US Army Corps of Engineers®

Charleston District

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

Charleston, South Carolina

ENGINEERING APPENDIX - B

AUGUST 2021

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4. COASTAL MODELING 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.

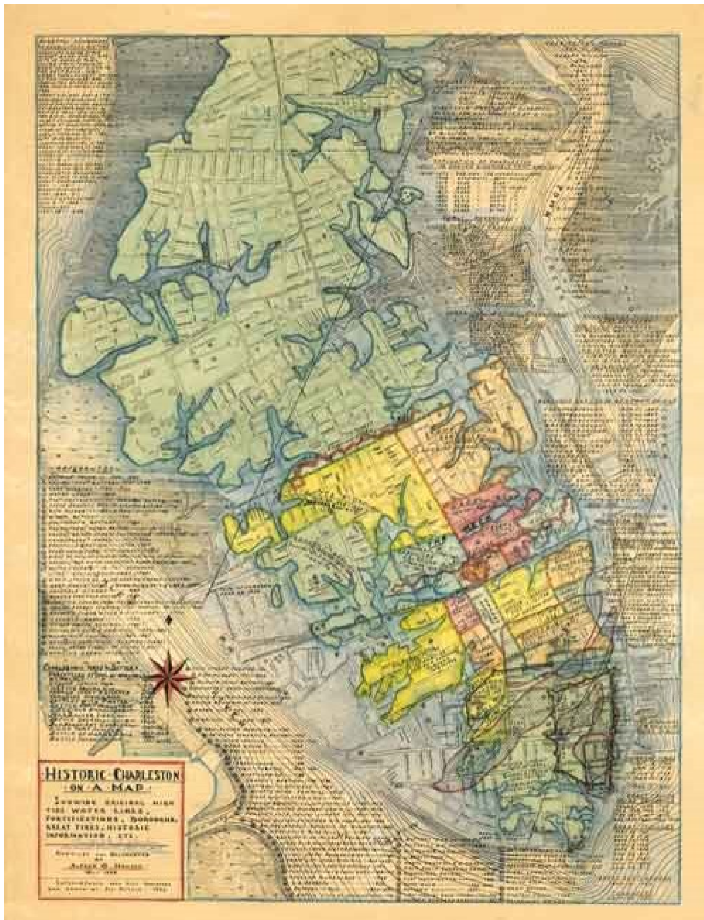

Figure 1.2: 1844 Map of Charleston

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." The city is presently raising the low battery to match the high battery at elevation 9NAVD88.

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.

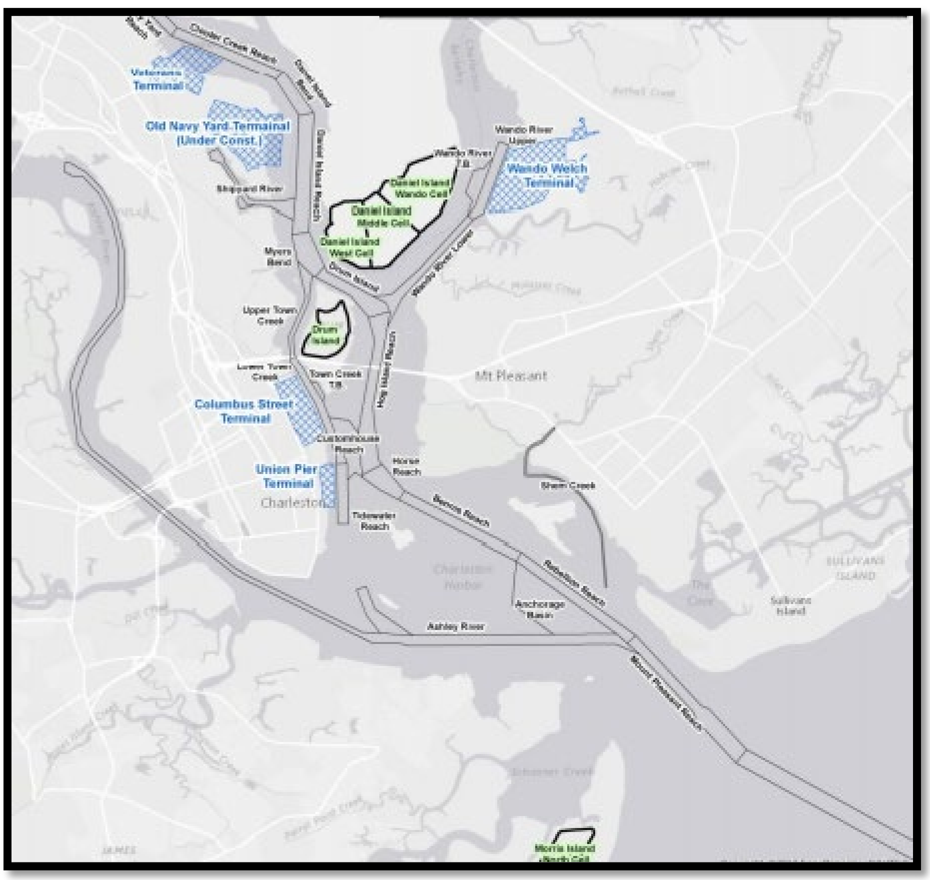

Figure 1.3 Charleston Harbor Navigation Channel

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.

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.

