US Army Corps of Engineers®

Charleston District

CHARLESTON PENINSULA, SOUTH CAROLINA, A COASTAL STORM RISK MANAGEMENT STUDY

Charleston, South Carolina

ENGINEERING APPENDIX - B

February 2022

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5. COST ENGINEERING SUB-APPENDIX

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CHAPTER 1 INTRODUCTION

1.1. DESCRIPTION OF PROJECT AREA AND VICINITY

Centrally located along the coast of South Carolina, the Charleston Peninsula project area is approximately 8 square miles, located between the Ashley and Cooper Rivers (Figure 1.1). The two rivers join to form the Charleston Harbor before discharging into the Atlantic Ocean. Charleston Harbor is formed by the confluence of the Cooper, Ashley, and Wando Rivers. It includes the tidal estuary of the lower 12 miles of the Cooper River and the four miles of open bay between the confluence of the Ashley and Cooper Rivers and the Atlantic Ocean. The Cooper River contributes most of the freshwater inflow to the system and is the largest of the estuaries, extending about 57 miles from the harbor entrance to the Jefferies Hydroelectric Station at Pinopolis, SC. The Charleston Harbor is sheltered by barrier islands.

Figure 1.1 Study Area

The first European settlers arrived in Charleston around 1670. Since that time, the peninsula city has undergone dramatic shoreline changes, predominantly by landfilling of the intertidal zone. Early maps show that over one-third of the peninsula has been “reclaimed.” Much of the landfilling occurred on the southern tip of Charleston, behind a seawall and promenade, known as the Battery and along the western shoreline. Figure 1.2 shows the Halsey Map of 1844 which depicts the original shoreline of the Charleston Peninsula.

Figure 1.2: 1844 Map of Charleston

The southern tip of the peninsula has a battery wall. The battery area is distinguished by elevation. The high battery wall is presently at elevation 9 NAVD88. "The High Battery consists of two distinct designs: The original High Battery (reconstructed in 1893 to 1894) is comprised of a stone wall (seaward) backed by two masonry/concrete walls approximately 10 feet apart. The space between the two walls is backfilled with soil and the top is capped with stone slabs to create a walkway or promenade.

The second part of the High Battery, "The Turn," was originally constructed in 1919. This portion of the Battery connected the High Battery to the Low Battery, creating a continuous seawall from north of Water Street to just south of Tradd Street.

Construction of the "Turn" used methods very similar to those used in the construction of the Low Battery. The repair of the "Turn" is the first phase of the overall seawall repair project and has been completed."

"The Low Battery was constructed as part of a large land reclamation project undertaken in two phases: The first section (1909 to 1911) extended from Tradd Street to King Street. The second phase (1917 to 1919) extended from King Street to the "Turn" at the intersection with East Battery Street. The concrete wall of the Low Battery was constructed on a timber deck supported by timber pilings. The seaward face of the Low Battery is skirted with concrete panels attached to timber sheeting and batter piles. This system formed a retaining wall system to retain the landside fill." During this feasibility study, the city has begun raising the low battery to be similar in height as the high battery.

The federal navigation channel is adjacent to the study area along the eastern side with Columbus Street Terminal and Union Pier Terminal (Figure 1.3). The federal navigation channel on the Ashley River to the west of the peninsula is still authorized but not maintained.

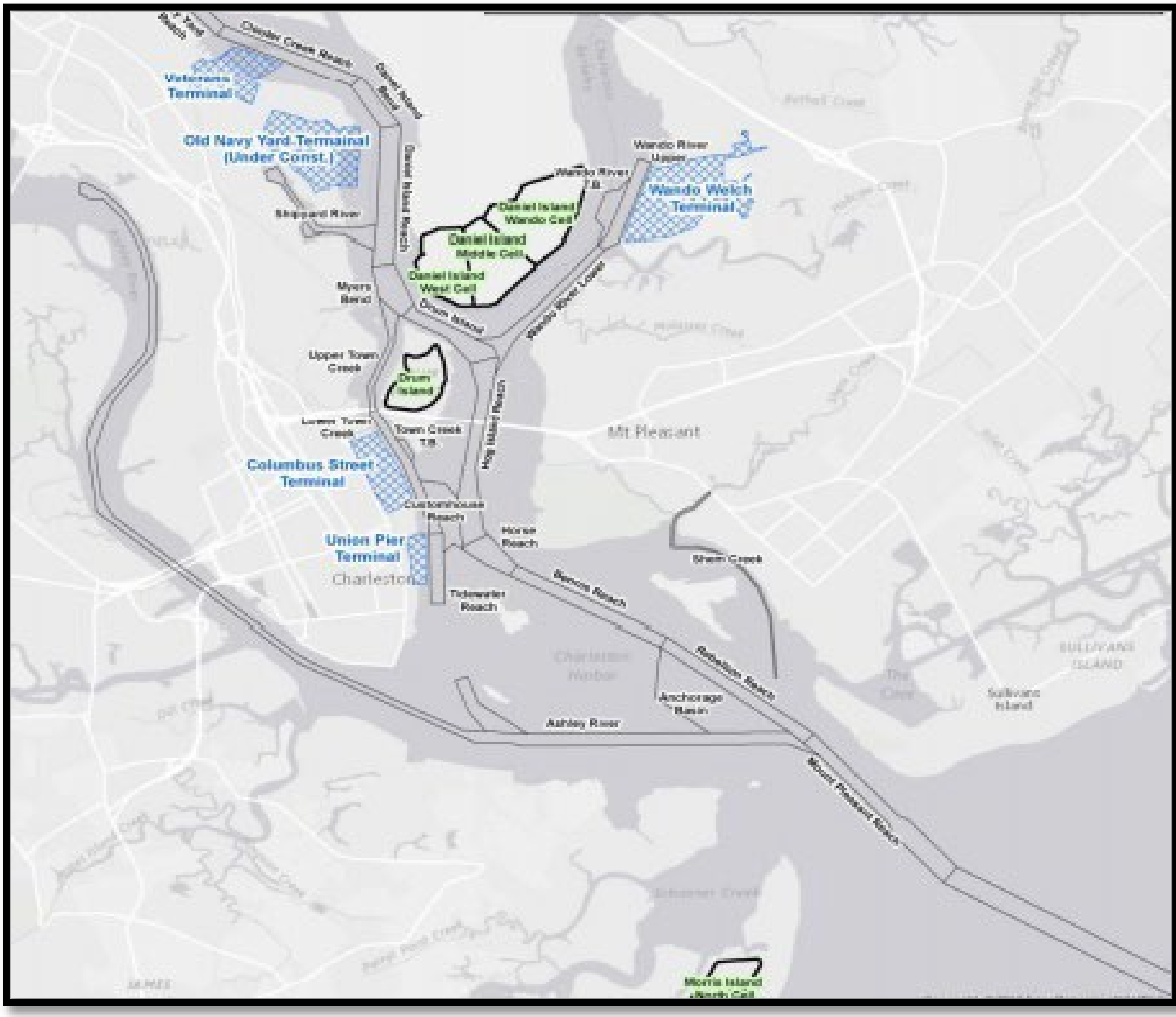


Figure 1.3 Charleston Harbor Navigation Channel

1.2. SCOPE OF ENGINEERING APPENDIX AND ENGINEERING ANALYSIS

The Charleston Peninsula Coastal Storm Risk Management (CSRM) study will address potential structural and non-structural solutions to mitigate coastal storm flood damages. This Engineering Appendix discusses the preliminary engineering and design work conducted of the structural elements and measures of the Charleston CSRM study. This includes the compilation and evaluation of existing geotechnical data for subsurface conditions, Coastal Storm surge numerical modeling modifying the FEMA ADCIRC and STWAVE models to enhance resolution of the study area, assess changes to the interior rainfall flooding by expanding and modifying the City of Charleston's HEC-RAS model, evaluation of the city's proposed "low" battery seawall modification, and the evaluation of floodwalls, berms, pump stations, breakwater, marsh resilience and other structural elements and measures that would meet the objectives and goals of the study. This appendix provides a general explanation of the preliminary engineering and design work that are further discussed in the sub-appendices from Structural Engineering, Geotechnical Engineering, Hydraulic Engineering of the Interior Hydrology, Coastal Engineering that supported the G2CRM Economic analysis, and Cost Engineering.

While not a typical engineered structure, the Natural and Nature-Based Features of the study are also described.

CHAPTER 2 EXISTING INFORMATION AND DATA

2.1. LIDAR

LIDAR collected by the South Carolina Department of Natural Resources in 2017 is being utilized in this study.

2.2. GEOLOGIC AND GEOTECHNICAL ASSESSMENTS

2.2.1 REGIONAL GEOLOGY

A compilation of geotechnical data was sent to SAW Geotechnical personnel from various consulting agencies within the public and private sector. Over 200 CPTs and SPTs were obtained and plotted into ArcMap. Borings were analyzed for easting and northing coordinates, depth of boring, and top of Cooper Marl Formation. Data plotted into ArcMap used coordinates provided on the logs; however, if easting and northing coordinates were not present, the borings were plotted visually from the maps provided by the consulting agencies. Based on the boring data collected, the top of the Cooper Marl Formation is depicted similarly to Figure 2.2.1. The Sub-appendix 2 Geologic and Geotechnical Engineering depicts the geologic setting and stratigraphy beneath the Charleston Peninsula.

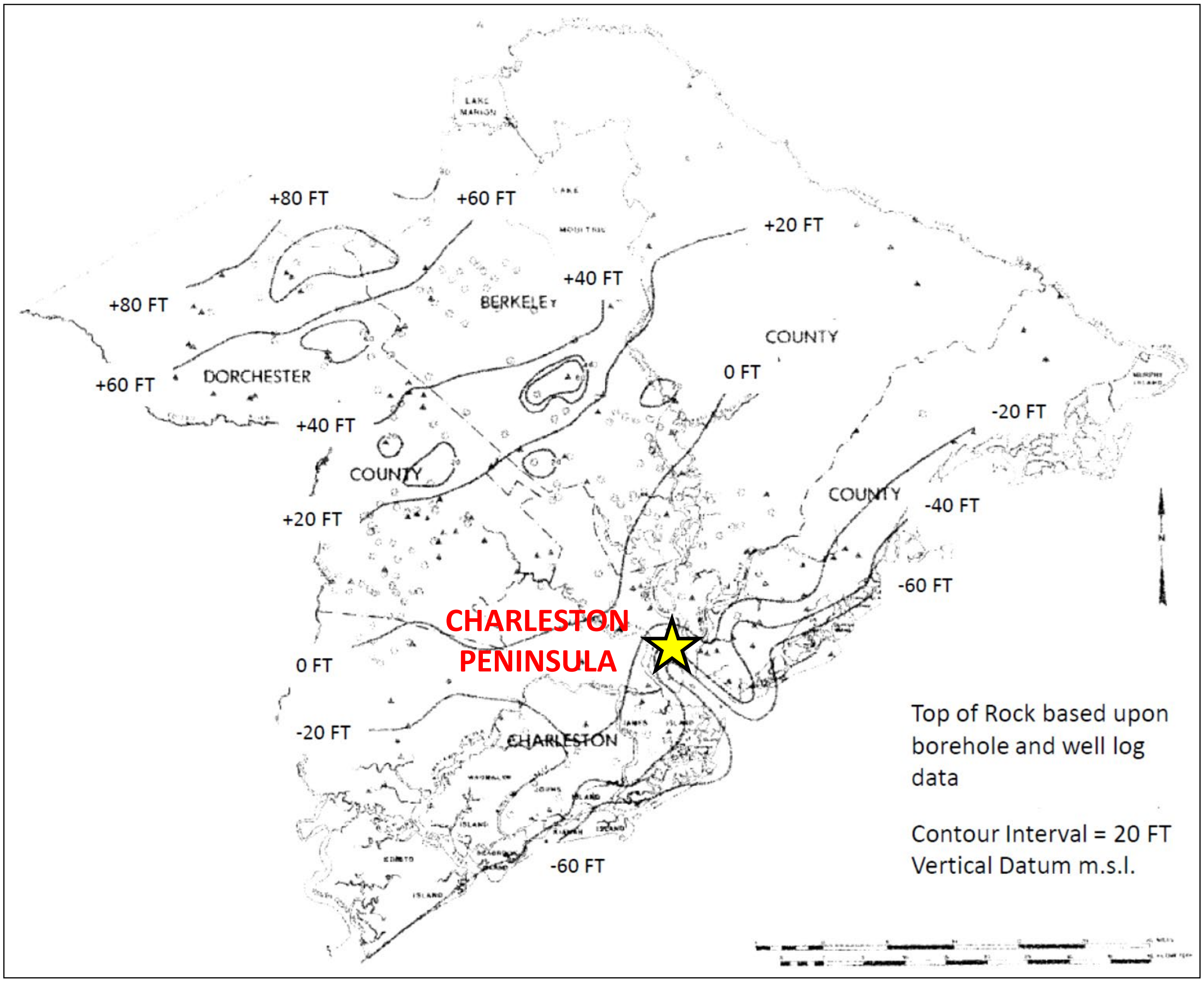


Figure 2.2.1: Structure contour map showing top of Cooper Formation, from Park (1985).

However, the term "Cooper Formation" (Toumey, 1848) is interchangeable with the term "Cooper Marl", which is the most recognized name for the material by the PDT. Further Explanation of the Geologic conditions is explained in Sub-AppendixSub-Appendix 2 Geologic and Geotechnical Engineering.

2.2.2 SEISMICITY

The Charleston Peninsula is located in a "hot spot" of high seismic activity and is deemed to be within a high seismic hazard zone as indicated in Figure 2.2.2.1. This area is known as the Middleton Place-Summerville Seismic Zone or the Charleston Seismic Zone. Additionally, Charleston, SC is also the site of the largest earthquake known to have occurred inthe southeastern United States, which occurred in 1886.

A seismic evaluation was completed as part of the feasibility study and the details are presented in ATTACHMENT 1 of the Sub Appendix 2 Geologic and Geotechnical Engineering.

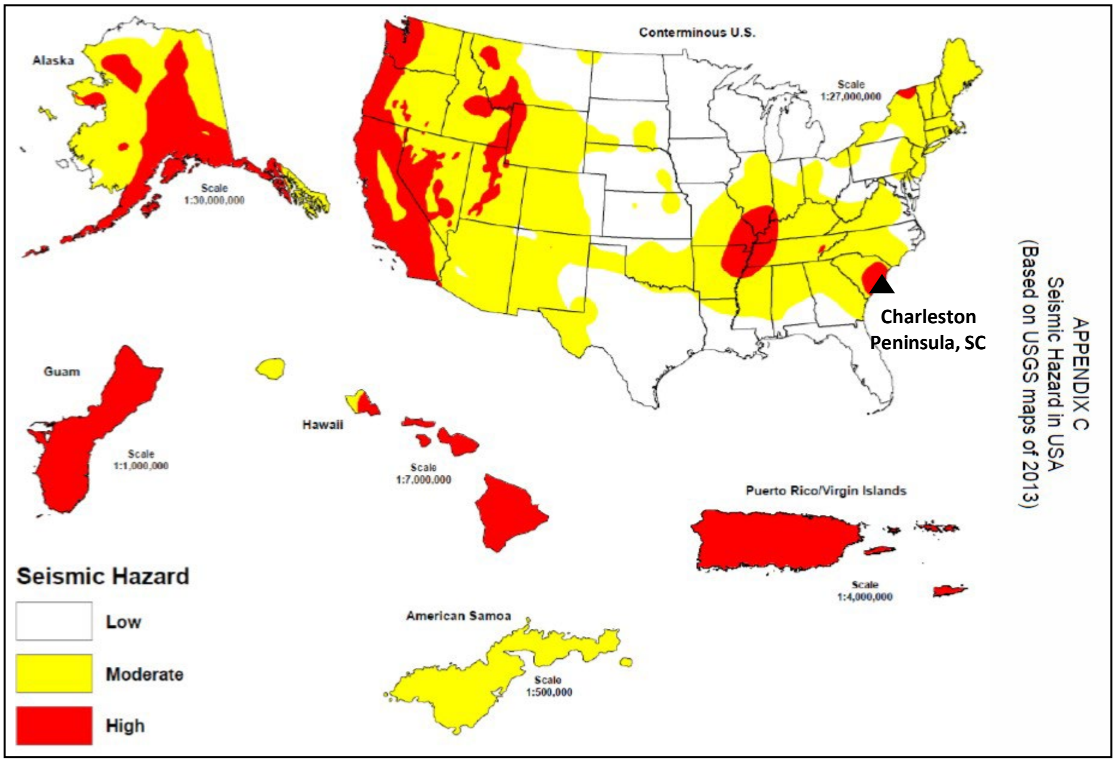


Figure 2.2.2.1: Project location shown on seismic hazard map of the USA, from ER-1110-2-1806.

2.2.2.1 Ground Motions

The seismic evaluation provided a range of ground motions for various events. An earthquake with a 2% probability of exceedance in 50 years could produce a PGA that ranges from 0.6 to 0.8g near the Charleston Peninsula [USGS 2014 seismic hazard map by Petersen et al. (2015)], shown in Figure 2.2.2.1.1. The site-predicted Peak ground acceleration (PGA) for an earthquake having a return period of 2,475 years is approximately 0.973g, which is slightly higher than the USGS seismic hazard map shown in Figure 2.2.2.1.1. Spectral ground motion on the Charleston Peninsula was also predicted by the Uniform Hazard Response Spectrum (Figure 2.2.2.1.2). Based upon probabilistic hazard mapping, the PGA at the site is predicted to be 0.8561g, but the largest and most likely damaging ground motion is 1.3972g at a spectral period of 0.2 seconds (Figure 2.2.2.1.2).

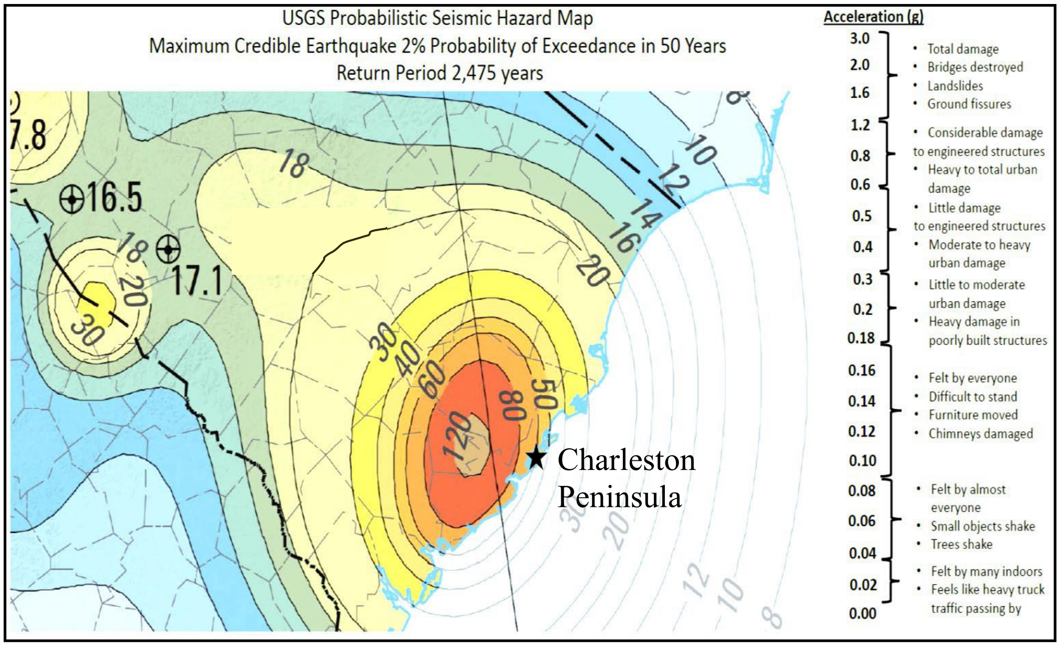


Figure 2.2.2.1.1: USGS Seismic Hazard Map, PGA, 2% Probability of Exceedance in 50 Years, from Peterson et al. (2015).

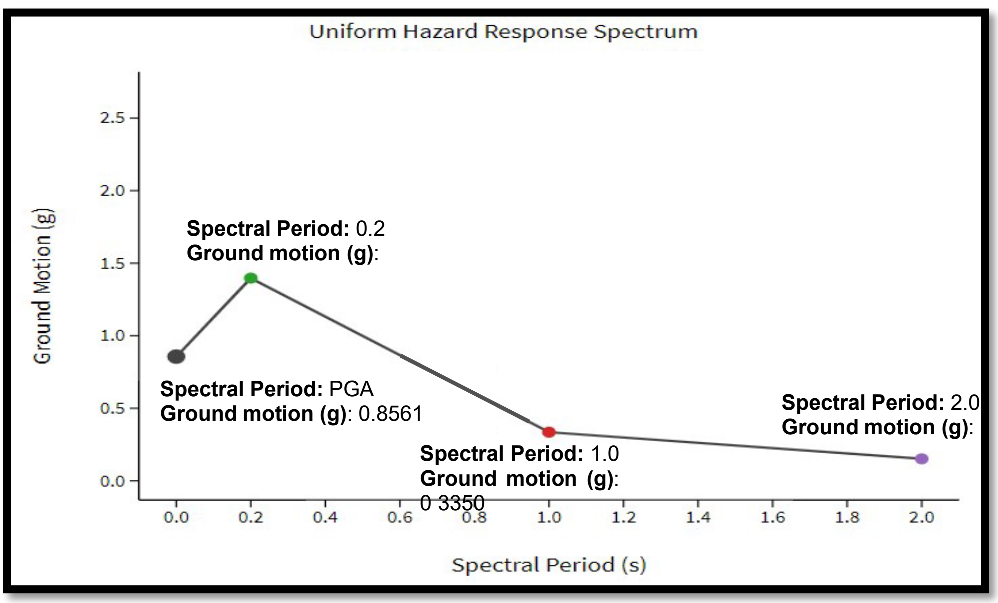


Figure 2.2.2.1.2: Uniform Hazard Response Spectrum predicted for the project site showing PGA with 2% in 50 years AEP (2,475 return period).

2.2.2.2 Maximum Credible Earthquake and an Operating Basis Earthquake

The Maximum Credible Earthquake (MCE) were deterministically derived. The MCE was determined to be an Mw = 7.3 and based upon the 1886 Charleston Earthquake event. The distance from the project site to the center of the MCE source zone is 10.00 km.

The Operating Basis Earthquake (OBE) was assessed using probabilistic methods that are informed by deterministic methods. An OBE PGA of 0.0548g and a SA of 0.09g (at 0.2 second period) is derived utilizing the USGS Unified Hazard Tool.

2.3. EXISTING NUMERICAL MODELS

2.3.1 COASTAL MODELS

There have been no past USACE Coastal Storm Risk Management Studies performed for the Charleston, Berkeley, Dorchester area, where city of Charleston Peninsula resides. Therefore, USACE reached out to SCDNR, the FEMA POC for Flood Insurance Studies (FIS) in the state of SC for available coastal models to minimize costs and improve efficiencies of the study. SCDNR contractor provided ADCIRC models, storm sets, SWAN runs, all the validation runs, production runs and input for their 2017 preliminary FIS, which was made effective January 2021. The ADCIRC /STWAVE mesh was modified for this study.

2.3.2 HYDROLOGIC and HYDRAULIC MODELS

USACE Engineer Regulation 1165-2-21 states "In urban or urbanizing areas, provision of a basic drainage system to collect and convey the local runoff to a stream is a non-Federal responsibility. This regulation should not be interpreted to extend the flood damage reduction program into a system of pipes traditionally recognized as a storm drainage system. "

While the storm drainage system is not a CSRM responsibility, any impacts to the interior hydrology due to the proposed project have to be evaluated and mitigated to the extent justified under USACE policy, if necessary. The City of Charleston contractor does not have a pipe network system coverage of the entire study area. The coverage they do have is in separate and different models based on drainage area.

The City of Charleston Contractor indicated they had majority of study area in HEC RAS 2D. They use the HEC RAS for rainfall and flow to the inlets for the drainage system and the pipe network model for conveyance to river or to the drywell/pump system depending upon drainage area (DA). They have provided the HEC RAS model. It does not cover the entire study area but with additional lidar, it was expanded. CESAC obtained concurrence from the MSC that the change in flood risk of a proposed project would be evaluated with the HECRAS 2D model only.

2.4. NOAA COOPER RIVER ENTRANCE TIDAL GAGE RECORD

The Cooper River Entrance Tidal Gage is Station 8665530 and is locally referred to as the Charleston Harbor or Custom's House gage. It was established September 13, 1899. It is located downtown on the peninsula in the vicinity of U.S. Custom House, along East Bay Street, and along Broad Street. The tide gage and staff are on the south end of the dock (Figure 2.4.1).

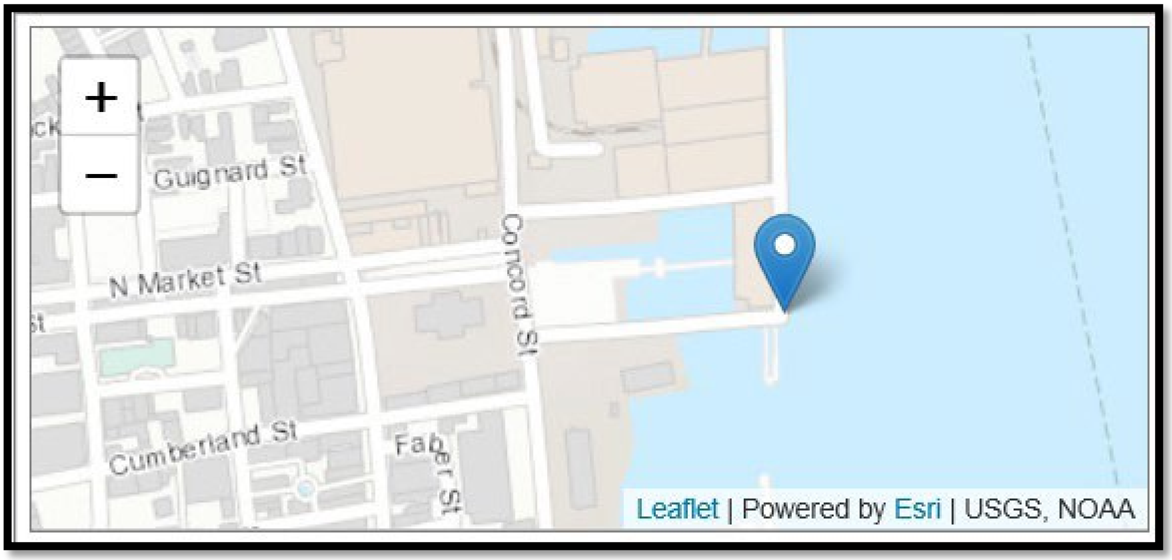


Figure 2.4.1 Location of NOAA Gage 8665530

Datum information provided by NOAA on their Tides and Currents website indicate a tide range of 5.76 feet (Figure 2.4.2 and Table 2.4.1). Mean Sea Level (MSL) of the tidal epoch between 1983 and 2001 is 2.92 feet above MLLW. The NAVD88 (North American Vertical Datum of 1988) is 0.22 above mean sea level. (https://tidesandcurrents.noaa.gov/datums.html?id=8665530)

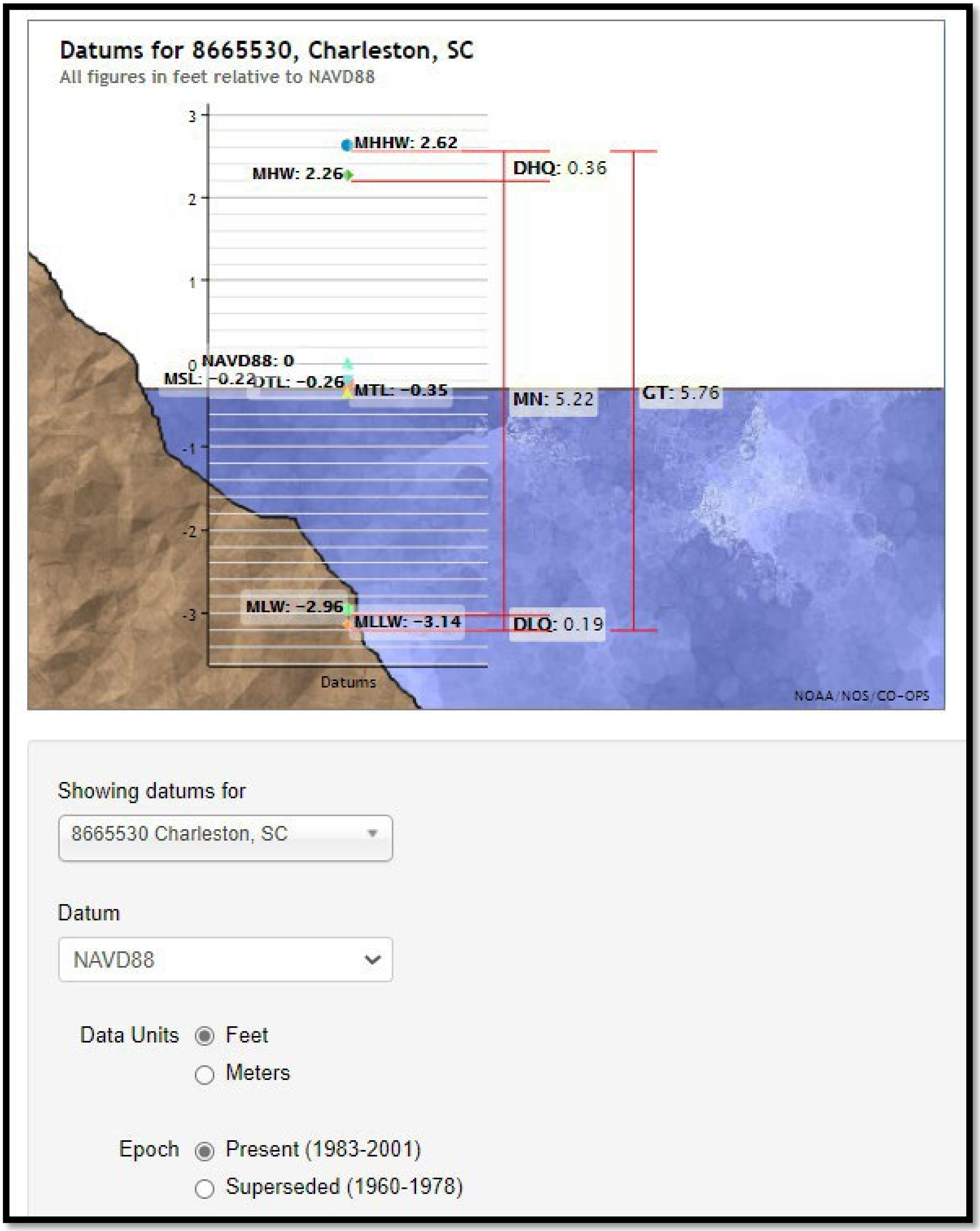


Figure 2.4.2 Tide Range Station 8665530

Table 2.4.1 Elevations on Mean Lower Low Water

Datum	Value	Description
MHHW	5.76	Mean Higher-High Water
MHW	5.4	Mean High Water
MTL	2.79	Mean Tide Level
MSL	2.92	Mean Sea Level
DTL	2.88	Mean Diurnal Tide Level
MLW	0.18	Mean Low Water
MLLW	0	Mean Lower-Low Water
NAVD88	3.14	North American Vertical Datum of 1988
STND	-2.77	Station Datum
GT	5.76	Great Diurnal Range
MN	5.22	Mean Range of Tide
DHQ	0.36	Mean Diurnal High Water Inequality
DLQ	0.19	Mean Diurnal Low Water Inequality
HWI	0.41	Greenwich High Water Interval (in hours)
LWI	6.63	Greenwich Low Water Interval (in hours)
Max Tide	12.52	Highest Observed Tide
Max Tide Date & Time	9/21/1989 23:42	Highest Observed Tide Date & Time
Min Tide	-4.09	Lowest Observed Tide
Min Tide Date & Time	3/13/1993 19:24	Lowest Observed Tide Date & Time
HAT	7.26	Highest Astronomical Tide
HAT Date & Time	10/16/1993 13:06	HAT Date and Time
LAT	-1.52	Lowest Astronomical Tide
LAT Date & Time	2/9/2001 7:24	LAT Date and Time

CHAPTER 3 PHYSICAL CONDITIONS

3.1. CLIMATE

Charleston SC has hot humid summers and fairly mild winters. Average annual high temperatures are approximately 75 degrees F and average annual low temperatures are approximately 53-degree F. Average annual precipitation is 44.29 inches with an average of 102 days of precipitation per year. Shown in Figure 3.1.1 and Table 3.1.1.

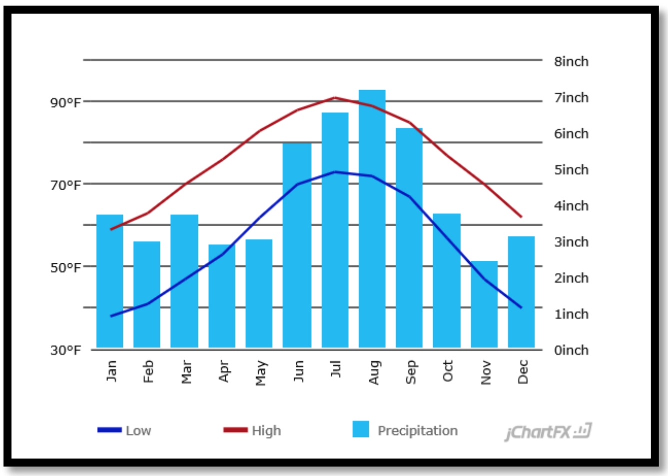


Figure 3.1.1 Charleston Temperature and Precipitation

Table 3.1.1 Charleston Temperature and Precipitation

Climate Charleston AFB - South Carolina

°C | °F

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Average high in °F:	59	63	70	76	83	88	91	89	85	77	70	62
Average low in °F:	38	41	47	53	62	70	73	72	67	57	47	40
Av. precipitation in inch:	3.7	2.95	3.7	2.91	3.03	5.67	6.54	7.17	6.1	3.74	2.44	3.11
Days with precipitation:	9	9	11	8	14	10	15	12	10	6	7	8
Hours of sunshine:	188	189	243	284	323	308	297	281	244	239	210	187

Source: https://www.usclimatedata.com/climate/charleston-afb/south-carolina/united-states/ussc0052

3.2. HORIZONTAL AND VERTICAL DATUMS

Horizontal datum for this study is tied to the State Plan Coordinate System using North American Datum of 1983 (NAD83, South Carolina 3900, international feet). Distances are in feet by horizontal measurement. The vertical datum for this study is tied to the North American Vertical Datum of 1988 (NAVD88), a requirement of ER 1110-2-8160. Elevations are in feet.

3.3. WINDS

Due to the geographic orientation of the peninsula with the Ashley River on the west and the Cooper River on the right, the western side and the northeastern side of the peninsula are generally sheltered from locally generated wind waves. The southern and southeastern portions are subject to local wind generated waves over the harbor. The Post45 Harbor Deepening study documented the following information, which is provided for general information.

.

Winds can be described by their speed, direction, and duration. The National Oceanic and Atmospheric Administration (NOAA) operates a weather station in Charleston Harbor which collects 6-minute wind data. This station records wind speed and direction at the shore. A wind rose was generated using the hourly averaged data recorded between January 2010 and December 2011 to visualize the distribution of winds which pass over Charleston Harbor (See Figure 3.3.1).

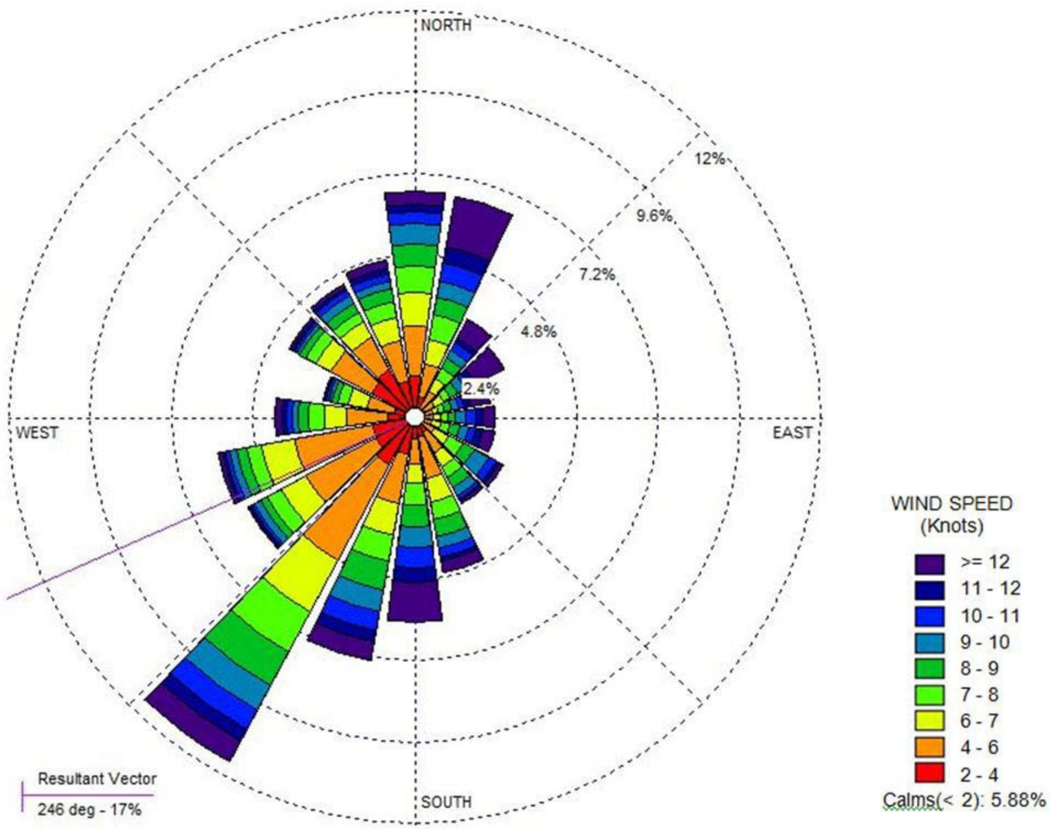


Figure 3.3.1. Wind Rose for Charleston Harbor Depicting Wind Direction and Speed Frequency

The distribution of wind speeds varies by direction (Refer to Figure 3.3.1. This figure is known as a wind rose). The total winds over Charleston Harbor, regardless of angle of approach, have the distribution by wind speed class shown in Figure 3.3.2. Three petals of the wind rose from Figure 1.5.1 are shown as frequency distributions in Figure 3.3.3. The petals selected reflect the three key directions: the largest number of winds, the highest speed winds, and those with longest fetch (distance to travel). The largest number of winds in Charleston Harbor come from the southwest, while the most high-speed winds (fastest 10% of winds) come from the north-northeast direction (Wando River). Winds entering the harbor from open ocean (south-east) have the potential to travel the furthest distance before reaching a shoreline.

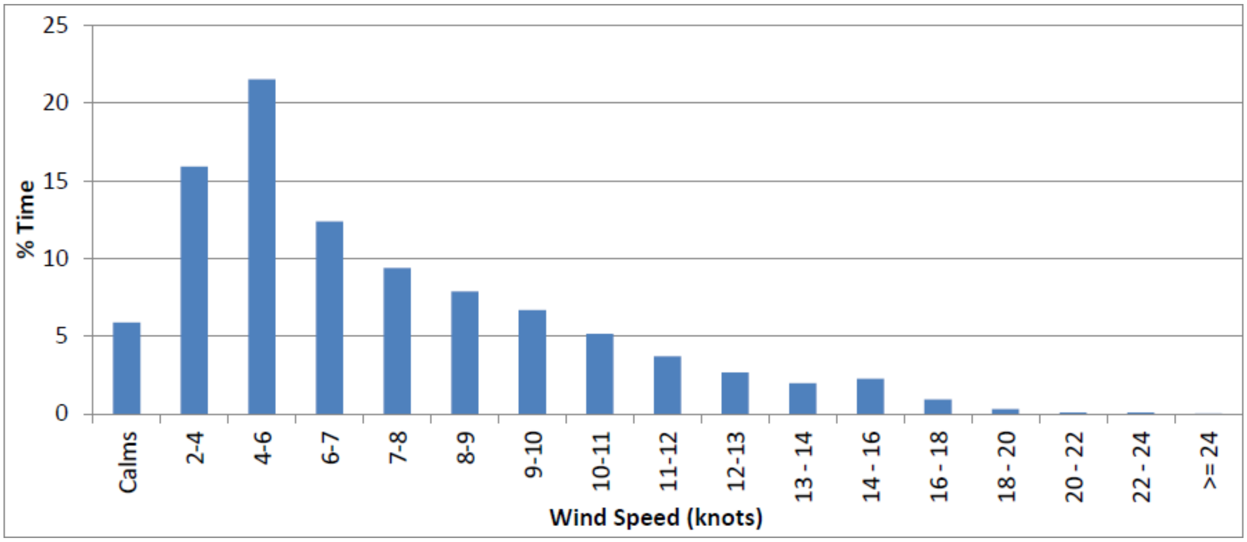


Figure 3.3.2 Wind Speed Frequency Distribution in Charleston Harbor from all directions

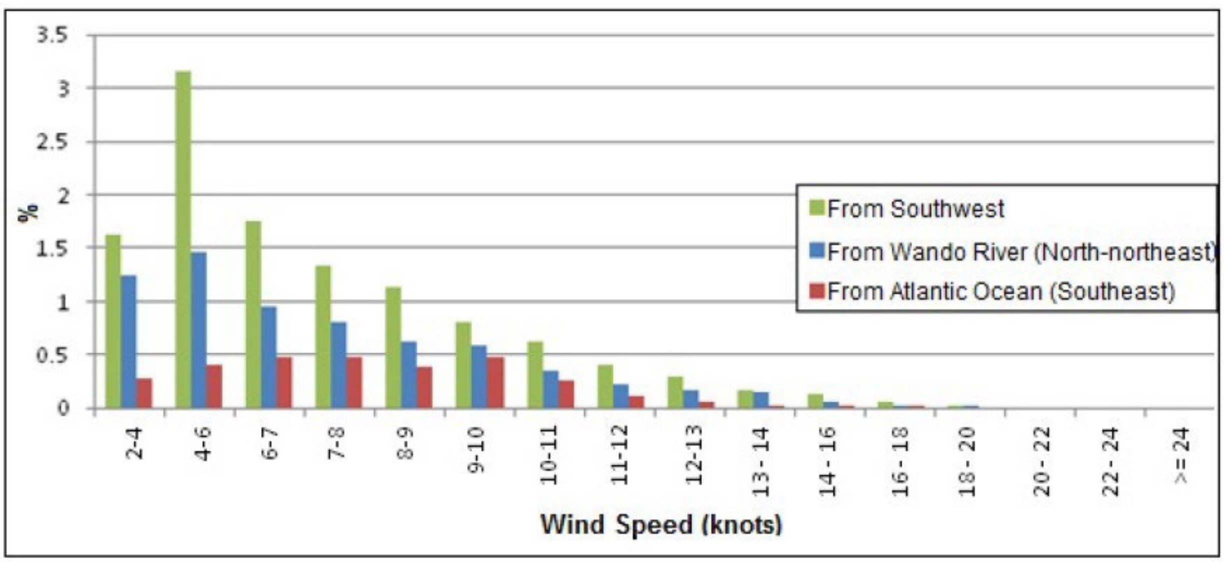


Figure 3.3.3 Wind Speed Frequency Distribution in Charleston Harbor comparing three key directions

3.4. ASTRONOMICAL TIDES & WATER LEVELS

3.4.1. ASTRONOMICAL TIDES

The Cooper River Entrance Tidal Gauge (8665530), also known as the Charleston Harbor or the Custom's House gauge is the most extensive and continuous record of tides for the City of Charleston.

3.4.2. WATER LEVELS

The Charleston Harbor tide gauge was established in 1899. In that nearly 100-year time span, local sea level has risen 1.07 ft (Fig 3.4.2.1) according to the 2017 assessment by NOAA. One way to track local impacts from sea level rise is documenting "minor coastal flooding". Commonly called nuisance, sunny day or high tide flooding, "minor coastal flooding" is a threshold from the National Weather Service that indicates when the tide has reached a certain height (7.0 ft MLLW in the Charleston Harbor). At this height, low-lying areas on land begin to flood. For example, Lockwood Blvd begins to flood at 7.2 ft MLLW (or 4.06 ft. NAVD88).

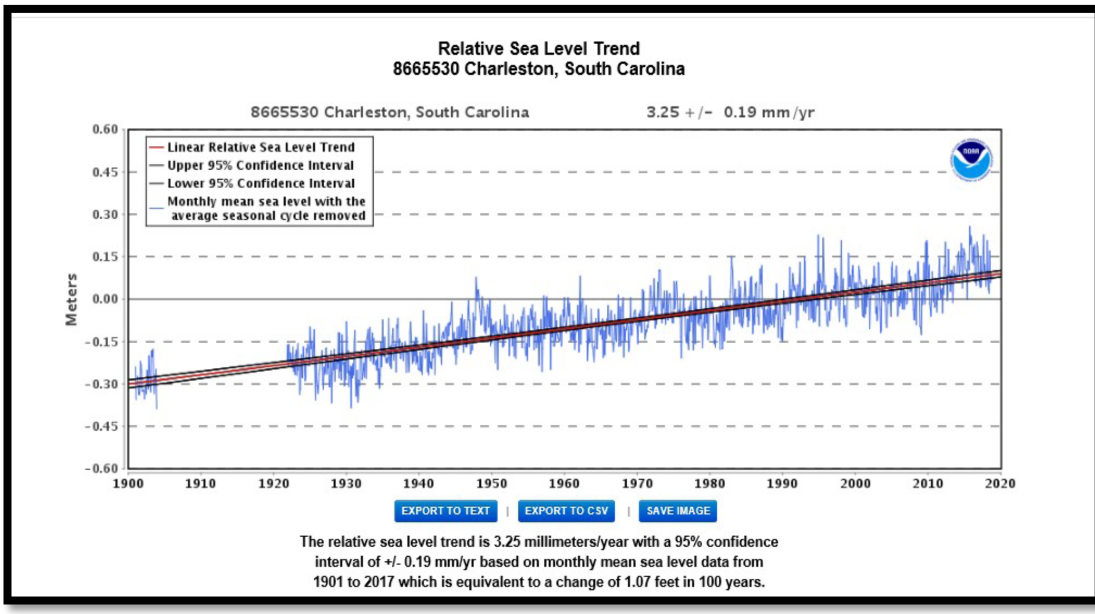


Figure 3.4.2.1 Observed Sea Level Rise at Charleston Harbor Gage

3.4.3 EXTREME WATER LEVELS

According to NOAA Tides and Currents explanation of Extreme Water Levels: Extremely high or low water levels at coastal locations are an important public concern and a factor in coastal hazard assessment, navigational safety, and ecosystem management. Exceedance probability, the likelihood that water levels will exceed a given elevation, is based on a statistical analysis of historic values. This product provides annual and monthly exceedance probability levels for select Center for Operational Oceanographic Products and Services (CO-OPS) water level stations with at least 30 years of data. When used in conjunction with real time station data, exceedance probability levels can be used to evaluate current conditions and determine whether a rare event is occurring. This information may also be instrumental in planning for the possibility of dangerously high or low water events at a local level.

Because these levels are station specific, their use for evaluating surrounding areas may be limited. A NOAA Technical Report, "Extreme Water Levels of the United States 1893-2010" describes the methods and data used in the calculation of the exceedance probability levels.

The extreme levels measured by the CO-OPS tide gauges during storms are called storm tides, which are a combination of the astronomical tide, the storm surge, and limited wave setup caused by breaking waves. They do not include wave run-up, the movement of water up a slope. Therefore, the 1% annual exceedance probability levels shown on this website do not necessarily correspond to the Base Flood Elevations (BFE) defined by the Federal Emergency Management Administration (FEMA), which are the basis for the National Flood Insurance Program. The 1% annual exceedance probability levels on this website more closely correspond to FEMA's Still Water Flood Elevations (SWEL). The peak levels from tsunamis, which can cause high-frequency fluctuations at some locations, have not been included in this statistical analysis due to their infrequency during the periods of historic record. (Source: https://tidesandcurrents.noaa.gov/est/)

High and low annual exceedance probability levels are shown relative to the tidal datum and the geodetic North American Vertical Datum (NAVD88), if available. The levels are in meters relative to the National Tidal Datum Epoch (1983-2001) Mean Sea Level datum at most stations or a recent 5-year modified epoch MSL datum at stations with rapid sea level rates in Louisiana, Texas, and Alaska. On the left of Figure 3.4.3.1 are the exceedance probability levels for the mid-year of the tidal epoch currently in effect for the station. Figure On the right are projected exceedance probability levels and tidal datum assuming continuation of the linear historic trend.

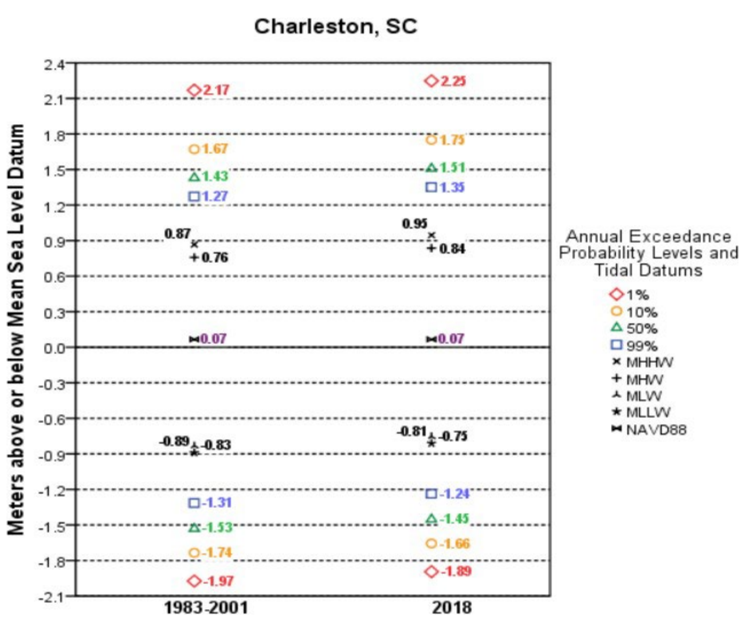
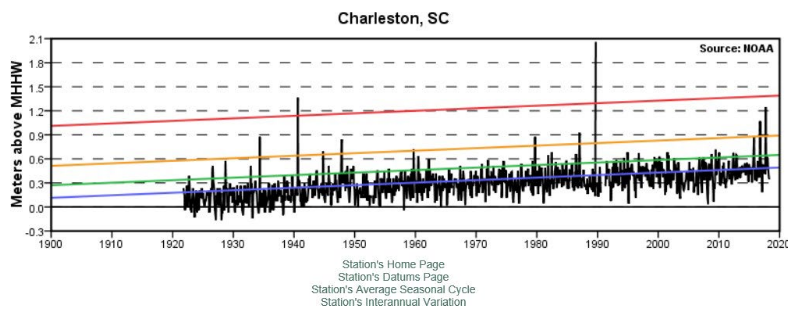


Figure 3.4.3.1 Exceedance Probability Levels and Tidal Datum of 8665530 Charleston, Cooper River Entrance, SC

Shown in Figure 3.4.3.2 the 1% level (red) indicates a 1 in 100 chance of occurring in any given year, the 10% level (orange) indicates a 10 in 100 chance of occurring in any given year , and the 50% level (green) indicates 50 in 100 chance of occurring in any given year. The 99% level (blue) indicates a high probability of occurrence every year. The level of confidence in the exceedance probability decreases with longer returns periods. Table 3.4.3.1 is tabulated in feet referenced to NAVD88. (source https://tidesandcurrents.noaa.gov/est/est_station.shtml?stnid=8665530)



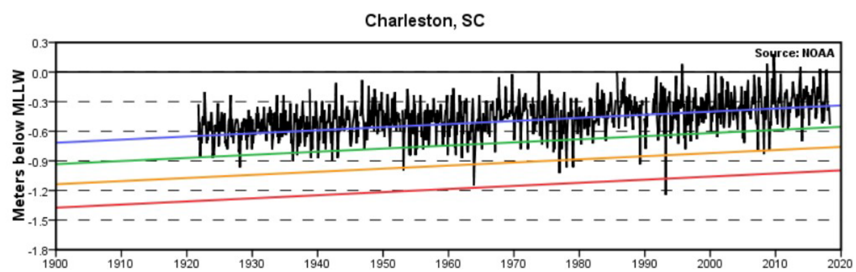


Figure 3.4.3.2 Seasonal and Interannual Variation of Gage 8665530 Extreme water Levels

Table 3.4.3.1Extreme Water levels and Tidal datum of 8665530 Charleston, Cooper River Entrance, SC

ID:	8665530
Reference Datum:	NAVD88
Name:	Charleston, SC
Established:	Sep 13, 1899
HAT:	4.12 (ft)
MHHW:	2.62 (ft)
MHW:	2.26 (ft)
MSL:	-0.22 (ft)
MLW:	-2.96 (ft)
MLLW:	-3.14 (ft)
NAVD88:	0.00 (ft)
1% AEP:	7.16 (ft)
10% AEP:	5.52 (ft)
50% AEP:	4.73 (ft)
99% AEP:	4.21 (ft)
Continuous record start:	1921
Continuous record end:	Present

Extreme events are documented by NOAA Tides and Currents website:
https://tidesandcurrents.noaa.gov/est/stickdiagram.shtml?stnid=8665530

3.5. STORMS

3.5.1. TROPICAL CYCLONES

Storms do not have to make landfall to have a flooding impact. Charleston experiences flooding from all three types of tropical cyclones: hurricanes, tropical storms, and tropical depressions. 22 storms passed within 100 nautical miles of Charleston between 2000 and present (Figure 3.5.1). The number of storms in the entire period of record will also be given, but an image would likely be too busy (156 storms passed the same area shown in the image).

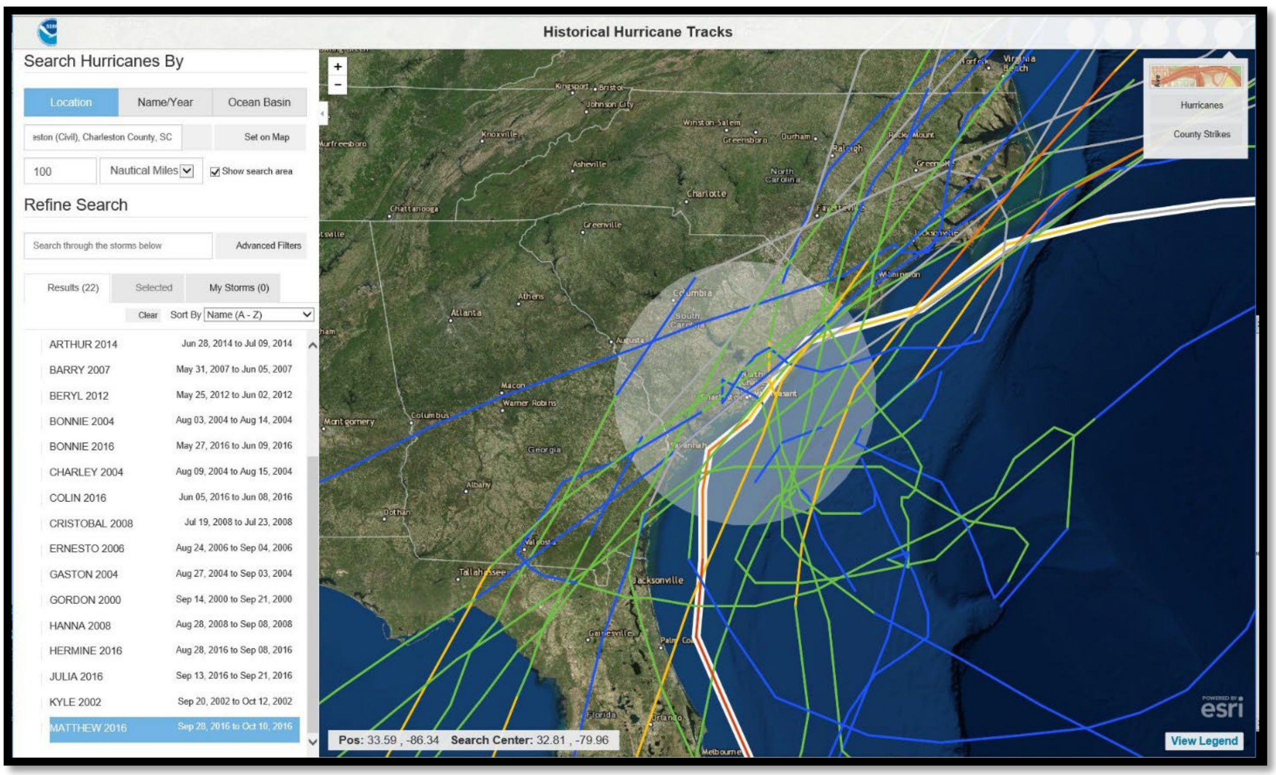


Figure 3.5.1 Twenty-two storms passed within 100 nautical miles of Charleston between 2000 and 2019.

3.5.2. HURRICANES

In the Colonial period tropical storms and hurricanes were known as "September gales," probably because the ones people remembered and wrote about were those which damaged or destroyed crops just before they were to be harvested.

One such storm that struck Charles Town on September 25, 1686, was "wonderfully horrid and destructive...Corne is all beaten down and lyes rotting on the ground... Aboundance of our hoggs and Cattle were killed in the Tempest by the falls of Trees..." The storm also prevented a Spanish assault upon Charles Town by destroying one of their galleys and killing the commander of the Spanish assault.

In autumn of 1700, "a dreadful hurricane happened at Charles Town which did great damage and threatened that total destruction of the Town, the lands on which it is built being low and level and not many feet about high water mark, the swelling sea rushed in with amazing impetuosity, and obliged the inhabitants to fly to shelter..." A ship, Rising Sun, out of Glasgow and filled with settlers had made port just prior to the storm's landfall. It was dashed to pieces and all on board perished.

Of a storm which passes inland along the coast September 7-9, 1854, Adele Pettigru Allston wrote from Pawley's Island, "The tide was higher than has been known since the storm of 1822. Harvest had just commenced and that damage to the crops in immense. From Waverly to Pee Dee not a bank nor any appearance of land was to be seen... (just) one rolling, dashing Sea, and the water was Salt as the Sea."

By 1893, major population centers could be telegraphically alerted to storms moving along the coast, but there were no warnings for the Sea Islands and other isolated areas. The "Great Storm of 1893" struck the south coast at high tide on August 28, pushing an enormous storm surge ahead of it and creating a "tidal wave" that swept over and submerged whole islands. Maximum winds in the Beaufort area were estimated to be 125 miles per hour, those in Charleston were estimated near 120 miles per hour. At least 2,000 people lost their lives, and an estimated 20,000-30,000 were left homeless and with no mean of subsistence.

Hazel (October 1954) and Gracie (September 1959) have been the most memorable storms in recent years. Hazel, a Category 4 storm, made landfall near Little River, S.C., with 106-miles per hour winds and 16.9-foot storm surge. One person was killed, and damage was estimated at $27 million.

Gracie (September 1959), a Category 4 hurricane, made landfall on St. Helena Island with 130 mph winds and continued toward the north-northwest. Heavy damage occurred along the coast from Beaufort to Charleston. Heavy rains caused flooding through much of the State and crop damage was severe. NOAA's Hurricane Re-analysis Project upgraded Gracie from a Category 3 to a Category 4 hurricane in June 2016. Tide level reached 5.0 feet NAVD88.

Hugo (September 1989) made landfall near Sullivan's Island with 120 knot winds. It continued on a northwest track at 25-30 miles per hour and maintained hurricane force winds as far inland as Sumter. Hugo exited the State southwest of Charlotte, N.C., before sunrise on September 22. The hurricane caused 13 directly related deaths and 22 indirectly related deaths, and it injured several hundred people in South Carolina. Damage in the State was estimated to exceed $7 billion, including $2 billion in crop damage. The forests in 36 counties along the path of the storm sustained major damage. Tide level reached 9.39' NAVD88. (Source https://tidesandcurrents.noaa.gov/waterlevels.html?id=8665530&units=standard&bdate=19890917&edate=19890925&timezone=GMT&datum=NAVD&interval=hl&action=)

From 1990 to 2015, South Carolina had only had five weak tropical cyclone landfalls along the coast: Tropical Storm Kyle (35 kts) in 2002, Hurricane Gaston (65 kts) and Hurricane Charley (70 kts) in 2004, Tropical Storm Ana (40 kts) in 2015, and Tropical Depression Bonnie (30 kts) in 2016. Bonnie developed north of the Bahamas and strengthened into a TS as it moves northwest toward the GA/SC coasts, eventually weakening to a TD before making landfall near Charleston. Produced heavy rainfall (widespread 3-7 inches with local amounts over 10 inches), mainly north of I-126, which led to significant flooding. During September 1999 Hurricane Floyd, a very large storm, came very close to the South Carolina coast, then made landfall near Cape Fear, North Carolina. Hurricane Floyd triggered mandatory coastal evacuations along the South Carolina coast. Heavy rain of more than 15 inches fell in parts of Horry County, S.C., causing major flooding along the Waccamaw River in and around the city of Conway for a month.

Mathew (October 2016) moved north and then northwest through the Caribbean Sea and then through the Bahamas while strengthening to a Category 4 hurricane. Tracked just off the east coast of FL and GA while weakening to a Category 1 storm before making landfall near McClellanville, SC with winds near 85 mph. Produced hurricane force wind gusts along the entire coast, significant coastal flooding from high storm tides (including a record level at Fort Pulaski), and very heavy rainfall (widespread 6 to 12 inches with locally higher amounts near 17 inches) which led to significant freshwater flooding. Tide level reached 6.14 feet NAVD88.

Irma (Sep 2017) made landfall in the Florida Keys as a Category 4 hurricane and then moved along the southwest coast of Florida as a Category 3 hurricane. The storm then moved north near the west coast of Florida while weakening to a tropical storm before moving into southwest Georgia and continuing to weaken. Produced significant coastal flooding, wind gusts near hurricane-force along with 4 tornadoes, flooding rainfall and river flooding across southeast SC/GA. NOAA tide level reached elevation 6.71 feet NAVD88.

Florence (Sept 2018) made landfall near Wrightsville Beach, NC as a Category 1 hurricane before slowing down and weakening to a TS. The storm then moved southwest near the northern SC coast before shifting west toward the SC Midlands and weakening to a TD. Produced some tropical storm force wind gusts and several inches of rain, mainly north of Charleston.

Michael (October 2018) made landfall near Mexico Beach, FL as a Category 4 hurricane and then moved northeast through southwest GA as a hurricane before weakening to a TS before reaching central SC. Produced tropical storm force winds and several inches of rainfall across much of southeast SC/GA which led to many fallen trees and some power outages.

Dorian (Sept 2019) strengthened to a Category 3 hurricane as it traveled along the Gulf Stream, offshore of the coasts of GA and SC. Produced sustained winds of 45 to 55 kt with gusts up to 77 kt. Storm surge created inundation in SC up to 4 ft and peak rainfall was measured to be 15.21 inches at Pawleys Island.

Isaias (August 2020) strengthened to a hurricane 100 nm south of Charleston and later made landfall at Ocean Isle, NC with a peak intensity of 80 kt. The storm produced 5-7 inches of rain in SC and over 7 ft of inundation in some areas.

3.5.3. HISTORICAL STORMS

A historic flooding event affected the Carolinas from October 1-5, 2015. A stalled front offshore combined with deep tropical moisture streaming northwest into the area ahead of a strong upper level low pressure system to the west and Hurricane Joaquin well to the east. This led to historic rainfall with widespread amounts of 15-20 inches and localized amounts over 25 inches, mainly in the Charleston tri-county area. Flash flooding was prevalent and led to significant damage to numerous properties and roads and many people having to be rescued by emergency personnel. In addition, tides were high due to the recent perigean spring tide and persistent onshore winds, exacerbating the flooding along the coast, especially in downtown Charleston.

3.6. CLIMATE CHANGE IMPACTS

Climate change is defined as a change in global or regional climate patterns. Climate change has already been observed globally and in the United States. These included increases and changes in air and water temperatures, reduced frost days, increased frequency and intensity of heavy downpours, a rise in sea level, and reduced snow cover, glaciers, permafrost, and sea ice. Climate change has the potential to affect all of the missions of the United States Army Corps of Engineers (USACE). USACE mission in regard to climate change is: “To develop, implement, and assess adjustments or changes in operations and decision environments to enhance resilience or reduce vulnerability of USACE projects, systems, and programs to observed or expected changes in climate”. The USACE’s Climate Change Program develops and implements practical, nationally consistent, and cost-effective approaches and policies, to reduce potential vulnerabilities to the Nation’s water infrastructure resulting from climate change and variability.

The Corps has the following guidance to assist in the assessment of Climate Change Impacts on a proposed project.:

- ER 1105-2-101 Risk Assessment for Flood Risk Management Studies, 2019.
- EM 1110-2-6056, Standards and Procedures for Referencing Project Elevation Grades to Nationwide Vertical Datums. 2010.
- EP 1100-2-1, Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation. 2020.
- ECB 2018-2, Implementation of Resilience Principles in the Engineering & Construction Community of Practice 2018.
- ECB 2018-14, Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects.

The Department of the Army Engineering Regulation 1100-2-8162 (31 Dec 2013) requires that future Relative Sea Level Change (RSLC) projections must be incorporated into the planning, engineering design, construction, and operation of all civil works projects. The structural components of the proposed alternatives in consideration of the “low”, “intermediate”, and “high” potential rates of future RSLC were evaluated. This range of potential rates of RSLC is based on the findings of the National Research Council (NRC, 1987) and the Intergovernmental Panel for Climate Change (IPCC, 2007).

3.6.1. EVALUATION OF RELATIVE SEA LEVEL CHANGE (RSLC)

RSLC considers the effects of (1) the eustatic, or global, average of the annual increase in water surface elevation due to the global warming trend, and (2) the “regional” rate of vertical land movement (VLM) that can result from localized geological processes, including the shifting of tectonic plates, the rebounding of the Earth’s crust in locations previously covered by glaciers, the compaction of sedimentary strata and the withdrawal of subsurface fluids (USGS 2013). A vertical Land Movement assessment at Sullivan’s Island by NASA/Jet Propulsion Lab indicated a very small (0.001 ft/yr) based on 1998-2004 data. Technical Report NOS CO-OPS 065, Estimating Vertical Land Motion from Long-Term Tide Gauge Records in 2013 indicated a -1.24mm/yr (0.004 ft/year) for Charleston.

The USACE Sea-Level Change Curve Calculator (Version 2021.12) is applied for the Charleston Gage 8665530 shown in figure 3.6.1. The year 1992 is used to start these curves because 1992 is the center year of the

NOAA National Tidal Datum Epoch of 1983–2001. The National Tidal Datum Epoch is the period used to define tidal datums (Mean High Water, for instance, and local MSL)

Figure 3.6.1 Location Charleston Gage 8665530

The historic rate of future RSLC (or USACE Low Curve) is determined directly from gage data gathered in the vicinity of the project area. RSLC is predicted to continue in the future as the global climate changes. According to National Oceanographic and Atmospheric Administration (NOAA) for the Charleston Gage 8665530, NOAA's 2006 Published Rate is 0.01033 feet/yr. However, more recent updates to the National Oceanographic and Atmospheric Administration (NOAA) for the Charleston Gage 8665530 is shown in Figure 3.6.2 for the period of record 1901 to 2017, which indicates 1.07 feet in 100 years. The rate for the "USACE Intermediate Curve" is computed from the modified NRC Curve I considering both the most recent IPCC projections and modified NRC projections with the local rate of vertical land movement added. The rate for the "USACE High Curve" is computed from the modified NRC Curve III considering both the most recent IPCC projections and modified NRC projections with the local rate of vertical land movement added. See the Coastal Sub-Appendix for more discussion on the methodology to compute the intermediate and high rates of sea level change.

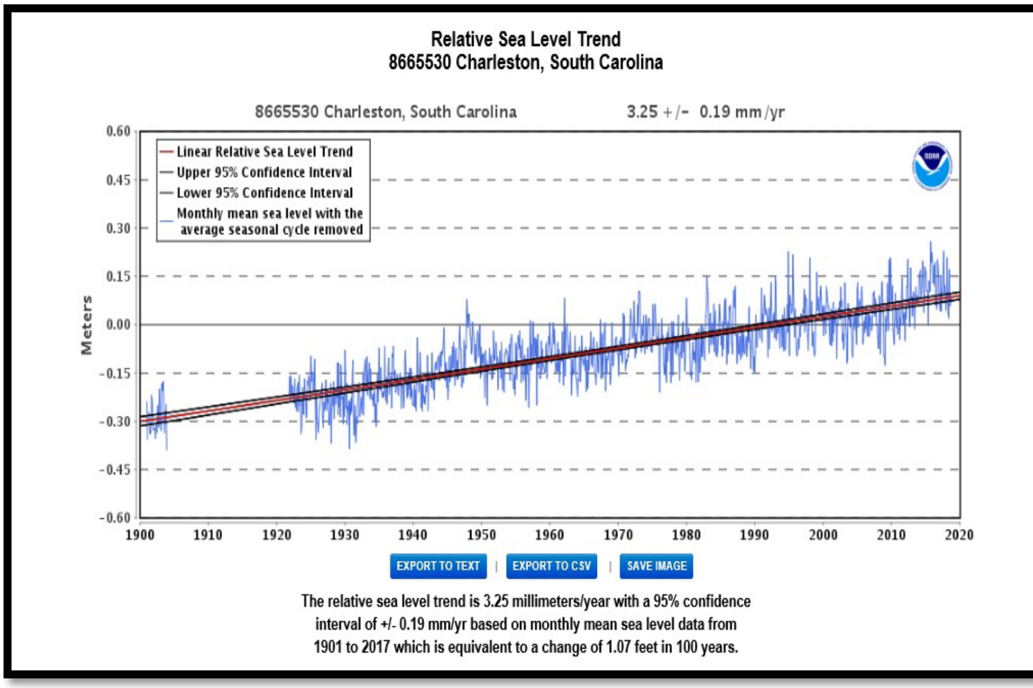


Figure 3.6.2 Relative Sea Level Trend

Figure 3.6.3 and Table 3.6.1 show the results of the Estimated Sea Level Change from the USACE Sea Level Change Curve Calculator (2021.12).

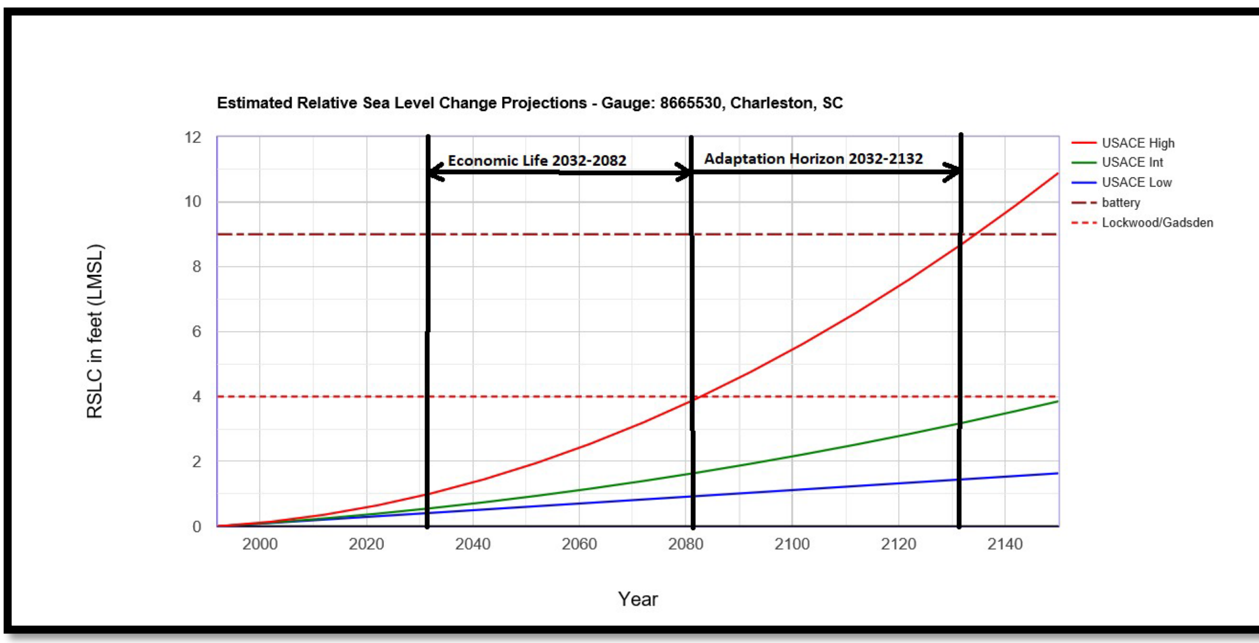


Figure 3.6.3 Low, Intermediate and High Sea Level Projection Gage 8665530

Table 3.6.1 Estimated Relative Sea Level Change

Gauge Status: Active and compliant tide gauge
Epoch: 1983 to 2001
8665530, Charleston, SC
NOAA's 2006 Published Rate: 0.01033 feet/yr
All values are expressed in feet relative to LMSL

Year	USACE Low	USACE Int	USACE High
1992	0.00	0.00	0.00
2002	0.10	0.11	0.14
2012	0.21	0.24	0.36
2022	0.31	0.39	0.64
2032	0.41	0.56	1.01
2042	0.52	0.74	1.44
2052	0.62	0.94	1.96
2062	0.72	1.16	2.54
2072	0.83	1.40	3.20
2082	0.93	1.65	3.93
2092	1.03	1.92	4.74
2102	1.14	2.21	5.62
2112	1.24	2.52	6.58
2122	1.34	2.85	7.61
2132	1.45	3.19	8.71

The proposed project has an estimated construction completion in the year 2032. That would be a change in sea level of 0.41 feet for low rate of sea level rise, 0.56 for intermediate rate of sea level rise and 1.01 feet for high rate of sea level rise. USACE guidance suggests a 50-year economic life and 100-year adaptation horizon. In 2082 (50-year economic life) the low rate of sea level change is 0.93 feet; the intermediate rate is 1.65 feet and the high rate of sea level rise is 3.93 feet. The 100-year adaptation horizon (year 2132) is projected to be 1.45 feet, 3.19 feet and 8.71 feet for the low, intermediate, and high, respectively. (Table 3.3.2.1).

Portions of Lockwood Dr, a primary road to the Medical District, are at elevation 5 NAVD88, with small portion at elevation 4 NAVD88. Gadsden Creek has connections to Hagood Ave and Fishburne, which have elevation 4 NAVD88. Based on the high rate of sea level change, high tide would flood these areas twice a day around the year 2085 (near the end of the economic life of the project) , and for the intermediate rate of sea level change in the year 2150. The battery is overtopped at every high tide with a high rate of sea level rise around the year 2035. Based on the NWS threshold for “King tides” at 3.46 NAVD88 would occur every tide by year 2145 based on an intermediate rate of SLC.

3.6.2 SELECTION OF SEA LEVEL CHANGE FOR ANALYSIS

ER 1100-2-8162 allows for the identification of a preferred alternative under one rate of sea level change and then evaluate performance under all three rates of sea level change. Consideration of sensitivity to sea level rise according to ER 1110-2-8162 and EP 1100-2-1 would not change the selection of an alternative since the alternatives were a wall with breakwater or wall without breakwater. The elevation of the wall and breakwater are scales of the alternatives. Using the different SLR only affects the exceedance probability of a selected elevation. There is not a targeted annual exceedance probability level for the project because the physical constraints of city infrastructure, bridges, topography, and ongoing “low” battery wall reconstruction, limit the maximum elevation considered in the study to elevation 12 NAVD88. Therefore, the determination of the recommended alternative and the measure of elevation was modeled using one rate of sea level change. It was determined that nonlinear effects of sea level rise in the Charleston area were negligible, so water levels resulting from the high and low USACE sea level change curves can be estimated by linear addition.

Using the USACE Sea Level Tracker (https://climate.sec.usace.army.mil/slr_app/) Figure 3.6.4 indicates trend of the last thirty years, which began lower than the historic trend and around 2006 to 2008 transitioned closer to the intermediate rate.

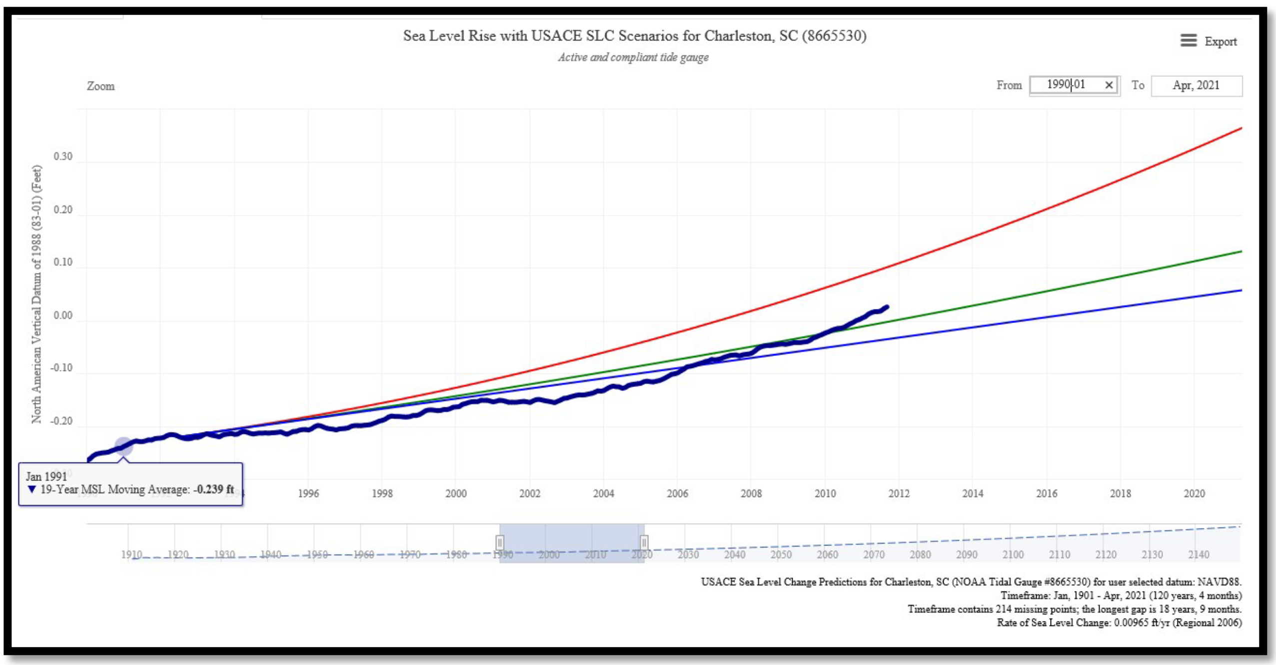


Figure 3.6.4 Sea Level Tracker Charleston SC (NOAA Station 8665530)

Alternatives were evaluated using the most likely SLR of the intermediate rate. Intermediate was selected because the historic trend is changing. Using a historic rate was not deemed prudent when it can be observed to be changing and increasing. Also, the relative sea level trend indicated a higher historic rate than the 2006 sea level trend – indicating a trend in increase but not sufficient to warrant using the high rate of sea level rise. Consideration of sensitivity to sea level rise according to ER 1110-2-8162 and EP 1100-2-1 would not change the selection of an alternative since the alternatives were a wall with breakwater or wall without breakwater. The elevation of the wall and breakwater are scales of the alternatives. Using the different SLR only affects the exceedance probability of a selected elevation, there is not a targeted annual exceedance probability level. The physical constraints of city infrastructure (bridges, topography, and ongoing battery wall work) limit the maximum elevation considered in the study to elevation 12NAVD88.

The future condition for the economic considerations is 50 years after construction completion which is estimated to be 2032. Table 3.6.1 indicates the incremental rate of sea level rise for the 50-year project life ending in 2082, as well as the 100-year project into the future (Year 2132) as 1.65 feet and 3.19 feet since

1992 for the intermediate rate of RSLC. All three sea level rise scenarios will be applied in G2CRM to address the benefits and damages of the selected wall elevation. These are discussed in the Economics Appendix.

CHAPTER 4 COASTAL STORM MODELING

4-1 MODELING

As previously stated, there were no existing USACE studies addressing Coastal Storm Risk Management. USACE reached out to SCDNR, the FEMA POC for Flood Insurance Studies (FIS) in the state of SC, for available coastal models to minimize costs and improve efficiencies of the study. FEMA/SCDNR contractor, AECOM, provided ADCIRC models, storm sets, SWAN, STWAVE runs, all the validation runs, production runs and input for their 2017 preliminary FIS. This data was provided to ERDC for analysis. In order to better capture the results of any structural measures of the study, the ADCIRC grid needed to be modified within the study area and ADCIRC rerun for a suite of storms. ERDC evaluated the suite of storms provided by AECOM and selected a subset of storms. The goal of storm selection was to find the optimal combination of storms given a predetermined number of storms to be sampled (e.g., 20 Tropical Cyclones (TC)), referred to as reduced storm set (RSS). In the process of selecting 20 TCs, it was determined that an RSS of this size adequately captured the storm surge hazard for the range of probabilities covered by the FEMA Storm Set (122 TCs). In order to also include high frequency events, five (5) additional storms were selected from the range of probabilities determined from EVA of water level measurements. Details are found in the ERDC Coastal Modeling subappendix.

ERDC was asked to run STWAVE and ADCIRC for three scenarios to generate time series still water elevations for input into the G2CRM model. The three scenarios were: existing, future without and future with a breakwater as a wave attenuator. This analysis is discussed in detail in the ERDC Coastal Modeling subappendix and ultimately led to elimination of the breakwater as an alternative.

Coastal analysis generates the still water elevation. As stated in the FIS, "the still water surge elevation is the water elevation due solely to the effects of the astronomical tides, storm surge, and wave setup on the water surface but which does not include wave heights. The inclusion of wave heights, which is the distance from the trough to the crest of the wave, increases the water-surface elevations. The height of a wave is dependent upon wind speed and duration, depth of water, and length of fetch. The wave crest elevation is the sum of the still water elevation and the portion of the wave height above the Stillwater elevation. "

As explained in the SOUTH CAROLINA STORM SURGE PROJECT DELIVERABLE 3: PRODUCTION RUNS, FINAL STATISTICS, AND RESULTS ANALYSIS report generated by URS for FEMA/SCDNR. "The tide range in South Carolina is up to 6 feet (ft), suggesting that the tide phase at the time of landfall may significantly influence the surge levels produced by a given storm." The report states that simulations were run to estimate the influences of steric effects on water levels throughout the project area and ultimately determined that these fluctuations could obtain a total increase of 2.75 inches above MSL. Therefore, steric effects were minimal compared to the magnitude of tides.

See the ERDC Coastal Modeling subappendix for the ERDC modeling report that includes the STWAVE and ADCIRC modeling used to select FWP alternatives.

The G2CRM was the tool used to evaluate the alternatives (stand-alone wall or wall plus breakwater) and scales of alternatives (different wall elevations and different breakwater sizes). Driving forces of the G2CRM are the still water hydrograph elevations generated in meters at MSL by ADCIRC and STWAVE. These were then converted to feet MSL for input into G2CRM. The G2CRM model then uses the difference in MSL to NAVD88 to keep all analyses to the NAVD88 datum. In addition to the driving forces from ADCIRC and STWAVE, G2CRM uses local tidal stations for the addition of tide, and the three USACE sea level formulas are embedded in G2CRM to include future sea level conditions. This data was then used to compare FWO conditions to the wall footprint at various measures of wall elevations. After evaluation of wall footprint and elevations as a stand-alone option (Alternative 2) and in conjunction with a breakwater wave attenuator (Alternative 3), it was concluded that the stand-alone wall at elevation 12NAVD88 was the recommended plan.

After Optimization of the footprint and selection of elevation 12 NAVD88 (discussed in Chapter 5) the recommended wall structure was incorporated into the ADCIRC/STWAVE models and evaluated for impacts outside the project area based on the year 2032 and 2082 rates of intermediate sea level rise.

4.2 RESULTS

After optimization of the footprint to reduce environmental impacts, minimize impacts to personal property while reducing costs by relocating the wall on high ground to utilize a T-wall rather than the combo wall, the wall at elevation 12 ft NAVD88 was added to the ADCIRC/STWAVE mesh for evaluation of impacts to surrounding areas.

The final recommended structures were incorporated into the ADCIRC and STWAVE models and evaluated for impacts outside the project area for the intermediate rate of sea level rise for the year 2032 (0.56 ft), after initial construction and for 2082 (1.65 ft), the end of its economic life. This methodology corresponds to the methodology used for the interior hydrology assessment detailed in Sub-AppendixSub-Appendix Interior Hydrology. Becausenonlinear residual (NLR) was proven to be very weak, effects shown by changes in sea level between the 2032and 2082 can be applied to other sea level rise scenarios.

ADCIRC was coupled with STWAVE to model 11 synthetic storms for each sea level rise scenario and each project condition, where the future without project (FWO) condition was modeled using the ADCIRC and STWAVE meshes described in Coastal Subappendix Section 4-2. The future with project (FWP) condition was modeled using the same ADCIRC and STWAVE meshes, manipulated to include a 12 ft NAVD88 wall surrounding the peninsula (Figure 4.1). The 11 storms were chosen from the storm suite to represent a wide distribution of storm sizes and patterns. This reduction in storm suite saved computational time and cost by reducing the required number of simulations to 44, while providing sufficient data to compare sea level rise scenarios and project conditions.

Figure 4.1. ADCIRC mesh used for FWP simulations with proposed 12 ft NAVD88 wall shown in light green.

Based on simulations completed using the FWO and FWP conditions, presence of the wall caused minimal effect on water levels due to storm surge in surrounding areas. Some simulations showed up to a 1 to 2-inch increase in water levels for the FWP condition in some surrounding areas. However, this change in water levels is within the accuracy of the model itself and can be considered minimal. These increases were only seen in small areas during simulations for larger storms that overtopped the wall (12+ ft of storm surge), soareas with an increase of 1 to 2 inches would typically already be experiencing several feet of inundation.

Other than these sparse cases of 1 to 2 inch increases, the increase in water levels to surrounding areas is typically less than 1 inch, while the reduction in water levels within the wall in the FWP condition is typically on the order of several feet.

Local wind waves within the Charleston riverine and estuary nearshore area will be limited in wave height and period by the limited fetches. These waves will be dissipated by marshes and shallow foreshore areas before encountering the wall which will scatter the remaining waves, causing them to dissipate within a few wavelengths. Scattering is due to directional/frequency spread of the short-period waves, irregularities in the wall, near-wall bathymetry, adverse wind (wind blowing against the reflected waves), and complex bathymetry of the far-field (river channels/nearshore). As supported by results in the STWAVE simulations, reflection and refraction of waves encountering the wall will have no effect on surrounding areas.

CHAPTER 5 ENGINEERING EVALUATION

5.1. GENERAL

Model Areas (MA) were needed by Economics to break city into manageable areas for G2CRM assessments. The determination of MA boundaries considered topography and the drainage pathways of the various areas, as well as land use (i.e. the Columbus Street Terminal had to remain whole). The Model Areas were identified by the primary land use of the area.

- Wagener Terrace: Identified as Wagener Terrace for the large residential area, covers the area from the upper limit of the study area on the Ashley side around the Wagener Terrace area to Citadel -which is high ground, - includes commercial, undeveloped, and residential land use.
- Marina: Identified as Marina due to the public marina along the shoreline, covers from Citadel to Low Battery (by the Coast Guard) and includes residential and hospital areas.
- Battery – identified as Battery because it follows the low and high battery walls, extends from Coast Guard to the end of the High Battery by the Historic Foundation and Yacht club. This area is characterized by much of the historic homes.
- Port: Identified based on the large SCPSA port facilities along the shoreline extends from High Battery end at the historical foundation/Yacht Club to just past Columbus Terminal. The area includes historic homes, commercial, port areas.
- Newmarket: identified by the historic creek that drains much of the areas extends from Columbus Terminal across Newmarket creek to the upper limit of the study area on the Cooper side. And includes - residential (low income), commercial properties.

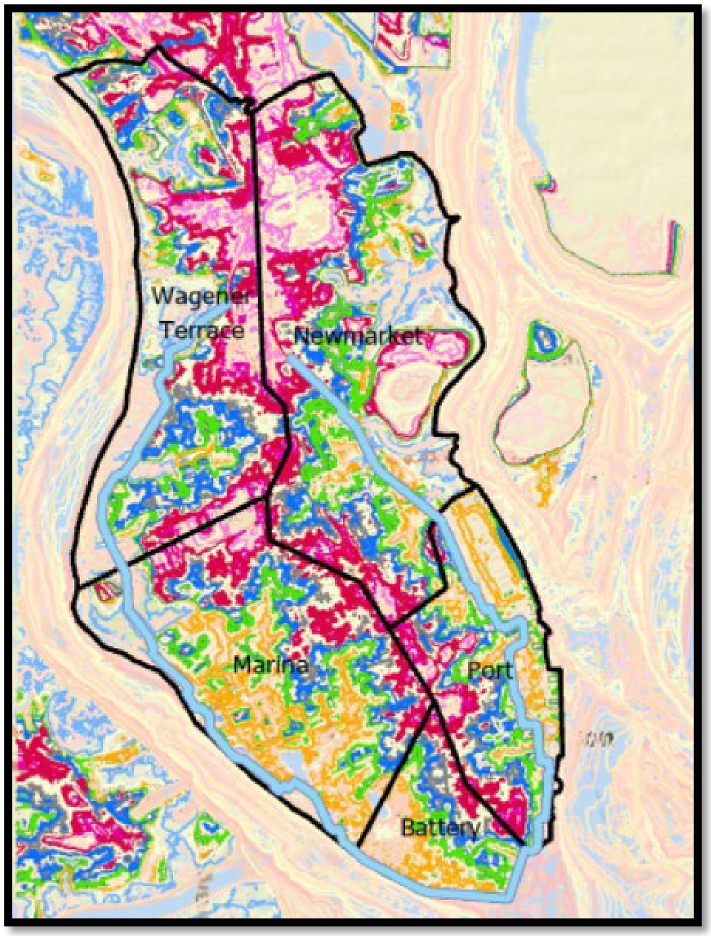


Figure 5.1.1 Map depicting Model Areas

5.2. ADCIRC WATER LEVELS

From the dataset of over 1000 points, 5 were selected to represent the Model Areas used for G2CRM (Figure 4.5). The G2CRM was the tool used to evaluate the alternatives (wall only or wall plus breakwater) and scales of alternatives (different wall elevations and different breakwater sizes). Based on the driving forces from ADCIRC and STWAVE storm hydrographs (surge and waves) levels generated with ADCIRC and STWAVE, combined with the tide information and intermediate rate of sea level rise the elevation of 12 NAVD88 was the scale of the alternative 2 (wall only) selected based on G2CRM analysis. G2CRM uses local tidal stations for the addition of tide and the three USACE sea level formulas are embedded in G2CRM to include future sea level conditions.

Figure 5.2.1 Location of Save points for the Model Areas

5.3. PROJECT ALIGNMENT

The primary criteria were to avoid personal property for footprint and avoid taking houses/businesses unless there is no other option. Only existing and known permitted structures were considered. Additional criteria were to take advantage of existing topography, consider the actions undertaken by the city and to consider the following construction and maintenance easements in Table 5.3.1. The elevation of the wall was selected to be Elevation 12 NAVD88 through the economic analysis. Further optimization of the footprint since the Tentatively Selected Plan to minimize wetland impacts and reduce construction costs resulted in relocating the wall to the final footprint. In addition, due to a request from the South Carolina Ports Authority, the wall alignment was adjusted slightly to include the Columbus Street and Union Pier terminals behind the protection of the wall. In some location the construction and maintenance easements were not met,however these small reaches can be accommodated with shoring of the trench, use of micropiles and other conditions in small, specific locations.

Table 5.3.1 Typical Permanent and Construction Easements.

	Permanent Easement		Construction	
Feature	**Riverside (from CL)**	**Landside (from CL)**	**Riverside (from CL)**	**Landside (from CL)**
T-Wall	16 feet	25 feet	35 feet	35 feet
Combo Wall	16 feet	25 feet	65 feet	35 feet

These criteria resulted in the following eliminations and assumptions:

1. Storm Surge Protection structure type: An earthen levee embankment was eliminated as a form of protection due to the footprint of an earthen levee. The study is limited to the peninsula of Charleston, where the land has been heavily developed, and available land is very scarce. Therefore, if an earthen levee were to be constructed, it would result in acquisition of many homeowners' propertiesbased on the following criteria:
   - Minimum top-width should be 10' (for access along top)
   - Side slopes should not be steeper than 1 vertical on 3 horizontal (1V:3H) for maintenance concerns; side slopes should be flattened if access may be limited or equipment tipping hazard exists (i.e. mowing equipment tipping and falling into adjacent body of water). (see Table 5.3.1)
   - Marsh soils would be unable to support an earthen embankment without reinforcement. To obtain the desired elevation, it would also have a large footprint with resulting adverse environmental impacts to marshes. The marshes provide valuable habitat and also provide reduction of shoreline erosion. The study wanted to minimize impacts to wetland marshes.
   - A vegetation-free zone (VFZ) is needed:
     - Provides reliable corridor of access / assures adequate access for inspections and flood-fighting.
     - Provides buffer between structure and vegetation so vegetation doesn't harm or reduces potential of harm on structure.
     - 15' beyond levee toes

Table 5.3.2 Levee Footprint Requirements

Berm Height (ft) Above Existing Grade	10 ft Top Width		8 ft Top Width	
	3H : 1V	**4H : 1V**	**3H : 1V**	**4H : 1V**
	Total Width (ft)	**Total Width (ft)**	**Total Width (ft)**	**Total Width (ft)**
1	46	48	44	46
2	52	56	50	54
3	58	64	56	62
4	64	72	62	70
5	70	80	68	78
6	76	88	74	86
7	82	96	80	94
8	88	104	86	102
9	94	112	92	110
10	100	120	98	118

11	106	128	104	126
12	112	136	110	134
13	118	144	116	142
14	124	152	122	150

* Total Widths include a Vegetation Free Zone (VFZ) of 15 ft on each side of the berm

2. Storm Surge Wall on Land: T-wall was assumed for all new construction, although during optimization consideration will be given to I-wall. Due to the poor nature of the soils in Charleston, it is assumed that the T-Wall will be founded on a deep pile foundation that will be embedded within the Cooper Marl stratum. Based on available data, this strata is roughly 60 to 80 feet below current finished grade and consists of medium dense silty sand to firm silty clay.
    - A vegetation-free zone (VFZ) is needed (see Figure 5.3.1):
        - Provides reliable corridor of access / assures adequate access for inspections and flood-fighting.
        - Provides buffer between structure and vegetation so vegetation doesn't harm or reduces potential of harm on structure.
        - 15' beyond footing the wall stem or 8' beyond the footing, whichever is greater.

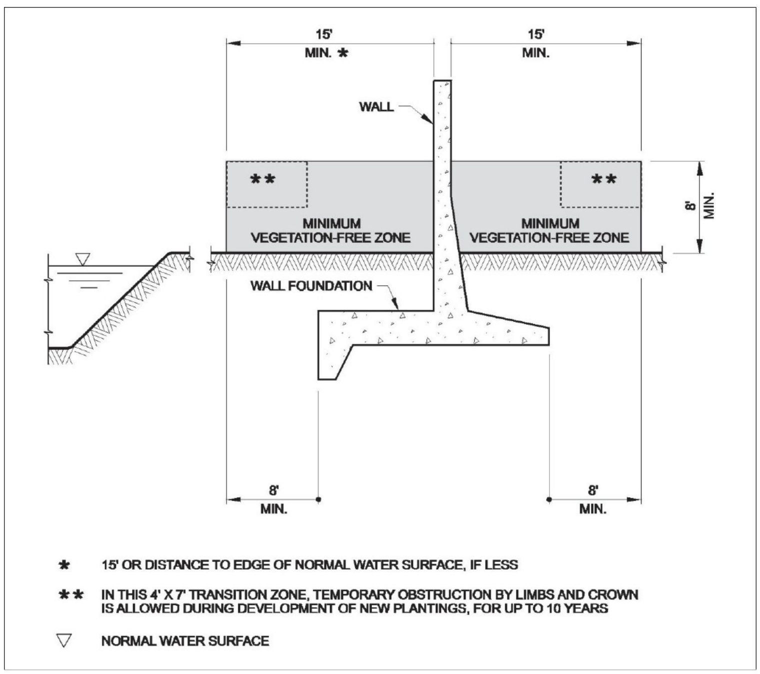


Figure A-16. Inverted-T Type Floodwall.

- Figure 5.3.1 Typical Vegetation-Free Zone Requirements for T-Wall (taken from USACE EP 1110-2-18, 1 May 2019)

3. Storm Surge Combo Wall in marsh: The excavation construction footprint in many areas would have required taking houses. Additionally, consideration of construction needs, proximity to homes, and vegetation free zone requirements lead to placement in the marsh for some areas. The type of wall assumed was a combo wall similar to the Norfolk and New Orleans projects. A Combo Wall is a combination of a large-diameter piles with sheet piles installed to form a surge barrier structure. Due to soil conditions and required loads, the Combo Wall will require batter piles to provide sufficient lateral support.

- A vegetation-free zone / vegetation-management (VFZ) is needed:
  - Prevents large trees from growing close to the wall so trees don't harm or reduces potential of harm on structure. Trees will be required to be removed within a zone of 15' on either side of the combo wall.
  - With combo wall being located in the marsh, the natural salt marsh vegetation (spartina or salt marsh cordgrass) will be allowed to grow naturally around the wall. It is not anticipated that the spartina will have a negative impact to the performance of the combo wall. There may be times in which the spartina is cut adjacent to the combo wall to facilitate inspections.

4. Bridge Clearances: Where the barrier goes under existing bridges, clearances for construction were taken into consideration when selecting a deep foundation system, as well as construction methods used. Micropiles will be utilized where clearance is low in the location of the T-Wall; and welding of steel sheet piles will be utilized where clearance is low for Combo Walls. Below are 3 locations where head clearance is a concern. While these solutions are more costly, it is anticipated that they are much for cost effective than altering the existing bridge path.

   - James Island Connector
   - US 17, along Lockwood Dr
   - US 17 Ravenel Bridge along Morrison Dr.

5. Utilities: Utility information obtained included water, sewer, storm drainage and gas. Additional details on utilities will need to be obtained during PED. The known utilities were considered when optimizing the project alignment. A high contingency in the cost estimate was included to account for the unknowns. The figure below represents that utility information obtained.

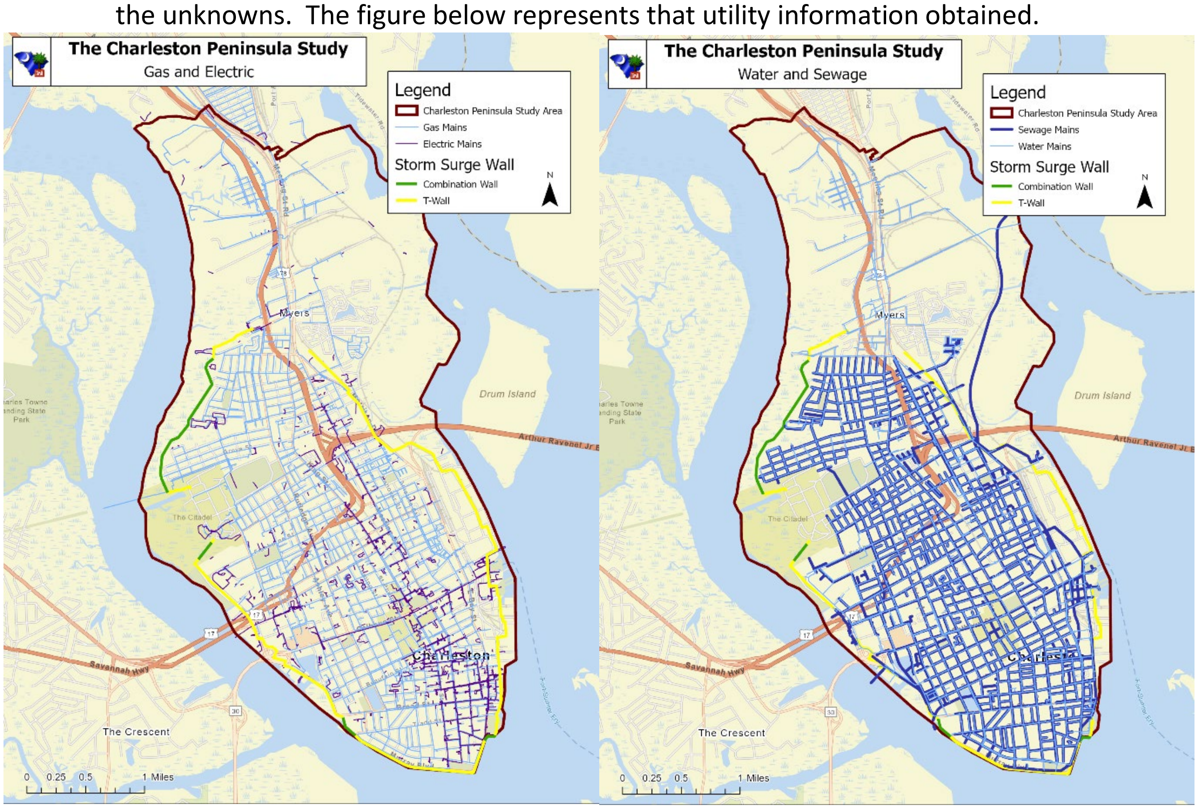


Figure 5.3.2. Utility Dataset Used in Project Alignment Optimization and Cost Considerations

The City provided a first draft of what they considered area to be inside the protection structure. It included only areas of existing development. All new development would be elevated per city and FEMA criteria, so there would not be damages; therefore, no additional benefits to the federal government. Additionally, WRDA 1990, Section 308 states that the Secretary shall not include in the benefit base for justifying Federal flood damage reduction projects – (1)(A) any new or substantially improved structure (other than a structure necessary for conducting a water-dependent activity) built in the 100-year flood plain with a first floor elevation less than the 100-year flood elevation after July 1, 1991.

Small areas of development within the study area such as Rosemont and BridgeView are excluded from being inside the wall, and are addressed by nonstructural solutions, such as floodproofing and elevating structures. Identified in Table 5.3.3, these were considered in the cost estimate.

Table 5.3.3 Nonstructural Solutions for the Study Area

Description
1. Bridgeview Community - Flood proof 38 condominium buildings that are approximately 250 feet in a rectangular structure. Water resistant sealant up to 3ft and replacing/waterproofing approximately 8 doors/16 windows in each structure.
2. Rosemont Community - Flood proof 43 homes with 3ft of water-resistant sealant and replacing 2 doors/4 windows in each structure. Approximately 1500 sq feet per home.
3. Rosemont Community – Raise 66 homes approximately 6 feet. All wooden structures. Approximately 1500 sq feet per home.
4. Lowndes Point – Raise pump station 12 feet.
5. City Marina – Flood Proof historic Rice Mill building with water resistant sealant and replacing approximately 4 doors and 12 windows. Approximately 5000 sq feet for lower level.

5.3.1 OPTIMIZATION CHANGES

After the release of the draft report which, the footprint of the wall was re-evaluated. Efforts were made to reduce costs and minimize impacts to wetlands by moving the wall out of the marsh. The effort to minimize real estate requirements of private property were addressed by utilizing city and state property by placing the footprint in parks and along roadways within SCDOT right of way.

The northwest end of the wall was relocated to land tying into Petty St. The wall in the marsh ties into land at the northern end of Citadel near Grier St. The next segment starts at the southern end of Citadel near Register Rd , crosses the marsh to connect near the Joe Baseball stadium, and then travels along Lockwood Dr under Highway 17, crosses the road to MUSC parking lot, passing under James Island connector to allow the James Island connector to remain accessible from James Island and back to the marina where it continues down Lockwood Dr. It then crosses into the marsh offshore of the Coast Guard base before connecting to the battery. From the high battery the wall will cross the water offshore of the Historic Charleston Foundation and Charleston Yacht Club, through the parking lot and along the East Bay Playground, along Concord St, Through the waterfront park and connects back to Concord St to Cumberland St. In response to a request from the SC Ports Authority, the alignment was adjusted to include Union Pier and Concord Street Terminals behind the wall.

This is the Corps' recommended footprint based on the information available during the study. As more information becomes available in PED phase and the sponsor chooses to evaluate other footprints, it may change. Figure 5.3.3 shows the recommended footprint evaluated.

Figure 5.3.3 Alignment of the Perimeter Storm Surge Wall

5.4 GEOTECHNICAL ASPECTS OF FEASIBILITY STUDY MEASURES

Due to the study area size, schedule and funding constraints, the geotechnical design is conceptual. It was developed based on assumptions made using information found within other CSRM project studies (Norfolk, Virginia and Galveston, Texas) and local geotechnical reports, along with engineering judgment. The geotechnical design is at a 10% conceptual level. Discussion are included on what future work is required during the Pre-construction Engineering and Design (PED) phase. The geotechnical aspects of the various feasibility study measures are discussed below.

5.4.1 T-WALL

The T-wall will be pile founded using both vertical and batter piles. A steel sheetpile cutoff will be installed to reduce underseepage and uplift on the wall. It was assumed that the sheetpile would be 20 feet long (depth) for the EL. 12 NAVD88 wall.

5.4.2 COMBO WALL

The king piles and batter piles for the Combo wall will be founded within the Cooper Marl formation. The steel sheetpile between the king piles will be installed to reduce underseepage. It was assumed that the sheetpile would be 40 feet long for EL. 12 wall.

5.4.3 PILES

Many structures on the peninsula are founded on piles. Review of various engineering reports received, the typical type was either steel H-piles or square, pre-stressed concrete piles, either 12” or 14” in size. These piles are driven to bear within the Cooper Marl formation, and it was assumed the embedment depth was 5 feet. The assumed top of Cooper Marl is presented below in Figure 5.4.1. Additional maps can be found in Attachment 2 of the Geologic and Geotechnical Sub-Appendix.

It is reported that there can be a dense sand/gravel layer above the Cooper Marl that can make it difficult to drive concrete piles through it. Additional investigation will be required during PED to determine if/where there are dense sand/gravel layers along the alignment.

Vibrations during pile driving is a concern as there will be many structures located adjacent to the CSRM project. Some of these structures have historical significance. There are methods to estimate distances but is dependent on soil stratigraphy, which detailed stratigraphy is unknown at this time. A general rule of thumb is that vibration damage is not likely to occur outside of 50 feet from the pile (either top or tip of pile, whichever is closer) for piles 50 feet or less in lengths or the length of the pile. With piles lengths approaching 90 feet and some piles being battered, preconstruction survey on properties within a 100-ft buffer from wall centerline was assumed. Additionally, vibration monitoring will be required during construction as various locations throughout the area but not at each residential structure.

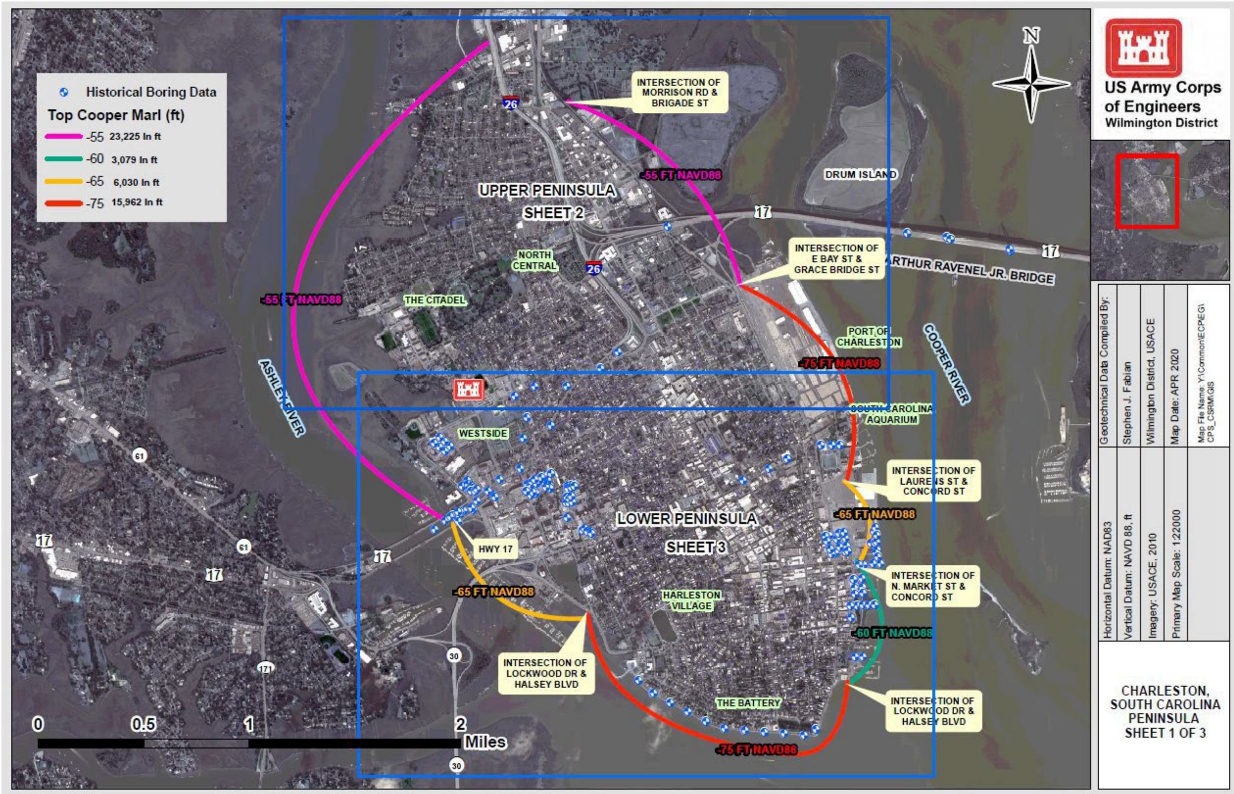


Figure 5.4.1: Assumed Top of Cooper Marl

5.5 STRUCTURAL ASPECTS OF THE FEASIBILITY MEASURES

Due to the poor nature of the soils in Charleston, all wall types are planned to be founded on deep piles that will be embedded into the Cooper Marl stratum which is located at elevations ranging from EL -55 NAVD 88 to El -75 NAVD 88. Cooper Marl consists of medium dense silty sand to firm silty clay and provides sufficient bearing capacity to support all structures.

5.5.1 I-WALL

I - Walls were ruled out as a viable option for flood wall construction. This type of barrier would consist of driven sheet pile walls with a concrete cap. I - Walls occupy a small footprint and would be desirable in areas where space is limited. I – Walls did not perform well in New Orleans during Hurricane Katrina and Corps design criteria was subsequently updated to limit the height of new I - Walls to a maximum of 4 feet above the current finish grade. The requirement for the new flood barrier to accommodate future raising rules out the I wall as a viable alternative.

5.5.2 T-WALL

For the purposes of this study, a T-Wall was assumed to be used where the barrier needed to be constructed on land, and not in the marsh or open water. T walls consist of a reinforced concrete stem, a reinforced concrete foundation, sheet pile cutoff wall, and vertical and batter piles. Steel sheet pile and H pile is shown in this sketch. Steel that is embedded in soil will not corrode. The steel sheets and the H pile will not displace as much soil during driving and will result in less vibration to mitigate risk of damage to nearby historic buildings. Due to the poor nature of the soils in Charleston, it is assumed that the T-Wall will be founded on a deep pile foundation that will be embedded within the Cooper Marl stratum. Based on available data, this strata is roughly 60 to 80 feet or more below current finished grade and consists of medium dense silty sand to firm silty clay.

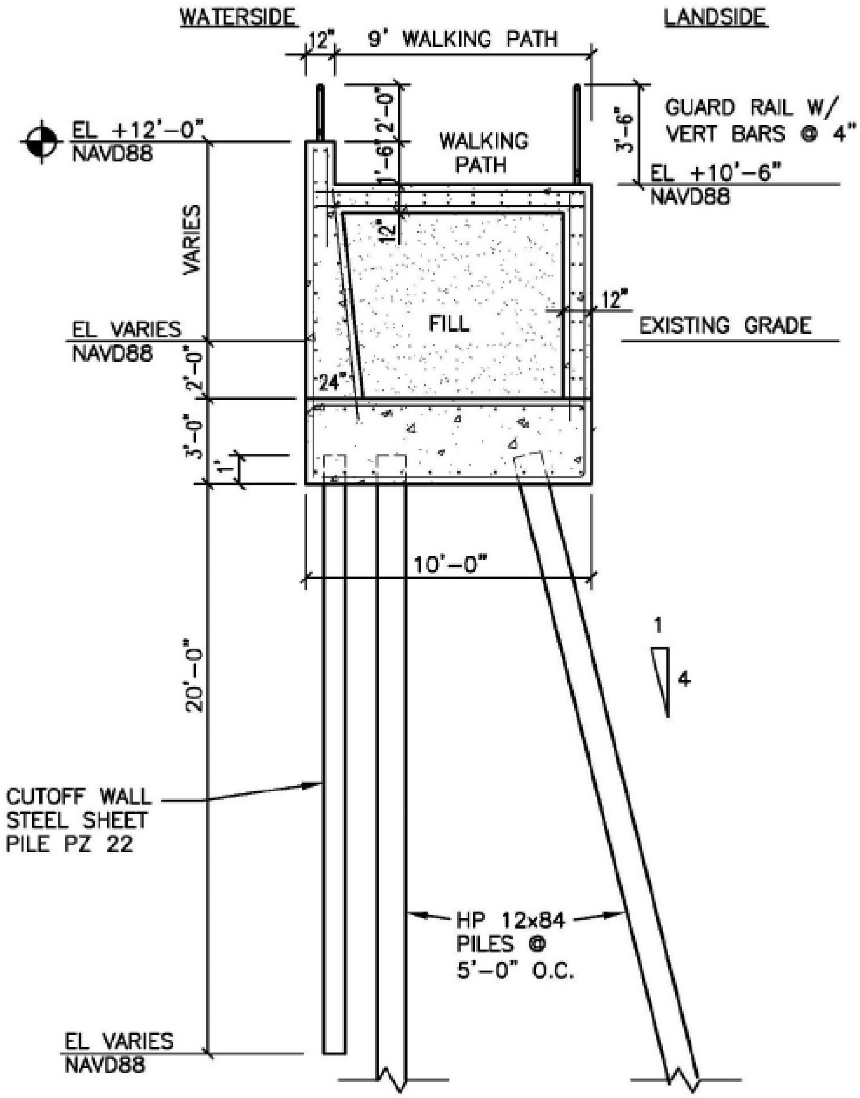


Figure 5.5.1 Typical T-Wall

T – Walls with walkways are planned to be constructed in scenic areas such as along Lockwood Blvd and Brittlebank Park, and to replace the existing High Battery wall. The T – Wall with walkway section is similar to the T wall except that the stem is moved to the waterside and the walking path is constructed over the remainder of the foundation. Stairs and Ramps will be required for pedestrian access to the T – Wall with walkway. Stairs and parallel Ramps would require a wider foundation than the typical T – Wall with walkway. "In Tandem" ramps would be identical to the typical T – Wall with walkway, sloping down to grade and would thus avoid the need for a wider foundation. Ramps would slope down at a rate of 1' vertical to 12' horizontal to meet all requirements for persons with disabilities including railing extensions, grab rails, and landings. See Structural Sub-Appendix for more details.

5.5.3 COMBO WALLS

Combo Walls are planned for reaches where the flood barrier will be constructed across water or wetland. Construction of this wall type presents a number of unique challenges such as: Wetland Impact, Construction access, and Exposure of materials to saltwater environment. A temporary work trestle was determined to be necessary to construct the combo wall, which will allow sufficient width to operate a crane and receive materials. A dredged access channel was considered but rejected due to the adverse environmental impact. Prestressed concrete was selected over steel piles for the combo wall to avoid the need for cathodic protection. The foundation could be precast in 10’ x 10’ sections and grouted into position to avoid the need for formwork. Precast units would include grouted keyways and post tensioning conduits to assure continuity and water-tightness.

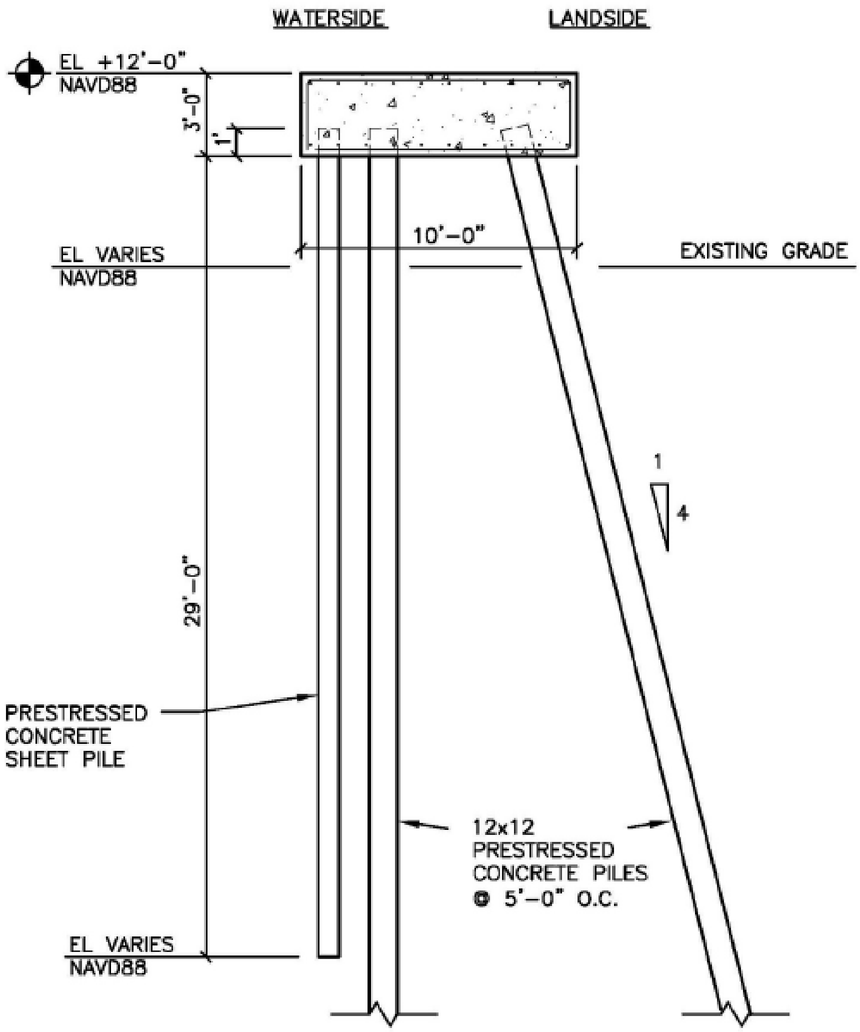


Figure 5.5.2 Typical Combo Wall

5.5.4 BRIDGE CLEARANCES

Where the barrier goes under existing bridges, clearances for construction were taken into consideration when selecting a deep foundation system, as well as construction methods used. Full height piles will not be able to be installed in areas with low vertical clearance. Piles would need to be installed in sections and spliced by welded or bolted connections. Micropiles will be utilized where clearance is low in the location of the T-Wall; Below are 3 locations where head clearance is a concern.

- James Island Connector - ~20 ft clearance from existing grade (T Wall)
- Ravenel Bridge - ~25 ft clearance from existing grade in the parking lot (T-Wall)
- Highway 17 at Lockwood ~ 17 feet from existing grade

5.5.5 LOADS

The load cases considered for this study were in accordance with Coastal Flood Wall requirements in EM 1110-2-2502. To date, analysis has not been completed, but engineering judgment and information from NAO's feasibility study were used at this stage. During optimization, preliminary analysis will be completed.

- C1: Surge Still Water Loading
- C2a: Nonbreaking Wave Loading
- C2b: Breaking Wave Loading
- C2c: Broken Wave Loading
- C3: Earthquake Loading
- C4: Construction Short-Duration Loading
- C5: Wind Loading
- C6: Debris/Boat/Barge Impact Loading

5.5.6 LOW BATTERY WALL

The Low Battery Wall is currently being renovated by the City of Charleston. The designer of record stated that the new Low Battery Wall was designed to be able to be modified in the future to provide a level of protection of EL 12 ft NAVD 88. The wall can be retrofitted by removing and replacing the existing post and railing and replacing with a solid wall. No other structural upgrades will be required to the Low Battery Wall to provide protection to EL 12 ft NAVD 88. Raising the LowBattery Wall in the future by an additional 3 feet would require additional structural analysis and structural upgrades. These upgrades may consist of, but are not limited to, foundation upgrades and additional lateral support. These upgrades will be very difficult to construct and may result in major demolition and reconstruction of the Low Battery Wall.

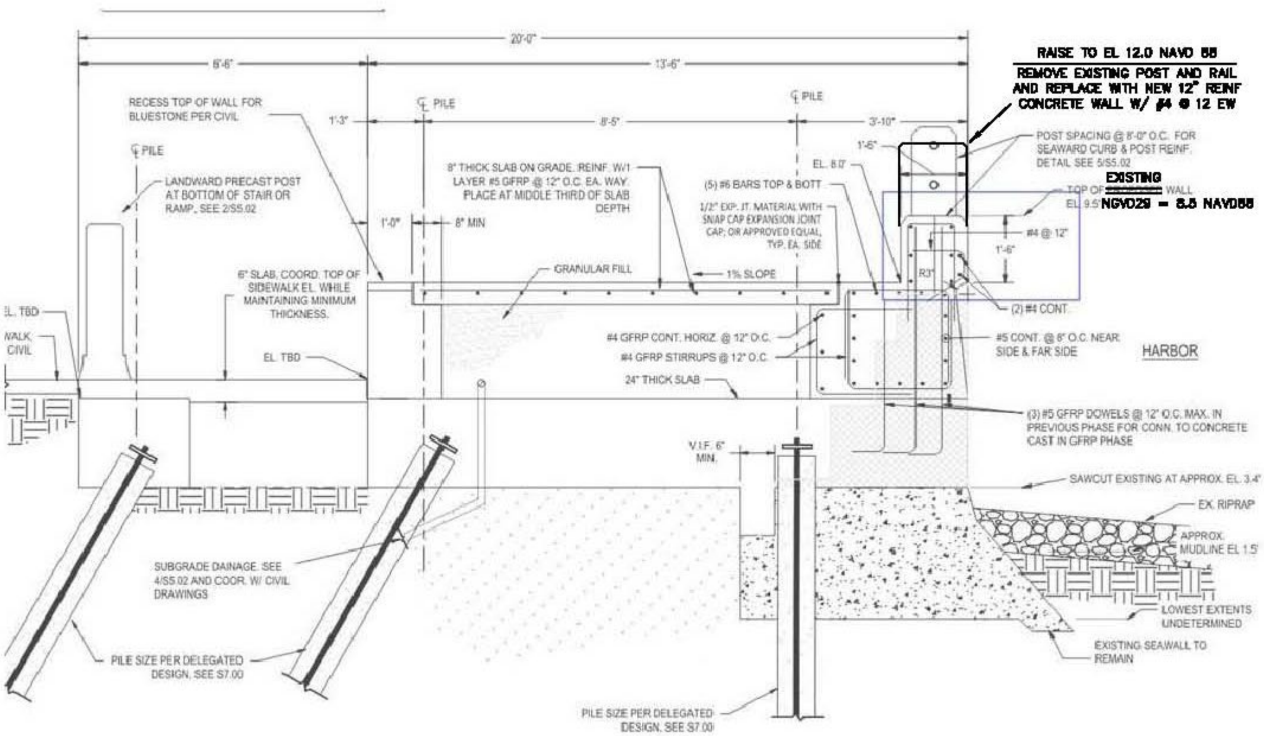


Figure 5.5.3 Typical section of Low Battery Wall Upgrade to EL 12.0 NAVD 88 flood protection

5.5.7 HIGH BATTERY WALL

The construction of the existing High Battery Wall is not sufficient to support raising the level of protection to EL 12.0 NAVD 88. Given its age and the assumed construction techniques used for the time period of which it was constructed, it is safe to assume that the high battery wall will not meet the criteria to be part of the Federal project. The High Battery Wall will be replaced with a new T-Wall withWalkway (Figure 5.5.1). There will still be a transition at the "turn" from the walking path elevation of the low battery to the new elevation at the high battery.

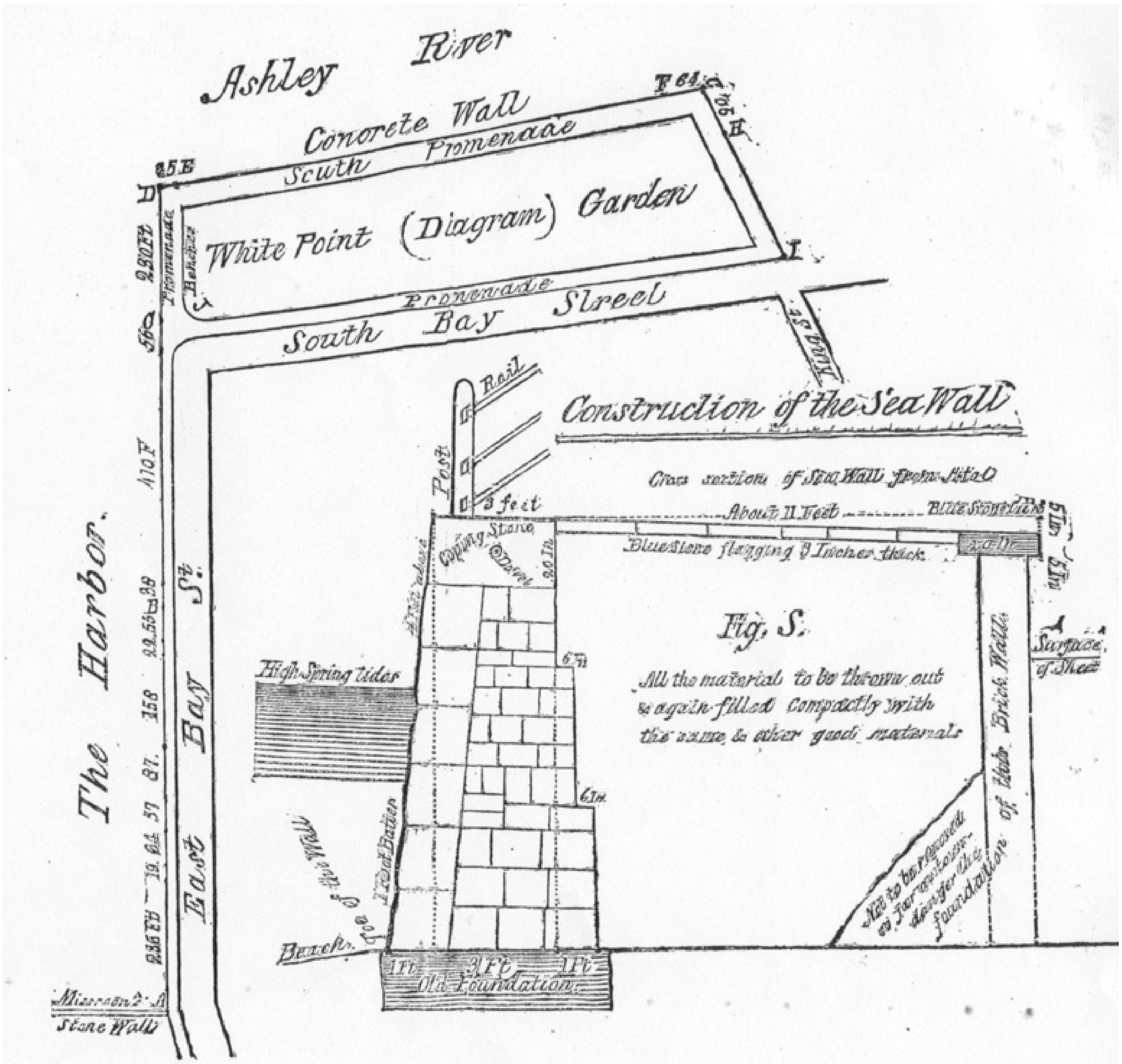


Figure 5.5.4 Existing High Battery Wall

5.5.8 UTILITY CROSSINGS

Consideration was given to a method of assuring the continuity of the sheet pile cutoff wall at utilitycrossings. Utilities would need to remain in service throughout the construction of the flood wall.

Utilities would need to be located prior to excavation or driving any piles. Sheet pile construction presents the greatest challenge since it must be continuous in order to function as a cutoff wall. Thesolution is to omit the sheet pile at any utility crossing and jet grout the cutoff wall panel around theutility as shown below.

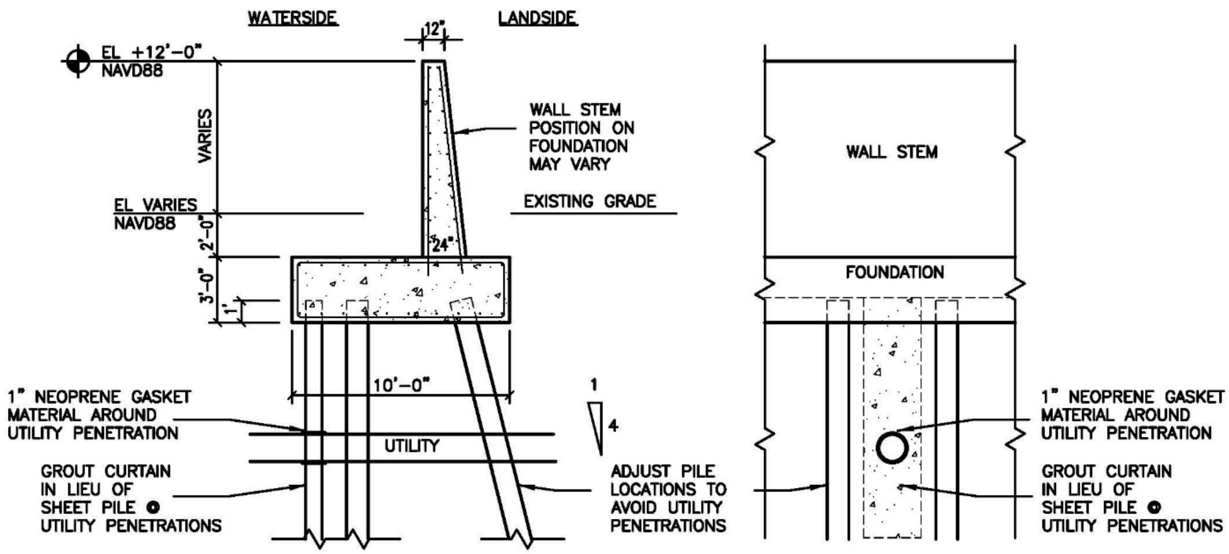


Figure 5.5.5 Grout Curtain Cutoff Wall Construction at Utility Crossings

5.6 GATES

Preliminary structural analysis and design was performed for the gates. Many types of gates will be required ranging from very long gates across roadways to very short gates across pedestrian access routes. There are a variety of gates required in the study area to ensure water tightness of the barrier during storm events. The different types of gates can be broadly broken into three categories: Vehicle Gates, Pedestrian Gates, and Storm Gates. There are different types of gates in each category depending on the exact location it will be installed. More detail is provided on each category and subcategory below. Typically, all of the gates will remain open the majority of the time and will only be closed when required due to a coastal storm event.

5.6.1 VEHICLE GATES

Where the new barrier/wall crosses existing roads, a watertight gate will have to be installed to seal the opening. The gates will be open the majority of the time and will only be closed for coastal storm events. In addition, vehicle gates that allow pedestrian passage will have to be Americans with Disabilities Act (ADA) compliant. There are approximately 59 vehicle gates required. Because of the simple design and lower maintenance costs, swing gates and slide gates will be prioritized. More information about each gate type is shown below:

Figure 5.6 1 Location of Vehicle Gates

5.6.1.1 SWING GATES

Swing gates are the simplest and easiest gates to install and operate. A large, reinforced metal gate is attached to the wall with hinges on one side. To close, the gate is simply swung around by the hinges and into place, then secured to the wall on the opposite side of the opening. Or, if needed to span a larger opening, two gates can be attached, one on each side of the wall and swung together in the middle. A removable support post is installed in the middle for the two gates to rest against. Compressible seals along the bottom and sides of the gate provide a watertight seal. If necessary, other removable supports will be installed on the dry side of the gate to provide structural integrity when resisting the hydraulic force of the water on the wet side of the gate. Typically swing gates do not require any powered equipment to open and close and can be done manually with enough people. Depending on the size, heavy lifting equipment may be needed for opening/closing and placing the additional, removable supports required. See an example photo of a single span swing gate from New Orleans, LA in Figure 5.6.2 below. The main drawbacks to swing gates are the large clearance required to be able to close them, and they remain exposed to the elements even when not in use. However, as the simplest to maintain and least expensive form of gate, swing gates will be prioritized and installed in all locations that have the necessary clearance.

Figure 5.6.2: Swing Gate Example (Photo by USACE Charleston District)

5.6.1.2 SLIDE GATES

Slide gates are also simple and relatively easy to install. A large reinforced metal gate slides across a track from one or both sides of the opening. When closed, compressible seals along the bottom and sides of the gate seal against the wall or each other to provide a watertight seal. Depending on the height and width, additional bracing can be placed on the dry side of the gate to help it withstand the pressure of the water. An advantage of slide gates is that they do not have the same clearance issues as swing gates, and also can potentially be stored within the wall itself, which keeps the gates, seals, and other moving parts out of the elements when not in use. See examples of a slide gate installed as part of a floodwall system in Norfolk, VA in Figures 5.6.3 and 5.6.4 below. Slide gates can have the option for a manual cranking system to close depending on size, but many times a motorized opening/closing device is required. Because of their simple design and operation, slide gates shall be prioritized for areas where there is not enough clearance for swing gates to operate properly.

Figure 5.6.3: Slide Gate Example (Photo courtesy of USACE Norfolk District)

Figure 5.6.4: Slide Gate Example (Photo courtesy of USACE Norfolk District)

5.6.1.3 RAILROAD GATES

Where the wall crosses over existing railroad tracks, a railroad gate will be required. These gates typically consist of a bulkhead type gate to provide a watertight seal as the rails prevent regular gates from closing and sealing properly. The modular sections will include special parts that are made to seal around the rail tracks. There are 3 locations where a railroad gate will be required. In these locations, modular sections of a gate will be stacked together within seats built into either side of the wall. For simplicity, the modular gate sections can be stored on or within the wall near the gate location. However, large equipment such as a small crane or frontend loader may be required to lift the sections into place.

Figure 5.6.5: Railroad Gate Locations

5.6.1.4 STORM DRAINAGE CHECK VALVES

There are numerous locations where the wall will cross over existing underground storm drainage lines. To prevent water from flowing back through the drainage lines, a check valve will be installed in the underground storm water system where the lines pass under the wall. Check valves restrict the flow of water to one direction, so they will allow stormwater to flow out through the system normally but will close and cut off all flow once water levels on the outlet reach a point that it would flow to the interior of the wall. They will then automatically open again to allow water to flow out from the interior of the wall once the water levels on the outlet are low enough. There are approximately 70 locations where a check valve is required. During PED phase, the drainage system will be evaluated and it will be determined the exact type of check valve and location is best suited for installation. Where possible, it may be best to install a flap gate style check valve on the stormwater outfall rather than at the location the wall crosses the drainage line. See the example photo below of a flap gate style check valve.

Figure 5.6.6 Example of Flap Gate Style Check Valve (Photo courtesy of USACE Norfolk District)

5.6.1.5 OTHER VEHICLE GATES

There are a variety of other gates such as pop-up gates, flood sensing automatic gates, modular section gates, etc. In general, these types of gates are more expensive, have more maintenance, and have more mechanical and electrical hardware. They will be considered on a case-by-case basis in the Preconstruction, Engineering and Design phase (PED) if swing and slide gates are not possible or practical, but the expectation is that the use of these types of gates will be minimal, if used at all.

5.6.1.6 COAST GUARD DOCK GATES

One unique gate that was studied will occur where a Combo Wall crosses the Coast Guard Dock. A portion of the dock will have to be removed to construct the Combo Wall and the pier will be restored along with addinga new 50’ wide swing gate (Figure 5.6.4 and Figure 5.6.5). The gate will be supported with intermediate diagonal frames to limit the span to 12.5 ft. see Structural Sub-appendix for more detail.

Figure 5.6.7 Aerial View at Coast Guard Base

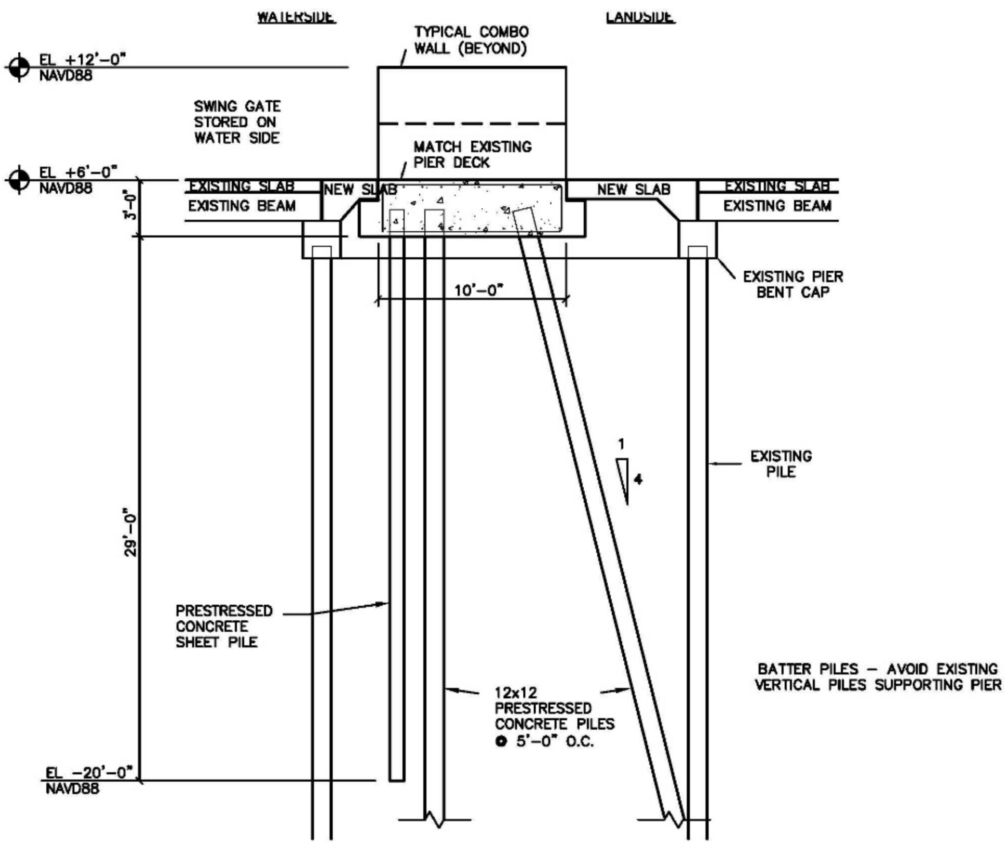


Figure5.6.8 Typical section of Combo Wall Crossing Coast Guard Dock

5.6.2 PEDESTRIAN GATES

Any place where the wall crosses an area such as a walking path, sidewalk in a parking lot, etc. a pedestrian gate will be required. This will allow foot traffic back and forth to specific areas that require access when not secured for a storm event. This includes access to marinas and private docks that have walk out access which will now be obstructed by the wall. Where possible, access will be provided by ADA compliant ramps going over the wall, which will eliminate the need for a gate. However, due to space and other constraints, a ramp will not be possible in all pedestrian locations. Therefore, watertight pedestrian gates will be installed in these locations. There are approximately 14 pedestrian gates required, and all pedestrian gates will be ADA compliant. A pedestrian gate simply consists of a hinged door or gate with compressible seals around the edges that provide a watertight seal when closed. Typically, the gates will remain open and will only be closed during a coastal storm event.

Figure 5.6.9 Pedestrian Gate Locations

5.6.3 STORM GATES

Storm gates is a broad term used to describe gates that will be installed in areas where water flows, such as creeks and marshes within the study area. The gates will remain open the majority of the time to allow normal passage of overland flow, ebb, and flow of the tide, etc. The gates will only be closed to protect against a coastal storm event, which is done to minimize the impact on the natural resources such as marshes and aquatic organisms. The exact size and type of gate installed depends on the individual location and area it is protecting, and the ecological conditions. In general, for areas that require wider spans of opening to account for more flow, several smaller gates will be utilized as opposed to one larger gate. This is for a variety of reasons, including access, operation, and maintenance reasons. Storm gates will primarily be sluice gate type for their simplicity and ease of operation. Figure 5.6.10 shows the Preliminary locations of Storm Gates. These are all located at tidal creeks, including creeks that are currently partially restricted by culverts.

Figure 5.6.10 Locations of Storm Gates

5.6.3.1 SLUICE GATES

Sluice gates were chosen for the primary storm gate as they are used in a variety of water control applications, including flood control, all over the world, and are relatively low maintenance due to their simple design. They also can be a variety of sizes or can be placed side by side to maximize the flow when open and minimize negative effects like flow restriction, scouring, etc. As shown in the example photo Figure 5.6.11, sluice gates function simply as a metal gate that can be raised and lowered on a track and seal an opening in the wall area. The sluice gates will be placed primarily in areas where a tidal creek or marsh flows in and out during normal tide cycles. The gate will remain open the majority of the time to minimize the impact to normal tide cycles and the surrounding environment. The gates will only be closed when needed for a coastal storm event. There are tentatively 6 areas that will require sluice gates in the study area (note one of the locations will have multiple gates for a total of 10 sluice gates).

Figure 5.6.11: An Example of a Pair of Sluice Gates (Photo courtesy of USACE Norfolk District)

5.6.3.1 OTHER GATES

Other storm gates that were considered and could be utilized are miter gates, tainter gates, stop logs, etc. These forms of gates are typically for larger spans and openings than needed and are more expensive and complicated, require more maintenance, etc. They will also require strong motors or heavy equipment to operate. For these reasons, sluice gates are intended as the primary form of storm gate, and these other forms of gate will only be used if absolutely needed.

5.6.4 GATE PREVENTATIVE MAINTENANCE

Monthly maintenance of the various gates will be required to keep them in proper working order. Maintenance of the vehicle and pedestrian gates will be fairly simple, and mostly involve making sure the seals are in working order and replacing them as needed. Other items such as slides and tracks will need to be cleaned and lubricated a few times a year to ensure they are in working order when needed. For any gates with motor operation, the motors will require normal maintenance and lubrication, etc. It is expected that the vehicle and pedestrian gates will last for the life of the project with regular maintenance and replacement of items such as seals as needed.

The storm gates will have all the same maintenance requirements as the pedestrian gates with the added requirement that the seats and seals will need to be regularly cleaned of vegetation, algae or debris to ensure that they can provide a proper seal when needed. Because of the exposure to the weather, including salt water, it is reasonable to assume that the storm gates will require at least one full replacement of the gate itself in the lifetime of the project. Other supporting structures such as the concrete casings will not need replacement, just the gates themselves.

A draft operations and maintenance manual has been started during this feasibility phase. This draft O&M manual provides a preliminary operations process, maintenance, and inspection timeline, etc. This draft plan will be further developed during the PED phase as more detailed information is developed about each individual gate during the design portion.

5.7. GATE CLOSURES

There are a variety of gates required in the study area to ensure water tightness of the barrier during storm events. As detailed above, the different types of gates can be broadly broken into four categories: Vehicle Gates, Pedestrian Gates, Storm Gates and Storm Drainage Check Valves. There are different types of gates in each category depending on the exact location itwill be installed.

Gate closure procedure will be finalized during PED phase and dictated in the Operation and Maintenance Plan, a draft version of which is provided as a Sub-Appendix. Typically, all of the gates will remain open and will only be closed when required due to a coastal stormflooding event. For the vehicular, pedestrian and railroad gate closings, it will be dependent on the time needed to close gates in reaction to water level so as to address operation and evacuation needs. This may result in different thresholds in the different areas of the city. Storm gates shall be closed when a significant storm surge event for the area is predicted by the National Weather Service, which will be described in the City of Charleston's overall emergency management plan. To maximize the availability of water storage, storm gates shall be closed during the last low tide cycle prior to the onset of predicted storm impacts. If the last low tide cycle is too close to the onset of impacts that personnel cannot efficiently and safely close the gates (to be defined in the City's emergency management plan) then the gates would be closed during the prior low tide cycle. Since low tide occurs approximately every 12 hours, the gates should not be closed for any longer than 24 hours prior to the onset of storm impacts.. At present that elevation is identified as 8 MLLW or 4.86 NAVD88.

Table 5.7.1 Major Water Level Thresholds for Charleston

Water Level Thresholds Established (Feet above MLLW)		**Feet above NAVD88**
Major Flooding (NOAA NWS) Widespread flooding occurs in Downtown Charleston with numerous roads flooded and impassable and some impact to structures	8.0	4.86

Terminology

Major Flooding: *Extensive inundation of structures and roads. Significant evacuations of people and/or transfer of property to higher elevations (NOAA NWS).*

5.8 CONSTRUCTION SCHEDULE

The construction schedule is based on the assumption that there will be a three year design period followed by a multi-phase construction period of approximately seven years. Construction phases will be broken down into the five different model areas, and each model area may require phasing as well. Early phases will focus on the model areas at the end of the peninsula such as the battery, marina, and port. Further detail on the construction schedule will be developed during the PED Phase.

5.9 WAVE ATTENUATION

During Optimization the large wave attenuator was evaluated and eliminated. The benefits derived did not justify the cost. During PED phase the exterior wall along the water's edge will be evaluated to determine if modifications to the face can attenuate wave overtopping.

5.10 INTERIOR DRAINAGE ASSESSMENT

5.10.1 STUDY OVERVIEW

The Charleston Peninsula Coastal Storm Risk Management Study is investigating coastal storm impacts on the Charleston peninsula. In partnership with the City of Charleston and its stakeholders, the Project Delivery Team (PDT) is exploring effective, economically viable and environmentally-sound solutions to mitigate risks and build enduring coastal storm resiliency.

The Tentatively Selected Plan (TSP) at Feasibility Phase includes both structural and non-structural flood risk management measures. The structural measure relevant for the interior drainage study is the proposal of a storm surge barrier with a design elevation of 12 feet NAVD88. This storm risk management measure greatly reduces the risk of flooding from coastal storm surge up to the level of design, however, areas protected from exterior flood elevations are subject to interior residual flooding from stormwater runoff. Thus, interior drainage facilities may be required to safely store and discharge the runoff to limit interior residual flooding. In the case of Downtown Charleston, SC, there are not many options for storing stormwater therefore allowing the stormwater to discharge via gravity flow (low exterior conditions) through the proposed wall and/or discharging the stormwater via pumping (typically during high exterior elevations) are the focus of the study. The interior areas were studied to determine the specific nature of flooding and to formulate drainage alternatives to implement as part of the alignment.

In accordance with USACE Engineer Manual (EM) 1110-2-1413, *Hydrologic Analysis of Interior Areas*, the interior drainage features are evaluated separately from the alignment to determine what project features are needed to provide interior flood relief such that during low exterior stages (gravity conditions) the local storm drainage system functions essentially as it did without the project in place up to that of the storm drainage design. The City of Charleston indicates much of the peninsula storm pipes reach capacity near the 10% AEP storm (rainfall) event with some areas designed to lesser pipe flow capacities. Such information suggests that surface-flow runoff becomes a larger component of drainage for rain events above the stated design capacities.

A study approach was defined and conducted using the Hydrologic Engineering Center's (CEIWR-HEC) River Analysis System (HEC-RAS) two-dimensional modeling software to determine the plans that can be implemented as part of the project alignment and that appropriately mitigate the interior residual flooding. The results for future without- and with-project conditions were incorporated into the Hydrologic Engineering Center's (CEIWR-HEC) Flood

Damage Reduction Analysis (HEC-FDA) to compute Equivalent Annual Damages (EAD) and Average Annual Damages (AAD) for describing the residual risk for the interior area. The HEC-FDA assessment served as an economic tool to determine the interior drainage features and their capacities (storm gates/pump stations) necessary for the project to perform acceptably and efficiently.

*All elevations in this report are referenced to the North American Vertical Datum of 1988 (NAVD88).

Figure 5.10.1 Interior Drainage Study Area

5.10.2 PROJECT AREA CONDITIONS

The project area conditions considers historical storm events (tropical cyclones, hurricanes, and rainfall) which were previously discussed in Section 3.5 of this appendix. Other project area conditions consider the rainfall-tide correlation (if any), the climate change effects to inland hydrology, and the City of Charleston's stormwater management systems (existing and future without-project).

5.10.2.1 RAINFALL-TIDE CORRELATION ASSESSMENT

For the with- and without-project conditions, the exterior stage is an important factor in the drainage of the interior stormwater runoff. The exterior stage is controlled by the tidal cycle and storm surge elevations during storm events. Inland, the interior surface runoff is conveyed out into the Cooper River and Ashley River, which join at the Charleston Harbor, via stormwater outfalls (pipes/culverts) and/or existing pump stations. If both sources of flooding occur at the same time the flooding effects are exacerbated, and pump stations become a major component of interior drainage relief until the high tides/storm surge recede and allows for gravity flow.

In the without-project condition, during high exterior stages (tide/storm surge) that rise above the outfall opening, the gravity driven outfalls may incur significant tidal backflow if no check valve is in place and/or cease to drain the interior area if a check valve is in place. Similarly, if a new coastal storm risk management structure is introduced (with-project condition) to reduce the risk of storm surge entering the study area, the existing outfalls, under high exterior (tailwater) stage conditions would incur tidal backflow and/or cease to drain as previously mentioned. Therefore, it is important to develop an understanding of whether there is a relationship between interior surface runoff and exterior tidal events in both with- and without-project conditions.

To understand the relationship between the interior and exterior stage conditions, if any, a correlation assessment needs to be performed. In accordance with EM 1110-2-1413, the correlation assessment should include a data assessment of the correlation, dependence, and coincidence of the interior and exterior stage relationship.

Rain-tide data was collected for thirty historic crest events to assess the correlation of the rainfall that may occur during storms that produce tidal flooding. Historic tide "crest" events were identified and tabulated with the corresponding rainfall events. Table 5.10.1 displays thirty historic crest events with daily rainfall data for the day of the peak crest and +/- one day before and after the peak crest. The tabulated data was also plotted and shown in Figure 5.10.2. The data for +/- one day was also included because stalled storms may bring multi-day rainfall events in which stalled storms may also produce longer duration high tide elevations.

Reviewing the data of the thirty historic crests events, two (8/11/1940 and 10/3/2015) contain accompanying 24-hr rainfall totals greater than a 10% AEP for the day the peak crest occurs. The 1940 event shows 7.66 inches of rainfall the day of the peak tide crest which is roughly a 4% AEP. The 2015 event shows 9.25 inches of rainfall the day of the peak tide crest which is roughly a 2% AEP. It is noted the 2015 historic rainfall event took place from approximately October 1-5, 2015 and produced rainfall totals upwards of 15 inches throughout the stalled storm.

Table 5.10.1 Historical Crest Events with Corresponding Rainfall

	Historic Crests (MLLW)	Historic Crests NAVD88	24-HR Rainfall on Day of Crest (inches)		24-HR Rainfall 1 Day Before Crest (inches)		24-HR Rainfall 1 Day After Crest (inches)		Max 24-HR Rainfall of 3 Days (inches)	Rainfall Sum over 3-Day Period (inches)
1	12.52	9.38	9/22/1989	0.87	9/21/1989	5.99	9/23/1989	0.15	5.99	7.01
2	10.23	7.09	8/11/1940	7.66	8/10/1940	0.03	8/12/1940	1.94	7.66	9.63
3	9.92	6.78	9/11/2017	4.53	9/10/2017	0.02	9/12/2017	0.01	4.53	4.56
4	9.29	6.15	10/8/2016	3.84	10/7/2016	4.36	10/9/2016	0	4.36	8.2
5	8.81	5.67	1/1/1987	0.97	12/31/1986	0.1	1/2/1987	0	0.97	1.07
6	8.76	5.62	11/24/2018	0.44	11/23/2018	0	11/25/2018	0	0.44	0.44
7	8.69	5.55	10/27/2015	0.51	10/26/2015	0.02	10/28/2015	0.28	0.51	0.81
8	8.64	5.5	5/28/1934	1.48	5/27/1934	0	5/29/1934	0.15	1.48	1.63
9	8.64	5.5	9/4/1979	6.06	9/3/1979	0.34	9/5/1979	0.7	6.06	7.1
10	8.46	5.32	11/2/1947	0.72	11/1/1947	0	11/3/1947	0	0.72	0.72
11	8.29	5.15	10/3/2015	9.25	10/2/2015	1.41	10/4/2015	2.49	9.25	13.15
12	8.27	5.13	10/28/2015	0.28	10/27/2015	0.51	10/29/2015	0	0.51	0.79
13	8.21	5.07	10/4/2015	2.49	10/3/2015	9.25	10/5/2015	0.14	9.25	11.88
14	8.15	5.01	10/15/1947	0.85	10/14/1947	0.58	10/16/1947	0.56	0.85	1.99
15	8.14	5	9/29/1959	3.96	9/28/1959	0.55	9/30/1959	0	3.96	4.51
16	8.14	5	11/23/2018	0	11/22/2018	0	11/24/2018	0.44	0.44	0.44
17	8.11	4.97	6/22/2009	0	6/21/2009	0	6/23/2009	0	0	0
18	8.08	4.94	8/30/2019	0	8/29/2019	0	8/31/2019	0.05	0.05	0.05
19	8.06	4.92	12/24/2019	2.78	12/23/2019	0	12/25/2019	0.34	2.78	3.12
20	8.06	4.92	6/23/2009	0	6/22/2009	0	6/24/2009	0	0	0
21	8.05	4.91	12/9/2018	1.06	12/8/2018	0.82	12/10/2018	0	1.06	1.88
22	8.05	4.91	9/29/2015	0	9/28/2015	0	9/30/2015	0.26	0.26	0.26
23	8.03	4.89	2/20/2019	0.25	2/19/2019	0.01	2/21/2019	0	0.25	0.26
24	8.02	4.88	8/29/2019	0	8/28/2019	0.06	8/30/2019	0	0.06	0.06
25	8.01	4.87	6/18/1982	4.27	6/17/1982	1.79	6/19/1982	0	4.27	6.06
26	8.01	4.87	12/31/1994	0	12/30/1994	0	1/1/1995	0.01	0.01	0.01
27	8.01	4.87	7/21/2001	0.03	7/20/2001	2.34	7/22/2001	0	2.34	2.37
28	8.01	4.87	9/28/2015	0	9/27/2015	0	9/29/2015	0	0	0
29	8	4.86	1/30/2010	1.06	1/29/2010	0	1/31/2010	0	1.06	1.06
30	7.28	4.14	9/15/1999	3.62	9/14/1999	0.77	9/16/1999	0	1.61	4.39

Denotes a greater than or equal to 10% AEP 24-hr rainfall event (yellow)
Denotes a greater than or equal to 10% AEP 3-day rainfall event (orange)

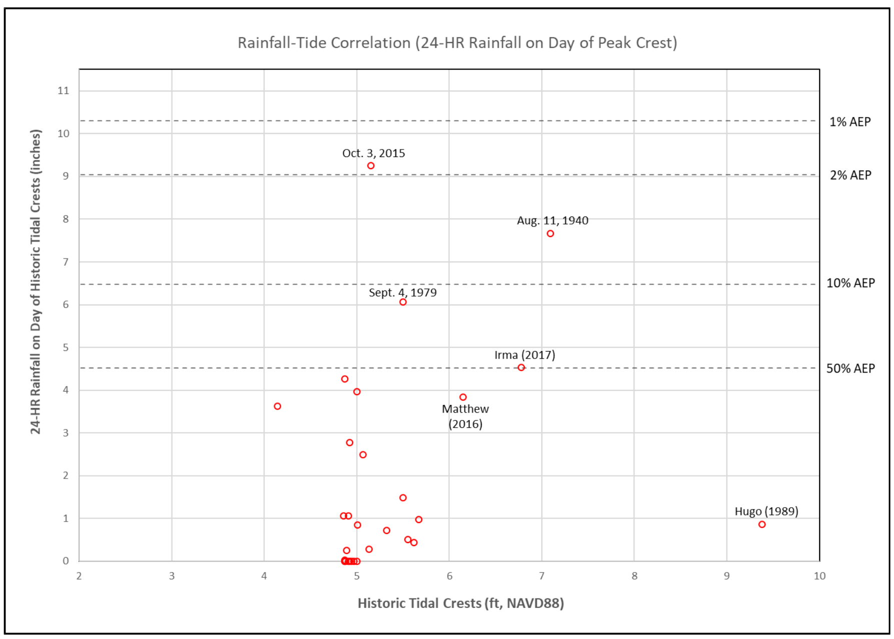


Figure 5.10.2 Historic Storms Rainfall-Tide Correlation Plot

In addition to correlation, further assessment was completed for dependence and coincidence. The review of the data displays some dependence for both rain-tide correlation and coincident timing. The interior drainage model for sizing pump stations assumes the storm gates are closed throughout the model simulation. This means pump stations are sized considering all interior rainfall throughout the modeled storm would be mitigated via pumping. In reality, it is unknown whether the rain and tide peaks occur at the same time. The storm gates could potentially be opened once the storm surge recedes and therefore assist in the drainage of interior storm water runoff. For this study, the future without-project conditions assume a constant Mean Higher High Water (MHHW) tide elevation with Sea Level Change (SLC) while applying the various rainfall frequencies onto the HEC-RAS model. The future with-project condition results for the same hydro-meteorological are compared to the future without-project results for assessing the specific nature of flooding between the conditions.

To describe the elevations at which flooding begins, the National Weather Service provides flood categories at specific elevations and details of the flooding at each elevation. Table 5.10.2 displays the flood categories as denoted by the NWS.

Table 5.10.2 National Weather Service Flood Categories

Flood Categories	MLLW (ft.)	NAVD88 (ft.)	*NAVD88 (ft.) Year 2032
Action Stage	6.5	3.36	3.92
King Tide	6.6	3.46	4.02
Minor Flooding	7.0	3.86	4.42
Moderate Flooding	7.5	4.36	4.92
Major Flooding	8.0	4.86	5.42
*The elevations for the year 2032 were simply estimated by adding 0.56 feet to the current elevations.			

NWS Flood Impacts

- At 8.0 ft MLLW (4.86 ft. NAVD88), major coastal flooding occurs. Widespread flooding occurs in Downtown Charleston with numerous roads flooded and impassable and some impact to structures. Impacts become more extensive all along the southeast South Carolina coast including erosion at area beaches, with limited or no access to docks, piers, and some islands.

- At 7.5 ft MLLW (4.36 ft. NAVD88), moderate coastal flooding occurs. In Downtown Charleston, additional impacted roads include HW-17 at HW-61, Market Street, East Bay, Rutledge, and areas around MUSC. Other impacted areas include Long Point Road near Palmetto Islands County Park, locations around the Naval Complex, 12th and 15th Streets on Isle of Palms, and the road leading to Bohicket Marina on Seabrook Island. In Beaufort County, flooding will impact Hunting Island and the Sea Island Parkway near Chowan Creek Bridge.

- At 7.0 ft MLLW (3.86 ft. NAVD88), minor coastal flooding typically begins. Minor flooding on roadways around Downtown Charleston occurs, possibly including Lockwood Drive, Wentworth and Barre, Fishburne and Hagood, and Morrison Drive. As the tide height approaches 7.5 ft MLLW, roads can become impassable and closed. Other impacts outside of Downtown Charleston include minor flooding of low-lying locations near area beaches including Isle of Palms, Sullivan's Island, Folly Beach, Kiawah Island, and Edisto Island.

The detailed assessment is provided in Section 2.2 of the Interior Drainage Sub-appendices (Hydraulics and Hydrology – HEC-RAS 2D Modeling).

5.10.2.2 CITY STORMWATER MANAGEMENT SYSTEMS

Most rainfall on the peninsula is collected in a subsurface pipe network system and routed to numerous outfall locations (pipes/culverts) and/or existing pump stations. The City of Charleston has stated the existing stormwater drainage systems (storm pipes) have design capacities of approximately 10% Annual Exceedance Probability (AEP) and some areas contain lesser capacities. Such capacities lead to frequent flooding during intense rainfall events. Such flooding is exacerbated when exterior stages are elevated limiting the conveyance of stormwater runoff. Another issue faced by the City of Charleston is sunny day or “nuisance” flooding which occurs during high tide events that cause backflow into the interior system and flood the streets. The City of Charleston has already begun the implementation of a check valve program in response. Check valves or “flap gates” are attached to peninsula outfall drains to prevent tidal backflow into the stormwater pipe network.

The City of Charleston currently does not have a complete storm water management model covering the entire study area. The coverage the City does have is spread across different models based on the drainage areas and service areas for the various individual projects listed below. The known storm drainage projects in the city include:

- Calhoun Street East Drainage to the Concord Street pump station which is complete.

- Market Street Drainage improvement project. Phase 1 and 2 is completed which connected the drainage to the Concord Pump station. Construction of Phase 3, will be the improvement of the surface drainage collection system to the previously installed new tunnel, expected in 2021. Phase 4 is in construction and phase 5 is pending all be completed for future without condition.

- Medical University of South Carolina (MUSC) pump station.

- Spring Fishburne Drainage Improvement which will improve drainage in an area that covers about 20% of the peninsula, areas - phase 2 completed, phase 3 (tunneling) is underway, completion 2020, Phase 4 (wet-well and outfall) expected to be complete by 2022, Phase 5 (pump station) expect completion by 2023.

- Wagener Terrace Storm Drainage - repair existing system – completed.

- Calhoun West – preliminary report is report is complete from a technical standpoint at this time, unknown if it will be completed by federal project.

- Huger King Street - Phase 1 design is complete with Department of Transportation (DOT) currently reviewing encroachment permits and construction expected in 2020. Phase 2 Outfall improvement and pump station is currently at 30% design with construction expected to be complete in 2022.

- Low Battery Project Phase 1 is ongoing, pile installation expected to be complete this month, construction of the phase expected to be complete in 2020. Phases 2- 5 will follow in each successive year. Low Battery raising to similar elevation of High Battery.

USACE Engineer Regulation 1165-2-21 states “In urban or urbanizing areas, provision of a basic drainage system to collect and convey the local runoff to a stream is a non-Federal responsibility. This regulation should not be interpreted to extend the flood damage reduction program into a system of pipes traditionally recognized as a storm drainage system.” While the storm drainage is not a USACE CFRM responsibility, any impacts to the interior drainage induced by the proposed project must be evaluated and mitigated to the extent justified under USACE policy. Creating and evaluating the system is outside the scope of the Feasibility study, however, it is assumed the pump stations proposed in this effort are to utilize existing storm pipes if accessible.

The PDT is to work with the City of Charleston during PED phase to appropriately incorporate the stormwater systems into the USACE recommended plan as result of the CFRM. The city’s contracted engineering firm (Davis & Floyd) have modeled the stormwater facilities previously discussed using the Storm Water Management Model (SWMM) but do not currently contain full coverage of the peninsula.

As part of the CFRM, three city-owned and operated pump stations are included in the interior drainage modeling. Those pump stations are the Concord Street Pump Station (active), MUSC Pump Station (active), and the Spring Fishburne Pump Station (construction). Also, included are existing drainage culverts that convey surface-flow stormwater runoff and/or provide routing of daily tidal fluctuations. HEC-RAS does not perform sub-surface modeling; therefore, the storm pipe network is not captured.

5.10.2.3 SPONSOR - FLOODING AND SEA LEVEL RISE STRATEGY

As indicated in the City of Charleston's "Flooding and Sea Level Rise Strategy" published in February 2019, one of the objectives is to address flooding while promoting a more resilient and sustainable future in the face of recurrent flooding, rising seas, and more frequent extreme weather. The City of Charleston indicates the intent to use the latest NOAA 2017 sea level rise projections for future considerations. One way to track sea level rise is to document "minor coastal flooding" commonly called nuisance, sunny day flooding. The City indicates a marked increase in the number of days of minor coastal flooding over time. The NOAA sea level change curves and other appurtenant information is provided throughout Chapter 3 of the Coastal Sub-Appendix B-4 report.

The City of Charleston's Sea Level Rise Strategy (2015) originally recommended a 1.5 to 2.5 foot elevation increase for new facilities and infrastructure. The City increased the recommendation to 2 to feet for the revised 2019 sea level rise strategy. The projection of a 2 to 3 foot rise in 50 years is higher than the NOAA intermediate rate of rise (+1.65 feet) being utilized for the USACE peninsula study in the year 2082 (50-year project life from end of construction). The City of Charleston uses the projection of 2-foot increase for less vulnerable infrastructure such as parking lots, while a 3-foot increase is for more critical long-term infrastructure, such as medical facilities.

In 2019, the City of Charleston began reconstructing and raising the Low Battery Seawall to account for sea level rise projections. The Low Battery Seawall is being raised to an elevation similar to that of the High Battery Seawall. The USACE study assumes this project for the future without-project conditions while the future with-project condition would need the wall raised again to 12 feet NAVD88. The City has also begun the Check Valve Program which is a plan to equip the peninsula outfalls with check valves. A check valve or "flap gate" prevents seawater from backing up into drainage infrastructure to mitigate tidal flooding, while still allowing the outfall to drain stormwater as usual when the tide recedes. Many outfalls in the City are gravity fed and drain to bodies of water that are tidally influenced. During high tides, seawater often enters storm water outfalls and water can back up far enough in low lying areas to result in backflow flooding on streets, even on a sunny day. The City has begun the installation and replacement of check valves on the outfalls. Some of the outfalls currently have a duck bill type check valve and are to be phased out in favor of in-line valves which function better and have less maintenance costs. USACE assumes all peninsula outfalls to be equipped with check valves by the USACE with-project end of construction year (2032).

In addition to tidal flooding and sea level rise, rainfall induced flooding is a significant challenge for the City, and flooding is exacerbated when rainfall and high tide/storm surge occur at the same time. While check valves on the outfalls work well to mitigate flooding from high tides entering the storm drains. Check valve or not, rain that falls during a high tide still has little room to drain and/or increased resistance to drain (as opposed to low tide) until the tide recedes. During such cases, the stormwater collects on the surface because the storm drains are full of seawater. In addition, if the check valve is in the closed position holding pressing seawaters then additional ponding on the streets may occur. The USACE PDT is proposing storm gates for the surface flow culverts that align with the proposed storm surge wall alignment. These culverts convey inland surface runoff and/or allow for daily tidal fluctuations. The storm gates placed on the culverts are to be closed only during predicted storm events that bring tidal flooding. There has been discussion between the City and USACE about the potential of upsizing culverts during construction of the storm surge wall. Further discussion on this matter is to take place during PED phase.

The City has begun and completed many other drainage improvements such as the Market Street improvement which connects to the Concord Street Pump Station. Another project is the Spring Fishburne Pump Station which is currently under construction. Further information about the City's existing and future drainage improvements are provided in Section 2.3 of the Interior Drainage Analysis Sub-Appendix. Along with drainage improvements are spotlights on drainage maintenance because a stormwater drainage system performs best when properly maintained. This includes procedures such as keeping drainage ditches, conveyance pipes, and storm drains as clean as possible. As of 2019, the City contracted with a group of engineers and subject area experts to form the new Stormwater Program Management Team. The team is to update the City's Master Drainage and Floodplain Management Plan, which was last revised in 1984. Another focus of the team is implementing GPS, GIS, and sub-surface camera technologies to schedule, inspect, and monitor both the surface flow drainage ditches and sub-surface stormwater drainage tunnels and pipes.

As mentioned, the City has many drainage improvements completed and/or underway. An important feature for both the City and the PDT are pump stations. The PDT has accounted for three City pump stations in the future without-project condition and in the future with-project condition conceptual plans and modeling. The PDT is proposing both permanent and temporary pump stations, meaning the permanent pump stations will contain permanent pump houses with larger pumping capacities while temporary pumps are deployed during storms and typically contain smaller pumping capacities. The City's Flooding and Sea Level Rise Strategy also dictates using strategically placed temporary pumps, with appropriate storm forecasting notice, to remove stormwater and tidal inundation to mitigate the risk of flooding to the inland area. The City's pump stations such as the Spring Fishburne are thoroughly described in the City' sea level rise strategy (2019). The pump stations contain storm pipes which bring stormwater to the stations which is then pumped underground to the surrounding rivers. The PDT plans to use the existing infrastructure for bringing stormwater to the pump stations. The City has stated the storm pipes accommodate no more than a 10% AEP rainfall event and, in some areas, a lesser capacity is provided by the storm pipes. Once the pipes become overwhelmed water begins to collect on the streets. Therefore, surface flow runoff becomes a larger component of drainage during heavy rainfall events and/or events where the storm drains are filled with backflowing seawater. This is an important aspect and assumption for the PDT as the hydraulic model (HEC-RAS 2D) for interior drainage is a surface flow only model and does not have the capability to model sub-surface flow. At this phase of the CFRM study, the City does not have full coverage or a complete model of the storm pipe network. During PED phase, further information about the storm pipe network will need to be incorporated to more appropriately size and place the PDT's recommended plan for pumps. The PDT has strategically placed pump stations using HEC-RAS 2D by assessing the natural drainage paths of surface flow runoff. In addition, the HEC-RAS modeling is supplemented with some of the City's GIS based layers for visual assistance in the placement of pump stations. These GIS layers provide an important information for the conceptual layout of the future with-project pump stations. The referenced layers include the storm pipe network (layout/inlets/outlets) and the peninsula watershed delineation. The watershed delineation refers to surface flow and further information about the storm pipe (sub-surface) delineation or pump servicing boundaries is needed during PED phase to ensure appropriate design.

The USACE PDT is to coordinate with the City engineers during PED phase to ensure the strategies, goals, and collaboration of the project features are adequately aligned.

Source: [(https://www.charleston-sc.gov/DocumentCenter/View/20521/Flooding-and-Sea-Level-Rise-Strategy-2019-printer-friendly?bidId=)](https://www.charleston-sc.gov/DocumentCenter/View/20521/Flooding-and-Sea-Level-Rise-Strategy-2019-printer-friendly?bidId=)

5.10.2.4 CLIMATE CHANGE IMPACTS TO INLAND HYDROLOGY

The USACE overarching climate adaptation policy requires consideration of climate change in all current and future studies to reduce vulnerabilities and enhance the resilience of USACE water resources infrastructure. To meet the USACE climate adaptation policy, project delivery teams (PDTs) must assess climate change impacts when a study involves inland hydrology, coastal analysis and/or a boundary condition impacted by sea level. The assessment should be carried out at an appropriate, scalable level based on the complexity, size and level of risk associated with the project. Sea Level Change (SLC), inland hydrology, and riverine hydrology are assessed to determine if the project is vulnerable to climate change.

The climate assessment for inland hydrology follows the USACE guidance of Engineering and Construction Bulletin (ECB 2018-14), Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects. For most USACE projects and studies, a qualitative assessment provides the necessary information to support the assessment of climate change risk and uncertainties to the project design or constructed project. Per the guidance, a hydrologic literature review of observed climate trends and projected climate trends in the project area is required. USACE and NWS hydrologic and meteorologic tools are used for this assessment. The tools are used to detect non-stationarities in sea level change, riverine hydrology, and meteorology (precipitation).

An in-depth assessment of sea level change is provided in Section 3.6.1 and 3.6.2. The selected sea level rates from that assessment are +0.56 feet for the year 2032 and +1.65 feet for the year 2082. These are considered "intermediate" rates. The hydraulic (HEC-RAS) model applies the selected SLR rates to the tidal boundary conditions to account for sea level rise for the interior drainage assessment.

As indicated in the City's Flooding and Sea Level Rise strategy, one way to track local impacts from sea level rise is documenting "minor coastal flooding". Commonly called nuisance, sunny day or high tide flooding, minor flooding is a threshold from the National Weather Service that indicates when the tide has reached a certain height (7.0 ft. MLLW in the Charleston Harbor or 3.86 feet NAVD88). At this height, low-lying areas on land begin to flood. Other flood thresholds are provided in Table 5.10.2, Section 2.2.1 of the interior drainage sub-appendices. The City of Charleston has experienced a marked increase in the number of days of minor coastal flooding over time. In response, the City has implemented the check valve program. In addition to tidal flooding and sea level rise, changes in precipitation trends are a focus of the City's strategy and some stormwater management features are being modified in response. Further discussion provided in Section 2.3 of the interior drainage sub-appendices.

The climate change to inland hydrology was assessed qualitatively following the three phases outlined in ECB 2018-14.

1) Scoping
2) Vulnerability assessment
3) Risk assessment

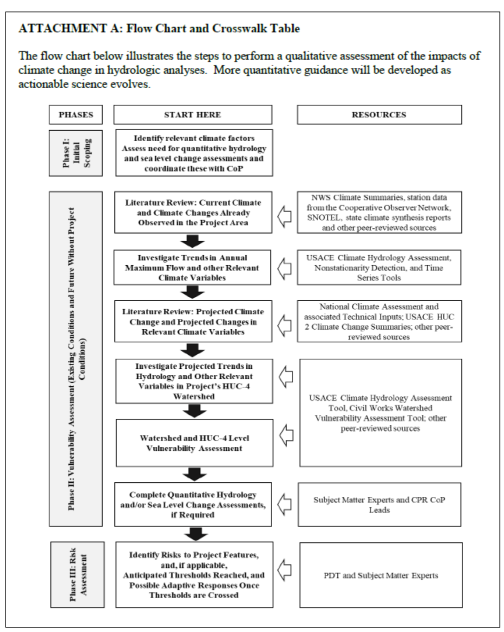


Figure 5.10.3 ECB 2018-14 Flow Chart for Climate Change Assessment

A review of the literature displays the project is most vulnerable to sea level rise, increases in air temperature, and increases in precipitation frequency and intensity. Per guidance in ECB 2018-14, Table 5.10.3 identifies risks resulting from changing climate conditions in the future. The table shows the major project feature, the trigger event (climate variable that causes the risk), the hazard (resulting dangerous environmental condition), the harms (potential damage to the project or changed project output), and a qualitative assessment of the likelihood and uncertainty of this harm.

The detailed qualitative assessment is provided in Appendix 1 of the Interior Drainage Sub-appendices (Hydraulics and Hydrology – HEC-RAS 2D Modeling). A summary of the Risk Assessment section of the assessment is included below.

Table 5.10.3 Climate Risk Register

Feature or Measure	Trigger	Hazard	Harm	Qualitative Likelihood
Flap Gated Peninsula Outfalls	Increased sea level	Increased water levels and wave heights seaward of the project. Limited discharge capacities of gravity outflow due to elevated exterior water levels above tops of pipe outfalls	Rainfall runoff may remain on the interior for longer durations and more frequently, potentially damaging the project and leading to more demand of pumps increasing maintenance and operational costs.	Likely
Storm Surge Design Elevation (12 ft. NAVD88)	Increased water levels from storm events (surge) due to sea level rise	Potential for overtopping of wall design elevation from wave overtopping or flood overtopping (SWL)	Increased SLR may increase frequency and magnitude of water level and wave loading on wall. Higher water levels that overtop design elevation causing a breach, flooding of interior areas and potential loss of life.	Moderate
Storm Gates	Increased sea level/Increased frequency of extreme events	Increased water levels and wave heights seaward of storm gates	Increased SLR may increase frequency of storm gate closure, increasing operational costs. Frequency and magnitude of water level may increase.	Likely
Pump Stations	Increased water levels from storm events (surge) due to sea level rise	Potential for wave overtopping and/or flood (SWL) overtopping	Potential of interior residual flood risks due to underperformance of interior project features leading to increased durations of interior flooding	Moderate

Pump Stations	Increased sea level	Increased water levels and wave heights seaward of the project. Limited discharge capacities of gravity outflow due to elevated exterior water levels above tops of pipe outfalls	More demand on pump stations, increasing maintenance and operational costs	Moderate

5.10.3 DEVELOPMENT OF HYDRAULIC MODEL

5.10.3.1 INTERIOR DRAINAGE METHODOLOGY

Areas protected from exterior flood elevations are subject to interior flooding from stormwater runoff. Thus, interior drainage facilities are required to safely discharge the runoff to limit interior residual flooding. In the case of Downtown Charleston, SC, there are not many options for storing stormwater therefore allowing the stormwater to discharge via gravity flow (low exterior conditions) through the proposed wall and discharging the stormwater via pumping (typically during high exterior elevations) are the focus of the study. The interior areas were studied to determine the specific nature of flooding and to formulate drainage alternatives to implement as part of the alignment.

For the with- and without-project conditions, the exterior stage is an important factor in the drainage of the interior stormwater runoff. The exterior stage is controlled by the tidal cycle and storm surge elevations during storm events. Inland, the interior surface runoff is conveyed out into the Cooper River, Ashley River, and Charleston Harbor via stormwater outfalls (pipes/culverts) and/or existing pump stations. When high tides occur, the disrupt the natural functions of the gravity driven drainage features in which pump stations become a significant feature for drainage.

In the without-project condition, during high exterior stages (tide/storm surge) that rise above the outfall opening, the gravity driven outfalls may incur significant tidal backflow if no check valve is in place and/or cease to drain the interior area if a check valve is in place. Similarly, if a new coastal storm risk management structure is introduced (with-project condition) to reduce the risk of storm surge entering the study area, the existing outfalls, under exterior (tailwater) stage conditions would incur tidal backflow and/or cease to drain as previously mentioned. Therefore, it is important to develop an understanding of whether there is a relationship between interior surface runoff and exterior tidal events in both with- and without-project conditions. The detailed tidal-rainfall correlation assessment is provided in Section 2.2 of the Interior Drainage sub-appendices.

Due to the uncertainties of the coincidental timing of peak rainfall runoff/tidal flooding events, the HEC-RAS model considers constant tidal boundary conditions throughout model simulations with- and without-project conditions. To study the potential interior ponding effect, a variety of scenarios were computed using a synthetic rainfall suite consisting of the 50%, 20%, 10%, 4%, 2%, and 1% Annual Exceedance Probabilities (AEP) of 24-hr durations. Each of these events were simulated for future without-project conditions and future with-project conditions for the years 2032 (end of project construction) and 2082 (50-year project life). The tidal boundary conditions assume the intermediate Sea Level Change (SLC) rates projected NOAA. The rates are +0.56 feet and +1.65 feet for the years 2032 (end of construction) and 2082 (50-yr project life), respectively. These tidal boundary conditions were assumed to be a constant 3.18 feet (2032) and 4.27 feet (2082) which are estimated from the current Mean Higher-High Water (MHHW) level of 2.62 feet NAVD88.

The stated conditions provide mean water levels to appropriately assess and size the storm surge gates and pump stations for comparison to future without-project conditions during non-storm surge conditions which allow for gravity flow although less efficient gravity flow considering high tide as compared to low tide gravity flow. In supplement to analyzing rainfall at projected MHHW levels, a variety of scenarios were also simulated to analyze the performance of the interior system (pump stations) during storm surge events that produce wave wash overtopping while also considering the rainfall, therefore events with rainfall plus wave wash overtopping and is further discussed in the Interior Drainage sub-appendices.

Because HEC-RAS 2D does not model sub-surface flow the current modeling performed for the Charleston Peninsula CFRM is performing a surface-flow only study. As previously mentioned, the City's existing storm pipe network becomes overwhelmed for events greater than the 10% AEP. At this point, an assumption is made that the surface-flow stormwater runoff becomes a much larger component of drainage for events at 10% AEP or greater. A surface-flow only model may be conservative in producing higher than expected water surface elevations, however, for both with- and without-project conditions providing the relative comparison between the with- and without-project. A surface flow only model may also under predict the stormwater flow rates to pump stations. Further due diligence is needed during PED phase to incorporate the City's storm pipe network system.

If the project is under-designed, water may pond on the interior, flooding homes and businesses. Alternatively, if the system is over-designed, the cost of the project will be inflated. Utilizing both HEC-RAS and HEC-FDA, the study approach is to assess the system from the hydraulics perspective and economics perspective. The RAS results may show differences in interior water surface elevations for with- versus without-project conditions but the FDA model provides the tools for describing the economic consequences and/or benefits of the differences in interior water surface elevations for with- versus without-project.

The RAS results for future without- and with-project conditions were incorporated into the FDA model to compute the EAD and AAD for describing the residual risk for the interior area. Detailed hydraulic and economics are provided in Chapters 4 and 5 of the Interior Drainage Sub-appendices (Hydraulics and Hydrology – HEC-RAS 2D Modeling)

5.10.3.2 EXISTING HYDRAULIC MODEL

The City of Charleston hired a contractor (Davis & Floyd Engineering) to perform HEC-RAS 2D modeling for the conceptual design of a local project, the Calhoun West Pumping Station. The contractors used one geometry file with a mesh size of 50-ft. x 50-ft. and breaklines to detail the road network and other raised features. The geometry used in the Calhoun West effort did not contain any features such as culverts or pump stations. The model was used for applying the rainfall runoff to observe the natural drainage paths of the runoff over the peninsula study area. For design of the stormwater pump stations, the surface-flow model is used for routing rainfall to the stormwater inlets. These flow rates at the inlets can be used for incorporation into the Storm Water Management Model (SWMM) for design of pump stations that consider sub-surface flow.

5.10.3.3 REFINEMENT OF HYDRAULIC MODEL

The model utilized for the City's Calhoun West pump station modeling was refined to perform the assessment for the Charleston Peninsula CFRM study. Revisions of the HEC-RAS 2D model from the originally provided model include separating the 2D mesh into an exterior and interior 2D and the incorporation of drainage features such as existing culverts which drain surface flow. Other features such as pump stations and the storm surge wall are incorporated and discussed in sections throughout this report. The breaklines for the road network are enforced into the 2D area. Breaklines were also applied to other appropriate locations to represent raised features in the model domain.

The HEC-RAS 2D model contains two 2D flow areas to represent the interior and exterior areas for each geometry condition including: existing, future without-project, and future with-project. The 2D flow areas are connected by SA/2D connections using the proposed storm surge wall alignment as the separator of the interior and exterior areas for all geometry conditions. Modeling each geometry condition in this manner allows for the cell mesh structure to be similar across each geometry. The 2D connections representing natural ground in the existing and future without-project geometry conditions allow the connection to read its weir station/elevation data directly from the underlying terrain (cut from terrain) while the 2D connections representing the proposed storm surge wall in the future with-project geometry uses elevation data of 12 feet NAVD88. The 2D connections at higher ground such as roads used weir coefficients of approximately 1 while low lying areas (tidal creeks) may use 0.5. The connection representing the storm surge wall use the default weir coefficient of 2.

As mentioned previously, the HEC-RAS 2D model does not have the capability to model sub surface flow. Therefore, all flow in the model is assumed to be surface flow and does not contain interior drainage features such as the storm pipes and/or sub-surface peninsula outfalls. A surface flow only model may over-estimate inland flood elevations in the absence of sub-surface storage, however, this is the case for all geometry conditions. Other uncertainties and model challenges are discussed in Section 6 of the Interior Drainage sub-appendices.

The projection for the RAS modeling is: “NAD_1983_StatePlane_South_Carolina_FIPS_3900_Feet_INTL”. The projection file is within in the RAS project folder labeled as “HEC_RAS_Projection.prj”.

Further information for model development is provided in Chapter 3 of the Interior Drainage Sub-appendices (Hydraulics and Hydrology – HEC-RAS 2D Modeling).

5.10.3.4 EXISTING CONDITIONS GEOMETRY

The existing conditions geometry contains interior drainage features such as culverts for surface flow drainage and/or fluctuations of tides. The included culverts are shown in Table 5.10.4 and Figure 5.10.4. The geometry also contains the Concord Street and MUSC pump stations which are owned and operated by the City of Charleston. The city owned pump stations are listed in Table 5.10.5. The locations of the city owned pump stations are provided in Section (Figure 5.10.8). The existing conditions geometry contains the current alignment and wall heights for the Low Battery and High Battery.

Table 5.10.4 Charleston Existing Culverts

Culvert Location	Culvert Type	Culvert Dimensions
10th Avenue	single box culvert	10ft. span, 2ft. rise
Near Joe Riley	single box culvert	12ft. span, 4ft rise
Gadsden Creek	single box culvert	9ft. span, 4ft. rise
Gadsden Creek Upstream 1	concrete pipe	24inch diameter
Gadsden Creek Upstream 2	concrete pipe	18inch diameter
Longpond	circular metal pipe	48inch diameter
Lockwood Wetland	circular metal pipe	36inch diameter
Newmarket Creek	double box culvert	8ft. span, 3ft rise
Newmarket Creek Upstream	concrete pipe	36inch diameter

Table 5.10.5 City of Charleston Pump Stations

Pump Location	PS (gpm)	PS (cfs)
MUSC (Active)	3 @ 17,000 each	3 @ 38 each
Concord Street (Active)	3 @ 42,000 each	3 @ 94 each
*Spring Fishburne (Construction)	3 @ 135,000 each	3 @ 300 each
**King/Huger (Design)	approximate capacity of 70,000	approximate capacity of 156
*Spring Fishburne - not included in existing conditions but is included in future conditions. **King/Huger - not included in RAS modeling.		

Figure 5.10.4 Charleston Existing Culverts

5.10.3.4.1 EXISTING CONDITIONS SIMULATION

The existing conditions scenario typically serves as a model calibration event, however, there are no inland gages nor are there known high water marks collected by the Charleston District (SAC) or the City of Charleston. The College of Charleston was contacted but was found to have no observed data. The Charleston based engineering firm (Davis & Floyd) also contains no recorded inland water surface elevations, however, indicated containing water level observations for Marina Lake and the Ashley River but were waiting on the final processed data. The HEC-RAS model, without calibration, still serves its inherent purpose of assessing the hydraulic response of future without-project and future with-project conditions for determining the specific nature of flooding for the various conditions.

The existing conditions scenario was computed using verified water levels produced by Hurricane Irma on September 11, 2017. These water levels were extracted from NOAA Tides & Currents webpage from the Charleston, Cooper River Entrance SC gage. The Station ID is listed as 8665530. Figure 5.10.5 displays the Hurricane Irma stage hydrograph (NAVD88). Table 5.10.6 displays the datums for the gage.

Table 5.10.6 Water Surface Elevations (WSEL)

Datum	*Elevations in NAVD88	Description	2032 Elev. (+0.56 feet)	2082 Elev. (+1.65 feet)
Max Tide	9.38	Highest Observed Tide	9.94	11.03
MHHW	2.62	Mean Higher-High Water	3.18	4.27
MHW	2.26	Mean High Water	2.82	3.91
MLW	-2.96	Mean Low Water	-2.4	-1.31
MLLW	-3.14	Mean Lower-Low Water	-2.58	-1.49
Note: There are uncertainties in projecting sea level change. The *current elevations were simply projected by adding the 2032 and 2082 sea level change rates.				

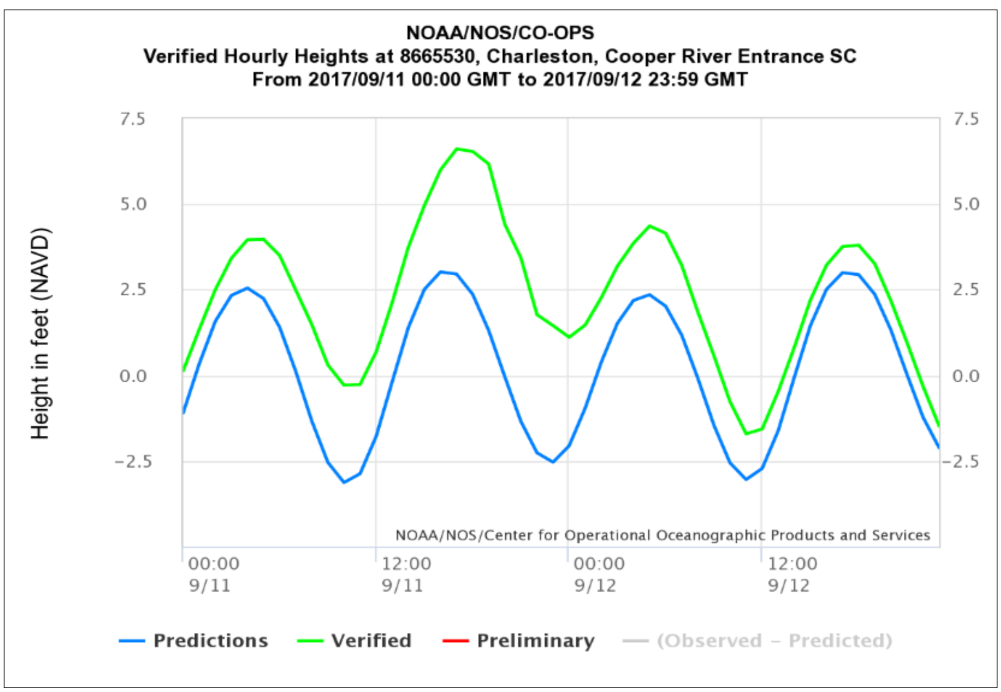


Figure 5.10.5 Charleston, Cooper River Entrance SC Tidal Gage (Hurricane Irma)

The existing conditions scenario in Figure 8 displays many areas inundated along the east and west side of the peninsula. The low side or west side of the Battery was overtopped by storm surge during this event. The Low Battery near the U.S. Coast Guard property was flanked by the surge before it was overtopped as seen in the modeling. Figure 5.10.6 displays the computed inundation for the 2017 Hurricane Irma event and compares it to the computed hypothetical inundation if it were to occur in 2032. The hypothetical 2032 Hurricane Irma event was computed by scaling up the 2017 stage hydrograph by an intermediate sea level change rate of +0.56 feet. Hurricane Irma peak water surface elevation was approximately 6.7 ft. NAVD88 therefore projecting this to the year 2032 assumes a peak water surface elevation of approximately 7.2 ft. NAVD88.

The purpose of Figure 5.10.6 is to provide visual representation of the potential increase in flooding for future storm events due to sea level change. There are significant uncertainties in estimating the evolution of future storm events, future storm surge, and the impacts of relative sea level change. Rainfall data was not included in this computation to show the inundation results from storm surge.

An additional existing conditions computation was computed for the 2017 Hurricane Irma event using the mentioned stage boundary condition and assuming a 24-hr 50% AEP rainfall. As shown in the rainfall-tide correlation assessment, the 2017 Irma event had approximately 4.5 inches of rainfall the day of the peak crest which corresponds roughly to a 10% AEP. The simulation is set to begin on 11Sept2017 at 0300 and end on 12Sept2017 at 1100. The Irma peak storm surge occurs at approximately 1700 on 11Sept2017. The 50% AEP rainfall is set to peak around 1500 on 11Sept2017.

Results from the 2017 Irma without rainfall simulation were compared to the 2017 Irma with rainfall. The results were compared where the inundations overlap (surge inundated areas) and show little to no difference between the simulations meaning the rainfall contributed little to peak water surface elevations for inland areas within that storm surge floodplain. Another factor to consider is the ability of rainfall runoff to discharge from inland areas at elevations greater than that of the storm surge floodplain during storm surge rise and fall.

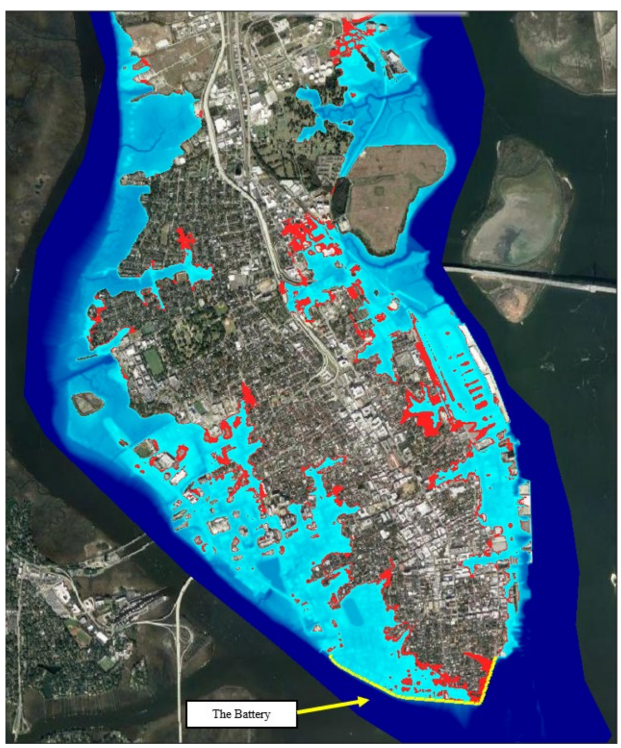


Figure 5.10.6 Hurricane Irma Simulation 2017 (Blue) vs Hurricane Irma Hypothetical Simulation 2032 (Red)

5.10.3.5 FUTURE WITHOUT-PROJECT GEOMETRY

The future without-project geometry contains interior drainage features such as culverts for surface flow drainage and/or fluctuations of tides. The geometry also contains the Concord Street (active), MUSC (active), and Spring Fishburne (construction) Pump Stations which are owned and operated by the City of Charleston. The King/Huger pump system is in design phase but is not included in this model.

The future without-project geometry assumes the City of Charleston to raise the Low Battery to be similar to the elevation of the High Battery.

5.10.3.6 FUTURE WITH-PROJECT GEOMETRY

The future with-project geometry contains the proposed storm surge wall with a design elevation of 12 feet NAVD88. The geometry also includes the existing culverts mentioned previously. The existing culverts which align with the perimeter of the proposed wall are assumed to be equipped with storm gates and part of the storm gate assessment. These storm gates would remain open and would only close when a storm surge is predicted. In addition to the existing culverts mentioned, other low lying tidal creek areas may require storm gates in the wall such as Halsey Creek and the creek near the Port.

The future with-project geometry also contains the city owned Concord Street, MUSC, and Spring Fishburne pump stations along with the USACE proposed pump stations. As previously mentioned, the King/Huger pump station is not included in the model.

Three alternative pump capacities at each pump station location have been assessed. Some pump stations are to be permanent, and some are to be temporary. The permanent pump station refers to those that would have permanent pump housing with larger capacities than the temporary pumps. Temporary or "portable" pumps refer to those that would not have permanent housing and would be deployed during storm events with appropriate notice. The pump stations would utilize existing storm pipe networks for routing water to the pumps. More information is provided in the Interior Drainage sub-appendices.

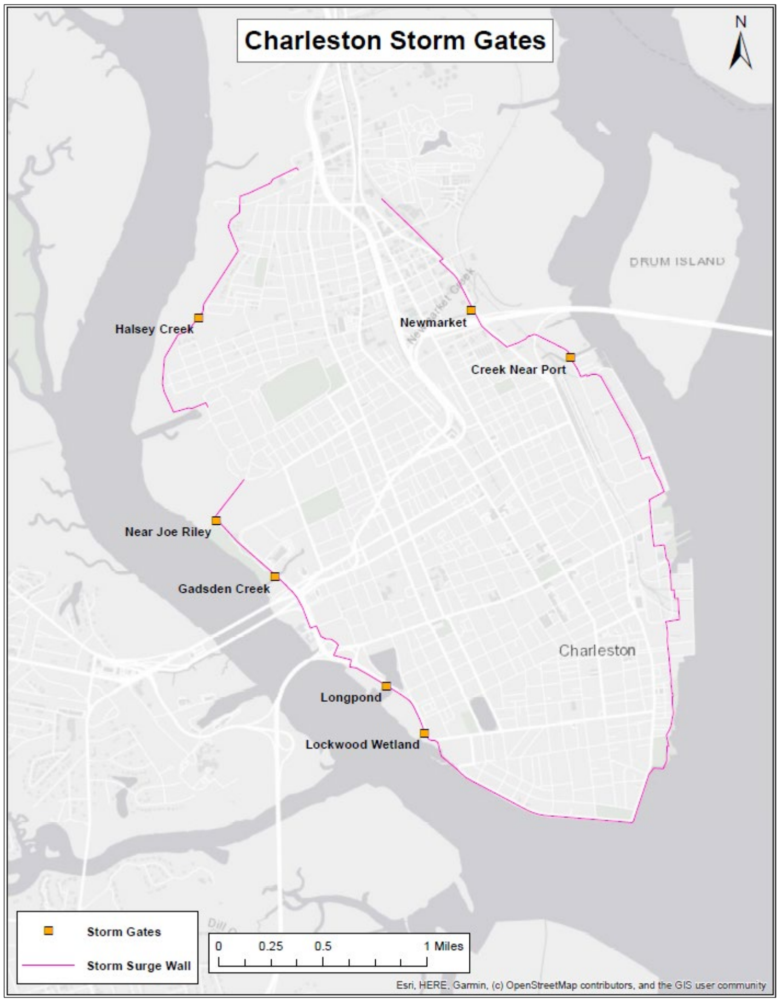


Figure 5.10.7 Charleston Storm Gates

Table 5.10.7 Charleston Storm Gate Dimensions

Storm Gate Location	Storm Gate Dimensions - # - (Width x Height) - feet
Halsey Creek	5 – 8’ x 15’ storm gates
*Near Joe Riley	1 – 4’ x 12’ box culvert
*Gadsden Creek	1 – 3’ x 9’ box culvert
*Longpond	1 – 4’ diameter circular pipe
*Lockwood Wetland	1 – 3’ diameter circular pipe
Vardell’s Creek	1 – 20’ x 80’ storm gate
*Newmarket Creek	2 – 8’ x 3’ double box culvert
Total	12 gates

*Existing culverts that are owned by the City of Charleston. During this phase of the study, the existing culverts are assumed to remain the same dimensions as they are currently. However, the culverts could potentially be upsized during construction of the proposed storm surge wall. The City of Charleston has also stated the possibility of upsizing some culverts. These details will be re-assessed during PED phase. Note that existing culvert dimensions are sized using the best available data and may not be exact.

Table 5.10.8 PDT Pump Station Alternatives

Pump Station	Pump Station Alt. 1	Pump station Alt. 2	Pump Station Alt. 3
	PS (cfs)	PS (cfs)	PS (cfs)
Halsey Creek (P)	60	90	150
Citadel near Joe Riley (P)	60	90	150
City Marina (P)	30	60	120
The Battery #1 (P)	30	60	120
The Battery #2 (T)	10	20	40
The Battery #3 (T)	10	20	40
Near Waterfront Park (T)	10	20	40
Port 1 (T)	10	20	40
Port 2 (T)	60	90	150
Newmarket Creek (P)	60	90	150
Totals	5 Permanent 5 Temporary	5 Permanent 5 Temporary	5 Permanent 5 Temporary
	340 cfs	560 cfs	1000 cfs
(P) Permanent (T) Temporary			

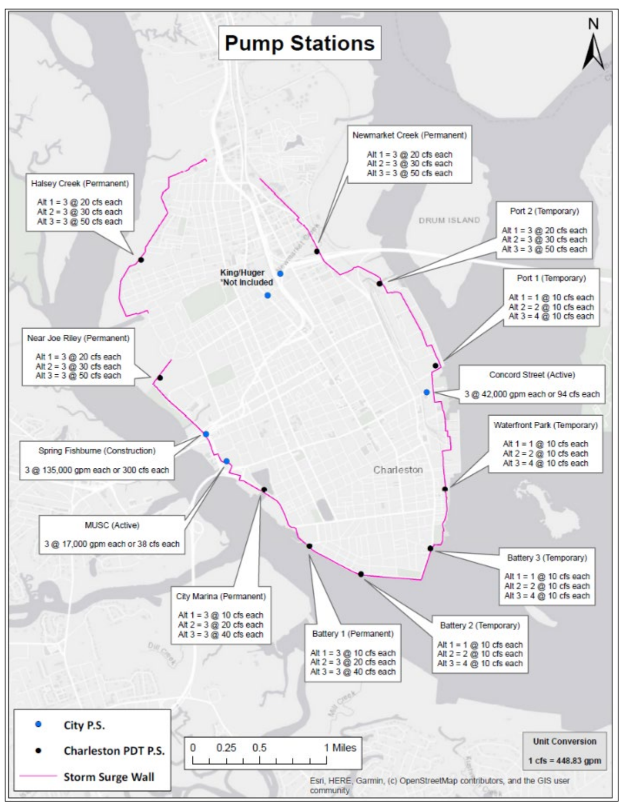


Figure 5.10.8 Pump Station Alternatives

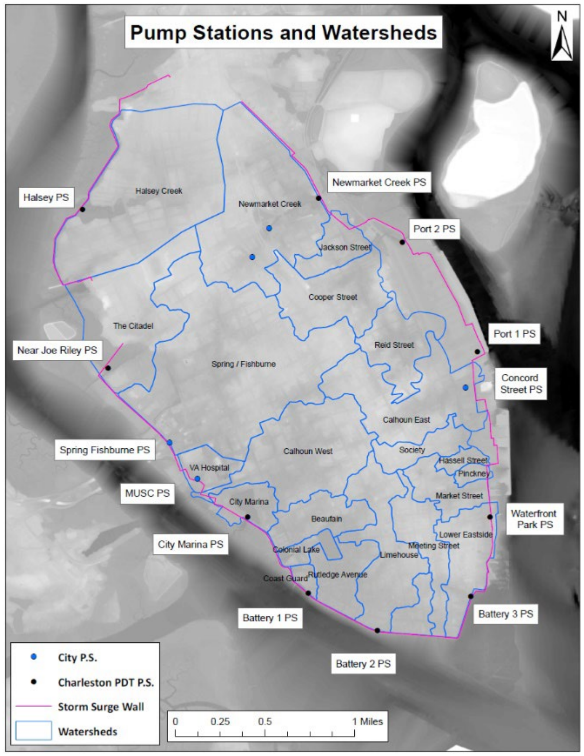


Figure 5.10.9 Pump Stations and Surface Flow Watersheds

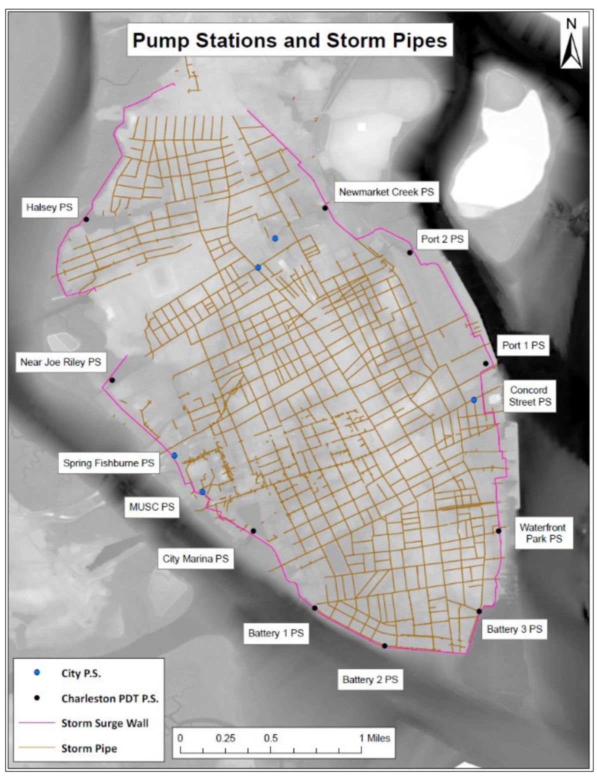


Figure 5.10.10 Pump Stations and Existing Storm Pipes

5.10.4 ASSESSMENT OF PROJECT ALTERNATIVES

The interior drainage assessment for the feasibility study assesses the proposed system from two perspectives: open system and closed system.

1. The open system assumes daily tide conditions or non-storm conditions. The storm gates remain open to allow daily tidal fluctuations. The gates open condition also assumes vehicle and pedestrian gates are open. These gates may not be significant for interior drainage however may provide some drainage relief during heavy rainfall events where water flows through the streets and along curbs before entering storm drains. In addition, if rainfall overwhelms the storm pipes or if seawater is in the storm pipes due to high tides street gates may have impacts on drainage. It is the assumption that check valves are to be installed on all outfalls/storm pipes, so seawater does not backflow into the system. For the open system conditions, the city owned pump stations are active while the USACE proposed pump stations are not active.

   The open system assumes inland runoff discharges via gravity flow overland to the exterior or to the City owned pump stations where it is discharged to the exterior. In reality, the runoff could drain through the storm pipes assuming the check valves are in place keeping seawater from backflowing and assuming the exterior elevations are low enough to allow the interior runoff to discharge through the outfalls.

2. The closed system assumes storm conditions meaning a storm surge has been forecasted that warrants storm gate closure. The model assumes the storm gates are closed at low tide prior to the arrival of a storm surge and to remain closed throughout the simulation. Interior rainfall runoff is to be removed via the city owned pump stations and USACE proposed pump stations. Runoff drains overland to the pump stations. In reality, the pipe system can bring stormwater to the pumps until the pipes reached capacity.

   a) The closed system condition assumes pre-storm water level drawdown meaning the interior system would be closed (storm gates) prior to the arrival of a storm surge. The gates are assumed to close at Mean Low Water (MLW) prior to the storm. This could allow the interior system additional storage for runoff accumulation. The closed system would have the option to flush the system prior to the storm if water is present in the system. The MLW levels for the year 2032 and 2082 are -2.4 feet and -1.31 feet NAVD88, respectively. The HEC-RAS model would assume these are the initial interior water levels at the beginning of model simulation.

      Due to the limitations of RAS 2D modeling, the interior inland conditions are bound by the underlying terrain (DEM) elevations. Majority of the inland terrain elevations are at greater elevations than the -2.4 and -1.31 feet MLW, therefore, the RAS model would not recognize the initial conditions and each 2D cell would use the higher underlying terrain elevation. The initial interior conditions for gates closed conditions in the years 2032 and 2082 are the same meaning no initial interior water surface is set and the initial interior condition is provided by the lowest terrain value for each 2D cell.

   b) There may be instances where the available storage in tidal creeks (gates closed conditions) on the interior are under-estimated due to the terrain capturing water surface elevations rather than the bottom of the creek bed.

   c) Sensitivity analyses were completed to assess the performance of the pumping operations during varying initial interior elevations. The assessment is provided in Appendix 2 of the Interior Drainage sub-appendices.

Per EM 1110-2-1413, the interior drainage system (storm gates and pump stations) should provide interior flood relief such that during low exterior stages (gravity conditions) the local storm drainage system functions essentially as it did without a levee in place for floods up to that of the storm drainage design. Using the guidance provided in EM 1413, the storm gates and pump stations were assessed for non-storm surge conditions meaning the future without-project geometry was computed using various rainfall frequencies while assuming constant high tide (MHHW). The future with-project geometries, both open system and closed system, were computed using various rainfall frequencies while assuming steady constant tide (MHHW) and the results were compared assess the potential interior "ponding" effect.

During a hurricane event with non-overtopping storm surge, the storm surge wall will greatly reduce water levels in the interior area regardless of pump capacity. The purpose of the pump stations is to remove rainfall accumulated during gates closed conditions. However, there may be instances where pumps are needed during gates open conditions if there are local depression areas that experience flooding due to the project.

To evaluate the performance of each with-project pump alternative, the inland water surface elevations are compared to the without-project without storm surge simulations. In addition to assessing the performance of the proposed system during non-storm surge conditions, the system was also assessed assuming extreme storm surge which causes wave wash overtopping. These simulations assess the performance of pumps for rainfall plus overtopping and compare the results to the performance of the pumps during rainfall only events. More information for storm surge overwash conditions is provided in the Interior Drainage sub-appendices.

As mentioned, the City's existing storm pipe network accommodates no more than a 10% AEP rainfall event meaning the system becomes overwhelmed for events of approximately and greater than this frequency. Surface flow becomes a major component of drainage during such instances. In some areas, the City indicates even lower capacities of stormwater routing. The interior drainage assessment signifies the performance of the system for the events of 50% AEP up to the 4% AEP due to the current capacities of the storm pipe network which brings stormwater to the pump stations.

5.10.4.1 HEC-RAS SIMULATIONS

HEC-RAS simulations were computed for open system conditions to assess the storm gates and closed system conditions to assess the pump stations. The simulations were computed using the 50%, 20%, 10%, 4%, 2%, and 1% AEP rainfall events combined with constant tidal boundary conditions.

<u>Note:</u> The following is a description of the approach for the comparison of events. The results of the events listed in the tables labeled FWO are compared to the respective event listed in the tables labeled FW for each plan alternative. Ex. FWO - exterior elevation (high tide) - interior rainfall (10% AEP) is to be compared to FW (storm gates open) – exterior elevation (high tide) – interior rainfall (10% AEP).

<u>Event Comparison Matrix (2032)</u>

Each event utilizes a constant stage boundary condition of the projected 2032 MHHW at 3.18 feet NAVD88. Table 5.10.9 displays the matrices for comparing Future Without (FWO) versus Future With (FW) for each condition and alternative plan.

Table 5.10.9 Event Comparison Matrix - 2032

Comparison matrix 2032	
FWO	
Exterior Elev	Interior rainfall
high tide	50% AEP rain
high tide	20% AEP rain
high tide	10% AEP rain
high tide	4 % AEP rain
high tide	2% AEP rain
high tide	1 % AEP rain

Comparison matrix 2032									
FW (storm gates open)		FW (storm gates closed)		FW (storm gates closed) P.S. alt 1		FW (storm gates closed) P.S. alt 2		FW (storm gates closed) P.S. alt 3	
Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall
high tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain
high tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain
high tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain
high tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain
high tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain
high tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain

<u>Event Comparison Matrix (2082)</u>

Each event utilizes a constant stage boundary condition of the projected 2082 MHHW at 4.27 feet NAVD88. Table 5.10.10 displays the matrices for comparing Future Without (FWO) versus Future With (FW) for each condition and alternative plan.

Table 5.10.10 Event Comparison Matrix 2082

Comparison matrix 2082	
FWO	
Exterior Elev	Interior rainfall
high tide	50% AEP rain
high tide	20% AEP rain
high tide	10% AEP rain
high tide	4 % AEP rain
high tide	2% AEP rain
high tide	1 % AEP rain

Comparison matrix 2082									
FW (storm gates open)		FW (storm gates closed)		FW (storm gates closed) P.S. alt 1		FW (storm gates closed) P.S. alt 2		FW (storm gates closed) P.S. alt 3	
Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall
high tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain
high tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain
high tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain
high tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain
high tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain
high tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain

5.10.4.2 SUMMARY OF HYDRAULIC MODEL RESULTS

Detailed results are provided throughout Chapter 4 of the Interior Drainage Sub-appendices (Hydraulics and Hydrology – HEC-RAS 2D Modeling).

Selected output locations were used to assess the impacts to interior water levels for the storm gates and pump station alternatives. Peak water surface elevations at 27 (Figure 5.10.11) selected output locations are tabulated to document the water surface elevation for each with-project condition to compare to the water surface elevations for the without-project conditions. The selected output locations provide a general sense of the performance of the system, however, areas not shown by these locations may experience different impacts.

The RAS results may show differences in interior water surface elevations for with- versus without-project conditions but the HEC-FDA model provides the tools for describing the economic consequences and/or benefits of those differences in interior water surface elevations. Further iterations between HEC-RAS and HEC-FDA are to be completed during PED phase to further assess the system on a site-by-site basis. The site-specific assessment will assist in appropriately designing each storm gate/pump station for the proposed interior drainage system and not to inflate the project cost due to over-design and/or under-design a portion of the system that could induce significant interior residual damages.

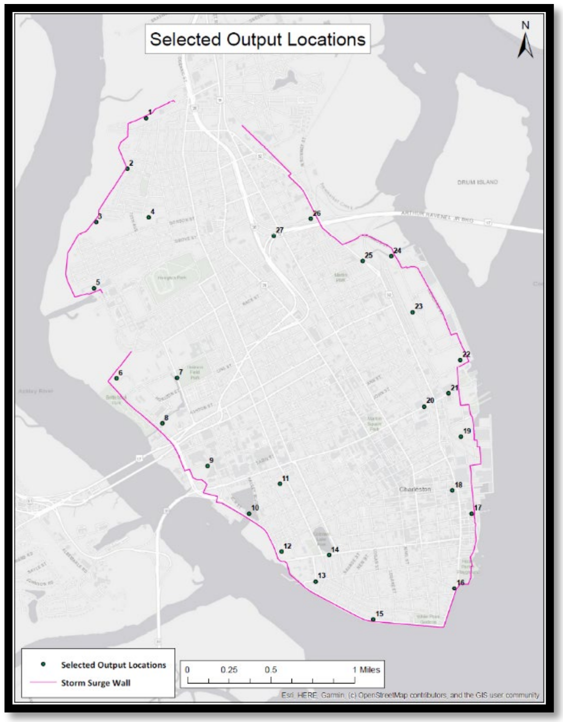


Figure 5.10.11 Selected Output Locations

The TSP must include the interior drainage facilities such that no induced flood damages occurs during low exterior stages (gravity conditions). In addition, other alternatives or enhancements can be assessed that could further improve interior drainage relief. Such instances relevant to the project study area could include the increased capacity of existing gravity outlets, increased pump station capacities, and increased stormwater pipe capacities. Other instances such as stormwater detentions are not realistically an option for the downtown area. During PED phase, a better understanding of the City's completed and planned stormwater improvements is necessary for appropriately designing the system along with more information regarding the existing storm pipe network which is assumed to be utilized for the USACE proposed pump stations.

As mentioned, the City's existing storm pipe network accommodates no more than a 10% AEP rainfall event meaning the system becomes overwhelmed for events of approximately and greater than this frequency. Surface flow becomes a major component of drainage during such instances. In some areas, the City indicates even lower capacities of stormwater routing. The interior drainage assessment signifies the performance of the system for the events of 50% AEP up to the 4% AEP due to the current capacities of the storm pipe network which brings stormwater to the pump stations.

Review of the future with-project storm gates open assessment displays the project produces similar water surface elevations as compared to the future without-project as most locations for the 50% and 20% AEP rain events for the years 2032 and 2082. The 10% AEP event displays that five locations incur increases greater than 0.5 feet for the year 2032 and two locations for the year 2082. The 4% AEP event displays that five locations incur increases greater than 0.5 feet for the year 2032 and four locations for the year 2082. The 2% AEP event displays that five locations incur increases greater than 0.5 feet for the year 2032 and five locations for the year 2082. The 1% AEP event displays that six locations incur increases greater than 0.5 feet for the year 2032 and five locations for the year 2082.

