Sabine Pass to Galveston Bay, Texas
Coastal Storm Risk Reduction and Ecosystem Restoration
Draft Integrated Feasibility Report and Environmental Impact Study

Draft Appendix D

Engineering Design, Cost Estimates, and Cost Risk Analysis

September 2015

US Army Corps
of Engineers
Galveston District

Sabine Pass to Galveston Bay, Texas
Coastal Storm Risk Management
and Ecosystem Restoration

Engineering Appendix

for

Draft Integrated Feasibility Report – Environmental Impact Statement

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ATTACHMENTS - ENGINEERING DESIGNS AND MAPS

List of Acronyms

ADCIRC	ADvanced CIRCulation (Model)
ADM	Agency Decision Milestone
AEP	Annual Exceedance Probability
ASTM	American Society for Testing and Materials
ATR	Agency Technical Review
CFR	Code of Federal Regulations
bpd	barrels per day
cfs	cubic feet per second
CPT	cone penetration test
CSRM	Coastal Storm Risk Management
DIFR-EIS	Draft Integrated Feasibility Report and Environmental Impact Statement
DOWSMM	Desktop Off-Channel Wetland Salinity Mitigation Model
EC	Engineering Circular
EM	Engineering Manual
EoC	End of Construction
EPA	U.S. Environmental Protection Agency
ER	Engineering Regulation
ERDC-CHL	Engineer Research and Design Center - Coastal Hydraulics Laboratory
ETL	Engineering Technical Letter
FEMA	Federal Emergency Management Agency
FM	Farm-to-Market Road
FoS	Factor of Safety
FRP	Fiber-Reinforced Polymer
FWOP	Future Without-Project
GIWW	Gulf Intracoastal Waterway
GIS	Geographic Information System
GLO	General Land Office
gpm	gallons per minute
HEC	Hydrological Engineering Center
H&H	Hydraulics and Hydrology
HFPP	Hurricane Flood Protection Project
HH&C	Hydrologic, Hydraulic and Coastal
HPTRM	high performance turf-reinforcement mattress

HQ-USACE	Headquarters - United States Army Corps of Engineers
HSDRRSDG	Hurricane and Storm Damage Risk Reduction System Design Guidelines
IPET	Interagency Performance Evaluation Taskforce
ksf	kips per square foot
ksi	kips per square inch
lf	lineal or linear feet
LiDAR	Light Detection and Ranging
LSAC	Levee Safety Action Classification
MII	Micro-Computer Aided Cost Estimating Software, Second Generation
MLLW	Mean Lower Low Water
MLRA	Major Land Resource Area
NAD	North American Datum
NAVD	North American Vertical Datum
NED	National Economic Development
NFIP	National Flood Insurance Program
NOAA	National Oceanic and Atmospheric Administration
O&M	operations and maintenance
pcf	pounds per cubic foot
PDT	Project Delivery Team
PED	Preconstruction Engineering and Design
PI	Periodic Inspection
PL	Public Law
POR	Period of Record
RMC	Risk Management Center
ROW	Right-of-Way or Rights-of-Way
RP	Rehabilitation Program
RSLC	relative sea-level change
RTK	Real Time Kinematic
SLC	Sea Level Change
SMART	Specific, Measurable, Attainable, Risk-Informed, and Timely
SPF	Standard Project Flood
SQRA	Semi-Quantitative Risk Assessment
SWE	Still Water Elevation
SWG	Southwestern Division, Galveston District
SWIF	system-wide improvement framework
SWL	Still Water Level

TCOON	Texas Coastal Ocean Observation Network
TNRIS	Texas Natural Resources Information System
TRRC	Texas Railroad Commission
TSP	Tentatively Selected Plan
USACE	United States Army Corps of Engineers
USGS	U.S. Geological Survey
UTM	Universal Transverse Mercator
VE	Value Engineering
VT	Vertical Team
WPG	Water at Project Grade

1 GENERAL

1.1 PURPOSE AND CONTEXT

This Engineering Appendix documents the preliminary engineering analyses and concept designs for the coastal storm risk management (CSRM) projects comprising the Tentatively Selected Plan (TSP), specifically for implementation within the Sabine and Brazoria study regions along the upper Texas coast. It supports the viability of the proposed projects, which are developed in the "Sabine Pass to Galveston Bay, Texas Coastal Storm Risk Management and Ecosystem Restoration" Draft Integrated Feasibility Report – Environmental Impact Statement (DIFR-EIS).

The Sabine and Brazoria regions comprise a broader coastal study area in southeast Texas that encompasses six counties. These two sub-regions are the subject of the first interim study where CSRM opportunities were formulated. Additional interim studies are planned for the future to investigate other opportunities where structural alternatives could be implemented to lower the risk of storm surge flooding within the delineated Galveston region and throughout the entire study area. Assessments of the existing and predicted storm surge climate conditions for the Sabine and Brazoria regions, and the concept designs for the proposed project features that came out of these assessments, factor into the economic analyses and are the basis for the costs of the alternatives that were compared during plan formulation. Thus, the preliminary engineering analyses validate the technical and economic viability of the projects from an engineering standpoint.

1.2 SCOPE OF EFFORT

The engineering design work is premised on a feasibility level of detail and analysis, consistent with the SMART planning process that is minimally necessary to substantiate the TSP baseline cost estimate. Thus, an appropriate level of engineering work was done in comparing alternatives during plan formulation. To the maximum extent possible, existing information that could be readily acquired was used to develop the plans and designs for the project features. Sources of available information included maps and imagery from the Federal Emergency Management Agency (FEMA), the U.S. Geological Survey (USGS), and Google; geospatial data; present and historical data from FEMA, the USGS, and the National Oceanic and Atmospheric Administration (NOAA); similar or related area studies; comparative studies; operations and maintenance (O&M) records; and damage risk assessment reports on existing systems. Limited field investigations conducted at a sufficient level of detail to validate the existing data and to provide more localized site-specific data, such as core borings to evaluate existing foundation conditions, complemented this information. Relevant technical references, empirical design guidance, and numerical modeling programs were also used in the engineering analyses.

The processed technical data for the analyses that were done, although adequate to assess project feasibility, are not complete and precise enough in detail to design the project features with the appropriate level of confidence; therefore, it should not to be construed as such. Further engineering analyses and refinements to the feature designs will be made after the Agency Decision Milestone (ADM) to more accurately assess the project's construction cost. The TSP cost presented in this appendix was developed only to the extent needed to equally compare alternatives in deciding the TSP.

1.3 ENGINEERING DOCUMENTATION

Narratives of the engineering analyses are broken out by discipline covering Hydrology and Hydraulics, Surveying and Mapping, Geotechnical engineering, Structural engineering, and Civil Design. The engineering effort level of detail and report content and format are consistent with guidance provided in ER 1110-2-1150. In general, the objective of this appendix is to provide enough supporting documentation to enable reviewers to understand the assumptions and models used to evaluate project benefits and to develop the costs necessary to derive the TSP projects.

The technical sections discuss the development of the preliminary designs for the CSRM features comprising the plans evaluated in the final array of alternatives. They detail the engineering information that was collected, design references and guidance used, computer programs used, the design criteria assumed, design parameters, assumptions made, and methods of analyses.

1.4 PROJECT AREAS

The discrete projects making up the first interim study for the Sabine and Brazoria sub-regions are in, or will be located in Orange, Jefferson, and Brazoria Counties, Texas, along the upper Texas coast (Figure 1-1). These counties are constituent counties of the larger six-county study area between Sabine Pass and the City of Freeport, Texas, that includes Galveston, Harris, and Chambers Counties.

The first interim study specifically looks at CSRM opportunities within the following defined areas:

- Orange-Jefferson County, Texas. Areas within the southern half of Orange County bounded by the Sabine and Neches Rivers and areas along the northeastern boundary of Jefferson County along the Neches River. Specific municipalities, industrial areas, and commercial facilities of interest within Orange County include the cities of Orange, West Orange, Pinehurst, Bridge City, Rose City, and Vidor, as well as surrounding unincorporated areas, an area known as "Chemical Row" along F.M. 1006, and Entergy Texas' Sabine Plant. In Jefferson County, specific areas include the cities of Port Neches and Nederland and industrial complexes along the west bank of

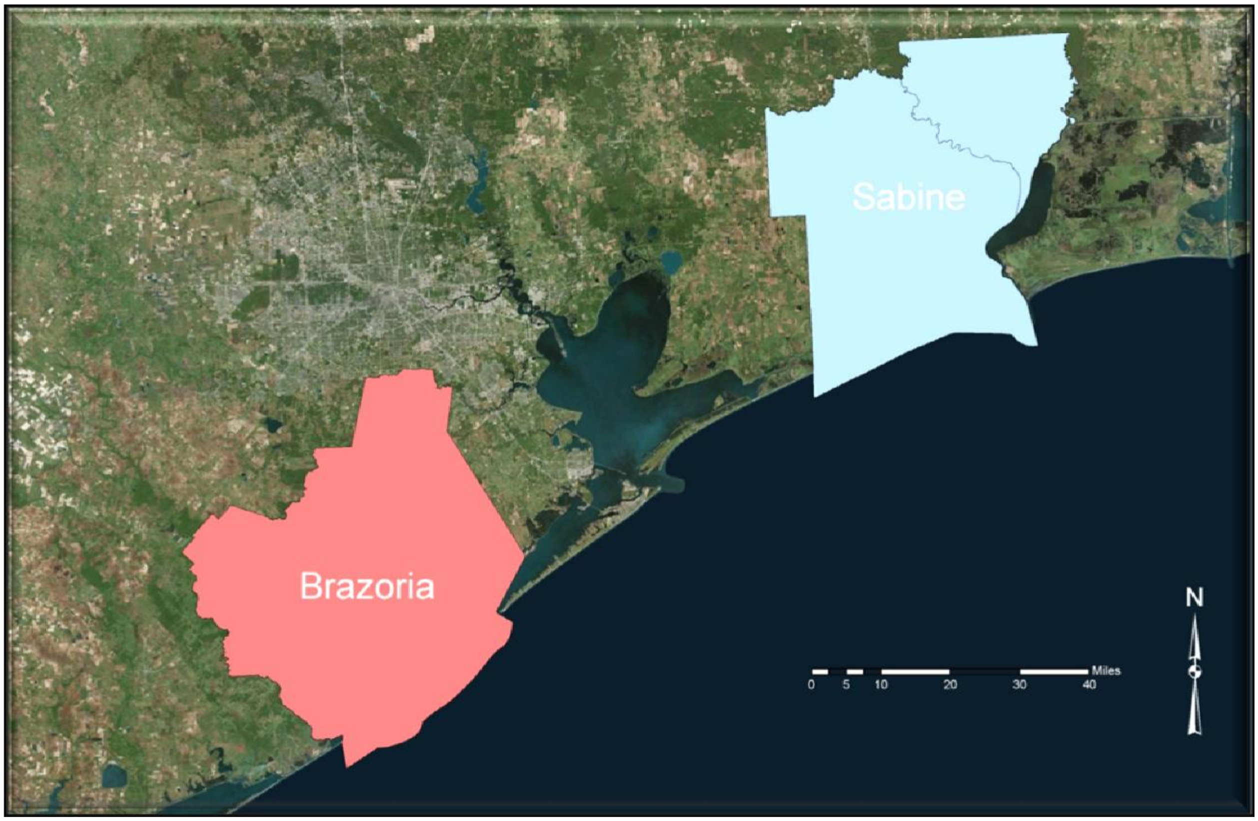

Figure 1-1: Sabine and Brazoria Regions encompassing Orange, Jefferson, and Brazoria Counties

the Neches River, which include the Du Pont Beaumont Works Industrial Park and ExxonMobil's Beaumont Chemical Plant.

- Port Arthur and Vicinity, Jefferson County, Texas. An existing HFPP protects the Port Arthur area from coastal storm surge events coming from the Gulf of Mexico. It also protects against flooding from the Sabine River. The levee system consists of 27.8 miles of earthen embankment and 6.6 miles of floodwall, including 3.5 miles of cantilever wall. There is also a wave barrier on Pleasure Island.

- Freeport and Vicinity, Brazoria County, Texas. An existing HFPP protects the Freeport area from coastal storm surge events coming from the Gulf of Mexico. The line of protection includes multiple structures that also serve as control structures and docks for the Dow Chemical Company, BASF, Conoco Philips, ExxonMobil, and Port Freeport. The system consists of approximately 43 miles of levees and wave barriers, seven pump stations, multiple gates, culverts, and related appurtenances.

1.5 PROJECT OVERVIEW

1.5.1 Project Need

A thorough discussion of the need for the CSRM projects is in Section 4 of the DIFR-EIS – Problems and Opportunities.

Approximately 2.26 million people within the six-county study area reside in areas subject to flooding from storm surge. The population within the study area is currently more than 5 million people and projected to grow to over 9 million people within the next 50 years. Industrially, this area encompasses three of the nine largest oil refineries in the world, 40 percent of the nation's petrochemical industry, 25 percent of the nation's petroleum-refining capacity, and three of the 10 largest U.S. seaports. The damages to the national economy that could be incurred from a one-month closure of the Houston Ship Channel alone are estimated to be upwards of $60 billion. It contains six major rivers and their watersheds, a nationally significant coastal estuary, a commercial fisheries industry, and tourism industry. Given the area's large population, nationally significant industries and facilities, and ecosystems that potentially could be harmed or damaged by storm surge flooding, governmental entities and interests over the years have identified and looked into possible CSRM opportunities in the context of local and regional strategies to address this.

Between 1900 and 2014, more hurricanes struck the upper Texas coast than along any other reach of the Texas coastline, which itself accounts for the second greatest total number of recorded hurricane strikes on the United States coastline, historically, behind only Florida. The probability that a hurricane will strike somewhere along the upper Texas coast in any given year ranges from 31 percent at Sabine Pass to 41 percent around Matagorda Bay. On average, the upper coast will see a major hurricane strike (Category 3 and greater) within a 6-year time frame (*source*: The Formation and Future of the Upper Texas Coast, John B. Anderson, published May 2007).

1.5.2 Project Objective

The tentatively recommended projects (for the Sabine and Brazoria sub-regions) seek to reduce the risk of potential flood damage from coastal storm surge to large at-risk population areas, public and industrial facilities and infrastructure, critical industries in the energy sector having tremendous implications on the economy and national security, and sensitive ecosystems along the Orange, Jefferson, and Brazoria County coastlines. Through the implementation of flood-protection measures, the risks to life safety and property damage associated with coastal storms can be minimized. From a purely economical standpoint, the level of protection sought is that which optimizes the costs of project construction relative to the potential reduction in economic losses that could be incurred over the 50-year period of economic analysis to derive the greatest net economic benefit. To this end, the economic damage assessments focus only on direct damages to structures. Incidental to reducing the life safety and property damage risks, opportunities were also looked into that preserve and enhance existing environmental habitats or reduce the damage potential to these habitats from storm surge flooding. The project areas covered by this appendix are not inclusive of all the study areas along the upper Texas coast where storm surge flooding is an issue (i.e., areas within Galveston, Harris, and Chambers Counties that are subject to storm surge inundation).

1.5.3 Project Coordination

The engineering assessments, analyses, and preliminary designs for the existing and proposed HFPPs were thoroughly coordinated and vetted with the Engineering Research and Development Center – Coastal Hydraulics Laboratory (ERDC-CHL), the Texas General Land Office (GLO), and other governmental agencies, such as FEMA, the EPA, representatives of Drainage District No. 7 (Port Arthur) and the Velasco Drainage District (Freeport), and county officials. Planning efforts were also coordinated with resource agencies and communities having an interest in the CSRM projects and with the Port of Orange. The interdisciplinary PDT collaborated with subject matter expert colleagues and seamlessly engaged the Vertical Team (VT - USACE District, Division, Headquarters, and Office of Water Project Review) in its efforts throughout the plan formulation process. Agency Technical Reviews (ATRs) and In-Progress Reviews were conducted at key development stages.

1.5.4 Project Background

The USACE, Galveston District, has been aware of the region's need to reduce the risk of flood damage from storm surge and the varied opportunities available in addressing the need given the size of the population at risk, importance of existing public and industrial facilities and infrastructure, and critically important ecosystems that need to be preserved. Consequently, the district, within USACE's Congressional authorization, explored possibilities of building projects that could benefit the region where there might also be a Federal interest in doing so, consistent with an integrated and coordinated approach to locating and implementing opportunities for CSRM and ER. With this as the objective, the district collaborated with the State of Texas and local governmental entities. Through such collaboration and pooling of resources, challenges can be more effectively met. Others have already laid much of the groundwork in evaluating the feasibility of pursuing specific new projects that can reduce flood damage from coastal surge. Considering this, the district leveraged studies, data, and models that have already been produced or are under development.

The DIFR-EIS goes into detail concerning the project's background and describes the focus areas that were advanced from the planning charrettes that were held to scope the feasibility study, along with discussion of the associated non-structural and structural measures that were considered to address CSRM. The focus areas for the Sabine and Brazoria regions and the structural alternatives formulated for these areas by combining selected measures (which are the bases for the TSP projects) are summarized in the paragraph entitled Design Analyses Process, below.

1.5.5 Project Datum

The horizontal and vertical datums used in the engineering analyses and models conform to the current Federal standard. Horizontal coordinates are referenced to North American Datum (NAD) of 1983. Elevations for upland areas and proposed upland features are referenced to the North American Vertical Datum of 1988 (NAVD 88). Water depths are referenced to Mean Lower Low Water (MLLW), unless otherwise stated.

1.6 GENERAL ASSUMPTIONS

1.6.1 Design Considerations

Currently, there is no Federal standard to provide a specified minimum level of flood protection. Historically however, the USACE goal for the design of flood-damage reduction projects protecting major urban areas was to accommodate the Standard Project Flood (SPF) or other "design floods." The SPF is defined as those "discharges that may be expected from the most severe combination of meteorological and hydrologic conditions that are considered reasonably characteristic of the geographical region involved, excluding extremely rare combinations." This goal was rarely achievable though, because the projects had to be economically justified as well and sizing them for SPF protection often was not economical. Although the SPF may have a useful role for application in a risk analysis approach to evaluating flood protection alternatives, it is no longer a valid design target in the context of ER 1105-2-101 (dated 03 January 2006) and risk analysis guidance, having been superseded by guidance that is more current. Instead, a full range of floods, including those that would exceed the SPF, is to be used in the formulation and evaluation of alternatives.

Since the early 1990s, USACE policy has mandated a balanced approach in planning flood-damage reduction projects in which the flood-damage reduction potential (i.e., physical performance), economic success, and residual risks associated with a particular alternative are assessed relative to its implementation cost. Accordingly, the total effect of uncertainty on the project's design and economic viability is examined and conscious decisions can be made that have taken into account the explicit tradeoff between risks and costs. Alternatives are evaluated and compared based on the scale of the flood damage reduction that reasonably maximizes the expected net benefits (expected benefits less expected costs). Those alternatives accomplishing this objective, with uncertainties in the key variables explicitly included, define the National Economic Development (NED) plans.

Increments in project scale beyond the NED plan can be considered to improve project performance and to manage residual risks to people and property, but existing policy governing such increments must be followed. For example, levee heights are often compared to that which would satisfy the FEMA National Flood Insurance Program (NFIP) base flood level for excluding the floodplain from mandatory flood insurance. In those urban settings where risk-based economics suggest a levee height less than the NFIP base flood protection, the heights are generally increased to that level so that the levees may be certified for NFIP purposes. The resulting project must still be economically justified and the non-Federal sponsor may be required to pay the cost for the increment of levee height between the NED plan project and the NFIP base flood protection project.

The flood-damage reduction potential for the various alternatives evaluated was assessed based on those damages that would be prevented under average still-water conditions only, without provision for relative sea level change (RSLC), wave set-up and run-up, and overtopping. Storm surge modeling results combined with existing information collected was used to determine the areas that would be inundated from storms with return periods varying from 10 to 1,000 years. Therefore, the economic viability of the TSP, to the point that it has been developed, is predicated only on the flood damages that would be prevented from the rise in water elevation due to storm surge. As the project features are further developed, they likely will be raised to accommodate future sea-level rise, wave run-up, and overtopping to the extent practicable. For the existing HFPPs at Port Arthur and Freeport however, the amount of raising that can be done practically is economically constrained by existing structures and facilities (e.g., floodwalls, gate structures, drainage structures, pipelines, utilities, pump stations), whose heights or capacities would have to be increased accordingly. Enlargement of the structures will increase the costs of the affected projects comprising the TSP.

1.6.2 Relative Sea-Level Change

RSLC is an important variable to consider in the design of HFPP structures, so much so that USACE policy requires its consideration in every USACE coastal activity as far inland as the extent of estimated tidal influence. This is because sea-level change (SLC) can potentially affect project and system performance. Therefore, the plans and designs for prospective projects, with

respect to SLC, need to consider how sensitive and adaptable human and engineered systems and natural and managed ecosystems are to climate change and other related global changes.
ER 1100-2-8162 requires that planning studies and engineering designs over the project life cycle, for both existing and proposed projects, consider a range of possible future rates of SLC when formulating and evaluating alternatives. This includes both structural and non-structural solutions. For the structural alternative comparisons, it was assumed that the applied range of future SLC would be the same across the system; therefore, there was no need to include or incorporate this criterion in the screening process. The structural alternatives compared are proposed modifications to the existing Port Arthur and Freeport HFPP levees and floodwalls and proposed new levees and floodwalls that would comprise the Orange-Jefferson County HFPP, which includes the Beaumont "A" component. For each system, the principal emphasis regarding the economics of the structural alternatives, for the different still-water levels (SWLs) assumed, was on the reaches and heights of protection that would yield the greatest net excess benefits. To this end, the existing systems and initially laid-out proposed levee system alignments were broken out into segments based on the areas that would be protected and existing topography. These separable elements constituted the structural alternatives that were evaluated to derive the TSP. Including sea-level rise in the alternatives analysis would not have affected the outcome of the plan selection process. SLC is not accommodated in the TSP concept designs as of yet, but is a consideration the PDT recognizes needs to be addressed in the project. This will be done when the TSP is further developed.

Although RSLC was not a determinant factor in deciding TSP, it was investigated nevertheless and is covered in the H&H section of this report. H&H's specific recommendations on how to address future RSLC in the project designs will be considered in developing the feature designs as they evolve. Those recommendations are to consider anticipatory and adaptive strategies to address or accommodate sea level rise. An anticipatory strategy would be to construct project features, including resiliency features, that would be capable of handling the maximum projected seal level rise ("high" scenario) over a given time horizon. An adaptive strategy, on the other hand, would be to construct project features lower than the maximum projected sea level rise at the "low" or "intermediate" scenario for the rate of change and adapting (i.e., raising the structures or building in resiliency) the features accordingly should the projected "low" or "intermediate" rate of change be greater than estimated. When designing the levees, floodwalls, and resiliency elements, both the 50-year and 100-year time horizons will be considered for RSLC. It may be that accommodating for the projected 50-year "high" scenario for sea level rise, for example, could be done at modest extra cost on a comparative basis, which would provide more accommodation than needed for the "low" scenario for the 100-year projected sea level rise. Adaptive strategies could be employed to address higher change potential for sea-level rise over the project's service life, which is assumed to extend well beyond the 50-year period assumed for the economic analysis. The project features will be optimally designed based on the projected ranges of RSLC combined with CSRM objectives and economic considerations.

1.7 DESIGN ANALYSES PROCESS

The preliminary design process utilizes design parameters and calculation methods described in the respective technical discipline sections of this Appendix. The feature designs presented are schematic in nature and represent systems and components currently in use in the United States and globally. The level of detail of the designs is sufficient to develop order of magnitude cost estimates suitable for the benefit-to-cost analysis in comparing the alternative plans. Primary features considered for the protection systems are:

- Earthen Levees
- Concrete Floodwall (T-Wall)
- Rights-of-Way Acquisitions
- Highway/Roadway Crossings - Non-Gated
- Pipeline and Major Utility Crossings
- Pump Stations
- Closure Gate Structures – Navigation
- Roadway and Railway Crossings – Gated
- Interior Drainage
- Resiliency Features

1.8 DESCRIPTION OF EXISTING PROJECT AREAS

1.8.1 Orange-Jefferson CSRM

The project areas in Orange and Jefferson Counties are characterized by nearly flat terrain that slightly dips towards the Gulf of Mexico, typical of the Gulf Coastal Plain in Texas. The general land slope is 0.05 percent and less with elevations ranging from 0 to 20 feet NAVD 88. Most of the area is less than 10 feet in elevation with a great percentage being less than 5 feet. Much of the areas are fronted by coastal inland marshes. Thus, use of the areas by wildlife is considerable. The major drainage systems in Orange County are the Neches River in the western part and the Sabine River in the eastern part. On the Jefferson County side, the major drainage system in the county is the Neches River. Both rivers, and other minor drainage systems, empty into the Gulf of Mexico at Sabine Lake. Noteworthy of the areas is that they are highly susceptible to rising sea level, with studies suggesting that regional relative sea-level rise rates, inferred from historical tide gauge data, have been significantly higher than presumed eustatic rise rates. A speculative reason as to why this may be is the down-warping of the earth's crust by sediment loading in the northern Gulf of Mexico.

Most of the areas beyond the coastal marshes and river floodplains are highly developed as evidenced by the several cities, industrial plants, commercial facilities, residential areas, dock facilities, and agricultural-use areas that depict the area. These cities include Orange, West Orange, Pinehurst, Bridge City, and Port Neches. Presently, no flood protection systems of significance are known to exist that would protect even portions of Orange County and the northeastern part of Jefferson County from storm surge inundation. The ExxonMobil Chemical Plant in Beaumont is protected by a barrier that was recently built after Hurricane Ike (2008) to replace a flood protection system that had failed during that storm. Figure 1-2 shows the project areas for the Orange-Jefferson CSRM.

Figure 1-2: Existing Conditions within Orange-Jefferson CSRM Project Area

1.8.2 Port Arthur and Vicinity CSRM

The Port Arthur and Vicinity Hurricane Protection Project (HFPP) protects the Port Arthur region from tropical storm events coming from the Gulf of Mexico. It also protects from riverine loading from the Sabine River. It is operated by Jefferson County Drainage District No. 7. This project was authorized by Flood Control Act of 1962, Public Law 87-874. Construction began in March 1966 and was completed in April 1982. The levee system consists of 27.8 miles of earthen embankment and 6.6 miles of floodwall. This includes 3.5 miles of coastal cantilever I-wall. The storm of record is Hurricane Ike (2008). This storm produced wave action that overtopped sections of floodwall adjacent to the Sabine-Neches Waterway (a deep-draft navigation channel whose authorized depth is 40 feet) for approximately 30 minutes. Figure 1-3 shows the project area for the Port Arthur and Vicinity CSRM.

Figure 1-3: Existing HFPP within Port Arthur and Vicinity CSRM Project Area

The Port Arthur and Vicinity project area has a preliminary Levee Safety Action Classification (LSAC) that has resulted in the Risk Management Center (RMC) initiating a Semi-Quantitative Risk Assessment (SQRA) to better define the systems risk. This classification was primarily driven by three main risk factors (probability of load, probability of failure, and nature of the consequences). The following paragraphs list the major engineering concerns for the Port Arthur and Vicinity CSRM:

I-wall stability analysis show concerns for performance during events that are at or above the top of the level of protection. These concerns are due to marginal factors of safety for stability and excessive anticipated deflection. Additional I-wall concerns are the lack of resiliency features for overtopping, no design considerations for vessel impact and embankment tie-ins that are susceptible to erosion during an overtopping event. Sections of T-wall type floodwall that were overtopped during Hurricane Ike experienced erosion and loss of supporting fill material due to as little as 30 minutes of overtopping. These T-wall sections are in the vicinity of the I-wall sections that are being evaluated under this study. It is expected that the foundation material at the I-walls will erode in a similar manner.

The Port Arthur system begins to experience overtopping at around a 150-year hydraulic loading event (0.67% ACE). This is a fairly frequent event for still water overtopping, wave overtopping will occur at more frequent events as was observed during Hurricane Ike. The Port Arthur region has a population of around 90,000 in the leveed area along with a large refinery presence (over one million barrels per day (bpd) production). This includes Motiva's 600,000 bpd plant (2012) as well as 60 percent of the nation’s jet-fuel production capacity. High economic consequences along with a large population, together with a high-return frequency for overtopping events that could lead to a catastrophic failure of the system, heavily influenced the LSAC rating.

Currently the system is minimally acceptable in the Rehabilitation Program (RP) under PL 84-99 and certifiable for FEMA accreditation under CFR 65.10, so the local sponsor does not have any plans to address the risk drivers for the LSAC. A Periodic Inspection (PI) was completed for the Port Arthur system in 2012. The Non-Federal Sponsor was provided a list of items to correct and is currently in the process of correcting them. The future without project conditions would result in no action being undertaken to reduce the risk that the system would suffer a catastrophic failure during a future hydraulic loading roughly equivalent or slightly prior to a 150-year event.

Fragility curves were developed for specific locations along the Port Arthur system to account for the anticipated system performance at those locations and were used to scope the reconstruction and resiliency features for the existing system. These curves were developed by the SWG Geotechnical and Structural Engineering section using existing information. Existing data used included performance history based on previous hydraulic loadings for the system, draft findings of the Freeport SQRA, draft findings of the Texas Coastal I-wall study, FY12 periodic inspection and Acradis I-wall analysis, USACE work on erosion for Herbert Hover Dike, Interagency Performance Evaluation Taskforce (IPET) report, ETL 575 criteria, along with influence from the LSAC screenings, the Hydrologic Engineering Center (HEC) Risk Analysis for Flood-Damage Reduction Projects, and RMC Internal Erosion Workshop. The curves focused on anticipated structural performance of the I-walls along with erosion at tie-in locations and reflect uncertainty in storm duration, size, landfall location, and wind-driven wave height.

The alternative selection process focused on selecting structural features that would significantly reduce the likelihood of a catastrophic failure of the system under extreme hydraulic loading. Features that would allow an increase in the level of protection were also evaluated. These alternatives consisted of overtopping erosion protection along with additional structural features to increase stability along with entire reconstruction/reconfiguration of the I-walls to T-walls. The additional structural features are a system of batter piles and walers along with stiffeners to provide the additional structural capacity to allow the system to be overtopped without catastrophic failure of the wall sections. Erosion control features would be constructed as needed to reduce the likelihood of significant erosion leading to system failure from an overtopping event.

TSP implementation will require additional project scoping to ensure the CSRM project functions as a system. This additional work will include assessing the entire existing system to ensure that the system is able to perform under the anticipated hydraulic event chosen for the TSP. Additionally the system will be evaluated and configured to accommodate relative sea-level rise as needed.

Risks associated with the TSP for the existing system are primarily situated with the use of existing geotechnical and structural information. Given the extensive work for the initial construction, along with additional information obtained by the Non-Federal Sponsor for 65.10 work, this risk should be manageable.

The draft findings of the SQRA for the Freeport system (discussed next in the section on Freeport) were applied to the plan formulation for the Port Arthur because the Port Arthur SQRA has not yet been done. For the Port Arthur system, the detailed description of the needs is similar to what will be presented in the Freeport HFPP section. However, the Port Arthur system is different because there are no known deferred maintenance issues related to this system at this time and general levee and foundation conditions at Port Arthur are less influenced by seepage concerns.

The formulation of alternatives for the Port Arthur and Vicinity CSRM began with defining reaches for the system. These were based on the failure locations identified by the levee safety program in the absence of an SQRA. These locations were included in plan formulation where improvements would positively impact the system's capacity for protection. The following lists the reaches at Port Arthur:

- 8- to 10-foot I-Wall
- Closure Structure
- I-Wall Near Valero
- Tank Farm

<u>8- to 10-foot I-Wall One-Foot Raise</u>

The existing HFPP in this reach was constructed in stages to allow for settlement and consolidation of the soft foundation material. During the second stage of construction, the embankment experienced significant settlement and the final required levee height could not be attained. A decision was made at the time to add an I-wall section on the top of the earthen embankment to obtain the required height. This I-wall section was put in place with excess height to accommodate any future settlement of the foundation or embankment. The additional stick-up height of the I-wall section makes it susceptible to failure for hydraulic loading events that exceed the system's capacity. Top of the I-wall is at elevation 23.5 feet while the top of the embankment is at elevation 16.5 feet. The I-wall does not meet current criteria for embedment.

1) No Fail: 3,500 LF of 15-foot-wide 6-inch scour pad would allow for the existing I-wall to have the excess stick-up height removed and erosion protection installed reducing the likelihood of a brittle failure if the system's capacity is exceeded.
2) One-Foot Raise: 7,500 LF of 15-foot-wide 6-inch scour pad would allow for the existing I-wall to have the excess stick-up height removed and erosion protection installed, reducing the likelihood of a brittle failure if the system's capacity is exceeded. This option would include adding capacity to the system in this reach by addressing low areas of the levee system, raising 2,000 lf of levee one foot and providing overtopping erosion protection.
3) Two-Foot Raise: 7,500 LF of 15-foot-wide 6-inch scour pad would allow for the existing I-wall to have the excess stick-up height removed and erosion protection installed, reducing the likelihood of a brittle failure if the system's capacity is exceeded. Additional 60,000 LF of levee raising 2 feet along with the raising of the State Highway 87 and Highway 73 levee crossings. Floodwalls at two (2) pump stations would be added along with 1,000 LF of floodwall reconstruction at the Taylors Bayou closure. This option would also require the replacement of a gravity drainage structure and vehicle closure structure.

<u>Closure Structure</u>

Top of the closure structure is at elevation 15.5 feet while the top of the embankment is at elevation 17 feet. It was assumed during the fragility curve exercise that a wave action load of 2 feet would occur along with still-water levels. Once the closure structure starts being overtopped, the chances of embankment erosion increase. The closure structure is a steel gate with concrete supports the tie into the adjacent earth embankments with steel sheet pile I-walls. This configuration was shown to have excessive erosion during overtopping during Hurricane Katrina. Similar performance is expected at this structure under the same hydraulic loading conditions.

1) No Fail: Construction of two 300 LF of 100-foot-wide 6-ichn scour pads, one on each side of the structure to provide erosion protection to reduce the likelihood of a brittle failure if the system's capacity is exceeded.
2) One-Foot Raise: Replace closure structure with a gate structure 12-foot height by 30-foot width including two 300 LF of 100-foot-wide 6-inch scour pads, one on each side of the structure to provide erosion protection to reduce the likelihood of a brittle failure if the system's capacity is exceeded. Additional raising of 12,000 LF of levee one foot.
3) Two-Foot Raise: Replace 2 closure structures gate structures are 12-foot height by 30-foot width. Including two (2) 300 LF of 100-foot-wide 6-inch scour pad at each closure structure. Raising 12,000 LF of levee one foot and adding floodwalls at 2 pump stations, 500 LF total (7 feet tall) along with reinforcing pump station walls at four (4) existing pump stations.

<u>I-Wall Near Valero</u>

I-wall near the Valero refinery is a steel sheet pile I-wall that has a stick-up height of 8 to 12+ feet, no scour protection and no protection from debris or vessel impact. Wall has been analyzed for stability and shows significant crack development, marginal FOS for rotational stability and large deflections. Fragility curve exercise was performed assuming a 2-foot wave along with development of still water level (SWL). Hydraulic loading at 15 feet is believed to be enough to cause exceedance in deflection past established thresholds in current criteria.

1) No Fail: Construction of 5,000 LF of 15-foot-wide 6-inch scour pad and batter piling and waler system will provide additional structural integrity and erosion protection to reduce the likelihood of a brittle failure if the system's capacity is exceeded.
2) One-Foot Raise: Construction of 5,000 LF of 15-foot-wide 6-inch scour pad and batter piling and waler system with one-foot rise will provide additional system capacity, increase structural integrity of the I-wall and provide erosion protection to reduce the likelihood of a brittle failure if the system's capacity is exceeded. Additionally, 3,000 LF of levee will need to be raised one foot.
3) Two-Foot Raise: This option will require significant reconstruction of the HFPP in the evaluated area including 5,000 LF of floodwall (15 feet tall), ten (10) closure structures

consisting of a 15-foot height and 20-foot width opening, 3,000 LF of levee raised 2 feet along with the work specified in the I-wall near Tank Farm #3 option and 8- to 10-foot I-wall #3 option.

Tank Farm

I-wall near the Tank Farm is a steel sheet pile I-wall that has a stick up height of 10 to 12+ feet, no scour protection and no protection from debris or vessel impact. Wall has been analyzed for stability and shows significant crack development, marginal FOS for rotational stability and large deflections. Wave action load of 2 feet was assumed during fragility curve exercise.

1) No Fail: Construction of 1,800 LF of 15-foot-wide 6-inch scour pad and batter piling and waler system will provide additional structural integrity and erosion protection to reduce the likelihood of a brittle failure if the system's capacity is exceeded.
2) One-Foot Raise: Construction of 1,800 LF of 15-foot-wide 6-inch scour pad and batter piling and waler system with one foot rise will provide additional system capacity, increase structural integrity of the I-wall and provide erosion protection to reduce the likelihood of a brittle failure if the system's capacity is exceeded. Additionally, 7,000 LF of levee will need to be raised one foot.
3) Two-Foot Raise: Construction of 2,000 LF of floodwall (15 feet tall) along with 9,000 LF of levee raised 2 feet, construct a floodwall at one pump station (200 LF at 7 feet tall) and raise an additional 10,400 LF of levee 2 feet and reconstruct 12,000 LF 15-foot-tall floodwall. Rebuilding four (4) existing pump stations at 1,100 cfs would also be required.

1.8.3 Freeport and Vicinity CSRM

The Freeport and Vicinity HFPP is in the coastal plains in southern Brazoria County, about 48 miles southwest of Galveston, Texas. The overall project for hurricane flood protection for Freeport and Vicinity, Texas, was authorized by the Flood Control Act of 23 October 1962, Public Law 87-874, substantially in accordance with House Document No. 495, 87th Congress, 2nd Session. The authorization provides for construction of improvements at Freeport and Vicinity, Texas, for protection against storm tides caused by tropical cyclones along the Gulf Coast of magnitudes up to and including the standard project hurricane. The existing HFPP at Freeport consists of approximately 43 miles of levees and wave barriers, seven (7) pump stations and multiple gates, culverts, and related appurtenances. Additionally, in the line of protection includes multiple structures that also serve as control structures and docks for the Dow Chemical Co., BASF, Conoco Philips, Exxon and Port Freeport. Figure 1-4 shows the project area for the Freeport and Vicinity CSRM.

The Freeport and vicinity system has a preliminarily LSAC that has resulted in the RMC conducting an SQRA to better define the systems risk. This classification was primarily driven by numerous factors. The primary structural factors that would have federal interest are seepage/slope stability of “sandy” levees, I-wall stability, and a “low” level of protection. These performance factors in an area that has very high consequences drove the LSAC.

Currently the system is unacceptable in the RP under PL 84-99 and not certifiable for FEMA accreditation under CFR 65.10, so the Non-Federal Sponsor has a SWIF plan in place to address the deferred maintenance issues and issues impeding 65.10 accreditation. The Sponsor does not plan on addressing the structural risk drivers for the LSAC due to the performance concerns coming at a more significant hydraulic loading event than the requirements under 65.10. The FWOP conditions would result in no action being undertaken to reduce the risk that the system would suffer a catastrophic failure during a future hydraulic loading roughly equivalent to or slightly before the 130-year event.

Figure 1-4: Existing HFPP within Freeport and Vicinity CSRM Project Area

Fragility curves were developed for specific locations along the existing systems in order to account for the anticipated system performance at those locations and were used to scope the reconstruction and resiliency features for the existing systems. These curves were developed by the SWG Geotechnical and Structural Engineering Section using existing information. Existing data used included performance history based on previous hydraulic loadings at each system, draft findings of the Freeport SQRA, draft findings of the Texas Coastal I-wall study, USACE work on erosion for Herbert Hover Dike, IPET report, ETL 575 criteria, along with influence from the LSAC screenings and the HEC Risk Analysis for Flood Damage Reduction Projects and RMC Internal Erosion Workshop. The curves were focused on erodeability of "sandy" levees, steady state seepage concerns, saturated levee stability concerns, anticipated structural performance of the I-walls, and erosion at tie-in locations. These curves reflect uncertainty in storm duration, size, landfall location, and wind driven wave height.

The alternative selection process focused on selecting structural features that would significantly reduce the likelihood of a catastrophic failure of the system under extreme hydraulic loading. Features that would allow an increase in the level of protection were also evaluated. These alternatives consisted of overtopping erosion protection along with additional structural features to increase stability along with entire reconstruction/reconfiguration of the I-walls to T-walls. The additional structural features are a replacement of a wall/drop in panel system at the Port Freeport and a closure of the Dow Barge Canal. Erosion control features would be constructed as needed to reduce the likelihood of significant erosion leading to system failure from an overtopping event.

TSP implementation will require additional project scoping to ensure the CSRM project functions as a system. This additional work will include assessing the entire existing system to ensure that the system is able to perform under the anticipated hydraulic event chosen for the TSP. Additionally the system will be evaluated and configured to accommodate relative sea-level rise as needed.

Risks associated with the TSP for the existing system are primarily situated with the use of existing geotechnical and structural information. Given the extensive work for the initial construction, along with additional information obtained by the sponsor for 65.10 work, this risk should be manageable.

The draft findings of the SQRA for the Freeport system show vulnerabilities primarily associated with steady-state seepage issues and floodwall and levee overtopping. Other performance issues identified during the SQRA were the result of deferred Sponsor maintenance, or alterations that local industrial stakeholders have constructed over time, these are not being evaluated under this project. Floodwall performance issues, at locations where the originally constructed floodwall is still in place and has been operated and maintained in an acceptable manner, are being evaluated to include stability and resiliency. Levee reaches that are non-uniform in height or otherwise susceptible to concentrated overtopping erosion during an event are being evaluated for raising or armoring to reduce the likelihood of breach.

The formulation of alternatives for the Freeport and Vicinity CSRM began with defining reaches for the system. These locations were then narrowed during formulation to those locations where improvements would positively impact the system's capacity for protection and to reduce any redundancies. For example, improvements to the Dow Barge Canal would negate any failures at the Dow Turning Basin. The following is the resulting list of reaches at the Freeport and Vicinity CSRM:

- South Storm Levee
- Freeport Dock
- Tide Gate I-Wall
- Old River at Dow Thumb
- Dow Barge Canal
- East Storm Levee
- Oyster Creek Levee

<u>South Storm Levee Raise</u>

The South Storm Levee is a frontal levee that has potential for direct wave impact from the Gulf of Mexico during storm loading. When this levee was originally constructed, the area south of the levee was very low in elevation and allowed surge and waves to reach it. Over the last 40 years, USACE has constructed dredge disposal areas for the deep-draft navigation channel in this low area. Continued use of the disposal areas has increased the elevation of the low area to a point that it is now higher than the South Storm Levee.

1) One-Foot Raise: Construction would include earth placement on top of the existing earth embankment for a 1-foot raise.
2) Two-Foot Raise: Construction would include earth placement on top of the existing earth embankment for a 2-foot raise.

<u>Freeport Dock Floodwall Raise</u>

The Freeport Dock floodwall is a 3-foot floodwall that was added to the dock face at the Port Freeport docks after Hurricane Ike under PL 84-99. This floodwall has drop-in panels that are removable to allow for "roll on - roll off" cargo loading. During evaluation of the HFPP for 65.10 the Non-Federal Sponsor noted that the wall/panels were structurally deficient. This deficiency was confirmed during the RMC-led Freeport SQRA.

1) Partial Fail: Construction includes replacing the drop-in panels and anchor system.
2) No Fail: Construction of 3,000 LF of floodwall to meet all USACE requirements for a wall/drop-in panel system located at a port facility.
3) One-Foot Raise: Construction would require complete reconstruction of the dock and floodwall assembly.

Tide Gate One-Foot Raise

The I-wall located at the Tide Gate was constructed as part of the original HFPP construction when the earth embankment section could not reach the design elevation. The very soft foundation materials present in the old river channel would not support the additional weight of the initially proposed embankment section. The original sheet pile/concrete cap I-wall in an embankment over very soft foundation material is of similar design to the poor performing I-wall sections in New Orleans, Louisiana. The stability concerns for this section of I-wall are at design exceedance events with water levels near or at the top of wall. The proposed construction will be to reconstruct the I-wall as a pile founded T-wall. The construction process will include demolishing the existing concrete monoliths and exposing the existing PZ27 sheet piles, driving concrete foundation piling, placing a concrete piling cap, placing scour protection, and placing the concrete stem of the T-wall. The overall length of the T-wall is approximately 362 feet.

1) No Fail: Construction of 362 LF of floodwall 10 feet tall.
2) One-Foot Raise: Construction of 700 LF of floodwall 11 feet tall along with 2,000 LF of levee raised one foot.
3) Two-Foot Raise: Construction of 700 LF of floodwall (12 feet tall) and 3,500 LF of levee raised 2 feet along with adding a floodwall at one pump station (200 LF at 7 feet tall). The Tide Gate structure adjacent to the I-wall will require significant modification or complete reconstruction to accommodate the 2-foot raise.

Old River at Dow Thumb

This reach of levee is an earth embankment that would be susceptible to erosion during an overtopping event. Updated modeling shows an area of this reach that has significant risk to large wave attack and overtopping from wave propagation along the adjacent deep draft navigation channel. This area has a very low FOS for global stability and is currently being evaluated under the General Re-evaluation Report for the navigation project. Any action taken under this planning study will not reduce the current FOS for global stability.

1) No Fail: Construction of 14,500 LF of high performance turf-reinforcement mattress (HPTRM) and 4,000 LF of 15-foot-wide 6-inch scour pad to provide erosion protection to reduce the likelihood of a brittle failure if the systems' capacity is exceeded.

2) One-foot Raise: Construction of 4,000 LF of 15-foot-wide 6-inch scour pad along with 3,000 LF of levee raised 1foot and 14,500 LF of HPTRM to “level up” the low spots and provide erosion protection to reduce the likelihood of a brittle failure if the systems’ capacity is exceeded.
3) Two-Foot Raise: Due to extremely low FOS for global stability raising the levee over existing heights by adding additional earth fill is not an option so for the 2-foot raise scenario the existing embankment would be removed and 12,000 LF of 10-foot-tall floodwall would be constructed. In areas that do not have stability issues 6,500 LF of levee would be raised 2 feet, one drainage structure would be replaced, and the saltwater intake at Dow A801 would be replaced.

Dow Barge Canal

The Dow Barge Canal levees are approximately 8.0 miles long and represent a significant risk to the HFPP performance at and above the design event. This risk is primarily from seepage and instability caused by seepage through the “sandy” levee and foundation material. Significant risk also exists with numerous pipeline penetrations, I-wall instability, and non-uniform levee heights. Taking into account the significant existing infrastructure within the existing embankment and the uncertainty in the performance of seepage control measures installed in such a complex environment, the PDT chose to utilize a closure structure and pump station constructed at the junction of the North Barge Canal and East Storm Levee. This structure will allow barge traffic to pass during routine operations and will have a pumping capacity of 2,000,000 gpm. The structure length will be approximately 500 feet long with two (2) sector gates totaling approximately 80 feet wide for vessel traffic. Additional tidal circulation will be provided by two (2) sluice gates approximately 15 feet wide each. The final configuration of this structure will match the proposed level of protection for the system.

East Storm Levee Raise

The East Storm Levee is a large earth embankment that faces the Gulf of Mexico and has direct wave and surge impacts from the gulf. The proposed construction procedure will include stripping topsoil, removal of a two lane asphalt road, placement of fill, replacement of a two-lane road, placement of a HPRTM, and turfing.

1) No Fail: Construction of 13,115 LF of HPTRM.
2) One-Foot Raise: Construction includes 13,115 LF of levee raised 1foot with HPTRM
3) Two-Foot Raise: Construction includes 19,115 LF of levee raised 2 feet with HPTRM and a floodwall at one pump station, 800 LF total (5 feet tall). Reinforcement of the pump station walls and raising FM 332 at the levee crossing would also be required.

Oyster Creek Levee Raise

The Oyster Creek Levee was constructed at varying elevations to account for changes in flood elevation as noted in the hydraulic modeling. The updated ADCIRC modeling showed a height deficiency over 3,500 LF where the updated modeling shows an elevated surge.

1) No Fail: The Oyster Creek Levee will be raised 2 feet over 3,500 LF in order to correct the height deficiency. The construction procedure will include stripping topsoil, removal of a 12-foot-wide asphalt road, placement of fill, replacement of a 12-foot-wide road, and turfing.
2) One-Foot Raise: Construction will include 3,500 LF of 3-foot levee raise and 10,000 LF of 1-foot levee raise for a total distance of 13,500 LF. The construction procedure will include stripping topsoil, removal of a 12-foot-wide asphalt road, placement of fill, replacement of a 12-foot-wide road and turfing.
3) Two-Foot Raise: Construction will include 3,500 LF of 4-foot levee raise and 1,000 LF of floodwall reconstruction along with raising FM 523 at the levee crossing. Additionally, 33,000 LF of levee raised 2 feet, reconstruction of one pump station, 1,100 cfs, and replacement of six (6) gravity structures would also be required.

1.9 PLAN EVALUATION, COMPARISON, AND SELECTION

The evaluation and comparison of the various alternative plans considered is described in the DIFR-EIS. The actual plan selection process is undertaken as one of the final steps in the planning process and is performed in conjunction with sponsor input and HQ-USACE reviews and approval. The selected plan is held to stringent standards. Consequently, it must be technically viable, economically feasible, and environmentally acceptable, all in accordance with governing agency regulations and associated Federal statutes.

A preliminary array of conceptual structural alternatives was compiled by combining planning measures. Through the plan formulation process, this array was culled to an evaluation array of four (4) concept structural alternatives that were carried forward for further development. These generalized alternatives were evaluated and refined accordingly to come up with the final array of structural alternatives summarized in Table 1-1 and shown on maps on Plates 1-1 through 1-6. These structural alternatives were then further evaluated at a preliminary level of design to determine the projects that would comprise the TSP. Incidental to this evaluation, alternative levels of protection were economically assessed to optimize the plans. Other alternatives screened in the early study phases are discussed in Appendix B. This appendix focuses on the final array evaluation.

