feature. Overtopping is shown at Location D. Wave-induced erosion (crenellation)

is shown at Location E, where there was no significant breach of the crest and through flow resulting in overtopping conditions.

139. With respect to timing of the breach development mechanisms, Figure 34 shows a timeline of storm surge elevation. Both Defense and Plaintiffs Experts agree that no erosion occurs with the storm surge below an elevation of +13 feet. Prior to overtopping (surge elevation equivalent to the crest elevation), wave-induced erosion is the dominant erosion mechanism (not fully evaluated by USA experts). To evaluate the susceptibility of erosion during this time period (approximately 5:30 am to 7 am) an analysis of grass cover armoring and erodibility from wave-generated velocities must occur (this was analyzed for our Phase II analysis at MR-GO Station 497+00).

140. Overtopping occurs with water levels exceeding the crest elevation, which occurred from approximately 6:30 to 9:00 am (note there is some overlap in time depending on crest elevation and actual surge experienced). To evaluate the susceptibility of erosion during this time period the analyses must include grass cover armoring and erosion from water cascading down the protected side. If no significant erosion has occurred by this time (9 am), the storm surge begins to subside and wave-induced erosion becomes the primary erosion mechanism again (this was analyzed for our Phase II analysis at MR-GO Station 497+00).

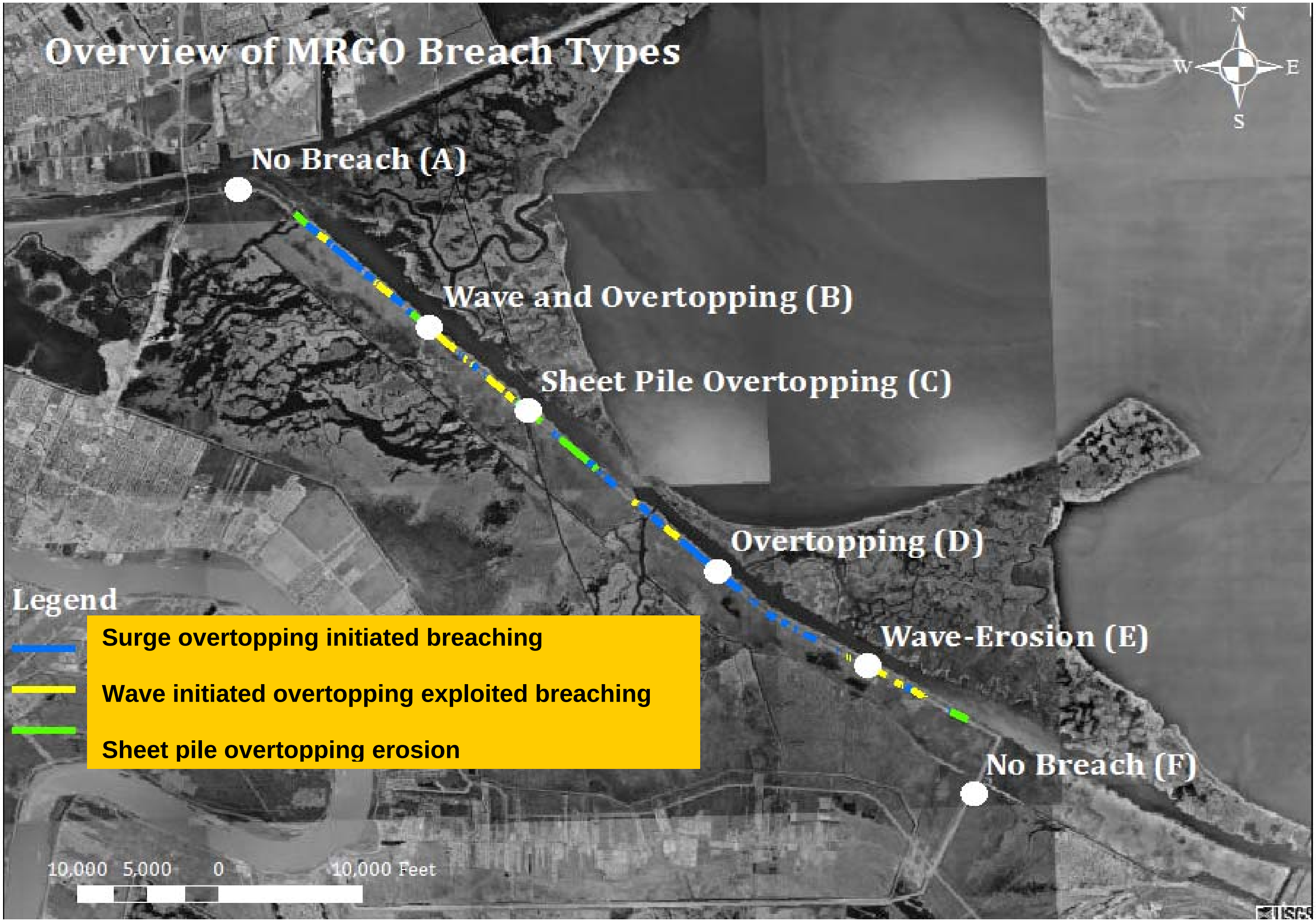


Figure 27: Phase I breach mechanism classification.

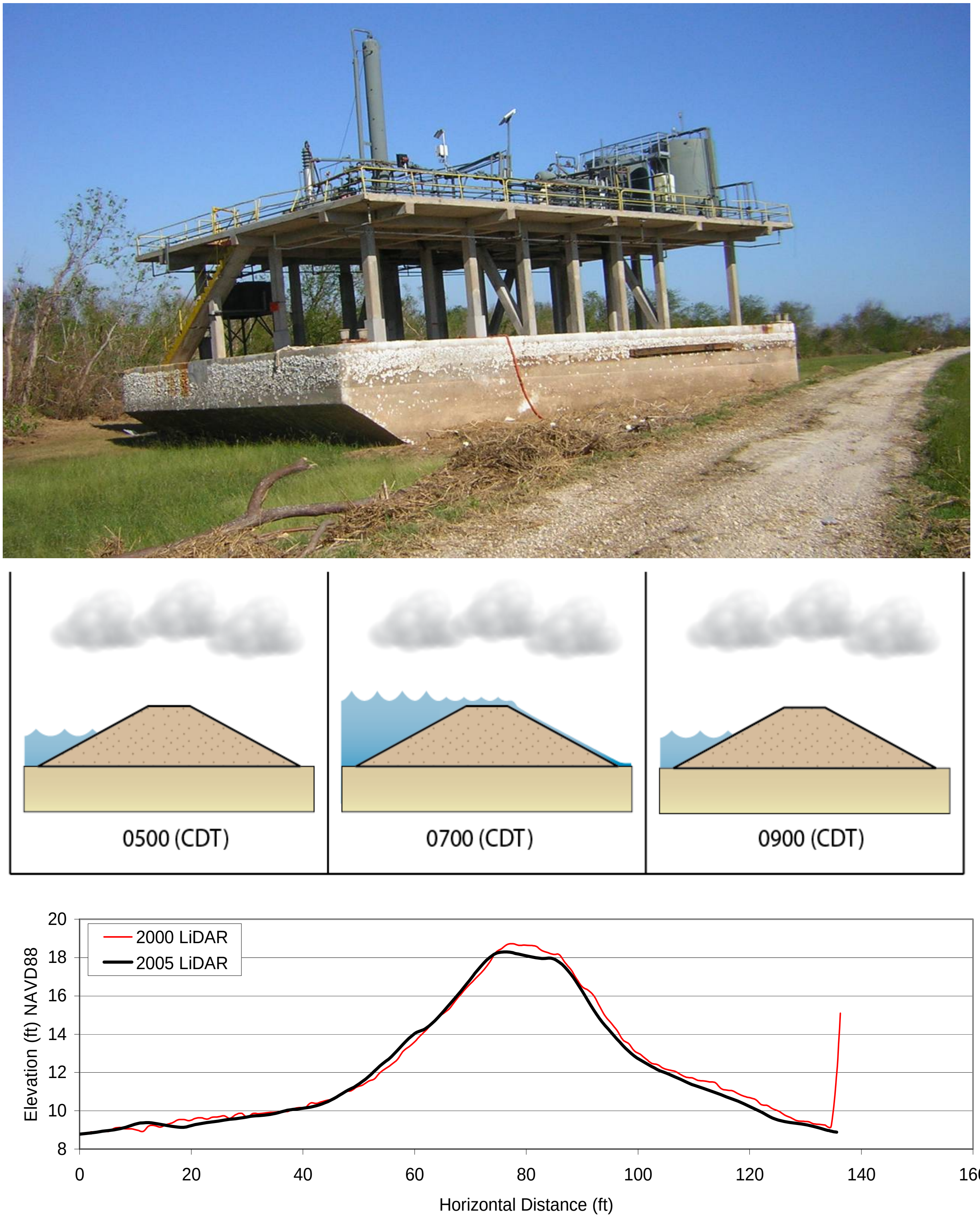


Figure 28: Location A – No Breaching.

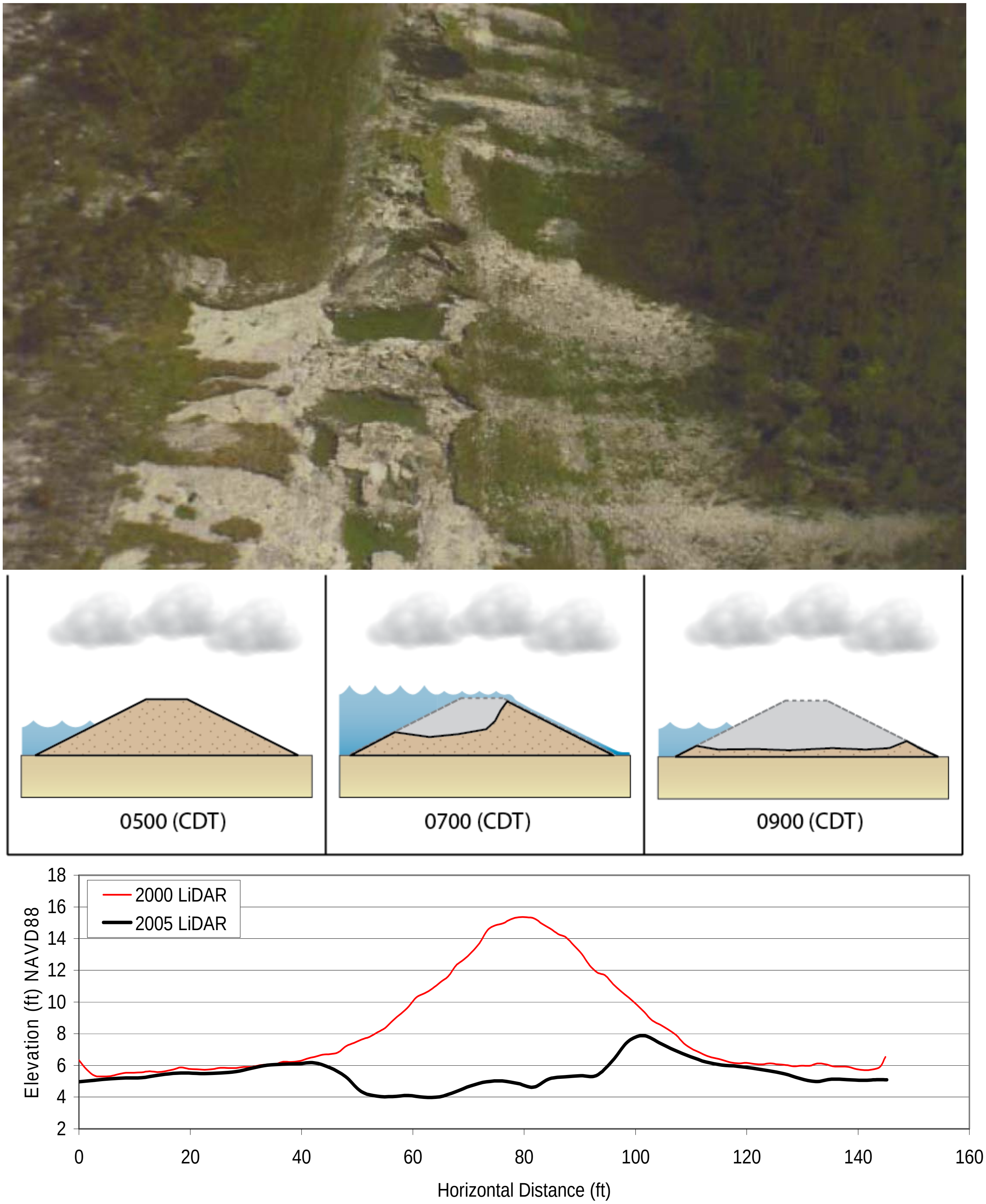


Figure 29: Location B – Wave initiated overtopping exploited breaching (flood side to protected side.

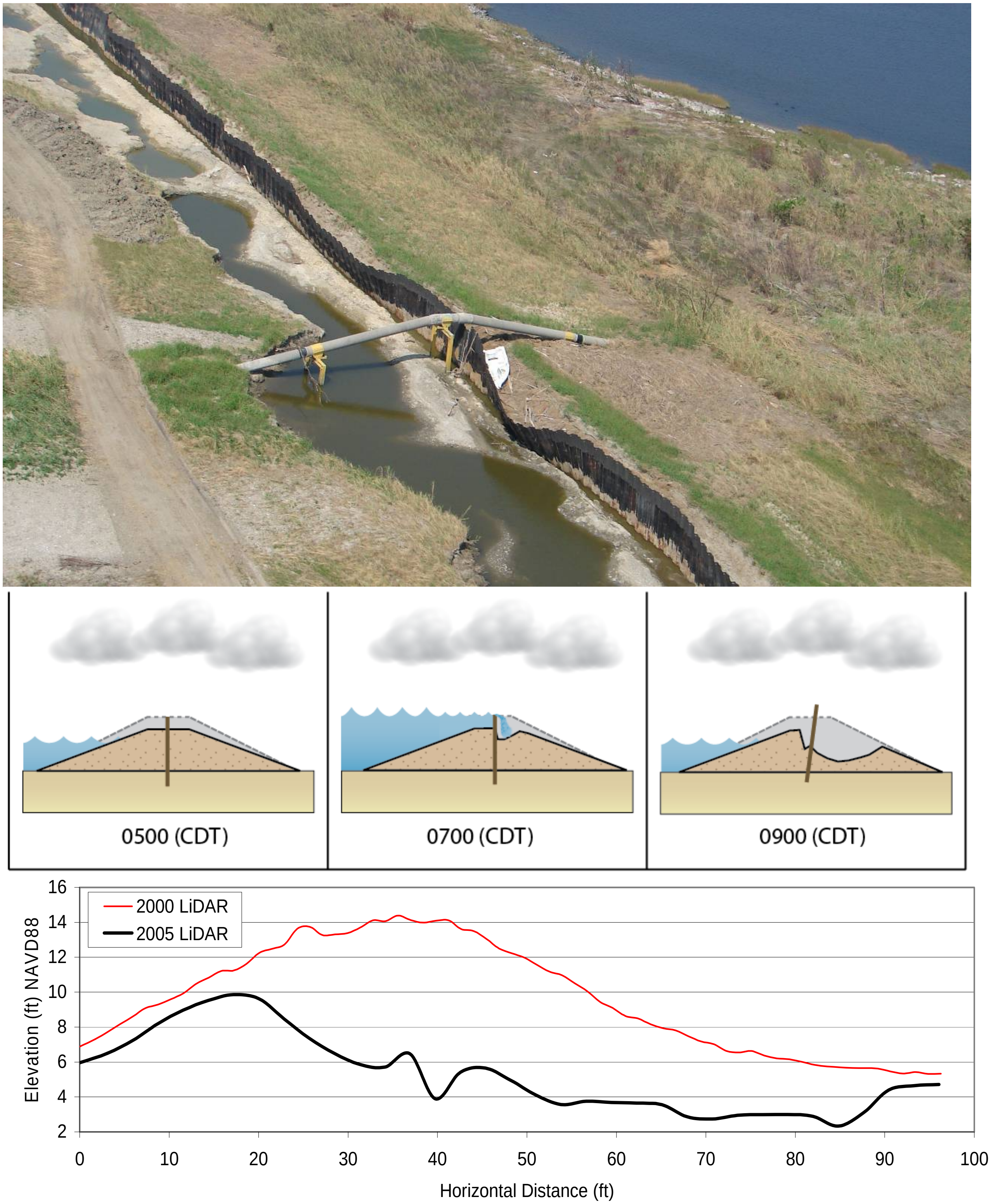


Figure 30: Location C – Sheet pile overtopping erosion and breaching.

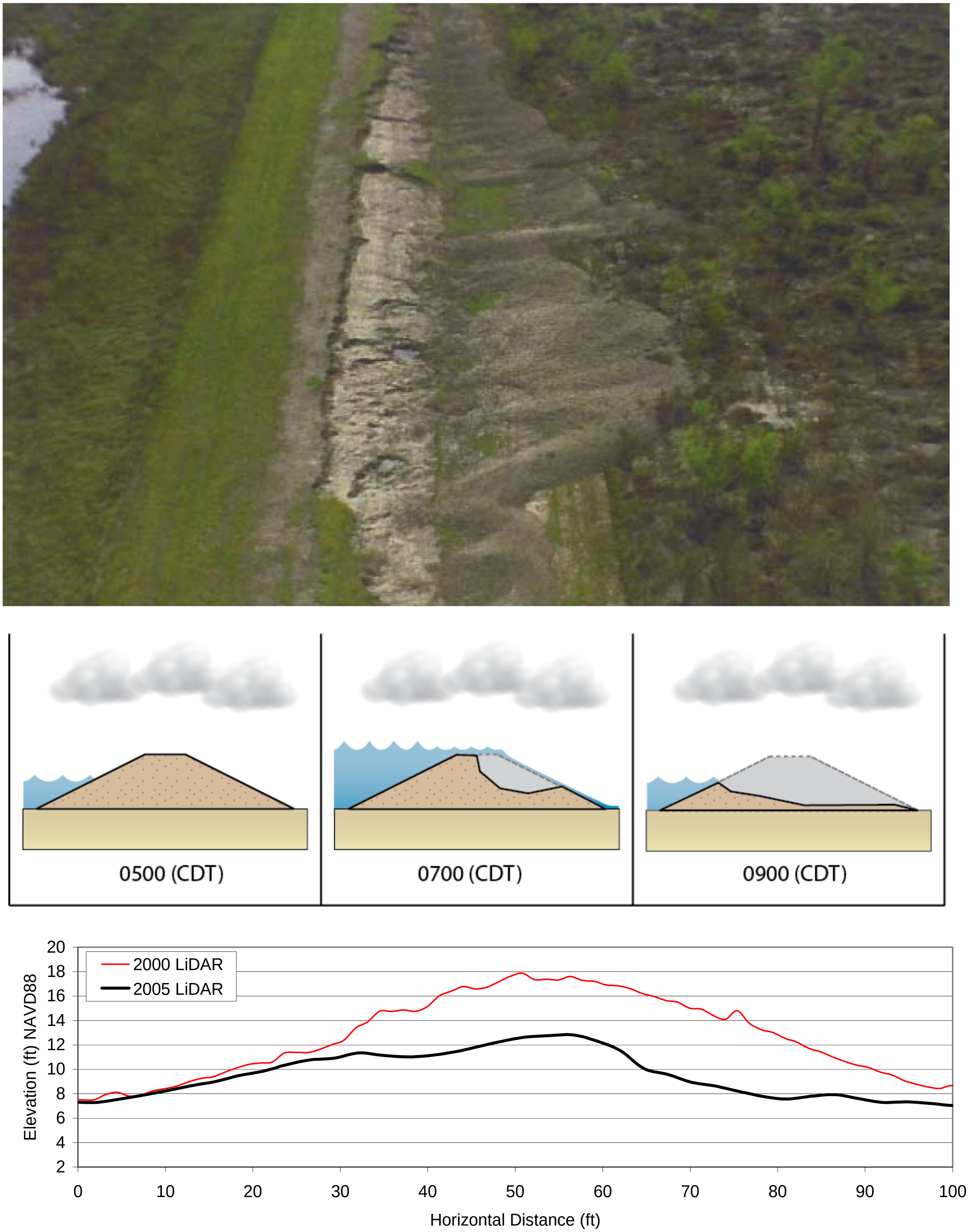


Figure 31: Location D – Surge overtopping initiated breaching (protected side to flood side).

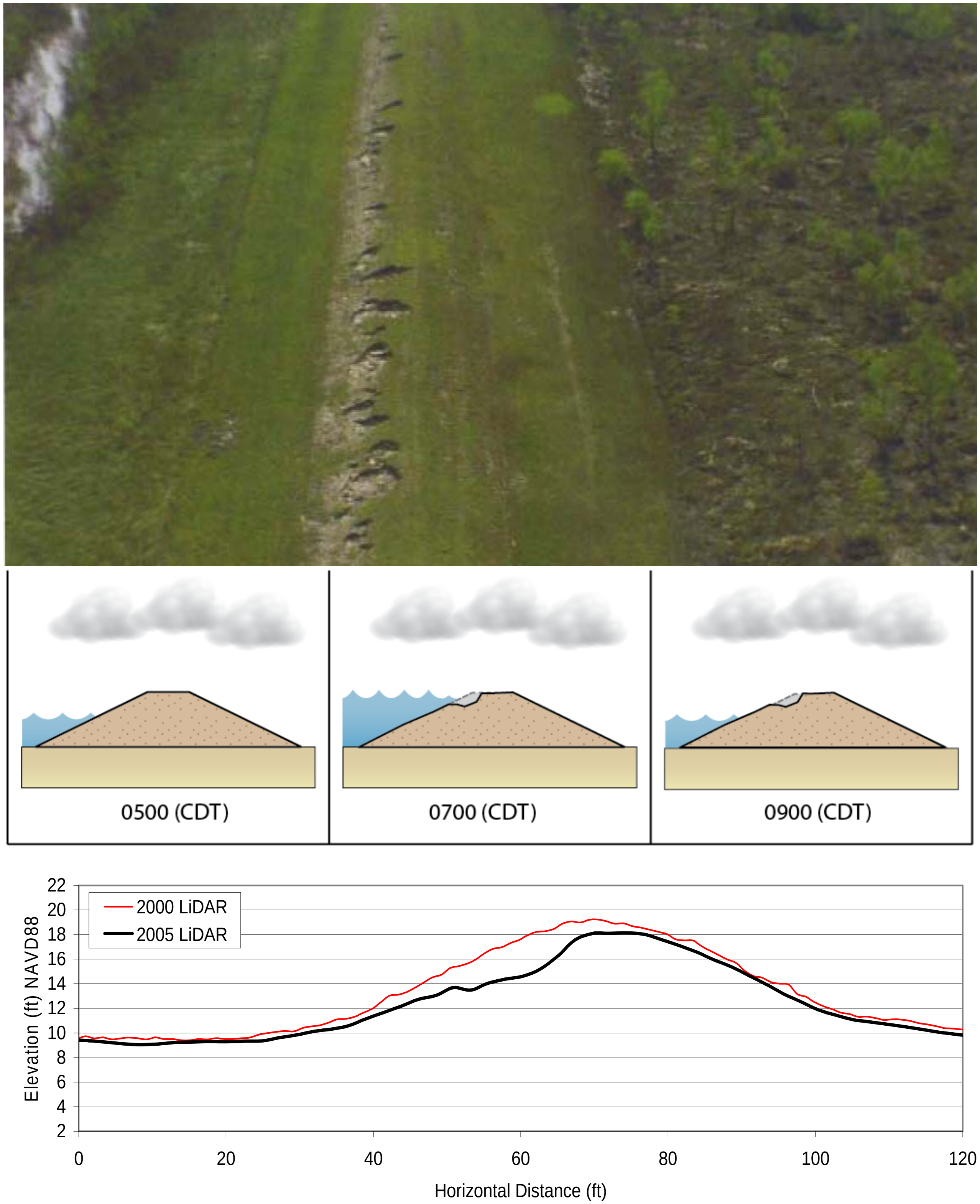


Figure 32: Location E – Wave head cutting (crenellation).