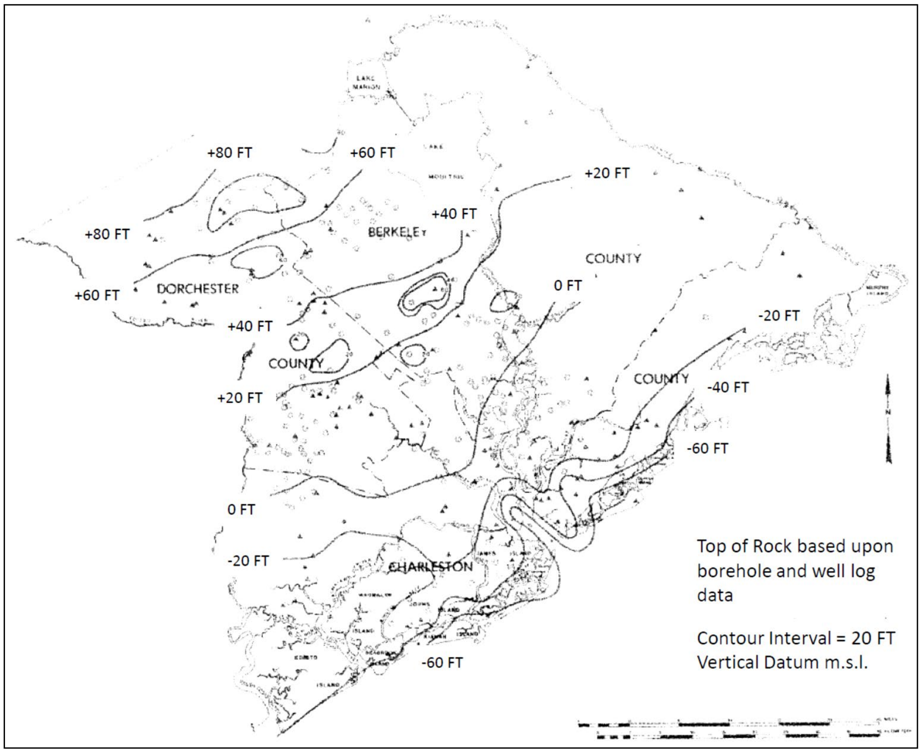

Figure 2.2.1: Structure contour map showing top of Cooper Formation, from Park (1985).

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 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 deem to be within a high seismic hazard zone as indicated in Figure 2.2.2.1. This area is known as 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 on 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.

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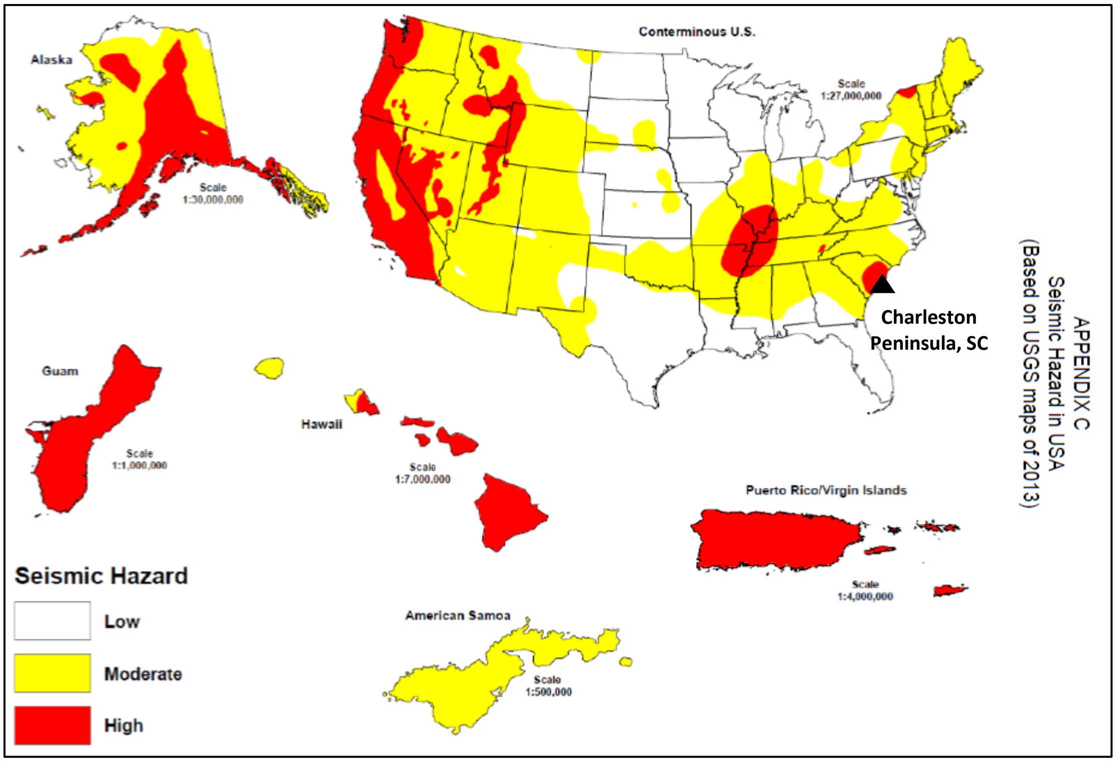

Figure 2.2.2.1: Project location shown on seismic hazard map of the USA, from ER-1110-2-1806.

2.2.2.1 Ground Motions

The seismic evaluation provided a range of ground motions for various events. A 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).