Table 1-1: Structural Coastal Storm Risk Management Alternatives Evaluated

Project Area	Final Array - Alt. Name	Description of CSRM Alternative	Optimization Alternatives
Orange-Jefferson CSRM	**Orange 1 New Levee**	Orange 1 begins at its north terminus in the City of Vidor north of U.S. Interstate 10 across from Church Street (which is south of the freeway). From there, the alignment runs southwesterly parallel to the freeway and nearly coincident with Rose City's north boundary line. Because of the limited availability of land area and considerable real estate hurdles that would have to be overcome, concrete floodwall is proposed for this leg. The alignment continues a little to the south across the freeway along Rose City's westernmost boundary, after having made a short jog to the north, and back east-northeasterly south of and parallel to an existing railway. After following most of the incorporated city boundaries, the alignment takes a sharp turn to the south-southeast, where it ends at an existing pipeline corridor about 1.5 miles away. The length of the entire reach is 8.2 miles. A levee system consisting of earthen levee and concrete floodwall was evaluated at four (4) incremental heights, in one-foot increments, that were correlated to corresponding reductions in the interior flood depths. As conceived, the system would consist of approximately 3.1 miles of concrete floodwall. Regardless of the structure heights assumed, Interstate 10 and the railroad would have to be accommodated by closure structures, or the freeway would have to be elevated, which is the more plausible alternative. Orange 1 reduces the surge flood-damage risk for Rose City and west Vidor.	All Alternatives Total new Levee length: 5.1 miles Total new Floodwall length: 3.1 miles Total new Reach length: 8.2 miles Incremental Levee Heights • New levee/floodwall, 11 feet high (not economically feasible) • New levee/floodwall, 12 feet high (not economically feasible) • New levee/floodwall, 13 feet high (not economically feasible) • New levee/floodwall, 14 feet high (not economically feasible)
	Orange 2 New Levee	Orange 2 generally runs east and south from the south end of the Orange 1 reach to an arbitrary high ground tie-in point at its southeast end less than a mile southwest of FM 1135, where it intersects FM 105 (Orangefield Road), midway between Rose City and Bridge City to the southeast. A levee system consisting of earthen levee was evaluated at four (4) incremental heights, in one-foot increments, that were correlated to corresponding reductions in the interior flood depths. No floodwalls are proposed. Orange 2 is approximately 6.6 miles long and protects incorporated areas within the City of Vidor and unincorporated areas of Vidor.	All Alternatives Total new Levee length: 6.6 miles Total new Reach length: 6.6 miles Incremental Levee Heights • New levee, 11 feet high (not economically feasible) • New levee, 12 feet high (not economically feasible) • New levee, 13 feet high (not economically feasible) • New levee, 14 feet high (not economically feasible)

Project Area	Final Array - Alt. Name	Description of CSRM Alternative	Optimization Alternatives
Orange-Jefferson CSRM	**Orange 3 New Levee**	Orange 3 runs southeasterly from the end of the Orange 2 reach to Bridge City. The alignment goes around the south side of the city and continues east-northeasterly to the Port of Orange. Along this segment, the alignment crosses Cow and Adams Bayous, both of which are used for navigation. From the port, the alignment runs northwesterly along the port's west property line (along Alabama Street) and to the north along much of the length of the Sabine River's west bank. The northeast terminus of the Orange 3 reach, and entire Orange County system reach, is at U.S. Interstate 10, on its south side, about 2/10ths of a mile east of the U.S. Highway 90 Business (Simmons Drive) intersection. (Continuation of the levee system beyond I-10 was considered, but eventually ruled out because of the associated environmental impacts, construction costs, and deduction that only modest damages would be incurred based on the assumed still-water elevations.) In total, the longest reach of the three (3) Orange reaches is 27.2 miles long. A levee system consisting of earthen levee and floodwall was evaluated at four (4) incremental heights, in one-foot increments, that were correlated to corresponding reductions in the interior flood depths. Because the levee system alignment traverses several industrial areas, sensitive environmental habitat areas, places where the land area is insufficient for levee construction, or there are other real estate constraints, and wide floodplains subject to constant inundation (where the foundation conditions are likely to be poor), the system assumes 6.1 miles of concrete floodwall. The alignment crosses many pipelines and utilities, several roadways, including State Highway 87, and two (2) navigation channels, all of which will have to be accommodated accordingly, relocated, or modified. Orange 3 provides the greatest amount of protection as it reduces the surge flood-damage risk to the Cities of Orangefield, Bridge City, Pinehurst, West Orange, and Orange, which together represent a large population area and heavily industrialized area. It also protects Entergy Texas Inc.'s Sabine (Power) Plant, Bridge City.	All Alternatives Total new Levee length: 21.1 miles Total new Floodwall length: 6.1 miles Total new Reach length: 27.2 miles Incremental Levee Heights • New levee/floodwall, 11 feet high (TSP) • New levee/floodwall, 12 feet high (NED Plan) • New levee/floodwall, 13 feet high (not economically feasible) • New levee/floodwall, 14 feet high (not economically feasible)
	Jefferson Main New Levee	This component of the Orange-Jefferson CSRM in Jefferson County begins near the northeastern terminus of the existing Port Arthur and Vicinity HFPP where it will tie into high ground. The alignment runs west-northwest nearly 11.0 miles, generally along the west bank of the Neches River to a point on the north side of the Du Pont Beaumont Works Industrial Park in Nederland. A levee system consisting of earthen levee and floodwall was evaluated at four (4) incremental heights, in one-foot increments, that were correlated to corresponding reductions in the interior flood depths. Because industrial facilities to be protected front the river, floodwall will have to be constructed along these reaches. As conceived, floodwall will comprise 3.1 miles of the system.	All Alternatives Total new Levee length: 7.6 miles Total new Floodwall length: 3.1 miles Total new Reach length: 10.7 miles Incremental Levee Heights • New levee/floodwall, 11 feet high (TSP) • New levee/floodwall, 12 feet high (NED Plan) • New levee/floodwall, 13 feet high (not economically feasible) • New levee/floodwall, 14 feet high (not economically feasible)

Project Area	Final Array - Alt. Name	Description of CSRM Alternative	Optimization Alternatives
Orange-Jefferson CSRM	**Beaumont A New Levee**	This reach provides protection exclusively for the ExxonMobil Beaumont Chemical Plant, which is among the top ten domestic producers of refined products. A 3.6-mile-long system is proposed that consists of 1.1 miles of earthen levee and 2.5 miles of concrete floodwall. The same four (4) one-foot increment heights assumed for all the Orange-Jefferson CSRM reaches were assumed for this reach.	All Alternatives Total new Levee length: 1.1 miles Total new Floodwall length: 2.5 miles Total new Reach length: 3.6 miles Incremental Levee Heights • New levee/floodwall, 11 feet high • New levee/floodwall, 12 feet high (TSP) • New levee/floodwall, 13 feet high (NED Plan) • New levee/floodwall, 14 feet high
	Beaumont B New Levee	This reach was intended to provide isolated protection for an electrical power distribution station initially thought to be actively operating. This station is no longer being used for that purpose. It is now only of historical significance.	Screened Out
	Beaumont C New Levee	This reach provides isolated protection for a sulfur terminal on the Neches River owned by Martin Midstream Partners. The proposed earthen ring levee is 1.3 miles long. Four (4) levee heights were evaluated at one-foot increments.	All Alternatives Total new Levee length: 1.3 miles Total new Reach length: 1.3 miles Incremental Levee Heights • New levee/floodwall, 11 feet high (not economically feasible) • New levee/floodwall, 12 feet high (not economically feasible) • New levee/floodwall, 13 feet high (not economically feasible) • New levee/floodwall, 14 feet high (not economically feasible)

Project Area	Final Array - Alt. Name	Description of CSRM Alternative	Optimization Alternatives
Port Arthur and Vicinity CSRM	**8-10 Feet I-Wall Raise**	The "8-10 ft I-Wall" is a section floodwall located along the southernmost corner of the existing Port Arthur and Vicinity Hurricane Flood Protection Project. This legacy wall does not meet the current criterion for embedment. Therefore, structural resiliency is proposed to protect the base of the wall from being scoured on the protected side from overtopping. The feature proposed to address this is a 15-foot-wide by 6-inch thick concrete scour pad along the protected side of the wall. Raising of the adjoining levee embankments is also proposed in combination with the floodwall scour protection because they are not high enough to accommodate the economically derived still-water level with a 2-foot wave applied. Future relative sea level rise will only compound this problem. Overtopping would effectively reduce the level of protection afforded for the local area and could severely erode the levee embankment. Adding more height to this reach of the system, beyond that premised on just the still-water level and assumed 2-foot wave alone, to account for future sea-level rise and wave run-up, as the TSP is further developed, will have impacts on other elements comprising the system. If these structures were raised, the structures near the Valero Refinery and the Tank Farm would have to be raised as well. Consequently, the incorporation of sea level rise and wave run-up into the project will substantially increase the lengths of levee requiring raising, the heights of raising necessary, and the lengths of floodwall requiring resiliency measures, reinforcement, or replacement, system-wide, which would expand the scope of the TSP as contemplated at this time.	• "No Fail" – Construct 3,500 LF of 15-foot-wide by 6-inch thick scour pad to protect I-wall against structural failure. • Construct 7,500 LF of 15-foot-wide by 6-inch scour pad and raise levee embankment one foot over a distance of 2,000 LF. (NED Plan/TSP) • Construct 7,500 LF of 15-foot-wide by 6-inch scour pad and raise levee embankment 2 feet over a distance of 60,000 LF. In addition, - replace 1,000 LF of steel I-wall (Taylors Bayou closure structure, 20 feet high); - elevate State Highways 87 (2,500 LF) and 73 (5,000 LF) at levee crossings; - construct 500 LF of new floodwall, 7 feet high, at two (2) pump stations; - replace a gravity drainage structure; and - replace a 30 feet wide by7 12 feet high vehicle closure structure.
	I-Wall Raise near Valero	This section of I-wall is at the south corner of the system near the Valero Refinery plant. The wall has been documented to not meet deflection criterion. Therefore, structural resiliency is proposed for this steel wall in which a 5,000-foot-long by 15-foot-wide by 6-inch thick concrete scour pad along the protected side of the floodwall would be constructed to prevent scouring at the base of the wall, which in turn will prevent the wall from over-deflecting when subjected to full loading. (The fragility-curve exercise for this section assumed a 2-foot wave on top of the still-water levels.) Levee raising in this area is also proposed because the existing levees are not high enough to accommodate the assumed water level with a 2-foot wave applied. Overtopping would effectively reduce the level of protection afforded for the local area and could severely erode the levee embankment. Adding more height to this reach of the system, beyond that premised on just the still-water level and assumed 2-foot wave alone, to account for future sea level rise and wave run-up, as the TSP is further developed, will have impacts on other elements comprising the system. The incorporation of these considerations into the project will substantially increase the lengths of levee requiring raising, the heights of raising necessary, and the lengths of floodwall requiring resiliency measures, reinforcement, or replacement, system-wide, which would expand the scope of the TSP as contemplated at this time.	• "No Fail" – Construct 5,000 LF of 15-foot-wide by 6-inch thick scour pad to protect I-wall against structural failure. • Construct 5,000 LF of 15-foot-wide by6-inch thick scour pad and raise levee embankment 1 foot over a distance of 3,000 LF. In addition, structurally reinforce the I-wall with batter piles and walers. (NED Plan/TSP) • Replace the I-wall with new concrete T-wall, 15 feet high; raise 3,000 LF of levee 2 feet; and replace ten (10) 20-foot-wide closure structures. Additionally, construct the work associated with raising the "8- to 10-foot I-Wall" and the I-wall near the Tank Farm 2 feet.

Project Area	Final Array - Alt. Name	Description of CSRM Alternative	Optimization Alternatives
Port Arthur and Vicinity CSRM	**I-Wall Raise near Tank Farm**	This section of I-wall is at the south corner of the system near an oil tank farm. The wall is documented to have global stability issues because of insufficient embedment. Therefore, structural resiliency is proposed for this wall in which a 1,800-foot-long by 15-foot-wide by 6-inch thick concrete scour pad along the protected side of the floodwall would be constructed to prevent scouring at the base of the wall. The wall would also be structurally reinforced with batter piling and a waler system. Levee raising in this area is also proposed due to the possibility of overtopping from the assumed still-water level with a 2-foot wave applied. Sheet flow over the top of the levee effectively compromises the level of protection afforded for the local area and could severely erode the embankment, which would further reduce the level of protection provided. Adding more height to this reach of the system, beyond that premised on just the still-water level and assumed 2-foot wave alone, to account for future sea level rise and wave run-up, as the TSP is further developed, will have impacts on other elements comprising the system. The incorporation of these considerations into the project will substantially increase the lengths of levee requiring raising, the heights of raising necessary, and the lengths of floodwall requiring resiliency measures, reinforcement, or replacement, system-wide, which would expand the scope of the TSP as contemplated at this time.	• "No Fail" – Construct 1,800 LF of 15-foot-wide by 6-inch thick scour pad and structurally reinforce the I-wall with batter piles and a waler system to protect against wall failure. • Construct 1,800 LF of 15-foot-wide by 6-inch thick scour pad, structurally reinforce the I-wall with batter piling and walers, and raise the levee embankment one foot over a distance of 7,000 LF. (NED Plan/TSP) • Replace the I-wall with 2,000 LF of new concrete T-wall, 15 feet high, and raise 19,400 LF of levee 2 feet. In addition, - install 200 LF of 7-foot-high floodwall at an existing pump station; - replace an additional 12,000 LF of floodwall with new concrete T-wall, 15 feet high; and - rebuild four (4) pump stations, each with capacity of 1,100 cfs.
	Closure Structure Raise	The Port Arthur closure structure, which consists of two (2) gated structures, is near the easternmost corner of the system. The top of the structure is at Elev. 15.5 feet, while the top of the adjacent levee embankment is at Elev. 17.0 feet. It was assumed during the fragility-curve exercise that a wave action load of 2 feet would occur along with still-water levels. Once the closure structure starts being overtopped, the chances of embankment erosion increase. To reduce the likelihood of a brittle failure due to erosion if the system's capacity is exceeded, 100-foot-wide by 6-inch concrete scour pads are proposed along both sides of the structure. The length of this protection would be about 300 LF. In conjunction with the scour protection, the adjoining levees would be raised to prevent localized overtopping. The length of levee requiring raising is approximately 12,000 LF.	• "No Fail" – Construct 300 LF of 100-foot-wide by 6-inch thick scour pad along both sides of two (2) closure structures to reduce the likelihood of a brittle failure due to erosion if the system's capacity is exceeded. • Construct 300 LF of 100-foot-wide by 6-inch thick scour pad along both sides of two (2) closure structures and raise levee embankment one foot over a distance of 12,000 LF. Additionally, replace a 30 feet wide by 12 feet high vehicle closure structure. (NED Plan/TSP) • Construct 300 LF of 100-foot-wide by 6-inch thick scour pad along both sides of two (2) closure structures and raise the levee embankment 2 feet over a distance of 12,000 LF. Additionally, replace two (2) 30 feet wide by 12 feet high vehicle closure structures, construct 500 LF of 7-foot floodwall at two (2) pump stations, and reinforce the existing pump station walls.

Project Area	Final Array - Alt. Name	Description of CSRM Alternative	Optimization Alternatives
Freeport and Vicinity CSRM	**South Storm Levee**	The South Storm Levee reach is on the south side of the Freeport and Vicinity Hurricane Flood Protection Project paralleling the Gulf Intracoastal Waterway. Because the levee embankment is not uniform in height and is therefore susceptible to concentrated overtopping erosion for the assumed design storm event, the reach was considered a candidate for levee raising. Levee raising was dropped from further consideration however because it was demonstrated not to be economically feasible.	• Raise levee embankment one foot (not economically feasible) • Raise levee embankment 2 feet (not economically feasible)
	Freeport Dock Floodwall	The existing floodwall along the Port of Freeport dock is not structurally competent to handle the additional hydrostatic loading and overtopping from the assumed design storm event. Consequently, structural reinforcement measures were considered that included replacing panels and anchor system, at a minimum, and structurally reinforcing the wall. The other alternative considered was to entirely reconstruct the dock and floodwall to a higher elevation.	• "Partial Fail" - Replace panels and anchor system. • "No Fail" - Reinforce 3,000 LF of floodwall. (NED Plan/TSP) • Reconstruct dock and floodwall (3,000 LF), raising by one foot. (not economically feasible)
	Tide Gate I-Wall	Because of floodwall performance issues related to stability and resiliency, the I-wall at the Tide Gate is proposed to be reconstructed as a pile-founded T-wall. The construction process will involve demolishing the existing concrete monoliths and exposing the existing PZ27 sheet piling, driving concrete piling for the foundation, constructing a concrete pile cap, constructing scour protection, and placing concrete for the T-wall stem. The overall length of the replaced wall would be 700 feet. Two thousand (2,000) LF of the adjacent levee embankment would also be raised one foot.	• "No Fail" – Replace 362 lf of I-wall with concrete T-wall, 10 feet high. • Replace 700 LF of I-wall with new concrete T-wall, 10 feet high, and raise levee embankment one foot over a distance of 2,000 LF. (NED Plan/TSP) • Replace 700 LF of I-wall with new concrete T-wall, 11 feet high, and raise levee embankment 2 feet over a distance of 3,500 LF. In addition, construct 200 LF of floodwall at one pump station, 7 feet high, and rebuild the Tide Gate structure.
	Old River Levee at Dow Thumb	The Old River North Levee at Dow Thumb is not high enough to prevent overtopping by waves during extreme surge events. Consequently, structural resiliency measures and/or structure height raises are proposed to prevent erosion of the levee embankment and scouring behind the existing floodwall. Construction activities to raise the existing earthen levee embankment will involve stripping the embankment side slopes, removing and replacing a 12-foot-wide asphalt road on the crown, placing and compacting fill atop the crown and along the interior slope, installing high-performance turf reinforcement mattress (HPTRM) on the slopes, and turfing the slopes.	• "No Fail" – Reinforce 14,500 LF of the levee embankment with HPTRM and construct 4,000 LF of 15-foot-wide by 6-inch thick concrete scour pad behind the I-wall - 3,000 LF feet along the "thumb" and 1,000 LF near the entrance of the Dow Barge Canal to provide structural resiliency. • Reinforce 14,500 LF of the levee embankment with HPTRM and construct 4,000 LF of 15-foot-wide by 6-inch thick concrete scour pad behind the I-wall to provide structural resiliency, and raise levee embankment one foot over a distance of 3,000 LF. (NED Plan/TSP) • Raise levee embankment 2 feet over a distance of 6,500 LF, replace 12,000 LF of floodwall, making it 10 feet high, replace one drainage structure, and construct a new saltwater intake structure. (not economically feasible)

Project Area	Final Array - Alt. Name	Description of CSRM Alternative	Optimization Alternatives
Freeport and Vicinity CSRM	**Dow Barge Canal**	A closure structure with a navigable opening and pump station is proposed at the junction of the North Barge Canal and East Storm Levee to obviate the need to resolve the seepage problem with the existing "sandy" levees when steady-state seepage is assumed (which is the condition required to be assumed versus transient seepage). It is not technically feasible to fix the underseepage problem given the existing infrastructure along the length of the levee. The closure structure will allow barge traffic to pass during routine operations. As conceived, the pump station would be capable of pumping of 2,000,000 gallons per minute. The structure will be approximately 500 feet long with two (2) sector gates to provide an 80-foot-wide opening for vessel traffic. Tidal circulation would be maintained with two (2) sluice gates, each approximately 15 feet wide.	• Rehabilitate levee along barge canal (~13 miles), including the installation of a seepage cut-off wall. (not technically feasible) • Construct a new gate structure with a navigable opening across the barge canal. The structure will be approximately 500 feet long with two (2) sector gates to provide an 80-foot-wide opening for vessel traffic, have two (2) sluice gates to maintain tidal circulation, and have an auxiliary pump station with a 2,000,000 gallons per minute pumping capacity. (NED Plan/TSP)
	East Storm Levee	The reach of levee along the system's east side is not high enough to prevent wave overtopping from the design surge event. Construction activities to raise the existing earthen levee embankment will involve stripping the embankment side slopes, removing and replacing a two-lane asphalt road on the crown, placing and compacting fill atop the crown and along the interior slope, installing HPTRM on the slopes, and turfing the slopes.	• "No Fail" – Install HPTRM on levee embankment slopes along a distance of 13,115 LF • Raise levee embankment one foot over a distance of 13,115 LF and install HPTRM on slopes. (NED Plan/TSP) • Raise levee embankment 2 feet over a distance of 19,115 lf and install HPTRM on slopes. In addition, construct 800 lf of new floodwall at one pump station, 5 feet high, reinforce the pump station walls, and raise FM 332 at levee crossing (2,500 lf).
	Oyster Creek Levee	The reach of levee flanking the south bank of Oyster Creek Levee is not high enough to prevent wave overtopping from the design surge event. Construction activities to raise the existing earthen levee embankment will involve stripping the embankment side slopes, removing and replacing a 12-foot wide asphalt road on the crown, placing and compacting fill atop the crown and along the interior slope, installing HPTRM on the slopes, and turfing the slopes.	• "No Fail" – Raise levee embankment two (2) feet over a distance of 3,500 LF. • Raise levee embankment one foot over a distance of 10,000 LF and 3 feet over a distance of 3,500 LF. (NED Plan/TSP) • Raise levee embankment 2 feet over a distance of 33,000 LF and 4 feet over a distance of 3,500 LF. In addition, - replace 1,000 LF of floodwall, making it 8 feet high; - build new pump station with 1,100 cfs pumping capacity; - replace six (6) gravity structures; - and raise FM 523 at levee crossing (2,500 LF).

1.10 DESCRIPTION OF TSP PROJECTS

1.10.1 Orange-Jefferson CSRM Project (reference Plate G-1)

1.10.1.1 Orange 3 Reach

The proposed Orange 3 reach is a levee/floodwall flood protection system that will serve to reduce the flood-damage potential from storm surge to much of the southern half of Orange County along the Sabine River and Bessie Heights Marsh (see Plate G-1). As conceived, the structures comprising the system will be built to an elevation capable of handling an 11-foot storm surge. At this time, there is no provision in the concept features assumed in developing the TSP project costs that accounts for future relative sea level rise, waves on top of the surge level, and additional minimum freeboard for wave overtopping. Therefore, the structure heights to accommodate this surge likely will end up being designed several feet higher than the heights assumed for the optimization analysis. (For the optimization analysis, it was assumed that sea-level rise, wave run-up, and overtopping could be uniformly applied across the system without affecting the TSP outcome.)

Orange 3 begins at I-10 at its northeast end, about 1.75 miles west of where the highway crosses the Sabine River. From there, the alignment roughly parallels the Sabine River to the south and then to the southwest to an industrial canal 1.65 miles northeast of the mouth of Cow Bayou. Along this reach, the alignment crosses Adams Bayou, which is used for navigation. There, the alignment turns sharply to the northwest, going up to a point near the south corner of the Du Pont Sabine River Works plant. From this point, the alignment runs southwesterly to Bridge City, crossing Cow Bayou along the way, which is also used for navigation. The alignment then wraps around Bridge City to the south, turning northwesterly along the north side of the Bessie Heights Marsh to an end point less than a mile southwest of FM 1135, where it intersects FM 105 (Orangefield Road). Along this reach, the system will protect an electrical power-generating plant. In total, the length of the system will be approximately 27.1 miles, 21.1 miles of which will consist of earthen levee and 6.1 miles of concrete floodwall. The proposed alignment crosses several secondary roads, State Highway 87, Adams and Cow Bayous, an industrial canal, power plant intake canal, and floodplains where the foundation conditions are poor. It also crosses numerous pipelines and utilities, industrial facilities, commercial properties, residential areas, and sensitive environmental habitats. Consequently, there are going to be several challenges in implementing this reach of the project that will add considerably to its construction cost.

Orange 3 reduces the flood-damage potential from storm surge within the Cities of Orange, West Orange, Pinehurst, Bridge City, and Orangefield. It also protects petrochemical plants and Entergy Texas Inc. Sabine (Power) Plant, Bridge City.

1.10.1.2 Jefferson Main Reach

The proposed Jefferson Main reach is an earthen levee/concrete floodwall protection system that will serve to reduce the flood-damage potential from storm surge along the northeastern boundary of Jefferson County defined by the Neches River (see Plate G-1). As conceived, the structures comprising the system will be built to an elevation capable of handling an 11-foot storm surge. At this time, there is no provision in the concept features assumed in developing the TSP project costs that accounts for future relative sea level rise, waves on top of the surge level, and additional minimum freeboard for wave overtopping. Therefore, the structure heights to accommodate this surge likely will end up being designed several feet higher that the heights assumed for the optimization analysis.

The alignment begins near the northeastern terminus of the existing Port Arthur and Vicinity HFPP where it will tie into high ground. It runs west-northwest generally along the west bank of the Neches River, nearly 11.0 miles, to a point on the north side of the Du Pont Beaumont Works Industrial Park in Nederland. Because industrial facilities to be protected front the river, approximately 3.1 miles of the system will consist of floodwall. The alignment crosses industrial plants and must provide for access to several docking facilities along the river. It circumvents a large upland wetland area, thereby avoiding adverse impacts to the wetland.

1.10.1.3 Beaumont "A" Reach

The proposed Beaumont "A" reach is an earthen levee/concrete floodwall protection system that will serve to protect ExxonMobil's Beaumont Chemical Plant (see Plate G-1). This 3.6-mile-long system, consisting of 1.1 miles of levee and 2.5 miles of floodwall, was envisioned to supplant the existing flood-protection system surrounding the plant to provide a level of protection that reduces the flood-damage risk against a 12-foot surge. It has been discovered since this alternative was forwarded to be included in the TSP as a project reach however that the recently industry-built barrier is approximately 3 to 4 feet higher than that being proposed based on the SWL alone. The proposed alignment generally follows the existing barrier alignment. When storm surge from Hurricane Ike inundated the plant in 2008, the plant was 6 to 10 feet under water. This resulted in a shutdown of the plant for more than a month. To prevent future damages from storm surge, ExxonMobil Corporation replaced its former levee system, making the barrier more robust and providing a higher level of protection.

1.10.2 Port Arthur and Vicinity CSRM Project (reference Plate G-2)

1.10.2.1 Port Arthur 8- to 10-foot I-Wall One-Foot Raise

The TSP for reconstruction and resiliency features for the existing HFPP in this reach was selected to be the one-foot raise plan. A one-foot raise will require 7,500 LF of 15-foot-wide by 6-inch thick concrete scour pad along with removing the excess stick-up height from the existing I-wall and adding erosion protection to reduce the likelihood of a brittle failure if the system's capacity is exceeded. This plan includes adding capacity to the system in this reach by addressing low areas of the levee system, raising 2,000 LF of levee one foot, and providing overtopping erosion protection at areas subject to excessive erosion.

1.10.2.2 Port Arthur Closure Structure

The TSP for reconstruction and resiliency features for the existing HFPP at the closure structure is the one-foot raise plan. The replaced closure structure gate structure is proposed to be 12 feet high by 30 feet wide. In addition to replacing the structure, 2,300 LF of 100-foot-wide by 6-inch thick concrete scour pad will be constructed, one on each side of the structure, to provide erosion protection to reduce the likelihood of a brittle failure if the system's capacity is exceeded and 12,000 LF of levee will be raised one foot.

1.10.2.3 I-Wall near Valero

The TSP for reconstruction and resiliency features for the existing HFPP at the I-wall near the Valero refinery is additional structural support and a one-foot raise. Construction of 5,000 LF of 15-foot-wide by 6-inch thick concrete scour pad and a batter piling and waler system with a one-foot raise will provide additional system capacity, assure the wall's structural integrity, and provide erosion protection to reduce the likelihood of a brittle failure if the system's capacity is exceeded. In conjunction with these reinforcement and resiliency measures, an additional 3,000 LF of adjacent levee will be raised one foot.

1.10.2.4 Port Arthur Tank Farm

The TSP for reconstruction and resiliency features for the existing HFPP at the I-wall near the Tank Farm is additional structural support and a one-foot raise. Construction of 1,800 LF of 15-foot-wide by 6-inch thick concrete scour pad and a batter piling and waler system with a one-foot raise will provide additional system capacity, assure the wall's structural integrity, and provide erosion protection to reduce the likelihood of a brittle failure if the system's capacity is exceeded. In conjunction with these reinforcement and resiliency measures, an additional 7,000 LF of adjacent levee will be raised one foot.

1.10.3 Freeport and Vicinity CSRM Project (reference Plate G-3)

1.10.3.1 South Storm Levee Raise

The South Storm Levee was determined not to be an economically viable option.

1.10.3.2 Freeport Dock Floodwall Raise

The TSP for reconstruction and resiliency features for the existing HFPP at the Freeport Dock floodwall is a reconstruction of the existing wall/drop in panel system. The construction will include 3,000 LF of floodwall to meet all USACE requirements for a wall/drop-in panel system for a port facility.

1.10.3.3 Tide Gate One-Foot Raise

The TSP for reconstruction and resiliency features for the existing HFPP at the I-wall located at the Tide Gate is a complete reconstruction of the I-wall and extension of the reconstructed floodwall to replace two (2) additional sections of floodwall at a one-foot raise in system capacity. Construction will consist of 700 LF of floodwall, 11 feet high, along with 2,000 LF of levee raised one foot.

1.10.3.4 Old River at Dow Thumb

The TSP for reconstruction and resiliency features for the existing HFPP along the Dow thumb levee reach is a one-foot raise in system capacity. Construction will include 4,000 LF of 15-foot-wide by 6-inch thick concrete scour pad along with 3,000 LF of levee raised one foot and 14,500 LF of high performance turf reinforcement mattress (HPTRM). Additional material will be added in order to “level up” the low spots of this reach of levee, which, in conjunction with the erosion protection, will to reduce the likelihood of a brittle failure if the system’s capacity is exceeded.

1.10.3.5 Dow Barge Canal Protection

The TSP for reconstruction and resiliency features for the existing HFPP at the Dow Barge Canal levee is a closure structure and pump station constructed at the junction of the North Barge Canal and East Storm Levee across the barge canal to the South Barge Canal Levee. This structure will allow barge traffic to pass during routine operations and will have a pumping capacity of 2,000,000 gpm. The structure will be approximately 500 feet in length with two (2) sector gates to provide for an 80-foot-wide navigable opening. Tidal circulation will be maintained by two (2) sluice gates, each approximately 15 feet wide. The structure’s final configuration will match the proposed level of protection for the system.

1.10.3.6 East Storm Levee Raise

The TSP for reconstruction and resiliency features for the existing HFPP at the East Storm Levee is a one-foot raise in system capacity. Construction includes 13,115 LF of levee raised one foot along with HPTRM installation for erosion protection.

1.10.3.7 Oyster Creek Levee Raise

The TSP for reconstruction and resiliency features for the existing HFPP at the Oyster Creek Levee reach is a one-foot raise in system capacity along with extending an elevation to account for changes in surge loading. Construction will include 3,500 LF of a 3-foot levee raise and 10,000 LF of a one-foot levee raise for a total raised distance of 13,500 LF. The construction procedure will include stripping, removal and replacement of a 12-foot-wide asphalt road, placement of compacted fill, and turfing.

2 HYDROLOGY AND HYDRAULICS

2.1 INTRODUCTION

This section discusses the Hydrologic and Hydraulic (H&H) data gathering efforts and engineering analyses for the Sabine to Galveston DIFR-EIS. The H&H analysis was included as part of the information used to select the NED plans and TSP.

The focus areas of the study, the Brazoria and Sabine Regions, contain a complex system of floodwalls, earthen levees, pump stations, culverts, sluice gates, flap gates, sector gates, hazardous material sites, small local levees, large adjacent rivers and vertical lift gates. Two large sector gates are recommended as part of the tentatively selected plan on Cow and Adams Bayous in Orange County. A complete SMART planning feasibility level of analysis of the storm surge, waves, run-up, and interior drainage features including culverts and pumps was performed. Additionally, RSLC and climate change were estimated and the impacts assessed in accordance with Engineering Regulation (ER) 1100-2-8162 and Engineering Technical Letter (ETL) 111-2-1. Levee height analyses were performed and overtopping rates calculated for all levee systems in the two study regions following guidance in planning guidance documents (as of 2015), Engineering Circular (EC) 1110-2-6067, ETL 1110-2-299, and Code of Federal Regulations (CFR) 44 and 45. The Brazoria Region contains Brazoria County and the Sabine Region contains Jefferson and Orange Counties. Some surrounding areas such as Southern Louisiana near the Sabine Region were also included in the current study to determine, assess, and minimize adjacent impacts.

An overall view showing the locations of the Brazoria and Sabine Regions is presented in Figures 2-1, 2-2, and 2-3. Three major rivers flow through these regions. The Neches River and Sabine River flow through the Sabine Region, with the Neches River delineating the county line between Jefferson and Orange Counties, and the Sabine River which marks the state line between Texas and Louisiana. In the Brazoria Region, the Brazos River flows along the west side of the existing Freeport and Vicinity HFPP. Additional important and also navigable waterways in the project study areas are discussed in further detail later in this section of the appendix.

2.1.1 Brazoria Region

The H&H analysis of Brazoria Region focused on detailed analysis of the existing Freeport levee system and its components, and how the existing system should be modified to prevent failure and increase the level of protection where needed. The Freeport levee footprint will change minimally or not at all, with some exceptions, between the with-project and without-project conditions with the exception of the proposed DOW barge canal gate. The system is being evaluated for an array of possible improvements and modifications. Improvements considered include raising the levees, changing the levee embankment geometry and height, acquiring rights-of-way (ROW), and constructing resiliency features (e.g., concrete scour pads on the interior side of the levee/floodwall and drainage features to manage overtopping flows). Additionally constructing a large closure structure with a navigable opening to block one of the longest and weakest sections of the system along a barge canal from being impacted by surge. Areas where these changes are recommended will have corresponding change in footpring and possibly change in the levee alignment. Detailed analysis of the Freeport levee system was performed is discussed in Section 2.9. Improvements to or replacement of existing drainage features, such as pumping stations, to accommodate changes in flows due to improvements/modifications were preliminarily evaluated. Features needed to create a resilient and robust levee system were identified. An aerial view of the existing Freeport and Vicinity HFPP is shown in Figure 2-1.

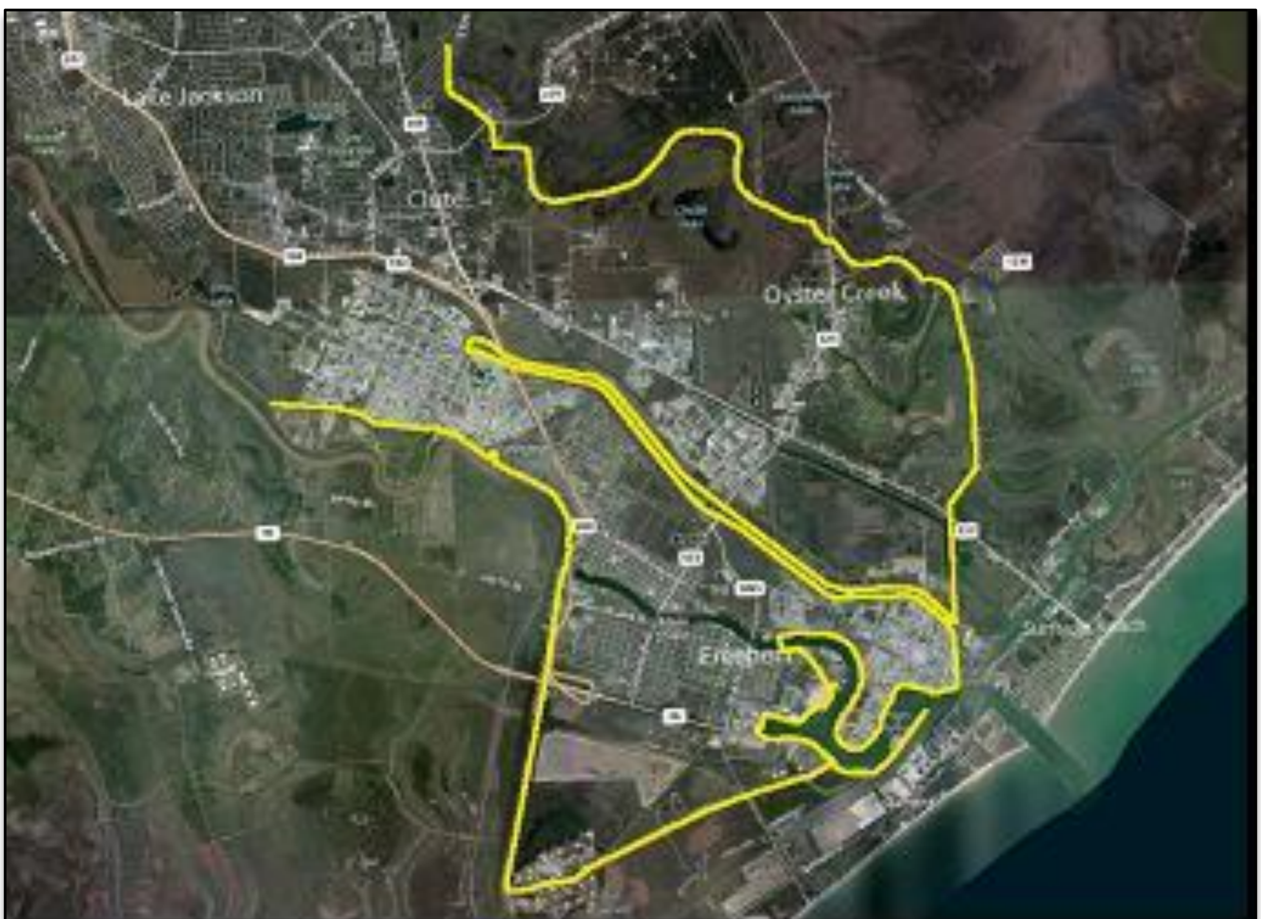

Figure 2-1: Existing Freeport and Vicinity HFPP

2.1.2 Sabine Region

The Sabine Region contains an existing levee system – the Port Arthur and Vicinity HFPP (Figure 2-2), which was evaluated from an H&H perspective using the same procedures and methodology as the existing Freeport levee system. The City of Port Arthur is in Jefferson County on the west bank of Sabine Lake. A deep-draft navigation channel is immediately in front of the Port Arthur levee frontal wall facing Sabine Lake (mostly reinforced-concrete floodwall, 15+ feet high). The Sabine Region study area also contains proposed new levee systems in Orange and Jefferson Counties (Figure 2-3). The new levee systems studied included levees protecting the majority of Orange County south of Interstate 10, as well as additional components in Jefferson County along the west bank of the Neches River. An analysis was performed on the proposed Orange system to determine levee heights, required drainage features, and adjacent impacts. This included new storm surge and wave modeling. A preliminary analysis and design of an interior drainage system and levee components for the proposed Orange County system was completed, including drainage structures such as culverts, gates, and pump structures for Adams and Cow Bayous.

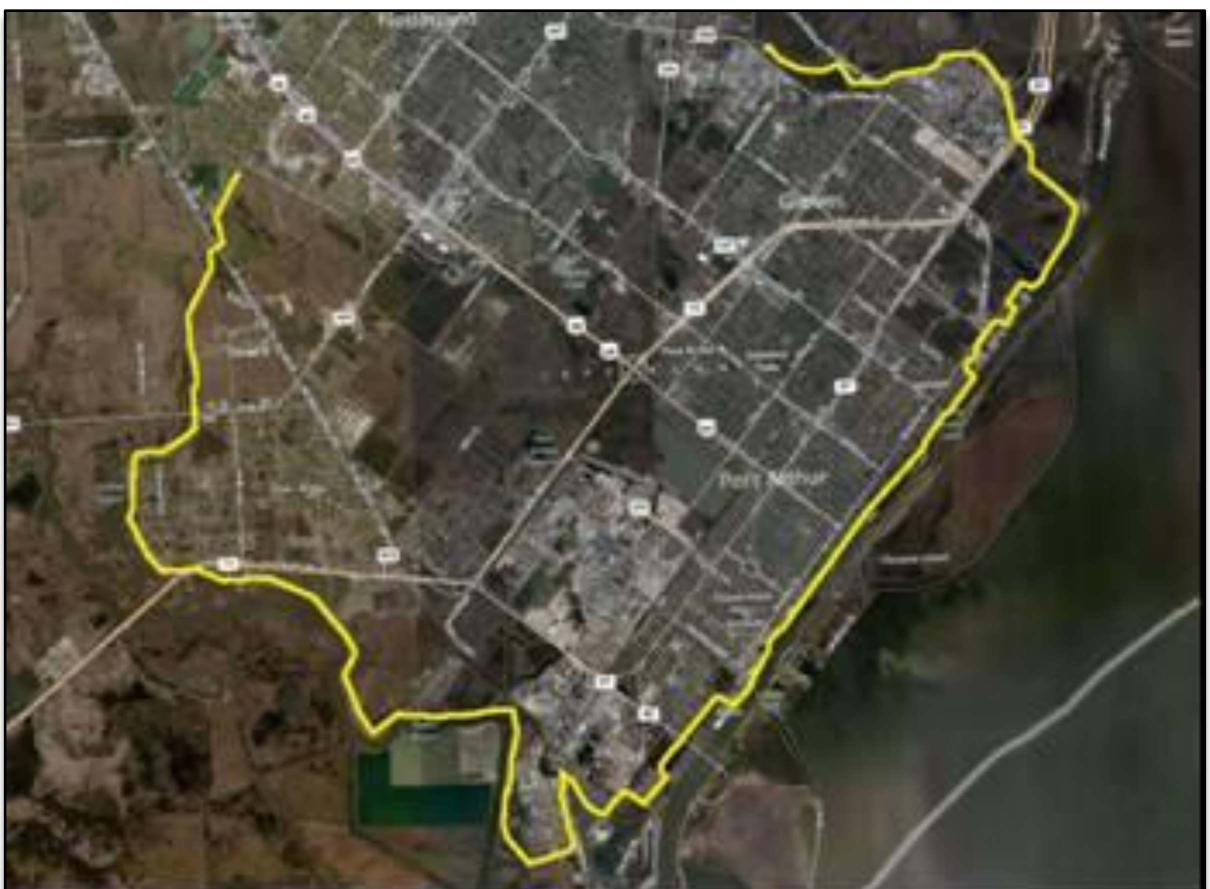

Figure 2-2: Existing Port Arthur and Vicinity HFPP

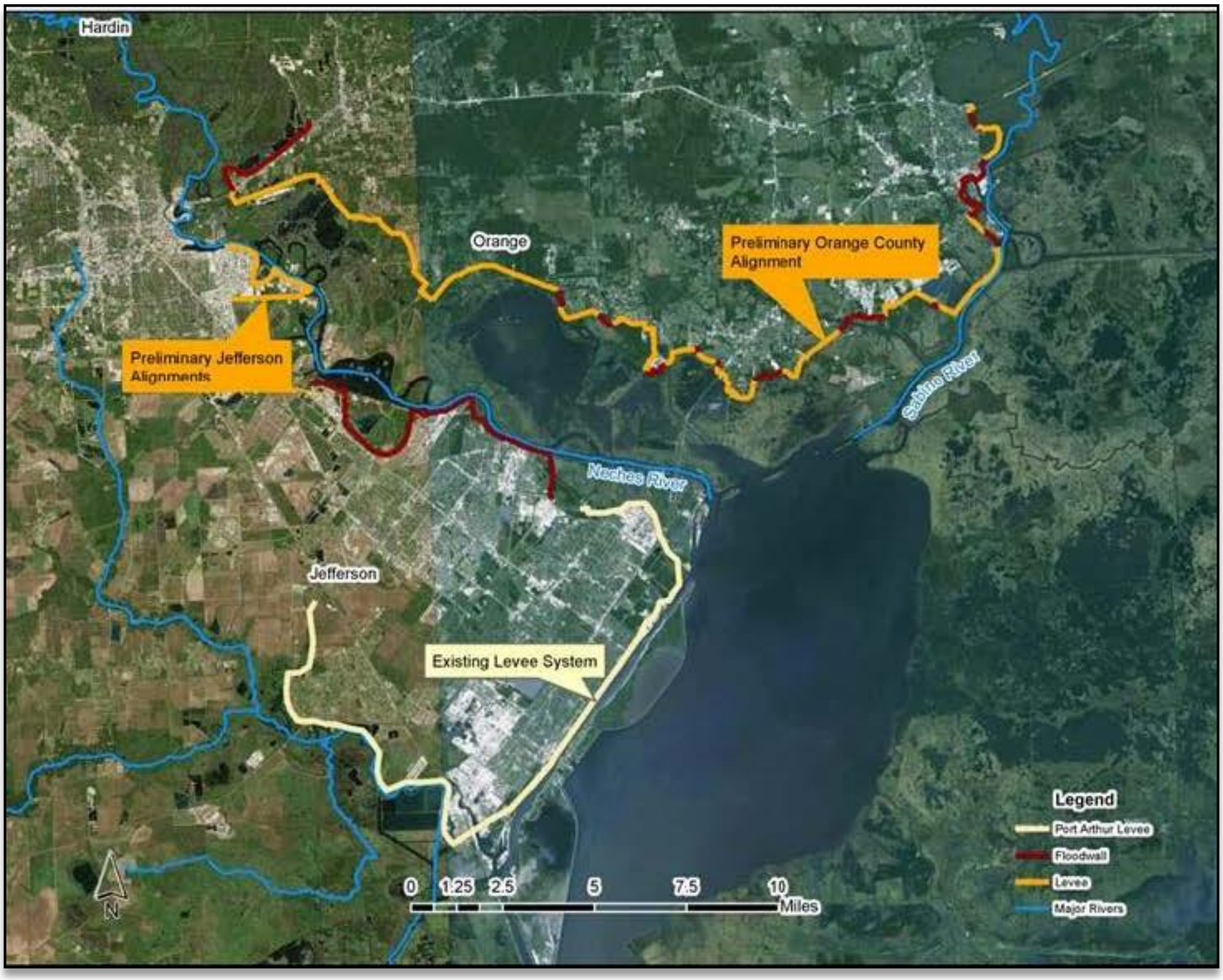

Figure 2-3: Maximum Extents of Proposed New Orange and Jefferson Counties Levee Systems Considered

New levee systems were also proposed and evaluated in Jefferson County across the Neches River from Orange County. Impacts from the proposed levee were assessed and levee heights were recommended accordingly. The locations and configurations of interior drainage features such as culverts and pumps were identified using existing plans and aerial photography. Deficient pumps or pump station requiring modification were identified with the help of levee periodic inspection reports and Jefferson Drainage District #7. The storm surge modeling is discussed in detail in Section 2.9. Interior drainage is discussed in Section 2.14.

2.2 REGIONAL DATA

2.2.1 Units and Coordinate System

All units are in International System (SI) of units unless stated otherwise. Vertical and elevation data is in feet, referenced to NAVD 88 datum unless noted otherwise. Horizontal coordinates shown are in Universal Transverse Mercator (UTM) Zone 15. The project horizontal datum is NAD 83.

2.2.2 Gage Data

Measured age data including water levels, flows, and velocities in the study area was obtained from NOAA. Local gages used to gather data include Gage 8772440 in the Brazoria Region and Gages 8770520 and 8770570 in the Sabine Region. Gage Locations are shown in Figures 2-4 and 2-5. Additionally Discharge data is available from the Neches River gage was utilized.

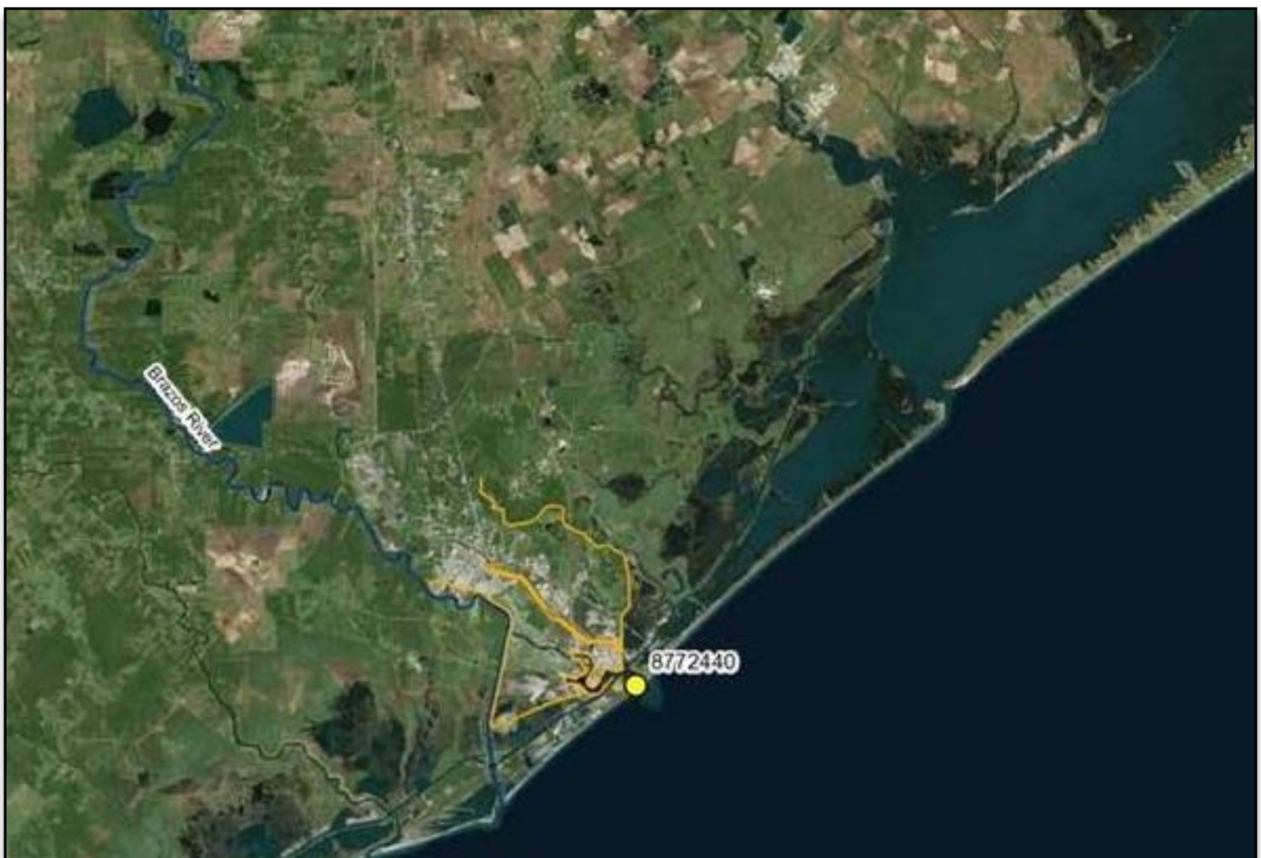

Figure 2-4: NOAA Gage 8772440 in Brazoria Region

Figure 2-5: NOAA Gages 8770570 and 8770520, Sabine Region

2.2.3 Datum and Tidal Information

Elevations in this report are referenced to NAVD 88 unless noted otherwise. Much of the data gathered used datums such as Mean Sea Level, Mean Low Tide, Mean Low Water, and Mean Lower Low Water. For easy reference, the conversion between datums for each region is presented in the Table 2-1 on the next page. For additional information on datum conversion, reference EM 1110-2-6056.

Tides in the Brazoria Region of the study area range from a low ebb tide of -0.28 feet (MLLW) to a high flood tide of 1.78 feet (MLLW). (NOAA Gage 8772440)

Tides in the Sabine Region of the study area range from a low ebb tide of 0.62 feet (MLLW) to a high flood tide of 1.22 feet (MLLW). (NOAA Gage 8770520)

Table 2-1: Datum Conversion Summary Table for Brazoria and Sabine Region

Datum	Freeport Region Gage 8772440 Value (feet)	Sabine Region Gage 8770520 (Rainbow Bridge) Value (feet)
MHHW*	5.61	4.96
MHW*	5.67	4.89
MTL*	5.00	4.45
MSL*	5.00	4.48
MLW*	4.32	4.00
MLLW*	4.05	3.90
NGVD29**	6.43	5.23
NAVD88**	3.41	3.72
Source - http://tidesandcurrents.noaa.gov/stations.html?type=Datums		
Conversion Source -USACE Survey, OPUS Control Monuments_Rizzo		

*Asterisk indicates water-based datum, **Asterisk indicates land-based datum

2.3 EXISTING DATA COLLECTION

2.3.1 Data Collection Summary

An extensive amount of existing data was gathered for both regions for use in this study. A wide variety of reports, models, design plans/as-built plans, feasibility studies, historical studies, watershed master plan studies, and multiple other various studies. The reference section of this document contains a list of data used for the HH&C analysis. Additionally, the latest available tools including the Coastal Hazard System, IWR Suite, the latest ERDC storm modeling capabilities, and ERDC environmental modeling capabilities, among others, were used in this study to the extent possible. It is recommended that some of the modeling be revisited when the project features are designed for construction as the models will be improved and site conditions could change.

2.3.2 Topographic, Bathymetric, and Survey Data

Detailed terrain data were obtained for the study area in the form of LiDAR data obtained from the Texas Natural Resources Information System (TNRIS). LiDAR data for Brazoria/Orange/Jefferson County, dated 2010, were at a 1-meter resolution. Additional terrain data were taken from existing surveys, USGS contour maps, and aerial photography. Historical reports, historical photography, and historical terrain maps were used where applicable. Bathymetry data for the study regions was obtained from the NOAA Geophysics Bathymetry and Global Relief World Data Services. Survey data in USACE's possession and ADCIRC mesh elevation data were also used. Recent survey data from other projects and construction plans were used to cross-reference the elevations in the data USACE is using. These data were compared to the latest ADCIRC mesh data. In areas where the gathered topography, survey and bathymetry data contained more recent and/or more accurate data, the ADCIRC mesh data were updated. Some previously used data, prior to USACE analysis, were found to be incorrect and those errors were fixed before modeling began.

2.3.3 FEMA Data

The FEMA Coastal Counties Report (2011) and associated electronic files were obtained and used in this study. The Coastal Counties Study covered the entire Texas coast. USACE obtained flood inundation maps from FEMA for the 10-, 50-, 100-, and 500-year still-water levels (SWL). These data were used in the screening for the Alternative Milestone Meeting (AMM) and Workshop (Reference Appendix B For information on screening). The data were later refined and stage-frequency data for the 10-, 20-, 50-, 100-, 200-, 500- and 1,000-year events were obtained utilizing ADCIRC modeling capabilities to determine the surge elevations for a range of different storms at points blanketing the area. ADCIRC coupled with STWAVE for the Brazoria Region was rerun. Statistical analysis of the data generated was conducted using StormSim, a computer program recently developed by ERDC to produce results that are more accurate. ADCIRC was coupled with STWAVE for the Sabine Region for the without-project and with-project conditions. The statistical analysis, again, was performed using StormSim. The ADCIRC runs for the Sabine Region contained additional storms not modeled in the 2011 FEMA study. The surge modeling and data were provided to assist in selecting the TSP. Additional discussion regarding the use of the FEMA data and storm surge modeling and accompanying analysis is discussed in Section 2.5.

2.3.4 Tropical Cyclones and Floods of Record

Both regions have a long history of being subjected to large topical events, including major hurricanes. The earliest documented hurricane in Texas made landfall at Matagorda Bay in 1527. For a complete list of documented hurricanes in Texas reference “Texas Hurricane History” authored by David Roth of the National Weather Service, last updated in 2010. The hurricane with the greatest known loss of life is the 1900 Galveston Storm with deaths estimated at between 6,000 and 12,000 people. The hurricane striking the Texas Gulf Coast with the greatest economic damages is Hurricane Ike, in 2008. A list of major tropical events that have impacted Texas since the 1850s is given in Table 2-2 (Reference: David Roth, NWS, 2010).