Review of the future with-project storm gates closed (pump station) assessment displays the project significantly increases interior water surface elevations at many locations for the condition with No USACE pump stations. The pump station alternatives display an increase of greater than 0.5 feet for 4-5 locations for the 50%, 20%, and 10% AEP events for the year 2032. The pump station alternatives display an increase of greater than 0.5 feet for 2-3 locations for the 50%, 20%, and 10% AEP events for the year 2082. The pump station alternatives display several locations with a greater than 0.5 feet increase for the events greater than the 10% AEP for both the years 2032 and 2082.

The pump station assessment also displays various locations with decreased elevations as a benefit of the project due to the improvement of drainage efficiency via pumping rather gravity flow and due to the storm surge wall prohibiting damages that may occur during MHHW tide elevations. As noted, the RAS model does not have the capability of modeling sub-surface drainage potentially computing higher than expected inland water surface elevations however for both without- and with-project conditions.

Many of the output locations that incur the noticeably increased water surface elevations, for storm gates open and storm gates closed, are at low-lying areas where increased elevations may not increase flood damages. For example, the Halsey Creek tidal area is shown to experience increased elevations within the banks of the creek but may not experience damage inducing elevations on land if those increased elevations are not producing out of bank inundations. In addition to low-lying creek areas, there are potentially low-lying isolated depression areas that could incur increased elevations as result of the storm surge wall.

At this phase of the study, the closed system conditions have been assessed holistically, i.e., the pump station alternative capacities have not been mixed and matched per location. In PED phase, the pump station alternatives can be mixed and matched for the site-specific analysis to appropriately accommodate the needed interior drainage relief per servicing area or watershed area. The site-specific assessment will assist in appropriately designing each storm gate/pump station for the proposed interior drainage system not to inflate the project cost due to over-design and not to under-design portions of the system that could induce significant interior residual damages.

While the RAS results display the differences in interior water surface elevations for with- versus without-project conditions, the HEC-FDA model provides the tools for describing the economic consequences and/or benefits for the differences in interior water surface elevations.

5.10.4.3 ECONOMIC ASSESSMENT OF PROJECT ALTERNATIVES

5.10.4.3.1 HEC-FDA METHODOLOGY

This section summarizes the HEC-FDA methodology and results. The detailed assessment is provided in Section 5.3 of the Interior Drainage Sub-Appendices (Hydraulics and Hydrology – HEC-RAS 2D Modeling).

The HEC-RAS results for future without- and with-project conditions were incorporated into the Hydrologic Engineering Center's (CEIWR-HEC) Flood Damage Reduction Analysis (HEC-FDA) to compute the Equivalent Annual Damages (EAD) and Average Annual Damages (AAD) to describe the risk of interior residual flooding. The HEC-FDA assessment served as an economic tool to determine the drainage features and their capacities (storm gates/pump stations) necessary for implementation into the project alignment.

The project alignment has undergone "re-alignment" in areas near the Port of Charleston since the FDA effort for interior drainage was completed. Some pump stations have been relocated to accommodate this project re-alignment along with the incorporation of a storm gate at the creek the Port of Charleston. Other changes that have been made to the TSP, since the FDA effort was completed, include the reduction of the number of storm gates in the Wagener Terrace area. The referenced project modifications have been incorporated into the RAS model. The RAS results presented in previous sections include those modifications, however, the FDA model has not since been updated. The FDA model reflects the results from the previous TSP prior to the mentioned modifications.

The FDA model computed damages to structures on a per Model Area (MA) basis. There are five model areas delineated for Economics: Model Area 1 (Battery), Model Area 2 (Port), Model Area 3 (Newmarket), Model Area 4 (Marina), and Model Area 5 (Wagener Terrace). To compute FDA, the software requires “index locations” with hydrologic data such as stage-discharge functions from the RAS model. For each MA, one index location was selected, therefore five total index locations. The RAS model was used to derive stage-discharge and discharge-frequency functions for each index location for each modeled scenario as presented in Section 5.1 of the Interior Drainage sub-appendices.

For example, the RAS model for the future with-project pump station alternative 2 scenario was used to extract the stage and flow at an index location for the 50% AEP, then for the 20% AEP, then for the 10% AEP, and so on. For each AEP, the stage and associated flow was tabulated to develop the stage-discharge and discharge-frequency functions. Also derived are the stage-damage functions.

Once the FDA model is setup, the damages (number of structures and associated dollar amounts) were computed for all without-project storm frequencies and all with-project storm frequencies and project alternatives for the years 2032 and 2082. Incremental damages are then calculated by comparing a future with-project alternative to the future without-project alternative for the same storm frequency. Incremental damages can be assessed for an entire project alternative and can be further assessed by reviewing the incremental damages occurring at a specific model area.

In addition to incremental damages for number of structures and associated dollar amounts, the storm frequencies, incremental probabilities, and stage-damage functions are used to compute the average annual damages. The average annual damages of the future without-project is compared to a future with-project alternative for determining the “Damage Reductions” and whether the project alternative induces or reduces the average annual damages.

HEC-RAS detailed water surface elevation grids and inundation boundary polygons were provided to the economics team member. A structural damage assessment was completed using a structures inventory layer, RAS water surface grids, and GIS tools to assess the structures getting “wet” for each scenario. “Heatmaps” are developed to present the inundated structures while attributing a range of dollar amounts associated to the inundated structures in the form of a color ramp with values and symbols.

5.10.4.3.2 SUMMARY OF ECONOMIC MODEL RESULTS

Storm Gates Open (USACE Pump Stations Inactive)

A review of the storm gates open assessment displays the project inundates one additional structure for the 50% AEP in 2032 and no additional for the 50% AEP in 2082. The 20% AEP inundates 2 additional structures in 2032 and 3 additional in 2082. The 10% AEP inundates 6 additional structures in 2032 and 8 additional in 2082. The 4% AEP inundates 9 additional in 2032 and 9 additional in 2082. The 2% AEP and 1% AEP inundate 8 to 12 additional in both 2032 and 2082. The results display the project with storm gates open increases the average annual damages by approximately $41,700 in the year 2032 and $45,000 in the year 2082.

Storm Gates Closed (USACE Pump Stations Active)

A review of the storm gates closed assessment for pump station alternatives displays the pump station alternatives 1, 2, and 3 significantly reduce the number of structures inundated and dollar damages for the 50% and 20% AEP rain events for the years 2032 and 2082. Pump station alternative 1 slightly reduces the number of structures

damaged for the 10% AEP while alternative 2 and 3 significantly reduce the number of structures damaged for the 10% AEP for the years 2032 and 2082.
Pump station alternative 1 displays reduced number of structures inundated and reduced structure damages in dollars for the 50% AEP to the 10% AEP while additional damages are induced for the events greater than the 10% AEP referring to both 2032 and 2082.

Pump station alternative 2 displays reduced number of structures inundated and reduced structure damages in dollars for the 50% AEP to the 4% AEP while additional damages are induced for the events greater than the 4% AEP referring to both 2032 and 2082.

Pump station alternative 3 displays reduced number of structures inundated and reduced structure damages in dollars for the 50% AEP to the 2% AEP while additional damages are induced for the 1% AEP referring to both 2032 and 2082.

The average annual damages for pump station alternative 1 display a decrease for the years 2032 and 2082 of approximately $248,600 and 316,500 respectively. The average annual damages for pump station alternative 2 display a decrease for the years 2032 and 2082 of approximately $613,500 and $737,800 respectively. The average annual damages for pump station alternative 3 display a decrease for the years 2032 and 2082 of approximately $1.01 million and $1.08 million.

5.10.4.3.3 TENTATIVELY SELECTED PLAN FOR INTERIOR DRAINAGE

Upon reviewing the results of the hydraulic assessment and economic assessment, the pump station alternative 2 capacities (Section 5.10.3.6) were selected to be implemented as part of the project alignment. For the overall system performance, pump station alternative 2 displays damage reductions in the average annual damages as presented in the previous sections. The pump station alternatives were assessed holistically as a system meaning the pump station alternative capacities were not mixed and matched for assessing each pump location individually. However, the FDA model does provide damage assessments on a per model area basis which provides into the performance of the pump stations. This approach assists in the selection of pump station alternative 2 for implementation to the TSP for the project alignment.

The storm gate dimensions and locations that have been assessed and chosen to be implemented as part of the TSP are displayed in Section 5.10.3.6. One alternative for storm gates were assessed in HEC-FDA. Further assessment is to be conducted during PED phase to refine storm gate dimensions if needed during the site-specific analysis.

While the economic assessment displays pump station alternative 2 provides a reduction in average annual damages (AAD) for the system, a site-specific assessment is to be completed during PED Phase to appropriately size each pump station and/or storm gate. Assessing the damages at a finer scale will be more insightful to individually sizing pump stations as some areas may overcompensate interior drainage relief while some may undercompensate. Further assessment may reveal some pump station capacities could be reduced from the current TSP while some pump station capacities may need increase. Typically, features that improve interior drainage above the without-project interior drainage are to be incrementally justified. The site-specific assessment is critical to not over-designing the system, inflating project cost, while also considering the effects of under-designing a system which may under-perform, leading to induced interior risk.

5.10.5 PUMP STATION INFORMATION

There are two types of pumps that are proposed, permanent pump stations and temporary pumps. More detail about each type and the potential locations for each is shown below. Locations of both the permanent pumps and the temporary pumps are shown in figure 5.10.12 below.

Figure 5.10.12 Locations of Permanent and Temporary Pump Stations

5.10.5.1 PERMANENT PUMPS

Permanent Pump Houses shown on Figure 5.10.1 are located at

- Halsey Creek
- Marsh behind the Baseball Stadium
- Longs Pond
- By the Coast Guard Base
- Newmarket Creek

Permanent pump stations would consist of a wet well installed in a low-lying area where water will likely collect and is connected to a pump house. The pump house will hold the electrical infrastructure, backup generator, etc. and will be elevated such that the electrical infrastructure is kept above the potential flood elevation (see Section 5.10.3 for more on the pump house). The wet well will be located in a low-lying area such as a marsh or tidal creek where water will naturally collect. The wet well consists of a concrete inlet box with mesh screens for debris and wildlife protection, hinged lid for pump removal for maintenance, etc. See Figure 5.10.2.2 below for an example drawing of the wet well. The outlet from the wet well will be routed to the wall and will either pass over the wall or through it with a check valve to prevent inflow from the river side.

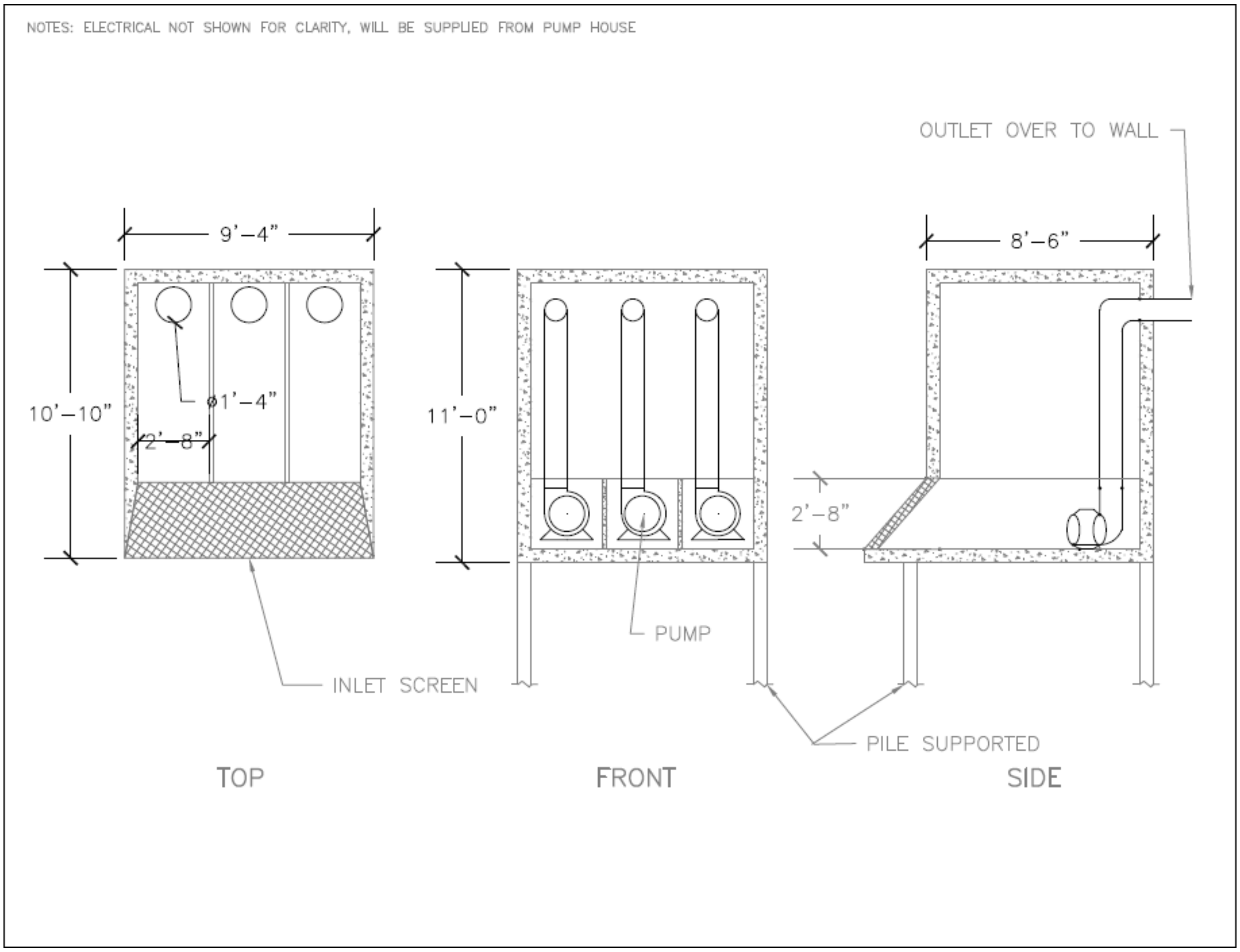


Figure 5.10.2.2 Pump Station Wet Well

The intent is to construct the pump stations with as minimal an impact to the marsh as possible. Therefore, the pump houses will be built on dry land with only the wet well-located marsh adjacent.

Pumps will be electric powered and will have a back-up diesel-powered generator located in the pump house, which will maximize the flexibility and ensure the pumps can run even if the storm has disrupted the electricity supply.

Each pump station will have a total of three equally sized pumps. This will allow for two-thirds redundancy where even if one pump fails, the station is still able to operate at two thirds capacity. The sizing of the pumps is based on modelling of expected rainfall during storm events to ensure there is adequate capacity in each model area to remove rainwater trapped by the new wall.

5.10.5.2 PORTABLE PUMPS

There are 5 locations that have been selected to receive temporary pumps. The locations based on the analysis performed are in the following locations shown on Figure 5.10.1 above. These may change in PED phase after the subsurface system is analyzed.

To pump out the rainfall in locations that do not have a natural, low lying area such as a tidal marsh, temporary pumps shall be utilized. These five proposed locations, which are along the battery on the south end of the peninsula and on the Cooper River side are also too congested with roads, houses, and other infrastructure for a pump house and wet well setup which is being used in other areas. Therefore, at the five predetermined locations, an inlet pipe will be installed which will tap into the existing storm drainage system for the peninsula, and an outlet pipe that goes over or through the wall. During storm events, a portable pump shall be brought to the location and hooked up to the inlet and outlet pipes to pump the rainfall in that area over the wall to avoid the bathtub effect the new wall will otherwise impart. Other than the inlet and outlet pipes, only a small pad with anchors and an electrical box to connect the pumps to grid power will be installed. This will minimize the needed real estate, visual impact, and overall effect on the proposed areas for the temporary pumps. Temporary pumps shall have built in backup diesel generators to allow them to function even if grid power fails. The pumps are sized based on modeling of rainfall for storm events to ensure there is adequate capacity to handle the projected flow. The pumps shall be trailer mounted and portable so they can be moved and stored off site when not needed.

5.10.5.2 PUMP HOUSES

Pump stations for interior drainage will be required in five locations. The systems will be automated and will not require a safe house for personnel. Equipment will need to be elevated above the flood elevation and contained in a building for protection. The city requirement is 2’ above BFE for items/areas that are not floodproofed. Typically, that is the electrical panels and controls for structures like pump houses. For our assessment the floor elevation is set at EL 17. Figure 5.10.3.1 shows the site plan, while Figure 5.10.3.2 shows the floor plan and Figure 5.10.3.3 shows the cross-section. Pump Stations were designed to accommodate an emergency generator; however, it was noted that existing City of Charleston pump stations do not contain emergency generators and are powered solely by the local utility company.

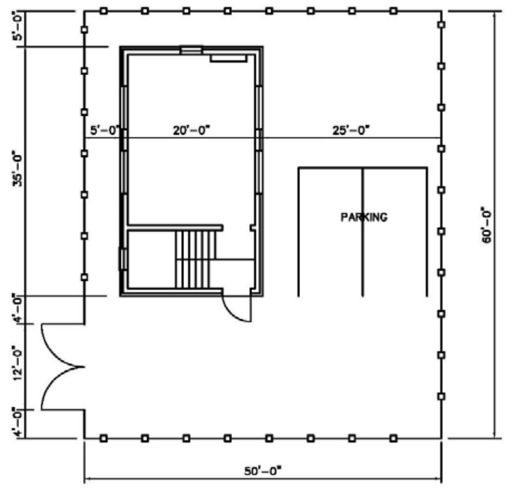


Figure 5.10.13 Site Plan of Pump Station

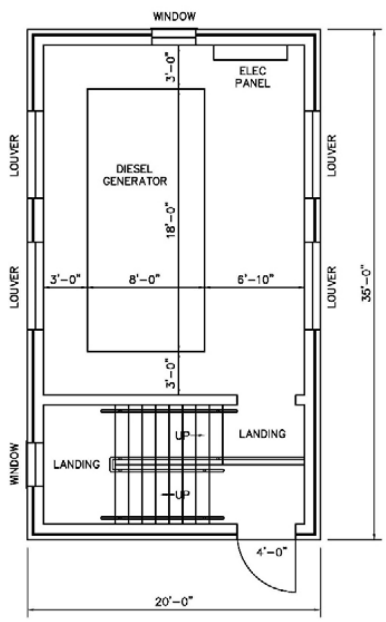


Figure 5.10.14 Floor Plan of Pump Station

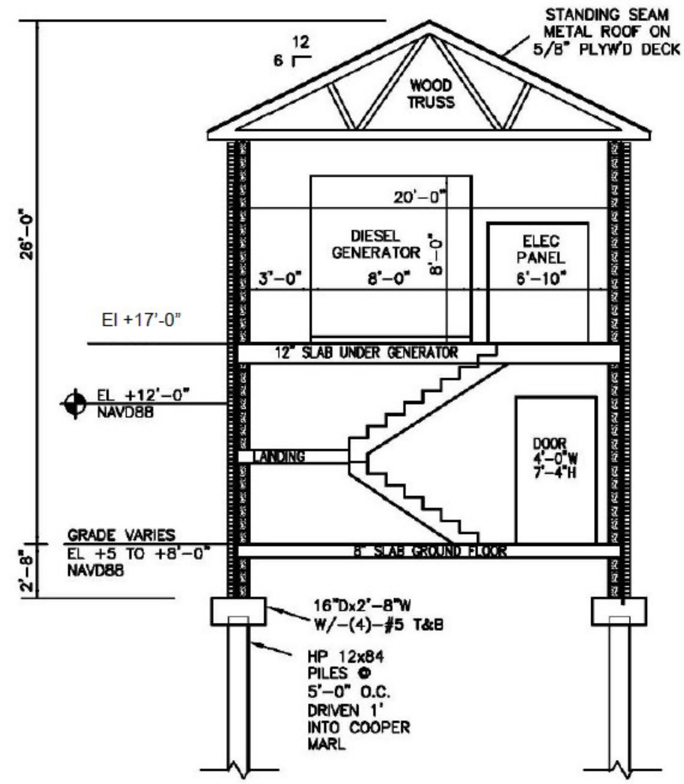


Figure 5.10.15 Typical Section of Pump Station

5.10.5.3 PUMP PREVENTIVE MAINTENANCE

To ensure the new pumps are fully functional when needed, a regimen of preventative maintenance will be required. A summary of preventative maintenance items is shown below and more detail will be provided in a full Operations and Maintenance manual, which will be further developed during PED Phase.

5.10.5.4.1 PERMANENT PUMPS

Regular maintenance of the pumps will include monthly exercising of them to check proper operation, adding grease, checking seals and gaskets and replacing as needed. The intake screens on the wells will need to be cleaned regularly to remove debris, algae, seaweed, etc. The frequency of maintenance will be monthly, with a major functionality check immediately before any large storms.

The median life expectancy of sump and well pumps according to ASHRAE is ten years. Due to the infrequent usage and low run hours, it is likely that the pumps will last for roughly twenty years which means that at least one if not two full replacements of the pumps themselves will be required during the life of the project.

5.10.5.4.2 PORTABLE PUMPS

Regular maintenance of the pumps will include monthly exercising of them to check proper operation, adding grease, checking seals and gaskets and replacing as needed. Additionally, the built-in diesel motor/generator set will require maintenance such as oil changes, filter changes, etc. The premade hookup locations will also require the intake screens cleaned at least monthly.

Because the temporary pumps are stored offsite and out of the weather, and coupled with the infrequent use, the expected life of the pumps is roughly thirty to forty years. Therefore, it is reasonable to assume that only some of the temporary pumps will require a full replacement during the life of the project. The remaining ones will last for the life of the project with regular maintenance and replacement of a few major components that fail.

5.11 WAVE OVERTOPPING

Structural design, discussed in the structural appendix, considered that the structure would be expected to withstand wind generated wave overtopping. Overtopping of the floodwall by the free flowing still water elevation is an indication of failure of defense, but not failure of the structure so long as the structure is designed for overtopping without structural failure.

Wave overtopping is primary concern for structures constructed to defend against flooding. Storm surge is driven by storm winds and waves as documented by Still Water Level (SWL). Peak surge elevations will be greater if the storm surge coincides with the tide. This is two parts of the total water level which includes still water, tide and wave runup. Local waves developing over inland water bodies such as the harbor can also develop. Waves running up the face of the wall can be high enough to pass over the crest of the wall and waves breaking on the structure can result in significant volume of splash. The following sections summarizes overtopping by still water elevation, dynamic still water level and overwash due to wave action that is explained in more detail in the Coastal Engineering Subappendix.

Figure 5.11.1 shows project area a red line showing flood wall with height +12 ft NAVD 88. White dots show 9 representing stations where statistical Still Water Level (SWL) and wave informationare available which are used to calculate wave overtopping flow using EUROTOP method. Methodologyis explained in Coastal Engineering Sub-Appendix.

Figure 5.11.1: Charleston Harbor Project Area with 9 SWL and wave data locations

Figure 5.11.2 shows location of representing stations with bathymetric depth. Here the numbers in black represent bathymetric depth in meter (NAVD88). Figure 5.11.3 shows still water level (SWL) at different points under different Annual Exceedance Probability (AEP). There is little variability in SWL among various points across the harbor. For example, for any representing station, 1% AEP (100-year return period) SWL (without considering sea level rise) is 3.1m (10.2 ft).

Figure 5.11.2: Representing Stations with Bathymetry Information

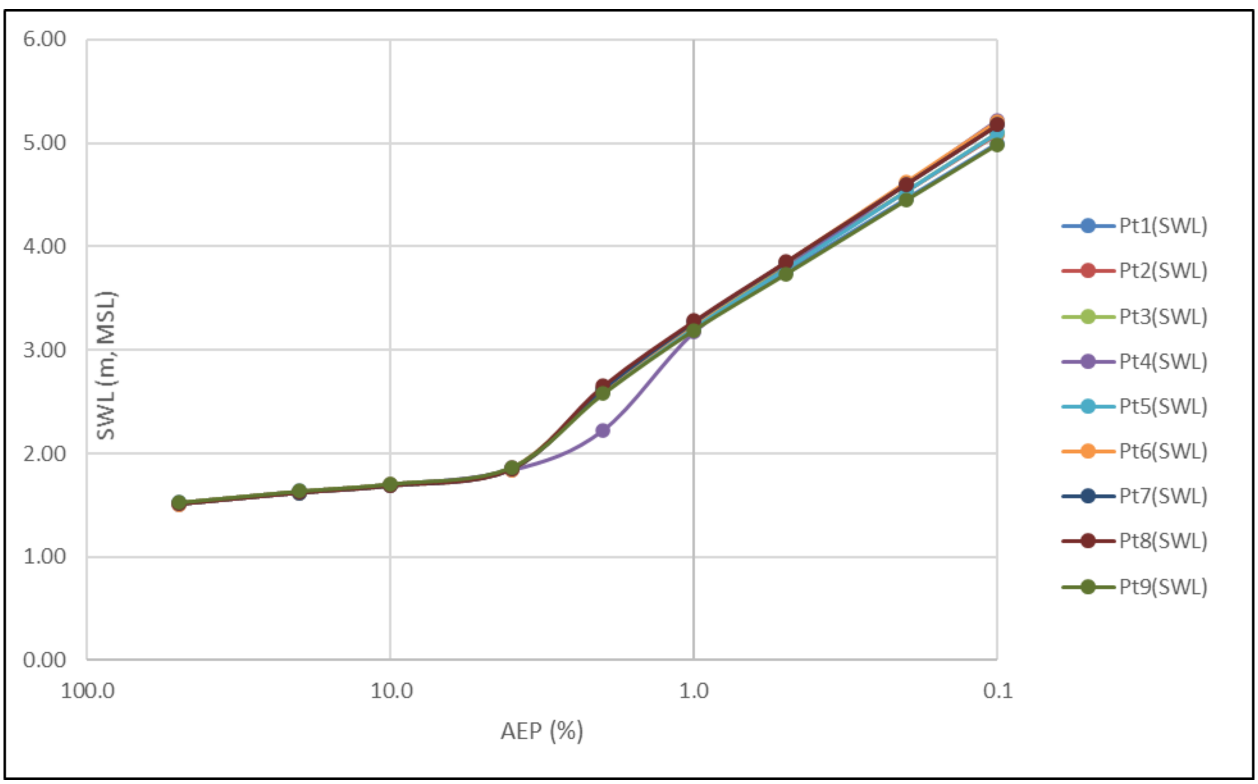


Figure 5.11.3: Still Water Level at Different Stations

Although SWL does not vary, significant wave height (HS) varies depending on the location. Figure 5.11.4(a)shows significant wave height along the Western side of the harbor where 1% AEP wave height is between 0.5 to 0.6m. Figure 5.11.4(b) shows significant wave height along the Eastern side of the harbor where 1% AEP wave height is between 0.8 to 1.4m. Figure 5.11.4(c) shows significant wave height along the Southern tip of harbor where 1% AEP wave height is between 0.7 to 1.2m. in general, due to larger water depth and long fetch, eastern and southern parts of the harbor experiences larger wave energy.

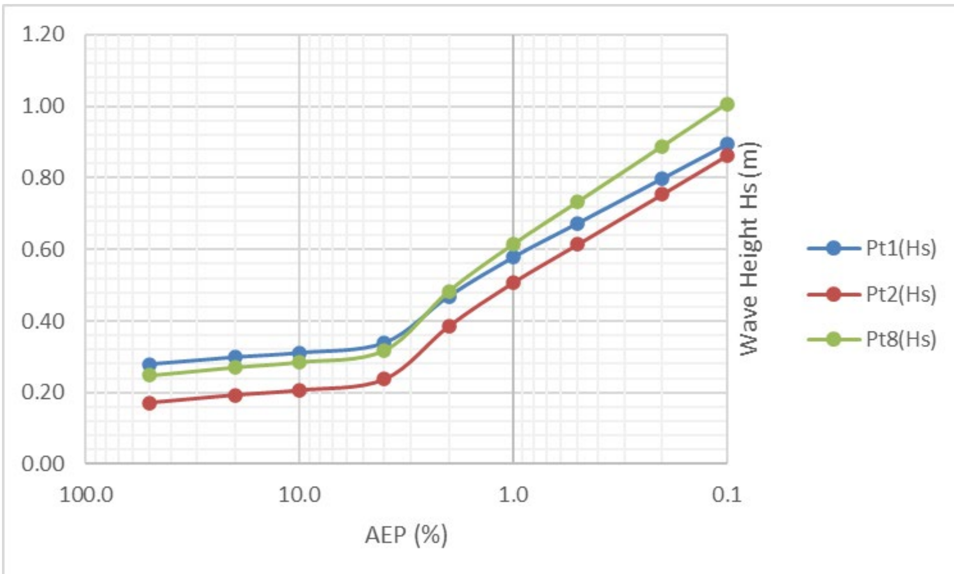


Figure 5.11.4(a): Significant Wave Height (HS) at Stations 1, 2, and 8

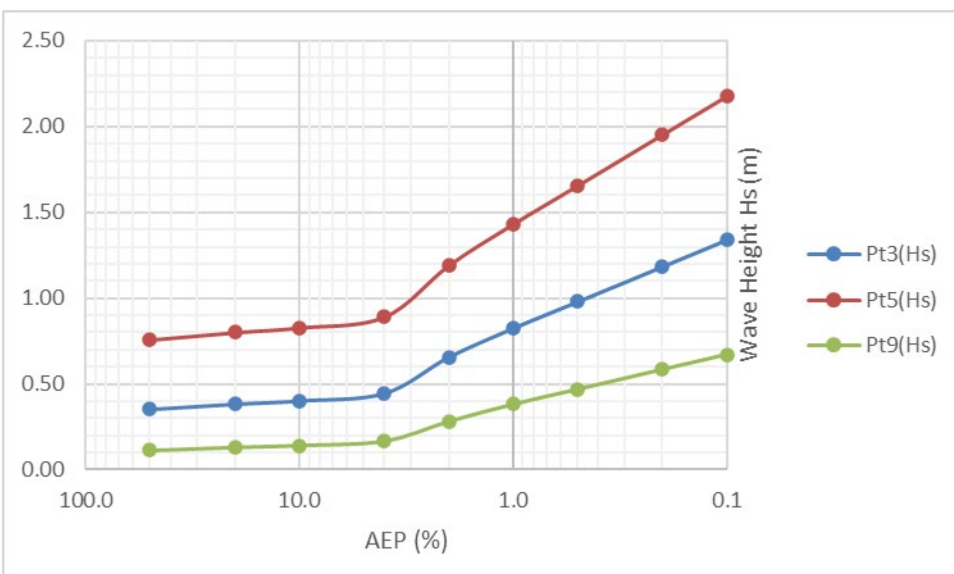


Figure 5.11.4(b): Significant Wave Height (HS) at Stations 3, 5, and 9

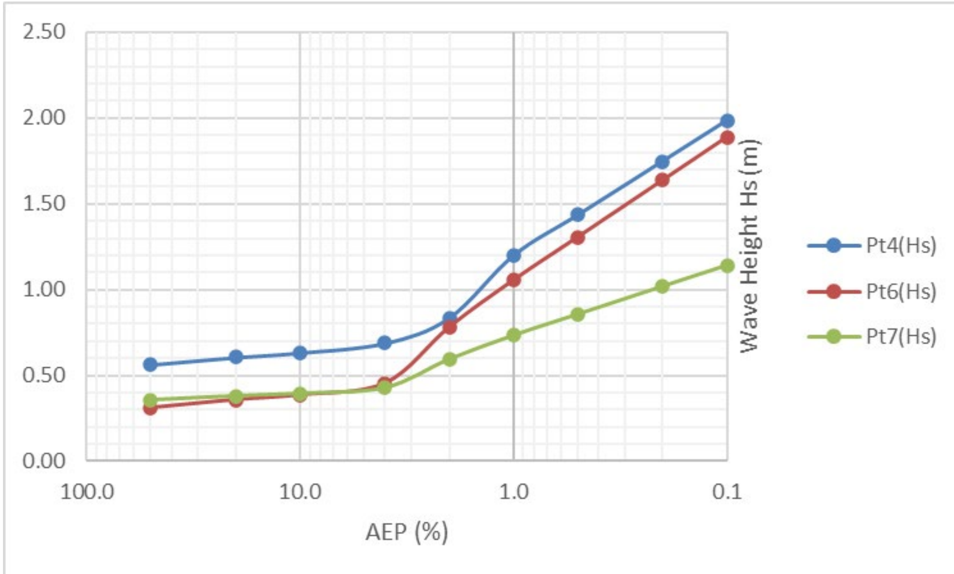


Figure 5.11.4(c): Significant Wave Height (HS) at Stations 4, 6, and 7

EUROTOP Methodology has been used to calculate overtopping flow (Figure 5.11.5). SWL has been adjusted for year 2082 with RSLC value = 1.65 ft and datum correction. Since floodwall elevation is set at +12 ft NAVD 88, when SWL is close to 12 ft, there will be free flow to be calculated as broad crested weirflow.

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.05\exp\left(-2.78\frac{R_c}{H_{m0}}\right) \quad \text{non-impulsive} \qquad 8.50$$

Impulsive conditions:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.011\left(\frac{H_{m0}}{hs_{m-1,0}}\right)^{0.5}\exp\left(-2.2\frac{R_c}{H_{m0}}\right) \quad \text{valid for } 0 < R_c/H_{m0} < 1.35 \qquad 8.51$$

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.0014\left(\frac{H_{m0}}{hs_{m-1,0}}\right)^{0.5}\left(\frac{R_c}{H_{m0}}\right)^{-3} \quad \text{valid for } R_c/H_{m0} \geq 1.35 \qquad 8.52$$

Figure 5.11.5: Key equations for overtopping flow calculation

Figure 5.11.6 shows wave overtopping flow calculated at Station 6. Here Red line shows AEP (2% in this case) at which point SWL considering RSLC plus one wave amplitude exceeds flood wall height of 12 ft NAVD. This happens roughly at 50-year return period. According to HSDRRS Guideline, for the 1% annual exceedance probability (1% AEP) still water, wave height and wave period, the maximum allowable average wave overtopping values are 0.1 cfs/ft at 90% level of assurance and 0.03 cfs/ft at50% level of assurance for floodwalls. For Station 6, we find this value to be 1.25 l/s/m or 0.013 cfs/ft. This is well below the HSDRRS limit state and hence considered tolerable.

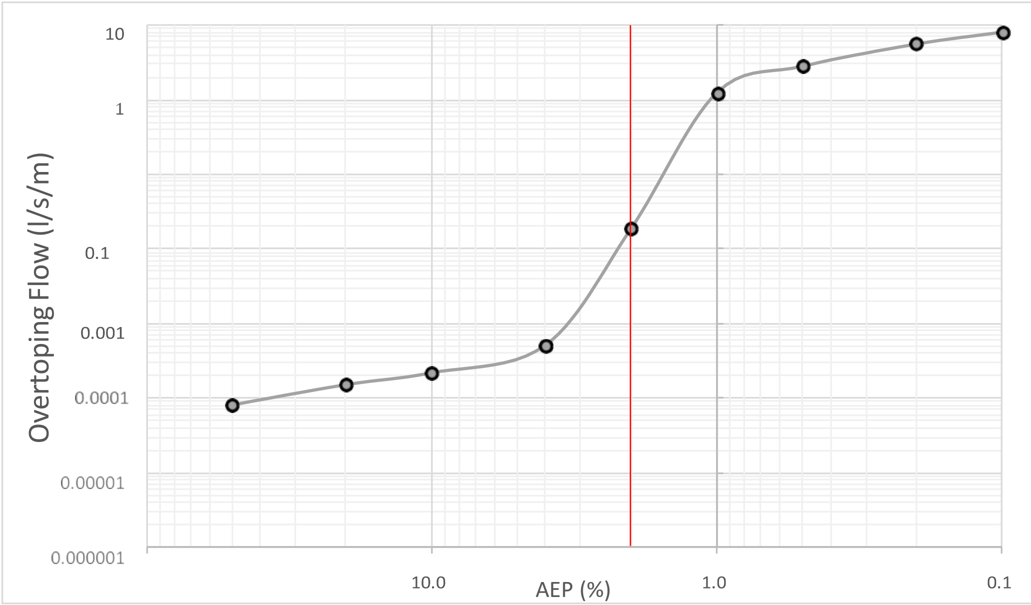


Figure 5.11.6: Overtopping Flow Calculated at Station 6.

Although overtopping flows are negligible and do not exceed limit state, figure 5.11.7 is presented to show estimated flow (1% AEP) that may be considered for drainage analyses. For simplicity, these flows are grouped into three regions – sheltered Western Region (stations 1, 2, 8, 9) where wave energy is low, Southern tip (Stations 4, 6, 7) where wave energy are relatively moderate and Eastern Section (3, 5) where wave energy are low to moderate. Accordingly, overtopping flows are shown in the following table (Table 5.11.X). Representing flood wall lengths should be multiplied with these flows to calculatetotal flow volume.

Table 5.11.1 Overtopping Flows

Reaches & Stations	Overtopping Flow (CFS/FT)
Western Region (stations 1, 2, 8, 9)	0.006
Southern tip (Stations 4, 6, 7)	0.013
Eastern Section (3, 5)	0.009

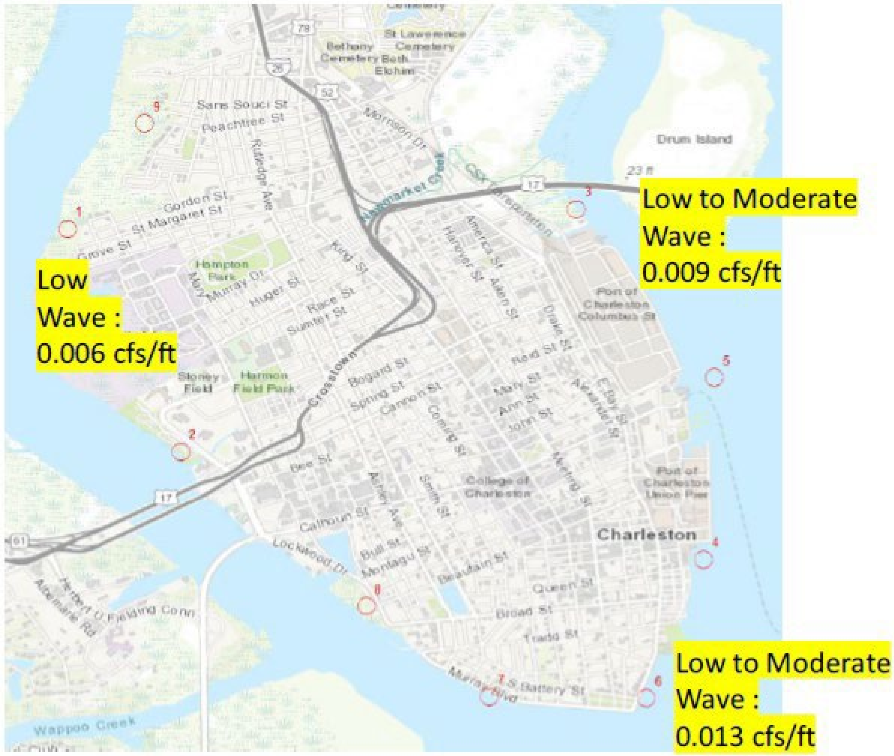


Figure 5.11.7: Overtopping Flow along Different Reaches

5.12 QUANTITY ESTIMATES

The following sections provide the quantities for each structural measure that were determined for the 12' NAVD88 elevation along the proposed alignment. Figure 5.12.1 shows the locations of each wall type, where the red is the T-wall and the green is the combo wall.

Figure 5.12.1 Wall types (yellow=Twall; Green =Combo Wall)

5.12.1 COMBO WALL QUANTITY CALCULATIONS

Prestressed Concrete Piles - 12" SQ

- Pile Spacing 5 ft
- Pile Embedment 6 ft (embedment depth in Cooper Marl)
- Pile Embedment 1 ft (embedment depth in Pile Cap)

Battered Pile Slope: 4 V 1 H

- Pile size 12 in. (Battered Pile)
- Pile size 12 in. (Vertical Pile)

Prestressed Concrete Sheet Piles

- Length 30 ft
- Width 10 in

Concrete Cap

- Cap Thickness 3 ft
- Cap Width 10 ft
- Cap Seg. Length 10 ft

12 FT ELEV (NVD88) WALL

Quantities used for construction cost estimate are detailed in Table 5.12.1.

Table 5.12.1 Combo Wall Quantities

			Description	Length (ft)	Cooper Marl Elev (ft)	Concrete Qty (CY)	Precast Cap Units (EA)	Vertical Pile Qty (EA)	Battered Pile Qty (EA)	Concrete Pile Pcs (EA)
Phase 1 - MARINA										
1+74.96	to	9+82.49	citadel to joe	807.53	-55	897	81	163	163	969
99+73.03	to	108+39.38	coast guard	866.35	-75	963	87	174	174	1040
				1,673.88		**1860**	**167**	**337**	**337**	**2009**
Phase 2 - BATTERY - NO COMBO WALL PROPOSED FOR THIS PHASE										
Phase 3 - PORT/NEV										
169+04.98	to	174+15.08	off high battery and CYC	510.1	-75	567	51	103	103	612
				510.1		**567**	**51**	**103**	**103**	**612**
Phase 4 - WAGNER 1										
8+25.	to	13+65.	Diesel Creek	540	-55	600	54	109	109	648
20+40.	to	49+37.69	Upper Wagner Terrace	2,897.69	-55	3220	290	581	581	3477
50+12.96	to	80+56.76	Halsey to lower	3,043.80	-55	3382	304	610	610	3653
				6,481.49		**7202**	**648**	**1299**	**1299**	**7778**

5.12.2 T WALL QUANTITY CALCULATIONS

T-WALL SLAB DIMENSIONS

- Width 10 FT
- Thickness 3 FT
- Soil Overfill 2 FT
- Min. Depth 5 FT (below grade)

T-WALL STEM DIMENSIONS

- Thicknesses:
- Base 2 FT
- Top 1 FT

H-PILE DIMENSIONS

- 12 x 84 H-Piles
- Embed Depth 6 FT (embedment depth in Cooper Marl)
- Embed Depth 1 FT (embedment depth in Slab)
- Spacing 5 FT
- Battered Pile Slope: 4V 1H

SHEET-PILE DIMENSIONS

- PZ 22 STEEL
- Depth 21 FT

EXCAVATION CALCULATIONS

- Excavation Width: 15 FT (assume 2.5 ft on each side)

Assume a 1V:2H side slope

Table 5.12.2. details the quantities used in the construction cost estimate.

Table 5.12.2. T wall Quantities

Stations			Description	Length (ft)	* Average Existing Grade Elev. (ft)	Average Excavation Depth (ft)	Bottom of Slab Elev. (ft)	Top of Slab Elev. (ft)	* Cooper Marl Elevation (ft)	Wall Height (ft)	Vertical H-Pile Qty (EA)	Battered H-Pile Qty (EA)	Total H-Pile Quantity (LF)	Total H-Pile Quantity (EA)	Total Sheet Pile Quantity (SF)
Phase 1 - MARINA															
0+00	to	1+74.96	Citadel	174.96	10.5	5	5.5	8.5	-55	4	36	36	4934	72	3674
9+82.49	to	77+60.	Brittlebank to Marina	6,777.51	4.9	5	-0.1	2.9	-55	9	1357	1357	170519	2713	142328
				6,952.47						**TOTALS:**	**1,392.5**	**1,392.5**	**175,453**	**2,785**	**146,002**
Phase 2 - BATTERY - NO STANDARD T-WALL PROPOSED FOR THIS PHASE															
Phase 3 - PORT/NEW MARKET															
174+15.08	to	201+32.29	CYC -waterfront park	2,717.20	5.5	5	0.5	3.50	-60	8.5	544	544	74630	1089	57061
201+32.29	to	300+73.68	otel -thru Columbus St	9,941.40	5.0	5	0.00	3.00	-65	9.00	1989	1989	290864	3979	208769
				12,658.60						**TOTALS:**	**2,534**	**2,534**	**365,495**	**5,067**	**265,831**
300+73.68	to	339+48.84	newmarket	3,875.16	5.3	5	0.3	3.30	-55	8.7	776	776	98182	1552	81378
339+48.84	to	371+47.2	newmarket	3,198.36	8.4	5	[illegible].4	6.40	-55	5.6	641	641	85089	1281	67166
371+47.2	to	371+87.26	Upper new market	40.06	12.85	5	7.85	10.85	-55	1.15	9	9	1278	18	841
				7,113.58						**TOTALS:**	**1,426**	**1,426**	**184,549**	**2,851**	**149,385**
				19,772.18							**3,959**	**3,959**	**550,044**	**7,919**	**415,216**
Phase 4 - WAGNER TERRACE															
0+00	to	8+25.	Diesel Creek	825.00	9.8	5	4.8	7.8	-55	4.2	166	166	22519	332	17325
13+65	to	20+40.	Lowndes Point	675.00	5.5	5	0.5	3.5	-55	8.5	136	136	17262	272	14175
49+38	to	50+12.96	park	75.27	3.6	5	-1.4	1.6	-55	10.4	16	16	1976	32	1581
80+56.76	to	90+36.46	Citadel	979.70	8.3	5	3.3	6.3	-55	5.7	197	197	26116	394	20574
				2,554.97						**TOTALS:**	**515**	**515**	**67,872**	**1,030**	**53,654**

5.12.3 T-WALL WALKING PATH

T-WALL SLAB DIMENSIONS

- Width 10 FT
- Thickness 3 FT
- Soil Overfill 2 FT
- Min. Depth 5 FT (below grade)

T-WALL STEM DIMENSIONS

- Thicknesses:
- Base 2 FT
- Top 1 FT

WALKWAY DIMENSIONS

- Wall Thickness: 1 FT
- Slab Thicknesses 8 IN
- Walkway Width 9 FT

H-PILE DIMENSIONS

- 12 x 84 H-Piles
- Embed Depth 6 FT (embedment depth in Cooper Marl)
- Embed Depth 1 FT (embedment depth in Slab)

- Spacing 5 FT
- Battered Pile Slope: 4V 1H

SHEET-PILE DIMENSIONS

- PZ 22 STEEL
- Depth 21 FT

EXCAVATION CALCULATIONS

- Excavation Width: 15 FT (assume 2.5 ft on each side)
- Assume a 1V:2H side slope

Table 5.12.3 Estimated T-Wall With Walking Path Quantities

Stations			Description	Length (ft)	* Average Existing Grade Elev. (ft)	Average Excavation Depth (ft)	Bottom of Fdn Elev. (ft)	Top of Fdn Elev. (ft)	* Cooper Marl Elevation (ft)	Wall Height (ft)	Excavation Volume (CY)	Concrete Volume - Fdn (CY)	Concrete Volume - Stem (CY)	Concrete Volume - LS Wall (CY)	Concrete Volume - WW Slab (CY)	Earth Fill Under WW Slab (CY)	Total H-Pile Quantity (LF)	Total H-Pile Quantity (EA)	Total Sheet Pile Quantity (SF)
Phase 1 - MARINA																			
77+60.	to	99+73.03	Marina to Lockwood/Broad	2,213.03	4.9	5	-0.1	2.9	-65	9	10,246	2459	1119	623	437	4546	64772	887	46474
				2,213.03						**TOTALS:**	**10,246**	**2,459**	**1,119**	**623**	**437**	**4,546**	**64,772**	**887**	**46,474**
Phase 2 - BATTERY																			
155+62.3	to	169+04.98	high battery	1,342.68	4.5	5	-0.5	2.5	-75	9.5	6216	1492	709	398	265	2758	44610	539	28196
				1,342.68						**TOTALS:**	**6,216**	**1,492**	**709**	**398**	**265**	**2,758**	**44,610**	**539**	**28,196**
Phase 3 - PORT/NEW MARKET - No T-Wall with Walking Path Proposed for this phase																			
Phase 4 - WAGNER TERRACE - No T-Wall with Walking Path Proposed for this phase																			

Additionally, there were gates, pumps, pump houses, utility crossings, and upgrade of the lower battery wall.

- Five pump houses with three pumps each
- 5 temporary pumps
- 10 storm surge gates (sluice gates)
- 59 vehicle gates
- 14 pedestrian gates
- 3 railroad gates

More details are in the Cost Sub-Appendix.

5.13 COST ESTIMATES

The baseline cost estimate for the proposed measures, tentative selected plan and the recommended plan were developed using MCACES in the Civil Works Work Breakdown Structure format. Quantities were calculated and provided by the designer engineers in the Charleston District. Real Estate costs for permanent and construction easements and acquisition were based on parcel data provided by the city and cost estimates were provided by USACE Real Estate personnel. Utility relocations and penetration through the wall were based on available data, more detailed data will be obtained in PED phase. The cost estimate for each feature was escalated to the midpoint of construction using the most current indices for Civil Works Construction Cost Index System (CWCCIS) EM 1110-2-1304. For this project a Cost and Schedule Risk Analysis (CSRA) was performed on a 5% design. Since the design level is so low (5% design), this could inherently result in cost uncertainties that are captured by higher cost contingencies. For more information on the Cost Estimates and the Total Project Cost Summary (TPCS) and CSRA performed on this project, refer to the Cost Engineering Sub-Appendix.

5.14 ENGINEERING RISK AND UNCERTAINITY

Risk is a measure of the probability (or likelihood) and consequences of uncertain future events. Risk analysis is a decision-making framework that explicitly evaluates the level of risk if no action is taken and recognizes the monetary and non-monetary costs and benefits of reducing risks when making decisions. A variety of variables and their associated uncertainties may be incorporated into the risk assessment of a coastal storm risk management study. Design conditions for major coastal and flood protection projects are often vague and design parameters contain large uncertainties.

5.14.1 LIFE SAFETY RISK ASSESSMENT

The Levee Safety Center completed an abbreviated Semi-Quantitive Risk Assessment (SQRA) for the planning study for the Charleston Peninsula in the Charleston District. The project consists of approximately 7.8 miles of floodwall and approximately 90. This risk assessment is in general accordance with ECB 2019-15 (InterimApproach for Risk-Informed Designs for Dam and Levee Projects), draft EC-1165-2-218 (Levee Safety Program – Policy and Procedures), and ER 1110-2-1156 (Safety of Dams – Policies and Procedures). This effort consisted of a facilitated Potential Failure Mode Analysis (PFMA) and a risk assessment of the potential failure modes judged to be risk drivers.

The incremental risk is plotted on a life safety risk matrix, this risk matrix is further described in Planning Bulletin 2019-04 and Engineering and Construction Bulletin 2019-15. A copy of the standard risk matrix is provided below.

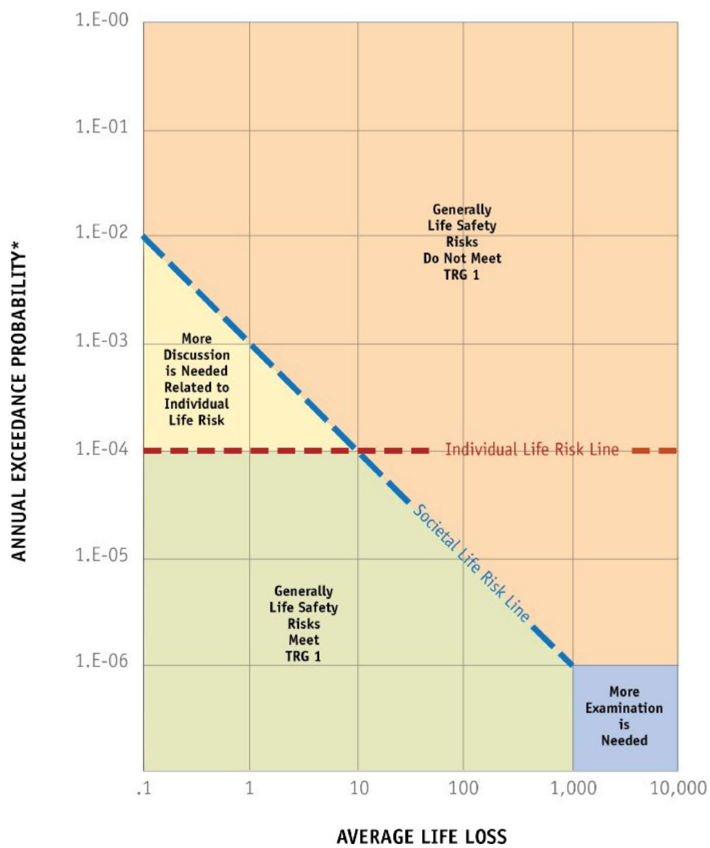


Figure 5.14.1 Standard Life Safety Risk Matrix

The incremental risk is judged to be primarily driven by the inability to install all of the closures and overtopping with breach. The estimated total annual probability of failure (APF) is between 1E-04 and 1E-03 failures per year (Figure 5.14.2). The probability is estimated to be straddling the tolerable risk guidelines. All other potential failure modes were judged to be well below tolerable risk and were excluded. There is a potential for large economic consequences as a result of a breach on this project. Life loss is estimated to be low as the loading scenario would be a hurricane event and there would likely be ample warning time to evacuate the site and there is a high evacuation rate. The estimated weighted average incremental life loss is between 1 and 10 lives per failure. Economic impacts were not developed specifically for this risk assessment; information on economic impacts is available in the feasibility report.

The primary incremental risk drivers for the levee system for the proposed design is the following

- Overtopping with breach
- Failure to close all of the gates

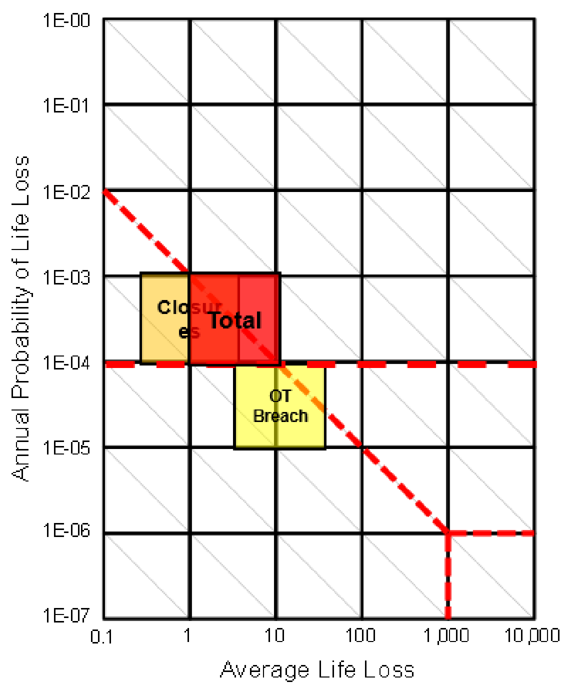


Figure 5.14.2 Societal Incremental Life Safety Risk Matrix

While the floodwall is not currently constructed, the team did estimate non-breach risk for the proposed floodwall. The non-breach risk for the floodwall is estimated to be between 3E-03 and 3E-02 with estimated life loss between 1 and 10. (Figure 5.14.3).

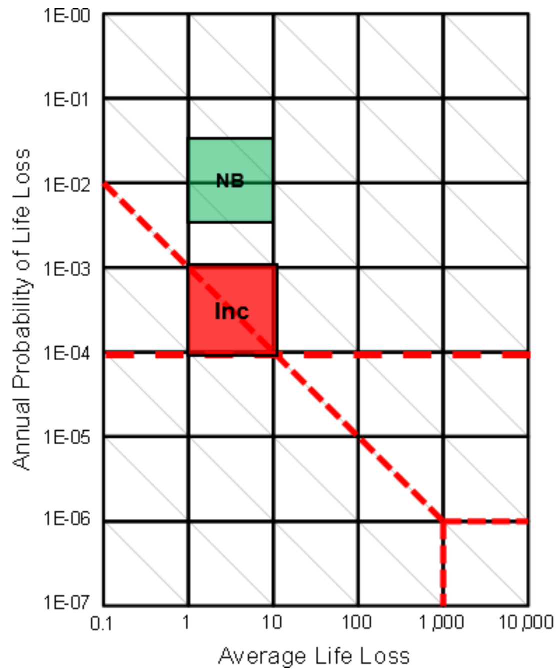


Figure 5.14.3 Levee Area Societal Life Safety Flood Risk Matrix
(Note: Matrix shows Non-Breach (NB) and Incremental (Inc) Risk)

None of the PFMs were judged to be risk drivers. The PFM consequence estimates are based on results from the G2CRM modeling efforts which considers with and without project conditions but does not consider breach of the floodwall. Based on additional modeling efforts using the LST methodology which incorporates floodwall breaches, the estimated life loss results for a floodwall breach were within the same order of magnitude estimate as those resulting from G2CRM. Therefore, the team was able to use the G2CRM life loss estimates. There were a few PFMs that were addressed in greater detail but were believed to all plot around the tolerable risk line. Those PFMs will be addressed in the following sections. The PFMs with the highest risk are:

- PFM 7: Overtopping, scour, and undermining of the T-Wall or Combo-Wall due to Wave Overtopping
- PFM 8: Overtopping, scour, and undermining of the T-Wall or Combo-Wall due to Still Water and Waves
- PFM 12: Misoperation of Gates
- PFM 20: Overtopping, scour, and undermining of Gates
- PFM 39: Mechanical Failure of Gate Operating System
- PFM 42: Pump Stations Fail and Gates will not Open to Release Storm Water

The team later combined PFM 7, 8 and 20 into a single PFM that addressed overtopping. The team determined the height of the wall based on the existing topography of the Peninsula. A wall height of greater than 12’ was not deemed practical due to a number of reasons. The 12’ height was chosen as the height due to the existing contours of the Peninsula. Looking at the contours of the Peninsula in the figures below it can be seen that while the wall could have tied in at the 13’ level in the neck area there was no tie in at the Citadel location. At the 14’ and 15’ contours show that there was no practical place to tie in at the neck area. There are some contours on each of the maps that are artificially elevated as they are buildings or the interstate, it was determined these were not the correct contours and tie in at those locations were not practicable. In addition to tie in at the 13’ and 14’ level significant modifications would have been required to the Ashley River bridge and the newly renovated low battery wall that the NFS is in the process of constructing. The team determined that effort to be cost prohibitive and not a practical solution. PFM 12 and 39were also combined into a single PFM for gate(s) not being closed. The team has discussed with the NFS the importance of gate maintenance and operations to reduce and will provide a draft O&M manual to reduce the risk associated with gate failure. More information can be found in the Charleston Planning Risk Assessment Sub-Appendix.

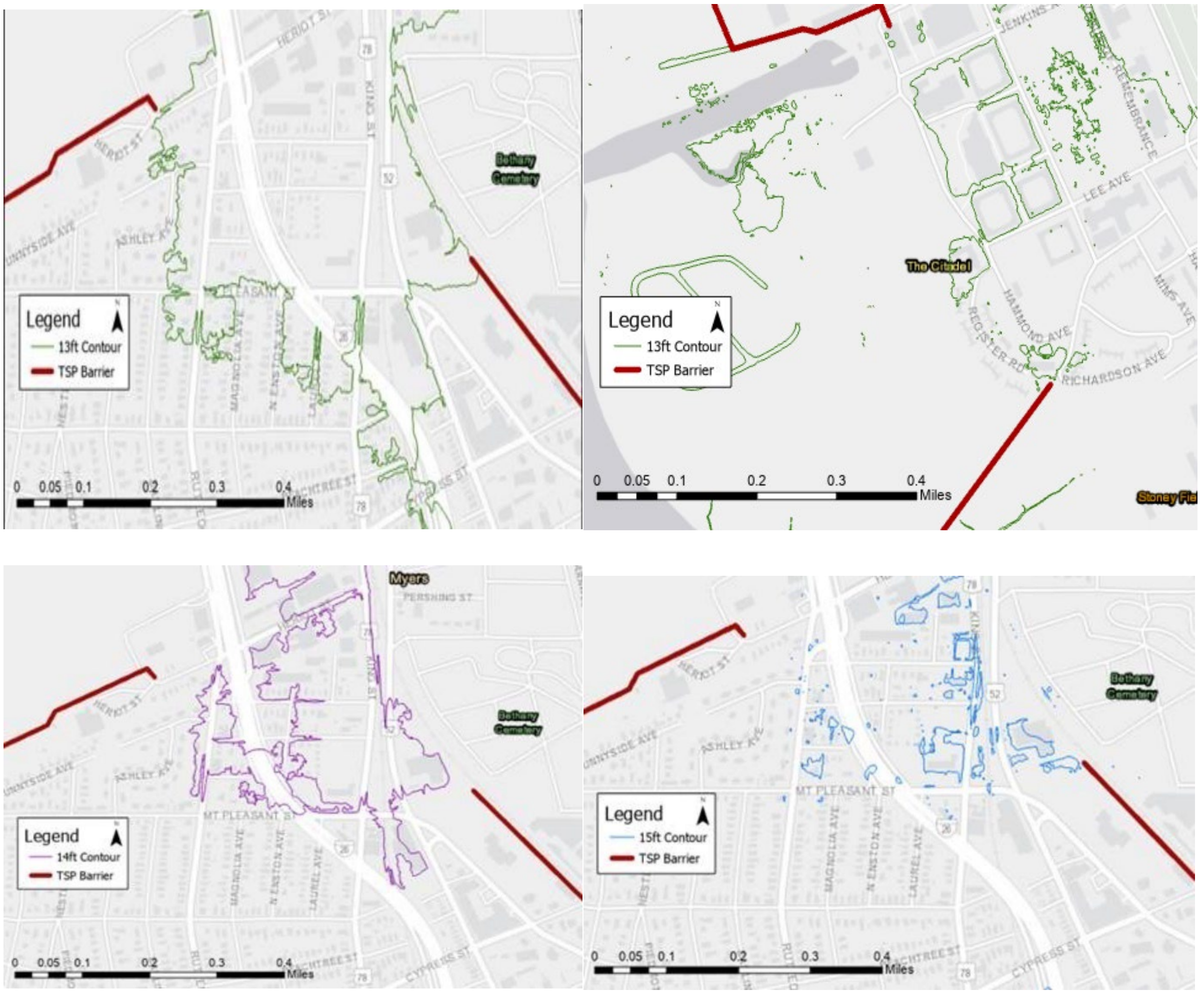


Figure 5.14.4 Example Contours of Different Heights Showing Lack of Tie-In Locations

The conclusion of the SQRA stated that, "The estimated average annual life loss for Future Without Project and Future With Project conditions are essentially equal," assuming a "highly effective evacuation rate", with the following caveats:

- "In the past the Charleston Peninsula area has had high evacuation rates and the community follows a formal evacuation plan. Over time, those evacuation rates have decreased, and it is uncertain if the population at risk will become overly confident in the ability of the floodwall to prevent inundation of the area and potentially have evacuation rates reduce further."
- "The incremental and non-breach risk will be higher ... if evacuation rates are reduced. Ineffective evacuation rates can in an increase in non-breach risk and an increase in incremental life loss above tolerable risk guidelines."

The two primary potential failure modes of overtopping of the wall and failure of the gates for a variety of reasons due pose some risk to the community. The 12’ wall height was chosen based on the existing topography and the pump stations have been designed to mitigate for wave overtopping but not surge overtopping or failure to operate properly. The team had determined it would not be feasible to justify the cost of pumps for surge overflows, however, additional analysis of wave overtopping and surge overtopping will be assessed during PED phase. The primary focus of the feasibility assessment demonstrated that the storm surge wall (with the interior drainage features) during a rainfall plus overwash events is less damaging to the inland area that the sae storm surge without a wall. The team has worked with the NFS to ensure the community understands evacuation will still be required during a storm event to minimize life safety risks as there is risk associated with these PFMs.

5.14.2 SWL CONFIDENCE LIMITS

Overtopping is primary concern for structures constructed to defend against flooding. Storm surge is driven by storm winds and waves as documented by Still Water Level (SWL). Peak surge elevations will be greater if the storm surge coincides with the tide. Local waves developing over inland water bodies such as the harbor can also develop. Waves running up the face of the wall can be high enough to pass over the crest of the wall and waves breaking on the structure can result in significant volume of splash. Overtopping of the floodwall by the free flowing still water elevation is an indication of failure defense but not failure of the structure so long as the structure is designed for overtopping without structural failure. The structure has been designed to withstand still water overtopping.

Wind generated wave overtopping analyses and the non-linearity assessment provided the justification for the method to determine probability of overtopping by still water elevation. Based on analysis, the maximum estimate for NLR was -0.15 m, which is a negative bias. The negative bias means that simple superposition of RSLC with storm surge model output will produce a higher water level estimate than compared to directly including RSLC within the storm surge model. Thus, the linear superposition of RSLC with storm surge model output can be used to estimate water levels for various probability storms under the effect of RSLC, which is a conservative approach.

Using FEMA still water elevation levels from the most recent Flood Insurance Study, ERDC generated an Annual Exceedance Probability (AEP) for each of the five save points requested (refer to Figure 5.1). Still water level values in MSL were converted to NAVD88 and sea level rates were applied. The still water surge elevation is the water elevation due solely to the effects of the astronomical tides, storm surge, and wave setup on the water surface, but which does not include wave heights. It is important to note, however, this differs from the base flood elevation because the still water level does not include wave regeneration that occurs over a large body of water before it reaches the shoreline.

Wave heights vary depending on direction and speed of the storm and the same storm will generate different wave heights on opposite sides of the peninsula, thus the probability of wave height is not directly associated with the probability of the storm.

ER 1105-2-101 states that the mean AEP values be used for economic analyses, but that when communicating project performance, the AEP values at the 90% confidence level should be used. AECOM, contractor for FEMA, provided confidence limit formulas to apply. Tables 5.2-5.4 list the AEP with the upper 90% confidence limit (UCL) at the 5 locations selected for model areas for the year 2032. Table 5.2 uses the USACE low sea level curve value, Table 5.3 uses the intermediate curve value, and Table 5.4 uses the high curve value. Figure 5.13 is the same information plotted. Probabilities are also tabulated in Table 5.5 based on Long-Term Exceedance Probabilities (LTEP), or probability of exceedance over each indicated time interval for the 50% assurance values. The table displays the mean AEP for exceedance of 12 ft NAVD88 at the five locations selected for model areas, using each USACE sea level change scenario at year 2032 and corresponding LTEP.

Table 5.2. Year 2032 annual exceedance with 90% confidence for USACE Low SLC curve

	SWL (ft NAVD88)	**2032**	**SLR=**	**0.41**	**ft**				
Location	AEP+SLR +90%UCL (%) 50	AEP+SLR +90%UCL (%) 20	AEP+SLR +90%UCL (%)10	AEP+SLR +90%UCL (%) 4	AEP+SLR +90%UCL (%) 2	AEP+SLR +90%UCL (%) 1	AEP+SLR +90%UCL (%) 0.5	AEP+SLR +90%UCL (%) 0.2	AEP+SLR +90%UCL (%) 0.1
Wagener Terrace	8.47	8.94	9.23	9.90	13.03	15.62	17.89	20.89	23.16
Marina	8.43	8.90	9.19	9.86	13.14	15.75	18.08	21.16	23.49
Newmarket	8.43	8.89	9.18	9.85	13.07	15.63	18.00	21.12	23.49
Port	8.40	8.86	9.15	9.81	13.02	15.59	17.99	21.18	23.59
Battery	8.39	8.85	9.14	9.80	13.02	15.69	18.10	21.29	23.71

Table 5.3. Year 2032 annual exceedance with 90% confidence for USACE Intermediate SLC curve

	SWL (ft NAVD88)	**2032**	**SLR =**	**0.56**	**ft**				
Location	AEP+SLR +90%UCL (%) 50	AEP+SLR +90%UCL (%) 20	AEP+SLR +90%UCL (%)10	AEP+SLR +90%UCL (%) 4	AEP+SLR +90%UCL (%) 2	AEP+SLR +90%UCL (%) 1	AEP+SLR +90%UCL (%) 0.5	AEP+SLR +90%UCL (%) 0.2	AEP+SLR +90%UCL (%) 0.1
Wagener Terrace	8.66	9.13	9.42	10.09	13.23	15.82	18.08	21.08	23.35
Marina	8.63	9.09	9.38	10.05	13.33	15.94	18.27	21.35	23.68
Newmarket	8.62	9.09	9.38	10.05	13.27	15.83	18.19	21.32	23.68
Port	8.59	9.05	9.34	10.00	13.21	15.78	18.19	21.37	23.79
Battery	8.58	9.04	9.33	10.00	13.21	15.88	18.29	21.48	23.90

Table 5.4. Year 2032 annual exceedance with 90% confidence for USACE High SLC curve

	SWL (ft NAVD88)	**2032**	**SLR=**	**1.01**	**ft**				
Location	AEP+SLR +90%UCL (%) 50	AEP+SLR +90%UCL (%) 20	AEP+SLR +90%UCL (%)10	AEP+SLR +90%UCL (%) 4	AEP+SLR +90%UCL (%) 2	AEP+SLR +90%UCL (%) 1	AEP+SLR +90%UCL (%) 0.5	AEP+SLR +90%UCL (%) 0.2	AEP+SLR +90%UCL (%) 0.1
Wagener Terrace	9.24	9.71	10.00	10.67	13.80	16.39	18.66	21.66	23.93
Marina	9.20	9.67	9.96	10.63	13.91	16.52	18.84	21.93	24.26
Newmarket	9.20	9.66	9.95	10.62	13.84	16.40	18.77	21.89	24.26
Port	9.16	9.63	9.92	10.58	13.79	16.36	18.76	21.95	24.36
Battery	9.16	9.62	9.91	10.57	13.79	16.46	18.87	22.06	24.48

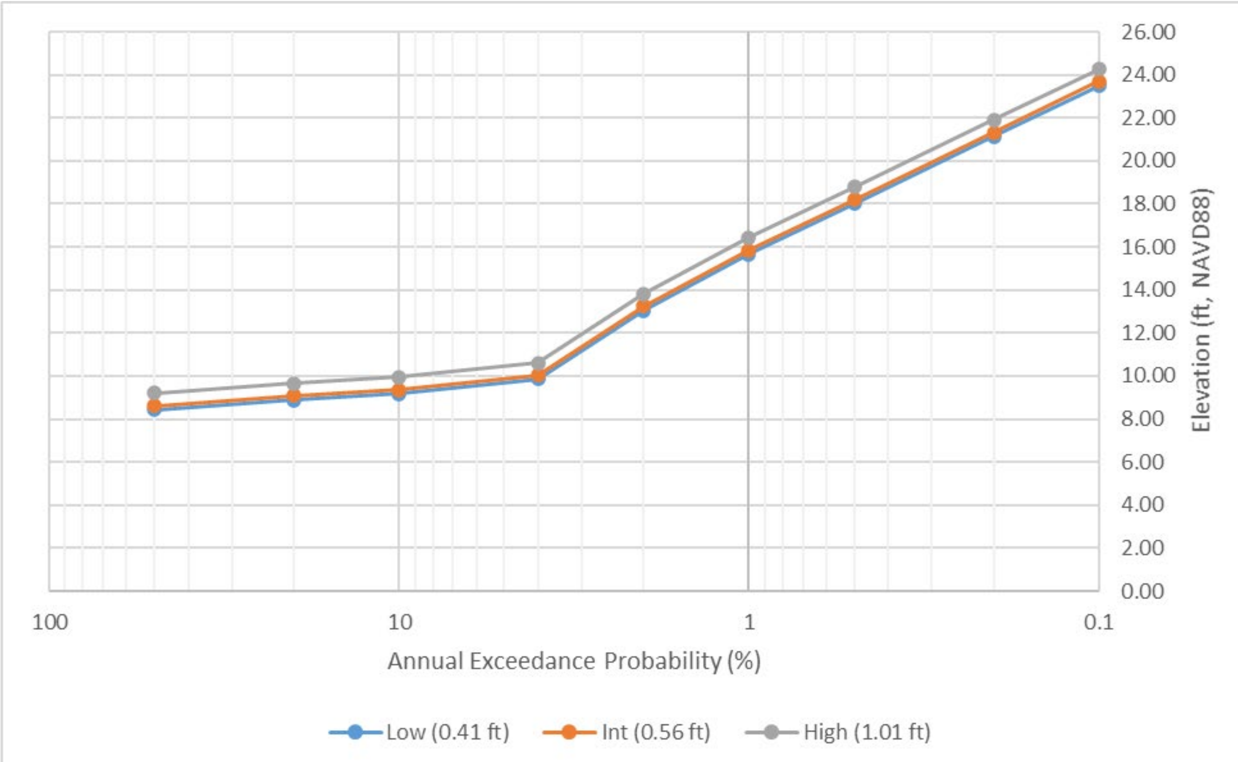


Figure 5.13 Year 2032 Annual Exceedance with 90% confidence

Table 5.5. Performance in 2032 described by mean AEP and LTEP

USACE SLC Scenario	Mean AEP at 12 ft NAVD88	LTEP (Probability of Exceedance Over Indicated Time)		
		10 Years	30 Years	50 Years
Low (0.41 ft)	0.007	0.07	0.18	0.29
Int (0.56 ft)	0.007	0.07	0.19	0.30
High (1.01 ft)	0.008	0.08	0.22	0.34

Tables 5.6-5.8 list the AEP with the upper 90% confidence limit (UCL) at the 5 locations selected for model areas for the year 2082. Table 5.6 uses the USACE low sea level curve value, Table 5.7 uses the intermediate curve value, and Table 5.8 uses the high curve value. Figure 5.14 is the same information plotted. Probabilities are also tabulated in Table 5.9 based on Long-Term Exceedance Probabilities (LTEP), or probability of exceedance over each indicated time interval for the 50% assurance values. The table displays the mean AEP for exceedance of 12 ft NAVD88 at the five locations selected for model areas, using each USACE sea level change scenario at year 2082 and corresponding LTEP.

Table 5.6. Year 2082 annual exceedance with 90% confidence for USACE Low SLC curve

	SWL (ft NAVD88)	**2082**	**SLR =**	**0.93**	**ft**				
Location	AEP+SLR +90%UCL (%) 50	AEP+SLR +90%UC L (%) 20	AEP+SLR +90%UCL (%)10	AEP+SLR +90%UCL (%) 4	AEP+SLR +90%UCL (%) 2	AEP+SLR +90%UCL (%) 1	AEP+SLR +90%UCL (%) 0.5	AEP+SLR +90%UCL (%) 0.2	AEP+SLR +90%UCL (%) 0.1
Wagener Terrace	9.14	9.60	9.90	10.57	13.70	16.29	18.55	21.55	23.83
Marina	9.10	9.57	9.86	10.53	13.80	16.41	18.74	21.82	24.16
Newmarket	9.09	9.56	9.85	10.52	13.74	16.30	18.66	21.79	24.16
Port	9.06	9.52	9.81	10.48	13.69	16.25	18.66	21.85	24.26
Battery	9.06	9.52	9.81	10.47	13.69	16.35	18.76	21.96	24.38

Table 5.7. Year 2082 annual exceedance with 90% confidence for USACE Intermediate SLC curve

	SWL (ft NAVD88)	**2082**	**SLR =**	**1.65**	**ft**				
Location	AEP+SLR +90%UCL (%) 50	AEP+SLR +90%UC L (%) 20	AEP+SLR +90%UCL (%)10	AEP+SLR +90%UCL (%) 4	AEP+SLR +90%UCL (%) 2	AEP+SLR +90%UCL (%) 1	AEP+SLR +90%UCL (%) 0.5	AEP+SLR +90%UCL (%) 0.2	AEP+SLR +90%UCL (%) 0.1
Wagener Terrace	10.06	10.53	10.82	11.49	14.62	17.21	19.48	22.48	24.75
Marina	10.02	10.49	10.78	11.45	14.73	17.34	19.66	22.75	25.08
Newmarket	10.02	10.48	10.77	11.44	14.66	17.22	19.59	22.71	25.08
Port	9.98	10.45	10.74	11.40	14.61	17.18	19.58	22.77	25.18
Battery	9.98	10.44	10.73	11.39	14.61	17.28	19.69	22.88	25.30

Table 5.8. Year 2082 annual exceedance with 90% confidence for USACE High SLC curve

	SWL (ft NAVD88)	**2082**	**SLR =**	**3.93**	**ft**				
Location	AEP+SLR +90%UCL (%) 50	AEP+SLR +90%UC L (%) 20	AEP+SLR +90%UCL (%)10	AEP+SLR +90%UCL (%) 4	AEP+SLR +90%UCL (%) 2	AEP+SLR +90%UCL (%) 1	AEP+SLR +90%UCL (%) 0.5	AEP+SLR +90%UCL (%) 0.2	AEP+SLR +90%UCL (%) 0.1
Wagener Terrace	12.98	13.45	13.74	14.41	17.54	20.13	22.40	25.40	27.67
Marina	12.95	13.41	13.70	14.37	17.65	20.26	22.59	25.67	28.00
Newmarket	12.94	13.41	13.69	14.36	17.59	20.15	22.51	25.64	28.00
Port	12.91	13.37	13.66	14.32	17.53	20.10	22.50	25.69	28.10
Battery	12.90	13.36	13.65	14.32	17.53	20.20	22.61	25.80	28.22

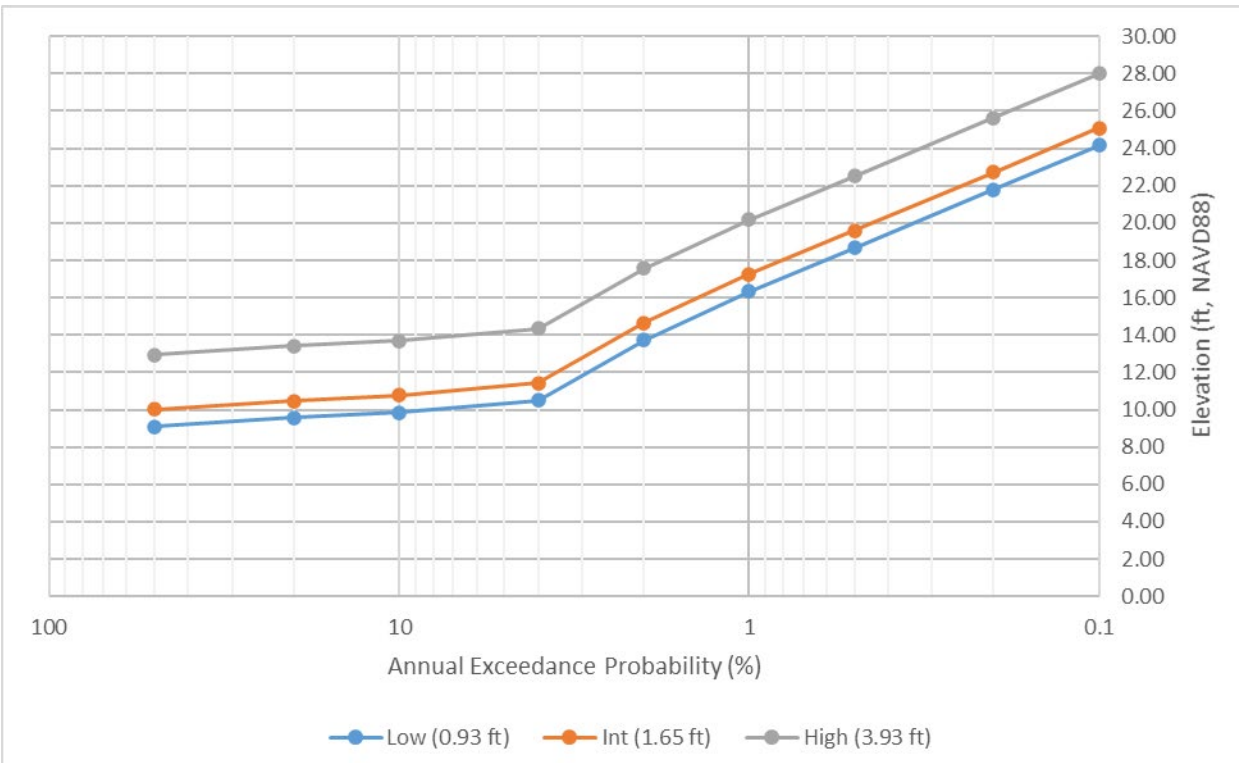


Figure 5.14 Year 2082 Annual Exceedance with 90% confidence

Table 5.9. Performance in 2082 described by mean AEP and LTEP

USACE SLC Scenario	Mean AEP at 12 ft NAVD88	LTEP (Probability of Exceedance Over Indicated Time)		
		10 Years	30 Years	50 Years
Low (0.93 ft)	0.008	0.08	0.22	0.34
Int (1.65 ft)	0.010	0.02	0.03	0.04
High (3.93 ft)	0.022	0.04	0.07	0.07

The tide range in Charleston is up to 6 feet, suggesting that the tide phase at the time of landfall may significantly influence surge levels produced by a given storm. Still water elevations were computed at MSL, therefore the risk of flooding at high tide must be considered when assessing risk and potential damages. This was considered in the G2CRM analysis ofdamages.

The existing still water elevation is documented in the FIS but it is not the Base Flood Elevation that is considered a better estimate of the flood hazard. To obtain the final Base Flood Elevations (BFEs), FEMA then uses WHAFIS, for the overland wave height analysis. The WHAFIS model can also cause wave regeneration if it goes over a sizable body of water. It can then dissipate as it passes over land as shown in Figure 5.15, obtained from FEMA contractor.

5.15 CONSTRUCTABILITY

The primary constructability issues for the Charleston Peninsula CSRM project are expected to be constructed adjacent to existing structures, construction near historic structures, construction in tidal marshes, soft soils and loose sands, man-made fill materials, unknown soil contamination, and traffic impacts.

Many construction activities produce potentially damaging vibration levels, including pile driving and removal, concrete and asphalt demolition, compacting soil with a vibratory compactor, and excavation. There will be many structures located adjacent to the construction, with some having historical significance. Most construction vibrations, except for pile driving, will dissipate relatively quickly. In general, vibratory pile drivers will produce lower vibration levels than impact pile drivers. Vibration damage from pile driving vibrations will not likely occur outside a radius equal to the length of length from the pile (either top or tip of pile, whichever is closer). With piles could be expected to approach 90 feet in length, preconstruction surveys will be required on structures within a 100-ft buffer from the wall centerline. Additionally, vibration monitoring will be required during construction as various locations throughout the area. In the case that dense sand and gravel layers are encountered above the Cooper Marl, or obstruction like large pieces of rubble, vibrations could increase in magnitude and the distance they travel.

Construction adjacent to existing structures also means that the temporary construction right-of-way must be minimized. Construction in tight quarters tends to take longer, which increases costs, and may be more dangerous for the workers.

Construction for the combo wall will occur in the tidal marshes. Access to the alignment will be limited. Dredging maybe required to get construction equipment in place. Tidal fluctuations may add difficulty to construction.

Soft clays or loose sands could be present at various locations throughout the peninsula. Loads placed on the soft clays will cause the foundation to consolidate. This could cause downdrag on piles or excessive settlement on adjacent shallow founded structures. Similarly, loose sands could be densified during the installation of piles, excessive settlement on adjacent shallow founded structures.

Man-made fill is likely to be present along the perimeter as it was used to expand the peninsula. The man-made fill could make pile driving difficult and could require pre-augering. These man-made fills could also be in loose states, causes settlement issues as described above.

If contaminated soil is encountered during excavation, it must be separated from uncontaminated soil and characterized and disposed of in a landfill licensed to accept the material. Contaminated soil is most likely to be encountered in areas with a history of industrial and railway use.

Construction will be near and along/across roadways and will negatively impact traffic and may require temporary lane or street closures and traffic monitors. The alignment also crosses numerous water access points.

During construction, weather could also impact work and schedule. Table 5.16.1 displays monthly anticipated adverse weather delays based on the National Oceanic and Atmospheric Administration (NOAA) or similar data for the project location and will constitute the baseline for monthly weather time evaluations.

Table 5.16.1 Monthly Anticipated Adverse Weather Delay Workdays Based on (5) day Work Week

JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
8	6	8	3	6	5	5	6	4	3	4	7

5.16 RESILIENCY & ADAPTABILITY -

Due to sea level rise and the harsh marine environment where the barrier is to be constructed, measures will be taken to ensure the barrier can adapt to our changing environment, as well as reduce required maintenance and ensure longevity.
All of the items listed below have been considered and will continue to be incorporated during the Preconstruction Engineering and Design (PED) Phase.

- Substructure and superstructure to accommodate future raising
- Plan for longevity and maintainability using durable materials like stainless steel
- Facilitate gate storage by storing nearby
- Facilitate gate deployment

5.16.1 INCREASING BARRIER HEIGHT

Since the I-Wall does not have any battered piles or major lateral resisting elements, an I-Wall would be the most difficult to increase the height if that needed to be done in the future. A toe on the concrete cap could be installed during initial construction, which would allow additional raising, but would add additional upfront costs. For that reason, an I-Wall is not part of the design.

In addition, the T-Wall and Combo Wall have battered piles which will be driven to the Cooper Marl stratum providing more lateral resistance. The load capacity of piles driven into Marl increases dramatically with each additional foot of penetration. The required pile embedment for the flood barrier with 3 feet of additional height has been calculated and accounted in the structural analysis for feasibility. During PED phase, the concrete reinforcement for all wall types should also accommodate the forces resulting from future increase in height. Future raising should only require dowelling into the top of the flood barrier to add rebar and to increase the height of the wall stem.

The low battery wall raising being done by the city puts a constraint on going any higher than elevation 12 NAVD88 due to the foundation design. Raising the Low Battery Wall in the future by an additional 3 feetwould require additional structural analysis and structural upgrades. These upgrades may consist of, butare not limited to, foundation upgrades and additional lateral support. These upgrades will be very difficult to construct and may result in major demolition and reconstruction of the Low Battery Wall.

There are some topographical constraints that would require more than just raising the wall. Additionally, the bridges into and out of the city on the Ashley River have height restriction now that the wall is passing under both highway 17 and James Island connector. Raising in these locations may require impacts to the bridges. The interstate I26 leading inland is a primary evacuation route and there are limited opportunities to connect to the abutment at any elevation higher than 12NAVD88, thus requiring a longer wall inland parallel to the interstate, gates across the interstate or raising the road.

5.16.2 CORROSION PREVENTION

This project is being built in the marsh, or near the ocean in a heavily corrosive environment. Therefore, corrosion prevention measures should be taken into consideration to reduce required maintenance and ensure longevity of the gates. Examples of corrosion prevention measures include:

- Noncorrosive rebar, such as galvanized, epoxy coated, or FRP composite
- Noncorrosive sheet pile, such as prestressed concrete, vinyl, or FRP composite
- Corrosion inhibiting admixtures for concrete
- Stainless steel for railings and hardware

5.17 MONITORING & INSPECTION

The project will have annual inspections by the USACE. Further description is found in Section 6.2 Operation and Maintenance.

CHAPTER 6 NATURAL AND NATURE-BASED FEATURES

6.1 OVERVIEW

Natural and nature-based features (NNBFs) refer to the use of landscape features to provide for flood risk management benefits that involve reducing damages to people and property from flooding and erosion, including the processes that contribute to these (Bridges et al. 2021). There are many terms related to NNBFs, such as such as natural flood management, engineering with nature, green infrastructure, and soft defenses, and they share some common elements, but they are not synonymous with NNBFs. NNBFs in coastal environments are a type of natural or nature-based solution that use landscape features that include beaches, dunes, wetlands, reefs, or islands (Bridges et al. 2021). Not all natural or nature-based solutions can be universally applied, and the appropriateness of particular NNBFs is dependent on the source of flooding and the environmental conditions.

For this study, NNBFs in the form of oyster reef-based living shoreline sills would be constructed in association with the storm surge wall to reduce erosional impacts. Shoreline erosion is caused by winds and wave action, which are increased during coastal storms. Erosion can leave upland bluffs exposed that slump into adjacent tidal creeks, leading to loss of gradually sloping shorelines and eventually upland. When hardened structures are placed in estuaries, they reflect wave energy seaward. The reflection can create turbulence, capable of suspending sediments and vegetation, leading to increased erosion, or scouring, of the marsh around the base of the structure. Approximately 9,300 linear feet of oyster reef-based living shoreline sills would be constructed in areas vulnerable to coastal erosion (see Figure 4-4 in the FR/EIS) and future scouring from storms, including areas where the wall would be sited along the Charleston Peninsula shoreline of the Ashley River near Lockwood Blvd, Brittlebank Park, and the Wagener Terrace neighborhood (Figure 6.1). The reef-based living shoreline sills would be placed parallel to the vegetated shoreline for the purpose of reducing wave energy between the sill and the wall, leading to reduced shoreline erosion and marsh scour, while trapping sediments to stabilize and enhance marshes. Erosion and scouring are discussed in more detail in Sections 4.2 and 6.2 of the FR/EIS.

Figure 6.1. Areas of shoreline planned for reef-based living shoreline sills.

6.2 CONSTRUCTION REQUIREMENTS AND CONSIDERATIONS

The sills would meet the definition and project standards for living shorelines found in sections R.30-1D(31) and R.30-12.Q of state regulations S.C. Code Sections 48-39-10 et seq. In coastal South Carolina, living shoreline sills involve techniques that stabilize estuarine shorelines at the marsh-water interface, or more specifically between the low and high tide lines (see Figure 6.2). Techniques commonly practiced in South Carolina incorporate natural materials such as oyster shells or coir logs, or materials that promote oyster growth such as oyster castles or manufactured wire reefs (e.g., concrete-coated crab traps) (SCDNR 2019). The specific technique and materials for the living shoreline sills for the proposed plan will be determined in the PED phase, but would not include coir logs since they do not lead to the formation of oyster reefs.

Because oysters thrive in the intertidal zone in South Carolina, not subtidal as in many other coastal states, they are extremely suitable for providing vertical relief for reducing wave energy and trapping sediments to stabilize shorelines at the marsh-water interface. Oyster recruitment to suitable substrate is high in South Carolina waters from April to September. The typical height of oyster reef-based living shoreline sills is 1-2 feet, depending on the materials used and vertical growth of the living reef over time (SCDNR 2019). Because the sills would be placed above the low tide line and in close proximity to the vegetated shoreline, they are not expected to interfere with navigation, although signage would likely be required.

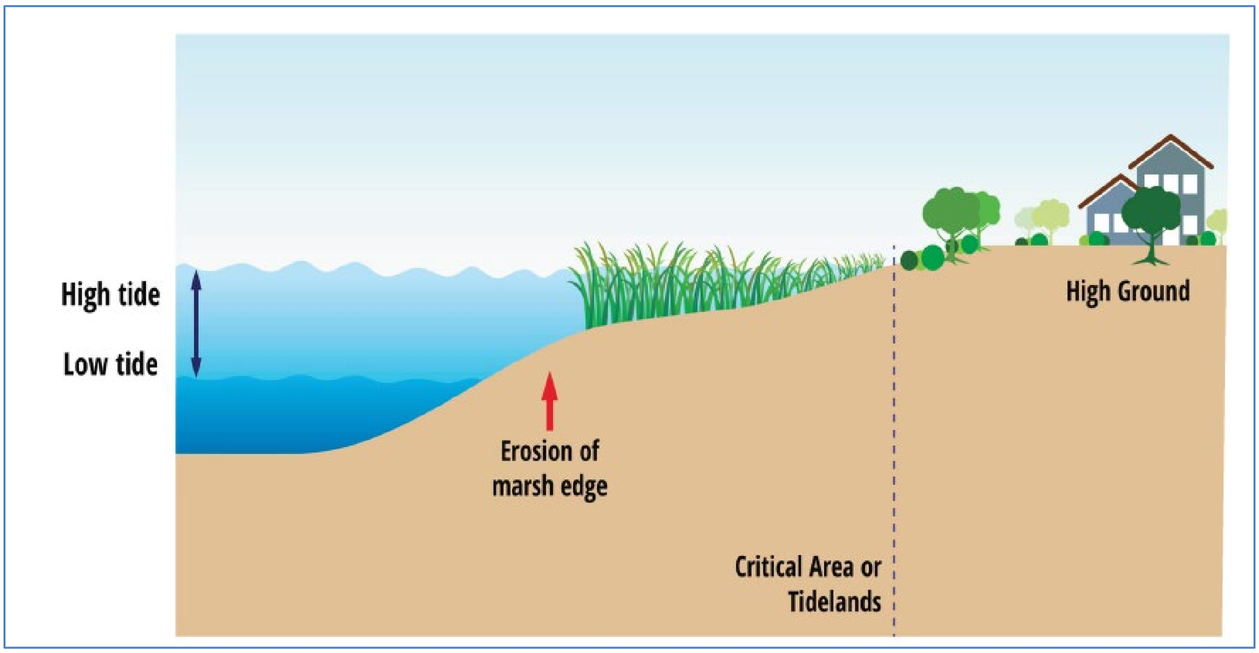


Figure 6.2. Profile of a typical South Carolina estuarine shoreline. The red arrow indicates the area of erosion concern where living shorelines sills would typically be placed in coastal South Carolina to reduce loss of the marsh edge. Source: SCDNR 2019

Construction of typical reef-based living shorelines in South Carolina is considered low-impact. Heavy equipment is not generally used. Construction would likely occur from the water-side with small, shallow boats to reach the intertidal zone to avoid damage to the marsh during construction. While unlikely, any lost marsh vegetation due to construction would be replaced. Construction is limited to times of low tide for proper placement. Some sediment disturbance is typical but has not required the use of devices or treatments to reduce water quality impacts. Sedimentation in the Ashley River and any turbidity plumes would be short-term and quickly dispersed. Some minor disturbance to micro and macro benthic fauna could occur. Fish, wading birds, and marine mammals would have limited access to the marsh edge temporarily during construction, yet the oyster reef-based living shorelines would provide considerable habitat benefits once the construction is complete. The potential for adverse effects on cultural resources would be minimal since the living shoreline materials are placed on the surface of the bank and no sediment excavation is involved. Any areas of known buried cultural resources along the shoreline would be avoided to the greatest extent practicable. Potential effects of the living shoreline sills on the environment, both adverse and beneficial, are discussed throughout Chapter 6 of the FR/EIS.

6.3 PERFORMANCE, MONITORING, AND ADAPTATION

For the proposed plan, the locations of the reef-based living shoreline sills along the Ashley River are approximate and site attributes would be evaluated for oyster suitability during the PED phase, including existing energy level from waves and currents, salinity, width and slope of the bank, sediment firmness, and sediment composition. These site attributes will also help to determine what substrate materials will be used for the reef-based living shoreline sills. However, the certainty for site suitability is high considering SCDNR has already constructed small scale living shoreline sills in some of the proposed locations, including along the shoreline of the Ashley River by Lockwood Blvd and Broad Street (Figure 6.3), and by Brittlebank Park (Figure 6.4). The living shoreline sills in the proposed plan would expand the spatial coverage of the existing reefs, protecting and stabilizing continuous areas of estuarine shoreline from wave attack and wave run-up, marsh scouring, and minimizing fragmentation of reef habitat.

Figure 6.3. Recently completed segment of living shoreline constructed by SCDNR from manufactured wire reef sills along the Ashley River near the corner of Lockwood Blvd. and Broad Street. Note the location from the low tide waterline and proximity to the vegetated shoreline. Source: photo courtesy of SCDNR.

Figure 6.4. Living shoreline sills constructed with different manufactured wire reef configurations along the Ashley River by Brittlebank Park. There are also some segments of bagged oyster shell reef sills in the vicinity (not shown in photo). Source: photo courtesy of SCDNR.

Typical performance metrics for living shoreline techniques used in South Carolina are changes in sediment type, vertical accretion of sediment behind the sill, lateral movement of the marsh toward the sill, and coverage of the reef substrate with oysters (SCDNR 2019). Natural adaptation of the living shorelines to sea level rise is expected over time with respect to sediment capture, vertical growth of the oyster reef structure, and marsh elevation to keep pace with the intertidal zone as it shifts, and natural succession of plants and animals which make for a healthy ecosystem. However, consideration would be given during the PED phase to design elements that may enhance resiliency such as planting of marsh grass behind the sill which can increase sediment capture and stimulate accretion, among other benefits. Consideration for intentional adaptation and maintenance will also be given during the PED phase and included in the Mitigation Plan, such as the need for additional substrate to enhance oyster settlement if needed. While living shorelines tend to be more resilient to storm damage over time than hardened erosion control structures such as bulkheads, any damaged sections of the living shoreline sills would be repaired.

CHAPTER 7 PRECONSTRUCTION ENGINEERING AND DESIGN (PED) CONSIDERATIONS

7.1 FUTURE WORK REQUIRED IN THE PRE-CONSTRUCTION ENGINEERING and DESIGN PHASE

Due to the study area size, schedule and funding constraints, there is much geotechnical analysis and design required during the PED phases. Some of this work, such as subsurface exploration, will need to start immediately at the beginning of PED in order to obtain the necessary information to complete geotechnical and structural analyses. The work required during PED is discussed in detail below.

7.1.1 SUBSURFACE EXPLORATION

Subsurface information will need to be gathered along the wall alignment and the breakwater alignment, if retained as part of the Recommended Plan. Along with determining stratigraphy along the wall alignment, it will be important to know if there is any man-made fill or construction debris that may affect construction and pile installation. When developing the soil exploration program, the PDT should determine areas where the presence of man-made fills is likely so additional exploration can be completed to define the type and extents of it. Soil exploration should be extended into the Cooper Marl, to a depth of at least 20 feet below the expected pile tip elevation (U.S. Department of Transportation Federal Highway Administration, Design and Construction of Driven Pile Foundations – Volume I, page 87). For the breakwater alignment, the soil exploration should be developed to provide information on the bearing capacity of the foundation. Soil exploration should consist of cone penetration test (CPT) soundings supplemented with standard penetration test (SPT) borings. The SPT borings will be used verify the soil behavior type determined during CPT data reduction. Additionally, undisturbed samples should be collected and tested. The testing should consist of both drained and undrained shear strength determination, consolidation, and soil classification tests (Atterberg limits and grain size distribution). The spacing between soil explorations will likely ranges from 250 to 1,000 feet.

If soil-structure interaction modeling will be required, in situ modulus values will need to be determined. Flat plate dilatometer or pressuremeter testing would be required. Additionally, the flat plate dilatometer could also be used to supplement the determination of shear strengths.

7.1.2 SEEPAGE ANALYSIS for T-WALL and COMBO WALL SECTIONS

Seepage analysis will need to be completed to determine the proper depth of seepage cutoff walls and the uplift pressures on the T-wall footing.

7.1.3 PILE DESIGN

The design of the piles will be required. The design will include selection of pile type (steel H-pile, concrete piles, micro piles, etc.) considering costs, drivability, vibration generation, constructability, and longevity (related to corrosion). Determination of both axial and lateral load capacity will be required along with downdrag calculations, where applicable. Pile load tests (dynamic, static, and lateral) should be evaluated to determine the appropriateness of completing that at various stages of design and construction.

In addition to the typical pile design, pile driving generated vibrations will need to be evaluate. Both magnitude and distance travel will need to be determined. Maximum allowable vibration amplitudes along with construction monitoring requirements will be needed.

7.1.4 LATERAL EARTH PRESSURE

It is anticipated in some locations the wall will also act as a retaining wall. Appropriate lateral earth pressures will need to be determined to be used in the design of the retaining wall.

7.1.5 I-WALL EVALUATION

There could be a cost savings potential if I-walls can replace T-walls and this should be evaluated along the project alignment where the exposed stem height is 4 feet or less. The PDT will need to realize that the design requirements for an I-wall are more intensive that T-walls and need to be considered this when developing the soil exploration program (smaller spacing) and design schedule.

7.1.6 VERIFY UTILITY LOCATIONS

Penetrations through the barrier will be necessary for utilities and stormwater drainage. These penetrations will need to be designed. The city provided their known utility layers but there is a significant amount of information missing such telephone, fiber optics, and property owner connections to city systems that will need to be identified and considered in the final design.

The PDT should consider determining utility corridors in which multiple utilities can penetrate the barrier in one designated segment. This would minimize the number of crosses.

7.1.7 DETAILED SURVEYS

There is insufficient detail in the topographic data to accurately place the wall and know impacts to things such as curbs along roadways. Detailed surveys of land features, utilities, trees, etc. will be done to finalize placement of wall.

7.1.8 FINAL INTERIOR HYDROLOGY ANALYSIS

For this Feasibility study the interior hydrology is based on the overland flow only. The subsurface drainage system is not considered. In PED phase the interior hydrology should be more accurate in determining impacts to insure the pumps are adequately sized and strategically placed. Detailed assessment of the timing of an overtopping scenario versus the opening and draining via gates in the wall.

7.1.9 GEOSPATIAL BATHYMETRIC AND TOPOGRAPHIC DATA

Coastal modeling was based on the FEMA model done in the second decade of the 21st century. Changes in bathymetry as well as topography should be evaluated to determine if there are changes to the hydrodynamic model and impacts of the proposed project.

7.1.10 RECOMMENDATIONS OF THE RISK ASSESSMENT

Consider all recommendations of the Charleston Planning Risk Assessment performed by the Life Safety Center.

- Consider debris and impact loading in project design (PFM 11, 15).

- Verify there is no damage to utility lines during installation of the project, possible resistivity or other non-destructive testing or camera inspection (PFM 18, 24).

- Minimize number of utility crossings and consider having adaptable utility corridors (PFM 18).

- Have secure buildings for storage of parts for gate structures (PFM 22).

- Consider using the same parts at all gate structures to the maximum extent possible and have spare parts available for critical components. Also consider having adequate information to fabricate critical parts without relying on proprietary systems (PFM 22).

- Verify anti-corrosion coating is placed properly, include O&M requirements to recoat as necessary over the life of the project and have a plan in place for higher risk zones. Consider long-term monitoring of coating for any critical elements (PFM 26).

- Perform pile load testing to select piles appropriately and to evaluate the installation methodology to minimize potential for improper installation of structure elements or overstressing or damaging during installation (PFM 28).

- Consider O&M requirements to have periodic and post-storm surveys of the marsh to evaluate any erosion and mitigate erosion areas as necessary; particularly in the area of the Coast Guard station which has higher potential for boat traffic (PFM 30).

• Discussion between environmental and geotechnical team members to understand the potential for limited geotechnical data along the alignment of the combo wall, the potential impacts to the project, and develop strategy to mitigate if possible (PFM 37).

• Consider bird deterrents for storm gates (PFM 40).

• Consider redundancy in ability to close gate structures using a manual or forced closure if mechanical equipment fails (PFM 12, 39).

• Perform a time and motion study to understand assumptions and limitations regarding closure time, manpower required, equipment requirements, etc. and integrate findings in the City's traffic plans for evacuation (PFM 12).

• O&M manual to include requirement to document the time required and manpower needed when routine closure testing is done to inform the closure plan. Re-evaluate the closure sequencing based on the results of this testing (PFM 12).

• In areas without landside erosion protection (concrete, asphalt, etc.), perform calculation on erosion potential of landside fill and include overtopping resiliency in the design (PFM 7, 8).

• Perform an analysis on duration needed to drain the City in the event the leveed area is inundated during an event and the gates are unable to be opened. If the study shows the need, the gate design considerations should include the ability to open the gates while under reverse loading. If interior flooding will be of a longer duration, develop a plan to evacuate the leveed area (PFM 42).

• If during design the depth of piles are limited/reduced, the overtopping failure modes need to be reassessed (PFM 7, 8, 20).

• It is anticipated during PED that unique foundation elements, such as micropiles, may be considered in areas where there is limited right-of-way or nearby structures that may be impacted by traditional pile installation. USACE presently does not have design criteria for non-traditional deep foundation elements. It is recommended that design criteria be developed during PED in coordination with USACE HQ structural and geotechnical Co-Op leads. Rock Island District (MVR) also developed a site-specific design criterion for micropiles for a project that could be considered as a starting point for development of design criteria for unique foundation elements specific to the implementation of the Charleston project.

7.1.11 TRANSPORATION STUDY

A Transportation Study will be done to assess the modifications to road widths and accessibility within the city to minimize real estate costs, reduce gates by rerouting access to side streets and potential impacts to structures. These might include making roads one way to allow more distance to structures, reducing number of lanes and relocating entrances.

7.2 OPERATION and MAINTENANCE MANUAL

A draft Operations and Maintenance (O&M) Manual has been started. This preliminary O&M manual will need to be further refined during PED phase as more information is gathered and the actual gates and pump stations are designed.

The local sponsor should be prepared to carry out maintenance activities on all flood risk management structures every year. Regular maintenance is critical because many types of problems will escalate exponentially when left unchecked. There are many ongoing requirements of which one should be aware. For example, debris and unwanted growth need to be removed from the areas adjacent to floodwalls. Local sponsor will need to periodicallyinstall closure structures as required by the inspection & levee safety program. Grass adjacent to floodwalls has to be cut low and maintained and no trees shall be planted on or within 15 feet of a structure.

Metal gates and other components need to be painted and greased periodically. Concrete damage needs to be identified and repaired early or it will get worse. Standard maintenance for cathodic protection systems will be needed as well. Beyond these examples of ongoing maintenance, there are also more significant repairs that will be necessary from time to time. On occasion, the local sponsor may have to add stone to control an erosion problem. Pump stations also need to be completely overhauled periodically. Routine maintenance is expected in any project and can be planned for in advanced. This is discussed in the sections under gates and pumps, as well as the draft O&M Plan. To assist with monitoring, certaintools and instruments are needed and measurements are required. Monitoring points and other instruments are needed to measure movement of the structures and periodic surveys are required to monitor for possible settlement.

SUB-APPENDICES

1. STRUCTURAL ENGINEERING SUB-APPENDIX
2. GEOLOGIC AND GEOTECHNICAL ENGINEERING SUB-APPENDIX
3. HYDRAULICS AND HYDROLOGY (HEC-RAS 2D Modeling) SUB-APPENDIX
4. COASTAL MODELING SUB-APPENDIX

4A. ERDC COASTAL MODELING SUB-APPENDIX

5. COST ENGINEERING SUB-APPENDIX

US Army Corps of Engineers®

Charleston District

CHARLESTON PENINSULA, SOUTH CAROLINA,
A COASTAL STORM RISK MANAGEMENT STUDY

Charleston, South Carolina

ENGINEERING APPENDIX - B STRUCTURAL SUB-APPENDIX

February 2022

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TABLE OF CONTENTS

INTRODUCTION

The structural engineering scope of this study is to identify various types of flood barriers and determine their feasibility to protect the Charleston Peninsula from flooding from a hurricane storm surge.

The route of the flood barrier and top elevation was determined in coordination the City of Charleston and others. The route of the flood barrier follows the perimeter of the city along the waterfront of the Ashley and Cooper Rivers. Right of way for construction is very limited throughout the route. There are environmental concerns with locating the barrier toward the water. The route must accommodate pedestrian and vehicular traffic, stormwater, and utilities.

The top elevation of the flood barrier was determined to be set at El 12.0 NAVD 88, which matches the highest elevation of the railing along the newly constructed Low Battery Seawall. In addition, the new flood barrier will be designed to accommodate sea level rising an additional 3 feet in the future.

Various barrier types were considered including Earth Berms (Levees), I-Walls, T-Walls, Combo Walls, Swing Gates and Stop Logs at pedestrian and vehicular crossings. Each barrier type has its own requirements, limitations and footprint requirements, which this report discusses in more detail.

USACE Norfolk District (NAO) completed a similar feasibility study for the City of Norfolk in Virginia. The soil conditions in Norfolk are similar to Charleston's soils, as the soils consist of roughly 50-65 ft of soft soils, and a harder layer below that is suitable for providing reliable structural support. The NAO feasibility study was referred to and guided the early evaluation of wall types.

For this feasibility study, the structural design effort was limited to only determining the basic requirements and to provide sufficient structural information to determine feasibility costs. The structural design will be further developed during Preconstruction Engineering and Design (PED) Phase.

REFERENCES

Structural design in the Corps of Engineers is governed by Engineering Regulations (ER's), Engineering Manuals (EM's), Engineering Technical Letters (ETL's), and Engineering Circulars (EC's). These documents are available online at usace.army.mil under the Library and Publications tabs. The following criteria documents are pertinent to the Charleston Peninsula flood barrier:

EM	1110-2-1901	Seepage Analysis and Control for Dams 9/30/1986
EM	1110-2-2000	Standard Practice for Concrete for Civil Works Structures 3/31/2001
EM	1110-2-2100	Stability Analysis of Concrete Structures 12/1/2005
EM	1110-2-2102	Waterstops and Other Preformed Joint Materials for Civil Works Structures 9/30/1995
EM	1110-2-2104	Strength Design for Reinforced Concrete Hydraulic Structures 11/30/2106
EM	1110-2-2502	Retaining and Flood Walls 9/29/1889
EM	1110-2-2503	Design of Sheet Pile Cellular Structures, Cofferdams, & Retaining Structures 6/11/1990
EM	1110-2-2504	Design of Sheet Pile Walls 3/31/1994
EM	1110-2-2906	Design of Pile Foundations 1/15/1991
ER	1110-2-1806	Earthquake Design and Evaluation for Civil Works Projects 5/31/2106
ETL	1110-2-584	Design of Hydraulic Steel Structures 6/30/2004
UFC	3-301-01	Structural Engineering 10/1/2019
IBC 2018		International Building Code
ASCE 7-16		Minimum Design Loads for Buildings and Other Structures

Note: Current editions of all criteria documents will be used during PED phase.
In the event of discrepancy between UFC 3-301-01 and IBC 2018 or ASCE 7-16 load criteria, the higher load will govern Structural engineering during PED phase.

Coastal Storm Risk Management Feasibility Study / Environmental Impact Statement
USACE Norfolk District, July 2018

LOAD CASES

The load cases considered for this study were in accordance with Coastal Flood Wall requirements in EM 1110-2-2502. A preliminary structural stability analysis was performed using Microsoft Excel to determine forces on the structures and reactions on the pile foundation. Detailed structural design of the walls will be accomplished during PED phase.

C1: Surge Still Water Loading
C2a: Nonbreaking Wave Loading
C2b: Breaking Wave Loading
C2c: Broken Wave Loading
C3: Earthquake Loading
C4: Construction Short-Duration Loading
C5: Wind Loading

ASSUMPTIONS AND LIMITATIONS

FOUNDATIONS

Due to the poor nature of the soils in Charleston, all wall types are planned to be founded on deep piles that will be embedded into the Cooper Marl stratum which is located at elevations ranging from EL -55 NAVD 88 to El -75 NAVD 88. Cooper Marl consists of medium dense silty sand to firm silty clay and provides sufficient bearing capacity to support all structures.

EARTH BERM (LEVEE)

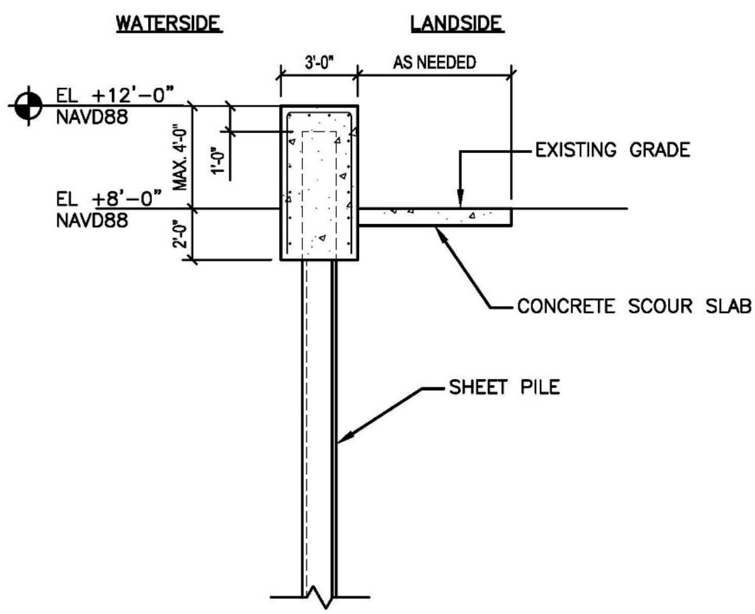
Earth Berms (Levees) were ruled out at a viable option due their large footprint requirement (i.e., 10 ft wide top, 3 to 1 slope, vegetative free zone on each side, etc.). The study is limited to the peninsula of Charleston, where the land has been heavily developed and available land is very scarce. Earth Berm construction would require taking many developed parcels of privately owned land and / or filling many acres of wetland. Refer to the table below for Total Width requirements for Earth Berms.

Berm Height (ft) Above Existing Grade	10 ft Top Width		8 ft Top Width	
	3H : 1V	4H : 1V	3H : 1V	4H : 1V
	Total Width (ft)	Total Width (ft)	Total Width (ft)	Total Width (ft)
1	46	48	44	46
2	52	56	50	54
3	58	64	56	62
4	64	72	62	70
5	70	80	68	78
6	76	88	74	86
7	82	96	80	94
8	88	104	86	102
9	94	112	92	110
10	100	120	98	118
11	106	128	104	126
12	112	136	110	134
13	118	144	116	142
14	124	152	122	150

* Total Widths include a Vegetation Free Zone (VFZ) of 15 ft on each side of the berm

I - WALL

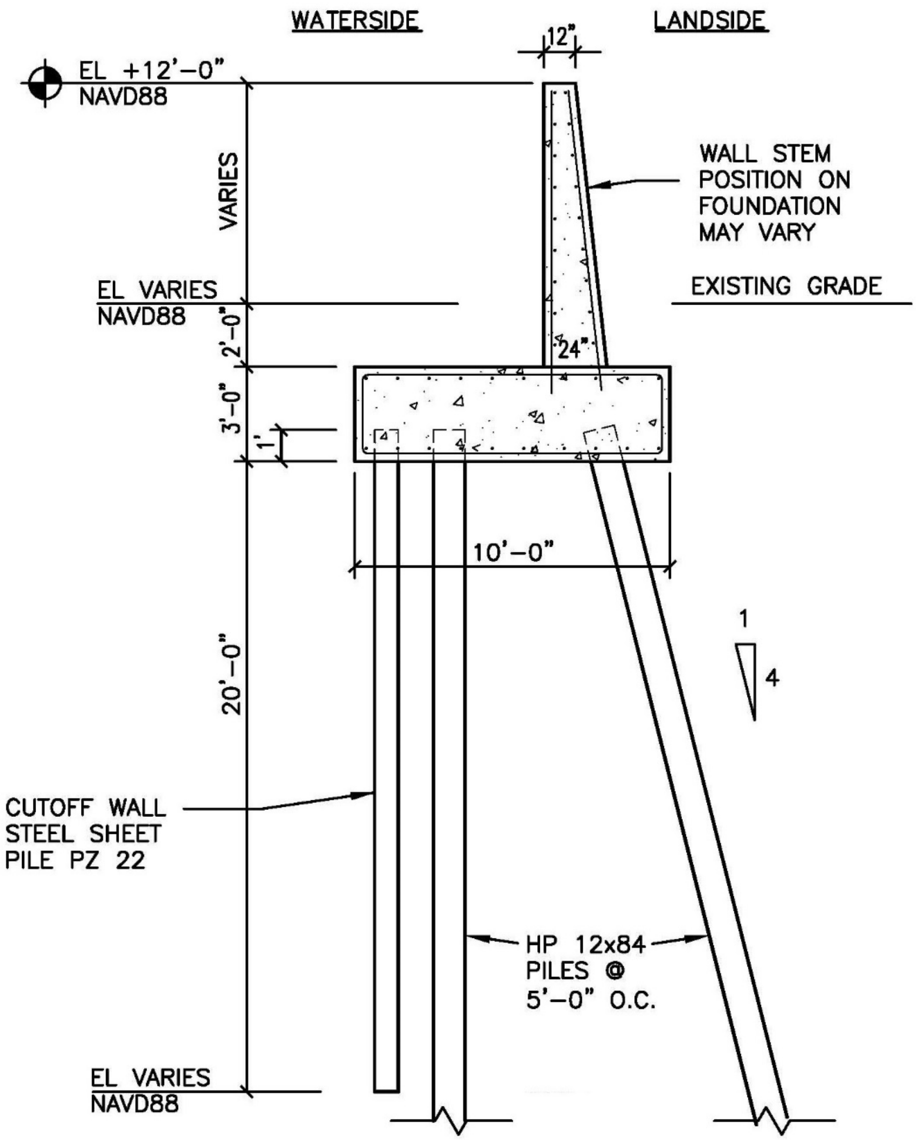
I - Walls were ruled out as a viable option for flood wall construction. This type of barrier would consist of driven sheet pile walls with a concrete cap. I - Walls occupy a small footprint and would be desirable in areas where space is limited. I – Walls did not perform well in New Orleans during Hurricane Katrina and Corps design criteria was subsequently updated to limit the height of new I - Walls to a maximum of 4 feet above the current finish grade. The requirement for the new flood barrier to accommodate 3 feet of future raising rules out the I wall as a viable alternative.



I – Wall Typical Section

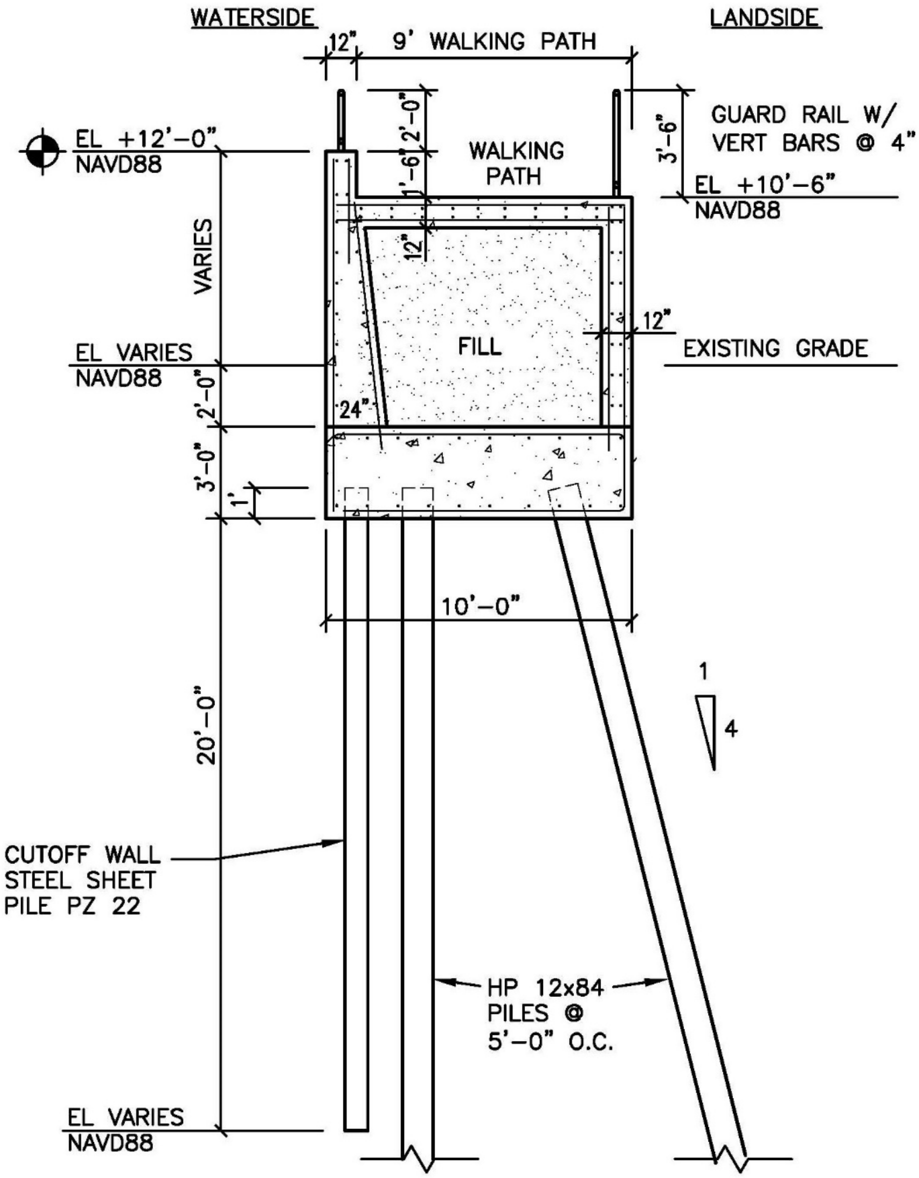
T - WALL

T - Walls are planned for reaches where the barrier can be constructed on land. T walls consist of a reinforced concrete stem, a reinforced concrete foundation, sheet pile cutoff wall, and vertical and batter piles. Steel sheet pile and H pile is shown in this sketch. Steel that is embedded in soil will not corrode. The steel sheets and the H pile will not displace as much soil during driving and will result in less vibration to mitigate risk of damage to nearby historic buildings.



T – Wall Typical Section

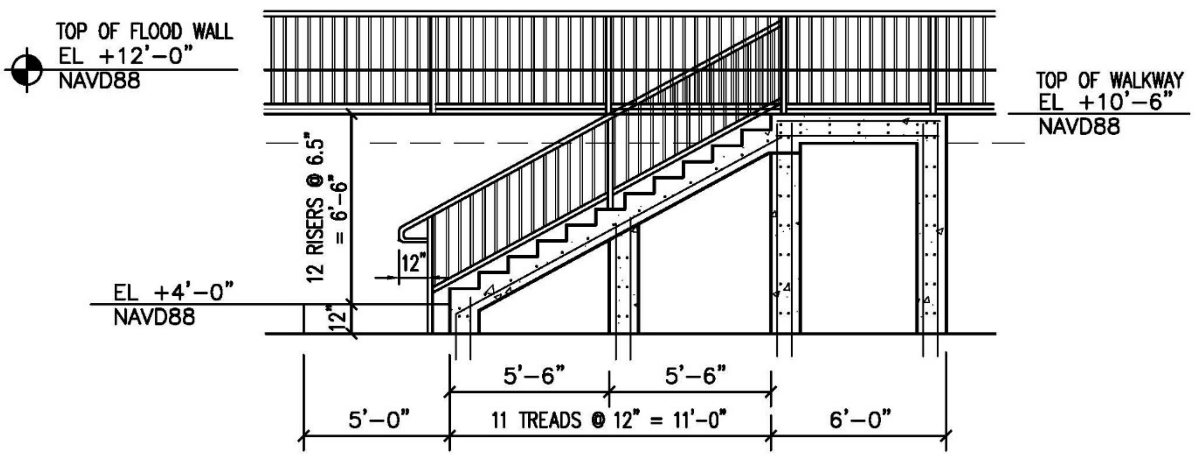
T – Walls with walkways are planned to be constructed in scenic areas such as along Lockwood Blvd and Brittlebank Park, and to replace the existing High Battery wall. The T – Wall with walkway section is similar to the T wall except that the stem is moved to the waterside and the walking path is constructed over the remainder of the foundation. This view shows the area under the walking path constructed over fill, but this area could be left unfilled to be used for storage.



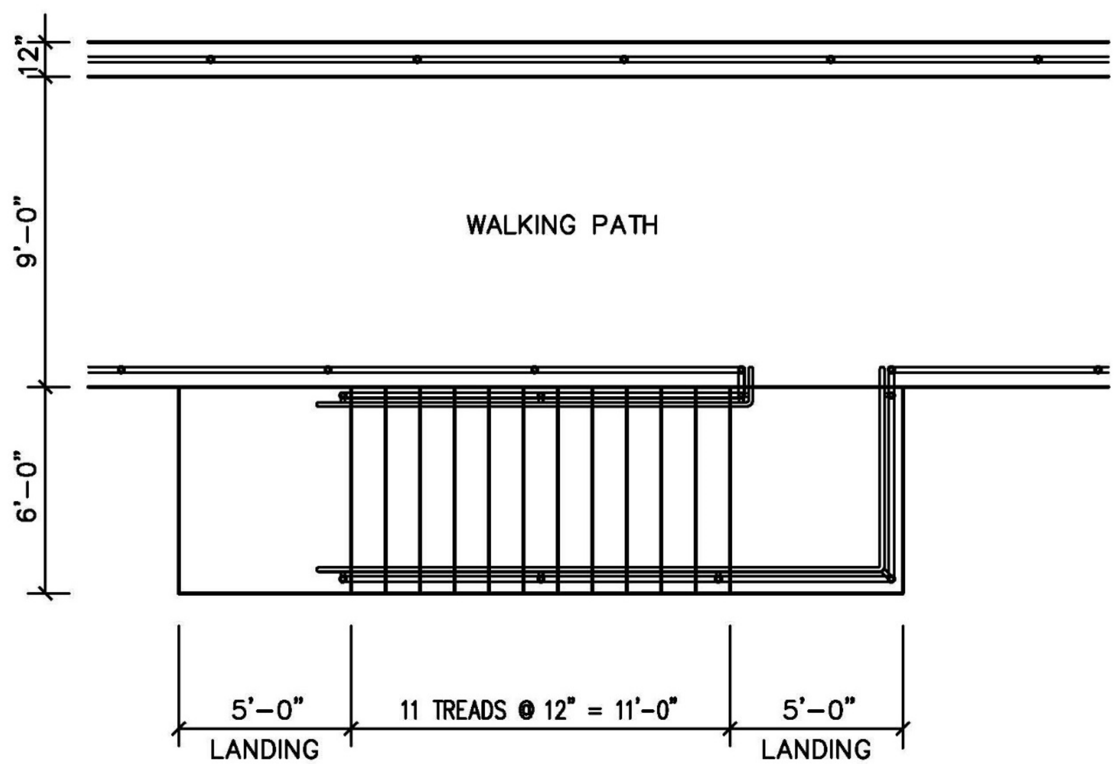
T - Wall with Walkway Typical Section

STAIRS and RAMPS

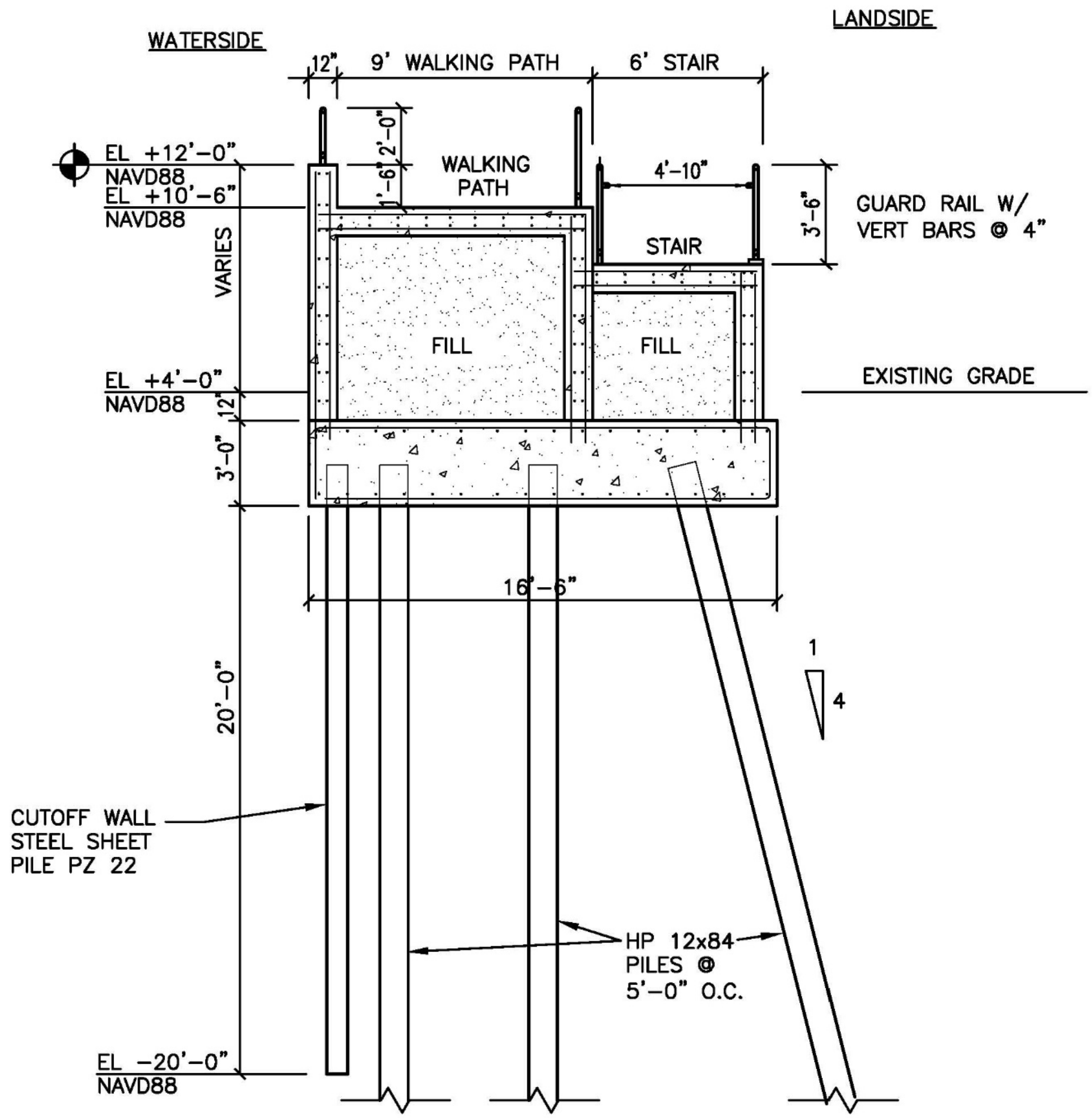
Stairs and Ramps will be required for pedestrian access to the T – Wall with walkway. Stairs and parallel Ramps would require a wider foundation than the typical T – Wall with walkway. "In Tandem" ramps would be identical to the typical T – Wall with walkway, sloping down to grade and would thus avoid the need for a wider foundation. Ramps would slope down at a rate of 1' vertical to 12' horizontal to meet all requirements for persons with disabilities including railing extensions, grab rails, and landings.



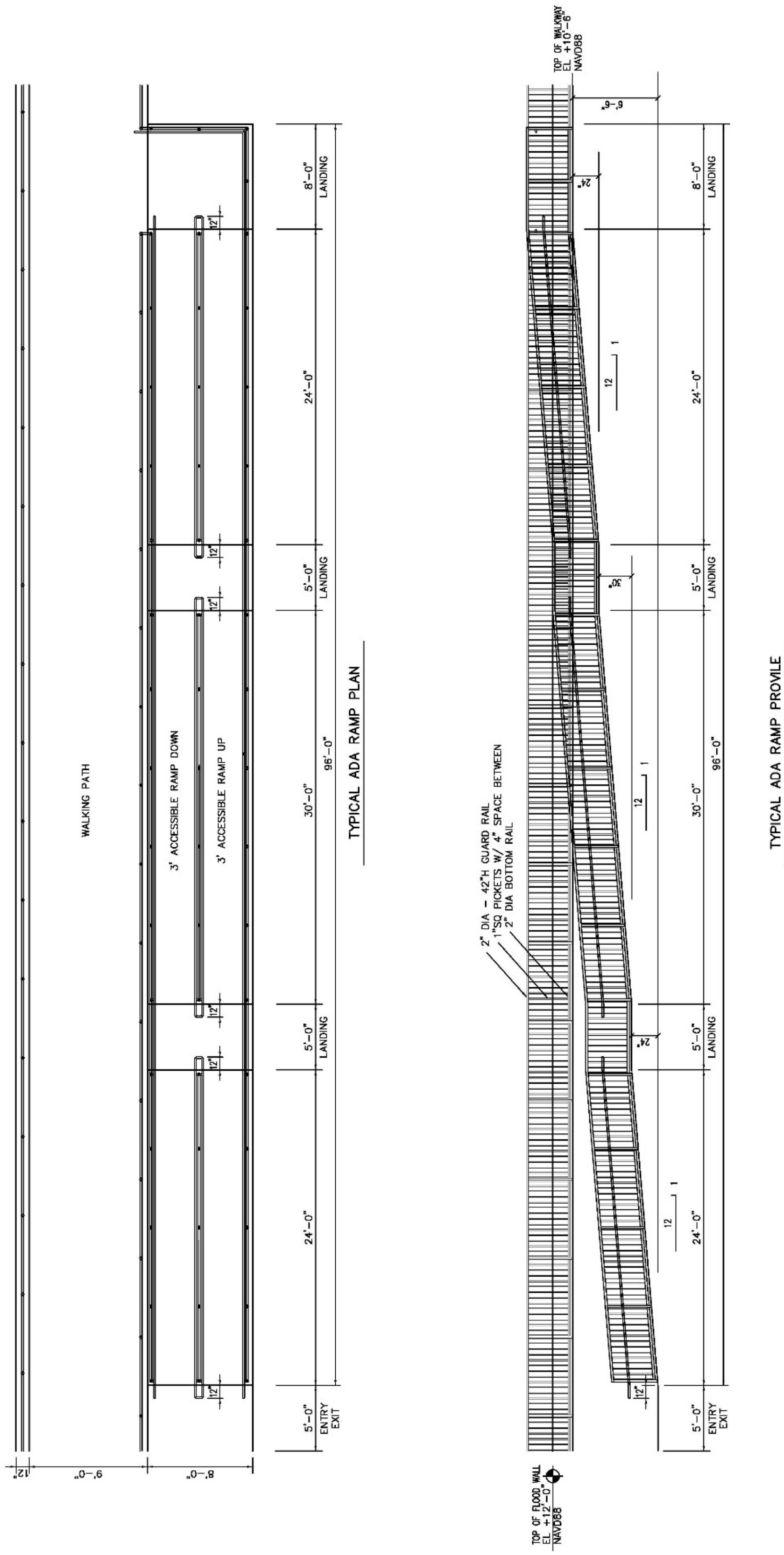
Stair Elevation



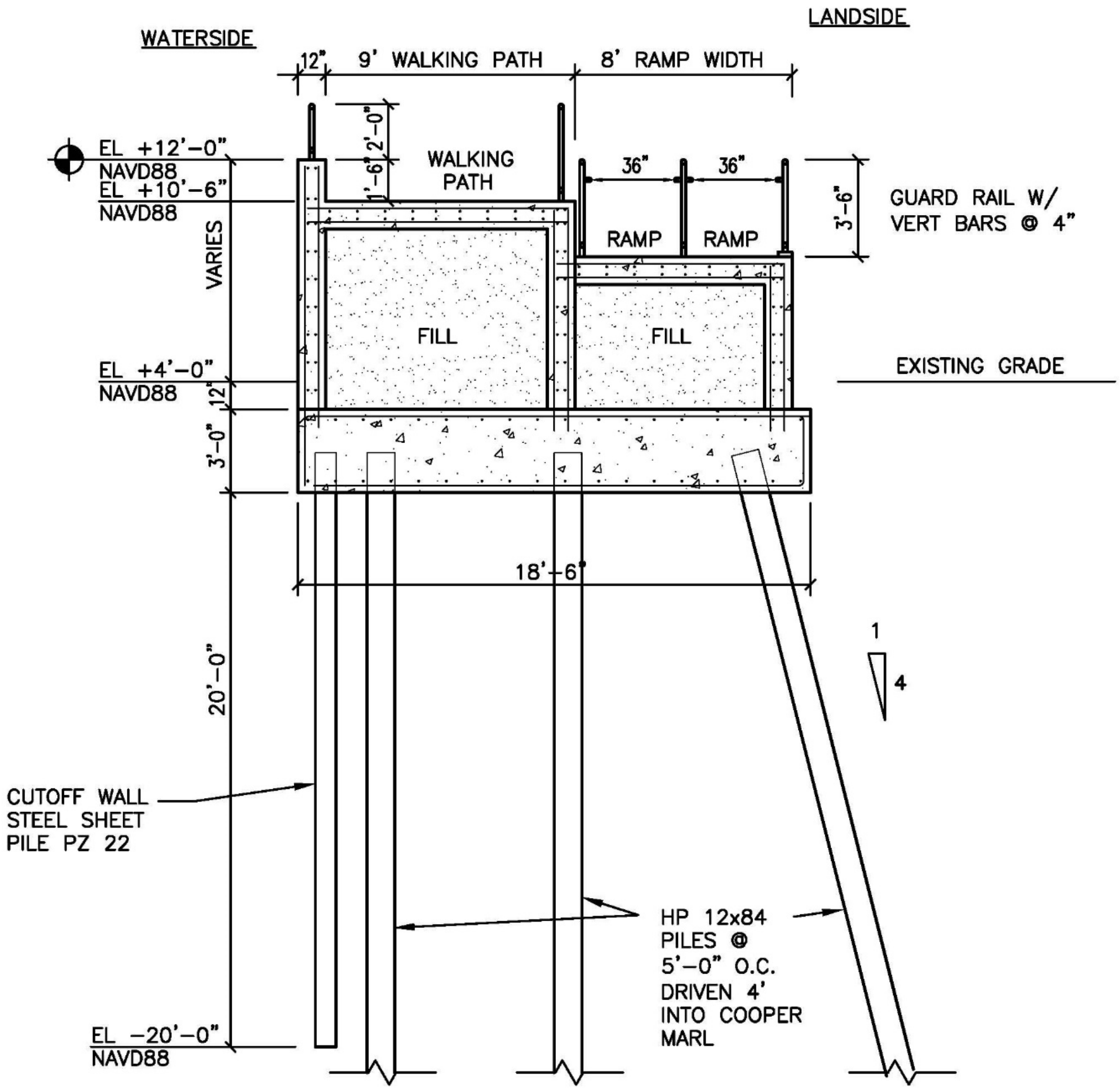
Stair Plan



T Wall with Walkway and Stair Typical Section



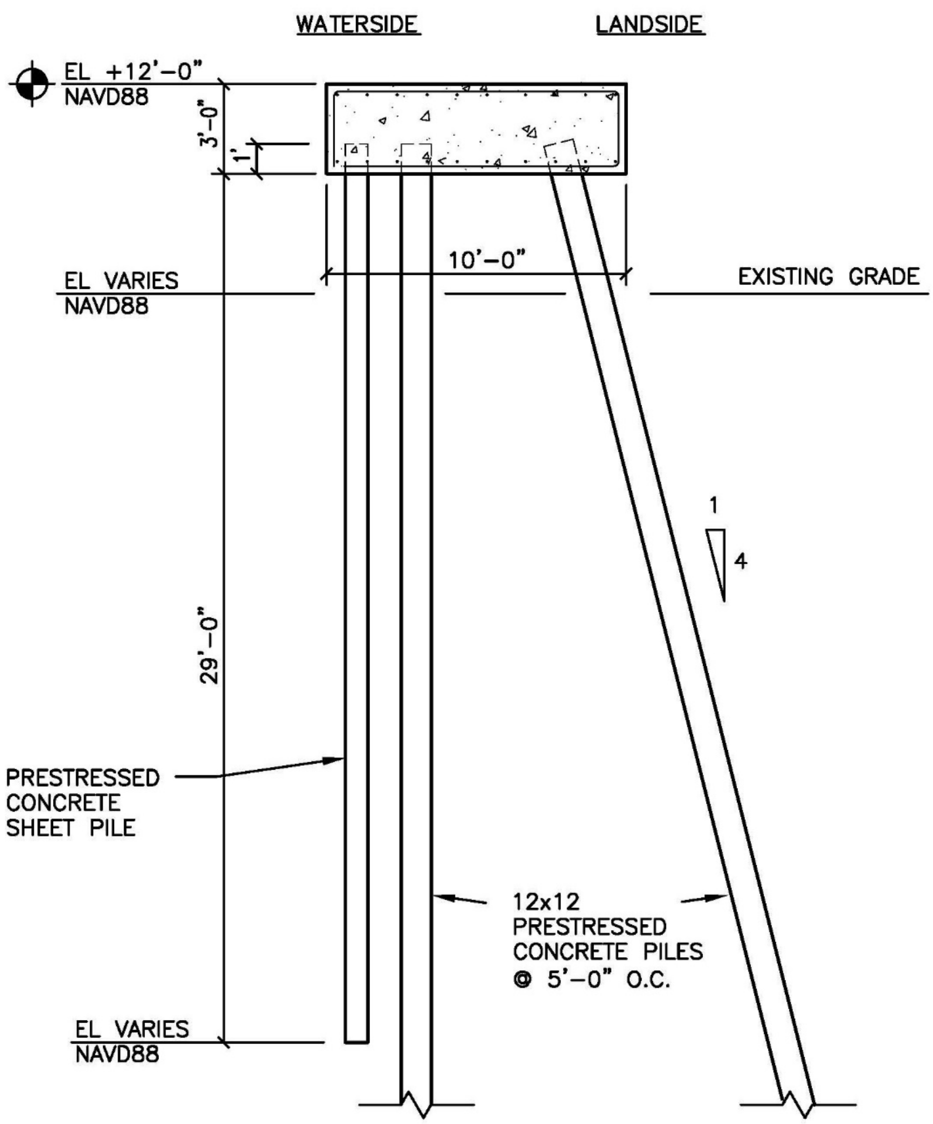
Parallel Ramp for Disabled Access



Parallel Ramp for Disabled Access Typical Section

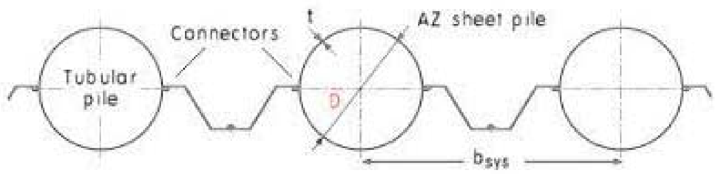
COMBO WALL

Combo Walls are planned for reaches where the flood barrier will be constructed across water or wetland. Construction of this wall type presents a number of unique challenges such as: Wetland Impact, Construction access, and Exposure of materials to saltwater environment. A temporary work trestle was determined to be necessary to construct the combo wall, which will allow sufficient width to operate a crane and receive materials. A dredged access channel was considered but rejected due to the adverse environmental impact. Prestressed concrete was selected over steel piles for the combo wall to avoid the need for cathodic protection. The foundation could be precast in 10' x 10' sections and grouted into position to avoid the need for formwork. Precast units would include grouted keyways and post tensioning conduits to assure continuity and watertightness.

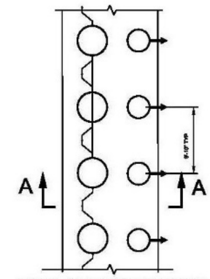


Typical section of Combo Wall

The original Combo Wall plan was to be a combination of large-diameter steel pipe piles with sheet piles between to form a flood barrier. A concrete cap at the top would transfer lateral force from the vertical piles to batter piles. The problem with using this concept is that all steel substructure members are in the wet and dry zone with saltwater exposure and very susceptible to corrosion and would require installing and maintaining a cathodic protection system over the life of the project. The Combo Wall plan was later modified to use precast concrete pile and sheet pile to avoid the need for cathodic protection.

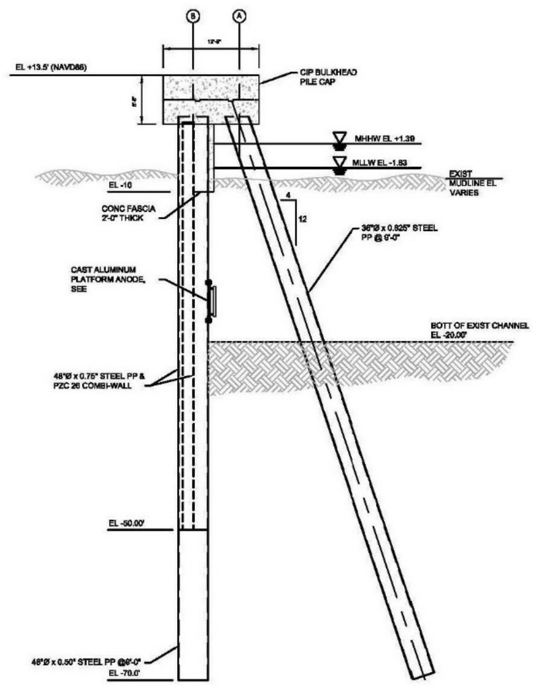


Plan of Tubular Pile with Sheet Pile Between



TYPICAL PILE AND PILE CAP LAYOUT

View of Constructed Combo Wall Substructure



SECTION A-A - COMBO WALL

WORK TRESTLE

Constructing the Combo Wall across wetland presents challenges for construction access. The Combo Wall structure is only 10 feet wide, which would be too narrow to operate a crane on, so a temporary work trestle was considered as an option. The City of Charleston constructed something similar between the US 17 bridges over the Ashley River as shown below.

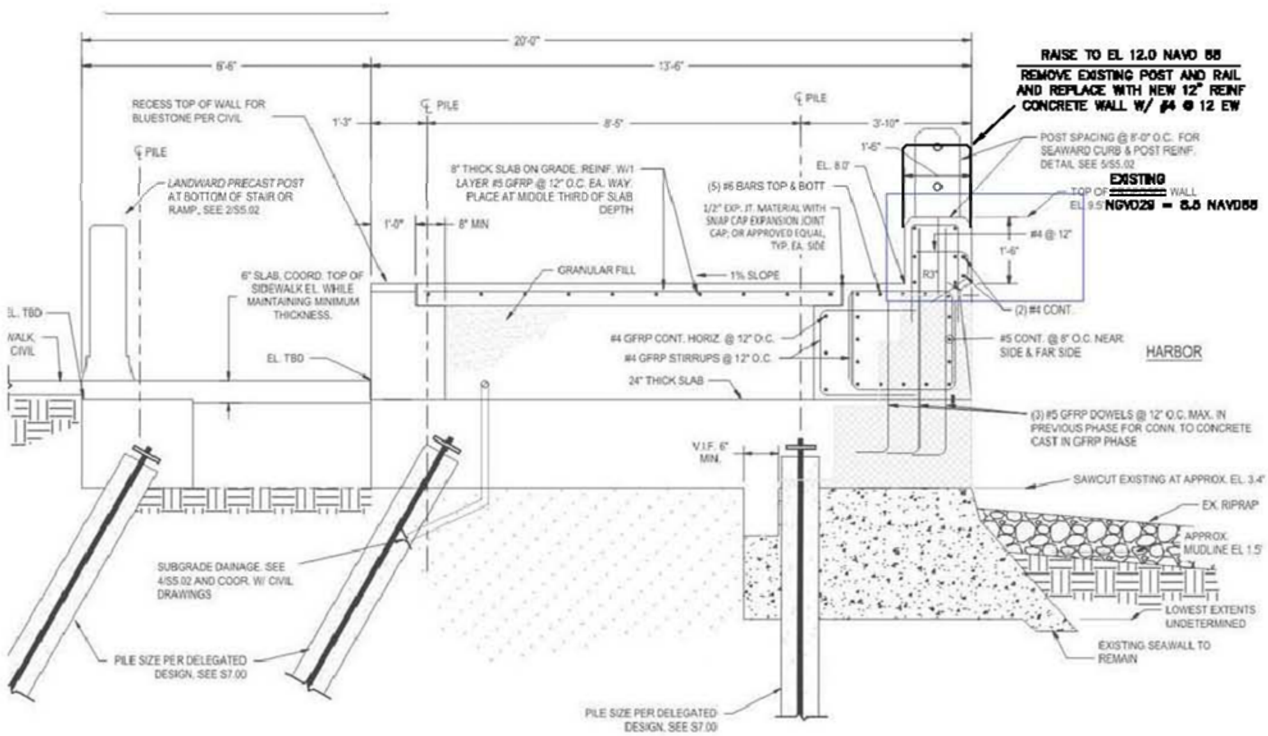
WORK TRESTLE

AERIAL VIEW OF WORK TRESTLE

VIEW FROM DECK OF WORK TRESTLE

LOW BATTERY WALL

The Low Battery Wall was recently renovated by the City of Charleston and provides a level of protection to EL 8.5 ft NAVD 88. The designer of record stated that the new Low Battery Wall was designed to support future raising to EL 12 ft NAVD 88. The wall can be retrofitted to provide a level of protection to EL 12 ft NAVD 88 by removing and replacing the existing post and railing and replacing with a solid wall. No other structural upgrades will be required to the Low Battery Wall to provide protection to EL 12 ft NAVD 88. Raising the Low Battery Wall in the future by an additional 3 feet would require additional structural analysis and structural upgrades. These upgrades may consist of, but are not limited to, foundation upgrades and additional lateral support. These upgrades will be very difficult to construct, and may result in major demolition and reconstruction of the Low Battery Wall.



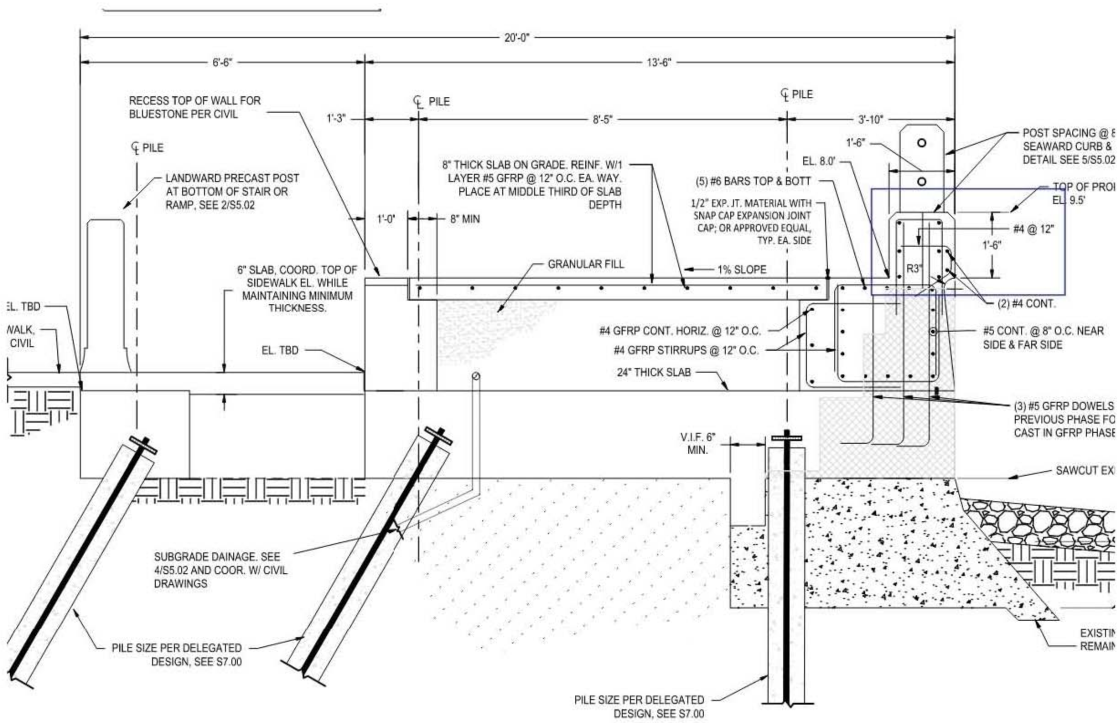
Typical section of Low Battery Wall Upgrade to EL 12.0 NAVD 88 flood protection

The low battery wall design was analyzed independently with top of wall at EL 12.0 NAVD88 using the same method and load combinations that were used for the T wall and Combo wall designs and the results confirmed the assurance given by the designer of record.

Lambert, Richard D CIV USARMY CESAC (USA)

From: O'Connor, Jim <JOconnor@jmt.com>
Sent: Monday, November 29, 2021 4:58 PM
To: Lambert, Richard D CIV USARMY CESAC (USA)
Cc: Mattie, Ryan; Newham, Frank; Kirk, Steve
Subject: [Non-DoD Source] RE: Low Battery Wall

Here is the detail and note from our plans identifying the future wall height.
The "1'-6"" we feel you refer to, as highlighted by the blue box below, can be replaced by a higher wall or added onto with the proper structural analysis and detailing.



WALL DESIGN

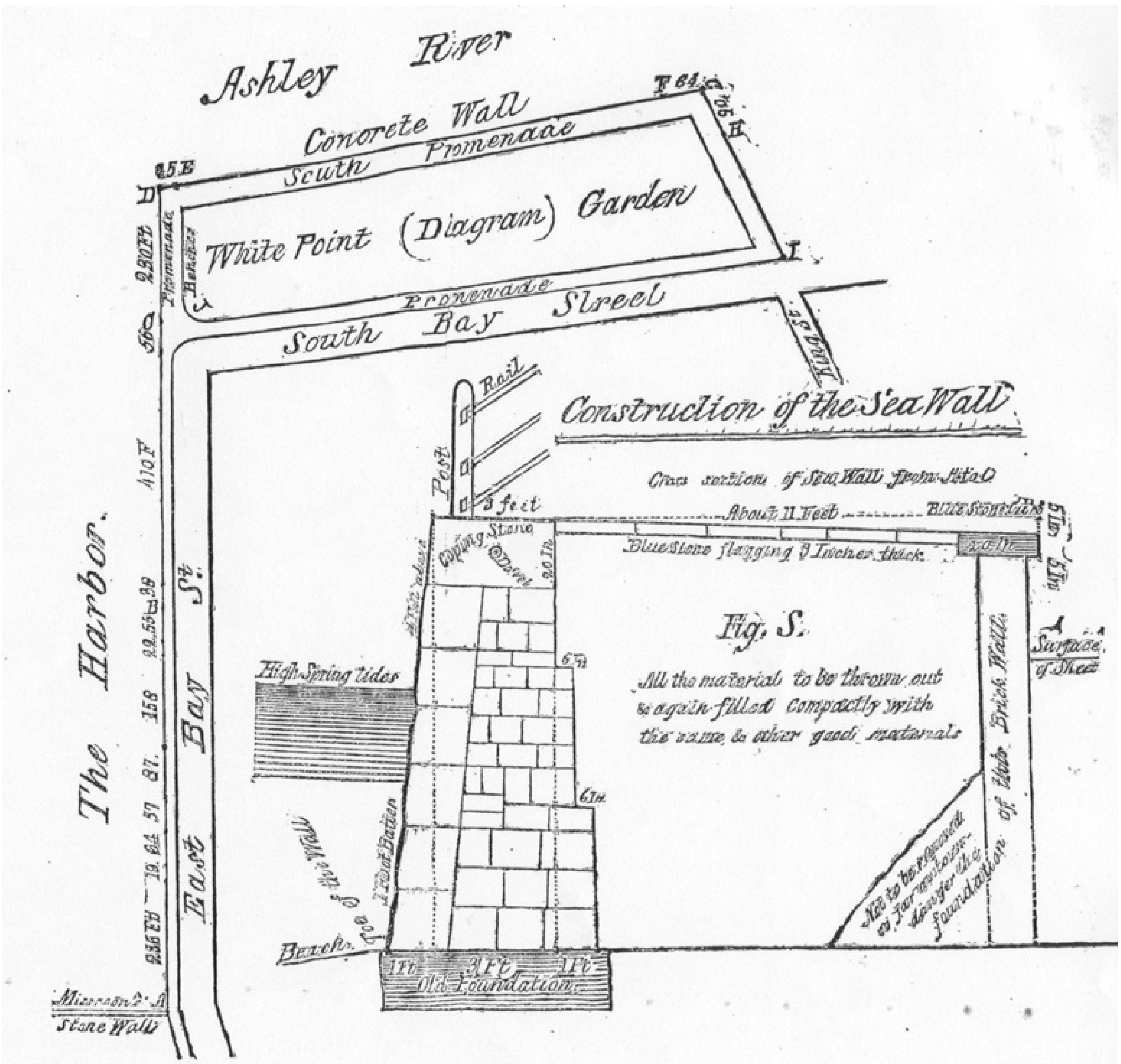
THE DESIGN OF THE SEA WALL ALLOWS FOR FUTURE MODIFICATION OF THE STRUCTURE TO ACCOMMODATE AN INCREASE OF THE WALL ELEVATION FROM 9.5 FT (NGVD29) TO 12.0 FT (NAVD88), AS PER RECOMMENDATIONS BY THE ARMY CORPS OF ENGINEER'S FEASIBILITY REPORT / ENVIRONMENTAL ASSESSMENT, DATED APRIL OF 2020.

Take Care,
Jim

James K. O'Connor, P.E., CEng MIEI
Vice President
Johnson, Mirmiran & Thompson, Inc.
An Employee Owned Company
235 Magrath Darby Boulevard, Suite 275
Mt. Pleasant, SC 29464
Direct. 843-779-3700
Mobile: 843-452-3266
Fax. 843-556-4329
joconnor@jmt.com

HIGH BATTERY WALL

The construction of the existing High Battery Wall is not sufficient to support raising the level of protection to EL 12.0 NAVD 88. Given its age and the assumed construction techniques used for the time period of which it was constructed, it is safe to assume that the high battery wall will not meet the criteria to be part of the Federal project. The High Battery Wall will be replaced with a new T-Wall with Walkway.



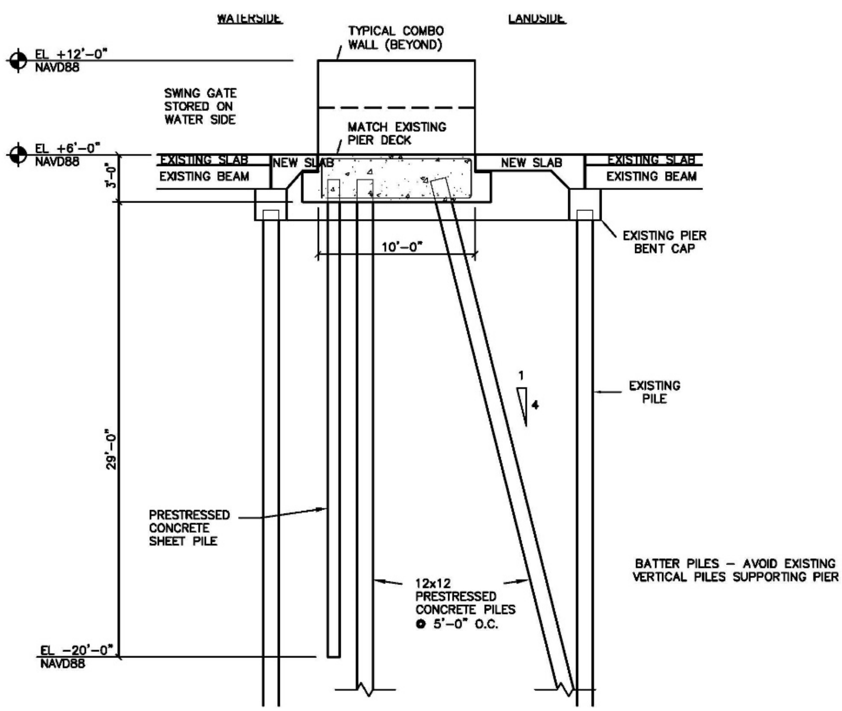
Existing High Battery Wall

GATES

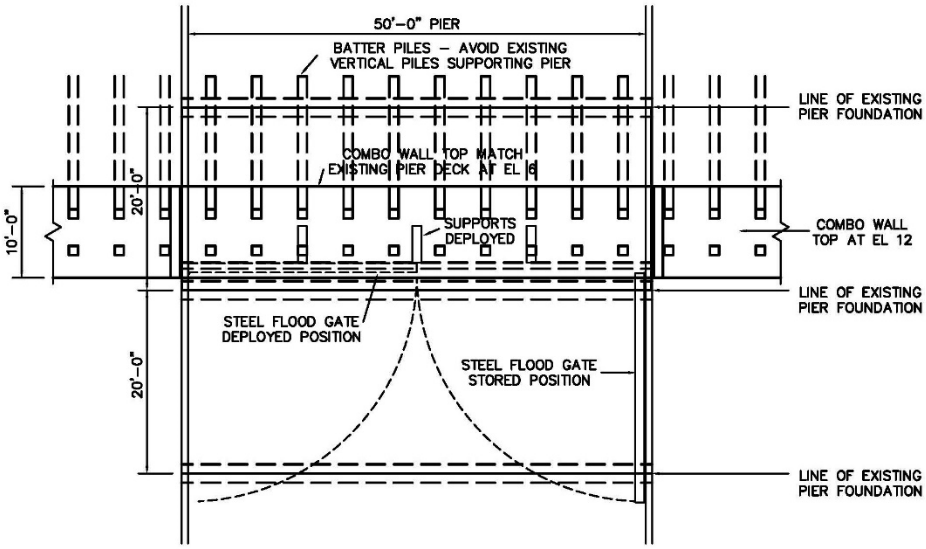
Preliminary structural analysis and design was performed for the gates. Many types of gates will be required ranging from very long gates across roadways to very short gates across pedestrian access routes.

One unique gate that was studied will occur where a Combo Wall crosses the Coast Guard Dock. A portion of the dock will have to be removed to construct the Combo Wall and the pier will be restored with load capacity equal to its original load rating along with adding a new 50’ wide swing gate. The gate will be supported with intermediate diagonal frames to limit the span to 12.5 ft.

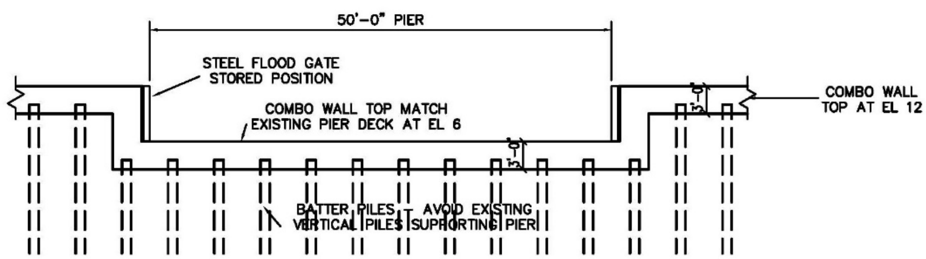
Aerial View at Coast Guard Base



Typical section of Combo Wall Crossing Coast Guard Dock



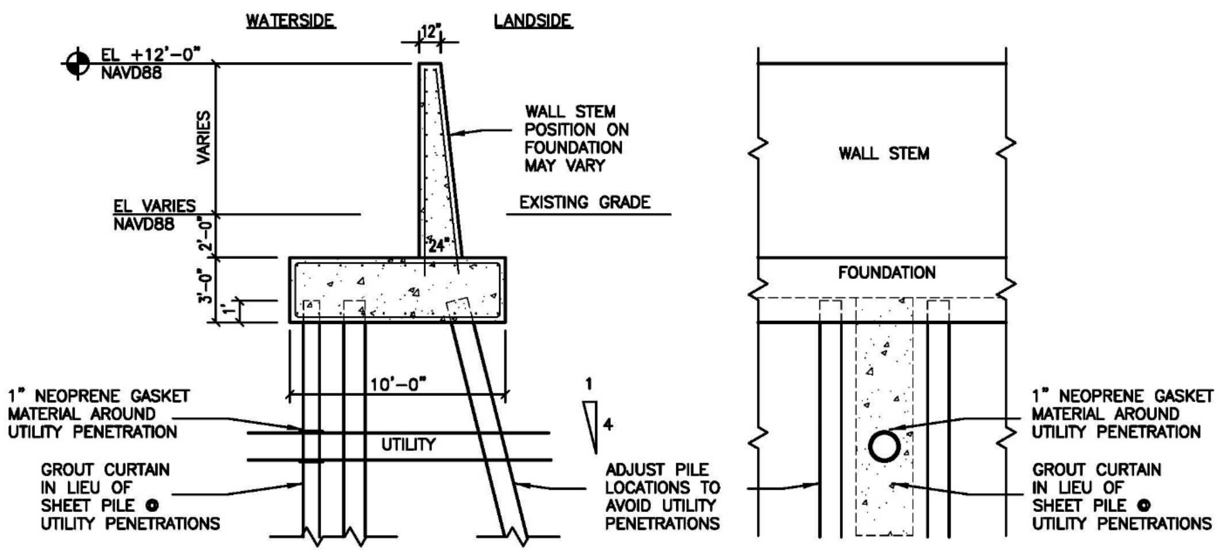
Plan of Combo Wall Crossing Coast Guard Dock



Elevation of Combo Wall Crossing Coast Guard Dock

UTILITY CROSSINGS

Consideration was given to a method of assuring the continuity of the sheet pile cutoff wall at utility crossings. Utilities would need to remain in service throughout the construction of the flood wall. Utilities would need to be located prior to excavation or driving any piles. Sheet pile construction presents the greatest challenge since it must be continuous in order to function as a cutoff wall. The solution is to omit the sheet pile at any utility crossing and jet grout the cutoff wall panel around the utility as shown below.



Grout Curtain Cutoff Wall Construction at Utility Crossings

BRIDGE CLEARANCES

Consideration has been given to special cases where the flood barrier will pass under existing bridges. Full height piles will not be able to be installed in areas with low vertical clearance. Piles would need to be installed in sections and spliced by welded or bolted connections. Micropiles could be used since they typically come in sections and are joined with threaded connections. Below are three locations where limited vertical clearance is a concern.

James Island Connector - ~20 ft clearance from existing grade (T-Wall)
Ravenel Bridge - ~25 ft clearance from existing grade in the parking lot (T-Wall)
Highway 17 at Lockwood - ~17 ft clearance from existing grade (T-Wall)

FUTURE DETAILING AND RESILIENCY

Due to sea level rise and the harsh marine environment where the barrier is to be constructed, measures should be taken to ensure the barrier can adapt to the changing environment, continue to perform well throughout the life of the project, reduce required maintenance, and ensure longevity. All of the items listed below have been considered and will continue to be incorporated during the Preliminary Engineering and Design (PED) Phase.

Oversize substructure and superstructure to accommodate future raising
Plan for longevity and maintainability using durable materials like stainless steel
Facilitate gate storage by storing nearby
Facilitate gate deployment

INCREASING BARRIER HEIGHT

The T-Wall and Combo Wall have vertical and battered piles which will be driven into the Cooper Marl stratum. The load capacity of piles driven into Marl increases dramatically with each additional foot of penetration. The required pile embedment for the flood barrier with 3 feet of additional height has been calculated and accounted in the structural analysis for feasibility. During PED phase, the concrete reinforcement for all wall types should also accommodate the forces resulting from future increase in height. Future raising should only require dowelling into the top of the flood barrier to add rebar and to increase the height of the wall stem.

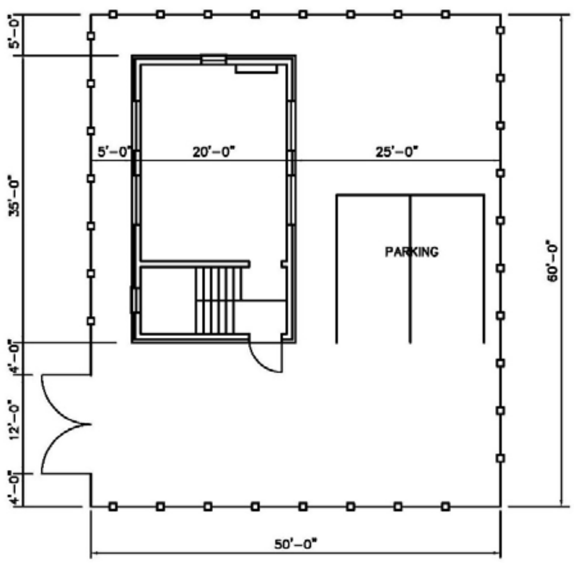
CORROSION PREVENTION

This project will be constructed around the perimeter of the Charleston Peninsula near saltwater and salt marshes and in a highly corrosive environment. Corrosion prevention measures should be taken into consideration to reduce required maintenance and ensure longevity. Examples of corrosion prevention measures include:

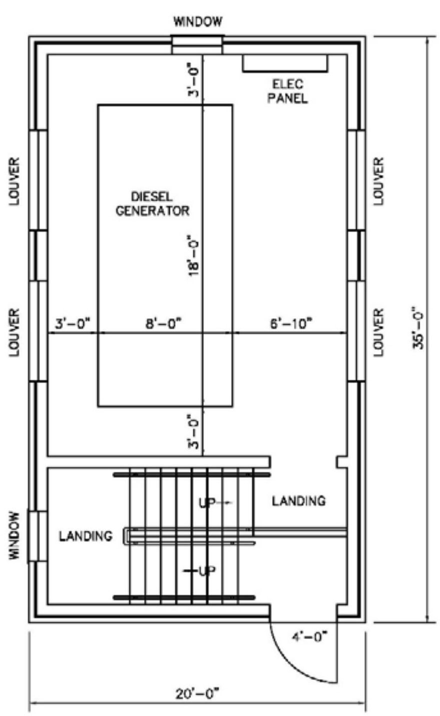
Noncorrosive rebar, such as galvanized, epoxy coated, or FRP composite
Noncorrosive sheet pile, such as prestressed concrete, vinyl, or FRP composite
Corrosion inhibiting admixtures for concrete
Stainless steel for railings and hardware

PUMP STATION

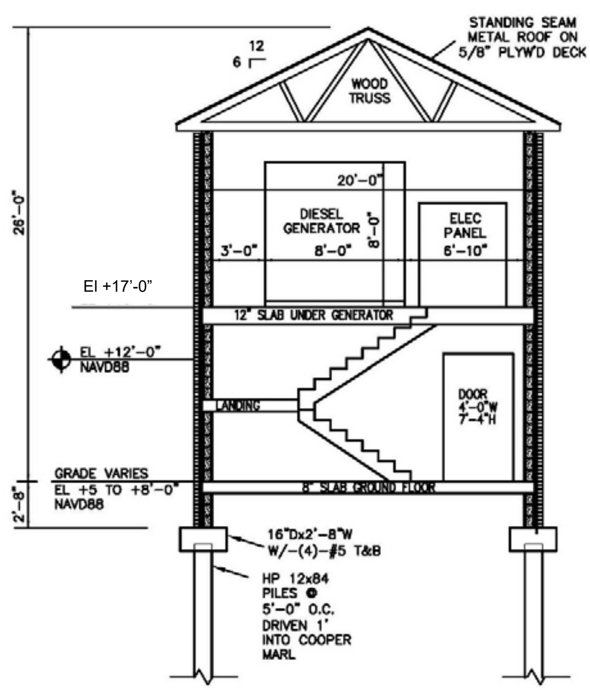
Pump stations for interior drainage will be required in several locations. Equipment such as electrical panels and controls will need to be elevated above the flood elevation and contained in a building for protection. The floor elevation is set at EL 17 to accommodate 3' future raising and an additional 2' above EL 12. Pump Stations were planned to accommodate an emergency generator; however, it was noted that existing City of Charleston pump stations do not contain emergency generators and are powered solely by the local utility company.



Site Plan of Pump Station



Floor Plan of Pump Station



Typical Section of Pump Station

ATTACHMENTS

Wind Load Calculations
Seismic Load Calculations
T Wall Calculations (Summary Sheet)
Combo Wall Calculations (Summary Sheet)
Pedestrian and Vehicle Gate Calculations
Trip Report – City of Charleston Work Trestle
Trip Report – Coast Guard Base

US Army Corps of Engineers®

TITLE: CHARLESTON PENINSULA FEASIBILITY STUDY

BLDG: ______ J.O.: ______ PAGE: ______ OF ______

BY: CRR DATE: 7/2019 CHKD: ______ DATE: ______

← USE ASCE 7-16 during PED

WIND ANALYSIS (ASCE 7-10, CHAPTER 26)

RISK CATEGORY III

$V = 154$ MPH

$q = 0.00256 K_z K_{zt} K_d V^2$

$K_d = 0.85$ (TABLE 26.6-1)

$K_z = 1.03$ (TABLE 27.3-1) EXPOSURE D

$K_{zt} = 1.0$

$q_h = (0.00256)(1.03)(0.85)(1.0)(154)^2 = 53.15$ PSF

$P = q_h G C_F$ (29.4-1)

$G = 0.85$

$S/H = 1$ (SOLID WALL)

ASPECT RATIO > 45 (VERY LONG WALL)

$\Rightarrow C_F = 1.3$

$p = (53.15 \text{ PSF})(0.85)(1.3) = \underline{58.73}$ PSF

$\therefore$ WIND PRESSURE FOR DESIGN OF BARRIER SHALL BE 58.73 PSF

US Army Corps of Engineers®

TITLE: CHARLESTON PENINSULA FEASIBILITY STUDY

BLDG: ______ J.O.: ______ PAGE: ______ OF ______

BY: CRB DATE: 7/2019 CHKD: ______ DATE: ______

USE ASCE 7-16 during PED

SEISMIC ANALYSIS (ASCE 7-10, CHAPTER 15)

$S_s = 1.58\ \%g$ (USED NWS CHARLESTON VALUES FROM UFC 3-301-01)

$S_1 = 0.53\ \%g$

SITE CLASS = E (ASSUMED) ← Note: consider if Site Class D may be used during PED

$S_{MS} = F_a S_s$ $F_a = 0.9$ $S_{MS} = (0.9)(1.58) = 1.42$

$S_{M1} = F_v S_1$ $F_v = 2.4$ $S_{M1} = (2.4)(0.53) = 1.27$

$S_{DS} = \frac{2}{3} S_{DS} = (2/3)(1.42) = 0.948$

$S_{D1} = \frac{2}{3} S_{D1} = (2/3)(1.27) = 0.848$

PER 11.6 SEISMIC DESIGN CATEGORY = E

RISK CATEGORY III ⇒ $I_e = 1.25$

$R = 2$ (ASSUMED, 11.5.4.1)

$C_s = 0.8 S_1 (R/I_e) = (0.8)(0.53)(2/1.25) = 0.678$

$T < 0.06$ SEC (ASSUMED) ← Note: verify this assumption revise if necessary in PED

$V = 0.30\, S_{DS}\, W I_e = (0.3)(0.948)(1.25) W$ $\boxed{= 0.36 W}$

∴ $V = 0.36W$ AND WILL BE APPLIED TO THE STEM AT MID-HEIGHT FROM TOP OF STEM TO FINISHED GRADE OR HORIZONTAL LEG OF WALL

US Army Corps of Engineers	DESIGN ANALYSIS	☐ BUDGET ☐ PRELIMINARY X FINAL ☐ OTHER	SHEET NO: ______ OF: ____ JOB NO:
PROJECT NAME:	CHARLESTON PENINSULA FEASIBILITY STUDY		DEPARTMENT STRUCT SHEET NO: OF:
PROJECT PART:	GEOMETRY AND MATERIAL PROPERTIES		COMPUTED BY: CRB DATE:
SPEC. DIVISION:			CHECKED BY: DATE:

DIMENSIONS

STEM:

bst	1	FT
bsb	2	FT
hs	10	FT (EXISTING GRADE IS 4 FT)

SLAB:

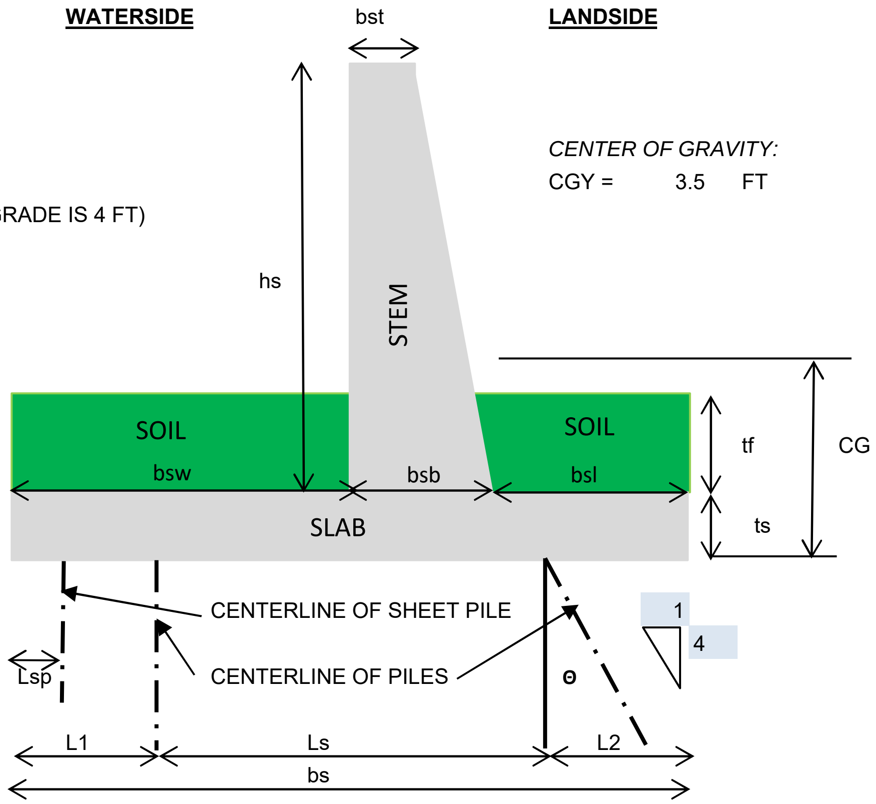
ts	3	FT
bs	10	FT
bsw	5	FT
bsl	3	FT

PILES:

Θ	14.04	DEG (4V:1H)
L1	3	FT
L2	3	FT
Ls	4	FT
s	5	FT (spacing)
Lsp	2	FT

SOIL:

tf	2	FT
ϕ	20	°
Ko	0.49	



CENTER OF GRAVITY:

CGY = 3.5 FT

LOADS

γc	150	PCF (WEIGHT OF CONCRETE)
γw	64	PCF (WEIGHT OF WATER)
γs	120	PCF (WEIGHT OF SOIL)
Eh	0.36	x WEIGHT OF T-WALL
W	58.73	PSF (WIND PRESSURE)

Uw1	810	PSF (UPLIFT PRESSURE WATERSIDE OF SHEET PILE)
Uw2	315	PSF (UPLIFT PRESSURE LANDSIDE OF SHEET PILE)

REQUIRED PILE CAPACITIES

LOAD CASE	VERT. PILE			BATT. PILE (VERT.)			LATERAL			BATT (TOTAL)	
UNLOADED	20.1	KIPS	*COMP*	23.3	KIPS	*COMP*	0.0	KIPS		24.0	KIPS
C1 - SURGE EL 12	-8.4	KIPS	*TEN*	31.1	KIPS	*COMP*	23.0	KIPS	*COMP*	38.7	KIPS
C2 - WAVE	-18.4	KIPS	*TEN*	41.1	KIPS	*COMP*	23.0	KIPS	*COMP*	47.1	KIPS
C3 - 2 FT OVER	-18.4	KIPS	*TEN*	41.0	KIPS	*COMP*	23.0	KIPS	*COMP*	47.0	KIPS
C4 - SEISMIC RT	9.5	KIPS	*COMP*	33.8	KIPS	*COMP*	12.2	KIPS	*COMP*	36.0	KIPS
C5 - SEISMIC LT	30.7	KIPS	*COMP*	-12.7	KIPS	TEN	-12.2	KIPS	TEN	-17.6	KIPS
C6 - WIND	14.8	KIPS	*COMP*	28.5	KIPS	*COMP*	2.3	KIPS	*COMP*	28.6	KIPS

* *SEISMIC RT = LOAD ACTING TOWARDS LANDSIDE*

* *SEISMIC LT = LOAD ACTING TOWARDS WATERSIDE*

Allowable 12" pile capacity, 4' into marl = 65 KIPS

This is greater than the highest pile load

US Army Corps of Engineers	DESIGN ANALYSIS ☐ BUDGET ☐ PRELIMINARY X FINAL ☐ OTHER	SHEET NO: ____ OF: ____ JOB NO:
PROJECT NAME: CHARLESTON PENINSULA FEASIBILITY STUDY		DEPARTMENT STRUCT SHEET NO: OF:
PROJECT PART: GEOMETRY AND MATERIAL PROPERTIES		COMPUTED BY: CRB DATE:
SPEC. DIVISION:		CHECKED BY: DATE:

WATERSIDE **LANDSIDE**

DIMENSIONS

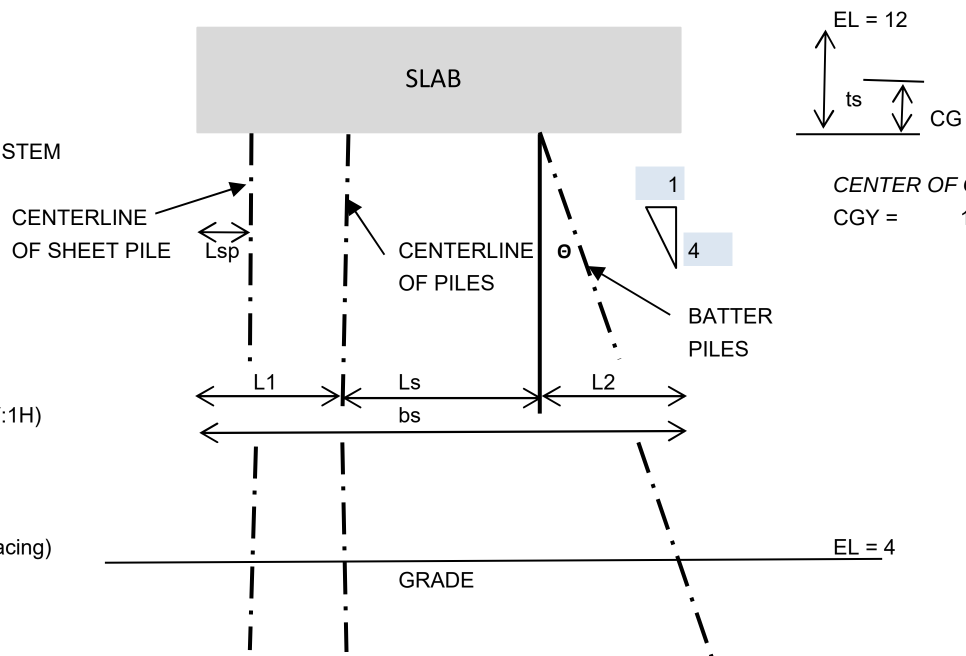
STEM:

bst	1	FT
bsb	2	FT
hs	0	FT

NO STEM

SLAB:

ts	3	FT
bs	10	FT
bsw	5	FT
bsl	3	FT



CENTER OF GRAVITY:
CGY = 1.5 FT

PILES:

ϴ	14.04	DEG (4V:1H)
L1	3	FT
L2	3	FT
Ls	4	FT
s	5	FT (spacing)
Lsp	2	FT

SOIL:

tf	0	FT
ϕ	20	°
Ko	0.49	

NO SOIL OVER SLAB

Top el	12
Grade el	4

LOADS

γc	150	PCF (WEIGHT OF CONCRETE)
γw	64	PCF (WEIGHT OF WATER)
γs	120	PCF (WEIGHT OF SOIL)
Eh	0.36	x WEIGHT OF T-WALL
W	58.73	PSF (WIND PRESSURE)

Uw1	192	PSF (UPLIFT PRESSURE WATERSIDE OF SHEET PILE)
Uw2	96	PSF (UPLIFT PRESSURE LANDSIDE OF SHEET PILE)

REQUIRED PILE CAPACITIES

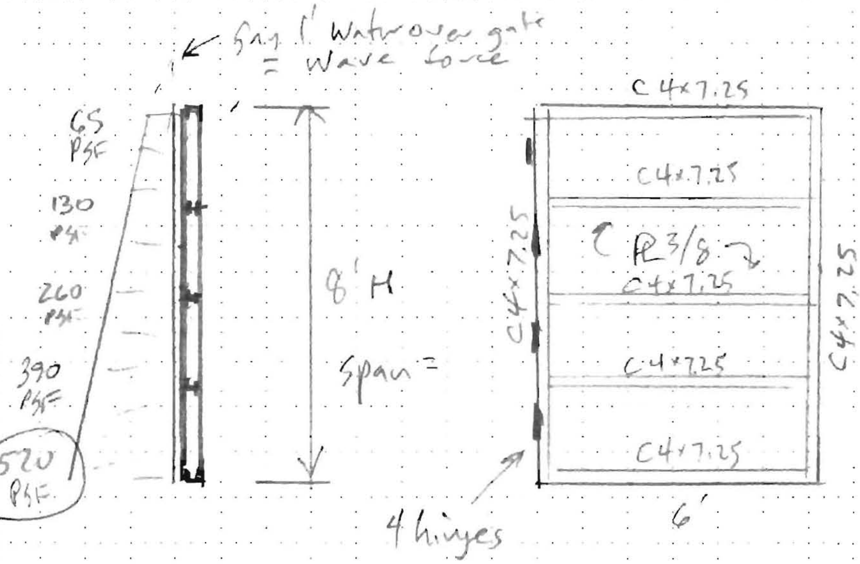
LOAD CASE	VERT. PILE			BATT. PILE (VERT.)			LATERAL			BATT (TOTAL)	
UNLOADED	11.3	KIPS	*COMP*	11.3	KIPS	*COMP*	2.8	KIPS	*COMP*	11.6	KIPS
C1 - SURGE EL 12	8.0	KIPS	*COMP*	12.6	KIPS	*COMP*	5.8	KIPS	*COMP*	13.9	KIPS
C2 - WAVE	6.8	KIPS	*COMP*	13.8	KIPS	*COMP*	10.8	KIPS	*COMP*	17.6	KIPS
C3 - 2 FT OVER	5.4	KIPS	*COMP*	13.9	KIPS	*COMP*	12.7	KIPS	*COMP*	18.8	KIPS
C4 - SEISMIC RT	8.2	KIPS	*COMP*	-14.3	KIPS	*TEN*	8.1	KIPS	*COMP*	16.4	KIPS
C5 - SEISMIC LT	14.3	KIPS	*COMP*	-8.2	KIPS	*TEN*	-8.1	KIPS	*TEN*	-11.5	KIPS
C6 - WIND	10.4	KIPS	*COMP*	-12.1	KIPS	*TEN*	1.6	KIPS	*COMP*	12.2	KIPS

* *SEISMIC RT = LOAD ACTING TOWARDS LANDSIDE*
* *SEISMIC LT = LOAD ACTING TOWARDS WATERSIDE*

Allowable 12" pile capacity, 4' into marl = 65 KIPS
This is greater than the highest pile load

Ped Gates: 6' W x 8' H

Single swing



Revise during PED:
9 x 62.4
= 562 PSF

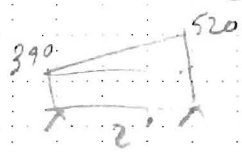
Bott channel
W = 520 L = 6 M = 2340 R = 1560
$S\ req'd = \frac{2340 \times 12}{22000} = 1.27$ C4 x 5.4 I = 1.93

Use C4x7.25 for all framing

* 1st beam
W = 390 x 2 = 780 L = 6 M = 3510 R = 2340
$S\ req'd = \frac{3510 \times 12}{22000} = 1.91$ C4x7.25 S = 2.29

Plate between beams

W =



say 455 avg
M = 228
$S\ req'd = \frac{228 \times 12}{22000} = 0.12$
$0.12 = \frac{12 \times t^2}{6}$ t = 0.25
Use 3/8" plate
0.375

ESTIMATE: Total wt: 5 x 7.25 x 6 = 218
2 x 7.25 x 8 = 116
$\frac{3/8}{12} \times 490 =$ 15.3 x 6 x 8 = 735
Total wt = 1069#

$\frac{1069^{\#}}{6 \times 8} = 22.3$ PSF

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CHARLESTON DISTRICT

Project Peninsula study
Subject vehicle gates
Design by RDL Check by
Date 10/13/20 Sheet 2 of 3

Vehicle Gates

Ht varies 3' then 7', 8', 9', 10' (12' at coast guard)
Use intermediate support frames
in the road between lanes, span = 12' max

10' high, 12' span

Revise during PED
11 × 62.4
= 686 PSF

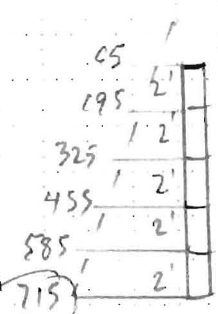
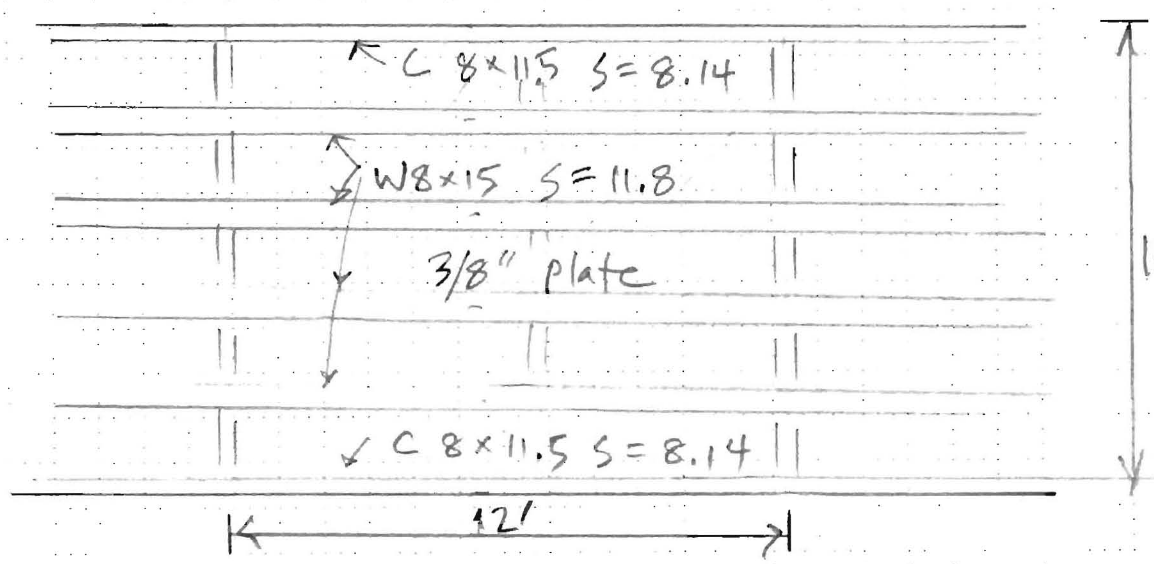


Plate
W = 650 AVG
L = 2'
M = 325
Srq'd = $\frac{325 \times 12}{22000}$ = 0.177 = $\frac{12 \times t^2}{6}$ t = 0.29   use 3/8" = 0.375

M @ 1st beam $585 \times 2 \times 12^2/8$ = 21.1 ftK
Srq'd = $\frac{21.1 \times 12}{22}$ = 11.5

M @ bottom $715 \times 12^2/8$ = 12.9 ftK
Srq'd = $\frac{12.9 \times 12}{22}$ = 7.02



Estimate:

Plate:	15.3 × 10 × 12	= 1836#
channel	2 × 11.5 × 12	= 276
Beam	4 × 15 × 12	= 720
	2 × 15 × 10	= 300
Diag	2 × 15 × 12	= 360
		3492#

$\frac{3492}{10 \times 12}$ = 29.1 psf

At coast guard dock, H = 12' span = 10'

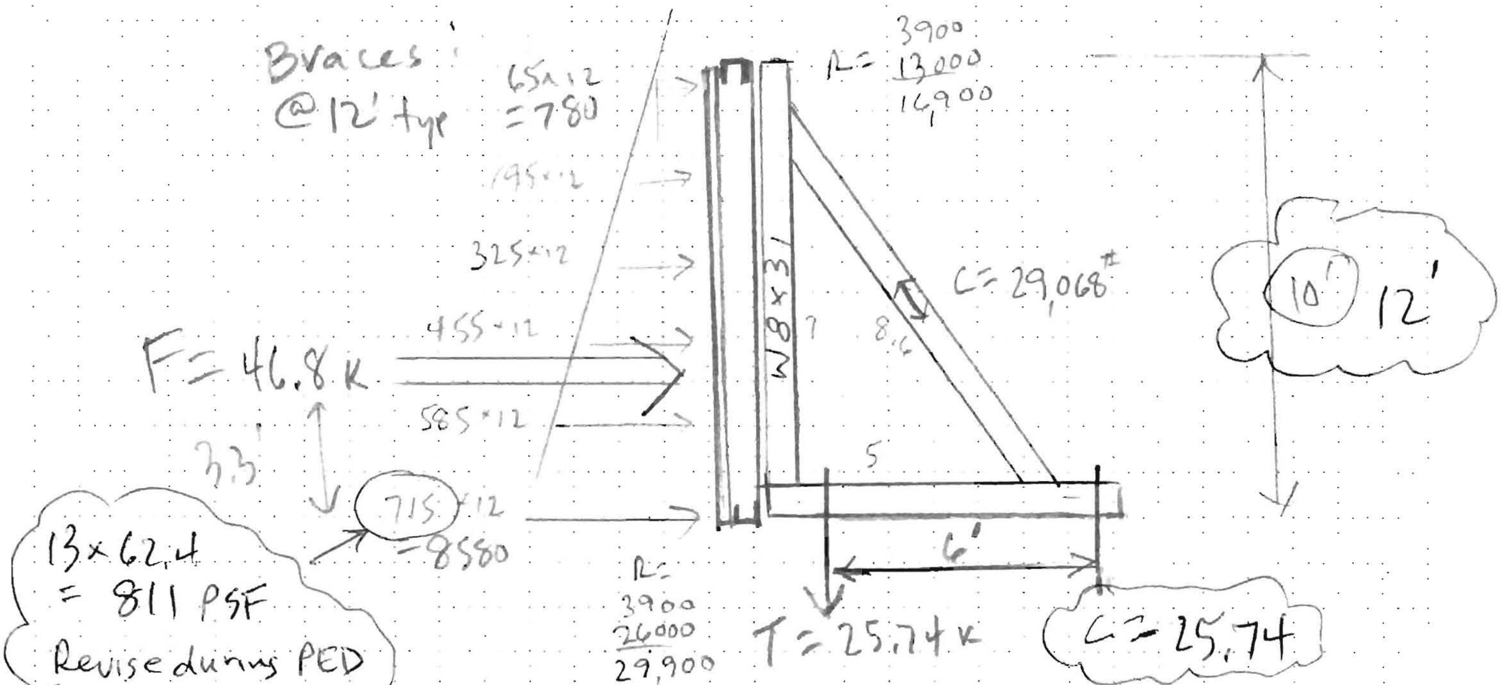
w @ 1st beam = 715

M = 715 × 2 × 10²/8 = 17.9 ftk < 21.1 for typical

W @ bottom = 845

M = 845 × 10²/8 = 10.6 ftK < 12.9 for typical

∴ use same member sizes for 12' high, 10' span at coast guard



Vert $M = \frac{780 \times 10^2}{8} + (.1283 \times 39{,}000 \times 10) = 59.8$ ftk W8×31 ✓

Diag C = 29.1 K use W8×31

Base use W8×31

Total wt = 31 × (10 + 8 + 9) = 837#

PROJECT SITE VISIT REPORT FORM

Project Name: City of Charleston stormwater outfall construction trestle
Project Location: between US 17 bridges, Charleston side of Ashley river
Date: 10-21-2020 **Time:** 1500
Approx. Temperature: 75°F **Weather:** Cloudy
Reported By: Rick Lambert, PE, CVS
Attendees: Sara Brown, Molly Holt, Steven Kirk (City of Charleston), various contractor personnel.

Currrent Condition:

1. Trestle was constructed to provide access to excavate a shored trench to construct a new pump station and outfall pipe. This is similar to what would be needed to construct a floodwall across marsh as part of the Charleston peninsula flood protection system.
2. Trestle is 30 feet wide and 529 feet long. Total cost reported by City of Charleston for design, construction and removal of the trestle was $2.75 Million. This equates to $173 / SF.

Observations:

1. Deck is 30' wide. Deck is comprised of large timbers. There is a curb and railing on the sides.
2. A 220 ton capacity Manitowoc crane with a very long boom was operating on the deck. The crane occupied nearly the full width of the trestle so a vehicle could not pass the crane.
3. Steven Kirk furnished drawings and construction photos for the trestle by email after the site visit.

Photos:

View of trestle deck and crane

Photos:

150’ deep wet well shaft in shored excavation

Photos:

Trestle allows access to construct dewatered trench for outfall
Note trestle, fall protection railing, shored sheet pile wall
We would not need shoring for flood wall construction

Photos:

Dewatering pump – we would not need this for floodwall construction

DEPARTMENT OF THE ARMY
Corps of Engineers, Charleston District
69A Hagood Avenue • Charleston • South Carolina • 29403-5107
www.sac.usace.army.mil

PROJECT SITE VISIT REPORT FORM

Project Name: Coast Guard Pier
Project Location: Coast Guard Base, Tradd Street, Charleston, SC
Date: 9-30-2020 **Time:** 0830
Approx. Temperature: 75°F **Weather:** Scattered clouds
Reported By: Rick Lambert, PE, CVS
Attendees: Sara Brown
Dave Garvis (Coast Guard) 843-614-0590

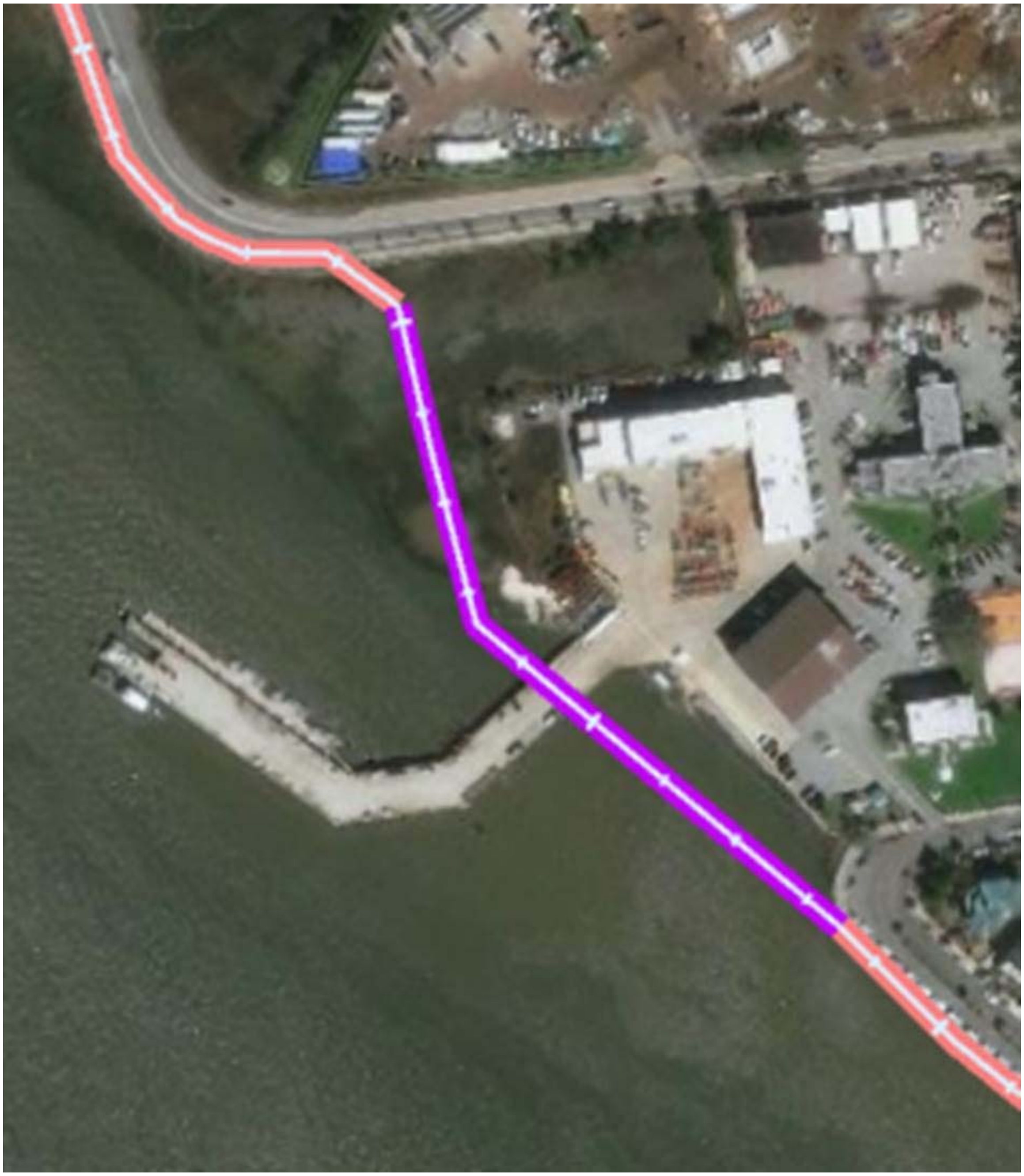
Currrent Condition:

1. The floodwall proposed for the Charleston Peninsula Study will traverse the Coast Guard Base Property.
2. The current plan has the combo wall crossing the Coast Guard Pier.
3. Coast Guard contact can obtain the original pier drawings.

Observations:

1. Pier is a bridge structure, 50' wide from outside to outside of curbs and has supporting bents spaced at 20 feet apart.
2. There are a lot of utilities serving the pier: Water, sewer, power, fuel, comm.
3. North side of the Coast Guard property is very low and floods frequently.

Aerial view:



1. Google Earth view of combo wall crossing Coast Guard Base

Photographs taken:

2. Coast Guard Pier looking at southwest toward the Ashley River

Photographs taken:

3. Coast Guard Pier looking at downstream side

US Army Corps of Engineers®

Charleston District

CHARLESTON PENINSULA, SOUTH CAROLINA, A COASTAL STORM RISK MANAGEMENT STUDY

Charleston, South Carolina

GEOLOGIC AND GEOTECHNICAL SUB-APPENDIX

May 2022

Version: Final, May 2022

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Charleston Peninsula
Coastal Flood Risk Management Feasibility Study
Geologic and Geotechnical Engineering Sub-Appendix

Version: Final, May 2022

Table of Contents

Figures

Tables

Attachments

1. INTRODUCTION

The Geology and Geotechnical report documents the subsurface conditions and the geotechnical engineering evaluation completed for the Charleston Peninsula Coastal Storm Risk Management (CHS CSRM) Feasibility Study.

1.1. Area Description

The study area is defined as the peninsula of Charleston. The structural alternative studied by the CHS CSRM was located along the edge of the peninsula, mainly in the tidal marsh areas.

1.2. Existing Data and Its Use

For the CHS CSRM Feasibility Study, no new geotechnical data were collected as part of this study due to funding and time constraints. Only existing and available geotechnical data were used. Various geotechnical reports were obtained from various engineering firms. This information was used in making design assumptions.

2. DATUMS

The horizontal and vertical datums used for the project are indicated below unless otherwise stated:

Horizontal Datum: South Carolina State Plane, North American Datum of 1983

Vertical Datum: North American Vertical Datum of 1988 (NAVD88)

3. REGIONAL GEOLOGY

A compilation of geotechnical data were sent to the study's geology and geotechnical team from various consulting agencies within the public and private sector. Over 200 Cone Penetration Tests (CPTs) and Standard Penetration Test (SPT) borings were obtained and plotted into ArcMap. Borings were analyzed for easting and northing coordinates, depth of boring, and top of Cooper Marl Formation. Data plotted in ArcMap used coordinates provided on the logs; however, if easting and northing coordinates were not present, the borings were plotted visually from the maps provided by the consulting agencies. Based on the boring data collected, the top of the Cooper Marl Formation is depicted similarly to Figure 1. The rest of the document depicts the geologic setting and stratigraphy beneath the Charleston Peninsula.

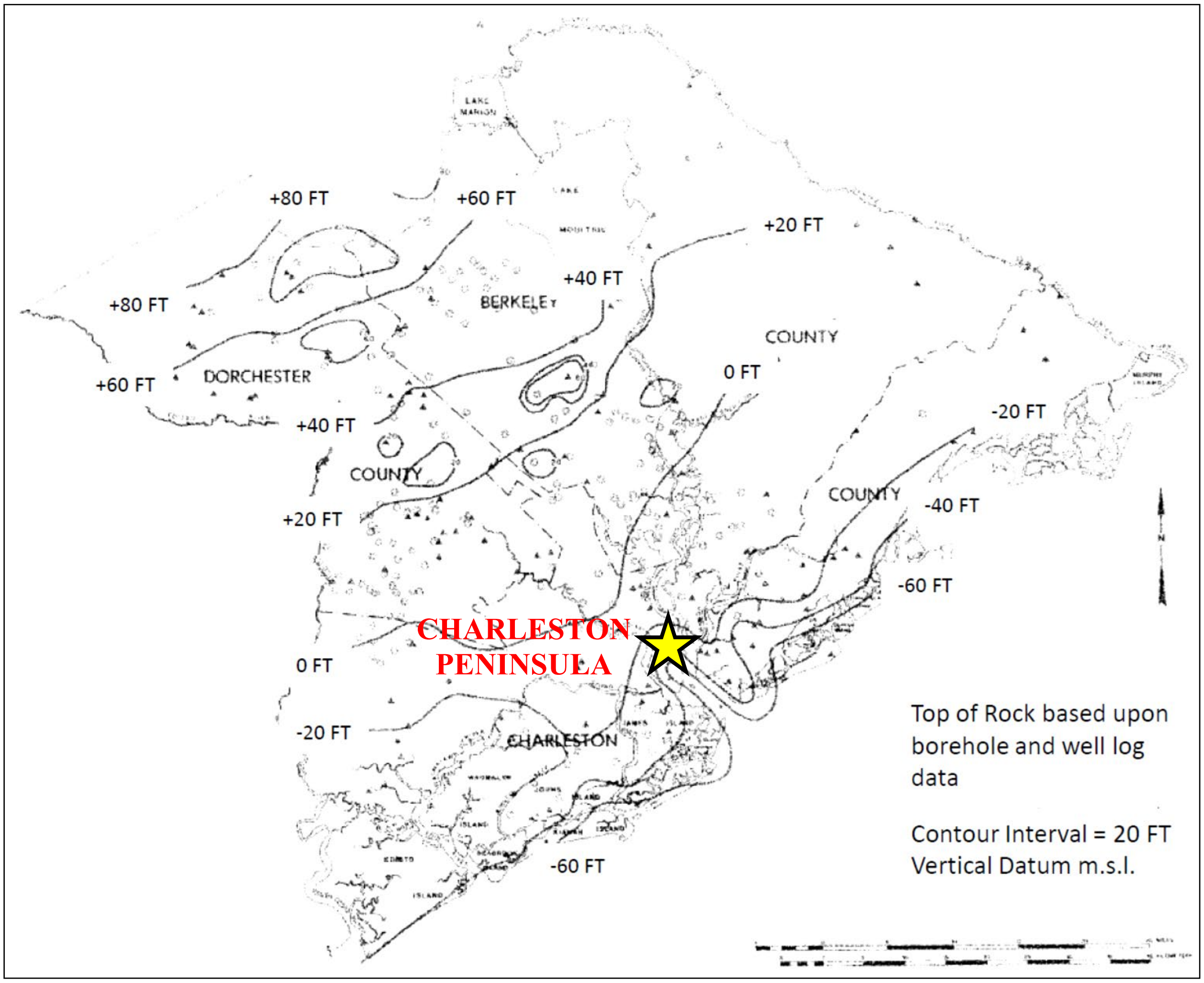

Figure 1: Structure contour map showing top of Cooper Formation, from Park (1985).

However, the term "Cooper Formation" (Toumey, 1848) is the most recognized name for the unit, and is hereby informally extended to encompass the Ashley and Chandler Bridge Formations described by Weems and Lemon (1993) and Weems and Lewis (2002). Therefore, for the purposes of this study, the term "Cooper Formation" will be used to describe the fine-grained, stiff to very stiff, low permeability strata that comprise much of the subsurface with the upper and lower harbor.

3.1. Geologic Setting

The Charleston Peninsula project site lies within the South Carolina Coastal Plain (Figure 2). Deep crustal faulting associated with Late Triassic rifting produced a subsiding depositional basin which contains Cretaceous and Tertiary sediments (Harris et al., 1979; Horton and Zullo, 1991; Harris et al., 2005). The stratigraphy of the South Carolina Coastal Plain consists of partially consolidated, unconformity bound, southeast dipping estuarine-marine shelf Tertiary deposits, which are overlain by unconsolidated Quaternary barrier and nearshore deposits. Superimposed upon this stratigraphy are escarpments and terraces that were carved into the strata as a result of interglacial sea-level fluctuation that began as early as 240,000 years ago (Weems and Lemon, 1993). The development of the modern shoreface with its barrier islands, inlets, and intertidal waters was strongly influenced by the geology and topography of resistant strata (Harris et al., 2005).

3.2. Stratigraphy

The stratigraphic units that are most significant to the project are Tertiary in age. Specifically, these units are the Black Mingo Group, Santee Limestone, Cooper Formation, Edisto Formation, and Marks Head Formation. These stratigraphic units are relevant because of their hydrogeologic properties or their occurrence within the project site (Figure 3). The units are lithologically distinct from each other and are disconformity bound. Pre-Cretaceous basement crystalline rocks and Cretaceous-age strata belonging to the Middendorf, Black Creek, and Pee Dee Formations lie at elevations of -3000 to -200 feet mean sea level (MSL), and are too deeply buried to be of engineering concern for this project. Quaternary units are generally found as surficial unconsolidated deposits along the shoreline and inland areas.

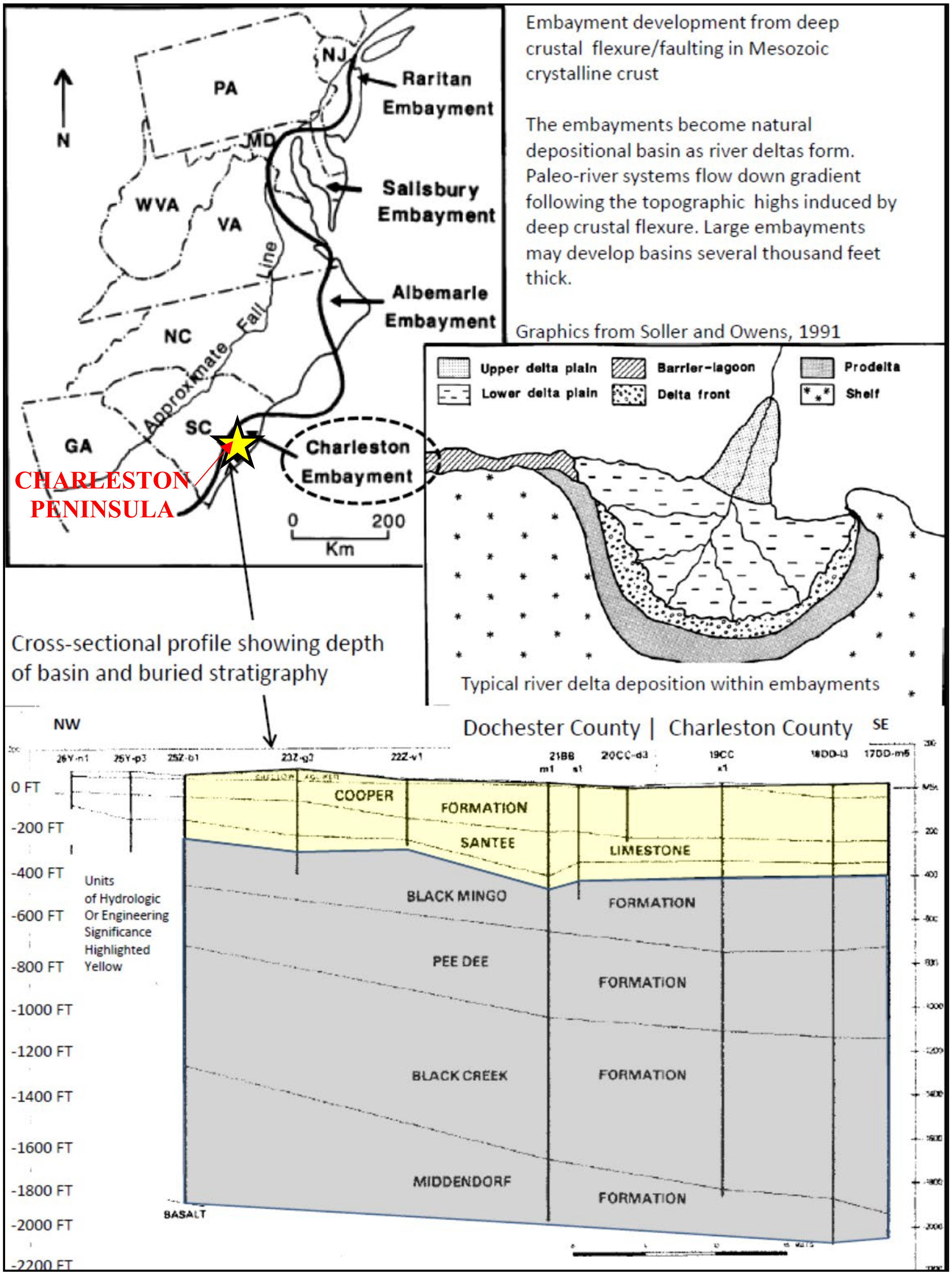

Figure 2: Regional geologic setting of the Charleston Embayment from Soller and Owens (1991) and Park (1985).

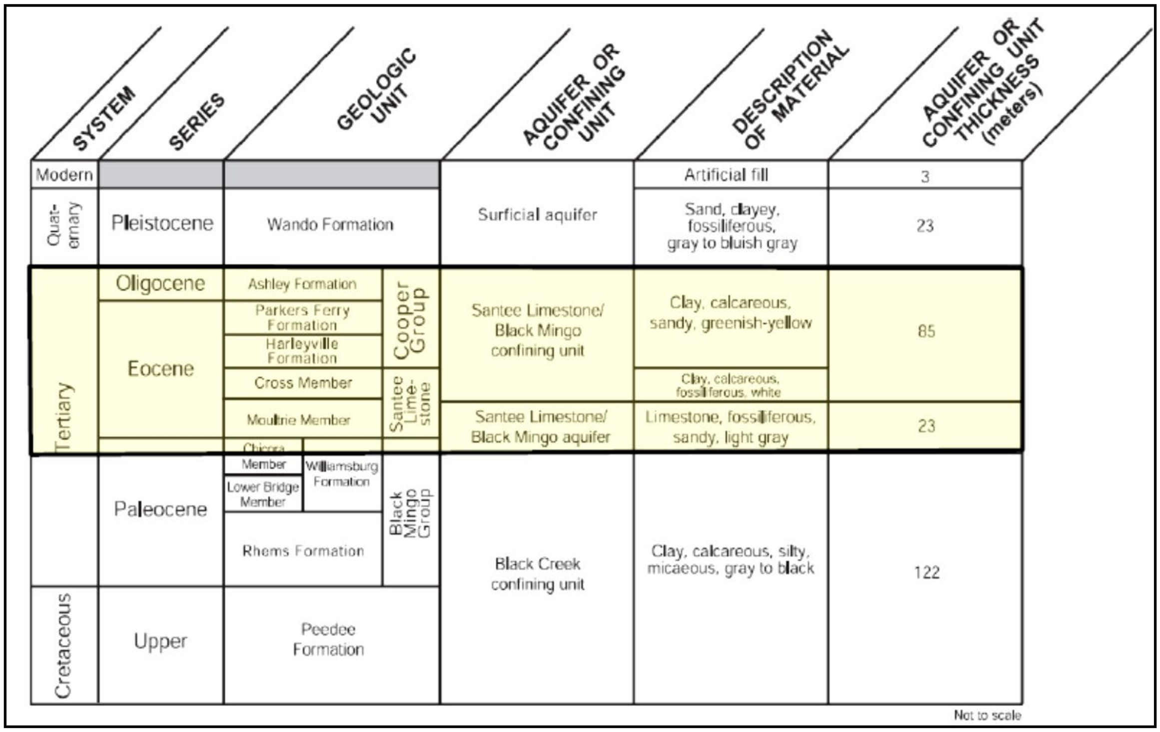

Figure 3: Project relevant stratigraphic & hydrogeologic units, from Petkewich et al. (2004).