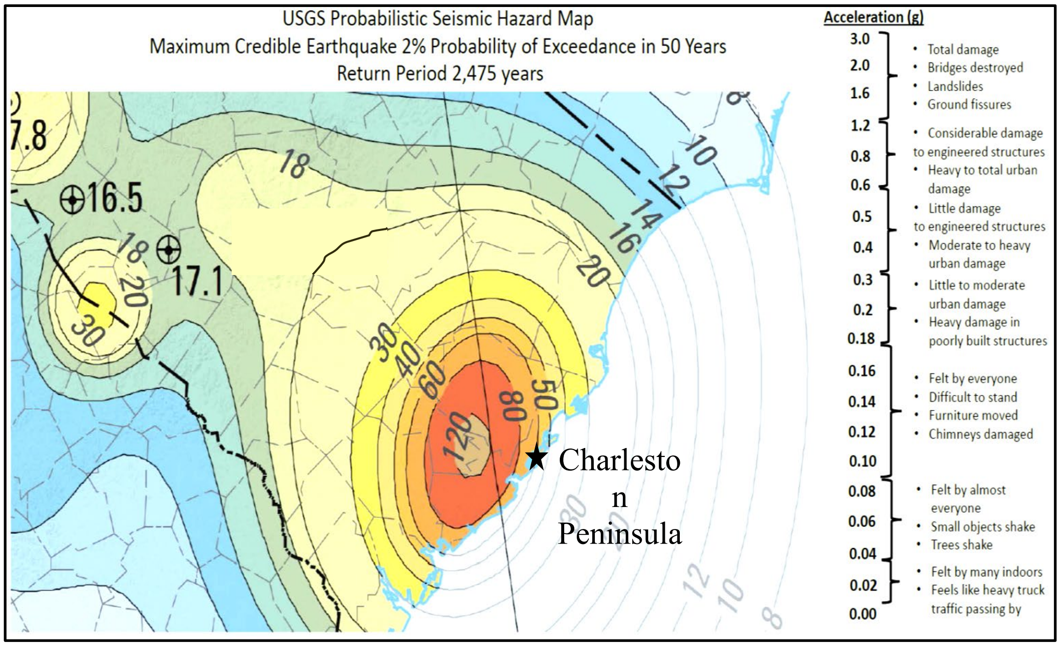

Figure 2.2.2.1.1: USGS Seismic Hazard Map, PGA, 2% Probability of Exceedance in 50 Years, from Peterson et al. (2015).

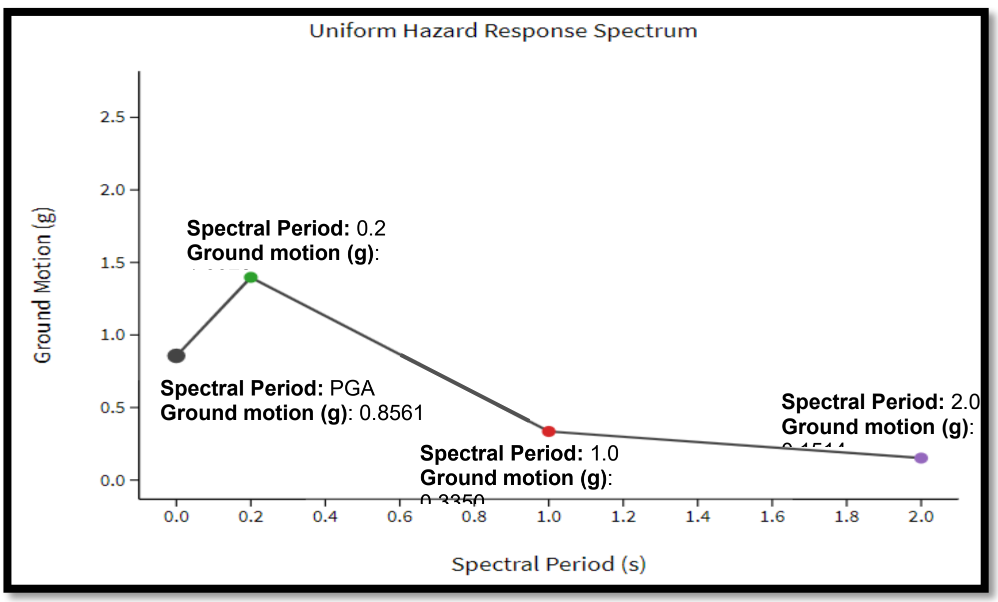

Figure 2.2.2.1.2: Uniform Hazard Response Spectrum predicted for the project site showing PGA with 2% in 50 years AEP (2,475 return period).

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).

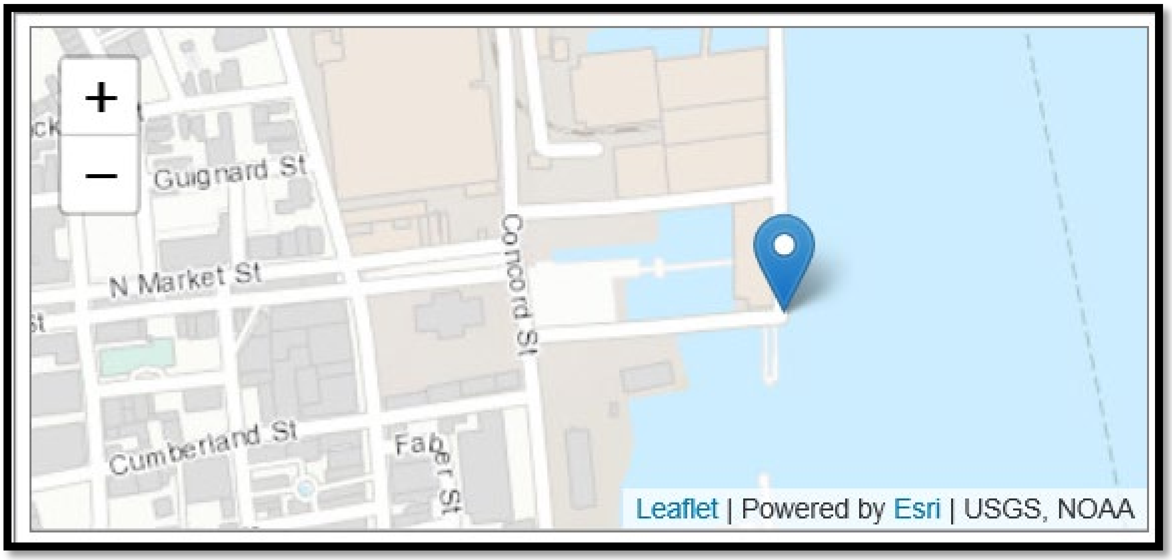

Figure 2.4.1 Location of NOAA Gage 8665530

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)

