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ASSESSMENT OF HURRICANE PROTECTION IN SOUTHEAST LOUISIANA

APPENDIX L
ENGINEERING INVESTIGATIONS

ASSESSMENT OF HURRICANE PROTECTION IN SOUTHEAST LOUISIANA
ENGINEERING INVESTIGATIONS APPENDIX

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L9	Reach 4 Typical Sections Ground El. 5.0 – Geotextile
L10	Reach 4 Typical Sections Ground El. 5.0 – Soil Mix
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LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT ENGINEERING INVESTIGATIONS APPENDIX

SECTION L.1 GENERAL

L.1.1 BACKGROUND

More than 1,000 people in Louisiana alone were killed by the wind and flooding brought by Hurricane Katrina in August 2005. Less than a month later, Hurricane Rita pushed into southwestern Louisiana, adding to the death toll and causing flooding throughout the coastal area. The entire nation felt the repercussions of this massive human tragedy. The structural damage brought by flooding in coastal Louisiana was so extensive it will take years for the region to fully recover.

This Engineering Appendix outlines the preliminary work of the Corps of Engineers New Orleans District to develop a plan to provide significantly improved flood protection to citizens and infrastructure in south Louisiana from the Mississippi to Texas borders. It includes plan formulation and hydrodynamic modeling, geotechnical investigations, levee design, structural design and coastal restoration features.

L.1.1.1 Currently Authorized Projects

The existing hurricane protection projects in the study area have varying levels of protection. The Lake Pontchartrain and Vicinity and West Bank and Vicinity projects are designed to provide protection for the Standard Project Hurricane (SPH). The SPH was developed by the National Weather Service in 1959 to serve as the design storm for hurricane protection projects in the New Orleans area. The SPH was developed prior to the creation of the Saffir-Simpson Hurricane Intensity Scale. In general terms, the SPH design storm is equivalent to a low strength Category 3 storm on the Saffir Simpson Scale. The Larose to Golden Meadow and the New Orleans to Venice projects are designed to provide 100-year frequency protection. In general, this provides the equivalent of Category 2 protection levels.

L.1.2 ALTERNATIVE ALIGNMENTS

L.1.2.1 General

Alternative alignments were developed in conjunction with other federal, state and local representatives. Details on how these alignments were developed can be found in Part 4 of the main report. The alignments shown in this report do not, however, preclude the discovery and design of alternative alignments that would represent significant improvements to the schedule, budget, and overall effectiveness of the project. It is

expected that a more thorough analysis will be conducted and that improvements will be incorporated during the design phase.

Confirmation of or amendments to the final construction alignment will be made with due deliberation and consideration of many significant factors, including the costs and benefits to the level of protection, the environment, constructability, schedule, hydraulics, economic and cultural resources, and other general and local issues. Innovative engineering will also be a key component in the alternative alignment analysis.

L.1.2.2 Modeling Alignments

All proposed alignments were distilled into five separate alignments for the purpose of ADCIRC modeling (Figures L-1 thru L-5) for the PTR. It was determined that the permutations of these alignments would not result in a significant difference in design elevations from the alignments chosen for modeling.

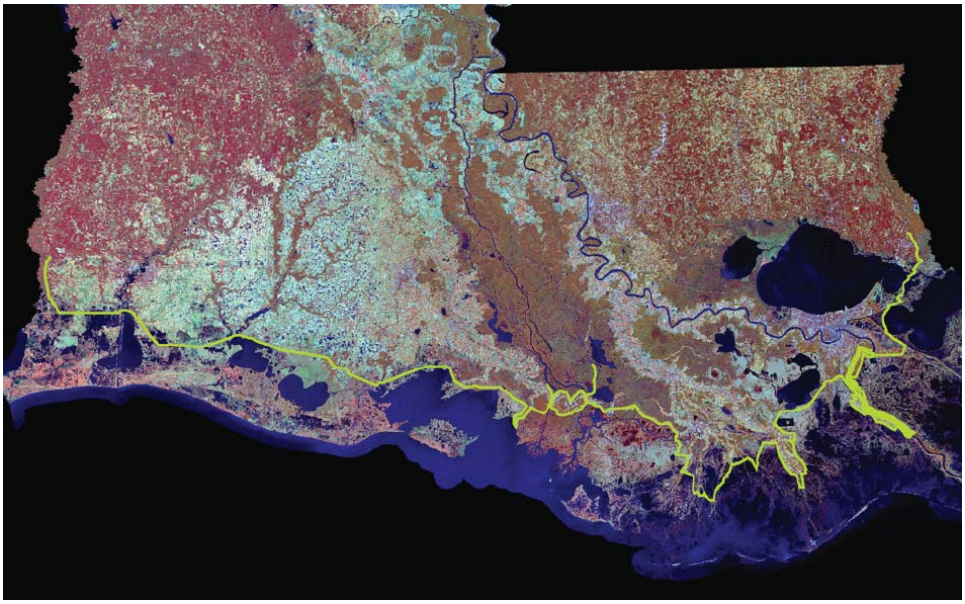


Figure L-1. Modeling Alignment 1.

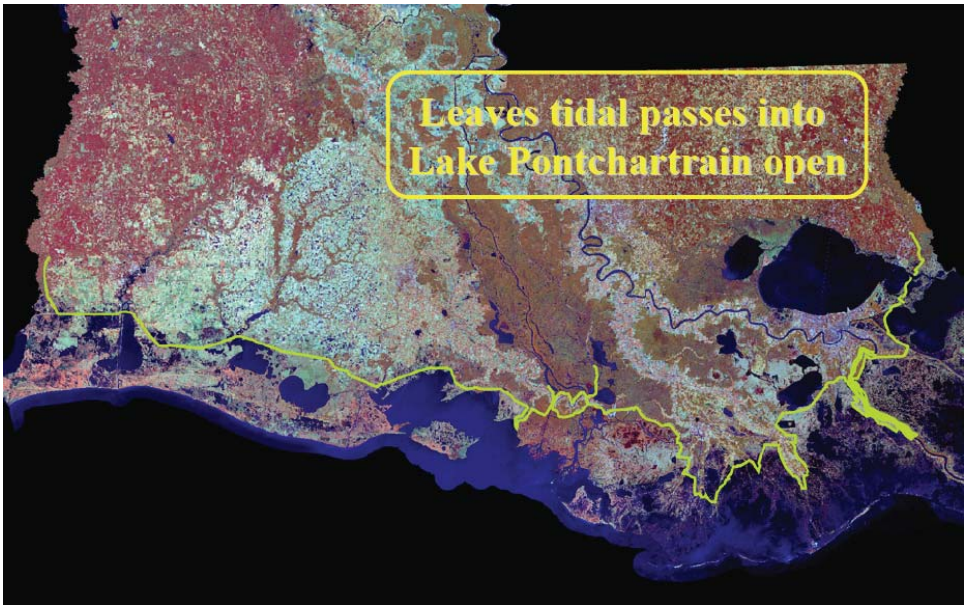


Figure L-2. Modeling Alignment 2.

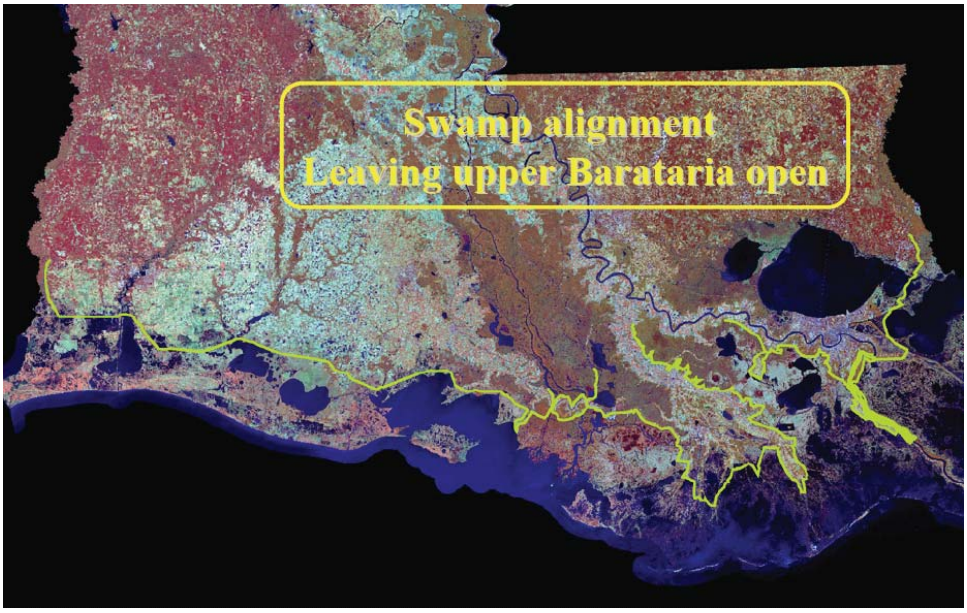


Figure L-3. Modeling Alignment 3.

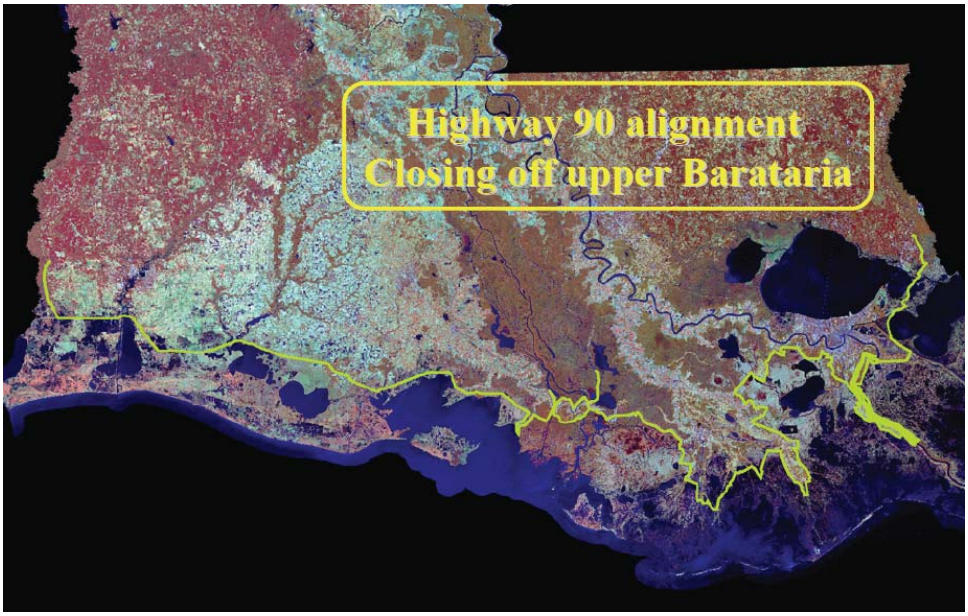


Figure L-4. Modeling Alignment 4.

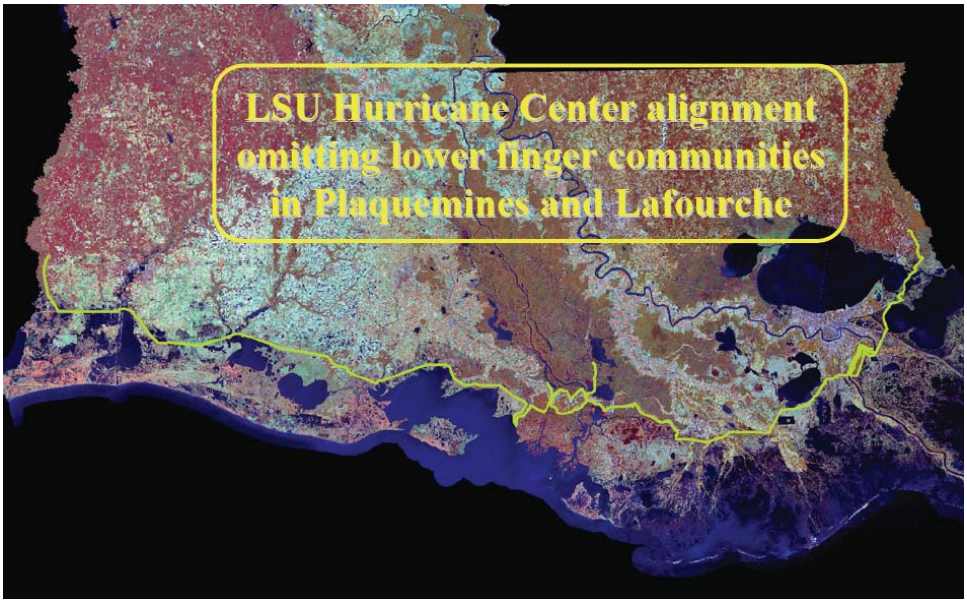


Figure L-5. Modeling Alignment 5.

L.1.2.2.1 Modeling Alignment 1

Alignment 1 places a barrier levee across Lake Pontchartrain along Hwy 90 and along the southwest edge of Lake Borgne. It then generally follows the alignments of existing and proposed hurricane protection projects to Morgan City and runs along the GIWW in the western part of the state. The protection ties into high ground near the Texas border. This alignment includes barrier structures across Lake Pontchartrain in the Rigolets and Chef Passes.

L.1.2.2.2. Modeling Alignment 2

Modeling Alignment 2 is the same as Alignment 1 except that the Rigolets and Chef Menteur Passes, the two major channels that connect Lake Pontchartrain to Lake Borgne and the Gulf of Mexico, are left open.

L.1.2.2.3 Modeling Alignment 3

Modeling Alignment 3 is the same as Alignment 1 except that the interior of the Barataria Basin is left open and highly developed areas are protected by levees extending up the wetland interface on both sides of the basin.

L.1.2.2.4 Modeling Alignment 4

Modeling Alignment 4 is the same as Alignment 1 except that the levee across the Barataria Basin follows Hwy 90.

L.1.2.2.5 Modeling Alignment 5

This alignment follows Alignment 1 except that it cuts through St. Bernard Parish and leaves out most of Plaquemines Parish, the Larose to Golden Meadow area and the Morganza to the Gulf area.

L.1.2.3 Design Alignments

Results from the modeling efforts were used to develop 7 different alignments for design purposes. These alignments incorporate permutations of the modeling alignments identified during the planning process. These alignments are shown in Figures L-6 through L-11 and described below.

Design Alignment 1 is the same as Modeling Alignment 1.

Design Alignment 2 is the same as Modeling Alignment 2.

Design Alignment 3 is the same as Modeling Alignment 1 with the alignment across the edge of Lake Pontchartrain following the existing railroad instead of Hwy 90. Other modifications include changes to tie into the boundary levees further south of the GIWW and cutting off the extreme north west corner of the proposed Morganza to the Gulf alignment.

Design Alignment 4 is the same as Modeling Alignment 4.

Design Alignment 5 has a modification to the Lake Pontchartrain barrier which runs along I-10, cuts off portions of New Orleans East and runs along the interior levee in St. Bernard Parish. The levee then follows the existing Plaquemines Parish levees and

the Barataria Basin interior alignment from Modeling Alignment 3. The levee then follows the existing Larose to Golden Meadow alignment and the Morganza to the Gulf alignment with some modifications to leave out areas. It then follows Modeling Alignment 1 to Morgan City. Between Morgan City and Freshwater Bayou the protection follows the interface with the developed areas. West of Freshwater Bayou the protection follows the south side of the GIWW and the wetland interface.

Design Alignment 6 is the same as Modeling Alignment 5.

Design Alignment 7 is the same as Modeling Alignment 1 until it reaches Freshwater Bayou. West of this point the protection follows the 10 ft contour.

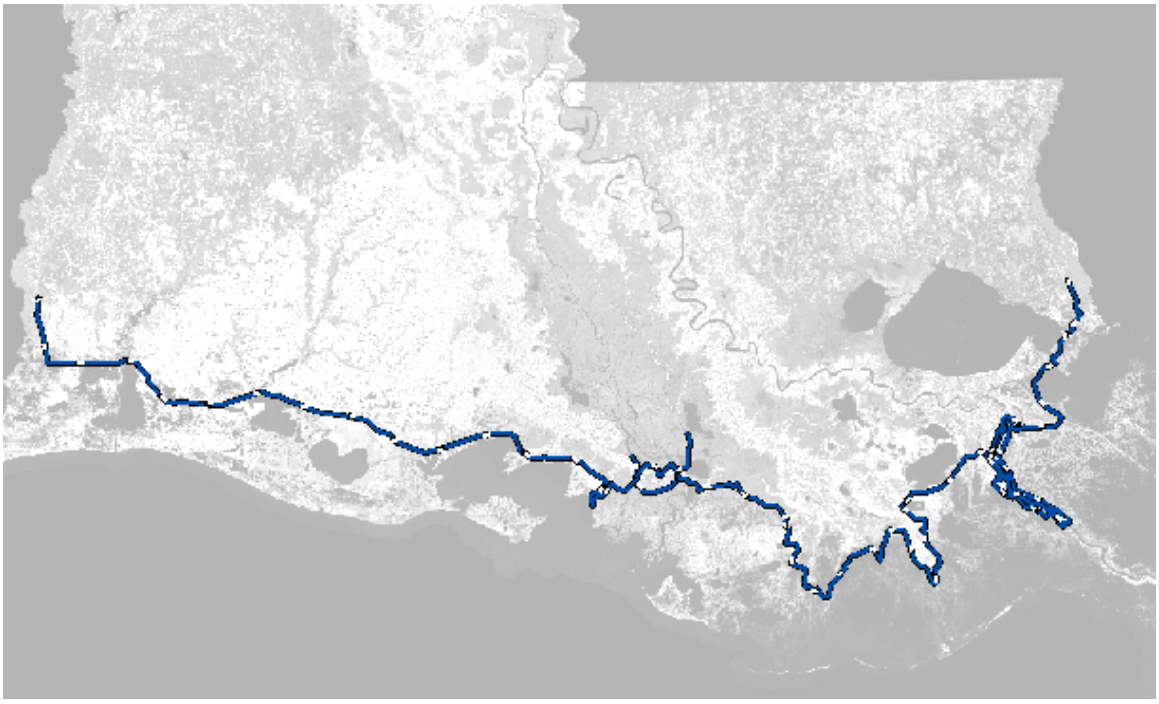


Figure L-6 Design Alignments 1 and 2

Figure L-7 Design Alignment 3

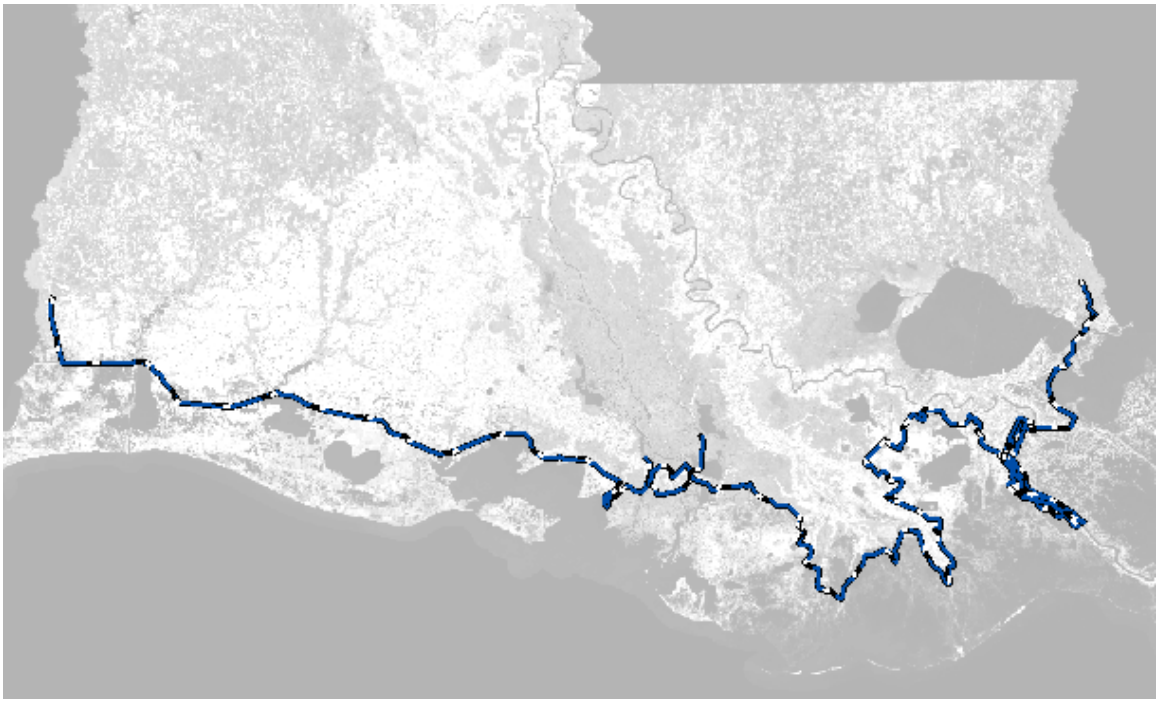


Figure L-8 Design Alignment 4

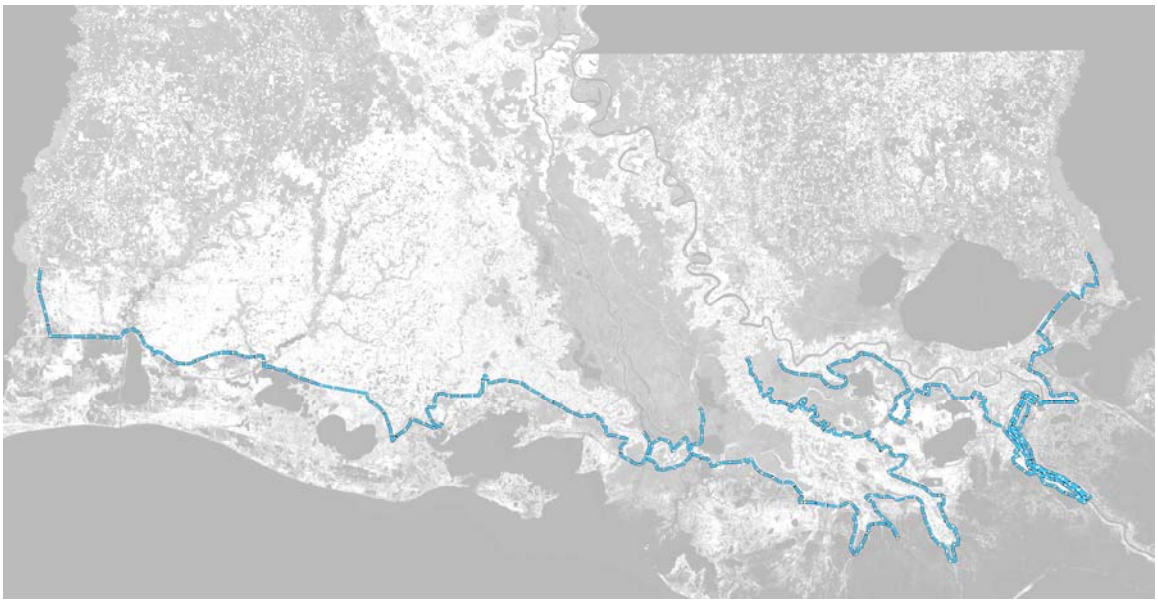


Figure L-9 Design Alignment 5

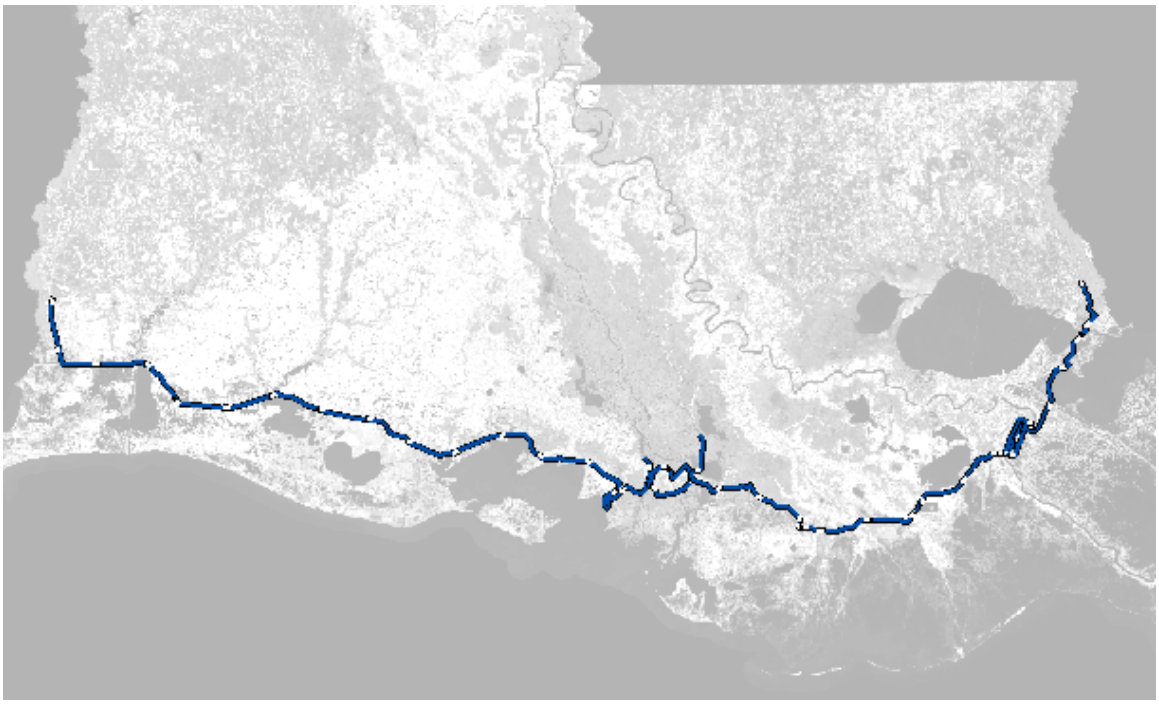


Figure L-10 Design Alignment 6

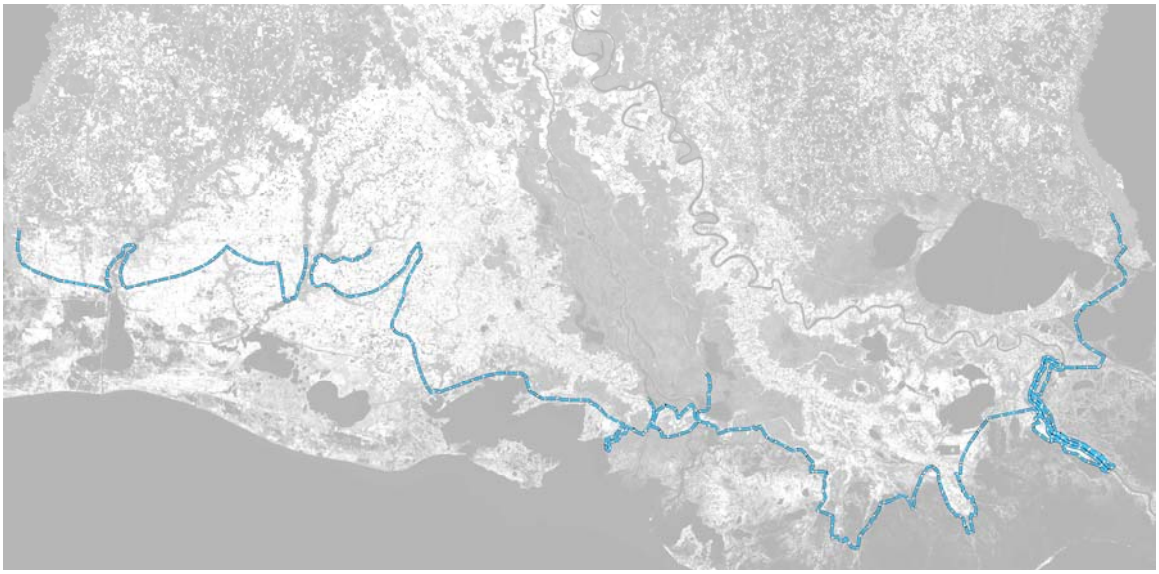


Figure L-11 Design Alignment 7

L.1.3 PROJECT REACHES

L.1.3.1 Planning Units

The project area was divided into different Planning Units based on hydrologic boundaries. These Planning Units extend as follows:

1. The area east of the Mississippi River (Pontchartrain Basin)
2. The area from the Mississippi River west to Bayou Lafourche (Barataria Basin)
3a. The area west of Bayou Lafourche to Bayou de West (Eastern Terrebonne Basin)
3b. The area west of Bayou de West to Freshwater Bayou (Atchafalaya River Influence Area)
4. The area west of Freshwater Bayou to the Sabine River (Chenier Plain)

The project area was further broken into sections based on geotechnical conditions and alternative alignments for ease in comparing alternatives. These reaches can be seen on Plates 1 and 2.

L.1.4 INNOVATIVE TECHNICAL APPROACHES

The need for a more thorough survey and analysis of innovative methods to find solutions to the challenges of the project became apparent because of the magnitude of the task of providing “Category 5” protection for the State of Louisiana. To achieve this goal a workshop (Engineering Technical Approaches and Innovations Workshop) was held in Vicksburg, Mississippi on March 2 and 3, 2006. More than a hundred geotechnical and structural engineers and experts from industry, academia and government took part in this

workshop. Participants came from across the U.S., Sweden, France, Great Britain and the Netherlands.

The workshop resulted in a variety of recommendations for improving the constructability and cost of levees in south Louisiana, including deep soil mixing, geofoam blocks, pop-up barriers, lightweight fill, hollow-core precast concrete sections, and use of geogrid. The summary report from this conference is included as Appendix G. Only one of these innovative recommendations has been examined for the Preliminary Technical Report: improvement of in situ foundation soils using deep soil mix columns.

The team will continue to review, study and evaluate recommendations from the workshop as the project progresses. Some of the participants from the workshop may be engaged to lend their expertise to advance suggested plans into schematic designs. More options will be explored and presented in more detailed analyses in the Final Technical Report.

L.2 HYDROLOGY AND HYDRAULICS

L.2.1 EXISTING CONDITIONS

L.2.1.1 Climate

The climate of the study area is subtropical with a strong maritime character. Proximity to the Gulf of Mexico provides a source of abundant moisture and rainfall. The 30-year annual normal temperature (1971-2000) for the study area is 68.4 °F. The 30-year annual normal precipitation (1971-2000) is 63.6 inches, according to the National Oceanic and Atmospheric Administration (NOAA). Prevailing wind direction is southerly during much of the year. Tropical storms and hurricanes in the summer produce the highest winds in the area.

L.2.1.2 Tides

Tides are mostly diurnal in the study area, with the exception of Terrebonne Basin, where tides can be diurnal or semi-diurnal, and the Atchafalaya Bay, where tides are mainly semi-diurnal. The normal range of tides along the coast portion of the study area is between 0.9 and 2.0 ft. (NOAA). In Lake Pontchartrain, the normal tidal range is 0.6 feet. In shallow water bodies, wind driven tides can be substantial and exceed the range of the astronomical tide.

L.2.1.3 Floods and Storms of Record

L.2.1.3.1 General

The official hurricane season for the Atlantic Ocean, the Caribbean Sea, and the Gulf of Mexico is from 1 June to 30 November. The peak of the season is from mid-August to late October. However, deadly hurricanes can occur anytime in the hurricane season.

The **Saffir-Simpson Hurricane Intensity Scale** is a scale classifying hurricanes by the velocity of their sustained winds. It was developed in 1969 by civil engineer Herbert Saffir and National Hurricane Center director Bob Simpson. Classifications are used to gauge the likely damage and flooding a hurricane will cause upon landfall. The Saffir-Simpson Hurricane Scale is used only to describe hurricanes forming in the Atlantic Ocean and northern Pacific Ocean east of the International Date Line.

The Saffir-Simpson Hurricane Intensity Scale is a 1 to 5 rating based on the hurricane's present wind speed. The magnitude of a hurricane storm surge is highly variable due to the effects of the slope of the continental shelf and the shape of the coastline in the landfall region. Storm surge height is not reliably predicted by the Saffir-Simpson scale. Table L-1 is developed from information by the NOAA National Hurricane Center/Tropical Prediction Center. Storm surge values presented in the table are general values; hurricanes with different ratings can have similar surge heights.

Table L-1
Saffir-Simpson Scale

Saffir-Simpson Category	Maximum sustained wind speed [1]			Minimum central pressure [2]	Storm surge [3]	
	mph	m/s	kts	mb	ft	m
1	74-95	33-42	64-82	> 980	3-5	1.0-1.7
2	96-110	43-49	83-95	979-965	6-8	1.8-2.6
3	111-130	50-58	96-113	964-945	9-12	2.7-3.8
4	131-155	59-69	114-135	944-920	13-18	3.9-5.6
5	156+	70+	136+	< 920	19+	5.7+

Notes:

1. The National Hurricane Center uses a 1 min averaging time for reporting the sustained (i.e. relatively long-lasting) winds. The maximum sustained wind mentioned in the advisories that NHC issues for tropical storms and hurricanes are the highest 1 min surface winds occurring within the circulation of the system. These "surface" winds are those observed (or, more often, estimated) to occur at the standard meteorological height of 10 m (33 ft) in an unobstructed exposure (i.e., not blocked by buildings or trees).
2. Classification by central pressure was ended in the 1990s, and wind speed alone is now used. These estimates of the central pressure that accompany each category are for reference only. 1 inch of mercury = 33.86 millibars.
3. These surge values are for reference only. The actual storm surge experienced will depend on offshore bathymetry and onshore terrain and construction.

From National Oceanic and Atmospheric Administration, Office of Oceanic and Atmospheric Research, http://www.aoml.noaa.gov/hrd/tcfaq/D1.html (last accessed May 2006).

NOAA has assigned categories to all hurricanes that have made direct hits on the mainland U.S. coastline since 1851. Between 1851 and 2004, 49 hurricanes Category 1 or greater have made direct hits on the Louisiana coastline. Eighteen hurricanes were Category 3 or greater.

L.2.1.3.2 Storm Descriptions

Many storms have affected the study area over the last 100 years. Several of the most intense hurricanes that affected southeastern Louisiana are described briefly in the following paragraphs.

The hurricane of 29 September 1915 had a central pressure of 944 mb (27.87 inches of mercury), an average forward speed of 10 knots, 94 miles per hour (mph) 5-minute sustained wind velocity, and an extreme wind velocity of 106 mph, based on National Weather Service records. This hurricane approached the study area from the south. Storm tides rose to 10-12 feet along the Mississippi-Louisiana coast. High water marks indicate that the water level in Lake Pontchartrain in the vicinity of West End rose to a height equivalent to 6.1 feet, National Geodetic Vertical Datum of 1929 (NGVD), and to 13 feet (NGVD) in the vicinity of Frenier.

The hurricane of 19 September 1947 had a central pressure of 967 mb (28.57 inches) and an average forward speed of 16 knots. The National Weather Service Station at Armstrong International Airport reported a special wind observation of 98 mph from the northeast and gusts to 112 mph. The direction of approach of this hurricane was approximately from the southeast. Stages at the Lake Pontchartrain West End were recorded at 5.5 feet (NGVD), and on the north shore at Mandeville at 7.0 feet (NGVD). Shell Beach experienced a high water elevation of 11.2 feet (NGVD).

On 9 September 1965, Hurricane Betsy, until recently the most destructive storm of record in Louisiana, struck the study area from the southeast. Betsy had a central pressure of 941 mb (27.79 inches0, an average forward speed of about 17 knots, maximum-recorded wind speed was 105 mph, and gusts estimated to be in excess of 160 mph. On the Mississippi River at New Orleans, the stage rose from a normal of about 4 feet to a crest stage of 12.40 feet (NGVD) on 9 September. At St. Francisville, 266 miles above Head of Passes on the Mississippi River, the water rose as much as 8 feet above normal. In Lake Pontchartrain, stages reported were: 5.5 feet at West End, 6.5 feet at Mandeville and 12.1 feet at Frenier, all in NGVD. A stage of 9.3 feet (NGVD) was recorded at Shell Beach and a high-water mark of 11.6 feet (NGVD) was established at the junction of the Inner Harbor Navigation Canal and the Mississippi River Gulf Outlet. A stage of 9.2 feet (NGVD) was recorded at the Paris Road Bridge across the Gulf Intracoastal Waterway (GIWW). In Rigolets and Chef Menteur Passes stages of 9 feet (NGVD) were measured.

Hurricane Camille, one of the most intense and destructive storms ever, struck the Mississippi Gulf Coast, but severely affected the lower Louisiana delta region. Camille had a central pressure of 901 mb (26.61 inches) and an average forward speed of about 13 knots. This storm made landfall just east of the Louisiana state line on August 18, 1969. As a Saffir-Simpson category 5 storm, Camille's maximum winds were estimated at 160 mph with gusts up to 200 mph. Peak stages include 11.1 feet (NGVD) at Shell Beach on 17 August and 10.0 feet (NGVD) at GIWW at Paris Road Bridge in the Mississippi River Gulf Outlet (MRGO) on 18 August. In Lake Pontchartrain at West End, a peak stage of 5.2 feet (NGVD) was recorded.

Hurricane Juan in October 1985 set several peak stage records due to its prolonged stay along the Louisiana coast. A minimal category 1 hurricane, Juan moved slowly and erratically along the Louisiana coast for several days. Juan's maximum sustained wind was 86 mph. Record stages set by this storm include: 8.0 feet (NGVD) at Bayou Bienvenue at Floodgate (East) on 28 October; 3.5 feet (NGVD) at Bayou Dupre at Floodgate (west) on 30 October; and 6.9 feet (NGVD) on Bayou Terre Aux Boeufs at Delacroix on 28 October. In Lake Pontchartrain the maximum high stages of record were measured at West End, 6.1 feet (NGVD) and at Mandeville, 7.6 feet (NGVD).

Hurricane Georges skirted the eastern marsh zones of Plaquemines and St. Bernard Parishes before making landfall near Biloxi, Mississippi on 28 September 1998 with maximum sustained winds of 105 mph. At landfall, Hurricane Georges had a minimal central pressure of 965 mb (28.5 inches) and a forward speed of approximately 7 mph. Rainfall totals averaged nearly 5 inches over metropolitan New Orleans. Maximum stages for this storm were 9.1 feet (NGVD) at the GIWW near Paris Road Bridge, 6.6 feet (NGVD) at the Mississippi River Gulf Outlet at Shell Beach, 4.6 ft. (NGVD) on Bayou Terre Aux Bouefs at Delacroix, 5.8 ft. (NGVD) at the Rigolets Lake Pontchartrain, 4.7 ft. (NGVD) at Mid Lake on the Lake Pontchartrain Causeway, 7.4 ft. (NGVD) at Bayou Bienvenue, 5.5 ft. (NGVD) at Lake Pontchartrain at West End, and 7.4 ft. (NGVD) at Bayou Bienvenue.

Hurricane Isidore weakened to a minimal tropical storm before entering the Gulf of Mexico. Isidore never redeveloped an inner core of strong winds prior to making landfall just west of Grand Isle on 26 September 2002. Belle Chasse Naval Air Station reported sustained winds of 58 mph with gusts of 69 mph. Isidore produced torrential rains throughout southeast Louisiana. Some of the storm totals were 18.50 inches at the New Orleans Algiers gage with 15.34 inches falling on the 26 September, Chalmette had a total of 14.82 inches with 11.02 on the 26th, the New Orleans Audubon gage had 11.83 inches with 9.21 inches observed on the 25th. As the storm continued north, it created a backlash tidal surge on Lake Pontchartrain that flooded the town of Mandeville and some subdivisions in St. Tammany Parish. Some of these maximum stages for this storm were 5.61 ft. (NGVD) at the Rigolets near Lake Pontchartrain, 6.0 ft. (NGVD) at Lake Pontchartrain at West End, and 6.84 ft. (NGVD) at Bayou Terre Aux Boeufs at Delacroix.

Hurricane Lili made landfall along the south-central coast of Louisiana near Intracoastal City on 3 October 2002 as a strong category 2 storm with winds of 120 mph and gusts of 150 mph. Areas along the storm's path received 8 inches or more of rain in parts of central Louisiana, with a band of 5 inches to 8 inches or more also occurring along the eastern portions of the southeast and Florida parishes. Heavy rains produced minor to moderate flooding along the lower sections of several rivers and streams. These include the Pearl, Tangipahoa, Tickfaw, and Amite Rivers in the Lake Pontchartrain Basin, as well as those in south central and southwestern Louisiana such as the Mermentau, Calcasieu, and Vermilion Rivers. Storm tide was reported as 10.6 ft. (NGVD) at Castille Pass near Morgan City, 12.3 ft. (NGVD) in Crewboat Channel near Calumet, 4.46 ft. (NGVD) in Grand Isle, 6.04 ft. (NGVD) at the Rigolets Lake Pontchartrain and 11.7 ft. (NGVD) in Vermillion Bay.

On 29 August 2005, Hurricane Katrina made landfall in Plaquemines Parish near Buras as a strong category 3 storm with winds of 127 miles per hour (mph). The central pressure at landfall, 920 mb (27.17 inches), was the lowest of record in the Atlantic Basin for a 127 mph storm and the fifth lowest of record overall. Gusts of over 100 mph were reported in New Orleans. The storm continued on its northerly track making its final landfall near Pearl River on the Louisiana/Mississippi border. The storm surge exceeded 18 ft. along the southeast Louisiana coast, overtopping and breaching protection levees, and flooding New Orleans metropolitan area, New Orleans East, and virtually all of Plaquemines and St. Bernard Parishes. St. Tammany Parish sustained significant wind damage and flooding. Jefferson Parish also experienced widespread flooding. Final storm damage will be in the billions of dollars, and casualties from Katrina, still incomplete, exceed 1300 dead. Katrina is one of the most costly and one of the deadliest storms on record. Most gages were damaged or destroyed by the storm. Storm rainfall accumulations were estimated to exceed 8 -10 inches along much of Katrina's path. Some of the estimated high water elevations (all at North American Vertical Datum of 1988 correction 2004.65 – NAVD88 2004.65) associated with the storm were 13.52 ft. at the Inner Harbor Navigation Canal Lock; 11.12 ft. inside the Coast Guard Station near the 17th Street Canal; 11.4 ft. on West Levee of the London Avenue Canal, north of Robert E. Lee Blvd; 18.7 ft. at the Bayou Bienvenue Floodgate; 12.6 ft. at the New Orleans Lakefront Airport; 18.7 ft. at Shell Beach; 14.7 ft. at Entergy Michoud Power Plant in New Orleans East; 10.9 ft. in St. Bernard Parish near the Courthouse; 16.0 ft. in Slidell at Highway 90 and Salt Bayou, and 14.8 ft. at Empire Lock. Grand Isle had a maximum tide reading of 12 ft. LMSL (Local Mean Sea Level). Katrina also sprung off 33 reported tornadoes as she tracked inland causing further damage to communities north of the Gulf coast.

As Hurricane Rita moved westward across the Gulf of Mexico during mid-September 2005, wind driven tides and surge raised water levels in Lake Borgne, Lake Pontchartrain, and the canals in New Orleans. The temporary repairs along the Inner Harbor Navigation Canal overtopped, and flooding occurred in some of the already heavily damaged areas of Orleans and St. Bernard Parishes. Additional flooding was reported in Slidell and Mandeville in St. Tammany Parish from high tides in Lake Pontchartrain. Rita made landfall on 24 September 2005 just east of the Texas/Louisiana

border near Johnson's Bayou with 120 miles per hour (mph) winds and a central pressure of 937 mb (27.67 inches). Rita's surge caused devastating damage all along the Louisiana and southeastern Texas coastal areas. Storm rainfall amounted to 2.07 inches in New Orleans, 9.30 inches in Baton Rouge, 6.24 inches in Lafayette, 8.13 inches in Cameron, and 7.68 inches in Lake Charles. Some of the Louisiana high water elevations were 8.7 ft. at Gueydan, 8.4 ft. at Lake Charles, 7.6 ft. at Moss Bluff, 4.0 ft. at Lake Arthur, 15.01 ft. at Cameron, 12.2 ft. at Windmill, and 10.4 ft. at Pecan Island (all readings at datum NAVD88 2004.65). Flooding in the Mermentau Basin remained for several weeks after the hurricane passed; unwatering of the basin was completed on 15th October. After coming ashore, Rita's circulation created an estimated 90 tornadoes that were reported in Alabama, Mississippi, Louisiana, and Arkansas.

L.2.2 HYDRODYNAMIC MODELING

L.2.2.1 General

Hydrodynamic modeling of storm surge and waves for the Preliminary Technical Report (PTR) was conducted to predict wave action at and water level response to several proposed levee alignments for input to engineering and design to assist in determination of preliminary levee height requirements and design. Due to time constraints on the preliminary report, a single screening storm was selected and simulated on ten separate tracks. It is recognized that the ten storm tracks do not provide comprehensive coverage to define water levels everywhere along the alignments. However, the results produced are augmented with engineering judgment to provide reasonable estimates for rough order of magnitude design purposes. The hydrodynamic response of the system with each proposed levee alignment is also compared to the existing condition to assist in the determination of unintended water level changes imparted by the proposed structures. This section describes the initial screening storm, the wind and atmospheric pressure modeling, the storm surge modeling, and the offshore and nearshore wave modeling. The model results were used to estimate maximum waves and water level along the five levee alignments. A more detailed description of this effort can be found in Annex 1.

L.2.2.2 Initial Screening Storm

The storm selected for rough order of magnitude design of the various alignments is based on the probable maximum hurricane (PMH) as documented in NOAA's Technical Report NWS 23 (1979). The PMH has a central pressure of 890 mb (26.28 inches of mercury). The PMH criteria for the Louisiana coast describe a storm of Category 5 intensity on the Saffir-Simpson Scale (SSC). The radius to maximum winds was approximately 11 nm, that of Hurricane Camille, and the average forward speed applied for the dynamic solution was set at 10 knots. The PMH was run on ten historical (or modified historical) tracks with landfalls across coastal Louisiana with different approach angles. For storm surge modeling, the storms were translated both at the historical hurricane track speed and at a constant 10 knots. The tracks were selected to result in Category 5 hurricane surge levels at locations across the proposed structural alignments. The selected tracks are summarized in Table L-2 and plotted in Figure L-12.

The number of tracks selected was limited because of time constraints on the Preliminary Technical Report schedule. A more comprehensive modeling assessment will be performed for the Final Technical Report (FTR).

L.2.2.3 Risk-Based Approach

Storm surge is a function of many factors including, but not limited to, wind speed, translation speed, landfall location, orientation of the storm track at landfall to the shoreline, and storm size. Therefore, there is a need to move away from an event-driven approach that considers only particular storms and move towards a risk-based approach that addresses how often assets and populations become inundated and how severe that inundation is for storm events of particular characteristics. Such an approach will be considered for the Final Technical Report.

Table L-2. Modeled Hurricane Tracks

Track	Description	Naming convention
1	Hurricane Katrina	T1
2	Hurricane Andrew shifted 1.0 deg east	T2
3	1947 storm shifted 0.25 deg south	T3
4	Hurricane Rita	T4
5	Hurricane Carmen	T5
6	1915 storm	T6
7	1893 storm shifted 0.5 deg west	T7
8	Hurricane Rita shifted 1.0 deg east	T8
9	Hurricane Camille shifted 0.5 deg west	T9
10	1893 storm shifted 2.5 deg west	T10

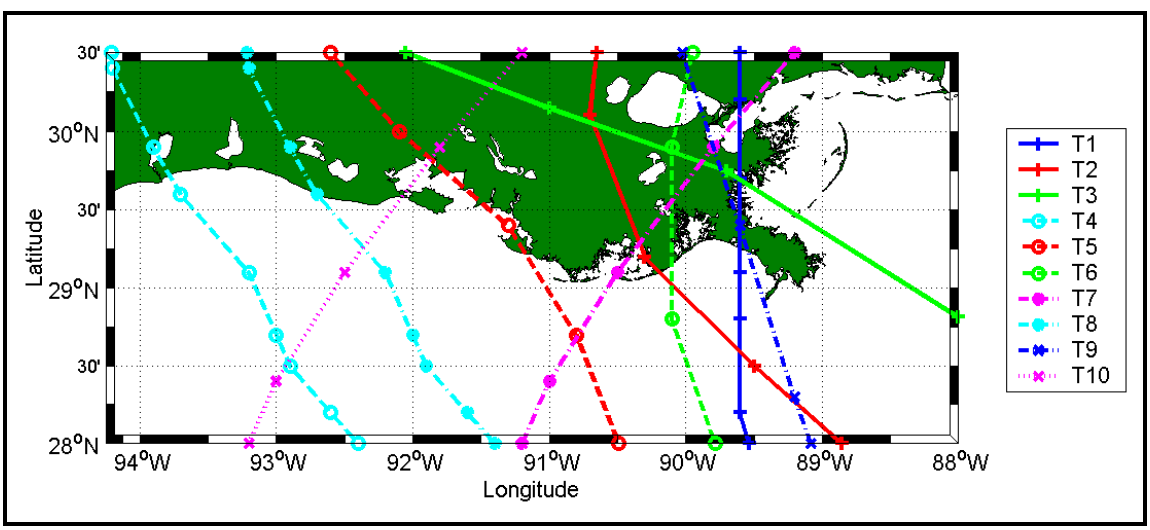


Figure L-12. Selected hurricane tracks.

L.2.2.4 Wind and Atmospheric Pressure

Accurate modeling of wave and storm surge levels requires accurate wind and pressure field input to the model. The wind and pressure fields were developed with a highly refined meso-scale vortex numerical model for the specification of surface wind and pressure fields in tropical cyclones. This model specifies the surface wind and pressure field in tropical cyclones and is referred to as the Planetary Boundary Layer Model (Thompson and Cardone 1996). Model inputs include the central pressure index (CPI), radius to maximum wind (RMW), forward velocity, and storm track locations. The dynamical model operates on these inputs and additional inputs required to produce a wind and pressure field. See Annex 2 for details of wind and pressure fields for the screening storm.

The maximum wind speed generated over space and time is approximately 135 mph. This speed is based on a 10-meter equivalent neutrally stable, 30-minute average wind speed. If the maximum wind speed is converted to a 1-minute average (the general average interval to quantify the Saffir-Simpson Hurricane Scale) the magnitude would be about 166 mph (a ratio of 1.23) or an intense Category 5 hurricane. The final 30-minute wind and pressure fields provided input to the surge and wave models.

L.2.2.5 Storm Surge Modeling

The ADvanced CIRCulation (ADCIRC) hydrodynamic model (Leuttich et al. 1992) was applied to estimate storm surge generated by the PMH. ADCIRC is a finite-element hydrodynamic circulation numerical model for the simulation of water level and current over an unstructured gridded domain. ADCIRC is a two-dimensional depth-integrated (2DDI) model that can simulate tide-, wind- and wave-driven circulation in coastal waters as well as hurricane storm surge and flooding.

ADCIRC was chosen for simulating the long-wave hydrodynamic processes in the study area. Imposing the wind and atmospheric pressure fields, the ADCIRC model can replicate tide-induced and storm-surge water levels and currents. In two dimensions, the model is formulated with the depth-averaged shallow-water equations for conservation of mass and momentum.

The ADCIRC model uses a finite-element algorithm in solving the defined governing equations over complicated bathymetry encompassed by irregular sea/shore boundaries. This algorithm allows for extremely flexible spatial discretizations over the entire computational domain and has demonstrated excellent stability characteristics. The advantage of this flexibility in developing a computational grid is that larger elements can be used in open-ocean regions where less resolution is needed, whereas smaller elements can be applied in the nearshore and estuary areas where finer resolution is required to resolve hydrodynamic details.

L.2.2.6 Methodology

The ADCIRC grid utilized during this study is that which was calibrated during the Interagency Performance Evaluation Team (IPET) Task 4 Hurricane Katrina study. The model incorporates the western North Atlantic Ocean, the Gulf of Mexico and Caribbean Sea to allow for full dynamic coupling between oceans, continental shelves, and the coastal floodplain. The grid is locally refined to resolve features such as inlets, rivers, navigation channels, levee systems and local topography/bathymetry. The grid applied and validated for the IPET Hurricane Katrina hindcast studies served as the existing condition for this study. The base grid was modified for each of the five proposed modeling alignments and the levee was set at a height to ensure that no overtopping occurred.

The storm surge modeling consists of 84 ADCIRC model simulations. All tracks were simulated on the existing condition grid. Combinations of levee alignments and PMH storm tracks were selected to produce the estimates of maximum surge, wave and runup for the proposed coastal Louisiana alignments. The alignments are similar along many sections of the Louisiana coast and not all storm tracks were required to be run for all alignments. Table L-3 summarizes the levee alignment/storm track combinations modeled. The storms were run at both the historical translation speed and at a constant 10 knot translation speed.

Table L-3
Levee Alignment/Storm Track Combinations Modeled with ADCIRC

Storm Track	Alignment 1	Alignment 2	Alignment 3	Alignment 4	Alignment 5
T1	X	X	X	X	X
T2	X		X	X	X
T3	X	X			X
T4	X				
T5	X				X
T6	X	X	X	X	X
T7	X	X	X	X	X
T8	X				
T9	X	X	X	X	X
T10	X				

L.2.2.7 Results

Hurricane surge water surface elevations were generated for each combination specified in Table L-3. Peak water surface elevations for each levee alignment were obtained by taking the maximum surge elevation from all ten storm tracks. The storm surge can be sensitive to the hurricane translation speed as a slower moving storm allows more time for the winds to push the water toward the coast. Therefore, the storms were run at both historical translation speeds and at a constant ten knots. Maximum surge elevations along the proposed levee alignments range from approximately 13 to 20 ft in Lake Pontchartrain to about 30 to 40 ft elsewhere along the coast. Track T10 produced the most severe conditions along the central and westerly reaches. Track T3 produced the worst conditions for the south shore of Lake Pontchartrain and on the easterly reaches north of English Turn while track T7 generated the greatest surge at the levees south of English Turn. Tracks T2 and T7 produced the most severe conditions in the Barataria reach and track T4 produced the most extreme conditions on the far western reaches.

The existing condition levees are overtopped and the placement of a high levee increases surge elevations by as much as approximately 15 ft for the storm and tracks simulated. The blocked water rises on the levee and spreads along the levee where possible. Model results also indicate that surges tend to be larger in levee "pockets", areas along the levee alignment with acute angles, sometimes termed the "funnel" effect. The levees in these areas confine the surge, not allowing it to spread along the coast. This effect is seen on a larger scale east of the river as the geography of the area "catches" the surge of hurricanes making landfall to the east. The sensitivity of surge potential is intensified on the south Louisiana coast due to the complex geometry of the levees and the coast. West of the river, surge potential is greatest for storms approaching from the southwest or perpendicular to the shoreline. The storm winds approaching from these angles push water toward the coast prior to landfall while southeast storms tend to push water away from the coast as they approach.

L.2.2.8 Surge Modeling Sensitivity

Sensitivity runs were performed to compare the surges estimated by the model for the PMH to historical storms to put the estimated surges for the screening storm into a historical context and to investigate the dependency of intense hurricane surge on drag law specification. The surges estimated by the model for the PMH track T1 were compared to that calculated for Hurricane Katrina and the PMH surges on track T9 were compared to those estimated for Hurricane Camille on track T9. Some of the stretches of the Louisiana levees most impacted by T1 and T9 are the south shore of Lake Pontchartrain and from Slidell to English Turn. Relative to Hurricane Katrina, the PMH surges were 1 to 2 ft lower from Slidell to English Turn and surges were 3 to 5 ft higher along the south shore of Lake Pontchartrain. Relative to a Hurricane Camille wind field translated along the same track, the PMH generated surges 1 to 3 ft higher from Slidell to English Turn and surges were 3 to 5 ft higher along the south shore of Lake Pontchartrain.

Sensitivity runs were also made to investigate the dependency of intense hurricane surge on drag law specification. The sensitivity was performed by simulating the screening storm on track T1 for Modeling Alignment 1 with the various drag coefficient cutoffs. The stretches of levee most impacted by T1 are those along the south shore of Lake Pontchartrain, from Slidell to English Turn, and the levees east of the river in lower Plaquemines Parish. The estimated peak surges for these areas are presented in Table L-4 relative to the simulation with no cutoff. The surge estimates show little sensitivity between the no cutoff and C_w =0.040 cases. The C_w =0.025 is an extreme case, reducing surges 1 to 5 ft at the levees. In offshore wave modeling, drag cutoffs of 0.030 to 0.040 are typically employed. The issue of limits on the specification of the drag coefficient for use in computing hurricane storm surge simulation will be investigated further during the second phase of the project.

Table L-4. Drag Cutoff Coefficient Sensitivity Analysis			
Location	**C_w=0.040 – No Cutoff (ft)**	**C_w=0.030 – No Cutoff (ft)**	**C_w=0.025 – No Cutoff (ft)**
South Shore	-0.5 to -1	-1 to -2.5	-1 to -4
Slidell to English Turn	0	-1 to -1.5	-1.5 to -2.5
Plaquemines	-0.5 to -1	-2 to -3	-3 to -5

Annex 3 contains the results of an Independent Technical Review on application of the ADCIRC model for storm surge predictions on the New Orleans, LA vicinity done by Robert Dean, Mark Powell and Robert Reid.

L.2.2.9 Offshore Wave Modeling

The generation of the wave field and directional wave spectra for the various hurricane storm tracks is based on the implementation of a third generation discrete spectral wave model called WAM (Komen et al. 1994). A nested grid approach was applied for the offshore wave simulations. This effectively reduces the computational demand and maximizes the use of higher resolution wind estimates in the coastal area. The purpose of the offshore wave simulations is to supply the nearshore wave modeling effort supported by STWAVE (Smith et al. 2001). The validation of the WAM was performed within the IPET Task 4 Katrina effort.

The wave model simulations reflect the time and spatial variation of one hurricane wind field projected onto various storm tracks. This will depict the growth and propagation of the wave energy in the target domains. A summary of the absolute maximum wave heights (H_{mo}) is documented in Table L-5.

Table L-5 Basin Maximum Wave Height Locations					
Trk #	**Description**	**H_{mo} (ft)**	**Storm Duration (Days)**	**Location**	
				Long	**Lat**
1	Hurricane Katrina	**56**	**4.50**	**-87.40**	**24.50**
2	Hurricane Andrew shifted 1.0-deg east	**59**	**3.50**	**-88.20**	**27.80**
3	1947 Storm shifted 0.25-deg south	**60**	**3.25**	**-88.05**	**29.10**
4	Hurricane Rita	**57**	**5.00**	**-85.80**	**24.50**
5	Hurricane Carmen	**56**	**4.25**	**-90.10**	**26.80**
6	1915 Storm	**56**	**4.75**	-88.40	**26.10**
7	1893 Storm shifted 1.0-deg east	**56**	**6.50**	-86.70	**25.60**
8	Hurricane Rita shifted 1.0-deg east	**55**	**4.75**	-90.50	**27.30**
9	Hurricane Camille shifted 0.5-deg west	**57**	**4.25**	-89.10	**28.60**
10	1893 Storm shifted 2.5-deg west	**53**	**6.50**	-92.60	**24.60**

The analysis continues into the Region-scale modeling domain. The overall maximum H_{mo} estimates are provided for each of the 10 hurricane track simulations are displayed in Table L-6.

Table L-6. Region Maximum Wave Height Locations					
Trk #	Description	H_{mo} (ft)	Storm Duration (Days)	Location	
				Long	Lat
1	Hurricane Katrina	**54**	**4.50**	**-89.40**	**28.65**
2	Hurricane Andrew shifted 1.0-deg east	**57**	**3.50**	**-89.25**	**28.55**
3	1947 Storm shifted 0.25-deg south	**60**	**3.25**	**-88.15**	**29.15**
4	Hurricane Rita	**48**	**5.00**	**-92.60**	**28.50**
5	Hurricane Carmen	**48**	**4.25**	**-90.40**	**28.50**
6	1915 Storm	**55**	**4.75**	-89.80	**28.50**
7	1893 Storm shifted 1.0-deg east	**45**	**6.50**	-90.50	**28.50**
8	Hurricane Rita shifted 1.0-deg east	**48**	**4.75**	-91.60	**28.50**
9	Hurricane Camille shifted 0.5-deg west	**58**	**4.25**	-89.15	**28.75**
10	1893 Storm shifted 2.5-deg west	**49**	**6.50**	-92.70	**28.50**

L.2.2.10 Nearshore Wave Modeling

Nearshore waves are required to calculate wave runup and overtopping on structures and wave forces on structures. The numerical model STWAVE (Smith, Sherlock, and Resio 2001; Smith and Smith 2001; Thompson, Smith, and Miller 2004) was applied to generate and transform waves to the shore.

STWAVE is a finite-difference model and calculates wave spectra on a rectangular grid. The model outputs zero-moment wave height, peak wave period (T_p), and mean wave direction (α_m) at all grid points and two-dimensional spectra at selected grid points. An option has been added to input variable wind and surge fields. The surge significantly alters the wave transformation and generation for the hurricane simulations in shallow areas (such as Lake Pontchartrain) and where low-laying areas are flooded. Spatially varying wind input is important to simulate the complex wind fields in hurricanes.

L.2.2.11 Methodology

STWAVE was applied on four grids for the southern Louisiana area: Pontchartrain, Louisiana Southeast, Louisiana South, and Louisiana West. The input for each grid includes the bathymetry (interpolated from the ADCIRC domain), surge fields (interpolated from ADCIRC output), and wind (interpolated from ADCIRC output). The first grid covers Lake Pontchartrain at a resolution of 656 ft (200 m). The domain is approximately 25.8 by 41.9 miles (41.6 by 67.4 km). The second, third, and fourth grids cover the entire Gulf of Mexico coastline of Louisiana at a resolution of 656 ft (200 m). The domain for the southeast grid is approximately 84.9 by 92.4 miles (136.6 by 148.8 km) and extends from Mississippi Sound in the northeast to the Mississippi River in the southwest. The domain for the south grid is approximately 82.5 by 104.2 miles (132.8 by 167.8 km) and extends from the Mississippi River in the east to the Atchafalaya River in the west. The domain for the west grid is approximately 100.7 by 216.2 miles (162 by 348 km) and extends from east of the Atchafalaya River to west of the Sabine River.

L.2.2.12 Simulations

The STWAVE simulations include ten storm tracks and five levee alignments. The ten storms, five alignments, and four grids give a potential for 200 STWAVE simulations. But, most of the alignment differences occur in the South grid, so not all storms were run for all alignments on all grids. Table L-7 provides a summary of which storms were run on which grids/alignments.

Table L-7
STWAVE Simulations by Grid (P=Pontchartrain, SE=Southeast, S = South, W = West)

Storm Track	Alignment 1	Alignment 2	Alignment 3	Alignment 4	Alignment 5
T1	P, SE, S, W	P, SE	S	S	SE, S
T2	P, SE, S, W		S	S	SE, S
T3	P, SE, S, W	P, SE			SE, S
T4	P, SE, S, W				
T5	P, SE, S, W				SE, S
T6	P, SE, S, W	P, SE		S	SE, S
T7	P, SE, S, W	P, SE	S	S	SE, S
T8	P, SE, S, W				
T9	P, SE, S, W	P, SE	S	S	SE, S
T10	P, SE, S, W				

L.2.2.13 Results

STWAVE was run for approximately a two-day period for each storm to capture the peak wave conditions. Because STWAVE is a steady-state model, spin-up time is not required for the simulations. To provide the wave height and period for runup calculations, the STWAVE output is processed to extract the largest significant wave height for each grid cell in each domain. Then, the maximum significant wave heights in representative sections seaward of the levees are selected for calculating runup. Maximum wave heights in Lake Pontchartrain were approximately 9 to 12 ft with peak wave periods of about 8 sec. Outside of Lake Pontchartrain, maximum wave heights were 10 to 15 ft depending on surge depths and wave periods were generally 14 sec. From Slidell to Chef Menteur, where waves are partially sheltered by the barrier islands, peak wave periods were approximately 11 sec. The significant wave height and peak wave period, along with the surge, and depth were output at specified save points near the structure toe. The save points are shown in Figures L-13 through L-22.

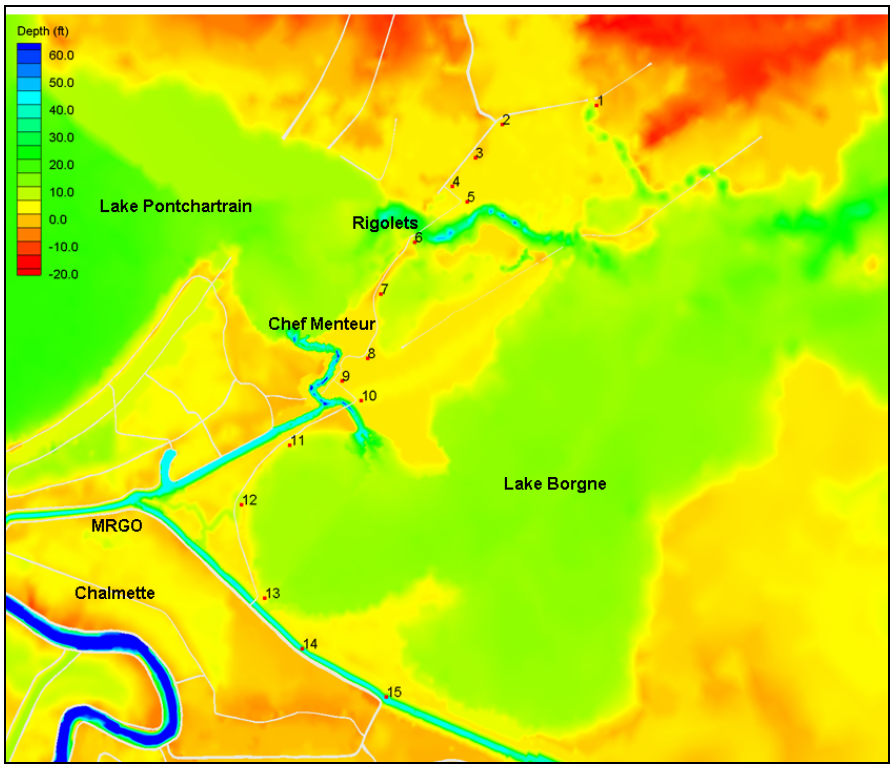


Figure L-13. Save Points 1-15 (STWAVE Southeast grid).

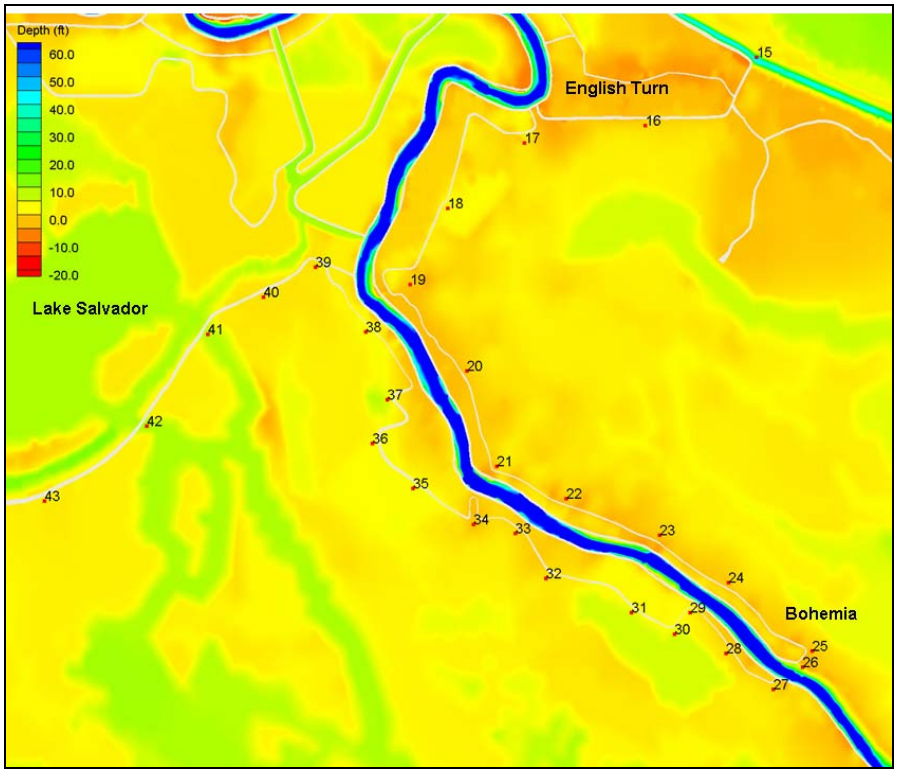


Figure L-14. Save Points 15-26 (STWAVE Southeast grid) and 27 to 43 (STWAVE South grid).

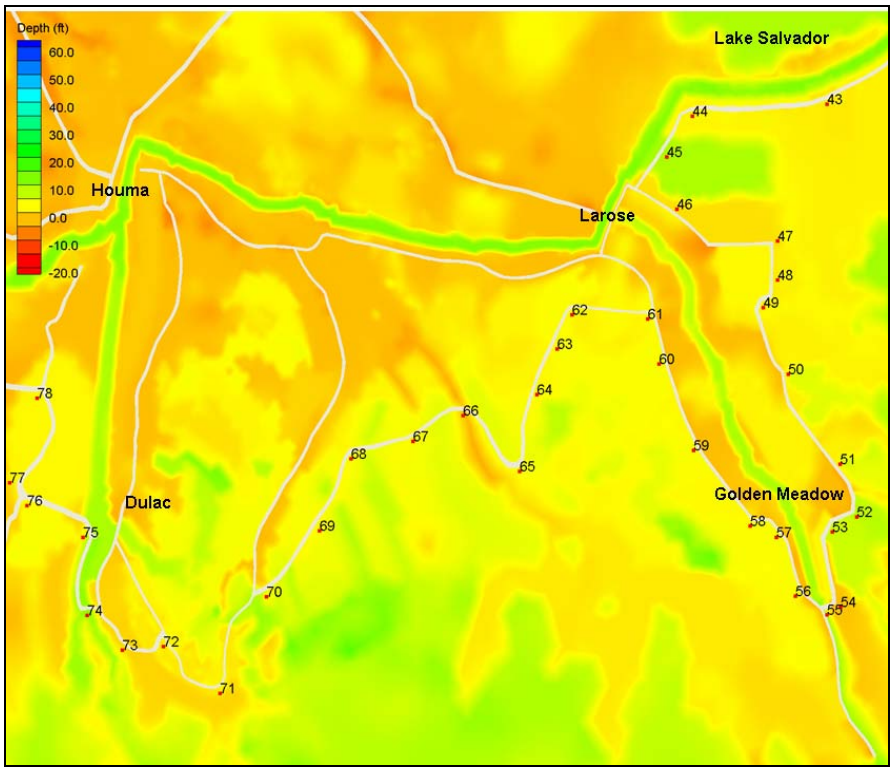


Figure L-15. Save Points 43 to 78 (STWAVE South grid) and 72 to 78 (STWAVE West grid).

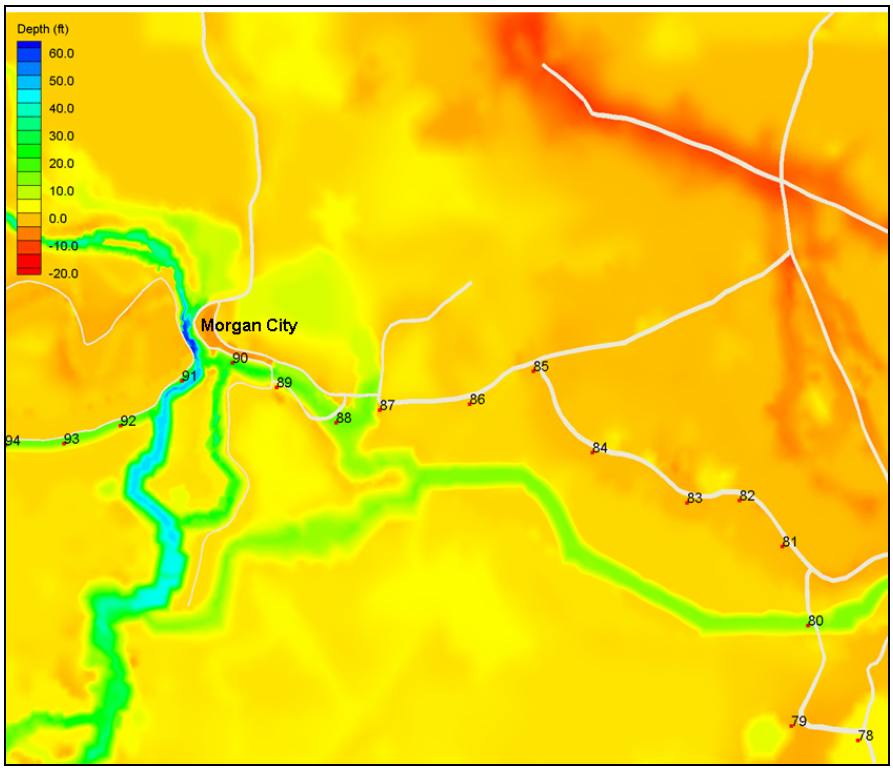


Figure L-16. Save Points 78 to 88 (STWAVE South grid) and 78 to 94 (STWAVE West grid).

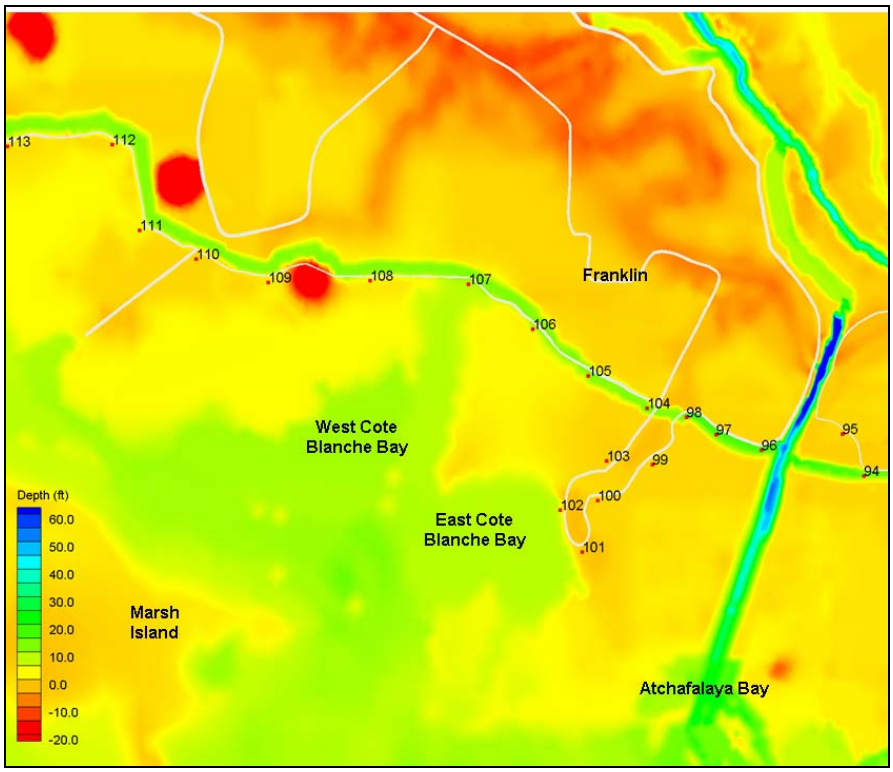


Figure L-17. Points 94 to 113 (STWAVE West grid).

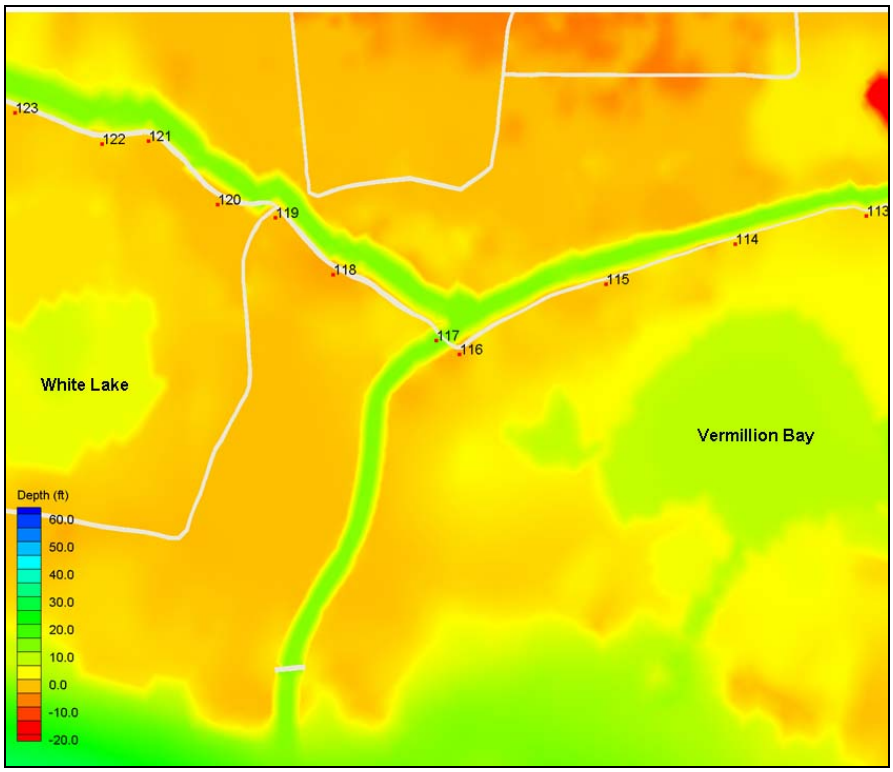


Figure L-18. Points 113 to 123 (STWAVE West grid).

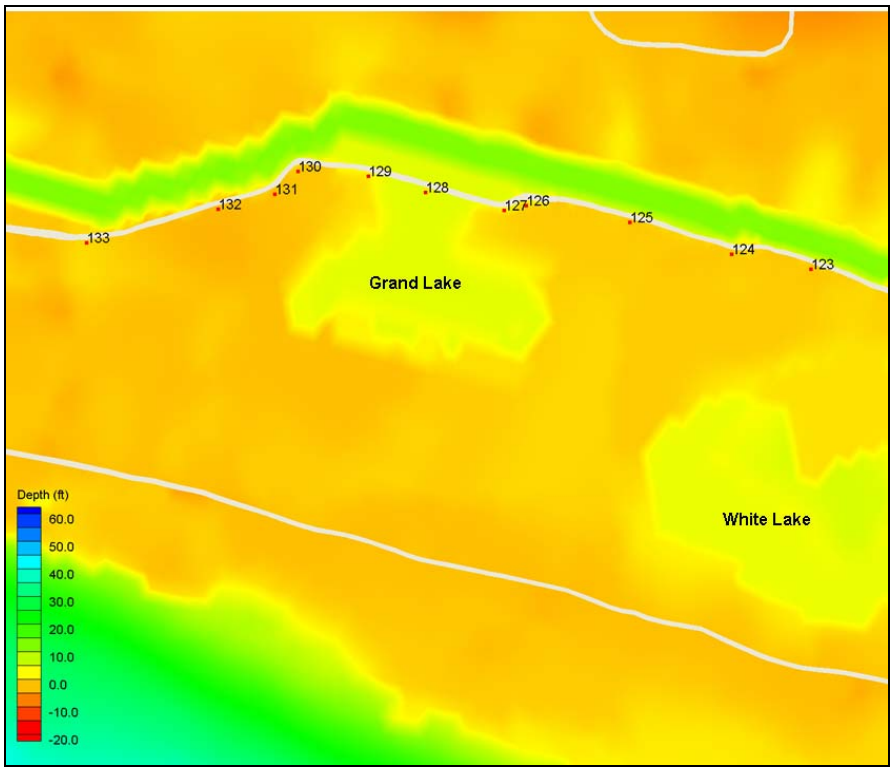


Figure L-19. Points 123 to 133 (STWAVE West grid).

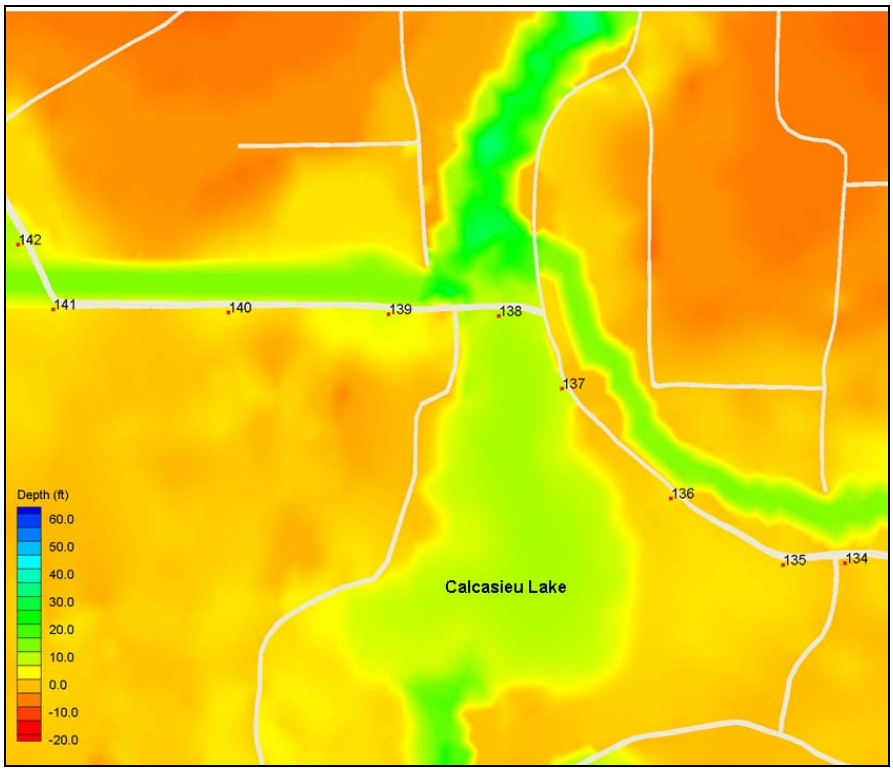


Figure L-20. Points 134 to 142 (STWAVE West grid).

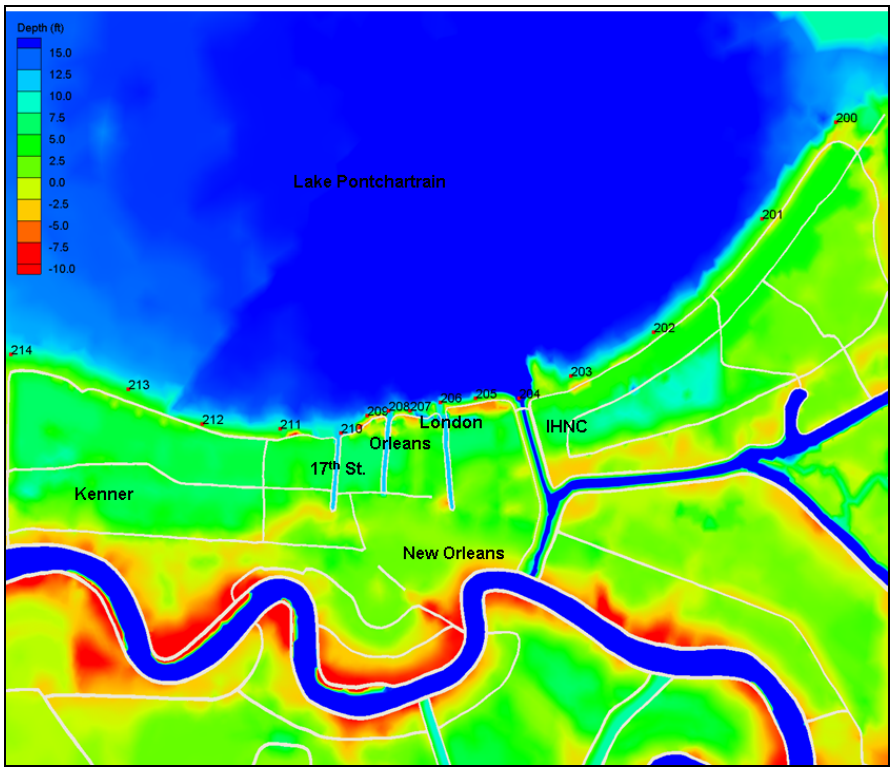


Figure L-21. Points 200 to 214 (STWAVE Pontchartrain grid).

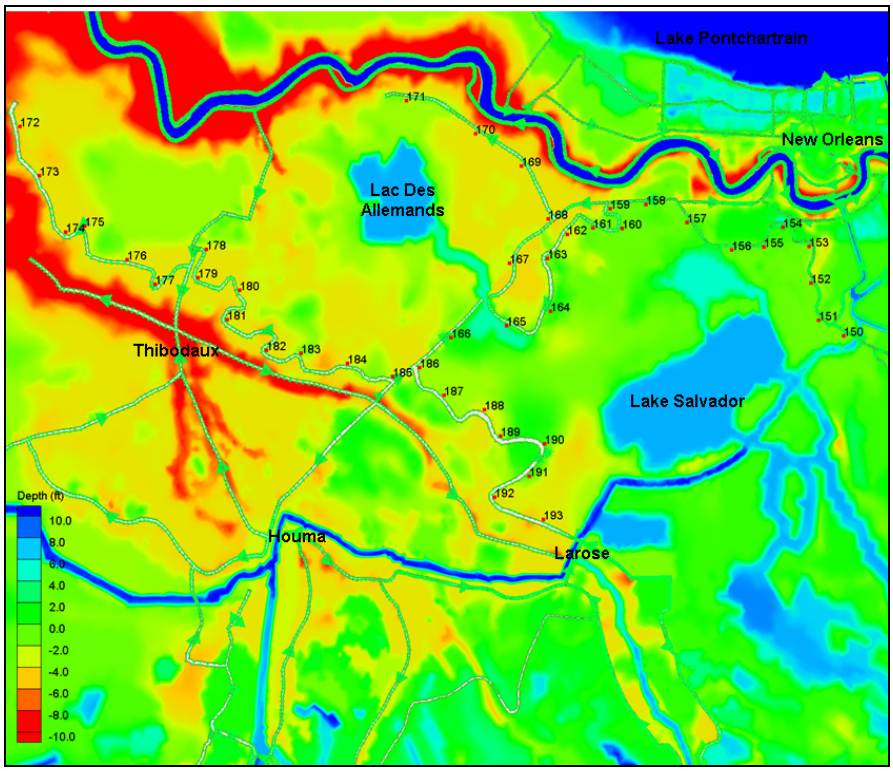


Figure L-22. Points 150-172 (STWAVE South Grid Alternatives 3 and 4).

L.2.2.14 Estimated Maximum Waves and Water Level

The storms simulated for the initial screening provide limited coverage across the Louisiana coast. The simulated storm tracks impact short reaches of the levee with the highest waves and surge. However, these impacts could occur over extensive reaches of the levee with changes in storm landfall location. Therefore, for rough order of magnitude design purposes, the estimated maximum surge elevations and waves were spread along the levee applying engineering judgment. Track T3 produced the worst conditions for the south shore of Lake Pontchartrain and on the easterly reaches north of English Turn, while track T7 generated the greatest surge at the levee south of English Turn. Tracks T2 and T7 produced the most severe conditions in the Barataria reach. Track T10 produced the most severe conditions along the central and westerly reaches and track T4 produced the most extreme conditions on the far western reaches. A more comprehensive modeling assessment with complete spatial coverage will be performed for the Final Technical Report.

A summary of the maximum wave and surge levels for each alignment is provided in Tables L-8 through L-12. Maximum surge elevations range from 15 to 20 ft in Lake Pontchartrain and 30 to 40 feet elsewhere along the coast. Maximum waves range from about 10 to 15 ft with peak wave periods of 8 to 14 sec. For the PMH storm, opening of the tidal passes decreases surges on the levee at the Pontchartrain land bridge by about 3 ft and increases water levels in the lake by about 1.5 ft. It should be noted, however, that the impact of allowing the tidal passes to remain open could increase water levels further in the lake for larger (radius to maximum winds), slower moving storms and additional analysis is required. Opening Barataria Basin (Modeling Alignments 3 and 4) reduces surges along the levees by several feet but requires substantially longer levees. Smoothing the levee alignment (Modeling Alignment 5) reduces the surges east of the river approximately 3 to 5 ft.

Table L-8. Summary of Waves and Water Levels for Modeling Alignment 1

Save Points	Surge (ft)	Wave Height (ft)	Wave Period (s)
1 – 10	33	12	11
11 – 26	36	13	14
27 – 33	33	12	14
34 – 44	30	11	14
45 – 51	33	12	14
52 - 53	36	13	14
54 – 104	40	15	14
105 - 126	36	13	14
127 – 134	33	12	14
135 - 142	30	11	14
200 - 214	13 - 20	9 - 12	8

Table L-9. Summary of Waves and Water Levels for Modeling Alignment 2

Save Points	Surge (ft)	Wave Height (ft)	Wave Period (s)
1 – 10	30	11	11
11 – 14	33	12	14
15 – 26	36	13	14
27 – 33	33	12	14
34 – 44	30	11	14
45 – 51	33	12	14
52 – 53	36	13	14
54 – 104	40	15	14
105 – 126	36	13	14
127 – 134	33	12	14
135 – 142	30	11	14
200 – 214	14 - 22	10 - 13	8

Table L-10. Summary of Waves and Water Levels for Modeling Alignment 3				Table L-11. Summary of Waves and Water Levels for Modeling Alignment 4			
Save Points	Surge (ft)	Wave Height (ft)	Wave Period (s)	Save Points	Surge (ft)	Wave Height (ft)	Wave Period (s)
1 – 10	33	12	11	1 – 10	33	12	11
11 – 26	36	13	14	11 – 26	36	13	14
27 – 33	33	12	14	27 – 33	33	12	14
34 – 37	30	11	14	34 - 37	30	11	14
38	26	10	14	38	26	10	14
39	23	9	14	39	23	8	14
46 - 49	26	10	14	46 – 49	26	10	14
50	30	11	14	50	30	11	14
51	33	12	14	51	33	12	14
52 - 53	36	13	14	52 - 53	36	13	14
54 – 104	40	15	14	54 – 104	40	15	14
105 – 126	36	13	14	105 – 126	36	13	14
127 – 134	33	12	14	127 – 134	33	12	14
135 – 142	30	11	14	135 – 142	30	11	14
150 – 164	20	8	14	150 – 156	20	7	14
165	17	6	14	157 - 162	23	8	14
166 – 184	10	4	14	163 - 164	20	7	14
185	13	5	14	165 - 166	17	6	14
186 – 190	17	6	14	186 - 190	17	6	14
191 – 193	26	10	14	191 – 193	26	10	14
200 - 214	13 - 20	9 - 12	8	200 - 214	13 - 20		8

Table L-12. Summary of Waves and Water Levels for Modeling Alignment 5			
Save Points	Surge (ft)	Wave Height (ft)	Wave Period (s)
1 – 19	33	12	11
39 – 46	33	12	14
61 - 70	33	12	14
71 – 98, 104	40	15	14
105 - 126	36	13	14
127 – 134	33	12	14
135 - 142	30	11	14
200 - 214	13 - 20	9 - 12	8

L.2.3 WAVE RUNUP AND OVERTOPPING AND STONE ARMOR STABILITY

L.2.3.1 General

The degree of wave action on and going over a coastal structure must be predicted in order to design the structure crest height. The structure crest height is usually set to provide a specific probability of exceedance of water going over the crest. This requires estimation of the expected incident wave and water level climate at the structure toe as well as the local subsidence, structure/foundation settlement rate, and the relative local sea level rise.

Wind-generated waves break and run up sloping coastal structures. This process is termed *wave runup*. If the structure crest is low enough, the runup can go over the structure crest. This process is termed wave *overtopping*. The degree of runup or overtopping is dependent on the incident wave height, wave steepness, wave angle of obliquity, slope of the structure, structure roughness and permeability. Incident waves are defined on the foreshore at the location of the structure toe, where the structure meets the local bathymetry. The foreshore is defined as the region of the bathymetry just seaward of the structure ending at the structure toe. Wave runup is computed as the upward vertical distance from the still water level. The probability of exceedance of wave runup is computed based on the percent of waves producing a runup higher than the given statistic. In other words, the exceedance is calculated based on the number of waves rather than the number of runups. Wave overtopping is defined by the volume of water going over the seaward edge of the structure crest per unit length of structure per unit time. These basic physical processes are described more fully in the Coastal Engineering Manual (CEM 2004).

A thorough analysis of wave runup and overtopping was done for the PTR. The detailed results of this analysis can be found in Annex 4. The results of this analysis are summarized in the following paragraphs.

L.2.3.2 Analysis Results

Waves and water levels for this preliminary design were taken from the results of the STWAVE and ADCIRC model runs. In summary, 10 storms with roughly the same intensity but following different tracks were used to generate waves and water levels near the structure toe along the five levee alignments. Significant wave height, peak wave period, surge, and depth were output at specified save points near the structure toe. These results are summarized in Tables L-8 to L-12.

For preliminary cross-sectional design of the levee system, the following parameters were calculated at each save point for each storm and each alignment:

1. Wave runup
2. Wave overtopping
3. Stone armor size
4. Stone armor layer thickness ($t = 2D_{n50}$)
5. Non-overtopping crest elevation ($h_c = h_s + R_{u2\%}$)
6. Crest elevation corresponding to varying levels of wave overtopping

Where $R_{u2\%}$ = wave runup height on the structure with 2 percent probability of exceedance and H_s = significant wave height.

Virtually all of the design guidance for levee armoring, runup, and overtopping is for uniform slopes. Some guidance exists for compound slopes with a fronting berm but there is no useful guidance for a structure with a shallower slope seaward of a steeper slope. Therefore, all computations summarized herein assume a uniform seaward slope

of a levee. Further, design guidance exists for wave overtopping erosion and for mean steady flow overtopping; however there is no guidance for erosion due to waves on top of a flood over the levee crest. Therefore, all surge levels are assumed to be below the levee crest. The maximum surge height was established using the ADCIRC storm surge model and levee elevations in the model were set artificially high so that the surge could not overtop. By doing so, the surge elevations computed in the ADCIRC model represented the maximum elevation that could be generated by the PMH at the location of the levee and for the storm track used. This maximum surge plus the wave height and periods computed in STWAVE were used in calculating the height of the levee crest. Also, all calculations are for rock armor. Other armoring will be evaluated for the FTR.

There are three levels of design crest height to be considered in this initial levee design:

1. Non-overtopped ($h_c = h_s + R_{u2\%}$) grass-covered levee crest
2. Lightly to moderately overtopped, lightly armored levee crest
3. Heavily overtopped by waves but not by steady surge flow, heavily armored levee crest

The overtopping rate corresponding to erosion of a grass levee is roughly q = 0.33 – 3.28 $ft^3/s/ft$. A heavily armored crest could be designed and constructed to withstand wave overtopping with a crest near the maximum flood height. The maximum flood elevation would be the sum of maximum surge, tide, and wave setup. For this simple analysis, the lowest crest height corresponding to the highest overtopping is assumed to be $h_c = 1.1h_s$ This crest height ranges from 30 to 50 ft in South Louisiana. The overtopping rate range for this lowest crest height was roughly 3 – 9 $ft^3/s/ft$. For light to moderate overtopping in the range of 0.3 – 0.9 $ft^3/s/ft$, the crest height is assumed to be $h_c = 1.2h_s$. For Lake Pontchartrain, the required crest heights are much lower than for the Gulf of Mexico shoreline...Required crest heights for several levels of protection are listed in Tables L-13 through L-17 for a slope of 1V:4H for Alignments 1 – 5 respectively. Table 18 gives results for a slope of 1V:8H for Alignment 1 for comparison. The resolution of the analysis for this preliminary design was not adequate to define a difference between the required crest heights of the two slopes for moderate to heavy wave overtopping. So the values listed in the right hand two columns of Tables L-13 and L-18 are identical. For all cases, save points were selected to yield a conservative design section for each alignment.

Table L-13. Summary of Crest Heights for Seaward Structure Slope 1V:4H for Alignment 1			
Save Points	Non-Overtopped $h_c = h_s + R_{u2\%}$ (ft)	Light to Moderate Wave Overtopping $h_c/h_s = 1.2$ (ft)	Massive Wave Overtopping $h_c/h_s = 1.1$ (ft)
1 – 10	49	39	36
11 – 26	56	43	40
27 – 34	50	39	36
35 – 44	46	35	32
45 – 51	50	39	36
52 – 53	55	43	40
54 – 104	60	47	43
105 - 126	55	43	40
127 – 134	50	39	36
135 – 142	46	35	32
200 – 214	23 – 35	18 - 26	16 - 24

Table L-14. Summary of Crest Heights for Seaward Structure Slope 1V:4H for Alignment 2			
Save Points	Non-Overtopped $h_c = h_s + R_{u2\%}$ (ft)	Light to Moderate Wave Overtopping $h_c/h_s = 1.2$ (ft)	Massive Wave Overtopping $h_c/h_s = 1.1$ (ft)
1 – 10	44	35	32
11 – 14	52	39	36
15 – 26	56	43	40
27 – 33	50	39	36
34 – 44	46	35	32
45 – 51	50	39	36
52 – 53	55	43	40
54 – 104	60	47	43
105 – 126	55	43	40
127 – 134	50	39	36
135 – 142	45	35	32
200 – 214	26 – 37	17 - 26	16 - 24

Table L-15. Summary of Crest Heights for Seaward Structure Slope 1V:4H for Alignment 3			
Save Points	Non-Overtopped $h_c = h_s + R_{u2\%}$ (ft)	Light to Moderate Wave Overtopping $h_c/h_s = 1.2$ (ft)	Massive Wave Overtopping $h_c/h_s = 1.1$ (ft)
1 – 10	50	40	36
11 – 26	55	44	40
27 – 33	50	40	36
34 – 37	46	36	33
38	41	32	29
39	35	28	26
46 – 49	40	32	29
50	46	36	33
51	50	40	36
52 – 53	55	43	40
54 – 104	60	48	43
105 – 126	55	43	40
127 – 134	50	40	36
135 – 142	46	36	33
150 – 164	31	24	22
165	26	20	18
166 – 184	16	12	11
185	20	16	15
186 – 190	26	20	18
191 – 193	40	32	29

Table L-16. Summary of Crest Heights for Seaward Structure Slope 1V:4H for Alignment 4			
Save Points	Non-Overtopped $h_c = h_s + R_{u2\%}$ (ft)	Light to Moderate Wave Overtopping $h_c/h_s = 1.2$ (ft)	Massive Wave Overtopping $h_c/h_s = 1.1$ (ft)
1 – 10	49	39	36
11 – 26	50	43	40
27 – 33	50	39	36
34 – 37	46	35	32
38	41	31	29
39	35	28	25
46 – 49	40	31	29
50	46	35	32
51	50	39	36
52 – 53	55	43	40
54 – 104	60	47	43

105 – 126	55	43	40
127 – 134	50	39	36
135 – 142	45	35	32
150 – 156	31	24	22
157 – 162	36	28	25
163 – 164	31	24	22
165 – 166	26	20	18
186 – 190	26	20	18
191 – 193	40	31	29

Table L-17. Summary of Crest Heights for Seaward Structure Slope 1V:4H for Alignment 5

Save Points	Non-Overtopped $h_c = h_s + R_{u2\%}$ (ft)	Light to Moderate Wave Overtopping $h_c/h_s = 1.2$ (ft)	Massive Wave Overtopping $h_c/h_s = 1.1$ (ft)
1 – 19	50	39	36
39 – 46	50	39	36
61 – 70	50	39	36
71 – 98, 104	60	47	43
105 - 126	55	43	40
127 – 134	50	39	36
135 – 142	46	35	32

Table L-18. Summary of Crest Heights for Seaward Structure Slope 1V:8H for Alignment 1

Save Points	Non-Overtopped $h_c = h_s + R_{u2\%}$ (ft)	Light to Moderate Wave Overtopping $h_c/h_s = 1.2$ (ft)	Massive Wave Overtopping $h_c/h_s = 1.1$ (ft)
1 – 10	43	39	36
11 – 26	48	43	40
27 – 34	43	39	36
35 – 44	39	35	32
45 – 51	44	39	36
52 – 53	48	43	40
54 – 104	52	47	43
105 - 126	48	43	40
127 – 134	43	39	36
135 – 142	39	35	32
200 – 214	19 – 29	18 - 26	16 - 24

Armor stone weight, under layer weight, and armor stone layer thickness along the levee system are shown in Tables L-19 through L-24.

Table L-19. Summary of Median Armor Stone Weight and Armor Layer Thickness for Seaward Structure Slope 1V:4H for Alignment 1			
Save Points	Armor Weight (lb)	Underlayer Weight (lb)	Armor Layer Thickness (ft)
1 – 10	5400	540	6.4
11 – 26	9800	980	7.8
27 – 34	7700	770	7.2
35 – 44	6400	640	6.7
45 – 51	7700	770	7.2
52 – 53	9800	980	7.8
54 – 104	12100	1210	8.4
105 - 126	9800	980	7.8
127 – 134	7700	770	7.2
135 – 142	6400	640	6.7
200 – 214	1600 - 3600	160 - 360	4.3 – 5.6

Table L-20. Summary of Median Armor Stone Weight and Armor Layer Thickness for Seaward Structure Slope 1V:8H for Alignment 1			
Save Points	Armor Weight (lb)	Underlayer Weight (lb)	Armor Layer Thickness (ft)
1 – 10	1900	190	4.5
11 – 26	3500	350	5.5
27 – 34	2800	280	5.1
35 – 44	2300	230	4.8
45 – 51	2800	280	5.1
52 – 53	3500	350	5.5
54 – 104	4300	430	5.9
105 - 126	3500	350	5.5
127 – 134	2800	280	5.1
135 – 142	2300	230	4.8
200 – 214	600 - 1300	60 - 130	3.0 – 3.9

Table L-21. Summary of Median Armor Stone Weight and Armor Layer Thickness for Seaward Structure Slope 1V:4H for Alignment 2			
Save Points	Armor Weight (lb)	Underlayer Weight (lb)	Armor Layer Thickness (ft)
1 – 10	4400	440	6.0
11 – 14	7700	770	7.2
15 – 26	9800	980	7.8
27 – 33	7700	770	7.2
34 – 44	6400	640	6.7
45 – 51	7700	770	7.2
52 – 53	9800	980	7.8
54 – 104	12100	121	8.4
105 – 126	9800	980	7.8
127 – 134	7700	770	7.2
135 – 142	6400	640	6.7
200 – 214	2045 - 4217	205 - 422	4.6 – 5.9

Table L-22. Summary of Median Armor Stone Weight and Armor Layer Thickness for Seaward Structure Slope 1V:4H for Alignment 3			
Save Points	Armor Weight (lb)	Underlayer Weight (lb)	Armor Layer Thickness (ft)
1 – 10	6800	680	6.9
11 – 26	9800	980	7.8
27 – 33	7700	770	7.2
34 – 37	6400	640	6.7
38	4300	430	5.9
39	2700	270	5.0
46 – 49	4300	430	5.9
50	6400	640	6.7
51	7700	770	7.2
52 – 53	9800	980	7.8
54 – 104	12100	1210	8.4
105 – 126	9800	980	7.8
127 – 134	7700	770	7.2
135 – 142	6330	630	6.7
150 – 164	1800	180	4.4
165	1000	100	3.6
166 – 184	200	20	2.1
185	500	50	2.7
186 – 190	1000	100	3.6
191 – 193	4300	430	5.9

Table L-23. Summary of Median Armor Stone Weight and Armor Layer Thickness for Seaward Structure Slope 1V:4H for Alignment 4			
Save Points	Armor Weight (lb)	Underlayer Weight (lb)	Armor Layer Thickness (ft)
1 – 10	5400	540	6.4
11 – 26	9800	980	7.8
27 – 33	7700	770	7.2
34 - 37	6400	640	6.7
38	4200	420	5.9
39	2700	270	4.3
46 – 49	4200	420	5.9
50	6400	640	6.7
51	7700	770	7.2
52 - 53	9800	980	7.8
54 – 104	12100	1210	8.4
105 – 126	9800	980	7.8
127 – 134	7700	770	7.2
135 – 142	6400	640	6.7
150 – 156	1800	180	4.4
157 - 162	2700	270	5.0
163 - 164	1800	180	4.4
165 - 166	1000	100	3.6
186 - 190	1000	100	3.6
191 – 193	4200	420	5.9

Table L-24. Summary of Median Armor Stone Weight and Armor Layer Thickness for Seaward Structure Slope 1V:4H for Alignment 5			
Save Points	Armor Weight (lb)	Underlayer Weight (lb)	Armor Layer Thickness (ft)
1 – 19	5400	540	6.4
39 – 46	7700	770	7.2
61 – 70	7700	770	7.2
71 – 98, 104	12100	1210	8.4
105 - 126	9800	980	7.8
127 – 134	7700	770	7.2
135 – 142	6400	640	6.7

L.2.4 DETERMINATION OF REQUIRED ELEVATIONS FOR PROTECTIVE STRUCTURES

The results of the wave runup and overtopping analysis were used to determine the required protection elevations for each section. Each area along each alignment was analyzed to determine if sufficient storage existed behind the protection to allow moderate or significant wave overtopping. Table L-25 shows the design elevations and degree of wave overtopping determined for each alignment.

Table L-25
Barrier Protection Design Elevations (ft)

Save Points	Alignment				
	1	2	3	4	5
1 – 10	36	32	36	36	36
11 – 14	43	39	43	43	39
15 – 19	48	48	48	48	44
20 – 21	48	48	48	48	x
22 – 26	48	48	48	48	x
27 – 33	39	44	44	44	x
34 – 37	39	39	39	39	x
38	39	39	35	36	x
39	32	39	35	35	36
40 – 44	32	32	x	x	36
45	44	44	x	x	39
46	44	44	x	x	39
47 - 49	44	44	35	35	x
50	44	44	39	39	x
51	44	44	44	44	x
52 - 53	44	44	44	44	x
54 – 60	44	44	44	44	x
61 – 70	43	43	43	43	36
71-97	43	43	43	43	43
97 - 104	47	47	47	47	43
105	40	40	40	40	40
106 - 126	40	40	40	40	40
127 – 134	36	36	36	36	36
135 – 142	32	32	32	32	32
150 – 156	x	x	31	31	x
157 - 162	x	x	31	36	x
163 - 164	x	x	31	31	x

165	x	x	26	26	x
166	x	x	X	26	x
167 – 184	x	x	16	x	x
185	x	x	20	x	x
186 - 190	x	x	26	26	x
191 – 193	x	x	35	35	x
200 – 214	23-35	26 – 37	23-35	23-35	23-35

Light to Moderate Wave Overtopping
Significant Wave Overtopping
No shading indicates no overtopping and assumes a 1:8 slope
1:4 slope no overtopping

Using these values, design elevations were determined for each section for each alignment.

A thorough analysis of the available storage capacity behind each levee will be performed for the FTR to determine the effects of the allowed overtopping on interior stages.

L.2.5 COASTAL FEATURES AND STORM SURGE

There is growing consensus among scientists and engineers involved with Louisiana coastal protection and restoration that future projects for reducing hurricane risks in New Orleans and along the Louisiana coast should include plans to sustain or enhance the wetland-dominated landscapes that surround the area. While these landscapes are widely recognized for their great value to the nation for the natural resources and ecosystem services they provide, they may also function to provide some level of protection from hurricane wave action and storm surge.

Coastal geologic features and associated vegetation, manifested in the presence of barrier islands, cheniers, maritime forest ridges, river tributary ridges, marshes, and wetlands, have potential to abate and restrict flow exchange between the estuarine and sea environments. Compared to open water, these coastal areas have increased drag and limited water depth facing inland, which slow water velocities and may reduce the effects of propagating storm surges and waves. Together, the elevation and vegetation of coastal features has potential to restrict the volume of water at areas landward of barriers.

Emergent canopies provided by forested wetlands have potential to diminish wind penetration, thereby reducing the wind stress available to generate surface waves and storm surge. The sheltering effect of these canopied areas also affects the fetch over which wave development takes place. Shallow water depths attenuate waves via bottom friction and breaking, while vegetation provides additional frictional drag and wave attenuation and also limits wave setup. Extracting energy from waves either by breaking or increased drag in front of levees would reduce the destructive storm wave action on the levees themselves.

Despite the qualitative knowledge of the potential effects of landscape features on hurricane protection, there has been minimal quantification of these effects in nature. A literature review was done to document studies that have measured storm surge elevations with the goal of understanding how landscape features and vegetation modify the surge elevation (Annex 5). In a Letter from the Chief of Engineers (1965) documenting an interim hurricane survey of Morgan City and vicinity, Louisiana, measurements of high-water marks due to hurricane surge were correlated with distance inland from the coast. Surge elevations at 16 locations near Morgan City due to seven hurricanes (Sep 1909, Aug 1915, Sep 1915, Aug 1926, Sep 1947, Sep 1956, and Jun 1957) were documented giving 42 data points. The report states that this area has numerous bays and marshes, but the data evaluated include the western part of Louisiana with cheniers (relatively high wooded ridges). Inconsistent results were obtained when attempting to correlate hurricane translation speed, surge hydrograph at the coast, and surge elevations inland. However, a trend was observed for the decrease in storm surge as a function of distance inland, and is independent of hurricane translation speed, wind speed, and direction. The relationship indicates that storm surge was reduced by 1 foot for every 2.75 miles inland.

Lovelace (1994) documented storm surge elevations after Hurricane Andrew in Louisiana. These data are being compiled into a GIS for future reference. Citing this study, the Louisiana Coastal Wetlands Conservation and Restoration Task Force and the Wetlands Conservation and Restoration Authority (2004) suggest that storm surge is reduced about 3-inch (0.25 ft) per mile of marsh along the central Louisiana coast.

Stone et al. (2003) modeled a Category 3 hurricane that made landfall in 1915 and compared wave and storm surge for the south-central Louisiana coast in 1950 (1.09 million acres of land) to that in 1990 (0.85 million acres of land). Models used were a hurricane planetary boundary model, ADCIRC circulation model, and SWAN wave model. Acreage impacted by a 7 ft and 12 ft surge increased to 69,000 and 49,000 acres, respectively, between 1950 and 1990. Surge levels greater than 15 ft were not significantly different between the two time periods.

The Louisiana Coastal Wetlands Conservation and Restoration Task Force and the Wetlands Conservation and Restoration Authority (2004; Chapter 6, p. 55) discuss that it is "commonly acknowledged that barrier islands and wetlands reduce the magnitude of hurricane storm surges and related flooding; however, there are scant data as to the degree of reduction." At the time the report was written, the best information documenting this phenomenon came from gauges measuring water elevations during the second landfall of Hurricane Andrew (data documented by Lovelace 1994), which occurred in the vicinity of Point Chevreuil, Louisiana on August 26, 1992. Gauge data from Cocodrie, Louisiana indicated a maximum water level elevation equal to 9.3 ft during this Category 3 Hurricane. Over a 23-mile stretch of marsh and open water from Cocodrie to the Houma Navigation Canal, the water elevation decreased from 9.3 ft to 3.3 ft, equating to a reduction in surge amplitude equal to 3.1 inch (0.26 ft) per mile of marsh and open water. A similar set of measurements showed reduction of the storm surge from 4.9 ft (1.5 m) at Oyster Bayou to 0.5 ft at Kent Bayou, located 19 miles north. This second set of measurements indicated

2.8-inch (0.23 ft) decrease in surge per mile over "fairly solid marsh." The report cautions that these represent measurements from one storm; other factors, such as storm characteristics, coastal geomorphology, and track of the storm influence the degree to which wetlands decrease storm surge.

The Working Group for Post-Hurricane Planning for Louisiana Coast (2006) wrote "barrier islands, shoals, marshes, forested wetlands and other features of the coastal landscape can provide a significant and potentially sustainable buffer from wind wave action and storm surge generated by tropical storms and hurricanes." ADCIRC results from Rick Luettich (Dec 30, 2005) indicated that wetland replacement east of the Mississippi River Gulf Outlet (MRGO) by 8 ft of open water would increase storm surge from Hurricane Katrina by 3 to 6 ft for St. Bernard Parish and Eastern New Orleans.

The role of wetlands and other coastal features in reducing storm surge and waves must be addressed. There is a need to quantitatively evaluate the role of wetlands and other coastal features in reducing storm surge and waves in coastal Louisiana. To address this need, the literature review will be continued and a set of idealized numerical modeling tests will be conducted to evaluate the reduction in surge as a function of landscape feature and vegetation type. A series of model runs will be conducted with ADCIRC and STWAVE to quantify the sensitivity of storm surge and waves to environmental features. The inputs required by the models include topography/bathymetry and frictional resistance of various vegetative covers for both hydro modeling and wind sheltering. Assessments will be made for various configurations of coastal features faced with some range of storm conditions. In making these assessments the specific effects of different landscape features will be considered. It is anticipated that the effects will range from simply effecting water depth, to imparting additional frictional resistance only in the water column to vegetation that penetrates the water column and effects on the overlying wind field. A surge elevation database is being developed within a GIS for determining relationships based on available measurements, as well as for comparison with numerical modeling results.

L.2.6 RISK ASSESSMENT

A detailed risk assessment will be done as part of the FTR. The object of this effort is to provide realistic probabilities of hurricane characteristics appropriate for modeling winds, waves, and surges for the entire coastal area of south Louisiana in order to assess the exposure risks to wave and surges in this area. Both "design storm" characteristics and probabilistic hurricane characteristics will be examined. This should allow any selected design storms to be interpreted probabilistically in terms of expected return periods for scalar variables (such as total surge level) and multivariate behavior (such as surge level plus wave run-up, plus rainfall, etc.). It should also provide objective information to help answer relevant questions regarding the selection of design levels in this region (for example, what is the difference between 100-year and Cat 5 levels of protection?). It is hoped that sufficient information will be provided to establish a more meaningful categorization of hurricanes in terms of potential surge levels rather than the present characterization embodied by the Saffir-Simpson Scale, which is suitable primarily for

estimates of on-land wind damage to structures. See Annex 6 for updated SPH and PMH indices.

A team of experts from government, academia, and private companies has been assembled to address this problem of evaluating hurricane exposure risks for the Louisiana Coastal Protection and Restoration effort. As part of this effort the following existing methods and extensions to existing methods will be examined for estimating hurricane-related risks: 1) the design storm method (including examination of both the SPH and PMH), 2) the Joint Probability Method (JPM), 3) a modified Empirical Simulation Technique (M-EST), and 4) the Synthetic Storm Method (SSM). All risk-based methods will treat the following set of parameters, considered to be of primary importance to the estimation of realistic wind fields in hurricanes: 1) surface-level wind speed, 2) radius to maximum wind speed, 3) forward storm velocity, 4) direction of storm heading, and 5) location of storm landfall. The treatment of additional parameters, such as the Holland's "B" parameter and the rate of change of storm intensity, will also be examined to determine how these factors may affect the statistics of waves and surges along the coast. This suite of methods will be distilled into a single "optimal" method for subsequent hurricane risk assessment in the south Louisiana coastal area.

As part of this effort, a number of careful analyses of historical storms will be undertaken, including careful analysis of the following:

1) limiting factors imbedded within the joint probabilities of hurricane parameters (i.e. factors that relate limits in storm size to storm intensity);
2) estimation of storm decay as it approaches the coast;
3) analysis of wind fields within historical storms of the last century or so, and
4) uncertainty in the risk estimates due to decadal-scale variations in storm frequency and intensity.

This effort will also investigate innovative means to reduce the number of storms required to be simulated while maintaining high fidelity in the simulations. Presently, the IPET study is simulating about 1300 storms to characterize storm surges in the New Orleans area alone. For this effort this number will be reduced to about 200-300. However, one of the issues that still must be addressed in these approaches is the large number of storms required to populate the probability matrices for a large coastal region, such as the entire Southern Louisiana coastline. The number of surge/wave runs required increases approximately linearly with the length of coastline included within this study.

L.2.7 INTERCEPTED DRAINAGE

L.2.7.1 General

The location and size of the levees for the various levee alignments that are under consideration presents considerable challenge for drainage designers. Where existing levees are being considered for increased levels of protection, existing drainage facilities such as pumping stations, culverts, water control structures etc. will need to be modified

or replaced due to the expected increase in the heights of the proposed levees and increased widths necessary for levee stability. Other design issues that must be considered concern the need to pump during a severe "Category 5" storm surge. Water levels and expected differential heads on even the newest modern pumping stations will severely reduce their existing pumping capacities, thus making the protected area vulnerable to flooding from rainfall.

L.2.7.2 Preliminary Analysis

For the preliminary report, it was not possible to undertake and complete interior drainage designs, as there was not enough time. As a first step the interior drainage design team conducted an inventory of all available numeric hydrodynamic models that have been developed for South Louisiana. The intent of the inventory was to determine the suitability of these models for drainage design purposes. Fortunately due to a robust Civil Works program in the New Orleans District, there has been a considerable body of hydrodynamic work done to support this program. Also, as in the case of the Pontchartrain Basin the academic community (University of New Orleans) has been active with their research program in the development of three dimensional hydrodynamic/water quality model for Lakes Pontchartrain and Borgne Basins. Other actions taken for the preliminary report included developing a list of all pumping stations, drainage structures and navigation structures located in South Louisiana that will be affected by any of the proposed levee alignments. The alignments under consideration at this time provide a continuous barrier across the entire State of Louisiana. With the exception of large rivers, for the most part where the line of protection crosses existing bayous, canals and navigation channels, it will be necessary to provide structures that can be closed when a hurricane approaches. For preliminary design purposes these structures were established using the types and sizes of structures proposed in Morganza to the Gulf project. The Morganza to the Gulf project provides a template for the rest of State except in cases where large tidal passes require much larger structures i.e. the Chef and Rigolets between lake Borgne and Lake Pontchartrain and in Barataria basin where the Proposed GIWW alignment crosses the bayou connecting Lake Salvador to Bayous Perot and Rigolets. At these locations these structure sizes were conservatively estimated so that their open area closely approximates the existing cross sectional available for flow.

L.2.7.3 Approach to the FTR

Where new levees are proposed, intercepted drainage and thus the existing flow patterns for the drainage system will be altered, as is the case typical of any levee construction. The scope of investigations for the intercepted drainage designs will because of time limitations necessarily be limited to less than feasibility scope. The intent of the FTR will be to evaluate the existing drainage systems to the extent that the primary drainage systems, i.e. rivers, bayous, tidal passes, canals and inlets are evaluated for their existing flow carrying capacities for a full spectrum of rainfall and runoff conditions, including astronomical and wind tides as produced by non-tropical frontal systems. Where new drainage, navigation, surge barrier structures are proposed we will size those facilities to insure that they provide to the maximum extent practical, flow carrying capacities and

velocities commensurate with the existing system. Where existing flow conditions are critical to survival of larvae state species, care will be taken to insure that no excessive turbulence above the ambient conditions occurs. In some cases where major structures are being considered in tidal passes, it will be necessary to also consider salinity regimes for the existing conditions and for the proposed project condition. Alteration of secondary and tertiary drainage systems will inevitably produce local environmental consequences and ecosystem responses if provisions are not taken to minimize these impacts. To that end, this report will recommend the use of environmental structures that will allow drainage through the proposed levees so that these adverse effects are minimized. For the purposes of this report, the environmental structures will be added to the levee alignments to try to minimize the secondary and tertiary drainage effects but it will not be possible to fully model all of them. Environmental structures will be designed for each appropriate location, but will generally consist of box culverts with sluice gates constructed through the base of the levee. Culverts will have inverts to match existing waterways to which they connect, typically El. -3 to -6. Culverts will be sized to preserve nearly the same quantity of without-project flow, generally 6x9 or 9x9 feet in size. Where larger flows need to be accommodated, multiple culverts can be provided.

There have been many prior studies involving numerical models by the Corps and others that have simulated the hydrodynamic characteristics of the various drainage basins in south Louisiana. Because of time limitations and the need to address intercepted drainage in this report it was decided to draw from this body of work the amassed knowledge of prior investigators and where possible enlist those individuals and their models in the evaluation of the intercepted drainage. Table L-26 below list the various models that have been identified and will be used to conduct the intercepted drainage designs for the Final Report.

TABLE L-26
Models For Interior Drainage Design

Location	Models	Source of Model
	Pontchartrain Basin	
Lake Pontchartrain/Lake Borgne	FVCOM/POM	University of New Orleans
Four Parish Area – Orleans, St Bernard, Jefferson, St Charles	HECRAS & HMS	IPET, New Orleans District COE
	Breton Basin	
Leveed areas - Plaquemine & St Bernard	HECRAS & HMS	IPET, New Orleans District COE
	Barataria Basin	
Leveed areas	HECRAS & HMS	Donaldsonville to the Gulf Feasibility Study, IPET
Un-Leveed/Leeved	TABS/RMA2&4	New Orleans District
	Terrebonne Basin	
Leveed & Un-Leveed Areas	ADH/HMS/TABS	Morganza to the Gulf Feasibility Study, ERDC & Others
	Atchafalaya Basin	
Un-Leveed	TABS/RMA2&4	New Orleans District
	Chenier Plain	
Leveed & Un-Leveed Areas	Mike 11 & 21 HMS	University of Louisiana Lafayette

L.3 GEOTECHNICAL

L.3.1 GEOLOGY

L.3.1.1 Geologic Formations

The history of the study area is dominated by deltaic growth. The Mississippi River deltaic plain is composed of two active and several inactive deltaic complexes extending some 180 miles across southeast Louisiana. Several major deltaic complexes have formed during the last 7,000 years and have been identified in coastal Louisiana. These complexes reflect changes in the course of the Mississippi River. From oldest to youngest, the abandoned delta complexes are the Maringouin, Teche, St. Bernard, Lafourche, and Plaquemine. The active complexes include the Atchafalaya and Modern deltas. The relative ages of these complexes are well established, but the absolute ages are less accurate. Ages were derived from radiocarbon data published in previous studies (Frazier, 1967 and McFarlan, 1961) as well as archeological evidence (McIntire, 1958).

Progradation of these deltas is responsible for the formation of two distinct geomorphic regions; the deltaic plain in the central and southeastern portions of coastal Louisiana and the chenier plain in the southwestern part of the state (see Figure L-23). Progradation of the present and former Mississippi River courses and deltas are responsible for creating the recent alluvial valley and deltaic plain of southeastern Louisiana. Each time the Mississippi River has built a major delta lobe seaward, it has subsequently been abandoned in favor of a shorter, more direct route to the sea. These meander belt changes in the alluvial valley and accompanying shifts in centers of deposition have resulted in the distribution of deltaic sediments along the coast of central and southeast Louisiana. Soon after a delta lobe is abandoned, marine transgression caused by compaction and subsidence of deltaic sediments begins. The end result of this long period of deltaic sedimentation has been the formation of a vast expanse of marsh and swamp separated by abandoned courses and distributaries.

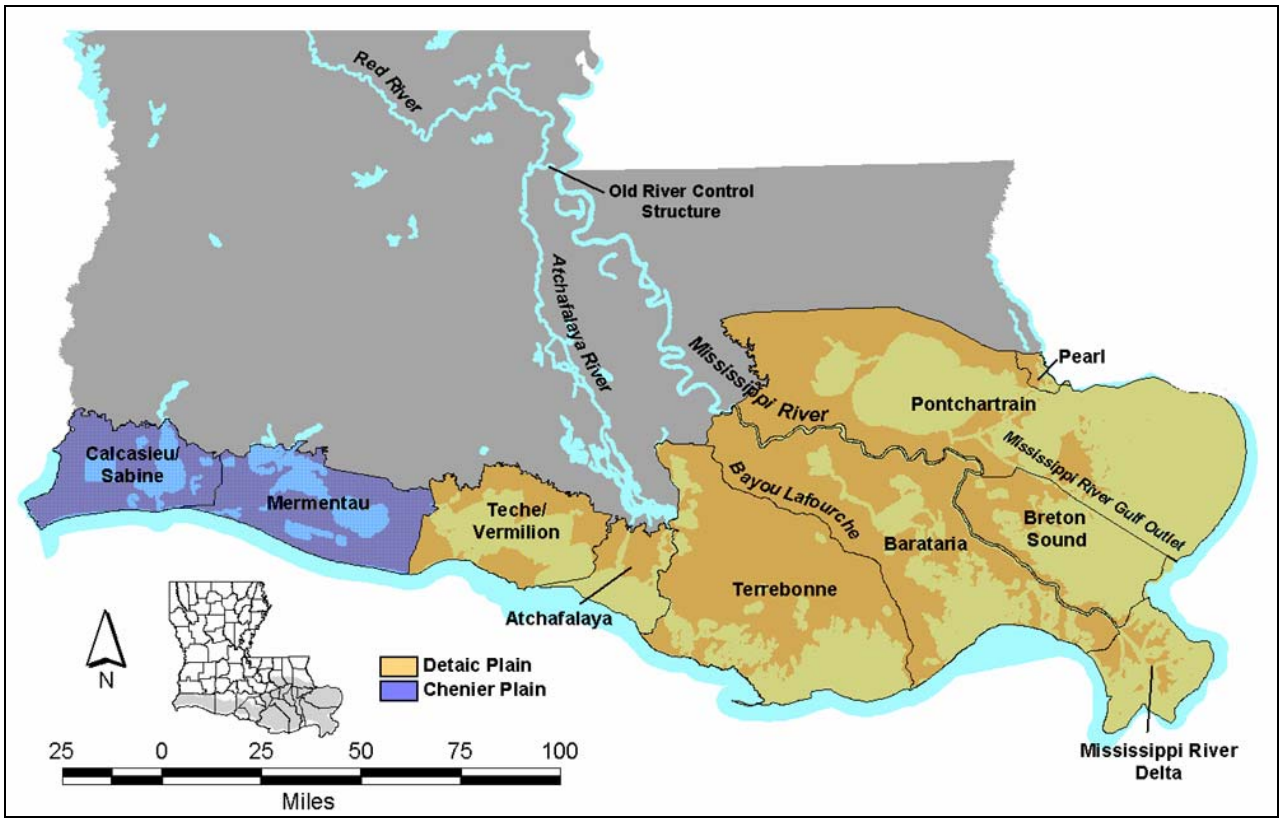


Figure L-23 Location of Chenier and Deltaic Plains.

The deltaic plain is generally composed of a characteristic suite of depositional environments which include marsh, swamp, natural levee, interdistributary, prodelta, and Pleistocene. At the surface and shallow subsurface, marsh, swamp, and natural levee deposits make up the majority of the depositional environments. Marsh deposits range from 1 to 20 feet thick and are generally composed of very soft, organic clays and peat, with high water content and low strengths. Swamp deposits vary widely in thickness, water content, and strength, and are characterized by soft to stiff clays with some silt, wood, and organics. Natural levee deposits border the hundreds of abandoned distributaries and courses located throughout the deltaic plain. They reach a maximum of 30 feet in thickness and are generally composed of oxidized, medium to stiff, clays and silty clays having relatively low water content and high strength. Located beneath marsh, swamp, and natural levee deposits are interdistributary deposits. Interdistributary deposits are up to 200 feet thick and are characterized by very soft to soft clays with some silt and shells. Interdistributary deposits have relatively high water content and low strength. Prodelta deposits are generally found beneath interdistributary deposits. They are composed of medium clay with relatively low water content and moderate strengths. Prodelta deposits are up to 120 feet thick in the deltaic plain. Prodelta clays sit stratigraphically above Pleistocene deposits. Pleistocene deposits represent the best foundation material in coastal Louisiana. They typically consist of interbedded, highly oxidized, stiff to very stiff, clay, silt, silty sand, and sand. These deposits have low water content and high strengths. In the central portion of the deltaic plain, where the ancestral

Mississippi River entrenchment is located, substratum sands are located beneath prodelta deposits.

In contrast to the deltaic plain, the chenier plain formed by longshore transport of fine-grained Mississippi River sediments that were deposited to the west of the deltaic plain. These sediments, transported by westward flowing nearshore currents, were eventually deposited along the existing shoreline as mudflats. When deposition ceased or declined due to shifting Mississippi River courses, these deposits were reworked by coastal processes, concentrating the coarse grained sediments, and forming shore-parallel ridges called "cheniers" (Gould and McFarlan, 1959; Byrne et al, 1959). Introduction of new sediment by westward shifts of the Mississippi River delta resulted in the isolation of these ridges by accretion of new material on the existing shoreline. Numerous cycles of deposition and erosion have been responsible for creating the alternating ridges separated by marshlands, which are characteristic of the chenier plain. The Atchafalaya River is currently supplying the chenier plain with fine sediments via westward flowing longshore currents.

In the northern portion of the chenier plain, marsh deposits up to 10 feet in thickness overly Pleistocene deposits. In the southern half, chenier ridges up to 10 feet thick, composed of sand and shell material separated by marsh deposits approximately 5 feet thick, overly approximately 20 feet of lacustrine, tidal flat, and Gulf bottom clays. These clays are generally soft with relatively high water contents and low strengths. These clays sit directly on Pleistocene deposits.

L.3.1.2 Groundwater

Groundwater is at or near the surface throughout most of the coastal zone. The silt and sand rich depositional environments such as point bar, intradelta, natural levee, beach, and nearshore gulf are generally connected hydraulically to the adjacent water body (i.e. river, lake, distributary channel) and the groundwater level in these deposits generally reflects the level/stage of the adjacent water body. This is especially true in deposits adjacent to the Mississippi and Atchafalaya Rivers.

Numerous deep regional aquifers exist in South Louisiana. The coastal lowlands aquifer system of Louisiana consists of alternating beds of sand, gravel, silt, and clay deposited under fluvial, deltaic, and marine conditions. The aquifer system is comprised of sediment from late Oligocene age to Holocene that thicken and dip toward the Gulf Coast. The sediments are highly heterogeneous with sand beds that are not traceable for more than a few miles. The Chicot aquifer underlies most of southwestern Louisiana and extends from central southwestern Louisiana to the Gulf of Mexico and from Sabine Lake to St. Mary Parish. The Chicot aquifer is up to 800 feet thick at its most northern extent and extends to an unknown depth beneath the Gulf of Mexico. The Southeastern Louisiana aquifer system, also known as the Southern Hills aquifer system, consists of about 30 named aquifers. The Southeastern aquifer extends approximately from the Mississippi River to the Pearl River in Louisiana. The aquifers range in thickness from 50 to 1100 feet with thickness increasing toward the south.

L.3.1.3 Relative Subsidence

The entire coastal zone is experiencing relative subsidence. Relative subsidence is defined here as the net effect of numerous processes that result in the downward displacement of the land surface relative to sea level. Relative subsidence is controlled by several factors which include eustatic sea level, geosynclinal downwarping, compaction of Holocene deposits, and faulting. Recent studies have shown that subsurface fluid withdrawal may also be a major contributor to relative subsidence and resulting wetland loss (Morton et al, 2002). An important man-made contributor to relative subsidence is drainage for agriculture, flood protection, and development.

Eustatic sea level refers to the global fluctuations in sea level primarily due to changes in the volume of major ice caps and glaciers, and expansion or contraction of seawater in response to temperature changes. Recent studies have predicted an increase in the rate of eustatic rise due to global warming (IPCC, 2001).

Downwarping of the Gulf Coast Geosyncline accounts for a small percentage of the observed relative subsidence in coastal Louisiana (Kolb and Van Lopik, 1958). For millions of years, fine sediments have been deposited along the continental margin downwarping the basement and creating a gradually subsiding trough. The downwarping continues as new sediments are added to the basin.

Compaction of Holocene deposits has been considered the primary contributor to relative subsidence in the coastal plain. The three major components of Holocene sediment compaction include 1)primary consolidation, 2) secondary compression, and 3) oxidation of organic matter (Terzaghi, 1943; Roberts, 1985). Primary consolidation occurs as the volume of the soil mass is reduced due to dewatering under a sustained load. Secondary compression results from a decrease in soil volume due to rearrangement of the internal soil structure. Oxidation of organic matter through chemical reactions reduces the soil volume.

Compaction of Holocene sediments varies widely throughout the coastal zone and is closely linked to the thickness and age of deposits. Fine-grained deposits with high water contents characterize the coastal zone. Where thicker the deposits are found, there is more interstitial water available for removal, which leads to high rates of relative subsidence as they compact. Older deposits have already undergone most of the primary consolidation and secondary compression and therefore exhibit lower relative subsidence rates than recently deposited sediments. The age, thickness, and to some extent the type of deposits are responsible for much of the variability in relative subsidence rates across the coast.

Movement on the downthrown side of deep-seated fault blocks is a well documented process in coastal Louisiana. However, there effects on the shallow subsurface and surface are poorly understood. Recent investigations (Gagliano, 2005) have identified likely areas of fault-induced subsidence but the magnitude and spatial extent of their

impact are still being investigated. The Baton Rouge fault is probably the best known example of an active fault that has caused some structural damage. This fault crosses the proposed alignment approximately 6 miles south of Slidell. Most fault planes in coastal Louisiana generally trend east-west and may contribute to increased maintenance at the location where they intersect the proposed project. A minor amount of movement along fault planes can have significant impacts on wetlands where marsh accretion barely exceeds relative subsidence.

Forced drainage of wetlands results in lowering of the water table resulting in accelerated compaction and oxidation of organic material. Areas under forced drainage can be found throughout coastal Louisiana.

Relative subsidence rates vary considerably across coastal Louisiana. In general, natural rates of relative subsidence are highest near the coast and at the mouth of the Mississippi River where young thick sediments are present. Reported rates range from less than 0.5 feet to over 5.0 feet per century. Relative subsidence rates of several feet have been documented in developed areas of Jefferson and Orleans parishes due mainly to forced drainage, and large areas of coastal land loss are found associated with failed land reclamation projects. Currently, no coastwide system for quantifying and predicting relative subsidence on a regional scale has been established. Therefore, relative subsidence rates are generally estimated using a combination of benchmark leveling, tide gauge measurements, and radiometric dating of buried marsh horizons.

L.3.1.4 Borrow Sources

Large volumes of clay, silt, and sand will be required for construction of the proposed project. Because the proposed project extends across the entire State, numerous borrow sources will be required. Potential silt and sand sources include the Mississippi, Atchafalaya, and Vermilion Rivers, as well as tidal deltas and offshore shoals in the Gulf of Mexico. Sources of silt and clay include natural levee deposits of past and present Mississippi River courses and Pleistocene terrace deposits.

Detailed geologic investigations focusing on foundation conditions, groundwater, relative subsidence, and borrow sources will be conducted for the FTR and during PED.

L.3.2 GEOTECHNICAL DESIGN

L.3.2.1 General

For the PTR, a compressed design schedule coupled with the large scope of work (from Sabine River to the Pearl River) necessitated a generalized assessment of the geologic foundation conditions in the project area. The alignment was divided into four similar reaches or areas where the soil layering (stratigraphy) remains relatively consistent. Soil unit weights and shear strengths of the strata were assigned to each of the four reaches based upon geotechnical engineering experience in the region with various projects in the vicinity. The objective to provide comparisons for three different levels of hurricane protection

levees and two different methods of levee design was achieved. Certainly, more detailed designs based upon site-specific foundation conditions will result in costs advantages in some reaches and cost disadvantages in others; however, the level of geotechnical reliability will be greatly enhanced.

L.3.2.2 Foundation Design Reaches

The three levels of protection analyzed were for El. +25, +30, and +40. Existing ground for all reaches was assumed to be zero for all alternatives. Reach 1 extended from the Lake Pontchartrain south shore at the Rigolets through Orleans, St. Bernard, Plaquemine, Jefferson, and Lafourche Parishes to Larose, Louisiana and consisted of a 30-foot Marsh deposit and a 30-foot Interdistributary deposit overlying a 60-foot Prodelta deposit and then Pleistocene clay. This reach consisted of the worst soils with regards to shear strength and settlement potential. Reach 2 extended from Larose through Terrebone Parish to Gibson, Louisiana and consisted of a 10-foot Marsh deposit, a 20-foot Interdistributary deposit, a 90-foot swamp deposit and then substratum sands. Reach 2 was considered to be slightly better soils than Reach 1 with regards to shear strength and settlement potential. Reach 3 extended from Gibson through St. Mary and Iberia Parishes to the Cote Blanche Salt Dome and consisted of a 15-foot Marsh deposit and then an 85-foot swamp deposit overlying substratum sands. Reach 3 was considered to be slightly better soils that Reach 2 with regards to shear strength and settlement potential. Reach 4 extended from Cote Blanche through Vermilion, Cameron, and Calcasieu Parishes to the Sabine River and consisted of a 10-foot Marsh deposit overlying Pleistocene clay. Reach 4 also was assumed to encompass from the north shore of Lake Pontchartrain to the Pearl River near Slidell, Louisiana due to similar foundation conditions as in southwest Louisiana. Reach 4 consisted of the best soils with regards to shear strength and settlement potential.

L.3.2.3 Typical Design Sections

The two methods of levee design considered for each reach were soil-cement columns and geotextile-reinforced embankment. See figures L-24 and L-25 and plates L-1 thru L-14 for typical sections for each reach. Each of these methods produces reduced berms and less levee footprint than a levee without these improvements. While the geotextile-reinforced embankment produces reduced berms and footprints by reinforcing the embankment, the soil-cement columns actually improves the foundation soils below the levee footprint by the Deep Mixing of cement in a series of “columns” and drastically improves the shear strength and compressibility of the clay soils.

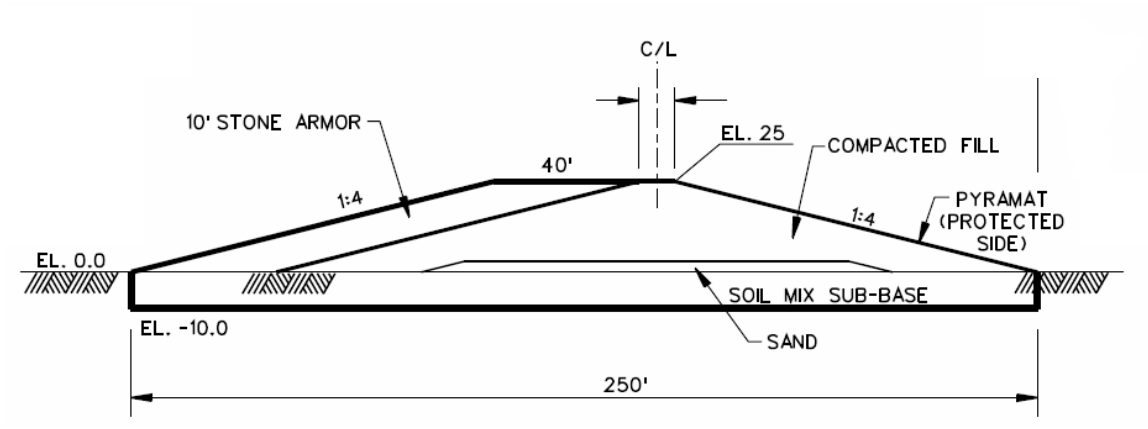


FIGURE L-24 Typical Soil-Cement Section

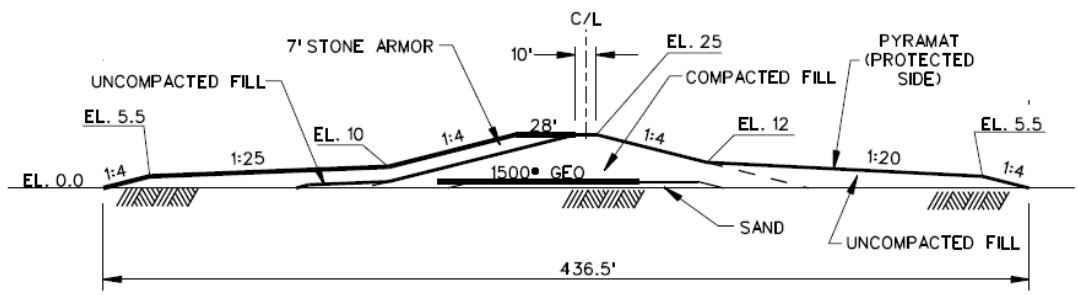


FIGURE L-25 Typical Geotextile-Reinforced Embankment Section

Deep Mixing Technology would be used to inject a binder mixture into the upper soft to very soft soils. In all reaches except Reach 4, a 60-foot depth column was used, while in Reach 4 the depth was reduced to 10-feet due to the presence of shallow Pleistocene clays at this depth. In this case, Type I Portland cement would be used as the binder, although some additives, such as fly ash or lime, could be considered.