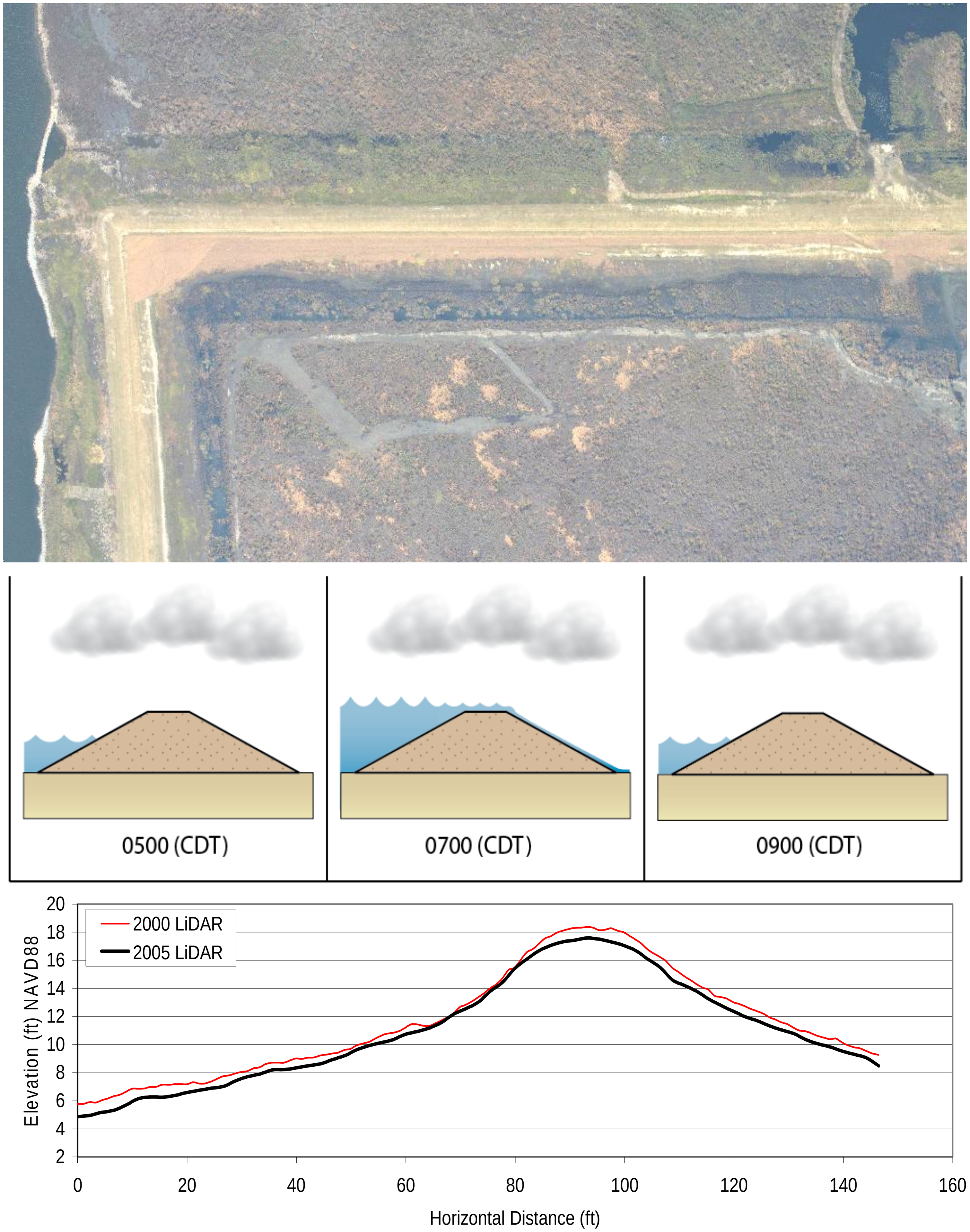


Figure 33: Location F – No breaching.

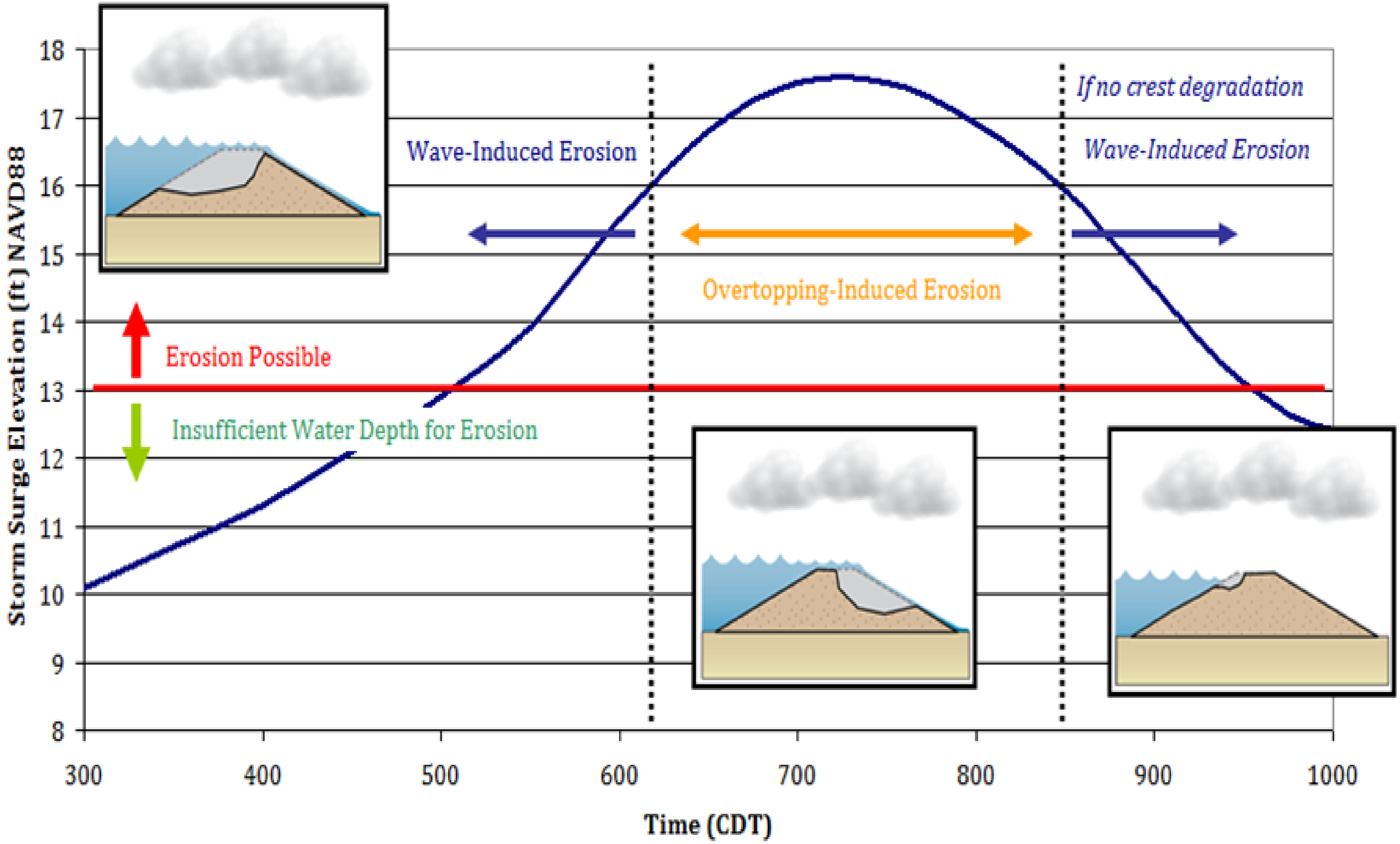


Figure 34: Overview of erosion mechanism timeline.

Conclusion

141. Table 5 summarizes the breaching analyses performed by the Defense and Plaintiffs Experts for each of the erosion mechanism time frames. Grass armor analyses are indicated with an "A," wave-induced erosion analyses performed are indicated with a "W," overtopping-induced erosion analyses performed are indicated with an "O," and if no evaluations were performed, the time period is marked with a "N.E." The significant 'incompleteness' of the breaching analyses documented in the Defense Expert Reports is obvious. As, well the breaching analyses documented in the Plaintiffs Expert Reports are not yet complete (Phase II).

Table 5: Summary of EBSB breaching analyses performed by Defense and Plaintiffs Experts

	Erosion Evaluation Time (CDT)							
Expert	500	530	600	630	700	730	800	830
Bea	A, W	A, W	A, W	A, W	A, O	A, O	A, O	A, O
Ebersole	A	A	A	A	A	A	A	A
Mosher	N.E.	N.E.	N.E.	N.E.	N.E.	N.E.	N.E.	N.E.
Resio	N.E.	N.E.	N.E.	N.E.	O	N.E.	N.E.	N.E.

NOTES

A	Armor analyses to determine if grass cover was compromised
W	Wave-induced erosion analyses to determine the magnitude of lateral erosion due to wave attack
O	Overtopping-induced erosion analyses
N.E.	No Evaluations or analyses performed/documented

142. A Phase I EBSB breach mechanism evaluation of the MR-GO Reach 2 has been completed. A Phase II breach mechanism evaluation has been performed at MR-GO Station 497+00 (Wave Breaching Study Location) and at another location immediately north of Bayou Bienvenue. In addition, a Phase II breach mechanism evaluation has been

performed at the additional locations discussed earlier: A through F (Figure 27). The details of these Phase 2 analyses are contained in Technical Report IV that accompanies this Declaration.

143. MR-GO The MR-GO EBSBs have a length of over 64,000 feet, with significant variability in materials and geometry over this length. The performance of the EBSBs is dictated by the ability of the EBSB to withstand the imposed storm surge and wind waves. There are a number of factors that impact the ability of the EBSB to resist storm surge and waves (riparian/wetland vegetation, armor, geometry, and materials) and factors that impose demands on the EBSBs (bathymetry, topography, storm surge level, and wind-waves). Our Phase I analyses for the MR-GO Reach 2 EBSB breach development mechanisms identified that <u>55% of the breaches could be attributed to surge overtopping initiated breaching</u> (protected side to flood side) and <u>45% could be attributed to wave initiated overtopping exploited breaching</u> (flood side to protected side) These percentages do not include un-breached segments, the sheet piling overtopping and breaching, and the breaches that developed at the EBSB interfaces with the navigation – water control structures at Bayou Dupre and Bayou Bienvenue.

144. <u>The Phase II analyses at the study locations have confirmed my the Phase I assessments.</u> MR-GO EBSBs highly heterogeneous with respect to topography, crest elevations (freeboard), grass cover (armor) characteristics, and soil erodibility characteristics. There was significant variation in these parameters, and a high-resolution analysis is required to capture and evaluate the impacts associated with these variations. Additionally, evaluating all important factors associate with the ‘Demands’ and ‘Capacities’ of the EBSBs are important in order to comprehensively evaluate performance. These analyses demonstrate

that the variable conditions along the MR-GO Reach 2 alignment provide a logical explanation for the varied performance and the presence of multiple breach mechanisms.

One very important insight developed during the Phase 2 analyses. The analyses detained in Technical Report IV Location D directly demonstrates that EBSB materials were of poor quality and susceptible to both wave-induced and overtopping-induced erosion breaching during Hurricane Katrina.

III. PERFORMANCE OF THE MR-GO REACH 1 FLOOD PROTECTION STRUCTURES ADJACENT TO THE LOWER 9TH WARD DURING HURRICANE KATRINA

145. The Defense Experts have cited a large number of concerns associated with my analyses of the breaches in the man-made flood protection structures along the portion of Reach 1 of the MR-GO adjacent to the Lower 9th Ward (Table 2) that developed during Hurricane Katrina. Key concerns documented by the Defense Experts include analyses of how the breaches developed and analyses of the hydraulic conductivity effects my analyses clearly show played major roles in development of both breaches (North and South Breaches).

North & South Breaches

146. The Defense Expert analyses of development of the North Breach are in substantial agreement with those documented in my July 2008 Declarations and Technical Reports. Both sets of analyses indicate that this breach developed before overtopping of the floodwall and developed very early during the morning of August 29, 2005. Both sets of analyses attribute initiation of development of the breach to lateral instability of the flood protection structure (concrete floodwall supported on sheet piling supported by the soil levee and foundation soils) exacerbated by the reduced cross section of the supporting levee at that location. The Plaintiffs Expert analyses also indicate that there was a high likelihood of important hydraulic uplift and seepage effects that contributed to development of this breach. <u>The primary disagreements concerning major causative factors involved in development of the North Breach are focused on the potential multiple interactive modes of failure involved</u>

in development of the North Breach. Further, the Defense Experts have not offered a defensible explanation of why the North Breach developed at the location that it did.

147. The Defense Expert's analyses of development of the South Breach are in substantial agreement with those documented in my July 2008 Declarations and Technical Reports. Both sets of analyses indicate that this breach developed after overtopping of the floodwall and developed very after the North Breach began its evolution during the early morning of August 29, 2005. Both sets of analyses attribute initiation of development of the breach to lateral instability of the flood protection structure (concrete floodwall supported on sheet piling supported by the soil levee and foundation soils) exacerbated by water intrusion into a 'tension gap' on the water side and additional hydrostatic forces generated on the inclined floodwall and supporting sheet piling. The Defense Expert's analyses indicate that the flood protection structure at the location of the South Breach lost its lateral support due to a deep erosion trench that had been eroded by the overtopping surge water in the IHNC. The Plaintiffs Expert's analyses indicate that while the loss of lateral support was a 'closely competing mode of failure,' the failure development 'horse race' was won by hydraulic conductivity uplift pressures developed on the less permeable overlying supporting levee soils. There was also evidence of seepage related hydraulic effects which lowered the lateral resistance of the flood protection structure. The primary disagreements concerning major causative factors involved in development of the South Breach are focused on the potential multiple interactive modes of failure involved in development of the South Breach. Further, the Defense Experts have not offered a defensible explanation of why the South Breach developed at the location that it did.

Hydraulic Conductivities and Effects on Lateral Stability of the Flood Protection Structures

148. My analyses and those performed by Seed et al (2008a, 2008b) show that hydraulic conductivity effects played a major role in development of both the North and South Breaches (Figure 35). These hydraulic conductivity effects are focused on three primary elements: a) the backfilled excavations adjacent to the wall that were associated with the USACE Lock Expansion Project EBIA (East Bank Industrial Area) site clearing activities (refer to Technical Report V), and b) the transmission of water pressures through the buried marsh- swamp layers and under the bottoms of the sheet piling that developed destabilizing forces (uplift) under the protected side of the flood protection structure (protected side portion of the supporting soil levee, and c) the transmission of water through the buried marsh-swamp layers causing seepage related degradations of the soil structure (reductions in strength, erosion, blow-outs).

149. <u>The Defense Experts do not address either the existence of or potential effects associated with the USACE Lock Expansion Project EBIA site clearing activities (poorly backfilled excavations) – this is a serious omission in their 'forensic engineering'</u>. Refer to Technical Report V for more details on the USACE Lock Expansion Project EBIA site clearing excavations. The Defense Expert's seepage and associated stability analyses do not take any account of potential effects of these flood side features. We have expended considerable resources in analyzing the two and three dimensional hydraulic conductivity effects (seepage and associated pressures) and their effects on the stability of this portion of the man-made flood protection structure (refer to Technical Report VI). This work has been and is being published in peer reviewed journal papers and discussions (Bea and Cobos-Roa 2008a, 2008b; Cobos-Roa and Bea 2008).

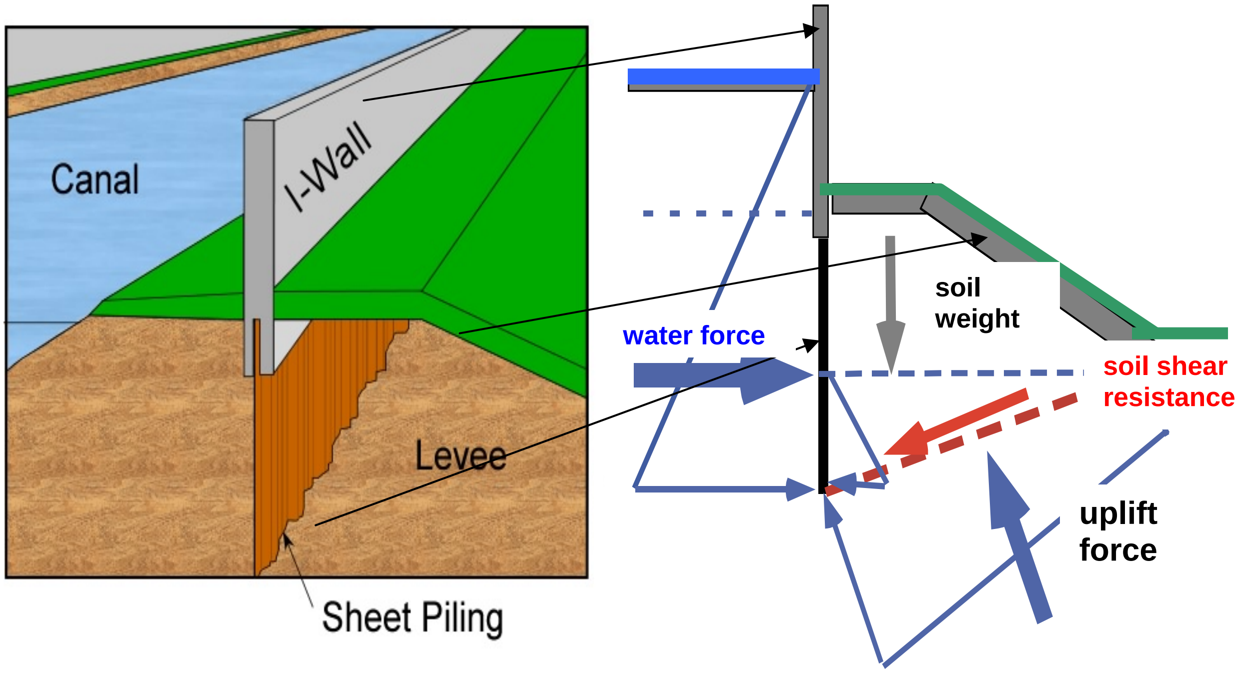


Figure 35: Analytical model illustrating 'demand' and 'capacity' forces involved in breach development (Bea 2008).

150. The Defense Experts (e.g. Dr. Mosher) have erroneously concluded that my analyses indicated the early flooding was associated solely with water 'seepage' under the sheet piling supporting the concrete wall and through the supporting foundation soils. I did not reach such a conclusion nor did I document such a conclusion in my July 2008 Expert Report. The conclusion cited by Dr. Mosher is an'erroneous deduction' based on his analyses of my analyses, conclusions, and opinions.

151. I did conclude that there was a high potential for 'blowout' at the toe of the reduced section of the levee at the Northern Breach that would result in water entering the Lower 9th Ward with the volume increasing with time. My analysis of this breach development 'sequence' included initial lateral differential displacements induced in the flood protection structure that opened up the vertical 'water-stops' separating sections of the concrete flood wall. Our analyses indicate that the first major volumes of flood waters most likely entered the Lower 9th Ward through the openings in the concrete flood wall. Our

breach development sequence indicates as the breach continued to develop, there were additional movements of the flood protection structure – including tilting of the concrete flood wall – sheet pile portion of the flood protection structure that resulted in development of additional paths for water entry from the water side. In addition, field inspections showed there also were failures of the sheet pile interlocks – connections between the sheet pile sections (Cushing2007). This mode of failure was particularly evident at the north end of the North Breach. This sequence of development and causation of the breach is very similar to that we identified as a result of our extensive analyses of the breach that developed at the 17th Street Canal – another failure that developed prior to overtopping (Bea 2008).

152. My analyses indicate that the rising surge flood waters and the associated increased water pressures communicated through the permeable saturated underlying marsh layers both via the adjacent backfilled excavations and the gap formed between the flood wall – sheet piling and the supporting soils. These water pressures were 'blanketed' with the less permeable fill and levee soils on the protected side – thus developing significant uplift pressures (forces) that acted to destabilize this portion of the flood protection structure (Figure 34). These effects have been ignored and not included in the analyses documented by Dr. Mosher in his December 2008 Expert Report.

153. A major point of disagreement developed very early between the results from the IPET (2007) and ILIT (2006) investigator's analyses of the North Breach development. This point of disagreement focused on the hydraulic conductivity properties of the buried marsh layers that comprised portions of the flood protection structure supporting foundation soils. The IPET investigator's analyses of seepage effects are essentially the same as those documented in the cited December 2008 Defense Expert Reports. I am unable to find any

important changes in these analyses since early 2006. This lack of change in the Defense Expert's analyses, conclusions, and opinions during a 2 year period is not indicative of a 'learning organization' (Bea 2000, 2005). Instead, it is symptomatic of an organizational bias: 'don't bother us with the facts, our mind is made up.'

154. Based on results from their very limited analyses, the Defense Experts conclude that seepage and hydraulic effects were not important and did not contribute significantly to development of the North and South Breaches. Unfortunately, they have addressed only one part of the important aspects associated with the hydraulic conductivity of the buried marsh and swamp layers that underlie the Lower 9th Ward (and much of the rest of the greater New Orleans area) (Dunbar 2008, Rogers 2008). I keep samples of these layers that I obtained from the Lower 9th Ward (obtained in 2006 from a crew working on repair of the sewage and water systems) on my desk to remind me of the unusual characteristics of these highly organic – silty soils. These soils 'outcrop' just to the north of the Lower 9th Ward out by the water treatment plant. Water can and does flow easily through these organic soils – like water through compacted 'dirty' sawdust.

155. Results from the later ILIT analyses (Seed et al 2008a, 2008b) and the analyses documented in my July 2008 Expert Report Declarations (II and III) and Technical Reports (II and III) indicate otherwise. Our analyses demonstrated there could be important hydraulic conductivity 'seepage' effects which could lead to 'blowouts'. Most importantly there were also hydraulic effects (pressures in the underlying marsh – swamp layers blanketed with the overlying much less permeable soils) which could lead to 'uplift forces' that could lead to destabilization of the flood protection structures (Figure 35, Bea 2008). My review of the IPET and Defense Expert Reports did not disclose evidence that these analyses

had accounted for such 'uplift forces' nor for the inter-related effects of the EBIA backfilled excavations in their analyses of lateral stability. In addition, my analyses indicate that the Defendant Experts performed seepage and stability analyses – but not a combination of the two. I was unable to detect that the hydraulic conductivity uplift pressure effects had been included in the lateral stability analyses performed by the Defendant Experts (same as work included in IPET final report in 2007). For many years, the existence of these hydraulic uplift effects have been recognized and addressed in design of dams and other structures (e.g. retaining walls, slopes) that must 'keep water friendly' (USACE 1995, 2000, 2002, 2003; Terzaghi and Peck 1948; Hough 1957, Lambe and Whittman 1969; Handy and Spangler, 2007).

156. Much of the deliberations concerning the importance of hydraulic conductivity effects as they contributed to development of both the North Breach and the South Breach have focused on the permeability (water transmissibility - conductivity) characteristics of the underlying marsh - swamp layers. Unfortunately, to my knowledge no appropriate measurements of the hydraulic conductivity characteristics of the buried marsh layers have been performed in this area:

> "*The installation and purpose of the piezometers along the IHNC floodwall were discussed with Mr. Rich Varuso, USACE Geotechmnical Branch Chief, along with the preliminary data obtained. Permeability tests which are being conducted were also discussed; the results of the permeability tests are anticipated by the end of the month and the data will be used in conjunction with seepage calculations to validate the 5.0 factor of safety.*" (Minutes of the Southeast Louisiana Flood Protection Authority – East Board Meeting, Thursday, February 21, 2008).

157. Unfortunately, after repeated requests for this data, I have not been able to get access to this information or to confirm that the measurements were actually performed. If the data were gathered and analyzed, it is curious that the information has not been produced by the Defense nor used by the Defense Experts – unless the information is not favorable to the analyses and conclusions reached by the Defense Experts as documented in the December 2008 Expert Reports. As a result of the lack of appropriate in situ measurements, both sets of analyses – by Defense and Plaintffs Experts - have been forced to rely on 'indirect' information and observations (e.g. results from laboratory and field tests performed on 'similar' soils.

158. In my July 2008 Expert Report Declaration (II) and Technical Report (II) and my associated Declarations and Technical Reports, I summarized the sources of the 'indirect' information used to provide input parameter quantifications for our numerical analyses. These 'indirect' sources of information include: a) results from laboratory tests performed on samples from marsh 'soils' thought to be comparable with those underlying the Lower 9th Ward, b) results from field tests performed in marsh soils comparable with those underlying the Lower 9th Ward, c) observations made at this location during performance of the soil borings for the ILIT project (Figure 36), d) observations made during the EBIA site clearing excavations (water filling excavations), e) observations made during construction of a flood protection drainage structure located north of the Lower 9th Ward (Dwyer Road, detailed in my July 2008 Expert Report) that encountered the marsh layers, and f) observations and measurements made in comparable buried marsh layers (Dunbar2008, Rigers et al 2008, IPET 2007, ILIT 2006) found in the vicinity of the 17th Street Canal Breach (Figure 36). The Defense Experts have chosen to ignore the 'indirect' sources of information concerning the

hydraulic conductivity characteristics of the buried marsh – swamp layers under the Lower 9th Ward,.

159. During performance of the ILIT soil borings at the sites of the North Breach and South Breach at the Lower 9th Ward the very high hydraulic connectivity of the marsh and swamp layers that underlie this entire area was made apparent (Figure 36). After completing one boring at the location, the soil boring rig was moved 200 feet away and a second boring was completed. Because of the variable soil characteristics, it was decided to drill a third soil boring midway between the first two borings. When the drilling unit encountered the buried marsh and swamp layers, water immediately began spouting from the two adjacent soil boring holes. Examination of the soil samples showed that the organic layers were very 'layered' – as one would expect from the accumulation of organic matter in the bottom of a marsh or swamp. The deposits were very anisotropic – the horizontal permeability was much greater than the vertical permeability. This observation showed that conventional sampling and laboratory testing methods (e.g. permeability determined from consolidation tests on samples from vertical cores) could not be relied upon to give realistic data regarding the horizontal permeabilities of these marsh - swamp layers.

