Technical Report No. II.

Analyses of Breaching at the MR-GO Bayou Dupre & Bayou Bienvenue Navigation Structures During Hurricane Katrina

Report to

Katrina Canal Breaches Consolidated Litigation

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

United States District Court

Eastern District of Louisiana

Pertains to MR-GO, Robinson [No. 06-2268]

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TABLE OF CONTENTS

1. SUMMARY 1

1.1 Overview 1

1.2 Analytical Models 4

1.2.1 Seepage – Hydraulic Conductivity Analyses 4

1.2.2 Slope Stability 5

1.3 FINDINGS 6

1.3.1 Seepage Analyses 6

1.3.2 Stability Analyses 7

2. DATA REVIEW 8

2.1 Navigation Structure - EBSB Wing-wall Tie-ins Construction History 8

2.1.1 First Lift, 1967 8

2.1.2 Second Lift, 1972 11

2.1.3 General Rise, 1985 12

2.1.4 General Rise, 1987 13

2.1.5 Installation of Sheet Piles and Final Rise, 1992 14

2.2 Bayou Bienvenue Control Structure 15

2.3 Bayou Dupre 17

3. OBSERVED EBSB RESPONSE 18

3.1 Bayou Bienvenue 18

3.2 Bayou Dupre 25

3.3 Observed Hydrograph 31

3.4 DESIGN CROSS SECTIONS vs LIDAR CROSS SECTIONS 32

3.4.1 Bayou Bienvenue 32
3.4.2 Bayou Dupre 34
4. STRATIGRAPHIC INTERPRETATIONS 37
4.1 BAYOU BIENVENUE 40
4.2 BAYOU DUPRE 40
5. SOILS DATA REVIEW 43
6. MATERIAL PROPERTIES 52
6.1 BAYOU BIENVENUE 52
6.2 BAYOU DUPRE 56
7. MARSH PERMEABILITY 60
8. SEEPAGE ANALYSES 64
8.1 Model Description 64
8.2 BAYOU BIENVENUE 68
8.2.1 Steady State 68
8.2.2 Transient Models 70
8.3 BAYOU DUPRE 70
8.3.1 Steady State 74
8.3.2 Transient Models 77
8.4 Potential for Piping Initiation 79
8.5 Seepage Modeling Observations 85
8.5.1 Bayou Bienvenue 85
8.5.2 Bayou Dupre 86

9. STABILITY ANALYSES 87

9.1 Model Description 87

9.2 BAYOU BIENVENUE 87

9.3 BAYOU DUPRE 91

10. SUMMARY OF RESULTS AND CONCLUSIONS 95

11. REFERENCES 96

1. SUMMARY

1.1 Overview

During Hurricane Katrina (August 29, 2005), the storm surge, waves, and currents induced many breaches along the Mississippi River Gulf Outlet (MR-GO) (USACE IPET 2007; ILIT 2006; Team Louisiana 2007). This report integrates available historical, geological and geotechnical information into comprehensive seepage – hydraulic conductivity and stability analysis models to evaluate development of the breaches at the wing-wall tie-ins on the south side of the Bayou Bievenue and north side of the Bayou Dupre navigation structures (Figures 1-1 through 1-6).

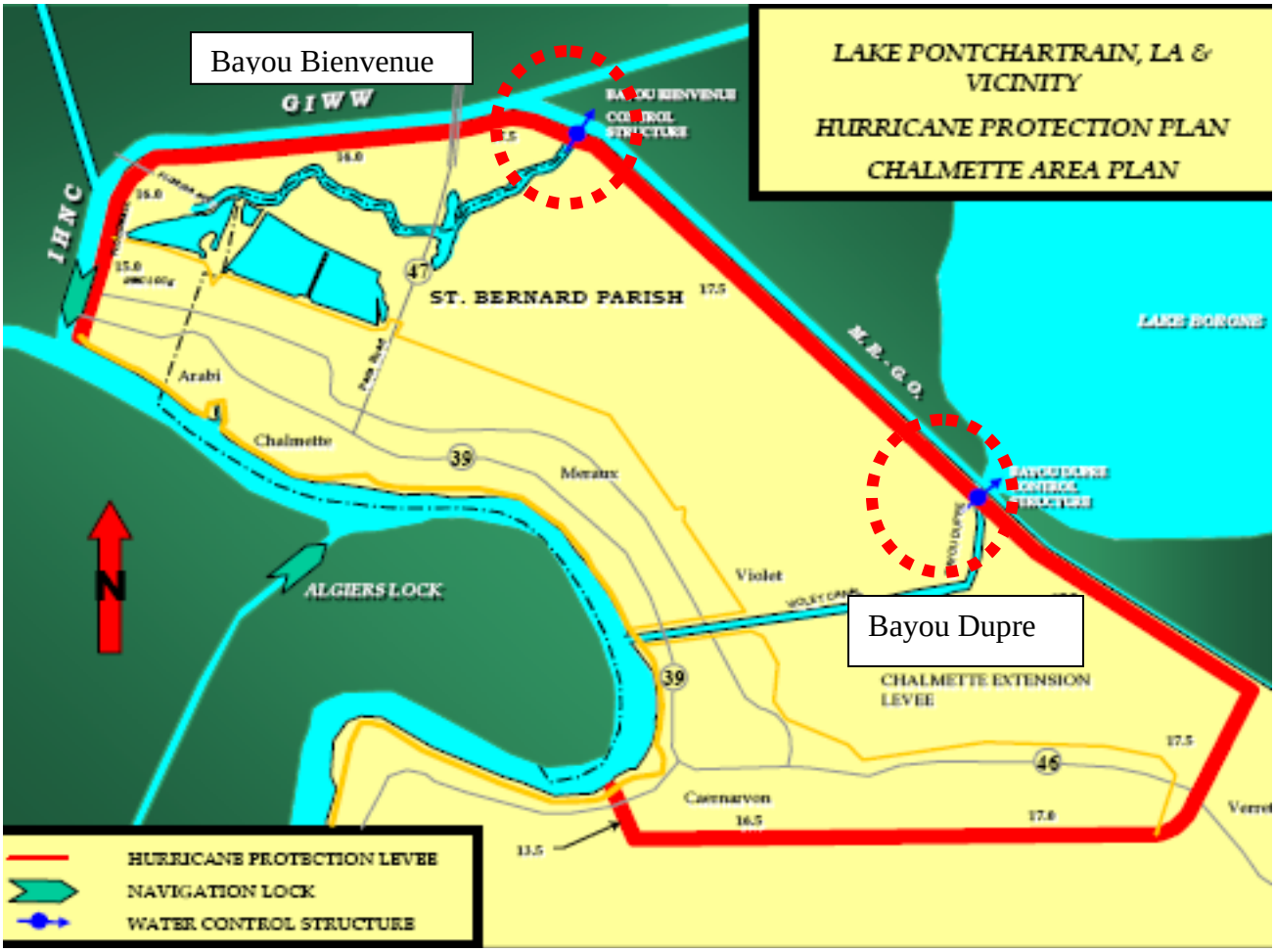


Figure 1-1. Bayou Bievenue and Bayou Dupre navigation structure locations (USACE IPET 2007).



Figure 1-2. Bayou Bievenue navigation structure south wing-wall breach (looking toward south). (EPA 2005)


Figure 1-3 Bayou Bievenue navigation structure South wing-wall breach (close-up looking toward West, September 1, 2005) (EPA 2005).



Figure 1-4. Large eroded breach at the contact between the south end of the concrete navigation lock structure at Bayou Bienvenue (ILIT 2006).



Figure 1-5. Breach at the contact between the North end of the concrete wing wall at Bayou Dupre Navigation structure (EPA 2005).


Figure 1-6. Bayou Dupre navigation structure north wing-wall breach (ILIT 2006).

1.2 Analytical Models

1.2.1 Seepage – Hydraulic Conductivity Analyses

In these analyses, soil pore water pressure development was assessed by means of Finite Element Modeling. The software SEEP\W, part of the computer analysis package GEOSTUDIO was employed for the task of calculating hydraulic heads, pore water pressures and hydraulic gradients through the EBSB[1] (Earthen Berm – Spoil Bank) and navigation structure wing-wall tie-in to EBSB fill materials.

The approach consisted on reviewing the geological information, borings, topographic surveys and stratigraphic profiles by DM03 and DM05 (USACE, 1966 and 1968), IPET (2006) and ILIT

[1] The term EBSB is used herein to describe the man-made structures composed of soil materials constructed along Reach 2 of the MR-GO. These man-made structures were composed primarily of uncompacted and unprotected dredged spoil materials.

(2006); in order to create a new geological, geotechnical and geometrical interpretation. Several design and construction documents were reviewed for the definition of excavation outlines, materials placed and dimensions of the EBSBs:

- Design Memorandum 3, Chalmette Area Plan – 1966.
- Design Memorandum 5, Chalmette Area Plan – Bayou Bienvenue and Bayou Dupre, 1968.
- Specification number DACW29-67-B-0134, Hurricane Protection EBSB First Lift, Station 368 – 570, 1967.
- Specification number DACW29-67-B-0136, Hurricane Protection EBSB First Lift, Station 594 – 770, 1967.
- Specification number DACW29-72-B-0137, Hurricane Protection EBSB Second Lift, Station 370 – 682, 1972.
- Specification number DACW29-85-B-0073, Hurricane Protection EBSB Second Enlargement, Station 380 – 692, 1985.
- Specification number DACW29-87-B-0014, Hurricane Protection EBSB Closures, Station 367 – 1005, 1987.
- Specification number DACW29-92-B-0061, Hurricane Protection EBSB Closures, Station 366 – 1007, 1992.

1.2.2 Slope Stability

The influence of the developed soil pore water pressures of a rising storm surge, and the distribution as a function of the relative permeabilities on slope stability was also assessed by calculating Factors of Safety (FOS) using Limit Equilibrium Analyses (LEA). Plots of FOS versus storm surge were developed for the three sites.

The effects of uncertainties of shear strengths, unit weights and storm elevations were assessed by calculating statistical parameters for all available field and laboratory test data. Slope stability analyses were coupled with Monte-Carlo simulations and Probability of Failure (Pf), Mean FOS

and Reliability Indices (β, $Pf \approx 10^{-\beta}$) were calculated based on the "expected values" and distributions of different parameters.

1.3 FINDINGS

1.3.1 Seepage Analyses

The seepage analyses show that soil pore water pressures develop quickly when the EBSB embankment is composed by permeable materials or excavations are backfilled with shell materials.

Whenever shell fills are present (i.e. Bayou Bienvenue and Bayou Dupre), saturation of these permeable materials is achieved quickly and high internal water flow velocities are developed. Potential for piping and internal erosion is triggered when soil pore water pressures exceed the overburden stresses of the soils near the EBSB toes. This was the case for Bayou Bienvenue and Bayou Dupre where the overburden stress was surpassed by the soil pore water pressures well before the surge reached the top of the EBSB, thus initiating erosive processes early the morning of August 29, 2005.

The design documentation for Bayou Bienvenue (DM05, 1968) shows that the bottom of the excavation for the control structure was not completely cut off by the installed sheet piles, thus leaving a space between the tips of the piles and the natural ground (banks and channel of buried bayou), which was filled with shell materials. The seepage analyses show that the dominant process at this site is seepage and high soil pore water pressures leading to internal erosion through the shell fill. No fine grained blanket is present at the toe of the EBSB, and hydraulic gradients (ratio of change in water pressure as function of distance) are smaller than 0.5 for the modeled storm surge conditions.

Bayou Dupre shows two shell fill horizons within the embankment, through which the storm can easily seep once the surge reaches them. Pressures develop rapidly inside these pervious materials, and computed internal velocities are larger than 10 feet /day inside the shell fill. No fine grained blanket is present at the toe of the EBSB, and hydraulic gradients are smaller than 0.5 for the modeled storm. These results indicate the main process that triggered instability was loss of shell material and progressive erosion of the rest of the EBSB.

1.3.2 Stability Analyses

The analytical models utilized in these analyses consider shallow slip surfaces on the protected side slope and other deeper surfaces passing through permeable materials (shell, sand, marsh). Circular, non-circular and translational shapes were evaluated for the two navigation structure wing-wall tie-ins to the EBSBs:

Bayou Bienvenue shows a critical failure surface originated near the waterside toe, passing through the shell fill and underneath the sheet pile tips. The bottom of the surface runs along the Marsh deposit until its exit landward of the EBSB toe. The computed Mean FOS for the water level at the top of the sheet pile (+17.5 feet) is FOS = 1.06, indicating incipient failure.

The computed factor of safety for Bayou Dupre for the storm surge at the EBSB crest (El. ~+19 feet) is 1.5. No critical surfaces were computed passing through the shell materials.

2. DATA REVIEW

2.1 Navigation Structure - EBSB Wing-wall Tie-ins Construction History

The review of the information on design and construction of the EBSBs along MR-GO has helped define five primary construction stages (DM05, 1968). These stages are useful in characterizing the EBSB materials and locating sites where sand or shell materials were used to fill EBSBs or closure structures. The results of this assessment were used in generating detailed cross sections, stratigraphy and seepage and stability analyses.

The five construction stages were:

- First lift, 1967
- Second lift, 1972
- General rise, 1985
- General rise, 1987
- Installation of Sheet piles and final rise, 1992

A brief description of each stage is described in the following sections.

2.1.1 First Lift, 1967

According to DM03 (1966), the first lift of EBSB along MR-GO was designed in this year. Its design crest elevation was +17.5 feet MSL (17.25 feet NAVD88). In document DACW29_67_B_0134 (USACE, 1967), the first lift of EBSB was performed to elevations between +11.75 feet and +14.75 feet (NAVD88) and was done between the following stations:

- 368+00 – 387+00. This is just downstream of the Bayou Bienvenue Control Structure. The rise was performed to Elevation +11.75 feet. The stretch of EBSB treated was 1,900 feet long (Figure 2-1).

- 472+40 – 516+00. 4,360 feet of EBSB were risen to Elevation +11.75 feet. This reach included Bayou Villere and the third analysis site of this report (Station 497+00).
- 522+00 – 570+00. The EBSB was raised to elevation +14.75 feet throughout 4,800 feet of EBSB.
- 594+00 – 682+00. Crest elevation of +14.25 feet was designed for this 8,800 foot stretch.
- 705+00 – 770+00. This reach is located 500 feet downstream of the Bayou Dupre Control Structure. The rise was designed to Elevation +14.25 feet. (Figures 2-2 and 2-3).

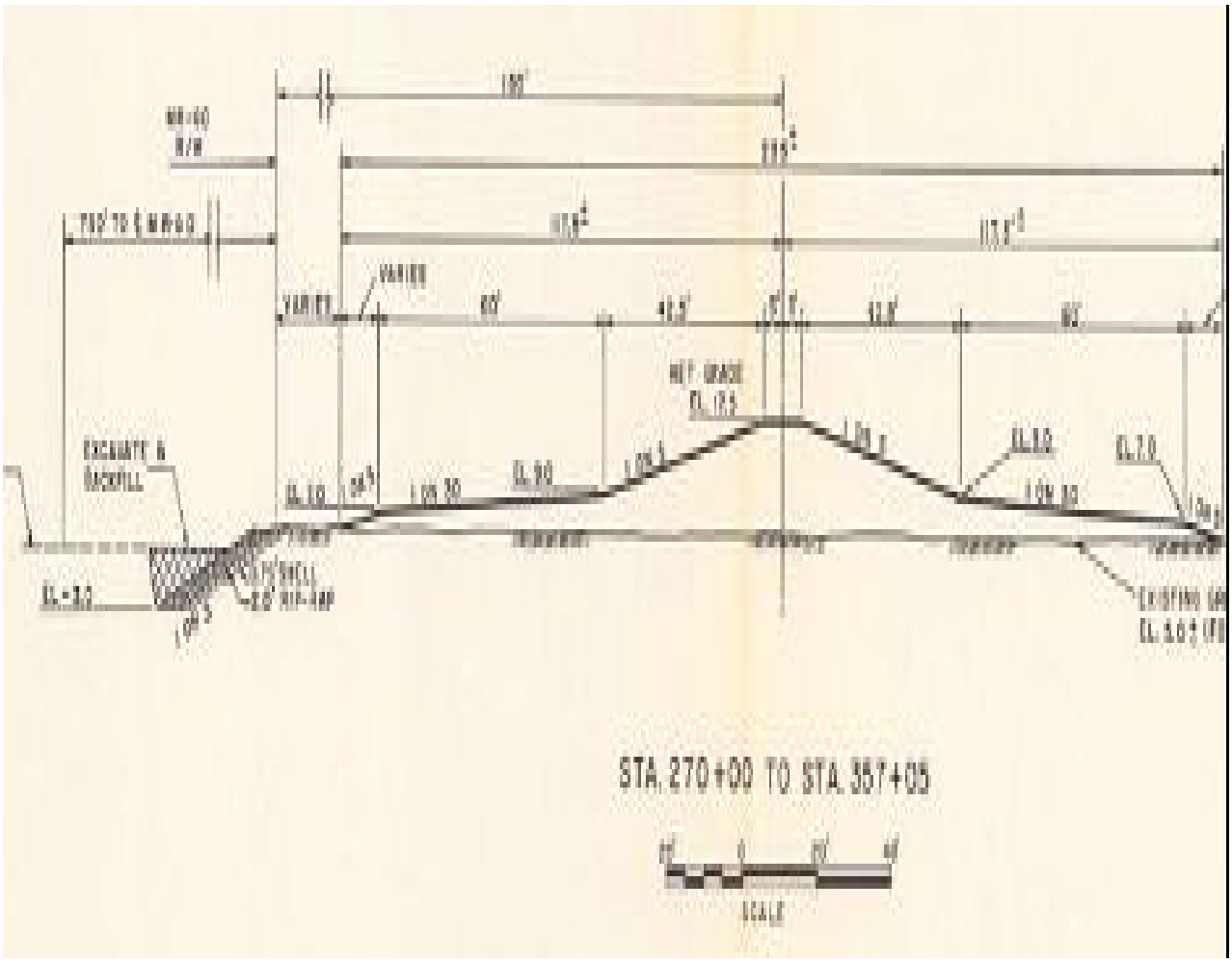


Figure 2-1. Bayou Bienvenue 1967 design cross section. Source: DM03, Chalmette Area Plan, 1966. Top of EBSB was designed to elevation +17.5ft MSL (+17.25ft NAVD88) and slopes of 1V : 5H. The centerline of MR-GO is shown approximately 900ft away from the EBSB centerline.

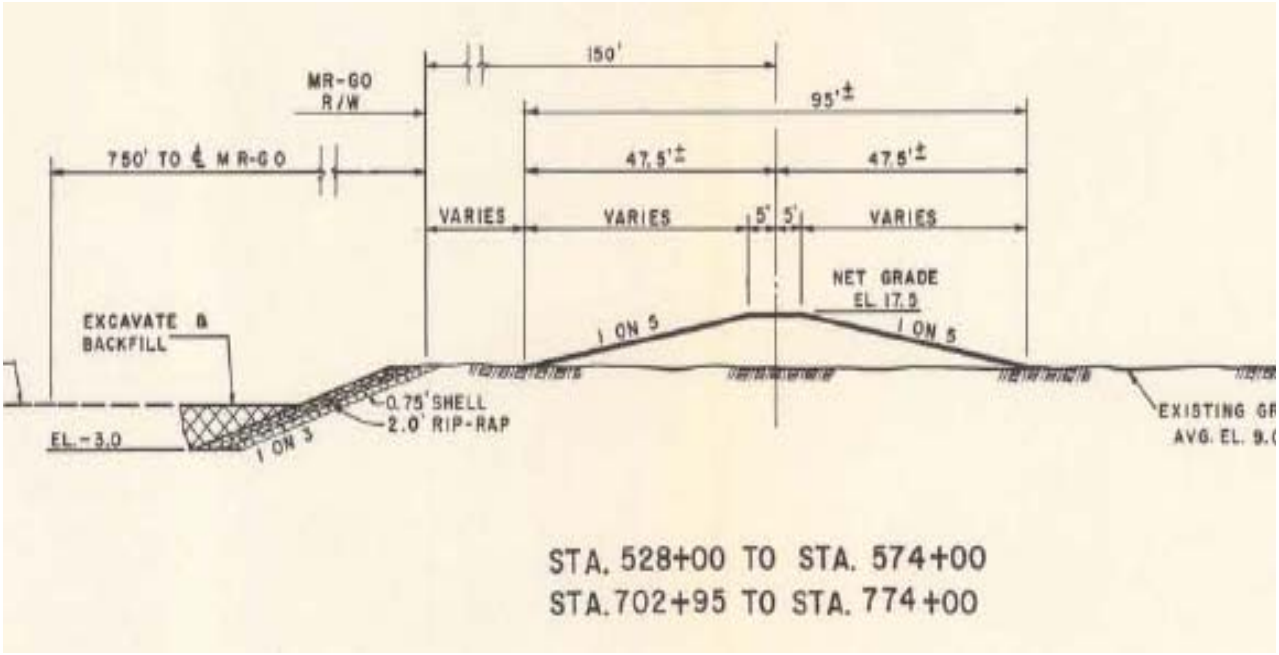


Figure 2-2. Bayou Dupre 1966 design cross section. Source: DM03, Chalmette Area Plan, 1966. Top of EBSB was designed to elevation +17.5ft MSL (+17.25ft NAVD88) and slopes of 1V : 5H. The centerline of MR-GO is shown approximately 900ft away from the EBSB centerline.

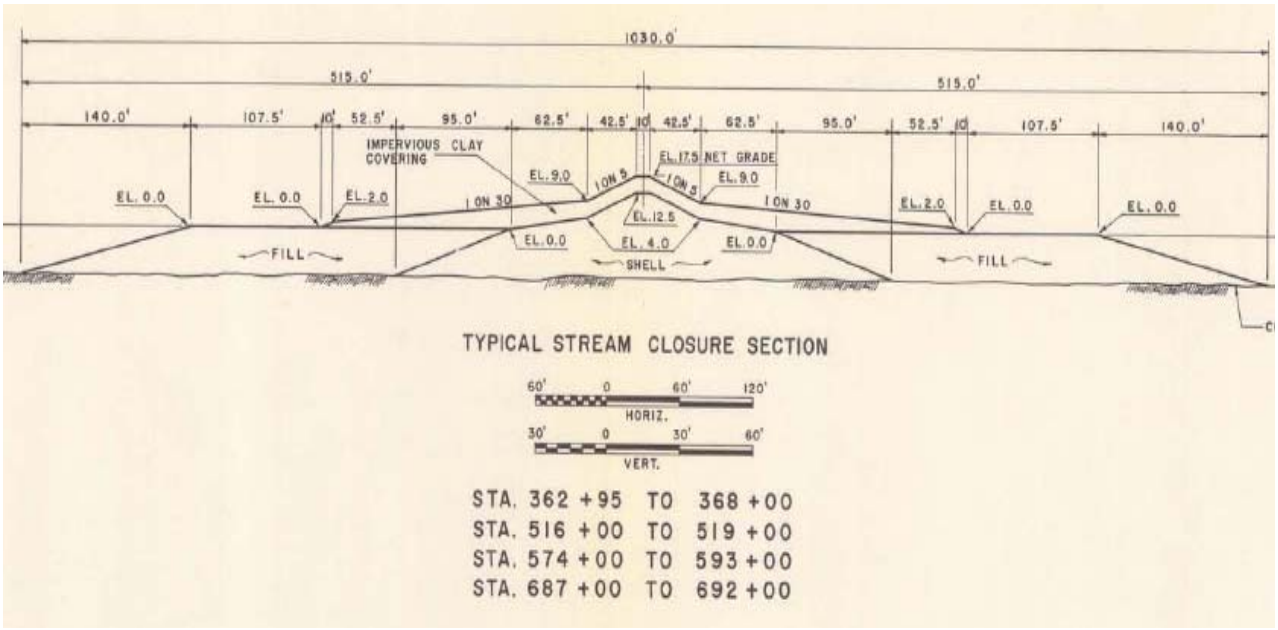


Figure 2-3. Typical 1966 design stream closure cross section. Source: DM03, Chalmette Area Plan, 1966. Top of EBSB was designed to elevation +17.5 feet MSL (+17.25 feet NAVD88) and slopes of 1 vertical : 5 horizontal. The shell core extended from the bottom of the bayou, approximately elevation –14 feet and was capped from elevation +12.25 feet by an "impervious clay covering". The side slopes were covered with hydraulic fill.

2.1.2 Second Lift, 1972

Document DACW29_72_B_0137 shows the designed lift for the general area between stations 370+00 to 682+00. This encompasses the area between Bayou Bienvenue and Dupre Control Structures. Most important, MR-GO was dredged down to elevation –70 feet in this section.

The design elevation for the new rise was +17.75 feet, except at the closure sections where the following elevations were left:

- 514+00 – 520+00. Bayou Villere closure. The existing +13.75 feet of crest elevation were not modified (Figure 2-4).
- 564+00 – 597+00. Natural Gas Pipeline Canal closure. The existing clay-capped shell core had a crest elevation of +10.25 feet. The new lift did not cover this reach.

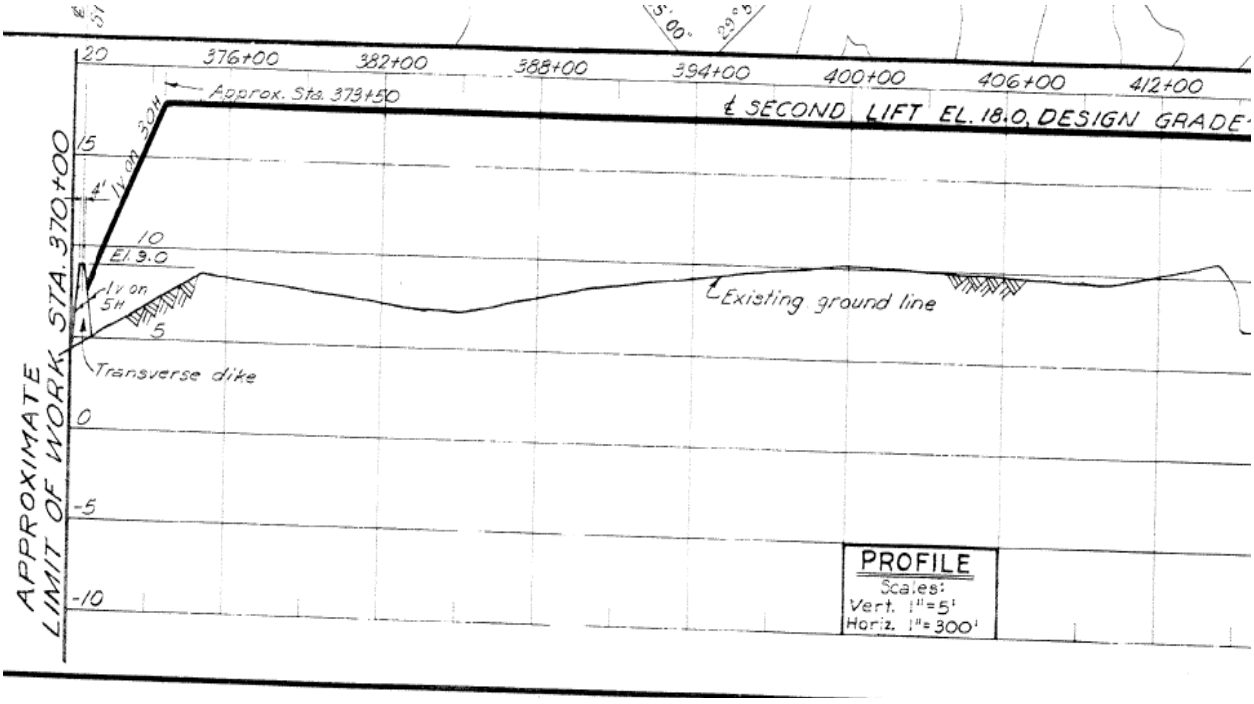


Figure 2-4. Typical 1972 second lift design elevation. Source: DACW29_72_B_0137. Top of EBSB was designed to elevation +18 feet MSL.

2.1.3 General Rise, 1985

The EBSBs were raised to elevation +20.3ft NAVD88 between stations 380+00 and 692+50; except at the following stations (The following stations for the 1985, 1987 and 1992 designs refer to the MR-GO stationing, which is larger than the EBSB stationing by 760 feet, therefore Bayou Bienvenue Control structure is located at Sta. 360+00 – EBSB- = 367+60 -MR-GO-)

- 522+00 – 531+00. These 900 feet of EBSB were treated in 1983 as part of work in Bayou Villere. The final elevation after this year was +18.8 feet (Figure 2-5).
- 580+00 – 604+00. The natural gas pipeline canal was also modified in 1983 and set to elevation +12.8 feet.

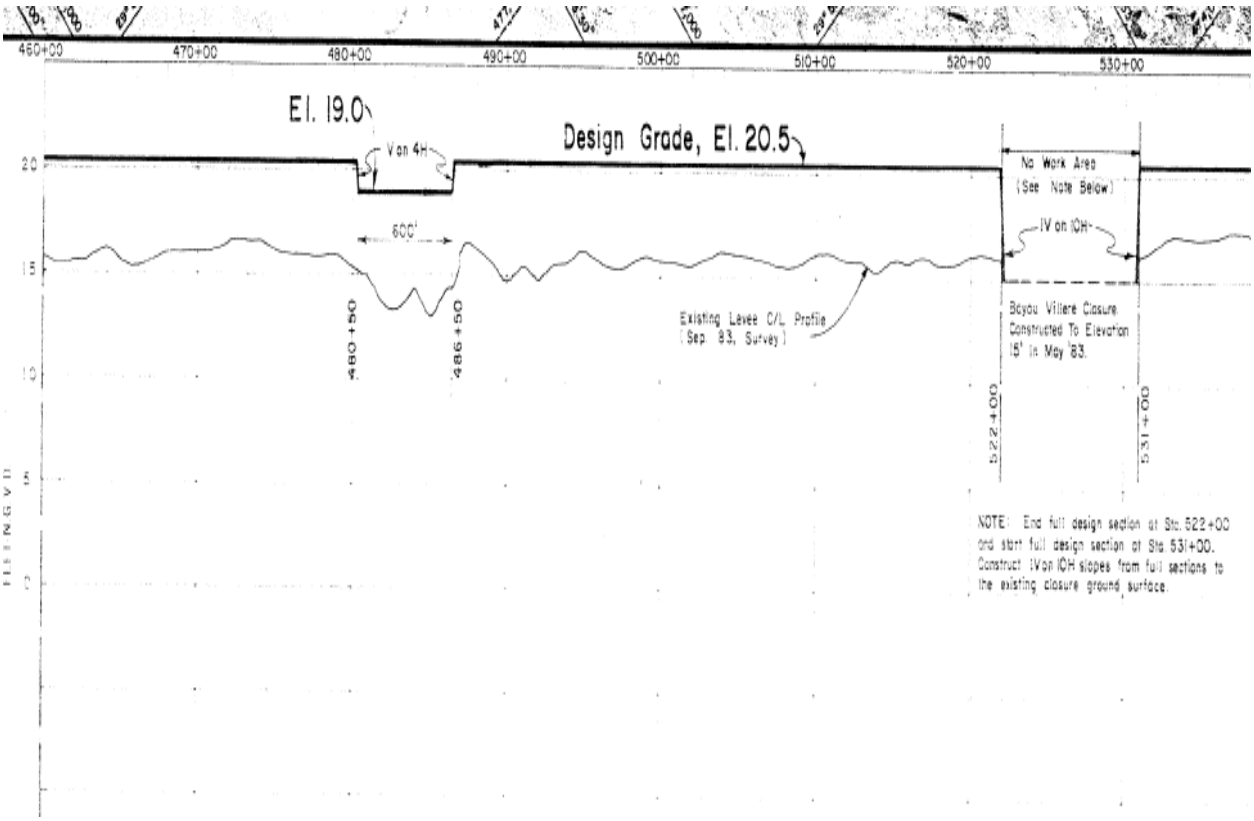


Figure 2-5. Typical 1985 design elevation. Source: DACW29_85_B_0073. Top of EBSB was designed to elevation +20.5 feet NGVD.

Additional borings were drilled in 1984 as part of the Hurricane Protection EBSB Second Enlargement MR-GO, Sta 380 to 692.

2.1.4 General Rise, 1987

The work performed in 1987 started just downstream of the T-walls at the Bayou Bienvenue control structure. The design consisted in raising the elevations of the closure structures to the following elevations:

- 367+00 – 381+00 (Bayou Bienvenue). Risen from +12.5 feet to +15 feet (Figure 2-6).
- 552+00 – 531+00 (Bayou Villere). Risen to +18.5 feet.
- 580+00 – 600+00 (Pipeline canal). Risen to +12.5 feet.
- 692+00 to 706+2 (Bayou Dupre). Raised from +12.5 feet to +18.5 feet.

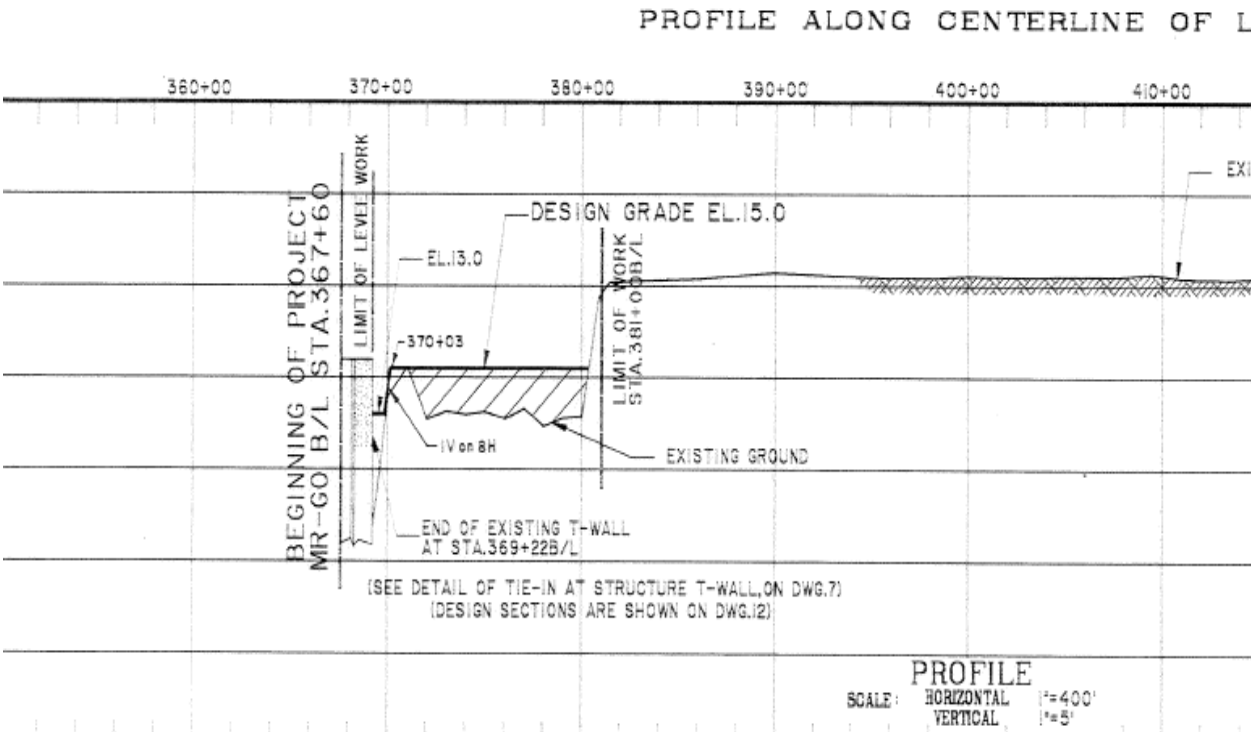


Figure 2-6. Typical 1987 EBSB crest elevation. Source: DACW29_87_B_0014. Top of EBSB remained at elevation +20.5 feet NGVD, and the closure sections were raised to the elevations cited above.