Two Deep Mixing processes are commonly used. In the Dry Mix process, the binder mixture is injected into the soil pneumatically through the mixing tool. The natural moisture content of the soil is used in this process to hydrate the cement, which gives the improved foundation its more competent characteristics. In the Wet Mix process, the end result is the same, but the method of mixing the binder with the soil is somewhat different. The binder is first mixed with water and then injected hydraulically. This creates a slurry column, which cures in a way similar to that from the Dry Mix process. Each method has advantages and disadvantages, but either would work for this application.

For all reaches and levels of levee embankment, 30% of the foundation soil would be replaced with soil cement columns. The columns would be approximately 2.6-feet in diameter and would be placed in an overlapping pattern in rows 7-feet on center. The rows would extend from the protected-side levee toe to the flood-side toe of the armored rock. Where berms are present and extend beyond the toe, the soil-cement rows would not continue past where the main levee slope on the protected-side or the armored rock slope on the flood-side would have intersected the natural ground if no berms were present. The columns would be installed perpendicular to the levee centerline to serve as hard clay shear walls, and would achieve shear strength of approximately 6,000-psf. The equivalent shear strength of the soil-column matrix beneath the levee would be 2, 000-psf for a 30% replacement ratio.

Immediately after the soil cement columns are installed, the compacted earthen levee could be constructed by conventional means. Construction lifts would have to be sequenced during the initial 28 days curing time to prevent overloading the columns before they reach their full design strength. This would be dictated by the site-specific in-situ foundation conditions. After 28 days curing time, the levee could be raised to its full height. A layer of geotextile varying in strength from 1460 to 1640 pounds-per-inch at 5% strain (depending upon the reach) is required for internal stability within the levee embankment only for the El. +40 alternative.

Flood-side slope protection was designed by the Coastal Structures Group of ERDC and consisted of varying thicknesses of rock from the levee crown down to the levee or berm toe. The rock thickness, since it was very considerable, was taken into account for all stability analyses. Although its thickness added to the active loading near the levee crown, it helped the levee stability in the flood-side berms. Therefore, construction of the levee will be staged, such that the levee embankment will be limited to a certain elevation (to be determined during detailed design) until the rock is placed in the flood-side berm, then levee embankment will be allowed to proceed to the final levee grade. Thickness of rock varied from 6- to 7-feet for the geotextile-reinforced levee alternative to 8- to 10-feet for the soil-cement levee alternative. The amount of rock cover is dependent upon the levee and berm slopes; the flatter the slope, the smaller the amount of rock required for protection due the wave-breaking effects of the longer slope. A separator fabric is also recommended between the rock armoring and the levee fill to prevent migration of rock into the fill material. Overtopping protection would also be provided on the crown and protected-side levee slope but was not considered significant enough to include in the stability analyses due to its limited height and extent over the levee.

For both alternative levee designs for all reaches, a three-foot sand base was included to provide a base for construction activities. The sand base was limited in extent such that 10-feet of clay cover would be provided above the sand due to seepage concerns. Semi-compacted clay fill was assumed for all levee and berm material. This will provide the necessary strength for the levee embankment height and for the berm to support the rock armoring protection layer.

The length of geotextile required for each section varied from 75- to 320-feet for the geotextile-reinforced alternative. The required strength at 5% strain of this geotextile varied from 200- to 2,020-pounds-per-inch. -. In some sections for both geotextile-reinforced levees and soil-cement columns, geotextile reinforcement is required to stabilize the flood-side berm. The length of geotextile required for some sections of the soil-cement alternative varied from 40- to 190-feet, and the required strength at 5% strain of this geotextile varied from 200- to 1,640-pounds-per-inch. This geotextile reinforcement was required for only the internal stability within the levee embankment for only the El. +40 alternative and for the flood-side berm.

All slope stability analyses were performed using the Method of Planes. The flood-side designs provided for a minimum design factor of safety of 1.3. The protected-side designs were evaluated for two water loadings. For the extreme case of water to the levee crest, a minimum design factor of safety of 1.2 was required. For the "operating" case with the water level 10-feet below the levee crest, a minimum design factor of safety of 1.3 was required. The stability analyses for the soil-cement alternative are shown on plates G1 to G24 for each of the four reaches and each of the three levee elevations. Only the controlling water load case is shown for the protected-side stability plates.

The Tables L-27 to L-29 show the required levee footprint for each of the three levee crowns considered. These footprint distances do not include any adjacent borrow pit requirements that may be utilized in areas where feasible and economical. The geotextile-reinforced alternative was not considered practical for the levee El. +40 in Reaches 1 and 2, given the excessive levee footprint of ~1,300-feet required in Reach 3. For levee crown El. +40, the levee required footprints range from 1,035- to 1,281-feet for the geotextile-reinforced alternative and range from 447- to 803-feet for the soil-cement alternative. The required levee footprints for the geotextile-reinforced alternative are 2.3 times larger than the soil-cement alternative.

Table L-27
Levee Footprints El. +40

Crown El. +40 Reach	Geotextile Reinforced Levee Footprint ft.	Soil-Cement Column Levee Footprint ft.
1	--	803
2	--	649
3	1281	553
4	1035	447

For levee crown El. +30, the required levee footprints range from 575- to 1183-feet for the geotextile-reinforced alternative and range from 290- to 440-feet for the soil-cement alternative. The required levee footprints for the geotextile-reinforced alternative range from 2 to 2.7 times larger than the soil-cement alternative.

Table L-28 Levee Footprints El. +30		
Crown El. +30 Reach	Geotextile Reinforced Levee Footprint ft.	Soil-Cement Column Levee Footprint ft.
1	1183	440
2	764	341
3	721	290
4	575	290

For levee crown El. +25, the required levee footprints range from 436- to 832-feet for the geotextile-reinforced alternative and are 250-feet for each of the soil-cement alternatives. The required levee footprints for the geotextile-reinforced alternative range from 1.7 to 3.3 times larger than the soil-cement alternative.

Table L-29 Levee Footprints El. +25		
Crown El. +25 Reach	Geotextile Reinforced Levee Footprint ft.	Soil-Cement Column Levee Footprint ft.
1	832	250
2	538	250
3	522	250
4	436	250

For all levee sections, both geotextile-reinforced and soil-cement alternatives, a minimum Lane's creep value of 3 was achieved. This value is considered acceptable for clay material; therefore, thru-levee seepage will not be an issue for these levee sections. When more site-specific geotechnical data are available, more detailed underseepage analyses will be conducted. These data include minimal areas where sand may exist at shallow levels underneath the proposed levee.

L.3.2.4 Settlement

Foundation settlement is a key component of the estimated levee construction cost, particularly for the geotextile-reinforced levee alternative, which would require numerous lifts to construct to the project design grade. Consolidation and lateral spread of the soft clays in the foundation would produce substantial settlement of the levee foundation. The settlement analyses considered ultimate consolidation settlement based upon empirical values relating liquid limits to the compression index. Additionally, lateral spread was assumed to be 25% of the ultimate consolidation values for the geotextile alternative and 15% for the soil-cement column alternative. Shrinkage settlement was assumed to be 10% of the fill height. The sum of these three values provides an estimate of the ultimate settlement of the levee. Table L-30 to L-32 summarize the estimated ultimate settlement of the proposed levee.

Table L-30 Ultimate Settlement Estimate El. +40		
Crown El. +40	Geotextile Reinforced Levee	Soil-Cement Column Levee
Reach	ft.	ft.
1	------	10.6
2	------	11.6
3	19.7	7.3
4	9.3	5.0

Table L-31 Ultimate Settlement Estimate El. +30		
Crown El. +30	Geotextile Reinforced Levee	Soil-Cement Column Levee
Reach	ft.	ft.
1	20.4	8.3
2	17.4	9.1
3	16.3	5.8
4	7.7	3.8

Table L-32 Ultimate Settlement Estimate El. +25		
Crown El. +25	Geotextile Reinforced Levee	Soil-Cement Column Levee
Reach	ft.	ft.
1	17.8	7.2
2	14.7	7.8
3	14.3	5.0
4	6.8	3.2

The geotextile-reinforced alternative produced settlements ranging from 1.9 to 2.9 times larger than those produced with the soil-cement column alternative.

An empirical relationship relating liquid-limit values to coefficient of consolidation (U.S. Navy, 1971) was used to estimate a hypothetical lift construction schedule for the soil-cement alternative. Table L-33 summarizes the results of this estimation:

Table L-33 CAT 5 Soil-Cement Column Alternative Settlement - Lift Requirements			
Reach	El. +40	El. +30	El. +25
1	2 - 6' Lifts @ t= 6 & 20 yrs.	1 - 7' Lift @ t= 9 yrs.	1 -6' Lift @ t = 11 yrs.
2	2 - 6' Lifts @ t= 5 & 20 yrs.	1 - 7' Lift @ t= 6 yrs.	1 - 7' Lift @ t= 7 yrs.
3	1 - 7' Lift @ t= 7 yrs.	1 - 6' Lift @ t= 8 yrs.	1 - 5' Lift @ t= 12 yrs.
4	1 - 5' Lift @ t= 9 yrs.	1 - 4' Lift @ t= 12 yrs.	1 - 3' Lift @ t= 12 yrs.

Reaches 1 and 2 with crest El. +40 are the only ones that require two subsequent lifts. All other reaches and elevations would only require one subsequent lift. Lift heights vary from 3-feet in Reach 4 El. +25 to 7-feet for Reaches 1 thru 3 at various elevations. The time to first lift varies from 5-years for Reach 2 El. +40 to 12-years for Reaches 3 and 4 at various elevations.

In areas where a straddle enlargement of an existing levee and berm is possible, the settlement is overestimated since it is based on zero initial ground surface elevation. Therefore, the number of lifts and the quantity of levee fill material described in this report is overestimated for those areas.

L.3.2.5 Additional Investigations

The following items will be investigated in the Final Technical Report: 1) increase geotechnical reliability with additional borings along the alignment, 2) study utilizing higher replacement ratio for soil-cement columns to alleviate berms for all reaches for El. +40 and for Reaches 1 and 2 for El. +30, 3) study the feasibility of adjacent borrow for more economical levee construction and effects upon the levee footprint, 4) perform underseepage analyses in areas where shallow sands may exist to determine if seepage berms are required, 5) reevaluate settlement for areas where the proposed levee will straddle an existing levee of substantial grade, and 6) reevaluate settlement to investigate whether soil-cement levee sections could be economically overbuilt to avoid lift construction.

L.4 LEVEE DESIGN

L.4.1 GENERAL

The five project basins were broken down into reaches and sub reaches for ease in comparing alternatives. These sub reaches are shown on plates 1 and 2. Existing LIDAR information was used to develop quantities for both the typical soil founded and soil

improvement sections for each reach. Quantities were developed for 25', 30' and 40' levees.

L.4.2 FLOOD PROTECTION ALTERNATIVES

For the PTR, two typical levee sections were considered. The first is a conventional earthen levee utilizing geotextile layers for load distribution and increased strength. The second is also an earthen levee design, but built upon an improved foundation of soil mix columns. Plates L-1 thru L-14 show typical sections for each reach and elevation.

L.4.2.1 Geotextile Alternative

The first alternative is a clay levee with geotextile reinforcement located in the center just above the base in two layers. The design levee section has 1V on 4H side slopes and a 10' wide crown. The design heights are 25', 30' and 40'. Floodside and landside stability berms are required to provide stability due to the soft foundation. The floodside slope and berm are protected from wave action by an 8' thick stone blanket. The levee crown, landside slope and the landside berm are protected from erosion with a high performance turf reinforcement mat anchored to the levee and berm designed to hold up under over topping conditions. For preliminary design purposes, the proprietary product Pyramat® was selected as representative of the required quality and performance characteristics. In the reaches where the levee is being constructed over wetlands a 3' sand base is placed down the center of the levee to provide a construction platform. Under the poorest foundation conditions the footprint widths for the 25', 30' and 40' high levees are approximately 437', 598' and 1035' respectively. This alternative is not practical to build in soils Reaches 1, 2, and 3 due to severe settlement that would be experienced in the compressible soils.

L.4.2.2 Soil Mix Alternative

The second design alternative is a clay levee supported by cement soil columns. The soil columns are 30" in diameter on 24" centers in rows perpendicular to the levee centerline on 7' centers. The depth of the columns for the best soils condition is 10' and in the poor soils it is 60'. The columns extend between the levee toes for levees without stability berms. Where there are stability berms the soil columns extend approximately 40' past the levee toes. The design section is also 1V on 4H with a 10' wide crown and the design heights are 25', 30' and 40'. On the poorest foundations stability berms are required for the 40' and 30' levees but for the 25' levee stability berms are not required. On the best foundation conditions stability berms are required for only the 40' levee and only on the protected side. The use of geotextile reinforcement is limited to a single layer located in the center just above the base of the levee and only for the 40' high levee. The flood side levee slopes and berm are protected by an 8' thick blanket of stone. On levees without flood side stability berms the stone blanket is increased to 10'. Under the poorest foundation conditions the footprint widths for the 25', 30' and 40' high levee are approximately 250', 440' and 800'. The levee crown, landside slope and the landside

berm are also protected from over topping with a high performance turf reinforcement mat.

L.4.3 CONSTRUCTION CONDITIONS

For the PTR, the project is divided into four soils reaches with Reach 1 having the poorest foundation condition and improving through Reach 4, which has the best foundation condition. Foundation conditions generally improve from east to west across the coastal area of south Louisiana from the Mississippi River delta to the Sabine River. Soil conditions are generally better in southwest Louisiana, where the underlying, firmer Pleistocene soils are found closer to the surface.

Another factor influencing construction conditions is the location of levee design alignments between wetlands and land above sea level, either distributary ridges or the pleistocene prairies that are found in Reach 4. The largest portion of proposed levee alignments falls in wetlands including parallel borrow pits. The second group of design alignments falls along the interface between wetlands and ridges or prairies, with the levee footprint falling in the wetlands and the parallel landside borrow areas falling on higher ground. For the final set of design alignments, both levee and borrow pits are on high ground.

Constructing levees completely founded in wetland requires a sand base along the levee centerline to serve as construction access and a dry working platform. This is typical of the levee alignments along the GIWW and south of the GIWW. In soils Reach 4 located west of New Iberia, the good foundation soils are close to the surface in the wetlands. For preliminary design, 80% of the required quantity of earthen levee material is assumed opposite cast with 20% hauled in from 30 miles away from the pleistocene prairie. Proposed levees east of New Iberia located totally in wetlands have a lower percentage opposite cast: around 40% with 60% haul. East of Golden Meadow the percent goes to 20% opposite cast and 80% haul with haul distances of 50 to 60 miles round trip. Sand for construction access is assumed to be barged from the Atchafalaya River area or Mississippi River. Truck haul to construction sites from the barge to placement is short since waterways run alongside or near the levee alignments. Levee alignments east of Golden Meadow will use sand from the Mississippi River and those to the west from the Atchafalaya River area. Stone armor will be barged in from out of state and will follow for the most part the same access used for the sand levee base from the point of barge delivery.

Construction along the interface between wetland and highland has the levee located on wetland which requires the placement of a sand base for access and to provide a working platform. Most of the borrow falls alongside and parallel to the levee on the protected side which is dry. High percentages of opposite cast are assumed (80% to 90%). The balance required material is assumed truck hauled for a 20-mile round trip from the tributary ridges or pliestocene prairie. Alignments in Plaquemines Parish have extremely limited local borrow sources resulting in an estimated round trip haul of 120 miles. The greatest portion of sand will be barged to the approximate area of the job site since few

waterways are near these alignments. Truck haul from the barge drop off ranges from 20 to 80 miles round trip. Alignments near the Atchafalaya River and Mississippi River sand sources will use truck hauls with 10 to 20 mile rounds trips. Armor stone will be barged in from out of state utilizing access similar to sand, where the material is moved by barge to the approximate area of the construction site.

Levees constructed on pliestocene soils found in Reach 4 have their alignments approximately following the 10' contour in the southwest part of the state. Favorable soil conditions classified as Reach 4 also occur at an additional short reach on the eastern end of the levee alignments on the north shore of Lake Pontchartrain north and east of Orleans Parish. All of these levees will be constructed using 90% opposite cast and 10% truck haul with a round trip of 10 miles. Borrow pits are all located on pliestocene soils. Sand is not required for construction access on these alignments. Armor stone for these alignments is truck hauled between 10 and 20 miles round trip from the nearest waterway which allows barge access.

L.4.4 AVAILABILITY OF BORROW

While assumptions have been made regarding the availability of local borrow for levee construction, this is a critical construction issue that warrants much closer analysis. The quantity of material required to build even the shortest of the proposed alignments will be in the hundreds of millions of cubic yards. Finding such a quantity of suitable material in south Louisiana would be difficult as well as environmentally undesirable. An investigation into borrow sources will be conducted for the Final Technical Report to both confirm the constructability of alternatives and to develop proper cost estimates

L.5 STRUCTURES

L.5.1 GENERAL

No hurricane protection project could span coastal Louisiana without crossing dozens of navigable waterways, distributaries, urban drainage canals, and roadways. More than one hundred locations will require structures as part of the LACPR project. For preliminary design, a set of typical structures was designed using a parametric approach to parallel the levee design effort. With final grades unknown until hydraulic analyses were completed, the team developed schematic designs for the necessary structures to assumed elevations of 35, 40 and 45 feet.

Structures designed for the proposed project generally fall into the following categories: sluice gate structures, sector gated structures, roller/swing gate structures, tainter gated structures, butterfly gate structures, locks, box culvert structures, new pumping stations, and hardening and fronting protection for existing pumping stations. A description of each type of structure is shown below. Plates S1 – S23 show typical sections and other details.

L.5.2 SLUICE GATE STRUCTURES

Drainage structures with sluice gates will provide drainage through the flood protection at various locations along the study area. Each structure will consist of a pile founded, reinforced concrete structure with trash screens, operating platforms, and provisions for dewatering. A 24-foot wide roadway will provide access across the structure. The sluice gate structures will connect into the existing flood protection on each side of the structure with a "T"-wall. The concept for these structures was taken from the 2003 Morganza to the Gulf of Mexico, Bayou Grand Caillou Alternative Study. The sluice gates will have the capability to be operated manually or will be mechanically actuated with portable motors.

Two different structures of this type are included in the study. One will have 4 - 10' x 10' sluice gates, an invert of El. –12.0 and will be utilized at several locations along the study alignment. A second structure of this type will have 4 or 5 – 8' x 10' gates and an invert at El. –8.0. This structure will be utilized at locations where the intersecting waterways that are smaller and shallower. These structures are also provided at locations where previous studies envisioned the use of multi-barreled box culverts. Given the levee sizes considered in the study, a pile founded, sluice gate structure appears to be much more practical and cost effective than long box culverts under wide and tall levees. A third type of sluice gate structure will be provided where only a few small openings are required to pass drainage flows. This structure will be a modified, inverted T-wall monolith with 5' x 5' surface mounted gates and an invert at El. –5.0. The number of gates at each site will be based on the flow requirements.

A detailed load analysis, considering both vertical and horizontal loads, was performed on each of the typical sluice gate structures for each of the three study levels of protection. For the T-type structure, the analysis considered an inverted T-wall structure with the top of the base at El. –5.0 and an increased wall depth to accommodate the gated openings at the base of the wall.

A foundation plan was developed for these structures using the Corps program CPGA (X0080) and the calculated vertical and horizontal loads. Typical pile capacities were assumed to be in excess of 100 tons per pile and were based on foundation capacities presented herein with a factor of safety of 3.0 for this level of study. Minor overstresses were allowed in the foundation piling.

L.5.3 SECTOR GATE STRUCTURES

Sector gated structures will be provided as part of the closure and flood protection to provide for normal navigation at many of the waterways intersecting the flood protection alignment. These structures were sized based on the apparent width of the existing waterway. The sill elevation at each location was selected based on the prevailing bottom elevation at the site and is typically either El. –9.0 or El. –15.0. Standard sector gate widths of either 56 feet or 110 feet were used in this study. A special structure, discussed elsewhere, that is 250 feet wide with a deeper sill will be provided at the

Mississippi River-Gulf Outlet and at the Houma Navigation Canal where larger vessels are anticipated. Each sector gate structure will be a pile founded, reinforced concrete structure at the required sill elevation and width to maintain navigation in the waterway. Double sector gates were provided at waterways with the highest levels of navigation and/or at major waterways where the number of vessels seeking safe harbor would be the highest. The double-gated structure will accommodate the higher traffic volume in either case. The structure will have emergency and/or maintenance stop logs and separate control houses on each wall. A timber guidewall with a protective cellular dolphin at the end will be provided on both sides of each approach channel to the structure.

A detailed load analysis, considering both vertical and horizontal loads, was performed on a typical interior monolith for several of the required sector gate structures. The detailed analyses considered two of the three study levels of protection. A foundation plan was developed for these structures using graphical methods and the calculated vertical and horizontal loads. Given the scope of the study, minor over-stresses were allowed in the foundation piling. Pile capacities were based on foundation capacities presented herein with a factor of safety of 3.0 for this level of study.

L.5.4 SPECIAL SECTOR GATE STRUCTURES

Special, 250-foot wide sector gated structures are assumed as part of the closure and flood protection at the Mississippi River-Gulf Outlet (MRGO) and at the Houma Navigation Canal (HNC) waterways to provide for the navigation needs in these waterways. The sill elevation at each location was selected based on the prevailing bottom elevation at the site and are, for the purposes of this preliminary analysis, El. –40.0 at the MRGO and El. –20.0 at the HNC. Each sector gate structure will be a pile founded, reinforced concrete structure at the required sill elevation with the appropriate width to maintain navigation in the waterway. The structure will have emergency and/or maintenance stop logs and separate control houses on each wall. Due to the nature of the navigation in these waterways, a cellular, concrete guidewall with protective dolphins at the end will be provided on both sides of each approach channel. These guidewalls will also be longer than those provided at the typical sector gated structure. At these locations, the overall depth of the structure will necessitate the use of adjacent, non-overflow sections until the channel slope becomes high enough to use conventional T-walls for the tie-in to the adjacent levees. At the deeper structures, with sill elevations at or below El. –20.0, the use of floodwalls is not adequate to “span” between the deep primary structure and the shallower foundation for the floodwall structure. Given their overall height, these structures must resist lateral loads nearly as high as the primary structure. The non–overflow monoliths are substantial structures with wide bases to bridge the deeper sections of the channel and/or excavation until the use of the standard floodwalls was deemed appropriate.

The main structure is envisioned as a single segment to be floated into position, however the adjacent non-overflow sections and the tie-in floodwalls were considered to be floated into position in separate phases after the navigation structure has been completed enough

to allow for some minimal navigation. The basis for this method of construction is the construction plan for the Inner Harbor Navigation Canal Lock Replacement.

A detailed load analysis, considering both vertical and horizontal loads, was performed on the structure required for an El. 45.0 level of protection. A foundation plan was developed for these structures using graphical methods and the calculated vertical and horizontal loads. Pile capacities were based on foundation capacities presented herein with a factor of safety of 3.0 for this level of study.

The MRGO continues to be the focus of much discussion regarding the hurricane protection system, with various interests proposing significantly modifying or closing the navigation channel. The design provided in the PTR assumes existing conditions will remain, which they will unless and until federal legislation directs otherwise. The FTR will address the implementation of any Post-Authorization Change that may result.

L.5.5 BUTTERFLY GATE STRUCTURE

A butterfly-gated structure will be provided at the Rigolets Pass as part of the closure complex for Lake Pontchartrain at this location. This structure was sized based on the available flow area in the Rigolets and is presently deemed necessary to allow for flows through the Rigolets for as long as possible. The sill elevation at the Rigolets was selected based on the prevailing bottom elevation in the channel. Butterfly gates open and close on their own in response to head and flow conditions due to the offset pintle location. The butterfly gate design allows for the automatic operation of the gates in case of a reversal in the direction of flows through the pass thus allowing for continual uninterrupted flow through the Rigolets Pass.

The tainter and butterfly gated structures at this location will be constructed separately so as not to cutoff flow through the pass. The structure will be constructed in the dry with a series of sheet pile cells for the cofferdam. The assumed number of gates across the pass was based upon a rough calculation of the end area of the pass at the nearby U.S. Hwy 90 bridge as presented in the 1973 Lake Pontchartrain and Vicinity Barrier Plan, Rigolets Control Structure and Closure Plan DDM. The concept for the butterfly-gated structure will be similar to that presented in the 1989 Lake Pontchartrain & Vicinity, High Level Plan, London Avenue Outfall Canal GDM.

The structure will be a pile founded, reinforced concrete structure at the required sill elevation with steel butterfly gates to maintain the flow conditions in the waterway. A two-lane highway bridge will be incorporated into the structure for access and maintenance. The structure will have emergency/maintenance stop logs and 4 cellular dolphins to protect the structure.

A typical section of a butterfly-gated structure is depicted in the study drawings.

A detailed load analysis, considering both vertical and horizontal loads, was performed on a typical interior monolith with 2-40-foot wide bays and 12-foot wide divider piers.

The detailed analyses considered two of the three study levels of protection. A foundation plan was developed for the structure using the Corps program CPGA (X0080) and the calculated vertical and horizontal loads. Typical pile capacities were based on foundation capacities presented herein with a factor of safety of 3.0 for this level of study.

L.5.6 SECTOR GATED LOCK STRUCTURES

Sector gated lock structures will be provided as part of the closure and flood protection at almost all of the larger, navigable waterways. The sill elevation at each location was selected based on the prevailing bottom elevation at the site. A lock width of 110 feet was generally used in this study. The gate monoliths at each lock structure will be a pile founded, reinforced concrete structure at the required sill elevation. The floodside gate monolith will extend to the required study elevation and will also serve, along with the tie-in floodwalls, as the main line of protection. The lock walls for the protected side gate and the lock chamber will extend only up to El. 9.0. In general the chamber sections will be earthen with timber guidewalls with rock dikes and riprap for scour protection. At Amelia and Calcasieu the design assumes a pile founded concrete chamber section. Each gate structure will have emergency and/or maintenance stop logs and separate control houses to operate the gates. Timber guidewalls and cellular dolphins that serve to protect the guidewalls will be provided on both sides of each approach channel.

A detailed load analysis, considering both vertical and horizontal loads, was performed on each of the gate monoliths at each lock. The detailed analyses considered two of the three study levels of protection. A foundation plan was developed for these structures using graphical methods and the calculated vertical and horizontal loads. Given the scope of the study, minor over-stresses were allowed in the foundation piling. Typically, steel pipe piles were used for the taller, floodside gate monoliths while 14” square prestressed concrete piles were provided for the foundations for the protected side gate monolith. Pile capacities were based on foundation capacities presented herein with a factor of safety of 3.0 for this level of study.

L.5.7 TAINTER GATE STRUCTURES

Tainter gated structures will be provided as part of the closure and flood protection at the larger waterways. These structures were sized based on the available flow area in the existing waterway. The sill elevation at each location was selected based on the prevailing bottom elevation at the site. Tainter gate widths of either 50 feet or 63 feet were used in this study. The gate radius was determined using the required sill elevation anticipated location of the trunnion girder. The basic layout for the tainter gate structures will be similar to the Old River Auxiliary Control Structure and the Red River Waterway dams. Each tainter gated structure will be a pile founded, reinforced concrete structure at the required sill elevation with the appropriate number of steel tainter gates with post-tensioned anchorages to maintain the flow conditions in the waterway. A two-lane highway bridge will be incorporated into the structure for access and maintenance. The structure will have emergency and/or maintenance stop logs, an exposed cantilevered

machinery platform and four cellular dolphins to serve as guardwalls for the structure. The location and size of the tainter gated structures in the Study are shown in Table L-34.

TABLE L-34
SUMMARY – TAINTER GATED STRUCTURES

STRUCTURE LOCATION	SILL ELEVATION	GATE WIDTH	NUMBER OF GATES
Bayou Grand Caillou	-12.0	50 feet	4
Bayou Perot	-16.0	50 feet	16
Mermentau River	-12.0	50 feet	5
Vermillion River	-12.0	50 feet	4
Calcasieu River	-25.0	63 feet	8
Chef Pass	-25.0	63 feet	6
Rigolets Pass	-30.0	63 feet	15

A detailed load analysis, considering both vertical and horizontal loads, was performed on a typical interior monolith for several of the required tainter gate structures. The detailed analyses considered two of the three study levels of protection. A foundation plan was developed for these structures using the Corps program CPGA (X0080) and the calculated vertical and horizontal loads. Minor over-stresses were allowed in the foundation piling. Typical pile capacities were assumed to be in excess of 110 tons per pile and were based on foundation capacities presented herein with a factor of safety of 3.0 for this level of study.

L.5.8 ROLLER/SWING GATE STRUCTURES

Preliminary design considerations indicated that the use of swing gates was not practical for the various gate sizes under consideration for this study. The gate hinge loads for this type of gate would be quite excessive due to the large gate weights and, in addition, moving such a large gate in windy conditions was deemed impractical and dangerous. For these reasons, all required highway and rail crossing structures were evaluated as conventional roller gate structures with both a combined storage monolith and tie-in floodwall structures on each side to transition into the full levee section.

Each gate will be supported by a pile founded, reinforced concrete base and walls. A cursory load analysis, considering both vertical and horizontal loads, was performed on a typical roller gate structure for each of the three study levels of protection. A foundation plan for the gate and storage structure, using 24-inch diameter pipe piles, was developed for these structures using graphical methods.

L.5.9 INVERTED “T” TYPE WALLS

Each T-wall structure will consist of a wide concrete base with walls to the required study elevations and will be supported by either 24 or 30-inch diameter pipe piles of the length required to develop the appropriate pile capacity. The use of 30-inch piles is

currently limited to the designs for protection to El. 45.0. A detailed load analysis, considering both vertical and horizontal loads, was performed on a typical 60-foot long monolith for each of the three study levels of protection. Horizontal loadings considered included both water to the top of the wall and unbalanced soil loads applied at the base. The wall design considered an impact force of 250-kips, applied at the top of the wall and spread out over a 5-foot width at the point of impact. A more accurate assessment of vessel impact will be required for the design criteria at each structure location. Wave loads were not considered at this time, but the final design will incorporate these loads and, quite possibly, a re-curve type of wall to better distribute and minimize the required wave forces.

A foundation plan was developed for these structures using the Corps program CPGA (X0080) and the calculated vertical and horizontal loads. Given the scope of the study, minor over-stresses were allowed in the foundation piling. Typical pile capacities were assumed to be in excess of 100 tons per pile and were based on foundation capacities presented herein with a factor of safety of 3.0 for this level of study.

L.5.10 EXISTING PUMPING STATIONS EQUIPPED WITH HORIZONTAL PUMPS

L.5.10.1 Closure Gates

L.5.10.1.1 Structures in Lake Pontchartrain and Vicinity Reach

Closure gates at the ends of the reinforced concrete discharge tubes are not provided at all existing horizontal pumps in the Lake Pontchartrain and Vicinity Reach. Structures in this reach are on the protected side of the front line protection and are exposed to a floodside stage of Elevation 15.0.

Installation of structurally framed, remotely operated closure gates at the discharge end of the discharge tubes will eliminate the possibility of backflow of floodwaters through the horizontal pumps with subsequent flooding of the protected side land area. Preliminary designs of the closure gates were performed.

L.5.10.1.2 Structures in the Mississippi River to Larose Reach

Existing horizontal pumps in the Mississippi River to Larose Reach which are along the front line protection will be exposed to hurricane surges from the Gulf of Mexico and were reviewed for surge heights of 30, 35 and 45 feet. Those on the protected side should not be exposed to hurricane surges and were reviewed for surges to 5 feet. The pumping station fronting protection for these existing pumping stations utilize an inverted T-wall structure supported by piling. The structures are located on the floodside of the pump station structures and extend across the width of the stations and tie into the levee 200 feet on each side of the pumping stations. The walls are designed with two ten foot by ten foot openings at each pump for pump discharge. Sluice gates are provided at each opening to provide closure during high water stages. A sheet pile cofferdam is required

for construction of the wall and riprap is provided adjacent to the structure to prevent erosion. A steel access platform is provided on the top of the wall for service and maintenance of the sluice gates.

L.5.10.2 PUMP STATION HARDENING

The pumping stations will be hardened to protect the structures and ancillary equipment and insure their operability during hurricane events. All openings such as windows, exhaust fans, glass doors, louvered roll-up doors, skylights, etc. will be covered or replaced to withstand the anticipated wind forces and impacts from wind borne debris. New steel doors, 16 gauge with sound deadening core, 1.75 inches thick will replace existing doors. New roll-up steel/aluminum louvers or fabricated steel shutters will guard the windows. Roof vents, fan turbines, exhaust louvers and skylights will be covered with missile barrier cages of 3/8 inch steel plate. Access roll-up louvered doors will be replaced. Where required, pump station buildings must be re-sided with heavier steel, 22 gage siding and additional structural support. In some instances building frames must be modified to support the hardening effort.

Roof hardening consist of providing a roof system to withstand design wind speed velocities. Where existing built-up roofs were constructed over concrete planks they were deemed adequate. If the existing built-up roofs were not constructed over concrete they are to be replaced with a metal roof system. All roofs which are being hardened will consist of a standing seam metal roof of material between 20 ga. and 22 ga, continuously supported on wood of thickness not less than 5/8” thick and supported on two-foot centered rafters or continuously supported on metal deck. The wood deck is sealed and waterproofed with underlayment of a non-asphaltic self-adhering rubberized material.

L.5.11 EXISTING PUMPING STATIONS EQUIPPED WITH VERTICAL PUMPS

L.5.11.1 Closure Valves

All existing vertical pump discharges consist of cylindrical large diameter fabricated steel pipes ranging from 48 to 96 inches in diameter. Visual inspection of the existing stations revealed some are equipped with closure valves at the discharge while others are not. The associated mechanical and electrical requirements for this phase of the work will consist of:

Closure valves at pump discharge.

Installation of new closure, electrically operated discharge valves.

Installation of a remotely operated electrical valve actuator for those pumps presently equipped with discharge valves.

All discharge valves will be remotely operated from the pump station control room.

L.5.11.2 Fronting Protection

The pumping station fronting protection for existing pumping stations utilize an inverted T-wall structure supported by piling. The structures are located on the floodside of the pump station structures and extend across the width of the stations and tie into the levee 200 feet from each side of the pumping station. The walls are designed with two ten foot by ten foot openings at each pump for pump discharge. Sluice gates are provided at each opening to provide closure during high water stages. A sheet pile cofferdam is required for construction of the wall and riprap is provided adjacent to the structure to prevent erosion. A steel access platform is provided on the top of the wall for service and maintenance of the sluice gates.

Inverted T-wall structures are used within the levee section where the pumping stations are set back from the levee and discharge pipes cross over the existing levee. Since the pump head cannot be changed, the levee will be excavated and replaced with the T-wall in order that the discharge pipe can remain at the same elevation by going through the wall.

The existing pumping stations in the New Orleans to City Price reach are outside the frontline protection and will be protected to the height of the existing protection system of which they are a part.

L.5.11.3 Pump Station Hardening

The pumping stations will be hardened similar to that discussed for existing pumping stations equipped with horizontal pumps.

L.5.12 NEW PUMPING STATION CONSTRUCTION

The typical pump station is a composite of civil, mechanical and electrical features. The civil portion of the work consists of a reinforced concrete basin structure with a steel framed building superstructure with precast concrete panels. The construction requires the use of a sheetpile cofferdam to accomplish the work. The new pump stations will have the fronting protection built within the pump station structure.

A typical horizontal pump design requires the use of reinforced concrete suction and discharge tubes, to lessen friction losses and increase efficiency of the pumping system. The tubes are an integral part of the concrete basin structure and dictate a more sophisticated construction scheme.

A typical vertical pump design does not require a suction and discharge arrangement as previously described. A typical vertical pump utilizes a built-in suction bell taking the

storm water from a suction basin and discharging through a fabricated steel discharge bell.

The size for a typical pumping unit (pump, reduction gear and drive), whether vertical or horizontal, is based on the required capacity, discharge elevation requirement and the existing suction elevation at the proposed site.

The scope of the electrical work is determined by the pumping arrangement, size and pumping drive mode i.e., whether electric drive or diesel drive. In addition, the electrical work is impacted by the degree of automation, power requirements and availability at site.

The ancillary equipment to be provided is an integral part of any pump station and is the support systems in the station such as vacuum, air, water, oil, fuel and associated piping that enable the pump station to operate.

Climber screens are provided to protect the pumps from damage by straining the floating debris coming into the station. The sizing of this ancillary equipment is commensurate with the pumping arrangement.

L.6 COASTAL RESTORATION FEATURES

Coastal restoration features have been identified as part of the comprehensive plan for hurricane protection. Two rationales (comprehensive plans) were developed by the State of Louisiana for each planning unit. The features identified with each rationale are described in Appendix K to the Main Report. No additional designs were done for these features for the PTR.

These rationales represent two options for comprehensive coastal restoration. More alternatives will be developed and analyzed for the FTR. More extensive design will be done on the identified features as well as any new features for the FTR.

L.7 RELOCATIONS

L.7.1 GENERAL

L.7.1.1 Scope and Purpose

Relocation data was collected and tabulated by U.S. Army Corps of Engineers, New Orleans District, Engineering Division, Relocations Section, for a feasibility level study. Relocations Section utilized the 1990 Louisiana Pipeline and Industrial Atlas, by Design Technics Corporation and information obtained through the Louisiana Department of Natural Resources, to identify oil and gas facilities located in the proposed project area. Also used were information gathered while working on previous feasibility studies and site visit information for other projects in the proposed project area. Relocations Section

considered project design features and their effects on existing facilities to determine project relocation requirements. No contacts were made with facility owners.

L.7.2. DESCRIPTION OF FACILITY RELOCATIONS

L.7.2.1 Method of Pipeline Relocation

Three methods of relocating affected pipelines were investigated:

(1) Placement of pipelines on the surface of the newly constructed levee.
(2) Construction of permanent pipeline bridges supported by pile founded piers.
(3) Directional drilling.

The conditions influencing the selection of a relocation method include the substantial settlement anticipated by the proposed hurricane protection levee alternatives addressed in this report. The hurricane protection alternatives include three to four construction lifts of earthen levees to achieve the required design section and account for substantial settlement, the depth of the soil-cement founded columns. The cost of each method was investigated, analyzed and compared for the best method of relocation for each design alternative chosen. Other factors considered in determining the best method include, initial cost, life cycle cost, maintenance, vandalism, potential impacts by future lifts and disruption of service.

Further discussion on the investigated methods of relocation follows:

Placement of Pipelines on the Levee Surface

This method of relocation is typically used for pipelines affected by main line Mississippi River and Atchafalaya Basin levees. Typically, a pipeline is installed on the levee net design section and follows the contour of the levee until it proceeds underground on the floodside and protected sides of the levee. Additional fill producing the gross grade is placed over the pipeline to provide protection. This method presented problems due to the anticipated settlement of levees on which pipelines would rest. Since the levee provides support for the pipeline, the anticipated levee settlement would result in undesirable stresses on the pipeline. The multiple lifts expected to meet project flood protection elevations also raised concerns about multiple relocations, a prospect which would not be welcomed by the pipeline owners. Although the cost for initial pipeline relocation by this method would probably be the least costly, future costs of subsequent adjustments, modifications, and relocations of the pipeline diminish this advantage.

Permanent Pipeline Bridge

This method of relocation involves installation of affected pipelines on bents supported by pile piers. The advantage of this method is that it diminishes the affects of the expected settlement. However, it is possible settlement of the permanent Pipeline Bridge will also cause stress on the pipeline, requiring additional modification of the bridge.

Another disadvantage is that the pipeline is exposed and vulnerable to vandalism. Additionally, the presence of a permanent structure over the levee impedes levee maintenance and construction of future levee lifts.

Directional Drilling

Directional drilling of pipelines under existing levees is an acceptable method of pipeline relocation provided the pipeline is drilled deep enough to avoid fracturing the levee from the pressure of the drilling fluid. To meet geotechnical criteria, pipelines are directionally drilled at least 100 feet under the base of the levee section and selected berm sections to avoid stresses from levee and berm settlement. The pipelines are offset approximately 15 feet from the existing pipeline alignment to allow workspace. The owners will need to perform excavation of a push/false ditch in order to weld together and test the new pipe segments and to utilize this ditch area to pull the pipe back into the hole. The owners will tie the new pipeline into the existing pipeline at each end and remove the old section of pipe in between. The advantage of this method is that it eliminates additional modification and/or relocation in the future. Additionally, the pipeline cannot be vandalized or damaged by levee maintenance or future construction. The initial cost of directional drilling is greater than that of the two previous relocation methods studied. However, the cost of future modification and/or relocations resulting from the use of the previous two methods would most likely make the final relocation costs comparable to directional drilling.

For the purposes of this report, we anticipate pipelines will be directionally drilled to accommodate the levee construction and to avoid any interference with that construction.

L.7.3 HIGHWAYS AND ROADS

The project alignment impacts several state and parish highways and roads. In some cases, information was obtained from the applicable transportation agencies to determine the physical characteristics of their existing facilities as well as traffic requirements. Traversing the proposed flood protection requires either relocating the highways and roads over the protection, or construction of permanent floodgates at the points of intersection of the protection and highways and roads. These structures remain open except for times of anticipated flooding, at which time they are closed and remain closed until the flood threat subsides. For purposes of this report, we anticipate the relocation of highways and roads atop earthen ramps as the preferred construction method. Relocation of the applicable roads and highways in this manner eliminates the need for floodgate operation during flood conditions, thus allowing egress from affected areas to areas outside of the flood protection for a longer period of time.

L.7.4 CRITERIA FOR RELOCATING FACILITIES

L.7.4.1 Utility Owners

The facility owners will accomplish the design and relocation of their facilities in accordance with Corps facility crossing criteria, the owners' individual criteria and in a manner that eliminates interference with the project.

L.7.4.2 Highways/Roads

Roads within the project area are of various width, surface and design standards. The minimum Design Standards for Rural Highways and Roads of the Louisiana Department of Transportation and Development had been reviewed to apply design standards to the highways/roads required by the proposed project. Required road relocations will be accomplished in a manner to eliminate interference with the project.

L.7.5 PROCEDURE FOR ACCOMPLISHING RELOCATIONS

Owners will prepare plans and specifications for their relocations in a manner that will not interfere with the project. The Government will review the owner's plans and specifications to determine if the owner's proposals are compatible with the project. The owners will construct their relocations, in most cases, prior to our project construction. The local assuring agency for the work will advise the affected owners to effect their relocations.

The relocations work for pipelines to be relocated by directional drilling will be performed prior to construction. All other relocations will be performed concurrent with construction of project features.

L.8 OPERATION, MAINTENANCE, REPAIR, REPLACEMENT AND REHABILITATION (OMRR&R)

OMRR&R estimates were not developed for the PTR. Estimates of OMRR&R for barrier features as well as individual structures will be developed and included in the FTR.

L.9 ACCESS ROADS

The majority of the levees and structures for this project are located in relatively remote areas. Parts of the alignment follow waterways with adjacent roads, but several lengths along the project are located in marsh or open water areas totally inaccessible by vehicle. It is anticipated that access to construction sites will be a significant part of the design and construction effort. Access after construction is necessary to maintain and operate floodgates and locks, and access to all of the alignment is needed to maintain and regularly inspect the levee system.

Access roads will also be required over project levees for property owners on the flood side of the protection system. In some locations, a roadway floodgate will not be practicable. For instance, several businesses operating along the Mississippi River will need roads over the levees to maintain their businesses. The quantity, locations and design of these access roads will be determined during the design phase.

An estimate of the location and miles of access roads required for the project has not been completed for the PTR.

L.10 PROJECT SECURITY

L.10.1 GENERAL

This project poses significant security issues that will be addressed during the design phase. While levees and floodgates for hurricane protection are only fully put into service when a storm threatens Southeast Louisiana, the potential for destruction resulting from even a single failed barrier must be viewed as tempting to terrorists and others seeking to cause harm. Additionally, day-to-day security of hundreds of sites and structures needs to be provided to protect from theft and vandalism. As most barriers will remain unmanned most of the time, the security plan will provide for measures that can be maintained without having personnel on site. We expect only locks to be staffed on a regular basis, and thus security of those sites will require additional attention.

L.10.2 LOCKS

Site security will be accomplished through maintenance of perimeter fences to control land access, and the use of standard locks to control access to building and storage areas to protect contents from theft. Entrance gates within the perimeter fences will normally be kept open to allow access to visitors while the Lockmaster or other day-shift personnel are on duty. Access to lock reservations after hours will be controlled at the discretion of lock operations personnel on duty. Special security measures will be implemented as ordered to mitigate the threats of terrorism or acts of aggression against the government. Effective key control for buildings and other secured areas will be the responsibility of the Lockmaster. Standard internal control practices will be applied for inventory control and the prevention of waste or abuse of government facilities or equipment.

Lock operation personnel will report damage to government property incidental to the lockage of watercraft as required by regulation. Reimbursement for damages will be sought through legal recourse. A video monitoring system will be used to monitor and videotape the passage of all vessels through locks. Video taped images that record incidents of damage to the lock or approach structures will be preserved as evidence to support claims for reimbursement from offending parties.

L.11 COST ESTIMATES

Preliminary costs developed to date are no more than rough order of magnitude and therefore are not included in this report.

L.12 FUTURE STUDIES

L.12.1 GENERAL

The PTR captures the work done to date concerning the effort to provide a plan for "Category 5" protection for all of coastal Louisiana. This design requirement was mandated in the authorizing legislation for this report. Because of the extreme event used to set design elevations for this initial phase of technical study, levee elevations required to block storm surges push the capability of convention levee design. Extraordinary methods such as deep soil mixing coupled with high strength geotextiles are required to satisfy the height and stability requirements for structures located on the soft foundation conditions that exist in south Louisiana. It is intuitively obvious that it may not be practical to employ this degree of protection for the entire state. Be that as it may, the PTR as presented here relied on a single "screening design storm" that satisfied the "Category 5" criteria.

As the project moves forward, engineers are pursuing several independent but coordinated efforts to address the problems now clearly understood at this point of the project. We anticipate conducting the following analyses and studies for use and inclusion in the Final Technical Report (FTR).

L.12.2 STORM MODELING

Following the precise instruction of the legislation, all design work has been based on hydraulic modeling results of a single, massive hurricane that is classified as a "Category 5" storm on the Saffir-Simpson Scale. The result is an enormous, and enormously expensive design for protection against a single storm event of indeterminate recurrence. For critical systems, our counterparts in the Netherlands and elsewhere use a risk-based approach that provides recurrence and a rational basis for establishing flood protection policies. The superior utility of this approach is apparent and warranted. For the FTR, the PDT is not limiting its efforts to a single design storm but is moving to a risk-based approach. This will enable decision makers to look at the flood risks associated with the various levee alignments and make informed decisions about the degree of protection that can be justified.

L.12.3 STORM SURGE STORAGE

Preparation of alternative plans focused on providing an impenetrable barrier to completely halt the storm surge and waves of the design storm. The result is a barrier with elevations not only higher than any ever constructed in south Louisiana, but larger than any existing levee in some locations by a factor of more than two. A cursory

examination of the various alignments under study reveals areas with potentially significant storage capacity on the protected side of the barrier. Where storm surge water can be stored, full height impenetrable barriers would not be necessary. Additionally, allowing water to move from the flood side to the protected side will reduce the volume of water to be repelled by the barrier, which may result in lower barrier elevations. The resulting design would be less expensive to build and be less imposing on the landscape. During the FTR phase engineers are looking at ways to manage the storm surge by incorporating into the design the ability to allow some part of the peak portion of the storm surge to overflow at selected "spillway" locations so that these volumes of water can move into designated storage areas. One promising example would be to allow a wave overtopping and/or freeflow over a portion of the barrier levees proposed for the area between the Chef Menteur and Rigolets Passes. This would allow a calculated volume of water to be stored in Lake Pontchartrain instead of being forced to pile up against a much higher levee. The intent here is to be able to reduce the heights of the adjacent barrier levee to an elevation that is more cost effective. Under this scenario, there may be additional levee works required to protect against the added stage caused by the controlled overflow. However, it can readily be illustrated that the cost per mile of two 25-foot high levees built on the soft soils of Louisiana is less than the cost per mile of one 35-foot high levee. Engineers are seeking to limit the heights of the levee and spillway system to provide for the maximum protection at the least cost. There are a number of locations where this concept is being examined.

L.12.4 STORM SURGE TRANSFER ACROSS THE MISSISSIPPI RIVER

As noted above, the construction of a tall, impenetrable barrier may not be the wisest way to resolve the storm surge problem at all locations. Hurricanes Katrina and Rita were especially harsh on Plaquemines Parish, where a slim strip of developed land hugs the east and west banks of the Mississippi River from Belle Chasse to Venice, Louisiana. Along the river, the cost of hurricane protection is literally doubled: levees must be provided to block the effects of hurricane storm surge attacking from both the river side and back side. Already expensive hurricane protection levees become twice as expensive through this region. Again, if storm surge water could be moved away from the protected side of the levees, stages could be lowered and a more feasible levee design may result. Storage on the protected side of levees in lower Plaquemines is severely limited, however. Engineers are investigating the use of spillways to manage the height of storm surges that stack up against the Mississippi River levees below River Mile 70 above Head of Passes. A review of ADCIRC model runs simulating storm surge affects shows that the continuous Mississippi River levee system, the authorized New Orleans to Venice, LA hurricane protection projects levees and other local levees completely block storm surges approaching from either direction. This not only results in higher water stages rise against levees in Plaquemines, but it ultimately produces a reflected wave that propagates north towards New Orleans and the Mississippi River Gulf Coast. By incorporating spillways at selected critical locations, a controlled volume of storm surge would be able to pass from one side of the Mississippi River delta to the other. By providing relief spillways, the surge would not build up as high and might make it possible to reduce levee heights for many miles of the hurricane protection system and still provide the same

degree of protection. It is hoped that the resulting plan will be less expensive to build and maintain and will thus be made available to protect the greatest number of citizens.

L.12.5 FRESHWATER OVERFLOW VIA SPILLWAYS

In addition to the storm surge reduction benefits that may be gained with the construction of storm surge spillways across the Mississippi River, it may be possible to incorporate a plan to allow overflow from the Mississippi River into the adjacent wetlands. Some minimal height of levee along the Mississippi River would be maintained to keep average flows in the channel, but high stages, particularly spring flood flows, could be allowed to regularly discharge into adjacent wetlands. The spillway concept described above will be studied, so the opportunity to provide much needed sediment and nutrients to sustain the wetlands will be included in that analysis.

L.12.6 HOLLOW CORE LEVEES

Significant expense and design obstacles of the barrier system will be addressed more thoroughly in the FTR. Engineers recognize the high cost for improving local soils is made necessary by the heavy loads imposed on the foundation by a large levee. The source for the large quantity of borrow material required to construct a large levee system is also a concern. This is just one of several innovative solutions were put forward at the Engineering Technical Approaches and Innovations Workshop held earlier this year. One that will be studied is the concept of constructing a hollow core levee. There are several ways in which this type of barrier could be constructed. One way would be to use large diameter precast concrete pipe covered with soil. Another option would be to manufacture precast concrete sections in the desired levee shape. All have the advantage of reducing the quantity of soil needed to build the levee, which simultaneously reduces the weight of the overburden on the foundation soils. There is the potential for significant cost savings and reduced detrimental environmental impacts. Parallel to the cost and construction advantages of a hollow core levee, the resulting hollow interior presents an interesting opportunity to accomplish other project goals. It has been suggested that the hollow core levee could be used as a conduit for freshwater diversion projects. Too, some significant design issues must also be addressed. Will the hollow core levee be strong enough to withstand the lateral force of storm surges? Will settlement present a problem, and how should it be handled? This will be investigated and reported in more detail in the FTR.

L.12.7 FOUNDATION SOIL IMPROVEMENT BY DEEP SOIL MIXING

For the PTR, engineers have already completed a preliminary design analysis of improving existing foundation soils using deep soil mixing technology. However, it must be recognized that this technology has limited real-world use in both hurricane protection systems and in the especially poor soils of southeast Louisiana. A number of general assumptions were made in this first phase of the project that will be further investigated and refined for the FTR. Of most value would be a full scale field test of a typical levee

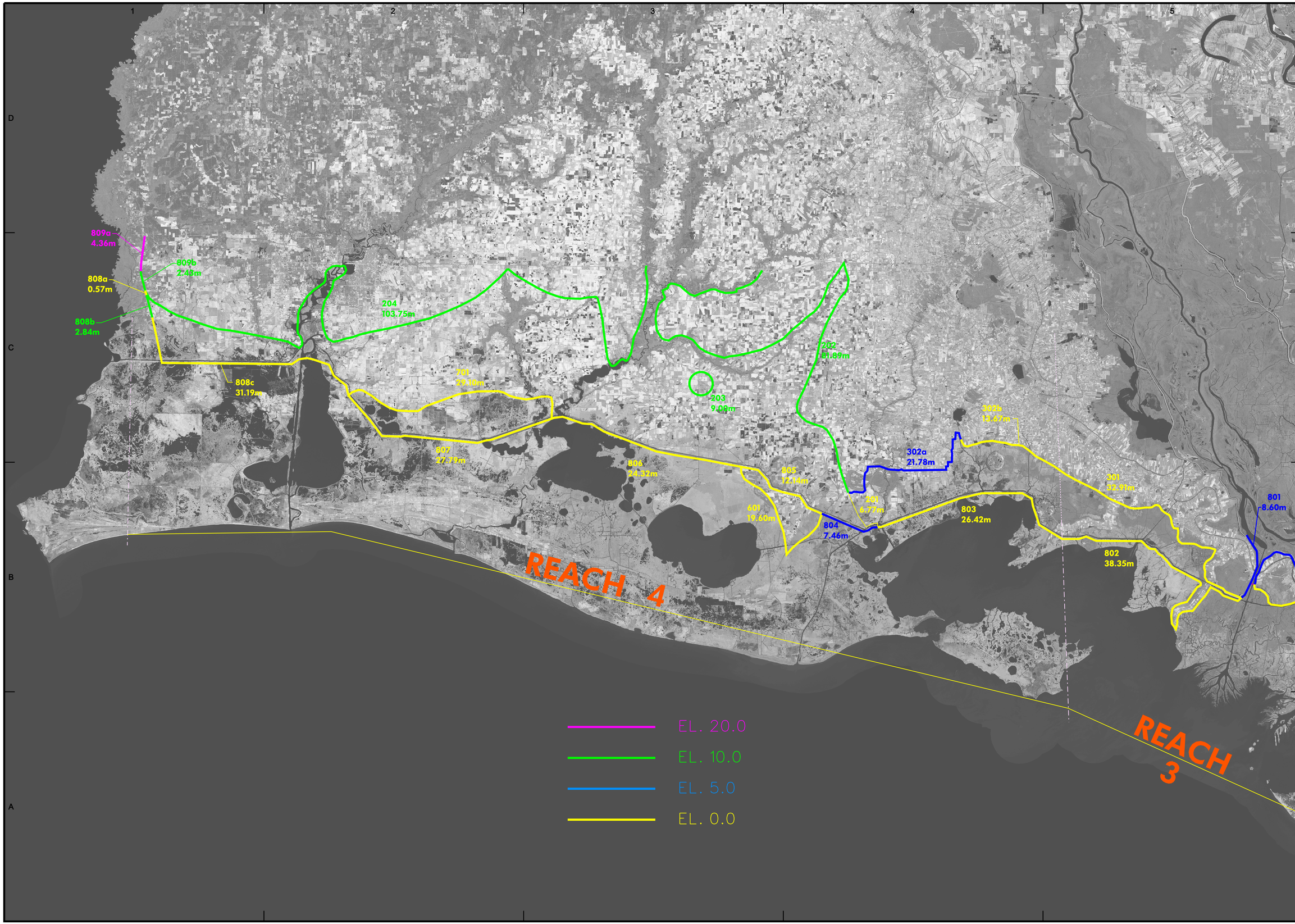
section. Engineers will further examine the design parameters and field techniques to provide the optimal recommendation for deep soil mixing.

L.12.8 STORM SURGE MODELING FOR UNINTENDED CONSEQUENCES

The screening storm used up to the preparation of this PTR does not provide full coverage of coastal storm surge effects. As the construction of barriers and coastal features change the landscape of Louisiana, so do they change the way water approaching the coast in a storm surge event. Water is neither absorbed nor destroyed by a hurricane protection system, but is merely held back or diverted to other locations. During the more detailed modeling for the FTR, engineers will be paying close attention to the corollary effects of the project on Louisiana and its neighbors. Care will be taken to avoid damaging unintended consequences. Additional modeling and analysis of specific cases may be warranted.

L.12.9 STORM SURGE BENEFITS OF ENVIRONMENTAL FEATURES

As noted, a literature review of the quantitative effects of environmental features revealed the dearth of scientific research and computational methods for measuring and predicting the hurricane protection benefits of environmental features. These benefits are widely acknowledged and supported only with very cursory “rules of thumb.” Given the high degree of risk inherent in hurricane protection, certainly a higher level of confidence in the quantification of environmental benefits is required. Engineers have already begun to examine past efforts and are seeking to find appropriate and effective computational methods to correctly characterize and measure the storm surge benefits of marsh, cheniers, barrier islands, and other coastal topographic features of interest.



US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

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CKD BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXX
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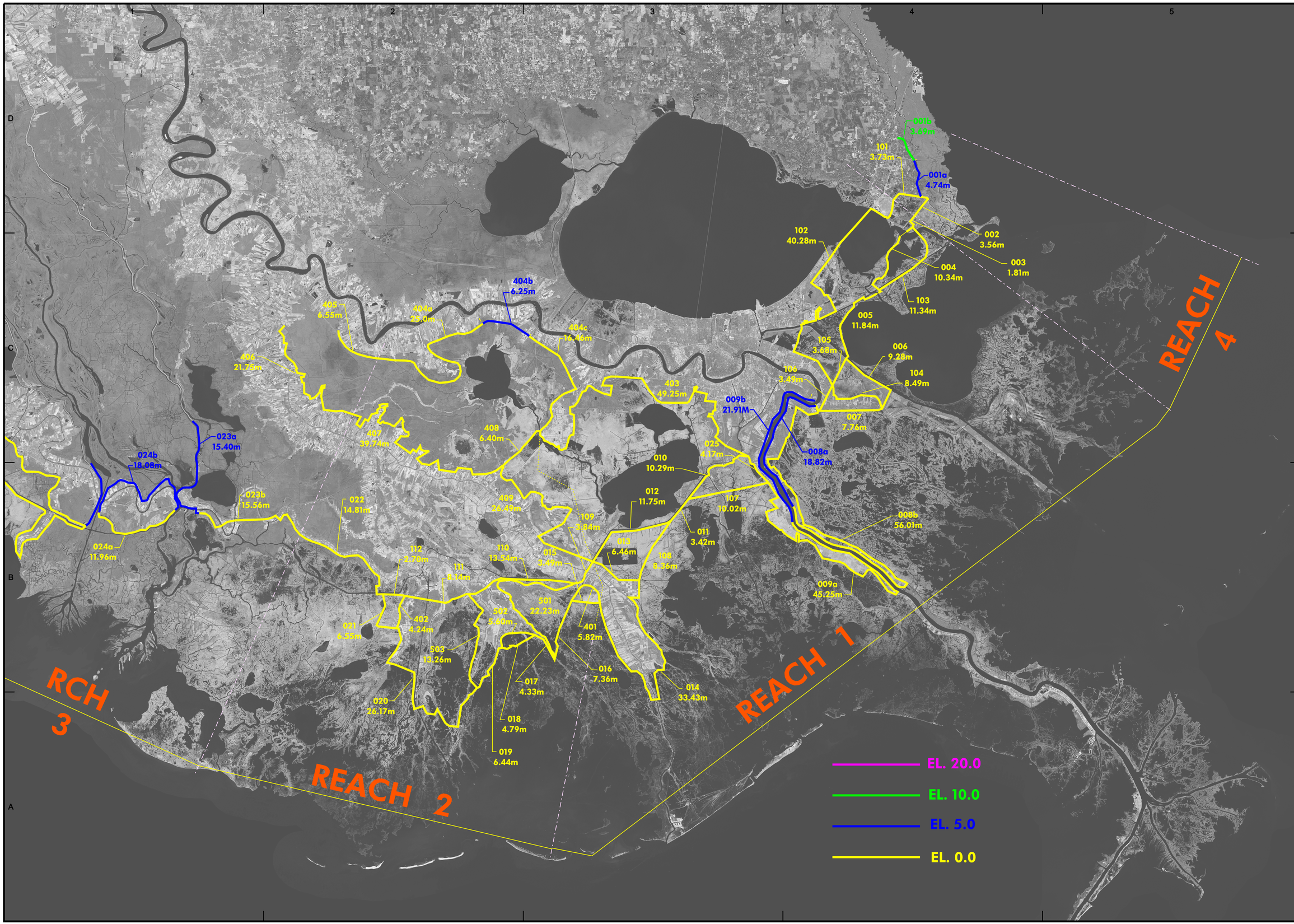
PLOT DATE: MMDDYYYY

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

WEST ALIGNMENTS AND SOIL REACHES

LOUISIANA

SHEET IDENTIFICATION NUMBER
1



US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

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LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

EAST ALIGNMENTS AND SOIL REACHES

LOUISIANA

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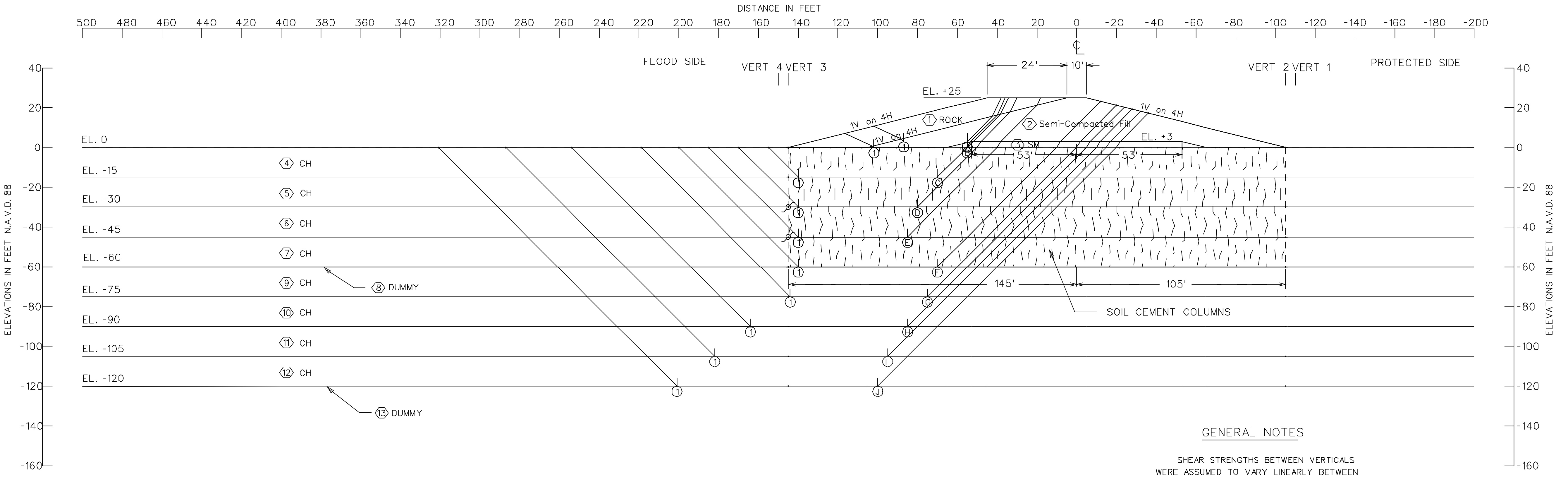


GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
(A) (1)	3.0	14631	12800	16417	28964	5977	43848	22987	1.91
(B) (1)	.0	16919	18800	15936	37302	5111	51655	32191	1.60
(C) (1)	-15.0	79623	140000	6000	86700	11098	225623	75602	2.98
(D) (1)	-30.0	139940	120000	12000	159568	43723	271940	115845	2.35
(E) (1)	-45.0	200903	110000	20250	259164	98846	331153	160318	2.07
(F) (1)	-60.0	261455	35000	33000	396070	176469	329455	219601	1.50
(G) (1)	-75.0	277045	44850	50235	540396	276197	372130	264199	1.41
(H) (1)	-90.0	297948	63200	71985	702978	398802	433133	304176	1.42
(I) (1)	-105.0	323123	82650	98235	884617	543924	504008	340693	1.48
(J) (1)	-120.0	351724	111100	128985	1088284	711545	591809	376739	1.57

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
2	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
3	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
4	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
5	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
6	CH	100	108	108	100	275	2000	2000	275	350	2000	2000	350	0
7	CH	100	108	108	100	425	2000	2000	425	500	2000	2000	500	0
8	DUMMY	100	100	100	100	500	500	500	500	500	500	500	500	0
9	CH	100	100	100	100	575	575	575	575	650	650	650	650	0
10	CH	100	100	100	100	725	725	725	725	800	800	800	800	0
11	CH	100	100	100	100	875	875	875	875	950	950	950	950	0
12	CH	100	100	100	100	1025	1025	1025	1025	1100	1100	1100	1100	0
13	DUMMY	100	100	100	100	1100	1100	1100	1100	1100	1100	1100	1100	0

NOTES

- ⬡ -- STRATUM NUMBER
- ○ -- WEDGE NUMBER
- -- CROSSOVER POINT
- φ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ∇ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT,REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

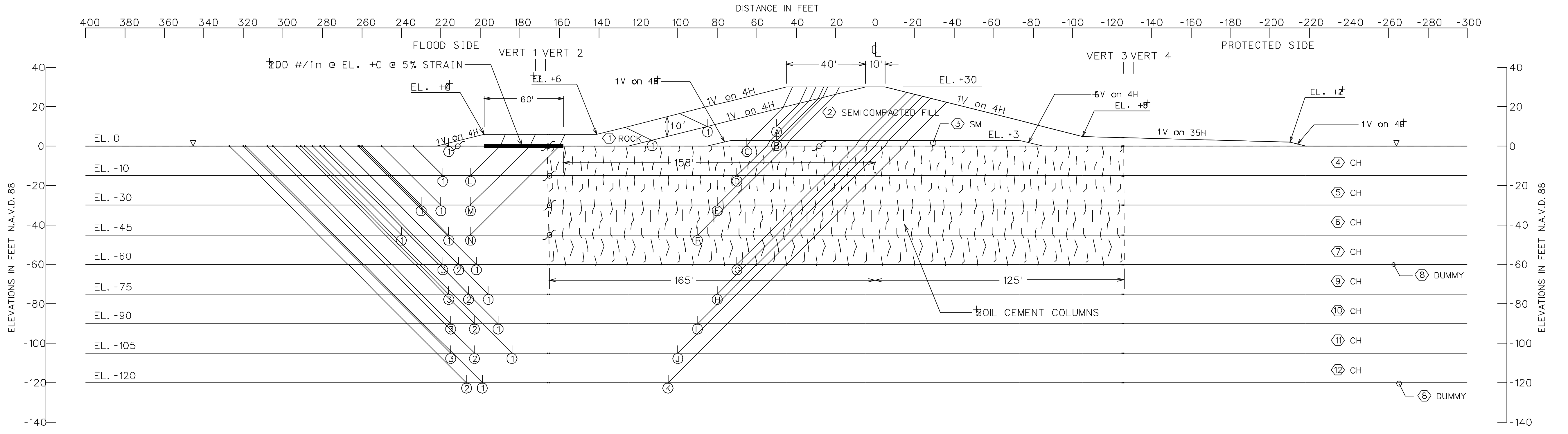
$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 1 - SOIL CEMENT COLUMNS
F/S - CROWN EL. +25
LOUISIANA

SHEET IDENTIFICATION NUMBER



ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
Ⓐ ①	10.0	12919	14000	15461	24859	4294	42380	20565	2.06
Ⓑ ①	3.0	18658	25200	15457	44057	4294	59315	39763	1.49
Ⓒ ①	.0	24483	105500	348	48817	97	130331	48720	2.68
Ⓓ ①	-15.0	84765	201679	6000	111339	10836	292444	100503	2.91
Ⓔ ①	-30.0	145297	181880	12000	192561	43377	339177	149184	2.27
Ⓕ ①	-45.0	206058	168656	20250	294867	99028	394964	195839	2.02
Ⓖ ①	-60.0	265755	66000	33000	443354	182655	364755	260699	1.40
Ⓖ ②	-60.0	265755	70500	33000	443354	178053	369255	265301	1.39
Ⓖ ③	-60.0	265755	74500	33000	443354	176206	373255	267148	1.40
Ⓗ ①	-75.0	282890	71125	50235	593900	287264	404250	306636	1.32
Ⓗ ②	-75.0	282890	77625	50235	593900	280402	410750	313498	1.31
Ⓗ ③	-75.0	282890	84125	50235	593900	276773	417250	317127	1.32
Ⓘ ①	-90.0	303840	80800	71985	763427	413845	456625	349582	1.31
Ⓘ ②	-90.0	303840	90400	71985	763427	404756	466225	358671	1.30
Ⓘ ③	-90.0	303840	100000	71985	763427	399609	475825	363818	1.31
Ⓙ ①	-105.0	329243	79800	98235	952084	564509	507278	387575	1.31
Ⓙ ②	-105.0	329243	97850	98235	952084	549877	525328	402207	1.31
Ⓙ ③	-105.0	329243	109250	98235	952084	544730	536728	407354	1.32
Ⓚ ①	-120.0	357843	103400	128985	1163311	720268	590228	443043	1.33
Ⓚ ②	-120.0	357843	112200	128985	1163311	715253	599028	448058	1.34

ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY	F.S. w/200 lb/in GEOSYNTHETIC AT 5% STRAIN
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING		
Ⓛ ①	-15.0	7858	2800	6000	24129	10836	16658	13293	1.25	1.43
Ⓜ ①	-30.0	13859	5000	12000	68632	43311	30859	25321	1.22	1.31
Ⓝ ①	-45.0	22109	12250	20250	135635	98434	54609	37201	1.47	

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
2	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
3	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
4	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
5	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
6	CH	100	108	108	100	275	2000	2000	275	350	2000	2000	350	0
7	CH	100	108	108	100	425	2000	2000	425	500	2000	2000	500	0
8	CH	100	100	100	100	500	500	500	500	500	500	500	500	0
9	CH	100	100	100	100	575	600	600	575	650	600	600	650	0
10	CH	100	100	100	100	725	725	725	725	800	800	800	800	0
11	CH	100	100	100	100	875	875	875	875	950	950	950	950	0
12	CH	100	100	100	100	1025	1025	1025	1025	1100	1100	1100	1100	0
13	CH	100	100	100	100	1100	1100	1100	1100	1100	1100	1100	1100	0

NOTE: REQUIRED STRENGTH OF GEOSYNTHETIC AT 5% STRAIN IS 200 LB\IN.

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

CONTROLING FAILURE WEDGE Ⓜ-①

T = DRIVING(F.S.) - RESISTING

T = (25321) (1.3) - 30859 = 2058 LB/FT

GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

NOTES

- ◯ -- STRATUM NUMBER
- ◯ -- WEDGE NUMBER
- -- CROSSOVER POINT
- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ∇ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT,REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

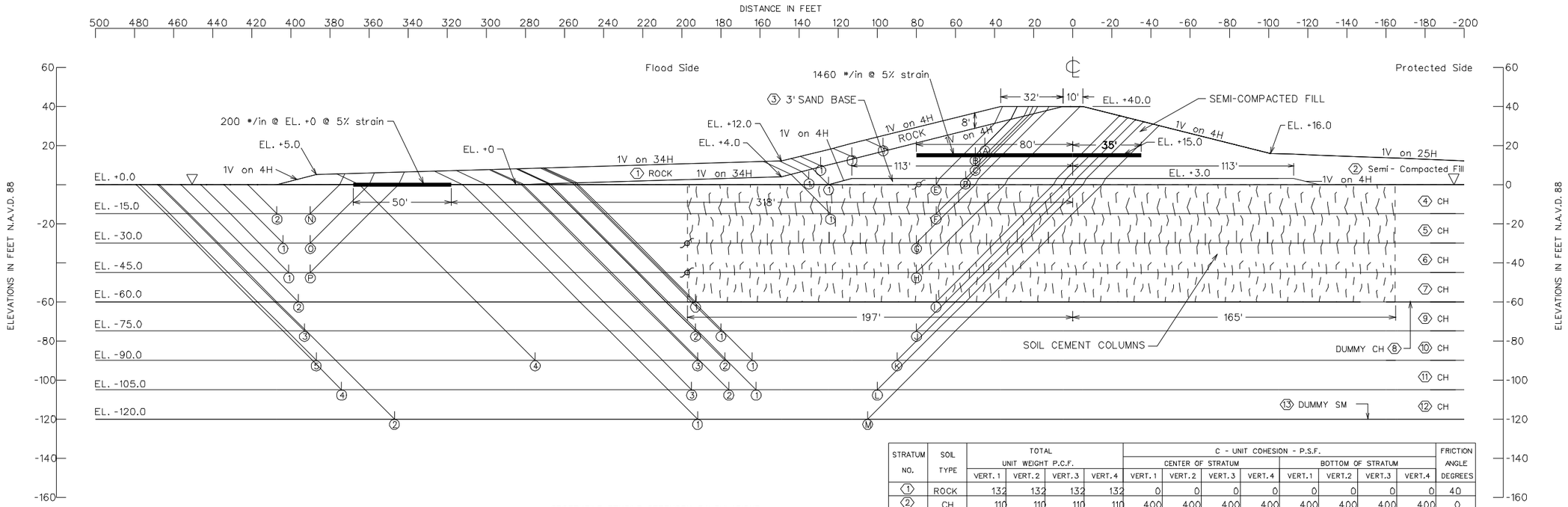
US ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT PRELIMINARY TECHNICAL REPORT TO CONGRESS

REACH 1 - SOIL CEMENT COLUMNS F/S - CROWN EL. +30 LOUISIANA

SHEET IDENTIFICATION NUMBER



STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT. 1	VERT. 2	VERT. 3	VERT. 4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	BOTTOM OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	FRICTION ANGLE DEGREES
①	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
②	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
③	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
④	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
⑤	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
⑥	CH	100	108	108	100	275	2000	2000	275	350	2000	2000	350	0
⑦	CH	100	108	108	100	425	2000	2000	425	500	2000	2000	500	0
⑧	DUMMY	100	100	100	100	500	500	500	500	500	500	500	500	0
⑨	CH	100	100	100	100	575	575	575	575	650	650	650	650	0
⑩	CH	100	100	100	100	725	725	725	725	800	800	800	800	0
⑪	CH	100	100	100	100	875	875	875	875	950	950	950	950	0
⑫	CH	100	100	100	100	1025	1025	1025	1025	1100	1100	1100	1100	0
⑬	DUMMY	100	100	100	100	1100	1100	1100	1100	1100	1100	1100	1100	0

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	F.S. w/1460 lb/in GEOSYNTHETIC AT 5% STRAIN
Ⓐ ①	20.0	12960	20800	7974	23872	610	41734	23262	1.79	
Ⓑ ①	15.0	16960	25200	8614	35681	1235	50774	34446	1.47	
Ⓒ ①	10.0	21290	31600	9255	51447	1948	62145	49499	1.26	1.61
Ⓓ ①	3.0	27101	32000	12797	76194	7447	71898	68747	1.05	1.30
Ⓔ ①	.0	33388	92887	16296	78091	15595	142571	62496	2.28	
Ⓕ ①	-15.0	95236	108000	75488	159538	52303	278724	107235	2.60	
Ⓕ ②	-15.0	95236	295977	6000	159538	10687	397213	148851	2.67	
Ⓖ ①	-30.0	156341	275378	12000	254328	43459	443719	210869	2.10	
Ⓗ ①	-45.0	218426	305378	20250	382164	99028	544054	283136	1.92	
Ⓘ ①	-60.0	274355	61500	48029	536074	256435	383884	279639	1.37	
Ⓘ ②	-60.0	274355	163000	33000	536074	178052	470355	358022	1.31	
Ⓙ ①	-75.0	290760	65000	65234	699453	384999	420994	314454	1.34	
Ⓙ ②	-75.0	290760	73450	64936	699453	373099	429146	326354	1.31	
Ⓙ ③	-75.0	290760	203450	50250	699453	279411	544460	420042	1.30	
Ⓚ ①	-90.0	311710	59200	87007	882062	537569	457917	344493	1.33	
Ⓚ ②	-90.0	311710	70400	86687	882062	524713	468797	357349	1.31	
Ⓚ ③	-90.0	311710	81600	86366	882062	511878	479676	370184	1.30	
Ⓚ ④	-90.0	311710	148000	79120	882062	483545	538830	398517	1.35	
Ⓚ ⑤	-90.0	311710	237600	72000	882062	405397	621310	476665	1.30	
Ⓛ ①	-105.0	337113	58900	112959	1083839	699927	508972	383912	1.33	

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

EL. +15.0 CONTROLLING FAILURE WEDGE Ⓓ - ①

T = DRIVING(F.S.) - RESISTING
T = (68747)(1.3) - 71898 = 17475 LB/FT

EL. +0.0 CONTROLLING FAILURE WEDGE Ⓒ - ①

T = DRIVING(F.S.) - RESISTING
T = (22709)(1.3) - 27957 = 1565 LB/FT

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	F.S. w/200 lb/in GEOSYNTHETIC AT 5% STRAIN
Ⓛ ②	-105.0	337113	72200	112639	1083839	686410	521952	397429	1.31	
Ⓛ ③	-105.0	337113	90250	111013	1083839	671010	538376	412829	1.30	
Ⓛ ④	-105.0	337113	260300	98250	1083839	559424	695663	524415	1.33	
Ⓜ ①	-120.0	365713	95700	140598	1308226	854056	602011	454170	1.33	
Ⓜ ②	-120.0	365713	266200	129000	1308226	747636	760913	560590	1.36	
Ⓝ ②	-15.0	7499	2400	6000	22630	11100	15899	11530	1.38	
Ⓞ ①	-30.0	13757	2200	12000	66614	43905	27957	22709	1.23	1.33
Ⓟ ①	-45.0	22285	2800	20250	133998	99771	45335	34227	1.32	

NOTES

- ⬡ -- STRATUM NUMBER
- ○ -- WEDGE NUMBER
- ⊗ -- CROSSOVER POINT
- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ▽ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS.

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

US ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

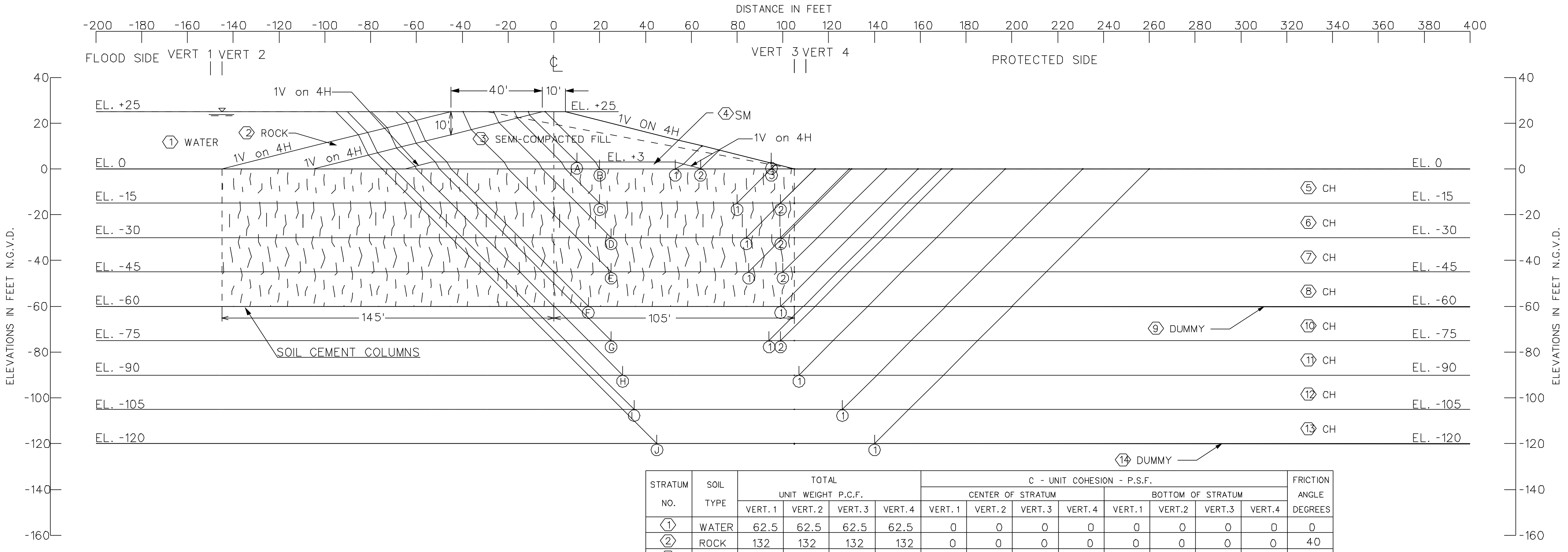
U.S. ARMY ENGINEER DISTRICT NEW ORLEANS, LOUISIANA
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXXXXXXXXXXXXXXX
DWN BY: XXXXXXXXXXXXXXXX
CKD BY: XXXXXXXXXXXXXXXX
SUBMITTED BY: XXXXXXXXXXXXXXXX
SCALE: AS SHOWN
DATE: MM/DD/YYYY
SUBMITTED: XXXXXXXXXXXXXXXX
REC'D BY: XXXXXXXXXXXXXXXX
DATE: MM/DD/YYYY
DATE: MM/DD/YYYY
SOLICITATION NO.: W912P8-YY-B-XXXX
CONTRACT NO.: W912P8-YY-C-XXXX
LOCATION: NEW ORLEANS, LOUISIANA
DESIGN FIRM: XXXXXXXXXXXXXXXX
FILE NAME: XXXXXXXXXXXXXXXX
SIZE: A1
PLOT SCALE: 1:1
PLOT DATE: MM/DD/YYYY

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

REACH 1 - SOIL CEMENT COLUMNS
F/S - CROWN EL.+48
LOUISIANA

SHEET IDENTIFICATION NUMBER



STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	WATER	62.5	62.5	62.5	62.5	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
3	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
6	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
7	CH	100	108	108	100	275	2000	2000	275	350	2000	2000	350	0
8	CH	100	108	108	100	425	2000	2000	425	500	2000	2000	500	0
9	DUMMY	100	100	100	100	500	500	500	500	500	500	500	500	0
10	CH	100	108	108	100	575	575	575	575	650	650	650	650	0
11	CH	100	100	100	100	725	725	725	725	800	800	800	800	0
12	CH	100	100	100	100	875	875	875	875	950	950	950	950	0
13	CH	100	100	100	100	1025	1025	1025	1025	1100	1100	1100	1100	0
14	DUMMY	100	100	100	100	1100	1100	1100	1100	1100	1100	1100	1100	0

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
A	1	3.0	16582	33963	-320	26381	11	50225	26370	1.90
B	1	.0	20213	21353	9582	30836	7086	51148	23750	2.15
B	2	.0	20213	26150	6611	30836	4598	52974	26238	2.02
B	3	.0	20213	33377	1600	30836	275	55190	30561	1.81
C	1	-15.0	78831	119892	61600	85682	19642	260323	66040	3.94
C	2	-15.0	78831	157452	6000	85682	11182	242283	74500	3.25
D	1	-30.0	137396	117828	66000	162343	53199	321224	109144	2.94
D	2	-30.0	137396	147452	12000	162343	43806	296848	118537	2.50
E	1	-45.0	195669	119825	72000	267045	109559	387494	157486	2.46
E	2	-45.0	195669	149467	20250	267045	98778	365386	168267	2.17
F	1	-60.0	252528	42000	33000	400221	176553	327528	223668	1.46
G	1	-75.0	268845	44850	50235	545888	286113	363930	259775	1.40
G	2	-75.0	268845	48100	50235	545888	276675	367180	269213	1.36
H	1	-90.0	288702	61600	71985	713639	398801	422287	314838	1.34
I	1	-105.0	313273	86450	98235	902135	543922	497958	358213	1.39
J	1	-120.0	343223	104500	128985	1108449	711542	576708	396907	1.45

NOTES

⬡ -- STRATUM NUMBER
○ -- WEDGE NUMBER
⌀ -- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT,REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

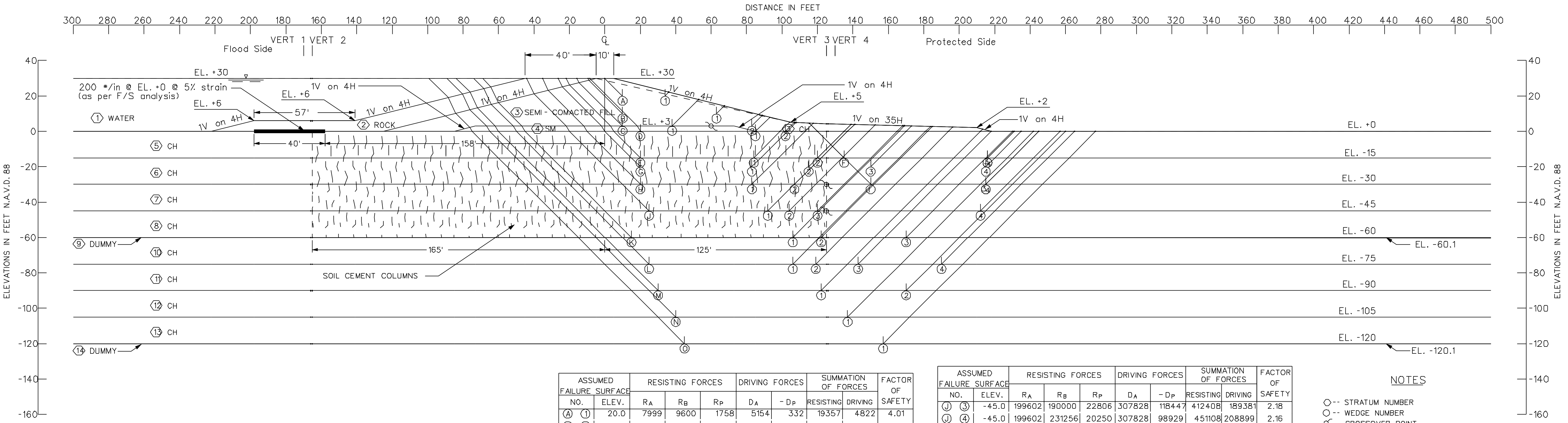
US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

REACH 1 - SOIL CEMENT COLUMNS
P/S - CROWN EL. +25, Water at EL. +25
LOUISIANA

SHEET IDENTIFICATION NUMBER



STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT. 1	VERT. 2	VERT. 3	VERT. 4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	BOTTOM OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	FRICTION ANGLE DEGREES
⟨1⟩	WATER	62	62	62	62	0	0	0	0	0	0	0	0	0
⟨2⟩	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
⟨3⟩	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
⟨4⟩	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
⟨5⟩	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
⟨6⟩	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
⟨7⟩	CH	100	108	108	100	275	2000	2000	275	350	2000	2000	350	0
⟨8⟩	CH	100	108	108	100	425	2000	2000	425	500	2000	2000	500	0
⟨9⟩	DUMMY	100	100	100	100	500	500	500	500	500	500	500	500	0
⟨10⟩	CH	100	100	100	100	575	575	575	575	650	650	650	650	0
⟨11⟩	CH	100	100	100	100	725	725	725	725	800	800	800	800	0
⟨12⟩	CH	100	100	100	100	875	875	875	875	950	950	950	950	0
⟨13⟩	CH	100	100	100	100	1025	1025	1025	1025	1100	1100	1100	1100	0
⟨14⟩	DUMMY	100	100	100	100	1100	1100	1100	1100	1100	1100	1100	1100	0

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
(A) (1)	20.0	7999	9600	1758	5154	332	19357	4822	4.01
(B) (1)	10.0	15227	21200	3519	21708	1330	39946	20378	1.96
(C) (1)	3.0	19837	11200	11999	40065	15465	43036	24600	1.75
(C) (2)	3.0	19837	27972	4799	40065	2474	52608	37591	1.40
(D) (1)	.0	23669	36325	6399	45991	4398	66393	41593	1.60
(D) (2)	.0	23669	40155	3953	45991	1492	67777	44499	1.52
(E) (1)	-15.0	81988	128000	64160	109414	27826	274148	81588	3.36
(E) (2)	-15.0	81988	200000	9224	109414	18780	291212	90634	3.21
(E) (3)	-15.0	81988	229079	6000	109414	10742	317067	98672	3.21
(F) (4)	-15.0	9751	16200	6000	19035	10742	31951	8293	3.85
(G) (1)	-20.0	101277	126000	83930	136011	40617	311207	95394	3.26
(G) (2)	-20.0	101277	190000	29224	136011	30773	320501	105238	3.05
(G) (3)	-20.0	101277	215879	10444	136011	27134	327600	108877	3.01
(G) (4)	-20.0	101277	228879	8000	136011	19186	338156	116825	2.89
(H) (1)	-30.0	140249	126000	123696	197673	72859	389945	124814	3.12
(H) (2)	-30.0	140249	174000	69179	197673	62893	383428	134780	2.84
(H) (3)	-30.0	140249	228880	12000	197673	43435	381129	154238	2.47
(I) (4)	-30.0	15751	13000	12000	58143	43435	40751	14708	2.77
(J) (1)	-45.0	199602	134000	129179	307828	134207	462781	173621	2.67
(J) (2)	-45.0	199602	158000	74912	307828	126498	432514	181330	2.39

ASSUMED FAILURE SURFACE NO.	ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
(J) (3)	-45.0	199602	190000	22806	307828	118447	412408	189381	2.18
(J) (4)	-45.0	199602	231256	20250	307828	98929	451108	208899	2.16
(K) (1)	-60.0	256187	45500	82783	447939	211174	384470	236765	1.62
(K) (2)	-60.0	256187	53500	35177	447939	200608	344864	247331	1.39
(K) (3)	-60.0	256187	77500	33000	447939	188247	366687	259692	1.41
(L) (1)	-75.0	272499	52650	52435	601601	317077	377584	284524	1.33
(L) (2)	-75.0	272499	61100	52145	601601	305599	385744	296002	1.30
(L) (3)	-75.0	272499	76700	50235	601601	298852	399434	302749	1.32
(L) (4)	-75.0	272499	107250	50235	601601	282084	429984	319517	1.35
(M) (1)	-90.0	292403	73600	72945	777381	430805	438948	346576	1.27
(M) (2)	-90.0	292403	112000	71985	777381	410990	476388	366391	1.30
(N) (1)	-105.0	317729	92150	98235	971201	569236	508114	401965	1.26
(O) (1)	-120.0	346593	123200	128985	1188371	728493	598778	459878	1.30

GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

NOTES

⬡ -- STRATUM NUMBER
◯ -- WEDGE NUMBER
⌀ -- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
▽ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

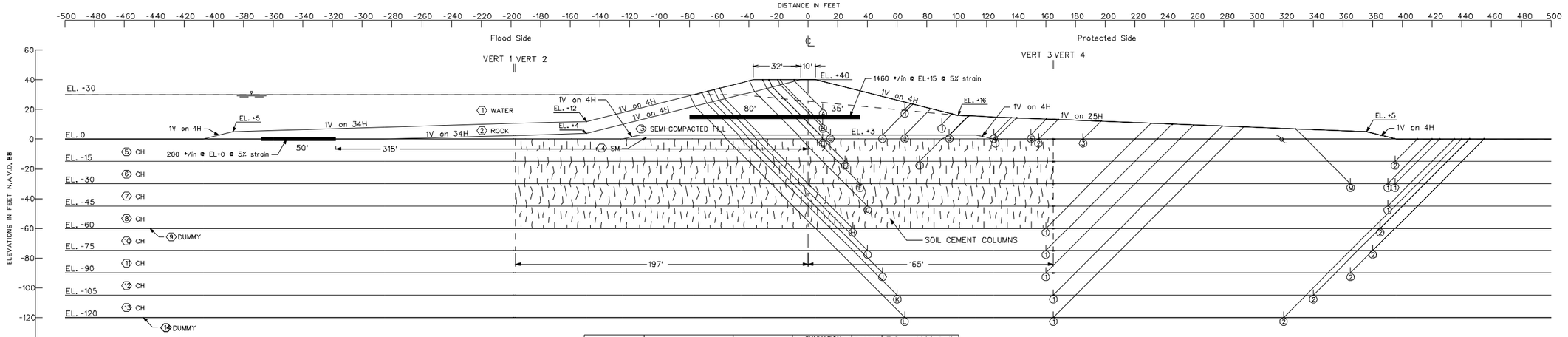
MARK | DESCRIPTION | DATE | APPR.

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXX | DWN BY: XXX | CKD BY: XXX | SUBMITTED BY: XXXXXXXXXXXXXXXX | DATE: MM/DD/YYYY | SOLICITATION NO.: W912P8-Ø-B-XXXX | CONTRACT NO.: W912P8-ØØ-C-XXXX | FILE NAME: XXXXXXXXXXXX-XXX-XXXXXX.DGN | FILE NUMBER: H-NN-NNNNN | SIZE: ANSI X | PLOT SCALE: 1"=500.0' | PLOT DATE: MM/DD/YYYY

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 1 - SOIL CEMENT COLUMNS
P/S - CROWN EL. +30 WATER EL. +30
LOUISIANA

SHEET IDENTIFICATION NUMBER



GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	WATER	62.5	62.5	62.5	62.5	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
3	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
6	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
7	CH	100	108	108	100	275	2000	2000	275	350	2000	2000	350	0
8	CH	100	108	108	100	425	2000	2000	425	500	2000	2000	500	0
9	DUMMY	100	100	100	100	500	500	500	500	500	500	500	500	0
10	CH	100	100	100	100	575	575	575	575	650	650	650	650	0
11	CH	100	100	100	100	725	725	725	725	800	800	800	800	0
12	CH	100	100	100	100	875	875	875	875	950	950	950	950	0
13	CH	100	100	100	100	1025	1025	1025	1025	1100	1100	1100	1100	0
14	DUMMY	100	100	100	100	1100	1100	1100	1100	1100	1100	1100	1100	0

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	F.S. w/1460 lb/in GEOSYNTHETIC AT 5% STRAIN
Ⓐ	1	20.0	15252	22000	3199	21708	1099	40451	20609	1.96	
Ⓑ	1	10.0	22065	32000	5600	49647	3368	59665	46279	1.29	1.67
Ⓒ	1	3.0	27477	14000	16479	74550	29171	57956	45379	1.28	1.66
Ⓒ	2	3.0	27477	20000	14080	74550	21294	61557	53256	1.16	1.48
Ⓒ	3	3.0	27477	32000	10185	74550	9686	69662	64864	1.07	1.34
Ⓒ	4	3.0	27477	43323	9264	74550	7669	80064	66881	1.20	1.46
Ⓒ	5	3.0	27477	53323	8496	74550	6451	89296	68099	1.31	
Ⓓ	1	.0	32331	46400	11538	88800	11898	90269	76902	1.17	1.40
Ⓓ	2	.0	32331	57809	10646	88800	10129	100786	78671	1.28	1.50
Ⓓ	3	.0	32331	68701	9723	88800	8448	110755	80352	1.38	
Ⓔ	1	-15.0	92331	100000	75503	162328	61899	267834	100429	2.67	
Ⓔ	2	-15.0	92331	326879	6000	162328	10701	425210	151627	2.80	
Ⓕ	1	-30.0	152241	306880	12000	259160	43325	471121	215835	2.18	
Ⓖ	1	-45.0	211757	329556	20250	383253	98929	561563	284324	1.98	
Ⓗ	1	-60.0	267578	65000	41642	536235	264804	374220	271431	1.38	
Ⓗ	2	-60.0	267578	177500	33000	536235	177720	478078	358515	1.33	
Ⓘ	1	-75.0	283850	78000	58416	699943	382275	420266	317668	1.32	
Ⓘ	2	-75.0	283850	221000	50235	699943	279698	555085	420245	1.32	
Ⓙ	1	-90.0	304333	88000	79705	882692	521294	472038	361398	1.31	
Ⓙ	2	-90.0	304333	252000	71985	882692	410612	628318	472080	1.33	

NOTE: REQUIRED STRENGTH OF GEOSYNTHETIC AT 5% STRAIN IS 1460 LB\IN.

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

CONTROLLING FAILURE WEDGE Ⓒ - ③

T = DRIVING(F.S.) - RESISTING

T = (64864X1.3) - 69662 = 1466ILB/FT

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓚ	1	-105.0	329202	99750	105340	1084592	679349	534292	405243	1.32
Ⓚ	2	-105.0	329202	266000	98235	1084592	572061	693437	512531	1.35
Ⓛ	1	-120.0	357222	110000	135629	1308951	861146	602851	447805	1.35
Ⓛ	2	-120.0	357222	280500	128985	1308951	754723	766707	554228	1.38
Ⓜ	1	-30.0	17531	5000	12000	65757	43806	34531	21951	1.57

NOTES

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$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

US ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT

MARK	DATE	DESCRIPTION

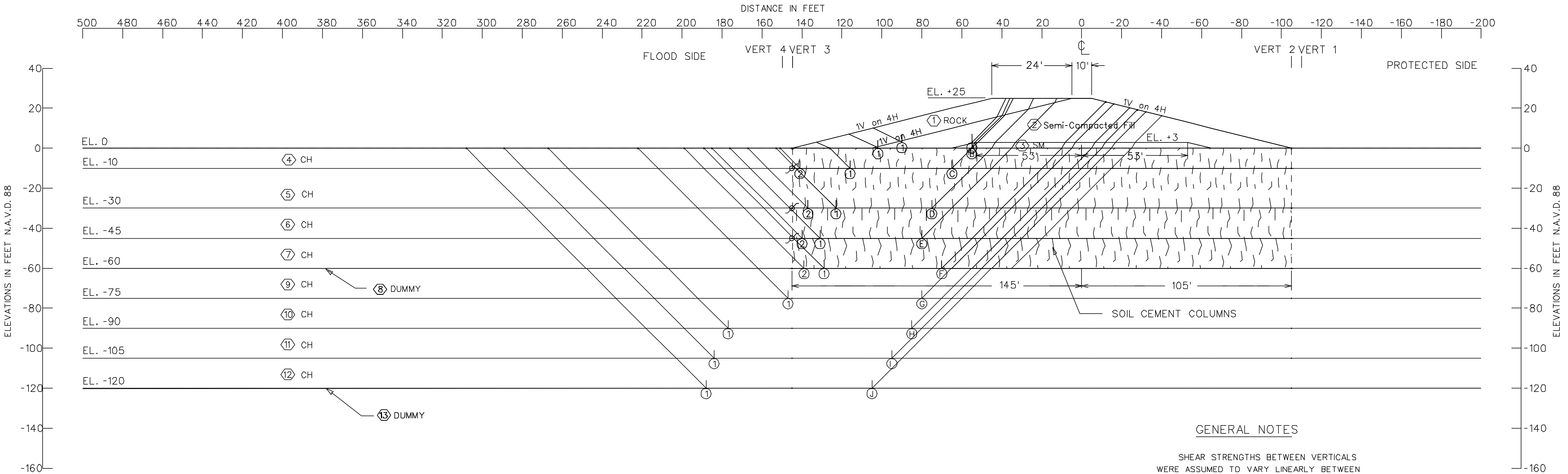
ISSUED BY: U.S. ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT MISSISSIPPI VALLEY DIVISION

DESIGN BY: DATE: 10-24-2000 SCALE: AS NOTED
DRAWN BY: DATE: XX-XX-XXXX
CHECKED BY: DATE: XX-XX-XXXX
APPROVED BY: DATE: XX-XX-XXXX
NO. 00-XX-XXXX
SHEET NO. XX OF XX
FILE NAME: XXXXXXXX.DGN

PRELIMINARY TECHNICAL REPORT TO CHANGES

REACH 1 - SOIL CEMENT COLUMNS
P/S - DROWN EL.+40, WATER EL.+30
LOUISIANA

SHEET IDENTIFICATION NUMBER



GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

ASSUMED FAILURE SURFACE			RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.		ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
(A)	(1)	3.0	14631	14000	15937	28964	5111	44568	23853	1.87
(B)	(1)	.0	16919	18800	15936	37302	5111	51655	32191	1.60
(C)	(1)	-10.0	59623	102000	43486	68195	14285	205109	53910	3.80
(C)	(2)	-10.0	59623	152000	4001	68195	5263	215624	62932	3.43
(D)	(1)	-30.0	140415	96000	84000	163944	56178	320415	107766	2.97
(D)	(2)	-30.0	140415	124000	16000	163944	46052	280415	117892	2.38
(E)	(1)	-45.0	201678	102000	76000	263689	108977	379678	154712	2.45
(E)	(2)	-45.0	201678	120000	27250	263689	101658	348928	162031	2.15
(F)	(1)	-60.0	261455	35400	87250	396070	190514	384105	205556	1.87
(F)	(2)	-60.0	261455	41400	43000	396070	180588	345855	215482	1.61
(G)	(1)	-75.0	280845	50250	63235	538631	281243	394330	257388	1.53
(H)	(1)	-90.0	303948	82800	87985	702978	404989	474733	297989	1.59
(I)	(1)	-105.0	332123	93450	117235	884616	551233	542808	333383	1.63
(J)	(1)	-120.0	364798	99600	150985	1085454	719979	615383	365475	1.68

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
2	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
3	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
4	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
5	CH	100	108	108	100	300	2000	2000	300	300	2000	2000	300	0
6	CH	100	108	108	100	375	2000	2000	375	450	2000	2000	450	0
7	CH	100	108	108	100	525	2000	2000	525	600	2000	2000	600	0
8	DUMMY	100	100	100	100	600	600	600	600	600	600	600	600	0
9	CH	100	100	100	100	675	675	675	675	750	750	750	750	0
10	CH	100	100	100	100	825	825	825	825	900	900	900	900	0
11	CH	100	100	100	100	975	975	975	975	1050	1050	1050	1050	0
12	CH	100	100	100	100	1125	1125	1125	1125	1200	1200	1200	1200	0
13	DUMMY	100	100	100	100	1200	1200	1200	1200	1200	1200	1200	1200	0

NOTES

- ⬡ -- STRATUM NUMBER
- ◯ -- WEDGE NUMBER
- -- CROSSOVER POINT
- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ∇ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY $= \frac{R_A + R_B + R_P}{D_A - D_P}$

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

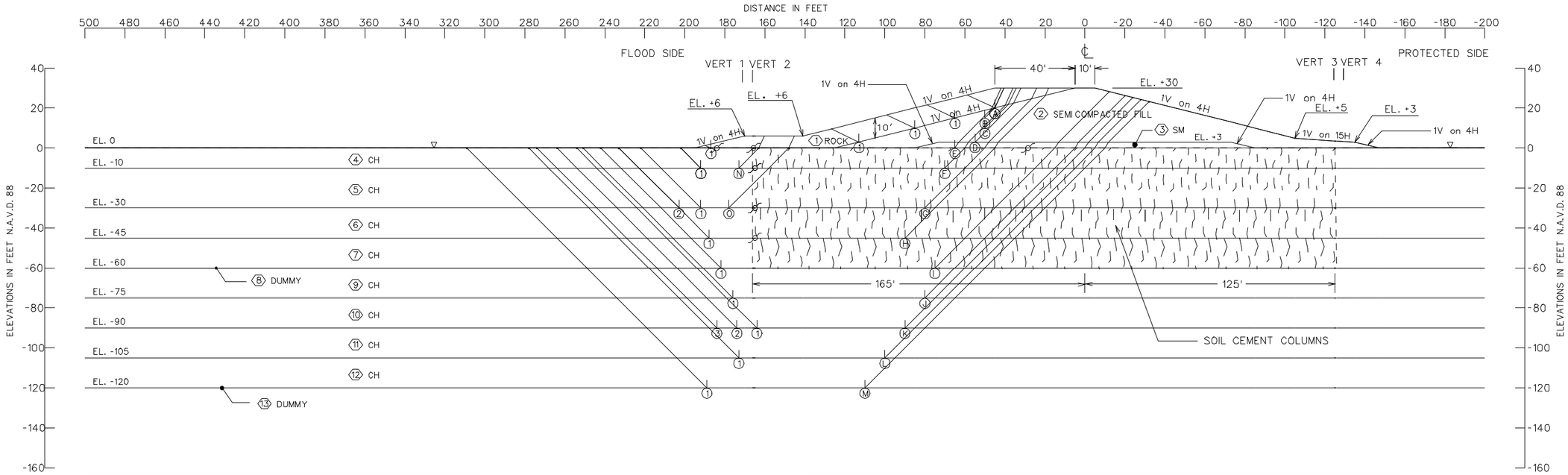
MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXXXX
FILE NAME: XXXXXXXXXXXXX-XXX-XXXXXX.DGN
SIZE: ANSI X
PLOT SCALE: 1"=5000'
DATE: MM/DD/YYYY
SOLICITATION NO.: V912P8-00-B-XXXX
CONTRACT NO.: V912P8-00-C-XXXX
FILE NUMBER: H-NN-NNNNN
PLOT DATE: MM/DD/YYYY

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 2 - SOIL CEMENT COLUMNS
F/S - CROWN EL. +25
LOUISIANA

SHEET IDENTIFICATION NUMBER



ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
Ⓐ ①	20.0	5163	0	15460	6597	4294	20623	2303	8.96
Ⓑ ①	15.0	9163	6000	15461	14227	4294	30624	9933	3.08
Ⓒ ①	10.0	12919	14000	15461	24859	4294	42380	20565	2.06
Ⓓ ①	3.0	18501	23200	15457	43373	4294	57158	39079	1.46
Ⓔ ①	.0	24483	99814	578	48817	161	124875	48656	2.57
Ⓕ ①	-10.0	64520	196280	4000	86864	5142	264800	81722	3.24
Ⓖ ①	-30.0	145297	178931	16000	192562	45141	340228	147421	2.31
Ⓗ ①	-45.0	206058	161107	27250	294868	102023	394415	192845	2.05
Ⓘ ①	-60.0	266610	64200	43000	441936	182671	373810	259265	1.44
Ⓙ ①	-75.0	285145	72000	63235	593900	286959	420380	306941	1.37
Ⓚ ①	-90.0	309095	66600	87985	763427	419637	463680	343790	1.35
Ⓚ ②	-90.0	309095	75600	87985	763427	411972	472680	351455	1.34
Ⓚ ③	-90.0	309095	84600	87985	763427	406905	481680	356522	1.35
Ⓛ ①	-105.0	337498	76650	117235	952083	558900	531383	393183	1.35
Ⓜ ①	-120.0	370173	94800	150985	1159799	720551	615958	439248	1.40
Ⓝ ①	-10.0	5859	3800	4000	15152	5142	13659	10010	1.36
Ⓞ ②	-30.0	53860	7500	16000	70518	44999	77360	25519	3.03

GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

NOTES

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$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
①	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
②	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
③	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
④	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
⑤	CH	100	108	108	100	300	2000	2000	300	300	2000	2000	300	0
⑥	CH	100	108	108	100	375	2000	2000	375	450	2000	2000	450	0
⑦	CH	100	108	108	100	525	2000	2000	525	600	2000	2000	600	0
⑧	DUMMY	100	100	100	100	600	600	600	600	600	600	600	600	0
⑨	CH	100	100	100	100	675	675	675	675	750	750	750	750	0
⑩	CH	100	100	100	100	825	825	825	825	900	900	900	900	0
⑪	CH	100	100	100	100	975	975	975	975	1050	1050	1050	1050	0
⑫	CH	100	100	100	100	1125	1125	1125	1125	1200	1200	1200	1200	0
⑬	DUMMY	100	100	100	100	1200	1200	1200	1200	1200	1200	1200	1200	0

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

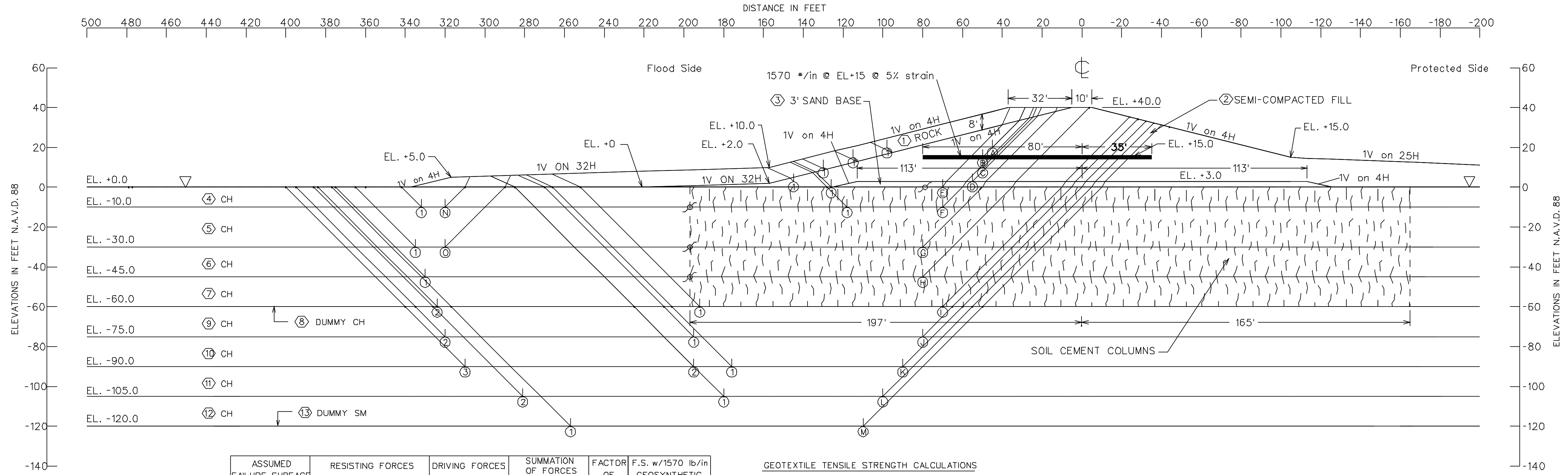
MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

ISSUED BY:
XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY:
XXXXXXXXXXXXXXXXXXXX
FILE NAME:
XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
PROJ. NO.
MM/DD/YYYY
SCALE: AS NOTED
SOLICITATION NO.
XXXXXXXXXX
CONTRACT NO.
XXXXXXXXXX
FILE NUMBER
N/A
ACT DATE: MM/DD/YYYY
PAGE X FOUND

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 2 - SOIL CEMENT COLUMNS
F/S - CROWN EL. +30
LOUISIANA

SHEET IDENTIFICATION NUMBER



ASSUMED FAILURE SURFACE			RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY	F.S. w/1570 lb/in GEOSYNTHETIC AT 5% STRAIN
NO.		ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING		
Ⓐ	①	20.0	12960	21200	7814	23872	468	41974	23404	1.79	
Ⓑ	①	15.0	16960	26000	8294	35681	911	51254	34770	1.47	
Ⓒ	①	10.0	21290	32000	9095	51447	1761	62385	49686	1.26	1.63
Ⓓ	①	3.0	27101	36000	11196	76194	4620	74297	71574	1.04	1.30
Ⓔ	①	.0	33388	94123	16135	78091	15166	143646	62925	2.28	
Ⓕ	①	-10.0	74418	96000	55815	129930	41906	226233	88024	2.57	
Ⓖ	①	-30.0	156341	275520	16000	254329	45065	447861	209264	2.14	
Ⓗ	①	-45.0	218426	293959	27250	382164	102055	539635	280109	1.93	
Ⓘ	①	-60.0	274355	73200	54002	536074	245872	401557	290202	1.38	
Ⓘ	②	-60.0	274355	152400	43000	536074	182782	469755	353292	1.33	
Ⓙ	①	-75.0	293760	86250	72561	699454	359233	452571	340221	1.33	
Ⓙ	②	-75.0	293760	180000	63250	699454	286009	537010	413445	1.30	
Ⓚ	①	-90.0	317710	77400	97674	882061	509214	492784	372847	1.32	
Ⓚ	②	-90.0	317710	94500	96009	882061	494960	508219	387101	1.31	
Ⓚ	③	-90.0	317710	198000	88000	882061	416304	603710	465757	1.30	
Ⓛ	①	-105.0	346113	84000	125259	1083839	665972	555372	417867	1.33	
Ⓛ	②	-105.0	346113	190050	117250	1083839	584257	653413	499582	1.31	
Ⓜ	①	-120.0	378788	176400	151000	1304671	773592	706188	531079	1.33	
Ⓝ	①	-10.0	5426	2400	4000	13372	5412	11826	7960	1.49	
Ⓞ	①	-30.0	17789	3000	16000	68432	45807	36789	22625	1.63	

NOTES

- ○ -- STRATUM NUMBER
- ○ -- WEDGE NUMBER
- ∫ -- CROSSOVER POINT
- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ▽ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

EL. +15.0 CONTROLLING FAILURE WEDGE Ⓓ - ①

T = DRIVING(F.S.) - RESISTING

T - (71574)(1.3) - 74297 - 18750 LB/FT

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
①	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
②	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
③	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
④	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
⑤	CH	100	108	108	100	300	2000	2000	300	300	2000	2000	300	0
⑥	CH	100	108	108	100	375	2000	2000	375	450	2000	2000	450	0
⑦	CH	100	108	108	100	525	2000	2000	525	600	2000	2000	600	0
⑧	DUMMY	100	100	100	100	600	600	600	600	600	600	600	600	0
⑨	CH	100	100	100	100	675	675	675	675	750	750	750	750	0
⑩	CH	100	100	100	100	825	825	825	825	900	900	900	900	0
⑪	CH	100	100	100	100	975	975	975	975	1050	1050	1050	1050	0
⑫	CH	100	100	100	100	1125	1125	1125	1125	1200	1200	1200	1200	0
⑬	DUMMY	100	100	100	100	1200	1200	1200	1200	1200	1200	1200	1200	0

GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS.

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

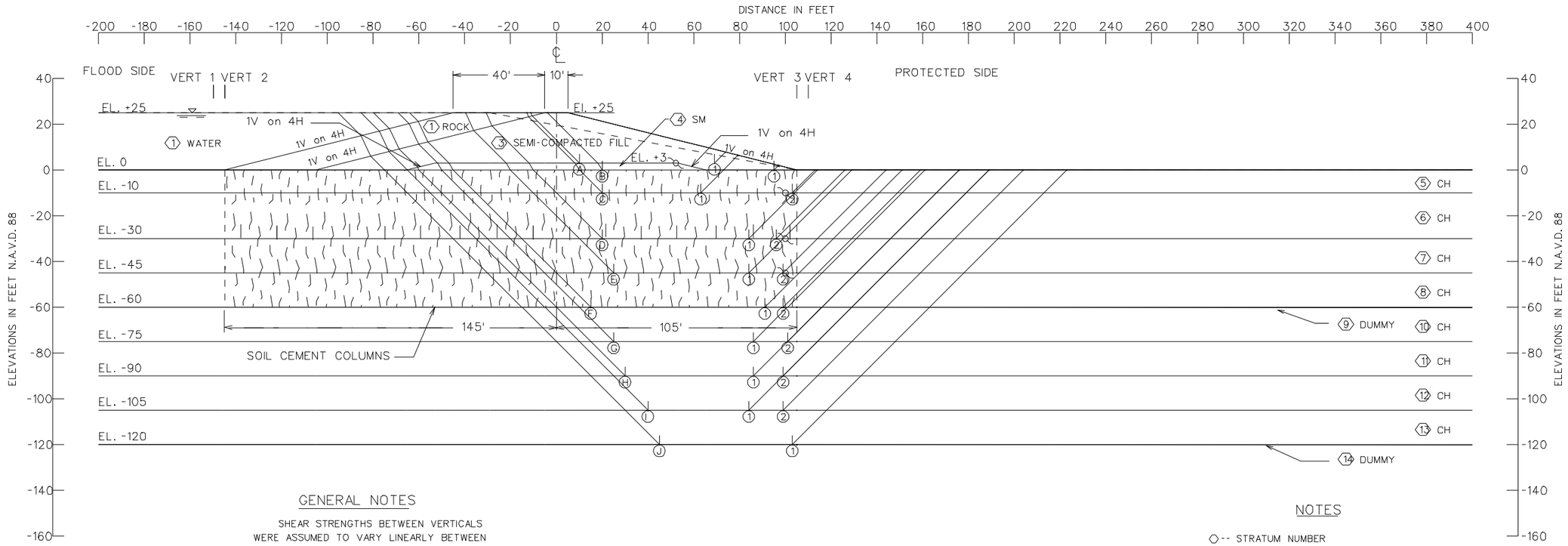
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXX | DWN BY: XXX | CKD BY: XXX | SUBMITTED BY: XXXXXXXXXXXXXXX | DATE: MM/DD/YYYY | SOLICITATION NO.: W912P8-00-B-XXXX | CONTRACT NO.: W912P8-00-C-XXXX | FILE NAME: XXXXXXXXXXXXX-XXX-XXXXXX.DGN | FILE NUMBER: H-NN-NNNNN | SIZE: ANSI X | PLOT SCALE: 1"=300' | PLOT DATE: MM/DD/YYYY

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

REACH 2 - SOIL CEMENT COLUMNS
F/S - CROWN EL. +40
LOUISIANA

SHEET IDENTIFICATION NUMBER



GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F.								FRICTION ANGLE DEGREES
						CENTER OF STRATUM				BOTTOM OF STRATUM				
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	WATER	62.5	62.5	62.5	62.5	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
3	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
6	CH	100	108	108	100	300	2000	2000	300	300	2000	2000	300	0
7	CH	100	108	108	100	375	2000	2000	375	450	2000	2000	450	0
8	CH	100	108	108	100	525	2000	2000	525	600	2000	2000	600	0
9	DUMMY	100	100	100	100	600	600	600	600	600	600	600	600	0
10	CH	100	100	100	100	675	675	675	675	750	750	750	750	0
11	CH	100	100	100	100	825	825	825	825	900	900	900	900	0
12	CH	100	100	100	100	975	975	975	975	1050	1050	1050	1050	0
13	CH	100	100	100	100	1125	1125	1125	1125	1200	1200	1200	1200	0
14	DUMMY	100	100	100	100	1125	1125	1125	1125	1200	1200	1200	1200	0

ASSUMED FAILURE SURFACE			RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.		ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
A	1	3.0	16582	23544	3839	26381	1583	43965	24798	1.77
B	1	.0	20213	33383	1599	30836	274	55195	30562	1.81
C	1	-10.0	59291	86000	45119	64612	18394	190410	46218	4.12
C	2	-10.0	59291	165966	4000	64612	5055	229257	59557	3.85
D	1	-30.0	136685	128000	84000	165363	54258	348685	111105	3.14
D	2	-30.0	136685	152000	16004	165363	46112	304689	119251	2.56
E	1	-45.0	195668	118000	76000	267046	111806	389668	155240	2.51
E	2	-45.0	195668	148000	27250	267046	101741	370918	165305	2.24
F	1	-60.0	252528	45600	87250	400221	188987	385378	211234	1.82
F	2	-60.0	252528	50400	43000	400221	180489	345928	219732	1.57
G	1	-75.0	271845	45750	63235	545889	293401	380830	252488	1.51
G	2	-75.0	271845	57000	63235	545889	281461	392080	264428	1.48
H	1	-90.0	294703	50400	87985	713640	417149	433088	296491	1.46
H	2	-90.0	294703	62100	87985	713640	405483	444788	308157	1.44
I	1	-105.0	323074	46200	117235	899223	564494	486509	334729	1.45
I	2	-105.0	323074	61950	117235	899223	551728	502259	347495	1.45
J	1	-120.0	355223	69600	150985	1108451	720034	575808	388417	1.48

NOTES

- ○ -- STRATUM NUMBER
- ○ -- WEDGE NUMBER
- ⌀ -- CROSSOVER POINT
- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ▽ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT,REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

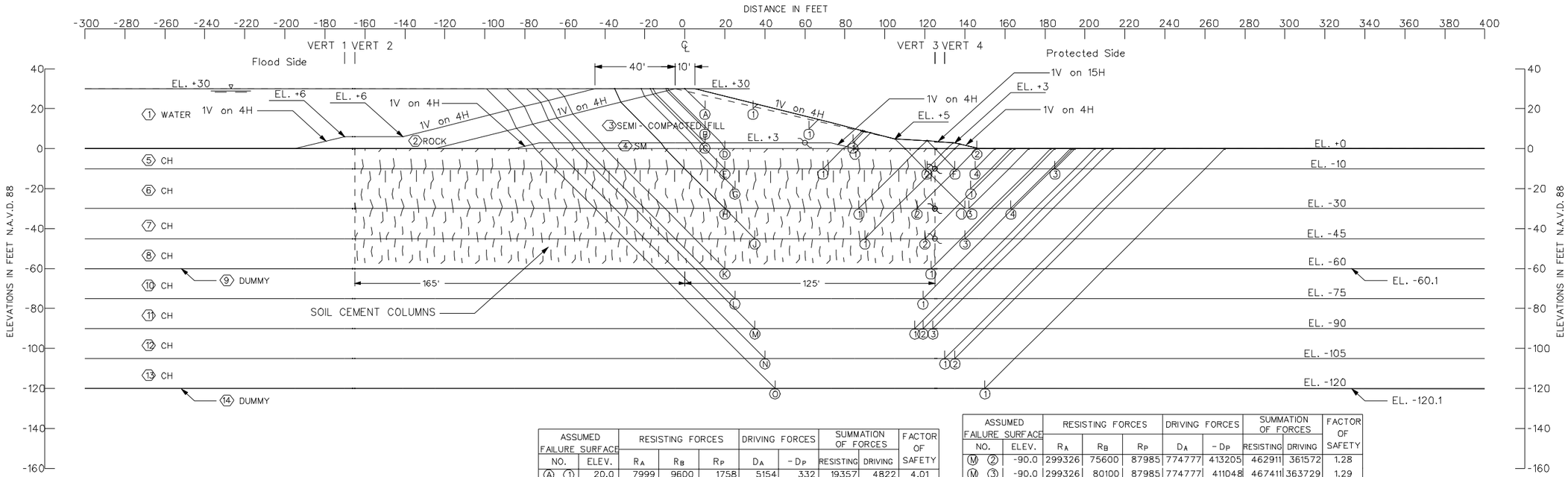
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXX
DWN BY: XXX
CKD BY: XXX
SUPERVISED BY: XXXXXXXXXXXXXXXX
DATE: MM/DD/YYYY
SCALE: AS NOTED
W912P8-00-B-XXXX
CONTRACT NO.: W912P8-00-C-XXXX
FILE NUMBER: XXXXXXXXXXXX-XXXX-XXXXXXXXXXXX
SIZE: ANSI 1
FSCM NO.: 96896
SHT. REF: MM/DD/YYYY

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

REACH 2 - SOIL CEMENT COLUMNS
P/S - CROWN EL.+25 WATER EL.+25
LOUISIANA

SHEET IDENTIFICATION NUMBER



STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT. 1	VERT. 2	VERT. 3	VERT. 4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	BOTTOM OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	FRICTION ANGLE DEGREES
1	WATER	62	62	62	62	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
3	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
6	CH	100	108	108	100	300	2000	2000	300	300	2000	2000	300	0
7	CH	100	108	108	100	375	2000	2000	375	450	2000	2000	450	0
8	CH	100	108	108	100	525	2000	2000	525	600	2000	2000	600	0
9	DUMMY	100	100	100	100	600	600	600	600	600	600	600	600	0
10	CH	100	100	100	100	675	675	675	675	750	750	750	750	0
11	CH	100	100	100	100	825	825	825	825	900	900	900	900	0
12	CH	100	100	100	100	975	975	975	975	1050	1050	1050	1050	0
13	CH	100	100	100	100	1125	1125	1125	1125	1200	1200	1200	1200	0
14	DUMMY	100	100	100	100	1200	1200	1200	1200	1200	1200	1200	1200	0

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
A	1	20.0	7999	9600	1758	5154	332	19357	4822	4.01
B	1	10.0	15227	20800	3679	21708	1454	39706	20254	1.96
C	1	3.0	19837	28359	4639	40065	2311	52835	37754	1.40
D	1	.0	23669	36325	6399	45991	4398	66393	41593	1.60
D	2	.0	23669	44388	160	45991	3	68217	45988	1.48
E	1	-10.0	62526	98000	47561	85484	25373	208087	60111	3.46
E	2	-10.0	62526	202000	6449	85484	9525	270975	75959	3.57
E	3	-10.0	62526	222880	4000	85484	5000	289406	80484	3.60
F	4	-10.0	7161	2000	4000	9473	5055	13161	4418	2.98
G	1	-20.0	101988	206230	10000	133135	20219	318218	112916	2.82
H	1	-30.0	140249	134000	123166	197673	69951	397415	127722	3.11
H	2	-30.0	140249	192000	16162	197673	54585	348411	143088	2.43
H	3	-30.0	140249	215931	16000	197673	45342	372180	152331	2.44
I	4	-30.0	56000	6900	16000	58561	44999	78900	13562	5.82
J	1	-45.0	200249	110000	145919	299176	133917	456168	165259	2.76
J	2	-45.0	200249	170000	27250	299176	109017	397499	190159	2.09
J	3	-45.0	200249	187507	27250	299176	101920	415006	197256	2.10
K	1	-60.0	256966	61800	43000	446582	186471	361766	260111	1.39
L	1	-75.0	275499	70500	63235	601602	289459	409234	312143	1.31
M	1	-90.0	299326	72000	87985	774777	422259	459311	352518	1.30

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
M	2	-90.0	299326	75600	87985	774777	413205	462911	361572	1.28
M	3	-90.0	299326	80100	87985	774777	411048	467411	363729	1.29
N	1	-105.0	326729	94500	117235	971201	554955	538464	416246	1.29
N	2	-105.0	326729	99750	117235	971201	553214	543714	417987	1.30
O	1	-120.0	358594	126000	150985	1188372	719980	635579	468392	1.36

NOTES

⬡ -- STRATUM NUMBER
○ -- WEDGE NUMBER
𝒸 -- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
▽ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT,REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

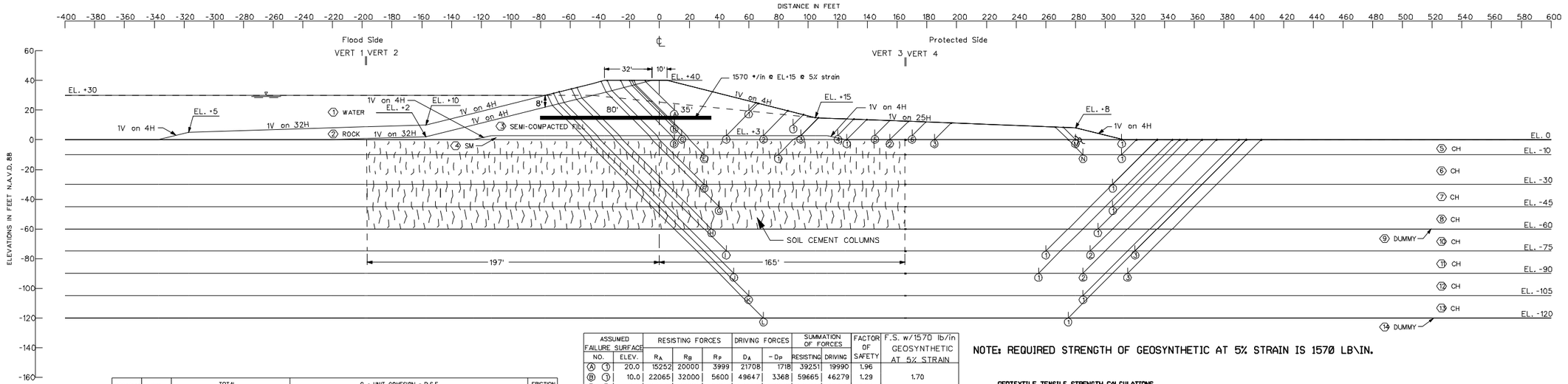
MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY: XXX
FILE NAME: XXXXXXXXXXXXXXXXXXXX
SCALE: AS NOTED
PROJ. MANAGER: XXXXXXXXXXXXXXXXXXXX
PROJECT ID: XXXXXXXXXXXXXXXXXXXX
CONTRACT NO.: XXXXXXXXXXXXXXXXXXXX
FILE NUMBER: XXXXXXXXXXXXXXXXXXXX
DATE: MM/DD/YYYY
PLOT DATE: MM/DD/YYYY

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 2 - SOIL CEMENT COLUMNS
P/S - CROWN EL.+30 WATER EL.+30
LOUISIANA

SHEET IDENTIFICATION NUMBER



STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT.1	VERT.2	VERT.3	VERT.4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT.1	VERT.2	VERT.3	VERT.4	BOTTOM OF STRATUM VERT.1	VERT.2	VERT.3	VERT.4	FRICTION ANGLE DEGREES
①	WATER	62.5	62.5	62.5	62.5	0	0	0	0	0	0	0	0	0
②	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
③	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
④	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
⑤	CH	100	108	108	100	200	2000	2000	200	200	2000	2000	200	0
⑥	CH	100	108	108	100	300	2000	2000	300	300	2000	2000	300	0
⑦	CH	100	108	108	100	375	2000	2000	375	450	2000	2000	450	0
⑧	CH	100	108	108	100	525	2000	2000	525	600	2000	2000	600	0
⑨	DUMMY	100	100	100	100	600	600	600	600	600	600	600	600	0
⑩	CH	100	100	100	100	675	675	675	675	750	750	750	750	0
⑪	CH	100	100	100	100	825	825	825	825	900	900	900	900	0
⑫	CH	100	100	100	100	975	975	975	975	1050	1050	1050	1050	0
⑬	CH	100	100	100	100	1125	1125	1125	1125	1200	1200	1200	1200	0
⑭	DUMMY	100	100	100	100	1200	1200	1200	1200	1200	1200	1200	1200	0

GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	F.S. w/1570 lb/in GEOSYNTHETIC AT 5% STRAIN
Ⓐ	①	20.0	15252	20000	3999	21708	1718	39251	19990	1.96	
Ⓑ	①	10.0	22065	32000	5600	49647	3368	59665	46279	1.29	1.70
Ⓒ	①	3.0	27478	12000	17279	74550	32072	56757	42478	1.34	
Ⓒ	②	3.0	27478	22000	13280	74550	18943	62758	55607	1.13	1.47
Ⓒ	③	3.0	27478	32000	9538	74550	9286	69016	65264	1.06	1.35
Ⓒ	④	3.0	27478	41116	8771	74550	6875	77365	67675	1.14	1.42
Ⓒ	⑤	3.0	27478	51033	8003	74550	5724	86514	68826	1.26	1.53
Ⓒ	⑥	3.0	27478	61033	7226	74550	4667	95737	69883	1.37	
Ⓓ	①	.0	32359	46369	10892	88800	10603	89620	78197	1.15	1.39
Ⓓ	②	.0	32359	57168	10000	88800	8937	99527	79863	1.25	1.48
Ⓓ	③	.0	32359	67369	9076	88800	7362	108804	81438	1.34	
Ⓔ	①	-10.0	72760	100000	55069	129131	42521	227829	86610	2.63	
Ⓕ	①	-30.0	152049	312831	16000	263629	45673	480880	217956	2.21	
Ⓖ	①	-45.0	211777	313757	27250	383254	101921	552784	281333	1.96	
Ⓗ	①	-60.0	268503	156000	43000	534928	183968	467503	350960	1.33	
Ⓘ	①	-75.0	287821	161250	63235	697811	313793	512306	384018	1.33	
Ⓘ	②	-75.0	287821	183750	63235	697811	287896	534806	409915	1.30	
Ⓘ	③	-75.0	287821	206250	63235	697811	281243	557306	416568	1.34	
Ⓙ	①	-90.0	310335	184500	87985	882689	442434	582820	440255	1.32	
Ⓙ	②	-90.0	310335	211500	87985	882689	415010	609820	467679	1.30	

NOTE: REQUIRED STRENGTH OF GEOSYNTHETIC AT 5% STRAIN IS 1570 LB\IN.