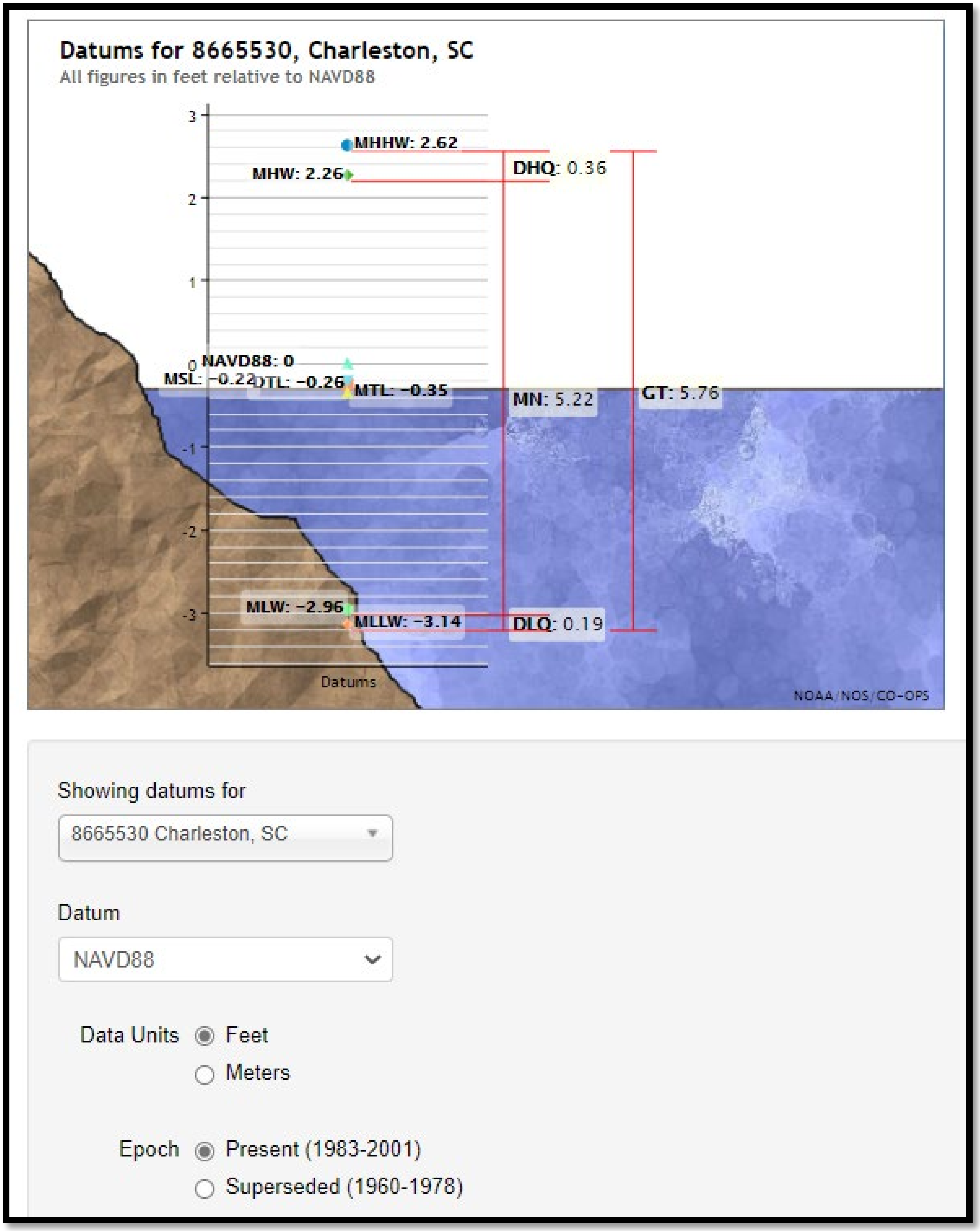

Figure 2.4.2 Tide Range Station 8665530

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.

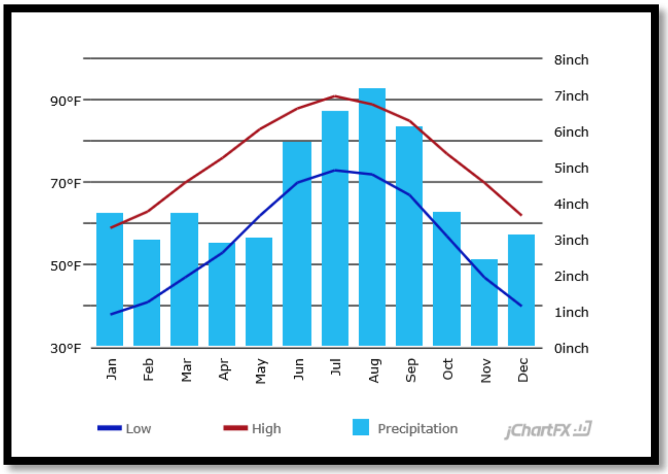

Figure 3.1.1 Charleston Temperature and Precipitation

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 2900). 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.

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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).