Table 2-2: Tropical Cyclone Strikes by Decade since 1850

Tropical Cyclone Strikes By The Decade			
Decade	**Hurricanes**	**Tropical Storms**	**Total**
1850's	3	1	4
1860's	4	1	5
1870's	2	4	6
1880's	8	3	11
1890's	3	3	6
1900's	4	2	6
1910's	7	1	8
1920's	2	3	5
1930's	5	4	9
1940's	8	6	14
1950's	2	5	7
1960's	3	3	6
1970's	2	7	9
1980's	5	4	9
1990's	1	4	5
2000's	5	5	10
Total	64	56	120
Avg Per Decade	4	3.5	7.5

Tropical cyclones impacting the study area have included major hurricanes such as the Galveston storms of 1900 and 1915, the unnamed hurricane in 1943, Hurricane Carla in 1962, Hurricane Rita in 2005, and Hurricane Ike 2008. These storms have caused considerable loss of life and extensive economic damages. As evident in Table 2-2, tropical cyclones along the Texas Gulf Coast occur frequently, averaging 7.5 per decade. This includes an average of four (4) hurricanes per decade over the last 150+ years. It is important to note that these are storms which made landfall in Texas. They do not include storms that made landfall in Louisiana or Mexico, but had significant impacts of surge and/or wind to the Texas coast, such as Hurricane Katrina in 2006.

2.4 SUMMARY OF PROJECT AREAS AND H&H OBJECTIVE

The coastal areas of Texas are low-lying and highly populated areas, and include a vast variety of recreation, population centers, residential areas, businesses, and industrial infrastructure. The area is subject to frequent tropical cyclones, which historically have caused great loss of life and high economic losses.

The objective of the CSRM study is to evaluate alternatives that would reduce coastal storm damages in the Brazoria and Sabine Regions. H&H analyses are key components in determining a plan to reduce damages from storm surge. This section of the engineering appendix presents an overview of the H&H efforts performed to inform, evaluate, and provide support to USACE efforts in determining the TSP to reduce damages from storms along the Texas coast.

2.5 ANALYSIS OF HYDRAULIC LOADING FOR FLOOD PROTECTION SYSTEMS

The following documents the analyses that were done to support the design of the proposed hurricane-flood protection systems. Hydraulic analyses were conducted to inform flood-protection system design. Analyses were conducted to support design of the levees and floodwalls. The following parameters were evaluated for design:

- Levees
  - Crest elevation
  - Side slopes
  - Overtopping rate

- Flood walls
  - Crest elevation
  - Side slopes
  - Overtopping rate

In order to calculate design parameters, the following general process was applied:

Gather existing data

1) Analyze storm surge and waves
   a. ADCIRC/STWAVE for large-scale storm surge and waves
   b. Output statistics for various return periods for waves and water levels along with quantification of uncertainty for surge and wave modeling results
2) Local wave transformations using CMS-Wave, where required
3) Overtopping analysis
   a. Existing structures: run-up and overtopping were calculated for the 1 % ACE
   b. Proposed structures: acceptable overtopping limits were specified to calculate freeboard
4) Recommend levee crest elevation based on hydraulic analyses presented

5) Assess any adjacent impacts due to new levee construction or modification to existing levees

2.6 EXISTING DATA SUMMARY

Existing data gathered and reviewed for each study region included:

- Previous surge and wave models and results
- Topographic and bathymetric data, aerial imagery, USGS maps
- Historical data including plans, modification studies, O&M manuals, and design memorandums, local hurricane flood protection plans, FEMA and other Texas coastal surge models and documentation
- Ongoing analysis and reports (for example, the Risk Management Center (RMC) draft report on the Freeport levee system)
- Existing levee alignment data and proposed modification from ports, districts, and cities

Data gathered were summarized using ArcMAP and Excel to simplify comparison and analysis.

2.6.1 Brazoria Region

Existing data were gathered for the Brazoria Region, focusing on data related to the existing Freeport and Vicinity HFPP, to enable hydraulic analyses. Figure 2-6 shows a map of some of the available data including LiDAR data. Readily available data gathered included:

- FEMA DEM of LIDAR data collected in 2007
- Existing levee alignment and elevation data collected for the National Levee Database
- 10-, 50-, 100-, and 500-year return period water levels calculated for the FEMA (2011) study
- STWAVE results from the FEMA (2011) study
- COULWAVE results from Lynett (2012)
- USACE channel survey data
- NOAA bathymetry surveys
- USGS, TCOON, and NOAA water data
- Aerial imagery from various sources
- USACE RMC Freeport Semi-Quantitative Risk Assessment (SQRA)
- Various other reports as referenced in this document

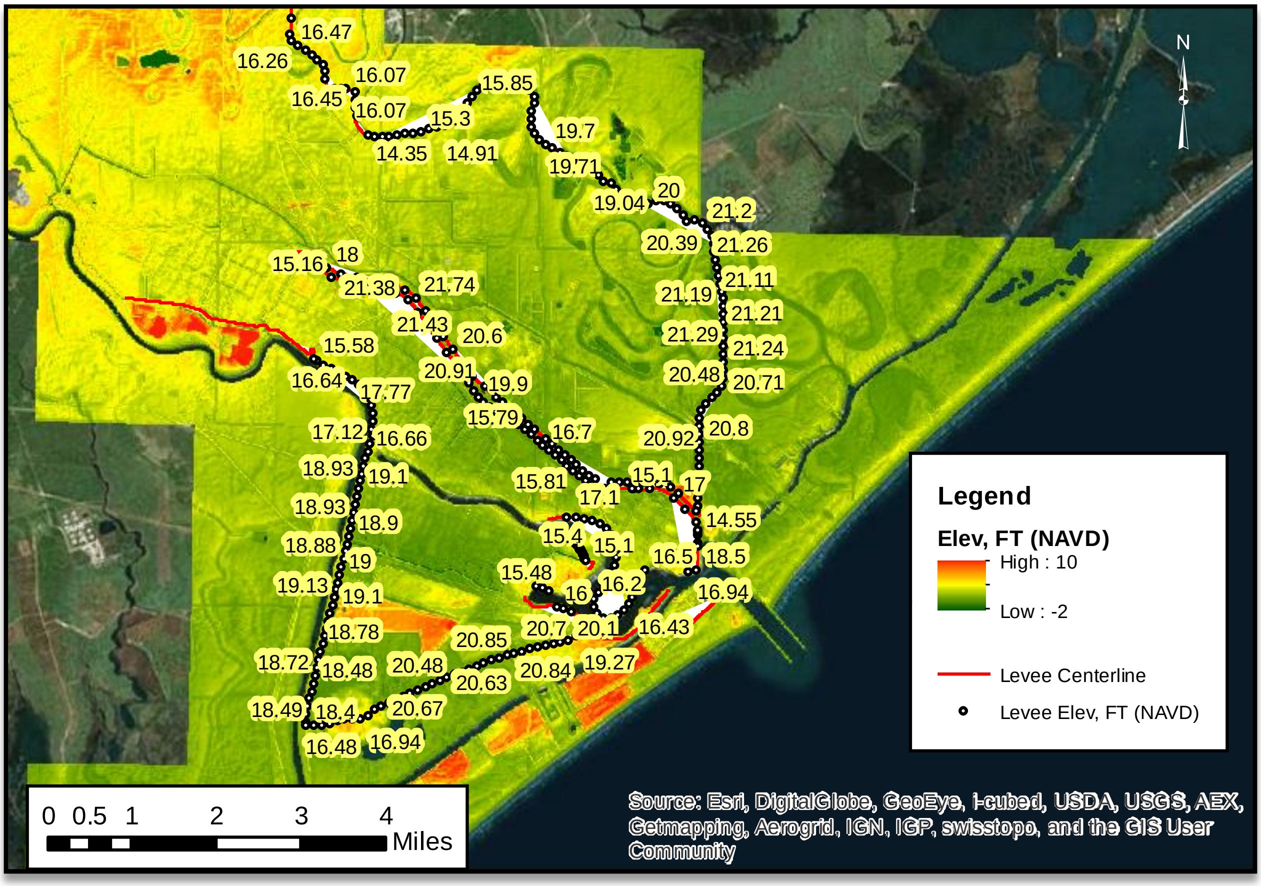

Figure 2-6: Map of Freeport Area with Select Data Gathered

2.6.2 Sabine Region

Existing data were gathered for the Sabine Region related to the storm-protection system to enable hydraulic analyses. Figure 2-7 shows a map of some of the available data including LiDAR data. Readily available data included:

- FEMA Digital Elevation Model (DEM) of LiDAR data collected in 2007
- Existing levee alignment and elevation data collected for the National Levee Database
- 10-, 50-, 100-, and 500-year return period water levels calculated for the FEMA (2011) study
- STWAVE results from the FEMA (2011) study
- Flood Protection Planning Study, Hurricane Flood Protection System, Orange County, Texas report (Carroll & Blackman, Inc. 2012)
- Neches River Corps Water Management System (CWMS) and Modeling, Mapping, and Consequences (MMC) hydraulic models
- Adams Bayou Hydraulic Modeling and Drainage Area Delineation for Cow and Adams Bayou (Carroll & Blackman, 2015)

- USGS, TCOON, and NOAA water level data
- USACE channel survey data
- NOAA bathymetric surveys
- Aerial imagery from various sources
- Various other reports as referenced in this document

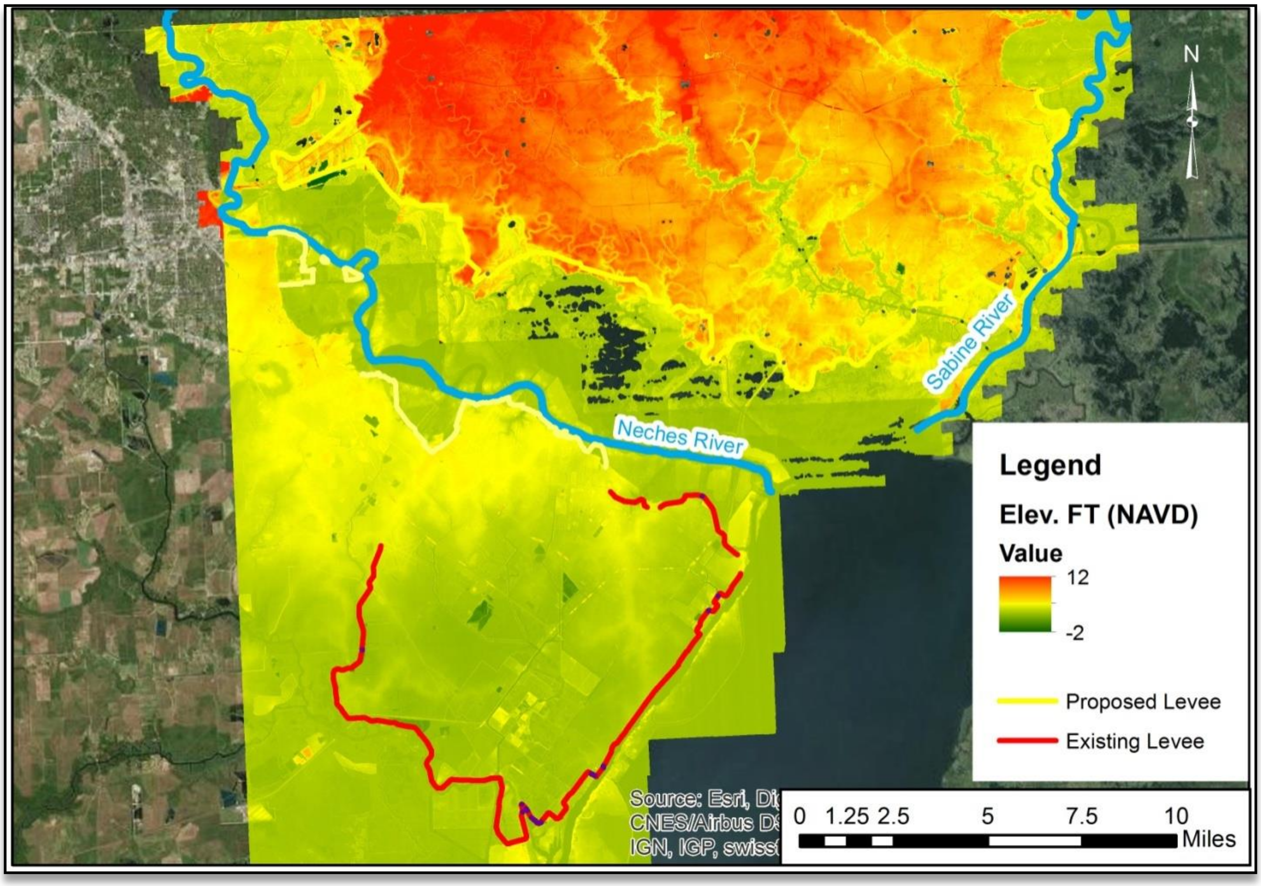

Figure 2-7: Map of Sabine Region with Select Data Gathered

2.7 ANALYSIS OF STORM WATER LEVEL AND WAVES

This section summarizes the modeling of storm surge and correlated waves for the with-project and without-project conditions.

2.7.1 Storm Modeling Approach

ERDC-CHL completed a storm modeling study of the Brazoria and Sabine Regions. The process applied and details of the model setup and accuracy are discussed in the ERDC report (Melby, et al 2015). The study took advantage of previous modeling efforts conducted for the FEMA Region VI NFIP Risk Mapping, Assessment and Planning (MAP) study reported in

FEMA (2011). In general, hydrodynamic modeling of the offshore waves and coupled surge and nearshore waves that was conducted for the FEMA study was used for the current study. However, all storm statistics as well as response statistics were recomputed using new software developed at ERDC called StormSim. In addition to improved statistical analysis, new modeling was conducted to improve some of the prior modeling and to model the with-project alternatives for the Sabine Region (including the proposed new levee systems in Jefferson and Orange Counties). A joint probability model of tropical storm parameters was developed specifically for this study. Storm recurrence rates for the coastline were defined. Previously-defined synthetic tropical storms, as summarized in FEMA (2011), were mapped onto the new joint probability model. These storms had a frequency range of roughly 1 in 10 years to 1 in 1,000 years. For the FEMA study, 446 synthetic tropical storms were defined and modeled for the Texas coast. These included 152 low-frequency storms and 71 high-frequency storms for both the north and south Texas regions. In addition, 152 Louisiana west low-frequency and 71 high-frequency storms defined in FEMA (2008) were used for the FEMA study.

2.7.1.1 Brazoria Region

For the Brazoria Region, nearshore waves and water levels modeled during the FEMA (2011) study were used to compute statistics for the Freeport area. There was no change in levee alignment for the with-project condition, so no additional with-project modeling was conducted for this area.

Extremal statistics for waves and water levels were computed using StormSim and the joint probability method. In addition, confidence levels were computed. Mean water level, wave height, and wave period responses were defined for return periods of 10, 20, 50, 100, 200, 500, and 1,000 years at save points in the Freeport area, including points surrounding and within the Freeport levee system. The average, standard deviation and 90, 95, and 98 percent levels of assurance were also calculated. Finally, the integrated uncertainty as used in the FEMA study was computed for comparison to the FEMA (2011) results to help verify quality. Figure 2-8 shows the 1 % ACE average water level at Freeport. Figure 2-9 shows the 1 % ACE upper 90 percent level of assurance at Freeport. Figure 2-10 shows plots of the return period of water level for some key locations around Freeport.

Inundation maps comparing the with-project and without-project conditions based on the FEMA (2011) 100-year storm level are presented in the Plate H-01 as part of this Engineering Appendix - H&H Support Documentation.

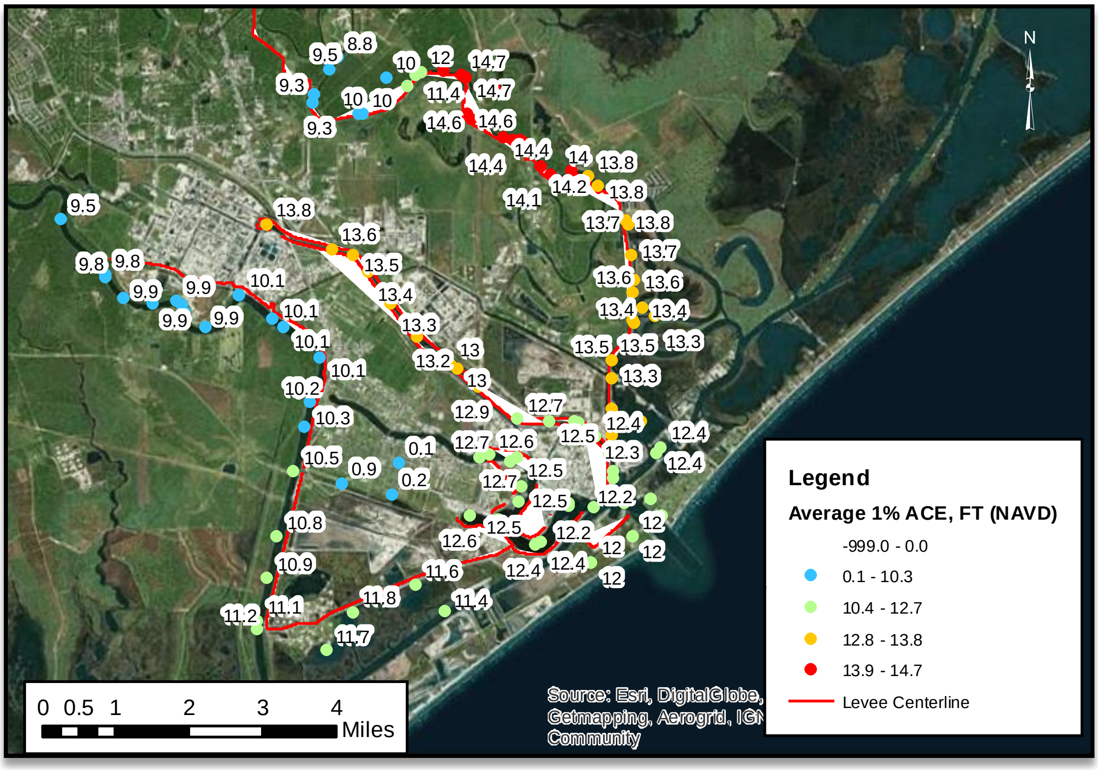

Figure 2-8: Average 1.0% ACE Still-Water Level at Freeport

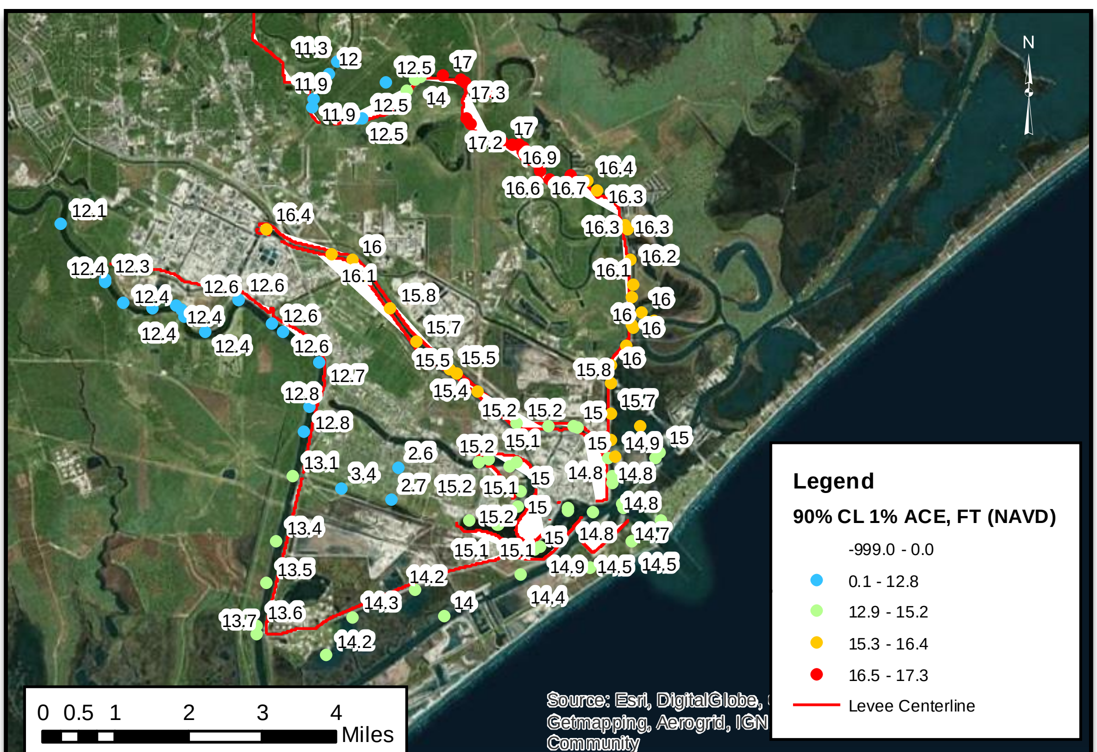

Figure 2-9: 90% Confidence Level for the 1.0% ACE Still-Water Level at Freeport

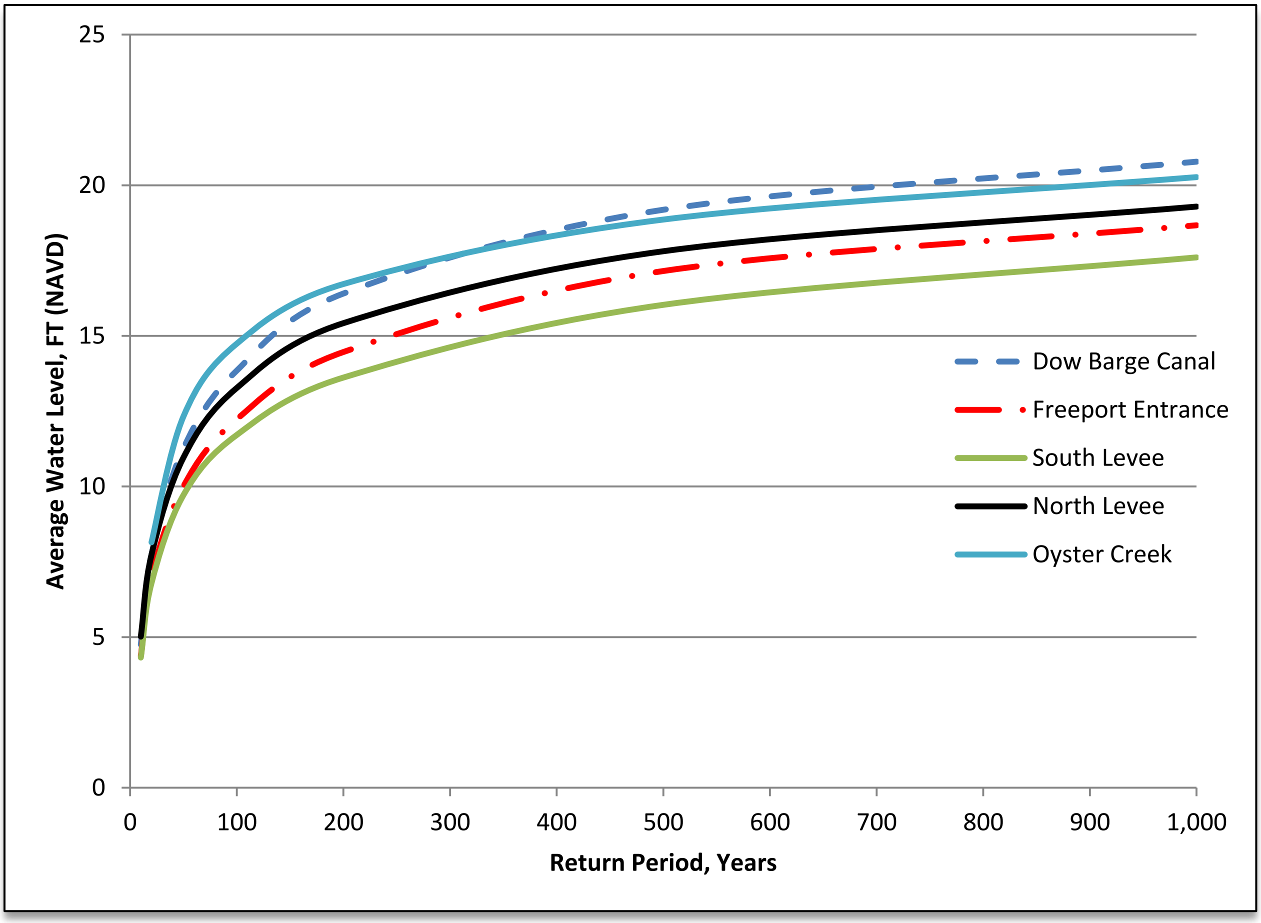

Figure 2-10: Example of Average Water-Level Return Periods at Freeport

2.7.1.2 Sabine Region

Nearshore waves and water levels modeled during the FEMA study were used to compute initial statistics. Comparison of the initial statistics with previous analyses indicated that some of the storms were previously run on the Louisiana grids only. Thus, there was concern that including these storms might dramatically increase the water levels for the Sabine Region projects. Additional model runs were conducted to include these storms and test this assumption. The results indicated that the additional storms did not increase the water levels.

New model grids were developed to include the proposed levee alignments. The same suite of storms was run on both grids and the same statistical analysis was run on both sets of results. Extremal statistics for waves and water levels were computed using StormSim and the joint probability method. Confidence levels were also computed. Mean water level, wave height, and wave period responses were defined for return periods of 10, 20, 50, 100, 200, 500, and 1,000 years for save points in the Sabine Region, including points surrounding and within the Port Arthur levee system. The standard deviation and 90 and 95 percent levels of assurance were also calculated. Finally, the integrated uncertainty as used in the FEMA study was computed for comparison to the FEMA (2011) results to help verify results. Figure 2-11 shows the 1 % ACE average water level in the Sabine Region. Figure 2-12 shows the 1 % ACE upper 90 percent level of assurance in the Sabine Region. Figure 2-13 shows plots of the return period of water level for some key locations around the Sabine Region.

Inundation maps comparing the with-project and without-project conditions based on the FEMA (2011) 100-year storm level are presented in Exhibit 2-11.

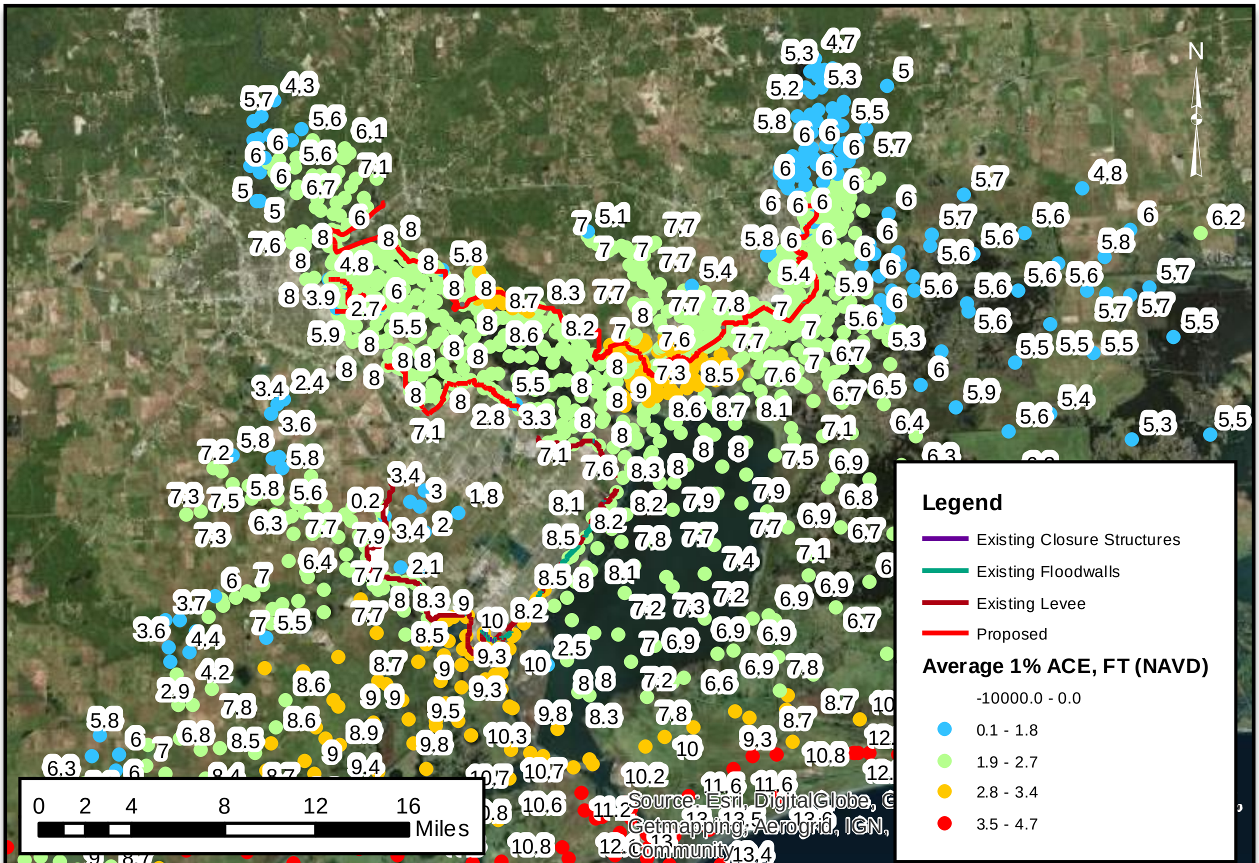

Figure 2-11: Average 1 % ACE Still-Water Level in Sabine Region

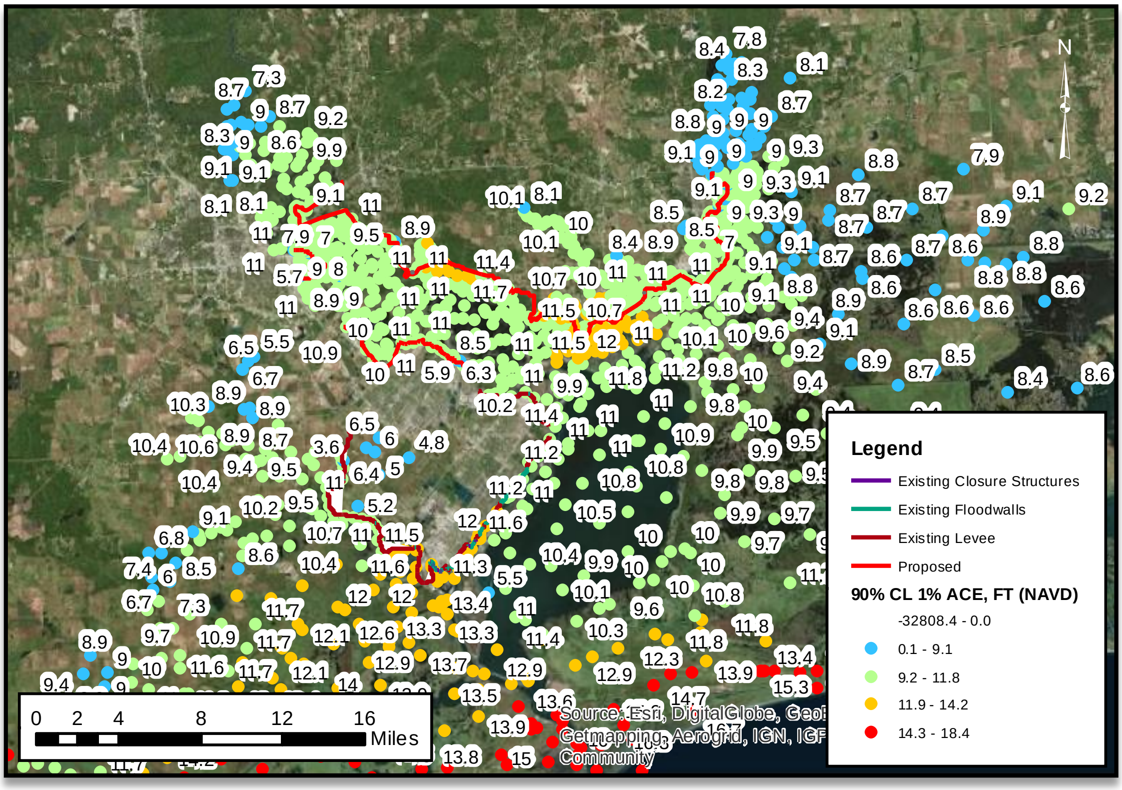

Figure 2-12: 90 percent Confidence Level for the 1 % ACE Still-Water Levels in Sabine Region

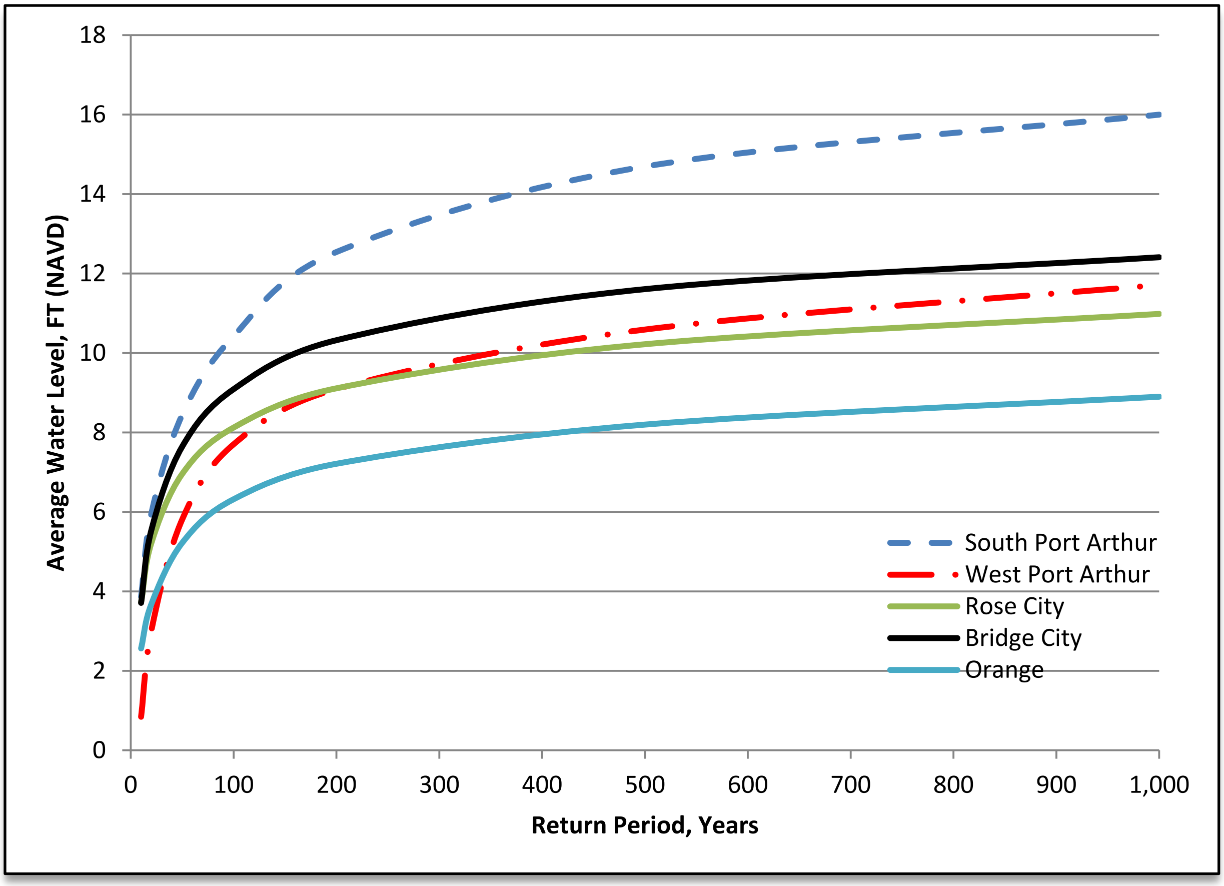

Figure 2-13. Example of Average Water-Level Return Periods in Sabine Region

2.8 LOCAL WAVE TRANSFORMATION AND CALCULATIONS

Local scale wave modeling was required to augment the WAM and STWAVE models run by ERDC. This section describes the local scale wave modeling.

2.8.1 Software Description

- STWAVE (STeady state spectral WAVE)
- STWAVE was applied to simulate wave propagation and generation from the larger scale STWAVE grid to the structure toe. STWAVE is a spectral wind-wave model capable of simulating growth, decay and transformation of wind generated waves, and swell in offshore and coastal areas (Massey, et al, 2011).
- Coastal Modeling System - Wave (CMS-Wave)
- CMS-Wave is a two-dimensional (2D) spectral wave model with energy dissipation and diffraction terms (Lin, et al 2011). The model was applied to help evaluate STWAVE results since no measured wave data were available.

2.8.2 Model Approach and Conventions

Numerical models were developed to simulate wave propagation and generation from the larger scale STWAVE grid to the structure toe.

2.8.3 Direction Convention

Wind and wave direction follow the meteorological convention for input and output, indicating direction of origin (direction from which it travels).

2.8.4 Model Parameters

The following model parameters were specified in the STWAVE model (CMS-Wave setup was similar):

- Depth type: Nontransient
- Source terms and propagation
- No current interaction
- Bottom friction: Manning's estimated and set at 0.03 based on known data
- Constant surge applied
    - Average 1 % ACE applied when waves on the boundary were known
    - 95 percent CL 1 % ACE applied when waves are generated by local winds only
- Winds
    - Based on ASCE (2010)
- 100-year return period
- 20-minute average
- Winds were applied in the worst-case direction to determine the worst-case waves using CMS-Wave and STWAVE.

2.8.5 Calibration and Verification

Measured or observed wave data at the project site were not available for calibration or verification. Since appropriate data for calibration and verification were not available, the solution from STWAVE was compared to results from CMS-Wave and the Automated Coastal Engineering System (ACES).

2.9 MODEL RESULTS

2.9.1 Brazoria Region (Freeport)

Large-scale STWAVE models run as part of the storm simulations for FEMA (2011) were not well resolved near the Freeport levees, requiring transformation of waves from nearshore to the levee toe. Waves inside the Freeport channels were also not resolved by the FEMA (2011) model runs, requiring calculation. Five (5) local grids were modeled with rectangular cells ranging in size from 2 to 20 meters (Figure 2-13). The figure shows the local model domains and the location of waves output from the large-scale storm model suite.

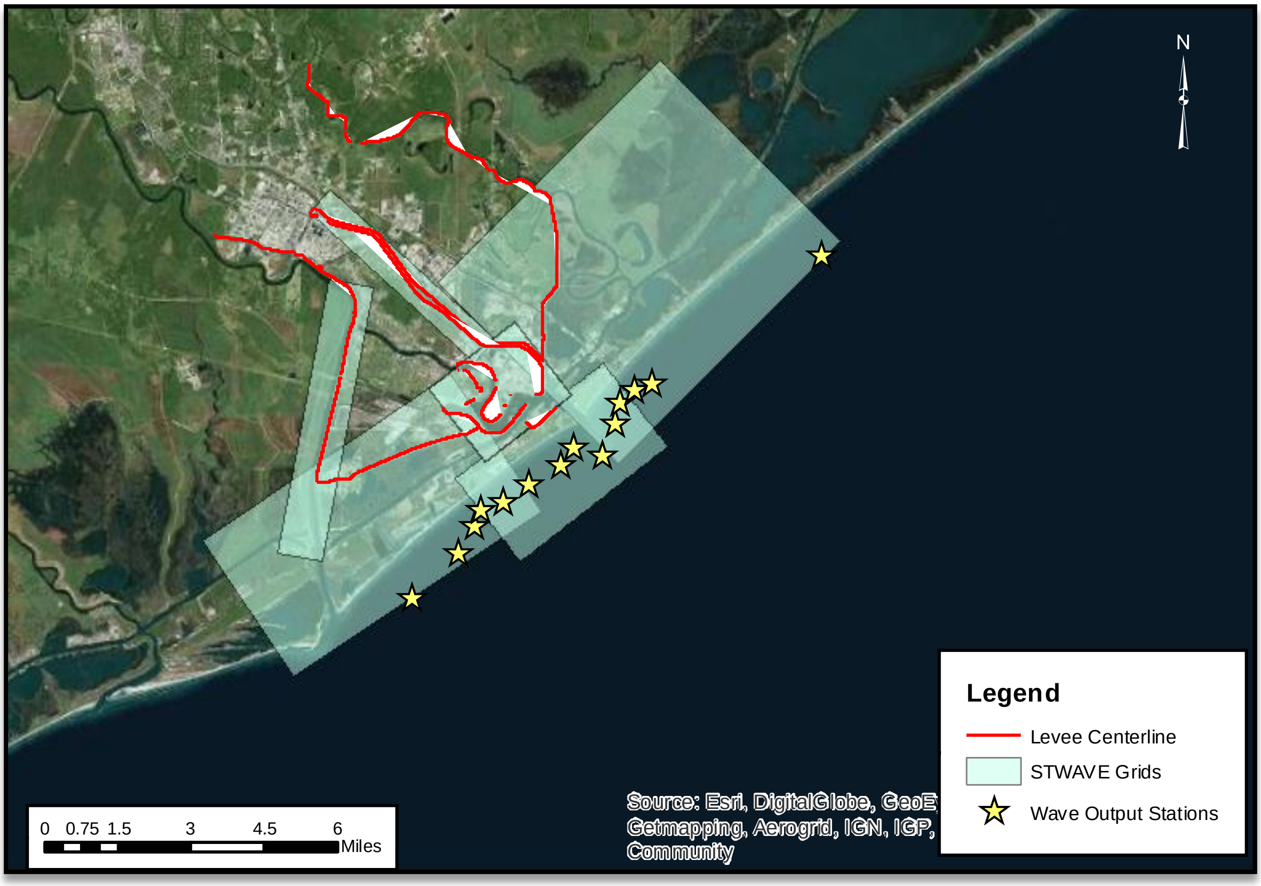

Figure 2-13: STWAVE Model Domains at Freeport

Each domain was forced with winds and offshore waves. The largest waves along each levee reach were summarized from all available runs. Figure 2-14 shows plots of the approximate locations of wave output used to inform the levee design. Table 2-3 lists the zero-moment spectral wave height (Hm0) and peak spectral wave period at each location (Figure 2-14).

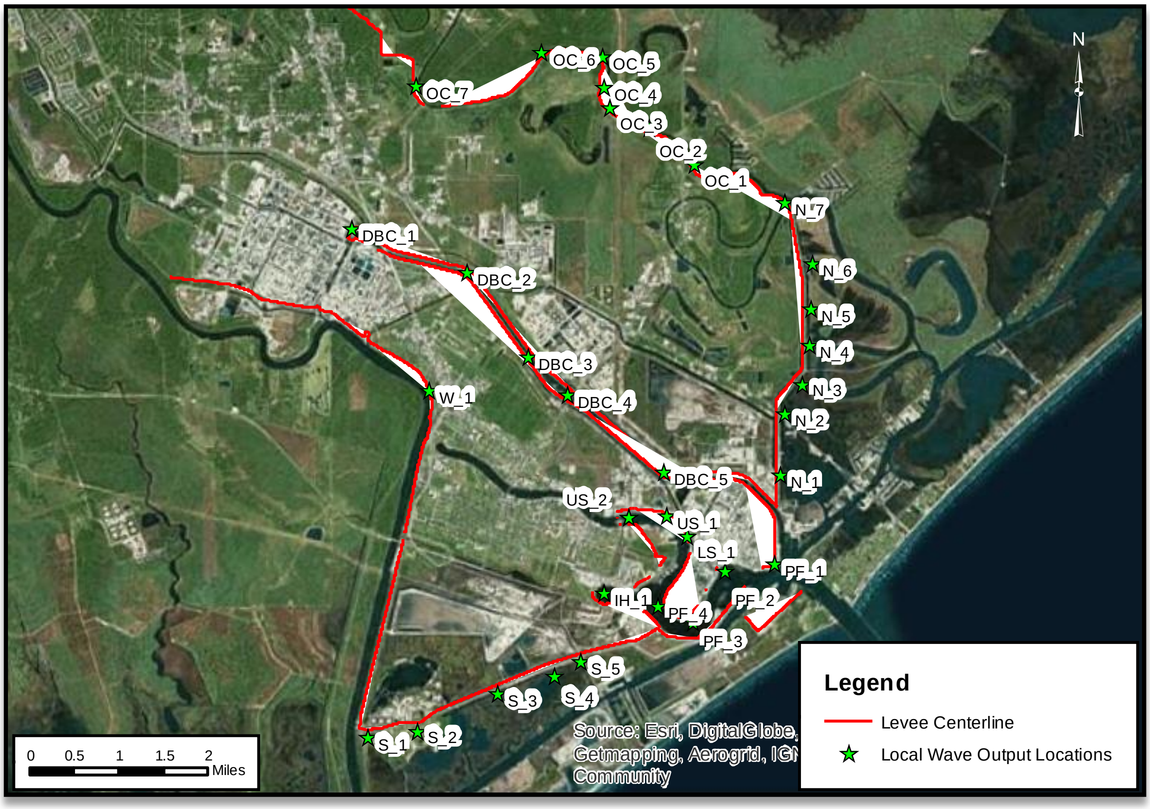

Figure 2-14: Freeport Local Wave Output Locations

Table 2-3: Summary of Local Wave Output for Freeport

Wave Output Station Name	H_{m0}, ft	T_p, s	Wave Output Station Name	H_{m0}, ft	T_p, s
DBC_1	1.28	2.8	PF_4 Phillips	2.23	2.80
DBC_2	1.38	2.8	PF_2	2.23	2.80
DBC_3	1.76	2.8	PF_1	2.53	8.57
DBC_3v2*	1.64	2.8	US_2	2.13	2.80
DBC_4	1.35	2.8	US_2v2*	0.69	2.80
DBC_4v2*	2.23	2.8	IH_1*	1.15	2.80
DBC_5	0.72	2.8	IH_1	2.36	2.80
DBC_5v2*	1.67	2.8	N_1	1.77	8.12
W_1	1.84	4.2	N_2	1.84	9.40
S_1	2.07	3.4	N_3	2.89	9.40
S_2	0.00		N_4	2.46	9.10
S_3	1.25	5.8	N_6	1.87	9.10
S_4	1.35	4.3	N_5	2.13	9.12
S_5	0.52	7	N_7	2.13	8.70
PF_3-wave barrier	2.79	3	LS_1	1.61	3.30
PF_3-thumb	1.51	2.8	US_1	2.33	2.80
PF_4 thumb*	2.00	2.8			

2.9.2 Sabine Region

Large-scale STWAVE models run as part of the storm simulations conducted in this study were not well resolved along all of the proposed and existing systems, requiring wave generation and transformation of waves from nearshore to the levee toe in some locations. Five (5) local grids were modeled with rectangular cells ranging in size from 2 to 20 meters (Figure 2-15). The figure shows the local model domains and the location of waves output from the large-scale storm model suite.

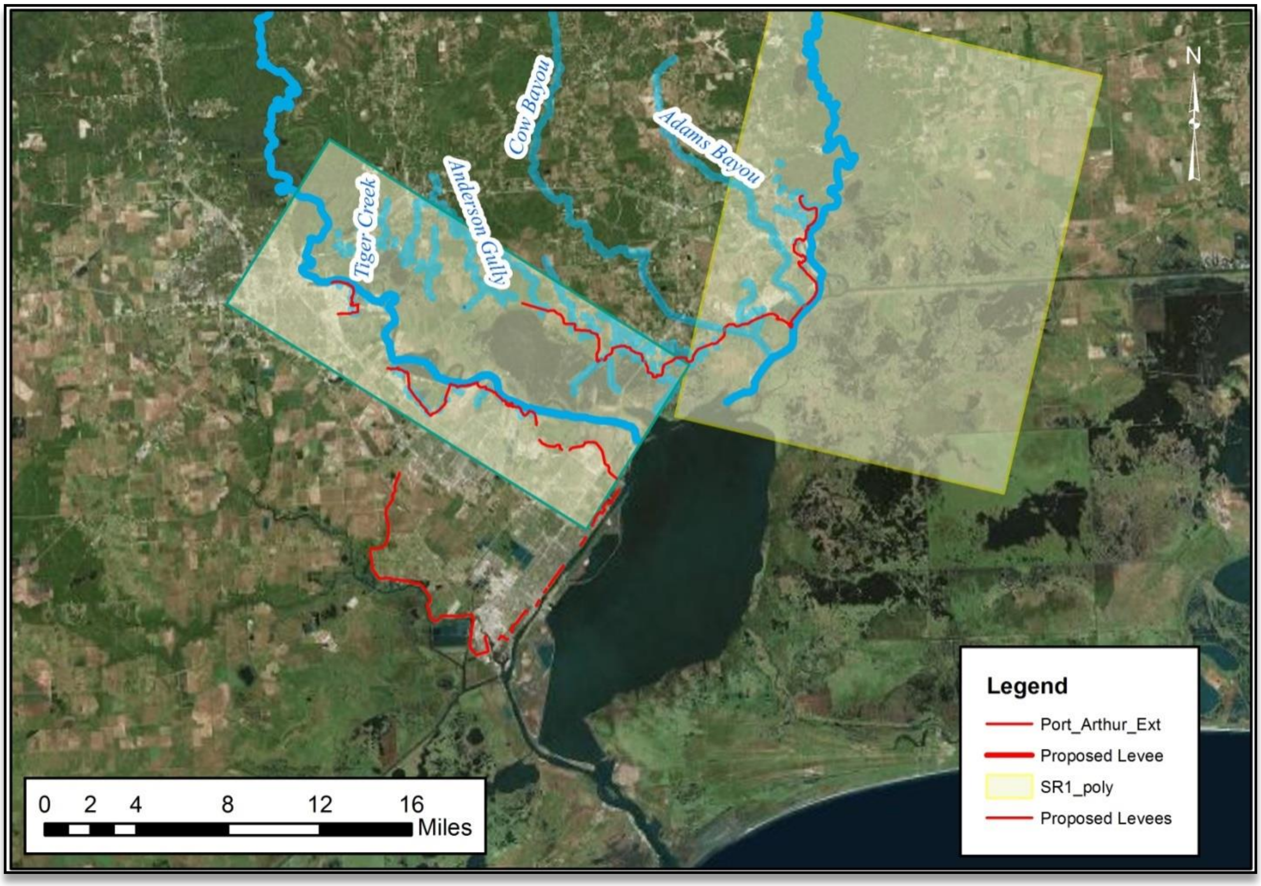

Figure 2-15: STWAVE Model Domains in Sabine Region

Each domain was forced with winds and offshore waves. The largest waves along each levee reach were summarized from all available runs. Figure 2-16 shows plots of the approximate locations of wave output used to inform the levee design. Wave output at the stations shown in the figure are included in Table 2-4. The values in the table are a combination of local wave model results and results from the storm simulation STWAVE models, where appropriate.

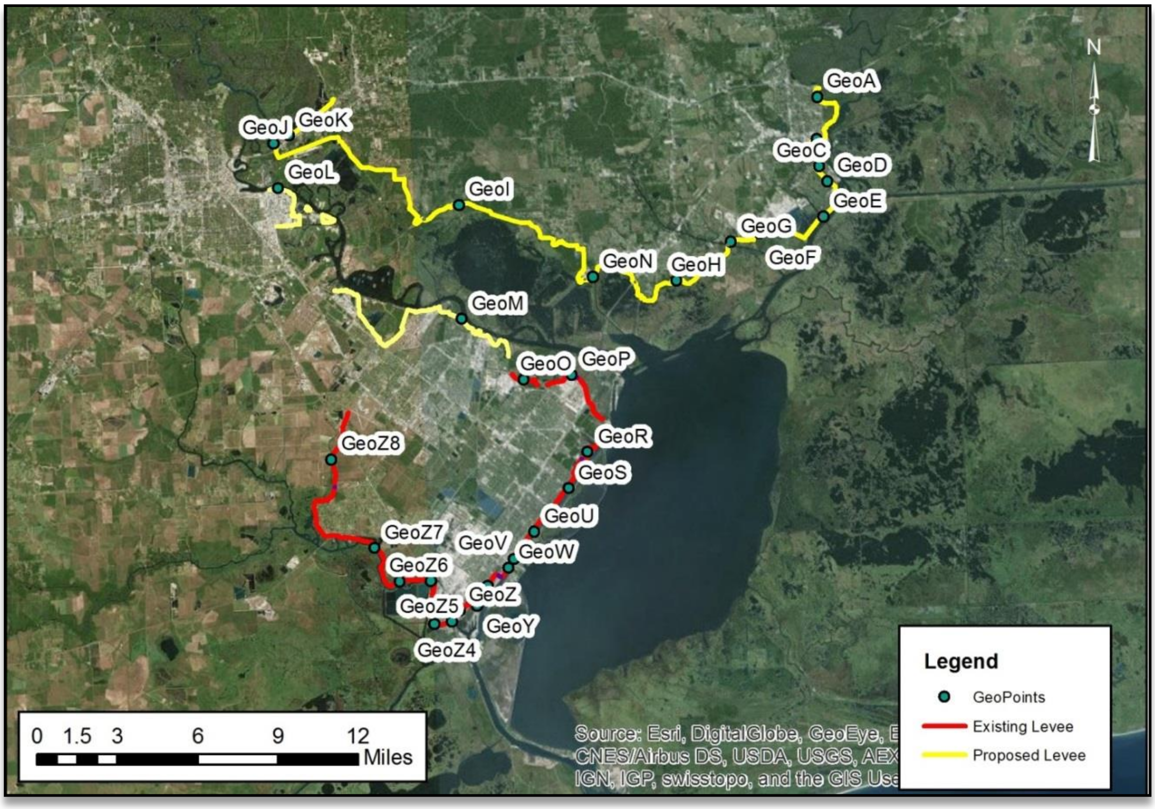

Figure 2-16: Sabine Local Wave Output Locations

Table 2-4: Summary of Wave Output for Sabine Region

Wave Output Station Name	Hm0, ft	Tm0, s	Wave Output Station Name	Hm0, ft	Tm0, s
GeoA	3.9	4.5	GeoR	2.1	5.0
GeoB	2.6	2.9	GeoS	2.3	4.3
GeoC	3.7	4.5	GeoT	2.3	4.9
GeoD	2.0	3.0	GeoU	2.3	4.9
GeoE	2.0	2.9	GeoV	2.3	5.0
GeoF	2.4	4.0	GeoW	2.3	5.0
GeoG	1.0	2.9	GeoX	2.1	4.6
GeoH	1.3	4.5	GeoY	2.3	2.9
GeoI	1.0	3.4	GeoZ	2.1	3.2
GeoJ	2.8	3.8	GeoZ1	2.9	2.9
GeoK	2.4	4.0	GeoZ2	2.9	2.9
GeoL	2.3	2.8	GeoZ3	2.2	3.2
GeoM	2.4	3.2	GeoZ4	1.4	3.0
GeoN	1.3	3.4	GeoZ5	2.0	3.4
GeoO	2.2	4.7	GeoZ6	1.9	3.4
GeoP	2.5	4.5	GeoZ7	2.1	3.4
GeoQ	2.1	4.9	GeoZ8	0.9	3.4

2.9.3 Wave Overtopping Analysis Methods

Wave overtopping calculations were conducted following the EurOtop Manual (Pullen, et al 2007). Both deterministic and probabilistic overtopping equations were calculated. In-house calculations were compared to the online overtopping calculator to verify results (http://www.overtopping-manual.com/). In addition to overtopping, 2.0 percent and maximum run-up values were calculated to inform comparison to typical FEMA requirements. Sensitivity analysis showed that various methods for calculating overtopping are not consistently conservative, meaning that selecting one method over another does not necessarily guarantee lower or higher values. For consistency, the EurOtop deterministic equations were applied to estimate overtopping rates for each reach. The locations shown for wave model output are used to summarize overtopping conditions.