3.2.1. Black Mingo Group

The Black Mingo Group was named for exposures of mudstone along the Black River and Black Mingo Creek by Sloan (1907). Other agency and private drill core data indicates that the unit is heterogeneous and comprised of interbedded sequences of laminated clay, mudstone, sand, and limestone. The base of the unit is predominantly composed of mudstone and silty-clay interbedded with calcareous sands with occasional limestone, whereas the top of the unit is predominantly fossiliferous limestone interbedded with quartz sand and occasional clay (Weems and Bybell, 1998; Edwards et al., 1999). The Black Mingo sediments are generally a mixture of clastic detrital material and volcanic ash that were deposited within inner shelf and marginal marine environments during the Late Paleocene to Early Eocene. Outcroppings of the formation occur in Monck’s Corner and surrounding counties, and it dips south-southwest into the subsurface to a depth of -600 feet MSL below southern Charleston County (Park, 1985).

3.2.2. Santee Limestone Formation

The Santee Limestone is named for exposures that occur along the Santee River in South Carolina where it underlies the Cooper Group (Sloan, 1908). The Santee Limestone is creamy-white to gray, fossiliferous, glauconitic, and has sand to mud-supported matrix. The unit is middle to late Eocene in age and disconformity bound (Park, 1985). Two members are generally recognized within the Santee Limestone; the middle Eocene Moultrie Member and middle to late Eocene Cross Member (Figure 3). The Moultrie Member of the Santee Limestone is approximately 7-feet thick (from recovered drill cores) and the limestone matrix tends to be coarse-grained, bioturbated, moldic, and sandy. The Cross Member is significantly thicker (39-feet thick from drill core) with a finer-grained, clayey matrix. Deposition of the Santee

Limestone occurred 45-41 million years before present, when shallow open marine-shelf environments were drowned and transformed into deeper outer continental shelf environments (Petkewich et al., 2004). The Santee Limestone is exposed in surficial exposures located along a 5-mile wide belt that extends across northern Dorchester, Berkeley, and Charleston Counties, and it dips into the subsurface towards the south-southeast (Figure 4). The top of the formation lies at elevation -300 feet MSL beneath Charleston Harbor. The unit thickens southwestward from 20-feet thick near Lake Moultrie to over 260-feet thick beneath Edisto Island (Park, 1985).

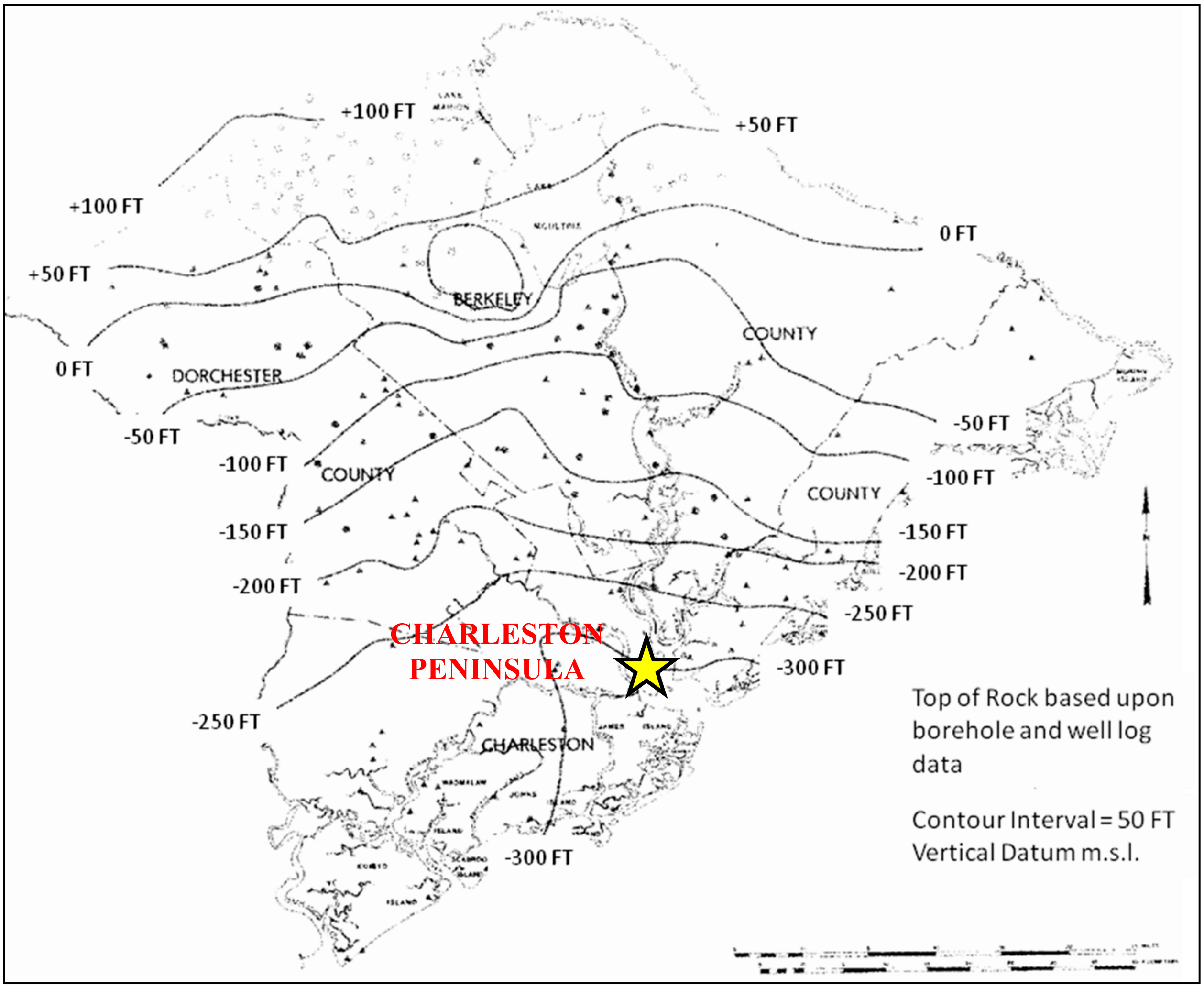

Figure 4: Structural contour map showing top of Santee Limestone, from Park (1985).

3.2.3. Cooper Formation

The Cooper Formation was originally termed “Cooper Marl” by Toumey (1848) for exposures of soft, very fine-grained, impure carbonate material found along the Cooper River and Ashley Rivers in South Carolina. This unit has been described by various workers in surficial exposures within the coastal plains of North Carolina, South Carolina, and Georgia (Toumey, 1848; Cooke, 1952; Malde, 1959; Weems and Lemmon, 1993; Weems and Lewis, 2002). Carbonate-rich sections of the unit were extensively studied and served as a source for agricultural lime production between 1867 and 1920. Upland exposures of the Cooper Formation are described as consisting of fine-grained calcareous foraminiferal shell material (Malde, 1959; Gohn et al., 1977; Park, 1985). In contrast, soil borings, grab samples, and surficial exposures of the Cooper Formation within Charleston Harbor resemble a consolidated and low permeability soil that ranges in composition from stiff clayey silt to dense silty sand. Weems and Lemon (1993) indicated that the Cooper Formation (Toumey, 1848) actually consists of a composite sequence of variably consolidated silt and clay, soft clayey and sandy limestones, and phosphatic deposits of Eocene-Oligocene age (Park, 1985; Weems and Lemon, 1993).

Structural contour maps indicate that the Cooper Formation dips into the subsurface toward the south-southeast at a gradient of 8 feet per mile (Figure 1). Beneath the city of Charleston, the top of the Cooper Group lies at an elevation of -20 feet MSL, but due to the dipping gradient and high subsurface relief, it plunges to a depth of -60 feet MSL near the mouth of the harbor. Parks (1985) determined that the stratum thickens to 280 feet beneath Charleston Harbor (Figure 5).

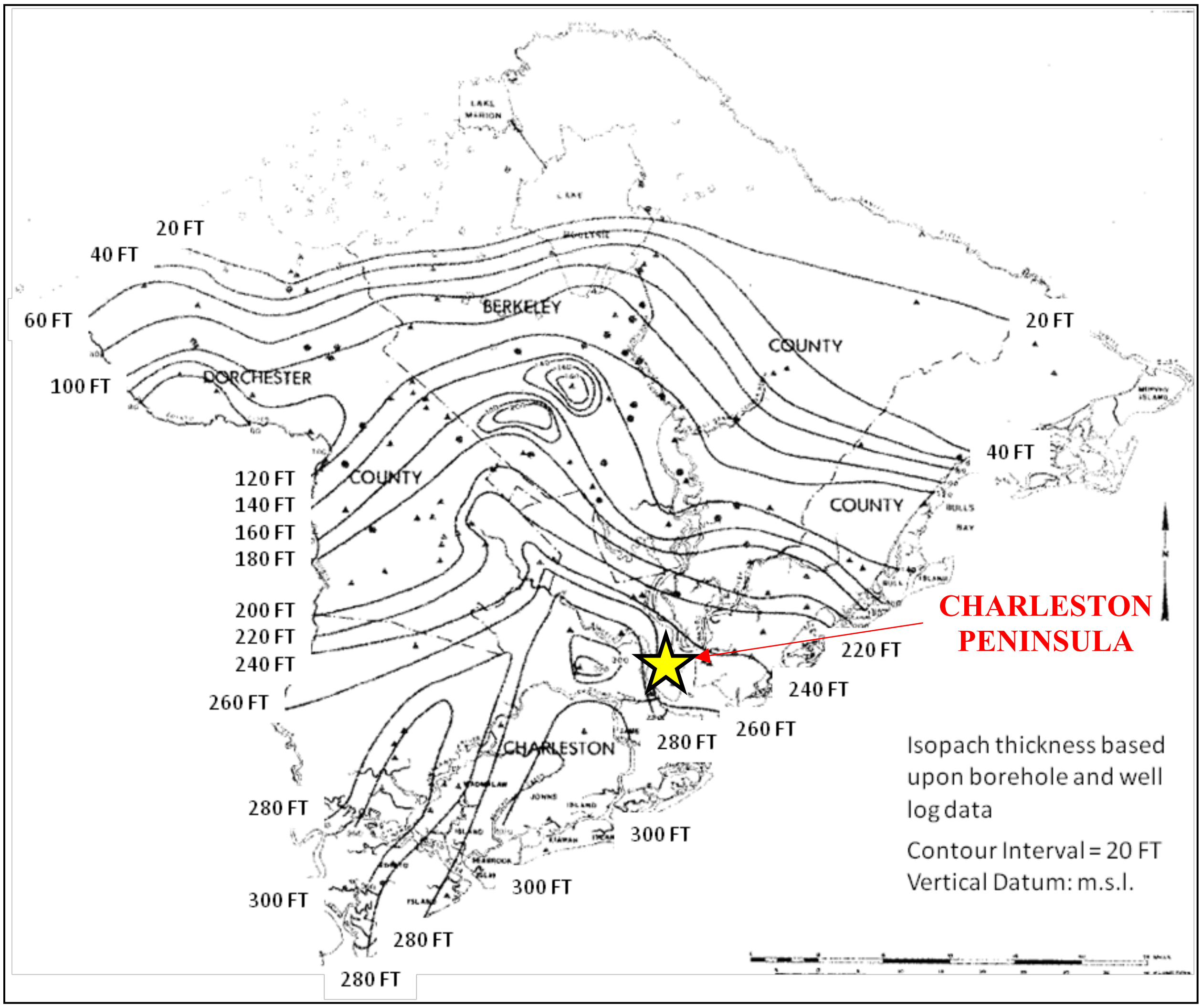

Figure 5: Isopach map showing thickness of the Cooper Formation, from Park (1985).

South Carolina Department of Natural Resources (SCDNR) describes the unit as a stiff, partially consolidated, calcareous silty-clay (SCDNR, Doars, personal communication, 2012). U.S. Army Corps of Engineers (USACE) drilling logs that penetrate into the Cooper Group describe the soil as a stiff to very stiff or hard, brown to greenish-colored, clayey, inorganic silt to silty clay, which had been classified as (MH, CH, ML, MH-CH, and ML-CL) per ASTM D2487. This material appears to grade into and out of medium dense clayey sand and stiff to hard lean clay. Brainard et al. (2009) states that, historically, tunnel construction in Charleston area was conducted within the Cooper Formation (Cooper Marl) because of the unit's optimal engineering characteristics of low permeability, stiffness, and the relative ease by which it can be excavated. However, several water-bearing sand lenses 30-feet thick have been encountered during tunnel excavation (Brainard et al., 2009).

The Cooper Formation is comprised of at least four major subunits: the Eocene Harleyville and Parkers Ferry Formations, and the upper Oligocene Ashley and Chandler Bridge Formations. Collectively, these units were deposited in shallow to open marine environment 30 to 38 million years ago. The strata range in composition from phosphatic clay, to sandy limestone, to fine-grained, silty-clayey, phosphatic sand (Ward et al., 1979; Weems and Lemon, 1984; Weems and Lemon, 1993). Harris et al. (2005) verified the top of the Cooper Formation at elevation -60 feet MSL by seismic profile in the vicinity of Folly Island (Figure 6).

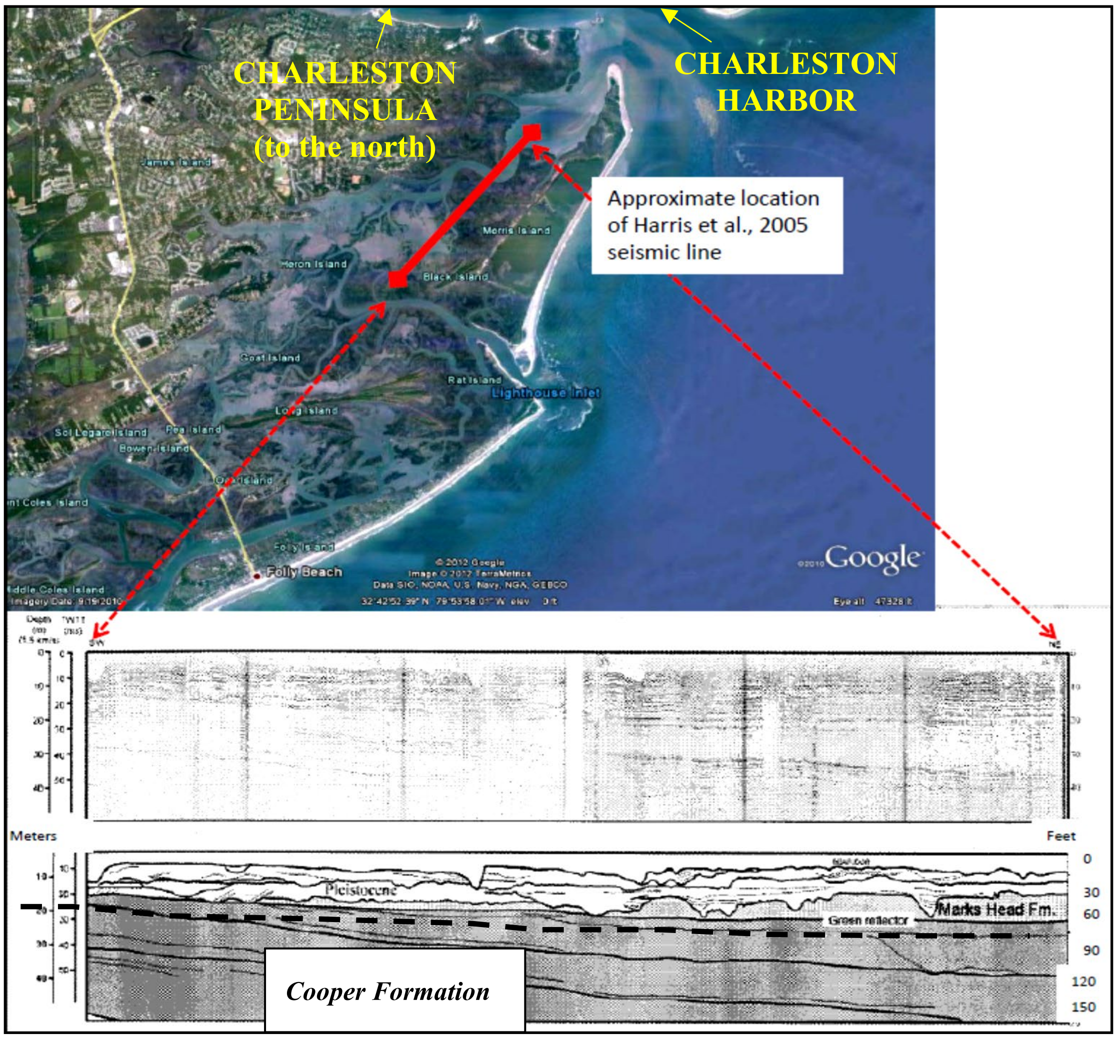

Figure 6: Seismic profile south of Charleston Harbor, from Harris et al. (2005).

3.2.4. Edisto Formation

Ward et al. (1979) applied the name "Edisto Formation" to sandy-shelly limestones of early Miocene age that unconformably overlie the Cooper Formation in southern South Carolina. Weems and Lemon (1993) describe the unit as consisting of light gray, fine-grained, calcareous sand to quartzose calcarenite[1] with locally abundant pelecypod shells. The Edisto Formation is generally composed of detrital, weakly-cemented sand, gravel, and shell hash. The unit was deposited in a shallow marine environment 24 million years ago during the Miocene-Oligocene. Weems and Lemon (1993) report the occurrence of phosphate nodules in land borings, but not in offshore borings. The Edisto Formation unconformably overlies the Cooper Formation within the study area; however, the stratigraphic contact was not observed in drill core and the thickness of the unit is unknown.

3.2.5. Marks Head Formation

The Marks Head Formation is described as fine-grained, quartz-phosphate sand that is Miocene-aged. The unit is known to lie unconformably atop the Cooper Formation and was deposited in shallow-brackish water conditions. Weems and Lemon (1993) indicate that the unit is discontinuous and only occurs in the near subsurface northeast of Charleston, beneath Mount Pleasant and Sullivan Island. South of Charleston, the unit is present from -30 to -60 feet MSL and is no more than 30-feet thick (Harris et al., 2005). The Marks Head Formation dips into the subsurface south and east from surficial outcroppings north of Charleston (Weems and Lewis, 2002). The base of the unit is present at elevations -20 to -80 feet MSL near Charleston Harbor. The shallowest occurrence of this stratum is likely to occur within the Ashley River near Duck Island and north of the confluence of the Cooper and Wando Rivers.

3.2.6. Undifferentiated Quaternary Units

Nearly all of the surficial deposits in the Charleston area are Quaternary in age, and they unconformably overlie the Tertiary strata. These sediments were deposited during sea-level fluctuations caused by multiple interglacial cycles throughout the Pleistocene. Based upon the presence of Pleistocene-aged terrace deposits and erosional shoreline escarpments, at least five different sea-level stands are recognized near Charleston. These geomorphologic features lie as far as 45-miles inland and mimic the shape of the modern coastline (Weems and Lemon, 1993; Harris et al., 2005). The Quaternary age strata generally consist of interbedded sequences of clay, clayey to clean quartz sand, and fossiliferous sand that may be capped by peat, clean sand, or tidal marsh deposits (Weems and Lemon, 1993).

---

[1] Calcarenite is a type of limestone that is composed predominantly (> 50 percent) of detrital (transported) sand-size (0.0625 to 2 mm in diameter), carbonate grains. This material is derived from corals, shells, fragments of older limestones, and other carbonate clasts. Calcarenite is the carbonate equivalent of a sandstone. They can consist of grains of carbonate that have accumulated either as coastal sand dunes (eolianites), beaches, offshore bars and shoals, turbidites, or other depositional settings. Reference: http://en.wikipedia.org/wiki/Calcarenite

4. SUBSIDENCE

4.1. General

Subsidence can be thought of as sinking of the ground surface. There are three main causes of subsidence which are crustal deformation, groundwater extraction, and compaction/compression of the soil.

4.1.1. Crustal Deformation

Crustal deformation is related to glaciation. During the last Ice Age, the mantle beneath the glacier was compressed which in turn caused the mantle beyond the edge of the glacier to rise. Once the glacier receded, the compressed mantle begins to rise while the edge settles. The settling of the mantle overtime causes subsidence. This is depicted in Figure 7 below.

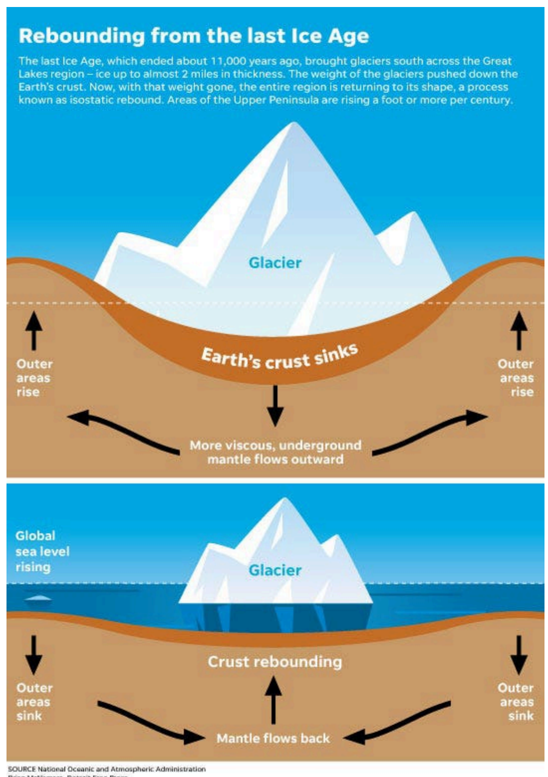

Figure 7: Crustal Deformation Caused be Glaciation (Matheny, Keith Source: National Oceanic and Atmospheric Administration, McNamara)

4.1.2. Groundwater Extraction

When excess amounts of groundwater is removed from the soil, lowering the groundwater table, the effective stress in the soil increases and causes soil particles to rearrange into a more compact state, reducing the volume of the soil, which is commonly referred to as consolidation settlement. On a large, regional scale, this consolidation settlement causes subsidence.

4.1.3. Compaction/Compression

Compaction/compression of the soil is a form of consolidation settlement as indicated above but is caused by additional loads or stresses placed on the soil, not lowering the groundwater table.

With that additional load, the soil particles rearrange into a more compact state, causing subsidence.

4.2. Cause of Subsidence in Charleston Area

Various research has indicated that subsidence in the Charleston Area is not being caused by crustal deformation or groundwater extraction. The past glaciation did not advance far enough towards Charleston to influence the mantle. Additionally, groundwater extraction in the region isn't great enough to lower the groundwater table.

Given this, subsidence has to be attributed to the compaction/compression of the surrounding soils. It is known that many low areas were filled in to extend the Charleston Peninsula out to the current shoreline. This fill in the low areas are likely attributing to compaction of the soils beneath it.

4.3. Rate of Subsidence

Multiple articles have indicated that over the last 100-years, subsidence has attributed around 5 inches, or 40%, of the 12 inches of sea level raise. Additionally, is it thought that the rate of subsidence will remain constant.

5. GROUNDWATER

Groundwater levels are relatively shallow within the Charleston Peninsula and will fluctuate with the tides, seasons, and precipitation. The CSRM features will be located along the exterior of the peninsula and the groundwater levels will be highly dependent on the tides. It should be anticipated that the groundwater table would be encountered at or near the elevation of the tide elevation. This relatively shallow groundwater table will likely require some dewatering during construction of the T-wall foundations. Steel and concrete elements will need to consider this in respect to corrosion.

6. SEISMICITY

The Charleston Peninsula is located in a "hot spot" of high seismic activity and is deemed to be within a high seismic hazard zone as indicated in Figure 8. This area is known as the Middleton Place-Summerville Seismic Zone or the Charleston Seismic Zone. Additionally, Charleston, SC is also the site of the largest earthquake known to have occurred in the southeastern United States, which occurred in 1886.

A seismic evaluation was completed as part of the feasibility study and the details are presented in ATTACHMENT 1.

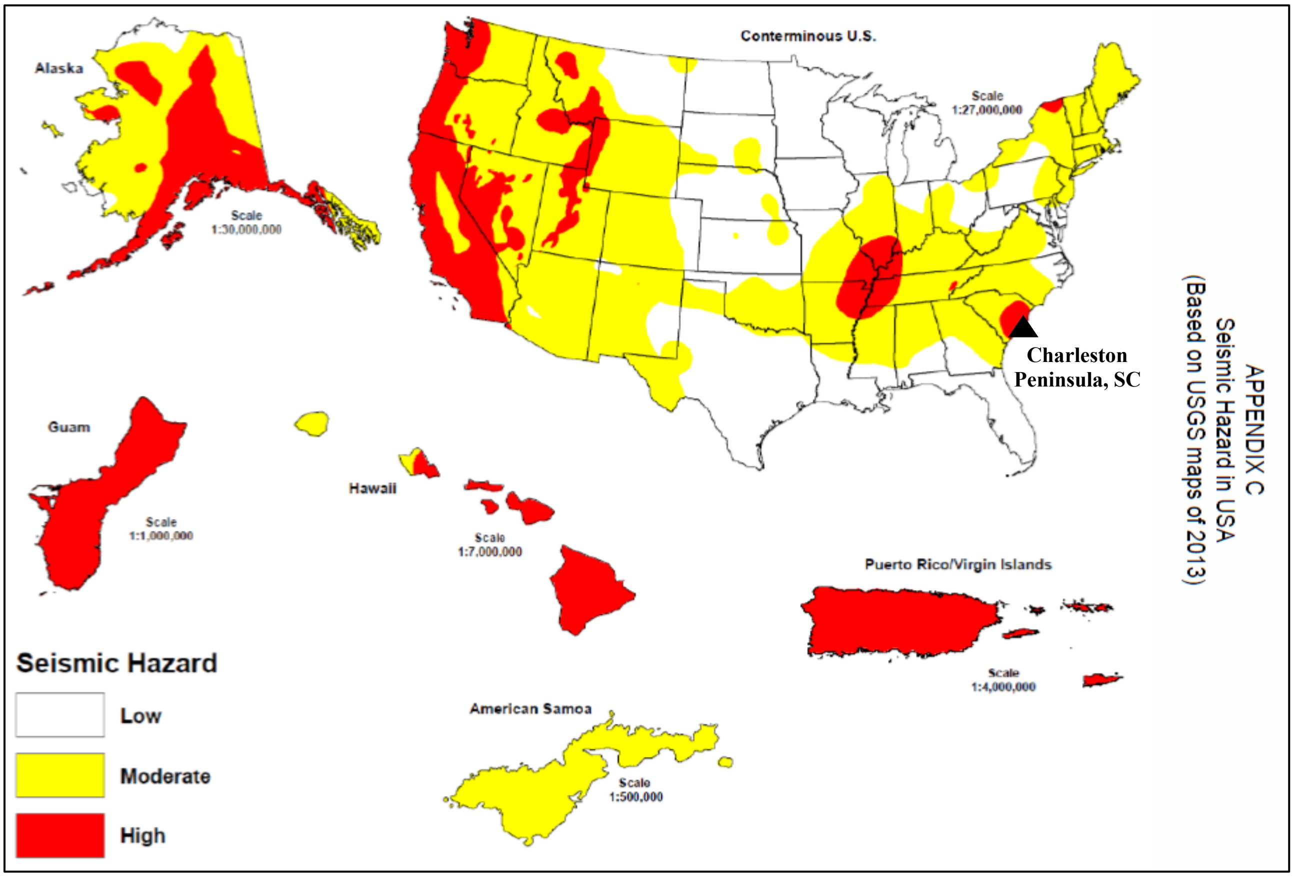

Figure 8: Project location shown on seismic hazard map of the USA, from ER-1110-2-1806.

6.1. Ground Motions

The seismic evaluate provided a range of ground motions for various events. An earthquake with a 2% probability of exceedance in 50 years could produce a PGA that ranges from 0.6 to 0.8g near the Charleston Peninsula [USGS 2014 seismic hazard map by Petersen et al. (2015)], shown in Figure 9. The site-predicted PGA for an earthquake having a return period of 2,475 years is approximately 0.973g, which is slightly higher than the USGS seismic hazard map shown in Figure 9. Spectral ground motion on the Charleston Peninsula was also predicted by the Uniform Hazard Response Spectrum (Figure 10). Based upon probabilistic hazard mapping, the PGA at the site is predicted to be 0.8561g, but the largest and most likely damaging ground motion is 1.3972g at a spectral period of 0.2 seconds (Figure 10).

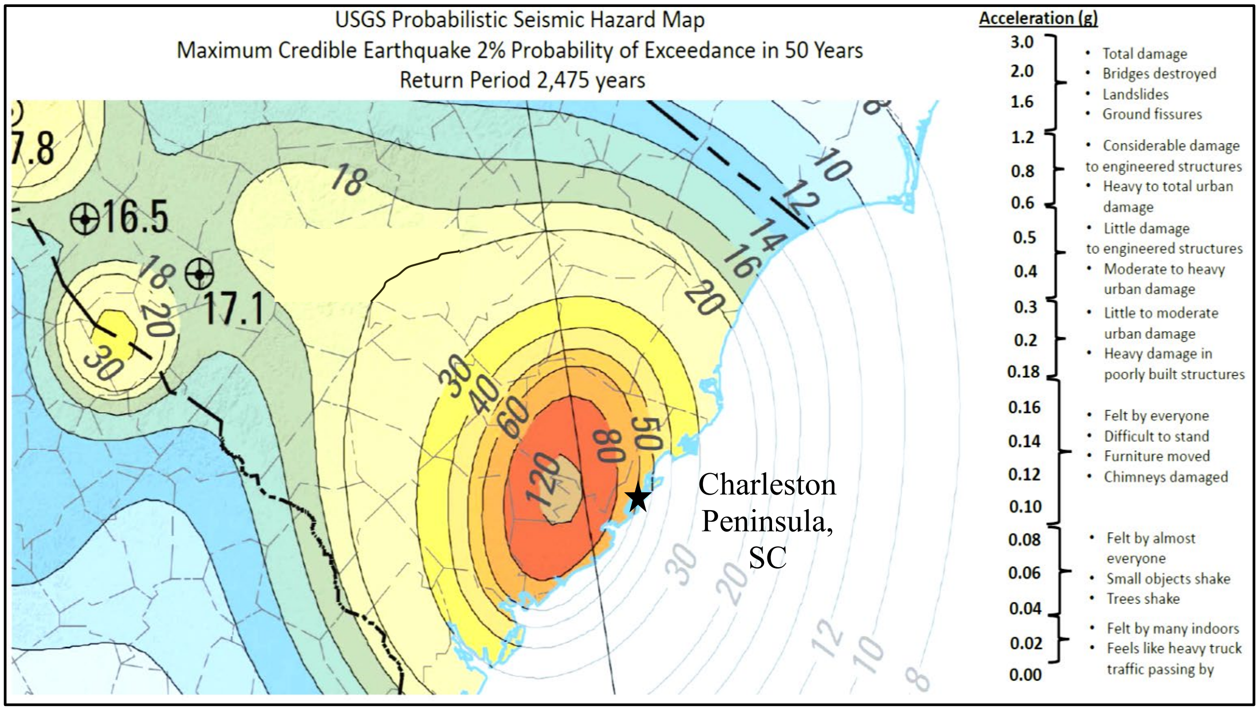

Figure 9: USGS Seismic Hazard Map, PGA, 2% Probability of Exceedance in 50 Years, from Peterson et al. (2015).

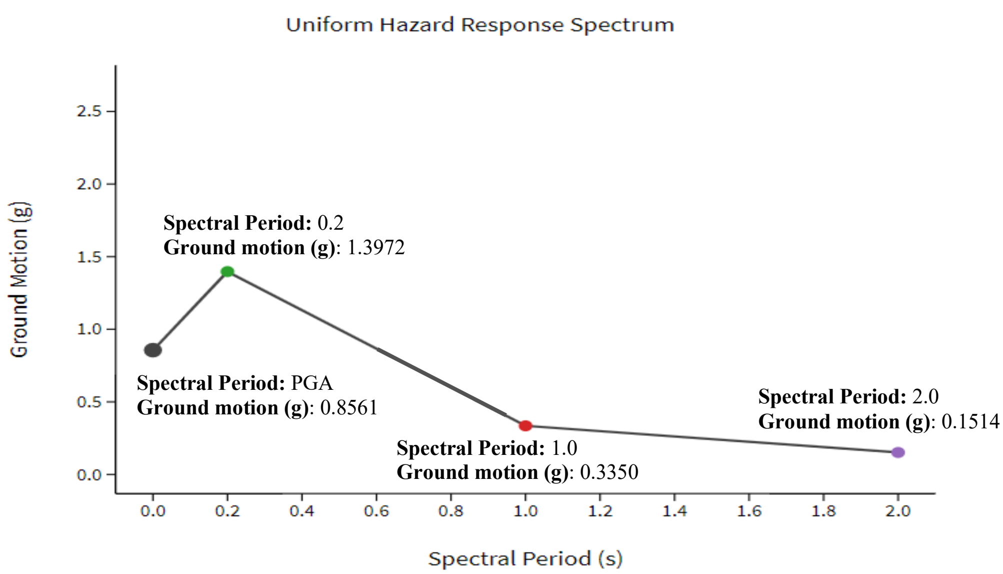

Figure 10: Uniform Hazard Response Spectrum predicted for the project site showing PGA with 2% in 50 years AEP (2,475 return period).

6.2. Maximum Credible Earthquake and an Operating Basis Earthquake

The Maximum Credible Earthquake (MCE) were deterministically derived. The MCE was determined to be an Mw = 7.3 and based upon the 1886 Charleston Earthquake event. The distance from the project site to the center of the MCE source zone is 10.00 km.

The Operating Basis Earthquake (OBE) was assessed using probabilistic methods that are informed by deterministic methods. An OBE PGA of 0.0548g and a SA of 0.09g (at 0.2 second period) is derived utilizing the USGS Unified Hazard Tool.

7. EXISTING FLOOD RISK MANAGEMENT STRUCTURES

The City of Charleston has two floodwalls, the Low Battery and High Battery Walls, which are located on the west and south sides of the peninsula, respectively. Both the Low and High Battery Walls have had their conditions assessed and recommendation for improvements developed. At the time of the feasibility study, the City of Charleston was pursuing a project to modify the Low Battery Wall and raise it to EL. 9 feet, which is the height of the High Battery Wall.

Due to the age and condition of the High Battery Wall, it was assumed that it would not meet USACE standards for design and performance and therefore a new floodwall would be required to be constructed as part of the CSRM project.

8. STUDY STRUCTURAL MEASURES

8.1. General

There were various structural measures considered during the CSRM study that fall into two main categories, levees and floodwalls.

8.2. Levees

Levees, including road raises and other earthen berms, were initially discussed as potential features but were not carried forward due to the larger footprint required by these features over that of floodwalls. Additionally, road raises create issues as ramps would be required to maintain access to connecting streets. The footprint of the access ramps would have an impact on adjacent properties.

8.3. Floodwalls

Multiple types were considered that included I-wall, double row sheetpile, combo wall, and concrete T-wall.

I-walls were initially considered in areas with exposed stem heights less than 4 to 6 feet. But due to the soft condition of the soils along with lack of any specific geotechnical data, I-walls were not carried forward for the feasibility study and replaced with a pile-founded concrete T-wall.

A double row sheetpile wall that was tied together was also initially considered along reaches adjacent to and within the tidal marsh area. This concept included placing fill within the double row sheetpile wall to allow for a walking path on top of the structure. The double row sheetpile was not carried forward as placement of fill material would be problematic due to the unconsolidated nature of the tidal marsh material and the compressibility of the foundation material which could lead to excess settlement and drawdrag on the sheetpiles.

T-walls and combo types were selected and used in the feasibility project. The T-Wall concept used was based on a typical design that included piles (both vertical and battered) for structural support and a sheetpile cutoff as part of seepage mitigation. T-walls were placed in locations where the alignment was on ground.

The combo wall concept is a wall comprised of large circular piles (also known as king piles) and batter piles, the later to provide additional lateral resistance. Sheetpile placed in-between the circular piles to provide a continuous structural wall. These components are tied together with a concrete cap. The combo walls were placed in the tidal marsh areas.

9. GEOTECHNICAL ASPECTS OF FEASIBILITY STUDY MEASURES

The geotechnical aspects of the various feasibility study measures are discussed below. Due to the study area size, schedule and funding constraints, the geotechnical design is conceptual. It was developed based on assumptions made using information found within other CSRM project studies (Norfolk, Virginia and Galveston, Texas) and local geotechnical reports, engineering judgment, and some analyses (preliminary pile capacity estimates and seepage analyses). The geotechnical design is at a 10% conceptual level. Discussion are included on what future work is required during the Preconstruction Engineering and Design (PED) phase.

From the perspective of selecting the NED plan, one alignment was looked at with various top of wall elevations. The same foundation conditions are being used/assumed for the various alternatives as they are all in the same location. Of importance for costs is the selection of the foundation type for the walls. It has been assumed that all walls will be pile-founded due to the poor quality of the foundation materials above the Cooper Marl. The variation in the foundation materials above the Cooper Marl are not expected to drastically affect the design of the walls and similarly the cost. If the alignment was changing for the various alternatives, determining if there was a major change in foundation conditions would be warranted.

The top of Cooper Marl has been estimated using existing data received from various companies and other published documents. Using this data and conservative assumptions, various top of Cooper Marl elevations were established for the study, being broken into 5-foot intervals to account for variability and uncertainty.

9.1. T-Wall

The T-wall will be pile founded using both vertical and battered piles. A steel sheetpile cutoff will was assumed to be installed to reduce underseepage and uplift on wall. It was assumed that the sheetpile would be 15 feet long for the EL. 7 and EL. 9 walls and 20 feet long for the EL. 12 wall. These assumptions were based on the Norfolk Study's depth. It was assumed that longer sheetpile would be required for the higher wall to EL. 12 and sheetpile was extended an additional 5 feet. Seepage analyses were completed to verify this assumption and described below.

A figure depicting the T-Wall concept can be found in Engineering Sub-Appendix 6. Selected Plan Drawings.

9.1.1. Seepage Analysis for T-wall

Steady state seepage analyses were conducted in the finite element software SEEP/W Version 10.0.0.17401, by GeoStudio (2019). All materials were extended to 450 feet to the landward side and 1100 feet to the river side of the seawall. The global mesh size was set at 10 feet, however, this was altered allow for more detailed computations around the wall and is increasingly coarse with distance from the wall. The region that extends to 50 feet on either side of the wall is set to have a mesh size of 1/10th of the global mesh, or 1 foot. The mesh size for the regions on both sides of the wall from 50 feet to 100 feet is set to 1/4th of the global mesh, or 2.5 feet. For the region between 100 feet to 450 feet on the landward side and 100 feet to 300 feet on the seaward side, the mesh size is set to ½ of the global mesh size. The mesh size for the region located on the river side between 300 feet and 1100 feet was set at the global mesh size.

A total head boundary condition of elevation 12 feet was applied to the river side of the model based on the top of wall design elevation. This was assumed to be the worst-case scenario and, therefore, was the only water level used in analysis. On the land side of the wall, a water rate boundary was applied with a flow rate of 0 cubic feet per second and the potential seepage face review option applied.

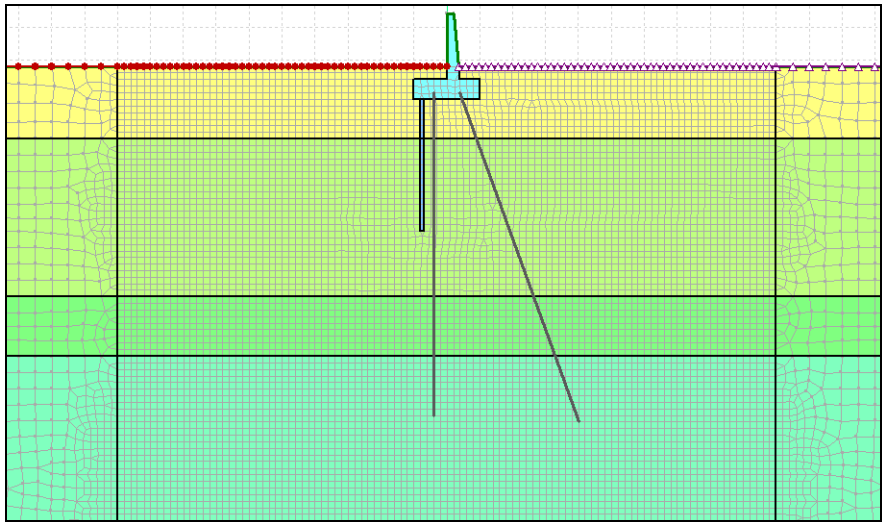

Figure 11: Example of Seep/W Model, Configuration and Mesh

Feasibility level seepage analyses were completed to evaluate the effectiveness of the sheet pile cutoff wall, placed 1-foot from the river side edge of the foundation, and to estimate values of the uplift pressure distribution on the foundation. Without site-specific data, two stratigraphic scenarios were analyzed to determine potential uplift pressures and exit gradients on the land side of the wall; one based on the "average" layer thicknesses and one based on a "lower" elevation of the Cooper Marl and maximum strata thicknesses. The layers modeled are labeled, from the ground surface to depth, as: Upper Sand, Marsh, Lower Sand, and the Cooper Marl. The average, minimum, and maximum thickness of each layer observed in the available geotechnical data was tabulated. The Upper Sand ranges from 11 feet to 17 feet thick, the Marsh materials ranges from 23 feet to 42 feet thick, the Lower Sand ranges from 9 feet to 10 feet thick. Using average layer thicknesses, the elevation of the top of the Cooper Marl became EL. -40. For the "Lower" model, the elevation of the top of the Cooper Marl was assumed EL. -65. The ground surface was assumed to be at EL. 4 feet and the base of the T-wall slab at EL. -1 feet.

Table 1: Summary of Stratigraphic Scenarios for T-Wall Seepage Analyses

Model	Thickness			Elevations			
	U Sand	Marsh/Muck	L Sand	Ground Surface	Upper Sand / Marsh/Muck Contact	Marsh/Muck / Lower Sand Contact	L Sand / Cooper Marl Contact
1 Ave	11	24	9	4	-7	-31	-40
2 Lower	17	42	10	4	-13	-55	-65

The permeability of each stratum is based on typical published values for the material type with some consideration given to the density of the deposit. All materials were modeled as saturated;

no attempt was made to account for changes in permeability with partial saturation. Values used in the analyses are: 5 x 10^{-2} cm/s for the upper sand unit, 1 x 10^{-3} cm/s for the marsh unit, 1 x 10^{-2} cm/s for the lower sand unit, and 1 x 10^{-5} cm/s for the Cooper Marl. The sheet pile wall and the concrete seawall were modeled with a permeability of 3 x 10^{-11} cm/s.

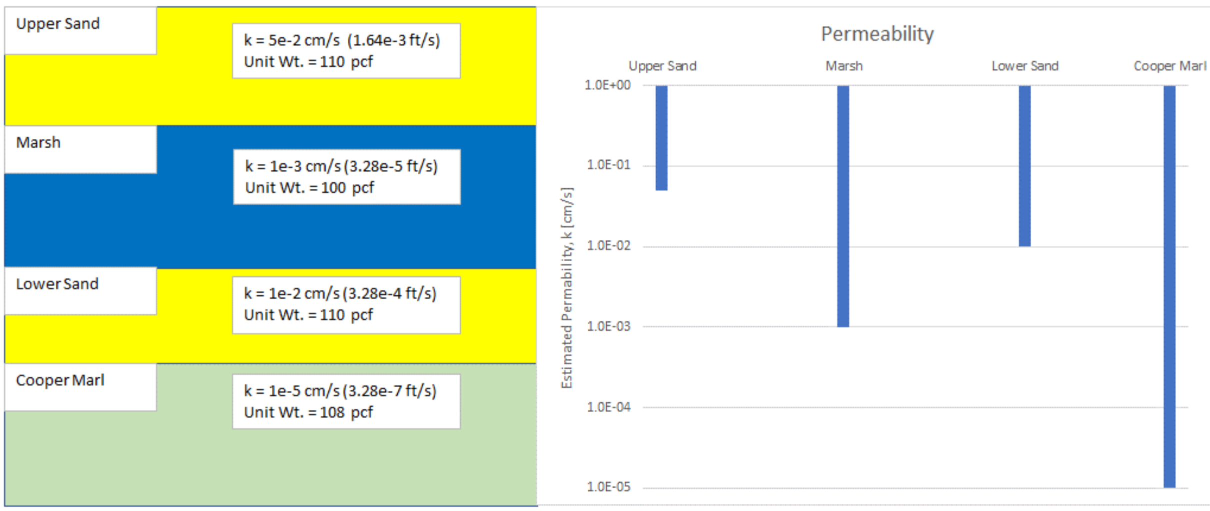

Figure 12. Permeability and Stratigraphy Model used in Analyses.

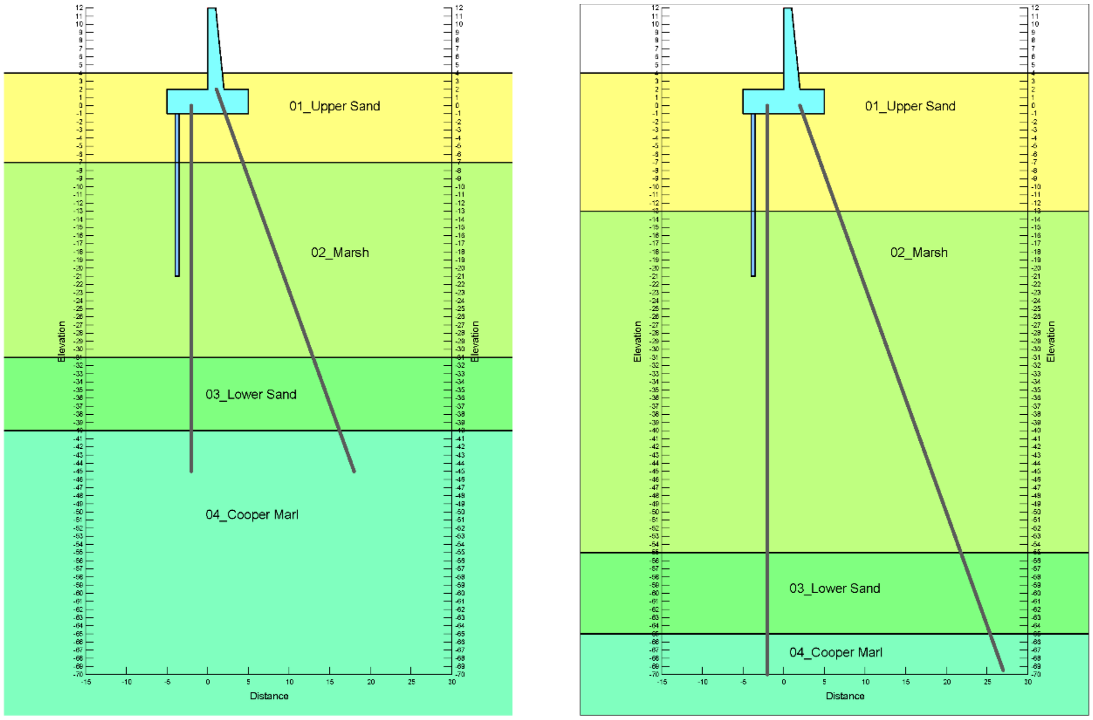

Figure 13. Seepage Model Scenarios. “Average” Thickness on left; “Lower” Copper Marl on Right

For the two scenarios analyzed, the potential for seepage induced quick condition or uplift on the land side heel of the T-wall was evaluated based on the methods described in Taylor (1948) and in Holtz, Kovacs, and Sheahan (2011). The methods used are based on the total boundary pore water pressures vs. total stress applied to the boundary and comparison of the measured exit gradient to the theoretical critical gradient. A summary of the calculations is provided below.

For the total boundary pore pressure vs. total stress on the boundary:

$$\boldsymbol{F_{water\uparrow} = u \times \Upsilon_w \times A}$$

$$\boldsymbol{F_{soil+water\downarrow} = (\Upsilon_{sat} \times T \times A) + (\Upsilon_w \times h_{water} \times A)}$$

Where:

$\mathbf{F}_{water\uparrow}$ = the resultant seepage force on the boundary

$\mathbf{F}_{soil\ +\ water\downarrow}$ = the total stress on the boundary

γ_w = unit weight of water

γ_{sat} = the saturated unit weight of the soil {assumed to be 110 pcf}

T = the thickness of the soil layer above the boundary

h_{water} = height of water above the ground surface

u = the pore water pressure at the boundary

A = the area acted upon {1 sq. ft.}

For the exit gradient method, when a quick condition, or boiling, is just possible:

$$(h + L)\gamma_w A = \frac{(G+1)}{(1+e)}\gamma_w LA$$

Where:

G = the specific gravity of the soil grains, *e* is the void ratio of the soil

h + L = the total head at a point

L = the thickness of the soil above the point

and all other variables are as previously defined. This equation can be simplified to the critical gradient:

$$\boldsymbol{i_c} = \frac{\Delta \boldsymbol{h}}{\Delta \boldsymbol{L}} = \frac{\boldsymbol{\Upsilon'}}{\boldsymbol{\Upsilon_w}}$$

the gradient obtained from the model is:

$$\boldsymbol{i} = \frac{\Delta \boldsymbol{h}}{\Delta \boldsymbol{L}}$$

Where:

Δh / ΔL = the change water head between a point and the ground surface over the distance between the point and the ground surface.

The factor of safety is determined based on the capacity demand model for each as:

$$FOS = \frac{Capacity}{Demand}$$

The critical exit gradient calculated for the analysis is 0.77. The exit gradient determined from the model was small, approximately 0.004 to 0.005, and the resulting factor of safety is 152. When evaluated using the total boundary forces, the resulting factor of safety is 1.75.

The uplift pressures for both stratigraphic scenarios were the same and are indicated below in Figure 14. An example of the Seep/W model output is shown below in Figure 15.

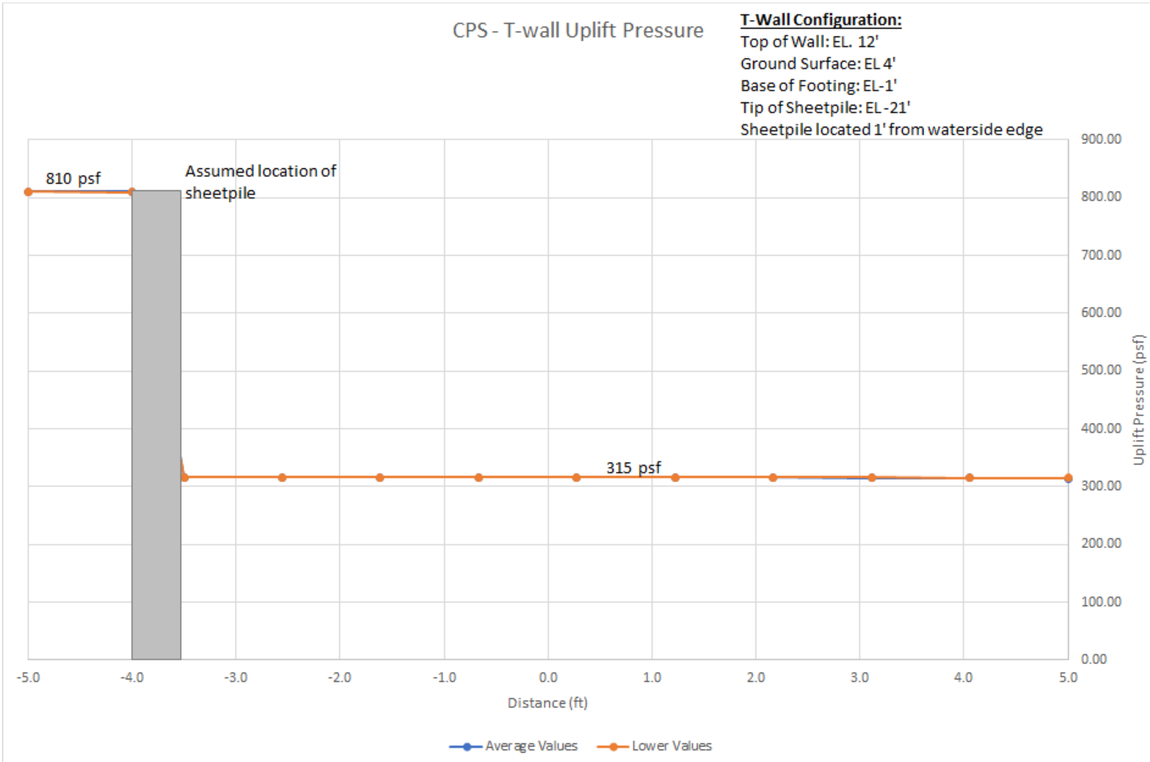

Figure 14: Uplift Pressures on T-Wall

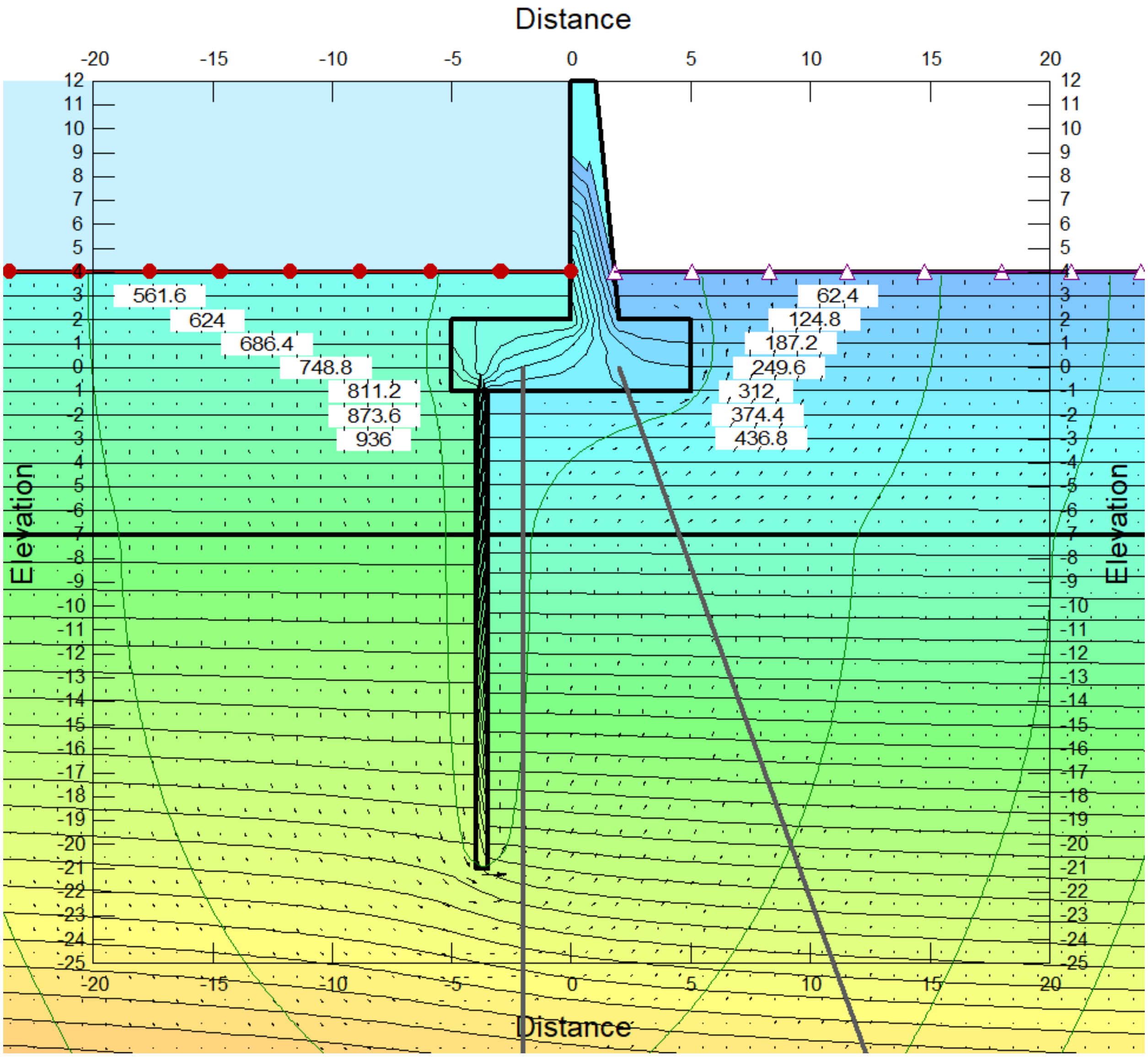

Figure 15. Seepage Analysis Results for "Average Model"

9.2. Combo Wall

The king piles and battered piles for the Combo wall will be founded within the Cooper Marl formation. The steel sheetpile between the king piles will be installed to reduce underseepage. It was assumed that the sheetpile would be 35 feet long for the EL. 7 and EL. 9 walls and 40 feet long for the EL. 12 wall. The sheetpile lengths for the Combo wall were adjusted such that the seepage path length for the Combo wall was equivalent to that of the T-Wall. A figure depicting the Combo wall concept can be found in Engineering Sub-Appendix 6. Selected Plan Drawings.

9.3. Piles

Many structures on the peninsula are founded on piles. Review of various engineering reports received, the typical type was either steel H-piles or square, pre-stressed concrete piles, either 12" or 14" in size. These piles are driven to bear within the Cooper Marl formation. For the feasibility study, it was assumed that steel H-piles would be used and the embedment depth was 5 feet in the Copper Marl.

The top of Cooper Marl varies across the peninsula. Using existing subsurface information obtained during the study, estimates of the top of Cooper Marl elevations were made. It was estimated that top of Cooper Marl ranges from EL. -55 feet to EL. -75 feet. The top of Cooper Marl within the various reaches/segments around the peninsula is presented below in Figure 16. Additional maps can be found in Attachment 2.

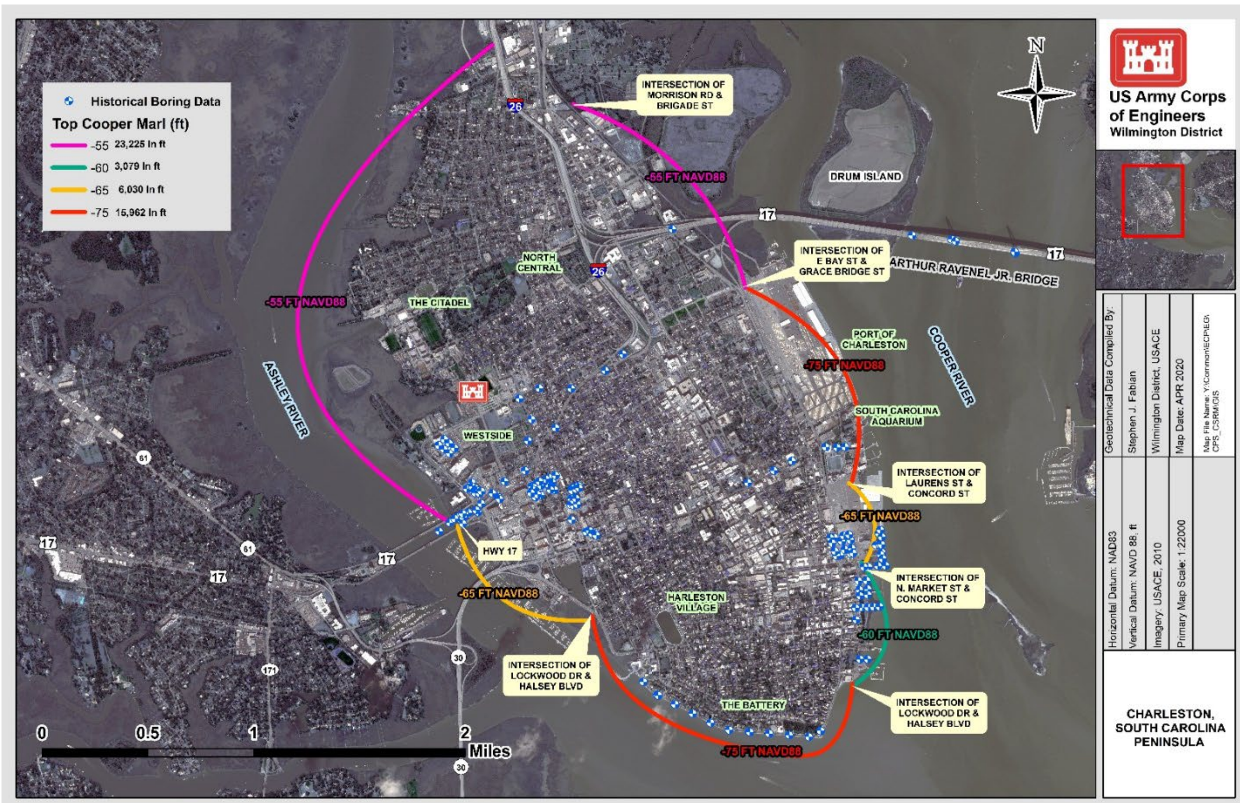

Figure 16: Top of Cooper Marl Within Various Reaches/Segments around the Charleston Peninsula

9.3.1. Determination of Top of Cooper Marl

The top of Cooper Marl was defined by taking a compilation of scientific literature and collection of over 200 SPTs and CPTs from 1977 to 2018. This geotechnical data was taken by several contractors and were not USACE affiliated projects. Prior to depicting the top of Cooper Marl, SPTs and CPTs were loaded into ArcGIS via "Go To XY" feature to locate each SPT and CPT. Following the input of each geotechnical point, a field was created in the attribute table called: "Top of Cooper Marl." The CPT data looked for differences in resistivity along stratigraphic boundaries indicating high plasticity silt and/or clay. Most of the CPTs taken were followed by a SPT in the same CPT location in order to ground truth the CPT collected. Majority of the SPTs drilled noted in the core descriptions specifically the top of the Cooper Marl (Figure 17). This was the primary way to delineate the top of Cooper Marl across the peninsula. Each elevation was inputted into the attribute table in ArcMap. Once this was done, the labels were turned "ON" and the elevations shown were used to mark the top of Cooper Marl in 5-foot intervals around the peninsula (Figure 16).

Moreover, in Figure 1, the Cooper Marl is shown dipping to the south with the top of the Copper Marl at EL. -60 feet underneath the Charleston Peninsula. The SPT data across the peninsula shows the top of Copper Marl ranging from EL. -55 to -75 feet with the top of the formation dipping from north to south. This coincides with Figure 1 showing this behavior. In addition, Figure 17, shows the southerly dipping trend in the top of the Copper Marl. The Port of Charleston which is located north of "The Battery" has the top of the Cooper Marl shallower than "The Battery." This is seen throughout the peninsula.

There is variability in the top of Cooper Marl but with combining the literature review and geotechnical data there is high confidence that the top of the Cooper Marl ranges from EL. -55 to EL. -75 feet across the peninsula. To account for variability and uncertainty, the top of Cooper Marl was grouped into reaches, each reach covering a 5-foot interval. The reaches were designated based on the lowest top of Cooper Marl in the group, meaning other exploration data indicated top of Cooper Marl between 1 to 4 feet higher. However, because of the data gaps along the outer edges of the peninsula the top of the Cooper Marl ranges drastically from one area to another. In order to achieve a better understanding of the in-situ soil conditions, additional exploratory SPTs and CPTs would need to be performed to delineate the top of Copper Marl more accurately.

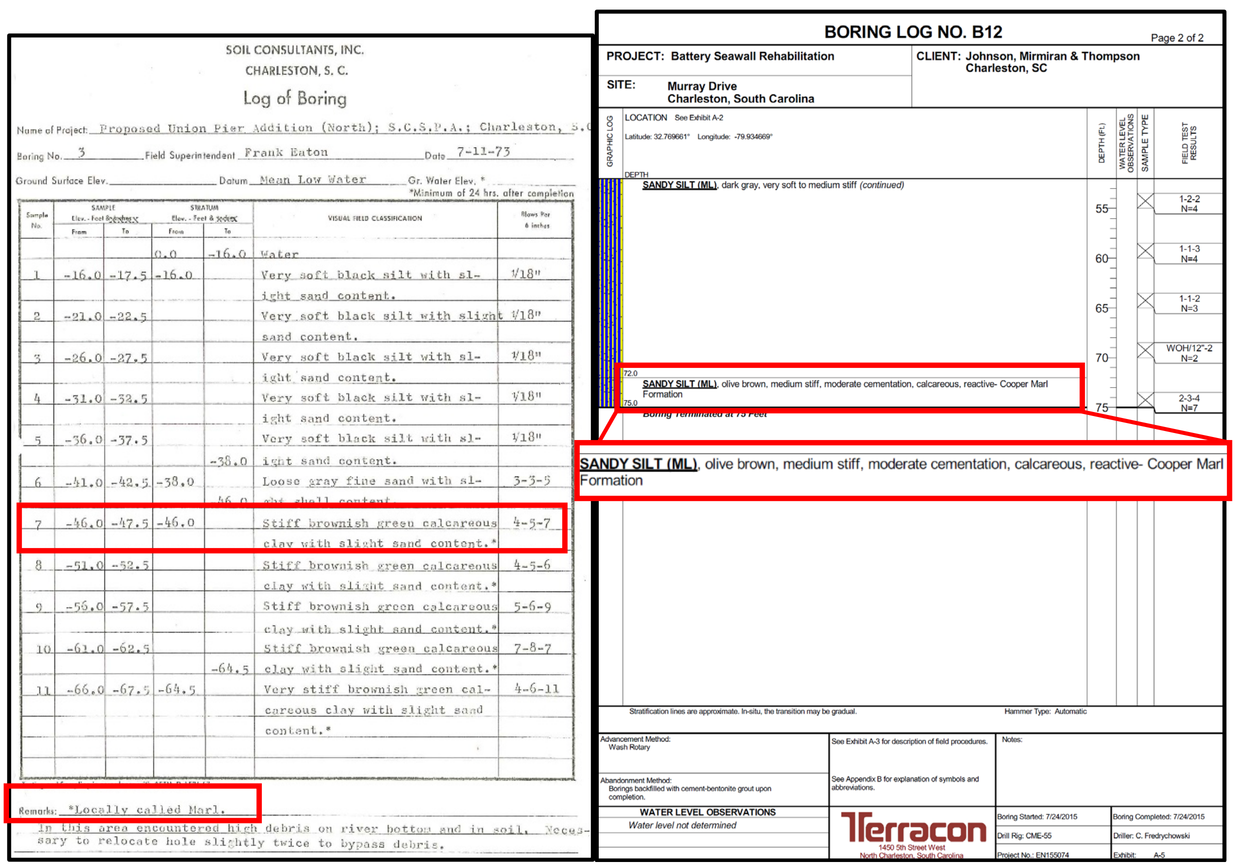

Figure 17. Two SPT logs from the Charleston Peninsula.

The log on the left was taken north of "The Battery" at the Port of Charleston while the log on the right was taken from 2015 along "The Battery." Refer to Figure 16 for locations.

9.3.2. Dense Sand / Gravel

For the feasibility study it was assumed that steel H-piles would be generally be used but further evaluation on most cost effective pile will be done during PED. It should be noted that various engineering reports received reported that there can be a dense sand/gravel layer above the Cooper Marl that can make it difficult to drive concrete piles, or any displacement type pile, through it. Additional investigation will be required during PED to determine if/where there are dense sand/gravel layers along the alignment that would preclude the use of displacement piles in certain reaches/segments of the project.

9.3.3. Pile Capacity Estimate

A preliminary analysis of the pile capacity was conducted using the soil profile from the seepage analysis associated with the top of Cooper Marl at EL. -65 feet. Capacities were calculated for both a 12-inch and 18-inch square prestressed concrete (PSC) piles. H-piles with a similar "box" perimeters as the concrete piles are expected to have similar overall axial capacities. Analyses for the h-piles were not performed.

The majority of the materials found in the foundation are fine grained and the Alpha method was used to determine the capacity per foot of depth of the piles. A spreadsheet was developed to calculate pile capacity. The spreadsheet was compared to local geotechnical reports received to verify that the methodology provided similar results to local experience. The strength of the foundation materials used in pile capacity estimating along with the elevations of the formations for both the "average" thickness profile and lowest Copper Marl are indicated below in Table 2. Please refer back to Figure 13 to see soil profile. Lateral loading and the resulting deflections were not considered. The pile capacity estimates are shown below and indicate that frictional resistance makes up the majority of the total capacity. The allowable pile capacities are base on a factor of safety of 2 which assume that static and dynamic pile load testing will be completed as part of the design and construction process.

Table 2: Soil Strength and Profile for Pile Capacity Estimates

Formation	***Su (psf)***	***Average Profile Elevation***	***Lower Profile Elevation***
Ground Surface		4	4
Upper Sand	300	4 to -7	4 to -13
Marsh/Muck	200	-7 to -31	-13 to -55
Sand	500	-31 to -40	-55 to -65
Silty Sand/Marl	2600	-40	-65

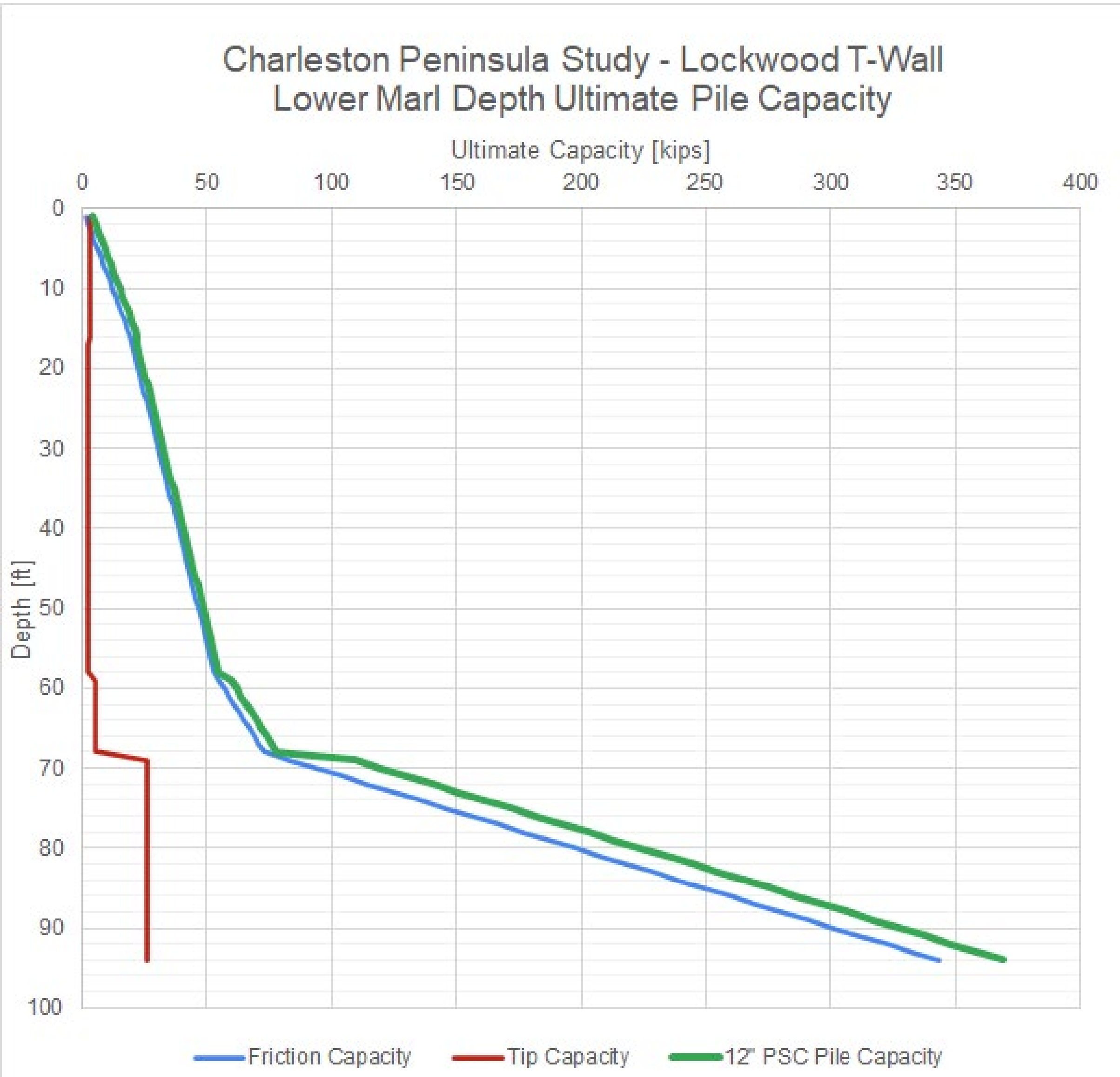

Figure 18. 12” Prestress Concrete Pile Ultimate Capacity Estimate for Cooper Marl at EL. -65 feet.

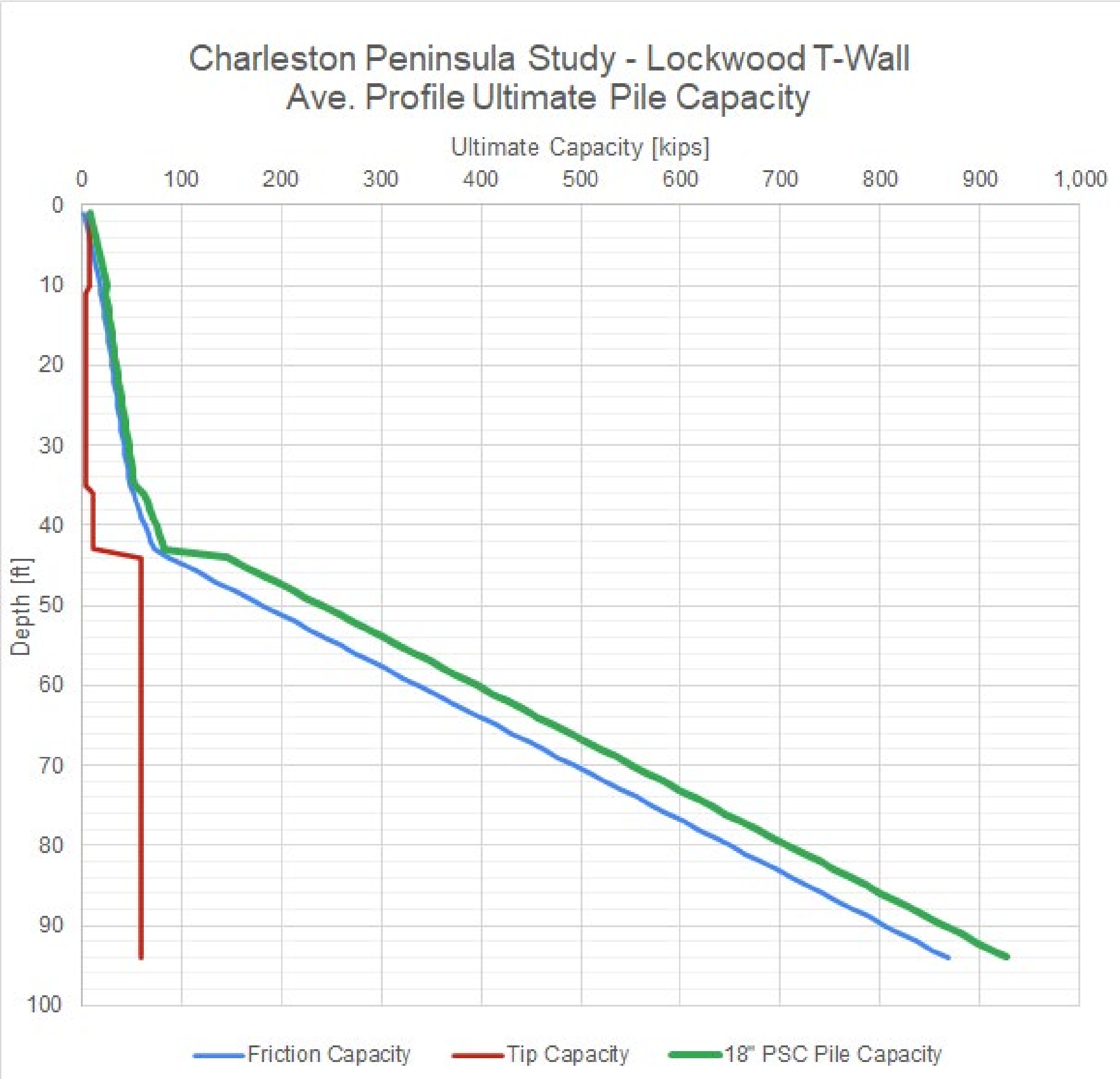

Figure 19. 18” Prestressed Concrete Pile Capacity Estimate for Cooper Marl at EL. -65 feet.

9.3.4. Vibrations During Pile Driving

Vibrations during pile driving is a concern as there will be many structures located adjacent to the CSRM project. Some of these structures have historical significance. There are methods to estimate distances but is dependent on soil stratigraphy, which detailed stratigraphy is unknown at this time. A general rule of thumb is that vibration damage is not likely to occur outside of 50 feet from the pile (either top or tip of pile, whichever is closer) for piles 50 feet or less in lengths or the length of the pile. With piles lengths approaching 90 feet and some piles being battered, preconstruction survey on properties within a 100-ft buffer from wall centerline was assumed. Additionally, vibration monitoring will be required during construction as various locations throughout the area but not at each residential structure.

9.4. Structural Steel Elements

Structural steel elements that are exposed to air and salt water will require cathodic protection.

9.5. Future Work Required during Design Pre-construction Engineering and Design Phase

Due to the study area size, schedule and funding constraints, there is much geotechnical analysis and design required during the PED phases. Some of this work, such as subsurface exploration, will need to start immediately at the beginning of PED in order to obtain the necessary information to complete geotechnical and structural analyses. All analyses and designs completed during PED should consider the findings and recommendations of the semi-quantitative risk assessment (SQRA) such that risks are not increased. The work required during PED is discussed in detail below.

9.5.1. Subsurface Exploration

Subsurface information will need to be gathered along the wall alignment and the breakwater alignment, if retained as part of the Recommended Plan. Exploration along the combo-wall alignment will require advance coordination with the Environmental Branch and the agencies as the exploration may impact the marsh. Along with determining stratigraphy along the wall alignment, it will be important to know if there is any man-made fill or construction debris that may affect construction and pile installation. When developing the soil exploration program, the PDT should determine areas where the presence of man-made fills are likely so additional exploration can be completed to define the type and extents of it. Soil exploration should be extended into the Cooper Marl, to a depth of at least 20 feet below the expected pile tip elevation (U.S. Department of Transportation Federal Highway Administration, Design and Construction of Driven Pile Foundations – Volume I, page 87). For the breakwater alignment, the soil exploration should be developed to provide information on the bearing capacity of the foundation.

Soil exploration should consist of CPT soundings supplemented with SPT borings. The SPT borings will be used verify the soil behavior type determined during CPT data reduction. Additionally, undisturbed samples should be collected and tested. The testing should consist of both drained and undrained shear strength determination, consolidation, and soil classification tests (Atterberg limits and grain size distribution). The results of the undisturbed testing can be used to determine the N coefficients that are used in relating cone resistance to undrained shear strength. The spacing between soil exploration will likely range from 250 to 1,000 feet.

If soil-structure interaction modeling will be required, in situ modulus values will need to be determined. Flat plate dilatometer or pressuremeter testing would be required. Additionally, the flat plate dilatometer could also be used to supplement the determination of shear strengths.

A preliminary subsurface investigation plan and estimate was developed. The details of this preliminary plan and estimate are in Attachment 5.

9.5.2. Seepage Analysis for T-wall and Combo-Wall Sections

Seepage analysis will need to be completed to determine the proper depth of seepage cutoff walls and the uplift pressures on the T-wall footing.

9.5.3. Pile Design

The design of the piles will be required. The design will include selection of pile type (steel H-pile, concrete piles, micro piles, etc.) considering costs, drivability, vibration generation, constructability, and longevity (related to corrosion). Determination of both axial and lateral load capacity with consideration of seismic loading will be required along with downdrag calculations, where applicable. Pile load tests (dynamic, static, and lateral) should be conducted during the early stages of PED to evaluate pile types and sizes, drivability, and vibration. The use of pile load tests should also be evaluated to determine the appropriateness of including them during various stages of construction.

Assessment of shear and bending stresses in battered piles causes be settlement and downdrag need to be assessed.

In addition to the typical pile design, pile driving generated vibrations will need to be evaluated. Both magnitude and distance travel will need to be determined. Maximum allowable vibration amplitudes along with construction monitoring requirements will be needed.

9.5.4. Lateral Earth Pressure

It is anticipated in some locations the wall will also act as a retaining wall. Appropriate lateral earth pressures will need to be determined to be used in the design of the retaining wall.

9.5.5. I-Wall Evaluation

There could be a cost savings potential if I-walls can replace T-walls and this should be evaluated along the project alignment where the exposed stem height is 4 feet or less. The PDT will need to realize that the design requirements for an I-wall are more intensive that T-walls and need to be considered this when developing the soil exploration program (smaller spacing) and design schedule. Additionally, the I-wall should be considered a major change in the project and be evaluated by a supplemental SQRA.

9.5.6. Penetrations Through Barrier

Penetrations through the barrier will be necessary for utilities and stormwater drainage. These penetrations will need to be designed.

The PDT should consider determining utility corridors in which multiple utilities can penetrate the barrier in one designated segment. This would minimize the number of crosses.

9.5.7. Operation, Maintenance, Repair, Replacement, and Rehabilitation Manual

An Operation, Maintenance, Repair, Replacement, and Rehabilitation Manual (O&M Manual) will be required. Once a functional portion of the project has been constructed, the Non-Federal Sponsor will be notified and their OMRR&R responsibilities will begin. Geotechnical input to the O&M Manual will be required during PED but mainly during and after construction.

10. CONSTRUCTABILITY

There are various constructability issues that could be encountered for the Charleston CSRM which are indicated and discussed below.

10.1. Pile Installation

Piles will be driven throughout the Peninsula Area, sometime very near to existing structures, with some having historical significance. Pile driving will cause vibrations and pre-construction surveys will need to be completed along with monitoring of vibrations.

If dense sand and gravel layers above the Cooper Marl are encountered during soil exploration, driving of displacement-type piles (i.e. square concrete piles or closed-end pipe piles) could be hampered and would require pre-augering. Additionally, driving displacement-type piles through very dense layers could increase the magnitude of vibrations and distance they travel.

10.2. Soft Soils

If soft soils are present, these could be problematic from the stand point that any additional load on the foundation will cause consolidation and downdrag on piles. Settlement and drawdrag will need to be considered during design.

Soft soils could also cause issues in which the soils cannot support construction equipment, or excess rutting occurs.

10.3. Loose Sands and Adjacent Shallow Foundations

If both loose sands and structures on shallow foundations are present along the alignment, pile driving and excess vibration may cause the loose sands to densify and lead to settlement of the shallow-founded structures. The density of foundation soils and type of structural foundation will need to be evaluated during design.

10.4. Man-Made Fills

Historically, the peninsula was expanded by placement of fill into the low areas around the perimeter. The man-made fills could make pile driving difficult and could require pre-augering.

11. DESIGN GUIDANCE

A list of anticipated design guidance documents that will facilitate design are as follows:

EC 1110-2-6066 Design of I-walls

EC1165-2-217 Review Policy of Civil Works

ECB 2018-15 Technical Lead for E&C Deliverables

ECB 2017-3 Design and Evaluation of I-Walls Including Sheet Pile Walls

EM 1110-1-1804 Geotechnical Investigations

EM 1110-1-1904 Settlement Analysis

EM 1110-1-1905 Bearing Capacity of Soils

EM 1110-2-1901 Seepage Analysis Control and for Dams

EM 1110-2-1902 Slope Stability

EM 1110-2-1906 Laboratory Soil Testing

EM 1110-2-1913 Design and Construction of Levees

EM 1102-2100 Stability Analysis of Concrete Structures

EM 1110-2-2502 Retaining and Flood Walls

EM 1110-2-2504 Design of Sheet Pile Walls

EM 1110-2-2902 Conduits, Culverts, and Pipes

EM 1110-2-2906 Pile Foundations

EM 1110-2-6050 Response Spectra and Seismic Analysis for Concrete Hydraulic Structures

EM 1110-2-6051 Time-History Dynamic Analysis of Concrete Hydraulic Structures

EM 1110-2-6053 Earthquake Design and Evaluation of Concrete Hydraulic Structures

EP 1110-2-18 Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures

ER 1110-1-12 Quality Management

ER 1110-1-261 Quality Assurance of Laboratory Testing Procedures

ER 1110-1-8100 Laboratory Investigation and Testing

ER 1110-2-401 Operation, Maintenance, Repair, Replacement, and Rehabilitation Manual for Projects and Separable Elements Managed by Project Sponsors

ER 1110-2-1802 Reporting Earthquake Effects

ER 110-2-1806 Earthquake Design and Evaluation for Civil Works Projects

ER 1110-2-8160 Planning and Design of Temporary Cofferdams and Braced Excavations

ETL 1110-2-39 Pile Foundations

ETL 1110-2-569 Design Guidance for Levee Underseepage

ETL 1110-2-575 Evaluation of I-walls

Naval Facilities Engineering Command Design Manual 7.02 Foundations and Earth Structures

Unified Facilities Criteria 3-220-01N Geotechnical Engineering Procedures for Foundation Design of Buildings and Structures

Federal Highway Administration. FHWA GEC 012 – Design and Construction of Drive Pile Foundations, Volume I. September 2016.

Federal Highway Administration. FHWA GEC 012 – Design and Construction of Drive Pile Foundations, Volume II. September 2016.

National Cooperative Highway Research Program. NCHRP 25-25/Task 74 Current Practices to Address Construction Vibration and Potential Effects to Historic Buildings Adjacent to Transportation Projects. September 2012.

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13. ATTACHMENTS

Additional details to various topics can be found in the following attachments:

Attachment 1: Seismic Evaluation
Attachment 2: Top of Cooper Marl and Existing Boring Locations
Attachment 3: T-wall Analyses
Attachment 4: Pile Capacity
Attachment 5: Preliminary Subsurface Investigation Estimate

Attachment 1: Seismic Evaluation

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Charleston Peninsula
Coastal Flood Risk Management Feasibility Study
Geology and Geotechnical Engineering Sub-Appendix
Attachment 1: Seismic Evaluation

Version: Final, May 2022

Table of Contents

Tables

Figures

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1.1. General

This seismic evaluation was conducted in accordance with the guidance established in ER 1110-2-1806 “Earthquake Design and Evaluation for Civil Works Projects” dated 31 May 2016. Guidance outlined in ECB 1110-2-6000 (DRAFT) “Selection of Design Earthquakes and Associated Ground Motions” was also utilized in the evaluation. This evaluation was augmented by products from the United States Geological Survey (USGS) and USACE, Risk Management Center (RMC) at (https://radsii.usace.army.mil/RMCResources.aspx).

1.2. Project Hazard Potential

The Charleston Peninsula is located at coordinates latitude N 32.787° and longitude W 79.937°. The Charleston Peninsula is a very densely populated area with thousands of resident and non-resident buildings. Because of its proximity to the coast and low elevation, the Charleston Peninsula is frequently inundated from tropical systems and the occasional perigean spring tide. Additionally, the impacts of harbor traffic and wind driven waves have caused extensive erosion around the peninsula resulting in loss of shoreline and relocation of infrastructure. The city has already started mitigation efforts to protect a portion of the southern peninsula by constructing a retaining wall. However, the increase in the frequency and intensity of tropical systems coupled with sea-level rise and harbor traffic has put a heavily populated area at greater risk. Additional protective measures need to be considered in order to not only protect the southern tip of the peninsula, but the peninsula as a whole. According to Table 1, the Charleston Peninsula has a High Hazard Potential Project rating, due to the presence of a residential population at risk (PAR).

Table 1: Hazard potential classification for the Charleston Peninsula based off ER 1110-2-1806.

Table B-1
HAZARD POTENTIAL CLASSIFICATION
FOR CIVIL WORKS PROJECTS

	Category[1]			
Hazard Potential Classification	Direct Loss of Life[2]	Lifeline Losses[3]	Property Losses[4]	Environmental Losses[5]
Low	None Expected	No disruption of services – repairs are cosmetic or rapidly repairable damage	Private agricultural lands, equipment, and isolated buildings	Minimal incremental damage
Significant	None Expected	Disruption of essential facilities and access	Major or extensive public and private facilities	Major or extensive mitigation required or impossible to mitigate
High	Probable (one or more)	Disruption of critical facilities and access	Extensive public and private facilities	Extensive mitigation cost or impossible to mitigate

Guidance established in ER 1110-2-1806 and ECB 1110-2-6000 states that projects having a “High Hazard Potential Project” shall have a Maximum Design Earthquake (MDE) that equals the Maximum Credible Earthquake (MCE). The required seawall performance under the MDE is damage control performance and under the MCE is collapse prevention performance.

1.3. Previous Seismic Evaluations

There have been no seismic considerations in the original design and no subsequent seismic evaluations have been performed prior to this document.

1.4. Seismotectonic Setting

1.4.1. General

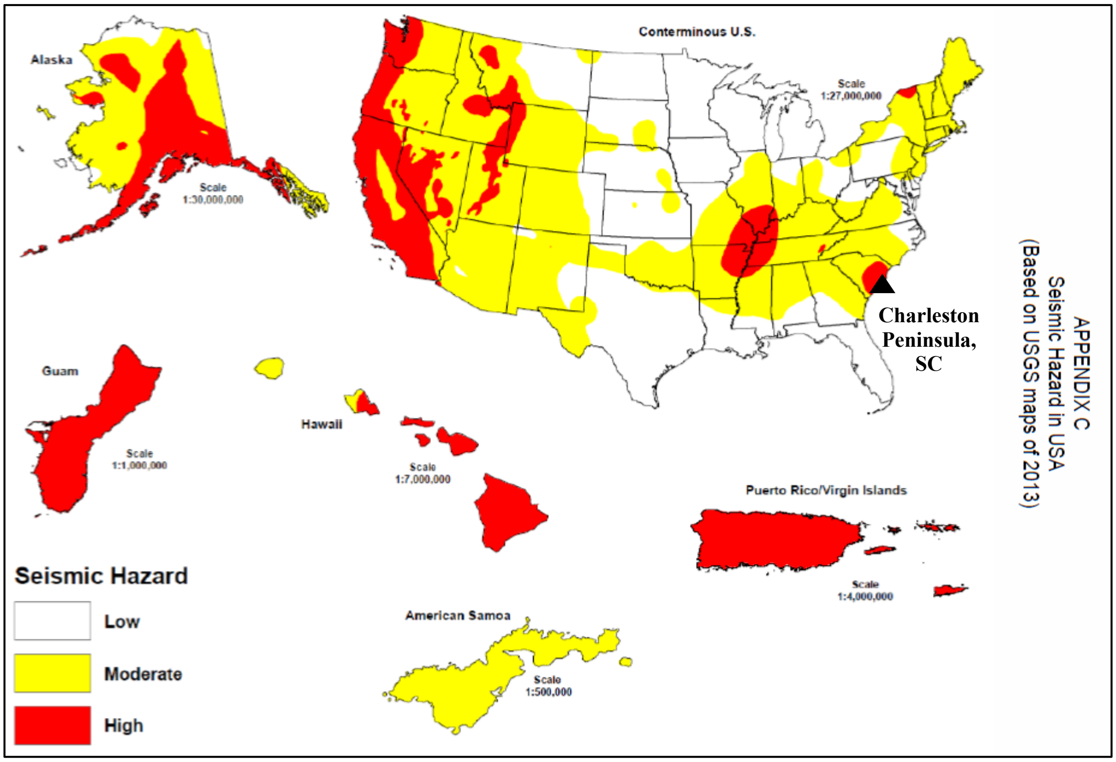

Figure 1: Project location shown on seismic hazard map of the USA, from ER-1110-2-1806.

ER 1110-2-1806 requires that the project site be located on the seismic hazard map (Figure 1). The Charleston Peninsula is within a high seismic hazard zone. While this map is generalized, it does indicate that ground motions will need to be considered as part of the construction design. Present state-of-the-art practice has moved toward methods that generate regional and site-specific (if needed) probabilistic and deterministic Peak Ground Acceleration (PGA), or response spectra analysis (Leyendecker et al., 2000; NEHRP, 2009; 2012; ASCE/SEI 7-10; International Building Code, 2012; USGS, 2014; 2016).

Seismic hazard maps presently available by the United States Geological Survey offer the best up-to-date seismic probability assessments. A more detailed USGS seismic hazard map, filtered and adjusted for seismicity in the southeastern U.S., is shown in Figure 2. This map shows the contoured peak ground acceleration (PGA) to be expected within southeastern U.S. from an earthquake having a return period of 2,475 years, or a 2% probability of exceedance in 50 years. This corresponds to the Maximum Credible Earthquake defined in ER 1110-2-1806, EM 1110-2-6053, and ECB 1110-2-6000. Ground motions and spectra will be described in detail later in this chapter. The USGS 2014 seismic hazard map by Petersen et al. (2015), shown in Figure 2, suggests that an earthquake with a 2% probability of exceedance in 50 years could produce a

PGA that ranges from 0.6 to 0.8g near the Charleston Peninsula. ER 1110-2-1806 and Krinitzsky (2003) discourage the use of probabilistic methods alone to estimate ground motion parameters because they may be much different from those using deterministic methods. This is due to inherent sampling bias and limited recorded history (<100 years) in probabilistic methods, which when extrapolated to longer time periods results in much larger ground motions than those using deterministic methods that utilize kinematic fault movement/history parameters. ER 1110-2-1806 states that the MCE should be developed by deterministic methods, but it also needs to be informed by probabilistic methods as well.

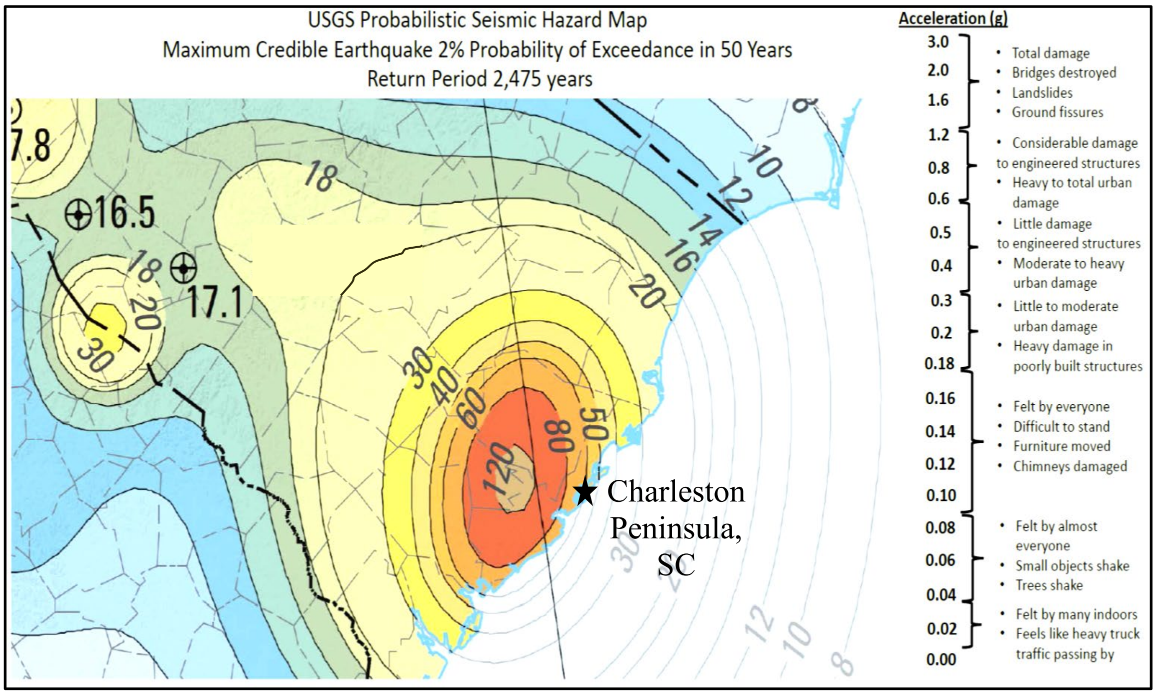

Figure 2: USGS Seismic Hazard Map, PGA, 2% Probability of Exceedance in 50 Years, from Peterson et al. (2015).

Note: Contours of peak acceleration expressed in percent of gravity (%g). Point values shown indicate local minimum %g.

Because the project lies within an area known to be highly influenced by a zone of high seismic activity, and that the project has been deemed to have a high hazard classification, the use of both deterministic and probabilistic methods is deemed to be appropriate in order to preserve engineering conservatism (ECB 1110-2-6000; ER 1110-2-1806; Krinitzsky, 2003).

1.4.2. Geology of the Central and Eastern U.S. Seismotectonic Zone

The project site lies within the Central and Eastern U.S. Seismotectonic Zone (CEUS), the seismotectonic zone is located hundreds of miles from active plate tectonic boundaries and is characterized by relatively low rates of seismicity. However, the Charleston Peninsula is a localized "hot spot" of high seismic activity. This area is known as the Middleton Place-Summerville Seismic Zone or the Charleston Seismic Zone. A generalized regional geologic map of the CEUS is shown in Figure 3. The CEUS is comprised of Pre-Cambrian stable interior cratonic crust, Paleozoic-aged imbricated, thrust sheet stacks of metamorphic, igneous, and metasedimentary sediments comprising the Appalachian chain, Mesozoic-aged rift basin sequences of intermediate and mafic intrusive igneous rocks, metavolcanic and sedimentary rocks, and younger Gulf Coast sedimentary rock. These areas have slightly different bulk rock seismic velocities, and slightly different rates of seismic occurrence, which may be due to effects of reactivation of pre-existing faults and planes of weakness in response to present-day remote tectonic forces.

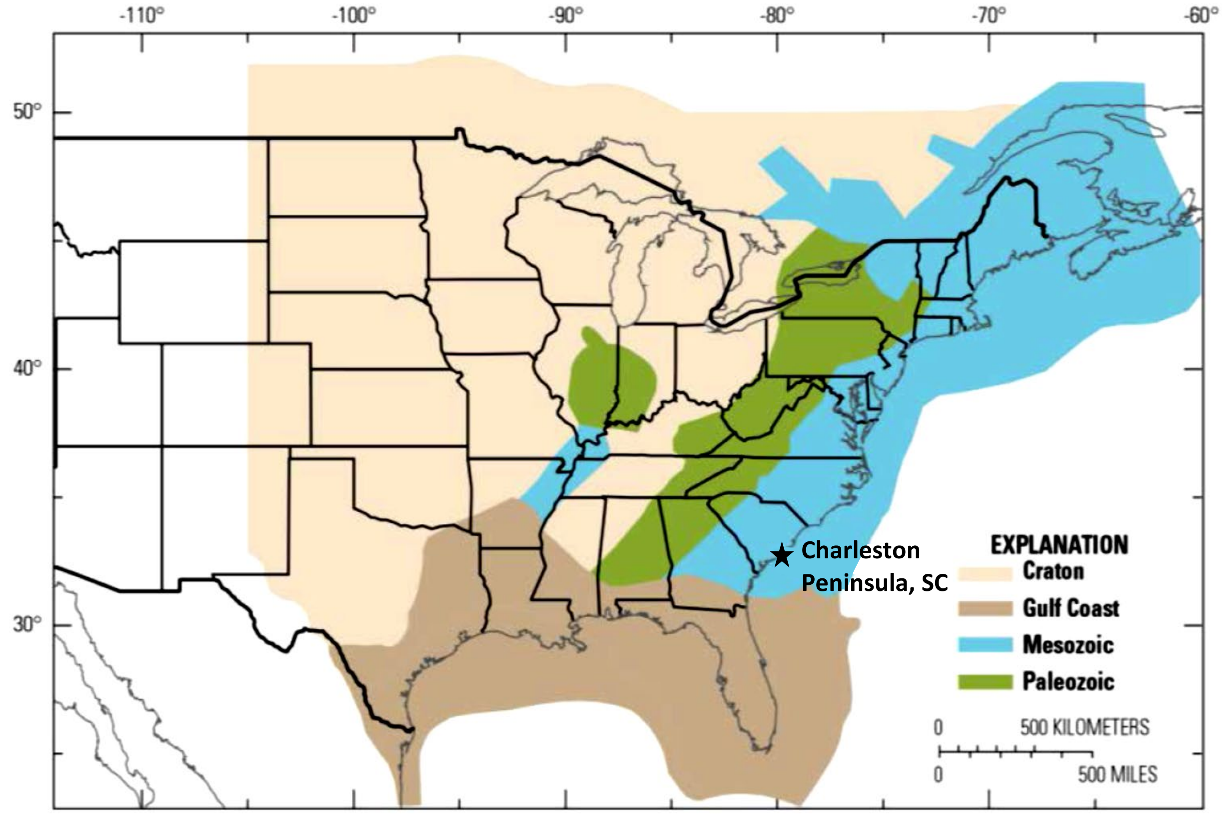

Figure 3: General geologic and seismic velocity structure of the CEUS seismotectonic zone.

There are no active surficially expressed, regional-scale transform faults such as the San Andreas Fault Zone, or active subduction zones. For the CEUS seismotectonic zone, most large scale fault movement had occurred during the Late Paleozoic and Mesozoic eras. The last major tectonic event was related to Mesozoic rifting and opening of the Atlantic Ocean. However, there is active low magnitude seismicity and large regional earthquakes have occurred in the CEUS in historic time. The fault source for many strong motion earthquakes in CEUS is generally not well defined because there is little to no surficial fault expression, as seen in the western U.S. While there is general agreement that large magnitude earthquakes in the CEUS are the result of shallow to deep basement crustal fault slippage, a clear association of even some of the largest historical earthquakes (e.g. the 1886 Charleston, S.C. earthquake) with a particular fault has been difficult to recognize. Therefore, in the CEUS, earthquake sources are generally defined as areas or volumetric source zones which is deemed acceptable in accordance with ER 1110-2-1806, EM 1110-2-6053, and ECB 1110-2-6000.

1.4.3. Earthquake Catalogue, 1964 - Present

An earthquake map and event catalogue was created for this evaluation using data from the Search Earthquake Catalog which is managed by the USGS. It can be accessed https://earthquake.usgs.gov/earthquakes/search/. Input parameters used to query the online application are tabulated in Table 2 below:

Table 2: Search parameters used to query the Search Earthquake Catalog for Charleston Peninsula, South Carolina.

ANSS PARAMETER DESCRIPTION	INPUT PARAMETERS
Start Date/ Time	January 01, 1800
End Date/ Time	December 18, 2019
Minimum Latitude	31.376
Minimum Longitude	-74.839
Maximum Latitude	38.465
Maximum Longitude	-84.990

Figure 4 shows the earthquake event map for all earthquakes recorded by the Search Earthquake Catalog using the input parameters denoted in Table 2. The query returned a total of 1,327 earthquakes that were measured from 1800 to present, and nearly all were less than moment magnitude (Mw) 5.0. It is assumed that the seismological record in the early 20th century is underrepresented due to the lack of seismological monitoring. Nonetheless, large magnitude earthquakes are documented to have occurred within the area, specifically, the Charleston, SC, 1886 earthquake (Mw = 7.3). Table 3 shows the number of earthquakes by strength, PGA, and relative effects from Figure 4. The majority of these earthquakes are very weak to weak and may not have been noticed by the public.

Table 3: Magnitude distribution of earthquakes from seismic catalogue query (1800 to present) within 300 km of the project site.

EQ Strength (Mw)	**# EQ Measured ANSS**	**Est. PGA Range (Epicentral g)**	**Observed Effects**
Magnitude 0 to 1	11- quakes	< 0.002g	Felt by very few people; barely noticeable.
Magnitude 1 to 2	80-quakes	0.002g – 0.008g	Felt by few people; mostly upper floors.
Magnitude 2 to 3	139-quakes	0.008g – 0.014g	Noticeable indoors, especially on upper floors, may not be recognized as an EQ.
Magnitude 3 to 4	31-quakes	0.014g – 0.039g	Felt by many indoors, few outdoors. Feels like a heavy truck passing.
Magnitude 4 to 5	12-quakes	0.039g – 0.18g	Felt by almost everyone, some people awakened. Small objects moved. Some plaster falls. Chimneys slightly damaged.
Magnitude 5 to 6 *1 quake >6.0 Mw	1-quakes	0.18g – 0.30g	Little to moderate urban damage. Heavy damage in poorly built structure.

The Middleton Place-Summerville (Charleston) Seismic Zone, however, is characterized by a dense clustering of earthquakes (1.0 < Mw < 5.0) that indicate active seismicity. Northwest of the project site, there is a broad zone of weak to moderate seismicity (Eastern Tennessee Source Zone) that is associated with the Western Blue Ridge and Valley and Ridge geologic provinces (see Figure 4). Seismicity within this area is unique to its geology and does not relate to the project site because, at >300 km away, it is outside of the maximum source to site consideration for this project (ER 1110-2-6000).

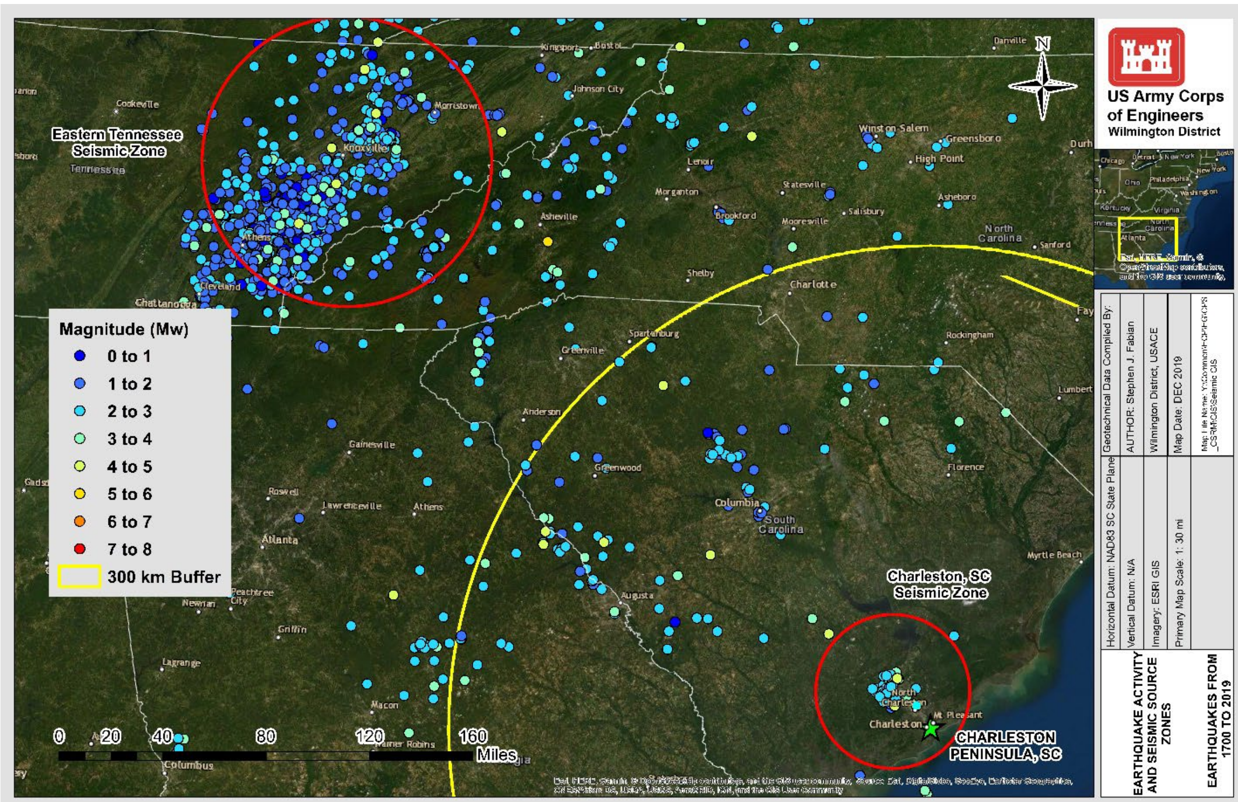

Figure 4: Map showing measured earthquake activity and seismic source zones within 300 km of the project site.

Note: Sourced data (1700-present) is from the seismic catalogue.

1.4.4. Regional Seismic Source Model Defined

The seismic source model used by the 2014 National Seismic Hazard Mapping Program for the CEUS (Peterson et al., 2014) considers both seismicity-based background sources and fault-based sources and utilizes data and models from the CEUS Seismic Source Characterization for Nuclear Facilities (CEUS–SSCN) project, which accounts for broader uncertainties and replaces older seismic source models. The Peterson et al. (2014) source model assumes that future large earthquakes are more likely to nucleate near previous earthquakes with M_w greater than or equal to 3.0 (see Figure 5). The model also distinguishes seismotectonic zones in the CEUS with distinct seismicity and maximum earthquake magnitudes in order to accommodate some possibility that the historical seismicity does not fully represent likely sources of background earthquakes. As shown in Figure 5, the project lies in an area characterized by high levels of weak to moderate seismicity (3 < Mw < 6) which is influenced by strong motion earthquakes originating from the Middleton Place-Summerville (Charleston) Seismic Zone or by local diffuse background sources.

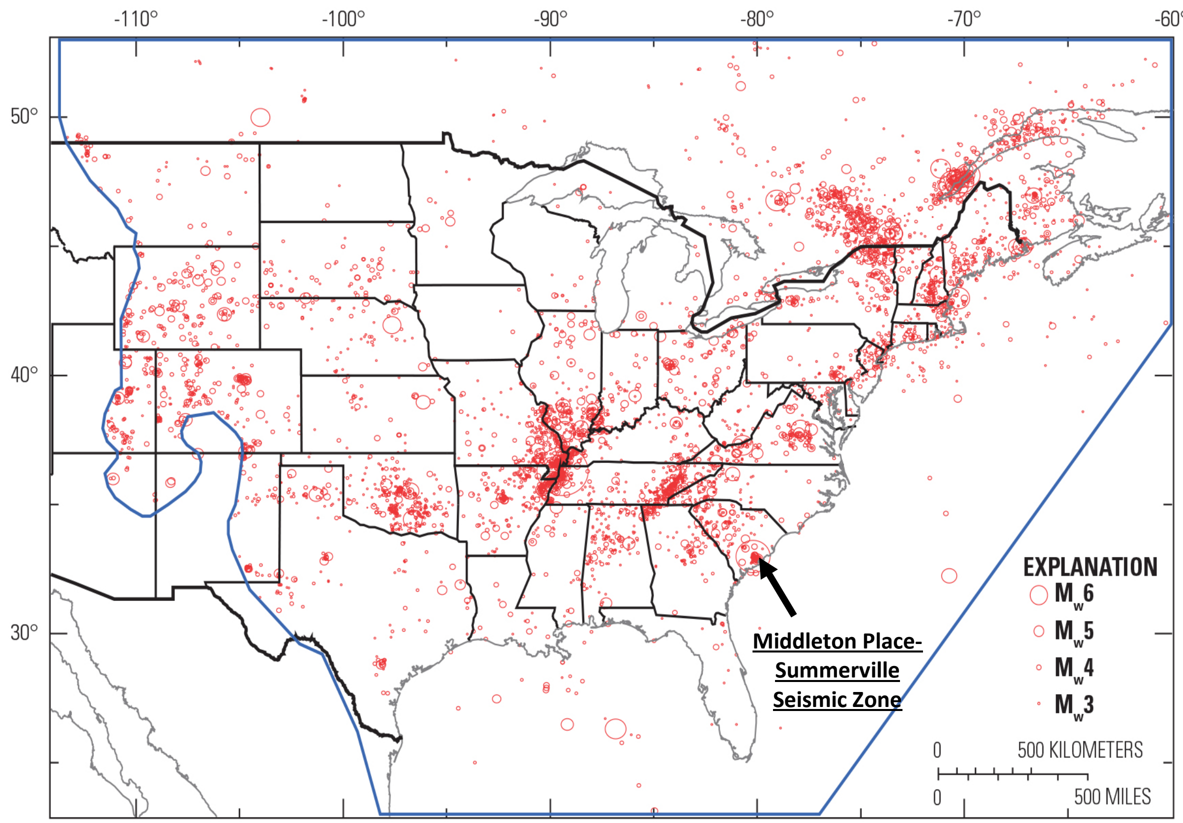

Figure 5: CEUS Earthquake data 1700 to 2012 used for USGS-sponsored seismic hazard mapping, from Peterson et al. (2014).

1.4.5. Review of USGS Quaternary Fault Database

Faults capable of producing a strong motion earthquakes (Mw > 5.0), that lie within a 50 km/31 mile radius of a project site, must be identified in accordance with ER 1110-2-1806 and ECB 1110-2-6000 (1898 Charleston Earthquake). The USGS Quaternary fault and fold database of the United States (http://earthquake.usgs.gov/hazards/qfaults/) was reviewed to locate any active Quaternary-aged (past 1,600,000 years) faults in close proximity to the Charleston Peninsula. No active Quaternary faults were found, but evidence of paleoliquefaction has been mapped along the coastal areas of North and South Carolina (see Figure 6)[1]. Guidance initially established by Krinitzsky (1995) and reinforced by ECB 1110-2-6000 states that if no active faults are found

[1] There is evidence of large strong motion earthquakes that have occurred within the last 15,000 years during the latter part of the Holocene, which are related to the Middleton Place-Summerville (Charleston) Seismic Zone. Liquefaction features such as sand boils and sand fissures, first recognized in the region following the 1886 earthquake, have been mapped and geochronologically dated throughout the coastal region of South Carolina. Though the liquefaction features demonstrate that strong prehistoric shaking occurred, they provide no information on specific source fault attributes such as azimuth, length, dip, sense of motion, or slip-rate (Wheeler, 1998).

within 50km/31 miles of a project site, then far-field attenuation curves shall be used to evaluate MCE ground motions to a maximum distance of 300 km/186 miles. The Charleston Peninsula lies in an area where a highly active source is present and capable of generating large magnitude earthquakes.

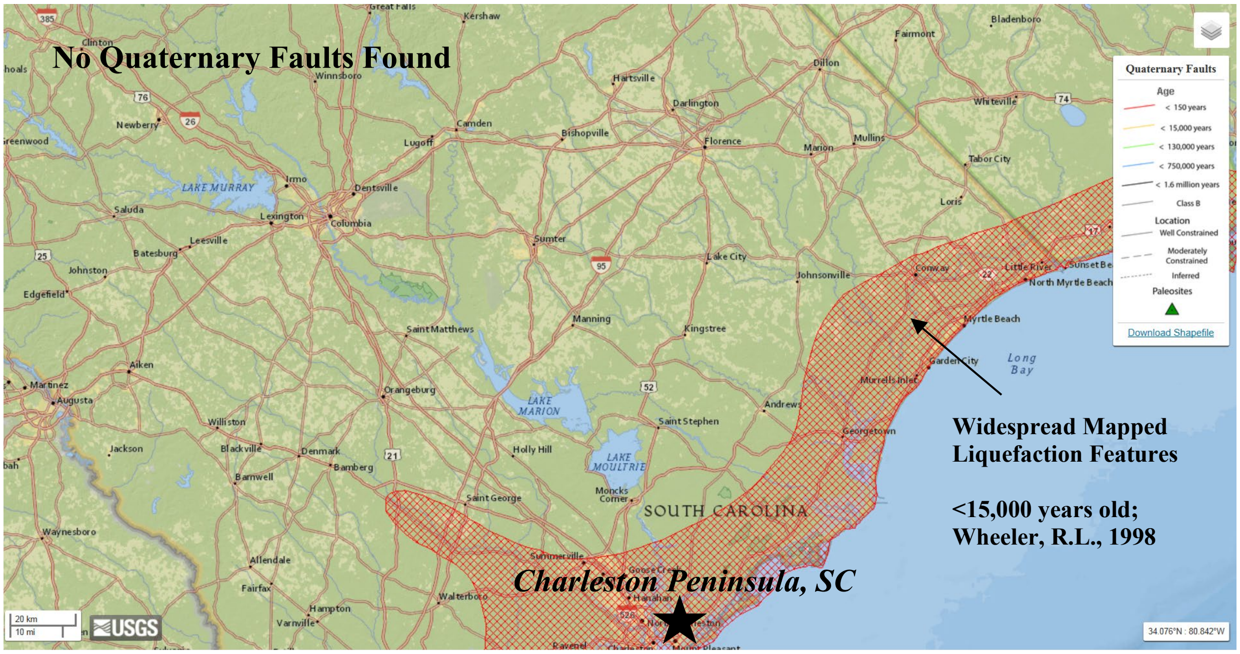

Figure 6: Location of paleoliquefaction features, from USGS Quaternary Fault and Fold Database of the U.S.

1.4.6. Middleton Place-Summerville (Charleston) Seismic Zone Defined

Middleton Place-Summerville (Charleston) Seismic Zone is a region of high seismic hazard centered 30 kilometers northwest of Charleston, South Carolina, where a large earthquake (Mw = 7.3) caused widespread damage in 1886. The 1886 Charleston earthquake is the largest earthquake known to have occurred in the southeastern United States and was likely due to a reactivated deeply buried basement fault (Rankin, 1977). Observations of earthquake activity within the Middleton Place-Summerville (Charleston) Seismic Zone suggest that it may be associated with a failed extensional rift basin[2] within the Mesozoic-aged extended crust (ECB 1110-2-6000). A detailed map showing the contoured seismicity, tectonic structure, and paleoliquefaction features within the epicentral region of the 1886 Charleston earthquake is shown in Figure 7. Previous workers have utilized mapping of sand boils (Amick et al., 1990), geologic well logs (Colquhoun et al., 1983; Weems and Lewis, 2002), seismic survey (Behrendt et al., 1983; Schilt et al., 1983; Marple and Miller, 2006), numerous kinematic and seismotectonic studies (chiefly, Dura-Gomez and Talwani, 2006), paleoseismic studies (Talwani

[2] Failed rift basins are deeply buried, sediment filled, faulted basins that are oriented at a high angle to adjacent oceanic plates or orogenic belts. They form by faulting from extensional tectonics and crustal thinning. These structures are thought to represent failed initiation points of ancient continental rifting and ocean basin formation.

and Schaffer, 2001) and even geomorphological mapping of the Ashley River (Marple and Talwani, 2000) to ascertain specific fault characteristics, but disagreements (e.g., Marple, 2011) among seismic workers forestall detailed fault modelling in this seismic evaluation. An MCE of 7.3 Mw + 1σ is selected for use in characterizing the Middleton Place-Summerville (Charleston) Seismic Zone.

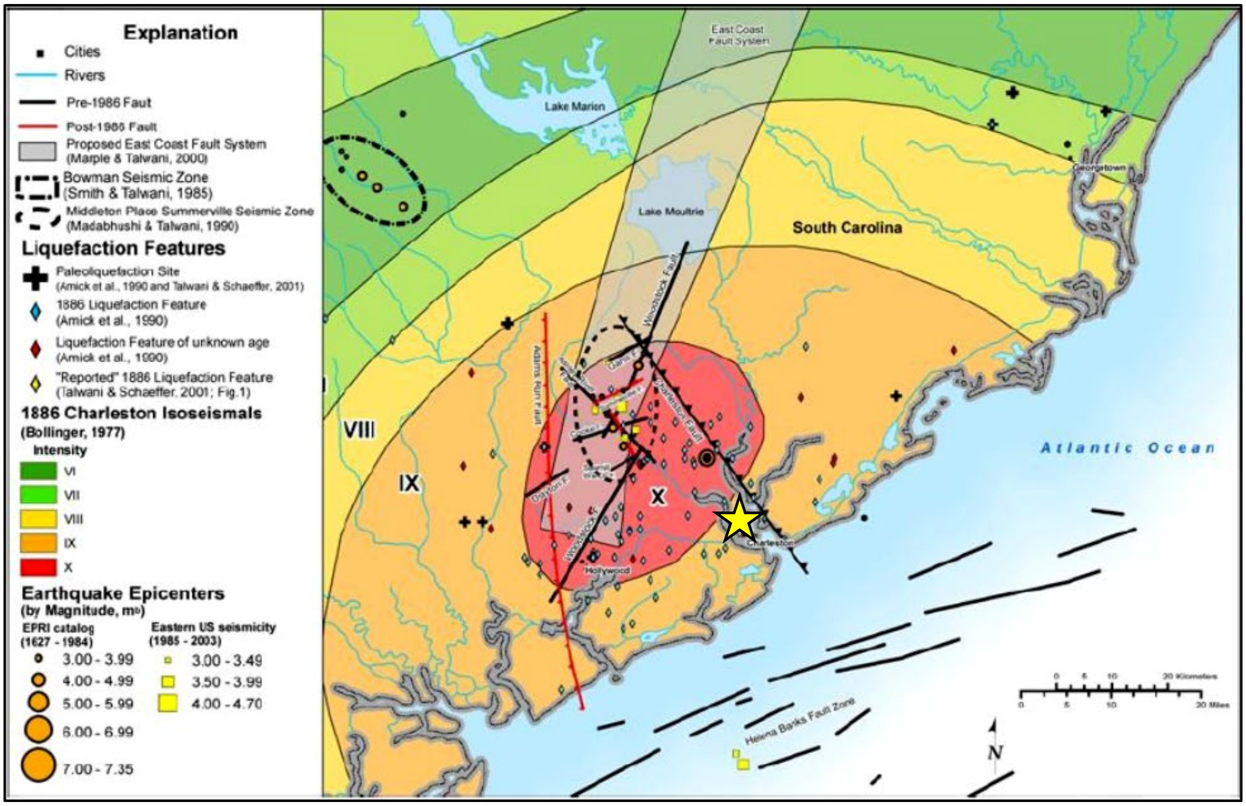

Figure 7: Map from ECB 1110-2-6000 showing seismicity, tectonic and paleoliquefaction, in the epicentral region of the 1886 Charleston earthquake, from Southern Nuclear Company (2007).

1.5. USGS Uniform Hazard Tool and Seismic Hazard Deaggregation for the Project Site

The USGS's Uniform Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/) was used to evaluate the seismic hazard to the site. Inputs to the tool include:

- USGS Probabilistic Seismic Hazard Map Edition: Dynamic conterminous U.S. 2014 (v4.1.1) was used because it was the only dataset capable of interacting with the deaggregation tool.
- Spectral Period: PGA, 0.2, 1.0, and 2.0 seconds evaluated.
- Latitude/Longitude Inputs: 32.787 Lat. / -79.937 Long.
- Time Horizon: Return period 2,475 year corresponding to a 2% in 50 years AEP selected.
- Site Class: Only one Vs30 site class was available in the application: B-C (760 m/s) designated "firm rock" and A (2000 m/s) which is designated "hard rock." Because the uppermost crustal strata in the region consists of loosely consolidated clayey sands underlain by dense silts and clays, a Site B-C boundary of 760 m/s was selected for use to initially evaluate the seismic hazard and corresponding deaggregation.

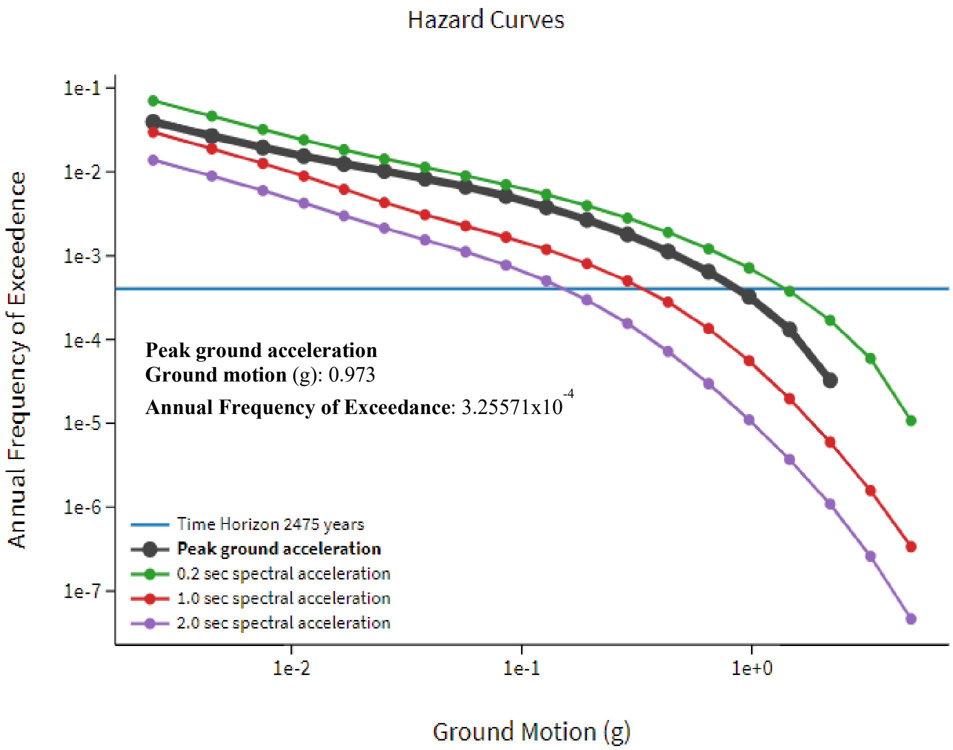

Figure 8: Probabilistic Seismic Hazard Curve showing PGA for 2% in 50 years AEP (2,475 year return period), using USGS 2014 Conterminous U.S. data.

Probabilistic seismic hazard curves showing the PGA 0.2, 1.0, and 2.0 second spectral acceleration predicted for the project site are shown in Figure 8. The site-predicted PGA for an earthquake having a return period of 2,475 years is approximately 0.973g, which is slightly higher than the USGS seismic hazard map shown in Figure 2 ranging from 0.6 to 0.8g. Spectral ground motion on the Charleston Peninsula was also predicted by the Uniform Hazard Response Spectrum (Figure 9). Based upon probabilistic hazard mapping, the PGA at the site is predicted to be 0.8561g, but the largest and most likely damaging ground motion is 1.3972g at a spectral period of 0.2 seconds (Figure 9).

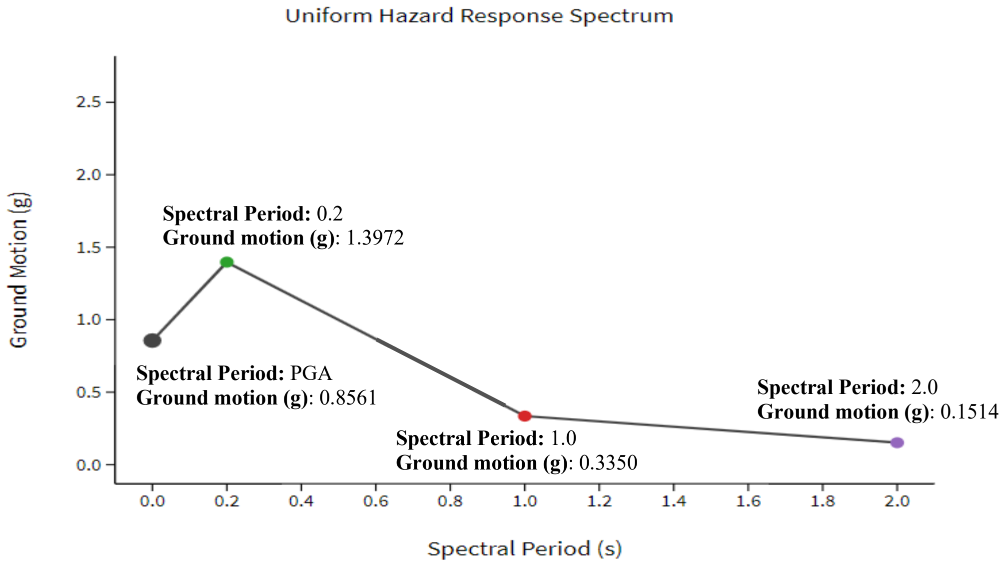

Figure 9. Uniform Hazard Response Spectrum predicted for the project site showing PGA with 2% in 50 years AEP (2,475 return period).

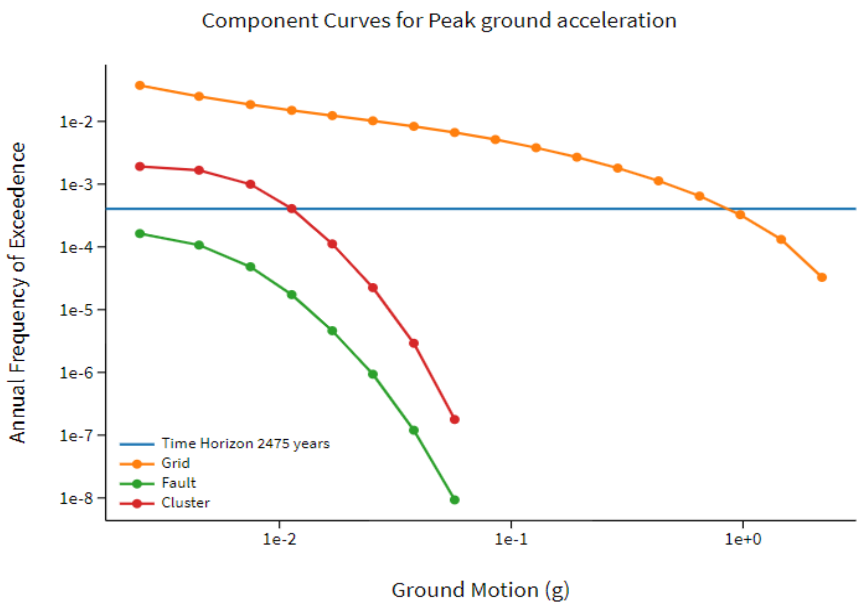

Figure 10: Component curves showing PGA differences based on different seismic sources as calculated in probabilistic hazard analysis.

Note, “gridded” seismic data produces the highest ground motions.

It should be noted that the probabilistic seismic hazard predicts the total seismic hazard by integrating all potential source magnitudes and distance, and applying a statistical prediction of the event return period. These return periods are generally greater than what is empirically supported by human observation. As a result, this may produce higher ground motions than what is geologically possible at a particular site, as discussed in Krinitzsky (2003). Figure 10 illustrates this by showing the PGA differences that arise during probabilistic seismic hazard analysis when considering point seismic sources, faults, or gridded seismic data. Probabilistic seismic hazard curves (see Figure 8 and Figure 9) display relatively high ground motions, but provide a good initial estimate of the total ground motion and associated seismic risk. Probabilistic analysis is used to inform site-specific deterministic seismic hazard analysis (ER 1110-2-1806; ECB 1110-2-6000), discussed later in this chapter.

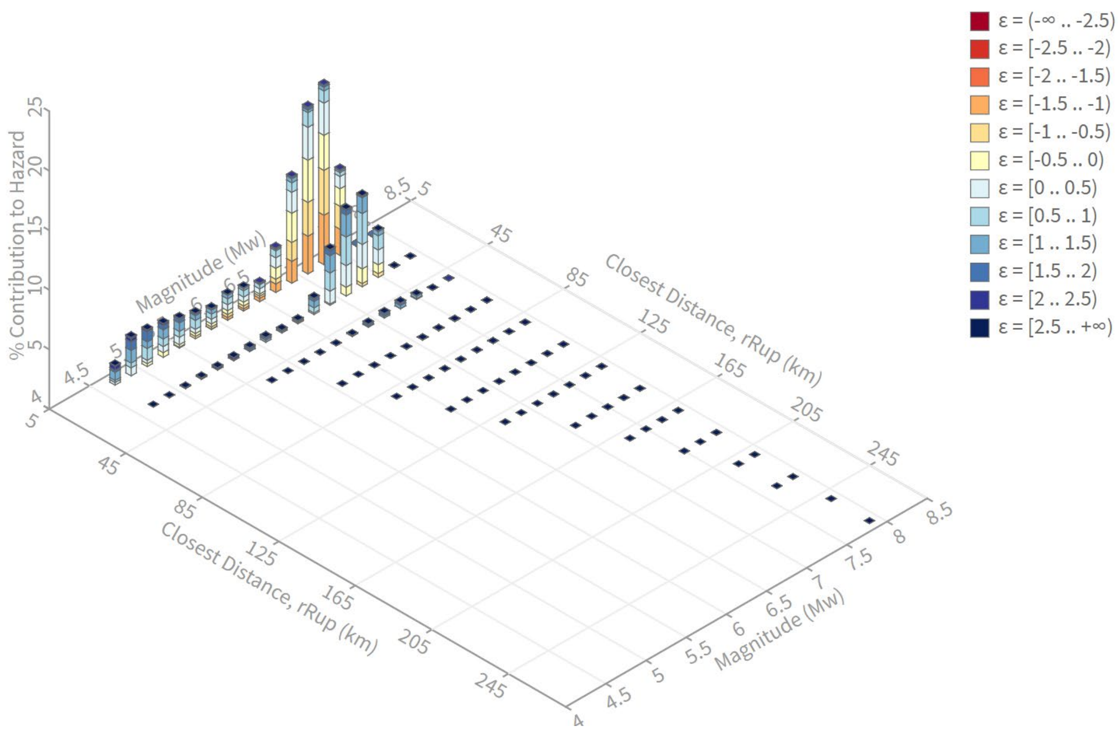

Figure 11: Deaggregation chart (USGS, 2014) showing itemized seismic hazard contribution to the Charleston Peninsula.

A seismic hazard deaggregation chart was constructed using USGS Uniform Hazard Tool. Deaggregation charts measure the hazard contribution from a number of seismic sources to the project site (see Figure 11). The deaggregation process takes integrated ground motion from all seismic events within the U.S., statistically itemizes it, and projects each seismic hazard based upon magnitude (Mw) and distance (rRup) to the project site. The percent contribution to the total seismic hazard is measured in terms of ε, which is the number of logarithmic standard deviations from which the itemized seismic hazard deviates from the total mean predicted ground motion.

Figure 11 shows a unimodal distribution in the total seismic hazard to the project site. The most prominent contribution to the seismic hazard is the Mw = 7.3 earthquake[3] ($\varepsilon_o = -1.33\sigma$), located at a distance of 10.72 km.

[3] This earthquake corresponds to the 1886 Charleston Earthquake, which had an estimated moment magnitude of 6.9-7.3. The earthquake caused 60 deaths and destroyed 2,000 buildings. Damage estimated to be between 5-6 million dollars.

1.6. Charleston Peninsula Vs30 Designation

The seismic velocity of the upper 30 meters of soil (Vs30) was initially estimated using the USGS Global slope-based Vs30 model (Wald and Allen, 2007; Allen and Wald, 2009), found at: https://earthquake.usgs.gov/data/vs30/. The values derived range between 180 to 330 m/s. After looking at the overall seismic velocities for the Charleston Peninsula, a velocity of 255 m/s was determined. A Vs30 of 255 m/s falls within the range of a seismic "Site Class D" classification.

1.7. MCE Deterministic Analysis

1.7.1. General

USACE design guidelines utilize a Maximum Credible Earthquake (MCE) and an Operating Basis Earthquake (OBE). The MCE is defined as the greatest earthquake magnitude that can reasonably be expected to be generated by a specific source based on seismological and geological evidence. The MCE has no defined return period. According to ER 1110-2-1806, an OBE is based on the event with a 50% probability of occurrence during the 100-year service life of the project. This translates to a 144-year return period. The MCE is determined by a deterministic seismic hazard analysis, while the OBE is determined by a probabilistic seismic hazard analysis (see Section 5.8).

1.7.2. Charleston MCE and Background Earthquake

Deterministically derived MCE were developed using the methods described by Krinitzsky (1995) and in ECB 1110-2-6000. An Mw = 7.3 MCE is established for the Middleton Place-Summerville (Charleston) Seismic Zone based upon the 1886 Charleston Earthquake event. The distance from the project site to the center of the MCE source zone is 10.00 km.

1.7.3. Ground Motion Prediction Equations and Source Attenuation

Ground motion prediction equations (GMPE) take into account bulk crustal seismic velocities and other components to attenuate ground motions as they are propagated to the project site. GMPE and attenuation curves are used to estimate the median site PGA that is propagated from an MCE epicenter or another designated seismic source. The GMPE of Boore and Atkinson (2006) is used because it was specifically developed for use within the eastern U.S. Furthermore, this GMPE was selected because it is readily available for use in open source, web-based ground-motion calculators. A +1 σ (standard deviation) was applied to the median PGA curves in order to account for uncertainty in assessing the MCE, and achieve the 84th percentile ground motion projection. Boore and Atkinson (2006) recommend that median ground acceleration values be multiplied by 10^ (log10 (ground acceleration) +0.3) to account for this uncertainty. Resultant ground motions for engineering consideration reflect Mean PGA+ 1σ. Figure 12 shows how the median PGA+1 σ is attenuated to the site from the epicenter.

1.7.4. OPENSHA Ground Motion Modeling and Attenuation Relationship Plotter

Ground motions were generated using OPENSHA, which is an open-source, web-based modeling and plotting program developed by Field et al. (2003): http://www.opensha.org/apps.

This web application is freely available through the USGS website and it is relatively easy to use. For the web application, the following inputs were used:

- A site seismic velocity (Vs30) of 255 m/s (site class D) was designated. The fault type was designated as "unknown" due to it being a deep crustal level feature. "Unknown" fault type yields the highest PGA, which is considered appropriate for conservatism.
- Charleston MCE was established at Mw = 7.3.
- X-axes were set to measure the shortest surficial distance to the surface rupture. Distances were set to 10.00 km for the Charleston MCE.
- Y axes were set to Median PGA.

1.7.5. Seismic Attenuation Curves

The median PGA output from OPENSHA was then plotted in Excel and the Median PGA +1 σ was calculated to generate the curve representing the +1 standard deviation or 84th percentile. Median PGA and +1 σ curves for the Charleston MCE is shown in Figure 12. The isoseismal contours 0.5g and 0.8g from the USGS seismic hazard map (Figure 2) are also plotted against the attenuation curves to compare the predicted ground motions for the project site by deterministic and probabilistic methods. The probabilistic site-specified PGA from Figure 8 is also plotted for reference. The distance from each epicenter, relative magnitude, and predicted attenuated PGA at the project site are given in Table 4.

Table 4: Seismic attenuation curves indicating median PGA and median PGA +1 σ at project site.

Seismic Source	**Distance to Project**	**Max Credible Earthquake (Mw)**	**Boore & Atkinson, 2006 Median PGA at Project**	**Boore and Atkinson, 2006 Median PGA +1 σ at Project**
Charleston, SC	10 km	7.3	0.28g	0.56g

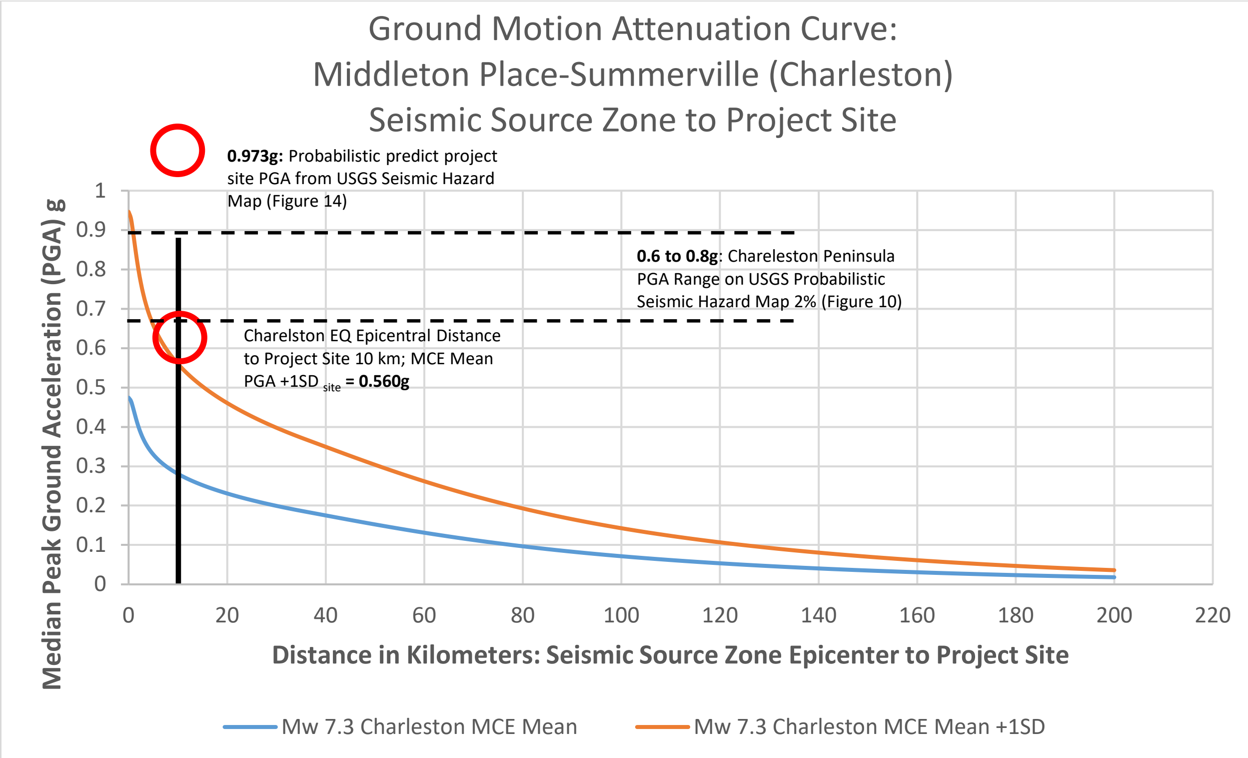

Figure 12: Attenuation curves for selected seismic source zones with respect to project site. Note: GMPE of Boore and Atkinson (2006) used to generate median PGA and median PGA +1σ. Curves generated using the attenuation relationship of Boore Atkinson (2006), with site Vs30 = 255 m/s. Mw Charleston MCE = 10 km distal source.

While attenuation curves are useful in understanding how ground motion is dampened with distance from the epicenter, site response is better understood by evaluating the spectral acceleration predicted for the site by the ground motion prediction equation. Furthermore, EM-1110-2-1806 mandates the evaluation of spectral periods between 0.2 and 5 seconds. The ground motion prediction equation of Boore and Atkinson (2006) was also used to predict the spectral wave for Charleston MCE events. The site response spectra was evaluated for an array of ground motions and is discussed in the following section.

1.7.6. Spectral Acceleration Ground Motion Response

OPENSHA was used to generate spectral acceleration data for the Charleston MCE, using the ground motion prediction equation of Boore and Atkinson (2006), for wave periods between 0 and 5 seconds. Input parameters for fault type, seismic velocity, event magnitude, and distance to epicenter are the same as discussed in Section 1.7.2. Spectral acceleration (SA) response curves for the Charleston MCE are shown in Figure 13. Table 5 shows the response spectra ordinates for the curves in Figure 13. A PGA of 1.261g at 0.30 sec period is selected as the design earthquake for follow-on liquefaction and stability analyses. The deterministically derived design PGA of 1.261g is considered to be conservative and agrees well with current USGS probabilistic seismic hazard data (see Figure 13) which yields a similar SA of 1.3972g at 0.2 seconds.

Table 5: Deterministic acceleration response spectra for median and median + 1σ ground motions generated from Middleton Place-Summerville (Charleston) Seismic Zone and Background Earthquake.

Period (Seconds)	**Middleton Place-Summerville (Charleston) Seismic Zone MCE Mw = 7.3**	**Middleton Place-Summerville Charleston Seismic Zone MCE Mw = 7.3 Median PGA +1 σ**
	Acceleration g (m/s^{2})	
0.05	0.387	0.772
0.10	0.466	0.930
0.20	0.616	1.230
0.30	0.632	1.261
0.50	0.576	1.151
1.00	0.355	0.709
2.00	0.183	0.366
3.00	0.120	0.240
4.00	0.087	0.174
5.00	0.070	0.141

Note: The spectral acceleration selected for design/liquefaction analysis is highlighted green.

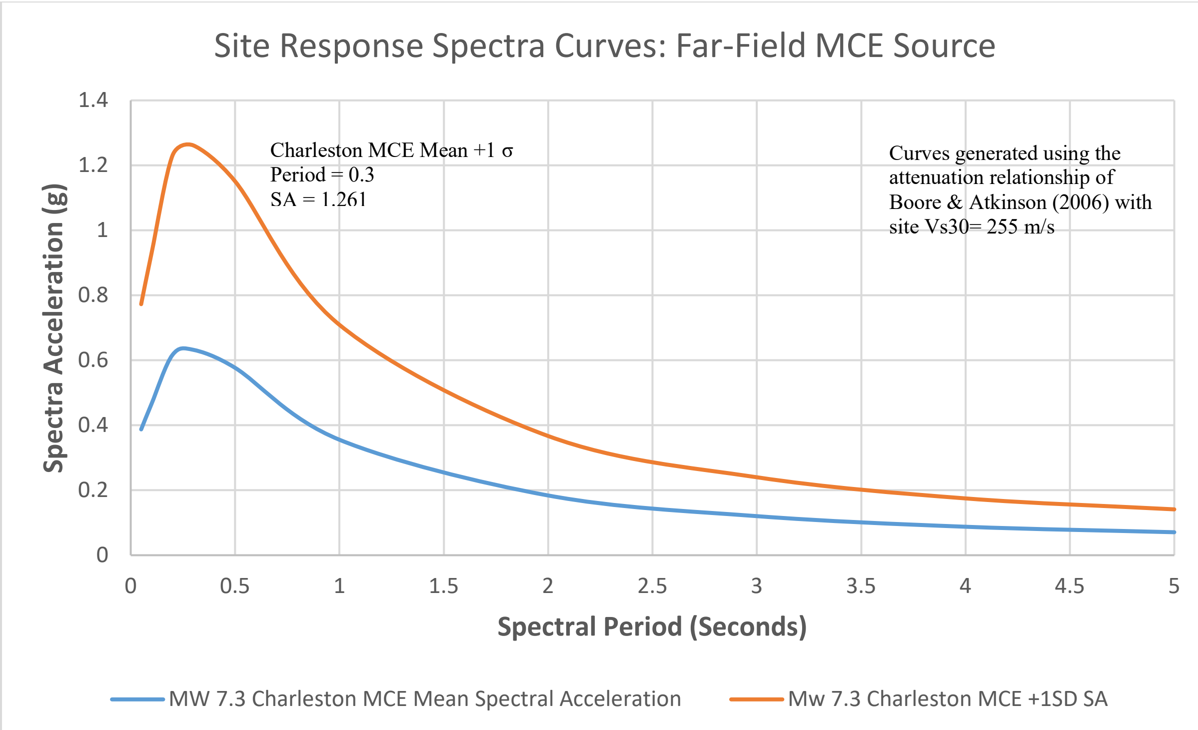

Figure 13: Deterministic acceleration response spectra curves for described ground motions.

1.7.7. Spectral Velocity Ground Motion Response

Spectral velocities from the Charleston MCE were interpolated using the following relationship: $V = V_0 + (a * t)$, where V = incremental ground velocity, V_0 = initial velocity, and t = period (sec). The relative velocities were calculated for the acceleration response spectra (see Table 5 and Figure 13) that were generated by the OPENSHA application using the ground motion prediction equation of Boore and Atkinson (2006). Table 6 contains the computed seismic velocities, the curves of which are plotted in Figure 14.

Table 6: Deterministic velocity response spectra for Middleton Place-Summerville (Charleston) Seismic Zone (Toro et al., 1997; USGS, 2003; Boore and Atkinson, 2006).

Period (Seconds)	**Middleton Place-Summerville (Charleston) Seismic Zone MCE Mw = 7.3**	**Middleton Place-Summerville (Charleston) Seismic Zone MCE Mw = 7.3 Median PGA +1 σ**
	Velocity (cm/s)	
0.05	1.936	3.86
0.10	6.599	13.16
0.20	18.93	37.77
0.30	37.89	75.61
0.50	66.74	133.1
1.00	102.2	204.0
2.00	139.0	277.4
3	175.1	349.4
4	210.2	419.4
5	245.5	489.9

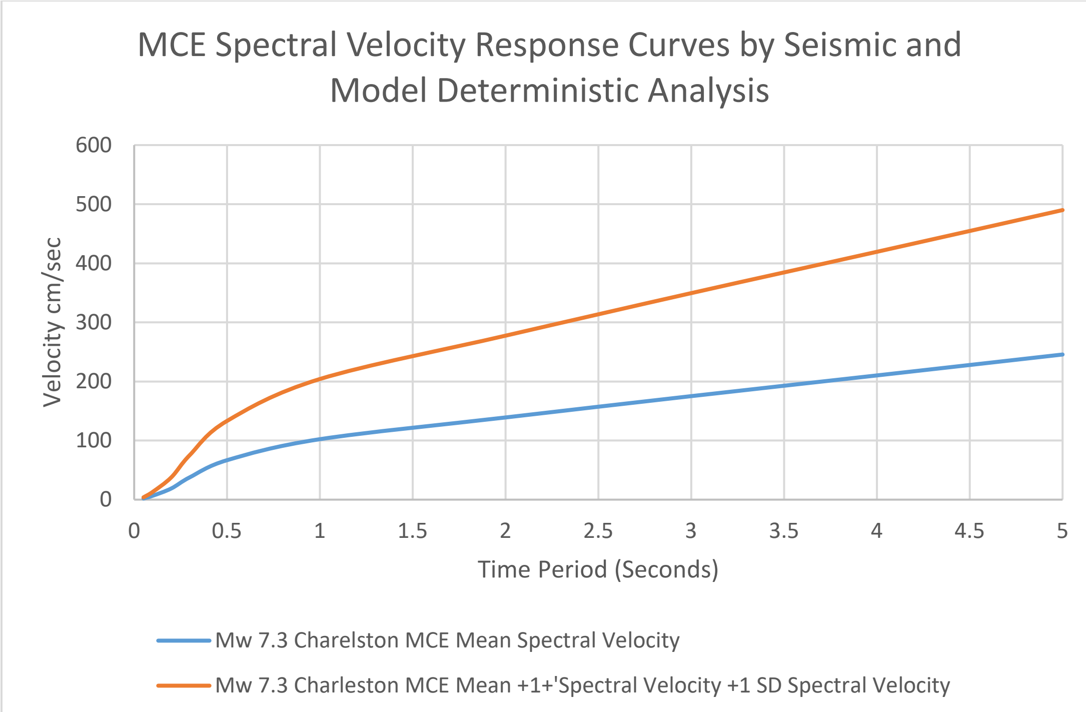

Figure 14: Deterministic velocity response spectra curves for described ground motions.

1.8. OBE Probabilistic Analysis

1.8.1. General and OBE Defined

The Operational Basis Earthquake (OBE) is defined in ER 1110-2-1806 as the earthquake that can reasonably be expected to occur within the service life of the project, typically a 50% probability of exceedance in 100 years (average return period of 144 years). The OBE is assessed using probabilistic methods that are informed by deterministic methods (see Section 5.7).

1.8.2. USGS Unified Hazard Tool Input Parameters

Probabilistic hazard characterization is based on existing USGS data by Frankel et al. (1996; 2002), and later revised by Petersen et al. (2015). Seismic hazard curves were generated using the USGS Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/). Input parameters utilized are nearly same as in Section 5.5, with exception to the return period as shown:

- USGS Probabilistic Seismic Hazard Map Edition: Dynamic conterminous U.S. 2014 (v4.1.1).
- Spectral Period: PGA, 0.2, 1.0, and 2.0 seconds evaluated.
- Latitude/Longitude Inputs: 32.787 Lat. / -79.937 Long.
- Time Horizon: Return period 144 years corresponding to a 50% in 100 years AEP.
- Site Class: Vs30 = 760 m/s (chosen for consistency with Section 1.5).

1.8.3. Hazard Response Spectrum Curves and OBE

Seismic hazard curves for the project site were generated for the PGA and spectral periods of 0.2, 1.0, and 2.0 seconds (Figure 15). The USGS Unified Hazard Tool utilizes seismic hazard curves to create the uniform hazard response spectrum (UHRS) curve shown in Figure 16. The UHRS curve is created (automatically by the tool) by selecting data points along each hazard curve corresponding to the 144-year return period. An OBE PGA of 0.0548g and an SA of 0.09g (at 0.2 second period) is derived utilizing the USGS Unified Hazard Tool.

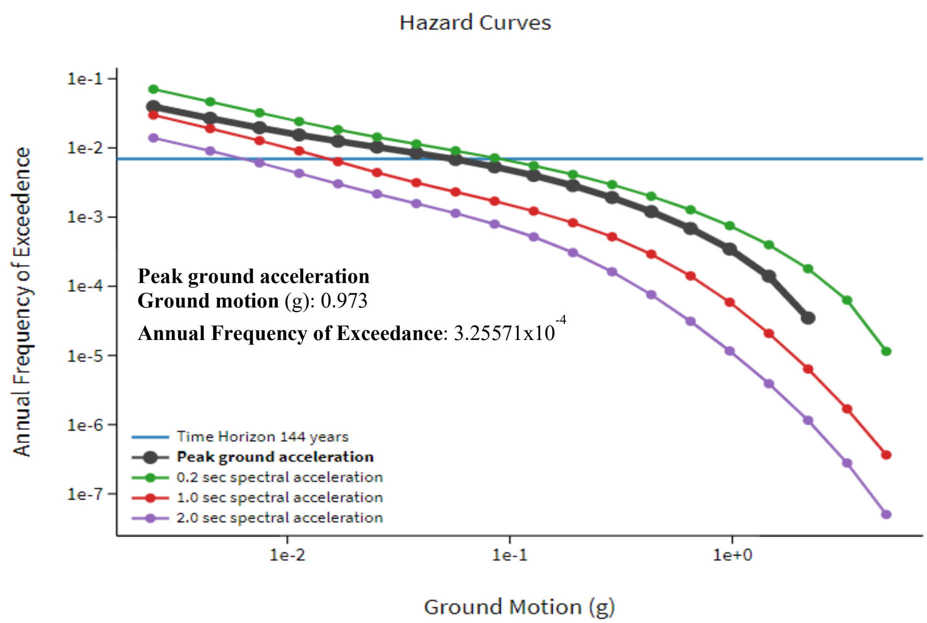

Figure 15: Site-specified seismic hazard curves showing ground motions for PGA and SA with 144-year return period.

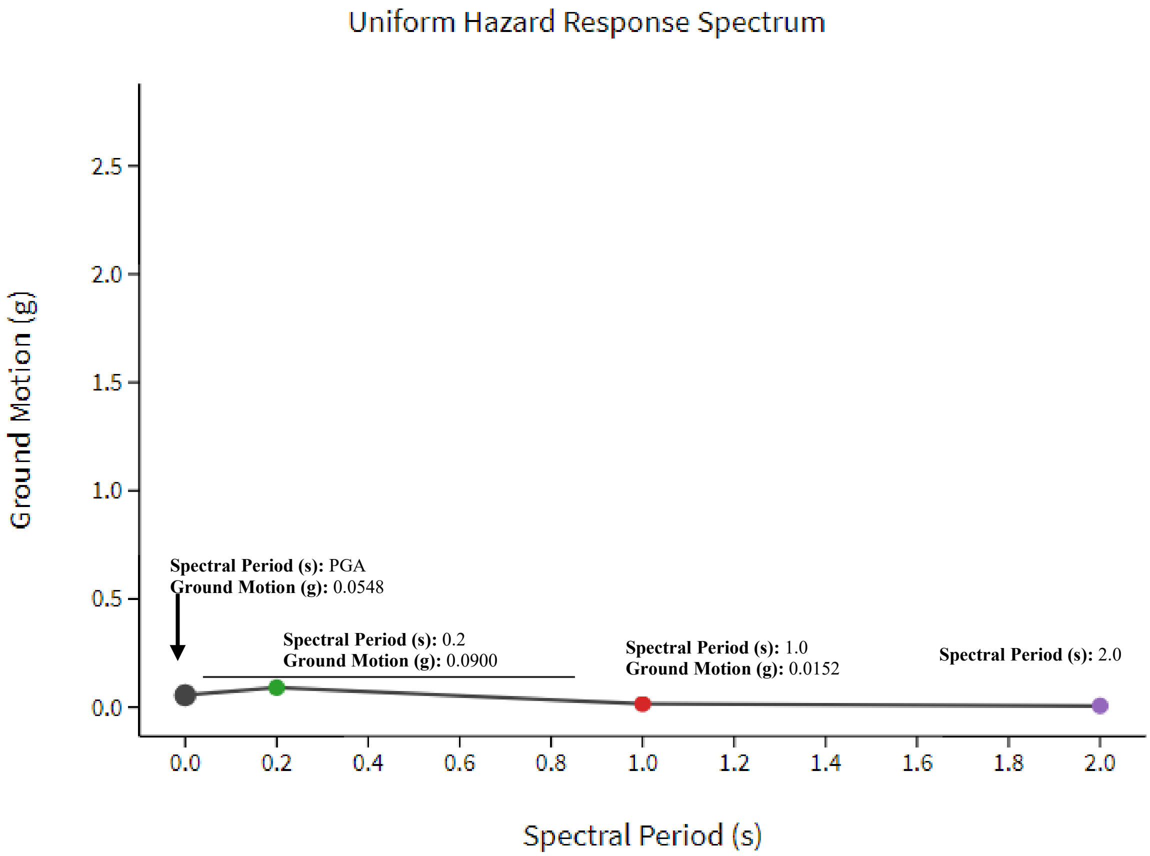

Figure 16: Site-specified uniform hazard response spectrum for the 144-year return period. PGA and SA Periods 0.2, 1.0, and 2.0 seconds shown.

1.9. Seismic Analysis Summary

1. The project site lies in an area that is subject to moderate to strong seismic activity. The largest earthquake recorded in the eastern U.S. occurred approximately 10 kilometers northwest of the project site. In accordance with ER 1110-2-1806, seismic ground motions must be accounted for in the seawall design. Deterministic methods, informed by probabilistic methods, were used to determine the design ground motion.

2. One ground motion was evaluated: Mw = 7.3 “MCE Charleston Earthquake”. The ground motion prediction equation of Boore and Atkinson (2006) was used with OPENSHA software to evaluate the median and median +1 σ PGA and SA from this event. Comparison of attenuation, spectral acceleration, and spectral velocity curves reveal significant attenuation, spectral acceleration, and spectral velocity. The Charleston earthquake of 1886 should be utilized for Maximum Design Earthquake.

3. Figure 13 indicates the highest spectral acceleration being +1 σ spectral acceleration = 1.261g at a period of 0.3 seconds. This spectral acceleration corresponds to a 7.3 Mw

Charleston earthquake event, templated to occur within a radius of 25 kilometers from the site. An OBE PGA of 0.0548g and an SA of 0.09g at 0.2 second period is also designated for the project site for 144-year return period.

4. Figure 12 compares the peak ground acceleration (g) between the USGS seismic hazard map (Figure 2) and the probabilistic seismic hazard curve (Figure 8). Figure 2 indicates a range of 0.6 to 0.8g PGA while Figure 8 indicates a higher PGA of 0.973g with the greatest spectral period being 1.3972g at 0.2 spectral period (Figure 9). The higher ground motion of 1.3972g at 0.2 spectral period should be taken into account when designing.

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Attachment 2: Top of Cooper Marl and Existing Boring Locations

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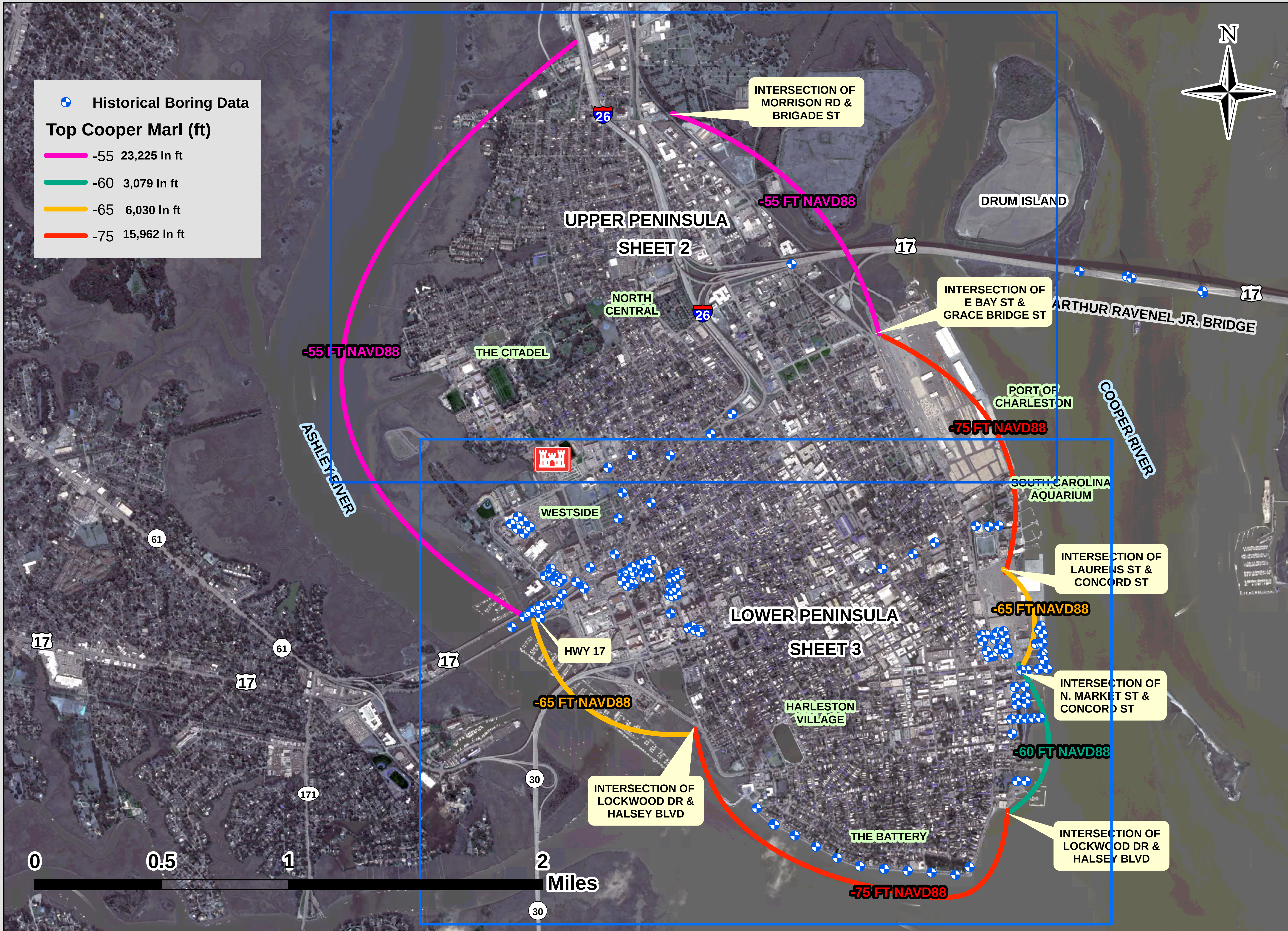

US Army Corps of Engineers
Wilmington District

Geotechnical Data Compiled By:
Stephen J. Fabian
Wilmington District, USACE
Map Date: APR 2020
Map File Name: Y:\Common\ECP\EG\CPS_CSRM\GIS

Horizontal Datum: NAD83
Vertical Datum: NAVD 88, ft
Imagery: USACE, 2010
Primary Map Scale: 1:22000

CHARLESTON, SOUTH CAROLINA PENINSULA SHEET 1 OF 3

Attachment 2, Pg. 1

US Army Corps of Engineers
Wilmington District

Geotechnical Data Compiled By:
Stephen J. Fabian
Wilmington District, USACE

Map Date: APR 2020

Map File Name: Y:\Common\ECP\EGI\CPS_CSRM\GIS

Horizontal Datum: NAD83
Vertical Datum: NAVD 88, ft
Imagery: USACE, 2010
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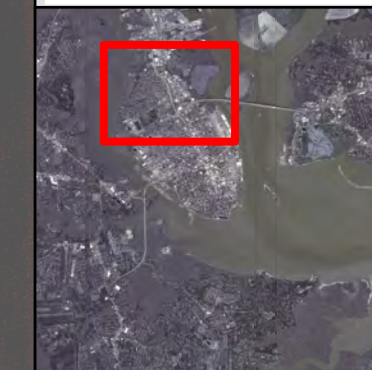
CHARLESTON, SOUTH CAROLINA
UPPER PENINSULA

SHEET 2 OF 3

Attachment 2, Pg. 2

US Army Corps of Engineers
Wilmington District



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Wilmington District, USACE
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CHARLESTON, SOUTH CAROLINA UPPER PENINSULA

SHEET 3 OF 3

Attachment 2, Pg. 3

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Attachment 3: T-wall Analyses

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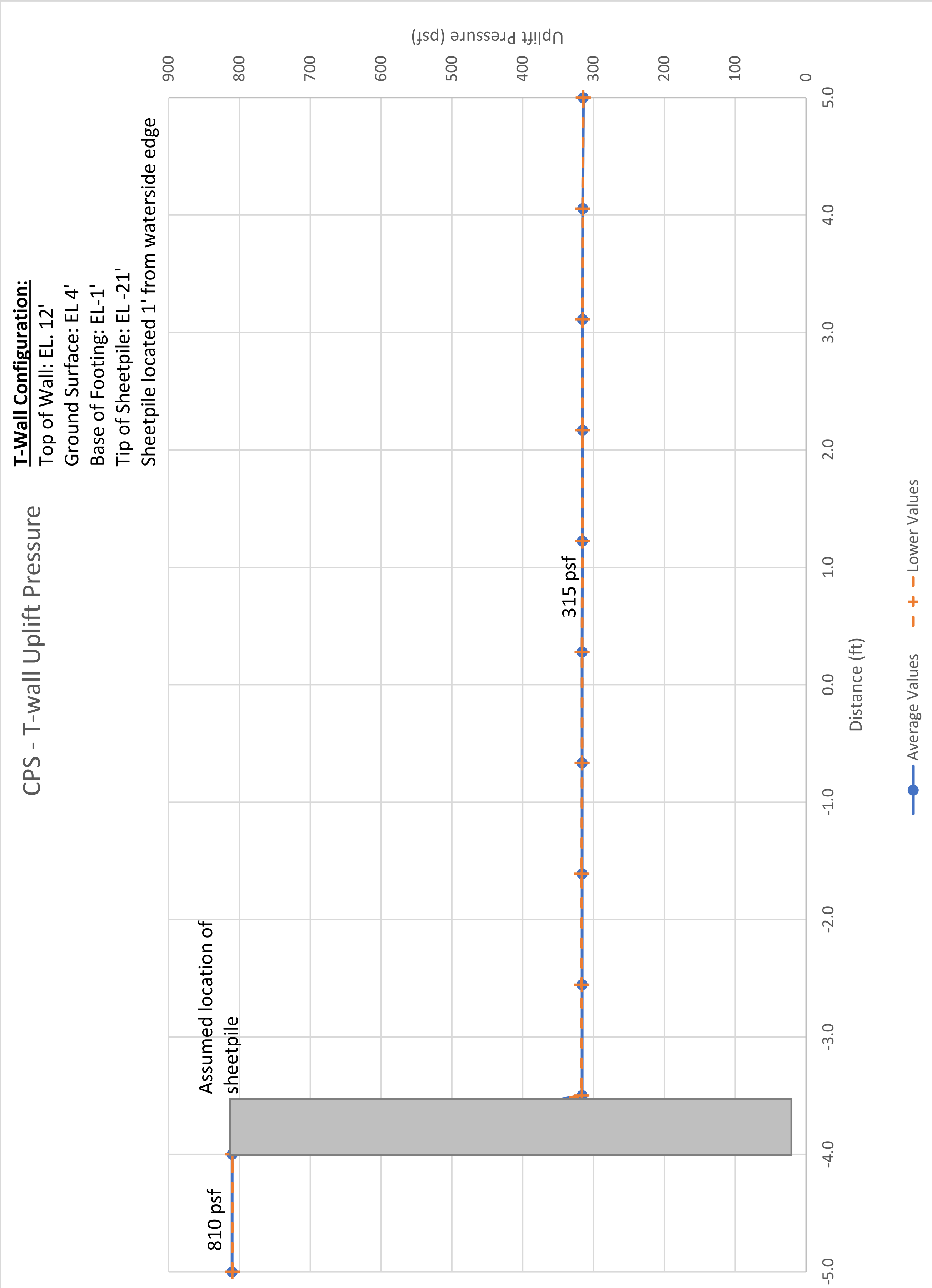

CPS - T-wall Uplift Pressure
T-Wall Configuration:
Top of Wall: EL. 12'
Ground Surface: EL 4'
Base of Footing: EL-1'
Tip of Sheetpile: EL -21'
Sheetpile located 1' from waterside edge
Uplift Pressure (psf)
900
800
700
600
500
400
300
200
100
0
315 psf
Assumed location of sheetpile
810 psf
5.0
4.0
3.0
2.0
1.0
0.0
-1.0
-2.0
-3.0
-4.0
-5.0
Distance (ft)
Average Values
Lower Values

Charleston Peninsula Study - T-wall Seepage
Jason Inskeep / Kurt Heckendorf
06/30/2020

Location: Lockwood

Ground Surface Elevation	4	ft NAVD88
Unit Wt. of Surficial soils	110	pcf
Bouyant Unit Wt. of Surficial Soils	47.6	pcf
Unit Wt. Water	62.4	pcf
Critical Gradient	0.76	

Average Values

Seepage

0 sec

Y (ft)	X (ft)	Water Total Head (ft)	Excess Head	Pore Pressure	Gradient	FOS Gradient	FOS Total Stress
2.00	3.95	4.00	0.00279	125.04	0.0014	548	1.76
2.00	4.00	4.00	0.00283	125.04	0.0014	538	1.76
2.00	4.94	4.01	0.00744	125.26	0.0037	205	1.76
2.00	**5.00**	**4.01**	**0.00772**	**125.28**	**0.0039**	**198**	**1.76**
2.00	**5.97**	**4.01**	**0.01005**	**125.36**	**0.0050**	**152**	**1.75**
2.00	6.94	4.01	0.01040	125.32	0.0052	147	1.75
2.00	7.91	4.01	0.01021	125.24	0.0051	149	1.75
2.00	8.91	4.01	0.00976	125.14	0.0049	156	1.75
2.01	9.90	4.01	0.00923	125.04	0.0046	165	1.75
2.01	10.91	4.01	0.00869	124.94	0.0044	175	1.76
2.01	11.93	4.01	0.00819	124.84	0.0041	186	1.76
2.01	12.94	4.01	0.00772	124.74	0.0039	197	1.76
2.01	13.95	4.01	0.00729	124.65	0.0037	208	1.76
2.01	14.96	4.01	0.00690	124.56	0.0035	220	1.76
2.01	15.98	4.01	0.00654	124.47	0.0033	232	1.76
2.01	16.99	4.01	0.00622	124.38	0.0031	244	1.76
2.01	17.86	4.01	0.00597	124.29	0.0030	254	1.76

Uplift

0 sec

X (ft)	Water Pressure (psf)
-5.0	810.37
-4.0	810.23
-3.5	316.34
-2.6	316.32
-1.6	316.27
-0.7	316.19
0.3	316.06
1.2	315.90
2.2	315.69
3.1	315.42
4.1	315.08
5.0	314.43

With Blocked Exit

Blocked Seepage 13' beyond footing on dryside.

0 sec

X (ft)	Water Total Head (ft)	Δ
3.95	4.00	0.00
4.00	4.00	0.00
4.94	4.01	0.00
5.00	4.01	0.00
5.97	4.02	0.01
6.94	4.02	0.01
7.91	4.02	0.01
8.91	4.02	0.01
9.90	4.02	0.01
10.91	4.02	0.01
11.93	4.02	0.01
12.94	4.02	0.01
13.95	4.02	0.01
14.96	4.02	0.02
15.98	4.03	0.02
16.99	4.03	0.03
17.86	4.04	0.03

0 sec

X (ft)	Water Pressure (psf)	Δ
-5.0	810.40	0.02
-4.0	810.26	0.03
-3.5	317.53	1.19
-2.6	317.52	1.19
-1.6	317.46	1.19
-0.7	317.37	1.19
0.3	317.24	1.18
1.2	317.06	1.17
2.2	316.84	1.15
3.1	316.55	1.13
4.1	316.16	1.08
5.0	315.39	0.96

Charleston Peninsula Study - T-wall Seepage
Jason Inskeep / Kurt Heckendorf
06/30/2020

Location: Lockwood

Lower Values

Seepage

0 sec

Y (ft)	X (ft)	Water Total Head (ft)	Excess Head	Pore Pressure	Gradient	FOS Gradient	FOS Total Stress
2.00	3.95	4.00	0.00299	125.05	0.0015	510	1.76
2.00	4.00	4.00	0.00305	125.05	0.0015	501	1.76
2.00	4.94	4.01	0.00801	125.30	0.0040	191	1.76
2.00	**5.00**	**4.01**	**0.00830**	**125.31**	**0.0042**	**184**	**1.76**
2.00	**5.95**	**4.01**	**0.01082**	**125.41**	**0.0054**	**141**	**1.75**
2.00	6.92	4.01	0.01125	125.37	0.0056	135	1.75
2.00	7.90	4.01	0.01110	125.29	0.0056	137	1.75
2.00	8.90	4.01	0.01069	125.20	0.0054	142	1.75
2.01	9.90	4.01	0.01018	125.10	0.0051	150	1.75
2.01	10.91	4.01	0.00965	125.00	0.0048	158	1.75
2.01	11.93	4.01	0.00915	124.90	0.0046	166	1.75
2.01	12.94	4.01	0.00867	124.80	0.0044	175	1.76
2.01	13.95	4.01	0.00823	124.71	0.0041	184	1.76
2.01	14.96	4.01	0.00783	124.61	0.0039	194	1.76
2.01	15.98	4.01	0.00745	124.52	0.0037	204	1.76
2.01	16.99	4.01	0.00710	124.43	0.0036	214	1.76
2.01	17.86	4.01	0.00681	124.35	0.0034	222	1.76

Uplift

0 sec

X (ft)	Water Pressure (psf)
-5.0	810.14
-4.0	809.97
-3.5	316.42
-2.6	316.40
-1.6	316.36
-0.7	316.28
0.3	316.16
1.2	316.01
2.2	315.81
3.1	315.56
4.1	315.22
5.0	314.57

With Blocked Exit

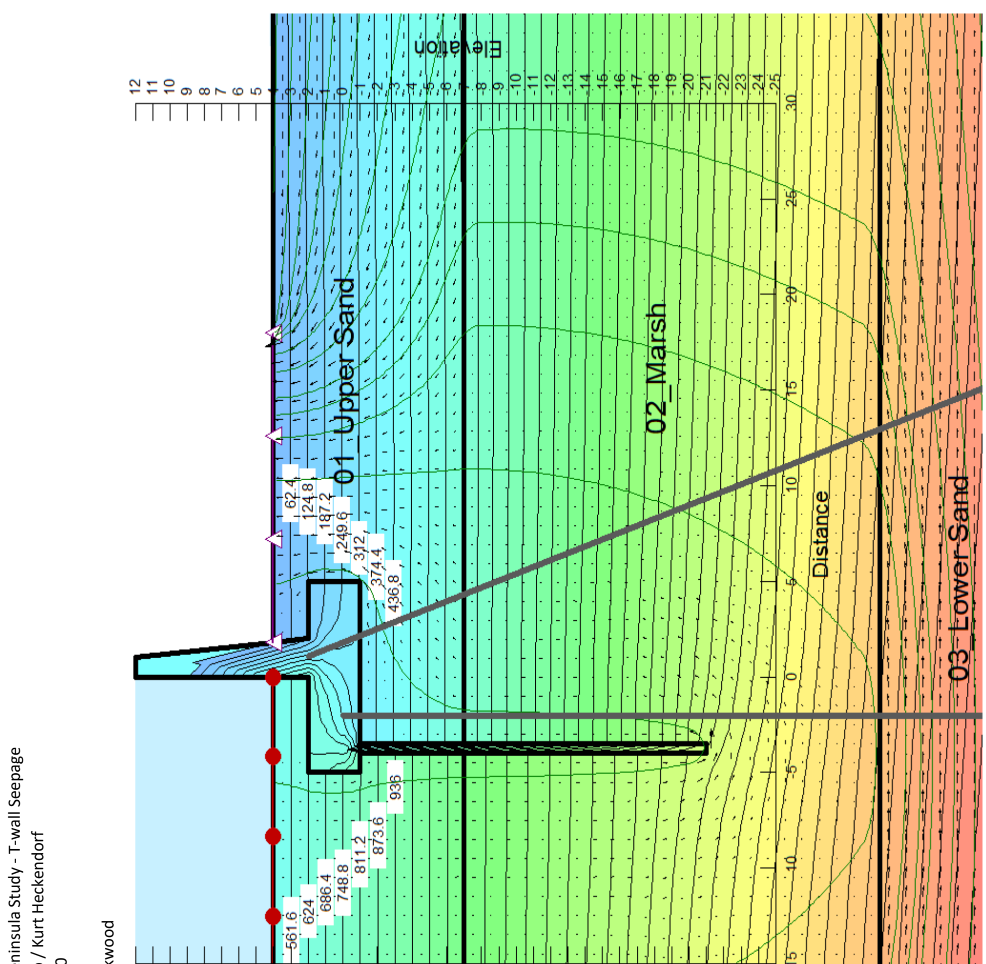
Blocked Seepage 13' beyond footing on dryside.

0 sec

X (ft)	Water Total Head (ft)	Δ
3.95	4.01	0.00
4.00	4.01	0.00
4.94	4.01	0.01
5.00	4.01	0.01
5.95	4.02	0.01
6.92	4.02	0.01
7.90	4.02	0.01
8.90	4.02	0.01
9.90	4.02	0.01
10.91	4.02	0.01
11.93	4.02	0.01
12.94	4.02	0.01
13.95	4.02	0.02
14.96	4.03	0.02
15.98	4.03	0.02
16.99	4.03	0.03
17.86	4.04	0.03

0 sec

X (ft)	Water Pressure (psf)	Δ
-5.0	810.17	0.02
-4.0	810.00	0.03
-3.5	318.61	2.19
-2.6	318.59	2.18
-1.6	318.53	2.18
-0.7	318.44	2.16
0.3	318.30	2.14
1.2	318.11	2.10
2.2	317.87	2.05
3.1	317.54	1.98
4.1	317.09	1.87
5.0	316.17	1.60


Charleston Peninsula Study - T-wall Seepage
Jason Inskeep / Kurt Heckendorf
06/30/2020
Location: Lockwood
01_Upper Sand
02_Marsh
03_Lower Sand
Distance
Elevation
62.4
124.8
187.2
249.6
312
374.4
436.8
561.6
624
686.4
748.8
811.2
873.6
936

Charleston Peninsula Study - Lockwood Soil Profiles
06/17/2020

Comparison Sheet
Jason Inskeep

Horizon	Bottom of Upper Sand	Top of Lower Sand	Bottom of Lower Sand	Top of Cooper Marl
LRV	5.5	36	42	48
HRV	11	39.5	46	56
Measurments	9	39	42	48
	8.5	38	44	52
	8	37	43.5	52
	8	37	46	55
	11	39	44	56
	9.5	39.5	45	56
	9.5	38.5	43.5	55
	5.5	36	43	55
	7.5	38	44	56
Average Value	8.5	38.0	43.9	53.9
Sample Std Dev.	1.5	1.1	1.1	2.7
2-sigma	3.1	2.3	2.3	5.4
3-sigma	4.6	3.4	3.4	8.1
LCV	3.9	34.6	40.5	45.7
HCV	13.1	41.4	47.3	62.0

Pumpstation

	Bottom of Upper Sand	Top of Lower Sand	Bottom of Lower Sand	Top of Cooper Marl
LRV	11	34	44	44
HRV	12	36	44	46
Measurments	11	34	44	45
	12	36	44	44
	11	36	44	46
Average Value	11.3	35.3	44.0	45.0
Sample Std Dev.	0.6	1.2	0.0	1.0
2-sigma	1.2	2.3	0.0	2.0
3-sigma	1.7	3.5	0.0	3.0
LCV	9.6	31.9	44.0	42.0
HCV	13.1	38.8	44.0	48.0

22 Westedge

	Bottom of Upper Sand	Top of Lower Sand	Bottom of Lower Sand	Top of Cooper Marl
LRV	8	38	48	49.5
HRV	13.5	43	50	52.5
Measurments	12	38.5	48	49.5
	12	40	49	51
	9	38.5	50	50
	13.5	38.5	50	50
	8	43	50	52.5
	11.5	38		
Average Value	11	39.4	49.4	50.6
Sample Std Dev.	2.1	1.9	0.9	1.2
2-sigma	4.1	3.8	1.8	2.4
3-sigma	6.2	5.6	2.7	3.6
LCV	4.8	33.8	46.7	47.0
HCV	17.2	45.1	52.1	54.2

Thickness

	U Sand	U Sand	U Sand
Lower	4	10	5
Upper	13	13	17
Average	8	11	11
	Muck	Muck	Muck
Lower	30.7	22.3	29.0
Upper	28.3	25.7	27.8
Average	29.5	24.0	28.4
	L Sand	L Sand	L Sand
Lower	5.9	12.1	12.9
Upper	5.9	5.2	7.0
Average	5.9	8.7	10.0

		Thickness			Elevations			
Model		U Sand	Muck	L Sand	**Ground Surface**	**U Sand**	**Marsh/Muck**	**L Sand**
1 (Ave)	**Ave**	**11**	**24**	**9**	**4**	**-7**	**-31**	**-40**
(LCV)	Min	4	22	6	4	0	-22	-28
(HCV)	Max	17	30	13	4	-13	-43	-56
	Measured/reported values all fall between average and the upper values calculated here.							
Lower REV01		17	42	10	4	**-13**	**-55**	**-65**

*refers to base elevation of formations

as stated in reports {Landfill materials are assumed to be sand and gravelly sand as indicated in the boring logs. Lower Sand layer is said to terminate at the top of Marl in all 3 reports.}

	Lockwood: Bottom of Upper Sand	Top of Lower Sand	Top of Cooper Marl	Pumpstation: Bottom of Upper Sand	Top of Lower Sand	Top of Cooper Marl	22 Westedge: Bottom of Upper Sand	Top of Lower Sand	Top of Cooper Marl
	7	36	50	7	40	45	8	38	50
	10	39	56	10		50	12	43	
Mins		26	11		30	5		26	7
Max		32	20		33	10		35	12

U Sand Bottom Elevation {Assumes ground surface at approximately 4 ft. NAVD88 based on Ryan's analysis and report data.}

	Lockwood	Pumpstation	22 Westedge
Mins	-3	-3	-4
Max	-6	-6	-8

L Sand Bottom Elevation {Assumes ground surface at approximately 4 ft. NAVD88 based on Ryan's analysis and report data.}

	Lockwood	Pumpstation	22 Westedge
Mins	-32	-36	-34
Max	-35		-39

Marl Elevation {Assumes ground surface at approximately 4 ft. NAVD88 based on Ryan's analysis and report data.}

	Lockwood	Pumpstation	22 Westedge
Mins	-46	-41	-46
Max	-52	-46	

Charleston Peninsula Study - Lockwood Soil Profiles
06/17/2020
Jason Inskeep

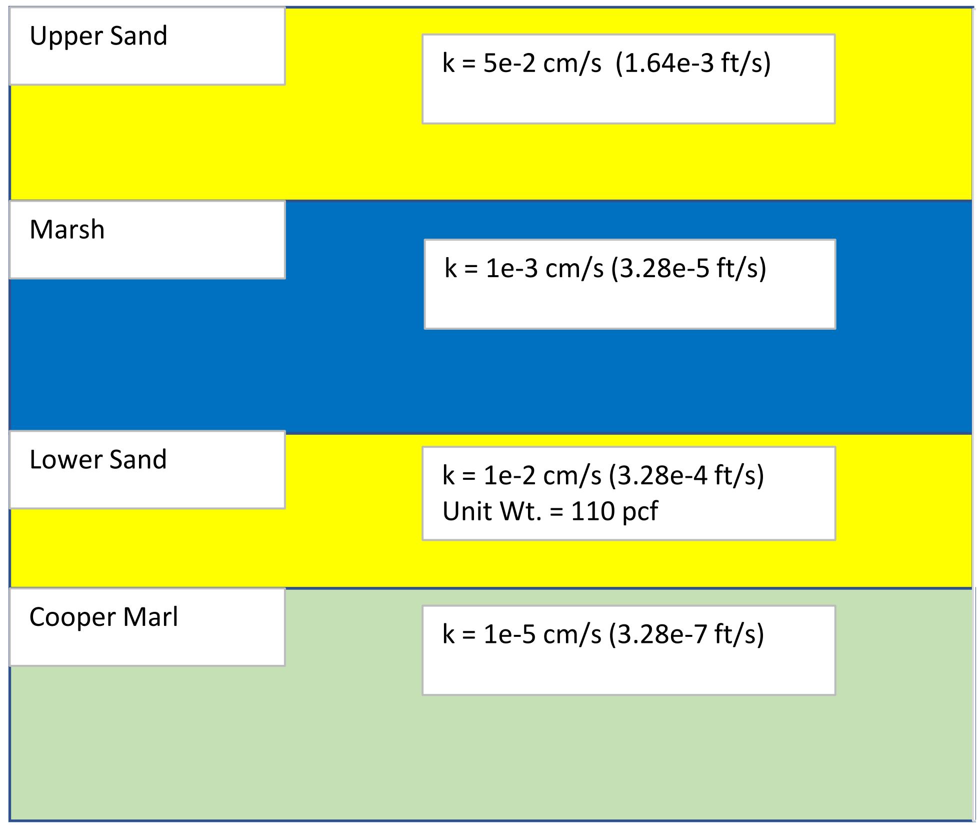
Permeability Values from USBR	cm/sec		m/day		m/sec		ft/sec
Fine to Medium Sand	1.15E-03	1.15E-02	1.00E+00	1.00E+01	1.15E-05	1.15E-04	3.77E-04
Clean Sand and Gravel			1.00E+02		1.15E-03		3.77E-03
Silt and Clay	1.15E-04	1.15E-05	1.00E-01	1.00E-02	1.15E-06	1.15E-07	3.77E-07
Cooper Marl	1.15E-07	1.15E-08	1.00E-04	1.00E-05	1.15E-09	1.15E-10	3.77E-10

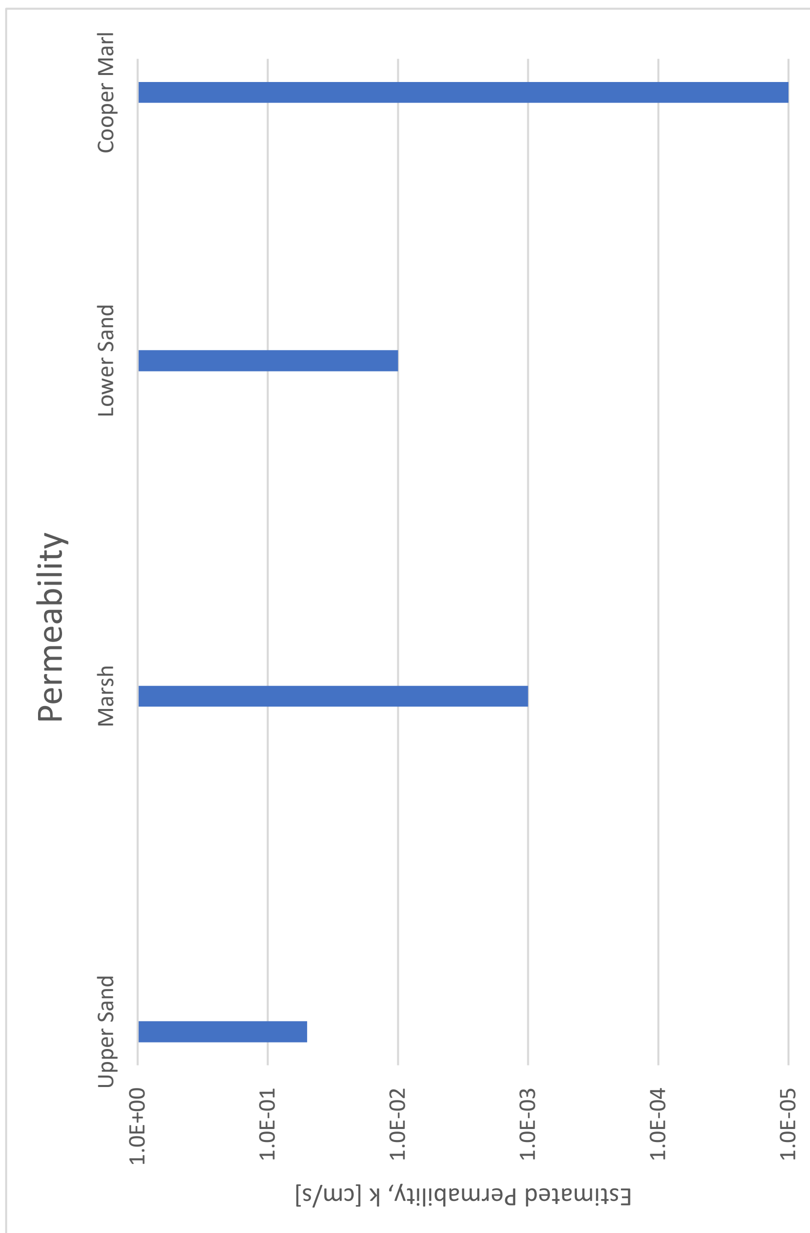
Dewatering and Groundwater Control (Table in Chapt. 3)							
Uniform Sand (Upper and Lower Sand)	2.00E-01	5.00E-03			2.00E-03	5.00E-05	1.64E-04
Silty Sand (Upper and Lower Sand)	5.00E-03	1.00E-03			5.00E-05	1.00E-05	3.28E-05
Clayey Sand (Upper and Lower Sand)	1.00E-03	1.00E-04			1.00E-05	1.00E-06	3.28E-06
Silt (Marsh Deposit)	1.00E-04	5.00E-05			1.00E-06	5.00E-07	1.64E-06
Clay (Marl and Marsh Deposit)	1.00E-05	1.00E-08			1.00E-07	1.00E-10	3.28E-10

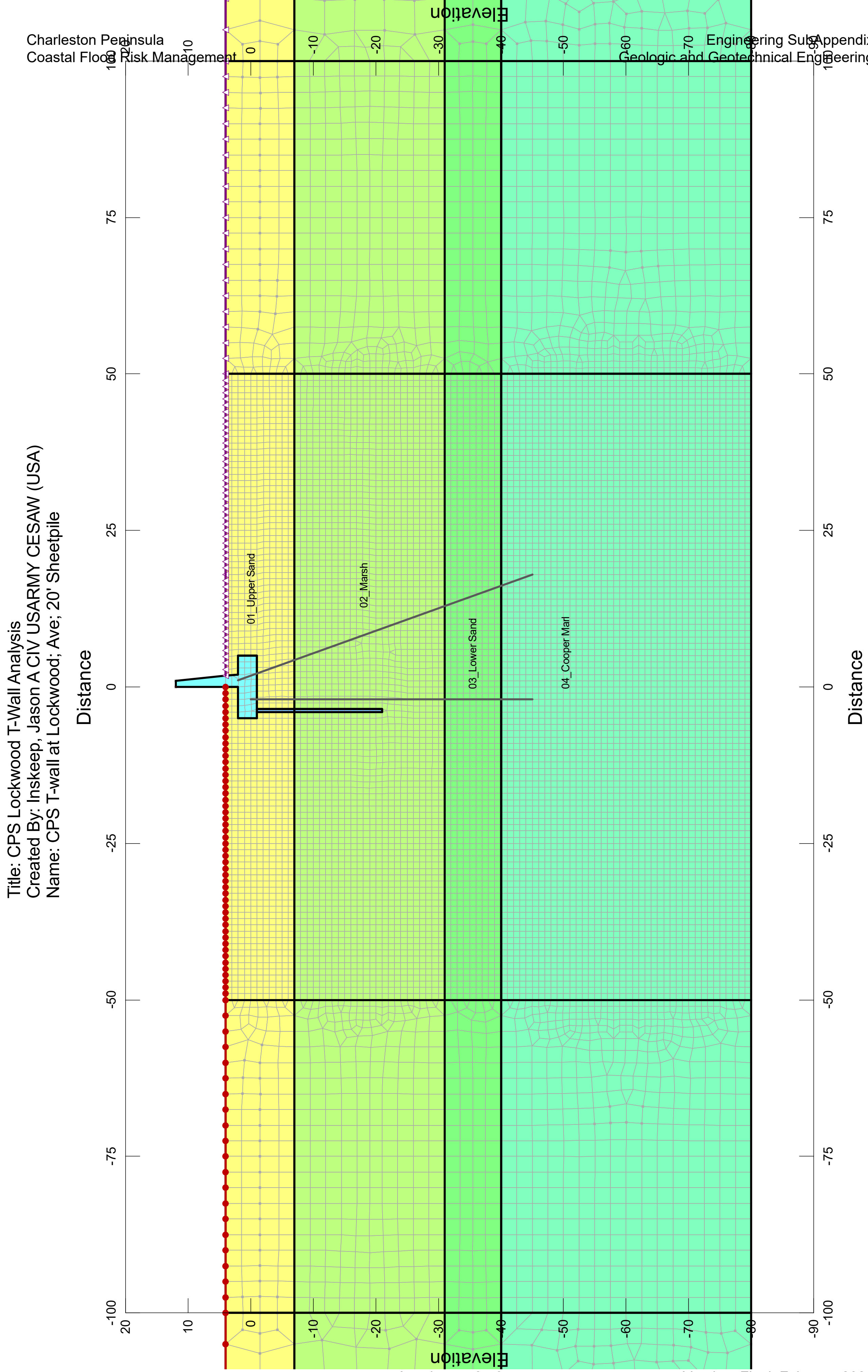
Dewatering and Groundwater Control (Figures 3.7a-3.7c; Prugh method based on density/consistency from CPT)							
Clean Loose Sands (Upper Sand)	1.00E-01	2.00E-02			1.00E-03	2.00E-04	6.56E-04
Well Graded, Loose, Sand (Upper)	4.00E-02	2.00E-02			4.00E-04	2.00E-04	6.56E-04
Clean, Medium Dense, Sand (Lower and Upper)	4.00E-01	2.00E-02			4.00E-03	2.00E-04	6.56E-04
Well Graded, Medium Dense, Sand (Lower and Upper)	6.00E-02	1.00E-02			6.00E-04	1.00E-04	3.28E-04
Clean, Dense, Sand (Lower)	8.00E-02	1.00E-02			8.00E-04	1.00E-04	3.28E-04
Well Graded, Dense, Sand (Lower)	6.00E-02	1.00E-02			6.00E-04	1.00E-04	3.28E-04

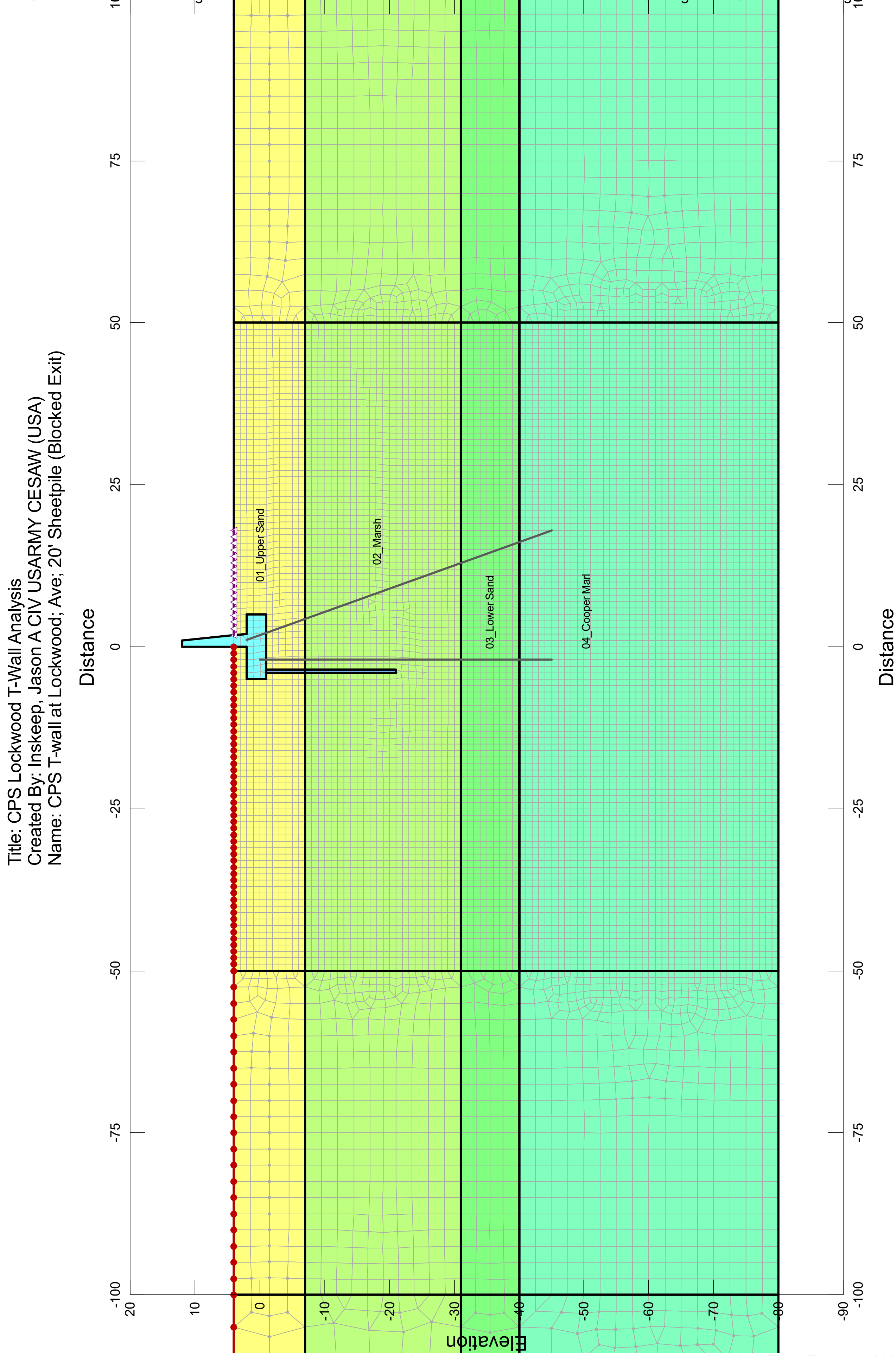
Duncan		
Fine Sand	1.00E-01	1.00E-03
Silty Sand	1.00E-03	1.00E-05
Silt	1.00E-05	1.00E-07
Clay	1.00E-07	

Permeability	m/s	cm/s	ft/s
Upper Sand	5.0E-04	5.0E-02	1.64E-03
Marsh	1.0E-05	1.0E-03	3.28E-05
Lower Sand	1.0E-04	1.0E-02	3.28E-04
Cooper Marl	1.0E-07	1.0E-05	3.28E-07




Permeability
Upper Sand
Marsh
Lower Sand
Cooper Marl
1.0E+00
1.0E-01
1.0E-02
1.0E-03
1.0E-04
1.0E-05
Estimated Permability, k [cm/s]


Title: CPS Lockwood T-Wall Analysis
Created By: Inskeep, Jason A CIV USARMY CESAW (USA)
Name: CPS T-wall at Lockwood; Ave; 20' Sheetpile
Distance
Elevation
01_Upper Sand
02_Marsh
03_Lower Sand
04_Cooper Marl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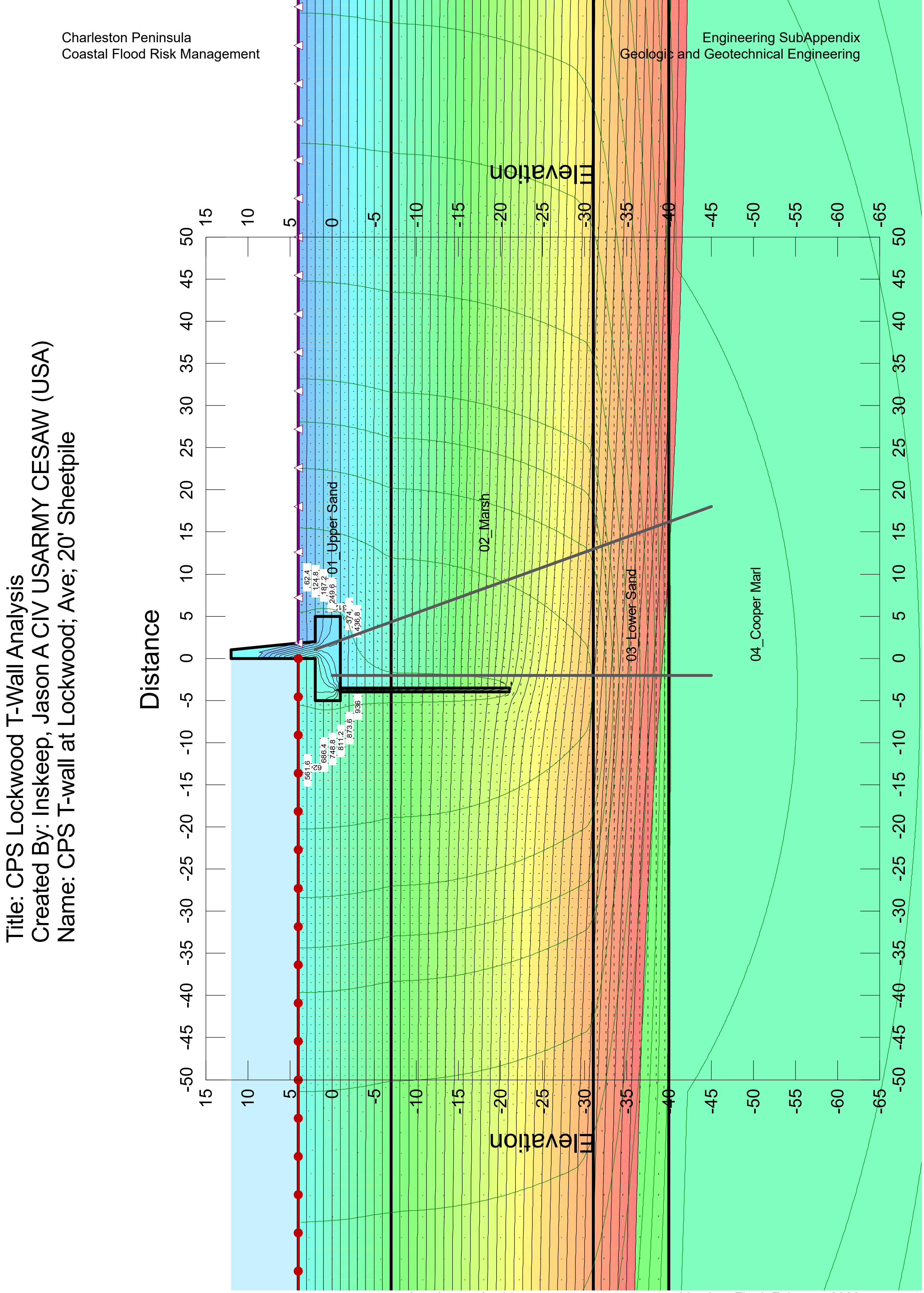
Created By: Inskeep, Jason A CIV USARMY CESAW (USA)
Name: CPS T-wall at Lockwood; Ave; 20' Sheetpile (Blocked Exit)
Distance
Elevation
01_Upper Sand
02_Marsh
03_Lower Sand
04_Cooper Marl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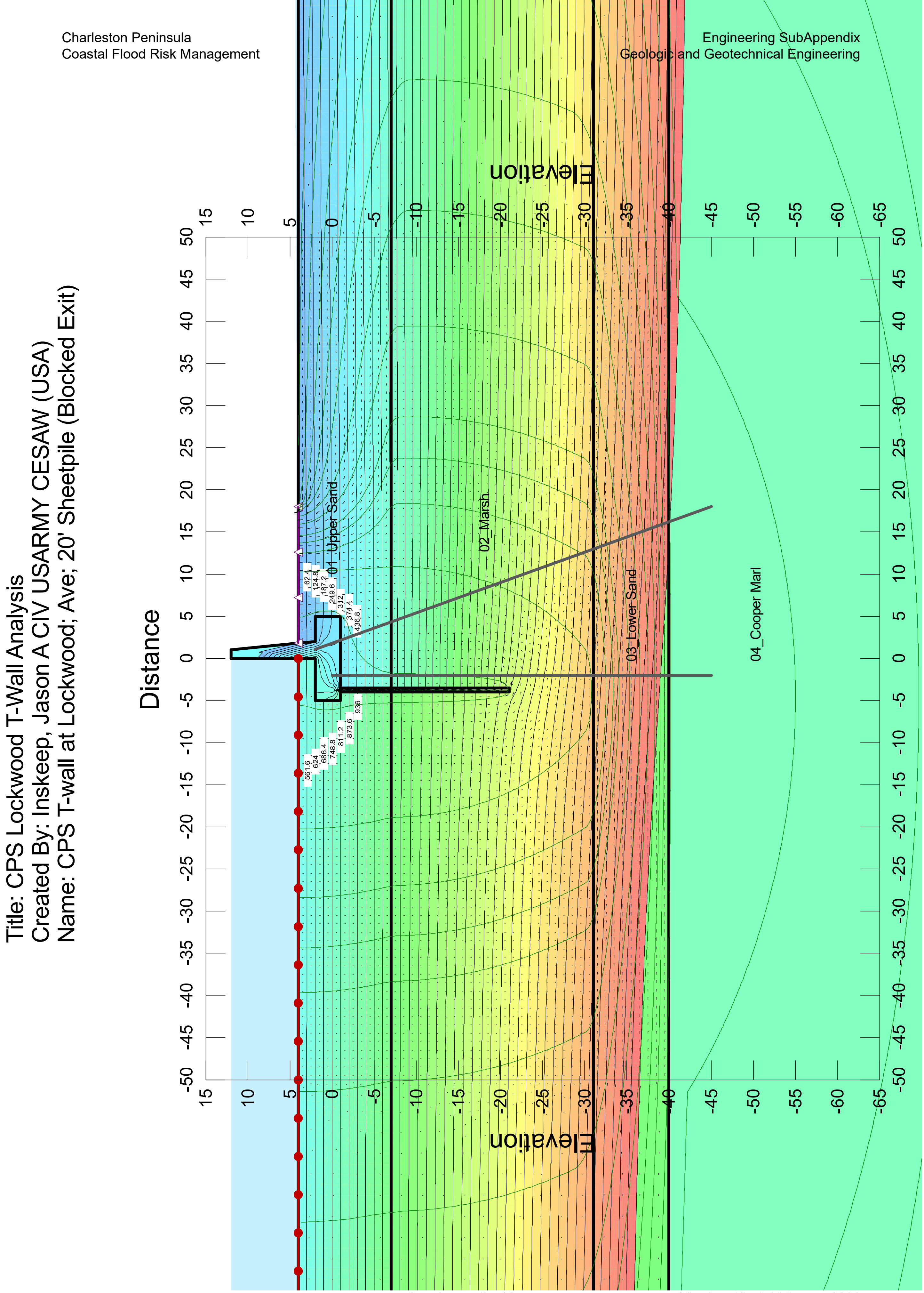
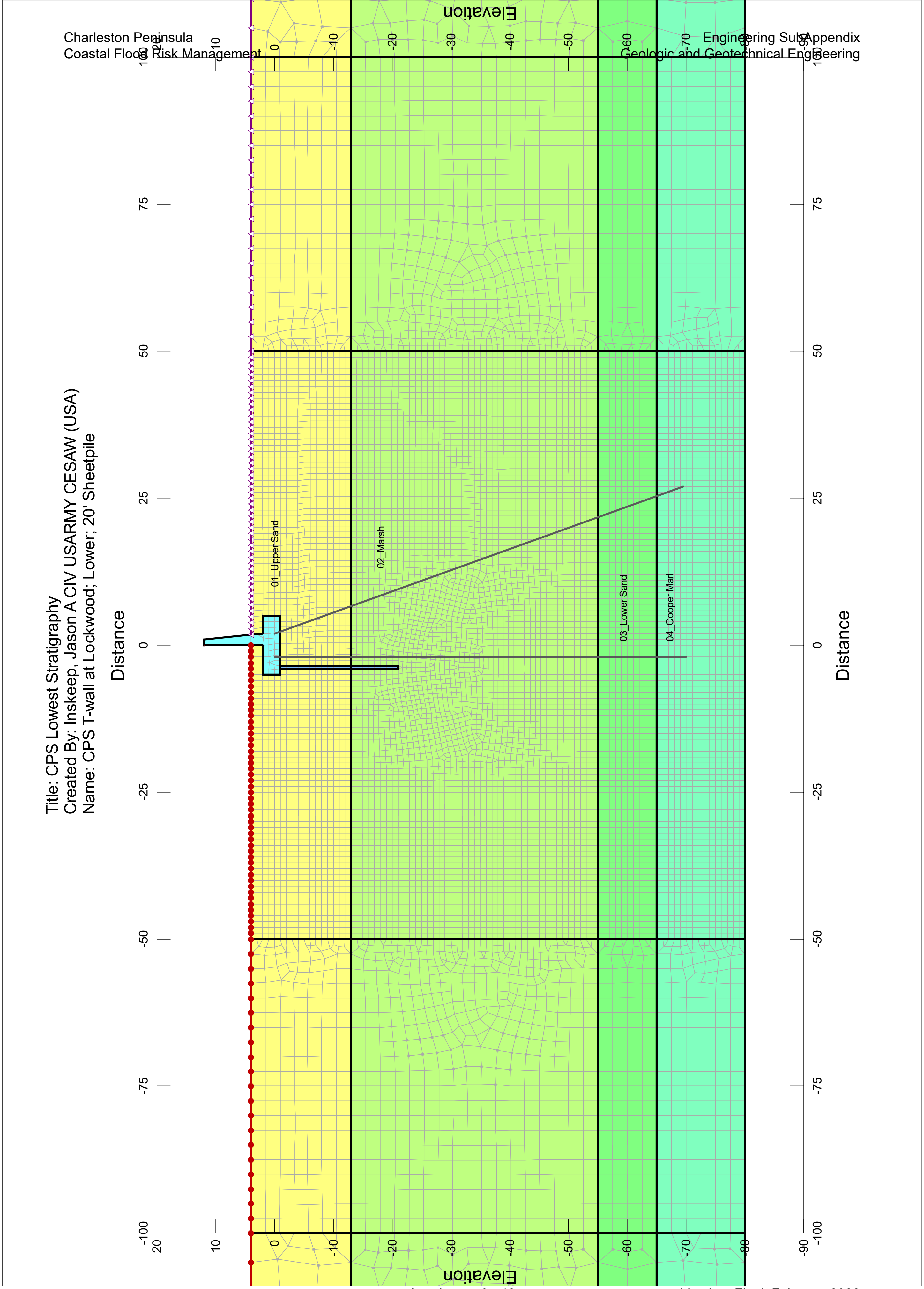
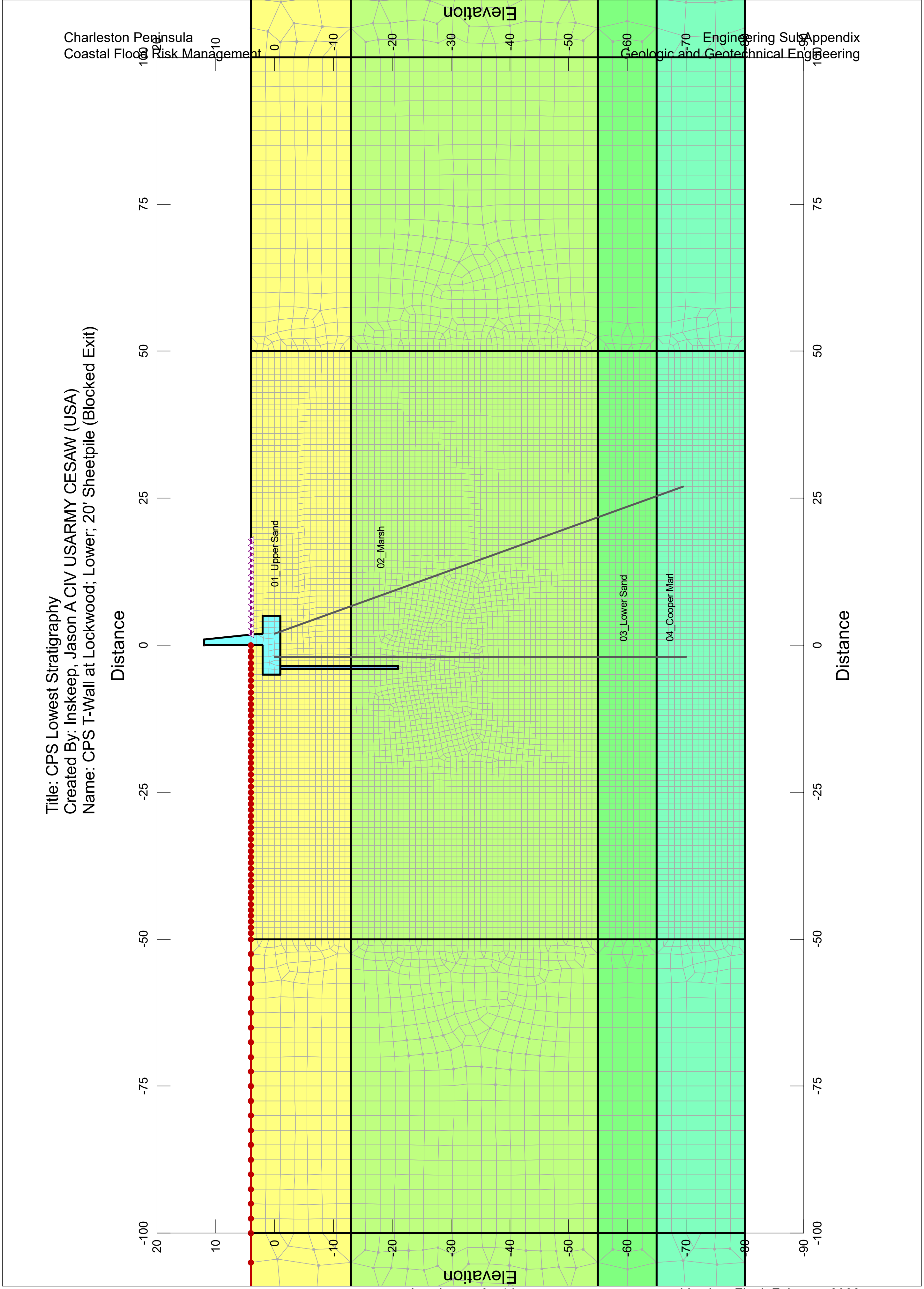
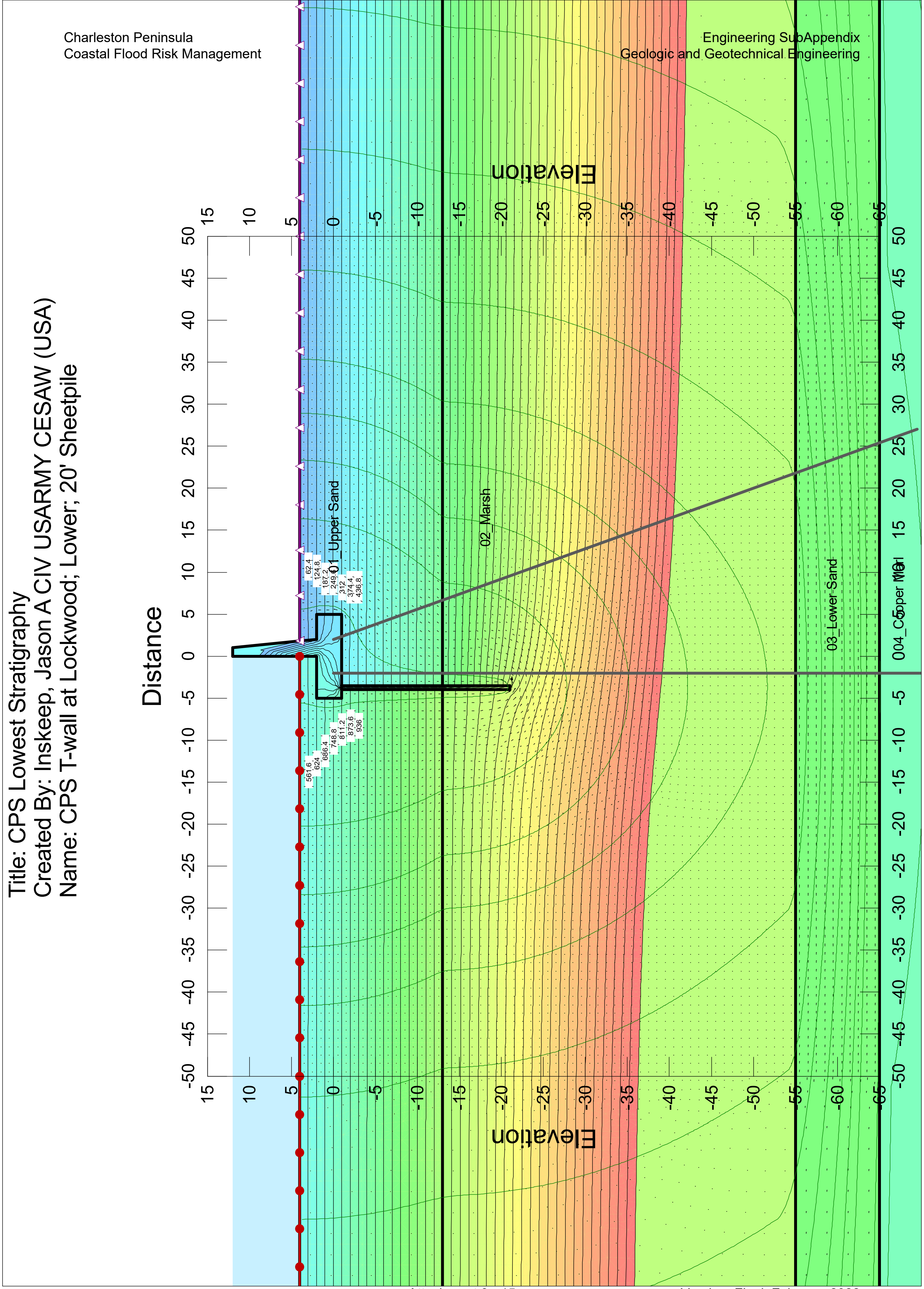
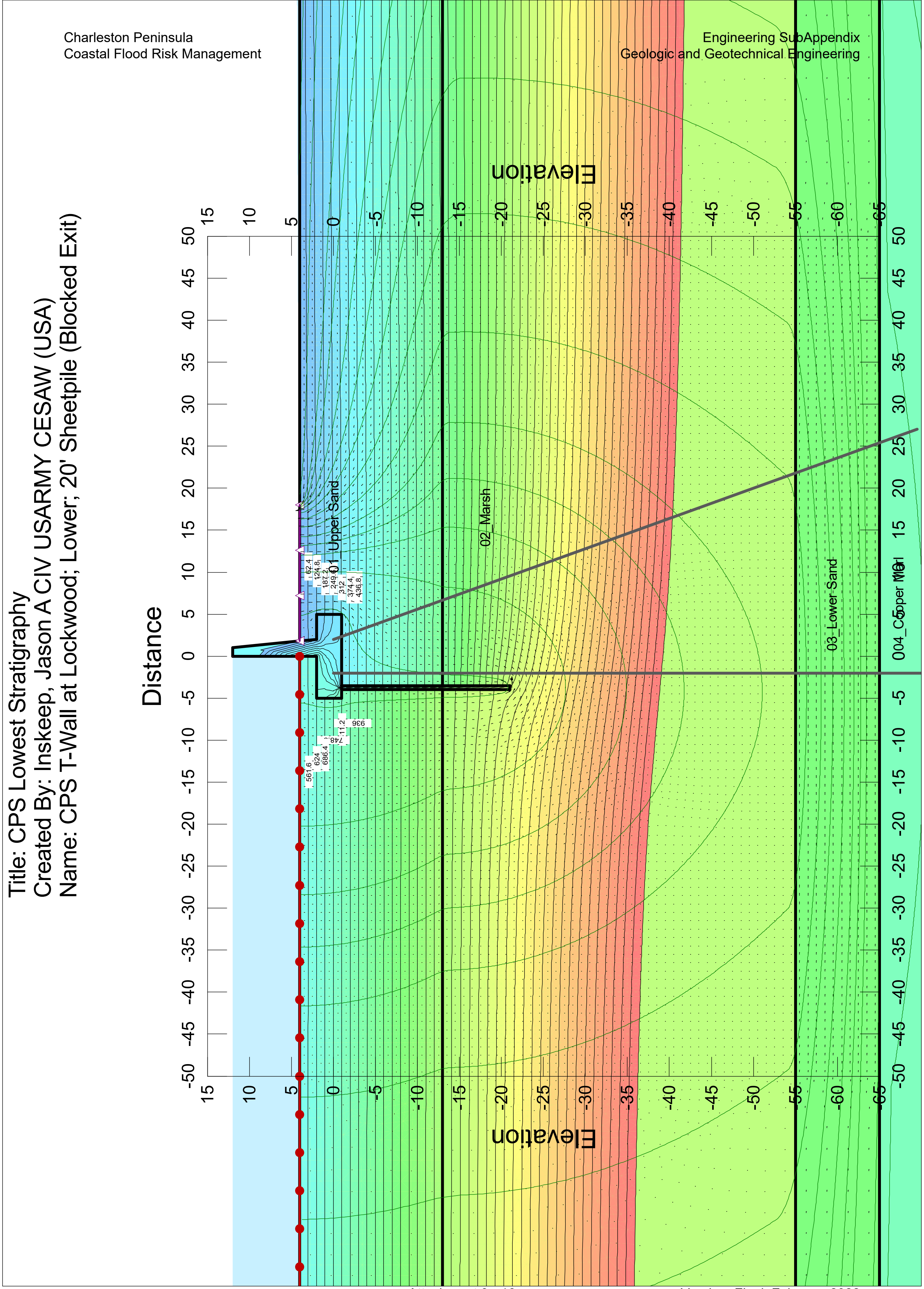
Created By: Inskeep, Jason A CIV USARMY CESAW (USA)
Name: CPS T-wall at Lockwood; Ave; 20' Sheetpile
Distance
Elevation
01_Upper Sand
02_Marsh
03_Lower Sand
04_Cooper Marl