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

CONTROLLING FAILURE WEDGE Ⓒ - ③

T = DRIVING(F.S.) - RESISTING
T = (65264(1.3) - 69816 = 15827LB/FT

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY
Ⓙ	③	-90.0	310335	238500	87985	882689	404988	636820	477701	1.33
Ⓚ	①	-105.0	338202	236250	117235	1084589	561256	691687	523333	1.32
Ⓛ	①	-120.0	370572	246000	150985	1305686	738511	767557	567175	1.35
Ⓜ	①	.0	6664	3478	160	3664	3	10302	3661	2.81
Ⓝ	①	-10.0	10831	5200	4000	17358	5014	20031	12344	1.62

NOTES

○ -- STRATUM NUMBER
○ -- WEDGE NUMBER
⊘ -- CROSSOVER POINT
Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
▽ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT,REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

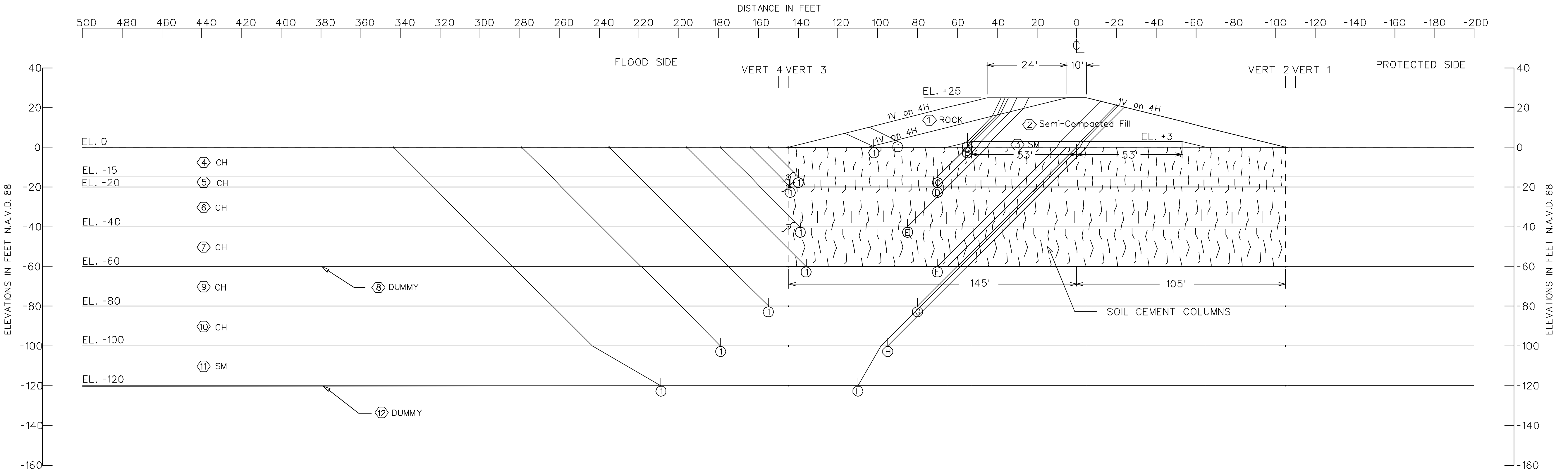
MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

U.S. ARMY ENGINEER DISTRICT
NEW ORLEANS, LOUISIANA
MISSISSIPPI VALLEY DIVISION

DESIGN BY: MG
DRAWN BY: MG/CT/TT
CHECKED BY: RS/DEB/RJ
SUBMITTED BY: RS/DEB/RJ
DATE: 10/2/2000
SCALE: 1:120
CONTRACT NO.:
LAST REVISION: 10/2/2000
FILE NAME: 10/2/2000/XXXX/XXXX
SHEET NO.: 7 OF 8
DWG. NO.: 7-2-98

JEFFERSON COUNTY, MISSISSIPPI, AND VICINITY (IN PROJECT
PRELIMINARY TECHNICAL REPORT TO CHANGES
REACH 2 - SOIL CEMENT COLUMNS
F/S - CROWN EL.+48 WATER EL.+38
LUDSWAN

SHEET IDENTIFICATION NUMBER



ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
A 1	3.0	14631	14000	15937	28964	5111	44568	23853	1.87
B 1	.0	16919	18800	15936	37302	5111	51655	32191	1.60
C 1	-15.0	79623	140000	6000	86700	11098	225623	75602	2.98
D 1	-20.0	99940	148000	9250	111122	19078	257190	92044	2.79
E 1	-40.0	180415	108000	27250	221815	78153	315665	143662	2.20
F 1	-60.0	261455	49500	53233	396070	177395	364188	218675	1.67
G 1	-80.0	293790	71250	87230	591923	314554	452270	277369	1.63
H 1	-100.0	334943	96600	129230	820098	493048	560773	327050	1.71
I 1	-120.0	417671	301111	300015	1078214	715940	1018797	362274	2.81

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
2	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
3	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
4	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
5	CH	100	108	108	100	325	2000	2000	325	350	2000	2000	350	0
6	CH	100	108	108	100	450	2000	2000	450	550	2000	2000	550	0
7	CH	100	108	108	100	650	2000	2000	650	750	2000	2000	750	0
8	DUMMY	100	100	100	100	750	750	750	750	750	750	750	750	0
9	CH	100	100	100	100	850	850	850	850	950	950	950	950	0
10	CH	100	100	100	100	1050	1050	1050	1050	1150	1150	1150	1150	0
11	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
12	DUMMY	122	122	122	122	0	0	0	0	0	0	0	0	30

NOTES

- ⬡ -- STRATUM NUMBER
- ○ -- WEDGE NUMBER
- -- CROSSOVER POINT
- ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ▽ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

GENERAL NOTES

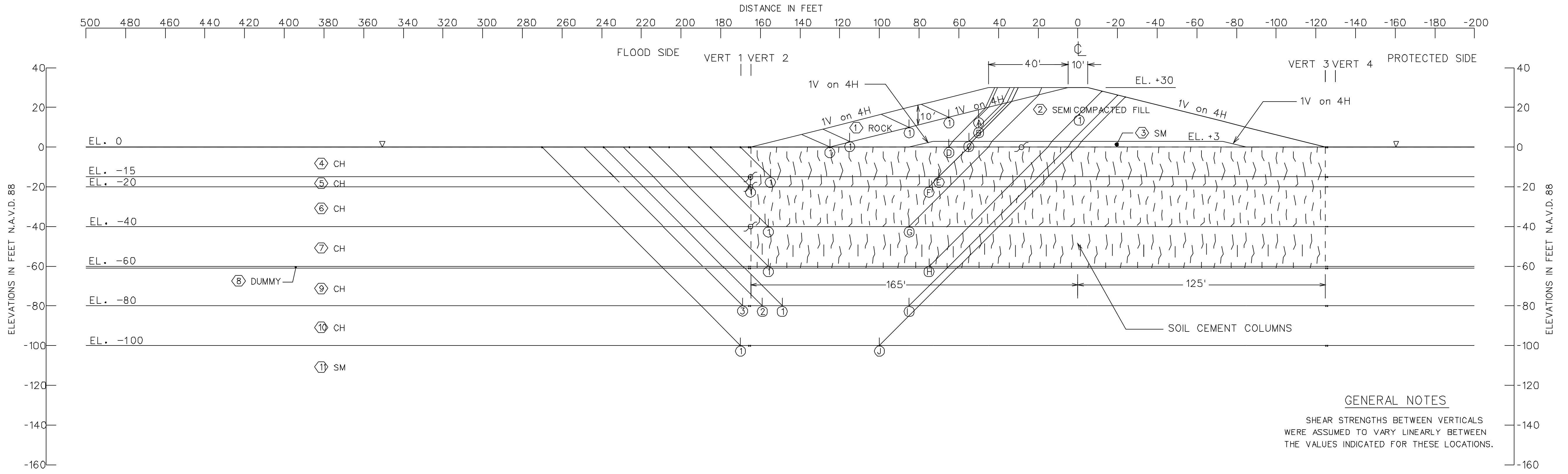
SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 3 - SOIL CEMENT COLUMNS
F/S CROWN EL. +25 WATER EL. +0
LOUISIANA

SHEET IDENTIFICATION NUMBER
X



GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

ASSUMED FAILURE SURFACE NO.	ASSUMED FAILURE SURFACE ELEV.	RESISTING FORCES R_A	RESISTING FORCES R_B	RESISTING FORCES R_P	DRIVING FORCES D_A	DRIVING FORCES $-D_P$	SUMMATION OF FORCES RESISTING	SUMMATION OF FORCES DRIVING	FACTOR OF SAFETY
Ⓐ ①	15.0	9163	6000	15442	14226	4289	30605	9937	3.08
Ⓑ ①	10.0	12919	14000	15430	24859	4286	42349	20573	2.06
Ⓒ ①	3.0	18501	24000	15087	43371	3765	57588	39606	1.45
Ⓓ ①	.0	24481	74478	15400	48811	4279	114359	44532	2.57
Ⓔ ①	-15.0	84765	170000	60000	111329	13768	314765	97561	3.23
Ⓕ ①	-20.0	104764	179999	9250	134766	18999	294013	115767	2.54
Ⓖ ①	-40.0	186058	142000	27243	260140	77308	355301	182832	1.94
Ⓗ ①	-60.0	266609	60750	53230	441923	172307	380589	269616	1.41
Ⓘ ①	-80.0	298708	60800	87050	647504	323986	446558	323518	1.38
Ⓘ ②	-80.0	298708	70300	87050	647504	305567	456058	341937	1.33
Ⓘ ③	-80.0	298708	79800	87050	647504	304992	465558	342512	1.36
Ⓙ ①	-100.0	339907	80500	129050	884622	478987	549457	405635	1.35

NOTES

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- ○ -- WEDGE NUMBER
- ⌀ -- CROSSOVER POINT
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$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

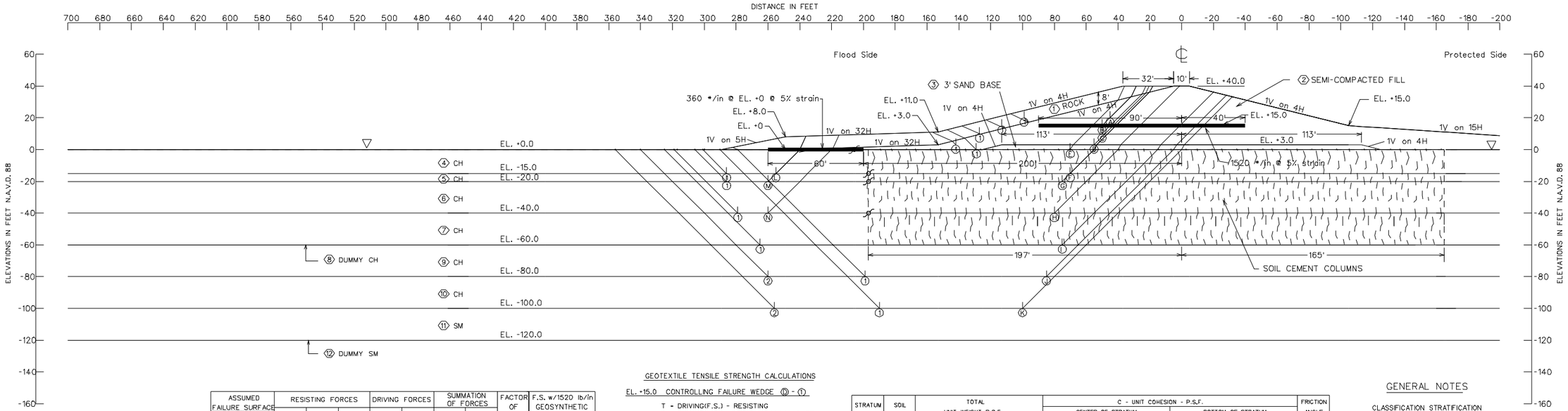
STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F. VERT. 1	VERT. 2	VERT. 3	VERT. 4	C - UNIT COHESION - P.S.F. CENTER OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	BOTTOM OF STRATUM VERT. 1	VERT. 2	VERT. 3	VERT. 4	FRICTION ANGLE DEGREES
1	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
2	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
3	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
4	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
5	CH	95	108	108	95	325	2000	2000	325	350	2000	2000	350	0
6	CH	95	108	108	95	450	2000	2000	450	550	2000	2000	550	0
7	CH	95	108	108	95	650	2000	2000	650	750	2000	2000	750	0
8	DUMMY	100	100	100	100	750	750	750	750	750	750	750	750	0
9	CH	100	100	100	100	850	850	850	850	950	950	950	950	0
10	CH	100	100	100	100	1050	1050	1050	1050	1150	1150	1150	1150	0
11	SM	122	122	122	122	0	0	0	0	0	0	0	0	30

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 3 - SOIL CEMENT COLUMNS
F/S CROWN EL. +30 WATER EL. +0
LOUISIANA

SHEET IDENTIFICATION NUMBER
X



NOTES

○ -- STRATUM NUMBER
○ -- WEDGE NUMBER
ℴ -- CROSSOVER POINT
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P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY = $\frac{R_A + R_B + R_P}{D_A - D_P}$

ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY	F.S. w/1520 lb/in GEOSYNTHETIC AT 5% STRAIN
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING		
Ⓐ ①	20.0	12960	21600	7654	23872	331	42214	23541	1.79	
Ⓑ ①	15.0	16960	25200	8614	35682	1235	50774	34447	1.47	
Ⓒ ①	10.0	21290	30800	9575	51448	2337	61665	49111	1.26	1.63
Ⓓ ①	3.0	27101	34800	11852	76195	5459	73753	70736	1.04	1.30
Ⓔ ①	.0	33388	97738	15656	78092	13914	146782	64178	2.29	
Ⓕ ①	-15.0	95236	286102	6000	159539	10806	387338	148733	2.60	
Ⓖ ①	-20.0	115236	288259	9250	187165	19181	412745	167984	2.46	
Ⓗ ①	-40.0	198339	290618	27250	336981	78880	516207	258101	2.00	
Ⓘ ①	-60.0	275210	142500	53250	534442	183660	470960	350782	1.34	
Ⓙ ①	-80.0	307510	108300	87915	757406	391625	503725	365781	1.38	
Ⓙ ②	-80.0	307510	166250	87250	757406	325653	561010	431753	1.30	F.S. w/360 lb/in GEOSYNTHETIC AT 5% STRAIN
Ⓚ ①	-100.0	348710	103500	129250	1012027	593546	581460	418481	1.39	
Ⓚ ②	-100.0	348710	179400	129250	1012027	507421	657360	504606	1.30	
Ⓛ ①	-15.0	9585	6000	6000	30783	10898	21585	19885	1.09	1.30
Ⓜ ①	-20.0	12835	7000	9250	43315	20131	29085	23184	1.25	1.44
Ⓝ ①	-40.0	31352	9350	27250	124960	79461	67952	45499	1.49	

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

EL. -15.0 CONTROLLING FAILURE WEDGE Ⓓ - ①

T = DRIVING(F.S.) - RESISTING
T = (70736)(1.3) - 73753 = 18204 LB/FT

EL. +0.0 CONTROLLING FAILURE WEDGE Ⓒ - ①

T = DRIVING(F.S.) - RESISTING
T = (19885)(1.3) - 21585 = 4266 LB/FT

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
①	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
②	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
③	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
④	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
⑤	CH	100	108	108	100	325	2000	2000	325	350	2000	2000	350	0
⑥	CH	100	108	108	100	450	2000	2000	450	550	2000	2000	550	0
⑦	CH	100	108	108	100	650	2000	2000	650	750	2000	2000	750	0
⑧	DUMMY	100	100	100	100	750	750	750	750	750	750	750	750	0
⑨	CH	100	100	100	100	850	850	850	850	950	950	950	950	0
⑩	CH	100	100	100	100	1050	1050	1050	1050	1150	1150	1150	1150	0
⑪	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
⑫	DUMMY	122	122	122	122	0	0	0	0	0	0	0	0	30

GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS.

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

SYM.	DESCRIPTION	DATE	APR.	CHK.	DSGN.

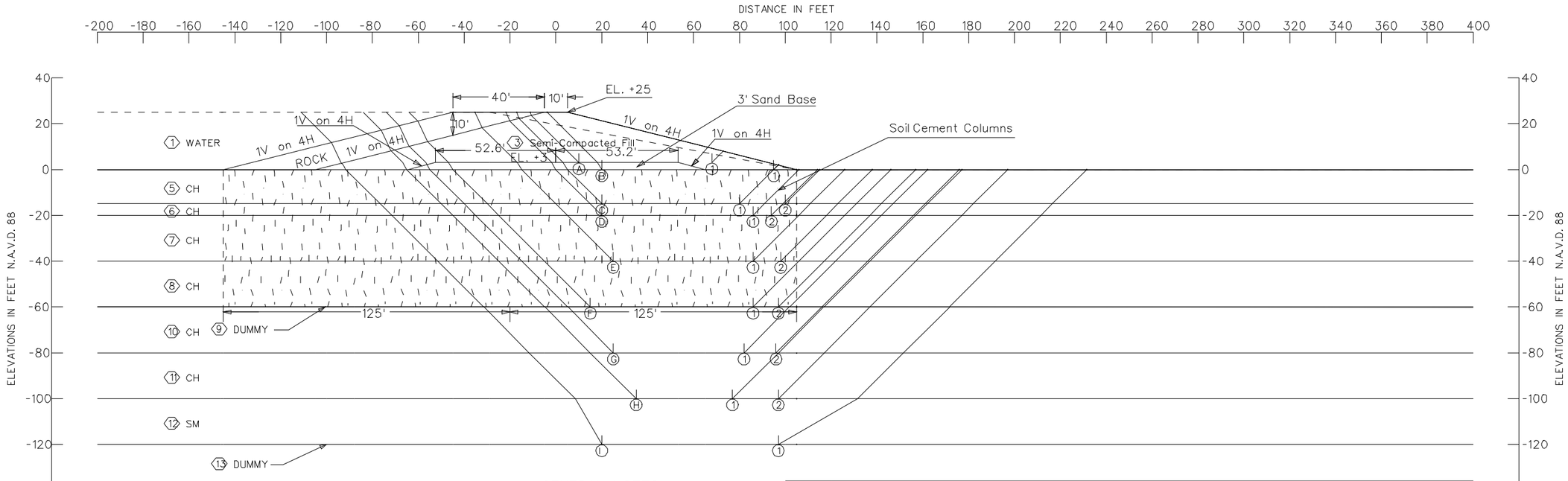
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: MARK OTTYN
DRAWN BY: JOE BIN
CHECKED BY: TOM
SUBMITTED BY: LAWRENCE BURT
FILE NAME: 0000000000000000
SIZE: 36 X 48
SCALE: NONE
DATE: 00/00/0000
FILE NO.: 0000000000
REV. NO.: 0
SUBMITTED: 00/00/0000
REV. DATE: 00/00/0000

LEVEE FOR FEDERAL PROJECT
PRELIMINARY, CONTRACT, REPORT TO CONGRESS

REACH 3 - SOIL CEMENT COLUMNS
F/S CROWN EL. 4.8 WATER EL. 1.0

SHEET IDENTIFICATION NUMBER
X



GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS.

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

NOTES

⬡ -- STRATUM NUMBER
○ -- WEDGE NUMBER
⊘ -- CROSSOVER POINT
Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
▽ -- STATIC WATER SURFACE
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P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
Ⓐ ①	3.0	16582	23144	3999	26381	1717	43725	24664	1.77
Ⓑ ①	.0	20213	33383	1599	30836	274	55195	30562	1.81
Ⓒ ①	-15.0	78831	120000	61599	85683	19639	260430	66044	3.94
Ⓒ ②	-15.0	78831	160000	6000	85683	11030	244831	74653	3.28
Ⓓ ①	-20.0	98139	132000	80000	109419	26559	310139	82860	3.74
Ⓓ ②	-20.0	98139	148000	26000	109419	21799	272139	87620	3.11
Ⓔ ①	-40.0	176136	122000	89250	229335	88820	387386	140515	2.76
Ⓔ ②	-40.0	176136	146000	27250	229335	78233	349386	151102	2.31
Ⓕ ①	-60.0	252527	53250	107250	400218	190517	413027	209701	1.97
Ⓕ ②	-60.0	252527	61500	53250	400218	176936	367277	223282	1.64
Ⓖ ①	-80.0	284670	54150	87230	600286	332922	426050	267364	1.59
Ⓖ ②	-80.0	284670	67450	87230	600286	316221	439350	284065	1.55
Ⓗ ①	-100.0	324833	48300	129230	836652	514922	502363	321730	1.56
Ⓗ ②	-100.0	324833	71300	129230	836652	493928	525363	342724	1.53
Ⓘ ①	-120.0	404199	0	299168	1123282	716448	703367	406834	1.73

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
⬡1	WATER	62.5	62.5	62.5	62.5	0	0	0	0	0	0	0	0	0
⬡2	RIPRAP	132	132	132	132	0	0	0	0	0	0	0	0	40
⬡3	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
⬡4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
⬡5	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
⬡6	CH	100	108	108	100	325	2000	2000	325	350	2000	2000	350	0
⬡7	CH	100	108	108	100	450	2000	2000	450	550	2000	2000	550	0
⬡8	CH	100	108	108	100	650	2000	2000	650	750	2000	2000	750	0
⬡9	CH	100	100	100	100	750	750	750	750	750	750	750	750	0
⬡10	CH	100	100	100	100	850	850	850	850	950	950	950	950	0
⬡11	CH	100	100	100	100	1050	1050	1050	1050	1150	1150	1150	1150	0
⬡12	SM	122	122	122	122	0	0	0	0	0	0	0	0	30

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

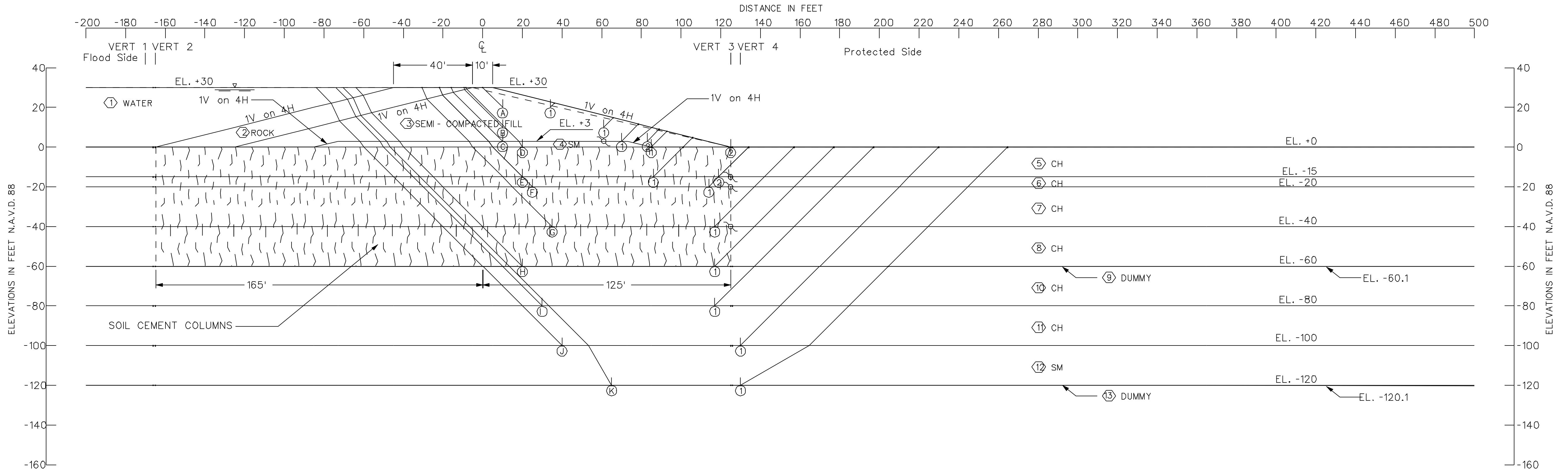
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXXXXXXXX
FILE NAME: XXXXXXXXXXXX-XXX-XXXXXXXXXX
AREA X
DATE: MM/DD/YYYY
CONTRACT NO.: W912P8-00-B-XXXX
W912P8-00-C-XXXX
FILE NUMBER: H:\N-XNNNNN
SHEET NO.: P-XXXX

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

REACH 3 - SOIL CEMENT COLUMNS
P/S CROWN EL. +25 WATER EL. +25
LOUISIANA

SHEET IDENTIFICATION NUMBER
X



STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F.								FRICTION ANGLE DEGREES
						CENTER OF STRATUM				BOTTOM OF STRATUM				
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	WATER	62	62	62	62	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
3	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
6	CH	100	108	108	100	325	2000	2000	325	350	2000	2000	350	0
7	CH	100	108	108	100	450	2000	2000	450	550	2000	2000	550	0
8	CH	100	108	108	100	650	2000	2000	650	750	2000	2000	750	0
9	DUMMY	100	100	100	100	750	750	750	750	750	750	750	750	0
10	CH	100	100	100	100	850	850	850	850	950	950	950	950	0
11	CH	100	100	100	100	1050	1050	1050	1050	1150	1150	1150	1150	0
12	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
13	DUMMY	122	122	122	122	0	0	0	0	0	0	0	0	30

ASSUMED FAILURE SURFACE			RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.		ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
A	1	20.0	7999	9600	1758	5154	332	19357	4822	4.01
B	1	10.0	15227	20400	3834	21708	1580	39461	20128	1.96
C	1	3.0	19837	23698	6872	40065	5074	50407	34991	1.44
C	2	3.0	19837	28133	4790	40065	2465	52760	37600	1.40
D	1	.0	23680	36780	6389	45991	4386	66849	41605	1.61
D	2	.0	23680	42787	0	45991	0	66467	45991	1.45
E	1	-15.0	81992	132000	63827	109414	26689	277819	82725	3.36
E	2	-15.0	81992	198000	6000	109414	11166	285992	98248	2.91
F	1	-20.0	101992	178000	26000	133135	21772	305992	111363	2.75
G	1	-40.0	180655	164000	27250	258825	78419	371905	180406	2.06
H	1	-60.0	256966	72750	53250	446583	176917	382966	269666	1.42
I	1	-80.0	289244	82650	87230	656335	315413	459124	340922	1.35
J	1	-100.0	329398	103500	129230	902288	493050	562128	409238	1.37
K	1	-120.0	408150	215774	300018	1160335	715944	923942	444391	2.08

GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

NOTES

○ -- STRATUM NUMBER
○ -- WEDGE NUMBER
-- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

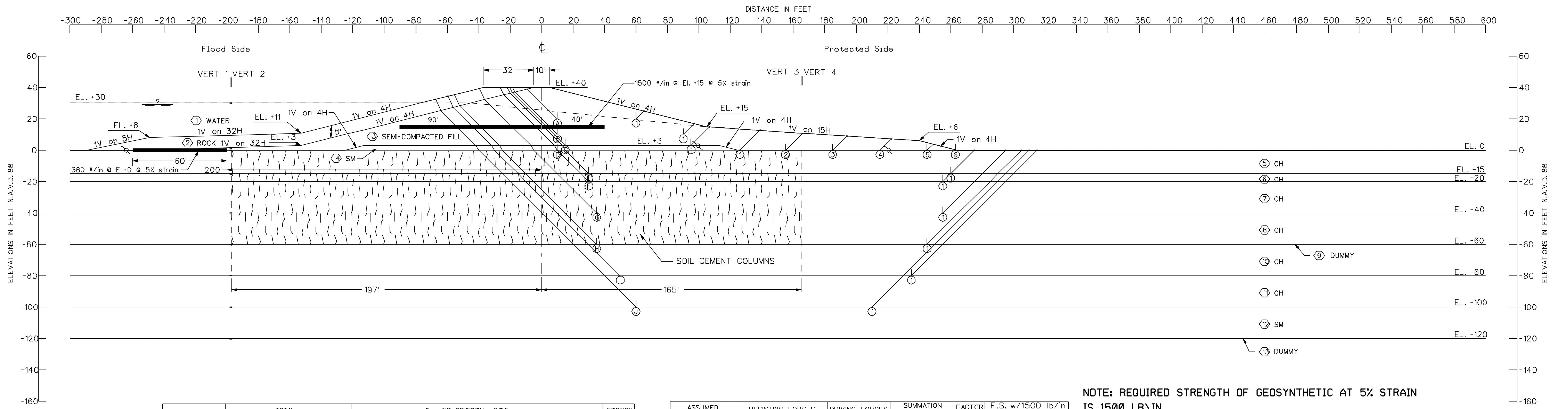
FACTOR OF SAFETY $= \frac{R_A + R_B + R_P}{D_A - D_P}$

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 3 - SOIL CEMENT COLUMNS
P/S CROWN EL. +30 WATER EL. +30
LOUISIANA

SHEET IDENTIFICATION NUMBER
X



GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	WATER	62.5	62.5	62.5	62.5	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
3	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
6	CH	100	108	108	100	325	2000	2000	325	350	2000	2000	350	0
7	CH	100	108	108	100	450	2000	2000	450	550	2000	2000	550	0
8	CH	100	108	108	100	650	2000	2000	650	750	2000	2000	750	0
9	DUMMY	100	100	100	100	750	750	750	750	750	750	750	750	0
10	CH	100	100	100	100	850	850	850	850	950	950	950	950	0
11	CH	100	100	100	100	1050	1050	1050	1050	1150	1150	1150	1150	0
12	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
13	DUMMY	122	122	122	122	0	0	0	0	0	0	0	0	30

ASSUMED FAILURE SURFACE NO.		ELEV.	RESISTING FORCES R_A	R_B	R_P	DRIVING FORCES D_A	$-D_P$	SUMMATION OF FORCES RESISTING	DRIVING	FACTOR OF SAFETY	F.S. w/1500 lb/in GEOSYNTHETIC AT 5% STRAIN
A	1	20.0	15252	20000	3999	21708	1718	39251	19990	1.96	
B	1	10.0	22065	32000	5600	49647	3368	59665	46279	1.29	1.68
C	1	3.0	27478	32000	9500	74550	9280	68978	65270	1.06	1.33
D	1	.0	32359	46284	10200	88800	9536	88843	79264	1.12	1.35
D	2	.0	32359	56331	8750	88800	7017	97440	81783	1.19	1.41
D	3	.0	32359	65105	7249	88800	4817	104713	83983	1.25	1.46
D	4	.0	32359	72233	5749	88800	3029	110341	85771	1.29	1.50
D	5	.0	32359	77651	3039	88800	992	113049	87808	1.29	1.49
D	6	.0	32359	78884	160	88800	3	111403	88797	1.25	1.46
E	1	-15.0	92508	289880	6000	158591	10907	388388	147684	2.63	
F	1	-20.0	112359	302306	9250	190863	20175	423915	170688	2.48	
G	1	-40.0	191777	310209	27250	342594	78674	529236	263920	2.01	
H	1	-60.0	268503	157500	53250	534928	181020	479253	353908	1.35	
I	1	-80.0	301586	175750	87250	755293	325862	564586	429431	1.31	
J	1	-100.0	341350	172500	129250	1013095	524059	643100	489036	1.32	

NOTE: REQUIRED STRENGTH OF GEOSYNTHETIC AT 5% STRAIN IS 1500 LB\IN.

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

CONTROLLING FAILURE WEDGE (C) - (1)

T = DRIVING(F.S.) - RESISTING
T = (65270)(1.3) - 68978 = 15873LB/FT

NOTES

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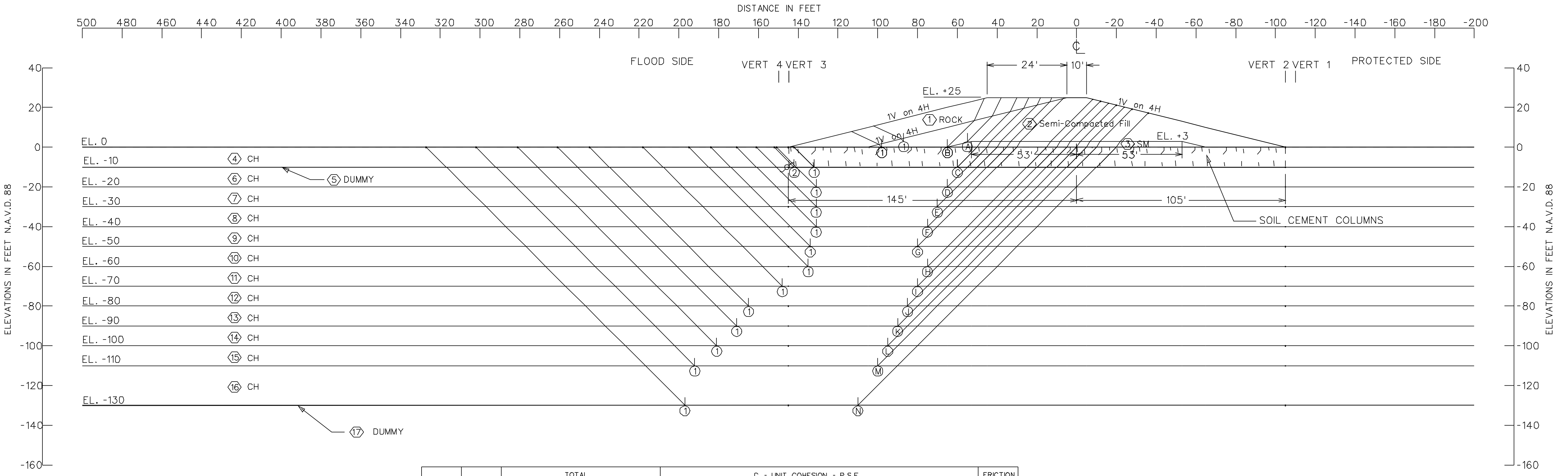
$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 3 - SOIL CEMENT COLUMNS
P/S CROWN EL. +40 WATER EL. +30
LOUISIANA

SHEET IDENTIFICATION NUMBER
X



ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
Ⓐ ①	3.0	14631	12800	16417	28964	5977	43848	22987	1.91
Ⓑ ①	.0	16510	37457	16576	32396	6277	70543	26119	2.70
Ⓒ ①	-10.0	59940	72000	40087	70689	8061	172027	62628	2.75
Ⓒ ②	-10.0	59940	82000	4000	70689	4898	145940	65791	2.22
Ⓓ ①	-20.0	80415	66000	24011	113473	23530	170426	89943	1.89
Ⓔ ①	-30.0	102893	67100	45990	168637	52526	215983	116111	1.86
Ⓕ ①	-40.0	128668	67200	70990	233450	90768	266858	142682	1.87
Ⓖ ①	-50.0	156730	70200	97990	308891	139260	324920	169631	1.92
Ⓗ ①	-60.0	185300	84000	126990	402578	200142	396290	202436	1.96
Ⓘ ①	-70.0	215445	102000	157990	500195	268623	475435	231572	2.05
Ⓙ ①	-80.0	247600	128000	190990	608081	352850	566590	255231	2.22
Ⓚ ①	-90.0	281800	137700	225990	726258	448577	645490	277681	2.32
Ⓛ ①	-100.0	317953	154800	262989	854667	555803	735742	298864	2.46
Ⓜ ①	-110.0	355878	174800	301989	993195	674530	832667	318665	2.61
Ⓝ ①	-130.0	433729	174000	381989	1300967	946482	989718	354485	2.79

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
2	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
3	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
4	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
5	DUMMY	95	95	95	95	1000	1000	1000	1000	1000	1000	1000	1000	0
6	CH	115	115	115	115	1050	1050	1050	1050	1100	1100	1100	1100	0
7	CH	115	115	115	115	1150	1150	1150	1150	1200	1200	1200	1200	0
8	CH	115	115	115	115	1250	1250	1250	1250	1300	1300	1300	1300	0
9	CH	115	115	115	115	1350	1350	1350	1350	1400	1400	1400	1400	0
10	CH	115	115	115	115	1450	1450	1450	1450	1500	1500	1500	1500	0
11	CH	115	115	115	115	1550	1550	1550	1550	1600	1600	1600	1600	0
12	CH	115	115	115	115	1650	1650	1650	1650	1700	1700	1700	1700	0
13	CH	115	115	115	115	1750	1750	1750	1750	1800	1800	1800	1800	0
14	CH	115	115	115	115	1850	1850	1850	1850	1900	1900	1900	1900	0
15	CH	115	115	115	115	1950	1950	1950	1950	2000	2000	2000	2000	0
16	CH	115	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0
17	DUMMY	115	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0

GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

NOTES

- ⬡ -- STRATUM NUMBER
- ○ -- WEDGE NUMBER
- -- CROSSOVER POINT
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- ∇ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
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$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

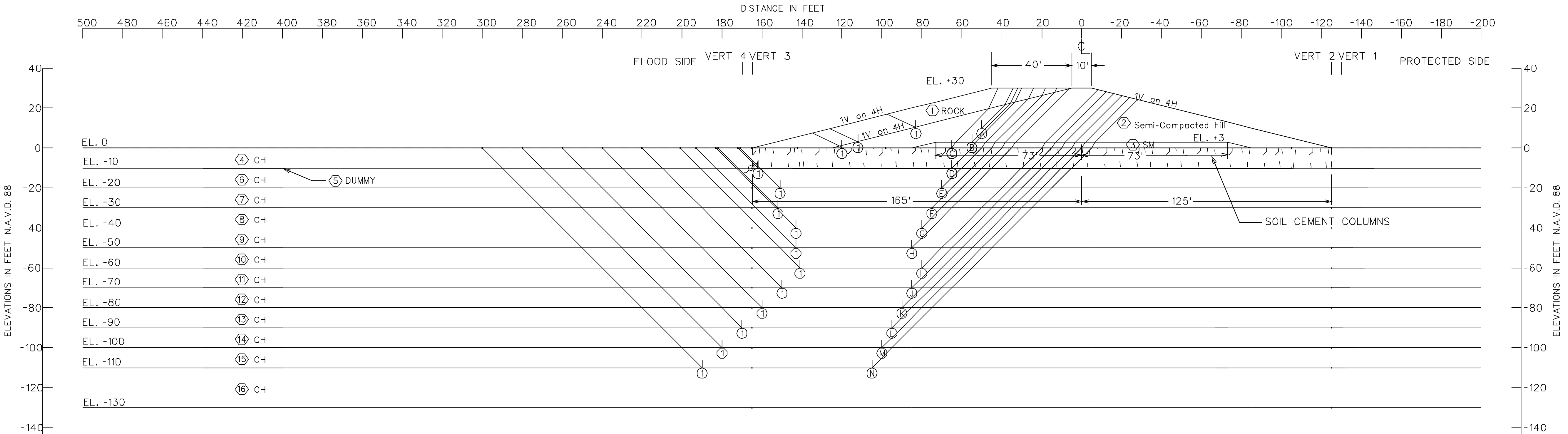
US ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXX | DWN BY: XXX | CKD BY: XXX | SUBMITTED BY: XXXXXXXXXXXXXXXX | FILE NAME: XXXXXXXXXXXXX-XXX-XXXXXX.DGN | SIZE: ANSI X | PLOT SCALE: 1"=5000' | DATE: MM/DD/YYYY | SOLICITATION NO.: W912P8-00-B-XXXX | CONTRACT NO.: W912P8-00-C-XXXX | FILE NUMBER: H-NN-NNNNN | PLOT DATE: MM/DD/YYYY

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 4 - SOIL CEMENT COLUMNS
F/S CROWN EL. +25 WATER EL. +0
LOUISIANA

SHEET IDENTIFICATION NUMBER
X



GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
(A) (1)	10.0	12919	13200	15780	24859	4833	41899	20026	2.09
(B) (1)	3.0	18501	22800	15617	43373	4561	56918	38812	1.47
(C) (1)	.0	24483	70518	16255	48817	5683	111256	43134	2.58
(D) (1)	-10.0	64765	97000	4000	89904	4898	165765	85006	1.95
(E) (1)	-20.0	86287	89100	24999	138660	24510	200386	114150	1.76
(F) (1)	-30.0	110048	92400	47990	198003	50794	250438	147209	1.70
(G) (1)	-40.0	135823	81900	72990	267740	95517	290713	172223	1.69
(H) (1)	-50.0	163884	81200	99990	348032	145246	345074	202786	1.70
(I) (1)	-60.0	192455	91500	128990	447868	207992	412945	239876	1.72
(J) (1)	-70.0	222600	104000	159990	550336	274931	486590	275405	1.77
(K) (1)	-80.0	254745	119000	192990	663027	354562	566735	308465	1.84
(L) (1)	-90.0	288900	135000	227990	785988	448578	651890	337410	1.93
(M) (1)	-100.0	325100	152000	264990	919242	555805	742090	363437	2.04
(N) (1)	-110.0	363253	170000	303990	1062722	674532	837243	388190	2.16

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
2	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
3	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
4	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
5	DUMMY	95	95	95	95	1000	1000	1000	1000	1000	1000	1000	1000	0
6	CH	115	115	115	115	1050	1050	1050	1050	1100	1100	1100	1100	0
7	CH	115	115	115	115	1150	1150	1150	1150	1200	1200	1200	1200	0
8	CH	115	115	115	115	1250	1250	1250	1250	1300	1300	1300	1300	0
9	CH	115	115	115	115	1350	1350	1350	1350	1400	1400	1400	1400	0
10	CH	115	115	115	115	1450	1450	1450	1450	1500	1500	1500	1500	0
11	CH	115	115	115	115	1550	1550	1550	1550	1600	1600	1600	1600	0
12	CH	115	115	115	115	1650	1650	1650	1650	1700	1700	1700	1700	0
13	CH	115	115	115	115	1750	1750	1750	1750	1800	1800	1800	1800	0
14	CH	115	115	115	115	1850	1850	1850	1850	1900	1900	1900	1900	0
15	CH	115	115	115	115	1950	1950	1950	1950	2000	2000	2000	2000	0
16	CH	115	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0

NOTES

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$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

US ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXXXXX
FILE NAME: XXXXXXXXXXXXX-XXX-XXXXXX.DGN
SIZE: ANSI X
PLOT SCALE: 1"=5000'
DATE: MM/DD/YYYY
SOLICITATION NO.: W912P8-00-B-XXXX
CONTRACT NO.: W912P8-00-C-XXXX
FILE NUMBER: H-NN-NNNNN
PLOT DATE: MM/DD/YYYY

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 4 - SOIL CEMENT COLUMNS
F/S CROWN EL. +30 WATER EL. +0
LOUISIANA

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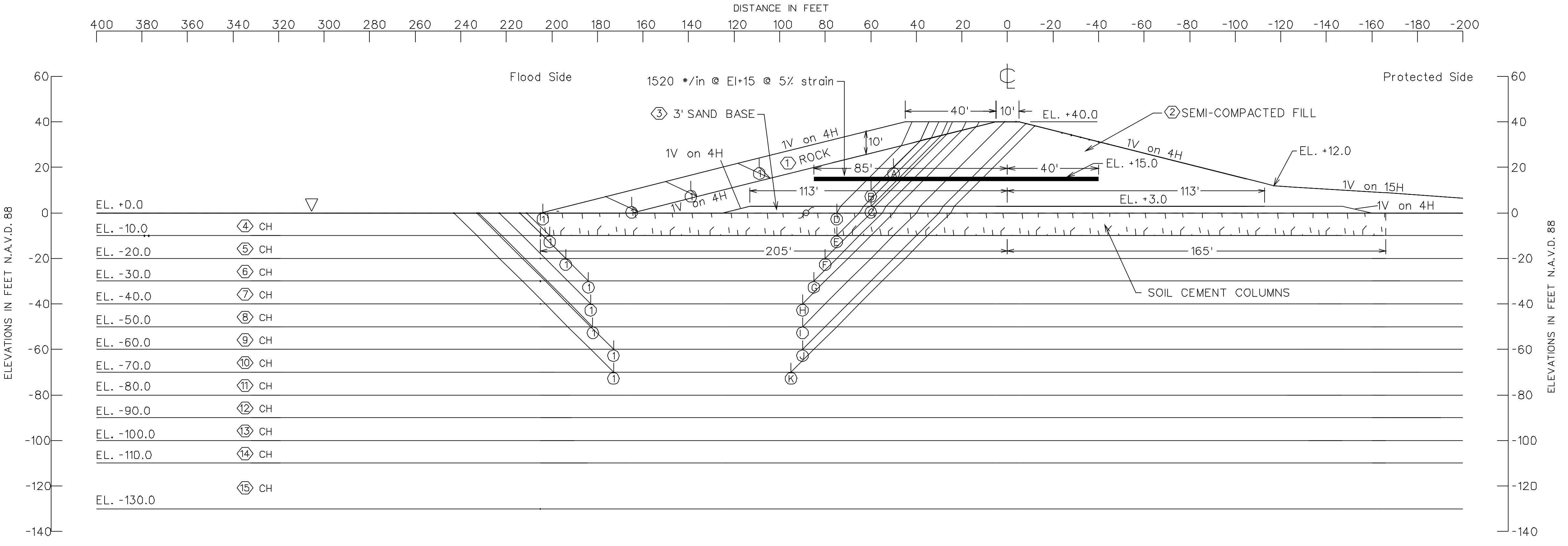


ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY	F.S. w/1520 lb/in GEOSYNTHETIC AT 5% STRAIN
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING		
Ⓐ ①	20.0	12918	23600	11620	24859	-457	48138	25316	1.90	
Ⓑ ①	10.0	20919	31600	13219	51805	1138	65738	50667	1.30	
Ⓒ ①	3.0	26658	42000	13536	78703	1524	82194	77179	1.06	1.30
Ⓓ ①	.0	34133	147614	10	82290	3	181757	82287	2.21	
Ⓔ ①	-10.0	74415	138600	4001	134377	5014	217016	129363	1.68	
Ⓕ ①	-20.0	95946	125400	25000	192778	23296	246346	169482	1.45	
Ⓖ ①	-30.0	119764	118800	48000	261805	56624	286564	205181	1.40	
Ⓗ ①	-40.0	145869	120900	73000	341465	95582	339769	245883	1.38	
Ⓘ ①	-50.0	174480	128800	100000	437120	146073	403280	291047	1.39	
Ⓙ ①	-60.0	202775	124500	129000	543807	216787	456275	327020	1.40	
Ⓚ ①	-70.0	232920	124800	160000	656059	289535	517720	366524	1.41	

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

EL. +15.0 CONTROLLING FAILURE WEDGE Ⓒ - ①

T = DRIVING(F.S.) - RESISTING

T = (77179)(1.3) - 82194 = 18140 LB/FT

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F.								FRICTION ANGLE DEGREES
						CENTER OF STRATUM				BOTTOM OF STRATUM				
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
①	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
②	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
③	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
④	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
⑤	CH	115	115	115	115	1050	1050	1050	1050	1100	1100	1100	1100	0
⑥	CH	115	115	115	115	1150	1150	1150	1150	1200	1200	1200	1200	0
⑦	CH	115	115	115	115	1250	1250	1250	1250	1300	1300	1300	1300	0
⑧	CH	115	115	115	115	1350	1350	1350	1350	1400	1400	1400	1400	0
⑨	CH	115	115	115	115	1450	1450	1450	1450	1500	1500	1500	1500	0
⑩	CH	115	115	115	115	1550	1550	1550	1550	1600	1600	1600	1600	0
⑪	CH	115	115	115	115	1650	1650	1650	1650	1700	1700	1700	1700	0
⑫	CH	115	115	115	115	1750	1750	1750	1750	1800	1800	1800	1800	0
⑬	CH	115	115	115	115	1850	1850	1850	1850	1900	1900	1900	1900	0
⑭	CH	115	115	115	115	1950	1950	1950	1950	2000	2000	2000	2000	0
⑮	CH	115	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0

NOTES

- ◇ -- STRATUM NUMBER
- ○ -- WEDGE NUMBER
- ⌿ -- CROSSOVER POINT
- Φ -- ANGLE OF INTERNAL FRICTION, DEGREES
- C -- UNIT COHESION, P.S.F.
- ▽ -- STATIC WATER SURFACE
- D -- HORIZONTAL DRIVING FORCE IN POUNDS
- R -- HORIZONTAL RESISTING FORCE IN POUNDS
- A -- AS A SUBSCRIPT,REFERS TO ACTIVE WEDGE
- B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
- P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

GENERAL NOTES

CLASSIFICATION STRATIFICATION SHEAR STRENGTHS AND UNIT WEIGHTS OF THE SOIL WERE BASED ON THE RESULTS OF THE UNDISTURBED BORINGS.

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

US ARMY CORPS OF ENGINEERS NEW ORLEANS DISTRICT

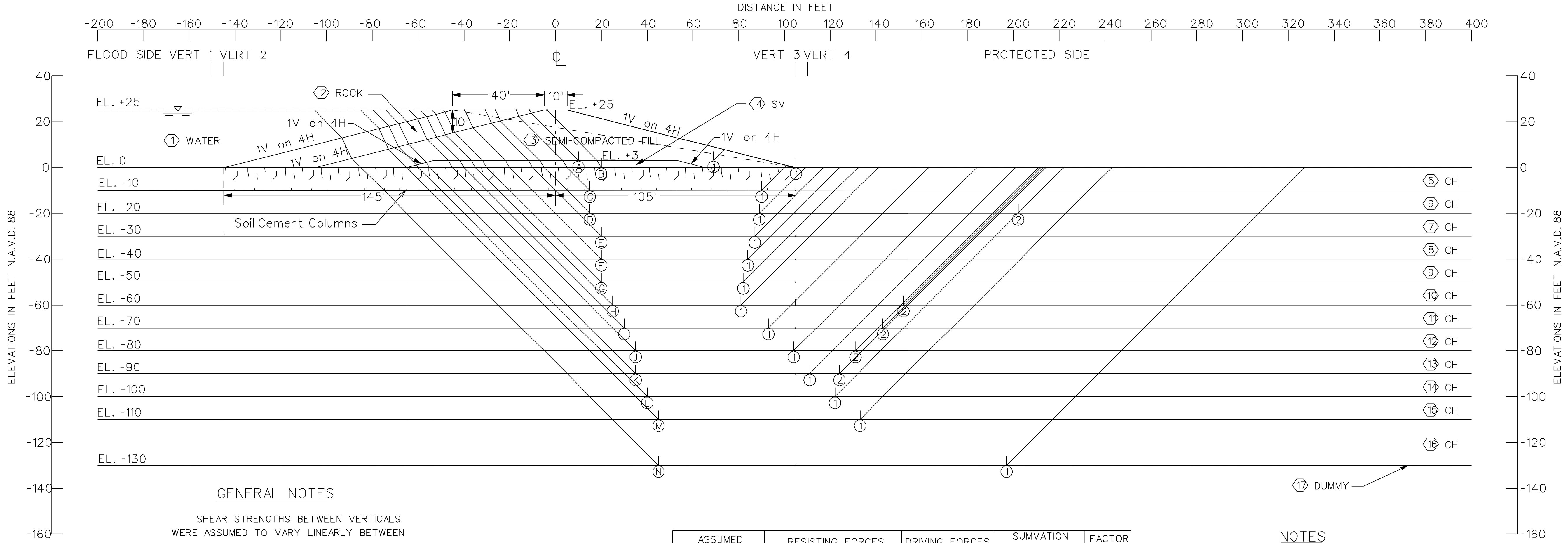
MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXX | DWN BY: XXX | CKD BY: XXX | DATE: MM/DD/YYYY | SOLICITATION NO.: W912P8-00-B-XXXX | SUBMITTED BY: XXXXXXXXXXXXXXXX | CONTRACT NO.: W912P8-00-C-XXXX | FILE NAME: XXXXXXXXXXXX-XXX-XXXXXX.DGN | FILE NUMBER: H-NN-NNNNN | SIZE: ANSI X | PLOT SCALE: 1"=5000' | PLOT DATE: MM/DD/YYYY

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 4 - SOIL CEMENT COLUMNS
F/S CROWN EL. +40 WATER EL. +0
LOUISIANA

SHEET IDENTIFICATION NUMBER
X



GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F.								FRICTION ANGLE DEGREES
						CENTER OF STRATUM				BOTTOM OF STRATUM				
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	WATER	62.5	62.5	62.5	62.5	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
3	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
6	CH	115	115	115	115	1050	1050	1050	1050	1100	1100	1100	1100	0
7	CH	115	115	115	115	1150	1150	1150	1150	1200	1200	1200	1200	0
8	CH	115	115	115	115	1250	1250	1250	1250	1300	1300	1300	1300	0
9	CH	115	115	115	115	1350	1350	1350	1350	1400	1400	1400	1400	0
10	CH	115	115	115	115	1450	1450	1450	1450	1500	1500	1500	1500	0
11	CH	115	115	115	115	1550	1550	1550	1550	1600	1600	1600	1600	0
12	CH	115	115	115	115	1650	1650	1650	1650	1700	1700	1700	1700	0
13	CH	115	115	115	115	1750	1750	1750	1750	1800	1800	1800	1800	0
14	CH	115	115	115	115	1850	1850	1850	1850	1900	1900	1900	1900	0
15	CH	115	115	115	115	1950	1950	1950	1950	2000	2000	2000	2000	0
16	CH	115	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0
17	DUMMY	115	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0

ASSUMED FAILURE SURFACE			RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.		ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
A	1	3.0	16581	23600	3839	26381	1583	44020	24798	1.78
B	1	.0	20426	36433	0	30836	0	56859	30836	1.84
C	1	-10.0	59085	82500	40799	66723	8216	182384	58507	3.12
D	1	-20.0	79019	81400	61000	111962	25466	221419	86496	2.56
D	2	-20.0	79019	205700	24999	111962	19999	309718	91963	3.37
E	1	-30.0	101529	80400	48000	166763	53800	229929	112963	2.04
F	1	-40.0	125415	83200	73000	235725	93657	281615	142068	1.98
G	1	-50.0	151109	86800	100000	316418	144615	337909	171803	1.97
H	1	-60.0	179238	84000	129000	404812	206510	392238	198302	1.98
H	2	-60.0	179238	190500	128999	404812	195994	498737	208818	2.39
I	1	-70.0	209389	100800	160000	503766	272020	470189	231746	2.03
I	2	-70.0	209389	180800	160000	503766	268742	550189	235024	2.34
J	1	-80.0	241528	117300	193000	613281	353004	551828	260277	2.12
J	2	-80.0	241528	163200	192999	613281	352989	597727	260292	2.30
K	1	-90.0	274652	136800	228000	736966	448737	639452	288229	2.22
K	2	-90.0	274652	160200	228000	736966	448737	662852	288229	2.30
L	1	-100.0	310718	155800	265000	867794	555984	731518	311810	2.35
M	1	-110.0	348839	176000	303999	1009147	674730	828838	334417	2.48
N	1	-130.0	425638	304000	383999	1331810	946720	1113637	385090	2.89

NOTES

⬡ -- STRATUM NUMBER
○ -- WEDGE NUMBER
-- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
∇ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

FACTOR OF SAFETY $= \frac{R_A + R_B + R_P}{D_A - D_P}$

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

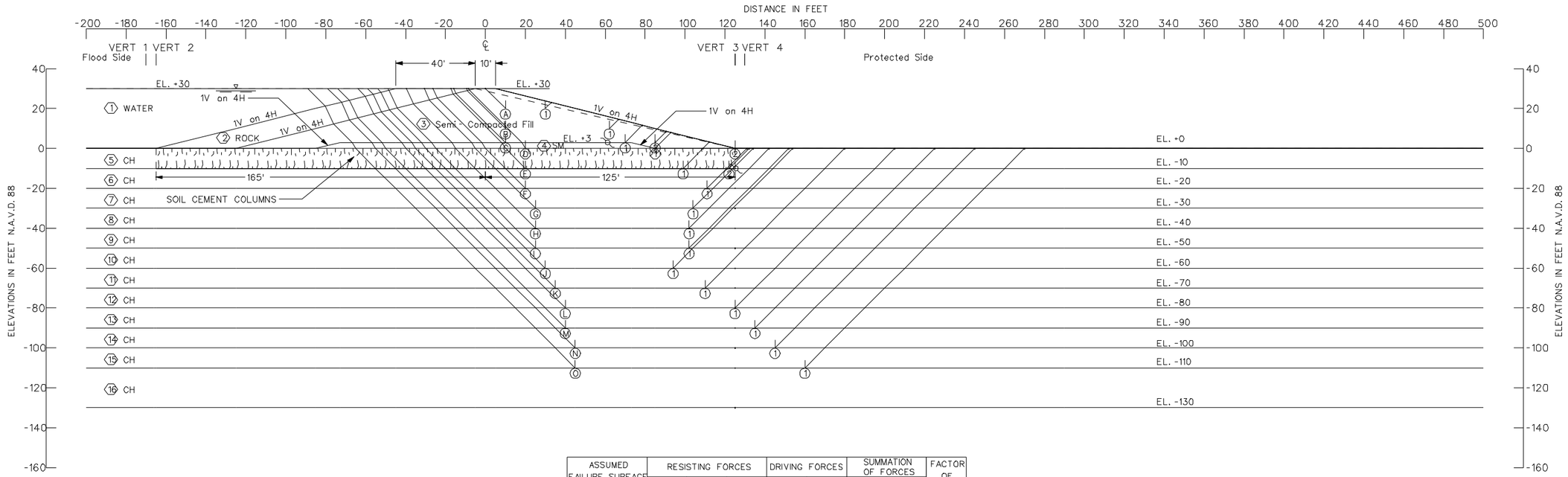
DESIGNED BY: XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXXXXXX
FILE NAME: XXXXXXXXXXXXX-XXX-XXXXXX.DGN
SIZE: ANSI X
PLOT SCALE: 1'=500'
DATE: MM/DD/YYYY
SOLICITATION NO.: W912P8-00-B-XXXX
CONTRACT NO.: W912P8-00-C-XXXX
FILE NUMBER: H-NN-NNNNN
PLOT DATE: MM/DD/YYYY

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

REACH 4 - SOIL CEMENT COLUMNS
P/S CROWN EL. +25 WATER EL. +25
LOUISIANA

SHEET IDENTIFICATION NUMBER
X



STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	WATER	62	62	62	62	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
3	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
6	CH	115	115	115	115	1050	1050	1050	1050	1100	1100	1100	1100	0
7	CH	115	115	115	115	1150	1150	1150	1150	1200	1200	1200	1200	0
8	CH	115	115	115	115	1250	1250	1250	1250	1300	1300	1300	1300	0
9	CH	115	115	115	115	1350	1350	1350	1350	1400	1400	1400	1400	0
10	CH	115	115	115	115	1450	1450	1450	1450	1500	1500	1500	1500	0
11	CH	115	115	115	115	1550	1550	1550	1550	1600	1600	1600	1600	0
12	CH	115	115	115	115	1650	1650	1650	1650	1700	1700	1700	1700	0
13	CH	115	115	115	115	1750	1750	1750	1750	1800	1800	1800	1800	0
14	CH	115	115	115	115	1850	1850	1850	1850	1900	1900	1900	1900	0
15	CH	115	115	115	115	1950	1950	1950	1950	2000	2000	2000	2000	0
16	CH	115	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0

ASSUMED FAILURE SURFACE		RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY
NO.	ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING	
A 1	20.0	7999	8000	2398	5154	618	18397	4536	4.06
B 1	10.0	15227	20800	3679	21708	1454	39706	20254	1.96
C 1	3.0	19837	23701	6879	40065	5083	50417	34982	1.44
C 2	3.0	19837	28927	4479	40065	2155	53243	37910	1.40
D 1	.0	23680	36796	6399	45991	4398	66875	41593	1.61
D 2	.0	23680	42834	0	45991	0	66514	45991	1.45
E 1	-10.0	62532	86900	42560	85483	11877	191992	73606	2.61
E 2	-10.0	62532	112200	4000	85483	4874	178732	80609	2.22
F 1	-20.0	82279	100100	25011	136360	23994	207390	112366	1.85
G 1	-30.0	104655	94800	48000	195944	55411	247455	140533	1.76
H 1	-40.0	128898	100100	73000	270406	94870	301998	175536	1.72
I 1	-50.0	155196	107800	100000	356598	144619	362996	211979	1.71
J 1	-60.0	183463	96000	129000	449795	213106	408463	236689	1.73
K 1	-70.0	213714	120000	160000	553557	274436	493714	279121	1.77
L 1	-80.0	245966	144500	193000	667891	352992	583466	314899	1.85
M 1	-90.0	279264	171000	228000	797240	448739	678264	348501	1.95
N 1	-100.0	315341	190000	265000	933053	555987	770341	377066	2.04
O 1	-110.0	352494	230000	304001	1083871	674734	886495	409137	2.17

NOTES

○ -- STRATUM NUMBER
○ -- WEDGE NUMBER
-- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
▽ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT, REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

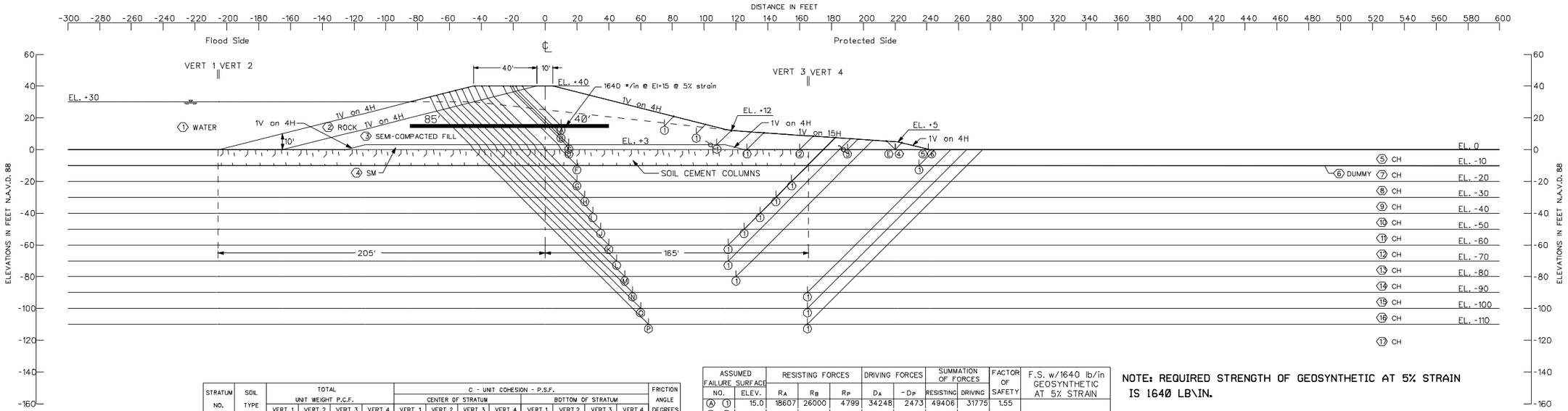
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

ISSUED BY:
XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY:
XXXXXXXXXXXXXXXXXX
FILE NAME:
XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
PROJ X FOOM

DATE:
MM/DD/YYYY
SCALE:
AS SHOWN
SOLICITATION NO.:
XXXXXXXXXXXXXXXX
CONTRACT NO.:
XXXXXXXXXXXXXXXX
FILE NUMBER:
MM/DD/YYYY
PLOT SCALE: 1=1
PLOT DATE: MM/DD/YYYY

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 4 - SOIL CEMENT COLUMNS
P/S CROWN EL.+30 WATER EL.+30
LOUISIANA

SHEET IDENTIFICATION NUMBER
X



GENERAL NOTES

SHEAR STRENGTHS BETWEEN VERTICALS WERE ASSUMED TO VARY LINEARLY BETWEEN THE VALUES INDICATED FOR THESE LOCATIONS.

STRATUM NO.	SOIL TYPE	TOTAL UNIT WEIGHT P.C.F.				C - UNIT COHESION - P.S.F. CENTER OF STRATUM				C - UNIT COHESION - P.S.F. BOTTOM OF STRATUM				FRICTION ANGLE DEGREES
		VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	VERT. 1	VERT. 2	VERT. 3	VERT. 4	
1	WATER	62.5	62.5	62.5	62.5	0	0	0	0	0	0	0	0	0
2	ROCK	132	132	132	132	0	0	0	0	0	0	0	0	40
3	CH	110	110	110	110	400	400	400	400	400	400	400	400	0
4	SM	122	122	122	122	0	0	0	0	0	0	0	0	30
5	CH	95	108	108	95	200	2000	2000	200	200	2000	2000	200	0
6	DUMMY	115	115	115	115	1050	1050	1050	1050	1050	1050	1050	1050	0
7	CH	115	115	115	115	1050	1050	1050	1050	1100	1100	1100	1100	0
8	CH	115	115	115	115	1150	1150	1150	1150	1200	1200	1200	1200	0
9	CH	115	115	115	115	1250	1250	1250	1250	1300	1300	1300	1300	0
10	CH	115	115	115	115	1350	1350	1350	1350	1400	1400	1400	1400	0
11	CH	115	115	115	115	1450	1450	1450	1450	1500	1500	1500	1500	0
12	CH	115	115	115	115	1550	1550	1550	1550	1600	1600	1600	1600	0
13	CH	115	115	115	115	1650	1650	1650	1650	1700	1700	1700	1700	0
14	CH	115	115	115	115	1750	1750	1750	1750	1800	1800	1800	1800	0
15	CH	115	115	115	115	1850	1850	1850	1850	1900	1900	1900	1900	0
16	CH	115	115	115	115	1950	1950	1950	1950	2000	2000	2000	2000	0
17	CH	115	115	115	115	2000	2000	2000	2000	2000	2000	2000	2000	0

ASSUMED FAILURE SURFACE			RESISTING FORCES			DRIVING FORCES		SUMMATION OF FORCES		FACTOR OF SAFETY	F.S. w/1640 lb/in GEOSYNTHETIC AT 5% STRAIN
NO.		ELEV.	R_A	R_B	R_P	D_A	$-D_P$	RESISTING	DRIVING		
Ⓐ	①	15.0	18607	26000	4799	34248	2473	49406	31775	1.55	
Ⓑ	①	10.0	22065	34000	4799	49648	2473	60864	47175	1.29	1.71
Ⓒ	①	3.0	27477	37109	7200	74551	5567	71786	68984	1.04	1.32
Ⓓ	①	.0	32590	43934	8500	87126	6622	85024	80504	1.06	1.30
Ⓓ	②	.0	32590	53194	6850	87126	4301	92634	82825	1.12	1.36
Ⓓ	③	.0	32590	59884	5349	87126	2623	97823	84503	1.16	1.39
Ⓓ	④	.0	32590	64927	3519	87126	1290	101036	85836	1.18	1.41
Ⓓ	⑤	.0	32590	66571	160	87126	3	99321	87123	1.14	1.36
Ⓔ	⑥	.0	4398	1644	160	1552	3	6202	1549	4.00	
Ⓕ	①	-10.0	72434	166897	4000	135806	5423	243331	130383	1.87	
Ⓖ	①	-20.0	93110	148500	31099	198122	44068	272709	154054	1.77	
Ⓗ	①	-30.0	115999	144000	54099	268927	82896	314098	186031	1.69	
Ⓘ	①	-40.0	141036	136500	79100	350681	133958	356636	216723	1.65	
Ⓙ	①	-50.0	167867	126000	106099	442962	197252	399966	245710	1.63	
Ⓚ	①	-60.0	196068	112500	135100	545099	272970	443668	272129	1.63	
Ⓛ	①	-70.0	226193	112000	165599	657480	353760	503792	303720	1.66	
Ⓜ	①	-80.0	258269	119000	197849	780120	442049	575118	338071	1.70	
Ⓝ	①	-90.0	292296	198000	228000	913025	497491	718296	415534	1.73	
Ⓞ	①	-100.0	328275	199500	265000	1056192	604738	792775	451454	1.76	
Ⓟ	①	-110.0	366159	200000	304001	1209581	723485	870160	486096	1.79	

NOTE: REQUIRED STRENGTH OF GEOSYNTHETIC AT 5% STRAIN IS 1640 LB\IN.

GEOTEXTILE TENSILE STRENGTH CALCULATIONS

CONTROLLING FAILURE WEDGE Ⓓ - ①

T = DRIVING(F.S.) - RESISTING
T = (88504X1.3) - 85024 = 19631LB/FT

NOTES

○ -- STRATUM NUMBER
○ -- WEDGE NUMBER
⌀ -- CROSSOVER POINT
ϕ -- ANGLE OF INTERNAL FRICTION, DEGREES
C -- UNIT COHESION, P.S.F.
▽ -- STATIC WATER SURFACE
D -- HORIZONTAL DRIVING FORCE IN POUNDS
R -- HORIZONTAL RESISTING FORCE IN POUNDS
A -- AS A SUBSCRIPT,REFERS TO ACTIVE WEDGE
B -- AS A SUBSCRIPT REFERS TO CENTRAL BLOCK
P -- AS A SUBSCRIPT REFERS TO PASSIVE WEDGE

$$\text{FACTOR OF SAFETY} = \frac{R_A + R_B + R_P}{D_A - D_P}$$

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

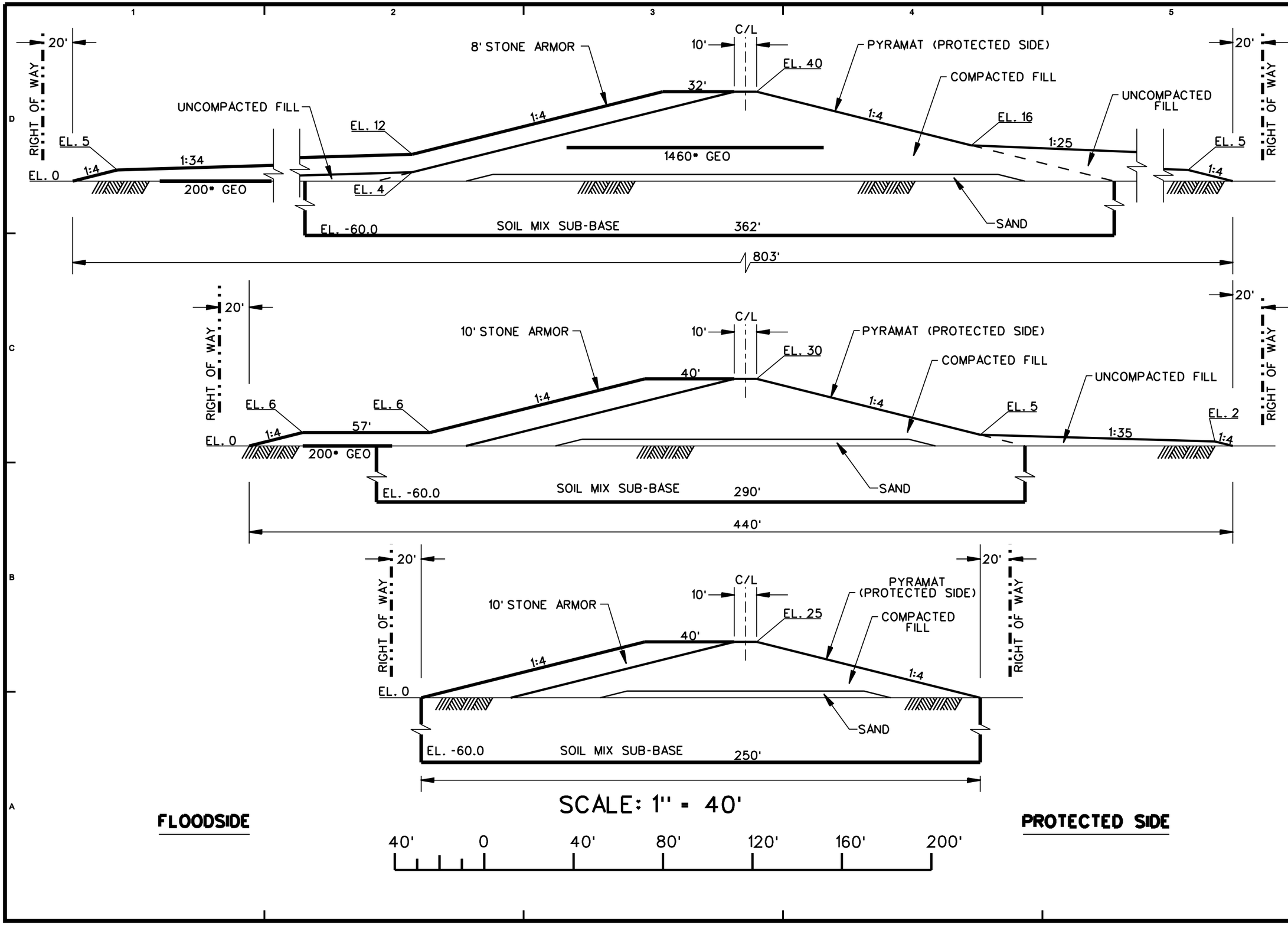
MARK	DATE	DESCRIPTION	BY	CHK

MARK	DATE	DESCRIPTION	BY	CHK

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS
REACH 4 - SOIL CEMENT COLUMNS
F/S CROWN EL.+40 WATER EL.+30
LAFOURCHE

SHEET IDENTIFICATION NUMBER
X



US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXXX | DATE: MM/DD/YYYY
DWN BY: XXXX | SPECIFICATION: W912PL-00-B-XXXX
CKD BY: XXXX | CONTRACT NO.: W912PL-00-C-XXXX
SUBMITTED BY: XXXX | FILE NUMBER: H-NN-NNNNN
FILE NAME: XXXXXXXXXXXXXXXXXXXX.DGN
SIZE: ANSI X | PLOT SCALE: 1"=20.00' | PLOT DATE: MM/DD/YYYY

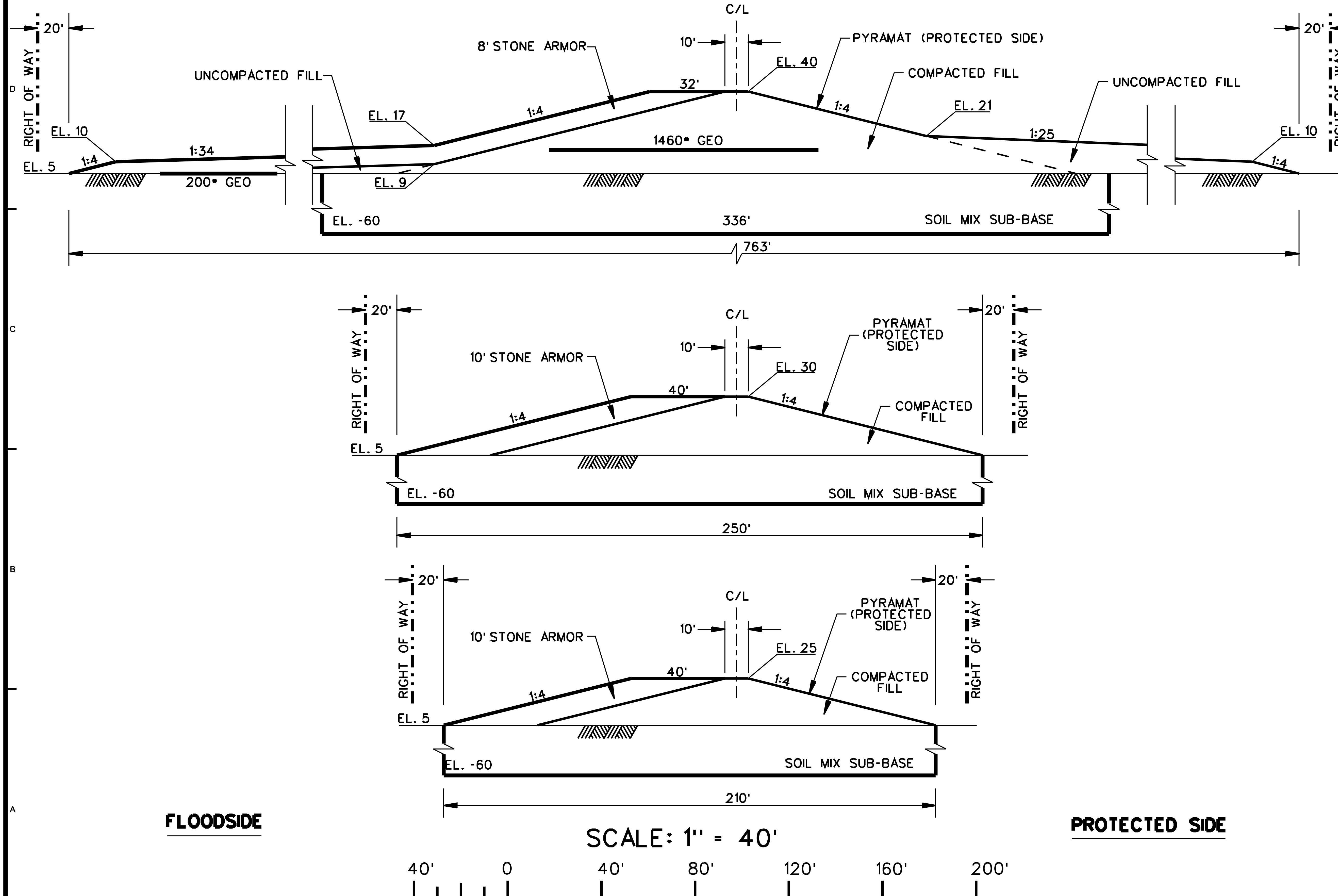
Louisiana Coastal Protection and Restoration Project
Hurricane Katrina, Report to Congress

REACH 1 TYPICAL SECTIONS

GROUND EL. 0.0 - SOIL MIX

LOUISIANA

SHEET IDENTIFICATION NUMBER
L-1



US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

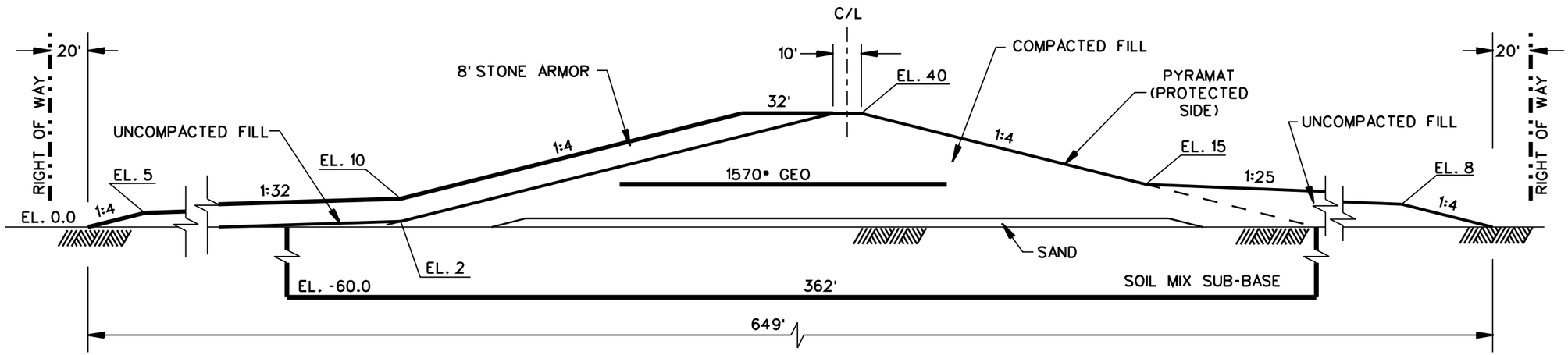
MARK	DESCRIPTION	DATE	APPR.

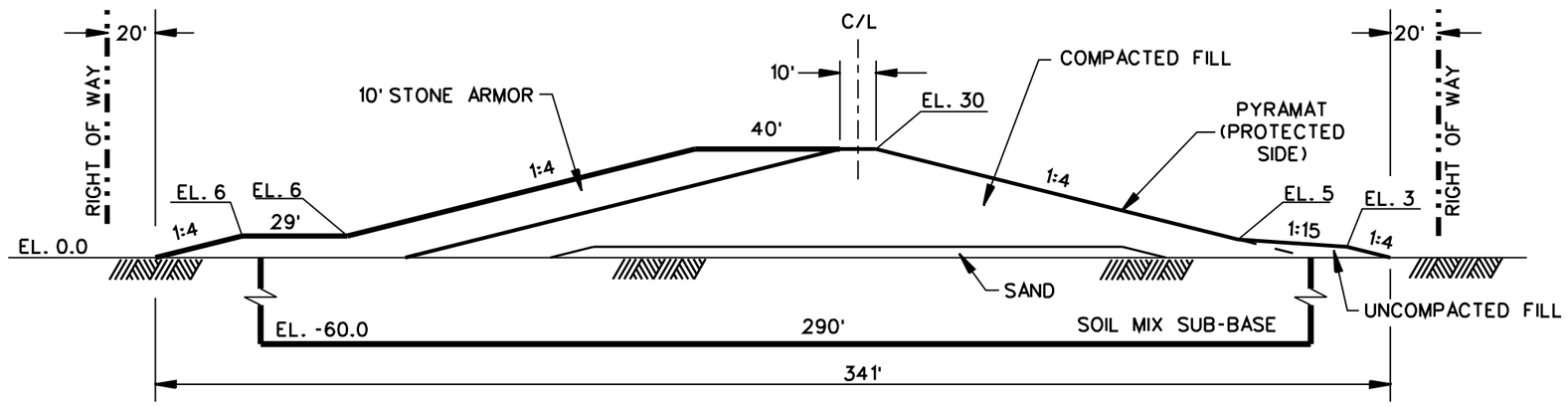
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

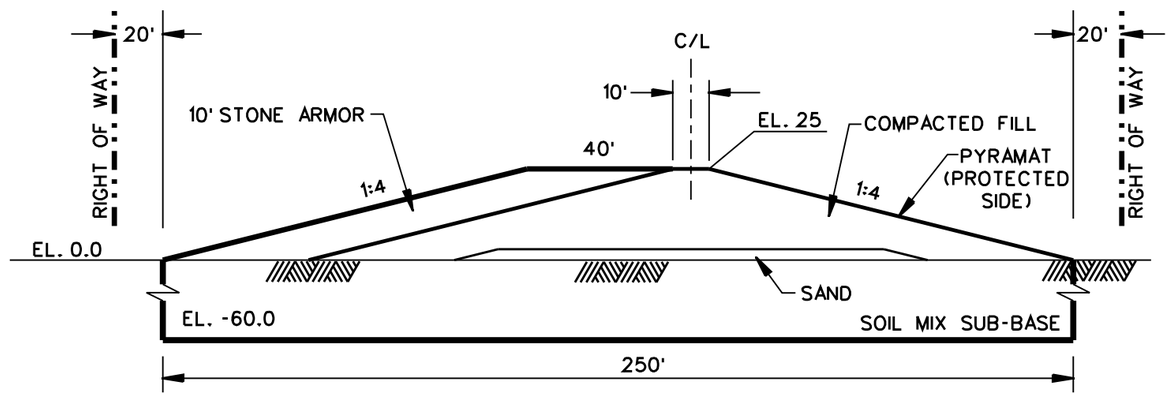
DESIGNED BY: XXX		DATE: MM/DD/YYYY
DWN BY: XXX	CKD BY: XXX	SOLICITATION NO.: W912P8-00-B-XXXX
SUBMITTED BY: XXXXXXXXXXXXXXXXX		CONTRACT NO.: W912P8-00-C-XXXX
FILE NAME: XXXXXXXXXXXX-XXX-XXXXXXX.DGN		FILE NUMBER: H-NN-NNNNN
SIZE: ANSI X	PLOT SCALE: 1"=5000'	PLOT DATE: MM/DD/YYYY

REACH 1 TYPICAL SECTIONS
GROUND EL. 5.0 - SOIL MIX
LOUISIANA

SHEET IDENTIFICATION NUMBER
L-2







FLOODSIDE

PROTECTED SIDE

SCALE: 1" = 40'

40' 0 40' 80' 120' 160' 200'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

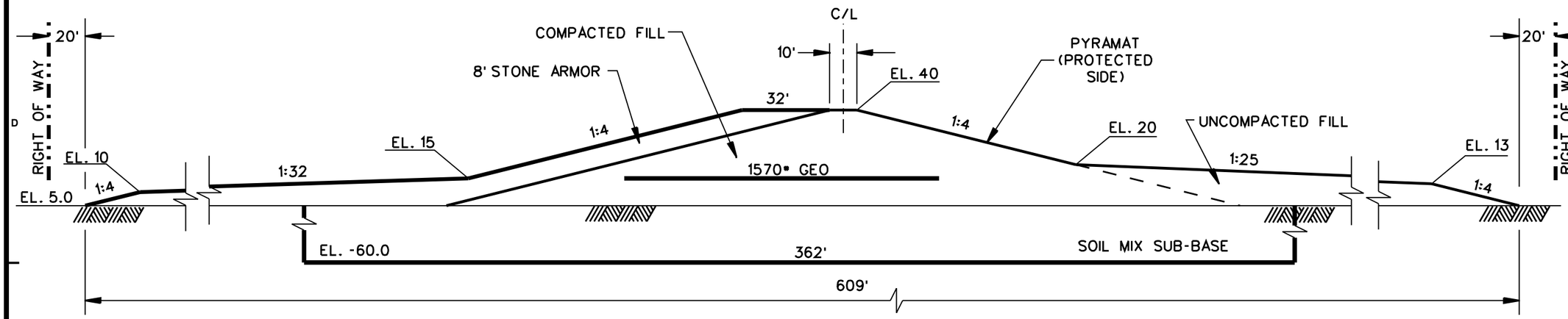
MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

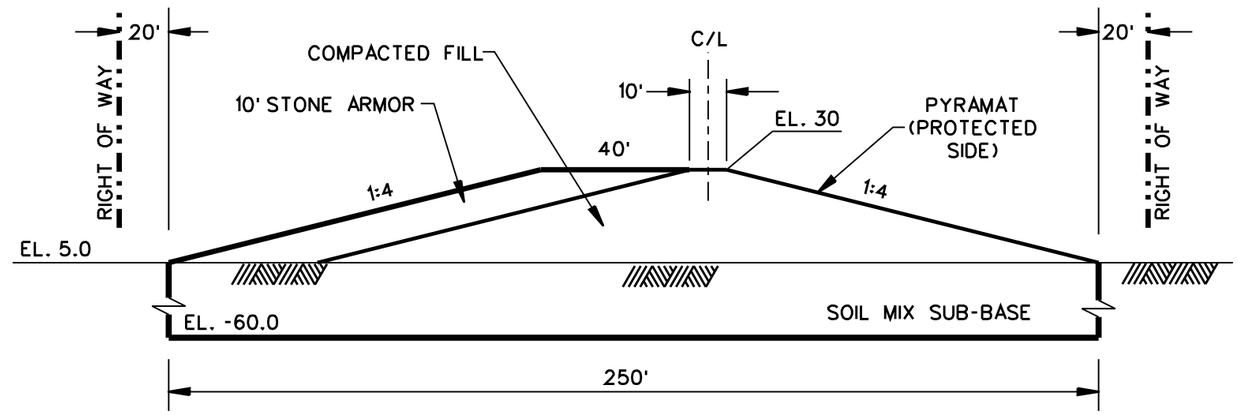
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

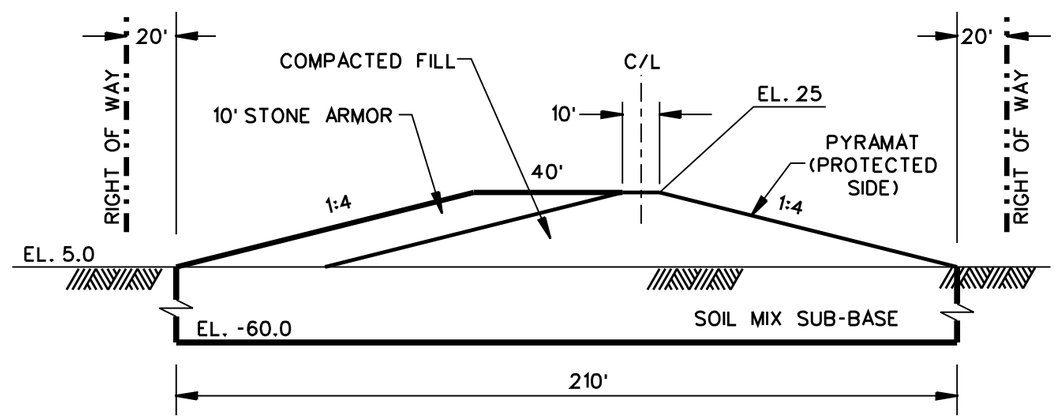
DESIGNED BY: XXX
DRAWN BY: XXX
CKD BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXXX
DATE: MM/DD/YYYY
SOLICITATION NO.: W912PB-XX-B-XXXX
CONTRACT NO.: W912PB-XX-C-XXXX
FILE NUMBER: H-N-NNNNN
FILE NAME: XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX.DGN
SIZE: ANSI X
PLOT SCALE: 1"=5000'
PLOT DATE: MM/DD/YYYY

REACH 2 TYPICAL SECTIONS
GROUND EL. 0.0 - SOIL MIX
LOUISIANA

SHEET IDENTIFICATION NUMBER
L-3







FLOODSIDE

PROTECTED SIDE

SCALE: 1" = 40'

40' 0 40' 80' 120' 160' 200'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

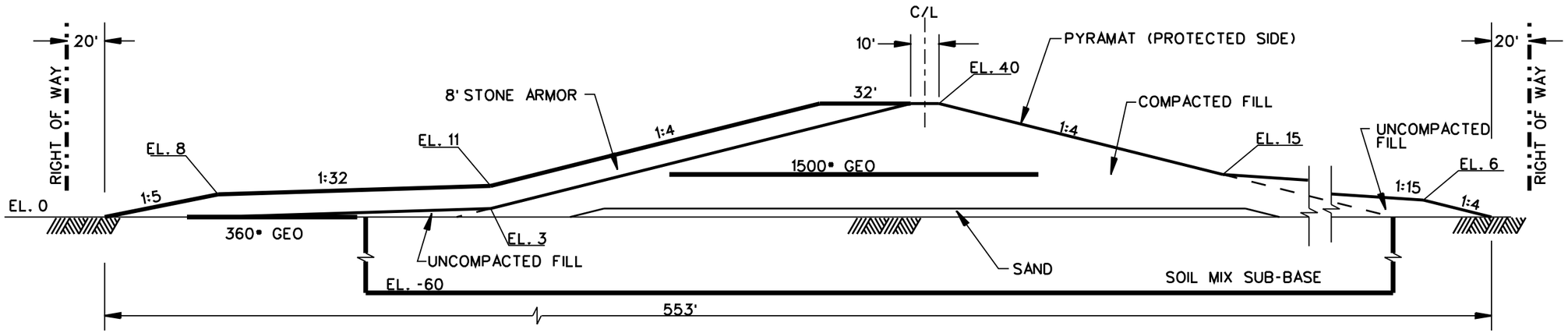
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

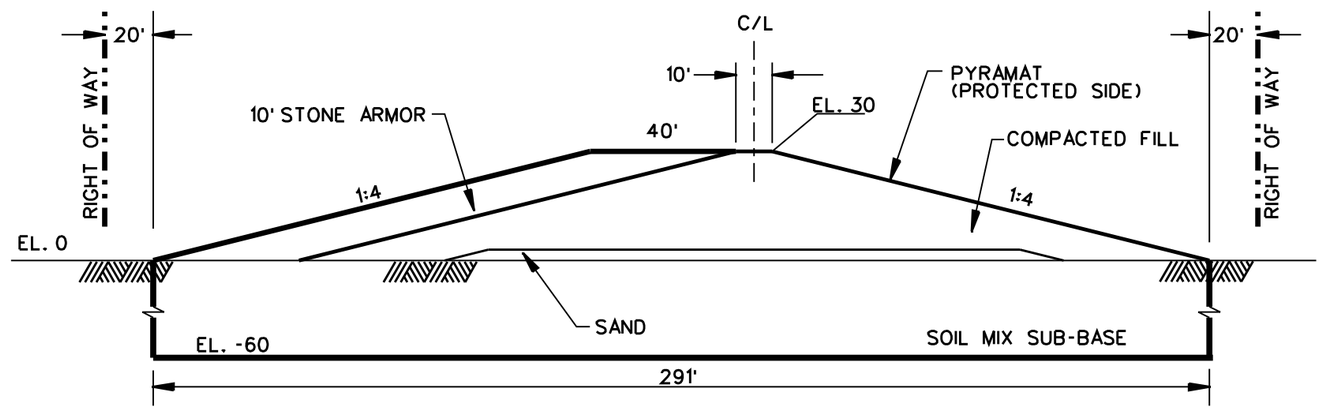
DESIGNED BY: XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXXXX
FILENAME: XXXXXXXXXXXXXXXXXXXX.DGN
SIZE: ANSI X
PLOT SCALE: 1"=5000'
DATE: MM/DD/YYYY
SOLICITATION NO.: W912P8-XX-B-XXXX
CONTRACT NO.: W912P8-XX-C-XXXX
FILE NUMBER: H-N-NNNNNN
PLOT DATE: MM/DD/YYYY

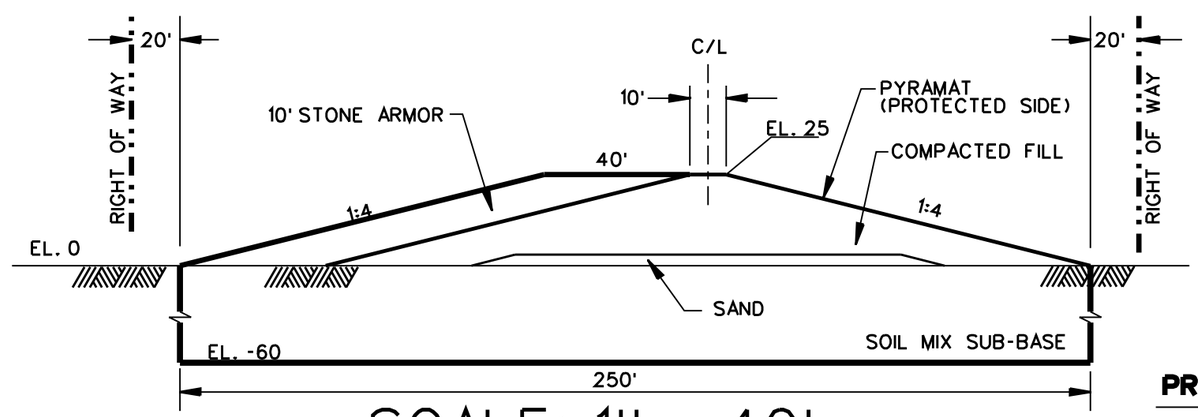
Uniform Coastal Protection and Restoration Project
Hollywood Beach, Broward Co Florida

REACH 2 TYPICAL SECTIONS
GROUND EL. 5.0 - SOIL MIX
LOUISIANA

SHEET IDENTIFICATION NUMBER
L-4







FLOODSIDE

PROTECTED SIDE

SCALE: 1" = 40'

40' 0 40' 80' 120' 160' 200'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

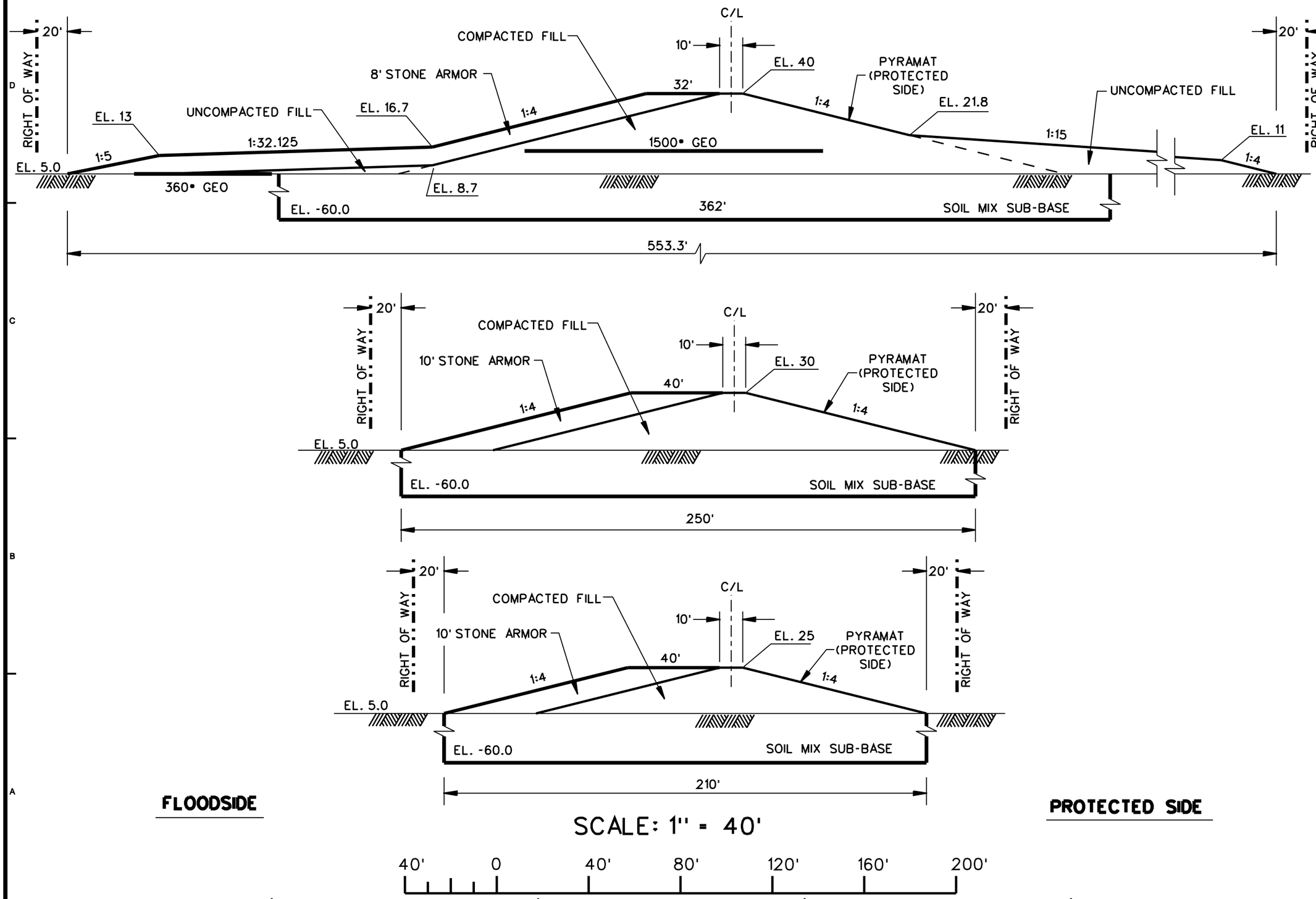
MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXX
DRAWN BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXXXX
FILE NAME: XXXXXXXXXXXXXXXXXXXXXXXXXXXX
SIZE: ANSI X
DATE: MM/DD/YYYY
SOLICITATION NO.: W912P8-XX-B-XXXX
CONTRACT NO.: W912P8-XX-C-XXXX
FILE NUMBER: H-NN-NNNNN
PLOT DATE: MM/DD/YYYY
SCALE: 1"=40'

REACH 3 TYPICAL SECTIONS
GROUND EL. 0 - SOIL MIX
LOUISIANA

SHEET IDENTIFICATION NUMBER
L-5



US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

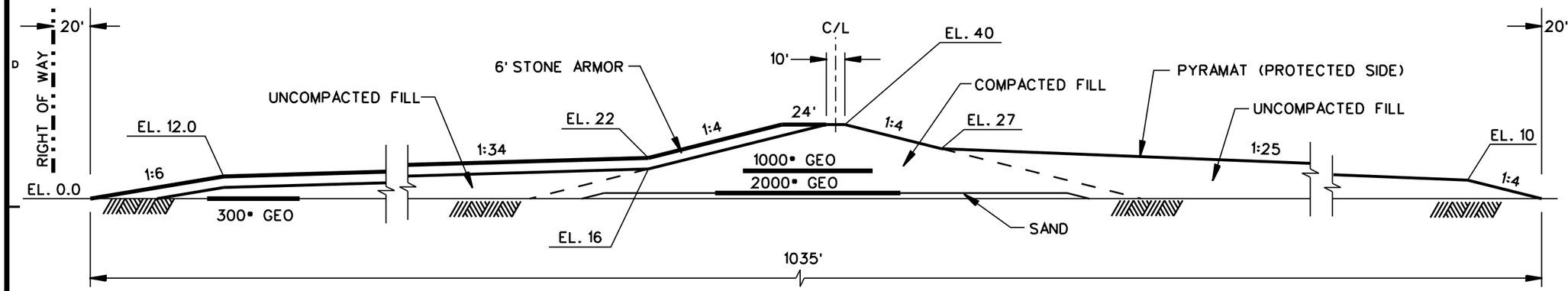
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

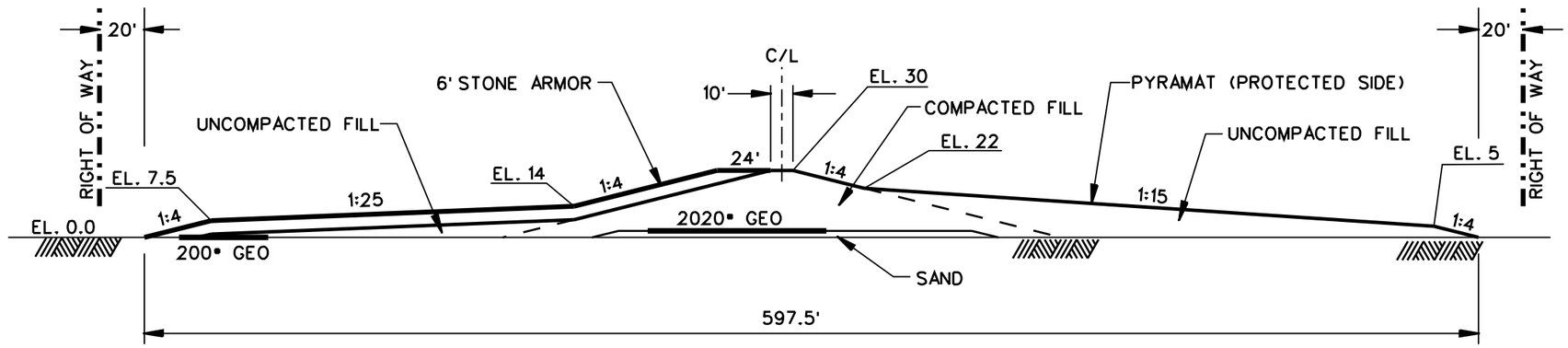
DESIGNED BY: XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXXXXXX
FILENAME: XXXXXXXXXXXXXXXXXXXX.DGN
SIZE: ANSI X
PLOT SCALE: 1"=5000'
DATE: MM/DD/YYYY
SOLICITATION NO.: W912P8-00-B-XXXX
CONTRACT NO.: W912P8-00-C-XXXX
FILE NUMBER: H.H.H.H.H.H.H.H.
PLOT DATE: MM/DD/YYYY

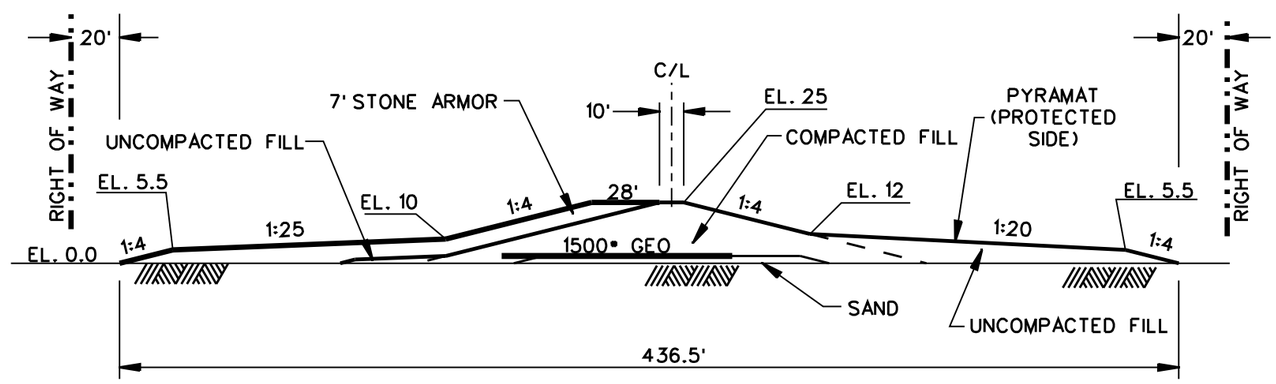
MARK | DESCRIPTION | DATE | APPR.
MARK | DESCRIPTION | DATE | APPR.

REACH 3 TYPICAL SECTIONS
GROUND EL. 5.0 - SOIL MIX
LOUISIANA

SHEET IDENTIFICATION NUMBER
L-6







FLOODSIDE

PROTECTED SIDE

SCALE: 1" = 60'

60' 0 60' 120' 180' 240' 300'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

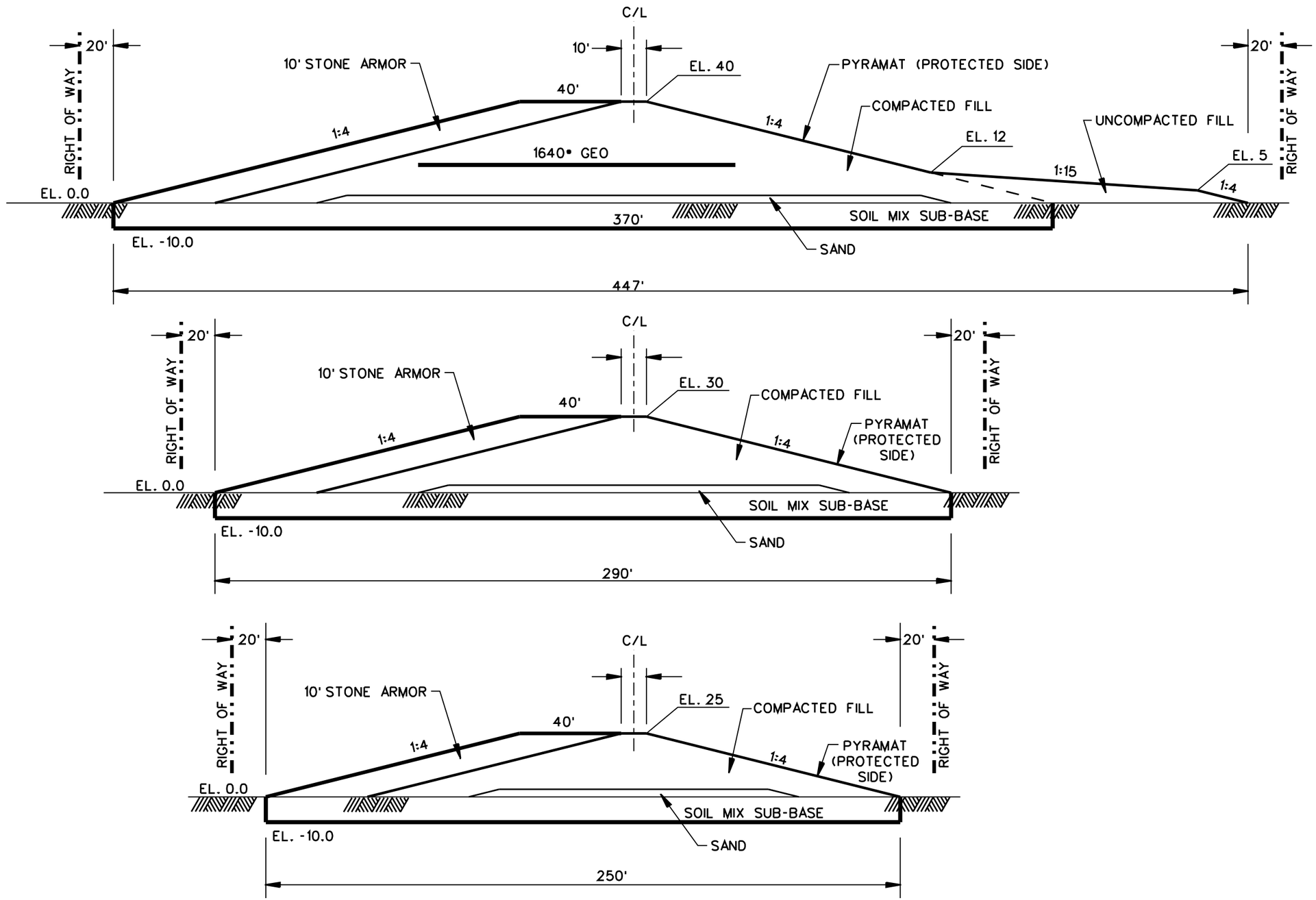
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXXX
DWN BY: XXXX
CKD BY: XXXX
SUBMITTED BY: XXXXXXXXXXXXXXXX
DATE: MM/DD/YYYY
SOLICITATION NO.: W912P8-09-B-XXXX
CONTRACT NO.: W912P8-09-C-XXXX
FILE NAME: XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
FILE NUMBER: H-N-N-N-N-N-N-N
SIZE: ANSI X
FOR SCALE: 1"=5000'
PLOT DATE: MM/DD/YYYY

HURRICANE CONSTITUTION PROJECT
PHILADELPHIA, MISSISSIPPI, NO CONTRACT

REACH 4 TYPICAL SECTIONS
GROUND EL. 0.0 - GEOTEXTILE
LOUISIANA

SHEET IDENTIFICATION NUMBER
L-7



SCALE: 1" = 40'

40' 0 40' 80' 120' 160' 200'

FLOODSIDE

PROTECTED SIDE

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

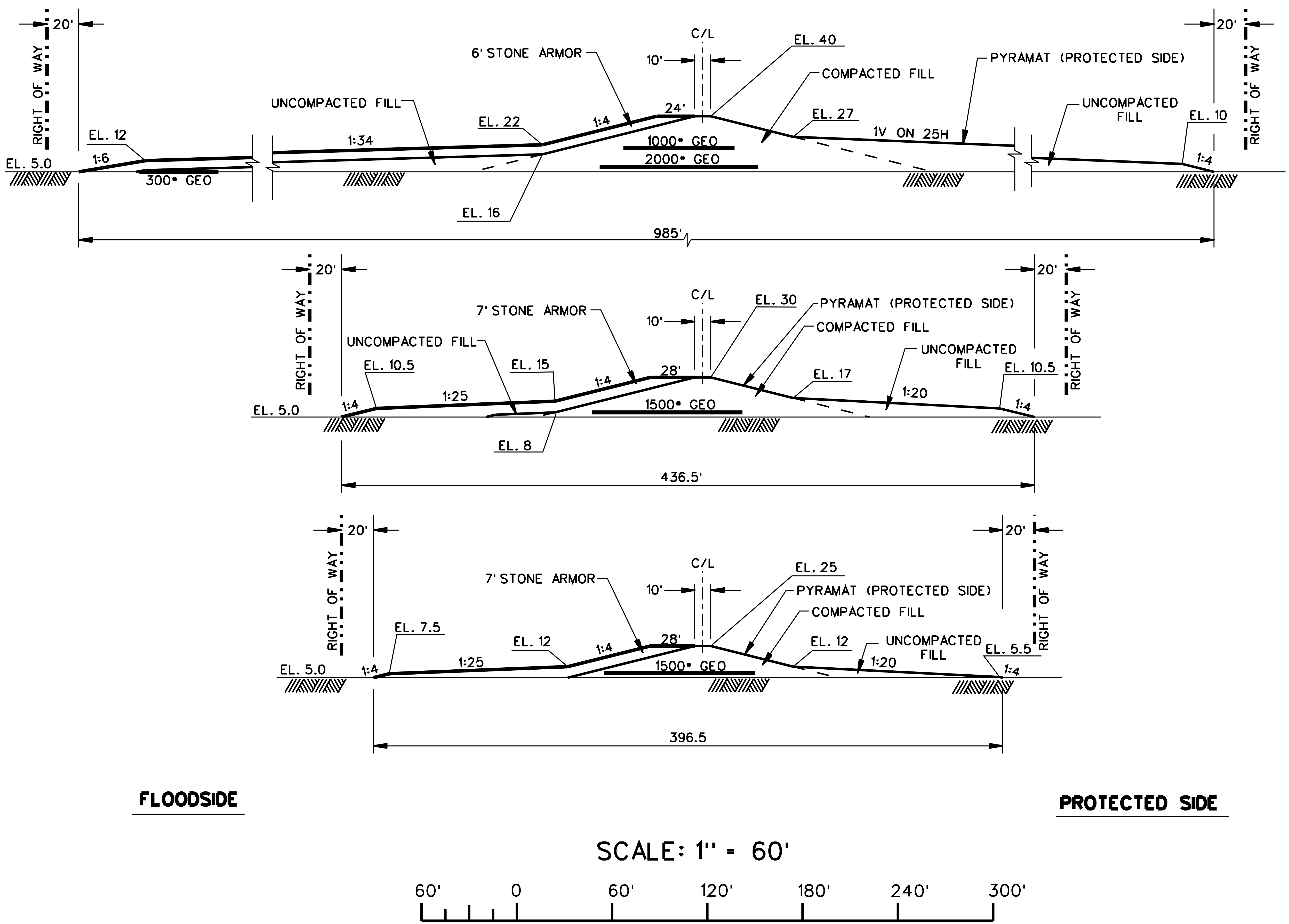
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXX | DATE: MM/DD/YYYY
DWN BY: XXX | CKD BY: XXX | SOLICITATION NO.: W912P8-04-B-XXXX
SUBMITTED BY: XXXXXXXXXXXXXXX | CONTRACT NO.: W912P8-04-C-XXXX
FILE NAME: XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX.DGN | FILE NUMBER: H-NN-NNNNN
SIZE: ANSI X | PLOT SCALE: 1"=50'0" | PLOT DATE: MM/DD/YYYY

Louisiana Coastal Protection and Restoration Project
Hurricane Protection Office, Report to Congress

REACH 4 TYPICAL SECTIONS
GROUND EL. 0.0 - SOIL MIX
LOUISIANA

SHEET IDENTIFICATION NUMBER
L-8



US ARMY CORPS
OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

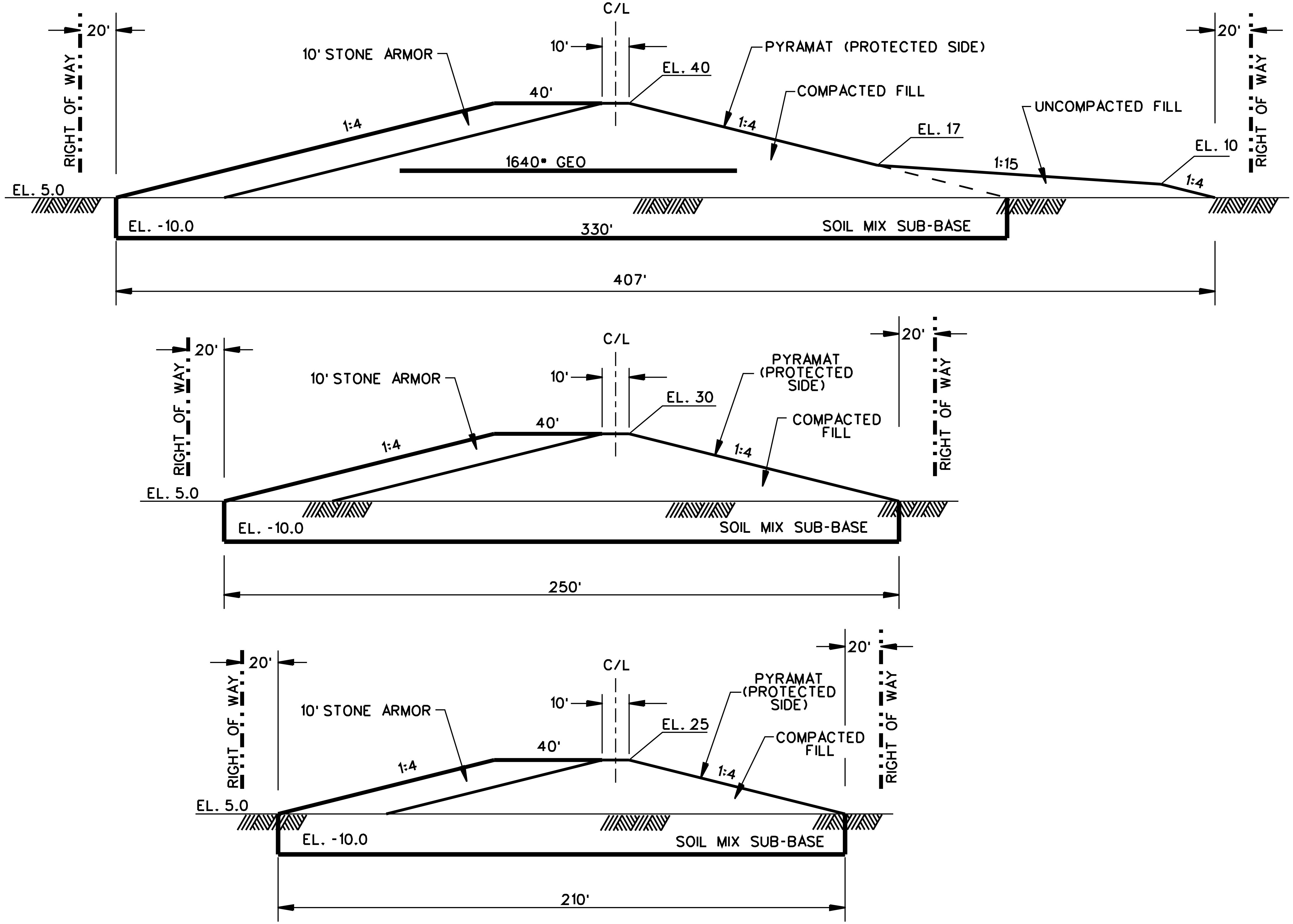
MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXX	DATE: MM/DD/YYYY	
DWN BY: XXX	CKD BY: XXX	SOLICITATION NO.: W912P8-00-B-XXXX
SUBMITTED BY: XXXXXXXXXXXXXXXX		CONTRACT NO.: W912P8-00-C-XXXX
FILE NAME: XXXXXXXXXX-XXX-XXXXXXX.DGN		FILE NUMBER: H-NN-NNNNN
SIZE: ANSI X	PLOT SCALE: 1"=5000'	PLOT DATE: MM/DD/YYYY

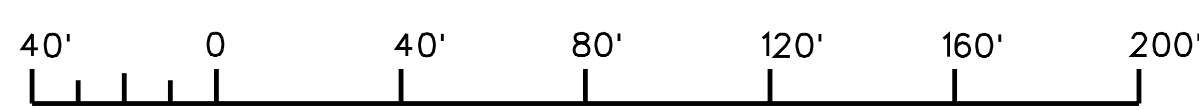
REACH 4 TYPICAL SECTIONS
GROUND EL. 5.0 - GEOTEXTILE
LOUISIANA

SHEET
IDENTIFICATION
NUMBER
L-9



FLOODSIDE

SCALE: 1" = 40'



PROTECTED SIDE

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

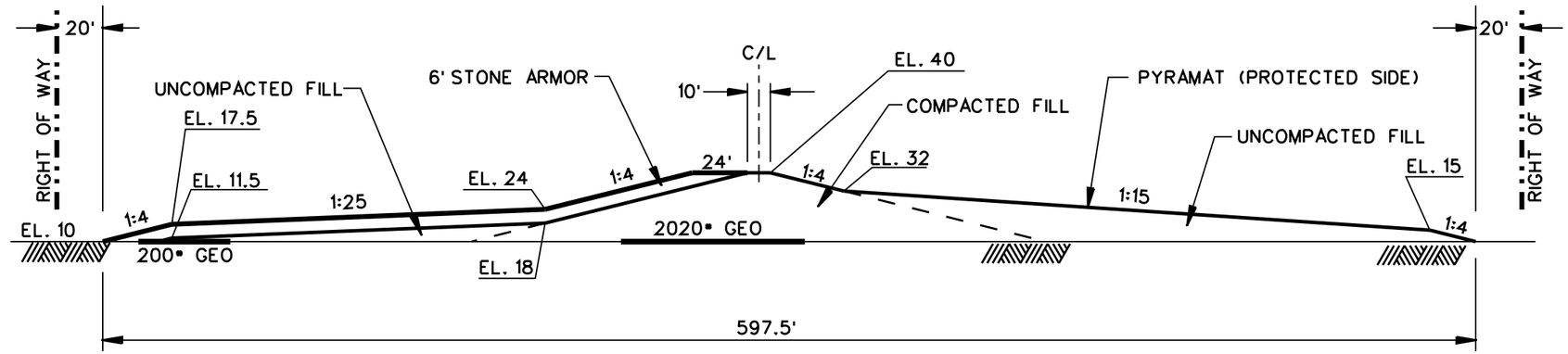
MARK	DESCRIPTION	DATE	APPR.

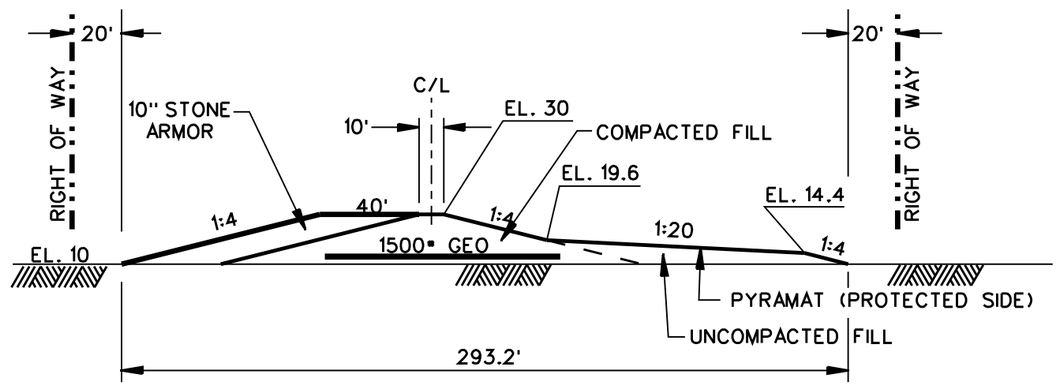
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

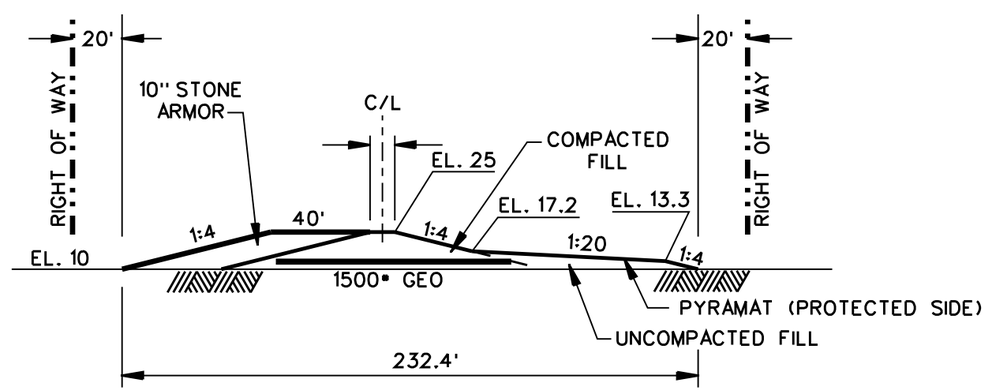
DESIGNED BY: XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXXXXX
FILE NAME: XXXXXXXXXXXX-XXX-XXXXXXX.DGN
SIZE: ANSI X
PLOT SCALE: 1"=5000'
DATE: MM/DD/YYYY
SOLICITATION NO.: W912P8-00-B-XXXX
CONTRACT NO.: W912P8-00-C-XXXX
FILE NUMBER: H-NN-NNNNN
PLOT DATE: MM/DD/YYYY

REACH 4 TYPICAL SECTIONS
GROUND EL. 5.0 - SOIL MIX
LOUISIANA

SHEET IDENTIFICATION NUMBER
L-10







FLOODSIDE

PROTECTED SIDE

SCALE: 1" = 60'

60' 0 60' 120' 180' 240' 300'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

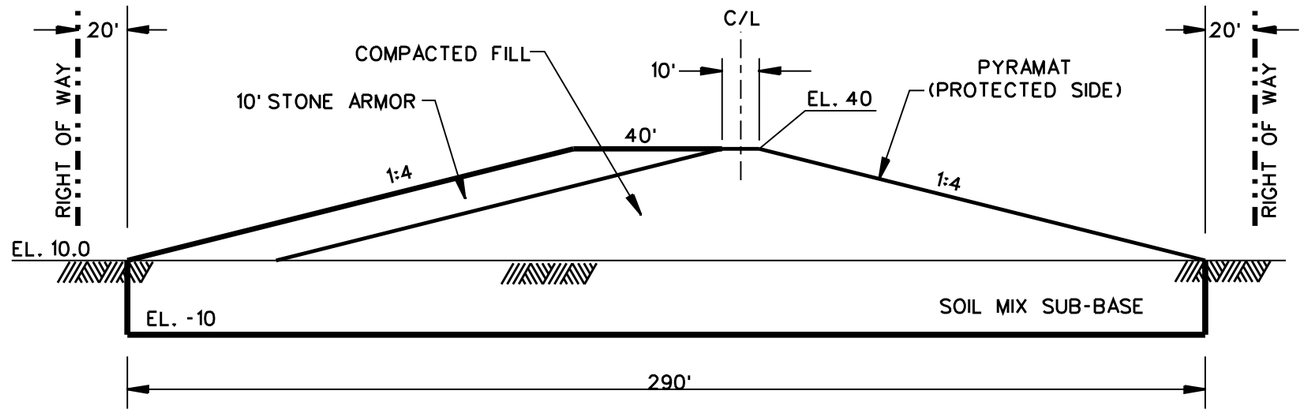
MARK	DESCRIPTION	DATE	APPR.

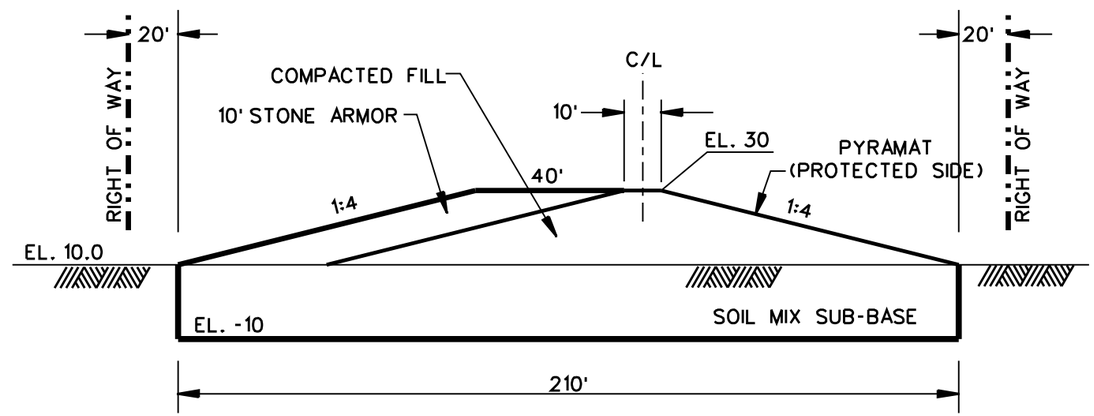
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

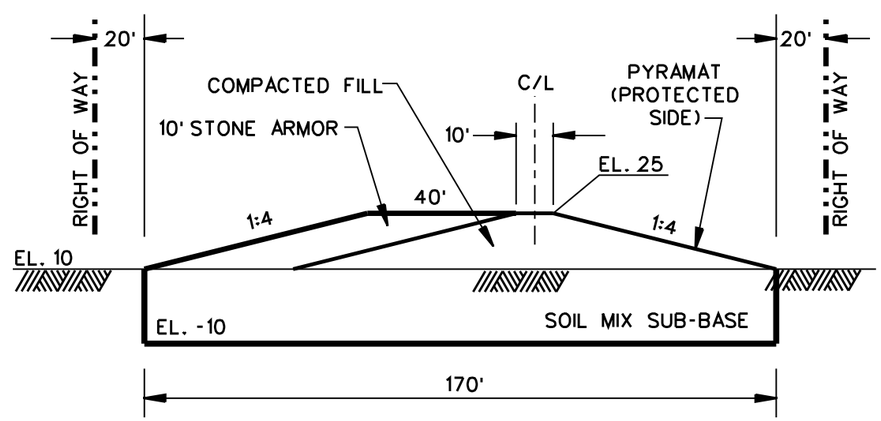
DESIGNED BY: XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXXXXXX
FILE NAME: XXXXXXXXXXXXXXXXXXXXXXXX.DGN
SIZE: ANSI X
PLOT SCALE: 1"=500'
DATE: MM/DD/YYYY
SOLICITATION NO.: W912P8-00-B-XXXX
CONTRACT NO.: W912P8-00-C-XXXX
FILE NUMBER: HH-NNNNNN
PLOT DATE: MM/DD/YYYY

REACH 4 TYPICAL SECTIONS
GROUND EL. 10.0 - GEOTEXTILE
LOUISIANA

SHEET IDENTIFICATION NUMBER
L-11







FLOODSIDE

PROTECTED SIDE

SCALE: 1" = 40'

40' 0 40' 80' 120' 160' 200'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

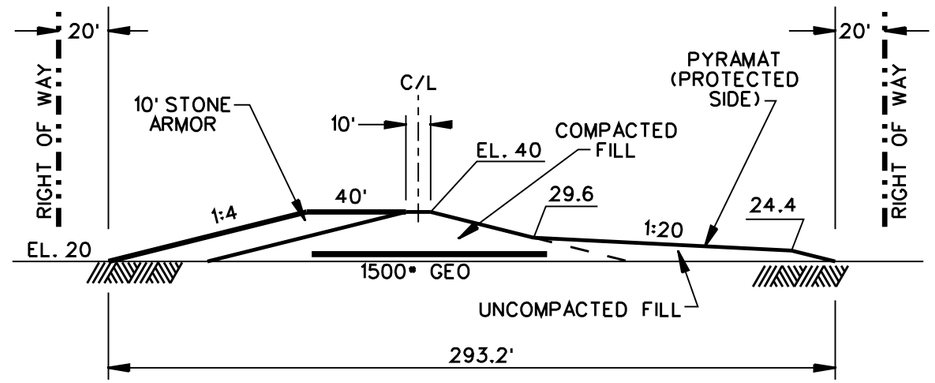
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

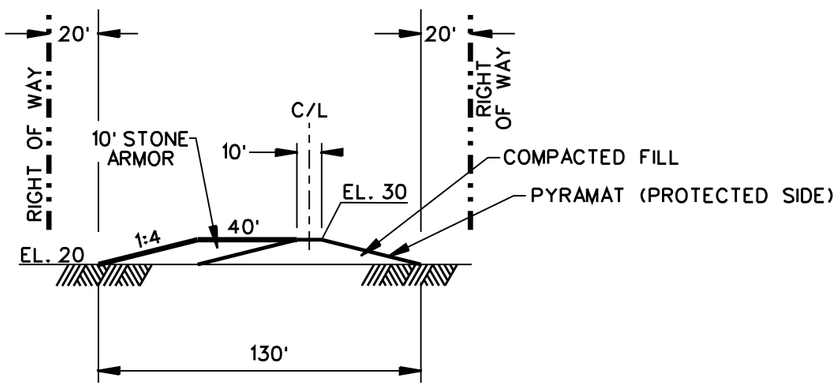
DESIGNED BY: XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXXXXXXXX
DATE: MM/DD/YYYY
SOLICITATION NO.: W912P8-09-B-XXXX
CONTRACT NO.: W912P8-09-C-XXXX
FILE NUMBER: HHH-NNNNN
FILE NAME: XXXXXXXXXXXXXXXXXXXXXXXX.DGN
SIZE: ANSI X
PLOT SCALE: 1"=5000'
PLOT DATE: MM/DD/YYYY

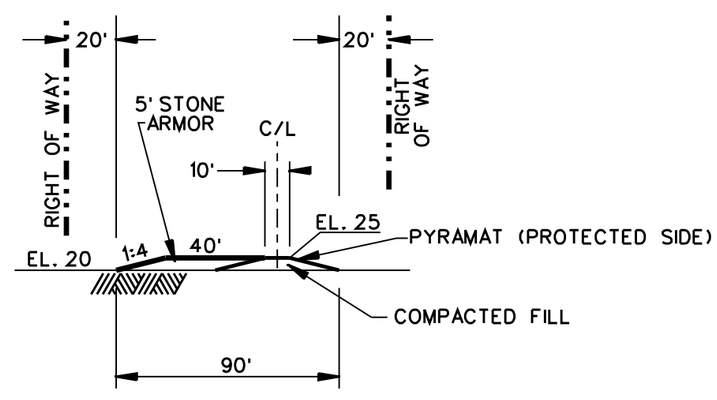
Hurricane Control, Protection and Restoration Project
Multiparties Hydrocarbon Report to CONGRESS

REACH 4 TYPICAL SECTIONS
GROUND EL. 10.0 - SOIL MIX
LOUISIANA

SHEET IDENTIFICATION NUMBER
L-12







FLOODSIDE

PROTECTED SIDE

SCALE: 1" = 60'

60' 0 60' 120' 180' 240' 300'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

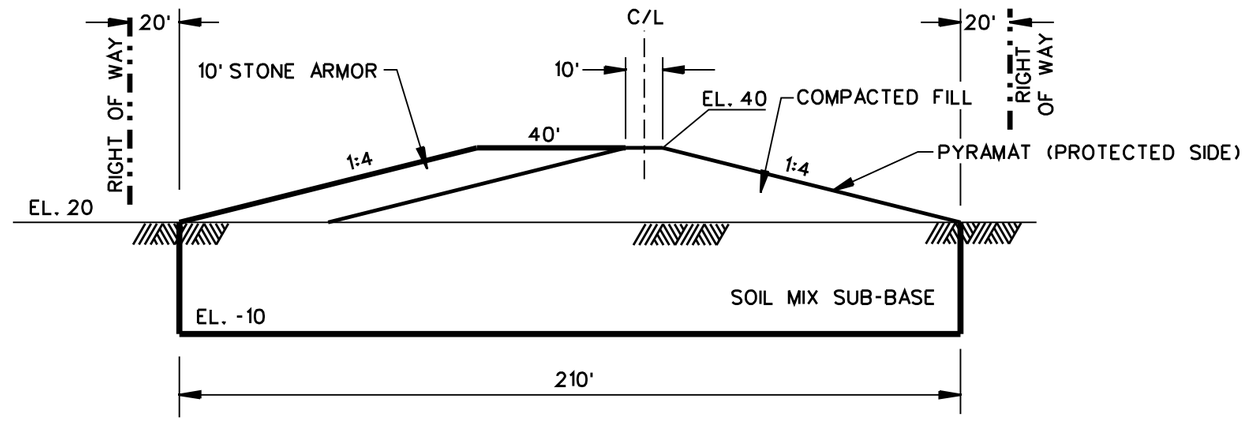
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

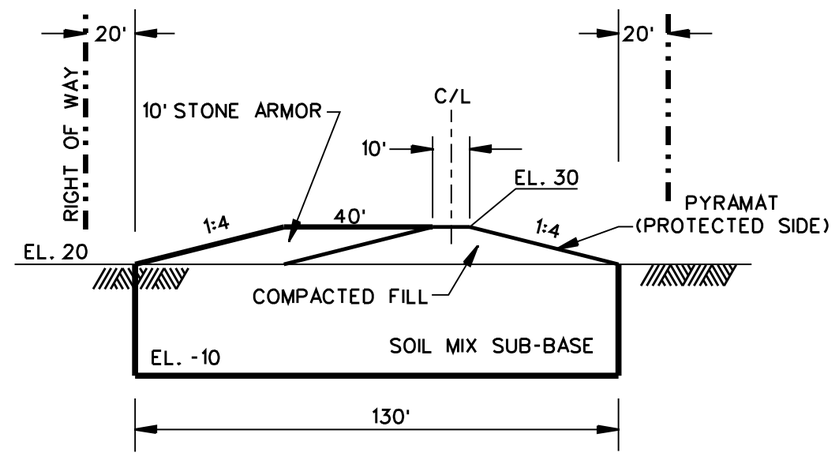
DESIGNED BY: XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXXXX
DATE: MM/DD/YYYY
SOLICITATION NO.: W912P8-10-B-XXXX
CONTRACT NO.: W912P8-10-C-XXXX
FILE NAME: XXXXXXXXXXXXXXXXXXXX.DGN
FILE NUMBER: H-H-H-H-H-H-H
SIZE: ANSI X
PLOT SCALE: 1"=5000'
PLOT DATE: MM/DD/YYYY

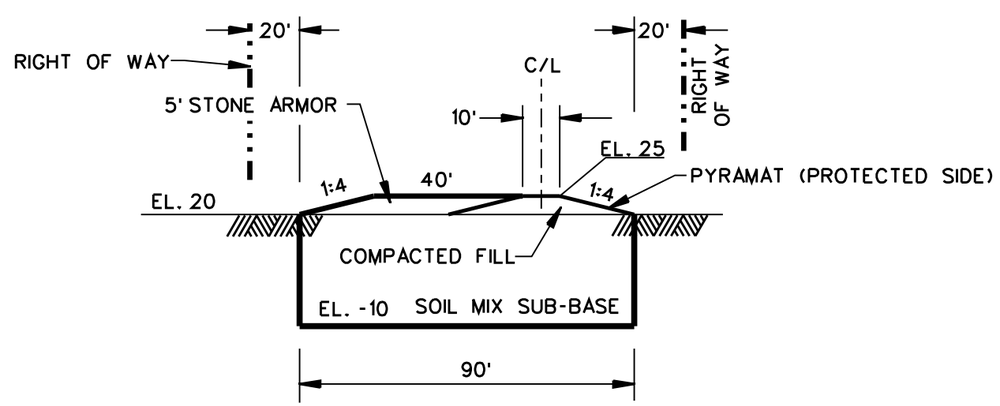
[illegible]

REACH 4 TYPICAL SECTIONS
GROUND EL. 20.0 - GEOTEXTILE
LOUISIANA

SHEET IDENTIFICATION NUMBER
L-13







FLOODSIDE

PROTECTED SIDE

SCALE: 1" = 40'

40' 0 40' 80' 120' 160' 200'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

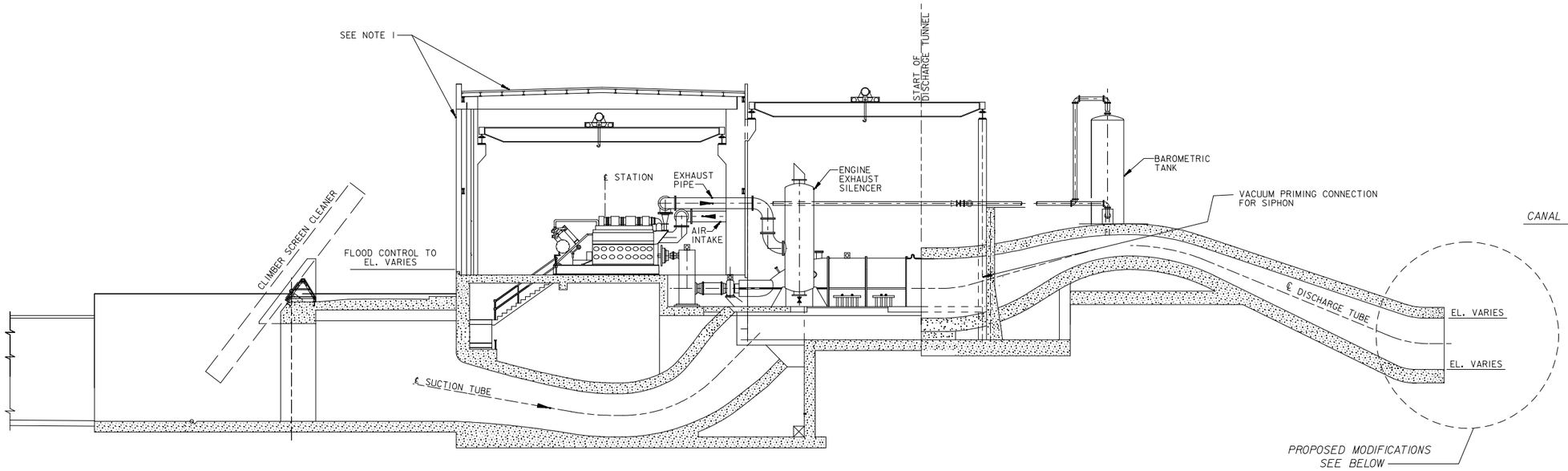
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: XXX
DWN BY: XXX
CKD BY: XXX
SUBMITTED BY: XXXXXXXXXXXXXXXX
FILENAME: XXXXXXXXXXXXXXXXXXXX.DGN
SIZE: ANSI X
PLOT SCALE: 1"=5000'
DATE: MM/DD/YYYY
SOLICITATION NO.: W912P8-00-B-XXXX
CONTRACT NO.: W912P8-00-C-XXXX
FILE NUMBER: H.H.H.H.H.H.H.H.
PLOT DATE: MM/DD/YYYY

Mississippi River and Tributaries Project
Feliciana Bend, Tensas Co. Louisiana

REACH 4 TYPICAL SECTIONS
GROUND EL. 20.0 - SOIL MIX
LOUISIANA

SHEET IDENTIFICATION NUMBER
L-14



EXISTING CONDITIONS

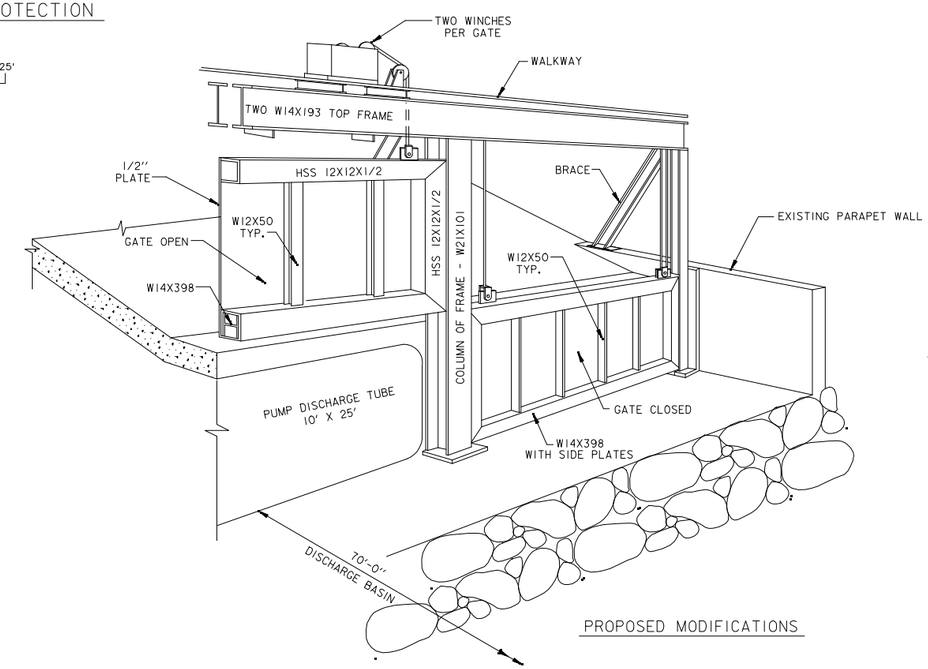
TYPICAL EXISTING HORIZONTAL PUMPING STATION WITH ADDED FRONTING PROTECTION

SCALE: 1/8" = 1'- 0"

0 5' 10' 15' 20' 25'

NOTES:

1. HARDEN BUILDING TO WITHSTAND HURRICANE FORCE WINDS.
2. PUMP DISCHARGE FLOOD PROTECTION WALL CONSIST OF TWO (2) STEEL FABRICATED CLOSURE GATES IN A STRUCTURAL STEEL GUIDED FRAME FOR A TYPICAL HORIZONTAL PUMP.
3. CLOSURE GATE WILL REMAIN NORMALLY IN THE UP POSITION. LOWERING WILL OCCUR DURING EXTREME WEATHER CONDITIONS.