The Hurricane and Storm Damage Risk Reduction System (HSDRRS) design guidelines (USACE 2012) criteria were applied to estimate required freeboard and determine levee crest elevation. The following criteria were applied:

- For the 1.0 percent exceedance still water, wave height, and wave period, the maximum allowable average wave overtopping of 0.01 cfs/ft at 50 percent level of assurance for grass-covered levees.
- For the 1.0 percent exceedance still water, wave height, and wave period, the maximum allowable average wave overtopping of 0.03 cfs/ft at 50 percent level of assurance for floodwalls with erosion-appropriate protection on the back side.

The HSDRRS criteria also include design guidance for the 90 percent level of assurance of no more than 0.1 cfs/ft for both floodwalls and earthen levees. The level of detail in the analysis conducted for this study does not allow for accurate estimation of the 90 percent level of assurance for overtopping. Therefore, the 50 percent level of assurance will be applied for this feasibility study and the 90 percent level of assurance will be checked during the project engineering and design (PED) phase.

2.10 LEVEE ANALYSIS

2.10.1 General Process

Waves and water levels from the ERDC-CHL storm surge and local wave modeling results were combined with the existing data gathered and analyzed to determine required levee heights based on EC 1110-2-6067 and CFR 2000 Title 44, and additional ERs, EMs, and ECs referenced below. The processes used for determining the required levee crest elevations were:

1. In areas subject to waves, levee crest elevation was calculated based on the maximum of:

    a. Freeboard associated with average overtopping rate plus the average 1 % ACE SWL
    b. 90 percent assurance level for the 1 % ACE SWL
    c. Freeboard associated with average overtopping rate plus FEMA required freeboard plus average 1 % ACE SWL (For coastal levees subject to overtopping, minimum required freeboard is 1 foot.)

2. In areas not subject to waves, levee crest elevation was calculated based on the maximum of:

    a. 90 percent Assurance Level for the 1 % ACE SWL
    b. FEMA required freeboard plus average 1 % ACE SWL (For levees subject to SWL overtopping only, required freeboard is 3 feet.)

After the required levee height was determined as described above, RSLC was added to determine the final recommended height. Recommendation is to add 50 years of projected RSLC at the low rate to earthen levees and the intermediate rate to floodwalls. RSLC is added linearly in this analysis. As discussed in Section 2.13, this is recommended because it is much easier to adapt the earthen crest elevation than the floodwall elevation. Recommended levee crest elevations based on other RSLC rates were also calculated to support planning scenario analysis for climate change.

2.10.2 Recommended Elevations for Brazoria Region (Freeport)

Based on the analyses, crest elevations are recommended. Tables 2-5 and 2-6 summarize recommended elevations for earthen levees subject to waves and water-level overtopping only, respectively.

Table 2-5: Recommended Earthen Levee Elevations Subject to Waves for Freeport as a Function of RSLC

Wave Output Station Name	Recommended Elevation in feet, NAVD without RSLC	Recommended Elevation in feet, NAVD with Low RSLC	Recommended Elevation in feet, NAVD with Intermediate RSLC	Recommended Elevation in feet, NAVD with High RSLC
DBC_1	16.6	17.3	17.9	19.6
DBC_2	16.2	16.9	17.5	19.2
DBC_3	15.9	16.6	17.2	19.0
DBC_3v2*	15.9	16.6	17.2	19.0
DBC_4	15.7	16.5	17.0	18.8
DBC_4v2*	15.7	16.5	17.0	18.8
DBC_5	15.4	16.1	16.7	18.5
DBC_5v2*	15.4	16.1	16.7	18.5
W_1	12.9	13.6	14.2	15.9
S_1	13.9	14.6	15.1	16.9
S_2	14.5	15.2	15.7	17.5
S_3	14.5	15.2	15.8	17.6
S_4	14.4	15.1	15.7	17.4
S_5	14.4	15.1	15.7	17.4
PF_3 - wave barrier	16.5	17.3	17.8	19.6
PF_3 - thumb	15.2	15.9	16.4	18.2
PF_4 - thumb*	15.6	16.3	16.8	18.6
PF_2	15.6	16.3	16.9	18.7
PF_1	19.1	19.8	20.3	22.1
US_2	15.4	16.1	16.6	18.4
US_2v2*	15.4	16.1	16.6	18.4
IH_1*	15.4	16.1	16.6	18.4
IH_1	15.4	16.1	16.6	18.4
N_1	17.9	18.7	19.2	21.0
N_2	18.4	19.1	19.7	21.5
N_3	21.3	22.0	22.5	24.3
N_4	20.1	20.8	21.4	23.2
N_6	18.7	19.4	20.0	21.7
N_5	19.3	20.0	20.6	22.4
N_7	19.5	20.2	20.7	22.5
LS_1	15.3	16.0	16.5	18.3
US_1	15.3	16.0	16.5	18.3

Table 2-6: Recommended Earthen Levee Elevations Subject to Water Level Overtopping Only (Oyster Creek Section of Freeport Levee) as a Function of RSLC

Wave Output Station Name	Recommended Elevation in feet, NAVD without RSLC	Recommended Elevation in feet, NAVD with Low RSLC	Recommended Elevation in feet, NAVD with Intermediate RSLC	Recommended Elevation in feet, NAVD with High RSLC
OC_1	17.2	17.2	17.2	17.2
OC_2	17.4	17.4	17.4	17.4
OC_3	17.6	17.6	17.6	17.6
OC_4	17.7	17.7	17.7	17.7
OC_5	17.7	17.7	17.7	17.7
OC_6	15.0	15.0	15.0	15.0
OC_7	12.3	12.3	12.3	12.3

2.10.3 Recommended Elevations for Sabine Region

Based on the analyses, crest elevations are recommended. Tables 2-7 and 2-8 summarize recommended elevations for earthen levees and floodwalls, respectively.

Table 2-7: Recommended Earthen Levee Elevations for Sabine as a Function of RSLC

Wave Output Station Name	Recommended Elevation in feet, NAVD without RSLC	Recommended Elevation in feet, NAVD with Low RSLC	Recommended Elevation in feet, NAVD with Intermediate RSLC	Recommended Elevation in feet, NAVD with High RSLC
GeoA	16.6	17.5	18.1	19.9
GeoB	9.5	10.4	11.0	12.8
GeoC	12.7	13.7	14.2	16.0
GeoD	11.2	12.1	12.7	14.5
GeoE	11.3	12.2	12.8	14.6
GeoF	11.7	12.6	13.2	14.9
GeoG	10.9	11.8	12.4	14.2
GeoH	13.5	14.4	15.0	16.7
GeoI	11.9	12.8	13.4	15.2
GeoJ	12.6	13.5	14.1	15.9
GeoK	14.2	15.1	15.7	17.4
GeoL	12.8	13.7	14.3	16.1
GeoM	13.2	14.1	14.7	16.4
GeoN	12.2	13.1	13.6	15.4
GeoO	10.7	11.6	12.2	13.9

Wave Output Station Name	Recommended Elevation in feet, NAVD without RSLC	Recommended Elevation in feet, NAVD with Low RSLC	Recommended Elevation in feet, NAVD with Intermediate RSLC	Recommended Elevation in feet, NAVD with High RSLC
GeoP	10.7	11.6	12.2	14.0
GeoQ	11.2	12.2	12.7	14.5
GeoR	11.3	12.2	12.8	14.5
GeoS	15.4	16.4	16.9	18.7
GeoT	15.6	16.5	17.0	18.8
GeoU	12.2	13.1	13.7	15.5
GeoV	16.2	17.2	17.7	19.5
GeoW	16.0	16.9	17.5	19.3
GeoX	12.8	13.8	14.3	16.1
GeoY	13.1	14.0	14.6	16.3
GeoZ	Floodwall Only			
GeoZ1	Floodwall Only			
GeoZ2	Floodwall Only			
GeoZ3	13.5	14.4	15.0	16.7
GeoZ4	12.3	13.2	13.7	15.5
GeoZ5	12.2	13.1	13.7	15.5
GeoZ6	12.0	12.9	13.5	15.2
GeoZ7	11.5	12.4	13.0	14.8
GeoZ8	11.0	11.9	12.5	14.3

Table 2-8: Recommended Floodwall Elevations for Sabine as a Function of RSLC

Wave Output Station Name	Recommended Elevation in feet, NAVD without RSLC	Recommended Elevation in feet, NAVD with Low RSLC	Recommended Elevation in feet, NAVD with Intermediate RSLC	Recommended Elevation in feet, NAVD with High RSLC
GeoA	16.4	17.3	17.9	19.6
GeoB	11.6	12.6	13.1	14.9
GeoC	13.5	14.4	15.0	16.8
GeoD	10.5	11.4	11.9	13.7

2.11 ADJACENT IMPACTS

This section discusses impacts to areas adjacent to the proposed new levees, including changes in surge water surface elevations, run-up, and waves. The ERDC surge model was run with the full "maximum" footprint for the Freeport, Port Arthur, Jefferson, and Orange levees being considered (see Figure 2-3). The model results showed induced impacts could reach levels of nearly 1 foot for the full configuration along the Neches River and the Orange County levee. The existing systems of Port Arthur and Freeport showed negligible impacts during a 100-year event.

In the area up the Neches and Sabine, some induced flooding was determined, particularly along the Neches River. The TSP removed a large part of the original levee system causing adjacent impacts due to lack of economic justification. This drastically reduced induced impacts along the Neches River from 1-foot levels to an insignificant impact on the order of 0.5 foot. Most areas impacted are vacant areas of grasslands with some trees and wetlands. It is recommended that some minor surge protection features be created to block induced damages in developed or sensitive areas. These would be small features with a height of approximately 6 inches. These could be small earthen berms, small I-walls, small rock walls, or other features. During design, the TSP configuration should be modeled in ADCIRC/STWAVE.

2.12 TSP/NED PLANS COMPARED TO RECOMMENDED FEMA CERTIFICATION AND USACE ACCREDITATION HEIGHTS

It is important to note that a planning study recommends the project that provides the most net excess benefits. That is what is being done at this level of the study to determine the limits of what items and percentages the Government would cost share. Reference the Main Report for cost share information. The NED plans use the average SWL and wave information without any application of statistics for the HH&C inputs (confidence levels, etc.). Additional consideration includes whether the system will meet FEMA CFR requirements and USACE Accreditation requirements.

Elevations required to meet FEMA CFR 65.10, as well as USACE Accreditation elevation requirements, are included in this Section. In many cases, the elevation recommended by the NED plan is in fact the highest recommended elevation. Many are also equal. For areas where the NED plan recommends a top levee elevation lower than FEMA or USACE required elevation for that location it is recommended that USACE move forward with the NED plan, and take the information the Non-Federal Sponsors to see if they desire or can pay the cost to add the extra elevation to the top of levee. This is a considerable benefit to them, and with the wall already close to the required height they would have to pay relatively little (example: many areas would just require a raise of a foot or two, which compared to a 12-foot pile-founded T-Wall is a rather small cost.

The level of detail in the analysis conducted for this study does not allow for accurate estimation of the 90 percent level of assurance for overtopping. The 50 percent level of assurance will be applied for this feasibility study as required by planning studies, and the 90 percent level of assurance will be checked during the Preliminary Engineering and Design (PED) phase. For additional information on the topics in this section, reference the DIFR-EIS and Geotechnical Engineering section of this appendix.

2.13 CLIMATE CHANGE

2.13.1 Relative Sea-Level Change

This document uses current USACE guidance to assess relative sea level change (RSLC). Current USACE guidance - ER 1100-2-8162, December 2013, and ETL 1100-2-1, June 2014, specifies the procedures for incorporating climate change and RSLC into planning studies and engineering design projects. Projects must consider alternatives that are formulated and evaluated for the entire range of possible future rates of RSLC for both existing and proposed projects. USACE guidance specifies evaluating alternatives using "low," "intermediate," and "high" rates of future sea level change.

- Low - Use the historic rate of local mean sea-level change as the "low" rate. The guidance further states that historic rates of sea level change are best determined by local tide records (preferably with at least a 40-year data record).
- Intermediate - Estimate the "intermediate" rate of local mean sea-level change using the modified NRC Curve I. It is corrected for the local rate of vertical land movement.
- High - Estimate the "high" rate of local mean sea-level change using the modified NRC Curve III. It is corrected for the local rate of vertical land movement.

USACE (ETL 1100-2-1, 2014) recommends an expansive approach to considering and incorporating RSLC into civil works projects. It is important to understand the difference between the period of analysis (POA) and planning horizon. Initially, USACE projects are justified over a period of analysis, typically 50 years. However, USACE projects can remain in service much longer than the POA. The climate for which the project was designed can change over the full lifetime of a project to the extent that stability, maintenance, and operations may be impacted, possibly with serious consequences, but also potentially with beneficial consequences. Given these factors, the project planning horizon (not to be confused with the economic period of analysis) should be 100 years, consistent with ER 1110-2-8159. Current guidance considers both short- and long-term planning horizons and helps to better quantify RSLC. RSLC must be included in plan formulation and the economic analysis, along with USACE expectations of climate change and RSLC, and their impacts. Some key expectations include:

- At minimum 20-, 50-, and 100-year planning horizons should be considered in the analysis.

- Reinforces the concept that a thorough physical understanding of the project area and purpose is required to effectively assess the projects sensitivity to RSLC.
- Sea level changes should be incorporated into models at the mean and extreme events.
- Identification of thresholds by the project delivery team and tipping points within the impacted project area will inform both the selection of anticipatory, adaptive, and reactive options selected and the decision/timing strategies.

2.13.2 Historical RSLC

Historical rates are taken from the Center for Operational Oceanographic Products and Services (CO-OPS) at NOAA, which has been measuring sea level for over 150 years. Changes in MSL have been computed using a minimum 30-year span of observations at each location. These measurements have been averaged by month to eliminate the effect of higher frequency phenomena such as storm surge, in order to compute an accurate linear sea-level trend.

The MSL trends presented are local relative trends as opposed to the global (eustatic) sea-level trend. Tide gauge measurements are made with respect to a local fixed reference level on land; therefore, if there is some long-term vertical land motion occurring at that location, the relative MSL trend measured there is a combination of the global sea-level rate and the local vertical land motion, also known as RSLC.

Historical rates of local RSLC can be obtained from local tide records. The tide gage nearest to the Sabine Lake system, with over 40 years of record, is at Sabine Pass (NOAA Gage 8770570). The NOAA MSL trend at this site (from 1958 to 2008) is equal to 5.66 mm/yr with a 95 percent confidence interval of +/- ± 0.98 mm/yr. Using the estimated historic eustatic rate equals that given for the modified NRC curves (1.70 mm/yr). This results in an observed subsidence rate of 3.96 mm/yr (5.66 mm/yr - 1.70 mm/yr). A vicinity map for NOAA Gage 8770570 is shown in Figure 2-17 on the next page.

The tide gage nearest to the Galveston coast region, with over 40 years of record, is at the Galveston Pleasure Pier, which is on the Gulf of Mexico coast side of Galveston Island (NOAA Gage 8771510). The NOAA MSL trend at this site (from 1957 to 2008) is equal to 6.84 mm/yr with a 95 percent confidence interval of ± 0.74 mm/yr. If the estimated historic eustatic rate equals that given for the modified NRC curves, the observed subsidence rate would be 5.14 mm/yr (6.84 mm/yr - 1.70 mm/yr). A vicinity map for NOAA Gage 871510 is shown in Figure 2-18.

Figure 2-17: NOAA Gage 8770570 Vicinity Map

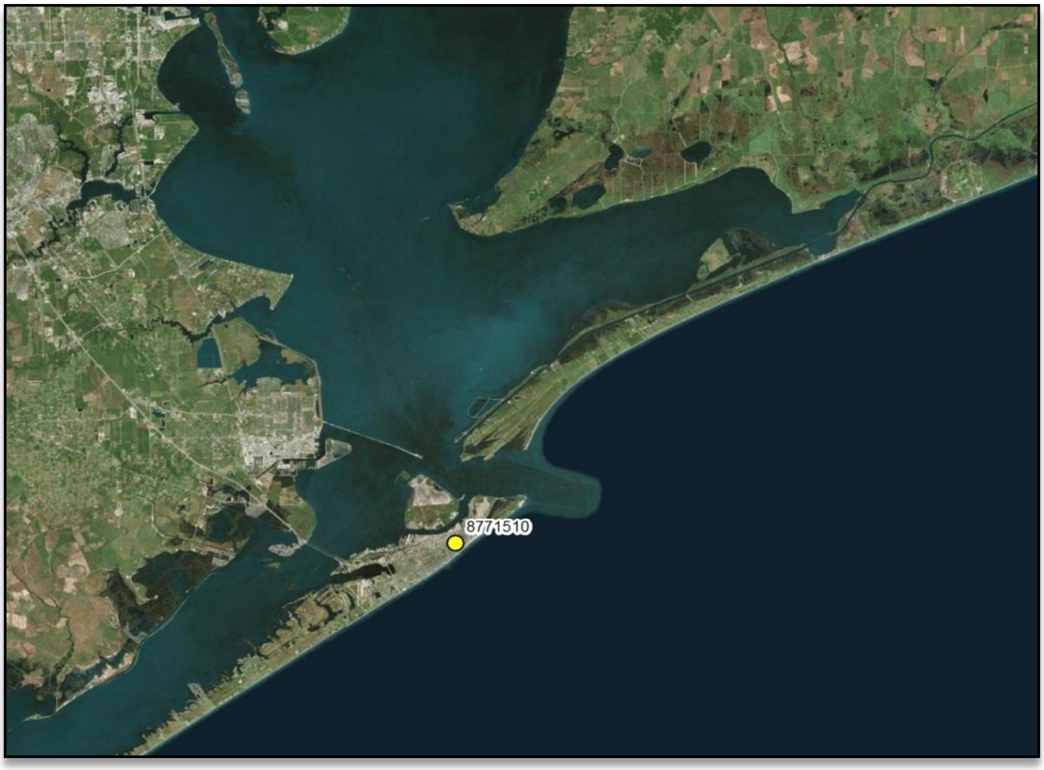

Figure 2-18: NOAA Gage 877150 Vicinity Map

The tide gage nearest to the Brazos River system, with over 40 years of record, is located at Freeport, TX Island (NOAA Gage 8772440). The NOAA MSL trend at this site (from 1954 to 2006) is equal to 4.35 mm/yr with a 95 percent confidence interval of ± 1.12 mm/yr. If the estimated historic eustatic rate equals that given for the modified NRC curves, the observed subsidence rate would be 2.65 mm/yr (4.35 mm/yr - 1.70 mm/yr). A vicinity map for NOAA gage 8772440 is shown in Figure 2-19.

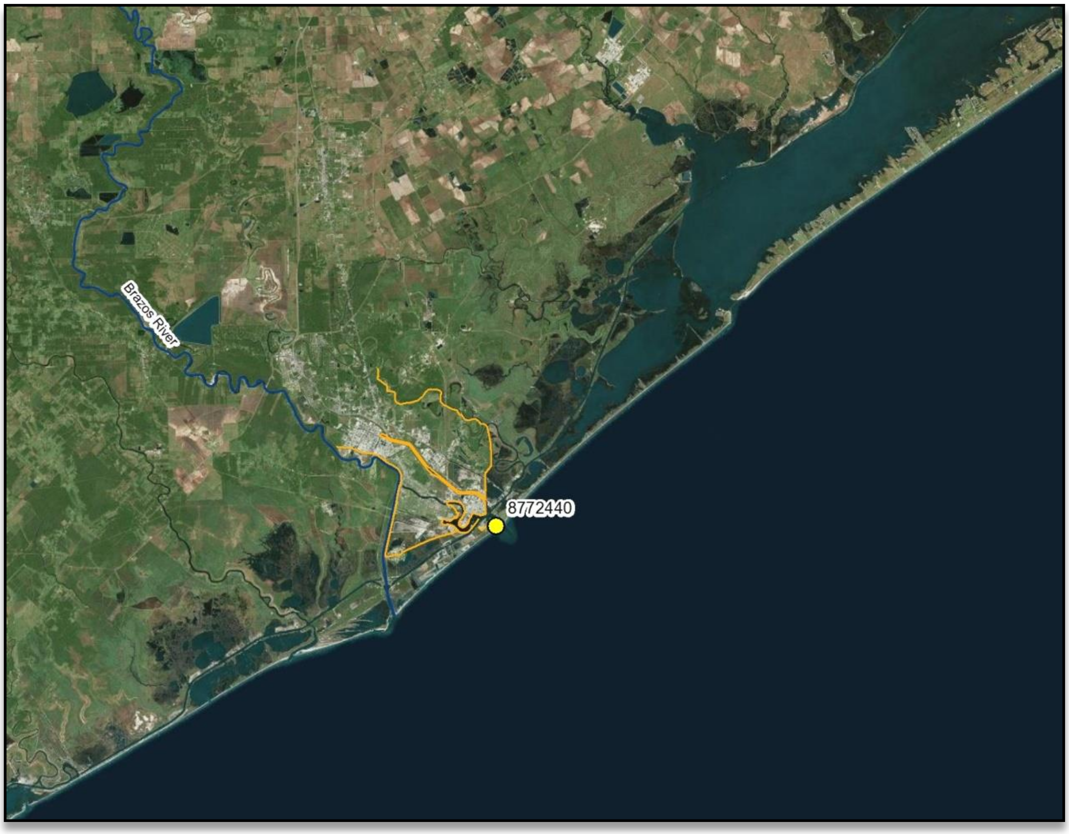

Figure 2-19: NOAA Gage 8772440 Vicinity Map

2.13.3 Predicted Future Rates of RSLC for 20-Year Period of Analysis

The computed future rates of RSLC in this section give the predicted change between the years 2030 and 2050 for the Sabine Lake, Galveston Bay, and Brazos River systems. RSLC values for this 20-year period are summarized in Table 2-9 and plotted for each of the systems in Figures 2-20, 2-21, and 2-22.

Table 2-9: Estimated RSLC over the First 20 Years of the Project Life (2030 - 2050)

Tide Gage	Measured Relative SLR Rate (NOAA)	Low (ft)	Intermediate (ft)	High (ft)
Sabine Pass, TX	5.66 mm/yr	0.37	0.54	1.08
Galveston Pleasure Pier, TX	6.84 mm/yr	0.45	0.62	1.16
Freeport, TX	4.35 mm/yr	0.29	0.46	1.00

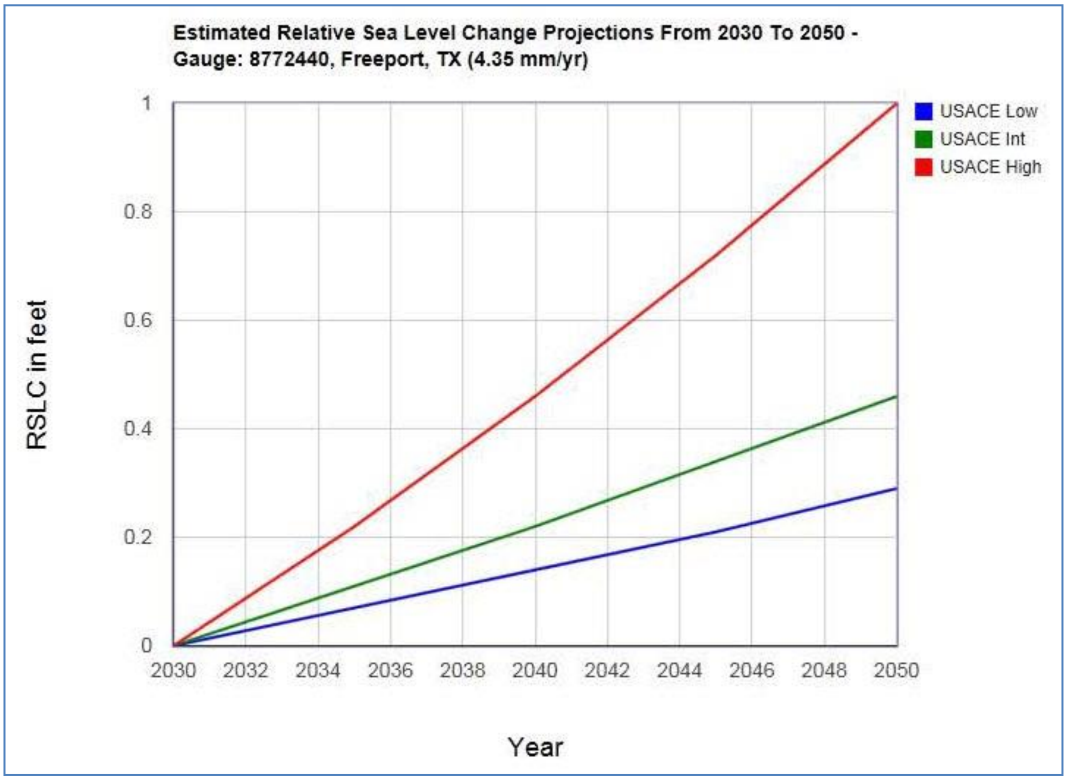

Figure 2-20: RSLC at Sabine, Texas over 20-Year Period of Analysis

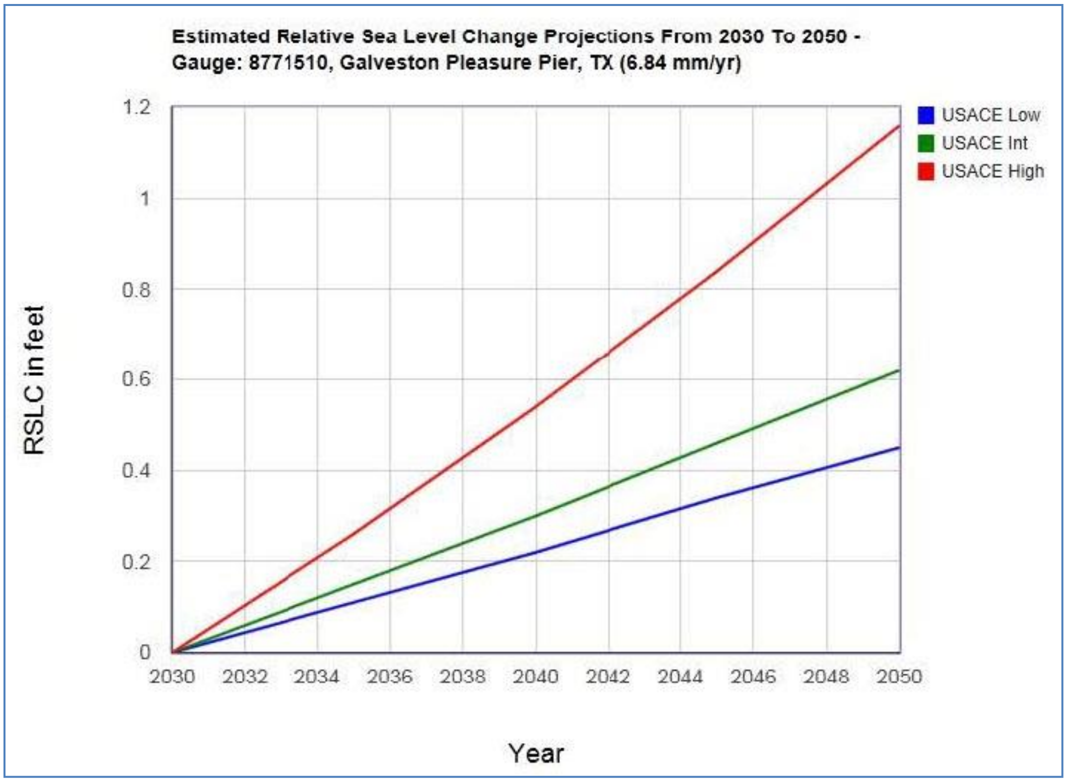

Figure 2-21: RSLC at Galveston, Texas over 20-Year Period of Analysis

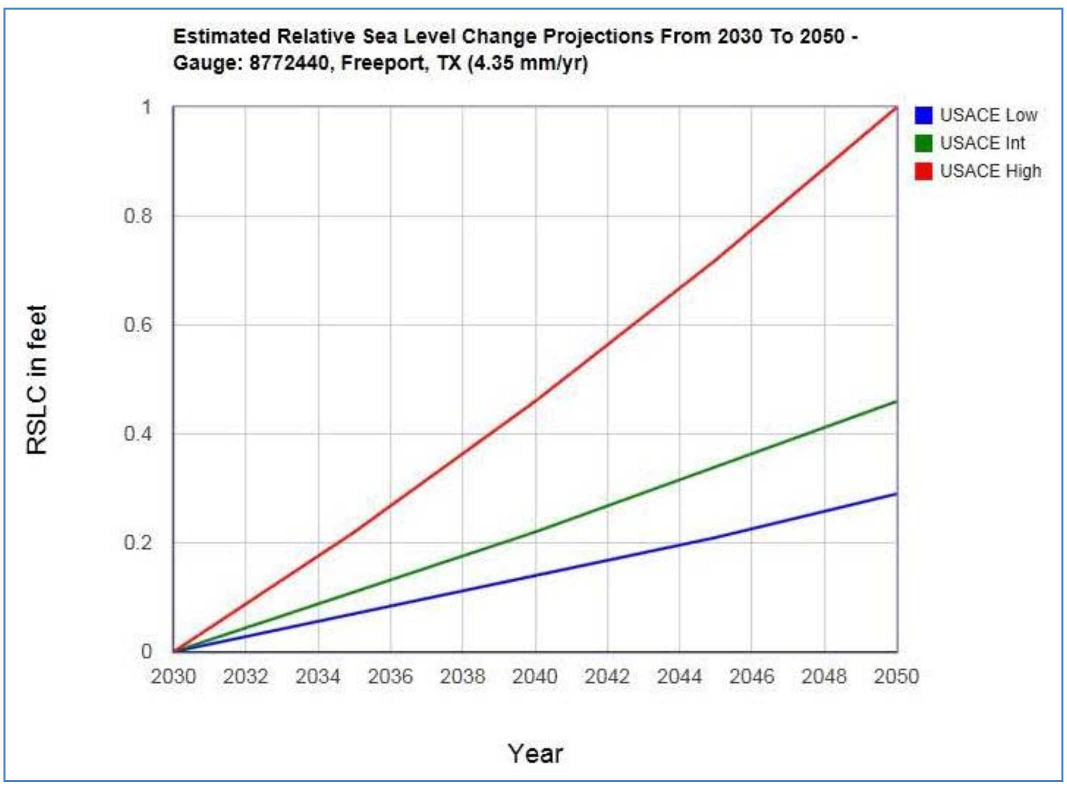

Figure 2-22: RSLC at Freeport, Texas over 20-Year Period of Analysis

2.13.4 Predicted Future Rates of RSLC for 50-Year Period of Analysis

The computed future rates of RSLC given here assume a 50-year period of analysis, and give the predicted change between the years 2030 and 2080 for the Sabine Lake, Galveston Bay, and Brazos River systems. Relative sea level change values for the 50-year period of analysis are summarized in the table below.

Table 2-10: Estimated RSLC over the First 50 Years of the Project Life (2030 - 2080)

Tide Gage	Measured Relative SLR Rate (NOAA)	Low (ft)	Intermediate (ft)	High (ft)
Sabine Pass, TX	5.66 mm/yr	0.93	1.49	3.26
Galveston Pleasure Pier, TX	6.84 mm/yr	1.12	1.68	3.46
Freeport, TX	4.35 mm/yr	0.72	1.27	3.05

Figure 2-23 shows in graphical form the computed sea level change for the Sabine Lake system based on the latest guidance for the "low," "intermediate," and "high" rates of change.

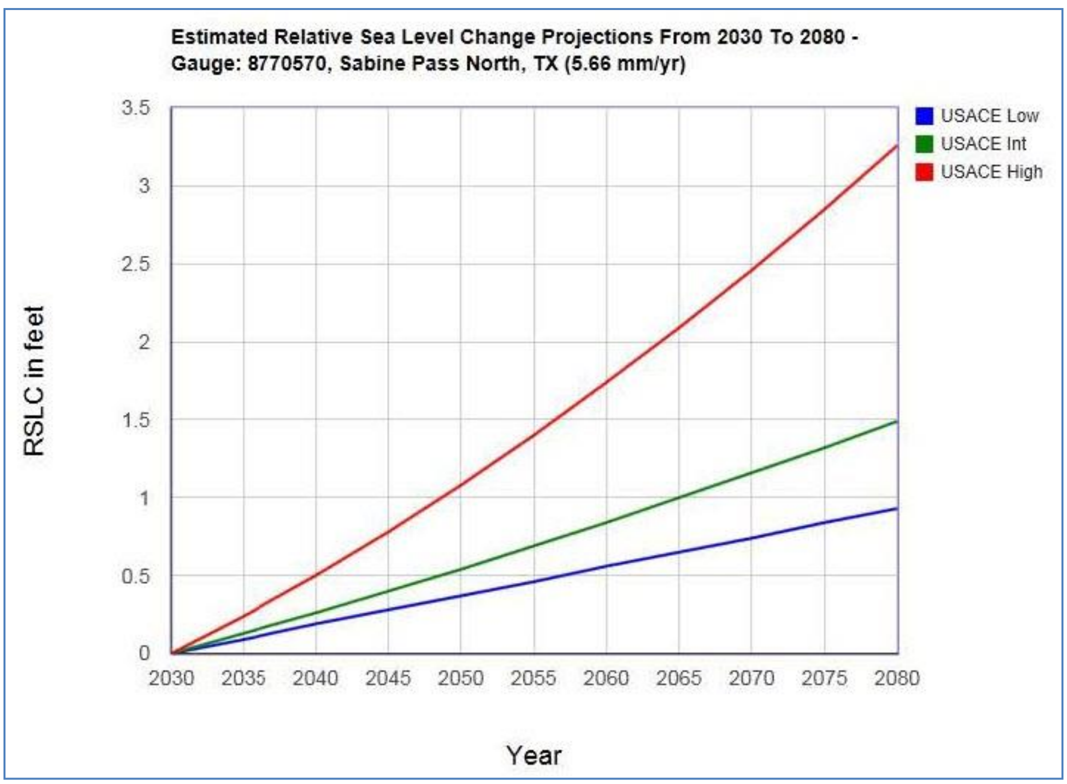

Figure 2-23: RSLC at Sabine Pass, Texas over 50-Year Period of Analysis

Using NOAA's Sea Level Rise and Coastal Flooding Impacts Viewer, it is apparent that much of the land around Sabine Pass in Jefferson County is low-lying. With just 1 foot of sea level rise, much of the coastline from Sabine Pass west to the Chambers County line is inundated during high tide. Areas as far as 10 miles inland experience increased inundation under this sea level rise scenario. There is substantial risk for lowland flooding without consideration of storm surge and waves. Adding surge and wave attack from a large hurricane to even the lowest SLR estimate yields substantial additional flooding. At 2 feet of sea level rise, the coastline west of Sabine Pass in Jefferson County becomes inundated without consideration of storm surge and waves, as does much of Port Arthur. Beaumont experiences more widespread flooding as well (Figure 2-24).

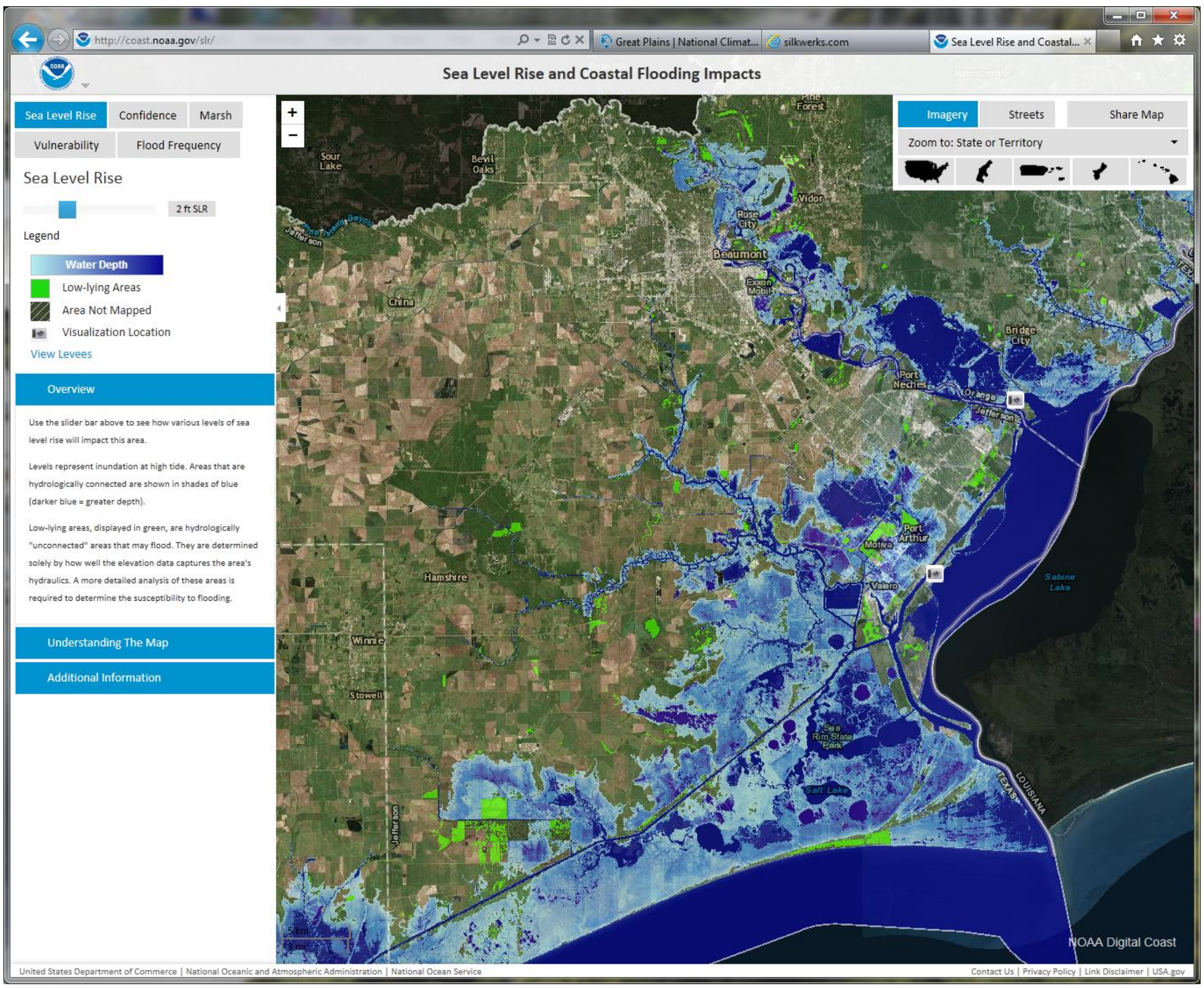

Figure 2-24: Extent of inundation at Sabine Pass, Texas with 2-foot sea level rise

Figure 2-25 shows the computed sea level change for the Galveston Bay system based on the current USACE guidance for “low,” “intermediate,” and “high” rates of change.

The Galveston County coastline is elevated higher than Jefferson County’s coastline. At a 2-foot rise in sea level, the centerline of Bolivar Peninsula west of Rollover Bay remains above water. Chambers County, on the other hand, experiences significant flooding around Galveston Bay. However, the majority of Galveston County is mostly unaffected (Figure 2-26).

Figure 2-27 shows the computed sea level change for the Brazos River system based on the current USACE guidance for “low,” “intermediate,” and “high” rates of change.

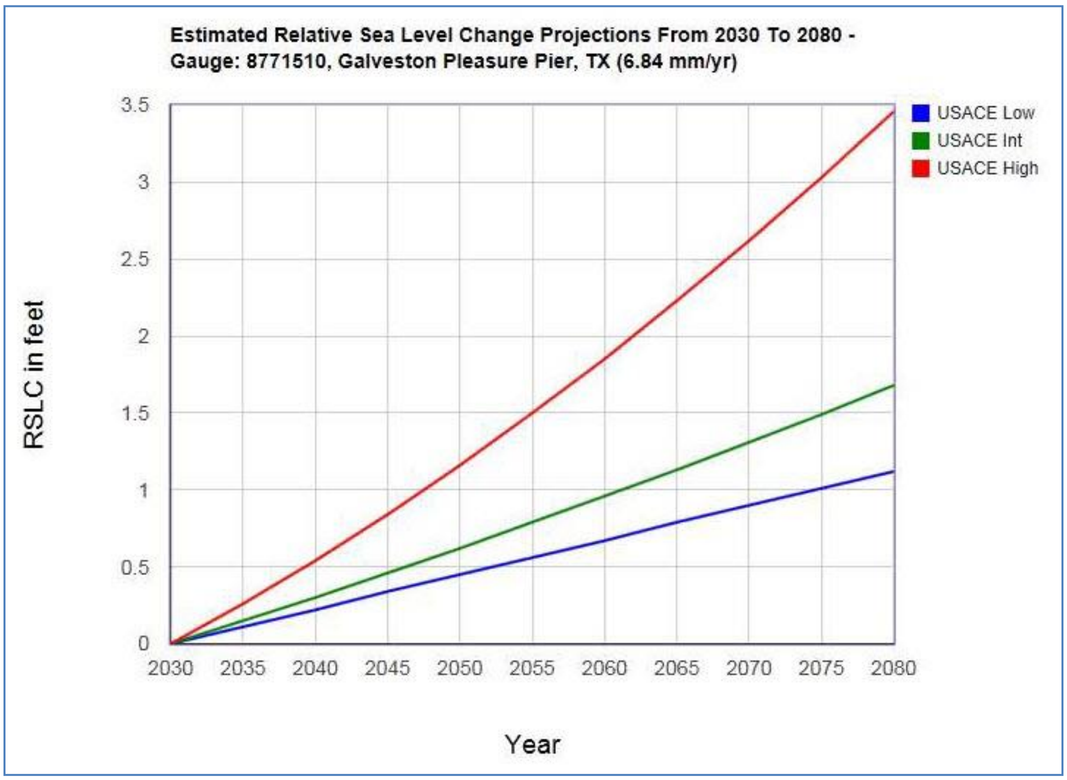

Figure 2-25: RSLC at Galveston Bay, Texas over 50-Year Period of Analysis

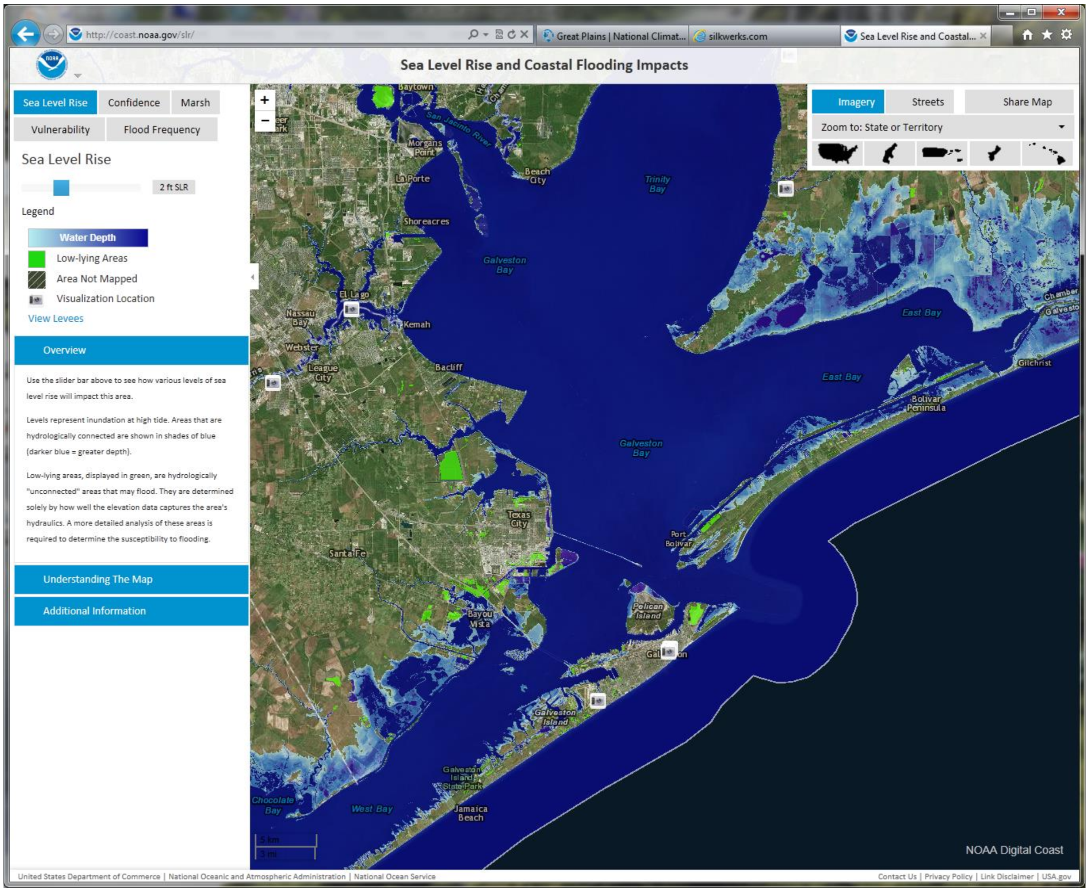

Figure 2-26: Extent of inundation at Galveston, Texas with 2-foot sea level rise

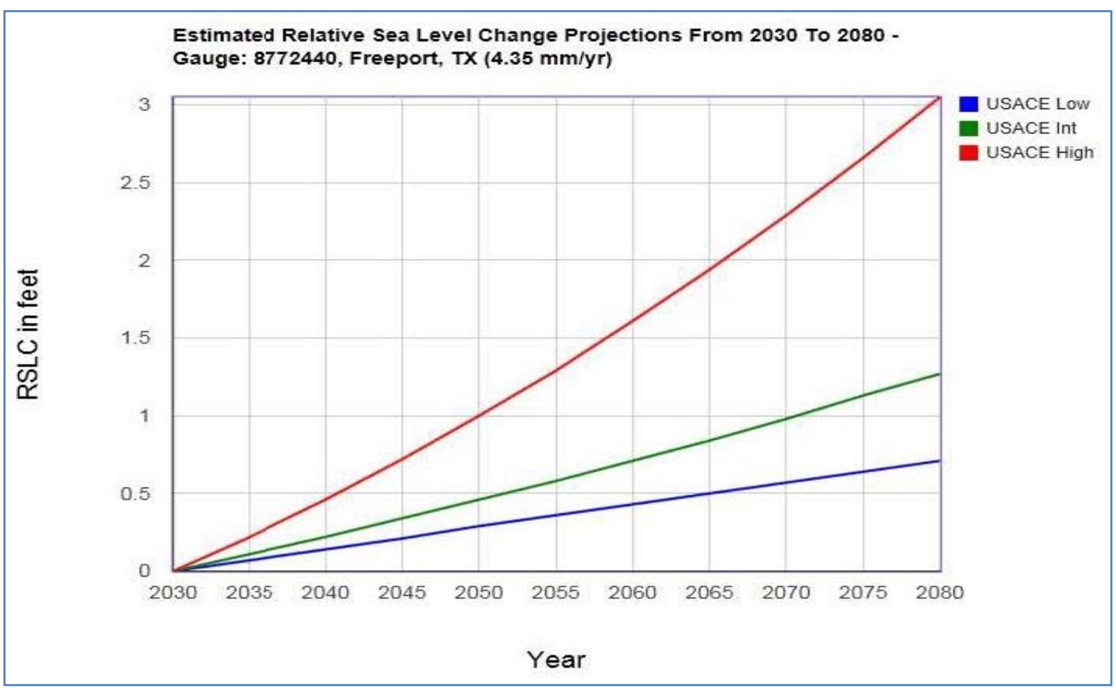

Figure 2-27: RSLC at Freeport, Texas over 50-Year Period of Analysis

Much of the area in the Freeport vicinity is low-lying. The majority of these low-lying areas are undeveloped, consisting of empty plots of land, some including marshes and wetlands. At 1 foot of sea level rise, several of these plots are inundated. It is important to note that water has begun to impact the Surfside Beach community just east of the Freeport Entrance Channel under this sea level rise scenario. At 2 feet, water begins to flood some central parts of Surfside Beach, inundating dozens of homes. For all considered sea level rise scenarios, safety from storm surge and wave attack for low-lying areas consistently decreases (Figure 2-28).

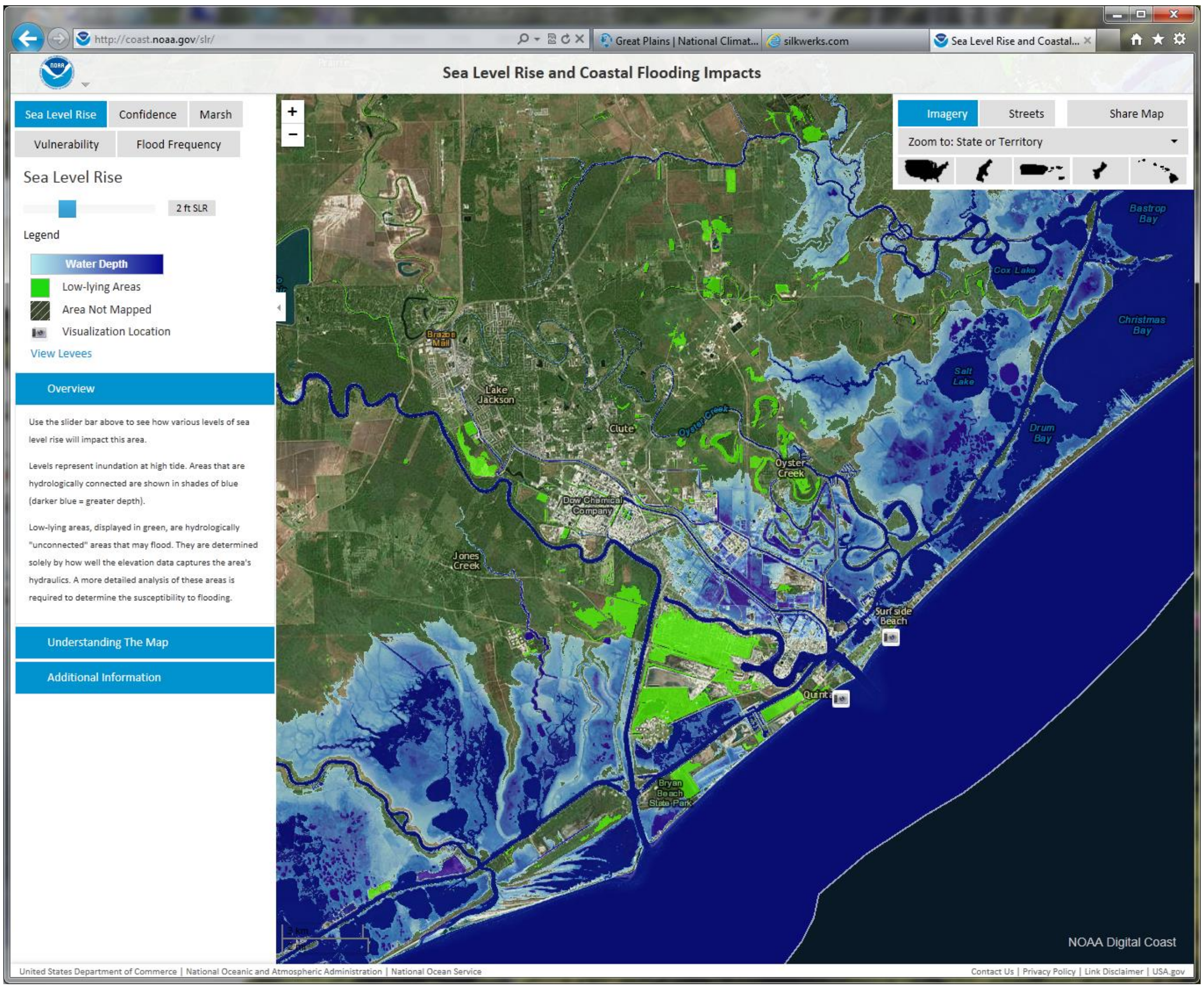

Figure 2-28: Extent of inundation at Freeport, Texas with 2-foot sea level rise

2.13.5 Predicted Future Rates of RSLC – 100-year Sea-Level Change

The planning, design, and construction of a large water resources infrastructure project can take decades. Though initially justified over a 50-year economic period of analysis, USACE projects often remain in service much longer. The climate for which the project was designed can change over the full lifetime of the project to the extent that stability, maintenance, and operations may be affected. These changes can cause detrimental or beneficial consequences. Given these factors, the project planning horizon (not to be confused with the economic period of analysis) should be 100 years, consistent with ETL-1110-2-1.