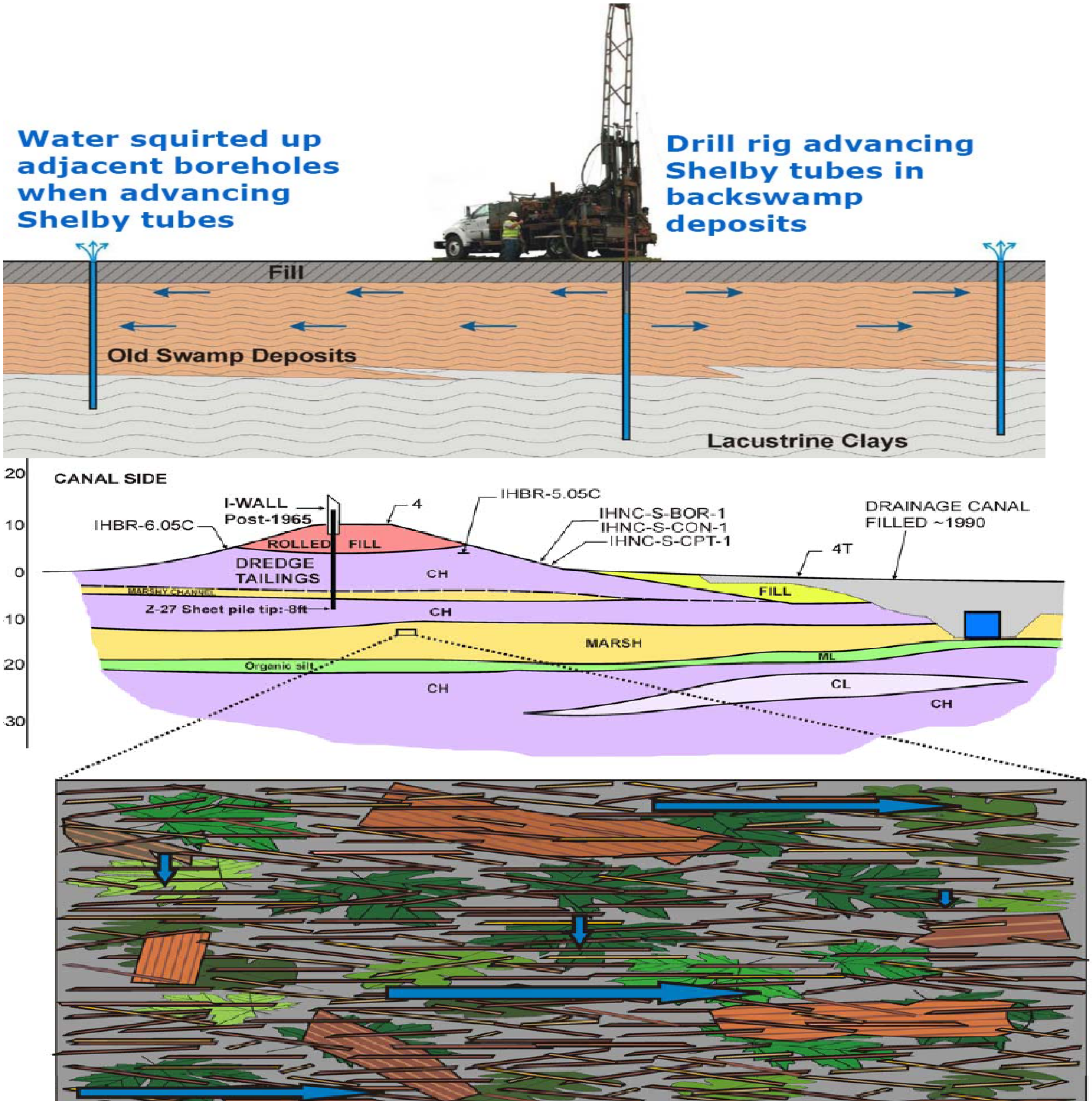


Figure 36: Observations made during soil borings performed at Lower 9th Ward breach locations that penetrated the buried marsh layers (Bea 2008 from Rogers 2007).

160. The piezometer and water level measurements made by the USACE in the buried marsh layers that underlie the 17th Street Canal (SSC) Breach (Figure 37) had particular importance in development of our understanding of the hydraulic conductivity and hydraulic pressure response characteristics of these buried marsh soils. As part of the

USACE's study of "Safe Water Levels" for the SSC (USACE 2007), piezometers were installed on the Orleans Parish and Jefferson Parish sides of the SSC in the immediate vicinity of the SSC breach that developed during Hurricane Katrina. These piezometers (identified by USACE incorrectly as SSP-1A and SSP-2A; based on the piezometer installation records and report the correct designation is SSP-1B and SSP-2B) were installed in the peat (Marsh) layers at elevations – 14.24 feet and –15.9 feet (NAVD88), respectively. These piezometers were monitored manually from January to June 2006. The reading were adjusted for barometric pressure changes. Piezometer SSP-1B was tipped in a marsh - swamp layers under the Orleans Parish levee centerline. Piezometer SSP-2B was tipped in the marsh – swamp layers under the Jefferson Parish side of the canal at the protected side levee toe. As for SSP-2B, the piezometer readings varied between El. –5 feet to –6 feet (NAVD88) during the measurement period. Both piezometers showed similar trends. A summary of the SSP-1B readings is given in Figure 36. Some very important information is contained in these records. As indicated in Figure 36, there is direct correspondence between changes in the water level in the canal and those in the buried marsh layer/s. Several instances are shown in Figure 17 where there is an immediate and direct correlation between sudden changes in the canal water level and that in the buried marsh layer/s. This indicates a direct connection between the canal water and that in the marsh layer/s. Similar marsh layers underlie the Lower 9th Ward and have similar hydraulic 'connections' with the adjacent Inner Harbor Navigation Canal (IHNC). Refer to Technical Report IV for details on how these hydraulic connections were developed during the USACE IHNC Lock Expansion Project EBIA site clearing activities.

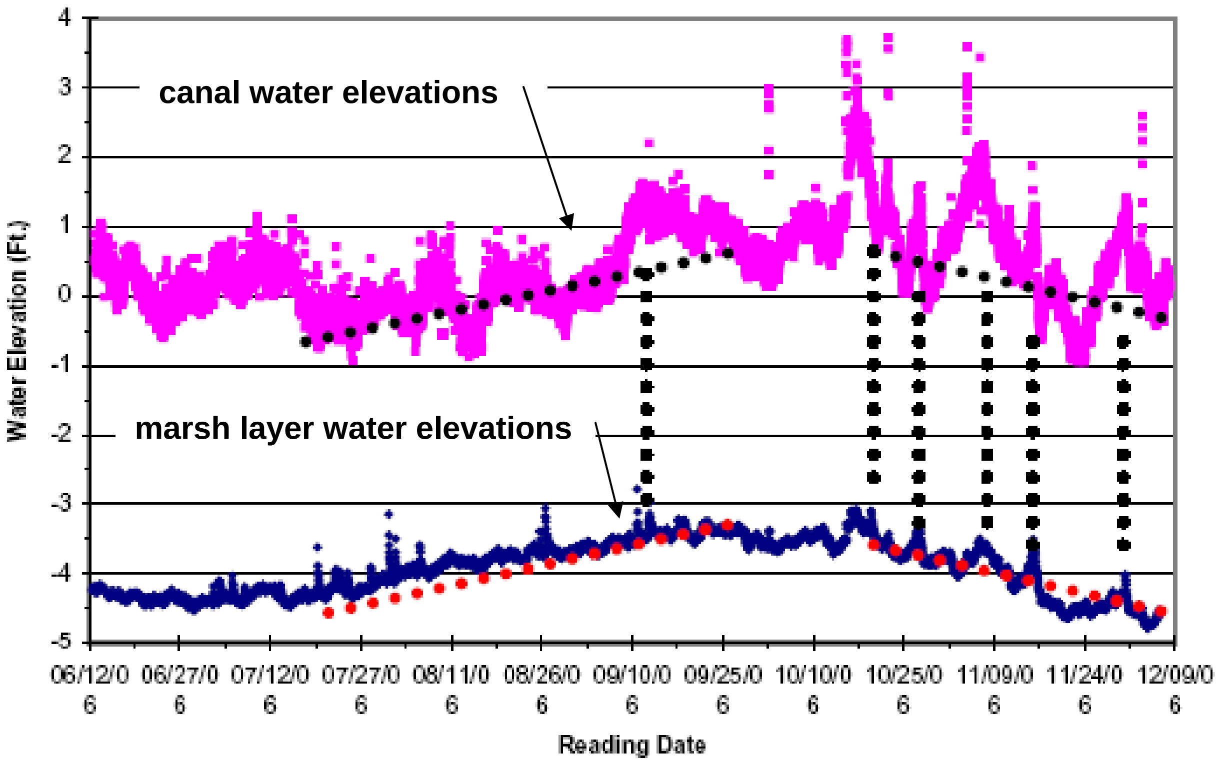


Figure 37: Piezometer (water pressure elevations) measurements made in the buried marsh layers adjacent to the 17th Street Canal breach and canal water elevations recorded during a six month period (USACE 2007 17th Street Canal Safe Water Levels Report).

161. These data show how high water levels in the adjacent IHNC can lead rapidly to high water pressures in the underlying marsh layer/s. As shown by the analyses summarized my July 2008 Expert Report Declaration (II) and Technical Report (III), these high water pressures if exerted on an overlying 'blanket' (relatively impermeable) layers can develop upward pressures that act to reduce the normal downward pressures exerted by the overlying soils that are important to stability of the flood protection structures.

162. Similar uplift pressure – flood protection structure concerns developed during our recent investigations of the breaches that developed in the mid-west levees during July 2008 (Storesund et al 2008). Potential breach development exacerbating effects of these uplift pressures were identified by USACE engineers from the St. Louis District (Figure 38). The uplift pressures reduce the 'effective' weight of the soils that provide a substantial

portion of the lateral load resistance of the flood protection structure – levee. As a consequence, there were many instances of breaches in the levees that developed without and well before overtopping (e.g. Figure 39). It is interesting to note that in many cases it was initially reported that the majority of major breaches had occurred due to overtopping – the public was told that when a levee was overtopped “the levee failed” (Storesund et al 2008).

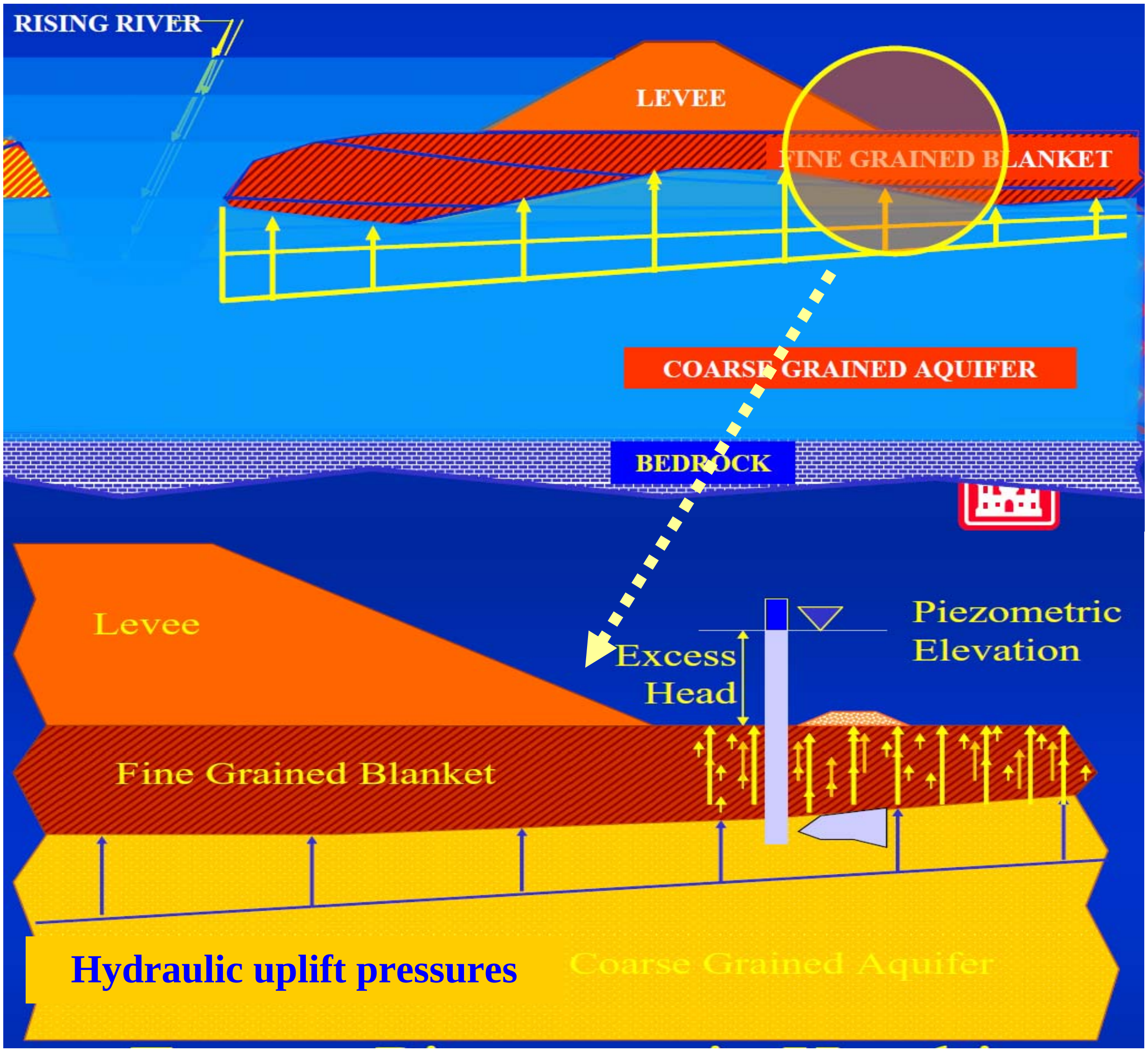


Figure 38: Hydraulic conductivity – underseepage uplift pressures from “Underseepage – The Silent Killer of Levees” (Conroy 2008).

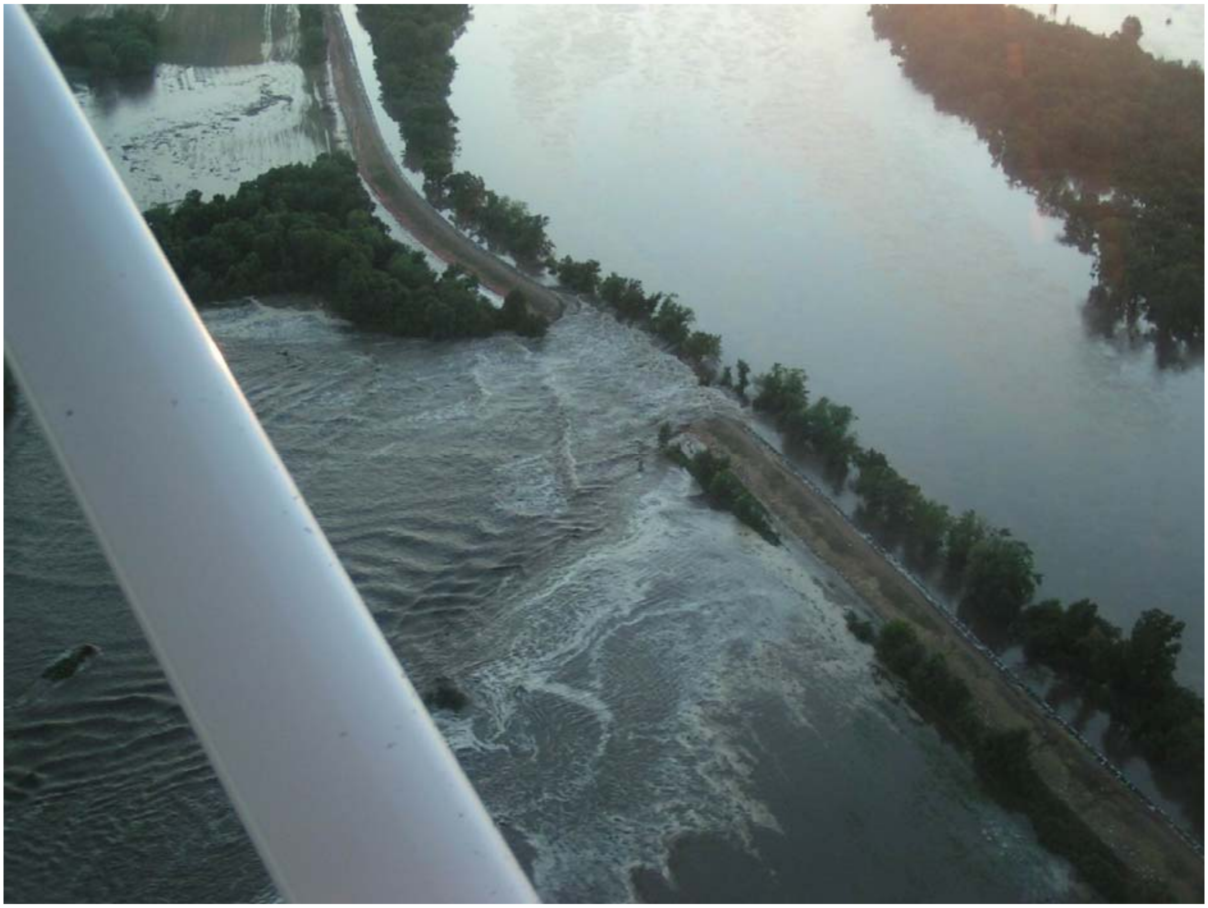


Figure 39: Levee breached before overtopping

163. The photograph shown in Figure 40 was taken on a farm next to the river during the high water stages of the July 2008 mid-west flood. The windmill in the photograph was used to pump water from the shallow (20 feet below surface) sand aquifer under the cornfield. This sand aquifer connects to the nearby river. At normal water stages, the water had to be pumped to the surface. It is evident that when this photograph was taken (high water stage in the river. The water pressures in this shallow sand aquifer obviously are very high – a 'gusher' of sediment laden water is being forced almost to the top of the windmill. After this photograph was taken, a breach developed in a section of the adjacent levee (Figure 41) (Storesund et al 2008).

Figure 40: Shallow aquifer water pressures developed during flood stage

Figure 41: Levee adjacent to windmill developed breach prior to overtopping

164. Assembly of the available background cited herein and in my July 2008 Expert Report indicates that the marsh – swamp layers that underlie the Lower 9th Ward could have a very wide range in 'insitu' (in the actual marsh layers) permeabilities (hydraulic conductivities) (see Table 6 for the range of values considered, also consult Mitchell 1976). Generally, there are important differences between permeabilities determined based on results from laboratory and field tests (e.g. differences caused by layering of the organic matter, sampling disturbances, laboratory test induced effects). There are similar potential differences between insitu horizontal and vertical permeabilities – due principally to the macroscale characteristics (layering) of the organic materials in marsh layers. Complicating this already complicated 'picture' of the marsh layer permeabilities are the natural variabilities one could expect in the properties of buried marsh layers and the additional variabilities one could expect in the properties as influenced by 'man-made' activities (e.g. increased overburden stresses due to the flood protection structure, excavations, and placement of underground utilities). Thus, this 'indirect' information had to be assembled in a reasonable way to enable development of realistic understanding and insights into the potential hydraulic effects. This assembly, its use in parametric studies (upper bound to lower bound), and the results for our 'best estimate' case are summarized in my July 2008 Expert Report Declarations and Technical Reports. <u>During my study of the background summarized in the December 2008 Defense Expert Reports, I was not able to find any similar parametric studies; single values for parameters were used in these analyses and then general conclusions were drawn from the 'restricted' analyses.</u>

Table 6: In situ intrinsic horizontal permeabilities of earthen materials (after Bear 1972)

<table>
<tr><td>K (cm/s)</td><td>10^2</td><td>10^1</td><td>$10^0=1$</td><td>10^{-1}</td><td>10^{-2}</td><td>10^{-3}</td><td>10^{-4}</td><td>10^{-5}</td><td>10^{-6}</td><td>10^{-7}</td><td>10^{-8}</td><td>10^{-9}</td><td>10^{-10}</td></tr>
<tr><td>K (ft/day)</td><td>10^5</td><td>10,000</td><td>1,000</td><td>100</td><td>10</td><td>1</td><td>0.1</td><td>0.01</td><td>0.001</td><td>0.0001</td><td>10^{-5}</td><td>10^{-6}</td><td>10^{-7}</td></tr>
<tr><td>Relative Permeability</td><td colspan="4">Pervious</td><td colspan="4">Semi-Pervious</td><td colspan="5">Impervious</td></tr>
<tr><td>Aquifer</td><td colspan="5">Good</td><td colspan="4">Poor</td><td colspan="4">None</td></tr>
<tr><td>Unconsolidated Sand & Gravel</td><td colspan="2">Well Sorted Gravel</td><td colspan="3">Well Sorted Sand or Sand & Gravel</td><td colspan="4">Very Fine Sand, Silt, Loess, Loam</td><td colspan="4"></td></tr>
<tr><td>Unconsolidated Clay & Organic</td><td colspan="4"></td><td colspan="2">Peat</td><td colspan="3">Layered Clay</td><td colspan="4">Fat / Unweathered Clay</td></tr>
<tr><td>Consolidated Rocks</td><td colspan="4">Highly Fractured Rocks</td><td colspan="3">Oil Reservoir Rocks</td><td colspan="2">Fresh Sandstone</td><td colspan="2">Fresh Limestone, Dolomite</td><td colspan="2">Fresh Granite</td></tr>
</table>

165. Initially, the ILIT project investigators performed seepage analyses (transient and steady-state) based on an in situ horizontal permeability of the marsh layers of 10^{-2} centimeters per second (cm/sec). Dr. Mosher erroneously makes a direct connection between the initial ILIT analyses (May 2006) and the analyses documented in my July 2008 Expert Report. <u>This erroneous connection is then propagated to develop additional erroneous conclusions regarding the analyses performed by the Plaintiffs Experts – again, the wrong dots are connected in the wrong ways resulting in the wrong conclusions.</u>

166. The IPET investigators considered the initial value of permeability used in the ILIT seepage analyses was much too high. Subsequent to publication of the ILIT report (May 2006), the IPET investigators founded their seepage analyses on a constant value of 10^{-6} cm/sec for the in situ horizontal permeability of the marsh layers. It is these results that are referenced in the cited Defense Expert Reports (e.g. December 2008 Expert Report by Dr. Mosher). Following publication of the original ILIT analyses, the ILIT analyses were repeated and expanded to include a much wider range of potential – plausible - insitu

horizontal permeabilities; 10^{-3} to 10^{-6} cm/sec (Seed et al 2008a, 2008b). This range of permeabilities were studied in the analyses documented in my July 2008 Expert Report. As would be expected, for horizontal permeabilities in the lower range there were much less significant hydraulic 'seepage' effects. However, the analyses indicated hydraulic uplift pressures developed under the relatively less permeable levee soils were relatively insensitive to the range in horizontal permeabilities analyzed (10^{-3} to 10^{-5} cm/sec).

167. Our work (Bea and Cobos-Roa 2008, Cobos-Roa and Bea 2008) has clearly shown that there are two categories of effects that are associated with hydraulic conductivity: a) seepage, and b) uplift pressures. Seepage involves underground transmission of significant quantities of water. Development of uplift pressures does not (saturated soils, incompressible water-soil system). Seepage can develop several deleterious effects on soils; e.g. weakening the soils and transporting significant quantities of the soil thereby developing expanded conduits for water transmission – causing 'blowouts'. As discussed previously, uplift pressures act to decrease the 'effective' weight of the soils and thereby reduce the ability of the soils to resist lateral loadings. Initially, our primary concern was with the seepage effects, but as we learned more about the behavior of the soils and the performance of the flood protection structures during Hurricane Katrina, our primary concern focused on the effects of the hydraulic uplift pressures on lateral stability of the flood protection structures (Conroy 2008).

Fog Created by Faulty Forensic Engineering Analyses

168. As a consequence, the conclusions reached and opinions documented by Dr. Mosher in his December 2008 Expert Report are misguided and incorrect as they relate to my analyses of hydraulic conductivity effects. His conclusion that:

> *"Because the unrealistic permeability assigned to the marsh material for Dr. Bea seepage analyses was at least 1,000 times too high, the results of the seepage analyses described in that his report do not reflect the real seepage conditions in the field. Because it was assigned such a high permeability, the marsh layer appeared in those analyses to have very low resistance to seepage, and to respond very quickly to the rise in canal water level. This behavior is not consistent with the actual behavior of marsh material and peat, especially when consolidated under the weight of the levees. Based on this mistaken choice of marsh permeability, and the ensuing unrepresentative analytical results, Dr. Bea's report offer a misguided conclusion for the p3rformacne of the I-walls along the IHNC adjacent to the Lower 9th Ward. He further tries to imply that his unrealistic results from seepage analyses have effect on the slope stability analyses he performed. Because he used undrained shear strengths for the marsh soils, he is implicitly stating the marsh soils are so impervious that they will not drain when sheared and strength is not affected by pore pressure during the analysis" (page 183, Mosher, December 2008 Expert Report).*

169. The analysis performed by Dr. Mosher (essentially the same as documented in the IPET Report Chapter V Appendices 2006-2007) and the conclusions reached on the basis of this analysis clearly show how flawed premises when combined with flawed analytical methods can and will lead to deeply flawed conclusions and opinions. <u>First, Dr. Mosher does</u>

not know nor can he define objectively the 'true' in situ values of permeability. He must rely on the same body of knowledge and technology that I have used. However, he chooses to ignore in situ measurements and data gathered by the USACE in similar marsh and swamp soils adjacent to the breach that developed at the 17th Street Canal (Orleans Parish side) – and on the other side of the canal (Jefferson Parish side) in the same layers (USACE 17th Street Canal Safe Water Levels report, 2007). In addition, he ignores the reported and observed rapid influx of water and seepage associated with the EBIA Lock Expansion Project excavations and with the excavations made north of the Lower 9th Ward during construction of the Dwyer Road drainage structures.