Available borings for this design stage included the previously mentioned, and a new set of exploration performed in 1987 for the Hurricane Protection EBSB Closures, MR-GO Sta 367 to 1005.

2.1.5 Installation of Sheet Piles and Final Rise, 1992

Sheet Piles were installed and crest elevations raised at the following locations:

- 366+00 - 381+00 (Bayou Bienvenue). Sheet pile top at elevation +17.5 feet and tip elevation at -5.2 feet. The EBSB crest was maintained at elevation +15 feet (Figure 2-7).
- 479+95 – 487+48 (Bayou Villere). 14 foot long piles (from elevation +18.3 feet to +4.3 feet) were installed along the 750 feet of this reach. The EBSB crest was also maintained at elevation +14.8 feet.
- 579+90 – 604+00 (Pipeline Canal). 21.5 feet long sheet piles were installed from elevation +18.3 feet to -3.2 feet. The existing closure EBSB (composed by a shell fill) was capped with a "semicompacted" fill to elevation +14.8ft (DACW29_92_B_0061).
- 620+00 – 647+00 and 652+86 – 656+45. (North of Bayou Dupre). 21.5 foot long sheet piles were installed, and EBSBs rose to elevation +14.8 feet.

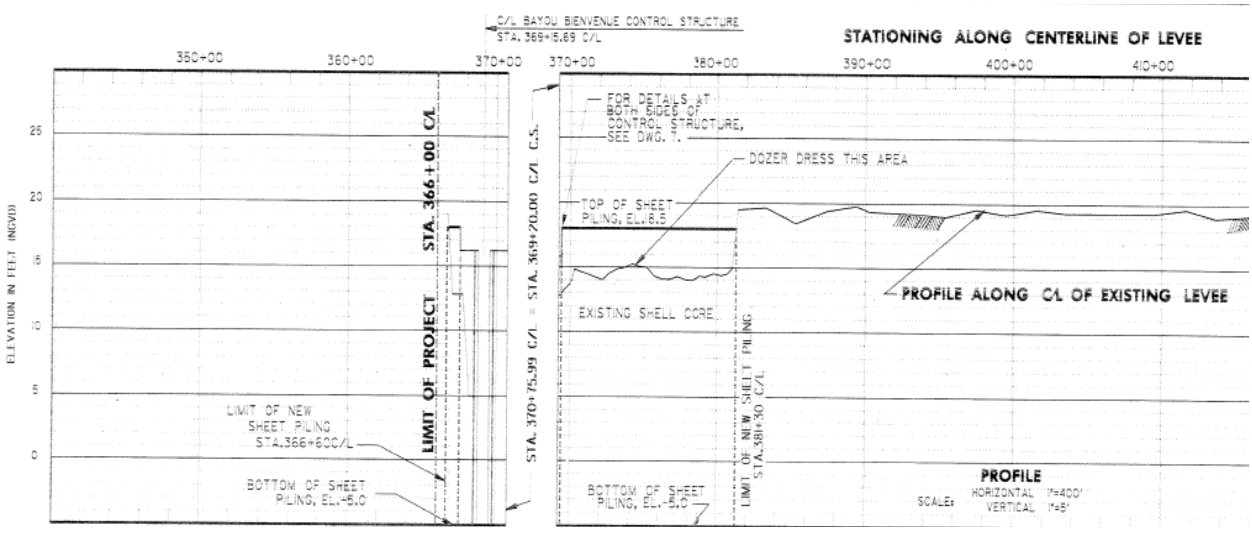


Figure 2-7. 1992 sheet pile installation south of Bayou Bienvenue. Source: DACW29_92_B_0061.

The final rise of the EBSBs along MR-GO was designed using the previous information and yet another set of borings drilled in 1988.

Five main stages of EBSB design along MR-GO took place since 1967. The existing EBSBs, originally topped at elevation 4 to 5ft, were to be raised to elevations +14.7, +17.8 and +18 feet, in 1967, 1972 and 1985, respectively.

Stream closure sections are composed by a shell core, capped with an approximately 5 feet thick clay layer and hydraulic fill on the side slopes.

The longitudinal and cross sections created for this report show approximate boundaries between these fill stages and approximate cross sections for the closure sites near the three sites of analyses.

2.2 Bayou Bienvenue Control Structure

The control structure at Bayou Bienvenue (Station 360+00) ties into the EBSBs to the north and south with concrete I-walls with top elevation of approximately 17.5 feet (Figure 2-8).

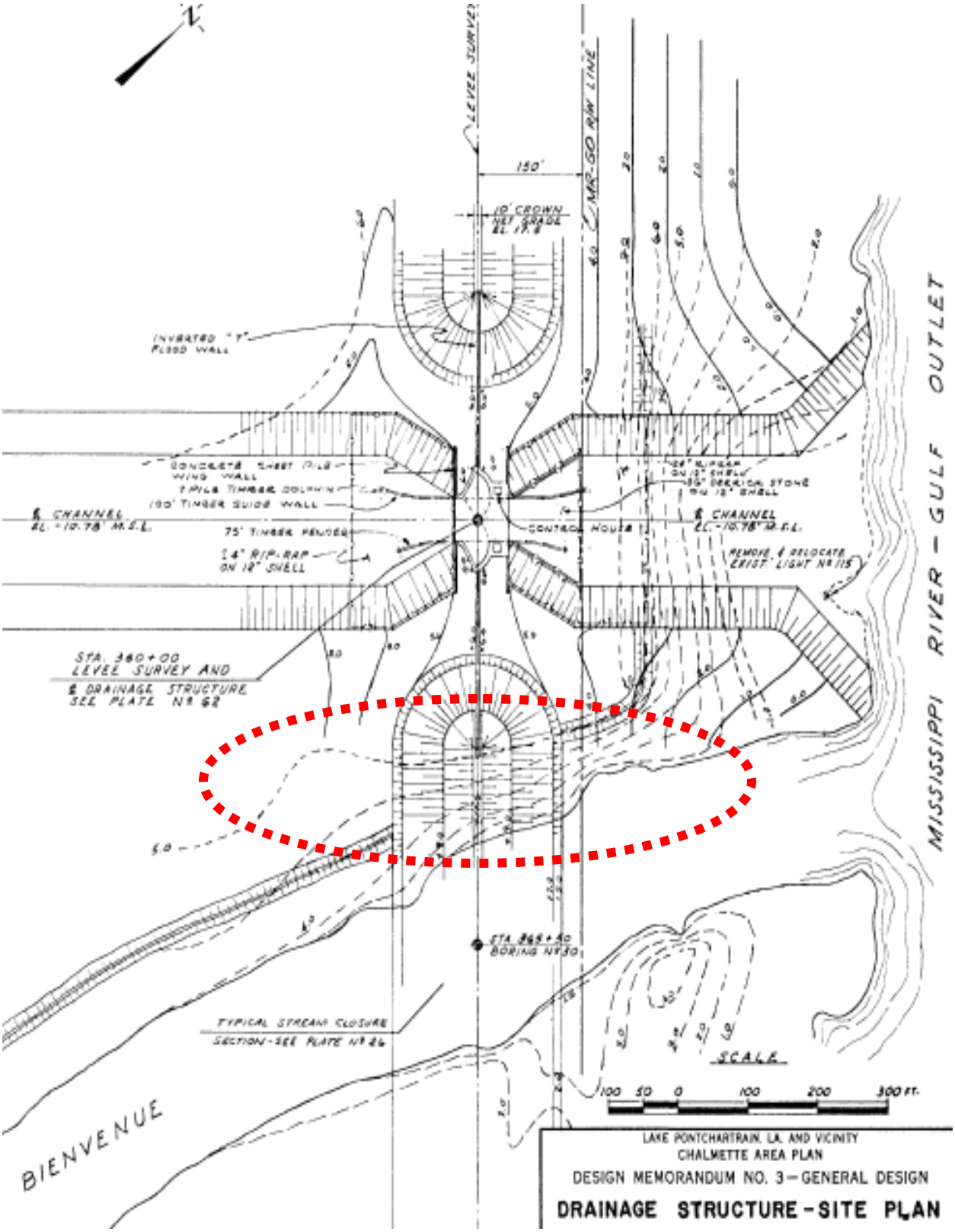


Figure 2-8. Bayou Bienvenue Control Structure. Source: Plate 60, DM03 – Chalmette Area Plan, 1966.

2.3 Bayou Dupre

The control structure at Bayou Dupre (Figure 2-9) keys into the adjacent EBSBs with concrete sheet piles topped at elevation 17.5 feet. The EBSB which it keys into is more than 1ft taller than the top of the concrete structure.

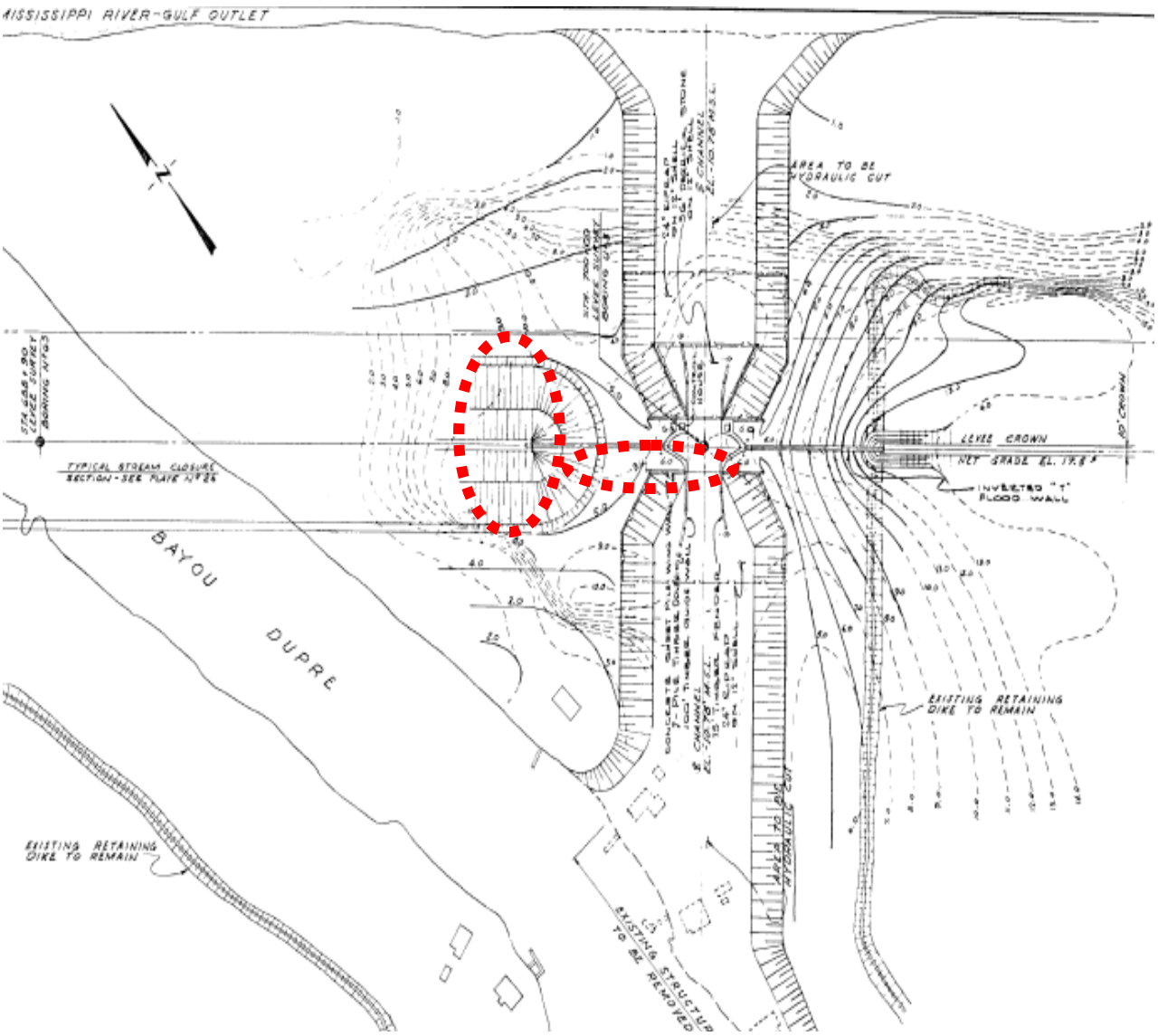


Figure 2-9. Bayou Dupre Control Structure. Source: Plate 61, DM03 – Chalmette Area Plan, 1966.

3. OBSERVED EBSB RESPONSE

3.1 Bayou Bienvenue

Figure 3-1 shows pre and post failure conditions on the Bayou Bienvenue flood control structure. It is seen from the pre-storm conditions that the earthen EBSB stretch north of the Bienvenue channel connected to the concrete structure by a steel I-wall. South of the channel, the EBSB connects to the concrete structure by a shorter sheet pile; and the transition is followed by a steel sheet pile that extends several hundred feet south.

The post-Katrina picture shows that several erosion mechanisms occurred on this site, including approximately 1000 feet of overtopping scour on the protected side slope, just downstream of the sheet pile due to overtopping.

The floodwall to the north of the channel also suffered some erosion due to overtopping and turbulence in the concrete wing wall – earthen EBSB interface. However, the major breach at this location occurred just south of the channel, where the foundation soils catastrophically failed.

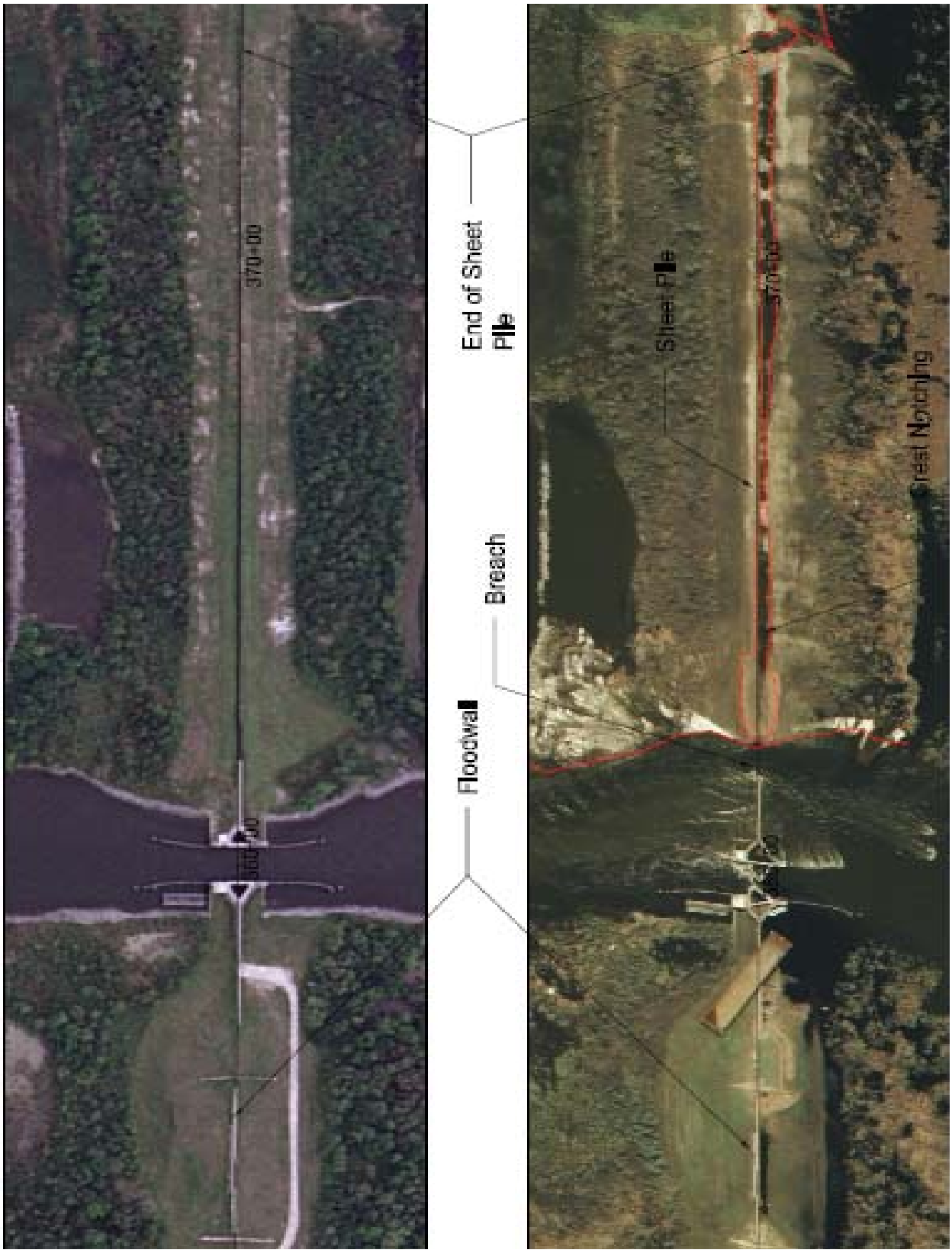


Figure 3-1. Pre and Post failure conditions on Bayou Bienvenue.

IPET (2006) estimated that 28,600 cubic yards of granular fill would be necessary to close the scour hole generated by the breach. The plan area of the scour hole was calculated to be 5,246 sq. yards, therefore a volume of 28,600 cubic yards translates into a depth of approximately 17 feet of scour. Figure 3-2 shows an oblique view of the breach on Bayou Bienvenue.



Figure 3-2. Aerial picture of the breach on Bayou Bienvenue. Shading shows the extents of the scour hole (EPA 2005).

The post failure conditions indicate that the direction of breaching follows the arrows shown in Figure 3-2. The shell material ejecta (white soil on the bottom right corner) describes the preferential flow direction.

Figure 3-3 shows a longitudinal section from Plate IV-36 of DM05 (USACE, 1968) with the design elevations of the different elements of the control structure.

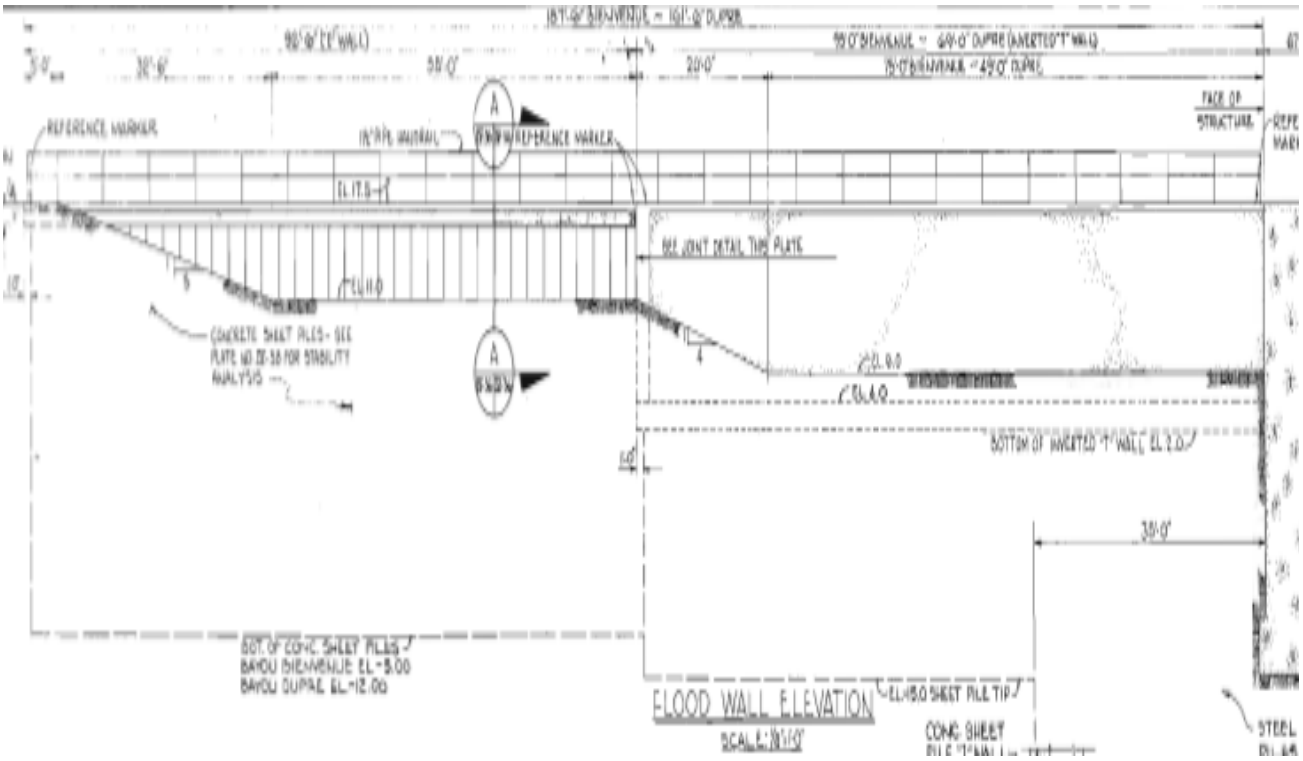


Figure 3-3. Longitudinal section through control structure. Source DM05, plate IV-36.

The T-wall (intact concrete wall in Figure 3-2) south of the southern closure structure is founded on 88 foot long timber piles. The piles next to the T-wall are steel sheet piles driven down to elevation -5.2 feet (NAVD88). Failure is believed to have initiated in the interface between this short pile and the earthen EBSB which it keys into. Had this T-wall been founded on piles, it is believed it too would have failed following the underseepage induced erosion through the southern embankment. Figure 3-4a shows the plan view of the excavation for the control structure, which was subsequently filled with shell materials.

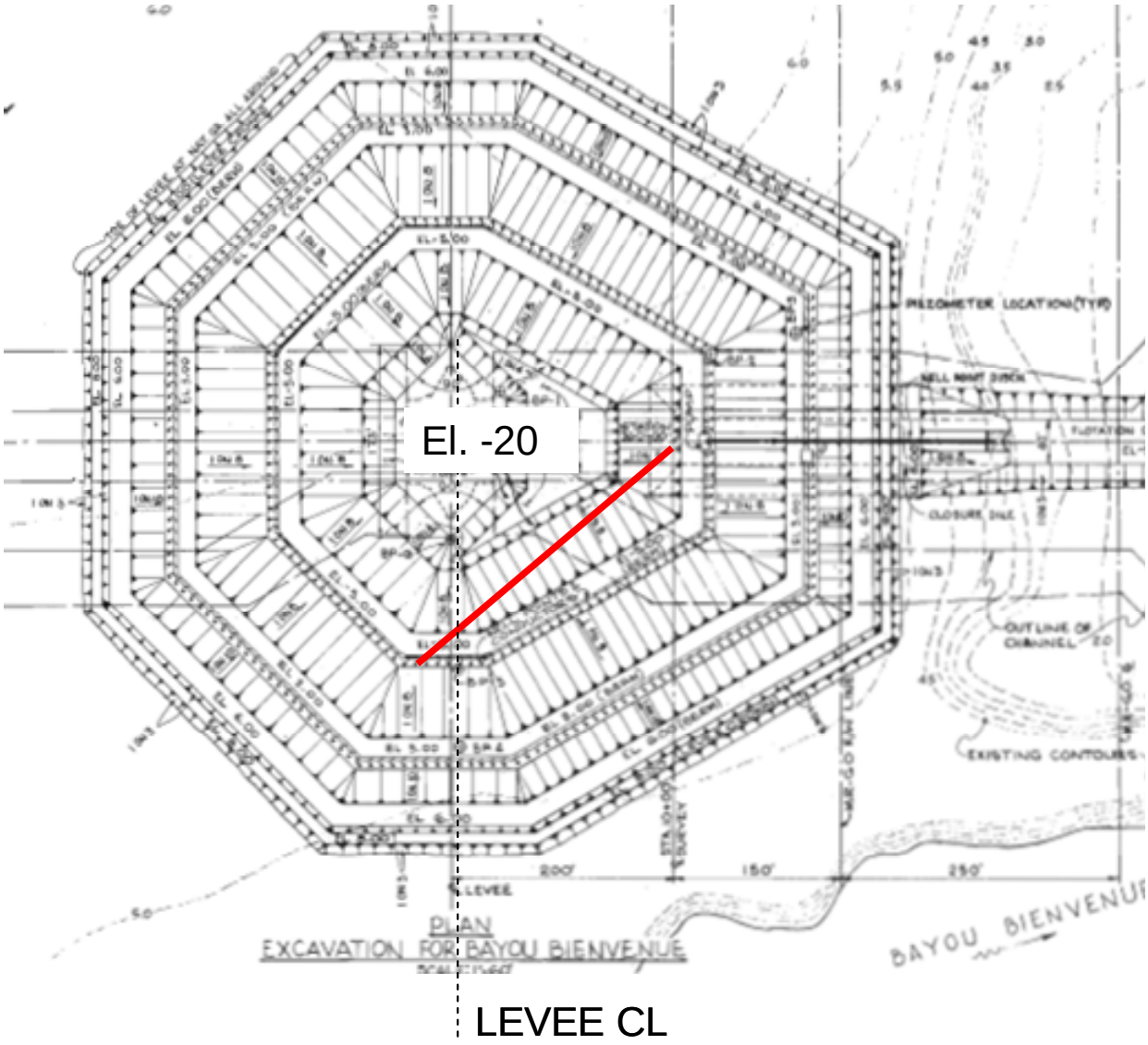


Figure 3-4a. Excavation at Bayou Bienvenue. The red line shows the approximate location of the cross section for analysis. Source: DM05.

When superimposing the excavation outline shown in Figure 3-4b, it can be seen that the line falls below the bottom of the sheet pile (elevation -5.2 feet). This leaves a small "window" of permeable material, of about 5 feet in thickness for underseepage to act adversely. The analyses cross sections in this report will be drawn through this feature (Figure 3-5).

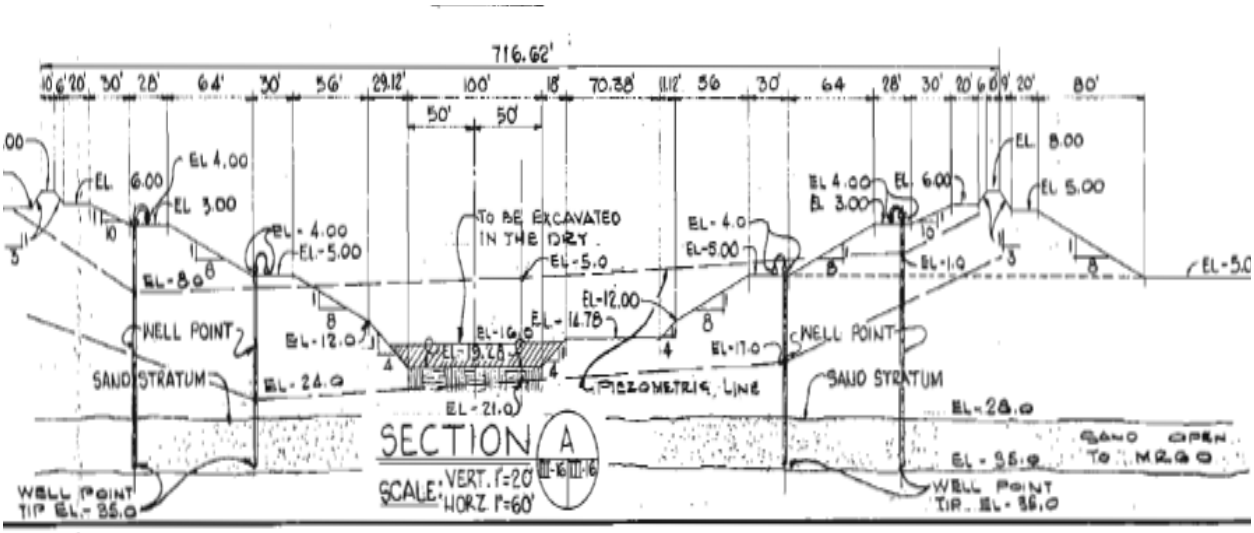


Figure 3-4b. Excavation outline for Bayou Bienvenue. Source: DM03, Plate IV-13.

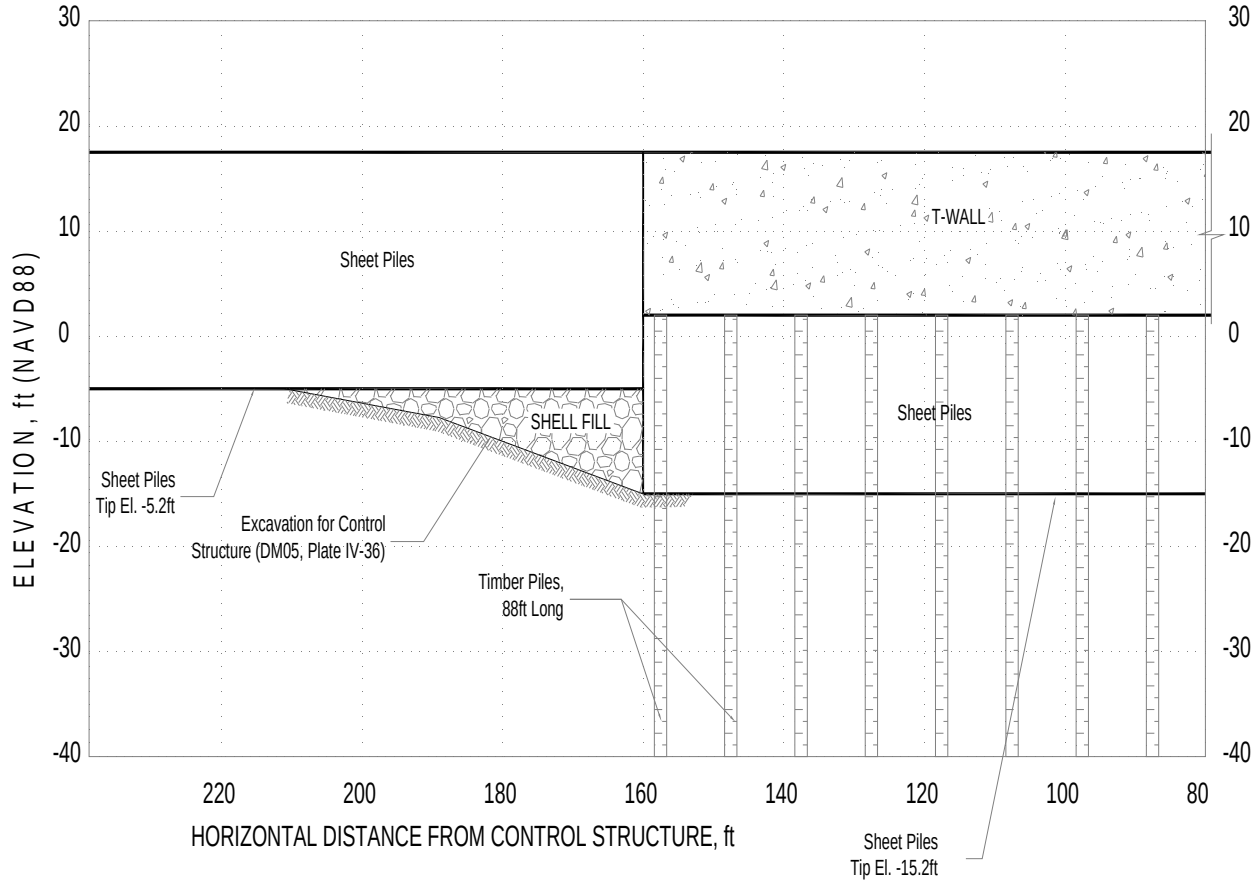


Figure 3-5. Superimposed sections showing gap between shell materials and bottom of sheet pile.

Figures 3-6 and 3-7 show extents and characteristics of the failures on Bayou Bienvenue.



Figure 3-6. Oblique view of the failure at Bayou Bienvenue (ILIT 2006).



Figure 3-7. Oblique view of the failure at Bayou Bienvenue. Note the shorter steel sheet piles bent and separated from the T-Wall (ILIT 2006).

3.2 Bayou Dupre

The control structure at Bayou Dupre showed a smaller breach, this time, to the north of the control structure. Figure 3-8 shows pre and post-Katrina conditions of the structure and adjacent EBSB, where the main scour hole is located on the waterside of the T-Wall, and the soils near the EBSB/wall interface were washed away, as well as a segment of T-wall panels.

The scour pattern indicates that erosion may have started in the interface soil/concrete, accentuated by the 2.0 foot elevation difference between them (see Lidar cross-sections in Section 2.4).

Other erosion induced damages to the EBSBs near the Dupre control structure include several hundred feet of landside and waterside erosion in what appears to be granular materials (deposited lighter soils in Figure 3-9). The series of photos in Figures 3-9 through 3-11 show the extents and characteristics of the failure on this site.