The period of economic analysis for USACE projects has generally been limited to 50 years because economic forecasts beyond that time frame were not considered reliable. However, the potential impacts of SLC over a 100-year period can be used in the formulation of alternatives and for robustness and resiliency comparisons. ETL 1100-2-1 recommends that predictions of how the project or system might perform, as well as its ability to adapt beyond the typical 50-year economic analysis period, be considered in the decision-making process.

The initial assessment that evaluates the exposure and vulnerability of the project area over the 100-year planning horizon was used in assisting planners and engineers in determining the long-term approach that best balances risks for the project. The three (3) general approaches are anticipatory, adaptive, and reactive strategies. These strategies can be combined or they can change over the life cycle of the project. Key factors in determining the approach include consequences, the cost, and risk. This consideration is of particular importance under a climate change condition where loading and response mechanisms are likely to transition over the life of the project.

Using the high SLC curve elevation at 100 years, the potential future affected area has been approximately defined. This includes both the vertical and the horizontal extents of potential SLC impacts. Since this is an initial screening level, detailed modeling has not occurred yet. This basic approach will provide a first-level assessment of how the project and project area might be impacted, and follows the guidance in ETL-2-1. More detailed engineering analyses will be conducted during PED.

The future affected areas, as defined by the 100-year high rate of RSLC, can impact resources, including economics. These resources can be identified and quantified, such as critical infrastructure (schools, roads, water supply, community buildings, etc.), impacted property, life-safety concerns, and environment and ecosystems. The consideration of the potentially larger area of impact facilitates discussion of what actions may need to be considered at certain trigger points. Community, as well as other stakeholder expectations will be better defined. Evaluation of coastal storm-damage risk reduction in the context of RSLC may also involve societal thresholds. Potential system and cumulative effects should be explored qualitatively when formulating plans.

An essential element of developing a good understanding of the project area's exposure and vulnerability is assessing how quickly the individual scenarios might necessitate an action due to thresholds and tipping points. It is important to identify key milestones in the project timeline when impacts are expected. This involves inputs from all members of the PDT as the threshold or tipping point could be a vast variety of different items or combinations of items.

Response strategies for the project planning horizon range from a conservative anticipatory approach, which constructs a resilient project at the beginning to last the entire life cycle (and possibly beyond), to a reactive approach, which would simply be to do nothing until impacts are experienced. Between these extremes is an adaptive management strategy, which incorporates new assessments and actions throughout the project life based on timeframes, thresholds and triggers. A plan may include multiple measures adaptable over a range of SLC conditions and over the entire timeline, with different measures being executed as necessitated.

For a feasibility-level design, it is important to identify potential cost-risk items and adaptation costs to the stakeholders and decision makers. Further detailed design and analysis may be undertaken during the pre-construction engineering and design phase to optimize project features sensitive to relative sea level change. In this phase, the question of further adaptability beyond the 50-year economic analysis period may be addressed as part of the design optimization. The economic and cost formulation for the project should account for uncertainty in critical design items.

Utilizing the online sea-level calculator referenced in ER 1100-2-8162, estimates of 100-year RSLC, assuming a starting date of 2030 and ending date of 2130, were determined for this project. Table 2-11 lists the predicted low, medium, and high RSLC values for 2130. Figures 2-29, 2-30, and 2-31 graphically show the RSLC information generated by the calculator.

Table 2-11: Predicted RSLC over the First 100 Years of the Project Life (2030 - 2130)

Tide Gage	Measured Relative SLR Rate (NOAA)	Low (ft)	Intermediate (ft)	High (ft)
Sabine Pass, TX	5.66 mm/yr	1.86	3.42	8.38
Galveston Pleasure Pier, TX	6.84 mm/yr	2.24	3.81	8.77
Freeport, TX	4.35 mm/yr	1.43	2.99	7.95

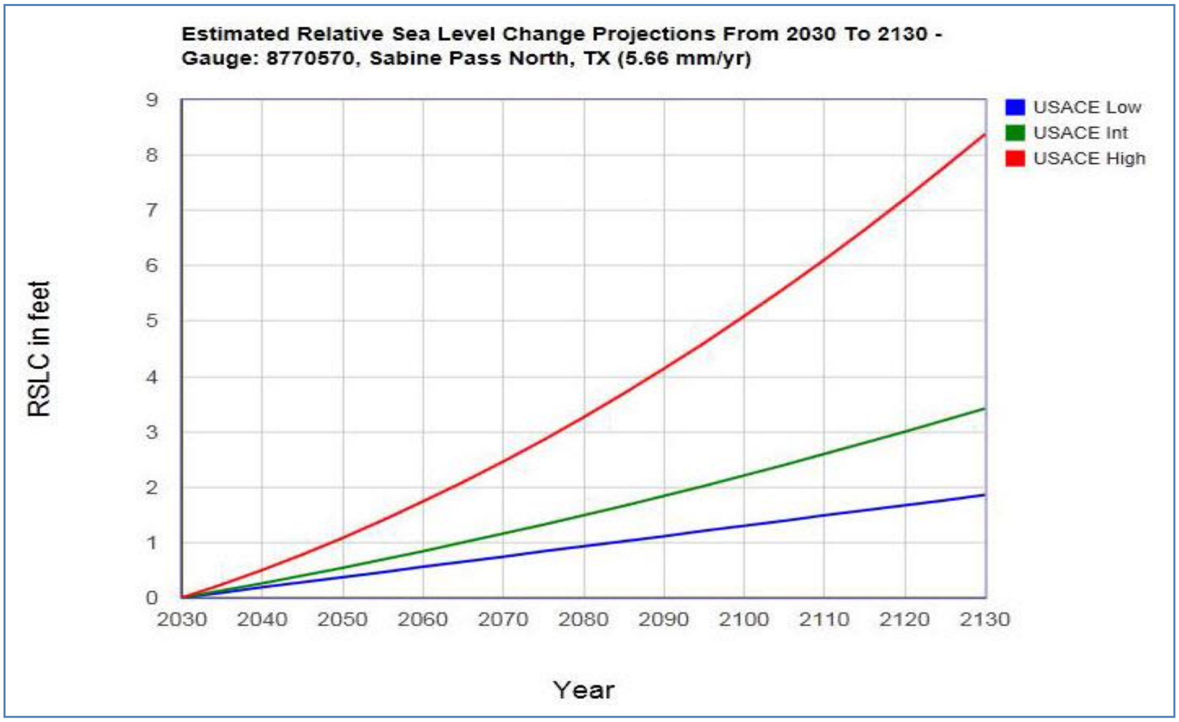

Figure 2-29: Predicted RSLC at Sabine, Texas over 50-Year Period of Analysis

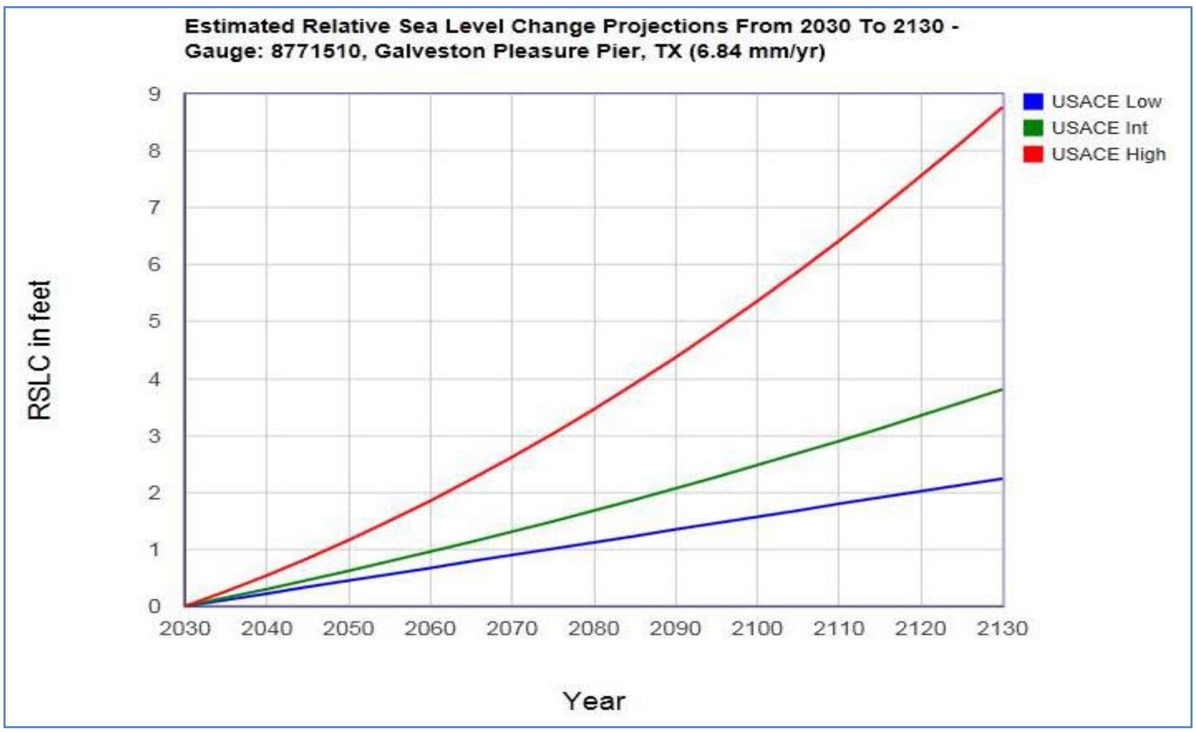

Figure 2-30: Predicted RSLC at Galveston, Texas over 100-Year Period of Analysis

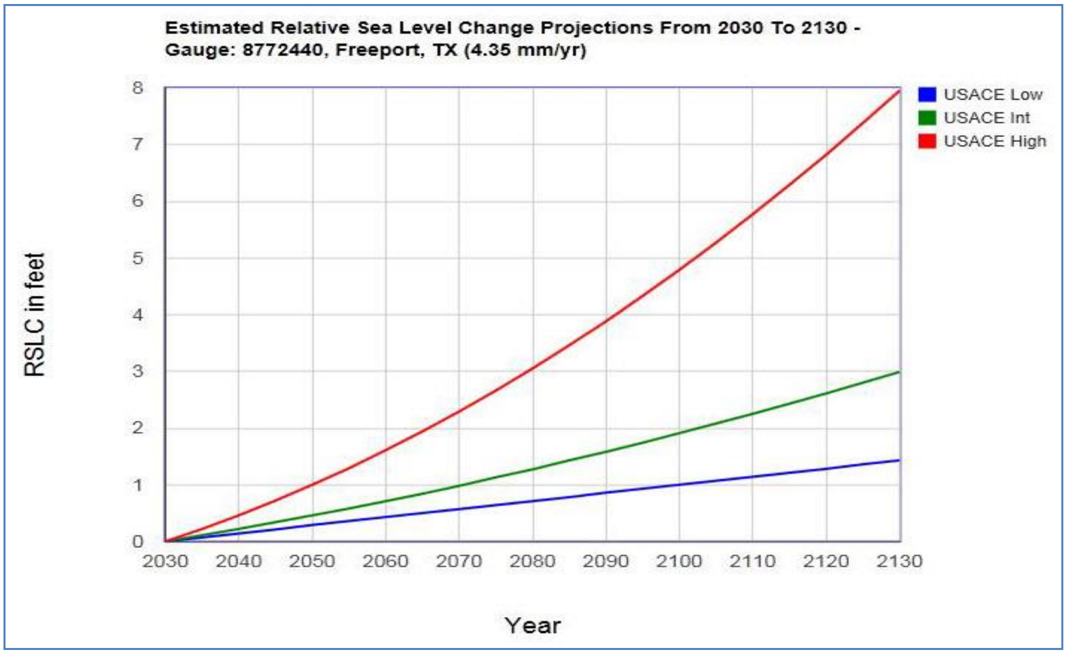

Figure 2-31: Predicted RSLC at Freeport, Texas over 100-Year Period of Analysis

2.13.6 Total Water Level Approach

The Total Water Level Approach framework is designed to emphasize several key principles described in USACE policy and guidance. Procedures to Evaluate the Magnitudes and Effects of Total Water Levels at USACE Projects include:

- Linking Tidal and Geodetic Datums
- Identifying Coastal Design Performance Factors
- Applying Coastal Forcing/Total Water Level Approach
- Applying RSLC and future scenarios
- Scaled analysis and decision making

For the TSP an initial Total Water Level approach was performed to:

- Identify Coastal Design Performance Factors
- Apply Coastal Forcing/Total Water Level Approach
- Observe RSLC and impacts on future scenarios
- Scaled analysis and decision making, such as when a threshold elevation may be reached or levee height may need modification.

Figures 2-32 and 2-33 show the low, intermediate, and high RSLC curves, and the storm surge. The plot shows the impact that RSLC will likely have on surge levels; by 2080 the surge levels are estimated to be 4 feet higher than currently due to RSLC. Figure 2-34 shows the different components that together make up the total water forcing. This information will be used to determine when modifications will need to be made, when threshold elevations are at risk of being reached, and other design and O&M purposes.

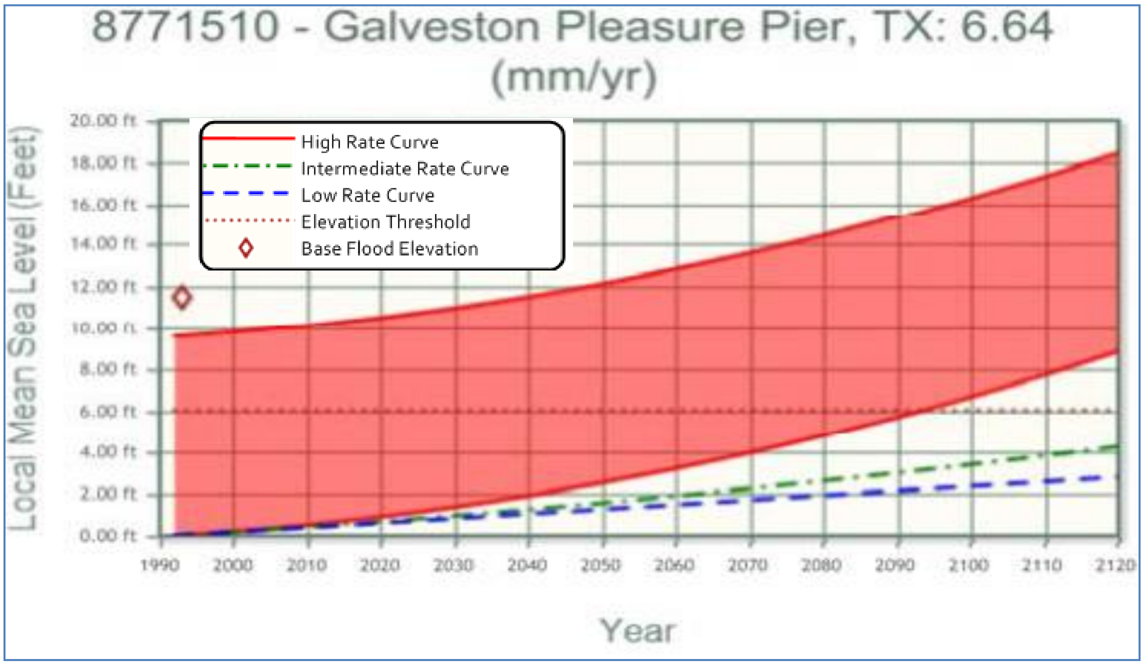

Figure 2-32: RSLC and Total Water Level Plot for Brazoria Region

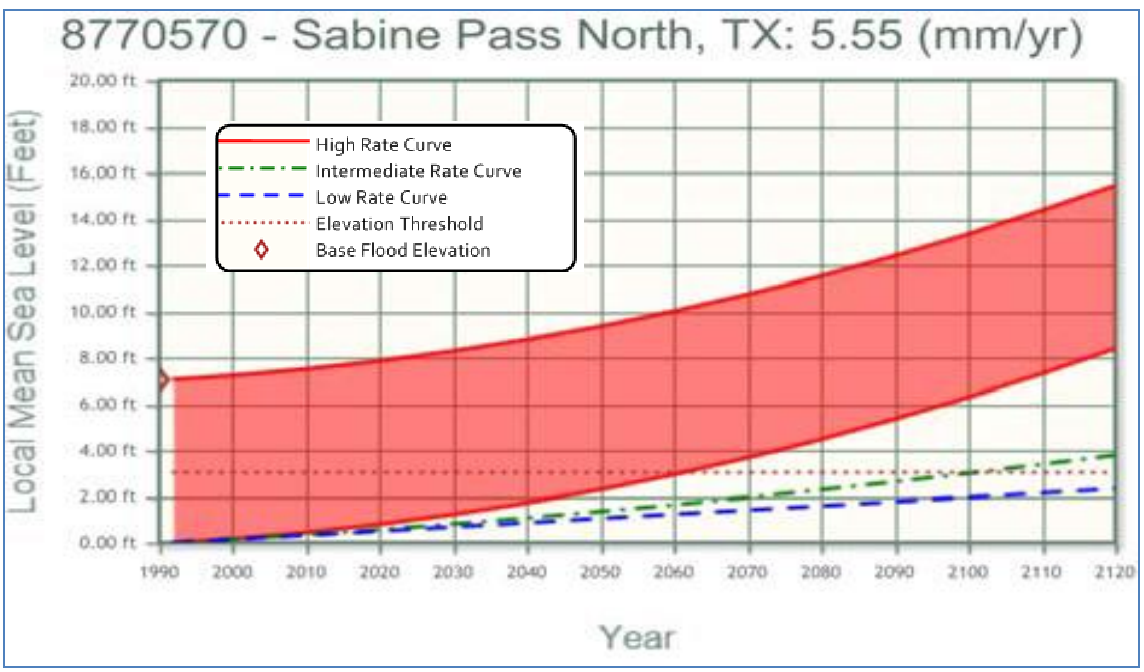

Figure 2-33: RSLC and Total Water Level Plot for Sabine Region

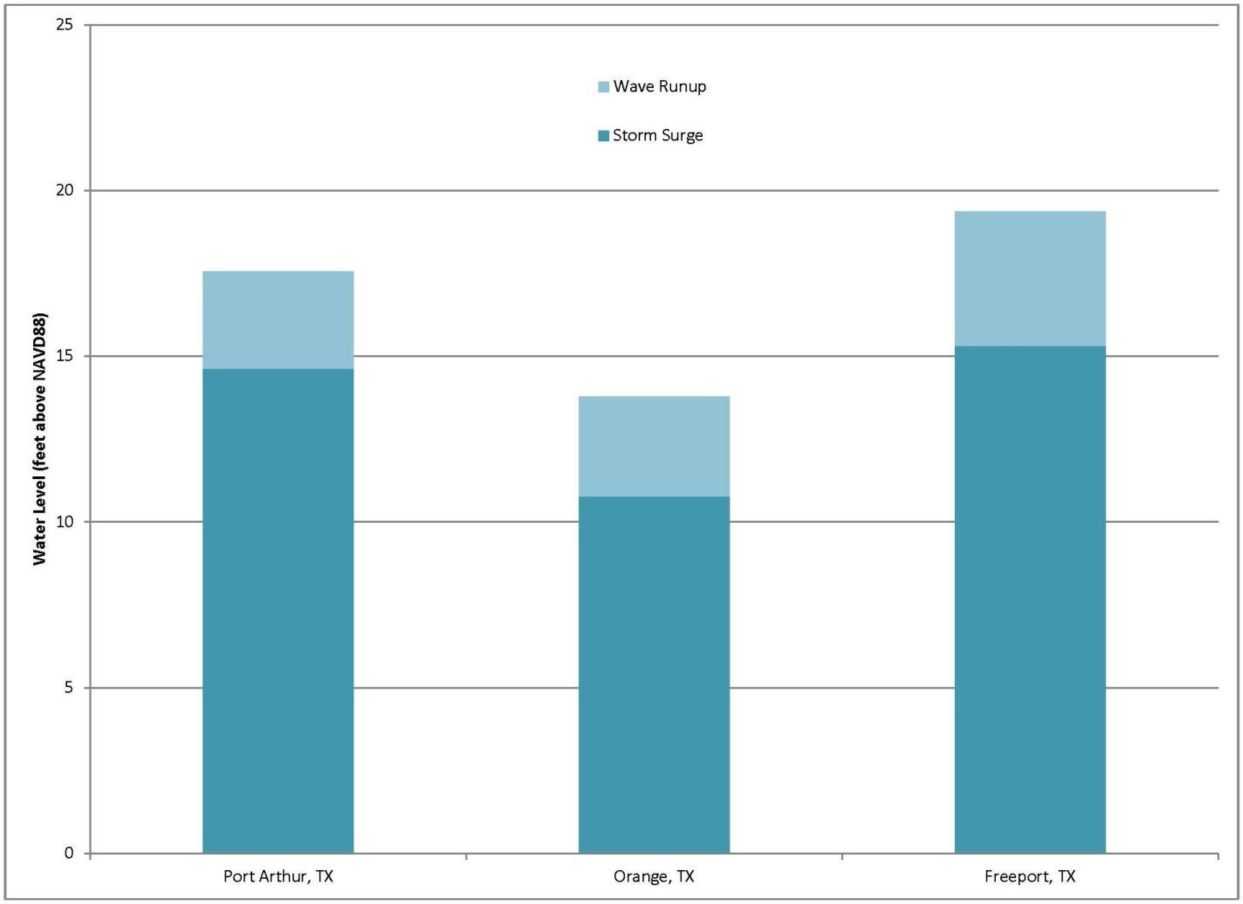

Figure 2-34: Bar Charts to Determine Total Water Level Forcing

Tides are not shown in Figure 2-34 as they are included in the surge model elevations.

2.13.7 Additional Climate Change Considerations

This section discusses future climate change expected based on current scientific evidence and studies. Climate change is expected to pose several challenges along the Texas coast. It is expected to vary greatly along the extensive Texas coast from the Mexican border to the Louisiana border. USACE will be pressed to 1) find ways to resolve increasing competition for land, water, and energy resources, 2) conserve ecological systems, and 3) enhance the resilience of people to the impacts of climate extremes (USACE 2014). These challenges will unfold against a backdrop that includes a growing urban population, incentives for energy production, and advances in technology.

For the current study area, the primary climatic forces with potential to affect the project are changes in temperature, sea and water levels, precipitation, storminess, ocean acidity, and ocean circulation. Air temperatures in the Houston-Galveston mean statistical area, on average, increased about 1 degree over the past 20 years, a pattern that is expected to continue. Sea surface temperatures have risen and are expected to rise at a faster rate over the next few decades. Global average sea level is rising and has been doing so for more than 100 years, and greater rates of sea-level rise are expected in the future (Parris 2012). Higher sea levels cause more coastal erosion, changes in sediment transport and tidal flows, more frequent flooding from higher storm surges, and saltwater intrusion into aquifers and estuaries.

Patterns of precipitation change are affecting coastal areas in complex ways. The Texas coast saw a 10 to 15 percent increase in annual precipitation between 1991 and 2012 compared to the 1901-1960 average (Figure 2-35). Texas coastal areas are predicted to experience heavier runoff from inland areas, with the already observed trend toward more intense rainfall events continuing to increase the risk of extreme runoff, flooding, and possibly creating life safety issues.

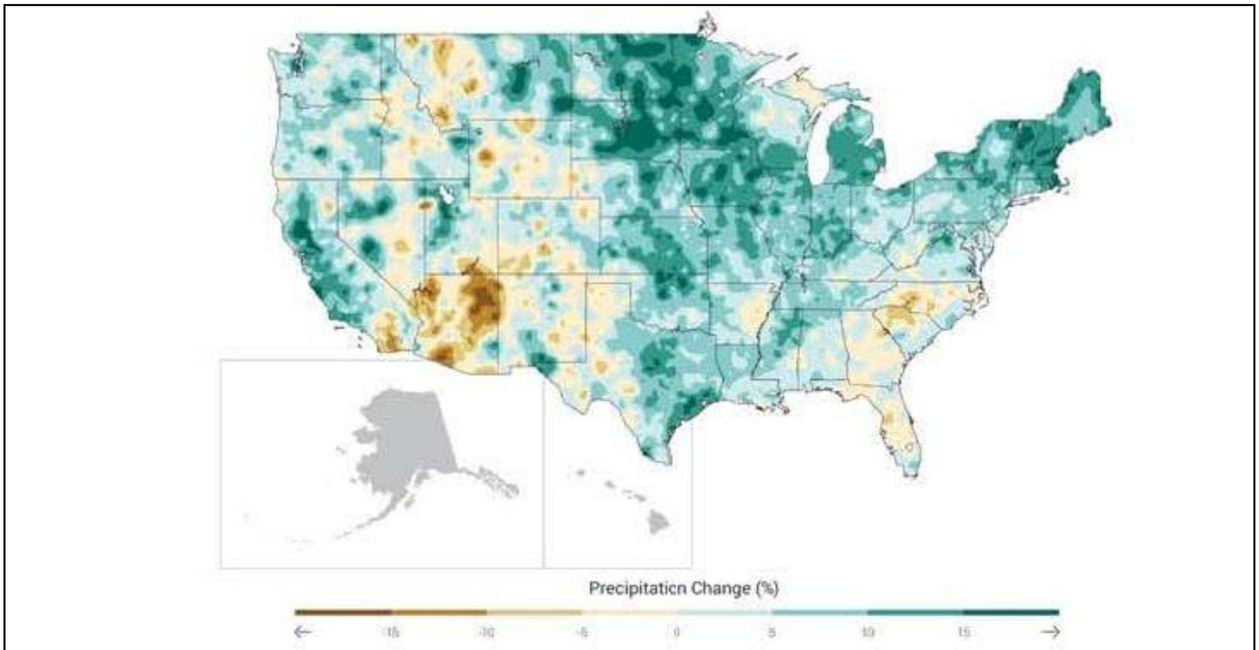

Figure 2-35: Percent Change in Annual Precipitation for 1991-2012 Compared to 1901-1960

Texas' Gulf Coast historically averages three (3) tropical storms or hurricanes every four (4) years, generating coastal storm surges and sometimes bringing heavy rainfall and damaging winds hundreds of miles inland. The estimated change in sea level will result in the potential for greater damage from storm surge along the Texas Gulf coast and miles inland. Tropical storms have increased in intensity in the last few decades. Future projections suggest increases in hurricane rainfall and intensity (with a greater number of the strongest - Category 4 and 5 - hurricanes) (Melillo 2014).

As the concentration of carbon dioxide in the atmosphere increases, the oceans will continue to absorb CO_2, resulting in increased ocean acidification. This threatens coral reefs and shellfish (Hoegh-Guldberg 2007). Coastal fisheries are also affected by rising water temperatures and climate-related changes in oceanic circulation. Wetlands and other coastal habitats are threatened by sea-level change, especially in areas of limited sediment supply or where barriers prevent onshore migration. The combined effects of saltwater intrusion, reduced precipitation, and increased evapotranspiration will elevate soil salinities and lead to an increase in salt-tolerant vegetation (Craft 2009). For additional information, reference the Environmental section of the DIFR-EIS.None of these changes operate in isolation. The combined effects of climate changes with other human-induced stresses make predicting the effects of climate change on coastal systems challenging. However, it is certain that these factors will create increasing hazards to the Texas coast. Heavily industrialized cities and ports containing critical infrastructure along the Texas coast, including Freeport, Port Arthur, Galveston, Corpus Christi, Matagorda, Brazos Island Harbor, Houston, Port Orange, and additional areas will be adversely affected by climate change.

The projected change in sea level will result in the potential for greater damage from storm surge along the Texas coast. About a third of the GDP for the state of Texas is generated in coastal counties. Coastal areas in Alabama, Mississippi, Louisiana, and Texas already face losses that annually average $14 billion from hurricane winds, land subsidence, and sea-level change. According to a recent study, projected sea-level change increases average annual losses from hurricanes and other coastal storms (Building 2010).

Diminishing water supplies and rapid population growth are critical issues in Texas. Along the coast, climate change-related saltwater intrusion into aquifers and estuaries poses a serious risk to local populations. In 2011, many locations in Texas experienced more than 100 days over 100°F, as the state set high temperature records. Rates of water loss were double the long-term average, depleting water resources. This contributed to more than $10 billion in direct losses to agriculture alone (Melillo 2014). Typically, many of the water shortages occur in the drier east parts of Texas.

The agricultural economy along the Texas coast, including livestock, rice, cotton, and citrus cultivation, is threatened by the combination of salt or brackish water from sea-level change and reduced freshwater levels from changes in temperature and precipitation. Coastal ecosystems are particularly vulnerable to climate change because many have already been dramatically altered by human interventions creating additional stresses. Climate change will result in further reduction or loss of functions these ecosystems provide.

Successful adaptation of human and natural systems to climate change will require commitment to addressing these challenges. Regional-scale planning and local-to-regional implementation will prove beneficial. Finding a way to mainstream climate planning into existing processes will save time and money. It is important that information be continually shared among decision-makers to facilitate the alignment of goals.

2.13.8 Conclusions and Recommendations regarding RSLC

The RLSC values in Tables 2-12, 2-13, and 2-14 should be included in all aspects of the feasibility designs and analyses for all three levee systems and study area. This includes modifications of existing levees and design of proposed levees and gate structures. Additionally, these values should be used when evaluating drainage features and for the environmental assessment of the project. RSLC estimates for 20-year, 50-year, and 100-year planning horizons, following guidance in ER 1100-2-8162, are summarized in the following tables.

Table 2-12: Estimated RSLC over the First 20 Years of the Project Life (2030 - 2050)

Tide Gage	Low (ft)	Intermediate (ft)	High (ft)
Sabine Pass, TX	0.37	0.54	1.08
Galveston Pleasure Pier, TX	0.45	0.62	1.16
Freeport, TX	0.29	0.46	1.00

Table 2-13: Estimated RSLC over the First 50 Years of the Project Life (2030 - 2050)

Tide Gage	Low (ft)	Intermediate (ft)	High (ft)
Sabine Pass, TX	0.93	1.49	3.26
Galveston Pleasure Pier, TX	1.12	1.68	3.46
Freeport, TX	0.71	1.27	3.05

Table 2-14: Estimated RSLC over the First 100 Years of the Project Life (2030 - 2050)

Tide Gage	Low (ft)	Intermediate (ft)	High (ft)
Sabine Pass, TX	1.86	3.42	8.38
Galveston Pleasure Pier, TX	2.24	3.81	8.77
Freeport, TX	1.43	2.99	7.95

Consideration should be given to raising levee crown heights to the optimum elevation based on the range of RSLC combined with CSRM and economic considerations. This includes assessing the benefits and costs associated with the different scenarios to select the TSP.

Design of the various components studied in the project may require alteration due to RSLC. There are several possible ways to integrate RSLC and climate change into the design of the project features being studied. This section recommends some possible solutions to the PDT, while there are many more and other solutions that could be analyzed during PED.

- For the existing levee systems in Port Arthur and Freeport:

    - Threshold elevations should be determined by the PDT and used to assess what actions need to be taken due to RSLC alone. The threshold elevation is the elevation at which RSLC will impact the functionality of the existing projects. Examples of a threshold elevation could be the toe of a levee, components of gated structures, or the elevations and configuration of drainage structures.
    - When evaluating the existing levee systems, consideration should be given to raising levee crown heights for future coastal storms combined with sea level change over the projects life. An adaptive approach is recommended for earth embankment portions of the existing levees to raise the crest as needed at different times in the projects life. For the earthen levee sections, additional ROW could be purchased to allow for expansion of the levee footprint and raising the levee height. The side slopes of the existing earthen levee embankments should be evaluated for possible increases in steepness that would allow for raising the crest height within the current ROW. It should be taken into consideration that steeper levees are more easily overtopped by waves. Floodwall sections should be assessed for their vulnerability to the possible relative sea-level rise scenarios. Raising of floodwalls may be justified to protect against increased coastal storm damages due to higher surge heights when RSLC is considered. If floodwalls are to be replaced entirely for structural or other reasons, it is advised to set the top elevation of the new floodwalls to account for sea-level rise.

- For new levee systems in Orange and Jefferson Counties:

    - An adaptive approach or an anticipatory/conservative approach is recommended for the proposed new levee components where justified.

- An adaptive approach for earthen levees could involve acquiring additional ROW to utilize when raising levees, providing a larger footprint than would be needed. The crown width could be set greater than the width required. Current regulations require a 10-foot crown. A larger crown width, for example 15 feet, would provide the opportunity to raise the levee with a decrease in the crown width. Earthen embankments could be constructed with flatter side slopes than required. This would provide the opportunity to steepen the sides slope in the future when raising the levees, but may alter the levees performance during surge events. For earthen levees, if ROW is available and not expensive, an anticipatory approach should be considered.
- An anticipatory approach is recommended for sections of floodwall, which should be analyzed and designed to include RSLC.
- Gate designs for Adams and Cow Bayous should evaluate the impacts of the RSLC scenarios and be designed to accommodate possible RSLC scenarios. The large costs of these gates make this a critical design parameter for the project.
- A conservative/anticipatory design is recommended for floodwall sections. Adaptive designs are recommended for earth embankment sections. A reactive approach is not recommended for any component of the proposed system. Anticipatory approaches are the most costly initially, but may yield the highest overall benefit-to-cost ration and net excess benefits over the project design life.

➢ Drainage structures:

- It is recommended that the design of drainage structures incorporate the potential effects of RSLC and climate change.
- To account for expected increase in the precipitation intensity on the upper Texas coast, the size of the culverts should be increased by 5-10 percent. This is to allow the interior drainage to function as needed and allow increased discharges.
- To accommodate RSLC, culvert slopes will be conservative to account for RSLC impacts. The downstream flow-line for many culverts will be adjusted as needed to account for at least the low RSLC condition to prevent inundation behind the line of protection.
- Other alternatives for meeting the recommendation of incorporating RSLC and climate change include flap gates on culvert outfalls with positive closure that open to discharge interior runoff when the interior hydraulic head is sufficient to push water out. Culverts may be constructed with an adaptive flow-line breaching the line of protection. This option may be most useful in environmentally sensitive areas and tidally influenced areas.

- It is recommended that pumps be designed conservatively to provide adequate capacity for future coastal storm events that may bring more rainfall runoff than previously experienced due to climate change.

➢ Additional Considerations:

- The jetties for the Freeport Entrance Channel and Sabine Neches Waterway are critical for safe navigation into the project areas. If the sea-level change reaches an elevation that overtops the jetties, measures would need to be taken to modify the jetties and continue to provide safe navigation.
- The Freeport Entrance Channel jetties have a threshold elevation of 6.6 feet NAVD 88 (7.24 feet MLLW). This is the height of both jetties after repairs were made to return the storm-damaged jetties to their pre-Hurricane Ike elevations. Based on this data, it was determined that there will be no need for modification of the jetties over the 50-year period of analysis or for an extended period afterwards.
- The Sabine Neches Waterway jetties have a threshold elevation of 7.5 feet NAVD 88 (7.6 feet MLLW). This is the elevation of both jetties after repairs were made to return the storm-damaged jetties to their pre-Hurricane Ike condition. Based on this data, it was determined that there will be no need for modification of the jetties over the 50-year period of analysis or for an extended period afterwards.

This summary of RSLC is provided to the PDT to assist in making critical designs regarding the project feature designs and economic damages assessments. This includes economic analysis, plan formulation, and the incorporation of this information when determining the TSP.

2.14 INTERIOR DRAINAGE

The existing and proposed levee systems need to maintain drainage of rainfall-induced runoff of the levee interior area during normal rainfall events, and also during tropical events during which the system will be closed to reduce storm surge damages. The following describes the preliminary interior drainage analysis performed and drainage plan for the existing and proposed levee components.

For the proposed new levee systems, flows were calculated using rational and regression methods. Drainage structures were conservatively sized based on the 50-year and 100-year flow events. The calculated flows were increase by a magnitude of 10 percent to account for climate change and the anticipated increases in precipitation predicted. This is further discussed in Section 2.13 - Climate Change.

For the existing systems at Port Arthur and of Freeport, the existing drainage systems and possible modifications or improvements that could be made were evaluated. Known deficiencies in interior drainage structures (pipes, pumps, etc.) are not addressed in this evaluation of existing systems as they would be Operations, Maintenance, Repair, Replacement and Rehabilitation (OMRR&R) issues. Only improvements and modifications with economic benefits that establish a Federal interest were considered for the existing systems. Section 2.15 details the study efforts of interior drainage for both regions.

2.15 EXISTING LEVEE SYSTEMS INTERIOR DRAINAGE

An extensive data collection effort was performed for the Regions of study. Data collected included USACE Design Memorandums, Plans and Specifications, levee inspection report, Hydrologic and hydraulic models from counties, the state of Texas, Cities, Ports, and Levee Districts, among other sources including consultants were gathered and used when appropriate.

2.15.1 Brazoria Region Data (Freeport and Vicinity)

Levee construction plans, hydrologic and hydraulic studies, FEMA data, TXDOT data, county data, Periodic Inspection Reports, and FEMA data for FEMA studied bodies of water were used. Maps of the area and historical data regarding water and inundation paths were used to identify areas of concern. Additionally data from the on-going USACE Risk Management Center Freeport SQRA was obtained including surge, drainage, and other hydrologic and hydraulic models to use. Preliminary investigation determined that current interior drainage infrastructure is adequate and there are no known deficiencies. Some interior drainage components will need reconstruction or repair as work on the TSP plan dictates. One new drainage structure will be needed for the TSP. At the DOW barge canal the TSP calls for a sector gate structure, which will require and expanded pump location to drain water past the proposed gate structure. More detailed study of this structure is recommended during PED. During PED it is recommended that all major waterways, creeks, bayous, and other waterways within or adjacent to the levee system be surveyed and hydrologic/hydraulic models be update or recreated.

2.15.2 Sabine Region Data (Port Arthur and Vicinity)

Data from DMs, plans and specifications other studies for existing Port Arthur and the surrounding areas, Periodic Inspection Reports, the current Alligator Bayou Pump Station 408 study, and FEMA data for the area was used to investigate the interior drainage of Port Arthur. Preliminary hydrologic and hydraulic investigation determined that the current interior drainage infrastructure is adequate and there are no known deficiencies or need to change the interior drainage features besides some of the pump stations. Five pump stations that discharge rainfall runoff during surge events need reconstruction or repair as work on the TSP plan dictates. These pumps are shown below in Figure 2-36.

Figure 2-36: Port Arthur Pump Stations in Need of Rehabilitation

The pumps will need constructed with the capability to discharge 10 percent more cfs than the current pumps. More detailed study of this structure is recommended during PED. During PED it is recommended that all major waterways, creeks, bayous, and other waterways within or adjacent to the levee system be surveyed and hydrologic/hydraulic models be updated or created if they do not or no longer exist.

2.15.3 Preliminary Screening of Neches River Gate Alternative

An alignment for a closure gate on the mouth of the Neches River recommended by Orange County Study was selected for additional H&H study and analysis. The original Orange County Report alignment was refined to minimize environmental and navigation impacts, and identify a more specific location for a pumping station. This pumping station would need to have the ability to pump discharges from the inland Neches River into Sabine Lake.

Both alternatives were configured to provide the same level of protection (and benefits). USACE must recommend a plan that maximizes net excess benefits. With the benefits for each alternative providing the same benefits the alternatives could be compared based on cost data. A joint probability analysis of riverine discharge and storm surge on the Neches River was conducted to assist in determining the feasibility of an alternative that would place a closure gate on the Neches River. The analysis calculates the probability of discharge with respect to extreme water level. This analysis is appropriate for initial screening. This preliminary screening was performed to determine if the Neches River gate continued to remain a viable alternative or would be dropped from further consideration.

Figure 2-37 shows the location of gages that provided data used in this analysis. The nearest gage measuring river discharge was an USGS gage at the Neches River Saltwater Barrier. The USGS gage has a period of record (POR) from 04 June/2003 until present. The nearest gage measuring water level was a NOAA gage at Rainbow Bridge. The NOAA gage has a POR from 01 January 1993 to present.

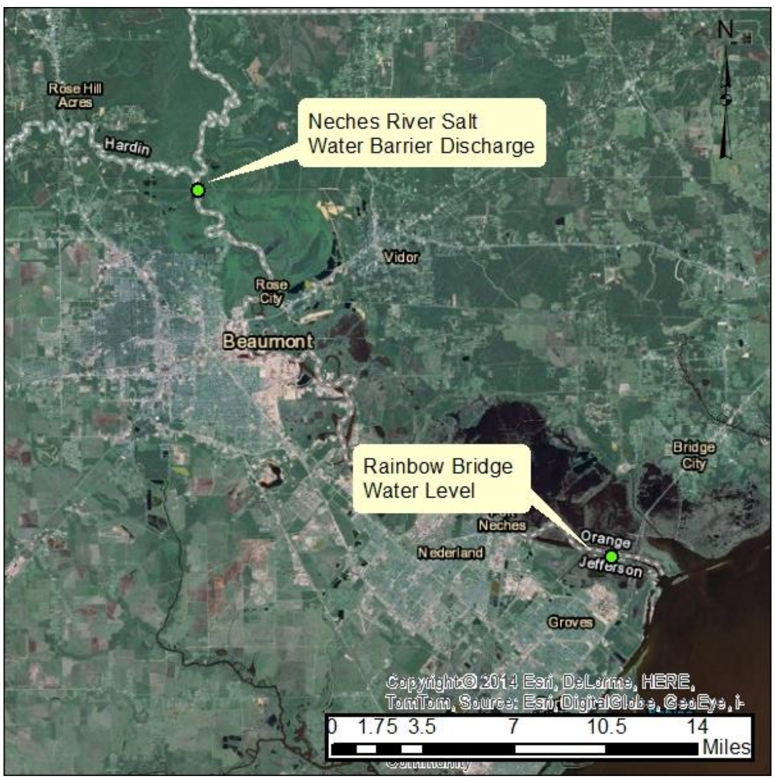

Figure 2-37: Location of Gages Used in this Analysis

Figure 2-38 plots all available overlapping water level and discharge data. The time period from 2003 to present was used for this analysis. Figure 2-39 plots discharges and water levels during Hurricane Rita (2005). Rita made landfall in the study area as a Category 3 Hurricane. Since water levels and discharges are both influenced directly by tropical storms, a joint probability analysis is required to determine the probability of discharge volumes during variable surge events that would require the gate to be closed.

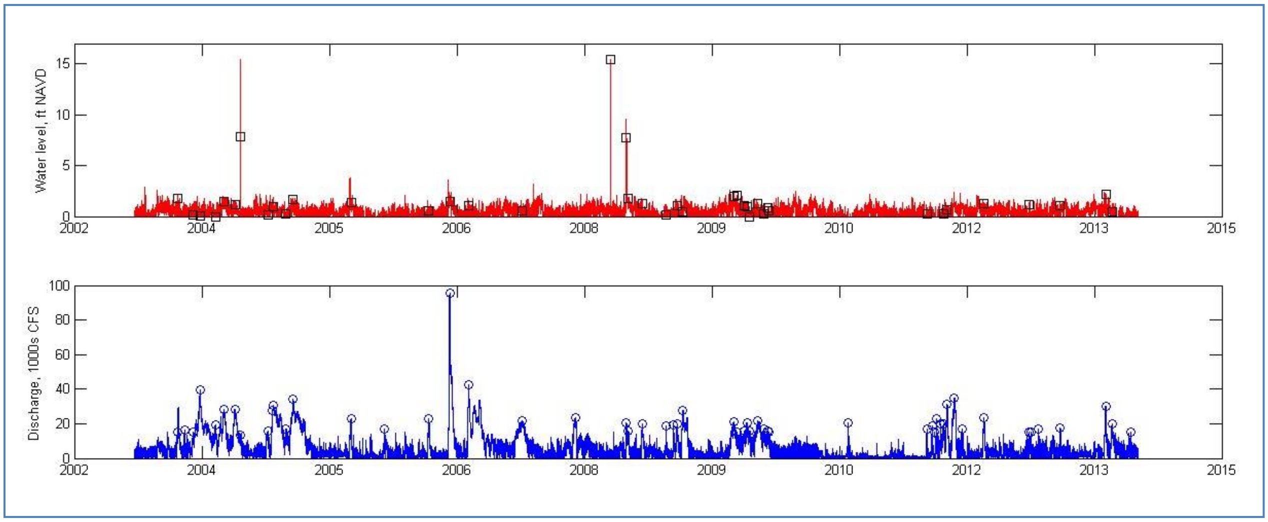

Figure 2-38: Water Level and Discharge Data from 2003 to Present

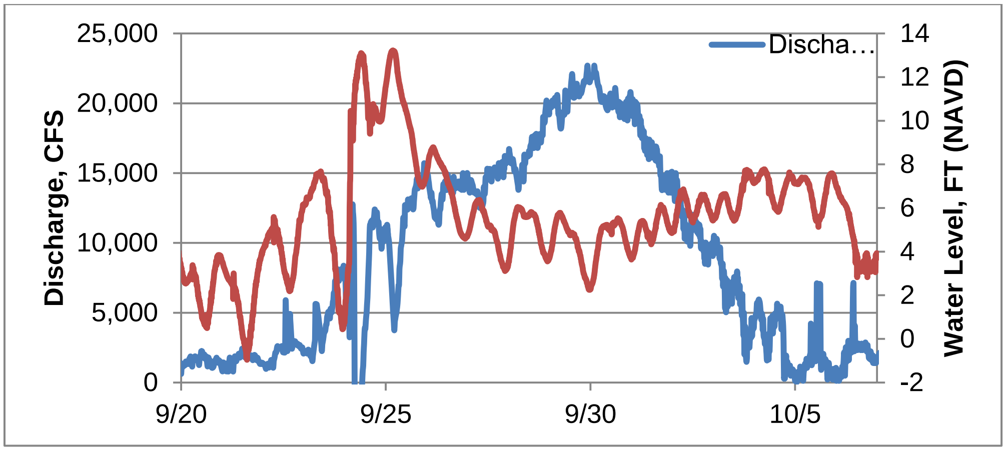

Figure 2-39: Discharge and Water Level during Hurricane Rita

The data gathered from NOAA and USGS gages were utilized in performing a joint probability analysis of discharge and water level. First, the peak over threshold approach was used to determine extreme water level and discharge events. Water levels over 3.3 feet and discharges over 15,000 CFS were specified as extreme and included for analysis. Frank and empirical Copula bivariate distribution functions were then fit to the data to calculate the distribution following the method applied in Michalsen (2014). Figure 2-41 plots the probability distribution calculated with the Frank (top) and empirical (bottom) Copula functions. The colors and contours show probability in return period. Figure 2-40 shows that the highest discharges tend to occur at lower water level. It is important to point out that this is a relatively short data record.

A longer data record would need to be synthesized to attempt to improve the accuracy of these results. It is also noted the Neches River downstream of the USGS gage would incorporate additional drainage area and runoff so it should be expected that flow near the proposed gate location would be higher.

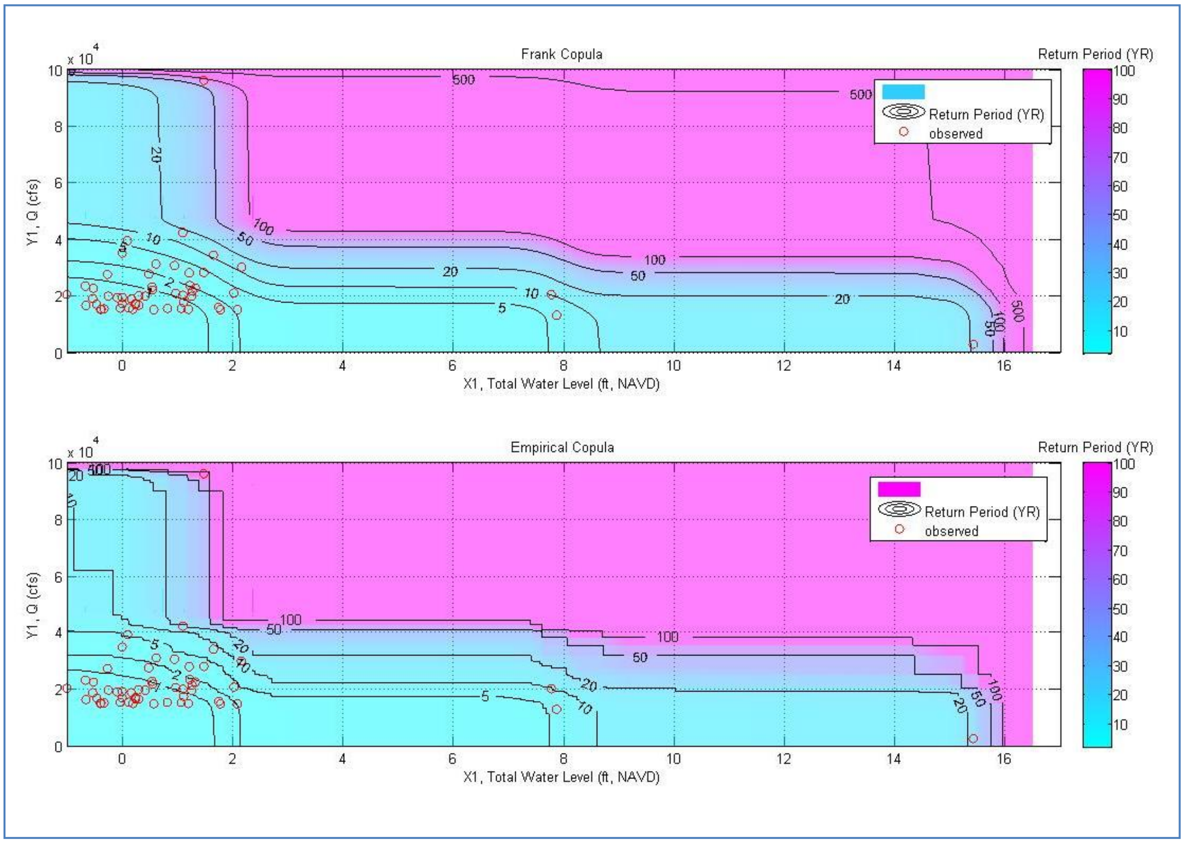

Figure 2-40: Joint Probability of Discharge (Q) and Water Level

In summary, the analysis leads to the following general conclusions:

- 100-year return period Neches River discharge could reach levels up to 40,000 cfs for coincident riverine/surge events that would occur during gate closure.
- If the gate remains the preferred alternative, more detailed analysis of joint probability of water level and discharge should be conducted to inform operations and design.

2.15.4 Consideration of Storm Surge Damages in Port/City of Beaumont

A preliminary investigation was conducted by USACE to evaluate the Beaumont areas vulnerability to storm surge, and if the alternatives being considered could provide beneficial protection in the area. For this investigation USACE collected data, studied inundation maps for a range of storm frequencies, and contacted local and national entities with knowledge of flooding issues in the area. During this investigation USACE contacted the Port of Beaumont, Jefferson County Drainage Districts 6 and 7, the FEMA local flood administrator, FEMA Regions VI and IV, the City of Beaumont Water Utilities Department, and the Sabine Neches Drainage District.

The Port of Beaumont stated that surge in the Port did occur during Hurricane Ike, but it caused only minor damage to docks and a few electrical systems. The Port of Beaumont was fully operable 2 to 3 days after Hurricane Ike passed. Hurricane Rita caused almost no surge at the Port. Damages from Rita in Beaumont were almost entirely due to wind.