Title: CPS Lockwood T-Wall Analysis
Created By: Inskeep, Jason A CIV USARMY CESAW (USA)
Name: CPS T-wall at Lockwood; Ave; 20' Sheetpile (Blocked Exit)
Distance
Elevation
01_Upper Sand
02_Marsh
03_Lower Sand
04_Cooper Marl
62.4
124.8
187.2
249.6
312
374.4
436.8
561.6
624
686.4
748.8
811.2
873.6
936


Title: CPS Lowest Stratigraphy
Created By: Inskeep, Jason A CIV USARMY CESAW (USA)
Name: CPS T-wall at Lockwood; Lower; 20' Sheetpile
01_Upper Sand
02_Marsh
03_Lower Sand
04_Cooper Marl
Distance
Elevation


Title: CPS Lowest Stratigraphy
Created By: Inskeep, Jason A CIV USARMY CESAW (USA)
Name: CPS T-Wall at Lockwood; Lower; 20' Sheetpile (Blocked Exit)
Distance
Elevation
01_Upper Sand
02_Marsh
03_Lower Sand
04_Cooper Marl


Title: CPS Lowest Stratigraphy
Created By: Inskeep, Jason A CIV USARMY CESAW (USA)
Name: CPS T-wall at Lockwood; Lower; 20' Sheetpile
Distance
Elevation
01_Upper Sand
02_Marsh
03_Lower Sand
004_Cooper Marl


Title: CPS Lowest Stratigraphy
Created By: Inskeep, Jason A CIV USARMY CESAW (USA)
Name: CPS T-Wall at Lockwood; Lower; 20' Sheetpile (Blocked Exit)
Distance
Elevation
01_Upper Sand
02_Marsh
03-Lower-Sand
004_Cooper Marl

Attachment 4: Pile Capacity

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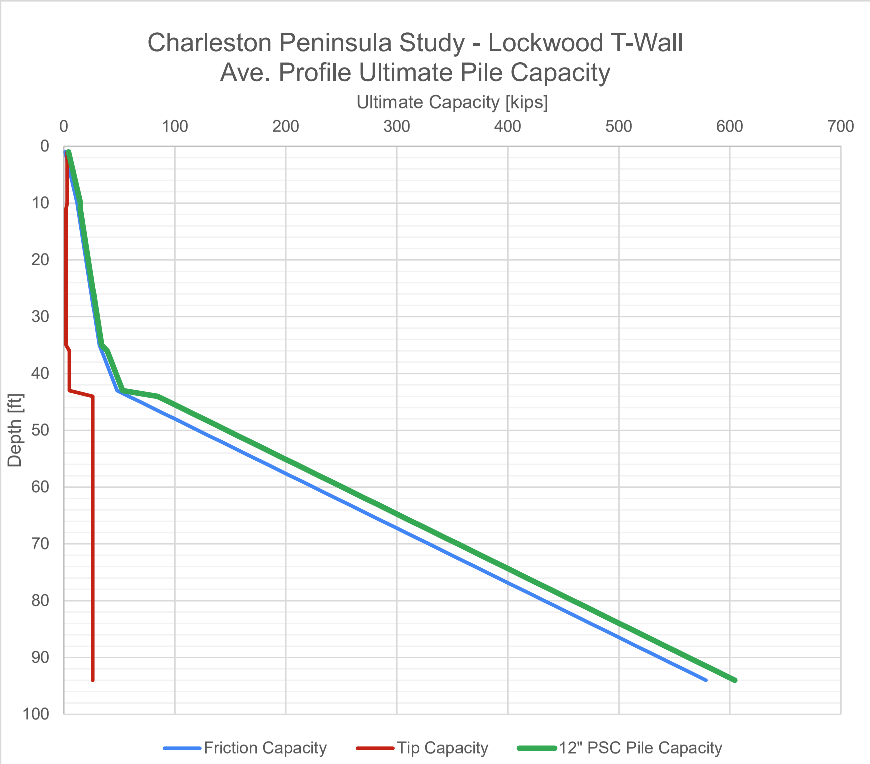
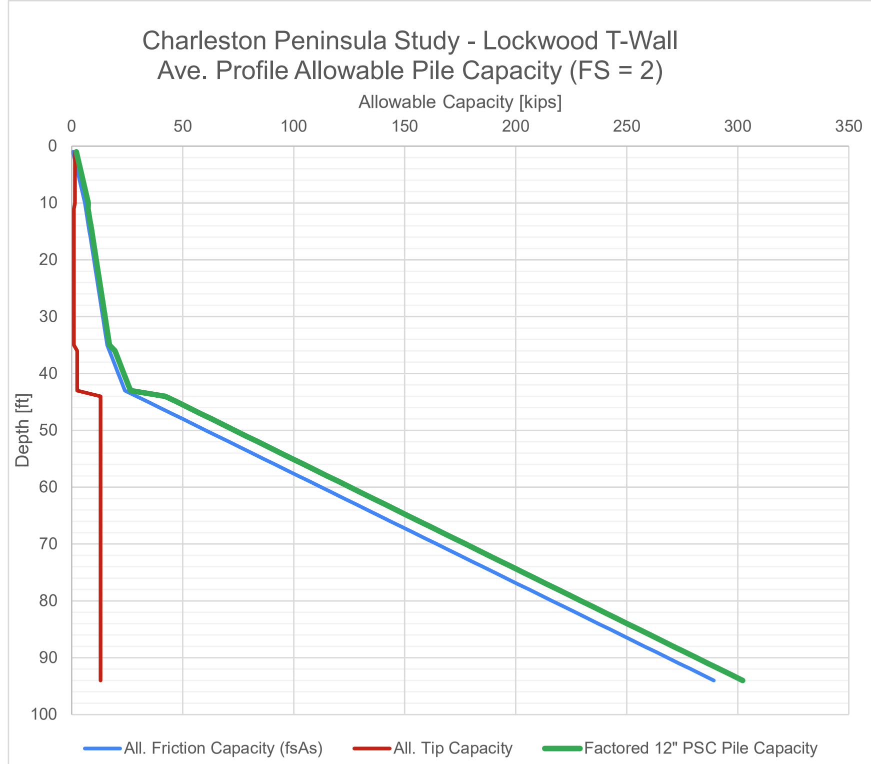
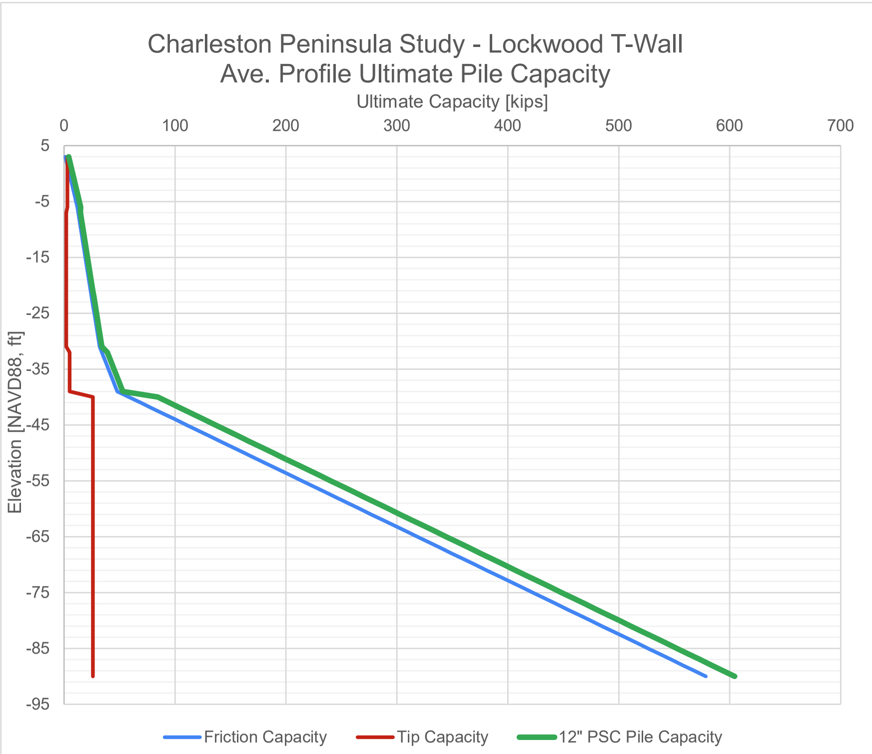
Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Average Marl Depth, Top of Marl at EL. -40 FT
Computed By: JAI
Date: 07/02/2020
Revised By: JAI
Date Revised: 07/06/2020
Reviewed By: KAH
Date Reviewed: 07/02/2020

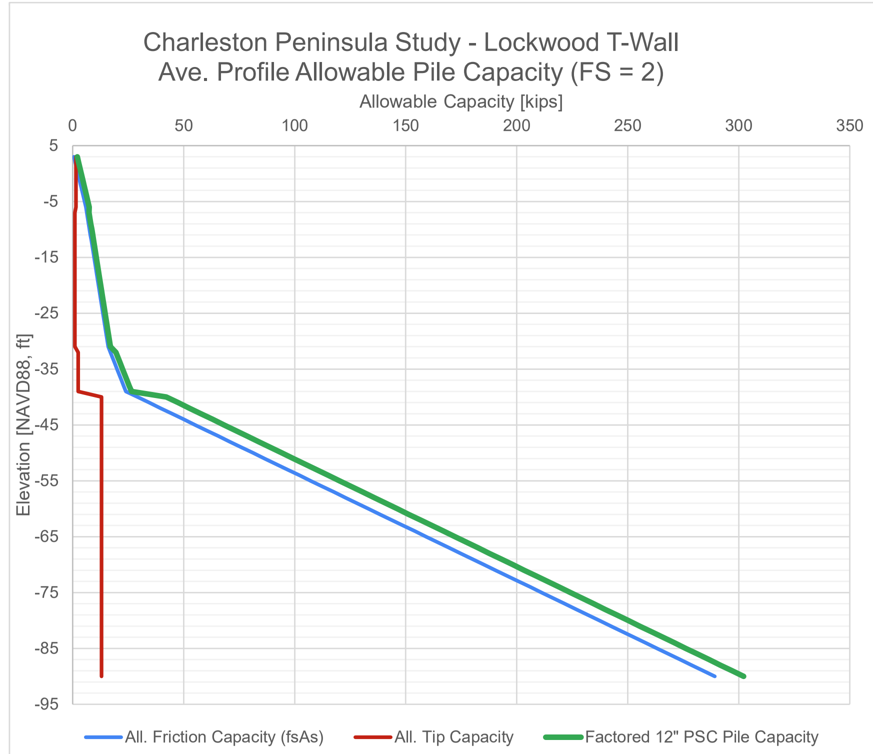




Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Average Marl Depth, Top of Marl at EL. -40 FT
Computed By: JAI
Date: 07/02/2020
Revised By: JAI
Date Revised: 07/06/2020

Reviewed By: KAH
Date Reviewed: 07/02/2020





Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Average Marl Depth, Top of Marl at EL. -40 FT
Computed By: JAI
Date: 07/02/2020
eference Project 22 Westedge

Revised By: JAI
Date Revised: 07/06/2020

Reviewed By: KAH
Date Reviewed: 07/02/2020

Boring		
Drill Rig	CPT	
Depth to Water	2	
Su/sigmaz' NC	0.22	
Factor of Safety	2	
Nc*	10	
Ground Surface	4	ft NAVD88

Pile Section

	12-in. PSC	
Pile area	144	in^2
	1.000	ft^2
Depth of Section	12	ft
Flange Width, bf	12	ft
Pile Perimeter	4.00	ft

Formation	***Su (psf)***	***Average Profile Elevation***	***Lower Profile Elevation***
Ground Surface		4	4
Upper Sand	300	4 to -7	4 to -13
Marsh/Muck	200	-7 to -31	-13 to -55
Sand	500	-31 to -40	-55 to -65
Silty Sand/Marl	2600	-40	-65

Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	12" PSC Pile Capacity	Factored 12" PSC Pile
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
4	0	Upper Sand										
3	1	Upper Sand	300	1.0	300	1.2	0.6	3000	3	1.5	4	2
2	2	Upper Sand	300	1.0	300	2.4	1.2	3000	3	1.5	5	3
1	3	Upper Sand	300	1.0	300	3.6	1.8	3000	3	1.5	7	3
0	4	Upper Sand	300	1.0	300	4.8	2.4	3000	3	1.5	8	4
-1	5	Upper Sand	300	1.0	300	6.0	3.0	3000	3	1.5	9	5
-2	6	Upper Sand	300	1.0	300	7.2	3.6	3000	3	1.5	10	5
-3	7	Upper Sand	300	1.0	300	8.4	4.2	3000	3	1.5	11	6
-4	8	Upper Sand	300	1.0	300	9.6	4.8	3000	3	1.5	13	6
-5	9	Upper Sand	300	1.0	300	10.8	5.4	3000	3	1.5	14	7
-6	10	Upper Sand	300	1.0	300	12.0	6.0	3000	3	1.5	15	8
-7	11	Marsh/Muck	200	1.0	200	12.8	6.4	2000	2	1.0	15	7
-8	12	Marsh/Muck	200	1.0	200	13.6	6.8	2000	2	1.0	16	8
-9	13	Marsh/Muck	200	1.0	200	14.4	7.2	2000	2	1.0	16	8
-10	14	Marsh/Muck	200	1.0	200	15.2	7.6	2000	2	1.0	17	9
-11	15	Marsh/Muck	200	1.0	200	16.0	8.0	2000	2	1.0	18	9
-12	16	Marsh/Muck	200	1.0	200	16.8	8.4	2000	2	1.0	19	9
-13	17	Marsh/Muck	200	1.0	200	17.6	8.8	2000	2	1.0	20	10
-14	18	Marsh/Muck	200	1.0	200	18.4	9.2	2000	2	1.0	20	10
-15	19	Marsh/Muck	200	1.0	200	19.2	9.6	2000	2	1.0	21	11
-16	20	Marsh/Muck	200	1.0	200	20.0	10.0	2000	2	1.0	22	11
-17	21	Marsh/Muck	200	1.0	200	20.8	10.4	2000	2	1.0	23	11
-18	22	Marsh/Muck	200	1.0	200	21.6	10.8	2000	2	1.0	24	12
-19	23	Marsh/Muck	200	1.0	200	22.4	11.2	2000	2	1.0	24	12
-20	24	Marsh/Muck	200	1.0	200	23.2	11.6	2000	2	1.0	25	13
-21	25	Marsh/Muck	200	1.0	200	24.0	12.0	2000	2	1.0	26	13
-22	26	Marsh/Muck	200	1.0	200	24.8	12.4	2000	2	1.0	27	13
-23	27	Marsh/Muck	200	1.0	200	25.6	12.8	2000	2	1.0	28	14
-24	28	Marsh/Muck	200	1.0	200	26.4	13.2	2000	2	1.0	28	14
-25	29	Marsh/Muck	200	1.0	200	27.2	13.6	2000	2	1.0	29	15
-26	30	Marsh/Muck	200	1.0	200	28.0	14.0	2000	2	1.0	30	15
-27	31	Marsh/Muck	200	1.0	200	28.8	14.4	2000	2	1.0	31	15
-28	32	Marsh/Muck	200	1.0	200	29.6	14.8	2000	2	1.0	32	16
-29	33	Marsh/Muck	200	1.0	200	30.4	15.2	2000	2	1.0	32	16
-30	34	Marsh/Muck	200	1.0	200	31.2	15.6	2000	2	1.0	33	17
-31	35	Marsh/Muck	200	1.0	200	32.0	16.0	2000	2	1.0	34	17
-32	36	Sand	500	1.0	500	34.0	17.0	5000	5	2.5	39	20
-33	37	Sand	500	1.0	500	36.0	18.0	5000	5	2.5	41	21
-34	38	Sand	500	1.0	500	38.0	19.0	5000	5	2.5	43	22
-35	39	Sand	500	1.0	500	40.0	20.0	5000	5	2.5	45	23
-36	40	Sand	500	1.0	500	42.0	21.0	5000	5	2.5	47	24
-37	41	Sand	500	1.0	500	44.0	22.0	5000	5	2.5	49	25
-38	42	Sand	500	1.0	500	46.0	23.0	5000	5	2.5	51	26
-39	43	Sand	500	1.0	500	48.0	24.0	5000	5	2.5	53	27
-40	44	Silty Sand/Marl	2600	1.0	2600	58.4	29.2	26000	26	13.0	84	42
-41	45	Silty Sand/Marl	2600	1.0	2600	68.8	34.4	26000	26	13.0	95	47
-42	46	Silty Sand/Marl	2600	1.0	2600	79.2	39.6	26000	26	13.0	105	53
-43	47	Silty Sand/Marl	2600	1.0	2600	89.6	44.8	26000	26	13.0	116	58
-44	48	Silty Sand/Marl	2600	1.0	2600	100.0	50.0	26000	26	13.0	126	63
-45	49	Silty Sand/Marl	2600	1.0	2600	110.4	55.2	26000	26	13.0	136	68
-46	50	Silty Sand/Marl	2600	1.0	2600	120.8	60.4	26000	26	13.0	147	73
-47	51	Silty Sand/Marl	2600	1.0	2600	131.2	65.6	26000	26	13.0	157	79
-48	52	Silty Sand/Marl	2600	1.0	2600	141.6	70.8	26000	26	13.0	168	84
-49	53	Silty Sand/Marl	2600	1.0	2600	152.0	76.0	26000	26	13.0	178	89
-50	54	Silty Sand/Marl	2600	1.0	2600	162.4	81.2	26000	26	13.0	188	94
-51	55	Silty Sand/Marl	2600	1.0	2600	172.8	86.4	26000	26	13.0	199	99
-52	56	Silty Sand/Marl	2600	1.0	2600	183.2	91.6	26000	26	13.0	209	105
-53	57	Silty Sand/Marl	2600	1.0	2600	193.6	96.8	26000	26	13.0	220	110
-54	58	Silty Sand/Marl	2600	1.0	2600	204.0	102.0	26000	26	13.0	230	115
-55	59	Silty Sand/Marl	2600	1.0	2600	214.4	107.2	26000	26	13.0	240	120
-56	60	Silty Sand/Marl	2600	1.0	2600	224.8	112.4	26000	26	13.0	251	125
-57	61	Silty Sand/Marl	2600	1.0	2600	235.2	117.6	26000	26	13.0	261	131
-58	62	Silty Sand/Marl	2600	1.0	2600	245.6	122.8	26000	26	13.0	272	136
-59	63	Silty Sand/Marl	2600	1.0	2600	256.0	128.0	26000	26	13.0	282	141
-60	64	Silty Sand/Marl	2600	1.0	2600	266.4	133.2	26000	26	13.0	292	146
-61	65	Silty Sand/Marl	2600	1.0	2600	276.8	138.4	26000	26	13.0	303	151

Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Average Marl Depth, Top of Marl at EL. -40 FT
Computed By: JAI
Date: 07/02/2020
Reference Project 22 Westedge
Revised By: JAI
Date Revised: 07/06/2020
Reviewed By: KAH
Date Reviewed: 07/02/2020

Boring		
Drill Rig	CPT	
Depth to Water	2	
Su/sigmaz' NC	0.22	
Factor of Safety	2	
Nc*	10	
Ground Surface	4	ft NAVD88

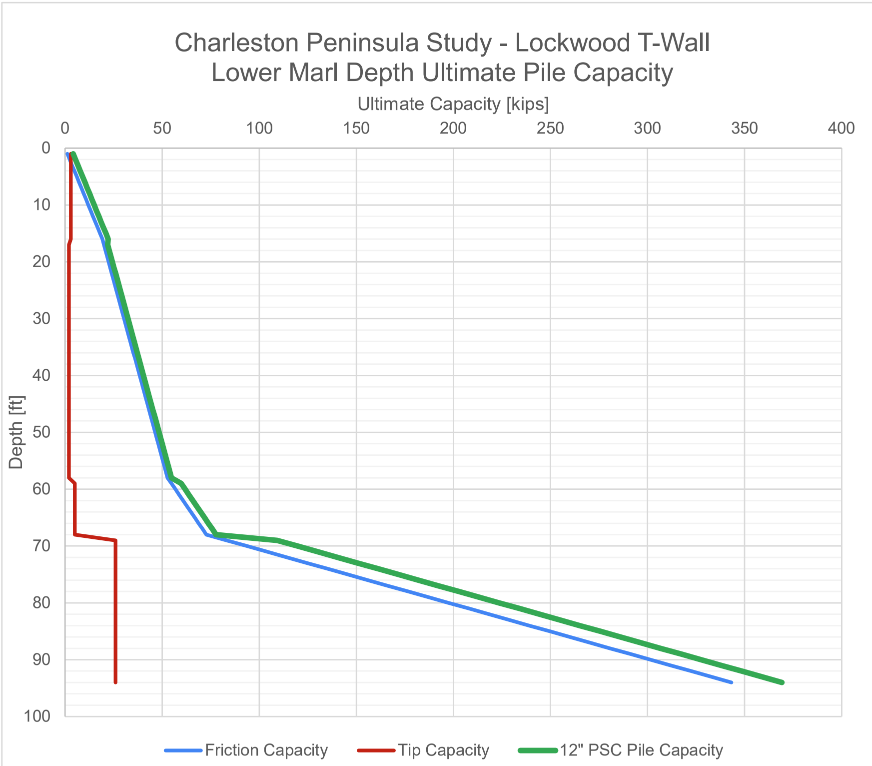
Pile Section

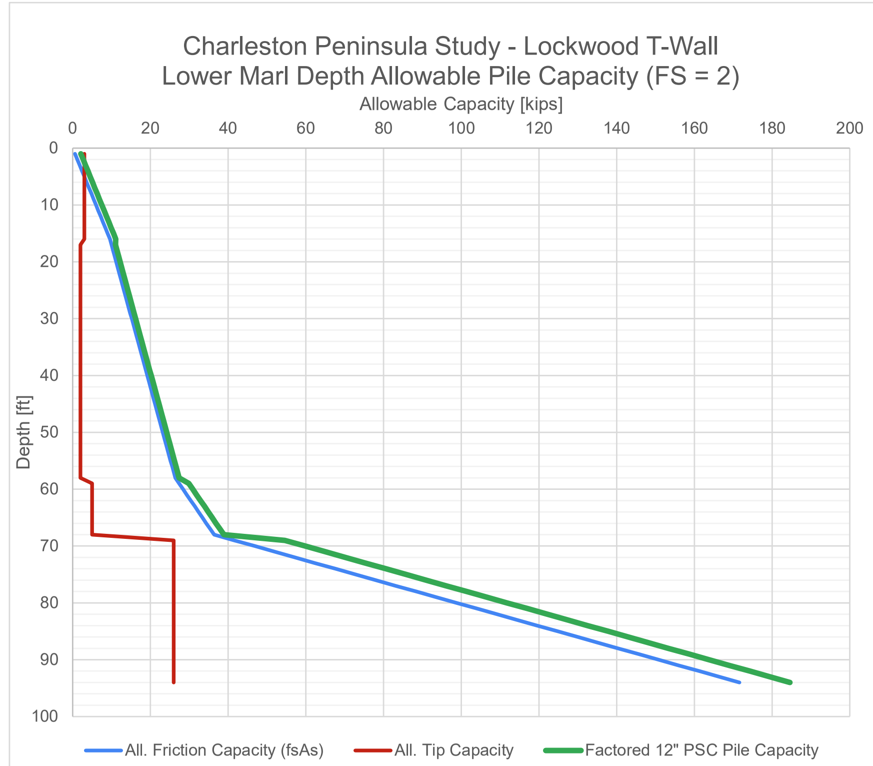
	12-in. PSC	
Pile area	144	in^2
	1.000	ft^2
Depth of Section	12	ft
Flange Width, bf	12	ft
Pile Perimeter	4.00	ft

Formation	***Su (psf)***	***Average Profile Elevation***	***Max Profile Elevation***
Ground Surface		4	4
Upper Sand	300	4 to -7	4 to -13
Marsh/Muck	200	-7 to -31	-13 to -55
Sand	500	-31 to -40	-55 to -65
Silty Sand/Marl	2600	-40	-65

Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	12" PSC Pile Capacity	Factored 12" PSC Pile
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
-62	66	Silty Sand/Marl	2600	1.0	2600	287.2	143.6	26000	26	13.0	313	157
-63	67	Silty Sand/Marl	2600	1.0	2600	297.6	148.8	26000	26	13.0	324	162
-64	68	Silty Sand/Marl	2600	1.0	2600	308.0	154.0	26000	26	13.0	334	167
-65	69	Silty Sand/Marl	2600	1.0	2600	318.4	159.2	26000	26	13.0	344	172
-66	70	Silty Sand/Marl	2600	1.0	2600	328.8	164.4	26000	26	13.0	355	177
-67	71	Silty Sand/Marl	2600	1.0	2600	339.2	169.6	26000	26	13.0	365	183
-68	72	Silty Sand/Marl	2600	1.0	2600	349.6	174.8	26000	26	13.0	376	188
-69	73	Silty Sand/Marl	2600	1.0	2600	360.0	180.0	26000	26	13.0	386	193
-70	74	Silty Sand/Marl	2600	1.0	2600	370.4	185.2	26000	26	13.0	396	198
-71	75	Silty Sand/Marl	2600	1.0	2600	380.8	190.4	26000	26	13.0	407	203
-72	76	Silty Sand/Marl	2600	1.0	2600	391.2	195.6	26000	26	13.0	417	209
-73	77	Silty Sand/Marl	2600	1.0	2600	401.6	200.8	26000	26	13.0	428	214
-74	78	Silty Sand/Marl	2600	1.0	2600	412.0	206.0	26000	26	13.0	438	219
-75	79	Silty Sand/Marl	2600	1.0	2600	422.4	211.2	26000	26	13.0	448	224
-76	80	Silty Sand/Marl	2600	1.0	2600	432.8	216.4	26000	26	13.0	459	229
-77	81	Silty Sand/Marl	2600	1.0	2600	443.2	221.6	26000	26	13.0	469	235
-78	82	Silty Sand/Marl	2600	1.0	2600	453.6	226.8	26000	26	13.0	480	240
-79	83	Silty Sand/Marl	2600	1.0	2600	464.0	232.0	26000	26	13.0	490	245
-80	84	Silty Sand/Marl	2600	1.0	2600	474.4	237.2	26000	26	13.0	500	250
-81	85	Silty Sand/Marl	2600	1.0	2600	484.8	242.4	26000	26	13.0	511	255
-82	86	Silty Sand/Marl	2600	1.0	2600	495.2	247.6	26000	26	13.0	521	261
-83	87	Silty Sand/Marl	2600	1.0	2600	505.6	252.8	26000	26	13.0	532	266
-84	88	Silty Sand/Marl	2600	1.0	2600	516.0	258.0	26000	26	13.0	542	271
-85	89	Silty Sand/Marl	2600	1.0	2600	526.4	263.2	26000	26	13.0	552	276
-86	90	Silty Sand/Marl	2600	1.0	2600	536.8	268.4	26000	26	13.0	563	281
-87	91	Silty Sand/Marl	2600	1.0	2600	547.2	273.6	26000	26	13.0	573	287
-88	92	Silty Sand/Marl	2600	1.0	2600	557.6	278.8	26000	26	13.0	584	292
-89	93	Silty Sand/Marl	2600	1.0	2600	568.0	284.0	26000	26	13.0	594	297
-90	94	Silty Sand/Marl	2600	1.0	2600	578.4	289.2	26000	26	13.0	604	302

Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Lower Marl Depth, Top of Marl at EL. -65 FT
Computed By: JAI
Date: 07/02/2020
Revised By: JAI
Date Revised: 07/06/2020
Reviewed By: KAH
Date Reviewed: 07/02/2020





Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Lower Marl Depth, Top of Marl at EL. -65 FT
Computed By: JAI
Date: 07/02/2020
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Date Revised: 07/06/2020
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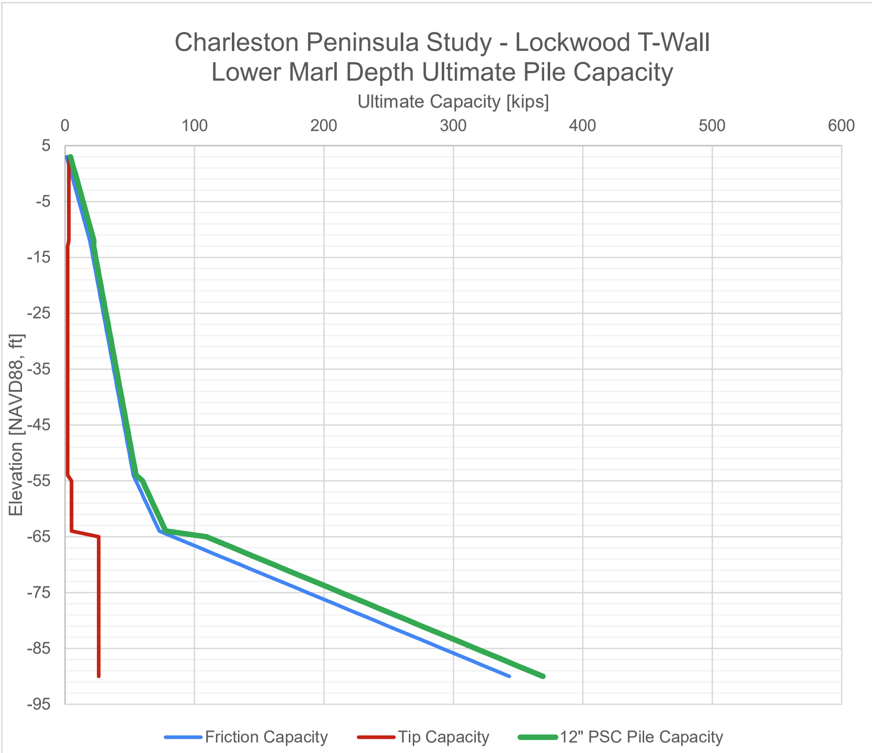
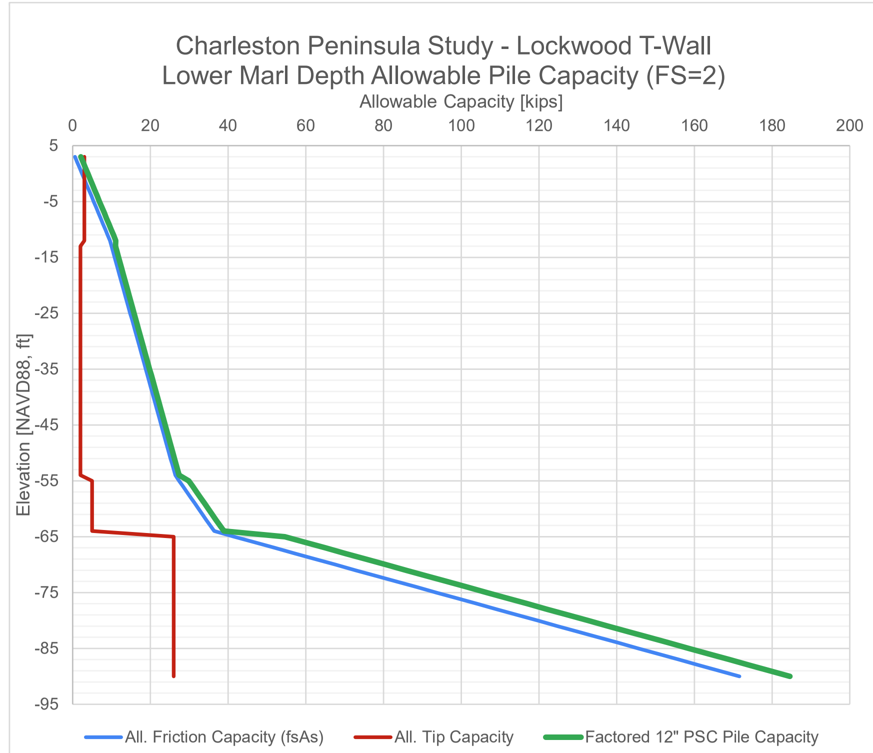
Date Reviewed: 07/02/2020





Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Lower Marl Depth, Top of Marl at EL. -65 FT
Computed By: JAI
Date: 07/02/2020
ference Project 22 Westedge

Revised By: JAI
Date Revised: 07/06/2020

Reviewed By: KAH
Date Reviewed: 07/02/2020

Boring	
Drill Rig	CPT
Depth to Water	2
Su/sigmaz' NC	0.22
Factor of Safety	2
Nc*	10
Ground Surface	4 ft NAVD88

Pile Section	12-in. PSC
Pile area	144 in^2
	1.000 ft^2
Depth of Section	12 ft
Flange Width, bf	12 ft
Pile Perimeter	4.00 ft

Formation	Su (psf)	Elevation of Bottom of Layer
Upper Sand	300	-13
Marsh/Muck	200	-55
Sand	500	-65
Silty Sand/Marl	2600	

Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	12" PSC Pile Capacity	Factored 12" PSC Pile
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
4	0	Upper Sand										
3	1	Upper Sand	300	1.0	300	1.2	0.6	3000	3	1.5	4	2
2	2	Upper Sand	300	1.0	300	2.4	1.2	3000	3	1.5	5	3
1	3	Upper Sand	300	1.0	300	3.6	1.8	3000	3	1.5	7	3
0	4	Upper Sand	300	1.0	300	4.8	2.4	3000	3	1.5	8	4
-1	5	Upper Sand	300	1.0	300	6.0	3.0	3000	3	1.5	9	5
-2	6	Upper Sand	300	1.0	300	7.2	3.6	3000	3	1.5	10	5
-3	7	Upper Sand	300	1.0	300	8.4	4.2	3000	3	1.5	11	6
-4	8	Upper Sand	300	1.0	300	9.6	4.8	3000	3	1.5	13	6
-5	9	Upper Sand	300	1.0	300	10.8	5.4	3000	3	1.5	14	7
-6	10	Upper Sand	300	1.0	300	12.0	6.0	3000	3	1.5	15	8
-7	11	Upper Sand	300	1.0	300	13.2	6.6	3000	3	1.5	16	8
-8	12	Upper Sand	300	1.0	300	14.4	7.2	3000	3	1.5	17	9
-9	13	Upper Sand	300	1.0	300	15.6	7.8	3000	3	1.5	19	9
-10	14	Upper Sand	300	1.0	300	16.8	8.4	3000	3	1.5	20	10
-11	15	Upper Sand	300	1.0	300	18.0	9.0	3000	3	1.5	21	11
-12	16	Upper Sand	300	1.0	300	19.2	9.6	3000	3	1.5	22	11
-13	17	Marsh/Muck	200	1.0	200	20.0	10.0	2000	2	1.0	22	11
-14	18	Marsh/Muck	200	1.0	200	20.8	10.4	2000	2	1.0	23	11
-15	19	Marsh/Muck	200	1.0	200	21.6	10.8	2000	2	1.0	24	12
-16	20	Marsh/Muck	200	1.0	200	22.4	11.2	2000	2	1.0	24	12
-17	21	Marsh/Muck	200	1.0	200	23.2	11.6	2000	2	1.0	25	13
-18	22	Marsh/Muck	200	1.0	200	24.0	12.0	2000	2	1.0	26	13
-19	23	Marsh/Muck	200	1.0	200	24.8	12.4	2000	2	1.0	27	13
-20	24	Marsh/Muck	200	1.0	200	25.6	12.8	2000	2	1.0	28	14
-21	25	Marsh/Muck	200	1.0	200	26.4	13.2	2000	2	1.0	28	14
-22	26	Marsh/Muck	200	1.0	200	27.2	13.6	2000	2	1.0	29	15
-23	27	Marsh/Muck	200	1.0	200	28.0	14.0	2000	2	1.0	30	15
-24	28	Marsh/Muck	200	1.0	200	28.8	14.4	2000	2	1.0	31	15
-25	29	Marsh/Muck	200	1.0	200	29.6	14.8	2000	2	1.0	32	16
-26	30	Marsh/Muck	200	1.0	200	30.4	15.2	2000	2	1.0	32	16
-27	31	Marsh/Muck	200	1.0	200	31.2	15.6	2000	2	1.0	33	17
-28	32	Marsh/Muck	200	1.0	200	32.0	16.0	2000	2	1.0	34	17
-29	33	Marsh/Muck	200	1.0	200	32.8	16.4	2000	2	1.0	35	17
-30	34	Marsh/Muck	200	1.0	200	33.6	16.8	2000	2	1.0	36	18
-31	35	Marsh/Muck	200	1.0	200	34.4	17.2	2000	2	1.0	36	18
-32	36	Marsh/Muck	200	1.0	200	35.2	17.6	2000	2	1.0	37	19
-33	37	Marsh/Muck	200	1.0	200	36.0	18.0	2000	2	1.0	38	19
-34	38	Marsh/Muck	200	1.0	200	36.8	18.4	2000	2	1.0	39	19
-35	39	Marsh/Muck	200	1.0	200	37.6	18.8	2000	2	1.0	40	20
-36	40	Marsh/Muck	200	1.0	200	38.4	19.2	2000	2	1.0	40	20
-37	41	Marsh/Muck	200	1.0	200	39.2	19.6	2000	2	1.0	41	21
-38	42	Marsh/Muck	200	1.0	200	40.0	20.0	2000	2	1.0	42	21
-39	43	Marsh/Muck	200	1.0	200	40.8	20.4	2000	2	1.0	43	21
-40	44	Marsh/Muck	200	1.0	200	41.6	20.8	2000	2	1.0	44	22
-41	45	Marsh/Muck	200	1.0	200	42.4	21.2	2000	2	1.0	44	22
-42	46	Marsh/Muck	200	1.0	200	43.2	21.6	2000	2	1.0	45	23
-43	47	Marsh/Muck	200	1.0	200	44.0	22.0	2000	2	1.0	46	23
-44	48	Marsh/Muck	200	1.0	200	44.8	22.4	2000	2	1.0	47	23
-45	49	Marsh/Muck	200	1.0	200	45.6	22.8	2000	2	1.0	48	24
-46	50	Marsh/Muck	200	1.0	200	46.4	23.2	2000	2	1.0	48	24
-47	51	Marsh/Muck	200	1.0	200	47.2	23.6	2000	2	1.0	49	25
-48	52	Marsh/Muck	200	1.0	200	48.0	24.0	2000	2	1.0	50	25
-49	53	Marsh/Muck	200	1.0	200	48.8	24.4	2000	2	1.0	51	25
-50	54	Marsh/Muck	200	1.0	200	49.6	24.8	2000	2	1.0	52	26
-51	55	Marsh/Muck	200	1.0	200	50.4	25.2	2000	2	1.0	52	26
-52	56	Marsh/Muck	200	1.0	200	51.2	25.6	2000	2	1.0	53	27
-53	57	Marsh/Muck	200	1.0	200	52.0	26.0	2000	2	1.0	54	27
-54	58	Marsh/Muck	200	1.0	200	52.8	26.4	2000	2	1.0	55	27
-55	59	Sand	500	1.0	500	54.8	27.4	5000	5	2.5	60	30
-56	60	Sand	500	1.0	500	56.8	28.4	5000	5	2.5	62	31
-57	61	Sand	500	1.0	500	58.8	29.4	5000	5	2.5	64	32
-58	62	Sand	500	1.0	500	60.8	30.4	5000	5	2.5	66	33
-59	63	Sand	500	1.0	500	62.8	31.4	5000	5	2.5	68	34
-60	64	Sand	500	1.0	500	64.8	32.4	5000	5	2.5	70	35
-61	65	Sand	500	1.0	500	66.8	33.4	5000	5	2.5	72	36
-62	66	Sand	500	1.0	500	68.8	34.4	5000	5	2.5	74	37
-63	67	Sand	500	1.0	500	70.8	35.4	5000	5	2.5	76	38

Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Lower Marl Depth, Top of Marl at EL. -65 FT
Computed By: JAI
Date: 07/02/2020
Reference Project: 22 Westedge

Revised By: JAI
Date Revised: 07/06/2020

Reviewed By: KAH
Date Reviewed: 07/02/2020

Boring	
Drill Rig	CPT
Depth to Water	2
Su/sigmaz' NC	0.22
Factor of Safety	2
Nc*	10
Ground Surface	4 ft NAVD88

Pile Section	12-in. PSC	
Pile area	144	in^2
	1.000	ft^2
Depth of Section	12	ft
Flange Width, bf	12	ft
Pile Perimeter	4.00	ft

Formation	Su (psf)	Elevation of Bottom of Layer
Upper Sand	300	-13
Marsh/Muck	200	-55
Sand	500	-65
Silty Sand/Marl	2600	

Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	12" PSC Pile Capacity	Factored 12" PSC Pile
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
-64	68	Sand	500	1.0	500	72.8	36.4	5000	5	2.5	78	39
-65	69	Silty Sand/Marl	2600	1.0	2600	83.2	41.6	26000	26	13.0	109	55
-66	70	Silty Sand/Marl	2600	1.0	2600	93.6	46.8	26000	26	13.0	120	60
-67	71	Silty Sand/Marl	2600	1.0	2600	104.0	52.0	26000	26	13.0	130	65
-68	72	Silty Sand/Marl	2600	1.0	2600	114.4	57.2	26000	26	13.0	140	70
-69	73	Silty Sand/Marl	2600	1.0	2600	124.8	62.4	26000	26	13.0	151	75
-70	74	Silty Sand/Marl	2600	1.0	2600	135.2	67.6	26000	26	13.0	161	81
-71	75	Silty Sand/Marl	2600	1.0	2600	145.6	72.8	26000	26	13.0	172	86
-72	76	Silty Sand/Marl	2600	1.0	2600	156.0	78.0	26000	26	13.0	182	91
-73	77	Silty Sand/Marl	2600	1.0	2600	166.4	83.2	26000	26	13.0	192	96
-74	78	Silty Sand/Marl	2600	1.0	2600	176.8	88.4	26000	26	13.0	203	101
-75	79	Silty Sand/Marl	2600	1.0	2600	187.2	93.6	26000	26	13.0	213	107
-76	80	Silty Sand/Marl	2600	1.0	2600	197.6	98.8	26000	26	13.0	224	112
-77	81	Silty Sand/Marl	2600	1.0	2600	208.0	104.0	26000	26	13.0	234	117
-78	82	Silty Sand/Marl	2600	1.0	2600	218.4	109.2	26000	26	13.0	244	122
-79	83	Silty Sand/Marl	2600	1.0	2600	228.8	114.4	26000	26	13.0	255	127
-80	84	Silty Sand/Marl	2600	1.0	2600	239.2	119.6	26000	26	13.0	265	133
-81	85	Silty Sand/Marl	2600	1.0	2600	249.6	124.8	26000	26	13.0	276	138
-82	86	Silty Sand/Marl	2600	1.0	2600	260.0	130.0	26000	26	13.0	286	143
-83	87	Silty Sand/Marl	2600	1.0	2600	270.4	135.2	26000	26	13.0	296	148
-84	88	Silty Sand/Marl	2600	1.0	2600	280.8	140.4	26000	26	13.0	307	153
-85	89	Silty Sand/Marl	2600	1.0	2600	291.2	145.6	26000	26	13.0	317	159
-86	90	Silty Sand/Marl	2600	1.0	2600	301.6	150.8	26000	26	13.0	328	164
-87	91	Silty Sand/Marl	2600	1.0	2600	312.0	156.0	26000	26	13.0	338	169
-88	92	Silty Sand/Marl	2600	1.0	2600	322.4	161.2	26000	26	13.0	348	174
-89	93	Silty Sand/Marl	2600	1.0	2600	332.8	166.4	26000	26	13.0	359	179
-90	94	Silty Sand/Marl	2600	1.0	2600	343.2	171.6	26000	26	13.0	369	185

Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: 22 Westedge Calibration
Computed By: JAI
Date: 07/02/2020
Reference Project 22 Westedge

Revised By: JAI
Date Revised: 07/06/2020

Reviewed By: KAH
Date Reviewed: 07/02/2020

Boring	
Drill Rig	CPT
Depth to Water	2
Su/sigmaz' NC	0.22
Factor of Safety	2
Nc*	10
Ground Surface	4 ft NAVD88

Pile Section	12-in. PSC	
Pile area	144	in^2
	1.000	ft^2
Depth of Section	12	ft
Flange Width, bf	12	ft
Pile Perimeter	4.00	ft

	Su	Depth to Bottom of Layer
Upper Sand	300	12
Marsh/Muck	200	43
Sand	500	50
Silty Sand/Marl	2700	

Used greatest values of depths to change in material type as reported in the original report.

Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	Pile Capacity	Factored Pile Capacity
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
4	0	Upper Sand										
3	1	Upper Sand	300	1.0	300	1.2	0.6	3000	3	1.5	4	2
2	2	Upper Sand	300	1.0	300	2.4	1.2	3000	3	1.5	5	3
1	3	Upper Sand	300	1.0	300	3.6	1.8	3000	3	1.5	7	3
0	4	Upper Sand	300	1.0	300	4.8	2.4	3000	3	1.5	8	4
-1	5	Upper Sand	300	1.0	300	6.0	3.0	3000	3	1.5	9	5
-2	6	Upper Sand	300	1.0	300	7.2	3.6	3000	3	1.5	10	5
-3	7	Upper Sand	300	1.0	300	8.4	4.2	3000	3	1.5	11	6
-4	8	Upper Sand	300	1.0	300	9.6	4.8	3000	3	1.5	13	6
-5	9	Upper Sand	300	1.0	300	10.8	5.4	3000	3	1.5	14	7
-6	10	Upper Sand	300	1.0	300	12.0	6.0	3000	3	1.5	15	8
-7	11	Upper Sand	300	1.0	300	13.2	6.6	3000	3	1.5	16	8
-8	12	Upper Sand	300	1.0	300	14.4	7.2	3000	3	1.5	17	9
-9	13	Marsh/Muck	200	1.0	200	15.2	7.6	2000	2	1.0	17	9
-10	14	Marsh/Muck	200	1.0	200	16.0	8.0	2000	2	1.0	18	9
-11	15	Marsh/Muck	200	1.0	200	16.8	8.4	2000	2	1.0	19	9
-12	16	Marsh/Muck	200	1.0	200	17.6	8.8	2000	2	1.0	20	10
-13	17	Marsh/Muck	200	1.0	200	18.4	9.2	2000	2	1.0	20	10
-14	18	Marsh/Muck	200	1.0	200	19.2	9.6	2000	2	1.0	21	11
-15	19	Marsh/Muck	200	1.0	200	20.0	10.0	2000	2	1.0	22	11
-16	20	Marsh/Muck	200	1.0	200	20.8	10.4	2000	2	1.0	23	11
-17	21	Marsh/Muck	200	1.0	200	21.6	10.8	2000	2	1.0	24	12
-18	22	Marsh/Muck	200	1.0	200	22.4	11.2	2000	2	1.0	24	12
-19	23	Marsh/Muck	200	1.0	200	23.2	11.6	2000	2	1.0	25	13
-20	24	Marsh/Muck	200	1.0	200	24.0	12.0	2000	2	1.0	26	13
-21	25	Marsh/Muck	200	1.0	200	24.8	12.4	2000	2	1.0	27	13
-22	26	Marsh/Muck	200	1.0	200	25.6	12.8	2000	2	1.0	28	14
-23	27	Marsh/Muck	200	1.0	200	26.4	13.2	2000	2	1.0	28	14
-24	28	Marsh/Muck	200	1.0	200	27.2	13.6	2000	2	1.0	29	15
-25	29	Marsh/Muck	200	1.0	200	28.0	14.0	2000	2	1.0	30	15
-26	30	Marsh/Muck	200	1.0	200	28.8	14.4	2000	2	1.0	31	15
-27	31	Marsh/Muck	200	1.0	200	29.6	14.8	2000	2	1.0	32	16
-28	32	Marsh/Muck	200	1.0	200	30.4	15.2	2000	2	1.0	32	16
-29	33	Marsh/Muck	200	1.0	200	31.2	15.6	2000	2	1.0	33	17
-30	34	Marsh/Muck	200	1.0	200	32.0	16.0	2000	2	1.0	34	17
-31	35	Marsh/Muck	200	1.0	200	32.8	16.4	2000	2	1.0	35	17
-32	36	Marsh/Muck	200	1.0	200	33.6	16.8	2000	2	1.0	36	18
-33	37	Marsh/Muck	200	1.0	200	34.4	17.2	2000	2	1.0	36	18
-34	38	Marsh/Muck	200	1.0	200	35.2	17.6	2000	2	1.0	37	19
-35	39	Marsh/Muck	200	1.0	200	36.0	18.0	2000	2	1.0	38	19
-36	40	Marsh/Muck	200	1.0	200	36.8	18.4	2000	2	1.0	39	19
-37	41	Marsh/Muck	200	1.0	200	37.6	18.8	2000	2	1.0	40	20
-38	42	Marsh/Muck	200	1.0	200	38.4	19.2	2000	2	1.0	40	20
-39	43	Marsh/Muck	200	1.0	200	39.2	19.6	2000	2	1.0	41	21
-40	44	Sand	500	1.0	500	41.2	20.6	5000	5	2.5	46	23
-41	45	Sand	500	1.0	500	43.2	21.6	5000	5	2.5	48	24
-42	46	Sand	500	1.0	500	45.2	22.6	5000	5	2.5	50	25
-43	47	Sand	500	1.0	500	47.2	23.6	5000	5	2.5	52	26
-44	48	Sand	500	1.0	500	49.2	24.6	5000	5	2.5	54	27
-45	49	Sand	500	1.0	500	51.2	25.6	5000	5	2.5	56	28
-46	50	Silty Sand/Marl	2700	1.0	2700	62.0	31.0	27000	27	13.5	89	45
-47	51	Silty Sand/Marl	2700	1.0	2700	72.8	36.4	27000	27	13.5	100	50
-48	52	Silty Sand/Marl	2700	1.0	2700	83.6	41.8	27000	27	13.5	111	55
-49	53	Silty Sand/Marl	2700	1.0	2700	94.4	47.2	27000	27	13.5	121	61
-50	54	Silty Sand/Marl	2700	1.0	2700	105.2	52.6	27000	27	13.5	132	66
-51	55	Silty Sand/Marl	2700	1.0	2700	116.0	58.0	27000	27	13.5	143	72
-52	56	Silty Sand/Marl	2700	1.0	2700	126.8	63.4	27000	27	13.5	154	77
-53	57	Silty Sand/Marl	2700	1.0	2700	137.6	68.8	27000	27	13.5	165	82
-54	58	Silty Sand/Marl	2700	1.0	2700	148.4	74.2	27000	27	13.5	175	88
-55	59	Silty Sand/Marl	2700	1.0	2700	159.2	79.6	27000	27	13.5	186	93
-56	60	Silty Sand/Marl	2700	1.0	2700	170.0	85.0	27000	27	13.5	197	99
-57	61	Silty Sand/Marl	2700	1.0	2700	180.8	90.4	27000	27	13.5	208	104
-58	62	Silty Sand/Marl	2700	1.0	2700	191.6	95.8	27000	27	13.5	219	109
-59	63	Silty Sand/Marl	2700	1.0	2700	202.4	101.2	27000	27	13.5	229	115
-60	64	Silty Sand/Marl	2700	1.0	2700	213.2	106.6	27000	27	13.5	240	120
-61	65	Silty Sand/Marl	2700	1.0	2700	224.0	112.0	27000	27	13.5	251	126

Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: 22 Westedge Calibration
Computed By: JAI
Date: 07/02/2020
Revised By: JAI
Reviewed By: KAH
ference Project 22 Westedge
Date Revised: 07/06/2020
Date Reviewed: 07/02/2020

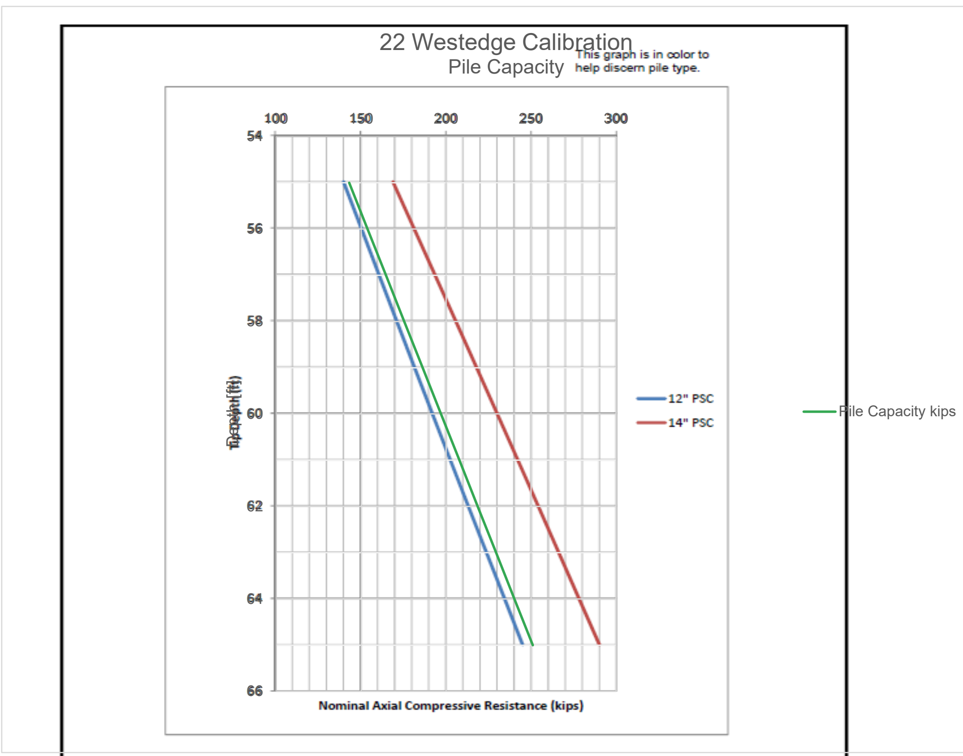
Boring		
Drill Rig	CPT	
Depth to Water	2	
Su/sigmaz' NC	0.22	
Factor of Safety	2	
Nc*	10	
Ground Surface	4	ft NAVD88

Pile Section	12-in. PSC	
Pile area	144	in^2
	1.000	ft^2
Depth of Section	12	ft
Flange Width, bf	12	ft
Pile Perimeter	4.00	ft

	Su	Depth to Bottom of Layer
Upper Sand	300	12
Marsh/Muck	200	43
Sand	500	50
Silty Sand/Marl	2700	

Used greatest values of depths to change in material type as reported in the original report.

Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	Pile Capacity	Factored Pile Capacity
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
-62	66	Silty Sand/Marl	2700	1.0	2700	234.8	117.4	27000	27	13.5	262	131
-63	67	Silty Sand/Marl	2700	1.0	2700	245.6	122.8	27000	27	13.5	273	136
-64	68	Silty Sand/Marl	2700	1.0	2700	256.4	128.2	27000	27	13.5	283	142
-65	69	Silty Sand/Marl	2700	1.0	2700	267.2	133.6	27000	27	13.5	294	147
-66	70	Silty Sand/Marl	2700	1.0	2700	278.0	139.0	27000	27	13.5	305	153
-67	71	Silty Sand/Marl	2700	1.0	2700	288.8	144.4	27000	27	13.5	316	158
-68	72	Silty Sand/Marl	2700	1.0	2700	299.6	149.8	27000	27	13.5	327	163
-69	73	Silty Sand/Marl	2700	1.0	2700	310.4	155.2	27000	27	13.5	337	169
-70	74	Silty Sand/Marl	2700	1.0	2700	321.2	160.6	27000	27	13.5	348	174
-71	75	Silty Sand/Marl	2700	1.0	2700	332.0	166.0	27000	27	13.5	359	180
-72	76	Silty Sand/Marl	2700	1.0	2700	342.8	171.4	27000	27	13.5	370	185
-73	77	Silty Sand/Marl	2700	1.0	2700	353.6	176.8	27000	27	13.5	381	190
-74	78	Silty Sand/Marl	2700	1.0	2700	364.4	182.2	27000	27	13.5	391	196
-75	79	Silty Sand/Marl	2700	1.0	2700	375.2	187.6	27000	27	13.5	402	201
-76	80	Silty Sand/Marl	2700	1.0	2700	386.0	193.0	27000	27	13.5	413	207
-77	81	Silty Sand/Marl	2700	1.0	2700	396.8	198.4	27000	27	13.5	424	212
-78	82	Silty Sand/Marl	2700	1.0	2700	407.6	203.8	27000	27	13.5	435	217
-79	83	Silty Sand/Marl	2700	1.0	2700	418.4	209.2	27000	27	13.5	445	223
-80	84	Silty Sand/Marl	2700	1.0	2700	429.2	214.6	27000	27	13.5	456	228
-81	85	Silty Sand/Marl	2700	1.0	2700	440.0	220.0	27000	27	13.5	467	234
-82	86	Silty Sand/Marl	2700	1.0	2700	450.8	225.4	27000	27	13.5	478	239
-83	87	Silty Sand/Marl	2700	1.0	2700	461.6	230.8	27000	27	13.5	489	244
-84	88	Silty Sand/Marl	2700	1.0	2700	472.4	236.2	27000	27	13.5	499	250
-85	89	Silty Sand/Marl	2700	1.0	2700	483.2	241.6	27000	27	13.5	510	255
-86	90	Silty Sand/Marl	2700	1.0	2700	494.0	247.0	27000	27	13.5	521	261
-87	91	Silty Sand/Marl	2700	1.0	2700	504.8	252.4	27000	27	13.5	532	266
-88	92	Silty Sand/Marl	2700	1.0	2700	515.6	257.8	27000	27	13.5	543	271
-89	93	Silty Sand/Marl	2700	1.0	2700	526.4	263.2	27000	27	13.5	553	277
-90	94	Silty Sand/Marl	2700	1.0	2700	537.2	268.6	27000	27	13.5	564	282



Notes:
1. Recommended minimum pile tip depth is 55 ft below the ground surface elevation on March 27, 2017.
2. Values presented are nominal values. An appropriate safety factor should be applied.
3. Nominal axial tension resistance is 75% of compressive resistance.
4. Load testing is required for an ASD safety factor of 2.
5. Pile structural capacity was not considered in our analyses and is the responsibility of the Structural Engineer.
6. The minimum recommended center-to-center pile spacing is 3 pile diameters, and the efficeintcy factor is 1.0.
7. PSC – Prestressed Concrete

Scale: Not to Scale	S&ME	**Pile Resistance Curves** 22 West Edge Charleston, South Carolina	Figure No.
Drawn by: WLF			**3**
Checked by: MSU			
Date: 04/7/2017		Project No.: 1413-16-141	

Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Horizon Project Bldg 1A Calibration
Computed By: JAI
Date: 07/02/2020
Revised By: JAI
Reviewed By: KAH
ference Project Horizon Project Bldg 1A
Date Revised: 07/06/2020
Date Reviewed: 07/02/2020

Boring			
Drill Rig	CPT		
Depth to Water	2		
Su/sigmaz' NC	0.22		
Factor of Safety	2		
Nc*	10		
Ground Surface	4	ft NAVD88	

Pile Section	12-in. PSC	
Pile area	144	in^2
	1.000	ft^2
Depth of Section	12	ft
Flange Width, bf	12	ft
Pile Perimeter	4.00	ft

	Su	Depth to Bottom of Layer
Upper Sand	400	10
Marsh/Muck	200	39
Sand	500	56
Silty Sand/Marl	2700	

Used greatest values of depths to change in material type as reported in the original report.

Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	Pile Capacity	Factored Pile Capacity
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
4	0	Upper Sand										
3	1	Upper Sand	400	1.0	400	1.6	0.8	4000	4	2.0	6	3
2	2	Upper Sand	400	1.0	400	3.2	1.6	4000	4	2.0	7	4
1	3	Upper Sand	400	1.0	400	4.8	2.4	4000	4	2.0	9	4
0	4	Upper Sand	400	1.0	400	6.4	3.2	4000	4	2.0	10	5
-1	5	Upper Sand	400	1.0	400	8.0	4.0	4000	4	2.0	12	6
-2	6	Upper Sand	400	1.0	400	9.6	4.8	4000	4	2.0	14	7
-3	7	Upper Sand	400	1.0	400	11.2	5.6	4000	4	2.0	15	8
-4	8	Upper Sand	400	1.0	400	12.8	6.4	4000	4	2.0	17	8
-5	9	Upper Sand	400	1.0	400	14.4	7.2	4000	4	2.0	18	9
-6	10	Upper Sand	400	1.0	400	16.0	8.0	4000	4	2.0	20	10
-7	11	Marsh/Muck	200	1.0	200	16.8	8.4	2000	2	1.0	19	9
-8	12	Marsh/Muck	200	1.0	200	17.6	8.8	2000	2	1.0	20	10
-9	13	Marsh/Muck	200	1.0	200	18.4	9.2	2000	2	1.0	20	10
-10	14	Marsh/Muck	200	1.0	200	19.2	9.6	2000	2	1.0	21	11
-11	15	Marsh/Muck	200	1.0	200	20.0	10.0	2000	2	1.0	22	11
-12	16	Marsh/Muck	200	1.0	200	20.8	10.4	2000	2	1.0	23	11
-13	17	Marsh/Muck	200	1.0	200	21.6	10.8	2000	2	1.0	24	12
-14	18	Marsh/Muck	200	1.0	200	22.4	11.2	2000	2	1.0	24	12
-15	19	Marsh/Muck	200	1.0	200	23.2	11.6	2000	2	1.0	25	13
-16	20	Marsh/Muck	200	1.0	200	24.0	12.0	2000	2	1.0	26	13
-17	21	Marsh/Muck	200	1.0	200	24.8	12.4	2000	2	1.0	27	13
-18	22	Marsh/Muck	200	1.0	200	25.6	12.8	2000	2	1.0	28	14
-19	23	Marsh/Muck	200	1.0	200	26.4	13.2	2000	2	1.0	28	14
-20	24	Marsh/Muck	200	1.0	200	27.2	13.6	2000	2	1.0	29	15
-21	25	Marsh/Muck	200	1.0	200	28.0	14.0	2000	2	1.0	30	15
-22	26	Marsh/Muck	200	1.0	200	28.8	14.4	2000	2	1.0	31	15
-23	27	Marsh/Muck	200	1.0	200	29.6	14.8	2000	2	1.0	32	16
-24	28	Marsh/Muck	200	1.0	200	30.4	15.2	2000	2	1.0	32	16
-25	29	Marsh/Muck	200	1.0	200	31.2	15.6	2000	2	1.0	33	17
-26	30	Marsh/Muck	200	1.0	200	32.0	16.0	2000	2	1.0	34	17
-27	31	Marsh/Muck	200	1.0	200	32.8	16.4	2000	2	1.0	35	17
-28	32	Marsh/Muck	200	1.0	200	33.6	16.8	2000	2	1.0	36	18
-29	33	Marsh/Muck	200	1.0	200	34.4	17.2	2000	2	1.0	36	18
-30	34	Marsh/Muck	200	1.0	200	35.2	17.6	2000	2	1.0	37	19
-31	35	Marsh/Muck	200	1.0	200	36.0	18.0	2000	2	1.0	38	19
-32	36	Marsh/Muck	200	1.0	200	36.8	18.4	2000	2	1.0	39	19
-33	37	Marsh/Muck	200	1.0	200	37.6	18.8	2000	2	1.0	40	20
-34	38	Marsh/Muck	200	1.0	200	38.4	19.2	2000	2	1.0	40	20
-35	39	Marsh/Muck	200	1.0	200	39.2	19.6	2000	2	1.0	41	21
-36	40	Sand	500	1.0	500	41.2	20.6	5000	5	2.5	46	23
-37	41	Sand	500	1.0	500	43.2	21.6	5000	5	2.5	48	24
-38	42	Sand	500	1.0	500	45.2	22.6	5000	5	2.5	50	25
-39	43	Sand	500	1.0	500	47.2	23.6	5000	5	2.5	52	26
-40	44	Sand	500	1.0	500	49.2	24.6	5000	5	2.5	54	27
-41	45	Sand	500	1.0	500	51.2	25.6	5000	5	2.5	56	28
-42	46	Sand	500	1.0	500	53.2	26.6	5000	5	2.5	58	29
-43	47	Sand	500	1.0	500	55.2	27.6	5000	5	2.5	60	30
-44	48	Sand	500	1.0	500	57.2	28.6	5000	5	2.5	62	31
-45	49	Sand	500	1.0	500	59.2	29.6	5000	5	2.5	64	32
-46	50	Sand	500	1.0	500	61.2	30.6	5000	5	2.5	66	33
-47	51	Sand	500	1.0	500	63.2	31.6	5000	5	2.5	68	34
-48	52	Sand	500	1.0	500	65.2	32.6	5000	5	2.5	70	35
-49	53	Sand	500	1.0	500	67.2	33.6	5000	5	2.5	72	36
-50	54	Sand	500	1.0	500	69.2	34.6	5000	5	2.5	74	37
-51	55	Sand	500	1.0	500	71.2	35.6	5000	5	2.5	76	38
-52	56	Sand	500	1.0	500	73.2	36.6	5000	5	2.5	78	39
-53	57	Silty Sand/Marl	2700	1.0	2700	84.0	42.0	27000	27	13.5	111	56
-54	58	Silty Sand/Marl	2700	1.0	2700	94.8	47.4	27000	27	13.5	122	61
-55	59	Silty Sand/Marl	2700	1.0	2700	105.6	52.8	27000	27	13.5	133	66
-56	60	Silty Sand/Marl	2700	1.0	2700	116.4	58.2	27000	27	13.5	143	72
-57	61	Silty Sand/Marl	2700	1.0	2700	127.2	63.6	27000	27	13.5	154	77
-58	62	Silty Sand/Marl	2700	1.0	2700	138.0	69.0	27000	27	13.5	165	83
-59	63	Silty Sand/Marl	2700	1.0	2700	148.8	74.4	27000	27	13.5	176	88
-60	64	Silty Sand/Marl	2700	1.0	2700	159.6	79.8	27000	27	13.5	187	93
-61	65	Silty Sand/Marl	2700	1.0	2700	170.4	85.2	27000	27	13.5	197	99

Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Horizon Project Bldg 1A Calibration
Computed By: JAI
Date: 07/02/2020
Revised By: JAI
Reviewed By: KAH
ference Project Horizon Project Bldg 1A
Date Revised: 07/06/2020
Date Reviewed: 07/02/2020

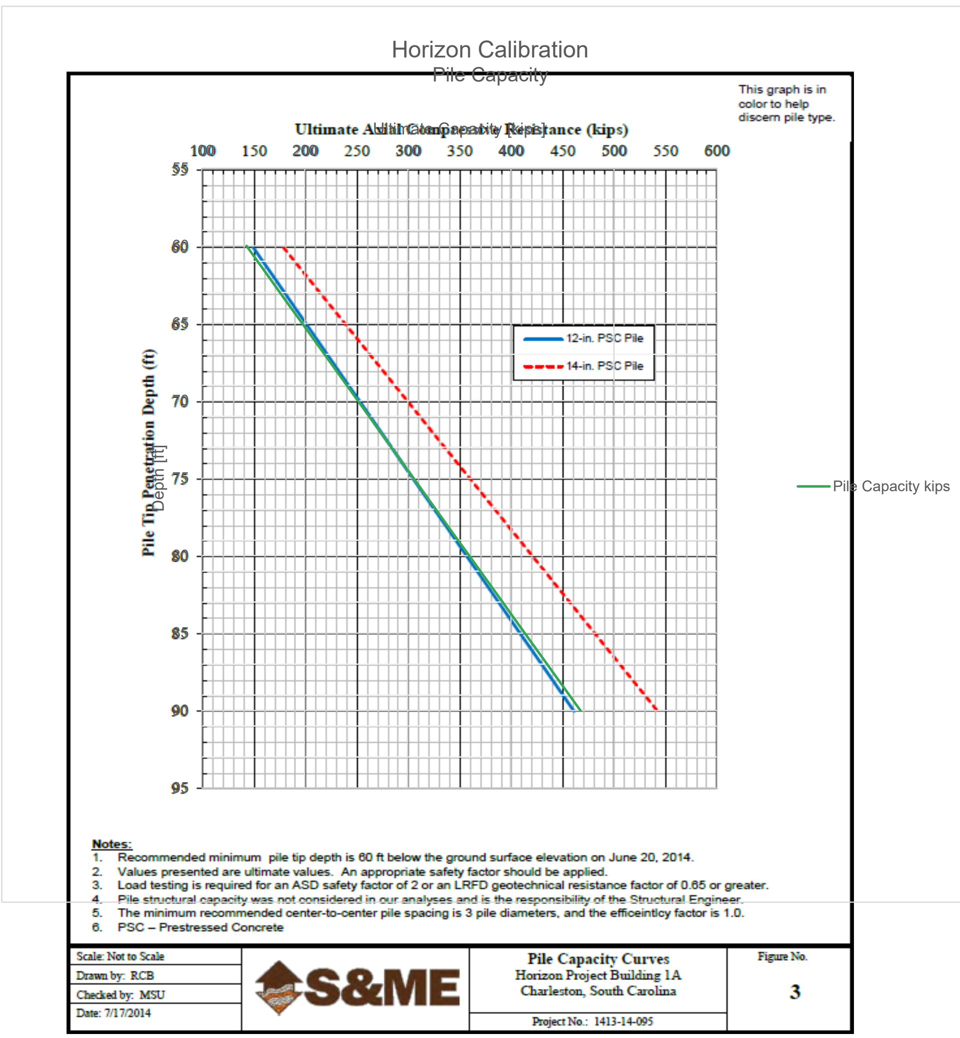
Boring	
Drill Rig	CPT
Depth to Water	2
Su/sigmaz' NC	0.22
Factor of Safety	2
Nc*	10
Ground Surface	4 ft NAVD88

Pile Section	12-in. PSC	
Pile area	144	in^2
	1.000	ft^2
Depth of Section	12	ft
Flange Width, bf	12	ft
Pile Perimeter	4.00	ft

	Su	Depth to Bottom of Layer
Upper Sand	400	10
Marsh/Muck	200	39
Sand	500	56
Silty Sand/Marl	2700	

Used greatest values of depths to change in material type as reported in the original report.

Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	Pile Capacity	Factored Pile Capacity
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
-62	66	Silty Sand/Marl	2700	1.0	2700	181.2	90.6	27000	27	13.5	208	104
-63	67	Silty Sand/Marl	2700	1.0	2700	192.0	96.0	27000	27	13.5	219	110
-64	68	Silty Sand/Marl	2700	1.0	2700	202.8	101.4	27000	27	13.5	230	115
-65	69	Silty Sand/Marl	2700	1.0	2700	213.6	106.8	27000	27	13.5	241	120
-66	70	Silty Sand/Marl	2700	1.0	2700	224.4	112.2	27000	27	13.5	251	126
-67	71	Silty Sand/Marl	2700	1.0	2700	235.2	117.6	27000	27	13.5	262	131
-68	72	Silty Sand/Marl	2700	1.0	2700	246.0	123.0	27000	27	13.5	273	137
-69	73	Silty Sand/Marl	2700	1.0	2700	256.8	128.4	27000	27	13.5	284	142
-70	74	Silty Sand/Marl	2700	1.0	2700	267.6	133.8	27000	27	13.5	295	147
-71	75	Silty Sand/Marl	2700	1.0	2700	278.4	139.2	27000	27	13.5	305	153
-72	76	Silty Sand/Marl	2700	1.0	2700	289.2	144.6	27000	27	13.5	316	158
-73	77	Silty Sand/Marl	2700	1.0	2700	300.0	150.0	27000	27	13.5	327	164
-74	78	Silty Sand/Marl	2700	1.0	2700	310.8	155.4	27000	27	13.5	338	169
-75	79	Silty Sand/Marl	2700	1.0	2700	321.6	160.8	27000	27	13.5	349	174
-76	80	Silty Sand/Marl	2700	1.0	2700	332.4	166.2	27000	27	13.5	359	180
-77	81	Silty Sand/Marl	2700	1.0	2700	343.2	171.6	27000	27	13.5	370	185
-78	82	Silty Sand/Marl	2700	1.0	2700	354.0	177.0	27000	27	13.5	381	191
-79	83	Silty Sand/Marl	2700	1.0	2700	364.8	182.4	27000	27	13.5	392	196
-80	84	Silty Sand/Marl	2700	1.0	2700	375.6	187.8	27000	27	13.5	403	201
-81	85	Silty Sand/Marl	2700	1.0	2700	386.4	193.2	27000	27	13.5	413	207
-82	86	Silty Sand/Marl	2700	1.0	2700	397.2	198.6	27000	27	13.5	424	212
-83	87	Silty Sand/Marl	2700	1.0	2700	408.0	204.0	27000	27	13.5	435	218
-84	88	Silty Sand/Marl	2700	1.0	2700	418.8	209.4	27000	27	13.5	446	223
-85	89	Silty Sand/Marl	2700	1.0	2700	429.6	214.8	27000	27	13.5	457	228
-86	90	Silty Sand/Marl	2700	1.0	2700	440.4	220.2	27000	27	13.5	467	234
-87	91	Silty Sand/Marl	2700	1.0	2700	451.2	225.6	27000	27	13.5	478	239
-88	92	Silty Sand/Marl	2700	1.0	2700	462.0	231.0	27000	27	13.5	489	245
-89	93	Silty Sand/Marl	2700	1.0	2700	472.8	236.4	27000	27	13.5	500	250
-90	94	Silty Sand/Marl	2700	1.0	2700	483.6	241.8	27000	27	13.5	511	255


Horizon Calibration
Pile Capacity
This graph is in color to help discern pile type.
Ultimate Axial Compressive Resistance (kips)
Ultimate Capacity kips
100 150 200 250 300 350 400 450 500 550 600
55 60 65 70 75 80 85 90 95
Pile Tip Penetration Depth (ft)
Depth [ft]
12-in. PSC Pile
14-in. PSC Pile
Pile Capacity kips
Notes:
1. Recommended minimum pile tip depth is 60 ft below the ground surface elevation on June 20, 2014.
2. Values presented are ultimate values. An appropriate safety factor should be applied.
3. Load testing is required for an ASD safety factor of 2 or an LRFD geotechnical resistance factor of 0.65 or greater.
4. Pile structural capacity was not considered in our analyses and is the responsibility of the Structural Engineer.
5. The minimum recommended center-to-center pile spacing is 3 pile diameters, and the efficeintlcy factor is 1.0.
6. PSC – Prestressed Concrete
Scale: Not to Scale
Drawn by: RCB
Checked by: MSU
Date: 7/17/2014
S&ME
Pile Capacity Curves
Horizon Project Building 1A
Charleston, South Carolina
Project No.: 1413-14-095
Figure No.
3

Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Lockwood Pumpstation Calibration
Computed By: JAI
Date: 07/02/2020
ference Project Lockwood Pumpstation

Revised By: JAI
Date Revised: 07/06/2020

Reviewed By: KAH
Date Reviewed: 07/02/2020

Boring	
Drill Rig	CPT
Depth to Water	2
Su/sigmaz' NC	0.22
Factor of Safety	2
Nc*	10
Ground Surface	4 ft NAVD88

Pile Section	12-in. PSC
Pile area	144 in^2
	1.000 ft^2
Depth of Section	12 ft
Flange Width, bf	12 ft
Pile Perimeter	4.00 ft

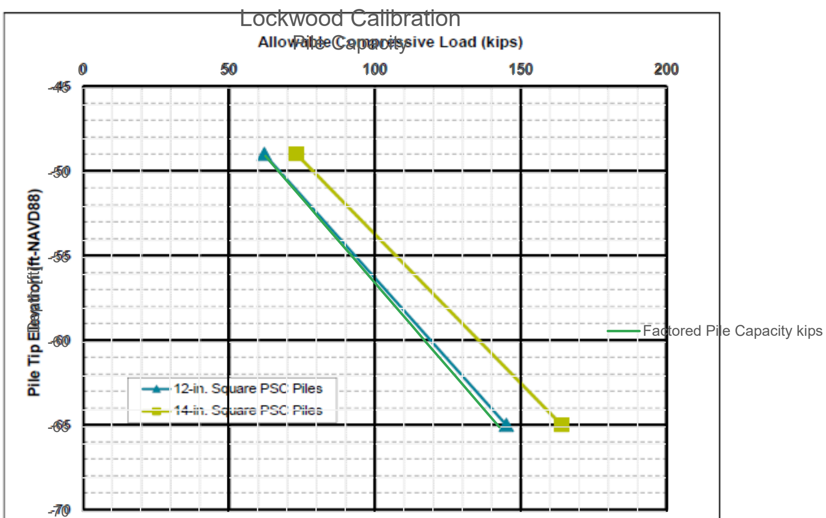
	Su
Upper Sand	200
Marsh/Muck	200
Sand	500
Silty Sand/Marl	2500

Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	Pile Capacity	Factored Pile Capacity
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
4	0	Upper Sand										
3	1	Upper Sand	200	1.0	200	0.8	0.4	2000	2	1.0	3	1
2	2	Upper Sand	200	1.0	200	1.6	0.8	2000	2	1.0	4	2
1	3	Upper Sand	200	1.0	200	2.4	1.2	2000	2	1.0	4	2
0	4	Upper Sand	200	1.0	200	3.2	1.6	2000	2	1.0	5	3
-1	5	Upper Sand	200	1.0	200	4.0	2.0	2000	2	1.0	6	3
-2	6	Upper Sand	200	1.0	200	4.8	2.4	2000	2	1.0	7	3
-3	7	Upper Sand	200	1.0	200	5.6	2.8	2000	2	1.0	8	4
-4	8	Upper Sand	200	1.0	200	6.4	3.2	2000	2	1.0	8	4
-5	9	Upper Sand	200	1.0	200	7.2	3.6	2000	2	1.0	9	5
-6	10	Upper Sand	200	1.0	200	8.0	4.0	2000	2	1.0	10	5
-7	11	Upper Sand	200	1.0	200	8.8	4.4	2000	2	1.0	11	5
-8	12	Upper Sand	200	1.0	200	9.6	4.8	2000	2	1.0	12	6
-9	13	Upper Sand	200	1.0	200	10.4	5.2	2000	2	1.0	12	6
-10	14	Upper Sand	200	1.0	200	11.2	5.6	2000	2	1.0	13	7
-11	15	Upper Sand	200	1.0	200	12.0	6.0	2000	2	1.0	14	7
-12	16	Marsh/Muck	200	1.0	200	12.8	6.4	2000	2	1.0	15	7
-13	17	Marsh/Muck	200	1.0	200	13.6	6.8	2000	2	1.0	16	8
-14	18	Marsh/Muck	200	1.0	200	14.4	7.2	2000	2	1.0	16	8
-15	19	Marsh/Muck	200	1.0	200	15.2	7.6	2000	2	1.0	17	9
-16	20	Marsh/Muck	200	1.0	200	16.0	8.0	2000	2	1.0	18	9
-17	21	Marsh/Muck	200	1.0	200	16.8	8.4	2000	2	1.0	19	9
-18	22	Marsh/Muck	200	1.0	200	17.6	8.8	2000	2	1.0	20	10
-19	23	Marsh/Muck	200	1.0	200	18.4	9.2	2000	2	1.0	20	10
-20	24	Marsh/Muck	200	1.0	200	19.2	9.6	2000	2	1.0	21	11
-21	25	Marsh/Muck	200	1.0	200	20.0	10.0	2000	2	1.0	22	11
-22	26	Marsh/Muck	200	1.0	200	20.8	10.4	2000	2	1.0	23	11
-23	27	Marsh/Muck	200	1.0	200	21.6	10.8	2000	2	1.0	24	12
-24	28	Marsh/Muck	200	1.0	200	22.4	11.2	2000	2	1.0	24	12
-25	29	Marsh/Muck	200	1.0	200	23.2	11.6	2000	2	1.0	25	13
-26	30	Marsh/Muck	200	1.0	200	24.0	12.0	2000	2	1.0	26	13
-27	31	Marsh/Muck	200	1.0	200	24.8	12.4	2000	2	1.0	27	13
-28	32	Marsh/Muck	200	1.0	200	25.6	12.8	2000	2	1.0	28	14
-29	33	Marsh/Muck	200	1.0	200	26.4	13.2	2000	2	1.0	28	14
-30	34	Marsh/Muck	200	1.0	200	27.2	13.6	2000	2	1.0	29	15
-31	35	Marsh/Muck	200	1.0	200	28.0	14.0	2000	2	1.0	30	15
-32	36	Marsh/Muck	200	1.0	200	28.8	14.4	2000	2	1.0	31	15
-33	37	Marsh/Muck	200	1.0	200	29.6	14.8	2000	2	1.0	32	16
-34	38	Marsh/Muck	200	1.0	200	30.4	15.2	2000	2	1.0	32	16
-35	39	Sand	500	1.0	500	32.4	16.2	5000	5	2.5	37	19
-36	40	Sand	500	1.0	500	34.4	17.2	5000	5	2.5	39	20
-37	41	Sand	500	1.0	500	36.4	18.2	5000	5	2.5	41	21
-38	42	Sand	500	1.0	500	38.4	19.2	5000	5	2.5	43	22
-39	43	Sand	500	1.0	500	40.4	20.2	5000	5	2.5	45	23
-40	44	Sand	500	1.0	500	42.4	21.2	5000	5	2.5	47	24
-41	45	Sand	500	1.0	500	44.4	22.2	5000	5	2.5	49	25
-42	46	Sand	500	1.0	500	46.4	23.2	5000	5	2.5	51	26
-43	47	Sand	500	1.0	500	48.4	24.2	5000	5	2.5	53	27
-44	48	Sand	500	1.0	500	50.4	25.2	5000	5	2.5	55	28
-45	49	Silty Sand/Marl	2500	1.0	2500	60.4	30.2	25000	25	12.5	85	43
-46	50	Silty Sand/Marl	2500	1.0	2500	70.4	35.2	25000	25	12.5	95	48
-47	51	Silty Sand/Marl	2500	1.0	2500	80.4	40.2	25000	25	12.5	105	53
-48	52	Silty Sand/Marl	2500	1.0	2500	90.4	45.2	25000	25	12.5	115	58
-49	53	Silty Sand/Marl	2500	1.0	2500	100.4	50.2	25000	25	12.5	125	63
-50	54	Silty Sand/Marl	2500	1.0	2500	110.4	55.2	25000	25	12.5	135	68
-51	55	Silty Sand/Marl	2500	1.0	2500	120.4	60.2	25000	25	12.5	145	73
-52	56	Silty Sand/Marl	2500	1.0	2500	130.4	65.2	25000	25	12.5	155	78
-53	57	Silty Sand/Marl	2500	1.0	2500	140.4	70.2	25000	25	12.5	165	83
-54	58	Silty Sand/Marl	2500	1.0	2500	150.4	75.2	25000	25	12.5	175	88
-55	59	Silty Sand/Marl	2500	1.0	2500	160.4	80.2	25000	25	12.5	185	93
-56	60	Silty Sand/Marl	2500	1.0	2500	170.4	85.2	25000	25	12.5	195	98
-57	61	Silty Sand/Marl	2500	1.0	2500	180.4	90.2	25000	25	12.5	205	103
-58	62	Silty Sand/Marl	2500	1.0	2500	190.4	95.2	25000	25	12.5	215	108
-59	63	Silty Sand/Marl	2500	1.0	2500	200.4	100.2	25000	25	12.5	225	113
-60	64	Silty Sand/Marl	2500	1.0	2500	210.4	105.2	25000	25	12.5	235	118
-61	65	Silty Sand/Marl	2500	1.0	2500	220.4	110.2	25000	25	12.5	245	123

Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Lockwood Pumpstation Calibration
Computed By: JAI
Date: 07/02/2020
Revised By: JAI
Reviewed By: KAH
ference Project Lockwood Pumpstation
Date Revised: 07/06/2020
Date Reviewed: 07/02/2020

Boring		Pile Section	12-in. PSC			Su	
Drill Rig	CPT	Pile area	144	in^2		Upper Sand	200
Depth to Water	2		1.000	ft^2		Marsh/Muck	200
Su/sigmaz' NC	0.22	Depth of Section	12	ft		Sand	500
Factor of Safety	2	Flange Width, bf	12	ft		Silty Sand/Marl	2500
Nc*	10	Pile Perimeter	4.00	ft			
Ground Surface	4 ft NAVD88						

Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	Pile Capacity	Factored Pile Capacity
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
-62	66	Silty Sand/Marl	2500	1.0	2500	230.4	115.2	25000	25	12.5	255	128
-63	67	Silty Sand/Marl	2500	1.0	2500	240.4	120.2	25000	25	12.5	265	133
-64	68	Silty Sand/Marl	2500	1.0	2500	250.4	125.2	25000	25	12.5	275	138
-65	69	Silty Sand/Marl	2500	1.0	2500	260.4	130.2	25000	25	12.5	285	143
-66	70	Silty Sand/Marl	2500	1.0	2500	270.4	135.2	25000	25	12.5	295	148
-67	71	Silty Sand/Marl	2500	1.0	2500	280.4	140.2	25000	25	12.5	305	153
-68	72	Silty Sand/Marl	2500	1.0	2500	290.4	145.2	25000	25	12.5	315	158
-69	73	Silty Sand/Marl	2500	1.0	2500	300.4	150.2	25000	25	12.5	325	163
-70	74	Silty Sand/Marl	2500	1.0	2500	310.4	155.2	25000	25	12.5	335	168
-71	75	Silty Sand/Marl	2500	1.0	2500	320.4	160.2	25000	25	12.5	345	173
-72	76	Silty Sand/Marl	2500	1.0	2500	330.4	165.2	25000	25	12.5	355	178
-73	77	Silty Sand/Marl	2500	1.0	2500	340.4	170.2	25000	25	12.5	365	183
-74	78	Silty Sand/Marl	2500	1.0	2500	350.4	175.2	25000	25	12.5	375	188
-75	79	Silty Sand/Marl	2500	1.0	2500	360.4	180.2	25000	25	12.5	385	193
-76	80	Silty Sand/Marl	2500	1.0	2500	370.4	185.2	25000	25	12.5	395	198
-77	81	Silty Sand/Marl	2500	1.0	2500	380.4	190.2	25000	25	12.5	405	203
-78	82	Silty Sand/Marl	2500	1.0	2500	390.4	195.2	25000	25	12.5	415	208
-79	83	Silty Sand/Marl	2500	1.0	2500	400.4	200.2	25000	25	12.5	425	213
-80	84	Silty Sand/Marl	2500	1.0	2500	410.4	205.2	25000	25	12.5	435	218
-81	85	Silty Sand/Marl	2500	1.0	2500	420.4	210.2	25000	25	12.5	445	223
-82	86	Silty Sand/Marl	2500	1.0	2500	430.4	215.2	25000	25	12.5	455	228
-83	87	Silty Sand/Marl	2500	1.0	2500	440.4	220.2	25000	25	12.5	465	233
-84	88	Silty Sand/Marl	2500	1.0	2500	450.4	225.2	25000	25	12.5	475	238
-85	89	Silty Sand/Marl	2500	1.0	2500	460.4	230.2	25000	25	12.5	485	243
-86	90	Silty Sand/Marl	2500	1.0	2500	470.4	235.2	25000	25	12.5	495	248
-87	91	Silty Sand/Marl	2500	1.0	2500	480.4	240.2	25000	25	12.5	505	253
-88	92	Silty Sand/Marl	2500	1.0	2500	490.4	245.2	25000	25	12.5	515	258
-89	93	Silty Sand/Marl	2500	1.0	2500	500.4	250.2	25000	25	12.5	525	263
-90	94	Silty Sand/Marl	2500	1.0	2500	510.4	255.2	25000	25	12.5	535	268

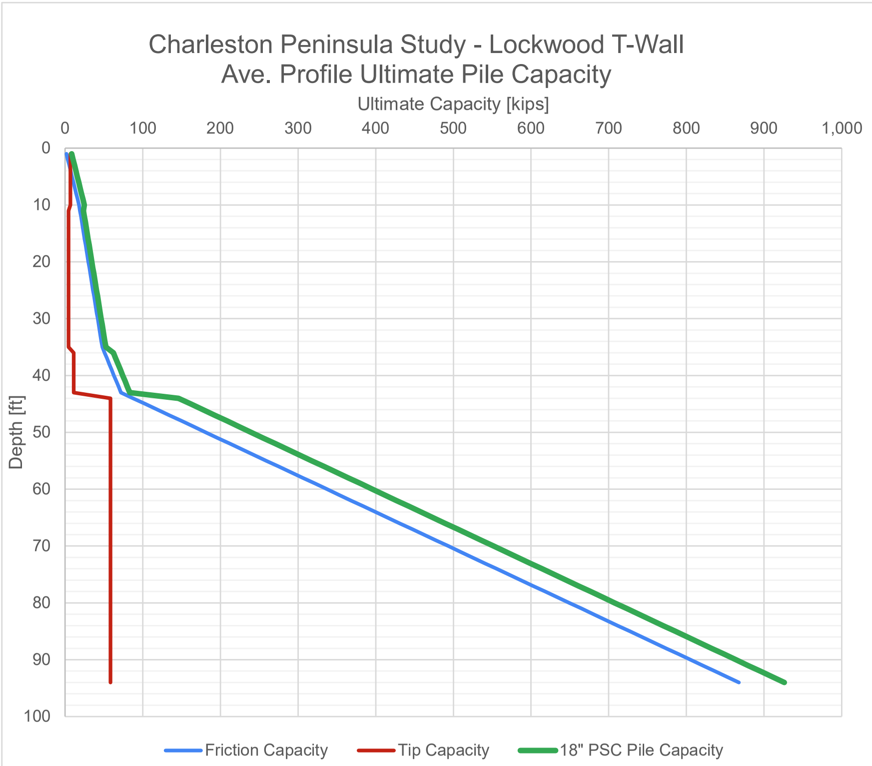


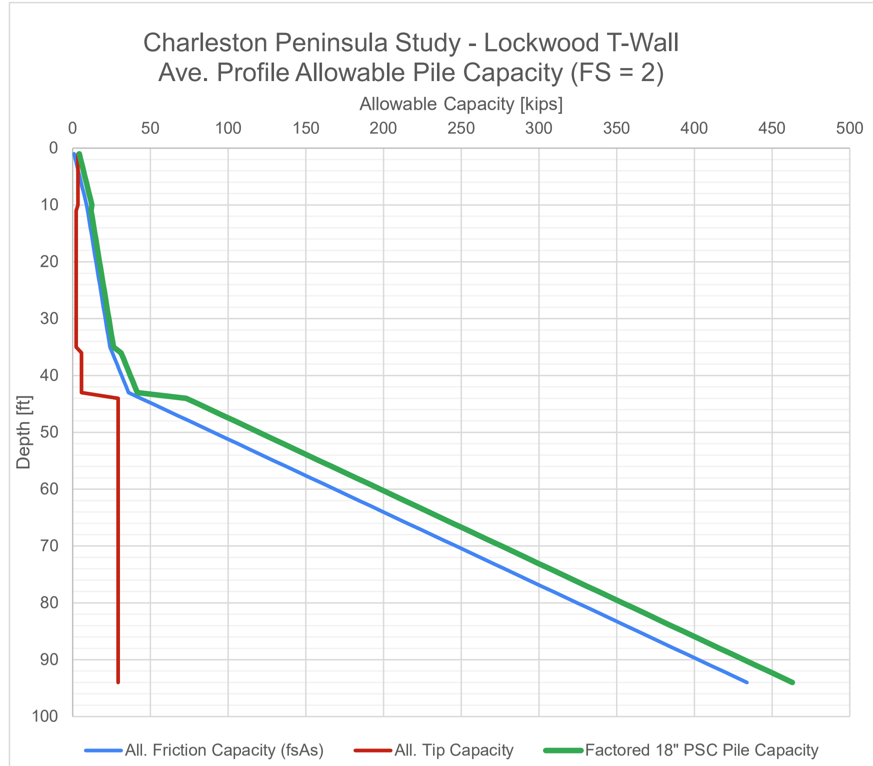
NOTES:

1) A minimum pile center-to-center spacing of three (3) pile diameters is recommended.
2) Piles should penetrate the Cooper Marl at least 5 ft. The subsurface exploration indicated that marl was encountered at an elevation of approximately -44 ft-NAVD88; therefore, a minimum pile tip elevation of -49 ft-NAVD88 is required.
3) To estimate the allowable compressive resistance, a safety factor of 2 was applied to the ultimate skin friction and end bearing resistances. This assumes dynamic test measurements (PDA) and CAPWAP analyses are performed to confirm design assumptions.
4) An allowable uplift resistance can be assumed as ⅔ of the allowable compressive resistance shown in the plot.
5) The allowable resistances presented on the figure are based on the soil strength only. The structural capacity of the pile has not been considered in our analysis and should be evaluated by the project structural engineer.

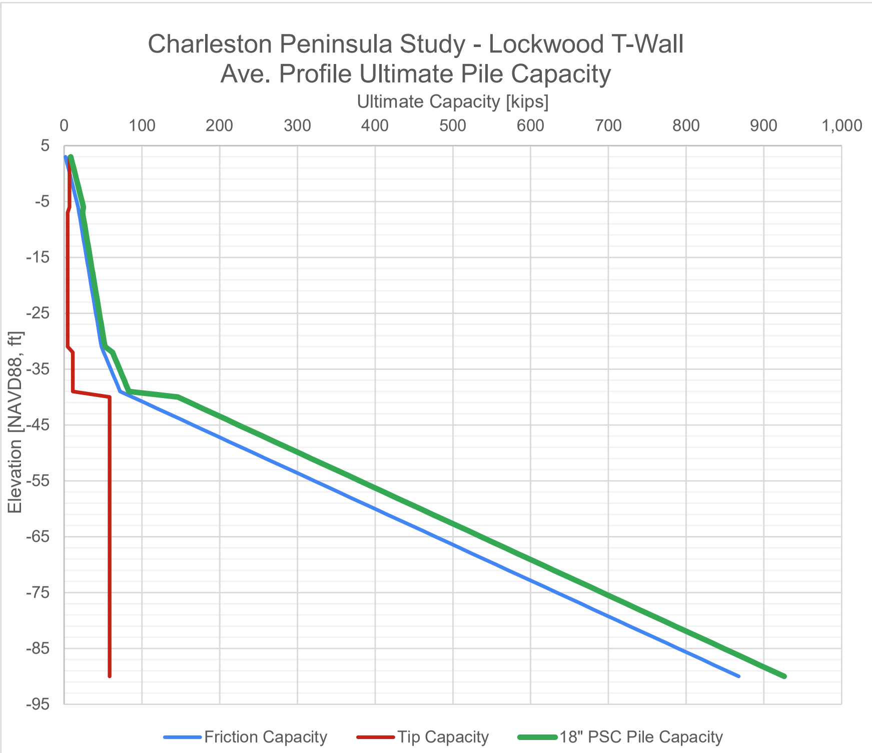
Figure 3 – Allowable Axial Compressive Pile Loads (12-in. & 14-in. Square PSC Piles)

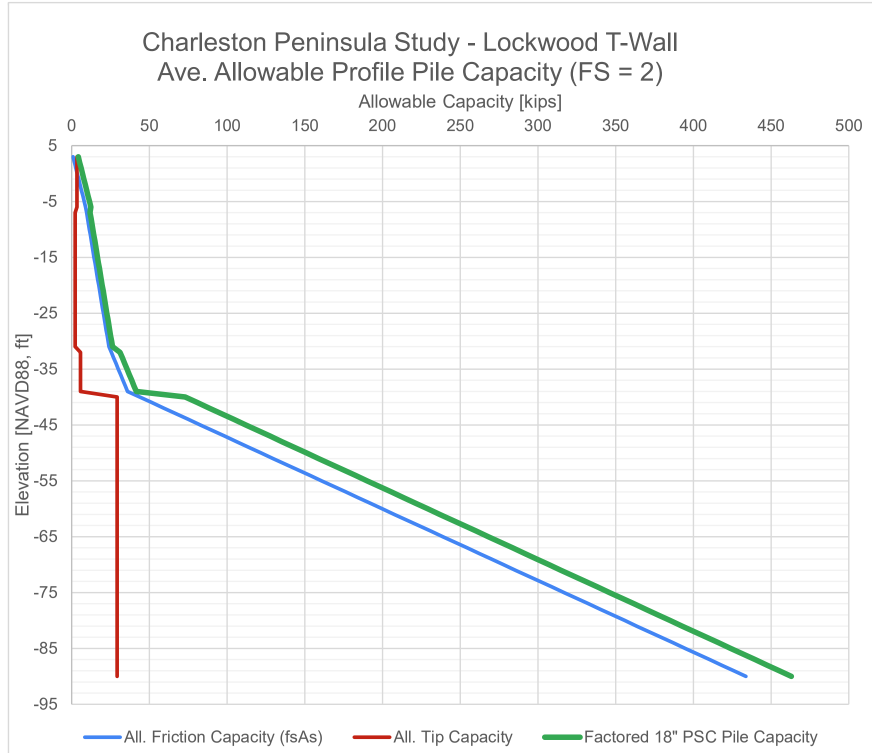
Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Average Marl Depth, Top of Marl at EL. -40 FT
Computed By: JAI
Date: 07/02/2020
Revised By: JAI
Date Revised: 07/06/2020
Reviewed By: KAH
Date Reviewed: 07/02/2020





Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Average Marl Depth, Top of Marl at EL. -40 FT
Computed By: JAI | Revised By: JAI | Reviewed By: KAH
Date: 07/02/2020 | Date Revised: 07/06/2020 | Date Reviewed: 07/02/2020





Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Average Marl Depth, Top of Marl at EL. -40 FT
Computed By: JAI
Date: 07/02/2020
ference Project 22 Westedge

Revised By: JAI
Date Revised: 07/06/2020

Reviewed By: KAH
Date Reviewed: 07/02/2020

Boring		
Drill Rig	CPT	
Depth to Water	2	
Su/sigmaz' NC	0.22	
Factor of Safety	2	
Nc*	10	
Ground Surface	4	ft NAVD88

Pile Section	18-in. PSC	
Pile area	324	in^2
	2.250	ft^2
Depth of Section	18	ft
Flange Width, bf	18	ft
Pile Perimeter	6.00	ft

Formation	Su (psf)	Elevation of Bottom of Layer
Upper Sand	300	-7
Marsh/Muck	200	-31
Sand	500	-40
Silty Sand/Marl	2600	

Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	18" PSC Pile Capacity	Factored 18" PSC Pile
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
4	0	Upper Sand										
3	1	Upper Sand	300	1.0	300	1.8	0.9	3000	7	3.4	9	4
2	2	Upper Sand	300	1.0	300	3.6	1.8	3000	7	3.4	10	5
1	3	Upper Sand	300	1.0	300	5.4	2.7	3000	7	3.4	12	6
0	4	Upper Sand	300	1.0	300	7.2	3.6	3000	7	3.4	14	7
-1	5	Upper Sand	300	1.0	300	9.0	4.5	3000	7	3.4	16	8
-2	6	Upper Sand	300	1.0	300	10.8	5.4	3000	7	3.4	18	9
-3	7	Upper Sand	300	1.0	300	12.6	6.3	3000	7	3.4	19	10
-4	8	Upper Sand	300	1.0	300	14.4	7.2	3000	7	3.4	21	11
-5	9	Upper Sand	300	1.0	300	16.2	8.1	3000	7	3.4	23	11
-6	10	Upper Sand	300	1.0	300	18.0	9.0	3000	7	3.4	25	12
-7	11	Marsh/Muck	200	1.0	200	19.2	9.6	2000	5	2.3	24	12
-8	12	Marsh/Muck	200	1.0	200	20.4	10.2	2000	5	2.3	25	12
-9	13	Marsh/Muck	200	1.0	200	21.6	10.8	2000	5	2.3	26	13
-10	14	Marsh/Muck	200	1.0	200	22.8	11.4	2000	5	2.3	27	14
-11	15	Marsh/Muck	200	1.0	200	24.0	12.0	2000	5	2.3	29	14
-12	16	Marsh/Muck	200	1.0	200	25.2	12.6	2000	5	2.3	30	15
-13	17	Marsh/Muck	200	1.0	200	26.4	13.2	2000	5	2.3	31	15
-14	18	Marsh/Muck	200	1.0	200	27.6	13.8	2000	5	2.3	32	16
-15	19	Marsh/Muck	200	1.0	200	28.8	14.4	2000	5	2.3	33	17
-16	20	Marsh/Muck	200	1.0	200	30.0	15.0	2000	5	2.3	35	17
-17	21	Marsh/Muck	200	1.0	200	31.2	15.6	2000	5	2.3	36	18
-18	22	Marsh/Muck	200	1.0	200	32.4	16.2	2000	5	2.3	37	18
-19	23	Marsh/Muck	200	1.0	200	33.6	16.8	2000	5	2.3	38	19
-20	24	Marsh/Muck	200	1.0	200	34.8	17.4	2000	5	2.3	39	20
-21	25	Marsh/Muck	200	1.0	200	36.0	18.0	2000	5	2.3	41	20
-22	26	Marsh/Muck	200	1.0	200	37.2	18.6	2000	5	2.3	42	21
-23	27	Marsh/Muck	200	1.0	200	38.4	19.2	2000	5	2.3	43	21
-24	28	Marsh/Muck	200	1.0	200	39.6	19.8	2000	5	2.3	44	22
-25	29	Marsh/Muck	200	1.0	200	40.8	20.4	2000	5	2.3	45	23
-26	30	Marsh/Muck	200	1.0	200	42.0	21.0	2000	5	2.3	47	23
-27	31	Marsh/Muck	200	1.0	200	43.2	21.6	2000	5	2.3	48	24
-28	32	Marsh/Muck	200	1.0	200	44.4	22.2	2000	5	2.3	49	24
-29	33	Marsh/Muck	200	1.0	200	45.6	22.8	2000	5	2.3	50	25
-30	34	Marsh/Muck	200	1.0	200	46.8	23.4	2000	5	2.3	51	26
-31	35	Marsh/Muck	200	1.0	200	48.0	24.0	2000	5	2.3	53	26
-32	36	Sand	500	1.0	500	51.0	25.5	5000	11	5.6	62	31
-33	37	Sand	500	1.0	500	54.0	27.0	5000	11	5.6	65	33
-34	38	Sand	500	1.0	500	57.0	28.5	5000	11	5.6	68	34
-35	39	Sand	500	1.0	500	60.0	30.0	5000	11	5.6	71	36
-36	40	Sand	500	1.0	500	63.0	31.5	5000	11	5.6	74	37
-37	41	Sand	500	1.0	500	66.0	33.0	5000	11	5.6	77	39
-38	42	Sand	500	1.0	500	69.0	34.5	5000	11	5.6	80	40
-39	43	Sand	500	1.0	500	72.0	36.0	5000	11	5.6	83	42
-40	44	Silty Sand/Marl	2600	1.0	2600	87.6	43.8	26000	59	29.3	146	73
-41	45	Silty Sand/Marl	2600	1.0	2600	103.2	51.6	26000	59	29.3	162	81
-42	46	Silty Sand/Marl	2600	1.0	2600	118.8	59.4	26000	59	29.3	177	89
-43	47	Silty Sand/Marl	2600	1.0	2600	134.4	67.2	26000	59	29.3	193	96
-44	48	Silty Sand/Marl	2600	1.0	2600	150.0	75.0	26000	59	29.3	209	104
-45	49	Silty Sand/Marl	2600	1.0	2600	165.6	82.8	26000	59	29.3	224	112
-46	50	Silty Sand/Marl	2600	1.0	2600	181.2	90.6	26000	59	29.3	240	120
-47	51	Silty Sand/Marl	2600	1.0	2600	196.8	98.4	26000	59	29.3	255	128
-48	52	Silty Sand/Marl	2600	1.0	2600	212.4	106.2	26000	59	29.3	271	135
-49	53	Silty Sand/Marl	2600	1.0	2600	228.0	114.0	26000	59	29.3	287	143
-50	54	Silty Sand/Marl	2600	1.0	2600	243.6	121.8	26000	59	29.3	302	151
-51	55	Silty Sand/Marl	2600	1.0	2600	259.2	129.6	26000	59	29.3	318	159
-52	56	Silty Sand/Marl	2600	1.0	2600	274.8	137.4	26000	59	29.3	333	167
-53	57	Silty Sand/Marl	2600	1.0	2600	290.4	145.2	26000	59	29.3	349	174
-54	58	Silty Sand/Marl	2600	1.0	2600	306.0	153.0	26000	59	29.3	365	182
-55	59	Silty Sand/Marl	2600	1.0	2600	321.6	160.8	26000	59	29.3	380	190
-56	60	Silty Sand/Marl	2600	1.0	2600	337.2	168.6	26000	59	29.3	396	198
-57	61	Silty Sand/Marl	2600	1.0	2600	352.8	176.4	26000	59	29.3	411	206
-58	62	Silty Sand/Marl	2600	1.0	2600	368.4	184.2	26000	59	29.3	427	213
-59	63	Silty Sand/Marl	2600	1.0	2600	384.0	192.0	26000	59	29.3	443	221
-60	64	Silty Sand/Marl	2600	1.0	2600	399.6	199.8	26000	59	29.3	458	229
-61	65	Silty Sand/Marl	2600	1.0	2600	415.2	207.6	26000	59	29.3	474	237

Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Average Marl Depth, Top of Marl at EL. -40 FT
Computed By: JAI
Date: 07/02/2020
ference Project 22 Westedge

Revised By: JAI
Date Revised: 07/06/2020

Reviewed By: KAH
Date Reviewed: 07/02/2020

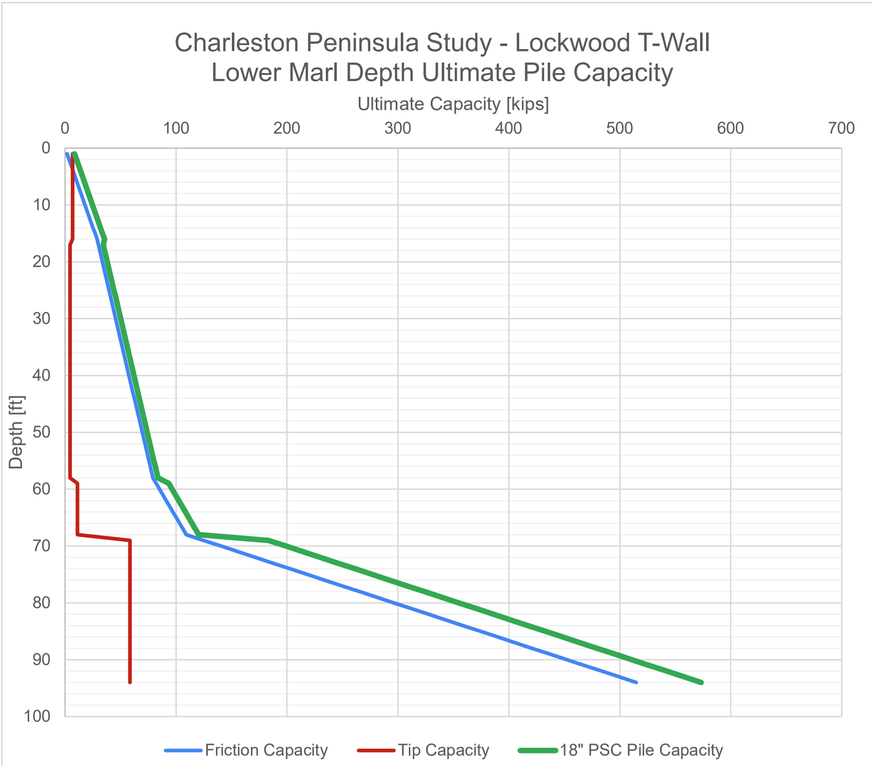
Boring	
Drill Rig	CPT
Depth to Water	2
Su/sigmaz' NC	0.22
Factor of Safety	2
Nc*	10
Ground Surface	4 ft NAVD88

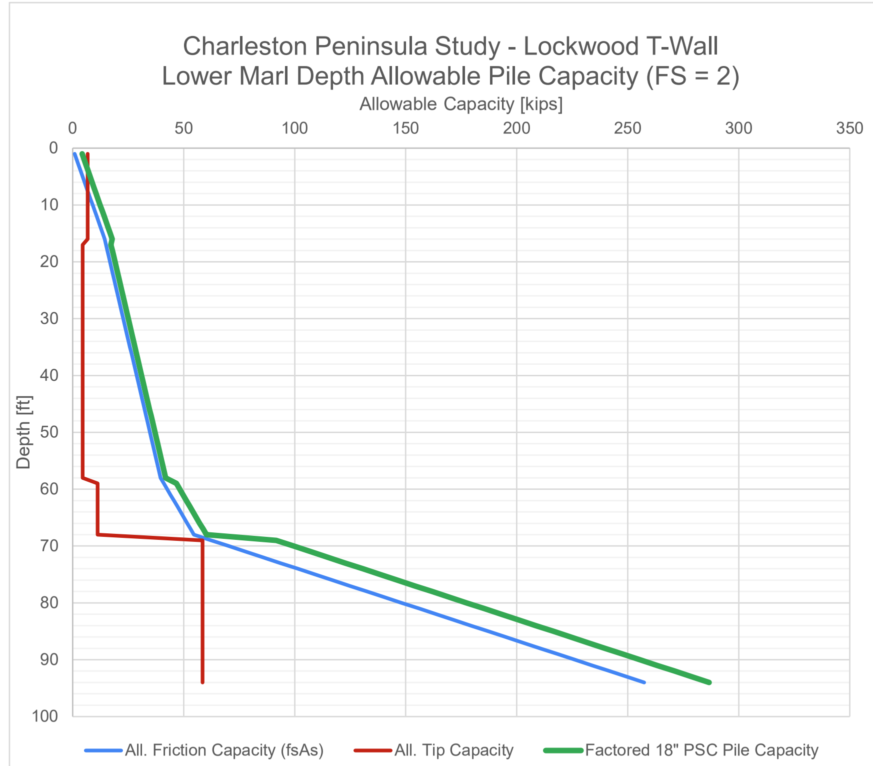
Pile Section	18-in. PSC
Pile area	324 in^2
	2.250 ft^2
Depth of Section	18 ft
Flange Width, bf	18 ft
Pile Perimeter	6.00 ft

Formation	Su (psf)	Elevation of Bottom of Layer
Upper Sand	300	-7
Marsh/Muck	200	-31
Sand	500	-40
Silty Sand/Marl	2600	

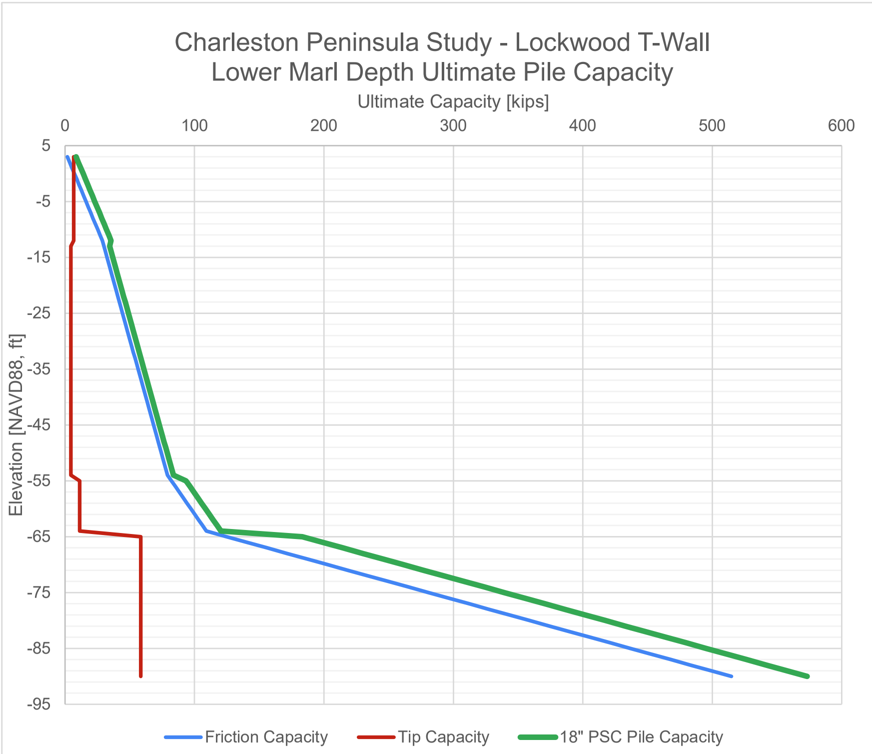
Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	18" PSC Pile Capacity	Factored 18" PSC Pile
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
-62	66	Silty Sand/Marl	2600	1.0	2600	430.8	215.4	26000	59	29.3	489	245
-63	67	Silty Sand/Marl	2600	1.0	2600	446.4	223.2	26000	59	29.3	505	252
-64	68	Silty Sand/Marl	2600	1.0	2600	462.0	231.0	26000	59	29.3	521	260
-65	69	Silty Sand/Marl	2600	1.0	2600	477.6	238.8	26000	59	29.3	536	268
-66	70	Silty Sand/Marl	2600	1.0	2600	493.2	246.6	26000	59	29.3	552	276
-67	71	Silty Sand/Marl	2600	1.0	2600	508.8	254.4	26000	59	29.3	567	284
-68	72	Silty Sand/Marl	2600	1.0	2600	524.4	262.2	26000	59	29.3	583	291
-69	73	Silty Sand/Marl	2600	1.0	2600	540.0	270.0	26000	59	29.3	599	299
-70	74	Silty Sand/Marl	2600	1.0	2600	555.6	277.8	26000	59	29.3	614	307
-71	75	Silty Sand/Marl	2600	1.0	2600	571.2	285.6	26000	59	29.3	630	315
-72	76	Silty Sand/Marl	2600	1.0	2600	586.8	293.4	26000	59	29.3	645	323
-73	77	Silty Sand/Marl	2600	1.0	2600	602.4	301.2	26000	59	29.3	661	330
-74	78	Silty Sand/Marl	2600	1.0	2600	618.0	309.0	26000	59	29.3	677	338
-75	79	Silty Sand/Marl	2600	1.0	2600	633.6	316.8	26000	59	29.3	692	346
-76	80	Silty Sand/Marl	2600	1.0	2600	649.2	324.6	26000	59	29.3	708	354
-77	81	Silty Sand/Marl	2600	1.0	2600	664.8	332.4	26000	59	29.3	723	362
-78	82	Silty Sand/Marl	2600	1.0	2600	680.4	340.2	26000	59	29.3	739	369
-79	83	Silty Sand/Marl	2600	1.0	2600	696.0	348.0	26000	59	29.3	755	377
-80	84	Silty Sand/Marl	2600	1.0	2600	711.6	355.8	26000	59	29.3	770	385
-81	85	Silty Sand/Marl	2600	1.0	2600	727.2	363.6	26000	59	29.3	786	393
-82	86	Silty Sand/Marl	2600	1.0	2600	742.8	371.4	26000	59	29.3	801	401
-83	87	Silty Sand/Marl	2600	1.0	2600	758.4	379.2	26000	59	29.3	817	408
-84	88	Silty Sand/Marl	2600	1.0	2600	774.0	387.0	26000	59	29.3	833	416
-85	89	Silty Sand/Marl	2600	1.0	2600	789.6	394.8	26000	59	29.3	848	424
-86	90	Silty Sand/Marl	2600	1.0	2600	805.2	402.6	26000	59	29.3	864	432
-87	91	Silty Sand/Marl	2600	1.0	2600	820.8	410.4	26000	59	29.3	879	440
-88	92	Silty Sand/Marl	2600	1.0	2600	836.4	418.2	26000	59	29.3	895	447
-89	93	Silty Sand/Marl	2600	1.0	2600	852.0	426.0	26000	59	29.3	911	455
-90	94	Silty Sand/Marl	2600	1.0	2600	867.6	433.8	26000	59	29.3	926	463

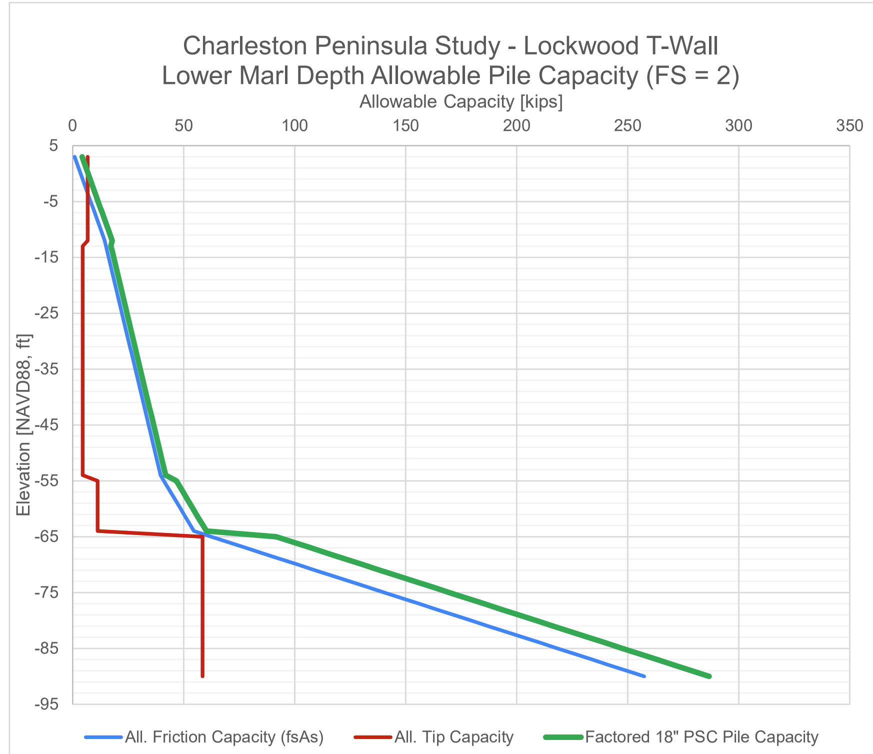
Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Lower Marl Depth, Top of Marl at EL. -65 FT
Computed By: JAI | Revised By: JAI | Reviewed By: KAH
Date: 07/02/2020 | Date Revised: 07/06/2020 | Date Reviewed: 07/02/2020





Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Lower Marl Depth, Top of Marl at EL. -65 FT
Computed By: JAI
Date: 07/02/2020
Revised By: JAI
Date Revised: 07/06/2020
Reviewed By: KAH
Date Reviewed: 07/02/2020





Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Lower Marl Depth, Top of Marl at EL. -65 FT
Computed By: JAI
Date: 07/02/2020
Reference Project 22 Westedge

Revised By: JAI
Date Revised: 07/06/2020

Reviewed By: KAH
Date Reviewed: 07/02/2020

Boring		
Drill Rig	CPT	
Depth to Water	2	
Su/sigmaz' NC	0.22	
Factor of Safety	2	
Nc*	10	
Ground Surface	4	ft NAVD88

Pile Section	18-in. PSC	
Pile area	324	in^2
	2.250	ft^2
Depth of Section	18	ft
Flange Width, bf	18	ft
Pile Perimeter	6.00	ft

Formation	Su (psf)	Elevation of Bottom of Layer
Upper Sand	300	-13
Marsh/Muck	200	-55
Sand	500	-65
Silty Sand/Marl	2600	

Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	18" PSC Pile Capacity	Factored 18" PSC Pile
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
4	0	Upper Sand										
3	1	Upper Sand	300	1.0	300	1.8	0.9	3000	7	3.4	9	4
2	2	Upper Sand	300	1.0	300	3.6	1.8	3000	7	3.4	10	5
1	3	Upper Sand	300	1.0	300	5.4	2.7	3000	7	3.4	12	6
0	4	Upper Sand	300	1.0	300	7.2	3.6	3000	7	3.4	14	7
-1	5	Upper Sand	300	1.0	300	9.0	4.5	3000	7	3.4	16	8
-2	6	Upper Sand	300	1.0	300	10.8	5.4	3000	7	3.4	18	9
-3	7	Upper Sand	300	1.0	300	12.6	6.3	3000	7	3.4	19	10
-4	8	Upper Sand	300	1.0	300	14.4	7.2	3000	7	3.4	21	11
-5	9	Upper Sand	300	1.0	300	16.2	8.1	3000	7	3.4	23	11
-6	10	Upper Sand	300	1.0	300	18.0	9.0	3000	7	3.4	25	12
-7	11	Upper Sand	300	1.0	300	19.8	9.9	3000	7	3.4	27	13
-8	12	Upper Sand	300	1.0	300	21.6	10.8	3000	7	3.4	28	14
-9	13	Upper Sand	300	1.0	300	23.4	11.7	3000	7	3.4	30	15
-10	14	Upper Sand	300	1.0	300	25.2	12.6	3000	7	3.4	32	16
-11	15	Upper Sand	300	1.0	300	27.0	13.5	3000	7	3.4	34	17
-12	16	Upper Sand	300	1.0	300	28.8	14.4	3000	7	3.4	36	18
-13	17	Marsh/Muck	200	1.0	200	30.0	15.0	2000	5	2.3	35	17
-14	18	Marsh/Muck	200	1.0	200	31.2	15.6	2000	5	2.3	36	18
-15	19	Marsh/Muck	200	1.0	200	32.4	16.2	2000	5	2.3	37	18
-16	20	Marsh/Muck	200	1.0	200	33.6	16.8	2000	5	2.3	38	19
-17	21	Marsh/Muck	200	1.0	200	34.8	17.4	2000	5	2.3	39	20
-18	22	Marsh/Muck	200	1.0	200	36.0	18.0	2000	5	2.3	41	20
-19	23	Marsh/Muck	200	1.0	200	37.2	18.6	2000	5	2.3	42	21
-20	24	Marsh/Muck	200	1.0	200	38.4	19.2	2000	5	2.3	43	21
-21	25	Marsh/Muck	200	1.0	200	39.6	19.8	2000	5	2.3	44	22
-22	26	Marsh/Muck	200	1.0	200	40.8	20.4	2000	5	2.3	45	23
-23	27	Marsh/Muck	200	1.0	200	42.0	21.0	2000	5	2.3	47	23
-24	28	Marsh/Muck	200	1.0	200	43.2	21.6	2000	5	2.3	48	24
-25	29	Marsh/Muck	200	1.0	200	44.4	22.2	2000	5	2.3	49	24
-26	30	Marsh/Muck	200	1.0	200	45.6	22.8	2000	5	2.3	50	25
-27	31	Marsh/Muck	200	1.0	200	46.8	23.4	2000	5	2.3	51	26
-28	32	Marsh/Muck	200	1.0	200	48.0	24.0	2000	5	2.3	53	26
-29	33	Marsh/Muck	200	1.0	200	49.2	24.6	2000	5	2.3	54	27
-30	34	Marsh/Muck	200	1.0	200	50.4	25.2	2000	5	2.3	55	27
-31	35	Marsh/Muck	200	1.0	200	51.6	25.8	2000	5	2.3	56	28
-32	36	Marsh/Muck	200	1.0	200	52.8	26.4	2000	5	2.3	57	29
-33	37	Marsh/Muck	200	1.0	200	54.0	27.0	2000	5	2.3	59	29
-34	38	Marsh/Muck	200	1.0	200	55.2	27.6	2000	5	2.3	60	30
-35	39	Marsh/Muck	200	1.0	200	56.4	28.2	2000	5	2.3	61	30
-36	40	Marsh/Muck	200	1.0	200	57.6	28.8	2000	5	2.3	62	31
-37	41	Marsh/Muck	200	1.0	200	58.8	29.4	2000	5	2.3	63	32
-38	42	Marsh/Muck	200	1.0	200	60.0	30.0	2000	5	2.3	65	32
-39	43	Marsh/Muck	200	1.0	200	61.2	30.6	2000	5	2.3	66	33
-40	44	Marsh/Muck	200	1.0	200	62.4	31.2	2000	5	2.3	67	33
-41	45	Marsh/Muck	200	1.0	200	63.6	31.8	2000	5	2.3	68	34
-42	46	Marsh/Muck	200	1.0	200	64.8	32.4	2000	5	2.3	69	35
-43	47	Marsh/Muck	200	1.0	200	66.0	33.0	2000	5	2.3	71	35
-44	48	Marsh/Muck	200	1.0	200	67.2	33.6	2000	5	2.3	72	36
-45	49	Marsh/Muck	200	1.0	200	68.4	34.2	2000	5	2.3	73	36
-46	50	Marsh/Muck	200	1.0	200	69.6	34.8	2000	5	2.3	74	37
-47	51	Marsh/Muck	200	1.0	200	70.8	35.4	2000	5	2.3	75	38
-48	52	Marsh/Muck	200	1.0	200	72.0	36.0	2000	5	2.3	77	38
-49	53	Marsh/Muck	200	1.0	200	73.2	36.6	2000	5	2.3	78	39
-50	54	Marsh/Muck	200	1.0	200	74.4	37.2	2000	5	2.3	79	39
-51	55	Marsh/Muck	200	1.0	200	75.6	37.8	2000	5	2.3	80	40
-52	56	Marsh/Muck	200	1.0	200	76.8	38.4	2000	5	2.3	81	41
-53	57	Marsh/Muck	200	1.0	200	78.0	39.0	2000	5	2.3	83	41
-54	58	Marsh/Muck	200	1.0	200	79.2	39.6	2000	5	2.3	84	42
-55	59	Sand	500	1.0	500	82.2	41.1	5000	11	5.6	93	47
-56	60	Sand	500	1.0	500	85.2	42.6	5000	11	5.6	96	48
-57	61	Sand	500	1.0	500	88.2	44.1	5000	11	5.6	99	50
-58	62	Sand	500	1.0	500	91.2	45.6	5000	11	5.6	102	51
-59	63	Sand	500	1.0	500	94.2	47.1	5000	11	5.6	105	53
-60	64	Sand	500	1.0	500	97.2	48.6	5000	11	5.6	108	54
-61	65	Sand	500	1.0	500	100.2	50.1	5000	11	5.6	111	56
-62	66	Sand	500	1.0	500	103.2	51.6	5000	11	5.6	114	57
-63	67	Sand	500	1.0	500	106.2	53.1	5000	11	5.6	117	59

Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Lower Marl Depth, Top of Marl at EL. -65 FT
Computed By: JAI
Date: 07/02/2020
Reference Project 22 Westedge

Revised By: JAI
Date Revised: 07/06/2020

Reviewed By: KAH
Date Reviewed: 07/02/2020

Boring	
Drill Rig	CPT
Depth to Water	2
Su/sigmaz' NC	0.22
Factor of Safety	2
Nc*	10
Ground Surface	4 ft NAVD88

Pile Section	18-in. PSC
Pile area	324 in^2
	2.250 ft^2
Depth of Section	18 ft
Flange Width, bf	18 ft
Pile Perimeter	6.00 ft

Formation	Su (psf)	Elevation of Bottom of Layer
Upper Sand	300	-13
Marsh/Muck	200	-55
Sand	500	-65
Silty Sand/Marl	2600	

Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	18" PSC Pile Capacity	Factored 18" PSC Pile
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
-64	68	Sand	500	1.0	500	109.2	54.6	5000	11	5.6	120	60
-65	69	Silty Sand/Marl	2600	1.0	2600	124.8	62.4	26000	59	29.3	183	92
-66	70	Silty Sand/Marl	2600	1.0	2600	140.4	70.2	26000	59	29.3	199	99
-67	71	Silty Sand/Marl	2600	1.0	2600	156.0	78.0	26000	59	29.3	215	107
-68	72	Silty Sand/Marl	2600	1.0	2600	171.6	85.8	26000	59	29.3	230	115
-69	73	Silty Sand/Marl	2600	1.0	2600	187.2	93.6	26000	59	29.3	246	123
-70	74	Silty Sand/Marl	2600	1.0	2600	202.8	101.4	26000	59	29.3	261	131
-71	75	Silty Sand/Marl	2600	1.0	2600	218.4	109.2	26000	59	29.3	277	138
-72	76	Silty Sand/Marl	2600	1.0	2600	234.0	117.0	26000	59	29.3	293	146
-73	77	Silty Sand/Marl	2600	1.0	2600	249.6	124.8	26000	59	29.3	308	154
-74	78	Silty Sand/Marl	2600	1.0	2600	265.2	132.6	26000	59	29.3	324	162
-75	79	Silty Sand/Marl	2600	1.0	2600	280.8	140.4	26000	59	29.3	339	170
-76	80	Silty Sand/Marl	2600	1.0	2600	296.4	148.2	26000	59	29.3	355	177
-77	81	Silty Sand/Marl	2600	1.0	2600	312.0	156.0	26000	59	29.3	371	185
-78	82	Silty Sand/Marl	2600	1.0	2600	327.6	163.8	26000	59	29.3	386	193
-79	83	Silty Sand/Marl	2600	1.0	2600	343.2	171.6	26000	59	29.3	402	201
-80	84	Silty Sand/Marl	2600	1.0	2600	358.8	179.4	26000	59	29.3	417	209
-81	85	Silty Sand/Marl	2600	1.0	2600	374.4	187.2	26000	59	29.3	433	216
-82	86	Silty Sand/Marl	2600	1.0	2600	390.0	195.0	26000	59	29.3	449	224
-83	87	Silty Sand/Marl	2600	1.0	2600	405.6	202.8	26000	59	29.3	464	232
-84	88	Silty Sand/Marl	2600	1.0	2600	421.2	210.6	26000	59	29.3	480	240
-85	89	Silty Sand/Marl	2600	1.0	2600	436.8	218.4	26000	59	29.3	495	248
-86	90	Silty Sand/Marl	2600	1.0	2600	452.4	226.2	26000	59	29.3	511	255
-87	91	Silty Sand/Marl	2600	1.0	2600	468.0	234.0	26000	59	29.3	527	263
-88	92	Silty Sand/Marl	2600	1.0	2600	483.6	241.8	26000	59	29.3	542	271
-89	93	Silty Sand/Marl	2600	1.0	2600	499.2	249.6	26000	59	29.3	558	279
-90	94	Silty Sand/Marl	2600	1.0	2600	514.8	257.4	26000	59	29.3	573	287

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Attachment 5: Preliminary Subsurface Investigation Estimate

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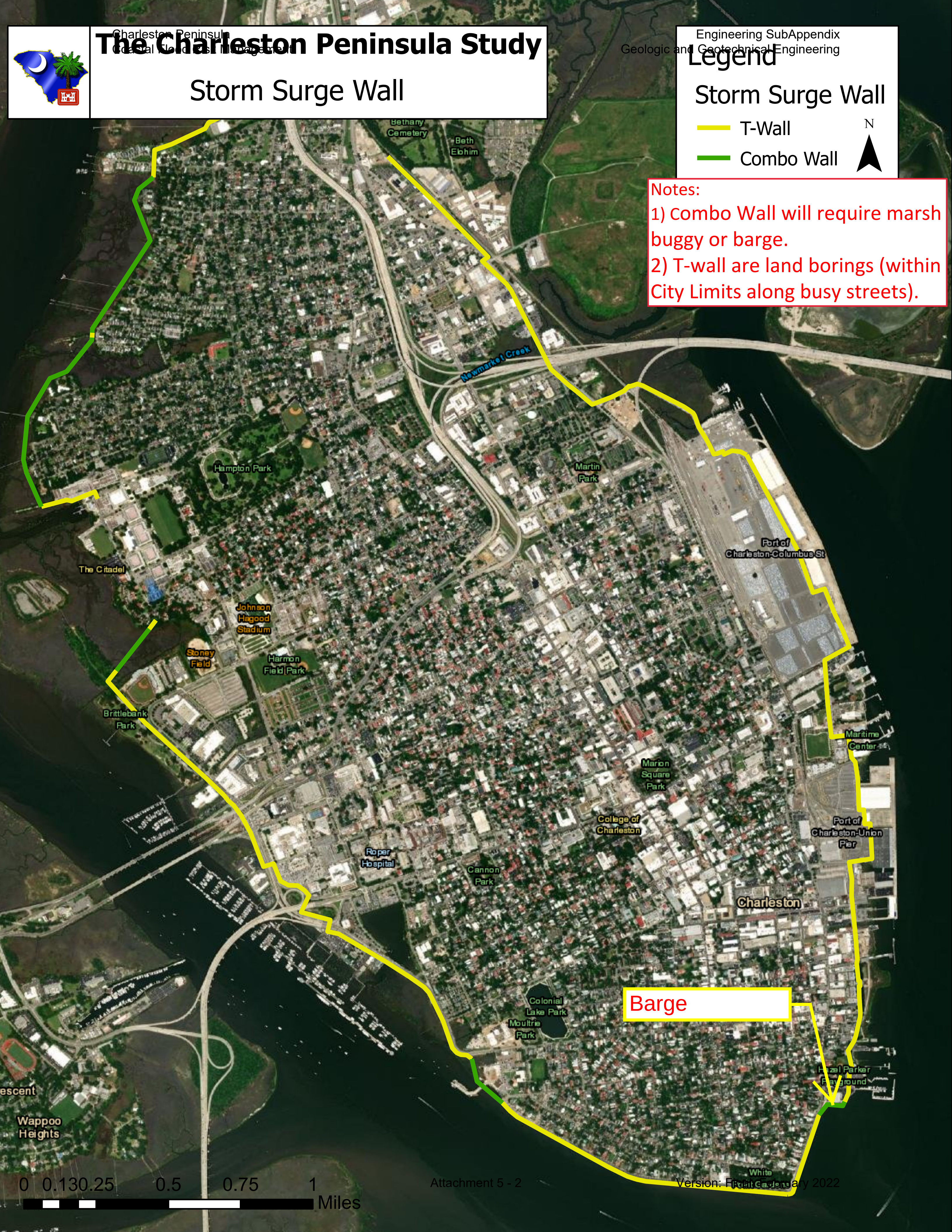
Charleston Peninsula Study
PED Phase Soil Exploration Estimate
Date: 12/02/2021
Estimator: Heckendorf

Subsurface Investigation	$	1,456,800.00
Lab Testing	$	308,084.00
Pressuremeter Testing	$	200,000.00
Soil Exploration Total:	**$**	**1,964,884.00**

Length in Marsh	8,665	ft
Width Disturbed	25	ft
Area disturbed	5.0	acres

Pile Load Testing **$2 million to $5 million**



Charleston Peninsula
Coastal Flood Risk Management
The Charleston Peninsula Study
Storm Surge Wall
Engineering SubAppendix
Geologic and Geotechnical Engineering
Legend
Storm Surge Wall
T-Wall
Combo Wall
N
Notes:
1) Combo Wall will require marsh buggy or barge.
2) T-wall are land borings (within City Limits along busy streets).
Bethany Cemetery
Beth Elohim
Newmarket Creek
Hampton Park
Martin Park
Port of Charleston-Columbus St
The Citadel
Johnson Hagood Stadium
Stoney Field
Harmon Field Park
Brittlebank Park
Maritime Center
Marion Square Park
College of Charleston
Port of Charleston-Union Pier
Roper Hospital
Cannon Park
Charleston
Colonial Lake Park
Moultrie Park
Barge
Hazel Parker Playground
White Point Garden
Wappoo Heights
escent
0 0.130.25 0.5 0.75 1
Miles
Attachment 5 - 2
Version: Final February 2022

Charleston Peninsula Study
Project Reaches & Segments
Date: 12/02/2021
Estimator: Heckendorf

Order	MA	foundation	type of wall	Length	start station	end station	Description	Spacing	Cross Sections	2 Hole Section	CPTs	Borings	GS	Cooper Marl	Length	Land	Land CPT	Land Boring	Barge	Barge CPT	Barge Boring	Marsh Buggy	Buggy CPT	Buggy Boring	LF of Marsh	Notes
1	WagTer	land	T-Wall	825	0+00	8+25.	Diesel Creek	500	2	1	3	1	9.8	-55	90.8	x	272	91								
2			combo wall	540	8+25	13+65.		250	3	2	5	1	6	-55	87							x	435	87	540	
3			T-Wall	675	13+65	20+40.		500	2	1	3	1	5.5	-55	86.5	x	260	87								
4		marsh	combo wall	2,898	20+40.	49+37.69	Upper Wagner Terrace	500	6	3	9	2	6	-55	87							x	783	174	2,898	
5		land	T-Wall	75	49+37.69	50+12.96	park	250	1	1	2		3.6	-55	84.6	x	169	0								
6		marsh	combo wall	3,044	50+12.96	80+56.76	Halsey to lower WT	500	7	4	11	2	6	-55	87							x	957	174	3,044	
7		land	T-Wall	980	80+56.76	90+36.46	Citadel	500	2	1	3		8.3	-55	89.3	x	268	0								
8	Wagner total			9,036					23	13	36	7					969	177		0	0		2,175	435	6,481	
9																										
10	Marina	land	T-Wall	175	0+00	1+74.96	Citadel	250	1	1	2		10.5	-55	91.5	x	183	0								
11		marsh	combo wall	808	1+74.96	9+82.49	citadel to joe	500	2	1	3	1	6	-55	87							x	261	87	808	
12		land	Twall	6,778	9+82.49	77+60.	Brittlebank to Marina	500	14	7	21	4	4.9	-55	85.9	x	1,804	344								
13		land	T-Wall with walkway	2,213	77+60.	99+73.03	Marina to Lockwood/Broad	500	5	3	8		4.9	-65	95.9	x	767	0								
14		marsh	combo	866	99+73.03	108+39.38	coast guard	500	2	1	3	1	6	-75	107							x	321	107	866	
15	Marina total			10,839					24	13	37	6					2,754	344		0	0		582	194	1,674	
16																										
17	battery	land	T-Wall	4,723	108+39.38	155+62.3	new city wall - low battery																			City Designed
18		land	T-Wall	1,343	155+62.3	169+04.98	high battery	500	3	1	4	1	4.5	-75	105.5				x	422	105.5					
19	Battery total			6,066					3	1	4	1					0	0		422	106		0	0	0	
20																										
21	port	marsh	combo wall w/backfill	510	169+04.98	174+15.08	off high battery and CYC	500	2	1	3	1	6	-75	107							x	321	107	510	
22		land	T-Wall	2,717	174+15.08	201+32.29	CYC -waterfront park	500	6	3	9	2	5.5	-60	91.5	x	824	183								
23		land	T-Wall	9,941	201+32.29	300+73.68	Lowes Hotel -thru Columbus St terminal	500	20	10	30	6	5	-65	96	x	2,880	576								
24	port total			13,169					28	14	42	9					3,704	759		0	0		321	107	510	
25																										
26	newmark	land	T-Wall	3,875	300+73.68	339+48.84	newmarket	500	8	4	12	2	5.3	-55	86.3	x	1,036	173								
27		land	T-Wall	3,198	339+48.84	371+47.2	newmarket	500	7	4	11	2	8.4	-55	89.4	x	983	179								
28		land	T-Wall	40	371+47.2	371+87.26	Upper new market	250	1	1	2		12.85	-55	93.85	x	188	0								
29	newmark total			7,114					16	9	25	4					2,207	351		0	0		0	0	0	
30																										
31			Total Length	46,224	8.8 miles			492	94	50	144	27					9,633	1,631		422	106		3,078	736	8,665	

Notes:

5.3 Borings per CPT

Spacing noted in Geo Appendix: 250 to 1,000 LF
Each reach has atleast 2 cross sections to analyze, unless length is less than 250 feet.
Cross sections have atleast 1 subsurface investigation; Each section has 1 subsurface investigations
Average spacing of soil exploration cross section is 492 feet.
Borings to include undisturbed samples to be used in CPT correlation.
Ratio of CPT to SPT is 5.3.
Depth of Exploration is 20 feet below expected tip elevation. Minimum of 6' embedment into Cooper Marl.

Land	Land
106	18
Barge	Barge
4	1
Marsh Buggy	Marsh Buggy
34	8

Charleston Peninsula Study
PED Phase Subsurface Investigation
12/02/2021
Drilling Estimate

Estimator: Heckendorf & Widincamp

Number

Type	Land	Barge	Marsh Buggy	**TOTAL**
CPT	106	4	34	144
Borings	18	1	8	27
Undisturbed Sample Boring	18	1	8	27

Length by Type

	Land	Barge	Marsh Buggy	**TOTAL**
CPT Length	9,633	422	3,078	**13,133**
Boring Length	1,631	106	736	**2,473**

TASK DESCRIPTION	Quantity	Unit	Rate	Cost	Notes	
1 SPT & 1 CPT Crew Mobilizations						
Mob	2	day	$2,500.00	$5,000.00		
Demob	2	day	$2,500.00	$5,000.00		
			Task Total:	***$10,000.00***		
Land CPTs, SPTs, and Undisturbed Sampling						
Daily Rate						
Drill Rig (no inspector)	36	day	$4,000.00	$144,000.00	1	per day
CPT Rig	36	day	$3,800.00	$136,800.00	3	per day
Traffic Control (includes CT fees)	1	lump	$46,000.00	$46,000.00	(1)	
			Task Total:	***$326,800.00***		
Marsh CPTs, SPTs, and Undisturbed Sampling						
Marsh Buggy Rental						
Mob/Demob	2	trip	$18,000.00	$36,000.00		
Setup (Assembly, rig placement, labor)	14	lump	$16,000.00	$224,000.00	(2)	
Monthly Rate	2	lump	$32,000.00	$64,000.00		
Breakdown (Disassembly, rig removal, labor)	14	lump	$16,000.00	$224,000.00		

Charleston Peninsula Study
PED Phase Subsurface Investigation
12/02/2021
Drilling Estimate

Estimator: Heckendorf & Widincamp

Number

Type	Land	Barge	Marsh Buggy	**TOTAL**
CPT	106	4	34	144
Borings	18	1	8	27
Undisturbed Sample Boring	18	1	8	27

Length by Type

	Land	Barge	Marsh Buggy	**TOTAL**
CPT Length	9,633	422	3,078	**13,133**
Boring Length	1,631	106	736	**2,473**

TASK DESCRIPTION	Quantity	Unit	Rate	Cost	Notes	
Contracting Costs	1	lump	$10,000.00	$10,000.00		
			Subtotal:	***$558,000.00***		
Barge Rental						
Mob/Demob	2	trip	$15,000.00	$30,000.00		
Setup (Assembly, rig placement, labor)	2	lump	$20,000.00	$40,000.00	(3)	
Daily Rate	6	day	$20,000.00	$120,000.00		
Breakdown (Disassembly, rig removal, labor)	2	lump	$20,000.00	$40,000.00		
Contracting Costs	1	lump	$10,000.00	$10,000.00		
			Subtotal:	***$240,000.00***		
SPT/UD/CPT					(4)	
Drill Rig (no inspector)	32	day	$4,000.00	$128,000.00	1	per day
CPT Rig	45	day	$3,800.00	$171,000.00	2	per day
			Subtotal:	***$299,000.00***		
			Task Total:	**$1,097,000.00**		

Charleston Peninsula Study
PED Phase Subsurface Investigation
12/02/2021
Drilling Estimate

Estimator: Heckendorf & Widincamp

Number

Type	Land	Barge	Marsh Buggy	**TOTAL**
CPT	106	4	34	144
Borings	18	1	8	27
Undisturbed Sample Boring	18	1	8	27

Length by Type

	Land	Barge	Marsh Buggy	**TOTAL**
CPT Length	9,633	422	3,078	**13,133**
Boring Length	1,631	106	736	**2,473**

TASK DESCRIPTION	Quantity	Unit	Rate	Cost	Notes
Miscellaneous					
Supplies	1	lump	$3,000.00	$3,000.00	
Data Analysis/QA Oversight	20	day	$1,000.00	$20,000.00	
			Task Total:	**$23,000.00**	
			TOTAL:	**$1,456,800.00**	
			10% Contingency:	$145,680.00	
			GRAND TOTAL:	**$1,602,480.00**	

Notes:
(1) Extensive traffic control required due to land borings being on / adjacent to highly traveled roadways.
(2) Marsh buggy can not travel into open water around docks therefore it will need to be broken down, moved, and reassemblied at next location. Estimate 7 different segements between docks. Two setups for each segment, one for CPT and one for SPT.
(3) One barge with one setup for CPT and one setup for SPT.
(4) Duration includes 1 day for each setup required.

Geotechnical - Technical Support Cost Estimate

Project: Charleston Peninsula Study Undisturbed and Disturbed Samples Testing

Date: 11/23/2021

Estimator: Heckendorf

Note: FY22 - OCT 2021 Testing Price Schedule

Borings: 18
Length: 1631
Sample Interval for Testing: 10
Samples to Test 164
Tubes per Boring 4
Tubes 72

Soil Testing for Disturbed & Undisturbed Samples					
Cost Item	**Cost Code**	**Quantity**	**Unit Cost**	**Total Cost**	Notes
Sieve Analysis with hydrometer - ASTM D422	SI101	195	$ 171.00	$ 33,345.00	75% Jar samples; 100% Tubes
Sieve Analysis without hydrometer - ASTM D422 or D6913 (6-7 sieve set)	SI102		$ 96.00	$ -	
Sieve Analysis (10-14 sieves) - ASTM D6913	SI103	41	$ 171.00	$ 7,011.00	25% Jar samples
Wash No. 200 for Soil - ASTM D1140	SI104		$ 40.00	$ -	
Visual % Shell Determination	SI105		$ 17.00	$ -	
Atterberg Limits One Point - ASTM D4318	SI201		$ 113.00	$ -	
Atterberg Limits Multi Point - ASTM D4318	SI202	195	$ 171.00	$ 33,345.00	75% Jar samples; 100% Tubes
Organic Content - ASTM D2974 Method C	SI203		$ 57.00	$ -	
Water Content - ASTM D2216	SI301	195	$ 17.00	$ 3,315.00	75% Jar samples; 100% Tubes
Specific Gravity - ASTM D854	SI302		$ 68.00	$ -	
Classification of Soil - ASTM D2487	SI303	236	$ 23.00	$ 5,428.00	100% Jar and Tubes
Visual Classification of Undisturbed Samples - ASTM D2488	SI304		$ 98.00	$ -	
Visual Classification of Jar Samples - ASTM D2488	SI305		$ 23.00	$ -	
Water Content by Microwave Oven - ASTM D4643	SI306		$ 17.00	$ -	
Organic Content - ASTM D2974 Method C	SI307		$ 57.00	$ -	
Carbnate Content - ASTM D4373	SI308		$ 57.00	$ -	
pH Determination - ASTM D4972	SI309	124	$ 57.00	$ 7,068.00	50% Jars
Sulfates content	SI310	82	$ 33.00	$ 2,706.00	50% Jars
Chlorides content	SI311	82	$ 33.00	$ 2,706.00	50% Jars
Electrical Resistivity - ASTM G187	SI312		$ 110.00	$ -	
Standard Compaction - ASTM D698 Method A	SE101		$ 341.00	$ -	
Standard Compaction - ASTM D698 Method B & C	SE102		$ 420.00	$ -	
Modified Compaction - ASTM D1557 Method A	SE201		$ 375.00	$ -	
Modified Compaction - ASTM D1557 Method B & C	SE202		$ 454.00	$ -	
Specific Gravity of Plus No. 4 - ASTM C127 (for oversize correction ASTM D4718)	SE203		$ 111.00	$ -	
CBR Single Point at Specified %Compaction & Moisture - ASTM D1883	SE204		$ 317.00	$ -	
Permeability Constant Head - ASTM D2434 (granular)	SE301	41	$ 332.00	$ 13,612.00	25% Jar samples
Permeability Falling Head Rising Tail - ASTM D5084 (fine)	SE302	18	$ 426.00	$ 7,668.00	25% Tubes
Unconfined Compression clayey materials - ASTM D2166 (soil)	SE401		$ 171.00	$ -	
Unconfined Compression indurated clays, lthified materials - ASTM D2166 (soil)	SE402		$ 199.00	$ -	
Denisty in Shelby Tube - ASTM D7263 Method B	GGe16		$ 68.00	$ -	
UU Triaxial - ASTM D2850 (single points)	SE601	216	$ 244.00	$ 52,704.00	100% Tubes; 3 specimens per test
CU Triaxial - ASTM D4767 (single points)	SE602	216	$ 398.00	$ 85,968.00	100% Tubes; 3 specimens per test
Consolidation 10 loads, 8 curves - ASTM D2435	SE501	72	$ 739.00	$ 53,208.00	100% Tubes
Consolidation Additional Loading - ASTM D2435	SE502		$ 74.00	$ -	
Dispersive Clay Double Hydrometer - D4221	SE510		$ 110.00	$ -	
Dispersive Clay PinHole Dispersion - D4647	SE511		$ 440.00	$ -	
Direct Shear of soil (single point) - ASTM D3080	SE701		$ 284.00	$ -	
Residual Shear of soil (single point) repeatative shear - ASTM D3080	SE702		$ 671.00	$ -	
Sample Handling, Processing, Blending & Remolding	GGe16		$ 55.00	$ -	
Labor Cost for Senior Engineer Consulting	GGe42	0	$ 168.00		
Shipping Materials and cost	GGe16		$ 550.00	$ -	
Admin expenses, Reporting & Consulting	GGe42	1		$ -	
Overall Soil Testing Cost:				$ 308,084.00	

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US Army Corps of Engineers®

Charleston District

CHARLESTON PENINSULA, SOUTH CAROLINA, A COASTAL STORM RISK MANAGEMENT STUDY

Charleston, South Carolina

ENGINEERING APPENDIX - B
Hydraulics and Hydrology SUB-APPENDIX 3
(Interior Drainage Analysis)

February 2022

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Contents

List of Tables

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CHAPTER 1 - INTRODUCTION

1.1 STUDY OVERVIEW

The Charleston Peninsula Coastal Storm Risk Management Study (CSRM) is investigating coastal storm impacts on the Charleston peninsula. In partnership with the City of Charleston and its stakeholders, the Project Delivery Team (PDT) is exploring effective, economically viable and environmentally-sound solutions to mitigate risks and build enduring coastal storm resiliency.

The Tentatively Selected Plan (TSP) at Feasibility Phase includes both structural and non-structural flood risk management measures. The structural measure relevant for the interior drainage study is the proposal of a storm surge barrier with a design elevation of 12 feet NAVD88. This flood risk management measure greatly reduces the risk of flooding from coastal storm surge up to the level of design, however, areas protected from exterior flood elevations are subject to interior residual flooding from stormwater runoff. Thus, interior drainage facilities may be required to safely store and discharge the runoff to limit interior residual flooding. In the case of Downtown Charleston, SC, there are not many options for storing stormwater therefore allowing the stormwater to discharge via gravity flow (low exterior conditions) through the proposed wall and/or discharging the stormwater via pumping (typically during high exterior elevations) are the focus of the study. The interior areas were studied to determine the specific nature of flooding and to formulate drainage alternatives to implement as part of the alignment.

In accordance with USACE Engineer Manual (EM) 1110-2-1413, *Hydrologic Analysis of Interior Areas*, the interior drainage features are evaluated separately from the alignment to determine what project features are needed to provide interior flood relief such that during low exterior stages (gravity conditions) the local storm drainage system functions essentially as it did without the project in place up to that of the storm drainage design. The City of Charleston indicates much of the peninsula storm pipes reach capacity near the 10% AEP storm (rainfall) event with some areas are designed to lesser pipe flow capacities. Such information suggests that surface-flow runoff becomes a larger component of drainage for rain events above the stated design capacities.

A study approach was defined and conducted using the Hydrologic Engineering Center's (CEIWR-HEC) River Analysis System (HEC-RAS) two-dimensional modeling software to determine the plans that can be implemented as part of the project alignment and that appropriately mitigate the interior residual flooding. The results for future without- and with-project conditions were incorporated into the Hydrologic Engineering Center's (CEIWR-HEC) Flood Damage Reduction Analysis (HEC-FDA) to compute Equivalent Annual Damages (EAD) and Average Annual Damages (AAD) for describing the residual risk for the interior area. The HEC-FDA assessment served as an economic tool to determine the interior drainage features and their capacities (storm gates/pump stations) necessary for the project to perform acceptably and efficiently.

A rainfall-tide correlation assessment was completed and documented in Section 2.2. A climate change to inland hydrology assessment was completed and documented in Appendix 1 of this report.

*All elevations in this report are referenced to the North American Vertical Datum of 1988 (NAVD88).

1.2 DESCRIPTION OF PROJECT AREA AND VICINITY

Centrally located along the coast of South Carolina, the Charleston Peninsula project area is approximately 8 square miles, located between the Ashley and Cooper Rivers (Figure 1.2.1). Charleston Harbor is formed by the confluence of the Cooper, Ashley, and Wando Rivers before discharging into the

Atlantic Ocean. It includes the tidal estuary of the lower 12 miles of the Cooper River and the four miles of open bay between the confluence of the Ashley and Cooper Rivers and the Atlantic Ocean. The Cooper River contributes most of the freshwater inflow to the system and is the largest of the estuaries, extending about 57 miles from the harbor entrance to the Jefferies Hydroelectric Station at Lake Moultrie dam in Pinopolis, SC. The Cooper River flows are controlled under a contractual agreement with USACE to reduce shoaling in Charleston Harbor federal navigation channel. The flows are limited to a 4,500 cfs (cubic feet per second) daily average. The Charleston Peninsula consists of urban drainage infrastructure such as stormwater pipes, deep drainage shafts and tunnels, culverts, pump stations, and a battery seawall located near the southern tip.

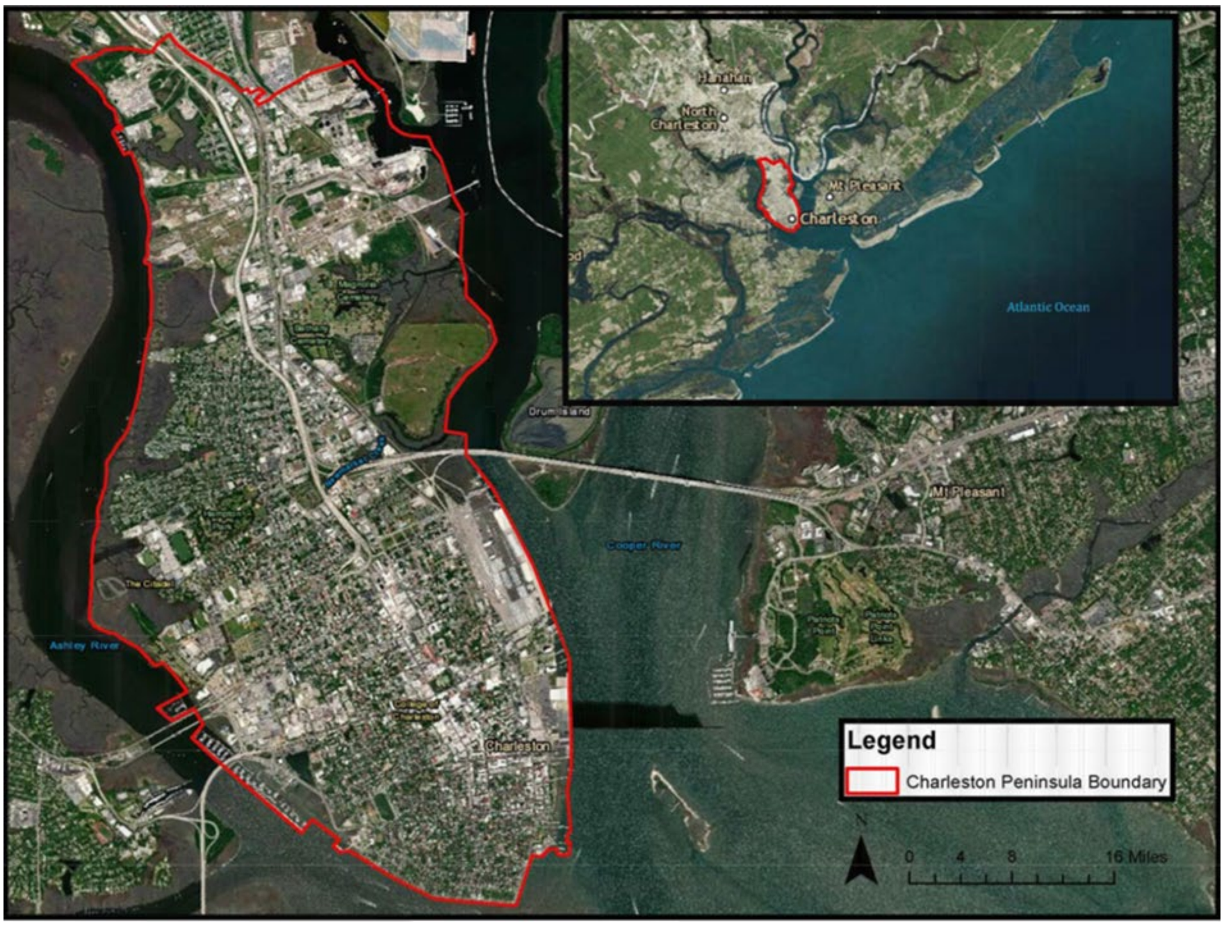

Figure 1.2.1 Charleston Study Area

Figure 1.2.2. Interior Drainage Study Area

1.3 SOFTWARE APPLICATIONS

HEC-RAS 6.0
The Hydrologic Engineering Center's (CEIWR-HEC) River Analysis System (HEC-RAS) is used to model the complex flow of rainfall runoff within the interior area and evaluate different hydraulic alternatives, such as storm gates and pumps. HEC-RAS 2D features were utilized in this effort.

ESRI ArcMap 10.7.1

The Environmental Systems Reach Institute (ESRI) Geographic Information Systems (GIS) layers and RAS Mapper tools are used to geo-reference structures and features within the HEC-RAS model. ArcMap is also used to create maps for visualization of project features and/or inundation maps.

HEC-FDA 1.4.2

The Hydrologic Engineering Center's (CEIWR-HEC) Flood Damage Reduction Analysis (HEC-FDA) software was used by the economics team member to compute the Expected Annual Damage (EAD), equivalent annual damages, and average annual damages for describing the residual risk for the interior area.

CHAPTER 2 - PROJECT AREA CONDITIONS

2.1 HISTORICAL STORMS

Hazel (October 1954) and Gracie (September 1959) have been the most memorable storms in recent years. Hazel, a Category 4 storm, made landfall near Little River, S.C., with 106-miles per hour winds and 16.9-foot storm surge. One person was killed, and damage was estimated at $27 million.
Gracie (September 1959), a Category 4 hurricane, made landfall on St. Helena Island with 130 mph winds and continued toward the north-northwest. Heavy damage occurred along the coast from Beaufort to Charleston. Heavy rains caused flooding through much of the State and crop damage was severe. NOAA's Hurricane Re-analysis Project upgraded Gracie from a Category 3 to a Category 4 hurricane in June 2016. Tide level reached 5.0 feet NAVD88.

Hugo (September 1989) made landfall near Sullivan's Island with 120 knot winds. It continued a northwest track at 25-30 miles per hour and maintained hurricane force winds as far inland as Sumter. Hugo exited the State southwest of Charlotte, N.C., before sunrise on September 22. The hurricane caused 13 directly related deaths and 22 indirectly related deaths, and it injured several hundred people in South Carolina. Damage in the State was estimated to exceed $7 billion, including $2 billion in crop damage. The forests in 36 counties along the path of the storm sustained major damage. Tide level reached 9.39' NAVD88.

Source:
https://tidesandcurrents.noaa.gov/waterlevels.html?id=8665530&units=standard&bdate=19890917&edate=19890925&timezone=GMT&datum=NAVD&interval=hl&action=)

From 1990 to 2015, South Carolina had only had five weak tropical cyclone landfalls along the coast: Tropical Storm Kyle (35 kts) in 2002, Hurricane Gaston (65 kts) and Hurricane Charley (70 kts) in 2004, Tropical Storm Ana (40 kts) in 2015, and Tropical Depression Bonnie (30 kts) in 2016. Bonnie developed north of the Bahamas and strengthened into a TS as it moves northwest toward the GA/SC coasts, eventually weakening to a TD before making landfall near Charleston. Produced heavy rainfall (widespread 3-7 inches with local amounts over 10 inches), mainly north of I-126, which led to significant flooding. During September 1999 Hurricane Floyd, a very large storm, came very close to the South Carolina coast, then made landfall near Cape Fear, North Carolina. Hurricane Floyd triggered mandatory coastal evacuations along the South Carolina coast. Heavy rain of more than 15 inches fell in parts of Horry County, S.C., causing major flooding along the Waccamaw River in and around the city of Conway for a month.

Mathew (October 2016) moved north and then northwest through the Caribbean Sea and then through the Bahamas while strengthening to a Category 4 hurricane. Tracked just off the east coast of FL and GA while weakening to a Category 1 storm before making landfall near McClellanville, SC with winds near 85 mph. Produced hurricane force wind gusts along the entire coast, significant coastal flooding from high storm tides (including a record level at Fort Pulaski), and very heavy rainfall (widespread 6 to 12 inches with locally higher amounts near 17 inches) which led to significant freshwater flooding. Tide level reached 6.14 feet NAVD88.

Irma (Sep 2017) made landfall in the Florida Keys as a Category 4 hurricane and then moved along the southwest coast of Florida as a Category 3 hurricane. The storm then moved north near the west coast of Florida while weakening to a tropical storm before moving into southwest Georgia and continuing to weaken. Produced significant coastal flooding, wind gusts near hurricane-force along with 4 tornadoes, flooding rainfall and river flooding across southeast SC/GA. NOAA tide level reached elevation 6.71 feet NAVD88.

Florence (Sept 2018) made landfall near Wrightsville Beach, NC as a Category 1 hurricane before slowing down and weakening to a TS. The storm then moved southwest near the northern SC coast before shifting west toward the SC Midlands and weakening to a TD. Produced some tropical storm force wind gusts and several inches of rain, mainly north of Charleston.

Michael (October 2018) made landfall near Mexico Beach, FL as a Category 4 hurricane and then moved northeast through southwest GA as a hurricane before weakening to a TS before reaching central SC. Produced tropical storm force winds and several inches of rainfall across much of southeast SC/GA which led to many fallen trees and some power outages.

A historic flooding event affected the Carolinas from October 1-5, 2015. A stalled front offshore combined with deep tropical moisture streaming northwest into the area ahead of a strong upper-level low pressure system to the west and Hurricane Joaquin well to the east. This led to historic rainfall with widespread amounts of 15-20 inches and localized amounts over 25 inches, mainly in the Charleston tri-county area. Flash flooding was prevalent and led to significant damage to numerous properties and roads and many people having to be rescued by emergency personnel. In addition, tides were high due to the recent perigean spring tide and persistent onshore winds, exacerbating the flooding along the coast, especially in downtown Charleston.

2.2 RAINFALL-TIDE CORRELATION ASSESSMENT

For the with- and without-project conditions, the exterior stage is an important factor in the drainage of the interior stormwater runoff. The exterior stage is controlled by the tidal cycle and storm surge elevations during storm events. Inland, the interior surface runoff is conveyed out into the Cooper River and Ashley River, which join at the Charleston Harbor, via stormwater outfalls (pipes/culverts) and/or existing pump stations. If both sources of flooding occur at the same time, the flooding effects are exacerbated, and pump stations become a major component of interior drainage relief until the high tides/storm surge recede.

In the without-project condition, during high exterior stages (tide/storm surge) that rise above the outfall opening, the gravity driven outfalls may incur significant tidal backflow if no check valve is in place and/or cease to drain the interior area if a check valve is in place. Similarly, if a new coastal flood risk management structure is introduced (with-project condition) to reduce the risk of storm surge entering the study area, the existing outfalls, under high exterior (tailwater) stage conditions would incur tidal backflow and/or cease to drain as previously mentioned. Therefore, it is important to develop an understanding of whether there is a relationship between interior surface runoff and exterior tidal events in both with- and without-project conditions.

To understand the relationship between the interior and exterior stage conditions, if any, a correlation assessment needs to be performed. In accordance with EM 1110-2-1413, the correlation assessment should include a data assessment of the correlation, dependence, and coincidence of the interior and exterior stage relationship.

2.2.1 DATA SOURCES

The National Oceanic and Atmospheric Administration (NOAA) online tools were utilized for the assessment by collecting precipitation (rainfall) and tide data. The tools are referenced in the following paragraphs.

Data from two rainfall gages were gathered for the assessment: Station ID (WBAN:13880) – Charleston Intl. Airport and Station ID (USW00013782) – Downtown Charleston, SC. Hourly rainfall data was retrieved from the Charleston Intl. Airport station for the time 2012 to 2020. Hourly rainfall data and daily rainfall data was retrieved for the Downtown Charleston station from 1948 to 2014 (hourly) and from 1930 to 2020 (daily) The data was retried using the following tool: (https://www.ncdc.noaa.gov/cdo-web/datatools/findstation). NOAA indicates no hourly rainfall data is collected at the downtown station after 2014.

The tide elevation data for Station 8665530 (Charleston, Cooper River Entrance SC), was obtained directly from NOAA's Tides and Currents web site (http://tidesandcurrents.noaa.gov/). Peak tide elevations from 30 historic events were gathered. The daily rainfall data correlating to the date of the peak tide were tabulated and plotted to describe the correlation between tidal flooding and rainfall.

In addition to the rain-tide correlation for historic flooding, a rain-tide correlation was also completed for daily rain and tide from 2010 to 2021. NOAA Tides and Currents has limits for retrieving tide data. There are work arounds that may require extensive effort or knowledge for scripting, querying, and cleaning data for appropriate correlation of rain-tide. Using NOAA's Data API (Application Programming Interface), the high/low tide data from station 8665530 were retrieved but limited to one-year increments for the years 2010 through 2021. The high/low tide data was filtered to only present the peak daily tides. The data was cleaned to remove duplicate data points and or missing data points. The peak daily tides were then correlated to daily rainfalls to present a daily rain-tide correlation for the years 2010 through 2021. This is not an extensive dataset however the historic rain-tide correlation and daily rain-tide correlation provide insight into the correlation, coincidence, and dependence. The source for NOAA's Data API: (https://tidesandcurrents.noaa.gov/api-helper/url-generator.html)

Other sources of data are presented in this section of the report to describe flood categories and storm frequencies. The National Weather Service (NWS) provides flood categories per tide elevation at the Charleston tide gage (Table 2.2.1). The NWS currently depicts flood categories for Charleston, SC with moderate flooding occurring at tidal elevations 4.36 feet NAVD88 and major flooding at 4.86 feet NAVD88. As part of the Charleston Peninsula CSRM, ERDC generated Stillwater elevation frequencies using the Advanced Circulation (ADCIRC) model (Table 2.2.2) at five locations around the peninsula.

Based on the ADCIRC generated Stillwater elevations, a 4.86 feet Stillwater elevation is approximately a 50% AEP (Annual Exceedance Probability) elevation at present day. For rainfall frequencies, NOAA Atlas 14 provides rainfall frequency estimates with 90% confidence intervals for AEP's ranging from 99% to 0.1% AEP (Table 2.2.3).

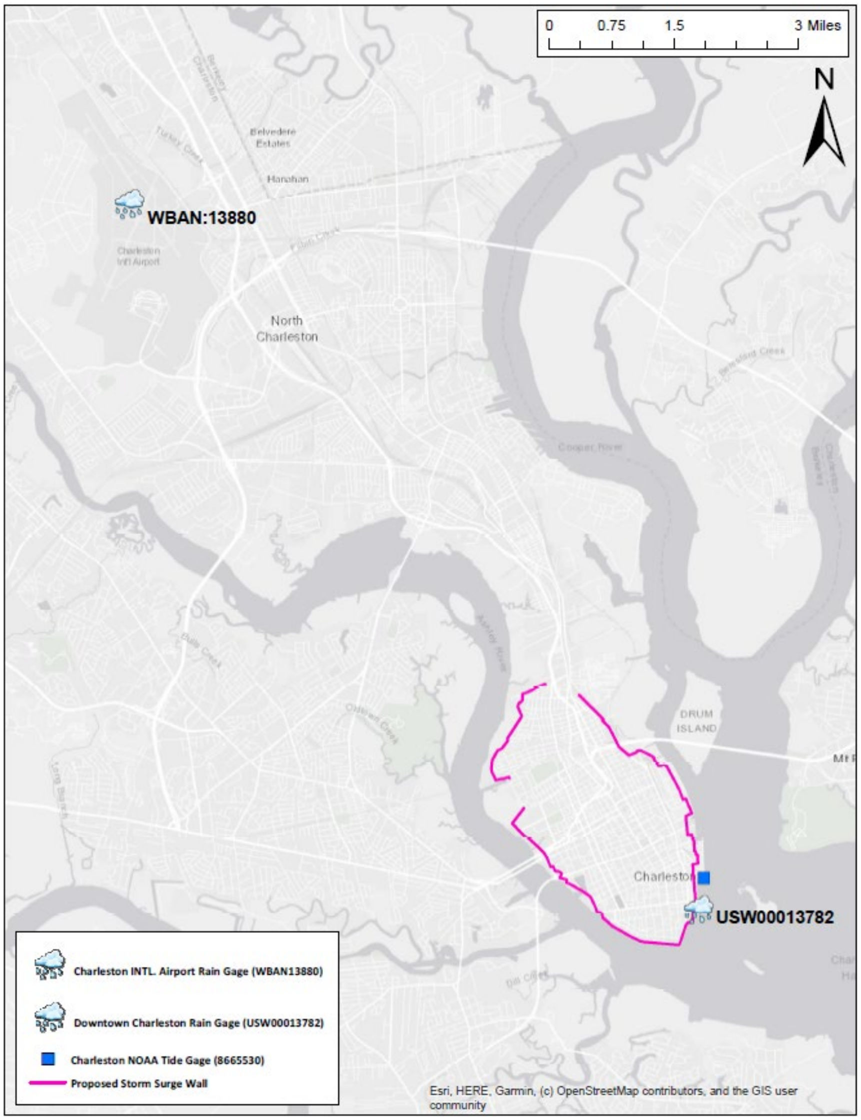

Figure 2.2.1 Study Area and Gage Station Locations

Table 2.2.1 National Weather Service Flood Categories

Flood Categories	MLLW (ft.)	NAVD88 (ft.)	*NAVD88 (ft.) Year 2032
Action Stage	6.5	3.36	3.92
King Tide	6.6	3.46	4.02
Minor Flooding	7.0	3.86	4.42
Moderate Flooding	7.5	4.36	4.92
Major Flooding	8.0	4.86	5.42
*The elevations for the year 2032 were simply estimated by adding 0.56 feet to the current elevations.			

Table 2.2.2 FEMA/ERDC Annual Exceedance Probability (AEP) Stillwater Levels

	Stillwater Levels (Existing without SLR)								
	AEP (%)								
area	50	20	10	4	2	1	0.5	0.2	0.1
Wagener Terrace	4.7	5.1	5.3	5.9	8.3	10.3	12.1	14.4	16.2
Marina	4.7	5.1	5.3	5.8	8.4	10.4	12.2	14.6	16.5
Newmarket	4.7	5.1	5.3	5.8	8.3	10.3	12.2	14.6	16.5
Port	4.7	5.0	5.3	5.8	8.3	10.3	12.2	14.7	16.5
Battery	4.7	5.0	5.3	5.8	8.3	10.4	12.3	14.7	16.6
	Stillwater Levels (2032 SLR +0.56 ft.)								
	AEP (%)								
area	50	20	10	4	2	1	0.5	0.2	0.1
Wagener Terrace	5.3	5.7	5.9	6.4	8.9	10.9	12.6	15.0	16.8
Marina	5.3	5.6	5.9	6.4	8.9	11.0	12.8	15.2	17.0
Newmarket	5.3	5.6	5.9	6.4	8.9	10.9	12.7	15.2	17.0
Port	5.2	5.6	5.8	6.3	8.9	10.9	12.7	15.2	17.1
Battery	5.2	5.6	5.8	6.3	8.8	10.9	12.8	15.3	17.2
	Stillwater Levels (2082 SLR +1.65 ft.)								
	AEP (%)								
area	50	20	10	4	2	1	0.5	0.2	0.1
Wagener Terrace	6.4	6.8	7.0	7.5	10.0	12.0	13.7	16.1	17.9
Marina	6.4	6.7	7.0	7.5	10.0	12.1	13.9	16.3	18.1
Newmarket	6.4	6.7	6.9	7.5	10.0	12.0	13.8	16.3	18.1
Port	6.3	6.7	6.9	7.4	9.9	11.9	13.8	16.3	18.2
Battery	6.3	6.7	6.9	7.4	9.9	12.0	13.9	16.4	18.3

Table 2.2.3 Precipitation Frequency Estimates (NOAA Atlas 14)

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)1										
Duration	Annual Exceedance Probabiltiy (AEP %)									
	99	50	20	10	4	2	1	0.5	0.2	0.1
24-hr	**3.48**	**4.52**	**5.47**	**6.47**	**7.89**	**9.04**	**10.3**	**11.5**	**13.3**	**14.8**
2-day	**4.06**	**4.92**	**6.29**	**7.41**	**8.98**	**10.3**	**11.6**	**13**	**15**	**16.6.**
3-day	**4.33**	**5.24**	**6.67**	**7.81**	**9.42**	**10.7**	**12.1**	**13.5**	**15.5**	**17.1**

*The 10% AEP values for 24-hr and 3-day are highlighted for reference to the historic tide events with corresponding rainfall shown in Table 2.2.4.

2.2.2 CORRELATION

Historic tide “crest” events were identified and tabulated with the corresponding rainfall events. Table _ displays thirty historic crest events with daily rainfall data for the day of the peak crest and +/- one day before and after the peak crest. The tabulated data was also plotted and shown in Figures 2.2.2 and 2.2.3. The data for +/- one day was also included because stalled storms may bring multi-day rainfall events in which stalled storms may also produce longer duration high tide elevations.

Reviewing the data of the thirty historic crests events, two (8/11/1940 and 10/3/2015) contain accompanying 24-hr rainfall totals greater than a 10% AEP for the day the peak crest occurs. The 1940 event shows 7.66 inches of rainfall the day of the peak tide crest which is roughly a 4% AEP. The 2015 event shows 9.25 inches of rainfall the day of the peak tide crest which is roughly a 2% AEP. It is noted the 2015 historic rainfall event took place from approximately October 1-5, 2015 and produced rainfall totals upwards of 15 inches throughout the stalled storm.

Table 2.2.4 Historical Crest Events with Corresponding Rainfall

	Historic Crests (MLLW)	Historic Crests NAVD88	24-HR Rainfall on Day of Crest (inches)		24-HR Rainfall 1 Day Before Crest (inches)		24-HR Rainfall 1 Day After Crest (inches)		Max 24-HR Rainfall of 3 Days (inches)	Rainfall Sum over 3-Day Period (inches)
1	12.52	9.38	9/22/1989	0.87	9/21/1989	5.99	9/23/1989	0.15	5.99	7.01
2	10.23	7.09	8/11/1940	7.66	8/10/1940	0.03	8/12/1940	1.94	7.66	9.63
3	9.92	6.78	9/11/2017	4.53	9/10/2017	0.02	9/12/2017	0.01	4.53	4.56
4	9.29	6.15	10/8/2016	3.84	10/7/2016	4.36	10/9/2016	0	4.36	8.2
5	8.81	5.67	1/1/1987	0.97	12/31/1986	0.1	1/2/1987	0	0.97	1.07
6	8.76	5.62	11/24/2018	0.44	11/23/2018	0	11/25/2018	0	0.44	0.44
7	8.69	5.55	10/27/2015	0.51	10/26/2015	0.02	10/28/2015	0.28	0.51	0.81
8	8.64	5.5	5/28/1934	1.48	5/27/1934	0	5/29/1934	0.15	1.48	1.63
9	8.64	5.5	9/4/1979	6.06	9/3/1979	0.34	9/5/1979	0.7	6.06	7.1
10	8.46	5.32	11/2/1947	0.72	11/1/1947	0	11/3/1947	0	0.72	0.72
11	8.29	5.15	10/3/2015	9.25	10/2/2015	1.41	10/4/2015	2.49	9.25	13.15
12	8.27	5.13	10/28/2015	0.28	10/27/2015	0.51	10/29/2015	0	0.51	0.79
13	8.21	5.07	10/4/2015	2.49	10/3/2015	9.25	10/5/2015	0.14	9.25	11.88
14	8.15	5.01	10/15/1947	0.85	10/14/1947	0.58	10/16/1947	0.56	0.85	1.99
15	8.14	5	9/29/1959	3.96	9/28/1959	0.55	9/30/1959	0	3.96	4.51
16	8.14	5	11/23/2018	0	11/22/2018	0	11/24/2018	0.44	0.44	0.44
17	8.11	4.97	6/22/2009	0	6/21/2009	0	6/23/2009	0	0	0
18	8.08	4.94	8/30/2019	0	8/29/2019	0	8/31/2019	0.05	0.05	0.05
19	8.06	4.92	12/24/2019	2.78	12/23/2019	0	12/25/2019	0.34	2.78	3.12
20	8.06	4.92	6/23/2009	0	6/22/2009	0	6/24/2009	0	0	0
21	8.05	4.91	12/9/2018	1.06	12/8/2018	0.82	12/10/2018	0	1.06	1.88
22	8.05	4.91	9/29/2015	0	9/28/2015	0	9/30/2015	0.26	0.26	0.26
23	8.03	4.89	2/20/2019	0.25	2/19/2019	0.01	2/21/2019	0	0.25	0.26
24	8.02	4.88	8/29/2019	0	8/28/2019	0.06	8/30/2019	0	0.06	0.06
25	8.01	4.87	6/18/1982	4.27	6/17/1982	1.79	6/19/1982	0	4.27	6.06
26	8.01	4.87	12/31/1994	0	12/30/1994	0	1/1/1995	0.01	0.01	0.01
27	8.01	4.87	7/21/2001	0.03	7/20/2001	2.34	7/22/2001	0	2.34	2.37
28	8.01	4.87	9/28/2015	0	9/27/2015	0	9/29/2015	0	0	0
29	8	4.86	1/30/2010	1.06	1/29/2010	0	1/31/2010	0	1.06	1.06
30	7.28	4.14	9/15/1999	3.62	9/14/1999	0.77	9/16/1999	0	1.61	4.39

Denotes a greater than or equal to 10% AEP 24-hr rainfall event

Denotes a greater than or equal to 10% AEP 3-day rainfall event

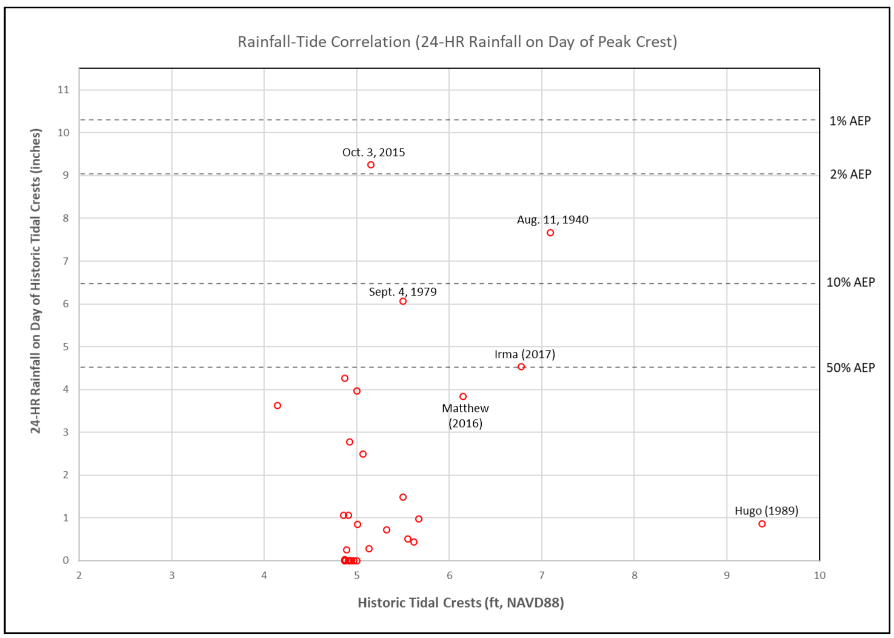

Figure 2.2.2 Historic Storms Rainfall-Tide Correlation Plot

The rainfall-tide correlation for peak daily tides and daily rainfall was plotted (Figure 2.2.3) for the years 2010 through 2021. As mentioned, NOAA tides and currents webpages contains data retrieval limitations which leads to extensive data gathering and/or strenuous work arounds for retrieval, therefore data for the most recent eleven years was gathered and cleaned (missing data/duplicate points removal) for correlation in Microsoft Excel to provide a visual representation of the rainfall-tide correlation for the daily data.

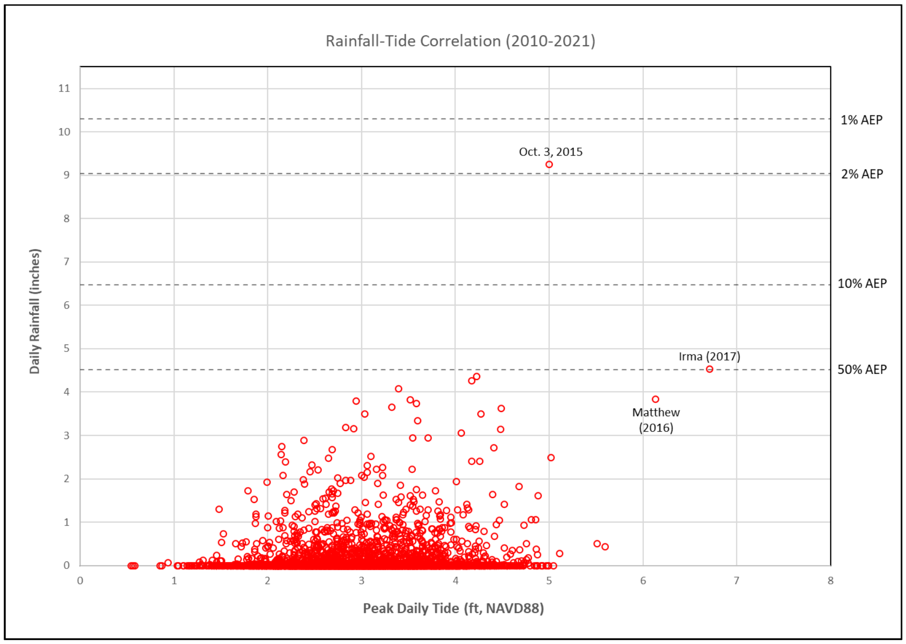

Figure 2.2.3 2010 to 2021 Rainfall-Tide Correlation Plot

2.2.3 DEPENDENCE

It is understood that the storms that typically produce tidal surges (i.e., hurricanes and nor'easters) can also produce somewhat significant rainfall. Likewise, high rainfall events may be accompanied by some degree of storm surge. If this were not true, the high surge events would not likely have any rainfall, and the paired data in Figures 2.2.3 and 2.2.2 would fall much closer to each axis. The historical data also shows that significant storm surge can occur without much rainfall (i.e., Hurricane Irma). As expected, the figures reveal some dependence between tide elevation and rainfall.

2.2.4 COINCIDENCE

The coincidence between the interior and exterior conditions involves the timing of peak rainfall and peak exterior stages. In the exterior conditions, the timing of the peak exterior stage is unpredictable because of the impacts of tidal fluctuations to the overall storm surge elevation. Therefore, predicting the coincidence of the peak exterior event and the peak interior flows is uncertain. Assuming the interior and exterior events occur at the same time could be considered the worst-case scenario or conservative approach for modeling coincidence. Some historic events were plotted to assess the coincident peaks of rainfall and tide.

The following figures show the rain/tide coincidence plots for four events. The plots reveal varying results for the coincidence of peak rain and tide.

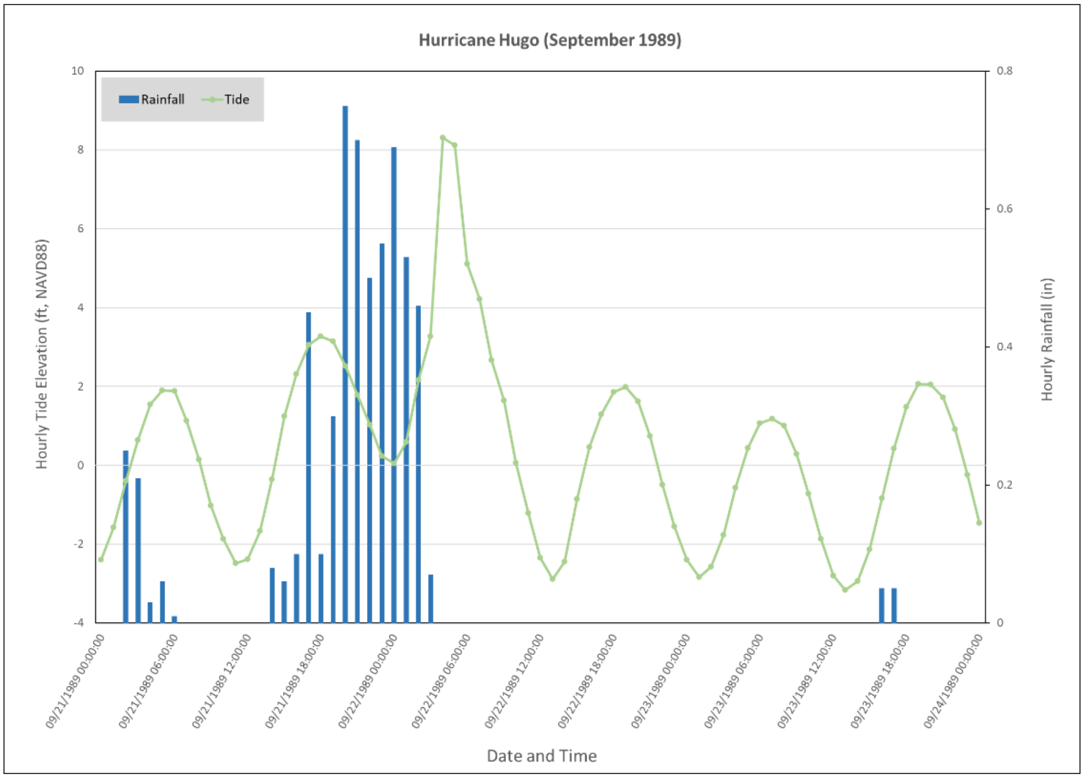

Figure 2.2.4 Hurricane Hugo – Rainfall/Tide Coincidence Plot

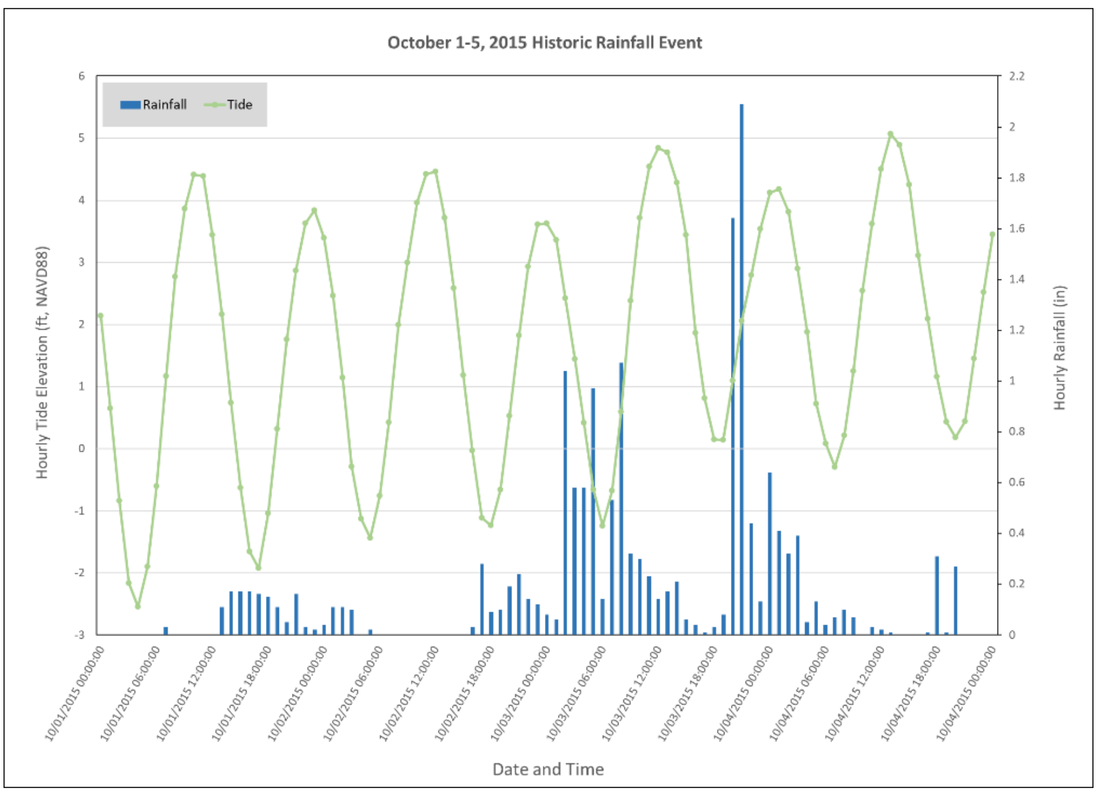

Figure 2.2.5 October 2015 Historical Rainfall Event – Rainfall/Tide Coincidence Plot

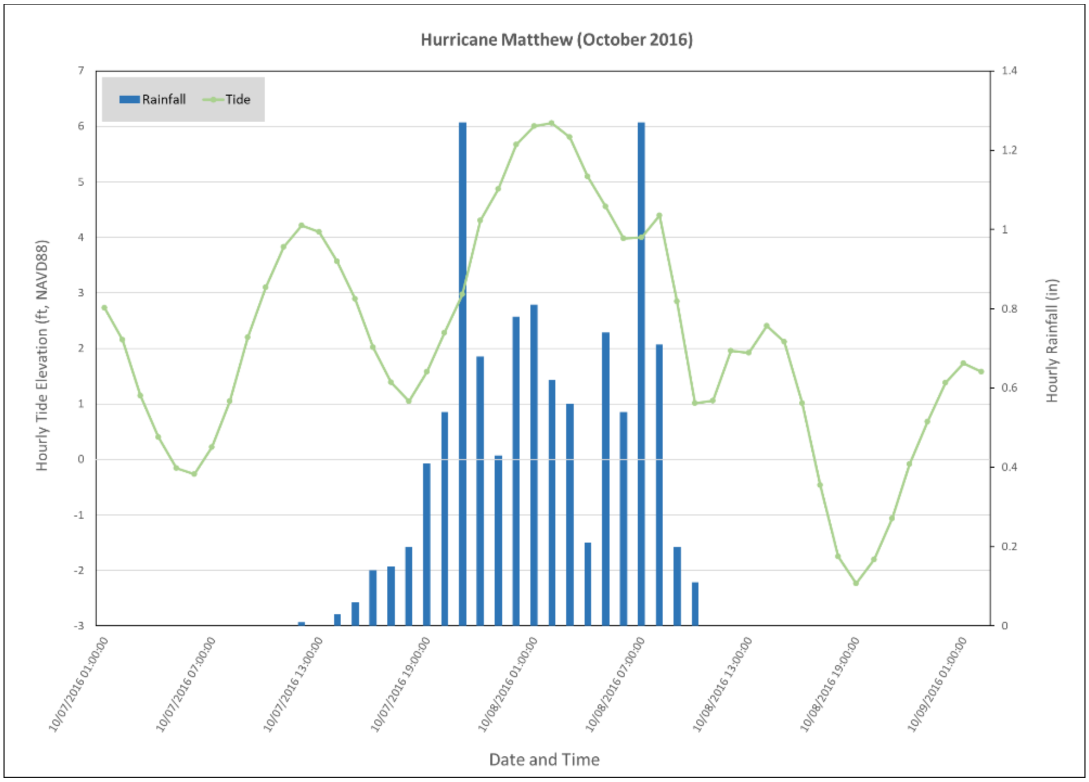

Figure 2.2.6 Hurricane Matthew – Rainfall/Tide Coincidence Plot

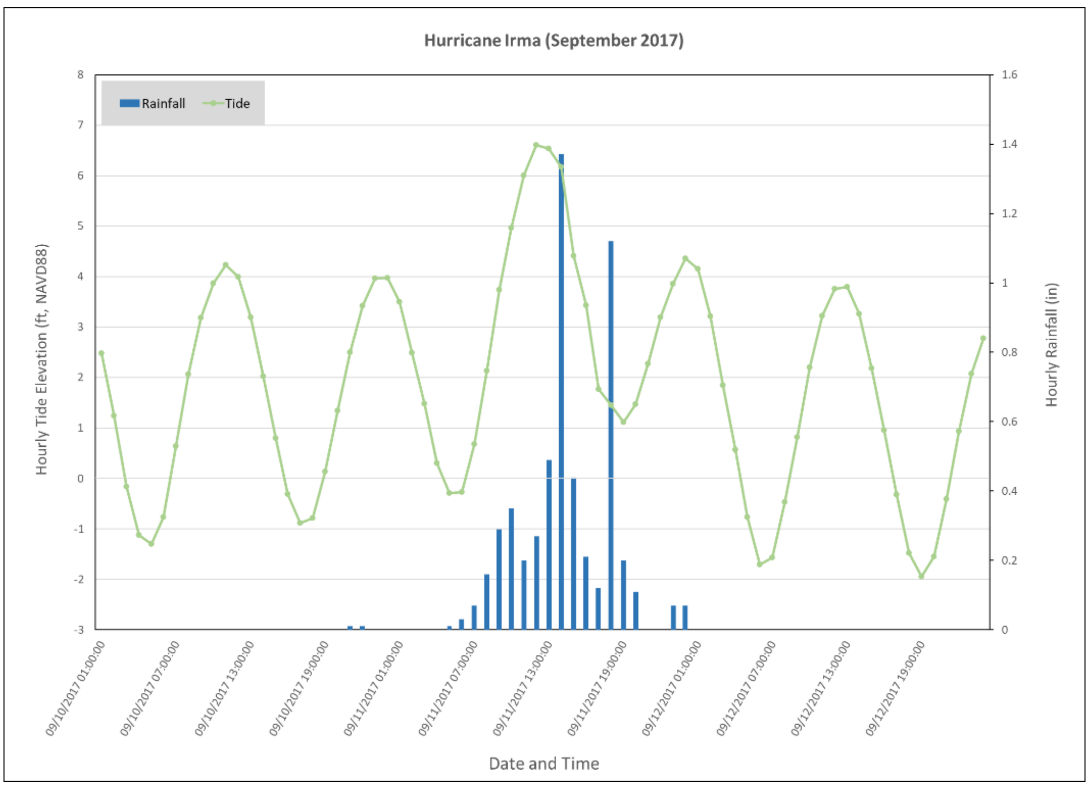

Figure 2.2.7 Hurricane Irma – Rainfall/Tide Coincidence Plot

2.2.5 RAINFALL-TIDE SUMMARY

The review of the data displays some dependence for both rain-tide correlation and coincident timing. The interior drainage model for sizing pump stations assumes the storm gates are closed throughout the model simulation. This means pump stations are sized considering all interior rainfall throughout the modeled storm would be mitigated via pumping. In reality, it is unknown whether the rain and tide peaks occur at the same time. The storm gates could potentially be opened once the storm surge recedes and therefore assist in the drainage of interior storm water runoff. For this study, the future without-project conditions assume a constant Mean Higher High Water (MHHW) tide elevation with Sea Level Change (SLC) while applying the various rainfall frequencies onto the HEC-RAS model. The future with-project condition results for the same hydro-meteorological are compared to the future without-project results for assessing the specific nature of flooding between the conditions.

2.3 CITY STORMWMATER MANAGEMENT SYSTEMS

Most rainfall on the peninsula is collected in a subsurface pipe network system and routed to numerous outfall locations (pipes/culverts) and/or existing pump stations. The City of Charleston has stated the existing stormwater drainage systems (storm pipes) have design capacities of approximately 10% Annual Exceedance Probability (AEP) and some areas contain lesser capacities. Such capacities lead to frequent flooding during intense rainfall events. Such flooding is exacerbated when exterior stages are elevated limiting the conveyance of stormwater runoff. Another issue faced by the City of Charleston is sunny day or "nuisance" flooding which occurs during high tide events that cause backflow into the interior system and flood the streets. The City of Charleston has already begun the implementation of a check valve program in response. Check valves or "flap gates" are attached to peninsula outfall drains to prevent tidal backflow into the stormwater pipe network. Further information is provided in Section 2.3.1
The City of Charleston currently does not have a complete storm water management model covering the entire study area. The coverage the city does have is spread across different models based on the drainage areas and service areas for the various individual projects listed below. The known storm drainage projects in the city include:

- Calhoun Street East Drainage to the Concord Street pump station which is complete.

- Market Street Drainage improvement project. Phase 1 and 2 is completed which connected the drainage to the Concord Pump station. Construction of Phase 3, will be the improvement of the surface drainage collection system to the previously installed new tunnel, expected in 2021. Phase 4 is in construction and phase 5 is pending all be completed for future without condition.

- Medical University of South Carolina (MUSC) pump station.

- Spring Fishburne Drainage Improvement which will improve drainage in an area that covers about 20% of the peninsula, areas - phase 2 completed, phase 3 (tunneling) is underway, completion 2020, Phase 4 (wet-well and outfall) expected to be complete by 2022, Phase 5 (pump station) expect completion by 2023.

- Wagener Terrace Storm Drainage - repair existing system – completed.

- Calhoun West – preliminary report is report is complete from a technical standpoint at this time, unknown if it will be completed by federal project.

- Huger King Street - Phase 1 design is complete with Department of Transportation (DOT) currently reviewing encroachment permits and construction expected in 2020. Phase 2 Outfall improvement and pump station is currently at 30% design with construction expected to be complete in 2022.

- Low Battery Project Phase 1 was completed in 2020. Phases 2-5 will follow in each successive year. Low Battery raising to similar elevation of High Battery.

USACE Engineer Regulation 1165-2-21 states “In urban or urbanizing areas, provision of a basic drainage system to collect and convey the local runoff to a stream is a non-Federal responsibility. This regulation should not be interpreted to extend the flood damage reduction program into a system of pipes traditionally recognized as a storm drainage system.” While the storm drainage is not a USACE CSRM responsibility, any impacts to the interior drainage induced by the proposed project must be evaluated and mitigated to the extent justified under USACE policy. Creating and evaluating the system is outside the scope of the Feasibility study, however, it is assumed the pump stations proposed in this effort are to utilize existing storm pipes if accessible.

The PDT is to work with the City of Charleston during PED phase to appropriately incorporate the stormwater systems into the USACE recommended plan as result of the CSRM. The city’s contracted engineering firm (Davis & Floyd) have modeled the stormwater facilities previously discussed using the Storm Water Management Model (SWMM) but do not currently have full coverage of the peninsula.

As part of the CSRM, three city-owned and operated pump stations are included in the interior drainage modeling. Those pump stations are the Concord Street Pump Station (active), MUSC Pump Station (active), and the Spring Fishburne Pump Station (construction). Also, included are existing drainage culverts that convey surface-flow stormwater runoff and/or provide routing of daily tidal fluctuations. HEC-RAS does not perform sub-surface modeling; therefore, the storm pipe network is not captured.

The implementation of existing features into the model are further discussed throughout this report. Maps showing the locations of existing drainage infrastructure are provided in Chapter 3.2 of this report. The City of Charleston has worked with and learned from the water community of the Dutch and documented plans and strategies as part of the Charleston and Dutch Dialogues. Some of these strategies play a role in the City’s Flooding and Sea Level Rise Strategy and the City’s newly revised Stormwater Design Standards Manual. More information about these strategies and design manuals in the following sections (2.3.1 and 2.3.2).

2.3.1 SPONSOR - FLOODING AND SEA LEVEL RISE STRATEGY

As indicated in the City of Charleston’s “Flooding and Sea Level Rise Strategy” published in February 2019, one of the objectives is to address flooding while promoting a more resilient and sustainable future in the face of recurrent flooding, rising seas, and more frequent extreme weather. The City of Charleston indicates the intent to use the latest NOAA 2017 sea level rise projections for future considerations. One way to track sea level rise is to document “minor coastal flooding” commonly called nuisance, sunny day flooding. The city indicates a marked increase in the number of days of minor coastal flooding over time. The NOAA Sea level change curves and other appurtenant information is provided throughout Chapter 3 of the Coastal Sub-Appendix B-4 report.

The City of Charleston’s Sea Level Rise Strategy (2015) originally recommended a 1.5-to-2.5-foot elevation increase for new facilities and infrastructure. The city increased the recommendation to 2 to feet for the revised 2019 sea level rise strategy. The projection of a 2-to-3-foot rise in 50 years is higher than the NOAA intermediate rate of rise (+1.65 feet) being utilized for the USACE peninsula study in the year 2082 (50-year project life from end of construction). The City of Charleston uses the projection of 2-foot increase for less vulnerable infrastructure such as parking lots, while a 3-foot increase is for more critical long-term infrastructure, such as medical facilities.

In 2019, the City of Charleston began reconstructing and raising the Low Battery Seawall to account for sea level rise projections. The Low Battery Seawall is to be raised to be a similar elevation as the High Battery Seawall. The USACE study assumes this project for the future without-project conditions while the future with-project condition would need the wall raised again to 12 feet NAVD88. The city has also begun the Check Valve Program which is a plan to equip the peninsula outfalls with check valves. A check valve or “flap gate” prevents seawater from backing up into drainage infrastructure to mitigate tidal flooding, while still allowing the outfall to drain stormwater as usual when the tide recedes. Many outfalls in the city are gravity fed and drain to bodies of water that are tidally influenced. During high tides, seawater often enters storm water outfalls and water can back up far enough in low lying areas to result in backflow flooding on streets, even on a sunny day. The city has begun the installation and replacement of check valves on the outfalls. Some of the outfalls currently have a duck bill type check valve and are to be phased out in favor of in-line valves which function better and have less maintenance costs. USACE assumes all peninsula outfalls to be equipped with check valves by the USACE with-project end of construction year (2032).

In addition to tidal flooding and sea level rise, rainfall induced flooding is a significant challenge for the city, and flooding is exacerbated when rainfall and high tide/storm surge occur at the same time. While check valves on the outfalls work well to mitigate flooding from high tides entering the storm drains. Check valve or not, rain that falls during a high tide still has little room to drain and/or increased resistance to drain (as opposed to low tide) until the tide recedes. During such cases, the stormwater collects on the surface because the storm drains are full of seawater. In addition, if the check valve is in the closed position holding pressing seawaters, then additional ponding on the streets may occur. The USACE PDT is proposing storm gates for the surface flow culverts that align with the proposed storm surge wall alignment. These culverts convey inland surface runoff and/or allow for daily tidal fluctuations. The storm gates placed on the culverts are to be closed only during predicted storm events that bring tidal flooding. There has been discussion between the City and USACE about the potential of upsizing culverts during construction of the storm surge wall. Further discussion on this matter is to take place during PED phase.

The city has begun and completed many other drainage improvements such as the Market Street improvement which connects to the Concord Street Pump Station. Another project is the Spring Fishburne Pump Station which is currently under construction. Along with drainage improvements are spotlights on drainage maintenance because a stormwater drainage system performs best when properly maintained. This includes procedures such as keeping drainage ditches, conveyance pipes, and storm drains as clean as possible. As of 2019, the City contracted with a group of engineers and subject area experts to form the new Stormwater Program Management Team. The team is to update the City’s Master Drainage and Floodplain Management Plan, which was last revised in 1984. Another focus of the team is implementing GPS, GIS, and sub-surface camera technologies to schedule, inspect, and monitor both the surface flow drainage ditches and sub-surface stormwater drainage tunnels and pipes.

As mentioned, the city has many drainage improvements completed and/or underway. An important feature for both the city and the PDT are pump stations. The PDT has accounted for three City pump stations in the future without-project condition and in the future with-project condition conceptual plans and modeling. The PDT is proposing both permanent and temporary (mobile) pump stations, meaning the permanent pump stations will contain permanent pump houses with larger pumping capacities while temporary pumps are deployed during storms and typically contain smaller pumping capacities. The City’s Flooding and Sea Level Rise Strategy also dictates using strategically placed temporary pumps, with appropriate storm forecasting notice, to remove stormwater and tidal inundation to mitigate the risk of flooding to the inland area. The City's pump stations such as the Spring Fishburne are thoroughly described in the City' sea level rise strategy (2019). The pump stations contain storm pipes which bring stormwater to the stations which is then pumped underground to the surrounding rivers. The PDT plans to use the existing infrastructure for bringing stormwater to the pump stations. The City has stated the storm pipes accommodate no more than a 10% AEP rainfall event and, in some areas, a lesser capacity is provided by the storm pipes. Once the pipes become overwhelmed water begins to collect on the streets. Therefore, surface flow runoff becomes a larger component of drainage during heavy rainfall events and/or events where the storm drains are filled with backflowing seawater. This is an important aspect and assumption for the PDT as the hydraulic model (HEC-RAS 2D) for interior drainage is a surface flow only model and does not have the capability to model sub-surface flow. At this phase of the CSRM study, the city does not have full coverage or a complete model of the storm pipe network. During PED phase, further information about the storm pipe network will need to be incorporated to more appropriately size and place the PDT's recommended plan for pumps. The PDT has strategically placed pump stations using HEC-RAS 2D by assessing the natural drainage paths of surface flow runoff. In addition, the HEC-RAS modeling is supplemented with some of the City's GIS based layers for visual assistance in the placement of pump stations. These GIS layers provide an important information for the conceptual layout of the future with-project pump stations. The referenced layers include the storm pipe network (layout/inlets/outlets) and the peninsula watershed delineation. The watershed delineation refers to surface flow and further information about the storm pipe (sub-surface) delineation or pump servicing boundaries is needed during PED phase to ensure appropriate design.

The USACE PDT is to coordinate with the city engineers during PED phase to ensure the strategies, goals, and collaboration of the project features are adequately aligned.