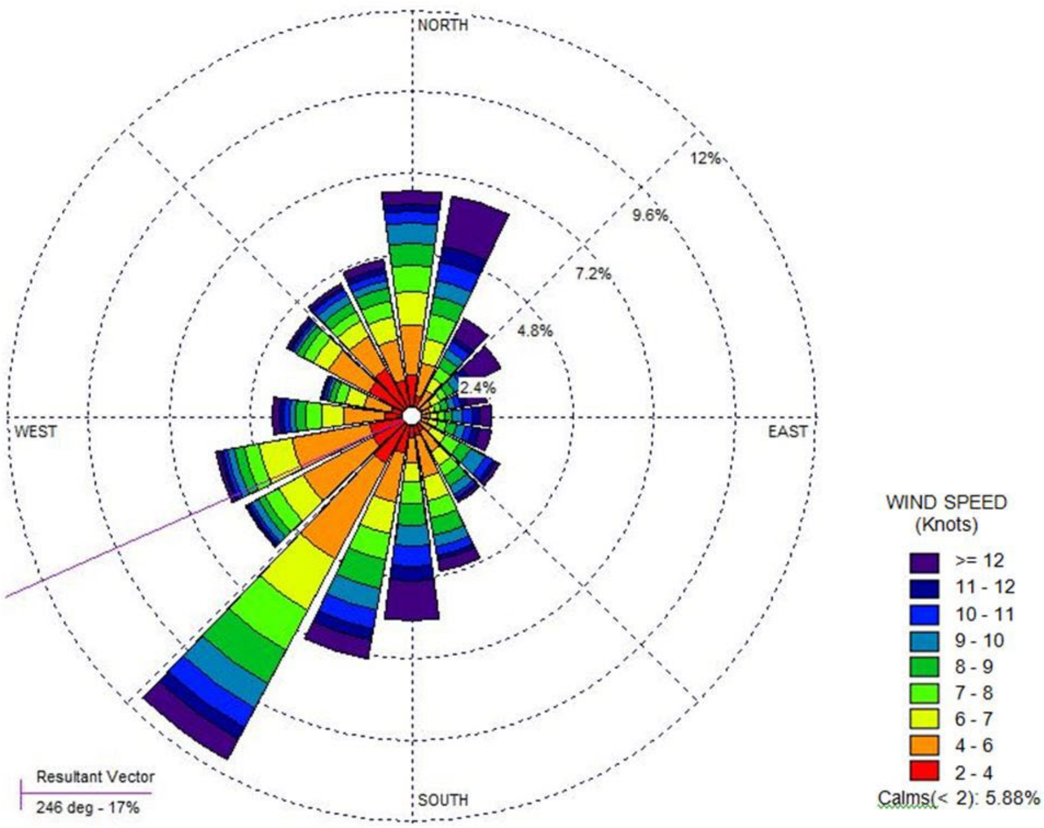


Figure 3.3.1. Wind Rose for Charleston Harbor Depicting Wind Direction and Speed Frequency

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.

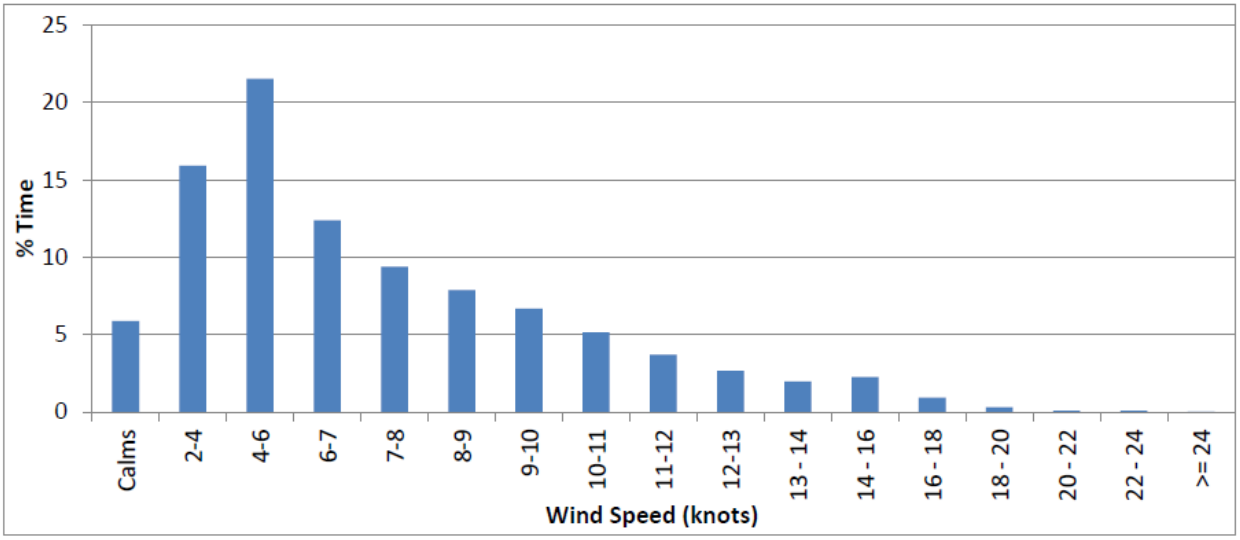


Figure 3.3.2 Wind Speed Frequency Distribution in Charleston Harbor from all directions

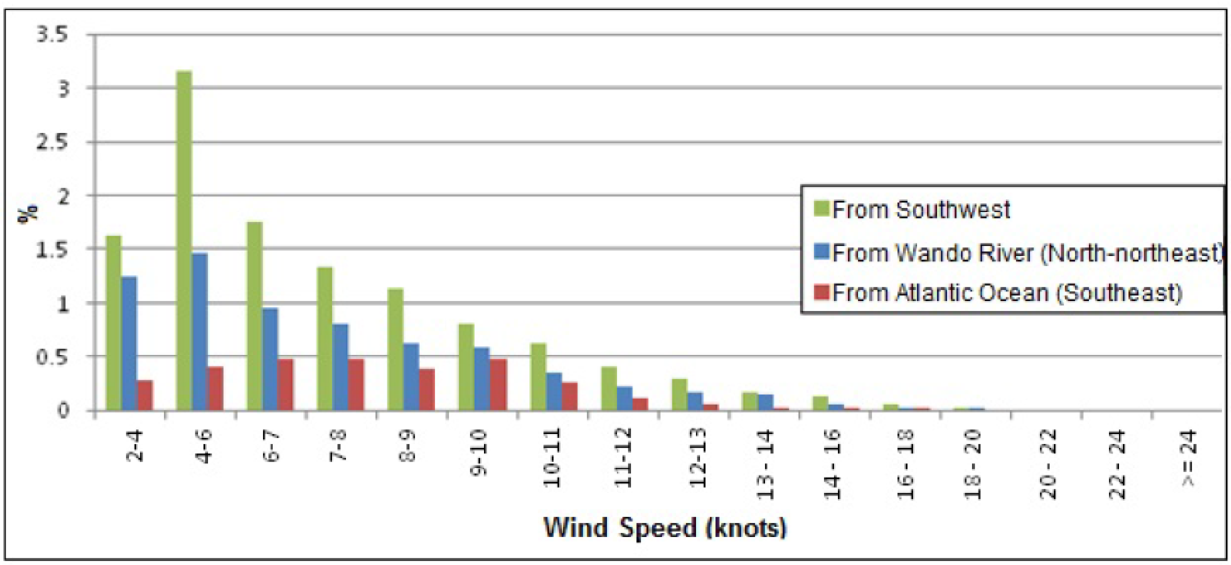


Figure 3.3.3 Wind Speed Frequency Distribution in Charleston Harbor comparing three key directions

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).