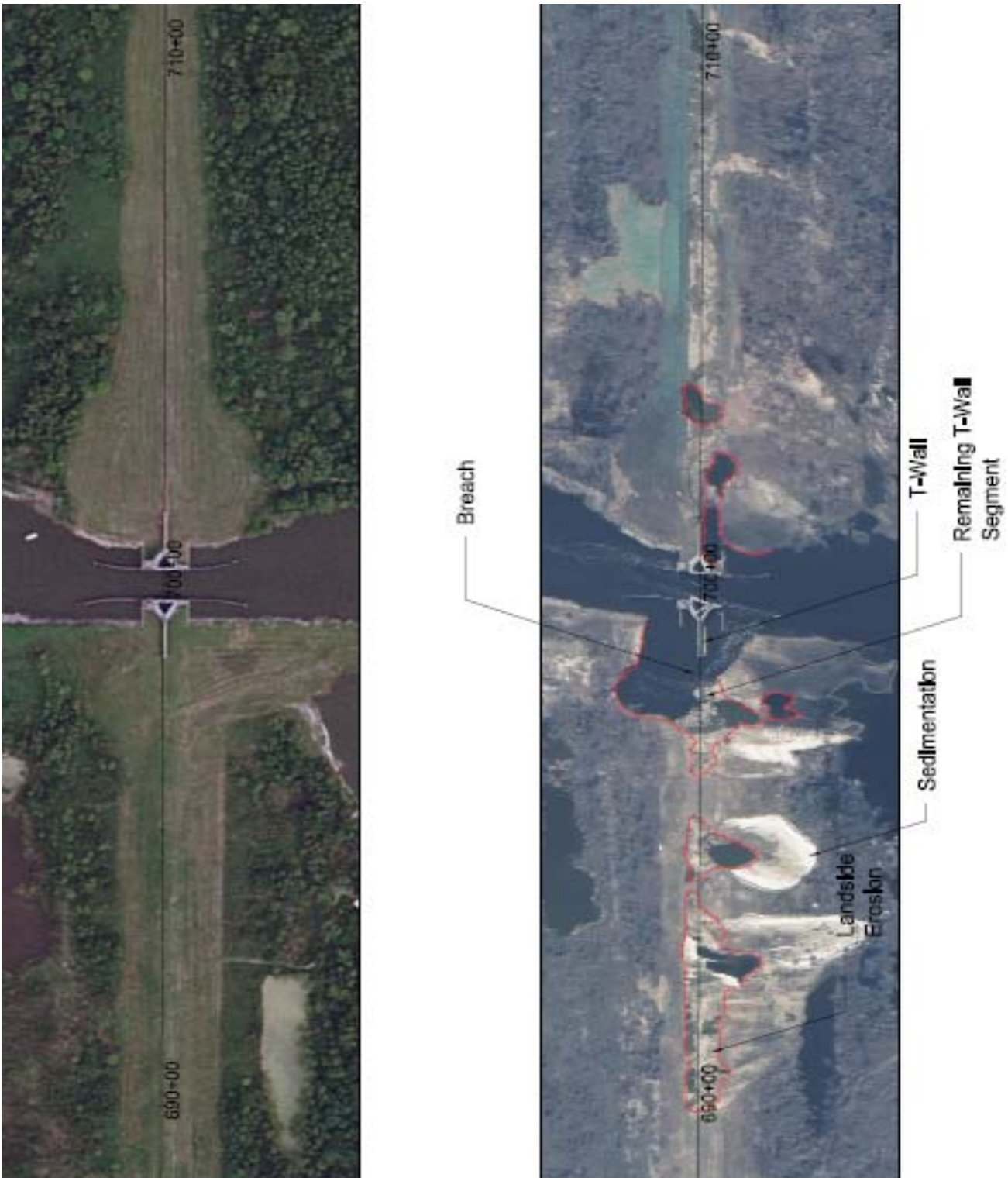


Figure 3-8 Pre and Post-Katrina conditions in Bayou Dupre.



Figure 3-9. Breach at Bayou Dupre. The main erosion site is located north of the structure, where a concrete panel is missing and the interface material between EBSB and concrete sheet pile was washed away (ILIT 2006).



Figure 3-10. Close-up near the main scour hole. Note the shell material near the concrete piles and in front of the man walking (ILIT 2006).



Figure 3-11. Detail of the EBSB material at the north end of the breach ("intact" EBSB begins). Note the clear horizon between shell materials (top) and intermixed shells with finer soils below (ILIT 2006).

Figure 3-12 shows the approximate location of the cross sections used in the analysis for this section over the design drawing from DM05.

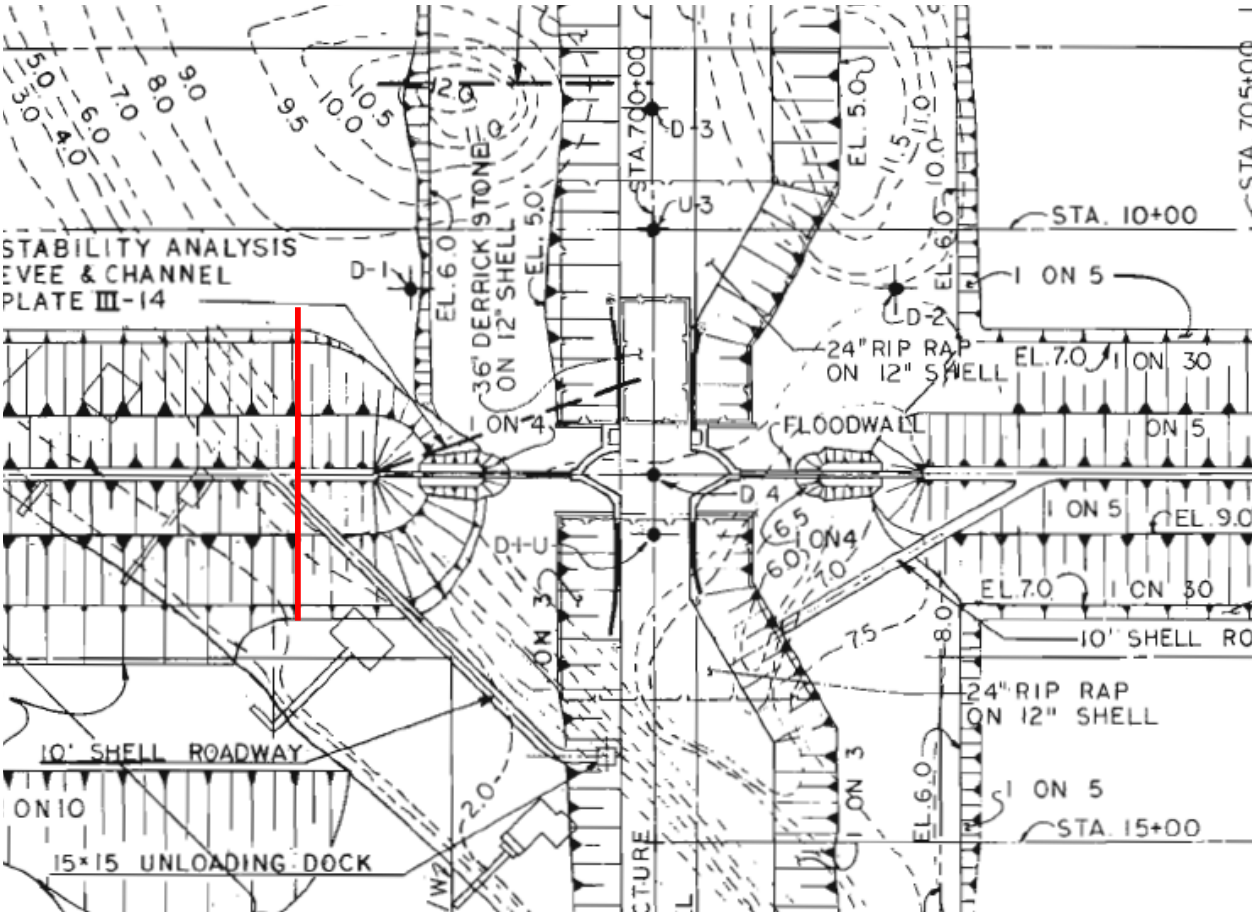


Figure 3-12. Approximate location of cross section through breach area at Bayou Dupre. Source: DM05 (USACE, 1968).

Figure 3-13 shows the design excavation from DM05 (USACE, 1968). From this plan view it can be seen that the excavation near the breach location is not as deep as the one at Bayou Bienvenue, and that the sheet piles tipped at Elevation –12 feet completely cut off the shell materials that compose the excavation fill. Figure 3-14 shows the interpreted longitudinal section near the breach.

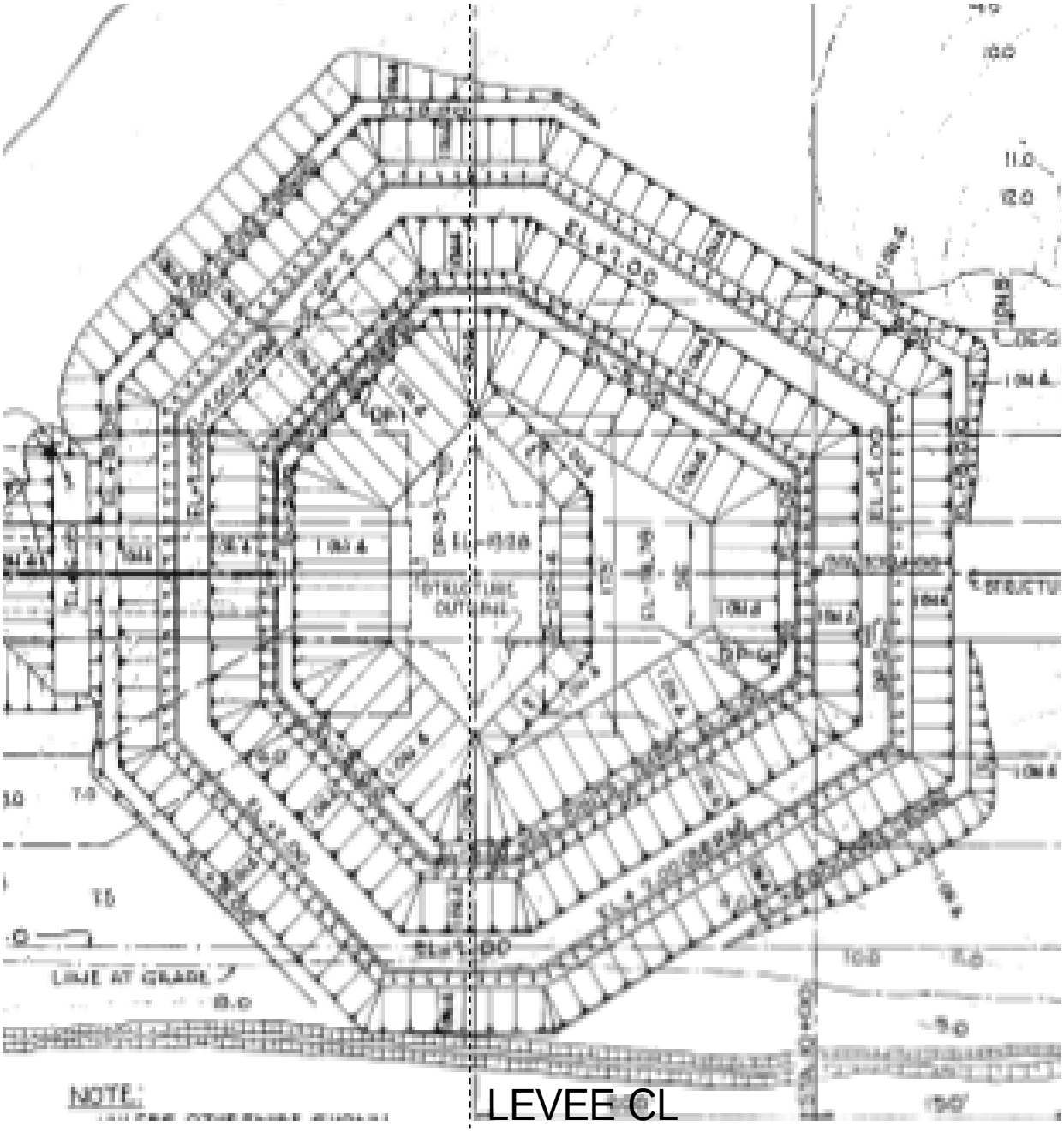


Figure 3-13. Plan view of design excavation on Bayou Dupre. Source: DM05 (USACE, 1968).

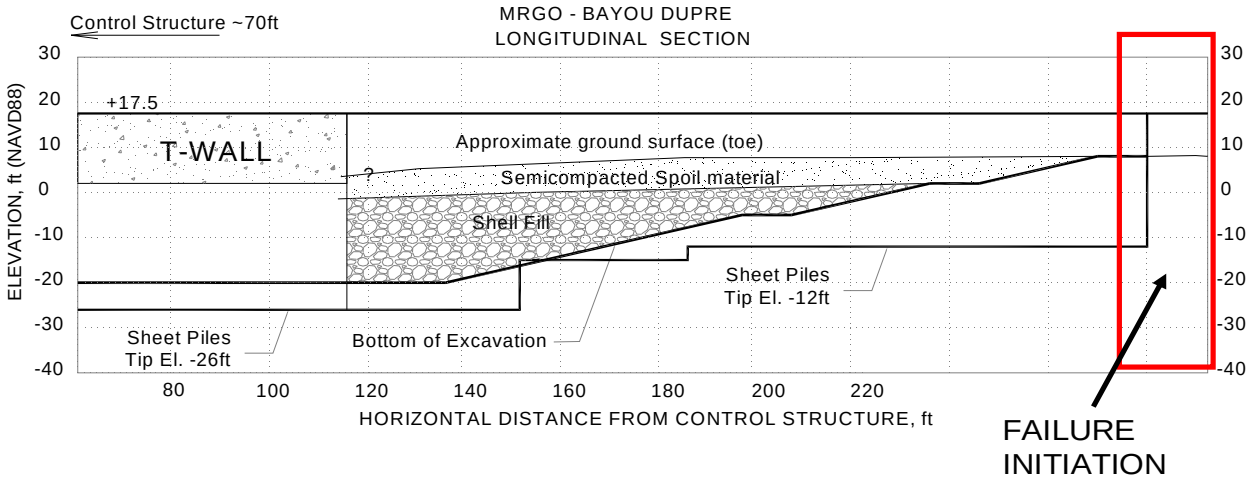


Figure 3-14. Longitudinal section through levee crest at Bayou Dupre.

3.3 Surge Hydrograph

The hurricane Katrina surge hydrographs (water elevations versus time) in Figure 3-15 were used as the input information for the seepage analyses to be discussed in subsequent parts of this report (van Heerden 2007).

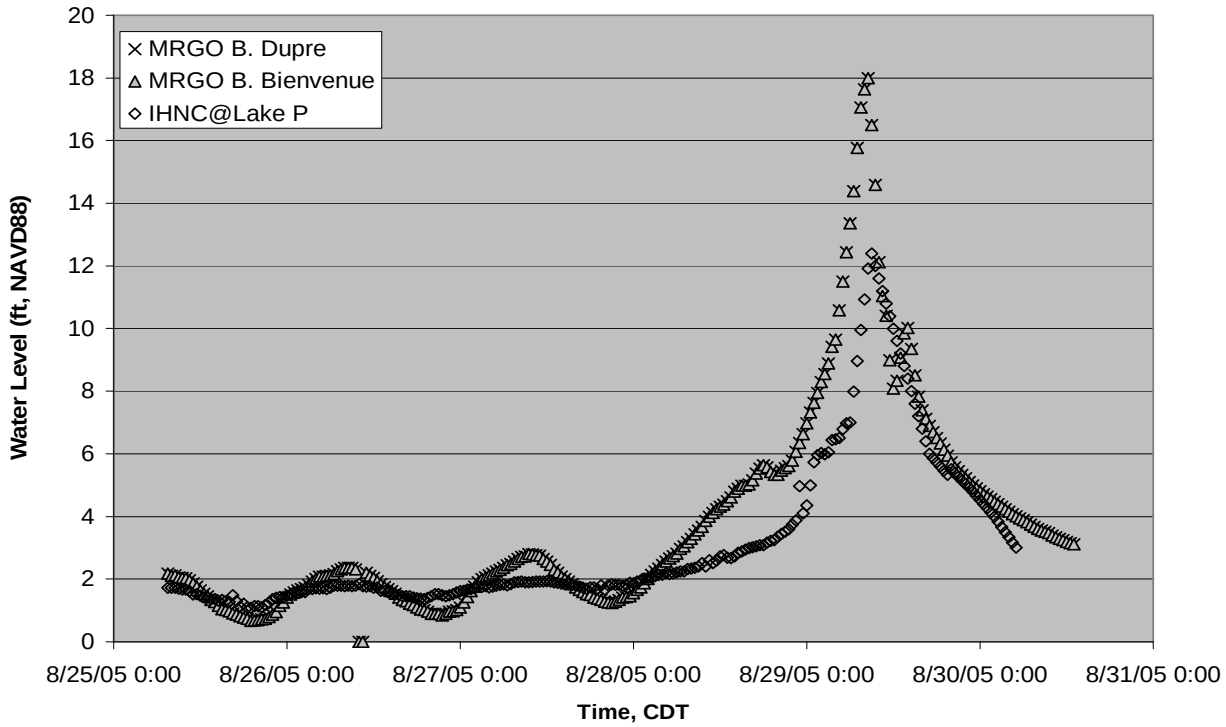


Figure 3-15. Hurricane Katrina surge hydrographs at Bayous Dupre and Bievenue.

3.4 DESIGN CROSS SECTIONS vs LIDAR CROSS SECTIONS

3.4.1 Bayou Bienvenue

Figure 3-16 shows a plan view of the LIDAR survey performed in 2000. The data from this figure was used to produce the profile in Figure 3-17, where the elevation difference between sheet pile and earthen EBSB is shown.

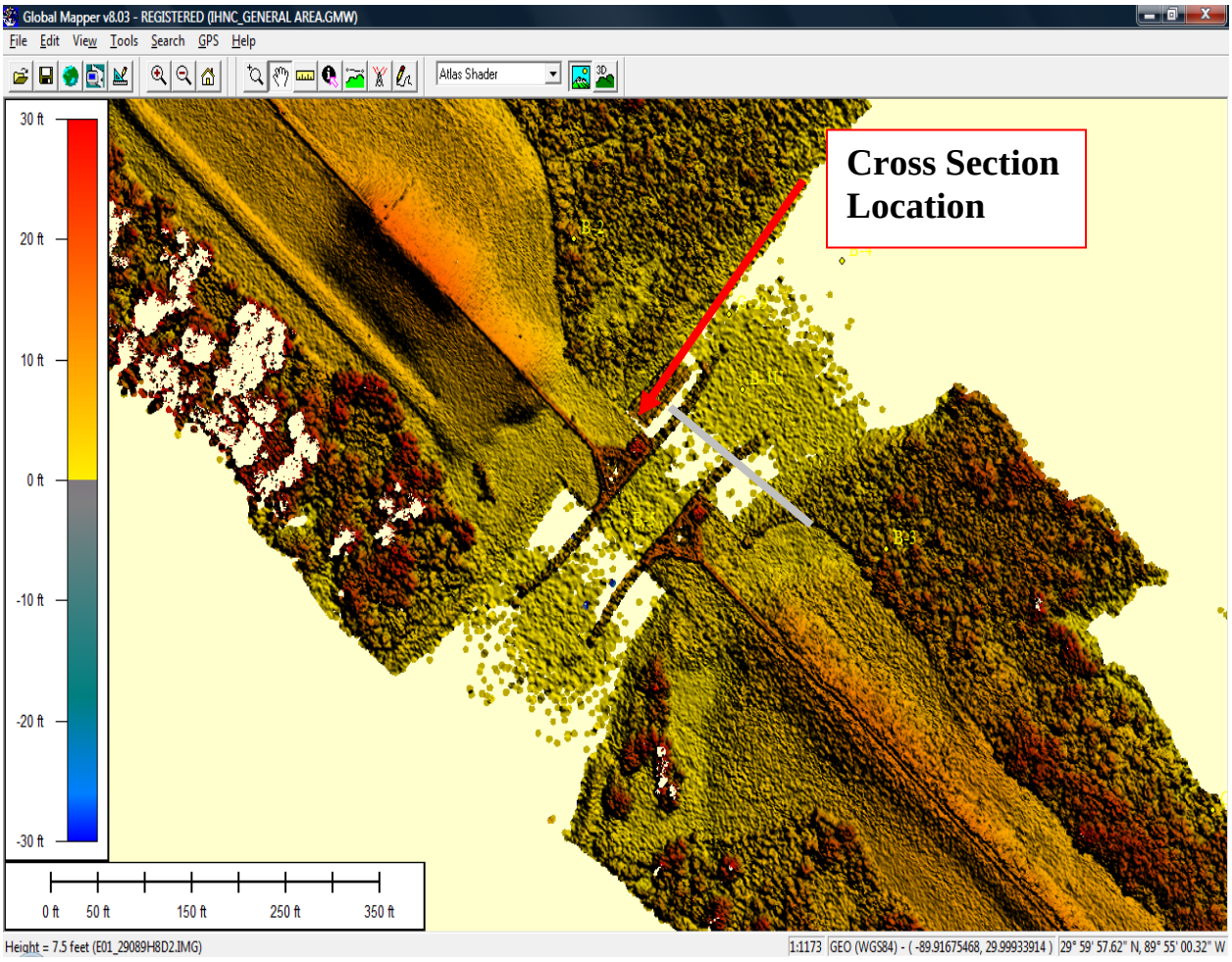


Figure 3-16. Pre-Katrina LIDAR survey information.

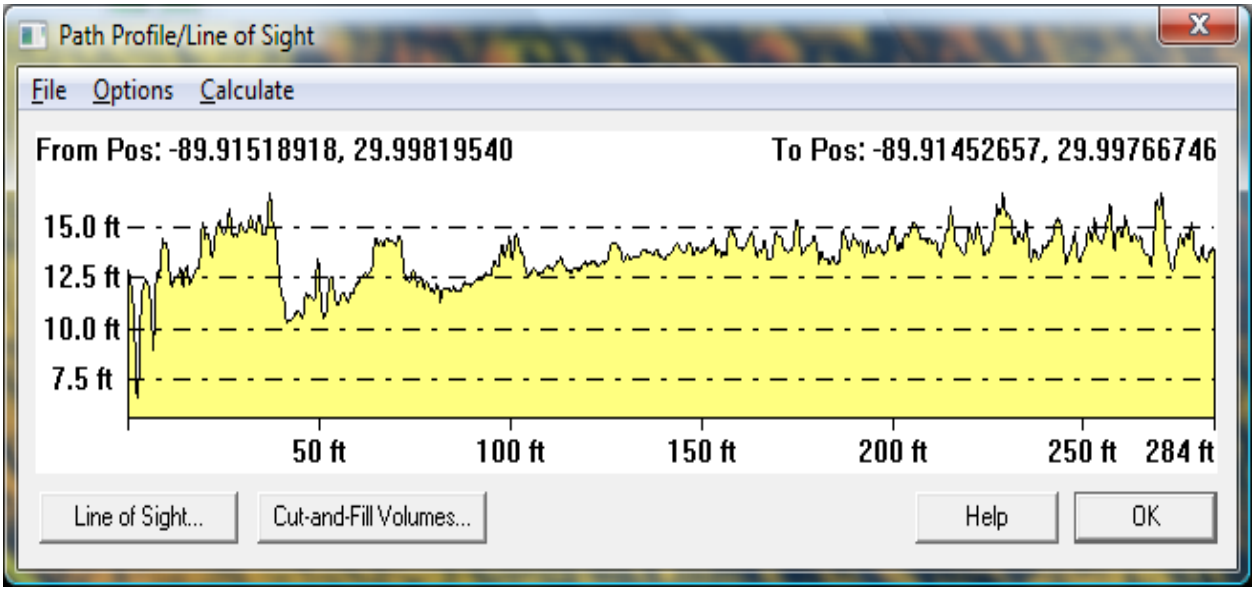


Figure 3-17. Longitudinal section along T-wall and steel sheet tops. The T-Wall (0 to 50 feet in horizontal scale) seems to be more than 1 foot higher than the transition to the steel sheet pile.

Figure 3-18 shows the design cross section from DM03 compared with the pre-Katrina LIDAR cross section at the site of the breach.

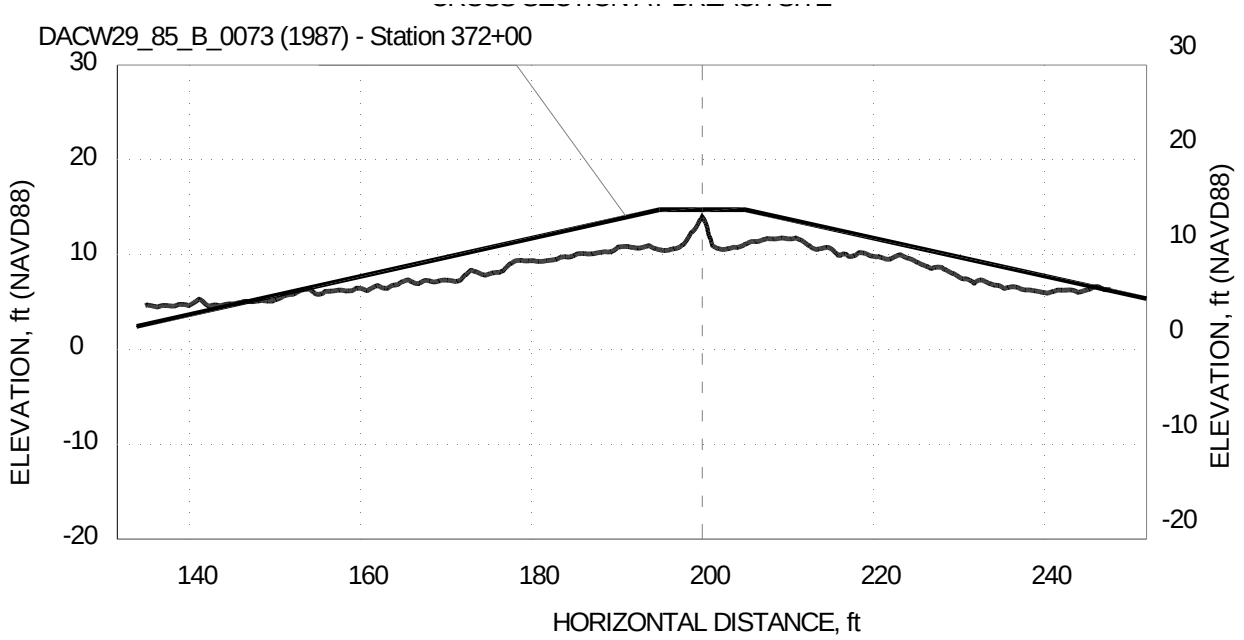


Figure 3-18. Cross section through breach site comparing design and pre-Katrina elevations.

3.4.2 Bayou Dupre

The 2000 survey data was used in the vicinity of the Bayou Dupre control structure. This time, the profile is located north of the structure and shows that the earthen EBSB is higher than the concrete sheet piles by more than 1 foot. Figures 3-19 through 3-21 show the LiDAR survey information.

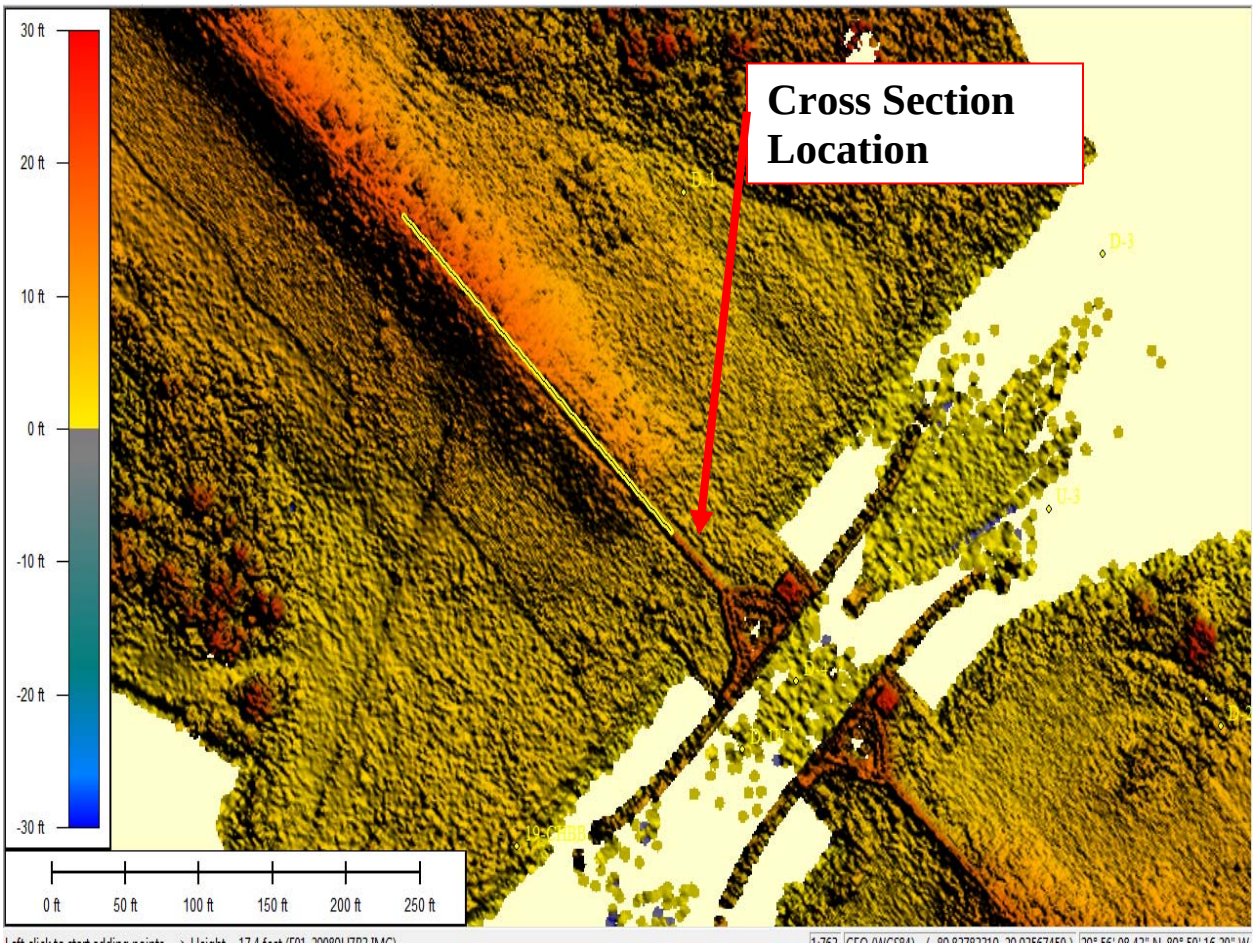


Figure 3-19. Pre-Katrina LIDAR survey information.

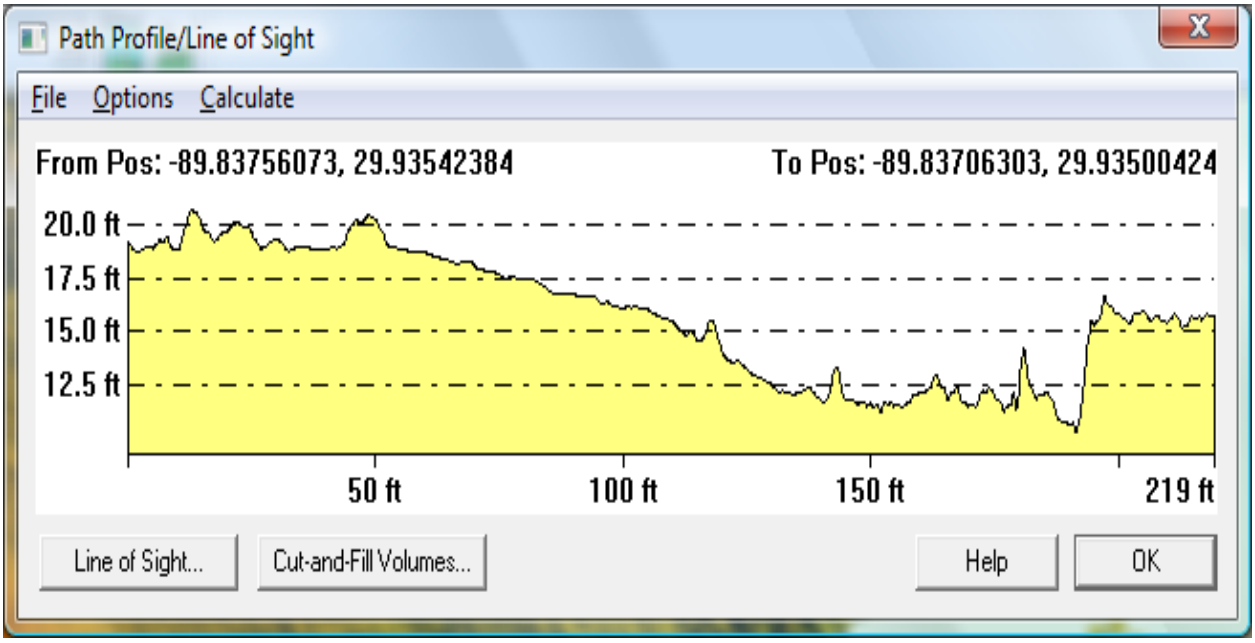


Figure 3-20. Pre-Katrina LIDAR and longitudinal section along Bayou Dupre. The T-wall (from 200ft in horizontal distance) is located at elevation 15ft, and the connecting EBSB is located above 18ft.

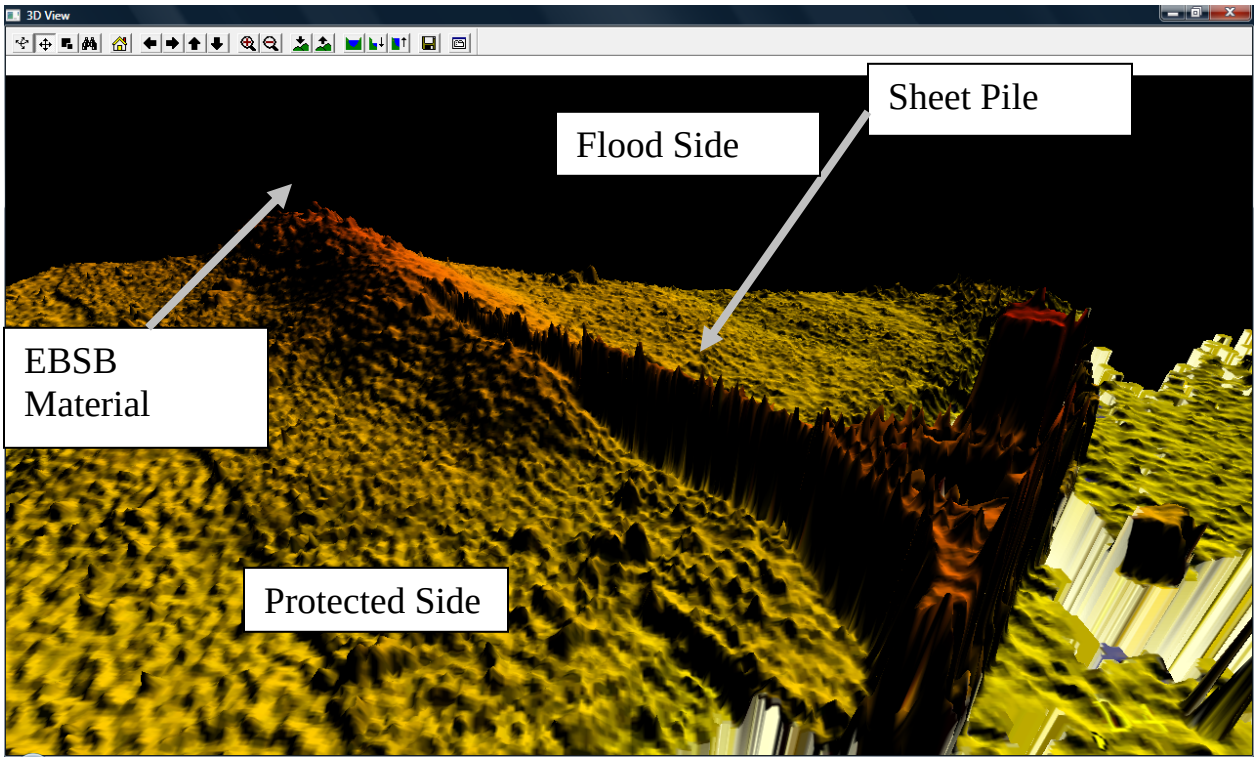


Figure 3-21. Three dimensional view of pre-storm conditions at Bayou Dupre. Note the elevation difference between the EBSB and concrete sheet piles.