Source: https://www.charleston-sc.gov/DocumentCenter/View/20521/Flooding-and-Sea-Level-Rise-Strategy-2019-printer-friendly?bidId=

2.3.2 SPONSOR – STORMWATER DESIGN STANDARDS MANAUAL

Overview

The City of Charleston's Department of Stormwater Management has been working to update the Stormwater Design Standards Manual (SWDSM). The prior 2013 SWDSM was updated in 2020 as the 2020 SWDSM. More information can be found at the following link: https://www.charleston-sc.gov/351/Stormwater-Design-Standards-Manual

The 2020 SWDSM Executive Summary states, "The objective of the City of Charleston Stormwater Design Standards Manual is to provide guidance on the design process during all phases of construction and the latest permanent construction stormwater management practices available to minimize the negative impacts of increasing stormwater runoff and its associated pollutants. Building on the previous version, this updated manual will help the City of Charleston take a comprehensive approach to stormwater management that integrates drainage design, stormwater quantity, and water quality considerations. The goal is to provide an effective tool for the City of Charleston and the development community to reduce both stormwater quality and quantity impacts and ensure protection of both upstream and downstream areas as well as receiving waters.

Stormwater management has entered a new era, the City of Charleston recognizes the need for more innovative policies and practices. The requirements for National Pollution Discharge Elimination System municipal and industrial permits, total maximum daily loads, and watershed assessments and the desire to protect human life, property, aquatic habitats, and the quality of life in the City of Charleston have brought home the pressing need to manage both stormwater quality and quantity from developed and developing areas."

Types of Flooding

Section 2.5 Water Quantity of the 2020 SWDSM mentions five types of flooding that occur in the Charleston area: Coastal and Tidal Flooding, Extreme Event Flash Flooding, Fluvial (Riverine) Flooding, Groundwater Flooding, and Surface Flooding. The focus of the USACE peninsula study is coastal storm surge with the interior drainage focusing on mitigating the residual risk of interior flooding from rainfall and/or storm surge overtopping the proposed wall with pump stations and storm gates.
The following sections were taken from the 2020 SWDSM to briefly describe the five types of flooding:

1. Coastal and Tidal Flooding

In areas along the coastline, factors including high tide, storm surge, and tailwater contribute to the risk of coastal flooding. High tide flooding, also referred to as sunny day flooding or nuisance flooding, occurs during higher-than-average high tide conditions in low-lying areas along the coast (NOAA 2018 and NOAA 2019). These higher-than-average high tide conditions are also called spring tides or king tides. High tide flooding may lead to more frequent road closures, overwhelmed storm drains, or deterioration of stormwater infrastructure. In some areas, land subsidence, or the sinking of land over time, has led to an increased frequency of high tide flooding. Another condition impacting coastal flooding is storm surge. Storm surge is the rising of coastal water levels because of strong winds and changing atmospheric pressure during hurricanes and tropical storms. Higher high tides and land subsidence can also lead to tailwater issues for stormwater drainage systems. Tailwater occurs when the water surface elevation of a receiving waterbody is higher than the discharge point of a stormwater system. When at this condition, there is not enough energy for the stormwater to be discharged out of the system, causing the stormwater system to become overloaded

2. Extreme Event (Flash) Flooding

Floods that develop within six hours of their immediate cause are considered to be extreme event floods. Extreme event floods are typically associated with mountainous regions where stormwater flows rush down mountainsides and overwhelm downstream communities. However, extreme event floods can occur in coastal areas under certain conditions, including intense rainfall during king tides; high-intensity rainfalls inland of a coastal community that drain toward the coast, leading to inundation of coastal river systems; and high-intensity rainfalls that occur in areas that are already partially inundated by previous storm events.

3. Fluvial (Riverine) Flooding

Another flooding risk experienced in coastal regions is fluvial flooding. This type of flooding occurs when water levels in stream channels rise and overtop the streambanks, causing water to flow into the floodplain. In natural landscapes, this process is an integral part of a stream ecosystem that reduces stress on the channel during high flows and helps add nutrients to the stream that boost the aquatic habitat. In developed landscapes, riverine flooding can cause damage to buildings, roadways, and other infrastructure that have been built too close to the stream. The frequency of fluvial flooding is often increased in developed, coastal areas due to multiple factors, including impervious area that increases stormwater runoff volume and intensity; persistent, intense rain events; and debris or log jams causing blockages in the stream channel. The National Weather Service classifies fluvial floods as minor, moderate, or major based on the projected water surface elevation and impacts along the river. Minor floods occur in low-lying areas adjacent to streams found in rural areas and secondary roads. Moderate flooding is characterized by water levels high enough to impact homes, businesses, and larger roads. This level of flood event may require evacuations for residents in the impacted City of Charleston Stormwater Design Standards Manual Conceptual Overview January 2020 2-15 areas. Major floods cause extensive flooding that may flood major traffic routes and isolate some neighborhoods. These events require evacuations of numerous homes to protect citizens from injury.

4. Groundwater Flooding

In pervious landscapes, the higher intensity and duration of storm events along the coast can cause groundwater recharge to occur faster than groundwater discharge. This leads to the water table rising and saturating subsurface soil layers, resulting in groundwater flooding. During this condition, previously permeable soil layers are no longer able to allow stormwater to infiltrate, causing ponding along the soil surface.

5. Surface Flooding

A common misconception is that flooding can only occur near bodies of water. Surface flooding occurs when the excessive stormwater flows from intense or extended rain events cause the ground to become saturated. This type of flooding is observed as standing water in City of Charleston Stormwater Design Standards Manual Conceptual Overview January 2020 2-16 grassed and impervious areas resulting from stormwater flows that saturate the soil and overwhelm the stormwater drainage system. Surface flooding does not typically have a significant flood depth but can cause property damage when combined with other sources of flooding.

Rainfall and Design storms

The 2020 SWDSM Section 3.4.2 Rainfall and Design Storms, discusses the 24-hour duration precipitation depths corresponding to various probabilities for exceedance in any given year are to be used for projects within the city. The USACE study is using the 24-hour duration precipitation depths for model simulations. The 2020 SWDSM provides a table of 24-hour design storm precipitation data for Charleston, South Carolina and each precipitation depth is applied a 10 percent safety factor to account for uncertainties in the design process and the increasing intensities of storms. The USACE study does not apply this 10 percent safety factor to the precipitation data however a suite of rainfall frequencies from a 50% AEP to 1% AEP are simulated to provide a full range of model outputs for assessing the performance of the proposed features. In addition, some model sensitivity has been completed and further model sensitivity is to be completed in PED phase for varying rainfall depths, durations, and intensities.

Summary

The USACE study focuses on coastal storm surge with the interior drainage focusing on mitigating the residual risk of interior flooding from rainfall and storm surge overtopping. The USACE study uses various 24-hour precipitation depths of varying frequencies for model simulations. These simulations are completed for future without-project and future with-project for various plan alternatives for assessment of project performance. The goal of the interior drainage study is to ensure the project does not induce flooding from the view of hydraulics and economics.

While there are different types of flooding occurrences as previously mentioned, coastal/tidal and rainfall flooding are the larger contributors to the flooding within the study area. Riverine flooding and groundwater flooding are not a focus for the USACE study. The study uses the Charleston tidal gage and coastal modeling for determining exterior stages and exterior flooding is primarily driven by the tidal stages not riverine events. Additionally, groundwater tables rising due to sea level rise would likely be the same for future without- and future with-project. The proposed storm surge wall will contain a sheet pile cutoff wall incorporated into the foundation. The purpose of the cutoff wall is to assure that water from hurricane storm surge (relatively short events, hours) does not migrate under the wall and undermine the foundation. The cutoff wall may also prohibit groundwater on the interior side from flowing outward. The city has existing and proposed pump stations which can be used to remove excess water, regardless of the source of flooding. USACE is also proposing pump stations to mitigate any wall-induced interior flooding which are to be used to remove excess water. While pump stations can be used to remove excess water, regardless of the source, the efficiency of pumps may vary with exterior stages meaning seawater directly flowing into a pump wet well would essentially mean a pump is pumping seawater against itself until the surge recedes. The proposed storm surge wall with storm gates and gated outfalls will greatly reduce the risk of storm surge flooding up to the design elevation of the wall. The USACE pumps are to primarily operate during storm surge events that warrant gate closure with concurrent rainfall events for which pumps become the major component of drainage. There may also be instances where less frequent rainfall events (>10% AEP) occur during non-storm surge conditions (storm gates open) for which the USACE pumps can be used to quickly remove rainfall runoff.

See Chapter 7 Feasibility Phase Conclusions for more discussion about PED phase considerations.

CHAPTER 3 – DEVELOPMENT OF HYDRAULIC MODEL

3.1 INTERIOR DRAINAGE METHODOLOGY

Areas protected from exterior flood elevations are subject to interior flooding from stormwater runoff. Thus, interior drainage facilities are required to safely discharge the runoff to limit interior residual flooding. In the case of Downtown Charleston, SC, there are not many options for storing stormwater therefore allowing the stormwater to discharge via gravity flow (low exterior conditions) through the proposed wall and discharging the stormwater via pumping (typically during high exterior elevations) are the focus of the study. The interior areas were studied to determine the specific nature of flooding and to formulate drainage alternatives to implement as part of the alignment.

For the with- and without-project conditions, the exterior stage is an important factor in the drainage of the interior stormwater runoff. The exterior stage is controlled by the tidal cycle and storm surge elevations during storm events. Inland, the interior surface runoff is conveyed out into the Cooper River, Ashley River, and Charleston Harbor via stormwater outfalls (pipes/culverts) and/or existing pump stations. When high tides occur, the disrupt the natural functions of the gravity driven drainage features in which pump stations become a significant feature for drainage.

In the without-project condition, during high exterior stages (tide/storm surge) that rise above the outfall opening, the gravity driven outfalls may incur significant tidal backflow if no check valve is in place and/or cease to drain the interior area if a check valve is in place. Similarly, if a new coastal storm risk management structure is introduced (with-project condition) to reduce the risk of storm surge entering the study area, the existing outfalls, under exterior (tailwater) stage conditions would incur tidal backflow and/or cease to drain as previously mentioned. Therefore, it is important to develop an understanding of whether there is a relationship between interior surface runoff and exterior tidal events in both with- and without-project conditions. The tidal-rainfall correlation assessment is provided in Section 2.2 of this report.

Due to the uncertainties of the coincidental timing of peak rainfall runoff/tidal flooding events, the HEC-RAS model considers constant tidal boundary conditions throughout model simulations with- and without-project conditions. To study the potential interior ponding effect, a variety of scenarios were computed using a synthetic rainfall suite consisting of the 50%, 20%, 10%, 4%, 2%, and 1% Annual Exceedance Probabilities (AEP) of 24-hr durations. Each of these events were simulated for future without-project conditions and future with-project conditions for the years 2032 (end of project construction) and 2082 (50-year project life). The tidal boundary conditions assume the intermediate Sea Level Change (SLC) rates projected NOAA. The rates are +0.56 feet and +1.65 feet for the years 2032 (end of construction) and 2082 (50-yr project life), respectively. These tidal boundary conditions were assumed to be a constant 3.18 feet (2032) and 4.27 feet (2082) which are estimated from the current Mean Higher-High Water (MHHW) level of 2.62 feet NAVD88.

The stated conditions provide mean water levels to appropriately assess and size the storm surge gates and pump stations for comparison to future without-project conditions during non-storm surge conditions which allow for gravity flow although less efficient gravity flow considering high tide as compared to low tide gravity flow. In supplement to analyzing rainfall at projected MHHW levels, a variety of scenarios were also simulated to analyze the performance of the interior system (pump stations) during storm surge events that produce wave wash overtopping while also considering the rainfall, therefore events with rainfall plus wave wash overtopping (Appendix 2.1 of this report).

Because HEC-RAS 2D does not model sub-surface flow the current modeling performed for the Charleston Peninsula CSRM is performing a surface-flow only study. As previously mentioned, the City's existing storm pipe network becomes overwhelmed for events greater than the 10% AEP. At this point, an assumption is made that the surface-flow stormwater runoff becomes a much larger component of drainage for events at 10% AEP or greater. A surface-flow only model may be conservative in producing higher than expected water surface elevations, however, for both with- and without-project conditions providing the relative comparison between the with- and without-project. A surface flow only model may also under predict the stormwater flow rates to pump stations. Further due diligence is needed during PED phase to incorporate the City's storm pipe network system.

If the project is under-designed, water may pond on the interior, flooding homes and businesses. Alternatively, if the system is over-designed, the cost of the project will be inflated. Utilizing both HEC-RAS and HEC-FDA, the study approach is to assess the system from the hydraulics perspective and economics perspective. The RAS results may show differences in interior water surface elevations for with- versus without-project conditions but the FDA model provides the tools for describing the economic consequences and/or benefits of the differences in interior water surface elevations for with- versus without-project.

The RAS results for future without- and with-project conditions were incorporated into the FDA model to compute the EAD and AAD for describing the residual risk for the interior area. More information is provided in Chapter 5.

3.2 EXISTING HYDRAULIC MODEL

The City of Charleston hired a contractor (Davis & Floyd Engineering) to perform HEC-RAS 2D modeling for the conceptual design of the Calhoun West Pumping Station. The contractors used one geometry file with a mesh size of 50-ft. x 50-ft. and breaklines to detail the road network and other raised features. The geometry used in the Calhoun West effort did not contain any features such as culverts or pump stations. The model was used for applying the rainfall runoff to observe the natural drainage paths of the runoff over the peninsula study area. For design of the stormwater pump stations, the surface-flow model is used for routing rainfall to stormwater inlets. The flows at the inlets are typically used for incorporation into a Storm Water Management Model (SWMM) for the design of pump stations in urban areas that consider sub-surface flow.

The projection used in the modeling is the NAD_1983_StatePlan_South_Carolina_FIPS_3900_Feet_Intl with a terrain file based on the 2009 Charleston County LIDAR. The 2011 NLCD data was used for characterizing the land cover of the peninsula.

Figure 3.2.1 Calhoun West HEC-RAS Model Extent

3.3 REFINEMENT OF HYDRAULIC MODEL

3.3.1 OVERVIEW

The model utilized for the City's Calhoun West pump station modeling was refined to perform the assessment for the Charleston Peninsula CSRM study. Revisions of the HEC-RAS 2D model from the originally provided model include separating the 2D mesh into an exterior and interior 2D and the incorporation of drainage features such as existing culverts which drain surface flow. Other features such as pump stations and the storm surge wall are incorporated and discussed in sections throughout this report. The breaklines for the road network are enforced into the 2D area. Breaklines were also applied to other appropriate locations to represent raised features in the model domain.

The HEC-RAS 2D model contains two 2D flow areas to represent the interior and exterior areas for each geometry condition including: existing, future without-project, and future with-project. The 2D flow areas are connected by SA/2D connections using the proposed storm surge wall alignment as the separator of the interior and exterior areas for all geometry conditions. Modeling each geometry condition in this manner allows for the cell mesh structure to be similar across each geometry. The 2D connections representing natural ground in the existing and future without-project geometry conditions allow the connection to read its weir station/elevation data directly from the underlying terrain (cut from terrain) while the 2D connections representing the proposed storm surge wall in the future with-project geometry uses elevation data of 12 feet NAVD88. The 2D connections at higher ground such as roads used weir coefficients of approximately 1 while low lying areas (tidal creeks) may use 0.5. The connection representing the storm surge wall use the default weir coefficient of 2.

As mentioned previously, the HEC-RAS 2D model does not have the capability to model sub surface flow. Therefore, all flow in the model is assumed to be surface flow and does not contain interior drainage features such as the storm pipes and/or sub-surface peninsula outfalls. A surface flow only model may over-estimate inland flood elevations in the absence of sub-surface storage, however, this is the case for all geometry conditions. Other uncertainties and model challenges are discussed in Section 6.

The projection for the RAS modeling is:

“NAD_1983_StatePlane_South_Carolina_FIPS_3900_Feet_INTL”. The projection file is in the RAS folder as “HEC_RAS_Projection.prj”.

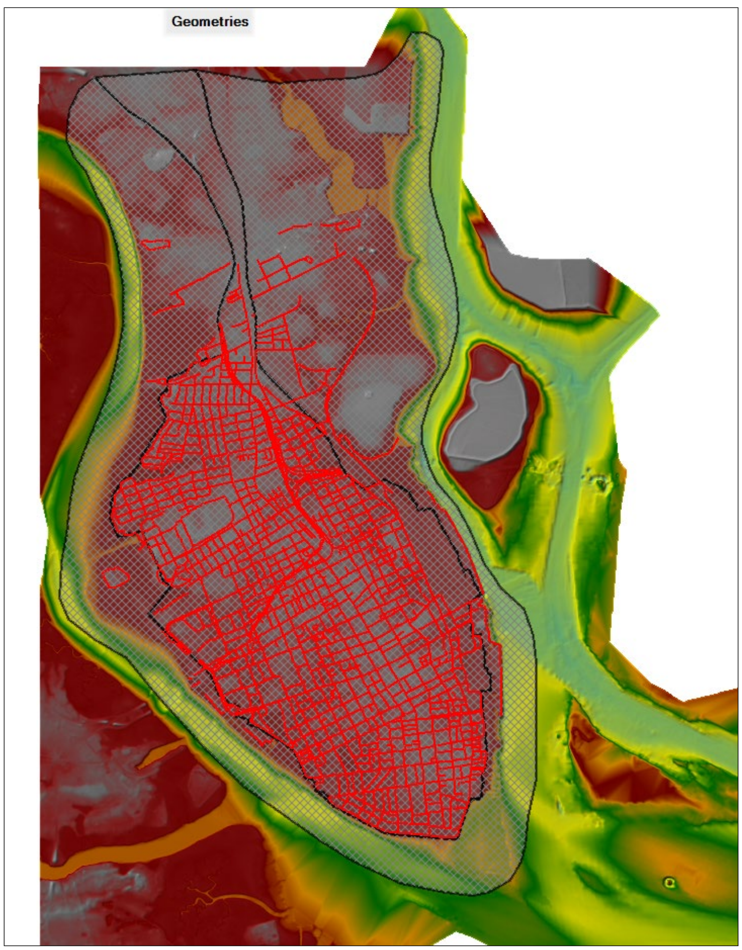

Figure 3.3.1 CSRM HEC-RAS Model Extent

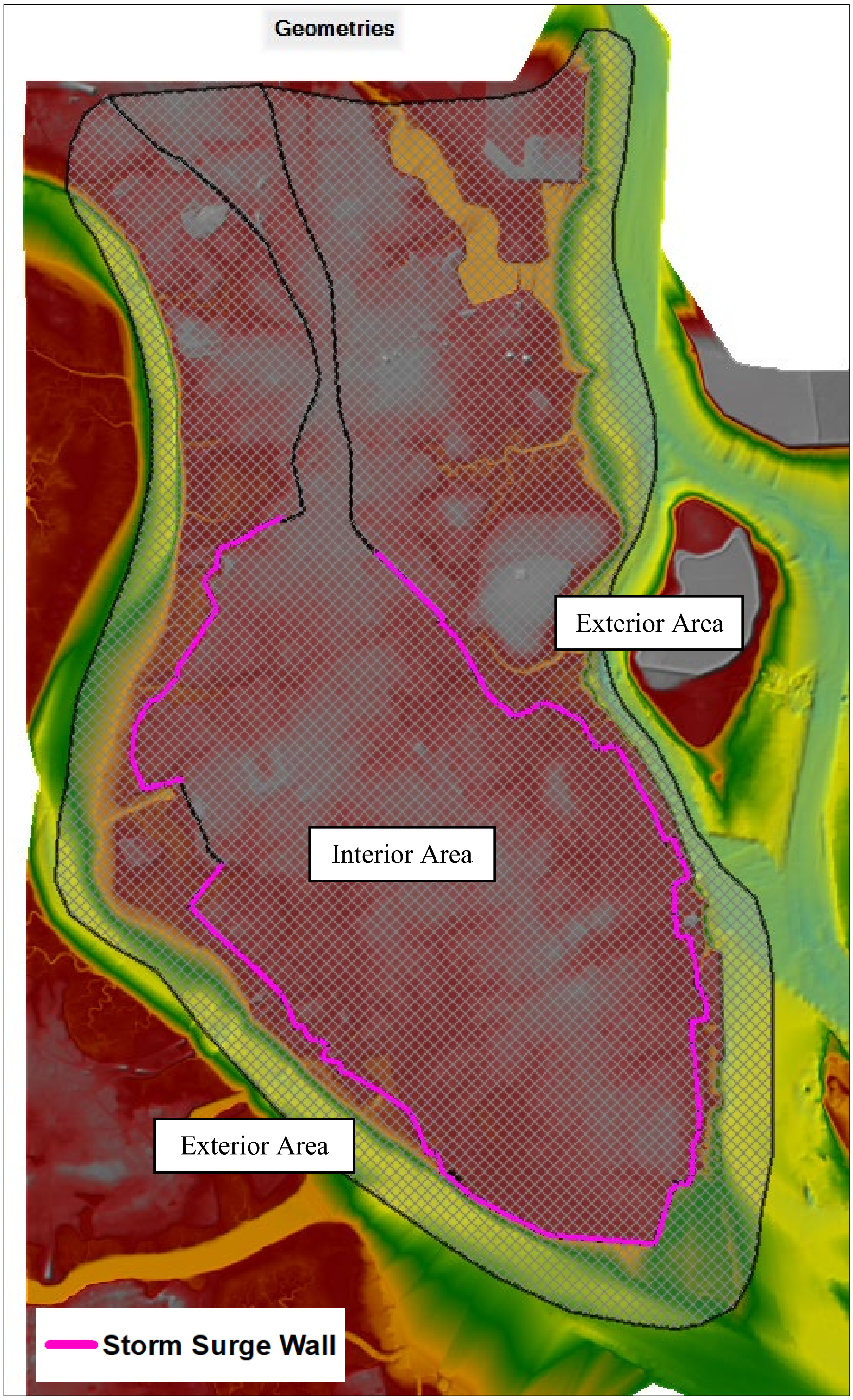

Figure 3.3.2 CSRM HEC-RAS Model with Storm Surge Wall Alignment

3.3.2 DIGITAL ELEVATION MODEL

The Digital Elevation Model (DEM) used in the model is LiDAR provided by the South Carolina Department of Natural Resources (SCDNR). The LiDAR is characterized as a single band raster with a 3ft x 3ft resolution and collected in 2017. The dataset originally wasn't spatially extensive enough to capture the entire study area, so the LiDAR was merged with the 2009 Charleston County raster data and tinned by the GIS team member to extract and "smooth" out the data at the merging boundary. The 2009 Charleston County raster provided terrain values into the Ashley/Cooper Rivers and into the Charleston Harbor which the SCDNR LiDAR did not capture. The SCDNR LiDAR covers the interior area while the Charleston County raster provided the buffer of the exterior area.

The LIDAR was resampled to 5ft x 5ft resolution when merged with the 2009 Charleston County raster. Buildings are not "extruded" in the LiDAR therefore the terrain is bare earth meaning buildings are not captured. The penetration of flood waters into buildings is accounted for using the landcover layer and a buildings shapefile layer. More discussion is provided in Section 3.3.

A sensitivity analysis was performed using a terrain with buildings extruded to compare the results from the current setup where buildings are not captured (extruded) in the terrain but otherwise accounted for using the Manning's n Landcover layer. The results for a 10% AEP rainfall show no meaningful difference in inland water surface elevations for the different terrains.

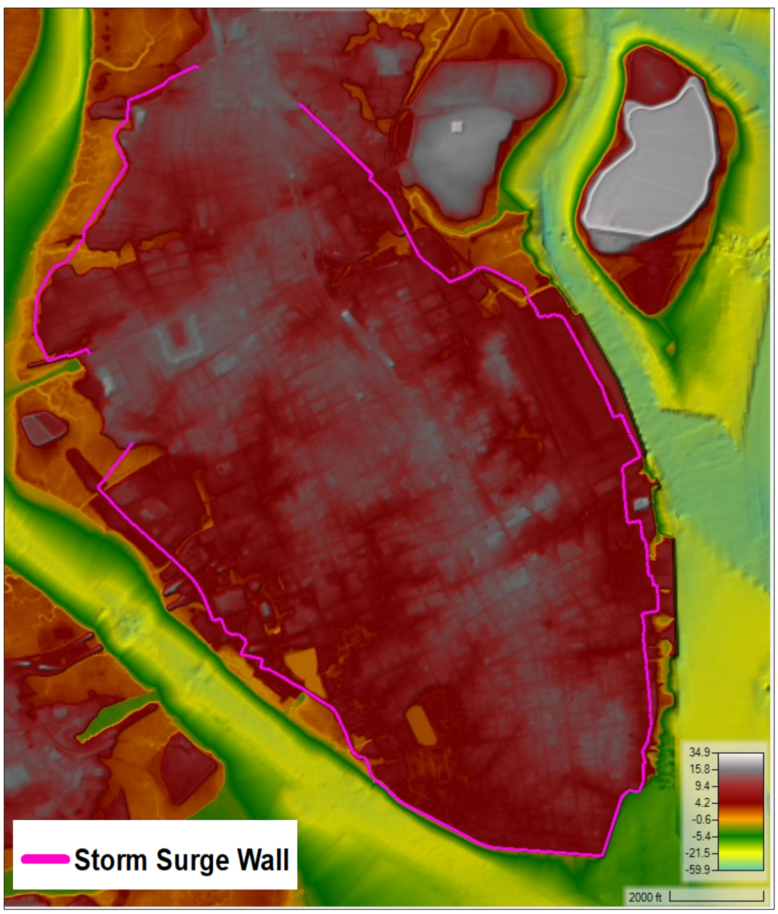

Figure 3.3.3 Digital Elevation Model (feet)

3.3.3 MANNING’S LANDCOVER LAYER

Figure 3.3.4 displays the Manning’s n values applied to the HEC-RAS 2D mesh. The 2016 National Land Cover is being used in this modeling effort. More information on this dataset is provided at http://www.mrlc.gov/. Manning’s n values were assigned to the various land coverage types.

The type of land displaying the Manning’s n value of 99 represents areas that at buildings. A GIS shapefile layer of the buildings on the Charleston Peninsula was provided by the City of Charleston. This layer was merged with the Manning’s n layer and assigned the 99 value to simulate the hydraulic effects of water penetrating a building and stagnating with little to no velocity.

LandCover

Color	Value	Name	Default Manning's n
	0	nodata	
	1	3550900011	99
	11	open water	0.03
	21	developed, open space	0.04
	22	developed, low intensity	0.05
	23	developed, medium intensity	0.06
	24	developed, high intensity	0.07
	31	barren land rock/sand/clay	0.035
	41	deciduous forest	0.15
	42	evergreen forest	0.15
	43	mixed forest	0.15
	52	shrub/scrub	0.1
	71	grassland/herbaceous	0.08
	81	pasture/hay	0.06
	82	cultivated crops	0.05
	90	woody wetlands	0.08
	95	emergent herbaceous wetla...	0.08

Figure 3.3.4. Manning’s n values applied to the HEC-RAS 2D model

3.3.4 BOUNDARY CONDITIONS

A stage tidal boundary condition is applied to the exterior 2D grid as an exterior Boundary Condition line using the stage hydrograph function.

- For the existing conditions simulation, the 2017 Hurricane Irma stage hydrograph was used. The stage hydrograph was extracted from the Charleston, Cooper River Entrance SC Tidal Gage via NOAA’s website.

- For the assessment in the year 2032, the stage hydrograph is set to the Mean Higher High-Water (MHHW) surface elevation of 3.18 ft. NAVD88. Currently the MHHW is 2.62 ft., and the value is increased by adding the intermediate sea level rate of +0.56 ft. for the year 2032.

- For the assessment in the year 2082, the stage hydrograph is set to the MHHW surface elevation of 4.27 ft. NAVD88. This is an increase from the current 2.62 ft. by adding the intermediate sea level rate of +1.65 feet for the year 2082.

- For the assessment of rainfall plus overtopping, a surge event at approximately the 2% AEP in the year 2082 was used as the stage boundary condition. The coastal model overtopping analysis provided the Annual Exceedance Probability (2% in this case) at which point the Still Water Level (SWL) considering Relative Sea Level Change (RSLC) plus one wave amplitude exceeds the flood wall height of 12 ft. NAVD88. More detail regarding the inputs and outputs of the overtopping assessment can be found in Section 9.1 of this report.

- The rainfall used in the assessment is uniformly applied to the 2D flow areas rather than spatially varied. Rainfall information is provided in Section 3.3.4.

3.3.5 RAINFALL APPLICATION

The City of Charleston contractor (Davis & Floyd Engineering) which developed the original RAS model for the Calhoun West pump station project also provided the rainfall data. The contracting team developed the rainfall data into a runoff time-series format. A runoff excel spreadsheet was used to develop the direct runoff based on SCS Type III methodology and an average CN Value of 88.

Rainfall data was provided for the 50%, 10%, 4%, 2%, and 1% AEP rainfall events . Using NOAA's Atlas 14, the 20% AEP cumulative rainfall amount for a 24-hr duration was estimated and input into the provided spreadsheet using the SCS Type 111, SCS Curve Number, and excess precipitation equations. The 24-HR cumulative direct runoff value (Qcn) for the 20% AEP was calculated to be 4.15 inches. More information on the precipitation data can be found within the excel spreadsheet associated with this report.

The rainfall data was applied to the 2D mesh uniformly, however, rainfall could vary spatially. Rainfall information is displayed in Table 3.3.1. The column labeled "24-HR Depth (in)" displays the cumulative rainfall distribution over the 24-hour period. The column labeled "Qcn (in)" displays the cumulative excess precipitation or runoff over the 24-hour period.

Table 3.3.1 24-HR Rainfall Time-Series Data

AEP	24-HR Depth (in)	Q_{cn} (in)
50%	4.5	3.20
20%	5.5	4.15
10%	6.5	5.11
4%	7.9	6.47
2%	9	7.55
1%	10.3	8.83

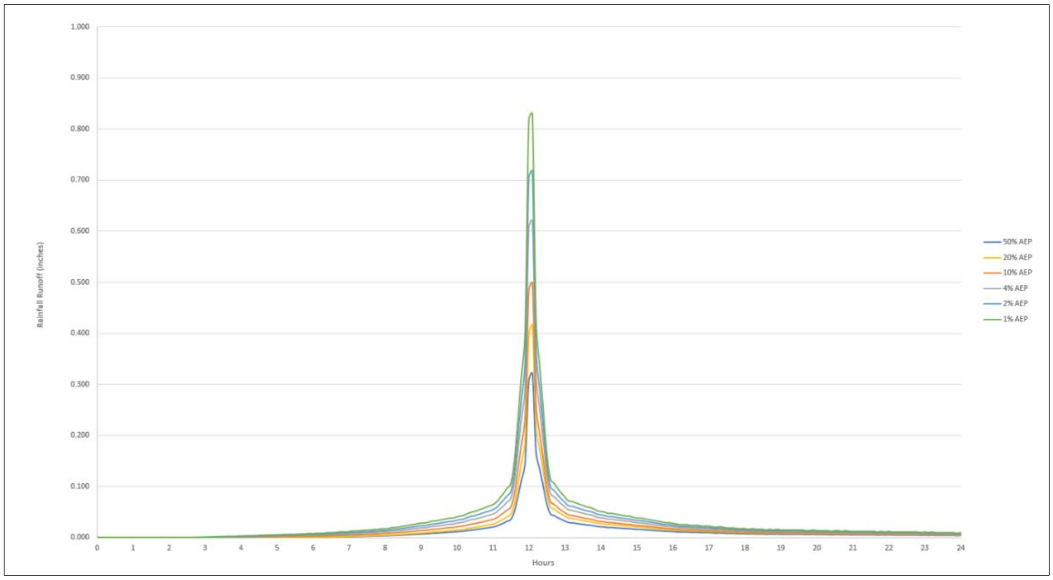

Figure 3.3.5 Direct Rainfall Runoff Time-Series Data (24-HR Durations)

3.4 EXISTING CONDITIONS GEOMETRY

The existing conditions geometry contains interior drainage features such as culverts for surface flow drainage and/or fluctuations of tides. The included culverts are shown in Table 3.4.1. The geometry also contains the Concord Street and MUSC pump stations which are owned and operated by the City of Charleston. The city owned pump stations are listed in Table 3.4.2. The locations of the city owned pump stations are shown in later sections of the report. The existing conditions geometry contains the current alignment and wall heights for the Low Battery and High Battery.

Table 3.4.1 Charleston Existing Culverts

Culvert Location	Culvert Type	Culvert Dimensions
10th Avenue	single box culvert	10ft. span, 2ft. rise
Near Joe Riley	single box culvert	12ft. span, 4ft rise
Gadsden Creek	single box culvert	9ft. span, 3ft. rise
Gadsden Creek Upstream 1	concrete pipe	24inch diameter
Gadsden Creek Upstream 2	concrete pipe	18inch diameter
Longpond	circular metal pipe	48inch diameter
Lockwood Wetland	circular metal pipe	36inch diameter
Newmarket Creek	double box culvert	8ft. span, 3ft rise
Newmarket Creek Upstream	concrete pipe	36inch diameter

Table 3.4.2 City of Charleston Pump Stations

Pump Location	PS (gpm)	PS (cfs)
MUSC (Active)	3 @ 17,000 each	3 @ 38 each
Concord Street (Active)	3 @ 42,000 each	3 @ 94 each
*Spring Fishburne (Construction)	3 @ 135,000 each	3 @ 300 each
**King/Huger (Design)	approximate capacity of 70,000	approximate capacity of 156
*Spring Fishburne not included in existing conditions but is included in future conditions. **King/Huger not included in RAS modeling.		

Figure 3.4.1 Charleston Existing Culverts

3.4.1 EXISTING CONDITIONS SIMULATION

The existing conditions scenario typically serves as a model calibration event, however, there are no inland gages nor are there known high water marks collected by the Charleston District (SAC) or the City of Charleston. The College of Charleston was contacted but was found to have no observed data. The Charleston based engineering firm (Davis & Floyd) also contains no recorded inland water surface elevations, however, indicated containing water level observations for Marina Lake and the Ashley River but were waiting on the final processed data. The HEC-RAS model, without calibration, still serves its inherent purpose of assessing the hydraulic response of future without-project and future with-project conditions for determining the specific nature of flooding for the various conditions.

The existing conditions scenario was computed using verified water levels produced by Hurricane Irma on September 11, 2017. These water levels were extracted from NOAA Tides & Currents webpage from the Charleston, Cooper River Entrance SC gage. The Station ID is listed as 8665530. Figure 3.4.2 displays the Hurricane Irma stage hydrograph (NAVD88). Table 3.4.3 displays the datums for the gage.

Table 3.4.3 Water Surface Elevations (WSEL)

Datum	*Elevations in NAVD88	Description	2032 Elev. (+0.56 feet)	2082 Elev. (+1.65 feet)
Max Tide	9.38	Highest Observed Tide	9.94	11.03
MHHW	2.62	Mean Higher-High Water	3.18	4.27
MHW	2.26	Mean High Water	2.82	3.91
MLW	-2.96	Mean Low Water	-2.4	-1.31
MLLW	-3.14	Mean Lower-Low Water	-2.58	-1.49
Note: There are uncertainties in projecting sea level change. The *current elevations were simply projected by adding the 2032 and 2082 sea level change rates.				

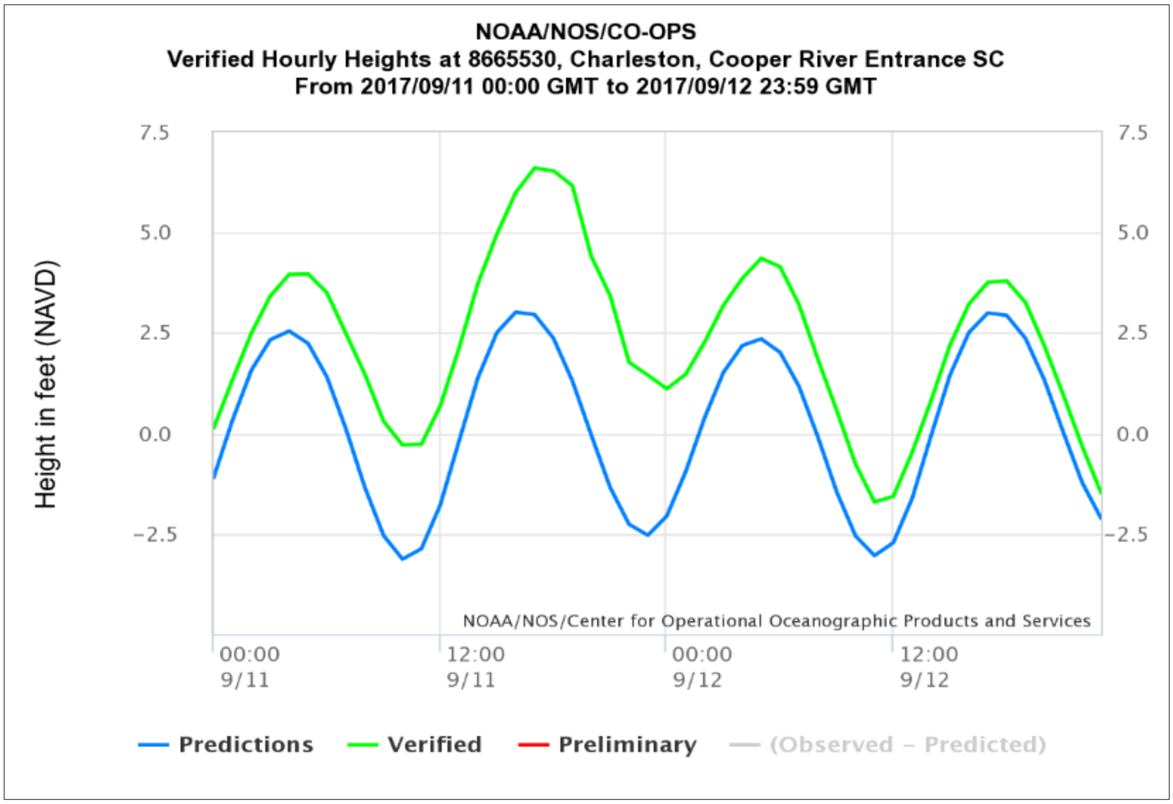

Figure 3.4.2 Charleston, Cooper River Entrance SC Tidal Gage (Hurricane Irma)

The existing conditions scenario in Figure 3.4.3 displays many areas inundated along the east and west side of the peninsula. The low side or west side of the Battery was overtopped by storm surge during this event. The Low Battery near the U.S. Coast Guard property was flanked by the surge before it was overtopped as seen in the modeling. Figure 3.4.3 displays the computed inundation for the 2017 Hurricane Irma event and compares it to the computed hypothetical inundation if it were to occur in 2032. The hypothetical 2032 Hurricane Irma event was computed by scaling up the 2017 stage hydrograph by an intermediate sea level change rate of +0.56 feet. Hurricane Irma peak water surface elevation was approximately 6.7 ft. NAVD88 therefore projecting this to the year 2032 assumes a peak water surface elevation of approximately 7.2 ft. NAVD88.

The purpose of Figure 3.4.3 is to provide visual representation of the potential increase in flooding for future storm events due to sea level change. There are significant uncertainties in estimating the evolution of future storm events, future storm surge, and the impacts of relative sea level change. Rainfall data was not included in this computation to show the inundation results from storm surge.

An additional existing conditions computation was computed for the 2017 Hurricane Irma event using the mentioned stage boundary condition and assuming a 24-hr 50% AEP rainfall. As shown in the rainfall-tide correlation assessment, the 2017 Irma event had approximately 4.5 inches of rainfall the day of the peak crest which corresponds roughly to a 10% AEP. The simulation is set to begin on 11Sept2017 at 0300 and end on 12Sept2017 at 1100. The Irma peak storm surge occurs at approximately 1700 on 11Sept2017. The 50% AEP rainfall is set to peak around 1500 on 11Sept2017.

Results from the 2017 Irma without rainfall simulation were compared to the 2017 Irma with rainfall. The results were compared where the inundations overlap (surge inundated areas) and show little to no difference between the simulations meaning the rainfall contributed little to peak water surface elevations for inland areas within that storm surge floodplain. Another factor to consider is the ability of rainfall runoff to discharge from inland areas at elevations greater than that of the storm surge floodplain during storm surge rise and fall.

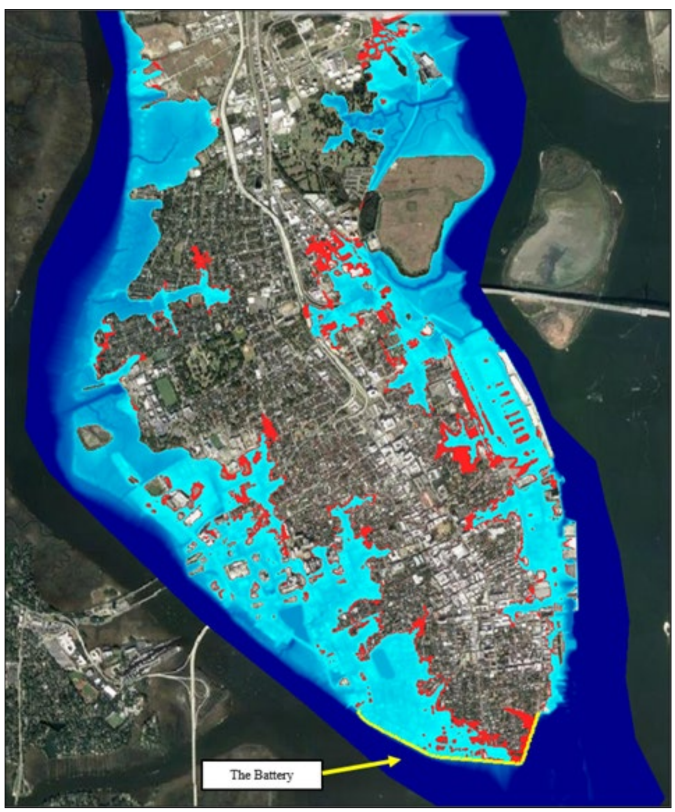

Figure 3.4.3 Hurricane Irma Simulation 2017 (Blue) vs Hurricane Irma Hypothetical Simulation 2032 (Red)

3.5 FUTURE WITHOUT-PROJECT GEOMETRY

The future without-project geometry contains interior drainage features such as culverts for surface flow drainage and/or fluctuations of tides. The geometry also contains the Concord Street (active), MUSC (active), and Spring Fishburne (construction) Pump Stations which are owned and operated by the City of Charleston. The King/Huger pump system is in design phase but is not included in this model.

The future without-project geometry includes the City of Charleston having raised the Low Battery to similar to the elevation of the High Battery.

3.6 FUTURE WITH-PROJECT GEOMETRY

The future with-project geometry contains the proposed storm surge wall with a design elevation of 12 feet NAVD88. The geometry also includes the existing culverts mentioned previously. The existing culverts which align with the perimeter of the proposed wall are assumed to be equipped with storm gates and part of the storm gate assessment. These storm gates would remain open and would only close when a storm surge is predicted. In addition to the existing culverts mentioned, other low lying tidal creek areas may require storm gates in the wall such as Halsey Creek and the creek near the Port.

The future with-project geometry also contains the city owned Concord Street, MUSC, and Spring Fishburne pump stations along with the USACE proposed pump stations. As previously mentioned, the King/Huger pump station is not included in the model.

Information for the USACE proposed pump stations is provided in the following tables and figures. Three alternative pump capacities at each pump station location have been assessed. Some pump stations are to be permanent, and some are to be temporary (mobile). The permanent pump station refers to those that would have permanent pump housing with larger capacities than the temporary pumps. Temporary or "portable" pumps refer to those that would not have permanent housing and would be deployed during storm events with appropriate notice. The pump stations would utilize existing storm pipe networks for routing water to the pumps. More information is provided throughout Chapter 4.

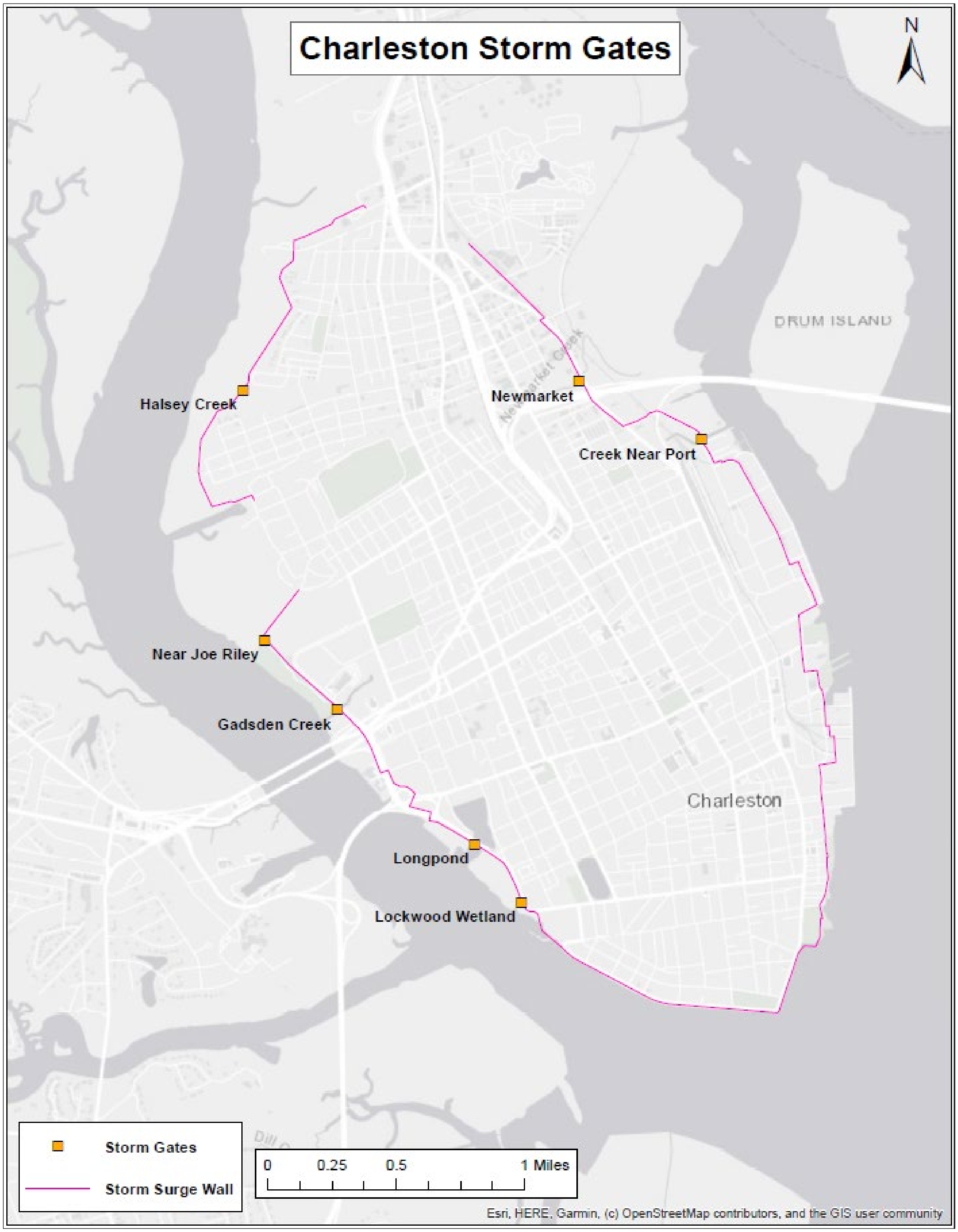

Figure 3.6.1 Charleston Storm Gates

Table 3.6.1 Charleston Storm Gate Dimensions

Storm Gate Location	Storm Gate Dimensions - # - (Width x Height) - feet
Halsey Creek	5 – 8’ x 10’ storm gates
*Near Joe Riley	1 – 4’ x 12’ box culvert
*Gadsden Creek	1 – 3’ x 9’ box culvert
*Longpond	1 – 4’ diameter circular pipe
*Lockwood Wetland	1 – 3’ diameter circular pipe
Creek Near Port	1 – 20’ x 10’ storm gate
*Newmarket Creek	2 – 8’ x 3’ double box culvert
Total	12 gates

*Existing culverts that are owned by the City of Charleston. During this phase of the study, the existing culverts are assumed to remain the same dimensions as they are currently. However, the culverts could potentially be upsized during construction of the proposed storm surge wall. The City of Charleston has also stated the possibility of upsizing some culverts. These details will be re-assessed during PED phase. Note that existing culvert dimensions are sized using the best available data and may not be exact.

Table 3.6.2 PDT Pump Station Alternatives

Pump Station	Pump Station Alt. 1	Pump station Alt. 2	Pump Station Alt. 3
	PS (cfs)	PS (cfs)	PS (cfs)
Halsey Creek (P)	60	90	150
Citadel near Joe Riley (P)	60	90	150
City Marina (P)	30	60	120
The Battery #1 (P)	30	60	120
The Battery #2 (T)	10	20	40
The Battery #3 (T)	10	20	40
Near Waterfront Park (T)	10	20	40
Port 1 (T)	10	20	40
Port 2 (T)	60	90	150
Newmarket Creek (P)	60	90	150
Totals	5 Permanent 5 Temporary	5 Permanent 5 Temporary	5 Permanent 5 Temporary
	340 cfs	560 cfs	1000 cfs

(P) Permanent (permanent housing)
(T) Temporary (mobile/no permanent housing)

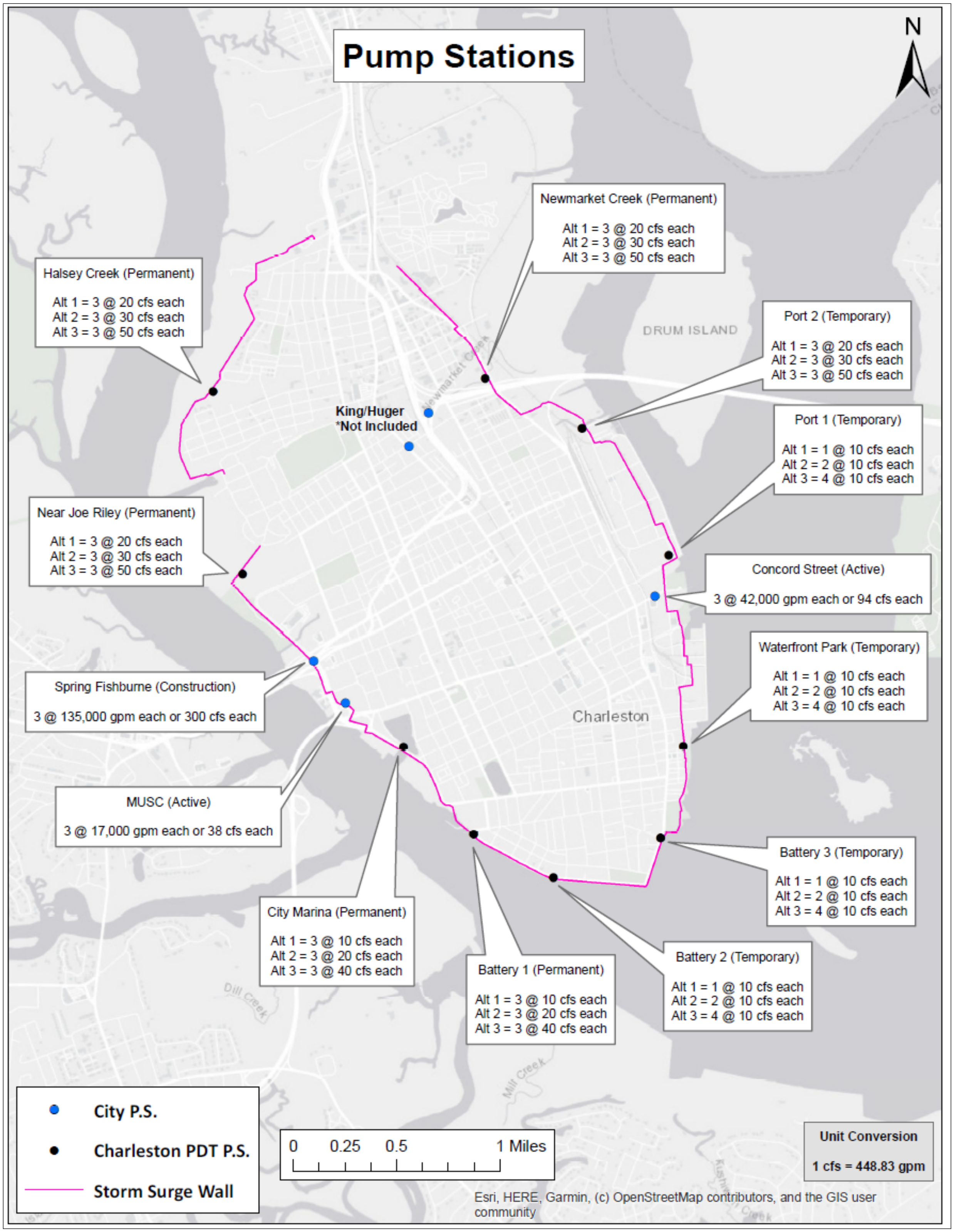

Figure 3.6.2 Pump Station Alternatives

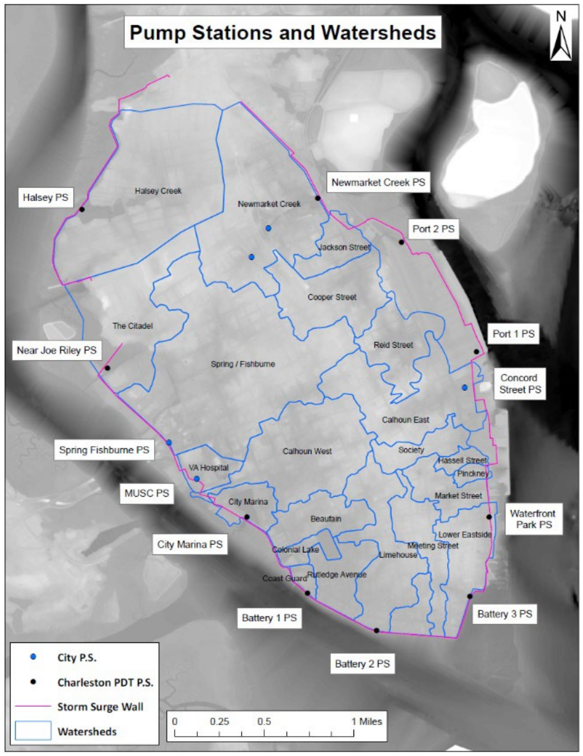

Figure 3.6.3 Pump Stations and Surface Flow Watersheds

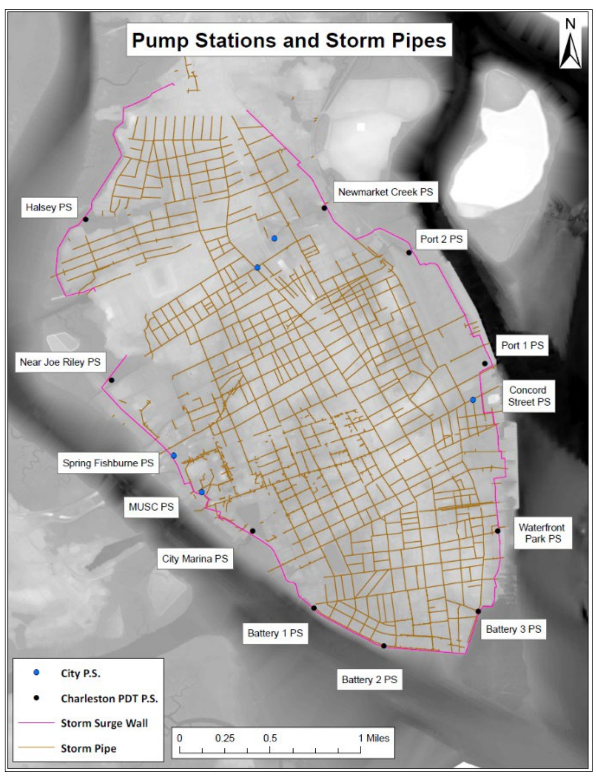

Figure 3.6.4 Pump Stations and Existing Storm Pipes

CHAPTER 4 – INTERIOR DRAINAGE ASSESSMENT OF PROJECT ALTERNATIVES

The interior drainage assessment for the feasibility study assesses the proposed system from two perspectives: open system and closed system.

1. The open system assumes daily tide conditions or non-storm conditions. The storm gates remain open to allow daily tidal fluctuations. The gates open condition also assumes vehicle and pedestrian gates are open. These gates may not be significant for interior drainage however may provide some drainage relief during heavy rainfall events where water flows through the streets and along curbs before entering storm drains. In addition, if rainfall overwhelms the storm pipes or if seawater is in the storm pipes due to high tides street gates may have impacts on drainage. It is the assumption that check valves are to be installed on all outfalls/storm pipes, so seawater does not backflow into the system. For the open system conditions, the city owned pump stations are active while the USACE proposed pump stations are not active.

   The open system assumes inland runoff discharges via gravity flow overland to the exterior or to the city owned pump stations where it is discharged to the exterior. In reality, the runoff could drain through the storm pipes assuming the check valves are in place keeping seawater from backflowing and assuming the exterior elevations are low enough to allow the interior runoff to discharge through the outfalls.

2. The closed system assumes storm conditions meaning a storm surge has been forecasted that warrants storm gate closure. The model assumes the storm gates are closed at low tide prior to the arrival of a storm surge and to remain closed throughout the simulation. Interior rainfall runoff is to be removed via the city owned pump stations and USACE proposed pump stations. Runoff drains overland to the pump stations. In reality, the pipe system can bring stormwater to the pumps until the pipes reached capacity.

   a) The closed system condition assumes pre-storm water level drawdown meaning the interior system would be closed (storm gates) prior to the arrival of a storm surge. The gates are assumed to close at Mean Low Water (MLW) prior to the storm. This could allow the interior system additional storage for runoff accumulation. The closed system would have the option to flush the system prior to the storm if water is present in the system. The MLW levels for the year 2032 and 2082 are -2.4 feet and -1.31 feet NAVD88, respectively. The HEC-RAS model would assume these are the initial interior water levels at the beginning of model simulation.

      Due to the limitations of RAS 2D modeling, the interior inland conditions are bound by the underlying terrain (DEM) elevations. Majority of the inland terrain elevations are at greater elevations than the -2.4 and -1.31 feet MLW, therefore, the RAS model would not recognize the initial conditions and each 2D cell would use the higher underlying terrain elevation. The initial interior conditions for gates closed conditions in the years 2032 and 2082 are the same meaning no initial interior water surface is set and the initial interior condition is provided by the lowest terrain value for each 2D cell.

   b) There may be instances where the available storage in tidal creeks (gates closed conditions) on the interior are under-estimated due to the terrain capturing water surface elevations rather than the bottom of the creek bed.

c) Sensitivity analyses were completed to assess the performance of the pumping operations during varying initial interior elevations. The assessment is provided in Appendix 2 of this report.

Per EM 1110-2-1413, the interior drainage system (storm gates and pump stations) should provide interior flood relief such that during low exterior stages (gravity conditions) the local storm drainage system functions essentially as it did without a levee in place for floods up to that of the storm drainage design. Using the guidance provided in EM 1413, the storm gates and pump stations were assessed for non-storm surge conditions meaning the future without-project geometry was computed using various rainfall frequencies while assuming constant high tide (MHHW). The future with-project geometries, both open system and closed system, were computed using various rainfall frequencies while assuming steady constant tide (MHHW) and the results were compared assess the potential interior "ponding" effect.

During a hurricane event with non-overtopping storm surge, the storm surge wall will greatly reduce water levels in the interior area regardless of pump capacity. The purpose of the pump stations is to remove rainfall accumulated during gates closed conditions. However, there may be instances where pumps are needed during gates open conditions if there are local depression areas that experience flooding due to the project.

To evaluate the performance of each with-project pump alternative, the inland water surface elevations are compared to the without-project without storm surge simulations. In addition to assessing the performance of the proposed system during non-storm surge conditions, the system was also assessed assuming extreme storm surge which causes wave wash overtopping. These simulations assess the performance of pumps for rainfall plus overtopping and compare the results to the performance of the pumps during rainfall only events. More information for storm surge overwash conditions is provided in Section 9.1

As mentioned, the City's existing storm pipe network accommodates no more than a 10% AEP rainfall event meaning the system becomes overwhelmed for events of approximately and greater than this frequency. Surface flow becomes a major component of drainage during such instances. In some areas, the City indicates even lower capacities of stormwater routing. The interior drainage assessment signifies the performance of the system for the events of 50% AEP up to the 4% AEP due to the current capacities of the storm pipe network which brings stormwater to the pump stations.

Table 4.1 provides a list of the geometry conditions and assumptions.

Table 4.1 Geometry Conditions

Geometry Condition	Geometry Assumptions
FWO	Future without-project assumes Low Battery is raised to be similar to High Battery and three City of Charleston P.S. are active. No Subsurface pipes.
FW (gates open)	Future with-project contains 12 ft. NAVD88 storm surge wall, three City P.S. are active, and PDT storm gates are placed and open throughout entire simulation. All gates open. No USACE pumps. No Subsurface pipes.
FW (gates closed)	Future with-project contains 12 ft. NAVD88 storm surge wall, three City P.S. are active, storm gates are closed but no USACE pumps active. All gates closed. No Subsurface pipes.
FW (gates closed) P. S. alt 1	Future with-project contains 12 ft. NAVD88 storm surge wall, three City P.S. are active, USACE P.S. alt 1. All gates closed. No Subsurface pipes.
FW (gates closed) P. S. alt 2	Future with-project contains 12 ft. NAVD88 storm surge wall, three City P.S. are active, USACE P.S. alt 2. All gates closed. No Subsurface pipes.
FW (gates closed) P. S. alt 3	Future with-project contains 12 ft. NAVD88 storm surge wall, three City P.S. are active, USACE P.S. alt 3. All gates closed. No Subsurface pipes.

4.1 HEC-RAS SIMULATIONS

HEC-RAS simulations were computed for open system conditions to assess the storm gates and closed system conditions to assess the pump stations. The simulations were computed using the 50%, 20%, 10%, 4%, 2%, and 1% AEP rainfall events combined with constant tidal boundary conditions.

<u>Note:</u> The following is a description of the approach for the comparison of events. The results of the events listed in the tables labeled FWO are compared to the respective event listed in the tables labeled FW for each plan alternative.

Ex. FWO - exterior elevation (high tide) - interior rainfall (10% AEP) is to be compared to FW (storm gates open) – exterior elevation (high tide) – interior rainfall (10% AEP).

4.1.1 EVENT COMPARISON MATRIX (2032)

Each event utilizes a constant stage boundary condition of the projected 2032 MHHW at 3.18 feet NAVD88. Table 4.1.1 displays the matrices for comparing Future Without (FWO) versus Future With (FW) for each condition and alternative plan.

Table 4.1.1 Event Comparison Matrix - 2032

Comparison matrix 2032	
FWO	
Exterior Elev	Interior rainfall
high tide	50% AEP rain
high tide	20% AEP rain
high tide	10% AEP rain
high tide	4 % AEP rain
high tide	2% AEP rain
high tide	1 % AEP rain

Comparison matrix 2032									
FW (storm gates open)		FW (storm gates closed)		FW (storm gates closed) P.S. alt 1		FW (storm gates closed) P.S. alt 2		FW (storm gates closed) P.S. alt 3	
Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall
high tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain
high tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain
high tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain
high tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain
high tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain
high tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain

4.1.2 EVENT COMPARISON MATRIX (2082)

Each event utilizes a constant stage boundary condition of the projected 2082 MHHW at 4.27 feet NAVD88. Table 4.1.2 displays the matrices for comparing Future Without (FWO) versus Future With (FW) for each condition and alternative plan.

Table 4.1.2 Event Comparison Matrix 2082

Comparison matrix 2082	
FWO	
Exterior Elev	Interior rainfall
high tide	50% AEP rain
high tide	20% AEP rain
high tide	10% AEP rain
high tide	4 % AEP rain
high tide	2% AEP rain
high tide	1 % AEP rain

Comparison matrix 2082									
FW (storm gates open)		FW (storm gates closed)		FW (storm gates closed) P.S. alt 1		FW (storm gates closed) P.S. alt 2		FW (storm gates closed) P.S. alt 3	
Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall
high tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain
high tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain
high tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain
high tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain
high tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain
high tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain

4.2 SELECTED OUTPUT LOCATIONS

Selected output locations were used to assess the impacts to interior water levels for the storm gates and pump station alternatives. Peak water surface elevations at 27 (Figure 4.2.1) selected output locations are tabulated to document the water surface elevation for each with-project condition to compare to the water surface elevations for the without-project conditions. The selected output locations provide a general sense of the performance of the system, however, areas not shown by these locations may experience different impacts.

The RAS results may show differences in interior water surface elevations for with- versus without-project conditions but the HEC-FDA model provides the tools for describing the economic consequences and/or benefits of those differences in interior water surface elevations. Further iterations between HEC-RAS and HEC-FDA are to be completed during PED phase to further assess the system on a site-by-site basis. The site-specific assessment will assist in appropriately designing each storm gate/pump station for the proposed interior drainage system and not to inflate the project cost due to over-design and/or under-design a portion of the system that could induce significant interior residual damages.

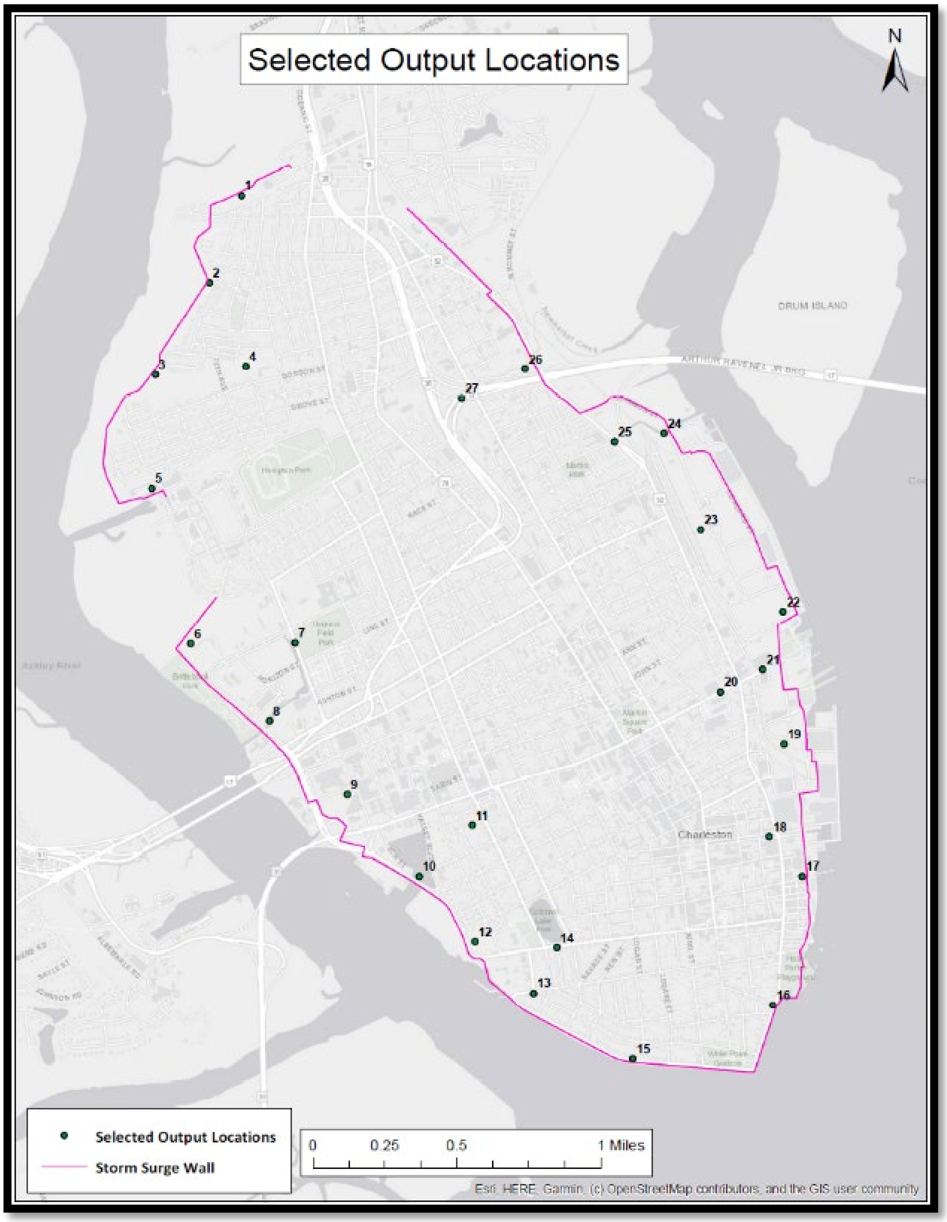

Figure 4.2.1 Selected Output Locations

4.3 FWO RESULTS @ MHHW

This section displays the results for future without-project conditions for the various rainfall frequencies while assuming the MHHW constant tide boundary condition. No changes are assumed in the rainfall data for the years 2032 and 2082. Table 4.3.1 displays the results at the 27 selected output locations for the years 2032 and 2082. Also, reported in the table is the difference in water surface elevation at each location from the year simulations in the year 2032 and 2082.

Table 4.3.1 Future Without-Project Peak Water Surface Elevations @ MHHW

FWO Peak Water Surface Elevations																		
Selected Output Locations	50% AEP Rain			20% AEP Rain			10% AEP Rain			4% AEP Rain			2% AEP Rain			1% AEP Rain		
	Peak Water Surface Elevation (ft. NAVD88)			Peak Water Surface Elevation (ft. NAVD88)			Peak Water Surface Elevation (ft. NAVD88)			Peak Water Surface Elevation (ft. NAVD88)			Peak Water Surface Elevation (ft. NAVD88)			Peak Water Surface Elevation (ft. NAVD88)		
	2032	2082	Diff.	2032	2082	Diff.	2032	2082	Diff.	2032	2082	Diff.	2032	2082	Diff.	2032	2082	Diff.
1	3.58	4.29	0.71	3.64	4.30	0.66	3.68	4.32	0.64	3.75	4.34	0.59	3.80	4.36	0.56	3.85	4.39	0.54
2	3.19	4.38	1.19	3.19	4.38	1.19	3.19	4.38	1.19	3.19	4.38	1.19	3.19	4.39	1.20	3.19	4.39	1.20
3	3.21	4.29	1.08	3.25	4.31	1.06	3.31	4.33	1.02	3.41	4.37	0.96	3.49	4.40	0.91	3.60	4.45	0.85
4	5.92	6.02	0.10	6.19	6.24	0.05	6.40	6.43	0.03	6.60	6.63	0.03	6.75	6.76	0.01	6.90	6.90	0.00
5	6.27	6.21	-0.06	6.35	6.30	-0.05	6.42	6.38	-0.04	6.50	6.46	-0.04	6.55	6.51	-0.04	6.60	6.56	-0.04
6	3.22	4.29	1.07	3.26	4.30	1.04	3.32	4.31	0.99	3.41	4.34	0.93	3.48	4.36	0.88	3.56	4.39	0.83
7	4.55	4.78	0.23	4.91	5.05	0.14	5.26	5.35	0.09	5.58	5.62	0.04	5.75	5.78	0.03	5.91	5.93	0.02
8	3.81	4.50	0.69	4.45	4.76	0.31	4.97	5.12	0.15	5.34	5.41	0.07	5.53	5.57	0.04	5.71	5.73	0.02
9	4.40	4.30	-0.10	4.45	4.47	0.02	4.52	4.53	0.01	4.60	4.62	0.02	4.65	4.71	0.06	4.78	4.83	0.05
10	4.29	4.82	0.53	4.73	4.98	0.25	4.98	5.10	0.12	5.16	5.23	0.07	5.25	5.31	0.06	5.35	5.39	0.04
11	4.70	4.88	0.18	4.90	5.08	0.18	5.10	5.26	0.16	5.37	5.47	0.10	5.53	5.60	0.07	5.69	5.73	0.04
12	3.28	4.30	1.02	3.40	4.33	0.93	3.61	4.35	0.74	4.10	4.41	0.31	4.36	4.47	0.11	4.48	4.54	0.06
13	4.83	4.83	0.00	5.04	5.04	0.00	5.18	5.19	0.01	5.36	5.36	0.00	5.47	5.47	0.00	5.60	5.61	0.01
14	4.83	4.83	0.00	5.04	5.04	0.00	5.19	5.19	0.00	5.36	5.36	0.00	5.47	5.47	0.00	5.60	5.61	0.01
15	5.11	5.11	0.00	5.30	5.30	0.00	5.47	5.47	0.00	5.67	5.67	0.00	5.79	5.79	0.00	5.92	5.92	0.00
16	6.18	6.18	0.00	6.37	6.37	0.00	6.58	6.58	0.00	6.81	6.81	0.00	6.97	6.97	0.00	7.15	7.15	0.00
17	6.18	6.18	0.00	6.25	6.25	0.00	6.34	6.34	0.00	6.48	6.48	0.00	6.55	6.55	0.00	6.62	6.62	0.00
18	6.01	6.01	0.00	6.20	6.20	0.00	6.38	6.38	0.00	6.60	6.60	0.00	6.77	6.77	0.00	6.97	6.97	0.00
19	4.96	4.96	0.00	5.08	5.08	0.00	5.23	5.23	0.00	5.39	5.39	0.00	5.51	5.51	0.00	5.66	5.66	0.00
20	5.55	5.55	0.00	5.72	5.72	0.00	5.88	5.88	0.00	6.06	6.06	0.00	6.18	6.18	0.00	6.32	6.32	0.00
21	4.58	4.57	-0.01	4.71	4.70	-0.01	4.85	4.84	-0.01	4.97	4.97	0.00	5.20	5.20	0.00	5.49	5.49	0.00
22	6.14	6.14	0.00	6.17	6.17	0.00	6.19	6.19	0.00	6.24	6.24	0.00	6.28	6.28	0.00	6.32	6.32	0.00
23	5.45	5.45	0.00	5.56	5.56	0.00	5.67	5.67	0.00	5.89	5.89	0.00	6.03	6.03	0.00	6.17	6.17	0.00
24	3.24	4.32	1.08	3.27	4.33	1.06	3.32	4.35	1.03	3.45	4.41	0.96	3.58	4.48	0.90	3.75	4.56	0.81
25	3.96	4.34	0.38	4.05	4.36	0.31	4.14	4.40	0.26	4.20	4.51	0.31	4.25	4.62	0.37	4.35	4.76	0.41
26	3.79	4.49	0.70	4.10	4.65	0.55	4.48	4.88	0.40	4.90	5.21	0.31	5.35	5.54	0.19	5.88	6.01	0.13
27	4.92	5.10	0.18	5.31	5.46	0.15	5.83	5.85	0.02	6.20	6.21	0.01	6.44	6.46	0.02	6.69	6.71	0.02

1. The table displays the peak water surface elevations at 27 locations around the interior area of the peninsula for future without-project conditions at Mean Higher-High Water (MHHW). The year 2032 uses a constant tide boundary condition of 3.18 feet NAVD88 where the year 2082 uses a constant tide boundary condition of 4.27 feet NAVD88. No change is assumed for rainfall between the years.

2. The table shows the difference in water surface elevation when assuming the year 2032 versus the year 2082. The column labeled "Diff" is the result of column "2082" minus "2032", meaning a positive value denotes an increase in water surface elevation from the year 2032 to 2082..

4.4 STORM GATE EVALUATION

The tables in Section 4.4 display the peak water surface elevations collected from the selected output locations for the storm gates open simulations. The tables include a column for the "Selected Output Locations" and another column to describe the nearest drainage feature influencing each selected output location.

The tables include the with-project water surface elevations and an additional column labeled "Difference from without-project condition (ft.)". This additional column displays if the project alternative is increasing (+) or decreasing (-) the peak water surface elevation at the selected output locations. Increasing the interior water surface would be considered a negative effect of the project alternative and decreasing the interior water surface could be considered a positive effect of the project alternative. In addition, the difference from without-project column is color coated (gold) to signify the locations with a greater than 0.5-foot (6 inch) increase occurring due to the project.

In addition to the output tables, hydrographs are presented for some output locations.

Storm Gate Operations

Storm gates remain open throughout the simulation. The storm gates which are considered existing culverts are to remain the same culvert dimensions that they were for future without-project geometry. This means the RAS model assumes those same culvert characteristics for invert elevations, culvert manning's n, etc. as they were in the future without-project geometry. Assuming this provides an appropriate comparison of the results when using the same culvert characteristics for both geometries.

The storm gates at Halsey Creek and the creek near the Port (which would be new gates and are not existing culverts) use the RAS gate editor rather than the culvert editor. The RAS storm gate weir coefficient applied to these gates is a gated weir coefficient of 1. Additional sensitivity analyses were completed by varying the storm gate coefficient and documented in Appendix 2.3 of this report.

4.4.1 STORM GATE RESULTS (2032)

Table 4.4.1 Future With-Project (Storm Gates Open) @ MHHW 2032

Selected Output Locations	Nearest Drainage Feature Influence (Storm Gates/City PS)	FW 2032 (gates open)											
		50% AEP		20% AEP		10% AEP		4% AEP		2% AEP		1% AEP	
		Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)
1	**NA**	7.31	3.73	7.88	4.24	8.07	4.39	8.30	4.55	8.44	4.64	8.65	4.80
2	**NA**	2.70	-0.49	3.02	-0.17	3.70	0.51	4.54	1.35	5.13	1.94	5.74	2.55
3	**Halsey Creek Gates**	3.31	0.10	3.57	0.32	3.99	0.68	4.65	1.24	5.22	1.73	5.89	2.29
4	**Halsey Creek Gates**	5.92	0.00	6.19	0.00	6.41	0.01	6.62	0.02	6.76	0.01	6.90	0.00
5	**NA**	6.41	0.14	6.55	0.20	6.68	0.26	6.80	0.30	6.89	0.34	6.97	0.37
6	**Joe Riley Creek Gate**	3.47	0.25	3.68	0.42	3.96	0.64	4.33	0.92	4.62	1.14	5.13	1.57
7	**Gadsden Creek**	4.56	0.01	4.93	0.02	5.29	0.03	5.61	0.03	5.78	0.03	5.95	0.04
8	**Gadsden Creek**	3.85	0.04	4.52	0.07	5.03	0.06	5.40	0.06	5.60	0.07	5.79	0.08
9	**MUSC PS**	4.40	0.00	4.45	0.00	4.51	-0.01	4.59	-0.01	4.75	0.10	4.94	0.16
10	**Longpond Gate**	4.30	0.01	4.74	0.01	5.04	0.06	5.31	0.15	5.48	0.23	5.66	0.31
11	**Longpond Gate**	4.71	0.01	4.90	0.00	5.12	0.02	5.40	0.03	5.58	0.05	5.76	0.07
12	**Lockwood Gate**	3.30	0.02	3.47	0.07	4.69	1.08	5.29	1.19	5.52	1.16	5.75	1.27
13	**NA**	4.83	0.00	5.04	0.00	5.18	0.00	5.40	0.04	5.58	0.11	5.79	0.19
14	**Lockwood Gate**	5.10	0.27	5.04	0.00	5.18	-0.01	5.40	0.04	5.58	0.11	5.79	0.19
15	**NA**	5.00	-0.11	5.30	0.00	5.47	0.00	5.67	0.00	5.79	0.00	5.92	0.00
16	**NA**	6.18	0.00	6.37	0.00	6.58	0.00	6.81	0.00	6.97	0.00	7.15	0.00
17	**NA**	6.23	0.05	6.31	0.06	6.41	0.07	6.54	0.06	6.66	0.11	6.82	0.20
18	**NA**	6.18	0.17	6.33	0.13	6.50	0.12	6.73	0.13	6.91	0.14	7.12	0.15
19	**Concord St. PS**	5.05	0.09	5.20	0.12	5.35	0.12	5.53	0.14	5.65	0.14	5.80	0.14
20	**Concord St. PS**	5.57	0.02	5.74	0.02	5.90	0.02	6.08	0.02	6.20	0.02	6.35	0.03
21	**Concord St. PS**	4.55	-0.03	4.68	-0.03	4.84	-0.01	5.01	0.04	5.12	-0.08	5.37	-0.12
22	**NA**	6.15	0.01	6.18	0.01	6.22	0.03	6.28	0.04	6.34	0.06	6.39	0.07
23	**Port Creek Gate**	5.41	-0.04	5.53	-0.03	5.65	-0.02	5.86	-0.03	6.01	-0.02	6.15	-0.02
24	**Port Creek Gate**	3.22	-0.02	3.23	-0.04	3.31	-0.01	3.57	0.12	3.92	0.34	4.35	0.60
25	**Port Creek Gate**	3.97	0.01	4.07	0.02	4.17	0.03	4.22	0.02	4.32	0.07	4.63	0.28
26	**Newmarket Creek Gate**	3.73	-0.06	3.99	-0.11	4.31	-0.17	4.79	-0.11	5.29	-0.06	5.84	-0.04
27	**Newmarket Creek Gate**	4.92	0.00	5.31	0.00	5.83	0.00	6.20	0.00	6.44	0.00	6.69	0.00

1. The column labeled "Difference from without-project condition" shows the difference between the with-project minus the without-project.
2. A positive (+) means the project increases the wsel where a negative (-) means the project decreases the wsel.

Denotes a > 0.5 ft. increase induced by the project.

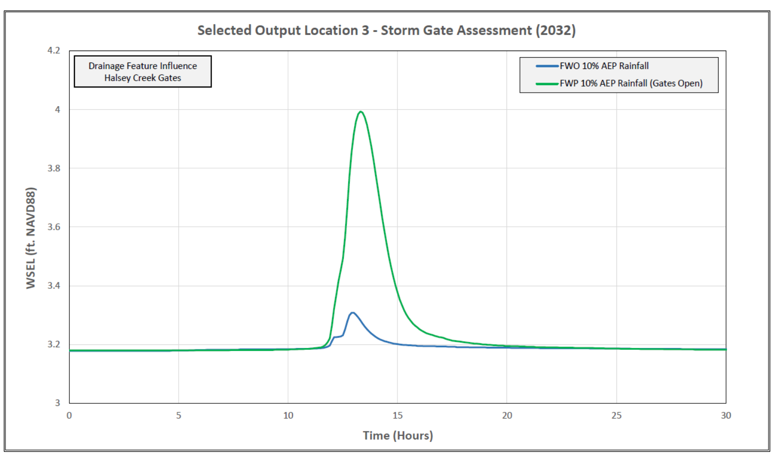

Figure 4.1.1 Storm Gate Assessment – Output Location 3 – 2032

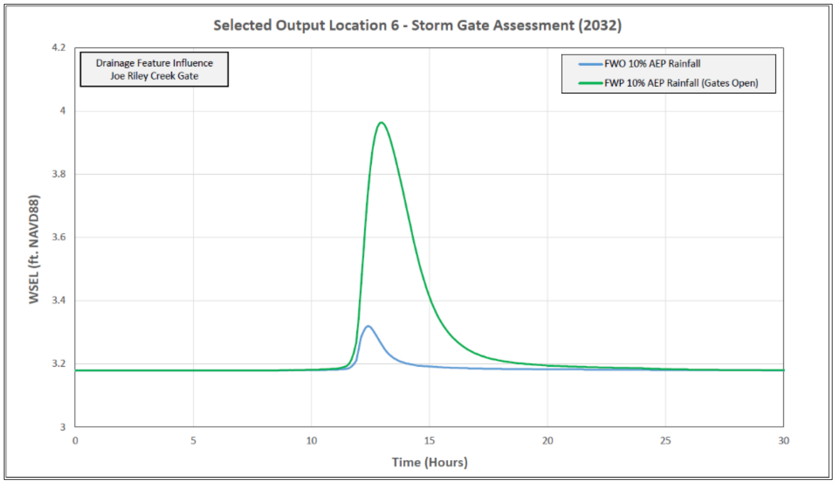

Figure 4.1.2 Storm Gate Assessment – Output Location 6 – 2032

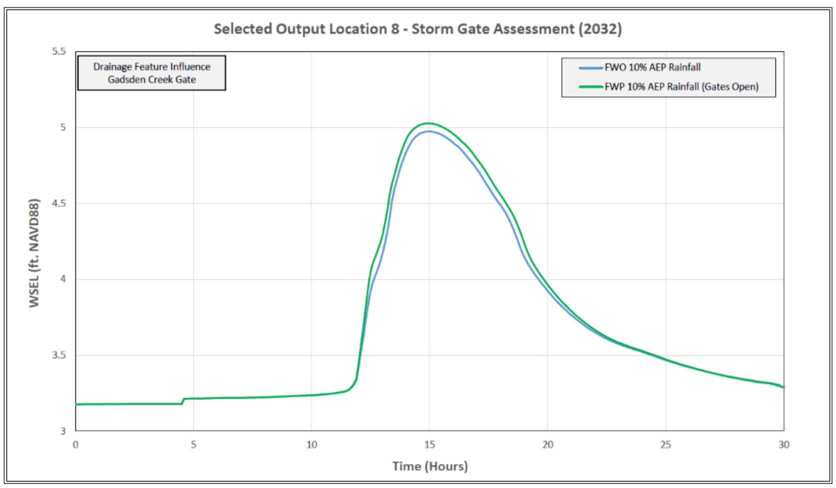

Figure 4.1.3 Storm Gate Assessment – Output Location 8 – 2032

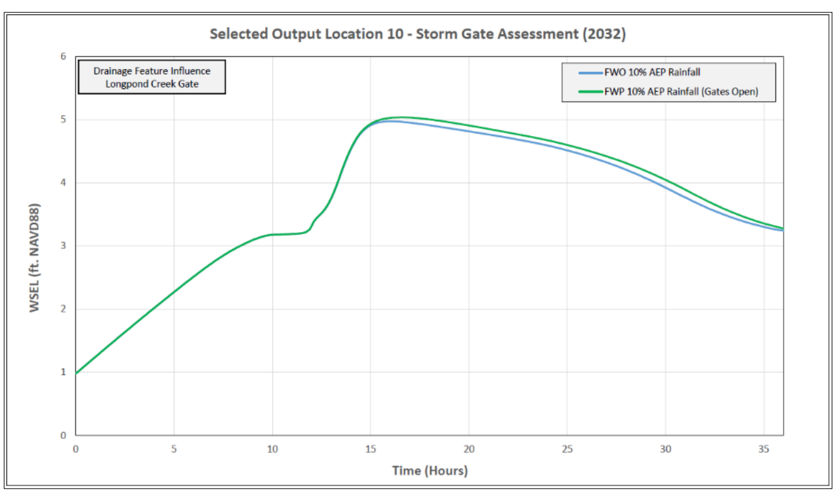

Figure 4.1.4 Storm Gate Assessment – Output Location 10 – 2032

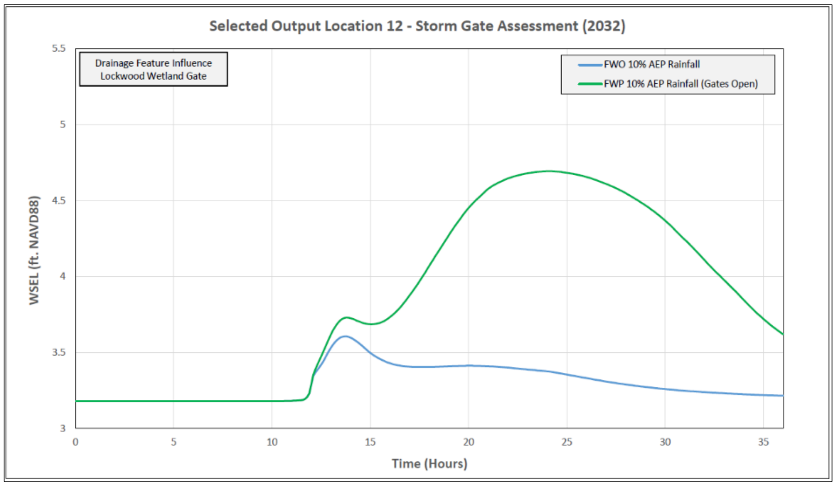

Figure 4.1.5 Storm Gate Assessment – Output Location 12 – 2032

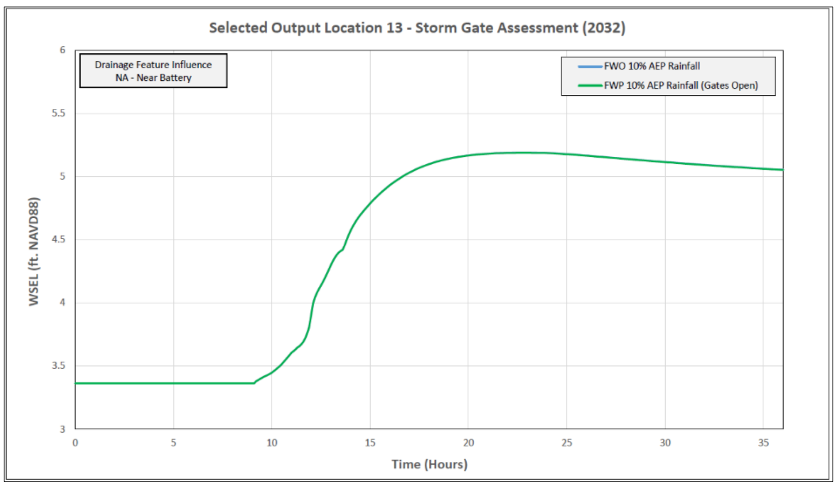

Figure 4.1.6 Storm Gate Assessment – Output Location 13 – 2032

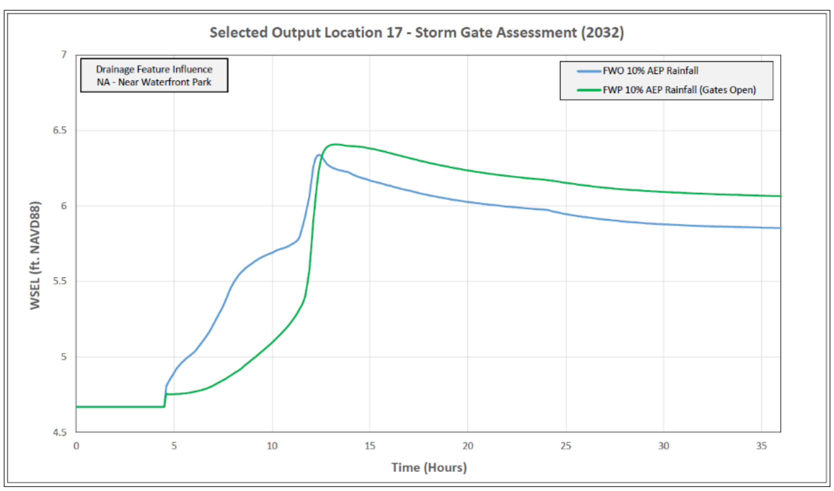

Figure 4.1.7 Storm Gate Assessment – Output Location 17 – 2032

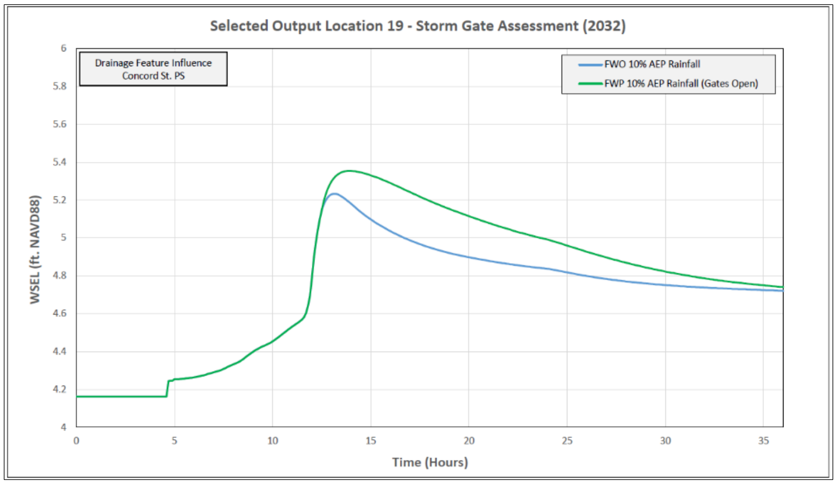

Figure 4.1.8 Storm Gate Assessment – Output Location 19 - 2032

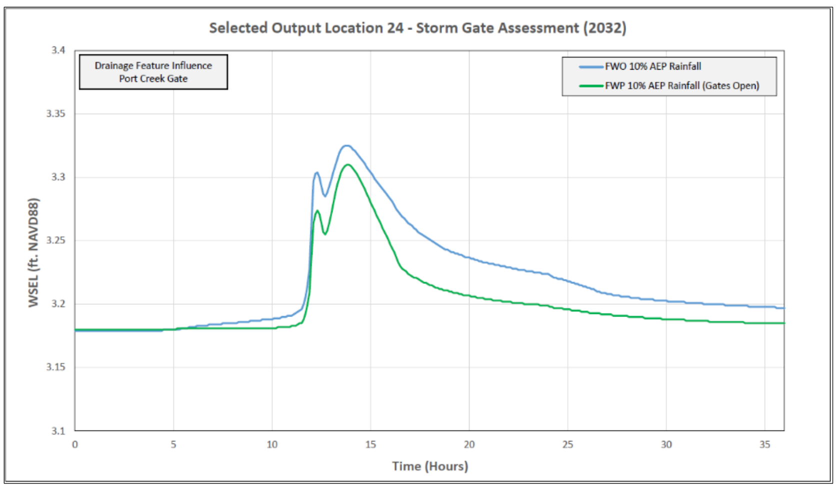

Figure 4.1.9 Storm Gate Assessment – Output Location 24 – 2032

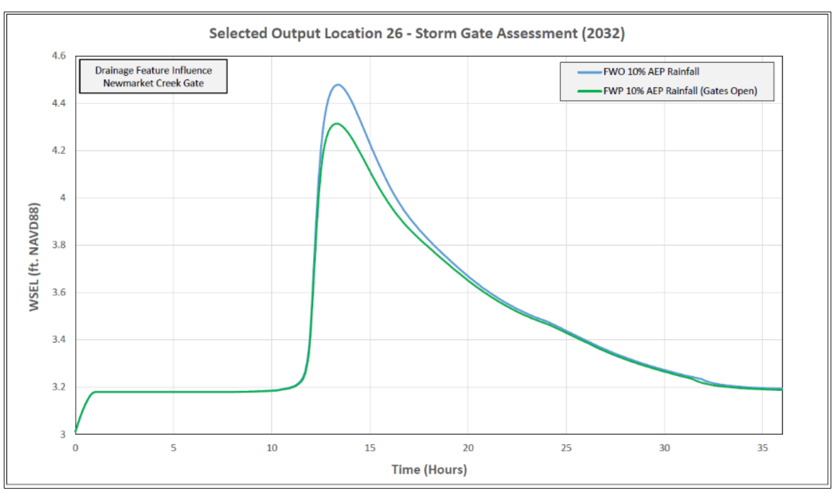

Figure 4.1.10Storm Gate Assessment – Output Location 26 – 2032

4.4.2 STORM GATE RESULTS (2082)

Table 4.4.2 Future With-Project (Storm Gates Open) @ MHHW 2082

Selected Output Locations	Nearest Drainage Feature Influence (Storm Gates/City PS)	FW 2082 (gates open)											
		50% AEP		20% AEP		10% AEP		4% AEP		2% AEP		1% AEP	
		Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)
1	**NA**	7.31	3.02	7.88	3.58	8.07	3.75	8.30	3.96	8.44	4.08	8.65	4.26
2	**NA**	2.63	-1.75	2.95	-1.43	3.64	-0.74	4.50	0.12	5.11	0.72	5.85	1.46
3	**Halsey Creek Gates**	4.35	0.06	4.49	0.18	4.75	0.42	5.21	0.84	5.69	1.29	6.14	1.69
4	**Halsey Creek Gates**	6.02	0.00	6.25	0.01	6.45	0.02	6.65	0.02	6.77	0.01	6.90	0.00
5	**NA**	6.41	0.20	6.55	0.25	6.68	0.30	6.81	0.35	6.88	0.37	6.96	0.40
6	**Joe Riley Creek Gate**	4.41	0.12	4.54	0.24	4.73	0.42	4.98	0.64	5.20	0.84	5.55	1.16
7	**Gadsden Creek**	4.78	0.00	5.07	0.02	5.38	0.03	5.65	0.03	5.81	0.03	5.98	0.05
8	**Gadsden Creek**	4.53	0.03	4.81	0.05	5.17	0.05	5.47	0.06	5.64	0.07	5.83	0.10
9	**MUSC PS**	4.40	0.10	4.45	-0.02	4.51	-0.02	4.69	0.07	4.84	0.13	5.03	0.20
10	**Longpond Gate**	4.84	0.02	5.05	0.07	5.23	0.13	5.45	0.22	5.59	0.28	5.74	0.35
11	**Longpond Gate**	4.89	0.01	5.10	0.02	5.31	0.05	5.53	0.06	5.68	0.08	5.84	0.11
12	**Lockwood Gate**	4.51	0.21	4.76	0.43	5.10	0.75	5.43	1.02	5.63	1.16	5.84	1.30
13	**NA**	4.83	0.00	5.04	0.00	5.23	0.04	5.49	0.13	5.67	0.20	5.87	0.26
14	**Lockwood Gate**	4.83	0.00	5.04	0.00	5.23	0.04	5.49	0.13	5.67	0.20	5.87	0.26
15	**NA**	5.11	0.00	5.30	0.00	5.47	0.00	5.67	0.00	5.79	0.00	5.92	0.00
16	**NA**	6.18	0.00	6.37	0.00	6.58	0.00	6.81	0.00	6.97	0.00	7.15	0.00
17	**NA**	6.21	0.03	6.30	0.05	6.39	0.05	6.53	0.05	6.68	0.13	6.83	0.21
18	**NA**	6.18	0.17	6.33	0.13	6.50	0.12	6.73	0.13	6.91	0.14	7.12	0.15
19	**Concord St. PS**	5.05	0.09	5.20	0.12	5.35	0.12	5.53	0.14	5.66	0.15	5.80	0.14
20	**Concord St. PS**	5.57	0.02	5.74	0.02	5.90	0.02	6.08	0.02	6.21	0.03	6.35	0.03
21	**Concord St. PS**	4.56	-0.01	4.69	-0.01	4.83	-0.01	5.01	0.04	5.13	-0.07	5.38	-0.11
22	**NA**	6.13	-0.01	6.17	0.00	6.22	0.03	6.28	0.04	6.33	0.05	6.39	0.07
23	**Port Creek Gate**	5.41	-0.04	5.53	-0.03	5.65	-0.02	5.86	-0.03	6.00	-0.03	6.13	-0.04
24	**Port Creek Gate**	4.29	-0.03	4.30	-0.03	4.32	-0.03	4.43	0.02	4.59	0.11	4.80	0.24
25	**Port Creek Gate**	4.31	-0.03	4.33	-0.03	4.38	-0.02	4.54	0.03	4.74	0.12	4.98	0.22
26	**Newmarket Creek Gate**	4.47	-0.02	4.60	-0.05	4.79	-0.09	5.10	-0.11	5.48	-0.06	5.98	-0.03
27	**Newmarket Creek Gate**	5.09	-0.01	5.45	-0.01	5.85	0.00	6.21	0.00	6.46	0.00	6.70	-0.01

1. The column labeled "Difference from without-project condition" shows the difference between the with-project minus the without-project.
2. A positive (+) means the project increases the wsel where a negative (-) means the project decreases the wsel.
Denotes a > 0.5 ft. increase induced by the project.

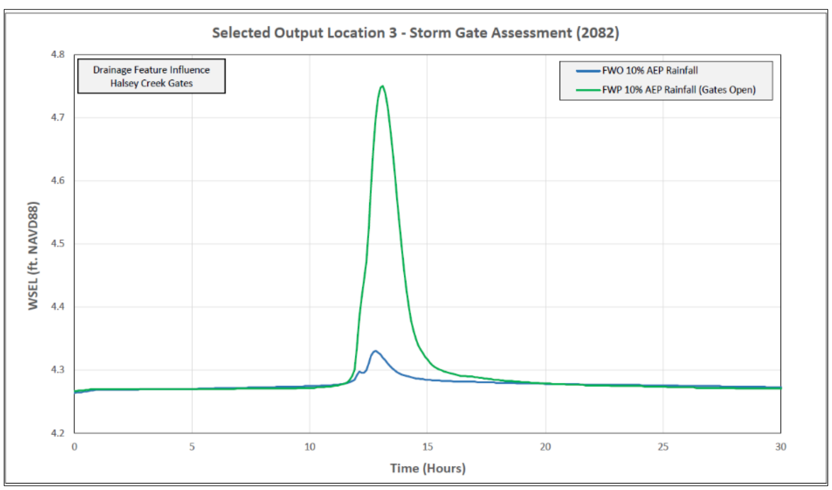

Figure 4.4.11 Storm Gate Assessment – Output Location 3 – 2082

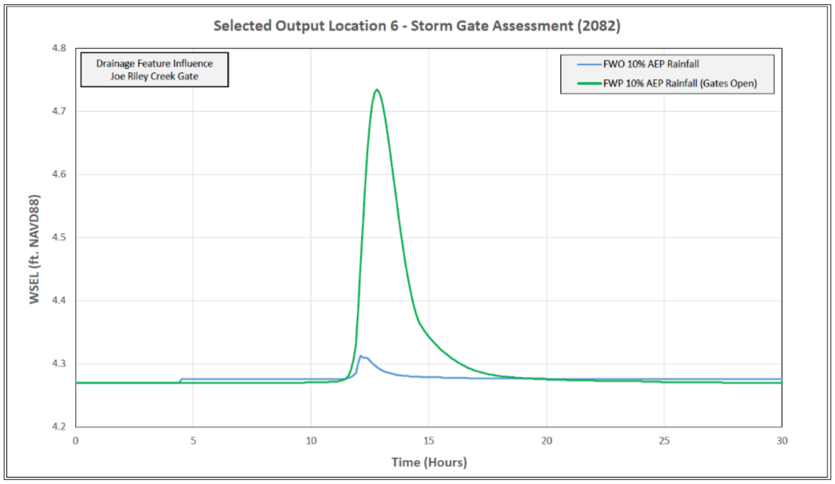

Figure 4.4.12 Storm Gate Assessment – Output Location 6 – 2082

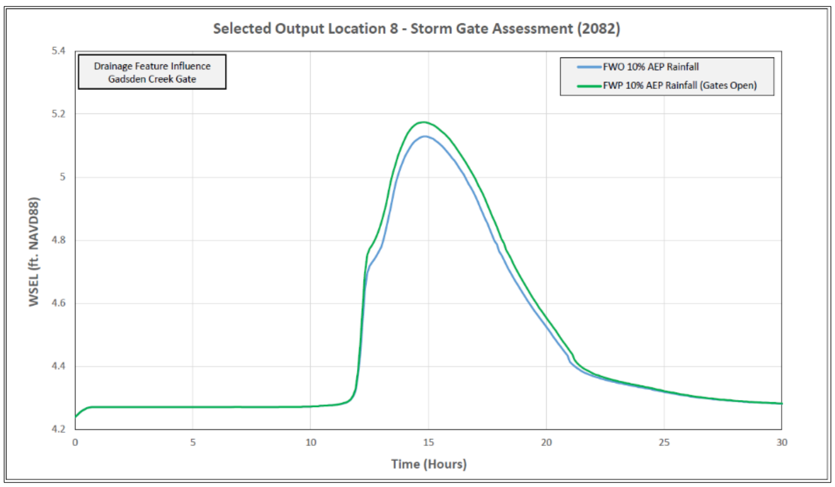

Figure 4.4.13 Storm Gate Assessment – Output Location 8 – 2082

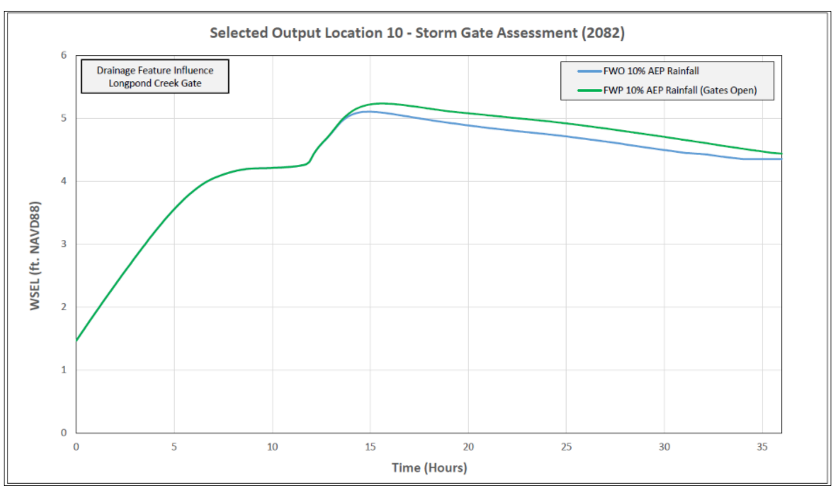

Figure 4.4.14 Storm Gate Assessment – Output Location 10 – 2082

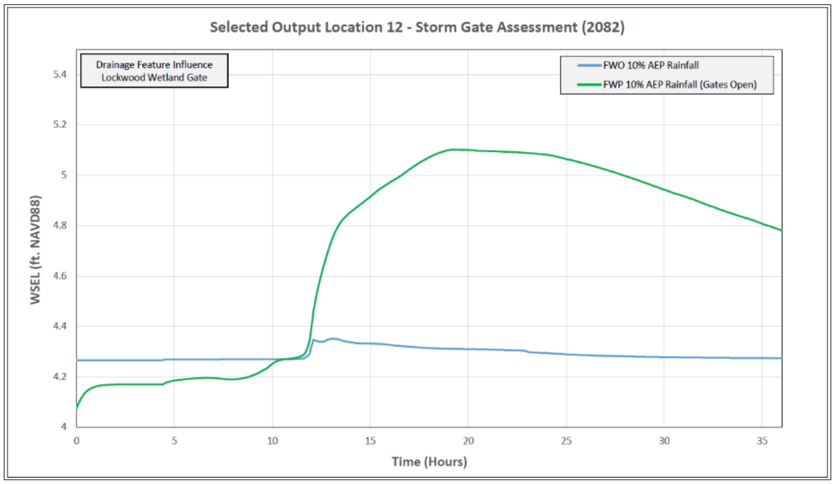

Figure 4.4.15 Storm Gate Assessment – Output Location 12 – 2082

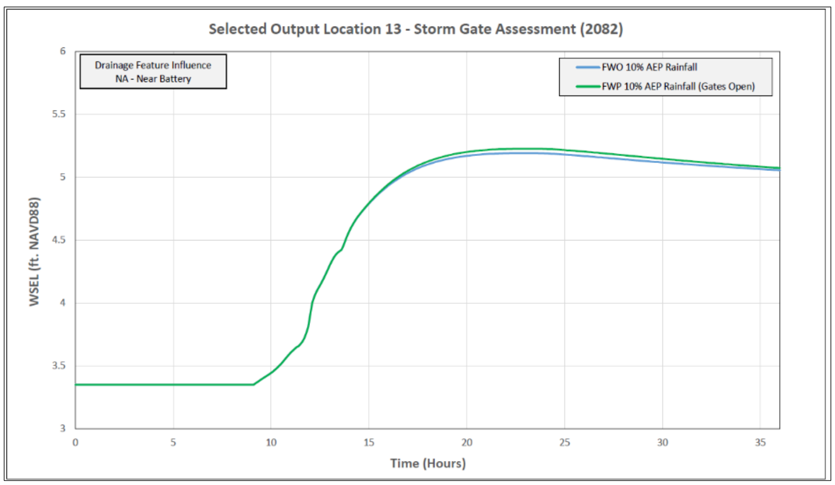

Figure 4.4.16 Storm Gate Assessment – Output Location 13 – 2082

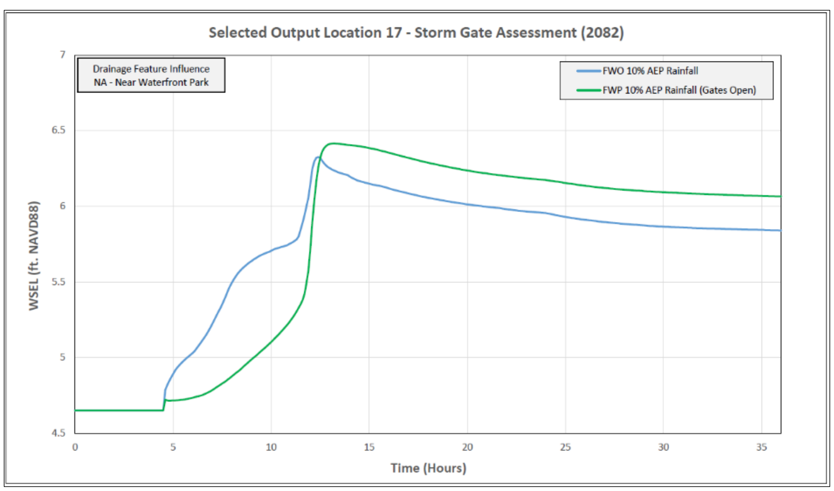

Figure 4.4.17 Storm Gate Assessment – Output Location 17 – 2082

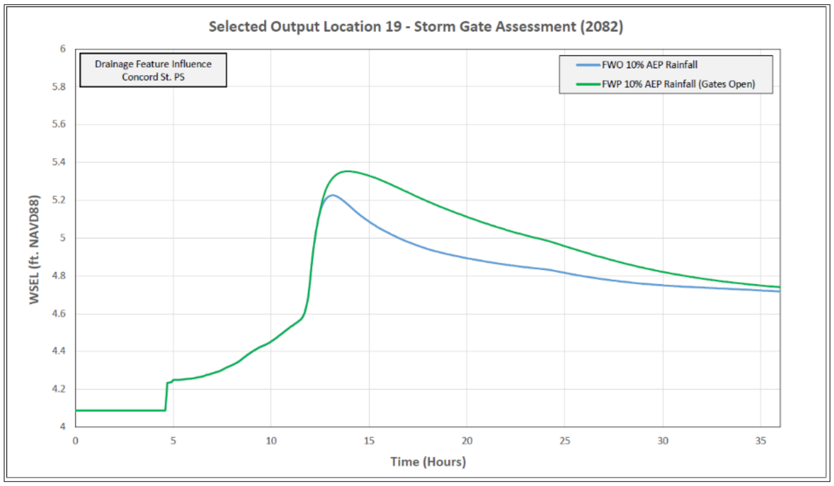

Figure 4.4.18 Storm Gate Assessment – Output Location 19 – 2082

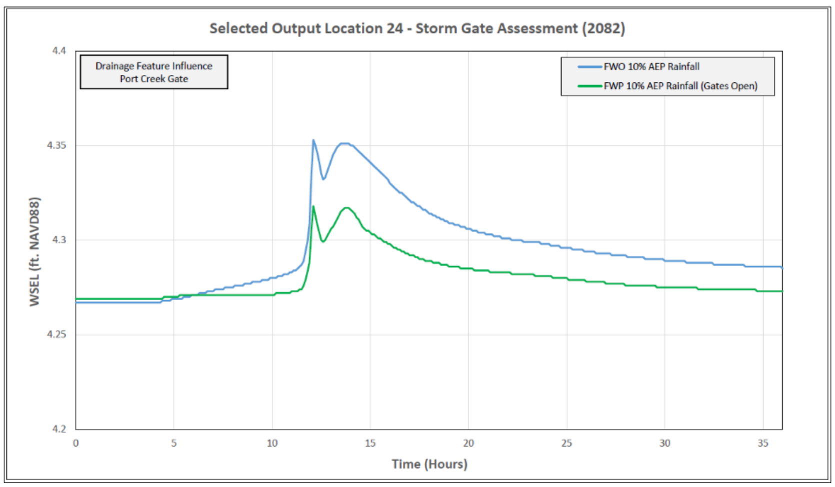

Figure 4.4.19 Storm Gate Assessment – Output Location 24 – 2082

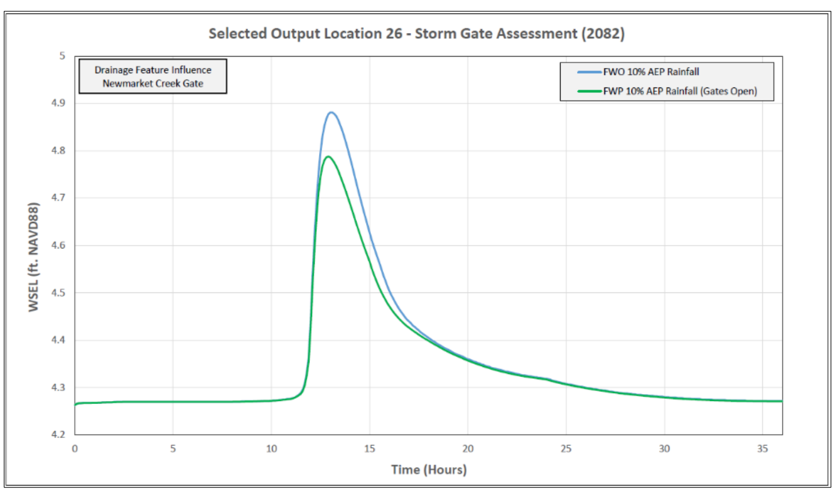

Figure 4.4.20 Storm Gate Assessment – Output Location 26 - 2082

4.5 PUMP STATION EVALUATION

The tables in Section 4.5 display the peak water surface elevations collected from the selected output locations for the storm gates closed simulations. Storm gates are closed throughout model simulation. The tables include a column for the "Selected Output Locations" and another column to describe the nearest drainage feature influencing each selected output location.

The tables include the with-project water surface elevations and an additional column labeled "Difference from without-project condition (ft.)". This additional column displays if the project alternative is increasing (+) or decreasing (-) the peak water surface elevation at the selected output locations. Increasing the interior water surface would be considered a negative effect of the project alternative and decreasing the interior water surface could be considered a positive effect of the project alternative. In addition, the difference from without-project column is color coated (gold) to signify the locations with a greater than 0.5-foot (6 inch) increase occurring due to the project.

In addition to the output tables, hydrographs are presented for some output locations.

Pump Operations

The pump stations in the HEC-RAS model are set to turn on at the NWS currently denoted "Action Stage" of 3.36 feet NAVD88. Due to the RAS 2D model being limited to the underlying terrain elevations, pump stations with inlet land elevations greater than 3.36 feet NAVD88 are set to turn on (WS Elev. on, ft.) at an elevation slightly above the terrain elevation at the inlet. The pump stations are set to turn off (WS Elev. off, ft.) at a low elevation of -1 feet NAVD88. For most pump stations in the model, this assumes the pumps are to remain on throughout the remainder of the simulation once the turn on elevation is achieved. The pump stations are provided inlet and outlet reference points and the inlet reference point controls the pump on/off elevations. Pump stations may iterate through on/off operations as the 2D cell encompassing the inlet reference point iterates through dry/wet condition as stormwater is discharged via the pumping.

The pump stations are applied a 10-minute start up (warm-up to max flow) time once the on elevation is achieved and a 1-minute shutdown time if the 2D cell goes dry or if the off elevation is achieved. The pump stations are assumed to operate at maximum efficiency (after the 10-minute warm up) meaning the HEC-RAS model contains the max flow (cubic feet per second, cfs) for each head elevation along the pump efficiency curve. In reality, pump operations are more complex, and efficiencies may increase/decrease as the tailwater/exterior head elevations fluctuate with the tide/storm surge.

The tables in Sections 4.5.1 and 4.5.2 contain the gates closed conditions for the alternative with no USACE pumps, Pump Station Alternative 1, Pump Station Alternative 2, and Pump Station Alternative 3.

4.5.1 PUMP STATION RESULTS (2032)

Table 4.5.1 Future With-Project (Gates Closed/No USACE Pumps) -2032

Selected Output Locations	Nearest Drainage Feature Influence (City PS)	FW 2032 (gates closed) City Pumps Active/No USACE Pumps											
		50% AEP		20% AEP		10% AEP		4% AEP		2% AEP		1% AEP	
		Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)
1	**NA**	7.32	3.74	7.89	4.25	8.08	4.40	8.31	4.56	8.46	4.66	8.68	4.83
2	**NA**	3.42	0.23	5.86	2.67	6.63	3.44	7.35	4.16	7.81	4.62	8.25	5.06
3	**NA**	5.46	2.25	6.00	2.75	6.62	3.31	7.36	3.95	7.81	4.32	8.24	4.64
4	**NA**	5.90	-0.02	6.18	-0.01	6.63	0.23	7.38	0.78	7.82	1.07	8.25	1.35
5	**NA**	6.40	0.13	6.55	0.20	6.68	0.26	7.38	0.88	7.82	1.27	8.26	1.66
6	**NA**	5.37	2.15	5.67	2.41	5.90	2.58	6.18	2.77	6.37	2.89	6.56	3.00
7	**NA**	5.47	0.92	5.66	0.75	5.90	0.64	6.18	0.60	6.36	0.61	6.55	0.64
8	**NA**	5.47	1.66	5.66	1.21	5.89	0.92	6.16	0.82	6.34	0.81	6.52	0.81
9	**MUSC PS**	4.40	0.00	4.45	0.00	4.52	0.00	4.74	0.14	4.91	0.26	5.31	0.53
10	**NA**	4.74	0.45	4.99	0.26	5.22	0.24	5.45	0.29	5.60	0.35	5.78	0.43
11	**NA**	4.74	0.04	4.99	0.09	5.22	0.12	5.45	0.08	5.60	0.07	5.78	0.09
12	**NA**	4.54	1.26	5.04	1.64	5.27	1.66	5.53	1.43	5.70	1.34	5.89	1.41
13	**NA**	4.83	0.00	5.06	0.02	5.28	0.10	5.55	0.19	5.71	0.24	5.90	0.30
14	**NA**	4.83	0.00	5.06	0.02	5.28	0.09	5.55	0.19	5.71	0.24	5.90	0.30
15	**NA**	5.11	0.00	5.29	-0.01	5.47	0.00	5.67	0.00	5.79	0.00	5.92	0.00
16	**NA**	6.18	0.00	6.37	0.00	6.57	-0.01	6.81	0.00	6.97	0.00	7.14	-0.01
17	**NA**	6.84	0.66	6.87	0.62	6.92	0.58	7.06	0.58	7.23	0.68	7.43	0.81
18	**NA**	6.48	0.47	6.57	0.37	6.70	0.32	6.93	0.33	7.12	0.35	7.34	0.37
19	**Concord St. PS**	5.05	0.09	5.20	0.12	5.35	0.12	5.55	0.16	5.69	0.18	5.84	0.18
20	**Concord St. PS**	5.56	0.01	5.74	0.02	5.89	0.01	6.07	0.01	6.20	0.02	6.34	0.02
21	**Concord St. PS**	4.56	-0.02	4.69	-0.02	4.83	-0.02	5.01	0.04	5.13	-0.07	5.37	-0.12
22	**NA**	6.12	-0.02	6.16	-0.01	6.21	0.02	6.35	0.11	6.55	0.27	6.78	0.46
23	**NA**	5.45	0.00	5.71	0.15	6.05	0.38	6.35	0.46	6.55	0.52	6.79	0.62
24	**NA**	5.29	2.05	5.72	2.45	6.05	2.73	6.35	2.90	6.55	2.97	6.78	3.03
25	**NA**	5.29	1.33	5.72	1.67	6.05	1.91	6.35	2.15	6.55	2.30	6.78	2.43
26	**NA**	5.43	1.64	6.18	2.08	6.69	2.21	7.21	2.31	7.45	2.10	7.68	1.80
27	**NA**	5.43	0.51	6.18	0.87	6.69	0.86	7.21	1.01	7.45	1.01	7.68	0.99

1. The column labeled "Difference from without-project condition" shows the difference between the with-project minus the without-project.
2. A positive (+) means the project increases the wsel where a negative (-) means the project decreases the wsel.
Denotes a > 0.5 ft. increase induced by the project.

Table 4.5.2 Future With-Project (Gates Closed/Pump Station Alternative 1) -2032

Selected Output Locations	Nearest Drainage Feature Influence (City PS/USACE PS)	FW 2032 (gates closed) City Pumps Active/USACE Pump Station Alternative 1											
		50% AEP		20% AEP		10% AEP		4% AEP		2% AEP		1% AEP	
		Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)
1	**NA**	7.33	3.75	7.89	4.25	8.08	4.40	8.31	4.56	8.46	4.66	8.68	4.83
2	**Halsey PS**	2.63	-0.56	2.97	-0.22	3.88	0.69	6.52	3.33	7.12	3.93	7.69	4.50
3	**Halsey PS**	3.76	0.55	4.69	1.44	5.81	2.50	6.51	3.10	7.12	3.63	7.69	4.09
4	**Halsey PS**	5.90	-0.02	6.18	-0.01	6.40	0.00	6.62	0.02	7.12	0.37	7.69	0.79
5	**Halsey PS**	6.46	0.19	6.59	0.24	6.72	0.30	6.85	0.35	7.14	0.59	7.69	1.09
6	**Joe Riley PS**	3.38	0.16	3.60	0.34	5.13	1.81	5.94	2.53	6.20	2.72	6.44	2.88
7	**Joe Riley/SF PS**	5.47	0.92	5.60	0.69	5.74	0.48	5.95	0.37	6.20	0.45	6.44	0.53
8	**SF PS**	5.47	1.66	5.60	1.15	5.74	0.77	5.94	0.60	6.19	0.66	6.42	0.71
9	**MUSC PS**	4.40	0.00	4.45	0.00	4.51	-0.01	4.65	0.05	4.82	0.17	5.04	0.26
10	**City Marina PS**	3.96	-0.33	4.67	-0.06	5.00	0.02	5.29	0.13	5.48	0.23	5.68	0.33
11	**City Marina PS**	4.72	0.02	4.91	0.01	5.11	0.01	5.34	-0.03	5.51	-0.02	5.71	0.02
12	**Potential Temp PS**	4.43	1.15	4.66	1.26	4.98	1.37	5.31	1.21	5.52	1.16	5.75	1.27
13	**Battery 1 PS**	4.40	-0.43	4.60	-0.44	4.93	-0.25	5.31	-0.05	5.53	0.06	5.76	0.16
14	**Battery 1 PS**	4.41	-0.42	4.61	-0.43	4.93	-0.26	5.31	-0.05	5.53	0.06	5.76	0.16
15	**Battery 2 PS**	4.66	-0.45	4.92	-0.38	5.28	-0.19	5.54	-0.13	5.70	-0.09	5.85	-0.07
16	**Battery 3 PS**	5.98	-0.20	6.22	-0.15	6.45	-0.13	6.70	-0.11	6.87	-0.10	7.05	-0.10
17	**Waterfront Park PS**	5.89	-0.29	6.29	0.04	6.65	0.31	6.90	0.42	7.08	0.53	7.30	0.68
18	**Waterfront Park PS**	6.25	0.24	6.44	0.24	6.62	0.24	6.85	0.25	7.05	0.28	7.28	0.31
19	**Concord St. PS**	5.05	0.09	5.20	0.12	5.35	0.12	5.54	0.15	5.67	0.16	5.83	0.17
20	**Concord St. PS**	5.57	0.02	5.74	0.02	5.90	0.02	6.08	0.02	6.21	0.03	6.35	0.03
21	**Concord St. PS**	4.73	0.15	4.79	0.08	4.93	0.08	5.11	0.14	5.23	0.03	5.43	-0.06
22	**Port 1 PS**	6.12	-0.02	6.15	-0.02	6.18	-0.01	6.20	-0.04	6.27	-0.01	6.40	0.08
23	**Port 2 PS**	5.43	-0.02	5.55	-0.01	5.68	0.01	5.91	0.02	6.17	0.14	6.42	0.25
24	**Port 2 PS**	3.37	0.13	3.56	0.29	4.61	1.29	5.78	2.33	6.13	2.55	6.41	2.66
25	**Port 2 PS**	3.93	-0.03	4.02	-0.03	4.62	0.48	5.79	1.59	6.14	1.89	6.41	2.06
26	**Newmarket PS**	3.42	-0.37	3.79	-0.31	4.82	0.34	6.16	1.26	6.72	1.37	7.14	1.26
27	**Newmarket PS**	4.90	-0.02	5.29	-0.02	5.84	0.01	6.25	0.05	6.72	0.28	7.14	0.45

1. The column labeled "Difference from without-project condition" shows the difference between the with-project minus the without-project.
2. A positive (+) means the project increases the wsel where a negative (-) means the project decreases the wsel.
Denotes a > 0.5 ft. increase induced by the project.

Table 4.5.3 Future With-Project (Gates Closed/USACE Pump Station Alt. 2) -2032

Selected Output Locations	Nearest Drainage Feature Influence (City PS/USACE PS)	FW 2032 (gates closed) City Pumps Active/USACE Pump Station Alternative 2											
		50% AEP		20% AEP		10% AEP		4% AEP		2% AEP		1% AEP	
		Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)
1	**NA**	7.33	3.75	7.89	4.25	8.08	4.40	8.31	4.56	8.46	4.66	8.68	4.83
2	**Halsey PS**	2.79	-0.40	3.13	-0.06	3.78	0.59	6.22	3.03	6.87	3.68	7.48	4.29
3	**Halsey PS**	3.47	0.26	4.19	0.94	5.37	2.06	6.28	2.87	6.85	3.36	7.48	3.88
4	**Halsey PS**	5.90	-0.02	6.18	-0.01	6.40	0.00	6.61	0.01	6.76	0.01	7.48	0.58
5	**Halsey PS**	6.47	0.20	6.61	0.26	6.73	0.31	6.86	0.36	6.91	0.36	7.48	0.88
6	**Joe Riley PS**	3.37	0.15	3.43	0.17	4.24	0.92	5.73	2.32	6.10	2.62	6.38	2.82
7	**Joe Riley/SF PS**	5.47	0.92	5.60	0.69	5.74	0.48	5.90	0.32	6.11	0.36	6.38	0.47
8	**SF PS**	5.47	1.66	5.60	1.15	5.74	0.77	5.90	0.56	6.11	0.58	6.36	0.65
9	**MUSC PS**	4.40	0.00	4.45	0.00	4.51	-0.01	4.59	-0.01	4.77	0.12	4.99	0.21
10	**City Marina PS**	3.43	-0.86	4.25	-0.48	4.83	-0.15	5.20	0.04	5.40	0.15	5.60	0.25
11	**City Marina PS**	4.72	0.02	4.91	0.01	5.11	0.01	5.34	-0.03	5.48	-0.05	5.67	-0.02
12	**Potential Temp PS**	4.43	1.15	4.65	1.25	4.84	1.23	5.15	1.05	5.37	1.01	5.63	1.15
13	**Battery 1 PS**	4.39	-0.44	4.55	-0.49	4.70	-0.48	5.09	-0.27	5.37	-0.10	5.63	0.03
14	**Battery 1 PS**	4.40	-0.43	4.56	-0.48	4.72	-0.47	5.10	-0.26	5.37	-0.10	5.63	0.03
15	**Battery 2 PS**	4.60	-0.51	4.66	-0.64	4.90	-0.57	5.40	-0.27	5.60	-0.19	5.78	-0.14
16	**Battery 3 PS**	5.70	-0.48	6.07	-0.30	6.35	-0.23	6.62	-0.19	6.80	-0.17	6.99	-0.16
17	**Waterfront Park PS**	4.61	-1.57	4.65	-1.60	4.67	-1.67	6.46	-0.02	6.95	0.40	7.20	0.58
18	**Waterfront Park PS**	6.25	0.24	6.43	0.23	6.62	0.24	6.84	0.24	7.02	0.25	7.24	0.27
19	**Concord St. PS**	5.05	0.09	5.20	0.12	5.35	0.12	5.54	0.15	5.67	0.16	5.82	0.16
20	**Concord St. PS**	5.55	0.00	5.73	0.01	5.88	0.00	6.06	0.00	6.18	0.00	6.32	0.00
21	**Concord St. PS**	4.74	0.16	4.78	0.07	4.92	0.07	5.09	0.12	5.22	0.02	5.42	-0.07
22	**Port 1 PS**	6.09	-0.05	6.13	-0.04	6.15	-0.04	6.16	-0.08	6.19	-0.09	6.27	-0.05
23	**Port 2 PS**	5.43	-0.02	5.55	-0.01	5.68	0.01	5.84	-0.05	6.06	0.03	6.31	0.14
24	**Port 2 PS**	3.34	0.10	3.40	0.13	3.87	0.55	5.35	1.90	5.95	2.37	6.28	2.53
25	**Port 2 PS**	3.97	0.01	4.07	0.02	4.16	0.02	5.35	1.15	5.95	1.70	6.28	1.93
26	**Newmarket PS**	3.40	-0.39	3.49	-0.61	4.13	-0.35	5.62	0.72	6.40	1.05	6.92	1.04
27	**Newmarket PS**	4.90	-0.02	5.29	-0.02	5.83	0.00	6.22	0.02	6.46	0.02	6.93	0.24

1. The column labeled "Difference from without-project condition" shows the difference between the with-project minus the without-project.
2. A positive (+) means the project increases the wsel where a negative (-) means the project decreases the wsel.
Denotes a > 0.5 ft. increase induced by the project.

Table 4.5.4 Future With-Project (Gates Closed/USACE Pump Station Alt. 3) -2032

Selected Output Locations	Nearest Drainage Feature Influence (City PS/USACE PS)	FW 2032 (gates closed) City Pumps Active/USACE Pump Station Alternative 3											
		50% AEP		20% AEP		10% AEP		4% AEP		2% AEP		1% AEP	
		Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)
1	**NA**	7.33	3.75	7.89	4.25	8.07	4.39	8.30	4.55	8.46	4.66	8.68	4.83
2	**Halsey PS**	2.94	-0.25	3.29	0.10	3.91	0.72	5.36	2.17	6.50	3.31	7.16	3.97
3	**Halsey PS**	3.38	0.17	3.68	0.43	4.64	1.33	6.02	2.61	6.48	2.99	7.15	3.55
4	**Halsey PS**	5.90	-0.02	6.18	-0.01	6.40	0.00	6.62	0.02	6.75	0.00	7.16	0.26
5	**Halsey PS**	6.41	0.14	6.55	0.20	6.68	0.26	6.81	0.31	6.88	0.33	7.16	0.56
6	**Joe Riley PS**	3.36	0.14	3.39	0.13	3.52	0.20	4.71	1.30	5.75	2.27	6.22	2.66
7	**Joe Riley/SF PS**	5.47	0.92	5.60	0.69	5.74	0.48	5.90	0.32	6.02	0.27	6.24	0.33
8	**SF PS**	5.47	1.66	5.60	1.15	5.74	0.77	5.90	0.56	6.02	0.49	6.26	0.55
9	**MUSC PS**	4.40	0.00	4.45	0.00	4.51	-0.01	4.59	-0.01	4.71	0.06	4.90	0.12
10	**City Marina PS**	3.38	-0.91	3.49	-1.24	4.31	-0.67	4.99	-0.17	5.26	0.01	5.49	0.14
11	**City Marina PS**	4.71	0.01	4.91	0.01	5.11	0.01	5.33	-0.04	5.47	-0.06	5.63	-0.06
12	**Potential Temp PS**	4.43	1.15	4.65	1.25	4.80	1.19	4.98	0.88	5.19	0.83	5.42	0.94
13	**Battery 1 PS**	4.38	-0.45	4.53	-0.51	4.68	-0.50	4.85	-0.51	5.06	-0.41	5.39	-0.21
14	**Battery 1 PS**	4.40	-0.43	4.55	-0.49	4.71	-0.48	4.89	-0.47	5.08	-0.39	5.40	-0.20
15	**Battery 2 PS**	4.57	-0.54	4.62	-0.68	4.69	-0.78	4.92	-0.75	5.35	-0.44	5.62	-0.30
16	**Battery 3 PS**	4.90	-1.28	5.67	-0.70	6.10	-0.48	6.44	-0.37	6.64	-0.33	6.85	-0.30
17	**Waterfront Park PS**	4.64	-1.54	4.65	-1.60	4.68	-1.66	4.75	-1.73	5.20	-1.35	6.91	0.29
18	**Waterfront Park PS**	6.25	0.24	6.43	0.23	6.62	0.24	6.84	0.24	7.01	0.24	7.22	0.25
19	**Concord St. PS**	5.05	0.09	5.20	0.12	5.35	0.12	5.54	0.15	5.67	0.16	5.82	0.16
20	**Concord St. PS**	5.56	0.01	5.73	0.01	5.89	0.01	6.07	0.01	6.20	0.02	6.36	0.04
21	**Concord St. PS**	4.45	-0.13	4.58	-0.13	4.72	-0.13	4.91	-0.06	5.02	-0.18	5.36	-0.13
22	**Port 1 PS**	6.10	-0.04	6.14	-0.03	6.14	-0.05	6.15	-0.09	6.16	-0.12	6.20	-0.12
23	**Port 2 PS**	5.43	-0.02	5.55	-0.01	5.68	0.01	5.87	-0.02	5.99	-0.04	6.19	0.02
24	**Port 2 PS**	3.36	0.12	3.36	0.09	3.36	0.04	4.14	0.69	5.29	1.71	6.01	2.26
25	**Port 2 PS**	3.97	0.01	4.06	0.01	4.16	0.02	4.25	0.05	5.30	1.05	6.01	1.66
26	**Newmarket PS**	3.38	-0.41	3.43	-0.67	3.57	-0.91	4.51	-0.39	5.64	0.29	6.48	0.60
27	**Newmarket PS**	4.90	-0.02	5.29	-0.02	5.83	0.00	6.20	0.00	6.44	0.00	6.69	0.00

1. The column labeled "Difference from without-project condition" shows the difference between the with-project minus the without-project.
2. A positive (+) means the project increases the wsel where a negative (-) means the project decreases the wsel.
Denotes a > 0.5 ft. increase induced by the project.

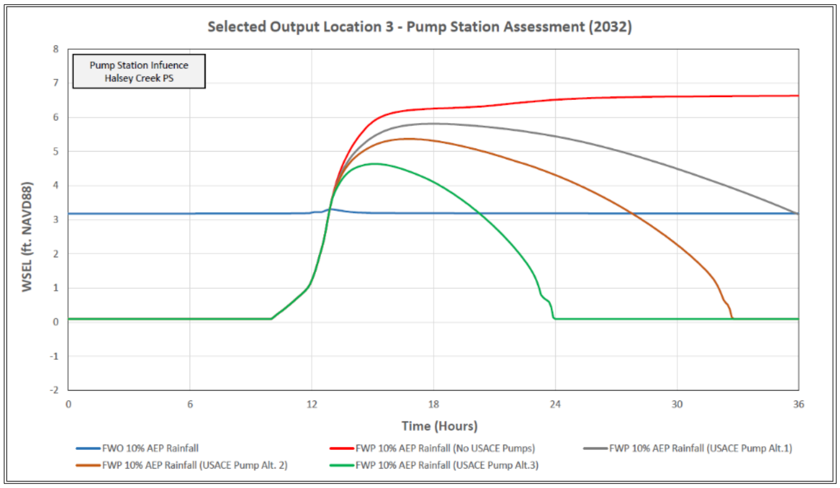

Figure 4.5.1 Pump Station Assessment – Output Location 3 – 2032

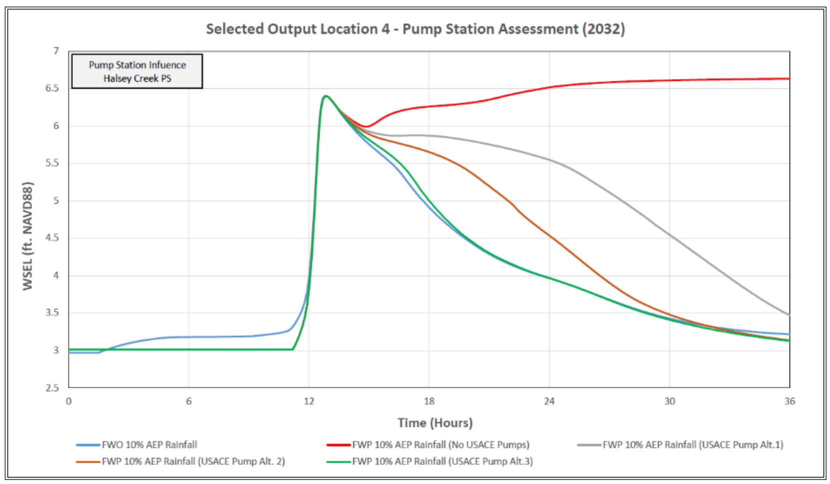

Figure 4.5.2 Pump Station Assessment – Output Location 4 – 2032

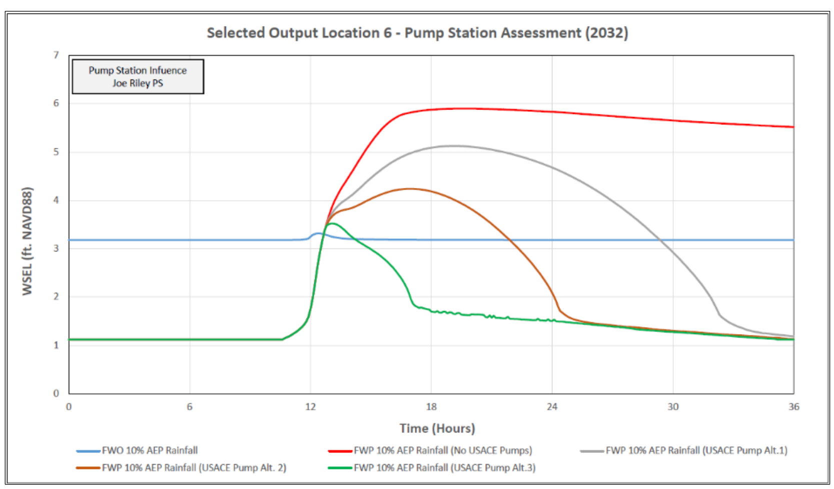

Figure 4.5.3 Pump Station Assessment – Output Location 6 – 2032

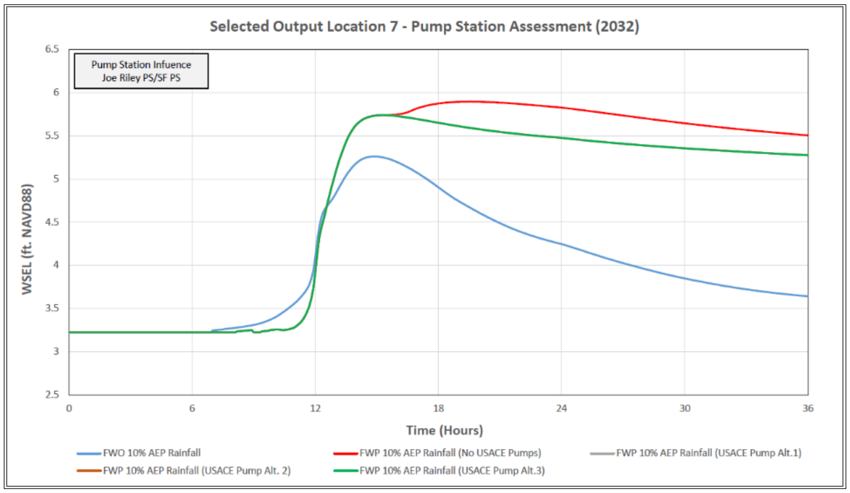

Figure 4.5.4 Pump Station Assessment – Output Location 7 – 2032

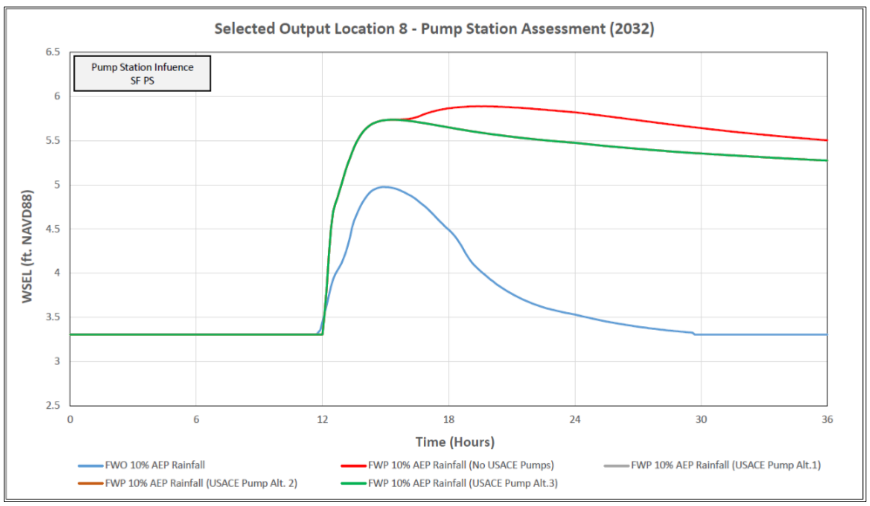

Figure 4.5.5 Pump Station Assessment – Output Location 8 – 2032

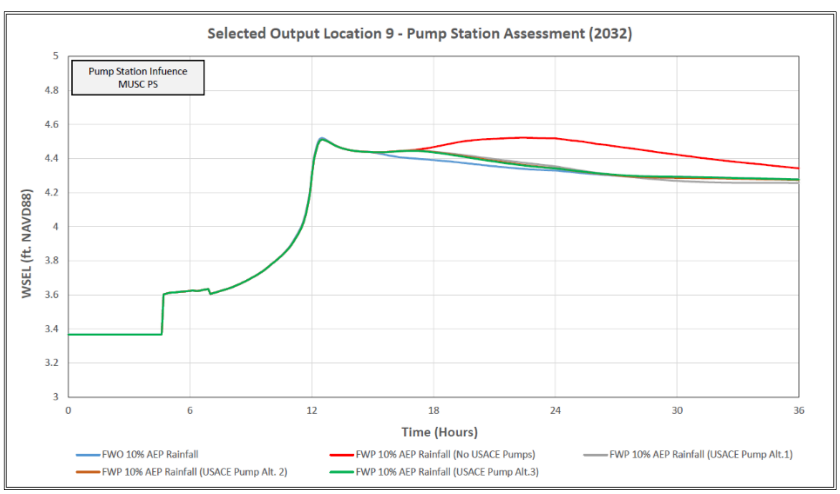

Figure 4.5.6 Pump Station Assessment – Output Location 9 – 2032

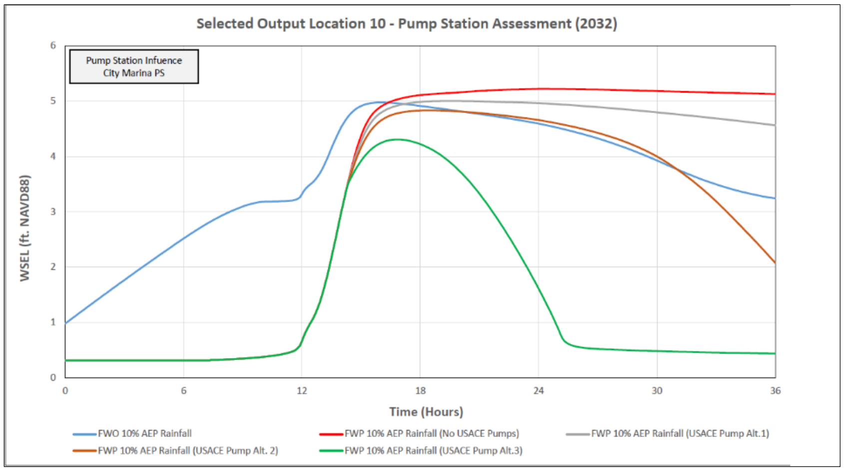

Figure 4.5.7 Pump Station Assessment – Output Location 10 – 2032

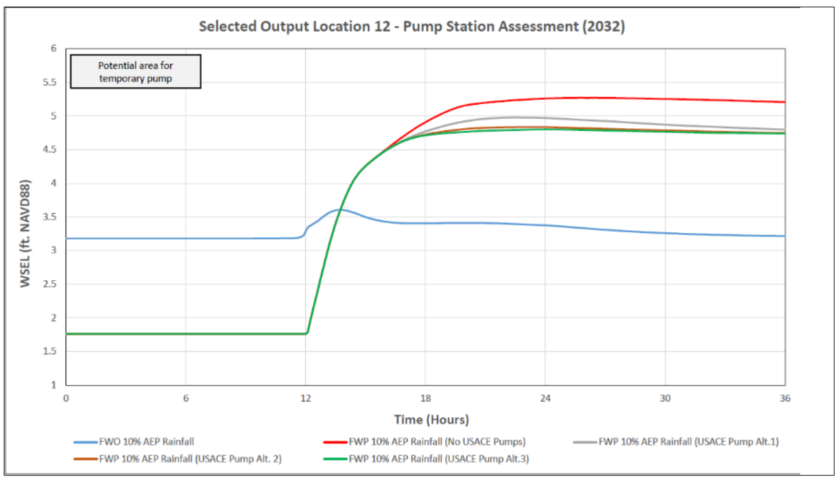

Figure 4.5.8 Pump Station Assessment – Output Location 12 – 2032

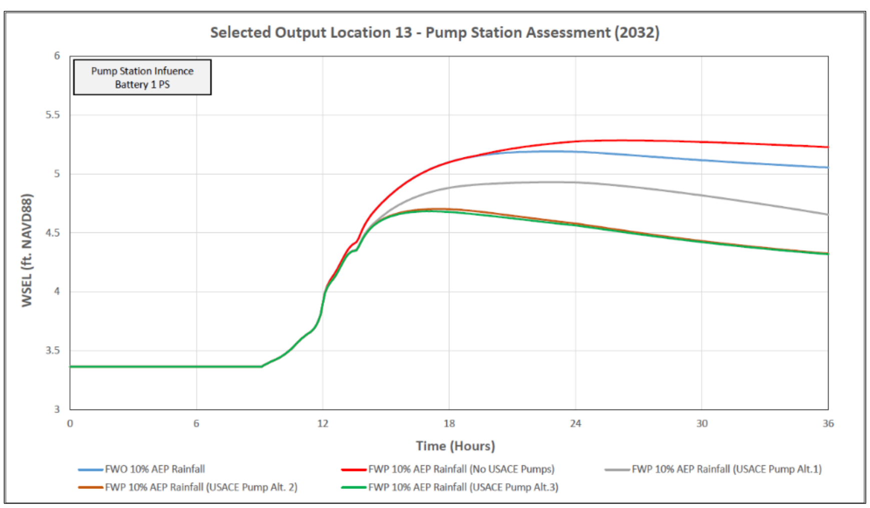

Figure 4.5.9 Pump Station Assessment – Output Location 13 – 2032

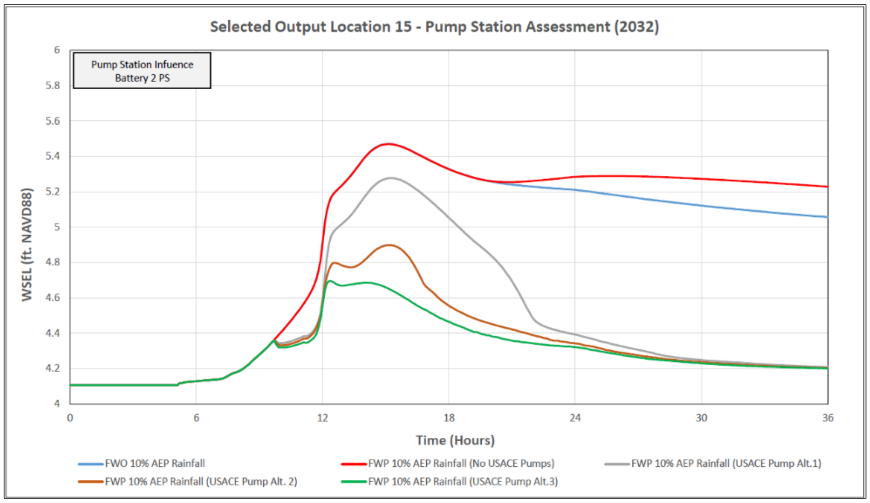

Figure 4.5.10 Pump Station Assessment – Output Location 15 – 2032

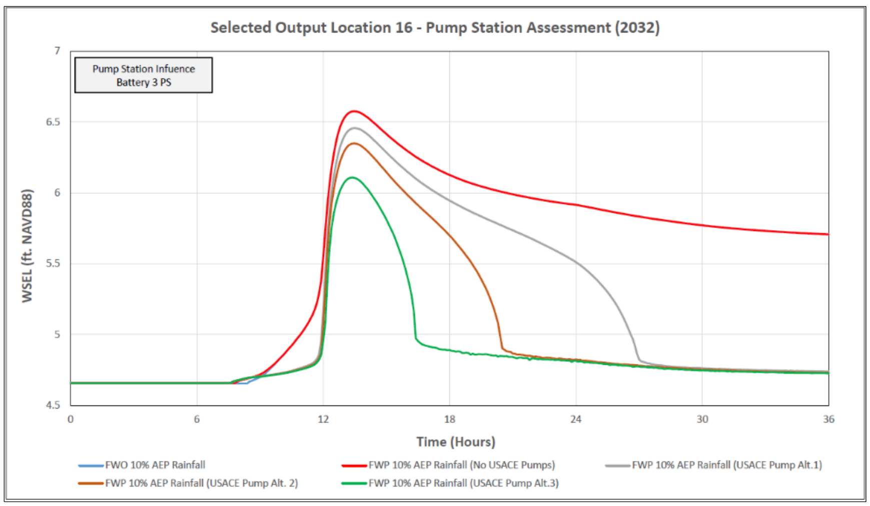

Figure 4.5.11 Pump Station Assessment – Output Location 16 – 2032

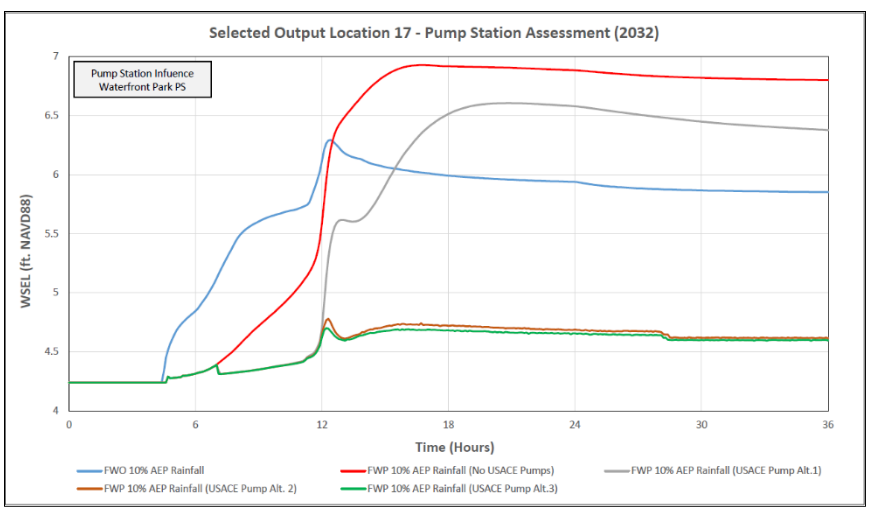

Figure 4.5.12 Pump Station Assessment – Output Location 17 – 2032

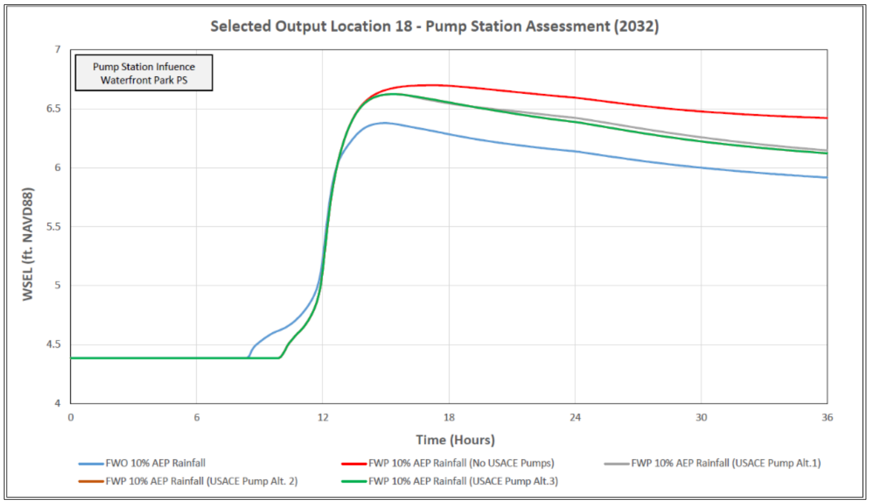

Figure 4.5.13 Pump Station Assessment – Output Location 18 - 2032

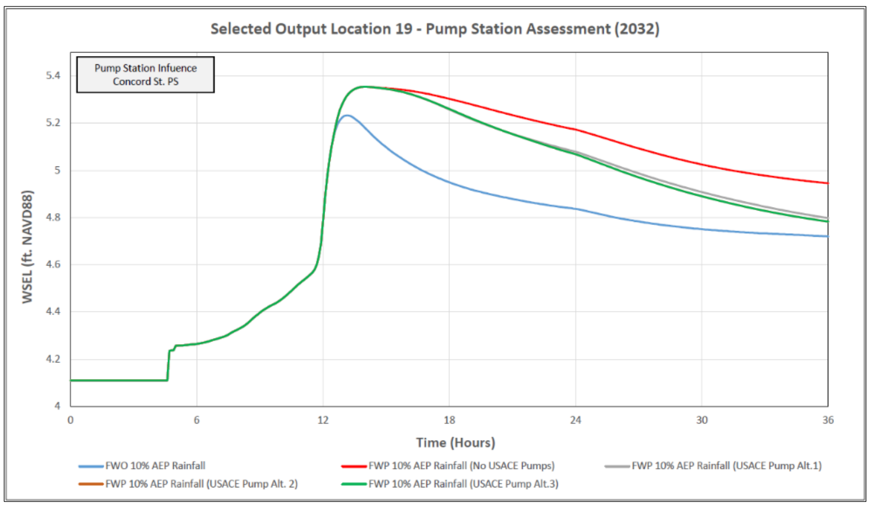

Figure 4.5.14 Pump Station Assessment – Output Location 19 – 2032

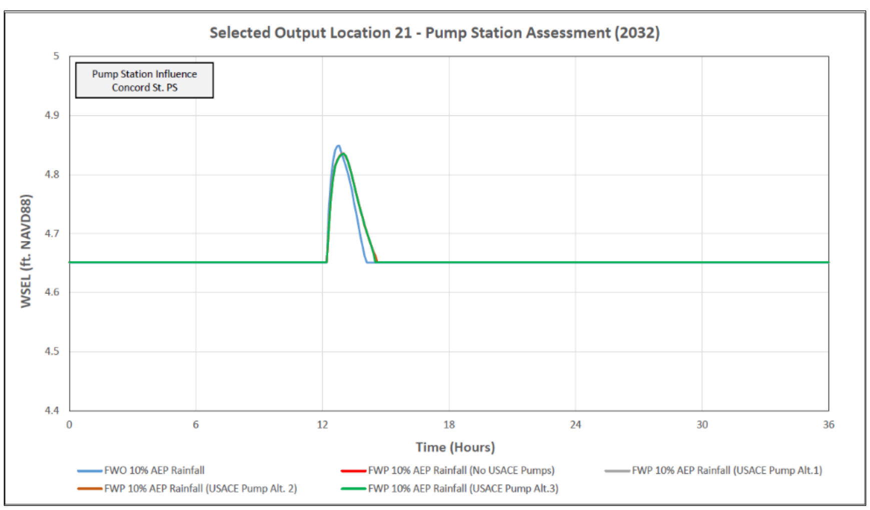

Figure 4.5.15 Pump Station Assessment – Output Location 21 – 2032

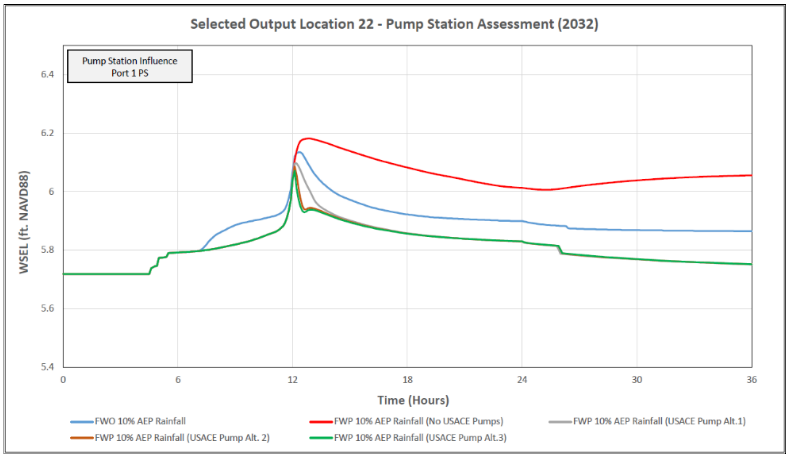

Figure 4.5.16 Pump Station Assessment – Output Location 22 – 2032

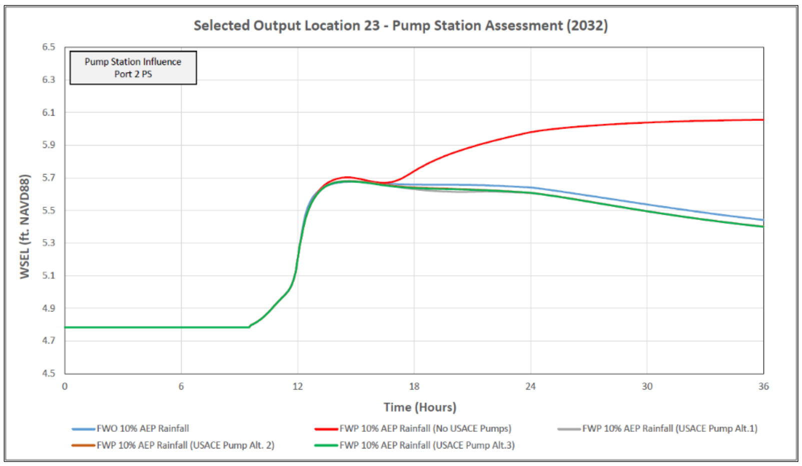

Figure 4.5.17 Pump Station Assessment – Output Location 23 – 2032

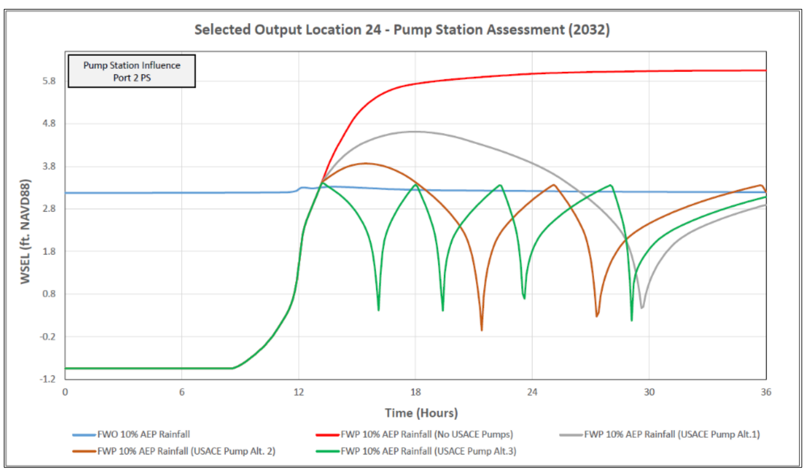

Figure 4.5.18 Pump Station Assessment – Output Location 24 – 2032

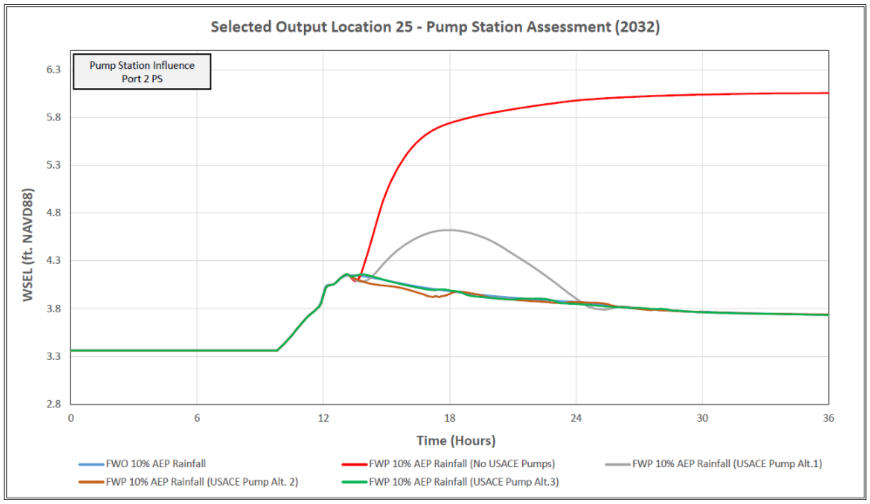

Figure 4.5.19 Pump Station Assessment – Output Location 25 - 2032

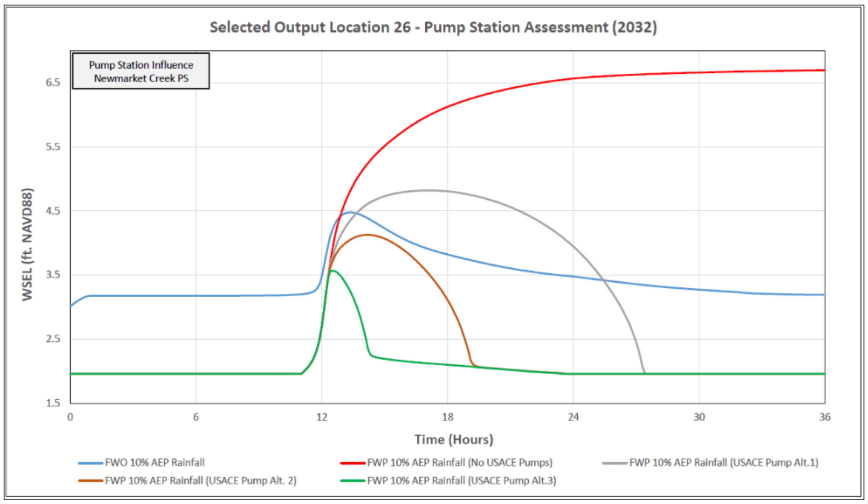

Figure 4.5.20 Pump Station Assessment – Output Location 26 - 2032

4.5.2 PUMP STATION RESULTS (2082)

Table 4.5.5 Future With-Project (Gates Closed/No USACE Pumps) -2082

Selected Output Locations	Nearest Drainage Feature Influence (City PS)	FW 2082 (gates closed) City Pumps Active/No USACE Pumps											
		50% AEP		20% AEP		10% AEP		4% AEP		2% AEP		1% AEP	
		Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)
1	NA	7.32	3.03	7.89	3.59	8.08	3.76	8.31	3.97	8.46	4.10	8.68	4.29
2	NA	3.42	-0.96	5.86	1.48	6.63	2.25	7.35	2.97	7.81	3.42	8.25	3.86
3	NA	5.46	1.17	6.00	1.69	6.62	2.29	7.36	2.99	7.81	3.41	8.24	3.79
4	NA	5.90	-0.12	6.18	-0.06	6.63	0.20	7.38	0.75	7.82	1.06	8.25	1.35
5	NA	6.40	0.19	6.55	0.25	6.68	0.30	7.38	0.92	7.82	1.31	8.26	1.70
6	NA	5.37	1.08	5.67	1.37	5.90	1.59	6.18	1.84	6.37	2.01	6.56	2.17
7	NA	5.47	0.69	5.66	0.61	5.90	0.55	6.18	0.56	6.36	0.58	6.55	0.62
8	NA	5.47	0.97	5.66	0.90	5.89	0.77	6.16	0.75	6.34	0.77	6.52	0.79
9	MUSC PS	4.40	0.10	4.45	-0.02	4.52	-0.01	4.74	0.12	4.91	0.20	5.31	0.48
10	NA	4.74	-0.08	4.99	0.01	5.22	0.12	5.45	0.22	5.60	0.29	5.78	0.39
11	NA	4.74	-0.14	4.99	-0.09	5.22	-0.04	5.45	-0.02	5.60	0.00	5.78	0.05
12	NA	4.54	0.24	5.04	0.71	5.27	0.92	5.53	1.12	5.70	1.23	5.89	1.35
13	NA	4.83	0.00	5.06	0.02	5.28	0.09	5.55	0.19	5.71	0.24	5.90	0.29
14	NA	4.83	0.00	5.06	0.02	5.28	0.09	5.55	0.19	5.71	0.24	5.90	0.29
15	NA	5.11	0.00	5.29	-0.01	5.47	0.00	5.67	0.00	5.79	0.00	5.92	0.00
16	NA	6.18	0.00	6.37	0.00	6.57	-0.01	6.81	0.00	6.97	0.00	7.14	-0.01
17	NA	6.84	0.66	6.87	0.62	6.92	0.58	7.06	0.58	7.23	0.68	7.43	0.81
18	NA	6.48	0.47	6.57	0.37	6.70	0.32	6.93	0.33	7.12	0.35	7.34	0.37
19	Concord St. PS	5.05	0.09	5.20	0.12	5.35	0.12	5.55	0.16	5.69	0.18	5.84	0.18
20	Concord St. PS	5.56	0.01	5.74	0.02	5.89	0.01	6.07	0.01	6.20	0.02	6.34	0.02
21	Concord St. PS	4.56	-0.01	4.69	-0.01	4.83	-0.01	5.01	0.04	5.13	-0.07	5.37	-0.12
22	NA	6.12	-0.02	6.16	-0.01	6.21	0.02	6.35	0.11	6.55	0.27	6.78	0.46
23	NA	5.45	0.00	5.71	0.15	6.05	0.38	6.35	0.46	6.55	0.52	6.79	0.62
24	NA	5.29	0.97	5.72	1.39	6.05	1.70	6.35	1.94	6.55	2.07	6.78	2.22
25	NA	5.29	0.95	5.72	1.36	6.05	1.65	6.35	1.84	6.55	1.93	6.78	2.02
26	NA	5.43	0.94	6.18	1.53	6.69	1.81	7.21	2.00	7.45	1.91	7.68	1.67
27	NA	5.43	0.33	6.18	0.72	6.69	0.84	7.21	1.00	7.45	0.99	7.68	0.97

1. The column labeled "Difference from without-project condition" shows the difference between the with-project minus the without-project.
2. A positive (+) means the project increases the wsel where a negative (-) means the project decreases the wsel.

Denotes a > 0.5 ft. increase induced by the project.

Table 4.5.6 Future With-Project (Gates Closed/USACE Pump Station Alt. 1) -2082

Selected Output Locations	Nearest Drainage Feature Influence (City PS/USACE PS)	FW 2082 (gates closed) City Pumps Active/USACE Pump Station Alternative 1											
		50% AEP		20% AEP		10% AEP		4% AEP		2% AEP		1% AEP	
		Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)
1	NA	7.33	3.04	7.89	3.59	8.08	3.76	8.31	3.97	8.46	4.10	8.68	4.29
2	Halsey PS	2.63	-1.75	2.97	-1.41	3.88	-0.50	6.52	2.14	7.12	2.73	7.69	3.30
3	Halsey PS	3.76	-0.53	4.69	0.38	5.81	1.48	6.51	2.14	7.12	2.72	7.69	3.24
4	Halsey PS	5.90	-0.12	6.18	-0.06	6.40	-0.03	6.62	-0.01	7.12	0.36	7.69	0.79
5	Halsey PS	6.46	0.25	6.59	0.29	6.72	0.34	6.85	0.39	7.14	0.63	7.69	1.13
6	Joe Riley PS	3.38	-0.91	3.60	-0.70	5.13	0.82	5.94	1.60	6.20	1.84	6.44	2.05
7	Joe Riley/SF PS	5.47	0.69	5.60	0.55	5.74	0.39	5.95	0.33	6.20	0.42	6.44	0.51
8	SF PS	5.47	0.97	5.60	0.84	5.74	0.62	5.94	0.53	6.19	0.62	6.42	0.69
9	MUSC PS	4.40	0.10	4.45	-0.02	4.51	-0.02	4.65	0.03	4.82	0.11	5.04	0.21
10	City Marina PS	3.96	-0.86	4.67	-0.31	5.00	-0.10	5.29	0.06	5.48	0.17	5.68	0.29
11	City Marina PS	4.72	-0.16	4.91	-0.17	5.11	-0.15	5.34	-0.13	5.51	-0.09	5.71	-0.02
12	Potential Temp PS	4.43	0.13	4.66	0.33	4.98	0.63	5.31	0.90	5.52	1.05	5.75	1.21
13	Battery 1 PS	4.40	-0.43	4.60	-0.44	4.93	-0.26	5.31	-0.05	5.53	0.06	5.76	0.15
14	Battery 1 PS	4.41	-0.42	4.61	-0.43	4.93	-0.26	5.31	-0.05	5.53	0.06	5.76	0.15
15	Battery 2 PS	4.66	-0.45	4.92	-0.38	5.28	-0.19	5.54	-0.13	5.70	-0.09	5.85	-0.07
16	Battery 3 PS	5.98	-0.20	6.22	-0.15	6.45	-0.13	6.70	-0.11	6.87	-0.10	7.05	-0.10
17	Waterfront Park PS	5.89	-0.29	6.29	0.04	6.65	0.31	6.90	0.42	7.08	0.53	7.30	0.68
18	Waterfront Park PS	6.25	0.24	6.44	0.24	6.62	0.24	6.85	0.25	7.05	0.28	7.28	0.31
19	Concord St. PS	5.05	0.09	5.20	0.12	5.35	0.12	5.54	0.15	5.67	0.16	5.83	0.17
20	Concord St. PS	5.57	0.02	5.74	0.02	5.90	0.02	6.08	0.02	6.21	0.03	6.35	0.03
21	Concord St. PS	4.73	0.16	4.79	0.09	4.93	0.09	5.11	0.14	5.23	0.03	5.43	-0.06
22	Port 1 PS	6.12	-0.02	6.15	-0.02	6.18	-0.01	6.20	-0.04	6.27	-0.01	6.40	0.08
23	Port 2 PS	5.43	-0.02	5.55	-0.01	5.68	0.01	5.91	0.02	6.17	0.14	6.42	0.25
24	Port 2 PS	3.37	-0.95	3.56	-0.77	4.61	0.26	5.78	1.37	6.13	1.65	6.41	1.85
25	Port 2 PS	3.93	-0.41	4.02	-0.34	4.62	0.22	5.79	1.28	6.14	1.52	6.41	1.65
26	Newmarket PS	3.42	-1.07	3.79	-0.86	4.82	-0.06	6.16	0.95	6.72	1.18	7.14	1.13
27	Newmarket PS	4.90	-0.20	5.29	-0.17	5.84	-0.01	6.25	0.04	6.72	0.26	7.14	0.43

1. The column labeled "Difference from without-project condition" shows the difference between the with-project minus the without-project.
2. A positive (+) means the project increases the wsel where a negative (-) means the project decreases the wsel.

Denotes a > 0.5 ft. increase induced by the project.

Table 4.5.7 Future With-Project (Gates Closed/USACE Pump Station Alt. 2) -2082

Selected Output Locations	Nearest Drainage Feature Influence (City PS/USACE PS)	FW 2082 (gates closed) City Pumps Active/USACE Pump Station Alternative 2											
		50% AEP		20% AEP		10% AEP		4% AEP		2% AEP		1% AEP	
		Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)
1	NA	7.33	3.04	7.89	3.59	8.08	3.76	8.31	3.97	8.46	4.10	8.68	4.29
2	Halsey PS	2.79	-1.59	3.13	-1.25	3.78	-0.60	6.22	1.84	6.87	2.48	7.48	3.09
3	Halsey PS	3.47	-0.82	4.19	-0.12	5.37	1.04	6.28	1.91	6.85	2.45	7.48	3.03
4	Halsey PS	5.90	-0.12	6.18	-0.06	6.40	-0.03	6.61	-0.02	6.76	0.00	7.48	0.58
5	Halsey PS	6.47	0.26	6.61	0.31	6.73	0.35	6.86	0.40	6.91	0.40	7.48	0.92
6	Joe Riley PS	3.37	-0.92	3.43	-0.87	4.24	-0.07	5.73	1.39	6.10	1.74	6.38	1.99
7	Joe Riley/SF PS	5.47	0.69	5.60	0.55	5.74	0.39	5.90	0.28	6.11	0.33	6.38	0.45
8	SF PS	5.47	0.97	5.60	0.84	5.74	0.62	5.90	0.49	6.11	0.54	6.36	0.63
9	MUSC PS	4.40	0.10	4.45	-0.02	4.51	-0.02	4.59	-0.03	4.77	0.06	4.99	0.16
10	City Marina PS	3.43	-1.39	4.25	-0.73	4.83	-0.27	5.20	-0.03	5.40	0.09	5.60	0.21
11	City Marina PS	4.72	-0.16	4.91	-0.17	5.11	-0.15	5.34	-0.13	5.48	-0.12	5.67	-0.06
12	Potential Temp PS	4.43	0.13	4.65	0.32	4.84	0.49	5.15	0.74	5.37	0.90	5.63	1.09
13	Battery 1 PS	4.39	-0.44	4.55	-0.49	4.70	-0.49	5.09	-0.27	5.37	-0.10	5.63	0.02
14	Battery 1 PS	4.40	-0.43	4.56	-0.48	4.72	-0.47	5.10	-0.26	5.37	-0.10	5.63	0.02
15	Battery 2 PS	4.60	-0.51	4.66	-0.64	4.90	-0.57	5.40	-0.27	5.60	-0.19	5.78	-0.14
16	Battery 3 PS	5.70	-0.48	6.07	-0.30	6.35	-0.23	6.62	-0.19	6.80	-0.17	6.99	-0.16
17	Waterfront Park PS	4.61	-1.57	4.65	-1.60	4.67	-1.67	6.46	-0.02	6.95	0.40	7.20	0.58
18	Waterfront Park PS	6.25	0.24	6.43	0.23	6.62	0.24	6.84	0.24	7.02	0.25	7.24	0.27
19	Concord St. PS	5.05	0.09	5.20	0.12	5.35	0.12	5.54	0.15	5.67	0.16	5.82	0.16
20	Concord St. PS	5.55	0.00	5.73	0.01	5.88	0.00	6.06	0.00	6.18	0.00	6.32	0.00
21	Concord St. PS	4.74	0.17	4.78	0.08	4.92	0.08	5.09	0.12	5.22	0.02	5.42	-0.07
22	Port 1 PS	6.09	-0.05	6.13	-0.04	6.15	-0.04	6.16	-0.08	6.19	-0.09	6.27	-0.05
23	Port 2 PS	5.43	-0.02	5.55	-0.01	5.68	0.01	5.84	-0.05	6.06	0.03	6.31	0.14
24	Port 2 PS	3.34	-0.98	3.40	-0.93	3.87	-0.48	5.35	0.94	5.95	1.47	6.28	1.72
25	Port 2 PS	3.97	-0.37	4.07	-0.29	4.16	-0.24	5.35	0.84	5.95	1.33	6.28	1.52
26	Newmarket PS	3.40	-1.09	3.49	-1.16	4.13	-0.75	5.62	0.41	6.40	0.86	6.92	0.91
27	Newmarket PS	4.90	-0.20	5.29	-0.17	5.83	-0.02	6.22	0.01	6.46	0.00	6.93	0.22

1. The column labeled "Difference from without-project condition" shows the difference between the with-project minus the without-project.
2. A positive (+) means the project increases the wsel where a negative (-) means the project decreases the wsel.

Denotes a > 0.5 ft. increase induced by the project.

Table 4.5.8 Future With-Project (Gates Closed/USACE Pump Station Alt. 3) -2082

Selected Output Locations	Nearest Drainage Feature Influence (City PS/USACE PS)	FW 2082 (gates closed) City Pumps Active/USACE Pump Station Alternative 3											
		50% AEP		20% AEP		10% AEP		4% AEP		2% AEP		1% AEP	
		Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)
1	NA	7.33	3.04	7.89	3.59	8.07	3.75	8.30	3.96	8.46	4.10	8.68	4.29
2	Halsey PS	2.94	-1.44	3.29	-1.09	3.91	-0.47	5.36	0.98	6.50	2.11	7.16	2.77
3	Halsey PS	3.38	-0.91	3.68	-0.63	4.64	0.31	6.02	1.65	6.48	2.08	7.15	2.70
4	Halsey PS	5.90	-0.12	6.18	-0.06	6.40	-0.03	6.62	-0.01	6.75	-0.01	7.16	0.26
5	Halsey PS	6.41	0.20	6.55	0.25	6.68	0.30	6.81	0.35	6.88	0.37	7.16	0.60
6	Joe Riley PS	3.36	-0.93	3.39	-0.91	3.52	-0.79	4.71	0.37	5.75	1.39	6.22	1.83
7	Joe Riley/SF PS	5.47	0.69	5.60	0.55	5.74	0.39	5.90	0.28	6.02	0.24	6.24	0.31
8	SF PS	5.47	0.97	5.60	0.84	5.74	0.62	5.90	0.49	6.02	0.45	6.26	0.53
9	MUSC PS	4.40	0.10	4.45	-0.02	4.51	-0.02	4.59	-0.03	4.71	0.00	4.90	0.07
10	City Marina PS	3.38	-1.44	3.49	-1.49	4.31	-0.79	4.99	-0.24	5.26	-0.05	5.49	0.10
11	City Marina PS	4.71	-0.17	4.91	-0.17	5.11	-0.15	5.33	-0.14	5.47	-0.13	5.63	-0.10
12	Potential Temp PS	4.43	0.13	4.65	0.32	4.80	0.45	4.98	0.57	5.19	0.72	5.42	0.88
13	Battery 1 PS	4.38	-0.45	4.53	-0.51	4.68	-0.51	4.85	-0.51	5.06	-0.41	5.39	-0.22
14	Battery 1 PS	4.40	-0.43	4.55	-0.49	4.71	-0.48	4.89	-0.47	5.08	-0.39	5.40	-0.21
15	Battery 2 PS	4.57	-0.54	4.62	-0.68	4.69	-0.78	4.92	-0.75	5.35	-0.44	5.62	-0.30
16	Battery 3 PS	4.90	-1.28	5.67	-0.70	6.10	-0.48	6.44	-0.37	6.64	-0.33	6.85	-0.30
17	Waterfront Park PS	4.64	-1.54	4.65	-1.60	4.68	-1.66	4.75	-1.73	5.20	-1.35	6.91	0.29
18	Waterfront Park PS	6.25	0.24	6.43	0.23	6.62	0.24	6.84	0.24	7.01	0.24	7.22	0.25
19	Concord St. PS	5.05	0.09	5.20	0.12	5.35	0.12	5.54	0.15	5.67	0.16	5.82	0.16
20	Concord St. PS	5.56	0.01	5.73	0.01	5.89	0.01	6.07	0.01	6.20	0.02	6.36	0.04
21	Concord St. PS	4.45	-0.12	4.58	-0.12	4.72	-0.12	4.91	-0.06	5.02	-0.18	5.36	-0.13
22	Port 1 PS	6.10	-0.04	6.14	-0.03	6.14	-0.05	6.15	-0.09	6.16	-0.12	6.20	-0.12
23	Port 2 PS	5.43	-0.02	5.55	-0.01	5.68	0.01	5.87	-0.02	5.99	-0.04	6.19	0.02
24	Port 2 PS	3.36	-0.96	3.36	-0.97	3.36	-0.99	4.14	-0.27	5.29	0.81	6.01	1.45
25	Port 2 PS	3.97	-0.37	4.06	-0.30	4.16	-0.24	4.25	-0.26	5.30	0.68	6.01	1.25
26	Newmarket PS	3.38	-1.11	3.43	-1.22	3.57	-1.31	4.51	-0.70	5.64	0.10	6.48	0.47
27	Newmarket PS	4.90	-0.20	5.29	-0.17	5.83	-0.02	6.20	-0.01	6.44	-0.02	6.69	-0.02

1. The column labeled "Difference from without-project condition" shows the difference between the with-project minus the without-project.
2. A positive (+) means the project increases the wsel where a negative (-) means the project decreases the wsel.

Denotes a > 0.5 ft. increase induced by the project.

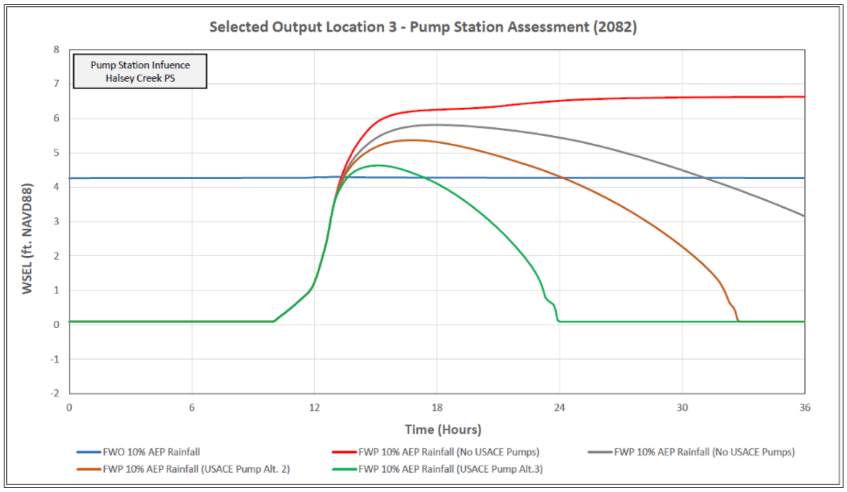

Figure 4.5.21 Pump Station Assessment – Output Location 3 – 2082

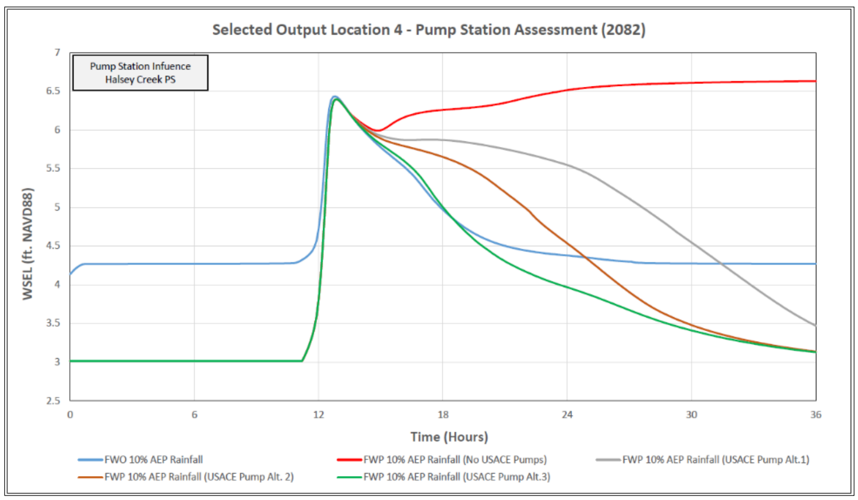

Figure 4.5.22 Pump Station Assessment – Output Location 4 – 2082

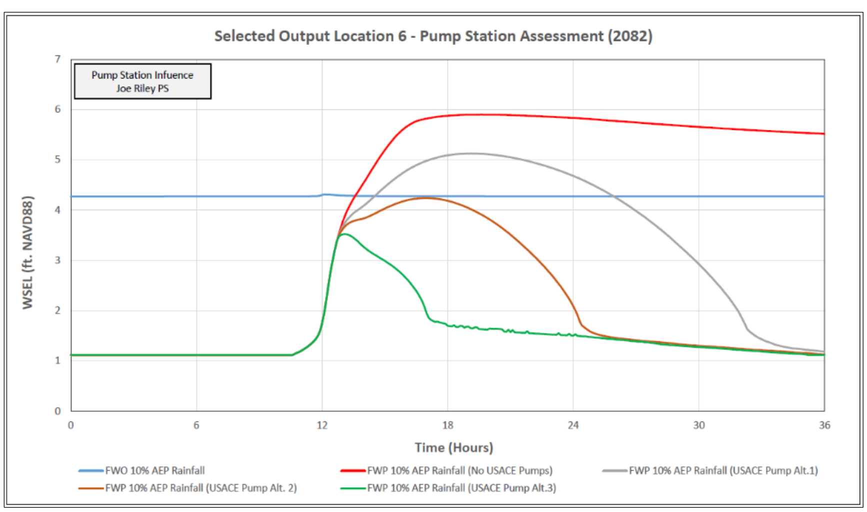

Figure 4.5.23 Pump Station Assessment – Output Location 6 – 2082

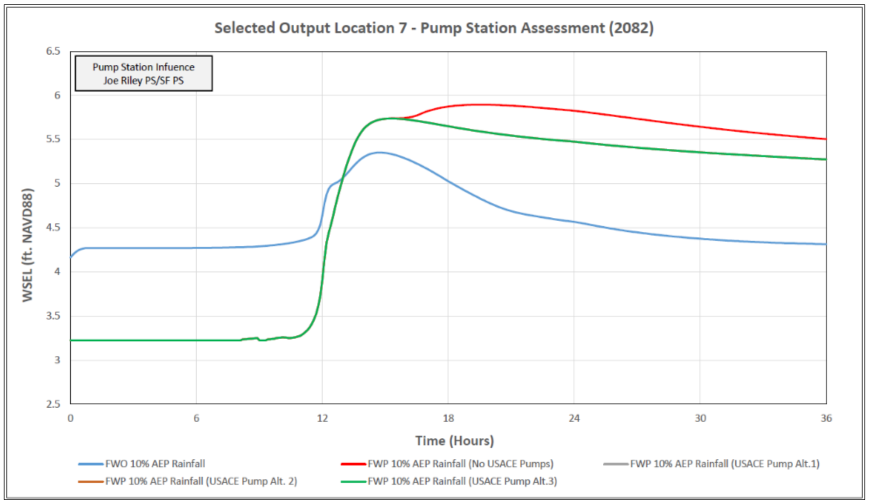

Figure 4.5.24 Pump Station Assessment – Output Location 7 – 2082

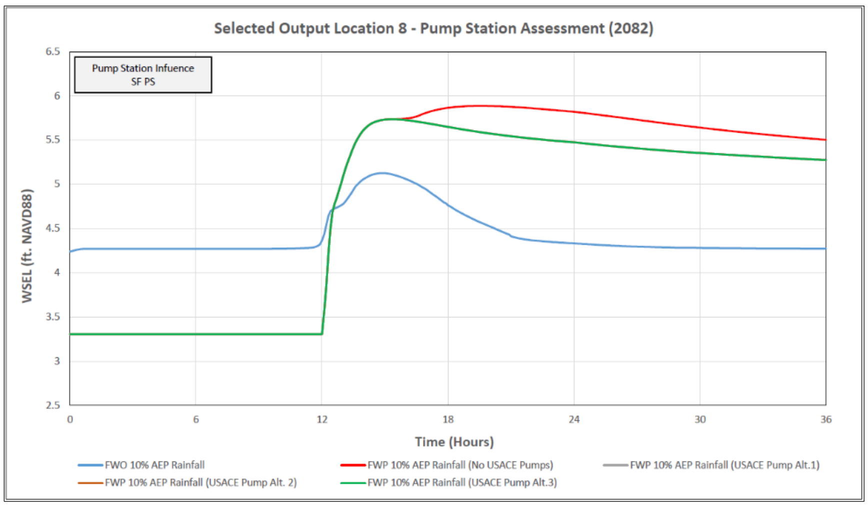

Figure 4.5.25 Pump Station Assessment – Output Location 8 – 2082

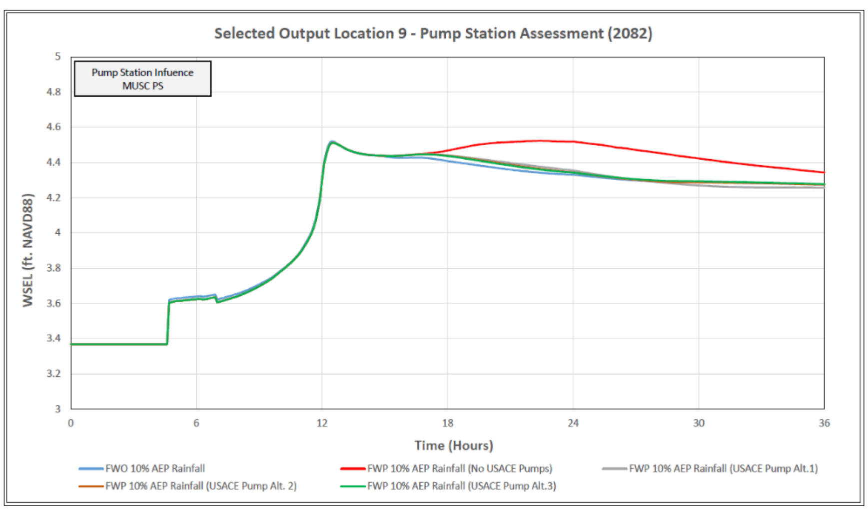

Figure 4.5.26 Pump Station Assessment – Output Location 9 – 2082

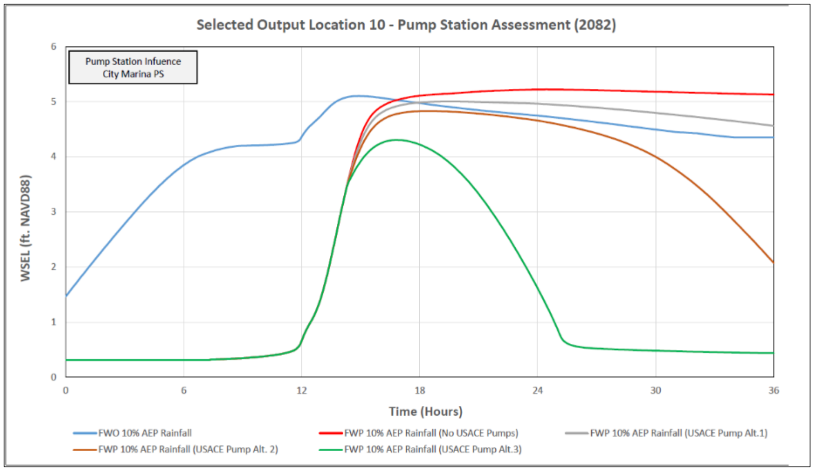

Figure 4.5.27 Pump Station Assessment – Output Location 10 – 2082

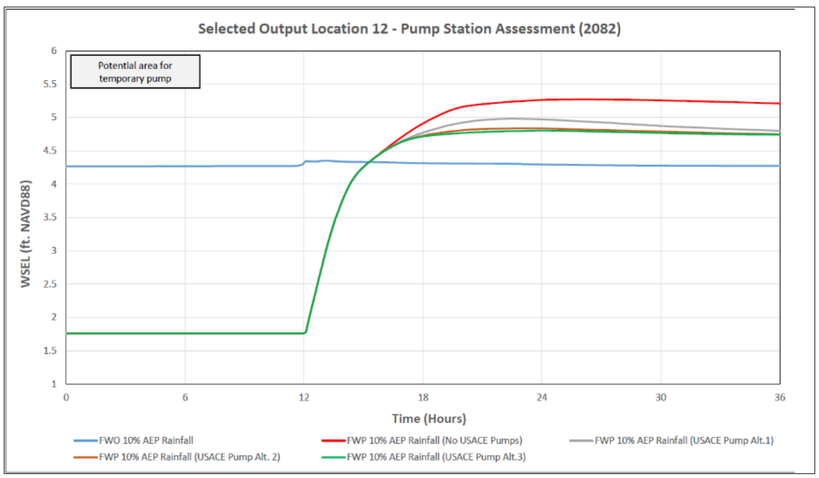

Figure 4.5.28 Pump Station Assessment – Output Location 12 – 2032

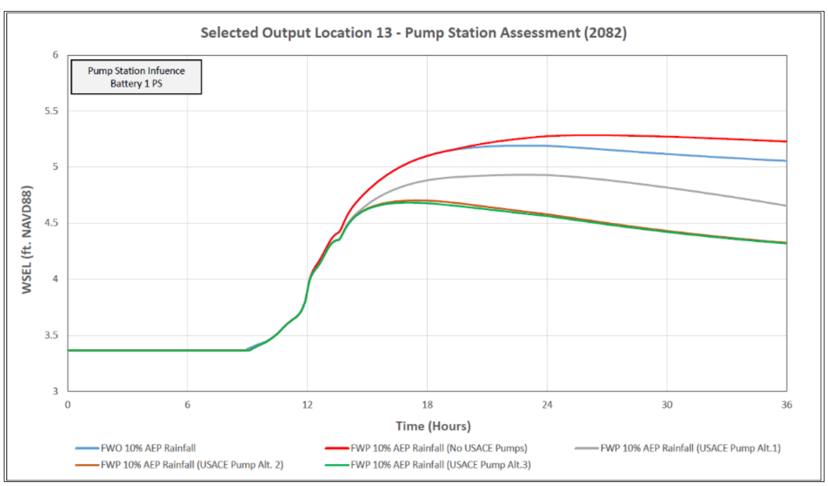

Figure 4.5.29 Pump Station Assessment – Output Location 13 - 2082

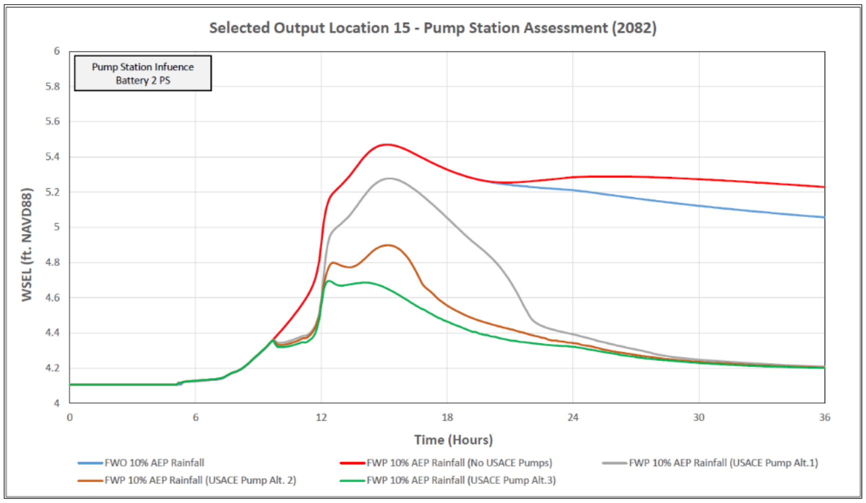

Figure 4.5.30 Pump Station Assessment – Output Location 15 – 2082

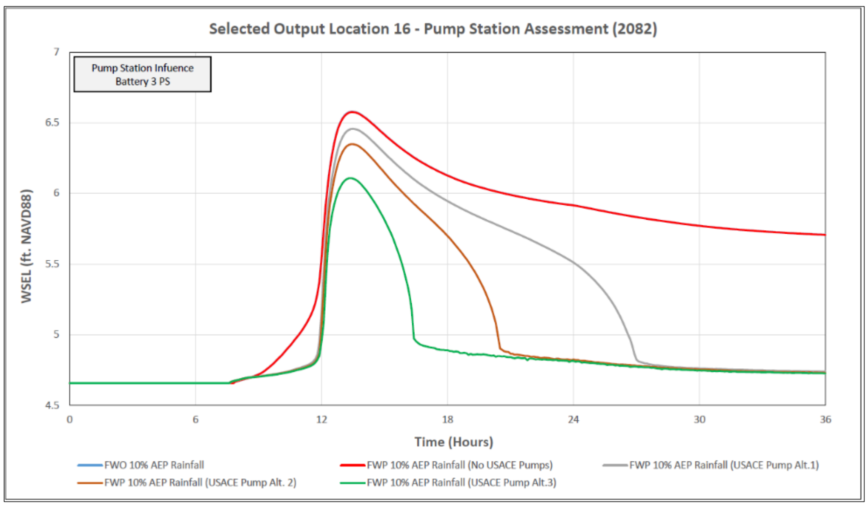

Figure 4.5.31 Pump Station Assessment – Output Location 16 – 2082

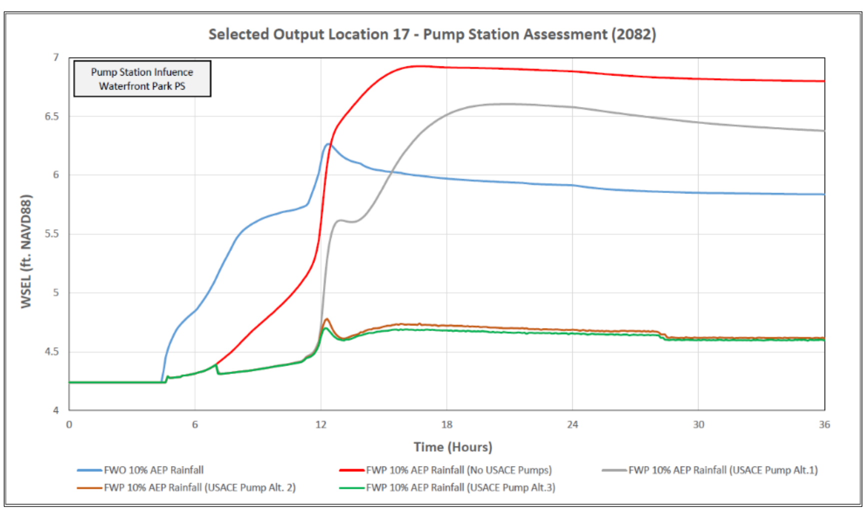

Figure 4.5.32 Pump Station Assessment – Output Location 17 – 2082

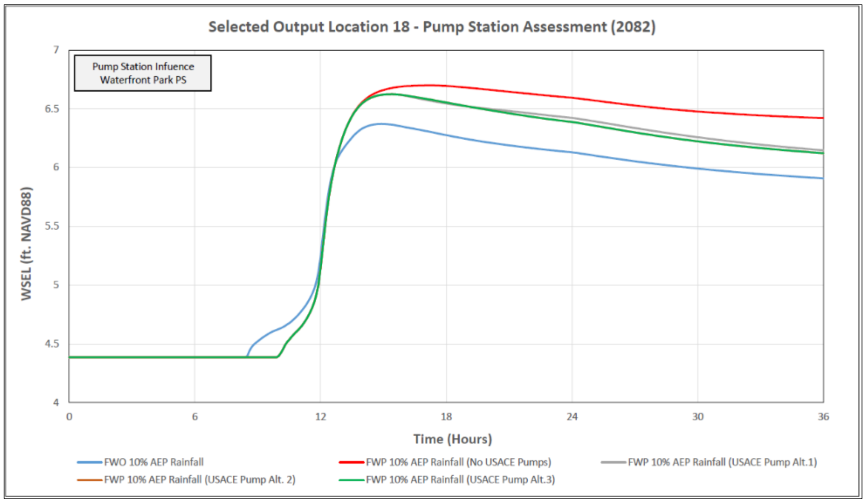

Figure 4.5.33 Pump Station Assessment – Output Location 18 - 2082

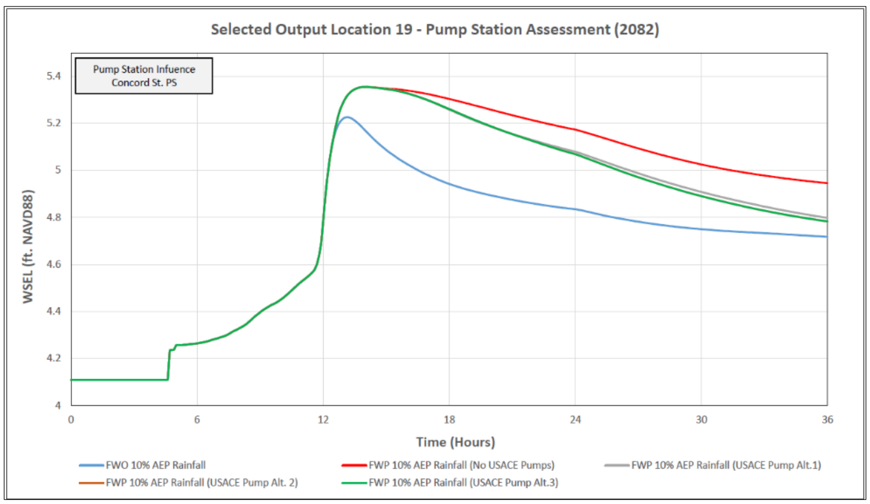

Figure 4.5.34 Pump Station Assessment – Output Location 19 – 2082

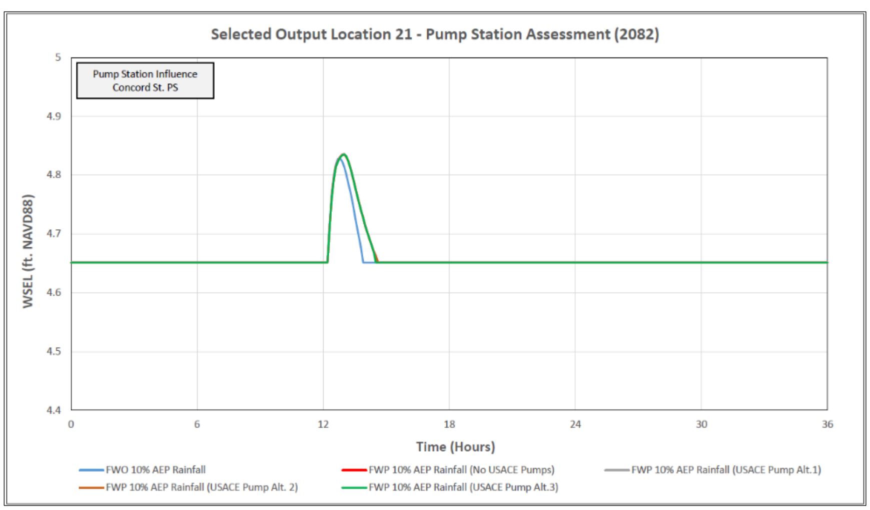

Figure 4.5.35 Pump Station Assessment – Output Location 21 – 2082

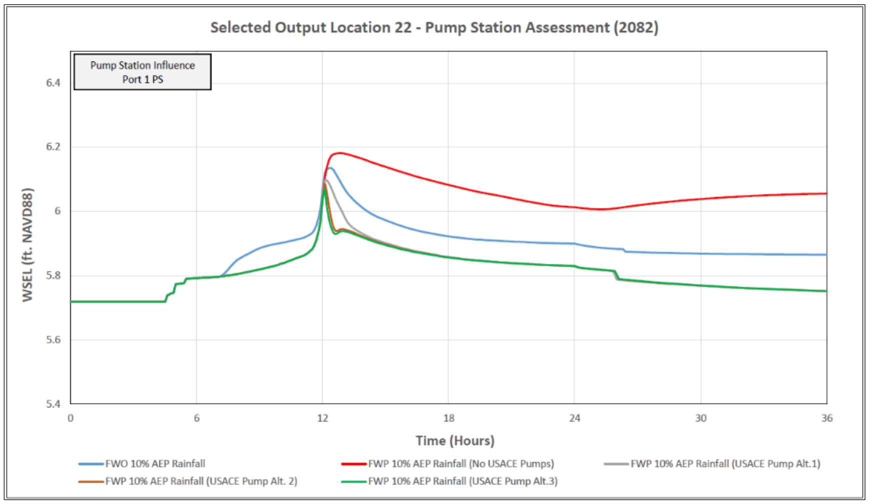

Figure 4.5.36 Pump Station Assessment – Output Location 22 – 2082

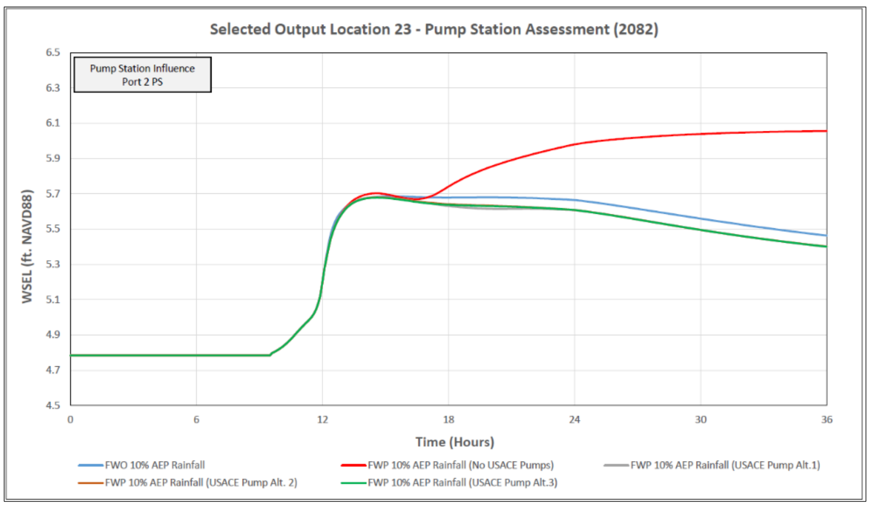

Figure 4.5.37 Pump Station Assessment – Output Location 23 – 2082

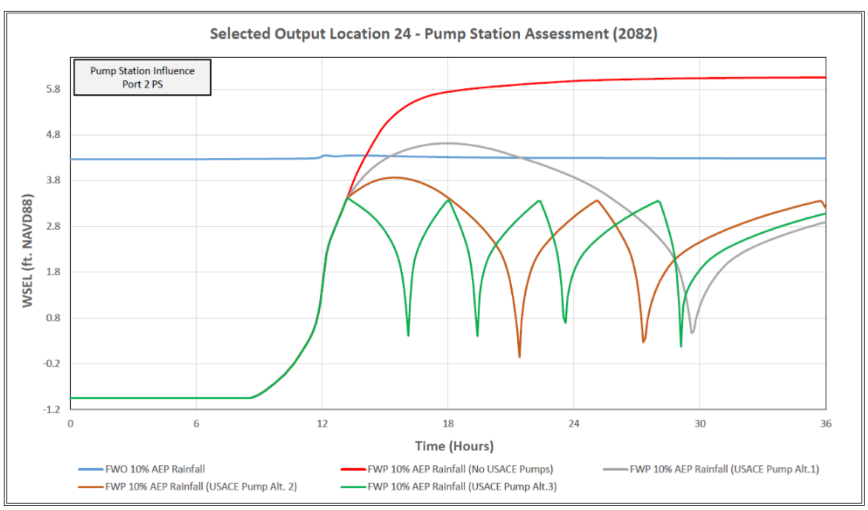

Figure 4.5.38 Pump Station Assessment – Output Location 24 – 2082

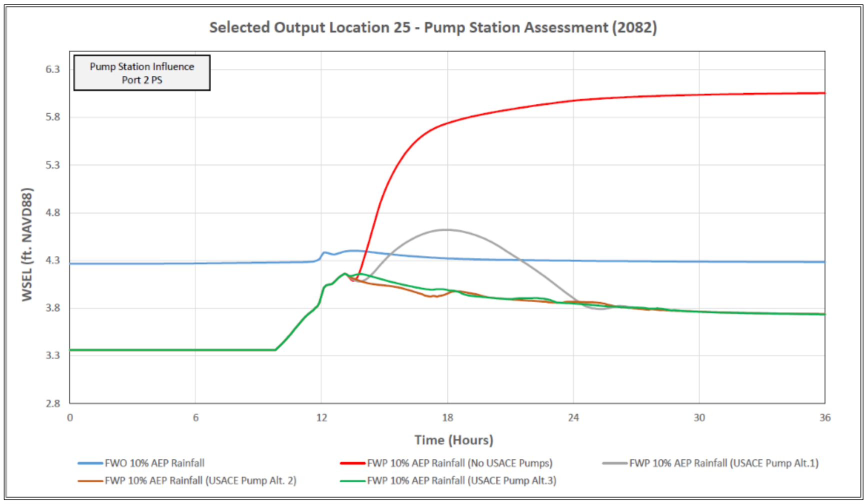

Figure 4.5.39 Pump Station Assessment – Output Location 25 - 2082

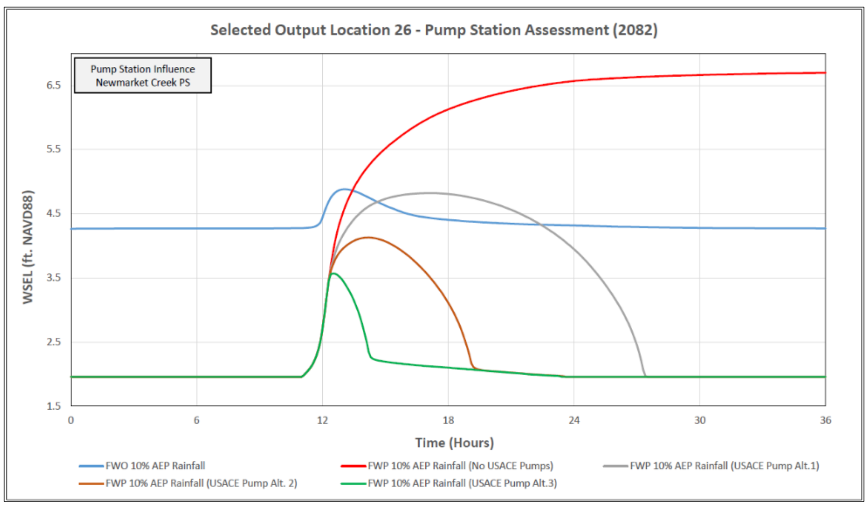

Figure 4.5.40 Pump Station Assessment – Output Location 26 - 2082

4.6 SUMMARY OF HYDRAULIC MODEL RESULTS

The TSP must include the interior drainage facilities such that no induced flood damages occurs during low exterior stages (gravity conditions). In addition, other alternatives or enhancements can be assessed that could further improve interior drainage relief. Such instances relevant to the project study area could include the increased capacity of existing gravity outlets, increased pump station capacities, and increased stormwater pipe capacities. Other instances such as stormwater detentions are not realistically an option for the downtown area. During PED phase, a better understanding of the City's completed and planned stormwater improvements is necessary for appropriately designing the system along with more information regarding the existing storm pipe network which is assumed to be utilized for the USACE proposed pump stations.

As mentioned, the City's existing storm pipe network accommodates no more than a 10% AEP rainfall event meaning the system becomes overwhelmed for events of approximately and greater than this frequency. Surface flow becomes a major component of drainage during such instances. In some areas, the City indicates even lower capacities of stormwater routing. The interior drainage assessment signifies the performance of the system for the events of 50% AEP up to the 4% AEP due to the current capacities of the storm pipe network which brings stormwater to the pump stations.

Review of the future with-project storm gates open assessment displays the project produces similar water surface elevations as compared to the future without-project as most locations for the 50% and 20% AEP rain events for the years 2032 and 2082. The 10% AEP event displays that five locations incur increases greater than 0.5 feet for the year 2032 and two locations for the year 2082. The 4% AEP event displays that five locations incur increases greater than 0.5 feet for the year 2032 and four locations for the year 2082. The 2% AEP event displays that five locations incur increases greater than 0.5 feet for the year 2032 and five locations for the year 2082. The 1% AEP event displays that six locations incur increases greater than 0.5 feet for the year 2032 and five locations for the year 2082.

Review of the future with-project storm gates closed (pump station) assessment displays the project significantly increases interior water surface elevations at many locations for the condition with No USACE pump stations. The pump station alternatives display an increase of greater than 0.5 feet for 4-5 locations for the 50%, 20%, and 10% AEP events for the year 2032. The pump station alternatives display an increase of greater than 0.5 feet for 2-3 locations for the 50%, 20%, and 10% AEP events for the year 2082. The pump station alternatives display several locations with a greater than 0.5 feet increase for the events greater than the 10% AEP for both the years 2032 and 2082.

The pump station assessment also displays various locations with decreased elevations as a benefit of the project due to the improvement of drainage efficiency via pumping rather gravity flow and due to the storm surge wall prohibiting damages that may occur during MHHW tide elevations. As noted, the RAS model does not have the capability of modeling sub-surface drainage potentially computing higher than expected inland water surface elevations however for both without- and with-project conditions.
Many of the output locations that incur the noticeably increased water surface elevations, for storm gates open and storm gates closed, are at low-lying areas where increased elevations may not increase flood damages. For example, the Halsey Creek tidal area is shown to experience increased elevations within the banks of the creek but may not experience damage inducing elevations on land if those increased elevations are not producing out of bank inundations. In addition to low-lying creek areas, there are potentially low-lying isolated depression areas that could incur increased elevations as result of the storm surge wall.

At this phase of the study, the closed system conditions have been assessed holistically, i.e., the pump station alternative capacities have not been mixed and matched per location. In PED phase, the pump station alternatives can be mixed and matched for the site-specific analysis to appropriately accommodate the needed interior drainage relief per servicing area or watershed area. The site-specific assessment will assist in appropriately designing each storm gate/pump station for the proposed interior drainage system not to inflate the project cost due to over-design and not to under-design portions of the system that could induce significant interior residual damages.

While the RAS results display the differences in interior water surface elevations for with- versus without-project conditions, the HEC-FDA model provides the tools for describing the economic consequences and/or benefits for the differences in interior water surface elevations. The Economics assessment is provided in Chapter 5.

CHAPTER 5 – ECONOMIC ASSESSMENT FOR INTERIOR DRAINAGE

5.1 HEC-FDA METHODOLOGY

The HEC-RAS results for future without- and with-project conditions were incorporated into the Hydrologic Engineering Center's (CEIWR-HEC) Flood Damage Reduction Analysis (HEC-FDA) to compute the Equivalent Annual Damages (EAD) and Average Annual Damages (AAD) to describe the risk of interior residual flooding. The HEC-FDA assessment served as an economic tool to determine the drainage features and their capacities (storm gates/pump stations) necessary for implementation into the project alignment.

The project alignment has undergone "re-alignment" in areas near the Port of Charleston since the FDA effort for interior drainage was completed. Some pump stations have been relocated to accommodate this project re-alignment along with the incorporation of a storm gate at the creek the Port of Charleston. Other changes that have been made to the TSP, since the FDA effort was completed, include the reduction of the number of storm gates in the Wagener Terrace area. The referenced project modifications have been incorporated into the RAS model. The RAS results presented in previous sections include those modifications, however, the FDA model has not since been updated. The FDA model reflects the results from the previous TSP prior to the mentioned modifications. The following figures display the layout of the previous TSP (reflected in FDA results) and the current "Final Feasibility" TSP (reflected in RAS results but not yet in FDA results).