170. His reference to my use of a permeability that is "1,000 times too high" is not correct. That value was initially used during the ILIT studies (ILIT 2006). Subsequent analyses by the ILIT investigators have examined a much wider range of permeabilities (Seed et al 2008). We have also used a wide range of permeabilities – and we have not used the referenced permeability of 10^{-2} cm/sec as our 'best estimate' permeability; we have used this value as an "upper bound". The "lower bound" permeability was established as 10^{-6} cm/s (e.g. Table 6). The most regrettable element in the 'fog' created by Dr. Mosher in his December 2008 Expert Report, is that he fails to recognize that we have found that the important hydraulic uplift pressures are virtually independent of the permeability values assigned to the marsh and swamp layers. Certainly, the permeability values affect the hydraulic seepage (water transmission) gradients and their potential for creation of 'blowouts.' But, that mode of failure is not the primary mode of failure of concern in my analyses of the lateral stability of the flood protection structure at either the North Breach or the South Breach. All of this experience was documented in my July 2008 Expert Report.

171. <u>Second, Dr. Mosher's analysis of our forensic engineering analyses is deeply flawed as it relates to the characterization of the strength properties of the soils involved in the lateral stability analyses.</u> He criticizes the use of undrained shear strengths for these soils, yet he uses undrained shear strengths in his (ILIT's) analyses (e.g. page 76 December 2008 Expert Report by Dr. Mosher). He ignores the fact that the characterization of the soil shear strengths must involve a variety of 'corrections' to laboratory and in situ field tests that incorporate recognition of disturbance, rate of loading, state of stress and strain, and other important effects which he chooses to ignore. Drained (soil water content changes) versus undrained (soil water content does not change) behavior in these soils is not only function of the soil permeability, but as well on the following factors: cohesive (pore water pressure independent) and frictional (pore water pressure dependent) components of the soil strength, intergranular pressures, intermediate principal stresses, speed of shearing, colloidal phenomena, and degree of progressive straining. Interrelated factors include the soil stress history, drainage conditions, the intrinsic pressures, consolidation pressures, apparent friction angles, void ratio, water content, and air or gas content (Bea 1960, 1963, 1981, Taylor 1948, Lambe and Whitman 1969). In these shallowly buried marsh and swamp soils, air and / or gas (methane generated by decomposition of the organic matter) can assume particular importance in determining whether or not the soil behaves in situ in a drained or undrained manner (Bea 1975).

172. The methods we have used to determine the 'effective' soil shear strengths are detailed in the refereed journal publications by Seed et al (2007, 2008, cited in my 2009 Vita and herein), Bea (1960. 1963, 2008), Bea and Cobos-Roa (2008), and Bea, Storesund, and Cobos-Roa (2008). These methods were also applied by Brandon, Vroman, and Duncan in

their report to the USACE titled "Evaluation of 17th St. Canal DSS (Direct Simple Shear) data" (part of the USACE documentation accompanying the report on the 17th Street Canal Safe Water Levels). Determination of the effective shearing strength of the soil is dependent on the analysis method (total stress or effective stress which takes into account pore water pressure changes), the methods used to 'sense' the soil shear strength properties (in situ testing and laboratory tests performed on samples retrieved from soil boring – coring). <u>Contrary to the assertions by Dr. Mosher, we have used 'state of the art' methods to determine the appropriate strength characteristics for the soils and analytical methods used in our seepage, stability, and combined seepage and stability analyses.</u>

173. The 'blindness' and focus on 'single mechanisms' to explain failures contained in the IPET work and documented in Dr. Mosher's December 2008 Expert Report was identified very early by the American Society of Civil Engineers Hurricane Katrina External Review Panel (2006, 2007) and by National Research Council (NRC) Committee on New Orleans Regional Hurricane Protection Projects (2006):

> *"<u>There is no quantitative assessment of sample and test quality</u>. Measurements of undrained shear strength for each of the key low-permeability soil layers comprising compacted fill, marsh, peat, and lacustrine clay units are aggregated."*
>
> *"It is important for the IPET to integrate the results of field observations, limiting equilibrium analyses (<u>using both circular and planar sliding surfaces</u>), finite element simulations, and centrifuge tests <u>to show the most likely failure mechanisms</u> in a more convincing way, with greater consistency among the physical and analytical models, field data, and failure observations on-site."*

"The IPET should also consider the emerging results from the study being conducted by scientists sponsored by the National Science Foundation (ILIT). Moreover, the IPET team should be aware of alternative failure mechanisms and assess the potential for instability at other locations along the levee system, using all failure mechanisms that are appropriate for the soil conditions and levee geometry at hand."

"Examples of other lingering issues regarding alternative failure mechanisms include the impacts of large differences in settlement across the protection system, the toppling of large trees that had encroached into levees, and the presence of soft clay layers not identified in present geotechnical investigations. Without considering these types of factors, the proposal of a single failure mechanism could lead future designers to focus on narrowly drawn conclusions, leading to neglect of other, equally plausible failure modes. The IPET final report should include a broader discussion of these other possible modes of failure." (underlines added for emphasis)

174. Contrary to the recommendation provided by the NRC Committee, in his December 2008 Expert Report (based on the IPET investigation results), Dr. Mosher continues to rely on limit equilibrium circular sliding plane methods as the primary method to analyze the mechanics of the complex failures that developed during evolution of the North and South Breaches:

"The analyses described here were performed using the computer program UTEXAS4[5]*. Critical circular slip surfaces were located for each case using the search routines available in UTEXAS4."* (page 86 December 2008 Expert Report by Dr. Mosher).

175. <u>In his December 2008 Expert Report, Dr. Mosher develops an incorrect link between an unrealistic characterization of the permeability of the soil and my characterization of the soil strength properties. Further, he completely ignores hydraulic uplift pressure effects.</u> Most importantly, the shear strengths and other soil properties used and employed in a particular lateral stability analytical model must be properly validated with prototype performance information. <u>Dr.Mosher has not offered any such validations for his analyses.</u> However, we have documented (in peer reviewed journal publications) such validations based on analyses of field experimental test results (e.g. E99 floodwall tests) and analyses to determine the water levels at which the breaches developed at the 17th Street Canal (a very similar geotechnical setting) and at the Lower 9th Ward breaches. Dr. Mosher has not been documented similar validations. We have performed validation analyses using the results from the prototype field experiments performed by the USACE in the Atachafayla River Basin – identified as the E-99 Pile Load Tests (Bea and O'Reilly 2008; Declaration on 17th Street Canal breaching by Dr. Bea 2007).

176. Dr. Mosher's analyses of development of the breaches that occurred at the Lower 9th Ward during Hurricane Katrina are deeply flawed. In addition, Dr. Mosher's analyses of the forensic engineering analyses I have performed to identify and determine the causative factors involved in development of these breaches are similarly deeply flawed. <u>Dr. Mosher connects the wrong 'dots' in the wrong ways for the wrong reasons.</u>

177. A prime example of Dr. Mosher's deeply flawed 'rush to judgment' is illustrated as follows:

> *"This calls into question the rest of Dr. Bea's investigation of the breaches along the IHNC east bank Lower Ninth Ward. While Dr. Bea tries to use unclear and*

compromised evidence from observations to bolster their (his) underseepage hypothesis, he overlooked the direct comparison of field observations between the south breach at the east bank of the IHNC Lower Ninth Ward, Figure B-21, and the I-wall on Citrus back levee along the GIWW, Figures B-22, B-23, and B-24" (page 183, December 2008 Expert Report by Dr. Mosher).

<u>Contrary to Dr. Mosher's observations and conclusions, I did perform and document corroborating analyses involving comparisons of the South Breach and performance of a very similar flood protection structure at the Citrus Back Levee (see my July Expert Report pages 95 – 98, photographs of the same areas cited by Dr. Mosher are included in my July 2008 Expert Report).</u> When I did this work, I was concerned with the fact that both flood protection structures had been heavily overtopped with the result that an erosion trench with a similar depth was developed on the protected side of both flood protection structures. The question was whether or not the deep erosion trench was a primary 'player' in development of the South Breach. <u>My analyses showed that the primary difference between the two different flood protection structures was that one was supported on the marsh – swamp layers (South Breach at the Lower 9th Ward) and the other (Citrus Back Levee floodwall) was not.</u> The Citrus Back Levee wall that had been heavily overtopped and damaged (leaning, out of alignment) but not failed, was not underlain by buried marsh – swamp layers. These layers were not present and thus the very important hydraulic uplift pressures and forces and related seepage effects could not be developed. These conclusions also were supported by results from finite element analyses of the lateral stability of the flood protection structure at the location of the South Breach that included explicit analyses of the effects of the deep erosion trench

developed by the overtopping surge waters in the IHNC (Seed et al 2008, Expert Report by Bea July 2008).

178. Initially, in analysis of lateral stability of the flood protection structures, it was assumed that there were no significant effects on lateral stability of the flood protection structure associated with hydraulic uplift pressures developed in the marsh layers under the flood protection structures. Seepage analyses were performed to determine the potential for blow-out initiated failures. Lateral stability analyses were performed to determine the potential for lateral stability initiated failures. Initially, 'coupled' analyses (seepage and stability) were not performed. As our understanding developed, we performed and validated lateral stability analyses that included the hydraulic uplift pressures. As a result of the revised analyses that accounted for the hydraulic uplift pressures determined from the seepage analyses were able to get very close agreements between recorded and reported water levels in the 17th Street Canal at the time of the initiation of the breach with those based on results the lateral stability analyses. Summaries of results from this additional work have been published in peer reviewed conference and journal papers (Bea 2008; Bea and Cobos-Roa 2008, Cobos-Roa and Bea 2008).

179. The hydraulic conductivity seepage and pressure effects developed from the analyses of development of the breach at the 17th Street Canal and at the Lower 9th Ward were based on two-dimensional (2D) finite element analyses. Subsequently, this work has been extended to three-dimensional (3D) finite element analyses (Cobos-Roa and Bea 2008). This additional work has included study of the North and South Breaches at the Lower 9th Ward (Cobos-Roa and Bea 2009; see Technical Report VI for additional details of this work). The additional work has shown that the two-dimensional hydraulic conductivity

analyses tend to substantially underestimate seepage and hydraulic pressure effects (Figures 41 and 42). These findings have been corroborated by other investigators (e.g. Money 2006). The results of the 17th Street Canal failure analyses showed that the 2D hydraulic gradients were increased by about 60%. Computed hydraulic gradients determined for the North and South Breaches at the Lower 9th Ward increased by 30% to 50% , respectively (Cobos-Roa and Bea 2009).

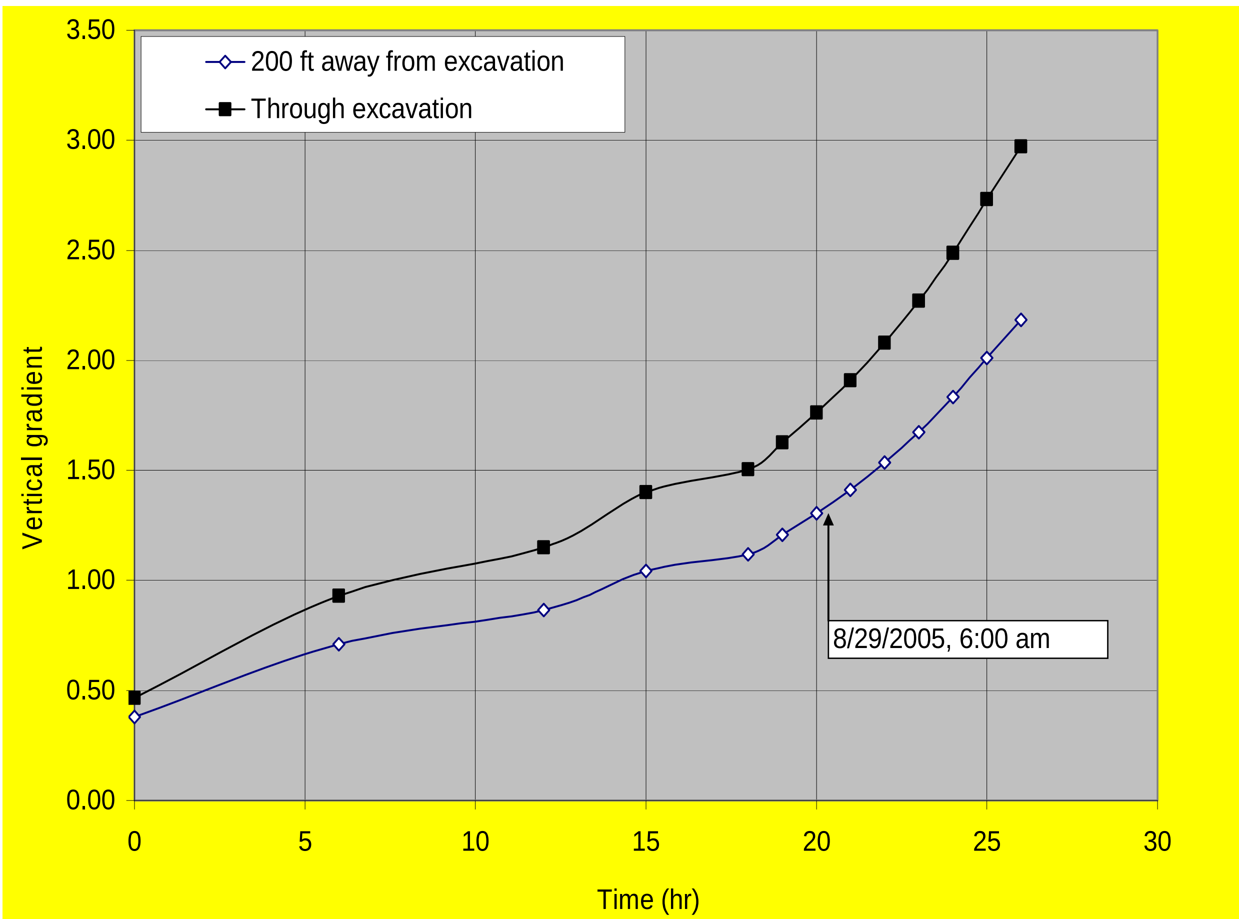


Figure 42: 2D and 3D hydraulic gradients for the North Breach

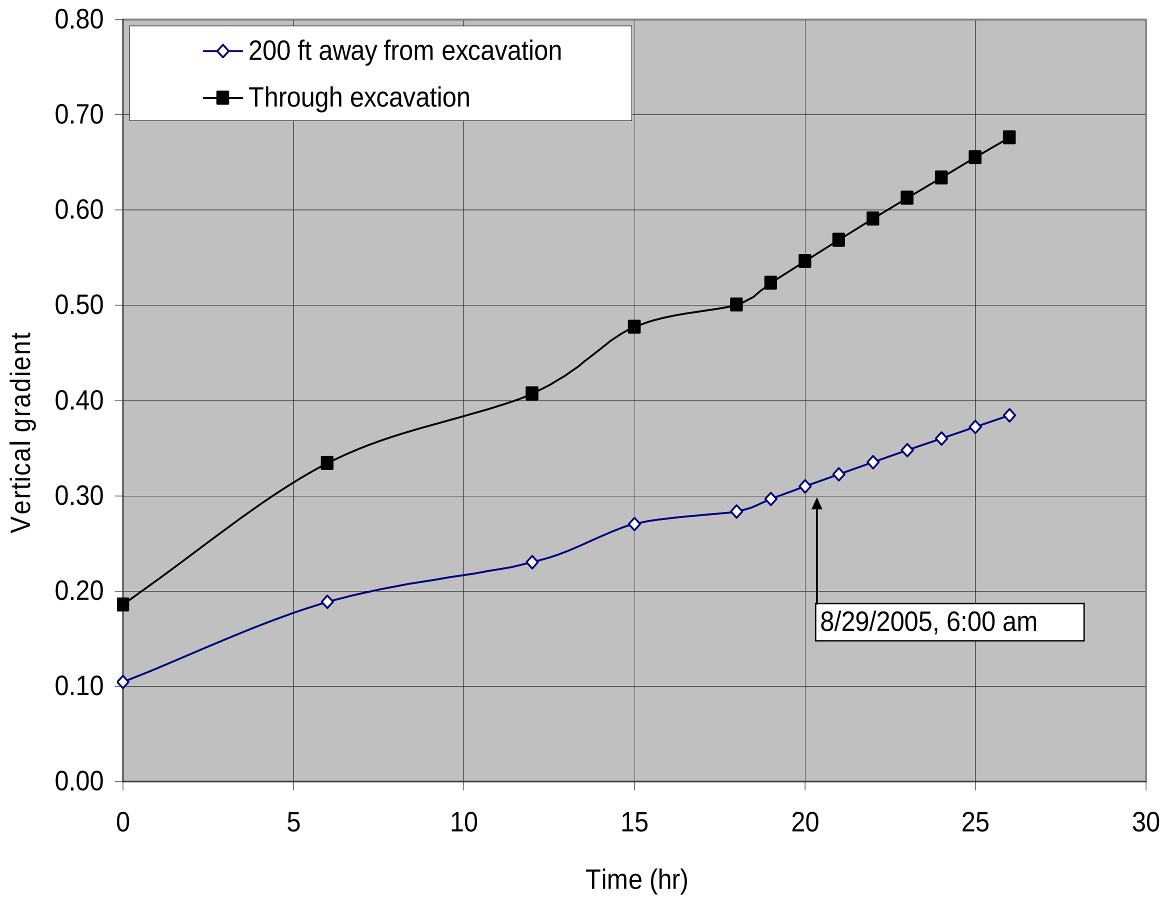


Figure 43: 2D and 3D hydraulic gradients for the South Breach.

180. The 2D versus 3D seepage analysis results are indicative of the effects of another important assumption embedded in the analytical work performed to this time (ILIT, IPET, and Plaintiffs Expert studies through July 2008). <u>All of the previous cited studies have relied on results from 2D finite element seepage analyses – the actual geometric and properties characteristics are 3D.</u> The 2D results are unconservative in that they do not fully characterize the hydraulic conductivity effects. There has not been any recognition of nor analyses of 3D seepage effects documented in the December 2008 Defense Expert reports. <u>Thus, the analyses cited by Dr. Mosher have underestimated the hydraulic conductivity – seepage effects by substantial amounts.</u>

181. Returning to the permeability characterizations applied in my studies of the breaches at the Lower 9th Ward, contrary to statements made by the cited Defense Experts, we did not base our results on a horizontal insitu permeability of 10^{-2} cm/sec. Our analyses were based on a much wider range in horizontal 'free field' (removed from overburden effects of soil levee component of the flood protection structure) permeabilities – 10^{-2} to 10^{-6} cm/sec (range of values between the 'lower bound' and 'higher bound' values). The vertical permeabilities were defined as a factor of 10 smaller than these values to recognize the layering characteristics found in samples of these soils during the ILIT studies. We reduced both vertical and horizontal permeabilities by a factor of 10 in the area under the soil levee that comprised part of the flood protection structure to recognize the overburden pressures effects developed by the levee.

182. The Defense Experts correctly observed that in my July 2008 Expert Report, I presented detailed results that referenced the seepage results associated with a 'best estimate' 'free-field' insitu horizontal permeability of 10^{-3} cm/sec. I analyzed the effects of the potential ranges in these results using a formal reliability based analysis of the Type I uncertainties (natural variabilities) of the soil properties. Details of these analyses are included as an appendix in my July 2008 Expert Report Technical Report (III). Most important, we based all of the stability analyses Factors of Safety on similar reliability based analyses that incorporated explicit analyses of the Type I uncertainties associated with the soil characteristics. The Factors of Safety that are portrayed are those based on the Mean Factors of Safety that were defined as a result of these analyses. The detailed analyses also resulted in definition of the uncertainties associated with the calculated Factors of Safety.

183. <u>Consequently, the Defense Experts are not correct in their observations concerning our 'focus' on 'single-valued' analyses. Unlike the results documented in the IPET report and summarized in the Defense Expert Reports, we have conducted formal reliability based analyses of the Type I uncertainties associated with the Factors of Safety that have been used to define the onset of the major sequence terminating in 'failure' (loss of lateral stability) that our analyses indicate resulted in development of the South and North Breaches (Figures 43 and 44).</u> These 'best estimate' lateral stability analyses result in definitions of the surge water levels that were present (observed) at the time of loss of lateral stability. As documented in my July 2008 Expert Report Declaration (I) and Technical report (I), the times associated with the failure sequence that led to the loss of lateral stability of the flood protection structures correlates well with the timing of observed and reported flooding in this area. The Defense Experts are encouraged to perform similar analyses to enable them to develop an understanding of the uncertainties associated with the results from their analyses.

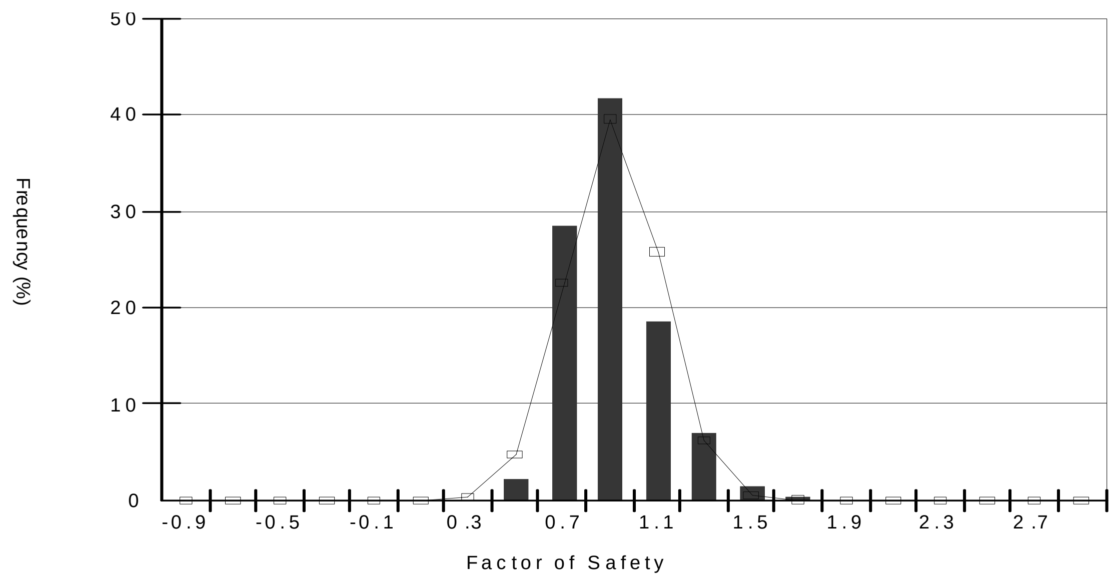


Figure 44: Type I uncertainties associated with computed lateral stability Factors of Safety at the South Breach (Mean Factor of Safety = 0.91, Coefficient of Variation = 28%).

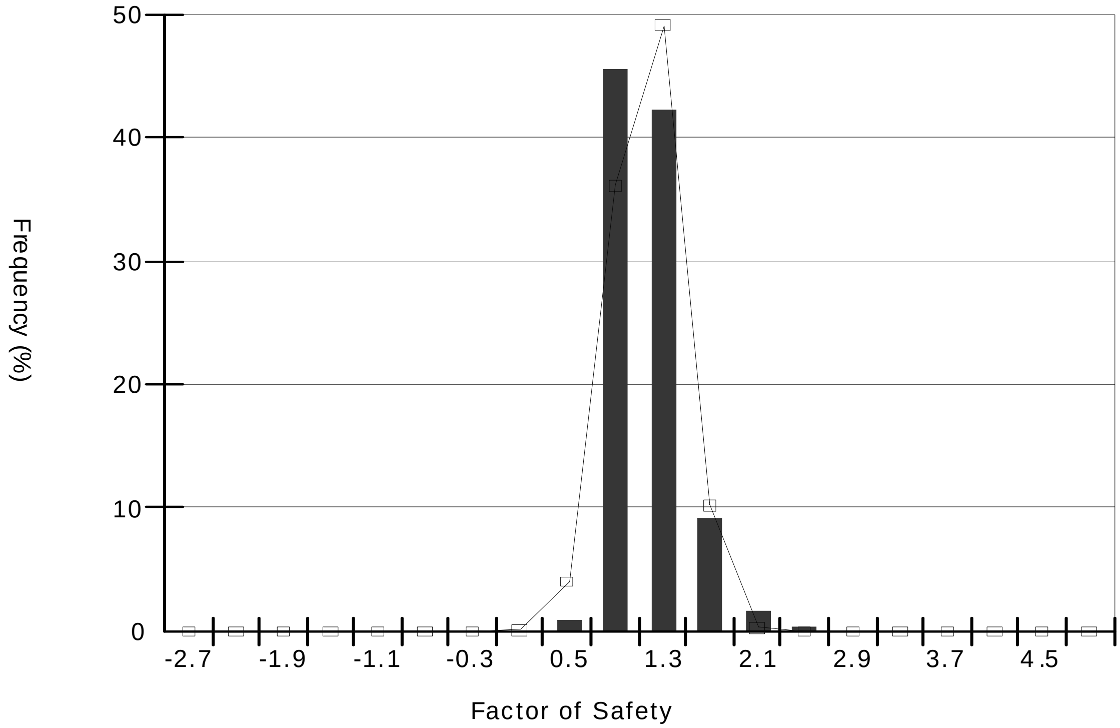


Figure 45: Type I uncertainties associated with computed lateral stability Factors of Safety at the North Breach for storm surge at 8 feet (Mean Factor of Safety = 1.2, Coefficient of Variation = 25%).