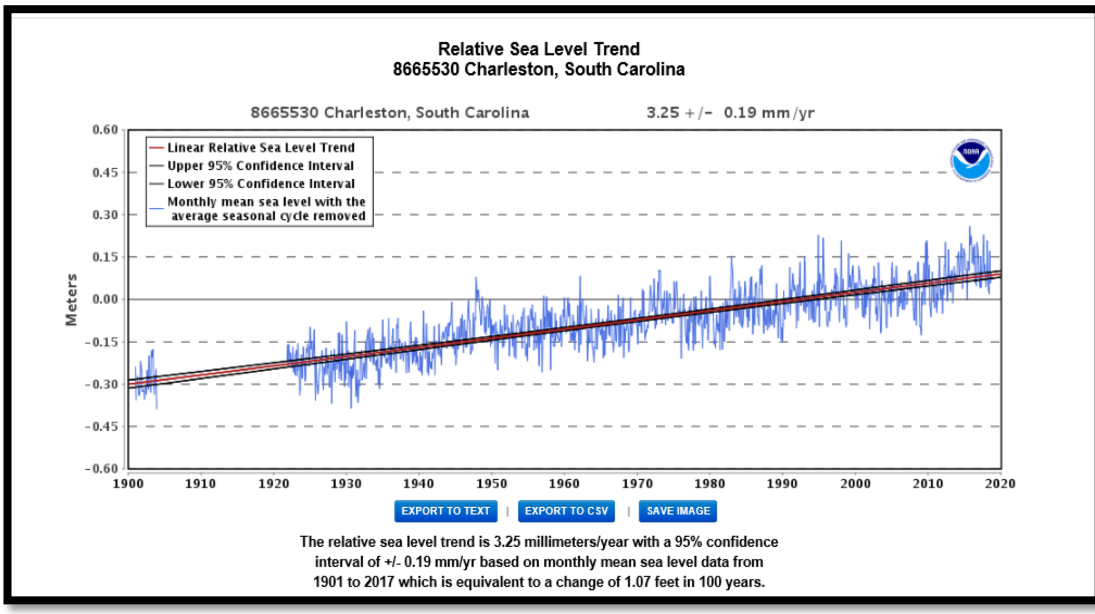


Figure 3.4.2.1 Observed Sea Level Rise at Charleston Harbor Gage

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.

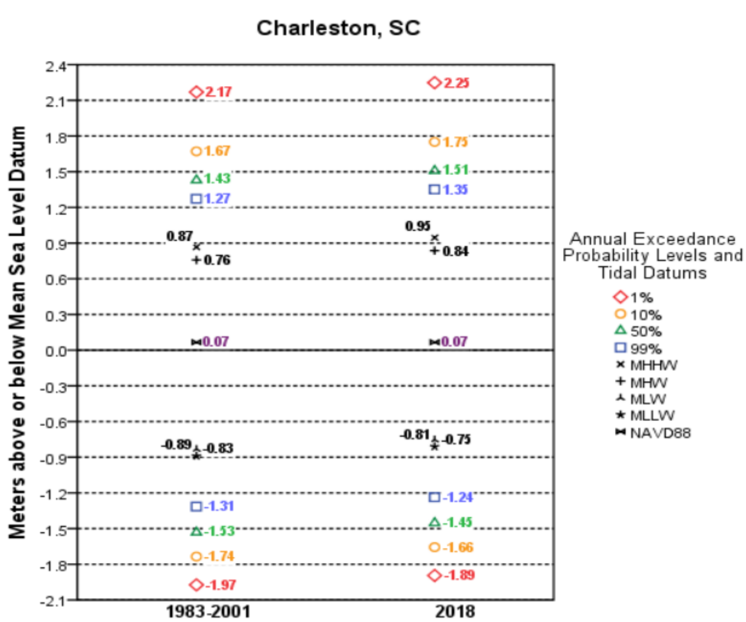
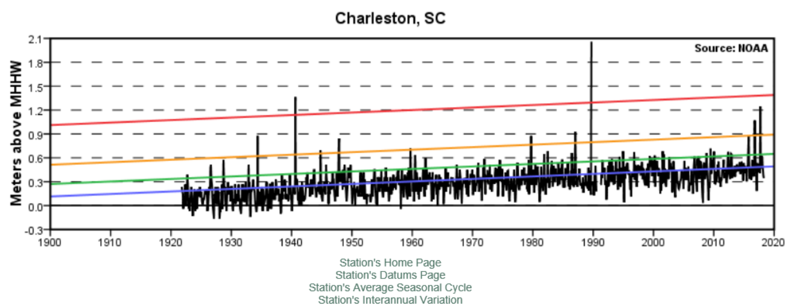


Figure 3.4.3.1 Exceedance Probability Levels and Tidal Datum of 8665530 Charleston, Cooper River Entrance, SC

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)



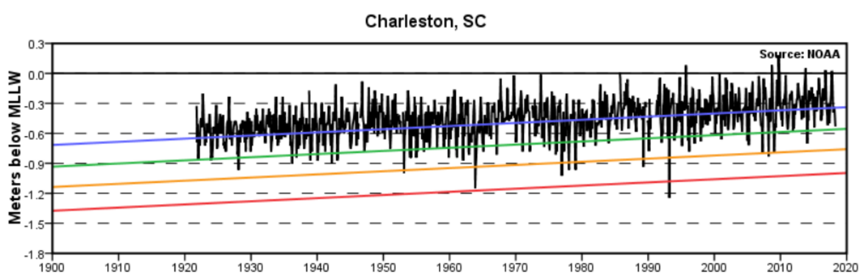


Figure 3.4.3.2 Seasonal and Interannual Variation of Gage 8665530 Extreme water Levels

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

Version of Data :	05/17/2017
ID:	8665530
Reference Datum:	NAVD88
Name:	Charleston, SC
HAT:	4.12 (ft)
MHHW:	2.62 (ft)
MHW:	2.27 (ft)
MSL:	-0.22 (ft)
MLW:	-2.95 (ft)
MLLW:	-3.14 (ft)
NAVD88:	0.00 (ft)
EWL Type:	NOAA GEV (NAVD88)
EWLs adjusted to 2019 using the historic rate.	
*1%:	7.18 (ft)
2%:	6.59 (ft)
5%r:	5.95 (ft)
10 %:	5.54 (ft)
20%:	5.18 (ft)
50%:	4.75 (ft)
Yearly:	4.23 (ft)
Monthly:	NaN (ft)
From:	1921
To:	2007
Years of Record:	86

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).