PROPOSED MODIFICATIONS

US ARMY CORPS OF ENGINEERS

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: DATE:
DWN BY: CKD BY: SOLICITATION NO.:
SUBMITTED BY: CONTRACT NO.:
FILE NAME: FILE NUMBER:
SIZE: PLOT SCALE: PLOT DATE:

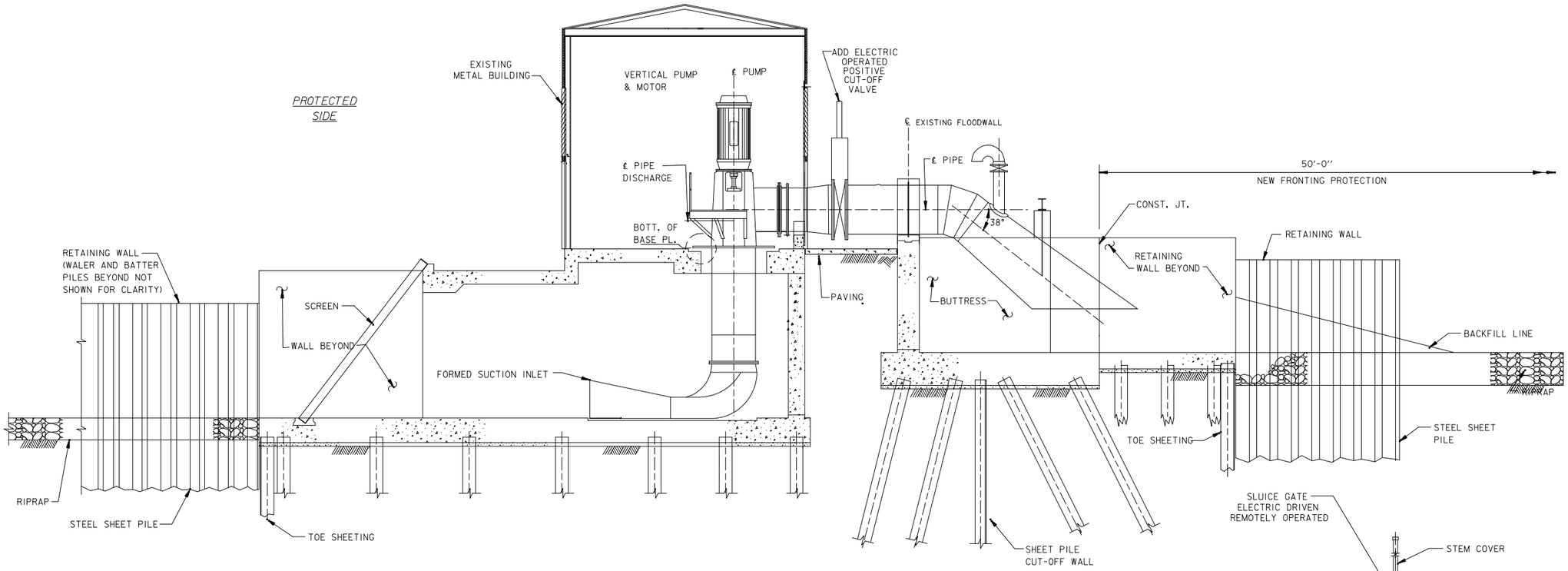
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

TYPICAL EXISTING HORIZONTAL PUMP STATION WITH ADDED FRONTING PROTECTION

SHEET IDENTIFICATION NUMBER
PLATE S-1

auto_username auto_design



NOTE:

1. PUMP DISCHARGE CUT OFF VALVE TO BE REMOTELY OPERATED FROM PUMPING STATION

TYPICAL EXISTING VERTICAL PUMPING STATION WITH ADDED FRONTING PROTECTION

SCALE: 1" = 5'

5' 0 5' 10' 15' 20'

NEW FRONTING PROTECTION

SCALE: 1" = 5'

5' 0 5' 10' 15' 20'

US ARMY CORPS OF ENGINEERS

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: DATE:
DWN BY: CKD BY: SOLICITATION NO.:
SUBMITTED BY: CONTRACT NO.: FILE NUMBER:
FILE NAME: PLOT DATE:
SIZE: PLOT SCALE:

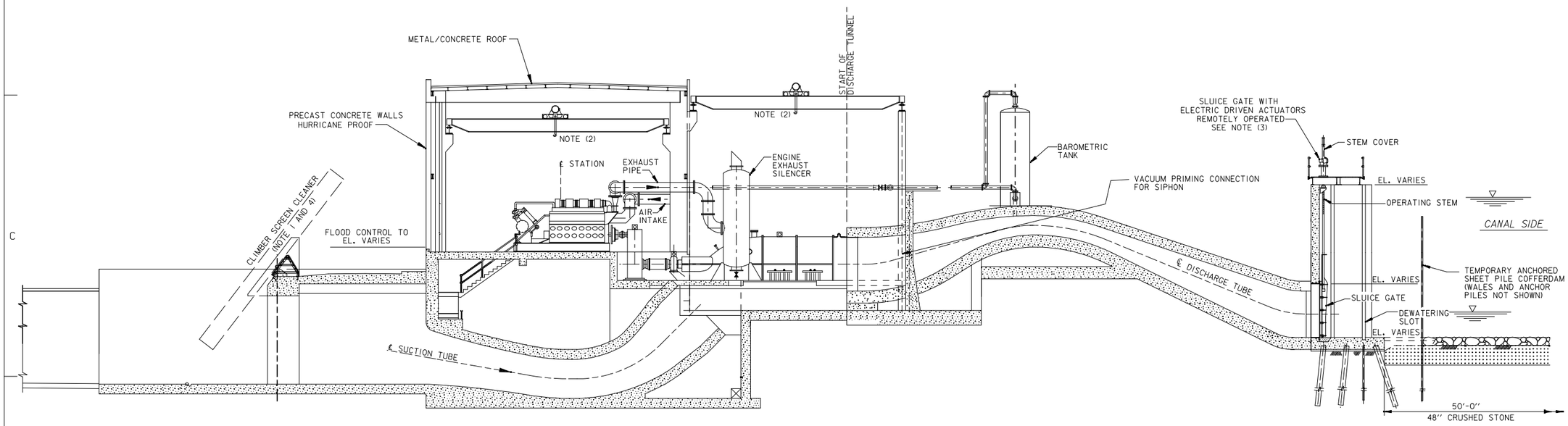
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

TYPICAL EXISTING VERTICAL PUMP STATION WITH ADDED FRONTING PROTECTION

SHEET IDENTIFICATION NUMBER
PLATE S-2

auto_design
auto_username



TYPICAL NEW HORIZONTAL PUMPING STATION WITH FRONTING PROTECTION

SCALE: 1/8" = 1'- 0"

0 5' 10' 15' 20' 25'

NOTES:

1. SIZE CLIMBER SCREEN CLEANER IN ACCORDANCE WITH DESIGN PUMPING CAPACITY.
2. OVERHEAD CRANE OF SUFFICIENT CAPACITY TO HANDLE LIFTING AND SETTING MAJOR EQUIPMENT.
3. SLUICE GATE ACTUATORS WILL BE ELECTRIC DRIVEN REMOTELY ACTIVATED FROM THE PUMPING STATION CONTROL ROOM.
4. FURNISH SCREEN CLEANERS WITH REMOTE CONTROLS TO ALLOW OPERATION FROM THE PUMPING STATION CONTROL ROOM AND LOCALLY AS WELL.
5. SLUICE GATES WILL BE RUN BETWEEN STEEL RAIL GUIDES IMBEDDED IN CONCRETE WALLS.

US ARMY CORPS OF ENGINEERS
district_name

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.: solicitation_num
SUBMITTED BY: | CONTRACT NO.: contract_num
FILE NAME: | FILE NUMBER:
SIZE: | PLOT SCALE: | PLOT DATE:

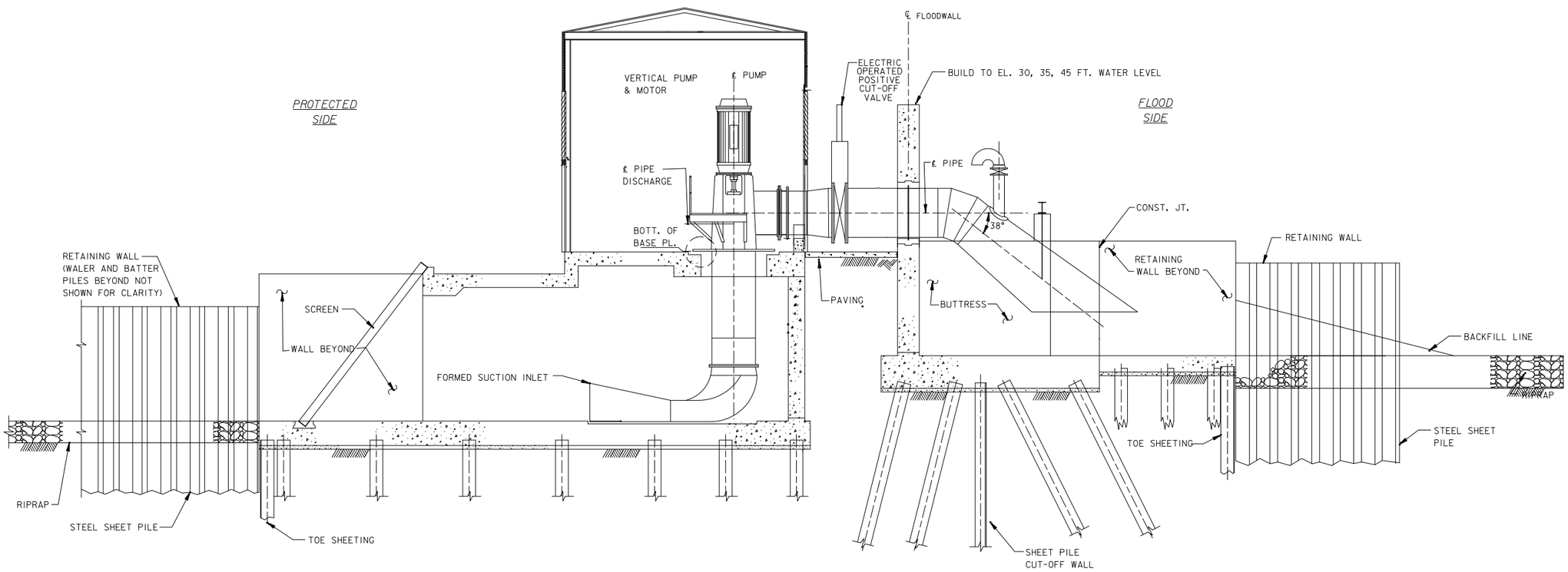
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

TYPICAL NEW HORIZONTAL PUMP STATION WITH FRONTING PROTECTION

SHEET IDENTIFICATION NUMBER
PLATE S-3

set_1d auto_username auto_design



NOTE:

1. PUMP DISCHARGE CUT OFF VALVE TO BE REMOTELY OPERATED FROM PUMPING STATION CONTROL ROOM.

TYPICAL NEW VERTICAL PUMPING STATION WITH FRONTING PROTECTION

SCALE: 1" = 5'

5' 0 5' 10' 15' 20'

US ARMY CORPS OF ENGINEERS
district_name

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.: solicitation_num
SUBMITTED BY: | CONTRACT NO.: contract_num
FILE NAME: | FILE NUMBER:
SIZE: | PLOT SCALE: | PLOT DATE:

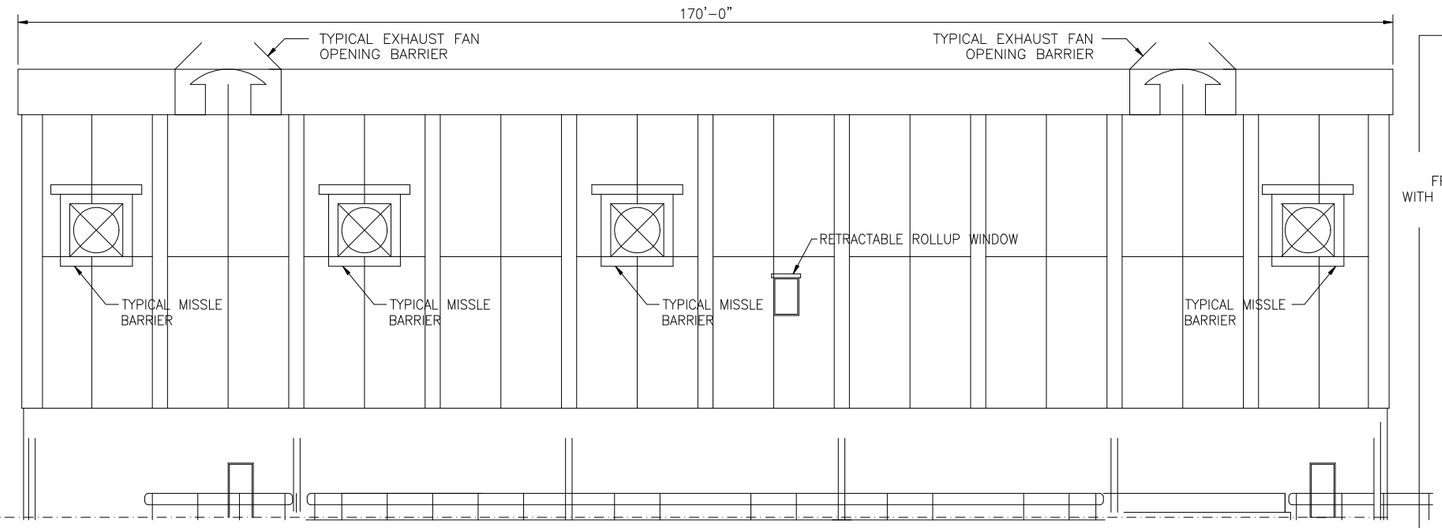
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

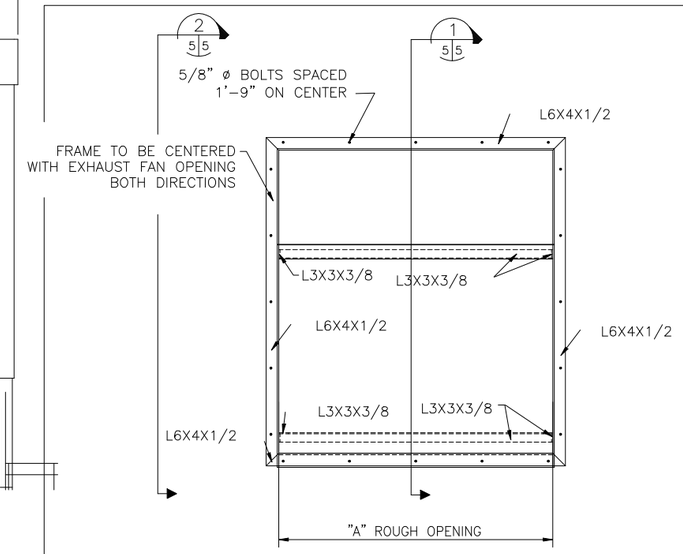
TYPICAL NEW VERTICAL PUMP STATION WITH FRONTING PROTECTION

SHEET IDENTIFICATION NUMBER
PLATE S-4

set_id auto_username auto_design

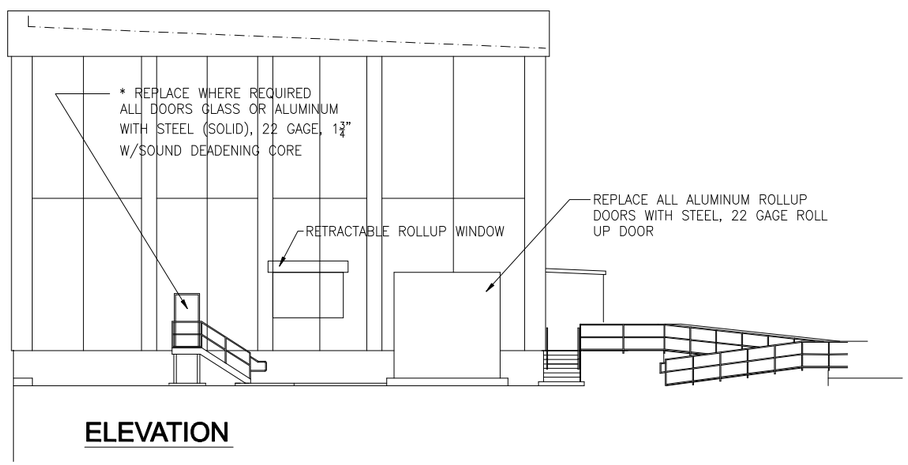


TYPICAL PUMP STATION HARDENING DETAILS



ELEVATION

N.T.S.

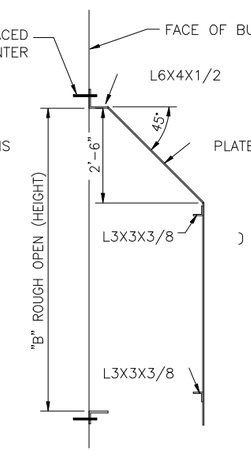


ELEVATION

NOTES:

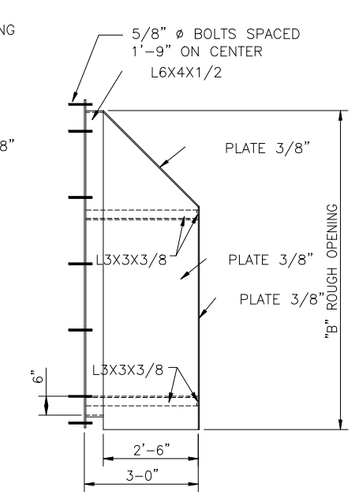
1) PROVIDE SEALANT BETWEEN FRAME AND WALL PANELS (ALL SIDES).

2) SEAL WELD ALL CONNECTIONS WITH 1/4" FILLET WELDS.



SECTION 1

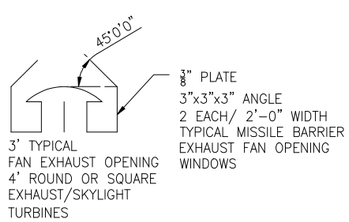
N.T.S.



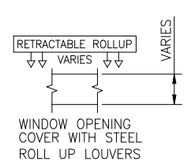
SECTION 2

N.T.S.

TYPICAL MISSLE BARRIER DETAILS



TYPICAL EXHAUST FAN OPENING BARRIER



TYPICAL WINDOW OPENING BARRIER

US ARMY CORPS OF ENGINEERS

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

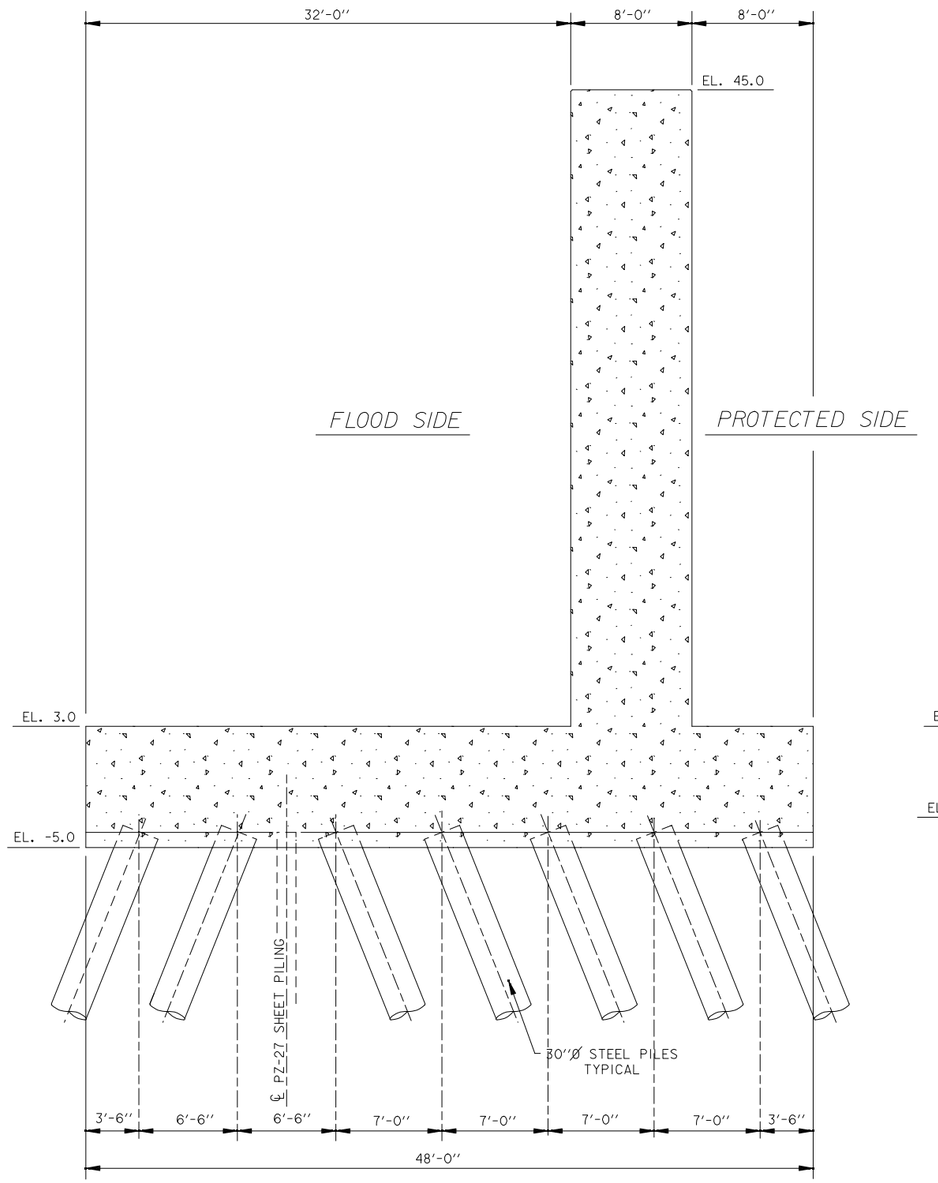
DESIGNED BY: DFH | DATE: MAY, 2008
DWN BY: JAY | CKD BY: RJG | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
FILE NAME: X0000CAT-0PUMP STATION-6 | FILE NUMBER:
SIZE: | PLOT SCALE: 1 = 10' | PLOT DATE:

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS

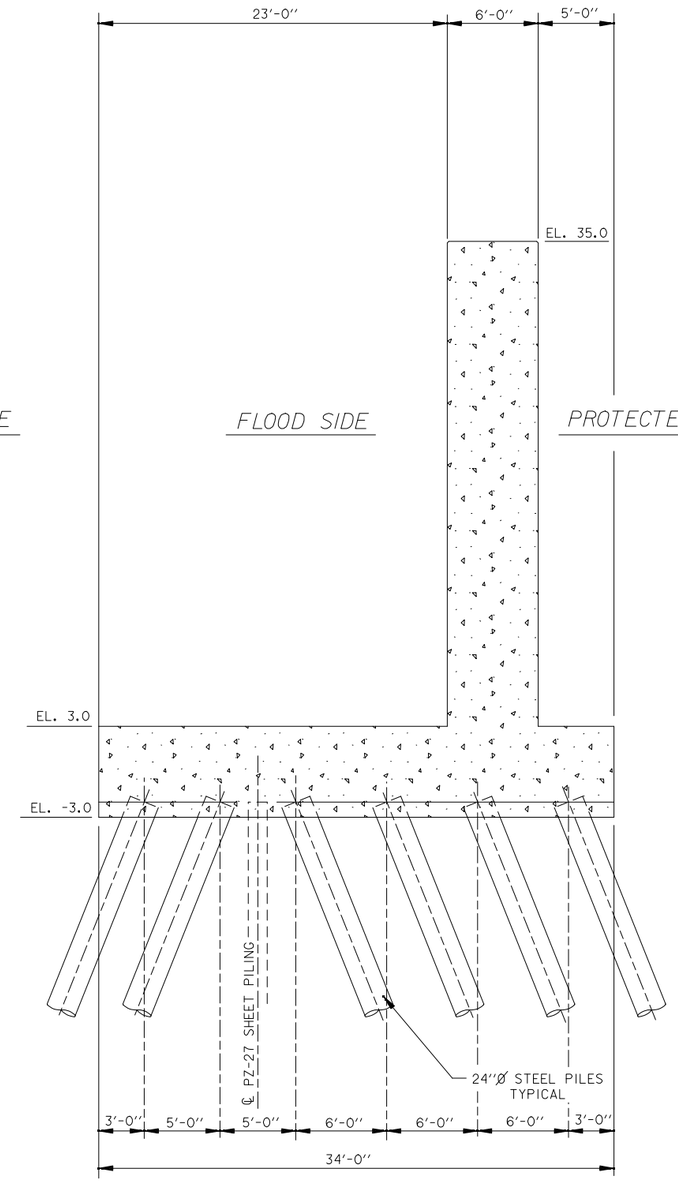
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

TYPICAL PUMP STATION HARDENING DETAILS

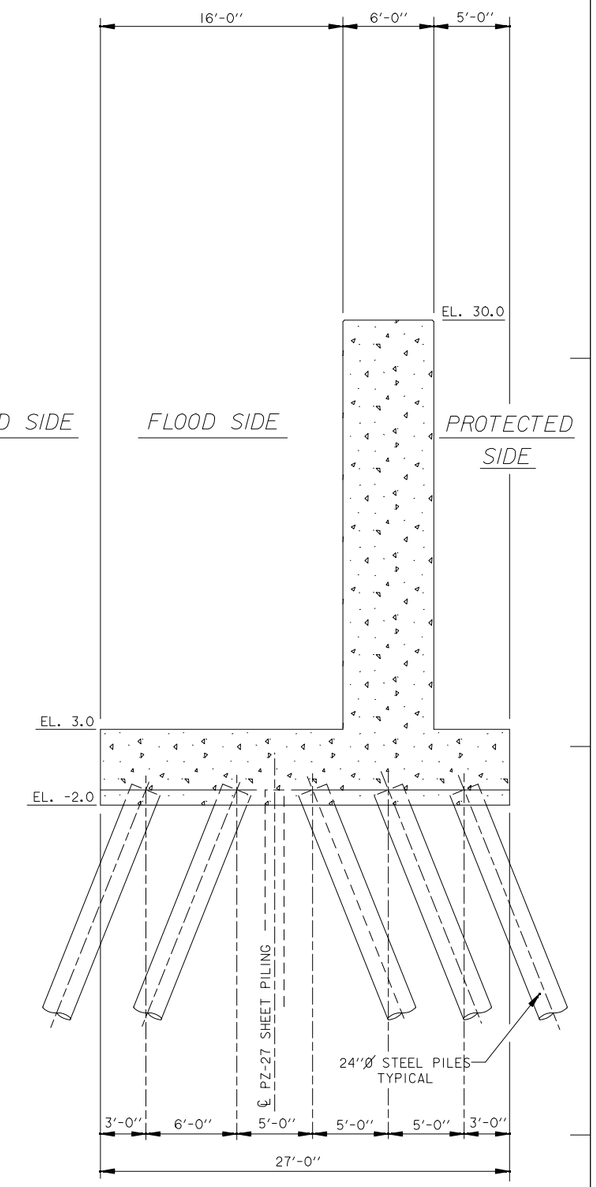
SHEET IDENTIFICATION NUMBER
PLATE S-5



PROTECTION TO EL. 45.0



PROTECTION TO EL. 35.0



PROTECTION TO EL. 30.0

SCALE: 1" = 5'

5' 0 5' 10' 15' 20'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

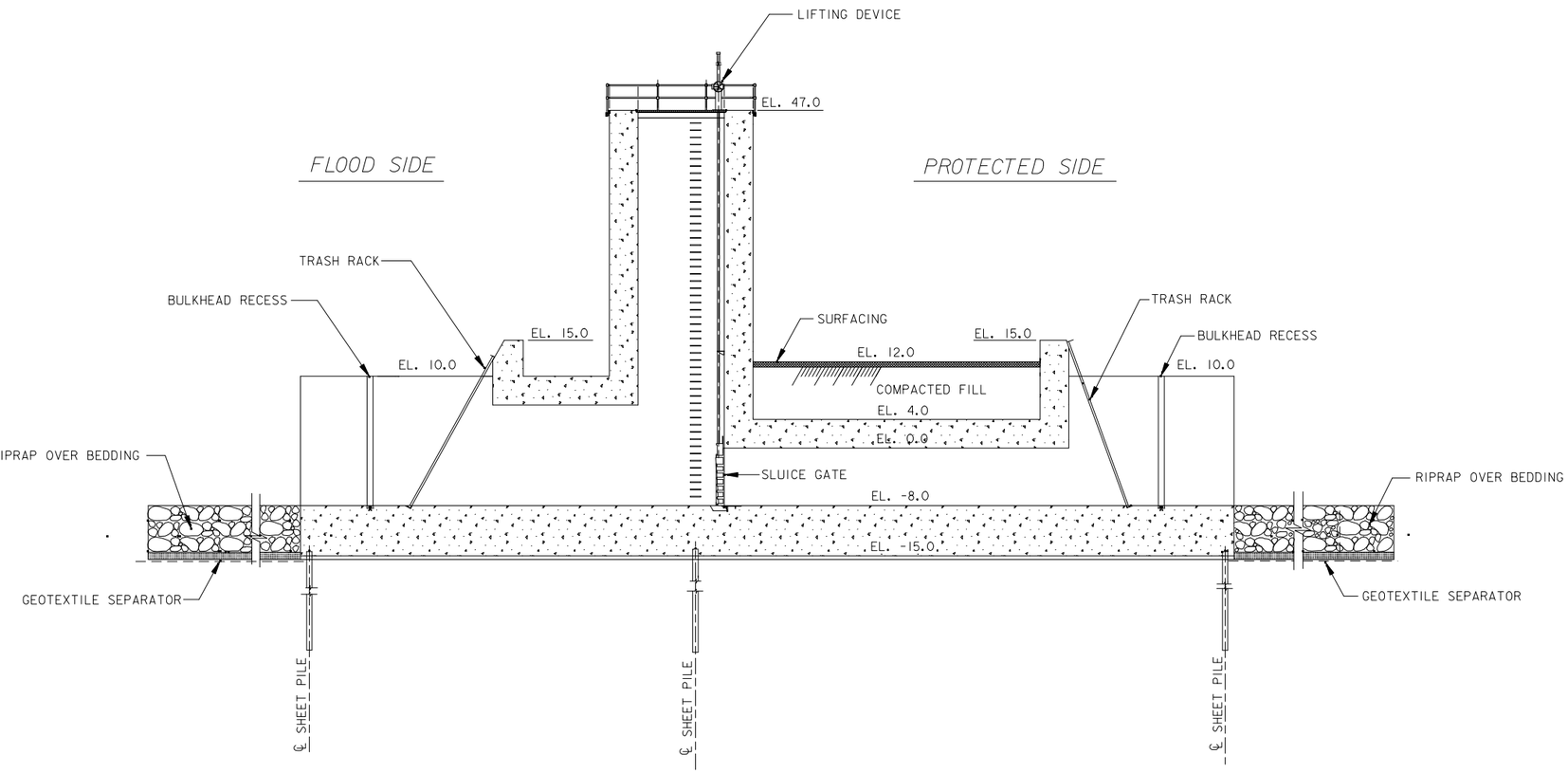
DESIGNED BY: DATE:
DWN BY: CKD BY: SOLICITATION NO.:
SUBMITTED BY: CONTRACT NO.:
FILE NAME: FILE NUMBER:
SIZE: PLOT SCALE: PLOT DATE:

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

T-WALL SECTIONS

SHEET IDENTIFICATION NUMBER
PLATE S-6



TYPICAL SLUICE GATE SECTION

SCALE: 1" = 10'

10' 0 10' 20' 30' 40'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

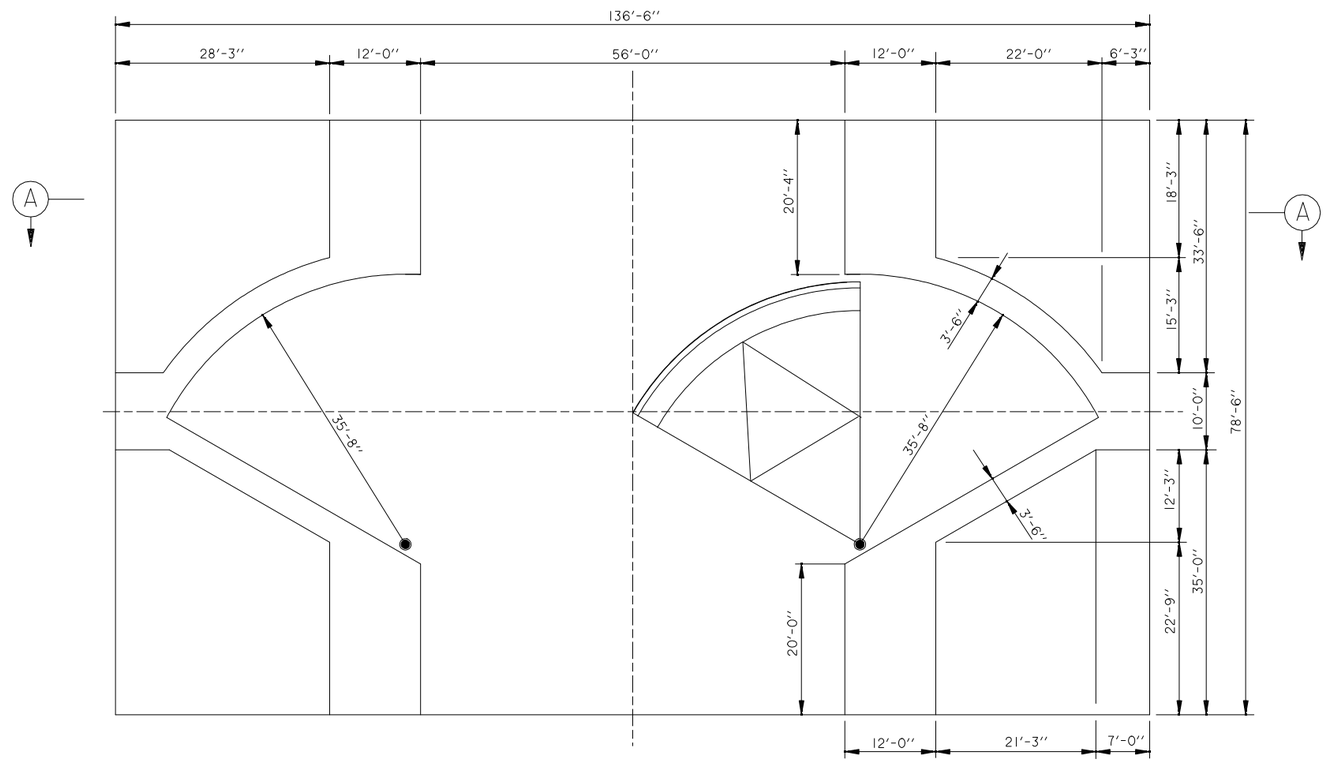
DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
FILE NAME: | FILE NUMBER:
SIZE: | PLOT SCALE: | PLOT DATE:

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

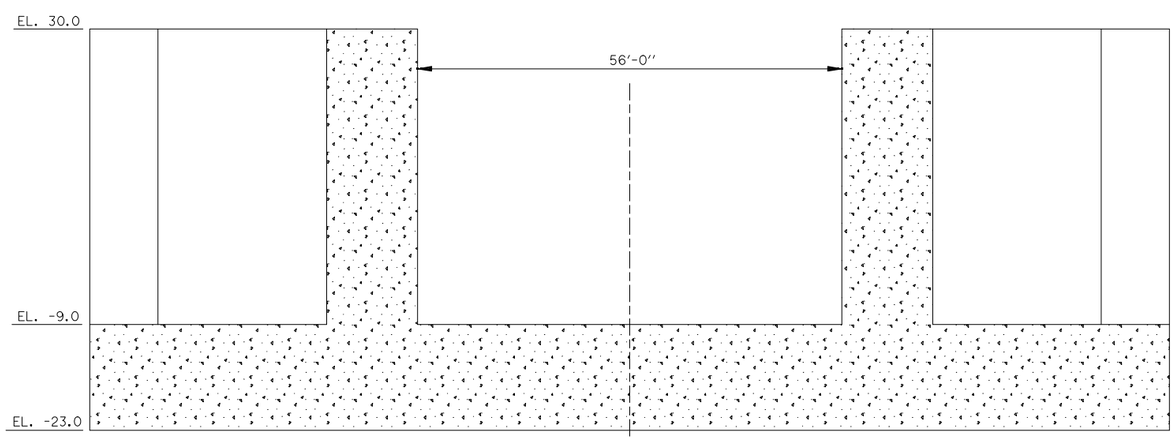
SLUICE GATE STRUCTURE PROTECTION TO EL. 45.0

SHEET IDENTIFICATION NUMBER
PLATE S-7



PLAN

SCALE: 1'' = 10'



SECTION (A)

SCALE: 1'' = 10'

SCALE: 1'' = 10'

10' 0 10' 20' 30' 40'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

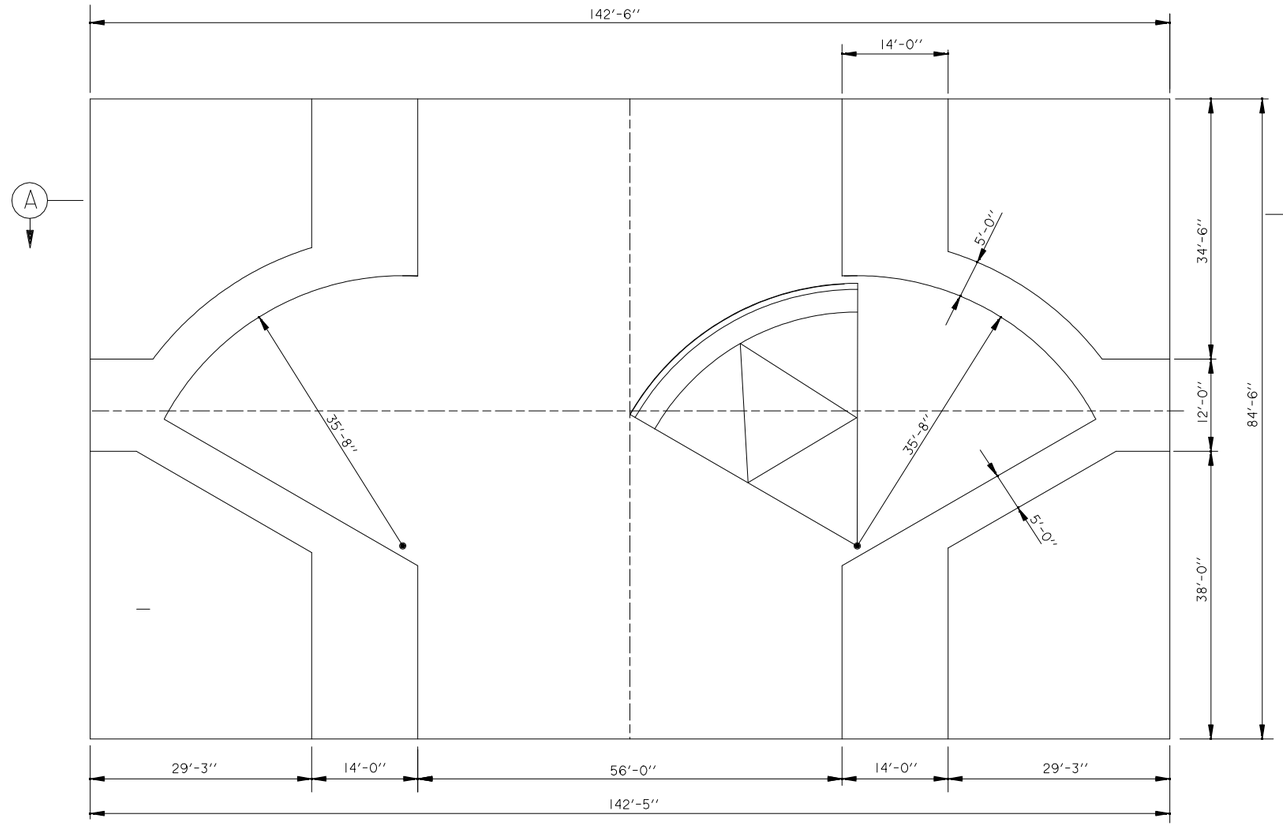
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: DATE:
DWN BY: CKD BY: SOLICITATION NO.:
SUBMITTED BY: CONTRACT NO.:
FILE NAME: FILE NUMBER:
SIZE: PLOT SCALE: PLOT DATE:

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

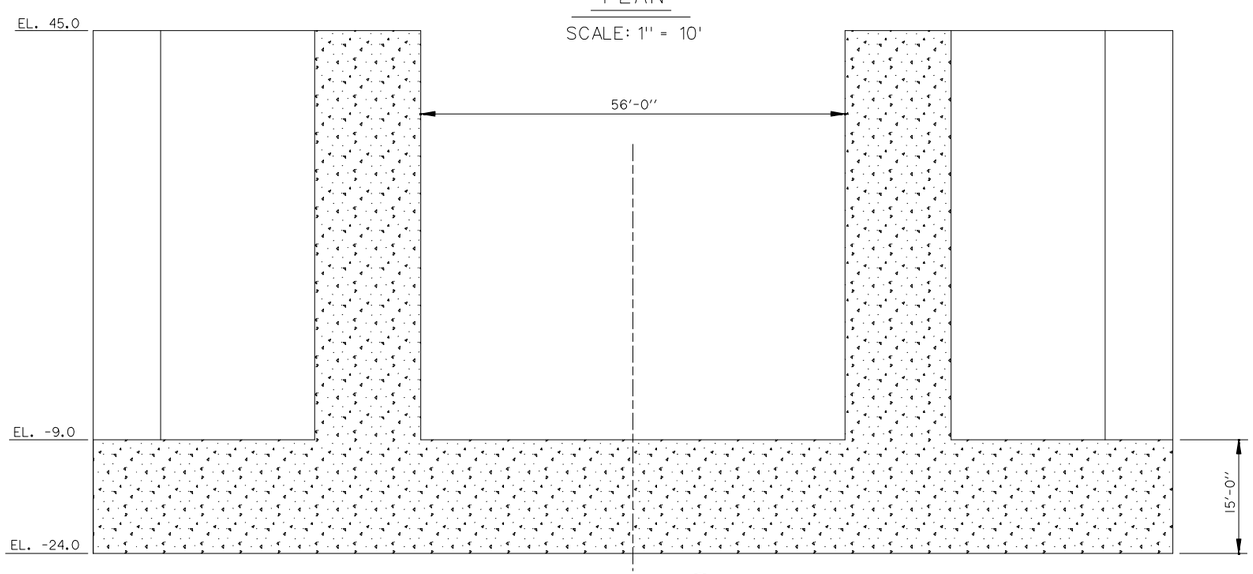
SECTOR GATE
56' WIDE - SILL EL. -9
PROTECTION TO EL. 30.0

SHEET IDENTIFICATION NUMBER
PLATE S-8



PLAN

SCALE: 1" = 10'



SECTION (A)

SCALE: 1" = 10'

SCALE: 1" = 10'

10' 0 10' 20' 30' 40'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

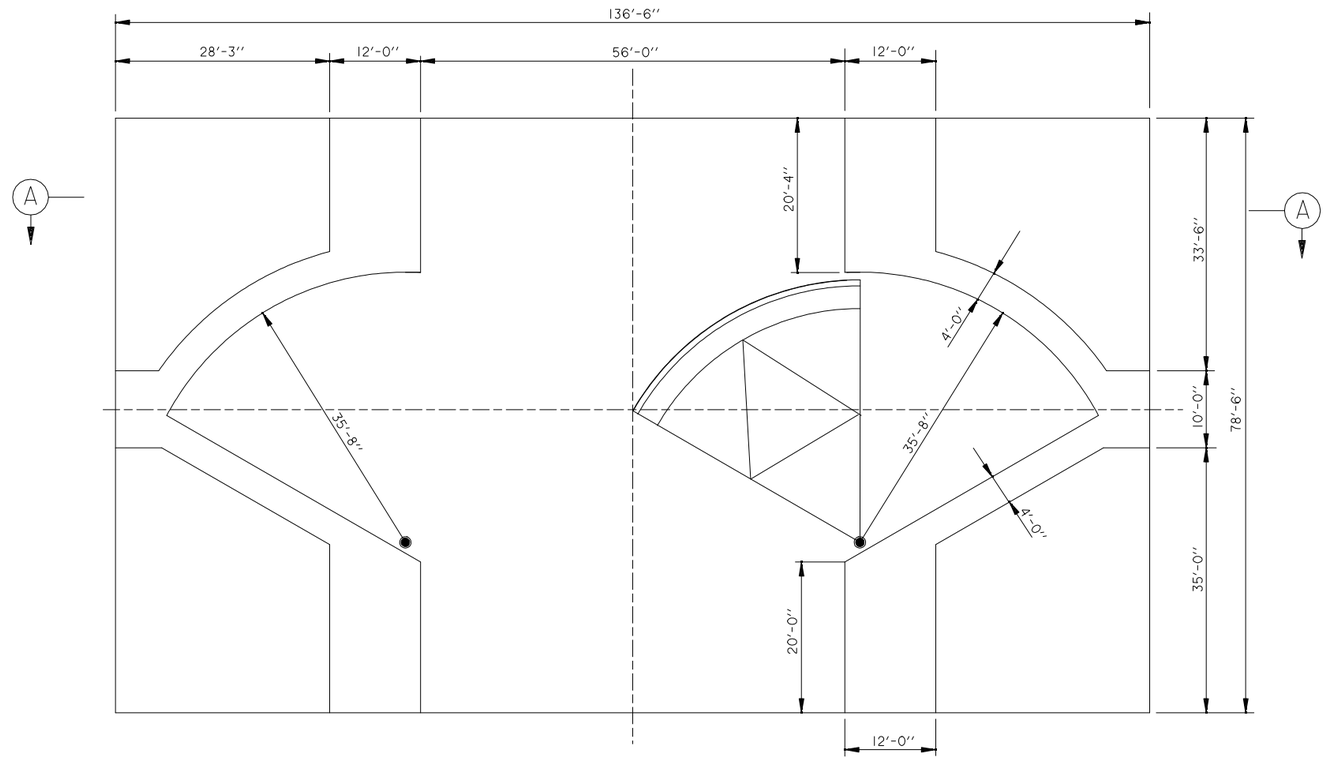
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: DATE:
DWN BY: CKD BY: SOLICITATION NO.:
SUBMITTED BY: CONTRACT NO.:
FILE NAME: FILE NUMBER:
SIZE: PLOT SCALE: PLOT DATE:

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

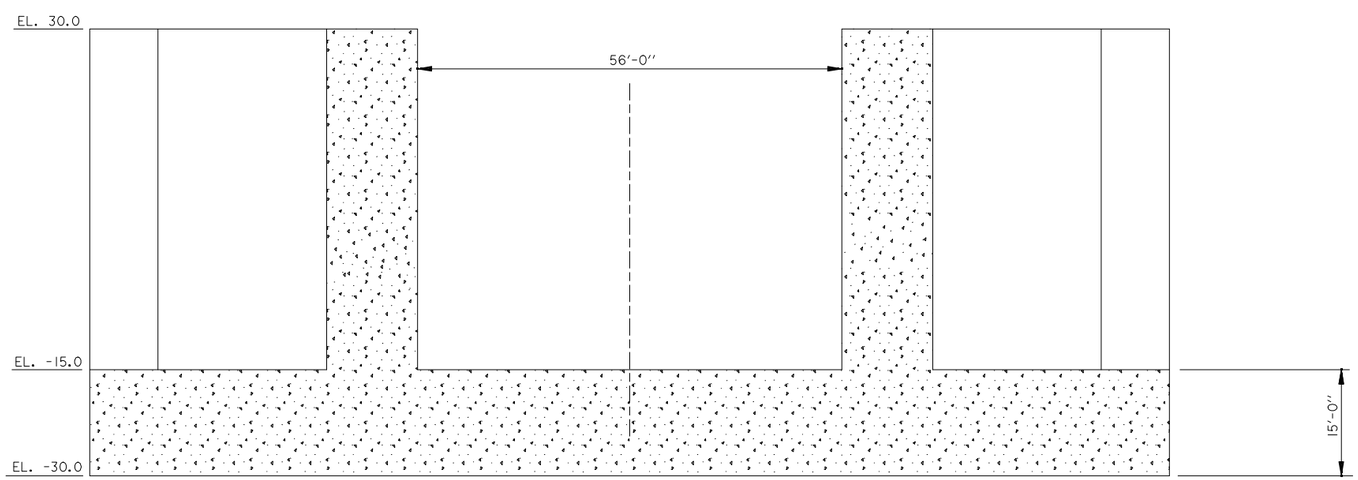
SECTOR GATE
56' WIDE - SILL EL. -9
PROTECTION TO EL. 45.0

SHEET IDENTIFICATION NUMBER
PLATE S-9



PLAN

SCALE: 1" = 10'



SECTION (A)

SCALE: 1" = 10'

SCALE: 1" = 10'

10' 0 10' 20' 30' 40'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

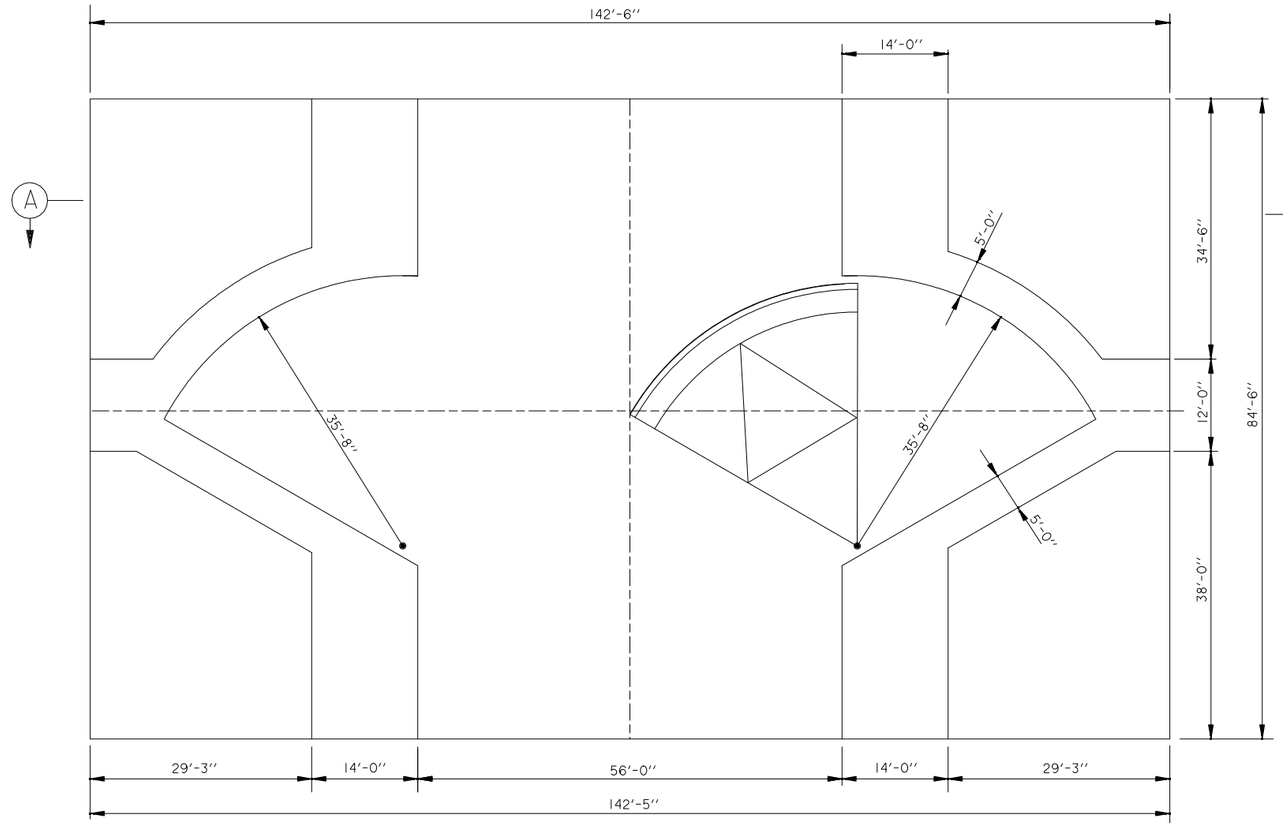
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
FILE NAME: | FILE NUMBER:
SIZE: | PLOT SCALE: | PLOT DATE:

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

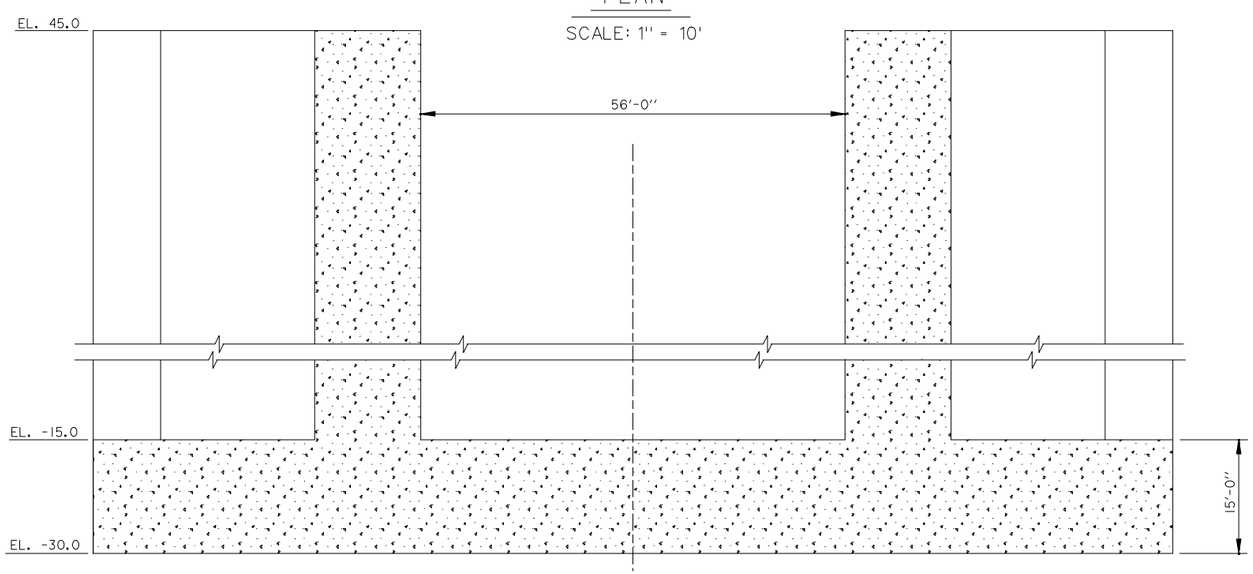
SECTOR GATE
56' WIDE - SILL EL. -15
PROTECTION TO EL. 30.0

SHEET IDENTIFICATION NUMBER
PLATE S-10



PLAN

SCALE: 1" = 10'



SECTION (A)

SCALE: 1" = 10'

SCALE: 1" = 10'

10' 0 10' 20' 30' 40'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

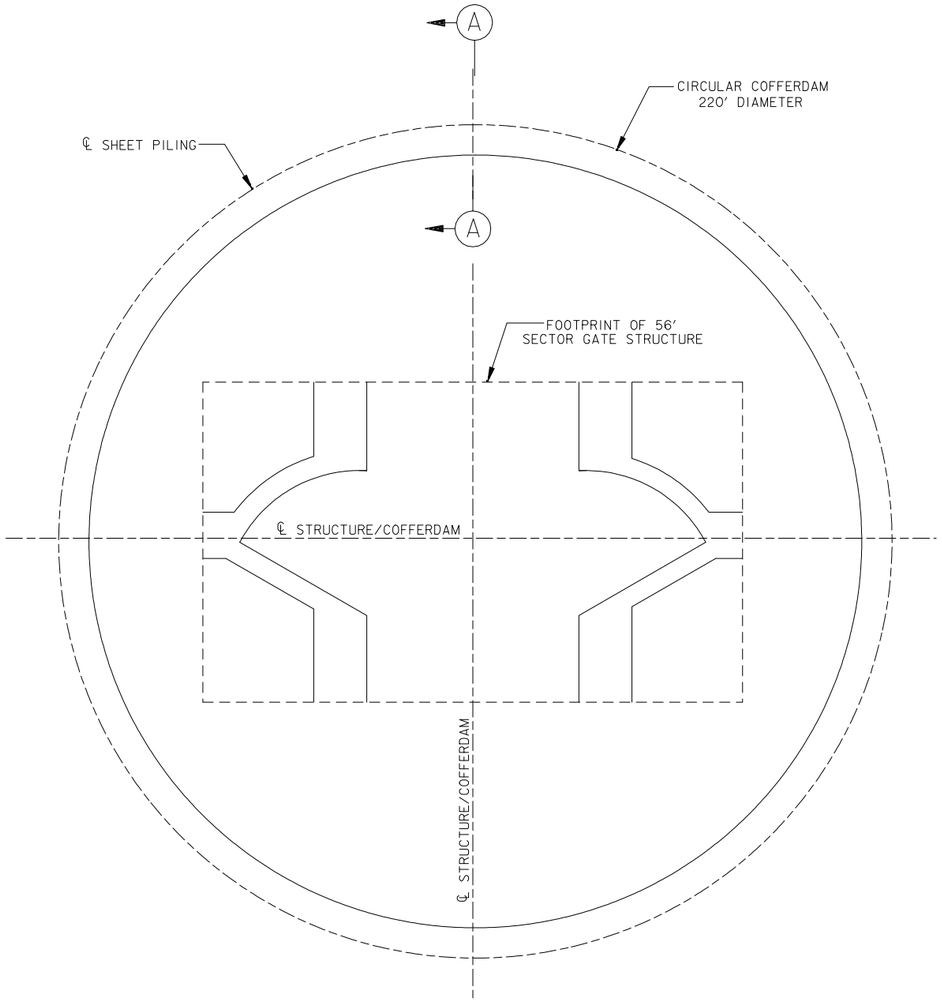
DESIGNED BY: DATE:
DWN BY: CKD BY: SOLICITATION NO.:
SUBMITTED BY: CONTRACT NO.:
FILE NAME: FILE NUMBER:
SIZE: PLOT SCALE: PLOT DATE:

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

SECTOR GATE 56' WIDE - SILL EL. -15 PROTECTION TO EL. 45

SHEET IDENTIFICATION NUMBER

PLATE S-11

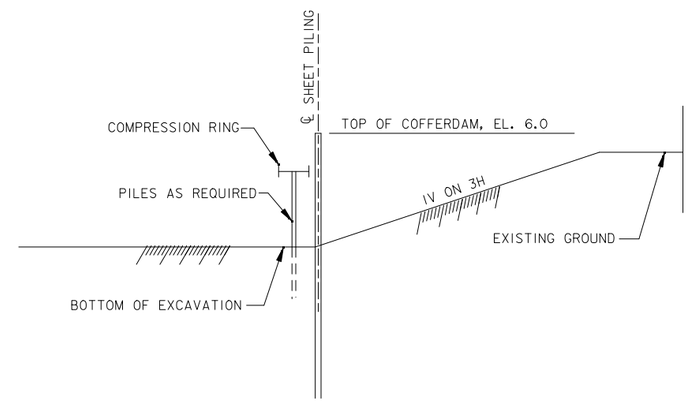


CIRCULAR COFFERDAM FOR 56' SECTOR GATES

PLAN

SCALE: 1" = 20'

20' 0 20' 40' 60' 80'



SECTION A

SCALE: 1" = 20'

20' 0 20' 40' 60' 80'

US ARMY CORPS
OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

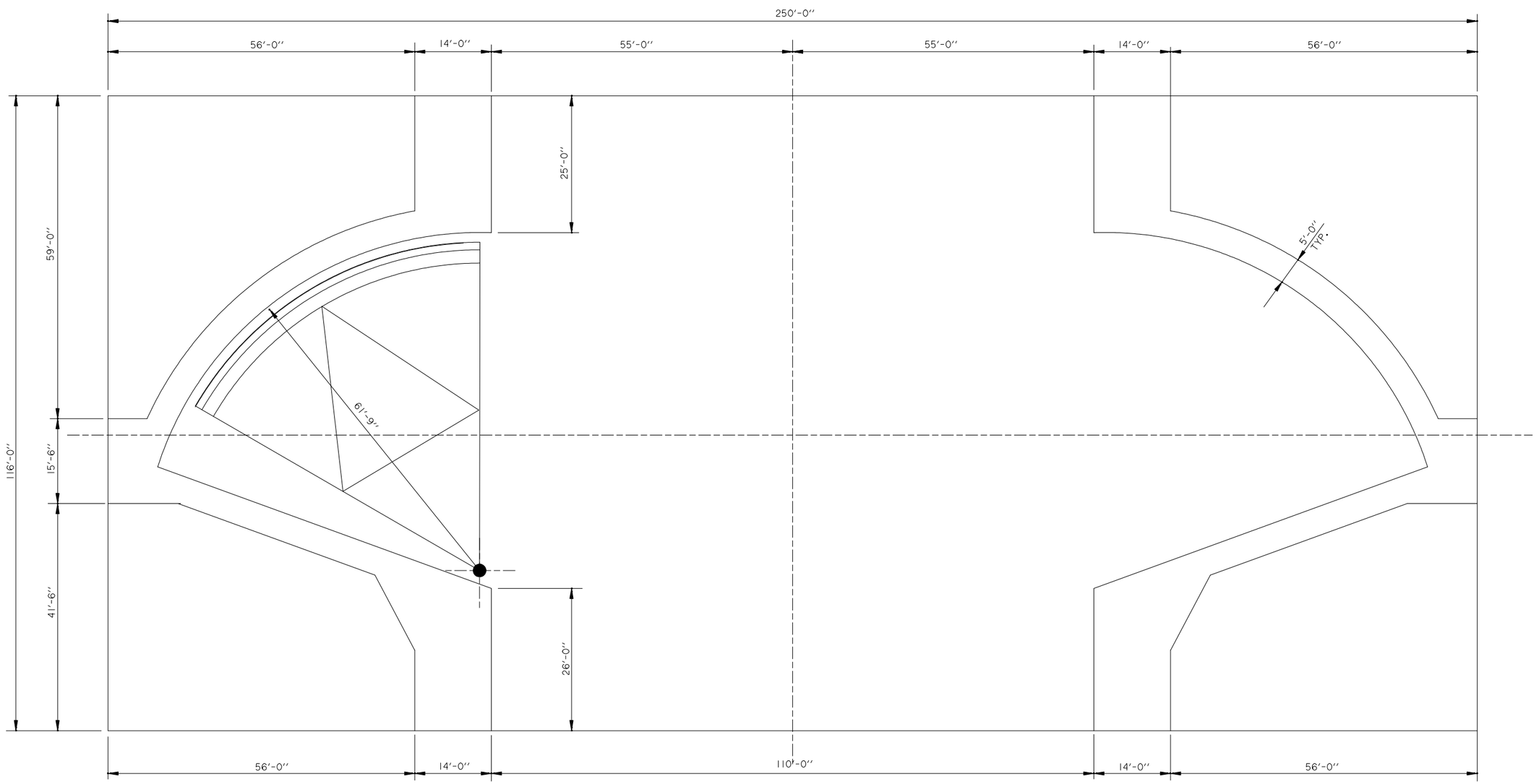
DESIGNED BY: DATE:
DWN BY: CKD BY: SOLICITATION NO.:
SUBMITTED BY: CONTRACT NO.:
FILE NAME: FILE NUMBER:
SIZE: PLOT SCALE: PLOT DATE:

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

CIRCULAR COFFERDAM 56' SECTOR GATE

SHEET IDENTIFICATION NUMBER

PLATE S-12



110' WIDE SECTOR GATE

SILL EL. -15.0

PLAN

SCALE: 1" = 10'

10' 0 10' 20' 30' 40'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

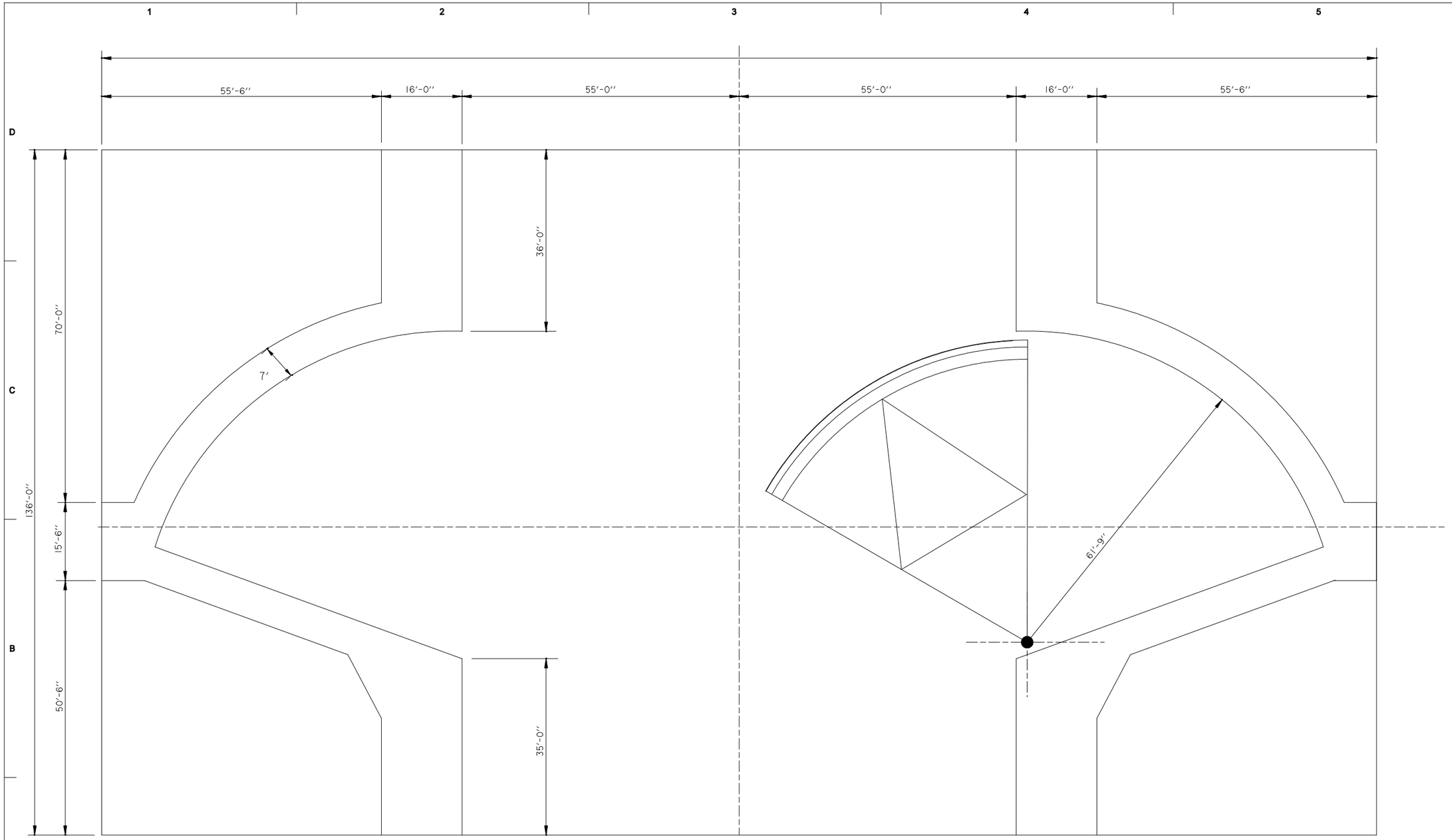
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: DATE:
DWN BY: CKD BY: SOLICITATION NO.:
SUBMITTED BY: CONTRACT NO.:
FILE NAME: FILE NUMBER:
SIZE: PLOT SCALE: PLOT DATE:

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

SECTOR GATE
110' WIDE - SILL EL. -15.0
PROTECTION TO EL. 30.0

SHEET IDENTIFICATION NUMBER
PLATE S-13



SECTOR GATE

110' WIDE - SILL EL. -15.0

PLAN

SCALE: 1" = 10'

10' 0 10' 20' 30' 40'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

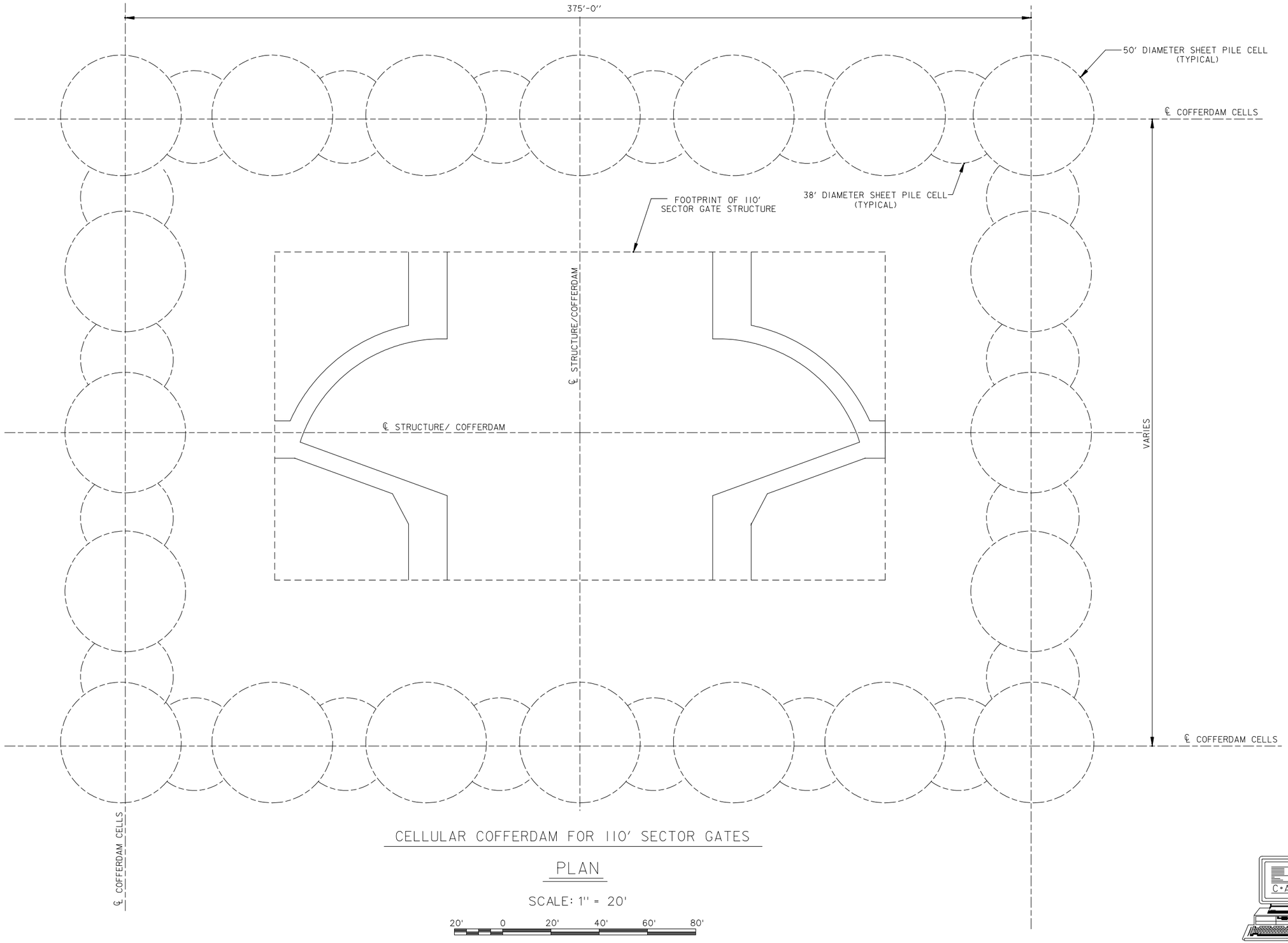
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: DATE:
DWN BY: CKD BY: SOLICITATION NO.:
SUBMITTED BY: CONTRACT NO.:
FILE NAME: FILE NUMBER:
SIZE: PLOT SCALE: PLOT DATE:

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

SECTOR GATE
110' WIDE - SILL EL. -15
PROTECTION TO EL. 45.0

SHEET IDENTIFICATION NUMBER
PLATE S-14



CELLULAR COFFERDAM FOR 110' SECTOR GATES

PLAN

SCALE: 1" = 20'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

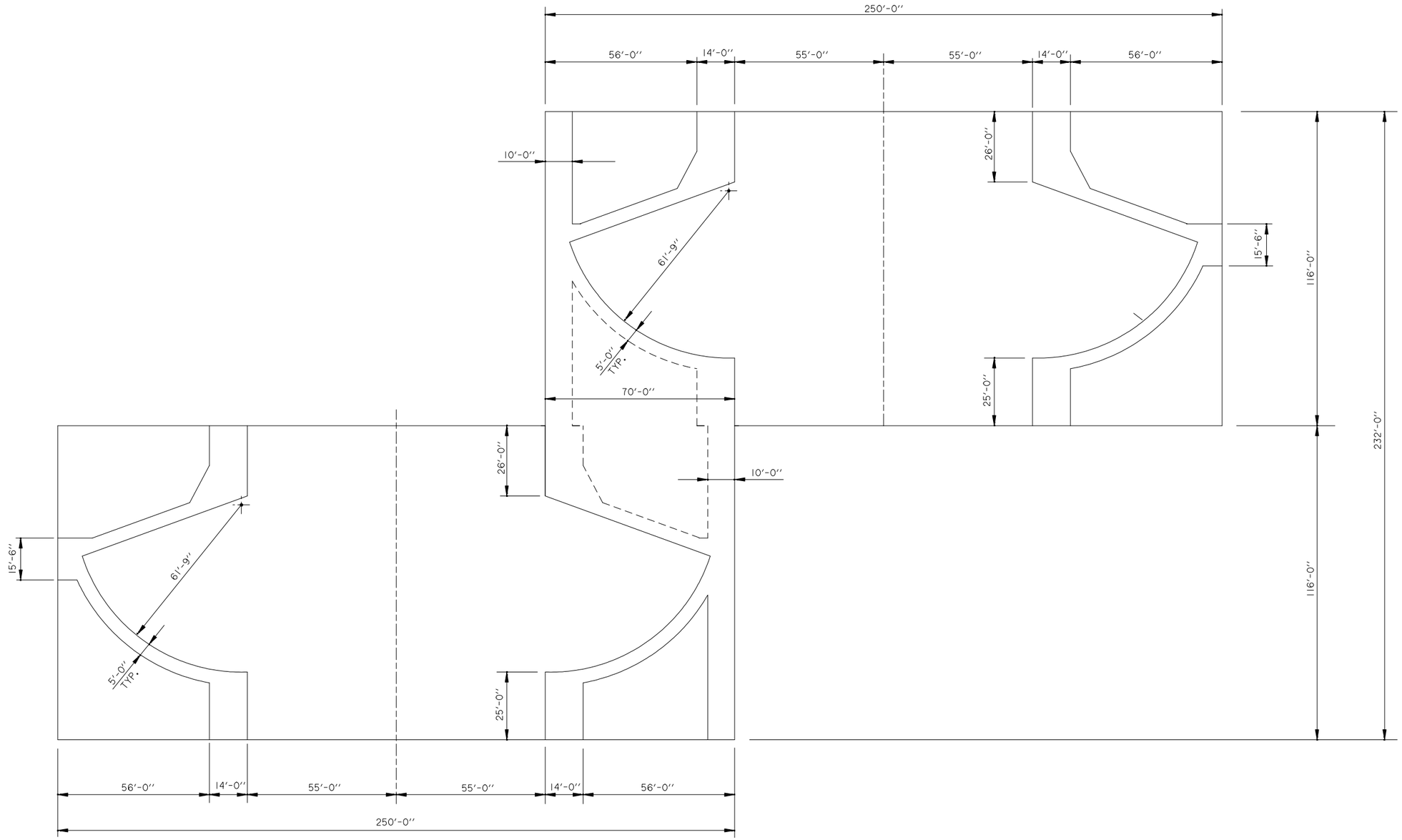
U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: | DATE:
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
FILE NAME: | FILE NUMBER:
SIZE: | PLOT SCALE: | PLOT DATE:

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

CELLULAR COFFERDAM 110' SECTOR GATE

SHEET IDENTIFICATION NUMBER
PLATE S-15



DOUBLE SECTOR GATE- 110' X 30' HIGH -15' SILL

PLAN

SCALE: 1'' = 20'

20' 0 20' 40' 60' 80'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

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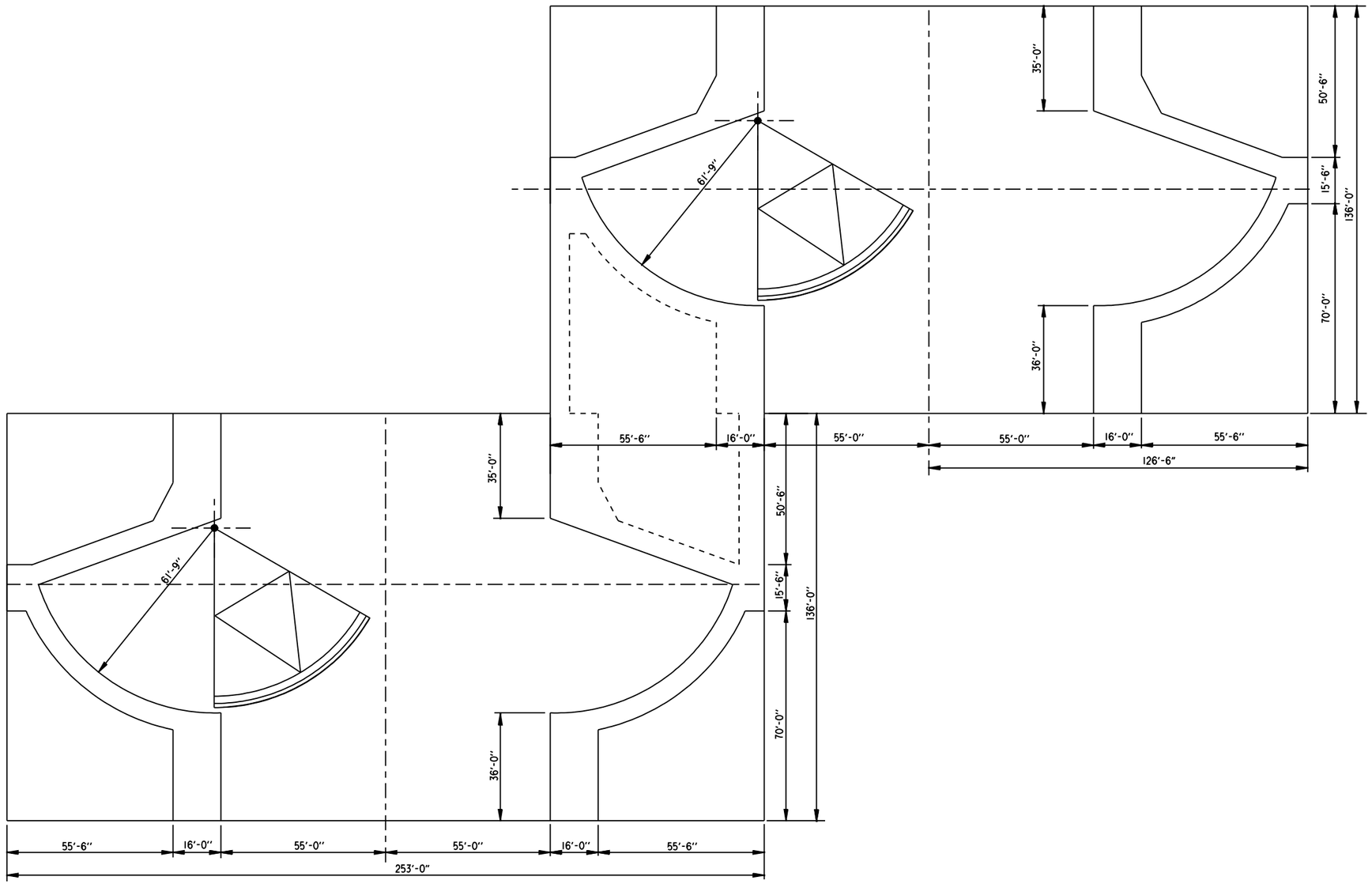
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

DOUBLE SECTOR GATE
110' WIDE - SILL EL. -15.0
PROTECTION TO EL. 30.0

SHEET IDENTIFICATION NUMBER
PLATE S-16



SECTOR GATE

110' WIDE - SILL EL. -15.0

PLAN

SCALE: 1" = 20'

20' 0 20' 40' 60' 80'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

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MISSISSIPPI VALLEY DIVISION

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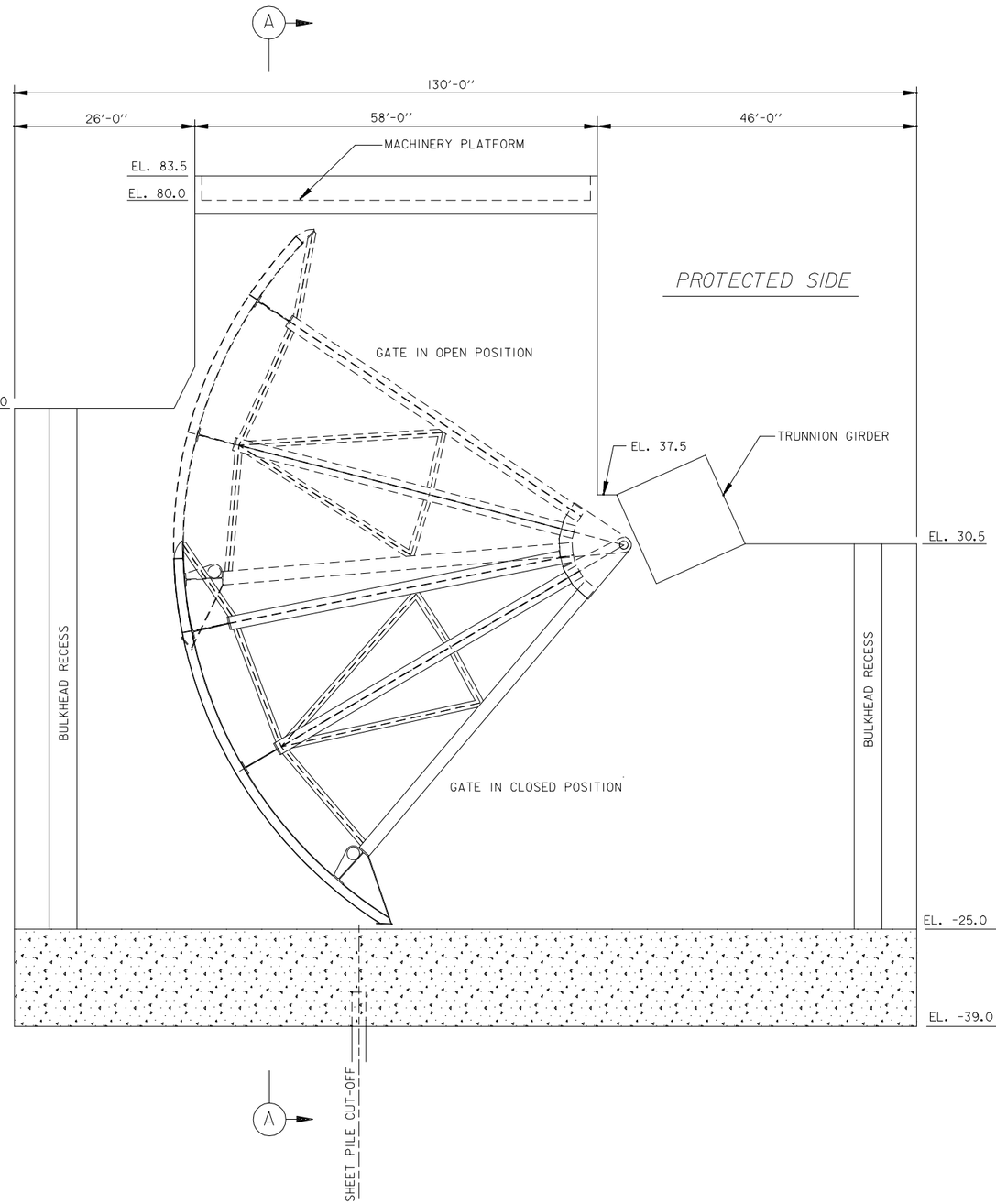
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

DOUBLE SECTOR GATE
110' WIDE - SILL EL. -15.0
PROTECTION AT EL. 45.0

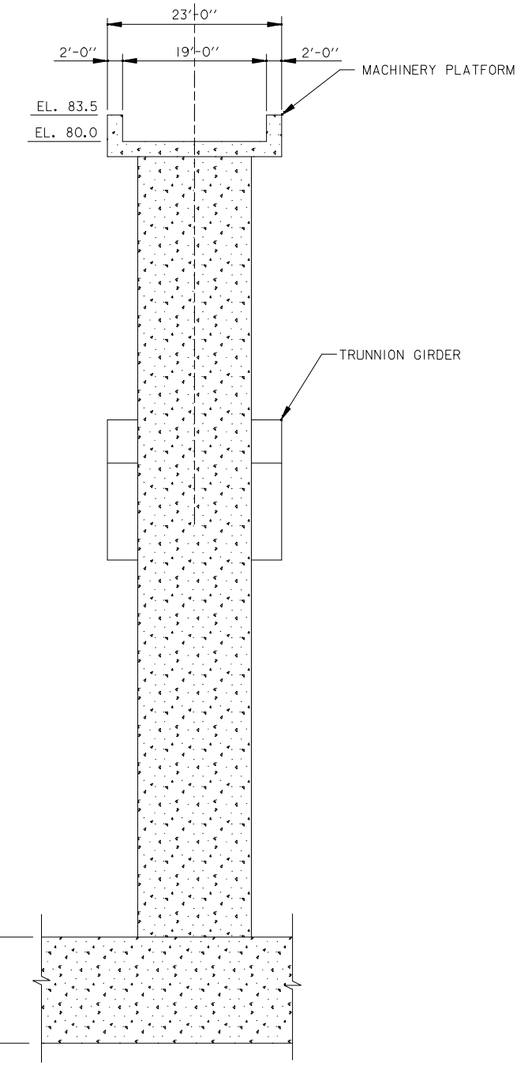
SHEET IDENTIFICATION NUMBER
PLATE S-17



TYPICAL TAINTER GATE STRUCTURE

SCALE: 1" = 10'

10' 0 10' 20' 30' 40'



SECTION A

SCALE: 1" = 10'

10' 0 10' 20' 30' 40'

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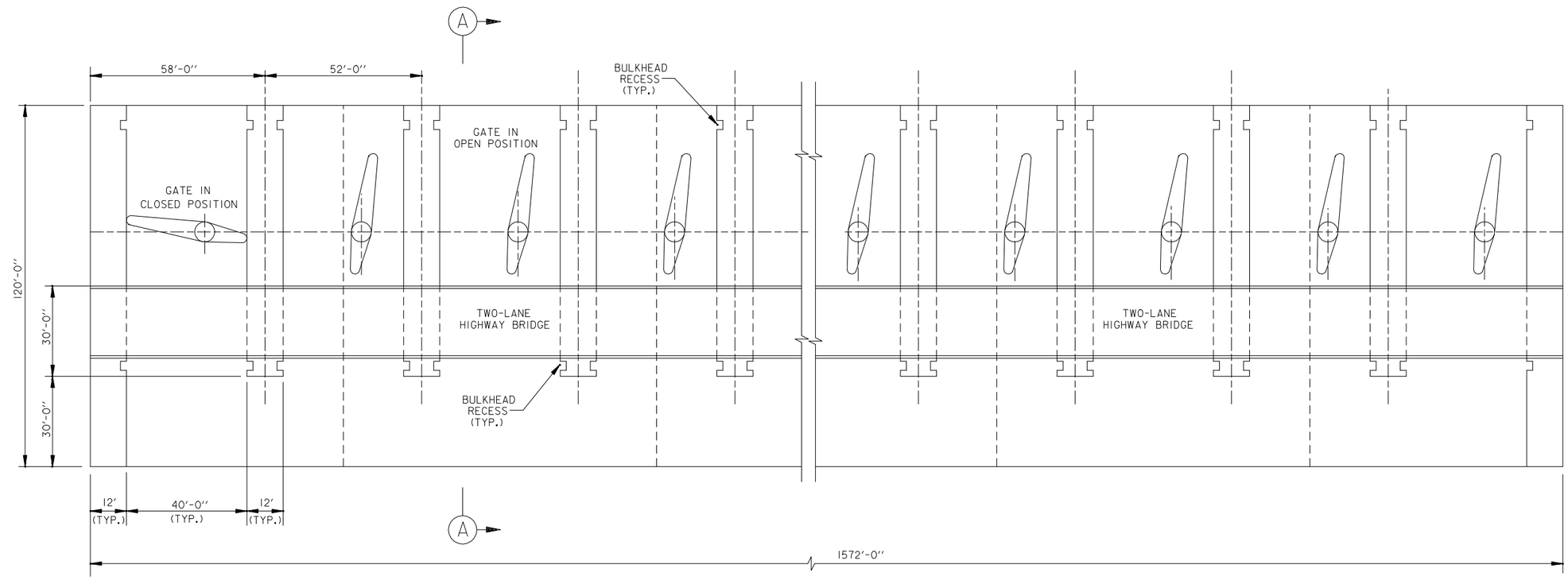
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

TAINER GATE STRUCTURE PROTECTION TO EL. 30.0

SHEET IDENTIFICATION NUMBER
PLATE S-18

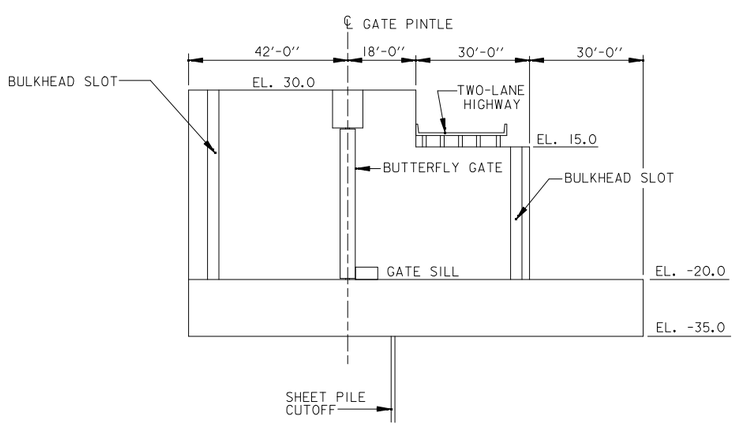


PLAN BELOW EL. 20.0

BUTTERFLY GATED STRUCTURE

SCALE: 1" = 20'

20' 0 20' 40' 60' 80'



SECTION (A)

SCALE: 1" = 20'

20' 0 20' 40' 60' 80'

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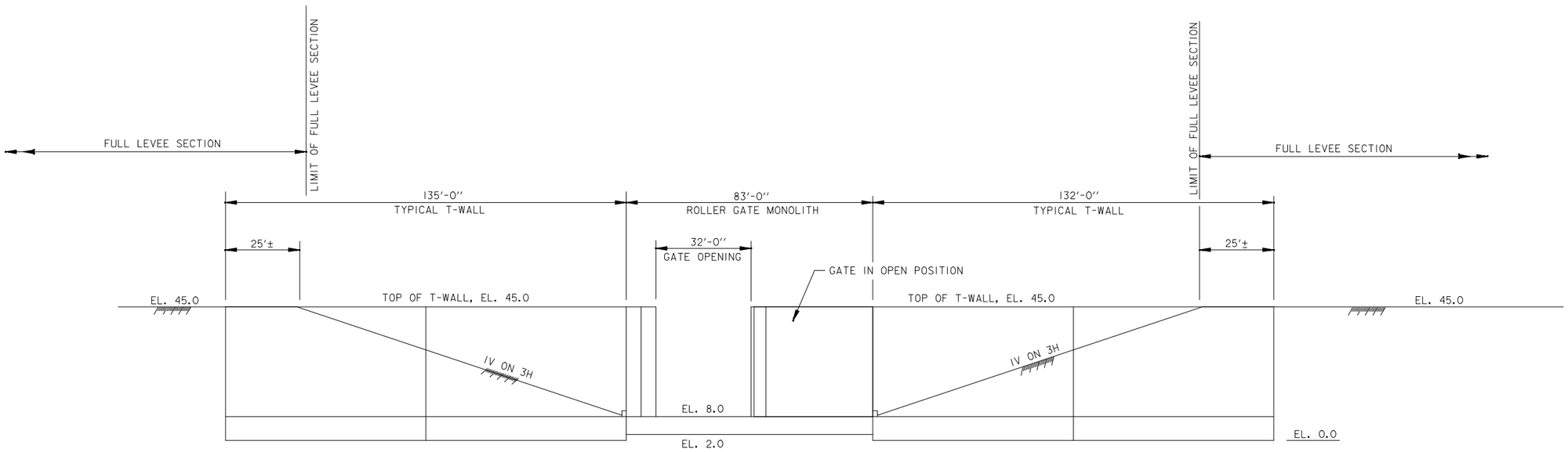
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

BUTTERFLY GATED STRUCTURE PLAN AND SECTION

SHEET IDENTIFICATION NUMBER
PLATE S-19



BOTTOM ROLLER GATE
FLOOD SIDE ELEVATION

SCALE: 1" = 20'

20' 0 20' 40' 60' 80'

US ARMY CORPS OF ENGINEERS
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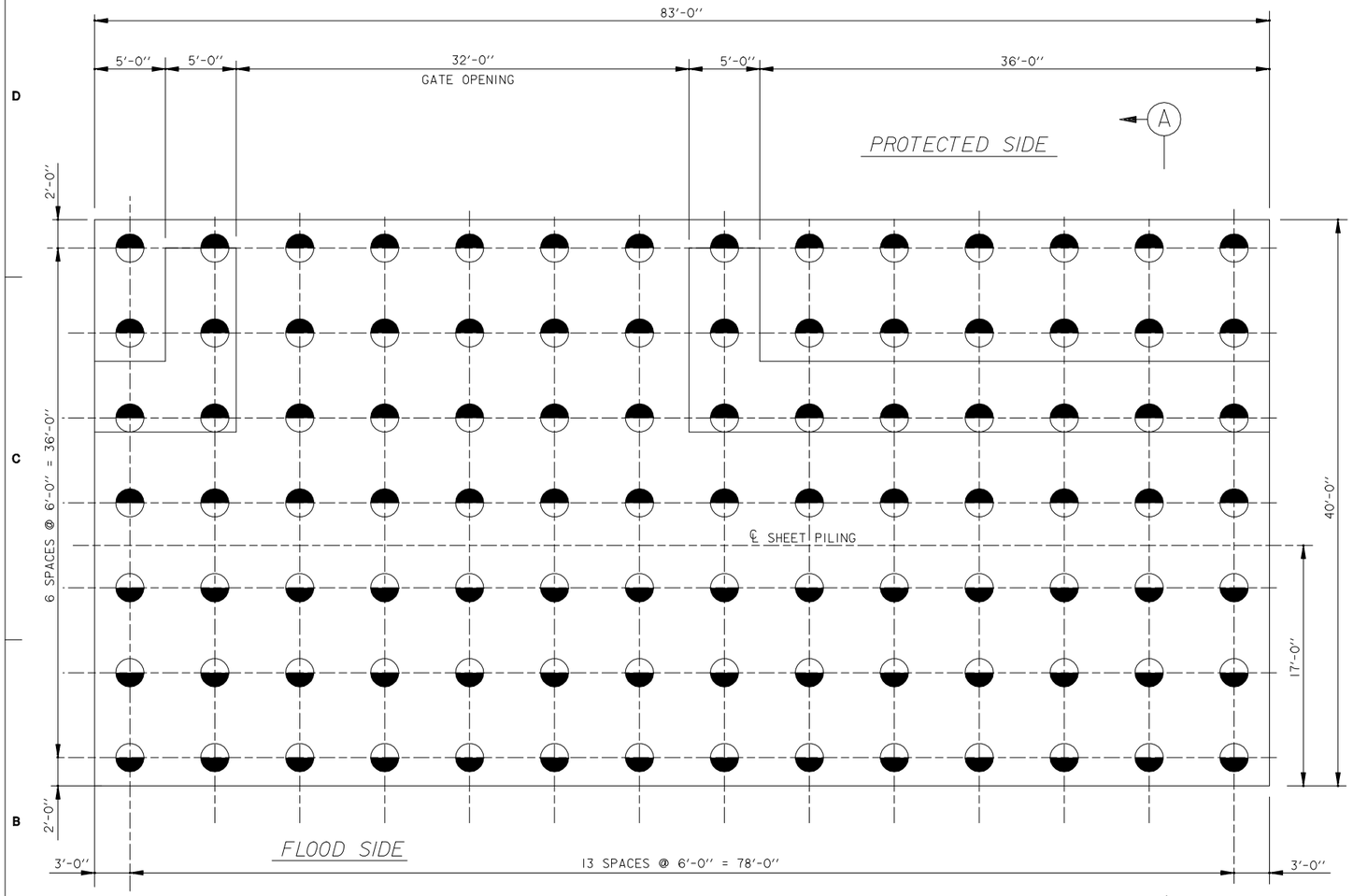
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

BOTTOM ROLLER GATE
32' WIDE - SILL EL. 8.0
PROTECTION TO EL. 45.0

SHEET IDENTIFICATION NUMBER
PLATE S-20



ROLLER GATE MONOLITH

PROTECTION TO EL. 45.0 - SILL EL. 8.0

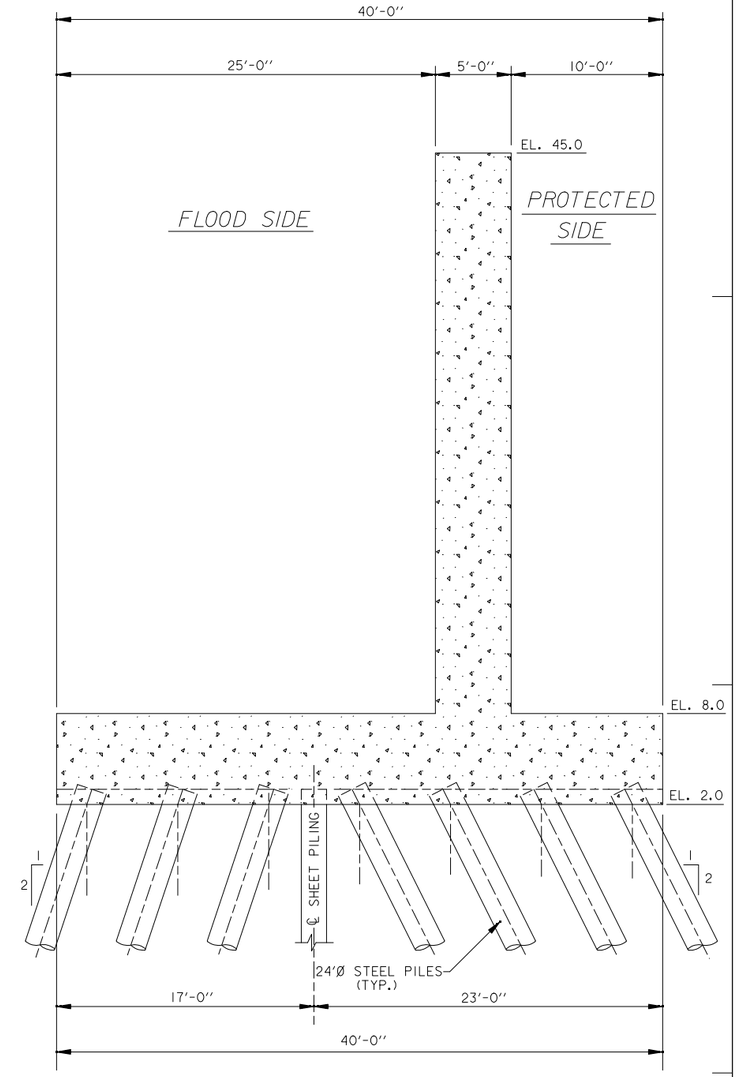
PILE LAYOUT

SCALE: 1'' = 5'

5' 0 5' 10' 15' 20'

24''Ø STEEL PILE

DIRECTION OF BATTER



NOTE:
ALL PILES BATTERED IV ON 2H.

SECTION (A)

SCALE: 1'' = 5'

5' 0 5' 10' 15' 20'

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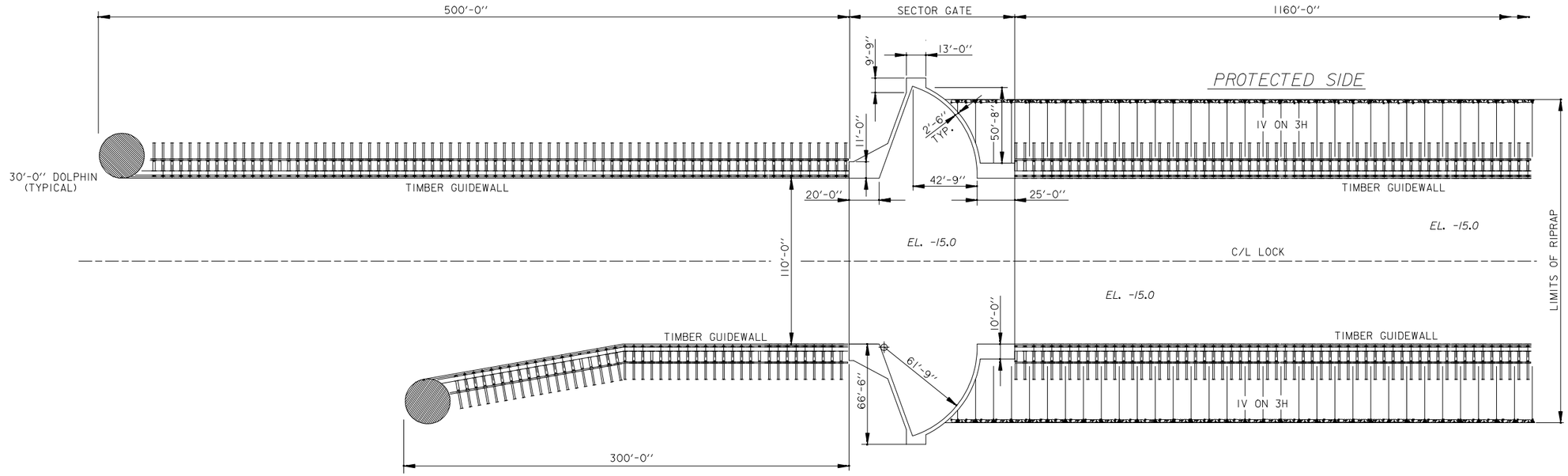
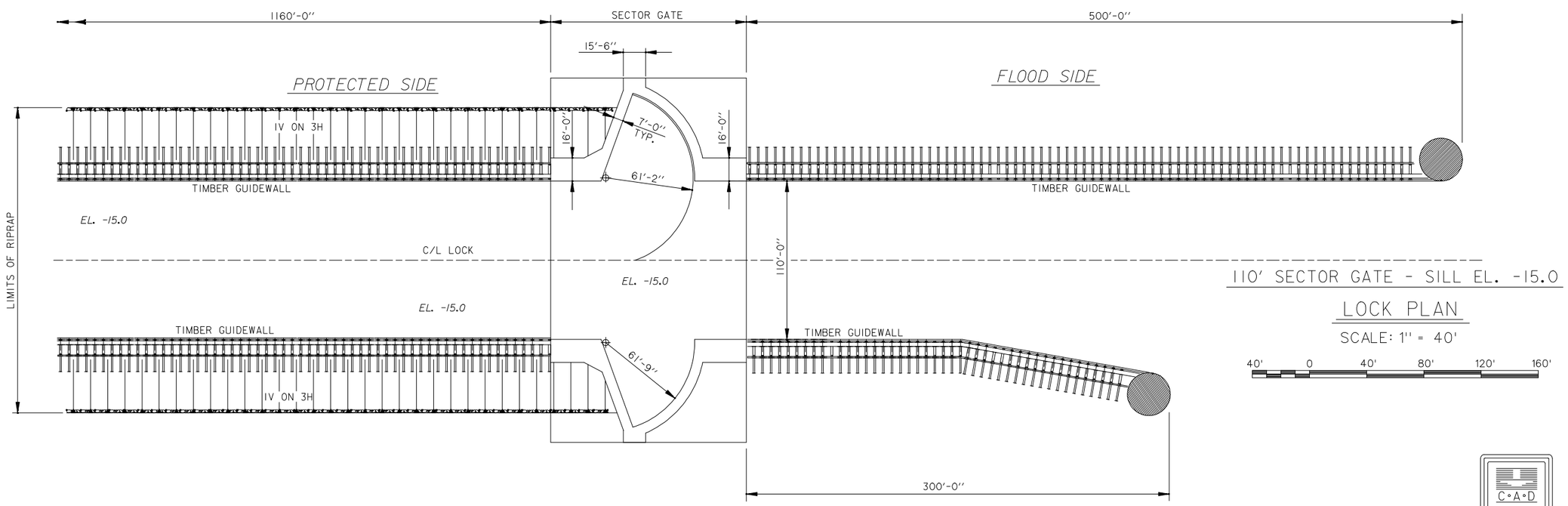
SIZE: | PLOT SCALE: | PLOT DATE:

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

BOTTOM ROLLER GATE
32' WIDE - SILL EL. 8.0
PROTECTION TO EL. 45.0

SHEET IDENTIFICATION NUMBER
PLATE S-21





110' SECTOR GATE - SILL EL. -15.0

LOCK PLAN

SCALE: 1" = 40'

40' 0 40' 80' 120' 160'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

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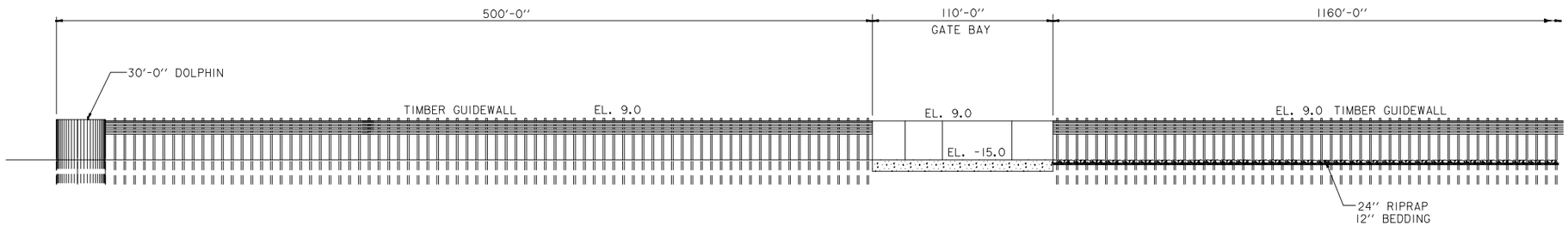
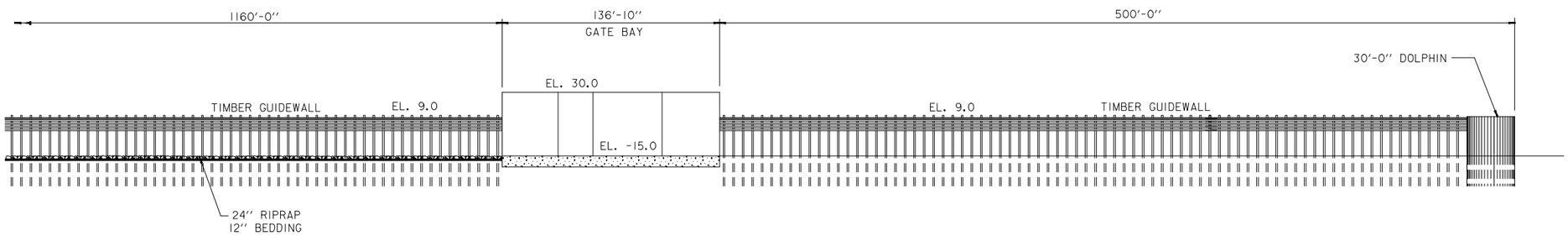
MISSISSIPPI VALLEY DIVISION

LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

LOCK PLAN
110' SECTOR GATE PROTECTION TO EL. 45.0

SHEET IDENTIFICATION NUMBER
PLATE S-22





110' SECTOR GATE - SILL EL. -15.0

LOCK PROFILE

SCALE: 1'' = 40'

40' 0 40' 80' 120' 160'

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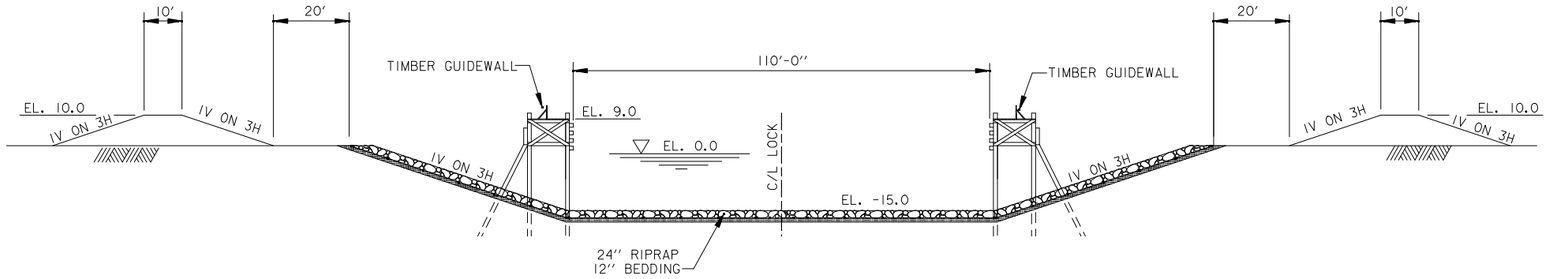
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

LOCK PROFILE 110' SECTOR GATE PROTECTION TO EL. 30.0

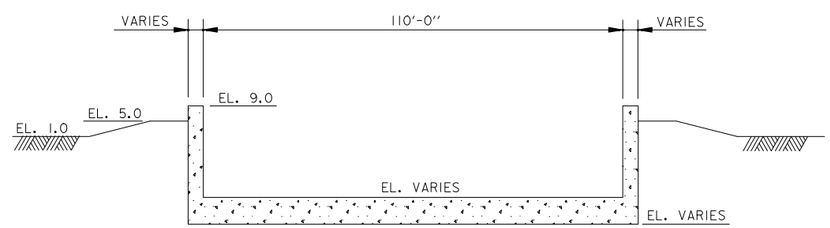
SHEET IDENTIFICATION NUMBER
PLATE S-23



EARTH CHAMBER SECTION

SCALE: 1" = 20'

20' 0 20' 40' 60' 80'



CONCRETE CHAMBER SECTION

SCALE: 1" = 20'

20' 0 20' 40' 60' 80'

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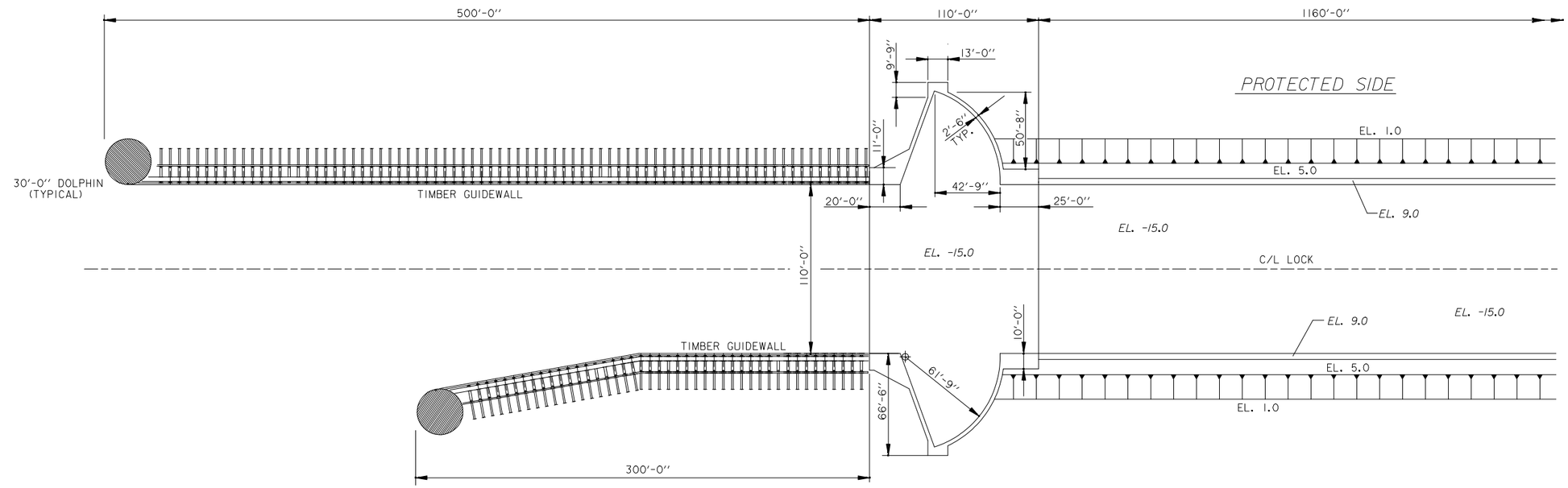
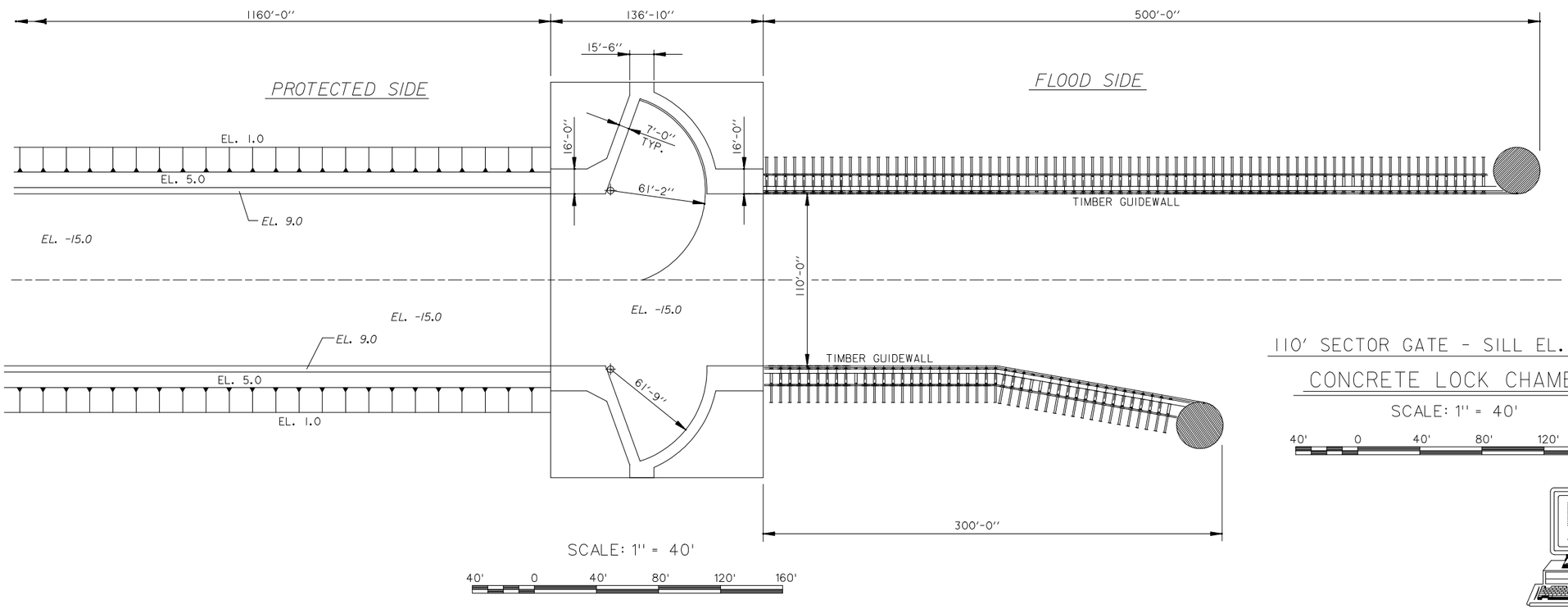
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

LOCK CHAMBER SECTIONS 110' WIDTH

SHEET IDENTIFICATION NUMBER
PLATE S-24





110' SECTOR GATE - SILL EL. -15.0

CONCRETE LOCK CHAMBER

SCALE: 1" = 40'

40' 0 40' 80' 120' 160'

US ARMY CORPS OF ENGINEERS
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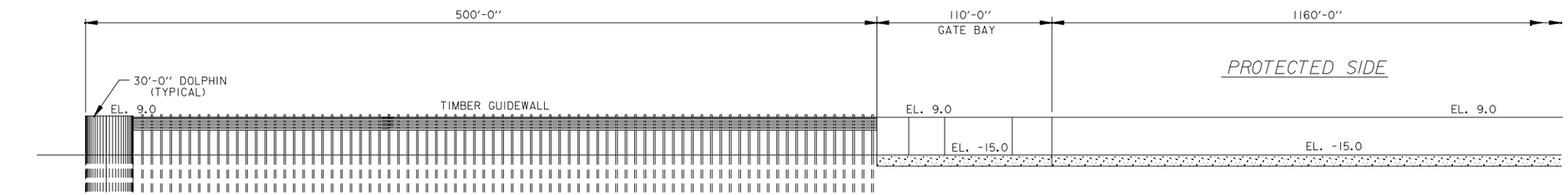
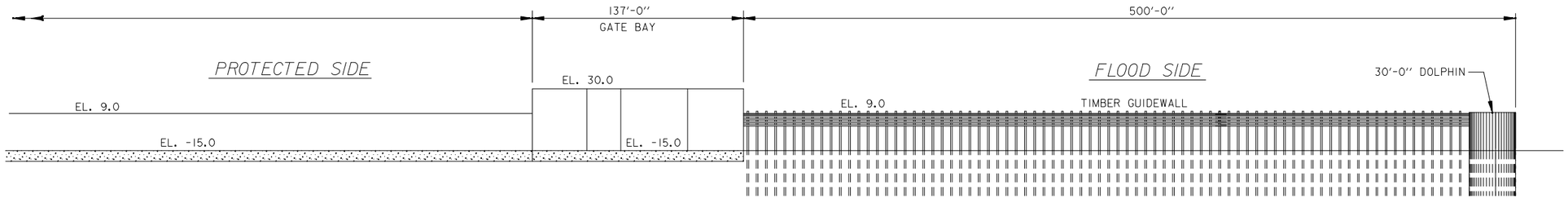
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

CONCRETE LOCK CHAMBER PLAN - 110' WIDTH PROTECTION TO EL. 30.0

SHEET IDENTIFICATION NUMBER
PLATE S-25





110' SECTOR GATE - SILL EL. -15.0

CONCRETE LOCK CHAMBER

SCALE: 1" = 40'

40' 0 40' 80' 120' 160'

US ARMY CORPS OF ENGINEERS
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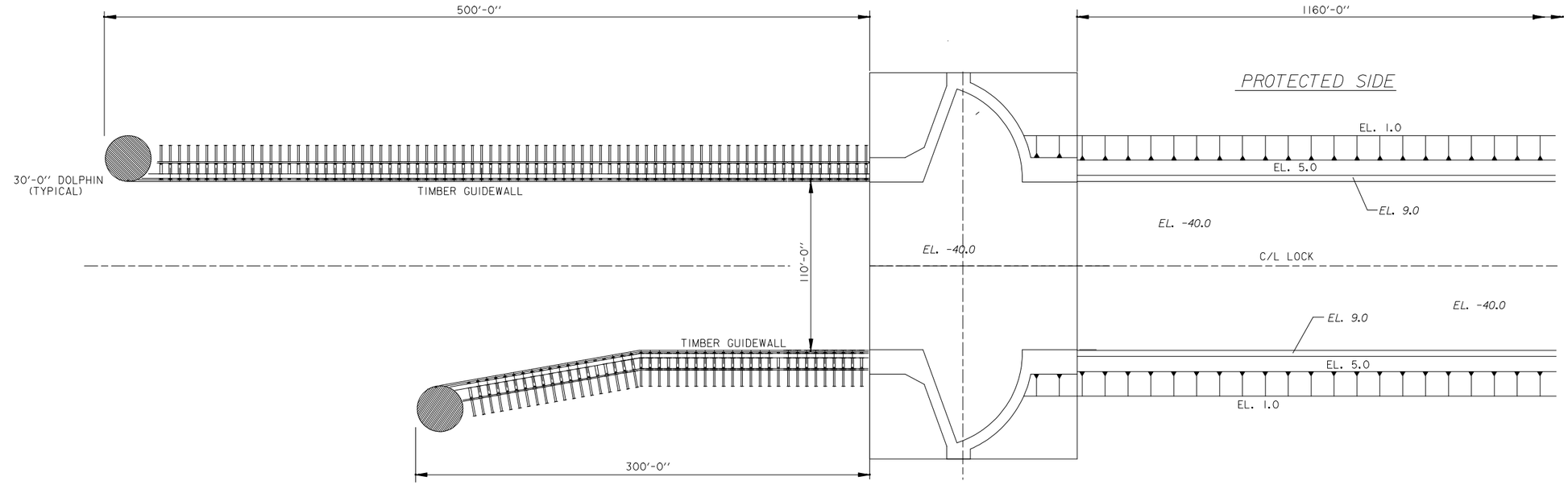
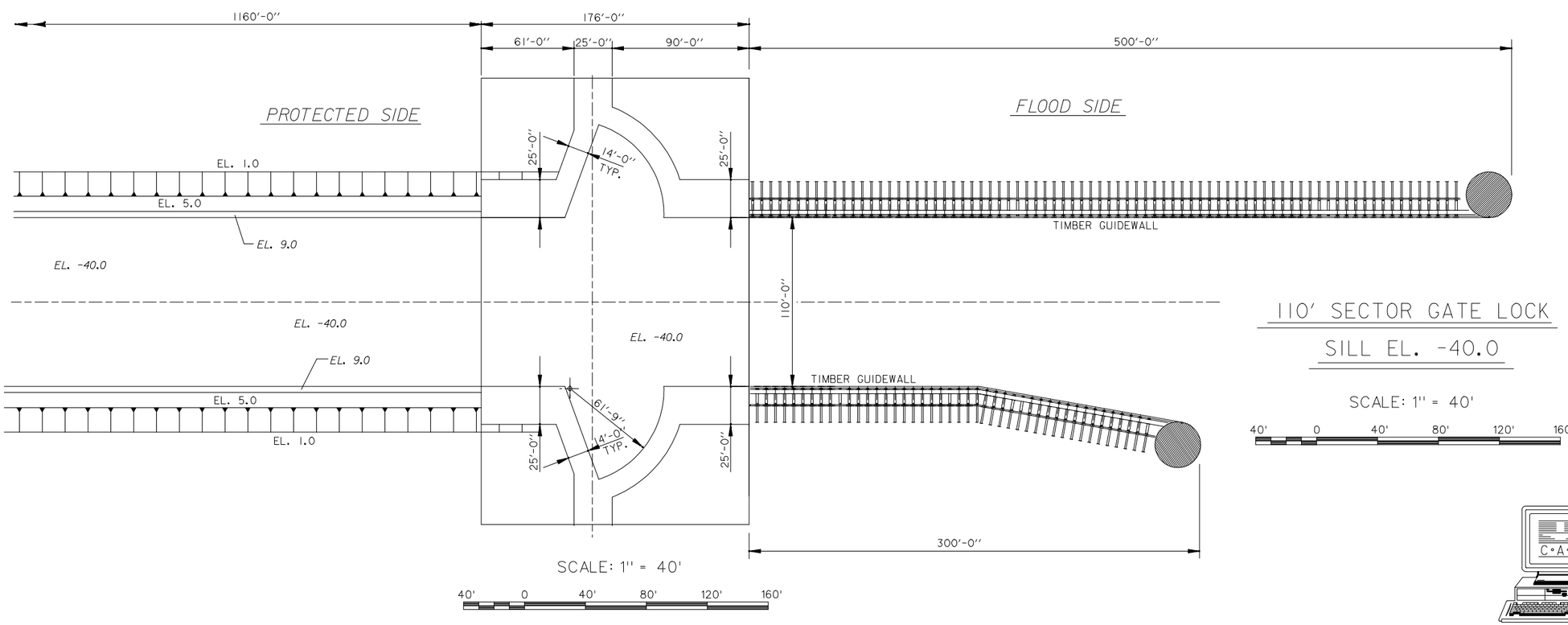
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

CONCRETE LOCK CHAMBER ELEVATION - 110' WIDTH PROTECTION TO EL. 30.0

SHEET IDENTIFICATION NUMBER
PLATE S-26





110' SECTOR GATE LOCK

SILL EL. -40.0

SCALE: 1" = 40'

40' 0 40' 80' 120' 160'

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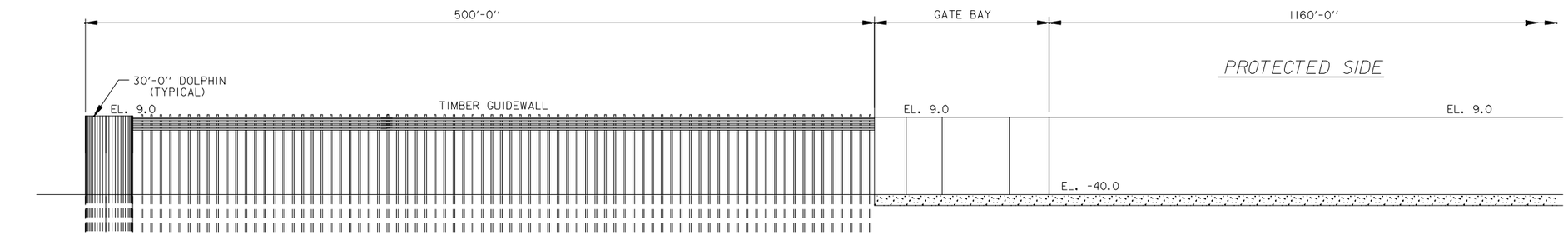
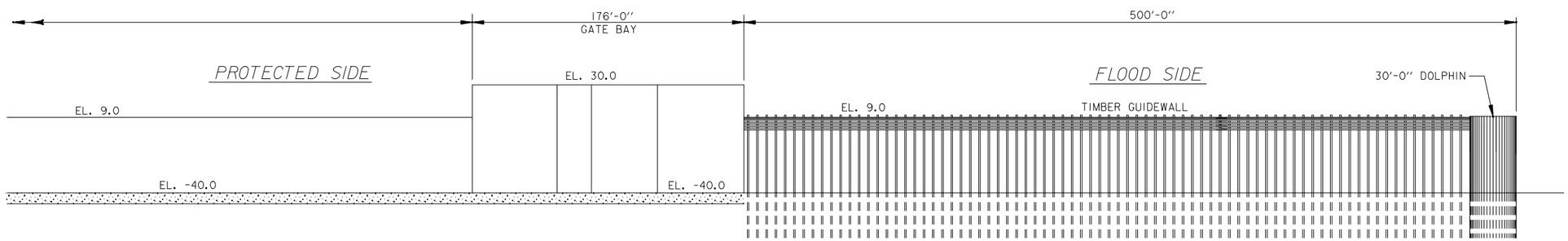
LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

CONCRETE LOCK CHAMBER PLAN - 110' WIDTH PROTECTION AT EL. 45.0

SHEET IDENTIFICATION NUMBER
PLATE S-27





110' SECTOR GATE LOCK

SILL EL. -40.0

SCALE: 1" = 40'

40' 0 40' 80' 120' 160'

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

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LOUISIANA COASTAL PROTECTION AND RESTORATION PROJECT
PRELIMINARY TECHNICAL REPORT TO CONGRESS

CONCRETE LOCK CHAMBER ELEVATION - 110' WIDTH PROTECTION TO EL. 30.0

SHEET IDENTIFICATION NUMBER
PLATE S-28

ANNEX 1 – HYDRODYNAMIC MODELING

APPENDIX L – ENGINEERING INVESTIGATIONS
ANNEX 1 – HYDRODYNAMIC MODELING

Hydrodynamic modeling of storm surge and waves for the preliminary technical report is being conducted to predict wave action at and water level response to several proposed levee alignments for input to engineering and design to assist in determination of preliminary levee height requirements and cost estimates. Due to time constraints on the preliminary report, a single screening storm was selected and simulated on ten separate tracks. It is recognized that the ten storm tracks do not provide comprehensive coverage to define water levels everywhere along the alignments. However, the results produced are augmented with engineering judgment to provide reasonable estimates for rough order of magnitude cost estimation purposes. The hydrodynamic response of the system with each proposed levee alignment is also compared to the existing condition to assist in the determination of unintended water level changes imparted by the proposed structures. This appendix describes the initial screening storm, the wind and atmospheric pressure modeling, the storm surge modeling, and the offshore and nearshore wave modeling. The model results are then used to estimate maximum waves and water level along the five levee alignments. The chapter concludes with a discussion of the potential impact of coastal features on storm surge.

Initial Screening Storm

The storm selected for rough order of magnitude cost estimation of the various alignments is based on the probable maximum hurricane (PMH) as documented in NOAA's Technical Report NWS 23 (1979). The PMH has a central pressure of 890 mb. The PMH criteria for the Louisiana coast describe a storm of Category 5 intensity on the Saffir-Simpson Scale (SSC). The radius to maximum winds was approximately 11 nm, that of Hurricane Camille, and the average forward speed applied for the dynamic solution was set at 10 knots. The PMH was run on ten historical (or modified historical) tracks with landfalls across coastal Louisiana with different approach angles. For storm surge modeling, the storms were translated both at the historical hurricane track speed and at a constant 10 knots. The tracks were selected to result in Category 5 hurricane surge levels at locations across the proposed structural alignments. The selected tracks are summarized in Table 1 and plotted in Figure 1.

The number of tracks selected was limited because of time constraints on the preliminary technical report schedule. A more comprehensive modeling assessment will be performed for the final technical report. Storm surge is a function of many factors including, but not limited to, wind speed, translation speed, landfall location, orientation of the storm track at landfall to the shoreline, and storm size. Therefore, there is a need to move away from an event-driven approach that considers only particular storms and move towards a risk-based approach that addresses how often assets and populations become inundated and how severe that inundation is for storm events of particular characteristics. Such an approach will be considered for the final technical report.

Table 1. Modeled Hurricane Tracks		
Track	**Description**	**Naming convention**
1	Hurricane Katrina	T1
2	Hurricane Andrew shifted 1.0 deg east	T2
3	1947 storm shifted 0.25 deg south	T3
4	Hurricane Rita	T4
5	Hurricane Carmen	T5
6	1915 storm	T6
7	1893 storm shifted 0.5 deg west	T7
8	Hurricane Rita shifted 1.0 deg east	T8
9	Hurricane Camille shifted 0.5 deg west	T9
10	1893 storm shifted 2.5 deg west	T10

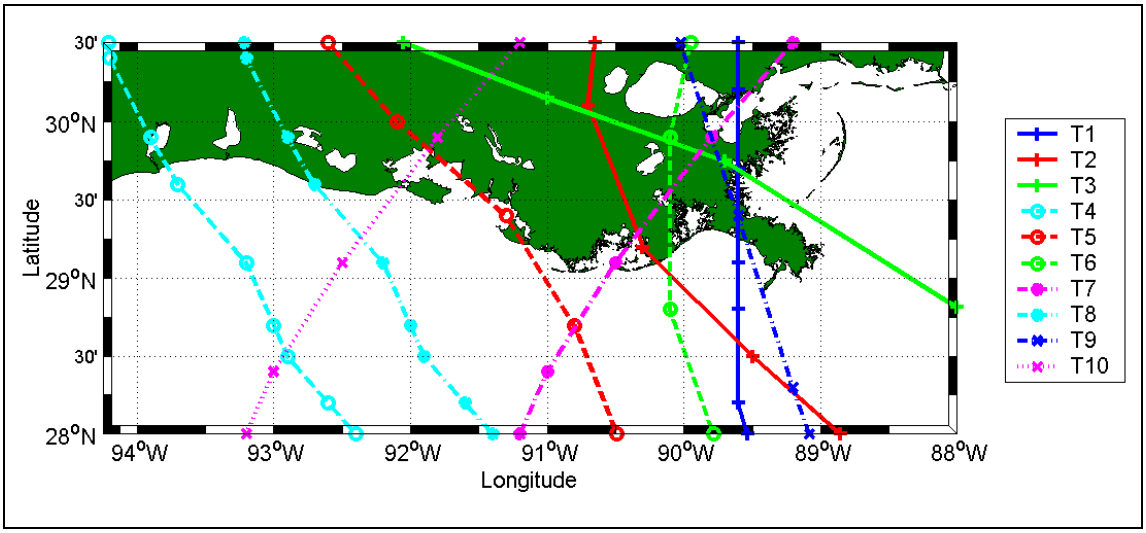

Figure 1. Selected hurricane tracks.

Wind and Atmospheric Pressure

Accurate modeling of wave and storm surge levels requires accurate wind and pressure field input to the model. This section describes the methodology to generate wind and pressure fields for the PMH which drive the storm surge simulations and the offshore and nearshore wave simulations.

Methodology

The wind and pressure fields were developed with a highly refined meso-scale vortex numerical model for the specification of surface wind and pressure fields in tropical cyclones. This model specifies the surface wind and pressure field in tropical cyclones and is referred to as the Planetary Boundary Layer Model (Thompson and Cardone 1996). Model inputs include the central pressure index (CPI), radius to maximum wind (RMW), forward velocity, and storm track locations. The inputs for the design storm for this phase of the study are given in the Initial Screening Storm

section. The dynamical model operates on these inputs and additional inputs required (defined below as calibration criteria) to produce a wind and pressure field. The simulation assumes that a hurricane is steady state offshore and does not begin to weaken until the center arrives at the coast. A filling model developed by Vickery et al. (1995), which describes the land effects in terms of CPI, was applied. The filling model takes the following form where t is time in hours and p_o is pressure in mb:

$$\Delta p(t) = \Delta p_o \exp(-at) \quad (1)$$

The filling constant a is given as:

$$a = a_o + a_1 \Delta p_o \quad (2)$$

Where a_o and a_1 are 0.035 and 0.00050 for Gulf Coast hurricanes.

Model calibration was required to explore and define additional storm criteria that are not included in the PMH criteria. The additional criteria include the pressure profile peakedness parameter, so called Holland's *B*, the ambient pressure field, peripheral pressure index, the boundary layer depth, the azimuth of the wind maximum, and the landfall-filling model described above. The calibration effort required a period of experimentation with recent Gulf of Mexico storms of intensity comparable to the PMH. Hurricanes Ivan (2004) and Katrina (2005) were selected as they affected the Louisiana coast and excellent kinematic descriptions of the wind fields already exist. The objective of the calibration is to ensure that the surface marine wind field specified is consistent with modern thinking as to the relationship between the storm criteria and inner core maximum surface winds for modern averaging intervals, including the standard 30-minute average, and the definition of "sustained" wind speed, which is a stochastic wind variable which may be defined as the median peak 1-minute wind speed within the 30-minute period.

The tropical cyclone boundary layer model was setup on a target domain that covers the range of two working grids. Winds and pressures are output on these two grid systems. The basin scale grid is 0.1 degree (~6 nm), covering the domain 18-30.8 deg N, 98-80 deg W, the fine scale grid is 0.025 degree (~1.2 nm), covering the domain 28.5-30.8 deg N, 94.25-88 deg W. Grid spacing of the fine domain is sufficient to resolve the radius of maximum wind (RMW) in the hindcast storms. Output is specified at a 15-minute time step.

Results

The maximum wind speed generated over space and time is approximately 135 mph. This speed is based on a 10-m equivalent neutrally stable, 30-minute average wind speed. If the maximum wind speed is converted to a 1-minute average (the general average interval to quantify the Saffir-Simpson Hurricane Scale) the magnitude would be about 166 mph (a ratio of 1.23) or an intense Category 5 hurricane. The final 30-minute wind and pressure fields provided input to the surge and wave models.

Storm Surge Modeling

The ADvanced CIRCulation (ADCIRC) hydrodynamic model (Leuttich et al. 1992) is being applied to estimate storm surge generated by the PMH. ADCIRC is a finite-element hydrodynamic circulation numerical model for the simulation of water level and current over an unstructured gridded domain. ADCIRC is a two-dimensional depth-integrated (2DDI) model that can simulate tide-, wind- and wave-driven circulation in coastal waters as well as hurricane storm surge and flooding. Extensive storm surge model development, application, and validation efforts have been conducted in southern Louisiana. Storm surge modeling capabilities within a physics-based framework have been improved through this work (Westerink et al. 2005, Feyen et al. 2005).

Computational Model

ADCIRC was chosen for simulating the long-wave hydrodynamic processes in the study area. Imposing the wind and atmospheric pressure fields, the ADCIRC model can replicate tide-induced and storm-surge water levels and currents. In two dimensions, the model is formulated with the depth-averaged shallow-water equations for conservation of mass and momentum. Furthermore, the formulation assumes the water is incompressible, hydrostatic pressure exists, and the Boussinesq approximation is valid. Using the standard quadratic parameterization for bottom stress and neglecting baroclinic terms and lateral diffusion/dispersion effects, the following set of conservation equations in primitive, nonconservative form, and expressed in a spherical coordinate system, are incorporated in the model (Flather 1988; Kolar et al. 1993):

$$\frac{\partial U}{\partial t} + \frac{1}{r\cos\phi} U \frac{\partial U}{\partial \lambda} + \frac{1}{R} V \frac{\partial U}{\partial \phi} - \left[\frac{\tan\phi}{R} U + f \right] V = - \frac{1}{R\cos\phi} \frac{\partial}{\partial \lambda} \left[\frac{p_s}{\rho_0} + g(\zeta - \eta) \right] + \frac{\tau_{s\lambda}}{\rho_0 H} - \tau_* U \tag{3}$$

$$\frac{\partial V}{\partial t} + \frac{1}{r\cos\phi} U \frac{\partial V}{\partial \lambda} + \frac{1}{R} V \frac{\partial V}{\partial \phi} - \left[\frac{\tan\phi}{R} U + f \right] U = - \frac{1}{R} \frac{\partial}{\partial \phi} \left[\frac{p_s}{\rho_0} + g(\zeta - \eta) \right] + \frac{\tau_{s\lambda}}{\rho_0 H} - \tau_* V \tag{4}$$

$$\frac{\partial \zeta}{\partial t} + \frac{1}{R\cos\phi} \left[\frac{\partial UH}{\partial \lambda} + \frac{\partial (UV\cos\phi)}{\partial \phi} \right] = 0 \tag{5}$$

where

t = time,

λ and φ = degrees longitude (east of Greenwich is taken positive) and degrees latitude (north of the equator is taken positive),

ζ = free surface elevation relative to the geoid,

U and V = depth-averaged horizontal velocities in the longitudinal and latitudinal directions, respectively,

R = the radius of the earth,

$H = \zeta + h$ = total water column depth,

h = bathymetric depth relative to the geoid,

$f = 2\Omega \sin \varphi$ = Coriolis parameter,

Ω = angular speed of the earth,

p_s = atmospheric pressure at free surface,

g = acceleration due to gravity,

η = effective Newtonian equilibrium tide-generating potential parameter,

ρ_0 = reference density of water,

$\tau_{s\lambda}$ and $\tau_{s\varphi}$ = applied free surface stresses in the longitudinal and latitudinal directions, respectively, and

τ = bottom shear stress and is given by the expression $C_f(U^2 + V^2)^{1/2} /H$ where C_f is the bottom friction coefficient.