Figure 3-22 shows the design cross section compared to the LIDAR section drawn from the available data. Figure 3-23 shows a three dimensional view of the Pre-Katrina LIDAR, the difference between the top of sheet pile and the earthen embankment is approximately 3feet.

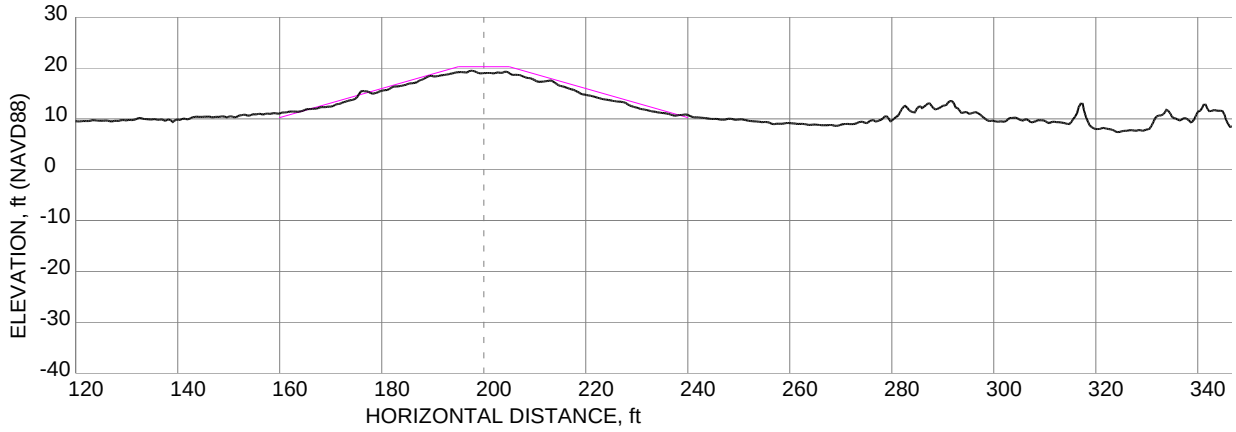


Figure 3-22. Design and LIDAR cross sections on Bayou Dupre near breach site.

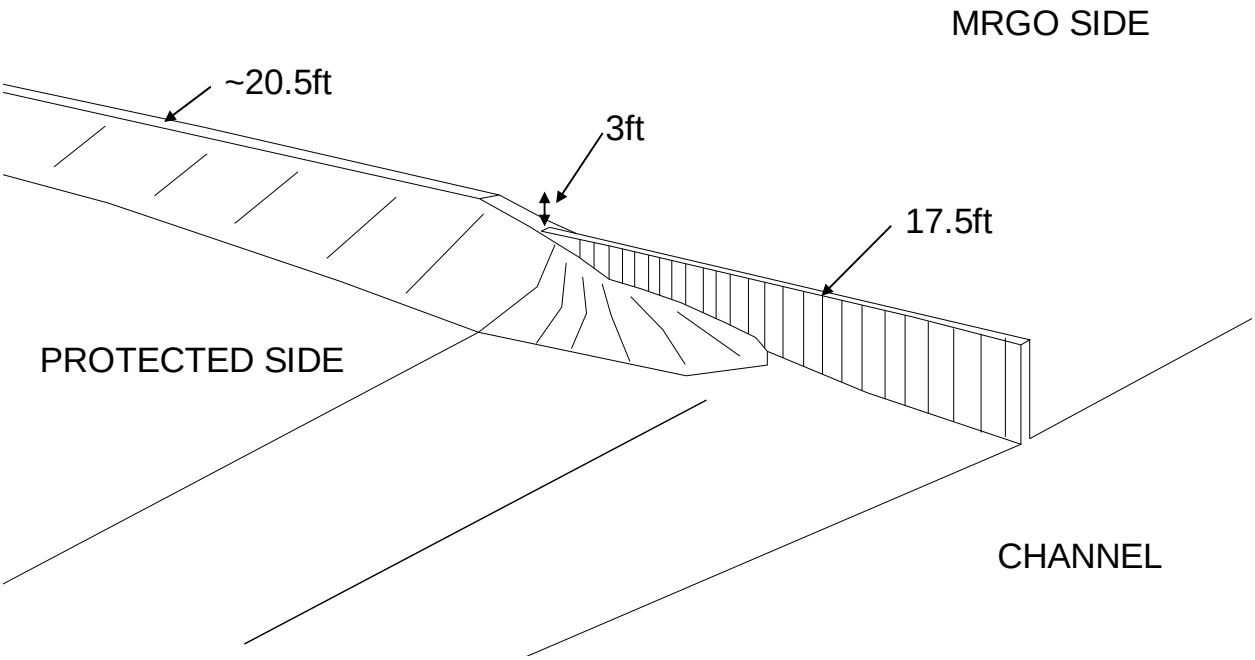


Figure 3-23. Three dimensional view of Bayou Dupre at the breach site. Note the 3foot difference between the concrete sheet piles and the earthen embankment.

4. STRATIGRAPHIC INTERPRETATIONS

Available borings and CPT probes, the locations of explorations were geo-referenced using a GIS interface. From the Pre and Post-Katrina LIDAR surveys and the DM03 ground elevations, a longitudinal profile was drawn, locating the elevations of the borings. To include the explorations into this report, the 1966 DM03 borings had to be "measured" from the image versions of this report, as well as the laboratory testing results. The 2005 IPET borings available in the IPET website (www.ipet.wes.army.mil).

Figures 4-1 and 4-2 show the complete longitudinal profile in a "profile/plan" format for Bayou Bienvenue and Bayou Dupre containing a pre-Katrina map. The stratigraphic interpretation is shown, depicting the main soil units in the area as 1) Surficial Clays, 2) Marsh deposits, 3) Deeper "Grey" clays, 4) Sands (relict beach deposits) and 5) Shell fills.

The stratigraphic interpretation clearly shows that the embankment fills that compose the EBSBs are mainly composed by clays (CL, CH), and an irregular Marsh (peat) layer underlies the EBSB. The Marsh layer appears to be continuous along the area. The Marsh is underlain by clays and silts down to elevation -30ft (NAVD88), where a thin sand layer occurs. These sands are, in turn, underlain by another clay deposit.

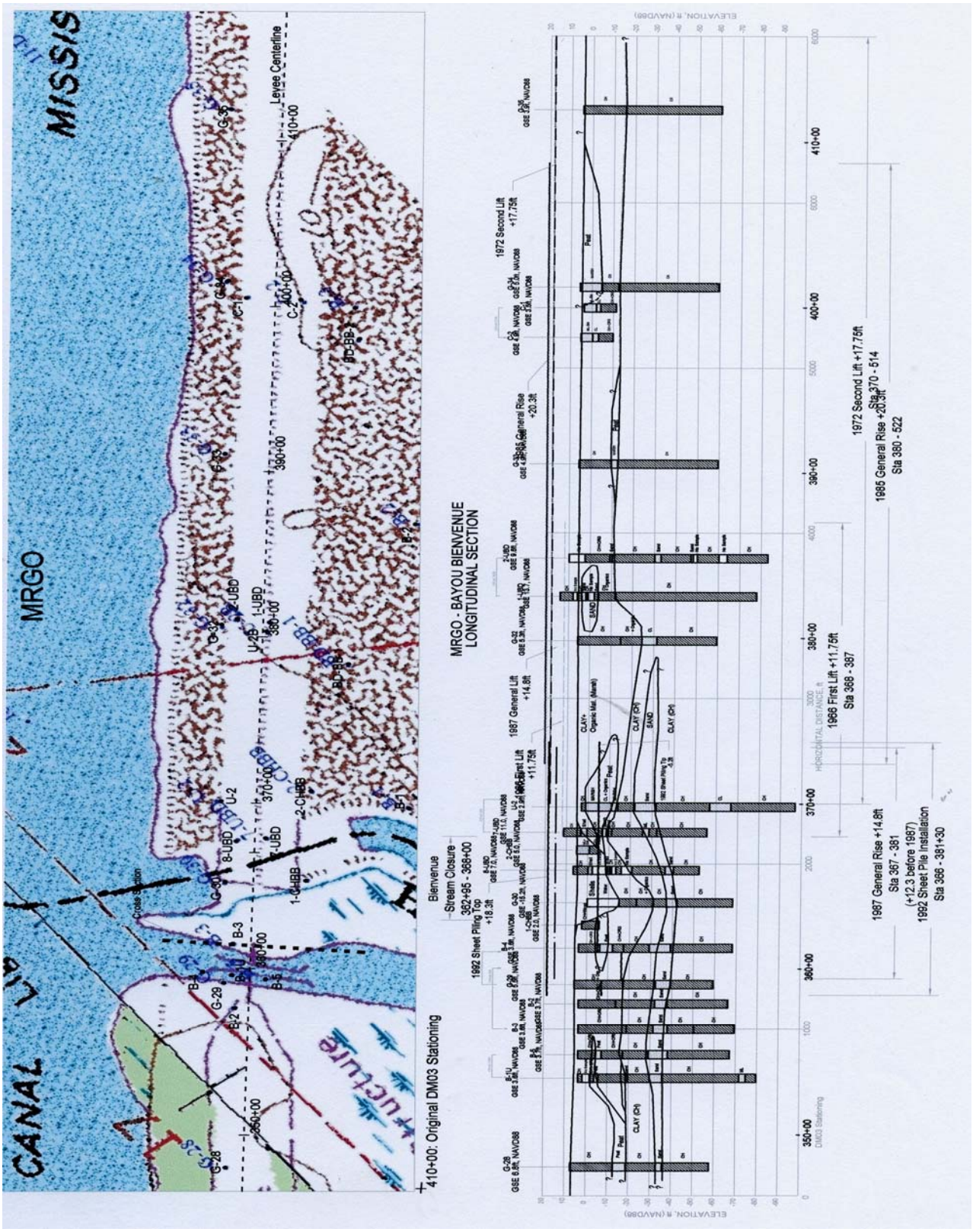


Figure 4-1. Longitudinal Profile along Bayou Bienvenue.

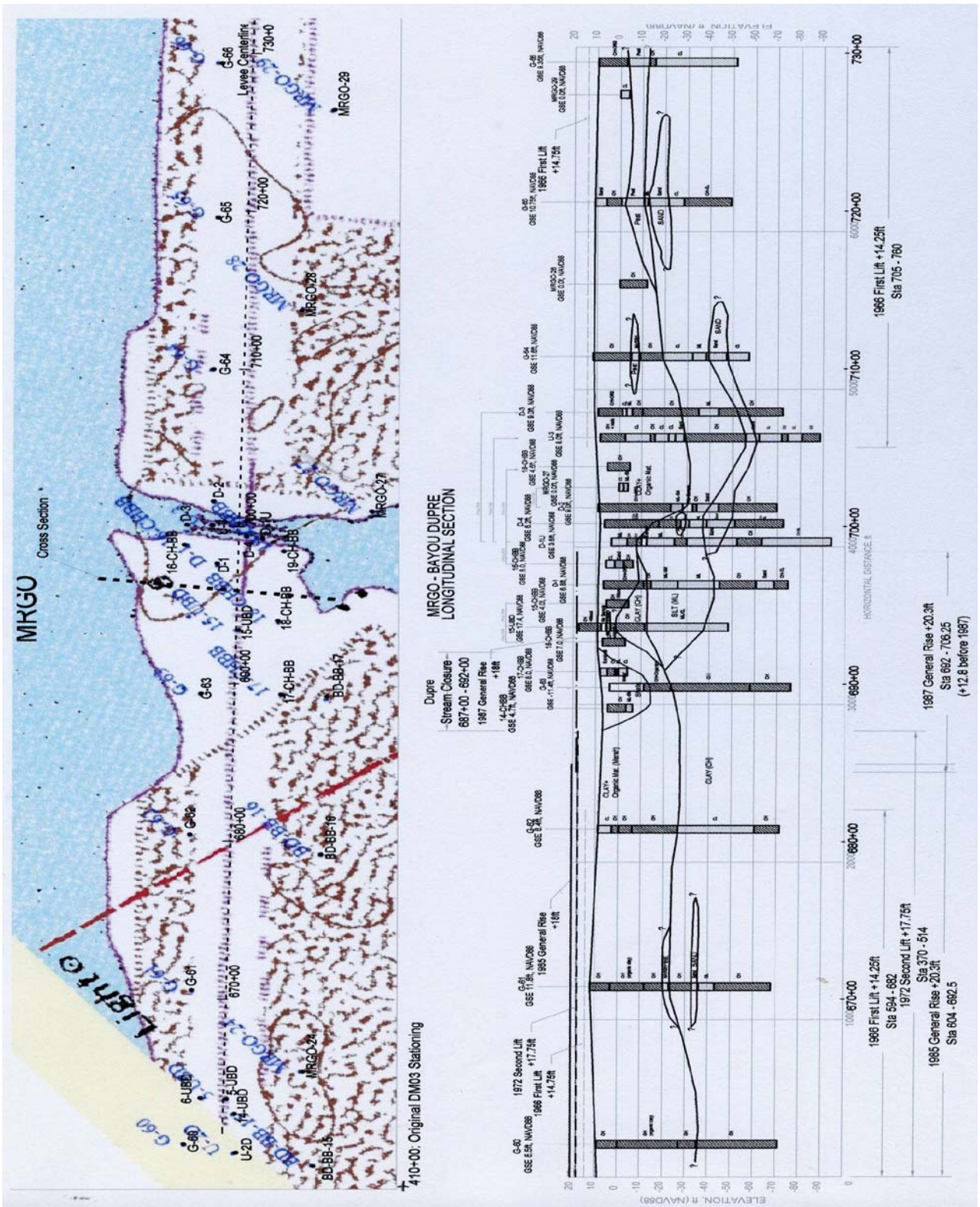


Figure 4-2. Longitudinal Profile along Bayou Dupre.

4.1 BAYOU BIENVENUE

The breach at Bayou Bienvenue is located south of the Bienvenue Control Structure (Station 360+00 of the original DM03 levee stationing).

The stratigraphic interpretation was based on borings G-29, G-30 and G-31 (1966), 1-CHBB and 2-CHBB (1987), B-1U through B-5 (2005), and U-2 (1966). The levee embankment is composed mainly by semi-compacted clays, underlain by a Marsh deposit, described as organic matter, organic clay, wood, roots and saturated-very low density material. Shell materials seem to have been placed to fill the excavations for the control structures. Given the poor descriptions presented in the borings from DM03 and the IPET (2006) report, samples that contained organic materials with high water contents, low density and low strengths were classified as Marsh samples.

Underlying the marsh deposit, a thick “fat” clay deposit occurs from elevation -30 to approximately –40 feet, located atop a sand deposit. The sands are in turn underlain by another clay and silt deposit, probably associated with Pleistocene age deposits. Figure 4-3 shows stick logs from borings near the breach site on Bayou Bienvenue. Figure 4-4 shows a close-up of the longitudinal profile created for the Bayou Bienvenue area, and Figure 4-5 shows the interpreted cross section for this area.

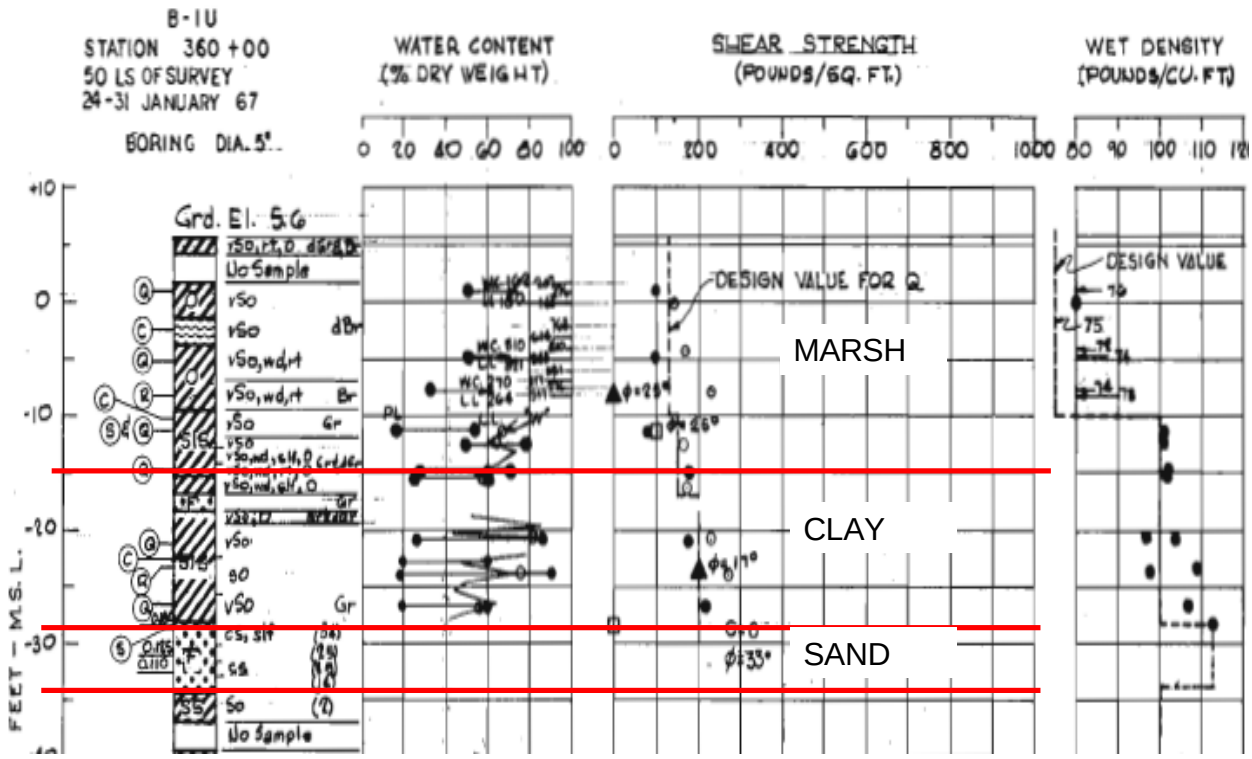


Figure 4-3. Longitudinal Profile along Station 497+00. Marsh layer is clearly shown between elevations 0 and -15ft (MSL).

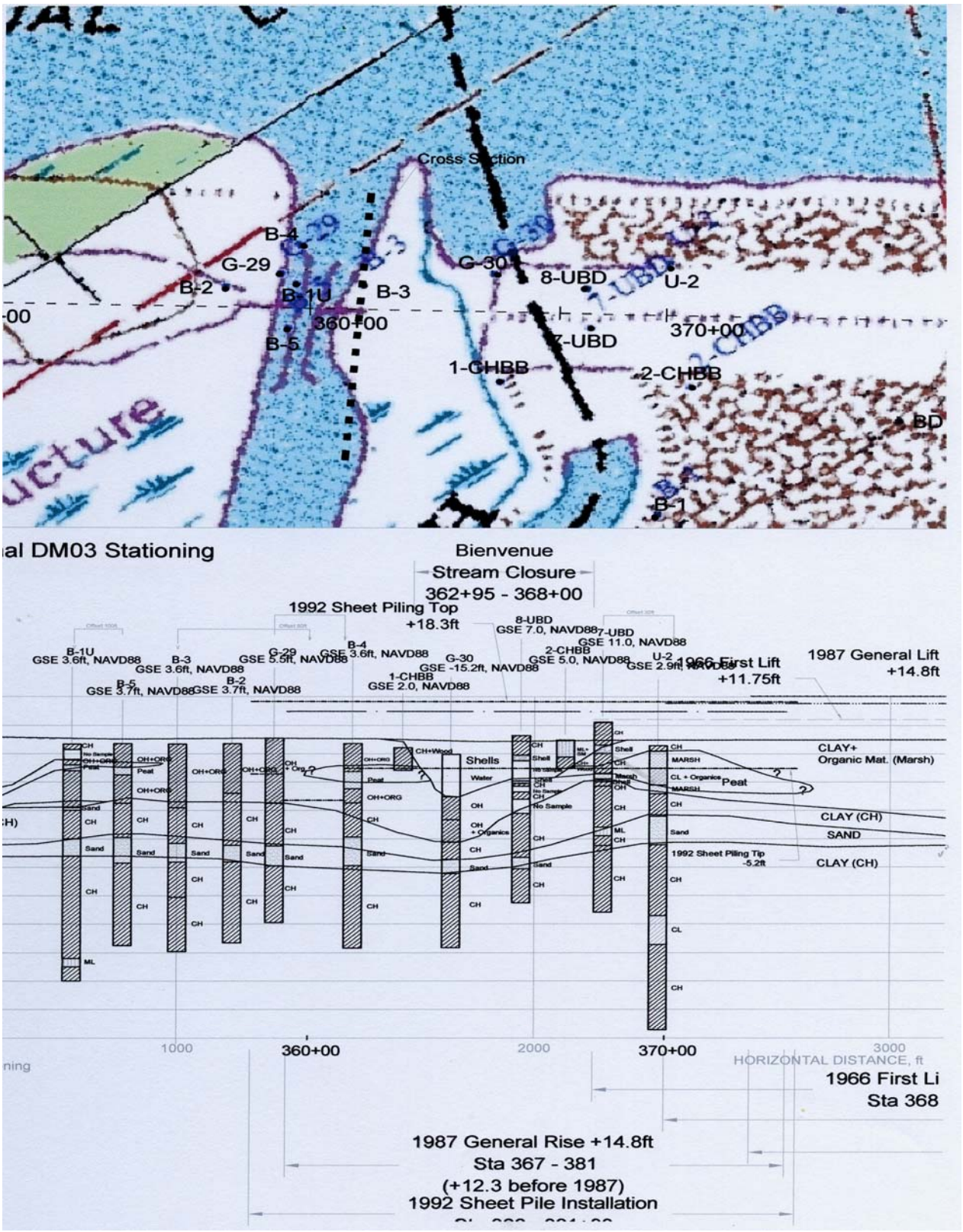


Figure 4-4. Geologic interpretation near the breach area.

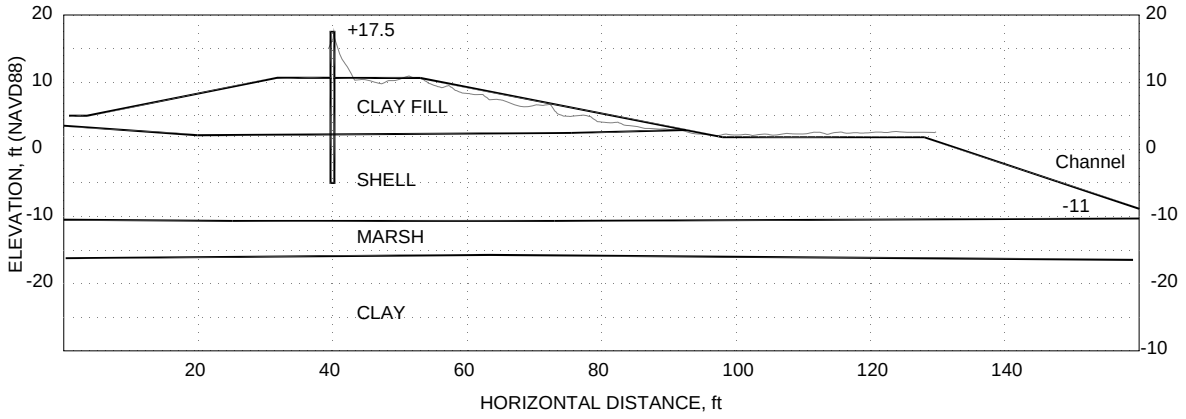


Figure 4-5. Interpreted cross section for Bayou Bienvenue.

4.2 BAYOU DUPRE

The main breach on this site took place just north of the control structure; at the interface between the concrete structure and the connecting earthen levee. The stratigraphic interpretation was based on borings G-63, G-64 (1966), 14-CHBB, 17-CHBB, 18-CHBB, 15-CHBB, 16-CHBB (1987), 15-UBD (1984), D-1 through D-4 (1967), and MRGO 27 (2005). According to the design documents, the levee embankment is composed mainly by semi-compacted clays. However, the pictures from Section 3 show that the levee was in fact composed by shells and mixed finer materials. The embankment is underlain by a Marsh deposit, described as organic matter, organic clay, wood, roots and saturated-very low density material.

Given the poor descriptions presented in the borings from DM03 and the IPET (2006) report, samples that contained organic materials with high water contents, low density and low strengths were classified as Marsh samples. Figure 4-6 shows the only available crest boring near the area of the breach at Bayou Dupre. Figure 4-7 shows close-up of the longitudinal profile created for

the Bayou Dupre area, and Figure 4-8 shows the interpreted cross section for the breach on this site.

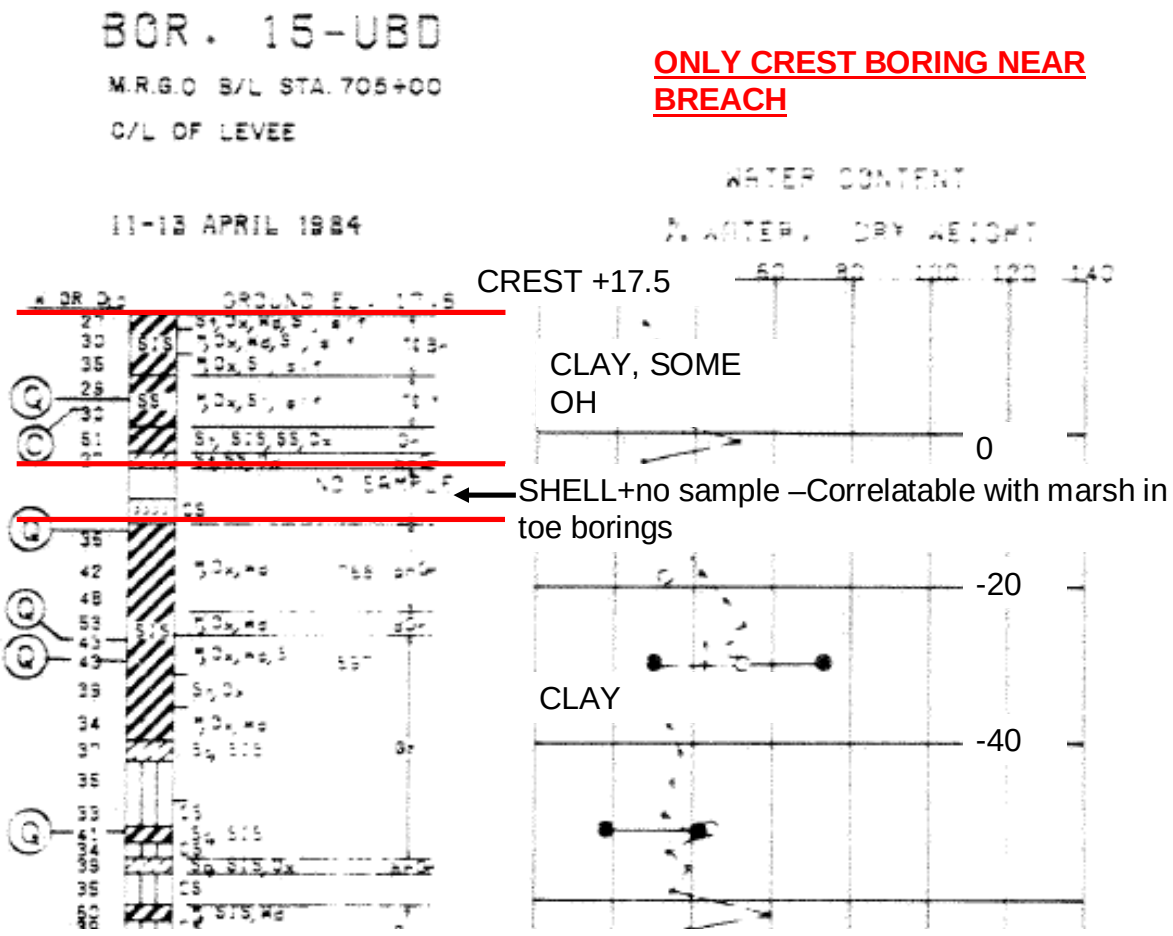


Figure 4-6. Available crest boring at Bayou Dupre.

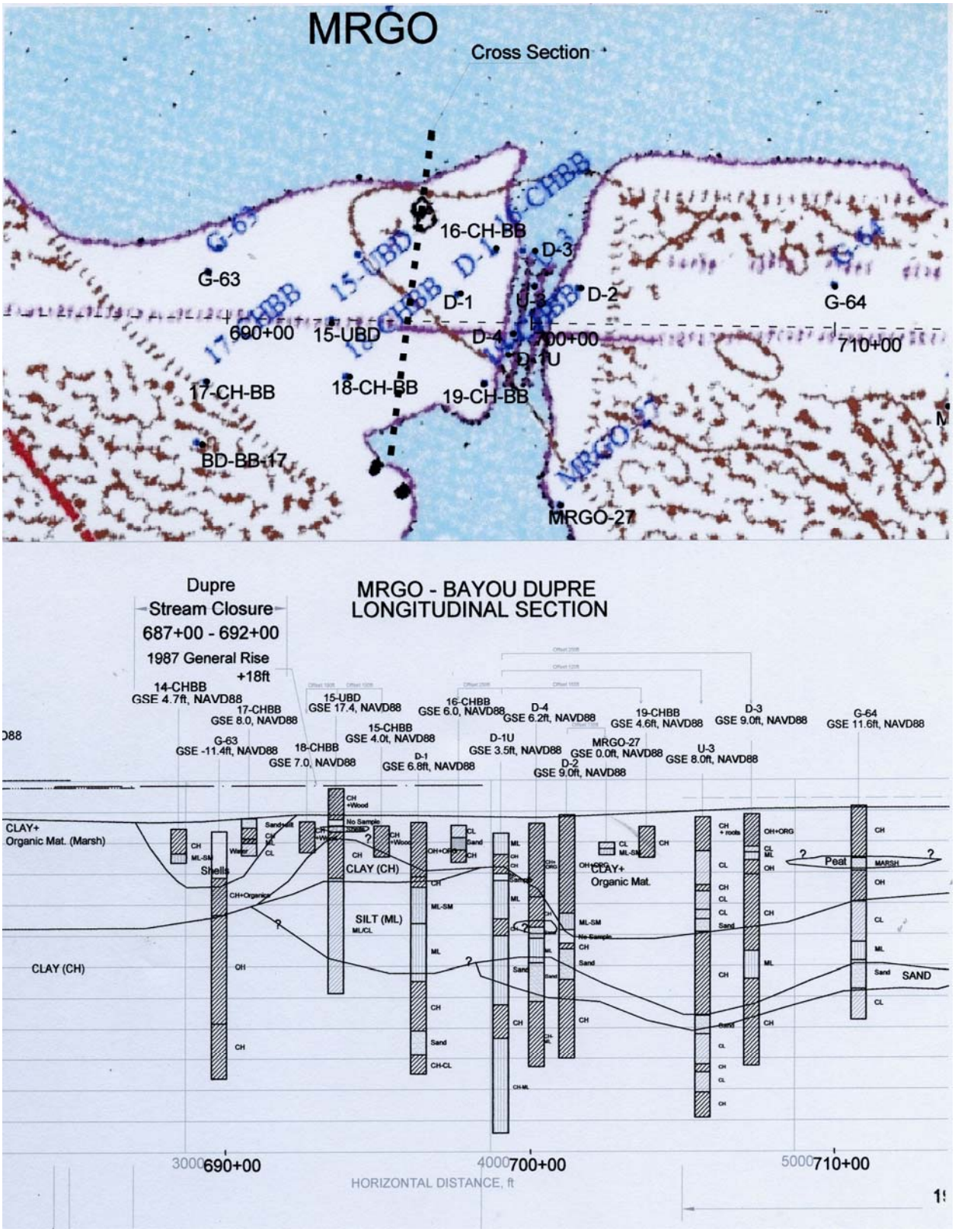


Figure 4-7. Geologic interpretation near the breach area.

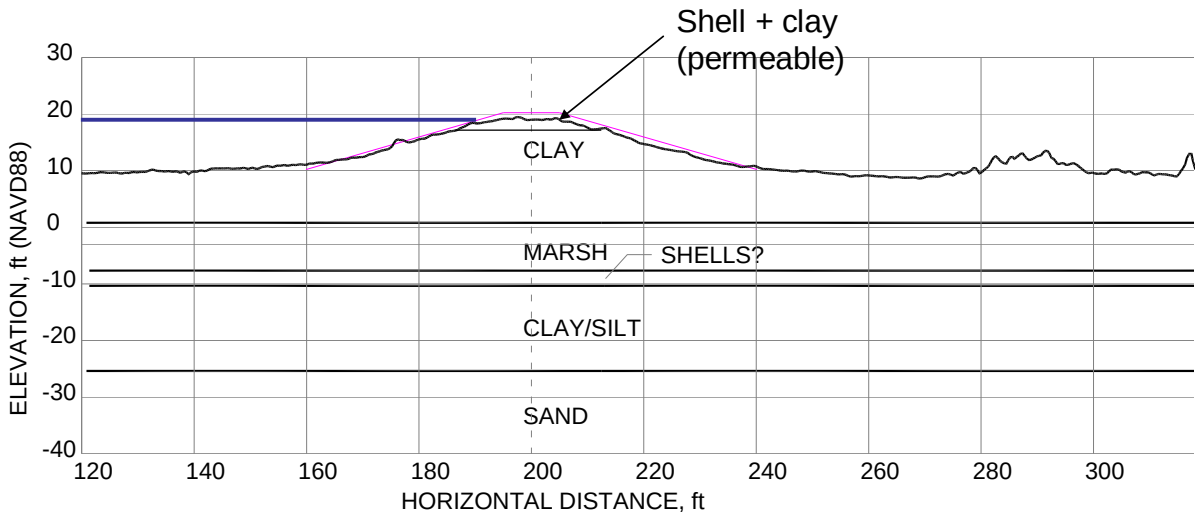


Figure 4-8. Cross section for Bayou Dupre.