The local floodplain administrator (Adena Ward) provided additional information on surge related damages in the Beaumont area. Ike flood damages were focused in two areas. The first area was along Pine Street in Beaumont, which experienced significant flooding. Most of the properties damaged at this location have since been bought out. The other location that was damaged by surge was the Exxon Mobil facility area located just south of the Port of Beaumont. FEMA Region VI provided damage claim amounts for the Ports of Orange and Beaumont during Hurricanes Ike and Rita. For Ike, 87 percent of damage claims were Category A (debris removal) with minimal damages to infrastructure or facilities. Damages claimed during Rita only amounted to $109,000.

Sabine Neches Navigation District (SNDD) had little info on Port of Beaumont. SNDD did relay that considerable damages had occurred along Taylors Bayou during Ike. They have since constructed a large diversion culvert to route water around the area and prevent flood water damages on Taylors Bayou during future rainfall events. The Water Utilities Department noted that the salt water barrier intake on the Neches River was damaged during Ike, but the cost of repair was minimal. Jefferson County Drainage District 6 provided data on high water marks during Ike that correlated well with the information other contacts had provided.

In summary, surge events have historically caused minimal damages to the Port and City of Beaumont. Drainage features have been constructed to alleviate flooding along Taylors Bayou, and buyouts have been performed in Beaumont to remove structures from flood-prone areas. The only area that has experienced damages and not been protected or removed from flood prone areas is the Exxon facility area (note that this area would be protected by the alternatives currently being considered in this USACE study). During surge events, water is almost entirely contained within the undeveloped Neches floodplain in the vicinity of Beaumont and causes minimal damages. The inundation in the vicinity of Beaumont from a 100-year frequency storm is displayed in Figure 2-41.

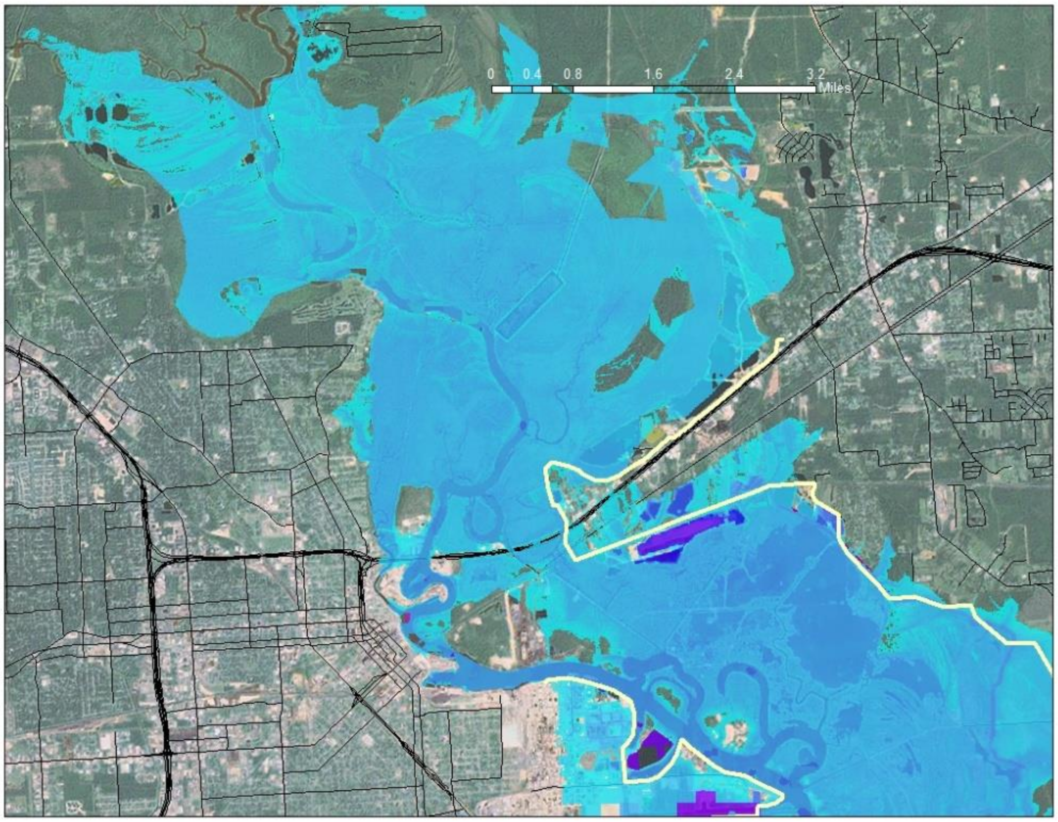

Figure 2-41: 100-year Frequency Inundation in Beaumont Vicinity

Orange County Levee/Gate Alternatives: Alignments and Preliminary Feasibility Approach

USACE used levee alignments established in the Orange County Flood Protection Planning Study (2012). The levee alignment was refined in some areas to maximize benefits, reduce cost, reduce environmental impacts, and protect critical infrastructure. Additionally, steps were taken to avoid major pipeline corridors/conflicts and identify potential properties for buyouts. For additional information and details on the alignment study, alignment selection, economics, cost, real estate, and other factors that led to the TSP alignment, please refer to those sections of this report.

2.15.5 Interior Drainage Analysis of Proposed New Levee Systems (TSP)

2.15.6 Drainage Area Delineation

Drainage areas for the interior of the levees were delineated using a combination of LiDAR, USGS maps, aerial maps, and documents gathered from the county and cities in the area. A closure gate on the Neches River was considered and eliminated due to cost. Drainage Area Maps for the entire proposed Orange, Beaumont, and Jefferson sections of the TSP are included in the H&H exhibits section of this H&H. These maps also contain preliminary drainage structure locations and sizes.

2.15.7 Drainage Area Discharge Estimates

Runoff was calculated using the Rational method for Areas under 200 acres, and using Regression methods for areas large than 200 acres. Details of the runoff calculations are summarized in the following sections. Measured gage data in the area is sparse, but what is available is being used. Hydrologic and Hydraulic Models of Cow and Adams Bayous were obtained from the Orange County Economic Development center were use.

2.15.8 Rational Method

For areas 200 acres or less, the Rational method was used to calculate runoff due to rainfall events. The Rational Equation is the simplest method to determine peak discharge for smaller drainage basin runoff. It is not as sophisticated as the SCS TR-55 method, but is the most common method used for sizing sewer systems and adequate for the purposes used in this study.

The Rational Equation is: $Q=CIA$

where:

Q = Peak discharge (cfs)

C = Rational method runoff coefficient

I= Rainfall intensity (inches/hour)

A = Drainage area (acres)

The Rational method runoff coefficient (C) is a function of the soil type and drainage basin slope. C is reflective of the type of land coverage. For example, C for pervious grassy/forest areas that allow infiltration and slow down rainfall runoff would have a C value in the range of 0.35. An impervious area such as a concrete lot or roadway would have a C value as high as 0.95, signifying increased runoff. Areas of mixed land coverage would have a value reflecting the coverage of that specific area. For each drainage area, a weighted C value was calculated to reflect each drainage areas land coverage.

The Rainfall intensity (I) is typically found from Intensity/Duration/Frequency curves for rainfall events in the geographical region of interest. The duration is usually equivalent to the time of concentration of the drainage area. The rainfall intensity was calculated using USGS pub (reference). The 2-hour rainfall in inches was used in this analysis. The Rational method requires a value in inches per hour at the outfall location. To obtain the intensity in inches per hour at the outfall location, the time of concentration was calculated for each watershed. The total rainfall in inches was divided by the time of concentration to obtain the intensity used in the runoff calculations.

Time of concentration is the time it takes for a point rainfall to travel to the outfall, and creates the maximum contribution of runoff from the drainage area. It is calculated by evaluating different travel paths to the outfall and calculating the time it would take for the entire drainage area to contribute to the discharge at the outfall.

As discussed previously, there is little to no data available to calibrate/verify/validate runoff estimates in many of these remote areas. Therefore, Rational method calculations were performed using a second method to calculate the intensity. The Intensity was based on IDF curves, time of concentration calculations, and runoff "C" Coefficients for the drainage areas. Using this independent method produced runoff results that showed good agreement (typically within 5-10 percent) with the previous flow estimates. This increased confidence that the previous estimates were fairly accurate and were adequate to be used later to size the structures associated with these areas. Table 2-15 tabulates the flows estimated for Orange County using the Rational method.

Table 2-15: Rational Method Calculated Discharges for the Sabine Region

Watershed ID	2-Year Discharge	5-Year Discharge	10-Year Discharge	25-Year Discharge	50-Year Discharge	100-Year Discharge
JN1	183	271	273	353	455	601
JN2	83	132	124	160	207	273
JN3	173	293	259	335	433	572
JN4	171	301	256	331	427	565
JN5	207	356	310	401	518	685
JN6	139	218	207	268	345	456
JN7	193	177	285	371	470	590
JN8	55	37	82	107	136	172
JN9	102	121	152	197	249	311
JS1	115	177	171	222	280	349
JS2	418	483	606	787	979	1,191
JS4	141	167	195	247	296	324
JS5	145	187	214	279	353	441
JS6	180	272	266	345	437	543
JS7	42	37	60	78	98	120
JS8	258	371	379	491	619	764
JS9	241	338	355	460	579	715
JS11	332	408	485	630	794	982
JS12	223	310	331	432	550	696
JS13	162	235	239	310	391	485

Watershed ID	2-Year Discharge	5-Year Discharge	10-Year Discharge	25-Year Discharge	50-Year Discharge	100-Year Discharge
JS14	83	84	122	158	199	247
OCE00	125	157	185	240	313	414
OCE0	150	187	220	287	372	491
OCE1	188	236	278	361	469	621
OCE2	92	114	134	174	226	299
OCE3	57	71	84	109	142	187
OCE4	180	223	263	342	444	585
OCE6	73	68	107	139	181	238
OCE7	94	79	137	178	231	304
OCE8	83	80	121	158	205	269
OCE9	46	37	66	86	112	147
OCE11	244	201	356	463	600	788
OCE12	127	100	185	240	311	409
OCE14	126	100	185	239	310	406
OCE15	85	70	125	161	210	276
OCE16	113	87	166	216	279	365
OCE17	83	68	122	159	206	270
OCE18	113	96	165	215	278	365
OCE19	61	43	90	117	151	197
OCE20	106	99	155	201	260	341
OCE22	167	154	245	318	412	541
OCE23	103	81	151	196	254	332
OCE24	116	101	170	221	286	374
OCE25	166	154	245	317	411	539
OCE26	83	62	122	159	206	269
OCW1	166	263	218	284	370	487
OCW3	183	316	295	384	499	656
OCW5	148	263	218	284	370	487
OCW6	178	309	262	341	443	584
OCW7	201	316	295	384	499	656
OCW8	128	151	186	243	315	413

2.15.9 Regression Method

Regression equations were used to calculate runoff for areas over 200 acres. Two Regression methods were used to limit uncertainty and compare results of the two methods. The first Regression method is one used by the FHWA.

The regression equation used for the FHWA Regression analysis is:

$$Q_T = P^c S^d \times 10^{[e\Omega + a + bA^{\lambda}]}$$

Q_T = peak discharge of recurrence interval T years (cfs)
P = mean annual precipitation in inches, from Texas Parks and Wildlife Board maps of the mean annual precipitation
S = dimensionless main channel slope
Ω = Omega value (Dimensionless parameter for specific areas of Texas, taken from TXDOT Hydraulic Manual 2014)
A, b, c, d, e = regression coefficients specific for the recurrence interval

The omega value for each location is taken from 2004 USGS maps for interior areas.

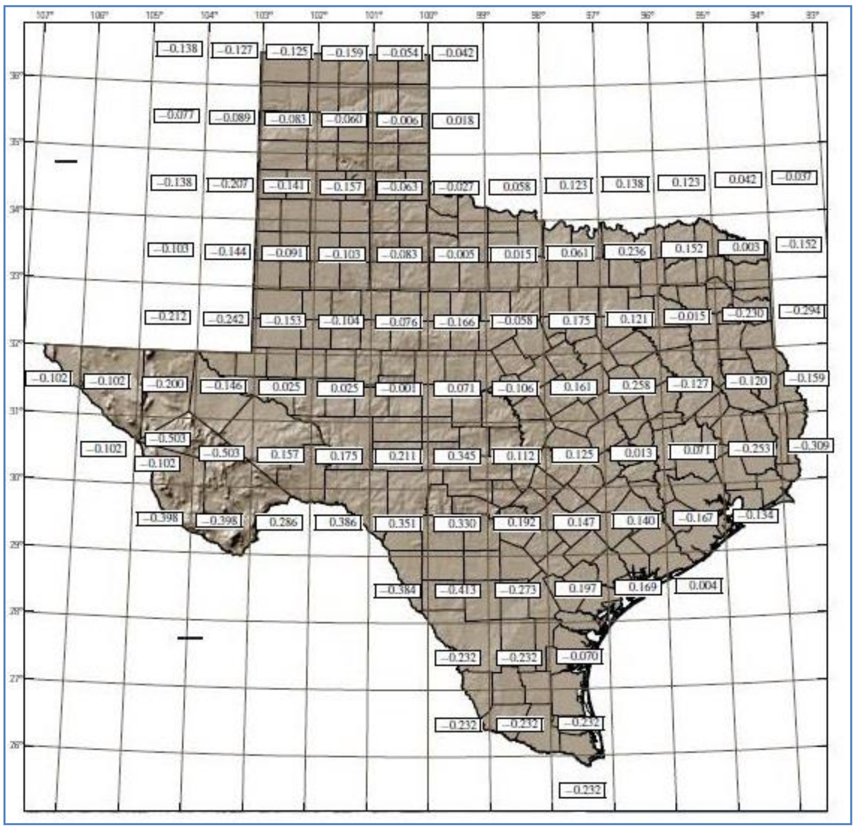

Figure 2-42: Mean Annual Precipitation Values from TXDOT 2014 Hydraulic Manual

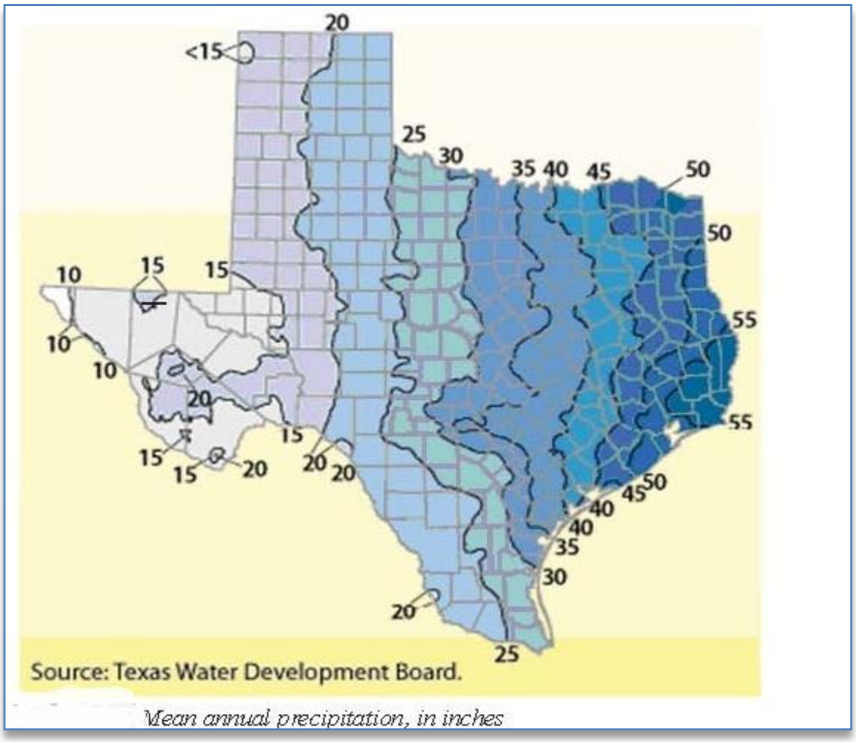

Figure 2-43: Omega Values Used for Regression Runoff Estimates

Table 2-16: The Equations Used for each Frequency Storm Event

Watershed ID	2-Year Discharge	5-Year Discharge	10-Year Discharge	25-Year Discharge	50-Year Discharge	100-Year Discharge
OCE5	158	234	280	351	396	665
OCE10	1,052	1,879	2,304	2,965	3,393	5,076
OCECB	2,009	3,596	4,329	5,542	6,228	8,637
OCE13	122	156	171	204	211	322
OCE21	121	175	213	266	303	528
OCW2	748	1,188	1,472	1,520	1,854	3,252
OCW4	214	316	380	435	502	867
OCW9	95	131	158	195	221	384
OCW10	260	402	483	601	679	1,111
OCW11	241	397	492	653	752	1,265
OCW12	81	114	140	182	210	376
OCW13	148	233	295	384	453	819
OCW14	359	604	738	999	1,124	1,770
OCW15	144	202	237	286	317	524
OCW18	196	321	403	550	636	1,088
OCW21	139	213	267	338	399	725
OCW29	133	205	258	330	390	708
JS3	88	132	170	230	278	531
JS10	335	574	739	901	1,099	1,997

*** All dischares in cfs**

Table 2-17: Regional Regression Discharge Summary Table

Regression Equations	RSE	Adj. R-squared	AIC statistic	PRESS statistic
$Q_2 = P^{1.398} S^{0.270} \times 10^{[0.776\Omega + 50.98 - 50.30A^{-0.0058}]}$	0.29	0.84	273	64.6
$Q_5 = P^{1.308} S^{0.372} \times 10^{[0.885\Omega + 16.62 - 15.32A^{-0.0215}]}$	0.26	0.88	122	49.1
$Q_{10} = P^{1.203} S^{0.403} \times 10^{[0.918\Omega + 13.62 - 11.97A^{-0.0289}]}$	0.25	0.89	86.5	46.6
$Q_{25} = P^{1.140} S^{0.446} \times 10^{[0.945\Omega + 11.79 - 9.819A^{-0.0374}]}$	0.26	0.89	140	49.5
$Q_{50} = P^{1.105} S^{0.476} \times 10^{[0.961\Omega + 11.17 - 8.997A^{-0.0424}]}$	0.28	0.87	220	55.6
$Q_{100} = P^{1.071} S^{0.507} \times 10^{[0.969\Omega + 10.82 - 8.448A^{-0.0467}]}$	0.30	0.86	320	64.8
$Q_{500} = P^{0.988} S^{0.569} \times 10^{[0.976\Omega + 10.40 - 7.605A^{-0.0554}]}$	0.37	0.81	591	98.7

Again, two different methods were used to check runoff estimates, as measured data are not available for calibration/verification/validation. Methodology for these estimates is based on equations and guidance outlined by the U.S. Department of Interior and U.S. Geological Survey (USGS) Water-Resources Investigations Report 96-4307.

The Regression Equation for this approach is: $Q_T = aA^bSH^cSL^d$

where:

A = Drainage Area (Mi2)

SH = Basin Shape Factor (SH) (Mi.2/Mi.2)

SL = Mean Channel Slope (SL) (Ft./Mi.)

A, b, c, d = Regional Regression Coefficients from the TXDOT 2004 Hydraulic Manual

Regional regression coefficients were taken from maps created by the USGS. Orange and Jefferson Counties are in Region 11, as seen in Figure 2-44. The Region 11 Regression Coefficients for 2-year, 5-year 10-year, 25-year, 50-year, and 100-year are listed in Table 2-18.

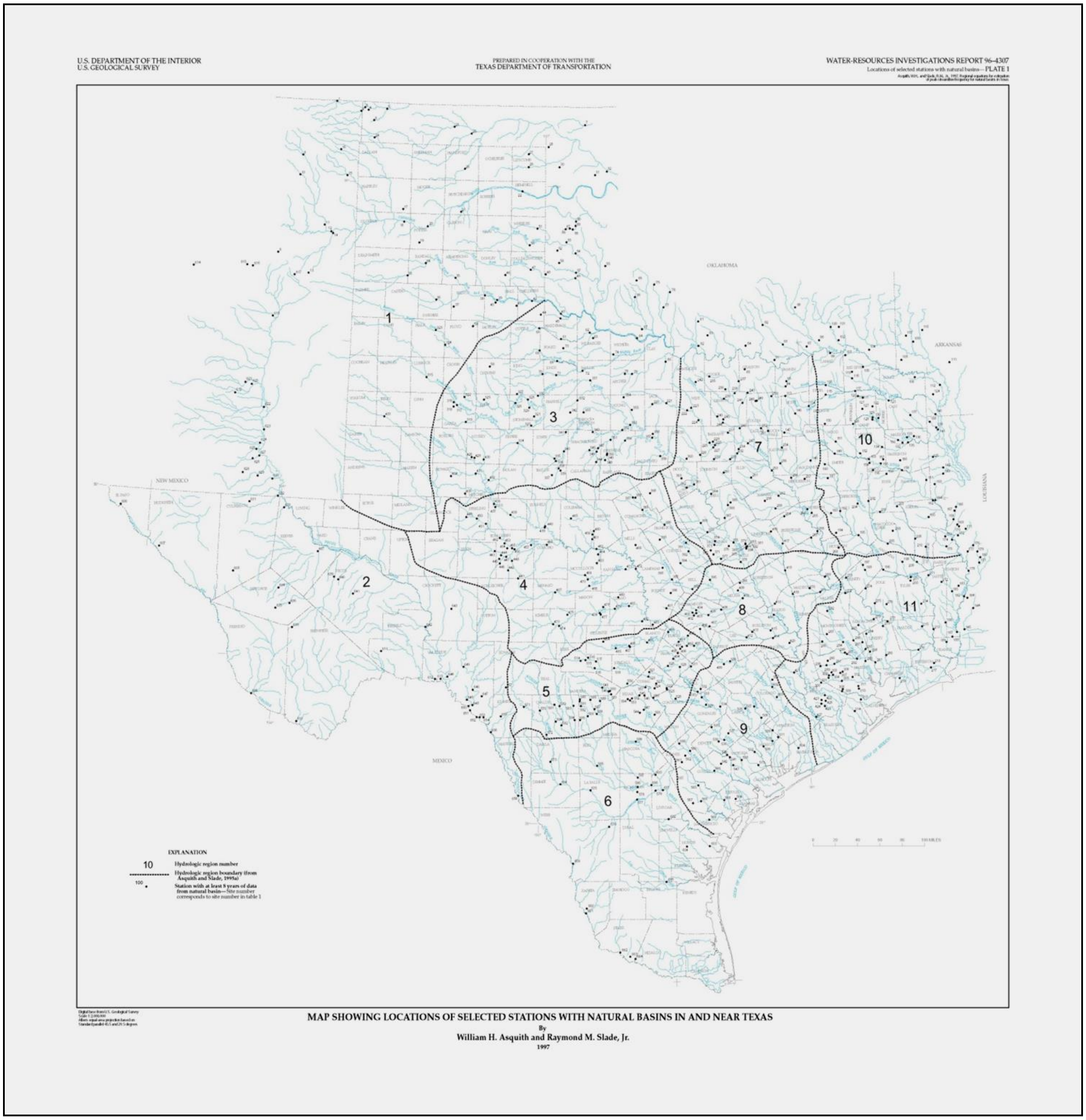

Figure 2-44: Region Map for the State of Texas

Table 2-18: Region 11 Regression Coefficients for Orange and Jefferson County

	a	b	c	d
2-Year	159	0.669	-0.262	0.000
5-Year	191	0.696	-0.186	0.130
10-Year	199	0.718	-0.151	0.221
25-Year	201	0.713	0.000	0.313
50-Year	207	0.735	0.000	0.380
100-Year	213	0.755	0.000	0.442

Results using the USGS/USDI Regression methods compared well with the Results calculated with the FHWA Regression method. The difference in flow estimates fit within the expected error of the previously used method, increasing confidence in the flow estimates. The Rational method can sometimes have considerable error and uncertainty, and being able to use two different Regression methods and obtain similar Discharge estimates provided verification that the results calculated using the Regression method were sufficient to use. These estimates were used to size structures for the new levee sections of the Sabine Region. Table 2-19 summarizes flows calculated by the Regression Methods.

Table 2-19: Average Regression Method Discharges Using USGS and FHWA Methodology

Watershed ID	2-Year Discharge	5-Year Discharge	10-Year Discharge	25-Year Discharge	50-Year Discharge	100-Year Discharge
OCE5	158	234	280	351	396	665
OCE10	1052	1879	2304	2965	3393	5076
OCECB	2009	3596	4329	5542	6228	8637
OCE13	122	156	171	204	211	322
OCE21	121	175	213	266	303	528
OCW2	748	1188	1472	1520	1854	3252
OCW4	214	316	380	435	502	867
OCW9	95	131	158	195	221	384
OCW10	260	402	483	601	679	1111
OCW11	241	397	492	653	752	1265
OCW12	81	114	140	182	210	376
OCW13	148	233	295	384	453	819
OCW14	359	604	738	999	1124	1770
OCW15	144	202	237	286	317	524
OCW18	196	321	403	550	636	1088
OCW21	139	213	267	338	399	725
OCW29	133	205	258	330	390	708
JS3	88	132	170	230	278	531
JS10	335	574	739	901	1099	1997

2.15.10 Pumps

When the system is closed during a tropical event, pumps will be required to discharge internal floodwaters. Preliminary requirements for pump discharges were estimated to offset any interior flooding due to the lack of outfalls during system closure. Currently, 11 new pump stations are proposed for the proposed new levee segments. The largest pumps will be located on Cow and Adams Bayous. Locations of pumps are shown on drainage area map plates in the engineering appendix. Many will require detention basins, routing features, and acquisition of land. These areas have been preliminarily identified and there is adequate space to construct these pump features.

For the proposed new levee sections, two existing pumps are currently in operation. An existing pump station exists along Coopers Gully, draining the majority of the City of Orange south of Interstate 10. An additional pump exists south of the Port of Beaumont and is operated and maintained by Exxon.

The Orange County pump has sufficient capacity to drain the watershed currently connected to it, and should only require minor modifications (1994 City of Orange Report). No increase in flows to the Orange pump will be created by the proposed project as the drainage area and runoff will not be altered by the proposed project. The pump is in need of maintenance and some part replacements; this is being addressed by the City of Orange. Some additional pump capacity (5-10 percent) may be needed for expected increases in precipitation (for climate change as discussed in Section 4 of this appendix).

The Exxon pump was constructed by Exxon after Hurricane Ike in 2008. The pump is known to be deficient and has failed multiple times since its construction due to rainfall runoff volumes exceeding the capacity of the pump. If the levee system proposed to protect the Exxon area remains as part of the selected plan, it will need to be replaced. The Exxon pump built after 2008 has not been tested during a hurricane and it is likely it will fail, as it was designed for a "Category 1" hurricane, as stated by Exxon.

2.15.11 Gate Structures

The proposed levee system would include two major gates structures with necessary appurtenances. The gates structures would be on Cow Bayou and Adams Bayou, which are both used for small ship navigation. The channel dimensions are 100 feet by 14 feet MLLW for Cow Bayou and 100 feet by 13 feet MLLW for Adams Bayou.

Preliminary gate configurations were determined and modeled in simple HEC-RAS models to determine and minimize impacts on the upstream floodplains. These models should be expanded upon and follow the guidance from HEC in the latest HEC-RAS manual. Regression equations were used to estimate flows. It is recommended that HEC-HMS models be constructed during PED. This was not possible during the feasibility phase due to the large and complex watersheds discharging into the Bayou

The Cow Bayou closure gate system would include a sector gate in the main channel and vertical lift gates on either side for rainfall runoff to pass through. The Adams Bayou gate structure would include a sector gate in the main channel and a vertical lift gate on the west sides of the channel. On the east side of the proposed Adams Bayou gate location, are two old placement areas that will be used as part of the levee system. Therefore, no vertical lift gate or other relief structure is proposed on the east side of the Adams Bayou gate. Both gates will require substantial pump stations to discharge interior runoff during a system closure. Orange County is currently working on these models and it is recommended that USACE team up with the county on the modeling efforts if possible. It is conceivable that Orange will have completed their HEC-HMS and HEC-RAS models of the Bayou by the time USACE enters the PED phase. If so, the models from Orange County should be utilized and modified as needed to meet USACE regulations and standards.

2.16 FLOODPLAIN IMPACT ANALYSIS

The preliminary design was performed in method to minimize impacts on all existing floodplains. Outfalls for interior drainage were sized to pass the 100-year storm plus a 10 percent increase in flow for climate change. The culvert outfalls were aligned and sized in a manner that mimics the existing waterways. Culvert spans and heights were chosen to provide widths that would prevent head build-up on the upstream side of the culverts, which could cause adverse impacts. This is because of the flat topography in the project area. The Cow and Adams Bayou gate structures were configured to allow a 100-year flood event to be discharged without impacting their respective floodplains.

For most of the waterways, only basic hydrologic and hydraulic models were created for outfalls. As the levee was refined, many outfalls were eliminated. Use of HEC-HMS and HEC-RAS (or similar acceptable H&H models) is recommended to model the final outfalls. Quantitative impacts for Neches River floodplain and a Qualitative assessment for Sabine River indicate minimal or insignificant impacts. The vast majority of the existing and proposed levee system is outside the banks of the rivers and is not in the 100-year floodplains. The Neches River passes between levee systems and the Sabine River passes along the east side of levee. Levees were kept out of the floodplains of the rivers as much as possible, minimizing impacts and with the intention of not disturbing the natural courses of the rivers.

The selected alignment, outfall location/type/size, and materials used were selected to minimize impacts on tidal areas, some of which are quite sensitive along the south portion of the Orange levee and adjacent Bessie Heights. No proposed levees infringe on Bessie Heights. This is further discussed in the H&H section on environmental considerations.

2.17 ENVIRONMENTAL IMPACTS OF PROPOSED DRAINAGE SYSTEM

Modifications to the existing systems and the construction of new levee systems have the potential to cause adverse environmental impacts. This section details studies done to assess possible impacts and discussions and decisions made to minimize impacts of drainage structures on environmental aspects of the levee systems.

New drainage components were evaluated for possible impacts on the existing environment in Orange and Jefferson Counties, and to the extent possible efforts were made to minimize any adverse impacts. ERDC-CHL conducted a study to determine possible impacts due to the closure gate structures that would be needed on Cow and Adams Bayou gate as part of the levee. Outfalls consisting of box culverts and concrete pipe culverts were sized and placed in a manner minimizing impacts on the environment. Overall, a least impact configuration was established through coordination and meetings with the EPA, NMS, USFW, TXDOT, TWDB, and TPWD.

The details discussed herein support the environmental assessment of the project as established during the study feasibility phase. The exact configuration and additional details regarding the proposed drainage structures design shall be refined as needed in PED. Continuous coordination with all involved environmental agencies shall occur through PED and construction. It is worth noting that the initial levee footprint shown to the environmental agencies has significantly decreased and so have the environmental impacts.

2.17.1 Gate Structures on Cow and Adams Bayous

ERDC-CHL conducted a study to qualitatively and quantitatively assess possible environmental impacts of the proposed sector gates on Cow and Adams Bayous (Gunkel and Brown, March 2015). The implementation of the gate structures on the bayous has the potential to constrict the inlets to both bayous (near the bayous confluence with the Sabine River). The constriction on the bayou inlets created by the proposed gates could impact velocities, salinity, and cause storm water impoundment. ERDC-CHL utilized a Desktop Off-Channel Wetland Salinity Mitigation Model (DOWSMM) to perform an analysis to quantitatively assess the impacts of the proposed gates. DOWSMM is a mass and energy balance model that assumes each bayou can be represented as a single control volume. This model was used in lieu of more sophisticated numerical models for several reasons. These include:

- The paucity of available data for the Bayous (especially velocity and discharge data) makes calibration and validation of a numerical model impossible without a significant data collection effort.
- The Bayous being evaluated fit the description of systems for which DOWSMM is designed (i.e. small, single inlet tidal systems with a single inflow and rainfall/evaporation).
- The DOWSMM analysis, in the absence of sufficient field data for calibration, is designed as a screening analysis only: i.e., bracketing potential impacts. The intent is to use the results of this analysis to determine whether or not a more significant investment in more detailed numerical modeling is necessary.

Figure 2-45 shows the Cow and Adams Bayous areas of interest. The blue lines indicate major and minor streams and the yellow line indicates the proposed structure alignment.

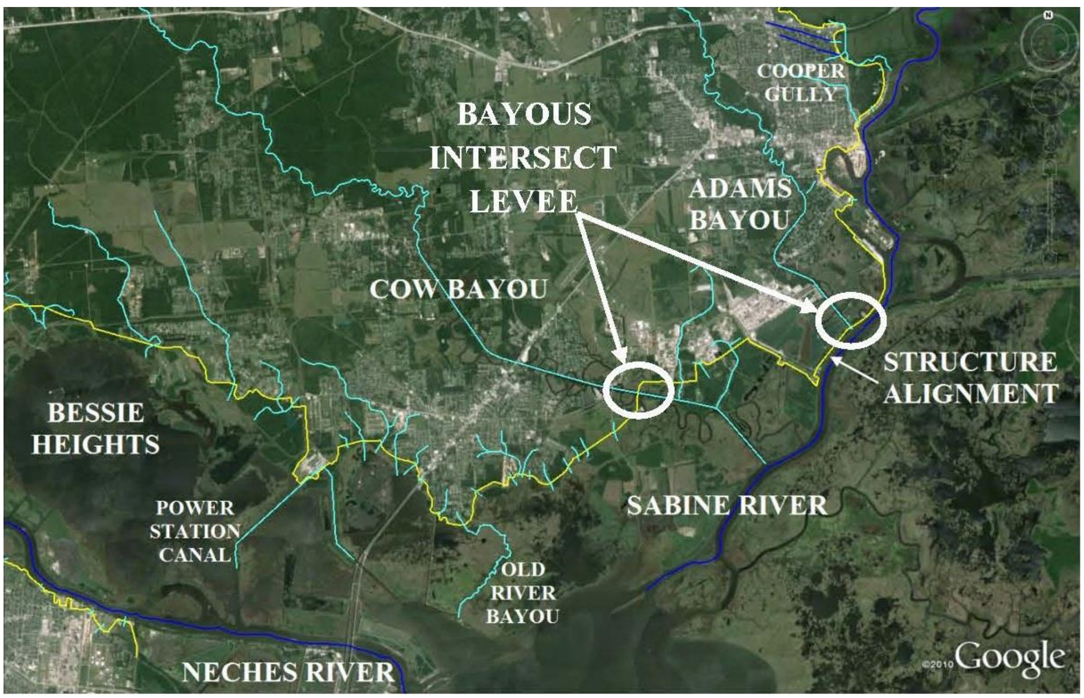

Figure 2-45: Cow and Adams Bayou Intersection with Gate Structures

DOWSMM requires time-varying boundary conditions. These include tide and salinity at the inlet, rainfall/evaporation over the wetland (or bayou) and freshwater inflow into the wetland (or bayou). For this study, these data were taken from a TABS-MDS model study performed in support of a larger feasibility study associated with proposed channel deepening in the Sabine Neches Estuary (Brown, et. al. 2006).

For this DOWSMM exercise, data associated with an observation point in close proximity to the intersection of the Sabine River and Adams and Cow Bayous were chosen for the tide and salinity boundary condition data. The data point is identified as Gage 9 (the location of Gage 9 is depicted in Figure 2-46). This gage was used to obtain low and median flow tide and salinity boundary conditions. Applied net rainfall data (rainfall minus evaporation) for low and median flow simulations were used to calculate discharge. This discharge was calculated by multiplying net rainfall rate by the surface area of the drainage area of each bayou.

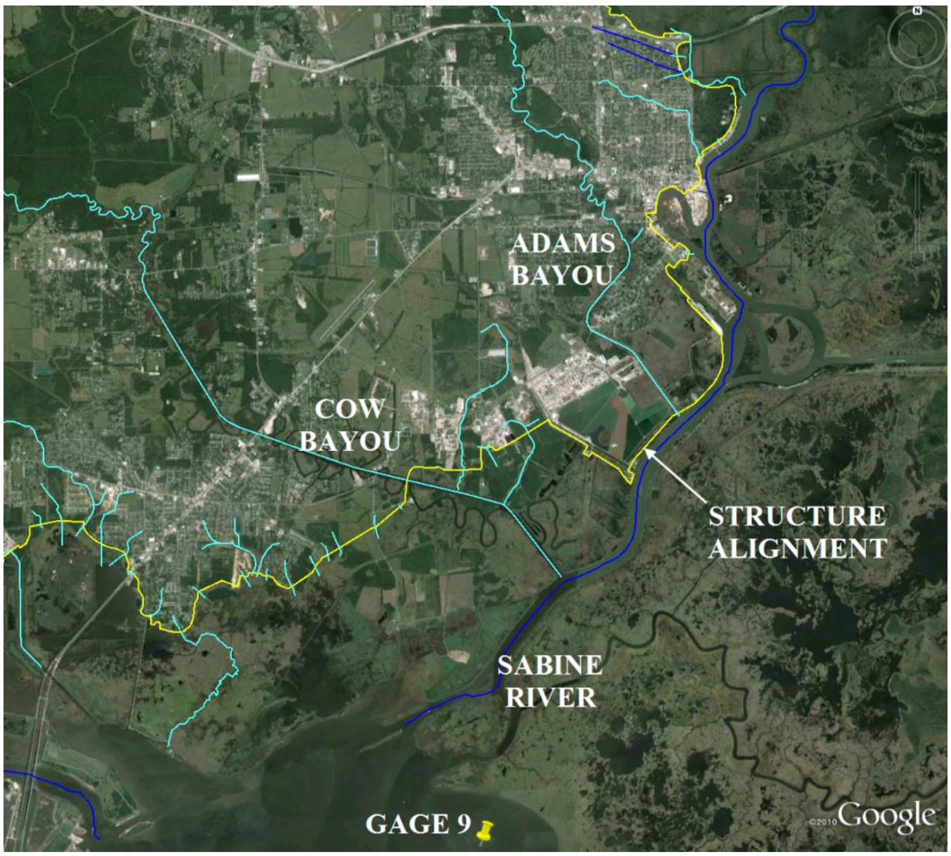

Figure 2-46: Gage 9 Location

2.17.2 Cow Bayou Analysis

The intersection of Cow Bayou and the proposed structure is approximately 2.5 miles north of the mouth where Cow Bayou meets Sabine River. Figures 2-45 and 2-46 show the location of the proposed gated structure on Cow Bayou. A series of varying inlet widths were tested, including 120 feet, 110 feet, 100 feet, 80 feet, and 50 feet widths. Table 2-20 contains all base parameters used for the Cow Bayou model runs.

Table 2-20: Cow Bayou DOWSMM Model Parameters

Parameter	Base
Wetland surface area (ft^2)	21000000
Wetland width (ft)	100
Wetted surface area factor (0=wet,1=dry)	0.75
Roughness height for the wetland	0.1
Width of the inlet (ft)	100
Elevation of the bed in the inlet (ft)	-11
Elevation of the top of the sill in the inlet (ft)	-11
Elevation of the bed in the wetland (ft)	-2.0
Gravitation constant (ft/s^2)	32.2

Low flow conditions inlet widths of 120 feet, 110 feet, 100 feet, 80 feet, and 50 feet were tested and results for 120 feet and 50 feet are provided here. Tables 2-21 and 2-22 show resulting statistics from the low flow runs. The minimum and maximum columns are the minimum and maximum values, respectively, of the respective parameters. The remaining 90, 50, and 10 percent columns represent percent exceedance values; meaning that 90 percent of the values for that parameter exceed the value.

Table 2-21: Cow Bayou Base Parameters: Low Flow, 120-foot Inlet Width

Parameter	Minimum	90 % exceedance	50 % exceedance	10 % exceedance	Maximum
Inlet Water Surface Elevation	-0.310	0.420	1.180	1.730	2.760
Inlet Salinity	5.040	8.050	16.420	20.170	22.260
Wetland Freshwater Inflow	0.000	0.000	0.000	134.758	1395.312
Inlet Discharge Magnitude	0.000	91.904	465.462	919.008	1552.101
Inlet Velocity Magnitude	0.000	0.062	0.321	0.632	1.150
Wetland Water Surface Elevation	-0.288	0.420	1.171	1.735	2.760
Wetland Salinity	0.479	5.232	13.915	19.207	20.565

Table 2-22: Cow Bayou Base Parameters: Low Flow, 50-foot Inlet Width

Parameter	Minimum	90 % exceedance	50 % exceedance	10 % exceedance	Maximum
Inlet Water Surface Elevation	-0.310	0.420	1.180	1.730	2.760
Inlet Salinity	5.040	8.050	16.420	20.170	22.260
Wetland Freshwater Inflow	0.000	0.000	0.000	134.758	1395.312
Inlet Discharge Magnitude	0.000	93.268	466.529	914.058	1547.946
Inlet Velocity Magnitude	0.000	0.152	0.775	1.507	2.759
Wetland Water Surface Elevation	-0.286	0.420	1.172	1.735	2.760
Wetland Salinity	0.486	5.221	13.910	19.207	20.562

Under low-flow conditions, the velocity at the inlet increases roughly in proportion to the degree of contraction of the inlet. This is because the energy loss across the inlet is negligible, and hence the discharge though the inlet is nearly the same for the 120 feet and 50 feet inlet width conditions. No significant impacts were seen for the salinity or water surface elevations. When comparing the results from the maximum inlet width tested of 120 feet to the minimum inlet width tested of 50 feet, the only differences seen in the water surface elevation or salinity are on the order of 0.001, and the differences in salinity are on the order of 0.05 ppt.

For median flow the same inlet widths were tested, and results for 120 feet and 50 feet are provided here. Tables 2-23 and 2-24 show resulting statistics from the median flow runs with the different inlet widths.

Table 2-23: Cow Bayou Base Parameters: Median Flow, 120-foot Inlet Width

Parameter	Minimum	90 % exceedance	50 % exceedance	10 % exceedance	Maximum
Inlet Water Surface Elevation	-0.930	0.530	1.320	1.960	3.050
Inlet Salinity	-0.130	0.160	2.370	11.360	17.010
Wetland Freshwater Inflow	0.000	0.000	0.000	195.938	5425.234
Inlet Discharge Magnitude	0.000	93.419	461.537	934.180	5759.486
Inlet Velocity Magnitude	0.000	0.063	0.314	0.637	3.837
Wetland Water Surface Elevation	-0.868	0.520	1.319	1.964	3.178
Wetland Salinity (ppt)	0.000	0.235	1.297	10.140	14.153

Table 2-24: Cow Bayou Base Parameters: Median Flow, 50-foot Inlet Wwidth

Parameter	Minimum	90 % exceedance	50 % exceedance	10 % exceedance	Maximum
Inlet Water Surface Elevation	-0.930	0.530	1.320	1.960	3.050
Inlet Salinity	-0.130	0.160	2.370	11.360	17.010
Wetland Freshwater Inflow	0.000	0.000	0.000	195.938	5425.234
Inlet Discharge Magnitude	0.000	95.524	459.812	926.542	5757.070
Inlet Velocity Magnitude	0.000	0.152	0.755	1.518	9.204
Wetland Water Surface Elevation	-0.866	0.522	1.318	1.966	3.298
Wetland Salinity (ppt)	0.000	0.235	1.295	10.144	14.144

According to these results, for a large rainfall event (the event tested is the observed rainfall associated with Tropical Storm Allison) only about a 0.10-foot impact was seen on the water surface elevation. Inlet constricts the outflow such that there is some increase in both the magnitude and duration of the stormwater flooding in the marsh. To mitigate this effect, any significant constriction of the inlet will include design elements (such as flap gates, vertical lift gates, equalizer culverts, or gated culverts) to allow stormwater release from behind the structure and allow outflow of water avoiding adverse impacts upstream of the gate structure.

2.17.3 Adams Bayou Analysis

The proposed structure on Adams Bayou is very near the inlet to the Sabine River. The same set of flows and salinity data collected from Gage 9 that were used for Cow Bayou were also used for Adams Bayou. Figure 2-43 shows the location of the structure alignment in relation to Adams Bayou and the Sabine River.

Base parameters for Adams Bayou were chosen from the HEC-RAS model provided by Carroll & Blackman, Inc. (CBI) for the width of the inlet, depth, and length of the section. These parameters were modified to some extent by information from Google Earth (2015). Figure 10 shows the wetland surface area that was approximated by using Google Earth (2015); the remaining parameters were estimated. A series of varying inlet widths were tested, the results from the maximum width of 350 feet and minimum width of 100 feet are shown here. Table 2-25 contains all base parameters used in the runs.

Table 2-25: Adams Bayou DOWSMM Model Parameters

Parameter	BASE
Wetland surface area (ft^2)	50000000
Wetland width (ft)	350
Wetted surface area factor (0=wet,1=dry)	0.75
Roughness height for the wetland	1.0
Width of the inlet (ft)	350
Elevation of the bed in the inlet (ft)	-5
Elevation of the top of the sill in the inlet (ft)	-5
Elevation of the bed in the wetland (ft)	-2.0
Gravitation constant (ft/s^2)	32.2

Low flow conditions modeling was performed as described previously in 6.2.2. Tables 2-26 and 2-27 contain the statistical results from the low flow runs for Adams Bayou.

Table 2-26: Adams Bayou Base Parameters: Low flow, 350-foot Inlet Width

Parameter	Minimum	90 % exceedance	50 % exceedance	10 % exceedance	Maximum
Inlet Water Surface Elevation (ft)	-0.310	0.420	1.180	1.730	2.760
Inlet Salinity (ppt)	5.040	8.050	16.420	20.170	22.260
Wetland Freshwater Inflow (cfs)	0.000	0.000	0.000	45.158	467.578
Inlet Discharge Magnitude (cfs)	0.000	235.329	1075.041	1910.239	2871.270
Inlet Velocity Magnitude (ft/sec)	0.000	0.111	0.509	0.896	1.239
Wetland Water Surface Elevation (ft)	-0.204	0.417	1.154	1.723	2.759
Wetland Salinity (ppt)	5.080	7.139	15.270	19.415	20.553

Table 2-27: Adams Bayou Base Parameters: Low Flow, 100-foot Inlet Width

Parameter	Minimum	90 % exceedance	50 % exceedance	10 % exceedance	Maximum
Inlet Water Surface Elevation (ft)	-0.310	0.420	1.180	1.730	2.760
Inlet Salinity (ppt)	5.040	8.050	16.420	20.170	22.260
Wetland Freshwater Inflow (cfs)	0.000	0.000	0.000	45.158	467.578
Inlet Discharge Magnitude (cfs)	0.000	261.733	1069.031	1771.663	2580.447
Inlet Velocity Magnitude (ft/sec)	0.000	0.407	1.754	2.944	3.857
Wetland Water Surface Elevation (ft)	-0.167	0.439	1.150	1.713	2.754
Wetland Salinity (ppt)	5.080	7.150	15.128	19.433	20.501

As with Cow Bayou, the velocity at the inlet increases roughly in proportion to the degree of contraction of the inlet. This is because the energy loss across the inlet is minimal, and hence the discharge though the inlet is nearly the same for the 350 ft and 100 ft inlet width conditions. Similar to Cow Bayou, no significant impacts are seen in the water surface elevation or salinity.

Median flow conditions were also modeled as described previously. Tables 2-28 and 2-29 contain the statistical results for the median flow runs for Adams Bayou.

Table 2-28: Adams Bayou Base Parameters: Median Flow, 350-foot Inlet Width

Parameter	Minimum	90 % exceedance	50 % exceedance	10 % exceedance	Maximum
Inlet Water Surface Elevation (ft)	-0.930	0.530	1.320	1.960	3.050
Inlet Salinity (ppt)	-0.130	0.160	2.370	11.360	17.010
Wetland Freshwater Inflow (cfs)	0.000	0.000	0.000	56.109	1553.590
Inlet Discharge Magnitude (cfs)	0.000	248.127	1064.117	1929.731	3507.700
Inlet Velocity Magnitude (ft/sec)	0.000	0.113	0.491	0.888	1.489
Wetland Water Surface Elevation (ft)	-0.751	0.532	1.305	1.952	3.058
Wetland Salinity (ppt)	0.067	0.337	1.767	10.387	14.075

Table 2-29: Adams Bayou Base Parameters: Median flow, 100-foot Inlet Width

Parameter	Minimum	90 % exceedance	50 % exceedance	10 % exceedance	Maximum
Inlet Water Surface Elevation (ft)	-0.930	0.530	1.320	1.960	3.050
Inlet Salinity (ppt)	-0.130	0.160	2.370	11.360	17.010
Wetland Freshwater Inflow (cfs)	0.000	0.000	0.000	56.109	1553.590
Inlet Discharge Magnitude (cfs)	0.000	265.851	1050.655	1813.801	3329.004
Inlet Velocity Magnitude (ft/sec)	0.000	0.412	1.693	2.900	5.021
Wetland Water Surface Elevation (ft)	-0.729	0.549	1.298	1.946	3.072
Wetland Salinity (ppt)	0.071	0.348	1.744	10.353	13.972

As with Cow Bayou, an increase in the water surface elevation associated with the storm event is observed. The effect of the constriction is qualitatively similar to that observed at Cow Bayou, but is less in magnitude. This is because Adams Bayou has more wetland surface area, and hence the impounded stormwater can spread over a larger area (the same storage volume requires less depth). The structure will designed with relief structures including vertical lift gate structures to allow additional flow to pass and mitigate any impacts on the upstream floodplain.

2.17.4 Neches River Gate

A closure gate on the Neches River near Sabine Lake was considered as an alternative during the screening process. The gate was eliminated from inclusion in the proposed plan based on benefit/cost considerations. Therefore, no detailed modeling was done to assess environmental impacts a gate on the Neches River would have. If the gate is later determined to be a Locally Preferred Alternative, an analysis of environmental impacts will need to be performed.

2.17.5 Discussion and Conclusions of Cow and Adams Proposed Gate Structures

It is likely that minimal potential impacts would be created by increased velocities in the inlets on larval transport into the bayous. This can be inferred from higher order model work that was performed at ERDC to investigate similar impacts for the Keith Lake Fish Pass Baffle (Brown and Lackey, 2011). In this work, it was demonstrated that the larval transport is not impeded by increases in velocity due to a constriction, but is impeded by the formation of large eddies associated with flow around blunt obstructions. Therefore, if the constrictions are designed with smooth transitions from wide to narrow, it is unlikely that the increased velocities alone will be a significant hindrance to larval recruitment.

Impacts imparted to the regular, tidal variation of water surface elevation and salinity within Adams and Cow Bayous from the proposed inlet constrictions were found to be negligible. This negligibility was determined by the sensitivity analysis conducted on the inlet size for each bayou. It was determined that the limited tidal prism associated with the bayous results in minimal energy loss across the connection between the bayous and the Sabine River; hence, constriction of this access point results in little change in the tidal energy passing into the bayou. The median flow simulation contains a significant rainfall event (Tropical Storm Allison), so the extent to which these constrictions tend to impound stormwater within the bayous is also examined. For both Cow Bayou and Adams Bayou, the inlet constriction causes some increase in both the magnitude and duration of the stormwater flooding in the marsh. This flooding will be mitigated by inclusion of design elements (flap gates, vertical lift gates, or gated culverts) that allow stormwater release from behind the structure.

Additional sensitivity analyses were conducted to investigate the impact of changes on each of the estimated parameters that define the bayous. The values were independently perturbed by 25-50 percent, and the results compared to the results given in this report for the “base” parameters. In no case were significant impacts observed.

Hence, in spite of the paucity of data available to properly define and calibrate/validate these desktop models of Adams and Cow Bayous, the insensitivity of the tidally varying water surface elevation and the salinity impacts on a wide range of estimated parameters give high confidence that the general conclusions of the ERDC study are robust. They are as follows:

- For general tidal conditions, the limited tidal prism associated with each of these bayous results in little energy loss across the inlet and hence, constriction of the inlet, even significant constriction, results in minimal impacts on water surface elevation and salinity within the bayous.
- For large freshwater inflow events, a significant constriction at Cow and Adams Bayous could result in additional impoundment of stormwater behind the structure. To mitigate for this additional stormwater drainage features will be utilized in the design.

2.17.6 Culverts and additional Interior Drainage Structures

Additional structures were preliminarily assessed to allow rainfall runoff from the interior of the proposed levee system to discharge rainfall runoff from the interior of the levees. Culverts and pump structures were placed along the proposed levee alignments and preliminary design parameters for these structures were determined. Existing outfalls and pumps were identified and data gathered on them where available. It is recommended more data be gathered in the field during PED. Locations and sizes of proposed culverts and pumps are discussed in Section 5 of this appendix. This section is focused on the environmental aspects of the proposed drainage system. The interior drainage structures were strategically placed to:

- Maintain existing drainage and minimize or eliminate any adverse impacts on floodplains
- Provide interior drainage via pump systems to evacuate floodwaters during a tropical event that would be associated with a system closure
- Maintain connectivity of environmental areas such as marshes and wetlands
- Minimize constriction of waterways by drainage structures and maintain cross sectional areas as best practicable
- Minimize environmental impact by utilizing and configuring the proposed drainage structures in a manner that aims to mimic existing without-project conditions

This section discusses how possible environmental impacts associated with the proposed interior drainage systems were considered and how possible impacts were addressed using BMPs and principles.