The FDA model computed damages to structures on a per Model Area (MA) basis. Figure _ shows the delineation of the FDA model areas. There are five model areas delineated for Economics: Model Area 1 (Battery), Model Area 2 (Port), Model Area 3 (Newmarket), Model Area 4 (Marina), and Model Area 5 (Wagener Terrace). To compute FDA, the software requires "index locations" with hydrologic data such as stage-discharge functions from the RAS model. For each MA, one index location was selected, therefore five total index locations. The RAS model was used to derive stage-discharge and discharge-frequency functions for each index location for each modeled scenario as presented in Section 4.1. For example, the RAS model for the future with-project pump station alternative 2 scenario was used to extract the stage and flow at an index location for the 50% AEP, then for the 20% AEP, then for the 10% AEP, and so on. For each AEP, the stage and associated flow was tabulated to develop the stage-discharge and discharge-frequency functions. Also derived are the stage-damage functions.

Once the FDA model is setup, the damages (number of structures and associated dollar amounts) were computed for all without-project storm frequencies and all with-project storm frequencies and project alternatives for the years 2032 and 2082. Incremental damages are then calculated by comparing a future with-project alternative to the future without-project alternative for the same storm frequency. Incremental damages can be assessed for an entire project alternative and can be further assessed by reviewing the incremental damages occurring at a specific model area.

In addition to incremental damages for number of structures and associated dollar amounts, the storm frequencies, incremental probabilities, and stage-damage functions are used to compute the average annual damages. The average annual damages of the future without-project is compared to a future with-project alternative for determining the "Damage Reductions" and whether the project alternative induces or reduces the average annual damages.

HEC-RAS detailed water surface elevation grids and inundation boundary polygons were provided to the economics team member. A structural damage assessment was completed using a structures inventory layer, RAS water surface grids, and GIS tools to assess the structures getting "wet" for each scenario.

"Heatmaps" are developed to present the inundated structures while attributing a range of dollar amounts associated to the inundated structures in the form of a color ramp with values and symbols.

5.2 EVOLUTION OF PROPOSED PROJECT ALIGNMENT

This section provides maps to show the evolution of the project alignment and drainage features, particularly after the completion of the FDA modeling. As mentioned, the wall was re-aligned near the Port of Charleston which required modifications to the locations of pump stations in the location while also incorporating a new storm gate at the creek near the Port. Other updates subsequent the FDA effort was completed are the removal of several storm gates in the Wagener Terrace area and the removal of a storm gate the Coast Guard office.

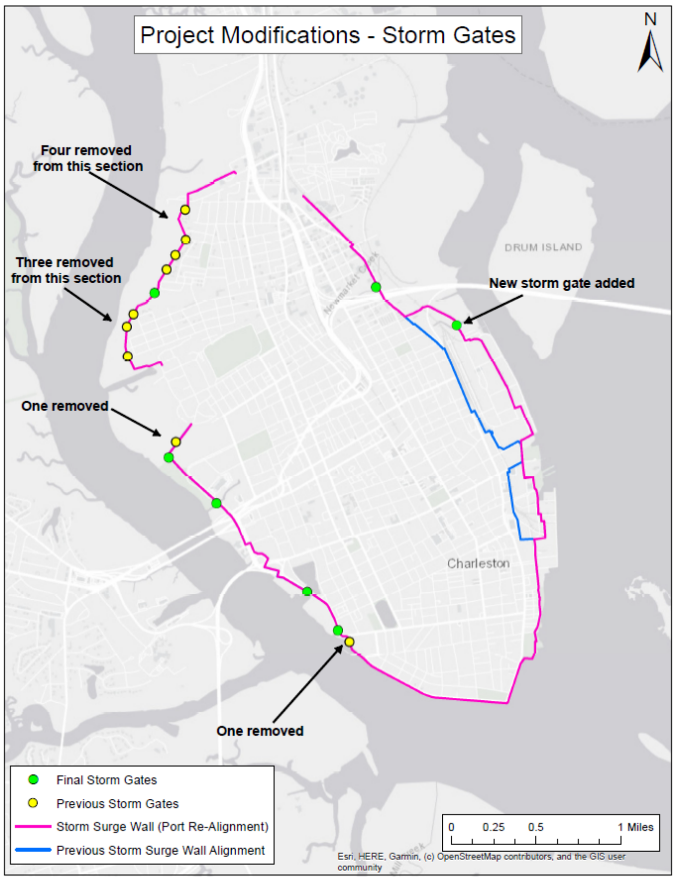

Figure 5.2.1 TSP Project Modification – Storm Gates

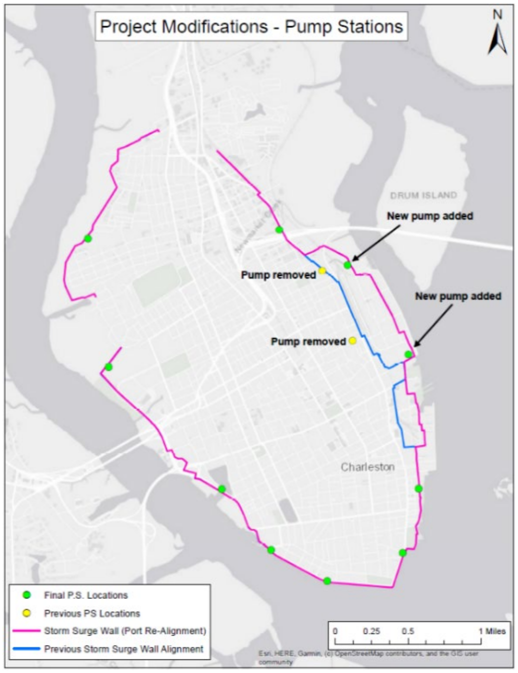

Figure 5.2.2 TSP Project Modification – Pump Stations

Figure 5.2.3 Previous TSP

Figure 5.2.4 Final "Feasibility" TSP

Figure 5.2.5 displays the five Model Areas used for the economics assessment and the RAS and FDA index locations used to describe the hydrologic data for each model area.

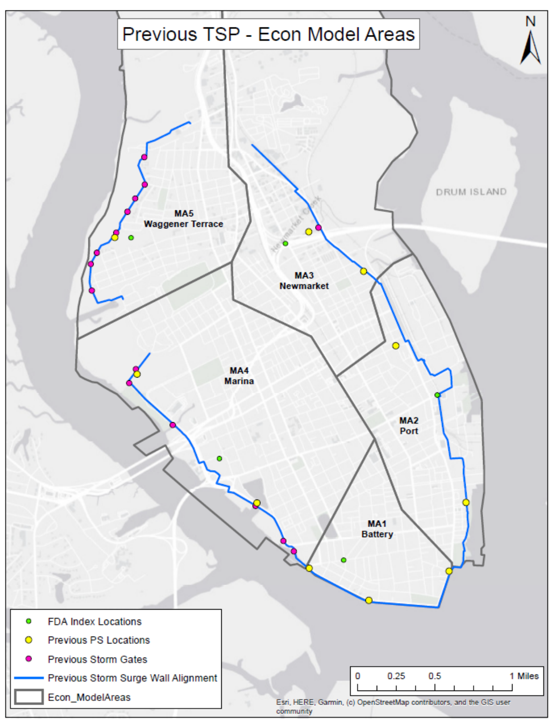

Figure 5.2.5 Previous TSP – ECON Model Areas

5.3 HEC-FDA EVALUATION

This section presents the results of the HEC-FDA model as it was computed using the previously selected TSP. Although not updated for current project alignment considerations, the economics assessment is beneficial in describing the performance of the project alternatives.

As mentioned, the RAS model computed gates open conditions (storm gates open/USACE pumps inactive/City pumps active) and gates closed conditions (storm gates closed/Usace pumps active/City pumps active) for the years 2032 and 2082. The future without-project conditions used a constant tide boundary condition of 3.18 feet for 2032 and 4.27 feet for 2082 NAVD88 for all rainfall frequencies. The future with- gates open also used the same tide boundary conditions for rainfall frequencies while the tide boundary condition for the gates closed conditions is ultimately insignificant (unless overtopping) for this assessment primarily because the RAS model assumes pump efficiency to be constant throughout a storm. In reality, pump station operations are more complex because the head differential throughout a storm would not be constant and therefore pump efficiency could fluctuate with tide/storm surge. Comparing the pump station conditions to future without-project conditions at low exterior stages (gravity conditions) allows for the proper sizing of pumps for the perspective of assessing the potential interior "ponding" effect.

<u>Recap of HEC-RAS Simulation Matrix:</u>

- 2032. Each event utilizes a stage boundary condition of the projected 2032 Mean Higher-High Water (MHHW) or high tide at 3.18 feet NAVD88. The matrices below is for comparing Future Without (FWO) versus Future With (FW) for each condition and alternative plan.

Comparison matrix 2032	
FWO	
Exterior Elev	Interior rainfall
high tide	50% AEP rain
high tide	20% AEP rain
high tide	10% AEP rain
high tide	4 % AEP rain
high tide	2% AEP rain
high tide	1 % AEP rain

←→

Comparison matrix 2032									
FW (storm gates open)		FW (storm gates closed)		FW (storm gates closed) P.S. alt 1		FW (storm gates closed) P.S. alt 2		FW (storm gates closed) P.S. alt 3	
Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall
high tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain
high tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain
high tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain
high tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain
high tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain
high tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain

- 2082. Each event utilizes a stage boundary condition of the projected 2032 Mean Higher-High Water (MHHW) or high tide at 3.18 feet NAVD88. The matrices below is for comparing Future Without (FWO) versus Future With (FW) for each condition and alternative plan.

Comparison matrix 2082	
FWO	
Exterior Elev	Interior rainfall
high tide	50% AEP rain
high tide	20% AEP rain
high tide	10% AEP rain
high tide	4 % AEP rain
high tide	2% AEP rain
high tide	1 % AEP rain

←→

Comparison matrix 2082									
FW (storm gates open)		FW (storm gates closed)		FW (storm gates closed) P.S. alt 1		FW (storm gates closed) P.S. alt 2		FW (storm gates closed) P.S. alt 3	
Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall
high tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain	High tide	50% AEP rain
high tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain	High tide	20% AEP rain
high tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain	High tide	10% AEP rain
high tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain	High tide	4 % AEP rain
high tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain	High tide	2% AEP rain
high tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain	High tide	1 % AEP rain

5.3.1 INCREMENTAL DAMAGES PER SCENARIO

This section provides the results for incremental damages of the future without-project conditions minus each future with-project condition and alternative. Each table displays the future without- condition paired to a future with- condition and the incremental results.

- Column labeled "Struct_Number". A negative value for the incremental number of structures damaged indicates the project increased the number of structures inundated. A positive value indicates the project decreased the number of structures inundated and could be said to provide net benefits.

- Column labeled "Struct_Damages". A parenthesis for the incremental dollar damages indicates the project increased the dollar damages. A positive value (no parenthesis) indicates the project reduced the dollar damages and could be said to provide net benefits.

Table 5.3.1 Incremental Damages (FWO vs FW Gates Open)

2032 High Tide - FWO vs FW (Gates Open - City PS Active - USACE PS Inactive) Incremental Damages			
FWO 2032		FW 2032 (gates open)	
Exterior Elevation	Interior rainfall	Struct_Number	Struct_Damages
high tide	50% AEP rain	1	$ 86,463
high tide	20% AEP rain	-2	$ (43,401)
high tide	10% AEP rain	-6	$ (233,173)
high tide	4% AEP rain	-9	$ (372,966)
high tide	2% AEP rain	-10	$ (455,640)
high tide	1% AEP rain	-10	$ (558,847)
1. "Struct_Number" A negative value means the project increased the number of structures getting wet. A positive value means the project decreased the number of structures getting wet.			
2. "Struct_Damages" A parenthesis () means the project increased the dollar amount of damages where no paranthesis means the project decreased the dollar amount of damages.			

2082 High Tide - FWO vs FW (Gates Open - City PS Active - USACE PS Inactive) Incremental Damages			
FWO 2082		FW 2082 (gates open)	
Exterior Elevation	Interior rainfall	Struct_Number	Struct_Damages
high tide	50% AEP rain	0	$ 77,041
high tide	20% AEP rain	-3	$ (55,483)
high tide	10% AEP rain	-8	$ (246,888)
high tide	4% AEP rain	-9	$ (392,514)
high tide	2% AEP rain	-8	$ (472,799)
high tide	1% AEP rain	-12	$ (621,252)
1. "Struct_Number" A negative value means the project increased the number of structures getting wet. A positive value means the project decreased the number of structures getting wet.			
2. "Struct_Damages" A parenthesis () means the project increased the dollar amount of damages where no paranthesis means the project decreased the dollar amount of damages.			

Table 5.3.2 Incremental Damages (FWO vs FW Gates Closed – No USACE PS)

2032 High Tide - FWO vs FW (Gates Open - City PS Active - USACE PS Inactive) Incremental Damages			
FWO 2032		FW 2032 (gates closed)	
Exterior Elevation	Interior rainfall	Struct_Number	Struct_Damages
high tide	50% AEP rain	-33	$ (1,278,481)
high tide	20% AEP rain	-50	$ (1,974,045)
high tide	10% AEP rain	-82	$ (3,341,110)
high tide	4% AEP rain	-144	$ (6,546,630)
high tide	2% AEP rain	-180	$ (9,285,817)
high tide	1% AEP rain	-231	$ (12,609,938)
1. "Struct_Number" A negative value means the project increased the number of structures getting wet. A positive value means the project decreased the number of structures getting wet.			
2. "Struct_Damages" A parenthesis () means the project increased the dollar amount of damages where no paranthesis means the project decreased the dollar amount of damages.			

2082 High Tide - FWO vs FW (Gates Open - City PS Active - USACE PS Inactive) Incremental Damages			
FWO 2082		FW 2082 (gates closed)	
Exterior Elevation	Interior rainfall	Struct_Number	Struct_Damages
high tide	50% AEP rain	-25	$ (976,743)
high tide	20% AEP rain	-43	$ (1,616,000)
high tide	10% AEP rain	-75	$ (2,999,489)
high tide	4% AEP rain	-138	$ (6,212,768)
high tide	2% AEP rain	-173	$ (8,945,795)
high tide	1% AEP rain	-230	$ (12,312,352)
1. "Struct_Number" A negative value means the project increased the number of structures getting wet. A positive value means the project decreased the number of structures getting wet.			
2. "Struct_Damages" A parenthesis () means the project increased the dollar amount of damages where no paranthesis means the project decreased the dollar amount of damages.			

Table 5.3.3 Incremental Damages (FWO vs FW Gates Closed – USACE PS Alt. 1)

2032 High Tide - FWO vs FW (Gates Open - City PS Active - USACE PS Alt1) Incremental Damages			
FWO 2032		FW 2032 (gates closed) P.S. Alt1	
Exterior Elevation	Interior rainfall	Struct_Number	Struct_Damages
high tide	50% AEP rain	37	$ 1,033,798
high tide	20% AEP rain	38	$ 1,236,583
high tide	10% AEP rain	2	$ 419,393
high tide	4% AEP rain	-40	$ (1,613,416)
high tide	2% AEP rain	-81	$ (3,712,456)
high tide	1% AEP rain	-128	$ (6,710,736)
1. "Struct_Number" A negative value means the project increased the number of structures getting wet. A positive value means the project decreased the number of structures getting wet.			
2. "Struct_Damages" A parenthesis () means the project increased the dollar amount of damages where no paranthesis means the project decreased the dollar amount of damages.			

2082 High Tide - FWO vs FW (Gates Open - City PS Active - USACE PS Alt1) Incremental Damages			
FWO 2082		FW 2082 (gates closed) P.S. Alt1	
Exterior Elevation	Interior rainfall	Struct_Number	Struct_Damages
high tide	50% AEP rain	43	$ 1,133,472
high tide	20% AEP rain	44	$ 1,395,504
high tide	10% AEP rain	8	$ 567,847
high tide	4% AEP rain	-35	$ (1,467,129)
high tide	2% AEP rain	-75	$ (3,558,902)
high tide	1% AEP rain	-128	$ (6,601,975)
1. "Struct_Number" A negative value means the project increased the number of structures getting wet. A positive value means the project decreased the number of structures getting wet.			
2. "Struct_Damages" A parenthesis () means the project increased the dollar amount of damages where no paranthesis means the project decreased the dollar amount of damages.			

Table 5.3.4 Incremental Damages (FWO vs FW Gates Closed – USACE PS Alt. 2)

2032 High Tide - FWO vs FW (Gates Open - City PS Active - USACE PS Alt2) Incremental Damages			
FWO 2032		FW 2032 (gates closed) P.S. Alt2	
Exterior Elevation	Interior rainfall	Struct_Number	Struct_Damages
high tide	50% AEP rain	49	$ 1,410,542
high tide	20% AEP rain	55	$ 1,821,629
high tide	10% AEP rain	32	$ 1,643,161
high tide	4% AEP rain	2	$ 515,111
high tide	2% AEP rain	-32	$ (1,183,585)
high tide	1% AEP rain	-90	$ (4,020,149)
1. "Struct_Number" A negative value means the project increased the number of structures getting wet. A positive value means the project decreased the number of structures getting wet.			
2. "Struct_Damages" A parenthesis () means the project increased the dollar amount of damages where no paranthesis means the project decreased the dollar amount of damages.			

2082 High Tide - FWO vs FW (Gates Open - City PS Active - USACE PS Alt2) Incremental Damages			
FWO 2082		FW 2082 (gates closed) P.S. Alt2	
Exterior Elevation	Interior rainfall	Struct_Number	Struct_Damages
high tide	50% AEP rain	49	$ 1,510,343
high tide	20% AEP rain	55	$ 1,978,919
high tide	10% AEP rain	32	$ 1,793,232
high tide	4% AEP rain	2	$ 663,539
high tide	2% AEP rain	-32	$ (1,027,919)
high tide	1% AEP rain	-90	$ (3,907,605)
1. "Struct_Number" A negative value means the project increased the number of structures getting wet. A positive value means the project decreased the number of structures getting wet.			
2. "Struct_Damages" A parenthesis () means the project increased the dollar amount of damages where no paranthesis means the project decreased the dollar amount of damages.			

Table 5.3.5 Incremental Damages (FWO vs FW Gates Closed – USACE PS Alt. 3)

2032 High Tide - FWO vs FW (Gates Open - City PS Active - USACE PS Alt3) Incremental Damages			
FWO 2032		FW 2032 (gates closed) P.S. Alt3	
Exterior Elevation	Interior rainfall	Struct_Number	Struct_Damages
high tide	50% AEP rain	49	$ 1,598,463
high tide	20% AEP rain	71	$ 2,370,779
high tide	10% AEP rain	51	$ 2,278,091
high tide	4% AEP rain	48	$ 2,350,371
high tide	2% AEP rain	19	$ 1,696,607
high tide	1% AEP rain	-28	$ (221,758)
1. "Struct_Number" A negative value means the project increased the number of structures getting wet. A positive value means the project decreased the number of structures getting wet.			
2. "Struct_Damages" A parenthesis () means the project increased the dollar amount of damages where no paranthesis means the project decreased the dollar amount of damages.			

2082 High Tide - FWO vs FW (Gates Open - City PS Active - USACE PS Alt3) Incremental Damages			
FWO 2082		FW 2082 (gates closed) P.S. Alt3	
Exterior Elevation	Interior rainfall	Struct_Number	Struct_Damages
high tide	50% AEP rain	55	$ 1,700,945
high tide	20% AEP rain	77	$ 2,530,390
high tide	10% AEP rain	57	$ 2,426,385
high tide	4% AEP rain	53	$ 2,498,623
high tide	2% AEP rain	25	$ 1,851,471
high tide	1% AEP rain	-28	$ (137,062)
1. "Struct_Number" A negative value means the project increased the number of structures getting wet. A positive value means the project decreased the number of structures getting wet.			
2. "Struct_Damages" A parenthesis () means the project increased the dollar amount of damages where no paranthesis means the project decreased the dollar amount of damages.			

5.3.2 ANNUAL DAMAGES PER SCENARIO

This section provides the results for Average Annual Damages (AAD) and Equivalent Annual Damages (EAD) for the future without- and future with-project conditions and alternative.

I.E., the average annual damages for the future without-project (2032) are computed for each AEP rainfall frequency (50%, 20%, 10%, 4%, 2%, and 1%). The average annual damages for each of those frequencies are summed to calculate the without project average annual damages for the future without-project condition in the year 2032. The same process is performed for the future without-project (2082) and all future with-project alternatives individually.

The individual project alternatives are compared back to the future without-project alternatives for the years 2032 and 2082 to describe whether the project increases the annual damages or decreases the annual damages, which could be said to provide net benefits.

Note: Each table contains a row near the bottom labeled "Damage Reductions". If the value in the cell for Damage Reductions is bound by a parenthesis, then the project alternative does not reduce the average annual damages but rather increases the average annual damages as compared to the without-project. If the value is positive (no parenthesis) then the project reduces the average annual damages as compared to the future without-project and could be said to provide net benefits.

Table 5.3.6 EAD 2032 (FWO vs FW Gates Open)

FWO 2032 - High Tide					
Frequency (yr)	*Percent (AEP)*	*Incremental Probability*	*Damage at Stage*	*Average Damage*	*Average Annual*
		0.010		$ 27,320,270	$ 273,203
100	1%		$ 54,640,540.58		
		0.010		$ 51,081,398	$ 510,814
50	2%		$ 47,522,254.61		
		0.020		$ 44,504,283	$ 890,086
25	4%		$ 41,486,310.44		
		0.060		$ 37,504,950	$ 2,250,297
10	10%		$ 33,523,589.76		
		0.100		$ 30,718,494	$ 3,071,849
5	20%		$ 27,913,397.54		
		0.300		$ 25,083,379	$ 7,525,014
2	50%		$ 22,253,360.97		
Without Project Average Annual Damages					$ 14,521,262
FW 2032 - High Tide - (Gates Open - City PS Active - USACE PS Inactive)					
Frequency (yr)	*Percent (AEP)*	*Incremental Probability*	*Damage at Stage*	*Average Damage*	*Average Annual*
		0.010		$ 27,599,694	$ 275,997
100	1%		$ 55,199,387.38		
		0.010		$ 51,588,641	$ 515,886
50	2%		$ 47,977,894.34		
		0.020		$ 44,918,586	$ 898,372
25	4%		$ 41,859,276.86		
		0.060		$ 37,808,020	$ 2,268,481
10	10%		$ 33,756,762.75		
		0.100		$ 30,856,781	$ 3,085,678
5	20%		$ 27,956,798.28		
		0.300		$ 25,061,848	$ 7,518,554
2	50%		$ 22,166,897.59		
With Project Average Annual Damages					$ 14,562,969
Damage Reductions					$ (41,706)

Table 5.3.7 EAD 2082 (FWO vs FW Gates Open)

FWO 2082 - High Tide					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*	*Average Damage*	*Average Annual*
		0.010		$ 27,469,063	$ 274,691
100	1%		$ 54,938,126.22		
		0.010		$ 51,458,010	$ 514,580
50	2%		$ 47,977,894.34		
		0.020		$ 44,918,586	$ 898,372
25	4%		$ 41,859,276.86		
		0.060		$ 37,862,244	$ 2,271,735
10	10%		$ 33,865,210.76		
		0.100		$ 31,068,327	$ 3,106,833
5	20%		$ 28,271,442.33		
		0.300		$ 25,413,271	$ 7,623,981
2	50%		$ 22,555,099.05		
		Without Project Average Annual Damages			**$ 14,690,191**

FW 2082 - High Tide - (Gates Open - City PS Active - USACE PS Inactive)					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*	*Average Damage*	*Average Annual*
		0.010		$ 27,779,689	$ 277,797
100	1%		$ 55,559,378.12		
		0.010		$ 51,947,227	$ 519,472
50	2%		$ 48,335,075.47		
		0.020		$ 45,273,881	$ 905,478
25	4%		$ 42,212,687.43		
		0.060		$ 38,162,393	$ 2,289,744
10	10%		$ 34,112,098.62		
		0.100		$ 31,219,512	$ 3,121,951
5	20%		$ 28,326,925.70		
		0.300		$ 25,402,492	$ 7,620,748
2	50%		$ 22,478,057.94		
		With Project Average Annual Damages			**$ 14,735,189**
			Damage Reductions		**$ (44,998)**

Table 5.3.8 EAD 2032 (FWO vs FW Gates Closed – No USACE PS)

FWO 2032 - High Tide					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*	*Average Damage*	*Average Annual*
		0.010		$ 27,320,270	$ 273,203
100	1%		$ 54,640,540.58		
		0.010		$ 51,081,398	$ 510,814
50	2%		$ 47,522,254.61		
		0.020		$ 44,504,283	$ 890,086
25	4%		$ 41,486,310.44		
		0.060		$ 37,504,950	$ 2,250,297
10	10%		$ 33,523,589.76		
		0.100		$ 30,718,494	$ 3,071,849
5	20%		$ 27,913,397.54		
		0.300		$ 25,083,379	$ 7,525,014
2	50%		$ 22,253,360.97		
		Without Project Average Annual Damages			**$ 14,521,262**

FW 2032 - High Tide - (Gates Closed - City PS Active - USACE PS Inactive)					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*		*Average Annual*
		0.010		$ 33,625,239	$ 336,252
100	1%		$ 67,250,478.52		
		0.010		$ 62,029,275	$ 620,293
50	2%		$ 56,808,071.54		
		0.020		$ 52,420,506	$ 1,048,410
25	4%		$ 48,032,940.59		
		0.060		$ 42,448,820	$ 2,546,929
10	10%		$ 36,864,699.43		
		0.100		$ 33,376,071	$ 3,337,607
5	20%		$ 29,887,442.80		
		0.300		$ 26,709,642	$ 8,012,893
2	50%		$ 23,531,841.83		
		With Project Average Annual Damages			**$ 15,902,384**
			Damage Reductions		**$ (1,381,122)**

Table 5.3.9 EAD 2082 (FWO vs FW Gates Closed – No USACE PS)

FWO 2082 - High Tide					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*	*Average Damage*	*Average Annual*
		0.010		$ 27,469,063	$ 274,691
100	1%		$ 54,938,126.22		
		0.010		$ 51,400,201	$ 514,002
50	2%		$ 47,862,276.34		
		0.020		$ 44,841,225	$ 896,824
25	4%		$ 41,820,173.09		
		0.060		$ 37,842,692	$ 2,270,562
10	10%		$ 33,865,210.76		
		0.100		$ 31,068,327	$ 3,106,833
5	20%		$ 28,271,442.33		
		0.300		$ 25,413,271	$ 7,623,981
2	50%		$ 22,555,099.05		
		Without Project Average Annual Damages			**$ 14,686,893**
FW 2082 - High Tide - (Gates Closed - City PS Active - USACE PS Inactive)					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*		*Average Annual*
		0.010		$ 33,717,212	$ 337,172
100	1%		$ 67,434,423.41		
		0.010		$ 62,212,646	$ 622,126
50	2%		$ 56,990,867.72		
		0.020		$ 52,604,446	$ 1,052,089
25	4%		$ 48,218,024.17		
		0.060		$ 42,638,540	$ 2,558,312
10	10%		$ 37,059,056.76		
		0.100		$ 33,572,783	$ 3,357,278
5	20%		$ 30,086,509.83		
		0.300		$ 26,910,085	$ 8,073,025
2	50%		$ 23,733,659.76		
		With Project Average Annual Damages			**$ 16,000,004**
		Damage Reductions			**$ (1,313,111)**

Table 5.3.10 EAD 2032 (FWO vs FW Gates Closed – USACE PS Alt 1)

FWO 2032 - High Tide					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*	*Average Damage*	*Average Annual*
		0.010		$ 27,320,270	$ 273,203
100	1%		$ 54,640,540.58		
		0.010		$ 51,081,398	$ 510,814
50	2%		$ 47,522,254.61		
		0.020		$ 44,504,283	$ 890,086
25	4%		$ 41,486,310.44		
		0.060		$ 37,504,950	$ 2,250,297
10	10%		$ 33,523,589.76		
		0.100		$ 30,718,494	$ 3,071,849
5	20%		$ 27,913,397.54		
		0.300		$ 25,083,379	$ 7,525,014
2	50%		$ 22,253,360.97		
		Without Project Average Annual Damages			**$ 14,521,262**
FW 2032 - High Tide - (Gates Closed - City PS Active - USACE PS Alt 1)					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*		*Average Annual*
		0.010		$ 30,675,638	$ 306,756
100	1%		$ 61,351,276.42		
		0.010		$ 56,292,994	$ 562,930
50	2%		$ 51,234,710.59		
		0.020		$ 47,167,218	$ 943,344
25	4%		$ 43,099,725.95		
		0.060		$ 38,101,961	$ 2,286,118
10	10%		$ 33,104,196.63		
		0.100		$ 29,890,506	$ 2,989,051
5	20%		$ 26,676,814.68		
		0.300		$ 23,948,189	$ 7,184,457
2	50%		$ 21,219,562.91		
		With Project Average Annual Damages			**$ 14,272,656**
		Damage Reductions			**$ 248,607**

Table 5.3.11 EAD 2082 (FWO vs FW Gates Closed – USACE PS Alt 1)

FWO 2082 - High Tide					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*	*Average Damage*	*Average Annual*
		0.010		$ 27,469,063	$ 274,691
100	1%		$ 54,938,126.22		
		0.010		$ 51,400,201	$ 514,002
50	2%		$ 47,862,276.34		
		0.020		$ 44,841,225	$ 896,824
25	4%		$ 41,820,173.09		
		0.060		$ 37,842,692	$ 2,270,562
10	10%		$ 33,865,210.76		
		0.100		$ 31,068,327	$ 3,106,833
5	20%		$ 28,271,442.33		
		0.300		$ 25,413,271	$ 7,623,981
2	50%		$ 22,555,099.05		
		Without Project Average Annual Damages			$ 14,686,893
FW 2082 - High Tide - (Gates Closed - City PS Active - USACE PS Alt 1)					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*		*Average Annual*
		0.010		$ 30,770,051	$ 307,701
100	1%		$ 61,540,101.45		
		0.010		$ 56,480,640	$ 564,806
50	2%		$ 51,421,178.05		
		0.020		$ 47,354,240	$ 947,085
25	4%		$ 43,287,302.57		
		0.060		$ 38,292,333	$ 2,297,540
10	10%		$ 33,297,364.15		
		0.100		$ 30,086,651	$ 3,008,665
5	20%		$ 26,875,938.35		
		0.300		$ 24,148,783	$ 7,244,635
2	50%		$ 21,421,626.73		
		With Project Average Annual Damages			$ 14,370,432
		Damage Reductions			**$ 316,461**

Table 5.3.12 EAD 2032 (FWO vs FW Gates Closed – USACE PS Alt 2)

FWO 2032 - High Tide					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*	*Average Damage*	*Average Annual*
		0.010		$ 27,320,270	$ 273,203
100	1%		$ 54,640,540.58		
		0.010		$ 51,081,398	$ 510,814
50	2%		$ 47,522,254.61		
		0.020		$ 44,504,283	$ 890,086
25	4%		$ 41,486,310.44		
		0.060		$ 37,504,950	$ 2,250,297
10	10%		$ 33,523,589.76		
		0.100		$ 30,718,494	$ 3,071,849
5	20%		$ 27,913,397.54		
		0.300		$ 25,083,379	$ 7,525,014
2	50%		$ 22,253,360.97		
		Without Project Average Annual Damages			$ 14,521,262
FW 2032 - High Tide - (Gates Closed - City PS Active - USACE PS Alt 2)					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*		*Average Annual*
		0.010		$ 29,330,345	$ 293,303
100	1%		$ 58,660,689.31		
		0.010		$ 53,683,264	$ 536,833
50	2%		$ 48,705,839.25		
		0.020		$ 44,838,520	$ 896,770
25	4%		$ 40,971,199.91		
		0.060		$ 36,425,814	$ 2,185,549
10	10%		$ 31,880,428.41		
		0.100		$ 28,986,098	$ 2,898,610
5	20%		$ 26,091,768.54		
		0.300		$ 23,655,666	$ 7,096,700
2	50%		$ 21,219,562.91		
		With Project Average Annual Damages			$ 13,907,765
		Damage Reductions			**$ 613,498**

Table 5.3.13 EAD 2082 (FWO vs FW Gates Closed – USACE PS Alt 2)

FWO 2082 - High Tide					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*	*Average Damage*	*Average Annual*
		0.010		$ 27,469,063	$ 274,691
100	1%		$ 54,938,126.22		
		0.010		$ 51,400,201	$ 514,002
50	2%		$ 47,862,276.34		
		0.020		$ 44,841,225	$ 896,824
25	4%		$ 41,820,173.09		
		0.060		$ 37,842,692	$ 2,270,562
10	10%		$ 33,865,210.76		
		0.100		$ 31,068,327	$ 3,106,833
5	20%		$ 28,271,442.33		
		0.300		$ 25,413,271	$ 7,623,981
2	50%		$ 22,555,099.05		
Without Project Average Annual Damages					$ 14,686,893
FW 2082 - High Tide - (Gates Closed - City PS Active - USACE PS Alt 2)					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*		*Average Annual*
		0.010		$ 29,422,866	$ 294,229
100	1%		$ 58,845,731.50		
		0.010		$ 53,867,964	$ 538,680
50	2%		$ 48,890,195.59		
		0.020		$ 45,023,415	$ 900,468
25	4%		$ 41,156,633.95		
		0.060		$ 36,614,306	$ 2,196,858
10	10%		$ 32,071,978.65		
		0.100		$ 29,182,251	$ 2,918,225
5	20%		$ 26,292,523.40		
		0.300		$ 23,668,640	$ 7,100,592
2	50%		$ 21,044,755.84		
With Project Average Annual Damages					$ 13,949,052
Damage Reductions					**$ 737,841**

Table 5.3.14 EAD 2032 (FWO vs FW Gates Closed – USACE PS Alt 3)

FWO 2032 - High Tide					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*	*Average Damage*	*Average Annual*
		0.010		$ 27,320,270	$ 273,203
100	1%		$ 54,640,540.58		
		0.010		$ 51,081,398	$ 510,814
50	2%		$ 47,522,254.61		
		0.020		$ 44,504,283	$ 890,086
25	4%		$ 41,486,310.44		
		0.060		$ 37,504,950	$ 2,250,297
10	10%		$ 33,523,589.76		
		0.100		$ 30,718,494	$ 3,071,849
5	20%		$ 27,913,397.54		
		0.300		$ 25,083,379	$ 7,525,014
2	50%		$ 22,253,360.97		
Without Project Average Annual Damages					$ 14,521,262
FW 2032 - High Tide - (Gates Closed - City PS Active - USACE PS Alt 3)					
Frequency	*Percent*	*Incremental Probability*	*Damage at Stage*		*Average Annual*
		0.010		$ 27,431,149	$ 274,311
100	1%		$ 54,862,298.99		
		0.010		$ 50,343,973	$ 503,440
50	2%		$ 45,825,647.37		
		0.020		$ 42,480,793	$ 849,616
25	4%		$ 39,135,939.29		
		0.060		$ 35,190,719	$ 2,111,443
10	10%		$ 31,245,499.10		
		0.100		$ 28,394,059	$ 2,839,406
5	20%		$ 25,542,618.45		
		0.300		$ 23,098,758	$ 6,929,627
2	50%		$ 20,654,897.72		
With Project Average Annual Damages					$ 13,507,844
Damage Reductions					**$ 1,013,419**

Table 5.3.15 EAD 2082 (FWO vs FW Gates Closed – USACE PS Alt 3)

FWO 2082 - High Tide					
Frequency	Percent	Incremental Probability	Damage at Stage	Average Damage	Average Annual
		0.010		$ 27,469,063	$ 274,691
100	1%		$ 54,938,126.22		
		0.010		$ 51,400,201	$ 514,002
50	2%		$ 47,862,276.34		
		0.020		$ 44,841,225	$ 896,824
25	4%		$ 41,820,173.09		
		0.060		$ 37,842,692	$ 2,270,562
10	10%		$ 33,865,210.76		
		0.100		$ 31,068,327	$ 3,106,833
5	20%		$ 28,271,442.33		
		0.300		$ 25,413,271	$ 7,623,981
2	50%		$ 22,555,099.05		
		Without Project Average Annual Damages			$ 14,686,893
FW 2082 - High Tide - (Gates Closed - City PS Active - USACE PS Alt 3)					
Frequency	Percent	Incremental Probability	Damage at Stage		Average Annual
		0.010		$ 27,537,594	$ 275,376
100	1%		$ 55,075,188.45		
		0.010		$ 50,542,997	$ 505,430
50	2%		$ 46,010,805.76		
		0.020		$ 42,666,178	$ 853,324
25	4%		$ 39,321,549.91		
		0.060		$ 35,380,188	$ 2,122,811
10	10%		$ 31,438,825.66		
		0.100		$ 28,589,939	$ 2,858,994
5	20%		$ 25,741,052.75		
		0.300		$ 23,297,603	$ 6,989,281
2	50%		$ 20,854,154.04		
		With Project Average Annual Damages			$ 13,605,216
		Damage Reductions			$ 1,081,677

5.3.3 SUMMARY OF ECONOMIC MODEL RESULTS

Storm Gates Open (USACE Pump Stations Inactive)
A review of the storm gates open assessment displays the project inundates one additional structure for the 50% AEP in 2032 and no additional for the 50% AEP in 2082. The 20% AEP inundates 2 additional structures in 2032 and 3 additional in 2082. The 10% AEP inundates 6 additional structures in 2032 and 8 additional in 2082. The 4% AEP inundates 9 additional in 2032 and 9 additional in 2082. The 2% AEP and 1% AEP inundate 8 to 12 additional in both 2032 and 2082. The results display the project with storm gates open increases the average annual damages by approximately $41,700 in the year 2032 and $45,000 in the year 2082.

Storm Gates Closed (USACE Pump Stations Active)
A review of the storm gates closed assessment for pump station alternatives displays the pump station alternatives 1, 2, and 3 significantly reduce the number of structures inundated and dollar damages for the 50% and 20% AEP rain events for the years 2032 and 2082. Pump station alternative 1 slightly reduces the number of structures damaged for the 10% AEP while alternative 2 and 3 significantly reduce the number of structures damaged for the 10% AEP for the years 2032 and 2082.

Pump station alternative 1 displays reduced number of structures inundated and reduced structure damages in dollars for the 50% AEP to the 10% AEP while additional damages are induced for the events greater than the 10% AEP referring to both 2032 and 2082.

Pump station alternative 2 displays reduced number of structures inundated and reduced structure damages in dollars for the 50% AEP to the 4% AEP while additional damages are induced for the events greater than the 4% AEP referring to both 2032 and 2082.

Pump station alternative 3 displays reduced number of structures inundated and reduced structure damages in dollars for the 50% AEP to the 2% AEP while additional damages are induced for the 1% AEP referring to both 2032 and 2082.

The average annual damages for pump station alternative 1 display a decrease for the years 2032 and 2082 of approximately $248,600 and 316,500 respectively. The average annual damages for pump station alternative 2 display a decrease for the years 2032 and 2082 of approximately $613,500 and $737,800 respectively. The average annual damages for pump station alternative 3 display a decrease for the years 2032 and 2082 of approximately $1.01 million and $1.08 million.

5.4 TENTATIVELY SELECTED PLAN FOR INTERIOR DRAINAGE

Upon reviewing the results of the hydraulic assessment and economic assessment, pump station alternative 2 selected to be implemented as part of the project alignment. Pump station locations and capacities are provided in Section 3.5. For the overall system performance, pump station alternative 2 displays damage reductions in the average annual damages as presented in the previous sections. The pump station alternatives were assessed holistically as a system meaning the pump station alternative capacities were not mixed and matched for assessing each pump location individually. However, the FDA model does provide damage assessments on a per model area basis which provides into the performance of the pump stations. This approach assists in the selection of pump station alternative 2 for implementation to the TSP for the project alignment.

The storm gate dimensions and locations that have been assessed and chosen to be implemented as part of the TSP are displayed in Section 3.5. One alternative for storm gates were assessed in HEC-FDA. Further assessment is to be conducted during PED phase to refine storm gate dimensions if needed during the site-specific analysis.

While the economic assessment displays pump station alternative 2 provides a reduction in average annual damages (AAD) for the system, a site-specific assessment is to be completed during PED Phase to appropriately size each pump station and/or storm gate. Assessing the damages at a finer scale will be more insightful to individually sizing pump stations as some areas may overcompensate interior drainage relief while some may undercompensate. Further assessment may reveal some pump station capacities could be reduced from the current TSP while some pump station capacities may need increase. Typically, features that improve interior drainage above the without-project interior drainage are to be incrementally justified. The site-specific assessment is critical to not over-designing the system, inflating project cost, while also considering the effects of under-designing a system which may under-perform, leading to induced interior risk.

CHAPTER 6 – MODELING CHALLENGES AND ASSUMPTIONS

- Vertical Datum used for modeling is NAVD88
- Sea Level Change (SLC) considered is NOAA's 2006 Published intermediate rates of +0.56 feet for 2032 and +1.65 for 2082
- The City of Charleston is to raise the Low Battery to be similar to the High Battery elevation.
- Three City of Charleston pump stations are included in the HEC-RAS modeling. The Concord Street, MUSC, and Spring Fishburne pump stations.
-
- The Charleston Sea Level Rise Strategy contains detailed descriptions for addressing sea level rise through various stormwater management measures. Details are provided in Section 2.3.1 of this report.

- HEC-RAS 2D does not have the capability of modeling sub-surface storm drainage. The HEC-RAS 2D model assesses surface-flow rainfall runoff only. A surface flow only model may compute higher than expected inland elevations in the absence of sub-surface drainage however this is the case for without- and with-project conditions.

- The City of Charleston indicates the stormwater pipe network contains max capacities approximately equal to a 10% AEP rainfall event and in some areas are limited to lesser capacities. The CSRM study focuses on the 10% AEP event for design of interior drainage features if the existing system only contains such capacities for bringing stormwater to the pump stations. It can be assumed that surface flow becomes a significant component of drainage when the pipes become overwhelmed.

- The HEC-RAS model could not be validated with a past event because no inland high-water marks or observed water surface elevations are available. Although there is uncertainty in interior water surface elevations due to the inability of sub-surface flow and absence of a validated model to observed elevations, the modeling provides a relative comparison between with and without-project alternatives.

- The rainfall time-series is applied uniformly across the 2D study area. Actual rainfall varies spatially across the study area.

- The pump stations are modeled using constant flow rates once turned on. The actual pump operations are more complex, and the total discharge depends on the interior and exterior stages and the efficiency of the pump discharges would fluctuate as the tide/storm surge fluctuates.

- Pre-storm interior water level drawdown was assumed for the gates closed pump station assessment. The storm gates are closed at low tide prior to the arrival of a storm surge. This operation will improve the performance of the system and reduce the likelihood of interior residual flooding. A sensitivity analysis was completed to assess the performance of the pump stations for situations in which the storm gates are closed at high tide and therefore the interior contains less available storage for rainfall runoff while the pumps are discharging into the exterior.

CHAPTER 7 – FEASIBILITY PHASE CONCLUSIONS

- HEC-RAS simulations were completed for the future without-project condition and future with-project condition for storm gates and pump station alternatives for the 50%, 20%, 10%, 4%, 2%, and 1% AEP rainfall events.

- The storm gate dimensions and pump station alternatives with various capacities are provided in Section 3.5.

- The project shows an increase in water surface elevations for some output locations at low-lying areas. Most output locations for areas at higher elevations show modest impacts. Some output locations show a decrease in water surface elevations for the project. More information is provided in Section 4.6.

- The economics assessment was completed using HEC-FDA. As result of the HEC-FDA effort, a recommended plan was defined to implement as part of the project alignment. Storm gate dimensions for the recommended plan are provided in Section 3.5. Pump Station alternative 2 was selected for implementation into the TSP. Pump Station alternative 2 capacities are listed in Section 3.5. Additional assessment will be completed during PED phase to further refine and assess the interior drainage features on a site-by-site basis.

- Ultimately, the PDT is to select a pump station and storm gate design that limits the increase in interior water surface elevations to tolerable levels particularly for the 10% AEP event as it has been indicated the existing storm pipes accommodate no more than a 10% AEP event. Although, the focus of design would be for such capacities, the effects are also considered for larger events which display greater impacts attributed to the project.

- The HEC-RAS modeling and HEC-FDA modeling provides information needed by the PDT to make an informed decision for total pumping capacity and drainage system (storm gates).

- Appendix 1 of this report contains a qualitative assessment for climate change impacts to inland hydrology following guidelines provided in ECB 2018-14.

- Appendix 2 of this report contains additional modeling scenarios to assess some factors that may have impacts to the performance of the project. Additional modeling and sensitivity analyses are completed to provide further insight into the performance of the interior drainage features while varying certain factors which influence the performance of the interior drainage.

- A wave overwash assessment was completed using coastal modeling outputs as inputs for the interior drainage hydraulic model to assess how the interior drainage system handles wave overtopping or waves splashing over the wall (Section 9.1) The coastal modeling wave overtopping analysis discovered that an approximately 2% AEP Stillwater elevation (10 feet NAVD88 in 2082) with one wave amplitude would cause wave overtopping of the TSP design elevation at 12 feet NAVD88.

- There are two perspectives of assessing overtopping – flood overtopping and wave (overwash) overtopping. Flood overtopping occurs when a continuous flow of a water elevation exceeds the design elevation of a wall. For overtopping by waves, or wave overtopping, the Stillwater elevation approaches but does not exceed the crest elevation. Instead, waves approaching the structure run up its profile and overtop the crest. The wave action can form an equivalent discharge per linear distance of the structure and can lead to erosion, potential failure of the structure, and can create ponding areas on the land side of a project alignment if pump stations are not considered. Wave overtopping is a function of the Stillwater elevation, wave height, period and direction, and structure slope, freeboard, and roughness.

- Additional assessments for flood overtopping are to be completed during PED phase to assess how the interior drainage system handles such overtopping flows above those that were assessed and documented in Section 9.1. In discussion with the City of Charleston, the existing pump stations do not have the capacities to pump when seawater is directly backflowing into the system. The check valve program is to address this concern and therefore the pump station assessment assuming overtopping flows would also assume the check valves do not allow the tidal backflow. In a scenario where the wall is overtopped, a detailed assessment of the timing of an overtopping event versus the opening of storm gates and draining of the interior area with pumps and gates can also be examined during PED phase.
- The USACE study focuses on the primary drivers of flooding which are coastal storm surge and the residual risk for the interior area from rainfall flooding and wall overtopping. Riverine flooding is not a focus of the study as the driver of exterior stages is the Charleston tidal gage

stages. Groundwater flooding is typically a much smaller contributor, by volume, to that of rainfall and tide/surge. The city has existing and proposed pump stations which can be used to remove excess water, regardless of the source as well as the USACE proposed pump stations. Generally, pump stations designed for less frequent rainfall events such as 10% AEP or greater would be adequate for dealing with any extra seepage from a groundwater source. However, the city may have more information about groundwater contributions to existing pump stations. During PED phase, the city could provide information about known groundwater contributions and whether pump station records indicate pumps have cycled on and off due to groundwater contributions (during periods without precipitation) and how significant such impacts are if known. In addition, pumps cycling on and off due to groundwater contributions may have some implications on pump performance during successive rain-storm events.

- During PED phase further model sensitivity is to be completed by varying model parameters such as rainfall depths, durations, and intensities. The rainfall depths can be used to simulate groundwater infiltration where saturated grounds may lead to higher surface runoff volumes for rainfall. Saturated grounds could be from successive storms leaving the ground "wet" or other effects such as less permeable, shallow groundwater tables. As studied so far, modeling the full suite of 24-hour rainfall frequencies provides a good overview into the performance of each project alternative amongst various storms. In addition, sensitivity analyses are completed by varying the initial interior water levels prior to the arrival of a storm. Meaning tidal creeks may be more saturated, have standing water, frequently higher sea levels, or just high tide that pushes into the system prior to the closing of storm gates. These assumptions are not all explicit to groundwater but do contain some groundwater considerations while showing the performance of pumps with the varied assumptions of the starting conditions to which pumps must mitigate.

- During PED phase, the PDT would seek to work further with the City to better understand the complex storm drainage pipes and capacities and the areas to which they service (take water off the surface and route to certain pump stations or outfalls). Also, more information about the stormwater management plans in updated stormwater design manuals, flooding and sea level rise strategies, and Dutch Dialogues Charleston report.

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APPENDIX 1 - CLIMATE CHANGE IMPACTS TO INLAND HYDROLOGY

The USACE overarching climate adaptation policy requires consideration of climate change in all current and future studies to reduce vulnerabilities and enhance the resilience of USACE water resources infrastructure. To meet the USACE climate adaptation policy, project delivery teams (PDTs) must assess climate change impacts when a study involves inland hydrology, coastal analysis and/or a boundary condition impacted by sea level. The assessment should be carried out at an appropriate, scalable level based on the complexity, size and level of risk associated with the project. Sea Level Change (SLC), inland hydrology, and riverine hydrology are assessed to determine if the project is vulnerable to climate change.

The climate assessment for inland hydrology follows the USACE guidance of Engineering and Construction Bulletin (ECB 2018-14), Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects. For most USACE projects and studies, a qualitative assessment provides the necessary information to support the assessment of climate change risk and uncertainties to the project design or constructed project. Per the guidance, a hydrologic literature review of observed climate trends and projected climate trends in the project area is required. USACE and NWS hydrologic and meteorologic tools are used for this assessment. The tools are used to detect non-stationarities in sea level change, riverine hydrology, and meteorology (precipitation).

An in-depth assessment of sea level change is provided in Engineering Appendix B Section 3.6.1 and 3.6.2. The selected sea level rates from that assessment are +0.56 feet for the year 2032 and +1.65 feet for the year 2082. These are considered "intermediate" rates. The hydraulic (HEC-RAS) model applies the selected SLR rates to the tidal boundary conditions to account for sea level rise for the interior drainage assessment.

As indicated in the City's Flooding and Sea Level Rise strategy, one way to track local impacts from sea level rise is documenting "minor coastal flooding". Commonly called nuisance, sunny day or high tide flooding, minor flooding is a threshold from the National Weather Service that indicates when the tide has reached a certain height (7.0 ft. MLLW in the Charleston Harbor or 3.86 feet NAVD88). At this height, low-lying areas on land begin to flood. Other flood thresholds are provided in Section 2.2.1. The City of Charleston has experienced a marked increase in the number of days of minor coastal flooding over time (Figure 3 of this section). In response, the city has implemented the check valve program. In addition to tidal flooding and sea level rise, changes in precipitation trends are a focus of the City's strategy and some stormwater management features are being modified in response. Further discussion provided in Section 2.3.

The climate change to inland hydrology was assessed qualitatively following the three phases outlined in ECB 2018-14.

1) Scoping
2) Vulnerability assessment
3) Risk assessment

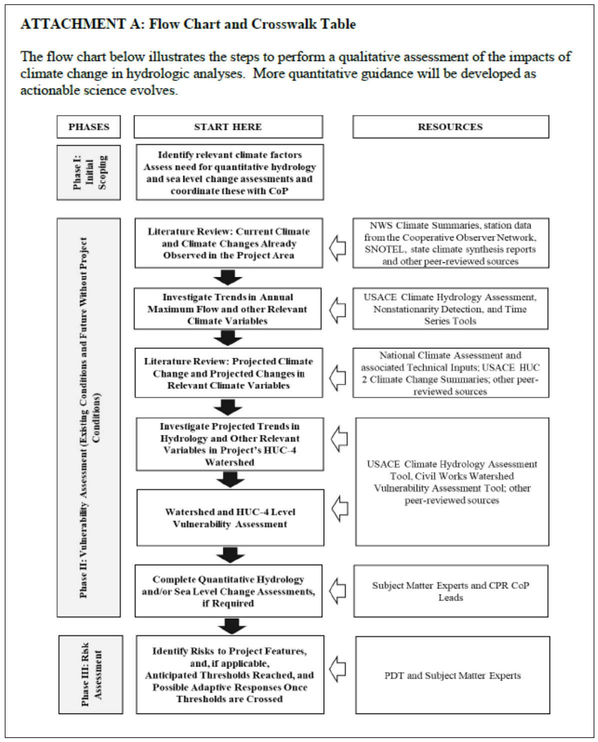

Figure 1. ECB 2018-14 Flow Chart for Climate Change Assessment

1.1 SCOPING

The purpose of this phase consists of two activities:

1) Understanding what climate variables are relevant to the assessment
2) Determining whether quantitative hydrology and/or sea level change assessments are needed.

For this project, it was determined that the following climate variables were the most relevant to the study area: sea level rise, precipitation, and streamflow. For most USACE projects and studies, a qualitative assessment will provide the necessary information to support the assessment of climate change risk and uncertainties to the project design or constructed project. Additionally, due to the project location (elevations less than 50 ft. NAVD88), and potential for sea level rise to affect riverine stages, it was determined that policy procedures outlined in ER 1100-2-8162 and ETL 1100-2-1 will apply. As mentioned, the sea level change assessment is provided in the Main Engineering Appendix.

1.2 VULNERABILITY ASSESSMENT

The vulnerability assessment includes a literature review of the current climate as well as observed and projected climate trends and includes the application of climate tools used to provide information on observed and projected climate trends relevant to the project area.

1.2.1 LITERATURE REVIEW

USACE is undertaking its climate change preparedness and resilience planning and implementation in consultation with internal and external experts using the best available – and actionable – climate science and climate change information. As required by ECB 2018-14, a hydrologic literature review was conducted to summarize peer reviewed literature on current climate and observed climate trends and projected climate trends in the project area.

The USACE January 2015 Civil Works Technical Series, CWTS-2015-03 for the South Atlantic-Gulf Region of the United States focuses on temperature, extreme precipitation events, stream flow trends, and both current and projected. In Section 2.2 of CWTS-2015-03, it was stated "A study by Dai et al. (2011), for a climate station in South Carolina (at the Santee Experimental Forest), identified a generally increasing, but not statistically significant, pattern in the number of extreme storm events over the past 60 years. Similarly, they demonstrate a generally increasing trend in total annual precipitation at their study site, but without statistical significance." While the Santee watershed is a different watershed that does not impact Charleston, South Carolina, it provides a general characterization of the precipitation trends in the region.

The CWTS-2015-03 contains a section titled "Summary of Future Climate Projection Findings". The section summarizes that projections of precipitation in the region are less certain than those associated with air temperature and that results of the studies are roughly evenly split with respect to projected increases vs. decreases in future annual precipitation. The South Atlantic-Gulf Region is noted to lie in a "transition zone" between the projected wetter conditions to the north and dryer conditions to the west. The study noted a "moderate" consensus that future storm events in the report may be more intense and more frequent compared to the recent past.

The trends and literary consensus of observed and project primary variables noted above are summarized for reference and comparison in Figure 2 below.

In general, the literature review shows that observed precipitation trends are denoted as "Small Increase" with low consensus amongst the ten relevant literature studies. The projected trend shows "No Change" with low consensus amongst six relevant literature studies.

In general, the literature review shows that observed precipitation extremes are denoted as "Small Increase" with eight relevant literature studies of the majority reporting similar trends while the projected trend also shows "Small Increase" with five relevant literature studies of the majority reporting similar trends.

If significant precipitation increases occur, potential project vulnerabilities may include increased interior rainfall runoff leading to increased rainfall flooding. This may require increased pump capacity and longer pumping times, which may lead to increased maintenance cost. Increasing interior flooding may also increase operational complexity if operators have more difficulty navigating flooded streets to access project sites.

In general, the literature review shows that observed hydrology/streamflow is denoted as "Small Decrease" with four relevant literature studies of the majority report similar trends. The projected trend shows no change with seven relevant literature reviews of low consensus.

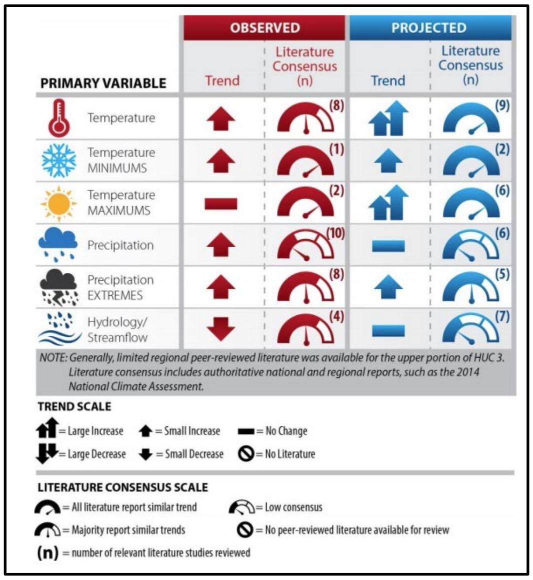

Figure 2. Summary Matrix of Observed and Projected Climate Trends (CWTS-2015-03)

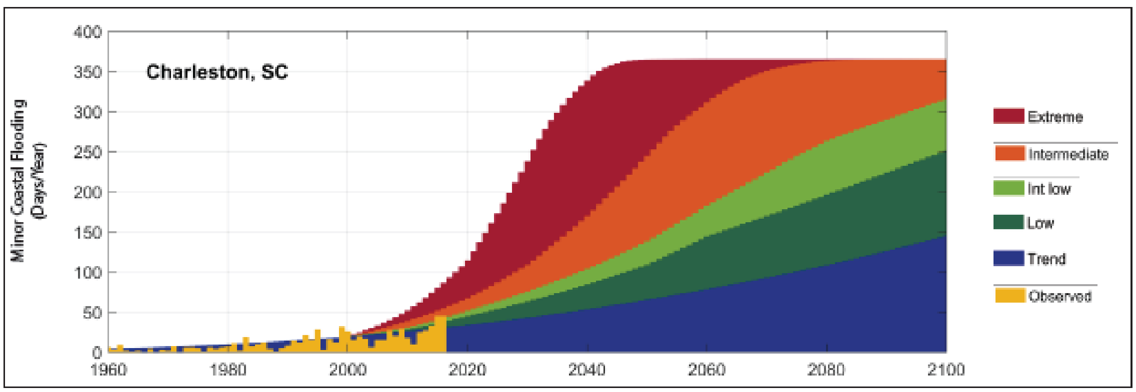

Figure 3. Observed and Predicted "Minor Coastal Flooding" in Charleston

1.2.2 CLIMATE TOOLS

In addition to a literature review, the vulnerability assessment includes a literature review and an application of climate tools to evaluate observed and projected climate trends. The following USACE CPR web-based tools were considered in the assessment:

1. NOAA's, Climate at a Glance, Time Series Tool
2. Climate Hydrology Assessment Tool (CHAT) – evaluate historic and projected climate trends.
3. Vulnerability Assessment Tool (VA) – provide qualitative information on projected climate conditions.
4. Nonstationary Detection Tool (NSD) – evaluate historic climate trends

The above tools are available on the USACE Climate Preparedness and Resilience CoP Applications web portal and NOAA's Climate Data Online.

The Nonstationary Detection Tool (NSD) does not contain stations within or upstream of the Charleston peninsula study area watersheds.

At this phase of the study (Feasibility), the VA tool has not been utilized. During PED phase, the PDT may work with USACE climate area experts for incorporating the tool into the assessment.

1.2.2.2 PRECIPITATION

Climate change can affect precipitation in many ways such as frequency, intensity, duration, or extremes. In initially scoping the observed trends, both the CWTS 2015-03 and the National Oceanic and Atmospheric Administration (NOAA) studies and tools were referenced. Figure 4 below displays the annualized precipitation data for Charleston, SC as provided using NOAA's "Climate at a Glance" time series tool. The tool depicts a slightly increasing trend in annual precipitation of +0.26 in/decade.

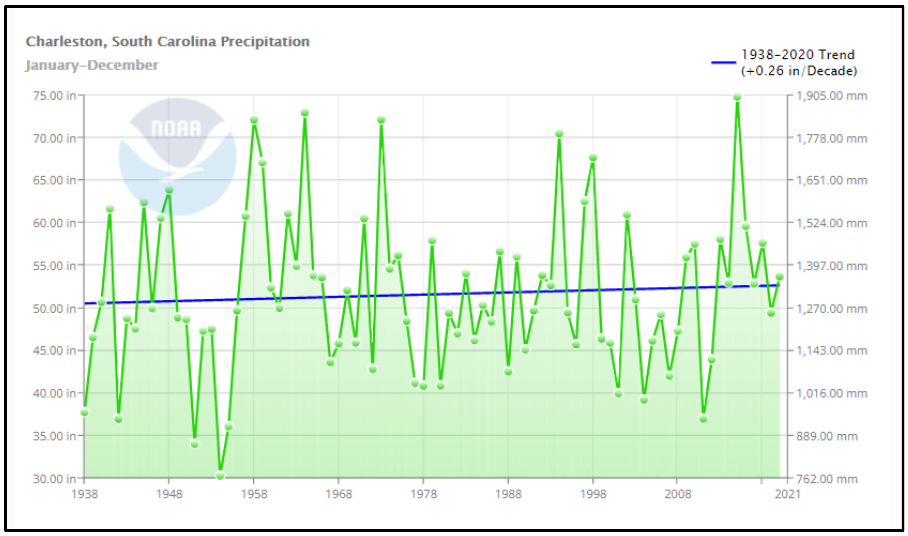

Figure 4. Precipitation Trend for Charleston, SC

As noted in CWTS 2015-03, the region is projected to experience a small increase in precipitation extremes.

The HEC-RAS model is utilizing NOAA's Atlas 14 precipitation frequency estimates with 90% confidence intervals. As mentioned in Section 3.4 of this report, Davis & Floyd Engineering (Charleston, SC) provided the rain on grid precipitation data for a 24-hour duration in increments of 6 minutes. The frequencies include the 50%, 10%, 4%, 2%, and 1% AEP rainfall events. Table 1 Section 3.4 displays the precipitation frequencies. These frequencies are coupled with various tidal boundary conditions to analyze the performance of future without-project and future with-project geometry conditions. A sensitivity analysis for varying rainfall duration intensities was completed and documented in Appendix 2 of this report. During PED phase, additional sensitivity analysis will be completed to further assess increased rainfall totals.

There are two important assumptions for the interior drainage modeling using HEC-RAS:
1. The City of Charleston's existing storm drainage system accommodates no more than a 10% AEP precipitation event with certain areas having lower capacities. The idea is that the USACE proposed pump stations will tie into the existing storm pipe network where appropriate. Due to the capacity of the existing drainage system at approximately a 10% AEP, surface flow becomes a larger component of drainage once the system becomes overwhelmed meaning the HEC-RAS model becomes a more appropriate condition of modeling.

2. The 2D HEC-RAS model does not have the capability to model the sub-surface drainage system, therefore it may be assumed that both future without project and future with project simulations produce conservative (higher than expected) water surface elevations.

1.2.2.3 RIVERINE HYDROLOGY

The Cooper River, Ashley River, and Wando River are the bounding rivers of the Charleston Peninsula that flow into the Charleston Harbor. The Cooper River which flows on the north side of the peninsula is controlled by an upstream hydropower facility at Lake Moultrie. The flood control structure at Lake Moultrie discharges flood releases into a different watershed, the Santee River watershed. Lake Moultrie does have an existing canal outlet utilized for navigation which drains into the West Branch Cooper River. Since flood releases from Lake Moultrie discharge into the Santee River watershed, there is no expectation to have a flooding incident on the peninsula due to riverine flooding stemming from the Cooper River.

FEMA does not document any riverine flooding on the Cooper, Ashley, or Wando rivers. All flooding incidences are primarily related storm surge-based flood risk, tidal events, or the overwhelming of the inland drainage system.

ECB 2018-14 provides tools such as the Climate Hydrology Assessment Tool (CHAT). "The Climate Hydrology Assessment Tool allows users to access data concerning past (observed) changes as well as potential future (projected) changes to relevant hydrologic inputs. This provides qualitative information about future climate conditions useful to decision-making officials and allows districts across the country to develop repeatable analytical results using consistent information. The tool reduces potential error while increasing the speed of information development so that data can be used earlier in the decision-making process, ideally in the development of risk registers." While CHAT is insightful for riverine projected conditions, it is the tidal gage at Charleston that most represents the water levels in the study area.

CHAT v1.0 contains information for the USGS Gage (02172002) Lake Moultrie Tailrace Canal at Moncks Corner, SC. This gage is roughly 40 plus river miles upstream of the Charleston Peninsula and is part of the navigation canal discharging into the West Branch Cooper River. The Lake Moultrie Tailrace Canal is primarily maintained for navigation and as mentioned the flood control project at Lake Moultrie discharges into a different watershed, the Santee River watershed therefore any increase to flood releases should be of minimal concern to the Cooper River watershed. Figure 6 shows the locations of the gage relative to Charleston, SC.

CHAT v2.0 contains information for the Ashley River. The tool produces trends in Mean Annual Max of Average Monthly Streamflow from 64 Climate-Changed Hydrology models. Figure 7 shows the trend in annual flow. While the flow is slightly increasing, the flow is small relative to tidal influences. Sensitivity Analyses were completed by scaling up riverine flow conditions while assuming a mean high tide. The sensitivity simulations show the riverine flows have little to no effect on exterior stages.

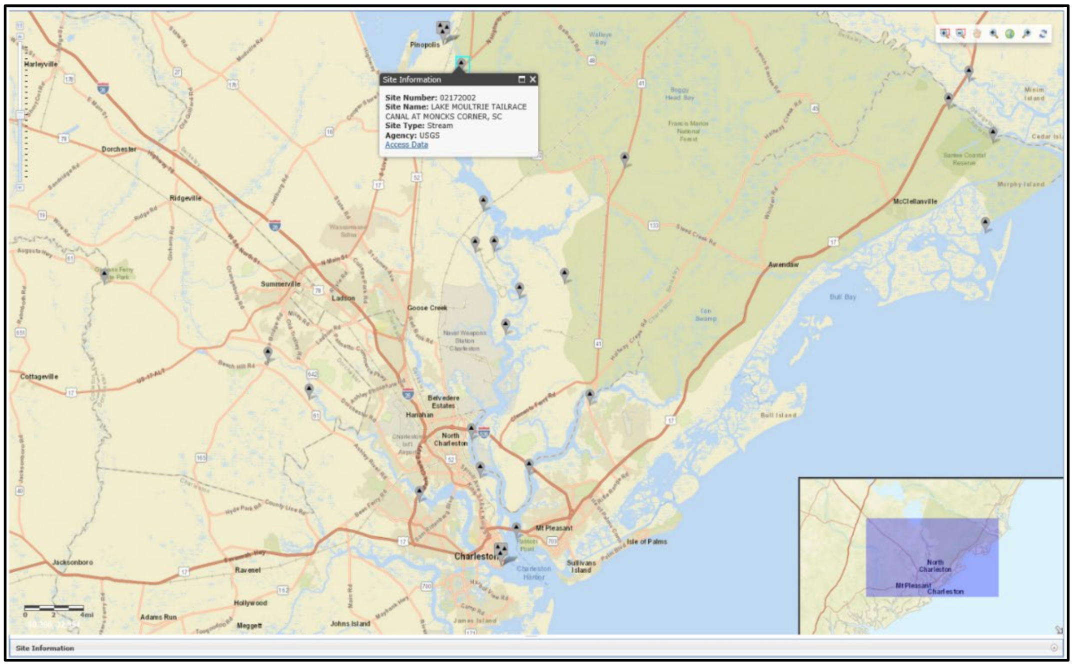

Figure 6. Lake Moultrie Tailrace Canal at Moncks Corner, SC

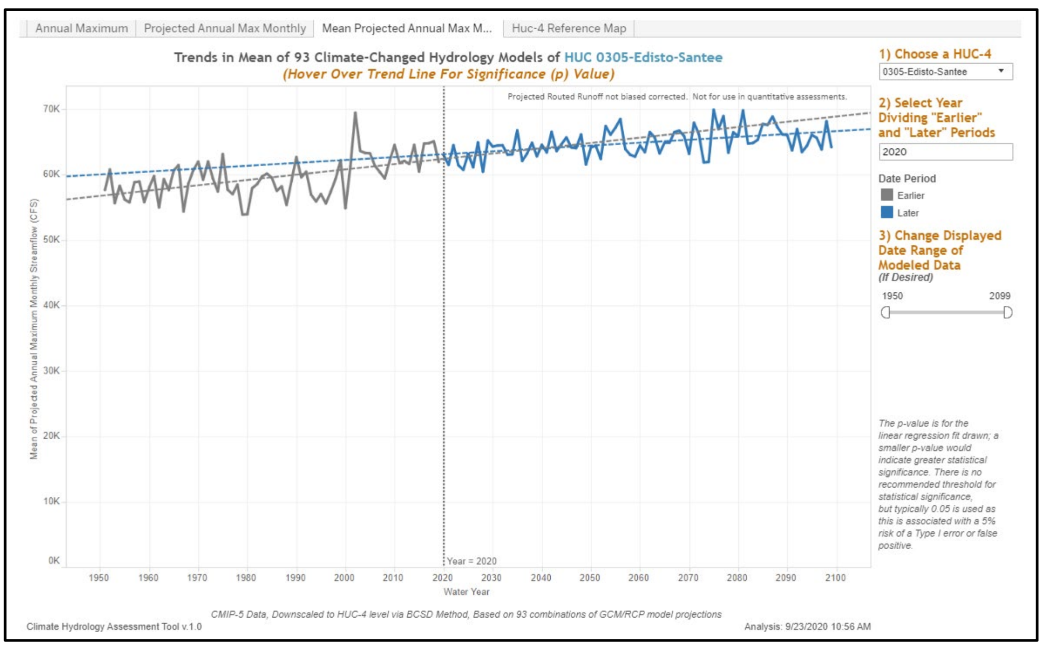

Figure 7. Projected Streamflow at Lake Moultrie Tailrace Canal USGS Gage (02172002)

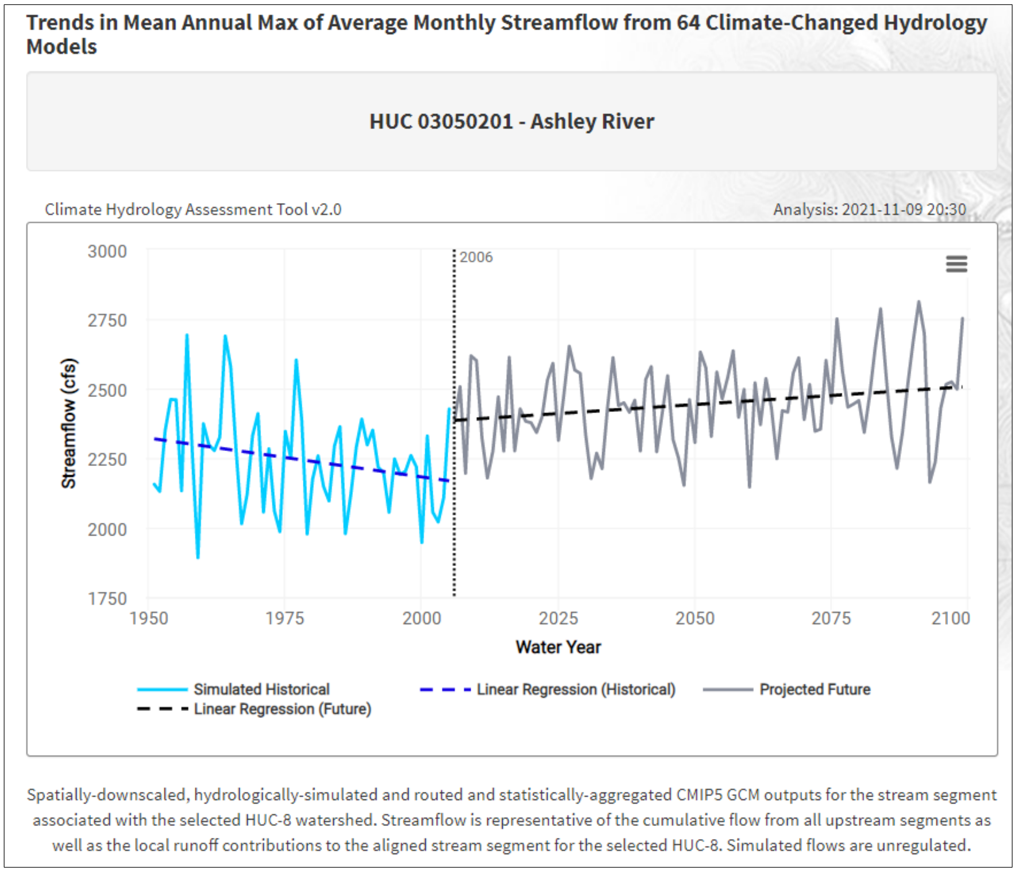

Figure 8. Projected Streamflow – HUC 03050201 – Ashley River

1.3 RISK ASSESSMENT

A review of the literature displays the project is most vulnerable to sea level rise, increases in air temperature, and increases in precipitation frequency and intensity. Per guidance in ECB 2018-14, Table 1 below identifies risks resulting from changing climate conditions in the future. The table shows the major project feature, the trigger event (climate variable that causes the risk), the hazard (resulting dangerous environmental condition), the harms (potential damage to the project or changed project output), and a qualitative assessment of the likelihood and uncertainty of this harm.

The review of existing conditions as reported, in Section 3.6 of the Engineering Appendix B, shows a relative sea level increase of approximately 1.07 feet over an approximate 100-year span since the establishment of the Charleston Harbor tide gage in 1899. The data also shows an increasing trend in observed minor coastal flooding beginning around 1960. The interior drainage model utilizes sea level rise rates for the tidal boundary conditions of +0.56 feet and +1.65 feet for the years 2032 and 2082, respectively, for future without-project and future with-project condition simulations.

Based on the knowledge of riverine hydrology, flood control discharges at upstream structures, and review of the literature, the project is not vulnerable to riverine hydrology nor significantly impacted by climate change but rather the inland flood elevations are more a function of the tide. The pattern of precipitation extremes are said to be generally increasing but not statistically significant. Similarly, a generally increasing trend in total annual precipitation is observed but without statistical significance.

Sensitivity analyses are completed by varying precipitation totals and further assessment of rainfall intensity is to be completed during PED phase.

Project benefits may change as a result of climate change effects to sea levels. In addition, project benefits may be impacted by climate change due to inland hydrology. Changes to benefits due to climate change may occur due to increases in flooding produced by sea level rise, or flooding produced by a combination of precipitation and sea level rise. There may be positive impacts to the project from increased air temperatures. The proposed hydrologic features for the project in relation to interior drainage include the storm surge wall, storm gates, and pump stations. The future resiliency and adaptation measures to the project as result of sea level rise and/or changes to precipitation frequencies/extremes could include:

- Increasing drainage capacity of existing gravity outlets and/or implementing new gravity outlets in collaboration with the City
- Increasing conveyance of stormwater pipes in collaboration with the city. The City of Charleston is already addressing the stormwater pipes by improving existing systems. See Section 2.3.
- Providing increased detention storage in low-lying tidal creek areas to aid in rainfall runoff storage by increasing or deepening the tidal creek areas which also increases conveyance capacities of said tidal creeks
- Increasing pump station capacities by adding pumps to existing stations or by deploying more temporary (mobile) pumps in addition those recommended as part of the TSP

<u>Other Considerations:</u>

- The primary goal of the PDT is not to address existing drainage issues faced by the City of Charleston but to ensure the proposed storm surge wall and interior drainage features do not induce residual interior flooding above the current level of flood protection. As mentioned throughout this report, the city’s storm pipe infrastructure accommodates no more than a 10% AEP rainfall event and surface flow becomes a larger component of drainage for greater events. Much of the projects interior drainage performance for smaller rainfall events depends on the City’s stormwater management systems therefore a deeper understanding of such functions are to be further addressed during PED phase.

- The PDT has declared that the TSP crest elevation is designed to not require adaptation to address future RSLC and other climate change effects. However, if the subject is reconsidered in the future there may be opportunities for temporarily adding height such as parapet walls. Assuming the parapet walls are temporary then parapet walls or other temporary measures could be implemented pending additional design analyses if raising the wall were to be revisited later in the project’s life.

- To combat sea level rise, the City of Charleston has implemented a check valve program to equip the peninsula tidal drains with flap gates to prevent tidal backflow. The city is also raising the Low Battery wall to be similar to the High Battery elevation. In addition to these measures, the city is continually increasing the integrity and performance of its interior drainage functions with the construction and re-service of pump stations and storm pipes. A summary of the Charleston Sea Level Rise Strategy is provided in Section 2.3.1.

- The city also indicates the plan to deploy temporary pumps with the appropriate notice of a storm surge event. USACE can collaborate with the city for developing best practices for deploying temporary pumps during flood fight efforts. These best practices may be implemented as emergency action plans and updated as necessary in response to the evolution of climate change to the project area conditions.

- USACE is currently dredging the navigation channel in the Cooper River. The dredging may increase conveyance capacities of the river and have some positive impacts on stages and flows in the area.

- Some rainfall sensitivity modeling was completed and documented in Appendix 2 of this report. Additional, rainfall sensitivity, including simulating increased rainfall totals above those currently being used, may be assessed to determine potential project impacts or to further refine drainage feature designs. The assessment will also provide additional insight for the possibility of additional resiliency and adaptation measures (additional from what is currently discussed) as well as providing a greater understanding of project performance if climate change due to inland hydrology and sea level rise were to occur.

-

Table 1. Climate Risk Register

Feature or Measure	Trigger	Hazard	Harm	Qualitative Likelihood
Flap Gated Peninsula Outfalls	Increased sea level	Increased water levels and wave heights seaward of the project. Limited discharge capacities of gravity outflow due to elevated exterior water levels above tops of pipe outfalls	Rainfall runoff may remain on the interior for longer durations and more frequently, potentially damaging the project and leading to more demand of pumps increasing maintenance and operational costs.	Likely
Storm Surge Design Elevation (12 ft. NAVD88)	Increased water levels from storm events (surge) due to sea level rise	Potential for overtopping of wall design elevation from wave overtopping or flood overtopping (SWL)	Increased SLR may increase frequency and magnitude of water level and wave loading on wall. Higher water levels that overtop design elevation causing a breach, flooding of interior areas and potential loss of life.	Moderate
Storm Gates	Increased frequency of extreme events	Increased water levels and wave heights seaward of storm gates	Increased SLR may increase frequency of storm gate closure, increasing operational costs. Frequency and magnitude of water level may increase.	Likely

Pump Stations	Increased sea level/Increased water levels from storm events (surge) due to sea level rise	Potential for wave overtopping and/or flood (SWL) overtopping	Potential of interior residual flood risks due to underperformance of interior project features leading to increased durations of interior flooding	Moderate
Pump Stations	Increased sea level	Increased water levels and wave heights seaward of the project. Limited discharge capacities of gravity outflow due to elevated exterior water levels above tops of pipe outfalls	More demand on pump stations, increasing maintenance and operational costs	Moderate

APPENDIX 2 - ADDITIONAL INTERIOR DRAINAGE MODELING

2.1 WAVE OVERWASH

2.1.1 EVENT COMPARISON MATRIX (WAVE OVERWASH)

HEC-RAS is used to assess future without-project and future with-project conditions during a storm surge. An overtopping analysis was conducted by the USACE Galveston (CESWG) Hydraulics Branch. The results from that assessment are used as inputs for the HEC-RAS model.

The HEC-RAS model applies storm surge as a tidal boundary condition for without- and with-project conditions. The results from the Galveston overtopping assessment found that a 2% AEP plus one wave amplitude in the year 2082 overtops the proposed storm surge wall at elevation 12 feet NAVD88. The 2% AEP storm surge correlates to approximately a 10-foot NAVD88 still water level. A constant 10-foot tide condition was applied to the HEC-RAS model for future without-project geometry while also assuming the suite of rainfall events. The 10-foot NAVD88 tide condition along with the calculated overtopping flow rates were applied to the future with-project geometry (Gates Closed – Pump Station Alternative 2) while also assuming the suite of rainfall events. The assessment uses pump station alternative 2 due to it being the Tentatively Selected Plan (TSP).

Performing this assessment within RAS 2D provides insight into the performance of the with-project condition during a storm event versus a storm surge for the without-project condition. The primary focus of the assessment is to demonstrate that the storm surge wall with wave overwash and rainfall is less damaging to the inland area than the storm surge event without a wall.

Another goal of this assessment is to assess the performance of pump station alternative 2 for the rainfall only events versus the rainfall plus wave overwash events for gates closed conditions. The comparison will use the results from the future with-project (gates closed) pump station alternative 2 (rainfall only) to compare to the results of the future with-project (gates closed) pump station alternative 2 (rainfall plus overtopping).

The following tables display the event comparison matrices for this part of the assessment. The two assessments for storm surge and wave overtopping are as follows:

1. Future without-project versus future with-project (gates closed) pump station alternative 2.
   - Storm surge tide conditions that would generate wave overwash for with-project conditions

FWO with Storm Surge	
FWO	
Exterior Elev	**Interior rainfall**
Elev 10.0 NAVD88	50% AEP rain
Elev 10.0 NAVD88	20% AEP rain
Elev 10.0 NAVD88	10% AEP rain
Elev 10.0 NAVD88	4 % AEP rain
Elev 10.0 NAVD88	2% AEP rain
Elev 10.0 NAVD88	1 % AEP rain

Closed System (Rainfall and Wave Overwash)	
FW (gates closed) P.S. alt 2	
Exterior Elev	**Interior rainfall**
Elev 10.0 NAVD88	50% AEP rain +OT
Elev 10.0 NAVD88	20% AEP rain + OT
Elev 10.0 NAVD88	10% AEP rain + OT
Elev 10.0 NAVD88	4 % AEP rain +OT
Elev 10.0 NAVD88	2% AEP rain +OT
Elev 10.0 NAVD88	1 % AEP rain +OT

2. Future with-project (gates closed) pump station alternative 2 (rainfall only) versus Future with-project (gates closed) pump station alternative 2 (rainfall and wave overwash)

   - Comparing pump performance for rainfall only versus rainfall and wave overwash.

Closed System (Rainfall Only)	
FW (gates closed) P.S. alt 2	
Exterior Elev	**Interior rainfall**
High tide	50% AEP rain
High tide	20% AEP rain
High tide	10% AEP rain
High tide	4 % AEP rain
High tide	2% AEP rain
High tide	1 % AEP rain

Closed System (Rainfall and Wave Overwash)	
FW (gates closed) P.S. alt 2	
Exterior Elev	**Interior rainfall**
Elev 10.0 NAVD88	50% AEP rain +OT
Elev 10.0 NAVD88	20% AEP rain + OT
Elev 10.0 NAVD88	10% AEP rain + OT
Elev 10.0 NAVD88	4 % AEP rain +OT
Elev 10.0 NAVD88	2% AEP rain +OT
Elev 10.0 NAVD88	1 % AEP rain +OT

2.1.2 SUMMARY OF OVERTOPPING ANALYSIS

Himangshu Das (CESWG), conducted a coastal modeling wave overtopping analysis using statistical Still Water Levels (SWL) and wave information to calculate wave overtopping flow using the EUROTOP method. This coastal modeling analysis provided overtopping flow rates and durations to be utilized as inputs into the 2D RAS model.

Tables 2 represents the estimated (2% AEP) peak overtopping flow rates. Figure 9 represents the estimated (2% AEP) peak overtopping flow rates placed on a map providing a visual representation of the flow rates at locations around the peninsula.

The provided overtopping rates in Table 2 were incorporated into the RAS model as boundary condition lines on the interior of the wall alignment. The representing overtopping rates were multiplied by the floodwall lengths in which they overtop to calculate the total cubic feet per second flowing over the wall. The overtopping was set in the RAS model to occur for approximately 3 hours and was modeled as a hydrograph to simulate the variable overtopping rate. This duration assumption comes from the provided hydrograph shown in Figure 11.

The coastal modeling analysis divided the peninsula into 3 regions: Western Region where wave energy is low, Southern tip where wave energy is relatively moderate, and Eastern Region where wave energy is low to moderate.

Table 2. Overtopping Flow Rates at Different Regions

Reaches & Stations	Peak Overtopping Flow (CFS/FT)
Western Region (stations 1, 2, 8, 9)	0.006
Southern tip (Stations 4, 6, 7)	0.013
Eastern Region (3, 5)	0.009

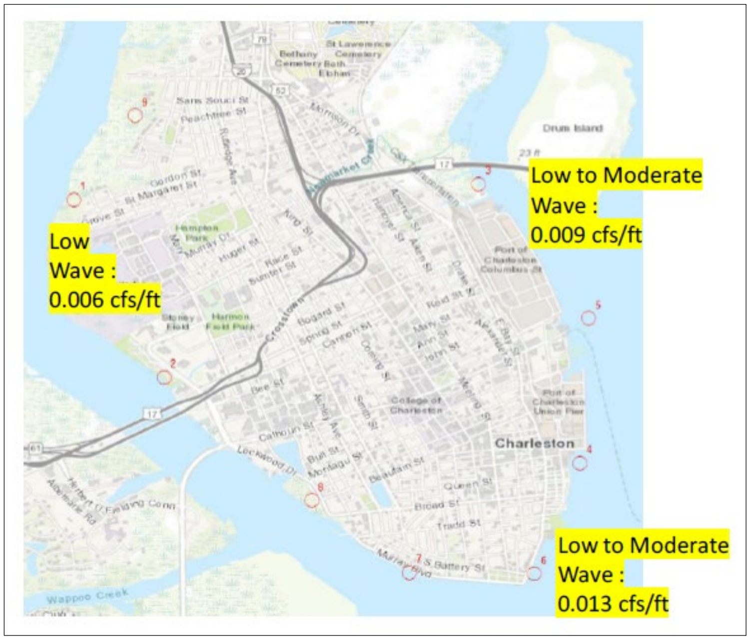

Figure 9. Overtopping flow along different reaches

The coastal model provided the Annual Exceedance Probability (2% in this case) at which point the SWL considering Relative Sea Level Change (RSLC) plus one wave amplitude exceeds the flood wall height of 12 ft. NAVD88. Figure 9 shows the wave overtopping flow calculated at Station 6 (2% AEP). Station 6 is located near the Battery. The coastal modeling report stated this occurred at roughly a 2% AEP. In the year 2082, a 2% AEP SWL is approximately at elevation 10 ft. NAVD88. The 10ft. SWL was used as the stage boundary condition within the RAS model when assessing overtopping.

The coastal model followed Hurricane & Storm Damage Risk Reduction System (HSDRRS) guidelines in the overtopping assessment. HSDRRS guideline provides allowable average wave overtopping rates for the 1% AEP SWL, wave height, and wave period. Those allowable values are 0.1 cfs/ft. at 90% level of assurance and 0.03 cfs/ft. at 50% level of assurance. As seen in Table 2, Station 6 produced an overtopping flow rate of 0.013 cfs/ft. which is well below the HSDRRS limit state. Although overtopping flows are negligible and do not exceed limit state as analyzed by the coastal modeling, the 1% AEP overtopping flow rates were provided and used within the interior drainage modeling.

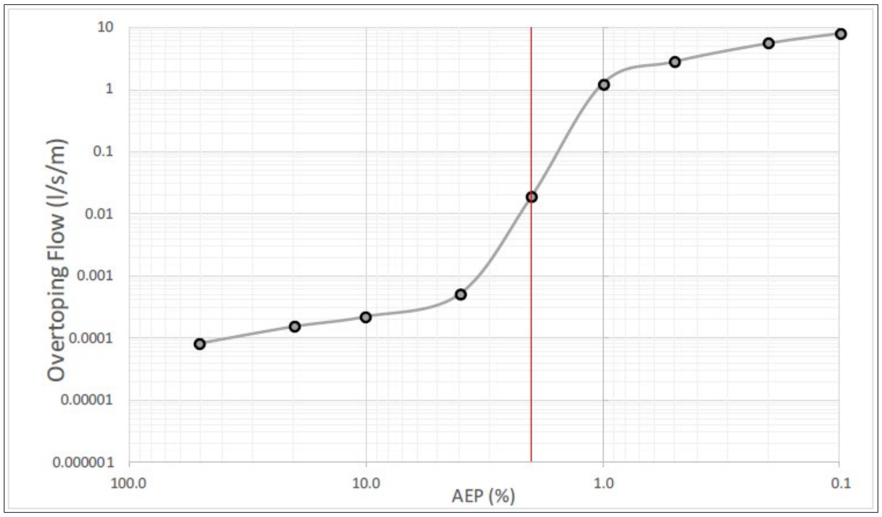

Figure 10. Overtopping Flow Calculated at Station 6

Figure 11 displays the duration of overtopping calculated at Station 6 as provided by the coastal modeler. The HEC-RAS model applies approximately 3 hours of overtopping flows into the interior area in the shape of the hydrograph. See below.

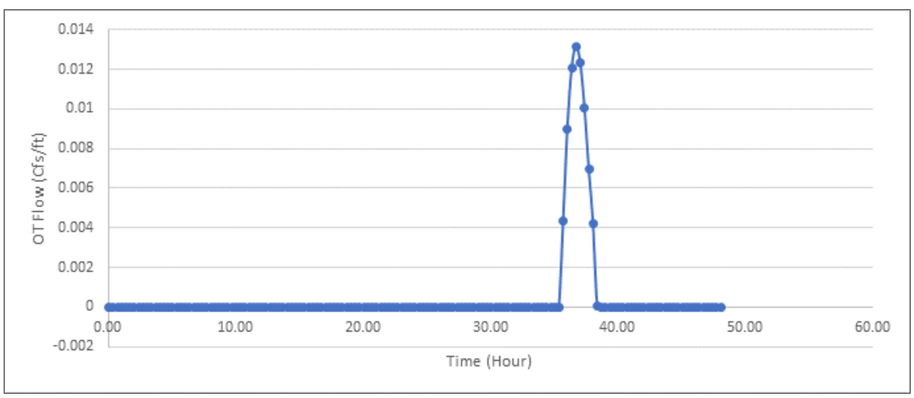

Figure 11. Duration of Overtopping Calculated at Station 6

2.1.3 HEC-RAS INPUTS

The HEC-RAS model applies interior 2D boundary condition lines to account for wave overwash flows for the gates closed assessment while also applying the rainfall conditions (rainfall plus wave overwash). The overtopping flows presented in the previous section for the wester, southern, and eastern region are appropriately applied around the peninsula on the interior of the wall. Table 3 and Figure 10 display the peak overtopping flows for each boundary condition line and the locations of the boundary condition lines.

The peak overtopping flow rates in cfs/ft. were multiplied by the total length (ft.) of the boundary condition lines in Table 3 to be applied to the interior 2D within the RAS model. This multiplication provides the total cfs applied to the boundary condition lines.

Table 3. Boundary Condition Lines for Overtopping Flows

RAS BC Line (Region)	RAS BC Line Length (FT.)	Peak Overtopping Flow (CFS)
BCpt9and1 (Western Region)	8,799.18	52.80
BCpt8and2 (Western Region)	10,222	61.33
BCpt6and7 (Southern Tip)	6,083.91	80.28
BCpt6and7(2) (Southern Tip)	378.1	4.99
BCpt6and7(3) (Southern Tip)	301.16	3.97
BCpt4 (Southern Tip)	3,671.21	48.44
BCpt3and5 (Eastern Region)	15648.4	140.84

Figure 10. Internal BC Lines for Wave Overwash

As mentioned in the previous section, the overtopping flows were applied to the HEC-RAS model in hydrograph form. The overtopping flow hydrograph in Figure 11 is an example for the flow applied at BC Line "BCpt6and7" and was assumed to start near the peak curve of the rainfall hyetograph also being applied to the model. This approach assumes coincidental peak rainfall and wave overwash occurring. I.e., as mentioned previously in the report the rainfall hyetograph has a duration of 24 hours with the peak rainfall occurring at the 12th hour of the hyetograph. The wave overwash flows are set to begin around hour 10 of the model simulation and set to peak between hour 11 and 12 applying almost coincidental peaks of the rainfall and flow conditions.

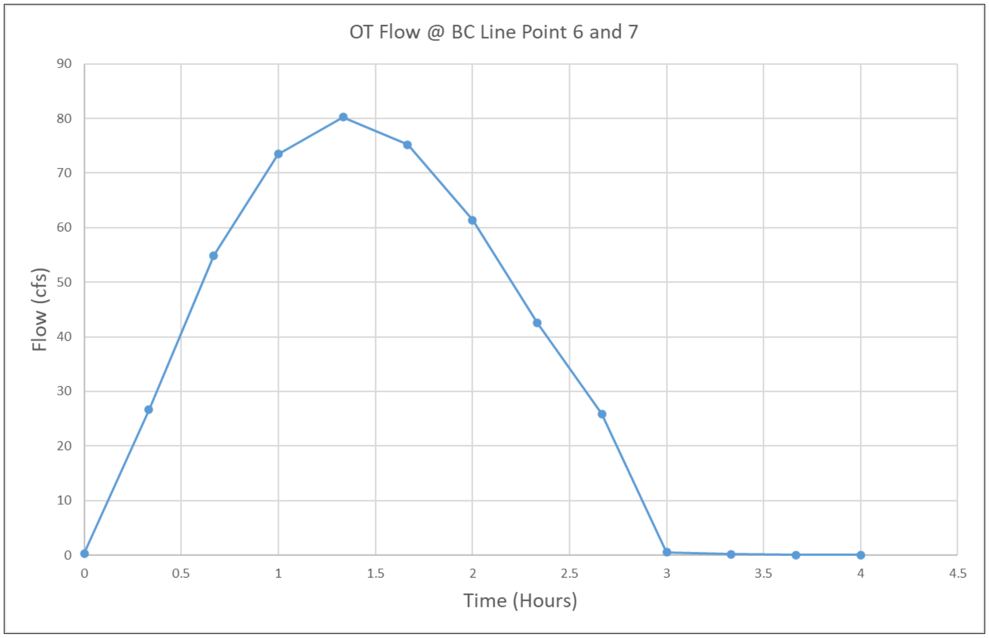

Figure 11. Example of Wave Overwash Hydrograph for BC Line Point 6 and 7 (Southern Tip)

2.1.4 HEC-RAS WAVE OVERWASH RESULTS

Only the 10% AEP rain and 1% AEP rain conditions with storm surge are documented to condense the results section for this assessment.

2.1.4.1 FWO (STORM SURGE) vs FWP (GATES CLOSED – RAINFALL AND WAVE OVERWASH)

This section displays the results for future without-project conditions with storm surge (10 ft. NAVD88 Still Water Level) and the 10% AEP Rain and 1% AEP Rain. Section 2.1.4.1 also displays the results for the future with-project (gates closed) conditions for the 10% AEP and 1% AEP rainfalls while applying wave overwash. As previously mentioned, the primary focus of this part of the assessment is to demonstrate that the storm surge wall with wave overwash and rainfall is less damaging to the inland area than a storm surge event occurring without a wall.

Reviewing the results show the storm surge wall greatly reduces the inland flood elevations during a wave overwashing event with rainfall when compared to the future without-project for the same storm surge and rainfall event.

Table 4. Future Without-Project Conditions with Storm Surge

Selected Output Locations	FWO 10ft NAVD88 Still Water	
	10% AEP Rain	1% AEP Rain
	Peak Water Surface Elevation (ft. NAVD88)	Peak Water Surface Elevation (ft. NAVD88)
1	10.02	10.02
2	10.02	10.02
3	10.03	10.05
4	10.05	10.09
5	10.02	10.03
6	10.02	10.03
7	10.03	10.04
8	10.02	10.03
9	10.02	10.03
10	10.02	10.03
11	10.02	10.03
12	10.02	10.03
13	10.02	10.03
14	10.02	10.03
15	10.02	10.03
16	10.02	10.03
17	10.02	10.03
18	10.03	10.05
19	10.03	10.05
20	10.03	10.05
21	10.03	10.05
22	10.02	10.03
23	10.03	10.04
24	10.02	10.03
25	10.02	10.03
26	10.02	10.03
27	10.03	10.07

Table 5. Future With-Project Conditions Pump Station Alternative 2 – Wave Overwash

Selected Output Locations	Nearest Drainage Feature Influence (City PS/USACE PS)	**FW (gates closed) City Pumps Active/ USACE Pump Station Alternative 2**			
		10% AEP Rainfall and Wave Overwash		1% AEP Rainfall and Wave Overwash	
		Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)	Peak Water Surface Elevation (ft. NAVD88)	Difference from without project condition (ft.)
1	**NA**	8.08	-1.94	8.68	-1.34
2	**Halsey PS**	3.78	-6.24	7.48	-2.54
3	**Halsey PS**	5.37	-4.66	7.48	-2.57
4	**Halsey PS**	6.40	-3.65	7.48	-2.61
5	**Halsey PS**	6.73	-3.29	7.48	-2.55
6	**Joe Riley PS**	4.24	-5.78	6.38	-3.65
7	**Joe Riley/SF PS**	5.74	-4.29	6.38	-3.66
8	**SF PS**	5.74	-4.28	6.36	-3.67
9	**MUSC PS**	4.51	-5.51	4.99	-5.04
10	**City Marina PS**	4.83	-5.19	5.60	-4.43
11	**City Marina PS**	5.11	-4.91	5.67	-4.36
12	**Potential Temp PS**	4.84	-5.18	5.63	-4.40
13	**Battery 1 PS**	4.70	-5.32	5.63	-4.40
14	**Battery 1 PS**	4.72	-5.30	5.63	-4.40
15	**Battery 2 PS**	4.90	-5.12	5.78	-4.25
16	**Battery 3 PS**	6.35	-3.67	6.99	-3.04
17	**Waterfront Park PS**	4.67	-5.35	7.20	-2.83
18	**Waterfront Park PS**	6.62	-3.41	7.24	-2.81
19	**Concord St. PS**	5.35	-4.68	5.82	-4.23
20	**Concord St. PS**	5.88	-4.15	6.32	-3.73
21	**Concord St. PS**	4.92	-5.11	5.42	-4.63
22	**Port 1 PS**	6.15	-3.87	6.27	-3.76
23	**Port 2 PS**	5.68	-4.35	6.31	-3.73
24	**Port 2 PS**	3.87	-6.15	6.28	-3.75
25	**Port 2 PS**	4.16	-5.86	6.28	-3.75
26	**Newmarket PS**	4.13	-5.89	6.92	-3.11
27	**Newmarket PS**	5.83	-4.20	6.93	-3.14

1. The "Difference from without-project condition" shows if the with-project decreases or increases the water surface elevation.
2. Positive (+) means the project increases the wsel where negative (-) means the project decreases the wsel.

2.1.4.2 FWP (GATES CLOSED – RAINFALL ONLY) vs FWP (GATES CLOSED – RAINFALL AND WAVE OVERWASH)

Table 6. Pump Station Alternative 2 – Rainfall Only vs Rainfall and Wave Overwash

Selected Output Locations	Nearest Drainage Feature Influence (City PS/USACE PS)	FW (gates closed) City Pumps Active/ USACE Pump Station Alternative 2		
		10% AEP Rainfall Only	10% AEP Rainfall and Wave Overwash	Rainfall and Wave Overwash minus Rainfall Only
		Peak Water Surface Elevation (ft. NAVD88)	Peak Water Surface Elevation (ft. NAVD88)	Difference (feet)
1	**NA**	8.08	8.21	0.13
2	**Halsey PS**	3.78	4.42	0.64
3	**Halsey PS**	5.37	5.51	0.14
4	**Halsey PS**	6.40	6.40	0.00
5	**Halsey PS**	6.73	6.73	0.00
6	**Joe Riley PS**	4.24	4.41	0.17
7	**Joe Riley/SF PS**	5.74	5.75	0.01
8	**SF PS**	5.74	5.75	0.01
9	**MUSC PS**	4.51	4.56	0.05
10	**City Marina PS**	4.83	4.86	0.03
11	**City Marina PS**	5.11	5.11	0.00
12	**Potential Temp PS**	4.84	4.88	0.04
13	**Battery 1 PS**	4.70	4.78	0.08
14	**Battery 1 PS**	4.72	4.79	0.07
15	**Battery 2 PS**	4.90	5.27	0.37
16	**Battery 3 PS**	6.35	6.42	0.07
17	**Waterfront Park PS**	4.67	6.09	1.42
18	**Waterfront Park PS**	6.62	6.66	0.04
19	**Concord St. PS**	5.35	5.43	0.08
20	**Concord St. PS**	5.88	5.88	0.00
21	**Concord St. PS**	4.92	5.00	0.08
22	**Port 1 PS**	6.15	6.15	0.00
23	**Port 2 PS**	5.68	5.77	0.09
24	**Port 2 PS**	3.87	4.23	0.36
25	**Port 2 PS**	4.16	4.26	0.10
26	**Newmarket PS**	4.13	4.53	0.40
27	**Newmarket PS**	5.83	5.83	0.00

1. The "Difference" column shows the "Rainfall and Wave Overwash" WSEL minus the "Rainfall Only" WSEL.
2. Positive (+) means the rainfall and wave overwash increases the wsel where negative (-) means the rainfall plus wave overwash

Denotes a > 0.5 ft. increase induced by the wave overwash.

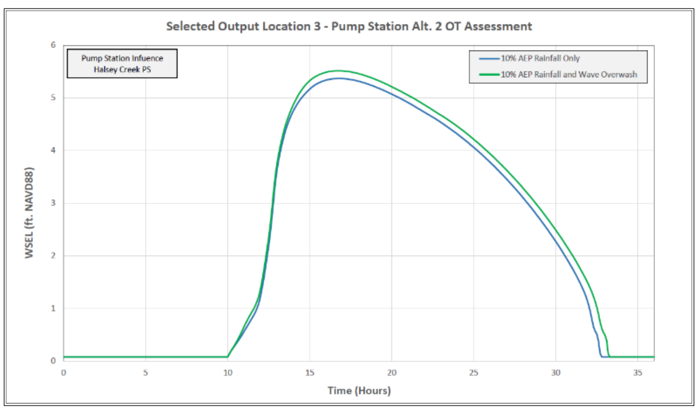

Figure 12. Pump Station Assessment – Rainfall Only vs Rainfall and Wave Overwash – Output Location 3

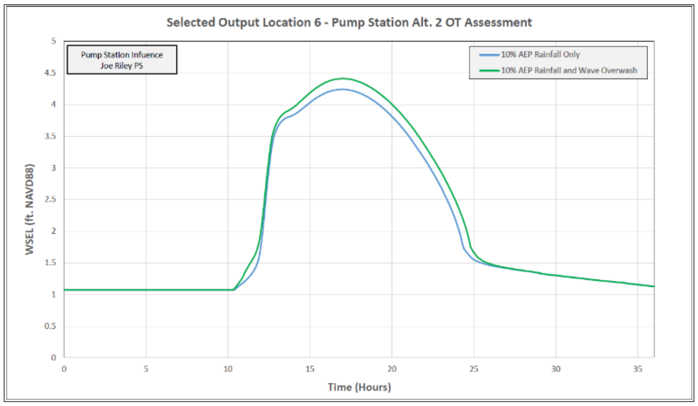

Figure 13. Pump Station Assessment – Rainfall Only vs Rainfall and Wave Overwash – Output Location 6

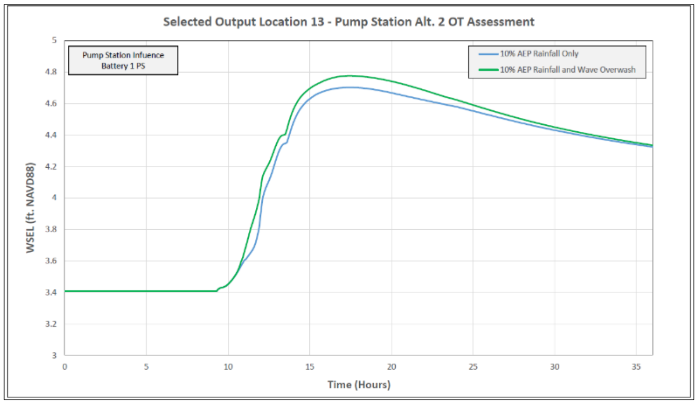

Figure 13. Pump Station Assessment – Rainfall Only vs Rainfall and Wave Overwash – Output Location 13

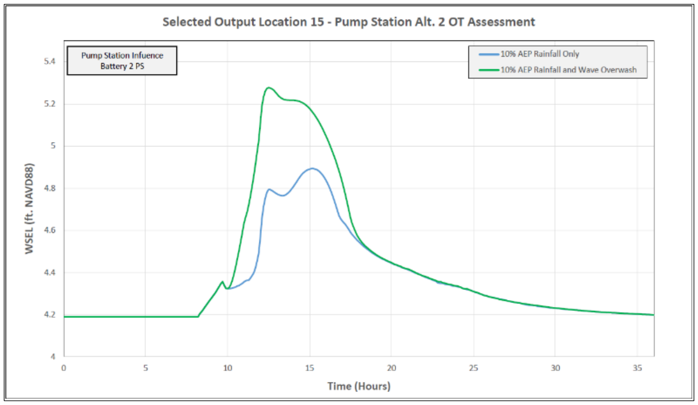

Figure 14. Pump Station Assessment – Rainfall Only vs Rainfall and Wave Overwash – Output Location 15

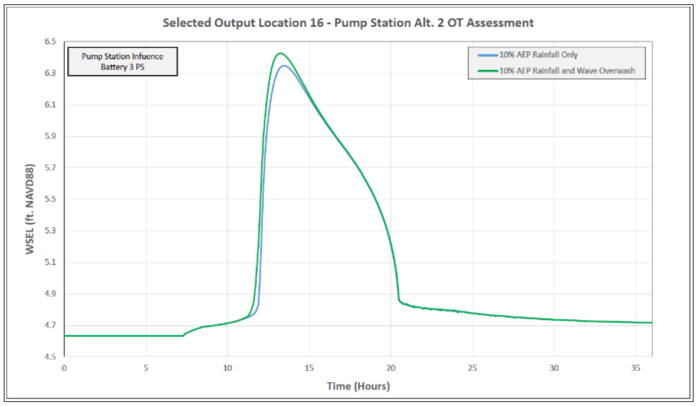

Figure 15. Pump Station Assessment – Rainfall Only vs Rainfall and Wave Overwash – Output Location 16

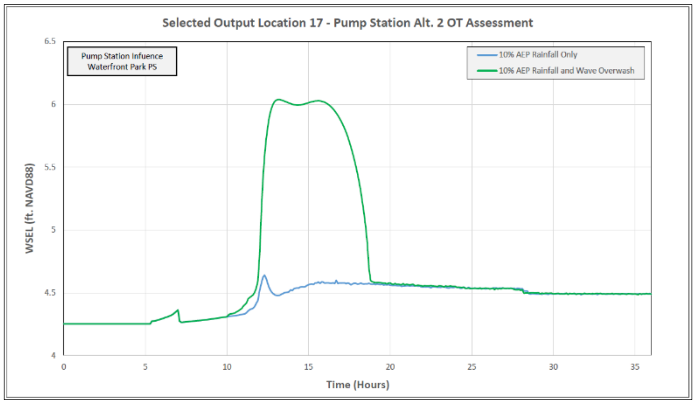

Figure 16. Pump Station Assessment – Rainfall Only vs Rainfall and Wave Overwash – Output Location 17

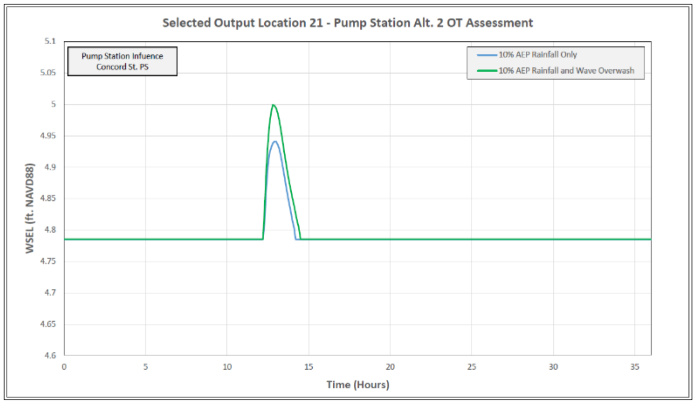

Figure 17. Pump Station Assessment – Rainfall Only vs Rainfall and Wave Overwash – Output Location 21

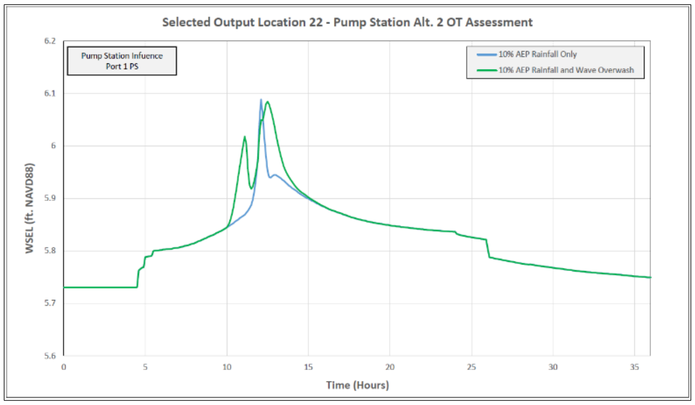

Figure 18. Pump Station Assessment – Rainfall Only vs Rainfall and Wave Overwash – Output Location 22

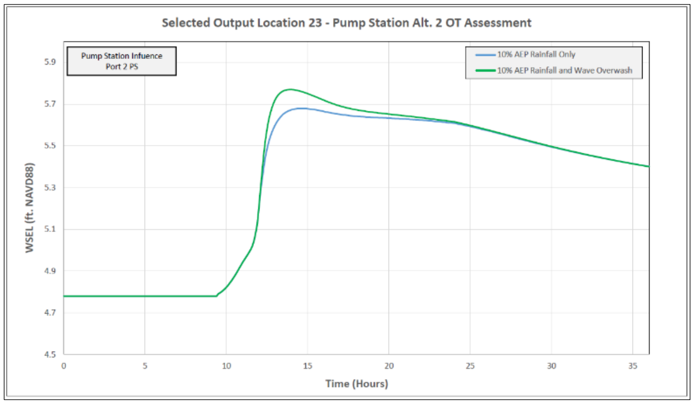

Figure 19. Pump Station Assessment – Rainfall Only vs Rainfall and Wave Overwash – Output Location 23

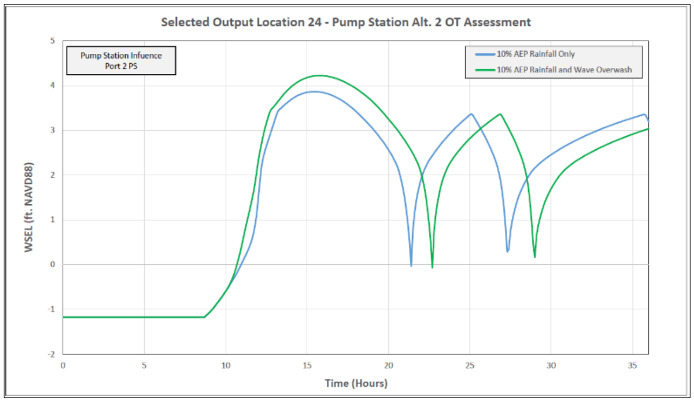

Figure 20. Pump Station Assessment – Rainfall Only vs Rainfall and Wave Overwash – Output Location 24

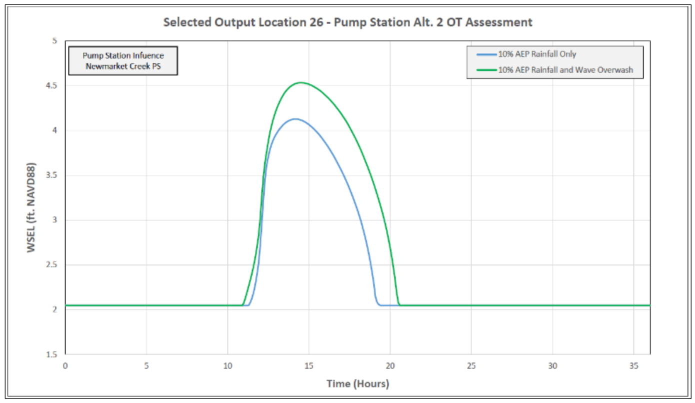

Figure 21. Pump Station Assessment – Rainfall Only vs Rainfall and Wave Overwash – Output Location 26

2.2 NWS – MAJOR FLOOD STAGE

The National Weather Service indicates major flooding occurs at 8 ft. (MLLW) which equates to 4.86 ft. (NAVD88). At present day, 4.86 ft. NAVD88 is approximately a 50% AEP Still Water Level. Projecting these conditions to the year 2032, RAS scenarios were computed for future without-project conditions and future with-project conditions with the storm gates closed. A stage boundary condition of 5.42 feet NAVD88 which includes +0.56 feet of sea level rise for 2032, was applied to the exterior 2D mesh boundary condition line for both the future without and future with conditions. No rainfall was included in these simulations to depict only the flooding occurring from the NWS Major Flood Stage.

Table 7. National Weather Service Flood Categories

Flood Categories	MLLW (ft.)	NAVD88 (ft.)	*NAVD88 (ft.) Year 2032
Action Stage	6.5	3.36	3.92
King Tide	6.6	3.46	4.02
Minor Flooding	7.0	3.86	4.42
Moderate Flooding	7.5	4.36	4.92
Major Flooding	8.0	4.86	5.42
*The elevations for the year 2032 were simply estimated by adding 0.56 feet to the current elevations.			

NWS Flood Impacts

- At 8.0 ft MLLW (4.86 ft. NAVD88), major coastal flooding occurs. Widespread flooding occurs in Downtown Charleston with numerous roads flooded and impassable and some impact to structures. Impacts become more extensive all along the southeast South Carolina coast including erosion at area beaches, with limited or no access to docks, piers, and some islands.

- At 7.5 ft MLLW (4.36 ft. NAVD88), moderate coastal flooding occurs. In Downtown Charleston, additional impacted roads include HW-17 at HW-61, Market Street, East Bay, Rutledge, and areas around MUSC. Other impacted areas include Long Point Road near Palmetto Islands County Park, locations around the Naval Complex, 12th and 15th Streets on Isle of Palms, and the road leading to Bohicket Marina on Seabrook Island. In Beaufort County, flooding will impact Hunting Island and the Sea Island Parkway near Chowan Creek Bridge.

- At 7.0 ft MLLW (3.86 ft. NAVD88), minor coastal flooding typically begins. Minor flooding on roadways around Downtown Charleston occurs, possibly including Lockwood Drive, Wentworth and Barre, Fishburne and Hagood, and Morrison Drive. As the tide height approaches 7.5 ft MLLW, roads can become impassable and closed. Other impacts outside of Downtown Charleston include minor flooding of low-lying locations near area beaches including Isle of Palms, Sullivan's Island, Folly Beach, Kiawah Island, and Edisto Island.

Figure 25 displays the modeled inundations for the major flood stage event. This event will display the potential inundation difference between the future without-project and the future with-project in the year 2032 if the NWS major flood stage were to occur with the sea level change rate applied. There are significant uncertainties in estimating the evolution of future storm events, storm surge, and the impacts of relative sea level change.

The inundation-colored red is depicting the future without-project inundation. This inundation shows widespread flooding all along the west side of the peninsula and flooding on the east side of the peninsula in the areas of the Waterfront Park, the Port, and Newmarket Creek.

The inundation-colored blue is depicting the future with-project inundation with gates closed. No inundation is shown on the interior of the wall while many of the areas outside of the wall are impacted. Rainfall data was not included in this computation; therefore, the computed inundation is only a result of the stage hydrographs.

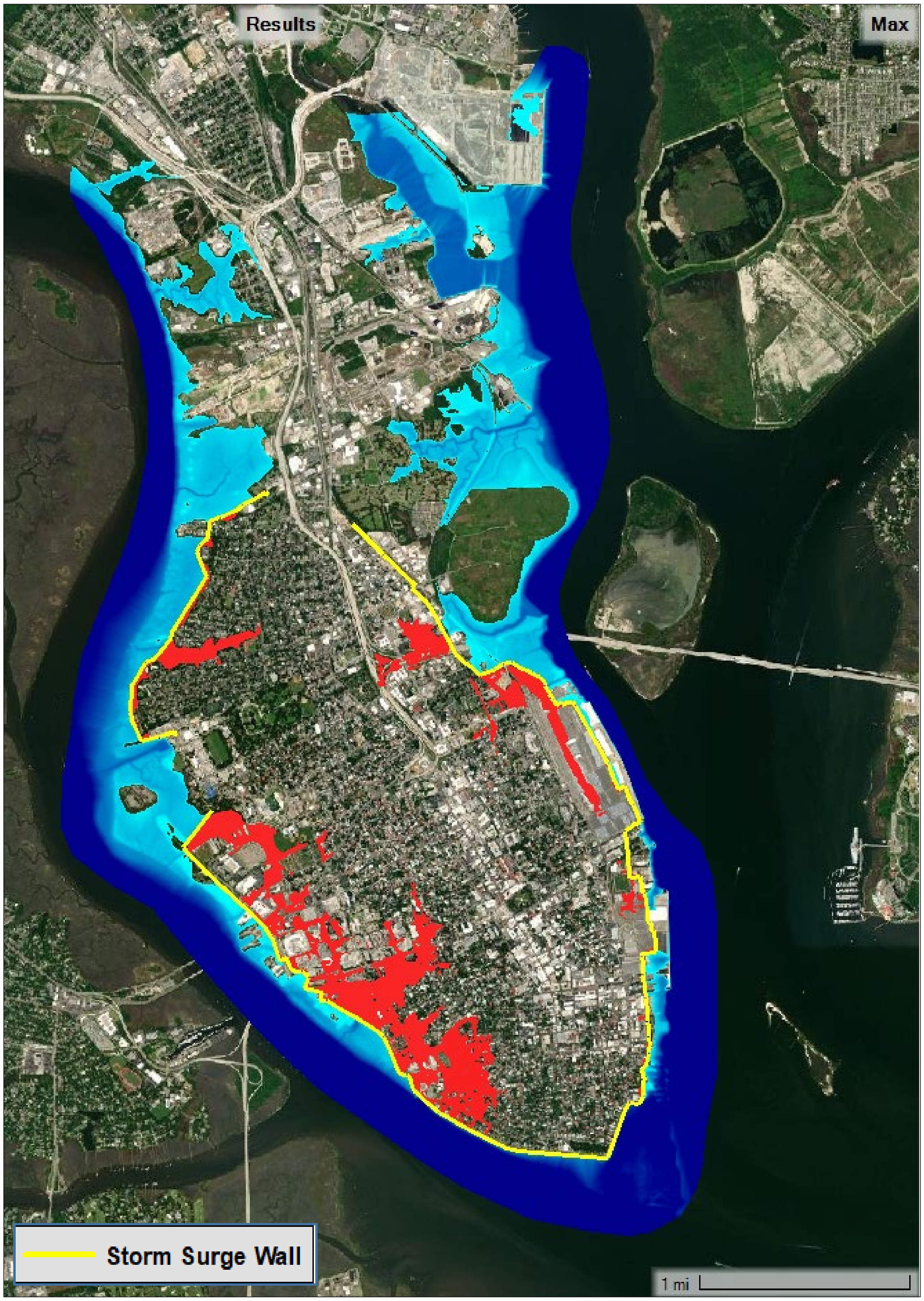

Figure 22. NWS Denoted Major Flooding Event – 2032 with SLR – FWO (Red) vs FWP (Blue)

2.3 STORM GATE - WEIR COEFFICIENT SENSITIVITY

This section was focused on the storm gate closures at Halsey Creek and the creek near the Port. The storm gates were modeled with SA/2D connection gate features. The gated weir coefficient was varied from 0.5 to 2 to test the efficiency of the storm gates and assess the change in interior water surface elevation. The greater the weir coefficient the more efficient the discharges through the gate.

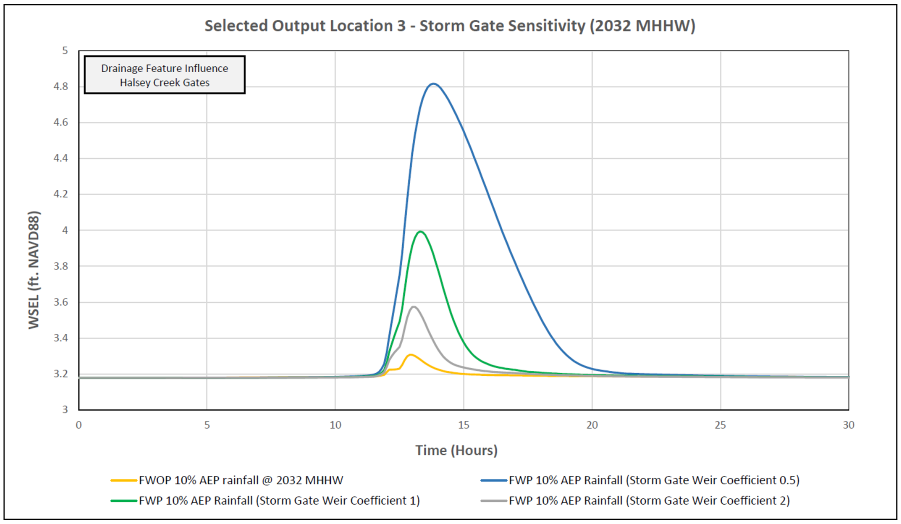

Figure 23. Output Location 3 – Storm Gate Weir Coefficient Sensitivity

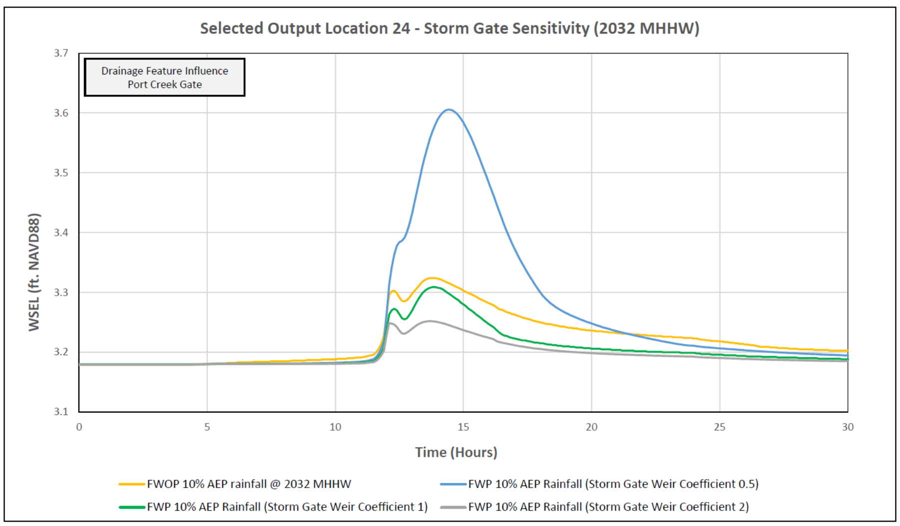

Figure 24. Output Location 24 – Storm Gate Weir Coefficient Sensitivity

2.4 PUMP STATION – INITIAL INTERIOR ELEVATION SENSITIVITY

This section assesses the performance of pump stations during varying initial interior water elevations. There are changes that storm gates are not closed at low tide and therefore higher interior water elevations may be present during the arrival of a storm surge. The current average high tide in Charleston is 5.5 feet MLLW or 2.36 feet NAVD88. This elevation was used as an initial interior condition for pump sensitivity. In addition, the MHHW is currently 2.62 feet NAVD88 and adjusting this elevation for the year 2032 could be estimated using the 0.56 feet sea level rise rate to equate to an elevation of 3.18 feet NAVD88. This elevation was used as another interior elevation for initial conditions. These scenarios were compared against the design scenario reported in earlier sections of the report where the system assumes storm gates are closed and therefore the initial condition reads the terrain elevation.

As expected, the sensitivity shows the greater the initial interior elevation the less efficient the pump stations. Therefore, if a high wind event occurs that pushes tide into the interior before gates are closed then pump performance may not be as efficient as times where storm gates are closed as planned (low tide).

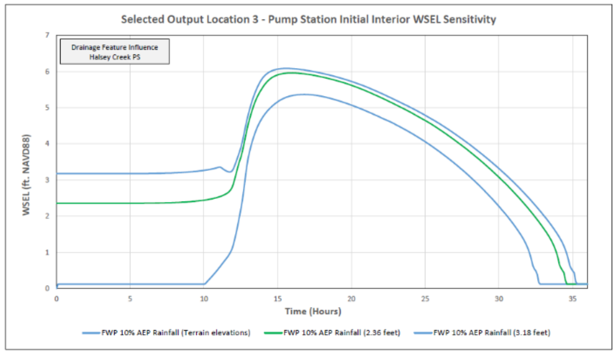

Figure 25. Output Location 3 – Initial Interior Sensitivity

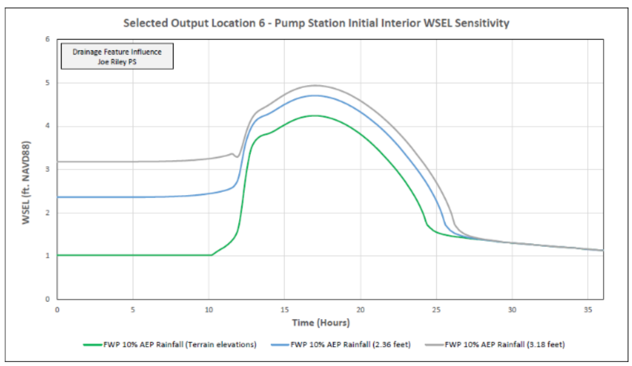

Figure 26. Output Location 6 – Initial Interior Sensitivity

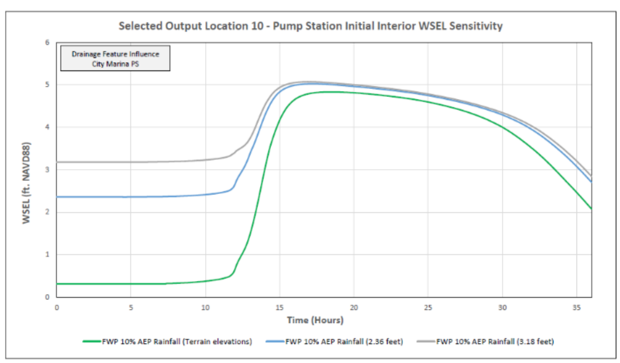

Figure 27. Output Location 10 – Initial Interior Sensitivity

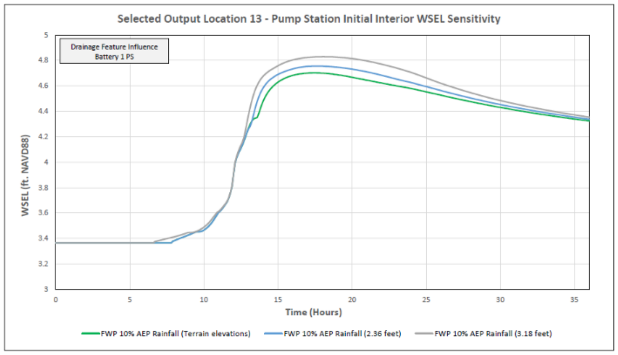

Figure 28. Output Location 13 – Initial Interior Sensitivity

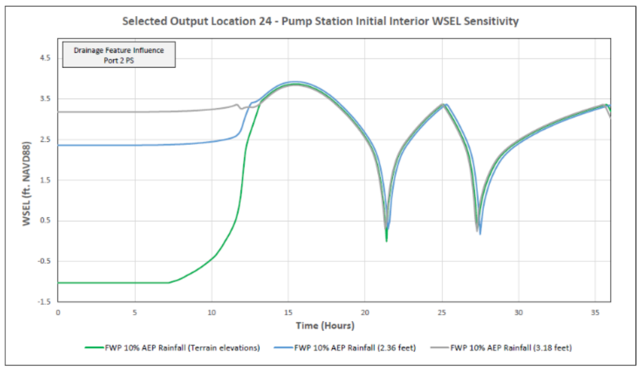

Figure 29. Output Location 24 – Initial Interior Sensitivity

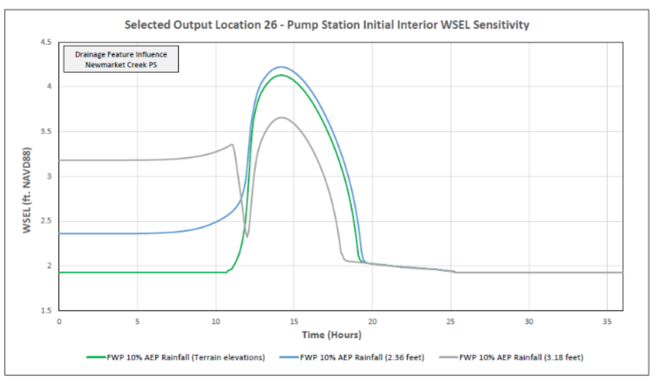

Figure 30. Output Location 26 – Initial Interior Sensitivity

2.5 PUMP STATION – RAINFALL DURATION SENSITIVITY

This sensitivity adjusts the duration of rainfall for the 10% AEP from 24 hours to 12, 6, and 3 hours. The 24 hours 10% AEP rainfall time-series was scaled in excel and then simulated in RAS. The sensitivity is not an exact estimation of rainfall durations but provides insightful information for varying rainfall durations. The 10% AEP rainfall for the various durations are provided from NOAA Atlas 14, then the rainfall runoff-time series spreadsheet for the 24-hour storm is used to scale to the other durations.

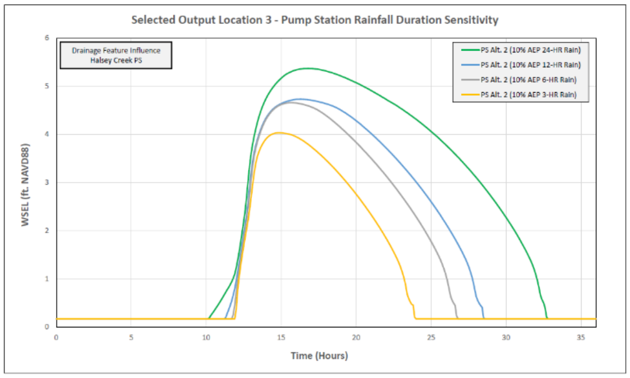

Figure 31. Output Location 3 – Rain Duration Sensitivity

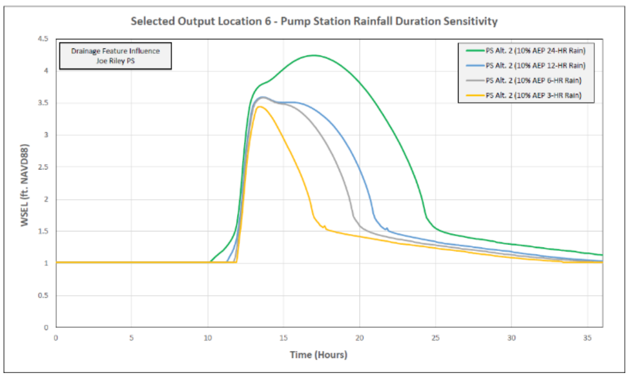

Figure 32. Output Location 6 – Rain Duration Sensitivity

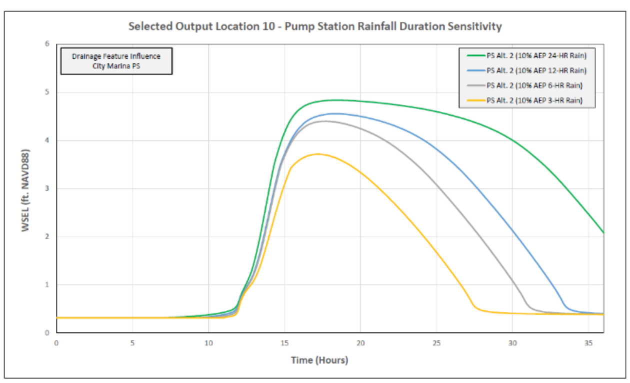

Figure 33. Output Location 10 – Rain Duration Sensitivity

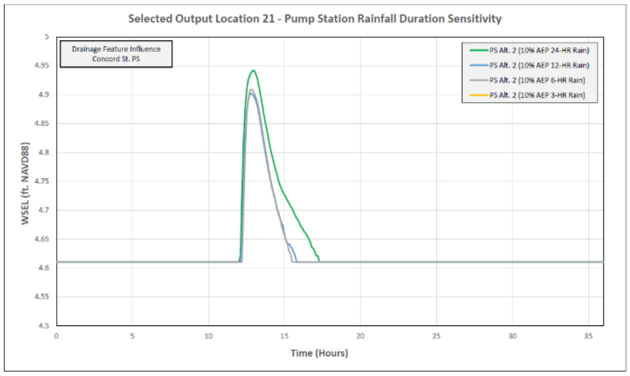

Figure 34. Output Location 21 – Rain Duration Sensitivity

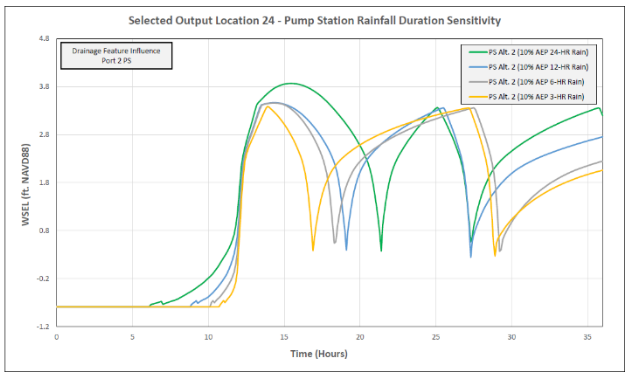

Figure 35. Output Location 24 – Rain Duration Sensitivity

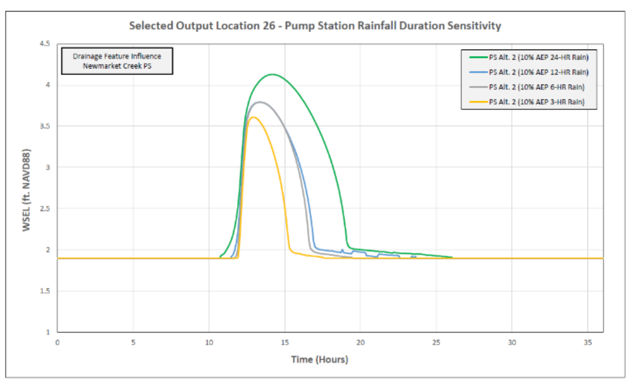

Figure 36. Output Location 26 – Rain Duration Sensitivity

2.6 PUMP STATION – STORM GATES DUAL OPERATION SENSITIVITY

This sensitivity assesses the 10% AEP rainfall with the MHHW in the year 2032 for future without-project, future with-project gates only, future with-project gates closed pump station alternative 2, and future with-project gates and pump stations. There could be instances where pump stations are used to quickly remove rainfall during non-storm surge conditions meaning storm gates are open and gravity conditions are present.

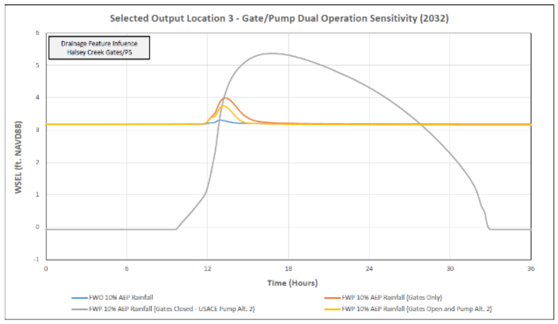

Figure 37. Output Location 3 – Dual Operations Sensitivity

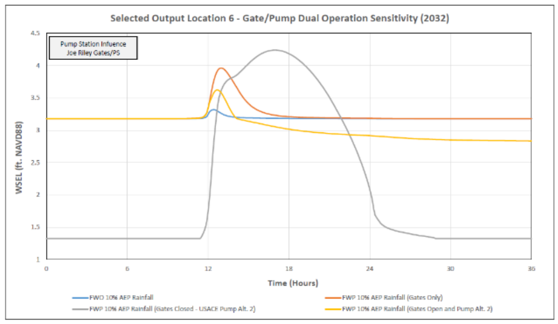

Figure 38. Output Location 6 – Dual Operations Sensitivity

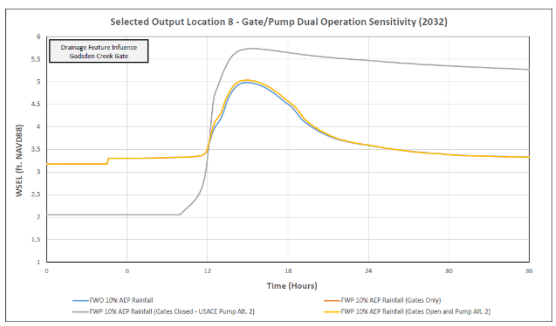

Figure 39. Output Location 8 – Dual Operations Sensitivity

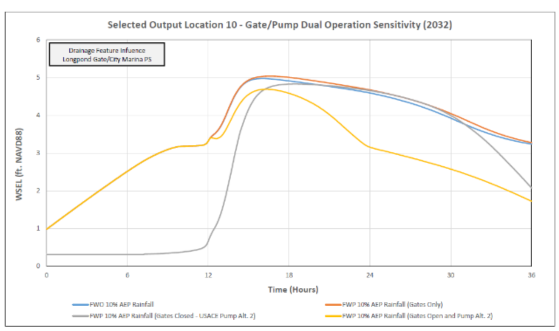

Figure 40. Output Location 10 – Dual Operations Sensitivity

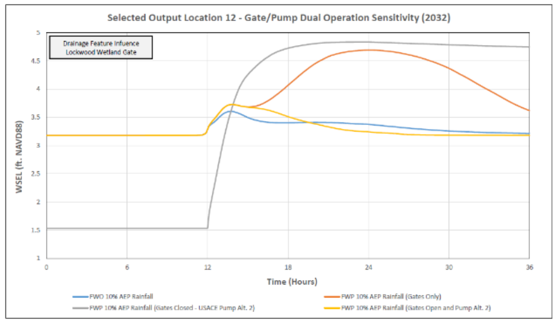

Figure 41. Output Location 12 – Dual Operations Sensitivity

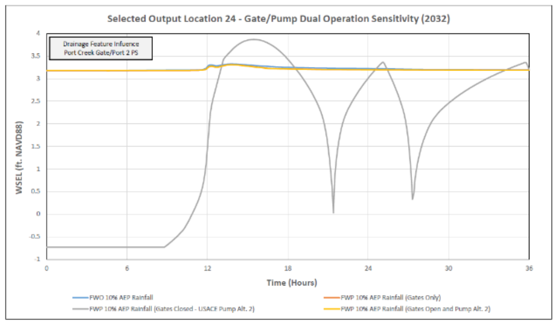

Figure 42. Output Location 24 – Dual Operations Sensitivity

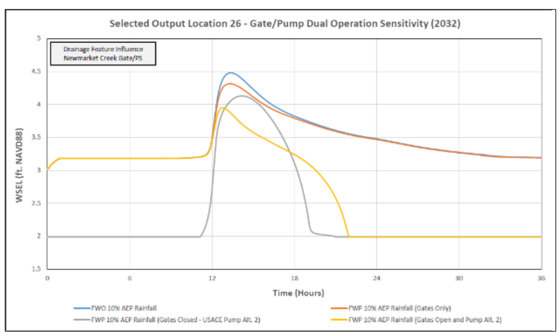

Figure 43. Output Location 26 – Dual Operations Sensitivity

US Army Corps of Engineers®

Charleston District

CHARLESTON PENINSULA, SOUTH CAROLINA, A COASTAL STORM RISK MANAGEMENT STUDY

Charleston, South Carolina

ENGINEERING APPENDIX - B
COASTAL SUBAPPENDIX 4

February 2022

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Contents

CHAPTER 1 – INTRODUCTION

1.1. DESCRIPTION OF PROJECT AREA AND VICINITY

Centrally located along the coast of South Carolina, the Charleston Peninsula project area is approximately 8 square miles, located between the Ashley and Cooper Rivers (Figure 1.1.1). Charleston Harbor is formed by the confluence of the Cooper, Ashley and Wando Rivers before discharging into the Atlantic Ocean. It includes the tidal estuary of the lower 12 miles of the Cooper River and the four miles of open bay between the confluence of the Ashley and Cooper Rivers and the Atlantic Ocean. The Cooper River contributes most of the freshwater inflow to the system and is the largest of the estuaries, extending about 57 miles from the harbor entrance to the Jefferies Hydroelectric Station at Lake Moultrie dam in Pinopolis, SC. The Cooper River flows are controlled under a contractual agreement with USACE to reduce shoaling in Charleston Harbor federal navigation channel. They are limited to a 4500 cfs daily average by week.

The Charleston Harbor is sheltered by barrier islands at the entrance. (see inset in Figure 1.1.1)

Figure 1.1.1 Charleston Peninsula Study Boundary

The first European settlers arrived in Charleston around 1670. Since that time, the peninsula city has undergone dramatic shoreline changes, predominantly by landfilling of the intertidal zone. Early maps show that over one-third of the peninsula has been “reclaimed.” Much of the landfilling occurred on the southern tip of Charleston, behind a seawall and promenade, known as the Battery and along the western shoreline. Figure 1.1.2 shows the Halsey Map of 1844 which depicts the original shoreline of the Charleston Peninsula.

Figure 1.1.2 Halsey Map of 1844

The federal navigation channel is adjacent to the study area along the eastern side with Columbus Street Terminal and Union Pier Terminal (Figure 1.1.3). The federal navigation channel on the Ashley River to the west of the peninsula is still authorized but not maintained.

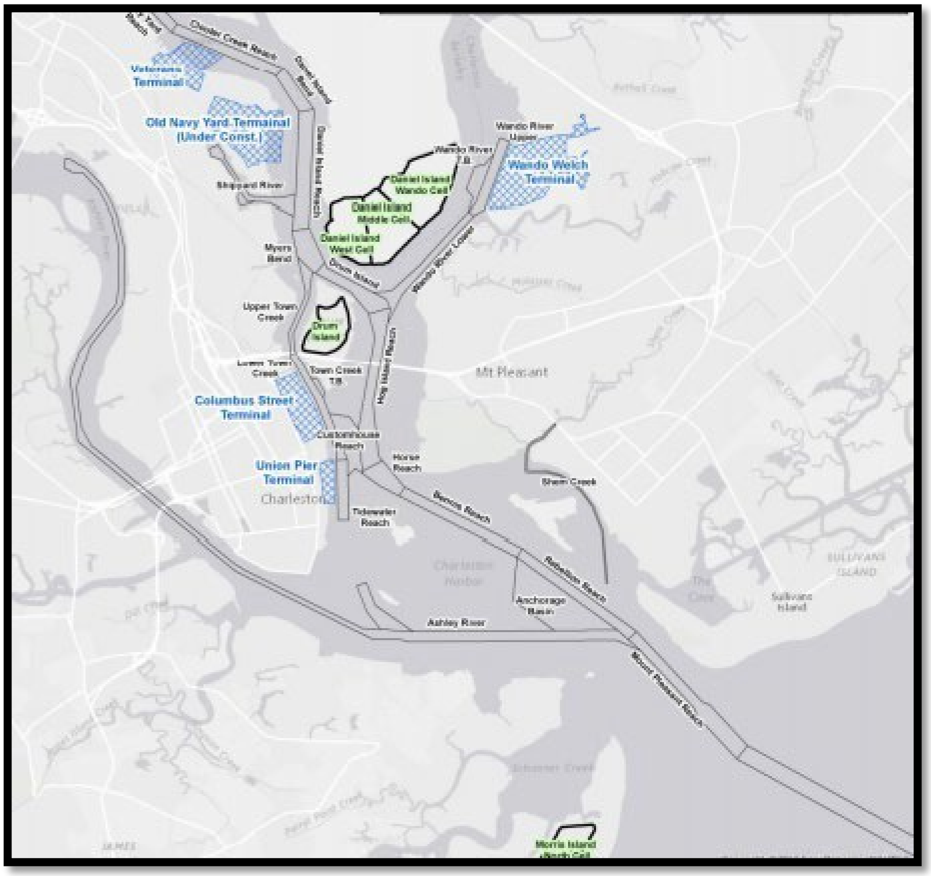


Figure 1.1.3 Charleston Harbor Navigation Channel

1.2. NOAA COOPER RIVER ENTRANCE TIDAL GAUGE RECORD

The Cooper River Entrance Tidal Gauge is Station 8665530 and is locally referred to as the Charleston Harbor orCustom’s House gauge. It was established September 13, 1899. It is located downtown on the peninsula in thevicinity of U.S. Custom House, along East Bay Street, and along Broad Street. The tide gauge and staff are on the south end of the dock. Shown in Figure 1.2.1.

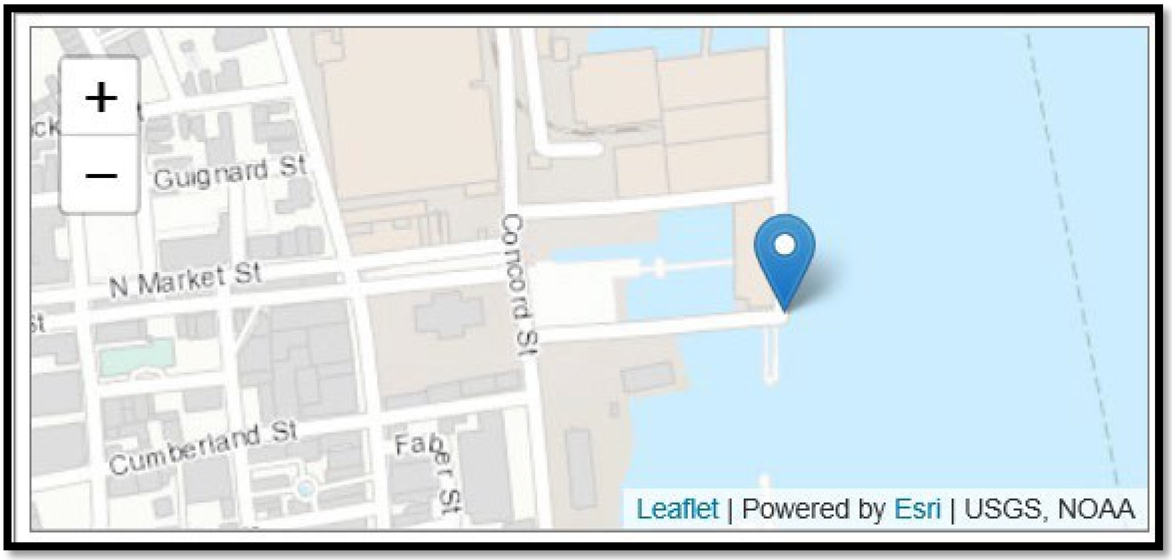


Figure 1.2.1 Location of NOAA Gauge 8665530

Datum information provided by NOAA on their Tides and Currents website indicate a tide range of 5.76 feet (https://tidesandcurrents.noaa.gov/datums.html?id=8665530). Shown in Figure 1.2.2 and Table 1.2.1. Mean Sea Level (MSL) of the tidal epoch between 1983 and 2001 is 2.92 feet above MLLW. The NAVD88 (North American Vertical Datum of 1988) is 0.22 above mean sea level.

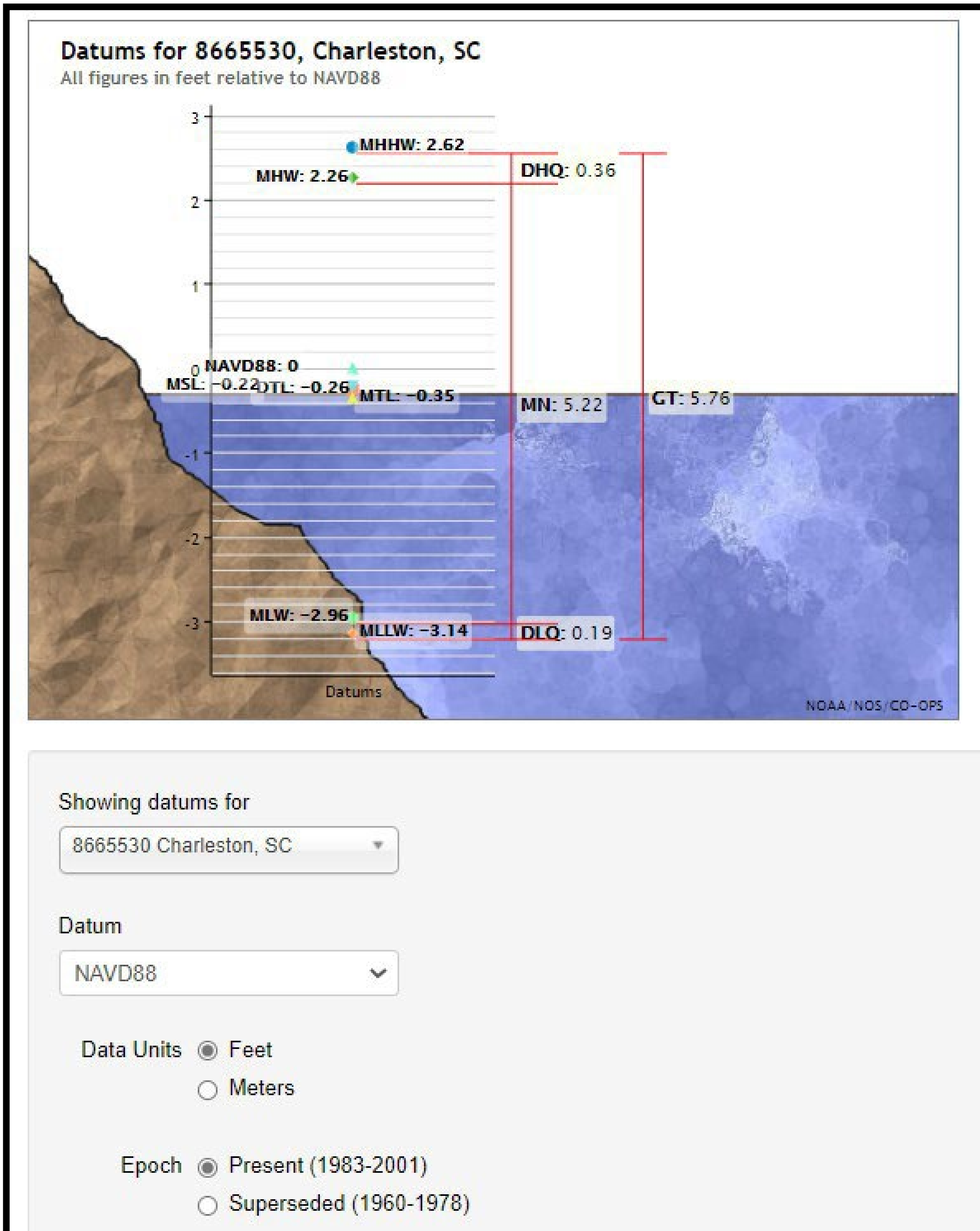


Figure 1.2.2 Tide Range Station 8665530 (Epoch 1983-2001)

Table 1.2.1 Elevations on Mean Lower Low Water

Datum	Value	Description
MHHW	5.76	Mean Higher-High Water
MHW	5.4	Mean High Water
MTL	2.79	Mean Tide Level
MSL	2.92	Mean Sea Level
DTL	2.88	Mean Diurnal Tide Level
MLW	0.18	Mean Low Water
MLLW	0	Mean Lower-Low Water
NAVD88	3.14	North American Vertical Datum of 1988
STND	-2.77	Station Datum
GT	5.76	Great Diurnal Range
MN	5.22	Mean Range of Tide
DHQ	0.36	Mean Diurnal High Water Inequality
DLQ	0.19	Mean Diurnal Low Water Inequality
HWI	0.41	Greenwich High Water Interval (in hours)
LWI	6.63	Greenwich Low Water Interval (in hours)
Max Tide	12.52	Highest Observed Tide
Max Tide Date & Time	9/21/1989 23:42	Highest Observed Tide Date & Time
Min Tide	-4.09	Lowest Observed Tide
Min Tide Date & Time	3/13/1993 19:24	Lowest Observed Tide Date & Time
HAT	7.26	Highest Astronomical Tide
HAT Date & Time	10/16/1993 13:06	HAT Date and Time
LAT	-1.52	Lowest Astronomical Tide
LAT Date & Time	2/9/2001 7:24	LAT Date and Time

Tidal Datum information provided from the NOAA website:
https://tidesandcurrents.noaa.gov/datums.html?id=8665530

1.3. CLIMATE

Charleston SC has hot humid summers and fairly mild winters. Average Annual high temperatures is approximately 75 degrees F and average annual low temperatures are approximately 53 degree F. Average annual precipitation is 44.29 inches with an average of 102 days of precipitation per year. Shown in Figure 1.3.1 and Table 1.3.1.

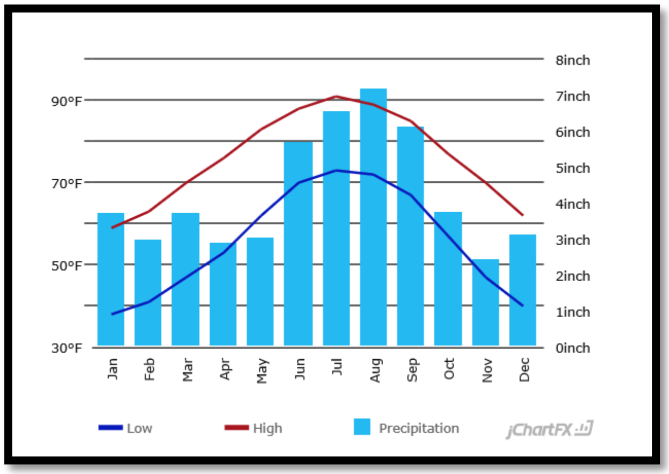


Figure 1.3.1 Charleston Temperature and Precipitation

Table 1.3.1 Charleston Temperature and Precipitation

Climate Charleston AFB - South Carolina

°C | °F

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Average high in °F:	59	63	70	76	83	88	91	89	85	77	70	62
Average low in °F:	38	41	47	53	62	70	73	72	67	57	47	40
Av. precipitation in inch:	3.7	2.95	3.7	2.91	3.03	5.67	6.54	7.17	6.1	3.74	2.44	3.11
Days with precipitation:	9	9	11	8	14	10	15	12	10	6	7	8
Hours of sunshine:	188	189	243	284	323	308	297	281	244	239	210	187

Source: https://www.usclimatedata.com/climate/charleston-afb/south-carolina/united-states/ussc0052

1.4 HORIZONTAL AND VERTICAL DATUMS

Horizontal datum for this study is tied to the State Plan Coordinate System using North American Datum of 1983 (NAD83, South Carolina 3900). Distances are in International Feet by horizontal measurement. The vertical datum for this study is tied to the North American Vertical Datum of 1988 (NAVD88), a requirement of ER 1110-2-8160. Elevations are in feet.

1.5 WINDS

Due to the geographic orientation of the peninsula with the Ashley River on the west and the Cooper River on the right, the western side and the northeastern side of the peninsula are generally sheltered from locally generated wind waves. The southern and southeastern portions are subject to local wind generated waves over the harbor. The Post45 Harbor Deepening study documented the following information.

1.5.1 Winds in Charleston Harbor

Winds can be described by their speed, direction, and duration. The National Oceanic and Atmospheric Administration (NOAA) operates a weather station in Charleston Harbor which collect 6-minute wind data. This station records wind speed and direction at the shore. A wind rose was generated using the hourly averaged data recorded between January 2010 and December 2011 to visualize the distribution of winds which pass over Charleston Harbor (See Figure 1.5.1).

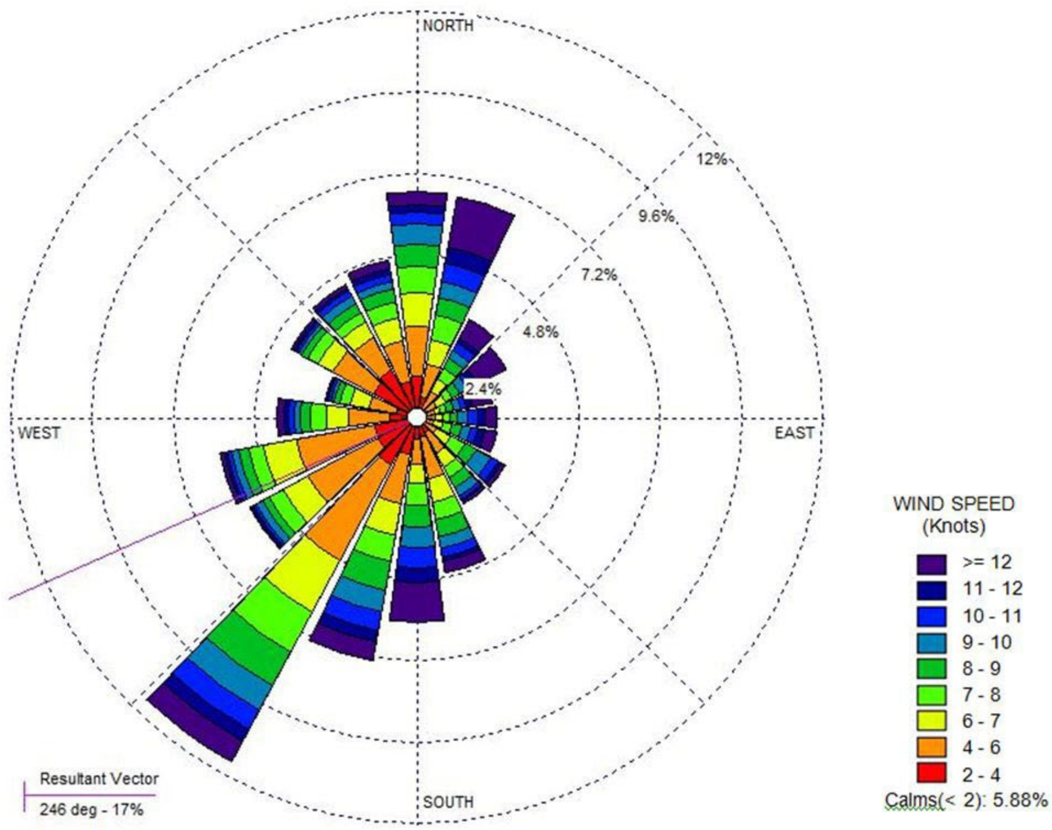


Figure 1.5.1. Wind Rose for Charleston Harbor Depicting Wind Direction and Speed Frequency

The distribution of wind speeds varies by direction (Refer to Figure 1.5.1. This figure is known as a wind rose). The total winds over Charleston Harbor, regardless of angle of approach, have the distribution by wind speed class shown in Figure 1.5.2. Three petals of the wind rose from Figure 1.5.1 are shown as frequency distributions in Figure 1.5.3. The petals selected reflect the three key directions: the largest number of winds, the highest speed winds and those with longest fetch (distance to travel). The largest number of winds in Charleston Harbor come from the southwest, while the most high-speed winds (fastest 10% of winds) come from the north-northeast direction (Wando River). Winds entering the harbor from open ocean (south-east) have the potential to travel the furthest distance before reaching a shoreline.

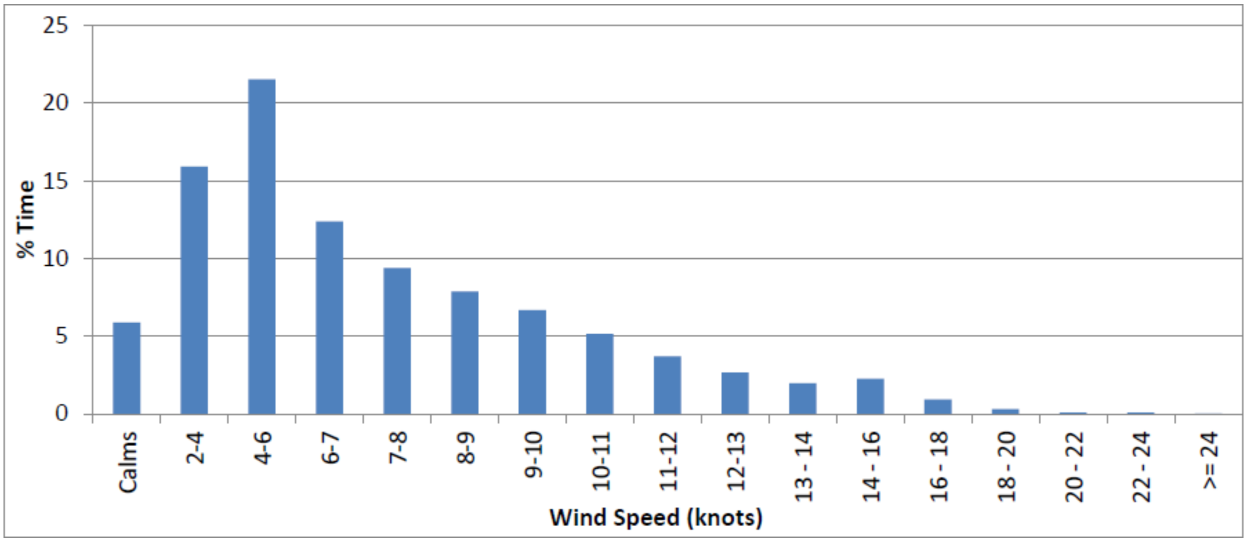


Figure 1.5.2 Wind Speed Frequency Distribution in Charleston Harbor from all directions

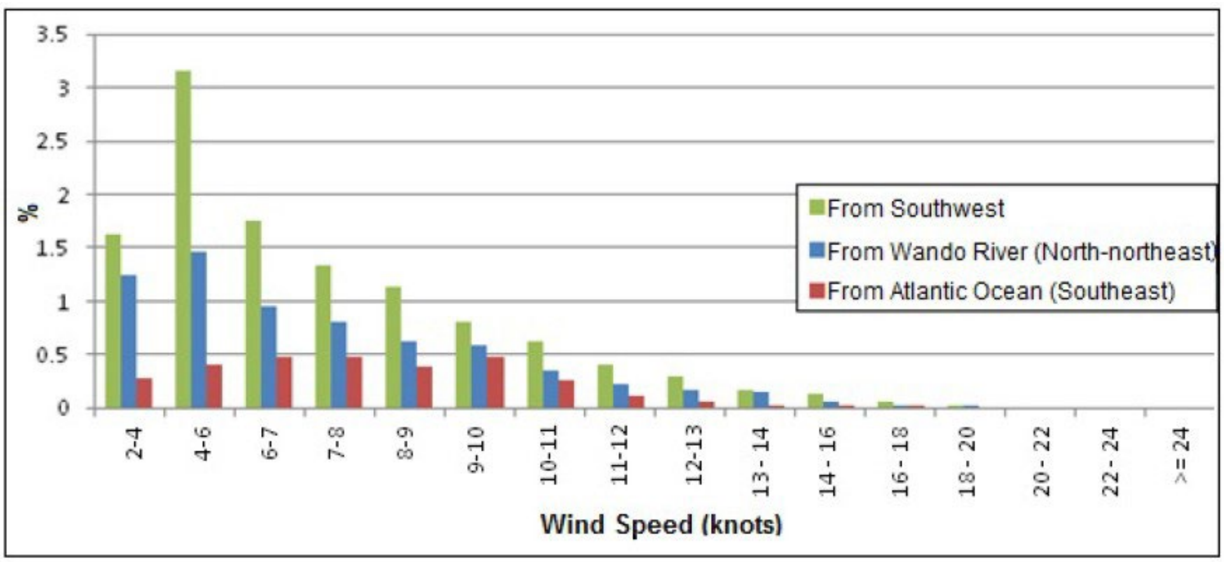


Figure 1.5.3 Wind Speed Frequency Distribution in Charleston Harbor comparing three key directions

1.6 ASTRONOMICAL TIDES & WATER LEVELS

1.6.1 Astronomical Tides

The Cooper River Entrance Tidal Gauge (8665530), or the Charleston Harbor or Custom's House gauge is themost extensive and continuous record of tides for the City of Charleston.

1.6.2. Water Levels

The Charleston Harbor tide gauge was established in 1899. In that nearly 100-year time span, local sea level has risen 1.07 ft (Figure 1.6.2.1). One way to track local impacts from sea level change is documenting “minor coastal flooding”. Commonly called nuisance, sunny day or high tide flooding, “minor coastal flooding” is a threshold from the National Weather Service that indicates when the tide has reached a certain height (7.0 ft MLLW in the Charleston Harbor). At this height, low-lying areas on land begin to flood. For example, Lockwood Blvd begins to flood at 7.2 ft MLLW (or 4.06 ft. NAVD88).

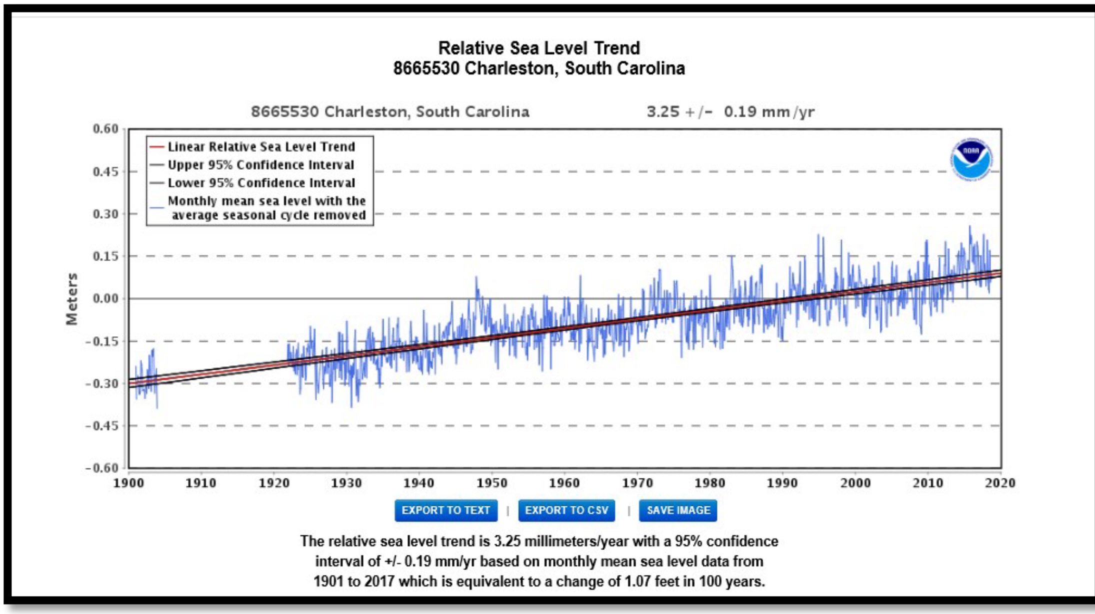


Figure 1.6.2.1 Observed Sea Level Change at Charleston Harbor Gauge

1.6.3 Extreme Water Levels

According to NOAA Tides and Currents explanation of Extreme Water Levels: Extremely high or low water levels at coastal locations are an important public concern and a factor in coastal hazard assessment, navigational safety, and ecosystem management. Exceedance probability, the likelihood that water levels will exceed a given elevation, is based on a statistical analysis of historic values. This product provides annual and monthly exceedance probability levels for select Center for Operational Oceanographic Products and Services (CO-OPS) water level stations with at least 30 years of data. When used in conjunction with real time station data, exceedance probability levels can be used to evaluate current conditions and determine whether a rare event is occurring. This information may also be instrumental in planning for the possibility of dangerously high or low water events at a local level. Because these levels are station specific, their use for evaluating surrounding areas may be limited. A NOAA Technical Report, "Extreme Water Levels of the United States 1893-2010" describes the methods and data used in the calculation of the exceedance probability levels.

The extreme levels measured by the CO-OPS tide gauges during storms are called storm tides, which are a combination of the astronomical tide, the storm surge, and limited wave setup caused by breaking waves. They do not include wave run-up, the movement of water up a slope. Therefore, the 1% annual exceedance probability levels shown on this website do not necessarily correspond to the Base Flood Elevations (BFE) defined by the Federal Emergency Management Administration (FEMA), which are the basis for the National Flood Insurance Program. The 1% annual exceedance probability levels on this website more closely correspond to FEMA's Still Water Flood Elevations (SWEL). The peak levels from tsunamis, which can cause high-frequency fluctuations at some locations, have not been included in this statistical analysis due to their infrequency during the periods of historic record. (Source: https://tidesandcurrents.noaa.gov/est/)

High and low annual exceedance probability levels are shown relative to the tidal datum and the geodetic North American Vertical Datum (NAVD88), if available. The levels are in meters relative to the National Tidal Datum Epoch (1983-2001) Mean Sea Level datum at most stations or a recent 5-year modified epoch MSL datum at stations with rapid sea level rates in Louisiana, Texas, and Alaska. On the left of Figure 1.6.2.2 are the exceedance probability levels for the mid-year of the tidal epoch currently in effect for the station. On the right are projected exceedance probability levels and tidal datum assuming continuation of the linear historic trend.

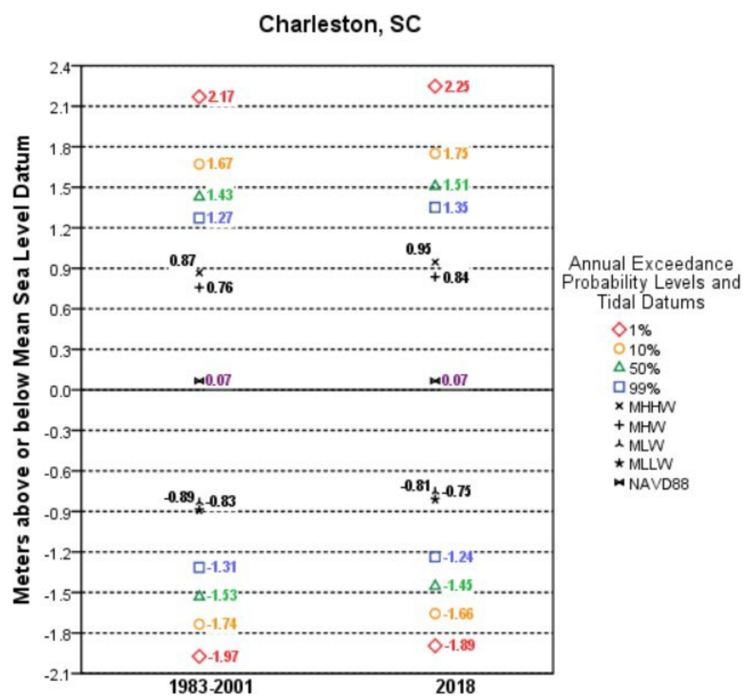
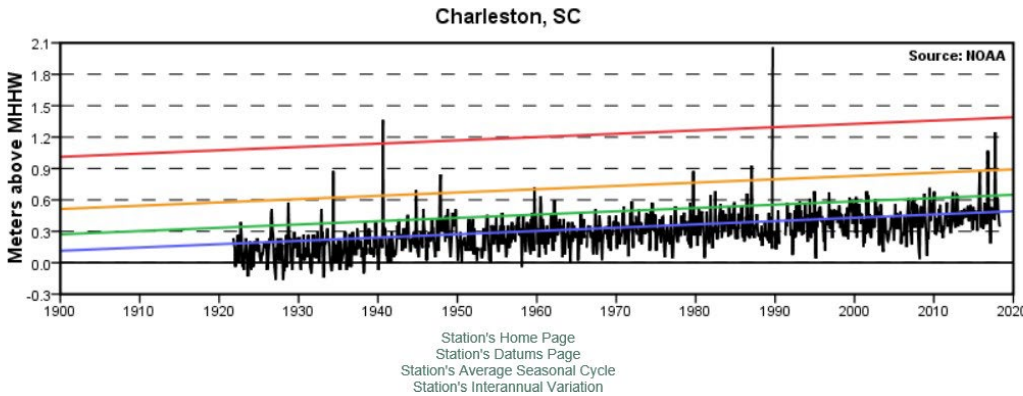


Figure 1.6.2.2 Exceedance Probability Levels and Tidal datum of 8665530 Charleston, Cooper River Entrance, SC

Shown in Figure 3.4.3.2 the 1% level (red) indicates a 1 in 100 chance of occurring in any given year, the 10% level (orange) indicates a 10 in 100 chance of occurring in any given year, and the 50% level (green) indicates 50 in 100 chance of occurring in any given year. The 99% level (blue) indicates a high probability of occurrence every year.

The level of confidence in the exceedance probability decreases with longer return periods. Table 1.6.2.1 is tabulated in feet referenced to NAVD88. (source https://tidesandcurrents.noaa.gov/est/est_station.shtml?stnid=8665530)



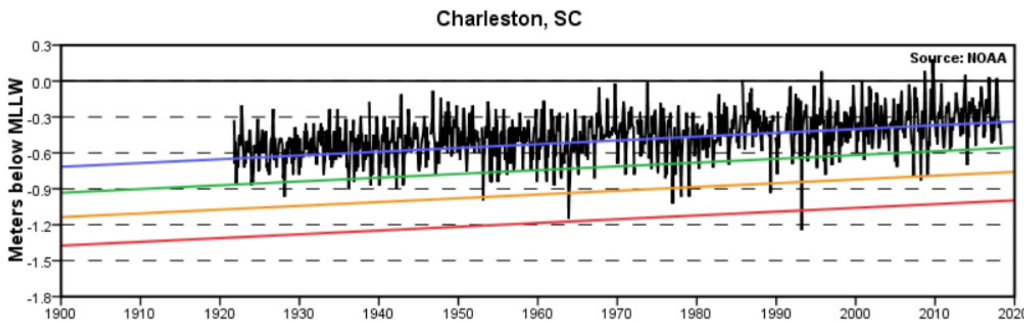


Figure 1.6.2.3 Seasonal and Interannual Variation of Gauge 8665530 Extreme water Levels

Table 1.6.2.1 Extreme Water levels and Tidal datum of 8665530 Charleston, Cooper River Entrance, SC

ID:	8665530
Reference Datum:	NAVD88
Name:	Charleston, SC
Established:	Sep 13, 1899
HAT:	4.12 (ft)
MHHW:	2.62 (ft)
MHW:	2.26 (ft)
MSL:	-0.22 (ft)
MLW:	-2.96 (ft)
MLLW:	-3.14 (ft)
NAVD88:	0.00 (ft)
1% AEP:	7.16 (ft)
10% AEP:	5.52 (ft)
50% AEP:	4.73 (ft)
99% AEP:	4.21 (ft)
Continuous record start:	1921
Continuous record end:	Present

1.7 STORMS

1.7.1. Tropical Cyclones

Storms do not have to make landfall to have a flooding impact. Charleston experiences flooding from all three types of tropical cyclones: hurricanes, tropical storms and tropical depressions. 22 storms passed within 100 nautical miles of Charleston between 2000 and present Figure 1.7.1.1). The number of storms in the entire period of record will also be given, but an image would likely be too busy (156 storms passed the same area shown in the image).

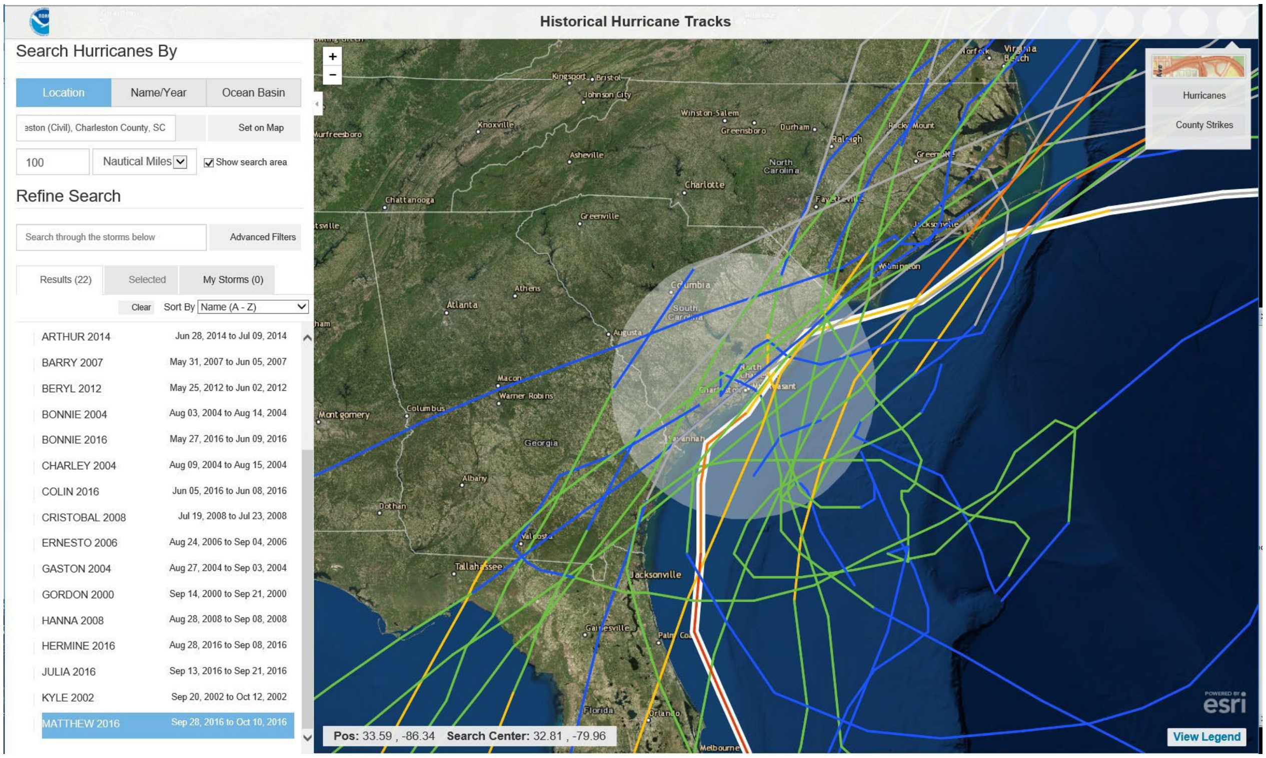


Figure 1.7.1.1 Twenty two storms passed within 100 nautical miles of Charleston between 2000 and 2019.

1.7.2. Hurricanes

In the Colonial period tropical storms and hurricanes were known as "September gales," probably because the ones people remembered and wrote about were those which damaged or destroyed crops just before they were to be harvested.

One such storm that struck Charles Town on September 25, 1686, was "wonderfully horrid and destructive...Corne is all beaten down and lyes rotting on the ground... Aboundance of our hoggs and Cattle were killed in the Tempest by the falls of Trees..." The storm also prevented a Spanish assault upon Charles Town by destroying one of their galleys and killing the commander of the Spanish assault.

In autumn of 1700, "a dreadful hurricane happened at Charles Town which did great damage and threatened that total destruction of the Town, the lands on which it is built being low and level and not many feet about high water mark, the swelling sea rushed in with amazing impetuosity, and obliged the inhabitants to fly to shelter..." A ship, Rising Sun, out of Glasgow and filled with settlers had made port just prior to the storm's landfall. It was dashed to pieces and all on board perished.

Of a storm which passes inland along the coast September 7-9, 1854, Adele Pettigru Allston wrote from Pawley's Island, "The tide was higher than has been known since the storm of 1822. Harvest had just commenced and that damage to the crops in immense. From Waverly to Pee Dee not a bank nor any appearance of land was to be seen... (just) one rolling, dashing Sea, and the water was Salt as the Sea."

By 1893, major population centers could be telegraphically alerted to storms moving along the coast, but there were no warnings for the Sea Islands and other isolated areas. The "Great Storm of 1893" struck the south coast at high tide on August 28, pushing an enormous storm surge ahead of it and creating a "tidal wave" that swept over and submerged whole islands. Maximum winds in the Beaufort area were estimated to be 125 miles per hour, those in Charleston were estimated near 120 miles per hour. At least 2,000 people lost their lives, and an estimated 20,000-30,000 were left homeless and with no mean of subsistence.

Hazel (October 1954) and Gracie (September 1959) have been the most memorable storms in recent years. Hazel, a Category 4 storm, made landfall near Little River, S.C., with 106-miles per hour winds and 16.9 foot storm surge. One person was killed and damage was estimated at $27 million.

Gracie, a Category 4 hurricane, made landfall on St. Helena Island with 130 mph winds and continued toward the north-northwest. Heavy damage occurred along the coast from Beaufort to Charleston. Heavy rains caused flooding through much of the State and crop damage was severe. NOAA's Hurricane Re-analysis Project upgraded Gracie from a Category 3 to a Category 4 hurricane in June, 2016. Tide level reached 5.0 feet NAVD88.

Hugo (September 1989) made landfall near Sullivan's Island with 120 knot winds. It continued on a northwest track at 25-30 miles per hour and maintained hurricane force winds as far inland as Sumter. Hugo exited the State southwest of Charlotte, N.C., before sunrise on September 22. The hurricane caused 13 directly related deaths and 22 indirectly related deaths, and it injured several hundred people in South Carolina. Damage in the State was estimated to exceed $7 billion, including $2 billion in crop damage. The forests in 36 counties along the path of the storm sustained major damage. Tide level reached 9.39 feet NAVD88. (https://tidesandcurrents.noaa.gov/waterlevels.html?id=8665530&units=standard&bdate=19890917&edate=19890925&timezone=GMT&datum=NAVD&interval=hl&action=)

From 1990 to 2015, South Carolina had only had five weak tropical cyclone landfalls along the coast: Tropical Storm Kyle (35 kts) in 2002, Hurricane Gaston (65 kts) and Hurricane Charley (70 kts) in 2004, Tropical Storm Ana (40 kts) in 2015, and Tropical Depression Bonnie (30 kts) in 2016. Bonnie developed north of the Bahamas and strengthened into a TS as it move northwest toward the GA/SC coasts, eventually weakening to a TD before making landfall near Charleston. Produced heavy rainfall (widespread 3-7 inches with local amounts over 10 inches), mainly north of I-126, which led to significant flooding. During September 1999 Hurricane Floyd, a very large storm, came very close to the South Carolina coast, then made landfall near Cape Fear, North Carolina. Hurricane Floyd triggered mandatory coastal evacuations along the South Carolina coast. Heavy rain of more than 15 inches fell in parts of Horry County, S.C., causing major flooding along the Waccamaw River in and around the city of Conway for a month.

Mathew (October 2016) moved north and then northwest through the Caribbean Sea and then through the Bahamas while strengthening to a Category 4 hurricane. Tracked just off the east coast of FL and GA while weakening to a Category 1 storm before making landfall near McClellanville, SC with winds near 85 mph. Produced hurricane force wind gusts along the entire coast, significant coastal flooding from high storm tides (including a record level at Fort Pulaski), and very heavy rainfall (widespread 6 to 12 inches with locally higher amounts near 17 inches) which led to significant freshwater flooding. Tide level reached 6.06 feet NAVD88.

Irma (Sep 2017) made landfall in the Florida Keys as a Category 4 hurricane and then moved along the southwest coast of Florida as a Category 3 hurricane. The storm then moved north near the west coast of Florida while weakening to a tropical storm before moving into southwest Georgia and continuing to weaken. Produced significant coastal flooding, wind gusts near hurricane-force along with 4 tornadoes, flooding rainfall and river flooding across southeast SC/GA. NOAA tide level reached elevation 6.61 feet NAVD88.

Florence (Sept 2018) made landfall near Wrightsville Beach, NC as a Category 1 hurricane before slowing down and weakening to a TS. The storm then moved southwest near the northern SC coast before shifting west toward the SC Midlands and weakening to a TD. Produced some tropical storm force wind gusts and several inches of rain, mainly north of Charleston.

Michael (October 2018) made landfall near Mexico Beach, FL as a Category 4 hurricane and then moved northeast through southwest GA as a hurricane before weakening to a TS before reaching central SC. Produced tropical storm force winds and several inches of rainfall across much of southeast SC/GA which ledto many fallen trees and some power outages.

Dorian (Sept 2019) strengthened to a Category 3 hurricane as it traveled along the Gulf Stream, offshore of the coasts of GA and SC. Produced sustained winds of 45 to 55 kt with gusts up to 77 kt. Storm surge created inundation in SC up to 4 ft and peak rainfall was measured to be 15.21 inches at Pawleys Island.

Isaias (August 2020) strengthened to a hurricane 100 nm south of Charleston and later made landfall at Ocean Isle, NC with a peak intensity of 80 kt. The storm produced 5-7 inches of rain in SC and over 7 ft of inundation in some areas.

1.7.3. Historical Storms

A historic flooding event affected the Carolinas from October 1-5, 2015. A stalled front offshore combined with deep tropical moisture streaming northwest into the area ahead of a strong upper level low pressure system to the west and Hurricane Joaquin well to the east. This led to historic rainfall with widespread amounts of 15-20 inches and localized amounts over 25 inches, mainly in the Charleston tri-county area. Flash flooding was prevalent and led to significant damage to numerous properties and roads and many people having to be rescued by emergency personnel. In addition, tides were high due to the recent perigean spring tide and persistent onshore winds, exacerbating the flooding along the coast, especially in downtown Charleston.

CHAPTER 2 – PAST STUDIES

There have been no past USACE Coastal Storm Risk Management Studies performed for the Charleston, Berkeley, Dorchester area, where city of Charleston Peninsula resides.

There have been numerous navigation studies done on the federal navigation project in Charleston Harbor.

CHAPTER 3 – IMPACTS OF CLIMATE CHANGE

GUIDANCE

Climate change is defined as a change in global or regional climate patterns. Climate change has already been observed globally and in the United States. These included increases and changes in air and water temperatures, reduced frost days, increased frequency and intensity of heavy downpours, a rise in sea level, and reduced snow cover, glaciers, permafrost, and sea ice. Climate change has the potential to affect all of the missions of the United States Army Corps of Engineers (USACE). USACE mission in regard to climate change is: “To develop, implement, and assess adjustments or changes in operations and decision environments to enhance resilience or reduce vulnerability of USACE projects, systems, and programs to observed or expected changes in climate”. The USACE’s Climate Change Program develops and implements practical, nationally consistent, and cost-effective approaches and policies, to reduce potential vulnerabilities to the Nation’s water infrastructure resulting from climate change and variability.

The Corps has the following guidance to assist in the assessment of Climate Change Impactson a proposed project.:

- ER 1105-2-101 Risk Assessment for Flood Risk Management Studies, 2019.
- EM 1110-2-6056, Standards and Procedures for Referencing Project Elevation Grades to Nationwide Vertical Datums. 2010.
- EP 1100-2-1, Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation. 2020.
- ECB 2018-2, Implementation of Resilience Principles in the Engineering & Construction Community of Practice 2018.
- ECB 2018-14, Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects.

The Department of the Army Engineering Regulation 1100-2-8162 (31 Dec 2013) requires that future Relative Sea Level Change (RSLC) projections must be incorporated into the planning, engineering design, construction and operation of all civil works projects. The structural components of the proposed alternatives in consideration of the “low”, “intermediate”, and “high” potential rates of future RSLC were evaluated. This range of potential rates of RSLC is based on the findings of the National Research Council (NRC, 1987) and the Intergovernmental Panel for Climate Change (IPCC, 2007).

3.1 OBSERVED IMPACTS

The effects of Climate change are already observed in the study area with the increase in “nuisance” flooding. According to NOAA’s Ocean Service: high tide flooding, sometimes referred to as "nuisance" flooding, is flooding that leads to public inconveniences such as road closures (Figure 3.1.1). It is increasingly common as coastal sea levels change. As relative sea level increases, it no longer takes a strong storm or a hurricane to cause coastal flooding. Flooding now occurs with high tides in many locations due to climate-related sea level change, land subsidence, and the loss of natural barriers.

High tide flooding—which causes such public inconveniences as frequent road closures, overwhelmed storm drains and compromised infrastructure—has increased in the U.S. on average by about 50 percent since 20 years ago and 100 percent since 30 years ago.

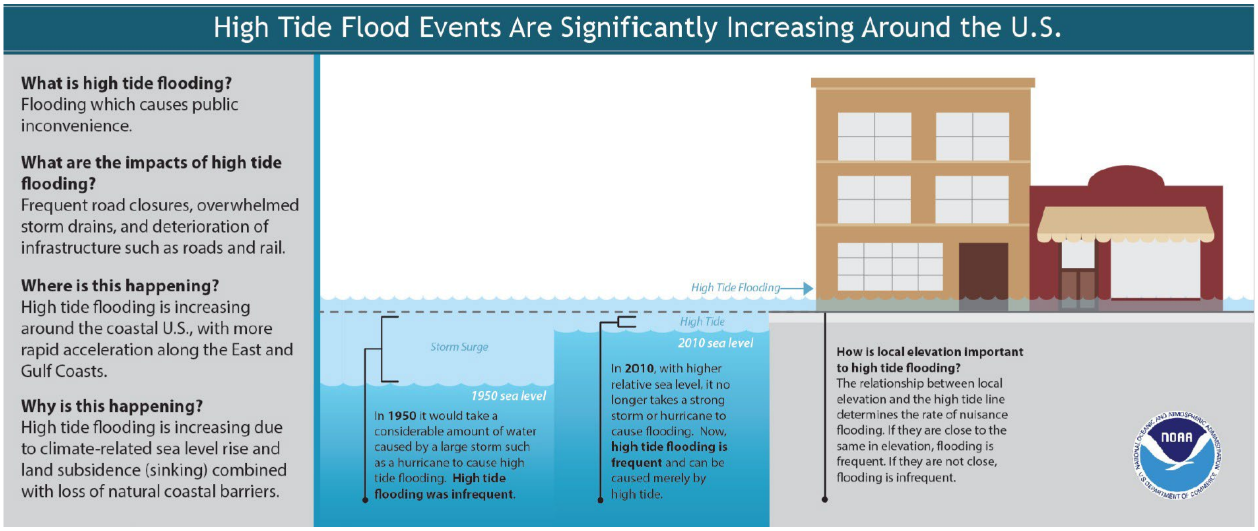


Figure 3.1.1. High Tide Flooding

NOAA Ocean Service further explains: A King Tide is a non-scientific term people often use to describe exceptionally high tides. Tides are long-period waves that roll around the planet as the ocean is "pulled" back and forth by the gravitational pull of the moon and the sun as these bodies interact with the Earth in their monthly and yearly orbits. Higher than normal tides typically occur during a new or full moon and when the Moon is at its perigee, or during specific seasons around the country.

SCDHEC is leading the South Carolina **King Tides** initiative to document the effect that extreme **tide** events have on our state's beaches, coastal waterways, private properties and public infrastructure on their MyCoast website (https://mycoast.org/sc). The effects of individual King Tides may vary considerably. King Tides may result in coastal erosion, flooding of low-lying areas, and road closures which may disrupt normal daily routines. This is particularly true when a King Tide coincides with significant precipitation because water drainage and runoff is impeded.

As an example: DHEC issues King Tide notifications to MyCoast members when water levels are predicted to reach 6.6 feet above mean lower low water (MLLW) (or 3.46 ft NAVD88) or higher at the Charleston Harbor Tide Station. NOAA's National Weather Service (NWS) Forecast Office in Charleston has established thresholds for minor (7.0 ft. MLLW), moderate (7.5 ft. MLLW), and major (8.0 ft. MLLW) flooding in the Charleston area. NOAA has also established a threshold for high tide flooding (HTF) in Charleston (7.6 ft. MLLW). Thresholds established for the Charleston area and terminology descriptions are provided in Table 3.1.1 below.

Table 3.1.1 Flooding Thresholds for Charleston, SC

Water Level Thresholds Established (Feet above MLLW)		**Feet above NAVD88**
Action Stage (NOAA NWS)	6.5	3.36
King Tide (SCDHEC)	6.6	3.46
Minor Flooding (NOAA NWS) Minor flooding on roadways around Downtown Charleston occurs, possibly including Lockwood Drive, Wentworth and Barre, Fishburne and Hagood, and Morrison Drive. As the tide height approaches 7.5 ft MLLW, roads can become impassable and closed	7.0	3.86
Moderate Flooding (NOAA NWS) In Downtown Charleston, additional impacted roads include HW-17 at HW-61, Market Street, East Bay, Rutledge, and areas around MUSC.	7.5	4.36
Major Flooding (NOAA NWS) Widespread flooding occurs in Downtown Charleston with numerous roads flooded and impassable and some impact to structures	8.0	4.86

Terminology

Action Stage: *The stage or level where the NWS or a partner/user needs to take action in preparation for possible significant hydrologic activity (NOAA NWS).*

King Tide: *A non-scientific term often used to describe exceptionally high tides (NOAA National Ocean Service).*

Minor Flooding: *Minimal or no property damage, but possibly some public threat (NOAA NWS).*

Moderate Flooding: *Some inundation of structures and roads. Some evacuations of people and/or transfer of property to higher elevations (NOAA NWS).*

Major Flooding: *Extensive inundation of structures and roads. Significant evacuations of people and/or transfer of property to higher elevations (NOAA NWS).*

High Tide Flooding (HTF): *Heights ranging from about 0.5 to 0.65 meters above mean higher high water and varying regionally with tide range. HTF height thresholds are based upon the minor-flood thresholds set by NWS Weather Forecasting Offices (WFOs) and on-the-ground local emergency managers who prepare for response to impending conditions (NOAA National Ocean Service).*

Further information on nuisance flooding can be found at https://oceanservice.noaa.gov/facts/nuisance-flooding.html).

High tides affect drainage systems by filling the stormwater collection systems, such that rainfall will pool and the runoff water will drain slower than if the systems were open. As Sea Level Rises in South Carolina, the occurrence of flooding associated with King Tides will also increase. Adapted from: Sweet, W. V., and J. Park, 2014: From the extreme to the mean: Acceleration and tipping points of coastal inundation from sea level rise, City of Charleston plotted "Observed and Predicted "Minor Coastal Flooding" in Charleston" (Figure 3.1.2) in their sea level rise strategy (City of Charleston, 2019). Charleston SC expects a significant increase based on trend and even more if sea level rise rate increases. Increases are expected along the entire South Carolina coast.

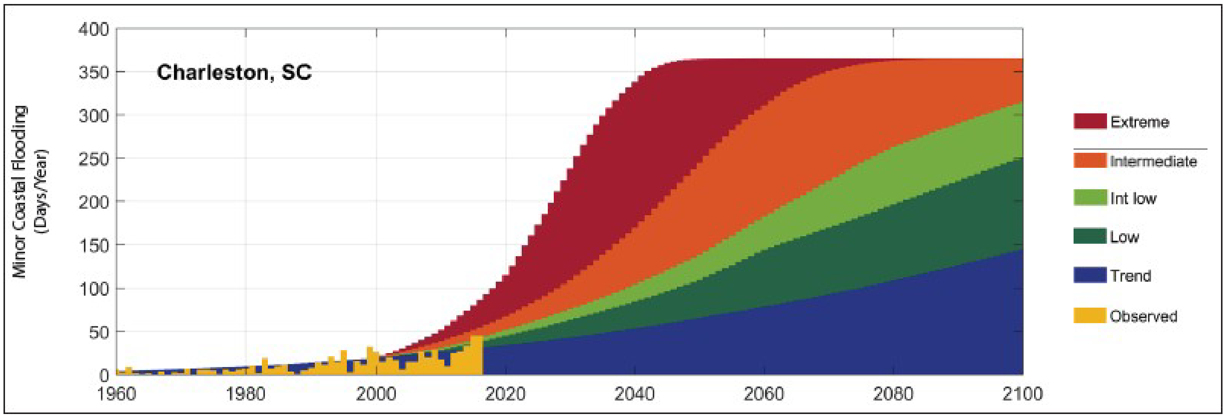


Figure 3.1.2. Observed and Predicted "Minor Coastal Flooding" in Charleston

NOAA sea level rise trends were used to develop Figure 3.1.2 and are displayed in Figure 3.1.3 and Table 3.1.2 below (USACE, 2021). Sea level rise predictions made by NOAA vary from USACE sea level rise curves, which were used throughout the rest of the study.

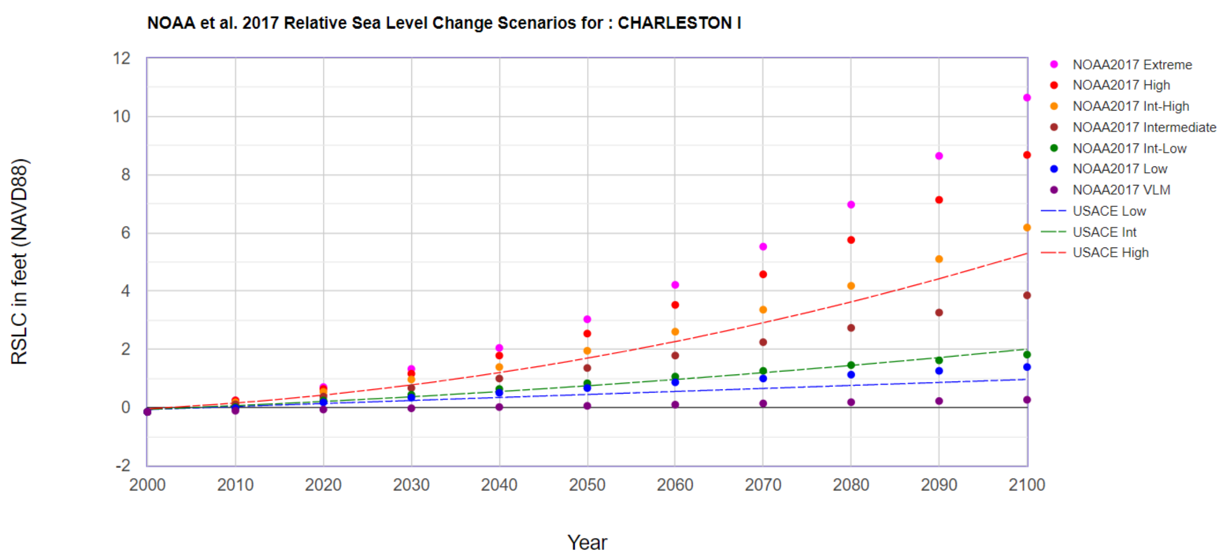


Figure 3.1.3. NOAA sea level rise curves compared to USACE curves

Table 3.1.2. NOAA sea level rise curves compared to USACE curves every decade in feet, NAVD88

Scenarios for CHARLESTON I
NOAA2017 VLM: 0.00417 feet/yr
All values are expressed in feet

Year	NOAA2017 VLM	NOAA2017 Low	NOAA2017 Int-Low	NOAA2017 Intermediate	NOAA2017 Int-High	NOAA2017 High	NOAA2017 Extreme	USACE Low	USACE Int	USACE High
2000	-0.15	-0.15	-0.15	-0.15	-0.15	-0.15	-0.15	-0.07	-0.06	-0.04
2010	-0.11	-0.02	0.02	0.11	0.18	0.25	0.25	0.04	0.07	0.16
2020	-0.06	0.18	0.25	0.38	0.54	0.64	0.70	0.14	0.21	0.43
2030	-0.02	0.34	0.44	0.67	0.97	1.16	1.33	0.24	0.37	0.78
2040	0.02	0.51	0.64	1.00	1.39	1.79	2.05	0.35	0.55	1.20
2050	0.06	0.67	0.84	1.36	1.95	2.54	3.03	0.45	0.75	1.70
2060	0.10	0.87	1.07	1.79	2.61	3.53	4.22	0.55	0.97	2.27
2070	0.14	1.00	1.26	2.25	3.36	4.58	5.53	0.66	1.20	2.91
2080	0.19	1.13	1.46	2.74	4.18	5.76	6.97	0.76	1.45	3.63
2090	0.23	1.26	1.62	3.26	5.10	7.14	8.64	0.86	1.72	4.43
2100	0.27	1.39	1.82	3.85	6.18	8.68	10.65	0.97	2.01	5.29

The City of Charleston has already taken steps to address the tidal filling of storm drains by adding check valves on many of the cities storm drainage pipelines and plans to continue. A check valve prevents seawater from backing up into drainage infrastructure to mitigate tidal flooding, while still allowing the outfall to drain stormwater as usual when the tide recedes. Overland flooding in areas such as Lockwood Boulevard are due to low-lying areas adjacent to the river and harbor which have a direct shoreline to increasing water levels.

3.2 COMPONENTS OF RELATIVE SEA LEVEL CHANGE

Sea Level Change is an increase in the volume of water in the world's ocean, resulting in an increase in sea level called global sea level change. The sea level change local to a specific area is called relative sea level change. Sea level change at specific locations (relative sea level change) may be more or less than the global average (global sea level change). Sea level change is attributed to global climate change by the added water from melting ice sheets and glaciers. Melting of floating ice shelves or icebergs at sea raises sea levels only slightly. Local factors such as subsidence of the land also impact local communities. Subsidence is the motion of the land surface as it shifts downward relative to a vertical datum.

3.3 LOCAL RATES OF RELATIVE SEA LEVEL CHANGE

RSLC considers the effects of (1) the eustatic, or global, average of the annual increase in water surface elevation due to the global warming trend, and (2) the "regional" rate of vertical land movement (VLM) that can result from localized geological processes, including the shifting of tectonic plates, the rebounding of the Earth's crust in locations previously covered by glaciers, the compaction of sedimentary strata and the withdrawal of subsurface fluids (USGS 2013). A vertical Land Movement assessment at Sullivan's Island by NASA/Jet Propulsion Lap indicated a very small change (0.001 ft/yr) based on 1998-2004 data. Technical Report NOS CO-OPS 065, Estimating Vertical Land Motion from Long-Term Tide Gauge Records in 2013 indicated a -1.24mm/yr (0.004 ft/year) for Charleston.

The year 1992 is used to start these curves because 1992 is the center year of the NOAA National Tidal Datum Epoch of 1983–2001. The National Tidal Datum Epoch is the period used to define tidal datums (Mean High

Water, for instance, and local MSL)

Figure 3.3.1 Location of Charleston Gauge 8665530

3.3.1 Historic Rate

The historic rate of future RSLC (or USACE Low Curve) is determined directly from gauge data gathered in the vicinity of the project area. RSLR is predicted to continue in the future as the global climate changes. The USACE Sea Level calculator uses the National Oceanographic and Atmospheric Administration (NOAA) for the Charleston Gauge 8665530, 2006 Published Rate of 0.01033 feet/yr. However, more recent updates to the National Oceanographic and Atmospheric Administration (NOAA) for the Charleston Gauge 8665530 are shownin Figure 3.3.1.1 for the period of record 1901 to 2017, which indicates 1.07 feet in 100 years. EC 1165-2-212 (*pdf, 845 KB*) and its successor ER 1100-2-8162 *(pdf, 317 KB)* were developed with the assistance of coastal scientists from the NOAA National Ocean Service and the US Geological Survey. Their participation on the USACE team allows rapid infusion of science into engineering guidance. EP 1100-2-1 (*pdf, 9.87 MB*), Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation.

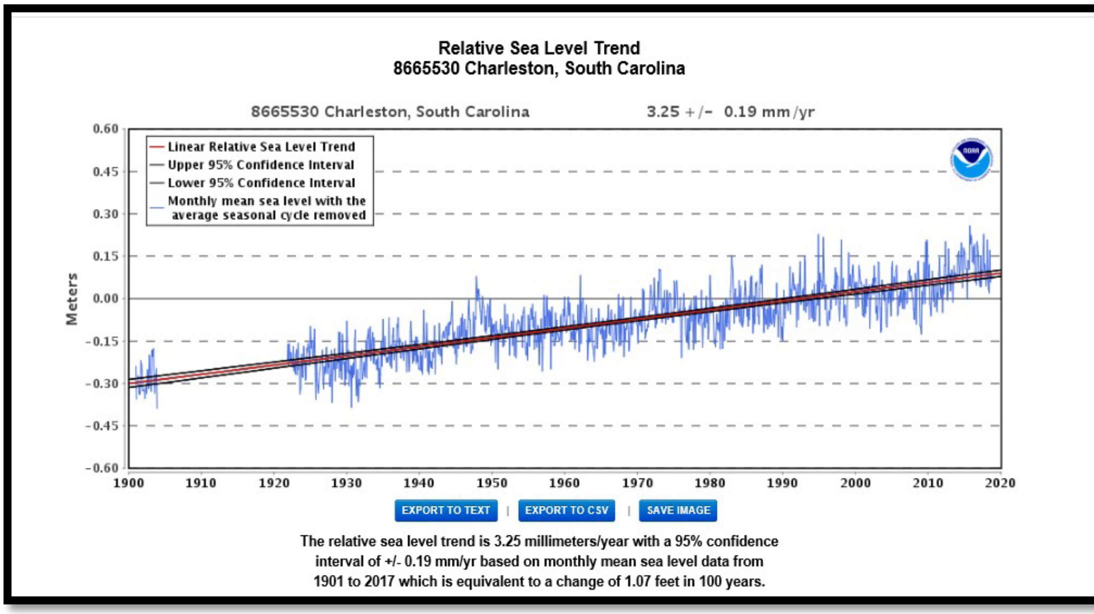


Figure 3.3.1.1 Mean Sea Level trend in Charleston 8665530 (source NOAA Tides and currents)

3.3.2 Intermediate and High Rate

The rate for the "USACE Intermediate Curve" is computed from the modified NRC Curve I considering both the most recent IPCC projections and modified NRC projections with the local rate of vertical land movement added.

The intermediate rate of local mean SLC is estimated by considering the modified NRC projections and adding the appropriate value to the local rate of vertical land movement. The intermediate rate of local sea level rise is based on the modified NRC Curve I since its value is comparable to that of the IPCC projection. The intermediate rate of sea level rise is computed using the equation

$$E(t2) - E(t1) = 0.0017(t2 - t1) + b(t2^2 - t1^2) + \text{local VLM}$$

where t1 and t2 represent the start and end dates of the projected time horizon in years, relative to 1992 (for both the intermediate and high rates of SLR, the NRC curves accelerate upward over time beginning in the year 1992 when the curves were developed; therefore, it is necessary to estimate SLR for a particular time horizon relative to 1992), and b is a constant value of 2.71E-5 for the intermediate rate.

The rate for the "USACE High Curve" is computed from the modified NRC Curve III considering both the most recent IPCC projections and modified NRC projections with the local rate of vertical land movement added.

The high rate of local mean SLR is estimated by determining the modified NRC Curve III value and adding it to the local rate of vertical land movement. This high rate scenario exceeds the 2001 and 2007 IPCC projections and considers the potential rapid loss of ice from Antarctica and Greenland. The NRC Curve III is also based on the general equation $E(t) = 0.0017t + bt^2$; however, the constant b changes to b = 1.13E-4, and has the same initial date of 1992.

3.3.3 Evaluation of Sea Level Change

According to National Oceanographic and Atmospheric Administration (NOAA) and using the USACE Sea-Level Change Curve Calculator (Version 2017.55) for the Charleston Gauge 8665530, the sea level change in 2100 for the low rate is 1.12 feet, intermediate rate is 2.15 feet and for high rate is 5.44 (Table 3.3.2.1).

Table 3.3.2.1 Estimates Sea Level Change 1990 to 2150

Gauge Status: Active and compliant tide gauge
Epoch: 1983 to 2001
8665530, Charleston, SC
NOAA's 2006 Published Rate: 0.01033 feet/yr
All values are expressed in feet relative to LMSL

Year	USACE Low	USACE Int	USACE High
1992	0.00	0.00	0.00
2002	0.10	0.11	0.14
2012	0.21	0.24	0.36
2022	0.31	0.39	0.64
2032	0.41	0.56	1.01
2042	0.52	0.74	1.44
2052	0.62	0.94	1.96
2062	0.72	1.16	2.54
2072	0.83	1.40	3.20
2082	0.93	1.65	3.93
2092	1.03	1.92	4.74
2102	1.14	2.21	5.62
2112	1.24	2.52	6.58
2122	1.34	2.85	7.61
2132	1.45	3.19	8.71
2142	1.55	3.55	9.89
2150	1.63	3.85	10.89

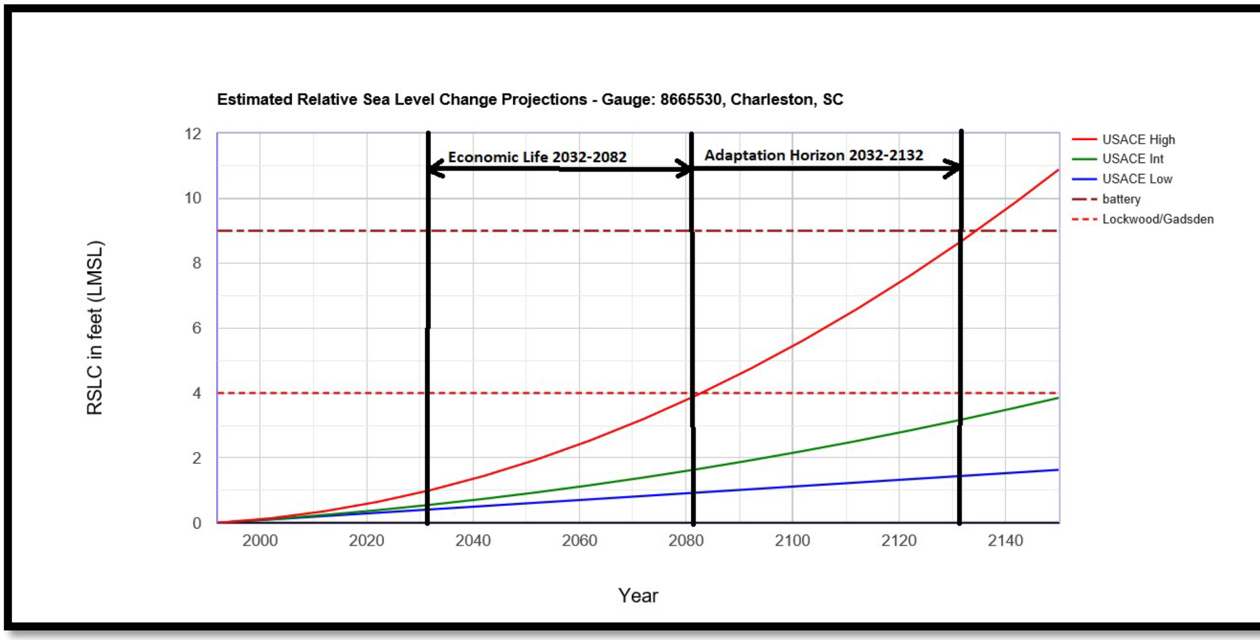


Figure 3.3.2.1 Low, Intermediate and High Sea Level Projection Gauge 8665530

The proposed project has an estimated construction completion in the year 2032. That would be a change in sea level of 0.41 feet for low rate of sea level rise, 0.56 for intermediate rate of sea level rise, and 1.01 feet for high rate of sea level rise, compared to the sea levels in 1992. USACE guidance suggests a 50 year economic life and 100 year adaptation horizon from the start of the project life. In 2082 (50 year economic life) the low rate of sea level change is 0.93 feet, the intermediate rate is 1.65 feet and the high rate of sea level rise is 3.93 feet. The 100 year adaptation horizon (year 2132) is projected to be 1.45 feet, 3.19 feet and 8.71 feet for the low, intermediate and high, respectively, compared to the sea levels in 1992 (Table 3.3.2.1).

Portions of Lockwood Dr, a primary road to the Medical District, are at elevation 5 feet NAVD88, with small portion at elevation 4 feet NAVD88. Gadsden Creek has connections to Hagood Ave and Fishburne, which have elevation 4 feet NAVD88. Based on the high rate of sea level change, high tide would flood these areas twice a day around the year 2085 (near the end of the economic life of the project), and for the intermediate rate of sea level change in the year 2150. The battery is overtopped at every high tide with a high rate of sea level rise around the year 2035. Based on the NWS threshold for "King tides" at 3.46 feet NAVD88 would occur every tide by year 2145 based on an intermediate rate of SLC.

3.4 SELECTION OF SEA LEVEL CHANGE FOR ANALYSIS

Using the USACE Sea Level Tracker (https://climate.sec.usace.army.mil/slr_app/) Figure 3.4.4 indicates trend of the last thirty years, which began lower than the historic trend and around 2006 to 2008 transitioned closer to the intermediate rate.

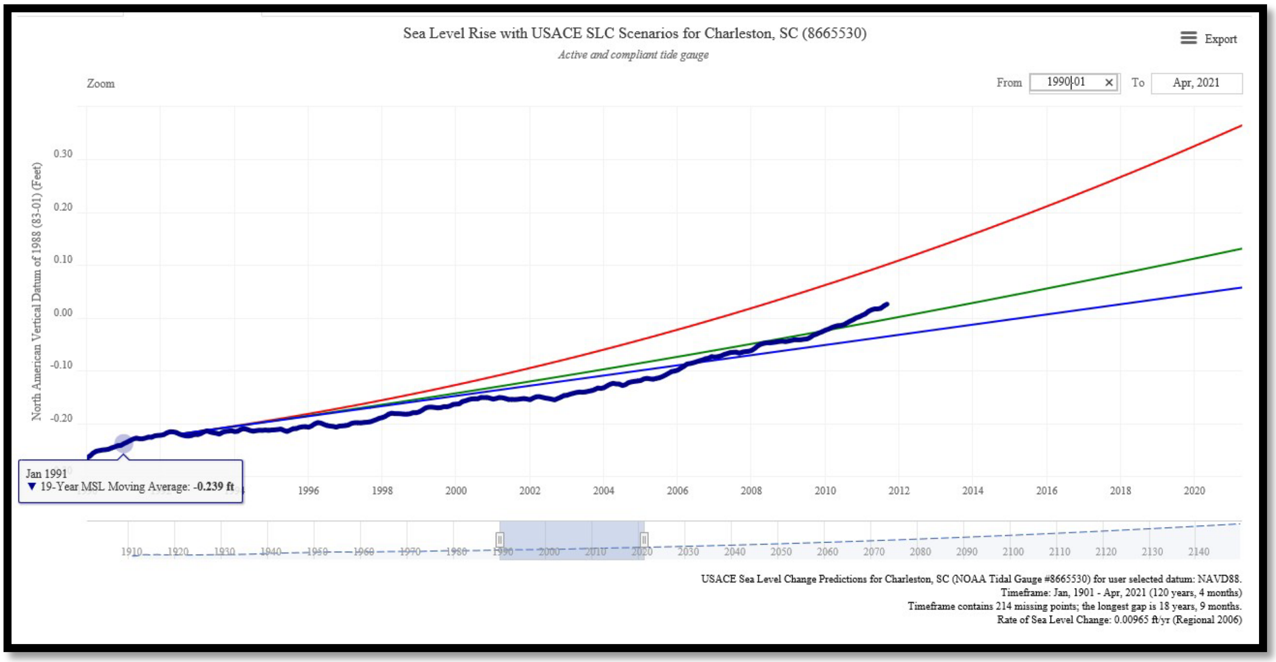


Figure 3.4.1 Sea Level Tracker Charleston SC (NOAA Station 8665530)

Consideration of sensitivity to sea level rise according to ER 1110-2-8162 and EP 1100-2-1 would not change the selection of an alternative since the alternatives were a wall with breakwater or wall without breakwater. The elevation of the wall and breakwater are scales of the alternatives. Using the different SLR only affects the exceedance probability of a selected elevation. There is not a targeted annual exceedance probability level for the project because the physical constraints of city infrastructure, bridges, topography and ongoing "low" battery wall reconstruction, limit the maximum elevation considered in the study to elevation 12 feet NAVD88.