5. SOILS DATA REVIEW

The information available for the interpretation and analysis process of the breaches on MR-GO was obtained from the following reports:

- DM03, Chalmette Area Plan (1966)
- DM05, Chalmette Area Plan – Bayou Bienvenue and Bayou Dupre (1968)
- IPET (2006) report, Volumes II and V (and appendices)
- ILIT (2006) report, Chapter 9

The boring data was classified by year and performing agency, as shown in Table 5-1. This table summarizes the available exploration near the breaches, and gives information on location, coordinates, approximate elevation and type of boring. Table 5-2 shows the same list of borings after the pre and post Katrina elevations from the LIDAR surveys were compared to the "design"

top of boring elevation (what was shown on the boring log). The right column shows the final elevation chosen for each boring incorporated in the models.

This section is intended to summarize for the reader on the available exploration information on the breach sites on MR-GO. A total of 80 borings were used in the interpretations and analyses.

Document	Boring Year	Datum	Boring ID	Boring Type	Top Elevation	Bottom Elevation	Boring Length	TOP EL. in NAVD88	Latitude	Longitude	Notes	Available Lab Tests?
BAYOU BIENVENUE												
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1966	MSL	U-2	5in Piston "undisturbed"	3.1	-96	99.1	2.85	29.996893	-89.912569		yes
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1965	MSL	G-28	3" ID Cove Barrel	7	-53	60.0	6.75	30.000884	-89.917918		no
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1965	MSL	G-29	3" ID Cove Barrel	6	-54	60.0	5.75	29.998849	-89.915237		no
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1965	MSL	G-30	3" ID Cove Barrel	-13	-63	50.0	-13.25	29.997726	-89.913830		no
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+01	1966	MSL	G-32	3" ID Cove Barrel	3	-57	60	2.75	29.994895	-89.910094		no
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1965	MSL	G-33	3" ID Cove Barrel	3	-57	60	2.75	29.992943	-89.907619		no
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1965	MSL	G-34	3" ID Cove Barrel	3	-57	60	2.75	29.990992	-89.905121		no
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1965	MSL	G-35	3" ID Cove Barrel	4	-36	40	3.75	29.989041	-89.902670		no
Mississippi River Gulf Outlet Design Memorandum No. 1-B, Channels, Mile 39.0	1956	MSL	10-D	1_7/8" Wire core barrel	2.78	-58	60.8	2.53	29.990491	-89.900469	Only available final 10ft o	no
Hurricane Protection Levee Second Enlargement MRGO B/L Sta 380+50 to Sta	1984	NGVD	1-UBD	5in Piston "undisturbed"	13.9	-77	90.9	11.9	29.994316	-89.910444		no
Hurricane Protection Levee Second Enlargement MRGO B/L Sta 380+50 to Sta	1984	NGVD	2-UBD	5in Piston "undisturbed"	9.8	-82	91.8	7.8	29.994659	-89.910120		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1984	NGVD	7-UBD	5in Piston "undisturbed"	13	-53	66	11	29.996776	-89.913537		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1984	NGVD	8-UBD	5in Piston "undisturbed"	9	-50	59	7	29.997110	-89.913299		no
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	B-2	n/a	5.70	-63.00	68.70	3.7	29.999020	-89.915686		no
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	B-1U	5in Piston "undisturbed"	5.60	-80.00	85.60	3.6	29.998680	-89.915204		yes
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	B-3	n/a	5.60	-65.00	70.60	3.6	29.998324	-89.914776		no
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1966	NGVD	B-4	n/a	5.60	-65.00	70.60	3.6	29.998965	-89.914913		no
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	B-5	n/a	5.70	-63.00	68.70	3.7	29.998356	-89.915538		no
Hurricane Protection Levee, Chalmette Area Plan, Sta 370+00 to Sta 682+00	1971	MSL	U-2B	5in Piston "undisturbed"	9	-70	79	8.75	29.994466	-89.910570		yes
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1987	NGVD	1-CHBB	1_7/8 ID Sampler	4	-4	8	2	29.996791	-89.914479		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1988	NGVD	2-CHBB	1_7/8 ID Sampler	7	-4	11	5	29.995744	-89.913242		no
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan, Hurricane Prote	2001	NGVD	BD-BB-1	1_7/8 ID Sampler	0	-7	7	-2	29.993930	-89.911730		no
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan, Hurricane Prote	2002	NGVD	BD-BB-2	1_7/8 ID Sampler	0	-10	10	-2	29.990180	-89.907100		no
Hurricane Protection Levee Second Enlargement MRGO B/L Sta 380+50 to Sta	1976	NGVD	C-1	n/a	3.7	-11	14.7	1.7	29.990947	-89.905533		no
Hurricane Protection Levee Second Enlargement MRGO B/L Sta 380+50 to Sta	1976	NGVD	C-2	n/a	5.1	-10	15.1	3.1	29.990357	-89.906085		no
Vicinity Chalmette Area Plan, Hurricane Protection Levee, Emergency Restorat	2005	NGVD	B-1	1_7/8 ID Sampler	0	-25	25	-2	29.994850	-89.914300		no
Vicinity Chalmette Area Plan, Hurricane Protection Levee, Emergency Restorat	2005	NGVD	B-2	1_7/8 ID Sampler	0	-25	25	-2	29.991580	-89.910320		no
Vicinity Chalmette Area Plan, Hurricane Protection Levee, Emergency Restorat	2005	NGVD	B-3	1_7/8 ID Sampler	0	-25	25	-2	29.989850	-89.906600		no

BAYOU DUPRE												
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-62	3" ID Cove Barrel	12	-48	60	11.75	29.940762	-89.843279		no
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-63	3" ID Cove Barrel	9	-51	60	8.75	29.938930	-89.840900		no
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-64	3" ID Cove Barrel	12	-48	60	11.75	29.933195	-89.833999		no
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-65	3" ID Cove Barrel	11	-49	60	10.75	29.931339	-89.831739		no
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-66	3" ID Cove Barrel	9.6	-50	59.6	9.35	29.929531	-89.829407		no
Mississippi River Gulf Outlet Design Memorandum No. 1-B, Channels, Mile 39.0	1956	MSL	17-D	1_7/8" Wire core barrel	4.78	-56	60.78	4.53	29.938072	-89.835842	Only available final 10ft of	no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1987	NGVD	14-CHBB	1_7/8 ID Sampler	6.7	-3	9.7	4.7	29.937264	-89.838615		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1987	NGVD	15-CHBB	1_7/8 ID Sampler	6	-3	9	4	29.936360	-89.837568		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1987	NGVD	16-CHBB	1_7/8 ID Sampler	8	-3	11	6	29.935527	-89.836474		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1987	NGVD	17-CHBB	1_7/8 ID Sampler	10	-3	13	8	29.936312	-89.839543		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1987	NGVD	18-CHBB	1_7/8 ID Sampler	9	-3	12	7	29.935479	-89.838401		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1987	NGVD	19-CHBB	1_7/8 ID Sampler	6.6	-3	9.6	4.6	29.934575	-89.837354		no
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan, Hurricane Prote	2001	NGVD	BD-BB-16	1_7/8 ID Sampler	0	-10	10	-2	29.937790	-89.842290		no
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan, Hurricane Prote	2001	NGVD	BD-BB-17	1_7/8 ID Sampler	0	-10	10	-2	29.935900	-89.839960		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1984	NGVD	15-UBD	5in Piston "undisturbed"	18	-48	66	16	29.936015	-89.838171		no
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+18	1966	MSL	U-3	5in Piston "undisturbed"	8.2	-92	100.2	7.95	29.935027	-89.836355		yes
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan Emergency Res	2005	NGVD	MRGO-27	1_7/8 ID Sampler	0	-25	25	-2	29.933170	-89.837500		no
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan Emergency Res	2005	NGVD	MRGO-28	1_7/8 ID Sampler	0	-12	12	-2	29.931520	-89.833830		no
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan Emergency Res	2005	NGVD	MRGO-29	1_7/8 ID Sampler	0	-4	4	-2	29.928880	-89.831070		no
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	D-1U	5in Piston "undisturbed"	5.50	-93.00	98.50	3.5	29.934707	-89.836927		yes
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	D-1		8.80	-73.00	81.80	6.8	29.935448	-89.837042		no
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	D-2		11.00	-70.00	81.00	9	29.934739	-89.836036		no
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	D-3		11.00	-73.00	84.00	9	29.935374	-89.836260		no
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	D-4		8.20	-71.00	79.20	6.2	29.934800	-89.836834		no

BAYOU DUPRE												
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-62	3" ID Cove Barrel	12	-48	60	11.75	29.940762	-89.843279		no
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-63	3" ID Cove Barrel	9	-51	60	8.75	29.938930	-89.840900		no
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-64	3" ID Cove Barrel	12	-48	60	11.75	29.933195	-89.833999		no
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-65	3" ID Cove Barrel	11	-49	60	10.75	29.931339	-89.831739		no
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-66	3" ID Cove Barrel	9.6	-50	59.6	9.35	29.929531	-89.829407		no
Mississippi River Gulf Outlet Design Memorandum No. 1-B, Channels, Mile 39.0	1956	MSL	17-D	1_7/8" Wire core barrel	4.78	-56	60.78	4.53	29.938072	-89.835842	Only available final 10ft of	no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1987	NGVD	14-CHBB	1_7/8 ID Sampler	6.7	-3	9.7	4.7	29.937264	-89.838615		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1987	NGVD	15-CHBB	1_7/8 ID Sampler	6	-3	9	4	29.936360	-89.837568		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1987	NGVD	16-CHBB	1_7/8 ID Sampler	8	-3	11	6	29.935527	-89.836474		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1987	NGVD	17-CHBB	1_7/8 ID Sampler	10	-3	13	8	29.936312	-89.839543		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1987	NGVD	18-CHBB	1_7/8 ID Sampler	9	-3	12	7	29.935479	-89.838401		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1987	NGVD	19-CHBB	1_7/8 ID Sampler	6.6	-3	9.6	4.6	29.934575	-89.837354		no
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan, Hurricane Prote	2001	NGVD	BD-BB-16	1_7/8 ID Sampler	0	-10	10	-2	29.937790	-89.842290		no
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan, Hurricane Prote	2001	NGVD	BD-BB-17	1_7/8 ID Sampler	0	-10	10	-2	29.935900	-89.839960		no
Chalmette Area Plan, Hurricane Protection Levee Closures, MRGO B/L Sta 367	1984	NGVD	15-UBD	5in Piston "undisturbed"	18	-48	66	16	29.936015	-89.838171		no
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+18	1966	MSL	U-3	5in Piston "undisturbed"	8.2	-92	100.2	7.95	29.935027	-89.836355		yes
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan Emergency Res	2005	NGVD	MRGO-27	1_7/8 ID Sampler	0	-25	25	-2	29.933170	-89.837500		no
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan Emergency Res	2005	NGVD	MRGO-28	1_7/8 ID Sampler	0	-12	12	-2	29.931520	-89.833830		no
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan Emergency Res	2005	NGVD	MRGO-29	1_7/8 ID Sampler	0	-4	4	-2	29.928880	-89.831070		no
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	D-1U	5in Piston "undisturbed"	5.50	-93.00	98.50	3.5	29.934707	-89.836927		yes
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	D-1		8.80	-73.00	81.80	6.8	29.935448	-89.837042		no
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	D-2		11.00	-70.00	81.00	9	29.934739	-89.836036		no
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	D-3		11.00	-73.00	84.00	9	29.935374	-89.836260		no
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Design Memorandur	1967	NGVD	D-4		8.20	-71.00	79.20	6.2	29.934800	-89.836834		no

Table 5-1. Available Borings in the vicinity of the breaches along MR-GO.

Document	Boring Year	Datum	Boring ID	Top Elevation	Bottom Elevation	TOP EL. in NAVD88	PRE-BREACH LIDAR ELEV.	POST-BREACH LIDAR ELEV.	FINAL BORING TOP ELEVATION, NAVD88
BAYOU BIENVENUE									
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1966	MSL	U-2	3.1	-96	2.85	8	3.8	2.85
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1965	MSL	G-28	7	-53	6.75	-	-	6.75
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1965	MSL	G-29	6	-54	5.75	0.7	1	5.75
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1965	MSL	G-30	-13	-63	-13.25	5.9	3.6	-13.25
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+01	1966	MSL	G-32	3	-57	2.75	5.7	2.7	2.75
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1965	MSL	G-33	3	-57	2.75	4.9	3	2.75
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1965	MSL	G-34	3	-57	2.75	-	4.1	2.75
Mississippi River Gulf Outlet Design Memorandum No. 1-B, Cha	1956	MSL	G-35	4	-36	3.75			3.75
Mississippi River Gulf Outlet Design Memorandum No. 1-B, Cha	1956	MSL	10-D	2.78	-58	2.53	-	-	2.53
Hurricane Protection Levee Second Enlargement MRGO B/L Sta	1984	NGVD	1-UBD	13.9	-77	11.9	19.1	10.5	11.9
Hurricane Protection Levee Second Enlargement MRGO B/L Sta	1984	NGVD	2-UBD	9.8	-82	7.8	3.9	3.7	7.8
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1984	NGVD	7-UBD	13	-53	11	8.7	7.5	11
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1984	NGVD	8-UBD	9	-50	7	20.6	2.8	7
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Des	1967	NGVD	B-2	5.7	-63	3.7	7.3	3.3	3.7
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Des	1967	NGVD	B-1U	5.6	-80	3.6	0.8	-	3.6
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Des	1967	NGVD	B-3	5.6	-65	3.6	4.5	1.6	3.6
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Des	1966	NGVD	B-4	5.6	-65	3.6	1.1	-	3.6
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Des	1967	NGVD	B-5	5.7	-63	3.7		-	3.7
Hurricane Protection Levee, Chalmette Area Plan, Sta 370+00 t	1971	MSL	U-2B	9	-70	8.75	17.8	11	8.75
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1987	NGVD	1-CHBB	4	-4	2	4.8	4.7	2
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1988	NGVD	2-CHBB	7	-4	5	2.2	4.2	5
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2001	NGVD	BD-BB-1	0	-7	-2	-	-	-2
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2002	NGVD	BD-BB-2	0	-10	-2	-	-	-2
Hurricane Protection Levee Second Enlargement MRGO B/L Sta	1976	NGVD	C-1	3.7	-11	1.7	-	3.8	1.7
Hurricane Protection Levee Second Enlargement MRGO B/L Sta	1976	NGVD	C-2	5.1	-10	3.1	-	4.8	3.1
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2005	NGVD	B-1	0	-25	-2	-	-	-
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2005	NGVD	B-2	0	-25	-2	-	-	
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2005	NGVD	B-3	0	-25	-2	-	-	
BAYOU DUPRE									
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-62	12	-48	11.75	-	-	11.75
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-63	9	-51	8.75	10.2	5.6	8.75
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-64	12	-48	11.75	26	8.8	11.75
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-65	11	-49	10.75	27.9	6.8	10.75
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+16	1966	MSL	G-66	9.6	-50	9.35	4.9	4.6	9.35
Mississippi River Gulf Outlet Design Memorandum No. 1-B, Cha	1956	MSL	17-D	4.78	-56	4.53	-	-	4.53
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1987	NGVD	14-CHBB	6.7	-3	4.7	5.9	-	4.7
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1987	NGVD	15-CHBB	6	-3	4	1.9	-	4
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1987	NGVD	16-CHBB	8	-3	6	7.1	-	6
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1987	NGVD	17-CHBB	10	-3	8	17.9	3.9	8
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1987	NGVD	18-CHBB	9	-3	7	6.4	9.7	7
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1987	NGVD	19-CHBB	6.6	-3	4.6	2.9	1.2	4.6
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2001	NGVD	BD-BB-16	0	-10	-2	-	-	-2
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2001	NGVD	BD-BB-17	0	-10	-2	-	-	-2
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1984	NGVD	15-UBD	18	-48	16	11.9	10.9	16
Hurricane Protection Levee, First Lift, Sta 594+00 - Sta 770+18	1966	MSL	U-3	8.2	-92	7.95	0.1	-	7.95
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan	2005	NGVD	MRGO-27	0	-25	-2	-	-	-2
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan	2005	NGVD	MRGO-28	0	-12	-2	25.6	-	-2
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan	2005	NGVD	MRGO-29	0	-4	-2	14.7	-	-2
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Des	1967	NGVD	D-1U	5.5	-93	3.5	1	0.7	3.5
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Des	1967	NGVD	D-1	8.8	-73	6.8	5.2	2.6	6.8
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Des	1967	NGVD	D-2	11	-70	9	5.9	6.2	9
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Des	1967	NGVD	D-3	11	-73	9	-	-	9
Lake Pontchartrain, LA. And Vicinity, Chalmette Area Plan, Des	1967	NGVD	D-4	8.2	-71	6.2	0.8	8.5	6.2
MRGO STATION 497									
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1966	MSL	G-41	5	-54	4.75	-	2	4.75
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1966	MSL	G-42	4	-56	3.75	-	2.7	3.75
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1966	MSL	G-43	4	-56	3.75	-	2.9	3.75
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1966	MSL	G-44	4	-55	3.75	-	3.5	3.75
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1966	MSL	G-45	5	-55	4.75	-	-	4.75
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1966	MSL	G-46	0	-60	-0.25	-	-	-0.25
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1966	MSL	G-47	9	-51	8.75	-	-	8.75
Hurricane Protection Levee, First Lift, Sta 368+00 - Sta 570+00	1966	MSL	G-48	9	-50	8.75	-	-	8.75
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2005	NGVD	B-8	0	-25	-2	-	-	
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2005	NGVD	B-11	0	-25	-2	-	-	
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2005	NGVD	MRGO-9	0	-25	-2	-	-	
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2005	NGVD	MRGO-10	0	-25	-2	-	-	
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2005	NGVD	MRGO-12	0	-25	-2	-	-	
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2005	NGVD	MRGO-13	0	-25	-2	-	-	
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2001	NGVD	BD-BB-6	0	-10	-2	-	-	-2
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2001	NGVD	BD-BB-7	0	-10	-2	-	-	-2
Lake Pontchartrain Louisiana and Vicinity Chalmette Area Plan,	2001	NGVD	BD-BB-8	0	-10	-2	-	-	-2
Hurricane Protection Levee Second Enlargement MRGO B/L Sta	1976	NGVD	C-5	5	-10	3	-	3	3
Hurricane Protection Levee Second Enlargement MRGO B/L Sta	1976	NGVD	C-6	5	-10	3	-	1.5	3
Mississippi River Gulf Outlet Design Memorandum No. 1-B, Cha	1956	MSL	13-D	2.78	-57	2.53	-	-	2.53
Hurricane Protection Levee Second Enlargement MRGO B/L Sta	1984	NGVD	11-UBD	14	-52	12	-	6.9	12
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1987	NGVD	3-CHBB	1	-4	-1	-	-	-1
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1987	NGVD	4-CHBB	5	-2	3	-	-	3
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1987	NGVD	5-CHBB	6	-3	4	-	-	4
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1987	NGVD	6-CHBB	8	-4	6	-	-	6
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1987	NGVD	7-CHBB	7	-4	5	-	-	5
Chalmette Area Plan, Hurricane Protection Levee Closures, MR	1987	NGVD	8-CHBB	8	-3	6	-	-	6
Mississippi River Gulf Outlet Design Memorandum No. 1-B, Cha	1956	MSL	2-U	-6	-61	-6.25	-	-	-6.25

NOTES ON ELEVATIONS:

1. Top elevations obtained from design documents
2. IPET borings (2005) appear to have "zero" elevations for all borings.
3. LIDAR elevations were determined for all borings, for Pre and Post-hurricane surveys. Where a "-" appears, the survey did not extend to the coordinates of the boring.

Table 5-2. Interpreted elevations for explorations along MR-GO

6. MATERIAL PROPERTIES

A limited number of borings with available laboratory testing were found in the literature review. Of the 80 reviewed borings for the three sites, only two borings for the breach near Bayou Bienvenue, two borings for Bayou Dupre, and one for the site at Station 497+00 had laboratory testing that could be used in the analyses. No Cone Penetrometer Test (CPT) results were found in the available information.

Material properties were determined from available laboratory and field testing. Section 13.1 contains all laboratory testing extracted from the borings. Since the 2006 investigations (IPET, ILIT) were performed after Katrina, no "new" samples could be obtained from soils below the embankment load. There are no crest borings in the reviewed documentation at the sites of interest.

6.1 BAYOU BIENVENUE

Figure 6-1 shows the available undrained shear strength (Su) for the soils near the toe of the EBSB near the Bayou Bienvenue control structure. Figures 6-2 and 6-3 show Atterberg Limits and Unit Weights for the same samples. The difference between the shear strengths for between the fine-grained soils near the toe of the EBSB, and under the embankment could not be established since no CPTs or crest tests were available. The stratigraphic interpretation and results from the laboratory testing show that the Marsh deposit extends from approximately the ground surface to elevation -15 to –20 feet, and is underlain by a soft clay layer that appears to be normally consolidated. A thin sand layer occurs approximately at elevation –30 feet, and is underlain by another stiffer clay layer to unknown depths.

Undrained shear strengths for the Marsh layer range from 50 to 250 pounds per square foot (psf), and for the underlying clays the test results range between 250 and 1200 psf, increasing with depth.

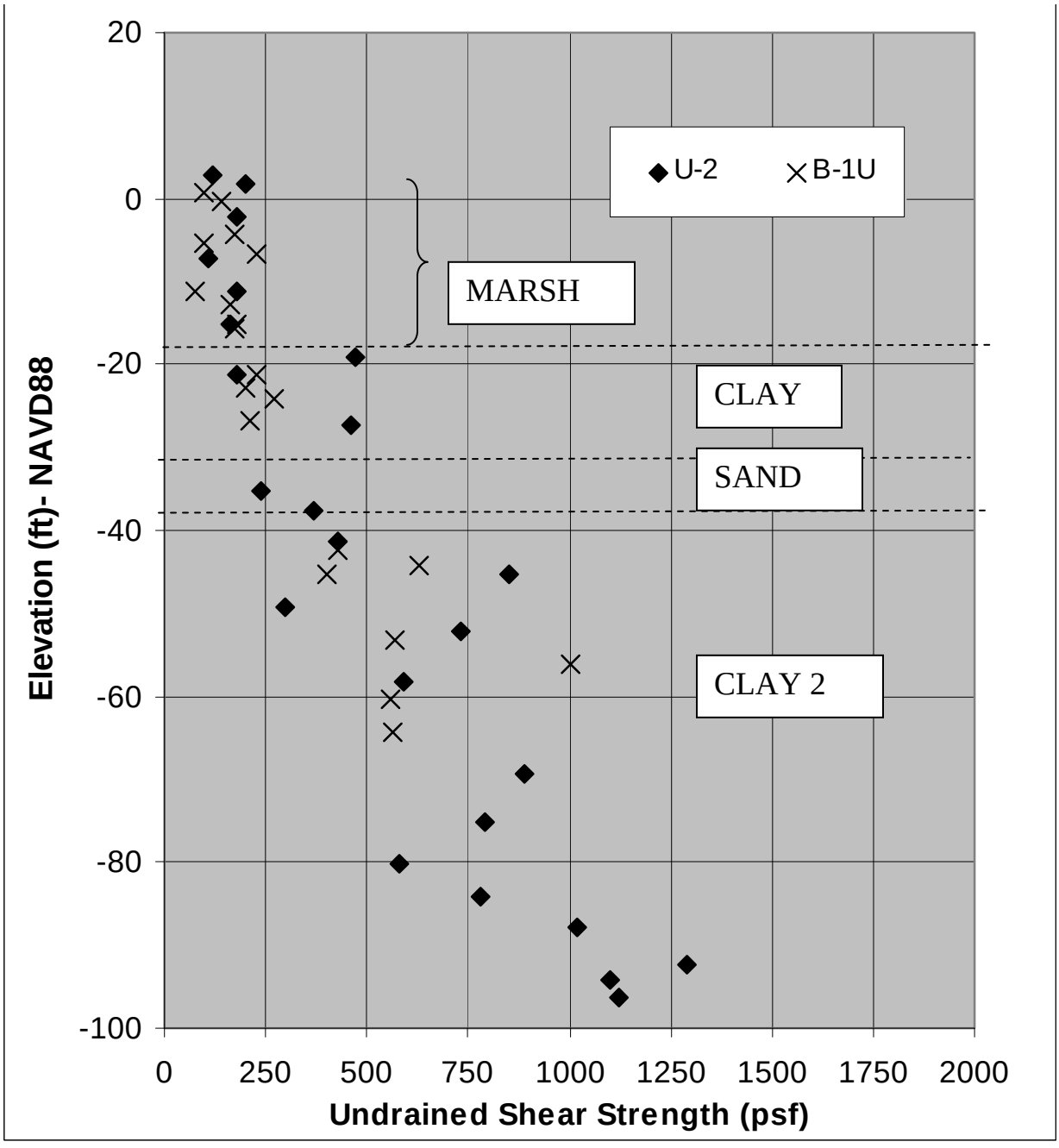


Figure 6-1. Undrained Shear strength values for Bayou Bienvenue, near the EBSB toe.

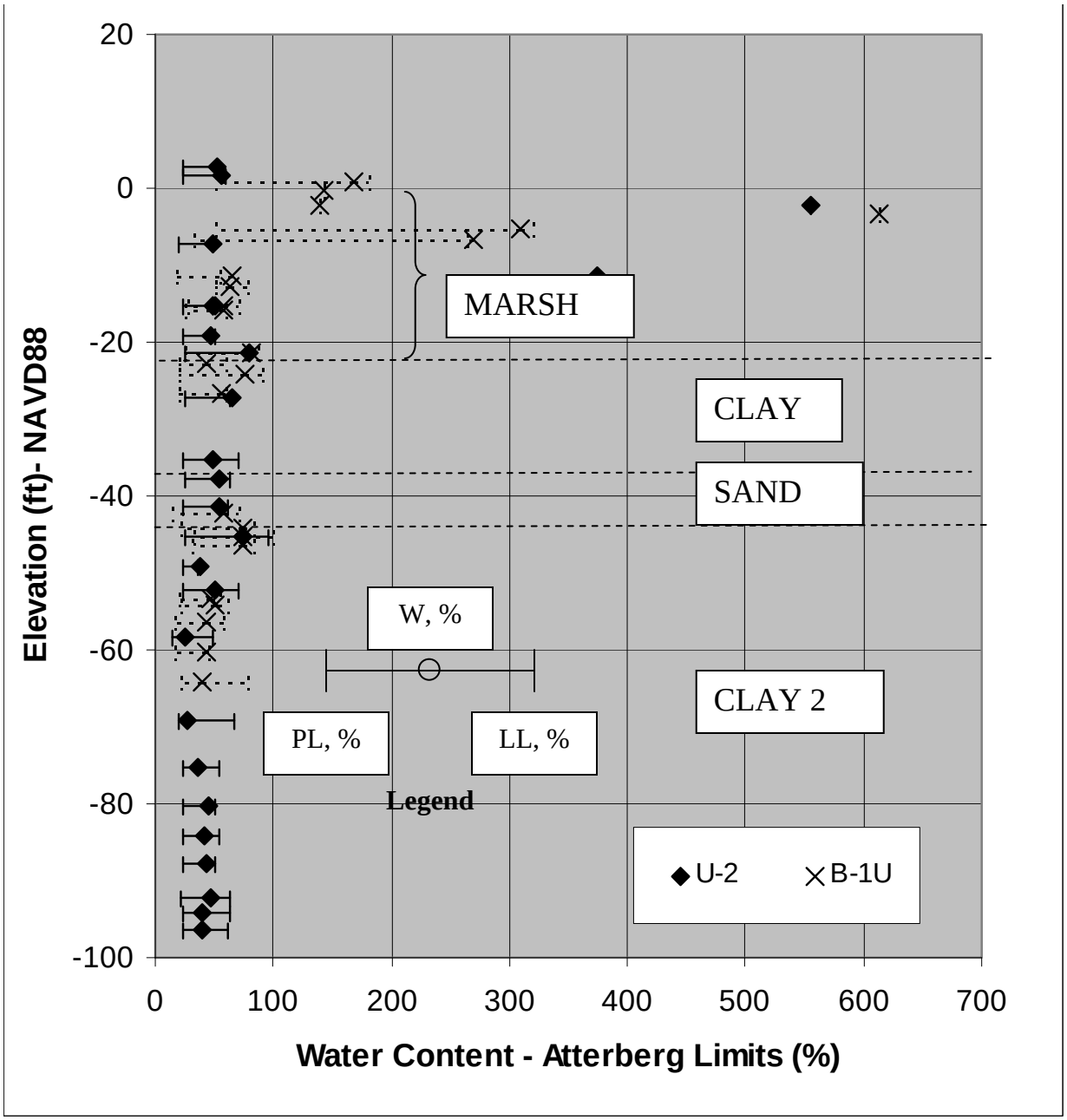


Figure 6-2. Atterberg Limits values for Bayou Bienvenue, near the EBSB toe.

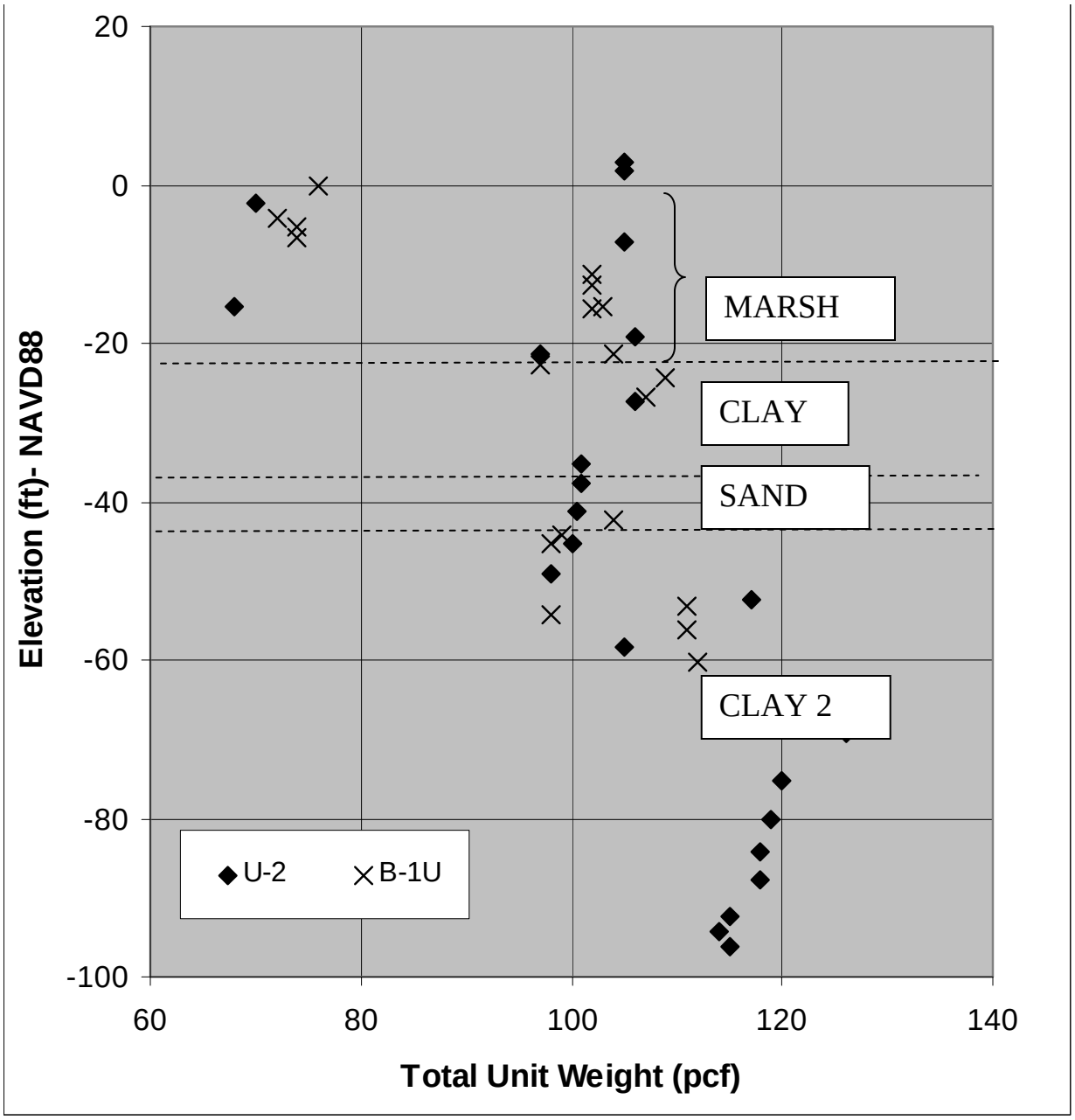


Figure 6-3. Unit Weight values for Bayou Bienvenue, near the EBSB toe.

Table 6-1 shows the summary of depths obtained from the cross section for the breach at Bayou Bienvenue. Based on these depth intervals, the available laboratory testing was grouped (undrained shear strengths, index tests) and statistical properties were developed for each soil

unit (i.e. median, coefficient of variation) to describe the uncertainties associated with the soil parameters.

Material	Elevation Interval, feet (NAVD88)	
	From	To
Clays (CL, CH)	+18	2
Shell (anthropogenic)	2	-11
Marsh (Peat)	2	-20
Grey Clay (CH)	-20	-30
Sand	-30	-34
Clays (CH)	-34	n/a

Table 6-1. Profile for Bayou Bienvenue. The shell layer was not tested, but was included in the models according to Section 2.

6.2 BAYOU DUPRE

Figure 6-4 shows the available undrained shear strength (Su) for the soils near the toe of the EBSB. Figure 6-5 and 6-6 show Atterberg Limits and Unit Weights for the same samples. The difference between the shear strengths for between the fine-grained soils near the toe of the EBSB, and under the embankment could not be established since no CPTs or crest tests were available. The stratigraphic interpretation, breach pictures and results from the laboratory testing show a layered embankment fill. It is composed by an approximately 4ft thick top layer of shells, followed by a soft clay (CH) layer extending to the ground surface (El. +5 feet). Boring 15-UBD shows a dropped sample and a shell sample just below the ground surface.

Below the embankment, a Marsh layer occurs down to elevation –10 feet, overlaying a deep irregular clay deposit that extends to unknown depths. Overconsolidation history is difficult to assess in this site, shear strengths are sparse and no CPTs are available for comparison.

Tested shear strengths range from 200 to 300 psf for the Marsh layer, and the clays show depth-dependent strengths ranging from 300 to 2000 psf below elevation –80 feet.