The momentum equations (Equations 3 and 4) are differentiated with respect to λ and t and substituted into the time differentiated continuity equation (Equation 5) to develop the following Generalized Wave Continuity Equation (GWCE):

$$\frac{\partial^2\zeta}{\partial t^2} + \tau_0\frac{\partial\zeta}{\partial t} - \frac{1}{R\cos\phi}\frac{\partial}{\partial\lambda}\left[\frac{1}{R\cos\phi}\left(\frac{\partial HUU}{\partial\lambda} + \frac{\partial(HUV\cos\phi}{\partial\phi}\right) - UVH\frac{\tan\phi}{R}\right]$$

$$\left[-2\omega\sin\phi\, HV + \frac{H}{R\cos\phi}\frac{\partial}{\partial\lambda}\left(g(\zeta - \alpha\eta) + \frac{p_5}{\rho}p_0\right) + \tau_* HU - \tau_0 HU - \tau_{s\lambda}\right]$$

$$-\frac{1}{R}\frac{\partial}{\partial\phi}\left[\frac{1}{R\cos\phi}\left(\frac{\partial HVV}{\partial\lambda} + \frac{\partial HVV\cos\phi}{\partial\phi}\right) + UUH\frac{\tan\phi}{R} + 2\omega\sin\phi\, HU\right] \qquad (6)$$

$$+\frac{H}{R}\frac{\partial}{\partial\phi}\left(g(\zeta - alfa\eta) + \frac{p_s}{\rho_0}\right) + \tau_* - \tau_0 HV - \frac{\tau_{s\lambda}}{\rho_0}]$$

$$-\frac{\partial}{\partial t}[\frac{VH}{R}\tan\phi] - \tau_0[\frac{VH}{R}\tan\phi] = 0$$

The ADCIRC model solves the GWCE in conjunction with the primitive momentum equations given in Equations 3 and 4. The GWCE-based solution scheme eliminates several problems associated with finite-element programs that solve the primitive forms of the continuity and momentum equations, including spurious modes of oscillation and artificial damping of the tidal signal. Forcing functions include time-varying water-surface elevations, wind shear stresses, and atmospheric pressure gradients.

The ADCIRC model uses a finite-element algorithm in solving the defined governing equations over complicated bathymetry encompassed by irregular sea/shore boundaries. This algorithm allows for extremely flexible spatial discretizations over the entire computational domain and has demonstrated excellent stability characteristics. The advantage of this flexibility in developing a computational grid is that larger elements can be used in open-ocean regions where less resolution is needed, whereas smaller elements can be applied in the nearshore and estuary areas where finer resolution is required to resolve hydrodynamic details.

Methodology

The ADCIRC grid utilized during this study is that which was calibrated during the IPET Task 4 Hurricane Katrina study (Figure 2). The model incorporates the western North Atlantic Ocean, the Gulf of Mexico and Caribbean Sea to allow for full dynamic coupling between oceans, continental shelves, and the coastal floodplain. The grid is locally refined to resolve features such as inlets, rivers, navigation channels, levee systems and local topography/bathymetry. The grid applied and validated for the IPET Hurricane Katrina hindcast studies served as the existing condition for this study. An example of the calibration and validation analysis is provided as Attachment A to this appendix. A complete description of the calibration and validation is found in Volume 4 of the IPET final report. The base grid was modified for each of the five proposed levee alignments and the levee was set at a height to ensure that no overtopping occurred.

The storm surge modeling consists of 84 ADCIRC model simulations. All tracks were simulated on the existing condition grid. Combinations of levee alignments and PMH storm tracks were selected to produce the estimates of maximum surge, wave and runup for the proposed coastal Louisiana alignments. The alignments are similar along many sections of the Louisiana coast and not all storm tracks were required to be run for all alignments. Table 2 summarizes the levee alignment/storm track combinations modeled. The storms were run at both the historical translation speed and at a constant 10 knot translation speed.

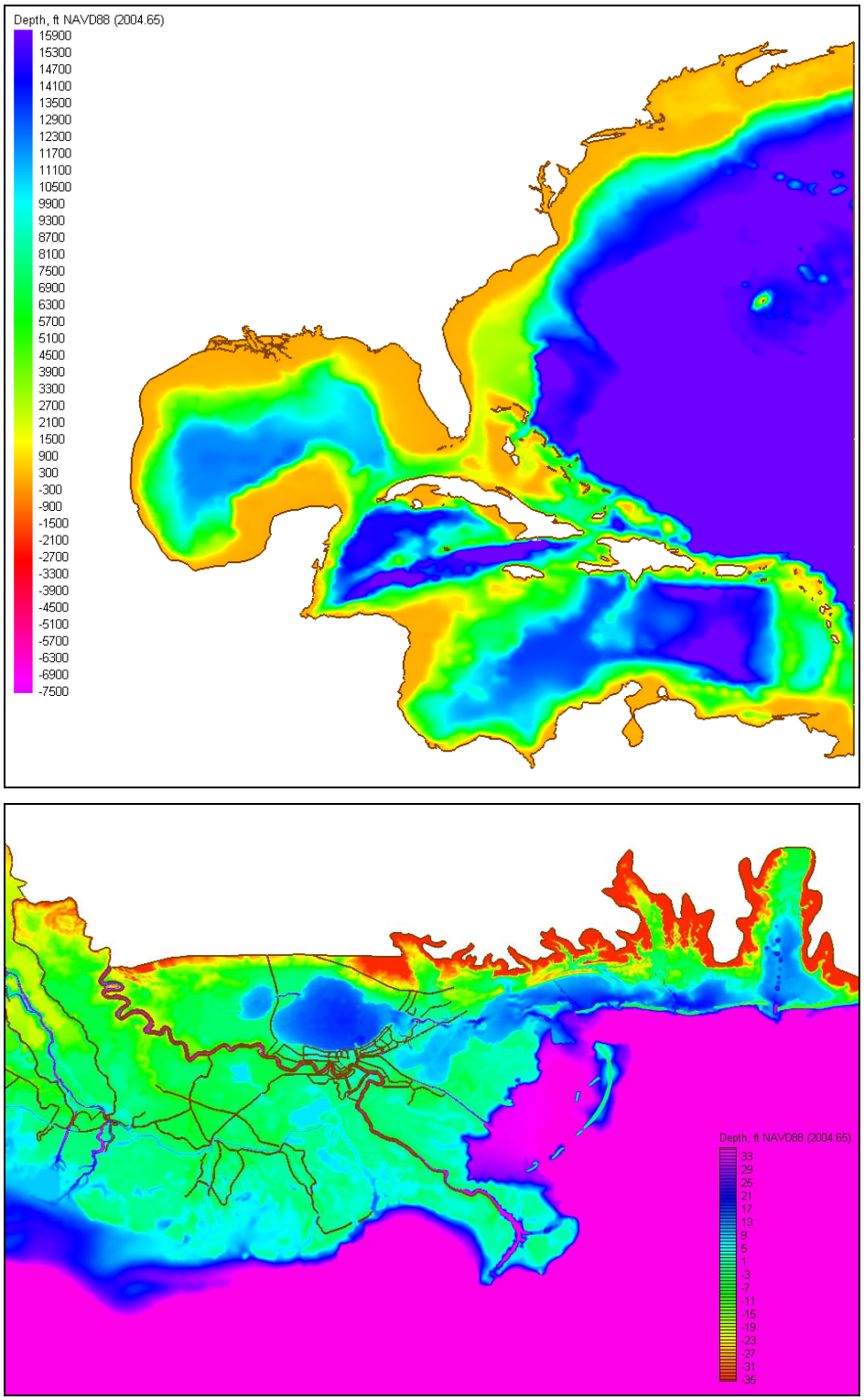

Figure 2. a. Computational domain. b. Detail of bathymetry and topography across southern Louisiana.

Table 2
Levee Alignment/Storm Track Combinations Modeled with ADCIRC

Storm Track	Alignment 1	Alignment 2	Alignment 3	Alignment 4	Alignment 5
T1	X	X	X	X	X
T2	X		X	X	X
T3	X	X			X
T4	X				
T5	X				X
T6	X	X	X	X	X
T7	X	X	X	X	X
T8	X				
T9	X	X	X	X	X
T10	X				

Results

Hurricane surge water surface elevations were generated for each combination specified in Table 2. Peak water surface elevations for each levee alignment were obtained by taking the maximum surge elevation from all ten storm tracks. The storm surge can be sensitive to the hurricane translation speed as a slower moving storm allows more time for the winds to push the water toward the coast. Therefore, the storms were run at both historical translation speeds and at a constant ten knots. The peak surges for the existing condition and each alignment for all the simulated storms are plotted in Figures 3-8. Maximum surge elevations along the proposed levee alignments range from approximately 13 to 20 ft in Lake Pontchartrain to about 30 to 40 ft elsewhere along the coast. Track T10 produced the most severe conditions along the central and westerly reaches. Track T3 produced the worst conditions for the south shore of Lake Pontchartrain and on the easterly reaches north of English Turn while track T7 generated the greatest surge at the levees south of English Turn. Tracks T2 and T7 produced the most severe conditions in the Barataria reach and track T4 produced the most extreme conditions on the far western reaches.

The existing condition levees are overtopped and the placement of a high levee increases surge elevations by as much as approximately 15 ft for the storm and tracks simulated. The blocked water rises on the levee and spreads along the levee where possible. For the screening storm, the model predicts water levels increase less than 1ft along the Mississippi coast due to the presence of the alignments modeled. However, the impact to Mississippi could be greater for storms with a larger radius to maximum winds (such as Hurricane Katrina) and additional analysis is required. Model results also indicate that surges tend to be larger in levee “pockets”, areas along the levee alignment with acute angles, sometimes termed the “funnel” effect. The levees in these areas confine the surge, not allowing it to spread along the coast. This effect is seen on a larger scale east of the river as the geography of the area “catches” the surge of hurricanes making landfall to the east. The sensitivity of surge potential is intensified on the south Louisiana coast due to the complex geometry of the levees and the coast. West of the river, surge potential is greatest for storms approaching from the southwest or perpendicular to the shoreline. The storm winds approaching from these angles push water toward the coast prior to landfall while southeast storms tend to push water away from the coast as they approach.

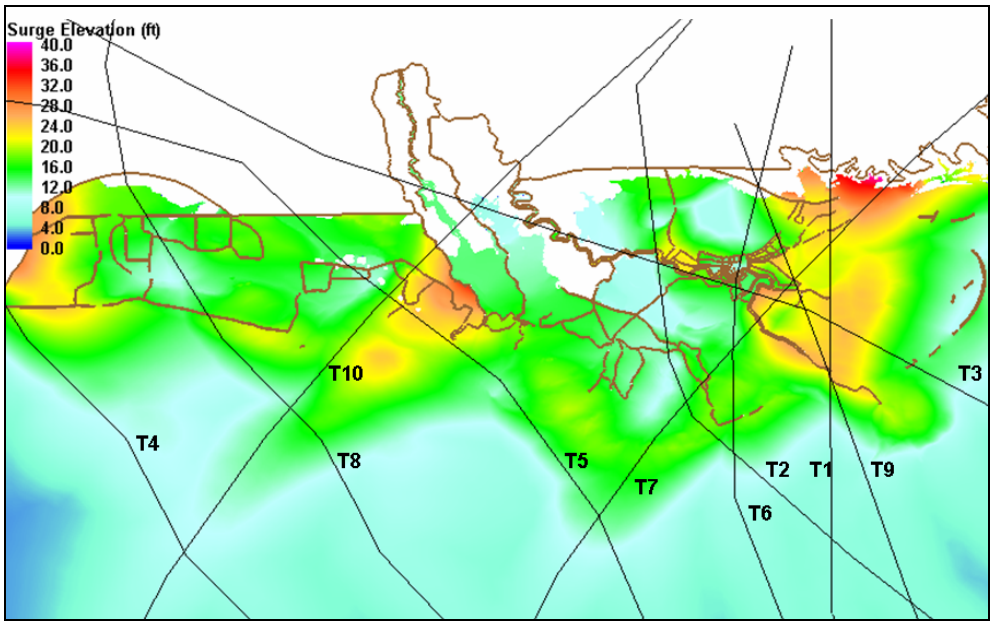

Figure 3. Peak surges for all storm tracks run on the existing condition.

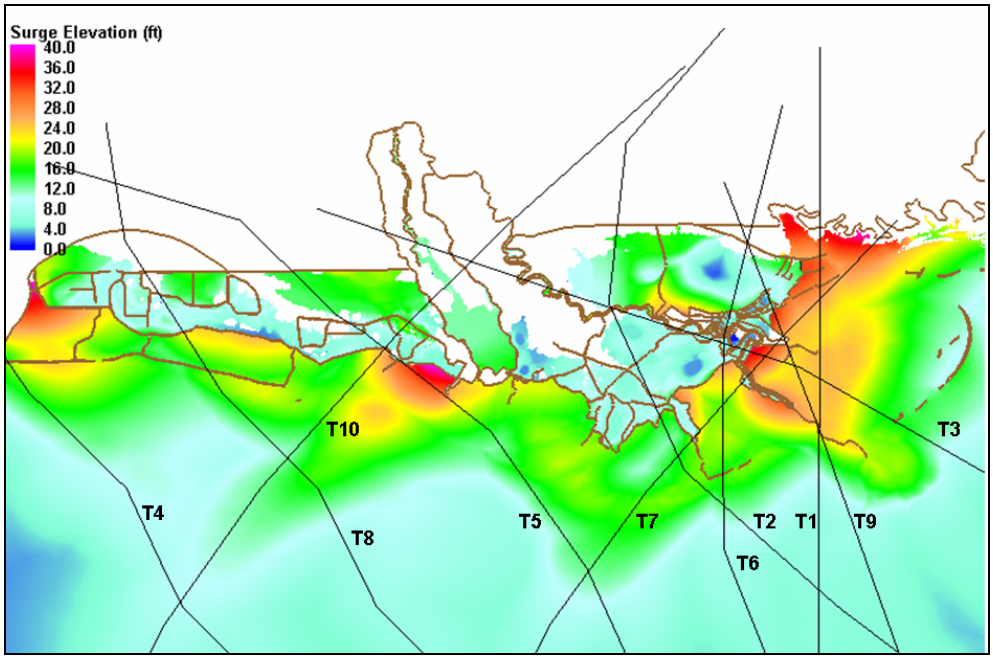

Figure 4. Peak surges for all storm tracks run on Alignment 1.

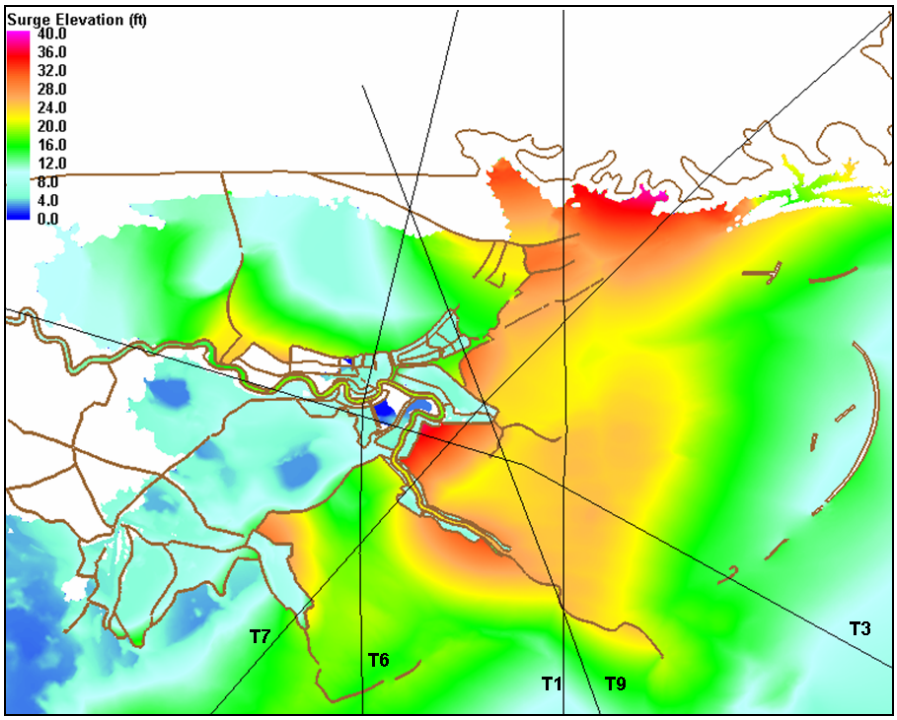

Figure 5. Peak surges for all storm tracks run on Alignment 2.

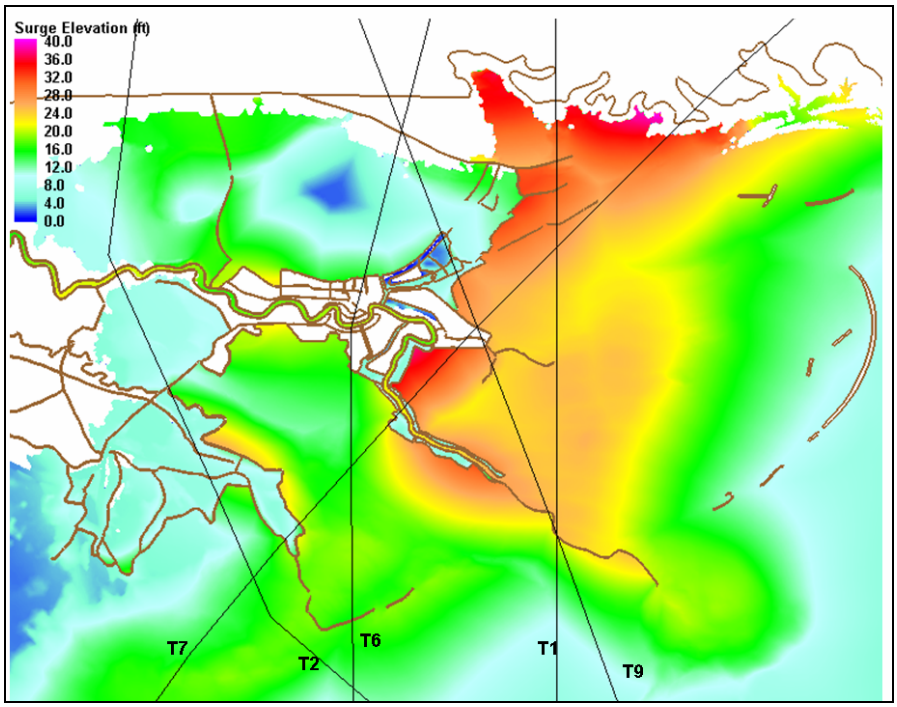

Figure 6. Peak surges for all storm tracks run on Alignment 3.

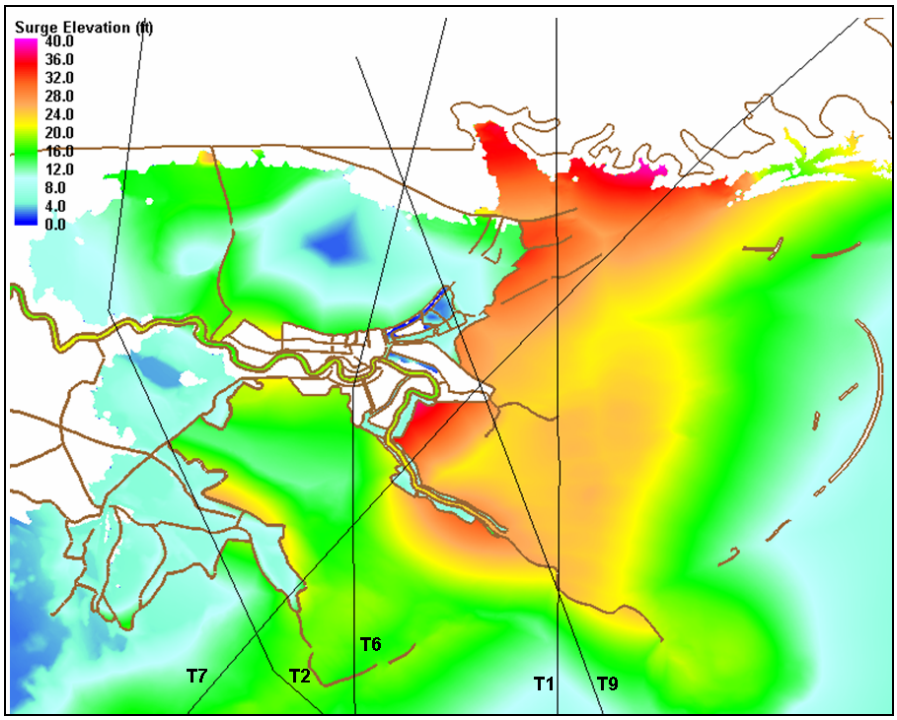

Figure 7. Peak surges for all storm tracks run on Alignment 4.

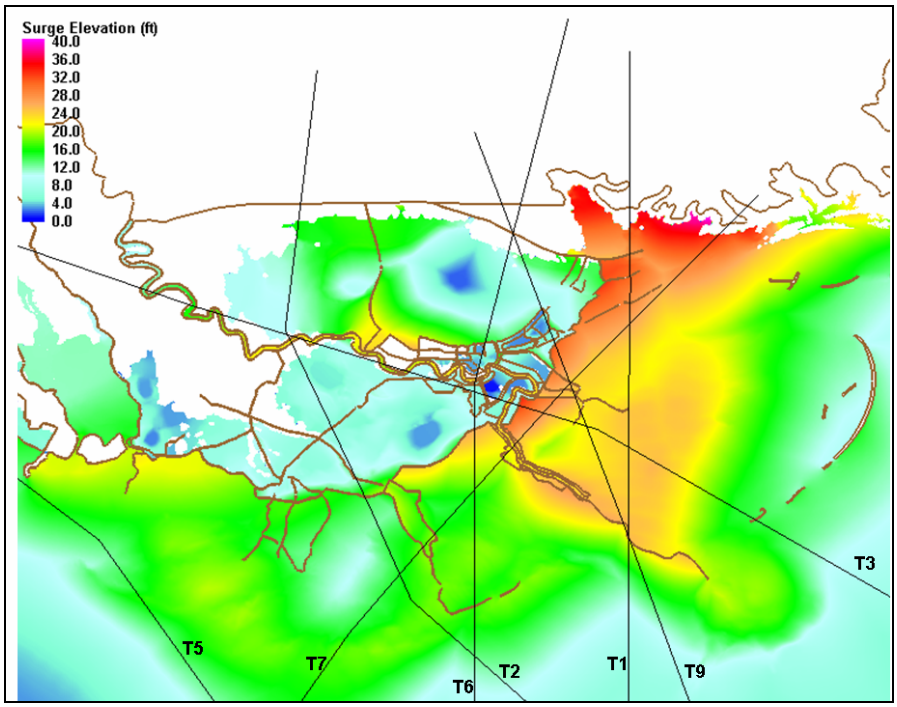

Figure 8. Peak surges for all storm tracks run on Alignment 5.

Surge Modeling Sensitivity

Sensitivity runs were performed to compare the surges estimated by the model for the PMH to historical storms to put the estimated surges for the screening storm into a historical context and to

investigate the dependency of intense hurricane surge on drag law specification. The surges estimated by the model for the PMH track T1 were compared to that calculated for Hurricane Katrina and the PMH surges on track T9 were compared to those estimated for Hurricane Camille on track T9. Difference plots for both Hurricane Katrina and Hurricane Camille on track T9 are given in Figures 9 and 10, respectively. Some of the stretches of the Louisiana levees most impacted by T1 and T9 are the south shore of Lake Pontchartrain and from Slidell to English Turn. Relative to Hurricane Katrina, the PMH surges were 1 to 2 ft lower from Slidell to English Turn and surges were 3 to 5 ft higher along the south shore of Lake Pontchartrain. Relative to a Hurricane Camille wind field translated along the same track, the PMH generated surges 1 to 3 ft higher from Slidell to English Turn and surges were 3 to 5 ft higher along the south shore of Lake Pontchartrain.

Sensitivity runs were also made to investigate the dependency of intense hurricane surge on drag law specification. The standard method for applying surface wind stress within storm surge models, such as ADCIRC, is the quadratic stress law via a surface drag coefficient $C_{w.}$ This coefficient is based on regression fits of field measurements, under conditions of moderate to strong wind speed, and has been found to be directly related to wind speed, wave state and atmospheric stability (Garratt, 1977, Large and Pond, 1981 and Trenberth et. al. 1989). Recent research (Powell, 2003) has found that under extreme winds, the linear extrapolation of the drag coefficient provides a clear overestimate of C_w and that the enforcement of a drag coefficient limit may be appropriate. The PMH storm generates extremely high winds and an upper limit of 0.040 for the drag coefficient was applied for the model simulations presented above. To assess the sensitivity of results to this parameter, a preliminary investigation into the dependency of hurricane surge on the drag law specification was investigated by specifying drag coefficient upper limits of 0.025 and 0.030 and applying no cutoff.

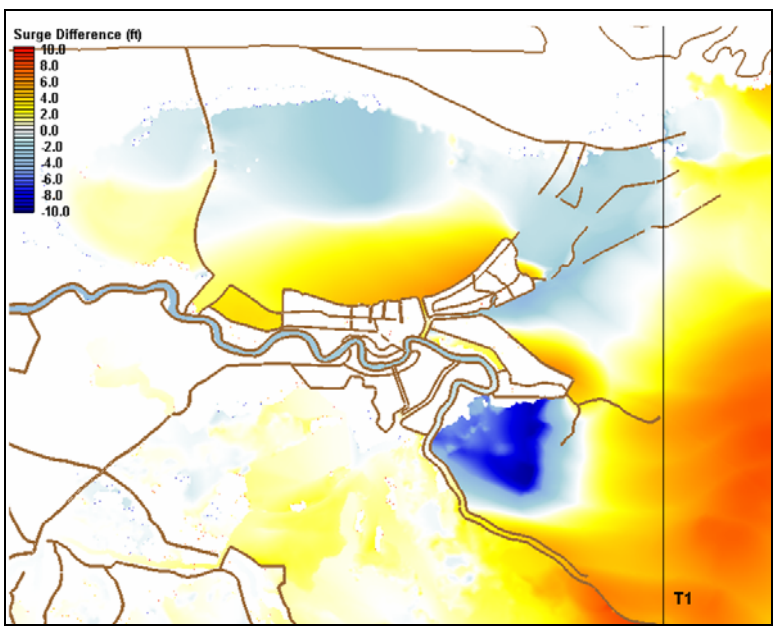

Figure 9. Surge difference between PMH on T1 and Hurricane Katrina (PMH-Katrina).

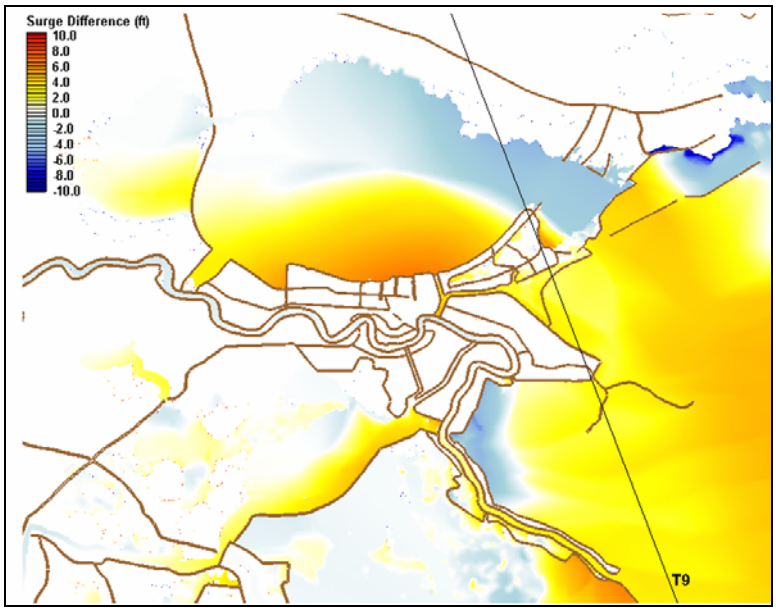

Figure 10. Surge difference between PMH on T9 and Hurricane Camille on T9 (PMH-Camille).

The sensitivity was performed by simulating the screening storm on track T1 for Alignment 1 with the various drag coefficient cutoffs. The stretches of levee most impacted by T1 are those along the south shore of Lake Pontchartrain, from Slidell to English Turn, and the levees east of the river in lower Plaquemines Parish. The estimated peak surges for these areas are presented in Table 3. relative to the simulation with no cutoff and difference plots are presented in Figures 11 to 13. The surge estimates show little sensitivity between the no cutoff and C_w =0.040 cases. The C_w =0.025 is an extreme case, reducing surges 1 to 5 ft at the levees. In offshore wave modeling, drag cutoffs of 0.030 to 0.040 are typically employed. The issue of limits on the specification of the drag coefficient for use in computing hurricane storm surge simulation will be investigated further during the second phase of this study.

Table 3.
Drag Cutoff Coefficient Sensitivity Analysis

Location	C_w=0.040 – No Cutoff (ft)	C_w=0.030 – No Cutoff (ft)	C_w=0.025 – No Cutoff (ft)
South Shore	-0.5 to -1	-1 to -2.5	-1 to -4
Slidell to English Turn	0	-1 to -1.5	-1.5 to -2.5
Plaquemines	-0.5 to -1	-2 to -3	-3 to -5

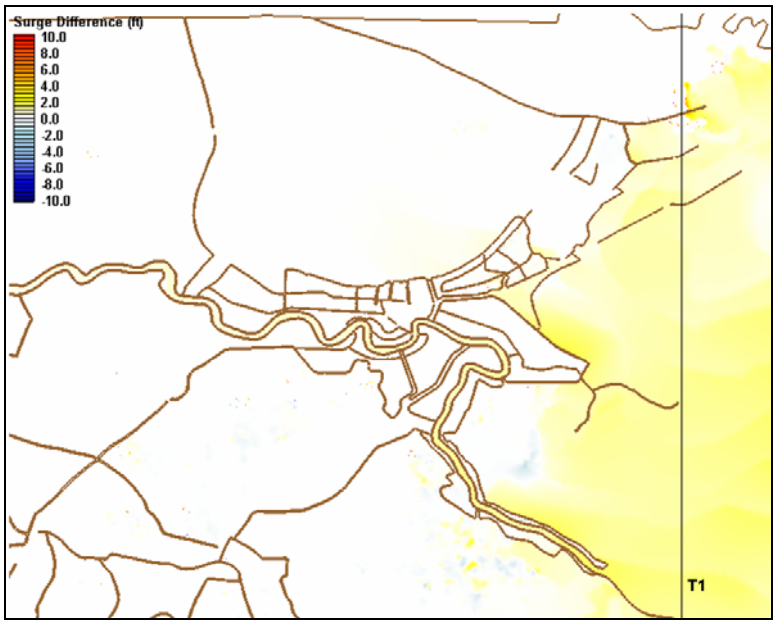

Figure 11. Surge difference between PMH on T1 applying no drag cutoff and C_w=0.040 (No cutoff - C_w=0.040).

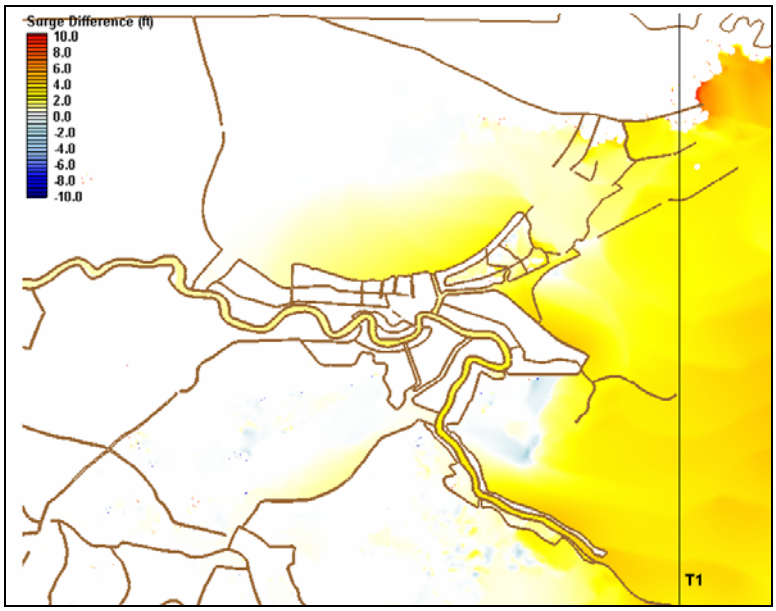

Figure 12. Surge difference between PMH on T1 applying no drag cutoff and C_w=0.030 (No cutoff - C_w=0.030).

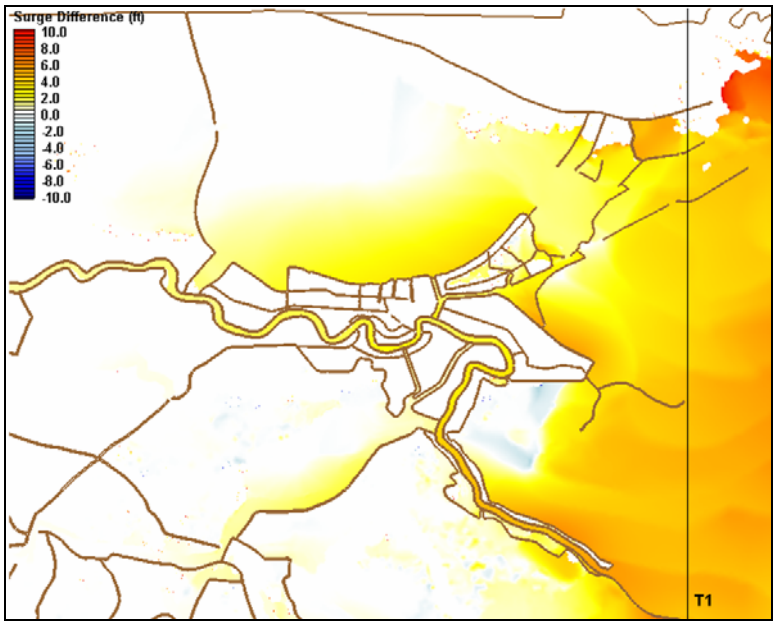

Figure 13. Surge difference between PMH on T1 applying no drag cutoff and C_w=0.025 (No cutoff- C_w=0.025).

Offshore Wave Modeling

The generation of the wave field and directional wave spectra for the various hurricane storm tracks is based on the implementation of a third generation discrete spectral wave model called WAM (Komen et al. 1994). This model solves the action balance equation:

$$\frac{\partial N}{\partial t} + \vec{c_G} \cdot \frac{\partial N}{\partial \vec{x}} = \omega^{-1} \cdot \sum_i S_i \qquad (7)$$

where: N is the action density defined by $F(f,\theta,x_i,t)/\omega$, where F is the energy density spectrum defined in frequency (f), direction (θ), over space (x_i), and time (t). The radial frequency (ω) is equal to $2\pi f$. S_i represent the source-sink terms:

$$\sum_i S_i = S_{in} + S_{nl} + S_{ds} + S_{w-b} + S_{bk} \qquad (8)$$

and S_{in} is the atmospheric input, S_{nl} represents the nonlinear wave-wave interactions, S_{ds} is the high frequency breaking (white-capping), $S_{w\text{-}b}$ is wave bottom effects (bottom friction), and S_{bk} is depth-limited wave breaking. The solution is solved for the spatial and temporal variation of wave action in frequency and direction over a fixed longitude-latitude geospatial grid.

Computationally Equation 7 is solved in two steps. The advection term (second term in Equation 1) is solved first accounting for the propagation of wave energy. Each packet of energy in frequency and direction is propagated based on the group speed for the frequency band and water depth. This assumes linear theory and superposition of wave packets. In a fixed longitude latitude grid system curvature effects are resolved where the energy is propagated in a spherical coordinate

system (or along great circle paths). As the water depth decreases, the full dispersion relationship is applied and wave shoaling and refraction effects the propagation of the energy packets.

After every propagation step the solution to the time rate change of the action density is solved including the source term integration. The wind field is read and the atmospheric input source (S_{in}) is applied. The nonlinear wave-wave interaction source term is the mechanism that self-stabilizes the spectral energy, transferring portions of the energy to the forward face and high-frequency tail. Dissipation (S_{ds}) removes portions of energy that become too energetic for the given frequency band. For application in arbitrary depths energy is removed via the wave-bottom sink ($S_{w\text{-}b}$) and ultimately in very shallow water the spectrum loses much of its energy due to breaking (S_{bk}). A more complete theoretical derivation and formulation of the source terms can be found in Komen et al. (1994).

Methodology

A nested grid approach was applied for the offshore wave simulations. This effectively reduces the computational demand and maximizes the use of higher resolution wind estimates in the coastal area. The two grids are shown in Figure 14 and documented in Table 4.

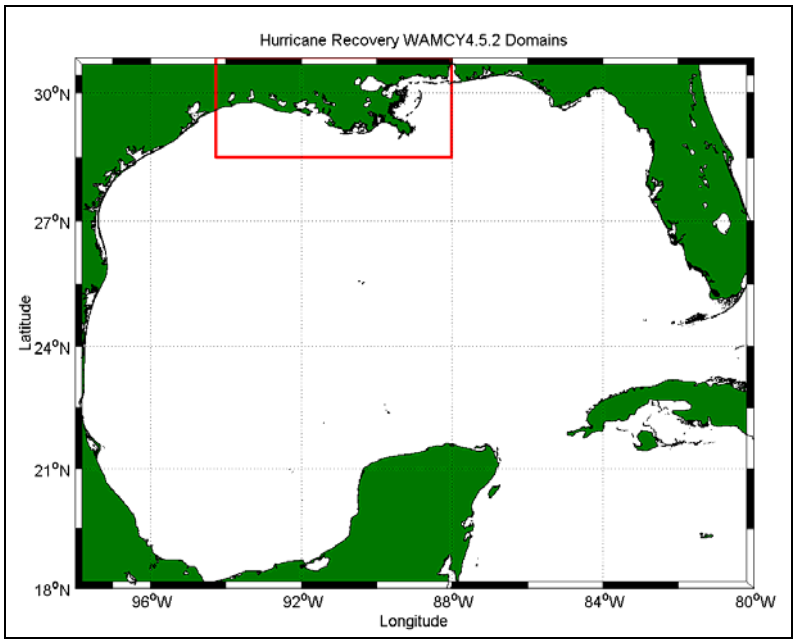

Figure 14. WAM Cycle 4.5.2 wave model grid domains, where the basin-scale is defined as the entire graphic and the region-scale is the red box.

Table 4. Wave Field Domain Characterization

Domain	Longitude (deg)		Latitude (deg)		Res. (deg)	Δt(prop) / Δt(source) (sec)
	West	East	South	North		
Basin	-98.00	-80.00	18.00	30.80	0.1	150/300
Region	-94.20	-88.00	28.50	30.50	0.05	75/300

Two time steps were applied in the wave model simulations. The propagation time step was set to attain numerical stability. The second time step for the source term integration was set to the physical processes and relaxation times of S_{in} , S_{nl} , S_{ds} , and $S_{w\text{-}b}$. In addition the time steps must be

integer multiples of the wind input, and for the fine-scale grid also evenly divisible of the basin-scale propagation time step.

All simulations were initiated from simple fetch laws using the first wind field. During the basin-scale simulation boundary condition information was generated at the defined propagation time step and consists of two-dimensional wave spectra (in frequency, and direction) along the red box defined in Figure 14. In addition wave field files were built to illustrate the time, and spatial variation of various wave parameters for each of the 10 hurricane track cases. Upon completion of each of the WAM basin-scale simulations, the regional simulations were executed. These model runs were forced with the higher resolution regional-wind fields and the boundary condition information derived from the basin level simulations.

The purpose of the offshore wave simulations is to supply the nearshore wave modeling effort supported by STWAVE (Smith et al. 2001). A total of 109 special output locations were defined in the region-scale WAM grid. The WAM directional wave spectra were output every 15 min at 28 discrete frequency bands (exponential distribution where $f_{n+1} = 1.1 \cdot f_n$ and $f_0 = 0.031384$) and 24 direction bands centered every 15 deg starting at $\theta_0 = 7.5$). These special output locations are displayed in Figure 15. The validation of the WAM was performed within the Task 4 Katrina effort. An example of the verification comparisons for wind and deep water waves is provided in Attachment B of this appendix.

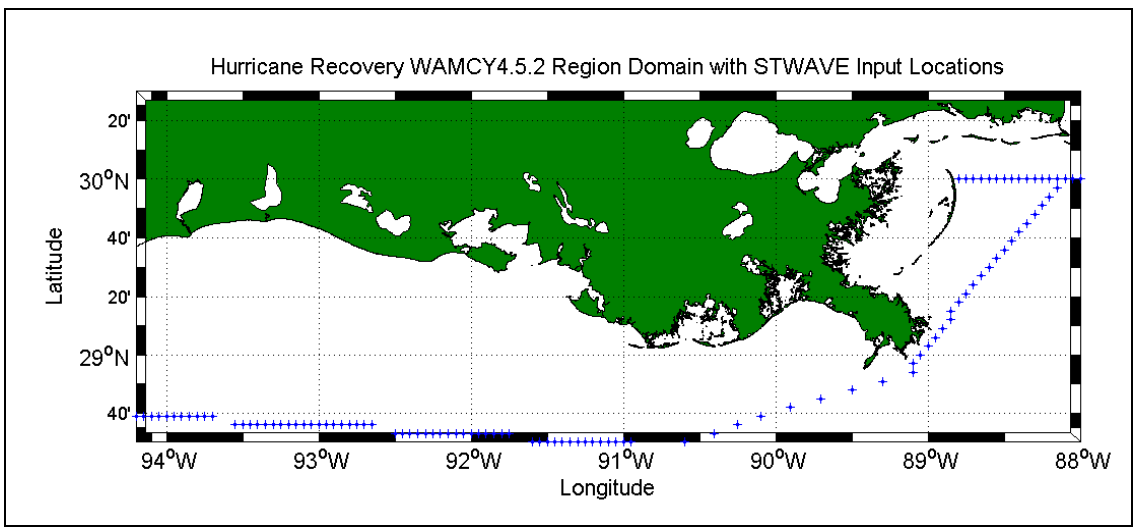

Figure 15. STWAVE output locations defined in the WAM Cycle 4.5.2 wave model regional grid domain.

Results

The wave model simulations reflect the time and spatial variation of one hurricane wind field projected onto various storm tracks. This will depict the growth and propagation of the wave energy in the target domains. A summary of the absolute maximum wave heights are documented in Table 5.

Table 5. Basin Maximum Wave Height Locations					
Trk #	Description	H_{mo} (ft)	Storm Duration (Days)	Location	
				Long	Lat
1	Hurricane Katrina	56	4.50	-87.40	24.50
2	Hurricane Andrew shifted 1.0-deg east	59	3.50	-88.20	27.80
3	1947 Storm shifted 0.25-deg south	60	3.25	-88.05	29.10
4	Hurricane Rita	57	5.00	-85.80	24.50
5	Hurricane Carmen	56	4.25	-90.10	26.80
6	1915 Storm	56	4.75	-88.40	26.10
7	1893 Storm shifted 1.0-deg east	56	6.50	-86.70	25.60
8	Hurricane Rita shifted 1.0-deg east	55	4.75	-90.50	27.30
9	Hurricane Camille shifted 0.5-deg west	57	4.25	-89.10	28.60
10	1893 Storm shifted 2.5-deg west	53	6.50	-92.60	24.60

The analysis continues into the Region-scale modeling domain. The overall maximum H_{mo} estimates are provided for each of the 10 hurricane track simulations are displayed in Table 6 and a plot of the location where the maxima occur are presented in Figure 16. Note that the southern extent and input boundary condition for the Region-scale WAM Cycle 4.5.2 is located at 28.5-deg Latitude. Six (4 through 8, and 10) of the ten wave model simulations place the maximum wave height on the input boundary to the regional domain.

Table 6. Region Maximum Wave Height Locations					
Trk #	Description	H_{mo} (ft)	Storm Duration (Days)	Location	
				Long	Lat
1	Hurricane Katrina	54	4.50	-89.40	28.65
2	Hurricane Andrew shifted 1.0-deg east	57	3.50	-89.25	28.55
3	1947 Storm shifted 0.25-deg south	60	3.25	-88.15	29.15
4	Hurricane Rita	48	5.00	-92.60	28.50
5	Hurricane Carmen	48	4.25	-90.40	28.50
6	1915 Storm	55	4.75	-89.80	28.50
7	1893 Storm shifted 1.0-deg east	45	6.50	-90.50	28.50
8	Hurricane Rita shifted 1.0-deg east	48	4.75	-91.60	28.50
9	Hurricane Camille shifted 0.5-deg west	58	4.25	-89.15	28.75
10	1893 Storm shifted 2.5-deg west	49	6.50	-92.70	28.50

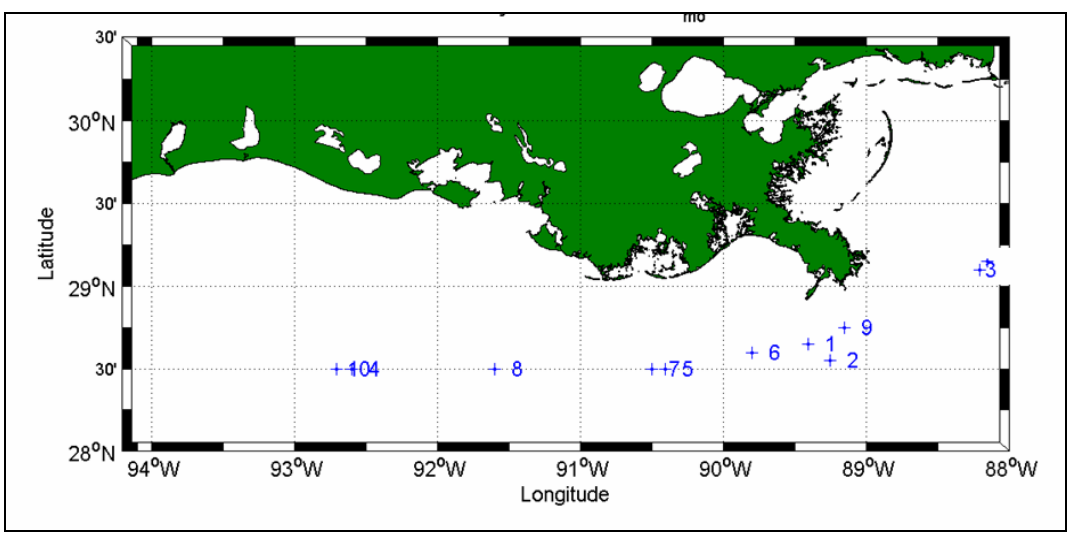

Figure 16. Maximum significant wave height location for all tracks in the region-scale grid.

Nearshore Wave Modeling

Nearshore waves are required to calculate wave runup and overtopping on structures and wave forces on structures. The numerical model STWAVE (Smith, Sherlock, and Resio 2001; Smith and Smith 2001; Thompson, Smith, and Miller 2004) was applied to generate and transform waves to the shore. STWAVE numerically solves the steady-state conservation of spectral action balance along backward-traced wave rays:

$$(C_{ga})_x \frac{\partial}{\partial x} \frac{C_a C_{ga} \cos(\mu - \alpha) E(f,\alpha)}{\omega_r} + (C_{ga})_y \frac{\partial}{\partial y} \frac{C_a C_{ga} \cos(\mu - \alpha) E(f,\alpha)}{\omega_r} = \sum \frac{S}{\omega_r} \tag{9}$$

where

C_{ga} = absolute wave group celerity

x,y = spatial coordinates, subscripts indicate x and y components

C_a = absolute wave celerity

μ = current direction

α = propagation direction of spectral component

E = spectral energy density

f = frequency of spectral component

ω_r = relative angular frequency (frequency relative to the current)

S = energy source/sink terms

The source terms include wind input, nonlinear wave-wave interactions, dissipation within the wave field, and surf-zone breaking. The terms on the left-hand side of Equation 9 represent wave propagation (refraction and shoaling), and the source terms on the right-hand side of the equation represent energy growth and decay in the spectrum.

The assumptions made in STWAVE are as follows:

a. Mild bottom slope and negligible wave reflection.

b. Steady waves, currents, and winds.

c. Linear refraction and shoaling.

d. Depth-uniform current.

STWAVE can be implemented as either a half-plane model, meaning that only waves propagating toward the coast are represented, or a full-plane model, allowing generation and propagation in all directions. Wave breaking in the surf zone limits the maximum wave height based on the local water depth and wave steepness:

$$H_{mo_{\max}} = 0.1L \tanh kd$$

where

H_{mo} = zero-moment wave height

L = wavelength

k = wave number

d = water depth

STWAVE is a finite-difference model and calculates wave spectra on a rectangular grid. The model outputs zero-moment wave height, peak wave period (T_p), and mean wave direction (α_m) at all grid points and two-dimensional spectra at selected grid points. An option has been added to input variable wind and surge fields. The surge significantly alters the wave transformation and generation for the hurricane simulations in shallow areas (such as Lake Pontchartrain) and where low-laying areas are flooded. Spatially varying wind input is important to simulate the complex wind fields in hurricanes.

a. Wave Model Inputs

The inputs required to execute STWAVE include:

a. Bathymetry grid (including shoreline position and grid size and resolution).

b. Incident frequency-direction wave spectra on the offshore grid boundary.

c. Current field (optional).

d. Surge and/or tide fields, wind speed, and wind direction (optional).

e. Bottom friction coefficients (optional).

b. Wave Model Outputs

The outputs generated by STWAVE include:

a. Fields of energy-based, zero-moment wave height, peak spectral wave period, and mean direction.

b. Wave spectra at selected locations (optional).

c. Fields of radiation stress gradients to use as input to ADCIRC (optional).

Methodology

STWAVE was applied on four grids for the southern Louisiana area: Pontchartrain, Louisiana Southeast, Louisiana South, and Louisiana West. The input for each grid includes the bathymetry (interpolated from the ADCIRC domain), surge fields (interpolated from ADCIRC output), and wind (interpolated from ADCIRC output).

a. Lake Pontchartrain Grid

The first grid covers Lake Pontchartrain at a resolution of 656 ft (200 m). The domain is approximately 25.8 by 41.9 miles (41.6 by 67.4 km). Lake Pontchartrain is run with the full-plane STWAVE to include generation and transformation along the entire lake shoreline. The grid parameters are given in Table 7. Figure 17 shows the bathymetry for the Lake Pontchartrain Grid relative to NAVD 88 (2004.65). Brown areas in the bathymetry plots indicate land areas at 0 ft or higher elevation.

Table 7 STWAVE Grid Specifications								
Grid	**State Plane**	**X origin ft**	**Y origin ft**	**Δx ft**	**Δy ft**	**Orient Deg**	**X cells**	**Y cells**
Lake Pontchartrain	LA South	3563779.5	690485.6	656	656	270	208	337
Louisiana Southeast	LA Offshore	4294586.6	1639491.5	656	656	141	683	744
Louisiana South	LA Offshore	3997126.0	1264895.0	656	656	108	664	839
Louisiana West	LA Offshore	3473720.5	1077723.1	656	656	86	810	1740

b. Louisiana Southeast, South, and West Grids

The second, third, and fourth grids cover the entire Gulf of Mexico coastline of Louisiana at a resolution of 656 ft (200 m). The domain for the southeast grid is approximately 84.9 by 92.4 miles (136.6 by 148.8 km) and extends from Mississippi Sound in the northeast to the Mississippi River in the southwest. The domain for the south grid is approximately 82.5 by 104.2 miles (132.8 by 167.8 km) and extends from the Mississippi River in the east to the Atchafalaya River in the west. The domain for the west grid is approximately 100.7 by 216.2 miles (162 by 348 km) and extends from east of the Atchafalaya River to west of the Sabine River. The southeast, south, and west grids are run with the half-plane STWAVE for computational efficiency. The grid parameters are given in Table 7. Figures 18 to 20 show the bathymetry for the southeast, south, and west grids, respectively. These simulations are forced with both the local winds and waves interpolated on the offshore boundary from the regional WAM model.

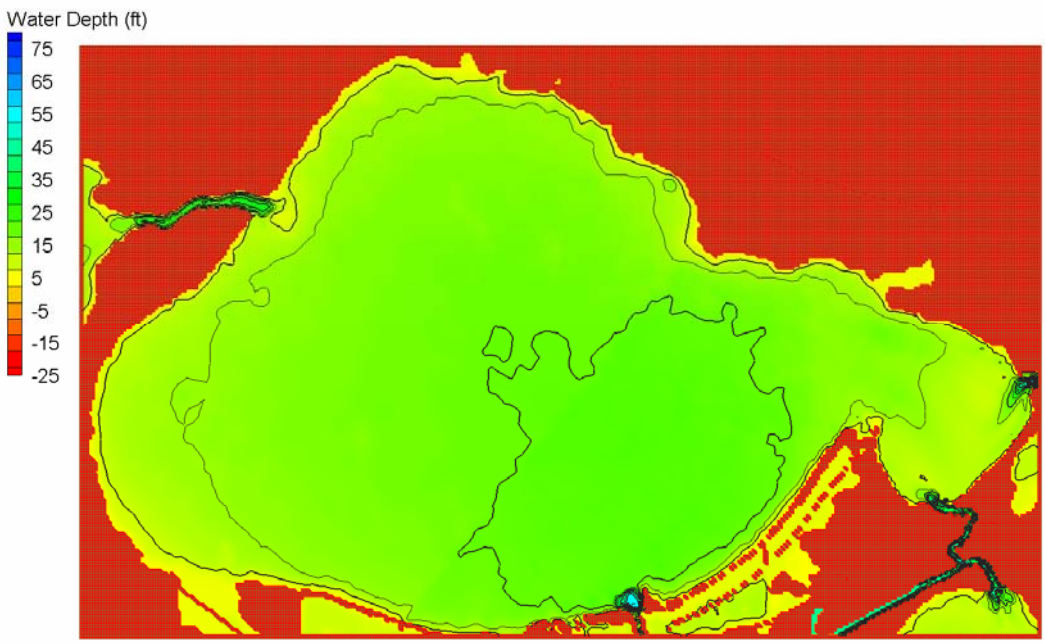

Figure 17. Lake Pontchartrain bathymetry grid (depths in feet, NAVD 88 2004.65).

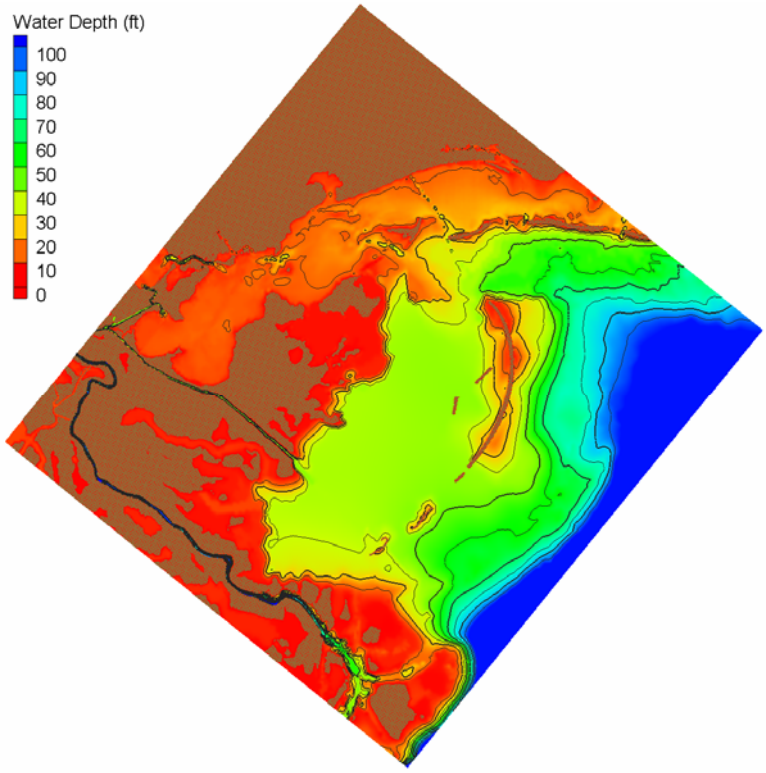

Figure 18. Louisiana Southeast bathymetry grid (depths in feet, NAVD 88 2004.65).

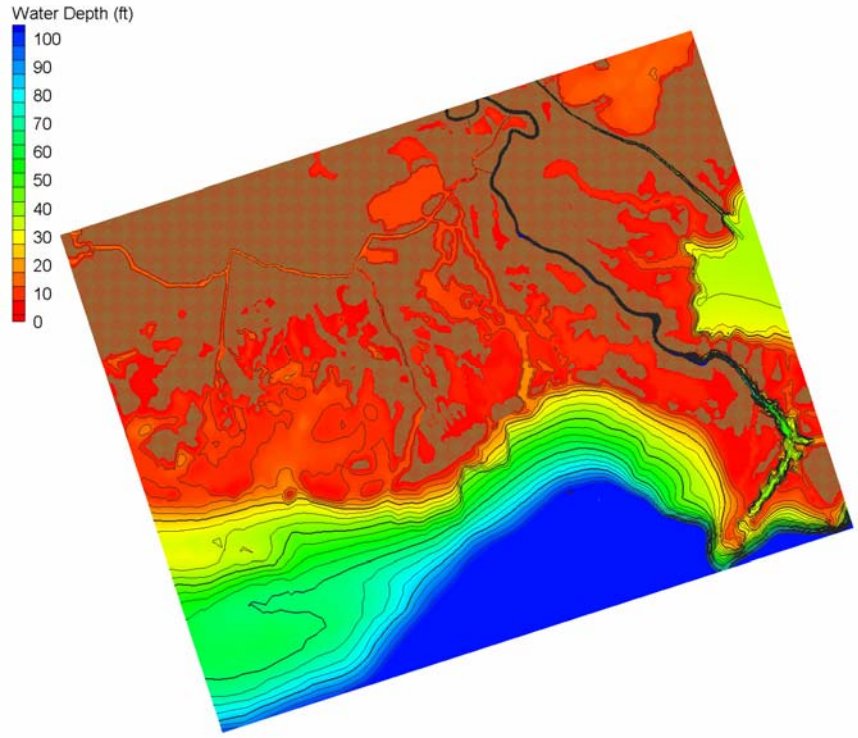

Figure 19. Louisiana South bathymetry grid (depths in feet, NAVD 88 2004.65).

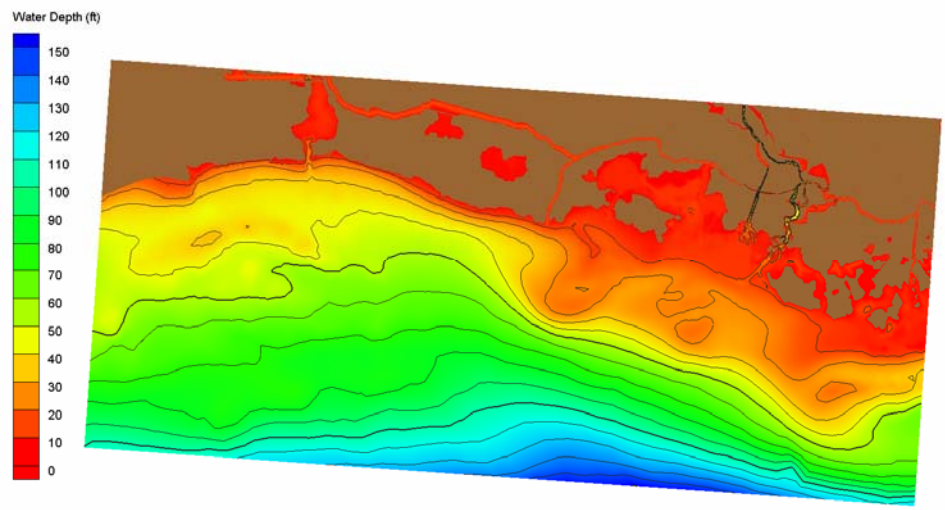

Figure 20. Louisiana West bathymetry grid (depths in feet, NAVD 88 2004.65).

Simulations

The STWAVE simulations include ten storm tracks and five levee alignments. The ten storms, five alignments, and four grids give a potential for 200 STWAVE simulations. But, most of the alignment differences occur in the South grid, so not all storms were run for all alignments on all grids. Table 8 provides a summary of which storms were run on which grids/alignments.

Table 8
STWAVE Simulations by Grid (P=Pontchartrain, SE=Southeast, S = South, W = West)

Storm Track	Alignment 1	Alignment 2	Alignment 3	Alignment 4	Alignment 5
T1	P, SE, S, W	P, SE	S	S	SE, S
T2	P, SE, S, W		S	S	SE, S
T3	P, SE, S, W	P, SE			SE, S
T4	P, SE, S, W				
T5	P, SE, S, W				SE, S
T6	P, SE, S, W	P, SE		S	SE, S
T7	P, SE, S, W	P, SE	S	S	SE, S
T8	P, SE, S, W				
T9	P, SE, S, W	P, SE	S	S	SE, S
T10	P, SE, S, W				

Results

STWAVE was run for approximately a two-day period for each storm to capture the peak wave conditions. Because STWAVE is a steady-state model, spin-up time is not required for the simulations. To provide the wave height and period for runup calculations, the STWAVE output is processed to extract the largest significant wave height for each grid cell in each domain. Then, the maximum significant wave heights in representative sections seaward of the levees are selected for

calculating runup. Maximum wave heights in Lake Pontchartrain were approximately 9 to 12 ft with peak wave periods of about 8 sec. Outside of Lake Pontchartrain, maximum wave heights were 10 to 15 ft depending on surge depths and wave periods were generally 14 sec. From Slidell to Chef Manteur, where waves are partially sheltered by the barrier islands, peak wave periods were approximately 11 sec. The significant wave height and peak wave period, along with the surge, and depth were output at specified save points near the structure toe. The save points are shown in Figures 21 – 30.

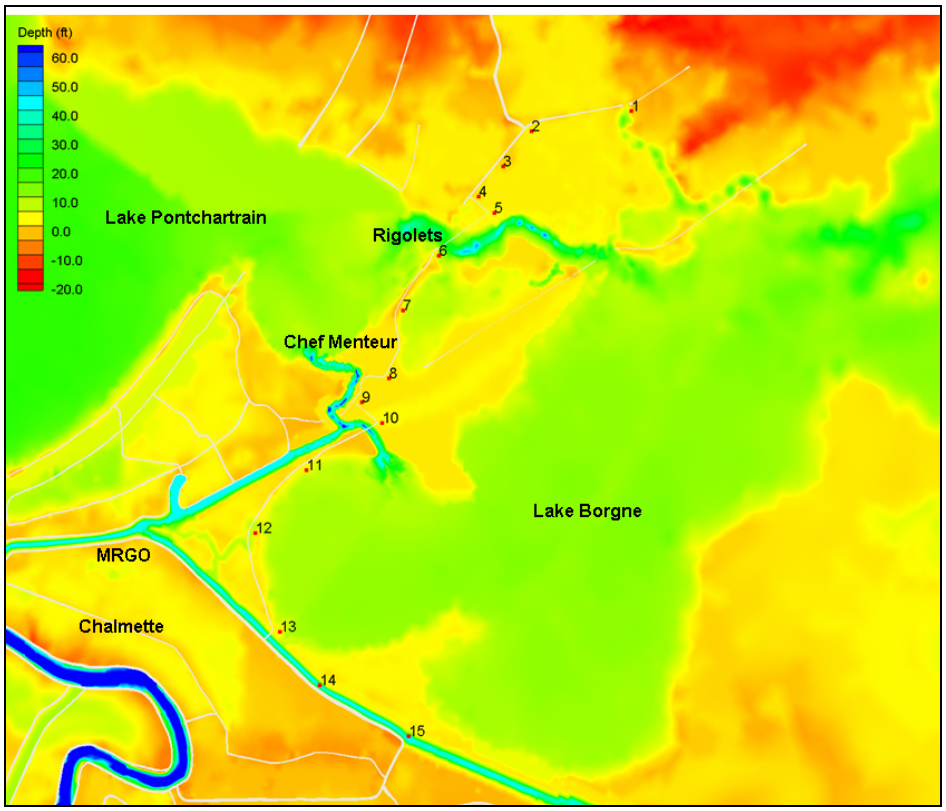

Figure 21. Save Points 1-15 (STWAVE Southeast grid).

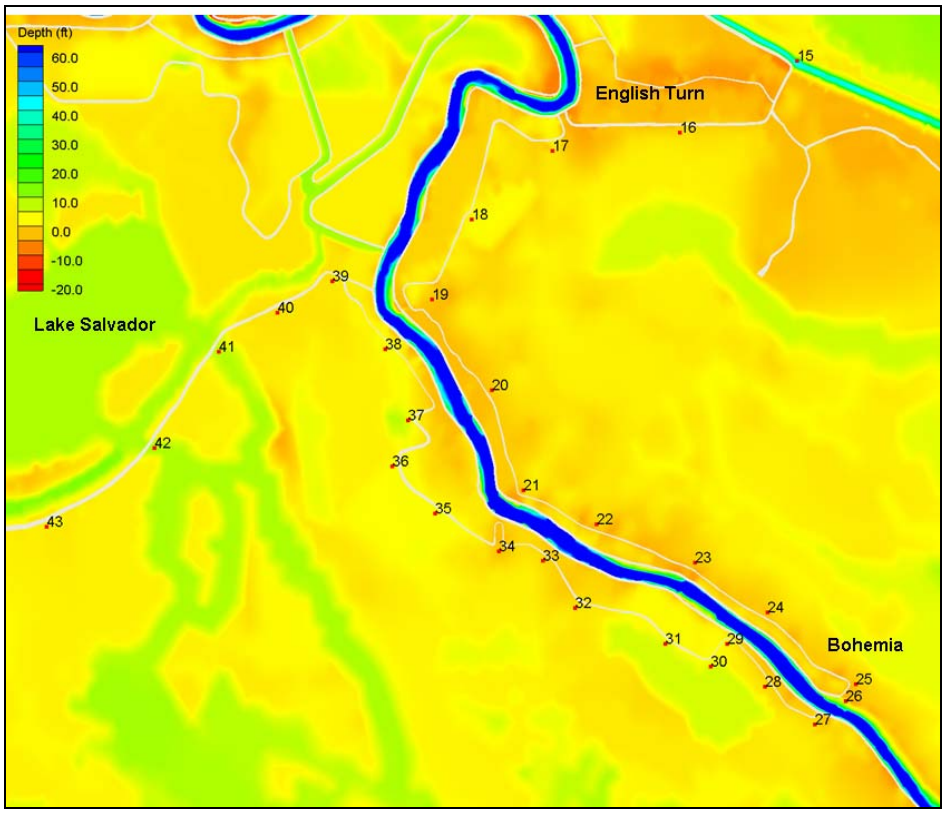

Figure 22. Save Points 15-26 (STWAVE Southeast grid) and 27 to 43 (STWAVE South grid).

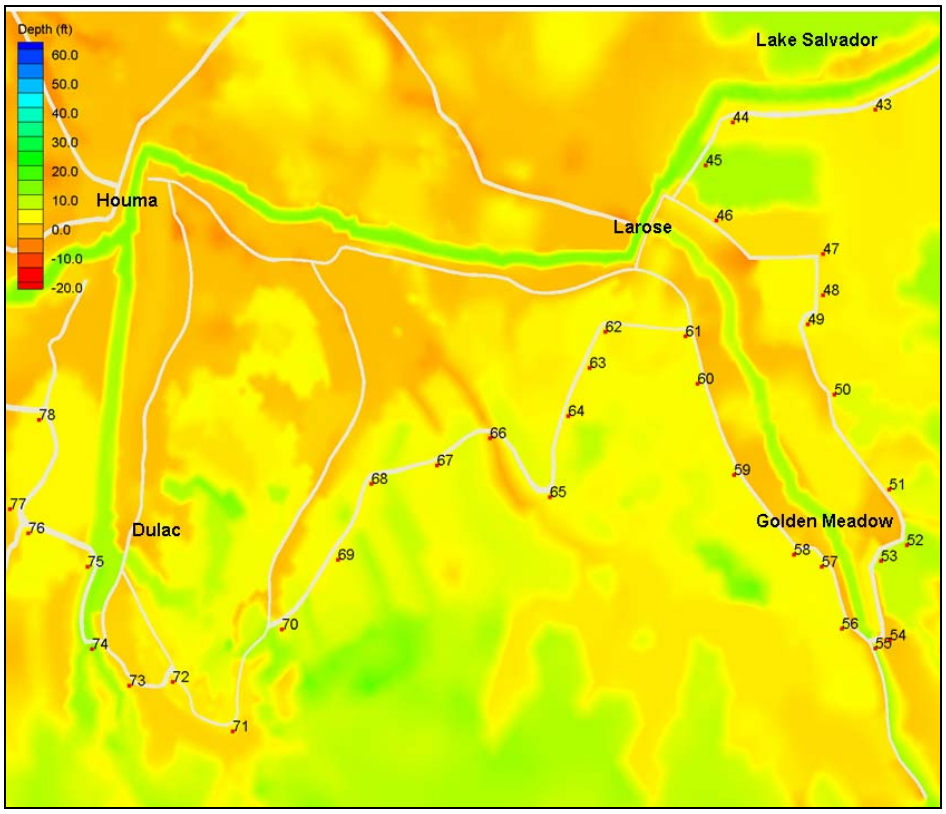

Figure 23. Save Points 43 to 78 (STWAVE South grid) and 72 to 78 (STWAVE West grid).

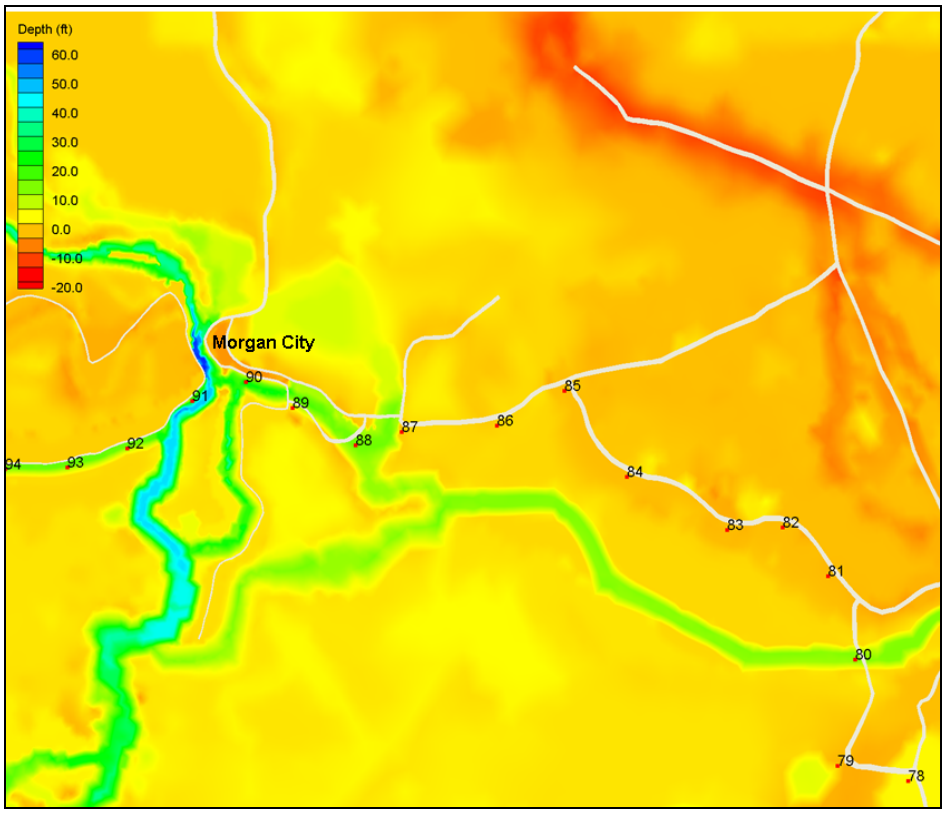

Figure 24. Save Points 78 to 88 (STWAVE South grid) and 78 to 94 (STWAVE West grid).

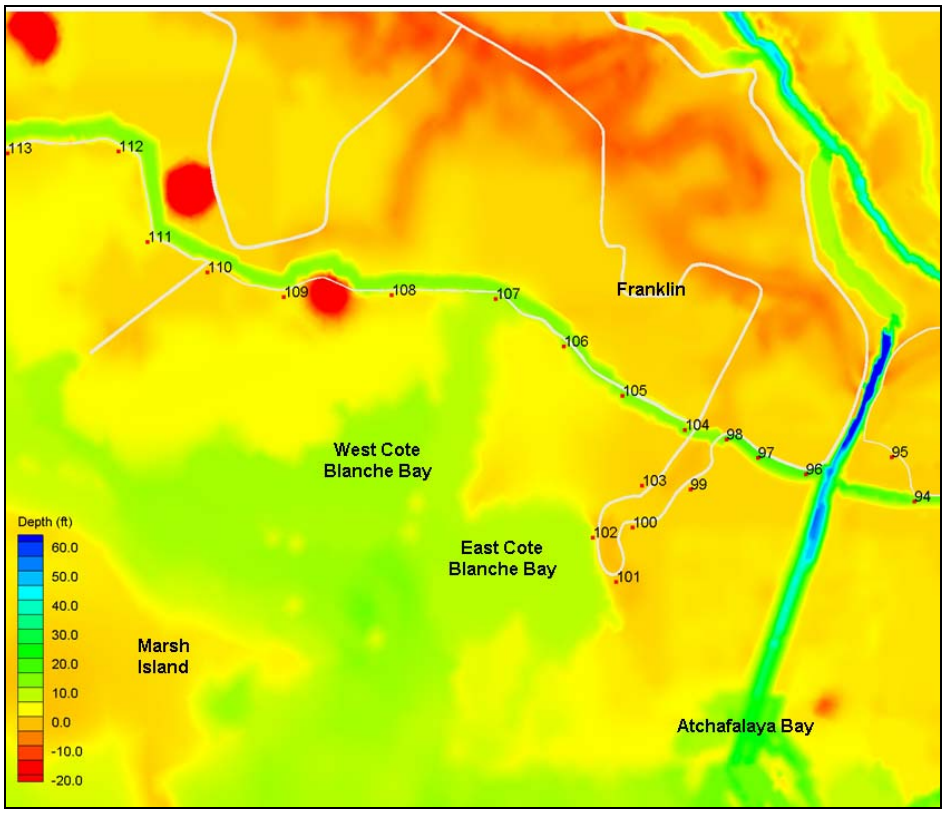

Figure 25. Points 94 to 113 (STWAVE West grid).

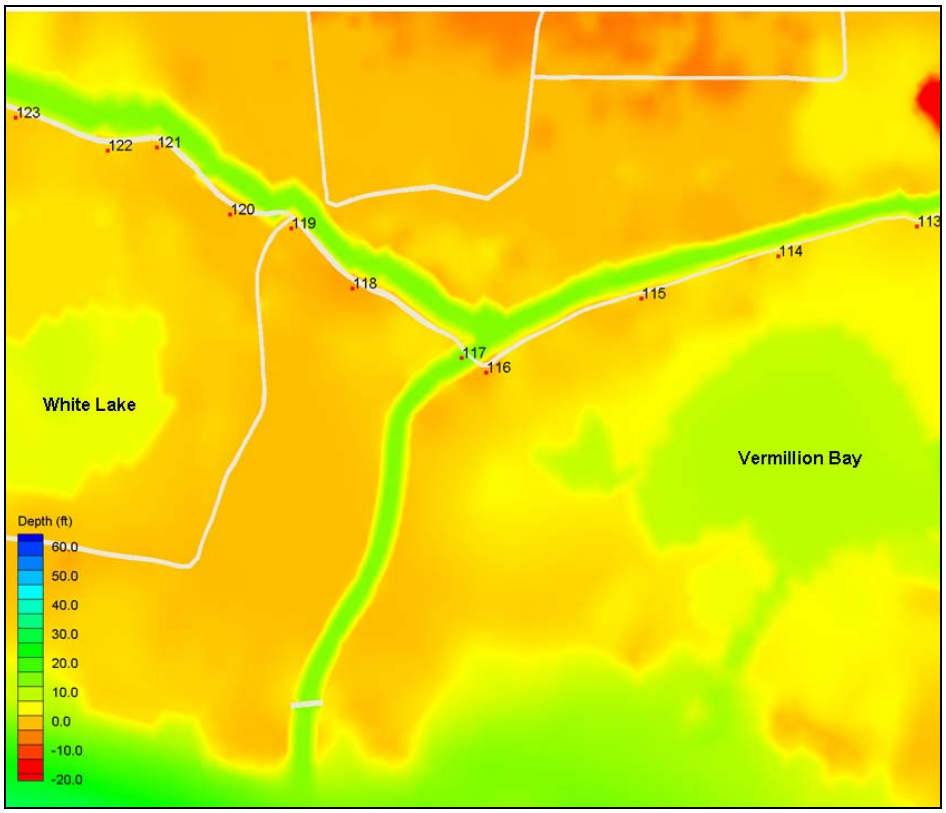

Figure 26. Points 113 to 123 (STWAVE West grid).

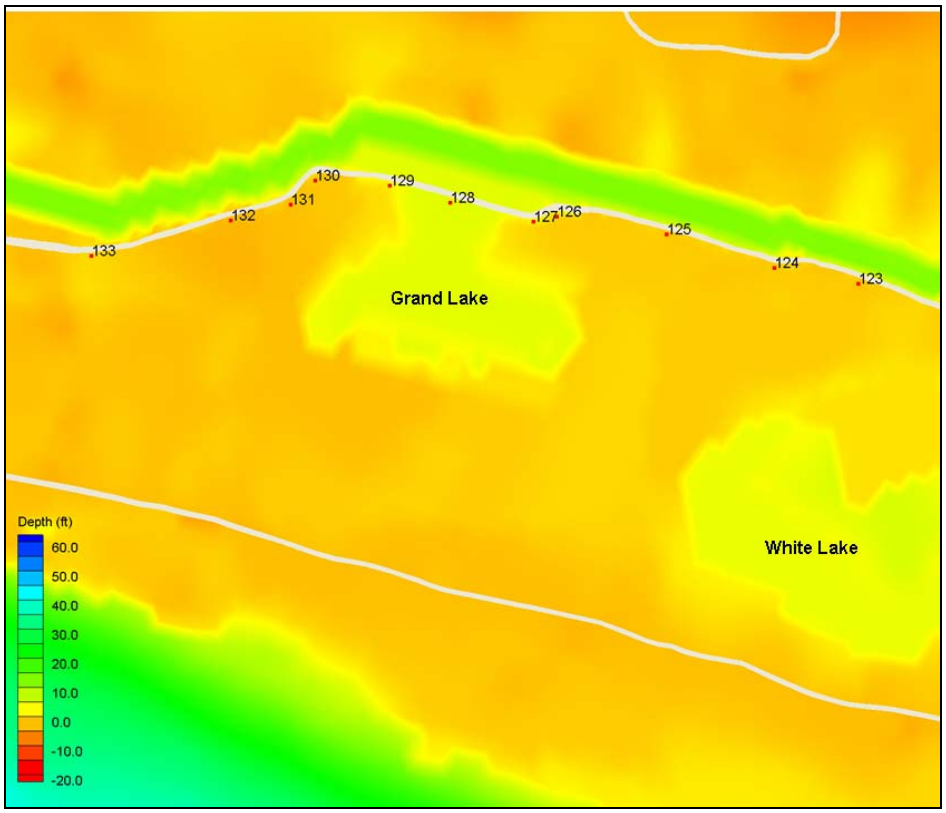

Figure 27. Points 123 to 133 (STWAVE West grid).

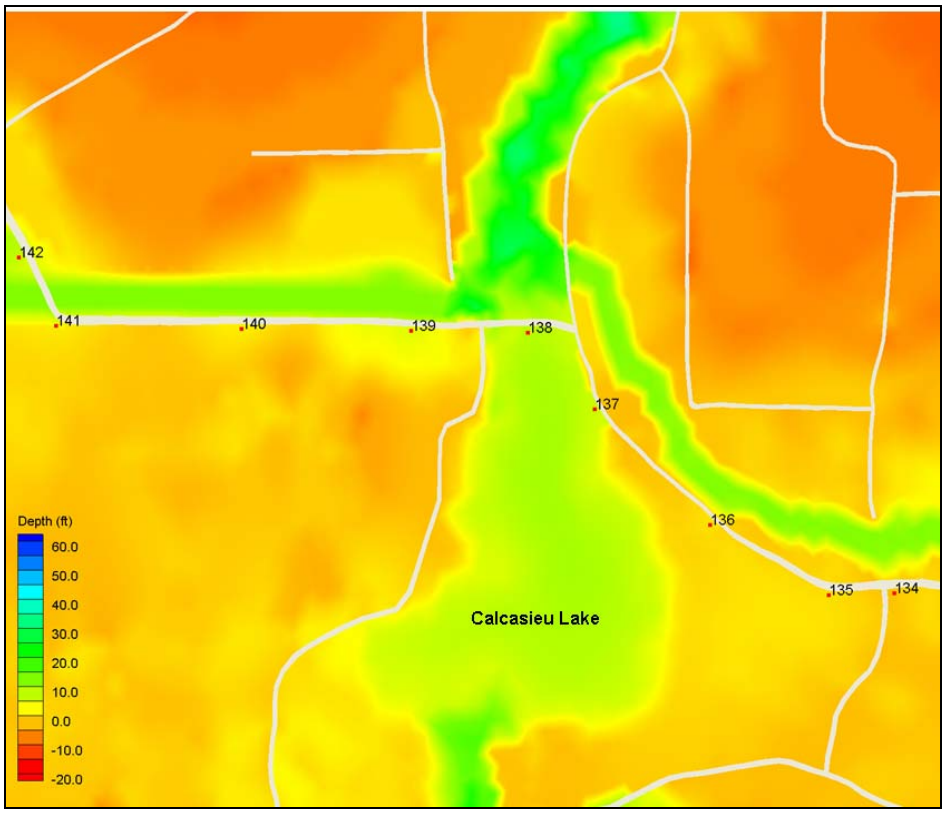

Figure 28. Points 134 to 142 (STWAVE West grid).

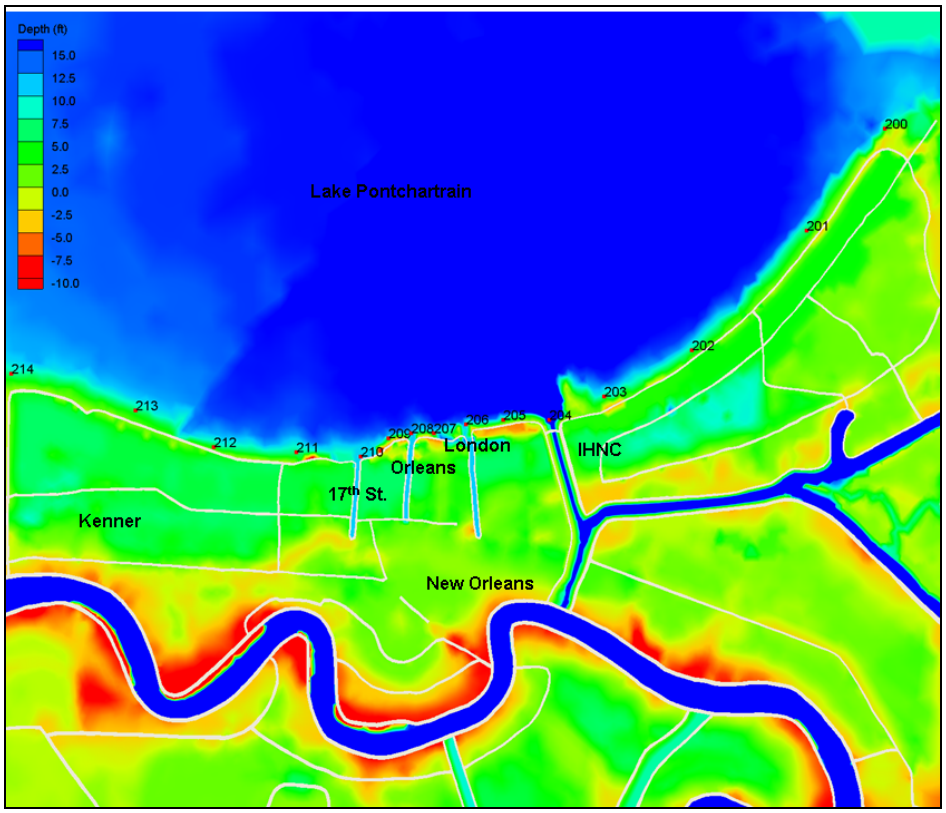

Figure 29. Points 200 to 214 (STWAVE Pontchartrain grid).

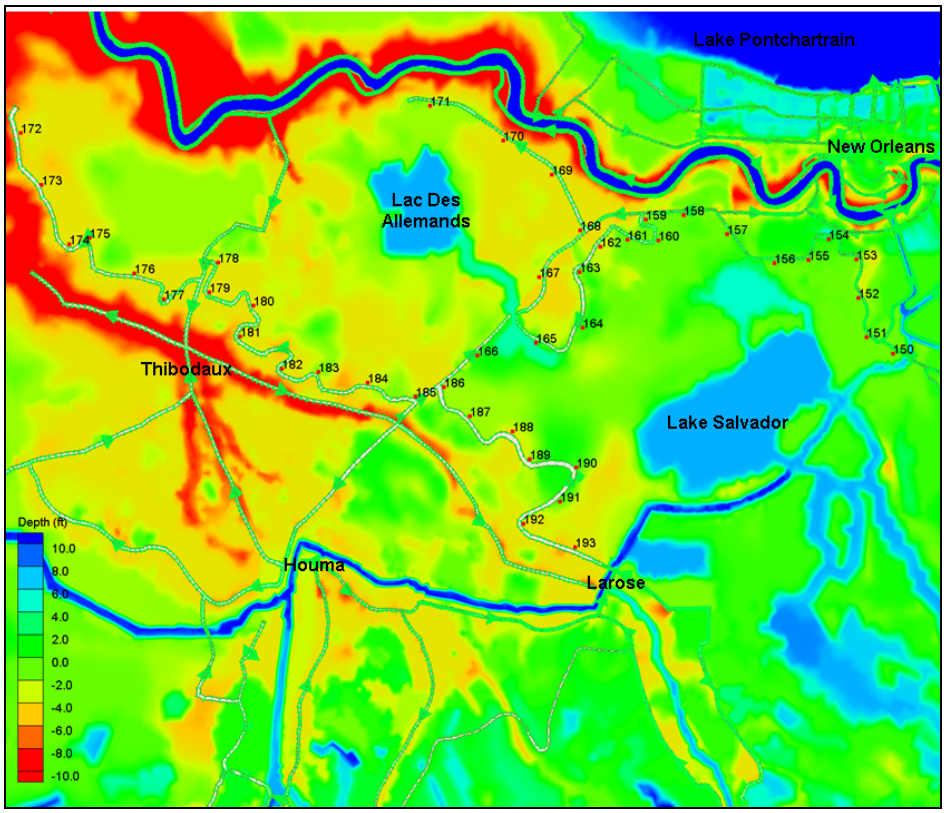

Figure 30. Points 150-172 (STWAVE South Grid Alternatives 3 and 4).

Estimated Maximum Waves and Water Level

The storms simulated for the initial screening provide limited coverage across the Louisiana coast. The simulated storm tracks impact short reaches of the levee with the highest waves and surge. However, these impacts could occur over extensive reaches of the levee with changes in storm landfall location. Therefore, for rough order of magnitude cost estimation purposes, the estimated maximum surge elevations and waves were spread along the levee applying engineering judgment. Track T3 produced the worst conditions for the south shore of Lake Pontchartrain and on the easterly reaches north of English Turn, while track T7 generated the greatest surge at the levee south of English Turn. Tracks T2 and T7 produced the most severe conditions in the Barataria reach. Track T10 produced the most severe conditions along the central and westerly reaches and track T4 produced the most extreme conditions on the far western reaches. A more comprehensive modeling assessment with complete spatial coverage will be performed for the final technical report.

A summary of the maximum wave and surge levels applied for the levee cost estimation for each alignment is provided in Tables 9-13. Maximum surge elevations range from 15 to 20 ft in Lake Pontchartrain and 30 to 40 feet elsewhere along the coast. Maximum waves range from about 10 to 15 ft with peak wave periods of 8 to 14 sec. For the PMH storm, opening of the tidal passes decreases surges on the levee at the Pontchartrain land bridge by about 3 ft and increases water levels in the lake by about 1.5 ft. It should be noted, however, that the impact of allowing the tidal passes to remain open could increase water levels further in the lake for larger (radius to maximum winds), slower moving storms and additional analysis is required. Opening Barataria Basin (Alignments 3 and 4) reduces surges along the levees by several feet but requires substantially longer levees. Smoothing the levee alignment (Alignment 5) reduces the surges east of the river approximately 3 to 5 ft.

Table 9.
Summary of Waves and Water Levels for Alignment 1

Save Points	Surge (ft)	Wave Height (ft)	Wave Period (s)
1 – 10	33	12	11
11 – 26	36	13	14
27 – 33	33	12	14
34 – 44	30	11	14
45 – 51	33	12	14
52 - 53	36	13	14
54 – 104	40	15	14
105 - 126	36	13	14
127 – 134	33	12	14
135 - 142	30	11	14
200 - 214	13 - 20	9 - 12	8

Table 10.
Summary of Waves and Water Levels for Alignment 2

Save Points	Surge (ft)	Wave Height (ft)	Wave Period (s)
1 – 10	30	11	11
11 – 14	33	12	14
15 – 26	36	13	14
27 – 33	33	12	14
34 – 44	30	11	14
45 – 51	33	12	14
52 – 53	36	13	14
54 – 104	40	15	14
105 – 126	36	13	14
127 – 134	33	12	14
135 – 142	30	11	14
200 – 214	14 - 22	10 - 13	8

Table 11.
Summary of Waves and Water Levels for Alignment 3

Save Points	Surge (ft)	Wave Height (ft)	Wave Period (s)
1 – 10	33	12	11
11 – 26	36	13	14
27 – 33	33	12	14
34 – 37	30	11	14
38	26	10	14
39	23	9	14
46 - 49	26	10	14
50	30	11	14
51	33	12	14
52 - 53	36	13	14
54 – 104	40	15	14
105 – 126	36	13	14
127 – 134	33	12	14
135 – 142	30	11	14
150 – 164	20	8	14
165	17	6	14
166 – 184	10	4	14
185	13	5	14
186 – 190	17	6	14
191 – 193	26	10	14
200 - 214	13 - 20	9 - 12	8

Table 12. Summary of Waves and Water Levels for Alignment 4			
Save Points	**Surge (ft)**	**Wave Height (ft)**	**Wave Period (s)**
1 – 10	33	12	11
11 – 26	36	13	14
27 – 33	33	12	14
34 - 37	30	11	14
38	26	10	14
39	23	8	14
46 – 49	26	10	14
50	30	11	14
51	33	12	14
52 - 53	36	13	14
54 – 104	40	15	14
105 – 126	36	13	14
127 – 134	33	12	14
135 – 142	30	11	14
150 – 156	20	7	14
157 - 162	23	8	14
163 - 164	20	7	14
165 - 166	17	6	14
186 - 190	17	6	14
191 – 193	26	10	14
200 - 214	13 - 20	9 - 12	8

Table 13. Summary of Waves and Water Levels for Alignment 5			
Save Points	**Surge (ft)**	**Wave Height (ft)**	**Wave Period (s)**
1 – 19	33	12	11
39 – 46	33	12	14
61 - 70	33	12	14
71 – 98, 104	40	15	14
105 - 126	36	13	14
127 – 134	33	12	14
135 - 142	30	11	14
200 - 214	13 - 20	9 - 12	8

Runup and overtopping were calculated from the estimated maximum wave and surge values. Three levels of design crest height to be considered in this initial cost estimation include a non-overtopped grass-covered levee crest, a lightly overtopped, lightly armored levee crest (defined as an overtopping rate of less than one cubic foot per second per foot of levee), and a heavily armored levee crest that is heavily overtopped by waves but not by steady surge flow. The runup was calculated from equations derived by Hughes (2003, 2004) for irregular wave runup using a momentum flux parameter. Wave overtopping volumes were calculated with formulae for bermed and straight impermeable slopes based on data from van der Meer and Janssen (1995). The complete methodology and results are presented the levee cross section evaluation appendix.

Coastal Features and Storm Surge

There is growing consensus among scientists and engineers involved with Louisiana coastal protection and restoration that future projects for reducing hurricane risks in New Orleans and along the Louisiana coast should include plans to sustain or enhance the wetland-dominated landscapes that surround the area. While these landscapes are widely recognized for their great value to the nation

for the natural resources and ecosystem services they provide, they may also function to provide some level of protection from hurricane wave action and storm surge.

Coastal geologic features and associated vegetation, manifested in the presence of barrier islands, cheniers, maritime forest ridges, river tributary ridges, marshes, and wetlands, have potential to abate and restrict flow exchange between the estuarine and sea environments. Compared to open water, these coastal areas have increased drag and limited water depth facing inland, which slow water velocities and may reduce the effects of propagating storm surges and waves. Together, the elevation and vegetation of coastal features has potential to restrict the volume of water at areas landward of barriers.

Emergent canopies provided by forested wetlands have potential to diminish wind penetration, thereby reducing the wind stress available to generate surface waves and storm surge. The sheltering effect of these canopied areas also affects the fetch over which wave development takes place. Shallow water depths attenuate waves via bottom friction and breaking, while vegetation provides additional frictional drag and wave attenuation and also limits wave setup. Extracting energy from waves either by breaking or increased drag in front of levees would reduce the destructive storm wave action on the levees themselves.

Despite the qualitative knowledge of the potential effects of landscape features on hurricane protection, there has been minimal quantification of these effects in nature. In a Letter from the Chief of Engineers (1965) documenting an interim hurricane survey of Morgan City and vicinity, Louisiana, measurements of high-water marks due to hurricane surge were correlated with distance inland from the coast. Surge elevations at 16 locations near Morgan City due to seven hurricanes (Sep 1909, Aug 1915, Sep 1915, Aug 1926, Sep 1947, Sep 1956, and Jun 1957) were documented giving 42 data points. The report states that this area has numerous bays and marshes, but the data evaluated include the western part of Louisiana with cheniers (relatively high wooded ridges). Inconsistent results were obtained when attempting to correlate hurricane translation speed, surge hydrograph at the coast, and surge elevations inland. However, a trend was observed for the decrease in storm surge as a function of distance inland, and is independent of hurricane translation speed, wind speed, and direction. The relationship indicates that storm surge was reduced by 1 foot for every 2.75 miles inland.

Lovelace (1994) documented storm surge elevations after Hurricane Andrew in Louisiana. These data are being compiled into a GIS for future reference. Citing this study, the Louisiana Coastal Wetlands Conservation and Restoration Task Force and the Wetlands Conservation and Restoration Authority (2004) suggest that storm surge is reduced about 3-inch (0.25 ft) per mile of marsh along the central Louisiana coast.

Stone et al. (2003) modeled a Category 3 hurricane that made landfall in 1915 and compared wave and storm surge for the south-central Louisiana coast in 1950 (1.09 million acres of land) to that in 1990 (0.85 million acres of land). Models used were a hurricane planetary boundary model, ADCIRC circulation model, and SWAN wave model. Acreage impacted by a 7 ft and 12 ft surge increased to 69,000 and 49,000 acres, respectively, between 1950 and 1990. Surge levels greater than 15 ft were not significantly different between the two time periods.

The Louisiana Coastal Wetlands Conservation and Restoration Task Force and the Wetlands Conservation and Restoration Authority (2004; Chapter 6, p. 55) discuss that it is "commonly acknowledged that barrier islands and wetlands reduce the magnitude of hurricane storm surges and related flooding; however, there are scant data as to the degree of reduction." At the time the report was written, the best information documenting this phenomenon came from gauges measuring water elevations during the second landfall of Hurricane Andrew (data documented by Lovelace 1994), which occurred in the vicinity of Point Chevreuil, Louisiana on August 26, 1992. Gauge data from Cocodrie, Louisiana indicated a maximum water level elevation equal to 9.3 ft during this Category 3

Hurricane. Over a 23-mile stretch of marsh and open water from Cocodrie to the Houma Navigation Canal, the water elevation decreased from 9.3 ft to 3.3 ft, equating to a reduction in surge amplitude equal to 3.1 inch (0.26 ft) per mile of marsh and open water. A similar set of measurements showed reduction of the storm surge from 4.9 ft (1.5 m) at Oyster Bayou to 0.5 ft at Kent Bayou, located 19 miles north. This second set of measurements indicated 2.8-inch (0.23 ft) decrease in surge per mile over "fairly solid marsh." The report cautions that these represent measurements from one storm; other factors, such as storm characteristics, coastal geomorphology, and track of the storm influence the degree to which wetlands decrease storm surge.

The Working Group for Post-Hurricane Planning for Louisiana Coast (2006) wrote "barrier islands, shoals, marshes, forested wetlands and other features of the coastal landscape can provide a significant and potentially sustainable buffer from wind wave action and storm surge generated by tropical storms and hurricanes." ADCIRC results from Rick Luettich (Dec 30, 2005) indicated that wetland replacement east of the Mississippi River Gulf Outlet (MRGO) by 8 ft of open water would increase storm surge from Hurricane Katrina by 3 to 6 ft for St. Bernard Parish and Eastern New Orleans.

The role of wetlands and other coastal features in reducing storm surge and waves must be addressed. There is a need to quantitatively evaluate the role of wetlands and other coastal features in reducing storm surge and waves in coastal Louisiana. To address this need, the literature review will be continued and a set of idealized numerical modeling tests will be conducted to evaluate the reduction in surge as a function of landscape feature and vegetation type. A series of model runs will be conducted with ADCIRC and STWAVE to quantify the sensitivity of storm surge and waves to environmental features. The inputs required by the models include topography/bathymetry and frictional resistance of various vegetative covers for both hydro modeling and wind sheltering. Assessments will be made for various configurations of coastal features forced with some range of storm conditions. In making these assessments the specific effects of different landscape features will be considered. It is anticipated that the effects will range from simply effecting water depth, to imparting additional frictional resistance only in the water column to vegetation that penetrates the water column and effects on the overlying wind field. A surge elevation database is being developed within a GIS for determining relationships based on available measurements, as well as for comparison with numerical modeling results.

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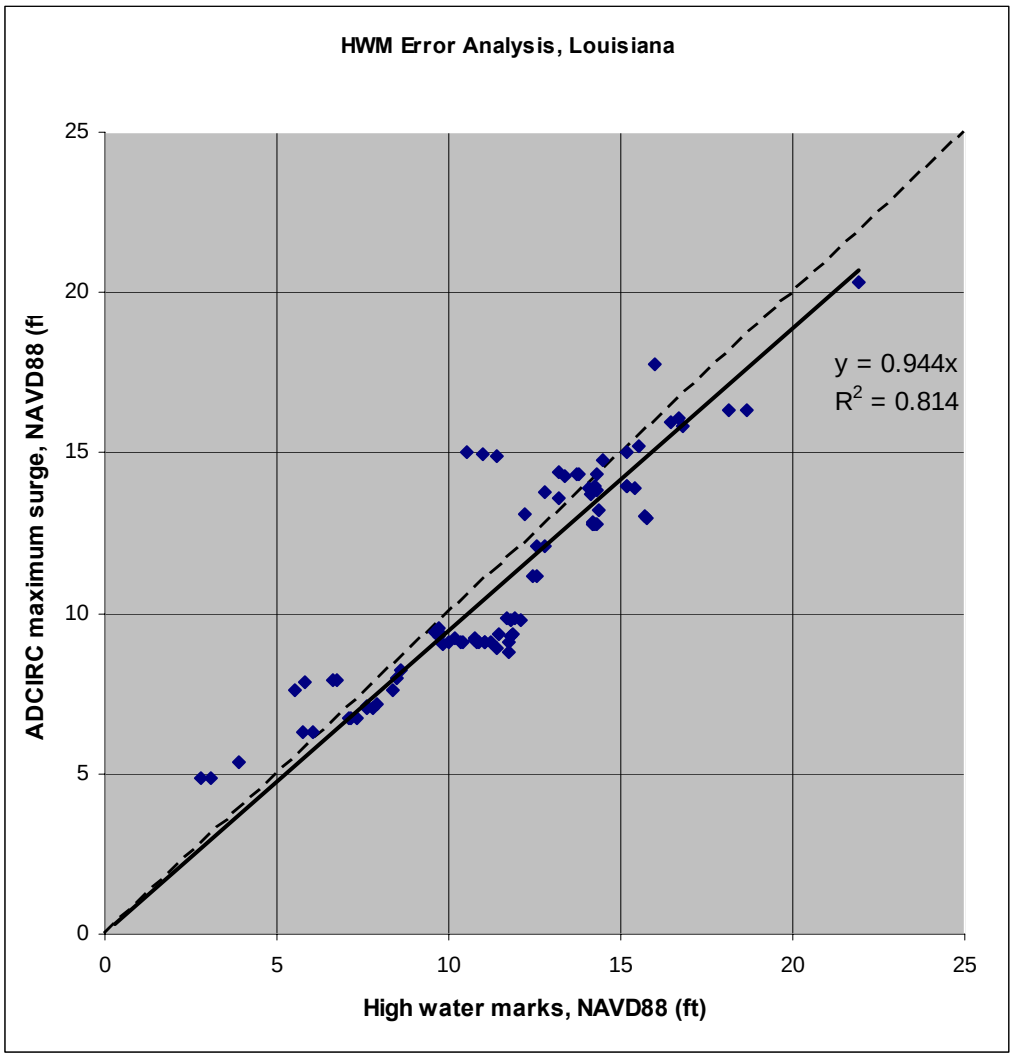
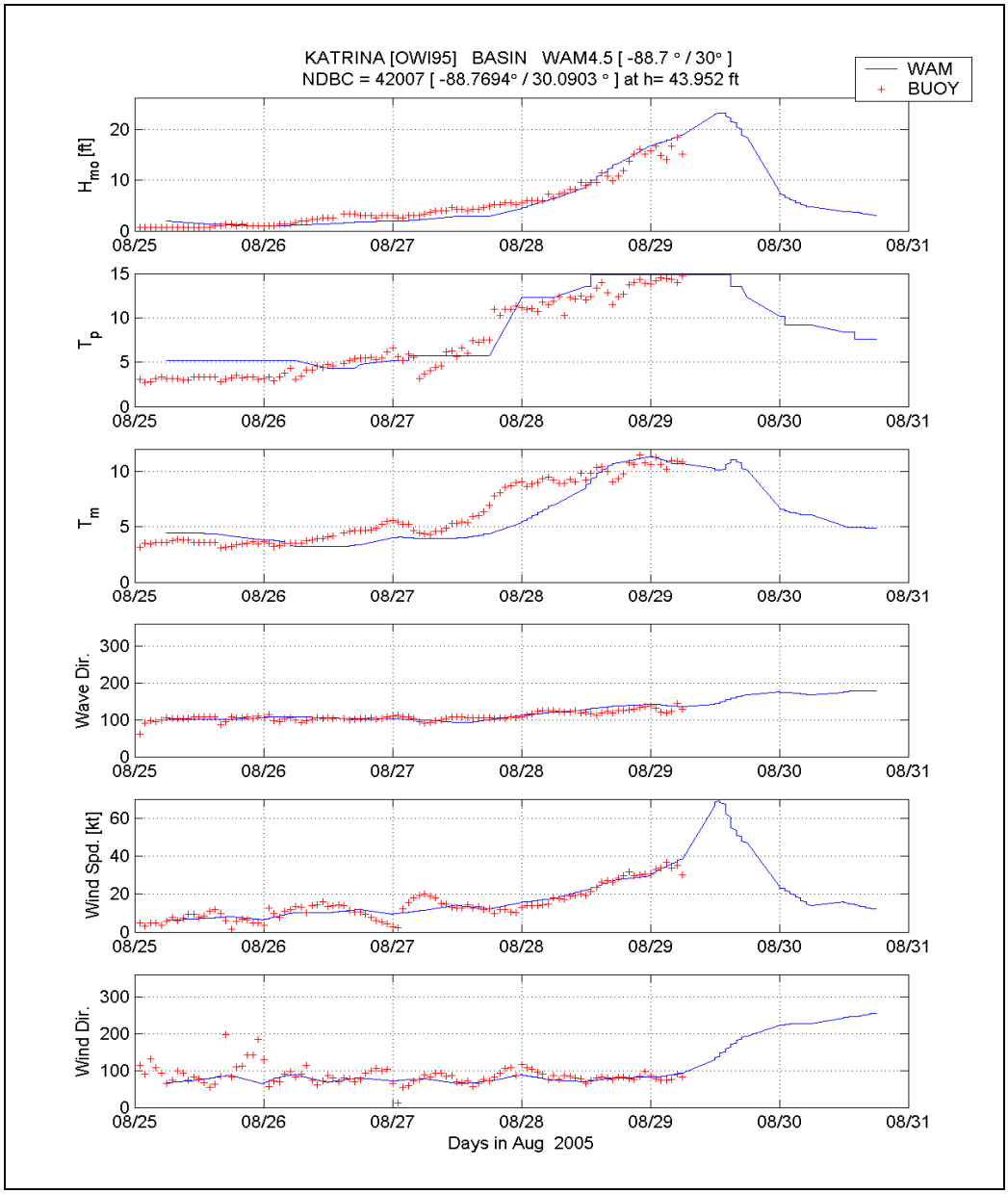
Attachment A: Storm Surge Calibration Analysis



High water mark error analysis for Louisiana conducted under Task 4 by the Interagency Performance Evaluation Team (IPET). A complete description of the calibration and validation is found in Volume 4 of the IPET final report.

Attachement B: WAM wave model validation with NDBC Buoy



Comparison of WAM model results to data measurements at NDBC buoy 42007. A complete description of the validation is found in Volume 4 of the IPET final report.

ANNEX 2 – WIND AND PRESSURE FIELDS

Specification of Hurricane Surface Marine Wind and Pressure Fields For the New Orleans and Louisiana Coast

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INTRODUCTION

The objective of this study is to develop a description of the time and space evolution of the surface marine wind and pressure fields for a suite of simulated design level hurricanes, first for New Orleans impact, and later for impacts along the entire Louisiana coastline. The wind and pressure fields will be used by ERDC (Engineer Research and Development Center) and possibly others to model storm generated water levels using ADCIRC (Advanced CIRCulation model) for the purposes of establishing preliminary infrastructure re-design criteria.

This preliminary study does not purport to develop in any systematic way a new specification of the design level storm criteria, such as central pressure index (CPI), radius parameter (Rp)/radius of maximum wind (RMW), forward velocity (Vf), storm track crossing locations and incidence angles. Rather, this study is intended mainly to specify the wind and pressure fields from established NOAA criteria for Standard Project Hurricane (SPH) and Probable Maximum Hurricane (PMH) in the form documented in NOAA's Technical Report NWS 23 (1979) with updated criteria supplied by ERDC. The PMH criteria for the stretch of the US coastline of interest here are sufficiently extreme that they also describe a storm of Category 5 intensity on the Saffir-Simpson Scale (SSC), and indeed a rather extreme Category 5 intensity. Projected tracks for the simulations are provided directly from ERDC based on previous work.

The method used to develop the wind fields in NWS23 from SPH and PMH criteria are out–of-date and have been supplanted over the past two decades by more accurate and robust dynamical and kinematical methods. For this project, the wind fields and pressure fields are developed using Oceanweather Inc.'s (OWI) highly refined meso-scale vortex numerical model for the specification of surface wind and pressure fields in tropical cyclones, often referred to at ERDC as the PBL (Planetary Boundary Layer) approach (Thompson and Cardone, 1996). However, there was still the need for a model calibration phase in order to explore and define additional storm criteria that are not included in the SPH and PMH criteria. These additional criteria include the pressure profile peakedness parameter, so called Holland's B (Holland, 1980), the boundary layer depth, the azimuth of the wind maximum and an updated landfall-filling model. This calibration effort required a period of experimentation with some real recent Gulf of Mexico storms of intensity comparable to those of the SPH and PMH for which excellent kinematic descriptions of the wind field already exist.

Soon after the base study described above commenced, there arose a requirement for the generation of a series of wind field for synthetic "sensitivity" storms in order for ERDC to test an idealized ADCIRC grid layout over a domain roughly the size of the Gulf of Mexico. Unlike the SPH/PMH studies where storm parameters such as CPI and RMW were drawn from climatology, the sensitivity runs were made using a pre-determined set of input parameters that attempt to model much of the dynamic range of land falling hurricanes in the Gulf of Mexico. Since much of the methodology, output data and documentation for these wind fields are in common with the SPH/PHM efforts, the wind fields were developed within this study and this report details the work performed in this regard.

Finally, to support ERDC's ADCIRC calibration efforts, a hindcast of the wind field for hurricane Camille (1969) along both its actual historical track and a shifted track was performed. Unlike the steady state solutions outlined above, this task involved running the actual storm history of model parameters as developed by OWI.

Methodology

OWI's tropical cyclone boundary layer model (henceforth referred to as TC96) was set up on a target domain the covers the range of the two working grids. Grid spacing of the fine domain is felt to be sufficient grid spacing to properly resolve the radius of maximum wind (RMW) in the hindcast storms (see below for grid details).

The SPH and PMH criteria provide part of the input suite needed by TC96. TC96, which is a dynamical model, operates on these inputs and additional inputs required and produce a wind and pressure field. The simulations assume that a hurricane is in steady state offshore and does not begin to weaken until the center arrives at the coast. Applications of the landfilling model are described below.

Real storms of intensity comparable to the CPI of SPH and PMH were used to obtain satisfactory agreement between the TC96 and available kinematic descriptions of the wind fields. The objective of this experimentation is to ensure that the surface marine wind field specified is consistent with modern thinking as to the relationship between the storm criteria and inner core maximum surface winds for modern averaging intervals, including the standard 30-minute

average, and the definition of "sustained" wind speed, which is a stochastic wind variable which may be defined as the median peak 1-minute wind speed within the 30-minute period.

The 2005 season contained 4 very strong storms: Dennis, Katrina, Rita and Wilma for which intensive Hwind analysis was available. This dataset covers most of the dynamic range required by the SPH, PMH and sensitivity runs. Figure 1 shows the maximum 1-min wind analyzed by Hwind vs. the total pressure drop (ΔP) in millibars (far-field pressure – central pressure). Central pressures were taken from the Hwind analysis (as determined by aircraft penetrations) while far-field pressures were estimated from synoptic weather maps by OWI. The polynominal fitted line describes a mean relationship between (ΔP) and maximum sustained wind speed. There is considerable scatter about this mean relationship. There are many potential sources of such scatter, some of which arise in real storm-by-storm variations in properties and some in statistical sampling variability (for instance, HWind analyses are strongly influenced by aircraft winds and it is always possible that on a given mission the aircraft flight path failed to observe the storm wind maxima). Several recent studies and our own analyses of intense historical Gulf hurricanes have established the tendency for the pressure profile mean peakedness parameter (usually referred to as Holland's B, though it was actually introduced by Graham and Hudson (1960)) to increase with increasing (ΔP). There may also be a dependence on latitude and storm scale though such effects are small for the purposes of this study, which addresses only relatively small and extreme hurricanes in the northern Gulf of Mexico. A mean dependence of B on (ΔP) has been incorporated into the initialization of TC96 and the maximum wind solution predicted there-from is shown in Figure 1 to lie slightly above the polynominal fit line, which is not unexpected since not all Hwind analysis contain the absolute storm maximum wind. The solution is well within the scatter of the data, but it is notable that even in a population of just four well developed storms the large range of maximum winds that can result from a single ΔP, though as noted above some of the scatter may be merely statistical.

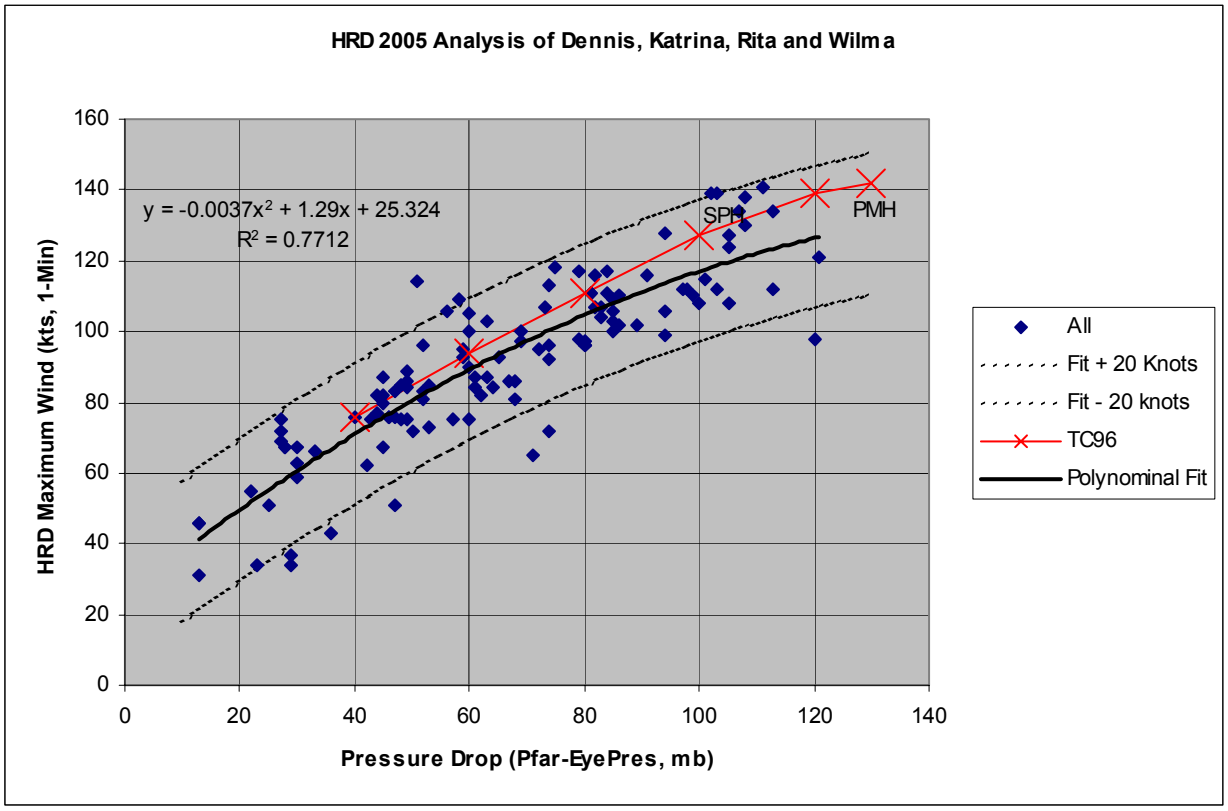
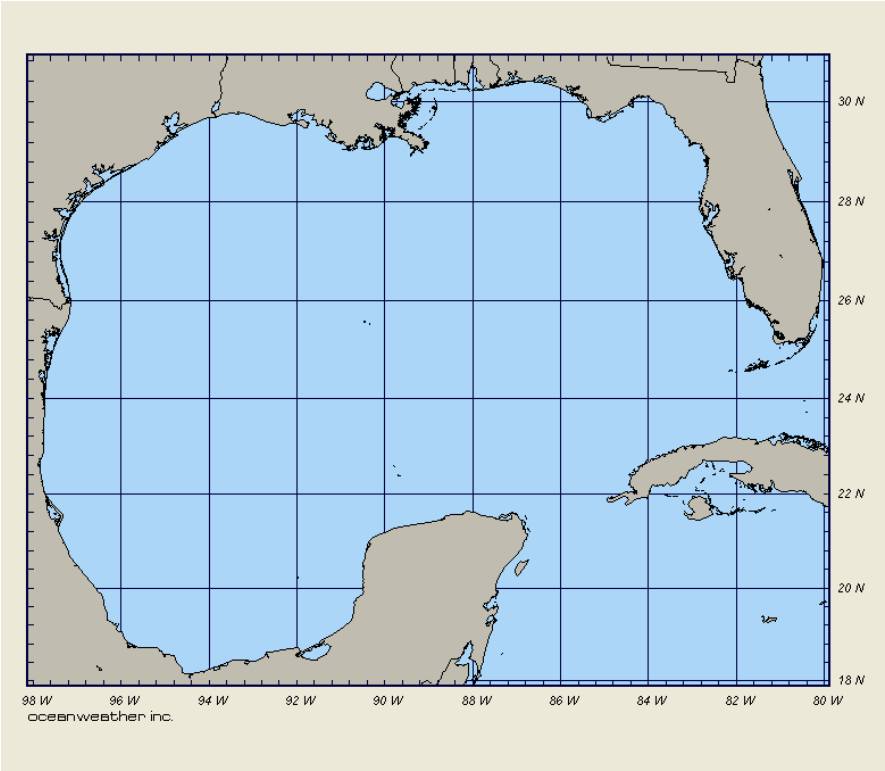


Figure 1. TC96 Maximum 1-minute wind (kts) solution compared with HWind 1-min maximum wind (kts) vs. pressure drop (mb)

Target Grids/Output

Hindcast deliverables are OWI WIN and PRE (see appendix A for details) format files containing winds (30-minute average, 10-meter neutral marine exposure) and sea level pressures for each storm. Two grid systems are used for the output winds and pressures. The basin scale grid is .1 degree, (~10 km) covering the domain 18-30.8N, 98-80W; the fine scale grid is .025 degree, (~2 km) covering the domain 28.5N-30.8N, 94.25W-88W (see Figure 2). All wind and pressure output is specified at a 15-minute time step.



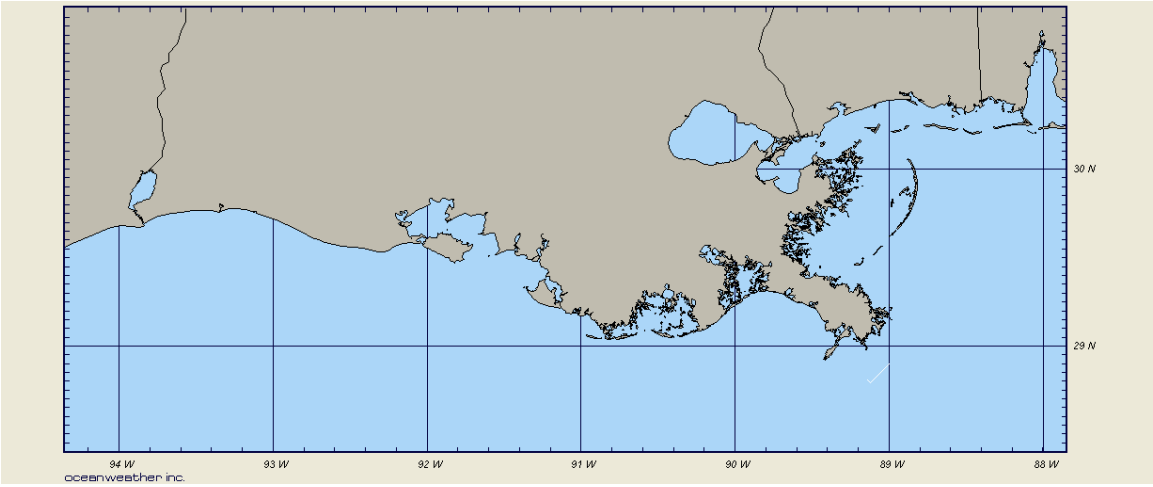


Figure 2 Tropical winds/pressure domains (basin above, fine below)

Landfilling Model

In NWS 23 a filling model for use in SPH/PMH wind fields in described in Chapter 15. However, its application is based on modification of the maximum wind rather than in CPI. Since the approach applied here relies on the CPI value for the SPH and PMH hurricanes, a land filling model in terms of CPI is required. A newer filling model developed by Vickery *et. al.* (1995) which describes the land effects in terms of CPI was applied. The filling model takes the following form where t is time in hours and p_o is in mb:

$$\Delta p(t) = \Delta\, p_o \exp(-at)$$

The filling constant a is given as:

$$a = a_o + a_1 \Delta p_o$$

Where a_o and a_1 are 0.035 and 0.00050 for Gulf Coast hurricanes. A 2-hour delay was incorporated into the decay model for Gulf Coast hurricanes. The SPH/PMH solutions each applied the landfilling model while the sensitivity runs did not (as requested by ERDC). For each SPH/PMH track provided by ERDC the landfall time was estimated to the nearest ½ hour and only one landfall point was considered.

STANDARD PROJECT HURRICANE CPI

The parameters of the SPH run in this study were provided by ERDC based on a new reanalysis of historical storms performed at the National Climatic Data Center. Parameters include a central pressure of 904 mb, far-field pressure of 1008 mb (ΔP=105 mb), radius of maximum winds 11 Nmi and forward speed of 10 knots. A Holland's B of 1.23 was specified in the tropical model. There were 3 tracks used to position the SPH solution: track A, C and F with translation speeds of 6, 5 and 11 knots respectively (Table 1). All solutions used the steady-state SPH solution that reflects a forward motion of 10-knots with filling modeled applied after landfall.

Table 1. SPH track list

Reference	ERDC Source Track	Start Date (CYMDH)	End Date (CYMDH)	Est. Landfall (DHM)
SPHCPIA	"A" Track from SPHTracks.xls (6 knots)	2005080100	2005080122	011000
SPHCPIC	"C" Track from SPHTracks.xls (5 knots)	2005080100	2005080203	010900
SPHCPIF	"F" Track from SPHTracks.xls (11 knots)	2005080100	2005080111	010400

MAXIMUM PROBABLE HURICANE CPI

NWS 23 provided most of the parameters required to describe the PMH storm. Parameters include a central pressure of 890 mb with far-field pressure of 1020 mb (ΔP=130 mb). The radius of maximum wind of 11 Nmi and forward speed of 10 knots were specified by ERDC. A Holland's B of 1.35 was specified in the tropical model. There were 33 tracks used to position the PMH solution. Tracks 1-16 contain variable forward speed, while tracks 17-33 used a fixed translation speed of 10 knots. All solutions used the steady-state PMH solution that reflects a forward motion of 10-knots with filling modeled applied after landfall.

Table 2. PMH track list

Reference	ERDC Source Track	Start Date (CYMDH)	End Date (CYMDH)	Est. Landfall (DHM)
PMHCPI01	Track_Rita.d	2005092009	2005092509	240730
PMHCPI02	Track_Katrina.d	2005082515	2005083009	291130
PMHCPI03	track_carmen.d	1974090100	1974091006	080900
PMHCPI04	track_1915.d	1915092600	1915093018	292000
PMHCPI05	Rita_1p0east_f.txt	2005092012	2005092509	240700
PMHCPI06	KatrinaTrack5_0p9711east-final.txt	2005082523	2005083009	291530
PMHCPI07	KatrinaTrack4_0p7341east-final.txt	2005082521	2005083009	291530
PMHCPI08	KatrinaTrack3_0p5682east-final.txt	2005082519	2005083009	291530
PMHCPI09	KatrinaTrack2_0p2852east-final.txt	2005082515	2005083009	291530
PMHCPI10	KatrinaTrack1_0p0943east-final.txt	2005082515	2005083009	291130
PMHCPI11	KatrinaTrack0_0p0814west-final.txt	2005082515	2005083009	291130
PMHCPI12	h1947_0p25south_f.txt	1947091712	1947092018	191130
PMHCPI13	h1893_0p5_west_f.txt	1893092718	1893100406	020600
PMHCPI14	Camille_0p5west_f.txt	1969081418	1969081900	172200
PMHCPI15	Andrew_1p0east_f.txt	1992082412	1992082800	260600
PMHCPI16	1893_2p5_f.txt	1893092718	1893100406	020900
PMHCPI17	1893_2p5_f.int	1893092719	1893100412	011900

PMHCPI18	Andrew_2p0east_f.int	1992082813	1992090205	031000
PMHCPI19	Camille_0p5west_f.int	1969081418	1969081906	171900
PMHCPI20	h1893_0p5_west_f.int	1893092719	1893100411	011600
PMHCPI21	h1947_0p25south_f.int	1947091909	1947092403	220000
PMHCPI22	KatrinaTrack0_0p0814west-final.int	2005082502	2005082919	280800
PMHCPI23	KatrinaTrack1_0p0943east-final.int	2005082503	2005082919	280800
PMHCPI24	KatrinaTrack2_0p2852east-final.int	2005082504	2005082919	280700
PMHCPI25	KatrinaTrack3_0p5682east-final.int	2005082506	2005082919	281500
PMHCPI26	KatrinaTrack4_0p7341east-final.int	2005082506	2005082919	281500
PMHCPI27	KatrinaTrack5_0p9711east-final.int	2005082508	2005082919	281500
PMHCPI28	Rita_1p0east_f.int	2005092019	2005092523	241100
PMHCPI29	track_1915.int	1915092612	1915100204	301300
PMHCPI30	track_carmen.int	1974090320	1974091010	090500
PMHCPI31	Track_Katrina.int	2005082502	2005082919	280800
PMHCPI32	Track_Rita.int	2005092013	2005092523	241200
PMHCPI33	Andrew_1p0east_10kt_f.txt	1992082807	1992090205	310100

SENSITIVITY RUNS

A series of baseline storms were required to test an idealized ADCIRC grid layout over a domain roughly the size of the Gulf of Mexico. To achieve this objective, TC96 was run for a selected set of CPI, RMW and forward speed parameters specified by ERDC and the results were delivered in a form such that ERDC could translate/rotate the solutions to simulate a variety of storm scenarios. ERDC requested snapshots for CPI's of 40, 60, 80, 100 and 120 mb, with RMW of 10, 20 and 30 Nmi for a northward storm track with forward speed of 10 knots, along with 3 additional cases for CPI=80 mb/RWM=20Nmi for forward speeds of 5, 15, and 20 knots. In all, 18 cases were run. All storms were translated due north at a longitude of 91W which is roughly in the middle of the fine scale grid to give the best coverage of the storm. As with the SPH/PMH storm fields, the model solution is a steady state solution. As requested by ERDC, no landfilling model was applied for these cases. It should be noted that certain combinations of CPI and RMW maybe lie outside observed ranges in nature, but determination of these constraints are well beyond the scope of this study.

Table 3 Sensitivity study storm list

Reference	ERDC Requested Parameters	Start Date (CYMDH)	End Date (CYMDH)	Est. Landfall (DHM)
P04R1V10	CPI=40mb, Radius=10Nmi, Speed=10knts	2005091118	2005091606	None
P04R2V10	CPI=40mb, Radius=20Nmi, Speed=10knts	2005091118	2005091606	None
P04R3V10	CPI=40mb, Radius=30Nmi, Speed=10knts	2005091118	2005091606	None
P06R1V10	CPI=60mb, Radius=10Nmi, Speed=10knts	2005091118	2005091606	None
P06R2V10	CPI=60mb, Radius=20Nmi, Speed=10knts	2005091118	2005091606	None
P06R3V10	CPI=60mb, Radius=30Nmi, Speed=10knts	2005091118	2005091606	None
P08R1V10	CPI=80mb, Radius=10Nmi, Speed=10knts	2005091118	2005091606	None
P08R2V05	CPI=80mb, Radius=20Nmi, Speed=05knts	2005090812	2005091712	None
P08R2V10	CPI=80mb, Radius=20Nmi, Speed=10knts	2005091118	2005091606	None
P08R2V15	CPI=80mb, Radius=20Nmi, Speed=15knts	2005091300	2005091518	None
P08R2V20	CPI=80mb, Radius=20Nmi, Speed=20knts	2005091312	2005091512	None
P08R3V10	CPI=80mb, Radius=30Nmi, Speed=10knts	2005091118	2005091606	None
P10R1V10	CPI=100mb, Radius=10Nmi, Speed=10knts	2005091118	2005091606	None
P10R2V10	CPI=100mb, Radius=20Nmi, Speed=10knts	2005091118	2005091606	None
P10R3V10	CPI=100mb, Radius=30Nmi, Speed=10knts	2005091118	2005091606	None
P12R1V10	CPI=120mb, Radius=10Nmi, Speed=10knts	2005091118	2005091606	None
P12R2V10	CPI=120mb, Radius=20Nmi, Speed=10knts	2005091118	2005091606	None
P12R3V10	CPI=120mb, Radius=30Nmi, Speed=10knts	2005091118	2005091606	None

CAMILLE 1969 HINDCAST

Hurricane Camille is an important case since the storm made landfall with a central pressure near 908 mb (1013 far-field pressure, ΔP = 105 mb) with radius of maximum winds near 10 Nmi. In the context of the PMH (ΔP=130 mb) and SPH (ΔP=105 mb) Camille is very close at landfall to the SPH storm although the Holland's B parameter selected for this hindcast (1.20) results in maximum winds just shy of the SPH solution. Land filling for Camille is based on observational analysis rather than a filling model. The 1969_03 hindcast starts on Aug-14-1969 12:00 GMT and ends Aug-18-1969 18:00 GMT and applies the same working grids as the SPH/PMH and sensitivity runs. The track used is based on OWI's own independent analysis as based on all available fix data and, therefore, may differ slightly from previously published tracks such as HURDAT. Most earlier analyses of Camille have assumed steady-state intensity in the 24-hour pre-landfall period. The present simulation incorporates the modeling of a slightly more intense vortex offshore (minimum CPI of 901 mb as based on US Navy dropsonde). A second wind field

for Camille was generated using the same TC96 parameters but along the Camille_0p5west_f.txt track (see PMH reference PMHCPI14) which is an ERDC provided shifted track (now referenced as 0P5West). Since this 0P5West's landfall is approximately 6 hours previous to the Camille track the central pressure time history was adjusted to keep the landfall central pressure and resulting filling over land consistent with the new landfall time.

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Thompson, E. F. and V. J. Cardone. 1996. Practical modeling of hurricane surface wind fields. ASCE J. of Waterway, Port,Coastal and Ocean Engineering. 122, 4, 195-205.

Vickery P.J. and Lawrence Twisdale. Wind-Field and Filling Models for Hurricane Wind-Speed Predictions. Journal of Structural Engineering, Nov. 1993 pp 1700-1709.

APPENDIX A Win/Pre File Formats

Oceanweather WIN and PRE File Formats

Winds and pressure data formats are similar. The header format is the same, but in the wind file the header is followed by U then V components while in the pressure file the header is followed by just pressures.

The file begins with a header indicating the starting and ending dates and is followed by a grid/date header for each time step and the u and v components of the wind in meters/second or pressures in millibars. Starting/Ending dates are in YYYYMMDDHH format where:

```
      YYYY Year
      MM   Month
      DD   Day
      HH   Hour
```

example win:

```
OWI WWS Wind Output Ucomp,Vcomp in m/s           Start:1995060600 End:1995060600
iLat=  67iLong=  67DX= 1.250DY=  .833SWLat=  22.500SWlon= -82.500Dt=199506060000

  -1.16856  -1.06439   -.84875  -1.03460  -1.50047  -2.09462  -2.80243  -3.55863
  -4.24125  -4.84273  -5.59486  -5.37088  -5.30224  -5.12534  -4.89537  -4.67412
  -4.49203  -4.35772  -4.26612  -4.20260  -4.14746  -4.08396  -4.00686  -3.92213
  -3.83615  -3.74765  -3.65182  -3.54998  -3.45299  -3.37660  -3.32959  -3.28037
  -3.10631  -2.67723  -2.08363  -1.53773  -1.12623   -.83526   -.62870   -.47371
[rest deleted]
```

iLat is the number of parallels
iLong is the number of meridions
DX is the grid spacing in degrees of longitude
DY is the grid spacing in degrees of latitude
SWLat is the latitude of the South West corner
SWlon is the longitude of the South West corner
Dt is the date/time in YYYYMMDDHHmm (same as master header date format but with mm Minutes as well)

The number of grid points is iLat*iLong, the u component of the winds in meters/second is followed by the v component.

Sample fortran to read a win file (first time step only):

```
 c     Read in begining/ending dates of win file
 10    format (t56,i10,t71,i10)
       read (20,10) date1,date2

 c     Read Grid Specifications/Date
 11    format (t6,i4,t16,i4,t23,f6.0,t32,f6.0,t44,f8.0,t58,f8.0,t69,i10,i2)
       read (20,11) iLat, iLong, dx, dy, swlat, swlong, lCYMDH, iMin

 c     Read U/V Components of the wind
 12    format (8f10.0)
       read (20,12) ((uu(i,j),i=1,ilong),j=1,ilat)
       read (20,12) ((vv(i,j),i=1,ilong),j=1,ilat)
```

Latitude Longitude for each point can be calculated as follows:

```
       do 20  icnt = 1,iLat
         slat(icnt) = SWlat + (icnt - 1) * DY
```

```
 20     continue

        do 30  jcnt = 1,iLong
          slon(jcnt) = SWlong + (jcnt - 1) * DX
 30     continue
```

Wind Speed/Meteorological Wind Direction
can be computed from the u/v components as follows:

```
      WS = sqrt(uu(iLong,iLat)**2 + vv(iLong,iLat)**2)
      WDIR = mod(180.+atan2d(UU(iLong,iLat),VV(iLong,iLat)),360.)
```

ANNEX 3 – REVIEW OF THE APPLICATION OF THE NUMERICAL “ADCIRC” FOR STORM SURGE PREDICTIONS IN THE NEW ORLEANS, LA VICINITY

REVIEW OF THE APPLICATION OF THE NUMERICAL "ADCIRC" FOR STORM SURGE PREDICTIONS IN THE NEW ORLEANS, LA VICINITY

January 31, 2004

Report Prepared For:

New Orleans District
U. S. Army Corps of Engineers
New Orleans, LA

Report Prepared By:

Technical Review Committee:

Robert G. Dean
Mark Powell
Robert O. Reid

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EXECUTIVE SUMMARY

This report presents the results of a four year program to develop and evaluate the long wave model ADCIRC for purposes of calculating storm surge elevations to assess existing levee elevations in the vicinity of New Orleans and, where necessary, establish new elevations. The program was sponsored by the New Orleans District of the U. S. Army Corps of Engineers, the modeling was conducted by Professors Joannes Westerink and Richard Luettich, Jr. of Notre Dame and the University of North Carolina, respectively, and guidance was provided and model assessment was conducted by the authors of this report who were requested by the Corps to serve as a Technical Review Committee. Although the Corps' interest extended to all geographic areas in the vicinity of New Orleans, the primary focus was storm surges in Lake Pontchartrain and the Mississippi River.

The modeling program comprised two phases: (1) A model refinement and calibration phase, and (2) A set of production runs. The first phase was based on examination of the governing physics of storm surge generation and comparison with measured storm surges generated by Hurricanes Betsy (1965) and Andrew (1992). This first phase resulted in several significant improvements both in the modeling and in the characterization of the physical system. Following this first phase, the model parameters were "frozen", ie not changed further. The second phase comprised the calculation of storm surges produced by eleven hurricanes including the two that were used in the refinement/calibration phase. The wind and pressure field forcings were parameterized via a Planetary Boundary Layer (PBL) model in the first phase and for all eleven hurricanes in the second phase. In addition, the second phase included surge predictions using wind and pressure field forcing based on a method developed by the Hurricane Research Division (HRD) of the National Oceanographic and Atmospheric Administration (NOAA). These fields are developed after the occurrence of the hurricanes and are based on all available data. Thus, the HRD winds should provide the better forcing; however, the design will be carried out with the PBL forcing.

Hydrographs were provided by the second phase of the modeling effort of the ADCIRC surge predictions and measured surges were added where available. The TRC analyzed these hydrographs and developed quantitative measures of the predictive skill of the combined ADCIRC model with PBL and HRD forcings. It was found that in two of the three geographic areas examined (Lake Pontchartrain and Mississippi River), the HRD winds produced significantly better agreement with the measurements whereas in the third broad area (Southwest of New Orleans), PBL provided better agreement. This is interpreted as possibly due to less emphasis in characterizing the physical system in this latter area.

Recommendations for further improvement of the modeling include incorporation of wave setup and accounting, in the atmospheric forcing, for systematic differences which appeared in this project between HRD and PBL forcing.

Methodology is recommended for application of the skill assessment results presented herein to design and four design examples are presented.

REVIEW OF THE APPLICATION OF THE NUMERICAL MODEL "ADCIRC" FOR STORM SURGE PREDICTIONS IN THE NEW ORLEANS, LA VICINITY

1.0 Introduction

1.1 Background

This report presents the results of a four year effort to improve and assess the capability of the long wave numerical model "**AD**vanced **CIRC**ulation" (ADCIRC) to predict storm surges in that part of Louisiana and Mississippi south of Baton Rouge with a specific focus on storm surges in Lake Pontchartrain, levees surrounding New Orleans, and waterways within its vicinity. This effort was sponsored by the New Orleans District of the U. S. Army Corps of Engineers (USACE). The ADCIRC modeling was carried out by Professors Joannes J. Westerink of Notre Dame University and Richard A. Luettich , Jr. of the University of North Carolina at Chapel Hill, the two developers of the model. Logistics, information resources and guidance were provided by Staff of the New Orleans District of USACE, notably Mr. Jay Combe, Dr. Harley Winer, Dr. Hasan Pourtaheri (now with the Wilmington District of the Corps), Mr. Vann Stutts, Mr. Burnell Thibodeaux and many others. Technical oversight was provided by the Authors of this report who were appointed by the Corps to serve on this effort as a Technical Review Committee (Hereinafter, TRC).

The motivation for this effort is the question of whether the Lake Pontchartrain and other levees are of adequate elevation to protect the City of New Orleans and vicinity from storm surges associated with severe hurricanes, and if not, what changes are required. To address this question, the New Orleans District decided in Spring of 2000 to carry out investigations of the capabilities of the state-of-the-art numerical long wave model ADCIRC (about version 40) by the developers of this model with the TRC serving as an independent advisory and evaluation committee. The charge to TRC encompassed only a portion of the overall design chain required to determine potential storm surge elevations and arrive at a decision as to whether or not the Lake Pontchartrain levees and other levees are of sufficient elevation. The following paragraphs discuss the overall components leading to such a design determination and delineate the elements on which recommendations by the TRC were requested.

The overall determination of whether the Lake Pontchartrian and other levees are of adequate elevation depends on a number of factors, including: (1) Design storm surge calculations (2) Hurricane statistics, (3) Wave runup, (4) Levee crest elevations relative to an appropriate datum including effects of regional and local subsidence, and (5) Possible non-technical issues. The responsibilities of the TRC are limited further to a subset of the issues relating to the design surge calculations as discussed below.

The overall components which influence the design surge calculations include: (1) Accuracy and uncertainty in joint surge and tide calculations for a given hurricane

structure, intensity, path, and time of landfall, (2) Accuracy and uncertainty of the model used for representing the wind and pressure structure, (3) Statistics of the parameters governing the hurricane model, (4) Method of accounting for the hurricane statistics [Joint Probability Method (JPM) or Empirical Simulation Technique (EST)]. The TRC responsibility is only in the first item although we do address Item 2 and include discussion of some of the other items. As will be evident, developing an assessment of the accuracy of the ADCIRC model in predicting storm surges involves both evaluation of computation results and judgment.