184. The Defense Experts have not addressed the Plaintiffs Experts analyses of the effects of the USACE IHNC Navigation Lock Expansion Project EBIA site clearing excavations, backfilling and other associated activities on the flood protection structures adjacent to the Lower 9th Ward. This is a very curious and important 'omission.' My July 2008 Expert Report, Declarations, and Technical Reports address these aspects extensively. Analyses of the breaches that developed at the Lower 9th Ward during Hurricane Katrina summarized by the Defense Experts have not changed substantially since the initial results of these analyses became available in early 2006 and were published in final form in 2007. However, the Plaintiffs Expert analyses of these breaches have evolved substantially since the early analyses and conclusions were published (May 2006). <u>This evolution has been the result of my search for more observations, information and data, performing and validating</u>

additional analyses of the I-wall failures that occurred during Hurricane Katrina, and reviewing the work done by other investigators. This has been a 'learning process' focused on developing and corroborating understandings of how and why these breaches developed. I am unable to find any evidence of a similar continuing 'search for the truth' in the work documented in the cited Expert Reports. I have diligently attempted to determine 'what is right', not 'who is right.' The Defense Expert Reports are primarily a recitation of the results published as a result of the USACE IPET investigations (IPET 2007).

Why Did the Breaches Develop Where They Did ?

185. One of the questions raised after the initial forensic engineering work had been completed in 2007 (e.g. USACE IPET, ILIT, Team Louisiana, National Institute of Standards and Technology) was: "why did these breaches develop where they did?" Substantial efforts had been devoted by the Defense and Plaintiffs Experts to determining how these breaches developed, but not to why they developed where they did. It was in the course of reviewing photographic evidence (e.g. Figures 45 - 48) gathered shortly after Hurricane Katrina, that the first 'clues' developed to help address the question. Figure 45 shows one such 'clue'. This is a frame from an aerial video survey that was performed on September 14, 2005 (before Hurricane Rita). This video frame clearly shows the trench and holes that were located outside of the concrete I-wall that comprised part of this flood protection structure. Several interesting 'features' that were perpendicular to and under the I-wall also were visible. This led to discovery of additional aerial and ground photographs that had been taken during the early stages of repair of the South Breach and North Breach (Figures 46 - 48). The basic question was: "why were these holes where they were and how did they get there?"

Figure 46: Video frame from aerial survey of Lower 9th Ward on September 14, 2005 – South Breach (USACE Video Survey). Light ground area to left is breach repair.

Figure 47: Aerial photograph showing ‘depressions’ *outside* the Lower 9th Ward floodwall before breach repair operations were initiated (USACE photograph).

Figure 48: South Breach during initial repair period before hole outside of the floodwall is covered (USACE photograph).

Figure 49a: North Breach during initial repair period before hole outside of the floodwall is covered (USACE photograph).


Figure 49b: North Breach during initial repair period before hole outside of the floodwall is covered (USACE photograph).

186. As detailed in my July 2008 Expert Report, Declarations and Technical Reports, detailed studies of aerial and ground photographs taken before Hurricane Katrina clearly indicated that a large number of 'excavations' were made during the EBIA site clearing activities that were located very close to the flood protection structures adjacent to the Lower 9th Ward. These excavations had been filled with soils from the EBIA and in many cases with sand brought to the site to backfill the excavations. Additional evidence regarding these EBIA excavations is presented in Technical Report V that accompanies this Expert Report. One key question about the excavations was related to the very large unbackfilled flooded soil borrow pit located midway between the North and South Breaches. Why didn't the flood protection structure fail at this location? The answer to this question was developed through reviews of the photographs and EBIA site clearing documentation. <u>Because of concerns about seepage</u>, before the borrow pit was allowed to flood with water from the

IHNC, the walls of this excavation were lined with clay to prevent water intrusion into the exposed marsh and swamp layers (Figure 50).

Figure 50: Clay lining placed on walls of borrow pit before flooding with water from IHNC.

187. Based on the available evidence, it was concluded that the deep depressions found in the early days after Hurricane Katrina were 'scour holes' that developed during and after the development of the North Breach and South Breach. The in-flowing and out-flowing surge waters eroded the coarse grained fill that had been used to fill the excavations that were made close to the flood protection structures (refer to Technical Report V for locations of these excavations). The subsequent analyses indicated that the backfilled excavations adjacent to the North and South Breaches provided 'conduits' that facilitated the hydraulic conductivity at these points thereby developing high hydraulic uplift and seepage pressures.

There were both very high potentials for development of ‘blowouts’ near the toes of the levees that comprised the flood protection structures and for high ‘uplift pressures’ to be developed under these levees. All of these hydraulic conductivity effects contributed significantly to the ‘destabilizing’ conditions and forces that led to the initiation and development of the North and South Breaches.

IV. SUMMARY AND CONCLUSIONS

188. There are many fundamental and important points of agreement between the Defense and Plaintiff Experts that background these analyses, conclusions, and expert opinions. There is much more agreement than disagreement. The Defense Experts are well qualified by training, knowledge, and experience to render their observations, analyses, assessments and conclusions. Based on the work documented in their Expert Reports, it is my assessment that the primary differences in the Expert's conclusions and opinions are focused in a few major issues of critical importance. To develop clear understanding of what most likely happened and what most likely should have happened, it is important to strive to "sort the wheat from the chaff."

189. In the context of the man-made flood protection structures existing at the time of Hurricane Katrina, my conclusion is that the fundamental differences between the Defense Experts and Plaintiffs Experts assessments are focused on an understanding of the most probable or likely modes of performance of the man-made flood protection structures during Hurricane Katrina (As Was, Neutral or Ideal Conditions).

190. Relative to the performance of the man-made flood protection structures adjacent to Reach 2 of the MR-GO during Hurricane Katrina, the Plaintiffs Experts have concluded that the major breaching of the man-made earthen flood protection structures was due in large measure to breaching initiated by water side wave erosion which was propagated to the final breach condition by overtopping flows and waves. The breaching that developed at the interfaces of the navigation – water control structures with the adjacent earthen flood protection structures was due primarily to a combination of wave and overtopping surge

erosion (Bayou Dupre north and south interfaces) and lateral instability caused by the surge water pressures developed on and under the structures (Bayou Bienvenue south interface).

191. The Defense Experts have concluded that the major breaching of the man-made earthen flood protection structures and that which developed at the interfaces between these structures and the two navigation – water control structures during Hurricane Katrina was due primarily to surge overtopping and wave erosion.

192. The primary differences between the opinions developed by the Defense Experts and the Plaintiffs Experts concerning development of the breaches that developed along Reach 2 of the MR-GO during Hurricane Katrina are centered in the water side wave erosion initiated development of the breaches in the earthen protection structures and at the interfaces with the navigation – water control structures (Bayou Dupre and Bayou Bienvenue) and the effects of the hydraulic conductivity on the lateral stability of the navigation structure – earthen structure interface at Bayou Bienvenue.

193. Relative to the performance of the man-made flood protection structures adjacent to the portion of Reach 1 of the MR-GO during Hurricane Katrina, the Plaintiffs Experts have concluded that the North Breach and South Breach that developed at this location were due to multiple causes including surge water pressures imposed on and developed under these structures, reduced cross section of the levee at the North Breach, with hydraulic conductivity effects at both breaches exacerbated by the backfilled excavations at the EBIA developed as a result of the IHNC Lock Expansion Project. The North Breach developed before overtopping and the South Breach developed after overtopping.

194. The Defense Experts have concluded that the North Breach developed as a result of surge water pressures imposed on the flood protection structure and the reduced

cross section of the levee at this location before overtopping and that the South Breach developed as a result of surge water pressures imposed on the flood protection structure and overtopping erosion of the supporting soils on the protected side. The North Breach developed before overtopping and the South Breach developed after overtopping.

195. The primary differences between the opinions developed by the Defense Experts and the Plaintiffs Experts concerning development of the breaches at the Lower 9th Ward during Hurricane Katrina are centered in the hydraulic conductivities of the marsh layers that underlie this area, the effects of the hydraulic conductivities on the lateral stability of the structures, consideration of and the effects of the back filled EBIA excavations, and the role of the overtopping erosion of the protected soils in development of the South Breach.

196. Relative to the performance of the man-made flood protection structures adjacent to Reach 2 of the MR-GO during Neutral ("do no harm") Hurricane Katrina conditions, the Plaintiffs Experts have concluded that there would not have been any major breaching of the man-made earthen flood protection structures. The breaching that developed at the interfaces of the navigation – water control structures with the adjacent earthen flood protection structures at Bayou Dupre would not have developed; however the breach at the south end of the Bayou Bienvenue navigation – water control structure would still develop due to lateral instability caused by the surge water pressures developed on and under the structure.

197. The Defense Experts have concluded that the major breaching of the MR-GO Reach 2 man-made earthen flood protection structures and that which developed at the interfaces between these structures and the two navigation – water control structures during

“Ideal MR-GO” Hurricane Katrina conditions would develop at similar times and ways as during the ‘actual’ Hurricane Katrina (as was) conditions.

198. The Defense Experts have concluded that the major breaching of the MR-GO Reach 1 man-made flood protection structures adjacent to the Lower 9th Ward during “Ideal MR-GO” Hurricane Katrina conditions would develop at similar times and ways as during the ‘actual’ Hurricane Katrina (as was) conditions.

199. My understanding as summarized above indicates that there are the following primary points of primary contention between the Defense Experts and the Plaintiffs Experts opinions:

- The roles of wave erosion in development of breaching of the man-made earthen flood protection structures during Hurricane Katrina,
- The roles of hydraulic conductivity in development of breaching of the man-made flood protection structures at the Lower 9th Ward and at the interface of the navigation – water control structure at Bayou Bienvenue (south side) with the adjacent earthen flood protection structure during Hurricane Katrina,
- The roles of the IHNC Lock Expansion Project EBIA backfilled excavations in development of the breaches (North Breach and South Breach) at the Lower 9th Ward during Hurricane Katrina,
- The roles of overtopping erosion of the protected side soils adjacent to the flood protection structures at the Lower 9th Ward during Hurricane Katrina,
- The conditions and characteristics that properly characterize “do no harm” MR-GO Hurricane Katrina conditions along Reach 2 and Reach 1 of the MR-GO, and

- The performance characteristics of the Reach 2 and Reach 1 man-made flood protection structures during "do no harm" MR-GO Hurricane Katrina conditions.

200. The Defense Experts have expressed concerns regarding the analytical models we have used in developing our quantitative assessments of wave erosion of the earthen flood protection structures. In this Declaration and the supporting Technical Reports, I have provided responses to these concerns that utilize multiple ways to validate the analytical models we have used (EBSB Wave Erosion Model). It has been shown that this model possesses both internal and external validity. Similar validation processes have been applied to the other analytical models we have used to evaluate the stability and performance characteristics of the flood protection structures.

201. The Defense Experts have expressed concerns regarding the parameters used in our analytical models of to determine the effects of hydraulic conductivity in development of breaching of the man-made flood protection structures at the Lower 9th Ward and at the interface of the navigation – water control structure at Bayou Bienvenue (south side) with the adjacent earthen flood protection structure during Hurricane Katrina. In this Declaration and the supporting Technical Reports, I have provided responses to these concerns that utilize multiple ways to validate the analytical parameters we have used (seepage and hydraulic uplift effects). It has been shown that these analytical model parameters possess both internal and external validity.

202. The Defense Experts have not addressed the roles of the IHNC Lock Expansion Project EBIA backfilled excavations in development of the breaches (North Breach and South Breach) at the Lower 9th Ward during Hurricane Katrina. In this Declaration I have addressed the importance and effects of these excavations in an attempt to

answer the question: why did these breaches develop where they did and not somewhere else? Thus far in these investigations, detailed analyses of the presence, locations, and effects of the EBIA excavations are the only way that the specific locations of the North Breach and South Breach have been explained. The Defense Experts have not offered any explanation for the development of the breaches at these specific locations.

203. In my previous July 2008 Expert Report, I specifically addressed the roles of overtopping erosion of the protected side soils adjacent to the flood protection structures at the Lower 9th Ward during Hurricane Katrina. These analyses included quantitative analyses of development of the erosion 'trenches' (using three different methods) and analyses of the effects of these erosion trenches on the lateral stability of the flood protection structure associated with the South Breach. It was determined that the erosion trenches could have contributed to and likely participated in the concluding phase of development of the South Breach. These analyses also addressed the performance of other very similar flood protection structures who had experienced the effects of development of very similar erosion trenches – in some cases deeper trenches. These other flood protection structures did not experience lateral stability failure – breaching. My analyses indicate that the major difference between the flood protection structures that did not fail and the structure that did fail and develop the South Breach was the lack of the water pressure hydraulically conductive marsh layers under the flood protection structures. The Defense Experts have not offered any explanation of the contrast between the failed and non-failed flood protection structures.

204. It is evident that the Defense Experts and Plaintiffs Experts have defined the "do no harm" MR-GO Hurricane Katrina conditions and characteristics along Reach 2 and Reach 1 differently (contrasting the Plaintiffs Experts Neutral MR-GO with the Defense

Experts Ideal MRGO conditions). In their definition of the Neutral MR-GO conditions and characteristics, the Plaintiffs have provided mitigations for all of the major deleterious effects that were developed during the life-cycle (design through decommissioning) of the MR-GO. These mitigations have been specifically cited and the reasons for their descriptions documented. As could be expected, the Defense Experts have not done likewise; their MR-GO mitigations have been far more ‘restrictive.’ As a result, an important series of “apples and oranges” analyses and conclusions develop that contrast the studies performed by the Defense Experts and the Plaintiffs Experts. These contrasts can only be resolved by continued deliberations of what properly defines the “do no harm” conditions and characteristics of the MR-GO.

205. The Plaintiffs Experts have addressed comprehensively the performance characteristics of the Reach 2 and Reach 1 man-made flood protection structures during “do no harm” MR-GO Hurricane Katrina conditions. The Plaintiffs Experts have concluded that the Reach 2 flood protection structures that existed at the time of Hurricane Katrina would perform acceptably under the Neutral MR-GO Hurricane Katrina conditions – no major breaches would develop and there would not be any substantial flooding of St Bernard Parish. The Defense Experts have concluded that the Reach 2 flood protection structures that existed at the time of Hurricane Katrina would perform in the Ideal MRGO conditions in a manner similar to that experienced during Hurricane Katrina and that the flooding of St Bernard Parish would be similar to or the same as experienced during Hurricane Katrina. In addition, the two groups of experts have used similar, but in some cases very different means to analyze the performance characteristics of the flood protection structures and develop conclusions and opinions based on the results of their analyses. The differences expert

conclusions and opinions can only be resolved when the differences in ‘inputs’ conditions and characteristics and analytical methods used to evaluate and assess performance characteristics of the flood protection structures – the ‘outputs’ are resolved.

206. My analyses, evaluations, assessments and conclusions have been based on reviews I made of the information provided in the cited Defense Expert Reports and supporting documentation during the period December 29 through January 13, 2009. The 12 working days available for these reviews, development of responses, and the documentation contained in this Declaration have not permitted a complete evaluation of either the contents of the cited Expert Reports nor preparation of a complete response to the contents of these Expert Reports. Consequently, I reserve the right to modify my analyses, evaluations, assessments, and conclusions as more time is provided to develop and document these elements and in the case that new or additional information becomes available in the future.

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 January 29, 2009 in Moraga, California.

Robert Bea, Ph.D, PE

V. REFERENCES

Declaration Text References

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VI. APPENDICES

Appendix A – Summary of Major Conclusions Developed by Experts

Defendant Experts

MR-GO Reach 2 Man-Made Earthen Flood Protection Structures

(1) The major breaching that was developed in the EBSBs during Hurricane Katrina was due primarily to overtopping by Hurricane Katrina's surge and waves.

(2) These conclusions are based primarily on results from analyses of observed data gathered following Hurricane Katrina (e.g. LiDAR surveys, photographs, field inspections), results from numerical analytical simulation models to determine the surge and wave overtopping velocities, and qualitative evaluations of the erodibility and breaching of the EBSBs based on their grass cover and the overtopping surge and wave velocities.

(3) The predicted Neutral MRGO Hurricane Katrina environmental conditions which would act on the EBSBs (waves and surge) are not substantially different than those experienced during Hurricane Katrina.

(4) Major breaching of the EBSBs similar to that developed during Hurricane Katrina would have developed during the Neutral Hurricane Katrina conditions. This breaching would be due primarily to overtopping by the storm's predicted surge and waves.

(5) These conclusions are based on results developed during the IPET project and on results from subsequent studies documented in the cited Defense Expert Reports.

(6) The Defense Experts conclude that the EBSBs existing at the time of Hurricane Katrina were 'proper' man-made hurricane flood protection structures. The Defense Experts conclude that the design, construction, operation, and maintenance of these man-made hurricane flood protection structures were within the appropriate authorizations and the applicable engineering standard of care.

MR-GO Reach 1 Lower 9th Ward flood protection structures

(7) The North Breach developed before overtopping due to Hurricane Katrina's surge water pressures and the reduced cross section – lateral resistance of the levee supporting the floodwall and including sheet piling.

(8) The South Breach developed after overtopping due to Hurricane Katrina's surge water pressures and reduction in the lateral resistance of the floodwall and sheet piling due to erosion of the soils by the overtopping water acting on the protected side of the supporting levee.

(9) These conclusions are primarily based on analyses of observed data (e.g. eyewitness reports, after failure photographs and inspections) and results from advanced numerical analytical models. These conclusions are based on results developed by the USACE sponsored IPET project and on results from subsequent studies performed by the Defense Experts.

(10) Analyses of the Hurricane Katrina environmental conditions indicate that the effects of a Neutral MR-GO are not substantially different than those associated with the As Was MR-GO conditions.

(11) The conclusions of this work indicate the man-made flood protection structures would have failed at the same times and in the same ways during the Neutral MR-GO Hurricane Katrina conditions.

(12) It is concluded that the flood protection structures that existed at the time of Hurricane Katrina were ‘proper’ man-made hurricane flood protection structures. The design, construction, operation, and maintenance of these man-made flood protection structures were within the appropriate authorizations and the applicable engineering standard of care.

Plaintiffs Experts

MR-GO Reach 2 Man-Made Earthen Flood Protection Structures

(1) The major breaching developed in the EBSBs during Hurricane Katrina was due primarily to water side wave erosion that developed through the crown or crest of the EBSBs followed and propagated by overtopping surge and wave erosion of the protected side.

(2) These conclusions are based primarily on analyses of observed data (LiDAR surveys, photographic evidence after Hurricane Katrina and before Hurricane Rita, field inspections) and results from numerical analytical model simulations of both front-side wave induced erosion and backside overtopping erosion by surge and waves. These quantitative analyses addressed the local effects of natural vegetation. These conclusions also are based on results from the USACE sponsored IPET project, from the NSF sponsored ILIT project, and from the Team Louisiana project.

(3) The predicted Neutral MR-GO Hurricane Katrina environmental conditions which would act on the EBSBs are substantially different than those experienced during

Hurricane Katrina. The primary differences involve the characteristics of the waves acting on the faces of the earthen structures, the effects of protective vegetation, and the intensities of surge and wave overtopping.

(4) The EBSBs would not have breached during the Neutral MR-GO Hurricane Katrina conditions.

(5) These conclusions are based primarily on analyses of observed data (LiDAR surveys, photographic evidence after Hurricane Katrina and before Hurricane Rita, field inspections) and results from numerical analytical model simulations of both front-side wave induced erosion and backside overtopping erosion by surge and waves. These conclusions also are based on results from the USACE sponsored IPET project, from the NSF sponsored ILIT project, and from the Team Louisiana project.

(6) It is concluded that the man-made earthen hurricane flood protection structures that existed at the time of Hurricane Katrina were those that existed at that time; these earthen flood protection structures represented a history of trade-offs and decisions that resulted in significant compromises of acceptable integrity, resilience, and reliability of these structures. There is not agreement that these man-made earthen hurricane flood protection structures were' proper' because of deficiencies and defects introduced during the design, construction, operation, and maintenance of the MR-GO. There is not agreement that the appropriate engineering standard of care was met for these earthen hurricane flood protection structures.

MR-GO Reach 1 Lower 9th Ward flood protection structures

(7) The North Breach developed before overtopping due to Hurricane Katrina's surge water pressures, the reduced cross section – lateral resistance of the levee supporting

the floodwall and sheet piling, and hydraulic seepage and pressure effects developed in the foundation soils. These effects were exacerbated by adjacent backfilled excavations developed during the USACE EBIA Navigation Lock Expansion Project.

(8) The South Breach developed after overtopping due to Hurricane Katrina's surge external water pressures and hydraulic seepage and uplift pressure effects developed in the foundation soils. These effects were exacerbated by adjacent backfilled excavations developed during the USACE EBIA Navigation Lock Expansion Project.

(9) The Plaintiffs Experts justify these conclusions primarily based on their analyses of 'observed data' (e.g. eyewitness reports, after failure photographs and videos, LiDAR surveys, contractor experience during excavations in nearby areas), and results from quantitative analytical models. These conclusions are based also on analyses of adjacent sections of the man-made flood protection structures and other similar parts of the flood protection structures that did not breach under the same or very similar environmental conditions. Further, these conclusions are based on results developed by the NSF sponsored ILIT project, the USACE sponsored IPET project, the Team Louisiana project, the NIST project, studies performed for the Ingram Barge PSLC litigation, and on results from subsequent studies performed by the Defense and Platintiff Experts.

(10) The Plaintffs Experts conclude that the effects of a 'Neutral MRGO' at this location are less severe than those associated with the 'As Was MRGO' during Hurricane Katrina (reduced surge elevations and durations). The Plaintiffs Experts conclude that the man-made flood protection structures would have failed in similar ways, times, and locations for both sets of Hurricane Katrina conditions.

(11) The Plaintiffs Experts conclude that the man-made hurricane flood protection structures that existed at the time of Hurricane Katrina were those that existed at that time; these flood protection structures represented a history of trade-offs and decisions that resulted in significant compromises in desirable integrity, resilience and reliability of these flood protection structures. These experts do not agree that these man-made hurricane flood protection structures were ‘proper’ and would meet the desirable engineering standard of care because of major deficiencies and defects introduced during the design, construction, operation, and maintenance of the flood protection structures including flaws and defects introduced into the flood protection structures during the USACE Navigation Lock Expansion Project EBIA site clearing activities.

Appendix B – Review of Expert Reports

The Defense Expert Reports reviewed in this Appendix are those written by Mr. Bruce Ebersole, Dr. Reed Mosher and Dr. Don Resio (2008). These review observations are specifically oriented towards our forensic engineering analyses of the wave-induced erosion of the MR-GO Reach 2 man-made earthen flood protection structures (EBSBs, Levees) documented in my July 2008 Expert Report, Declarations, and Technical Reports. Because of the severe limitations in time provided for these reviews and this documentation, the reviews have not been complete or exhaustive. In addition, this Appendix does not address review observations that pertain to development of the breaches at the portion of the MR-GO Reach 1 adjacent to the Lower 9th Ward.

A general review observation is that there is substantial agreement with the results from our analyses documented in the subject Defense Expert Reports. In many instances, this agreement is difficult to 'understand' because the Defense and Plaintiffs Experts 'talk past each other.' For example, Ebersole states that the overtopping velocity (15-25 feet per second) is greater than the wave-induced velocity (5-15 feet per second) on the MR-GO Levee and as a result, overtopping is a more critical breaching mechanism. While true, Ebersole is in general agreement with the Plaintiffs that the wave velocity values are on the order of 5-15 feet per second. He does not, however, directly analyze the potential for erosion for this velocity range. Further, there is no direct analysis of the erosion of the EBSBs based on the characteristics of the soils that comprise these earthen flood protection structures.

Another common issue was that the Defense Experts would address only some of the identified evaluation parameters used by the Plaintiffs Experts in developing their analyses and conclusions. For example, Dr. Resio only evaluates hydrodynamics (storm and waves) and Levee freeboard, separate from the surface cover and materials which comprise these earthen flood protection structures. However, both Mr. Ebersole and Dr. Mosher conclude these other parameters are important and require evaluation and analysis. Thus, evaluating all potentially important parameters should be evaluated in an integrated and coherent manner before conclusions are developed.