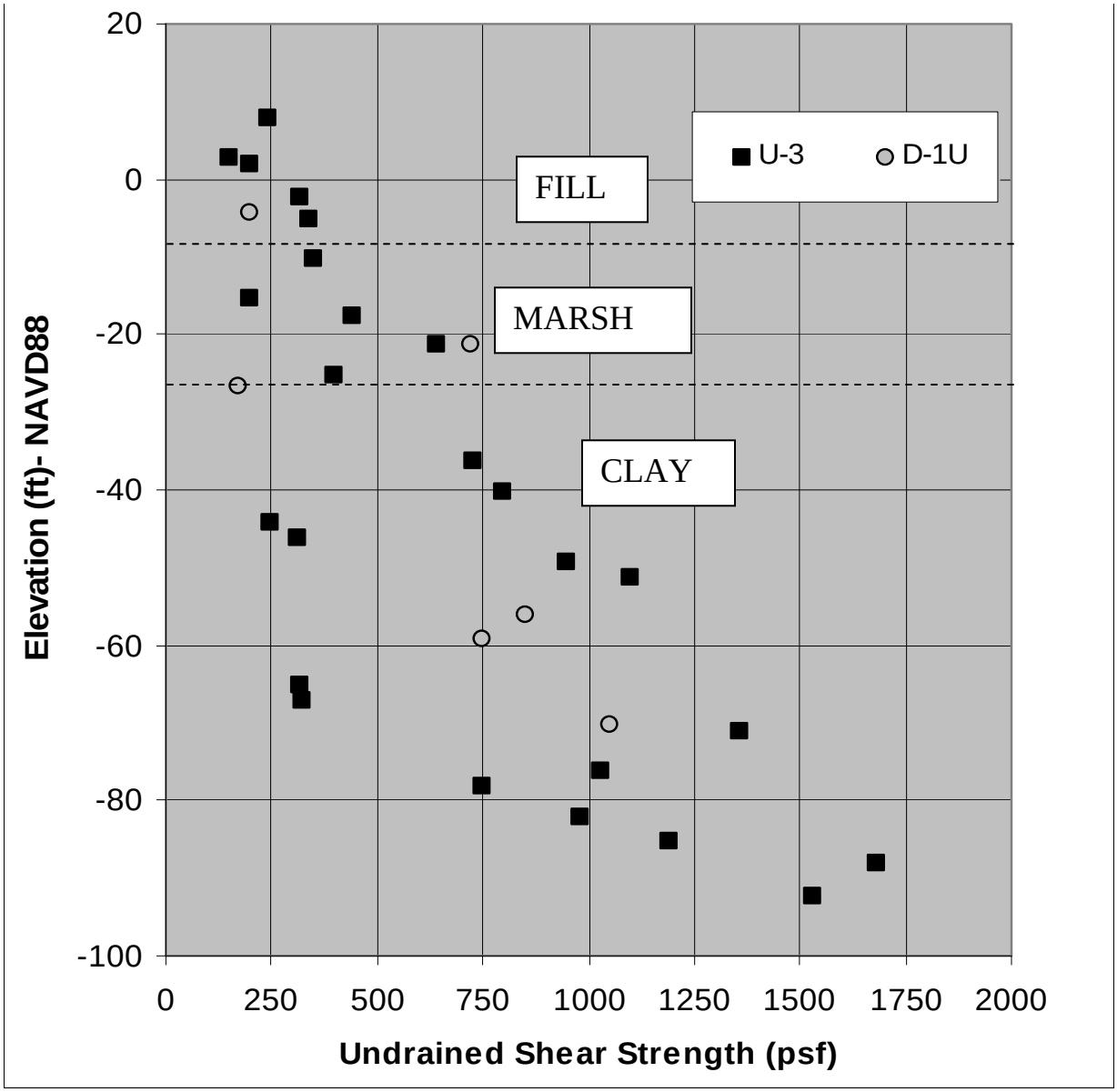


Figure 6-4. Undrained shear strength values for Bayou Dupre, near the EBSB toe.

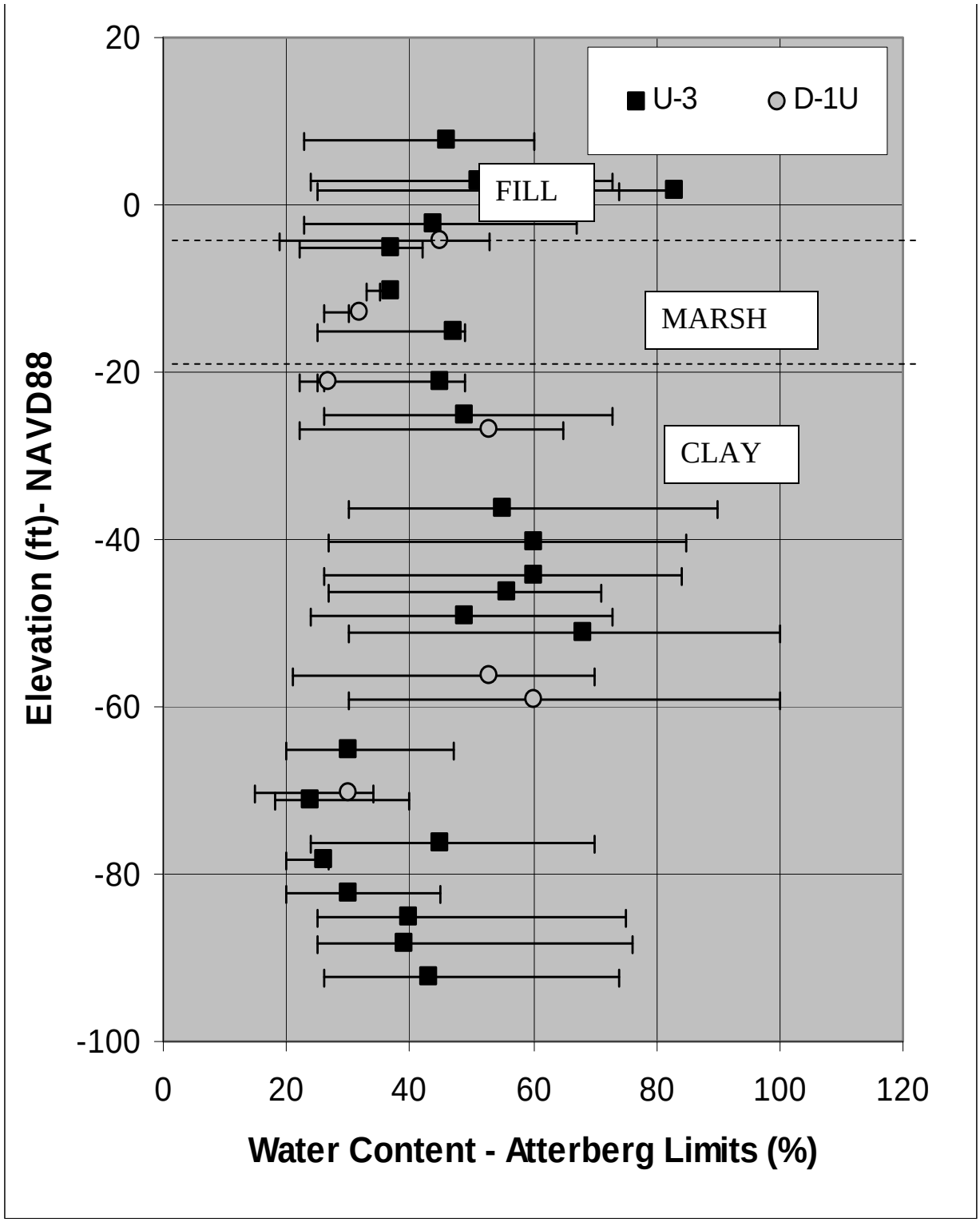


Figure 6-5. Atterberg Limit values for Bayou Dupre, near the EBSB toe.

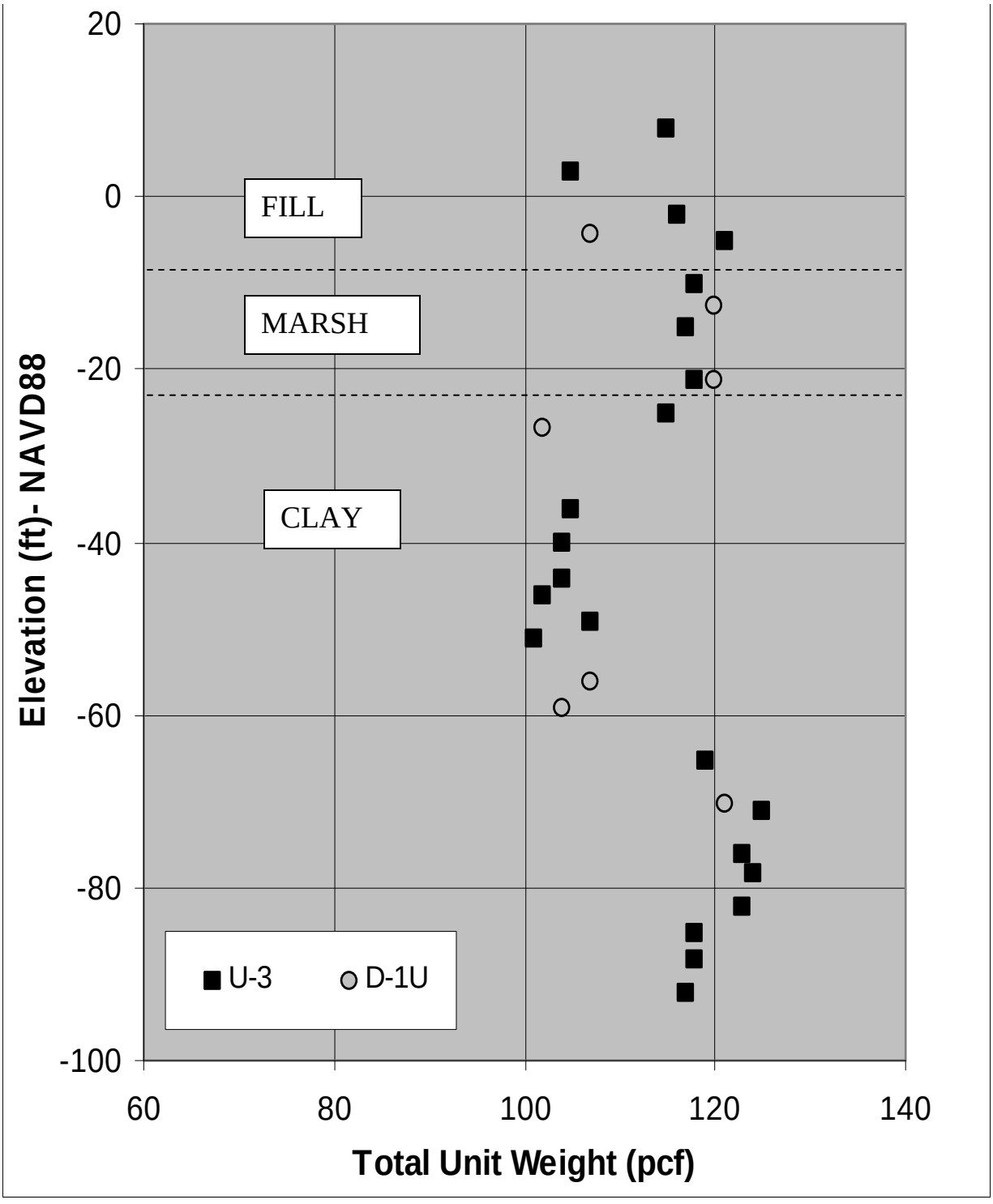


Figure 6-6. Unit Weight values for Bayou Dupre, near the EBSB toe.

Material	Elevation Interval, feet (NAVD88)	
	From	To
Shell Fill	+18	14
Clays (CL, CH)	14	6.5
Shell Fill	6.5	5
Marsh (Peat)	5	-11
Grey Clay (CH)	-11	n/a

Table 6-2. Profile for Bayou Dupre.

7. MARSH PERMEABILITY

Peat is a highly compressible material that exhibits rapid decrease on hydraulic conductivity as stresses increase and void spaces decrease (Mesri, 2007). However, the factors influencing the behavior of such soils depend not only on stresses, but on depositional mechanics, aging and decomposition, amount, type and orientation of fibers and saturation.

To date, it is still an open topic on how to approach Marsh hydraulic conductivity and that is the reason behind the parametric calculations in this report. The peats at the MR-GO have not been tested or even described in detail, therefore a correlation with published types of peat, and consequently the assignment of properties is very difficult.

Hogan et al, (2006) shows field tests performed on a peat-bog fen deposit. Using pumping and slug tests the investigators were able to determine widely scattered permeability values, ranging

from 10^{-3} to 10^{-4} cm/s, but with values as high as 10^{-2}. The results of the test program showed that permeability decreases with depth.

An important result from Hogan (2006) is that by using a load test, the time of dissipation of excess piezometric head occurred in just 2.5 hours, indicating high permeability values.

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

Table 7-1 summarizes the permeability values computed and/or tested by field and laboratory methods found in the researched publications.

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

Table 7-1. Summary of permeability values.

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

Beckwith et al (2003) have developed a new mechanism for extraction of "undisturbed" peat samples for permeability testing. Strong emphasis is made on the need to incorporate several samples at different depths inside a single deposit to capture the anisotropy (kv/kh) and variation of permeability with depth (and consequently stresses).

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

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

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

When selecting high permeability values for the Marsh deposit (10^{-3} cm/s), this unit acts as a draining layer, conducting pore pressures much faster beneath the EBSB. The effect of these

values does not significantly increase the calculated hydraulic gradients on the EBSB toe, but it allows the potential for underseepage and water exiting at the EBSB toe.

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

From the seepage analyses performed by IPET, on Appendix 17 of their Final Report, the best estimate for the permeability of the Marsh deposit is 10^{-4} cm/s. According to the plot on Figure V-17-15 (Figure 7-1), this permeability occurs when the soils are subject to overburden effective stresses from 0 to 500psf. This is a likely range for the shallow layer located at the toe of the EBSB, but the deeper Marsh underneath the EBSB is probably subject to effective stresses in the order of 0 to 1000 psf, with permeability of 10^{-5} to 10^{-7} cm/s.

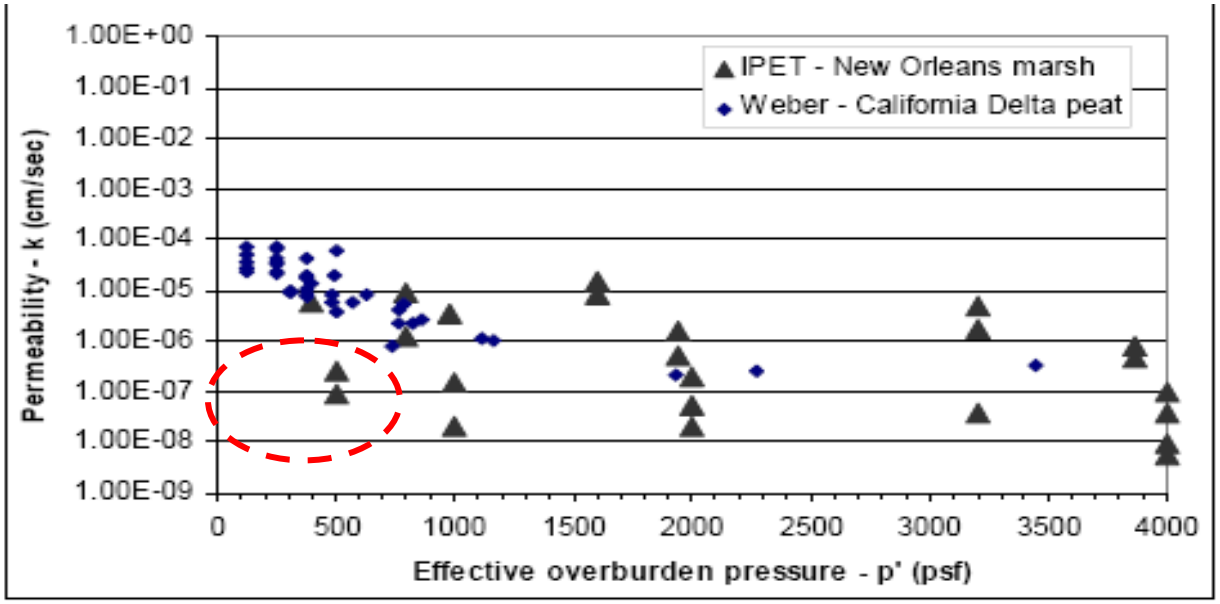


Figure 7-1. Hydraulic conductivity values for Peat. The range of effective stresses for the shallow (toe) peats ranges from 0 to 1000 psf (dashed circle).

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

It is difficult to come up with a best estimate for the permeability of the Marsh deposits at the breach sites on MR-GO. All analyses in this report were performed using a range of permeabilities from 10^{-3} to 10^{-6} cm/s.

8. SEEPAGE ANALYSES

As described in previous sections, Finite Element codes were used to perform steady state and transient seepage analyses.

8.1 Model Description

The software SEEP\W, part of the package GEOSTUDIO 2004 was used for seepage modeling. The program uses a finite element code to route the boundary conditions imposed as hydraulic heads on the boundaries of the problem through a porous media, calculating heads and pressures along the nodes of the finite element mesh.

The software requires the generation of permeability functions that include the decrease of permeability depending upon increases in negative pore pressures (suction).

Figure 8-1 shows a typical mesh used in SEEP\W (this case the section for Bayou Dupre) and Table 8-1 summarizes the hydraulic properties of the soil layers, including conductivity at saturation and anisotropy ratio.

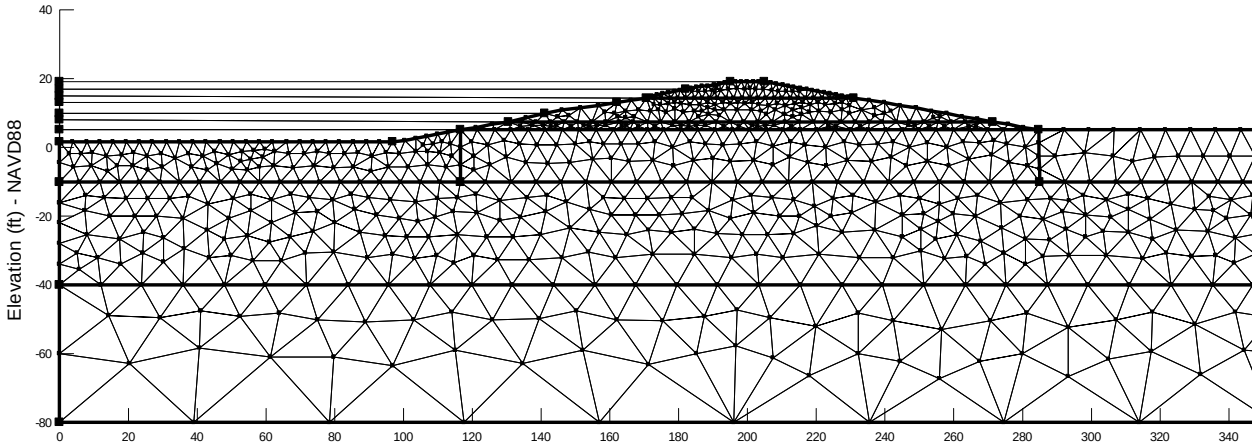


Figure 8-1. Typical finite element mesh for seepage calculations.

Figure 8-2 is the idealized hydrograph from Figure 3-15 as it was introduced into the model; Section 13.3 contains the hydraulic conductivity functions employed for every material in the SEEP-W models. Shell permeability is another key issue in the models, and no tests were found in the literature review. Given the size and behavior of this material, fine-gravel permeability was assigned to deposits composed by these materials. General references for fine gravels (approximate particle size of shell materials range between 1 and 100 cm/s (Table 8-3).

Material	K at saturation (cm/s)	Anisotropy Ratio (ky/kx)
Clays	10^{-6}	0.3
Sands	10^{-2}	0.25
Marsh (upper bound)	10^{-3}	0.10
Marsh (lower bound)	10^{-6}	0.10
Shell Fills	10	1.0

Table 8-2. Input values for seepage models.

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

Table 8-3. Gravel permeability values. Modified from Bear (1972). Source: www.answers.com.

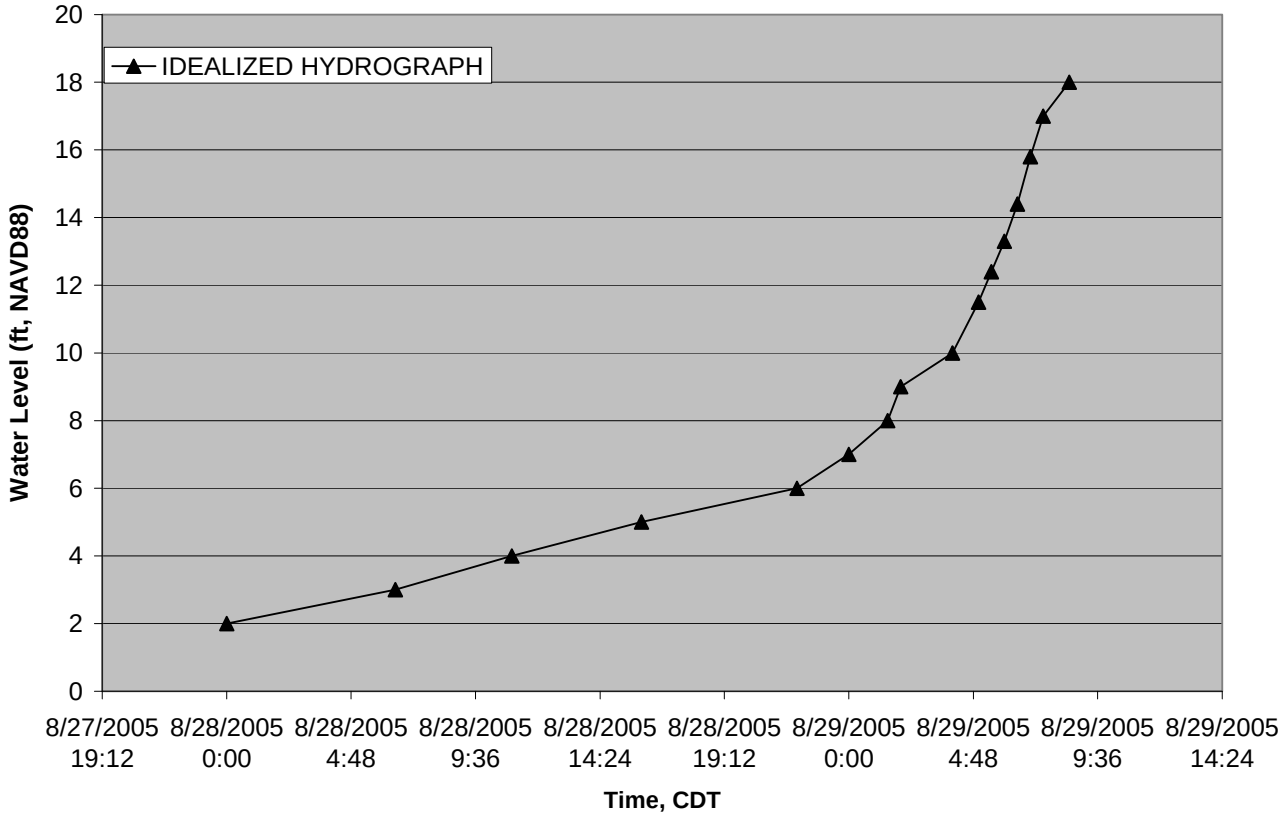


Figure 8-2. Idealized hydrograph for the transient seepage analyses. (H: total head in feet), time in seconds.

Boundary Conditions for the model include prescribing No-flow boundaries (or zero pressure) along the bottom, canal side edge, and landside EBSB slope and ground surface (the latter ones have the option of allowing water to flow out of the mesh, called review nodes). Total heads equal to different values of storm surge were applied to the waterside EBSB slope and canal bottom; and total heads with elevation equal to the ground surface elevation were applied at the nodes on the "far field" vertical edge of the model (landside).

8.2 BAYOU BIENVENUE

8.2.1 Steady State

A baseline steady state analysis was performed using a water elevation of +18.0 feet (NAVD88). These elevations were taken as reference of maximum development of pore pressures. Steady state implies that water has had enough time to flow through the media and a constant pressure regime has developed, therefore no flow is occurring. This does not mean that the pressures are hydrostatic. Often times, when less permeable deposits are at or near the surface, they act as "blankets" that do not permit the pore pressures to alleviate, creating excess heads and pore pressures that can cause uplift or blowout failures. This can be particularly critical on levees and embankments subject to relatively rapid hydraulic loading with thin fine grained soil layers located at or near the toe of the levee. Substantial differences in pore pressures can be seen when a steady state regime of pressures is calculated, compared with a simpler "sketched" phreatic surface that assumes hydrostatic pressures are developed underneath it. The shell materials present under clay fills allow quick development of pore pressures on the models.

From Figures 8-4 and 8-5 it can be observed that the flow regime is dominated by the presence of the permeable shell materials, and Marsh has a smaller influence on the hydraulic behavior of the system. The black arrows represent velocity vectors of subsurface flow, with the largest arrow with a value of over 30 feet per day. Figure 8-4 considers the Marsh deposit to have a permeability of 10^{-3} cm/s, and Figure 8-5 considers 10^{-6} cm/s for the same layer. It is clear that only minor head losses occur within the foundation soils until the tip of the sheet pile is reached.

It is seen that when the Marsh is treated as a clay (Figure 8-5), there is a substantial head loss before the water reaches the sheet pile; whereas almost no head loss occurs when the marsh is assigned a higher permeability, therefore making heads and pore pressures larger in this stratum.

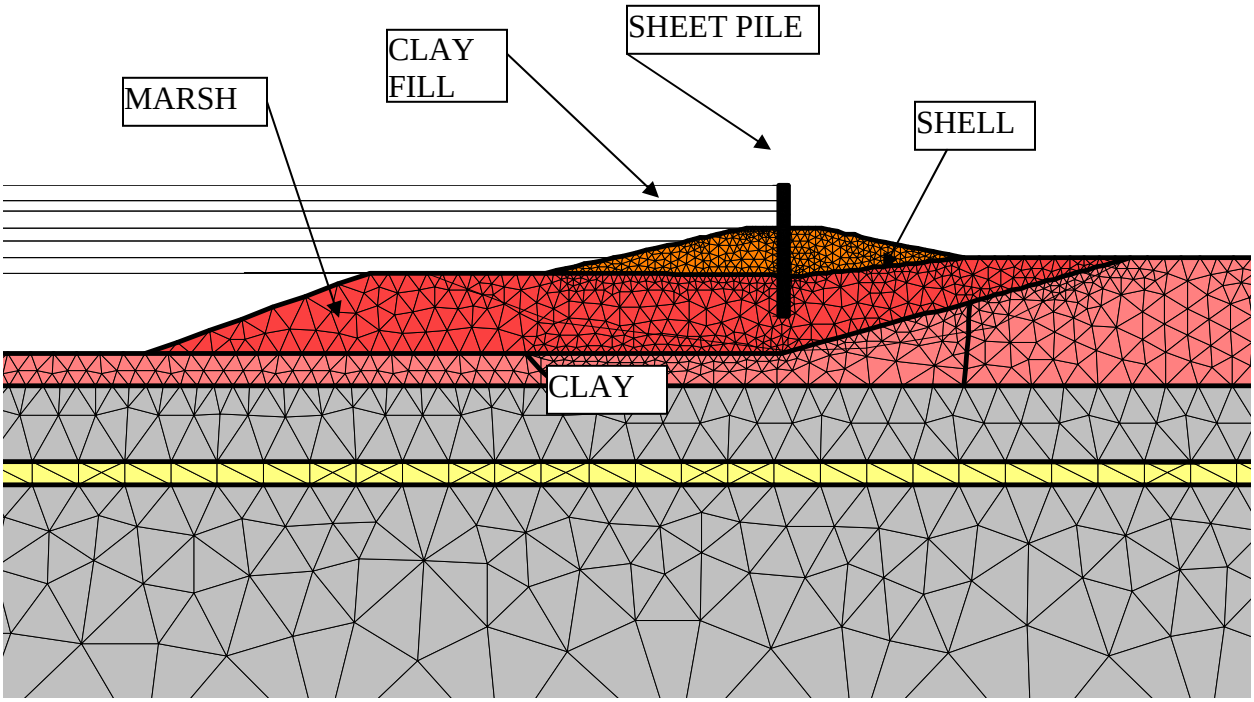


Figure 8-3. Cross Section and Finite Element Mesh used for the seepage analyses in Bayou Bienvenue.

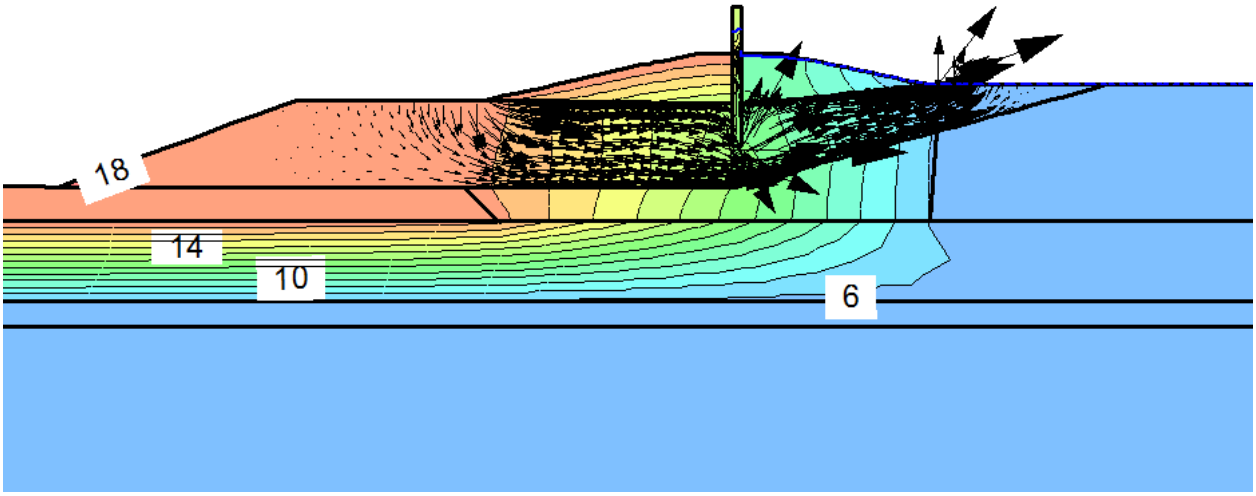


Figure 8-4. Head contours for the steady state condition. Storm surge at +18 feet. Marsh k: 10^{-3} cm/s.

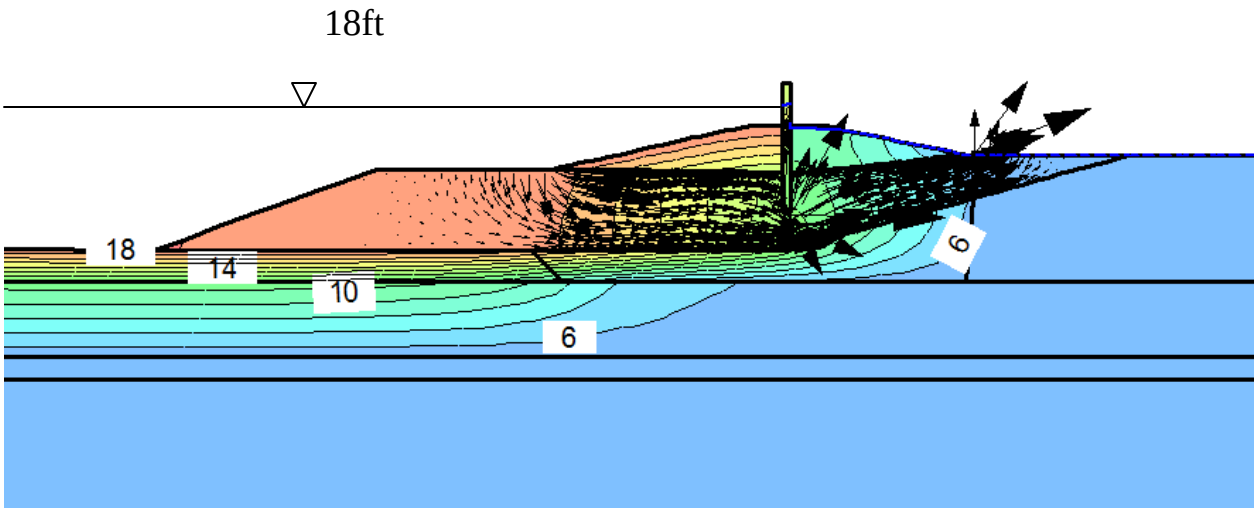


Figure 8-5. Head contours for the steady state condition. Storm surge at +18 feet. Marsh k: 10^{-6} cm/s

8.2.2 Transient Models

Transient analyses were performed for the rising water level on the flood side, according to the hydrograph from Figure 3-13. Figure 8-2 shows the idealized hydraulic head versus time acting on the canal side of the levee. This curve was input into the seepage models, which generated calculation steps beginning at a water elevation +2 feet.

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

The results of the analysis show that after 33 hours of modeled time (August 28, 2005 0:00 to August 29, 2005, 9:00am), high pore pressures near Steady State condition developed along the Marsh and Shells deposits. The transient models start from the “low” pre-storm generated steady

state at 2ft, generated by August 28, 2005 06:00 hours, and in 16 time increments, Elevation +18.0 feet (NAVD88) is reached on August 29, 2005 8:30 am (CDT). Results for every hour are plotted in these figures. Figure 8-6 shows the calculated phreatic (water table) surface and velocity vectors on August 29, 2005 8:30 am. It can be seen that the levee is not fully saturated, but the water level exits at the toe of the fill materials. The large arrows indicate high flow velocities concentrated at the toe of the levee, with the possibility for triggering of erosion of the shell materials.

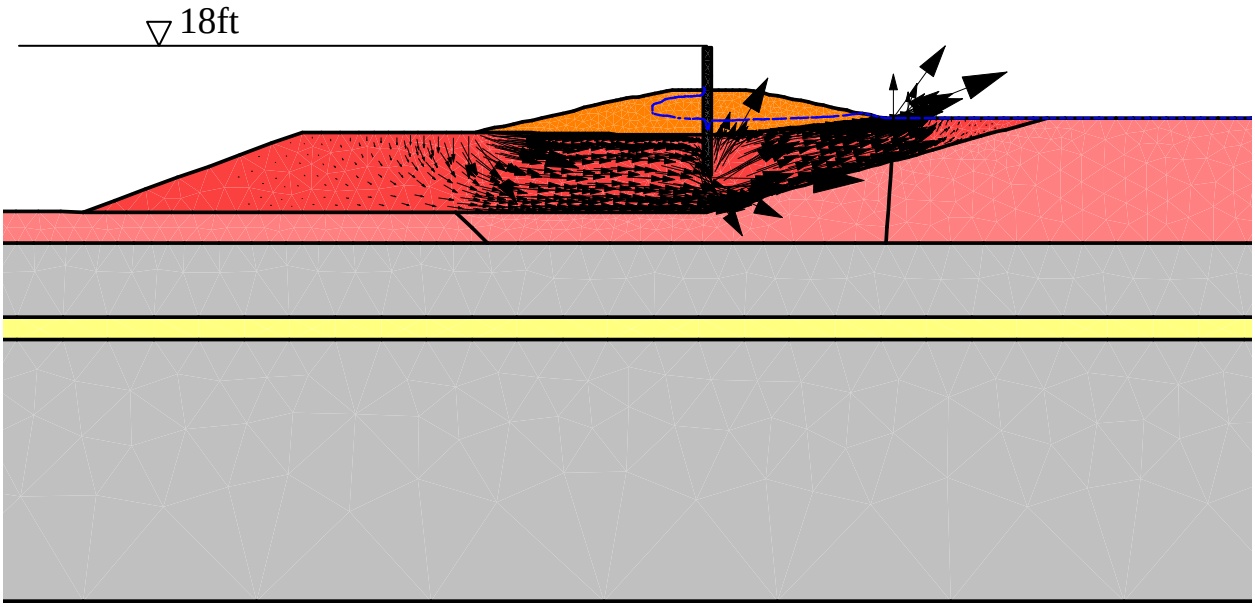


Figure 8-6. Results of the transient model, August 29, 2005 8:30am (CDT). Storm surge at +18 feet NAVD88.