To provide interior drainage of the proposed levee system, minimize impacts on areas that are tidally influenced, and maintain connectivity between environmentally linked areas, a preliminary plan was developed for the culverts and pumps that would be necessitated by the construction of the proposed levee. Bayous, creeks, gullys, and all waterways that cross the levee systems were identified. All watersheds that the levee systems intersect or impact were identified. Detailed drainage area maps are included in the Exhibits section of this appendix. These maps include locations of all proposed outfall and pump stations. Drainage areas were delineated using a combination of LiDAR, USGS maps, aerial maps, historical photos, survey data, construction plans and specifications, agency input from meetings and discussions with TPWD/USFWS/NMFS/USEPA/TXDOT/TWDB, and historical reports and documents.

Culvert locations were identified using all the information obtained from data gathering, state and agency input, and the interior drainage analysis performed (Section 5 of the H&H section of engineering appendix). Several distinct types of culverts were distinguished with regards to their required functionality draining the interior of the levee system, surrounding environments, upstream watershed characteristics, and characteristics of the land downstream of the outfall. The culverts were broken into four subgroups.

1. Culverts at high elevation, typically surrounded by dry land that would have minimal if any environmental impact.
2. Culverts that serve as drainage outfalls in lowlands and/or environmentally sensitive areas.
3. Culverts that are tidally influenced in low-lying coastal areas, including those that would need to allow tidal flow to pass through.
4. Culverts that serve as equalizer culverts were identified in discussions with state and agency representatives and by reviewing aerial maps. These culverts are not required to drain the interior. The purpose of these culverts is to maintain connectivity between environmentally sensitive area such as marshlands or wetlands, and would rarely, if ever, serve as a pathway for interior drainage.

Culverts have been strategically placed and sized to best mimic the natural waterways and environment as much as practicable. All culverts placed for interior drainage are of a multiple structure configuration (i.e., have numerous openings) with widths and locations intended to mimic the natural waterways. This includes utilizing culverts with more width than height in an attempt to maintain cross sectional areas and natural channel cross sections. This approach serves multiple purposes, including minimizing impacts on culverts located in tidal passes. Additionally, having larger span culverts will reduce the backwater (upstream head) that would occur during storm runoff events. The study area is in general very flat and high water surface elevation on the interior of the culvert is undesirable, as it could produce adverse impacts on the upstream floodplain.

Culverts will include will include shoreline baffles and/or ramps (e.g., rock rubble, riprap, articulated concrete mats) that slope to the structure invert to enhance organism passage where appropriate. Various ramp designs will be considered depending on site specific conditions. Culverts have been sized and preliminarily designed to produce velocities of 3 ft/s for a 2-year storm event to prevent siltation and prevent debris build-up, as is standard practice. Culverts will use sluice gates and flap gates to close openings during storm events. At all other times, the culverts should remain fully opened, with the exception of short duration closures for maintenance and inspections. The specific details and design of these riparian access features and culvert flow-lines will be refined during PED.

Drainage pathways, including sheet flow, over flow, and shallow concentrated flow, shall remain the same inside of the levee. The flows will be collected by toe drains at the base of the levee and directed to the outfall. Detailed design of these drains will occur during PED; it is anticipated the toe drains will vary on a case by case basis in term of material, slope, size, and configuration. Once the water is discharged through the multiple dispersed culverts crossing to the exterior of the levee, the relatively flat culverts will allow for the water to disperse rapidly and follow its natural course.

During conversations and meetings between environmental agencies and USACE, several areas were identified to place ‘equalizer’ culverts to maintain environmental connectivity and decrease environmental impacts and decrease mitigation needs. Most of these culverts will be 24 to 36-inch pipes, but three will be box culverts, including a sizable multiple box culvert equalizer culvert connecting the east side of the marsh near the Adams Bayou/Sabine River confluence.

Low lying tidally influenced culverts are sized to maintain tidal prisms/cross sectional areas, and minimize changes in velocities to the extent practicable. These culverts are to be designed to best match velocities correlating to flood and ebb tides. The current practice is to limit the maximum velocity of tidal velocities to 2.6 ft/s (preferably less, in the range of 0.5 ft/s to 0.82 ft/s) to minimize impacts on larval fish (NMFS 2008).

NMFS Fisheries 'Friendly Design and Operation Considerations for Hurricane and Flood Protection Water Control Structures' (NMFS 2008) was utilized for additional guidance on drainage structure placement and design considerations. This document was used to help identify design and operational guiding principles that would optimize passage of estuarine dependent marine fisheries species, or at least, minimize adverse impacts on their passage through hurricane and flood protection water control structures. The number of drainage structures and distances between structures closely match without-project conditions, and shall be further optimized to minimize migratory distances from the openings to enclosed wetland habitats during PED.

An operational plan will be completed in conjunction with the PED phase of the project. The plan will include the direction that culverts and gates will remain fully open at all times except during surge events, for short duration operational testing, and maintenance checks and inspections. The operational plan will also include direction on timing of closing and opening of culverts and gates as a storm approaches and passes. The operational plan shall include procedures to allow timely opening of culvert and gate structures in the absence of an off-site power source.

2.17.7 Pumps

Proposed pump stations will be designed and located to have minimal environmental impact. Preliminary selected locations for pump stations are included on the drainage area maps in the exhibits of this section of the engineering appendix. Pumps will not be used under normal conditions, only during storm events to evacuate water from the interior of the levee. One exception is the existing pump station used by the City of Orange. This pump station drains the majority of the City of Orange south of Interstate 10 via storm sewer, open ditches, and diversions. It is active during normal conditions for drainage of rainfall runoff and will remain so. This station will remain active during storm events when the levee system is closed, as it will need to continue to evacuate interior rainfall runoff.

Some areas will be disturbed or altered on the interior of the levee to place the proposed pump stations with associated features. The footprint and impacts will be optimized, reassessed, and detailed during the PED phase of the project. These features could include detention/retention ponds, toe drains along the levee to direct flow to the pumping stations, and other typical pump station ancillary features. The pumps shall be designed to avoid impacts on wetlands as much as possible. Smaller features such as riprap and stepped structures along the toe drains and other features for wildlife will be included as appropriate and detailed during PED.

For the existing Freeport and Port Arthur systems it is not anticipated that any new culverts, pumps, or other drainage changes besides some minor modifications and possibly some replacements/removal of older drainage structures may be needed. Existing pump stations currently have adequate capacity and will be studied in further detail during PED. More information and design calculations will be needed during PED to further assess existing pump stations.

2.17.8 Drainage Environmental Impacts Summary and Conclusions

The proposed levee system will require numerous drainage structures, including culverts, pumps, and two large closure gate structures. Best Management Practices were used extensively to minimize environmental impacts caused by the proposed levee system. Culverts were sized and placed in configurations that attempt to mimic the natural without-project characteristics to the extent practicable. This included using multiple structures at each drainage outfall for the proposed levee system. Additional reinforced concrete pipes were strategically placed to maintain connectivity of environmental area, with substantial coordination with the USEPA, USFW, TPWD, TXDOT, TBWD, and NMFS. These ‘connectivity’ culverts are not needed for discharging rainfall runoff from the interior of the proposed levees; their purpose is solely to minimize the environmental impacts of the proposed levee.

Closure gate structures will be required on Cow and Adams Bayou to prevent surge from entering the proposed levee interior. A variety of gate configurations were modeled by ERDC using DOWSMM to determine possible impacts on velocities, tidal prisms, and stormwater impoundment due to the gate structures. The study found that for general tidal conditions, the limited tidal prism associated with each of these bayous results in little energy loss across the inlet, and that even significant constriction results in minimal impacts on water surface elevation and salinity within the bayous. For large freshwater inflow events, a significant constriction at Cow and Adams Bayous could result in additional impoundment of stormwater behind the structure. To offset stormwater impoundment, additional drainage features will be constructed adjacent to the main gate structures (Gunkel and Brown 2015).

The drainage components of the levee system were optimized to minimize environmental impacts. Further refinement and optimization will occur during the PED phase of the project, and changes shall be documented and provided to the environmental agencies to review and submit recommendations.

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3 SURVEYING, MAPPING, AND OTHER GEOSPATIAL DATA REQUIREMENTS

FEMA LiDAR data from 2010 was used in the H&H storm surge modeling. This data was also imported into Bentley InRoads to create ground surfaces along the proposed new HFPP alignments that were used to determine the relative levee and floodwall heights and to calculate associated fill quantities for the levee construction. The voluminous data for the entire region was isolated to the project areas and thinned out to be within the program's capability of processing it. Elevations from the LiDAR surveys were compared to elevations physically taken at the ground surface of each core boring using Real Time Kinematic (RTK) system equipment. It was found that the LiDAR elevations near the core boring locations compared favorably with the core boring elevations, with differences typically being between 0.10 and 0.60 foot.

Other surveying, mapping, and geospatial information/tools came from the following resources:

- Satellite imagery and data published by Google Earth Pro
- Satellite imagery published by ESRI
- FEMA Flood Insurance Rate Maps
- Published tide and water level data from NOAA stations
- Oil and gas pipeline GIS database maintained by the Texas Railroad Commission (TRRC)

4 GEOTECHNICAL

4.1 SUMMARY

The objective of this document is to provide a detailed background of the geotechnical work performed throughout the preliminary design of a proposed Hurricane and Storm Damage Risk Reduction (HSDRR) system. The system will include levee embankment, floodwall and other features. This feasibility document focuses on foundation conditions and levee embankments, floodwall details are covered in Structural Appendix.

4.2 INTRODUCTION

This geotechnical documentation captures the concept design for construction of a new HSDRR system that would protect parts of Orange County and Jefferson County in the state of Texas.

The proposed system is a Hurricane and Storm Damage Risk Reduction system. The segments proposed are divided in three major areas (Orange, Beaumont and Jefferson) as shown and described in Figure 4-1 and Table 4-1 below. This document presents and describes the criteria, assumptions, procedures and results of the geotechnical procedures for the design of levees.

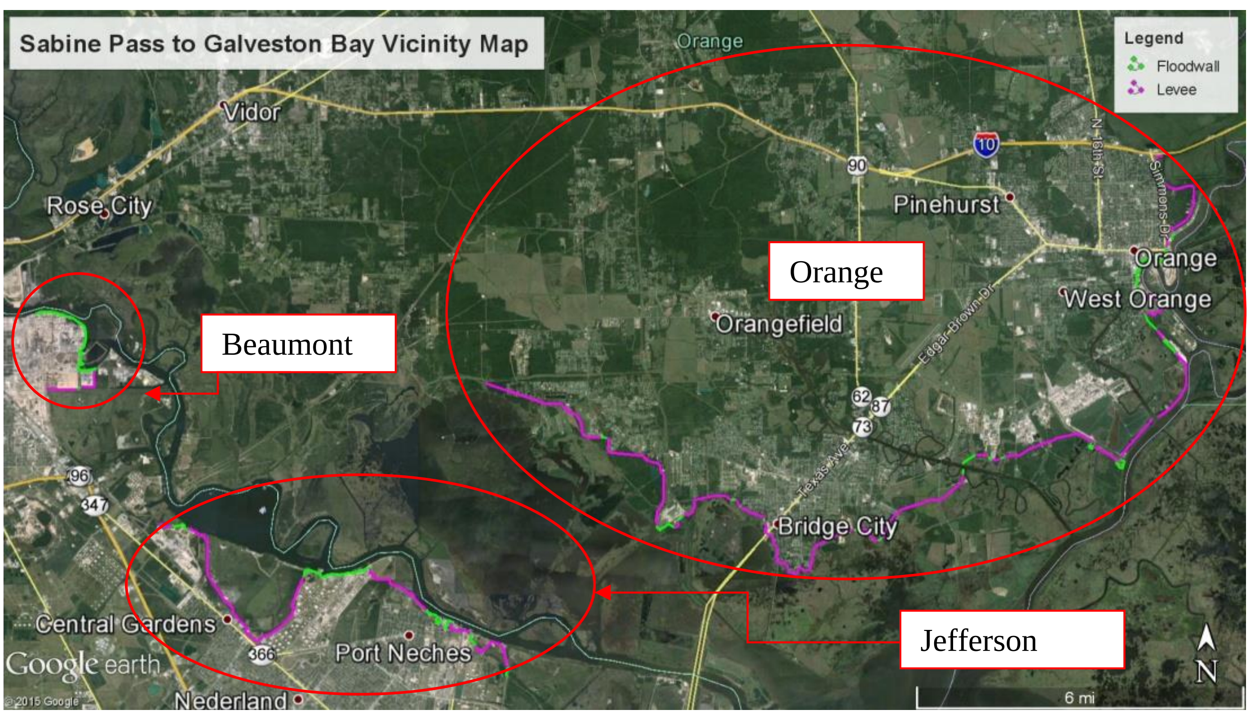

Figure 4-1: Sabine Pass to Galveston Bay Vicinity Map

Table 4-1: Approximate Lengths of Proposed Features per Area

Reach	Total Length (ft)	Length Of Features (ft)	
		Levee	Floodwall
Orange 3	143,400	113,571	29,829
Beaumont A	18,782	5,992	12,790
Jefferson Main	52,179	37,568	14,611

4.3 DESIGN CRITERIA

4.3.1 Selection of Design Criteria

Per Table B-1 of ER 1110-2-1806 [6], the proposed HSDRR system would be classified as a high hazard structure. Therefore, design efforts were required to conform with the most stringent applicable requirements. Several Engineering Manuals (EM) and other types of guidance published by USACE apply to the proposed structures. Main design criteria includes, but is not limited to:

- Hurricane and Storm Damage Risk Reduction System Design Guidelines, June 2012[7]
- EM 1110-2-1913, Design & Construction of Levees, April 2000[2]

Additional criteria applied are listed in Paragraph 4.4.

4.3.1 Historical Documentation and Input Data

Preliminary design included a search for historic, relevant, and useful documentation. Little to no relevant documentation was found for the proposed HSDRR system except for the "Flood Protection Planning Study Hurricane Flood Protection System Orange County, Texas", December 2012 which was prepared for Orange County and The Texas Water Development Board. No geotechnical investigations were performed for such study. Therefore, feasibility level design relies heavily on the geotechnical investigation performed in 2015.

4.3.2 Reference Documents

4.3.2.1 USACE Engineer Manuals

[1] EM 1110-2-1901 Seepage Analysis and Control for Dams, 30 April 1993

[2] EM 1110-2-1902 Slope Stability, 31 October 2003

[3] EM 1110-2-1913 Design & Construction of Levees, April 2000

4.3.2.2 USACE Engineer Technical Letters

[4] ETL 110-2-569 Design Guidance for Levee Underseepage, May 2005

4.3.2.3 USACE Engineering Regulations

[5] ER 1110-2-1150 Engineering and Design for Civil Works Projects, August 1999

[6] ER 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects, July 1995.

4.3.2.4 Other USACE Engineering Guidelines

[7] Hurricane and Storm Damage Risk Reduction System Design Guidelines – INTERIM, June 2012.

4.3.2.5 Drawings and Reports

[8] Flood Protection Planning Study Hurricane Flood Protection System, Orange County, Texas, December 2012.

4.3.3 Surficial Geology

Jefferson and Orange Counties are in three major land resource areas (MLRAs). About 50 percent of the area lies within the Gulf Coast Prairies MLRA. About 35 percent of the area is in the Gulf Coast Marsh MLRA. About 15 percent of the area lies within the Western Gulf Coast Flatwoods MLRA mostly in the northern part of Orange County. The Gulf Coast Prairie MLRA has mostly dark colored loamy and clayey soils that formed under prairie vegetation. The Gulf Coast Marsh is comprised of sandy, clayey, or loamy soils that are submerged for part of the time with saline or fresh water. The Western Gulf Coast Flatwoods MLRA has mostly light colored loamy and silty soils that formed under pine forest vegetation. The major land uses in the Gulf Coast Prairies include farming and ranching. The major land use for the Gulf Coast Marsh is wildlife. The major land use for the Western Gulf Coast Flatwoods is woodland.

4.3.4 Software

SEEP/W & SLOPE/W Version 8.12.3.7901

4.4 GEOTECHNICAL ANALYSIS

4.4.1 Foundation Conditions

Geotechnical investigations were performed in 2015 to document the general foundation conditions for the proposed features. Investigation consisted of 25 core borings and 46 cone penetration tests (CPT). General overview of the core boring plan and CPT plan are shown in

Figure 4-2 and 4-3, respectively. Investigation addressed portions of the system that have since been removed based on the risk assessment analysis performed at this stage. Draft results of these efforts were used in the performance of shear plots and subsequently in the demarcation of design reaches. The geotechnical investigation data and shear strength plots are not included with the appendix at this time, but can be made available upon request.

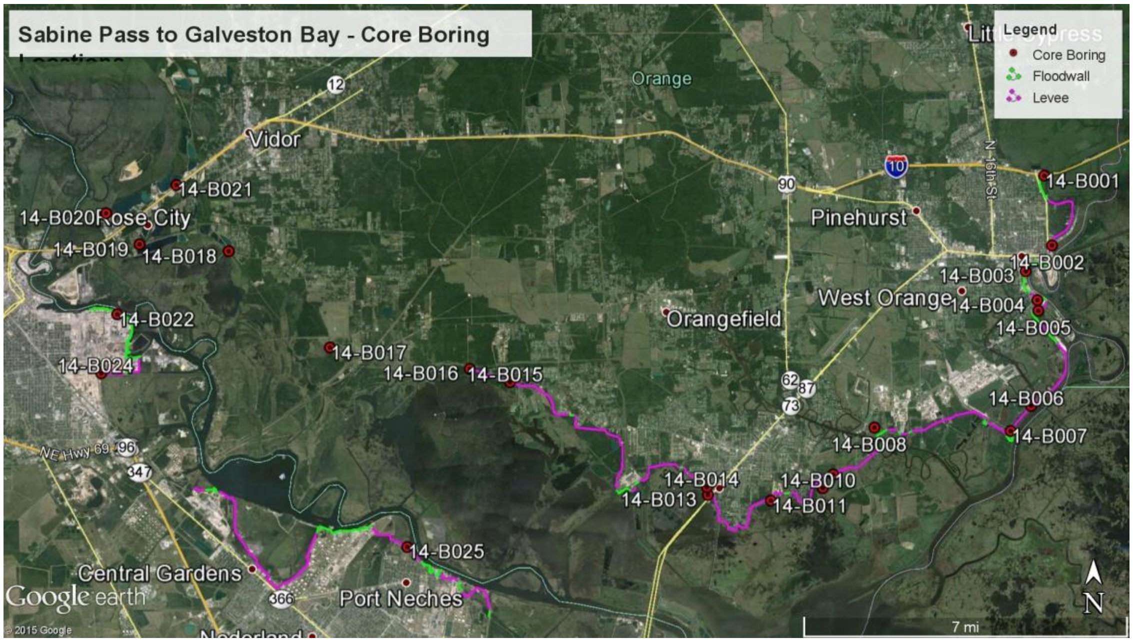

Figure 4-2: Core Boring Plan

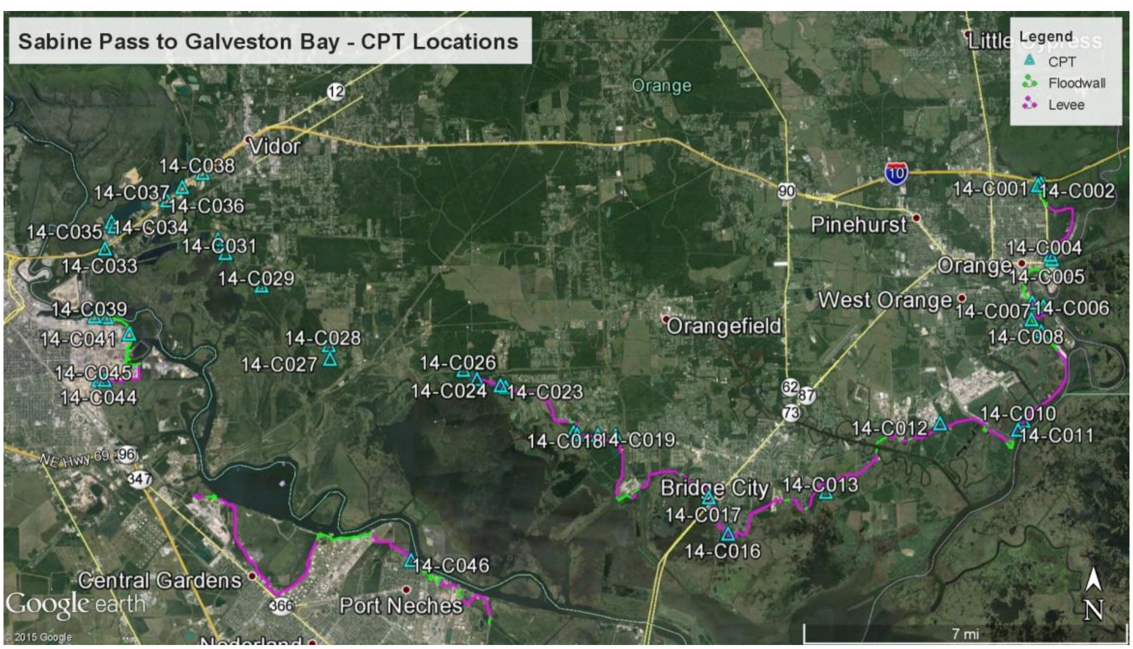

Figure 4-3: CPT Plan

4.4.2 Analysis Cross Section Selection and Soil Parameters

In order to come up with a representative design that would cover essential features of the project, controlling project feature geometry and soil configuration were identified along with representative sections. Three cross-sections were selected based on the shear strength developed with the information collected in the geotechnical investigations performed for the project throughout its alignment. Core boring and CPT were grouped based on proximity and their similarities of strata classification, configuration, and mechanical properties. Three cross-sections were used for the development of typical sections based on the performed analysis. Those cross sections are:

4.4.2.1 Orange Reach

Orange reach was developed within Orange 3 and shear strength plot IV, and is composed of Core Borings 14-B005, 14-B006 and 14-B007 along with CPT 14-C008, 14-C009, 14-C010 and 14-C011. The resulting plot exhibits the weakest soil configuration observed and was therefore identified as potentially producing unique features independent from other areas where the sub-surface conditions are better. Shear strength plot IV and its location can be seen in Figure 4-4, detailed plot is not included with this appendix, but can be obtained upon request.

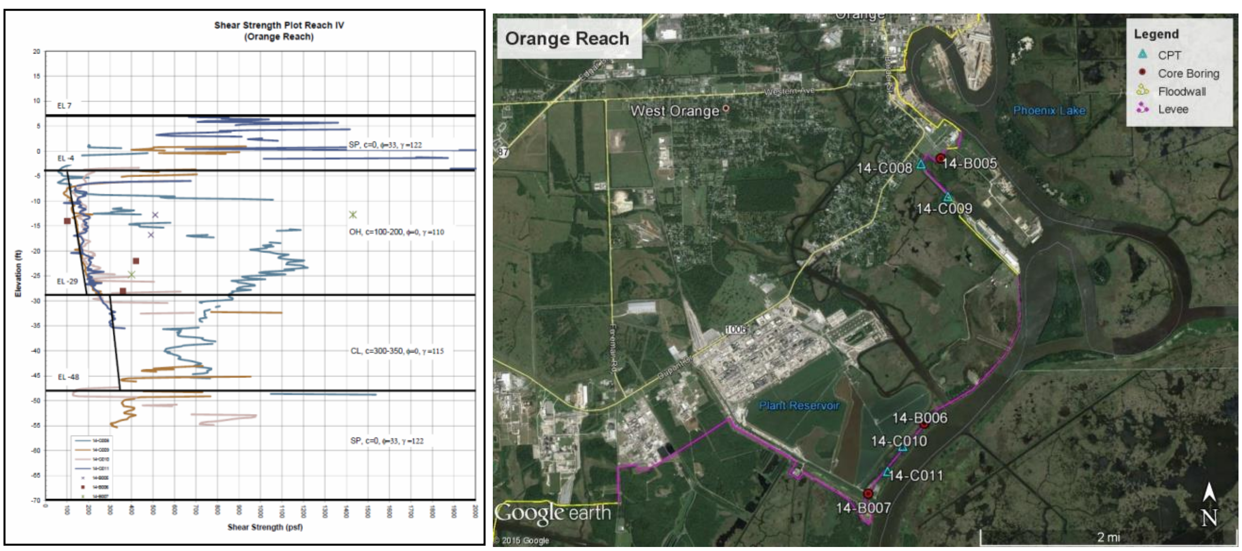

Figure 4-4: Orange Reach Represented by Shear Strength Plot IV (left) and its Location (right)

4.4.2.2 East Bridge City Reach

East Bridge City reach was developed within Orange 3 and shear strength plot V, and is comprised of borings 14-B008 and 14-B009 along with CPT 14-C012. The resulting plot exhibits stronger material strengths than the controlling section (Orange Reach) and was therefore identified for the development of a typical section where the soils were not as weak as other reaches. Shear strength plot V and its location can be seen in Figure 4-5, detailed plot is not included with this appendix, but can be obtained upon request.

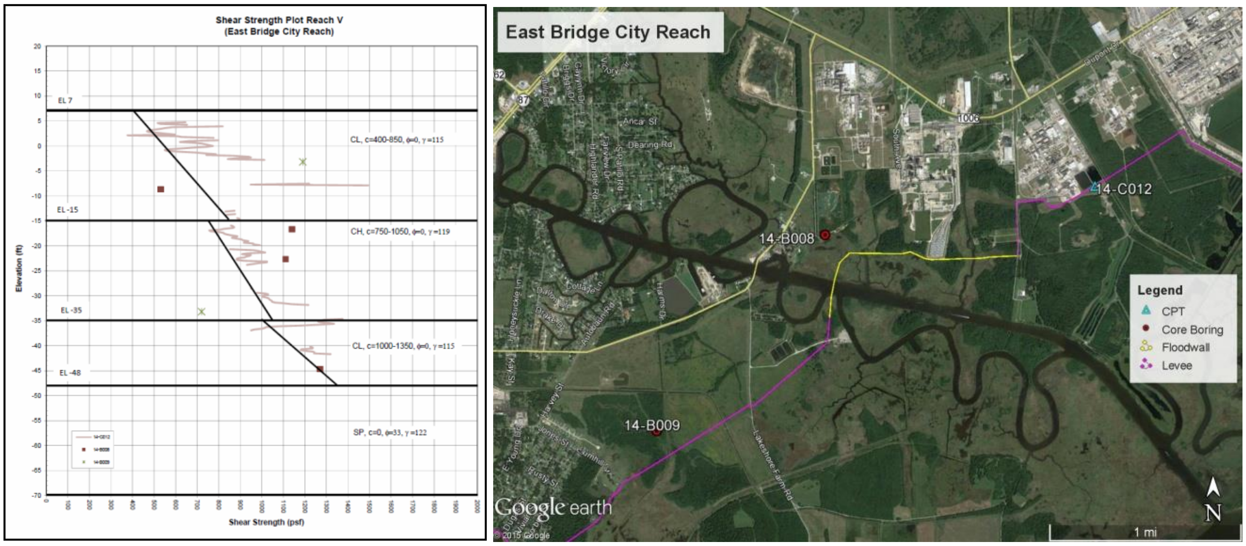

Figure 4-5: East Bridge City Reach Represented by Shear Strength Plot V (left) and its Location (right)

4.4.2.3 Jefferson Reach

Jefferson reach was identified within Jefferson Main and shear strength plot XIII, comprised of core boring 14-C025 and CPT 14-C046. Shear strength plot XIII and its location can be seen in Figure 4-6, detailed plot is not included with this appendix, but can be obtained upon request.

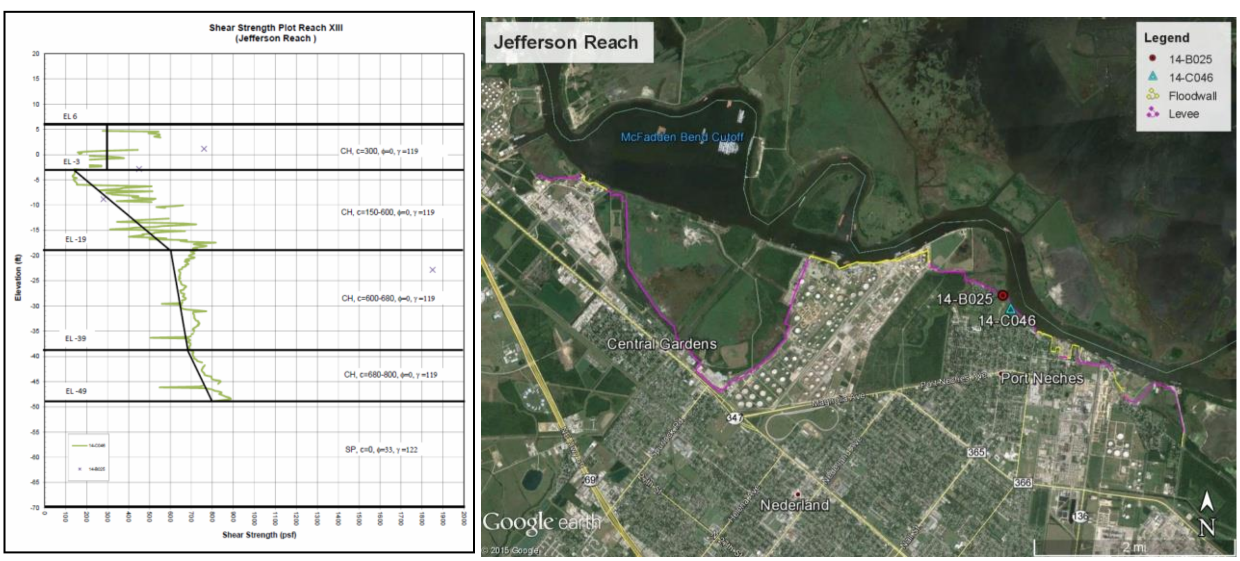

Figure 4-6: Jefferson Reach Represented by Shear Strength Plot XIII (left) and its Location (right)

4.4.3 Geotechnical Design Assumptions

Assumptions made throughout the performance of this study are:

- Steady-state conditions for all seepage analysis
- Proposed material on levee embankment is compacted impervious fill
- No seismic accelerations were considered, see Section 4.5.3 of this document

4.4.4 Material Properties

4.4.4.1 Saturated Unit Weights

- Water 62.4 pcf
- Impervious Embankment Fill 115.0 pcf
- Silt (ML) 117.0 pcf
- Fat Clay (CH) 119.0 pcf
- Lean Clay (CL) 115.0 pcf
- Organic Clay (OH) 110.0 pcf
- Sands (SP & SM) 122.0 pcf

4.4.4.2 Design Loads

The following load conditions were considered in all stability analyses unless stated otherwise.

4.4.4.3 Dead Load (D)

Dead load (D) includes the weight of superimposed water and embankment backfill. The unit weight of materials is listed in Paragraph 4.4.1.

4.4.4.4 Hydraulic Load (H)

Hydraulic load (H) includes hydrostatic pressure (Ps) and uplift pressure (U). These loads are developed from the hydraulic head produced from the design pool level listed in Table 4-2 for each load combination. Water elevations in Table 4-2 were developed with a 100-year return period at a 90 percent confidence as per HSDRR guideline and were used for all the design calculations described in this document.

Table 4-2: Hydrostatic Conditions

Case Condition	Hydraulic Condition per Reach and Loading Case (ft)		
	Orange	East Bridge City	Jefferson
End of Construction	-	-	-
Still Water Level (SWL)	12.70	14.96	14.67
Project Grade (WPG)	16.00	19.00	19.00

Water conditions described in Table 4-3 below is based on the results of the risk assessment analysis performed at this stage which identified 12 feet as the elevation that returned the most benefits. These elevations were only used in the recommendations section of this document (Paragraph 4.5).

Table 4-3: Hydrostatic Conditions Produced by Risk Assessment

Case Condition	Hydraulic Condition per Reach and Loading Case (ft)		
	Orange	East Bridge City	Jefferson
End of Construction	-	-	-
Still Water Level (SWL)	12	12	12
Project Grade (WPG)	16	16	16

Hydrostatic Pressure (Ps):

The hydrostatic pressure (Ps) is taken into account as a force acting perpendicular to the surface retaining water as in the following equation:

$$P_s = \gamma_w h$$

where: γ_w = unit weight of water
h = depth of water at calculation point

4.4.4.5 Earthquake Loads (EQ)

According to current USACE criteria, the project area is classified as a Zone 0 on Figure C-1 of ER 1110-2-1806, Earthquake Design and Evaluation for Civil Works Projects. [6] No earthquake loads need to be considered at a Zone 0 site. No seismic loads were taken into consideration as part of the preliminary design of levee embankment.

4.4.5 Seepage Analysis

Background, from EM 1110-2-1901, 30 Sep 86[1], Chapter 4-9 Paragraph B:

"The escape or exit gradient, i_e, is the rate of dissipation of head per unit of length in the area where seepage is exiting the porous media. For confined flow, the area of concern is usually along the uppermost flow line near the flow exit, e.g., at the downstream edge of a concrete or other impermeable structure. Escape gradients for flow through embankments may also be studied by choosing squares from the area of interest in the flow net (usually at or near the exit face and downstream toe) and calculating gradients. If the gradient is too great where seepage is exiting, soil particles may be removed from this area. This phenomenon, called flotation, can cause piping (the removal of soil particles by moving water) which can lead to undermining and loss of the structure. The gradient at which flotation of particles begins is termed the critical gradient, i_{cr}. Critical gradient is determined by the in-place unit weight of the soil and is the gradient at which upward drag forces on the soil particles equal the submerged weight of the soil particles."

For all seepage analysis performed for Sabine to Galveston exit gradients were obtained with the SEEP/W module of SLOPE/W, where modeling results values are displayed by selecting the area of interest. Exit gradient outputs were then used as input variables in the equation below. The quantitative values shown represent the factor of safety calculation for East Bridge City Reach for water to project grade (WPG).

$$\mathrm{FS}_{\mathrm{piping}} = \frac{i_{cr}}{i_e} = \frac{\gamma'/\gamma_w}{(dh/dl)} = \frac{(115-62.4)/62.4}{0.42} = \frac{0.842}{0.42} = 2.0$$

where:

dh, is head loss between each equipotential line.

dl, is the dimension of a flow net square or distance between equipotential lines.

Exit gradients were evaluated at, but not limited to, key feature locations such as land side slopes of the embankment and throughout all proposed stability berms slopes. The following table summarizes the seepage analyses results:

Table 4-4: Seepage Analysis Results

Reach	Exit Gradient		Factor Of Safety (*Required)	
	WPG	SWL	WPG (1.3*)	SWL (1.6*)
Orange	0.07	0.11	13.2	8.3
East Bridge City	0.42	0.33	2.0	2.5
Jefferson	0.37	0.32	2.4	2.8

4.4.6 Slope Stability Analysis

4.4.6.1 Slope Stability Design Criteria

The following factors of safety were considered in the stability analysis, using the Spencer method, according to Table 3.1 of the HSDRRSDG [7], based on criteria established by EM 1110-2-1902 Slope Stability [2]:

End of Construction	1.3
Design Hurricane (SWL)	1.5
Water at Project Grade (WPG)	1.4

4.4.6.2 Results of Stability Analysis

During preliminary design efforts geometric design of levee template was based on guidelines for stability and maintenance on EM 1110-2-1913 [3]. Maintaining a minimum of a 10-foot crown with slopes no steeper than 1H:3V. This geometry was then used with all the determined reaches (Paragraph 4.2). Modifications were made based on findings through the performed stability and seepage analysis. Such modifications are the inclusion of stability berms made of the same material as the proposed levees. The reaches that required stability berms in order to meet criteria are Orange Reach, 68 feet wide and the Jefferson Reach 42 feet wide. The results of the stability analyses are summarized in Table 4-5.

Table 4-5: Stability Analysis Results

Reach	Factors of Safety					
	Required			Computed		
	EoC	SWL	WPG	EoC	SWL	WPG
Orange	1.3	1.5	1.4	1.8	1.5	1.4
E. Bridge City				2.1	2.0	2.0
Jefferson				1.5	1.5	1.5

4.4.7 Settlement Analysis and Results

Settlement analysis was performed with preliminary data on several locations throughout the proposed alignment, results averaged in 4 feet of settlement. Overbuild of the same amount was considered on all design analysis.

4.5 RECOMMENDATIONS

Based on the geotechnical investigations and analysis performed at this feasibility stage, 10-foot crown along the 3H:1V slopes (Figure 4-7) were found to be adequate for the majority of the project where a levee is being proposed. However an area was identified as having weaker foundation configuration, for such area a stability berm is recommended. The suggested berm should span 3.2 miles of the project area, shown in Figure 4-8. Geometry details on such berm can be seen in Figure 4-9.

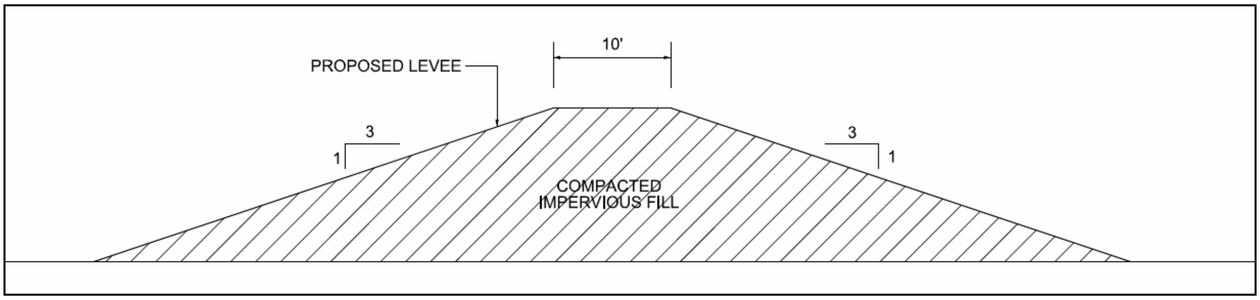

Figure 4-7: Typical Levee Section

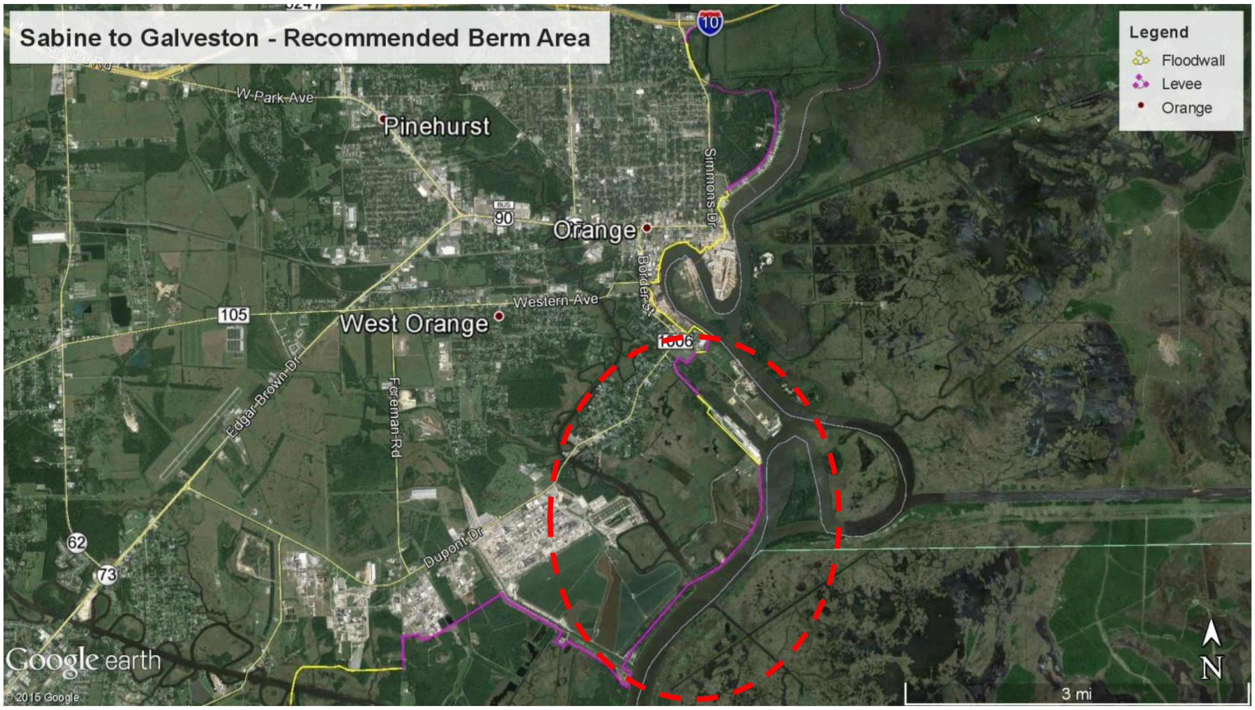

Figure 4-8: Recommended Berm Area in Orange 3, South City of Orange, TX

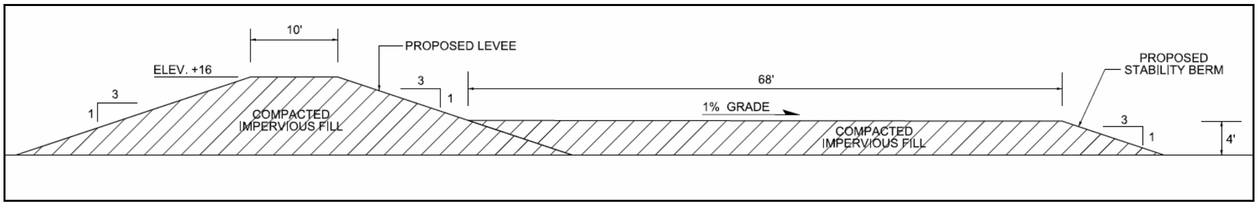

Figure 4-9: Typical Orange 3 Stability Berm Section

5 ENVIRONMENTAL IMPACTS AND ENGINEERING

5.1 USE OF ENVIRONMENTALLY RENEWABLE MATERIALS

Construction of the projects involves earthwork to build and raise levee embankments, and to excavate the ground for the floodwall footings; the placement of reinforced concrete for the floodwalls; installation of sheet pile cutoff walls; the building of gate structure and pump station housing; installation of steel gates; riprap placement; installation of resiliency elements; and pump installations. Soil excavated incidental to the levee and floodwall construction that is satisfactory for use as structural fill material will be used for this purpose. Beyond that, the fill material will be imported from local commercial sources. Stripped topsoil will be conserved and used to plate the levee embankment. Although there is little opportunity to incorporate renewable materials into the project due to the nature of its construction, much of the project will or can be built using materials that are reusable or recyclable, such as concrete, steel, and material made of polymers (e.g., high-performance turf reinforcement mattress (HPTRM), fiber-reinforced polymer (FRP) sheet piling, and high-density plastic drain pipe).

5.2 DESIGN OF POSITIVE ENVIRONMENTAL ATTRIBUTES INTO THE PROJECT

Incorporating positive environmental attributes into the project feature designs was a primary consideration in deciding the proposed alignments for the HFPPs in Orange and Jefferson Counties. These alignments were laid out, and the types of structures (i.e., levee or floodwall) tentatively decided taking into account the direct impacts the construction easements would have on existing forests and wetlands and indirect impacts the systems, as configured, would have on affected floodplains. Consequently, positive environmental attributes are largely built into the project designs by way of deliberately circumventing habitat areas and reducing the project footprints, which avoids adverse environmental impacts altogether or otherwise minimizes them (ER 1110-2-1150, Paragraph 13.6.8).

For the proposed HFPPs and existing Freeport and Port Arthur HFPPs to be modified, resiliency features are to be built in to withstand erosion forces from wave run-up and overtopping. To a considerable extent, these resiliency features will be constructed using high performance turf reinforcement mats wherever deemed acceptable, which is a more environmentally friendly, cost-effective alternative to hard armor systems. The levee slopes will be vegetated with native grass. Additionally, drainage structures will be strategically located and appropriately sized to maintain freshwater flow balances within affected habitats.

Because much of the HFPP construction will be in areas that are already developed, and avoids environmental habitats, there is little opportunity to design environmental attributes into the recommended plans that would enhance the existing project areas. Given the limited opportunities, the principal focus of the engineering effort related to designing-in environmental attributes was to avoid impacting critical habitats in the first place.

5.3 INCLUSION OF ENVIRONMENTALLY BENEFICIAL OPERATIONS AND MANAGEMENT FOR THE PROJECT

There is little opportunity or means of operating and managing the projects in such a way that will benefit the environment. The grassed levees will have to be regularly mowed and the floodwalls and gate structures routinely maintained. Occasional maintenance and repairs of the roadway on the levee crown will also be required. Other than that, the levees and floodwalls will remain in a passive state. Operationally, the functionality of the affected floodplains will not be compromised. Openings at closure points will be kept open and drainage structures allowed to flow freely when the system is not activated.

5.4 BENEFICIAL USES OF SPOIL OR OTHER PROJECT REFUSE DURING CONSTRUCTION AND OPERATION

While it is assumed that all fill material necessary to build the levees comprising the proposed Orange and Jefferson County HFPPs will be brought in from local commercial sources, it is anticipated that much of the material from required excavations can be used for some purpose in the construction. No material from the construction is expected to be reused for beneficial purposes. Spoil probably will be disposed of in placement areas that, as of yet, have not been identified. The plan for the disposal of this material however will avoid or minimize additional adverse impacts to the maximum extent practicable. The projects should not generate much refuse during their operation, but refuse and debris will collect at the gated drainage structures, mostly during flooding events. This debris will be removed and conventionally disposed of in approved landfills.

5.5 ENERGY SAVINGS FEATURES OF THE DESIGN

Other than the pump stations and navigation gate facilities, there are no features available where energy savings can be realized due to the nature of the CSRM projects. The projects are horizontal construction projects that consist of earthen and concrete barrier structures. Nevertheless, opportunities to economize maintenance of the levees and floodwalls will be explored when designing the project features.

5.6 MAINTENANCE OF ECOLOGICAL CONTINUITY WITH THE SURROUNDING AREA AND WITHIN THE REGION

The landscape/horizon of the Orange and Jefferson County project sites will be altered by the new levees and floodwalls. These levees and floodwalls will result in a prominent line of earthen embankment and concrete wall along ground surfaces that are relatively flat. Forest areas within the HFPP corridors will be cleared. Some existing drainage systems will be altered. To the extent practicable however, the project designs preserve existing forest and wetland habitats. Where environmental impact is unavoidable, the designs minimize the project footprints. The levee slopes will be turfed with native grass. Resiliency elements, to the maximum extent possible, will be designed to be "green" to blend in with the natural setting. Over time, the HFPPs will have less of an effect on the ecology of the area as the ecology adapts to the relatively modest changes in the environment.

Terrestrial wildlife would still be able to access habitats divided or cut off by earthen levee segments, as it does now across the Port Arthur HFPP levees. Floodwall segments would generally be located in developed areas and limited in length; wildlife would be able to utilize nearby levee segments for access as needed. Fisheries access would be maintained at FWOP levels. The recommended projects are not expected to impact migratory birds or their habitat within the project areas.

The recommended projects have been determined to be the best at satisfying the purpose and need of the projects to provide increased CSRM in a cost-effective manner with minimized impacts to the environment. They best meet the balance of providing a higher level of protection against coastal storm surge, minimizing environmental impacts, and being compatible with the surrounding environment.

5.7 CONSIDERATION OF INDIRECT ENVIRONMENTAL COSTS AND BENEFITS

Indirect environmental costs and benefits were considered in the preliminary layouts of the proposed HFPPs in Orange and Jefferson Counties. Their impacts on the environment are discussed in the DIFR-EIS. To the extent practicable, the proposed alignments were purposely routed to avoid environmental habitats. Where such avoidance was not possible, the project footprints were maximally reduced in several places by assuming more expensive floodwall segments where levees, which would have had a wider footprint, would have sufficed. Even with the deliberate efforts to avoid impacting forested areas and wetlands, the indirect environmental costs are considerable given the scale of the projects.

Section 7 of the DIFR-EIS describes each project's potential impact on soils, water quality, air quality, aquatic and terrestrial habitats, wildlife, threatened and endangered species, HTRW, and social and cultural resources. Essentially all of the environmental impacts are associated with the proposed Orange and Jefferson County HFPPs. The construction of these systems will negatively affect, indirectly, 2,264 acres of fisheries access to extensive marshes within the lower Cow and Adams Bayous floodplains (under the intermediate sea level rise scenario). In total, approximately 153 acres of forested wetland and 2,412 acres of coastal marsh would be impacted. Mitigation would be necessary to compensate for the consequential loss of these wetlands. Improvements to the existing Port Arthur and Freeport HFPPs are not expected to significantly disrupt the environment. However, potential exists for impacts due to buried hazardous and toxic materials during construction of improvements. The proposed projects do not impact Federally listed threatened or endangered species or their designated critical habitat.

5.8 INTEGRATION OF ENVIRONMENTAL SENSITIVITY

Project features will be designed with environmental sensitivity in mind.

5.9 CONSIDERATION OF ENVIRONMENTAL PROBLEMS ON SIMILAR PROJECTS WITH RESPECT TO THE ENVIRONMENTAL REVIEW GUIDE FOR OPERATIONS (ERGO)

Environmental issues/concerns on similar projects and lessons learned will be considered during the design of the project features to make sure that systems and facilities comply with applicable environmental laws and regulations and will comply with these laws and regulations during operations. Environmental concerns will be appropriately addressed/mitigated for in the project designs. Construction of the projects will not proceed until the Non-Federal Sponsor has provided a clean corridor free of any HTRW contamination.

5.10 INCORPORATION OF ENVIRONMENTAL COMPLIANCE MEASURES

As the projects comprising the TSPs are further developed and designed, environmental compliance measures will be incorporated into the feature designs.

5.11 ENVIRONMENTAL IMPACTS

Potential impacts to terrestrial and aquatic habitat and the need for environmental mitigation associated with the proposed projects were evaluated and coordinated with appropriate resource agencies. The initial assumption for the impact analysis and alternatives comparison was that all habitat within the construction easement would be destroyed or otherwise adversely impacted. Construction easements for the proposed HFPPs in Orange and Jefferson Counties were intentionally set to be considerably wide to account for possible modest shifting of the HFPP alignment and variation of the levee template during PED, as well as to be conservative in the environmental impact estimate. Essentially all the habitat within the assumed construction easements, most of which will also comprise permanent maintenance easements, will either be destroyed or disturbed during construction (where there will be significant clearing, grubbing, stripping, and equipment traffic). The disturbed areas, however, will be planted with native grasses following construction. Forest clearing would be conducted in such a manner as to avoid disturbing nesting bald eagles and migratory birds. Terrestrial wildlife would be able to cross earthen levee segments to access remaining habitat on either side, as it does now across the levees of the Port Arthur HFP. Floodwall segments would generally be located in developed areas and limited in length; wildlife would be able to utilize nearby levee segments for access as needed. Fisheries access would be maintained at FWOP levels.

The impacts of the proposed HFPPs to terrestrial habitats will come from new work construction in building earthen levees and concrete floodwalls having considerable footprints, particularly those of the levee, with the possibility that some levee sections will also include underseepage berms and landside-toe embankment sand drains. In some instances, these impacts will extend linearly several hundred feet, upwards to a few thousand feet, through forested areas. For the most part however, forests and wetlands are being circumvented and existing cleared corridors taken advantage of. Impacts will also come from the associated drainage, closure gate, and navigation gate structures that will have to be built. For the existing HFPPs, the impacts will result from lateral expansion of the levee footprint from the levee raise. The impacts resulting from levee raising will be limited to the immediate area flanking the levee, which is developed.

6 CIVIL DESIGN

This section presents the general civil design considerations that were made for the projects comprising the TSP.

6.1 SITE SELECTION AND PROJECT DEVELOPMENT

The general project areas were identified through the Plan Formulation process, discussed in Appendix B of the DIFR-EIS, where structural measures were developed to reduce the risk of flood damages from coastal storm surges and to provide environmental benefits. These structural measures included the construction of new regional HFPPs in Orange and Jefferson Counties and improvements to, and the reconstruction of existing HFPP elements comprising the existing Port Arthur and Freeport systems. For the Freeport HFPP, the structural measures also included adding a new navigation gate structure near the entrance of the Dow Barge Canal to serve as a flood barrier. After screening the measures, alternative structural plans were formulated for the Sabine, Galveston, and Brazoria Regions comprising the six-county study area. The initial arrays of structural plans for the Sabine and Brazoria Regions are listed in Tables 5-1 and 5-3, respectively, of Appendix B and in Table 1-1 of this appendix. Of these structural plans, Alternatives Nos. S5 (Sabine Inland Barrier CSRM Focus (Neches Gate/Sabine Levees/HFP)) and B2 (Brazoria Coastal Barrier CSRM Focus (revised)) were ultimately carried forward for further detailed analyses and development as the TSPs, where the plans were economically and environmentally optimized. The Neches River gate component of Alternative S5 was ultimately dropped from further consideration due to the very high difference in first cost between it and the construction of additional levees along southwest Orange County and the west bank of the Neches River in Jefferson County that would provide a comparable level of protection. Appendix B discusses the gate structure and the rationale for eliminating it from the alternative.