The expert reports also highlight the very complex nature of the man-made hurricane flood protection system, the many 'state of the art' analytic aspects that are required in order to address this complex engineered system and the environmental system in which it is embedded. The very large spatial extent of these engineered and environmental systems requires identification and management of a large number of uncertainties, heterogeneities, and places strains on analytical and computing resources to account for the multitude of complex inter-related and interactive factors. These challenges apply equally to the Defense and Plaintiffs Expert analyses.

Both the Defense and Plaintiffs Experts have presented their results specific to their areas of expertise. However, the aggregate 'picture' (understanding, insight) developed by combining the individual areas of technical expertise should generate a 'story' that explains the observed sequence of events, such as the observed flood volumes in St. Bernard Parish during Hurricane Katrina; the observed breaching mechanisms (overtopping-induced breaching, wave and/or overtopping-induced breaching, wave-induced erosion, and no-erosion/breaching); and the entry of large barges into and on top of the MR-GO EBSBs.

Finally, individual elements integrated into the forensic engineering analyses must be consistent (if overtopping is identified as a breaching mechanism, the erosion rate of the levee material at that location must yield the observed erosion for the identified overtopping duration). In my opinion, to this time, neither the Defense Experts nor the Plaintiffs Experts have done an adequate job of integrating these disparate aspects of the actual 'story' that developed during Hurricane Katrina.

Finally, both the Defense and Plaintiffs Experts agree full-scale prototype testing is the preferred means by which to validate analytic results. Unfortunately, this is not physically possible as the storm has already occurred and the damage done. Dr. Wolff best highlights the need for validated and calibrated models. This is a very important point and applies to both the Defense and Plaintiffs Experts analytical models and assessment – evaluation processes. In order to properly validate and calibrate the forensic engineering analytical models, actual field data is required. Unfortunately, very few instruments were installed in the greater New Orleans area, many of these instruments were limited in types of information collected (i.e. storm surge data may have been collected, but no information on wind wave heights, periods, and directions), and of the installed instruments, many of them did not capture data throughout the entire storm, resulting in an incomplete and insufficient 'data record' by which to reliably calibrate/validate any numeric models. Active attempts were made by the Plaintiffs Experts to validate the methods and models developed for wave-induced erosion analyses by applying to subsequent hurricanes in the MRGO Reach 2 area (Hurricane Gustav). It is not evident that the Defense Experts did the same and validated their models in the same fashion.

Expert Report by Mr. Bruce Ebersole

In his Expert Report, Mr. Ebersole addresses the areas of storm surge, wind waves, surf zone dynamics and implications for levee erosion, levee response (erodibility), erosion at floodwalls, and provides comments on the influence of the MRGO on critical levee and wall breaches. This Expert Report is in substantial agreement with the analyses completed by the Plaintiffs Experts. Ebersole confirms /agrees with the following:

- Some storm surge reduction is afforded with the presence of natural wetland (pre-MRGO);
- There were wave-induced uprush/downrush velocities on the MRGO levee face;
- Wave action was more pronounced with the storm surge level above El. +13 ft (NAVD88);
- Erodibility of levees is complex, with multiple factors requiring consideration to characterize erosion resistance or levee erosion "capacity";
- Construction method and compaction have a major impact on erodibility of levee materials; and
- Overtopping-induced erosion was more prevalent than wave and/or overtopping & wave-induced erosion, however these (wave and/or overtopping & wave-induced erosion) were still possible.

The primary criticism of the Plaintiff's work was the erodibilty assigned to the Levee – EBSB materials. Other, minor, criticisms are addressed in the 'notes' inserted into the

Ebersole expert report (PDF). There are three main response points relative to the erodibility comments highlighted by Ebersole:

- There was no direct analysis of wave-induced or overtopping-induced erosion in areas of catastrophic erosion. This is an omission as it does not confirm the range of erodibility rates estimated by Ebersole;

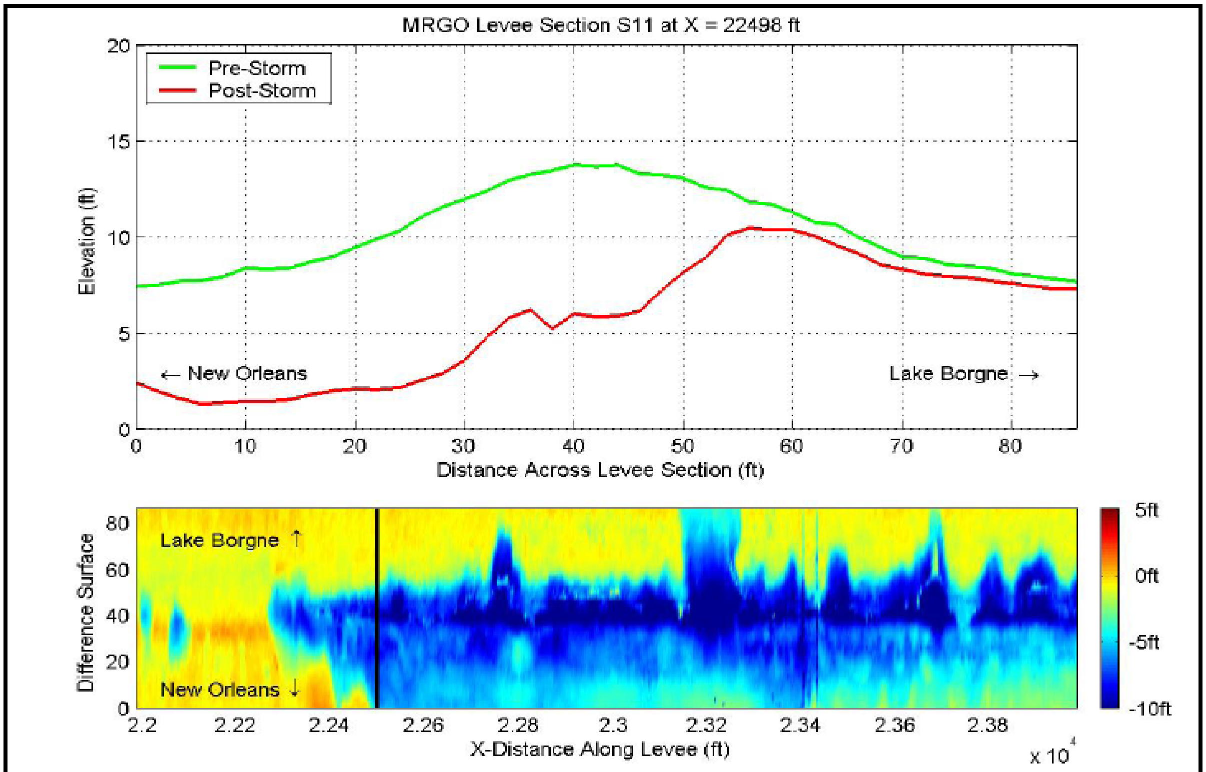
- Ebersole identifies a number of factors that are essential to be considered when analyzing erosion, however, his analyses do not account for these identified factors; and

- The eroded sediment volume evaluation did not account for the transport of sediment from the 'flood side' of the levee to the 'protected side' of the levee following breach of the EBSB crest and inrush of water during breaching. Furthermore, the analyses do not confirm that even for overtopping-induced erosion, the deposited sediment on the protected side matches the volume of eroded levee material.

These points are discussed in more detail below:

(1) There was no direct analysis of wave-induced or overtopping-induced erosion in areas of catastrophic erosion. This is an important omission as it does not confirm the range of erodibility rates estimated by Mr. Ebersole.

Mr. Ebersole presents a discussion of erodibility and states that "non-cohesive silts and sands would be expected to readily erode when subjected to … velocities of 6 to 8 feet per second; whereas denser clayey type sediments might not be eroded at all.." We generally agree with this, but there were VERY FEW locations on Reach 2 with DENSE clay…all indication are that the material was marginally dense to soft. Ebersole does not provide an estimate of critical velocity thresholds for these materials (which is the major soil type along Reach 2). This is an important omission. Furthermore, there is no analytic validation that Ebersole's erosion threshold velocities explain the observed events along MRGO Reach 2. Mr. Ebersole draws a cross section of erosion shape at MRGO Levee Section S11 (Figure B-2), however, the erosion rate and erodibility of the levee materials to result in this observed shape were not analyzed. This is an important omission.



Levee cross-sections at Location 4.

Figure B-2: Erosion shape was evaluated, but not the required erodibility characteristics of the levee material to result in this shape (Ebersole page 248/307).

(2) Mr. Ebersole identifies a number of factors that are essential to be considered when analyzing erosion, however, his analyses do not account for these identified factors.

Table B-1 presents a summary of factors (cited page numbers are shown in "( )" of Mr. Ebersole's Expert Report) identified by Mr. Ebersole as important parameters to be addressed when evaluating erosion / erodibility of earthen flood protection structures. These parameters were used to critique the Plaintiff's analyses. However, these factors were not addressed in Mr. Ebersole's Freeboard / Sediment Type analysis (pages 56/307 through 64/307). These parameters are substantially addressed in the Plaintiffs Expert analyses.

Table B-1. Summary of Ebersole Erodibility Parameters and Application in Mr. Ebersole's Freeboard/Sediment Analysis Method

	Ebersole Erodibility Evaluation Parameter Analysis	**Freeboard/Sediment Type Method**
Demands	Water depth/surge (45/307)	Addressed
	Incident wave energy (45/307)	Not Addressed
	Sea state (irregular vs. regular) (45/307)	Not Addressed
	Freeboard/crest elevation (54/307)	Addressed
	Shear velocity (55/307)	Not Addressed
Capacity	Soil type (56/307)	Addressed
	Soil density (compaction) (56/307)	Not Addressed
	Construction method (78/307)	Not Addressed
	Grass cover armoring (80/307)	Not Addressed
	Spatial gradients in sediment transport rates (66/307)	Not Addressed
	Difference between what is being transported into and out of sedimetn control volume through sheet flow, bed load, suspended load (66/307)	Not Addressed
	Rate at which sediment is being eroded and entrained locally (67/307)	Not Addressed
	Rate of sediment deposition (67/307)	Not Addressed
	Armoring due to local sorting and sediment fluxes (67/307)	Not Addressed

Mr. Ebersole does not address the important evaluation parameters he has identified in any coherent or integrated manner. These are important omissions.

(3) The eroded sediment volume evaluation did not account for the transport of sediment from the 'flood side' of the levee to the 'protected side' of the levee following breach of the levee crest and inrush of water during breaching. Furthermore, the analyses do not confirm that even for overtopping-induced erosion, the deposited sediment on the protected side matches the volume of eroded levee material.

Mr. Ebersole performs an analysis that evaluates the quantity of levee sediment that is deposited on the flood side of the levee (see Ebersole page 110/307). This analysis did not find that eroded sediment was deposited on the flood side.

More importantly, the study did not verify that locations of known overtopping-induced erosion deposited a volume of sediment equal to the eroded volume on the protected side of the levee. The study also did not address the lack of sediment 'spoil piles' on the protected side of the levee at many locations, such as the location shown in Figure B-3. These clayey sediments are readily carried by water and transported long distances by rushing moving water, thus 'leaving the scene of the crime.'

Furthermore, the analysis did not account for sediment over-wash, as shown in Figure B-3 (Figure 69 from Ebersole's report). In this instance, the erosion was the result of wave action, but the sediment was deposited on the protected side of the levee…implying (according to the sediment volume analysis logic) overtopping erosion.

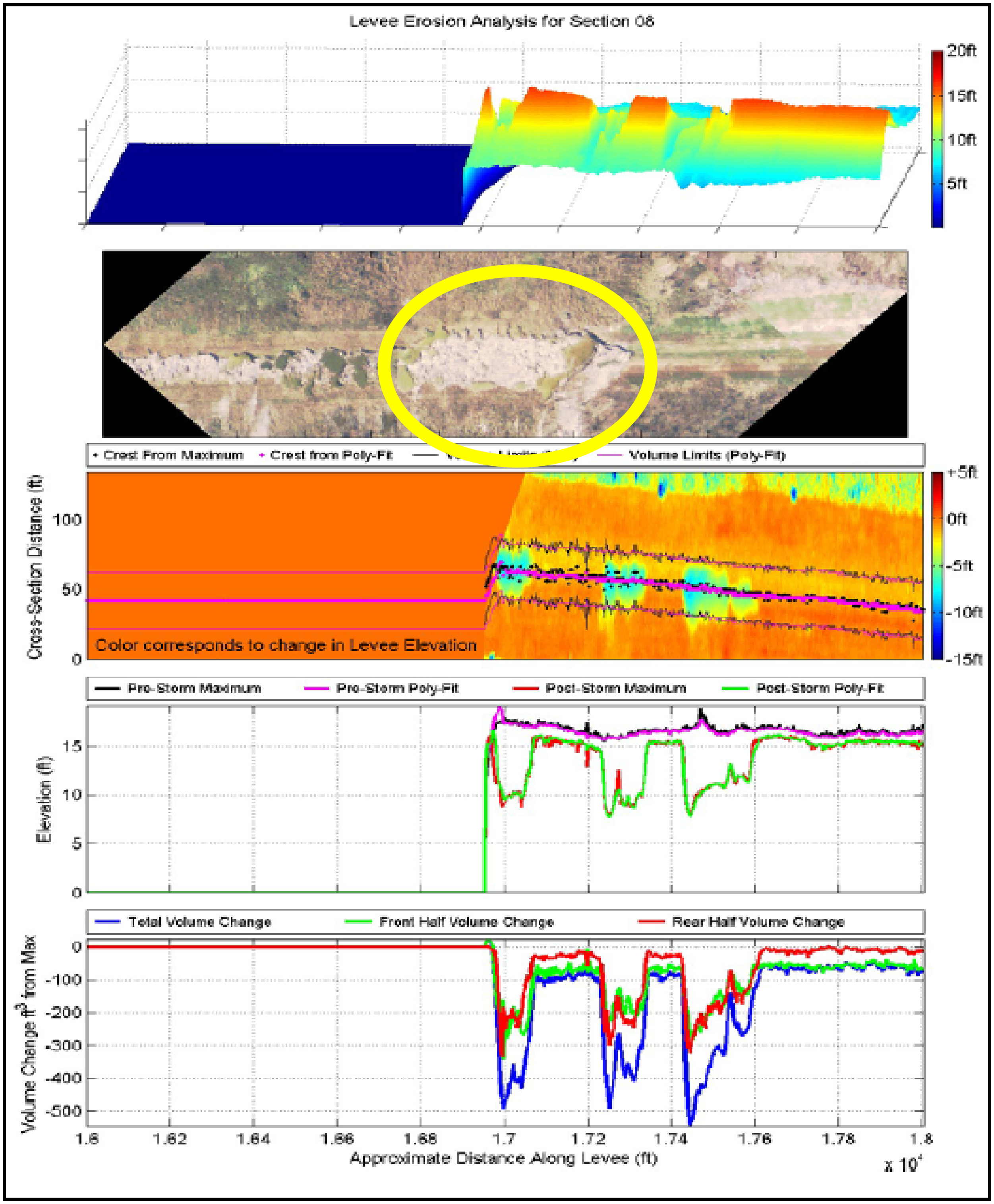


Figure B-3. Volume analyses at this location do not confirm overtopping OR wave induced erosion. The value of this analysis is unclear and ambiguous (Ebersole page 271/307).

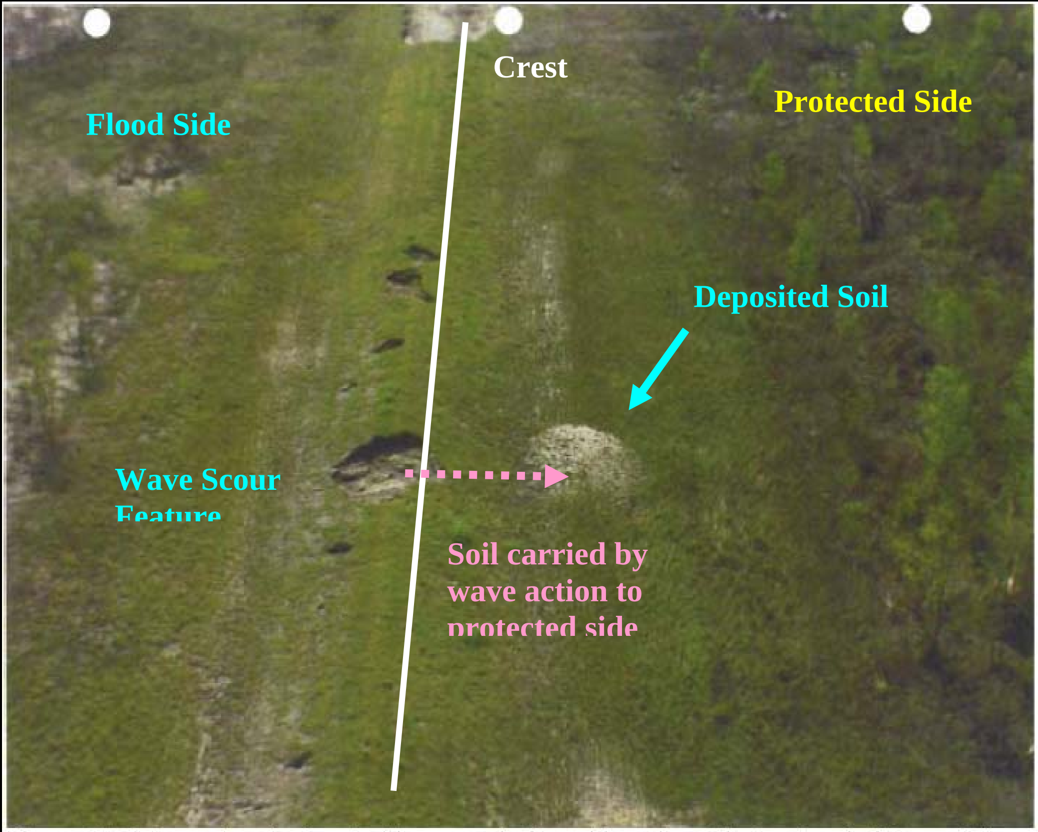


Figure 69. Example of a front side wave-induced head-cut that extends into and through the levee crest. Cut lead to minor overtopping and overwash of sediment.

Figure B-4. Here wave-induced erosion results in soil deposits being made on the protected side of the levee, counter to the imposed logic that wave-induced erosion must result in sediment deposition on the flood side of the levee (i.e. same side of the levee crest as the erosion feature).

In their December 2008 Expert Reports, Mr. Ebersole, Dr. Resio, and Dr. Mosher make the point that there was no evidence of substantial sections of the EBSBs being breached by water side wave induced erosion. Figures B-5 and B-6 show an aerial photograph of a section of the Reach 2 EBSB south of Bayou Bienvenue together with the analysis of what is evidenced in this photograph by Mr. Ebersole and Dr. Mosher (December 2008 Expert Reports.).

Figure A4. Severely degraded levee. Evidence of overtopping indicated by overwash sediment deposits on back side. Channel present just to south (top edge) of the barge. Backside elongated sediment deposit suggests high overflow. Traces of deposited sediment at front of the channel mouth also suggest outflowing water. Channel likley fomed on outflow. Lowest spots on the degraded levee likely served as outflow channels for water exiting the polder following storm's passage. Situation in lower half of photo is less clear.

Figure B-5. Analysis of EBSB breaching mechanics by Ebersole (December 2008).

Figure A4. Severely degraded levee. Evidence of overtopping indicated by overwash sediment deposits on back side. Channel present just to south (top edge) of the barge. Backside elongated sediment deposit suggests high overflow. Traces of deposited sediment at front of the channel mouth also suggest outflowing water. Channel likley fomed on outflow. Lowest spots on the degraded levee likely served as outflow channels for water exiting the polder following storm's passage. Situation in lower half of photo is less clear. Overwashed sediment evident, erosion pattern at bottom could be due to entering or exiting flow. Predominance of sediment deposits on the back side in the upper part of the photo suggests degradation caused by overtopping/head-cutting.

Figure B-6. Analysis of EBSB breaching mechanics by Mosher (December 2008.

I found their analyses of this photographic evidence to be very elucidating. The primary conclusion was that this breach developed due to overtopping (a theme developed by the USACE the day following Hurricane Katrina; Times-Picayune statements by Col. Waggenar). Early in my analyses of the available photographic evidence of the EBSB breaching in this vicinity, the presence of the cargo barge wedged on the water side of the EBSB drew my attention. This section and barge are located near the south terminus of the sheet pile repair section immediately south of Bayou Bienvenue (Figure B-7). A similar unloaded cargo barge was found following Hurricane Katrina impaled on the top of the EBSB immediately north of the Bayou Bienvenue navigation – water control structure (Figure B-7). South of the cargo barge on the EBSB south of Bayou Bienvenue, a large number of similar unloaded cargo barges were found several thousand feet away from the EBSBs on the protected side of the EBSBs – evidently they had been swept over the crests of the EBSBs during Hurricane Katrina (Figure B-8).

The post-Hurricane Katrina LiDAR surveys indicated the cargo barge had grounded on the EBSB crest which had been eroded to an elevation of between +11 and +12 feet (NAVD88) (Figure B-9). Photographs of the grounded cargo barge showed that it was sitting on top of the eroded breached section of the EBSB (Figures B-10 and B-11). It was important to obtain photographic evidence developed soon after Hurricane Katrina and before Hurricane Rita. This is because Hurricane Rita had important effects on the ‘features’ developed on the EBSBs during Hurricane Katrina. Also, it was very important to recognize that for the breaches with lower bottom elevations, that the features had been affected by the out-flowing surge waters that followed the peak surge period and the passage of Hurricane Katrina. In some cases, subsequent high storm and astronomical tidal flows had important effects on the erosion features.

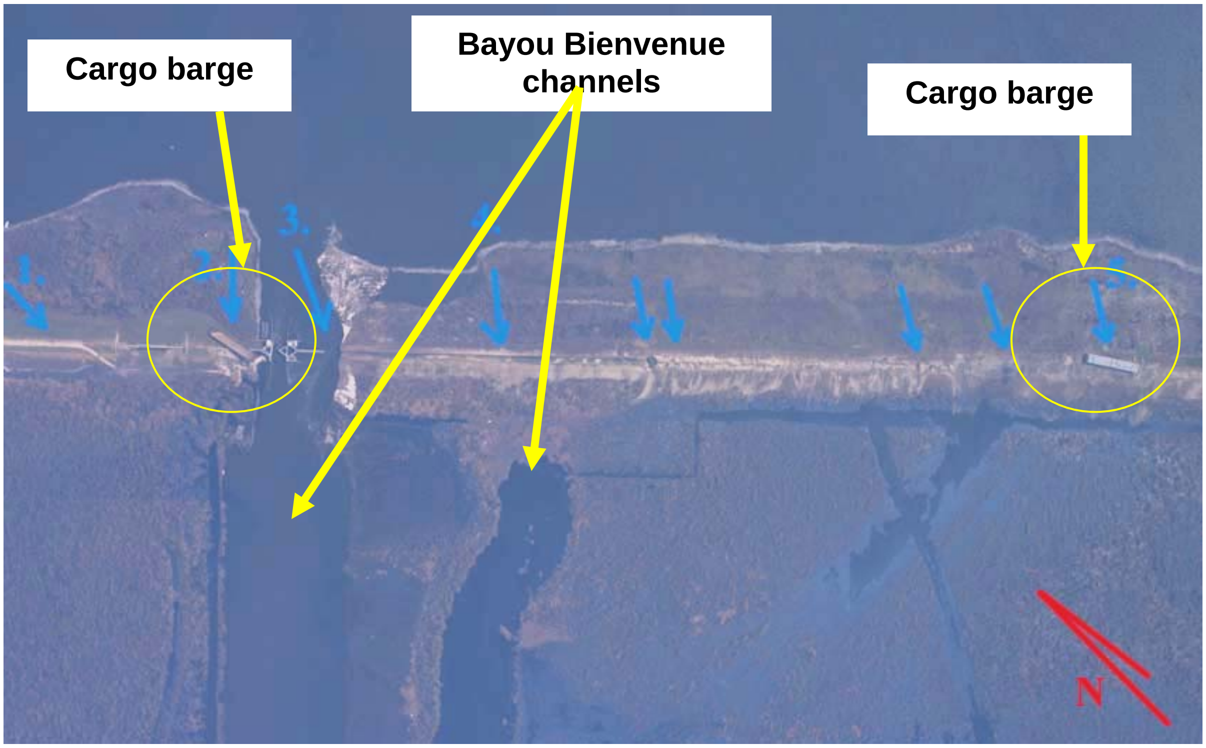


Figure B-7. Cargo Barges found impaled on crests of EBSBs in vicinity of Bayou Bienvenue following Hurricane Katrina (NOAA photograph 2005).