To the best of our knowledge, this comprehensive effort to develop/calibrate and evaluate a numerical storm surge model is unprecedented in its scope and level of effort.

1.2 The Process

In general, the process included refinement/calibration/development of ADCIRC through further consideration of the governing physics and comparison with data from two hurricanes, Betsy (1965) and Andrew (1992). This phase was conducted using the Planetary Boundary Layer (PBL) model for wind and barometric pressure forcing. Following this phase, the model was fixed and production runs were conducted with the hurricanes listed in Table 1.1. The production runs also included wind and barometric pressure forcing of Hurricanes Betsy, Isidore and Lili based on the NOAA Hurricane Research Division (HRD) methodology for producing gridded wind and barometric fields from available data for the particular hurricane.

Table 1.1
Summary of Hurricanes Included in Effort, Their Roles
(Calibration and/or Production Runs) and Characteristics of Wind Fields

Hurricane	Calibration (C) and/or Production (P)	Wind Fields Employed
Unnamed (1947)	(P)	PBL
Audrey (1957)	(P)	PBL
Hilda (1964)	(P)	PBL
Betsy (1965)	(C) & (P)	PBL & HRD
Camille (1969)	(P)	PBL
Edith (1971)	(P)	PBL
Carmen (1974)	(P)	PBL
Andrew (1992)	(C) & (P)	PBL
Georges (1998)	(P)	PBL
Isidore (2002)	(P)	PBL & HRD
Lili (2002)	(P)	PBL & HRD

The Modelers produced hydrographs for locations of standard Corps of Engineers and other agencies gages which included measured and calculated surge histories. These were inspected to identify significant peak surges which were then subjected to statistical analyses to evaluate the ADCIRC model using PBL and HRD forcing and establish a basis for application of ADCIRC predictions for design using PBL forcing.

1.3 A Look at the Future

Future modeling improvements will undoubtedly be made to the various elements (wind fields, wind stress coefficients, wave setup, etc) on which the surge predictions depend. Additionally, with time, the historical hurricane statistical data base will be enhanced and subsidence issues will be clarified/updated. At some stage in the future, it would be appropriate to repeat the effort here including the follow on design calculations based on model and data base improvements.

1.4 Bibliography

A bibliography comprising papers and reports related to storm surges is presented as Section 8 of this report. Although only a small number of these documents are cited directly in this present report, the greater number is included for general background purposes.

2.0 Capabilities of and Improvements to ADCIRC

2.1 Background

When this four year project was initiated, ADCIRC included the capability of wetting and drying as the water level rose to exceed or fell to be less than the elevations of the adjacent topography. Additionally, at that stage the model had been developed over a number of years, was documented extensively in the peer reviewed literature as the references to this report will demonstrate and had undergone substantial testing. However, the calibration/development phase, which involved further consideration of the governing physics and comparisons with the measured storm surges of Hurricanes Betsy (1965) and Andrew (1992) established that a number of further modifications could result in greater realism in the model predictions.

This section documents the modifications to ADCIRC that were carried out during the calibration phase and attempts to provide, where possible, the motivation for the modifications and discussion of improvements achieved.

2.2 Modifications to ADCIRC During Calibration/Refinement Phase

The various modifications that were carried out during the design phase are presented in Table 2.1 and are discussed further in the following sections.

2.2.1 Slip Boundary Condition

The version of ADCIRC available at the commencement of this project included a “no-slip” boundary condition at the model lateral boundaries. It was found, in cases where the flow being represented was in narrow waterways, that the effect of the no-slip lateral boundary condition tended to overly retard the overall flow. Thus, a slip lateral boundary condition was introduced which resulted in improved agreement between measured and calculated flows as well as producing a more realistic surface slope.

2.2.2 Upstream Radiation Boundary Condition

Comparison of measured and calculated storm surges in the Mississippi River during Hurricane Betsy established that in the lower reaches, the surges were quite similar; however with increasing upriver distances, the shape of the surges differed with the calculated surges characterized by a double peak in some cases. Further examination of these calculated surges established that this effect was due to a reflected wave from the upstream model boundary. Several approaches were investigated with the final modification being the incorporation of a radiation boundary condition at the upstream boundary of the ADCIRC model which allowed the upstream propagating surge to continue upstream with relatively small reflection. This significantly suppressed the reflection of the surge reaching the simulated location of Baton Rouge and produced much more realistic results at the downstream locations in the Mississippi River simulations with Hurricane Betsy and Andrew forcing. The effectiveness of this

boundary condition was verified further by comparison with results from a model which extended a great distance upriver.

Table 2.1

Summary of Various Modifications Made to ADCIRC During the Calibration Phase

Modification	Motivation for Modification	Description of Modification
Slip Boundary Condition	Initial no-slip lateral boundary condition tended to overly restrain flow	Apply no-slip boundary condition to lateral boundaries
Upstream Radiation Boundary Condition	Initial reflection boundary condition at upstream Mississippi River resulted in surges that differed from measured farther up river	Radiation boundary condition applied at upstream ADCIRC boundary to allow surge to continue upstream through boundary
Wind Stress Relationship	Recognition that design winds will exceed those included in study calibration phase and the large ranges of stress relationships	Adopted the Garratt wind stress relationship with no limitation of the maximum stress coefficient
Land Roughness Coefficients	Presence of large expanses of open water, much of which is vegetated and excessive predicted blowdowns in Lake Pontchartrain.	Development of a look-up table for incorporation of local roughness effects.
Under Canopy Drag	Recognition of the large reductions in stress applied to water surface for the case of vegetation protruding through the water surface	Set under canopy wind stresses to zero

2.2.3 Wind Stress Relationships

A significant number of wind stress relationships have been proposed and are available in the peer reviewed literature. Of particular interest to this effort are the wind stress coefficients at high wind speeds. Recent results obtained from GPS drop sondes in hurricanes (Powell, et al, 2003) suggest that after an initial increase with wind speed to hurricane force, the drag coefficient levels off or even decreases with further wind speed increase. However, these results are applicable only over the open ocean and the authors caution their use near the coast where other studies in non-hurricane conditions (Anctil and Donelan, 1996) suggest that breaking and shoaling waves may contribute to larger sea surface roughness and accompanying increase in drag coefficients. Based on considerable sensitivity testing and evaluation, a decision was made for purposes of this study, to use the Garratt wind stress coefficient with no cutoff for high winds.

2.2.4 Land Roughness Coefficients

Land roughness can be due to vegetation, buildings, levees, etc and, through increasing the boundary layer thickness, or through direct transfer to the roughness elements rather than to the water, the wind stress applied to the water surface and resulting water surface slopes can be reduced considerably. This issue also encompasses wind direction since wind directed from a heavily vegetated area will require a transition distance before reestablishment of the normal stress transfer mechanisms.

During the development phase, it was recognized that the winds over land and especially, the offshore flow regions were depicting open ocean flow conditions which had the effect of much larger than observed draw-downs to the west of New Orleans during Hurricane Betsy. It was also recognized that some of the coastal inundation areas comprised forest which would impact the drag coefficient. These issues were addressed by applying a method in use by HRD to convert marine flow to that over open terrain. The required Land Use/Land Cover (LULC) data were obtained by a combination of available information supplemented by efforts by New Orleans District personnel to improve the characterization of the LULC data. This information formed the basis of a look up table which allowed the surface roughness to be characterized by wind direction to 20 km upwind from each affected grid point.

2.2.5 Under Canopy Drag

In forested and heavily vegetated areas that might be flooded, it is recognized that the wind is significantly reduced within such regions and thus the wind stress on the water beneath such a canopy is likewise significantly reduced. Reid and Whitaker (1976) give formulas for the wind reduction factor in terms of the typical dimensions of the vegetation and their number per unit horizontal area. Based on a number of sensitivity tests and lacking detailed information for the vegetated areas in the study area for this project, the production runs set the wind stress equal to zero in vegetated areas.

3.0 Uncertainties in the Process

3.1 General

There are uncertainties associated with the process of evaluating the skill of ADCIRC in both the calibration and production runs and in the predictions on which future decisions regarding levee elevation will be based. Some of the uncertainties in the calibration and production runs should be reflected in the later design predictions and others should not. This section addresses these issues and discusses the appropriateness for inclusion of the various sources in the later calculations on which the current levee elevations will be evaluated and, where necessary, design modifications based.

3.2 Uncertainty Sources

The possible sources of identified uncertainties are presented in Table 3.1. Also indicated is the manner in which these sources contribute and whether they should be considered as contributing to the design process. Each of these sources is discussed below.

3.2.1 Wind and Pressure Fields

To provide a basis for estimating the portion of error associated with the PBL winds and barometric pressure fields used to force the long wave model ADCIRC, the Corps requested that NOAA's Hurricane Research Division (HRD) conduct a reconstruction of the Hurricane Betsy wind field and to provide additional meteorological information. Reconstruction of the Hurricane Lili and Isidore wind fields were added after the 2002 hurricane season. These reconstructions are based on input from all available sources including anemometers, barometers, research aircraft and satellites and the data contributing to the reconstruction are screened thoroughly for quality. The methodology has been applied to analyze the landfall characteristics of several major hurricanes as documented in peer reviewed literature (Powell, et. al., 1991, Powell and Houston, 1996, 1998, Powell, et. al. 1998). The resulting reconstructed wind fields depict the wind direction and speed over a fine resolution Cartesian grid, and depict the radius of maximum wind. The radius of maximum wind and the minimum sea-level pressure in the eye of the storm were specified every 3 hours for each storm track. The current accuracy to which wind centers of tropical cyclones may be located is approximately 1 km. It should be noted that the wind, pressure, and geometric eye centers of tropical cyclones do not always coincide and may vary be several km. In addition, hurricanes in the process of transforming into extratropical systems will frequently have considerable tilt in the wind centers of tens of km between the surface and the height where most reconnaissance is flown (3 km). The HRD representations should provide the better representations for forcing the long wave model. Uncertainties in the Hurricane Betsy wind field are considered greater than those for the Lili and Isidore fields since Betsy predated the era of high quality aircraft data communications.

Table 3.1

Sources of Uncertainties in the Overall Process and Their Significances

Uncertainty Source	Uncertainty Origin/ Explanation	Should be an Uncertainty Source in The Design Process?	Comment
Wind and Pressure Fields, Actual vs Model	Variability of Actual Wind Fields	Yes	Inherent in Variability Present in Production Runs
Any Inaccuracies in ADCIRC Model	Complexities in Hydrodynamics	Yes	Inherent in Variability Present in Production Runs
Inaccuracies in Representing Characteristics of Natural System	Complexity of Natural System,: Topography, Vegetation, Roughness, etc	Yes	Inherent in Variability Present in Production Runs
Inaccuracies in Hurricane Tracks	Tracks of Production Runs May Contain Inaccuracies	No, if the Historical Hurricane Data Base is Accurate	Inherent in Variability Present in Production Runs
Wind Stress Coefficients	Canopy Effects, Coefficients at High Wind Speeds, etc	Yes	Inherent in Variability Present in Production Runs
Historical Hurricane Data Base	Limited Number of Severe Strength Storms	Yes	This Will Be a Factor in the Later Design Calculations
Subsidence Effects			Not Inherent in Variability Present in Production Runs. Will Be a Factor in the Later Design Calculations
Steric Effects	Inherent Variability in Mean Seasonal Water Level Fluctuations	Yes	Variability Not Present in Production Runs
Wave Setup	Not in ADCIRC	Not if Incorporated in ADCIRC	Explicit Incorporation Would Reduce Uncertainty
JPM or EST	Method of Representing Storm Statistics	Yes	Consideration Required in Design Calculations

Comparisons of the PBL and HRD wind fields have identified several significant and systematic differences between these forcings. In particular, the PBL model has a

tendency to predict too broad a region of strong winds in most storms. Additionally, after landfall, the PBL model predicts too much broadening of the wind field. This appears to be a result of a built in dependency in the PBL model of the radius to maximum winds and the central pressure deficit. Because the PBL model will be used for levee elevation evaluation and design, these issues should be addressed in future refinements of the PBL model.

Obviously, the HRD wind fields will not be available for design purposes although it is possible that in the future some of the systematic characteristic differences found between the HRD and the PBL wind fields (as noted above) could be incorporated in the design phase to provide an improved model. At this stage, we consider that the benefits of the HRD fields will not be available for design purposes. Rather, these benefits will be developed and applied in the future and will provide improved meteorological forcing to ADCIRC and thus contribute to reducing the uncertainties in ADCIRC predictions, given a particular meteorological forcing. It will be shown later that the HRD wind fields generally (2 of 3 areas) produced better agreement than the PBL winds with the measured surges allowing for a constant factor difference which is interpreted as due to calibration based on PBL rather than HRD wind fields.

As noted, the PBL model (or a future improved model) will be used in the later design calculations. Although some questions remain regarding the appropriateness of the tracks and hurricane radii used in the production runs, if those tracks and radii were appropriate, the results of the comparisons of the production run surges and the measured surges based on the ADCIRC model will incorporate the proper uncertainty to be included in the design calculations.

3.2.2 Inaccuracies in the ADCIRC Model

Section 2 of this report detailed a number of substantial improvements that were made to the ADCIRC model during this four year effort. Undoubtedly other improvements will follow including some means of incorporating wave setup (discussed in more detail later) in the storm surge predictions. Any present inaccuracies in the ADCIRC model would be reflected in the statistics of the comparisons of the predicted and measured peak surges.

3.2.3 Inaccuracies in Characterizing Natural System

Efforts were made to characterize the natural system faithfully including the vegetation characteristics, relative ground levels at the times of the historical storms included in this effort, bathymetry, etc. However, future efforts may establish further improved characterizations of the natural system. Regardless, predictions of surges due to future hurricanes should include a conscientious effort to consider a range of relative ground elevations consistent with known rates of subsidence and ranges of times of storm occurrence. If relative sea level continues to rise at present rates and any corrective actions are relatively ineffective, future conditions will include more open water and less vegetation. Regardless, any inaccuracies in characterization of the present and past

system should be appropriately represented in the statistics of the comparison of the predicted and measured peak surges.

3.2.4 Inaccuracies in Hurricane Tracks

Some uncertainty remains as to whether some of the Production Runs were conducted with the best hurricane track descriptions. Any inaccuracies in the hurricane track should have the effect of increasing the differences between the predicted and measured peak surges. Thus, this should result in a recommended uncertainty to be considered in design which is greater than the actual uncertainty, ie the design calculations would produce overly conservative results through incorporation of uncertainties which are too large.

3.2.5 Wind Stress Coefficients

The Garratt wind stress coefficients were incorporated into the calculations. There are relatively few measurements at the higher severe storm associated wind speeds to be considered in design and this is an area of active research. Ai discussed in Section 2.23, recent investigations suggest that wind stress coefficients may level off or decrease at very high wind speeds. If the wind stress coefficients were constant with increasing wind speed, an inappropriate stress coefficient would be offset through calibration (probably adjusting the wind stress coefficient). However, for stress coefficients which vary with wind speed, such an adjustment is only appropriate for the wind speeds represented in the calibration and the surges in rare hurricanes could tend to be overestimated. For purposes here, the Garratt wind stress relationship was adopted without consideration of the limiting stress coefficients at high wind speeds.

3.2.6 Steric Effects

The steric contributions to mean water level are a result of the seasonally reduced density of the water due to thermal and fresh water effects. Thus, the steric contribution can be characterized by a monthly average and a statistical distribution about that average. As will be evident later, uncertainties in steric effects were avoided in the evaluation data base which comprised the differences between the peak surges (measured and computed) and their respective mean water levels immediately prior to the hurricane induced water levels. In design it will be necessary to include both the mean steric contribution (appropriate for the hurricane season months) and an approximation of the contribution due to the statistical uncertainty in components of steric water levels.

3.2.7 Historical Hurricane Data Base

The historical hurricane data base statistics will be used later in the surge calculations to determine whether or not levee elevations are adequate, and if not, the appropriate additional elevations. Although uncertainties in this data base do not affect the results of the present study, these uncertainties will contribute to assessment of the return periods of the peak surges determined at the design stage.

3.2.8 Subsidence

Past and future subsidence and eustatic sea level rise are of relevance to the later design stage. Subsidence includes both regional effects and local effects due to the weight of the levees. Considering the high relative sea level rise rates in the general Mississippi River delta region (order of 100 cm per century) and lower rates in the vicinity of New Orleans, current eustatic (world wide average) sea level rise rates of approximately 12 cm per century are relatively small. Any effects of subsidence are not incorporated into the uncertainties determined herein because we base the evaluation on the peak surge deviations from the pre hurricane effect water level and an attempt was made to base the relative sea level in the calculations on the relative water levels at the time of hurricane occurrence. However, appropriate incorporation of subsidence effects (regional and local, present and future) will be critical to the assessment of the adequacy of present levee elevations and the design for future levee elevations.

3.2.9 Wave Setup

Wave setup is the surge component resulting from the transfer of wave related momentum to the water column as a result of wave dissipation. The effect of this transfer is in the form of a force on the water column much like that due to a wind stress although the force due to wave momentum transfer is distributed throughout the vertical dimension of the water column rather than at the surface for wind stress. Although the basic physics of wave related setup are well understood, to date explicit incorporation into ADCIRC has been precluded by the complexities associated with this incorporation. Wave setup was not included explicitly in the calculated results presented herein.

3.2.10 JPM or EST

The Joint Probability Method (JPM) and the Empirical Simulation Technique (EST) are the two general approaches to incorporating the historical hurricane characteristics into the simulation of a series of storms and the quantification of the return periods of the resulting storm surges. This is not an issue in the present effort and questions regarding which of these two methods is more applicable will require addressing during the design phase. Since these two methods rely on the same historical hurricane data base, properly applied, they should yield the same approximate results. Considering the significance of the overall question of adequacy of levee elevations, at this stage, it appears reasonable to apply both methods in the design phase.

4.0 Development and Analysis of a Peak Surge Data Base

4.1 Effects Included in the Calculated Storm Surge Hydrographs

The physical effects included in the development of the calculated storm surge hydrographs included: (1) Astronomical tides, (2) Seasonal steric effects, (3) Overland wind reduction factors including effects of wind direction near water bodies, and (4) Wind reduction due to vegetation.

4.2 Development of a Peak Surge Data Base

The Modelers produced hydrographs at the locations of the available water level gages and plots were provided of the measured and calculated surges. These were analyzed as follows. The available hydrographs were examined to identify measured surges of significant magnitude. When such a peak surge was identified, at least four water levels were read from the hydrograph and entered into a data base. These water levels are shown in Figure 4.1 and included the two (measured and predicted) water levels before the effects of the hurricane became evident ($h_{m,o}$ and $h_{p,o}$) and the two peak surges ($h_{m,\max}$ and $h_{p,\max}$) where the subscripts "m", "p","max" and "o" denote "measured", "predicted", "maximum" and "initial", respectively. In those cases in which HRD winds were also used as forcing to ADCIRC, the two associated water levels (before hurricane effects and peak surge) were also read and entered into the data base. A total of 208 combinations of measured surge locations/hurricanes formed the data base. All of these 208 measured surges had an associated predicted surge based on the PBL model and 66 of the measured surges had an associated predicted value based on HRD forcing. The analysis presented below was based on the <u>peak surge anomaly</u>, ie the differences between the peak water levels and the water level prior to storm effects becoming evident, for example, for the measured surges: $\Delta h_m = h_{m,\max} - h_{m,o}$. The differences in the timing of the measured and calculated surges was not a factor in assessing the accuracy of the calculations. The data base is termed "ALLDAT.INP" and a listing is presented as Appendix B and has been provided to the Corps in digital form.

As noted above, no attempt was made to analyze the time shifts between the measured and calculated hydrographs; however, these were generally small and large time shifts usually corresponded to cases in which there was a relatively large difference between the measured and calculated peak surges. Finally, the astronomical tide signal was evident in both the measured and calculated hydrographs, particularly in the portions prior to the occurrence of the storm surges. No effort was made to compare the timing or magnitudes of the astronomical tidal signals for the purpose of model skill assessment as this issue had been addressed in detail by the Modelers during the Development/ Verification Phase of this overall effort.

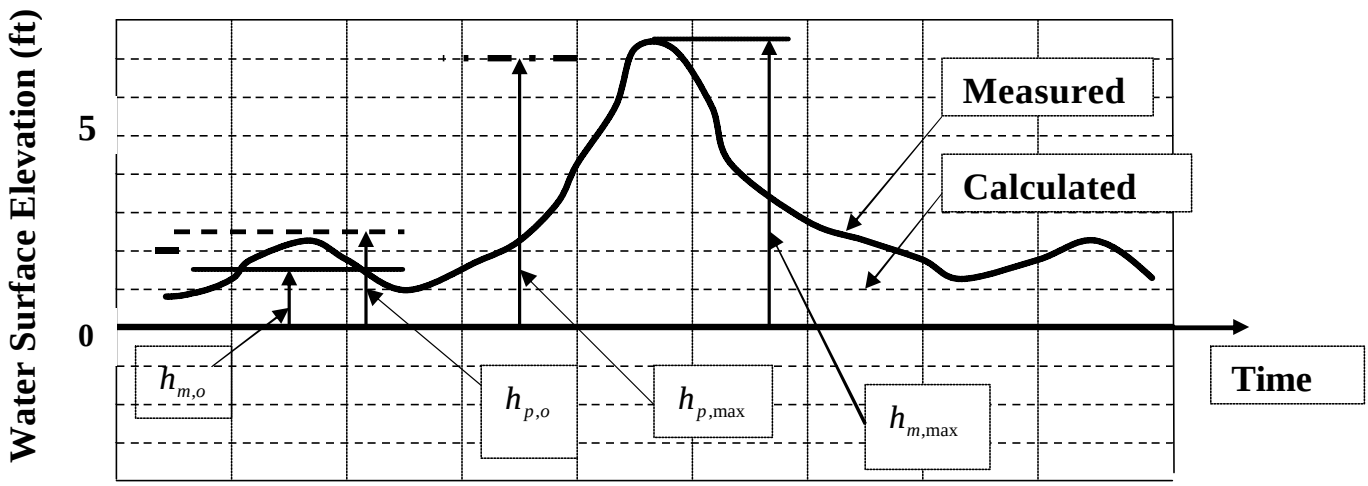

Figure 4.1. Method of Reading Surge Characteristics.

4.3 Regions Considered

Analyses were carried out and are presented for three separate regions: (1) Lake Pontchartrain, (2) Mississippi River, and (3) The area southwest of New Orleans. These three areas and the locations and numerical designations of the water level gages that will be referred to later are shown in Figures 4.2, 4.3 and 4.4. Not all gages were functional during all of the storms examined. In addition to these three areas, the data from all locations are combined and analyzed.

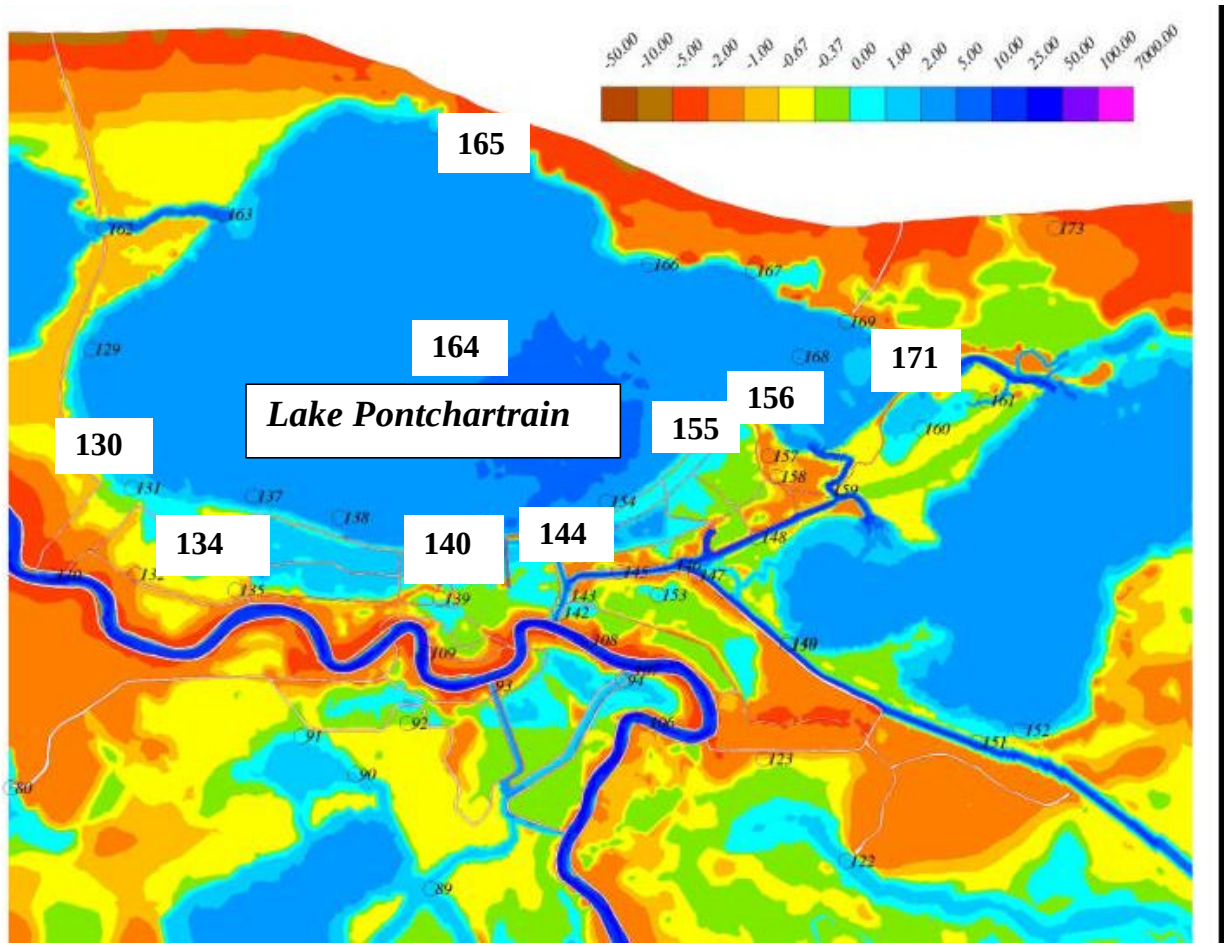

Figure 4.2 Locations of Lake Pontchartrain Gages Analyzed in This Report.

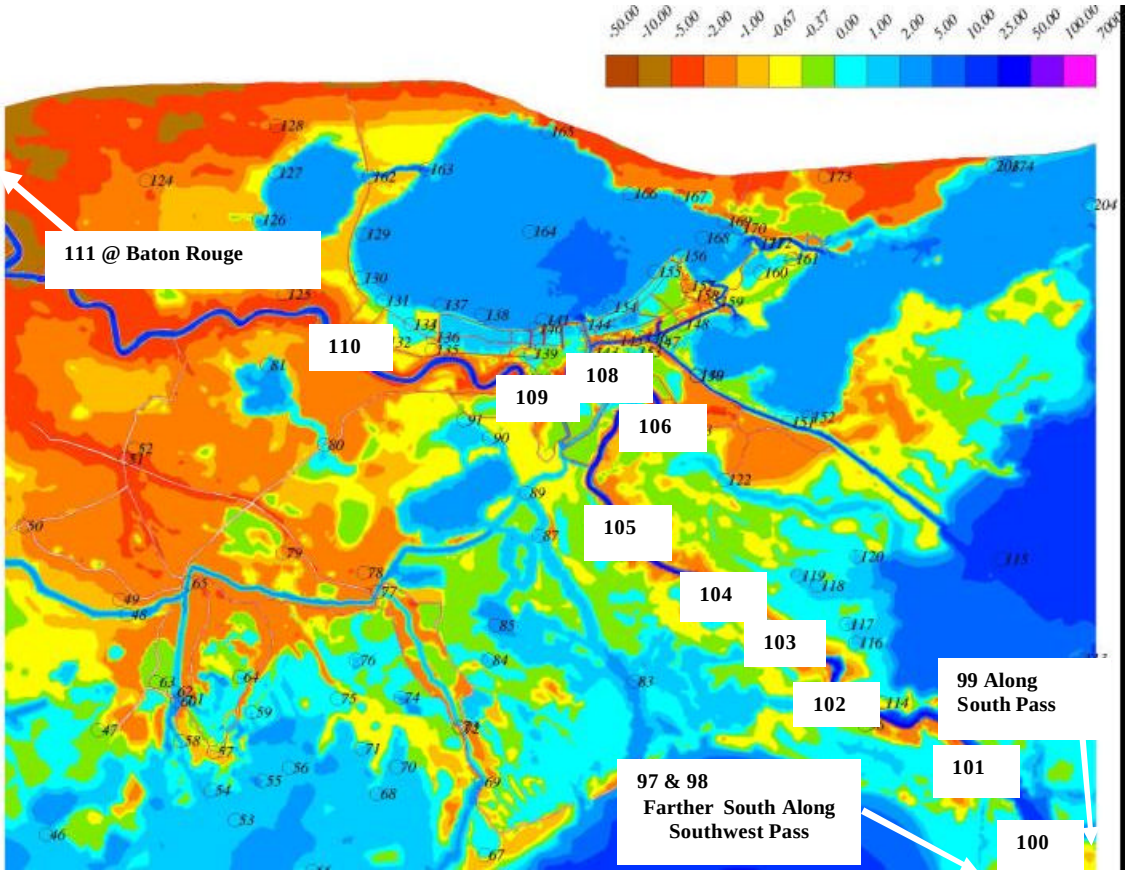

Figure 4.3 Locations of Mississippi River Gages Analyzed in This Report.

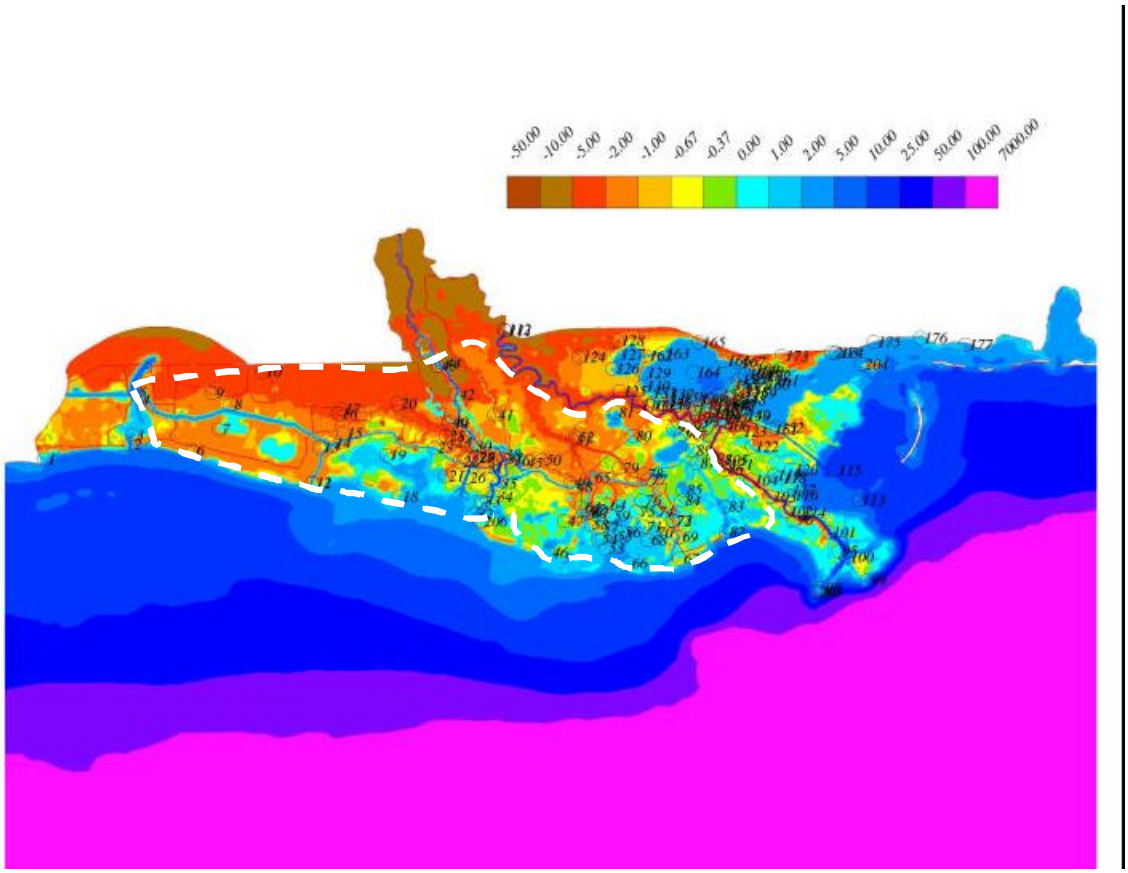

Figure 4.4 Locations of Southwest Area Gages Analyzed in This Report.

4.4 Statistical Analysis of ADCIRC Predictions

4.4.1 General

The production run results were subjected to a statistical analysis to quantify the capabilities of the combination of the ADCIRC model and the wind fields to predict storm surges in agreement with those measured. Various combinations of wind field representations and regional areas (ie, Lake Pontchartrain, Mississippi River and Southwest Area) were examined to determine the capabilities on a regional basis. Additionally, in some cases in which sufficient measurements were available, the data were stratified according to measured storm surge maxima because the larger storm surges are of primary interest. These results are presented in Table 4.1a,b,c,d and the various column entries are first discussed below followed by examination of the table results and the significance to the issue of ADCIRC modeling capabilities for design purposes. It is noted that of the total number of measured peak surge values (208), 34 peak surges are located in Lake Pontchartrain.

Table 4.1a. Lake Pontchartrain Results

Case/ Hurr	Model	Addl. Desc.	N	Gain	Std Dev (ft)	Surge rms (ft)	Diff of Means (ft)
Isidore	**PBL**	**Lake Pont**	**5**	**1.12**	**1.34**	**4.59**	**0.78**
Isidore	**HRD**	**Lake Pont**	**5**	**1.49**	**1.07**	**4.59**	**1.60**
Lili	**PBL**	**Lake Pont**	**4**	**1.43**	**1.21**	**3.57**	**1.25**
Lili	**HRD**	**Lake Pont**	**4**	**2.06**	**0.72**	**3.57**	**1.85**
Camille	**PBL**	**Lake Pont**	**6**	**0.90**	**1.12**	**4.4**	**- 0.10**
Betsy	**PBL**	**Lake Pont**	**7**	**0.79**	**0.87**	**3.72**	**- 0.73**
Betsy	**HRD**	**Lake Pont**	**7**	**1.49**	**0.73**	**3.72**	**1.26**
Georges	**PBL**	**Lake Pont**	**4**	**1.31**	**0.88**	**3.32**	**0.73**
Edith	**PBL**	**Lake Pont**	**3**	**0.49**	**0.64**	**1.39**	**- 1.20**
Hilda/Andrew/ 1947 Storm	**PBL**	**Lake Pont**	**5**	**0.84**	**0.91**	**3.23**	**- 0.48**
Δh **< 4 ft**	**PBL**	**Lake Pont Data**	**23**	**0.84**	**1.04**	**2.92**	**- 0.24**
Δh **>4 ft**			**11**	**0.99**	**1.59**	**4.99**	**0.50**
All Δh			**34**	**0.92**	**1.28**	**3.72**	**0.00**
All Δh	**HRD**	**All Lake Pont Data**	**16**	**1.56**	**0.96**	**3.98**	**1.51**
Same Hurricanes as HRD, All Δh	**PBL**	**Lake Pont Data**	**16**	**0.95**	**1.40**	**3.98**	**0.24**

Stations: 130, 134, 140, 144, 155, 156, 164, 165, 171

Table 4.1b. Mississippi River Results

Case/ Hurr	Model	Addl. Desc.	N	Gain	Std Dev (ft)	Surge rms (ft)	Diff of Means (ft)
Δh < 4 ft	PBL		29	1.04	1.06	2.90	0.42
Δh >4 ft			19	1.25	4.42	7.25	3.32
All Δh			48	1.19	2.92	5.37	1.59
Δh < 4 ft	PBL Except for Georges		28	1.03	0.87	2.87	0.32
Δh >4 ft			11	1.13	2.94	8.03	1.33
All Δh			39	1.10	1.74	4.91	0.60
Δh < 4 ft	HRD		12	1.45	0.63	3.49	1.17
Δh >4 ft			4	1.11	0.26	10.33	1.03
All Δh			16	1.17	0.84	5.98	1.13
Δh < 4 ft	PBL	Same Hurricanes as HRD	12	1.06	1.24	3.49	0.68
Δh >4 ft			4	1.52	0.26	10.33	3.50
All Δh			16	1.34	1.46	5.98	1.38

Stations: 97, 98, 99, 100, 101, 102, 103, 104, 105, 106, 107, 108, 109, 110, 111

Table 4.1c. Area Southwest of New Orleans

Case/ Hurr	Model	Add'l. Desc.	N	Gain	Std Dev (ft)	Surge rms (ft)	Diff of Means (ft)
Δh < 4 ft	PBL		65	0.56	1.51	2.82	-1.16
Δh >4 ft			27	0.90	1.35	5.34	-0.33
All Δh			92	0.70	1.67	3.74	-0.91
Δh < 4 ft	PBL Except for Georges		61	0.56	1.55	2.89	-1.21
Δh >4 ft			27	0.90	1.35	5.34	-0.33
All Δh			88	0.70	1.70	3.81	-0.94
Δh < 4 ft	HRD		15	1.07	1.08	3.25	0.52
Δh >4 ft			5	1.01	1.71	5.62	0.64
All Δh			20	1.04	1.27	3.98	0.55
Δh < 4 ft	PBL	Same Hurricanes as HRD	15	0.86	0.94	3.25	-0.23
Δh >4 ft			5	0.92	1.09	5.62	-0.40
All Δh			20	0.89	0.99	3.98	-0.28

Stations: 5 through 89

Table 4.1d. All Areas Combined

Case/ Hurr	Model	Addl. Desc.	N	Gain	Std Dev (ft)	Surge rms (ft)	Diff of Means (ft)
Δh < 4 ft	All PBL		135	0.67	1.49	2.86	- 0.54
Δh >4 ft			73	1.03	2.83	6.16	1.14
All Δh			208	0.86	2.21	4.32	0.05
Δh < 4 ft	All PBL Except Georges		124	0.56	1.49	2.91	- 0.62
Δh >4 ft			64	1.00	2.11	6.02	0.51
All Δh			188	0.84	1.89	4.24	- 0.24
Δh < 4 ft	All HRD		40	1.09	1.47	3.29	0.74
Δh >4 ft			26	1.17	1.74	6.46	1.49
All Δh			66	1.15	1.59	4.80	1.03
Δh < 4 ft	PBL	Same Hurricanes as HRD	40	0.87	1.35	3.29	0.02
Δh >4 ft			26	1.09	2.07	6.46	0.92
All Δh			66	1.01	1.73	4.80	0.37

4.4.2 Description of Entries in Table 4.1

4.4.2.1 First Three Columns

The combinations analyzed separately are defined in the first three columns in Table 4.1. For example, the first entry in Table 4.1a is for Hurricane Isidore peak surges in Lake Pontchartrain using the PBL model for forcing.

4.4.2.2 Column 4

Column 4 is the total number of storm surge measurements available for the case being examined. For example, for the first case in Table 4.1a, there are 5 peak surges in Lake Pontchartrain in Hurricane Isidore for which the PBL model calculations were available.

4.4.2.3 Column 5: "Gain"

The "Gain" is a factor by which the individual calculated surges must be increased to provide the best least squares agreement with the measured surges. Thus, for example, referring to the first case of Table 4.1a, it is seen that it is necessary to multiply the calculated PBL generated peak surges by a factor of 1.12 to provide the best least squares agreement with the measured peak surges. The gain will be denoted by "m".

4.4.2.4 Column 6. "Std Dev"

Std Dev is defined as the root mean square of the differences between the least squares results and the measurements. Stated differently, denoting the ith measured peak surge anomaly as $\Delta h_{m,i} = h_{m,\max,i} - h_{m,o,i}$ and the calculated peak surge anomaly as $\Delta h_{p,i} = h_{p,\max,i} - h_{p,o,i}$ and the gain as m, the Std Dev is defined as

$$Std\ Dev = \sqrt{\frac{1}{I}\sum_{1}^{I}[\Delta h_{m,i} - m\Delta h_{p,i}]^2}$$

where I is the number of data points included in the analysis in the specific entry in the table. The significance of the Std Dev is that the smaller this value, the closer the model predictions agree with the measurements within a factor (the gain). If this factor is reasonably constant over a range of conditions and storms, the implication is that there is a reasonable basis for applying the results to design with the correction factor. Complicating this interpretation is the fact that the calibration phase of ADCIRC employed the PBL winds. Thus, even though the PBL winds may yield smaller Std Dev values, this may be a reflection of the calibration basis rather than the accuracy of the PBL wind fields. This will be discussed in greater detail later. As an example, it is seen that the Std Dev associated with the first case in Table 4.1a is 1.34 ft.

4.4.2.5 Column 7. "Surge rms"

The "Surge rms" is simply a measure of the overall magnitude of the measured peak surge anomalies included in the specific analysis results, defined by

$$Surgerms = \sqrt{\frac{1}{I}\sum_{1}^{I}(\Delta h_{m,i})^2}$$

Thus, those data that are stratified to include only measured surges above the lower level and less than the upper level are included in the analysis. For a lower level of 4 feet, the Surge rms will always be greater than (or remotely possibly equal to) 4 feet. For example, for the "PBL, Lake Pontchartrain", Δh > 4 ft" case, the Surge rms is 4.99 ft.

4.4.2.6 Column 8. "Diff of Means"

The "Difference of Means" represents the average difference between the measured and calculated peak surge anomalies, ie

$$Differenceof\ Means = \frac{1}{I}\sum_{1}^{I}[\Delta h_{m,i} - \Delta h_{p,i}]$$

for example, for the first entry in Table 4.1a, for the 5 surges included in this analysis, the average of the measured surges is 0.78 ft greater than that of the ADCIRC calculated surges. Note that this difference is consistent with the Gain for this data set being greater than unity, ie it is necessary to increase the calculated surges to obtain the best least square agreement with the measured surges.

4.5 Interpretation of Statistical Results in Table 4.1

The following paragraphs discuss the results presented in Table 4.1

4.5.1 Table 4.1a. Lake Pontchartrain Results

Table 4.1a presents results for various combinations of storms, meteorological forcing and measured surge stratification levels. Figure 4.5 presents a plot of all of the measured and PBL based predicted storm surges analyzed for Lake Pontchartrain. Two storms, Carmen and Audrey did not cause storm surges in Lake Pontchartrain of sufficient magnitude to be included in the analysis. As an indication of the applicability of these results to design in Lake Pontchartrain, if no effect of surge level (stratification) is considered, a gain factor of 0.92 and a standard deviation of 1.28 ft would be applied.

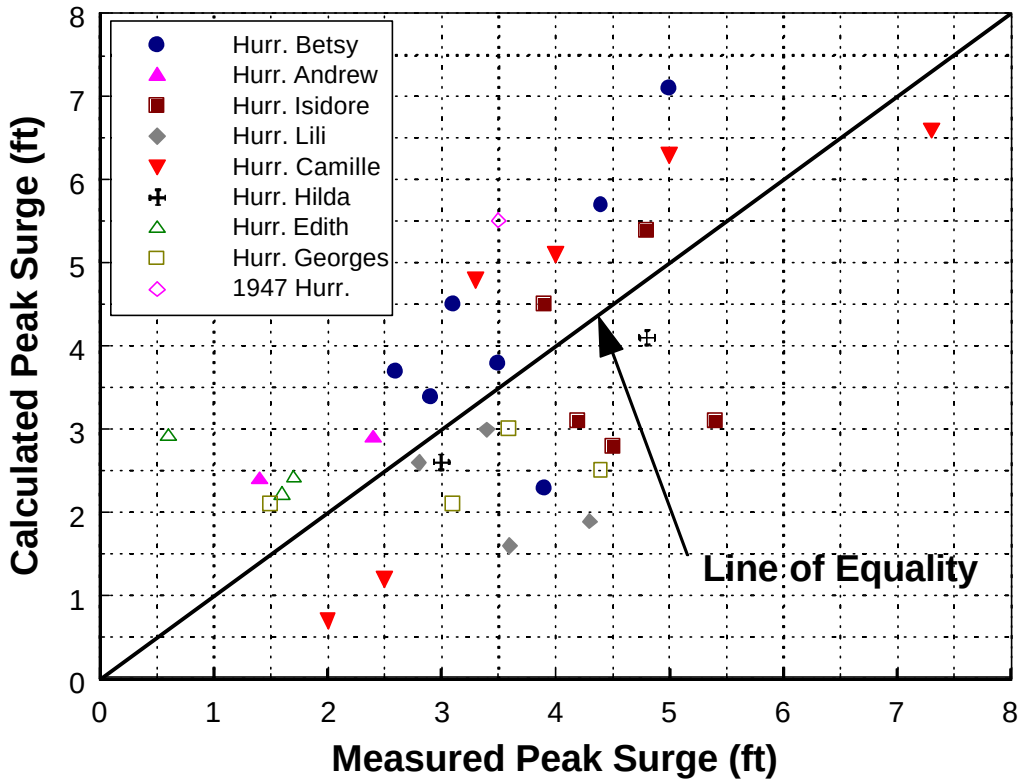

Figure 4.5 Plot of Measured vs Calculated Peak Surge Values for Lake Pontchartrain Data for All 34 Data Points Analyzed. PBL Winds.

Referring to the data in Table 4.1a for which the PBL calculations were conducted for the same hurricanes as available for the HRD calculations, it is of interest that although the ADCIRC model was refined/calibrated using Hurricanes Betsy and Andrew with the PBL winds, and the PBL winds for Betsy have a Gain closer to unity (0.79 for PBL winds vs 1.49 for HRD winds), the Std Dev is somewhat greater for the PBL winds than for the HRD winds (0.87 ft vs 0.73 ft). This issue will be addressed in greater detail later. As noted previously, it is our understanding that the PBL winds will be used in the design applications.

Examining all data for Lake Pontchartrian, it is seen that the results are consistent with the above findings: the Gain is greater for the HRD winds; however, the Std Dev values are less for the HRD winds.

4.5.2 Table 4.1b. Results for Mississippi River

These results are presented for: (1) All PBL data with and without Hurricane Georges, and (2) The PBL and HRD winds for Hurricanes Betsy, Isidore and Lili. All results are stratified for $\Delta h < 4$ ft and $\Delta h < 4$ ft and all Δh. The rationale for excluding Hurricane Georges in some of the PBL calculations was due to the anomalously low surges predicted by Hurricane Georges compared to the measured surges. The discussion here will focus on the results for the measured surge anomalies greater than 4 ft due to their greater relevance. As noted previously, the effects of the 8 surges in this range associated with Hurricane Georges are evident in comparison of the PBL results with and without Hurricane Georges. The Std Dev values are considerably greater for the case which includes the Hurricane Georges results (4.42 ft vs. 2.94 ft).

Comparison of results from PBL and HRD results for the common hurricanes (Betsy, Isidore and Lili) is not very meaningful when limiting the data to measured storm surges greater than 4 ft since there are only 4 values for which HRD values are available. However, for the "All Δh" category, the surge rms is nearly 6 ft, and the values of the Std Dev for HRD and PBL based surge anomalies are 0.84 ft and 1.46 ft, respectively. In fact, the PBL value of Std Dev would be even greater if one used a gain closer to unity.

4.5.3 Table 4.1c. Results for Southwest Area

This area comprised Station numbers 5 through 89 with this general area depicted in Figure 4.4. Also relevant, as noted previously, is that because the primary focus of this overall effort in the model calibration/refinement phases was on Lake Pontchartrain, and secondarily in the Mississippi River, the least care in land elevations, bathymetry and other details was directed to this area. As seen from Table 4.1c, there are 92 surges (of the 208 total) in this area, only four of which were caused by Hurricane Georges.

Inspecting Table 4.1c, it is seen that Hurricane Georges has little effect on the Std Dev values. Also of interest is that for the same storms, the HRD Std Dev values are greater than for the PBL results which differs from the two other regions examined previously.

4.5.4 Table 4.1d. All Data Results

The PBL data entering into Table 4.1d include 34 more surges than in the sum of the PBL entries in Tables 4.1a, 4.1b and 4.1c. Most of these additional locations are between the Mississippi River and the Mississippi River Gulf Outlet. Referring to Table 4.1d, for the PBL forcing and for measured storm surge anomalies greater than 4 ft including the 9 surge values due to Hurricane Georges, results in a gain of 1.03 and a Std Dev of 2.83 ft. For the same considerations except with the 9 surge values associated with Hurricane Georges excluded, the Gain is 1.00 and the Std Dev is 2.11 ft, somewhat greater than found for Lake Pontchartrain (1.59 ft.). For meteorological forcing by the HRD wind fields (which do not include Hurricane Georges), and for the 26 measured storm surge values greater than 4 feet, the gain is 1.17 and the Std Dev is 1.74 ft. Note that these

results are consistent with those found for surges limited to Lake Pontchartrain and the Mississippi River. That is, the Gains were greater based on HRD forcing and the Std Dev values were slightly less. It is also noted that for the available data set, the Surge rms is greater for this data set than for the data from Lake Pontchartrain. Of course, the results in Table 4.1d include those of all three regions discussed previously.

4.6 Interpretation of Results

Comparison of the peak surge anomaly statistics for the Lake Pontchartrian, Mississippi River and the Southwest Area have been reviewed. It is useful to attempt to develop a unified interpretation of these results for design application using PBL forcing. Referring to Tables 4.1abcd, it is seen that there may be correlations between the following parameters and surge elevation: Std Dev, Gain, and Difference of Means. The categories for PBL forcing are summarized in Table 4.2 and discussion and plots follow.

Table 4.2

Summary of Overall Characteristics of Fits to Data

Case/ Hurr	Model	Addl. Desc.	N	Gain	Std Dev (ft)	Surge rms (ft)	Diff of Means (ft)
$h < 4$ ft	PBL	Lake Pontchartrain Data	23	0.84	1.04	2.92	- 0.24
$h > 4$ ft			11	0.99	1.59	4.99	0.50
$h < 4$ ft	PBL Except for Georges	Mississippi River Data	28	1.03	0.87	2.87	0.32
$h > 4$ ft			11	1.13	2.94	8.03	1.33
$h < 4$ ft	PBL Except for Georges	Southwest Area	61	0.56	1.55	2.89	- 1.21
$h > 4$ ft			27	0.90	1.35	5.34	- 0.33

Figure 4.6 presents a plot of the Std Dev versus the RMS surge value. It is seen that a reasonable correlation exists with the following least squares relationship including the Mississippi River results

$$Std\ Dev = 0.135 + 0.315(Surge)_{rms}$$

however, the increase of Std Dev with Surge is due almost entirely to the one point from the Mississippi River for which Std Dev = 8.03 ft. Excluding the Mississippi River results reduces this trend significantly and is considered more reasonable.

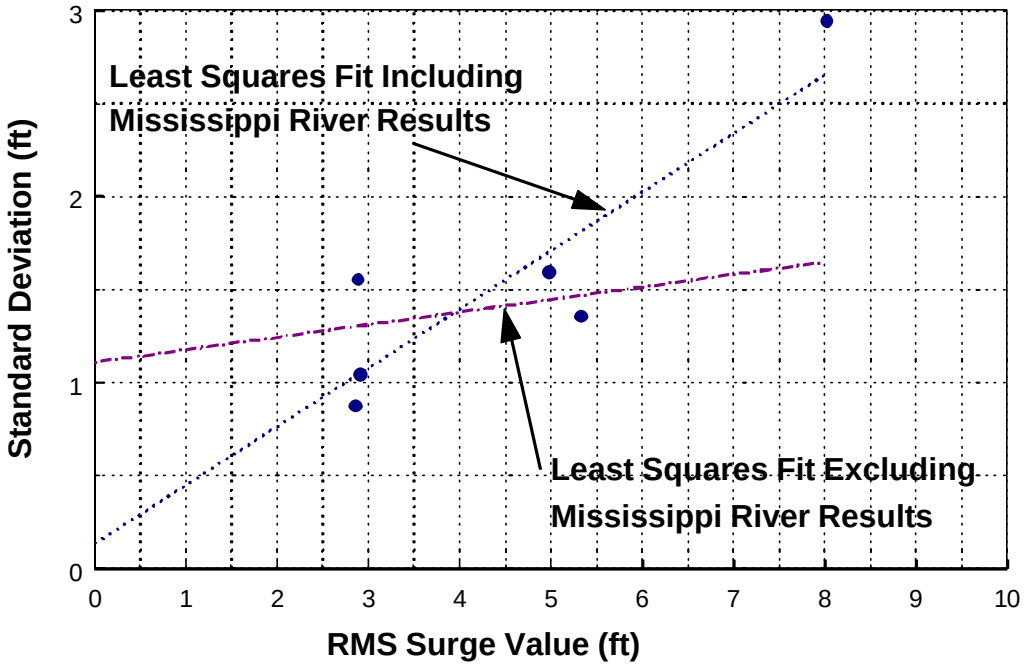

Figure 4.6. Relationship Between rms Surge Value and Standard Deviation. PBL Wind Model. With and Without Mississippi River Results.

Figure 4.7 presents a plot of the Gain versus rms surge values for the PBL winds. As for the Std Dev, there is an increasing value of the difference in means and the rms surge value. However, this trend is influenced (although to a lesser degree than the Std Dev) by the Mississippi River results. Least squares fits are presented for results including those from the Mississippi River and excluding the Mississippi River results.

Figure 4.8 presents a plot of the difference in means versus rms surge and demonstrates a weakly increasing trend with rms surge value. Because this trend is weak, it will be recommended later that the trend not be considered.

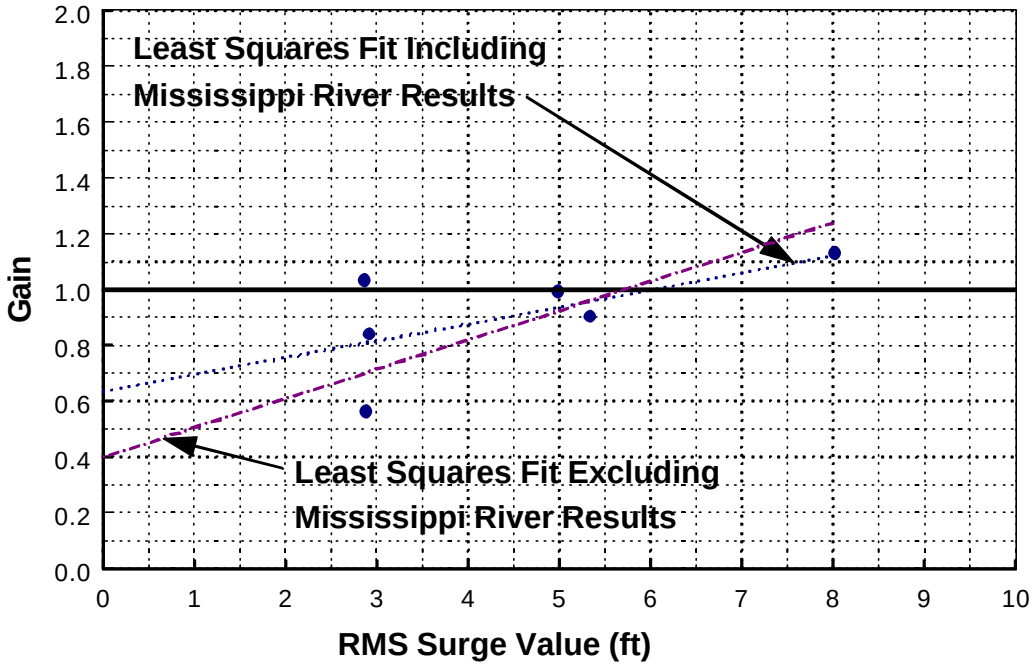

Figure 4.7 . Relationship Between Gain and RMS Surge.
PBL Wind Model. With and Without Mississippi River Results.

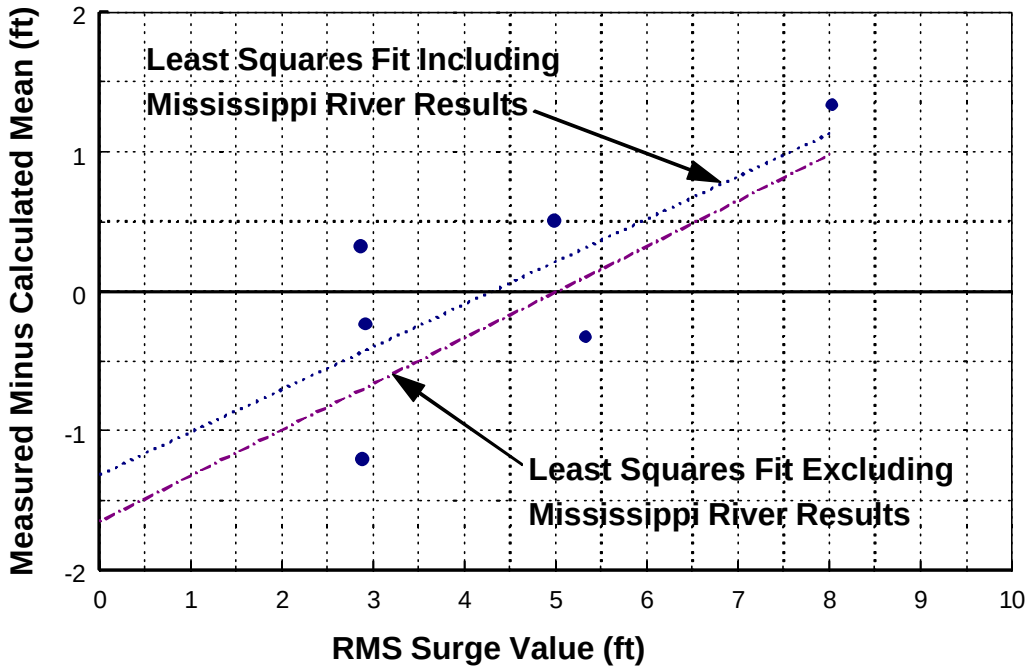

Figure 4.8 Relationship Between Difference in Means and RMS Surge.
PBL Wind Model. With and Without Mississippi River Results.

4.7 Summary of Regional Results

The results of the regional best fit results presented in Figures 4.6, 4.7 and 4.8 can be expressed as $y = ax + b$ where y is the variable of interest (Std Dev, Gain or Difference of Means) and x is the calculated surge based on ADCIRC with PBL forcing. The values of a and b are summarized in Table 4.3 for the three regions evaluated.

Table 4.3

Summary of Fits to Variables Based on Results in Figures 4.6, 4.7 and 4.8

Region	Variable	a	b
Lake Pontchartrain and Southwest Area	Gain	0.10	0.40
	Std Dev	0.07	1.11
	Difference of Means	0.33	-0.65
Mississippi River	Gain	0.17	0*
	Std Dev	0.36	0*
	Difference of Means	0.16	0*

. * Note: Because there were only two points available for the Mississippi River region, these values of b were forced to be zero due to provide greater "stability" to the relationships.

5.0 Added Value of Using HRD Rather Than PBL Winds

5.1 Introduction

The HRD winds are available for Hurricanes Betsy, Isidore and Lili and were based on a post-hurricane analysis of available meteorological data including barometric pressures, wind speeds and directions. Therefore, it is to be expected that the HRD predicted surges would be in better agreement with the measured surges than the surges predicted with PBL forcing. Complicating this issue is that the ADCIRC model refinement/ calibration phase was based on PBL forcing with one of the hurricanes (Betsy) that is used in comparing the HRD and PBL results.

5.2 Comparison Results

With the data available in Table 4.1, it is possible to quantify any added value in using the HRD winds compared to the PBL winds. The StdDev in Table 4.1 will be defined as s_{TOT} , ie the total standard deviation and includes “error “ or “difference” contributions from a number of sources. The total associated variance can be written as

$$s^2_{TOT,PBL} = s^2_{WF,PBL} + s^2_{Other}$$

in which $s^2_{TOT,PBL}$ represents the total variance associated with the PBL model (Column 6 in Table 4.1), $s^2_{WF,PBL}$ represents the variance contribution associated with the PBL wind field and s^2_{Other} is the variance associated with all other contributions. A similar equation can be expressed for the HRD winds

$$s^2_{TOT,HRD} = s^2_{WF,HRD} + s^2_{Other}$$

where s^2_{Other} is common to the two total variance terms. Subtracting the two equations yields

$$s^2_{TOT,PBL} - s^2_{TOT,HRD} = s^2_{WF,PBL} - s^2_{WF,HRD}$$

Which expresses the added value (in terms of variance reduction) associated with the HRD wind fields. We further express this as

$$s^2_{TOT,PBL} - s^2_{TOT,HRD} = s^2_{WF,PBL} - s^2_{WF,HRD} = e^2_{PBLWF}$$

or

$$e_{PBLWF} = \sqrt{s^2_{WF,PBL} - s^2_{WF,HRD}}$$

and e_{PBLWF} is the root mean square error in the PBL wind fields relative to the HRD wind fields. Table 5.1 presents the data entering into this analysis and Table 5.2 presents the analysis results for the Lake Pontchartrain and Mississippi River "all data" with Hurricane Georges removed. Note that this evaluation is based on common measured peak surges.

Table 5.1
Results Used in Wind Field Evaluation

Case/ Hurr	**Model**	**Region**	**N**	**Gain**	**Std Dev (ft)**	**Surge rms (ft)**	**Diff of Means (ft)**
Hurricanes Betsy, Isidore and Lili, All Δh	**PBL**	**Lake Pontchartrain Data**	**16**	**0.95**	**1.40**	**3.98**	**0.24**
	HRD		**16**	**1.56**	**0.96**	**3.98**	**1.51**
Hurricanes Betsy, Isidore and Lili, All Δh	**PBL**	**Mississippi River Data**	**16**	**1.34**	**1.46**	**5.98**	**1.38**
	HRD		**16**	**1.17**	**0.84**	**5.98**	**1.13**
Hurricanes Betsy, Isidore and Lili, All Δh	**PBL**	**Southwest Area**	**20**	**0.89**	**0.99**	**3.98**	**-0.28**
	HRD		**20**	**1.04**	**1.27**	**3.98**	**0.55**

Table 5.2

Additional Error, e, Associated With PBL vs HRD Models

Case/Hurr	Region	Incorporates Stations	$s_{TOT,HRD}$ (ft)	e_{PBLWF} (ft)
Hurricanes Betsy, Isidore and Lili, All Δh	Lake Pontchartrain	130,134,140,144,155, 156,164,165,171	0.96	1.02
Hurricanes Betsy, Isidore and Lili, All Δh	Mississippi River	97,98, 99, 100, 101, 102, 103, 104, 105, 106, 107, 108, 109, 110, 111	0.84	1.19
Hurricanes Betsy, Isidore and Lili, All Δh	Southwest Area	5 - 89	1.27	Not Possible to Calculate $s_{TOT,HRD} > s_{TOT,PBL}$

5.3 Interpretation

Referring to Table 5.2, $s_{TOT,PBL} > s_{TOT,HRD}$ for the Lake Pontchartrain and Mississippi River areas which are the two areas where the greatest effort was made in characterizing the natural system and in the modeling program. In the "Southwest Area", $s_{TOT,HRD} > s_{TOT,PBL}$ and thus it was not possible to calculate a value of e_{PBLWF}. The question is whether the HRD wind fields add a substantial value to the storm surge calculations. Although some degree of caution is warranted due to the limited amount of data, the results suggest the following. In view of the greater care exercised in the two areas where $s_{TOT,PBL} > s_{TOT,HRD}$, and the relatively large value of e_{PBLWF} relative to $s_{TOT,HRD}$, it is believed that the HRD wind and pressure fields do add a substantial quality value to the calculated surges. In the following, we discuss only the results for the Lake Pontchartrain and Mississippi River areas.

If we consider as a limiting case that the HRD winds are exact, the additional error in using the PBL winds can be interpreted as follows. In Lake Pontchartrain, the additional error in using PBL vs HRD winds is slightly greater than the total of all other errors combined. For the Mississippi River data, the additional error in using PBL winds is substantially greater (a factor of 1.4) than all of the other errors combined. If we recognize that the HRD winds also contain some error, the benefits of using HRD winds increase further relative to the errors which are not related to the forcing.

6.0 Summary, Conclusions and Recommendations

6.1 Summary

This report represents the results of a four year effort to quantify the capabilities of the numerical Model ADCIRC to predict accurately storm surges in the vicinity of New Orleans. This effort was sponsored by the New Orleans District of the U. S. Army Corps of Engineers. The ADCIRC modeling was carried out by Professors Joannes Westerink of Notre Dame and Richard Luettich Jr. of the University of North Carolina at Chapel Hill, the two developers of the model. Logistics, information resources and guidance were provided by Staff of the New Orleans District of the U. S. Army Corps of Engineers, notably Mr. Jay Combe, Dr. Harley Winer, Dr. Hasan Pourtaheri (now with the Wilmington District of the Corps), Mr. Vann Stutts, Mr. Burnell Thibodeaux and many others. Technical oversight was provided by the Authors of this report who were appointed by the Corps to serve on this effort as a Technical Review Committee.

The effort consisted of two broad phases: (1) A model calibration/refinement phase during which considerable improvements and refinements were made to ADCIRC and after which the model was “frozen”, ie no further model adjustments were made. This effort was conducted with Hurricanes Betsy and Andrew using the PBL wind model for meteorological forcing, and (2) A production phase which involved computations of storm surges for an additional 9 hurricanes.

Two bases for meteorological forcing were utilized in this study including the Planetary Boundary Layer (PBL) model and the Hurricane Research Division (HRD) forcing. Surges were calculated with PBL forcing for all 11 hurricanes and were based on HRD forcing for three hurricanes. It was found that in two of the three geographic regions examined, there is value added through use of the HRD wind fields; however, it is anticipated that the design calculations will be carried out with PBL wind fields.
The largest measured peak surge elevation found in the analysis was approximately 10.8 ft above the pre hurricane effects water level (this surge occurred in the Mississippi River).

There are a number of components required in the determination of whether existing levee heights are adequate and if they are not, the appropriate heights. These include, but are not limited to the following: wave runup and overtopping, the adequacy of the historical hurricane data base, past and future regional and local subsidence, the latter due to the weight of the levees, steric water level elevation effects, the method of applying the statistics of the historical hurricanes and any safety factors adopted. Two methods have been discussed for accounting for the historical hurricane statistics: The Joint Probability Method (JPM) and the Empirical Simulation Technique (EST) although this issue was beyond the charge to the TRC.

Separate analyses of the statistical differences between measured and predicted storm surges were carried out for the following regions: Lake Pontchartrain, Mississippi River and the Southwest Area. The parameters characterizing the skill of ADCIRC predictions

were also examined for different ranges of surge levels. It was found that the standard deviation, gain and difference between the averaged measured and calculated mean water levels all increased with increasing surge levels. However, some of the correlations for the Mississippi River appeared to differ significantly from those for the other two regions and the relationships for the Mississippi River were thus developed separately.

6.2 Conclusions

Based on analysis of the measured and ADCIRC calculated storm surges using the PBL for meteorological forcing, it was found that the relationships between the variables quantifying predictive skill were quite similar for Lake Pontchartrain and the Southwest Area; however, they differed considerably for the Mississippi River region. Although the cause of this difference is not known, it is recommended that regional considerations be included in the ADCIRC predictions with PBL forcing in the design phase. Additionally, adjustments based on the value of the predicted surge appear to be justified.

Comparisons were conducted between the surge predictions based on PBL and HRD forcing for the three hurricanes for which forcing by both bases was available. It was found that in two of the three regions, the HRD winds provided considerably greater predictive skill whereas in the third region (Southwest Area), the PBL predictions were slightly better. It is hypothesized that this may be a result of less effort in characterizing the physical features of this latter region.

Two areas in which the storm surge predictions could be improved are: better meteorological forcing and explicit incorporation of wave setup in ADCIRC and any associated appropriate adjustments to the other model elements which, in the present model development, have incorporated any wave setup effects implicitly.

The simulation of surges in the Mississippi River is apparently sensitive to possible error in the hurricane track in the vicinity of the river mouth, judging from the simulations carried out for Hurricane Georges.

6.3 Recommendations

In applying the model ADCIRC for design purposes with PBL meteorological forcing, it is recommended that the calculated surges be augmented with regional gain factors and standard deviations, the latter commensurate with the accepted risk. These regional factors have been summarized in Table 4.3 which is repeated as Table 6.1 here. It is not recommended that the “difference of means” by taken into account, as the gain factor accomplishes the same effect in the least squares sense.

It is also recommended that in the design calculations the total standard deviation be augmented to include the steric effects including monthly means and variability considering the steric variability to be uncorrelated with other causes resulting in the variability in the peak surges.

It is recommended that wave setup be incorporated into ADCIRC and an effort be made to modify the PBL model to provide improved meteorological forcing to address the systematic differences found in the comparison of the HRD and PBL wind fields. After these efforts, a pilot study should be conducted in which storm surges are calculated for several of the hurricanes used in the present effort. If the resulting predicted storm surges are sufficiently improved, it would be worthwhile to repeat the effort here to quantify revised design variables, especially standard deviations.

Finally, it is recommended that a study be carried out to evaluate the sensitivity of surge simulation in the Mississippi River to perturbations of tracks of those storms for which measurements show significant surges therein.

Table 6.1

Summary of Fits to Variables Based on Results in Figures 4.6, 4.7 and 4.8
(Note: This Table is a Duplicate of Table 4.3)

Region	Variable	a	b
Lake Pontchartrain and Southwest Area	Gain*	0.10	0.40
	Std Dev	0.07	1.11
Mississippi River	Gain*	0.17	0**
	Std Dev	0.36	0**

Notes:

*(1) Based on the data available here, a maximum gain of 1.2 is recommended.

**(2) These values of b were forced to be zero due to only two available points available for the Mississippi River region to provide greater “stability” to the relationships.

7.0 ACKNOWLEDGEMENTS

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8.0 Bibliography

8.1 Introduction

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APPENDIX A

EXAMPLE APPLICATIONS OF RECOMMENDED METHODOLOGY FOR DESIGN

APPENDIX A

A.0 Example Applications of Recommended Methodology for Design

A.1 General

The purpose of this section is to provide several examples of the application of the recommended methodology for adjusting storm surges calculated by ADCIRC using the present PBL model and determining the associated standard deviations. These examples do not include consideration of incorporating the steric contribution which will be required in the design process.

A.2 Methodology

The methodology for calculation of gains is as follows. Given a calculated peak storm surge anomaly, Δh_c the associated gain, m, is determined as m = a Δh_c + b where the appropriate a and b values are presented in Table 6.1. Note that a maximum gain of 1.2 is recommended for calculating the peak surge anomalies. The standard deviations are determined in a similar manner.

A.3 Examples

Table A.1 presents four cases of calculated peak storm surge anomalies, two cases each for Lake Pontchartrain (or the Southwest Area since the same methodology applies for these two regions) and the Mississippi River.

A.3.1 Case 1. Lake Pontchartrain or Southwest Area

For this case, the appropriate values of a and b for the gain (Table 6.1) are a = 0.10, b = 0.40. Thus the gain is 1.0 and the recommended surge is $\Delta h_{rec} = m\Delta h_c$ = 1.0 (6) = 6 feet, the same as calculated by ADCIRC. The appropriate a and b factors for the standard deviation are 0.07 and 1.11 ft, respectively. Thus, the recommended standard deviation is s_{rec} = 0.07(6)+1.11 = 1.53 ft.

A.3.2 Case 2. Lake Pontchartrain or Southwest Area

Repeating the procedure for the calculated storm surge anomaly of 10 feet, it is found that the calculated gain is 1.4; however, the maximum recommended gain for application is 1.2. Thus the recommended peak surge anomaly is 12 ft and the recommended standard deviation is 1.81 ft [= (0.07)(10 ft) +1.11 ft].

A.3.3 Case 3. Mississippi River Area

The procedure is the same for the Mississippi River; however, the coefficients (Table 6.1) differ. The gain, m = 0.17 (8) = 1.36 such that the recommended maximum value of

m (1.2) governs resulting in a recommended peak surge anomaly of 9.6 feet and a standard deviation, s_{rec} = 0.36 (8 ft) = 2.88 ft.

A.3.4 Case 4. Mississippi River Area

The recommended peak surge anomaly is based on a m value of 1.2 such that the recommended peak surge anomaly is 14.4 feet and the associated recommended standard deviation is 4.32 feet.

Table A.1

Examples of Methodology Recommended
for Design Application

Case	Region	Calculated Peak Storm Surge, Δh_c (ft)	Gain, m	Adjusted (Recommended) Peak Storm Surge (ft)	Adjusted Standard Deviation (ft)
1	Lake Pontchartrain	6	1.0	6	1.53
2	Lake Pontchartrain	10	1.2*	12	1.81
3	Mississippi River	8	1.2*	9.6	2.88
4	Mississippi River	12	1.2*	14.4	4.32

*Note: These values are limited by the maximum recommended value of m = 1.2

APPENDIX B

LISTING OF DATA ANALYZED FROM PRODUCTION RUNS

B.0 Listing of Data Analyzed From Production Runs

B.1 Description of Data

The following four pages present the data analyzed and reported on in this report. The column designations are as described in Table B.1. The description is aided by reference to Figure 4.1 repeated below as Figure B.1.

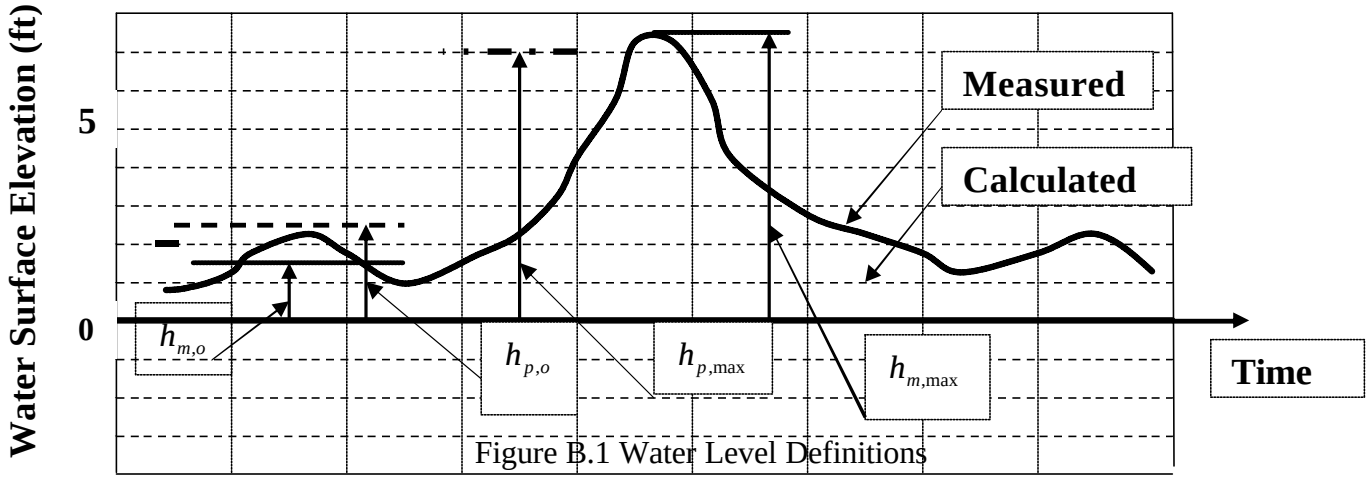

Figure B.1 Water Level Definitions

Table B.1

Description of Information Included in Table B.3

Column	Description
1	Storm Number (see Table 2)
2	Water Level Station
3	Measured Pre-Storm Water Level, $h_{m,o}$
4	Measured Peak Water Level, $h_{m,\max}$
5	Predicted Pre-Storm Water Level for PBL Forcing $h_{p,o}$ (Available for All Eleven Hurricanes)
6	Predicted Peak Water Level for PBL Forcing, $h_{p,\max}$ (Available for All Eleven Hurricanes)
7	Predicted Peak Water Level for HRD Forcing $h_{p,o}$ (Only Available for Three Hurricanes)
8	Predicted Peak Water Level for HRD Forcing, $h_{p,\max}$ (Only Available for Three Hurricanes)

Table B.2

Storm Numbering System in Data Base

Storm Number	Storm
1	Betsy (1965)
2	Andrew (1992)
3	Isidore (2002)
4	Lili (2002)
5	Camille (1969)
6	Carmen (1974)
7	Hilda (1964)
8	Edith (1971)
9	Georges (1998)
10	Audrey (1957)
11	1947 Hurricane (Unnamed)

Table B.3
Listing of Data Analyzed

Storm Number	Station Number	$h_{m,o}$ (ft)	$h_{m,\max}$ (ft)	PBL $h_{p,o}$ (ft)	PBL $h_{p,\max}$ (ft)	HRD $h_{p,o}$ (ft)	HRD $h_{p,\max}$ (ft)
1	21	1.0	3.5	1.0	3.0	1.0	3.5
1	23	0.5	2.9	1.0	3.7	1.0	3.7
1	37	0.6	4.4	1.0	4.1	1.0	4.4
1	82	1.3	8.2	1.3	7.2	1.3	9.6
1	87	1.3	7.0	1.2	6.0	1.2	3.4
1	94	3.6	4.3	1.3	7.4	1.4	7.7
1	99	2.7	5.4	1.3	2.6	1.3	3.3
1	104	2.3	15.0	2.3	10.5	2.0	13.1
1	107	3.0	12.3	2.7	9.1	2.6	11.3
1	108	2.9	12.5	3.1	9.4	2.7	11.4
1	109	3.0	12.3	3.0	9.0	3.0	11.3
1	140	2.4	5.3	1.7	5.1	2.0	4.4
1	140	2.4	5.0	1.7	5.4	2.0	3.8
1	144	2.3	6.2	1.7	4.0	1.7	3.4
1	144	2.3	5.8	1.7	5.5	1.7	3.7
1	146	2.3	9.9	1.5	11.5	1.5	10.8
1	164	2.2	5.3	1.6	6.1	1.6	4.0
1	165	2.0	6.4	1.6	7.3	1.6	5.0
1	171	2.0	7.0	1.6	8.7	1.6	4.5
1	176	1.0	8.5	1.0	9.7	1.0	5.0

Table B.3
Listing of Data Analyzed (Continued)

Storm Number	Station Number	$h_{m,o}$ (ft)	$h_{m,\max}$ (ft)	PBL $h_{p,o}$ (ft)	PBL $h_{p,\max}$ (ft)	HRD $h_{p,o}$ (ft)	HRD $h_{p,\max}$ (ft)
2	4	0.0	1.5	0.4	1.7	0.0	0.0
2	12	0.2	3.6	0.2	2.2	0.0	0.0
2	14	3.1	3.9	0.2	1.2	0.0	0.0
2	19	0.5	1.5	1.3	1.8	0.0	0.0
2	27	2.4	7.3	1.4	5.3	0.0	0.0
2	28	2.0	5.9	1.0	5.3	0.0	0.0
2	29	2.0	3.6	2.0	5.4	0.0	0.0
2	31	2.7	6.2	2.0	5.6	0.0	0.0
2	34	1.2	7.6	1.2	5.0	0.0	0.0
2	35	1.9	6.4	1.4	5.3	0.0	0.0
2	36	2.0	3.7	1.5	6.0	0.0	0.0
2	40	5.2	7.3	3.1	5.5	0.0	0.0
2	42	7.7	10.4	4.0	6.0	0.0	0.0
2	43	7.7	10.4	4.7	6.4	0.0	0.0
2	44	7.2	9.7	4.6	6.4	0.0	0.0
2	45	2.2	3.4	1.6	5.6	0.0	0.0
2	60	0.6	5.0	1.3	6.4	0.0	0.0
2	61	0.8	5.4	1.2	5.2	0.0	0.0
2	64	9.0	9.3	1.2	6.2	0.0	0.0
2	69	2.0	5.6	0.7	4.9	0.0	0.0
2	74	1.6	5.6	1.2	6.3	0.0	0.0
2	82	1.0	3.7	1.0	3.3	0.0	0.0
2	83	1.0	4.5	1.2	3.2	0.0	0.0
2	84	0.9	5.5	0.9	4.6	0.0	0.0
2	85	0.5	5.1	0.5	4.0	0.0	0.0
2	87	1.6	4.1	1.1	4.0	0.0	0.0
2	89	1.4	3.2	-0.5	4.6	0.0	0.0
2	102	1.8	4.6	1.5	4.3	0.0	0.0
2	103	0.8	5.0	0.9	5.1	0.0	0.0
2	104	3.1	5.2	3.3	5.6	0.0	0.0
2	105	2.8	5.5	3.2	6.1	0.0	0.0
2	108	4.7	6.7	4.2	6.2	0.0	0.0
2	110	5.5	7.2	6.4	8.5	0.0	0.0
2	111	14.0	16.8	13.7	15.2	0.0	0.0
2	116	0.2	5.2	0.7	4.5	0.0	0.0
2	119	1.0	5.0	2.0	5.3	0.0	0.0
2	120	0.2	5.4	0.4	4.0	0.0	0.0
2	146	3.0	6.0	3.0	6.0	0.0	0.0
2	156	1.2	3.6	1.6	4.5	0.0	0.0
2	171	1.7	3.1	1.7	4.1	0.0	0.0

Table B.3
Listing of Data Analyzed (Continued)

Storm Number	Station Number	$h_{m,o}$ (ft)	$h_{m,\max}$ (ft)	PBL $h_{p,o}$ (ft)	PBL $h_{p,\max}$ (ft)	HRD $h_{p,o}$ (ft)	HRD $h_{p,\max}$ (ft)
3	72	2.4	5.7	0.5	5.7	0.5	3.6
3	83	1.4	4.7	0.8	4.5	0.8	2.9
3	85	1.6	5.0	1.2	4.1	1.2	2.7
3	87	2.0	4.1	1.4	4.0	1.4	3.0
3	94	1.7	4.6	1.4	4.3	1.4	3.3
3	97	2.6	5.6	0.7	3.4	0.7	2.8
3	101	1.8	5.6	1.0	4.6	1.0	4.0
3	104	2.8	6.5	2.2	6.2	2.2	5.0
3	107	2.6	6.5	2.7	7.0	2.7	5.8
3	109	2.8	6.5	3.0	7.6	3.0	6.1
3	110	3.0	7.0	3.6	8.4	3.6	6.3
3	119	2.3	7.5	1.5	6.7	1.7	5.7
3	122	1.7	6.9	1.5	7.1	1.5	5.4
3	130	1.6	6.4	1.6	7.0	1.6	4.5
3	140	1.7	6.2	1.7	4.5	1.7	4.1
3	143	1.3	8.0	1.6	7.7	1.6	6.3
3	144	1.4	6.8	1.6	4.7	1.6	4.3
3	151	2.0	7.7	1.5	5.1	1.5	5.7
3	165	1.8	6.0	1.3	4.4	1.3	4.4
3	171	1.7	5.6	1.7	6.2	1.7	5.4
4	27	2.5	8.0	1.5	8.0	1.5	6.4
4	28	2.4	7.6	1.5	8.0	1.5	6.4
4	30	3.9	7.6	1.6	6.9	1.6	5.8
4	37	2.4	6.9	1.5	7.6	1.5	5.8
4	42	5.5	9.4	2.7	5.3	2.7	4.7
4	45	2.7	6.7	1.6	5.7	1.6	4.5
4	60	1.7	5.0	1.5	6.7	1.5	6.2
4	72	3.0	7.0	1.0	6.2	1.0	5.3
4	82	1.6	5.0	1.3	3.1	1.3	2.5
4	85	2.3	4.7	1.5	3.7	1.5	3.3
4	87	2.1	4.4	1.4	4.1	1.4	3.4
4	101	2.3	5.3	1.4	3.0	1.4	3.0
4	104	2.6	6.0	2.1	3.7	2.1	4.0
4	107	2.4	6.0	2.6	4.3	2.6	4.5
4	109	3.0	6.4	3.0	4.5	3.0	4.7
4	110	3.6	7.0	3.5	5.3	3.5	5.2
4	119	1.6	7.0	1.4	3.8	1.4	4.0
4	120	2.0	7.8	1.4	3.7	1.4	3.8
4	122	2.1	6.4	1.8	5.1	1.8	4.6
4	140	2.1	4.9	1.9	4.5	1.9	3.4
4	143	2.0	6.7	1.7	4.3	1.7	4.0
4	144	1.6	5.2	1.9	3.5	1.9	3.3
4	151	2.4	6.4	1.5	3.5	1.5	3.6
4	162	2.7	4.7	2.0	4.7	2.0	4.4
4	165	2.0	5.4	2.0	5.0	2.0	4.1
4	171	1.8	6.1	1.8	3.7	1.8	3.5

Table B.3
Listing of Data Analyzed (Continued)

Storm Number	Station Number	$h_{m,o}$ (ft)	$h_{m,\max}$ (ft)	PBL $h_{p,o}$ (ft)	PBL $h_{p,\max}$ (ft)	HRD $h_{p,o}$ (ft)	HRD $h_{p,\max}$ (ft)
5	97	3.2	5.0	1.0	3.2	0.0	0.0
5	102	2.5	7.0	2.5	13.5	0.0	0.0
5	104	3.1	11.9	3.1	10.4	0.0	0.0
5	107	3.7	10.5	3.7	9.5	0.0	0.0
5	108	3.7	11.3	3.7	9.2	0.0	0.0
5	109	4.1	10.6	4.1	9.3	0.0	0.0
5	140	1.9	5.2	2.7	7.5	0.0	0.0
5	144	2.3	6.3	2.6	7.7	0.0	0.0
5	146	2.5	9.7	3.6	5.7	0.0	0.0
5	151	1.4	10.0	1.4	10.2	0.0	0.0
5	156	2.0	7.0	2.4	8.7	0.0	0.0
5	159	2.0	8.7	3.3	5.0	0.0	0.0
5	164	2.0	4.0	2.7	3.4	0.0	0.0
5	165	1.9	4.4	1.6	2.8	0.0	0.0
5	171	1.7	9.0	1.5	8.1	0.0	0.0
6	28	1.9	5.4	1.9	9.4	0.0	0.0
6	29	2.0	2.7	2.0	9.3	0.0	0.0
6	30	2.5	5.6	2.5	8.4	0.0	0.0
6	31	1.5	4.5	2.0	10.0	0.0	0.0
6	33	2.0	4.0	2.0	8.1	0.0	0.0
6	35	2.0	4.8	2.0	9.4	0.0	0.0
6	38	2.0	2.0	2.0	9.4	0.0	0.0
6	54	2.0	8.5	2.0	8.4	0.0	0.0
6	65	3.5	4.5	2.0	10.7	0.0	0.0
6	69	1.5	5.7	1.5	6.0	0.0	0.0
7	5	0.0	2.5	0.3	2.6	0.0	0.0
7	23	2.3	-1.5	2.9	-2.3	0.0	0.0
7	23	-1.5	3.0	-2.3	3.3	0.0	0.0
7	23	-1.4	3.0	-2.4	2.7	0.0	0.0
7	28	2.3	-1.6	3.0	-2.0	0.0	0.0
7	28	-1.6	4.8	-2.0	6.6	0.0	0.0
7	28	2.2	5.0	2.6	6.7	0.0	0.0
7	30	2.7	0.2	2.7	0.3	0.0	0.0
7	30	0.2	5.5	0.3	6.0	0.0	0.0
7	30	0.0	7.5	0.0	8.0	0.0	0.0
7	37	2.4	-1.0	2.7	-0.2	0.0	0.0
7	37	-1.0	6.0	-0.2	7.1	0.0	0.0
7	37	2.2	5.8	2.6	8.2	0.0	0.0
7	69	1.5	5.5	1.0	6.3	0.0	0.0
7	94	2.4	5.1	2.9	7.4	0.0	0.0
7	94	2.9	5.0	2.9	7.3	0.0	0.0
7	97	2.0	3.4	0.7	2.1	0.0	0.0
7	97	2.0	3.5	1.4	2.1	0.0	0.0
7	104	2.1	4.0	1.9	3.9	0.0	0.0
7	104	1.9	4.0	1.9	3.9	0.0	0.0

Table B.3
Listing of Data Analyzed (Continued)

Storm Number	Station Number	$h_{m,o}$ (ft)	$h_{m,\max}$ (ft)	PBL $h_{p,o}$ (ft)	PBL $h_{p,\max}$ (ft)	HRD $h_{p,o}$ (ft)	HRD $h_{p,\max}$ (ft)
7	107	3.6	6.5	2.2	5.2	0.0	0.0
7	107	3.9	6.4	2.3	5.1	0.0	0.0
7	108	2.8	5.7	2.3	5.2	0.0	0.0
7	108	2.6	5.7	2.1	5.1	0.0	0.0
7	115	2.0	4.5	1.2	2.6	0.0	0.0
7	140	2.3	5.3	1.7	4.3	0.0	0.0
7	142	1.7	5.7	1.7	4.0	0.0	0.0
7	146	1.6	6.0	1.6	4.0	0.0	0.0
7	151	1.5	4.4	1.5	3.0	0.0	0.0
7	161	1.1	5.1	1.7	4.7	0.0	0.0
7	165	1.5	6.3	1.6	5.7	0.0	0.0
8	2	2.1	5.7	1.2	7.9	0.0	0.0
8	6	2.0	5.9	5.0	8.9	0.0	0.0
8	7	1.5	2.5	1.5	5.3	0.0	0.0
8	13	1.4	4.6	4.6	5.7	0.0	0.0
8	14	1.4	4.3	2.0	7.3	0.0	0.0
8	22	0.7	4.9	1.6	10.0	0.0	0.0
8	28	0.7	5.6	1.6	8.6	0.0	0.0
8	30	1.5	5.4	2.0	7.5	0.0	0.0
8	31	0.7	5.3	1.7	8.5	0.0	0.0
8	37	0.6	5.7	1.8	8.3	0.0	0.0
8	54	1.5	4.2	1.4	5.0	0.0	0.0
8	65	2.5	3.5	2.5	5.7	0.0	0.0
8	69	1.7	3.0	0.9	5.4	0.0	0.0
8	140	2.0	2.6	-0.5	2.4	0.0	0.0
8	156	1.8	3.4	1.8	4.0	0.0	0.0
8	171	2.0	3.7	2.0	4.4	0.0	0.0
8	176	0.1	3.6	0.5	4.7	0.0	0.0
9	21	1.5	3.5	0.7	2.3	0.0	0.0
9	22	1.7	2.4	-0.4	2.2	0.0	0.0
9	77	3.1	3.6	1.1	2.2	0.0	0.0
9	82	1.0	1.8	1.2	1.6	0.0	0.0
9	94	0.5	1.5	0.5	2.0	0.0	0.0
9	95	3.1	5.0	1.2	2.7	0.0	0.0
9	97	3.0	6.7	1.3	1.7	0.0	0.0
9	103	2.2	10.1	1.2	3.3	0.0	0.0
9	104	2.5	10.0	2.2	3.3	0.0	0.0
9	105	2.1	9.8	1.4	3.3	0.0	0.0
9	106	2.7	10.3	1.3	3.3	0.0	0.0
9	107	2.8	10.0	2.8	3.2	0.0	0.0
9	108	2.3	10.0	2.0	3.6	0.0	0.0
9	109	3.0	10.2	3.0	3.5	0.0	0.0
9	111	6.7	12.4	6.0	6.2	0.0	0.0
9	122	2.0	4.6	1.2	2.9	0.0	0.0
9	134	2.0	3.5	2.0	4.1	0.0	0.0
9	140	2.4	5.5	1.5	3.6	0.0	0.0
9	144	2.4	6.8	1.5	4.0	0.0	0.0
9	171	2.4	6.0	1.5	4.5	0.0	0.0

Table B.3
Listing of Data Analyzed (Continued)

Storm Number	Station Number	$h_{m,o}$ (ft)	$h_{m,\max}$ (ft)	PBL $h_{p,o}$ (ft)	PBL $h_{p,\max}$ (ft)	HRD $h_{p,o}$ (ft)	HRD $h_{p,\max}$ (ft)
10	22	2.5	6.0	2.5	8.4	0.0	0.0
10	32	1.5	7.9	1.0	5.1	0.0	0.0
10	66	4.2	6.9	0.7	3.4	0.0	0.0
10	82	1.0	3.4	0.8	2.8	0.0	0.0
11	23	-1.4	2.7	-1.6	2.7	0.0	0.0
11	28	-1.6	3.2	-1.6	3.0	0.0	0.0
11	98	1.5	3.1	1.0	2.4	0.0	0.0
11	101	1.4	6.0	1.0	4.4	0.0	0.0
11	140	2.0	5.5	2.0	7.5	0.0	0.0

ANNEX 4 – LEVEE CROSS SECTION EVALUATION

APPENDIX L – ENGINEERING INVESTIGATIONS
ANNEX 4 - LEVEE CROSS SECTION EVALUATION

Wave Runup and Overtopping on Revetments and Levees

Introduction

The degree of wave action on and going over a coastal structure must be predicted in order to design the structure crest height. The structure crest height is usually set to provide a specific probability of exceedance of water going over the crest. This requires estimation of the expected incident wave and water level climate at the structure toe as well as the local subsidence, structure/foundation settlement rate, and the relative local sea level rise.

Wind-generated waves break and run up sloping coastal structures. This process is termed *wave runup*. If the structure crest is low enough, the runup can go over the structure crest. This process is termed wave *overtopping*. The degree of runup or overtopping is dependent on the incident wave height, wave steepness, wave angle of obliquity, slope of the structure, structure roughness and permeability. Incident waves are defined on the foreshore at the location of the structure toe, where the structure meets the local bathymetry. The foreshore is defined as the region of the bathymetry just seaward of the structure ending at the structure toe. Wave runup is computed as the upward vertical distance from the still water level. The probability of exceedance of wave runup is computed based on the percent of waves producing a runup higher than the given statistic. In other words, the exceendance is calculated based on the number of waves rather than the number of runups. Wave overtopping is defined by the volume of water going over the seaward edge of the structure crest per unit length of structure per unit time. These basic physical processes are described more fully in the Coastal Engineering Manual (CEM 2004).

Wave runup and overtopping are usually computed using empirical relations derived from mostly small-scale physical model experiments. Historically, these experiments were conducted using monochromatic waves. During the 1970's and 1980's, it became common place to utilize irregular waves in laboratory experiments. This resulted in relations utilizing statistics of incident waves for predicting statistics of response. For runup, relations followed the simple form proposed by Battjes (1974) of

$$R_{u2\%} = H_s(A\xi + C)\gamma_f \gamma_b \gamma_h \gamma_\beta \tag{1}$$

$$\xi = \frac{\tan\alpha}{\sqrt{s}} \qquad s = \frac{H_s}{L} \qquad L = \frac{gT^2}{2\pi} \tag{2}$$

where $R_{2\%}$ = wave runup height on the structure with 2 percent probability of exceedance, A, C = empirical coefficients, H_s = significant wave height, ξ = Iribarren parameter or the surf similarity parameter, α = structure slope from horizontal, s = wave steepenss, L = wave length, g = acceleration of gravity, and T = wave period. Reduction factors are included for γ_r = influence of surface roughness, γ_b = influence of berm, γ_h = influence of shallow water (wave breaking), and γ_β = influence of angle of wave incidence. The CEM (Section VI-5-2) provides a summary of the studies conducted to generate coefficients for Equation 1 for varied structure slopes, wave conditions, and roughness characteristics. Some typical values for coefficients are γ_f = 1 for

smooth slope, γ_f = 0.5 - 0.6 for rock armor, γ_f = 0.9 – 1.0 for grass-covered levee, γ_b = 1 for non-bermed slope, γ_h = 1 for Rayleigh distributed waves, and γ_β = 1 for normally-incident waves.

For wave runup on structures, the CEM provides the following equations:

Smooth impermeable slopes

Ahrens (1981) for $1 \leq \cot \alpha \leq 4$:

$$\frac{R_{u2\%}}{H_{mo}} = 1.6\xi_{op} \qquad for \quad \xi_{op} \leq 2.5 \tag{3}$$

[1]

$$\frac{R_{u2\%}}{H_{mo}} = -0.2\xi_{op} + 4.5 \qquad for \quad 2.5 < \xi_{op} < 9 \tag{4}$$

De Wall & Van der Meer (1992) for $3 \leq \cot \alpha \leq 8$:

$$\frac{R_{u2\%}}{H_{mo}} = 1.5\xi_{op} \qquad for \quad 0.5 < \xi_{op} \leq 2.0 \tag{5}$$

$$\frac{R_{u2\%}}{H_{mo}} = 3.0 \qquad for \quad 2.0 < \xi_{op} \leq 3-4 \tag{6}$$

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{s_{op}}} \qquad s = \frac{H_{mo}}{L_{op}} \qquad L_{op} = \frac{gT_p^2}{2\pi} \tag{7}$$

Rough (rock) impermeable slopes

Van der Meer (1988) for $1.5 \leq \cot \alpha \leq 6$:

$$\frac{R_{u2\%}}{H_{mo}} = 0.96\xi_{om} \qquad for \quad 1.0 < \xi_{om} \leq 1.5 \tag{8}$$

$$\frac{R_{u2\%}}{H_{mo}} = 1.17\xi_{om}^{0.46} \qquad for \quad 1.5 < \xi_{om} \tag{9}$$

Rough (rock) permeable slopes

Van der Meer (1988) for $1.5 \leq \cot \alpha \leq 6$:

[1] All equations in this report are central fits to laboratory data.

$$\frac{R_{u2\%}}{H_{mo}} = 0.96\xi_{om} \qquad for \quad 1.0 < \xi_{om} \leq 1.5 \tag{10}$$

$$\frac{R_{u2\%}}{H_{mo}} = 1.17\xi_{om}^{0.46} \qquad for \quad 1.5 < \xi_{om} \leq 2.38 \tag{11}$$

$$\frac{R_{u2\%}}{H_{mo}} = 1.97 \qquad for \quad 2.38 < \xi_{om} \leq 7.5 \tag{12}$$

Here, the significant wave height is defined as $H_{mo} = 4(m_0)^{0.5}$ where m_0 = zeroth moment of the wave energy density spectrum. Herein, wave runup is related to H_{mo} and various wave period statistics. It is convenient to use the peak wave period T_p, corresponding to the peak frequency of the wave energy density spectrum, in runup and overtopping relations because this is the common output from numerical wave transformation models like STWAVE. The mean or average period T_m is also used. The spectral period, $T_{m\text{-}1.0} = m_{-1}/m_0$, where m_{-1} = first negative moment of the wave energy density spectrum, is proposed in TAW (2002) to describe wave runup. T_m and $T_{m\text{-}1.0}$ are not easily computed because details of the spectra are not usually available. However, if these parameters are used and nearshore wave data are not available, TAW recommends the simple relations $T_m = T_p/1.2$ and $T_{m\text{-}1.0} = T_p/1.1$. An alternative is to use hydrodynamic numerical models, such as those based on Boussinesq equations.

Figures 1 - 3 show equations 3 – 12 plotted versus structure slope for uniform smooth impermeable, rough impermeable, rough permeable slopes, respectively, for irregular unidirectional head-on waves. The wave and water level conditions plotted in Figures 1 – 3 are those roughly corresponding to Hurricane Katrina at the peak near the most exposed sections of levees. In Figures 1 – 3, the plotted results are limited to the application ranges of Equations3 – 12. Figures 1 - 3 show that runup decreases at a decreasing rate as the slope goes from steep to shallow sloping. Figures 1 -3 also show that Equations 3 and 4 may be overly conservative when compared to Equations 5 and 6 and do not apply to a very wide range of structure slopes. Figure 4 is similar to Figure 1 except that the slope is held constant at cot α = 8 and the wave period is varied over the range that was experienced by levee sections in Katrina. For Figure 4, the Iribarren parameter values are relatively small and restricted to a narrow range because of the shallow slope. The runup generally increases linearly over this range. Figures for shallow rough slopes are not shown because Equations 3 – 12 do not apply for a range of shallow slopes. Finally, Figures 5 – 7 are similar to Figure 1 – 3 and Figure 4 except the slope is held constant at cot α = 3. In this case, there is significant variability in the runup for the various equations with Equations 3 - 4 significant more conservative.

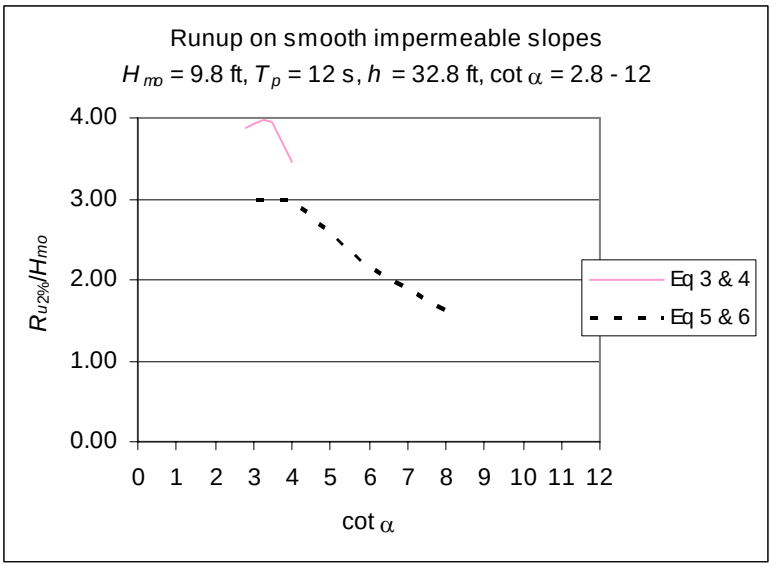

Figure 1. Runup as a function of structure slope for smooth impermeable uniform slopes exposed to unidirectional head-on incident waves. Wave conditions are constant so plot shows effect of changing slope.

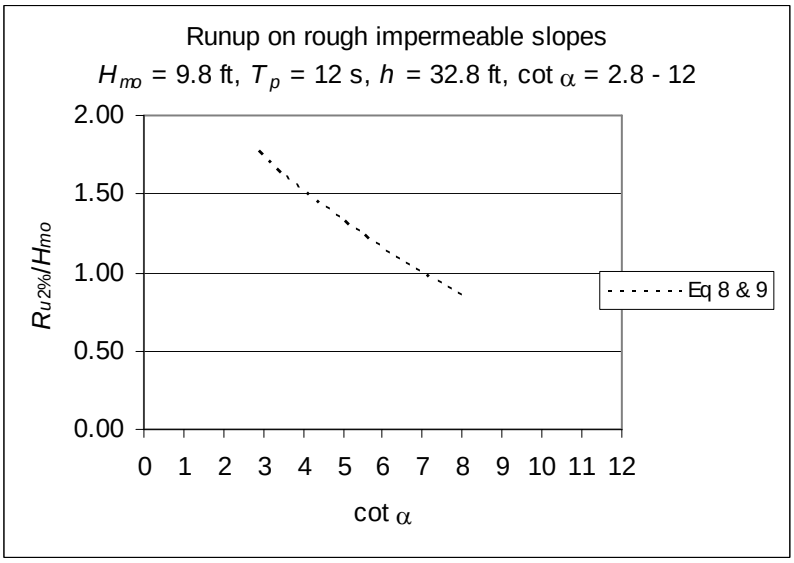

Figure 2. Runup as a function of structure slope for rough impermeable uniform slopes exposed to unidirectional head-on incident waves. Wave conditions are constant so plot shows effect of changing slope.

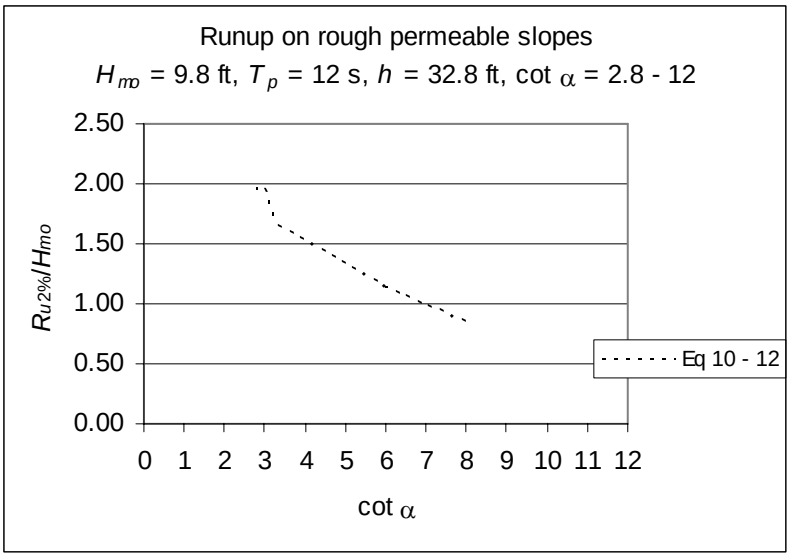

Figure 3. Runup as a function of structure slope for rough permeable uniform slopes exposed to unidirectional head-on incident waves. Wave conditions are constant so plot shows effect of changing slope.

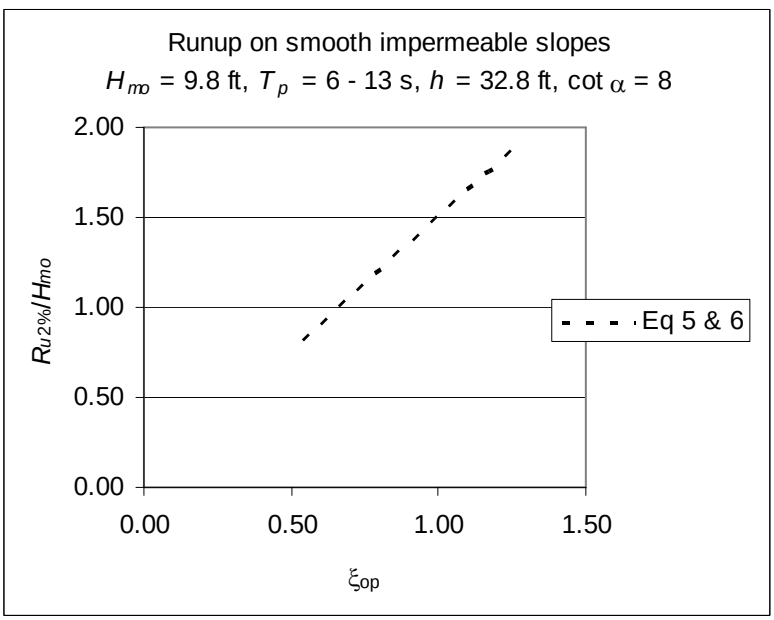

Figure 4. Runup as a function of Iribarren parameter for smooth impermeable uniform slopes exposed to unidirectional head-on incident waves. Shallow slope with variable wave period.

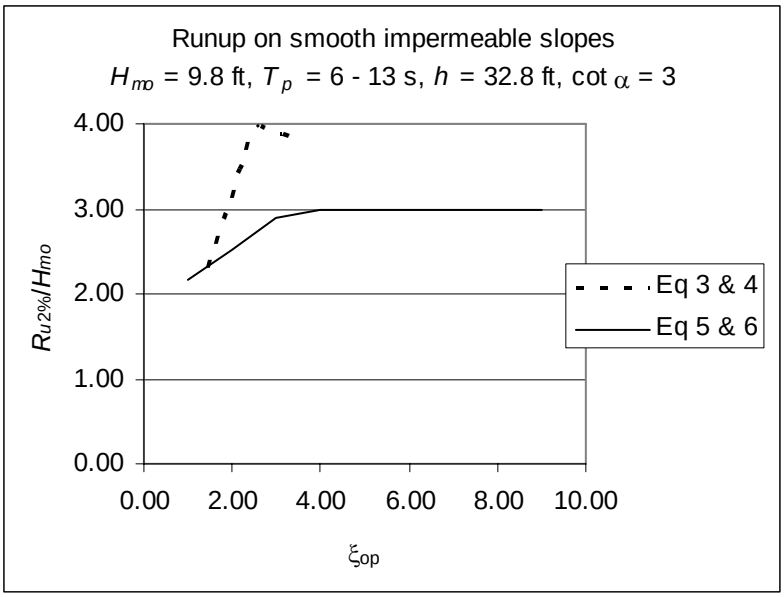

Figure 5. Runup as a function of Iribarren parameter for smooth impermeable uniform slopes exposed to unidirectional head-on incident waves. Steep slope with variable wave period.

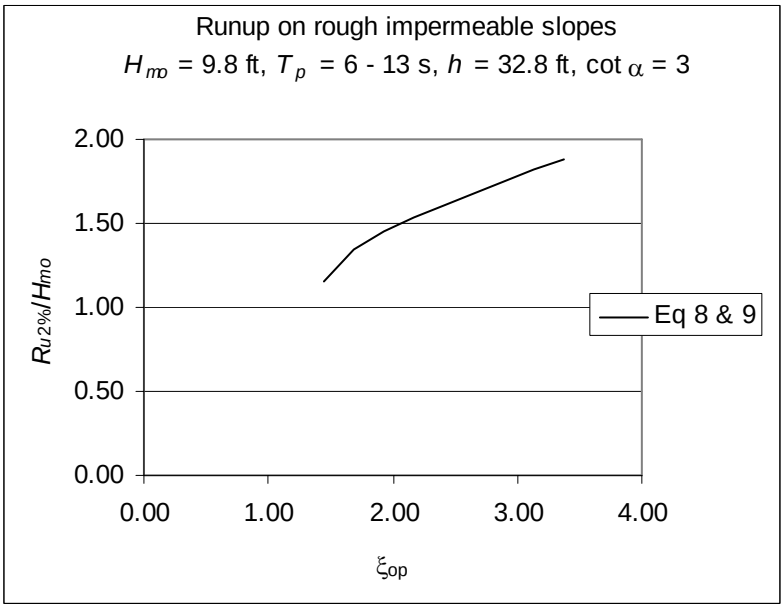

Figure 6. Runup as a function of Iribarren parameter for rough impermeable uniform slopes exposed to unidirectional head-on incident waves. Steep slope with variable wave period.

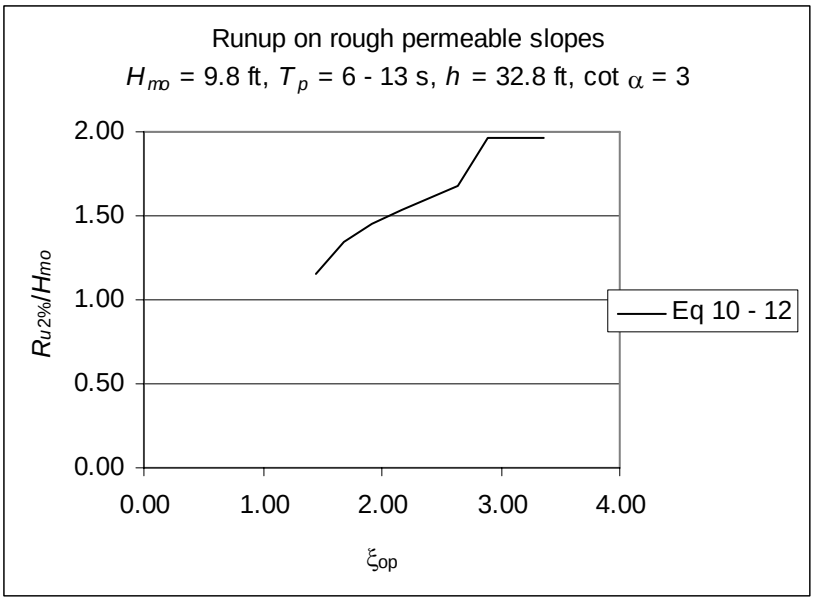

Figure 7. Runup as a function of Iribarren parameter for rough permeable uniform slopes exposed to unidirectional head-on incident waves. Steep slope with variable wave period.

TAW Wave Runup

TAW (2002) recommends that irregular wave runup on sloping structures be computed according to

$$\frac{R_{2\%}}{H_{mo}} = 1.65\gamma_f \gamma_\beta \xi_{om-1} \qquad 0.5 < \xi_{om-1} \le 1.8 \tag{13}$$

$$\frac{R_{2\%}}{H_{mo}} = \gamma_f \gamma_\beta (4.0 - 1.5/\sqrt{\xi_{om-1}}) \qquad \xi_{om-1} > 1.8 \tag{14}$$

$$\xi_{om-1} = \frac{\tan\alpha}{\sqrt{s_{om-1}}} \qquad s_{om-1} = \frac{H_{mo}}{L_{om-1}} \qquad L_{om-1} = \frac{gT_{m-1.0}^2}{2\pi} \tag{15}$$

TAW suggests that Equations 13 – 14 are valid for slopes 1V:1H to 1V:8H. Equations 13 – 14 can be applied to slopes as shallow as 1V:10H. They recommend a range $0.5 < \xi_{om-1} < 10$ with the range 8-10 being less accurate. For a reliability estimate, TAW recommends using a coefficient of variation of the central fit line of 0.07. $T_{m-1.0}$ can be calculated from measured wave data. However, measurements are rarely made in the nearshore zone. As stated previously, these equations are based on central fits to laboratory data. For deterministic design, TAW increases the empirical coefficients in the above equations (1.65, 4.0, and 1.5) by 6 - 7 percent.

For Equations 13 and 14, the following factors are recommended:

Wave Obliquity

$$\gamma_\beta = 1 - 0.0022|\beta| \qquad 0^\circ < |\beta| \le 80^\circ$$

$$\gamma_\beta = 1 - 0.0022 \cdot 80 \qquad |\beta| > 80^\circ$$

Slope Roughness

Slope Surface	γ_f
Concrete or Asphalt	1.0
Closed concrete block	1.0
Grass	1.0
Armorflex	0.9
Small blocks over 1/25 of surface	0.85
Small blocks over 1/9 of surface	0.80
Armor rock – single layer	0.70
Armor rock – two layers	0.55

Runup Based on Wave Momentum Flux

Various authors characterized the wave-structure interaction with a broad range of different types of wave statistics including time and frequency domain statistics, and mixes of the two in the same equation, as can be seen in the preceding relations. Most of the wave flume tests supporting the equations listed above utilized relatively deep water in the foreshore, so the incident waves were mostly nonbreaking. As shallow-water testing became more popular in the 1990's, alternate versions of the empirical equations were published (TAW 2002, CEM). The complex and nonlinear interaction between type of spectra, wave height, wave period and water depth in shallow water not accounted for in the simple form of Equation 1 resulted in relations that were dependent on the testing conditions. It is clear comparing the various equations in this report that there is still some migration of the parameters in the equations. This is evident when comparing CEM and TAW publications, published at roughly the same time. Relations with improved physics based on the nonlinear wave momentum flux have been developed that provide improved insight into the wave-structure interaction (Hughes 2003a,b, 2004a, b). These methods are summarized below.

Momentum Flux Parameter

Hughes (2003a,b, 2004a,b) proposed using the wave momentum flux to describe wave forces on coastal structures and the resulting response. He derived a wave momentum flux parameter that has the following attributes:

a. "The parameter is physically meaningful so it can be incorporated into theoretical models of specific physical processes in a rational way."
b. "The parameter applies to both periodic waves and nonperiodic waves."
c. "The parameter spans the range of relative depths from deep water to shallow water."

d. "The parameter is stable and robust."
e. "The parameter is easy to estimate so design guidance using the parameter can be programmed into computer spreadsheets or simple programs."

Assuming irrotational potential flow on a locally flat bottom in water depth h, the wave-averaged and depth-integrated radiation stress is given by

$$M = S_{xx} = \frac{1}{L}\int_0^L \int_{-h}^{\eta_x} (p_d + \rho_w u^2)\,dz\,dx \tag{16}$$

where L is the wave length, η_x is the free surface location, p_d is the dynamic pressure, ρ_w is the fluid density, u is the velocity in the x-direction, x is the horizontal coordinate, and z is the vertical coordinate. The maximum depth-integrated wave momentum flux is given by

$$(M_F)_{\max} = \int_{-h}^{\eta_x} (p_d + \rho u^2)\,dz \tag{17}$$

Using linear wave theory values for u and p_d yields

$$\left(\frac{(M_F)_{\max}}{\rho_w g h^2}\right)_{\max} = \frac{1}{2}\frac{H}{h}\frac{\tanh kh}{kh} + \frac{1}{8}\left(\frac{H}{h}\right)^2\left[1 + \frac{2kh}{\sinh 2kh}\right] \tag{18}$$

where g is the acceleration of gravity, H is the wave height, and $k = 2\pi/L$ is the wave number. In Equation 18, the first term on the right hand side is the dynamic pressure term while the second is the velocity term. In general, the pressure term will dominate. For example, for low steepness waves, the velocity term will only contribute 5 percent to the maximum momentum flux. For waves in shallow water at the steepness limit, the velocity term will provide the maximum contribution, roughly 30 percent of the momentum flux. Equation 18 assumes waves to be periodic and sinusoidal. But in shallow water, waves are nonlinear with peaked crests and shallow troughs. The wave forces from these nonlinear waves can be very different from those resulting from linear waves. Equation 18 will under predict the momentum under the nonlinear wave crest.

The maximum wave momentum flux is highly nonlinear for nonlinear waves - steep waves in shallow water. The linear wave momentum flux approximation does not include the portion of the wave momentum flux above the still water level. The estimate of wave momentum flux for shallow-water waves should include the portion above the still water level in order to provide an accurate solution. Hughes (2003a) described a non-linear wave momentum flux using a numerical Fourier solution. The resulting approximate relation was found to be

$$\left(\frac{M_F}{\rho_w g h^2}\right)_{\max} = A_0\left(\frac{h}{gT_p^2}\right)^{-A_1}$$

$$A_0 = 0.639\left(\frac{H_{mo}}{h}\right)^{2.026} \tag{19}$$

$$A_1 = 0.180\left(\frac{H_{mo}}{h}\right)^{-0.391}$$

For a runup calculation, Hughes replaced the regular wave parameters H and T by the spectral parameters H_{mo} and T_p.

Irregular Wave Runup

Hughes (2003b, 2004b) derived equations for irregular wave runup using the momentum flux parameter. He assumed that the weight of fluid in a triangular wedge on the structure during wave runup is proportional to the incident depth-integrated wave momentum flux at the structure toe. Equations for runup based on wave momentum flux were developed as follows:

Smooth impermeable slopes, non-breaking, surging or collapsing waves ($H_{mo}/L_p < 0.0225$)

$$\frac{R_{u2\%}}{h} = 1.75\left[1 - e^{-(1.3\cot\alpha)}\right]\left(\frac{M_F}{\rho_w g h^2}\right)^{1/2} \qquad for \quad 1.0 \le \cot\alpha \le 4.0 \tag{20}$$

Smooth impermeable slopes, plunging or spilling waves ($H_{mo}/L_p > 0.0225$)

$$\frac{R_{u2\%}}{h} = 1.75\left[1 + e^{-(0.47\cot\alpha)}\right]\left(\frac{M_F}{\rho_w g h^2}\right)^{1/2} \qquad for \quad 1.5 \le \cot\alpha \le 4.0 \tag{21}$$

Smooth impermeable slopes, plunging or spilling ($H_{mo}/L_p > 0.0225$). Slightly less accurate equation but simpler and covering a broader range of structure slopes.

$$\frac{R_{u2\%}}{h} = 4.4(\tan\alpha)^{0.7}\left(\frac{M_F}{\rho_w g h^2}\right)^{1/2} \qquad for \quad 1.5 \le \cot\alpha \le 30 \tag{22}$$

Rough impermeable slopes for all wave conditions from nonbreaking to breaking

$$\frac{R_{u2\%}}{h} = 4.4(\tan\alpha)^{0.7}\left(\frac{M_F}{\rho_w g h^2}\right)^{1/2}(0.505) \qquad for \quad 2.0 \le \cot\alpha \le 4.0 \qquad (23)$$

where h is the depth at the toe of the structure. Hughes argued that the runup on a rough slope is a function of both the effect of the roughness to reduce the runup and the porosity which stores fluid during the any runup cycle. The effect of this storage on runup will be influenced by the wave period, among other things. So a simple linear coefficient to account for roughness is relatively crude method for capturing the effect of this complex phenomenon. However, this method has been used with success for many years and it provides a reasonable approximation of the runup. So Hughes adopted this strategy in Equation 13.

Runup equations 3 – 14 and 20 – 23 are plotted in Figures 8 - 10. These figures are similar to Figures 1 - 3. Based on these figures, Equations 13 and 14 appear to provide a similar fit to Equations 5 – 6 and 9 – 12. However, the prediction is less conservative than Equations 3 and 4 for steep slopes. Equations 20 and 21 are valid for only steeper structure slopes. In this range, the equations tend to be the most conservative for the range of conditions evaluated for Figure 1. Equations 22 and 23 are valid over a wider range of conditions but are slightly less conservative than the other relations in Figures 8 and 9. However, in Figure 11, Equation 22 is significantly more conservative. Because Equations 22 and 23 are based on first-principles, it is expected that these equations would provide more insight and a broader range of applicability than the other equations. It is therefore recommended that these equations be used where applicable.

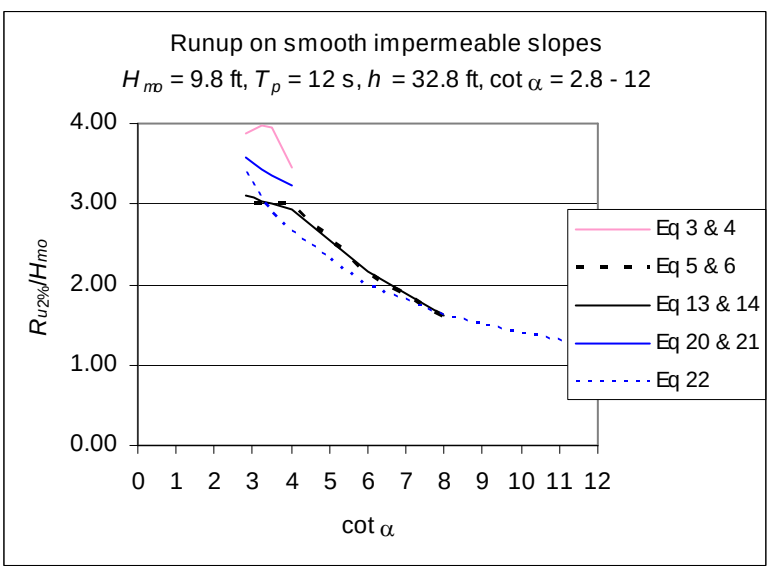

Figure 8. Runup as a function of structure slope for smooth impermeable uniform slopes exposed to unidirectional head-on incident waves. Wave conditions are constant so plot shows effect of changing slope.

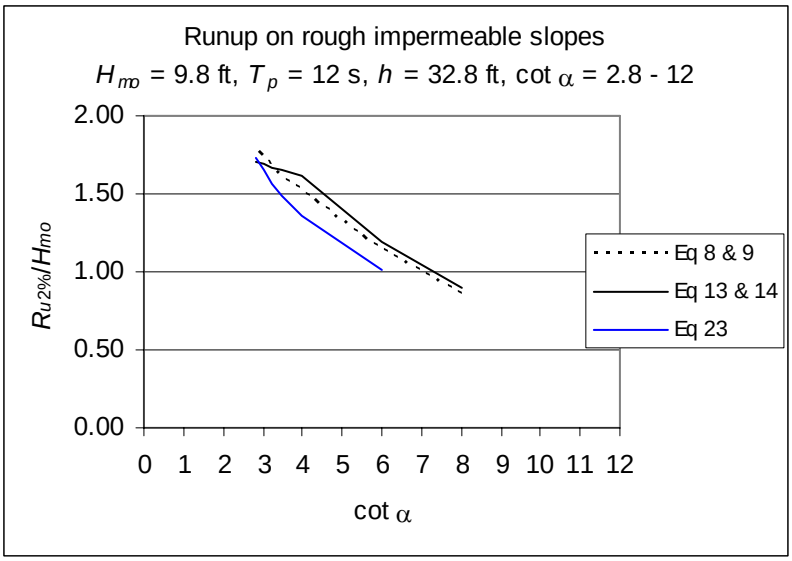

Figure 9. Runup as a function of structure slope for rough impermeable uniform slopes exposed to unidirectional head-on incident waves. Wave conditions are constant so plot shows effect of changing slope.

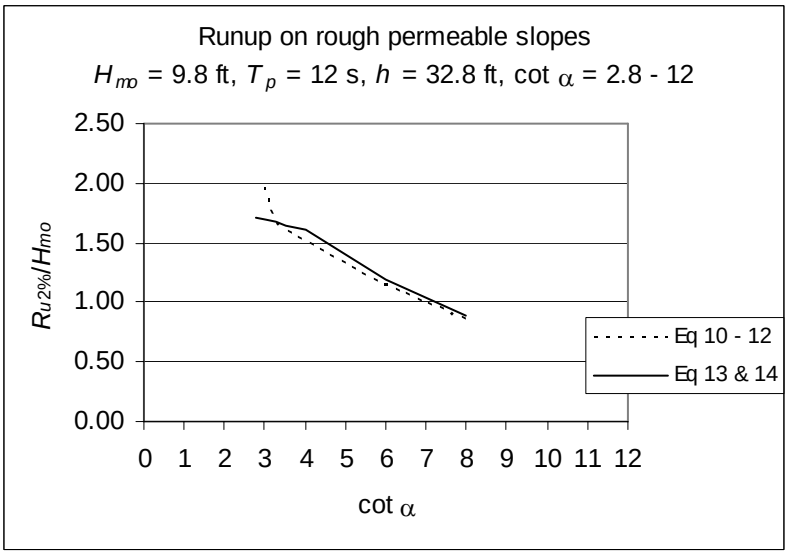

Figure 10. Runup as a function of structure slope for rough permeable uniform slopes exposed to unidirectional head-on incident waves. Wave conditions are constant so plot shows effect of changing slope.

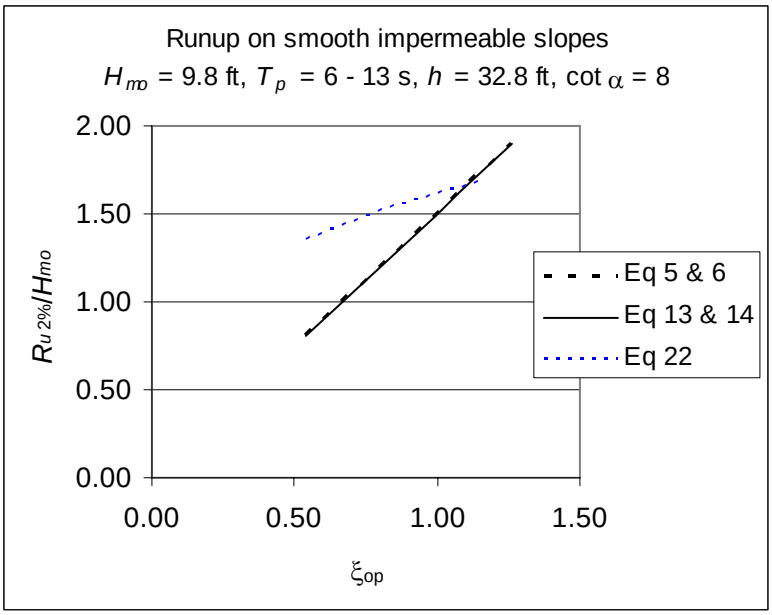

Figure 11. Runup as a function of Iribarren parameter for smooth impermeable uniform slopes exposed to unidirectional head-on incident waves. Steep slope with variable wave period.

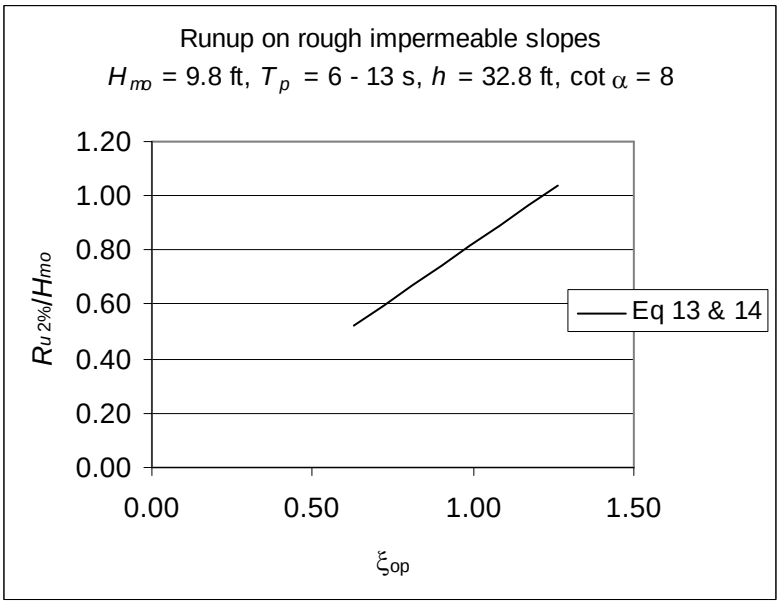

Figure 12. Runup as a function of Iribarren parameter for rough impermeable uniform slopes exposed to unidirectional head-on incident waves. Shallow slope with variable wave period.

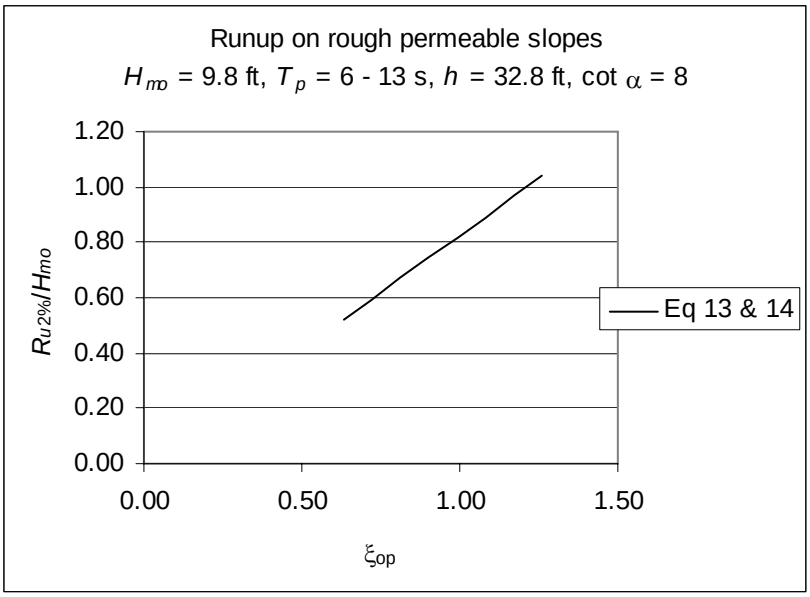

Figure 13. Runup as a function of Iribarren parameter for rough permeable uniform slopes exposed to unidirectional head-on incident waves. Shallow slope with variable wave period.

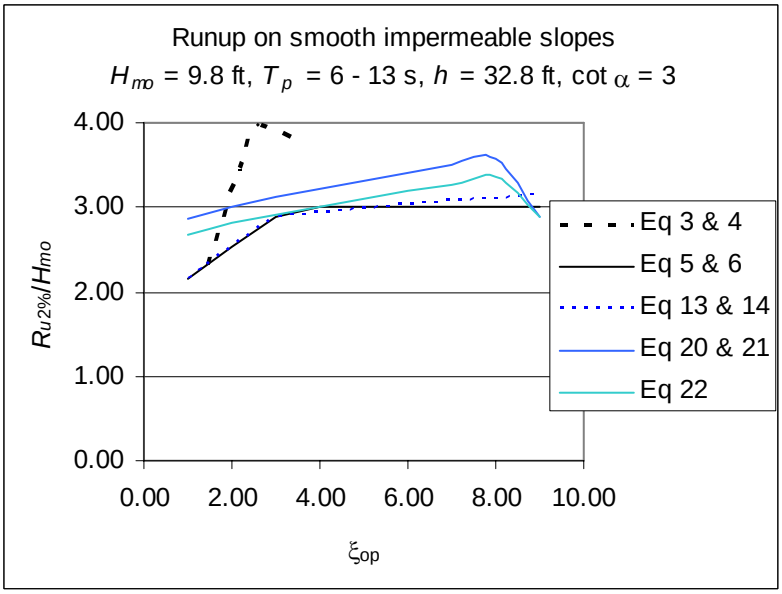

Figure 14. Runup as a function of Iribarren parameter for smooth impermeable uniform slopes exposed to unidirectional head-on incident waves. Steep slope with variable wave period.

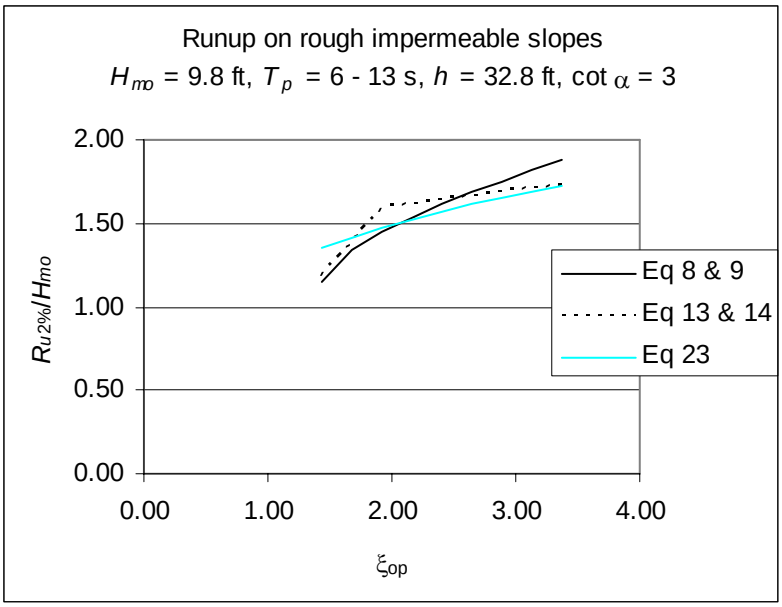

Figure 15. Runup as a function of Iribarren parameter for rough impermeable uniform slopes exposed to unidirectional head-on incident waves. Steep slope with variable wave period.

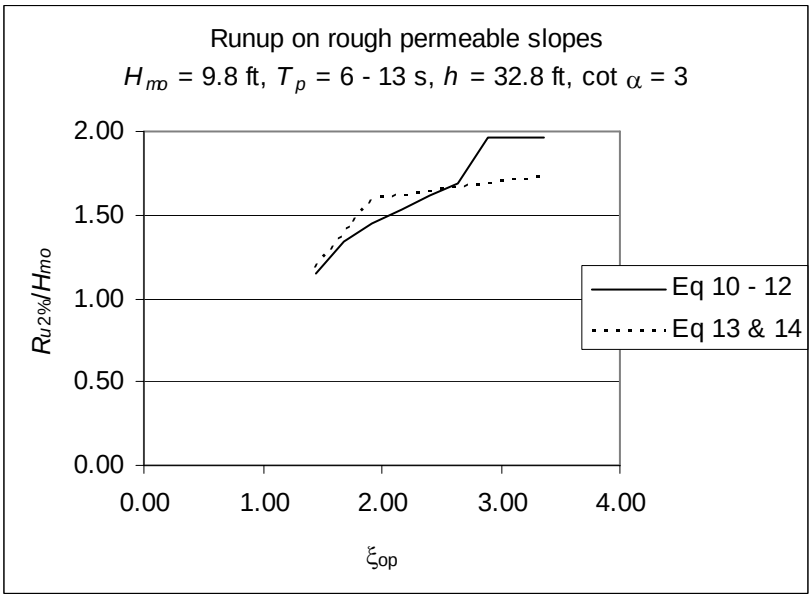

Figure 16. Runup as a function of Iribarren parameter for rough permeable uniform slopes exposed to unidirectional head-on incident waves. Steep slope with variable wave period.

Compound Slope Runup

The CEM provides equations for determining irregular wave runup on a compound slope. The equations (24 - 32) are given in the following section. Figure 17 shows a graphic idealization of a typical compound slope.

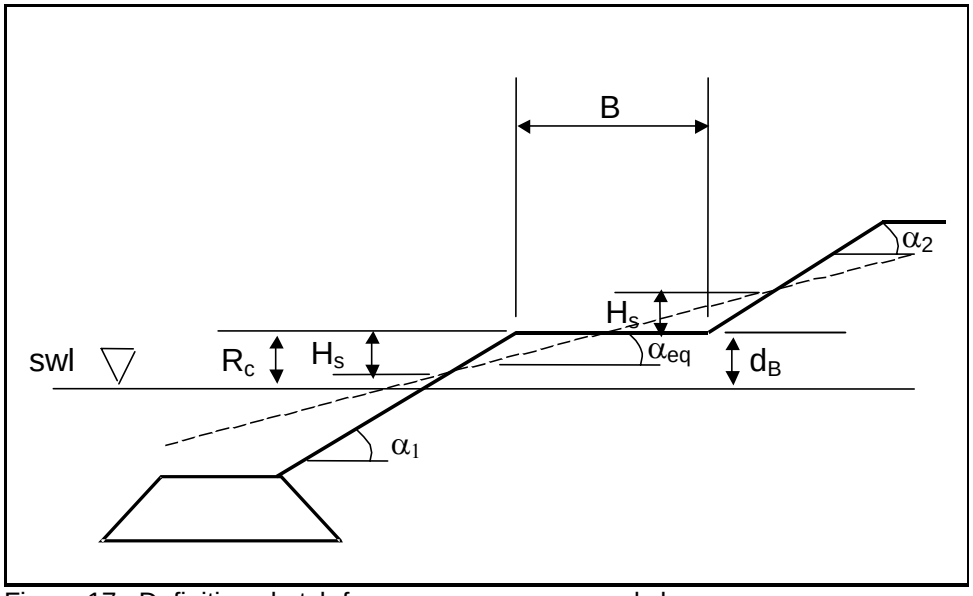

Figure 17. Definition sketch for runup on a compound slope

The runup relation for a compound slope is given by

$$\frac{R_{2\%}}{H_s} = \begin{cases} 1.5\xi_{eq}\gamma_f\gamma_h\gamma_\beta & for \quad 0.5 < \xi_{eq} \le 2 \\ 3.0\gamma_f\gamma_h\gamma_\beta & for \quad \xi_{eq} > 2 \end{cases} \tag{24}$$

where the equivalent Iribarren number is given by equations 25 – 32.

$$\xi_{eq} = \xi_{op}[1 - r_B(1 - r_{dB})] \qquad 0.6 \le \gamma_b \le 1.0 \tag{25}$$

$$r_B = 1 - \frac{\tan\alpha_{eq}}{\tan\bar{\alpha}} \tag{26}$$

$$r_{dB} = 0.5\left(\frac{d_B}{H_s}\right)^2 \qquad 0 \le r_{dB} \le 1 \tag{27}$$

$$\xi_{op} = \frac{\tan\alpha_1}{\sqrt{s_{op}}} \tag{28}$$

$$\tan\alpha_{eq} = \frac{2H_s}{B + H_s(\cot\alpha_1 + \cot\alpha_2)} \tag{29}$$

$$\tan\bar{\alpha} = \frac{2H_s}{(H_s - d_B)\cot\alpha_1 + (H_s + d_B)\cot\alpha_2} \qquad -H_s \le d_B \le H_s \quad (30)$$

$$\begin{aligned} \tan\bar{\alpha} &= \tan\alpha_1 \quad if\ d_B < -H_s \\ \tan\bar{\alpha} &= \tan\alpha_2 \quad if\ d_B > H_s \end{aligned} \quad (31)$$

where

R_c = dike crest freeboard
α_1 = lower structure slope
α_2 = upper structure slope
γ_b = berm influence factor = ξ_{eq}/ξ_{op}
d_B = depth of berm crest, negative if the reference still water level is below the berm crest

If $d_B < -H_s\sqrt{2}$ then $R_{u2\%} = R_c$.