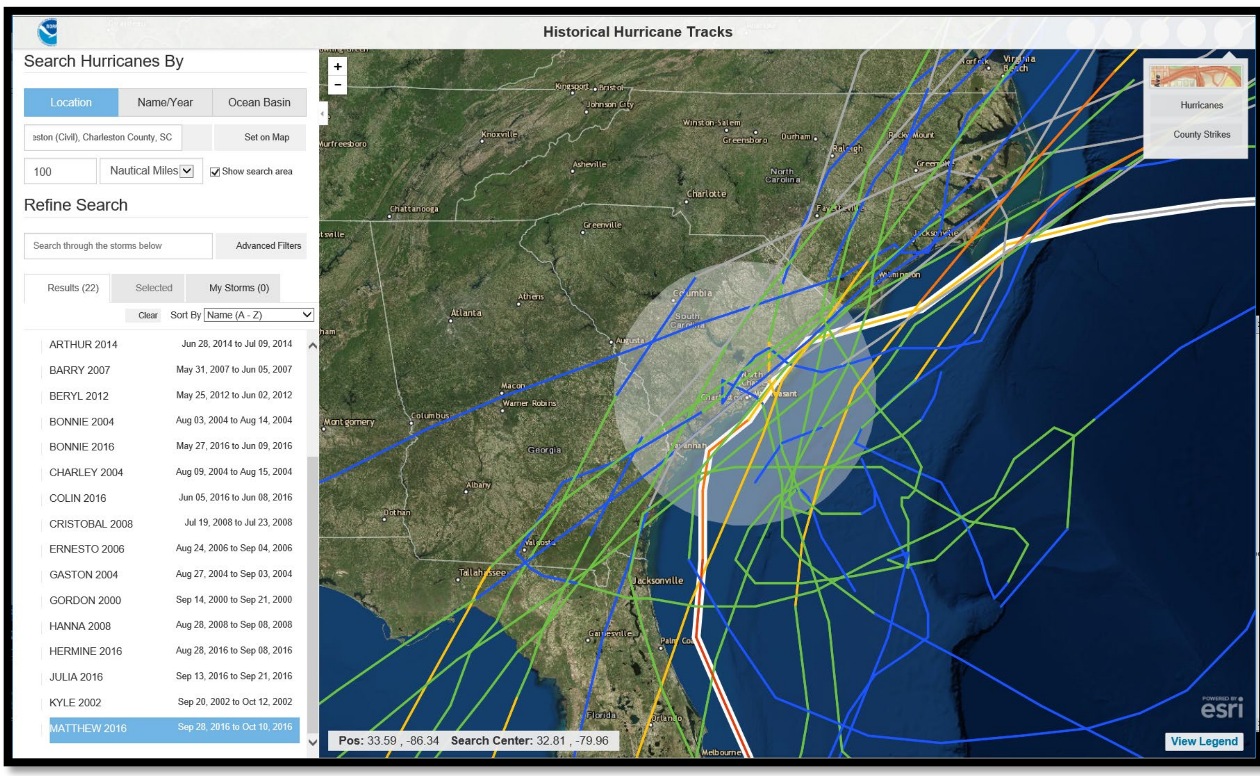


Figure 3.5.1 Twenty-two storms passed within 100 nautical miles of Charleston between 2000 and 2019.

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.

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.

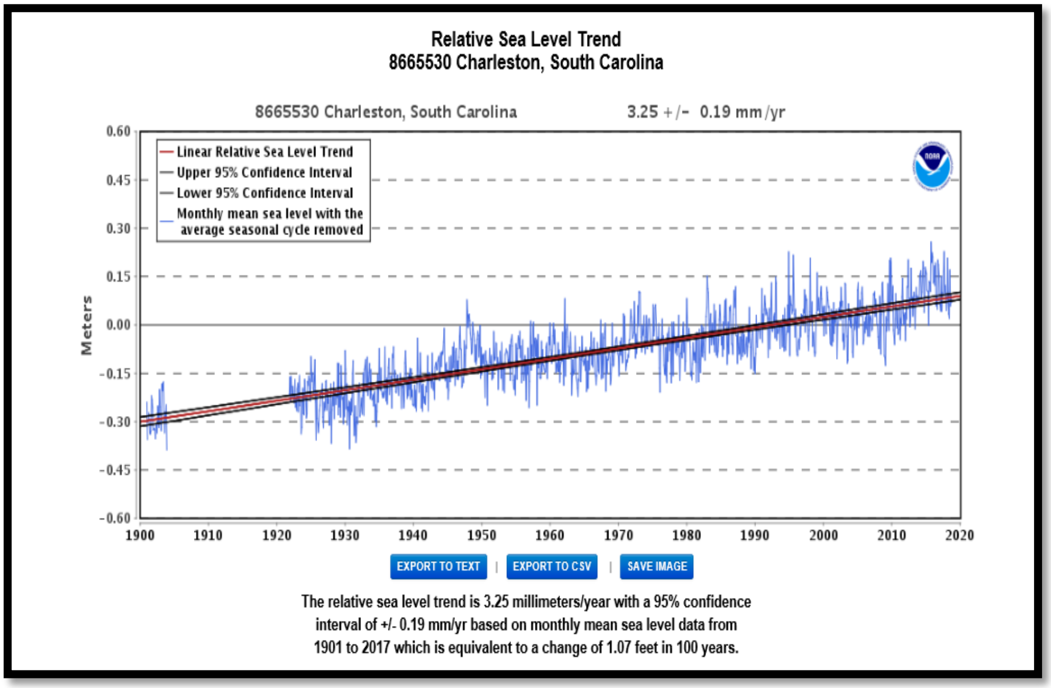


Figure 3.6.2 Relative Sea Level Trend

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).