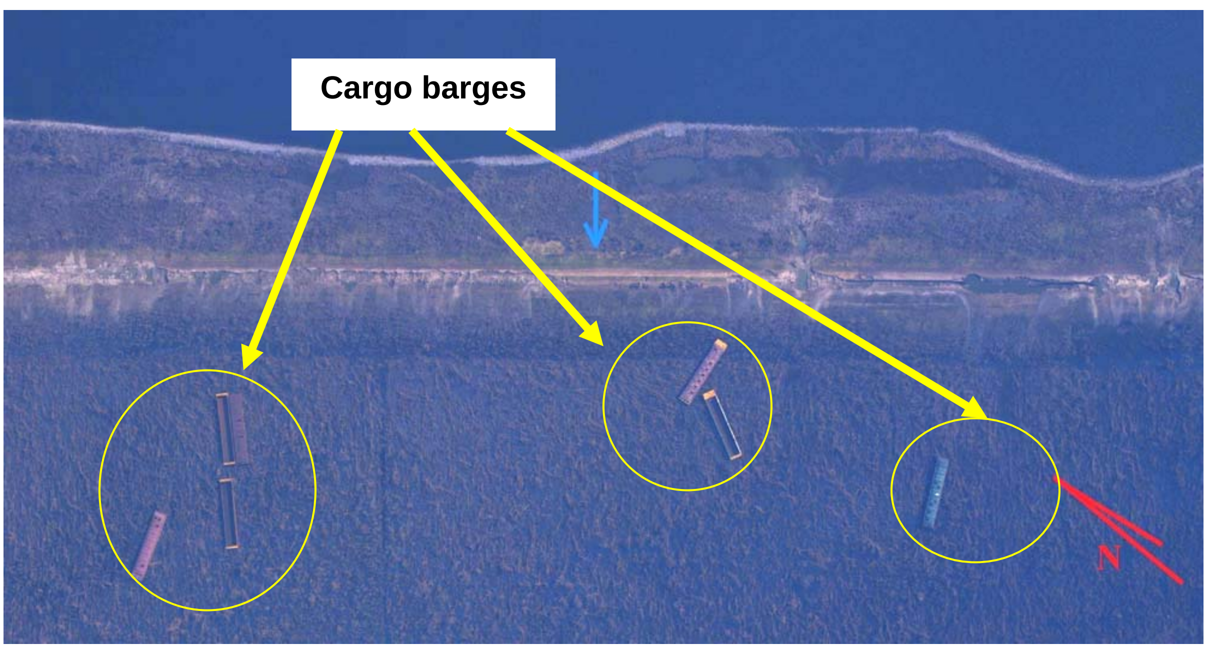


Figure B-8. Barges swept over crest of EBSBs during Hurricane Katrina (NOAA photograph 2005).

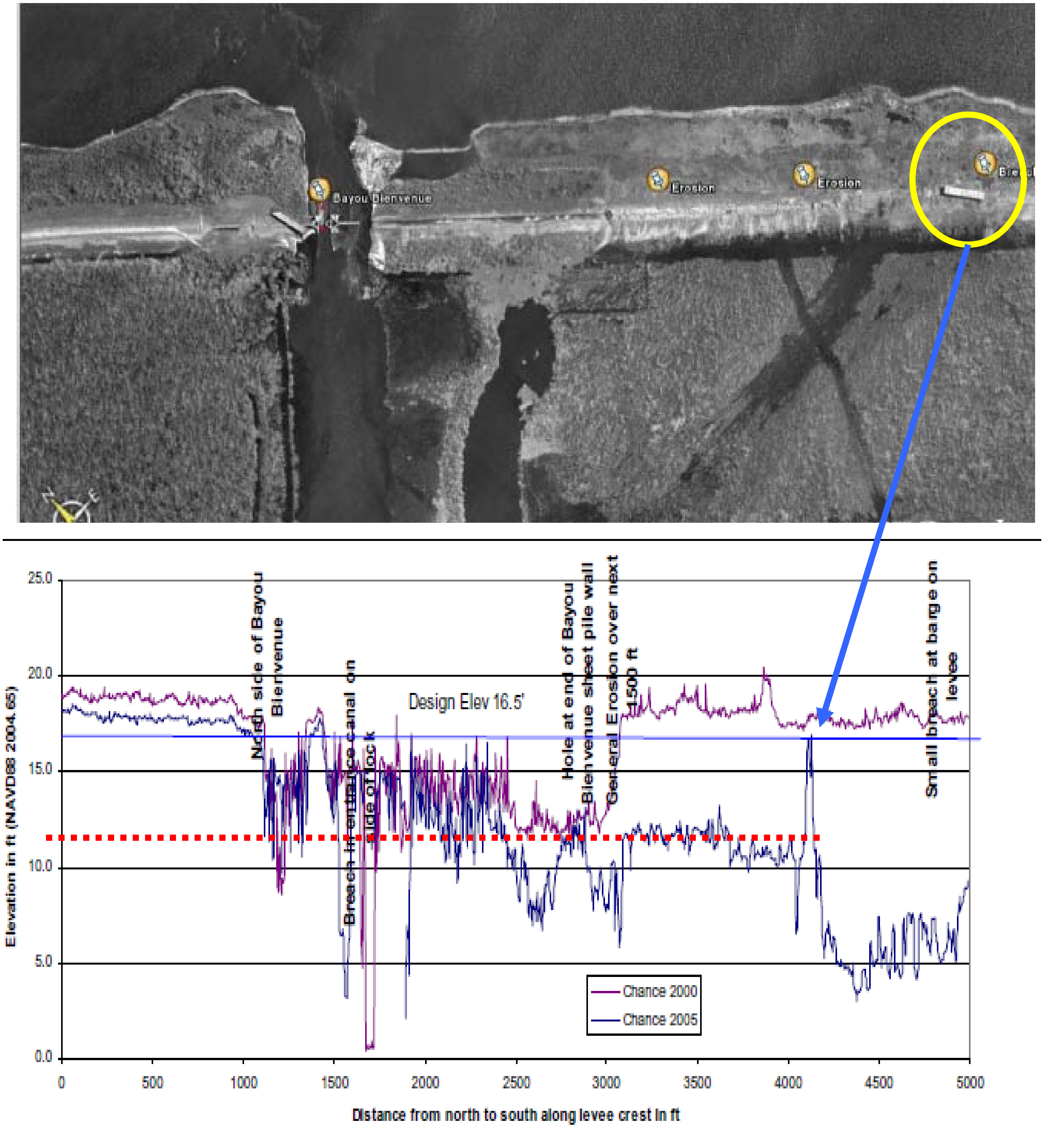


Figure B-9. Cargo barge position relative to crest elevations of EBSBs (from USACE IPET 2007).

Figure B-10. Video frame from USACE aerial survey (September 14, 2005).

Figure B-11.Video frame from USACE aerial survey (September 14, 2005).

The erosion features at the ends of the cargo barge indicate both surge in-flow and out-flow scour. Clearly, the barge was grounded before the peak surge arrived at this location. This observation is corroborated given that the unloaded cargo barge had a draft of approximately 2 to 3 feet and the elevation of the EBSB at the grounded location was approximately 12 feet, the water depth at the time of grounding would have been approximately 14 feet to 15 feet. Thus, there would be a water depth of about 3 feet that would flow around the barge ends as the surge continued to rise; thus developing the landward erosion features. The unique barge-end scour features were not present away from the barge ends. The barge is sitting on top of these erosion features. These features clearly show both wave and overtopping flow characteristics that had to be developed <u>before</u> the grounding of the barge. Thus, it would not be reasonable to conclude that these features were developed <u>after</u> surge overtopping.

These deductions were corroborated by observations associated with the unloaded cargo barges that were swept over the tops of the sheet pile repaired sections of the EBSBs located south of Bayou Bienvenue (Figure B-12). The unloaded cargo barges had a draft of approximately 2 to 3 feet. The tops of the sheet piling in this location were located at an elevation of approximately + 13 to +14 feet (NAVD88) (Figure B-13_. With the peak surge in this vicinity during Hurricane Katrina in the range of +17 to +18 feet (NAVD88), the barges could be easily swept over the tops of the sheet piling without grounding.

Figure B-12. Aerial photograph of grounded cargo barges on protected side of Reach 2 EBSBs (Morris 2008).

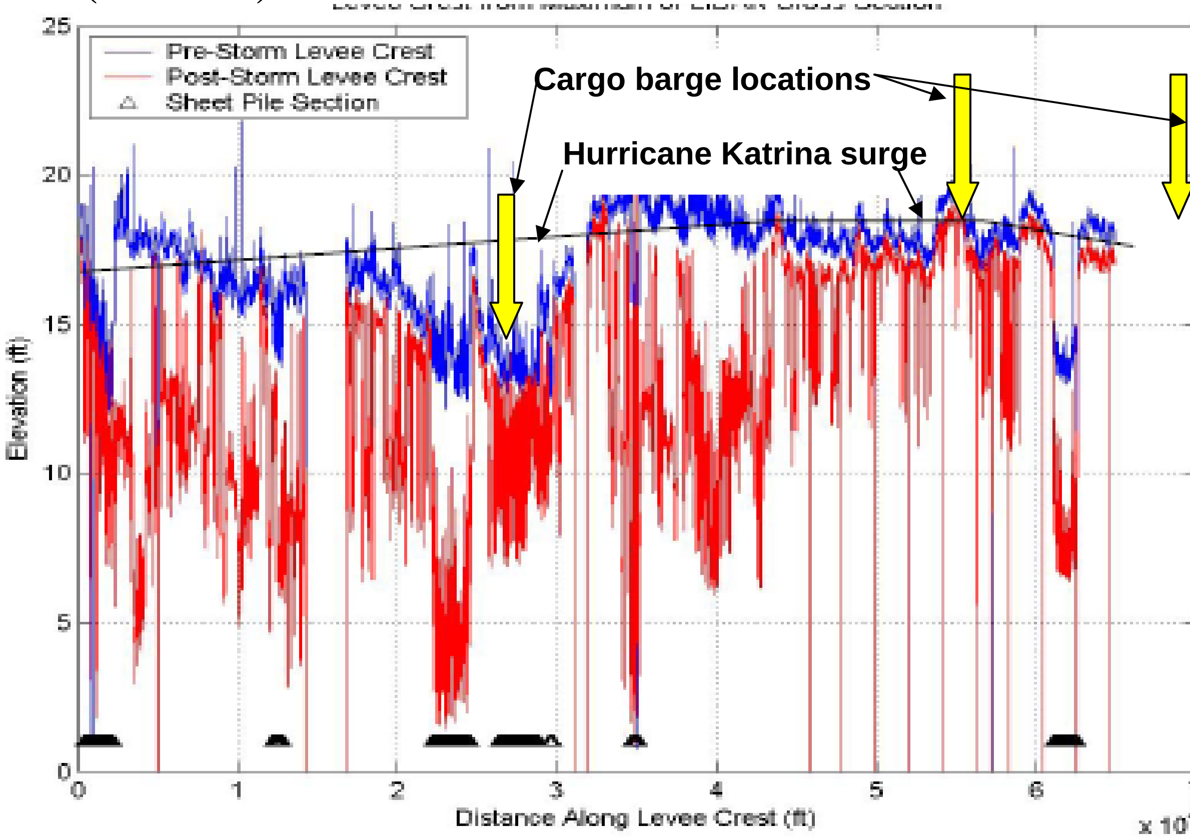


Figure B-13. Pre- and post-Hurricane Katrina LiDAR survey EBSB crest elevations, Hurricane Katrina peak surge elevations, and barge locations (from Resio December 2008 Expert Report).

This and other photographic evidence (e.g. Figures B-14 - B-17) clearly indicate the presence of water side wave erosion initiated breaching that was subsequently exploited by the rising surge waters. This evidence is not concordant with the observations developed by Mr. Ebersole and Dr. Mosher as documented in their December 2008 Expert Reports. Evidence clearly shows remnant features of extensive sections of the Reach 2 EBSBs that indicate water side wave induced erosion breaching exploited by the rising surge waters to develop the major breaches that were developed during the passage of Hurricane Katrina.

Figure B-14. High resolution pictograph of Reach 2 EBSBs showing wave erosion features (photograph provided by Dr. Paul Kemp).

Figure B-15. High resolution pictograph of Reach 2 EBSBs showing wave erosion features (photograph provided by Dr. Paul Kemp).

Figure B-16. High resolution pictograph of Reach 2 EBSBs showing wave erosion features (photograph provided by Dr. Paul Kemp).

Figure B-17a. High resolution pictograph of Reach 2 EBSBs showing wave erosion features (photograph provided by Dr. Paul Kemp).

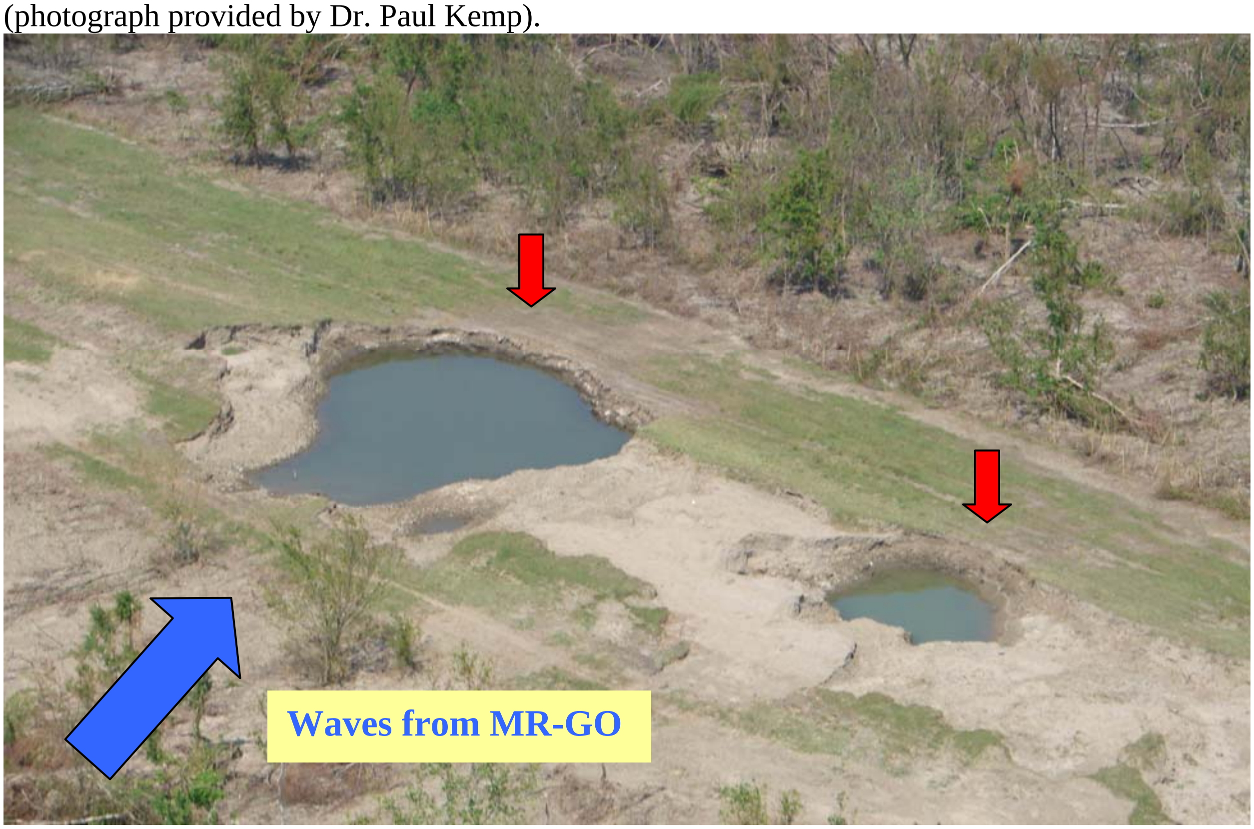


Figure B-17b. Aerial photograph of Reach 2 EBSB showing wave erosion features (photograph provided by Mr. Less Harder).

Expert Report by Dr. Reed Mosher

Dr. Mosher addresses the areas of levee failure patterns, soils and erodibility, post-Katrina erosion observations, and IHNC breach assessments. This expert report is in substantial agreement with the analyses completed by the Plaintiffs Experts. Dr. Mosher's analyses and observations agrees with the following:

- Overtopping was a more common breach mechanism than wave and/or overtopping-induced erosion;
- The hydraulic fill materials used were of poor quality (also implies support of higher erodibility characteristics than 'dense clay');
- Reduced crest elevations contributed to early breaching of the MRGO levee;
- Erodibility of levees is complex, with <u>multiple factors</u> requiring consideration to characterize erosion resistance or levee erosion "capacity"; and
- Construction method and compaction have a major impact on erodibility of levee materials.

The primary criticism of the Plaintiff Experts forensic engineering analyses was there was no photographic evidence to support wave and / or overtopping induced breaching. This is not a valid criticism. This criticism has been addressed earlier in this Appendix and further addressed in the body of this Declaration.

Dr. Mosher outlines parameters for the evaluation of erodibility, however he provides no analysis to evaluate the performance of the MR-GO EBSBs relative to his identified parameters. Without these analyses, what is the basis of his scientific and / or forensic engineering conclusions? Table B-2 presents a summary of Dr. Mosher's criteria to evaluate erodibility. These parameters were not analyzed nor evaluated by Dr. Mosher. This is an important omission.

Table B-2. Summary of Dr. Mosher's Erodibility Evaluation Parameters

	Mosher Erodibility Evaluation Parameter	Mosher Analyses
Demands	Storm surge and waves (pg 5)	Not Analyzed
	Turbulence (pg 30)	Not Analyzed
	Pore water chemistry (pg 30)	Not Analyzed
	Hydraulic stress (pg 30)	Not Analyzed
Capacity	Soil type (pg 26)	Not Analyzed
	Soil density (compaction) (pg 30)	Not Analyzed
	Construction method (pg 25)	Not Analyzed
	Moisture Content (pg 30)	Not Analyzed
	Soil structure (pg 30)	Not Analyzed
	Organic content (pg 30)	Not Analyzed
	In-situ shear strength (pg 30)	Not Analyzed
	Surface roughness (pg 31)	Not Analyzed
	Leeve geometry (pg 31)	Not Analyzed

Expert Report by Dr. Donald Resio

Dr. Resio addresses the areas of wave modeling, EBSB (Levee) failure and breach development patterns, soils and erodibility, post-Katrina erosion observations, and IHNC breach assessments. Dr. Resio's analyses and conclusions are in substantial agreement with a number of the analyses and conclusions reached by the Plaintiffs Experts Dr. Resio confirms /agrees that wave overtopping is strongly affected by levee freeboard.

Dr. Resio's primary criticisms of the Plaintiff's work are:

- Simulation of wave effects for only a single levee elevation (17.5 ft) and a single time series of waves and surges;
- Lack of demonstrated applicability of the LS-DYNA model for velocities; and

- Incorrect application of the LS-DYNA code for predicting run-up and down-rush velocities on the water side faces of the EBSBs / Levees .

These points are discussed in more detail in the following sections.

1. *Simulation of wave effects for only a single levee elevation (17.5 ft) and a single time series of waves and surges*

It is not clear that Dr. Resio understood that the wave-induced erosion analyses were specific to a particular location (the "EBSB Study Location"). The results from the wave-induced erosion analyses were used as a basis by which to evaluate the likelihood of breaching at the EBSB Study Location due to wave and overtopping. Based on information, data, and observations from a variety of sources (photographs before and after Katrina, video and LiDAR surveys of the EBSBs after Hurricane Katrina and before Hurricane Rita, multiple field ground, water, and air inspections performed following Hurricane Katrina, review of available data on EBSB soil characteristics), these results were then extrapolated to other Reach 2 locations. It was not the intent of the wave erosion – overtopping breaching analyses performed at the EBSB Study Location as documented in the Plaintiffs Expert Reports to be applicable to the entire Reach 2 alignment. Figure B-18 (From Bea Declaration No. I, format edits July 15, 2008) shows that wave-induced erosion was not implied over the entire Reach 2 alignment.

In addition, a single time series of waves and surges were not used in performing the wave erosion – breaching analyses. For a given storm characterization (different potential Hurricane Katrina conditions), multiple surge elevations and wave time histories until the point of surge overtopping were analyzed to develop the analyses. Also, multiple characterizations of the EBSB configurations, surface coverage, and soil properties were studied to develop an understanding of the parameteric uncertainties associated with results from the EBSB wave erosion models.

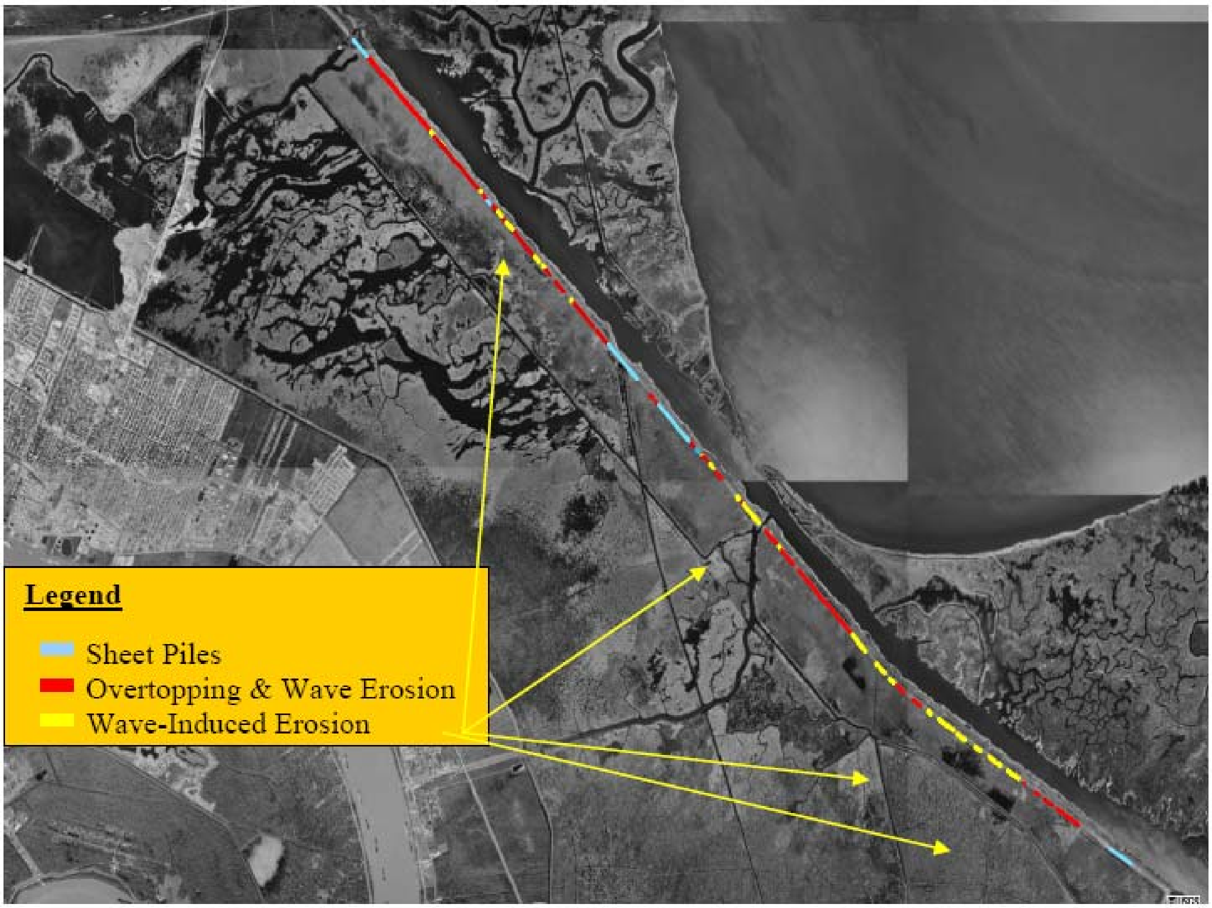


Figure 124: Identification of wave-induced breach sites based on visual examination of 2005 aerial LiDAR data (Bea 2008a, Bea and Storesund 2008a).

Figure B-18. Wave-induced erosion was only implied to occur at some locations, not the entire alignment.

2. *Lack of demonstrated applicability of the LS-DYNA model for velocities.*

This comment has been addressed in the validation studies (see Technical Report No. 2 included with this Declaration). Velocity profiles generated by LS-DYNA were compared with USACE laboratory studies and were found to be in substantial agreement (Figure B-19).

In addition, the velocities developed using LS-DYNA were in substantial agreement with empirical formulations as well as in substantial agreement with USACE (Ebersole, IPET) estimated values as well.

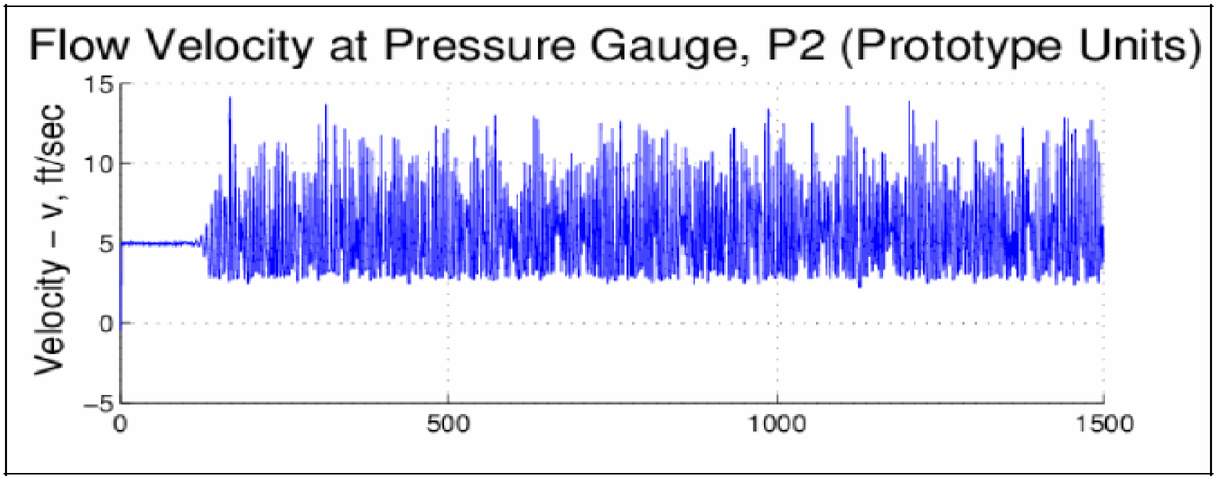


Figure 20: Velocity profile at P2 for a storm surge at +21 feet, a significant wave height of 3 feet and a period of 6 seconds (Hughes, 2008).