Figure 8-7 shows the evolution of pore water pressure at two different locations on the toe of the levee: 1) Center of shell layer, under toe of levee, and 2) Center of Marsh layer, under toe of levee.

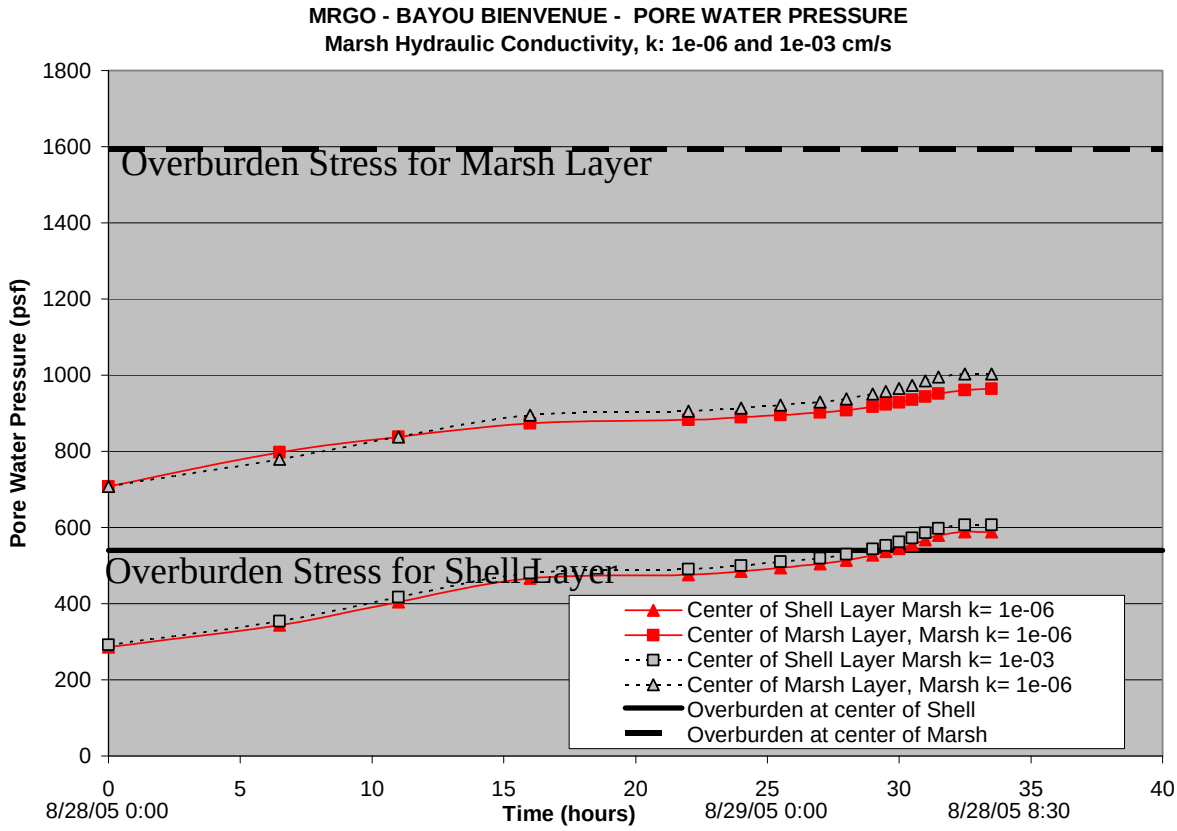


Figure 8-7. Transient model results. Pore pressure versus time on two points near the toe of the levee.

In Figure 8-7, the lines of calculated pore pressure are compared to the total overburden stress at the same points (thick horizontal lines). It can be seen that Piping can occur at the downstream toe when the pore water uplift pressures at a point are greater than the total vertical stress at that point. The subjects of piping and internal erosion are covered in textbooks such as Cedegren (1989). It is clearly seen that piping will initiate for the shell layer approximately 30 hours after the start of the rise in water level, between 6:00 and 7:00 am on August 29, 2005. This time frame is equivalent to a storm surge between elevations +10 feet and +12 feet NAVD88. The calculated overburden stresses at the Marsh layer show that no piping can trigger for this layer.

The influence of the marsh permeability on the pore pressures is small and piping inside the shell material occurs regardless of the value of permeability chosen for the underlying marsh soils.

Figure 8-8 repeats Figure 8-6, this time head contours are shown for the case of Marsh k: 10^{-3} cm/s, and Figure 8-9 shows the computed hydraulic gradients for the same case.

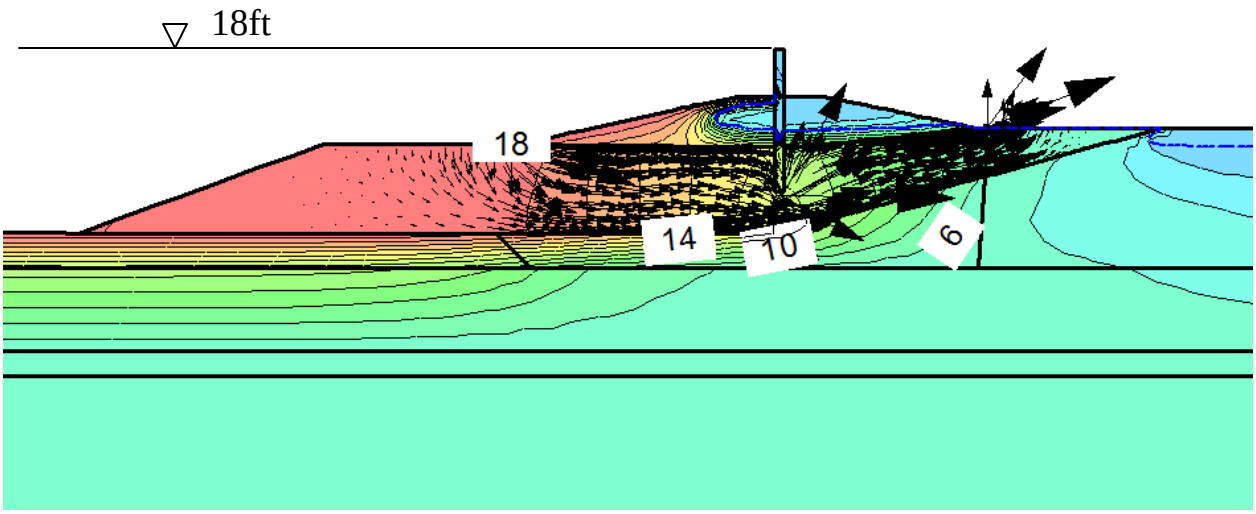


Figure 8-8. Transient model results. Total Heads on august 29, 2005 8:30 am. Storm surge at +17.5 feet NAVD88.

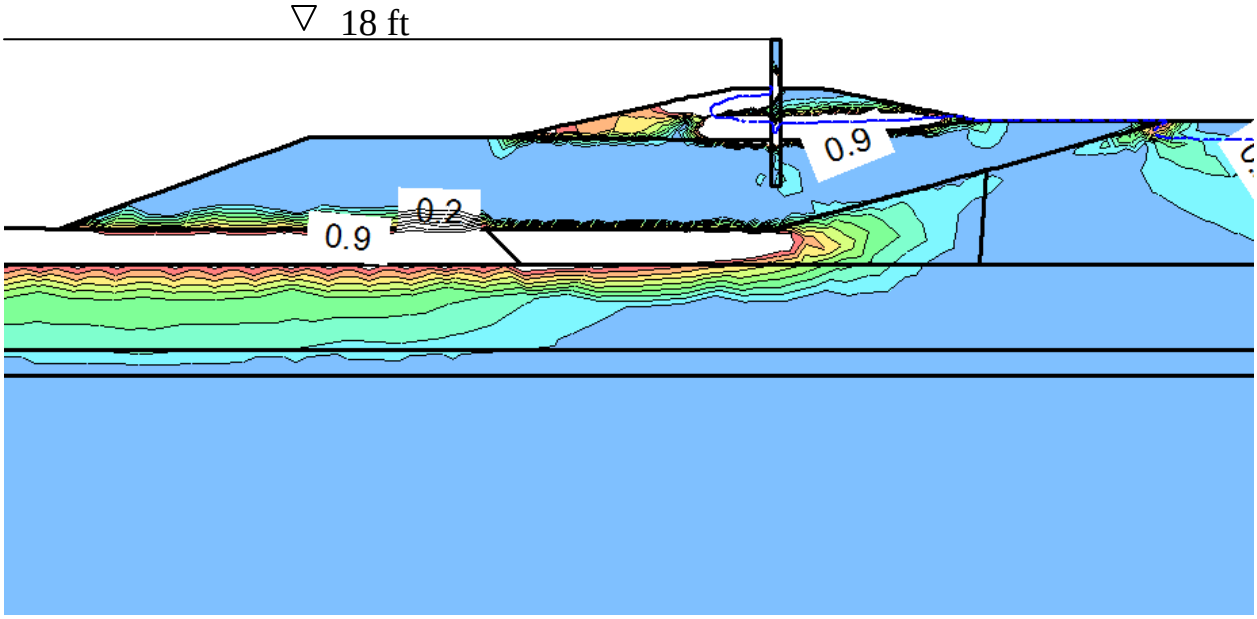


Figure 8-9. Transient model results. Hydraulic Gradients on august 29, 2005 8:30 am.

Figure 8-10 shows the advancement of the "wetting front" with time. The lowest line shows the initial condition of calculated phreatic surface established at elevation +2ft, and the uppermost is the final condition at 8:30 am on August 29, 2005. Near the toe, the water rises approximately 7ft until it exits at the toe of the levee. The water begins to seep out from the toe approximately on August 28, 2005 at 8:00 pm.

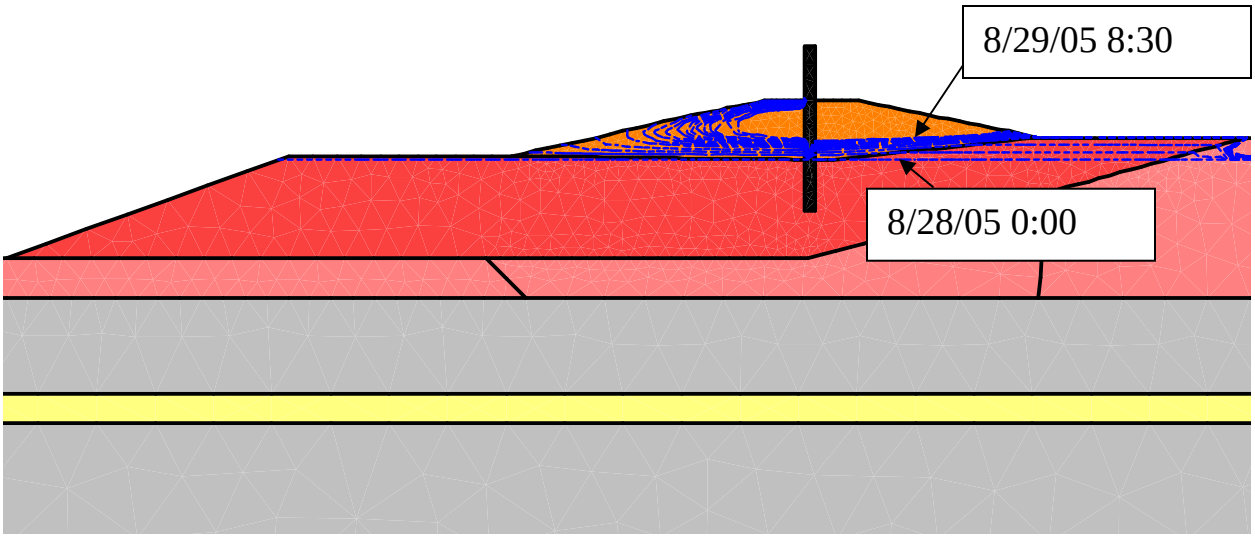


Figure 8-10. Transient model results. Advancement of the computed phreatic surfaces.

8.3 BAYOU DUPRE

8.3.1 Steady State

A baseline steady state analysis was performed using a water elevation of +18.0 feet (NAVD88). These elevations were taken as reference of maximum development of pore pressures. Figure 8-11 shows the cross section and mesh used for the analyses in this section.

From Figures 8-12 and 8-13 it can be observed that the flow regime is dominated by the presence of the permeable shell materials, inside and below the embankment. Marsh has a smaller influence on the hydraulic behavior of the system. The black arrows represent velocity vectors

of subsurface flow, with the largest arrow with a value of over 20 feet/day. Figure 8-13 considers the Marsh deposit to have a permeability of 10^{-6} cm/s, and Figure 8-15 uses 10^{-3} for the same layer.

It is clearly seen that when the Marsh is treated as a clay (Figure 8-12), there is a substantial head loss before the centerline of the EBSB is reached; whereas the head loss is smaller and more uniform when the marsh is assigned a higher permeability, therefore making heads and pore pressures larger in this stratum. The concentrated black arrows run inside the shell deposits.

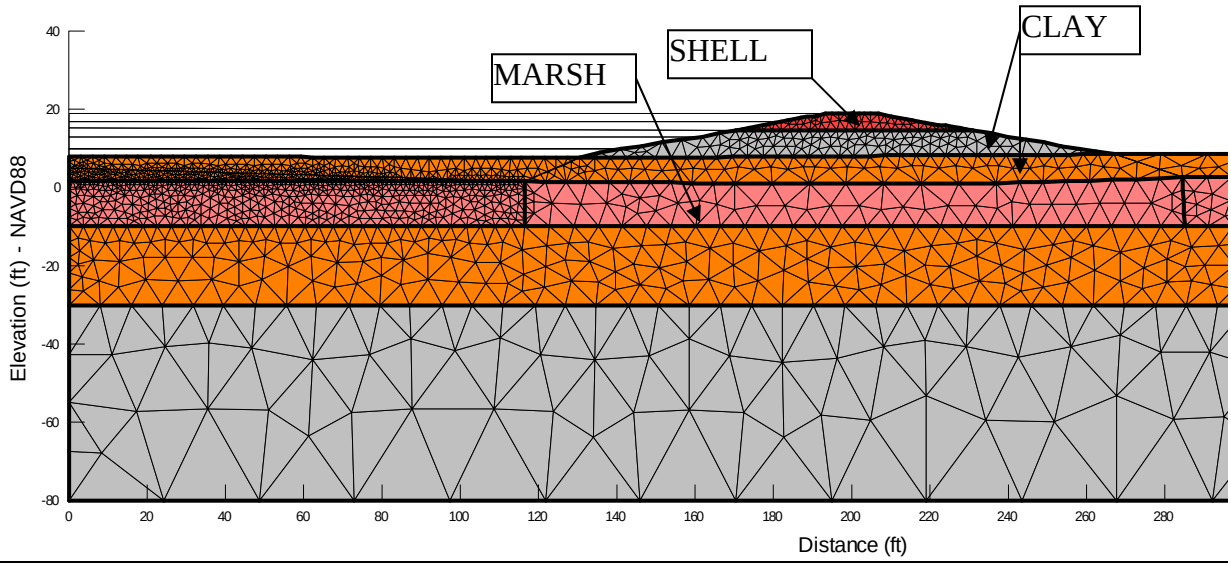


Figure 8-11. Cross section and mesh used for the analyses at Bayou Dupre.

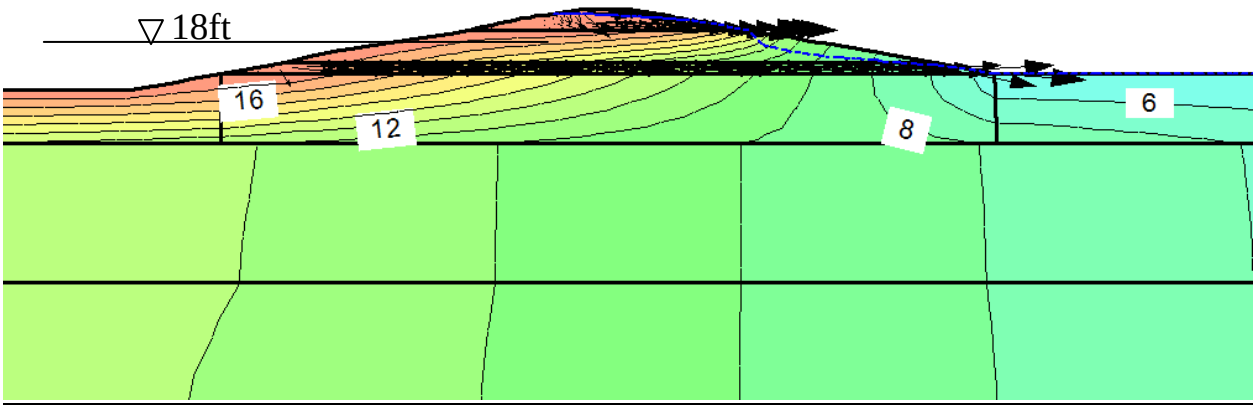


Figure 8-12. Steady State Total Heads at Bayou Dupre. Storm surge at +18ft. Marsh k: 10^{-6} cm/s.

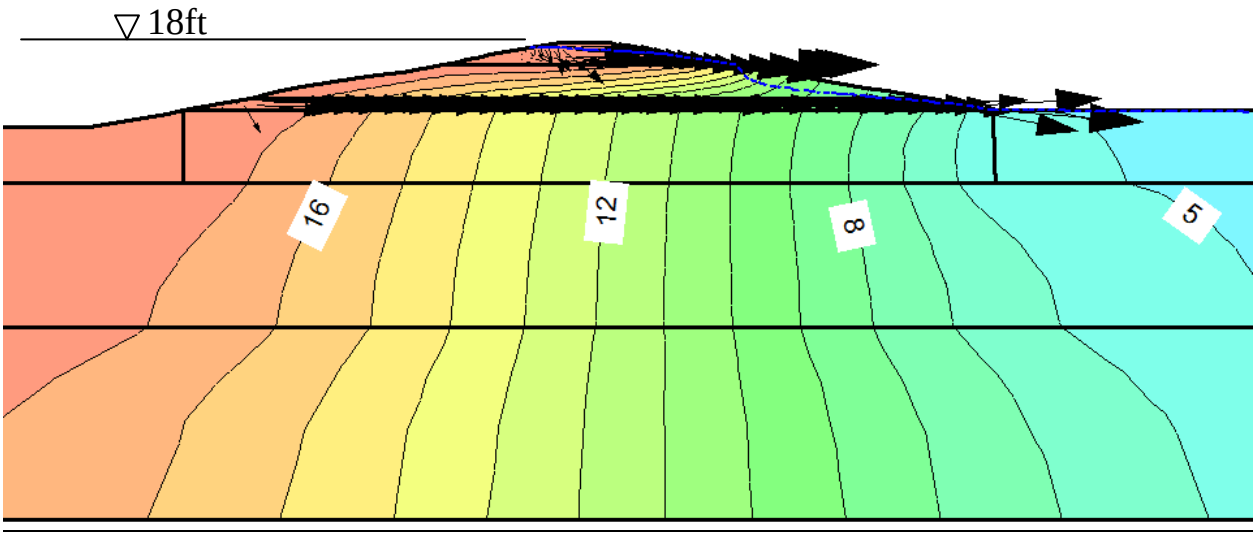


Figure 8-13. Steady State Total Heads at Bayou Dupre. Storm surge at +18ft. Marsh k: 10^{-3} cm/s.

Figure 8-13a shows a "plan view" of the seepage analysis, showing the top of EBSB composed by shell materials, and the transition to the sheet pile. Note the concentrated flow around the sheet pile, which can start erosion and scour around the structure.

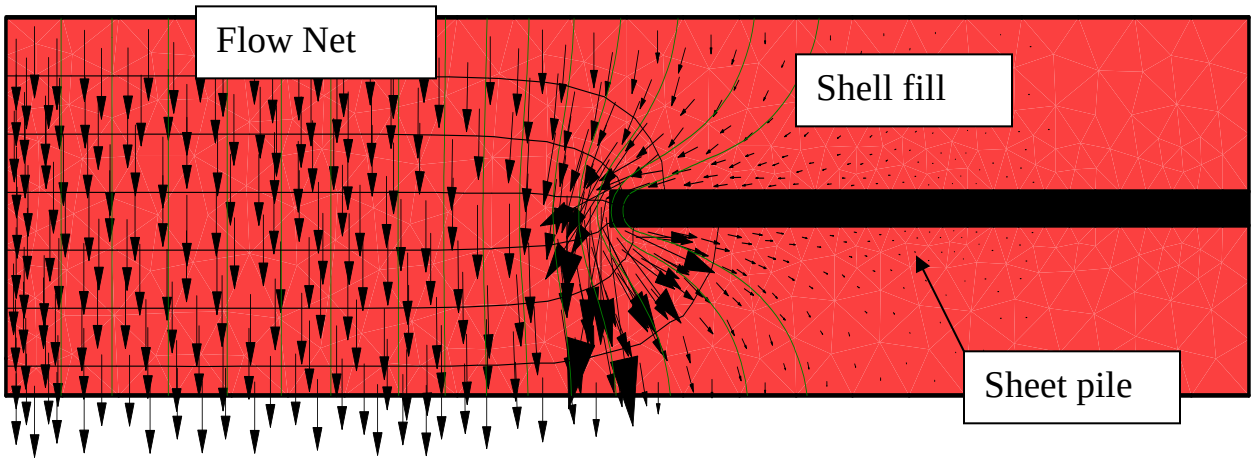


Figure 8-13a. Plan view showing effects of overtopping in the transition.

8.3.2 Transient Models

The same procedure used for Bayou Bienvenue was followed for the analyses of Bayou Dupre. Figure 8-14 shows the evolution of pore water pressure at the center of the Marsh layer below the levee fill.

According to this figure and the stratigraphic profile, there seems to not be a potential for underseepage in the Marsh materials beneath the toe of the levee, since computed pore pressures within this layer are lower than the overburden stress, and hydraulic gradients are lower than 0.5.

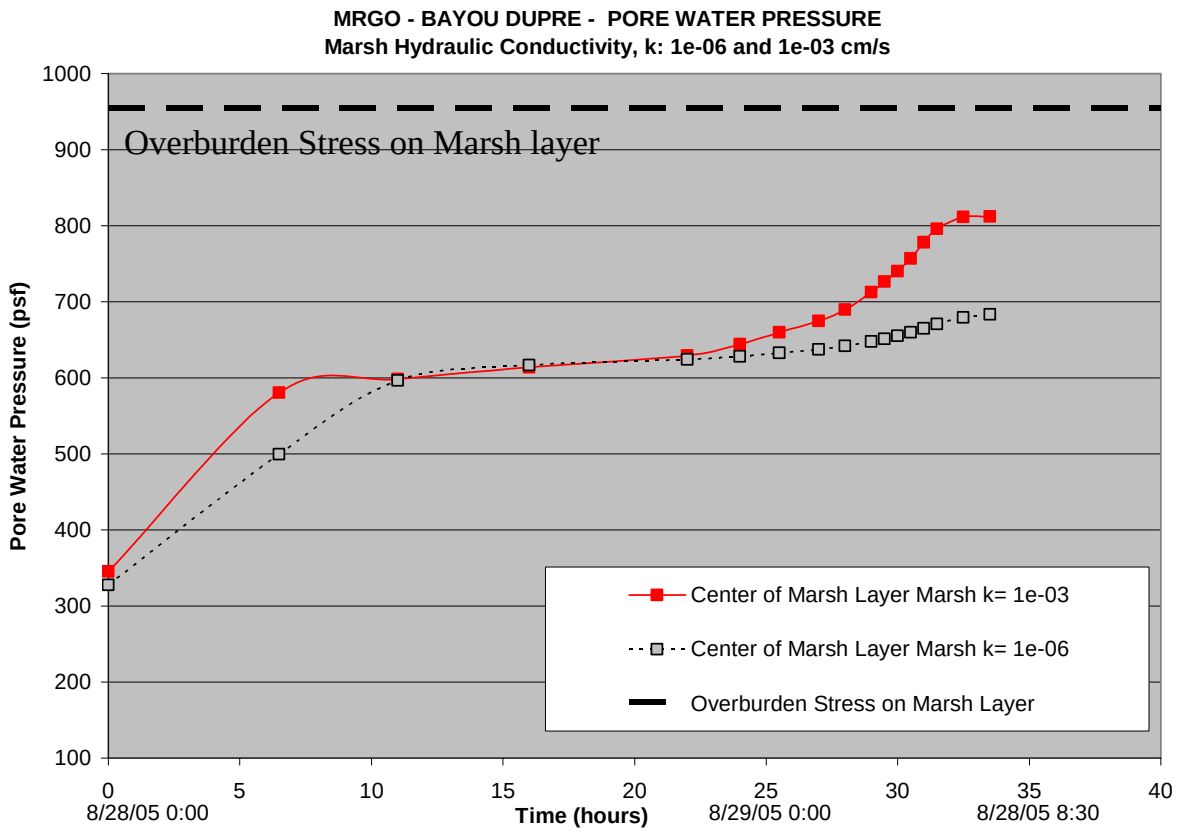


Figure 8-14. Transient model results. Pore pressure versus time near the toe of the levee.

In Figure 8-14, the lines of calculated pore pressure are compared to the total overburden stress at the same point. Blowout failure does not seem plausible at this site given the computed pore pressures. Figures 8-15 and 8-16 show the transient model results on August 29, 2005 8:30a.m., in terms of total heads and hydraulic gradients for the case of Marsh hydraulic conductivity of 10^{-3} cm/s.

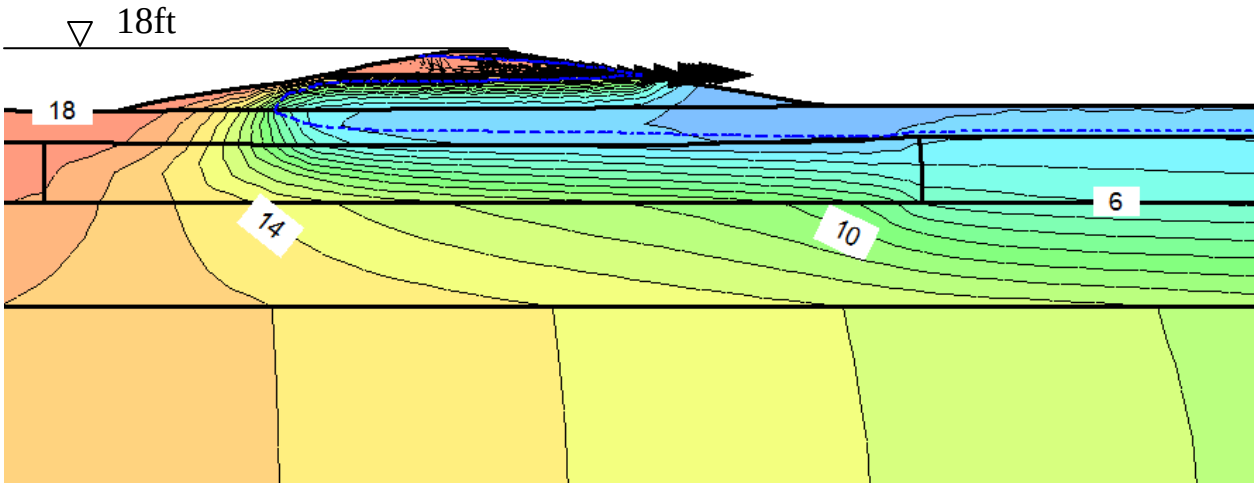


Figure 8-15. Transient model results. Total Heads on August 29, 2005 8:30. Marsh k: 10^{-3} cm/s.

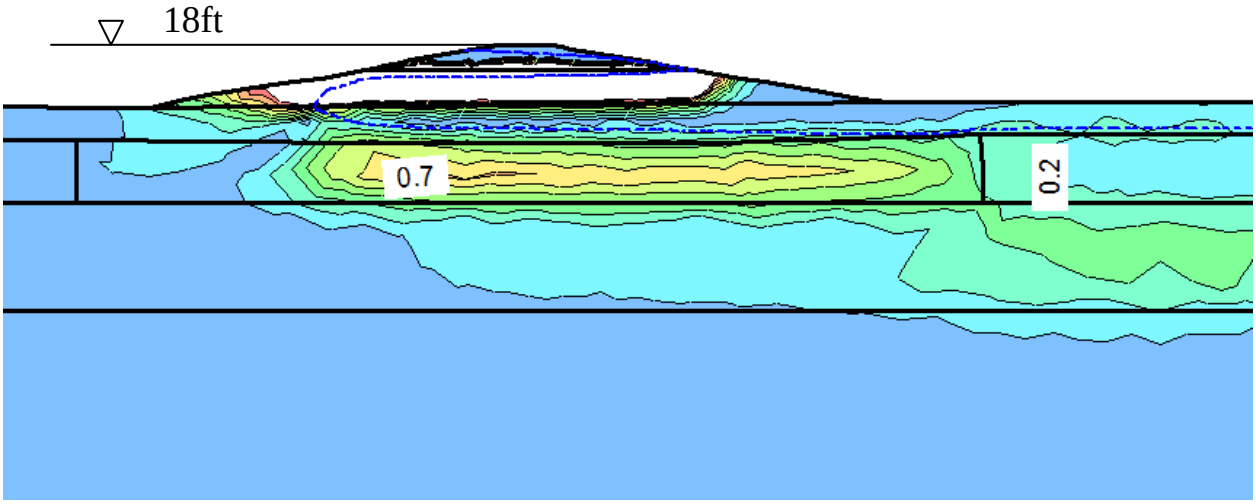


Figure 8-16. Transient model results. Hydraulic Gradients on august 29, 2005 8:30.

8.4 Potential for Piping Initiation

According to Fell et al. (2005), four conditions must exist for internal erosion and piping to occur:

- There must be a seepage flow path and a source of water

- There must be erodible material within the flow path and this material must be carried by the seepage flow
- There must be an unprotected exit from which the eroded material may escape and
- For a pipe to form, the material being piped, or the material directly above, must be able to form and support "roof" for the pipe.

These four conditions exist on the analyzed sections at Bayou Bienvenue and Bayou Dupre. A concentrated leak in the embankment (Fell et al., 2005) may be initiated by cracking due to differential settlements, hydraulic fracturing, and soft zones with high permeability as well as piping modes associated to the presence of conduits or walls.

It is useful to break up the process of internal erosion and piping into four phases – initiation and continuation of erosion, progression to form a pipe, and formation of a breach. This is shown in Figure 8-17 for piping through foundation.

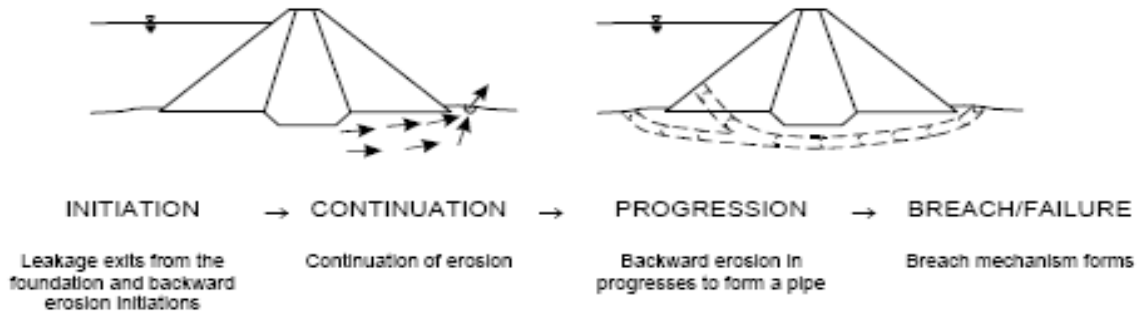


Figure 8-17. Piping in the foundation. Source: Fell et al., 2001.

The transient seepage analyses show that piping through the shell materials will initiate at water elevations lower than the maximum surge sustained by the EBSB for the sites at Bayou Bienvenue and Bayou Dupre. As early as midnight to 3:00 am on August 29, 2005 (storm surge

at between +7 feet and + 9 feet) for Bayou Bienvenue, and 6:00 am of the same day (storm surge elevation at +13.5 feet) for Bayou Dupre.

Another way to determine the initiation of internal erosion and piping has been proposed by Ojha (2003). This theoretical formulation is based on a thorough literature review and development of a new relationship between the fluid characteristics flowing through the porous media, and the properties of the grains that comprise the media.

Thevanayagan and Nesarajah (1998) used capillary models to calculate a value of critical head that could start particle movement. Their equation was re-arranged by Ojha (2003) as follows:

$$L / Hcrit = \frac{\gamma w}{4c}\left(1 - \frac{\gamma w * d^4}{512\mu^2 g * L}\frac{c}{\gamma w}\right) \qquad 8\text{-}1$$

$$c = 0.25\pi\eta\left(\frac{2d^3}{kL}\right)^{1/3}$$

where,

L= seepage length (base of the EBSB), (cm)

Hcrit= critical head for internal erosion initiation (cm)

γw= unit weight of water (gr/cm^3)

c= coefficient (unitless)

d= particle diameter (cm)

k= hydraulic conductivity, (cm/s)

μ= Viscosity of fluid – dynamic (gr/cm-s)

υ= Viscosity of fluid –absolute (gr/cm-s)

g= acceleration of gravity (cm/s^2)

n= porosity (unitless)

The calculations using this formulation were then corroborated by using another relationship based on capillary forces to initiate piping in coarse grained soils. This formula differs from

equation 8-1 mainly because it included the porosity *n* term, hugely important for coarse grained soils.

$$Hcrit = a\frac{(1-n)^2}{n^3} + b$$
$$a = \frac{150\upsilon}{gd}\left(\frac{L}{d}\right)\sqrt{\frac{cd}{\rho}} \qquad \text{8-2}$$
$$b = \frac{cd}{2\gamma w}$$
$$c = 0.25\pi\eta\left(\frac{2d^3}{kL}\right)^{1/3}$$

The above equations determine the potential for piping and internal erosion as a critical head value (units of length) above which the erosive processes would begin. The calculated critical heads were then compared to the heads computed by the seepage software in order to assess the triggering of internal erosion in the shell soils.

Given the limited information on shell permeability and porosity, parameters of medium to coarse sand were assigned to the shell materials and a suite of diameters (0.2 to 1.0cm) were assumed for the calculations.