Wave Overtopping

Both the CEM and TAW give overtopping formulae for bermed and straight impermeable slopes based on the data of van der Meer and Janssen (1995). The equations are similar. Therefore, only the relations from TAW will be given here. The average volume rate of irregular wave overtopping per unit length of structure q is given by

$$\frac{q}{\sqrt{gH_{mo}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b\xi_{om-1.0}\exp\left(-4.3\frac{R_c}{H_{mo}}\frac{1}{\xi_{om-1.0}\gamma_f\gamma_b\gamma_\beta}\right) \quad (32)$$

and

$$\frac{q}{\sqrt{gH_{mo}^3}} \le 0.2\exp\left(-2.6\frac{R_c}{H_{mo}}\frac{1}{\gamma_f\gamma_\beta}\right) \quad (33)$$

where R_c is the structure crest height above the still water level. These curves are central fits to laboratory data. For a reliability estimate, TAW recommends assuming that the coefficient 4.3 is normally distributed with a standard deviation of 0.5. Similarly, the standard deviation of the coefficient 2.6 is 0.35. According to CEM Table VI-5-6, significant damage to a non-paved levee crest will occur if $4.0 < q < 16.1$ gal/s/ft ($0.05 < q < 0.20$ m^3/s/m). For paved or lightly armored crests, the lower limit value is $q = 16.1$ gal/s/ft (0.2 m^3/s/m). Determining values of coefficients γ_r, γ_b, γ_h, γ_β was discussed in the preceding sections.

Stone Armor Stability

Background

The Hudson Equation is well known and has been used for years to determine armor stability. The equation in stability number form is given by

$$N_s = \frac{H_s}{\Delta D_{n50}} = (K_D \cot \alpha)^{1/3} \tag{34}$$

where $\Delta = S_r - 1$, $S_r = \rho_r/\rho_w$ = specific gravity of stone, ρ_r = density of stone, ρ_w density of water, $D_{n50} = (V_{50})^{1/3}$ = nominal stone diameter, $V_{50} = M_{50}/\rho_r$ = median volume of armor stone, M_{50} = median mass, K_D = empirical coefficient and cot α = structure seaward slope. K_D takes into account all parameters not in the equation. Recommended values of the coefficient K_D have been determined from laboratory data to fit the lower envelope of stability. The appropriate irregular wave height statistic has been discussed by many authors. Melby (2003) notes that recent guidance suggests H_s is reasonable if using K_D values published in the Shore Protection Manual (1984). The Hudson equation design assumes damage based on eroded volume of D% = 0 to 5.

As given in the CEM, Van der Meer (1987) proposed equations in the late 1980's that are based solely on irregular wave experiments and explicitly include more parameters. These relations were given as

For plunging waves where $\xi_{om} < \xi_{mc}$:

$$N_s = \frac{H_{2\%}}{\Delta D_{n50}} = 8.7P^{0.18}\left(\frac{S}{\sqrt{N_z}}\right)^{0.2} \xi_{om}^{-0.5} \tag{35}$$

For surging waves where $\xi_{om} > \xi_{mc}$:

$$N_s = \frac{H_{2\%}}{\Delta D_{n50}} = 1.4P^{-0.13}\left(\frac{S}{\sqrt{N_z}}\right)^{0.2} \sqrt{\cot\alpha}\,\xi_{om}^{P} \tag{36}$$

where

$$\xi_{mc} = [6.2P^{0.31}\sqrt{\tan\alpha}]^{\left(\frac{1}{P+0.5}\right)} \tag{37}$$

Here P = notional permeability, S = eroded area or damage, and N_z = storm duration/T_m. Damage levels given by S = 1 to 3 represent the start of damage and correspond to Hudson's D% = 0 to 5 percent. For an impermeable dike, P = 0.1. For a traditional multi-layer breakwater, P = 0.4.

Van der Meer suggested using two-percent exceedance value $H_{2\%}$ for application in shallow water. The stability equations for intermediate depth water are identical except that the Rayleigh relation $H_{2\%} = 1.4H_s$ has been used above. The statistic $H_{2\%}$ must be determined from actual measured time series of water surface elevation in shallow water. If $H_{2\%}$ is determined from a synthetic distribution, a Rayleigh distribution must be assumed. In this case, the equations using $H_{2\%}$ are identical to those based on H_s. Further, Equations 35 - 37 were developed from primarily

intermediate depth laboratory tests and do not explicitly incorporate depth. Therefore, Equations 35 - 37 are for primarily intermediate to deep-water applications.

Stability Equations Based on the Wave Momentum Flux

Melby and Hughes (2004) proposed stone stability equations specifically for both deep and shallow water applications. They derived stability equations based on the maximum momentum flux and fit to van der Meer's data. They noted that the equations were based on first principles and would therefore be applicable to a wider range of conditions. In the following, the background for the maximum wave momentum flux is given. Use of a nonlinear approximation for momentum flux is important because stability is at its minimum when the incident wave is the most nonlinear.

Two stability equations resulted from the fit to data. The recommended equations for armor stone stability were

<u>Plunging Waves</u>

$$N_m = 5.0(S / N_z^{0.5})^{0.2} P^{0.18} \sqrt{\cot\theta} \qquad s_m \geq s_{mc} \tag{38}$$

<u>Surging Waves</u>

$$N_m = 5.0(S / N_z^{0.5})^{0.2} P^{0.18} (\cot\theta)^{0.5-P} s_m^{\ -P/3} \qquad s_m < s_{mc} \tag{39}$$

where

$$s_{mc} = -0.0035 \cot\alpha + 0.028 \tag{40}$$

and

$$N_m = \left(\frac{K_a (M_F / \gamma_w h^2)_{\max}}{(S_r - 1)} \right)^{1/2} \frac{h}{D_n} \tag{41}$$

with $K_a = 1$.

Equations 38 - 40 are analogous to Equations 35 – 37. It is clear that the wave forcing portion given by equations 38 – 41 provides a more rigorous prediction of the incident wave effect on stability. The inclusion of depth explicitly in Equation 41 through the maximum momentum flux is a significant improvement. The wave-structure interaction portion described by the right hand side of Equations 38 and 39 is similar to that given in the van der Meer equations.

Accumulated Damage

Although Equations 38 – 41 provide a way to predict damage on a structure, the damage is for constant wave conditions – a single storm. The CEM summarize Melby and Kobayashi's method to predict the normalized eroded cross sectional area as a function of time for varying wave and water level conditions. The normalized eroded area as a function of time is given as

$$\bar{S}(t) = \bar{S}(t_n) + a_p N_{mo}^5 T_p^{-b}(t^b - t_n^b) \quad for \quad t_n \le t \le t_{n+1} \tag{42}$$

where $\bar{S}(t_n) = A_e/D^2_{n50}$ is the mean damage at time t_n, A_e is the mean eroded cross sectional area, $N_{mo} = H_{mo}/\Delta D_{n50}$ is the stability number, and a_p and b are empirical parameters. A similar equation uses time domain wave parameters. The calibrated parameter values are $a_p = 0.022$ and $b = 0.25$. Note that S can be thought of as the number of stones displaced from a D_{n50}-wide cross section. The standard deviation of S was given as a function of the mean $\bar{S}(t_n)$ by the relation $\sigma_S = 0.5\bar{S}^{0.65}$. This standard deviation describes the alongshore variability of damage. Also given were relations for maximum depth of erosion, minimum remaining cover depth, and length of the eroded hole. The maximum eroded depth is d_e, minimum remaining cover depth is d_c, and maximum eroded length is l_e. These three parameters are normalized to obtain $E = d_e/D_{n50}$, $C = d_c/D_{n50}$, and $L = l_e/D_{n50}$. Melby and Kobayashi (1998) expressed the key profile parameters as a function of the mean damage as follows: $\bar{E} = 0.46\bar{S}^{0.5}$, $\bar{C} = C_o - 0.1\bar{S}$, $\bar{L} = 0.44\bar{S}^{0.5}$ where C_o is the initial armor layer thickness.

A modification to Equation 42 was introduced by Melby and Kobayashi (1999) to allow for non-zero initial damage values. The modified equation is

$$\begin{aligned} \bar{S}(t) &= 0.011\, N_s^5 (N_e + \delta N)^{0.25} \\ N_e &= \left(\frac{\bar{S}(t_n)}{0.011\, N_s^5} \right)^4 \\ \delta N &= (t - t_n)/T_m \quad for \quad t_n \le t \le t_{n+1} \end{aligned} \tag{43}$$

This equation is similar to Equation 35 but predicted damage is not dependent on the time that the simulation begins. For a typical application, accumulated mean eroded area is predicted using Equation 43 if the zero damage condition characterized by Equations 38 - 41 is exceeded. The parameters E, C, and L can also be predicted if the zero-damage level is exceeded.

Rock-Armored Toe Stability

A rock-armored toe berm can be either emergent or submerged, depending on the water level. Stability equations are given in the CEM for a submerged toe berm, for a low-crested stand-alone structure, and for a submerged stand-alone but not for a sometimes-emergent toe berm. The toe stability equation is only applicable to deep toes. There is no guidance directly applicable to the emergent toe berm.

<u>Deeply-Submerged Toe</u>

The stable toe berm stability number is given by

$$N_s = \left(0.24\frac{h_b}{D_{n50}} + 1.6\right)N_{od}^{0.15} \tag{44}$$

or, rearranging, the nominal diameter is given by

$$D_{n50} = 0.625\frac{H_s}{\Delta N_{od}^{0.15}} - 0.15h_b \tag{45}$$

where

N_{od} = 1 for minimal damage
h_b = height of water surface above berm crest

Equations 37 - 38 are limited to the condition where $0.4 < h_b / h_s < 0.9$. Equation 45 requires an iterative solution which may not converge for some conditions. The design guidance provides no alternative if Equation 45 does not converge.

Barely Submerged Toe Berm

In the CEM, the submerged structure median stable weight W_{50} is determined using the following equations

$$\frac{h_c^{'}}{h} = (2.1 + 0.1S)\exp(-0.14N_s^*) \tag{46}$$

or, after rearranging,

$$N_s^* = -\left(\frac{1}{0.14}\right)\ln\left(\frac{h_c^{'}}{h}\frac{1}{(2.1+0.1S)}\right) \tag{47}$$

$$W_{50} = \gamma_r D_{n50}^3 = \gamma_r\left(\frac{H_s s_p^{-1/3}}{\Delta N_s^*}\right)^3 \tag{48}$$

$$s_p = \frac{H_s}{L_p} \qquad \Delta = S_r - 1 = \frac{\rho_r}{\rho_w} - 1 \tag{49}$$

where

h'_c = height of toe berm above bottom
h = water depth seaward of toe berm

N_s* = $H_s/(\Delta\, D_{n50}\, s_p^{-1/3})$ = spectral stability number
L_p = wave length corresponding to peak spectral frequency at toe of structure
W_{50} = median stone weight
γ_r = stone specific weight

Low-Crested Toe Berm and Low-Crested Stand-Alone Mound

The stable weight for a rubble mound structure was discussed earlier. Equations 38 – 41 assume a traditional two-stone thick armor layer and filter layers below the armor as well as a stable toe. These equations also assume little or no overtopping. The stable weights from Equations 38 – 41 can be modified for the condition of a heavily overtopped low structure crest. A modification is suggested by van der Meer (1991) and summarized in the CEM in Table VI-5-24. The modification reduces the stable armor weight by a small amount, the amount increasing as the crest approaches the still water level. The reduction relation is given by

$$f_i = \left(1.25 - 4.8\frac{R_c}{H_s}\sqrt{\frac{s_{op}}{2\pi}}\right)^{-1} \qquad 0.8 \le f_i \le 1.0$$
$$0 < \frac{R_c}{H_s}\sqrt{\frac{s_{op}}{2\pi}} < 0.052 \tag{50}$$

and is applied to the nominal median diameter. So in the stability equations, D_{n50} is replaced by $f_i D_{n50}$. The equation above is limited to $R_c > 0$.

Equation 50 is for emergent structures only. This method results in a more conservative stone size than if a submerged armor weight stability equation is used. For the condition where the water level is at the structure crest, $f_i = 0.8$ and the reduction in D_{n50} is 20 percent. As the freeboard increases, f_i approaches 1.0, depending on the wave height and wave period.

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Analysis Results

Waves and Water Levels for Design

Waves and water levels for this preliminary design were discussed in previous appendices. In summary, 10 storms with roughly the same intensity but following different tracks were used to generate waves and water levels near the structure toe along the five levee alignments. Significant wave height, peak wave period, surge, and depth were output at specified save points near the structure toe. The save points are shown in Figures 1 – 10.

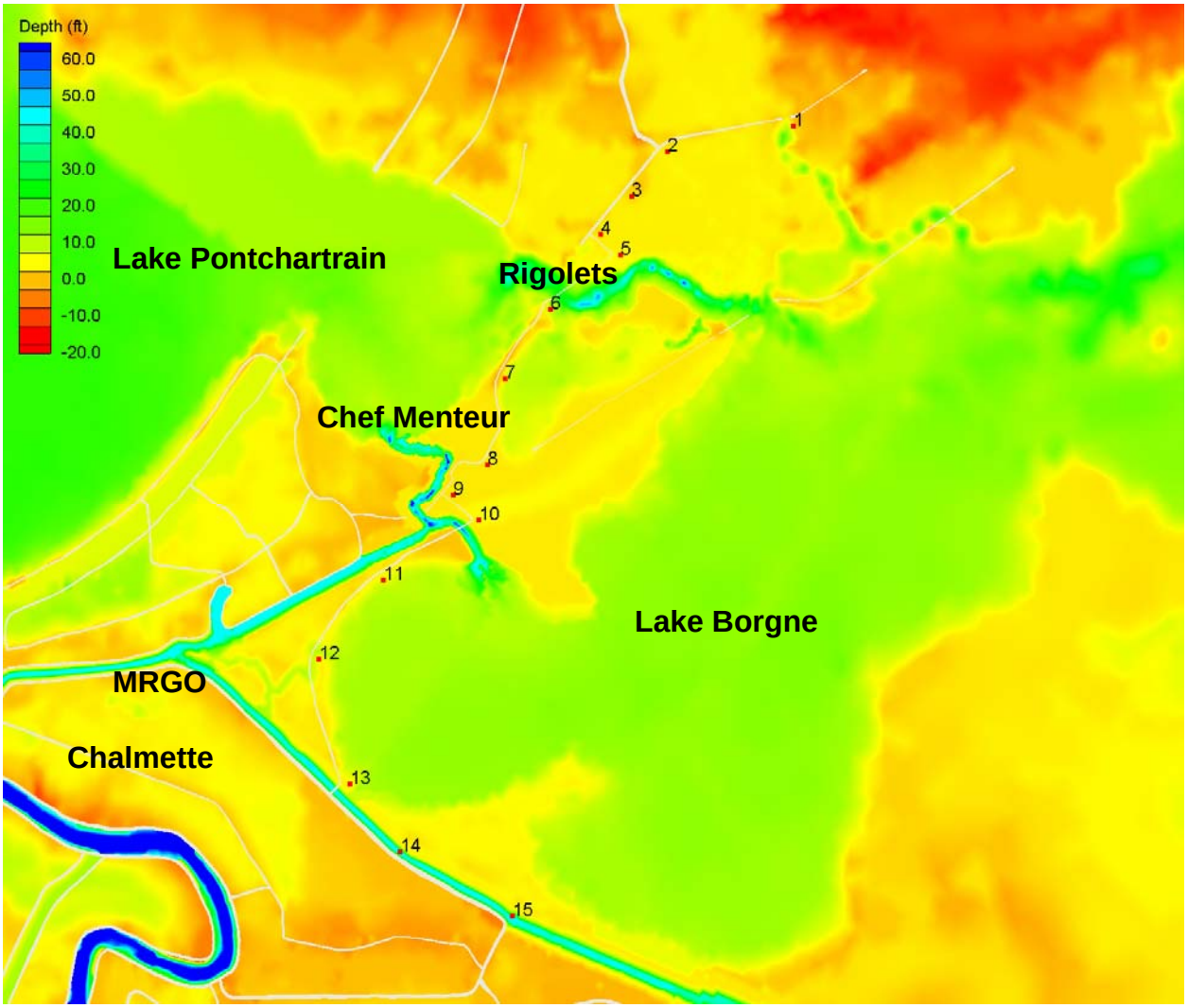


Figure 1. Save Points 1-15 (STWAVE Southeast grid)

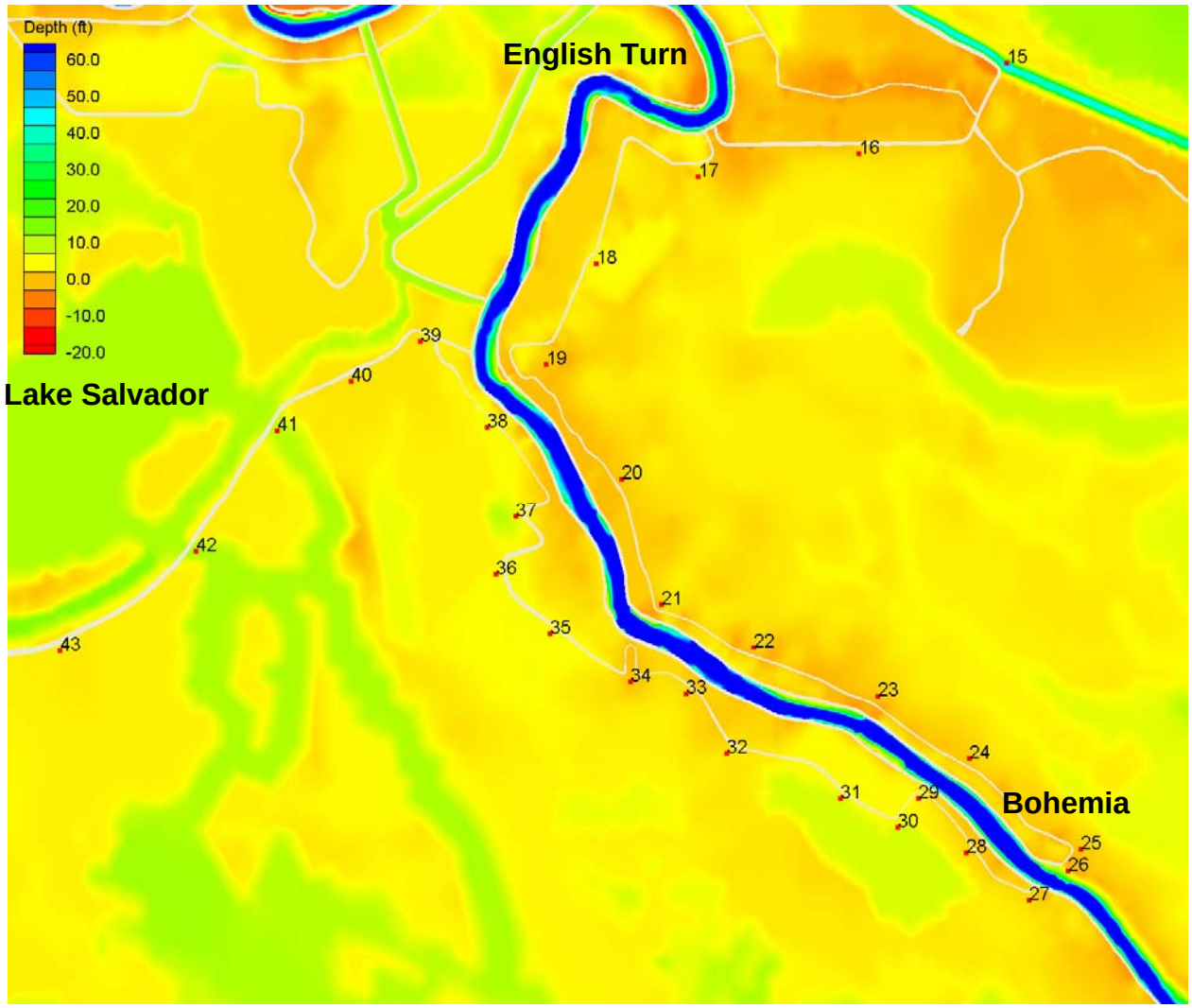


Figure 2. Save Points 15-26 (STWAVE Southeast grid) and 27 to 43 (STWAVE South grid)

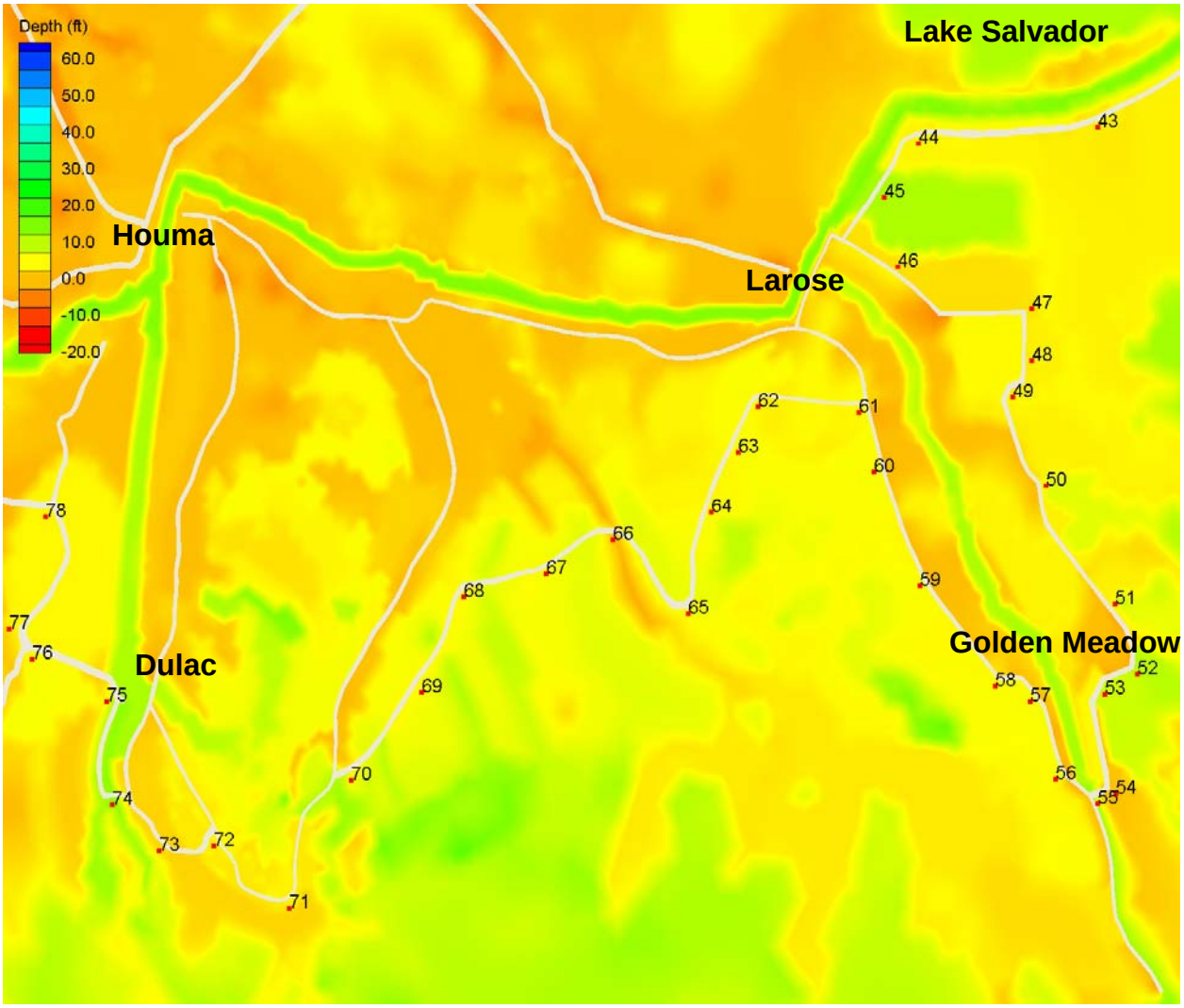


Figure 3. Save Points 43 to 78 (STWAVE South grid) and 72 to 78 (STWAVE West grid)

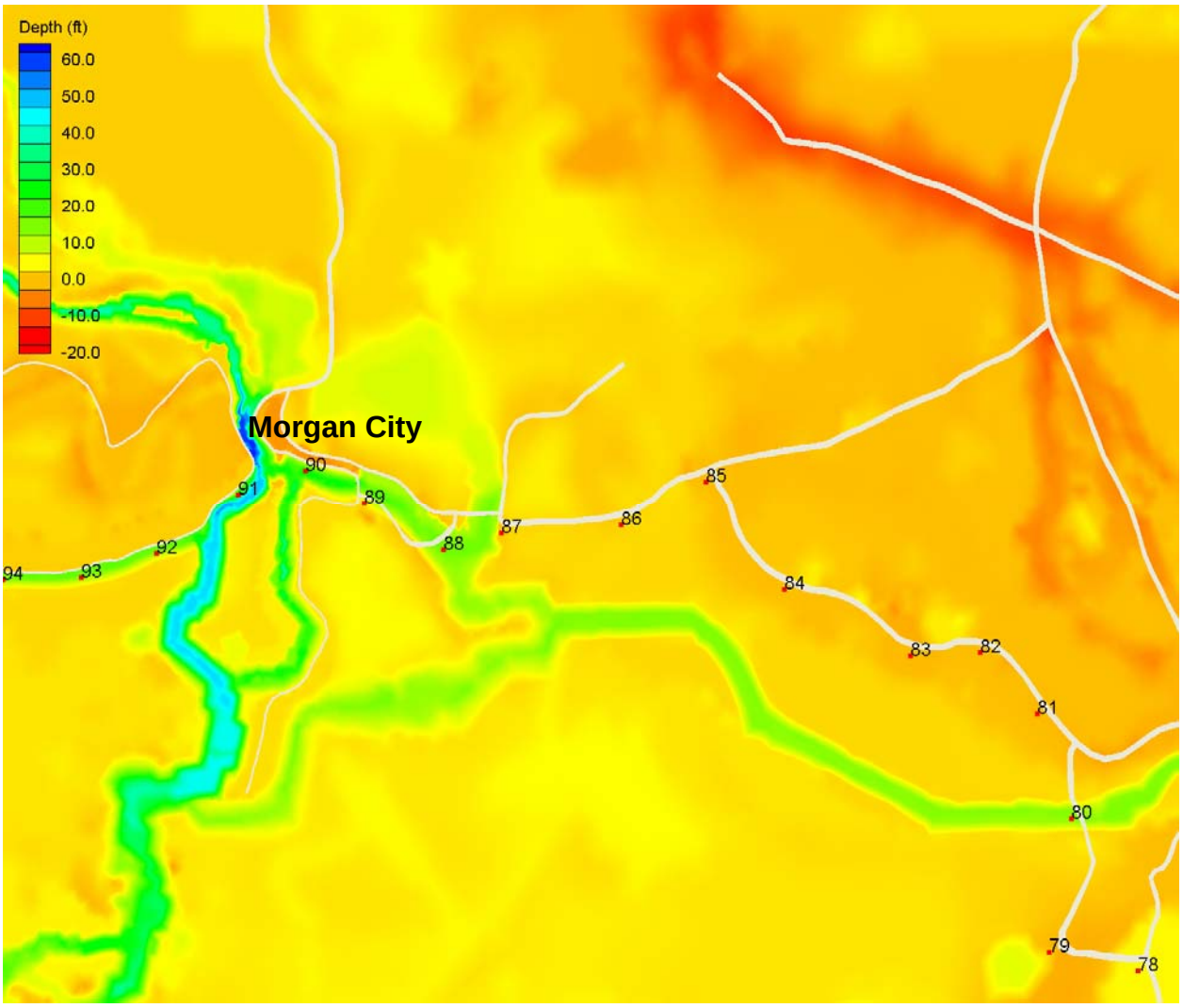


Figure 4. Save Points 78 to 88 (STWAVE South grid) and 78 to 94 (STWAVE West grid)

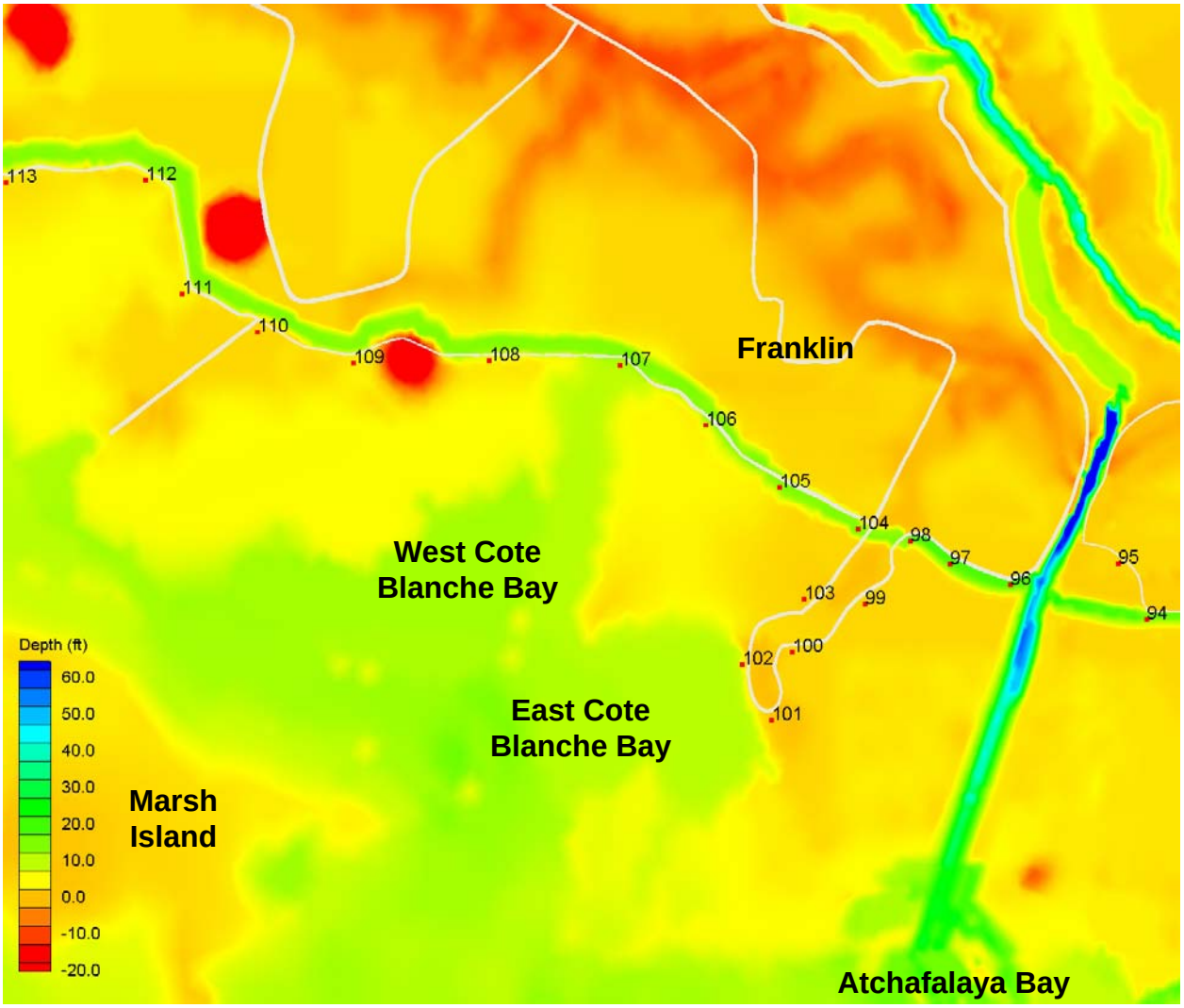


Figure 5. Points 94 to 113 (STWAVE West grid)

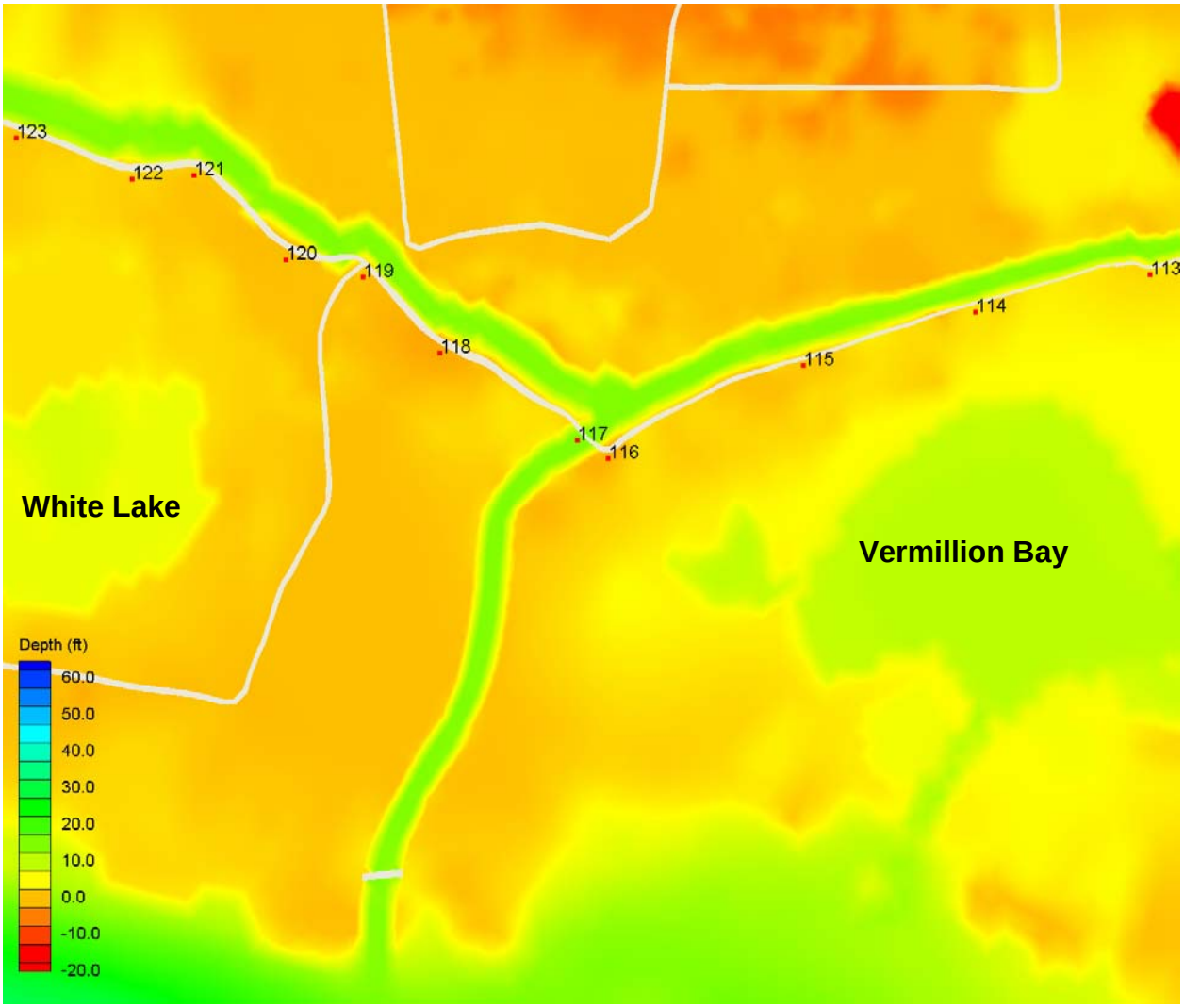


Figure 6. Points 113 to 123 (STWAVE West grid)

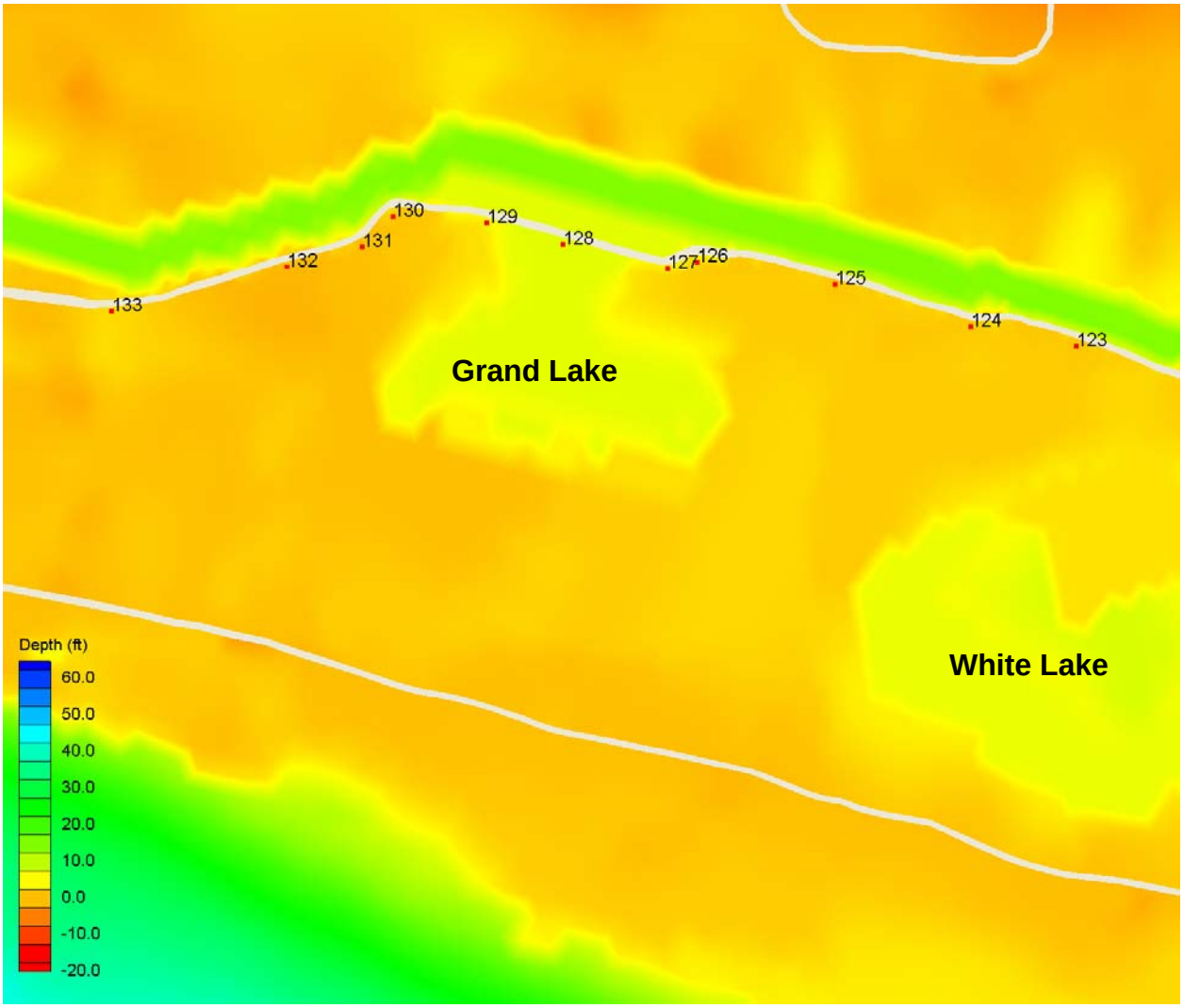


Figure 7. Points 123 to 133 (STWAVE West grid)

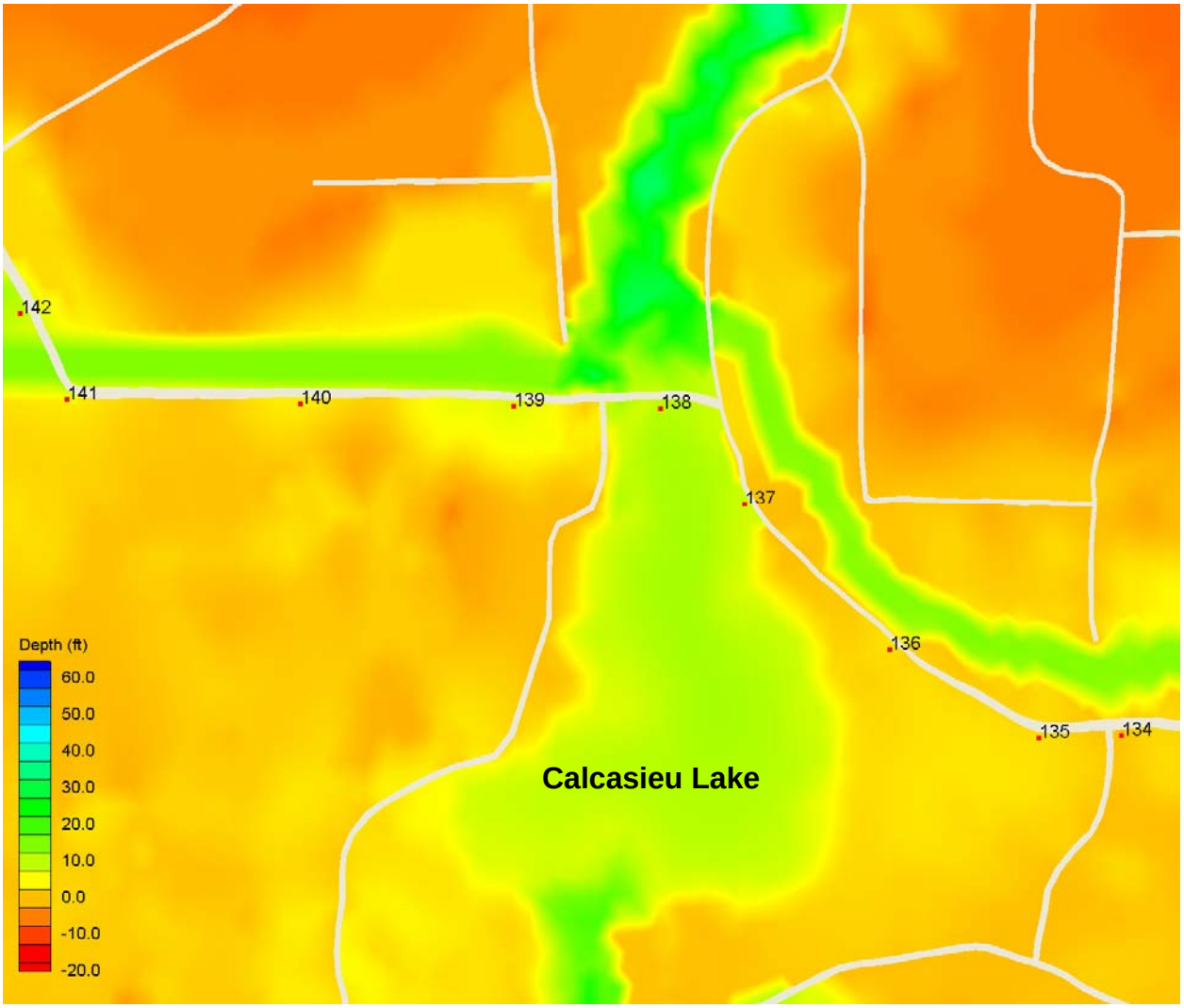


Figure 8. Points 134 to 142 (STWAVE West grid)

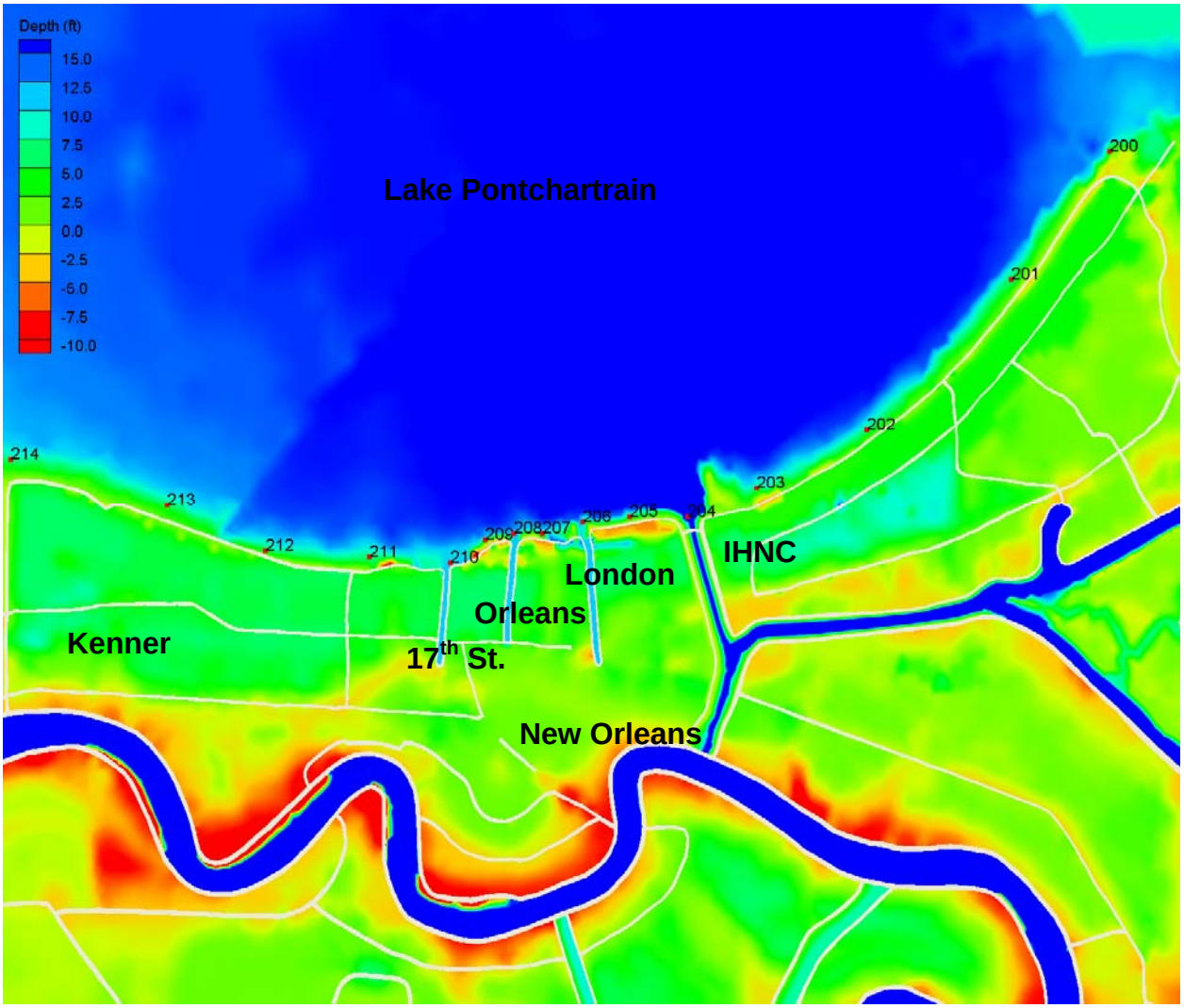


Figure 9. Points 200 to 214 (STWAVE Pontchartrain grid)

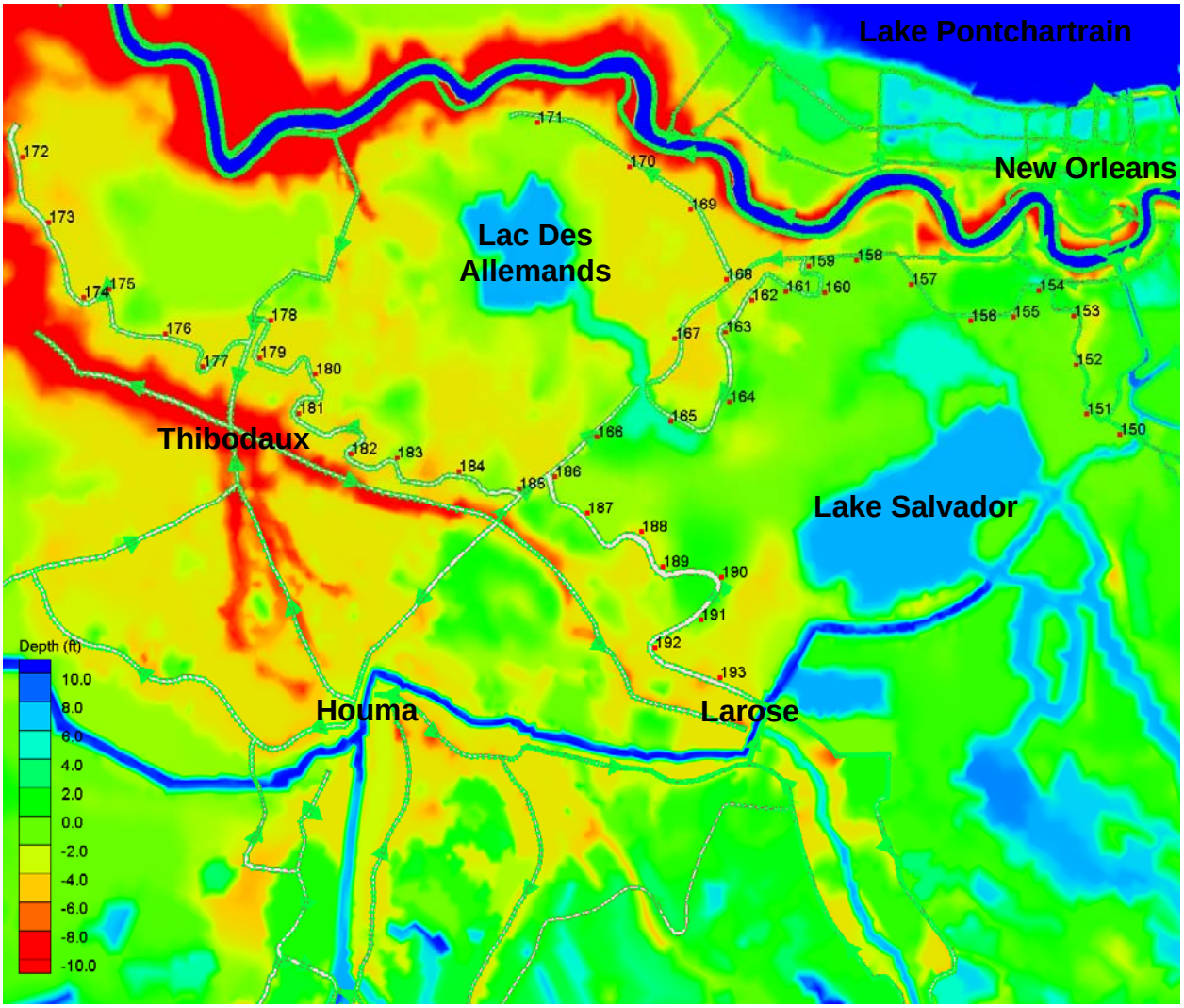


Figure 10. Points 150-172 (STWAVE South Grid Alternatives 3 and 4)

The wave and water level conditions produced by the ten storms on five alignments were evaluated to determine which storms produced the most severe conditions. The 1893 storm (T10) produced the most severe conditions along the central and westerly reaches while the 1947 event (T3) produced the worst conditions on the easterly reaches. Hurricane Andrew (T2) produced the most severe conditions in the Barataria reach and Hurricane Rita (T4) produced the most extreme conditions on the far western reaches. Although these storms impacted only short reaches with the highest waves and surges, for this preliminary design it was assumed that the impact of these storms could easily spread over extensive stretches of the levee system. As such, the highest waves and water levels from these storms were spread over the save points as listed in Tables 1 – 5, corresponding to alignments 1 - 5. These conditions are plotted in Figures 11 – 15 for Alignments 1 – 5, respectively.

Table 1. Summary of Waves and Water Levels Used for Levee Analysis for Alignment 1			
Save Points	Surge h_s (ft)	Wave Height H_{mo} (ft)	Wave Period T_p (s)
1 – 10	33	12	11
11 – 26	36	13	14
27 – 33	33	12	14
34 - 44	30	11	14
45 – 51	33	12	14
52 - 53	36	13	14
54 – 104	40	15	14
105 - 126	36	13	14
127 – 134	33	12	14
135 - 142	30	11	14
200 - 214	13 - 20	9 - 12	8

Table 2. Summary of Waves and Water Levels Used for Levee Analysis for Alignment 2			
Save Points	Surge h_s (ft)	Wave Height H_{mo} (ft)	Wave Period T_p (s)
1 – 10	30	11	11
11 – 14	33	12	14
15 – 26	36	13	14
27 – 33	33	12	14
34 – 44	30	11	14
45 – 51	33	12	14
52 – 53	36	13	14
54 – 104	40	15	14
105 – 126	36	13	14
127 – 134	33	12	14
135 – 142	30	11	14
200 – 214	14 - 22	10 - 13	8

Table 3. Summary of Waves and Water Levels Used for Levee Analysis for Alignment 3			
Save Points	Surge h_s (ft)	Wave Height H_{mo} (ft)	Wave Period T_p (s)
1 – 10	33	12	11
11 - 26	36	13	14
27 – 33	33	12	14
34 – 37	30	11	14
38	26	10	14
39	23	9	14
46 - 49	26	10	14
50	30	11	14
51	33	12	14
52 - 53	36	13	14
54 – 104	40	15	14
105 – 126	36	13	14
127 – 134	33	12	14
135 – 142	30	11	14
150 – 164	20	8	14
165	17	6	14
166 – 184	10	4	14
185	13	5	14
186 – 190	17	6	14
191 – 193	26	10	14
200 - 214	13 - 20	9 - 12	8

Table 4. Summary of Waves and Water Levels Used for Levee Analysis for Alignment 4			
Save Points	Surge h_s (ft)	Wave Height H_{mo} (ft)	Wave Period T_p (s)
1 – 10	33	12	11
11 - 26	36	13	14
27 – 33	33	12	14
34 - 37	30	11	14
38	26	10	14
39	23	8	14
46 – 49	26	10	14
50	30	11	14
51	33	12	14
52 - 53	36	13	14
54 – 104	40	15	14
105 – 126	36	13	14
127 – 134	33	12	14
135 – 142	30	11	14
150 – 156	20	7	14
157 - 162	23	8	14
163 - 164	20	7	14
165 - 166	17	6	14
186 - 190	17	6	14
191 – 193	26	10	14
200 - 214	13 - 20	9 - 12	8

Table 5. Summary of Waves and Water Levels Used for Levee Analysis for Alignment 5			
Save Points	Surge h_s (ft)	Wave Height H_{mo} (ft)	Wave Period T_p (s)
1 – 19	33	12	11
39 – 46	33	12	14
61 - 70	33	12	14
71 – 98, 104	40	15	14
105 - 126	36	13	14
127 – 134	33	12	14
135 - 142	30	11	14
200 - 214	13 - 20	9 - 12	8

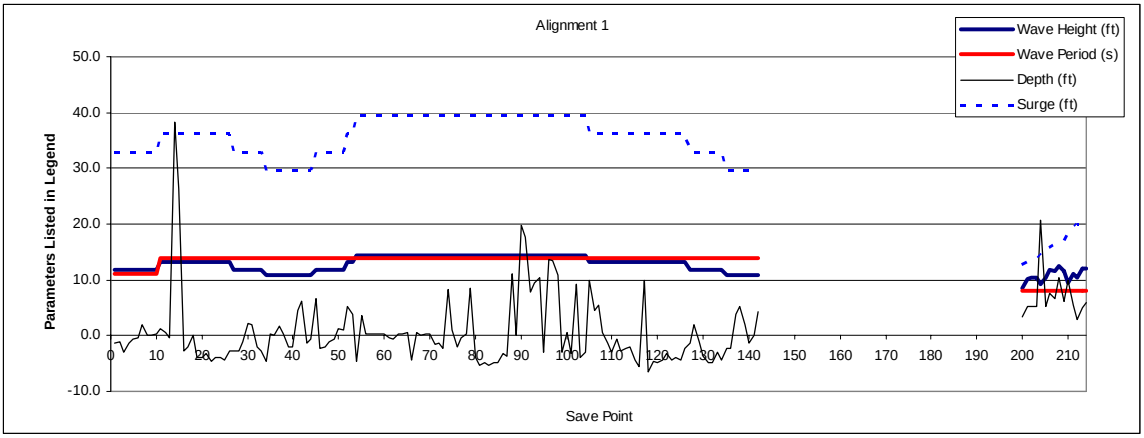


Figure 11. Alignment 1 wave and water level conditions and msl depth

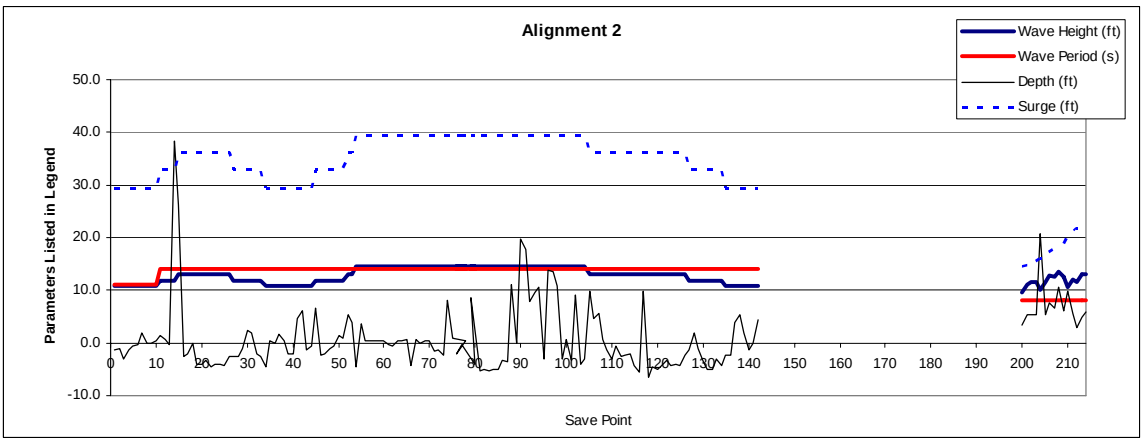


Figure 12. Alignment 2 wave and water level conditions and msl depth

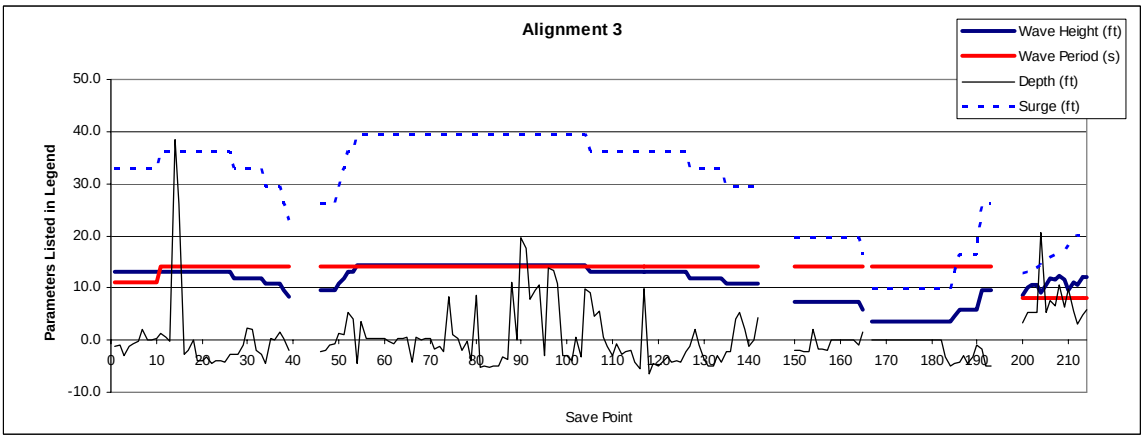


Figure 13. Alignment 3 wave and water level conditions and msl depth

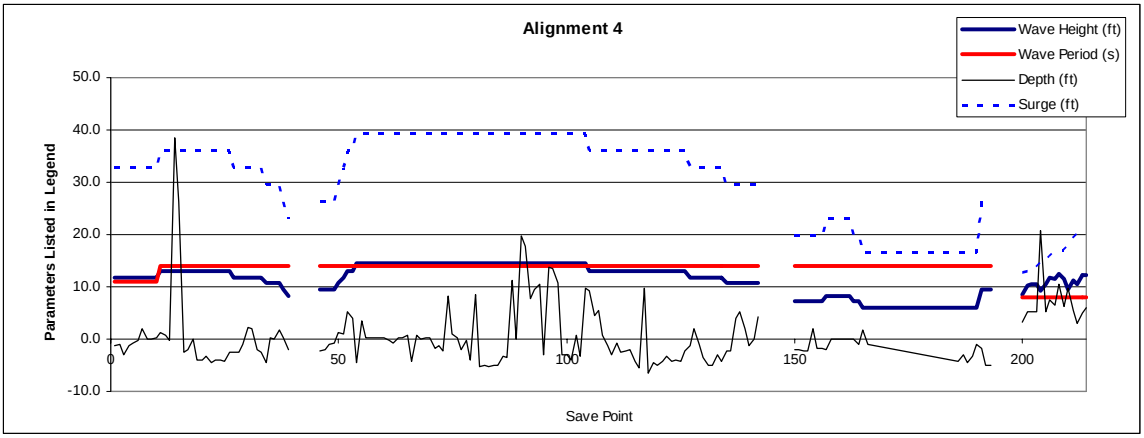


Figure 14. Alignment 4 wave and water level conditions and msl depth

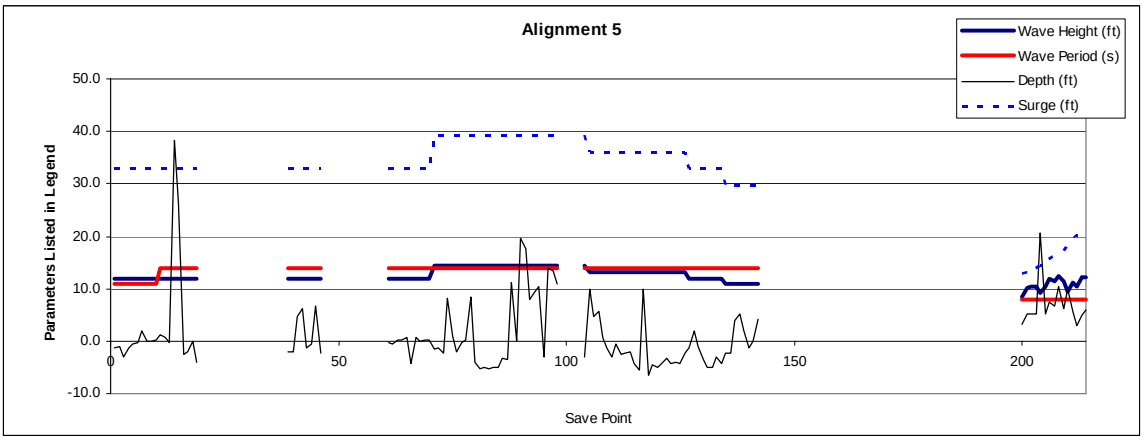


Figure 15. Alignment 5 wave and water level conditions and msl depth

For preliminary cross-sectional design of the levee system, the following parameters were calculated at each save point for each storm and each alignment:

1. Wave runup (Eq. 23)
2. Wave overtopping (Eq. 32 - 33)
3. Stone armor size (Eq. 34 - 41)
4. Stone armor layer thickness ($t = 2D_{n50}$)
5. Non-overtopping crest elevation ($h_c = h_s + R_{u2\%}$)
6. Crest elevation corresponding to varying levels of wave overtopping

Virtually all of the design guidance for levee armoring, runup, and overtopping is for uniform slopes. Some guidance exists for compound slopes with a fronting berm but there is no useful guidance for a structure with a shallower slope seaward of a steeper slope. Therefore, all computations summarized herein assume a uniform seaward slope of a levee. Further, design guidance exists for wave overtopping erosion and for mean steady flow overtopping; however there is no guidance for erosion due to waves on top of a flood over the levee crest. Therefore, all surge levels are assumed to be below the levee

crest. Also, all calculations are for rock armor. Other armoring is being evaluated for a separate study under Task Force Guardian but the results were not available in time to incorporate in this evaluation.

Figures 16 – 28 show results for Alignment 1 for 5 different structure seaward slopes. Figures 17, 19, 21, 23, and 26 include results for Alignments 3, 4, and 5 for Lake Pontchartrain because they are identical to those for Alignment 1. Figures 29 – 38 show results for Alignment 2. Figures 39 – 44 show results for Alignment 3, Figures 45 – 50 show results for Alignment 4, and Figures 51 – 56 show results for Alignment 5.

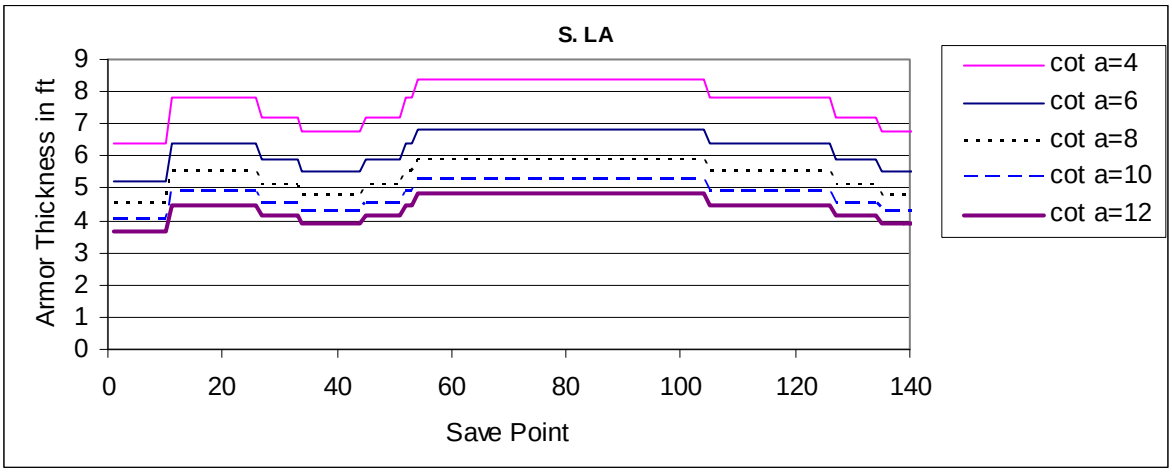


Figure 16. Armor layer thickness for Alignment 1 along South Louisiana levee system

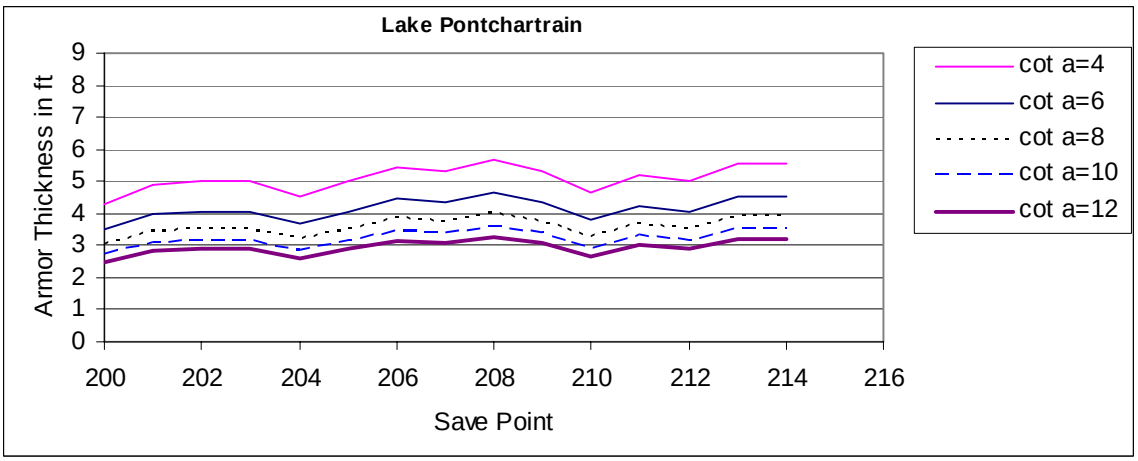


Figure 17. Armor layer thickness for Alignments 1, 3, 4, and 5 along Lake Pontchartrain south shore levee system

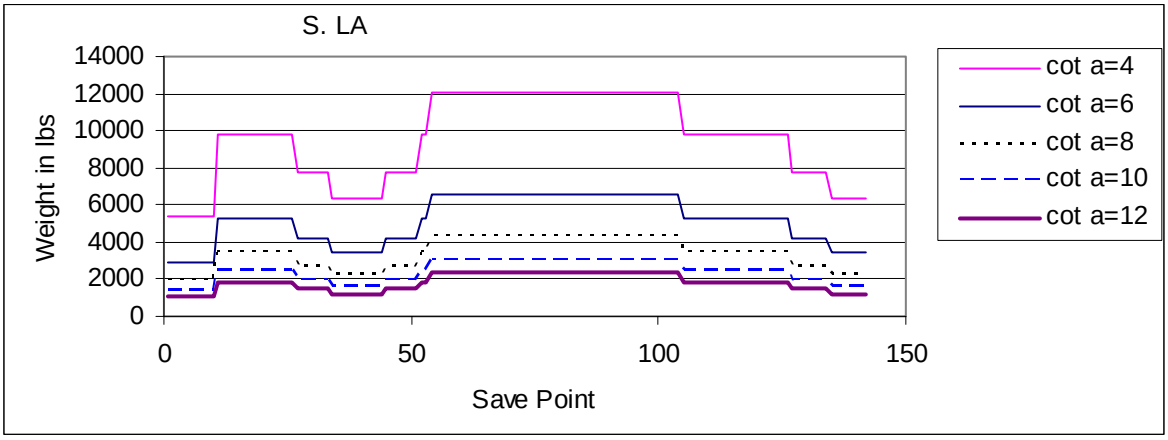


Figure 18. Armor weights for Alignment 1 along South Louisiana levee system

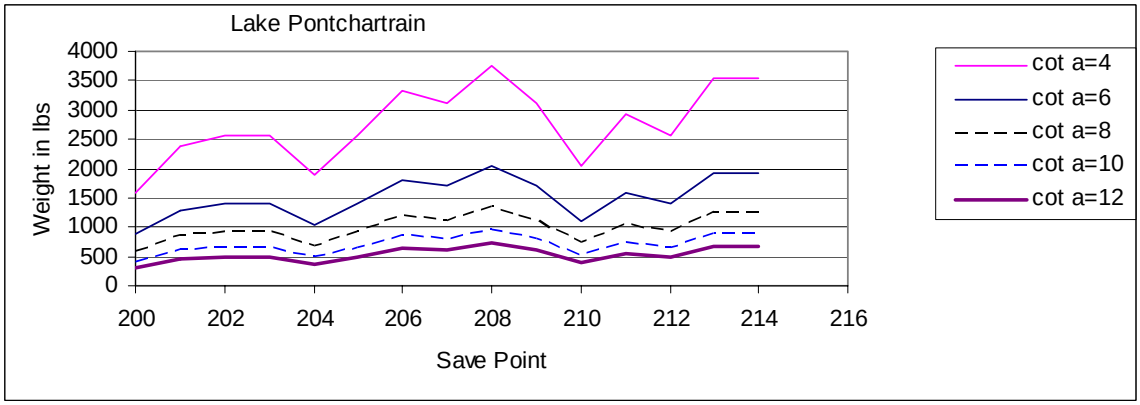


Figure 19. Armor weights for Alignments 1, 3, 4, and 5 along Lake Pontchartrain south shore levee system

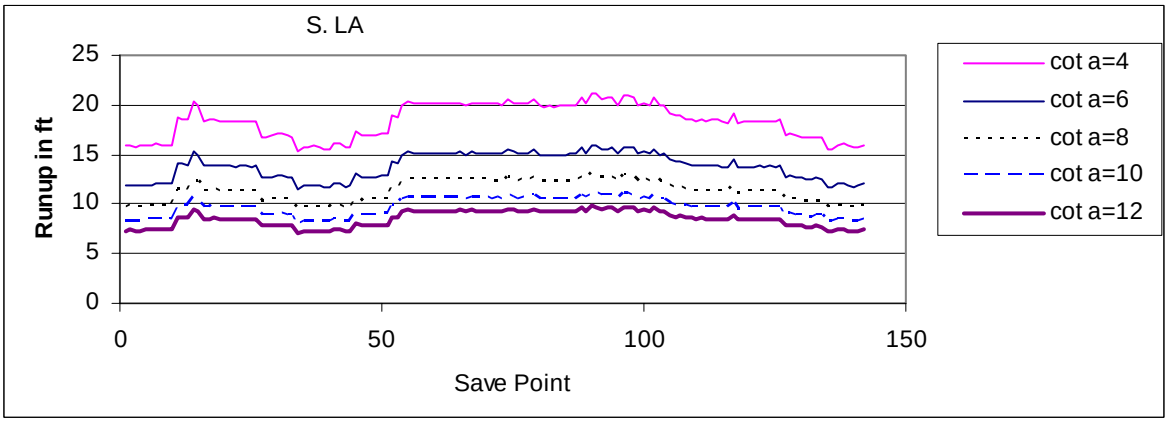


Figure 20. Runup for Alignment 1 along South Louisiana levee system

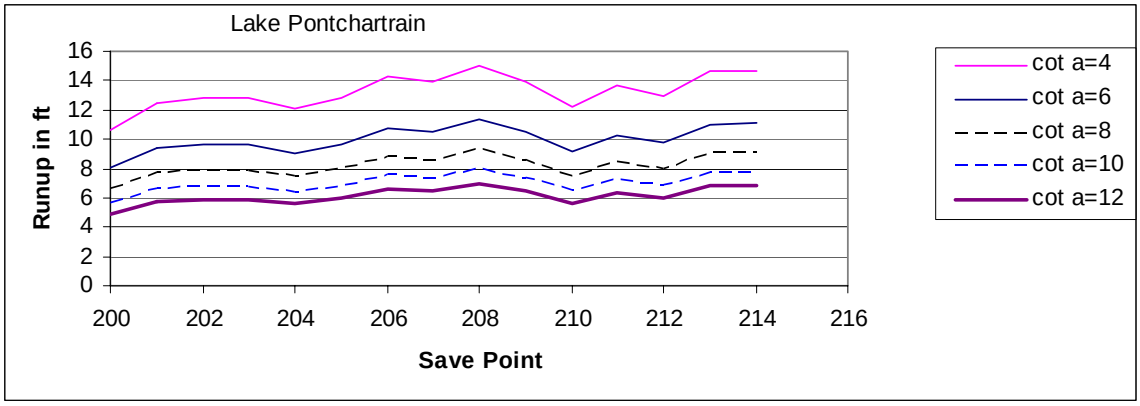


Figure 21. Runup for Alignments 1, 3, 4, and 5 along Lake Pontchartrain south shore levee system

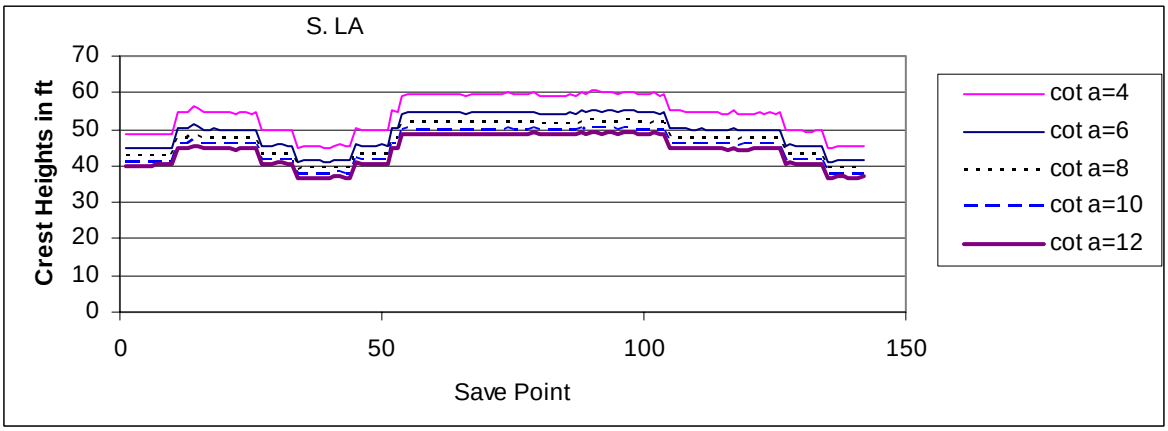


Figure 22. Non-overtopped crest height for Alignment 1 along South Louisiana levee system

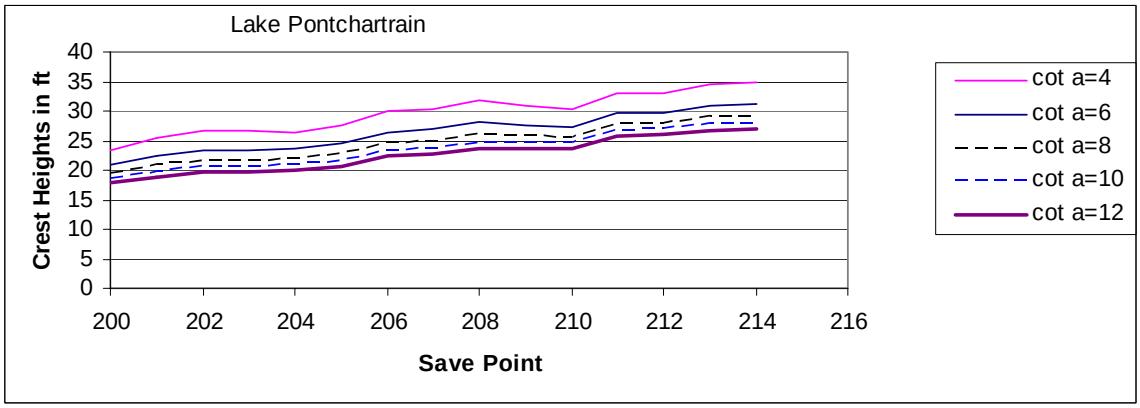


Figure 23. Non-overtopped crest height for Alignments 1, 3, 4, and 5 along Lake Pontchartrain south shore levee system

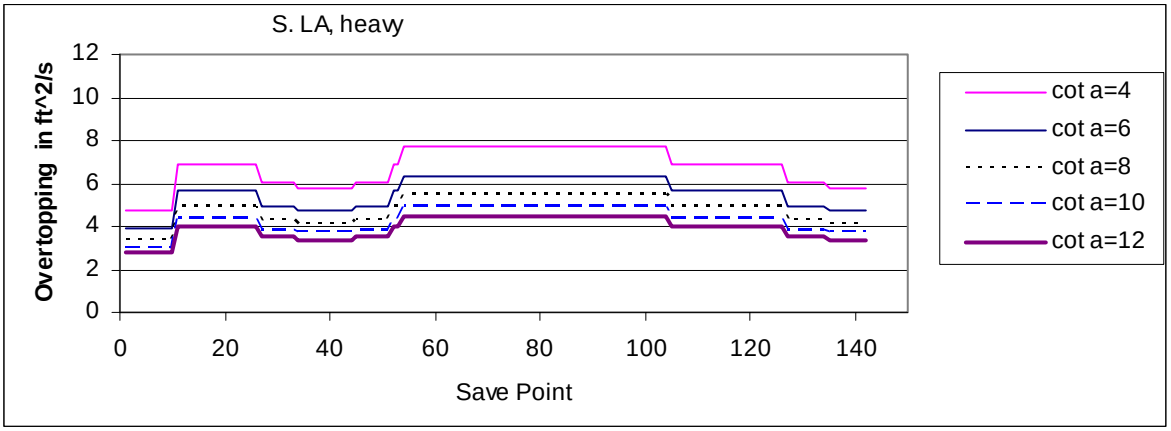


Figure 24. Wave overtopping for Alignment 1 along South Louisiana levee system for lowest crest height (32 ft – 43 ft crest height)

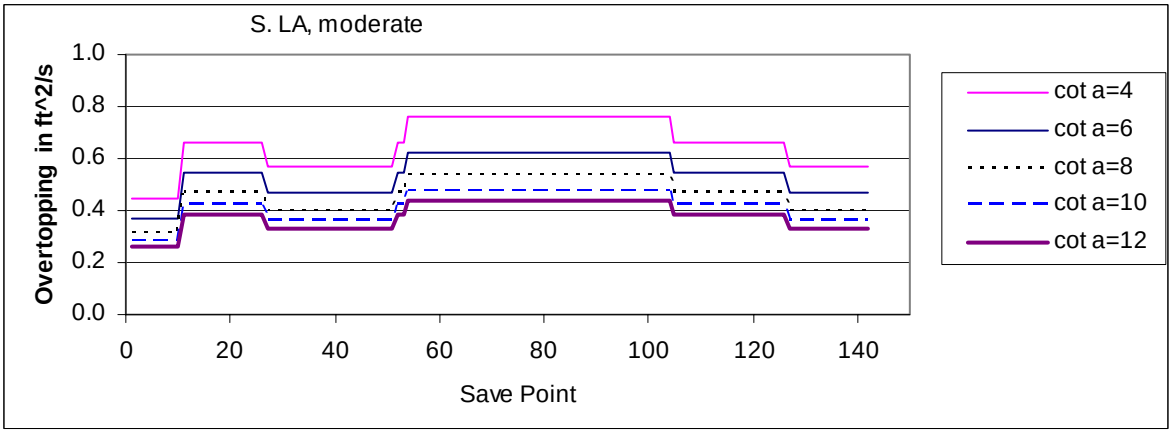


Figure 25. Wave overtopping for Alignment 1 along South Louisiana levee system for medium high crest (35ft – 47ft crest height)

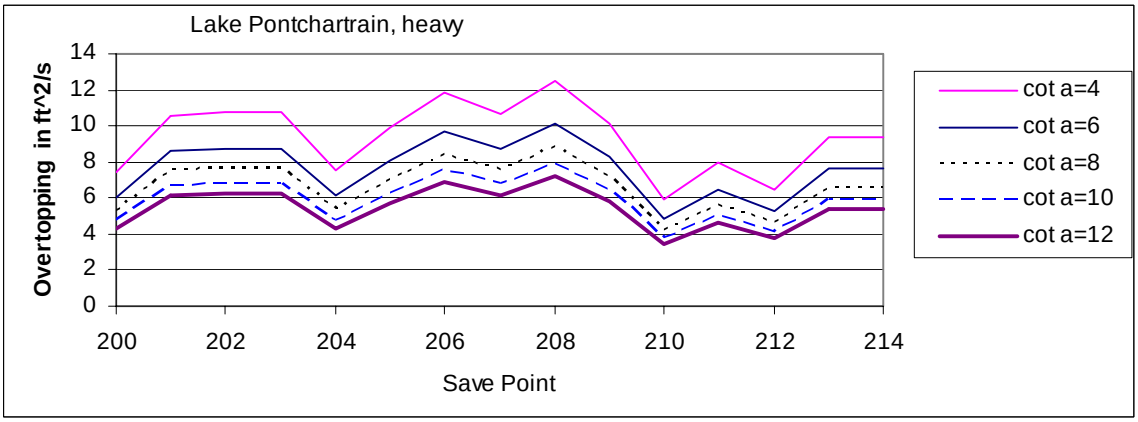


Figure 26. Wave overtopping for Alignments 1, 3, 4, and 5 along Lake Pontchartrain south shore levee system for 14 - 22 ft crest height

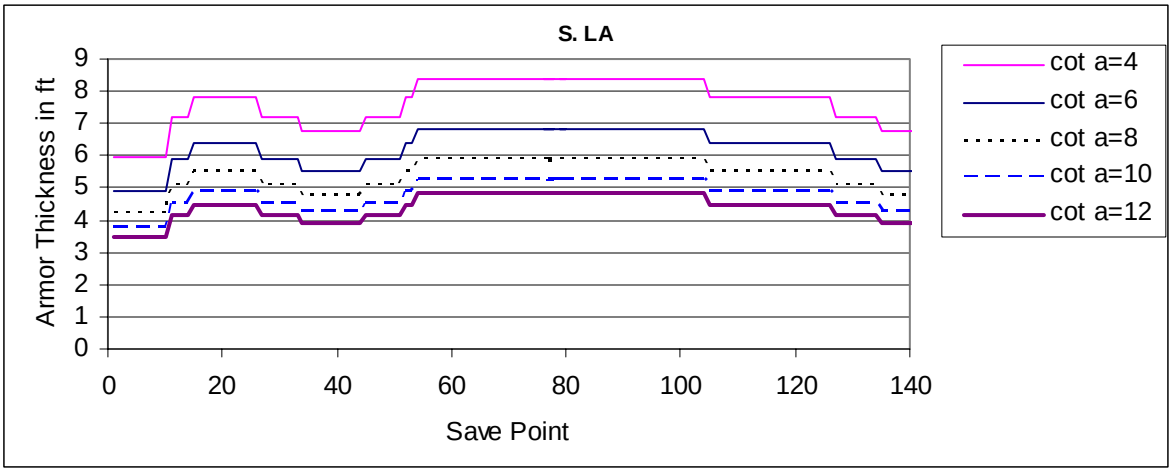


Figure 27. Armor layer thickness for Alignment 2 along South Louisiana levee system

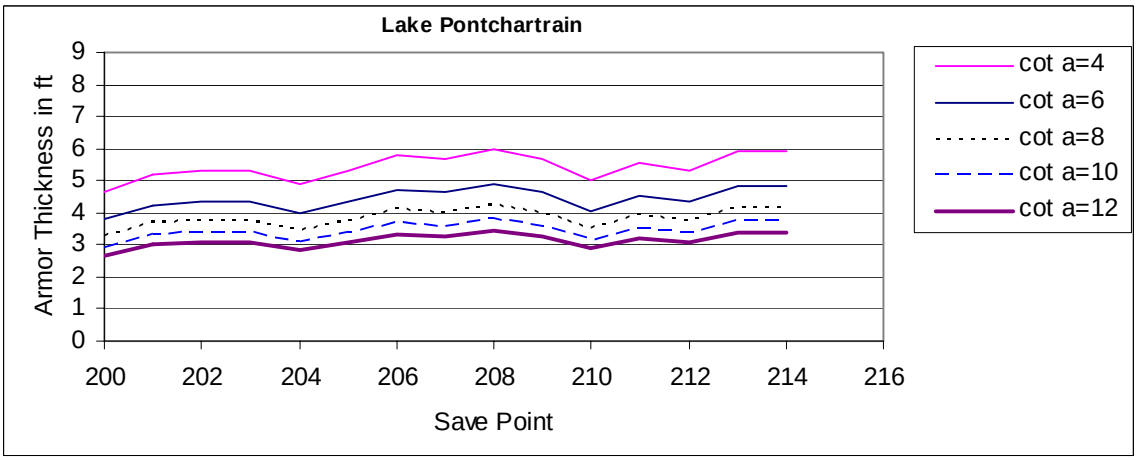


Figure 28. Armor layer thickness for Alignment 2 along Lake Pontchartrain south shore levee system

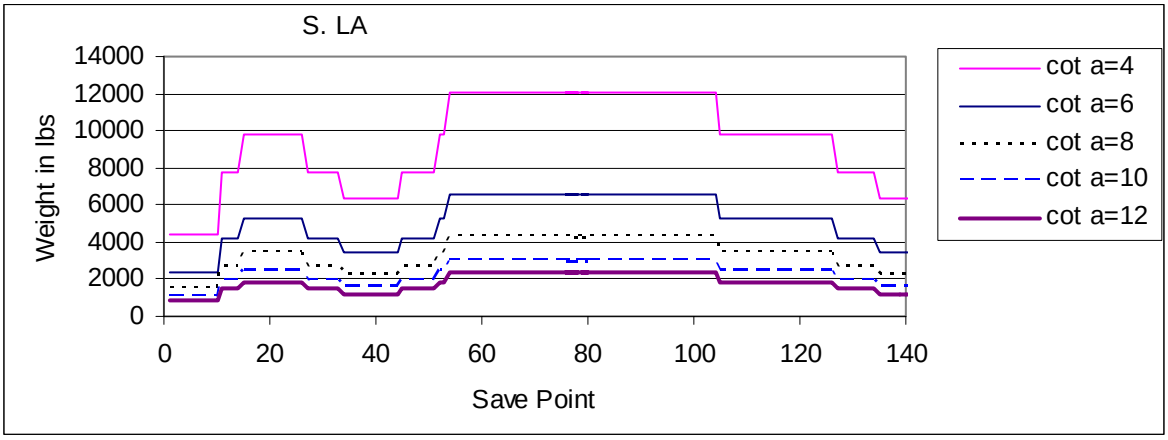


Figure 29. Armor weights for Alignment 2 along South Louisiana levee system

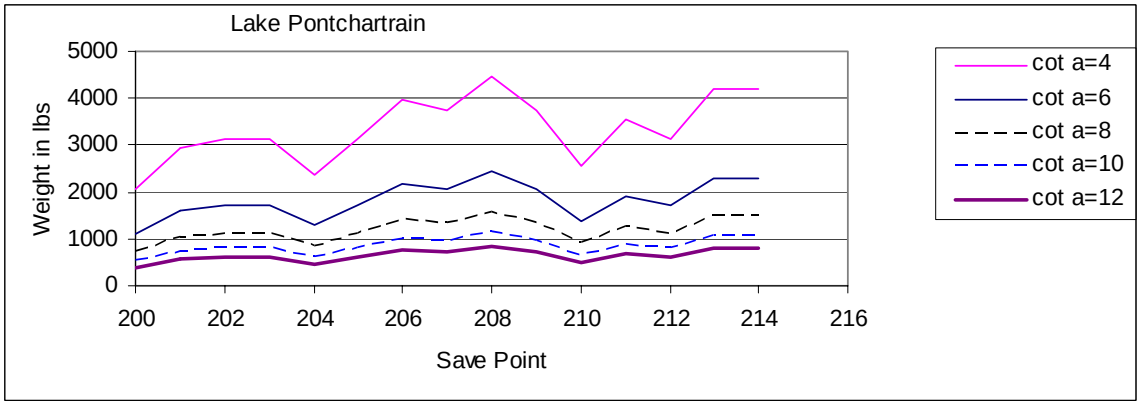


Figure 30. Armor weights for Alignment 2 along Lake Pontchartrain south shore levee system

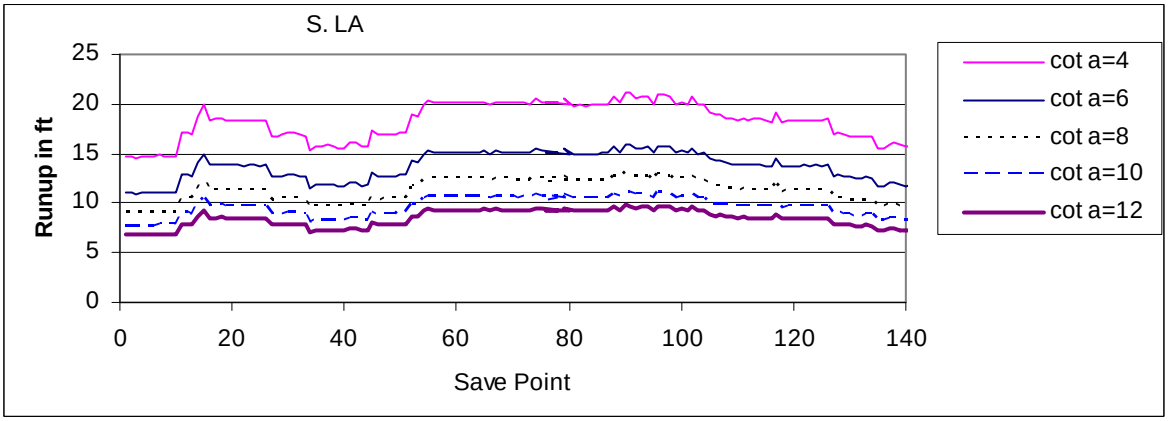


Figure 31. Runup for Alignment 2 along South Louisiana levee system

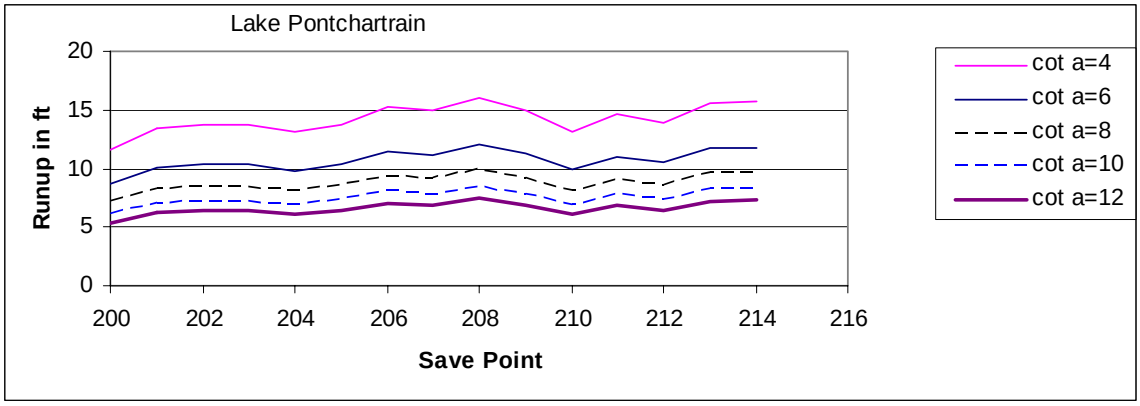


Figure 32. Runup for Alignment 2 along Lake Pontchartrain south shore levee system

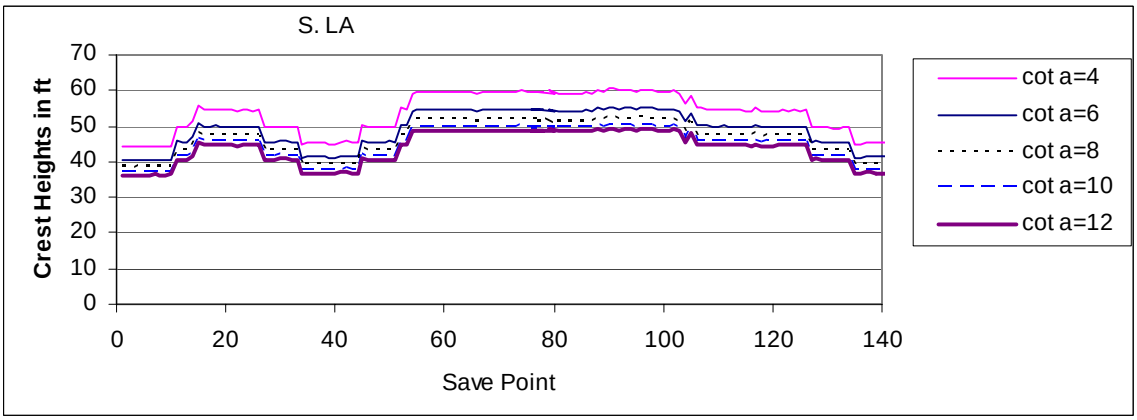


Figure 33. Non-overtopped crest height for Alignment 2 along South Louisiana levee system

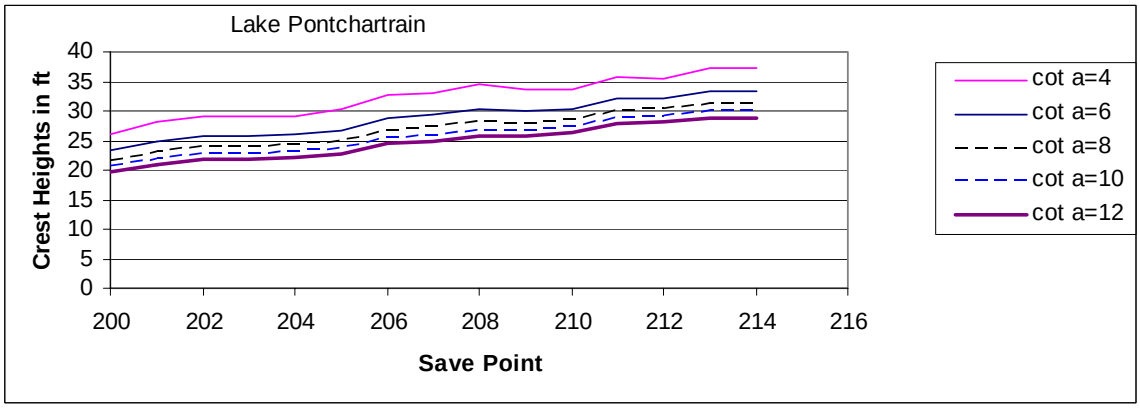


Figure 34. Non-overtopped crest height for Alignment 2 along Lake Pontchartrain south shore levee system

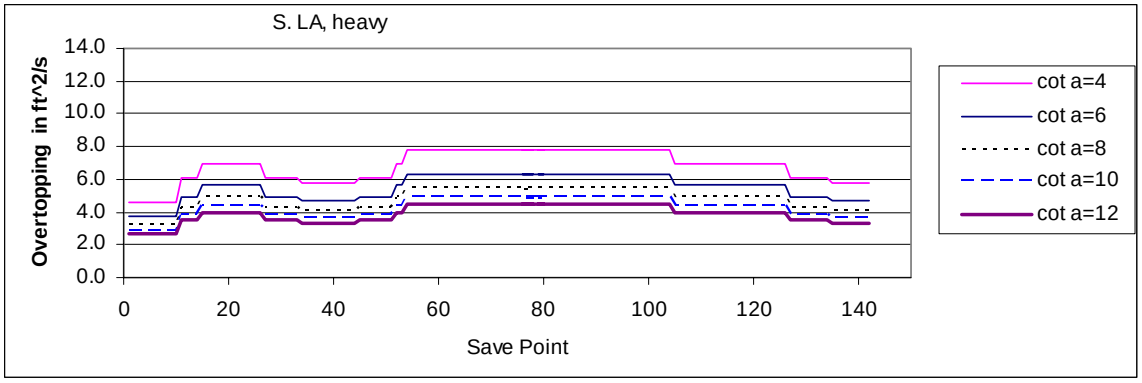


Figure 35. Wave overtopping for Alignment 2 along South Louisiana levee system for lowest crest height (32 ft to 43 ft crest height)

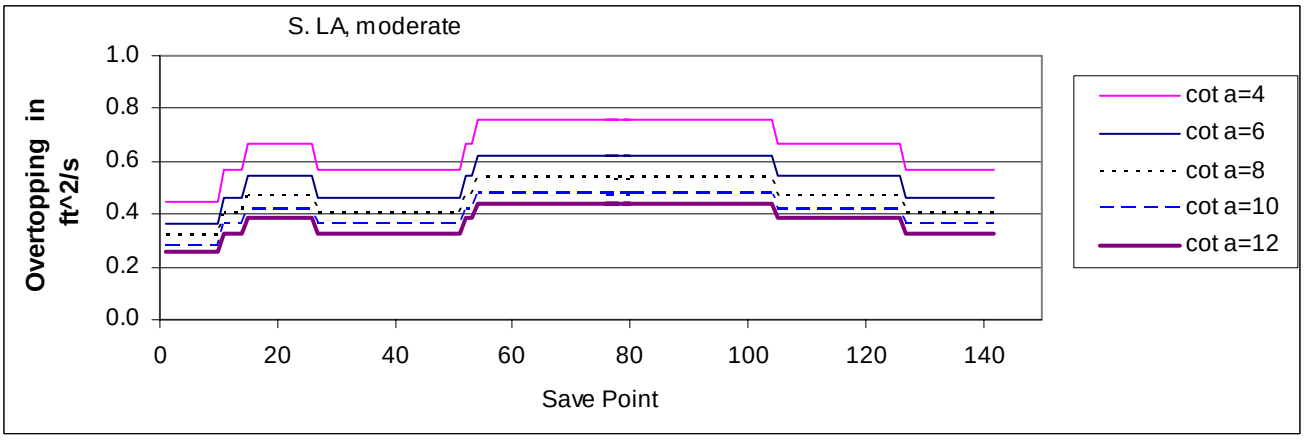


Figure 36. Wave overtopping for Alignment 2 along South Louisiana levee system for medium high crest (35 ft to 47 ft crest height)

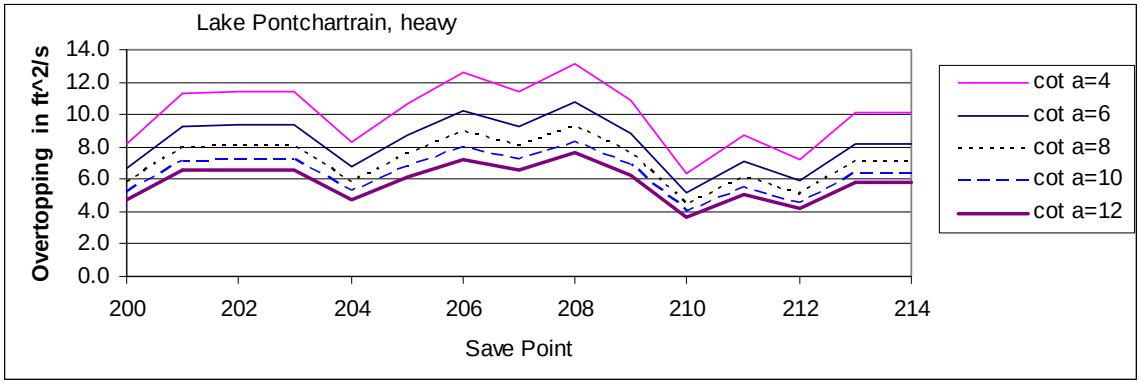


Figure 37. Wave overtopping for Alignment 2 along Lake Pontchartrain south shore levee system for lowest crest height (16 ft to 24 ft crest height)

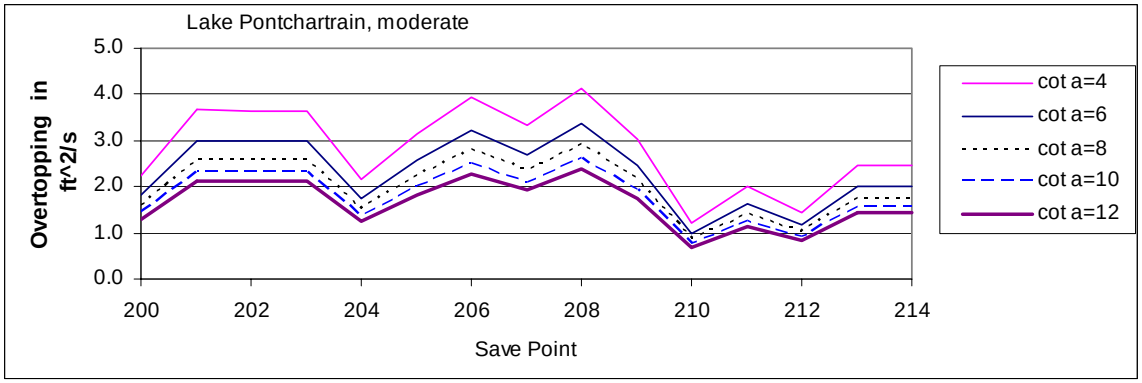


Figure 38. Wave overtopping for Alignment 2 along Lake Pontchartrain south shore levee system for medium crest height (17ft – 26 ft crest height)

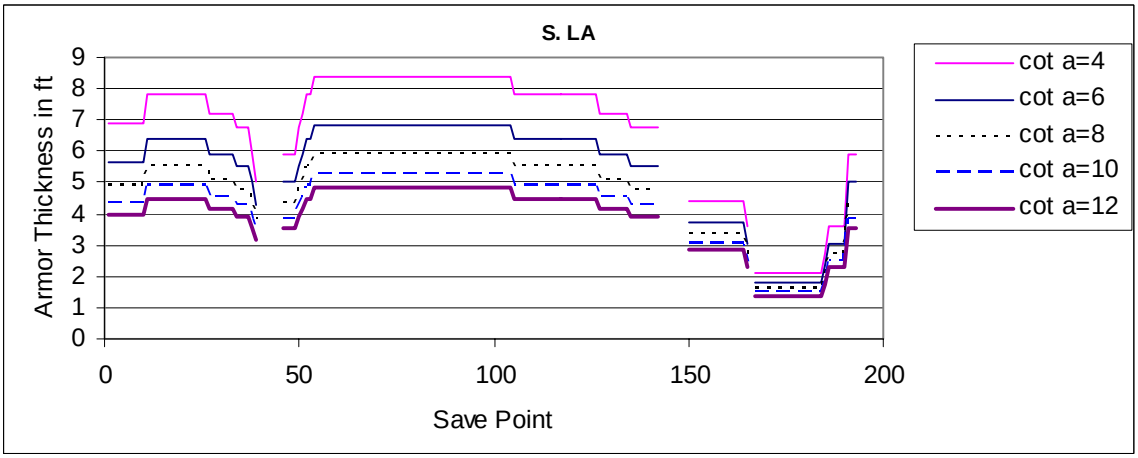


Figure 39. Armor layer thickness for Alignment 3 along South Louisiana levee system

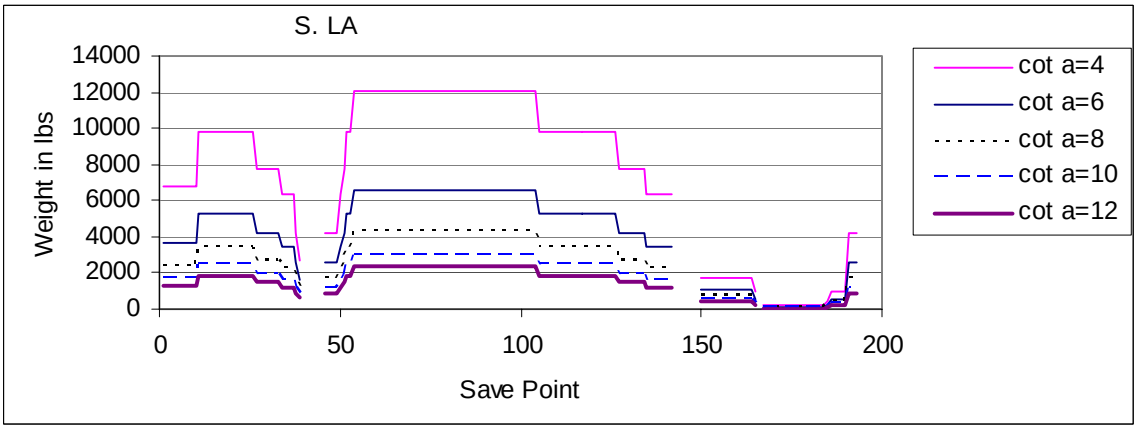


Figure 40. Armor weights for Alignment 3 along South Louisiana levee system

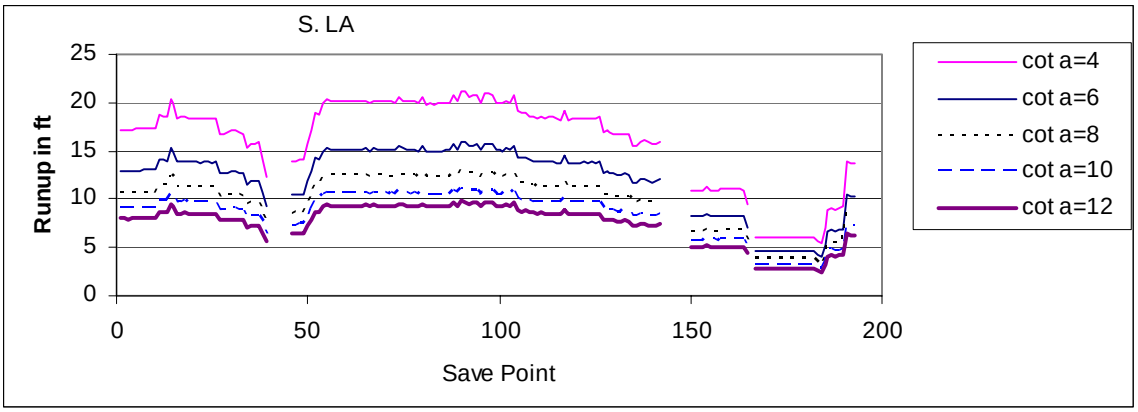


Figure 41. Runup for Alignment 3 along South Louisiana levee system

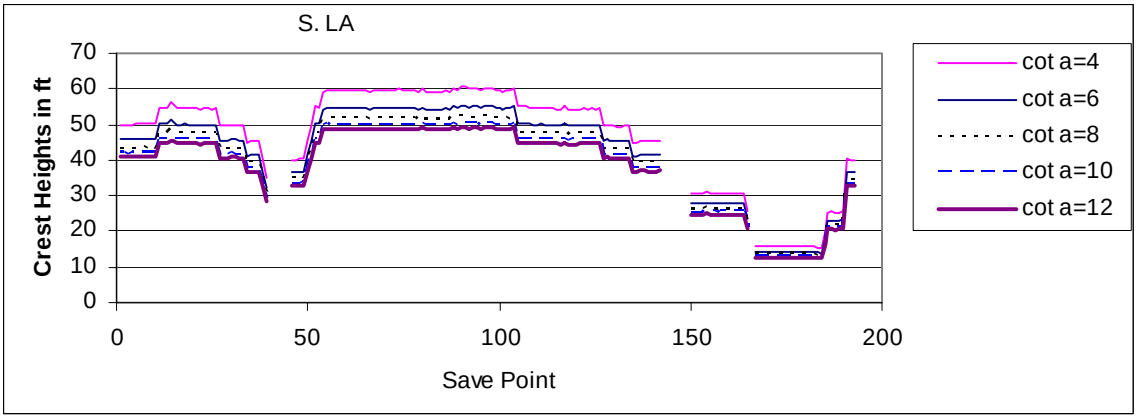


Figure 42. Non-overtopped crest height for Alignment 3 along South Louisiana levee system

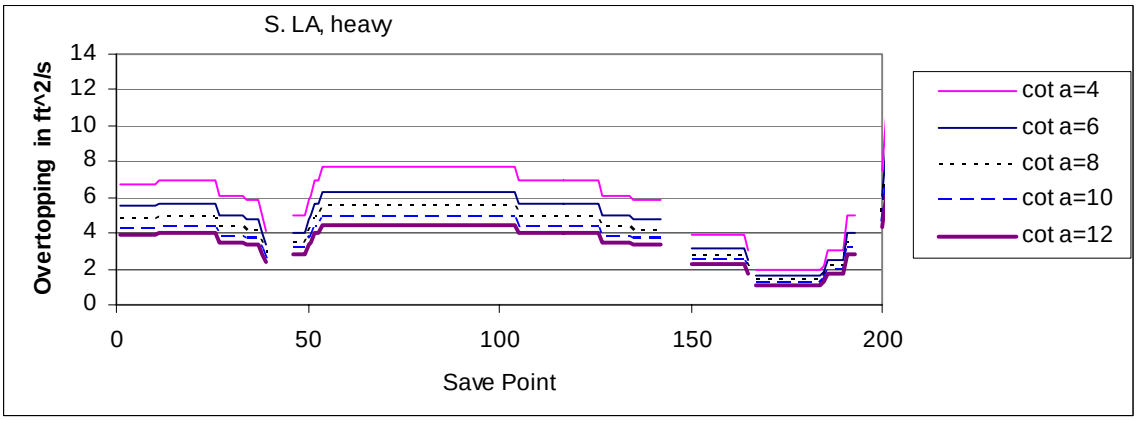


Figure 43. Wave overtopping for Alignment 3 along South Louisiana levee system for lowest crest height (11 ft to 44 ft crest height)

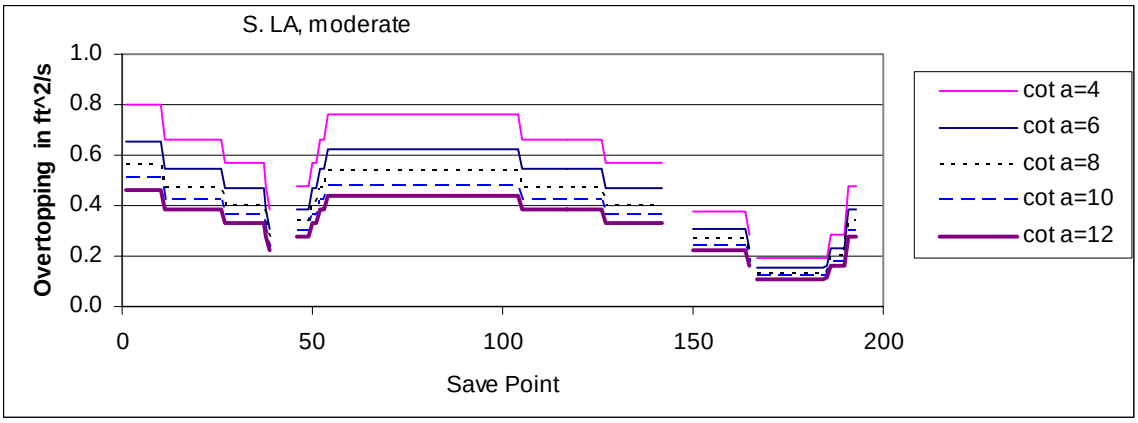


Figure 44. Wave overtopping for Alignment 3 along South Louisiana levee system for medium high crest (12 ft – 47 ft crest height)

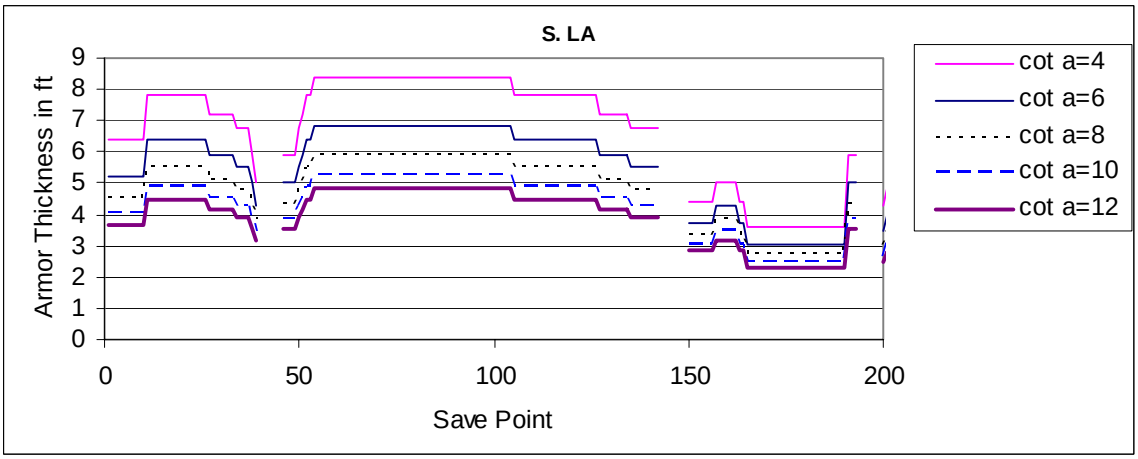


Figure 45. Armor layer thickness for Alignment 4 along South Louisiana levee system

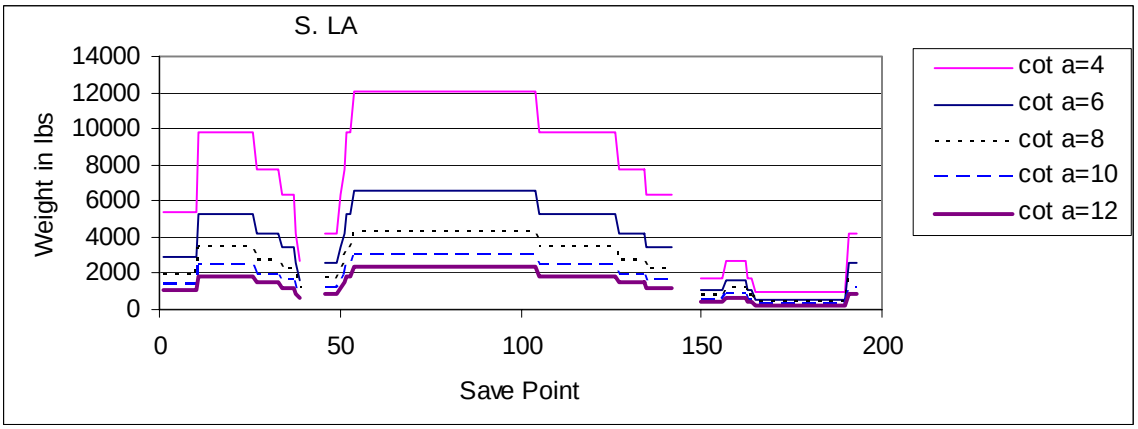


Figure 46. Armor weights for Alignment 4 along South Louisiana levee system

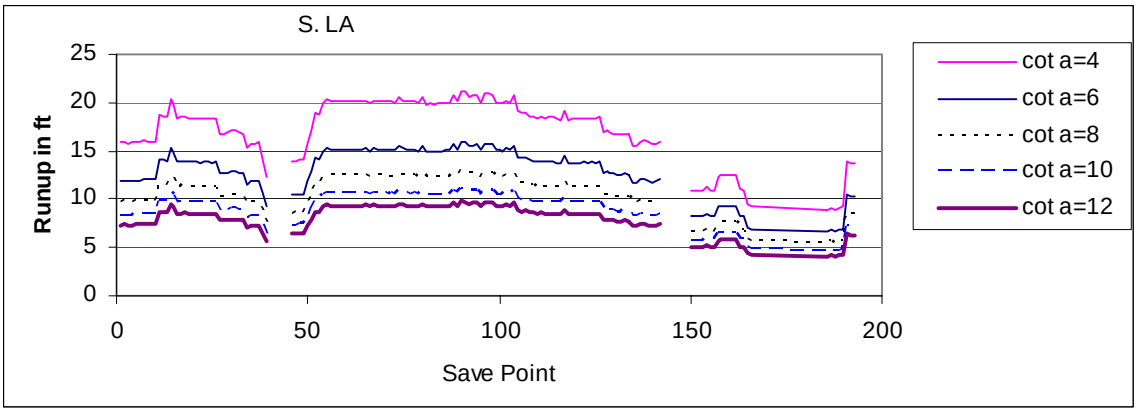


Figure 47. Runup for Alignment 4 along South Louisiana levee system

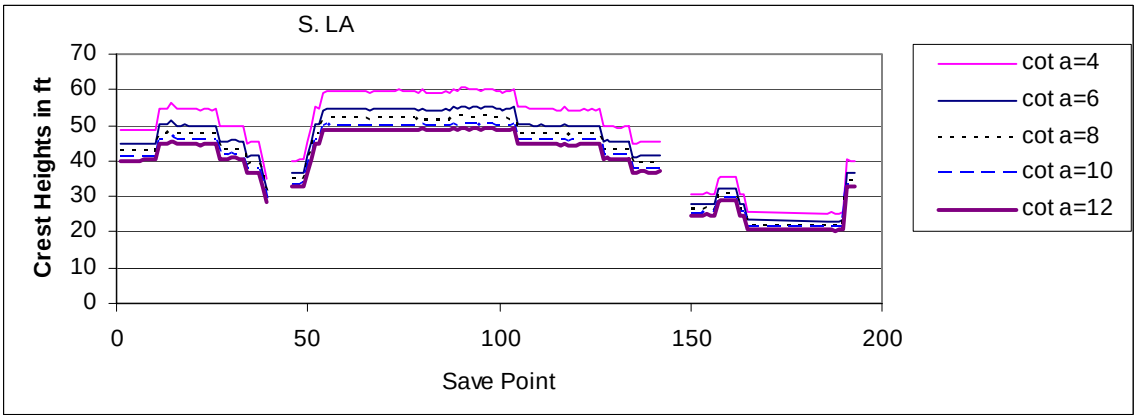


Figure 48. Non-overtopped crest height for Alignment 4 along South Louisiana levee system

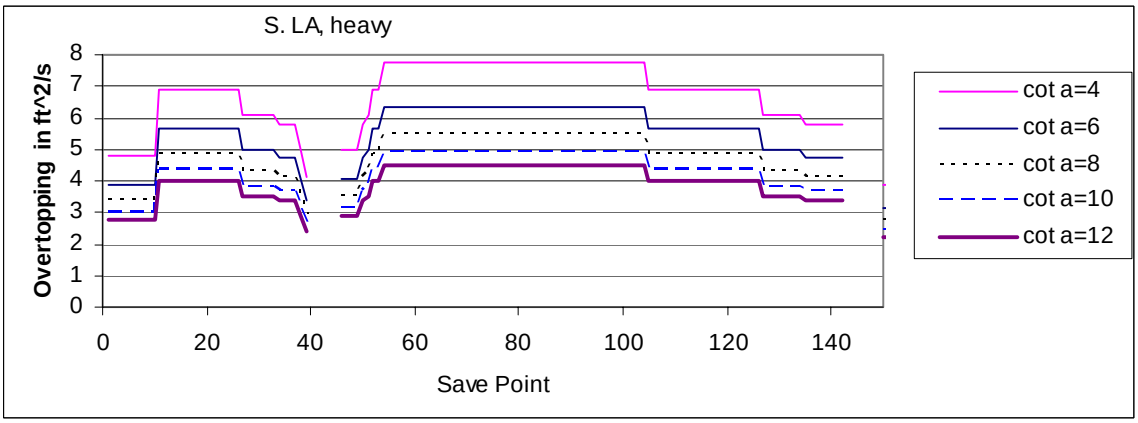


Figure 49. Wave overtopping for Alignment 4 along South Louisiana levee system for lowest crest height (18 ft – 40 ft crest height)

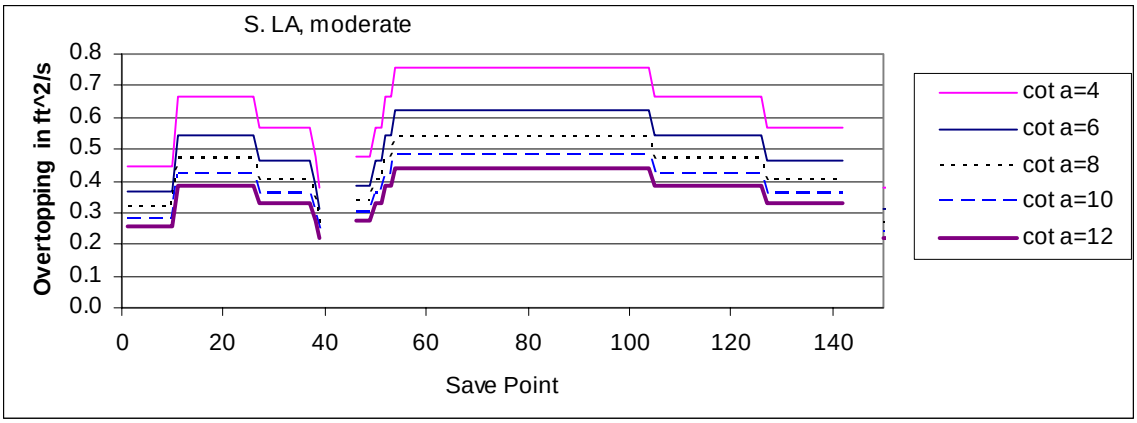


Figure 50. Wave overtopping for Alignment 4 along South Louisiana levee system for medium high crest (24 ft – 47 ft crest height)

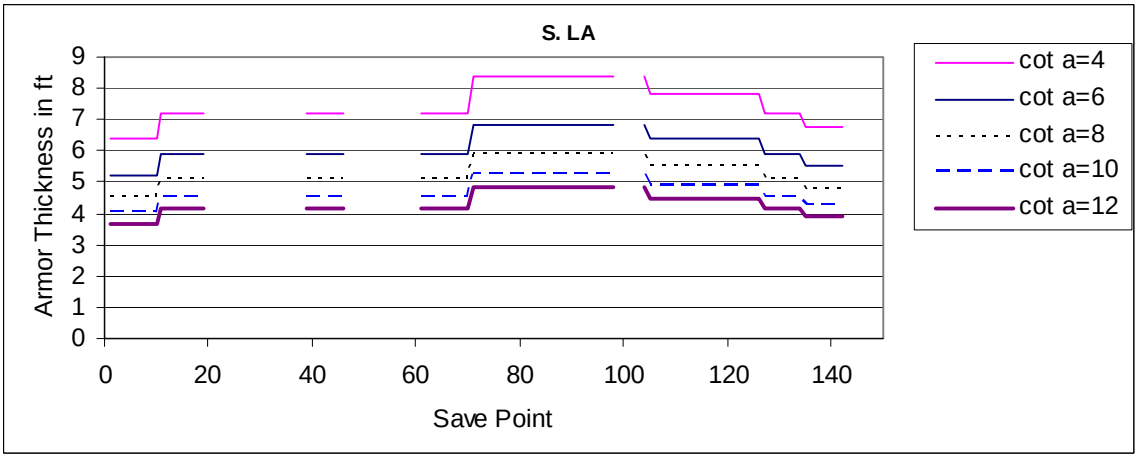


Figure 51. Armor layer thickness for Alignment 5 along South Louisiana levee system

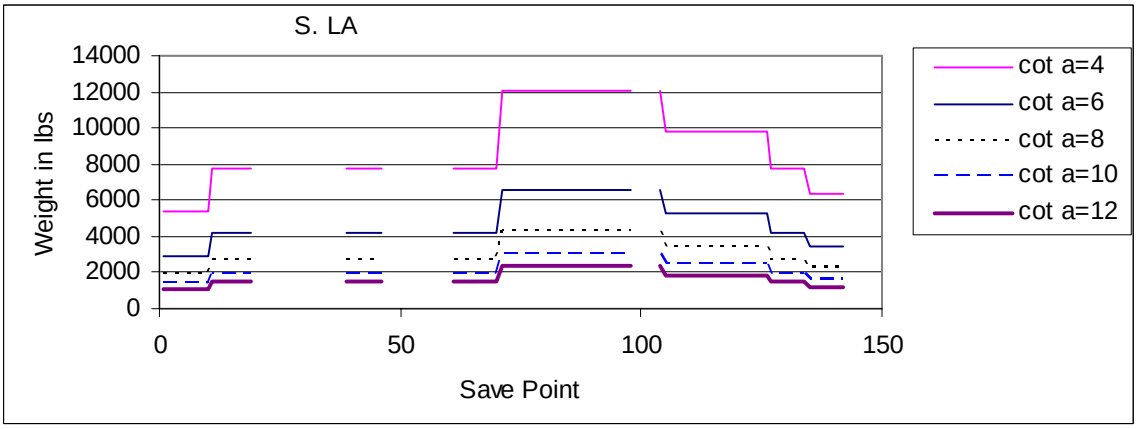


Figure 52. Armor weights for Alignment 5 along South Louisiana levee system

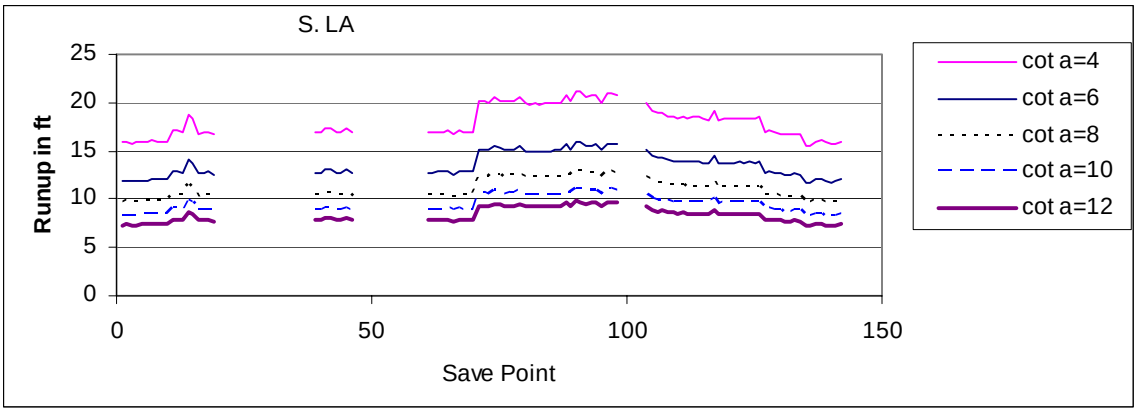


Figure 53. Runup for Alignment 5 along South Louisiana levee system

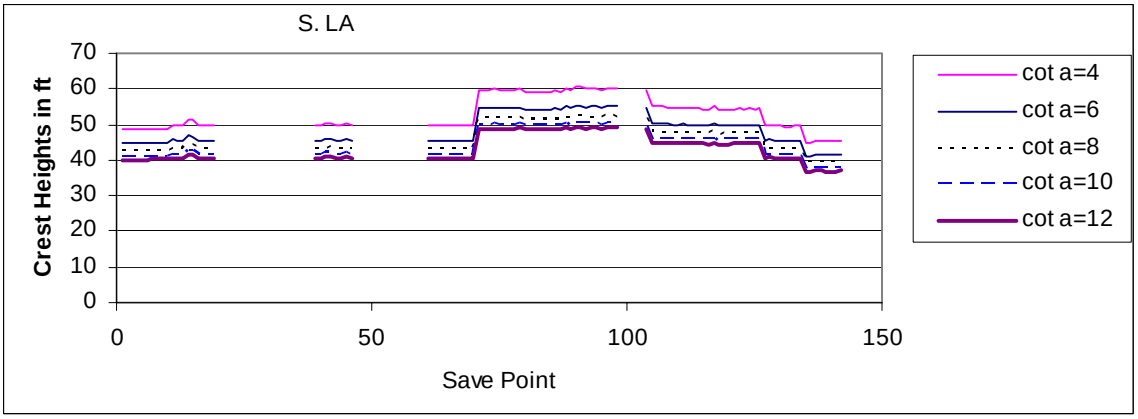


Figure 54. Non-overtopped crest height for Alignment 5 along South Louisiana levee system

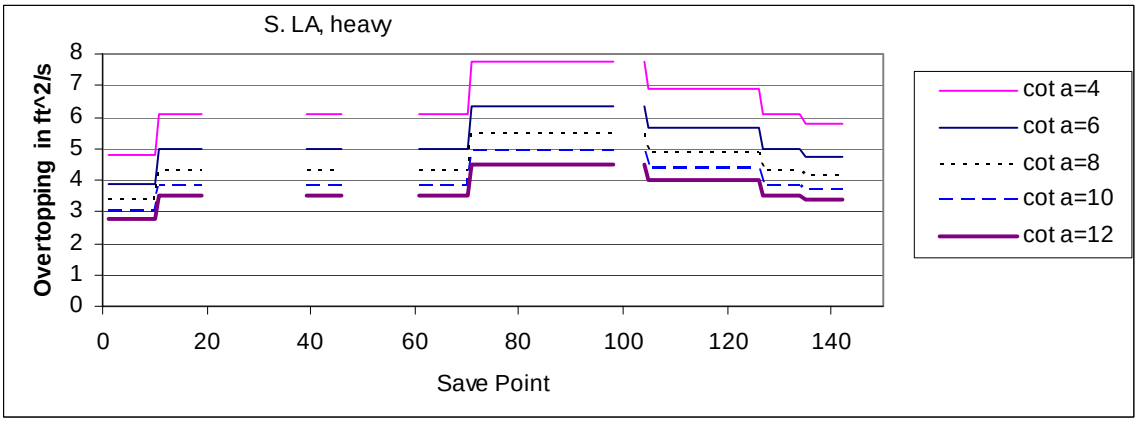


Figure 55. Wave overtopping for Alignment 5 along South Louisiana levee system for lowest crest height (32 ft – 43 ft crest height)

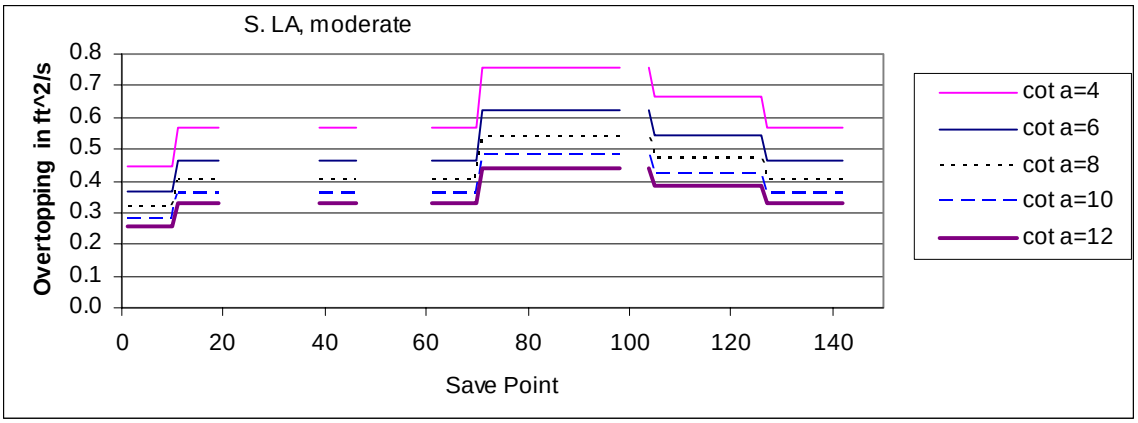


Figure 56. Wave overtopping for Alignment 5 along South Louisiana levee system for medium high crest (35ft – 47ft crest height)

There are three levels of design crest height to be considered in this initial levee design:

1. Non-overtopped ($h_c = h_s + R_{u2\%}$) grass-covered levee crest
2. Lightly to moderately overtopped, lightly armored levee crest
3. Heavily overtopped by waves but not by steady surge flow, heavily armored levee crest

The overtopping rate corresponding to erosion of a grass levee is roughly $q = 0.33 – 3.28$ $ft^3/s/ft$. Therefore, the levee crest height required with no armoring would vary as per the figures shown above. A heavily armored crest could be designed and constructed to withstand wave overtopping with a crest near the maximum flood height. The maximum flood elevation would be the sum of maximum surge, tide, and wave setup. For this simple analysis, the lowest crest height corresponding to the highest overtopping is assumed to be $h_c = 1.1h_s$ This crest height ranges from 30 to 50 ft in South Louisiana. The overtopping rate range for this lowest crest height was roughly 3 – 9 $ft^3/s/ft$. For light to moderate overtopping in the range of 0.3 – 0.9 $ft^3/s/ft$, the crest height is assumed to be $h_c = 1.2h_s$. For Lake Pontchartrain, the required crest heights are much lower than for the Gulf of Mexico shoreline, as shown in the figures. Required crest heights for several levels of protection are listed in Tables 6 through 10 for a slope of 1V:4H for Alignments 1 – 5 respectively. Table 11 gives results for a slope of 1V:8H for Alignment 1 for comparison. The resolution of the analysis for this preliminary design was not adequate to define a difference between the required crest heights of the two slopes for moderate to heavy wave overtopping. So the values listed in the right hand two columns of Tables 6 and 8 are identical. For all cases, save points were selected to yield a conservative design section for each alignment.

Table 6. Summary of Crest Heights for Seaward Structure Slope 1V:4H for Alignment 1			
Save Points	Non-Overtopped $h_c = h_s + R_{u2\%}$ (ft)	Light to Moderate Wave Overtopping $h_c/h_s = 1.2$ (ft)	Massive Wave Overtopping $h_c/h_s = 1.1$ (ft)
1 – 10	49	39	36
11 – 26	56	43	40
27 – 34	50	39	36
35 – 44	46	35	32
45 – 51	50	39	36
52 – 53	55	43	40
54 – 104	60	47	43
105 - 126	55	43	40
127 – 134	50	39	36
135 – 142	46	35	32
200 – 214	23 – 35	18 - 26	16 - 24

Table 7. Summary of Crest Heights for Seaward Structure Slope 1V:4H for Alignment 2			
Save Points	Non-Overtopped $h_c = h_s + R_{u2\%}$ (ft)	Light to Moderate Wave Overtopping $h_c/h_s = 1.2$ (ft)	Massive Wave Overtopping $h_c/h_s = 1.1$ (ft)
1 – 10	44	35	32
11 – 14	52	39	36
15 – 26	56	43	40
27 – 33	50	39	36
34 – 44	46	35	32
45 – 51	50	39	36
52 – 53	55	43	40
54 – 104	60	47	43
105 – 126	55	43	40
127 – 134	50	39	36
135 – 142	45	35	32
200 – 214	26 – 37	17 - 26	16 - 24

Table 8. Summary of Crest Heights for Seaward Structure Slope 1V:4H for Alignment 3			
Save Points	Non-Overtopped $h_c = h_s + R_{u2\%}$ (ft)	Light to Moderate Wave Overtopping $h_c/h_s = 1.2$ (ft)	Massive Wave Overtopping $h_c/h_s = 1.1$ (ft)
1 – 10	50	40	36
11 - 26	55	44	40
27 – 33	50	40	36
34 – 37	46	36	33
38	41	32	29
39	35	28	26
46 - 49	40	32	29
50	46	36	33
51	50	40	36
52 - 53	55	43	40
54 – 104	60	48	43
105 – 126	55	43	40
127 – 134	50	40	36
135 – 142	46	36	33
150 – 164	31	24	22
165	26	20	18
166 – 184	16	12	11
185	20	16	15
186 – 190	26	20	18
191 – 193	40	32	29

Table 9. Summary of Crest Heights for Seaward Structure Slope 1V:4H for Alignment 4			
Save Points	Non-Overtopped $h_c = h_s + R_{u2\%}$ (ft)	Light to Moderate Wave Overtopping $h_c/h_s = 1.2$ (ft)	Massive Wave Overtopping $h_c/h_s = 1.1$ (ft)
1 – 10	49	39	36
11 – 26	50	43	40
27 – 33	50	39	36
34 - 37	46	35	32
38	41	31	29
39	35	28	25
46 – 49	40	31	29
50	46	35	32
51	50	39	36
52 - 53	55	43	40
54 – 104	60	47	43
105 – 126	55	43	40
127 – 134	50	39	36
135 – 142	45	35	32
150 – 156	31	24	22
157 - 162	36	28	25
163 - 164	31	24	22
165 - 166	26	20	18
186 - 190	26	20	18
191 – 193	40	31	29

Table 10. Summary of Crest Heights for Seaward Structure Slope 1V:4H for Alignment 5			
Save Points	Non-Overtopped $h_c = h_s + R_{u2\%}$ (ft)	Light to Moderate Wave Overtopping $h_c/h_s = 1.2$ (ft)	Massive Wave Overtopping $h_c/h_s = 1.1$ (ft)
1 – 19	50	39	36
39 – 46	50	39	36
61 - 70	50	39	36
71 – 98, 104	60	47	43
105 - 126	55	43	40
127 – 134	50	39	36
135 - 142	46	35	32

Table 11. Summary of Crest Heights for Seaward Structure Slope 1V:8H for Alignment 1			
Save Points	Non-Overtopped $h_c = h_s + R_{u2\%}$ (ft)	Light to Moderate Wave Overtopping $h_c/h_s = 1.2$ (ft)	Massive Wave Overtopping $h_c/h_s = 1.1$ (ft)
1 – 10	43	39	36
11 – 26	48	43	40
27 – 34	43	39	36
35 – 44	39	35	32
45 – 51	44	39	36
52 – 53	48	43	40
54 – 104	52	47	43
105 - 126	48	43	40
127 – 134	43	39	36
135 – 142	39	35	32
200 – 214	19 – 29	18 - 26	16 - 24

Tables 12 and 13 list armor stone weight, under layer weight, and armor stone layer thickness along the levee system. These values were computed using Equations 35 – 37 although Equations 38 – 41 would normally be preferred. Equations 35 – 37 yielded almost identical results to Equations 38 – 41 for the range of conditions evaluated herein. Equation 34 yielded similar values although there was more variability for different slopes because of the uncertainty in the empirical coefficient K_D. For this study, $S = 4$ and $N_z = 540$ were used to size the stone. The specific weight for stone was assumed to be $\gamma_r = 165$ pcf and the permeability was $P = 0.1$.