6.1.1 HFPP Alignments

6.1.1.1 Orange-Jefferson County Alignments

Preliminary alignments for the proposed Orange-Jefferson County HFPP were initially laid out referencing preliminary layouts that a joint group of engineering firms had previously investigated and recommended for Orange County and eastern Jefferson County as a basis. This group conducted a flood-protection planning study for a countywide HFPP in Orange County, which is documented in the Orange County Report. The alignments recommended by the report are called the "Neches Crossing and Sabine River Alignment" and the "Industries Alignment". These alignments were chosen based on their ability to meet the requirements of providing countywide protection and protection to the major industrial area within Orange County.

Factoring into the decision on the Neches Crossing and Sabine River Alignment were assessments of the valuations of the properties to be protected and potential flood damage losses that would be prevented. In addition, this alignment would protect industrial areas and the majority of the land area within Orange and Jefferson Counties.

The TSP alignments draw on the "Protection System on East & West Bank of Neches River (SR + NR + NRW)" alternative alignments presented in the Orange County Report (see Figure 6-1 on the next page). To a considerable degree therefore, the alignments comprising the TSP generally coincide with the earlier recommended alignments. With these well-defined alignments as a starting point, a parametric approach was used to modify and refine the alignments to what became the TSP alignments. Site assessments were made based on existing information and aerial photos to selectively decide the property and environmental assets that would be worth protecting, and that would be economically viable of protecting. These assessments were augmented by site visits and limited site investigations. When routing the HFPP alignments, consideration was given to the environmental impacts to bottomland hardwood and wetland areas. This consideration significantly dictated the course of the alignment. Other principal considerations were taken into account as well. Considerations were made to avoid cutting through or getting too close to existing buildings and infrastructure, crossing low areas constantly under water, crossing areas with foundation conditions that are likely to be poor, water bodies, and crossing or paralleling too close to major pipeline corridors. All this was being done within the framework of the H&H modeling results in which the potential economic losses from flood damages corresponding to assumed SWEs were evaluated against the costs of providing the respective levels of protection.

After refining the HFPP alignments, the Orange County system was broken down into three (3) separable reaches, rationally based on existing geographical features and the areas that would be protected to maximize the economic benefits. These reaches (each of which was evaluated as an alternative in deriving the TSP) are called "Orange 1," "Orange 2," and "Orange 3." They are described in Table 1-1 and depicted on Plate C-1. The reaches were evaluated as discrete projects independent of the other reaches. They were not amalgamated in any combination of increments and evaluated in that manner. The "Jefferson Main" reach and Beaumont "A", "B", and "C" reaches were also evaluated as distinct alternatives. Each of these reaches is also described in Table 1-1 and shown on Plate C-1 as well. The alternative projects that produced net excess economic benefits became the Orange-Jefferson CSRM component of the TSP.

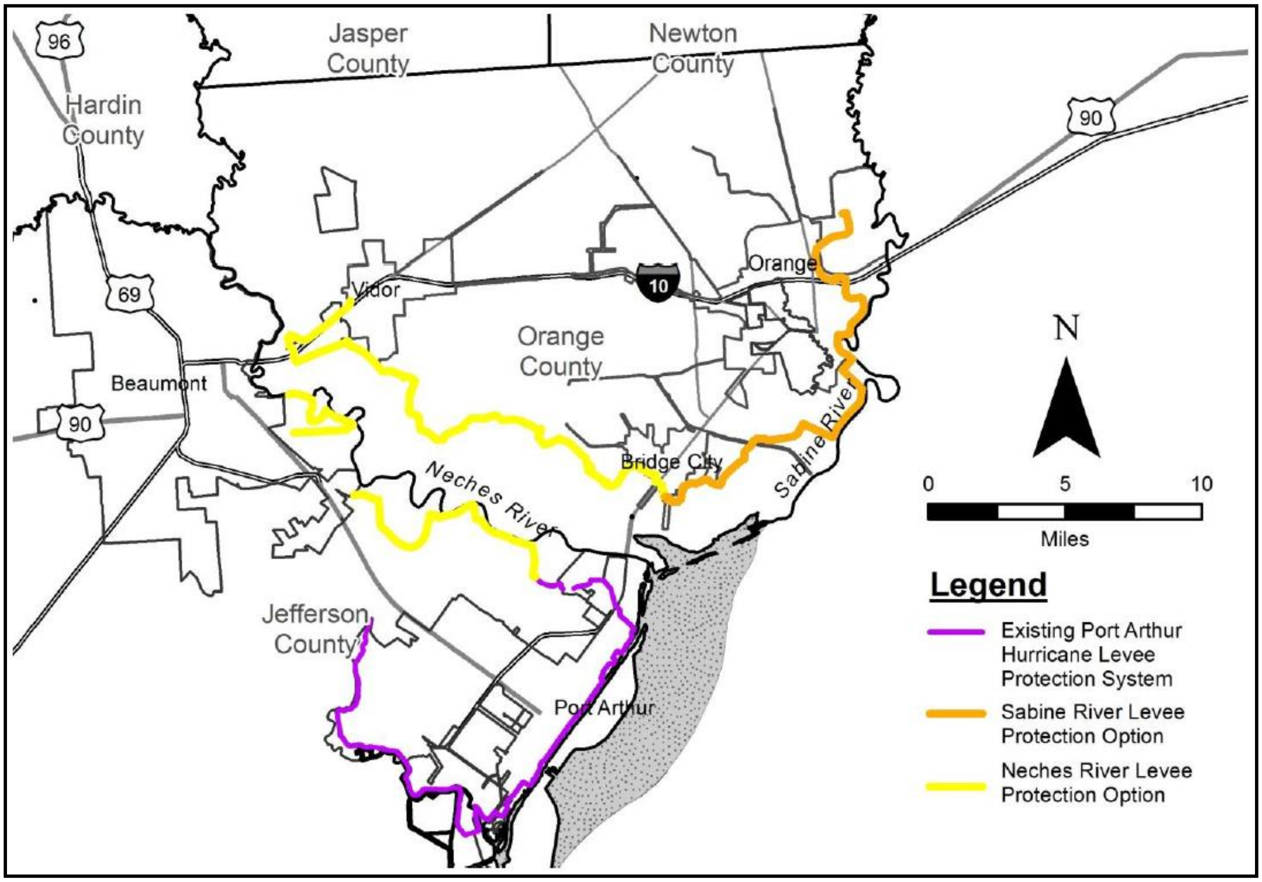

Figure 6-1: Levee Alignment to Provide Full County Protection-Levee on East and West Bank of Neches River (from Orange County Report)

6.1.1.2 Port Arthur HFPP Alignment

The Port Arthur HFPP is an established flood protection system. No realignments are proposed. The proposed HFPP along the south bank of the Neches River, that is to provide a greater level of flood protection to eastern Jefferson County, will tie into the Port Arthur system, which greatly extends the line of protection along the river.

6.1.1.3 Freeport HFPP Alignment

The Freeport HFPP is an established flood protection system. No realignments are proposed. Notably, however, that part of the system flanking the Dow Barge Canal, which is extensive (about 13 miles long), will effectively be rendered obsolete, as it will be cut off by the proposed navigation gate structure (with pump station).

6.1.2 Site Development

6.1.2.1 Orange-Jefferson County HFPP

Much of the areas traversed by the new levee system are highly or moderately developed, represented by industrial, residential, and commercial properties. The intent of a flood protection system is to reduce flood-damage risk to vulnerable populations and valuable assets, while at the same time minimize the environmental impacts; therefore, it is logical that levees and floodwalls would be constructed within and near sizably developed areas. The proposed HFPP alignments were appropriately routed with this in mind. All of the Jefferson Main reach in fact flanks or crosses industrial areas. Within developed areas, only a modest amount of clearing is anticipated. That said, several single-family homes, industrial and commercial buildings, structures, utilities (excluding pipelines), and pavements will have to be removed, relocated, or modified to provide necessary clearances and to construct the project. The number of affected structures was not counted for the TSP cost estimate. The default assumption however was that where a grouped number of structures or utilities would be adversely impacted by the construction of a levee, because of the embankment's large footprint, the more expensive floodwall structure, having a comparatively small footprint, would be built instead where reasonable for accommodation (assuming avoidance was not otherwise practical). This assumption was also made for situations in which there would be considerable impacts on sensitive environmental habitats. Assuming the more expensive floodwall structure option takes into consideration public acceptance and the political ramifications of the project where local public support for it is crucial. As the NED Plans are refined for the ADM, the number of structures and utilities impacted can be better assessed.

The proposed Orange County reach alignment will cut through several forested areas that will require a lot of clearing and grubbing within the entire area of the temporary construction easements. In deriving the TSP cost and NED Plan costs, the acreage of forests within the preliminary construction easements was not estimated. Instead, a uniform amount of clearing, grubbing, and stripping was assumed per lineal foot of levee/floodwall construction and parametric cost estimates applied. The assumption was made that all cleared materials will be hauled to and disposed of at offsite locations.

After clearing and grubbing the area and appropriately addressing structures within the temporary construction easements, the foundation areas within the levee embankment footprint, and 5 feet beyond the toes, will be stripped and generally excavated to a depth of one foot. The subgrade surface will then be proof-rolled and scarified afterwards before receiving fill. For the floodwall footing, the area within its footprint will be stripped and excavated to the required footing depth.

6.1.2.2 Port Arthur and Vicinity HFPP

The improvements proposed for the existing system are to provide scour protection along the backside (i.e., protected side) of three (3) discrete floodwall sections and a closure structure on the Sabine Lake side of the system and modestly raise the levees in these areas. The subject areas are considered soft points in the system vulnerable to possible structural failure (in the technical sense) and being breached from wave overtopping. Scour protection with reinforced-concrete pads is proposed along the 8- to 10-foot I-wall at the very south corner of the system; the I-walls near the Valero Refinery plant and tank farm, both at the south corner; and along the closure structure near the easternmost corner of the system. The levees will be raised by adding a fill lift atop the crown and interior-side slope. The completed improvements will be entirely within the existing perpetual easement. Where needed to accommodate construction activities however, additional temporary work area easements will be acquired. Because the existing system already is a developed project that is routinely maintained, no significant additional site development is anticipated to construct the improvements. The assumed construction operations will involve the stripping of surfaces to receive pavement and fill, modest excavations, removing road pavement (on the levee crown), placing concrete, placing select fill, constructing paved roadway, and turfing. These operations will be conducted in a developed environment.

6.1.2.3 Freeport and Vicinity HFPP

Four (4) of the six (6) improvements proposed for the existing system are modest levee raises of one foot. These raises will be done at potential failure points of the system located along the Old River Levee around the Dow "thumb," at the Tide Gate I-wall, along the East Storm Levee, and along the west bank of Oyster Creek. The Tide Gate I-wall will also have to be reconstructed to raise it a foot higher, as will existing floodwall along the Freeport dock. The TSP for the Freeport CSRM component also includes the construction of a new navigation structure near the entrance of the Dow Barge Canal. The levees will be raised by adding a fill lift atop the interior-side slope. This will be done entirely within existing rights-of-way. Where needed to accommodate construction activities however, temporary work area easements beyond the perpetual easement will be acquired. Except for the proposed navigation gate structure, little additional site development will be necessary to construct the improvements because the project is already a developed project that is maintained.

Construction of the navigation gate structure will require the construction of a cofferdam and provision for allowing the continued passage of barge traffic. Consequently, a considerable amount of logistics coordination and accommodation will go into the construction of this feature. Selecting the site for the structure will require the deliberate assessment of possible locations that would be practical from construction, functional integrity, and operational perspectives given the constraints of available land area and maintaining navigation on the barge canal, and to a lesser extent, the characteristics of existing foundation soils. Significant site development will be required because the structure will be constructed in the dry. A possible site location will be decided, and the construction scheme for a conceptual gate structure developed, after the ADM. The gate structure will be designed at the time the plans and specifications for this structure are prepared.

6.2 REAL ESTATE

6.2.1 General

For a feasibility study, USACE determines real estate requirements and associated cost based on the selected plan footprint and a gross appraisal. More exact real estate requirements are developed during the design phase following completion of the feasibility study.

According to Federal law, the non-Federal sponsor is responsible, within USACE guidance, to undertake (pay for) the required real estate actions. Real estate actions and acquisition required for project construction will begin prior to award of the individual construction contracts. The proposed Orange-Jefferson County HFPP will require a lot of pipeline and utility relocations, accommodation of existing roadways, and the removal of existing structures. USACE evaluates the compensable (or non-compensable) nature of the various utility relocations in conjunction with the sponsor. It then assigns relocation costs when the utility ownership and real estate rights information is adequate for a compensability determination. Utility relocation design details are developed during the design phase. For the alternative comparison analyses and determining the TSP projects, the associated project costs of pipeline and utility relocations and other infrastructure modifications were not considered.

6.2.2 Orange-Jefferson County HFPP

In developing the preliminary costs for the alternative plans for the separable project elements in Orange and Jefferson Counties, temporary construction easements and perpetual easements of uniform width were assumed for the new levees and floodwalls respectively. For the levees, a conservative 180-foot-wide temporary construction easement and 200-foot-wide perpetual easement were assumed. The temporary construction easement assumes minimum 30-foot-wide work areas along each side of the levee embankment as measured from its projected slope toe. The perpetual easement assumes 40-foot widths along each side of the levee for operations, maintenance, and monitoring. For the floodwalls, which have a smaller footprint than the levees (except for those 1 foot to 8 inches high or less), a uniform 60-foot-wide easement was assumed for both construction and permanent use.

The uniform 200- and 60-foot-wide construction easements assumed for the levee embankment and floodwall construction, respectively, are sufficient to construct the project. For most of the project though, particularly with regard to the levee construction, these widths are at least moderately conservative. That is because the levee and floodwall heights assumed in deriving the TSP easements were not related to the existing ground surface. They were instead referenced to an elevation of 0.0 foot. Taking the varying ground surface elevations into consideration, the final-design structure templates implemented for most of the project will be the same as or smaller than those assumed, even when adding structure height to account for future sea-level rise, wave run-up, freeboard, and foundation settlement and consolidation (overbuilding of the levee embankment). There will be instances too along the alignments where narrower-width work areas are necessary or preferred to minimize impacts on existing infrastructure, facilities, buildings, and environments or because of physical hindrances (e.g., bodies of water, soil conditions, limited construction easement). Conceivably, there could be instances where the easements would need to be made wider, although this is not expected. As the project features are further developed and refined relative to the existing terrain and physical features, the construction easements will be appropriately tailored along the alignment.

For the navigation gate structures proposed to be built across Adams and Cow Bayous, temporary construction easements and perpetual easements will need to be acquired.

It was also assumed that seven (7) temporary staging areas will be needed to construct the Orange 3 reach, three (3) will be needed to construct the Jefferson Main reach, and one staging area will be needed for the Beaumont “A” reach. All the staging areas were assumed to be 2.0 acres in size except for the Beaumont “A” staging area which was assumed at 3.0 acres.

6.2.3 Port Arthur and Vicinity HFPP

Except for possibly the closure structure, most, if not all, of the construction activities to construct the proposed improvements can be performed entirely within the existing rights-of-way. This will ultimately depend however on how high the levees are incrementally raised to accommodate wave run-up, future sea-level rise, and the minimum freeboard requirement. In developing the costs of the alternatives, it was conservatively assumed that 20-foot-wide temporary work area easements would be acquired along the interior-side boundary of the perpetual easement. Five (5) 2.0-acre temporary staging areas were also assumed would be needed for construction.

6.2.4 Freeport and Vicinity HFPP

Except for the navigation gate structure, most, if not all, of the construction activities to construct the proposed improvements can be performed entirely within the existing rights-of-way, depending on how high the levees are incrementally raised to accommodate wave run-up, future sea-level rise, and the minimum freeboard requirement. In developing the costs of the alternatives, it was conservatively assumed that a 20-foot-wide temporary work area easement would be acquired along one side of the existing perpetual easement. Ten (10) 2.0-acre temporary staging areas were also assumed would be needed for construction. For the gate structure with navigable opening, temporary construction easements and a perpetual easement will need to be acquired.

6.3 RELOCATIONS

6.3.1 Pipelines

Due to the prevalence of the petrochemical industry within the study areas, the projects cross or parallel, or will cross or parallel numerous pipelines carrying various petroleum products. The proposed new systems also cross several other utility conduits and storm drains as well. All such conduits are generically referred to in this report as “pipelines”. Plates C-2 through C-6 give the reader an appreciation for the number of pipelines crossed (most of which are multiple pipelines within closely spaced corridors or bundled pipelines).

In most instances, existing pipelines that will cross beneath or through a planned flood protection system, or be close to it, require relocation. This is because most pipelines are buried at a relatively shallow depth when surcharge loading is not anticipated and leaving them in place could cause serious damage to and structural compromise of the levees. The structural integrity of the pipelines can also be adversely affected when putting a large surcharge load over them, as will access to the pipelines. The pipe strength may not be enough to withstand the added loading and pipe joints may be unable to accommodate movements resulting from foundation settlement. The foundation design for the floodwalls may also preclude allowing pipelines to remain in place. As a general rule, pipelines beneath flood protection levees and floodwalls should be avoided altogether, particularly in the case of pressure lines.

When developing the projects to a greater level of design detail/confidence, the necessity for pipeline relocations will be re-evaluated. It may be that some pipelines can be allowed to remain in place within the levee foundation. In assessing this, the following principal items will be considered:

- levee height
- duration of high water stages against the levee
- susceptibility to piping and settlement of levee and foundation soils
- type of pipeline (low- or high-pressure line, or gravity drainage line)
- depth of the pipeline
- feasibility of providing closure in event of ruptured pressure lines, or in the event of failure of flap valves in gravity lines during high water
- ease and frequency of required maintenance and access
- cost of acceptable alternative systems
- possible consequences of piping or failure of the pipe

Since all but a few of the pipelines crossed by the proposed new levee/floodwall systems carry petroleum products and are buried at a relatively shallow depth, and given that their present condition and strength are unknown, the presumption will be that virtually all these pipelines will have to be relocated. Supporting this premise is that significant settlement and consolidation of the compressible foundation from the added levee surcharge load is expected will take place, which undoubtedly will greatly disturb/stress the pipelines embedded within it. The bottom of the concrete footings of the floodwalls will also be set a few feet below the existing grade and in some instances, batter piles may have to be driven for the foundation. Leaving the pipelines in place only increases the risk that either the structures composing the HFPP or the pipeline will eventually be structurally compromised.

6.3.1.1 Pipelines Crossing Orange-Jefferson County HFPPs

Relocations or modifications for the pipelines and utilities crossing the proposed Orange-Jefferson County HFPP alignments or near the alignments, and their associated costs, will be extensive. Essentially all the pipelines, if not all, will require relocation given their shallow burial depths and likely structural inadequacy to handle the greater overburden load, and because they will effectively serve as seepage conduits. Even under short-duration hydrostatic loading, seepage is a concern that needs to be examined. (Current requirement is that a steady-state seepage condition must be assumed for flood-damage reduction structures.)

Information on the pipelines crossing the Orange-Jefferson County alignments was obtained from an oil and gas GIS database maintained by the TRRC. This information included the pipeline's approximate location and orientation by coordinates, system and subsystem names, ownership, operator, diameter, product carried, and permit. However, it did not provide the pipeline depth. Because only a nominal amount of the Orange and Jefferson County project areas is within USACE's regulatory domain, no information on pipeline depths was immediately available that might have been included in as-installed permit records. There was no other expedient vehicle by which the pipeline depths could be readily assessed. Most oil and gas pipelines though are typically buried at a depth of 3 to 6 feet, as reported by the industry.

6.3.1.2 Pipelines crossing Port Arthur and Vicinity and Freeport and Vicinity HFPPs

While a number of pipelines cross the Port Arthur and Freeport systems, no pipeline and utility relocations or modifications are anticipated as being necessary except for one pipeline at Freeport. Therefore, other than for the one pipeline, no relocations are recommended at this time for the pipelines that cross the Port Arthur and Freeport levee reaches where 1-foot levee raisings are proposed. Modest raising of the levees is not expected to have an appreciable effect on the pipelines. This position could change however as the project is further developed after the ADM. Pipelines crossing both systems where the existing levee heights will remain as they are do not require relocation. These pipelines are not considered as posing a threat to the structural integrity of the levees.

6.3.2 Pipeline Relocation Method

At this point, it is envisioned that all pipelines requiring relocation will be removed by mechanical excavation (i.e., trenching) and then reinstalled at a deeper depth by way of horizontal directional drilling, which is ideally suited for deep burial depths. Directional drilling is a reliable method of relocation and can be done prior to constructing the levee. From a geotechnical perspective, the pipeline needs to be installed deep enough beneath the levee section and any berm sections to avoid stresses from levee and berm subsidence. The required depth and minimum distances the pipeline entry and exit points should be from the levee/floodwall centerline will be investigated after the TSP as the project details are further developed. That said, it is expected that the relocated pipelines will have to be buried at a depth of at least 30 feet below ground at the structure centerline with entry and exit points at least 500 feet away from the centerline.

7 STRUCTURAL REQUIREMENTS

7.1 INTRODUCTION

This section contains the conceptual design and preliminary analysis for the structural components of the proposed Orange-Jefferson CSRM levee system. The Orange 3, Jefferson Main, and Beaumont A Reaches have lengths of floodwall proposed to be included in their alignments.

The current proposed Orange 3 Reach of the Orange-Jefferson CSRM levee alignment crosses two existing navigation channels - Adams Bayou and Cow Bayou. Consequently, the system requires closure at each bayou with a navigation gate structure.

7.2 FLOODWALL

7.2.1 Assumptions

The floodwall will be designed as an inverted T-wall, which is the most appropriate type of floodwall for the project area. The inverted T-wall type also has no height limit and is the recommended type for areas with barge/boat impacts, which are possible with this proposed levee system.

Floodwall in the Orange-Jefferson CSRM levee reaches is being designed to an average top elevation of all portions of floodwall in the Reaches of the alignment. The top elevation includes added height for wave run-up and relative sea level rise, at the predicted intermediate level, on top of the modeled SWLs. For cost estimating purposes, the stem height was varied according to the variations in natural ground elevations, with the base geometry, concrete thicknesses, and pile arrangement remaining the same. Preliminary design parameters for the floodwall are based on hydraulic loadings (surge, wave run-up, etc.) provided by H&H and generalized local soil conditions along the alignment of the levee provided by Geotechnical. From the geotechnical investigations, sand stratums were found in areas the floodwall traversed. The floodwall will utilize a steel sheetpile cutoff (PZ22) for the entire length of floodwall, due to the assumption that a coastal storm event in this area would achieve steady state conditions. The project areas are in a non-seismic zone; therefore, no seismic analysis was done.

For areas of the alignment which require a tie-in between the earthen levee and floodwall, the T-wall will extend into the earthen embankment a distance past the design height of the earthen levee. This extension can be assumed to be 30 to 40 feet, but will depend on heights of the floodwall and adjacent earthen levee.

7.2.2 Structural Analysis

The design process for the floodwall used several EMs and the HSDRRDG. EM 1110-2-2502 was used for most of the floodwall design, except where newer EMs, such as EM 1110-2-2104, provided values that are more conservative.

7.2.2.1 Design Parameters

- Natural ground elevation: EL 5.0'
- K_o: 0.8 (at-rest earth pressure coefficient)
- Cohesion = 0.4 ksf (foundation)
- Φ = 23 degrees, for drained condition
- Unit weight (soil), γ = 110 pcf
- Unit weight (concrete), γ = 150 pcf
- Unit weight (water), γ = 64.0 pcf
- Still-Water Level = EL +11 feet (provided by H&H)
- Wave data for design purposes not provided at this stage of the project
- Concrete: normal weight, f'c = 4ksi
- Reinforcing steel: ASTM A615, fy = 60 ksi

The natural ground elevation was established from LiDAR survey data for the area. The ground elevation used for the design is an average value for all locations of floodwall. The average ground elevation was derived from the available LiDAR survey data by creating a profile along each of the Orange-Jefferson CSRM Reaches and calculating the average ground surface within all locations of floodwall. Geotechnical provided the foundation soil properties information (at-rest earth pressure coefficient, soil cohesion, phi angle, and unit weight) that was obtained through geotechnical investigations. (For more information on these input variables, including how they were derived, see the Geotechnical section of this appendix.) A higher unit weight of water than normally used (62.4 pcf) was assumed for the design due to the higher salinity of the water in this coastal location. The still-water level was provided from modeling performed by H&H.

7.2.2.2 Load Cases and Factors

Load cases for the floodwall design, as shown in Table 7-1, are based on EM 1110-2-2502 along with the HSDRRDG, which provides several additional load cases. The single load factor method, outlined in EM 1110-2-2104, was used to determine the factored moment and shear for the concrete structure. Where guidance overlapped, the more conservative approach was utilized for the design that would satisfy all applicable guidance.

Table 7-1: Floodwall Load Cases

Sabine to Galveston Floodwall Load Cases (Refer to EM 1110-2-2502 Table 4-3 & HSDRRSDG)				
Load Case No.	**C1**	**C4**	**C5**	**C6**
Title	Surge Stillwater	Construction	Wind	Stillwater to Top of Wall
Description	Backfill in place Water to stillwater Level (flood side) Waves excluded Uplift acting	Short Duration Loading Wind Included Construction Equipment Surcharge	Backfill in place Water at usual (non-storm) level Wind on land side	Backfill in place Water to top of wall Level (flood side) Waves excluded Uplift acting
Load Case Classification	Unusual	Unusual	Unusual	Extreme
Flood water Elevation	11.00	-----	-----	16.00
Wave Height	-----	-----	-----	-----
Backfill Elevation - Flood Side	5.00	5.00	5.00	5.00
Backfill Elevation - Land Side	5.00	5.00	5.00	5.00
Surcharge	-----	5 k/ft, 40 mph wind speed	140 mph wind speed	-----
Overturning (Min base in Compression)	100.0%	75.0%	75.0%	75.0%
Sliding FS Required	1.50	1.33	1.33	1.33
Minimum Bearing Capacity (FS)	3.00	2.00	2.00	2.00

*Load Cases C1, C2a, C2b, C2c, C4 and C5 are from EM 1110-2-2502, Table 4-3

**Load Case C6 is from New Orleans HSDRRSDG

***Load Case C6: values for sliding, overturning and bearing from I2, EM 1110-2-2502, Table 4-2

****Information for wave load cases (C2a, C2b, C2c) not provided at this stage (will update design when values are provided by H&H)

7.2.2.3 Design

The floodwall cross-section for the conceptual design (dimensions shown in Figures 7-1 and 7-2) was generalized for all locations of floodwall. The design assumes a stem height of 15 feet and ground elevation of +5 feet NAVD 88. The top of wall height was established based on SWLs of 11 feet NAVD 88, and includes additional height provision for wave run-up and intermediate sea level rise. The SWLs are premised on the economic analysis results that establish the NED Plans. The thicknesses of the concrete base and stem were optimized in the design process to provide the most appropriate cross section. Cohesive soil, having low permeability, will be used to back fill the flood side of the walls to reduce the potential of seepage under the wall. This material will also be used to backfill the land side of the walls where a concrete splash apron will be constructed to prevent scour from wave overtopping. Because of the cohesive nature of soil to be used as backfill and assumed conservative value for the soil foundation material (C = 0.4 ksf), the strength mobilization factor was not applied in the design calculations.

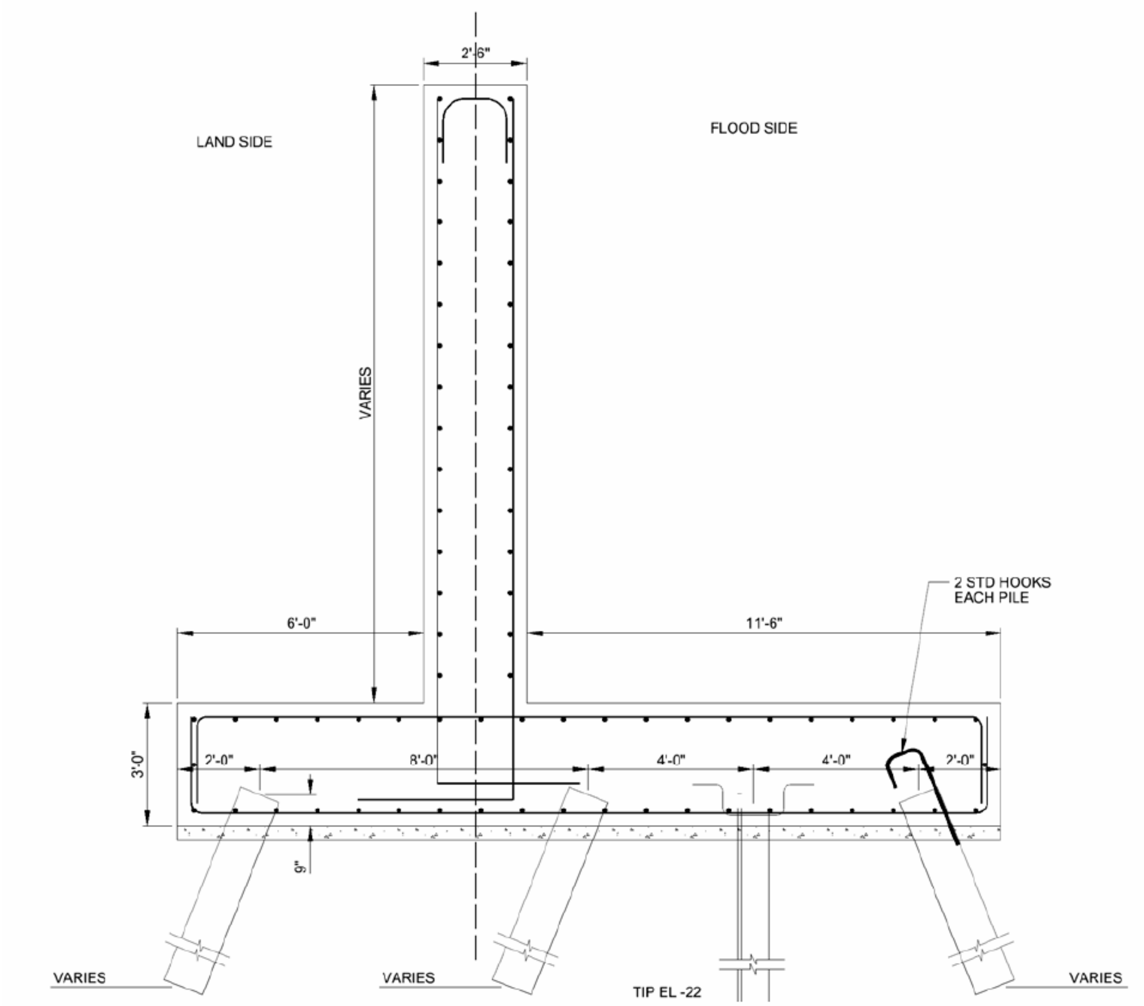

Figure 7-1: Floodwall - Conceptual Cross Section

Sabine to Galveston - Floodwall

Design Parameters

Floodwall dimensions (see figure)	
Top of Wall Elevation =	16
Ground EL (flood side) =	5
Ground EL (land side) =	5
stem height=	15
stem width=	2.5
base width=	20
base height=	3
stem position on base =	7.25
heel length=	11.50
toe length=	6.00
EL, top of base=	1
EL, bottom of base=	-2

Design Parameters		
$\gamma_{saturated}$ (lb/ft^3) =	110	
$\gamma_{submerged}$ (lb/ft^3) =	46	
γ_{water} (lb/ft^3) =	64	
$\gamma_{concrete}$ (lb/ft^3) =	150	
K_o =	0.8	
Wind Velocity (mph) =	140	*Load Case 5
$c_{undrained}$ (ksf) =	0.4	
$\phi_{undrained}$ =	0	
$N_{q,\ undrained}$ =	1.00	(EM 1110-2-2502, Table 5-1)
$N_{\gamma,\ undrained}$ =	0.00	(EM 1110-2-2502, Table 5-1)
$N_{c,\ undrained}$ =	5.14	(EM 1110-2-2502, Table 5-1)
$c_{drained}$ (ksf) =	0	
$\phi_{drained}$ =	23	
$N_{q,\ drained}$ =	8.66	(EM 1110-2-2502, Table 5-1)
$N_{\gamma,\ drained}$ =	4.82	(EM 1110-2-2502, Table 5-1)
$N_{c,\ drained}$ =	18.05	(EM 1110-2-2502, Table 5-1)

Load Factors (per EM 1110-2-2104, 3-3)	
Hydraulic =	1.3
Hydraulic (tension) =	1.65
Dead =	1.7
Live =	1.7
Wind =	0.75

Figure 7-2: Input for Floodwall Analysis

Several stability analyses were performed in accordance with the guidance of EM 1110-2-2502. These included evaluations of overturning, sliding, and bearing capacity. Each of these stability checks has an associated factor of safety for different load cases that the floodwall shall be designed to meet, which is provided in Table 7-1.

The figure below summarizes the design parameters and input values that were used in the stability analyses.

7.2.3 Conclusions

More extensive soils investigations will need to be conducted during PED to provide better understanding of the foundation conditions for the various floodwall sections along the proposed Orange-Jefferson CSRM levee alignment. Foundation analyses and design will be completed during the PED in accordance with applicable USACE engineering guidance.

Several sections of floodwall in the proposed alignment are near or along navigation channels; therefore marine vessels will be traveling near them. For those floodwalls that will be adjacent to the proposed navigation gates, fender systems will be installed to prevent vessel strikes (collisions) on the floodwalls during normal operations and during storms. The relative draft of a typical barge is 10 to 12 feet; therefore, with a top of wall surge loading at elevation +16.00 feet NAVD 88, the fender system will have to be built to a height that would be sufficient to keep vessels from striking the floodwall. The proposed floodwall along the Channel to Orange will also be exposed to vessel strikes. Consequently, this area will have to be designed accordingly to account for the possibility of vessel strikes. This design will be done in PED.

All floodwall locations were assumed to be constructed with protection against overtopping (splash apron) on the land side of the wall. Erosion protection (riprap) will be placed around floodwall-to-earthen levee tie-ins on both the flood and land sides of the levee.

During the design process, the floodwall was found to require piling due to the bearing stability check not meeting the minimum factor of safety for several load cases. As stated earlier, this assumption was made for the preliminary design. The conceptual design for the floodwall utilizes 16-inch square prestressed concrete piles for all locations, but it may be determined in future analyses that steel pipe or steel H-piles are more suitable for certain floodwall locations. A piling design will be required during PED for all locations which are determined to require piles. The additional soils investigations to be performed during PED will help to determine if floodwall locations will require piles and what type of piles are most suitable. Tests that will need to be performed during the PED phase for the piling design: *pile load tests,* which are intended to validate the computed capacity for a pile foundation and also to provide information for the improvement of design rational, and *field tests*, which are performed to verify or predict driving conditions and/or load capacity of service piles at the construction site. The preliminary configuration for the amount and spacing of the piles is 3 piles/row at 5-foot spacing. A pile group analysis will also need to be performed during PED to determine the most appropriate piling configuration.

For Still Water to Top of Wall loading (Load Case 6), the sliding stability check was found to not meet the minimum factor of safety. Consequently, the possibility of widening or sloping the base of the floodwall or providing a key in the base will need to be further explored during PED. The additional geotechnical investigations during PED will provide higher resolution of the in situ soils at the floodwall locations, which will yield a more accurate design. For the TSP/NED Plan however, the generalized floodwall cross section shown in Figure 7-2 was assumed to calculate quantities for the alternative cost estimates. Steel reinforcement for the floodwall was calculated to be #8 bars at 12 inches on centers. Due to the possibility of other load cases (i.e. the wave load cases) increasing the calculated loads for the reinforcement, for cost estimating purposes, 6 pounds of steel reinforcement per cubic foot of concrete was assumed.

7.3 NAVIGABLE GATE STRUCTURES

7.3.1 Concept Design

The navigable gate structures at Adams and Cow Bayous are proposed as a sector gate with adjacent vertical lift floodgates. The locations of these structures in relation to the alignment of the Orange 3 Reach are shown in Figure 7-3. The gate structures were modeled after the sector gate recently completed at the Caernarvon Canal as part of the New Orleans Hurricane Protection System project work. The structures will be built with an additional height of 2 feet relative to the height of the adjacent levees to provide structural superiority for the structures because of the difficult nature of their construction.

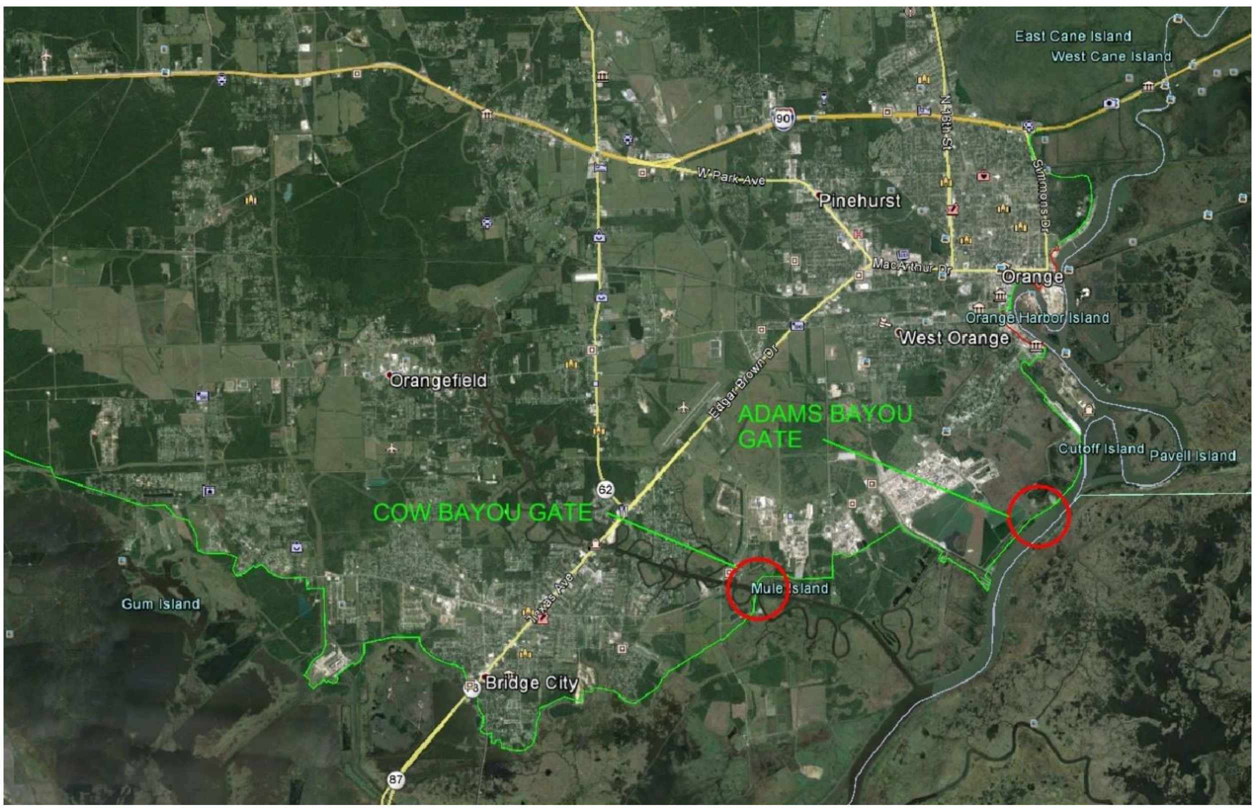

Figure 7-3: Overview Map of Orange 3 Reach – Gate Structures

It is assumed that both sector gate structures will be constructed in line with the existing authorized channels (see Plates S-01 and S-02 for representative plan views of the gate structures, respectively, and Figure 7-4 for a photo of a similar sector gate structure). Concrete floodwalls will be constructed adjacent to these structures and tie into the earthen levee.

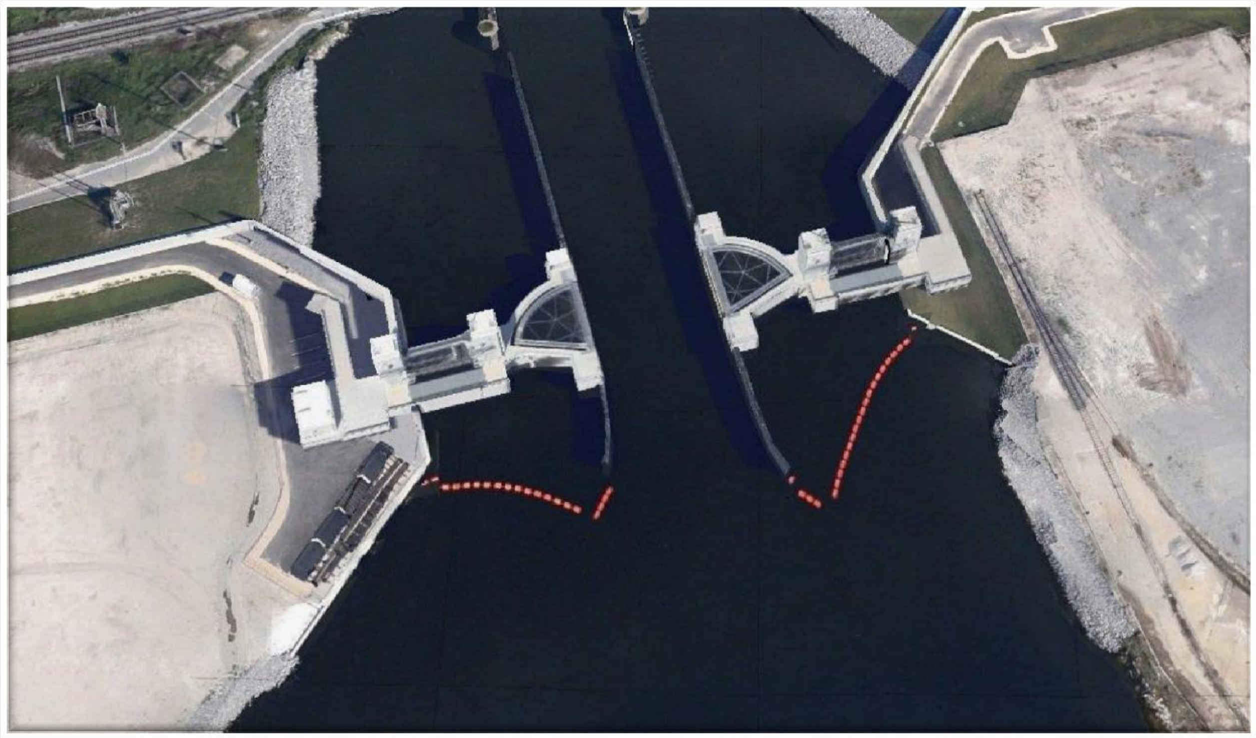

Figure 7-4: Aerial Photograph of Navigation Sector Gates with Adjoining Vertical Lift Gates

7.3.1.1 Adams Bayou

The Adams Bayou Gate Structure will be of the sector gate type with a navigable opening of 56 feet. Previous CSRM studies performed on the Orange County area utilized this type and size of structure for the navigable channel crossings. Without further investigations being performed by the PDT, this size will be utilized for the purpose of this study. A structure this size will be large enough to accommodate single barges and other moderate draft vessels likely to use the waterway. The sill of the sector gate structure is assumed to be at the current maintained channel depth of 13 feet MLLW.

Due to the narrowing of the bayou at the proposed crossing, an additional series of non-navigable flood gates are proposed to mitigate impacts to channel flow during normal upland rainfall events. This gate will be a 50-foot-wide vertical lift gate positioned on the west side of the sector gate structure. Its sill depth is assumed to mirror the current channel depth near the banks of the bayou (approximately 8 feet). The size of the vertical lift gate was determined by calculating the normal channel flow with a rainfall event and providing enough open area through the gate structures for this flow. For more information, see the Hydraulics and Hydrology section of this report. Because of the location where this structure is proposed to be sited, only one vertical lift gate is being proposed. On the east side of the channel, there is an abandoned placement area, which is assumed to be integrated into the proposed levee system. Therefore, the vertical lift gate is proposed to be situated on west side of the sector gate structure to avoid disturbing the placement area levee in which a gate structure would otherwise be constructed through it.

7.3.1.2 Cow Bayou

For the purpose of this study, the same sector gate proposed for Adams Bayou is proposed for the navigation structure at Cow Bayou due to similar site conditions at each proposed location (i.e., type of vessel usage, maintained channel width and depth). The sill of the sector gate structure is assumed to be at the current maintained channel depth of 14 feet MLLW.

Two 60-foot-wide non-navigable vertical lift floodgates will flank the Cow Bayou sector gate structure. These non-navigable floodgates are proposed to mitigate impacts to channel flow during normal upland rainfall events. The depth of the sill for the vertical lift gates is assumed to mirror the current depth of the channel near the banks of the bayou (approximately 8 feet). Cow Bayou is approximately 400 feet wide; therefore, a larger set of vertical lift gates will be required than for Adams Bayou to mitigate the impacts.

7.3.2 Conclusions

The gate structure analyses and design are beyond the scope of this study. Detailed design will be required for all aspects of the combined sector and vertical lift gate structures during PED. For the purpose of this study, only general concepts for these structures are provided, which are based on similar, previously constructed gates. Both gate structures will be constructed within channels used for navigation therefore, a cofferdam will have to be constructed to provide a dry construction area. The construction areas are currently assumed to be below the water table and would therefore require dewatering. Due to the current active use of both of these channels, they will be required to remain operational and therefore will need some type of diversion channel during the gate construction to not inhibit industry from utilizing the channels. It is understood that the foundation soils are very soft at the proposed locations for the gate structures; therefore, the foundation will require strengthening with piles to form a stable foundation for supporting the gate structures. The piles are assumed to be prestressed concrete for cost estimating purposes of this study, but may consist of concrete piles, steel H-piles, or pipe; which will be determined during PED. Floodwall will be directly adjacent to these structures and tie in to the earthen levee sections. Proper investigations should occur prior to study completion to determine the most appropriate size of the navigable opening for each of the sector gate structures.

7.4 REFERENCES

USACE Engineer Manuals

- EM 1110-2-2104 - Strength Design for Reinforced-Concrete Hydraulic Structures, 30 June 1992
- EM 1110-2-2502 - Retaining and Flood Walls, 29 September 1989

Other USACE Engineering Guidance

- Hurricane and Storm Damage Risk Reduction System Design Guidelines (HSDRRSDG) - Interim, June 2012

Drawings and Reports

- Flood Protection Planning Study Hurricane Flood Protection System, Orange County, Texas, December 2012.

8 HAZARDOUS AND TOXIC MATERIALS

A detailed discussion of HTRW and associated risk level definitions is provided in Appendix N of the DIFR-EIS.

8.1 ORANGE-JEFFERSON COUNTY CSRM

The Orange County levee and floodwall corridor (Orange 3 Reach) is approximately 27.2 miles in length from near the intersection of the Sabine River with Interstate 10, around the cities of Orange and Bridge City, Texas, and up to a point less than a mile southwest of FM 1135 where it intersects FM 105 (Orangefield Road) midway between Rose City and Bridge City. The corridor runs near numerous industrial facilities, including the Port of Orange, several shipbuilding yards, DuPont-Invista Sabine River Works, Lanxess Chemical Corporation, Firestone Polymers, and Chevron Phillips Chemical Company.

There are seven (7) major HTRW sites and facilities near or adjacent to the Orange County levee and floodwall alignment corridor identified from the research conducted. These sites consist of industrial facilities currently operating that are listed in databases identifying the generation, handling, or release of hazardous materials and waste. The largest of these, DuPont Chemical Corporation, maintains several large wastewater treatment and cooling ponds along the tentative alignment. The general HTRW risk for the levee alignment in Orange County is classified as *Low*, since no unresolved current or recent hazardous material releases were found, and no significant recent RCRA or CWA permit violations were identified. HTRW facilities in the area should be more thoroughly investigated with visual inspections and interviews with facility managers to confirm the potential HTRW risks along the levee alignment prior to construction or more detailed design.

The Jefferson Main reach levee and floodwall alignment corridor is approximately 14.3 miles in length around the City of Port Neches, Texas, and the ExxonMobil Chemical Plant in Beaumont, Texas. The corridor runs near seven (7) major petroleum and chemical facilities, including the ExxonMobil Chemical Plant, DuPont Chemical Corporation, and the Huntsman refinery, as well as numerous petroleum and chemical tank farms and loading facilities. There are numerous pipelines crossing the levee alignment in Jefferson County that transport hazardous materials for shipment or further processing. These pipeline crossings are identified as points of concern along the TSP alignment where special caution needs to be exercised during construction to avoid damage to the pipelines and release of hazardous materials into the environment.

Most of the refineries and chemical plants have had numerous Clean Air Act violations in the past or are currently in effect due to stack emissions. No currently active spills or land/water releases of hazardous materials were found however. The Star Lake Canal, emanating from the Huntsman Refinery, runs adjacent to the Jefferson County alignment and is listed as an active NPL site due to past release of wastewater and storm runoff containing hazardous materials. This canal is scheduled for remediation by USEPA with the proposed removal of contaminated bottom sediments. Due to the presence of toxic sediments, this canal would be considered *High* risk; however, minor construction on the existing levee along the canal would not expose any sediments within the canal. Therefore, despite the large number of potential HTRW release facilities, the Orange-Jefferson CSRM project area is classified as generally *Low* risk for HTRW. The HTRW facilities in the area need to be more thoroughly investigated with visual inspections and interviews with facility managers to confirm the potential HTRW risks along the alignment corridor prior to construction or more detailed design. Coordination with USEPA remediation of the Star Lake Canal would also be required.

8.2 PORT ARTHUR AND VICINITY CSRM

The existing Port Arthur HFPP corridor is approximately 34.4 miles around the cities of Port Arthur and Groves, Texas. The corridor runs near five (5) major petroleum and chemical facilities, including the Texaco refinery, the Total-BASF refinery, the Valero refinery, and the Motiva Enterprises refinery, as well as numerous petroleum and chemical tank farms and loading facilities.

Most of the refineries and chemical plants have had numerous Clean Air Act violations in the past or are currently in effect due to stack emissions. No currently active spills or land/water releases of hazardous materials were found however. Despite the large number of potential HTRW release facilities, the Port Arthur CSRM project area is classified as generally *Low* risk for HTRW. The HTRW facilities in the area need to be more thoroughly investigated with visual inspections and interviews with facility managers to confirm the potential HTRW risks along the alignment corridor prior to construction or more detailed design. Although some large refineries and chemical plants are more than 0.25 mile from the project alignment, they could still affect the project corridor as a result of a major release.

8.3 FREEPORT AND VICINITY CSRM

The general risk level for the Freeport area is indicated as *Low* risk, since no current or recent unresolved RCRA or CWA releases were identified for any of the industrial facilities in the Freeport area. Many of the facilities have ongoing CAA violations due to stack emissions. The HTRW facilities in the area need to be more thoroughly investigated with visual inspections and interviews with facility managers to confirm the potential HTRW risks along the alignment corridor prior to construction or more detailed design.

9 COST ESTIMATES

9.1 REFERENCES

ASTM E 2516-11 - Standard Classification for Cost Estimate Classification System

ER 1110-2-1302 – Civil Works Cost Engineering, 15 Sep 08

9.2 CLASSIFICATION

Cost Estimates for screening the different project reaches in Orange and Jefferson Counties (alternatives) were developed utilizing a Class 4 parametric approach using both historical and unit costs. From these screenings, the NED plans for the proposed Orange-Jefferson levee/floodwall system were determined. A similar approach was taken in screening alternatives for the alignment for the Port Arthur and Freeport Hurricane Flood Protection System.

9.3 SCOPE

Costs were developed only to the extent necessary to compare alternatives and determine which projects would comprise the TSP. Therefore, costs are not necessarily reflective of actual construction costs of a fully defined and developed project.

9.4 SCHEDULE FOR DESIGN AND CONSTRUCTION

Construction of the recommended HFPPs for the Sabine and Brazoria sub-regions is currently assumed will take 8 years to complete, in sequential phases. When the feature designs for the projects composing the TSP are further advanced after the ADM, the PDT can make more informed estimates of the amount of time it will take to design the projects and build them, and of the number of construction contracts it probably will take.

Prior to construction of the proposed HFPPs in Orange and Jefferson Counties, numerous pipelines and other utilities, including overhead power lines, that cross or are close to the structures will have to be relocated or modified to accommodate the levee embankments and floodwalls. For the alternative comparison analysis and TSP, it was assumed that all such pipelines would have to be removed and relocated. When constructing the closure structures with navigable opening for these particular systems, the construction contractor will have to provide for the continuity of vehicular traffic on secondary roads, rail traffic, and vessel traffic on Adams and Cow Bayous. Such accommodations will require significant coordination efforts given the agencies, public concern, and industry involved when possible disruptions are a possibility. Raising sections of the existing Port Arthur and Freeport HFPP levees likely will not involve any pipeline and utility relocations, except for one pipeline crossing the Freeport system.

When raising sections of the existing Freeport and Port Arthur HFPP levees and replacing floodwall, the construction contractor will be required to maintain the current flood-protection performance level provided by the system. Emergency response plans for the duration of the construction period, therefore, will have to be coordinated and prepared, reviewed by USACE, and then monitored and executed properly to address the potential for flooding.