Alternatives were evaluated using the intermediate SLC rate. The intermediate rate was chosen to balance the risk of formulating to the high or low scenario; however, all three sea level rise scenarios will be applied in G2CRM to address the benefits and damages of the selected wall elevation. These are discussed in the Economic Appendix.

The future condition for the economic considerations was performed using the intermediate rate of sea level rise for the 50 year economic life ending in 2082 as 1.65 feet for the purposes of hydraulic modeling. The 100 year adaptation range for the project intothe future (year 2132) would be 3.19 feet for the intermediate rate of RSLC and 8.71 ft for the high rate.

3.4.1 Extreme Water Level with Sea Level Change

The Extreme Water level can be added to the SLC curve and is based on NOAA data. According to the NOAA Tides and Currents website (https://tidesandcurrents.noaa.gov/est/index.shtml) “Extremely high or low water levels at coastal locations are an important public concern and a factor in coastal hazard assessment, navigational safety, and ecosystem management. Exceedance probability, the likelihood that water levels will exceed a given elevation, is based on a statistical analysis of historic values. This product provides annual and monthly exceedance probability levels for select CO-OPS water level stations with at least 30 years of data. When used in conjunction with real time station data, exceedance probability levels can be used to evaluate current conditions and determine whether a rare event is occurring. This information may also be instrumental in planning for the possibility of dangerously high or low water events at a local level. Because these levels are station specific, their use for evaluating surrounding areas may be limited. A NOAA Technical Report, "Extreme Water Levels of the United States 1893-2010" describes the methods and data used in the calculation of the exceedance probability levels. “

Adding a 10% Extreme water level (EWL) shown in Figure 3.4.2 to the intermediate rate of SLC, this is estimated to be 5.6 feet NAVD88 in 2032 and 6.69 feet NAVD88 in 2082. As indicated by the NWS thresholds (Table 3.1.1) the major flood stage is 4.86 feet NAVD88.

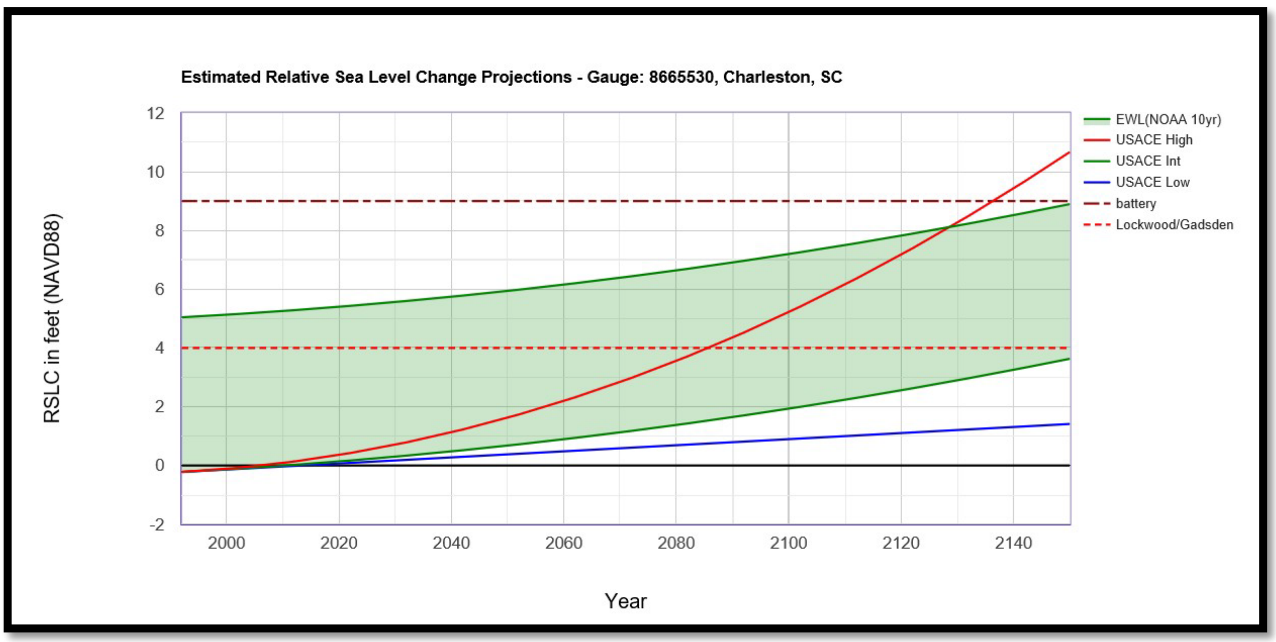


Figure 3.4.2 10% EWL on Intermediate rate of SLC.

The 1% EWL, Figure 3.4.3, added to the intermediate rate of SLC, is an estimated 7.23 feet NAVD88 in 2032 and 8.33 feet NAVD88 in 2082.

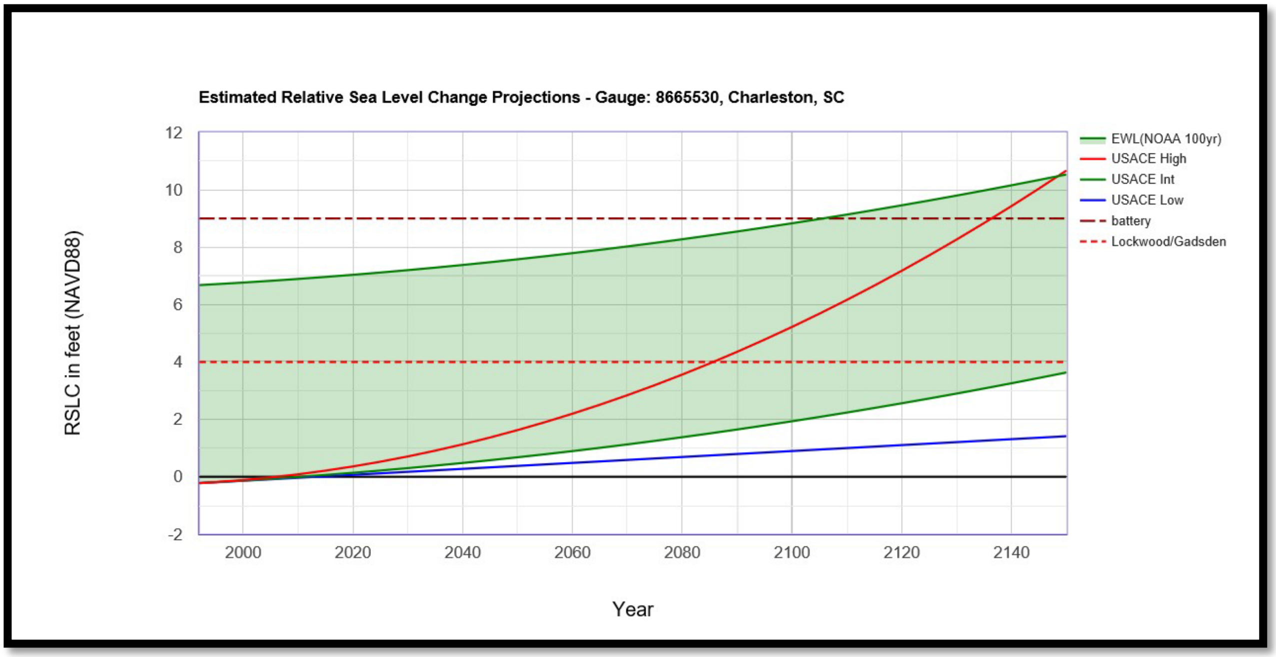


Figure 3.4.3 1% EWL with Intermediate Rate of SLC

3.5 SPONSOR SEA LEVEL CHANGE STRATEGY

As indicated in the City of Charleston's "Flooding and Sea Level Rise Strategy" published in February 2019, one of the objectives is to address flooding while promoting a more resilient and sustainable future in the face of recurrent flooding, rising seas, and more frequent extreme weather. The City of Charleston indicates the intent to use the latest NOAA 2017 sea level rise projections for future considerations (Figure 3.5.1 and Table 3.5.1). One way to track sea level rise is to document "minor coastal flooding" commonly called nuisance, sunny day flooding. The City indicates a marked increase in the number of days of minor coastal flooding over time. The NOAA sea level change curves in relation to USACE curves and other appurtenant information is provided throughout Chapter 3 of the Coastal Sub-Appendix B-4 report.

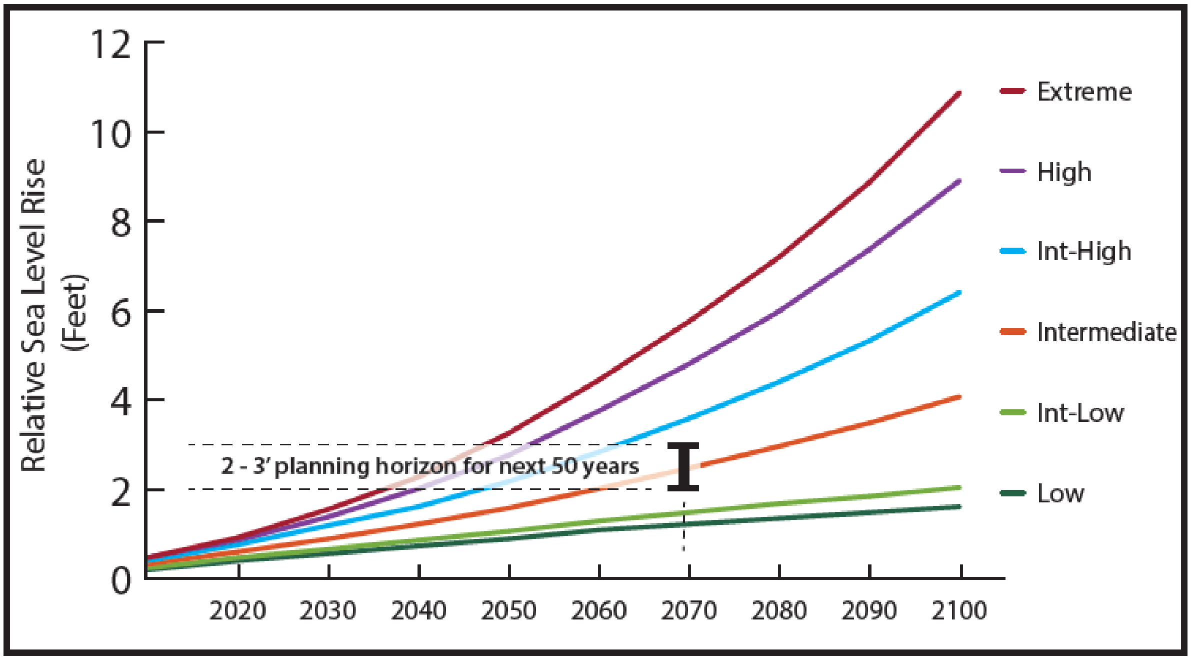


Figure 3.5.1 City of Charleston Planning Horizon for Relative Sea Level Change

Table 3.5.1 NOAA Relative Sea Level Change for Charleston

Charleston Peninsula
Scenarios for CHARLESTON I
NOAA2017 VLM: 0.00417 feet/yr
All values are expressed in feet

Year	NOAA2017 VLM	NOAA2017 Low	NOAA2017 Int-Low	NOAA2017 Intermediate	NOAA2017 Int-High	NOAA2017 High	NOAA2017 Extreme
2000	-0.15	-0.15	-0.15	-0.15	-0.15	-0.15	-0.15
2010	-0.11	-0.02	0.02	0.11	0.18	0.25	0.25
2020	-0.06	0.18	0.25	0.38	0.54	0.64	0.70
2030	-0.02	0.34	0.44	0.67	0.97	1.16	1.33
2040	0.02	0.51	0.64	1.00	1.39	1.79	2.05
2050	0.06	0.67	0.84	1.36	1.95	2.54	3.03
2060	0.10	0.87	1.07	1.79	2.61	3.53	4.22
2070	0.14	1.00	1.26	2.25	3.36	4.58	5.53
2080	0.19	1.13	1.46	2.74	4.18	5.76	6.97
2090	0.23	1.26	1.62	3.26	5.10	7.14	8.64
2100	0.27	1.39	1.82	3.85	6.18	8.68	10.65

The City of Charleston's Sea Level Rise Strategy (2015) originally recommended a 1.5 to 2.5-foot elevation increase for new facilities and infrastructure. The City increased the recommendation to 2 to 3 feet for the revised 2019 sea level rise strategy. The projection of a 2 to 3 foot rise in 50 years is higher than the USACE intermediate rate of rise (+1.65 feet) being utilized for the USACE peninsula study in the year 2082 (50-year period of analysis from end of construction). The City of Charleston uses the projection of 2-foot increase for less vulnerable infrastructure such as parking lots, while a 3-foot increase is for more critical long-term infrastructure, such as medical facilities.

In 2019, the City of Charleston began reconstructing and raising the Low Battery Seawall to account for sea level rise projections. The Low Battery Seawall is currently being raised to an elevation similar to that of the High Battery Seawall. The USACE study assumes this project for the future without-project conditions while the future with-project condition would need the wall raised again to 12 feet NAVD88. The City has also begun the Check Valve Program which is a plan to equip the peninsula outfalls with check valves. A check valve or “flap gate” prevents seawater from backing up into drainage infrastructure to mitigate tidal flooding, while still allowing the outfall to drain stormwater as usual when the tide recedes. Many outfalls in the City are gravity fed and drain to bodies of water that are tidally influenced. During high tides, seawater often enters storm water outfalls and water can back up far enough in low lying areas to result in backflow flooding on streets, even on a sunny day. The City has begun the installation and replacement of check valves on the outfalls. Some of the outfalls currently have a duck bill type check valve and are to be phased out in favor of in-line valves which function better and have less maintenance costs. USACE assumes all peninsula outfalls to be equipped with check valves by the USACE with-project end of construction year (2032).

In addition to tidal flooding and sea level rise, rainfall induced flooding is a significant challenge for the City, and flooding is exacerbated when rainfall and high tide/storm surge occur at the same time. While check valves on the outfalls work well to mitigate flooding from high tides entering the storm drains. Check valve or not, rain that falls during a high tide still has little room to drain and/or increased resistance to drain (as opposed to low tide) until the tide recedes. During such cases, the stormwater collects on the surface because the storm drains are full of seawater. In addition, if the check valve is in the closed position holding pressing seawaters then additional ponding on the streets may occur. The USACE PDT is proposing storm gates for the surface flow culverts that align with the proposed storm surge wall alignment. These culverts convey inland surface runoff and/or allow for daily tidal fluctuations. The storm gates placed on the culverts are to be closed only during predicted storm events that bring tidal flooding. There has been discussion between the City and USACE about the potential of upsizing culverts during construction of the storm surge wall. Further discussion on this matter is to take place during PED phase.

The City has begun and completed many other drainage improvements such as the Market Street improvement which connects to the Concord Street Pump Station. Another project is the Spring Fishburne Pump Station which is currently under construction. Further information about the City’s existing and future drainage improvements are provided in Section 1.3 of the Interior Drainage Analysis Sub-Appendix. Along with drainage improvements are spotlights on drainage maintenance because a stormwater drainage system performs best when properly maintained. This includes procedures such as keeping drainage ditches, conveyance pipes, and storm drains as clean as possible. As of 2019, the City contracted with a group of engineers and subject area experts to form the new Stormwater Program Management Team. The team is to update the City’s Master Drainage and Floodplain Management Plan, which was last revised in 1984. Another focus of the team is implementing GPS, GIS, and sub-surface camera technologies to schedule, inspect, and monitor both the surface flow drainage ditches and sub-surface stormwater drainage tunnels and pipes.

As mentioned, the City has many drainage improvements completed and/or underway. An important feature for both the City and the PDT are pump stations. The PDT has accounted for three City pump stations in the future without-project condition and in the future with-project condition conceptual plans and modeling. The PDT is proposing both permanent and temporary pump stations, meaning the permanent pump stations will contain permanent pump houses with larger pumping capacities while temporary pumps are deployed during storms and typically contain smaller pumping capacities. The City’s Flooding and Sea Level Rise Strategy also dictates using strategically placed temporary pumps, with appropriate storm forecasting notice, to remove stormwater and tidal inundation to mitigate the risk of flooding to the inland area. The City’s pump stations such as the Spring Fishburne are thoroughly described in the City’ sea level rise strategy (2019). The pump stations contain storm pipes which bring stormwater to the stations which is then pumped underground to the surrounding rivers. The PDT plans to use the existing infrastructure for bringing stormwater to the pump stations. The City has stated the storm pipes accommodate no more than a 10% AEP rainfall event and, in some areas, a lesser capacity is provided by the storm pipes. Once the pipes become overwhelmed water begins to collect on the streets. Therefore, surface flow runoff becomes a larger component of drainage during heavy rainfall events and/or events where the storm drains are filled with backflowing seawater. This is an important aspect and assumption for the PDT as the hydraulic model (HEC-RAS 2D) for interior drainage is a surface flow only model and does not have the capability to model sub-surface flow. At this phase of the CSRM study, the City does not have full coverage or a complete model of the storm pipe network. During PED phase, further information about the storm pipe network will need to be incorporated to more appropriately size and place the PDT's recommended plan for pumps. The PDT has strategically placed pump stations using HEC-RAS 2D by assessing the natural drainage paths of surface flow runoff. In addition, the HEC-RAS modeling is supplemented with some of the City's GIS based layers for visual assistance in the placement of pump stations. These GIS layers provide an important information for the conceptual layout of the future with-project pump stations. The referenced layers include the storm pipe network (layout/inlets/outlets) and the peninsula watershed delineation. The watershed delineation refers to surface flow and further information about the storm pipe (sub-surface) delineation or pump servicing boundaries is needed during PED phase to ensure appropriate design.

The USACE PDT will continue to coordinate with the City engineers during PED phase to ensure the strategies, goals, and collaboration of the project features are adequately aligned.

Source: https://www.charleston-sc.gov/DocumentCenter/View/20521/Flooding-and-Sea-Level-Rise-Strategy-2019-printer-friendly?bidId=

CHAPTER 4 -STORM SURGE AND WAVE DATA MODELING

4-1 Models

As previously stated, there were no existing USACE studies addressing Coastal Storm Risk Management. USACE reached out to SCDNR, the FEMA POC for Flood Insurance Studies (FIS) in the state of SC, for available coastal models to minimize costs and improve efficiencies of the study. FEMA/SCDNR contractor, AECOM, provided Advanced Circulation (ADCIRC) models, storm sets, SWAN runs, all the validation runs, production runs and input for their 2017 preliminary FIS (which became effective January 2021). These data were provided to ERDC for initial analysis.

The ADCIRC model is a high-performance, cross-platform, finite element numerical ocean circulation model popular in simulating storm surge, tides, and coastal circulation problems. The numerical model SWAN (Simulating WAves Nearshore), used for the computation of wave conditions in shallow water with ambient currents, is briefly described. The model is based on a fully spectral representation of the action balance equation with all physical processes modelled explicitly and is often coupled with ADCIRC.

STWAVE (STeady State Spectral WAVE) is a steady-state, finite difference, spectral model based on the wave action balance equation. STWAVE allows coastal project engineers to numerically model wave generation and transformation over complex bathymetry, interaction of waves with currents and structures, and propagation of waves in entrances and harbors. Available SWAN results, obtained from the FEMA contractor, were comprised of time series of bulk scalar parameters, including wave height, period, and direction.

The storm surge levels were determined by using the ADCIRC hydrodynamic model coupled with the Steady State Spectral Wave (STWAVE) model to complete a series of model runs with input data from artificial storms created using the Joint Probability Method (JPM) statistical analysis from FEMA. The outputs of FEMA SWAN node 16763 (Lat/Long: -79.5723836732 32.5145493492) served as the time series from which the spectra were constructed for the STWAVE.

ADCIRC and STWAVE are the high-fidelity storm surge and nearshore wave models combined to provide the driving forces of storm hydrographs (surge and waves) at locations needed for the Generation II Coastal Risk Model (G2CRM) analysis. G2CRM supports planning-level studies of hurricane protection systems (HPS). The G2CRM is distinguished from other models currently used for that purpose by virtue of its focus on probabilistic life cycle approaches. This allows for examination of important long-term issues including the impact of climate change and avoidance of repetitive damages. Key features of the model include the ability to use readily available data from existing sources and corporate databases and integration with geographic information systems (GIS). The G2CRM generates a wide variety of outputs useful for estimating damages and costs, characterizing and communicating risk, and reporting detailed model behavior, in the without-project condition and under various plan alternatives for the with-project conditions.

4-2 ADCIRC and STWAVE Modeling

To better capture the results of any structural measures of the study, the ADCIRC grid needed to be modified within the study area and rerun for a suite of storms (Figure 4.1 and Figure 4.2). ERDC evaluated the suite of storms provided by AECOM and selected a subset of storms. The goal of storm selection was to find the optimal combination of storms given a predetermined number of storms to be sampled (e.g., 20 TCs), referred to as reduced storm set (RSS). In the process of selecting 20 TCs, it was determined that an RSS of this size adequately captured the storm surge hazard for the range of probabilities covered by the FSS (122 TCs). In order to also include high frequency events, five (5) additional storms were selected from the range of probabilities determined from extreme value analysis (EVA) of water level measurements. Details are found in ERDC report.

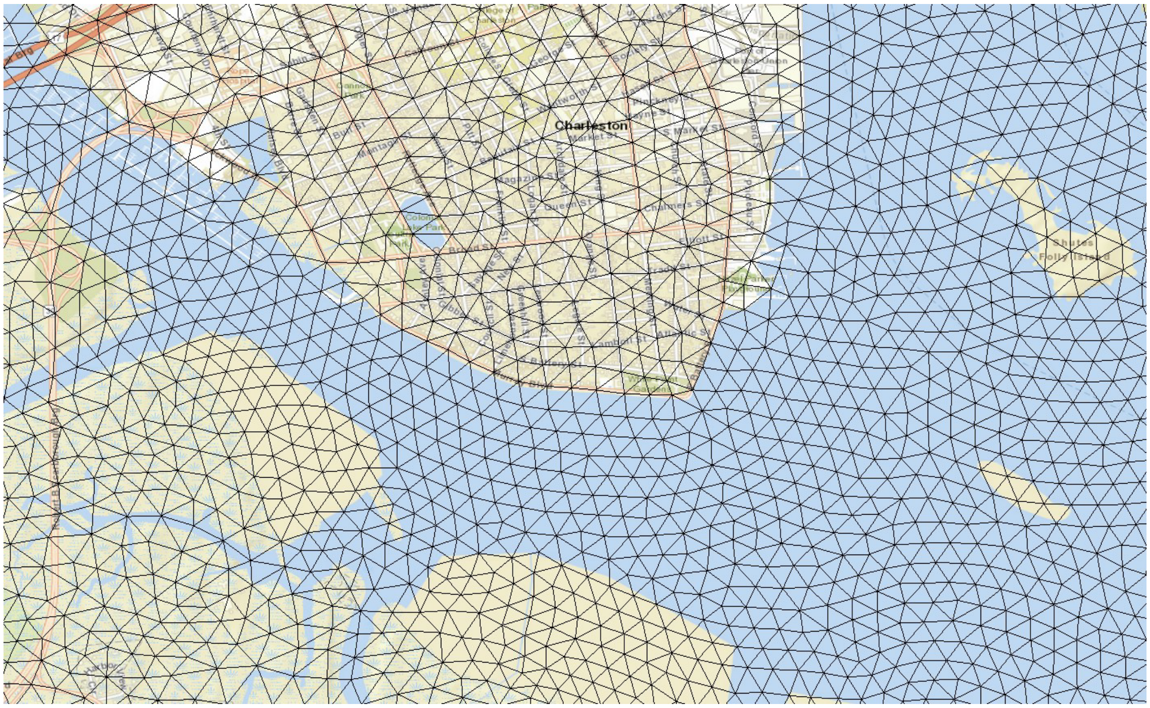


Figure 4.1 Zoom in to the Charleston Peninsula (before the grid refinement).

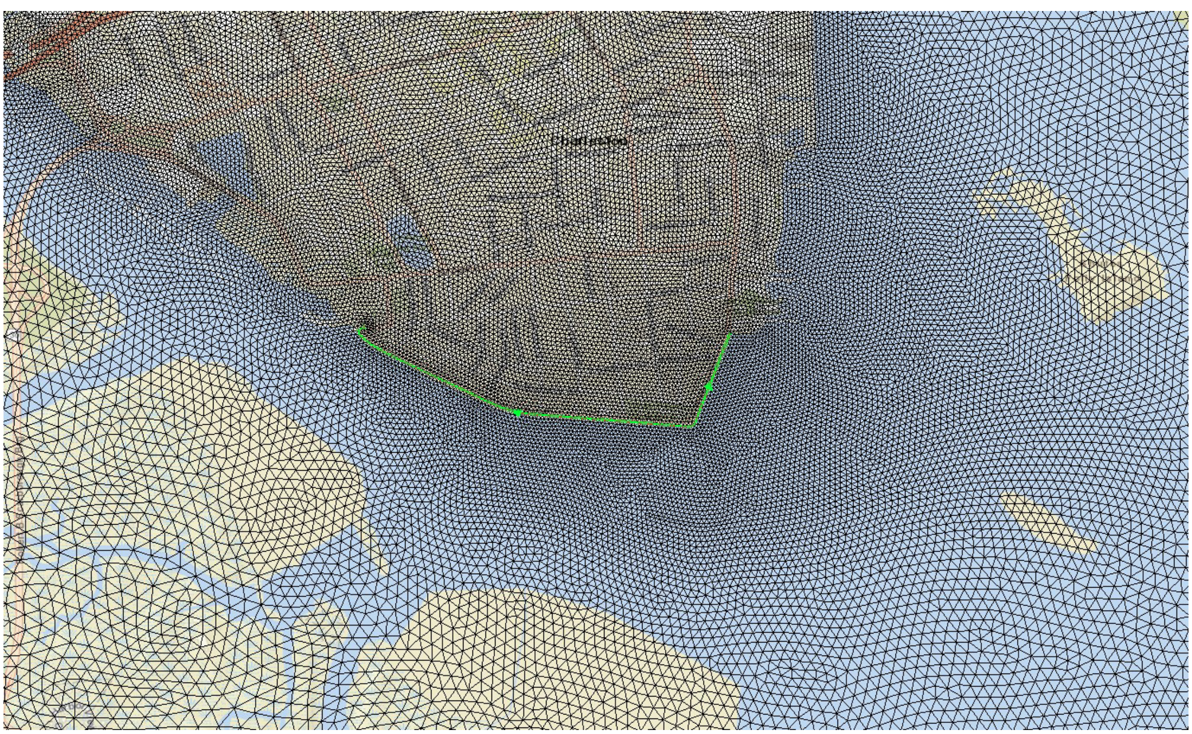


Figure 4.2 Zoom in to Charleston Peninsula after grid refinement and FWO condition of battery wall

Waves were simulated by coupling ADCIRC with STWAVE. Figure 4.3 shows the STWAVE domain for the analysis.

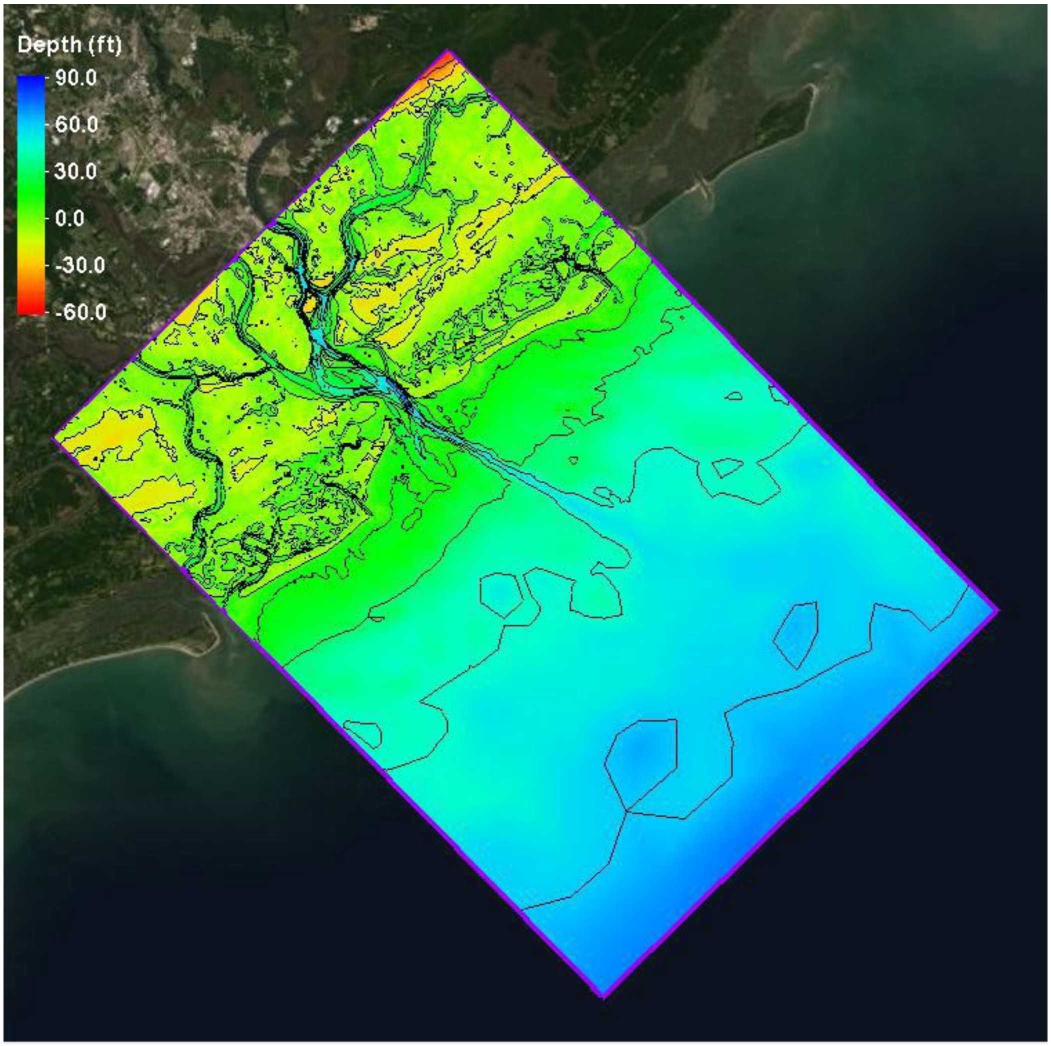


Figure 4.3 STWAVE Domain

ERDC was asked to run STWAVE and ADCIRC for three scenarios to generate time series still water elevations for input into the G2CRM model. The three scenarios were: existing, future without and future with a breakwater as a wave attenuator. This analysis is discussed in detail in the ERDC Coastal Modeling subappendix and ultimately led to elimination of the breakwater as an alternative.

Coastal analysis generates the still water elevation. As stated in the FIS, "the still water surge elevation is the water elevation due solely to the effects of the astronomical tides, storm surge, and wave setup on the water surface but which does not include wave heights. The inclusion of wave heights, which is the distance from the trough to the crest of the wave, increases the water-surface elevations. The height of a wave is dependent upon wind speed and duration, depth of water, and length of fetch. The wave crest elevation is the sum of the still water elevation and the portion of the wave height above the Stillwater elevation. "

As explained in the SOUTH CAROLINA STORM SURGE PROJECT DELIVERABLE 3: PRODUCTION RUNS, FINAL STATISTICS, AND RESULTS ANALYSIS report generated by URS for FEMA/SCDNR. "The tide range in South Carolina is up to 6 feet (ft), suggesting that the tide phase at the time of landfall may significantly influence the surge levels produced by a given storm." The report states that simulations were run to estimate the influences of steric effects on water levels throughout the project area and ultimately determined that these fluctuations could obtain a total increase of 2.75 inches above MSL. Therefore, steric effects were minimal compared to the magnitude of tides.

See the ERDC Coastal Modeling subappendix for the ERDC modeling report that includes the STWAVE and ADCIRC modeling used to select FWP alternatives.

4.3. G2CRM Collaboration

4.3.1 Models Areas

Model Areas (MA) were needed by Economics to break the city into manageable areas for G2CRM assessments. The determination of MA boundaries considered topography and the drainage pathways of the various areas, as well as land use (i.e. the Columbus Street Terminal had to remain whole). The Model Areas were identified by the primary land use of the area (Figure 4.4).

Wagener Terrace: Identified as Wagener Terrace for the large residential area, covers the area from the upper limit of the study area on the Ashley side around the Wagener Terrace area to Citadel -which is high ground, - includes commercial, undeveloped and residential land use.

Marina: Identified as Marina due to the public marina along the shoreline, covers from Citadel to Low Battery (by the Coast Guard) and includes residential and hospital areas.

Battery: Identified as Battery because it follows the low and high battery walls, extends from the Coast Guard to the end of the High Battery by the Historic Foundation and Yacht club. This area is characterized by much of the historic homes.

Port: Identified based on the large SCPSA port facilities along the shoreline extends from High Battery end at the historical foundation/Yacht Club to just past Columbus Terminal. The area includes historic homes, commercial, port areas.

Newmarket: identified by the historic creek that drains much of the areas extends from Columbus Terminal across Newmarket creek to the upper limit of the study area on the Cooper side. And includes - residential (low income), commercial properties.

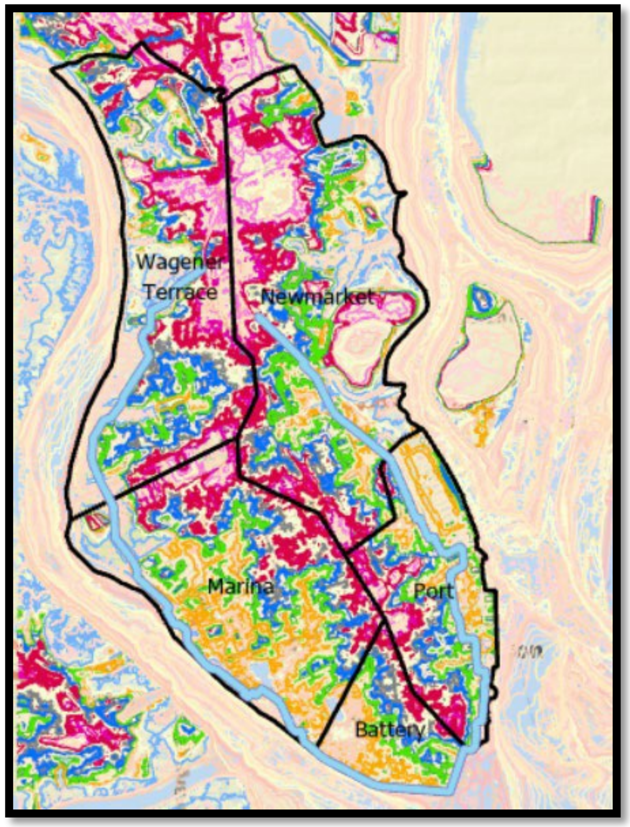


Figure 4.4 Map depicting Model Areas

4.3.2 ADCIRC Water Levels

From the dataset of over 1000 points, 5 were selected to represent the Model Areas used for G2CRM (Figure 4.5).

Figure 4.5 Location of Save points for the Model Areas

4.3.3 G2CRM Driving Forces

The G2CRM was the tool used to evaluate the alternatives (wall only or wall plus breakwater) and scales of alternatives (different wall elevations and different breakwater sizes). In addition to the driving forces from ADCIRC and STWAVE, G2CRM uses local tidal stations for the addition of tide and the three USACE sea level formulas are embedded in G2CRM to include future sea level conditions. Other data in the G2CRM model that required ERDC support include storm probabilities. As indicated in the ERDC Coastal Modeling subappendix on page 39, G2CRM estimates the wave's contribution to total water level by multiplying the wave height by 0.7 to find the adjusted wave contribution, then adds this value to the water level. Wave height data were calculated in STWAVE, so these values were imported directly and UseWaveDataAsIs was set to 1. The User's Manual indicates this is based on FEMA methods, which are acknowledged in this document "Wave Setup, FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report, February 2005" (https://www.fema.gov/sites/default/files/2020- 03/frm_p1wave1.pdf). See the Cost Engineering Sub-Appendix for more discussion on the G2CRM model specific to this project and the G2CRM User's Manual for more discussion.

Because ADCIRC uses MSL in meters as its vertical datum, the still water elevations are generated in meters at

MSL and were then converted to feet NAVD88 (to remain consistent with G2CRM input), as shown in Figure 4.6. The G2CRM model was then used to evaluate the wall footprint and elevations as a stand-alone option (Alternative 2) and in conjunction with a breakwater wave attenuator (Alternative 3). Based on the storm hydrograph levels (surge and waves) generated with ADCIRC and STWAVE, combined with the tide information and intermediate rate of sea level rise, the elevation of 12 feet NAVD88 (wall only) was selected as the scale of alternative 2 based on G2CRM analysis. The overtopping due to wave action and exceedance probability associated with elevation 12 feet NAVD88 with consideration of confidence limits is discussed in subsequent sections.

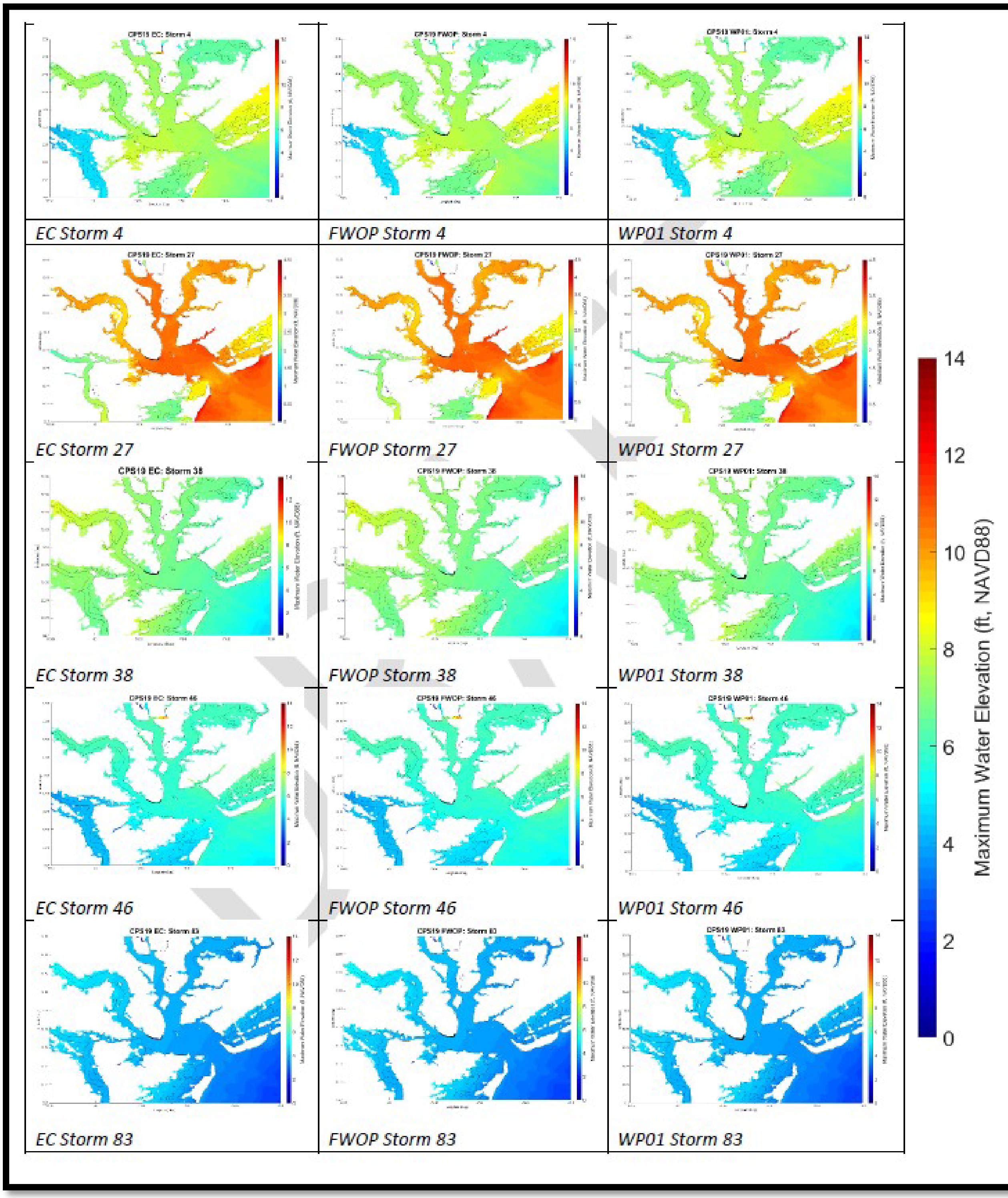


Figure 4.6 Plots of Still Water Elevations

CHAPTER 5 - ENGINEERING EVALUATION

5.1 Wave Overtopping and Non-Linearity

5.1.1 Hydrodynamic Condition

Figure 5.1 shows the project area with purple and pink lines representing the proposed flood wall with height +12 ft NAVD 88. Red dots show 9 representing stations where statistical Still Water Level (SWL) and wave information are available which are used to calculate wave overtopping flow using EUROTOP method.

Figure 5.1: Charleston Harbor Project Area

Figure 5.2 shows the location of representing stations with bathymetric depth. The numbers in black represent bathymetric depth in meters (NAVD88). Figure 5.3 shows still water level (SWL) at different points under different Annual Exceedance Probability (AEP). We notice little variability in SWL among various points across the harbor. For example, for any representing station, 1% AEP (formerly referred to as 100-year return period) SWL (without considering sea level rise) is 3.1m (10.2 ft).

Although SWL does not vary, significant wave height (HS) varies depending on the location. Figure 5.4(a) shows significant wave height along the Western side of the harbor where 1% AEP wave height is between 0.5 and 0.6 m. Figure 5.4(b) shows significant wave height along the Eastern side ofthe harbor where 1% AEP wave height is between 0.8 and 1.4 m. Figure 5.4(c) shows significant wave height along the southern tip of harbor where 1% AEP wave height is between 0.7 and 1.2 m.

In general, due to deeper water and long fetch, the eastern and southern parts of the harbor experience more wave energy.

Figure 5.2: Representing Stations with Bathymetry Information

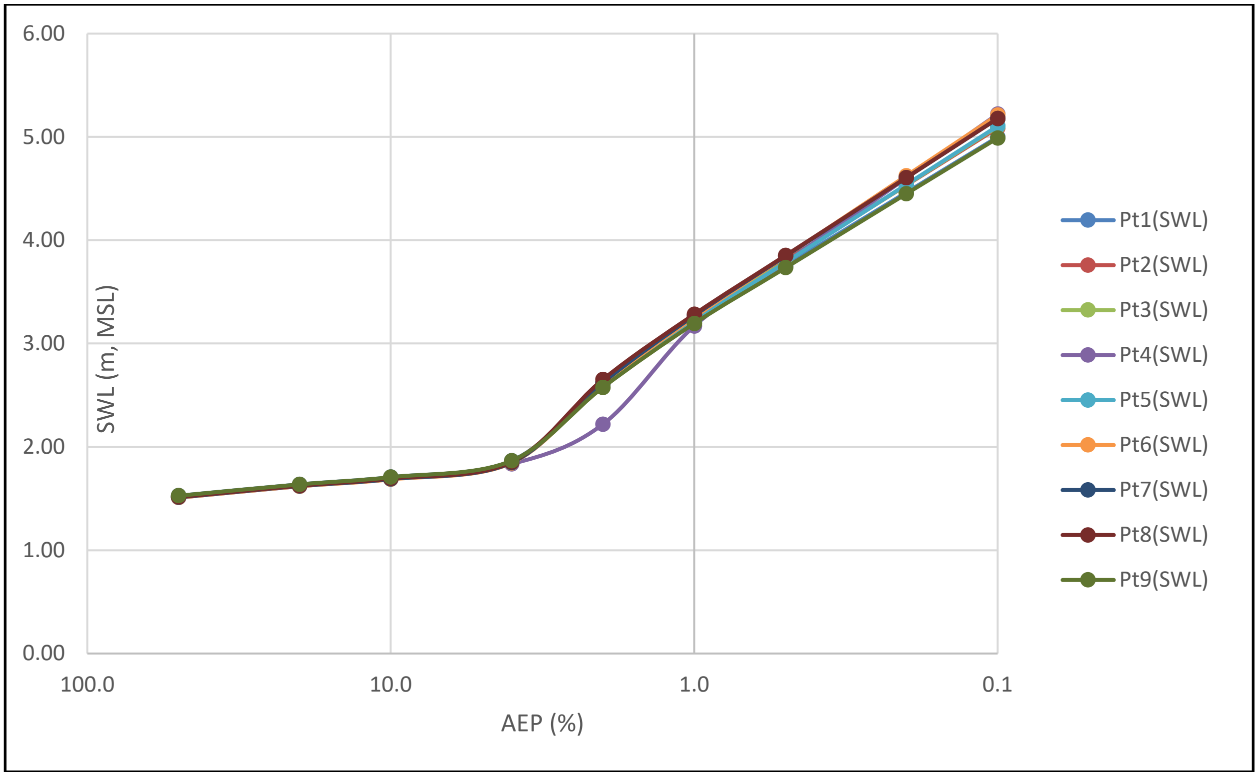


Figure 5.3: Still Water Level at Different Stations

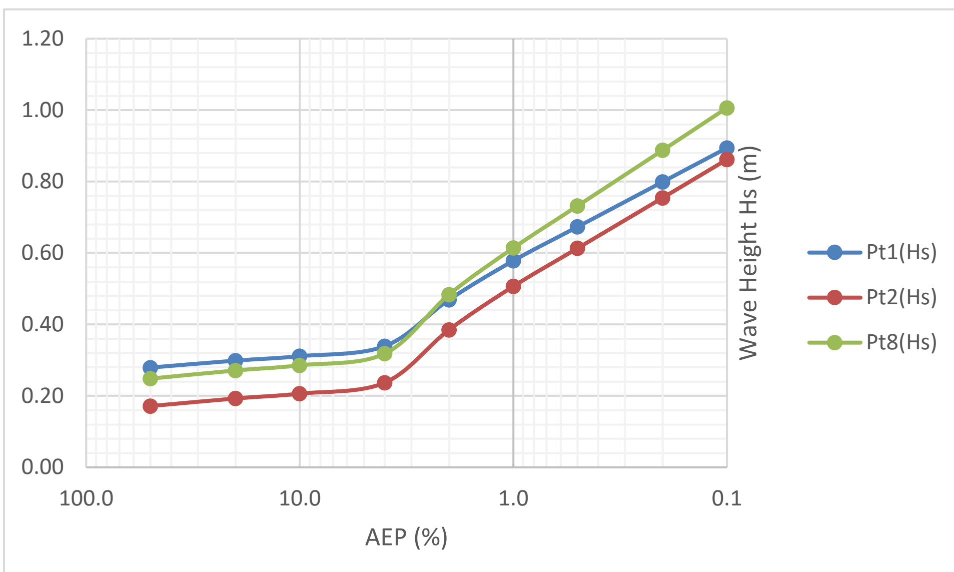


Figure 5.4 (a): Significant Wave Height (Hs) at stations 1, 2, and 8.

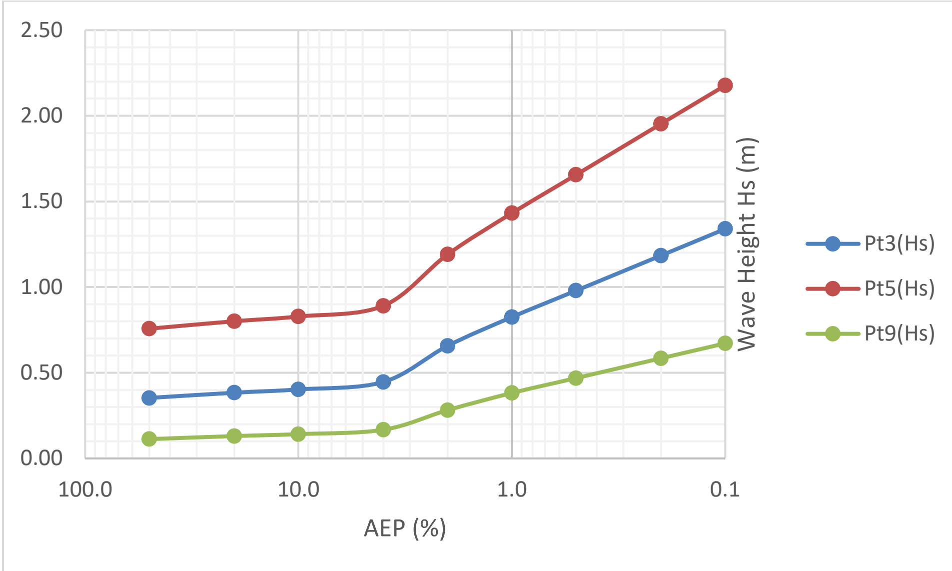


Figure 5.4 (b): Significant Wave Height (Hs) at stations 3, 5, and 9.

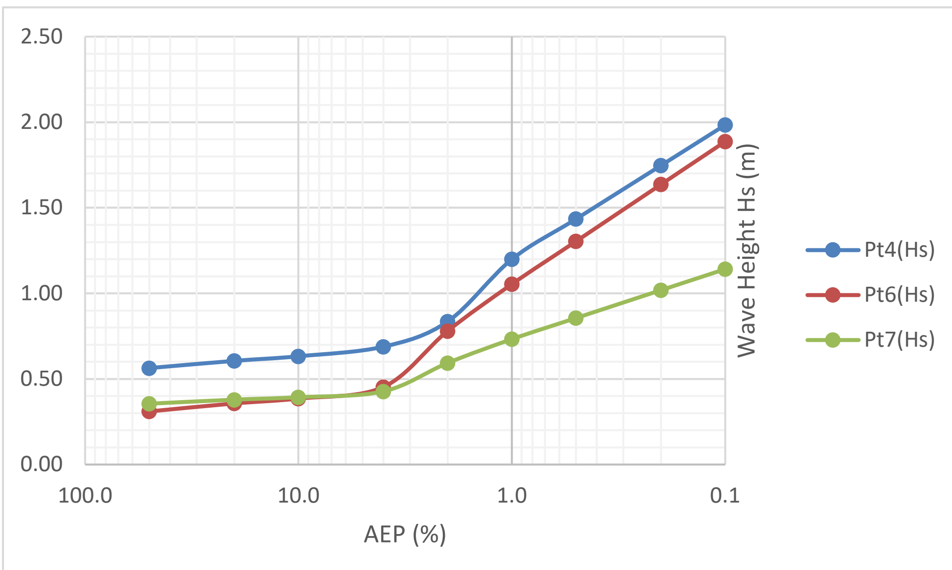


Figure 5.4 (c): Significant Wave Height (Hs) at stations 4, 6, and 7.

5.1.2 Methodology:

5.1.2.1 Non Linear Residual (NLR): Probabilistic water levels at a given year with a particular return period under a sea level curve scenario are usually calculated using linear superposition. It is common practice when assessing water levels in coastal studies to separately consider components, such as storm surge, tide, and RSLC, before combining them through linear superposition to determine the total water level. The use of linear superposition sometimes introduces an error due to the complex nonlinear interaction of the water level components. This error is referred to as the nonlinear residuals (NLR). The nonlinear residuals are added while calculating probabilistic storm surge at a location of interest under different sea level rise scenario. For SACS SA study, 3 cases were simulated which are:

SLR = 0 (Present Condition)
SLR 0.8321 m = 2.82 ft represents intermediate curve at year 2120
SLR 2.2404 m = 7.44 ft represents high curve at year 2120

In this analysis, NLR was calculated by subtracting SWL with linear superposition of RSLC from simulations using RSLC at the beginning of simulations. by proper adjustments using this formulation:

$$NLR = SWL_{RSLC} - (SWL+RSLC)$$

Figure 5.5(a) and 5.5(b) show the distribution of NLR at representing point 6 using approximately 1200 simulated storms. Interestingly, for this project area NLR is found to be negligible with slight negative bias. Mean NLR is found to be 3 cm and standard deviation is also 3 cm.

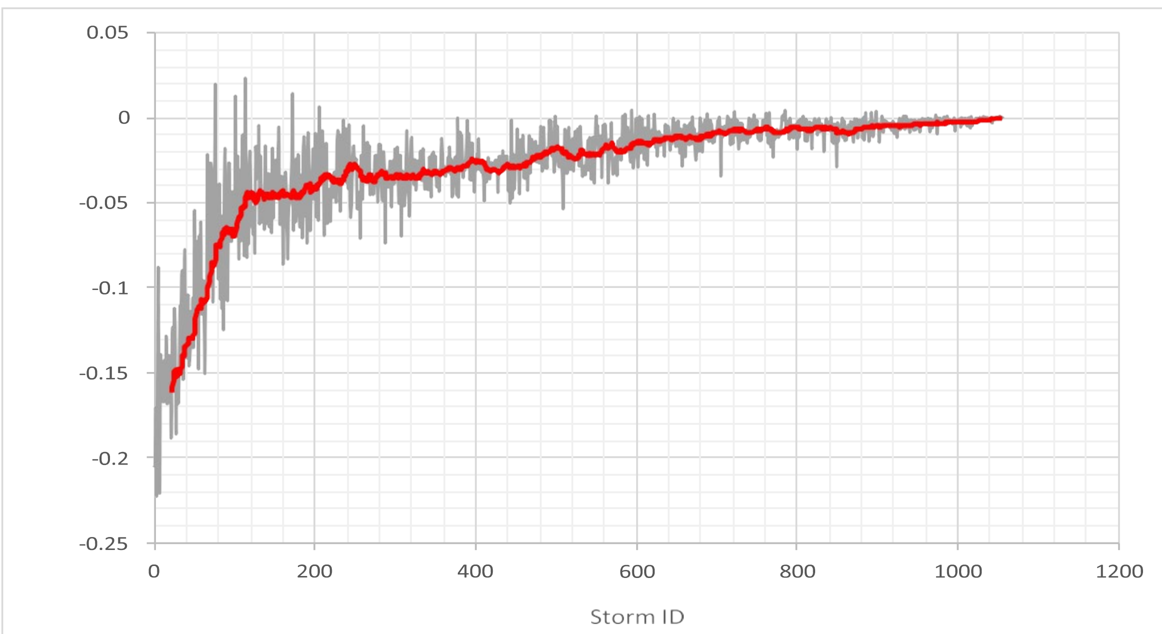


Figure 5.5(a): Distribution of NLR for Station 6

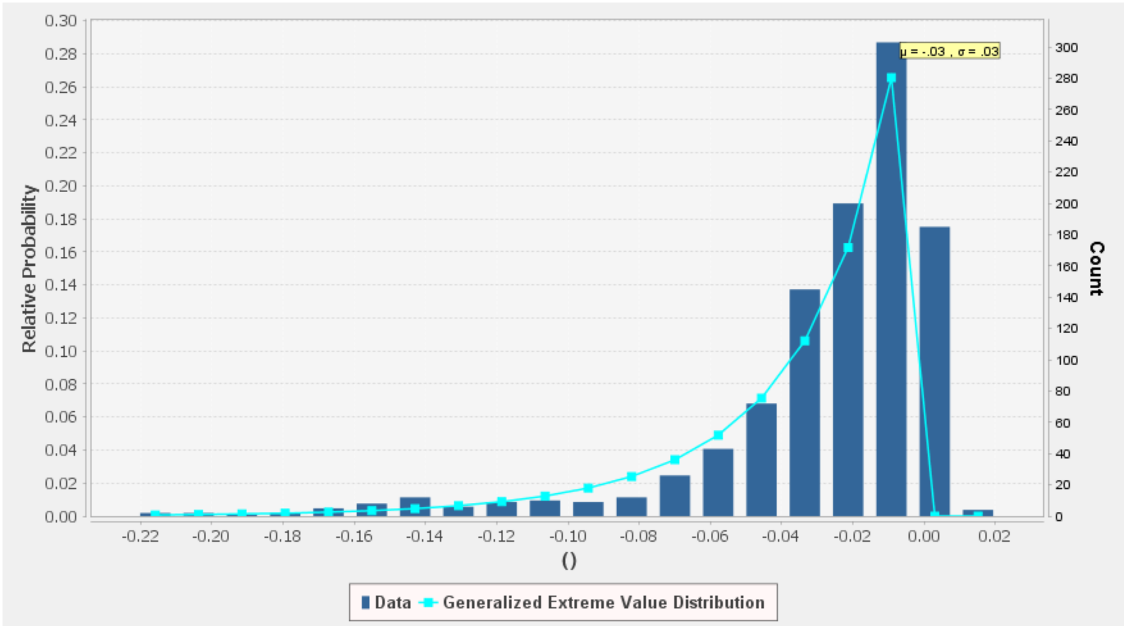


Figure 5.5(b): Frequency Distribution of NLR for Station 6

Negligible NLR are further investigated with Figure 5.6(a) and 5.6(b). These color contours show the wetted starting water column height under each of the SLR scenarios. Inundation extents are more pronounced away from the general area of the save points (black dots). Notice that around the save points, there isn't much extra inundation under the different SLR. However, the wetland areas north and south along the coast show a lot more inundation. This likely leads to the slightly negative nonlinear bias at the save points, as water is likely being redirected away from the save point locations.

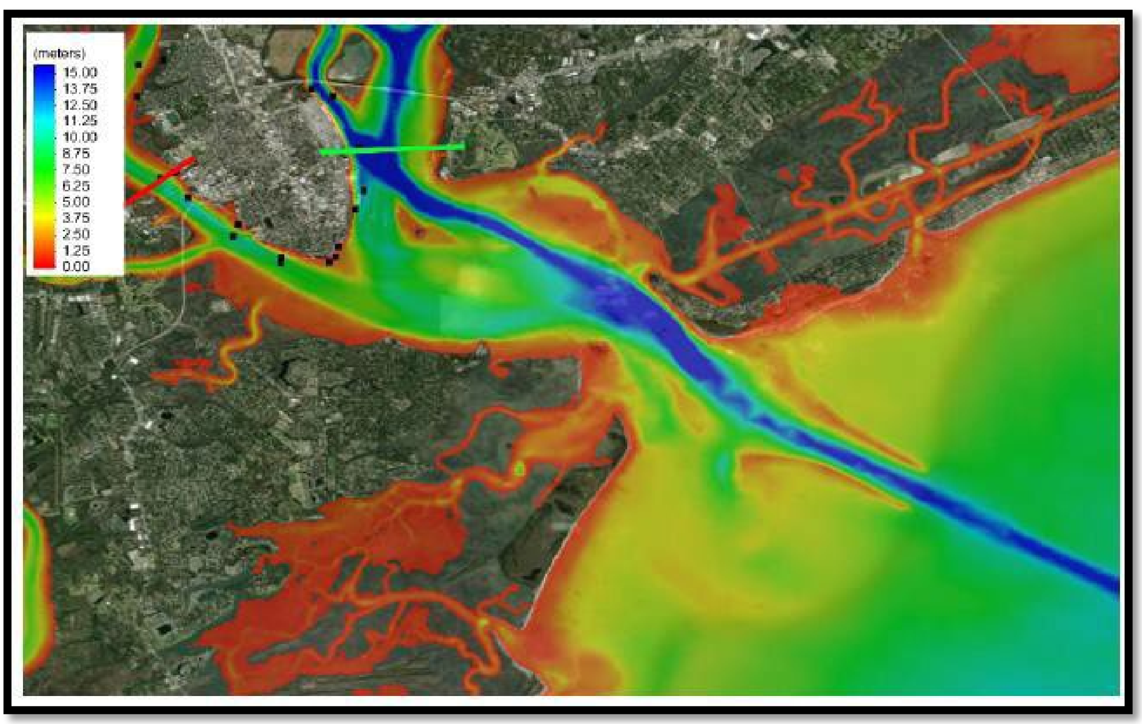


Figure 5.6(a): Water Column Height (SLC0 Case, SLR=0)

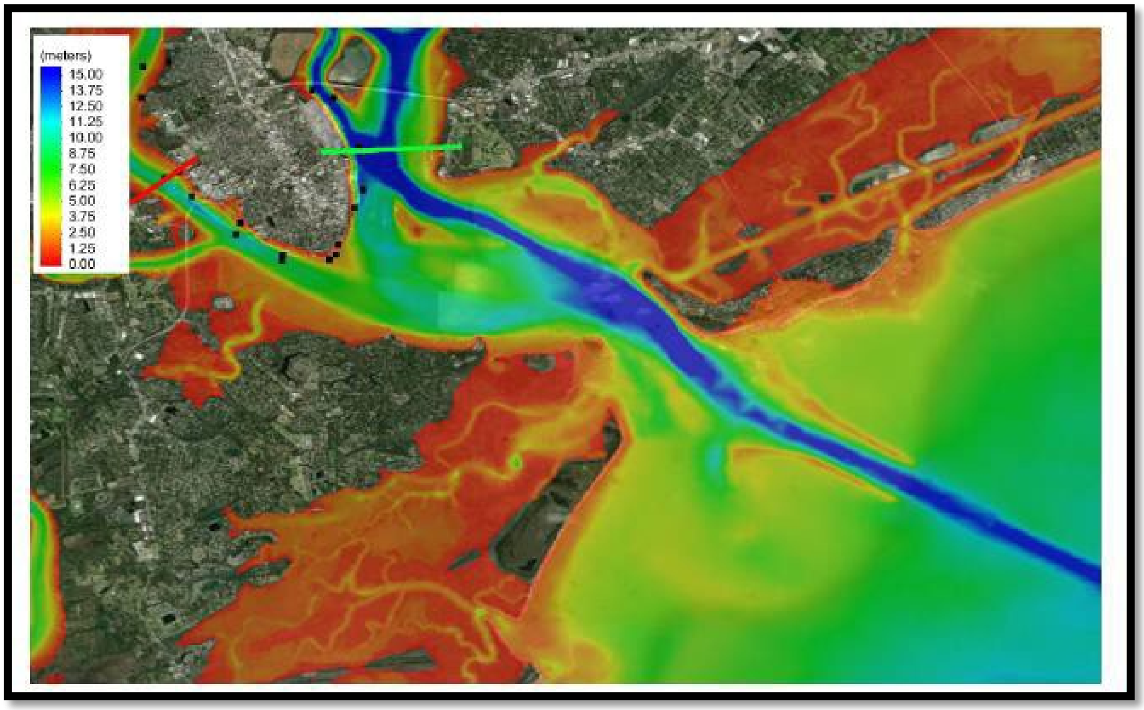


Figure 5.6(b): Water Column Height (SLC1 Case, SLR=2.82 ft)

Figure 5.7 shows the distribution of NLR at different stations across Charleston Harbor. NLR is found to be negligible (0.1 ft). Main factors influencing this negligible NLR influence compared to the Gulf is the size of this water body (relatively small) its close proximity to the open ocean and with a fairly wide/deep channel leading into it. As such, it has been concluded that NLR is very weak at those locations and is likely safe to proceed using a linear superposition at those locations without noticeable error.

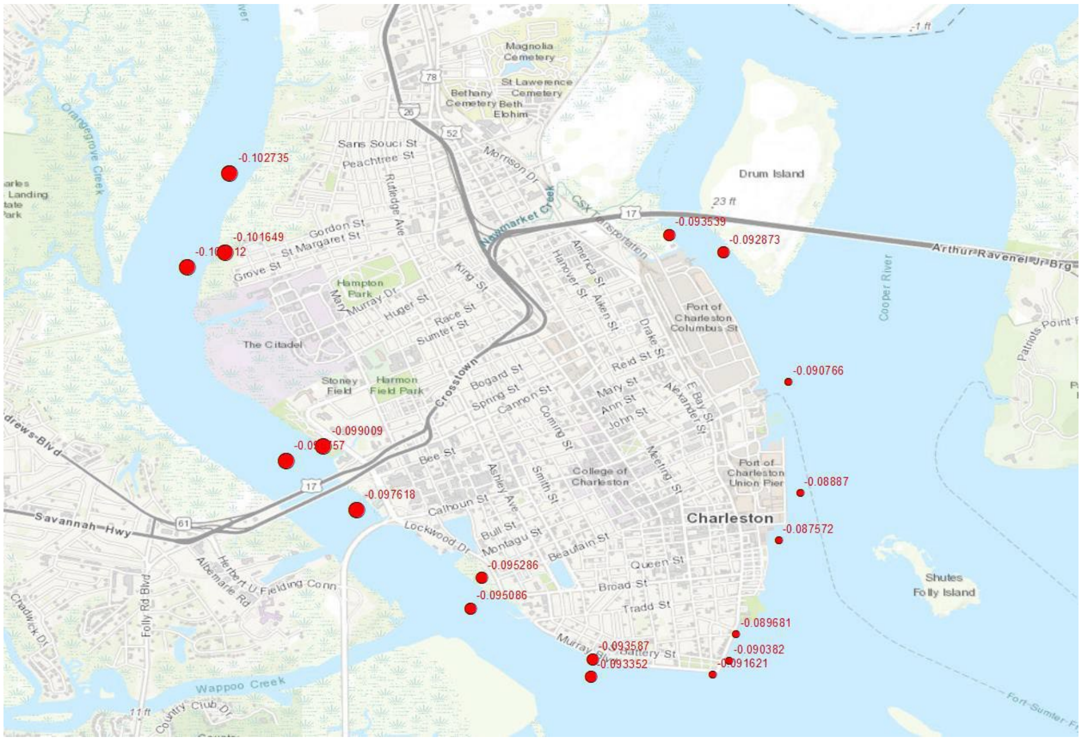


Figure 5.7: Distribution of NLR (In Ft) across Different Stations

5.1.2.2 Correction of Significant Wave Height for RSLC Condition

As wave statistics (HS) for RSLC condition are not available during this time, a correlation is needed to extrapolate HS values associated with SWL with RSLC values.

(1) Correlation between SWL and Wave: Figure 5.8 (a) below shows the location of extraction points. Point 6 is the most exposed location for waves.

Figure 5.8(a) Location of Extraction Points

(2) The graph below (Figure 5.8(b) shows a plot showing simulated SWL and significant wave height at present day condition for point 6. The data shows a linear trend with little scatter with correlation coefficient above 0.9. This justifies extracting wave information from the representing trendline from a given SWL with or without using RSLC.

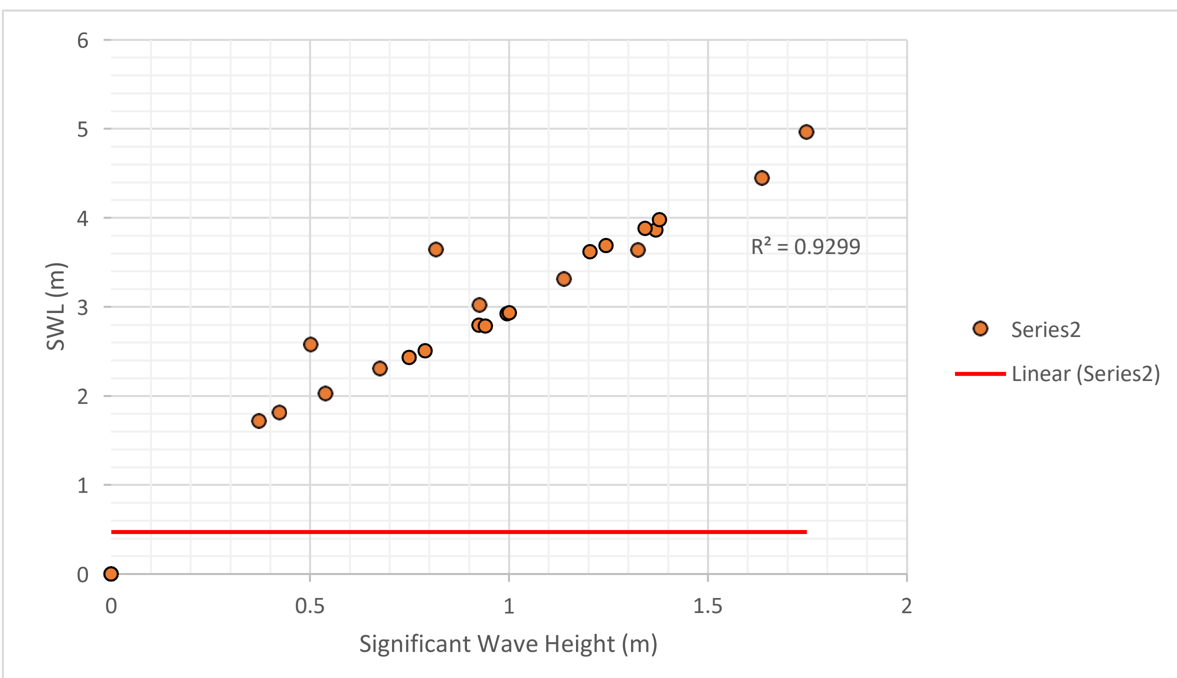


Figure 5.8(b) Simulated SWL and significant wave height Point 6

The graph below (Figure 5.8(c) shows a plot showing simulated SWL and significant wave height at present day condition for point 3 (sheltered location). Again, the data shows a linear trend with little scatter with correlation coefficient above 0.9. This justifies extracting wave information from the representing trendline from a given SWL with or without using RSLC.

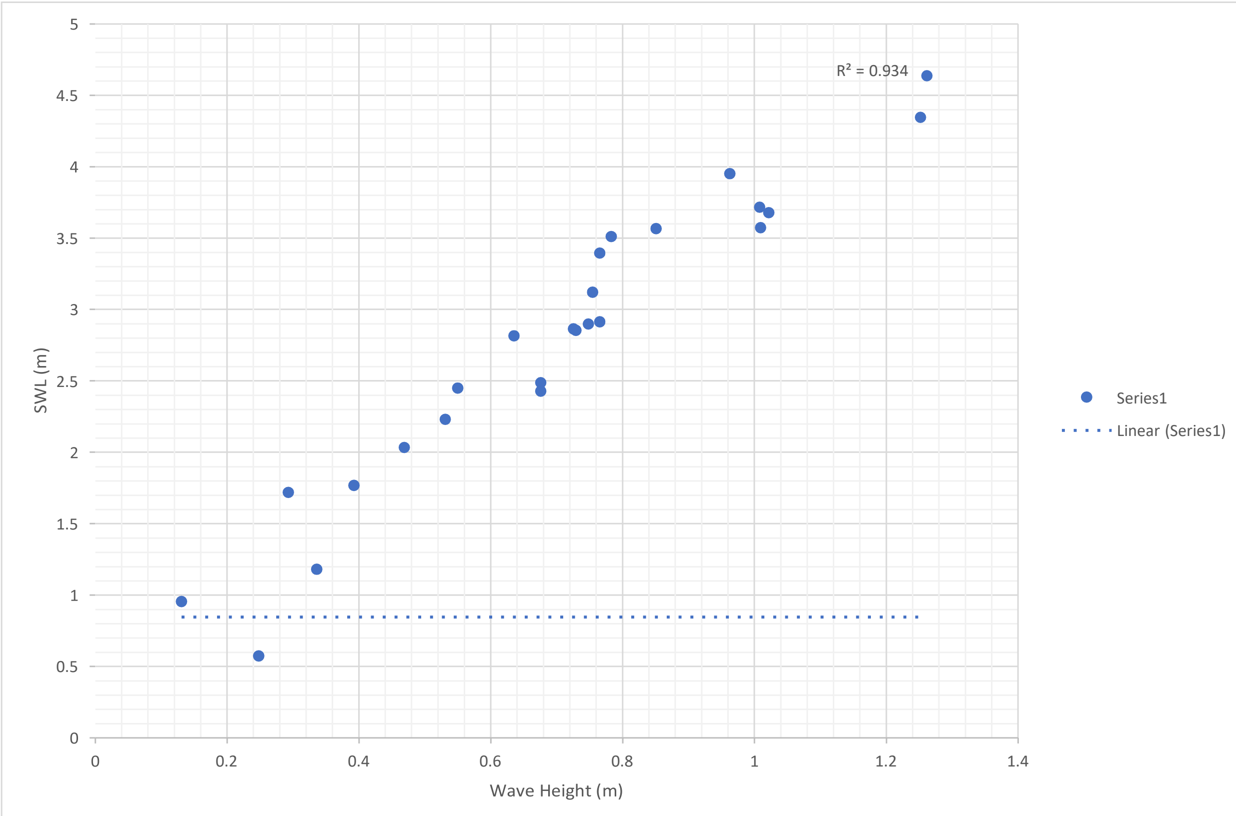


Figure 5.8(c) Simulated SWL and significant wave height point 3

(3) These linear trends are developed independently for all 9 extraction points which captures variation in site specific wave climates. Later these 9 trendlines are used to extract wave heights for SWL incorporating RSLC. As an example, Figure 5.8(d) below shows the extraction mechanism for point 3. Here blue points show the linear trend discussed above. Brown points are interpolated wave heights using RSLC condition. Note that these interpolations are done on separate extraction points and thus variations in the wave climate (sheltered vs. exposed) are captured in the trend lines eliminating (or reducing) uncertainty that you are referring. PDT already adopted a conservative approach (HIGH end) for estimating SWL incorporating RSLC using linear superposition and thus wave heights are also conservative.

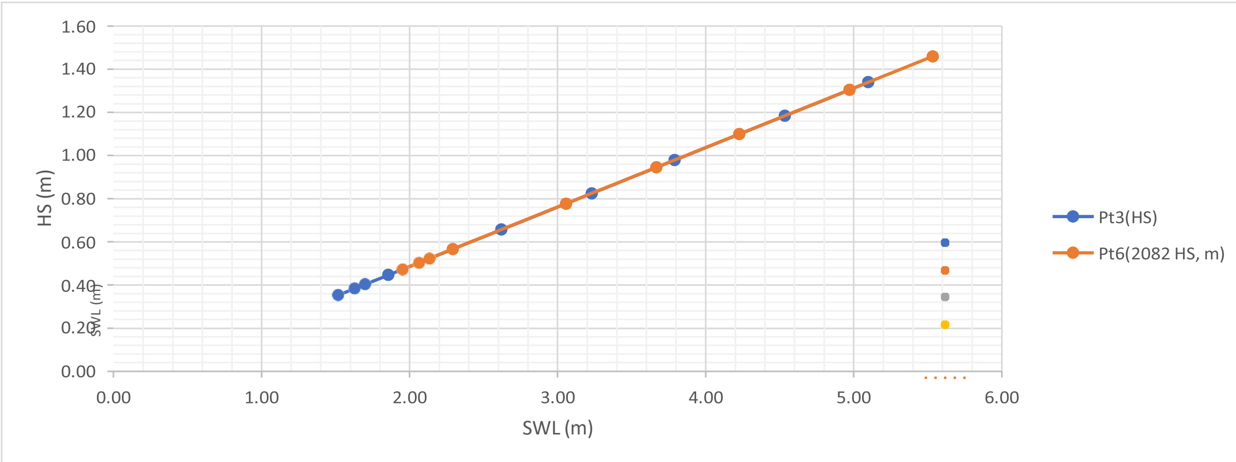


Figure 5.8 (d) Extraction Mechanism Point 3

Figure 5.9 is the correlation of SWL and Wave Height using 25 storms used in the analyses. Considering different locations, relation behaves in a linear fashion with R^2 values above 0.9. These linear relations are used to calculate representing wave heights associated with different SWL under RSLC condition.

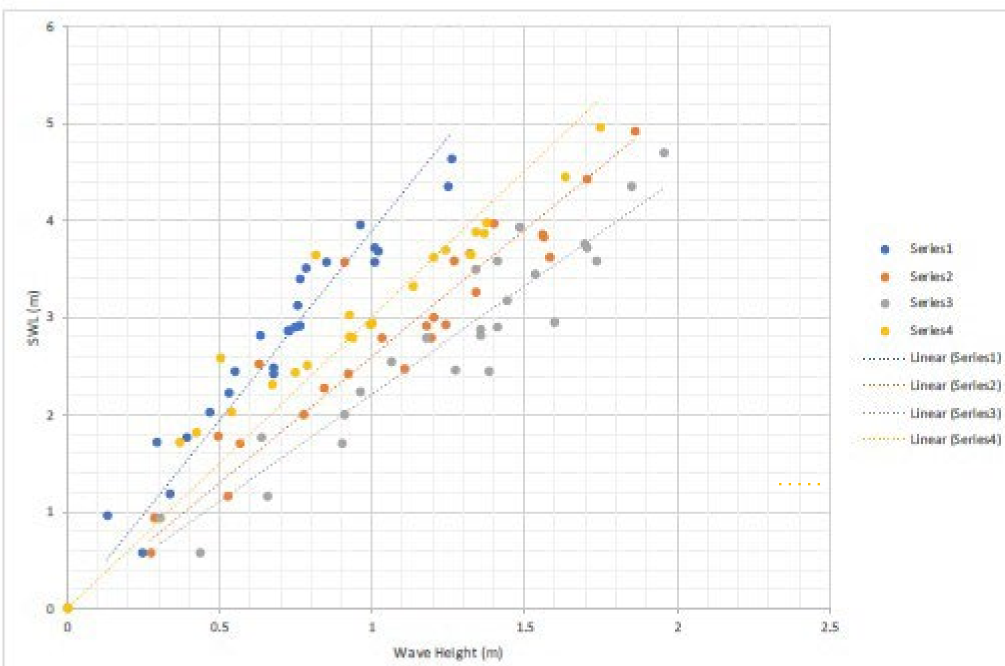


Figure 5.9: Correlation Between SWL (above MSL) and Wave Height

5.1.2.3 Overtopping Flow:

EUROTOP Methodology has been used to calculate overtopping flow (Figure 5.10). SWL has been adjusted for year 2082 with RSLC value = 1.65 ft and datum correction. Since floodwall elevation is set at +12 ft NAVD 88, when SWL is close to 12 ft, there will be free flow to be calculated as broad crested weir flow.

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.05\exp\left(-2.78\frac{R_c}{H_{m0}}\right) \text{ non-impulsive} \qquad 8.50$$

Impulsive conditions:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.011\left(\frac{H_{m0}}{hs_{m-1,0}}\right)^{0.5}\exp\left(-2.2\frac{R_c}{H_{m0}}\right) \qquad \text{valid for } 0 < R_c/H_{m0} < 1.35 \qquad 8.51$$

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.0014\left(\frac{H_{m0}}{hs_{m-1,0}}\right)^{0.5}\left(\frac{R_c}{H_{m0}}\right)^{-3} \qquad \text{valid for } R_c/H_{m0} \geq 1.35 \qquad 8.52$$

Figure 5.10: Key equations for overtopping flow calculation

Figure 11 shows wave overtopping flow calculated at Station 6. Here the red line shows AEP (2% in this case) at which point SWL considering RSLC plus one wave amplitude exceeds flood wall height of 12 ft NAVD88. According to HSDRRS Guideline, for the 1% AEP still water, wave height and wave period, the maximum allowable average wave overtopping values are 0.1 cfs/ft at 90% level of assurance and 0.03 cfs/ft at 50% level of assurance for floodwalls. For Station 6, we find this value to be 1.25 l/s/m or 0.013 cfs/ft. This is well below the HSDRRS limit state and hence considered tolerable.

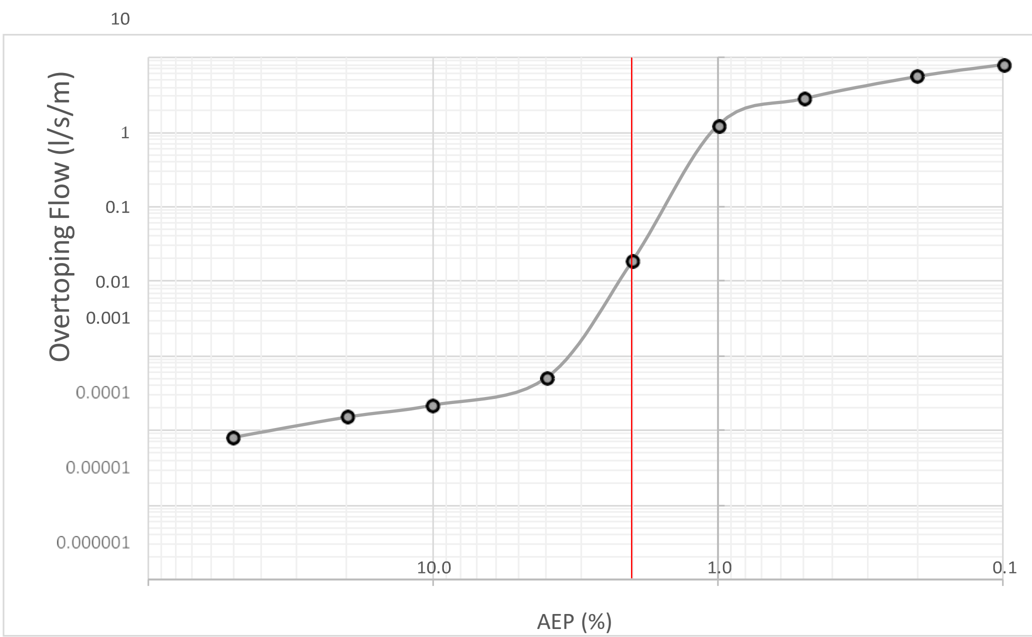


Figure 5.11: Overtopping Flow Calculated at Station 6.

Although overtopping flows are negligible and do not exceed limit state, figure 5.12 is presented to show estimated flow (1% AEP) that may be considered for drainage analyses. For simplicity, these flows are grouped into three regions – sheltered Western Region (stations 1, 2, 8, 9) where wave energy is low, Southern tip (Stations 4, 6, 7) where wave energy are relatively moderate and Eastern Section (3, 5) where wave energy are low to moderate. Accordingly, overtopping flows are shown in the following table (Table 5.1) . Representing flood wall lengths should be multiplied with these flows to calculate total flow volume.

Table 5.1 Overtopping Flows

Reaches & Stations	Overtopping Flow (CFS/FT)
Western Region (stations 1, 2, 8, 9)	0.006
Southern tip (Stations 4, 6, 7)	0.013
Eastern Section (3, 5)	0.009

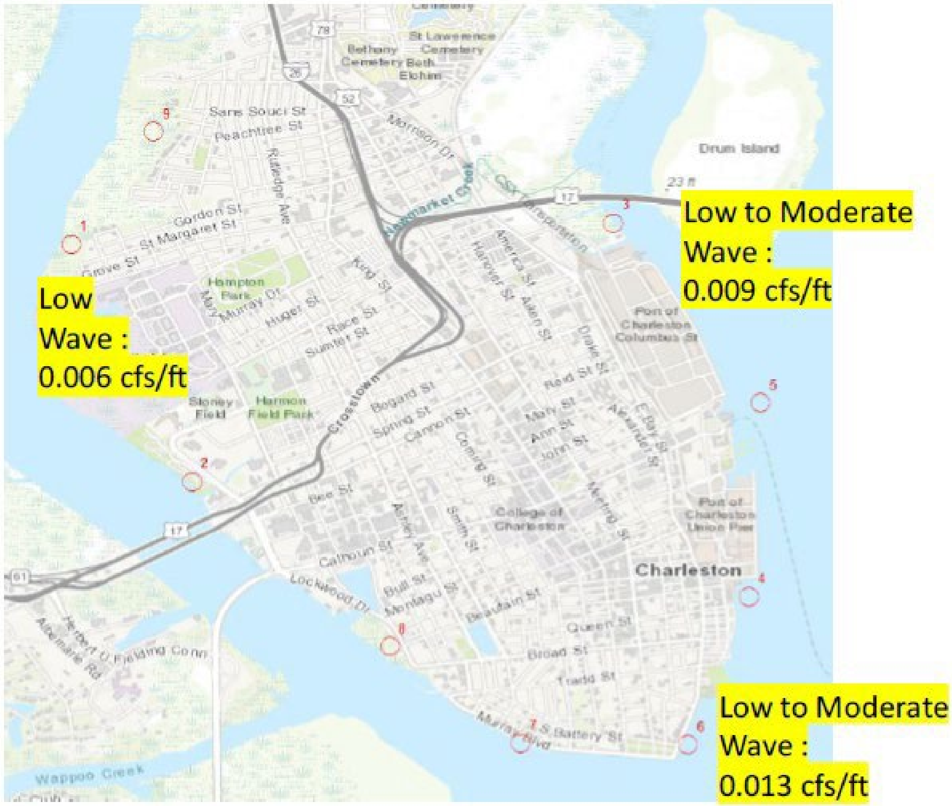


Figure 5.12: Overtopping Flow along Different Reaches

5.2 PROJECTED STILL WATER SURFACE ELEVATION WITH ANNUAL EXCEEDANCE PROBABILITY

Overtopping is primary concern for structures constructed to defend against flooding. Storm surge is driven by storm winds and waves as documented by Still Water Level (SWL). Peak surge elevations will be greater if the storm surge coincides with the tide. Local waves developing over inland water bodies such as the harbor can also develop. Waves running up the face of the wall can be high enough to pass over the crest of the wall and waves breaking on the structure can result in significant volume of splash. Overtopping of the floodwall by the free flowing still water elevation is an indication of failure defense but not failure of the structure so long as the structure is designed for overtopping without structural failure. The structure has been designed to withstand still water overtopping.

Wind generated wave overtopping was already presented in 5.1 and the non-linearity assessment provided the justification for the method to determine probability of overtopping by still water elevation. Based on analysis discussed in Section 5.1, the maximum estimate for NLR was -0.15 m, which is a negative bias. The negative bias means that simple superposition of RSLC with storm surge model output will produce a higher water level estimate than compared to directly including RSLC within the storm surge model. Thus, the linear superposition of RSLC with storm surge model output can be used to estimate water levels for various probability storms under the effect of RSLC, which is a conservative approach.

Using FEMA still water elevation levels from the most recent Flood Insurance Study, ERDC generated an Annual Exceedance Probability (AEP) for each of the five save points requested (refer to Figure 5.1). Still water level values in MSL were converted to NAVD88 and sea level rates were applied. The still water surge elevation is the water elevation due solely to the effects of the astronomical tides, storm surge, and wave setup on the water surface, but which does not include wave heights. It is important to note, however, this differs from the base flood elevation because the still water level does not include wave regeneration that occurs over a large body of water before it reaches the shoreline.

Wave heights vary depending on direction and speed of the storm and the same storm will generate different wave heights on opposite sides of the peninsula, thus the probability of wave height is not directly associated with the probability of the storm.

ER 1105-2-101 states that the mean AEP values be used for economic analyses, but that when communicating project performance, the AEP values at the 90% confidence level should be used. AECOM, contractor for FEMA, provided confidence limit formulas to apply. Tables 5.2-5.4 list the AEP with the upper 90% confidence limit (UCL) at the 5 locations selected for model areas for the year 2032. Table 5.2 uses the USACE low sea level curve value, Table 5.3 uses the intermediate curve value, and Table 5.4 uses the high curve value. Figure 5.13 is the same information plotted. Probabilities are also tabulated in Table 5.5 based on Long-Term Exceedance Probabilities (LTEP), or probability of exceedance over each indicated time interval for the 50% assurance values. The table displays the mean AEP for exceedance of 12 ft NAVD88 at the five locations selected for model areas, using each USACE sea level change scenario at year 2032 and corresponding LTEP.

Table 5.2. Year 2032 annual exceedance with 90% confidence for USACE Low SLC curve

	SWL (ft NAVD88)	**2032**	**SLR=**	**0.41**	**ft**				
Location	AEP+SLR +90%UCL (%) 50	AEP+SLR +90%UCL (%) 20	AEP+SLR +90%UCL (%)10	AEP+SLR +90%UCL (%) 4	AEP+SLR +90%UCL (%) 2	AEP+SLR +90%UCL (%) 1	AEP+SLR +90%UCL (%) 0.5	AEP+SLR +90%UCL (%) 0.2	AEP+SLR +90%UCL (%) 0.1
Wagener Terrace	8.47	8.94	9.23	9.90	13.03	15.62	17.89	20.89	23.16
Marina	8.43	8.90	9.19	9.86	13.14	15.75	18.08	21.16	23.49
Newmarket	8.43	8.89	9.18	9.85	13.07	15.63	18.00	21.12	23.49
Port	8.40	8.86	9.15	9.81	13.02	15.59	17.99	21.18	23.59
Battery	8.39	8.85	9.14	9.80	13.02	15.69	18.10	21.29	23.71

Table 5.3. Year 2032 annual exceedance with 90% confidence for USACE Intermediate SLC curve

	SWL (ft NAVD88)	**2032**	**SLR =**	**0.56**	**ft**				
Location	AEP+SLR +90%UCL (%) 50	AEP+SLR +90%UCL (%) 20	AEP+SLR +90%UCL (%)10	AEP+SLR +90%UCL (%) 4	AEP+SLR +90%UCL (%) 2	AEP+SLR +90%UCL (%) 1	AEP+SLR +90%UCL (%) 0.5	AEP+SLR +90%UCL (%) 0.2	AEP+SLR +90%UCL (%) 0.1

Wagener Terrace	8.66	9.13	9.42	10.09	13.23	15.82	18.08	21.08	23.35
Marina	8.63	9.09	9.38	10.05	13.33	15.94	18.27	21.35	23.68
Newmarket	8.62	9.09	9.38	10.05	13.27	15.83	18.19	21.32	23.68
Port	8.59	9.05	9.34	10.00	13.21	15.78	18.19	21.37	23.79
Battery	8.58	9.04	9.33	10.00	13.21	15.88	18.29	21.48	23.90

Table 5.4. Year 2032 annual exceedance with 90% confidence for USACE High SLC curve

	SWL (ft NAVD88)	**2032**	**SLR=**	**1.01**	**ft**				
Location	AEP+SLR +90%UCL (%) 50	AEP+SLR +90%UCL (%) 20	AEP+SLR +90%UCL (%)10	AEP+SLR +90%UCL (%) 4	AEP+SLR +90%UCL (%) 2	AEP+SLR +90%UCL (%) 1	AEP+SLR +90%UCL (%) 0.5	AEP+SLR +90%UCL (%) 0.2	AEP+SLR +90%UCL (%) 0.1
Wagener Terrace	9.24	9.71	10.00	10.67	13.80	16.39	18.66	21.66	23.93
Marina	9.20	9.67	9.96	10.63	13.91	16.52	18.84	21.93	24.26
Newmarket	9.20	9.66	9.95	10.62	13.84	16.40	18.77	21.89	24.26
Port	9.16	9.63	9.92	10.58	13.79	16.36	18.76	21.95	24.36
Battery	9.16	9.62	9.91	10.57	13.79	16.46	18.87	22.06	24.48

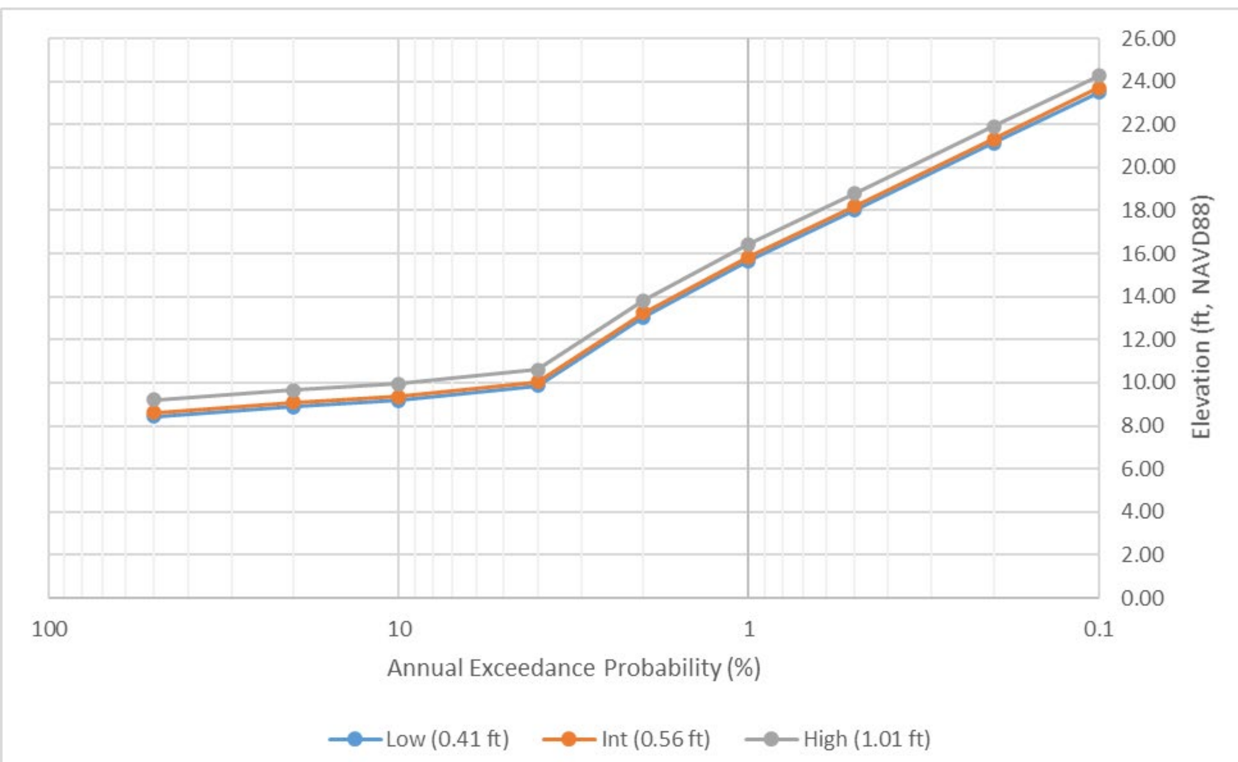


Figure 5.13 Year 2032 Annual Exceedance with 90% upper confidence

Table 5.5. Performance in 2032 described by mean AEP and LTEP

USACE SLC Scenario	Mean AEP at 12 ft NAVD88	LTEP (Probability of Exceedance Over Indicated Time)		
		10 Years	30 Years	50 Years
Low (0.41 ft)	0.007	0.07	0.18	0.29
Int (0.56 ft)	0.007	0.07	0.19	0.30
High (1.01 ft)	0.008	0.08	0.22	0.34

Tables 5.6-5.8 list the AEP with the upper 90% confidence limit (UCL) at the 5 locations selected for model areas for the year 2082. Table 5.6 uses the USACE low sea level curve value, Table 5.7 uses the intermediate curve value, and Table 5.8 uses the high curve value. Figure 5.14 is the same information plotted. Probabilities are also tabulated in Table 5.9 based on Long-Term Exceedance Probabilities (LTEP), or probability of exceedance over each indicated time interval for the 50% assurance values. The table displays the mean AEP for exceedance of 12 ft NAVD88 at the five locations selected for model areas, using each USACE sea level change scenario at year 2082 and corresponding LTEP.

Table 5.6. Year 2082 annual exceedance with 90% confidence for USACE Low SLC curve

	SWL (ft NAVD88)	**2082**	**SLR =**	**0.93**	**ft**				
Location	AEP+SLR +90%UCL (%) 50	AEP+SLR +90%UCL (%) 20	AEP+SLR +90%UCL (%)10	AEP+SLR +90%UCL (%) 4	AEP+SLR +90%UCL (%) 2	AEP+SLR +90%UCL (%) 1	AEP+SLR +90%UCL (%) 0.5	AEP+SLR +90%UCL (%) 0.2	AEP+SLR +90%UCL (%) 0.1
Wagener Terrace	9.14	9.60	9.90	10.57	13.70	16.29	18.55	21.55	23.83
Marina	9.10	9.57	9.86	10.53	13.80	16.41	18.74	21.82	24.16
Newmarket	9.09	9.56	9.85	10.52	13.74	16.30	18.66	21.79	24.16
Port	9.06	9.52	9.81	10.48	13.69	16.25	18.66	21.85	24.26
Battery	9.06	9.52	9.81	10.47	13.69	16.35	18.76	21.96	24.38

Table 5.7. Year 2082 annual exceedance with 90% confidence for USACE Intermediate SLC curve

	SWL (ft NAVD88)	**2082**	**SLR =**	**1.65**	**ft**				
Location	AEP+SLR +90%UCL (%) 50	AEP+SLR +90%UCL (%) 20	AEP+SLR +90%UCL (%)10	AEP+SLR +90%UCL (%) 4	AEP+SLR +90%UCL (%) 2	AEP+SLR +90%UCL (%) 1	AEP+SLR +90%UCL (%) 0.5	AEP+SLR +90%UCL (%) 0.2	AEP+SLR +90%UCL (%) 0.1
Wagener Terrace	10.06	10.53	10.82	11.49	14.62	17.21	19.48	22.48	24.75
Marina	10.02	10.49	10.78	11.45	14.73	17.34	19.66	22.75	25.08
Newmarket	10.02	10.48	10.77	11.44	14.66	17.22	19.59	22.71	25.08
Port	9.98	10.45	10.74	11.40	14.61	17.18	19.58	22.77	25.18
Battery	9.98	10.44	10.73	11.39	14.61	17.28	19.69	22.88	25.30

Table 5.8. Year 2082 annual exceedance with 90% confidence for USACE High SLC curve

	SWL (ft NAVD88)	**2082**	**SLR =**	**3.93**	**ft**				
Location	AEP+SLR +90%UCL (%) 50	AEP+SLR +90%UCL (%) 20	AEP+SLR +90%UCL (%)10	AEP+SLR +90%UCL (%) 4	AEP+SLR +90%UCL (%) 2	AEP+SLR +90%UCL (%) 1	AEP+SLR +90%UCL (%) 0.5	AEP+SLR +90%UCL (%) 0.2	AEP+SLR +90%UCL (%) 0.1
Wagener Terrace	12.98	13.45	13.74	14.41	17.54	20.13	22.40	25.40	27.67
Marina	12.95	13.41	13.70	14.37	17.65	20.26	22.59	25.67	28.00
Newmarket	12.94	13.41	13.69	14.36	17.59	20.15	22.51	25.64	28.00
Port	12.91	13.37	13.66	14.32	17.53	20.10	22.50	25.69	28.10
Battery	12.90	13.36	13.65	14.32	17.53	20.20	22.61	25.80	28.22

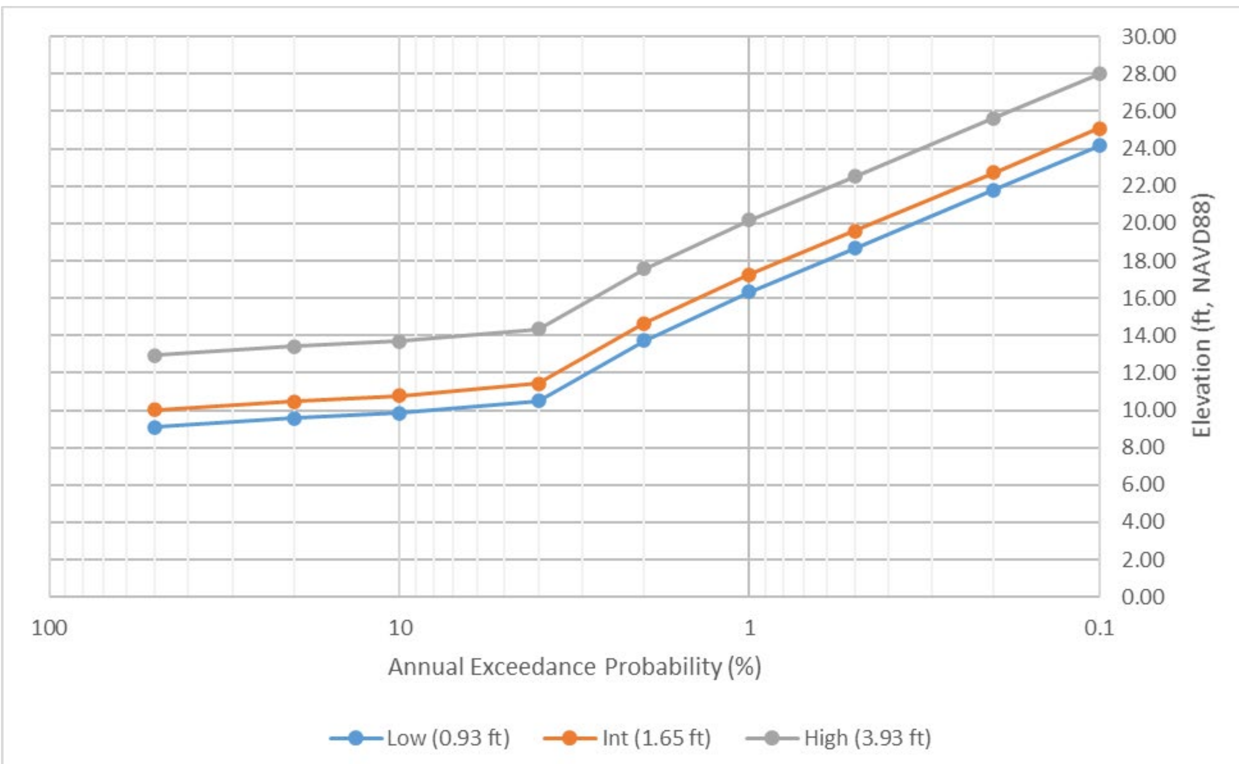


Figure 5.14 Year 2082 Annual Exceedance with 90% upper confidence

Table 5.9. Performance in 2082 described by mean AEP and LTEP

USACE SLC Scenario	Mean AEP at 12 ft NAVD88	LTEP (Probability of Exceedance Over Indicated Time)		
		10 Years	30 Years	50 Years
Low (0.93 ft)	0.008	0.08	0.22	0.34
Int (1.65 ft)	0.010	0.02	0.03	0.04

High (3.93 ft)	0.022	0.04	0.07	0.07

One other form of water level statistics required by ER 1105-2-101 is assurance. Assurance, also referred to as the conditional non-exceedance probability (CNP) is the likelihood that water levels will not reach a certain elevation within a given year. In this case, assurance is measured as the likelihood that water levels will not exceed 12 ft NAVD88 at various points along the wall for water levels with AEPs of 0.2, 0.1, 0.02, 0.01, 0.004, and 0.002. These values are displayed in tables 5.10-5.15 for each of the three USACE sea level rise curves during 2032 and 2082. For example, according to Table 5.11, which represents the USACE intermediate sea level curve for year 2032, water levels with an AEP of 0.01 (1%) would have an average of a 0.632 (63.2%) chance of overtopping.

Table 5.10. Assurance of still water level overtopping at 12 ft NAVD88 for the USACE low sea level curve, year 2032.

Save Point	0.2 Assurance	0.1 Assurance	0.02 Assurance	0.01 Assurance	0.004 Assurance	0.002 Assurance
Wagener Terrace	0.999	0.998	0.891	0.656	0.356	0.211
Marina	0.999	0.999	0.884	0.643	0.340	0.197
Newmarket	0.999	0.999	0.888	0.655	0.345	0.199
Port	0.999	0.999	0.892	0.660	0.345	0.196
Battery	0.999	0.999	0.892	0.649	0.336	0.190
AVG	0.999	0.999	0.890	0.653	0.344	0.199

Table 5.11. Assurance of still water level overtopping at 12 ft NAVD88 for the USACE intermediate sea level curve, year 2032.

Save Point	0.2 Assurance	0.1 Assurance	0.02 Assurance	0.01 Assurance	0.004 Assurance	0.002 Assurance
Wagener Terrace	0.999	0.998	0.878	0.636	0.341	0.201
Marina	0.999	0.998	0.870	0.623	0.326	0.188
Newmarket	0.999	0.998	0.875	0.635	0.331	0.189
Port	0.999	0.998	0.879	0.640	0.330	0.187
Battery	0.999	0.998	0.879	0.629	0.322	0.181
AVG	0.999	0.998	0.876	0.632	0.330	0.189

Table 5.12. Assurance of still water level overtopping at 12 ft NAVD88 for the USACE high sea level curve, year 2032.

Save Point	0.2 Assurance	0.1 Assurance	0.02 Assurance	0.01 Assurance	0.004 Assurance	0.002 Assurance
Wagener Terrace	0.996	0.994	0.833	0.576	0.299	0.173
Marina	0.997	0.995	0.823	0.563	0.285	0.162

Newmarket	0.997	0.995	0.829	0.575	0.290	0.163
Port	0.997	0.995	0.834	0.579	0.289	0.161
Battery	0.997	0.995	0.834	0.569	0.282	0.156
AVG	0.997	0.995	0.830	0.572	0.289	0.163

Table 5.13. Assurance of still water level overtopping at 12 ft NAVD88 for the USACE low sea level curve, year 2082.

Save Point	0.2 Assurance	0.1 Assurance	0.02 Assurance	0.01 Assurance	0.004 Assurance	0.002 Assurance
Wagener Terrace	0.997	0.995	0.841	0.586	0.306	0.178
Marina	0.997	0.996	0.832	0.573	0.292	0.166
Newmarket	0.997	0.996	0.838	0.585	0.297	0.168
Port	0.997	0.996	0.842	0.590	0.296	0.165
Battery	0.997	0.996	0.842	0.580	0.289	0.160
AVG	0.997	0.996	0.839	0.583	0.296	0.167

Table 5.14. Assurance of still water level overtopping at 12 ft NAVD88 for USACE intermediate sea level curve, year 2082.

Save Point	0.2 Assurance	0.1 Assurance	0.02 Assurance	0.01 Assurance	0.004 Assurance	0.002 Assurance
Wagener Terrace	0.989	0.984	0.758	0.493	0.246	0.140
Marina	0.989	0.985	0.747	0.480	0.234	0.131
Newmarket	0.989	0.985	0.754	0.492	0.238	0.132
Port	0.990	0.985	0.759	0.496	0.237	0.130
Battery	0.990	0.986	0.759	0.486	0.231	0.126
AVG	0.989	0.985	0.755	0.489	0.237	0.132

Table 5.15. Assurance of still water level overtopping at 12 ft NAVD88 for USACE high sea level curve, year 2082.

Save Point	0.2 Assurance	0.1 Assurance	0.02 Assurance	0.01 Assurance	0.004 Assurance	0.002 Assurance
Wagener Terrace	0.861	0.838	0.461	0.255	0.117	0.065
Marina	0.864	0.841	0.451	0.247	0.111	0.060
Newmarket	0.864	0.841	0.457	0.254	0.113	0.061
Port	0.867	0.844	0.462	0.257	0.113	0.060

Battery	0.868	0.845	0.462	0.251	0.110	0.058
AVG	0.865	0.842	0.458	0.253	0.113	0.061

The tide range in Charleston is up to 6 feet, suggesting that the tide phase at the time of landfall may significantly influence surge levels produced by a given storm. Still water elevations were computed at MSL, therefore the risk of flooding at high tide must be considered when assessing risk and potential damages. This was considered in the G2CRM analysis ofdamages.

The existing still water elevation is documented in the FIS but it is not the Base Flood Elevation that is considered a better estimate of the flood hazard. To obtain the final Base Flood Elevations (BFEs), FEMA then uses WHAFIS, for the overland wave height analysis. The WHAFIS model can also cause wave regeneration if it goes over a sizable body of water. It can then dissipate as it passes over land as shown in Figure 5.15, obtained from FEMA contractor.

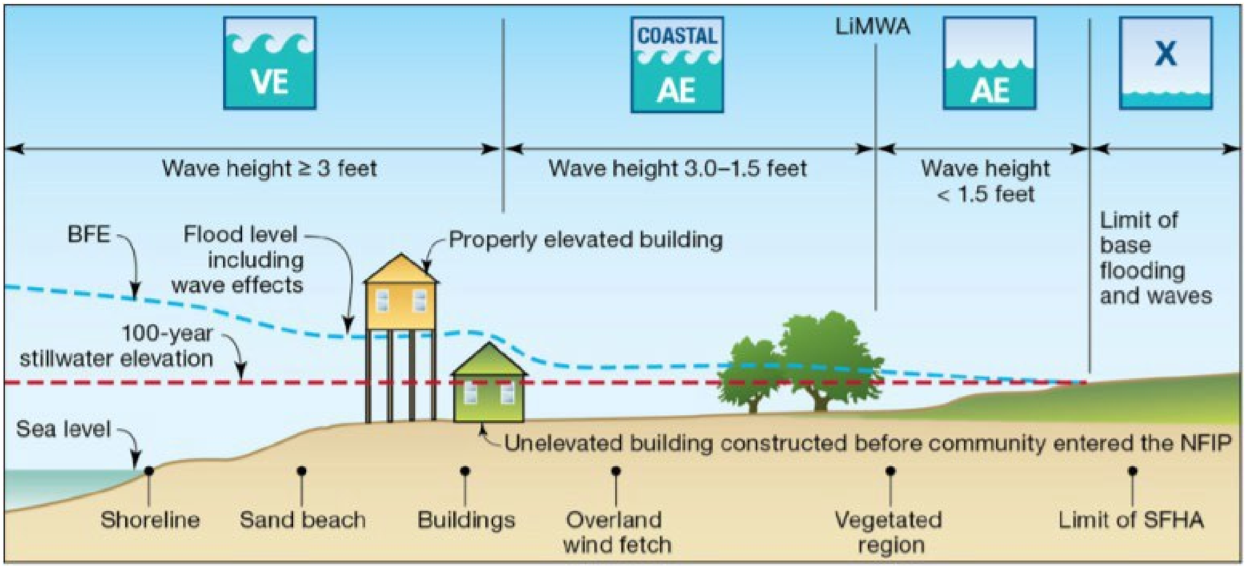


Figure 5.15 Demonstration of Stillwater elevation, BFE and various Special Flood Hazard Areas. (Source FEMA)

CHAPTER 6 – WAVE REFRACTION ON SURROUNDING AREAS

After optimization of the footprint to reduce environmental impacts, minimize impacts to personal property while reducing costs by relocating the wall on high ground to utilize a T-wall rather than the combo wall, the wall at elevation 12 ft NAVD88 was added to the ADCIRC/STWAVE mesh for evaluation of impacts to surrounding areas.

The final recommended structures were incorporated into the ADCIRC and STWAVE models and evaluated by the PDT for impacts outside the project area for the intermediate rate of sea level rise for the year 2032 (0.56 ft), after initial construction and for 2082 (1.65 ft), the end of its economic life. This methodology corresponds to the methodology used for the interior hydrology assessment detailed insub appendix Interior Hydrology. Because nonlinear residual (NLR) was proven to be very weak in Section 5.1.2.1, effects shown by changes in sea level for the intermediate rate at years 2032 and 2082 can be applied to other sea level rise scenarios.

ADCIRC was coupled with STWAVE to model 11 synthetic storms for each sea level rise scenario and each project condition, where the future without project (FWO) condition was modeled using the ADCIRC and STWAVE meshes described in Section 4-2 and shown in Figures 4.2 and 4.3. The future with project (FWP) condition was modeled using the same ADCIRC and STWAVE meshes, manipulated to include a 12 ft NAVD88 wall surrounding the peninsula (Figure 6.1). The 11 storms were chosen from the storm suite to represent a full distribution of storm sizes and patterns. Storm frequencies ranged from greater to 10% AEP to less than 0.02% AEP. This reduction in storm suite saved computational time and cost by reducing the required number of simulations to 44, while providing sufficient data to compare sea level rise scenarios and project conditions.

Figure 6.1. ADCIRC mesh used for FWP simulations with proposed 12 ft NAVD88 wall shown in light green.

Based on simulations completed using the FWO and FWP conditions, presence of the wall caused minimal effect on water levels due to storm surge in surrounding areas. Some simulations showed up to a 1 to 2 inch increase in water levels for the FWP condition in some surrounding areas. However, this change in water levels is within the accuracy of the model itself and can be considered minimal. These increases were only seen in small areas during simulations for larger storms that overtopped the wall (12+ ft of storm surge), so areas with an increase of 1 to 2 inches would already be experiencing several feet of inundation.

Other than these sparse cases of 1 to 2 inch increases, the increase in water levels to surrounding areas is typically less than 1 inch, while the reduction in water levels within the wall in the FWP condition is typically on the order of several feet.

Local wind waves within the Charleston riverine and estuary nearshore area will be limited in wave height and period by the limited fetches. These waves will be dissipated by marshes and shallow foreshore areas before encountering the wall which will scatter the remaining waves, causing them to dissipate within a few wavelengths. Scattering is due to directional/frequency spread of the short-period waves, irregularities in the wall, near-wall bathymetry, adverse wind (wind blowing against the reflected waves), and complex bathymetry of the far-field (river channels/nearshore). As supported by results in the STWAVE simulations, reflection and refraction of waves encountering the wall will have no effect on surrounding areas.

CHAPTER 7 – REFERENCES

o **ER 1105-2-100** (Appendix K): *Planning Guidance Notebook* (April 2000).
o **ER 1100-2-8162**: *Incorporating Sea Level Changes in Civil Works Programs* (December 2013).
o **ECB 2016-5**: *Using Non-NOAA Tide Gauge Records for Computing Relative Sea Level Change* (Jan 2016).
o **EP 1100-2-1**: *Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation* (June 2014).
o **ECB 2018-3**: *Using Non-NOAA Tide Gauge Records for Computing Relative Sea Level Change* (Feb 2018).

Additional important guidance is provided within the following documents:
o **ER 1110-2-8160**: *Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums* (March 2009).
o **EM 1110-2-6056**: S*tandards and Procedures for Referencing Project Elevation Grades to Nationwide Vertical Datums* (December 2010).
o **ER 1110-2-8159**: *Life Cycle Design and Performance* (October 1997).
o **ER 1110-2-1150**: *Engineering and Design for Civil Works Projects* ((August 1999).
o **ECB 2018-2:** *Implementation of Resilience Principles in the Engineering & Construction Community of Practice* (Jan 2018).
o **EP 1100-1-3:** *USACE Sustainability: Definition and Concepts Guide* (July 2018).
o EM 1110-2-1413 Hydrologic Analysis of Interior Areas
o Technical Report NOS CO-OPS 065, Estimating Vertical Land Motion from Long-Term Tide Gauge Records, 2013

Webpage

City of Charleston. 2019. Flooding and Sea Level Rise Strategy, Charleston, South Carolina. February 2019. https://www.charleston-sc.gov/DocumentCenter/View/20299/Flooding-and-Sea-Level-Rise-Strategy-2019-web-viewing?bidId=

NOAA Tides and Currents. https://tidesandcurrents.noaa.gov/stationhome.html?id=8665530.

NASA/JPL https://sideshow.jpl.nasa.gov/post/series.html

U.S. Army Corps of Engineers (2021). USACE Sea Level Change Curve Calculator (2021.12). https://cwbi-app.sec.usace.army.mil/rccslc/slcc_calc.html

US Army Corps of Engineers®

Charleston District

CHARLESTON PENINSULA, SOUTH CAROLINA, A COASTAL STORM RISK MANAGEMENT STUDY

Charleston, South Carolina

ENGINEERING APPENDIX – B

ERDC COASTAL MODELING SUB-APPENDIX 4A

February 2022

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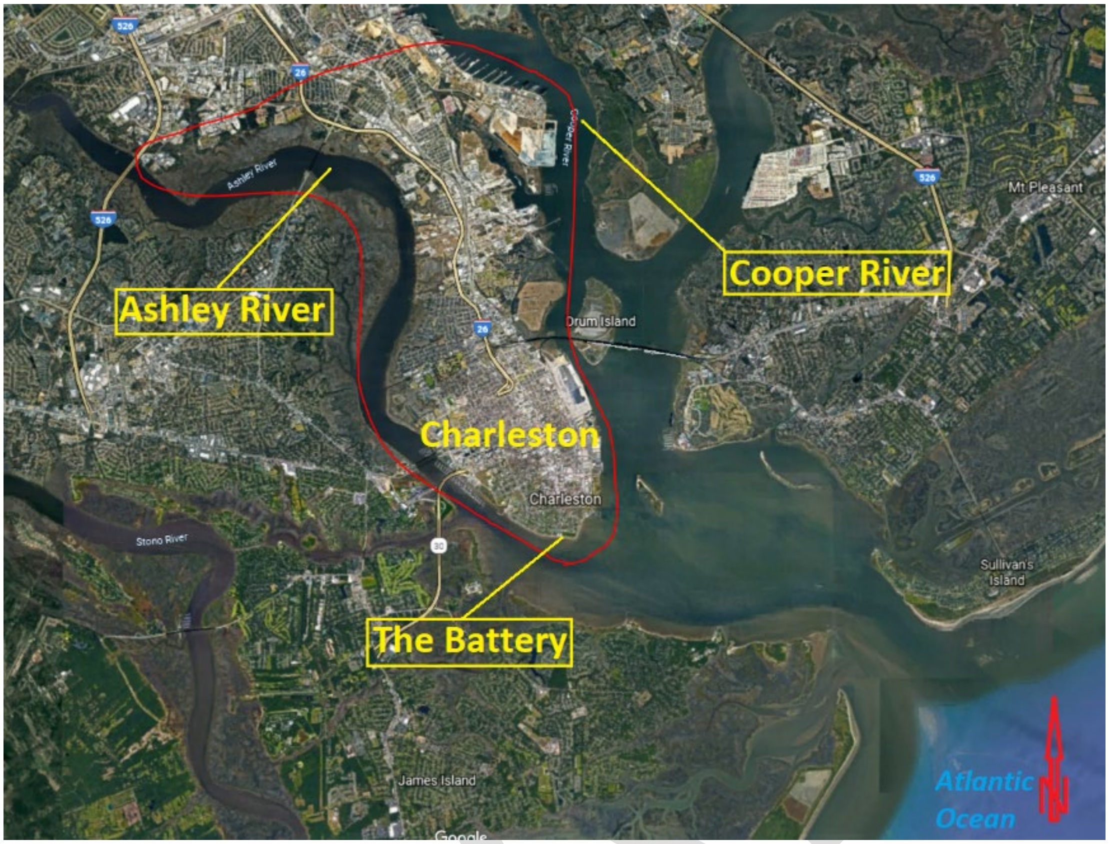
Charleston Peninsula Coastal Storm Risk Management Feasibility Study – Summary Report

by Gregory Slusarczyk, S.C. Dillon, Mary Anderson Bryant, Norberto Nadal-Caraballo, Rusty Permenter, Bradley Johnson

ABSTRACT: The U.S. Army Corps of Engineers (USACE), Wilmington (SAW) and Charleston (SAC) Districts are currently engaged in the Charleston Peninsula Coastal Storm Risk Management (CSRM) Feasibility Study. The U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Lab (CHL) conducted a numerical modeling study to evaluate the effectiveness of structural solutions to increase resilience and reduce risk from future storms and impacts from sea level change as a part of coastal flood risk management. The numerical modeling study includes the computation of water levels and wave heights for Existing Conditions (EC), Future Without Project (FWOP) and With Project (WP – breakwater) scenarios. Results from that numerical study are presented herein and provide the engineering inputs for the economics model, G2CRM.

INTRODUCTION: The Charleston Peninsula (**Figure 1**) is approximately 8 square miles, located between the Ashley and Cooper Rivers. The two rivers join at the Battery in Charleston to form Charleston Harbor before discharging into the Atlantic Ocean. Charleston Harbor is a natural tidal estuary sheltered by barrier islands. The first European settlers arrived in Charleston around 1670. Since that time, the peninsula city has undergone dramatic shoreline changes, predominantly by landfilling of the intertidal zone. Early maps show that over one-third of the peninsula has been "reclaimed." Much of the landfilling occurred on the southern tip and the western side of Charleston (predominant flooding is on the western side due to lower elevations), behind a seawall and promenade, known as the Battery. The Charleston Peninsula is the historic core and urban center of the City of Charleston and is home to 38,000 people.



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PROJECT OBJECTIVES: The Charleston Peninsula CSRM Study is a feasibility level study being conducted by SAC, with technical support by SAW, with the objective of reducing damages from coastal flooding that affects population, critical infrastructure, property, and ecosystems in the Charleston Peninsula area. Therefore the numerical modeling aspect of the Charleston Peninsula Study (CPS) is to provide estimates of waves and water levels for Existing Conditions (EC), Future Without Project (FWOP), and With-Project (WP - breakwater) scenarios to be evaluated by SAC. EC refers to the existing topography, FWOP is a condition in which the low battery wall is raised to a similar elevation as the high battery, and WP – breakwater is a condition in which the wall is changed to match the FWOP condition and an additional breakwater is included at the battery. These project scenarios reflect only physical changes in the study and not changes to sea level. In order to meet these objectives the following steps were taken:

- Selection of 25 tropical synthetic storms from the set of 122 synthetic and 3 historical tropical cyclones that were designed and simulated in a previous South Carolina Storm Surge Study conducted by FEMA (Federal Emergency Management Agency), 2013
- Modification to the FEMA ADCIRC grid to reflect without/with project conditions and development of corresponding STWAVE grids
- Simulation of waves and water levels. The simulations are in support of G2CRM and do not include tides or sea level changes since these are already included in G2CRM model

- Production of maximum water surface elevations, time series of water surface elevations at specified save point locations, maximum wave heights and time series, and data files (.h5) as part of post-processing of simulation results for the economics model, G2CRM.

STORM SELECTION METHODOLOGY: As part of the 2012 South Carolina Storm Surge Project (SCSSP) a joint probability method (JPM) storm suite of 122 synthetic tropical cyclones was developed by URS for SCDNR and FEMA (FEMA 2012, 2013). SAC obtained the SCSSP storm suite from AECOM, a FEMA contractor. The intent of this 122-storm suite was the generation of water levels corresponding to 2%, 1%, and 0.2% annual exceedance probability (AEP) for FEMA's flood hazard mapping program.

For the Charleston Peninsula Study (CPS), an initial reduced storm set of 20 synthetic tropical cyclones (TCs) was selected from the original SCSSP 122-storm suite (i.e., full storm set (FSS)). The number of storms to be selected was driven by schedule and budget constraints, and by knowledge gathered from other previous and ongoing USACE feasibility studies about the minimum number of storms required to adequately capture the storm surge hazard. The goal of storm selection was to find the optimal combination of storms given a predetermined number of storms to be sampled (e.g., 20 TCs), referred to as reduced storm set (RSS). In the process of selecting 20 TCs, it was determined that a RSS of this size adequately captured the storm surge hazard for the range of probabilities covered by the FSS (122 TCs).

The storm selection process was performed using the design of experiments (DoE) approach described in detail in Jia et al. (2015) and, more recently, Taflanidis et al. (2017) and Zhang et al. (2018). The DoE compares still water level (SWL), further in the text referenced as water elevation, hazard curves derived from the RSS to "benchmark" hazard curves corresponding to the FSS at a given number of save points within the study area. The difference between the RSS hazard curves and FSS benchmark curves is minimized in an iterative process considering multiple subsets of 20 TCs.

In summary, the general steps in this DoE approach for selecting a subset of storms are:

1. Identify a set of save points critical to a project or study area, where optimization will be performed.
2. Develop hazard curves for the FSS.
3. Select number of storms to be sampled.
4. Develop hazard curves for the RSS.
5. Choose the range of probabilities for which hazard curves will be compared. RSS versus FSS differences can be computed along the entire hazard curve, or by prioritizing a specific segment of the curves, e.g., 50 to 500 years.
6. Compute differences between RSS and FSS hazard curves.
7. An iterative sensitivity analysis is performed to determine the optimal combination of storms constituting the RSS.
8. Once the optimal combination of storms is determined, an optional analysis can be performed to evaluate the benefits of increasing storm subset size; finalize storm selection.

REDUCED STORM SET FOR CPS: For the CPS, a metamodel with recursive iterative implementation was used to select an optimal subsample of the 122 SCSSP storms. The method is based on the Gaussian process metamodeling described by Taflanidis et al. (2017) and Zhang et al. (2018). In this approach, an initial sample of 20 storms (i.e., RSS) is recursively obtained in the 122 storm FSS. A metamodel is produced for each of the 89 save points within the CPS area (**Figure 2**) based on these 20 events with hurricane JPM parameters as inputs and ADCIRC storm surge as output. Each metamodel is then used to predict the SWL hazard curves for each of the 89 save point locations. The metamodels of each 20-sample surrogate are trained, and hazard curves are produced at the 89 save point locations. The best 20-storm sample is determined by minimizing the error across the parameter space using a genetic algorithm where the error is between the reduced sample and the full 122 storm set. Many permutations of 20 events are sampled using a Monte Carlo sampling of the entire parameter space. This process is repeated until an optimal 20-event sample is defined that minimizes the error between the target (FSS) hazard curve and the sample (RSS).

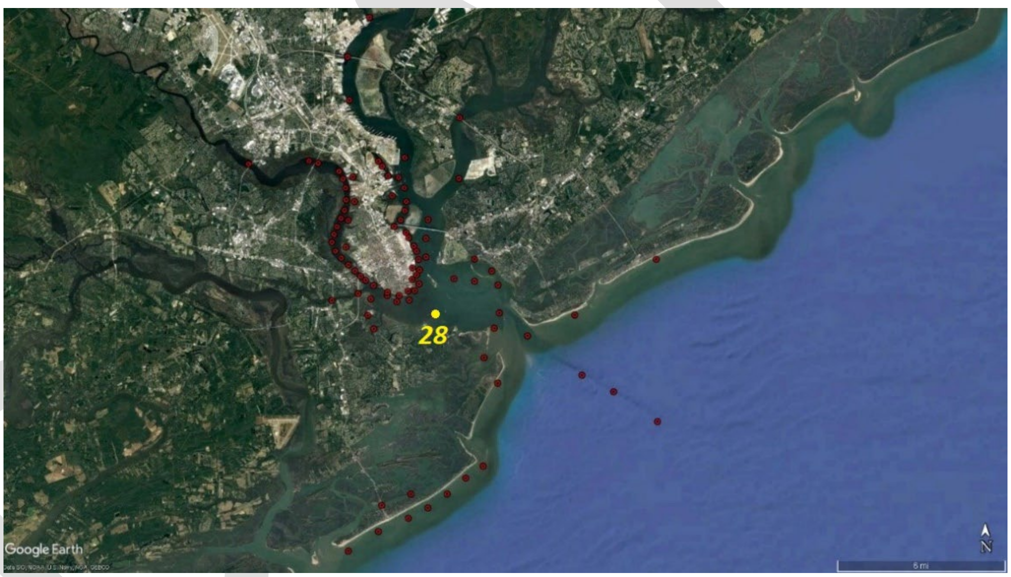
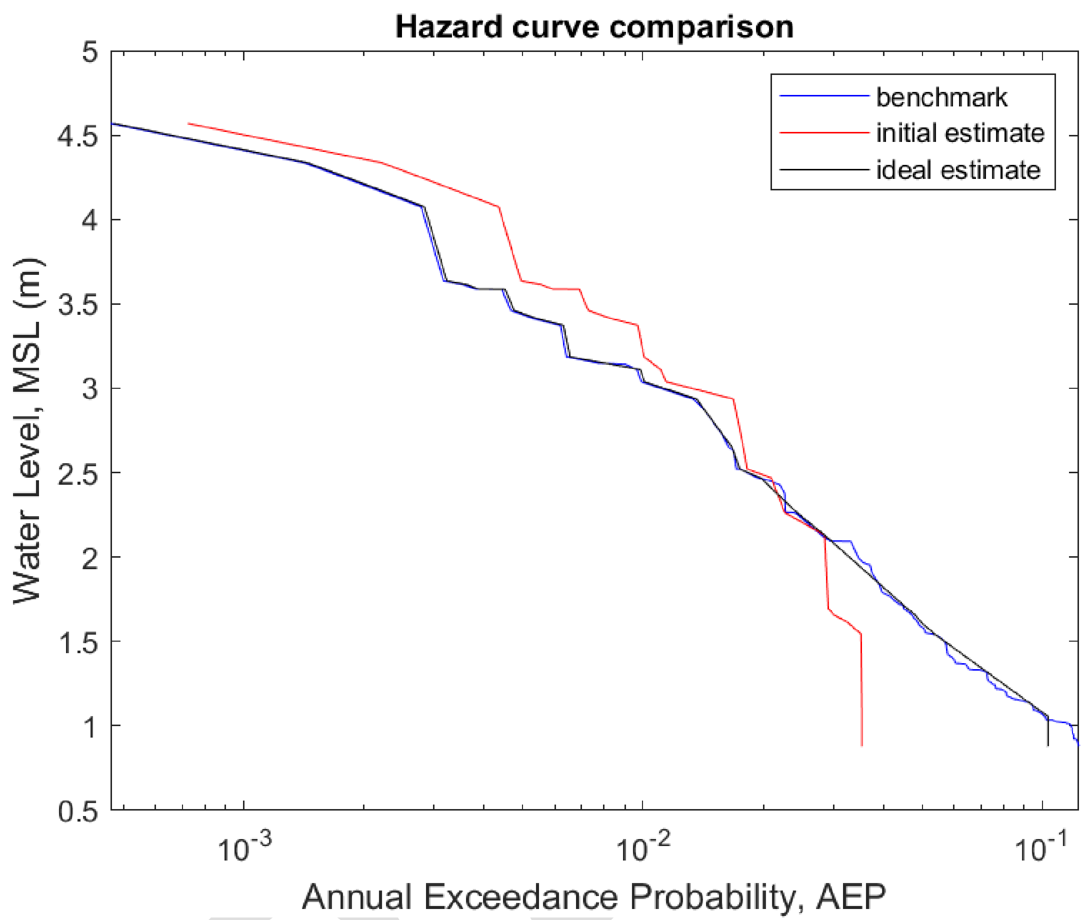


Figure 3 shows the results of the optimization process, from the initial random guess hazard curve (red) to the optimal 20-storm (RSS) hazard curve (black) matching the “benchmark” or full storm set (FSS) hazard curve. The figure illustrates that a sample of 20 storms converges and ultimately results in a hazard-curve error very close to zero for the intended range of AEPs of the full storm set (i.e., 2%, 1%, and 0.2%; or 50, 100, and 500 years).



The tracks and storm number for each of the 20 storms in the RSS are shown in **Figure 4**. As expected, a majority of the selected storms have paths to the left of the study area, since higher storm surge and flooding impacts are caused by the right side of the hurricanes due to their counterclockwise vorticity.

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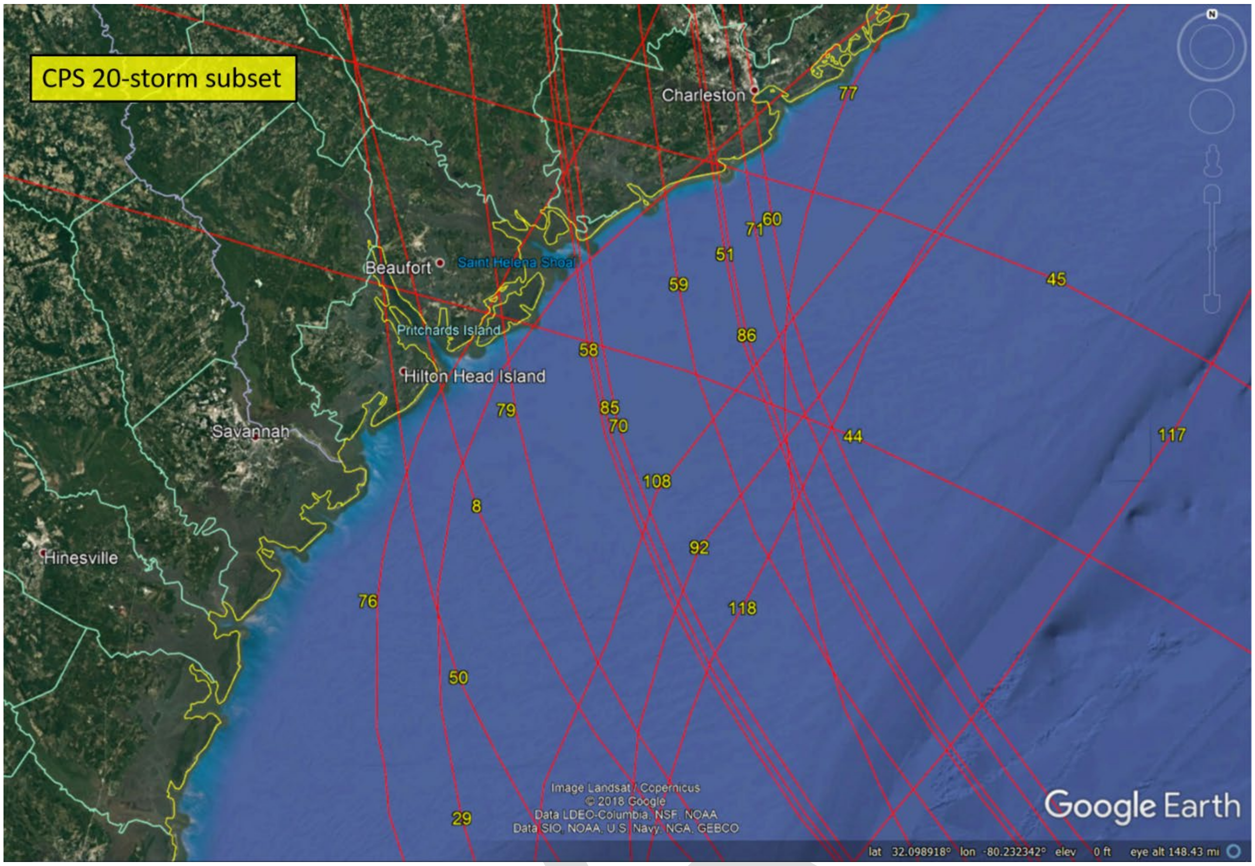
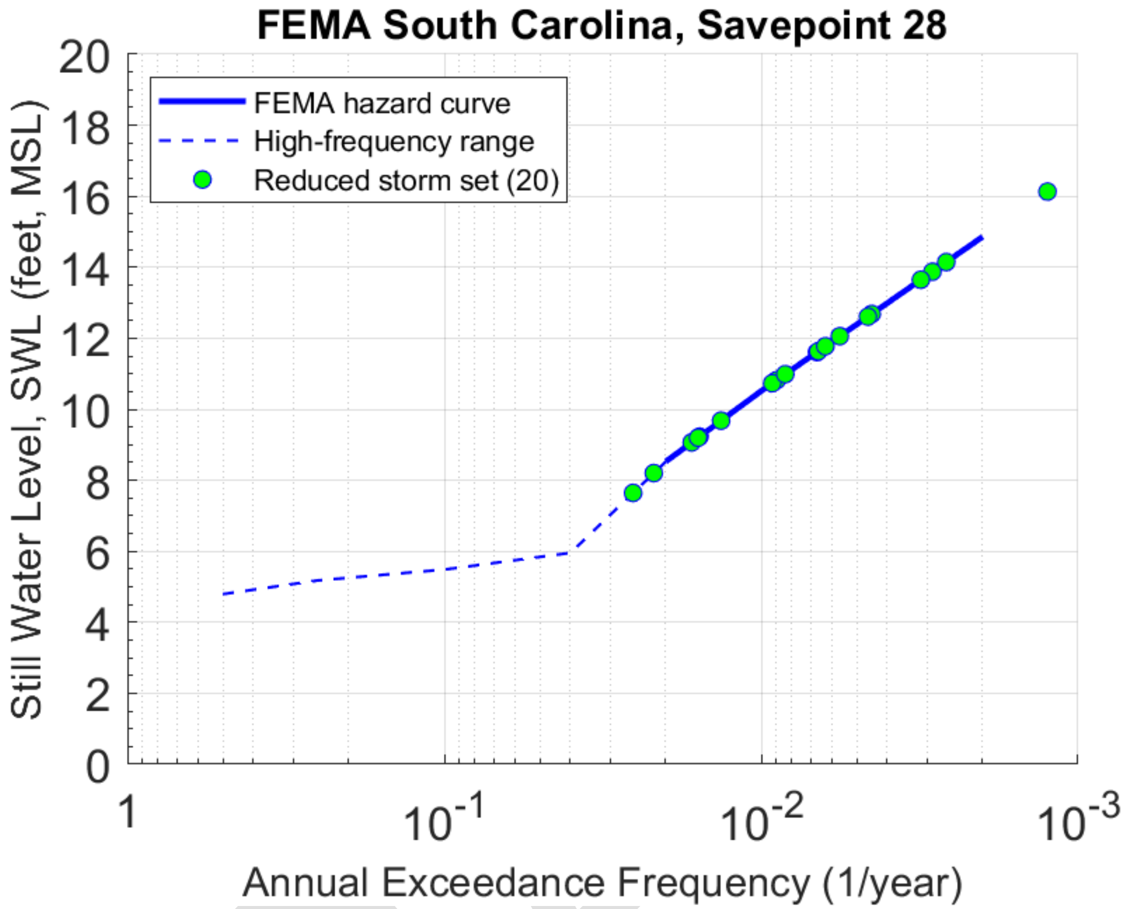
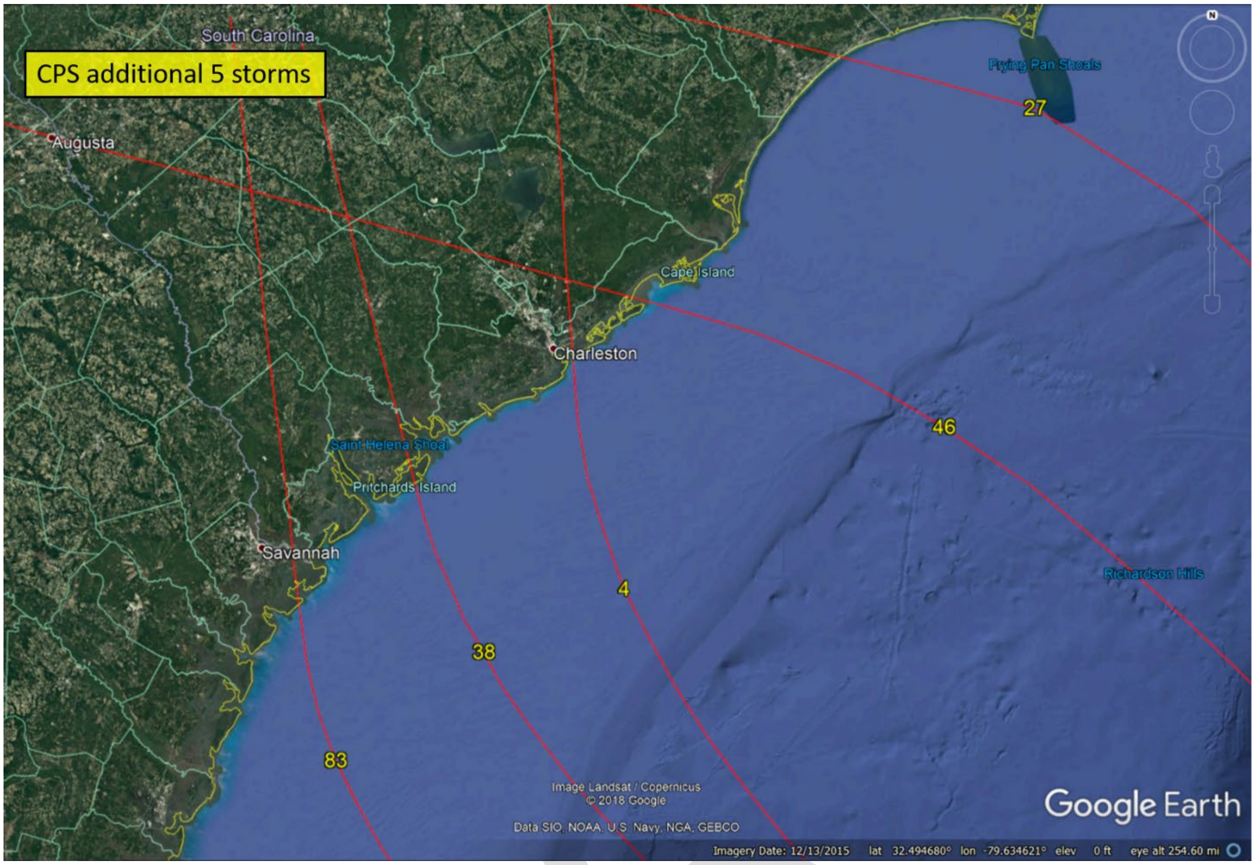


The FEMA SWL hazard curve for CPS save point no. 28 is shown in **Figure 5.** Also illustrated in this plot are the 20-storm subset and corresponding AEFs (x-axis) and SWL (y-axis).



In CPS, the need for storms representing the high-frequency range of the SWL hazard was later identified; this is, storms outside the range of regulatory FEMA SWL corresponding 2%, 1%, and 0.2% AEP. As discussed in the SCSSP report, although not required by FEMA, water levels corresponding to 50%, 20%, 10%, and 4% AEP were determined through extreme value analysis (EVA) of water levels recorded at tidal gages. Therefore, five (5) additional storms were selected from the range of probabilities determined from EVA of water level measurements.

The tracks and storm number for each of the additional five storms, and for the updated 25-storm subset are shown in **Figure 6** and **Figure 7**, respectively. As previously stated the additional five storms are intended to represent the high-frequency range of the SWL hazard. Although the FEMA SCSSP storms were not designed for this purpose, the five selected storms can serve as proxies for low-magnitude SWL responses due to a combination of relatively low intensities and/or long distances from the study area.


CPS additional 5 storms
South Carolina
Augusta
Charleston
Cape Island
Savannah
Saint Helena Shoal
Pritchards Island
Frying Pan Shoals
Richardson Hills
27
46
4
38
83
Image Landsat / Copernicus
© 2018 Google
Data SIO, NOAA, U.S. Navy, NGA, GEBCO
Google Earth
Imagery Date: 12/13/2015 lat 32.494680° lon -79.634621° elev 0 ft eye alt 254.60 mi

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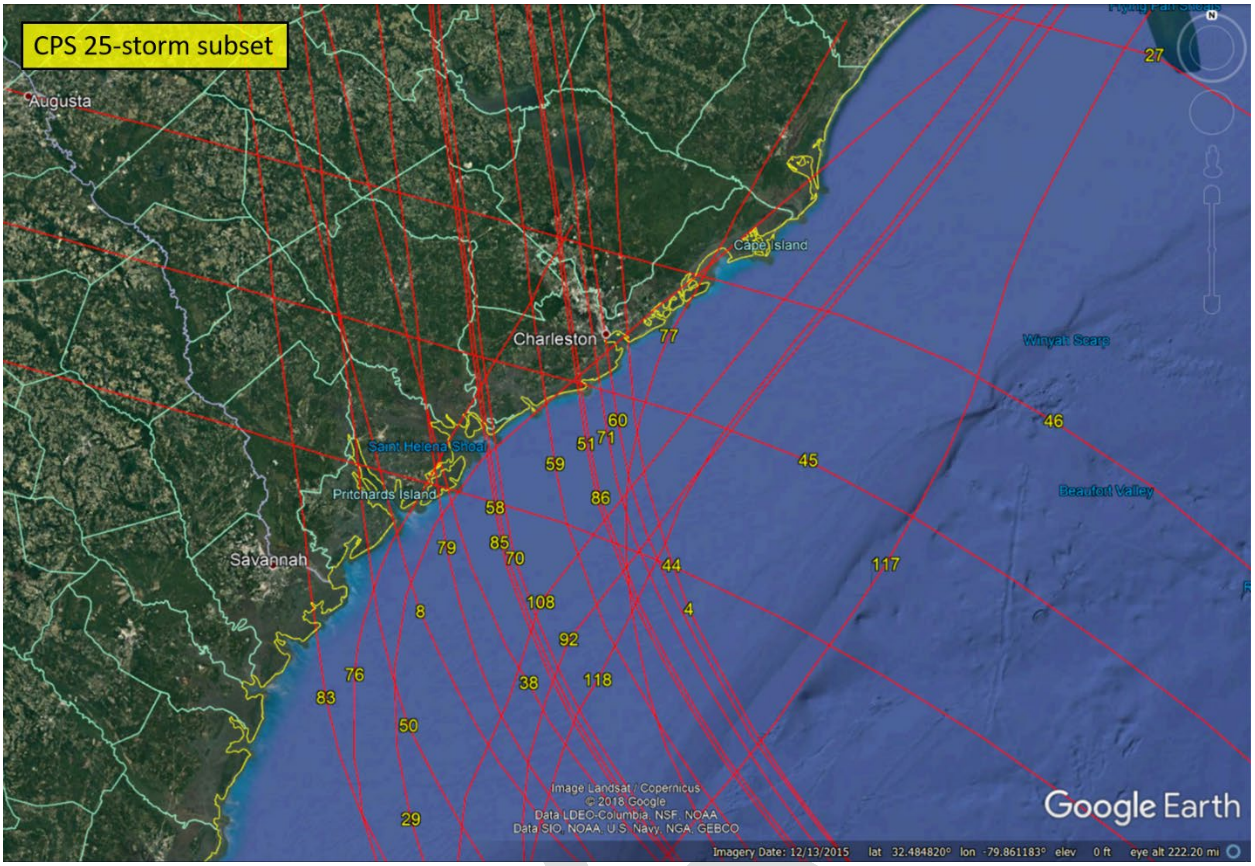


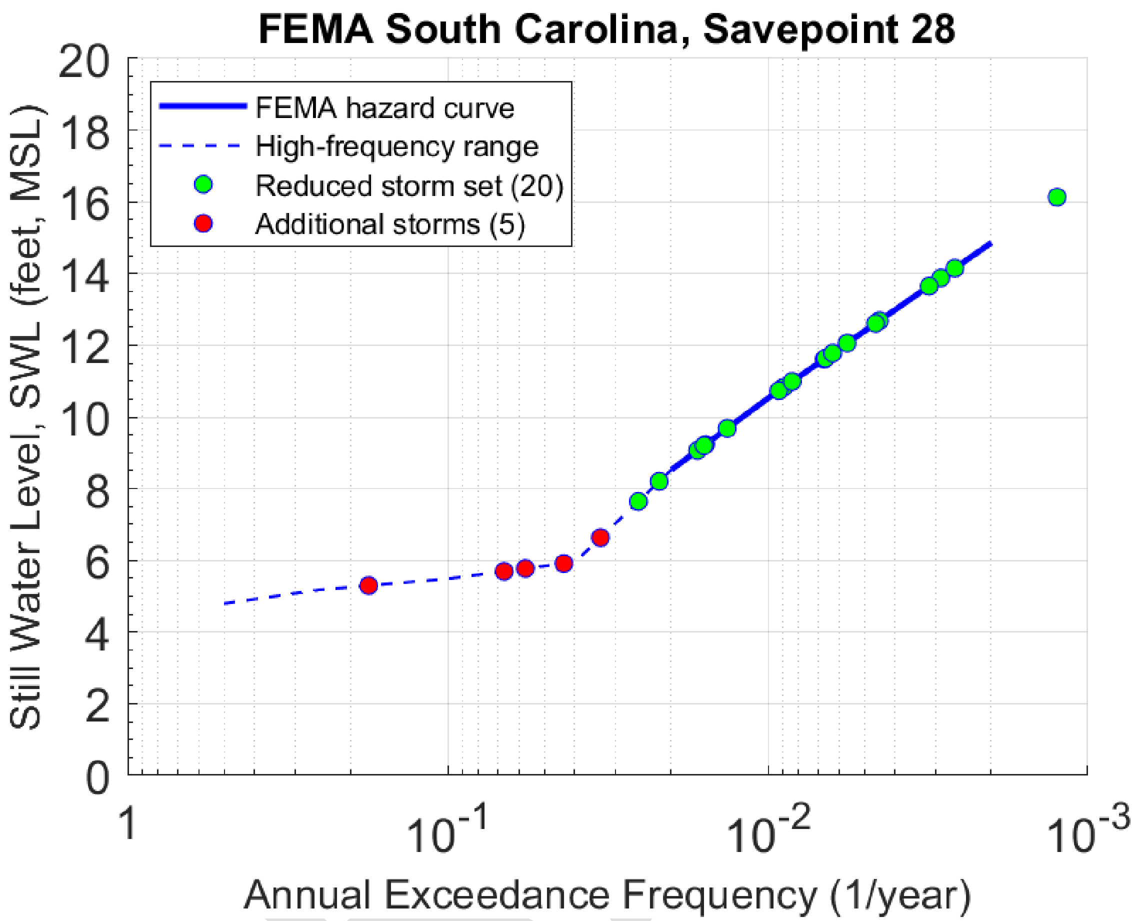
The FEMA SWL hazard curve for CPS save point no. 28, along with the 20-storm subset (red circles) and the additional 5 storms (red circles) are shown in **Figure 8**. The same information is depicted in **Figure 9** with the x-axis modified to show AEPs instead of AEFs.

Figure 9 seems to show an additional green circle not seen in **Figure 8**. This green circle corresponds to synthetic TC no. 117. At station no. 28 the water level produced by this storm has an AEP of 99.4%, as seen in **Figure 9**. The AEF of the water level associated with this storm is 5.12 year-1, which is outside the range of the x-axis in **Figure 8**.

Storm recurrence rates for each of the 25 storms were estimated for Monte Carlo sampling purposes within G2CRM based on:

i) the parameterization of hurricane climatology, as described in the SCSSP JPM-OS report (URS 2012) for the FSS and adjusted for the RSS following Zhang et al. 2018; and

ii) the interpolation of ADCIRC-simulated water level responses from the SCSSP hazard curves reproduced by AECOM and provided to SAC.



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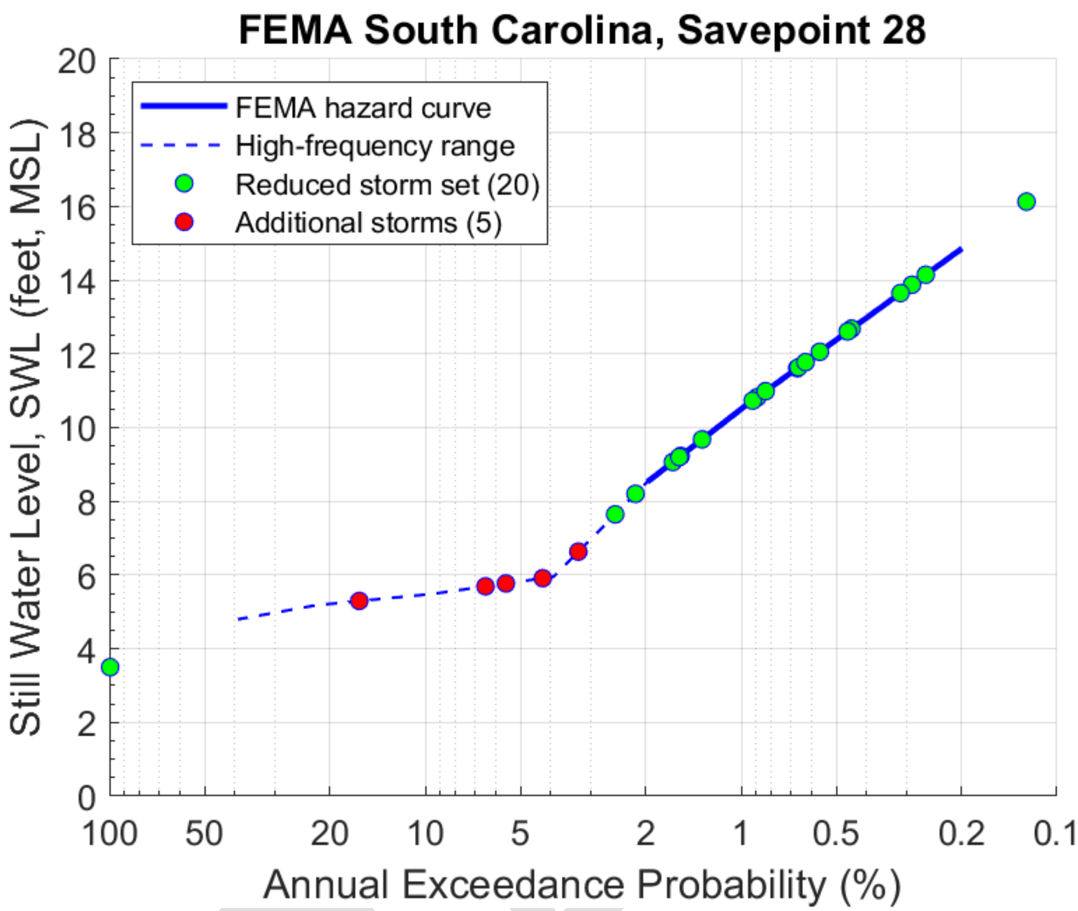
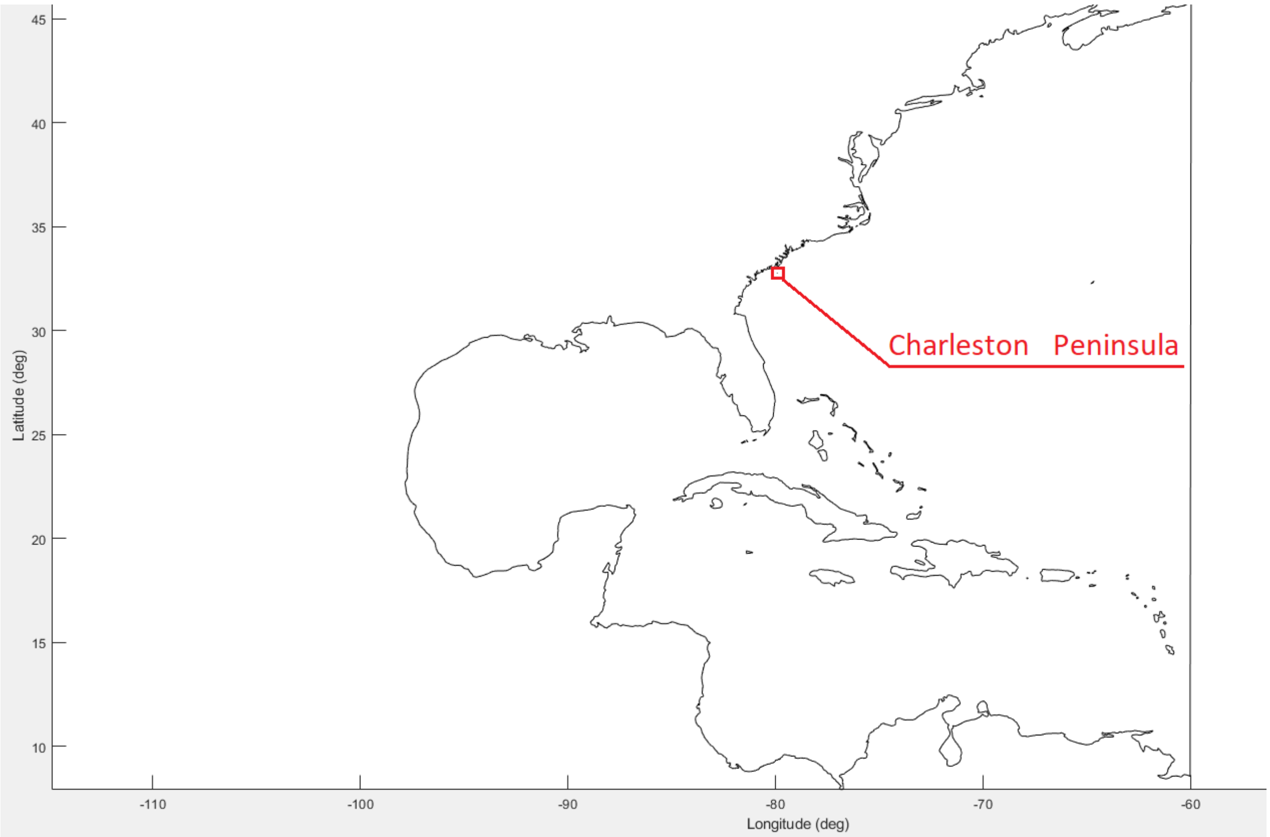


ADCIRC MESH DETAILS AND MODEL PARAMETERS: The computational domain for CPS, shown in **Figure 10**, was derived from South Carolina Storm Surge Study grid (FEMA 2013). While the original ADCIRC mesh boundary was maintained, the grid elements in the vicinity of the study area were refined by increasing the resolution of the mesh in order to provide more details in the region of interested. **Figure 11** shows the target area for the grid refinement.


Charleston Peninsula
Latitude (deg)
Longitude (deg)
45
40
35
30
25
20
15
10
-110
-100
-90
-80
-70
-60

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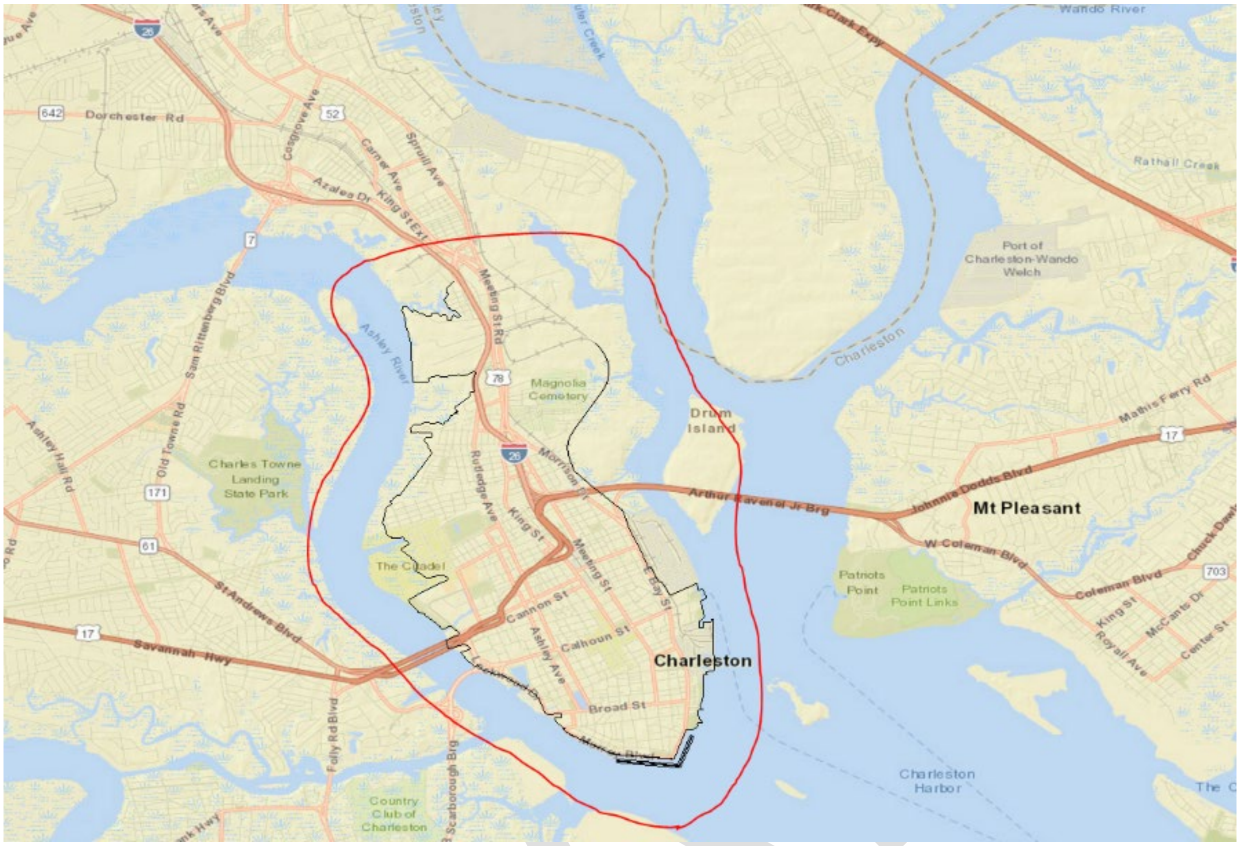

However due to grid instabilities caused by rapid change from element size of 150-200 m to 15-25 m, a buffer zone had to be created around the grid refinement target area (**Figure 12** and **Figure 13** show the CPS area before and after the grid refinement, respectively**). Figure 14, Figure 15,** and **Figure 16** show successive magnifications of the refined area to better illustrate the transition of the element size. This zone enabled gradual transition from the coarse grid to the refined grid inside the target area. Moreover to achieve additional grid stability, the element sizes in the target area were set to 25-35 m.

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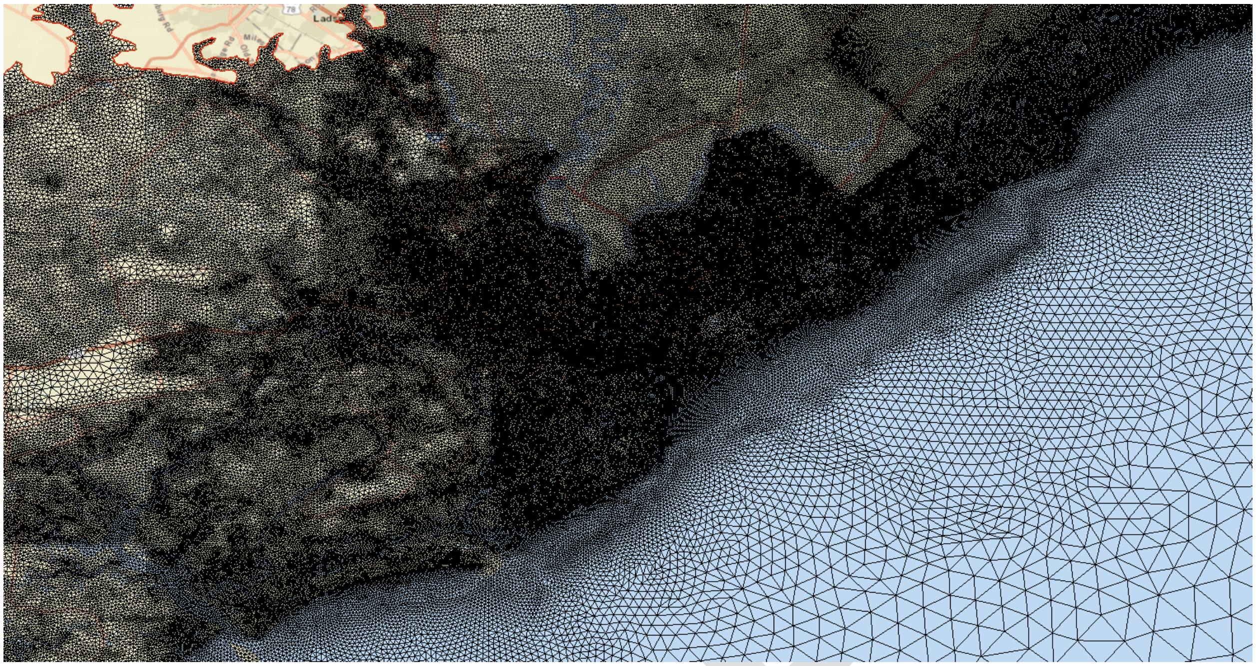

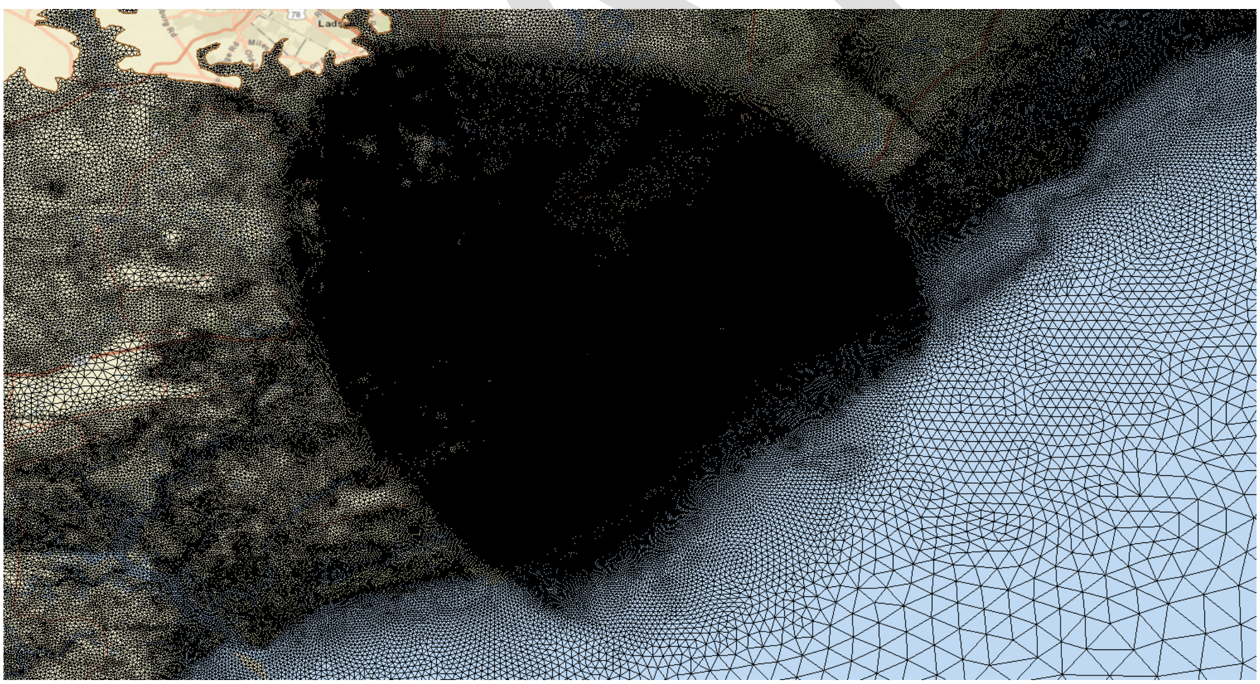

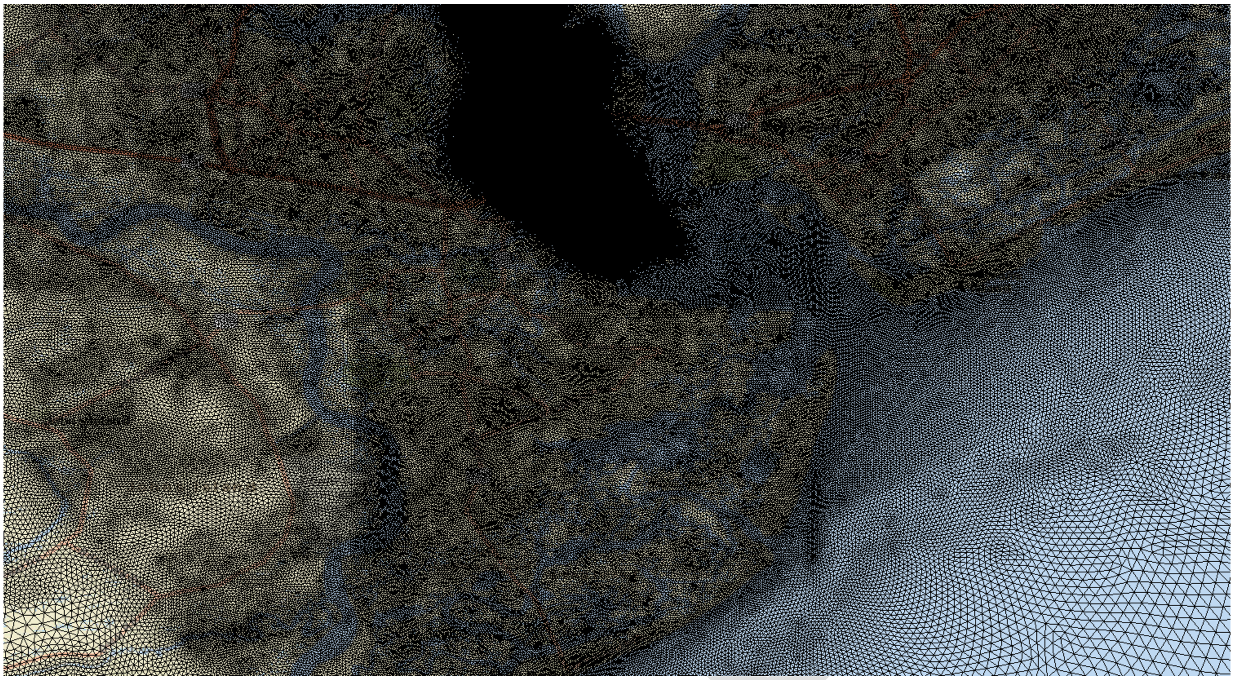

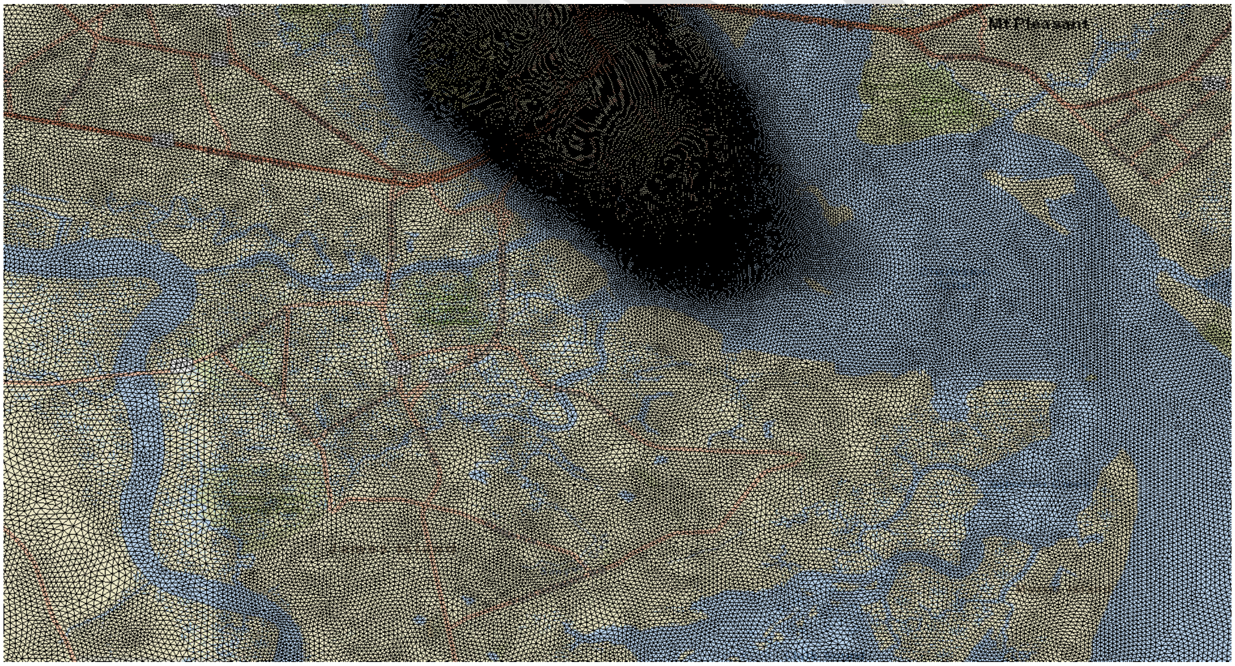
Mt Pleasant

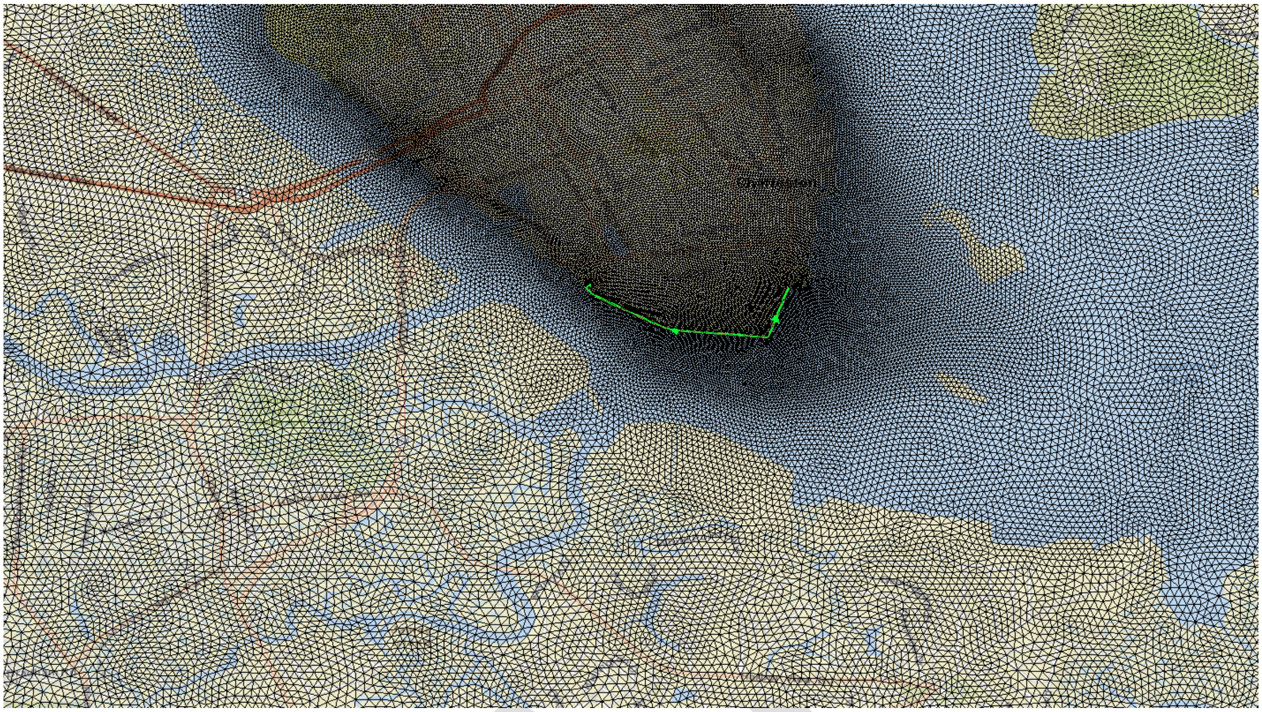

Figure 17 and **Figure 18** illustrate the difference in the mesh resolution before and after the refining procedure. The number of nodes increased from 542809 (South Carolina Storm Surge Study grid) to 793975 (Charleston Peninsula Study – EC grid). The green line in **Figure 18** indicates the Battery implemented as a weir-pair subgrid feature according to the specifications provided by SAC. The Battery in the original South Carolina Storm Surge Study FEMA grid was represented by topographic values.

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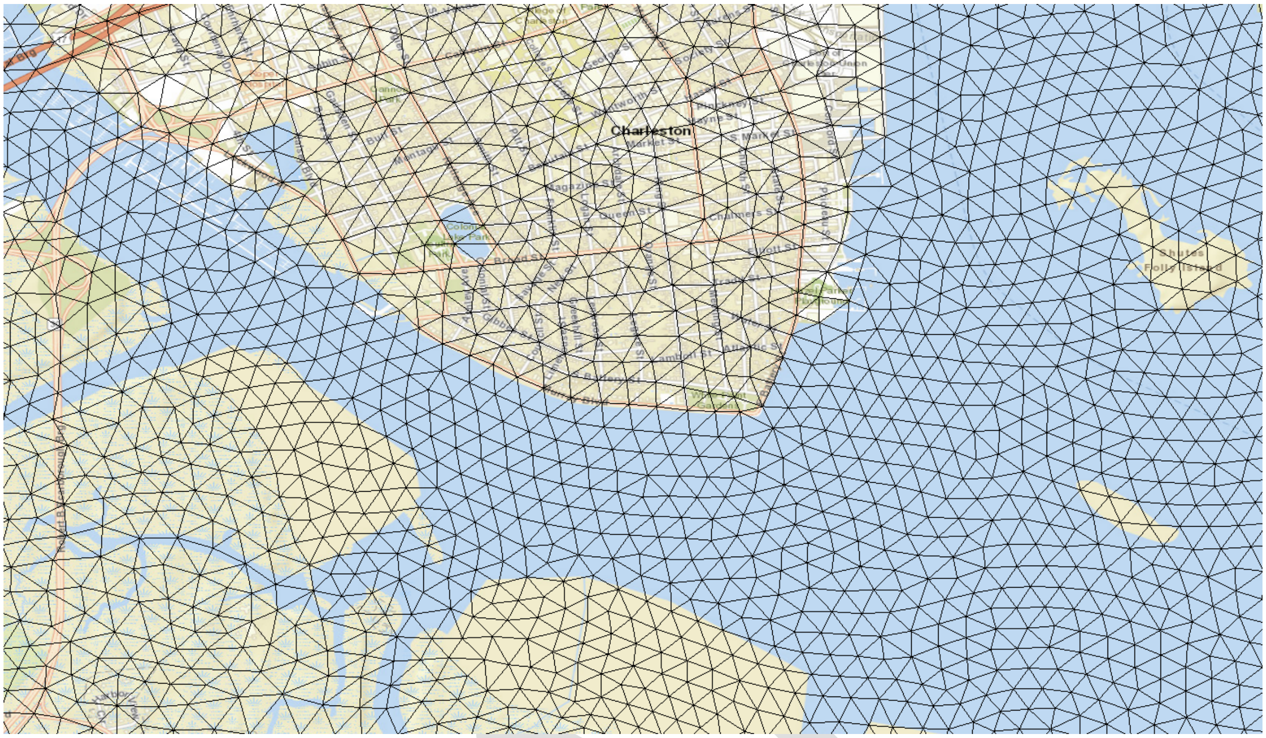

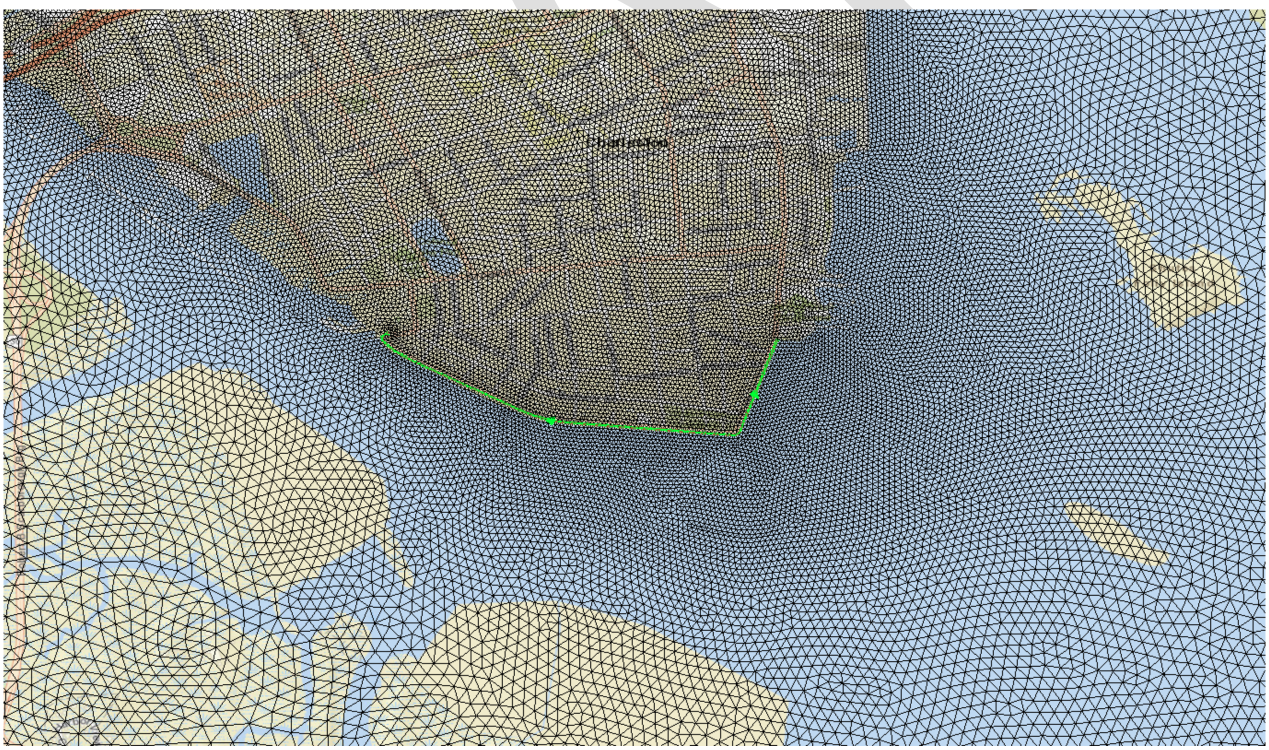

Finally, the grid topography/bathymetry (South Carolina Storm Surge Study – FEMA) had to be updated in the region distant and hydraulically independent from the study area shown in **Figure**

19 in order to maintain model stability. Because of the stability issue, smoothing (elimination of narrow creek and channels away from the area of study) and a slope limiter (selective limiting of the maximum water elevation within a mesh element) were applied and are also shown in **Figure 19**. Smoothing and implementation of the slope limiter did not contaminate the results of the study since they were applied 60 km away from Charleston Peninsula to the area that is hydraulically independent from the study area. The source of the updated bathymetry was the Northeast Florida Georgia Surge Study conducted by FEMA.

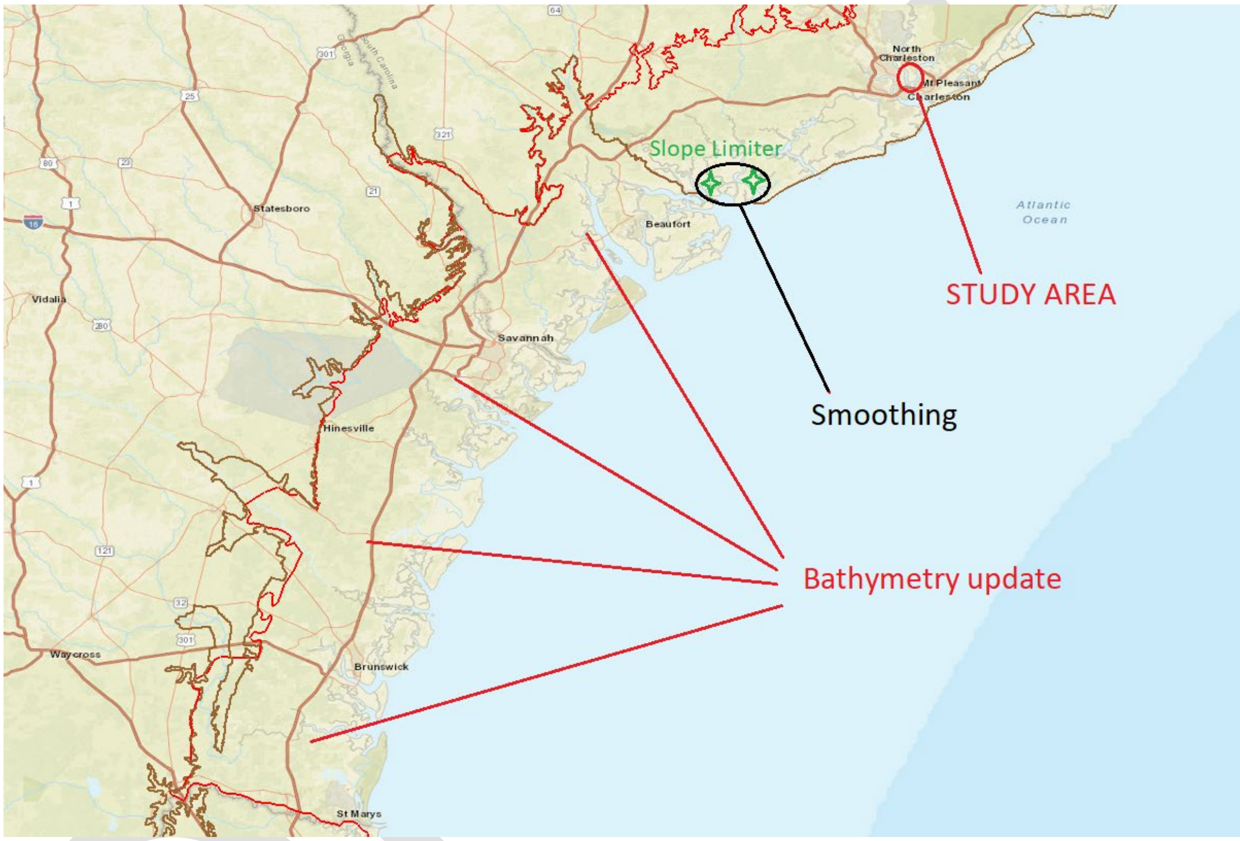

Figure 19: South Carolina Storm Surge Study FEMA grid with locations where bathymetry update, smoothing, and slope limiter were applied.

The resulting grid with the above modifications and the implemented Battery (shown in **Figure 20 -** red/black line) as a weir-pair subgrid feature served as Existing Condition (EC) grid. The elevation of the Battery (EC) was set to 9.1 ft., NAVD88 for the higher wall and 6.8 ft., NAVD88 for the existing low battery wall. Then the elevation of the Battery was modified according to the guideline provided by SAC (the higher and the existing low battery wall was set to 9.0 ft., NAVD88) to generate the Future WithOut Project (FWOP) conditions mesh. This mesh, in turn, served as the base for developing the final grid configuration which contained a breakwater. The breakwater (shown in **Figure 20** – magenta line), also implemented as a weir-pair subgrid feature, was set to 16.2 ft., NAVD88 (elevation of the crest). This grid is referred as the With Project condition (WP01).

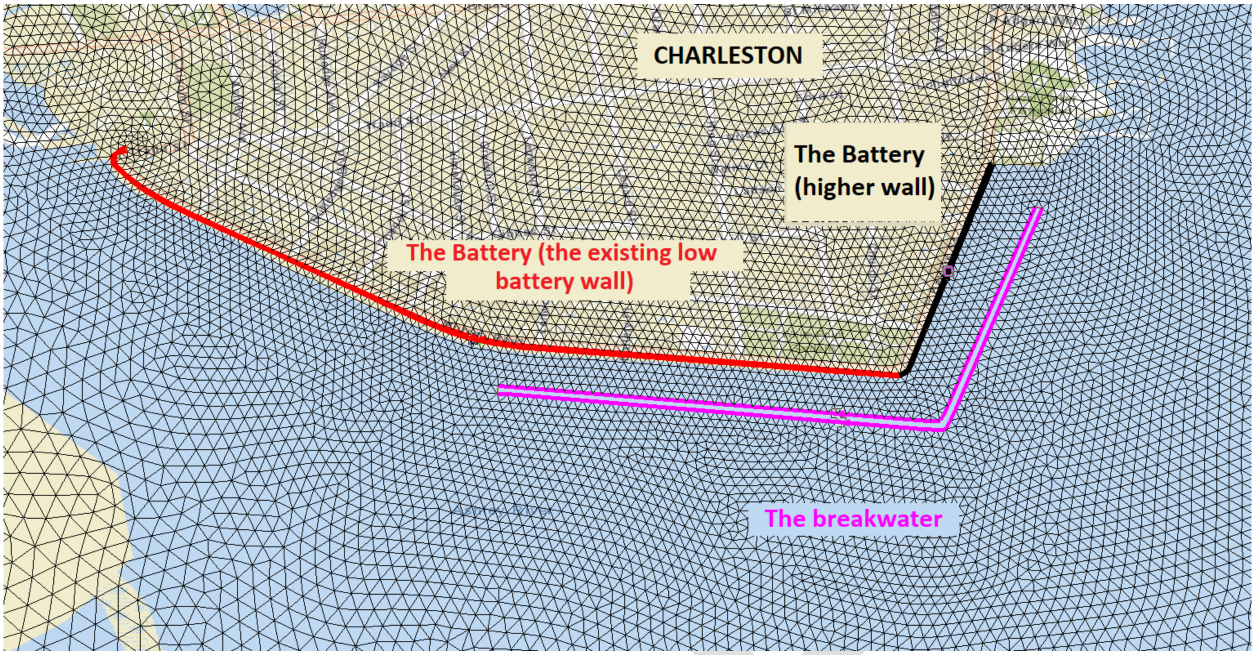

Figure 20: Charleston Peninsula with the Battery (the existing low battery wall: red, the higher wall: black) and the breakwater (magenta).

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ADCIRC OUTPUT FILES - MAXIMUM WATER ELEVATION AND TIME-SERIES OF WATER ELEVATION: The maximum water elevation and water elevation time-series plots presented in this section are based on a single storm event (Synthetic Tropical Storm #27) for the brevity of the document. All model output is referenced to MSL, meters, however at the request of SAC plots shown in this section are converted to NAVD88, feet. Note that water levels do not include tide or sea level change per the requirement of G2CRM; measures were included in G2CRM to conservatively account for nonlinear effects of increased water levels on ADCIRC output. The water elevation time-series illustrate results for save points preselected by SAC/SAW (Stations: 180, 598, 329, 699, and 976; shown **Figure 21**) that were also used for generation of the G2CRM .h5 files. The plots of maximum water elevation for the other four storm events (Storms: 4, 38, 46, and 83) are found in **Appendix A** of this document. The characteristics of all five storms are in **Appendix D**. Storm characteristics detailed in Appendix D include Rmax (the radius of maximum wind for a tropical cyclone), storm track, landfall location, Cp (Central pressure of a tropical cyclone), RAD1 (the scale pressure radius related to the radius of maximum winds), B1 (the peakedness of the primary wind maxima), and wind speed. Characteristics for all other storms can be found in the Appendix B of the FEMA document (FEMA 2012). **Figure 22, Figure 23,** and **Figure 24** show that the modification to the Battery elevation for the FWOP grid as well as the implementation of the breakwater had an insignificant effect on maximum storm surge water levels. That is, the three maximum surge envelopes show identical patterns of maximum water elevation. This observation is confirmed by hydrographs in figures:

- **Figure 25, Figure 26, and Figure 27** – time-series plots recorded at Station 976

- **Figure 28, Figure 29, and Figure 30** – time-series plots recorded at Station 699
- **Figure 31, Figure 32, and Figure 33** – time-series plots recorded at Station 598
- **Figure 34, Figure 35, and Figure 36** – time-series plots recorded at Station 329
- **Figure 37, Figure 38, and Figure 39** – time-series plots recorded at Station 180

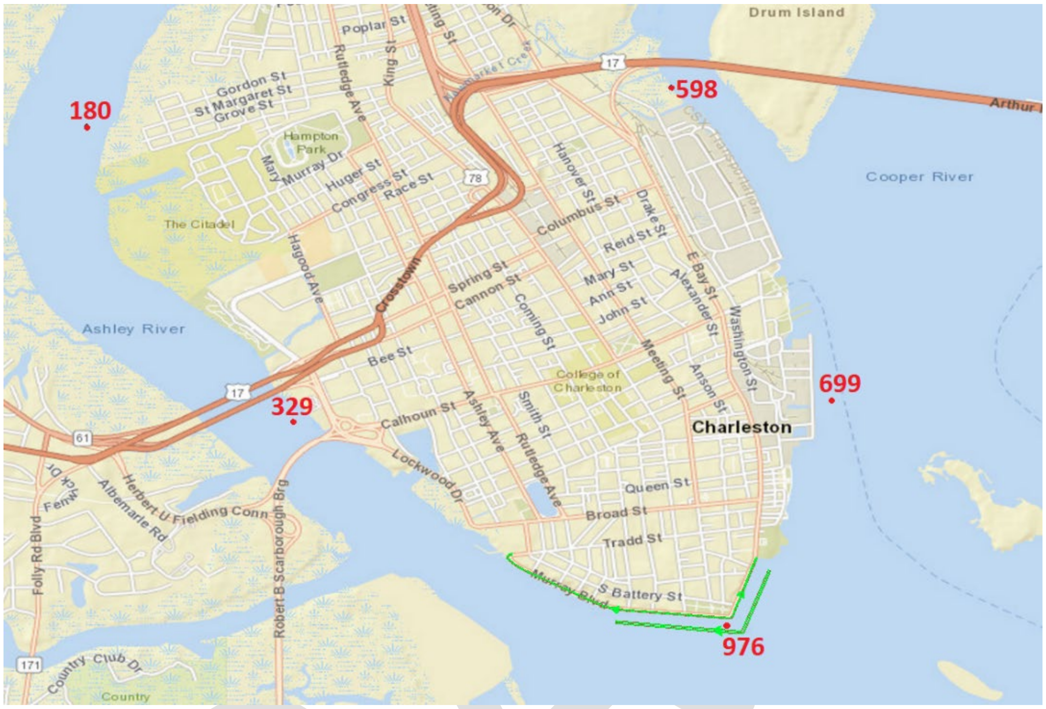

Figure 21: The locations of the save points: 976, 699, 598, 329, and 180.

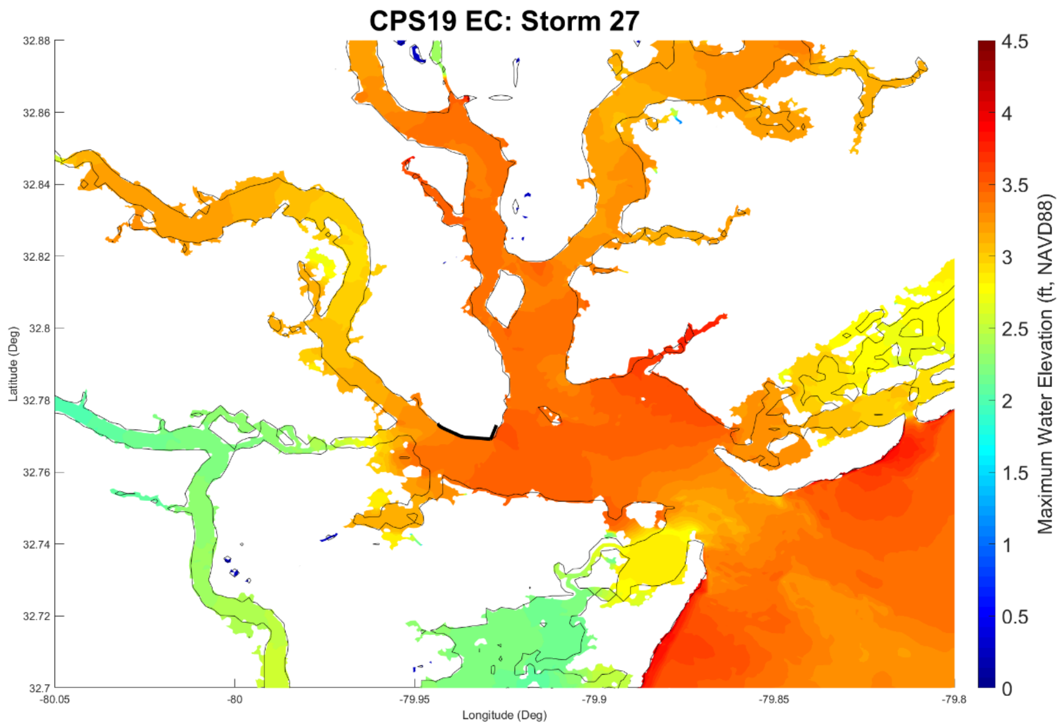

Figure 22: CPS EC grid, Storm 27.

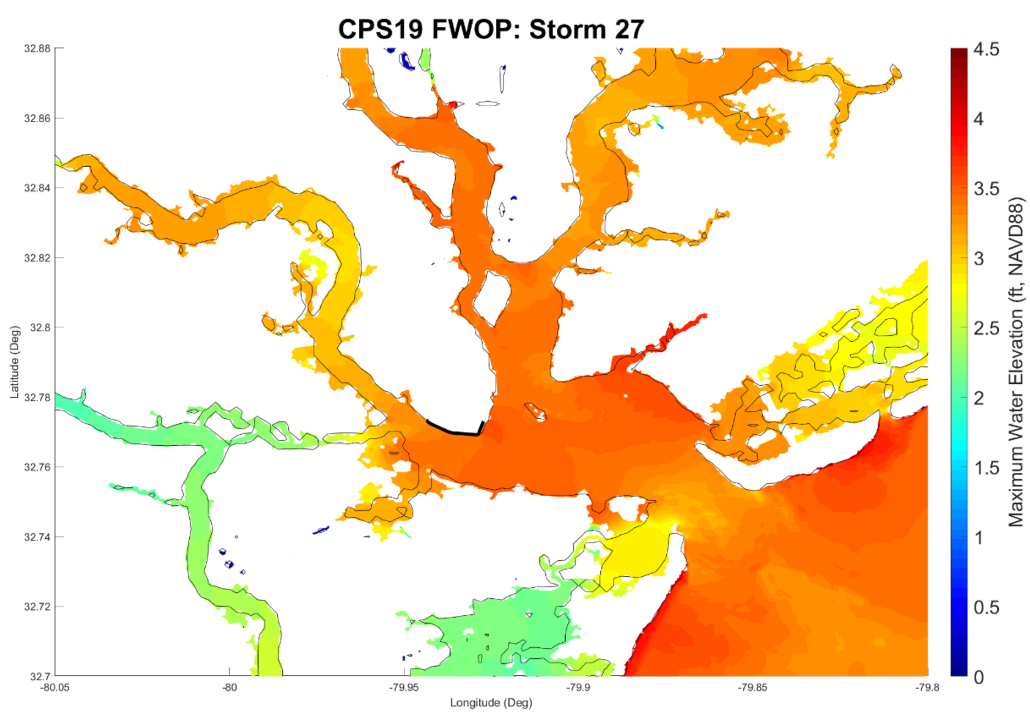

Figure 23: CPS FWOP grid, Storm 27.

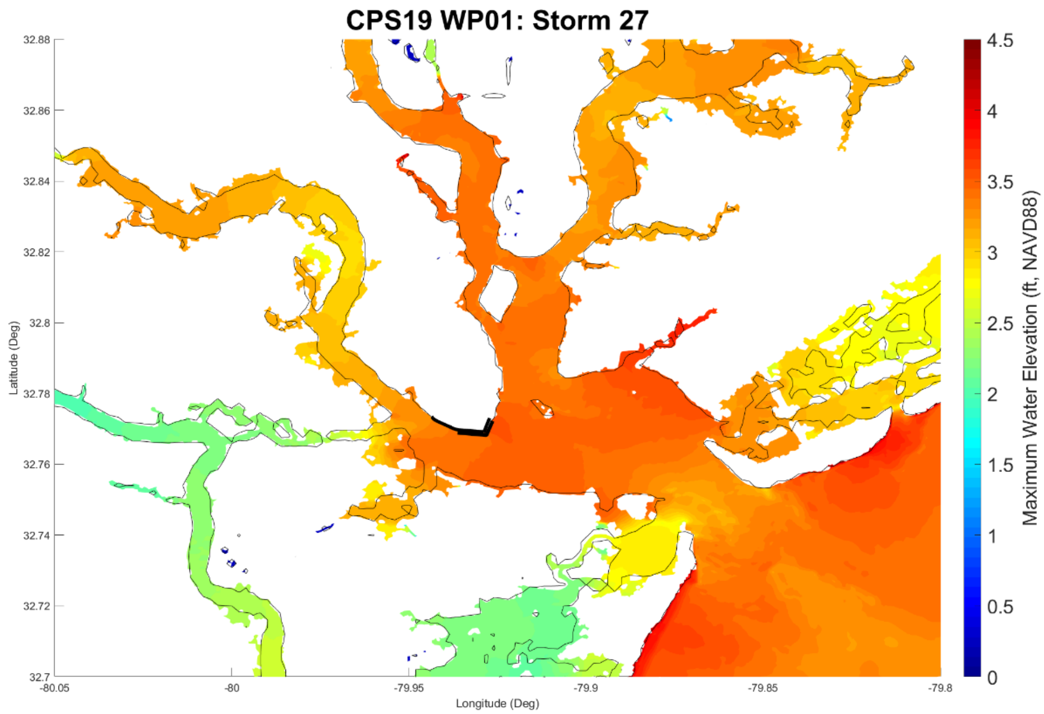

Figure 24: CPS WP01 grid, Storm 27.

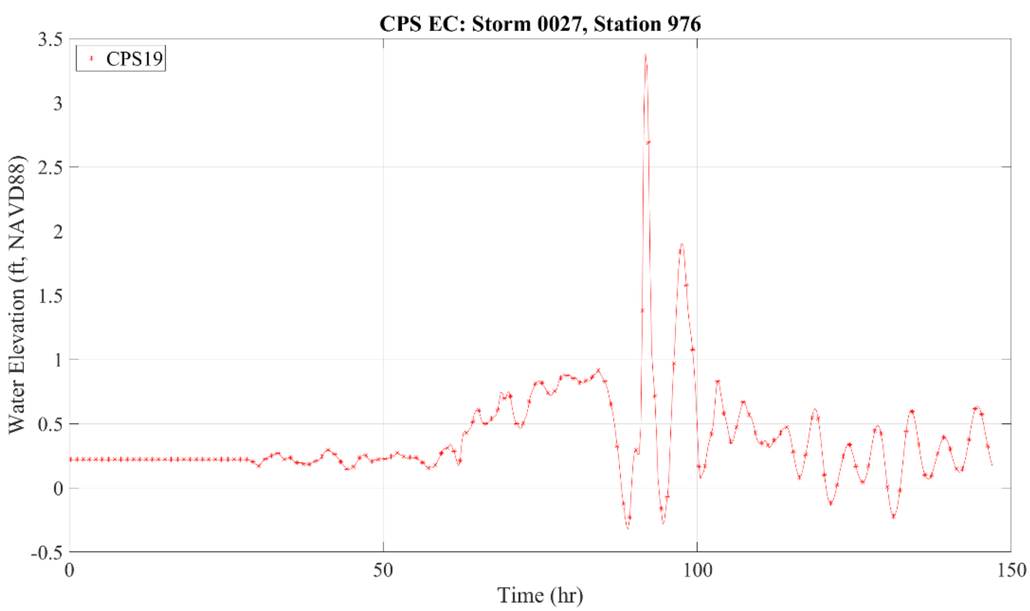

Figure 25: CPS EC grid, Storm 27, Station 976.

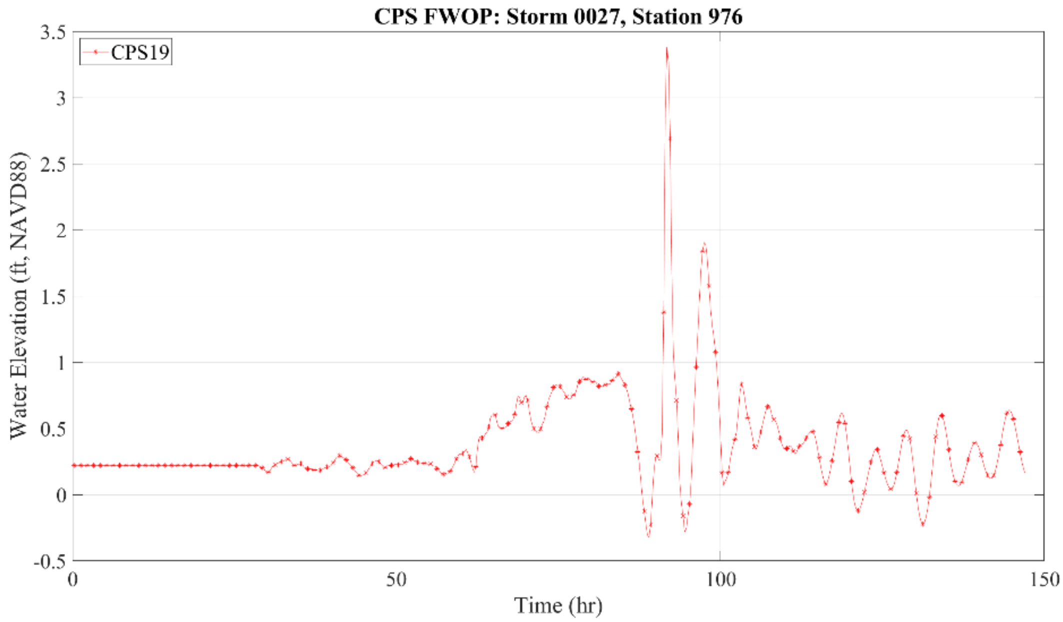

Figure 26: CPS FWOP grid, Storm 27, Station 976.

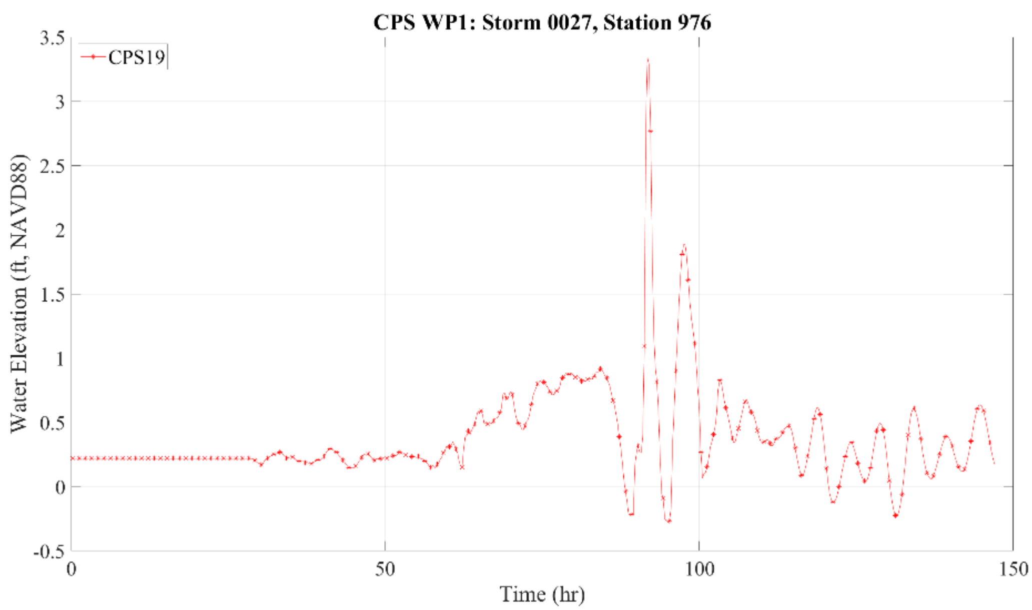

Figure 27: CPS WP01 grid, Storm 27, Station 976.

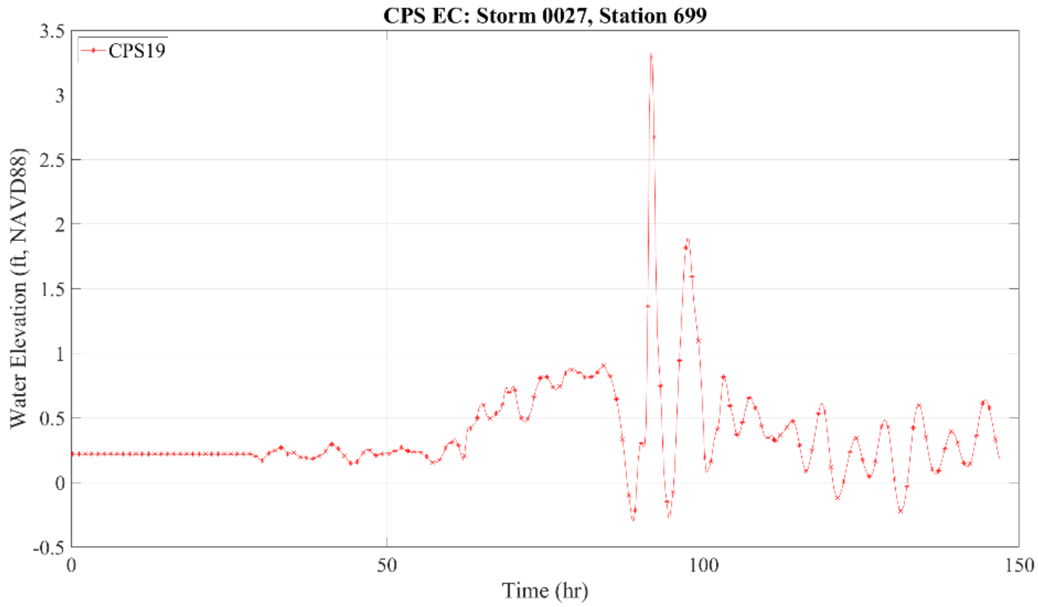

Figure 28: CPS EC grid, Storm 27, Station 699.

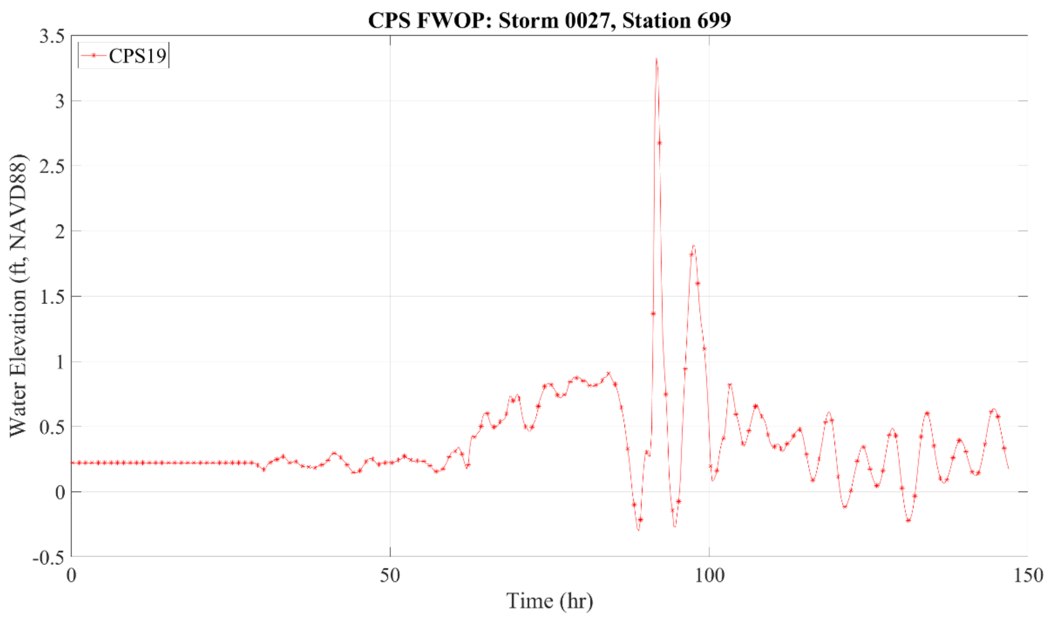

Figure 29: CPS FWOP grid, Storm 27, Station 699.

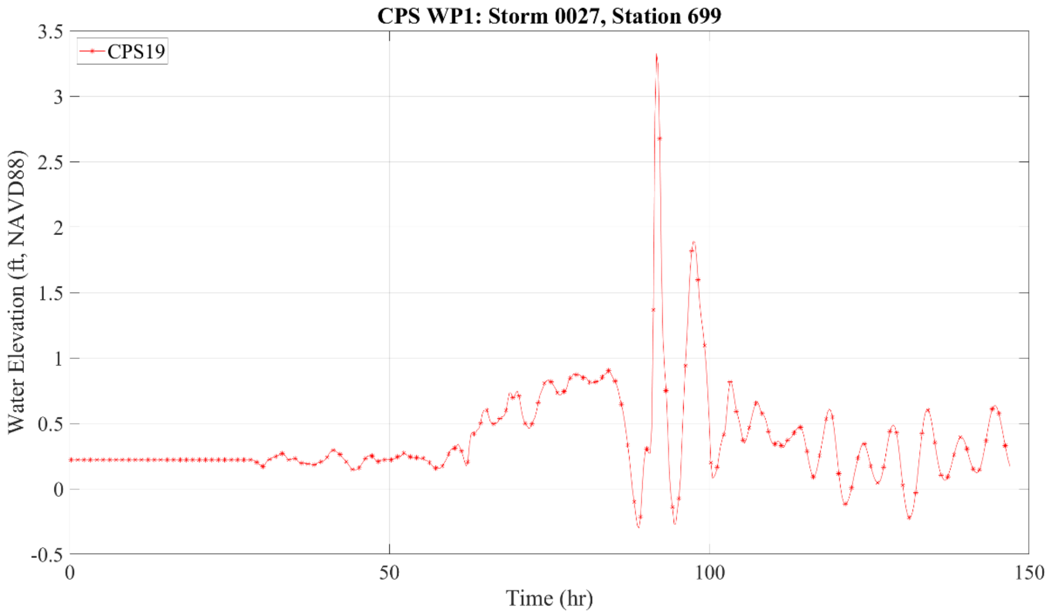

Figure 30: CPS WP01 grid, Storm 27, Station 699.

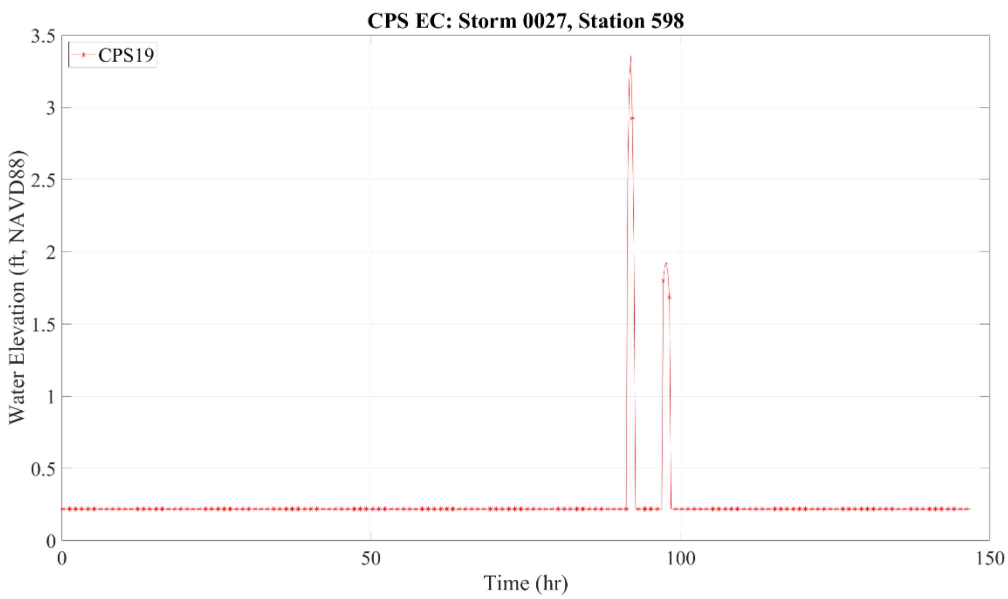

Figure 31: CPS EC grid, Storm 27, Station 598.