Table 12. Summary of Median Armor Stone Weight and Armor Layer Thickness for Seaward Structure Slope 1V:4H for Alignment 1			
Save Points	Armor Weight (lb)	Underlayer Weight (lb)	Armor Layer Thickness (ft)
1 – 10	5400	540	6.4
11 – 26	9800	980	7.8
27 – 34	7700	770	7.2
35 – 44	6400	640	6.7
45 – 51	7700	770	7.2
52 – 53	9800	980	7.8
54 – 104	12100	1210	8.4
105 - 126	9800	980	7.8
127 – 134	7700	770	7.2
135 – 142	6400	640	6.7
200 – 214	1600 - 3600	160 - 360	4.3 – 5.6

Table 13. Summary of Median Armor Stone Weight and Armor Layer Thickness for Seaward Structure Slope 1V:8H for Alignment 1			
Save Points	Armor Weight (lb)	Underlayer Weight (lb)	Armor Layer Thickness (ft)
1 – 10	1900	190	4.5
11 – 26	3500	350	5.5
27 – 34	2800	280	5.1
35 – 44	2300	230	4.8
45 – 51	2800	280	5.1
52 – 53	3500	350	5.5
54 – 104	4300	430	5.9
105 - 126	3500	350	5.5
127 – 134	2800	280	5.1
135 – 142	2300	230	4.8
200 – 214	600 - 1300	60 - 130	3.0 – 3.9

Table 14. Summary of Median Armor Stone Weight and Armor Layer Thickness for Seaward Structure Slope 1V:4H for Alignment 2			
Save Points	Armor Weight (lb)	Underlayer Weight (lb)	Armor Layer Thickness (ft)
1 – 10	4400	440	6.0
11 – 14	7700	770	7.2
15 – 26	9800	980	7.8
27 – 33	7700	770	7.2
34 – 44	6400	640	6.7
45 – 51	7700	770	7.2
52 – 53	9800	980	7.8
54 – 104	12100	121	8.4
105 – 126	9800	980	7.8
127 – 134	7700	770	7.2
135 – 142	6400	640	6.7
200 – 214	2045 - 4217	205 - 422	4.6 – 5.9

Table 15. Summary of Median Armor Stone Weight and Armor Layer Thickness for Seaward Structure Slope 1V:4H for Alignment 3			
Save Points	Armor Weight (lb)	Underlayer Weight (lb)	Armor Layer Thickness (ft)
1 – 10	6800	680	6.9
11 - 26	9800	980	7.8
27 – 33	7700	770	7.2
34 – 37	6400	640	6.7
38	4300	430	5.9
39	2700	270	5.0
46 - 49	4300	430	5.9
50	6400	640	6.7
51	7700	770	7.2
52 - 53	9800	980	7.8
54 – 104	12100	1210	8.4
105 – 126	9800	980	7.8
127 – 134	7700	770	7.2
135 – 142	6330	630	6.7
150 – 164	1800	180	4.4
165	1000	100	3.6
166 – 184	200	20	2.1
185	500	50	2.7
186 – 190	1000	100	3.6
191 – 193	4300	430	5.9

Table 16. Summary of Median Armor Stone Weight and Armor Layer Thickness for Seaward Structure Slope 1V:4H for Alignment 4			
Save Points	Armor Weight (lb)	Underlayer Weight (lb)	Armor Layer Thickness (ft)
1 – 10	5400	540	6.4
11 – 26	9800	980	7.8
27 – 33	7700	770	7.2
34 - 37	6400	640	6.7
38	4200	420	5.9
39	2700	270	4.3
46 – 49	4200	420	5.9
50	6400	640	6.7
51	7700	770	7.2
52 - 53	9800	980	7.8
54 – 104	12100	1210	8.4
105 – 126	9800	980	7.8
127 – 134	7700	770	7.2
135 – 142	6400	640	6.7
150 – 156	1800	180	4.4
157 - 162	2700	270	5.0
163 - 164	1800	180	4.4
165 - 166	1000	100	3.6
186 - 190	1000	100	3.6
191 – 193	4200	420	5.9

Table 17. Summary of Median Armor Stone Weight and Armor Layer Thickness for Seaward Structure Slope 1V:4H for Alignment 5			
Save Points	Armor Weight (lb)	Underlayer Weight (lb)	Armor Layer Thickness (ft)
1 – 19	5400	540	6.4
39 – 46	7700	770	7.2
61 - 70	7700	770	7.2
71 – 98, 104	12100	1210	8.4
105 - 126	9800	980	7.8
127 – 134	7700	770	7.2
135 - 142	6400	640	6.7

ANNEX 5 – WETLANDS, LANDSCAPE FEATURES AND STORM SURGE

APPENDIX L – ENGINEERING INVESTIGATIONS
ANNEX 5 - Wetlands, Landscape Features, and Storm Surge
A Review of Studies to Date in Louisiana

Introduction

As hurricanes and extratropical storms approach the coast, four storm-related phenomena can occur to modify local water levels: setup due to wind, low barometric pressure, set up due to wave forcing, and rainfall (Harris 1963). Storm winds force water towards the coast and typically create the greatest change in local water elevation. During hurricanes, winds create a positive storm surge on the right side of hurricanes in the Northern Hemisphere and negative surge on the left (Figure 1). Low barometric pressure provides a secondary effect, creating a bell-shaped bulge in the water surface that is symmetrical around the center of the storm. Wave forcing also creates a local setup on the coast, with highest waves on the right side of a hurricane in the Northern Hemisphere. A lower magnitude of wave setup may also occur on the left side of the storm, depending on the path, speed, and strength of the storm. Rapid storm rainfall can also increase the local water elevation. A fifth factor not related to the storm itself is the astronomical tide at the time the storm reaches the coast; a spring (high) tide occurring at the time of the storm will result in greater storm inundation than if the storm made landfall during a neap (low) tide.

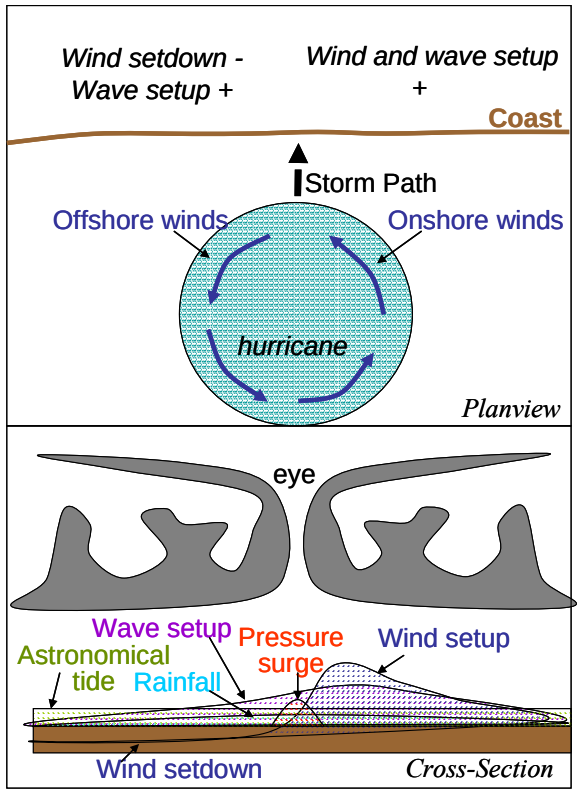


Figure 1. Storm surge components for hurricane landfall in Northern Hemisphere (relative magnitudes of surge are hypothetical)

Bathymetry and topography also modify the storm surge. A mildly sloping continental shelf, such as in the Gulf of Mexico, results in a higher storm surge as compared to a coast with a steeper bathymetry. The reason for this is, in deeper water, the surge can disperse downwards, whereas in shallow water, it cannot and is pushed inland by wind stresses. However, a milder slope reduces wave height as waves dissipate further offshore as compared to the steeper bathymetry.

Topography, landscape features, and vegetation also have the potential to reduce storm surge elevations. Land elevations greater than the storm surge elevation provide a physical barrier to the surge. Landscape features (e.g., ridges and barrier islands) and vegetation (e.g., maritime forests and wetlands) are typically below the surge elevation,

but they have the potential to create friction and slow the forward speed of the storm surge. The surge then has time to dissipate offshore and alongshore, reducing inland surge elevations.

The purpose of this literature review is to document studies that have measured storm surge elevations with the goal of understanding how landscape features and vegetation modify the surge elevation. Numerical modeling studies of this phenomenon are also reviewed. As illustrated in Figure 1, many factors control the elevation of the surge. To best characterize the influence of landscape features and vegetation on storm surge, ideal measurements are those that are (1) in line with the path of the storm, (2) on the same side of the storm, (3) not so far apart that processes (e.g., barometric pressure, winds, rainfall) are significantly different, (4) inside an enclosed space, to remove the influence of wave height on the measurements, and (5) representative of a homogeneous landscape feature (Figure 2). Information from the literature review is culled and near-ideal measurements and studies are identified to isolate the influence of these landscape features on storm surge elevations.

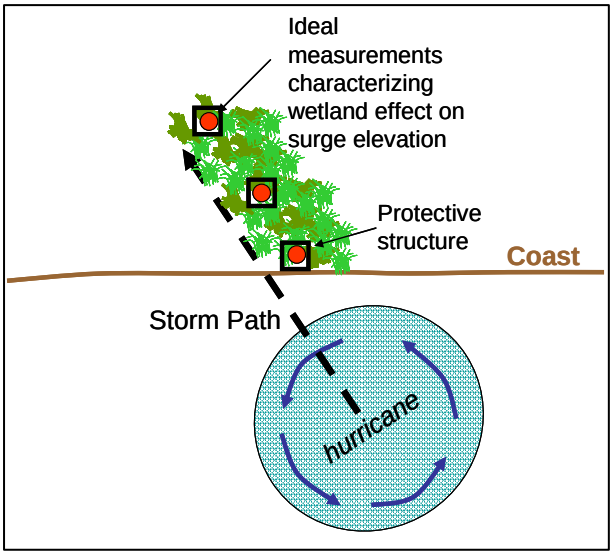


Figure 2. Ideal measurements for isolating the influence of landscape features on storm surge elevations

History of Storms Impacting the Louisiana Coast

The historical record of hurricanes and tropical storms brushing (within 96.6 km (60 miles) or making landfall at five cities on the Louisiana Gulf coast is shown in Table 1. This record of more than 130 years indicates that the Louisiana Gulf coast has experienced a tropical storm or hurricane approximately every 3 to 5 years, with a direct hurricane hit occurring every 8 to 17 years. The frequency of storms brushing the Louisiana coast is roughly equal for all locations, whereas hurricane landfall is more likely to occur in the central and eastern part of the coast. Table 2 shows the storm path, intensity, and identifies the landfall location for most storms listed in Table 1.

In an initial review of literature, maximum storm surge elevations due to the 1947 Hurricane (Sanders 1947), Hurricanes Betsy (U.S. Army Engineer District (USAED) New Orleans 1965, and USAED Mobile 1967), Camille (USAED Mobile 1970), Georges (USAE South Atlantic Division 1999), Ivan (USAED Mobile 2004), and Katrina (NOAA 2005) have been detailed for the Louisiana coast. These data are listed in Table A1, and are being incorporated into a Geographic Information System (GIS) of landscape features

Table 1. Storms within 96.6 km (60 mi) of Louisiana Cities 1872 through 2005*			
Location	**Storms t=tropical storm; b=brush; h=hurricane; d="backdoor" (coming over land from opposite coast)**	**Frequency of Occurrence (yr)**	
		Brush or hit	**Direct Hit**
Cameron	1879b, 1882t, 1886h, 1886t, 1888b, 1897h, 1898-2tb, 1905-2tb, 1918h, 1920b, 1932-37tb, 1938t, 1940h, 1940t, 1941b, 1943t, 1946t, 1957h, 1957t, 1971h, 1974bdt, 1978h, 1979t, 1982t, 1985-2b, 1986-89-92b, 2004tb, 2005h	4	17
Intracoastal City	1879b, 1888h, 1893h, 1897h, 1898t, 1905t, 1912t, 1914t, 1920h, 1923b, 1926b, 1931t, 1932-2t, 1934t, 1938t, 1959t, 1964h, 1965b, 1971b,1974h, 1977tb, 1985h, 1985t, 1992h, 2002h, 2005b	5	15
Morgan City	1877b, 1879, 1887b, 1888h, 1892tb, 1893h, 1897h, 1898t, 1905t, 1905tb, 1909h, 1912t, 1915b, 1920h, 1923h, 1926h, 1931t, 1932-2t, 1934h, 1936tb,1937t, 1939t, 1944t, 1946tb, 1947bd, 1948b, 1949tb, 1954b, 1956-57tb, 1959t,1964h, 1965h, 1971b, 1974, 1977t, 1979h, 1985b, 1985h, 1992h, 1998tb, 2002tb, 2002b, 2003t, 2004t	3	9
Grand Isle	1872t, 1875b, 1877h, 1879t, 1879b, 1885-3t, 1887h, 1889h, 1892t, 1893h, 1895t, 1897h, 1900t, 1901h, 1904tb, 1905tb, 1907t, 1909h, 1914bdt, 1915h, 1920h, 1934b, 1936t, 1939t, 1944t, 1946tb, 1947h, 1948h, 1949t, 1955tb, 1956, 1956tb, 1957tb, 1960b, 1965h, 1969h, 1971t, 1974b, 1979, 1985b, 1988, 1992b, 1997h, 1998t, 2002tb, 2002t, 2004tb, 2005-2h	3	8
New Orleans	1879h, 1879t, 1887h, 1888b, 1892t, 1893h, 1897b, 1900tb, 1901h, 1905t, 1907t, 1909h, 1914bdt, 1915h, 1916b, 1932t, 1934tb, 1936t, 1944tb, 1947h, 1948h, 1949t, 1955t, 1964bdt, 1965h, 1969b, 1979h, 1985b, 1988t, 1992b, 1998t, 2002-2 t, 2004tb, 2005t, 2005h	4	12
* http://www.hurricanecity.com/			

and vegetation for the Mississippi and Louisiana coasts. More storm surge elevation data may be available and will be incorporated into the GIS database as acquired.

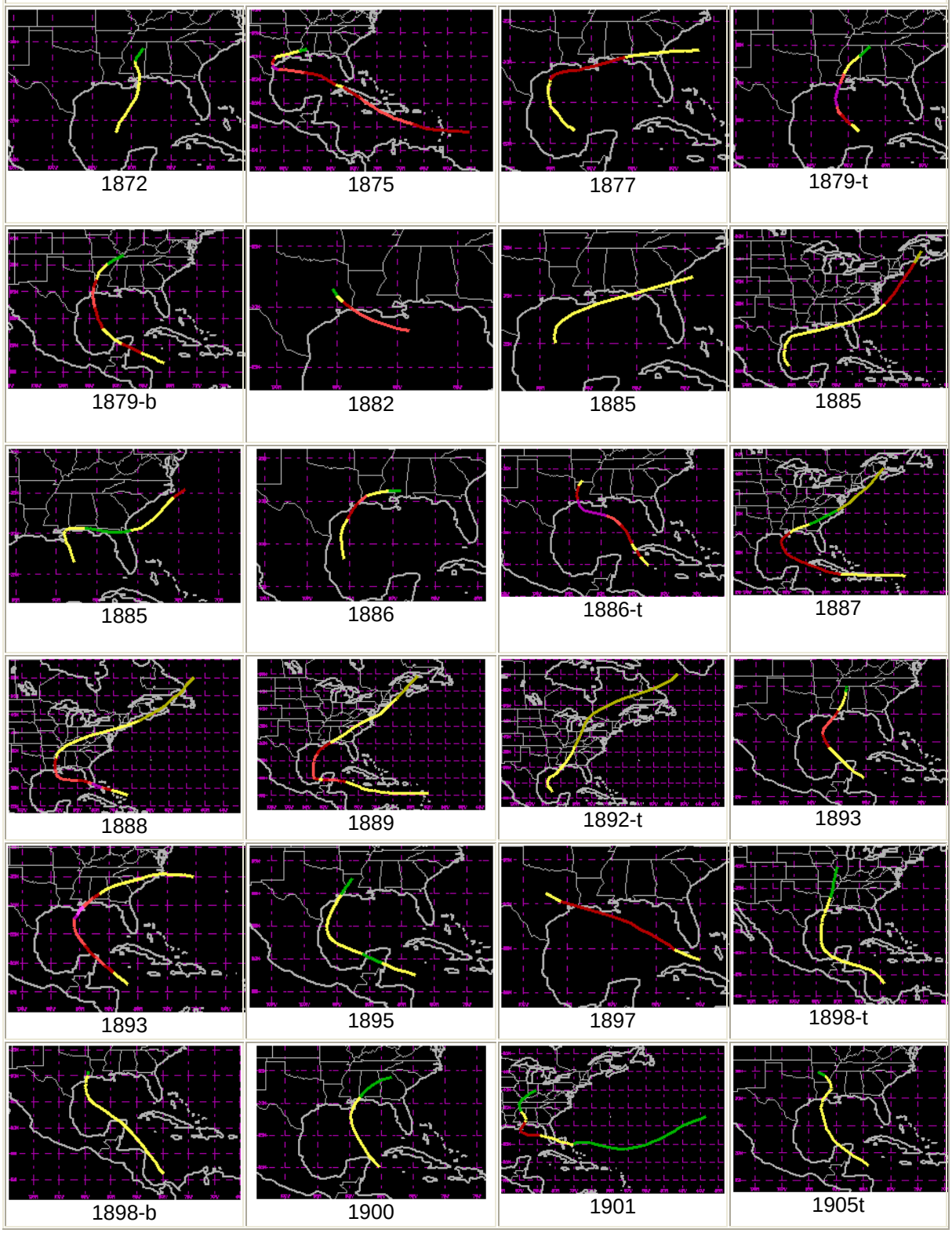
Existing Relationships

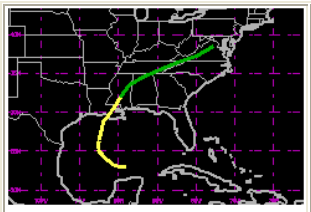
Relationships documenting the reduction in storm surge elevation due to landscape features and vegetation have been determined based on measurements in Louisiana. As more literature is reviewed and data are acquired, it is anticipated that more robust relationships will be developed.

In a Letter from the Chief of Engineers (1965) documenting an interim hurricane survey of Morgan City and vicinity, Louisiana, measurements of high water marks due to

hurricane surge were correlated with distance inland from the coast. Surge elevations at 16 locations near Morgan City due to seven hurricanes (Sep 1909, Aug 1915, Sep 1915,

Table 2. Storm Path for Hurricanes and Tropical Storms of Significance in Louisiana*

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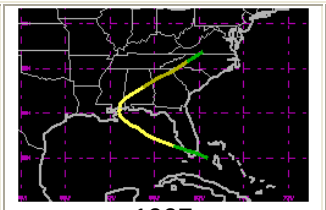
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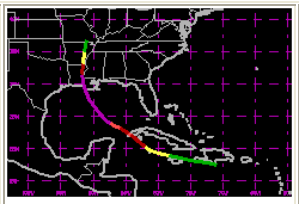
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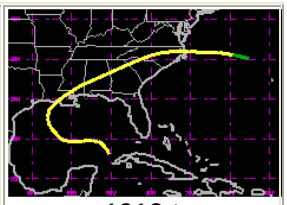
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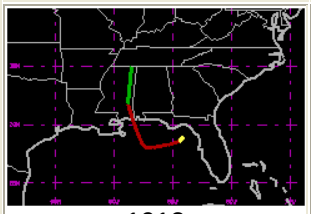
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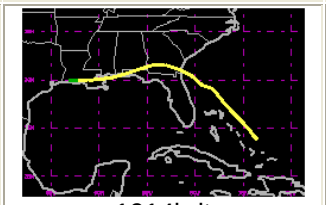
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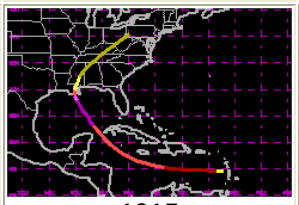
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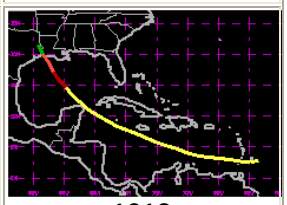
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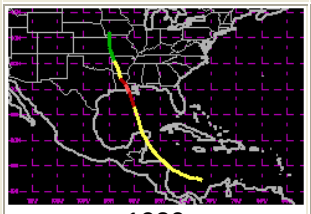
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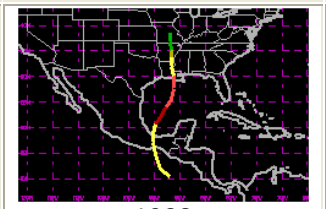
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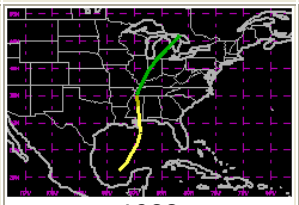
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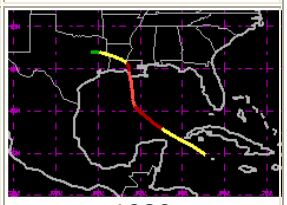
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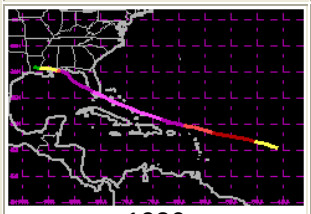
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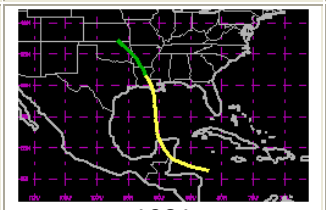
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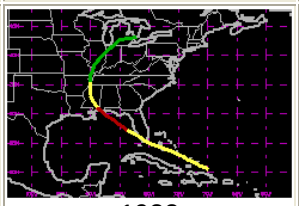
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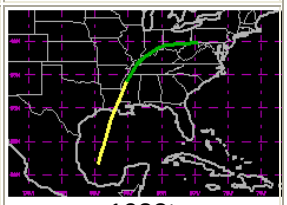
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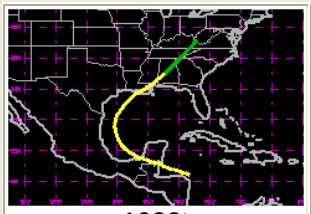
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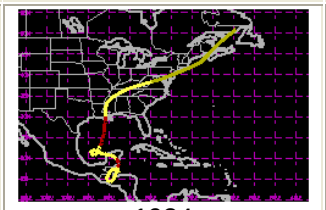
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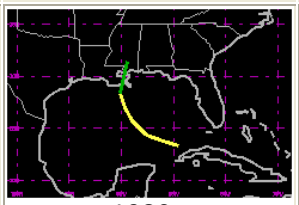
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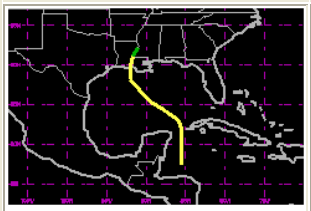
1936t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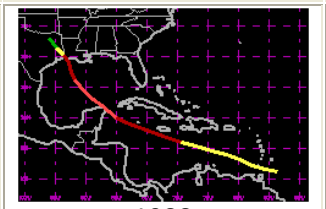
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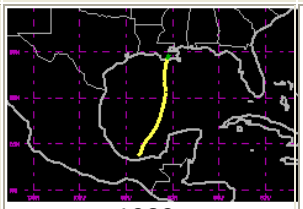
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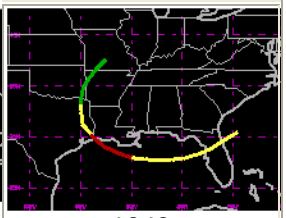
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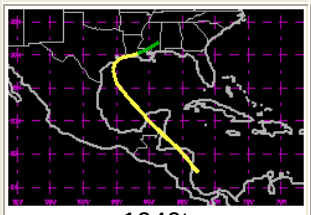
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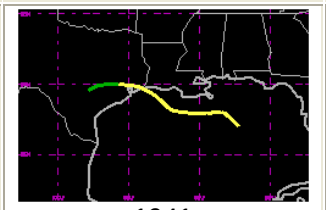
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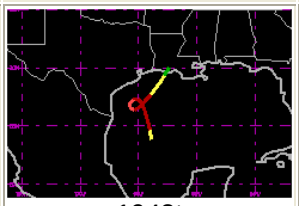
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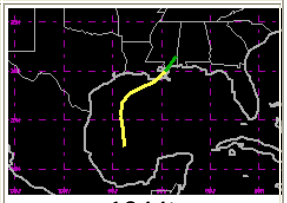
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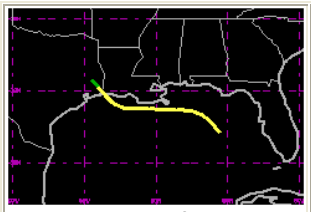
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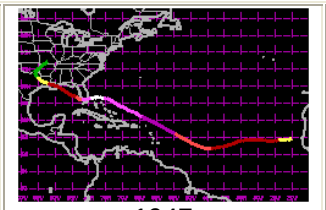
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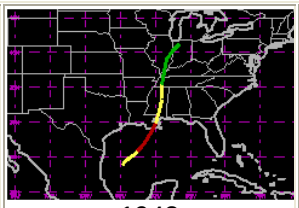
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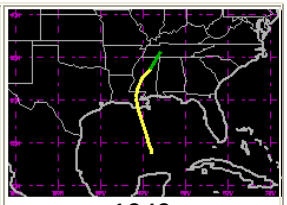
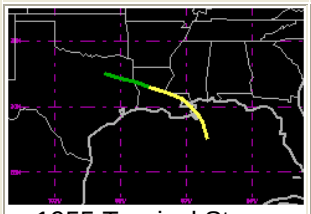
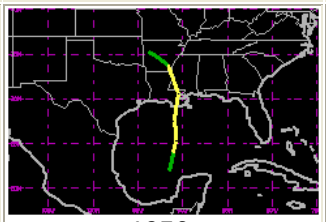

1949


1955 Tropical Storm Brenda 31 Jul – 3 Aug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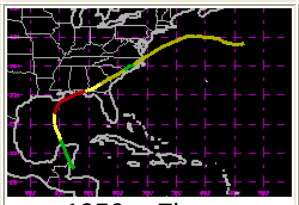
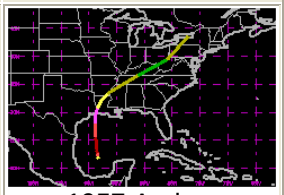
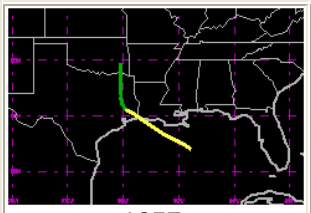

1956 -- Flossy


1957 Audrey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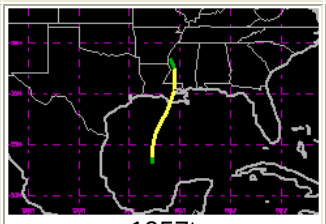
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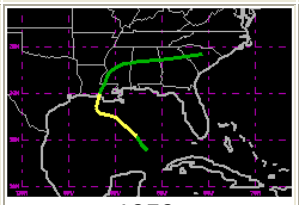
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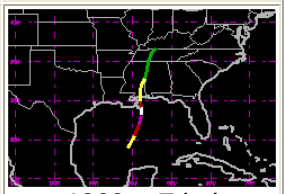
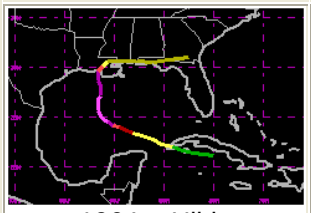
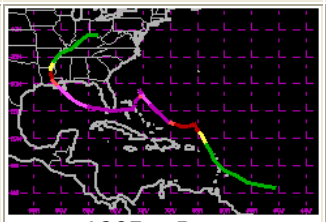
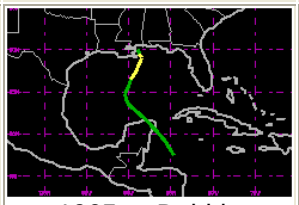
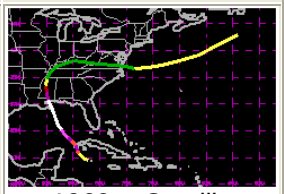
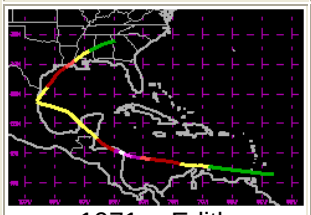

1960 -- Ethel

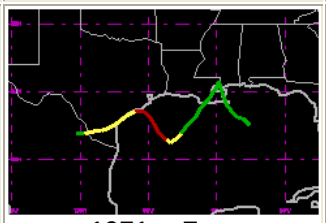

1964 -- Hilda

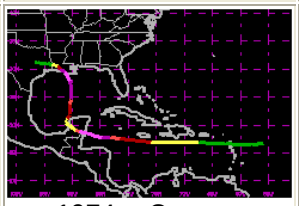

1965 -- Betsy

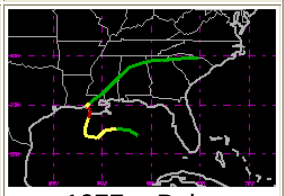

1965t -- Debbie

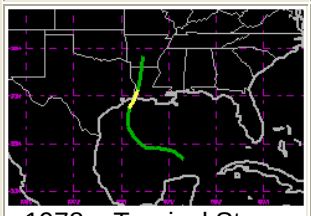

1969 -- Camille

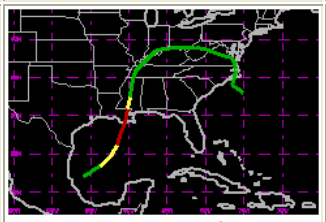

1971 -- Edith

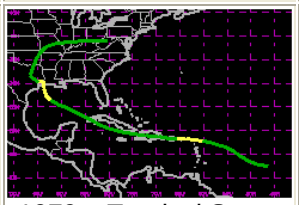

1971 -- Fern

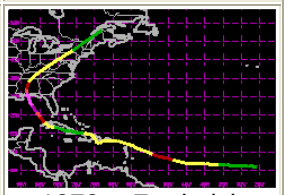

1974 -- Carmen

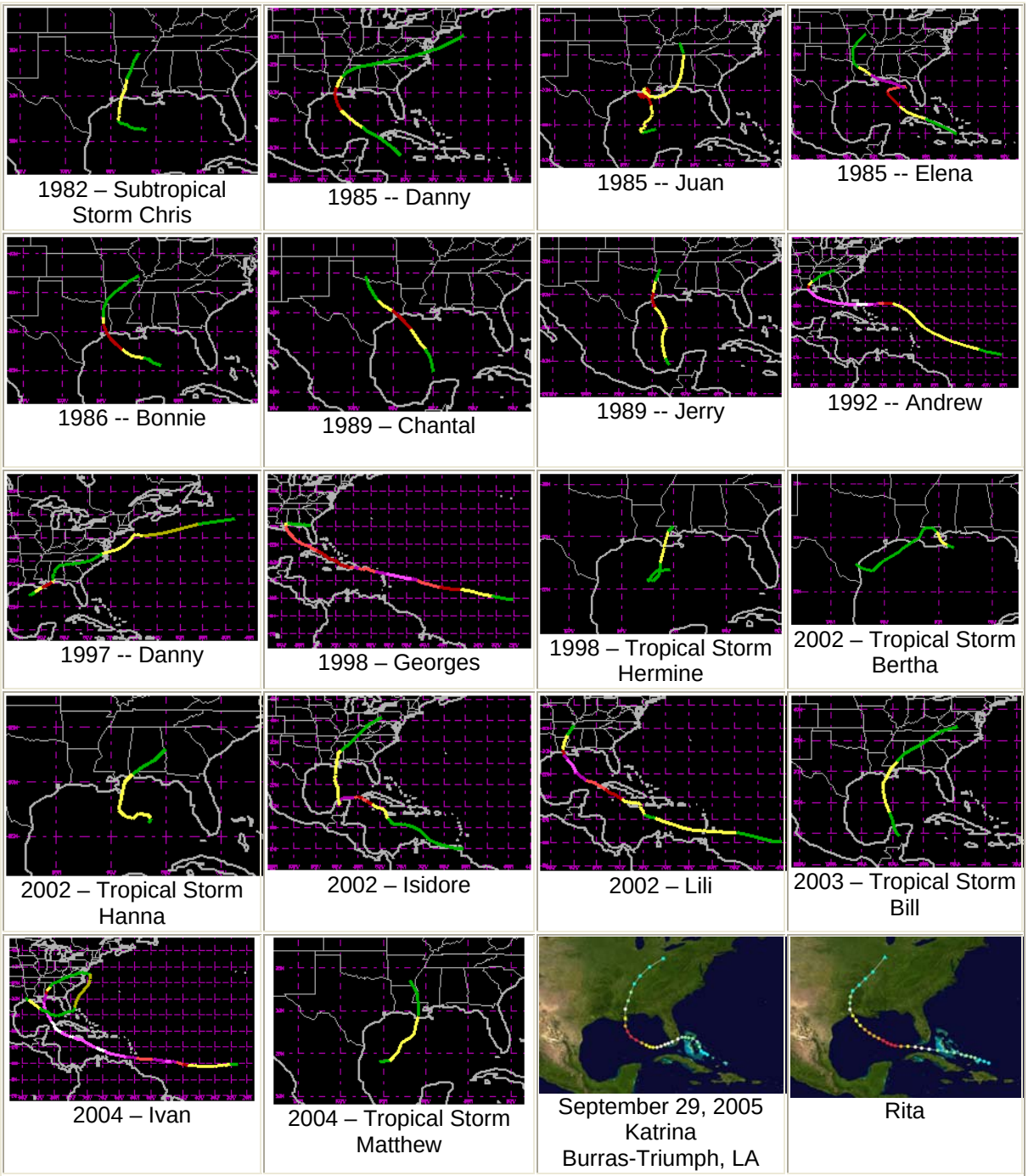

1977 -- Babe


1978 – Tropical Storm Debra


1979 -- Bob


1979 – Tropical Storm Claudette


1979 -- Frederick



* From http://weather.unisys.com/hurricane/atlantic/ and http://en.wikipedia.org/. Storm paths are not available for all storms listed in Table 1. Key for storms through 1998: green=tropical depression; yellow = tropical storm; maroon = Cat 1; red = Cat 2; purple = Cat 3; pink = Cat 4; white = Cat 5. Key for Ivan and Katrina: blue = tropical depression; turquoise = tropical storm; light yellow = Cat 1; yellow = Cat 2; gold = Cat 3; orange = Cat 4; red = Cat 5.

Aug 1926, Sep 1947, Sep 1956, and Jun 1957) were documented giving 42 data points (Figure 3). The report states that this area has numerous bays and marshes, but the data evaluated include the western part of Louisiana with cheniers (relatively high wooded ridges). Inconsistent results were obtained when attempting to correlate hurricane

translation speed, surge hydrograph at the coast, and surge elevations inland. However, a trend was observed for the decrease in storm surge as a function of distance inland, and is independent of hurricane translation speed, wind speed, and direction. The relationship indicates that storm surge was reduced by 1 foot for every 2.75 miles inland (1 cm decrease in storm surge per 145 m inland).

Lovelace (1994) documented storm surge elevations after Hurricane Andrew in Louisiana. These data are being compiled into a GIS for future reference. Citing this study, the Louisiana Coastal Wetlands Conservation and Restoration Task Force and the Wetlands Conservation and Restoration Authority (2004) suggest that storm surge reduces about 3-inch (0.25 ft) per mile (1 cm per 211 m) of marsh along the central Louisiana coast.

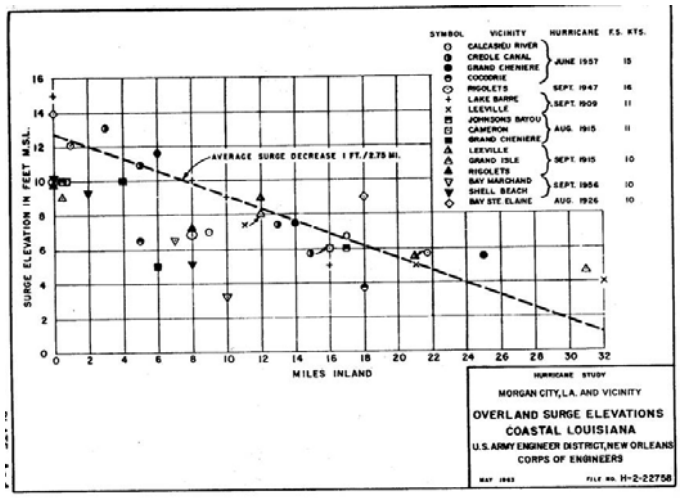


Figure 3. Observed maximum surge high water marks versus distance inland (USACE 1965)

Stone et al. (2003) modeled a Category 3 hurricane that made landfall in 1915 and compared wave and storm surge for the south-central Louisiana coast in 1950 (1.09 million acres of land) to that in 1990 (0.85 million acres of land). Models used were a hurricane planetary boundary model, ADCIRC circulation model, and SWAN wave model. Acreage impacted by a 2.1 m (7 ft) surge and 3.7 m (12 ft) increased to 69,000 and 49,000 acres, respectively, between 1950 and 1990 (Figure 4). Surge levels greater than 4.6 m (15 ft) were not significantly different between the two time periods.

The Louisiana Coastal Wetlands Conservation and Restoration Task Force and the Wetlands Conservation and Restoration Authority (2004; Chapter 6, p. 55) discuss that it is "commonly acknowledged that barrier islands and wetlands reduce the magnitude of hurricane storm surges and related flooding; however, there are scant data as to the degree of reduction." At the time the report was written, the best information documenting this phenomenon came from gages measuring water elevations during the second landfall of Hurricane Andrew (data documented by Lovelace 1994), which occurred in the vicinity of Point Chevreuil, Louisiana on August 26, 1992. Gage data from Cocodrie, Louisiana indicated a maximum water level elevation equal to 9.3 ft (2.8 m) during this Category 3 Hurricane. Over a 23-mile (37 km) stretch of marsh and open water from Cocodrie to the Houma Navigation Canal, the water elevation decreased from 9.3 ft (2.8 m) to 3.3 ft (1 m), equating to a reduction in surge amplitude equal to 3.1 inch (0.26 ft) per mile of marsh and open water (1 cm per 203 m). A similar set of measurements showed reduction of the storm surge from 4.9 ft (1.5 m) at Oyster Bayou to 0.5 ft (0.15 m) at Kent Bayou, located 19 miles (30.6 km) north. This second set of

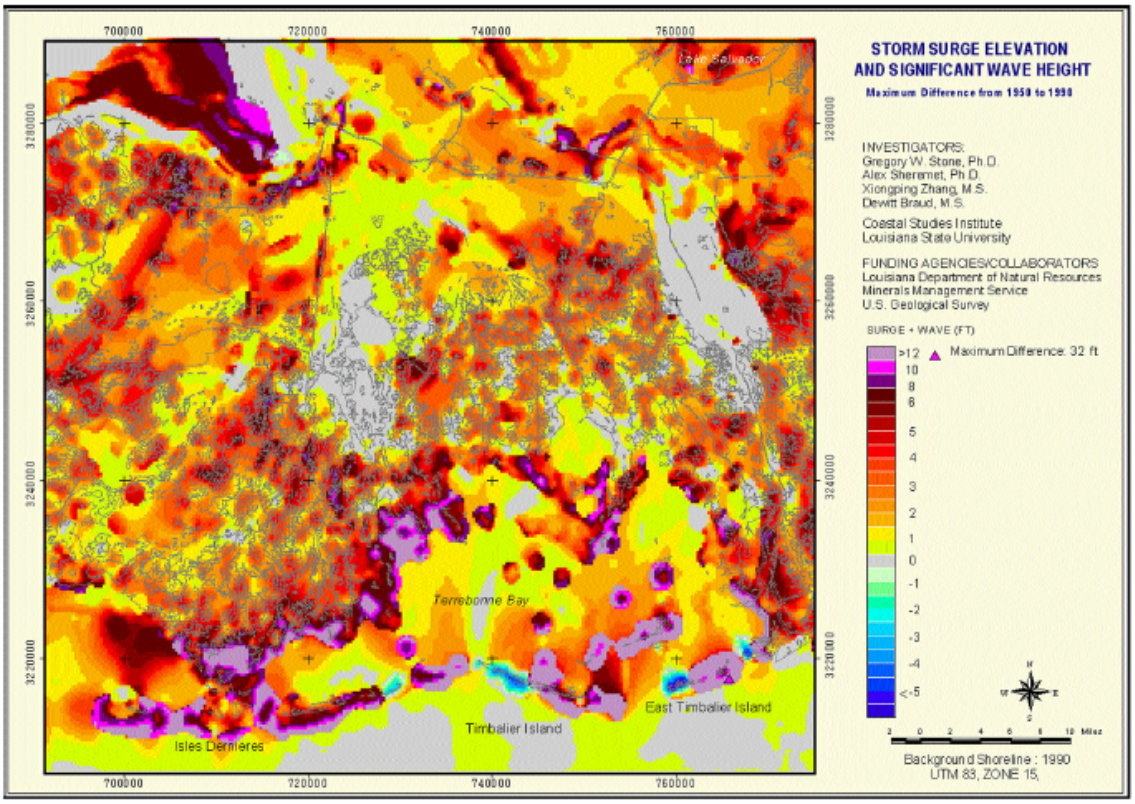


Figure 4. Difference between maximum surge plus significant wave height, 1950 to 1990 (Stone et al. 2003)

measurements indicated 2.8-inch (0.23 ft) decrease in surge per mile (1 cm per 230 m) over "fairly solid marsh." The report cautions that these represent measurements from one storm; other factors, such as storm characteristics, coastal geomorphology, and track of the storm influence the degree to which wetlands decrease storm surge.

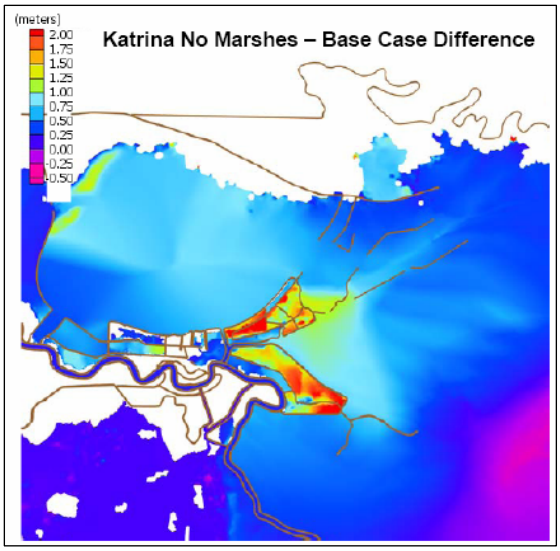


Figure 5. Differences in computed storm surge for Hurricane Katrina with the disappearance of wetland landscapes east of the Mississippi River Gulf Outlet (Working Group for Post-Hurricane Planning for the

The Working Group for Post-Hurricane Planning for Louisiana Coast (2006) wrote "barrier islands, shoals, marshes, forested wetlands and other features of the coastal landscape can provide a significant and potentially sustainable buffer from wind wave action and storm surge generated by tropical storms and hurricanes." ADCIRC results from Rick Luettich (Dec 30, 2005) indicated that wetland replacement east of the Mississippi River Gulf Outlet (MRGO) by 2.5-m (8-ft) of open water would increase storm surge from Hurricane Katrina by 1-2 m (3-6 ft) for St. Bernard Parish and Eastern New Orleans (Figure 5).

Engineering Relationships

Overview. Coastal landscape features affect the intensity and spatial patterns of storm winds, currents, waves, and water levels. These landscape features include wetlands, barrier islands, interior landscape ridges, navigation channels, bays and estuaries. This section presents a preliminary review of the engineering literature about the quantitative relationships between coastal landscape features and the characteristics of hurricane storms. The effects of each landscape feature on each of the hurricane storm characteristics are reviewed.

Wetlands contain a variety of vegetation types. The physical properties of wetlands that modify storm characteristics include the vegetation type, location, height and density. Vegetation has an effect on storm waves. Waves become depth limited, not fetch limited, over relatively short distance if the friction factor is high enough. Wind stress is also affected by land cover. The sediment geotechnical properties and morphology of each wetland can modify wave height and direction.

Barrier islands and interior landscape ridges modify storm surge as a function of location, elevation, width, vegetation cover, and foreshore slope. The degree to which a barrier island decreases storm surge elevation depends on whether the island is overtopped and if the adjacent tidal inlet cross sectional area is in equilibrium with the bay tidal prism. Inlet parameters include location, cross sectional area, depth, width, and frictional roughness.

Navigation channels are anthropogenic features that affect the landscape hydrology by their location, length, depth, width and roughness. Bays and estuaries affect bottom friction through their location, depth, bottom roughness, and bottom sediment shear strength. Suspended mud or a muddy seabed in the bay or estuary increases the rate of wave energy dissipation.

Winds. The strength and impact of hurricane winds in coastal areas is affected by landscape features in two distinct manners. First, the intensity of hurricane storms undergoes a significant decrease in intensity after landfall. This process, referred to as "filling," may actual be initiated before the eye of the storm crosses over land. The filling gradually reduces the wind velocity within the storm. The rate of wind speed reduction has been related to the numbers of hours after landfall and to the geographic region (NWS 23 1979). This rate of reduction is of highest category for the Louisiana coast, showing a reduction of the wind speed of about 15% at 5 hours after landfall and a reduction of about 30% at 10 hours after landfall.

Landscape features also affect hurricane winds because vegetation which extends above the water surface, both before and during flooding, reduces the speed of the wind at the water surface. This reduction in wind speed translates to a reduction in the wind stress which generates both storm waves and surges. The reduction in wind stress due to the presence of vegetation has been described with a "stress reduction factor" or SRF (Federal Emergency Management Agency (FEMA) 1985). The SRF is affected

differently by various land covers and the most important contribution is the areal distribution of the various land covers.

Wooded areas have the greatest effect, with the type, height and density of the trees being of primary importance. The SRF may be as low as 0.10, indicating a 90% reduction of the open water wind stress. The SRF for wooded areas is related to the fractional projected area of the trees. This fractional area is the area of the trees divided by the total flow area, with both areas being projected on a vertical plane perpendicular to the wind velocity. The effect of trees on the SRF is not linear. For a fractional projected area of 10% the SRF is 0.85, while for 40%, the SRF is 0.30. The effect decreases with higher fractional areas. At fractional areas equal to 60% and 80%, the SRF is 0.20 and 0.10, respectively.

Marsh grasses also affect the SRF, although this effect is very complex. Overall marsh grass has a smaller roughness than wooded areas, and has a smaller effect on wind velocity. Marsh grass is quite flexible and can be blown over during the hurricane. Also the marsh grasses can become inundated exposing the water surface to the full effect of the wind. The expected range in SRF for marsh is 0.70 to 0.90 with the higher value being used when the surge height is higher than the average height of the marsh grass. A value for 0.30 for the SRF has been used successfully by the USGS in the SWIFT2D hydrologic modeling of coastal wetlands (Swain 2005). The value of SRF equal to 0.30 was used for all computational grids having a Manning's coefficient greater than 0.10, implying that the vegetation is emergent.

Open water near land can experience a reduction in the wind stress when the wind is blowing offshore. This "downwind sheltering effect" results from the modification of the winds surface boundary layer as it passes a land surface having high roughness. This effect may extend to a distance of 2 to 10 nautical miles from the upwind land, and would be particularly important behind barrier islands. The approach used by FEMA is to linearly increase the wind stress from the reduced overland value to the open water value over a distance of from 2 to 10 nautical miles.

Waves. Storm waves are affected by several coastal landscape properties. These properties include the water depth (before and during flooding), bottom roughness or friction, water column friction, and bottom sediment characteristics.

The effect of water depth on waves becomes fundamental as waves propagate into shallow water and controls wave kinematics and dynamics (U.S. Army Corps of Engineers 2003). Shallow water wave processes includes generation, shoaling, refraction, diffraction, reflection, breaking, setup, run-up, bottom friction, water column friction, and dissipation of wave energy through wave/bottom interaction. The water depth and variations in water depth associated with coastal landscape features become particularly important when they cause wave breaking. Wave breaking occurs when the still water depth equals about 78% of the wave height and involves intense energy loss and can, for example, reduce wave heights by 90% over a distance of 10 meters. Wave run-up and

overtopping occur if the height of a barrier island or an interior ridge equals or is less than the still water elevation.

Bottom friction and wave/bottom interaction in shallow bays dissipates wave energy and can limit the height of waves to values considerably below the breaking criteria. This effect depends upon the type of bottom sediment in the bay. Muddy bottom sediments have a response that can involve actual motion of the bottom due to the elastic properties of clay and mud.

The wave energy loss through vegetation results from the drag force of the wave current on the plants (FIA 1984, FEMA 1988). The rate of energy loss depends upon the geometry of the individual plants and the density of the plants in a given area. For areas containing a variety of plant types, the number of plants of each type can be specified as the fraction of the total area covered by a plant type and the average number of plants per square foot in the fractional area. The total energy loss for all plants along a transect is the sum of the energy loss associated with all of the individual plant types. The time average energy loss, $E_{i,j}$ for all plants of all plant types is given by:

$$E_{i,j} = \frac{\int_0^T \int_0^{h_i} \left| F_{i,j} u \right| dz\, dt}{T} \tag{1}$$

where z is the elevation, $F_{i,j}$ is the drag force for the jth member of the ith plant type, h_i is the height of the submerged plant or the wave crest height if the plant is exposed, u is the horizontal wave current, and T is the total time being evaluated. The drag force on each individual plant is given as:

$$F_{i,j} = \frac{\rho C_D D_{i,j} |u| u}{2} \tag{2}$$

where ρ is the water mass density, C_D is the plant drag coefficient, and $D_{i,j}$ is the effective diameter of the jth member of the ith plant type. The drag coefficient generally varies with plant roughness and the Reynolds number, but is taken as 1.0 for most plants. The contribution from the flat parts of the plant leaves is generally ignored.

The growth or decay of wind waves propagating over vegetated areas can estimate the effects of high friction by adjusting the fetch length (Camfield 1977). In this analysis the friction factors associated with vegetation can be up to 100 times the friction factor associated with unvegetated shallow water. The friction factor for various vegetation types are given as a function of water depth for thick stands of marsh grass; dense grass, brush or bushy willows and scattered tress; and dense stands of trees. Based upon a water depth of 3 m (10 ft), the friction factor for marsh grass is 0.20, for dense grass and brush it is 0.48 and for dense stands of trees, 0.90. These values represent an increase over the unvegetated bottom friction by factors of 20, 48, and 90, respectively. An example can be cited of the effectiveness of vegetated wetlands to dissipate wave energy (U.S. Army Corps of Engineers 2003). Storm waves having an initial height of 3 m (10

ft) are predicted to be reduced to a height of 1.5 m (4.8 ft) after passing over 1000 m (3300 ft) of tall grass and brush.

Currents and Storm Surge Elevation. Currents and surge are affected by coastal landscape features through two mechanisms. Bottom friction is the generated by fluid shear stresses on the water bottom, while flow-drag resistance is generated by fluid stresses on objects extending through the water column (FEMA 1985). Bottom friction only occurs in bays whereas bottom friction and flow-drag resistance can occur in vegetated areas.

The most widely used formulation of bottom friction for flow in shallow water is the Manning-Chezy formula,

$$\tau = \frac{g|U|u}{C^2}, \ and \ \ C = \frac{1.486h^{1/6}}{N} \tag{3}$$

where τ is the bottom stress, $|U|$ is the flow speed, u is the vector velocity, C is the Chezy coefficient, h is the flow depth, and N is the Manning's coefficient. The Manning's coefficient is not a constant and varies with water depth and bottom roughness. For bays the Manning's coefficient has been represented as an exponential function of the water depth, by the following formula (FEMA 1985),

$$N = Ah^{-B} \tag{4}$$

where A and B are curve fitting parameters. Calibration data for various studies indicate B is about 0.5 and A varies between 0.08 and 0.12, with a mean value of 0.10. This formula indicates the Manning's coefficient decreases as the water depth increases, with values of N of about 0.044 for a depth of 1.5 m (5 ft), 0.032 for a depth of 3 m (10 ft) and 0.022 for a water depth of 6 m (20 ft). Since the Manning's N is typically used as a tuning factor in calibrating hydrodynamic models, in this formulation A can be used for the same purpose. For flooded wetlands, the Manning's N is assumed to be a constant that varies with vegetation type. Table 3 gives the range of values of Manning's N for various vegetation types.

Table 3. Estimated Values of Manning's Coefficient, *N*

Land Cover Type	Manning's *N*		
	Minimum	Mean	Maximum
Short grass	0.025	0.030	0.035
High grass	0.030	0.035	0.050
Scattered brush	0.035	0.050	0.070
Medium to dense brush	0.045	0.080	0.160
Marsh grass (0.3-1 m)	0.05	0.075	0.10
Marsh grass (1 – 2 m)	0.10	0.125	0.15
Marsh grass (>2m)	0.15	0.20	0.25

Flow-drag resistance also occurs in vegetated areas and represents flow resistance within the water column is a force that cannot be readily represented as a stress. Taking the approach that the flow-drag force on natural vegetation can be expressed as some the force on an equivalent cylinder, the total drag force for a given area of wetland can be given by

$$F_d = \frac{\rho C_d n D h_p V^2}{2} \tag{5}$$

where F_d is the drag force, C_d is the drag coefficient for the cylinder, n is the total number of plants, D is the diameter of each cylinder, h_p is the height of the submerged part of the cylinder, and V is the flow velocity. The drag coefficient C_d is not a constant and depends upon the size and proximity of each plant. An equivalent stress can be defined as the total drag force over an area, divided by the size of the area.

An alternative representation of the drag force on a number of plants is based upon the Darcy-Weisbach formulation,

$$F_d = \frac{\rho f V^2}{8} \tag{6}$$

where f is the Darcy-Weisbach resistance coefficient. This coefficient has been related to the "roughness concentration" given as

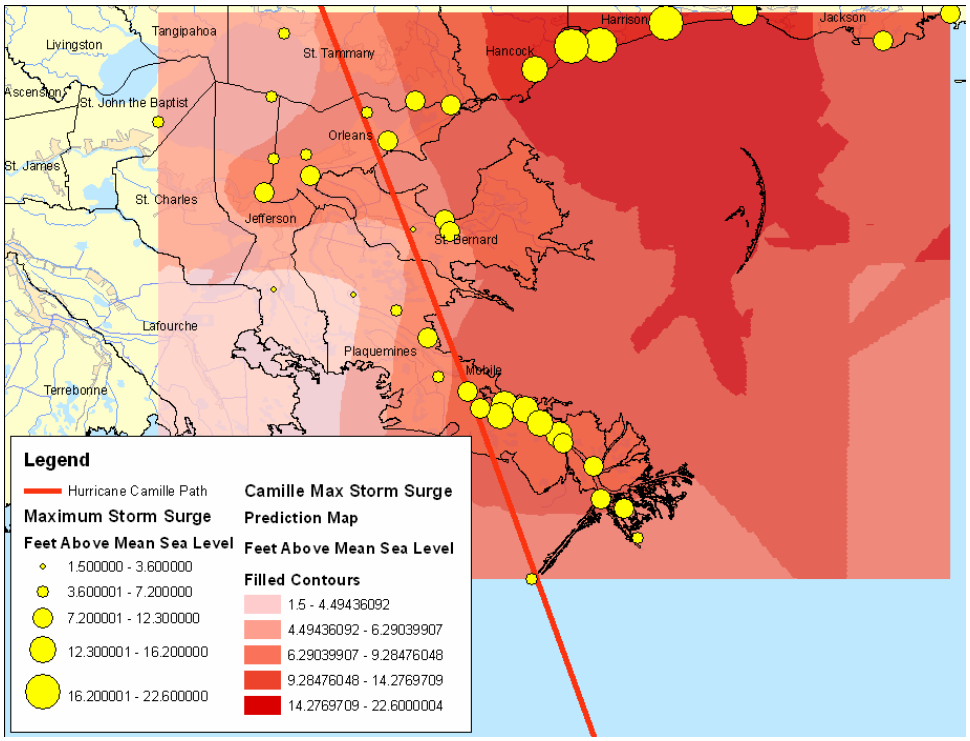
$$f = a\sigma^b, \; and \; \sigma = nDh_p \tag{7}$$

where σ is the roughness concentration, and a and b are calibration parameters.

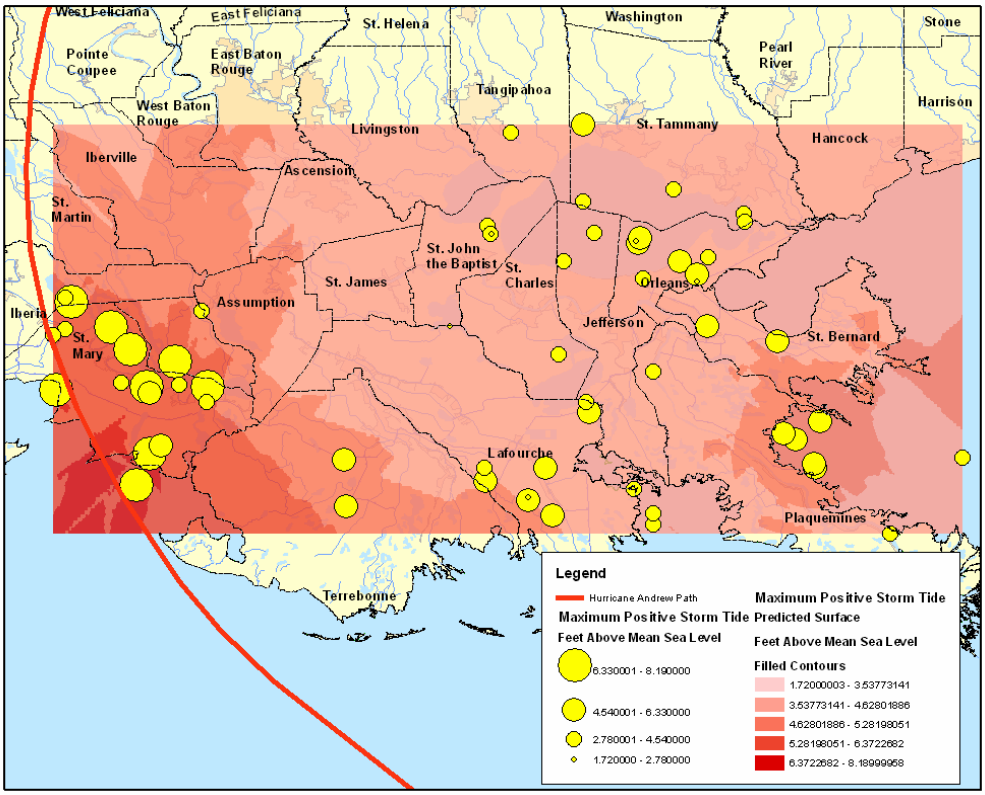
The effect of wetland vegetation density on the Manning's coefficient for overland flow was studied in a series of laboratory experiments (Hall 1994). The experiments involved placing bulrushes in various spatial densities in a 1.2 m (4 ft) wide channel and then subjecting them to discharges of 0.009, 0.026, 0.044 and 0.057 m^3/sec. The results of the tests indicated that for flow velocities in the range of 0.01 to 0.05 m/sec (0.03 to 0.16 ft/s), the Manning's N decreased as the average flow velocity increased, ranging about 0.3-0.9 at the lowest velocity to 0.2-0.3 at the highest velocity. A linear relationship was found between the density of plants and the Manning's N, with the value of N being about 0.6 for a density of 800 stems per square meter.

GIS Database

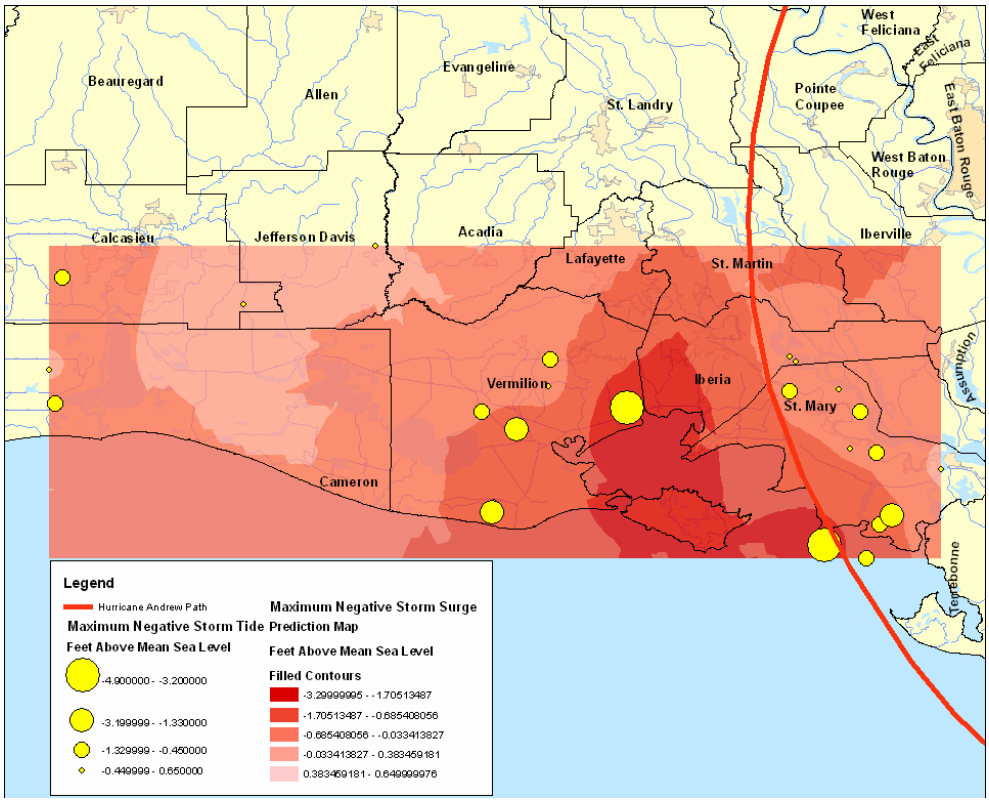
Measurements of storm surge elevation from Hurricanes Camille and Andrew have been incorporated into a GIS database. These data will be evaluated with the storm path and vegetation type to develop an understanding of how landscape features and vegetation modify storm surge elevation. Figure 6 shows preliminary contours of storm surge elevations measured during Hurricanes Camille and Andrew. Note that both positive and negative surge elevations were measured after Hurricane Andrew. More storm surge elevation data are available and will continue to be incorporated into the GIS.



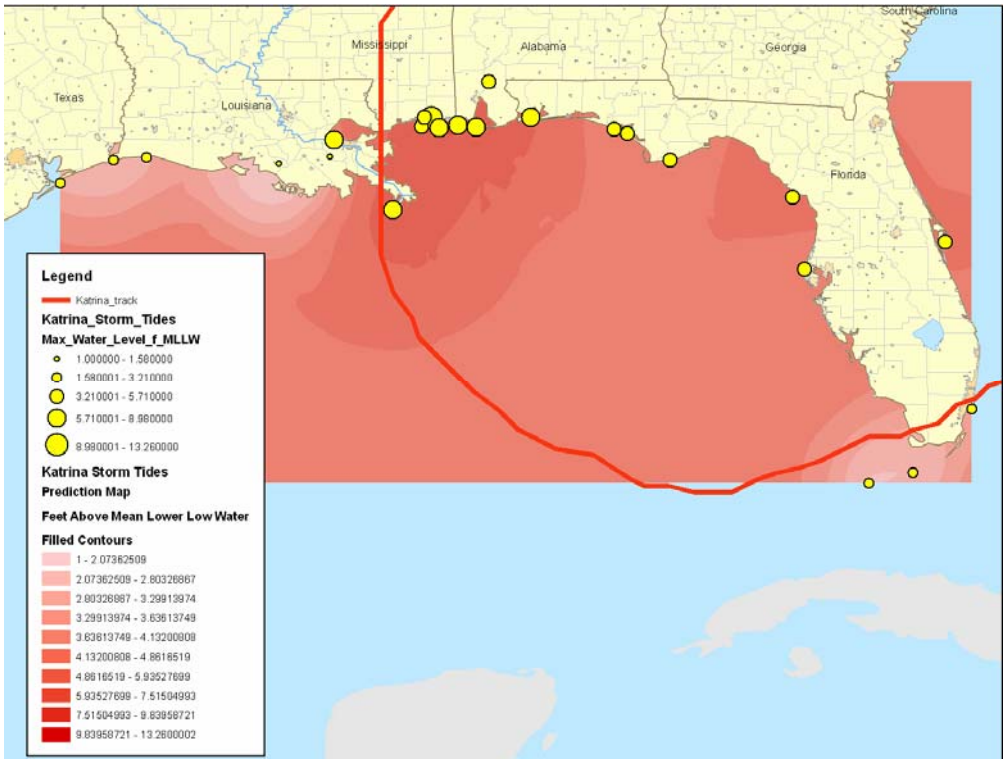
a. Hurricane Camille – positive storm surge



b. Hurricane Andrew – positive storm surge



c. Hurricane Andrew – negative storm surge



d. Hurricane Katrina – positive storm surge

Figure 6. Initial GIS Database

Preliminary Recommendations and Future Work

Table 3 summarizes the relationships that have been reviewed for storm surge reduction as a function of overland distance of landscape features and vegetation.

Table 3. Relationships for Storm Surge Reduction as a Function of Overland Distance

Distance Required for 1 cm Reduction in Surge Elevation (m)	Landscape Feature	Database	Reference and Notes
145	Cheniers, marsh, bays	7 hurricanes, 42 data points	USACE (1965); data on which relationship is based may represent both sides of storm path (see Figures 1 and 2 herein)
211	Central LA coast (assumed to be marsh and open water)	Hurricane Andrew; 2 data points	Lovelace (1994); more elevations are available and are being input to GIS database
203	Marsh and open water	Hurricane Andrew; 2 data points	The Louisiana Coastal Wetlands Conservation and Restoration Task Force and the Wetlands Conservation and Restoration Authority (2004), based on Lovelace (1994)
230	"Fairly solid marsh"	Hurricane Andrew; 2 data points	

Based on this preliminary review, it appears as if a conservative estimate might be 1 cm reduction in storm surge elevation for every 200-250 m of marsh. However, the location of each of these data points relative to the storm path and the quality of each data point must be evaluated. For example, data may be located on either side of the storm track and thus changes in elevation may represent differences in forcing conditions rather than a reduction in surge due to presence of a landscape feature. Elevations of the data point are also suspect as datums in the region have shifted through time. More recent measurements from Hurricanes Andrew and Katrina with more accurate datum control and broader coverage are available to infer relationships.

In their study of the south-central Louisiana coast, Stone et al. (2003) indicated that storm surge elevations greater than 4.6 m (15 ft) were not affected by changes in the landscape. From this finding, we might expect that the influence of *submerged* landscape features would decrease as storm surge increases. Landscape features only partially submerged would provide more resistance and thus reduce surge until they are submerged. Future work will continue with the literature review, conduct idealized numerical modeling tests to evaluate the reduction in surge as a function of landscape feature and vegetation type. A surge elevation database is being developed within a GIS for determining relationships based on available measurements, as well as for comparison with numerical modeling results.

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Appendix A

Table A1. Maximum Storm Surge Elevation Measurements (ongoing effort; this table represents a partial draft)

Storm	Location	Elevation (ft)	Reference and Datum
Unnamed Hurricane, 19 Sep 1947	Burrwood, LA	4	Sanders (1947) (no datum given, assumed to be mean sea level)
	Chandeleur Light, LA	14	
	Morgan City, LA	6	
	Bay St. Louis, MS	12	
	Biloxi, MS	12	
	Gulfport, MS	12	
	Pascagoula, MS	12	
Hurricane Betsy, 8-11 Sep 1965 (Partial listing of measure-ments) s=still high water mark; g=gage	Pascagoula, MS [(g)]	6.4	U.S. Army Engineer District, New Orleans (1965) (Mean Sea Level)
	Biloxi, MS [(g)]	8.6	
	Gulfport [(s)]	10.7	
	Pearlington, MS [(s)]	8.8	
	Lake St. Catherine (west side), LA [(s)]	10.6	
	Opening to Lake Pontchartrain [(g)]	7.0	
	Slidell, LA [(s)]	6.7	
	Lacombe, LA [(s)]	5.8	
	Pontchartrain Causeway (north), LA [(g)]	6.5	
	Lake Pontchartrain (north side), LA [(s)]	5.1	
	Lake Pontchartrain (west side), LA [(s)]	10.2	
	Pontchartrain Causeway (central), LA [(g)]	5.5	
	New Orleans on Lake Pontchartrain, LA [(g)]	5.0	
	New Orleans on east side, LA [(s)]	5.3	
	Algiers, on MS River [(g)]	12.6	
	Shell Beach, LA [(s)]	9.3	
	Yscloskey, LA [(g)]	11.7	
	Delacroix, LA [(s)]	11.0	
	Phoenix, LA [(g)]	8.3	
	Between Phoenix and Pointe a la Hache Bohemia (northeast side of river), LA [(s)]	8.8, 9.8, 11.9, 10.7 (from north to south)	
	Pointe a la Hache Bohemia, LA [(s)]	14.4	
	South of Port Sulphur, LA [(s)] (northeast and southwest sides of river, respectively)	13.7, 5.7	
	Empire, LA [(s)]	7.4	
	Ostrica (north side of river), LA [(s)]	13.6	
	Buras, LA [(s)]	7.7	
	Between Ostrica and Venice (south side of river), LA [(s)]	14.6	

	Venice, LA [(s)]	8.8	
	Head of Passes, LA [(s)]	6.6	
	Garden Island Bay, LA [(s)]	7.7	
	Port Eads, LA [(g)]	5.2	
	Burrwood, LA [(s)]	5.5	
	Grand Isle, LA [(g)]	8.8	
	Leeville, LA [(g)]	5.4	
	Pascagoula, MS [(s)]	6.4	U.S. Army Engineer District, Mobile (1967) (mean sea level)
	Biloxi, MS [(s)]	8.6	
	Gulfport, MS [(s)]	10.7	
	Long Beach, MS [(s)]	12.3	
	Pass Christian, MS [(s)]	10.8	
	Waveland, MS [(s)]	12.7	
	Clermont Harbor, MS [(s)]	12.0	
	Pearlington, MS [(s)]	8.8	
	Bay St. Louis, MS (seawall) [(s)]	12.5	
	St. Louis Bay, MS [(s)]	11.2	
Hurricane Camille, 17-18 Aug 1969; s=still high water mark; g=gage; d=debris line	Alabama border [(s)]	9.2	U.S. Army Corps of Engineers, Mobile District (1970, Plate 6) (mean sea level)
	Pascagoula, MS [(s)]	11.4	
	Biloxi, MS [(s)]	15.5	
	Gulfport [(s)]	21.0	
	Bay St. Louis (east side), MS [(s)]	22.6	
	Bay St. Louis (west side), MS [(s)]	21.7	
	Clermont Harbor, MS [(d)]	16.2	
	Lake St. Catherine (east side), LA [(s)]	12.3	
	Opening to Lake Pontchartrain [(g)]	9.0	
	Pontchartrain Causeway (north), LA [(g)]	4.6	
	Lake Pontchartrain (west side), LA [(g)]	4.6	
	Pontchartrain Causeway (central), LA [(g)]	4.1	
	New Orleans on Lake Pontchartrain, LA [(g)]	5.2	
	New Orleans on canal, LA [(g)]	6.5	
	Canal confluence with MS River, LA [(s)]	10.2	
	MS River near New Orleans, LA [(g)]	10.8	
	Between Lake Borgne and Lake Pontchartrain, LA [(g)]	8.7	
	Shell Beach, LA [(g)]	11.1	
	Yscloskey, LA [(d)]	2.6	
	End of Hwy. 46, LA [(s)]	8.9	
	Barataria, LA [(g)]	1.5	
	Phoenix, LA [(d)]	2.6	
	Between Phoenix and Pointe a la Hache Bohemia (northeast side of river), LA [(d)]	5.4	
	Pointe a la Hache Bohemia, LA [(d)]	11.0	

	Port Sulphur, LA [(s)]	5.2	
	Empire, LA [(s)]	10.9	
	Ostrica (north side of river), LA [(s)]	15.9	
	Buras, LA [(s)]	13.4	
	Between Ostrica and Venice (south side of river), LA [(s)]	14.6	
	Venice (north), LA [(d)]	15.9	
	Venice (south), LA [(s)]	9.1	
	Head of Main Pass, LA [(s)]	10.7	
	Head of Passes, LA [(s)]	12.0	
	Garden Island Bay, LA [(s)]	9.0	
	Port Eads, LA [(s)]	5.2	
	Burrwood, LA [(g)]	5.0	
	Grand Isle, LA [(d)]	3.6	
	Leeville, LA [(g)]	2.1	
	Pearlington, MS	5.2	
Hurricane Georges, 1988	Waveland, MS	6.6	U.S. Army Corps of Engineers, South Atlantic Division (1999) (NGVD 1929) See Figure A1
	Bay St. Louis, MS	5.8	
	Pass Christian, MS	7.9	
	Gulfport, MS	7.1	
	Biloxi – Pt. Cadet, MS	8.1	
	Biloxi – Back Bay, MS	8.3	
	Belle Fontaine Point, MS	11.0	
	Pascagoula – Hwy 90, MS	8.1	
	Pascagoula, MS	8.4	
	Pascagoula – MS Sound, MS	10.8	
	Pascagoula – Bayou Chico, MS	9.6	
	Bayou La Batre, AL	8.3	
	Dauphin Island, Gulf, AL	6.6	
	Dauphin Island, Bay, AL	5.0	
	Mobile Bay, Hollingers Island, AL	8.4	
	Downtown Mobile, AL	8.3	
	Mobile Bay – Causeway, AL	9.4	
	Weeks Bay, AL	6.5	
	Fort Morgan – Bay, AL	6.4	
	Pine Beach -- Bay, AL	8.5	
	Pine Beach – Gulf, AL	10.8	
	Gulf Shores, AL	9.5	
	Perdido Pass, AL	5.6	
	Ono Island, AL	5.4	
	Pensacola, FL	6.4	
	Pensacola Beach, FL	7.7	
	Gulf Breeze – Santa Rosa Sound, FL	4.5	

	Navarre – Santa Rosa Sound, FL	4.5	
	Choctawhatchee Bay, FL	5.2	
	Destin Harbor, FL	4.6	
	Grayton Beach, FL	4.6	
	Panama City Beach, FL	5.1	
	Panama City Harbor, FL	3.5	
	Port St. Joe – North side, FL	2.3	
	Apalachicola Bay, FL	4.5	
	Carrabelle, FL	4.6	
Hurricane Ivan, 2004 (partial listing of measure-ments)	Miss. Sound at Waveland, MS	4.56	U.S. Army Engineer District, Mobile http://chps.sam.usace.army.mil/USHESdata/Assessments/2004Storms/Ivan/slosh/table_1.htm (NGVD)
	Gulfport Harbor at Gulfport, MS	4.63	
	Mississippi Sound at Ship Island, MS	5.15	
	Biloxi Bay at Point Cadet, MS	4.23	
	W. Pascagoula river at Hwy. 90 at Gautier, MS	4.10	
	Pascagoula river at Pascagoula, MS	6.72	
	Miss. Sound at Pascagoula Pl – Rear Range	5.83	
	Miss. Sound at Petit Bois Island	4.83	
	Escatawpa River at I-10 near Orange Grove	3.93	
	Middle Gage at Bayou LaBatre	4.66	
	Mobile Bay at Cedar Point, AL	6.90	
	Dauphin island Bay at Dauphin Island	7.80	
	Mobile Bay at Dauphin island	8.00	
	Mobile River at Bucks, AL (Barry Steam Plant)	6.82	
	Mobile River at Mobile, AL	4.87	
Hurricane Katrina, 2005 *sensor malfunction did not record max **sensor malfunction at higher water levels	Waveland, MS*	8.98	NOAA (2005) (Mean Lower Low Water)
	Pilots Station, SW Pass, LA	7.75	
	Pensacola, FL	6.69	
	Dauphin Island, AL	6.37	
	Horn Island, MS*	6.23	
	East Bank, LaBranch, LA*	6.12	
	Grand Isle, LA**	5.71	
	Panama City Beach, FL	4.34	
	Biloxi, MS*	4.32	
	Lower Bryant Landing, AL**	3.89	
	Panama City, FL	3.83	
	Panama City, FL	3.83	

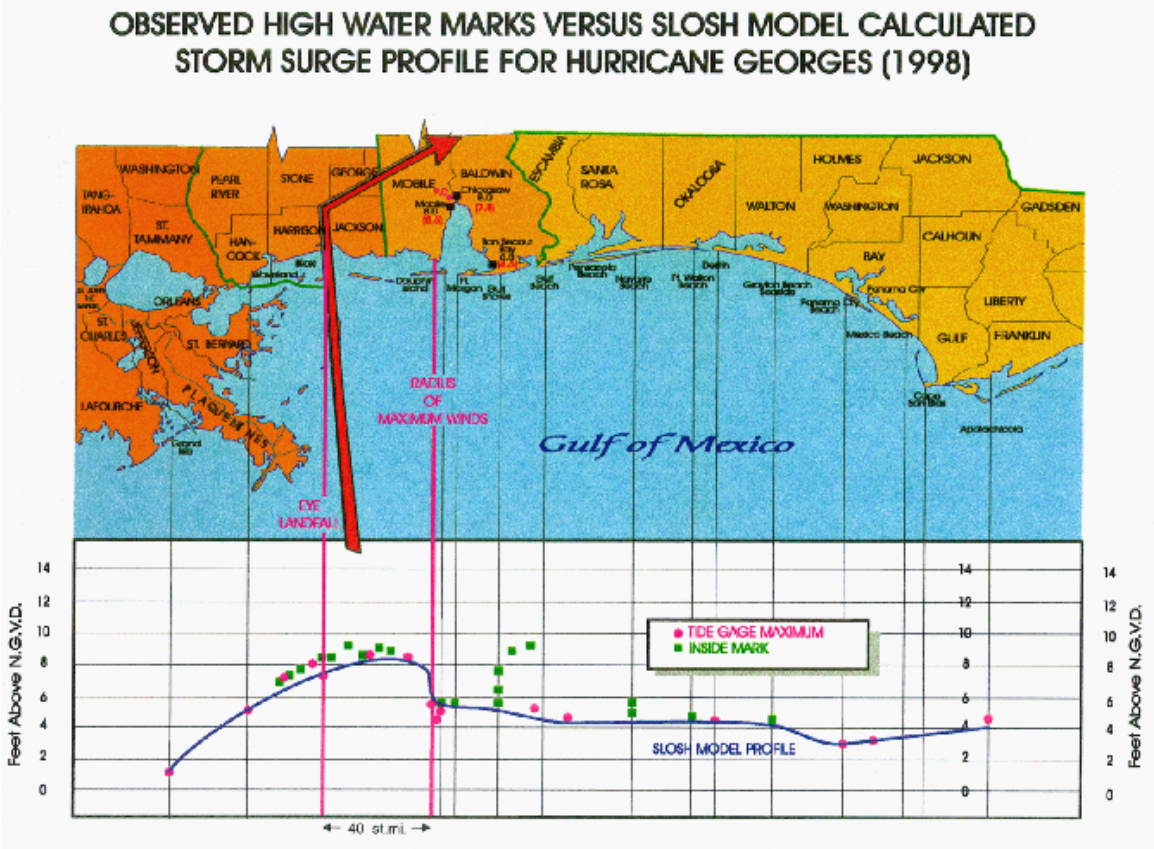
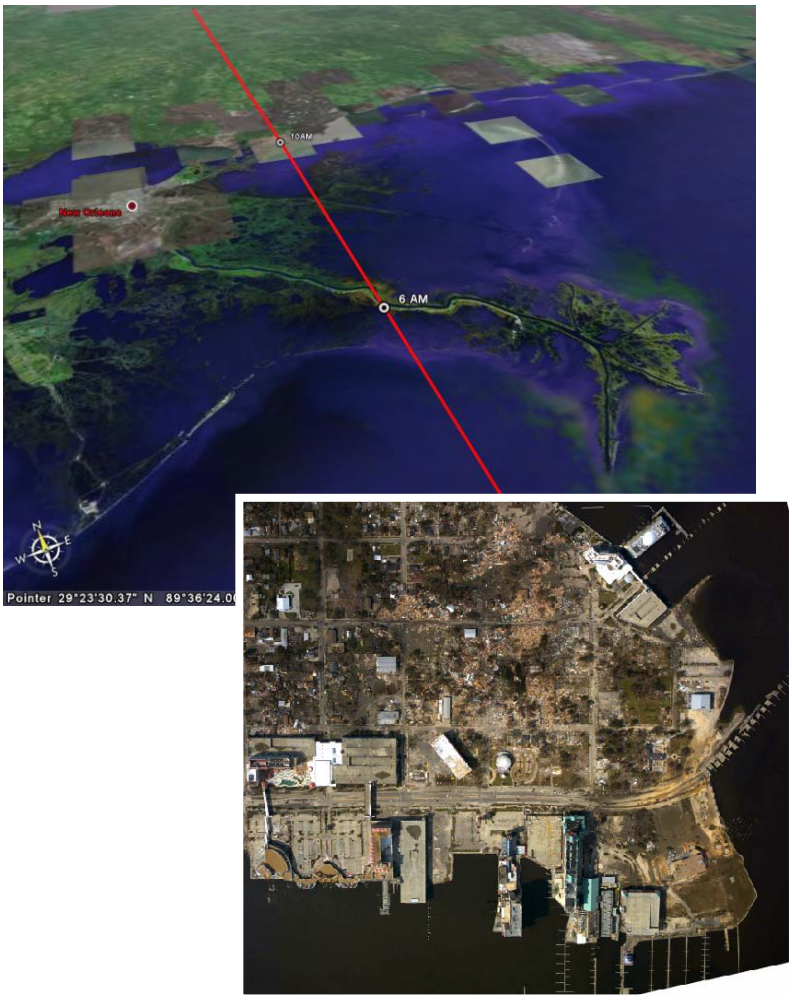


Figure A1. High water marks from Hurricane Georges and predicted with SLOSH model (Sea, Lake, and Overland Surges from Hurricanes; Jelesnianski et al. 1992)
http://chps.sam.usace.army.mil/USHESdata/Assessments/Georges/Chapter%202.htm

ANNEX 6 – UPDATED SPH & PMH INDICES

Update to the Standard Project Hurricane (SPH) Indices:
Preliminary Technical Report



May 15, 2006

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DRAFT TECHNICAL REPORT – CURRENTLY UNDER AGENCY REVIEW

1. Introduction:

The U.S. Army Corps of Engineers (USACE) uses meteorological criteria determined from historical hurricane intensities for its engineering projects along the nation's eastern coasts. The specifics of the sea level pressure, wind and other storm related parameters were determined using a standardized analysis of historical wind fields and the observed hurricane intensities that have impacted the Atlantic and Gulf of Mexico coasts of the U.S. The Standard Project Hurricane (SPH) "model" is one of two approaches the USACE currently uses to determine wind fields associated with tropical cyclones. The other model, termed the Probable Maximum Hurricane (PMH), is a more conservative approach than the SPH, and as such has a higher intensity (i.e. lower central pressure, and higher wind speed).

The SPH has been used for over 50 years by the USACE, and has been reanalyzed a number of times over that period. The first set of SPH indices was approved by USACE in a design study for Lake Okeechobee, Florida (U.S. Weather Bureau, March 1954). Detailed guidance on the selection of site-specific storm parameters was initially published in the *National Hurricane Research Project Report No. 33* (U.S. Weather Bureau, November 1959 – subsequently referred to as Report 33). The Weather Bureau and USACE jointly derived the specifications, criteria, procedures, and methods contained in report No. 33. The goal of the guidance was to provide generalized hurricane specifications that are consistent geographically, as well as meteorologically, in establishing design criteria for hurricane protection works.

The specifications for SPH were reviewed several times after 1959, and the Weather Bureau subsequently issued updates. After Hurricane Betsy in 1965, the Weather Bureau revised the wind field parameters, but did not modify the other characteristics of the SPH (U.S. Weather Bureau, August 1965, November 1965, and February 1966). Subsequent reanalysis using hurricane observations through 1975 were published as a NOAA National Weather Service updated technical report in 1979 (*NOAA NWS Technical Report No. 23* - subsequently referred to as Report 33). Guidance on the selection of site-specific storm parameters were updated and reissued in 1987 (*NOAA National Weather Service Technical Report, NWS No. 38*).

2. Definition of the Standard Project Hurricane (SPH):

The Standard Project Hurricane (SPH) indices were originally determined using hurricane data from 1900-1956. In Report No. 33, the SPH was defined as "the most severe storm that is considered reasonably characteristic of a region." The SPH indices were based on an analysis of past hurricanes of record along the Gulf of Mexico and Atlantic coasts, and were delineated geographically into zones. The purpose was to use the observed characteristics of hurricanes and correlate them with intensity criteria, location, and other features.

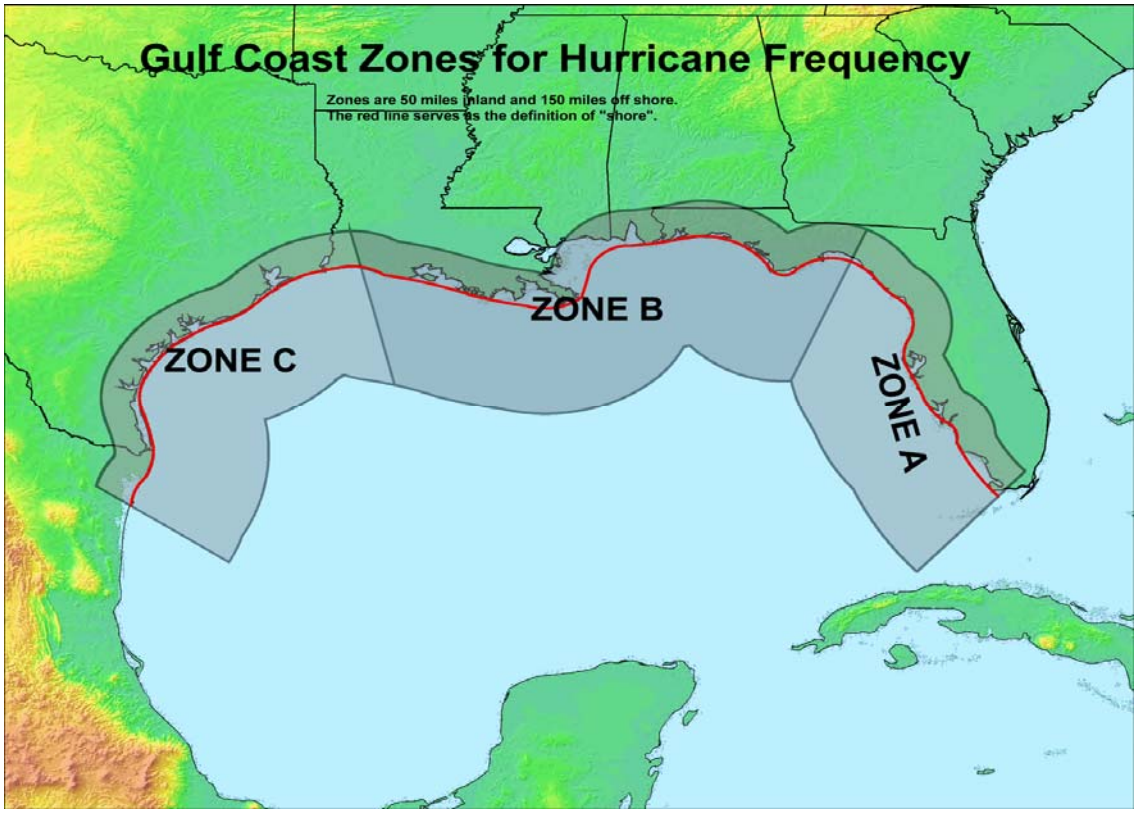

Figure 1. Gulf of Mexico coast zones defined in Report No. 33 used for determining hurricane frequency. Note that each zone extends 50 miles inland and 150 offshore using an idealized coastline (red line).

In Report No. 33, the Central Pressure Index (CPI) was the principal intensity criterion for defining the SPH index. The 1% frequency was selected as the CPI that defined the SPH index. Three Gulf coast zones were identified; most of coastal Louisiana was contained within Zone B, a 400-mile zone extending from Cameron, LA, to Pensacola, FL (see Fig. 1). For each zone, a frequency analysis was performed on the central pressure index of all storms with a CPI of less than or equal to 29 inches of mercury (~980 mb or 98.0 kPa) that passed through the zone during the period 1900-1956. The 1% frequency CPI for each zone was plotted along the Gulf coast at the geographic center of each zone, and a line drawn connecting the three points. The SPH for any specific location along the Gulf of Mexico coast was determined from this plot.

In 1979, Report 23 was published and it contained revised criteria for the SPH. Report No. 23 defined the SPH as "a steady state hurricane having a severe combination of values of meteorological parameters that will give high sustained wind speeds reasonably characteristic of *a specified coastal location*." Reasonably characteristic means, "only a few hurricanes of record over a large region have had more extreme values of the meteorological parameters." As already mentioned, specific guidance on the selection of site-specific storm parameters were updated and reissued in 1987 (*NOAA National Weather Service Technical Report, NWS No. 38*).

In Report 23, single limiting values of CPI and peripheral pressure were the criteria used to define the SPH index, and from which the maximum gradient wind speed equation was

developed. The approach used in Report 23 study was less statistically bound in setting the level of the CPI. Alongshore variations were determined by developing estimates within each of more than 60 overlapping zones along the East and Gulf coasts and smoothing between designated points. Each of the historical CPIs was plotted using a distance marker (nautical mile-marker or kilometer-marker) corresponding to each hurricane's location along the Gulf coast, and the CPI for the Louisiana coast and the New Orleans metropolitan area were determined from this plot. Because of this methodology, a frequency of occurrence for the SPH could not be determined based solely on the frequency of the CPI. However, Report 23 specified that "a frequency may be determined for any combination of values of meteorological parameters that define an SPH wind field". It is also important to note that for the revisions presented here no trend analysis was performed on the data to determine the SPH indices, and specifically the CPI. Changes in the SPH indices presented herein were the result of the time periods included in each study and the methods applied to analyze the tropical cyclones.

3. Data and Methods:

Prior to the mid-1960s and the satellite era, tropical cyclone detection was reliant on observations from land stations, ship reports, and occasional aircraft reconnaissance. It is possible that some storms may have been missed completely, although this is unlikely in the relatively confined space of the Gulf of Mexico. Reconnaissance flights officially commenced in 1944 (although aircraft traffic increased dramatically beginning in 1941 due to WWII), which enhanced the quality and quantity of data available, but it was not until the advent of geostationary satellite observations in the 1960's that all tropical storms and hurricanes were reliably detected in the Atlantic basin.

In recent years NOAA has undertaken the task to reanalyze North Atlantic hurricanes into a single database incorporating the best possible data available. This database, which runs from 1851 to 2003, is called HURDAT (Landsea et al.), and was used for this reanalysis of Gulf of Mexico storms. Operational data for 2004 and 2005 from NOAA's National Hurricane Center was added to make the record complete.

The data analysis involved first ascertaining which storms entered the three Gulf of Mexico coastal study areas (Zones A, B, and C, as shown in Fig. 1), and assessing the lowest central pressure within each zone for each tropical cyclone. This index is called the Central Pressure Index (CPI), which has been defined as the estimated (or observed) minimum pressure (p_O) for each hurricane within each zone. In addition, the maximum wind speeds were also determined for each hurricane within these three zones. Generalized Extreme Value (GEV) analyses were then performed for each zone's dataset which, together with an assessment of peripheral pressures (using the North American Regional Re-analysis – NARR data) completed the study.

3.1 Generalized Extreme Value (GEV) Analyses:

The results presented herein include the CPI (p_o) and maximum sustained wind speed (V_{max}) data for all tropical cyclones within each zone from 1900 to 2005, which were separated into two categories; one with hurricanes only (sustained winds ≥64 kt) and the other including all hurricanes and tropical storms (sustained winds ≥34 kt). Those tropical cyclones that had pressure data in HURDAT were then analyzed to assess the return periods of different central pressures, with the first step to assess the annual minimum pressure (p_o) and maximum sustained wind speed in each zone.

It was apparent that, while the wind-speed record was reasonably complete over the period 1900 to 2005, the minimum central pressure values were missing or undetermined for many tropical cyclones prior to 1965. There were, of course, years in which no tropical storm or hurricane entered a particular zone. However, it was also clear that a bias to the statistics would be introduced if we bundled these years together with those years with tropical cyclones but no observational data. Therefore, extreme value analyses of wind-speed included the whole period of record, while the minimum central pressure was analyzed only for more recent decades with adequate data.

The extreme value analysis used in this study was the GEV Type I (Gumbel 1941) :

$$G(x) = \exp[-\exp\{-(x - u)/\alpha\}] \qquad -\infty \leq x \geq \infty$$

where the parameters are $\alpha = (\sqrt{6}\ x\ s) / \pi$ (s is standard deviation)

and $u = \bar{u} - 0.5772\alpha$ ($\bar{u}$ is the mean)

So if $y = (x - u)/\alpha$, $G(x) = \exp[-\exp(-y)]$, and $y = -\ln[\ln\{1/G(x)\}]$

and for return period, T ,

$$G(x_T) = (T - 1)/T$$

so $y_T = -\ln[\ln\{T/(T - 1)\}]$

However, provision must be made for years with no storm, and defining this probability as P(0),

$$Y_T = P(0) + [1 - P(0)]y_T$$

and

$$x_T = u + \alpha Y_T$$

The T-year minimum pressure or maximum wind = x_T mb or kts

To obtain the confidence interval of the return period first an assessment of the standard error must be calculated, which can be found using the expression (Kite, 1977):

$$s_e = s[(1 + 1.1396K_T + 1.1K_T^2)/n]^{0.5}$$

$$\text{where } K_T = -\sqrt{6}\{0.5772 + \ln[\ln(T/(T-1))]\}/\pi \quad \text{(Chow, 1953)}$$

Then the limits of confidence of x_T are $x_T \pm s_e z_\alpha$

where z_α is the normal variate and α is the two tailed significance level (in this case a 90% confidence interval was required, so $z_\alpha = 1.645$).

The results from this analysis presented in this report were confirmed using the GEV sub-routines in the commercially available MatLab software, and also verified by USACE personnel, who applied a slightly different statistical model. The USACE applied a mixed GEV/Poisson distribution model, using only hurricanes which had CPIs lower than 980 mb (~29 inches of mercury). These results will be presented in section 4.2 below.

3.2 Peripheral Pressure Assessment:

The peripheral pressure (p_W) is the sea-level pressure at the outer limits of the hurricane circulation. It is used along with the central pressure to compute the pressure drop (differential pressure, ΔP), which is used in numerical models to determine the hurricane's wind field In Report No. 23, the peripheral pressure was defined in two slightly different ways: 1) p_W was defined as the average pressure around the hurricane circulation where the isobars change from cyclonic to anticyclonic, and 2) p_W was defined as the average pressure of the last closed isobar around the hurricane's circulation. Both of these methods for determining p_W must coincide with the observation time of the hurricane's minimum central pressure (p_O). For the results presented, the average value of the last closed isobar was determined to be the more accurate method for determining p_W.

Values of p_W were analyzed for all hurricanes in Zone B over the period 1976-2005. In order to conduct this analysis, digital data of the mean sea level pressure associated with each hurricane was obtained from the North American Regional Reanalysis (NARR) dataset. The differential pressure (which is the peripheral pressure minus central pressure) was found to have a high correlation with central pressures and it was, therefore, possible to provide a good estimate of the 1 in 100-year value of p_W. The results of this analysis are shown in Figure 2, which is a comparison of p_O and ΔP for all hurricanes in Zone B since 1976. A total of 21 hurricanes were observed in Zone B since 1976, and using the results for p_W from the NARR dataset, a value for p_W was found to be 1007.9 mb. This was extremely close to the value of p_W=1008.1 mb that was originally determined in Report No. 23.

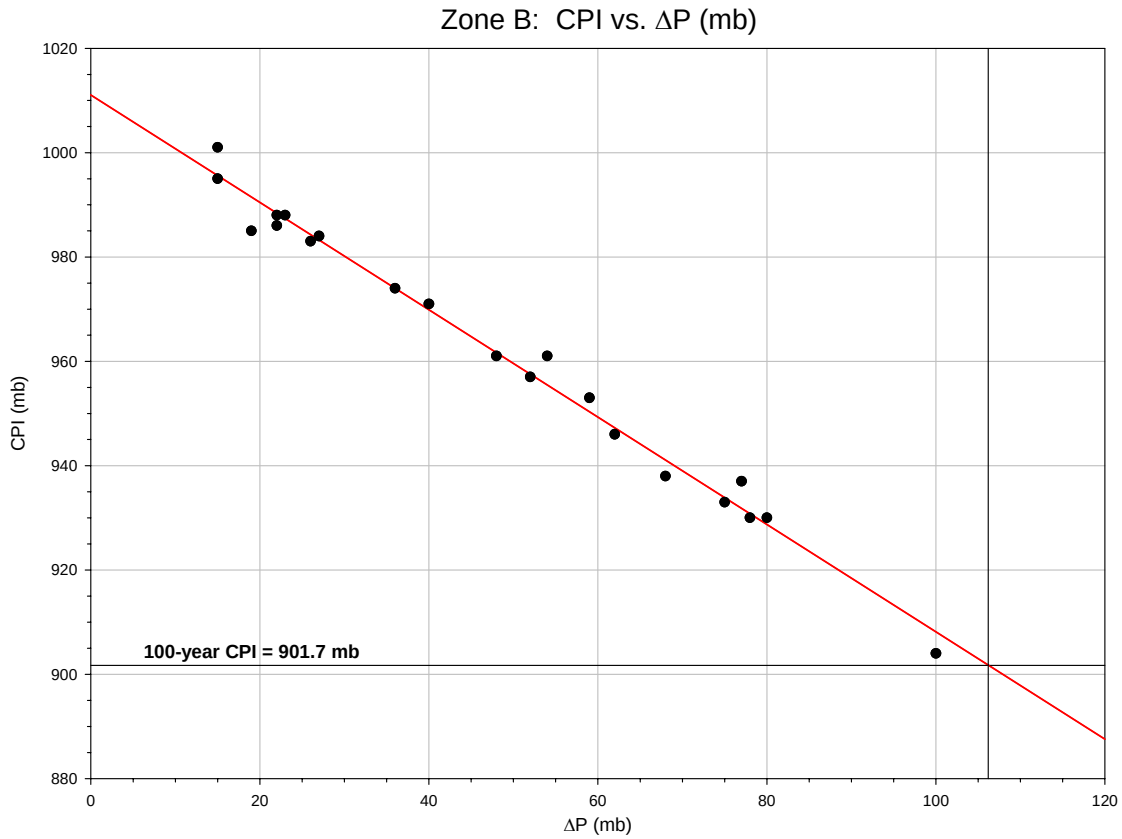

Figure 2. A comparison of the Central Pressure Index (CPI in mb) and the differential pressure (ΔP in mb) for all hurricanes in Zone B along the central Gulf of Mexico coast and covering the period 1976-2005 (a total of 21 hurricanes). Intersection of the linear regression (red line) with the 1 in 100-year CPI (901.7 mb) yielded a ΔP = **106.2 mb**. This value was then added to the CPI to yield a peripheral pressure p_w = **1007.9 mb** (or p_w = 29.763 inches). The resulting peripheral pressure was extremely close to the value determined in NWS Tech Report #23 (p_w = 29.77 inches or p_w = 1008.128 mb).

3.3 Comparison between Original Tech Report and Reanalysis:

Using the central pressure data from the HURDAT database, and the analysis of the CPI published in Report No. 23, a comparison of the 1900-1975 data was produced (see Figure 3 below).

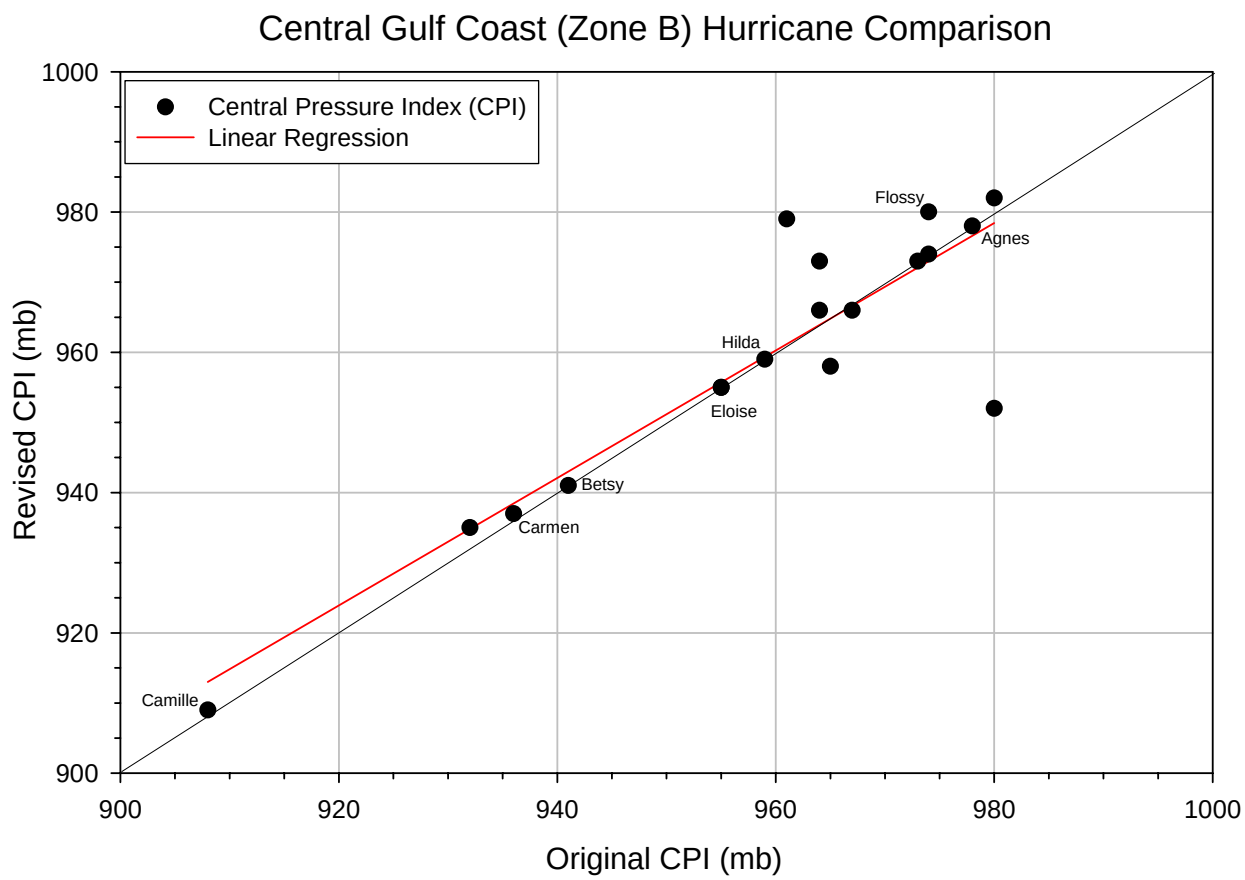

Figure 3. A comparison of the CPI (in mb) observed in Zone B over the period 1900-1975. The "Original CPI" refers to the CPI data published in Report No. 23 and the "Revised CPI" values are those analyzed using the CPI data in HURDAT. Note that a total of 23 hurricanes occurred during this 76-year period, but 5 of these had no reliable CPI data in HURDAT.

In general, there was very good agreement between the original analysis of the CPIs for Zone B and the reanalysis using the data in HURDAT. However, there were a number of obvious discrepancies between the two results. Of the 23 hurricanes that were determined to be in Zone B covering the period 1900-1975, five of these had no central pressure data in the HURDAT database. This was likely due to the lack of reliable central pressure estimates for these particular hurricanes, and therefore these were not used in this comparison. The remaining 18 hurricanes over that period are shown in Figure 3 below, where the y-axis is the revised CPI and the x-axis the original CPI as published in Report No. 23. The two strongest hurricanes that made landfall in Zone B during that time period, Camille and Carmen, both had a CPI that was 1 mb higher

than in the analysis published in Report 23. These slight adjustments of the central pressures were the result of the reanalysis conducted with the development of HURDAT. The other CPI discrepancies between the analysis in Report 23 and HURDAT occurred at higher pressures (i.e., weaker hurricanes), although there is certainly close agreement between the two independent sets of data.

4. Reanalysis of Central Pressure Index (CPI):

Central Pressure Indices (CPI) for all Gulf of Mexico hurricanes were analyzed from the HURDAT database and partitioned into the three zones that cover the Gulf coast region (Zones A, B, and C as originally defined in Report #33). These zones are shown in Figure 1. The CPI for each hurricane was determined within each zone, beginning in 1900 and extending through the 2005 season.

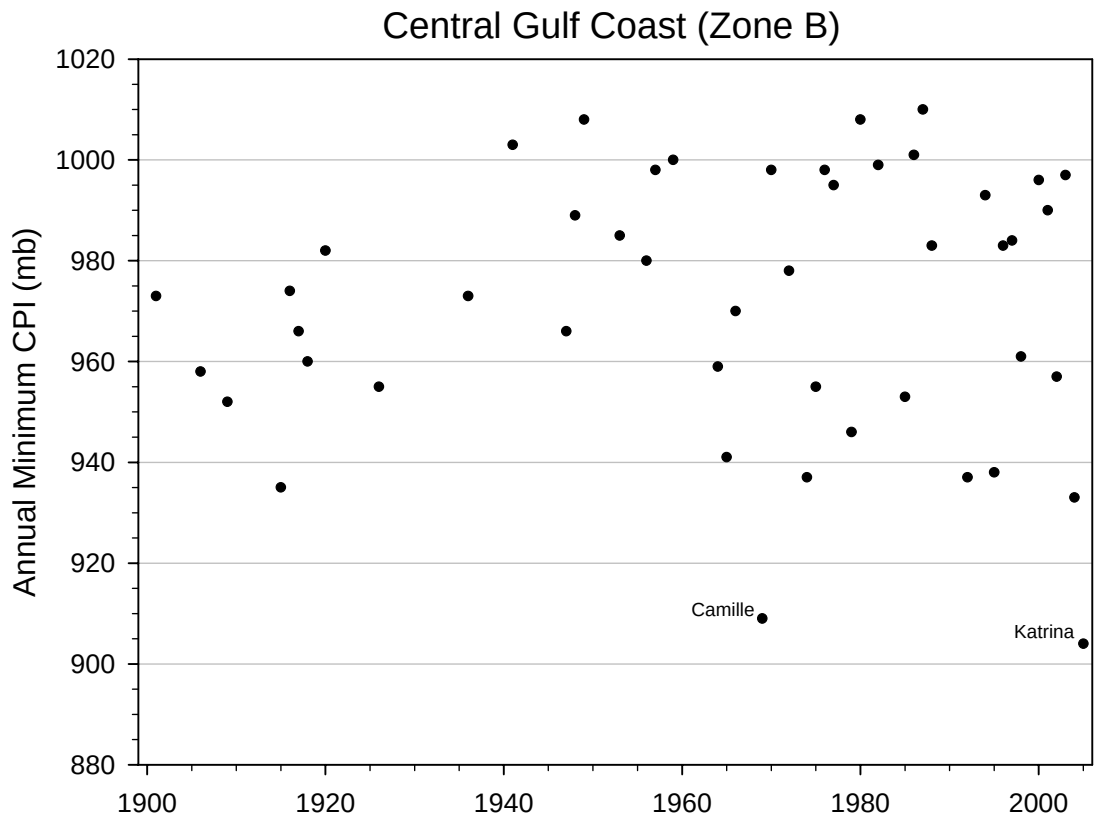

Figure 4. Annual values of the Central Pressure Index (CPI in mb) in Zone B determined over the period 1900-2005. Estimates of the CPI were determined from HURDAT over the period 1900-2003, with supplemental track data from NOAA's National Hurricane Center for the 2004 and 2005 hurricanes seasons.

4.1 Determining the 1% (1 in 100-year) CPI Value:

Different statistical techniques were used to determine the CPI value that accurately corresponds with the 1 in 100-year return period. All of these methods use similar Generalized Extreme Value (GEV) techniques for determining the 1 in 100-year event, with variations the result of the input data, period of record of the input data and the inclusion criteria chosen for each analysis. Note that there is no singular method that can be suggested as the "correct" one when estimating probabilities, since there are likely justifications for each. However, it is essential to use accurate and reliable input data covering the time-frame in question, and therefore a realistic sample of tropical cyclones are the most important criteria for achieving representative results.

The different datasets and analyses that were considered in determining the 1 in 100-year CPI (p_o) were as follows (as shown in Figure 6):

1. NCDC reanalysis using a mixed-Poisson/Gumbel approach with data covering the period 1941-2005, and excluding values > 980 mb.

2. NCDC reanalysis using a mixed-Poisson/GEV Type III approach with data covering the period 1941-2005, and excluding values > 980 mb.

3. Ocean Weather, Inc. (OWI) analysis using a mixed-Poisson/Gumbel approach with data covering the period 1941-2005, and excluding values > 980 mb (OWI used a slightly different area than Zone B, which included all hurricanes north of 27°N).

4. NCDC reanalysis using a Gumbel annual minimum approach, with data covering the period 1976-2005.

5. NCDC reanalysis using a Gumbel annual minimum approach, with data covering the period 1965-2005.

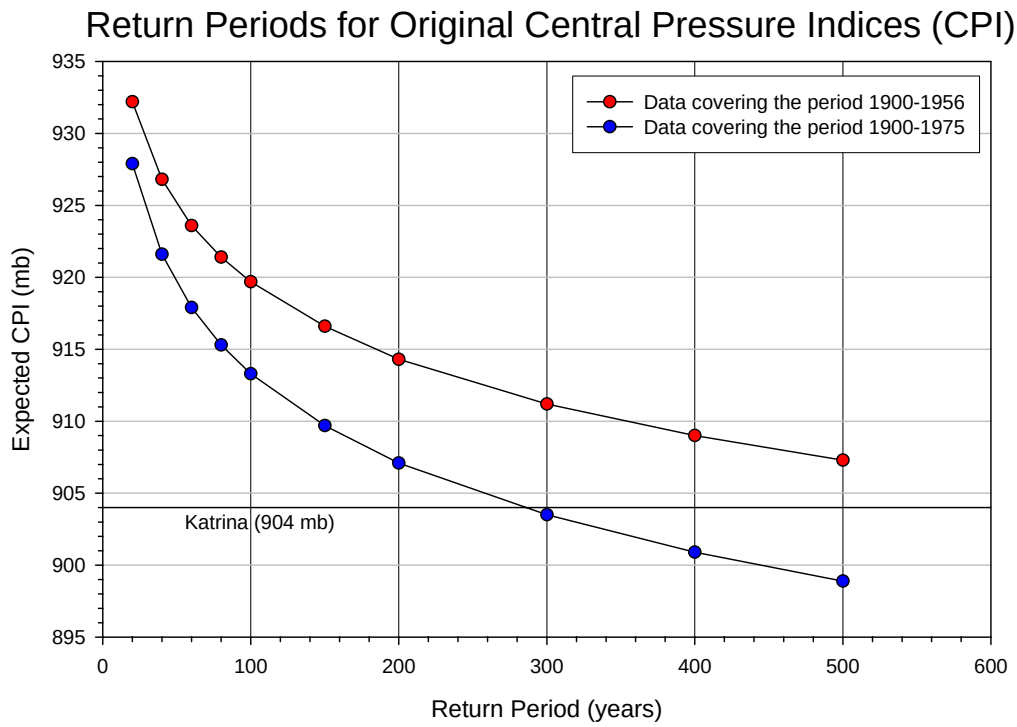

Figure 5. Return periods for CPIs in Zone B determined using Generalized Extreme Value (GEV) analysis and based on the original CPI data used for determining the SPH: 1) 1900-1956 covers the original period of record used in Report No. 33 (1959), and 2) 1900-1976 covers the period of record used in Report No. 23 (1979).

4.2 <u>Results of CPI Analysis:</u>

The expected return period minimum central pressures, together with 90% confidence intervals are shown in Figure 7 below. The expected return periods were calculated using the Gumbel (1941) Generalized Extreme Value (i.e., GEV Type I) analysis method outlined in section 3.1. The appropriate adjustments were applied for no tropical cyclones in a particular year and the annual minimum pressure values in Zone B for all tropical storms and hurricanes from 1965 to 2005. This resulted in a 1 in 100 year CPI return period in Zone B of **901.7 mb**

The USACE, using their mixed model, calculated the 1 in 100-year return period CPI to be 903.85 mb using GEV Type I, and 902.14 mb with the GEV Type III. Either of these values are extremely close to the results from the analysis produced by NCDC, and adequately verified the 1 in 100-year return period CPI value of **901.7 mb**.

Note that the official estimates from NOAA of Hurricane Katrina's lowest central pressure were 904 mb, which yields an approximately 1 in 90-year return period of a hurricane of this intensity in Zone B. However, Katrina had "filled" (weakened) slightly to about 915 mb at landfall, but a value of 904 mb was used as the CPI since that was the lowest central pressure within Zone B. The second lowest CPI in Zone B in the historical record was Hurricane

Camille, which severely damaged the Mississippi and Alabama Gulf coasts in 1969. Camille had a CPI in the official NOAA archive of 909 mb. However, there has been some debate regarding the issue of Camille's central pressure immediately prior to landfall, as a U.S. Navy dropsonde measured a central pressure as low as 901 mb (V. Cardone 2006, personal communication). However, this observation has yet to be substantiated and the analysis presented here used the central pressure value of 909 mb that is in HURDAT.

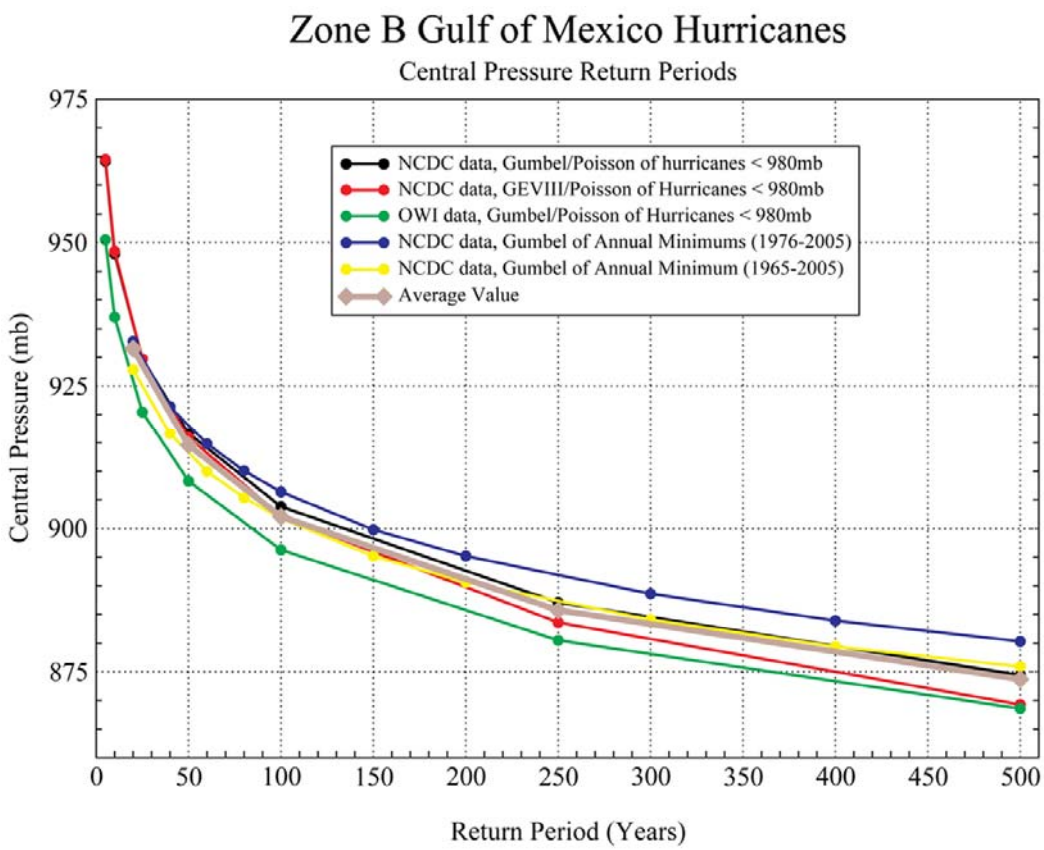

Figure 6. A comparison of the different return periods of the CPI determined independently in Zone B along the Central Gulf of Mexico coast. Each of the curves shown above corresponds to the GEV analysis technique applied and the period of record of the data. The grey line denotes the average of the 5 methods shown.

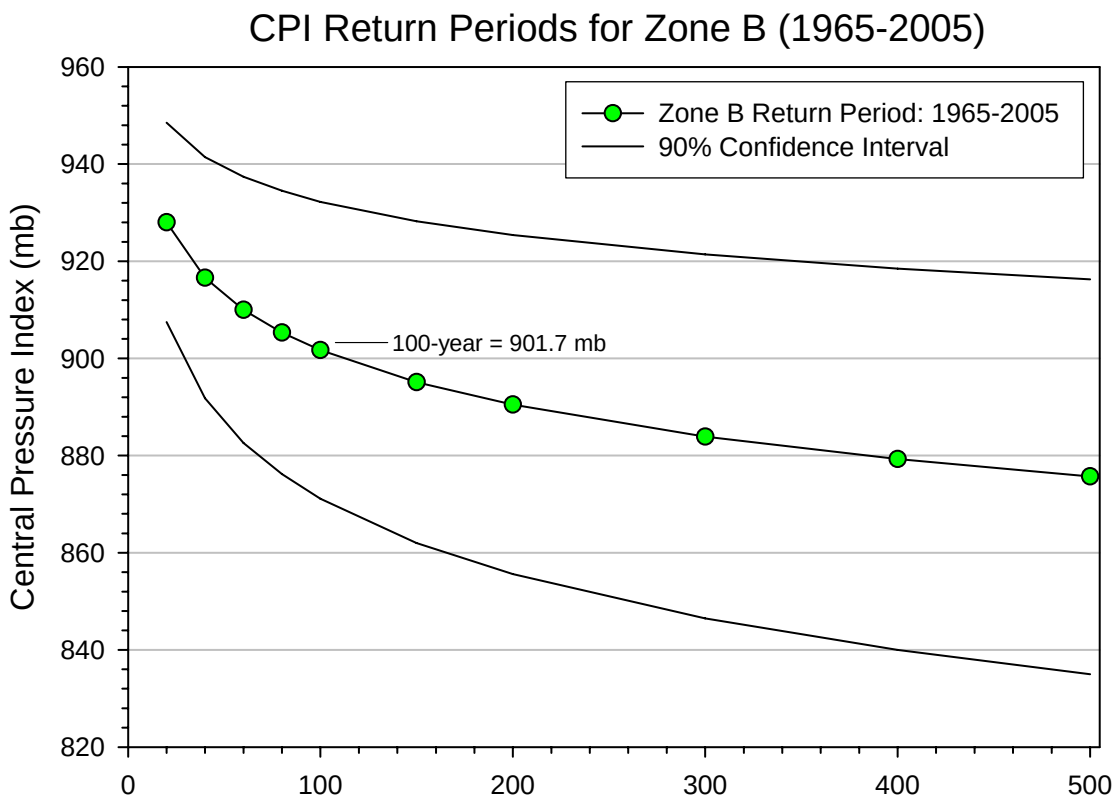

Figure 7. The return periods of the CPI in Zone B determined using GEV analysis (Gumbel 1941) and hurricane track data covering the period 1965-2005. Also plotted are the 90% confidence intervals determined for this analysis. Note that using this 41-year period of record resulted in a 1 in 100 year return period for the CPI in Zone B corresponding to 901.7 mb.

4.3 Results of Wind Speed Return Period Analysis:

An analysis of the return periods of maximum sustained winds (V_{max}) was also performed for the three zones along the Gulf of Mexico coast. The wind analysis was not specifically required by the USACE, but is useful in putting the analysis of the central pressure return periods into context. The method used to produce these results was the same as above for the CPI, but with maximum sustained wind data and covering the entire period from 1900 to 2005. These results are shown in Figure 8 below.

NOAA's National Hurricane Center estimated Hurricane Katrina's maximum winds at 140 kts, and using the results presented here is equivalent to about a return period wind speed of 1 in 125-year in Zone B. However, despite a slightly higher central pressure the estimated maximum winds associated with Hurricane Camille were even stronger than Katrina's, and according to the official archive reached 165 kts. This would result in a return period wind speed of greater than 1 in 500-years in Zone B.

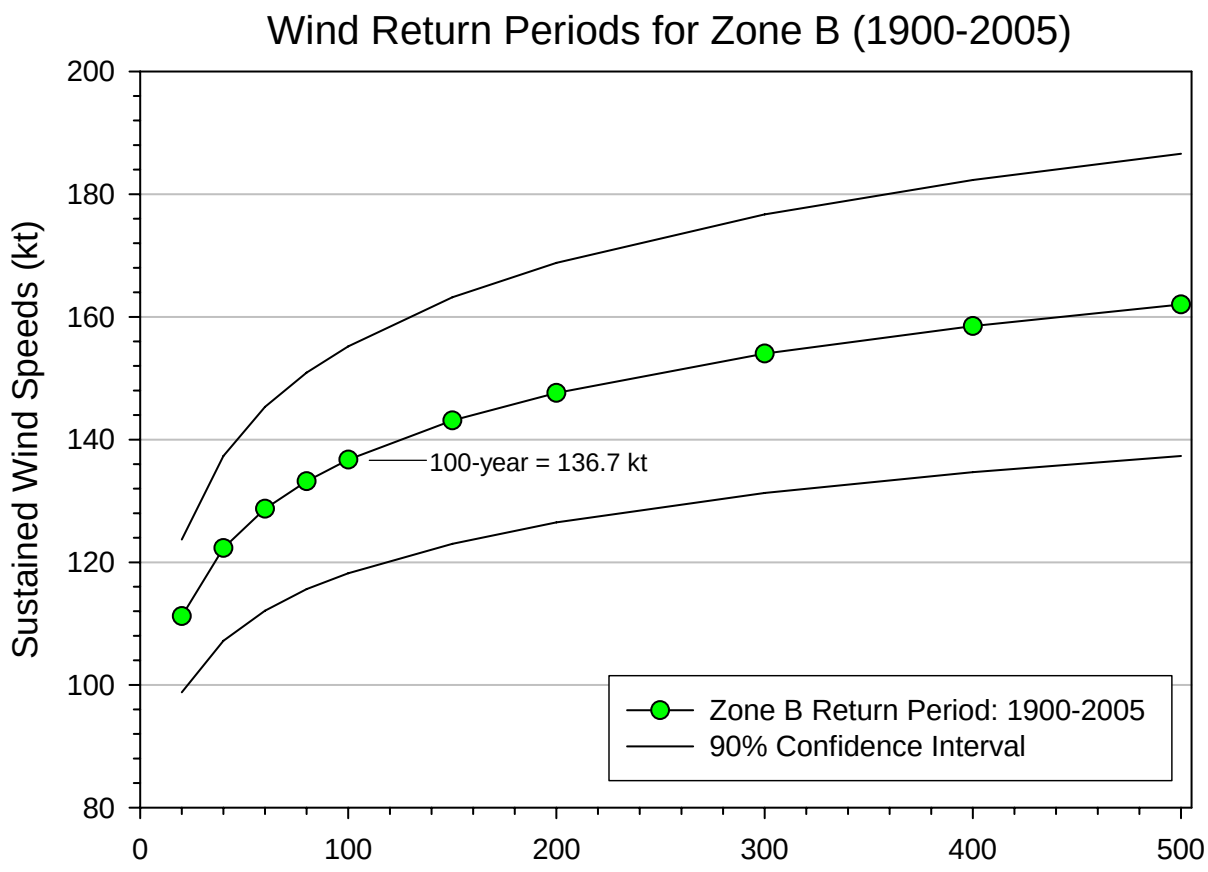

Figure 8. The return period for the sustained winds in Zone B determined using GEV analysis (Gumbel 1941) and hurricane track data covering the period 1900-2005. Also plotted are the 90% confidence intervals determined for this analysis. Note that using this 106-year period of record resulted in a 1 in 100 year return period for the sustained winds in Zone B that corresponded to 136.7 kt.

It has been widely reported that although Katrina weakened slightly before landfall, its coastal crossing location to the east of New Orleans definitely was significant and likely increased the storms impacts on New Orleans. As the hurricane's circulation was moving slowly northward, the N to NE winds that blew across Lake Ponchatrain were approximately 120 to 130 kt, and resulted in the longest possible fetch over the lake. Therefore, its location and the resulting wind fetch severely impacted the lake's south shore levees.

5. Summary:

This report summarized the results from a scientific study that reanalyzed the Standard Project Hurricane (SPH) indices, and specifically the Central Pressure Index (p_0) and peripheral pressure (p_w), along the central Gulf of Mexico coast for the U.S. Army Corps of Engineers. It should be noted that this work is ongoing, and the intent of this report was to present the analyses to-date, focusing on those results that were most complete and of greatest necessity to the U.S. Army Corps.

Detailed analysis of the historical tropical cyclone track data was used to determine the 1% (1 in 100-year) value of the CPI along the central Gulf of Mexico coast, in what is termed the "Zone B" region. Generalized Extreme Value (GEV) theory was used to analyze the 1 in 100-year CPI value for Zone B, and the results were verified using a variety of input data and GEV techniques. The verification was conducted both internally and externally to NOAA to ensure independent confirmation of the results.

There were two primary objectives related to the reanalysis of the SPH indices. First, the findings and methodologies used in the two original Technical Reports (DOC-NHRP Report No. 33 and NOAA/NWS Report No. 23) were reviewed, and comparisons generated showing how the original estimates of the CPI compared with those in the HURDAT reanalysis dataset, which is the official NOAA hurricane track archive for the North Atlantic basin. Second, updated analyses were generated for the 1 in 100-year values of the CPI and peripheral pressure estimates along the central Gulf of Mexico coast. These revisions and updates clearly showed that the 1 in 100-year CPI value had declined dramatically in comparison to the values published in Report 33 and Report 23 (Table 1 below). The significantly lower CPI was primarily due to observations from several strong hurricanes over the past 30-40 years, including Hurricane Katrina which had the lowest CPI in Zone B in the entire historical record.

Meteorological Parameter (units of inches of mercury)		*Report 33*	*Report 23*	*NOAA/NCDC Reanalysis*
Central Pressure Index	CPI or p_0	27.60 inches	27.35 inches	**26.627 inches**
Peripheral Pressure	p_w	29.92 inches	29.77 inches	**29.763 inches**

Meteorological Parameter (units of millbars)		*Report 33*	*Report 23*	*NOAA/NCDC Reanalysis*
Central Pressure Index	CPI or p_0	934.6 mb	926.2 mb	**901.7 mb**
Peripheral Pressure	p_w	1013.2 mb	1008.1 mb	**1007.9 mb**

Table 1. A comparison of the results for the Central Pressure Index (CPI or p_0) and the peripheral pressures (p_w) between the original Technical Report No. 33 (1959), the revisions published in NWS Technical Report No. 23 (1979), and the NOAA/NCDC Reanalysis presented in this preliminary report. The results are presented in two different units of pressure: top) inches of mercury, and bottom) millibars (mb).

6. Future Work:

In addition to the CPI and peripheral pressure, analyses are currently in progress related to the rate of overland filling associated with hurricanes along the central Gulf of Mexico coast in the historical record. The preliminary results from this part of the project will be submitted in a separate supplemental report to the U.S. Army Corps.

An additional effort has been recently proposed to analyze the CPI based on the distance-marker approach used in Report No. 23. To accomplish this task, digital location data of historical hurricane tracks will be input into ArcGIS software to produce the required analysis. Distance-marker and hurricane CPI (p_0) locations from Report No. 23 will be compared with the point data in the official NOAA archives. Replication of the original results will be produced over the 1900-1976 period, along with results data covering the period 1976-2005, in order to include those hurricanes that have occurred in the Gulf of Mexico over the past 30 years. The results from this study will also be submitted in a separate supplemental report to the U.S. Army Corps.

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ANNEX 7– SEA LEVEL RISE

APPENDIX L – ENGINEERING INVESTIGATIONS
ANNEX 7 – SEA LEVEL RISE

Variations and trends in the relationship between local mean sea level (LMSL) and land elevations are important considerations in the planning and design of structures in areas that are currently tidally influenced or that could become tidally influenced in the future. When we consider how to incorporate relative LMSL (RLMSL) changes in project planning and design, we have to recognize the differences between local and eustatic (world-wide) changes in mean sea level (MSL) and the differences between historic rates of change and future rates of change.

Changes in RLMSL can be due to many factors, including eustatic changes in MSL and local changes in land elevation. Eustatic MSL is commonly recognized to be rising, though the rate at which it is rising is difficult to determine. In its 1987 report, *Responding to Changes in Sea Level: Engineering Implications*, the National Research Council (NRC) stated that reasonably-based estimates of historic rates of eustatic MSL rise varied from 1.12 – 2.3 mm/year or 0.37 – 0.76 ft/century. While the rate of eustatic MSL rise has an effect on the rate of RLMSL change, it is often not the most important factor. Other activities that can effect RLMSL change include crustal subsidence or uplift; tectonic activity; human-induced subsidence from structural loading or groundwater, oil or natural gas extraction; auto-subsidence from consolidation of native sediments; and climatic fluctuations such as El Nino Southern Oscillation (ENSO).

Historic rates of RLMSL change around the United States are generally based on either geologic records or tide gauge records. Of these, tide gauge records are commonly considered to provide the more accurate measure of recent rates of RLMSL rise. In Louisiana three tide gauges provide records that extend back beyond 1959. These gauges are located at Grand Isle, Eugene Island, and Sabine Pass and have records extending from 1947, 1939, and 1958 (respectively) to the present. Rates of RLMSL rise for these three gauges were determined by the National Ocean Service (NOS)(2001) to be as shown in the following table.

Table 1. Linear RLMSL trends for monthly data up to 1999

Station Name	First Year	Year Range	MSL Trend and Standard Error (mm/yr)		Standard Error of Model (mm)
Grand Isle	1947	53	9.85	0.35	51.0
Eugene Island	1939	36	9.74	0.63	52.8
Sabine Pass	1958	42	6.54	0.72	72.6

These trends show a maximum historic RLMSL rise rate of 9.85 $\pm$ 0.35 mm/year (3.23 $\pm$ 0.11 ft/century) for southern Louisiana, based on historic tide records at these three tide gauges.

Future rates of RLMSL rise could be considerably different than the rates calculated by NOS. Due to concerns related to global warming, many people are predicting an acceleration in the rate of eustatic MSL rise, which would cause a corresponding acceleration in rates of RLMSL rise. The 1987 NRC study included a review of various estimates of eustatic MSL rise. This study provides three different curves showing three different scenarios for accelerations in eustatic MSL rise rates. These curves are shown below in figure 1 and represent scenarios that result in eustatic MSL rising 0.5 meters, 1.0 meters and 1.5 meters between the years 1986 and 2100.

Estimating rise in eustatic MSL based on these curves requires one to use the following equation

$$E(t) = 0.0012t + bt^2$$

in which $E(t)$ is the eustatic MSL rise for a given period of time starting in 1986, t is the number of years from 1986, and b is a coefficient that is different for Curves 1, 2, and 3. Accounting for the period of time between 1986 and 2006 (the present), and using the equations for Curves 1, 2,and 3, results in eustatic MSL rise rates for various periods of time as shown below in Table 2.

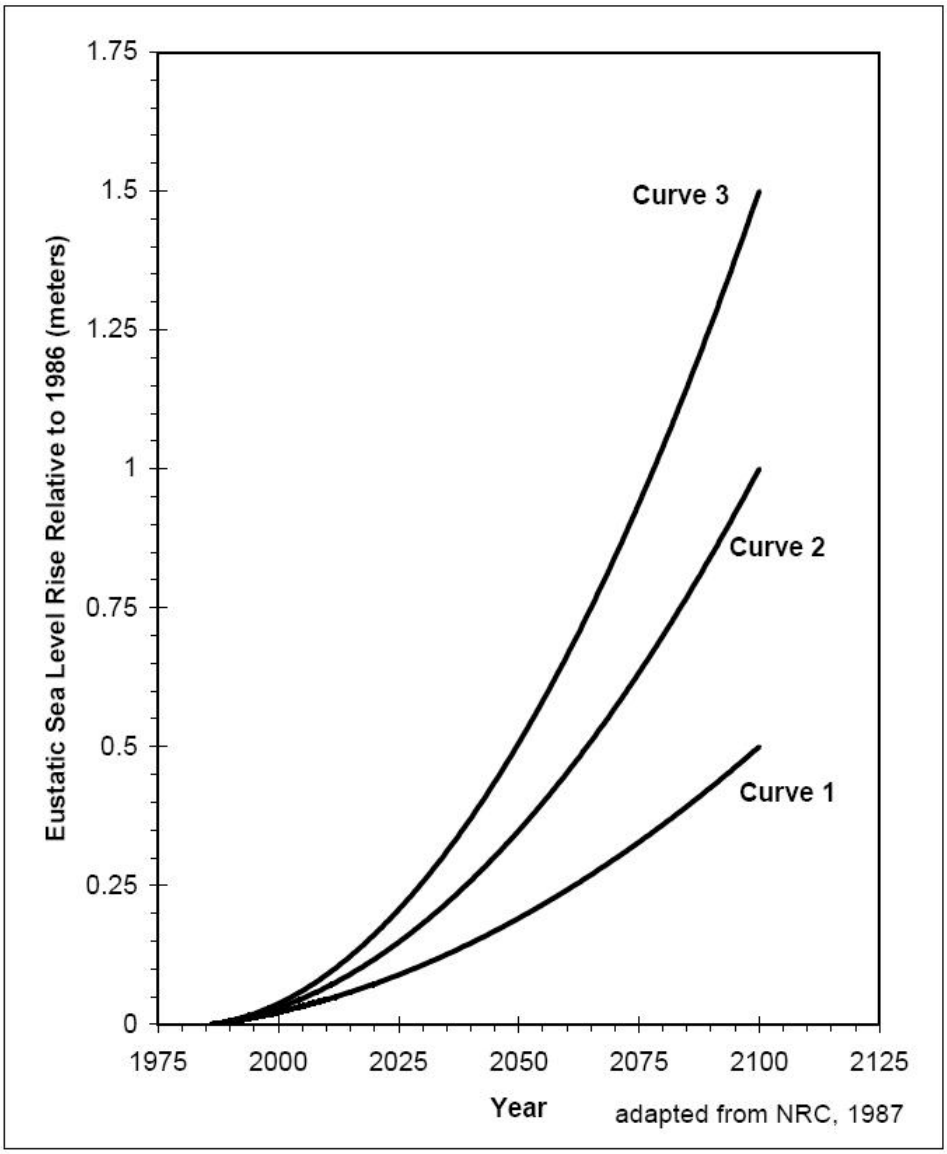

Figure 1. NRC accelerated eustatic MSL rates

Table 2. Eustatic MSL rise between 2006 and various future years, based on 1987 NRC report				
Year	Years from 2006	Eustatic MSL (meters)		
		Curve 1	Curve 2	Curve 3
2011	5	0.0124	0.0212	0.0301
2016	10	0.0263	0.0459	0.0655
2021	15	0.0415	0.0739	0.1062
2026	20	0.0582	0.1052	0.1523
2031	25	0.0764	0.1400	0.2037
2036	30	0.0959	0.1782	0.2605
2041	35	0.1169	0.2197	0.3226
2046	40	0.1393	0.2646	0.3901
2051	45	0.1632	0.3130	0.4629
2056	50	0.1884	0.3647	0.5411
2061	55	0.2151	0.4197	0.6246
2066	60	0.2432	0.4782	0.7134
2071	65	0.2728	0.5401	0.8076
2076	70	0.3038	0.6053	0.9071
2081	75	0.3362	0.6739	1.0120
2086	80	0.3700	0.7459	1.1222
2091	85	0.4052	0.8213	1.2378
2096	90	0.4419	0.9001	1.3587
2101	95	0.4800	0.9823	1.4850
2106	100	0.5196	1.0678	1.6166

Corps of Engineers policy, as described in ER 1105-2-100 (Planning Guidance Notebook dated 22 April 2000), states that Corps of Engineers projects should consider potential sea level change "as far inland as the new head of tide." This policy also states that the effect of varying rates of relative sea level rise on engineering designs should be based on a sensitivity analysis that includes (as a minimum) using historical rates as the low level of relative sea level rise and Curve 3 from the 1986 NRC report as the high level of relative sea level rise. Other rates of local relative sea level rise may also be considered and used in the sensitivity analysis.

For the Louisiana Coastal Protection and Restoration project, this policy will be used as guidance for how to incorporate future relative sea level change in project design. The historic values for RLMSL change at the three tide gauges listed in table 1 will be updated and other tidal datum trends at these three tide gauges will also be examined. The possibility of historical data being available for additional locations will be considered and investigated. Based on proximity to the tide gauges with significant periods of record and geologic conditions, historic rates of relative sea level change will

be assigned to all areas of southern Louisiana. These rates will be extrapolated to obtain design water levels for the low end rate of relative sea level change. The values for the NRC Curve 3 in table 2, as shown above, will be used to obtain a design water level for the high end estimate of relative sea level change. Other possible rates will be considered after examining more recent research in the areas of global climate change and coastal Louisiana geology.