Tables 8-4 and 8-5 and Figures 8-24 and 8-25 show the results of the foregoing formulations.

INTERNAL EROSION TRIGGERING FOR SHELL FILLS

Shell d50	Hcrit
(cm)	(ft)
1	9
0.9	5.9
0.8	3.4
0.7	2.55
0.6	1.95
0.5	1.5
0.4	1.2
0.2	0.8

n= 0.4

Table 8-4. Results from the critical head calculations for Bayou Bienvenue.

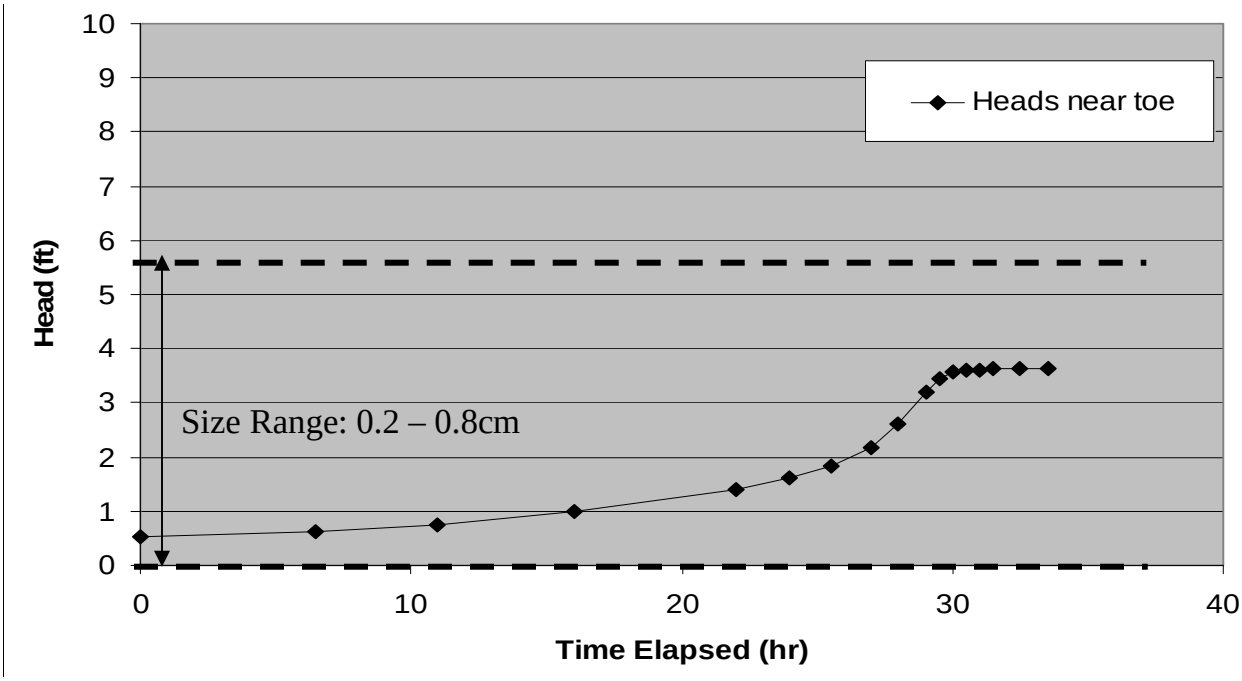


Figure 8-24. Critical head values for Bayou Bienvenue.

From the values in Table 8-3 and Figure 8-24 it can be seen that internal erosion could have been triggered for shell particles of diameters between 0.1 and 0.8 cm. Particles in excess of 0.9cm would have not moved.

INTERNAL EROSION TRIGGERING FOR SHELL FILLS

Shell d50	Hcrit
(cm)	(ft)
1	11
0.9	8.75
0.8	6.5
0.7	6
0.6	5.5
0.5	5.1
0.4	4.5
0.2	3.2

n= 0.4

Table 8-5. Results from the critical head calculations for Bayou Dupre.

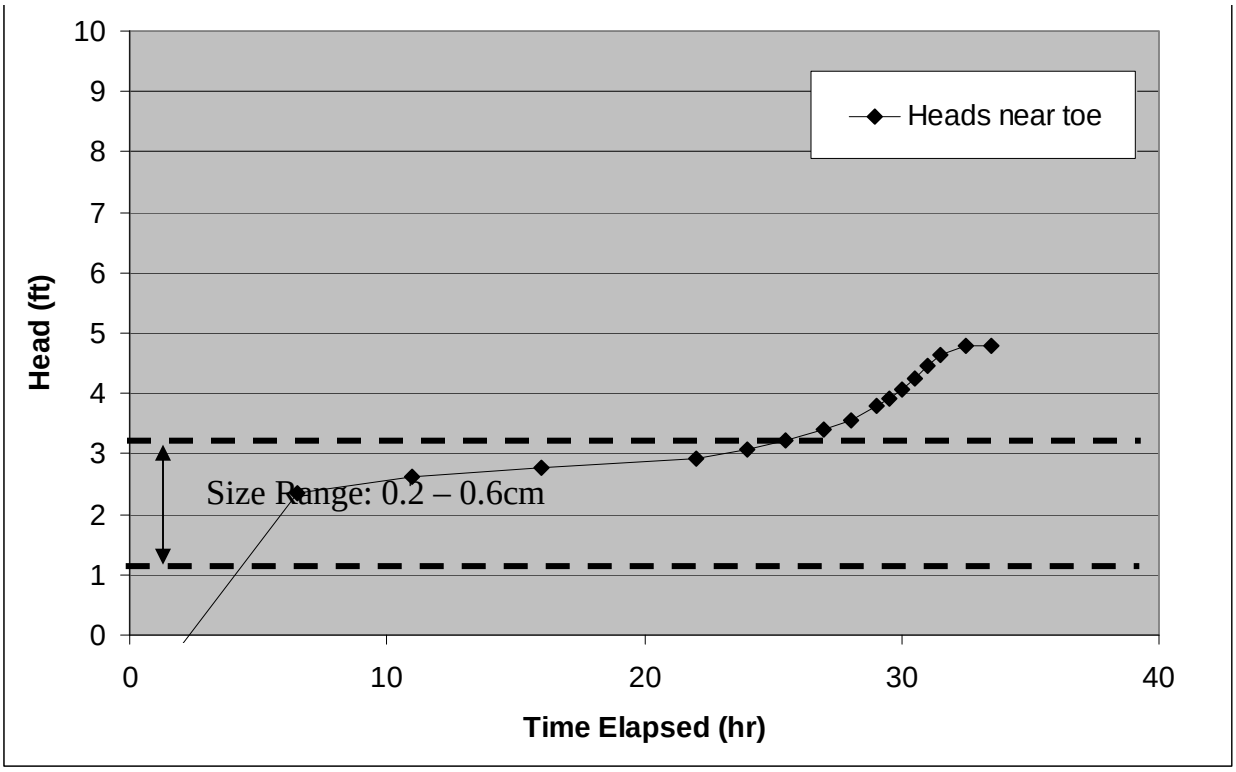


Figure 8-25. Critical head values for Bayou Dupre.

Similarly, for Bayou Dupre the range of particle diameters that could have started to move with the computed hydraulic heads near the toe of the EBSB is somewhat narrower than Bayou Bienvenue, with particles from 0.2 to 0.6 cm in diameter.

8.5 Seepage Modeling Observations

8.5.1 Bayou Bienvenue

The seepage analyses at Bayou Bienvenue show that no uplift instability occurred during Katrina, since no fine-grained deposits (blankets) were present. However, the permeable shell materials placed to backfill excavations played a key role in allowing the flow to seep and exit at the EBSB toe, allowing for the initiation of erosion and loss of material that resulted in the breach.

Marsh permeability is not the dominant factor in the behavior of the EBSB at Bayou Bienvenue, since the hydraulic conductivity of this material is 100 times less permeable than the shell fills.

It can be concluded that the gap between the bottom of the excavation and the tip of the sheet pile, which allowed a window of shell fills to be open is the most important factor in the hydraulic instability of the EBSB at Bayou Bienvenue.

Piping and internal erosion could have been triggered as early as midnight on August 28, 2005. The calculated pore pressures are in excess of the total overburden stress near the toe of the EBSB, and the critical heads for the shell materials found beneath the embankment are exceeded by the heads produced by the surge.

8.5.2 Bayou Dupre

The interpreted section at Bayou Dupre shows that the embankment and the first couple of feet of foundation soils contain shell materials. As with the analyses at Bayou Bienvenue, no blanket layers are subject to uplift pressures, since the pore pressure and heads generated by the models are dissipated by permeable materials daylighting at the toe of the EBSB (shell and marsh). The triggering of erosion processes inside the shell layers is given by high internal velocities (>20 feet per day) and concentrated flow. The shell materials present at this section are also associated with the backfilling of the excavations of the control structures.

Piping and internal erosion are triggered as early as 6:00 am on August 29, 2005. The calculated pore pressures are in excess of the total overburden stress near the toe of the EBSB, and the critical heads for the shell materials found beneath the embankment are exceeded by the heads produced by the surge.

9. STABILITY ANALYSES

9.1 Model Description

The next step on this study was to perform stability analyses that incorporate the hydraulic responses of the EBSB. The software SLOPE\W was employed for this task. The pore water pressures generated by the SEEP\W finite element analyses are used by SLOPE\W when calculating Factors of Safety.

The software uses Limit Equilibrium Analysis to calculate Factors of Safety by a number of methods. The results included in this report use the Spencer method.

The critical failure surface search resulted in shallow non-circular surfaces, passing through shell layers for Bayou Bienvenue and Bayou Dupre.

9.2 BAYOU BIENVENUE

Figure 9-1 shows the stability analysis section for Bayou Bienvenue from DM05 (USACE, 1968). The computed factors of safety are larger than 1.3 for all surfaces considered, and soil properties are similar to the calculated in this report. The difference is centered in the hydraulic response of the section, which seems to have been calculated with hydrostatic pore pressures in the original DM.

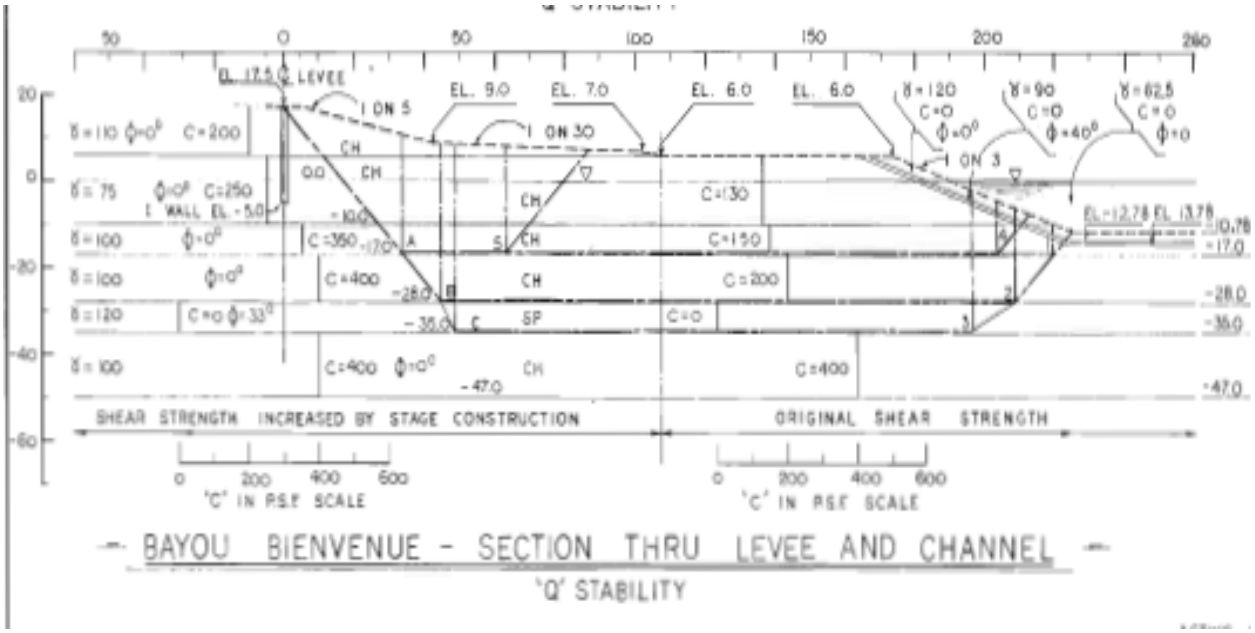


Figure 9-1. Original stability analysis for Bayou Bienvenue. Source: DM05, plate IV-36.

The soil properties determined in Section 6 (median values) for unit weight, undrained shear strengths and friction angles were assigned to the different layers according to the stratigraphic interpretation on Section 4. As discussed in previous sections, the available borings are "toe" borings, therefore no shear strengths under the EBSB embankment are available. Based on recent reports (IPET, 2006; ILIT, 2006), the difference between shear strengths between the two conditions ranges between 30 % and 60 %. A 40% increase will be applied to the Marsh beneath the EBSB embankment. The clays beneath the Marsh deposit are believed to be nearly normally consolidated, and a value of Su/p' (ratio of undrained shear strength to effective overburden pressure) of 0.26 will be applied, as calculated for the clays (of the same mineralogy and origin) in the canal district (ILIT, 2006). Table 9-1 shows the material properties used in the slope stability calculations. Figure 9-2 shows the input geometry used for the stability analyses of the breach.

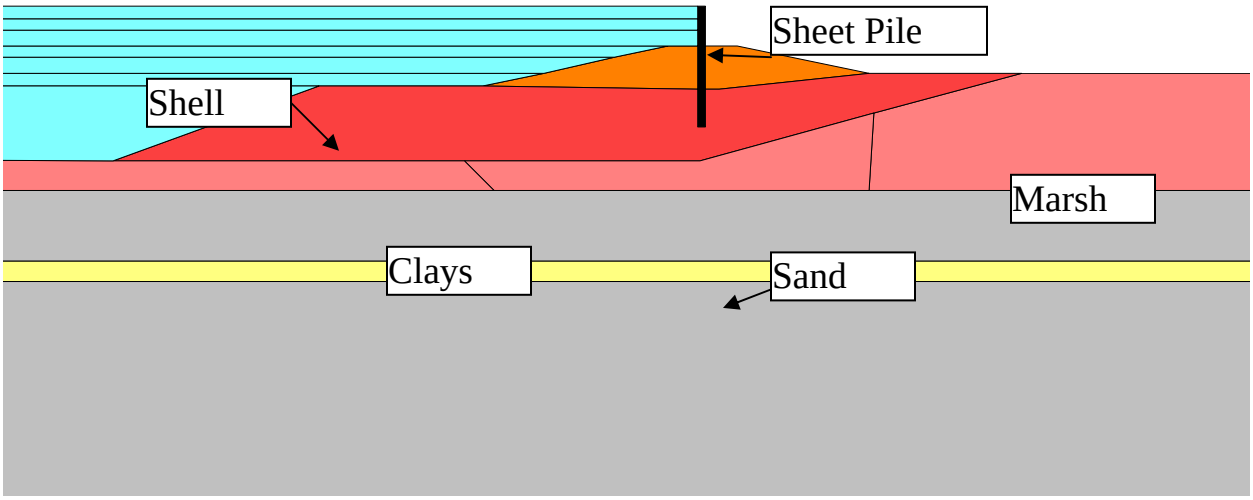


Figure 9-2. Analysis cross section for Bayou Bienvenue.

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

Table 9-1. Material properties used for the slope stability models, Bayou Bienvenue.

Figure 9-2 shows the "Mean values" of results for factor of safety for the breach at Bayou Bienvenue. The results from the analyses show that the section is marginally stable when the water is located at the sheet pile top (Elevation +18 feet).

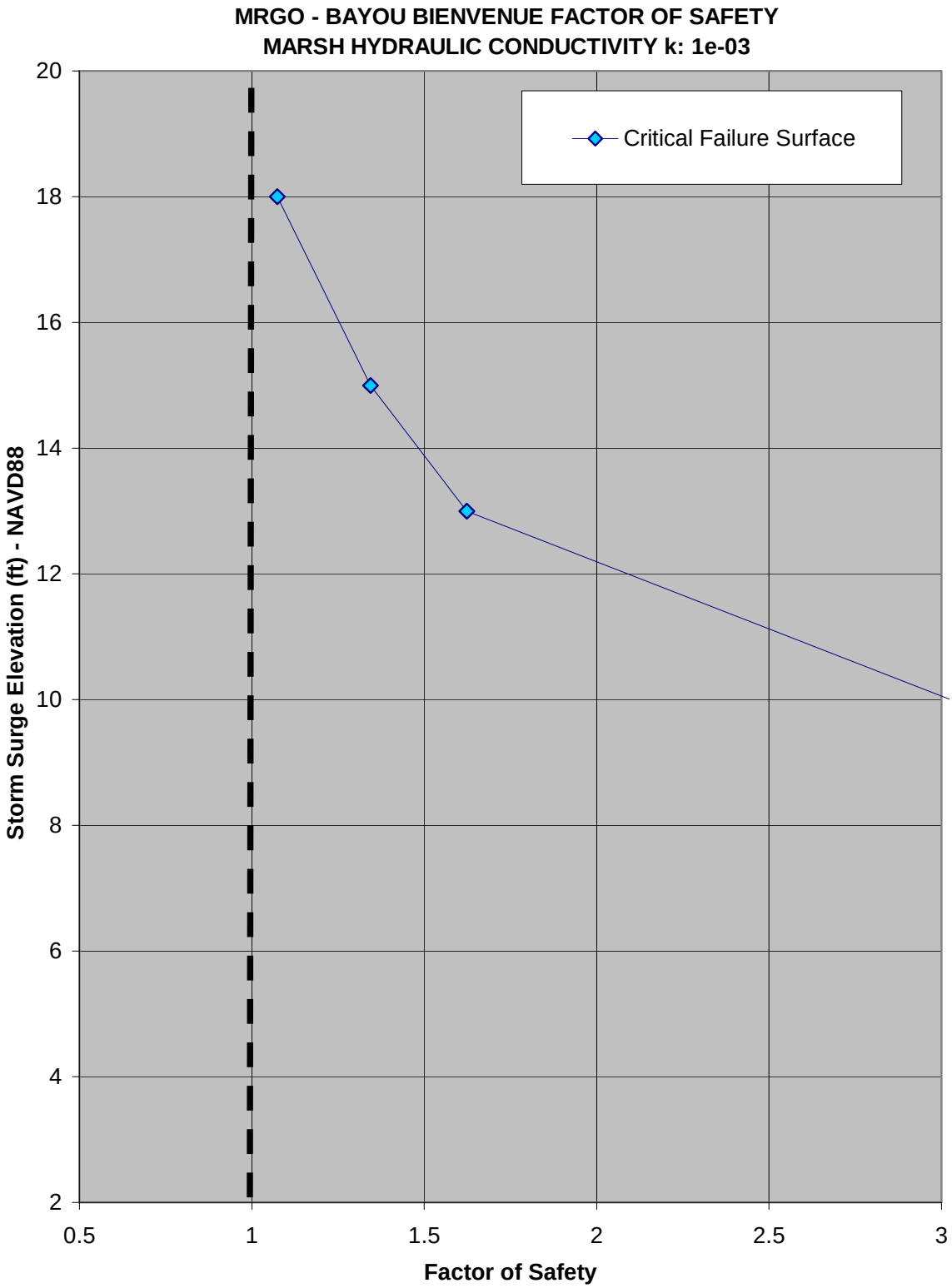


Figure 9-3. FOS versus storm surge elevation for Bayou Bienvenue.

Critical failure surfaces were determined to be non-circular (wedge like) surfaces starting at the waterside slope and progressing towards the landside toe along the shell and Marsh deposits. Figure 9-3 shows the critical surface using Spencer's method for a storm surge of 18ft.

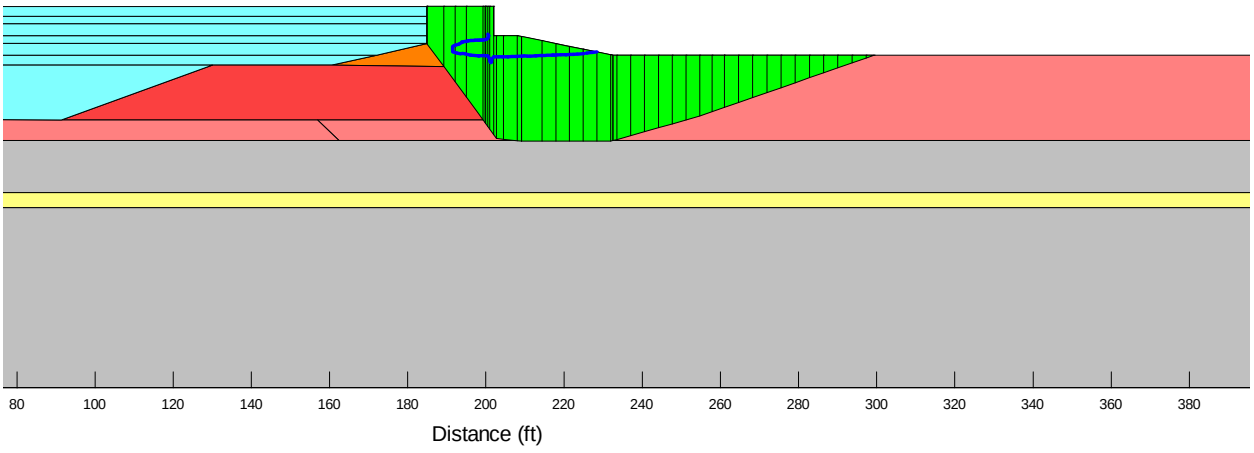


Figure 9-4. Critical shear surface for Bayou Bienvenue, storm surge 18 feet, FOS: 1.06.

9.3 BAYOU DUPRE

Figure 9-5 shows the stability analysis section for Bayou Bienvenue from DM05 (USACE, 1968). The computed factors of safety are larger than 1.3 for all surfaces considered, and soil properties are similar to the calculated in this report. The difference is centered in the hydraulic response of the section, which seems to have been calculated with hydrostatic pore pressures in the original DM.

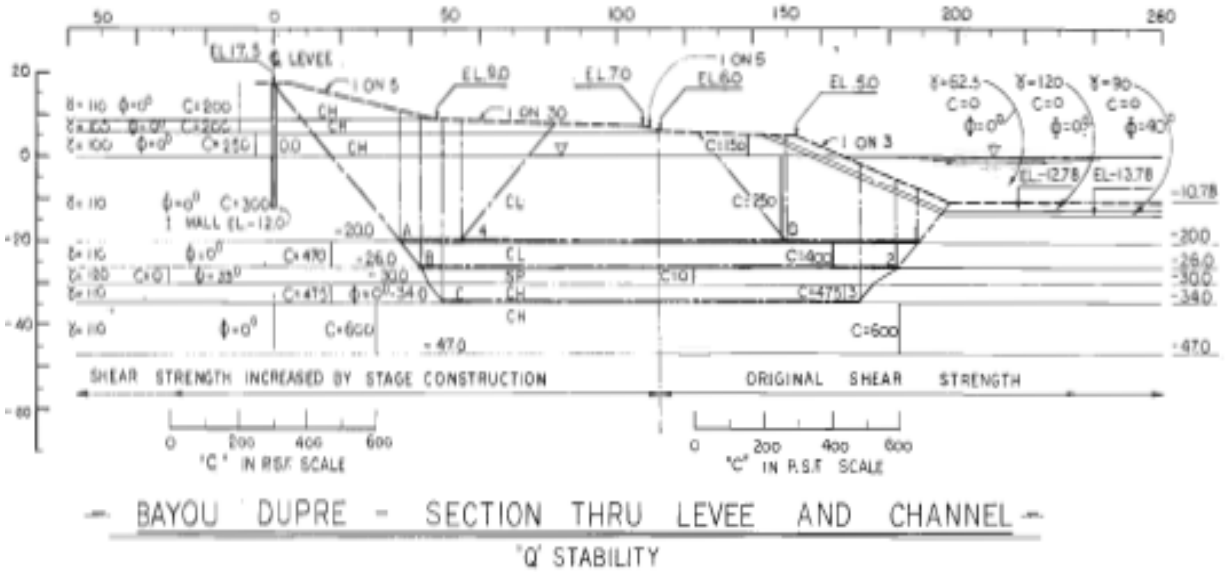


Figure 9-5. Original stability analysis for Bayou Bienvenue. Source: DM05, plate IV-36.

The soil properties determined in Section 5 (median values) for unit weight, undrained shear strengths and friction angles were assigned to the different layers according to the stratigraphic interpretation on Section 4. As discussed in previous sections, the available borings are "toe" borings, therefore no shear strengths under the EBSB embankment are available. Based on recent reports (IPET, 2006; ILIT, 2006), the difference between shear strengths between the two conditions ranges between 30 % and 60 %. A 40 % increase will be applied to the Marsh beneath the EBSB embankment. The clays beneath the Marsh deposit are believed to be nearly normally consolidated, and a value of Su/p' of 0.26 will be applied, as calculated for the clays (of the same mineralogy and origin) in the canal district (ILIT, 2006).

Table 9-2 shows the material properties used in the slope stability calculations. Figure 9-5 shows the input geometry used for the stability analyses of Bayou Dupre.

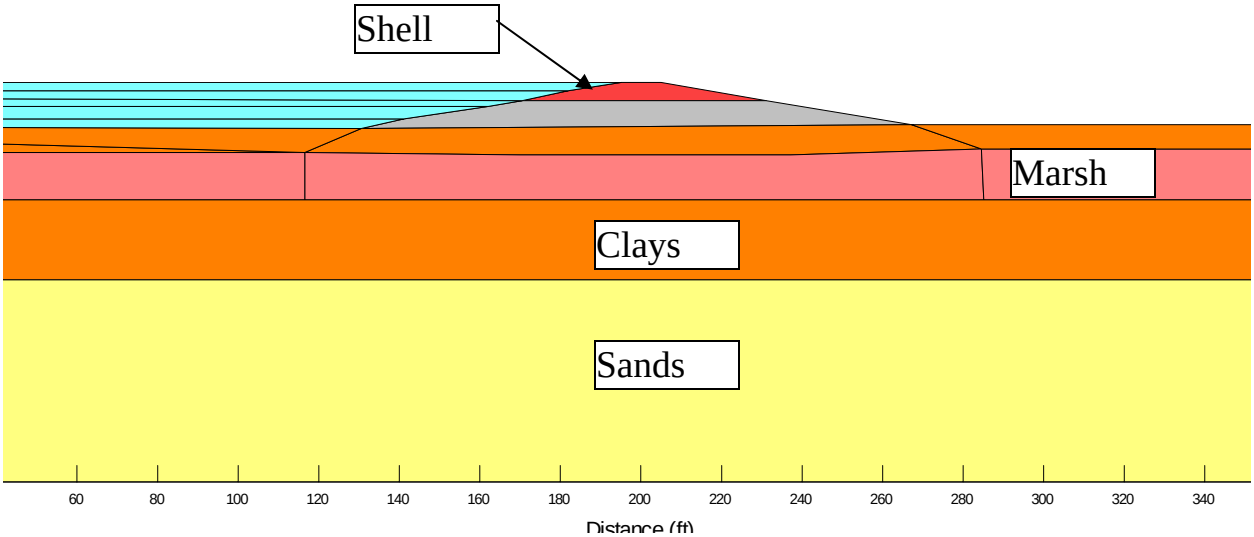


Figure 9-6. Analysis cross section for Bayou Dupre.

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

[1]: Assumed values

Table 9-2. Material properties used for the slope stability models, Bayou Dupre.

Figure 9-6 shows the "Median value" results of factor of safety for Bayou Dupre, and Figure 9-7 shows the critical shear surface for a storm surge of 18ft, approximately at 8:30am on August 29, 2005. There appears to not be a clear mass instability problem on this section. Computed factors of safety are above 1.4 for all potential surfaces considered.

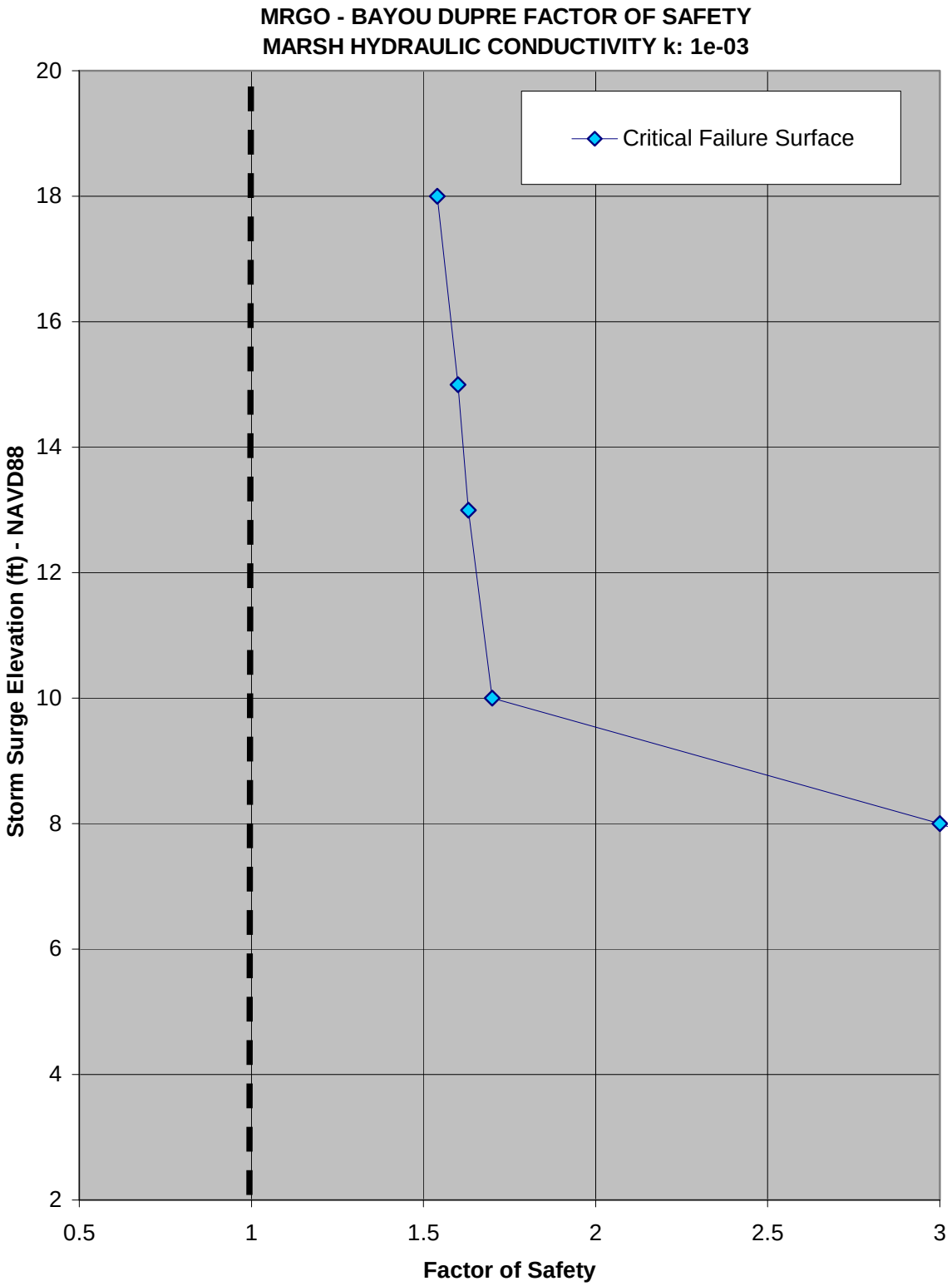


Figure 9-7. Computed Factors of Safety for Bayou Dupre.

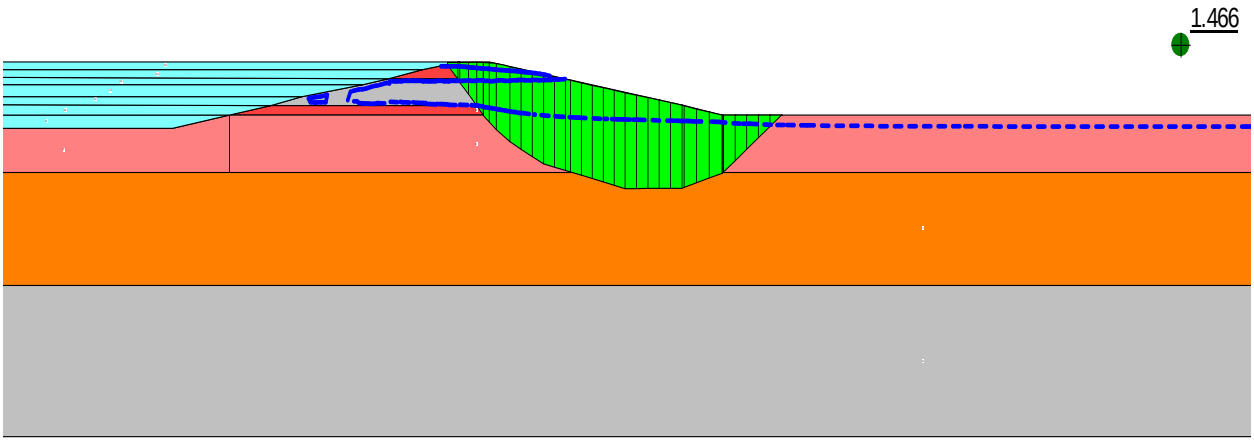


Figure 9-8. Critical shear surface for Bayou Dupre, storm surge 18ft, Factor of Safety = 1.46.

10. SUMMARY OF RESULTS AND CONCLUSIONS

The seepage analyses show that pore pressures develop quickly and the shell materials present at Bayou Bienvenue and Bayou Dupre are susceptible to piping and internal erosion several hours before the storm surge reaches the top of the EBSB.

The analyses at Bayou Bienvenue showed that the EBSB could have been undermined through the shell materials underneath the sheet pile, starting the erosive process (as early as midnight to 3:00 am on August 29, 2005) that resulted in the breach. The slope stability analysis show that a non-circular failure surface through the underlying marsh deposit is marginally stable with the water surface at the top of the sheet pile, however, the erosion process could have initiated before.

Bayou Dupre shows a similar hydraulic behavior, with the potential for internal erosion of the shell materials present within the embankment and in the foundation soils, but triggering several hours later (6:00am on the same day). The stability analyses did not recreate a mass instability

process in this EBSB. The failure can be attributed to erosion from overtopping and internal erosion from underseepage.

The stability analyses indicate that the EBSB wing-wall tie-in at the Bayou Bienvenue navigation structure is the only one of the sites that shows a marginal factor of safety against instability. This breach appears to have developed as a result of a combination of seepage – undermining and stability.

The stability analyses do not indicate that the EBSB wing-wall tie-in at the Bayou Dupre navigation structure approached factors of safety that would indicate instability. This brach appears to have developed as a result of seepage-undermining and overtopping erosion – scour.

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