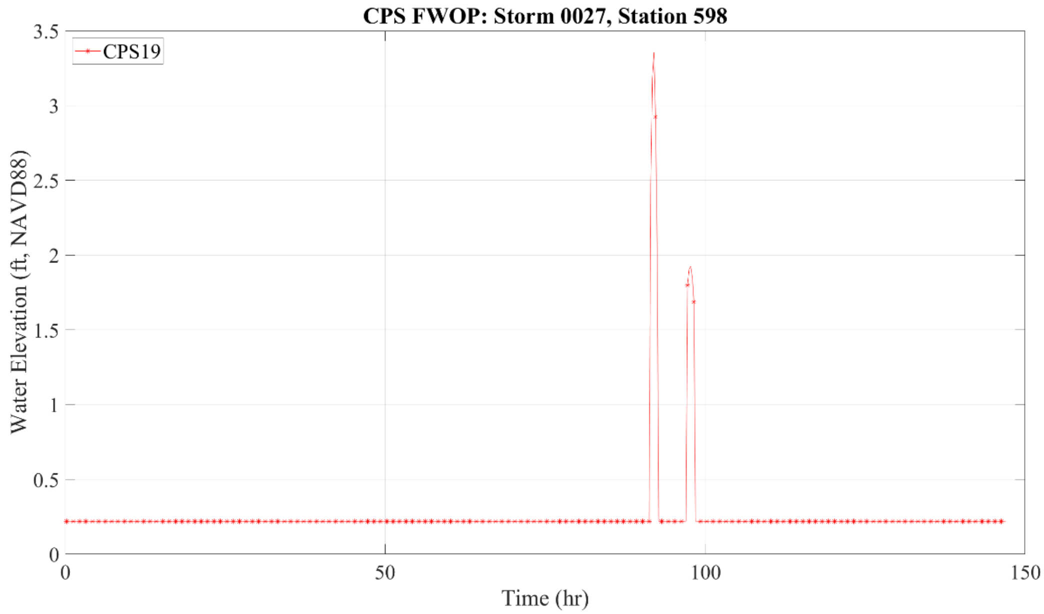

Figure 32: CPS FWOP grid, Storm 27, Station 598.

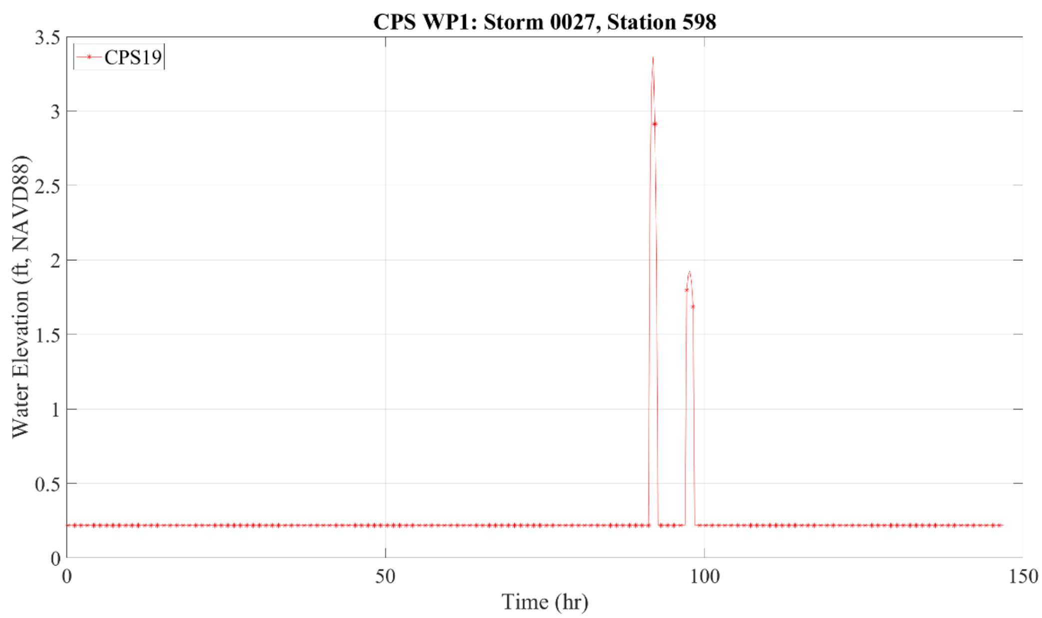

Figure 33: CPS WP01 grid, Storm 27, Station 598.

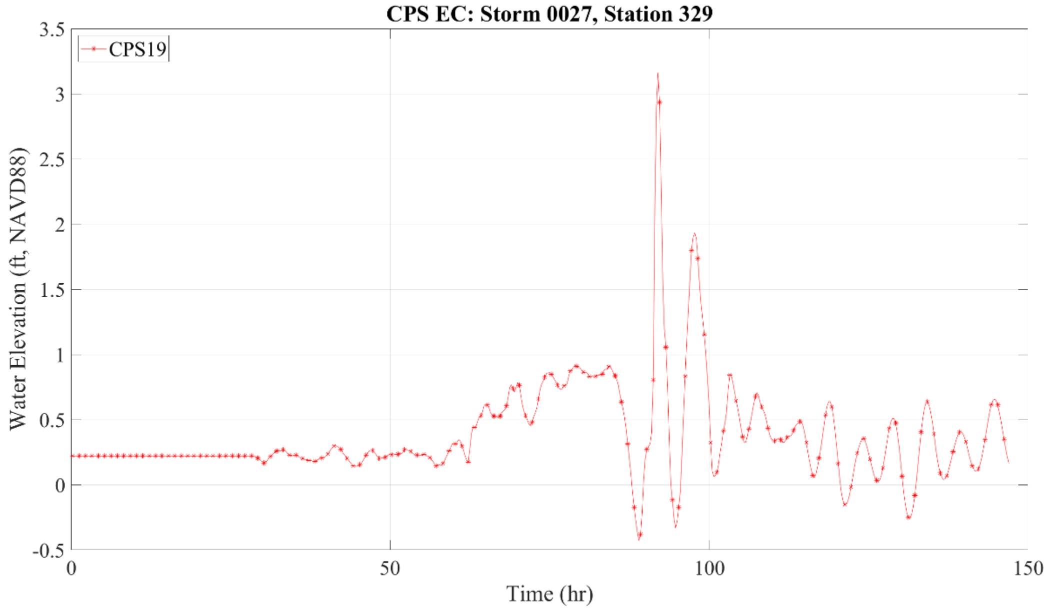

Figure 34: CPS EC grid, Storm 27, Station 329.

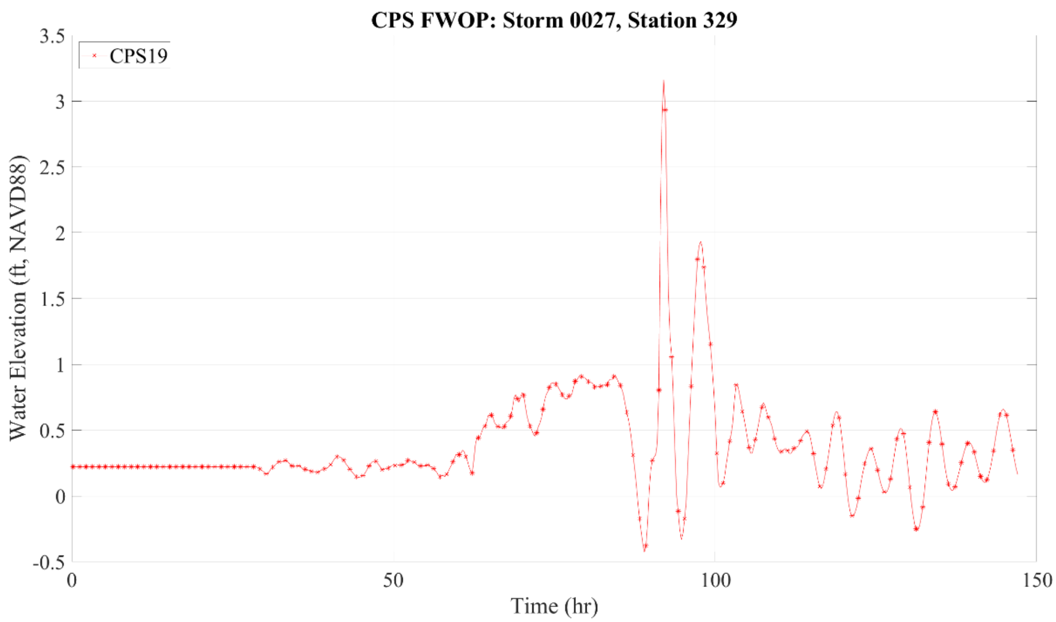

Figure 35: CPS FWOP grid, Storm 27, Station 329.

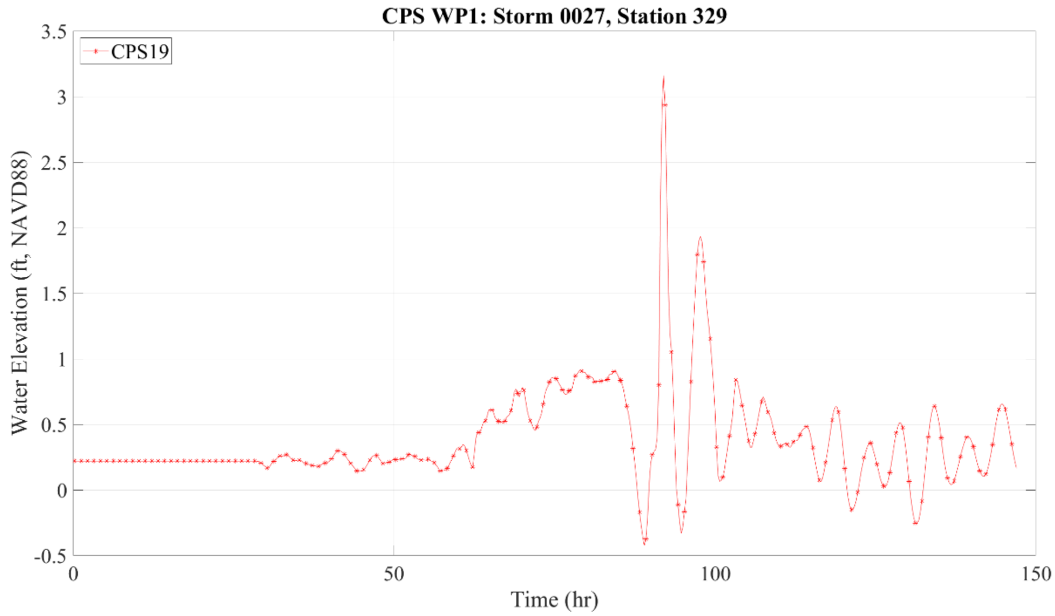

Figure 36: CPS WP01 grid, Storm 27, Station 329.

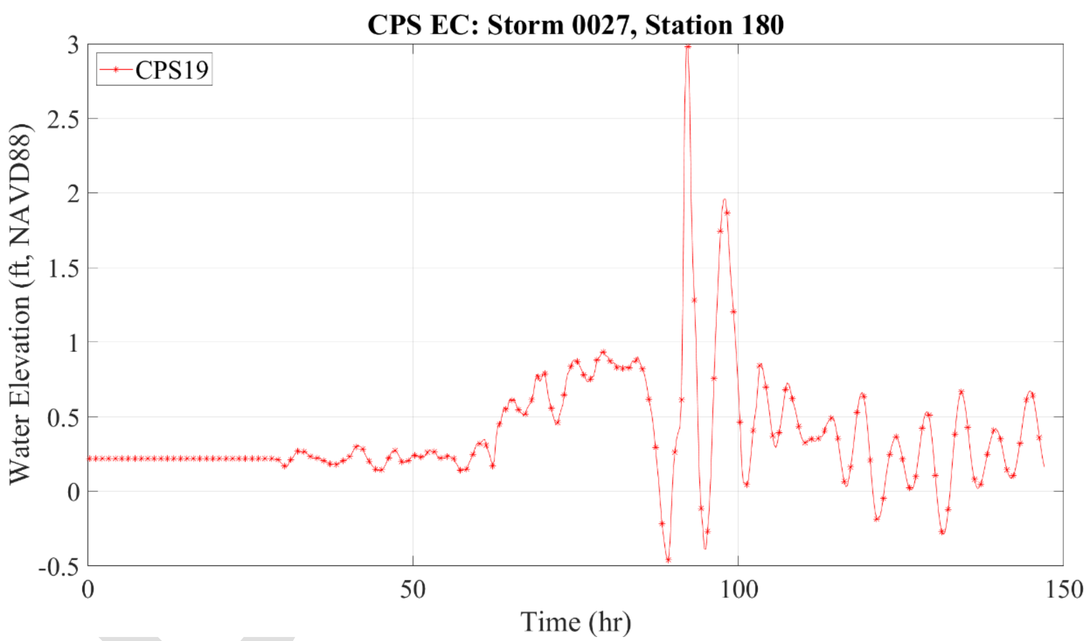

Figure 37: CPS EC grid, Storm 27, Station 180.

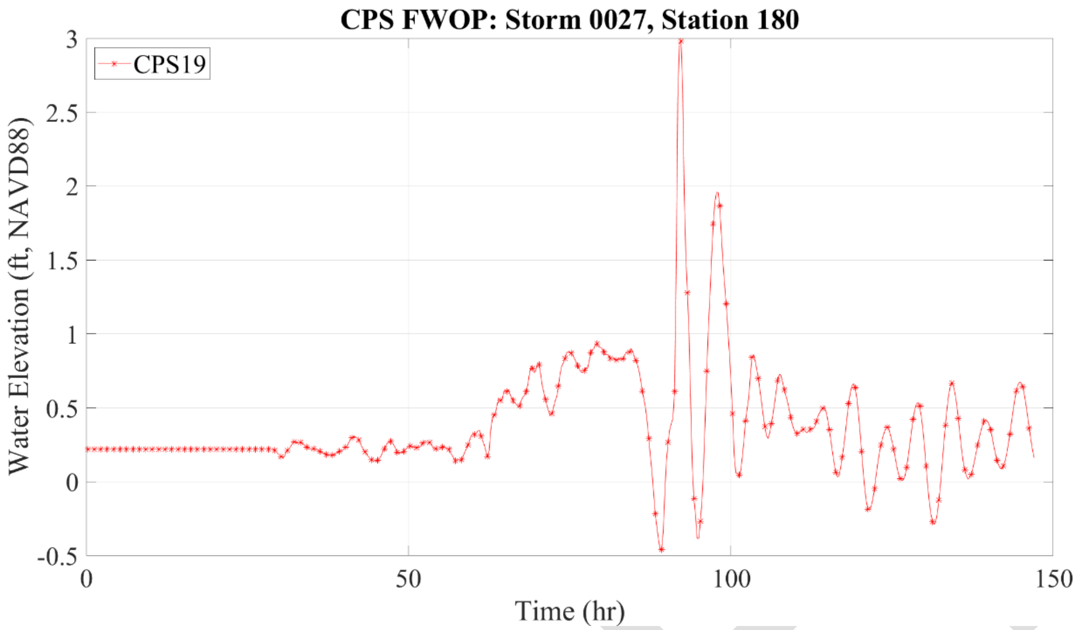

Figure 38: CPS FWOP grid, Storm 27, Station 180.

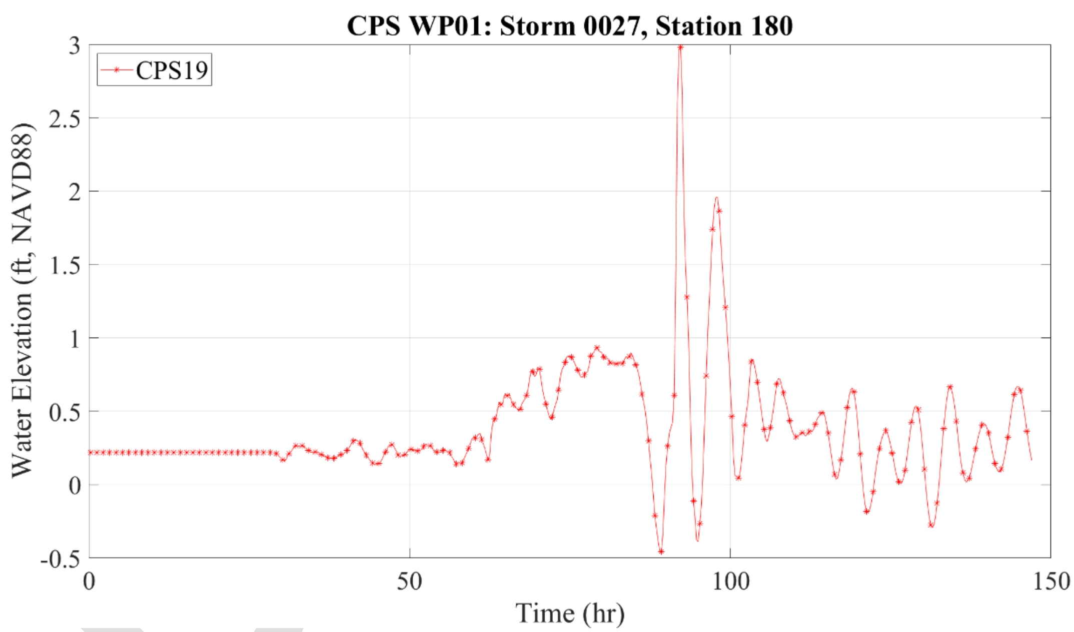

Figure 39: CPS WP01 grid, Storm 27, Station 180.

STWAVE: To incorporate the effects of waves for each storm, the steady state spectral wave model, STWAVE, was used to simulate nearshore wave generation, propagation, transformation, and dissipation (Smith et al. 2001, Smith 2007, Massey et al. 2011). STWAVE numerically solves the steady-state conservation of spectral wave action along backward-traced wave rays:

$$\left(C_g\right)_i \frac{\partial}{\partial x_i} \frac{C C_g \cos \alpha E(\sigma, \theta)}{\sigma} = \sum \frac{S}{\sigma} \tag{1}$$

where i is tensor notation for x- and y- components, C_g is group celerity, θ is wave direction, C is wave celerity, σ is wave angular frequency, E is wave energy density, and S is energy source and sink terms. Source and sink mechanisms included surf-zone wave breaking, wind input, wave-wave interaction, whitecapping, and bottom friction. STWAVE is formulated on a Cartesian grid, with the x-axis oriented in the cross-shore direction (I) and the y-axis oriented alongshore (J), generally parallel with the shoreline. Angles are measured counterclockwise from the grid x-axis.

GRID DEVELOPMENT: The STWAVE grid extended alongshore from Folly Beach, SC to the south to Dewees Island, SC to the north, and seaward to a depth of approximately 82 ft (25 m) to allow for transformation of waves from the offshore boundary into the nearshore. The Cartesian grid was approximately 49 ft (15 m) in resolution and was comprised of 3386 cells in the cross-shore direction (I) and 2383 cells in the alongshore direction (J). The projection of the grid was State Plane Coordinate System, South Carolina (FIPS 3900). **Table 1** provides the properties of the STWAVE domain.

Table 1. STWAVE Grid Properties.

Horizontal Projection	Grid Origin (x,y) [m]	Azimuth [deg]	Δx/Δy [ft]	Number of Cells	
				I	J
South Carolina (FIPS 3900)	(754814.787125, 88860.170325)	134.43	49	3386	2383

The bathymetry, topography, and bottom friction Manning's *n* values to populate the STWAVE domain were interpolated from the ADCIRC mesh. The final STWAVE domain overlaid on aerial imagery is shown in **Figure 40**. Although the area of interest for this study was a smaller area near the Charleston Peninsula, the STWAVE domain extents were designed to capture wave transformation from the offshore to the nearshore and limit boundary effects in the area of interest.

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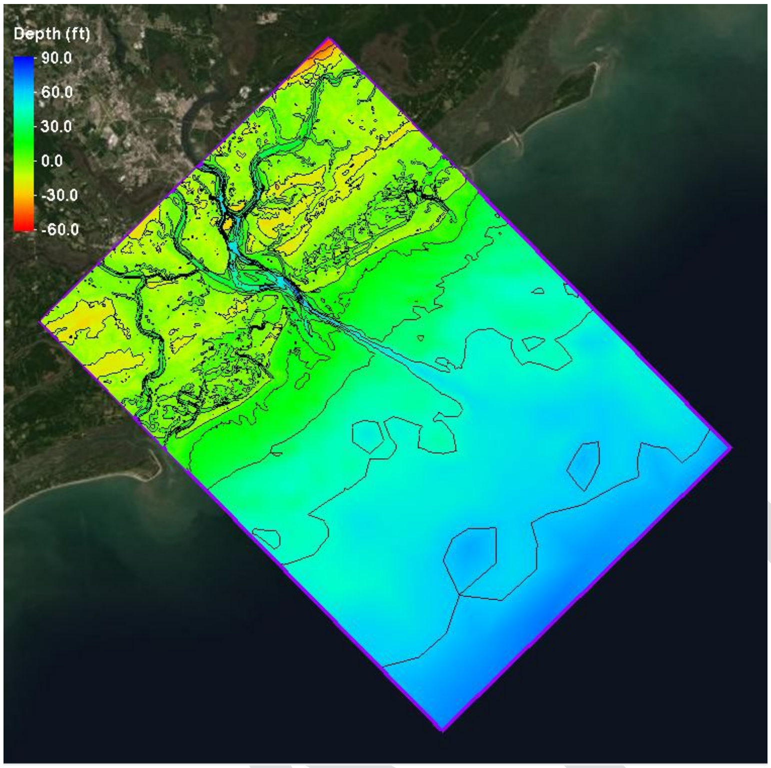

Figure 40: STWAVE domain extents.

In the area of interest, two different structural features were implemented under three conditions. The STWAVE domain was updated for the three conditions, existing condition (EC), future without project (FWOP) and with project (WP01), by interpolating from the ADCIRC mesh with the added features. **Figure 41** depicts the bathymetry of the area of interest, with the locations of the implemented features circled in black.

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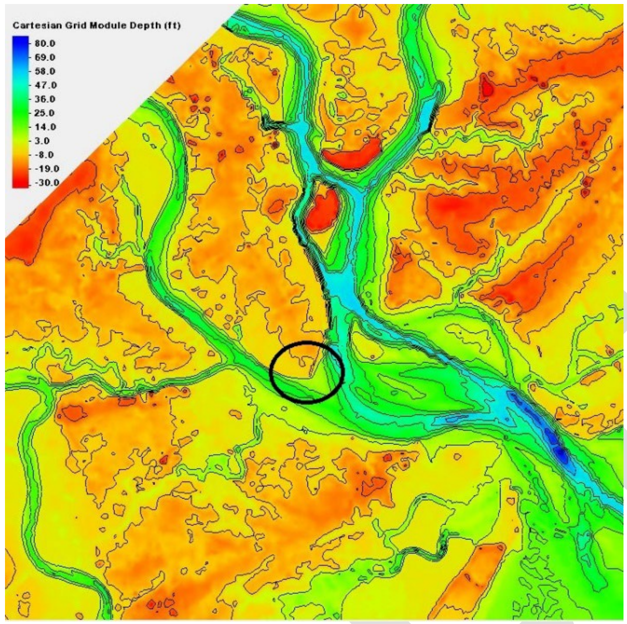

Figure 41: Bathymetry around the area of interest. The locations of the implemented features are located at the seaward tip of the peninsula (circled in black).

As stated in the ADCIRC section, the Existing Condition (EC) featured a battery wall with a height of 6.8 ft NAVD88 (existing low battery wall) which transitioned to a height of 9.1 ft NAVD88 (higher wall). The Future Without Project (FWOP) also included the battery wall, but with a uniform, elevated height of 9.0 ft. NAVD88. In addition to the elevated battery wall, the With Project (WP01) condition included the addition of a 16.2 ft NAVD88 breakwater located in the foreshore of the peninsula.

OFFSHORE BOUNDARY SPECTRA: Available SWAN results, obtained from the FEMA contractor, were comprised of time series of bulk scalar parameters, including wave height, period, and direction. The STWAVE model, however, required explicit specification of input spectra including variation of wave energy in frequency and direction for this application. To construct the spectral boundary forcing, it is assumed that the detailed spectra are well-represented by the established Texel-Marsen-Arsloe [TMA, Bouws *et al.* 1985] spectral shape, and overall energy conservation is prescribed as

$$H_s = 4\left\{\int P_{\eta\eta}(f)\right\}^{1/2} \tag{2}$$

where $H_s$is the provided SWAN model result and $P_{\eta\eta}$ is frequency-dependent power spectral density. A TMA spectrum is a JONSWAP (Joint North Sea Wave Project) spectrum modified for shallow water. Directional dependence is computed as in Goda 2000, with a symmetric distribution around the peak angle

$$P_{\eta\eta}(f,\alpha) = G(\alpha)P_{\eta\eta} = G_0 \cos^{2s}(\frac{\alpha - \alpha_p}{2})P_{\eta\eta}(f) \quad (3)$$

where the scalar G_0 is numerically determined to normalize the directional function G, and s is a user-defined empirical parameter as provided in Goda 2000. The outputs of FEMA SWAN node 16763 served as the time series from which the spectra was constructed.

The resulting resolved spectra were represented by 35 frequency bands, ranging from 0.029 Hz (34.4 sec) to 0.32 Hz (3.1 sec), and 72 angle bands, from an angle of 0 degrees to 355 degrees with respect to the grid azimuth. Frequency and angular resolution were 0.00881 Hz and 5 degrees, respectively. To match the FEMA ADCIRC/SWAN modeling effort, the time interval for STWAVE spanned the last two days of the simulation, from 7-13-2000 12:00:00 to 7-15-2000 12:00:00, with regularly spaced intervals of 20 minutes. For coupling in CSTORM, STWAVE must start on a whole hour. Since the first output from the FEMA modeling effort began 20 minutes after the hour, the first spectra was duplicated for the STWAVE modeling, resulting in a total of 145 time steps per storm.

MODEL EXECUTION: Tight two-way coupling between ADCIRC and STWAVE was facilitated with the CSTORM-MS, a physics-based modeling capability for simulating tropical and extratropical storm, wind, wave, and water level response. During the two-way coupling process, a single instance of ADCIRC passes water elevations and wind fields to STWAVE and all wave parameters are updated throughout the STWAVE domain. Upon completion, STWAVE passes wave radiation stress gradients to ADCIRC to drive wave-induced water level changes (e.g., wave set-up and setdown). Each STWAVE simulation conducted used the full-plane mode of STWAVE to allow for wave generation and transformation in a 360-degree plane. The full-plane version of STWAVE uses an iterative solution process that requires user-defined convergence criteria to signal a suitable solution. Boundary spectra information is propagated from the boundary throughout the domain during the initial iterations. Once this stage converges, winds and surges are added to the forcing, and this final stage iteratively executes until it also reaches a convergent state. The convergence criteria for both stages include the maximum number of iterations to perform per time-step, the relative difference in significant wave height between iterations, and the minimum percent of cells that must satisfy the convergence criteria (i.e., have values less than the relative difference.) Convergence parameters were selected based on a previous study by Massey et al. (2011) in which the sensitivity of the solution to the final convergence criteria was examined. The relative difference and minimum percent of cells were set as (0.1, 100.0) and (0.05, 99.8) for the initial and final iterations, respectively. STWAVE was set up with parallel in-space execution whereby each computational grid was divided into different partitions (in both the x- and y-direction), with each partition executing on a different computer processor. The number of partitions in the x direction was 60 and the number of partitions in the y direction was 41. The maximum number of initial and final iterations was set to a value of 74 iterations, higher than the largest partition size.

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Additionally, 921 station locations, or save points, were identified within the STWAVE domain from the ADCIRC station list. During the simulations, these stations recorded the significant wave height, mean wave period, mean wave direction, peak wave period, wind magnitude, wind direction, and water elevation for each time step. Out of the 921 stations, five stations were identified to be of particular interest and were used for input into the G2CRM input .h5 files. The locations of these stations are included in **Table 2** and in **Figure 42.**

Table 2: Locations of the STWAVE stations of interest.

FID	STWAVE Station Number	Longitude	Latitude	X coordinate (FIPS 3900, m)	Y coordinate (FIPS 3900, m)
180	818	-79.9704403374	32.8008995586	706020.67461907	107784.70541894
329	690	-79.9571539187	32.7818545302	707286.24719876	105685.37069113
598	425	-79.9327239290	32.8034032096	709549.91419664	108098.15509148
699	324	-79.9224045444	32.7832094125	710539.63362569	105868.99971441
976	47	-79.9291526483	32.7687239390	709924.19701562	104256.20754288

Figure 42: Location of the five stations of interest within the study area. The points are labeled with their correlating STWAVE station number.

RESULTS: Two types of figures were generated for each storm, a plot of maximum significant wave height and time series plots at the five stations of interest. For discussion, the plots and results from Storm 27 are highlighted in **Figure 43** - **Figure 45**, with the plots for the other four high-frequency storms events provided in **Appendix B** and **Appendix C**. Although not evident in Storm 27, some of the other storms, such as Storm 4 in the Appendix, saw 'blocking' in the maximum significant wave height solutions. This behavior is a well documented behavior of the parallel STWAVE model, as noted in Massey et al. (2011). Additionally, the way these maximum wave height plots are computed tend to exaggerate the differences between grid partitions – e.g., the maximum significant wave height is the largest wave that occurred during the entire simulation at that particular grid cell and could occur at different times during the simulation.

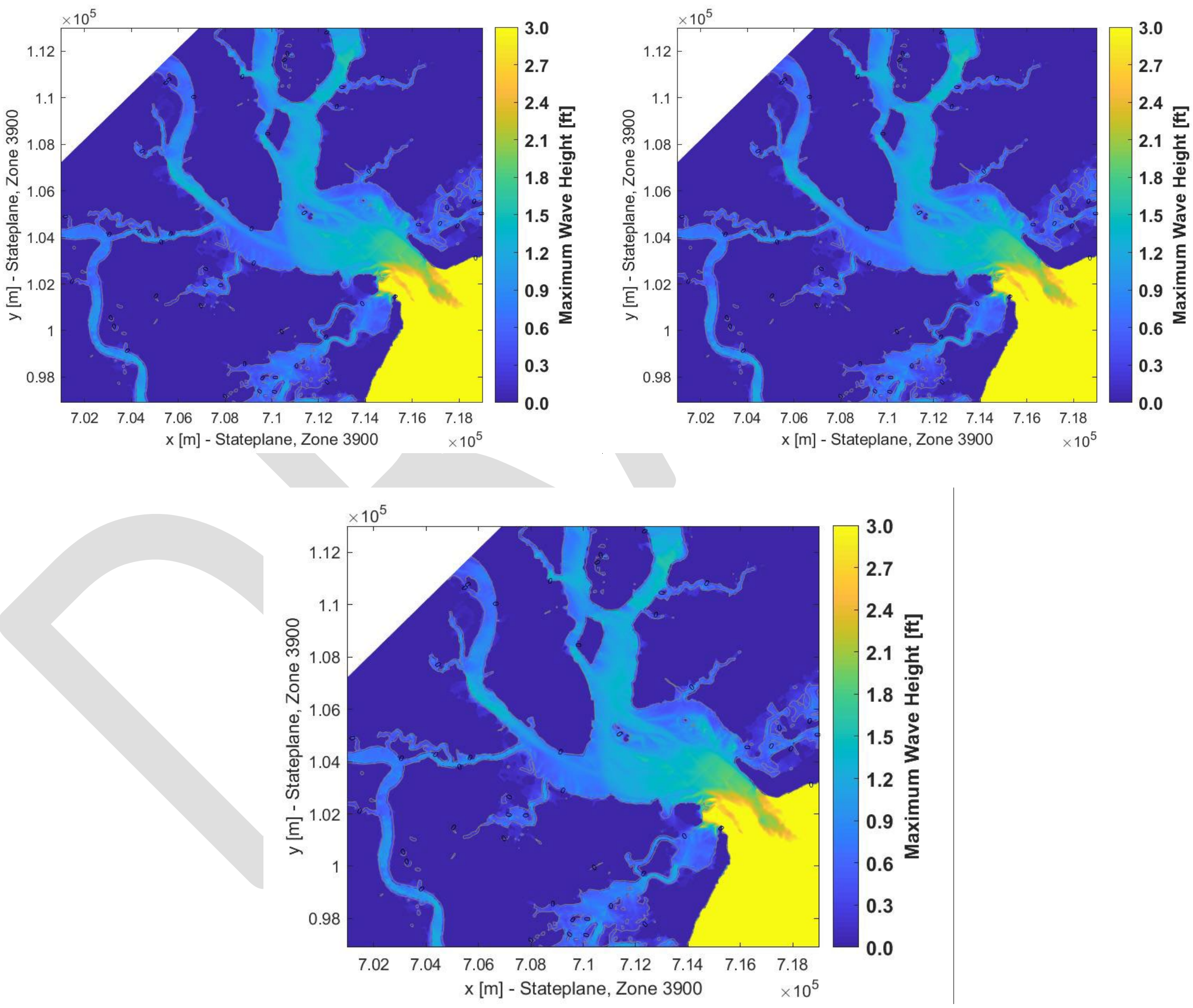

Figure 43: Maximum wave height (ft) of Storm 27 for EC (upper left), FWOP (upper right), WP01 (bottom).

For all three conditions, the maximum wave heights were greater than 3 ft at the opening of the inlet and along the open coast boundaries. These waves reduce to heights between 1 to 2 feet as they propagate into Charleston Harbor. Little difference is observed between the EC and the FWOP condition, which is expected as elevating the battery wall (the only change included in FWOP compared to EC) would have little impact on the waves around the peninsula. However, the addition of the breakwater in WP01 results in slight changes in the maximum wave height field in the immediate vicinity of the breakwater. Wave heights are slightly increased offshore of breakwater but are smaller lee of the breakwater. The difference in maximum wave height between WP01 and EC is shown in **Figure 44** for the area of interest. Smaller wave heights immediately behind the breakwater are expected. Additionally, differences in wave height in the vicinity of the breakwater are also anticipated given the breakwater changed the local bathymetry of the domain. For Storm 27, the increase in the maximum wave height due to the breakwater is marginal, on the order of 0.2 ft or 2-3 inches.

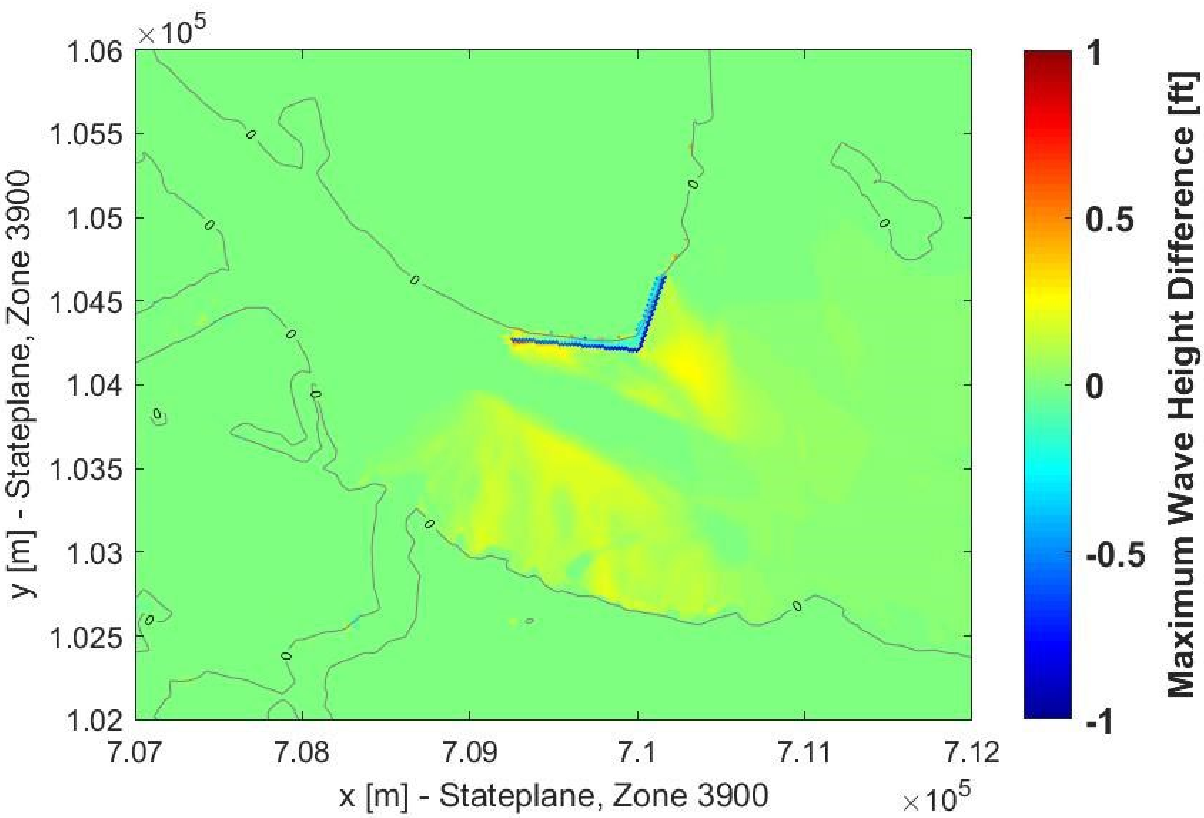

Figure 44: Difference between WP01 and EC [WP01-EC] maximum wave heights in feet, where warm colors indicated increases in wave height and cool colors indicate decreases in wave height. The breakwater is shown in dark blue.

The time series plots, such as that shown in **Figure 45**, show the growth and decay of wave heights through the simulated storm event at each of the selected locations. Here, we see that the highest waves grow to a little over 1 ft tall at the points of interest, with the highest wave heights occurring at STWAVE stations 818, 690, and 324. These stations are located at the upper western side, the central western side, and the central eastern side of the peninsula, respectively. Whereas there is little difference between the wave heights experienced at these sites between the EC and the FWOP, the wave heights at Station 47 are slightly smaller for WP01 than EC due to the presence of the breakwater. The wave heights at the other stations, which are further from the breakwater, are similar between EC and WP01.

It is important to note that the maximum wave height plots are useful for assessing overall conditions during a storm, but not necessarily for assessing wave climate as a factor of time, such as determining greatest reductions in wave height by condition. For instance, at station 47, the maximum wave heights for EC and FWOP, which would be included in the maximum wave height plot, occur during the beginning of the simulation while the maximum wave height for WP01 occurs more towards the end of the simulation. These differences in the time of occurrence for the maximum wave height are shown in **Figure 45** in the black boxes. The greatest difference in wave heights between the EC/FWOP and WP01 (0.6 ft. EC/FWOP to 0.1 ft. WP01) occurred at the time circled in red. As shown, the time of this greatest difference does not correlate with any of the occurrences of the maximum wave heights and, therefore, will not be evident in the maximum wave height plots. In summary, the greatest reduction in wave height between conditions may not be captured in a direct comparison between the maximum wave heights because these values are independent of time. Rather, a comparison of the time series will better show the effect of each condition on the wave climate through the entire simulation.

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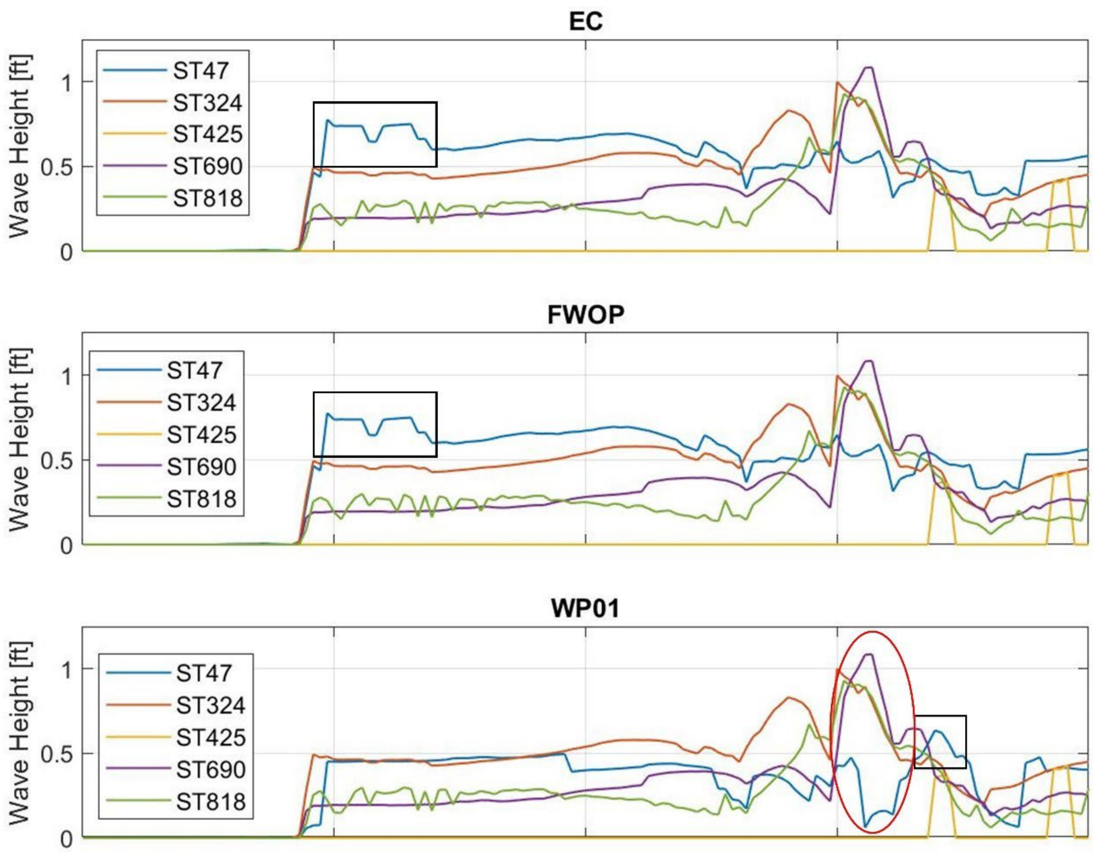

Figure 45. Time series of significant wave height at the five selected stations for Storm 27.

G2CRM: G2CRM is a Probabilistic Life Cycle Analysis model that applies a suite of storm surges and wave conditions to a study area to quantify the damages expected during a 50-year life cycle. Capturing the whole range of possible life cycles requires multiple iterations of the 50-year life cycle with randomly selected storms determined by the likelihood of the storms. G2CRM uses the selected storm surges to calculate the damages/life loss suffered by each modeled area. These damages can be calculated with and without a protective element, and allow stakeholders to determine the value added by constructing different scenarios.

Save Point Selection: For the purposes of this study, Charleston Peninsula was divided into five separate modeled areas. The modeled area is the region over which G2CRM aggregates damages and reports overall statistics. These regions are divided for either geographic or political reasons. Each modeled area is driven by an individual set of modeled data containing the surge data and wave data for each storm in the study. Multiple save stations account for variations in the hydraulic conditions in the areas being inundated by the storms. In this case the save stations selected were the stations immediately adjacent to the shoreline of the modeled area with the lowest lying area. Each save station was exported from the detailed modeling performed with

ADCIRC and converted into an .h5 format. Each .h5 file contains the metadata required to identify the time step and modeled area associated with the storm suite. The .h5 data is supplemented with an input file containing the details needed for G2CRM to apply the correct interpretation of the files given. The excel file contains the columns listed in **Table 3**.

Table 3: Inputs for H5 Metadata Excel File.

Column	Description
IsStwaveFormat	Format of the H5 file (1= STWAVE, 0 = ADCIRC)
ModeledStormSetTextId	Unique (within the representation) text identifier for the storm set
ModeledStormSetDescription	Description of the modeled storm set
StormDatumToAssetInventoryDatumConversion	Vertical conversion from storm datum to asset inventory datum (conversion from MSL to NAVD88)
MllwToStormDatumConversion	Vertical conversion from MLLW to MSL storm datum
UseWaveDataAsIs	1 = Use wave data from H5, 0 = auto-generate wave data in model

H5 METADATA AND .h5 FILE: The ModeledStormSetTextId is the unique identifier for a particular storm set. It needs to match the names internal to G2CRM. ModeledStormSetDescription describes the particular data set and serves as a tool to indicate any changes to the modeling. The StormDatumToAssetInventoryDatumConversion and MllwtoStormDatumConversion columns are critical in that they allow for G2CRM to convert from the various datums to NAVD88. This conversion is necessitated by the use of MSL in the ADCIRC modeling used as forcing for G2CRM. UseWaveDataAsIs indicates when G2CRM should search for wave data within the .h5 file. If the wave data is being used (UseWaveDataAsIs of 1), G2CRM will add 0.7 of the given wave height to the overall inundation level, a practice consistent with methods (FEMA 2005). If UseWaveDataAsIs is zero G2CRM will calculate the maximum possible depth limited wave height for each storm condition and then apply 0.7 of the resulting wave height to the total inundation level. If no waves are desired the user can input zero wave heights into the .h5 file and indicate that the zero wave heights are to be used as input. For the Charleston study, the waves from Stwave were applied so the IsStwaveFormat and UseWaveDataAsIs columns were both set to 1. The ModeledStormSetTextID was set from 1 to 5 depending on the modeled area for each set of data. The H5 Metadata excel file also requires the storms to keep tab. This tab is an artifact of the intended application of Coastal Hazards System (https://chs.erdc.dren.mil) (Nadal-Caraballo et al. 2020) data for G2CRM studies. The Coastal Hazards System provides a full suite of synthetic storms that capture the full probability space for locations across the United States' coastline. The storm names on the tab match the names in the .h5 file. For the specific case in Charleston, the datum conversions from the storm to inventory was set to 0.22 ft (to convert between NAVD88 and MSL), and the MLLW to storm datum conversion was -2.92 ft (to convert between MLLW and MSL). The .h5 files contain the water elevations and wave heights for all selected storms. For simplicity, the save points were renamed to 1-5 as shown in the **Table 4**.

Table 3: Save point correspondence.

Save Point Correspondence	
Original	ERDC
976	1

699	2
598	3
329	4
180	5

SEASONS: The seasons excel file for G2CRM delineates the given seasons for each type of storm given. Typically, this would include both tropical and extratropical storms. In the Charleston case the storms were all synthetic tropical storms. The seasons are usually broken down by months, and a probability of storm occurrence for each season/storm type condition is provided by the user. The maximum storms per season gives an upper extreme for the number of storms in a simulation. For Charleston, a value of 100 was applied.

STORMS: The storms excel spreadsheet lays out the relative probability of each storm, the time of year possible for each storm, and the basis year for the storm. The recurrence probability for each tropical cyclone was calculated by URS (2012), and the basis year gives the year that was used to calculate water levels. Sea level change is calculated based on the three Corps curves, and the net change from the storm basis year and the year of storm occurrence in an iteration is added to the water levels from the .h5 file. The Charleston study applies the present day water levels.

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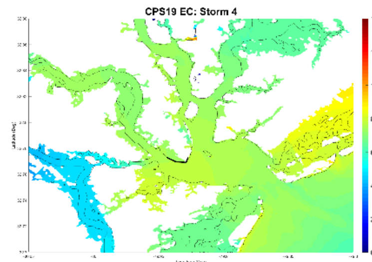
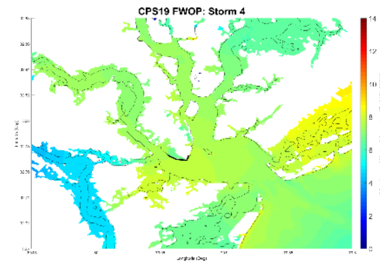
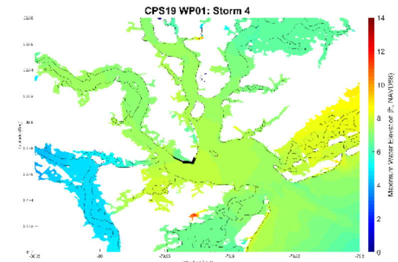
APPENDIX A: Maximum Water Elevation Plots



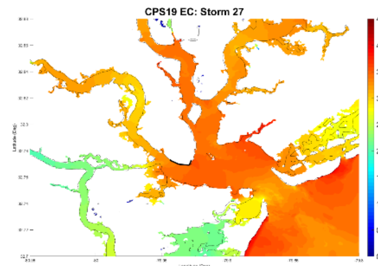
EC Storm 4



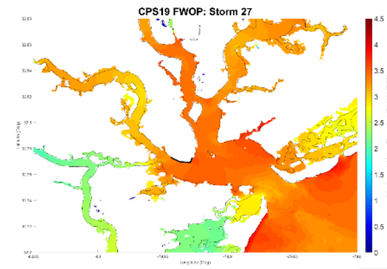
FWOP Storm 4



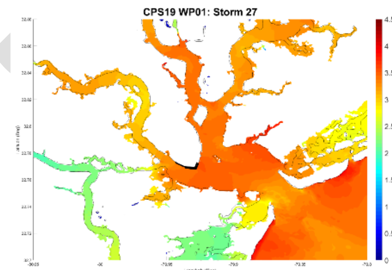
WP01 Storm 4



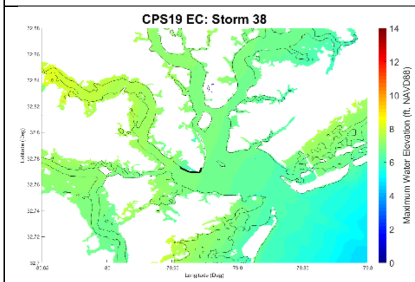
EC Storm 27



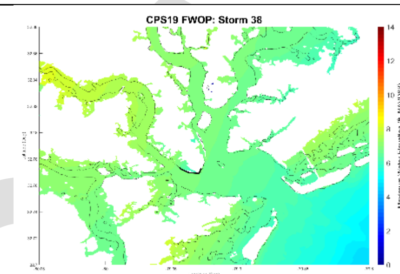
FWOP Storm 27



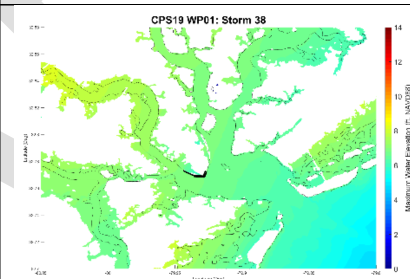
WP01 Storm 27



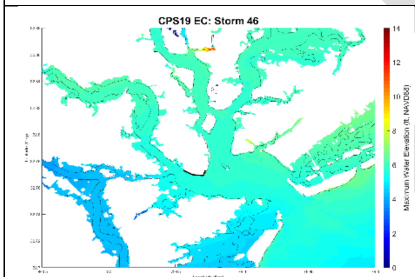
EC Storm 38



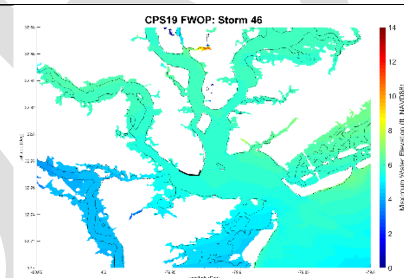
FWOP Storm 38



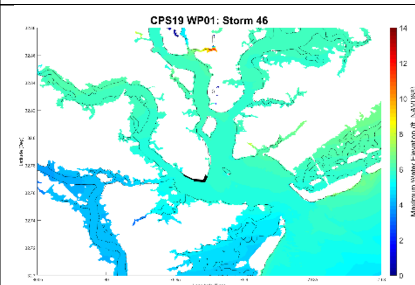
WP01 Storm 38



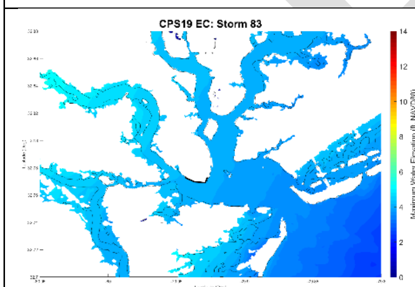
EC Storm 46



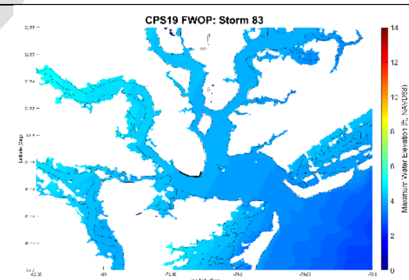
FWOP Storm 46



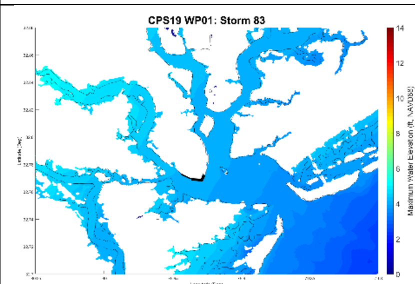
WP01 Storm 46



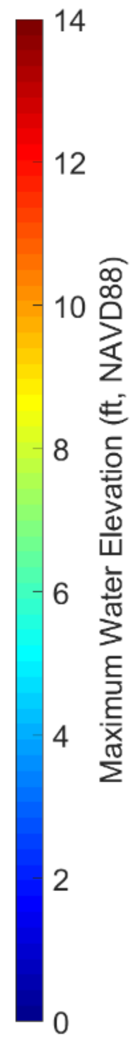
EC Storm 83



FWOP Storm 83



WP01 Storm 83



APPENDIX B: Maximum Wave Plots

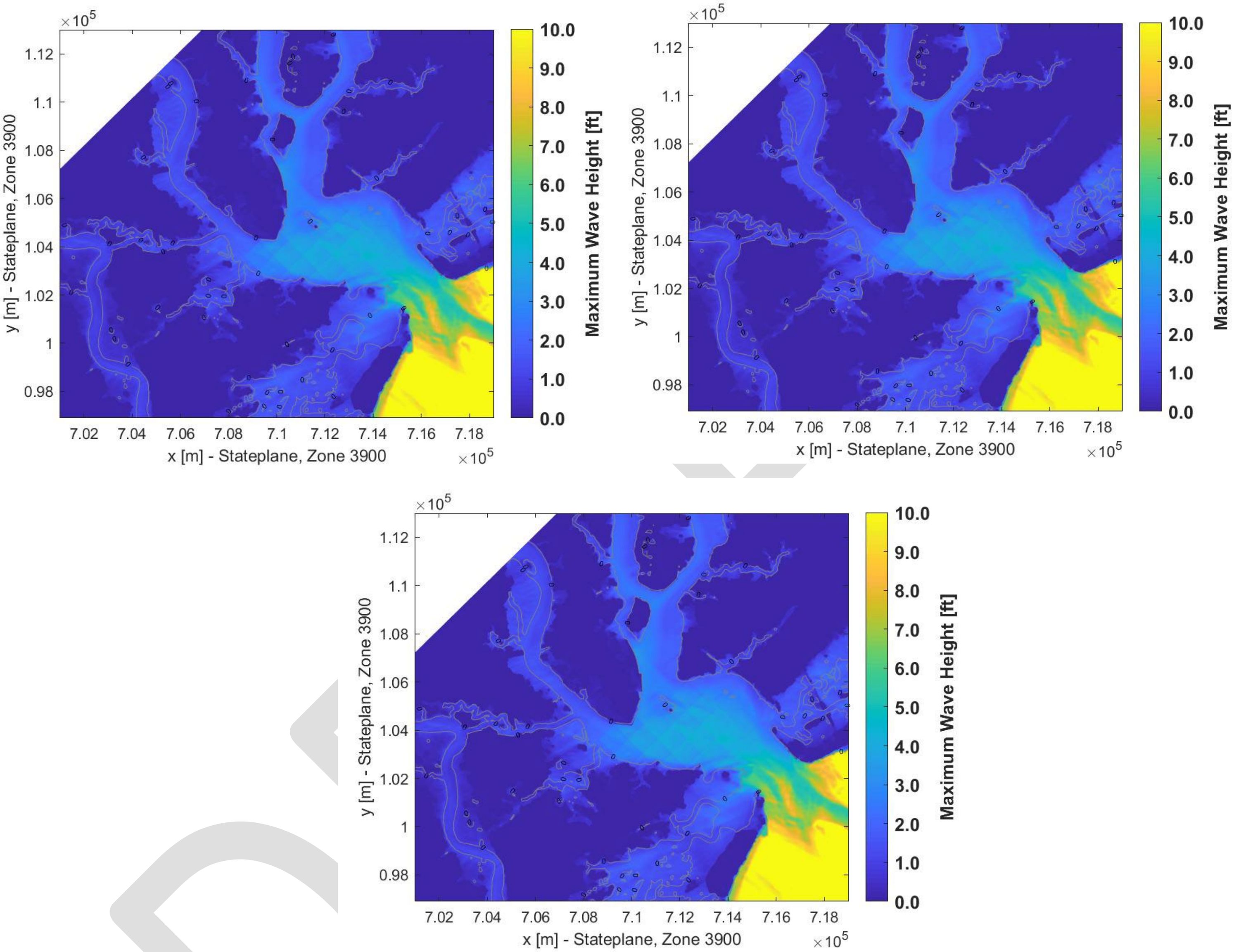


Figure 46. EC (upper left), FWOP (upper right), and WP01 (bottom) for Storm 4.

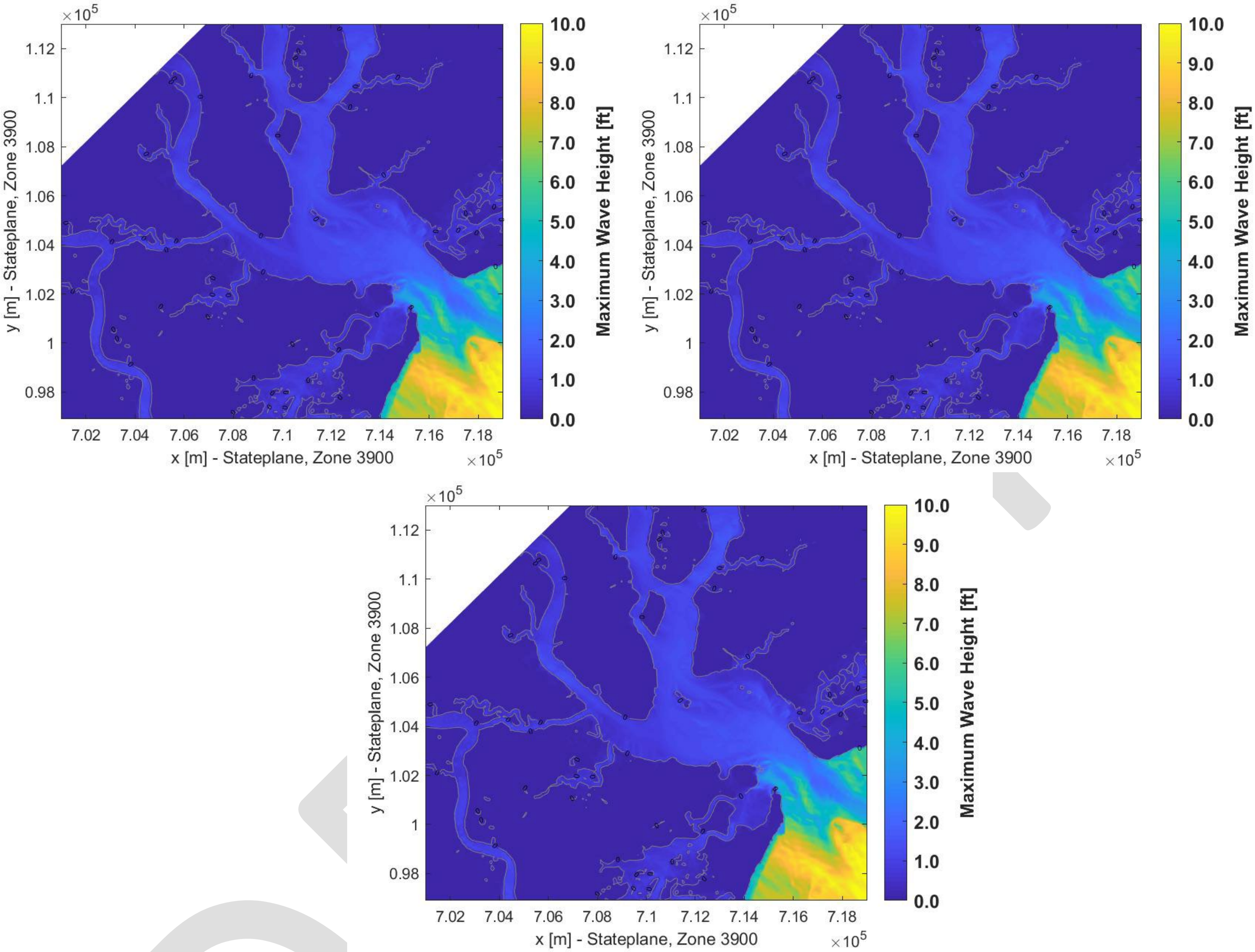


Figure 47. EC (upper left), FWOP (upper right), and WP01 (bottom) for Storm 27.

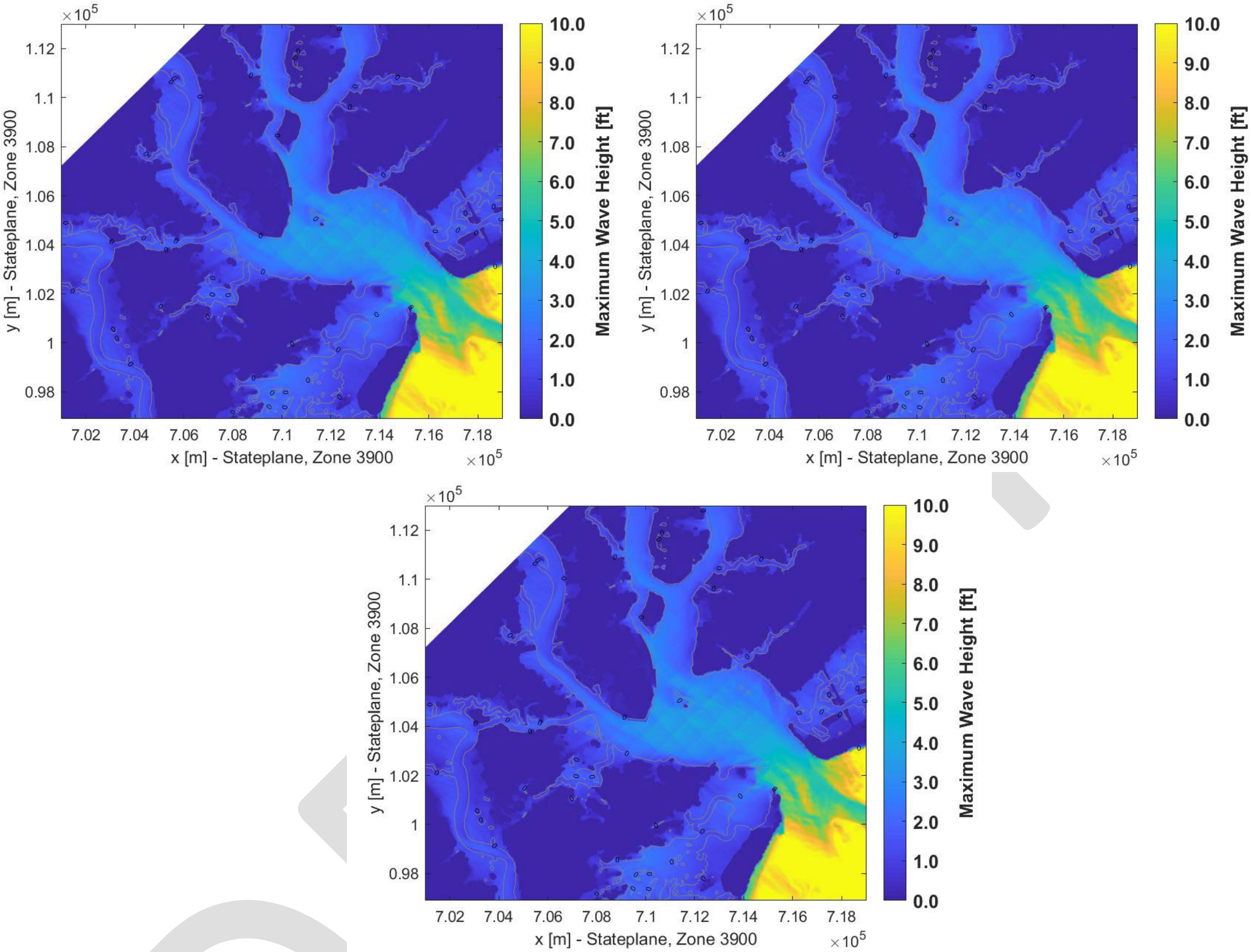


Figure 48. EC (upper left), FWOP (upper right), and WP01 (bottom) for Storm 38.

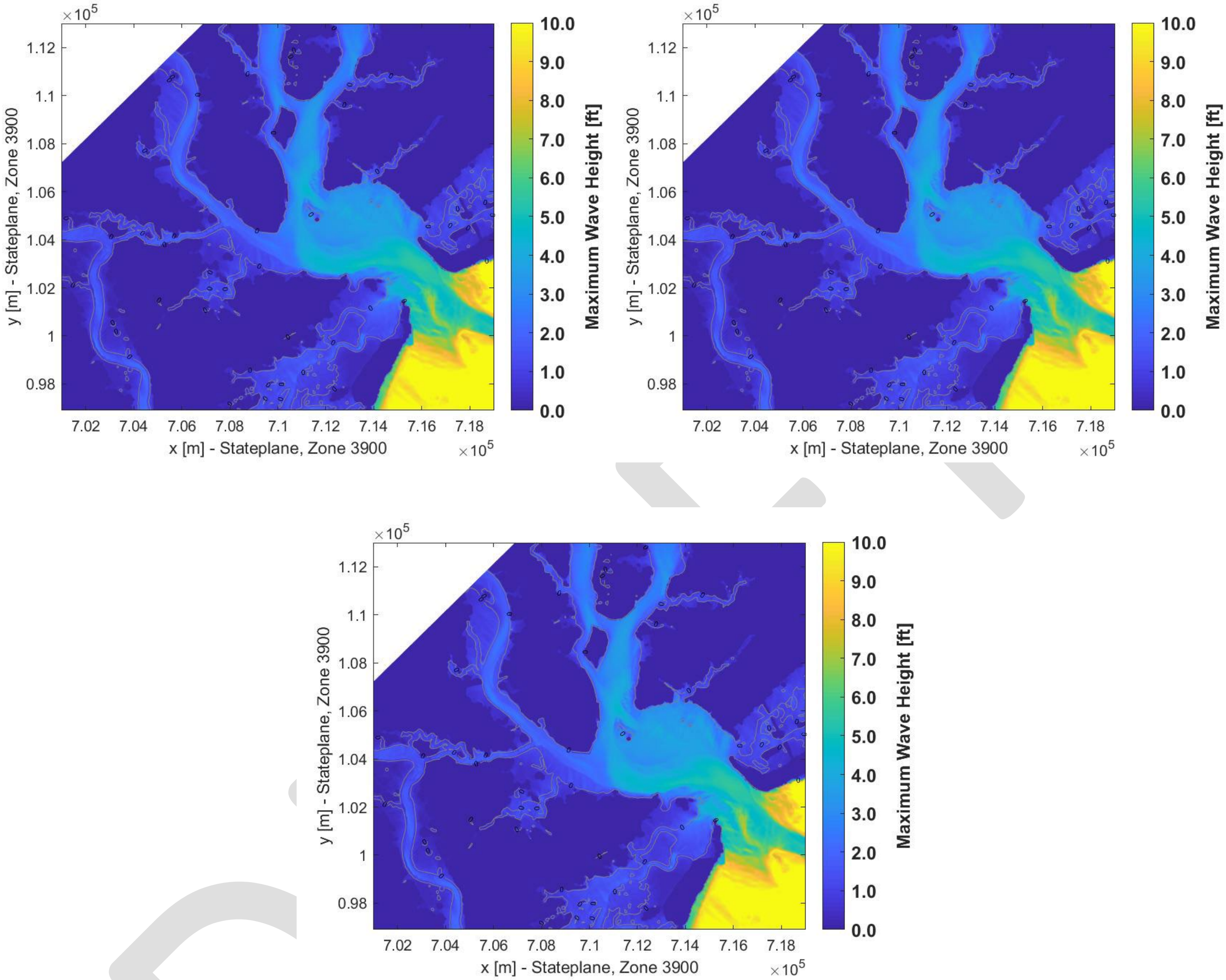


Figure 49. EC (upper left), FWOP (upper right), and WP01 (bottom) for Storm 46.

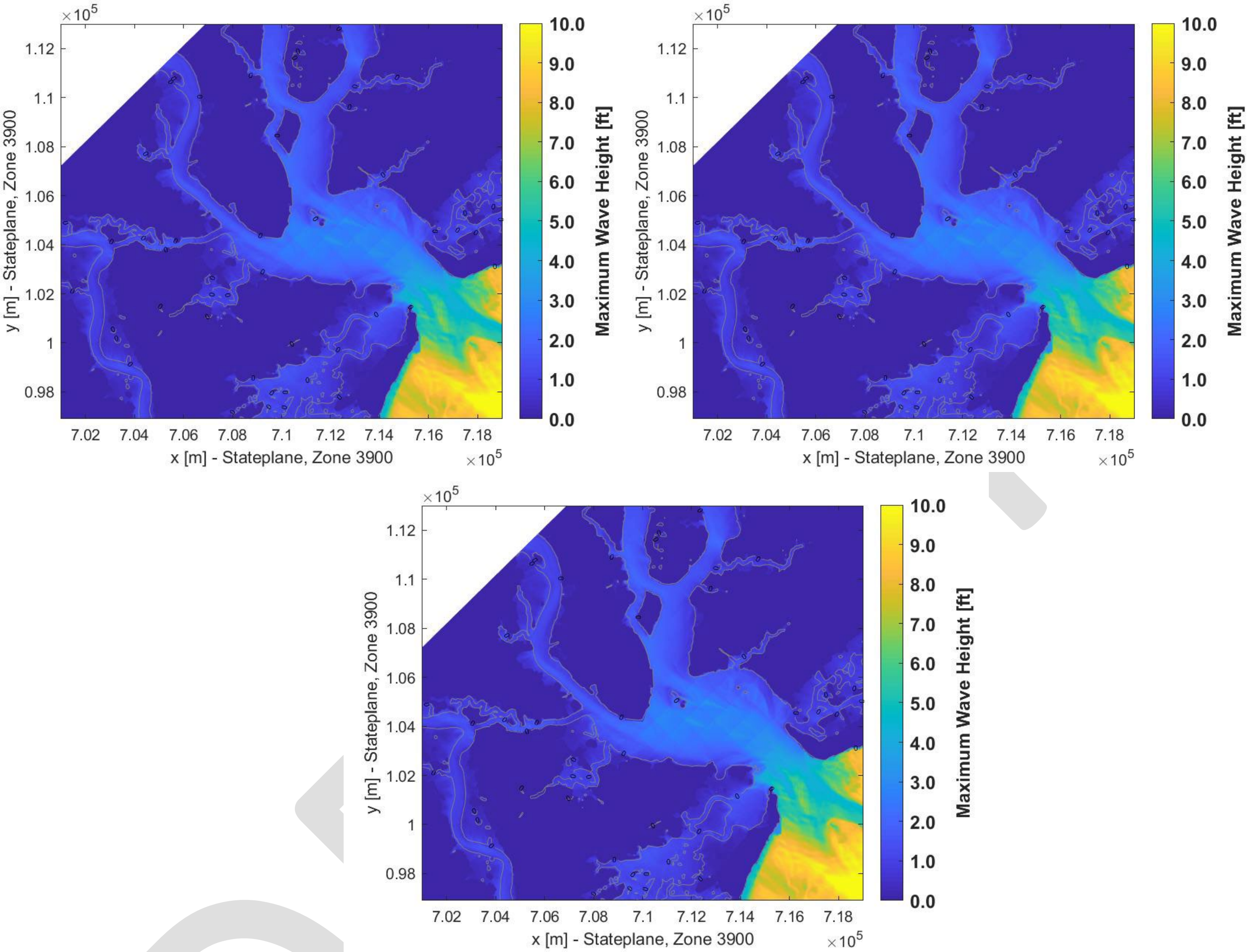
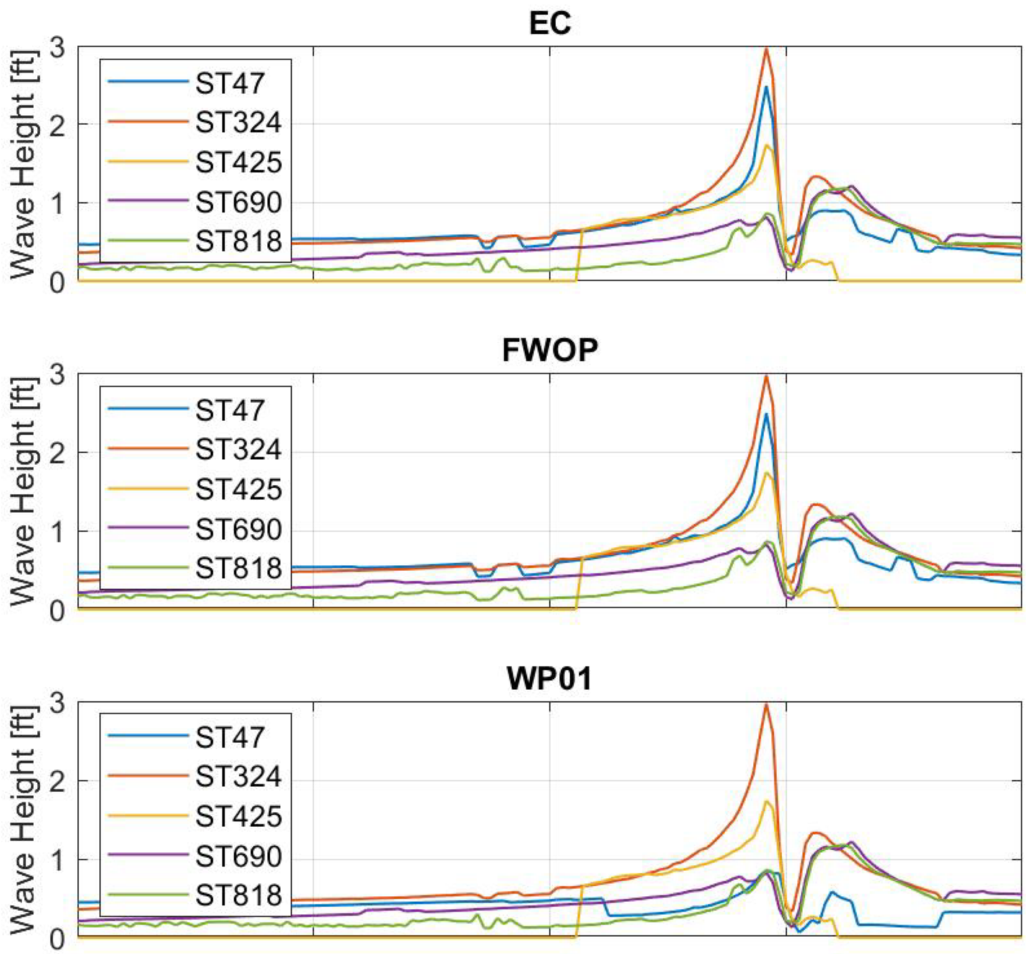
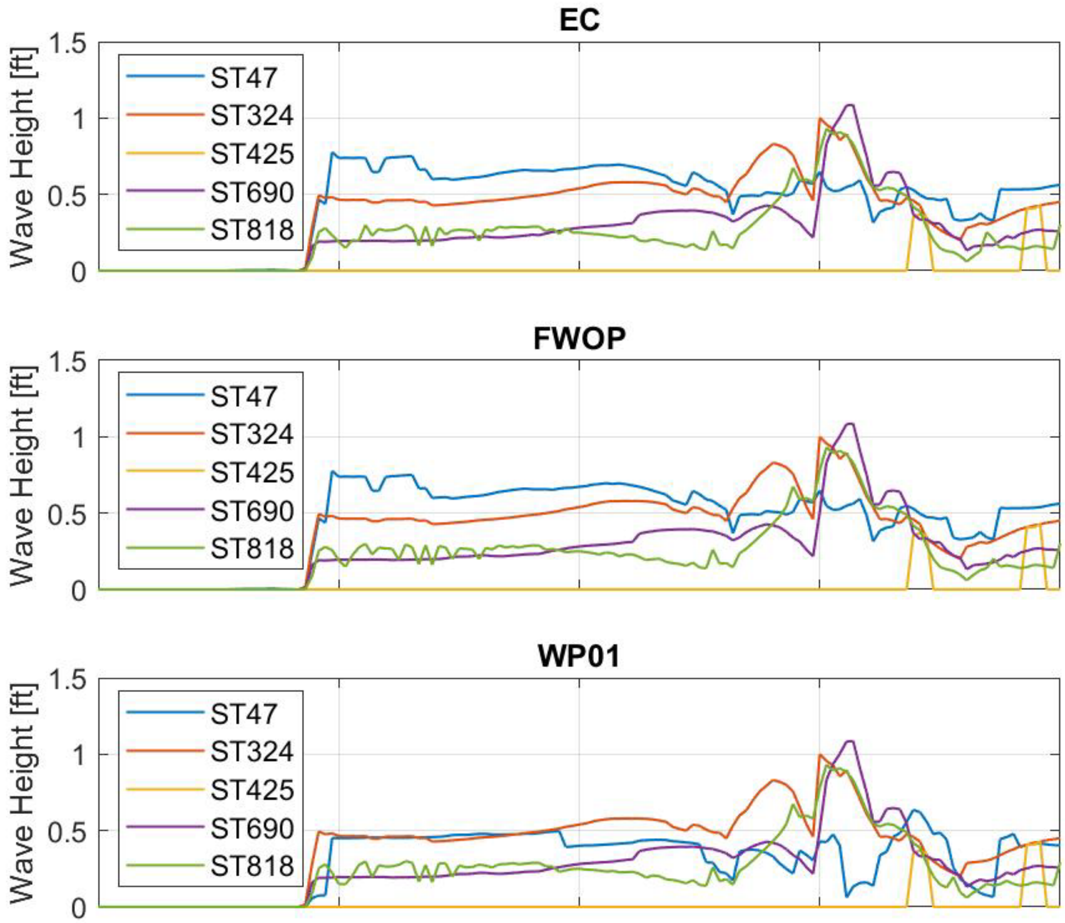
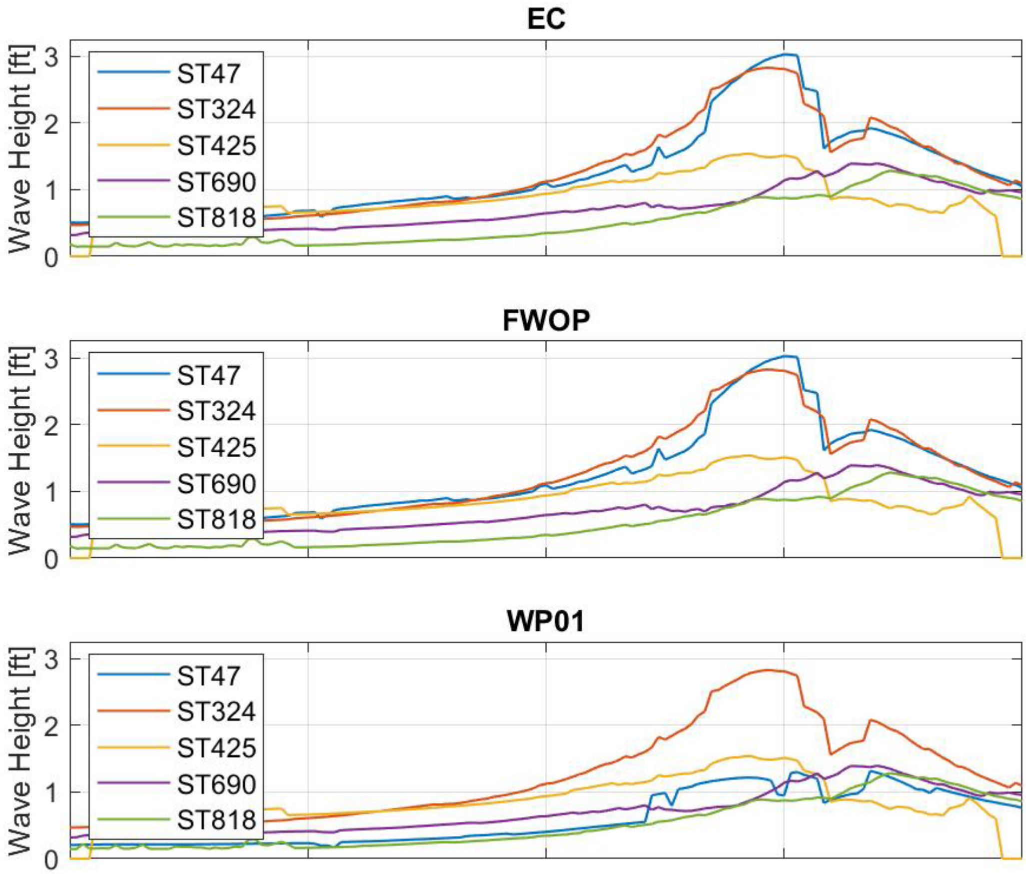


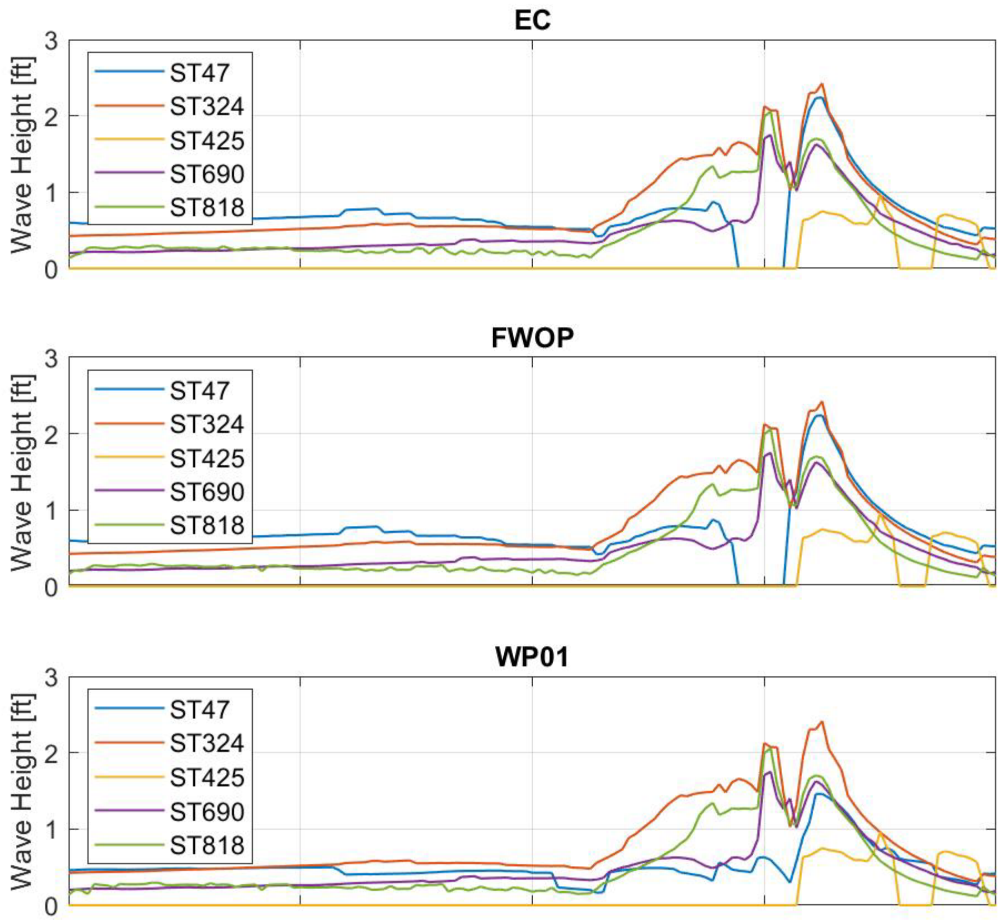
Figure 50. EC (upper left), FWOP (upper right), and WP01 (bottom) for Storm 83.

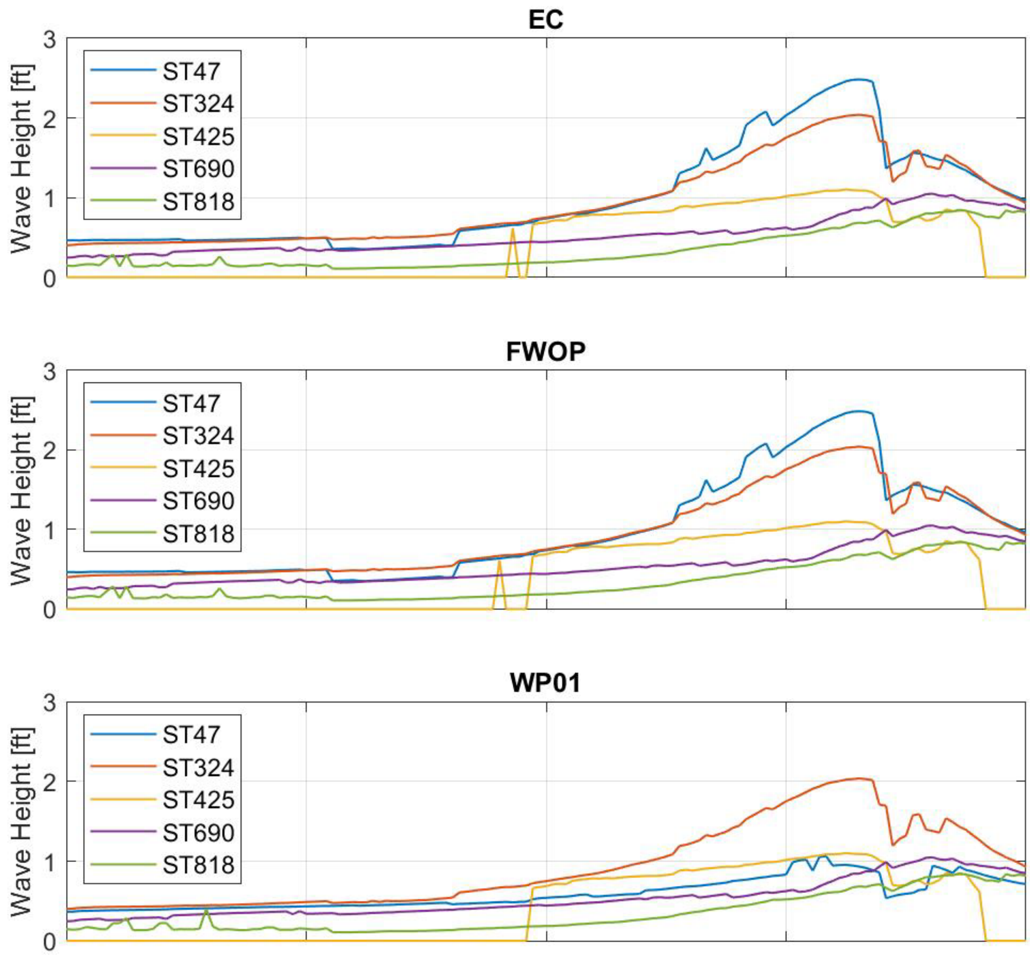
Appendix C: Time Series Plots



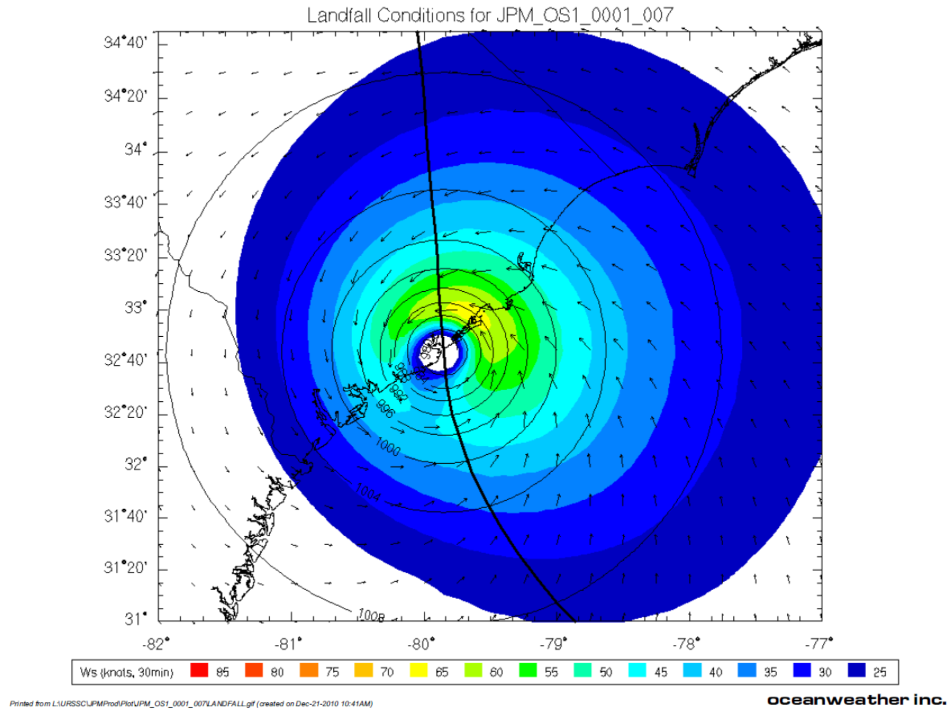

EC
Wave Height [ft]
1.5
1
0.5
0
ST47
ST324
ST425
ST690
ST818
FWOP
Wave Height [ft]
1.5
1
0.5
0
ST47
ST324
ST425
ST690
ST818
WP01
Wave Height [ft]
1.5
1
0.5
0
ST47
ST324
ST425
ST690
ST818

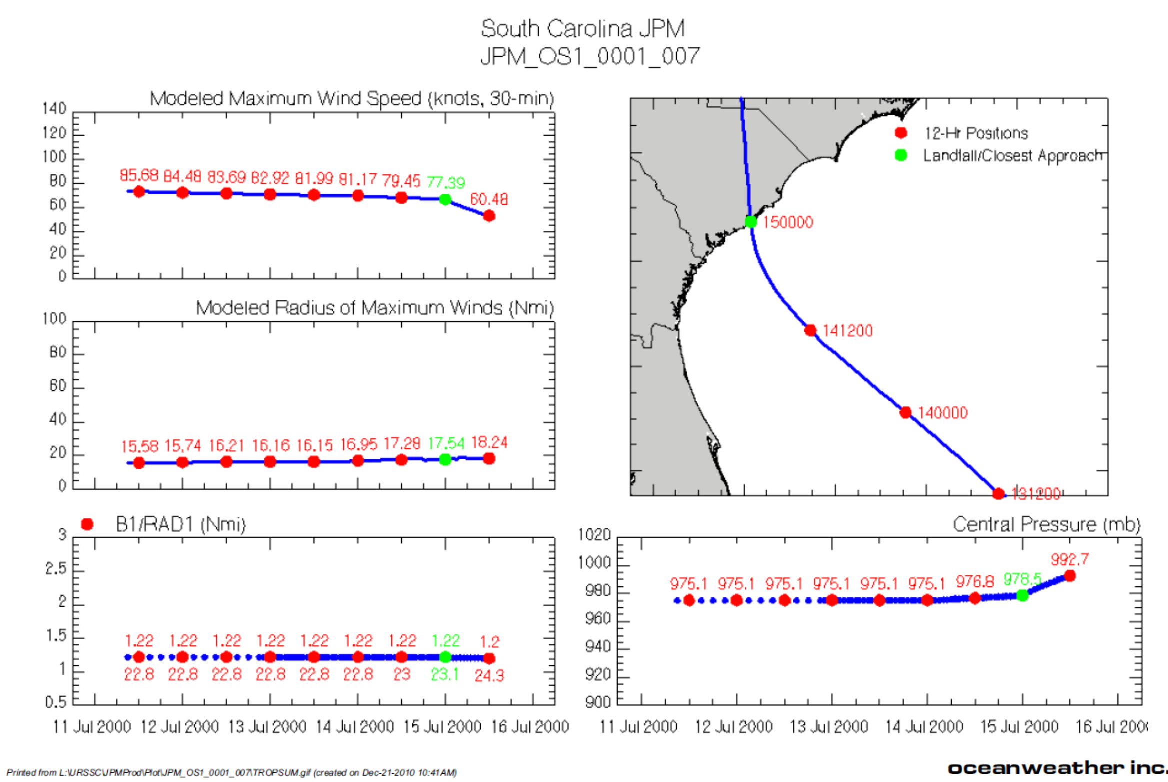

EC
FWOP
WP01
Wave Height [ft]
0
1
2
3
ST47
ST324
ST425
ST690
ST818

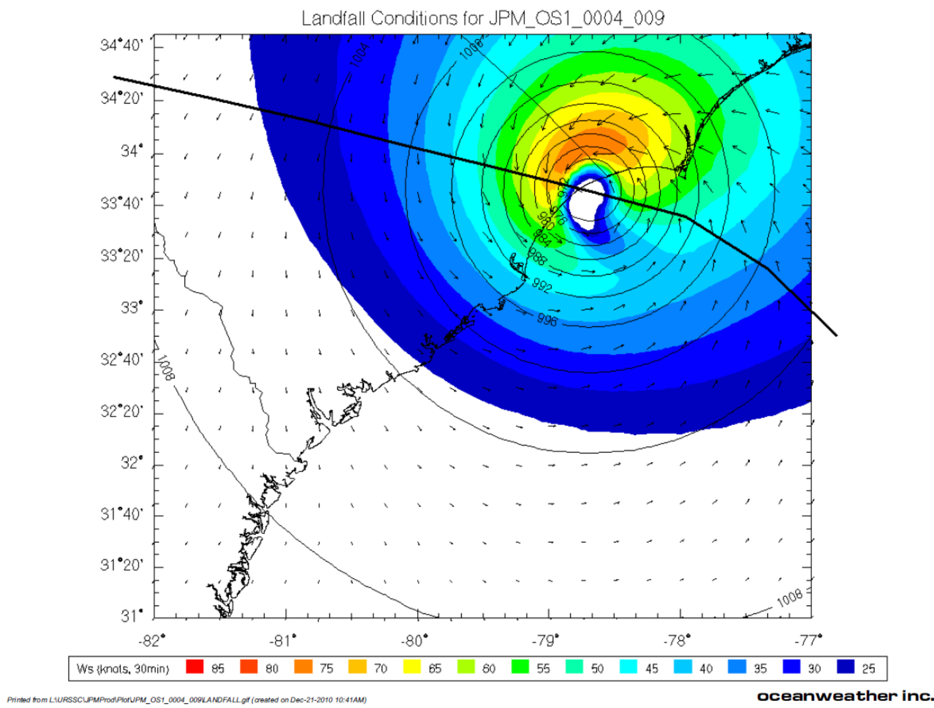

EC
FWOP
WP01
Wave Height [ft]
0
1
2
3
ST47
ST324
ST425
ST690
ST818

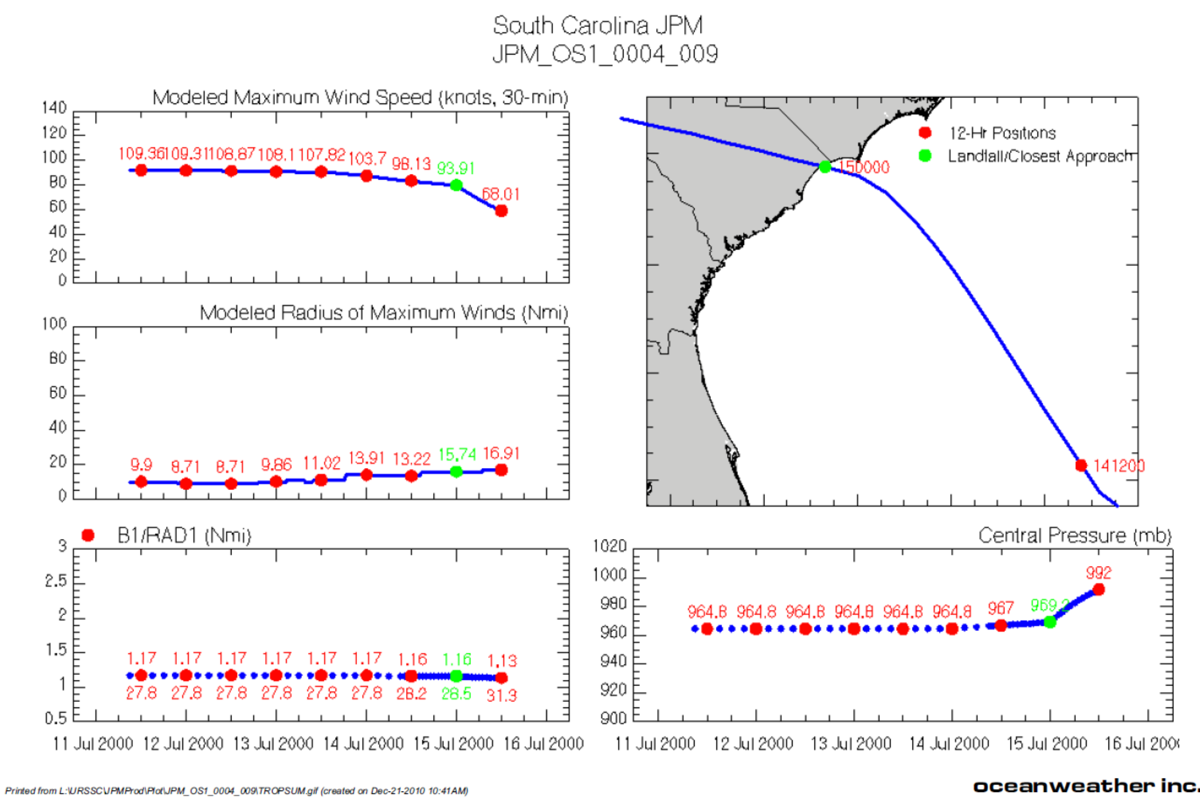

EC
Wave Height [ft]
0
1
2
3
ST47
ST324
ST425
ST690
ST818
FWOP
Wave Height [ft]
0
1
2
3
ST47
ST324
ST425
ST690
ST818
WP01
Wave Height [ft]
0
1
2
3
ST47
ST324
ST425
ST690
ST818

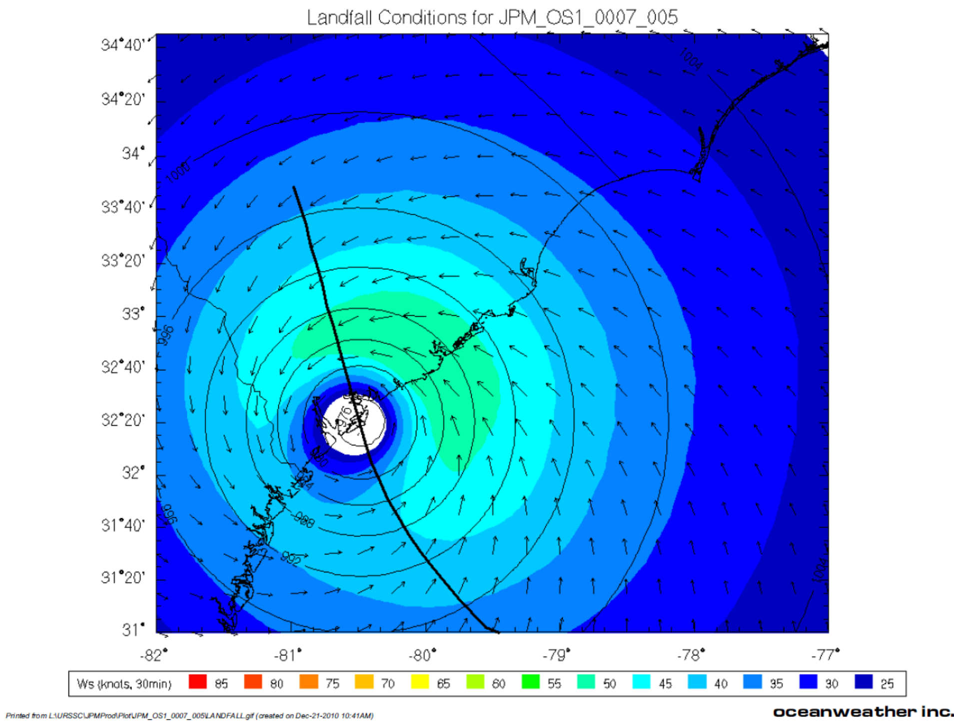
Appendix D: Characteristics of the Synthetic Tropical Storms - Appendix B (FEMA 2012)






Landfall Conditions for JPM_OS1_0004_009
34°40'
34°20'
34°
33°40'
33°20'
33°
32°40'
32°20'
32°
31°40'
31°20'
31°
-82°
-81°
-80°
-79°
-78°
-77°
1004
1008
992
996
Ws (knots, 30min) 85 80 75 70 65 60 55 50 45 40 35 30 25
oceanweather inc.


South Carolina JPM
JPM_OS1_0004_009
Modeled Maximum Wind Speed (knots, 30-min)
109.36 109.31 106.87 108.1 107.82 103.7 98.13 93.91 68.01
Modeled Radius of Maximum Winds (Nmi)
9.9 8.71 8.71 9.86 11.02 13.91 13.22 15.74 16.91
B1/RAD1 (Nmi)
1.17 1.17 1.17 1.17 1.17 1.17 1.16 1.16 1.13
27.8 27.8 27.8 27.8 27.8 27.8 28.2 28.5 31.3
12-Hr Positions
Landfall/Closest Approach
150000
141200
Central Pressure (mb)
964.8 964.8 964.8 964.8 964.8 964.8 967 969 992
11 Jul 2000 12 Jul 2000 13 Jul 2000 14 Jul 2000 15 Jul 2000 16 Jul 2000
oceanweather inc.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Ws (knots, 30min) 85 80 75 70 65 60 55 50 45 40 35 30 25
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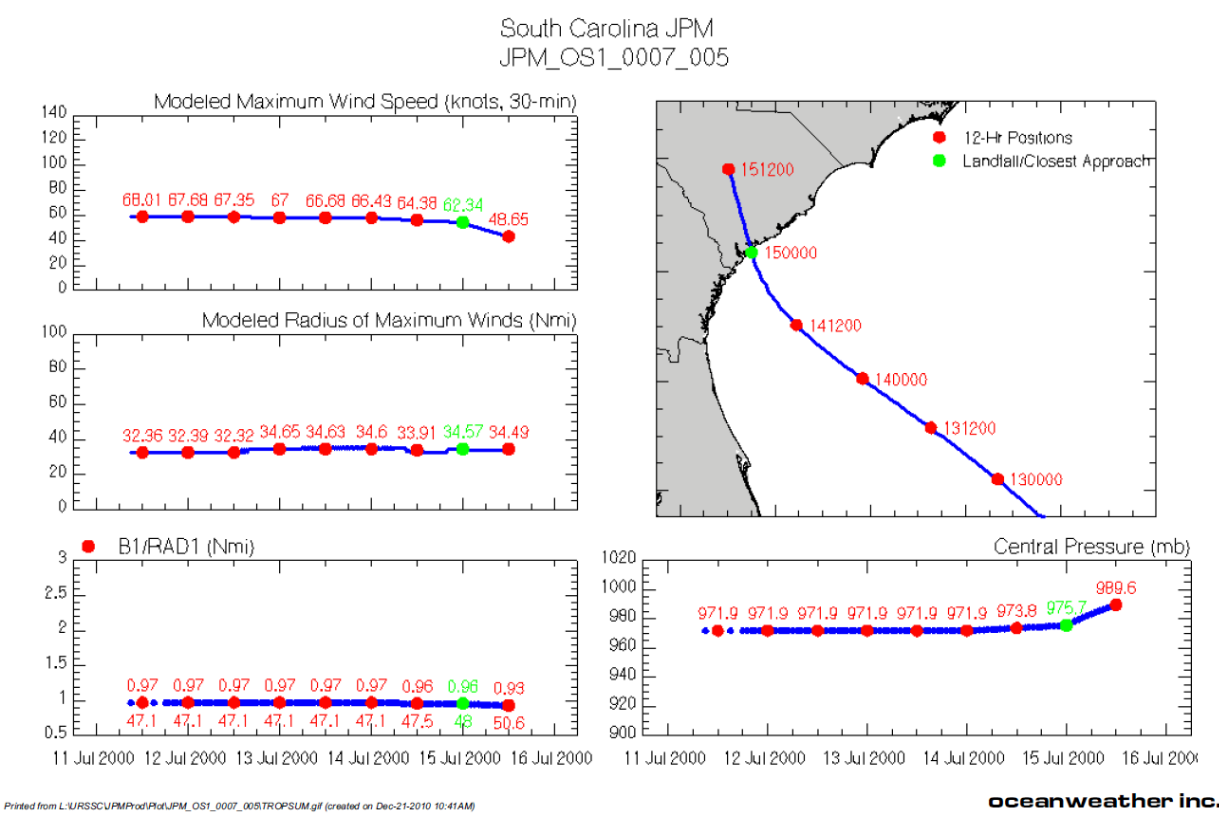
oceanweather inc.


South Carolina JPM
JPM_OS1_0007_005
Modeled Maximum Wind Speed (knots, 30-min)
68.01 67.69 67.35 67 66.68 66.43 64.38 62.34 48.65
Modeled Radius of Maximum Winds (Nmi)
32.36 32.39 32.32 34.65 34.63 34.6 33.91 34.57 34.49
B1/RAD1 (Nmi)
0.97 0.97 0.97 0.97 0.97 0.97 0.96 0.96 0.93
47.1 47.1 47.1 47.1 47.1 47.1 47.5 48 50.6
12-Hr Positions
Landfall/Closest Approach
151200
150000
141200
140000
131200
130000
Central Pressure (mb)
971.9 971.9 971.9 971.9 971.9 971.9 973.8 975.7 989.6
11 Jul 2000 12 Jul 2000 13 Jul 2000 14 Jul 2000 15 Jul 2000 16 Jul 2000
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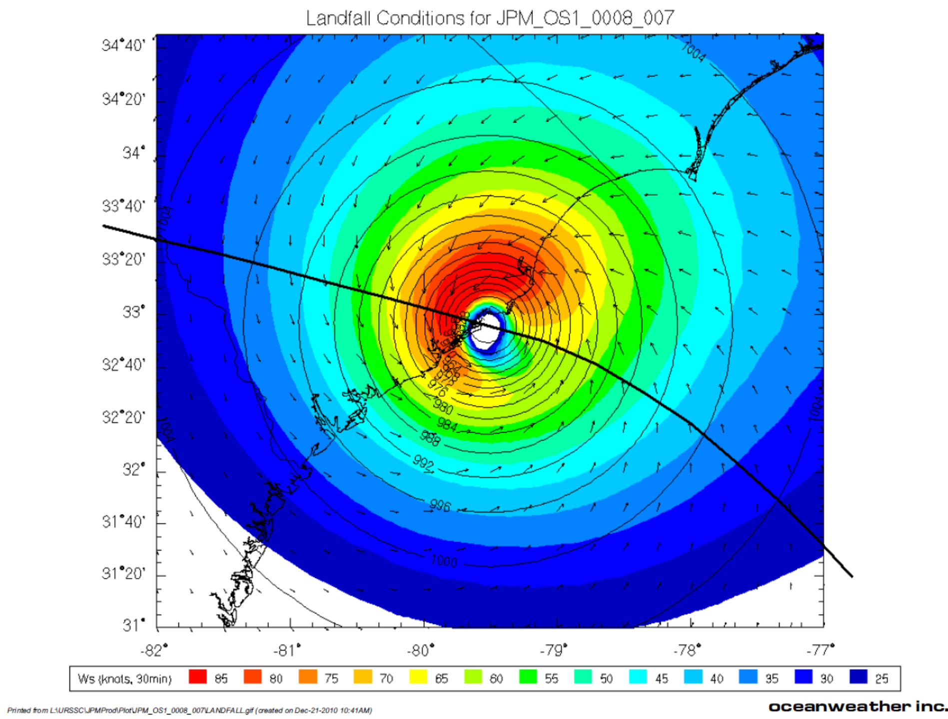
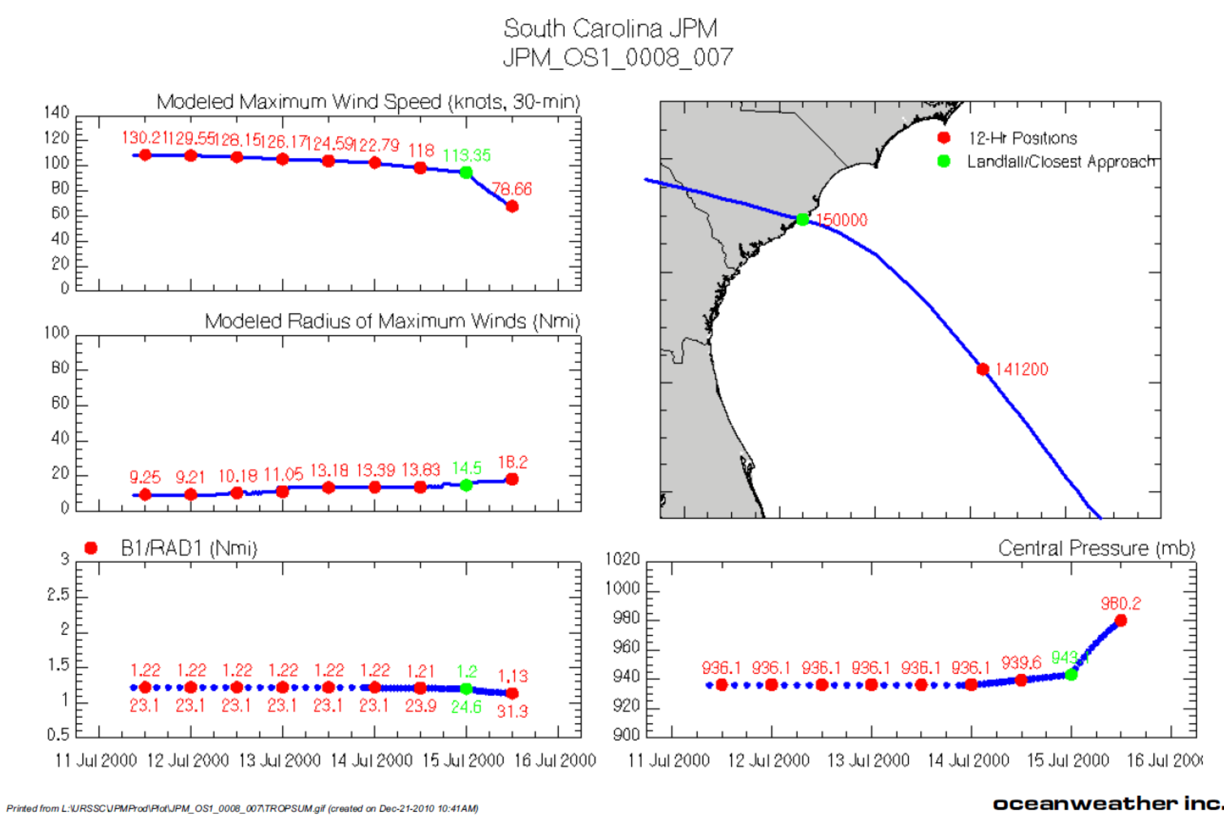
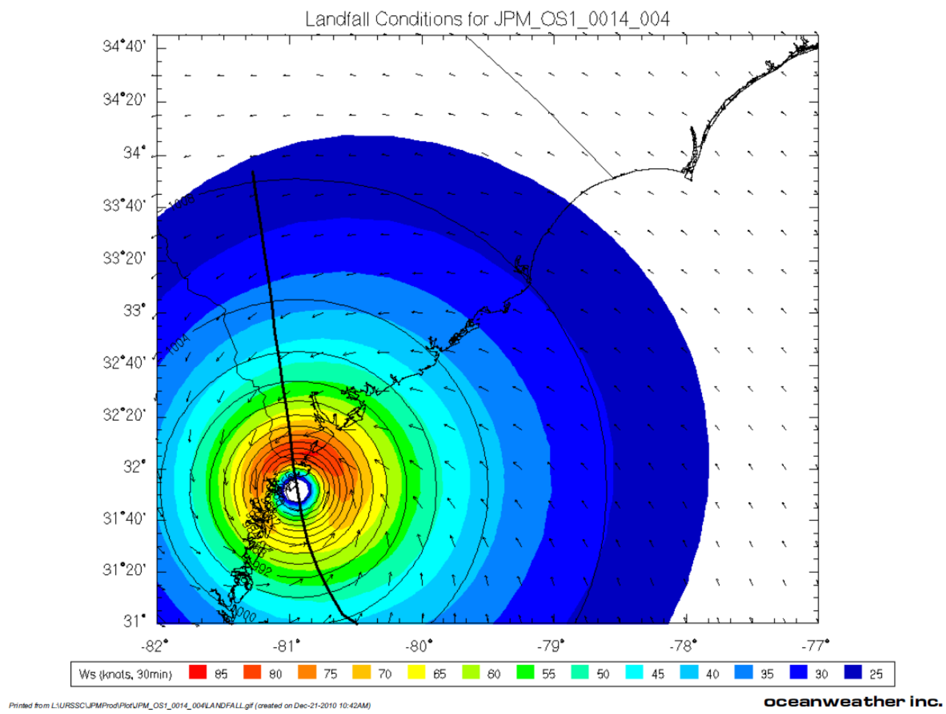
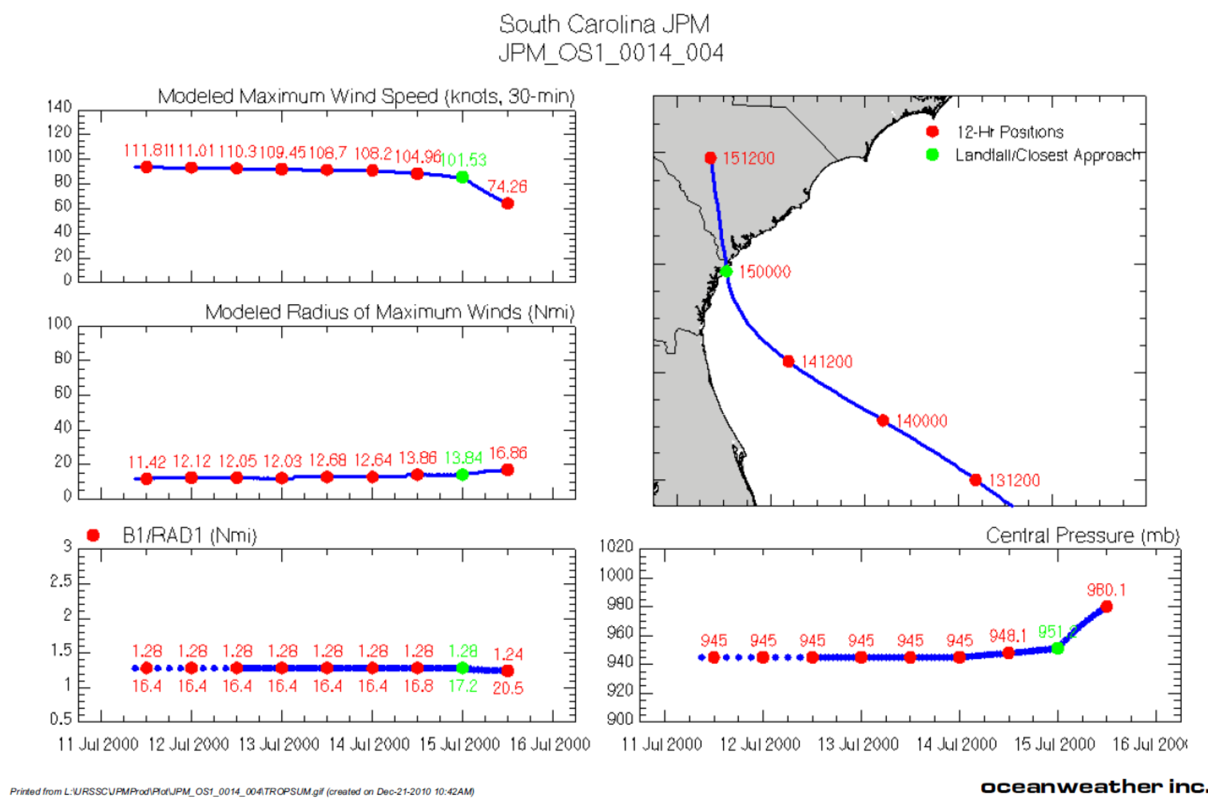
oceanweather inc.


Landfall Conditions for JPM_OS1_0008_007
Ws (knots, 30min) 85 80 75 70 65 60 55 50 45 40 35 30 25
oceanweather inc.


South Carolina JPM
JPM_OS1_0008_007
Modeled Maximum Wind Speed (knots, 30-min)
Modeled Radius of Maximum Winds (Nmi)
B1/RAD1 (Nmi)
Central Pressure (mb)
12-Hr Positions
Landfall/Closest Approach
150000
141200
11 Jul 2000 12 Jul 2000 13 Jul 2000 14 Jul 2000 15 Jul 2000 16 Jul 2000
oceanweather inc.


Landfall Conditions for JPM_OS1_0014_004
34°40'
34°20'
34°
33°40'
33°20'
33°
32°40'
32°20'
32°
31°40'
31°20'
31°
-82°
-81°
-80°
-79°
-78°
-77°
Ws (knots, 30min) 85 80 75 70 65 60 55 50 45 40 35 30 25
oceanweather inc.


South Carolina JPM
JPM_OS1_0014_004
Modeled Maximum Wind Speed (knots, 30-min)
111.81 111.01 110.31 109.45 108.7 108.2 104.96 101.53 74.26
Modeled Radius of Maximum Winds (Nmi)
11.42 12.12 12.05 12.03 12.68 12.64 13.86 13.84 16.86
12-Hr Positions
Landfall/Closest Approach
151200
150000
141200
140000
131200
B1/RAD1 (Nmi)
1.28 1.28 1.28 1.28 1.28 1.28 1.28 1.28 1.24
16.4 16.4 16.4 16.4 16.4 16.4 16.8 17.2 20.5
Central Pressure (mb)
945 945 945 945 945 945 948.1 951 980.1
11 Jul 2000 12 Jul 2000 13 Jul 2000 14 Jul 2000 15 Jul 2000 16 Jul 2000
oceanweather inc.

COST ENGINEERING SUB - APPENDIX

US Army Corps of Engineers®

Charleston District

CHARLESTON PENINSULA, SOUTH CAROLINA, A COASTAL STORM RISK MANAGEMENT STUDY

Charleston, South Carolina

COST ENGINEERING SUB -APPENDIX

February 2022

Summary of Scope of Work:

The Recommended Plan for the Charleston Peninsula Study includes the following civil works feature accounts:

- Account 01 - Land and Damages. For both structural and nonstructural features of work, real estate costs due to construction impacts are assessed and provided by SAS Real Estate Division and are shown in a tab called “01 - RE Total Costs” in the “Basis of Estimate – CPS Final Draft IPR_EIS FEB-2022” Excel file. The CSRA shows the current real estate costs have a 45% contingency on the acquisition costs and 25% on administration & relocation costs. The Real Estate BCE includes a 45% contingency on the acquisition of land and improvements, as determined by the Gross Appraisal. The 45% was projected by the Appraisal for incremental costs due to hidden and unforeseen aspects of property and improvements, potential development and zoning changes, negotiation latitude and eminent domain litigation. The Gross Appraisal was reviewed and certified by SAS Chief of Appraisers. The Real Estate BCE includes a separate 25% contingency on NFS and FED Administrative and Relocation costs. The Real Estate BCE was reviewed by Senior Realty Specialist and Chief of Acquisition.
- Account 02 - Relocations. It is anticipated that at a minimum, three types of utilities will be impacted: storm water, sanitary sewer, and potable water pipe lines. Quantity-takeoffs using GIS were done to determine number of pipe crossings and the distance from the crossing to the nearest possible connection. All pipes (old and new) are assume 8 inches diameter and are buried 6 ft deep. It is also assumed that 60% of the excavated materials is reusable and 50% of the remaining 40% is potential contaminated soil. Hauling and disposal of contaminated soils are included in this portion of the estimate. Quantities shown on the tabs with “02” prefix in the “Basis of Estimate – CPS Final Draft IPR_EIS FEB-2022” Excel file.
- Account 06 - Fish & Wildlife Facilities. Natural Resources Mitigation Costs are included for Wetland compensatory mitigation, Living shorelines (to mitigate impacts), and Environmental monitoring. Cost were assessed and provided by SAC Planning & Environmental Branch and are shown in a tab called “06 - Natural Resources Mitigation” in the “Basis of Estimate – CPS Final Draft IPR_EIS FEB-2022” Excel file.
- Account 11 - Levees and Floodwalls. The proposed project alignment shows Elements of Measures that include wall construction for multiple areas. As far as flood wall construction goes, T-wall, T-wall with walking path, and Combo wall were used. Combo wall design includes prestressed concrete piles and prestressed concrete sheet piles with precast pile cap. Length of wall and draft detail drawings for the walls were provided by Charleston District structural engineer. Preliminary quantity take-offs for the wall were conservatively estimated based on the detail drawings and the proposed lengths for wall, assuming averaged elevation of the project alignment will be the same as the constant desired height for the proposed wall. See all tabs with the prefix “11” in the “Basis of Estimate – CPS Final Draft IPR_EIS FEB-2022” Excel file. Street intersections in busy

parts of town where project alignment is crossing may need traffic control, which is estimated by assuming that new traffic signals, vehicle barriers, and flagmen may be needed. All costs in connection with construction work for floodwalls were estimated in MII using MII software, Cost Book Library 2016, latest national Davis Bacon wage rates and fuel prices.

- Account 13 - Pumping Plant. The NAO preliminary estimate for a permanent pump station in Freemanson, Norfolk VA with two 48" pumps (45,000 gpm) at 3rd quarter, 2014 price level was used to parametrically estimate pump stations for some of the areas in the project alignment. The size of concrete sump chamber, sluice gates, pipes, electrical, and other appropriate items are adjusted to accommodate the new number of pumps. Price level is escalated to current price using CWCCIS Escalation Calculation dated 30 Sep 2017 for account 13 from Q3 2014 to Q3 2018. See "Wall Alignment" tab in the "CPS 12' Optimized Basis of Estimate 01-JUL-2021" Excel file for quantities and estimated locations of pump stations. Estimate includes permanent sumps and outfalls for portable pumps as well as purchase of portable pumps.
- Account 18 - Cultural Resource Preservation. The cost for archaeological mitigation was conservatively estimated and provided by a Charleston District cultural resources PDT member. See "18 - Cultural Resources Mitigat" tab in the "Basis of Estimate – CPS Final Draft IPR_EIS FEB-2022" Excel file.
- Account 19 – Buildings, Grounds & Utilities. This account includes non-structural cost provided by PM based on the "19 - Non-structural" and "Storage Facility Cost" tabs in the "Basis of Estimate – CPS Final Draft IPR_EIS FEB-2022" Excel file.

References

EM 1110-2-1304, Civil Works Construction Cost Index Systems (CWCCIS), 30 September 2019

ER 1105-2-100, Planning Guidance Notebook, 20 November 2007

ER 1110-2-1150, Engineering and Design for Civil Works Projects, 31 August 1999

ER 1110-1-1300, Cost Engineering Policy and General Requirements, 26 March 1993

ER 1110-2-1302, Civil Works Cost Engineering, 30 June 2016

EP 1110-1-8 Volume 2, Construction Equipment Ownership and Operating Expense Schedule -Region II, 2016

PMBoK Guide, published by Project Management Institute (PMI)

Classification of Estimate and Expected Accuracy

Recommended Plan costs within this study have been prepared to an Estimate Class 3 Budget Authorization level of accuracy per AACE International Recommended Practice No. 56R-08 (see Table 1; also similar to ASTM E 2516-06, Standard Classification for Cost Estimate Classification System). These costs are intended for budget planning purposes.

	Primary Characteristic	*Secondary Characteristic*			
ESTIMATE CLASS	**LEVEL OF PROJECT DEFINITION** Expressed as % of complete definition	**END USE** Typical purpose of estimate	**METHODOLOGY** Typical estimating method	**EXPECTED ACCURACY RANGE** Typical variation in low and high ranges	**PREPARATION EFFORT** Typical degree of effort relative to least cost index of 1
Class 5	0% to 2%	Concept Screening	Capacity Factored, Parametric Models, Judgment or Analogy	L: -20% to -50% H: +30% to +100%	1
Class 4	1% to 15%	Study or Feasibility	Equipment Factored or Parametric Models	L: -15% to -30% H: +20% to +50%	2 to 4
Class 3	10% to 40%	Budget Authorization, or Control	Semi-Detailed Unit Costs with Assembly Level Line Items	L: -10% to -20% H: +10% to +30%	3 to 10
Class 2	30% to 70%	Control or Bid/ Tender	Detailed Unit Cost with Forced Detailed Take-Off	L: -5% to -15% H: +5% to +20%	4 to 20
Class 1	50% to 100%	Check Estimate or Bid/Tender	Detailed Unit Cost with Detailed Take-Off	L: -3% to -10% H: +3% to +15%	5 to 100

Table 1: AACE International Recommended Practice No. 56R-08[1]

Construction Cost Estimate:

The following methodology is used in the preparation of the cost estimate for Charleston Peninsula, South Carolina, A Coastal Storm Risk Management Study.

a. The estimate is in accordance with the guidance contained in ER 1110-2-1302, Civil Works Cost Engineering.

b. The estimate is presented in Civilworks Work Breakdown Structure.

c. The price level for the estimate is in 1st Quarter of FY2022.

d. Construction costs developed by General Engineering Section, Engineering Division,

[1] Source: www.aacei.org.

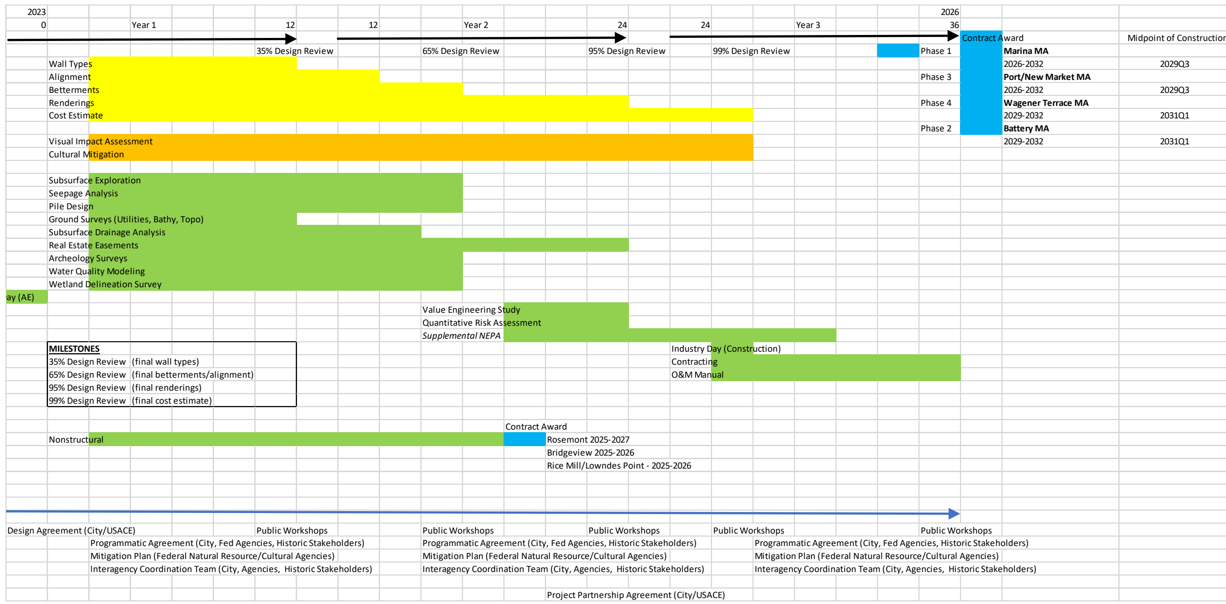
Charleston District are based on a concept design developed by SAC Engineering team. Unit costs are developed using the M-CACES Second Generation (MII) software containing the 2016 English Cost Book Library which was used as a starting point. Historical cost data from similar projects are used for parametric estimate, and vendor quotes were used for non-Cost Book data. The estimate is documented with notes to explain the assumed construction methods, crews, productivity, and other specific information. The intent is to provide or convey a "fair and reasonable" estimate that which depicts the local market conditions.

e. Labor costs are based on the 2021 National Labor Library. An overtime markup was added to all the t-wall and combo wall folders for working five ten hour days per week in lieu of the standard five eight hours days.

f. Bid competition: No contracting plan is done at this point. Bidding competition is assumed to be unrestricted since the overall work is typical to the area and the massive size of the project will likely draw multiple national level large size contractors to bid on the project. This assessment is reflected in the Cost and Schedule Risk Analysis.

g. Contract Acquisition Strategy: At this point of the feasibility study, the assumed acquisition strategy for design is based on full funding available to award one unrestricted large business architect-engineering contract. The contract will last between 3-4 years in duration and will design the entire Peninsula Coastal Storm Risk Management project. The design will be separated into 4 phases or separate plans/specs for construction awards. The assumed acquisition strategy for construction is based on full funding available to award multiple firm fixed price construction contracts via unrestricted best value solicitations. Due to the construction duration for phase 1 (Marina) and phase 3 (Port/New Market) it is assumed these contracts will be awarded concurrently after the design is completed and last approximately 6-7 years. The next two construction contracts for phase 2 (Battery) and phase 4 (Wagener Terrace) will be awarded between 2-3 years after award of the phase 1 (Marina) and phase 3 (Port/New Market) due to their anticipated construction duration. Contracts for pump houses/pumps/natural-nature based features will be included in their respective construction phases. In addition, non-structural contracts will be a stand-alone AE and construction contract and will begin concurrently with other design and phase 1 construction efforts.

h. Schedule:



i. Labor Shortages: It is assumed that there will be a normal labor market

j. Materials: Most material costs are from the Cost Book Library. Vendor quotes were used for non-Cost Book items. Assumptions include:

1. Rent materials will be part of the construction contract. No government furnished materials are assumed. Quoted delivery charge is used for hauling cost.
2. Materials will be rented from local nearest available sources.
3. Hauling: most hauling will be done by trucks. For trucking, it is assumed that the average speed is 30 mph factoring traffic hours in often congested major routes.

j. Equipment: Rates used are based from the latest USACE EP-1110-1-8, Region III. Adjustments are made for fuel and facility capital cost of money (FCCM). Judicious use of owned verses rental rates was considered based on typical contractor usage and local equipment availability. Full FCCM/Cost of Money rate is latest available; MII program takes EP recommended discount, no other adjustments have been made to the FCCM.

k. Fuels (gasoline, on and off-road diesel) were based on local market averages for on-road and off-road fuels in Charleston, SC. See Fuel Prices tab in the "Basis of Estimate – CPS Final Draft IPR_EIS FEB-2022.xlsx" file.

l. Major crew and productivity rates were developed and studied by senior USACE estimators familiar with the type of work. All of the work is typical to the Charleston District. The crews and productivities were checked by local SAC estimators, discussions with contractors and comparisons with historical cost data.

m. Most crew work hours are assumed to be 8 hrs. 5 days/week which is typical to the area. It is anticipated that no overtime is required for reasons such as time of year restriction because there is none.

n. Mobilization and demobilization: Contractor mobilization and demobilization are based on the assumption that most of the contractors will take about one 8 hrs day to mobilize and one 8 hrs day to demobilize. Mob and demob cost is estimated from 1% to 5% of total construction costs depending on the size of work.

o. Field Office Overhead: Typically, civil works project has field office overhead ranging from 10% to 15%. Since this project is a larger than the norm, 18% was used for Job Office Overhead. Overhead assumptions may include: Superintendent, office manager, pickups, periodic travel, costs, communications, temporary offices (contractor and government), office furniture, office supplies, computers and software, as-built drawings and minor designs, tool trailers, staging setup, camp and kitchen maintenance and utilities, utility service, toilets, safety equipment, security and fencing, small hand and power tools, project signs, traffic control, surveys, temp fuel tank station, generators, compressors, lighting, and minor miscellaneous.

p. Home Office Overhead: Due to large size of project a little less than typical percentage was used (5%) for HOOH. The rates are based upon estimating and negotiating experience, and consultation with local construction representatives.

q. Profit: Since the Construction Cost Estimate is currently in a budgetary phase, profit is typically included at 10% for Prime Contractor. However, due to large size of project and general expectation that there will be some competition, 8% profit was used for Prime and Prime's Profit on Sub's work. Sub-contractors' profit is mostly 8%.

r. Sales Tax: Only State sales tax was applied. No local sales tax was included in the estimate.

s. Bond: Bond is calculated at 0.64% using Bond Table in MII for the Prime contractor.

t. Contingency: Contingency is based the outcome of the Cost and Schedule Risk Analysis for the project.

u. Escalation: No escalation to midpoint of construction according to tentative construction start dates is included in the MII estimate and non-MII estimates provided by SAC. Escalation will only be included in the Total Project Cost Summary (TPCS) to avoid duplicates.

TOTAL PROJECT COST SUMMARY (TPCS)

****** TOTAL PROJECT COST SUMMARY ******

PROJECT: **Charleston Peninsula Study- Optimized Plan w/ Port Realignment**
PROJECT NO: P2 474899
LOCATION: Charleston, South Carolina

DISTRICT: SAC District **PREPARED: 2/7/2022**
POC: CHIEF, COST ENGINEERING, Lance Mahar

This Estimate reflects the scope and schedule in report; Charleston Peninsula Study

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)						TOTAL PROJECT COST (FULLY FUNDED)			
						Program Year (Budget EC): 2023 Effective Price Level Date: 1 OCT 22									
WBS NUMBER *A*	Civil Works Feature & Sub-Feature Description *B*	COST ($K) *C*	CNTG ($K) *D*	CNTG (%) *E*	TOTAL ($K) *F*	ESC (%) *G*	COST ($K) *H*	CNTG ($K) *I*	**TOTAL ($K)** *J*	Spent Thru: **1-Oct-21** ($K)	**TOTAL FIRST COST** ($K) *K*	INFLATED (%) *L*	COST ($K) *M*	CNTG ($K) *N*	FULL ($K) *O*
02	RELOCATIONS	$10,854	$3,907	36.0%	$14,762	3.2%	$11,198	$4,031	$15,230	$0	$15,230	22.4%	$13,711	$4,936	$18,646
06	FISH & WILDLIFE FACILITIES	$19,694	$7,090	36.0%	$26,784	3.2%	$20,319	$7,315	$27,633	$0	$27,633	25.4%	$25,472	$9,170	$34,642
11	LEVEES & FLOODWALLS	$459,905	$165,566	36.0%	$625,471	3.2%	$474,494	$170,818	$645,311	$0	$645,311	23.9%	$587,858	$211,629	$799,486
13	PUMPING PLANT	$34,289	$12,344	36.0%	$46,632	3.2%	$35,376	$12,735	$48,112	$0	$48,112	24.2%	$43,934	$15,816	$59,750
18	CULTURAL RESOURCE PRESERVATION	$62,589	$22,532	36.0%	$85,121	3.2%	$64,574	$23,247	$87,821	$0	$87,821	24.3%	$80,266	$28,896	$109,161
19	BUILDINGS, GROUNDS & UTILITIES	$39,291	$14,145	36.0%	$53,435	3.2%	$40,537	$14,593	$55,130	$0	$55,130	24.0%	$50,256	$18,092	$68,348
	CONSTRUCTION ESTIMATE TOTALS:	$626,622	$225,584		$852,206	3.2%	$646,498	$232,739	$879,237	$0	$879,237	24.0%	$801,496	$288,539	$1,090,035
01	LANDS AND DAMAGES	$87,327	$38,878	44.52042%	$126,206	3.2%	$90,097	$40,112	$130,209	$0	$130,209	4.7%	$94,347	$42,004	$136,351
30	PLANNING, ENGINEERING & DESIGN	$43,864	$15,791	36.0%	$59,654	3.1%	$45,223	$16,280	$61,504	$0	$61,504	4.7%	$47,348	$17,045	$64,393
31	CONSTRUCTION MANAGEMENT	$43,864	$15,791	36.0%	$59,654	2.5%	$44,960	$16,186	$61,146	$0	$61,146	19.2%	$53,579	$19,288	$72,867
	PROJECT COST TOTALS:	$801,676	$296,044	36.9%	$1,097,720		$826,779	$305,317	**$1,132,096**	$0	**$1,132,096**	20.5%	$996,770	$366,876	$1,363,646

MAHAR.LANCE.RICHARD.1268025770 Digitally signed by MAHAR.LANCE.RICHARD.1268025770 Date: 2022.02.17 11:29:59 -05'00'
CHIEF, COST ENGINEERING, Lance Mahar

WILSON.WESLEY.1293910789 Digitally signed by WILSON.WESLEY.1293910789 Date: 2022.02.17 11:52:03 -05'00'
PROJECT MANAGER, Wesley Wilson

Digitally signed by Ralph J. Werthmann Date: 2022.02.17 13:23:47 -05'00'
CHIEF, REAL ESTATE, Ralph J. Werthmann

CHIEF, PLANNING, Nancy Parrish

CHIEF, ENGINEERING, Carole Works

CHIEF, OPERATIONS, Joe Moran

CHIEF, CONSTRUCTION, David Dodds

CHIEF, CONTRACTING, Charlenne Figgins

CHIEF, PM-PB, Brian Williams

CHIEF, DPM, Lisa Metheney

ESTIMATED TOTAL PROJECT COST: $1,363,646

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Charleston Peninsula Study- Optimized Plan w/ Port Realignment
LOCATION: Charleston, South Carolina
This Estimate reflects the scope and schedule in report; Charleston Peninsula Study

DISTRICT: SAC District
POC: CHIEF, COST ENGINEERING, Lance Mahar
PREPARED: 2/7/2022

Civil Works Work Breakdown Structure		**ESTIMATED COST**				**PROJECT FIRST COST (Constant Dollar Basis)**				**TOTAL PROJECT COST (FULLY FUNDED)**				
		Estimate Prepared:			**1-Oct-21**	Program Year (Budget EC):			2023					
		Effective Price Level:			1-Oct-21	Effective Price Level Date:			1 OCT 22					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	RISK BASED CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	INFLATED (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	Phase 1 - MA - Marina													
02	RELOCATIONS	$687	$247	36.0%	$934	3.2%	$708	$255	$963	2029Q3	22.0%	$864	$311	$1,175
06	FISH & WILDLIFE FACILITIES	$7,820	$2,815	36.0%	$10,635	3.2%	$8,068	$2,905	$10,973	2029Q3	22.0%	$9,842	$3,543	$13,385
11	LEVEES & FLOODWALLS	$116,482	$41,934	36.0%	$158,416	3.2%	$120,177	$43,264	$163,440	2029Q3	22.0%	$146,599	$52,776	$199,374
13	PUMPING PLANT	$10,000	$3,600	36.0%	$13,600	3.2%	$10,317	$3,714	$14,031	2029Q3	22.0%	$12,586	$4,531	$17,116
18	CULTURAL RESOURCE PRESERVATION	$17,398	$6,263	36.0%	$23,662	3.2%	$17,950	$6,462	$24,412	2029Q3	22.0%	$21,897	$7,883	$29,780
19	BUILDINGS, GROUNDS & UTILITIES	$645	$232	36.0%	$877	3.2%	$665	$240	$905	2029Q3	22.0%	$812	$292	$1,104
	CONSTRUCTION ESTIMATE TOTALS:	$153,032	$55,092	36.0%	**$208,124**		$157,886	$56,839	$214,725			$192,599	$69,336	$261,935
01	LANDS AND DAMAGES	$87,327	$38,878	44.52%	$126,206	3.2%	$90,097	$40,112	$130,209	2024Q3	4.7%	$94,347	$42,004	$136,351
30	PLANNING, ENGINEERING & DESIGN													
7.0%	of Construction Estimate Totals	$10,712	$3,856	36.0%	$14,569	3.1%	$11,044	$3,976	$15,020	2024Q3	4.7%	$11,563	$4,163	$15,726
31	CONSTRUCTION MANAGEMENT													
7.0%	of Construction Estimate Totals	$10,712	$3,856	36.0%	$14,569	2.5%	$10,980	$3,953	$14,933	2029Q3	17.6%	$12,907	$4,647	$17,554
	CONTRACT COST TOTALS:	$261,784	$101,683		$363,467		$270,008	$104,879	**$374,887**			$311,417	$120,149	**$431,565**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Charleston Peninsula Study- Optimized Plan w/ Port Realignment
LOCATION: Charleston, South Carolina
This Estimate reflects the scope and schedule in report; Charleston Peninsula Study

DISTRICT: SAC District
POC: CHIEF, COST ENGINEERING, Lance Mahar
PREPARED: 2/7/2022

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared:			**1-Oct-21**	Program Year (Budget EC):			2023					
		Effective Price Level:			1-Oct-21	Effective Price Level Date:			1 OCT 22					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	INFLATED (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	Phase 2 - MA - Battery													
02	RELOCATIONS	$144	$52	36.0%	$196	3.2%	$148	$53	$202	2031Q1	27.7%	$189	$68	$258
06	FISH & WILDLIFE FACILITIES	$164	$59	36.0%	$223	3.2%	$169	$61	$230	2031Q1	27.7%	$216	$78	$293
11	LEVEES & FLOODWALLS	$18,262	$6,574	36.0%	$24,837	3.2%	$18,842	$6,783	$25,625	2031Q1	27.7%	$24,054	$8,660	$32,714
13	PUMPING PLANT	$7,315	$2,634	36.0%	$9,949	3.2%	$7,547	$2,717	$10,265	2031Q1	27.7%	$9,635	$3,469	$13,104
18	CULTURAL RESOURCE PRESERVATION	$17,919	$6,451	36.0%	$24,370	3.2%	$18,488	$6,656	$25,143	2031Q1	27.7%	$23,602	$8,497	$32,099
	CONSTRUCTION ESTIMATE TOTALS:	$43,805	$15,770	36.0%	**$59,574**		$45,194	$16,270	$61,464			$57,697	$20,771	$78,468
30	PLANNING, ENGINEERING & DESIGN													
7.0%	of Construction Estimate Totals	$3,066	$1,104	36.0%	$4,170	3.1%	$3,161	$1,138	$4,299	2024Q3	4.7%	$3,310	$1,192	$4,501
31	CONSTRUCTION MANAGEMENT													
7.0%	of Construction Estimate Totals	$3,066	$1,104	36.0%	$4,170	2.5%	$3,143	$1,131	$4,274	2031Q1	22.2%	$3,840	$1,382	$5,222
	CONTRACT COST TOTALS:	$49,937	$17,977		$67,915		$51,499	$18,539	**$70,038**			$64,846	$23,345	**$88,191**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Charleston Peninsula Study- Optimized Plan w/ Port Realignment
LOCATION: Charleston, South Carolina
This Estimate reflects the scope and schedule in report; Charleston Peninsula Study

DISTRICT: SAC District
POC: CHIEF, COST ENGINEERING, Lance Mahar
PREPARED: 2/7/2022

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared:			**1-Oct-21**	Program Year (Budget EC):			2023					
		Effective Price Level:			1-Oct-21	Effective Price Level Date:			1 OCT 22					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	INFLATED (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	***B***	***C***	***D***	***E***	***F***	***G***	***H***	***I***	***J***	***P***	***L***	***M***	***N***	***O***
	Phase 3 - MA - Port													
02	RELOCATIONS	$5,631	$2,027	36.0%	$7,658	3.2%	$5,809	$2,091	$7,901	2029Q3	22.0%	$7,087	$2,551	$9,638
11	LEVEES & FLOODWALLS	$123,148	$44,333	36.0%	$167,481	3.2%	$127,054	$45,739	$172,793	2029Q3	22.0%	$154,988	$55,796	$210,783
13	PUMPING PLANT	$1,658	$597	36.0%	$2,254	3.2%	$1,710	$616	$2,326	2029Q3	22.0%	$2,086	$751	$2,837
18	CULTURAL RESOURCE PRESERVATION	$14,192	$5,109	36.0%	$19,301	3.2%	$14,642	$5,271	$19,913	2029Q3	22.0%	$17,861	$6,430	$24,291
	CONSTRUCTION ESTIMATE TOTALS:	$144,628	$52,066	36.0%	**$196,694**		$149,215	$53,718	$202,933			$182,022	$65,528	$247,550
30	PLANNING, ENGINEERING & DESIGN													
7.0%	of Construction Estimate Totals	$10,124	$3,645	36.0%	$13,769	3.1%	$10,438	$3,758	$14,195	2024Q3	4.7%	$10,928	$3,934	$14,862
31	CONSTRUCTION MANAGEMENT													
7.0%	of Construction Estimate Totals	$10,124	$3,645	36.0%	$13,769	2.5%	$10,377	$3,736	$14,113	2029Q3	17.6%	$12,199	$4,392	$16,590
	CONTRACT COST TOTALS:	$164,876	$59,355		**$224,231**		$170,030	$61,211	**$231,241**			$205,149	$73,854	**$279,002**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Charleston Peninsula Study- Optimized Plan w/ Port Realignment
LOCATION: Charleston, South Carolina
This Estimate reflects the scope and schedule in report; Charleston Peninsula Study

DISTRICT: SAC District
POC: CHIEF, COST ENGINEERING, Lance Mahar
PREPARED: 2/7/2022

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared: 1-Oct-21 Effective Price Level: 1-Oct-21				Program Year (Budget EC): 2023 Effective Price Level Date: 1 OCT 22				FULLY FUNDED PROJECT ESTIMATE				
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	INFLATED (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	Phase 3 - MA - Newmarket													
02	RELOCATIONS	$3,682	$1,326	36.0%	$5,008	3.2%	$3,799	$1,368	$5,167	2029Q3	22.0%	$4,634	$1,668	$6,302
06	FISH & WILDLIFE FACILITIES	$160	$58	36.0%	$218	3.2%	$165	$59	$225	2029Q3	22.0%	$201	$72	$274
11	LEVEES & FLOODWALLS	$65,933	$23,736	36.0%	$89,669	3.2%	$68,024	$24,489	$92,513	2029Q3	22.0%	$82,980	$29,873	$112,853
13	PUMPING PLANT	$9,315	$3,354	36.0%	$12,669	3.2%	$9,611	$3,460	$13,071	2029Q3	22.0%	$11,724	$4,221	$15,945
18	CULTURAL RESOURCE PRESERVATION	$5,492	$1,977	36.0%	$7,468	3.2%	$5,666	$2,040	$7,705	2029Q3	22.0%	$6,911	$2,488	$9,400
19	BUILDINGS, GROUNDS & UTILITIES	$24,877	$8,956	36.0%	$33,832	3.2%	$25,666	$9,240	$34,905	2029Q3	22.0%	$31,309	$11,271	$42,580
	CONSTRUCTION ESTIMATE TOTALS:	$109,459	$39,405	36.0%	**$148,864**		$112,931	$40,655	$153,586			$137,760	$49,593	$187,353
30	PLANNING, ENGINEERING & DESIGN													
7.0%	of Construction Estimate Totals	$7,662	$2,758	36.0%	$10,420	3.1%	$7,900	$2,844	$10,744	2024Q3	4.7%	$8,271	$2,977	$11,248
31	CONSTRUCTION MANAGEMENT													
7.0%	of Construction Estimate Totals	$7,662	$2,758	36.0%	$10,420	2.5%	$7,854	$2,827	$10,681	2029Q3	17.6%	$9,232	$3,324	$12,556
	CONTRACT COST TOTALS:	$124,783	$44,922		**$169,705**		$128,684	$46,326	**$175,010**			$155,263	$55,895	**$211,157**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Charleston Peninsula Study- Optimized Plan w/ Port Realignment
LOCATION: Charleston, South Carolina
This Estimate reflects the scope and schedule in report; Charleston Peninsula Study

DISTRICT: SAC District
POC: CHIEF, COST ENGINEERING, Lance Mahar
PREPARED: 2/7/2022

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared:			**1-Oct-21**	Program Year (Budget EC):			2023	FULLY FUNDED PROJECT ESTIMATE				
		Effective Price Level:			1-Oct-21	Effective Price Level Date:			1 OCT 22					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	INFLATED (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	Phase 4 - MA - Wagner Terrace													
02	RELOCATIONS	$711	$256	36.0%	$967	3.2%	$733	$264	$997	2031Q1	27.7%	$936	$337	$1,273
06	FISH & WILDLIFE FACILITIES	$11,550	$4,158	36.0%	$15,708	3.2%	$11,916	$4,290	$16,206	2031Q1	27.7%	$15,213	$5,477	$20,690
11	LEVEES & FLOODWALLS	$136,080	$48,989	36.0%	$185,069	3.2%	$140,397	$50,543	$190,940	2031Q1	27.7%	$179,237	$64,525	$243,762
13	PUMPING PLANT	$6,000	$2,160	36.0%	$8,160	3.2%	$6,190	$2,229	$8,419	2031Q1	27.7%	$7,903	$2,845	$10,748
18	CULTURAL RESOURCE PRESERVATION	$7,588	$2,732	36.0%	$10,320	3.2%	$7,829	$2,818	$10,647	2031Q1	27.7%	$9,994	$3,598	$13,592
19	BUILDINGS, GROUNDS & UTILITIES	$13,769	$4,957	36.0%	$18,726	3.2%	$14,206	$5,114	$19,320	2031Q1	27.7%	$18,136	$6,529	$24,665
	CONSTRUCTION ESTIMATE TOTALS:	$175,698	$63,251	36.0%	**$238,950**		$181,272	$65,258	$246,529			$231,419	$83,311	$314,730
30	PLANNING, ENGINEERING & DESIGN													
7.0%	of Construction Estimate Totals	$12,299	$4,428	36.0%	$16,726	3.1%	$12,680	$4,565	$17,245	2024Q3	4.7%	$13,276	$4,779	$18,055
31	CONSTRUCTION MANAGEMENT													
7.0%	of Construction Estimate Totals	$12,299	$4,428	36.0%	$16,726	2.5%	$12,606	$4,538	$17,145	2031Q1	22.2%	$15,401	$5,544	$20,945
	CONTRACT COST TOTALS:	$200,296	$72,107		**$272,403**		$206,558	$74,361	**$280,919**			$260,096	$93,635	**$353,730**

Cost and Schedule Risk Analysis (CSRA)

US Army Corps
of Engineers®

Charleston Peninsula

Project Cost and Schedule Risk Analysis Report

Prepared for:

U.S. Army Corps of Engineers,
Charleston District

Prepared by:

U.S. Army Corps of Engineers
Cost Engineering Center of Expertise, Walla Walla, Wash.

January 31, 2022

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EXECUTIVE SUMMARY

The US Army Corps of Engineers (USACE), Charleston District, presents this cost and schedule risk analysis (CSRA) report regarding the risk findings and recommended contingencies for the Charleston Peninsula project. In compliance with Engineer Regulation (ER) 1110-2-1302 Civil Works Cost Engineering, dated September 15, 2008, a *Monte Carlo*-based risk analysis was conducted by the Project Development Team (PDT) on remaining costs. The purpose of this risk analysis study is to present the cost and schedule risks considered, those determined and respective project contingencies at a recommended 80% confidence level of successful execution to project completion.

The Charleston Peninsula Project is primarily concerned with the construction of an elevated 12' (North American Vertical Datum of 1988) wall around the city of Charleston. The wall alignment was chosen to avoid personal property for footprint and to avoid taking houses / businesses unless there is no other option (only existing and known permitted structures were considered). Additional criteria were to take advantage of existing topography, consider the actions undertaken by the city, and to consider construction and maintenance easements. Through the economic analysis, the elevation of the wall was selected to be Elevation 12 NAVD88. Further optimization of the footprint is required, as the Tentatively Selected Plan to minimize wetland impacts and reduce construction costs resulted in relocating the wall to the final footprint. In some locations, the construction and maintenance easements were not met; however, these small reaches can be accommodated with shoring of the trench, use of micropiles, and other conditions in small, specific locations.

The current project base cost for the Charleston Peninsula Project estimate is approximately $714M, excluding contingency and expressed in FY 2022 dollars. This CSRA study included all estimated construction costs, Planning, Engineering, Design and Construction Management costs. Based on the results of the analysis, the Cost Engineering Center of Expertise (MCX, located in the Walla Walla District) recommends a contingency value of $257M or approximately 36% of the base project cost at an 80% confidence level of successful execution.

Cost estimates fluctuate over time. During this period of study, minor cost fluctuations can and have occurred. For this reason, contingency reporting is based in cost and per cent values. Should cost vary to a slight degree with similar scope and risks, contingency percent values will be reported, cost values rounded.

Table ES-1. Construction Contingency Results

Base Estimate ->	$714,350,000	
Confidence Level	**Contingency Value**	**Contingency**
0%	142,870,000	20%
10%	200,018,000	28%
20%	207,161,500	29%
30%	214,305,000	30%
40%	228,592,000	32%
50%	235,735,500	33%
60%	242,879,000	34%
70%	250,022,500	35%
80%	**257,166,000**	**36%**
90%	271,453,000	38%
100%	342,888,000	48%

KEY FINDINGS / OBSERVATIONS / ASSUMPTIONS & RECOMMENDATIONS

The PDT worked through the risk register in June 2021. The key risk drivers identified through sensitivity analysis suggest a cost contingency of $257.2M and schedule risks adding a potential 37.2 months; all at an 80% confidence level.

Cost Risks: From the CSRA, the key or greater Cost Risk items include:

- <u>34 Lack of Detail for Estimate / Variable Quantities</u> – Scope and details provided to the estimator are very preliminary. Variable quantities and additional details are likely to cause costs to increase.

- <u>8 Scope Refinement</u> – Changes to the wall alignment, the wall height, the number and dimensions of the control structures, aesthetic changes, etc., are all possible. Changes of this nature will also likely cause cost growth.

- <u>20 Vibration Impacts on Historic Structures</u> – Construction activities could damage adjacent buildings by way of vibration. With proper planning, it is hoped that this risk can be avoided. At the present project phase, no geotechnical analysis is available. Potential structural and aesthetic damage to historical structures is thought to be unlikely; however, if damages are realized, they would cause significant impacts to cost.

- 36 Possible Inaccurate Wall Unit Pricing – Pricing data from constructed sites was not available. The wall unit pricing used for the cost was taken from another cost estimate that is currently under development.

- 27 Living Shoreline – The base estimate includes costs for construction of a shoreline based on a model area. The development of costs associated with this feature is new and has not been verified by actual costs.

Moderate risks, when combined, can also become a cost impact.

- 28 Aesthetic Features – Wall construction is a major shift in the aesthetic beauty, land use, user activity, etc., of the area. Considerations for aesthetic features are currently unknown. Alignment adjustments during Planning, Engineering, and Design (PED) could help mitigate.

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- 8 Scope Refinement – Alterations to the scope could potentially cause the redesign of various elements and delays to the design process.

- 17 Real Estate Acquisition Schedule – If funding is not provided up front, the sponsor may have trouble acquiring real estate until the 95% design is completed. The sponsor could acquire property, but this action is at their own risk until the Project Partnership Agreement (PPA) is signed. The base schedule assumes real estate acquisition is completed within the 3-year timeline.

Recommendations: The CSRA study serves as a “road map” towards project improvements and reduced risks over time. The PDT must include the recommended cost and schedule contingencies and incorporate risk monitoring and mitigation on those identified risks. Further iterative study and update of the risk analysis throughout the project life-cycle is important in support of remaining within an approved budget and appropriation.

MAIN REPORT

1.0 PURPOSE

Within the authority of the US Army Corps of Engineers (USACE), Charleston District, this report presents the efforts and results of the cost and schedule risk analysis for the Charleston Peninsula Project. The report includes risk methodology, discussions, findings and recommendations regarding the identified risks and the necessary contingencies to confidently administer the project, presenting a cost and schedule contingency value with an 80% confidence level of successful execution.

2.0 BACKGROUND

The City of Charleston will begin an extensive reconstruction project of the iconic Low and High Battery Seawalls to replace and raise (to 12 NAVD88) the seawall to account for sea level rise projections. The existing seawalls were built over 100 years ago, and the new seawall will be engineered and built to last another century. This presents a once-in-a-lifetime opportunity to create a signature public space worthy of Charleston's character and history, while also strengthening the City against regular flooding, storm surge, and imminent sea level rise. New construction is anticipated to begin where the wall is in the poorest condition, which is on the western side at Tradd Street, and then progress to White Point Gard.

3.0 REPORT SCOPE

The scope of the risk analysis report is to identify cost and schedule risks with a resulting recommendation for contingencies at the 80 percent confidence level using the risk analysis processes, as mandated by U.S. Army Corps of Engineers (USACE) Engineer Regulation (ER) 1110-2-1150, Engineering and Design for Civil Works, ER 1110-2-1302, Civil Works Cost Engineering, and Engineer Technical Letter (ETL) 1110-2-573, Construction Cost Estimating Guide for Civil Works. The report presents the contingency results for cost risks for construction features. The CSRA does not include consideration for life cycle costs.

3.1 Project Scope

The formal process included extensive involvement of the PDT for risk identification and the development of the risk register. The analysis process evaluated the Micro Computer Aided Cost Estimating System (MCACES) cost estimate, project schedule, and funding profiles using Crystal Ball software to conduct a *Monte Carlo* simulation and statistical sensitivity analysis, per the guidance in ETL 1110-2-573, Construction Cost Estimating Guide for Civil Works, dated September 30, 2008.

The project technical scope, estimates and schedules were developed and presented by the District. Consequently, these documents serve as the basis for the risk analysis.

The scope of this study addresses the identification of concerns, needs, opportunities and potential solutions that are viable from an economic, environmental, and engineering viewpoint.

3.2 USACE Risk Analysis Process

The risk analysis process for this study follows the USACE Headquarters requirements as well as the guidance provided by the Cost Engineering MCX. The risk analysis process reflected within this report uses probabilistic cost and schedule risk analysis methods within the framework of the Crystal Ball software. Furthermore, the scope of the report includes the identification and communication of important steps, logic, key assumptions, limitations, and decisions to help ensure that risk analysis results can be appropriately interpreted.

Risk analysis results are also intended to provide project leadership with contingency information for scheduling, budgeting, and project control purposes, as well as to provide tools to support decision making and risk management as the project progresses through planning and implementation. To fully recognize its benefits, cost and schedule risk analysis should be considered as an ongoing process conducted concurrent to, and iteratively with, other important project processes such as scope and execution plan development, resource planning, procurement planning, cost estimating, budgeting and scheduling.

In addition to broadly defined risk analysis standards and recommended practices, this risk analysis was performed to meet the requirements and recommendations of the following documents and sources:

- Cost and Schedule Risk Analysis Process guidance prepared by the USACE Cost Engineering MCX.
- Engineer Regulation (ER) 1110-2-1302 CIVIL WORKS COST ENGINEERING, dated September 15, 2008.
- Engineer Technical Letter (ETL) CONSTRUCTION COST ESTIMATING GUIDE FOR CIVIL WORKS, dated September 30, 2008.

4.0 METHODOLOGY / PROCESS

The Cost Engineering MCX performed the Cost and Schedule Risk Analysis, relying on local District staff to provide expertise and information gathering. The District PDT conducted initial risk identification via meetings with the Walla Walla Cost Engineering MCX facilitator in June 2021. The initial risk identification meeting also included qualitative analysis to produce a risk register that served as the draft framework for the risk analysis.

Participants in the risk identification meeting on June 29 and 30, 2021 included:

Name	Office	Role / Discipline	6.29.21 AM	6.29.21 PM	6.30.21
Bethney Ward	SAC	Environmental Lead	X	X	X
Brian Clouse	SAC	Cost Engineer	X	X	X
Carter Rucker	SAW	Coastal Engineer	X	X	X
Corrine Stetzel	SPK	Lead Planner		X	X
Diane Perkins	SAC	Aesthetic Mitigation	X		X
Dorothy Steinbeiser	SAS	Realty Specialist	X	X	X
James Elliott	MVK	H&H			
Jennifer Kist	SAC	Geographer			
Jonathan Jellema	SAC	Office of Counsel			X
Kurt Heckendorf (SAW GEO)	SAW	Geotech	X	X	X
Lance Mahar	SAC	Technical Lead	X	X	
Meredith Moreno	SAJ	Cultural Resources	X	X	X
Nancy Parrish	SAC	Chief Planning & Env			X
Rick Lambert	SAC	Structural	X		X
Stephen Phillips	SAM	Economist			X
Wesley Wilson	SAC	Project Manager	X	X	X

The risk analysis process for this study is intended to determine the probability of various cost outcomes and quantify the required contingency needed in the cost estimate to achieve the desired level of cost confidence. Per regulation and guidance, the P80 confidence level (80% confidence level) is the normal and accepted cost confidence level. District Management has the prerogative to select different confidence levels, pending approval from Headquarters, USACE.

In simple terms, contingency is an amount added to an estimate to allow for items, conditions or events for which the occurrence or impact is uncertain and that experience suggests will likely result in additional costs being incurred or additional time being required. The amount of contingency included in project control plans depends, at least in part, on the project leadership's willingness to accept risk of project overruns. The less risk that project leadership is willing to accept the more contingency should be applied in the project control plans. The risk of overrun is expressed, in a probabilistic context, using confidence levels.

The Cost MCX guidance for cost and schedule risk analysis generally focuses on the 80-percent level of confidence (P80) for cost contingency calculation. It should be noted that use of P80 as a decision criteria is a risk averse approach (whereas the use of P50 would be a risk neutral approach, and use of levels less than 50 percent would be risk seeking). Thus, a P80 confidence level results in greater contingency as compared to a P50 confidence level. The selection of contingency at a particular confidence level is ultimately the decision and responsibility of the project's District and / or Division management.

The risk analysis process uses *Monte Carlo* techniques to determine probabilities and contingency. The *Monte Carlo* techniques are facilitated computationally by a commercially available risk analysis software package (Crystal Ball) that is an add-in to Microsoft Excel. Cost estimates are packaged into an Excel format and used directly for cost risk analysis purposes. The level of detail recreated in the Excel-format schedule is sufficient for risk analysis purposes that reflect the established risk register, but generally less than that of the native format.

The primary steps, in functional terms, of the risk analysis process are described in the following subsections. Risk analysis results are provided in Section 6.

4.1 Identify and Assess Risk Factors

Identifying the risk factors via the PDT is considered a qualitative process that results in establishing a risk register that serves as the document for the quantitative study using the Crystal Ball risk software. Risk factors are events and conditions that may influence or drive uncertainty in project performance. They may be inherent characteristics or conditions of the project or external influences, events, or conditions such as weather or economic conditions. Risk factors may have either favorable or unfavorable impacts on project cost and schedule.

A formal PDT meeting was held with the District office and project owners for the purposes of identifying and assessing risk factors. The meeting included capable and qualified representatives from multiple project team disciplines and functions, including project management, cost engineering, design, environmental compliance, real estate, construction, contracting and representatives of the sponsoring agencies.

The initial formal meetings focused primarily on risk factor identification using brainstorming techniques, but also included some facilitated discussions based on risk factors common to projects of similar scope and geographic location. Additionally, numerous conference calls and informal meetings were conducted throughout the risk analysis process on an as-needed basis to further facilitate risk factor identification, market analysis, and risk assessment.

4.2 Quantify Risk Factor Impacts

The quantitative impacts (putting it to numbers of cost and time) of risk factors on project plans were analyzed using a combination of professional judgment, empirical data and analytical techniques. Risk factor impacts were quantified using probability distributions (density functions) because risk factors are entered into the Crystal Ball software in the form of probability density functions.

Similar to the identification and assessment process, risk factor quantification involved multiple project team disciplines and functions. However, the quantification process relied more extensively on collaboration between cost engineering and risk analysis team members with lesser inputs from other functions and disciplines. This process used an iterative approach to estimate the following elements of each risk factor:

- Maximum possible value for the risk factor
- Minimum possible value for the risk factor
- Most likely value (the statistical mode), if applicable
- Nature of the probability density function used to approximate risk factor uncertainty
- Mathematical correlations between risk factors
- Affected cost estimate and schedule elements

The resulting product from the PDT discussions is captured within a risk register as presented in section 6 for both cost and schedule risk concerns. Note that the risk register records the PDT's risk concerns, discussions related to those concerns, and potential impacts to the current cost and schedule estimates. The concerns and discussions support the team's decisions related to event likelihood, impact, and the resulting risk levels for each risk event.

4.3 Analyze Cost Estimate and Schedule Contingency

Contingency is analyzed using the Crystal Ball software, an add-in to the Microsoft Excel format of the cost estimate and schedule. *Monte Carlo* simulations are performed by applying the risk factors (quantified as probability density functions) to the appropriate estimated cost and schedule elements identified by the PDT. Contingencies are calculated by applying only the moderate and high level risks identified for each option (i.e., low-level risks are typically not considered, but remain within the risk register to serve historical purposes as well as support follow-on risk studies as the project and risks evolve).

For the cost estimate, the contingency is calculated as the difference between the P80 cost forecast and the baseline cost estimate. Each option-specific contingency is then allocated on a civil works feature level based on the dollar-weighted relative risk of each feature as quantified by *Monte Carlo* simulation. Standard deviation is used as the feature-specific measure of risk for contingency allocation purposes. This approach results in a relatively larger portion of all the project feature cost contingency being allocated to features with relatively higher estimated cost uncertainty.

5.0 PROJECT ASSUMPTIONS

The following data sources and assumptions were used in quantifying the costs associated with the project.

a. The District provided estimate files electronically. The files transmitted and resulting independent review, served as the basis for the final cost and schedule risk analyses.

b. The cost comparisons and risk analyses performed and reflected within this report are based on design scope and estimates that are at the feasibility level of design.

c. Schedules are analyzed for impact to the project cost in terms of delayed funding, uncaptured escalation (variance from OMB factors and the local market) and unavoidable fixed contract costs and / or languishing federal administration costs incurred throughout delay.

d. The Cost Engineering MCX guidance generally focuses on the eighty-percent level of confidence (P80) for cost contingency calculation. For this risk analysis, the eighty-percent level of confidence (P80) was used. It should be noted that the use of P80 as a decision criteria is a moderately risk averse approach, generally resulting in higher cost contingencies. However, the P80 level of confidence also assumes a small degree of risk that the recommended contingencies may be inadequate to capture actual project costs.

e. Only high and moderate risk level impacts, as identified in the risk register, were considered for the purposes of calculating cost contingency. Low level risk impacts should be maintained in project management documentation, and reviewed at each project milestone to determine if they should be placed on the risk “watch list”.

6.0 RESULTS

The cost and schedule risk analysis results are provided in the following sections. In addition to contingency calculation results, sensitivity analyses are presented to provide decision makers with an understanding of variability and the key contributors to the cause of this variability.

6.1 Risk Register

A risk register is a tool commonly used in project planning and risk analysis. The actual risk register is provided in Appendix A – Risk Register. The complete risk register includes low level risks, as well as additional information regarding the nature and impacts of each risk.

It is important to note that a risk register can be an effective tool for managing identified risks throughout the project life cycle. As such, it is generally recommended that risk registers be updated as the designs, cost estimates, and schedule are further refined, especially on large projects with extended schedules. Recommended uses of the risk register going forward include:

- Documenting risk mitigation strategies being pursued in response to the identified risks and their assessment in terms of probability and impact.
- Providing project sponsors, stakeholders, and leadership / management with a documented framework from which risk status can be reported in the context of project controls.
- Communicating risk management issues.
- Providing a mechanism for eliciting feedback and project control input.
- Identifying risk transfer, elimination, or mitigation actions required for implementation of risk management plans.

6.2 Cost Contingency and Sensitivity Analysis

The result of risk or uncertainty analysis is quantification of the cumulative impact of all analyzed risks or uncertainties as compared to probability of occurrence. These results, as applied to the analysis herein, depict the overall project cost at intervals of confidence (probability).

Table 1. Construction Cost Contingency Summary provides the construction cost contingencies calculated for the P80 confidence level and rounded to the nearest thousand. The construction cost contingencies for the P5, P50 and P90 confidence levels are also provided for illustrative purposes only.

Table 1. Construction Cost Contingency Summary

Base Estimate ->	**$714,350,000**	
Confidence Level	**Contingency Value**	**Contingency**
0%	142,870,000	20%
10%	200,018,000	28%
20%	207,161,500	29%
30%	214,305,000	30%
40%	228,592,000	32%
50%	235,735,500	33%
60%	242,879,000	34%
70%	250,022,500	35%
80%	**257,166,000**	**36%**
90%	271,453,000	38%
100%	342,888,000	48%

Contingency on Base Estimate	**80% Confidence Project Cost**	
Base Estimate ->	**$714,350,000**	
Estimate Contingency ->	**$257,166,000**	**36%**
Base Estimate w/ Contingency (80% Confidence) ->	**$971,516,000**	

6.2.1 Sensitivity Analysis

Sensitivity analysis generally ranks the relative impact of each risk / opportunity as a percentage of total cost uncertainty. The Crystal Ball software uses a statistical measure (contribution to variance) that approximates the impact of each risk / opportunity contributing to variability of cost outcomes during *Monte Carlo* simulation.

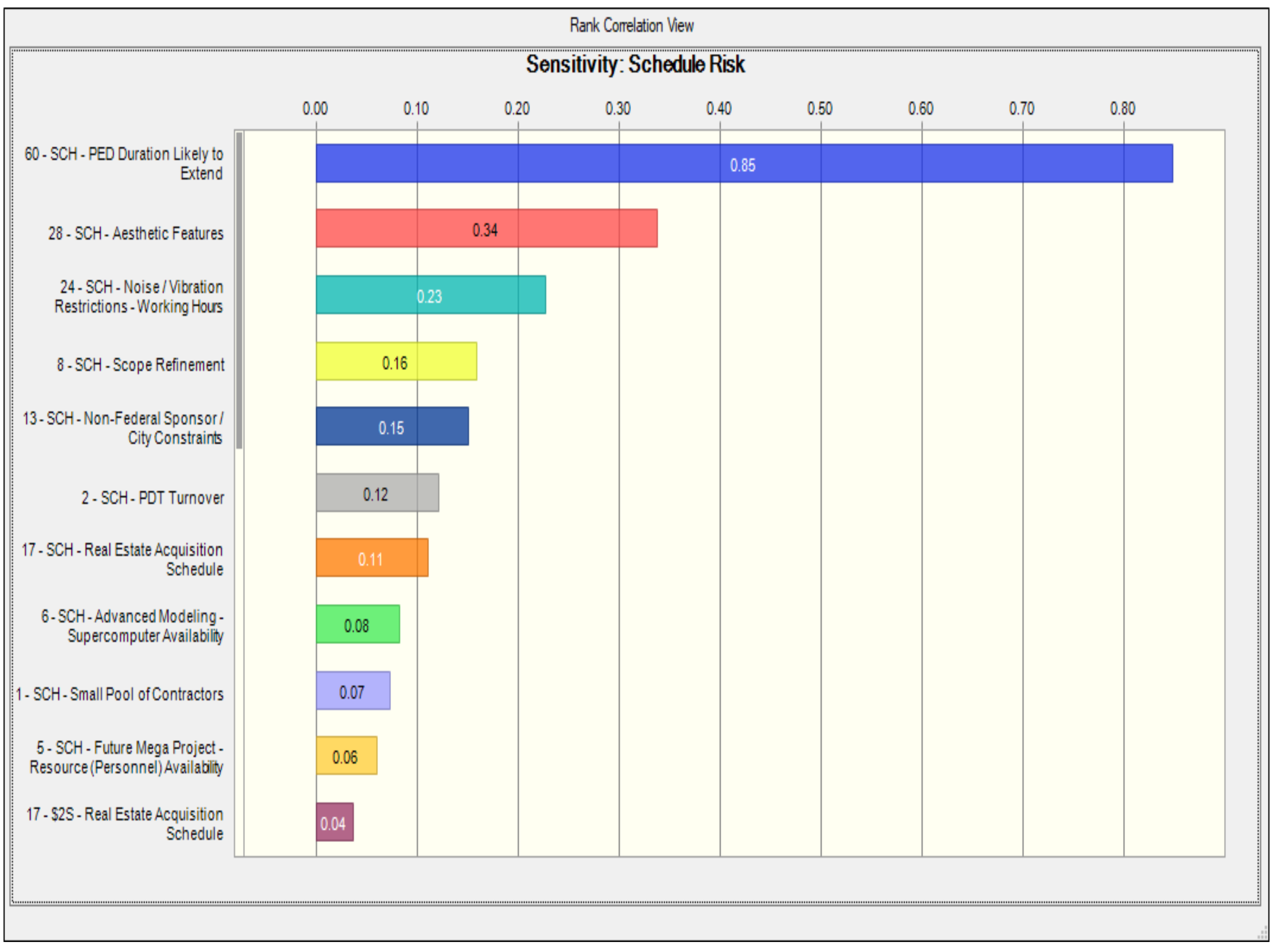
Key cost drivers identified in the sensitivity analysis can be used to support development of a risk management plan that will facilitate control of risk factors and their potential impacts throughout the project lifecycle. Together with the risk register, sensitivity analysis results can also be used to support development of strategies to eliminate, mitigate, accept or transfer key risks.

6.2.2 Sensitivity Analysis Results

The risks / opportunities considered as key or primary cost drivers and the respective value variance are ranked in order of importance in contribution to variance bar charts. Opportunities that have a potential to reduce project cost and are shown with a negative sign; risks are shown with a positive sign to reflect the potential to increase project cost. A longer bar in the sensitivity analysis chart represents a greater potential impact to project cost.

Figure 1. Cost Sensitivity Analysis presents a sensitivity analysis for cost growth risk from the high-level cost risks identified in the risk register. Likewise, Figure 2. Schedule Sensitivity Analysis presents a sensitivity analysis for schedule growth risk from the high-level schedule risks identified in the risk register.

Figure 1. Cost Sensitivity Analysis



6.3 Schedule and Contingency Risk Analysis

The result of risk or uncertainty analysis is quantification of the cumulative impact of all analyzed risks or uncertainties as compared to probability of occurrence. These results, as applied to the analysis herein, depict the overall project duration at intervals of confidence (probability).

Table 2. Schedule Duration Contingency Summary provides the schedule duration contingencies calculated for the P80 confidence level. The schedule duration contingencies for the P50 and P90 confidence levels are also provided for illustrative purposes.

These contingencies were used to calculate the projected residual fixed cost impact of project delays that are included in the Table 1. Construction Cost Contingency Summary presentation of total cost contingency. The schedule contingencies were calculated by applying the high level schedule risks identified in the risk register for each option to the durations of critical path and near critical path tasks.

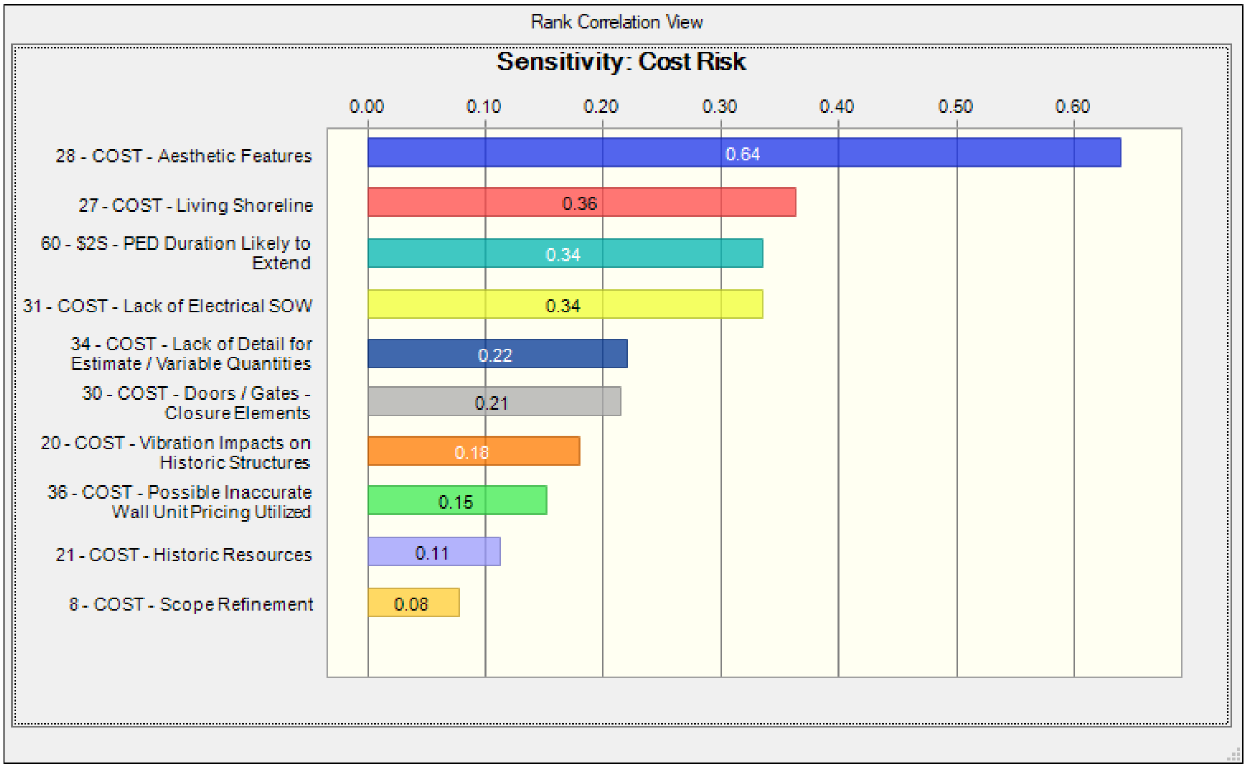
The schedule was not resource loaded and contained open-ended tasks and non-zero lags (gaps in the logic between tasks) that limit the overall utility of the schedule risk analysis. These issues should be considered as limitations in the utility of the schedule contingency data presented. Schedule contingency impacts presented in this analysis are based solely on projected residual fixed costs.

Table 2. Schedule Duration Contingency Summary

Base Schedule Duration ->	**120.0 Months**	
Confidence Level	**Contingency Value**	**Contingency**
0%	22.8 Months	19%
10%	30.0 Months	25%
20%	31.2 Months	26%
30%	32.4 Months	27%
40%	33.6 Months	28%
50%	33.6 Months	28%
60%	34.8 Months	29%
70%	36.0 Months	30%
80%	**37.2 Months**	**31%**
90%	38.4 Months	32%
100%	45.6 Months	38%

Contingency on Base Schedule	80% Confidence Project Schedule	
Base Schedule Start Date ->	October 1, 2022	
Base Schedule Finish Date ->	September 30, 2032	
Base Schedule Duration ->	120.0 Months	
Schedule Contingency Duration ->	37.2 Months	31%
Base Schedule w/ Contingency (80% Confidence) ->	157.2 Months	
Base Finish Date w/ Contingency (80% Confidence) ->	November 5, 2035	

Figure 2. Schedule Sensitivity Analysis



7.0 MAJOR FINDINGS / OBSERVATIONS / RECOMMENDATIONS

This section provides a summary of significant risk analysis results that are identified in the preceding sections of the report. Risk analysis results are intended to provide project leadership with contingency information for scheduling, budgeting, and project control purposes, as well as to provide tools to support decision making and risk management as projects progress through planning and implementation. Because of the potential for use of risk analysis results for such diverse purposes, this section also reiterates and highlights important steps, logic, key assumptions, limitations, and decisions to help ensure that the risk analysis results are appropriately interpreted.

7.1 Major Findings / Observations

Project cost and schedule comparison summaries are provided in Table 3. Construction Cost Comparison Summary (Uncertainty Analysis) and Table 4.

Construction Schedule Comparison Summary (Uncertainty Analysis) respectively. Additional major findings and observations of the risk analysis are listed below.

The PDT worked through the risk register in June 2021. The key risk drivers identified through sensitivity analysis suggest a cost contingency of $257.2M and schedule risks adding a potential 37.2 months; all at an 80% confidence level.

Cost Risks: From the CSRA, the key or greater Cost Risk items include:

- 34 Lack of Detail for Estimate / Variable Quantities – Scope and details provided to the estimator are preliminary. Variable quantities and additional details are likely to cause costs to increase.

- 8 Scope Refinement – Changes to the wall alignment, the wall height, and the number and dimensions of the control structures, as well as aesthetic changes, etc., are all possible. Changes of this nature will also likely cause cost growth.

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Moderate risks, when combined, can also become a cost impact.

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- 47 Lack of Staging Areas / Working Space – The lack of staging areas throughout the city will likely cause the need for just-in-time deliveries, (compounding sequencing issues), and an imported workforce due to the job size (with additional hotel costs).

Schedule Risks: From the CSRA, the key or greater Schedule Risk items include:

- 60 Planning, Engineering, and Design Duration is Likely to Extend – Schedule slips are likely to occur within the PED phase due to design milestone reviews, environmental coordination and surveying, changes in design, supplemental NEPA assessments, railroad crossing coordination, and changes in aesthetic features.

- 8 Scope Refinement – Alterations to the scope could potentially cause the redesign of various elements and delays to the design process.

- 17 Real Estate Acquisition Schedule – If funding is not provided up front, the sponsor may have trouble acquiring real estate until the 95% design is completed. The sponsor could acquire property, but this action is at their own risk until the PPA is signed. The base schedule assumes real estate acquisition is completed within the 3-year timeline.

Table 3. Construction Cost Comparison Summary (Uncertainty Analysis)

Base Estimate ->	$714,350,000	
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Contingency on Base Estimate	80% Confidence Project Cost	
Base Estimate ->	$714,350,000	
Estimate Contingency ->	$257,166,000	36%
Base Estimate w/ Contingency (80% Confidence) ->	$971,516,000	

Table 4. Construction Schedule Comparison Summary (Uncertainty Analysis)

Base Schedule Duration ->	120.0 Months	
Confidence Level	**Contingency Value**	**Contingency**
0%	22.8 Months	19%
10%	30.0 Months	25%
20%	31.2 Months	26%
30%	32.4 Months	27%
40%	33.6 Months	28%
50%	33.6 Months	28%
60%	34.8 Months	29%
70%	36.0 Months	30%
80%	**37.2 Months**	**31%**
90%	38.4 Months	32%
100%	45.6 Months	38%

Contingency on Base Schedule	80% Confidence Project Schedule	
Base Schedule Start Date ->	October 1, 2022	
Base Schedule Finish Date ->	September 30, 2032	
Base Schedule Duration ->	120.0 Months	
Schedule Contingency Duration ->	37.2 Months	31%
Base Schedule w/ Contingency (80% Confidence) ->	157.2 Months	
Base Finish Date w/ Contingency (80% Confidence) ->	November 5, 2035	

7.2 Recommendations

Risk Management is an all-encompassing, iterative, and life-cycle process of project management. The Project Management Institute's (PMI) *A Guide to the Project Management Body of Knowledge (PMBOK® Guide)*, *4th edition*, states that "project risk management includes the processes concerned with conducting risk management planning, identification, analysis, responses, and monitoring and control on a project." Risk identification and analysis are processes within the knowledge area of risk management. Its outputs pertinent to this effort include the risk register, risk quantification (risk analysis model), contingency report, and the sensitivity analysis.

The intended use of these outputs is implementation by the project leadership with respect to risk responses (such as mitigation) and risk monitoring and control. In short, the effectiveness of the project risk management effort requires that the proactive management of risks not conclude with the study completed in this report.

The Cost and Schedule Risk Analysis (CSRA) produced by the PDT identifies issues that require the development of subsequent risk response and mitigation plans. This section provides a list of recommendations for continued management of the risks identified and analyzed in this study. Note that this list is not all inclusive and should not substitute a formal risk management and response plan.

The CSRA study serves as a “road map” towards project improvements and reduced risks over time. The PDT must include the recommended cost and schedule contingencies and incorporate risk monitoring and mitigation on those identified risks. Further iterative study and update of the risk analysis throughout the project life-cycle is important in support of remaining within an approved budget and appropriation.

Risk Management: Project leadership should use of the outputs created during the risk analysis effort as tools in future risk management processes. The risk register should be updated at each major project milestone. The results of the sensitivity analysis may also be used for response planning strategy and development. These tools should be used in conjunction with regular risk review meetings.

Risk Analysis Updates: Project leadership should review risk items identified in the original risk register and add others, as required, throughout the project life-cycle. Risks should be reviewed for status and reevaluation (using qualitative measure, at a minimum) and placed on risk management watch lists if any risk’s likelihood or impact significantly increases. Project leadership should also be mindful of the potential for secondary (new risks created specifically by the response to an original risk) and residual risks (risks that remain and have unintended impact following response).

Appendix A – Risk Register

Meeting 7/1/2021 Updated Cost 1/28/2022					Project Cost			Project Schedule		
REF	**Risk Type**	**Risk/Opportunity Event**	**Risk Event Description**	**PDT Discussions on Impact and Likelihood**	**Likelihood (C)**	**Impact (C)**	**Risk Level (C)**	**Likelihood (S)**	**Impact (S)**	**Risk Level (S)**
1	01 - Project & Program Management (PM)	Small Pool of Contractors	• Concern that there are not enough contractors in the area that can do this type of work. • Impacts schedule primarily. • Potentially there will be low amounts of competition. • Contracting plan is totally undeveloped.	• Thought to be likely that there is a schedule impact. Current thought is that companies will "staff up" as these projects hit the street. •Depending on size of contracts, it is likely that large, nationwide contractors will also bid on the projects and use "local" contractors as subcontractors - unless the contracting plan indicates that small businesses need to be used. •Likely/Negligible cost impact suspected. • Likely/Moderate schedule impact suspected.	Likely	Negligible	**Low**	Likely	Moderate	**Medium**
2	01 - Project & Program Management (PM)	PDT Turnover	Long project duration will likely impact: • PDT composition (personnel turnover)	• Viewed as a schedule risk. • Very likely/marginal.	Very Likely	Negligible	**Low**	Very Likely	Marginal	**Medium**
3	01 - Project & Program Management (PM)	Sponsor Change of Will / Lack of Funding.	Long project duration will likely impact: • Local will for project completion. City could change after partial completion. • Sponsor may not have funding.	• May simply cancel the project. •This is a game changing risk. If it occurs, the project is likely canceled. Not modeled as a result.	Possible	Negligible	**Low**	Possible	Negligible	**Low**
4	01 - Project & Program Management (PM)	Competing Projects	• Other large-scale Coastal Storm Risk Management (CSRM) projects within USACE. • May impact schedule.	• Related to small pool of contractors. Will this small pool be overly busy due to multiple USACE projects? • Assumed that this risk is captured above (Small Pool of Contractors)	Possible	Negligible	**Low**	Possible	Negligible	**Low**
5	01 - Project & Program Management (PM)	Future Mega Project - Resource (Personnel) Availability	• Concern of having adequate resources to support mega project. Charleston district is historically smaller. • Schedule delay primarily.	• Primarily schedule risk. • A&E contract assumed during PED. Still will require technical oversight. • May have cost increase (per diem). Maybe an increase in cost due to differing locality rate. Cost change thought to be negligible. Schedule change thought to be moderate	Possible	Negligible	**Low**	Possible	Moderate	**Medium**
6	01 - Project & Program Management (PM)	Advanced Modeling - Supercomputer Availability	• Could delay design due to lack of supercomputing availability. Large line of projects for these computers. • Computing time can be costly.	• Not thought to cost an excessive amount. • Schedule risk primarily. Could cost moderate amount of time (3-4months). Schedule risk of possible/moderate	Possible	Negligible	**Low**	Possible	Moderate	**Medium**
7	01 - Project & Program Management (PM)	Update to Survey Data - Elevations	• Not considered a risk at this point. • Schedule already contains time for this effort.	• Charleston personnel are less familiar with land-based survey. • Assume that this may be a cost to hire resources needed. • Assumed to be within PED costs. Additional cost assumed to be negligible.	Possible	Negligible	**Low**	Possible	Negligible	**Low**

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8	02 - Scope and Objectives (SC)	Scope Refinement	• Changes in alignment • Changes in wall height • Changes in control structures • Changes for aesthetic reasons. • Additional environmental mitigation efforts. • Changing foundation elevation • City input may cause change • Public input may also cause changes. • Changes in future city development will also impact alignment. • New survey elevation data.	• This is thought to be one of the biggest risks of the project. • Wall height not likely to change because it will change benefits. • Dated information - only permitted projects utilized (11/2018). • Both positive and negative changes will occur. Could be a cost wash in the end. • Given that PED is 3 years, if changes are not captured at the start of the project, redesign later in the project will have impacts. If the redesign is significant and completed after an Agency Technical Review or Independent External Peer Review, the redesign will have to go through that process again. Given this, PED schedule could be extended months. Assume Moderate schedule impact with PED costs due to schedule slip.	Very Likely	Significant	**High**	Very Likely	Significant	**High**
9	03 - Ability to Execute (AB)	Funding Constraints & City Cost Sharing	• Inability to obtain funding to get this completed in a reasonable amount of time. • Related to competing projects with PM.	The base schedule assumes funding is available when needed without funding constraints. PED is assumed to start in FY23, and the project has a 10-year duration. The city's ability to cost-share (35% on regular construction and 100% on betterments) could also delay the schedule. LV: No change from base assumptions (funding received in FY23) ML: No change from base assumptions (funding received in FY23) HV: May never get funded. Identified and documented but **not modeled due to it being a "game-changing risk".**	Very Likely	Critical	**High**	Very Likely	Critical	**High**
10	04 - External Risks (EX)	Potential Lawsuit	• A lawsuit could drastically affect the implementation schedule and related costs due to pushing the project out in time.	Depending on the nature of the lawsuit, **this could be another "game-changer" risk that is documented but not anticipated to occur at this time.**	Unlikely	Critical	**Medium**	Unlikely	Critical	**Medium**
11	04 - External Risks (EX)	Public / Stakeholders	• Public Pushback • A 12' wall around the city will likely have dissenters that enjoy the current views. • New stakeholders could emerge, or mitigation scope could possibly change based on opinions as design progresses.	Early coordination will help mitigate some of the risk by working with the designers and having continuous discussions with the stakeholders to help capture proposed changes. NEPA requirements will be satisfied upon completion of the environmental impact statement.	Possible	Negligible	**Low**	Possible	Negligible	**Low**
12	04 - External Risks (EX)	Volume of Work in Same Area	• Large project may occupy contractor(s) in area for years making pricing jump.	Same risk as congested work area within construction risks. Not considered here since it is duplicated.	Unlikely	Negligible	**Low**	Unlikely	Negligible	**Low**
13	04 - External Risks (EX)	Non-Federal Sponsor / City Constraints	• City of Charleston may demand prohibitive work constraints. • Mayor could change in 6 years in the middle of construction. • City requests changes that could change our design / cost / schedule. • Political pressures. • Coordination of reviews with the city.	There are risks related to design features that the city is not completely satisfied with, which could cause schedule delays. The city will be coordinated with along the way during the milestone reviews. Political pressure could be the biggest risk, and worst case, the sponsor could pull out. LV / ML: No change from base schedule. HL: 3 Mo delay.	Possible	Negligible	**Low**	Possible	Moderate	**Medium**

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14	04 - External Risks (EX)	Restrictions on Types of Funding & PPA	Several projects have seen restrictions based on the type of funding prior to the PPA being signed which could also impact reimbursements back to the sponsor or stakeholder.	Not viewed as likely to occur in this case.	Unlikely	Negligible	**Low**	Unlikely	Negligible	**Low**
15	04 - External Risks (EX)	Real Estate Contingency	Contingency on real estate ($131M with contingency) could vary from the current assumptions.	The current real estate costs have a 45% contingency on the acquisition costs and 25% on administration & relocation costs. This risk is to capture the base assumptions and document the potential variation until design is progressed further related to the alignment, etc., since contingencies are provided by real estate. Risk not modeled due to contingencies provided which is to help capture some of the variations.	Likely	Significant	**High**	Likely	Negligible	**Low**
16	04 - External Risks (EX)	Staging Areas	If there are four construction contracts, some of which may overlap, this could cause a concern on whether there are adequate staging areas for the contractors to use.	The estimate doesn't currently include the contractor acquiring property which could increase costs. The estimate will be adjusted to capture these potential costs of leasing lands off of the peninsula therefore making this a low-risk item.	Likely	Negligible	**Low**	Likely	Negligible	**Low**
17	04 - External Risks (EX)	Real Estate Acquisition Schedule	Real estate acquisition is currently scheduled to be completed in 2 years on a 3-year PED schedule, but condemnations could cause the schedule to slip.	If funding is not provided up front, the sponsor may have trouble acquiring real estate until the 95% design is completed. The sponsor could acquire property, but it is at their own risk until the PPA is signed. The base schedule assumes real estate acquisition is completed within the 3-year timeline. COST: See item 15. Real estate already provides contingency for additional areas. Given that Real Estate Acquisition needs to start 1 year after PED begins, the designs of the projects will not be well defined. This will mean that conservative assumptions will need to be made on the project footprint, increasing the amount of real estate acquired.	Unlikely	Negligible	**Low**	Likely	Significant	**High**
18	05 - Contract Acquisition Risks (CA)	Contracting Planning	• Total project will likely be a series of consecutive and sequential contracts. Contracting plan is totally undeveloped.	• Primarily a schedule risk. • Assume related to small pool of contractors. • Not modeled because assumed captured under PM risks.	Likely	Negligible	**Low**	Unlikely	Negligible	**Low**
19	05 - Contract Acquisition Risks (CA)	Contracting Officer Warrant	• Charleston district is smaller. Necessary warrant for project of this size may not be possessed within district.	• Schedule risk. • May need to get outside help to accomplish necessary contract. •Assumed major schedule impact will not be realized since this has been identified early as a risk. Proper planning can avoid this risk.	Possible	Negligible	**Low**	Possible	Marginal	**Low**

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20	09 - Environmental & Cultural/Historical Resources (EC)	Vibration Impacts on Historic Structures	• Vibrations could impact historical structures and foundations.	• No soil data on hand (need Geotech analysis). • May need to conduct residential repairs. • Hope that vibration will be minimized by proper planning / design. • Assume unlikely to occur, with large costs if damage does occur. • Not assumed that this would impact schedule - not on critical path. • Pre-construction survey, post-construction survey, and vibration monitoring during construction will be added to the construction estimate. •This risk is very unlikely to occur but carries a significant risk due to the historical structures. Pile driving operations are >100' away from the nearest homes along the low and high battery walls. Seen as a risk that should be modeled due to the low chance of occurrence but high costs if realized.	Unlikely	Significant	**Medium**	Unlikely	Negligible	**Low**
21	09 - Environmental & Cultural/Historical Resources (EC)	Historic Resources	• Unknown scope, quantities, and areas requiring mitigation for historical resources.	The base estimate assumes some buildings and areas to be mitigated (~$15M) but this will not become clear until the design of the wall is complete. Assumes no impact to the critical path of the schedule.	Very Likely	Marginal	**Medium**	Very Likely	Negligible	**Low**
22	09 - Environmental & Cultural/Historical Resources (EC)	Unanticipated Discoveries During Construction	This is related to archeological sites that we haven't identified yet.	The base estimate assumes some costs in the $15M mitigation costs but this risk item could lead to schedule delays if things are discovered during construction. This could be modeled as an event risk in case this does happen (maybe 25% of the time) and could impact the project during construction (may not be critical path because contractor could move to another area and come back to complete the work). At this time, it is believed that costs are covered within the estimate and that schedule delays will not impact the critical path.	Possible	Negligible	**Low**	Possible	Negligible	**Low**
23	09 - Environmental & Cultural/Historical Resources (EC)	Wetland Mitigation	• Compensable wetland mitigation quantities are well defined but there is uncertainty on the approach to do the compensation (1-purchase bank credits, 2-own restoration). • Wetland mitigation efforts are likely to change and evolve throughout project. • Likely to occur after construction is finished.	The base estimate ($9M for 35 acres) assumes purchasing bank credits (more conservative) versus doing our own restoration. Own restoration may lead into purchasing real estate which is currently not included in the real estate estimate. Would have to purchase the wetland credit at the beginning of construction but not anticipated to cause a schedule delay. The risk associated with our ability to go with the least cost per policy is that it's possible the credits are no longer available in the future so then we'd have to go with the next least cost bank, etc. There's also risk that acreage calculation is off slightly. Here are costs for the 35 acres that need to be compensated for the three different banks: LV (Murry Hill bank) = $7,602,400 = -$1.5M Variance ML (Point Farm bank) = $9,068,160 = $0 Variance HV (Clydesdale bank) = $9,391,200 = $330K Variance Variance from estimate is so low that this element is not modeled.	Possible	Negligible	**Low**	Possible	Negligible	**Low**

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24	09 - Environmental & Cultural/Historical Resources (EC)	Noise / Vibration Restrictions - Working Hours	• Some areas will be constructed near hospital and/or residential areas. • Sheet pile driving will be noisy and create vibration • May impact marine mammals (dolphins) • City has noise ordinance which may restrict pile driving to only day-time work • Marine mammals (pile-driving under water) may require some sound barriers or work performed at low-tide (one time window during the daytime) to avoid impacting marine mammals.	Underwater noise not an issue when constructing in the Wagner Terrace MA because the water will be very shallow there. So will only be an issue in Marine MA and Port MA where wall sections underwater will be relatively short. It might be more sensible to restrict working to low tide (half day) for these short wall segments than requiring expensive sound buffering equipment. For the T-Wall, the main risk will be associated also with limited workday to daylight hours. This risk is thought to be possible to be mitigated with the necessary contractual restrictions.	Likely	Marginal	**Medium**	Likely	Moderate	**Medium**
25	09 - Environmental & Cultural/Historical Resources (EC)	Water Quality Mitigation	Will need to address water quality impacts. It is undetermined at this time whether we will model water quality to see if we need to mitigate, or we could go ahead and do mitigation for water quality (clean the water at the pump stations) or do nothing and monitor for now.	Risk thought to be possible. Impacts to cost and schedule are thought to be negligible moving forward.	Possible	Negligible	**Low**	Possible	Negligible	**Low**
26	09 - Environmental & Cultural/Historical Resources (EC)	Additional HTRW Remediation Site Survey	One site was assumed to have a Phase I and Phase II site assessment for hazardous waste, but there may be add'l sites.	As of right now, we are still only assuming one site for hazardous, toxic, and radioactive waste (HTRW) (identified by EPA). There is a landfill area that could potentially be a 2nd HTRW site. Waste thought to only be household waste. Doubtful that this will be a 2nd HTRW site not finding a compelling enough reason to evaluate a second HTRW site, but it is a possible risk. No remediation costs are expected at either site. PED: HTRW Survey Cost = $290K Cost is negligible, and surveys will occur during PED.	Possible	Negligible	**Low**	Possible	Negligible	**Low**
27	09 - Environmental & Cultural/Historical Resources (EC)	Living Shoreline	The approach for calculating the costs is based on a new product from "The Measures and Cost Library" developed by SAD.	The base estimate includes costs for construction of a shoreline based on a model area. The low range was used b/c it has a less structural component to it. This could potentially be a subcontractor to the prime contractor.	Possible	Critical	**High**	Possible	Negligible	**Low**
28	12 - Architectural and Interior (AI)	Aesthetic Features	• Wall construction is a major shift in the aesthetic beauty, land use, user activity, etc. of the area. • Considerations for aesthetic features are currently unknown. • Risks during PED phase for agency determination, public interest. • Alignment adjustments during PED could help mitigate	The base estimate assumes ~$5.5M for labor for conducting a detailed assessment of the alignment, determining the major options of mitigation, assessing the various options, and assessing the cost effectiveness. The alignment, gate openings, etc. could also affect these assumptions. There are currently two wall types, combo wall in the marsh & T-wall on land, with a $40M placeholder on how that design translates to construction.	Very Likely	Critical	**High**	Very Likely	Significant	**High**

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29	14 - Structural (SD)	Railroad Crossings	• Coordination with railways has historically been difficult. • There are several railroad (RR) crossings. • RR are historically difficult to work with.	The PED schedule has 3-years for design. Some of the gates could potentially go away with future re-development plans. There are risks for schedule delays, which are captured in the PED risk item. Assume RR closures or any other difficult area that requires a lot of coordination could potentially be separated out from the larger projects and completed on an extended schedule to mitigate risk of schedule delays.	Possible	Negligible	**Low**	Possible	Negligible	**Low**
30	14 - Structural (SD)	Doors / Gates - Closure Elements	• There will be several gates and/or doors that will need to be closed during storm events. Current quantities available are preliminary. • Number of gates could vary. • Storage of gates and building for maintenance.	Base estimate assumes a number of swing gates, stoplog closures, etc. The size, type, and number of gates could vary as design progresses. Will try to minimize number of gates because they are failure points.	Likely	Moderate	**Medium**	Likely	Negligible	**Low**
31	15 - Electrical (EE)	Lack of Electrical SOW	• Street lighting, sidewalk lighting • Gate operability • Pump stations (no add'l transmission lines assumed to be needed)	Scope of work (SOW) is currently unknown other than identifying certain scope items that may be needed. Base estimate assumes a 6% ($40M) allowance which can vary.	Very Likely	Critical	**High**	Very Likely	Negligible	**Low**
32	16 - Mechanical (ME)	Gates	• Approximately 90 gates in this project. • Additional opportunity for leakage. • Manual closed gates are anticipated to be utilized as much as possible.	• More of a performance-based risk. • Variation in cost and schedule is included in Risk #30.	Unlikely	Negligible	**Low**	Unlikely	Negligible	**Low**
33	18 - Hazardous Materials (HZ)	Landfill Material	• Potential for unsuitable foundations along Ashley River. This area was formerly a landfill. • Disturbed HTRW may need remediation.	Walls are pile founded, reducing the risk that unsuitable soils near the surface will affect overall design and construction; the effect would be potential difficultly driving. This could be remedied by auguring or using longer sheet pile cutoff. Project Cost: Likelihood = Possible; Impact = Marginal Project Schedule: Likelihood = Possible; Impact = Marginal	Possible	Marginal	**Low**	Possible	Marginal	**Low**
34	19 - Estimate and Schedule Risks (ES)	Lack of Detail for Estimate / Variable Quantities	• Changing quantities will likely change costs and schedules.	•Assume up to a 3%-7% cost increase as scope becomes more defined.	Very Likely	Significant	**High**	Very Likely	Negligible	**Low**
35	19 - Estimate and Schedule Risks (ES)	Alignment Changes	• Causes cascading impact as scope changes due to alignment change. • Change to alignment occurs due to a wide variety of factors. • Interconnected to real estate.	• Alignment may be somewhat constrained. • City may desire alignment changes for betterments. • Newly collected data may alter alignment. • See 02 - Scope and Objectives (SC) - scope refinement. Captured there.	Unlikely	Negligible	**Low**	Unlikely	Negligible	**Low**
36	19 - Estimate and Schedule Risks (ES)	Possible Inaccurate Wall Unit Pricing Utilized	• T-Walls and combo walls utilized by estimate are from site that has not been constructed to date. • Mitigation pricing is even less certain.	• Assume a possible 3%-10% price increase. • T-Wall = $4K per linear foot (LF) at 28,500 LF •T-Wall with walking path = $6,500 at 3,600 LF • Combo Wall = $13K/LF at 8,700 LF	Possible	Critical	**High**	Unlikely	Negligible	**Low**

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37	19 - Estimate and Schedule Risks (ES)	Escalation Forecast	• Project is long and accurate escalation will be difficult to forecast.	•Assume that economic disruptions due to COVID 19 will stabilize in the future. Additional escalation not thought to be a major bust at this time.	Unlikely	Negligible	**Low**	Unlikely	Negligible	**Low**
38	19 - Estimate and Schedule Risks (ES)	Pump Station Assumptions	• Interior flooding assessment was completed prior to pump sizing and quantities used within estimate. • Assessment to be remodeled during PED. Changes may occur as modeling refined.	•Better pump station pricing obtained since ARA. •Estimate contains 1-90CFS $5.7M, 1-60CFS $3.8M •It is thought possible that a 3rd pump station may be needed at a later date. •Not thought to impact schedule assuming that it can be completed simultaneous to the critical path.	Possible	Moderate	**Medium**	Possible	Negligible	**Low**
39	19 - Estimate and Schedule Risks (ES)	Material Shortages / Long Lead	• Covid 19 economic disruptions. • Connected to competing projects. Concern that there may be other projects in the area that could potentially utilize materials.	• Assume that economic disruptions due to COVID 19 will stabilize in the future. Additional escalation not thought to be a major bust at this time.	Possible	Negligible	**Low**	Possible	Negligible	**Low**
40	20 - Utilities (UT)	Utility Crossings	• Additionally utility location. • Changes in footprint and dimensions. • Lack of information and inaccurate information provided.	• No solid data at this point. Data obtained from the city, but coordination with all utilities is incomplete. • Old city likely has utilities without full utility as-builts. • ID of utilities assumed to be simultaneous with other design phases. • Viewed as cost risk primarily during PED and then construction to relocate. • Cannot simply put new pipeline or utility line under wall. Care must be taken to address any seepage concerns along or through utility crossing.	Very Likely	Significant	**High**	Very Likely	Negligible	**Low**
41	22 - General Technical Risks (GR)	Unexploded Ordinance (UXO)	• Area near Ft. Sumpter could possibly have UXO.	Assumed to be a low risk at this time.	Unlikely	Negligible	**Low**	Unlikely	Negligible	**Low**
42	24 - Equipment List (EQ)	Unique Equipment Required	• No unique equipment anticipated.		Unlikely	Negligible	**Low**	Unlikely	Negligible	**Low**
43	26 - Safety (SA)	Over Water Work	• Many areas will require work over water.	• Need to include longshoreman's insurance (already in estimate)	Unlikely	Negligible	**Low**	Unlikely	Negligible	**Low**
44	27 - Construction Risks (CR)	Congested Traffic	• Construction occurs parallel to major roads.	• Lower productivity. •Estimate currently includes productivity markup to account for this. •Schedule is very high level at this time, but it is assumed the lowered productivity is also present in this element.	Certain	Negligible	**Relook at Basis of Estimate**	Certain	Negligible	**Relook at Basis of Schedule**
45	27 - Construction Risks (CR)	Coordination and Event Planning	• Road closures. • Utility outages. • Controlled access to areas.	• Needs to be considered during PED. • Some (but incomplete) information available. • Eastern side appears to be more risky in terms of utilities. • Some private docks are outside of the area where they legally should be. • This is a known / known - things to be coordinated and figured out. •Estimate currently includes productivity markup to account for this. •Schedule is very high level at this time, but it is assumed the lowered productivity is also present in this element.	Certain	Negligible	**Relook at Basis of Estimate**	Certain	Negligible	**Relook at Basis of Schedule**

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46	27 - Construction Risks (CR)	Private Dock Access Construction	• Private docks cross current alignment (9 docks) • Access to marina docks	• Some private docks are outside the area where they legally should be. • This is a known / known - things to be coordinated and figured out. • Legal counsel will need to be consulted if the USACE will have to rebuild private docks (or provide compensation). • Line where dock is permitted is highly variable. • USACE believes states' rights to own marsh land. Assuming compensation to build access up / over wall. Already in real estate contingency. • Contract may have construction for access to private docks. • Assumed to be low cost.	Likely	Negligible	**Low**	Likely	Negligible	**Low**
47	27 - Construction Risks (CR)	Lack of Staging Areas / Working Space	• Require just in time delivery • Workers will need to be brought in from other areas.	• Assumed possible / moderate risk. • Assumed cost risk primarily. Schedule should account for this type of work.	Possible	Moderate	**Medium**	Possible	Negligible	**Low**
48	27 - Construction Risks (CR)	Lodging/Commute for workers	• Workers will need to be brought in from other areas. • Worker parking probably not available. Large workforce may need to be bused on site.	•Lower productivity and additional costs for work force. •Job Office Overhead (JOOH) costs increase. •Needs to be in estimate. •Not considered schedule risk. •Estimate updated to include $15/hr. for subsistence for all trades.	Certain	Negligible	**Relook at Basis of Estimate**	Unlikely	Negligible	**Low**
49	27 - Construction Risks (CR)	Differing Site Conditions	• Seem to happen • Proper planning and surveys need to be undertaken to limit this risk.	• Assumed to be unlikely/marginal.	Unlikely	Marginal	**Low**	Unlikely	Marginal	**Low**
50	27 - Construction Risks (CR)	Long Duration Storm Event	• Impacts to both costs and schedule	• Twenty-five-year event. • Assume 3 month delay.	Unlikely	Negligible	**Low**	Unlikely	Moderate	**Low**
51	27 - Construction Risks (CR)	Short Duration Storm Event	• Impacts to both costs and schedule	• Assume 3 year event. • Assume 2 week delay.	Possible	Negligible	**Low**	Possible	Negligible	**Low**
52	27 - Construction Risks (CR)	Congested Work Area	• Lower productivity and congested working conditions should be included within the estimate.	• Should be included in the estimate. •Estimate currently includes productivity markup to account for this. •Schedule is very high level at this time, but it is assumed the lowered productivity is also present in this element.	Certain	Negligible	**Relook at Basis of Estimate**	Certain	Negligible	**Relook at Basis of Schedule**
53	27 - Construction Risks (CR)	Navigation Traffic Congestion Due to Construction	• Barges filled with construction materials may impact other navigation traffic. • West Ashley location	• Should be included in the estimate. •Estimate currently includes productivity markup to account for this. •Schedule is very high level at this time, but it is assumed the lowered productivity is also present in this element.	Certain	Negligible	**Relook at Basis of Estimate**	Certain	Negligible	**Relook at Basis of Schedule**
54	29 - Turnover (TO)	Future O&M Implications	• System maintenance should be considered.	Will there be resistance from City to take over operation and maintenance (O&M) on substantially completed segments of the projects prior to the projects being entirely completed? If so, there could be additional cost incurred by the government for O&M during construction.	Possible	Moderate	**Medium**	Possible	Negligible	**Low**

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55	30 - Real Estate	Unknown Alignments	• Final alignment is unknown. • Need to obtain lands and easements for alignment and construction activities. •	• See 02 - Scope and Objectives (SC) - scope refinement. Captured there.	Unlikely	Negligible	**Low**	Unlikely	Negligible	**Low**
56	31 - Geotech/Geology	Soil Stability & Foundations	• Little to no effort has been made to explore the site for soil conditions. This area likely has many fine grains that drain slowly.	Due to expedited (3x3) planning process, soil explorations could not be completed so existing subsurface information was used to estimate top of Copper Marl along the alignment. Copper Marl is bearing layer for pile founded structures and piles will be embedded in formation, assumed 5 ft. Top elevation variation: LV & ML: 5 feet: HV: 10 feet. Project Cost: Likelihood = Likely; Impact = Marginal Using $3M - $6M as possible bracket. Project Schedule: Likelihood = Possible; Impact = Marginal	Likely	Marginal	**Medium**	Possible	Marginal	**Low**
57	31 - Geotech/Geology	Lack of Soil Data on Alignment	• Potential for unsuitable soil along Ashley River. This area was formerly a landfill. • Concern with under seepage. Sheet pile may need to be deeper. Potential difficulty driving piles with rubble type material. • High rise construction exists in this area. The Joe Riley Jr. Park is sinking and requires routine maintenance.	Assumed similar to 33. Already captured above in similar concept.	Unlikely	Negligible	**Low**	Unlikely	Negligible	**Low**
58	31 - Geotech/Geology	Advanced Modeling - Soil Structure Interaction	• Complex design analysis. • Requires special software with a subject matter expert (SME) to run analysis.	• Schedule risk primarily. • USACE has large internal Geotech community. Assume that PDT will be able to find USACE SME. Risk not modeled but noted as a risk to consider. Soil Structure Interaction (SSI) will require soil data including both in situ and laboratory testing to be completed which will delay the start of SSI modeling. SSI modeling durations: LV & MV: 3 to 6 months; HL: 9 months. Project Cost: Likelihood = Possibly; Impact = Marginal (SSI may indicate larger or deeper piles are needed) Project Schedule: Likelihood = Possible; Impact = Marginal	Possible	Marginal	**Low**	Possible	Marginal	**Low**
59	32 - Life Safety	Gate Closures	• Good equipment will be required to be certain that gates will close. • Potential for equipment increases (cost).	• Variation in cost and schedule is included in Risk #30.	Unlikely	Negligible	**Low**	Unlikely	Negligible	**Low**

Meeting 7/1/2021 Updated Cost 1/28/2022					Project Cost			Project Schedule		
REF	**Risk Type**	**Risk/Opportunity Event**	**Risk Event Description**	**PDT Discussions on Impact and Likelihood**	**Likelihood (C)**	**Impact (C)**	**Risk Level (C)**	**Likelihood (S)**	**Impact (S)**	**Risk Level (S)**
60	22 - General Technical Risks (GR)	PED Duration Likely to Extend	Could be delays due to… • Design milestone reviews • ENV coordination/surveying during design • Potential changes in design • Supplemental NEPA • Aesthetic features • RR Crossings and Coordination	The base schedule assumes 3-years of PED. LV: Assume 3-years of PED = 0 Mo. Variance ML: Assume 3 Mo. Delay HV: Assume 6-Mo. delay which may be mitigated by the real estate schedule duration as well. It is believed that this risk is simply a summary of all other risks. It will not be modeled as a result.	Likely	Moderate	**Medium**	Likely	Significant	**High**

Cost MCX ATR Certification

WALLA WALLA COST ENGINEERING MANDATORY CENTER OF EXPERTISE

COST AGENCY TECHNICAL REVIEW

CERTIFICATION STATEMENT

For Project No. 474899

SAC – Charleston Peninsula Coastal Flood Risk Management Study

The Charleston Peninsula Coastal Flood Risk Management Study, as presented by Charleston District, has undergone a successful Cost Agency Technical Review (Cost ATR), performed by the Walla Walla District Cost Engineering Mandatory Center of Expertise (Cost MCX) team. The Cost ATR included study of the project scope, report, cost estimates, schedules, escalation, and risk-based contingencies. This certification signifies the products meet the quality standards as prescribed in ER 1110-2-1150 Engineering and Design for Civil Works Projects and ER 1110-2-1302 Civil Works Cost Engineering.

As of February 16, 2022, the Cost MCX certifies the estimated total project cost:

FY22 Project First Cost: $1,132,096,000
Fully Funded Amount: $1,363,646,000

Cost Certification assumes Efficient Implementation (Funding). It remains the responsibility of the District to correctly reflect these cost values within the Final Report and to implement effective project management controls and implementation procedures including risk management through the period of Federal Participation.

®

Michael P. Jacobs, PE, CCE
Chief, Cost Engineering MCX
Walla Walla District