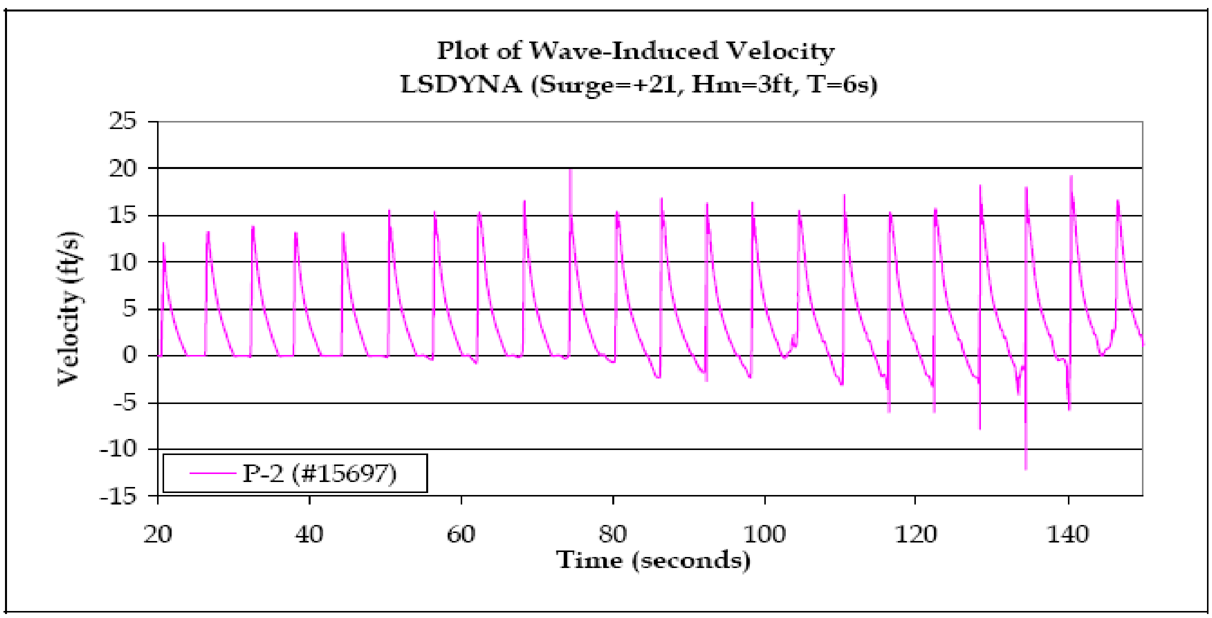


Figure 21: Velocity profile as determined by LSDYNA. The maximum velocity is approximately 20-30% higher than determined by Hughes (2008).

Figure B-19. Velocity comparisons between laboratory test results and LS-DYNA.

3. *Incorrect application of the LS-DYNA code for predicting run-up and down-rush velocities on the water side faces of the EBSBs / Levees .*

The nature of this criticism / observation is unclear. We agree that wave energy was simplified through a regular sea state, however how is this an incorrect application of LS-DYNA code? Also, the presented wave equations do not explicitly outline the algorithm employed by the LS-DYNA code, rather these were scaling components to generate the desired hydrodynamics (as described by Resio's Equation 2 on page 27 of his report).

In his report, Dr. Resio presents new results that have been developed using the computer program COULWAVE. This computer program was used to analyze levee overtopping velocities during the USACE IPET investigations and analyses. These velocities were compared with those developed by Bea in his July 2008 Expert Report. Excellent agreements were found.

Dr. Resio cited a large number of references that provided background for the theoretical formulations incorporated in COULWAVE and for its validations. These references were not provided with his Expert Report. A request was made by Dr. Bea to obtain electronic copies of these references. This request was rejected by the Defense on the grounds that the references were "publicly available." Unfortunately, the majority of the references were not publicly available; special access privileges were required. This lack of professional courtesy is very unfortunate. Dr. Bea previously furnished electronic copies of <u>all</u> the important references cited in his Plaintiffs Expert Reports. As a result of the 'obstruction' provided by the Defense, all of the important references providing important background on the development, theoretical bases, and validations of the COULWAVE computer program could not be obtained and reviewed.

The references that were obtained and reviewed indicate the COULWAVE computer program is founded on sound theoretical principles and has been appropriately validated with results from other similar analytical approaches, results from laboratory experiments, and results from field experiments.

In Dr. Resio's Expert Report, a "typical profile" (cross-section) for the Reach 2 EBSBs (Levees) was used as a basis for the analyses and calculations of the water velocities acting on the water side and protected side of the EBSBs (Figure B-20). Figure B-21 shows the profile at the EBSB Wave Erosion Study Location identified in Dr. Bea's Expert Report. Figure B-22 is a photograph of a section of the Reach 2 EBSB in the vicinity of the EBSB Wave Erosion Study Location. Figure B-23 provides additional details pertaining to the Reach 2 EBSB cross-sections. It is immediately evident that the "typical profile" used by Dr. Resio is highly idealized and omits many of the important local features that have important effects on the hydrodynamic interactions between the water and the EBSBs. Of particular importance is the omission of vegetation between the banks of the MR-GO and the 'toes' of the EBSBs. The 'idealization' of the EBSB cross-section used in the COULWAVE analyses points out an important aspect of the COULWAVE computer program analyses. While the COULWAVE computer program appears to have been adequately validated and calibrated, it is equally if not more important that the data and information input to the computer program is realistic and that the output from the computer program is properly analyzed and interpreted; "garbage in – garbage out".

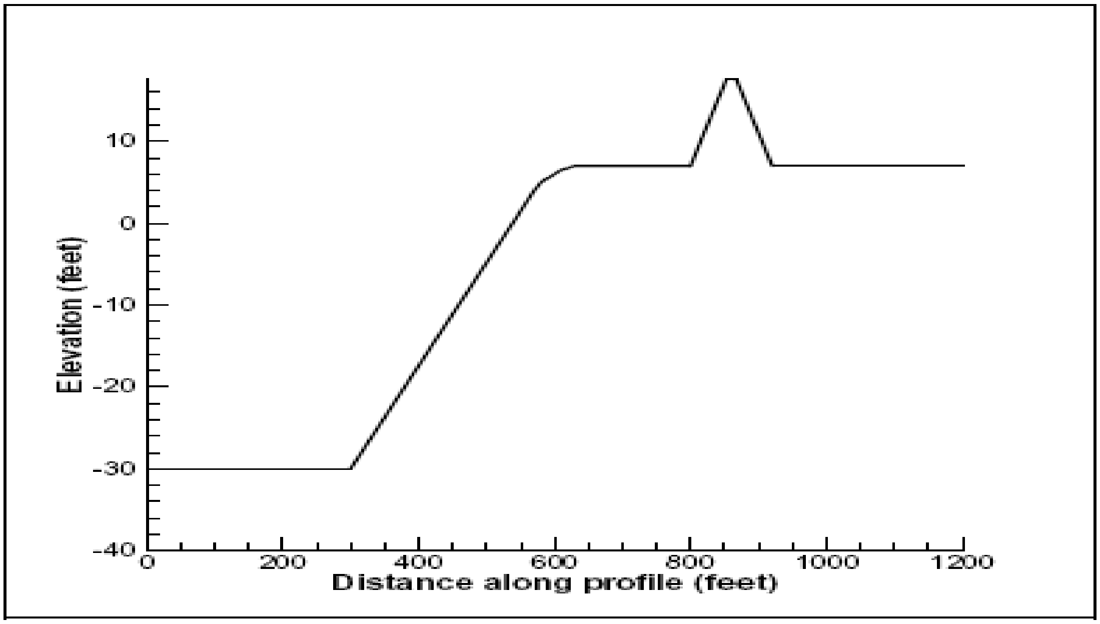


Figure 23. Generic profile for Boussinesq simulations with COULWAVE.

Figure B-20. Reach 2 EBSB cross-section used in COULWAVE fluid-structure interaction analyses (from Resio December 2008 Expert Report).

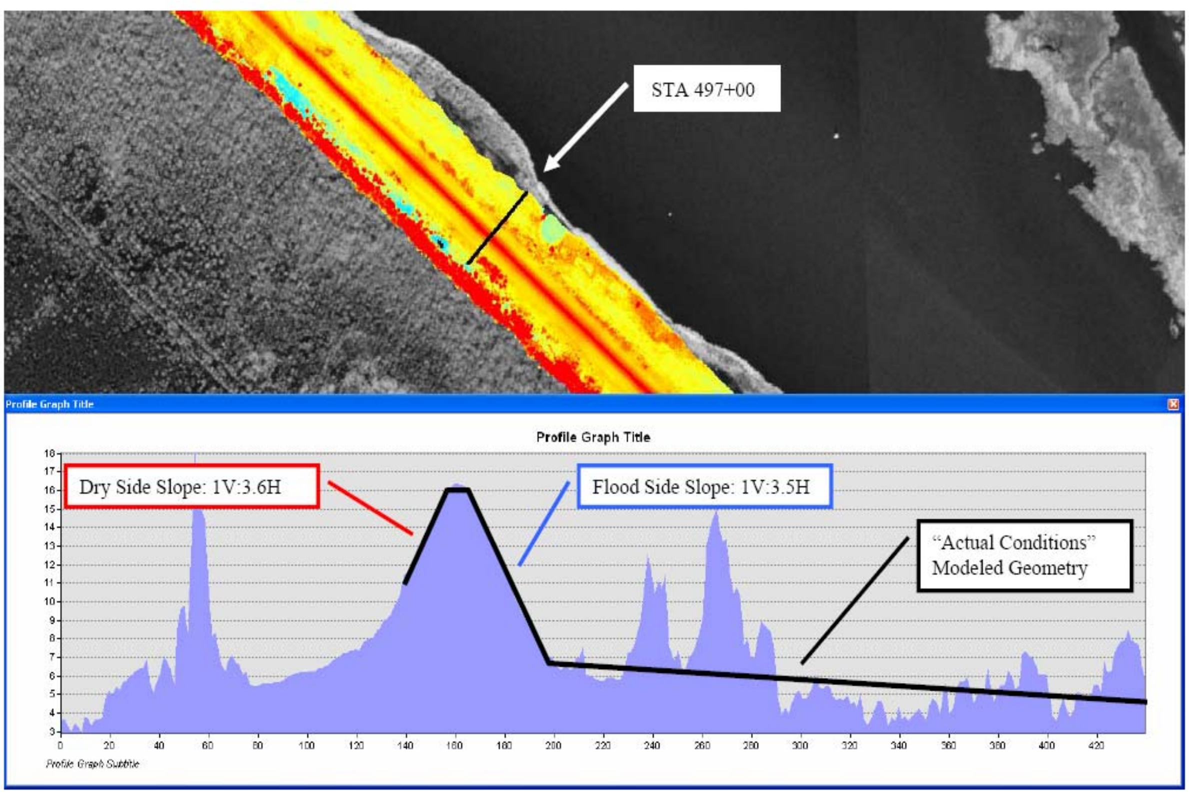


Figure B-21. Reach 2 EBSB Wave Erosion Study Location cross-section used in wave breaching analyses (from Bea July 2008 Expert Report.

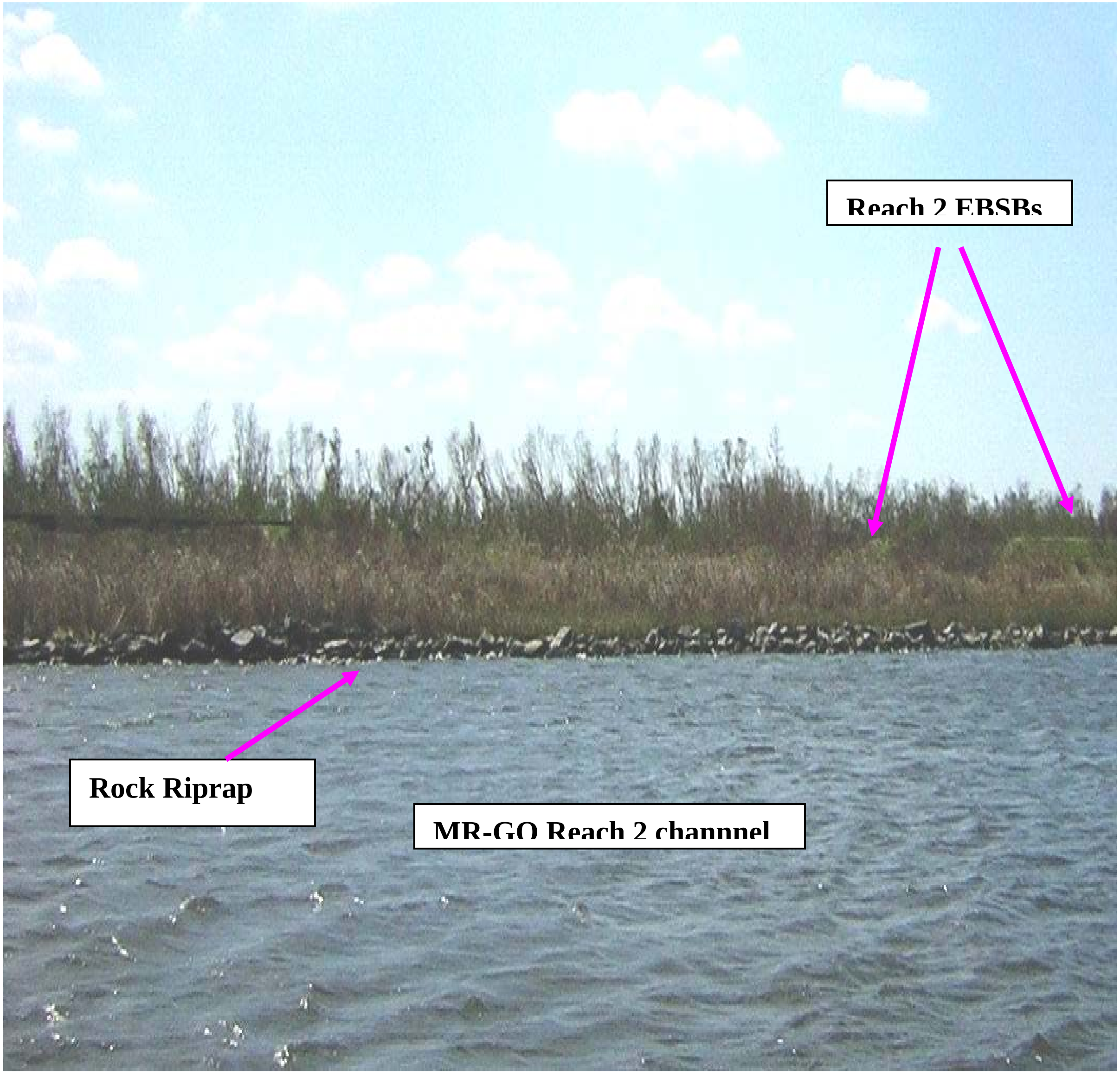


Figure B-22. Photograph of the vegetation and rock riprap between the toes of the EBSBs and the MR-GO channel mid way between Bayou Dupre and Bayou Bienvenue (looking toward the Reach 2 EBSBs from the MR-GO channel).

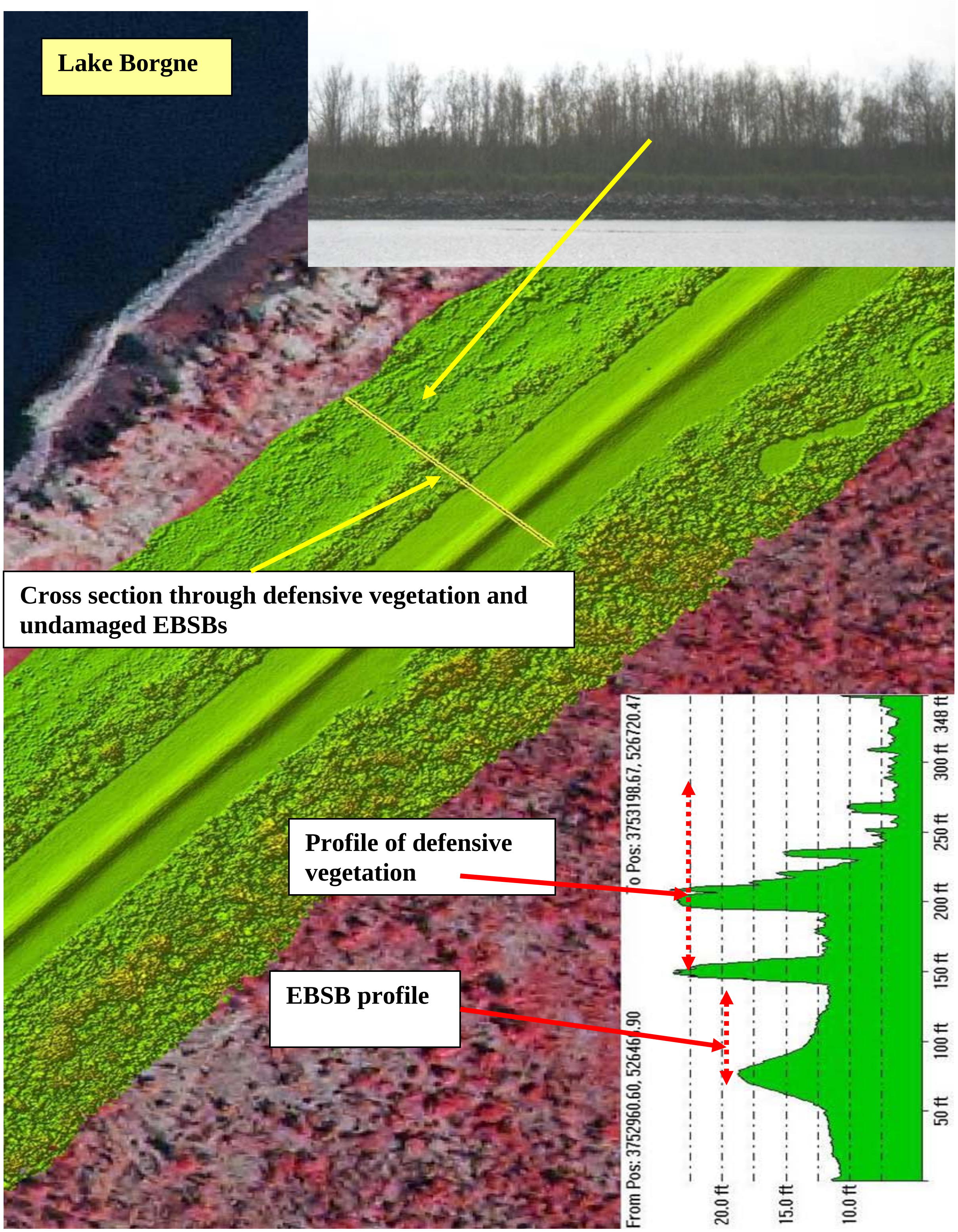


Figure B-23. Processed LiDAR post-Hurricane Katrina survey data showing the substantial defensive line of vegetation outboard of the MR-GO Reach 2 EBSBs and the undamaged profile of the EBSBs in this section of the alignment.

In his December 2008 Expert Report, Dr. Resio states that COULWAVE analyses were performed for a range of surge elevations – below and above the crest (crown) of the idealized EBSB (Levee) incorporated into the analyses. Figure 24 shows results from Dr. Resio's Expert Report. These results and those subsequently documented in his expert report pertain solely to conditions in which the surge elevations exceed the crest elevations of the EBSBs (Levees). As noted earlier, comparisons developed and documented in Dr. Bea's July 2008 Expert Report show excellent agreements with the results from the COULWAVE analyses for surge overtopping conditions.

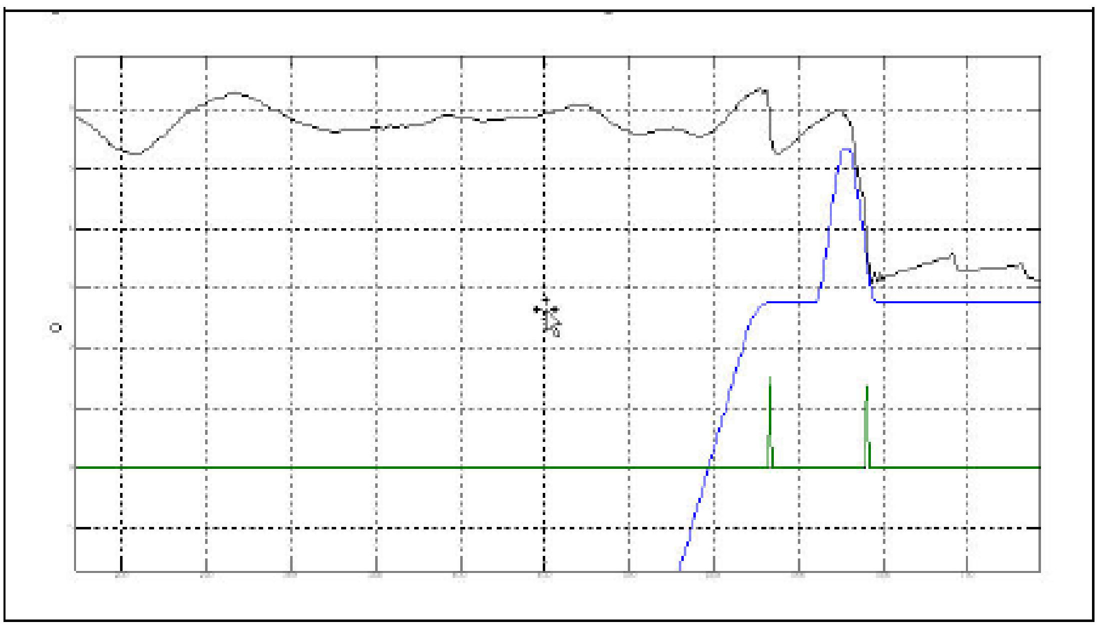


Figure 24. Snapshot of water surface from Boussinesq simulation over generic profile The green spikes denote strong wave breaking episodes.

Figure B-24. Results from COULWAVE analyses of fluid – structure interactions with idealized EBSB cross-section (from December 2008 Expert Report by Dr. Resio).

It is unfortunate, that Dr. Resio did not provide any results (e.g. water velocity characteristics) for the water side – wave attack conditions before surge overtopping. The report written by Dr. Resio indicates that these analyses were performed. Omission of documentation and discussion of these results is a very curious and important omission because a primary point of contention is whether or not water side wave erosion before surge overtopping could have initiated breaching of the EBSBs. I have requested that these analytical results be provided for my review. I have not received the requested information. Based on the results documented previously in the USACE IPET Hurricane Katrina report and in the documentation that could be obtained and reviewed on the validations of the COULWAVE computer program, the water side wave induced velocities that are developed before surge overtopping during the Hurricane Katrina conditions are very high and are capable of developing wave induced breaches in the EBSBs – EBSB crest crenellation exploited by overtopping flows to develop breaches.

Expert Report by Dr. Thomas Wolff

Dr. Dr. Wolff addresses the definition of a levee, presents a background on the design and construction of the MRGO levees, the design and construction of the Bayou Bienvenue and Bayou Dupre control structures, MRGO bank erosion, presents opinions of the USAE MRGO levee design, provides a description of the MRGO levees at the time of Hurricane Katrina, reviews erosion testing and modeling, and also evaluates the floodwall performance at the IHNC. This expert report is in substantial agreement with the analyses completed by the Plaintiffs. Dr. Wolff confirms /agrees with the following:

- Uncompacted soils were used to construct the MRGO levee;
- No formal erosion analyses as a result of wave action from the MRGO Deep Water Navigation Channel were completed; and
- Model validation and calibration are very important.

Overall, Dr. Wolff's observations and comments are excellent contributions. These contributions identify improvements needed for the completed analyses to be 'perfect.' However, these comments apply equally to both the Plaintiffs and Defense Experts. A primary reason that neither the Defense Experts or the Plaintiffs Experts can meet these 'ideal' criteria is the lack of required 'perfect' information and knowledge needed to perform and evaluate results from the analyses.

For example, Dr. Wolff states his opinion that the analyses do not directly prove that something DID happen, rather that the scenario was possible. What Dr. Wolff does not highlight, is that the required information by which to fully complete the analyses requires information not available, to either the Plaintiffs or the Defense Experts.