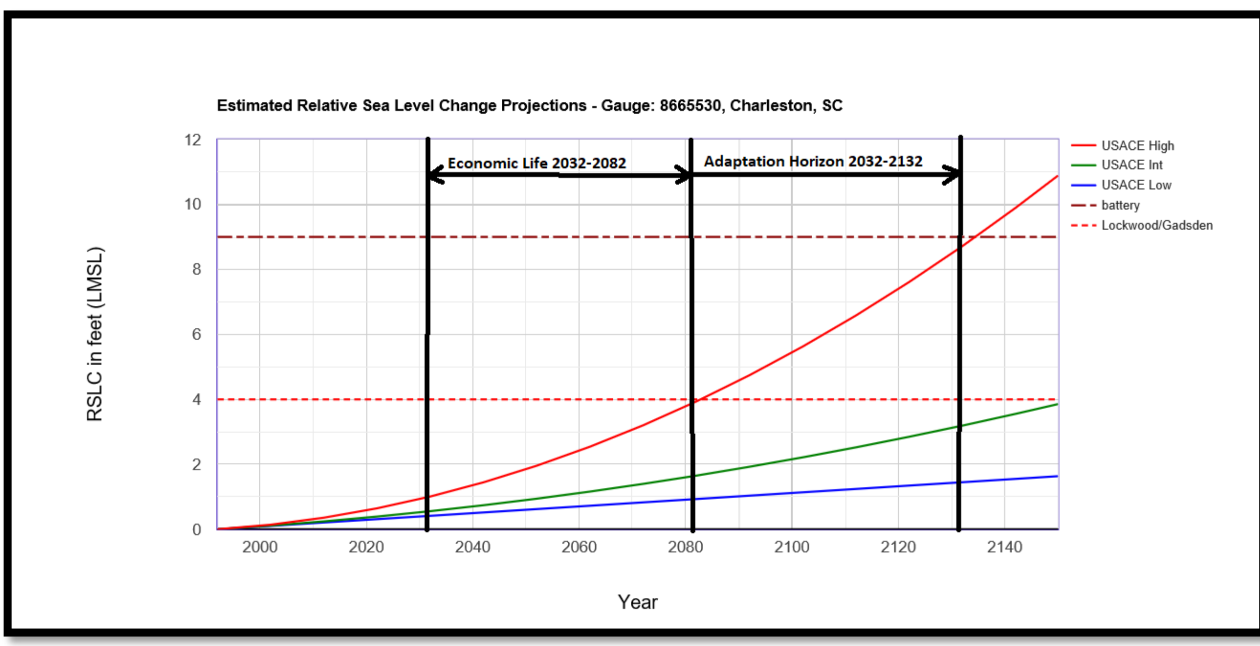


Figure 3.6.3 Low, Intermediate and High Sea Level Projection Gage 8665530

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 study used one rate of sea level change for determination of the recommended alternative and the measure of elevation.

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.

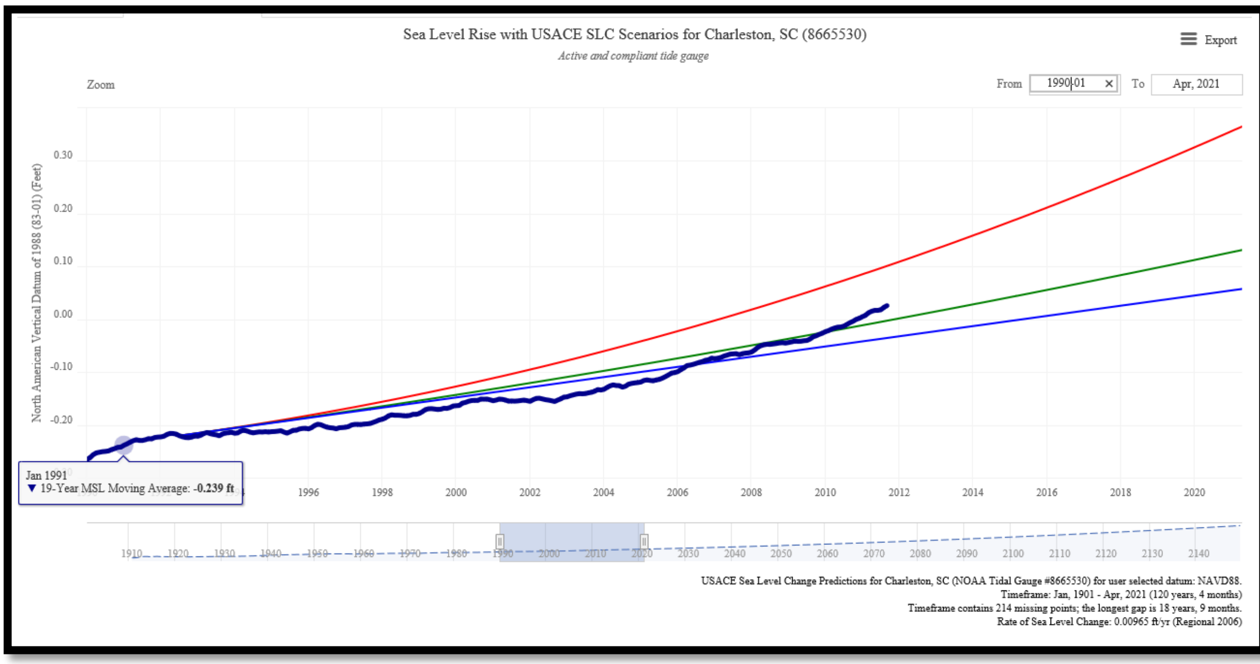


Figure 3.6.4 Sea Level Tracker Charleston SC (NOAA Station 8665530)

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 ERDC report located in Sub-Appendix 5 COASTAL MODELING SUB-APPENDIX.

ERDC was asked to run STWAVE and ADCIRC to generate time series still water elevations for input into the G2CRM model. 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.

- Existing Condition is the topography and bathymetry that was provided by FEMA for the latest Flood Insurance Study. The only modification was the mesh in the area of the peninsula (Figure 4.1 and 4.2). Results are mean sea level (MSL).

- Future Without condition only included the raising of the existing low battery wall to the same elevation (9 NAVD88) as the existing high battery wall. From the city of Charleston Flooding and Sea Level Rise Strategy: "*In early 2019, the City will begin an extensive reconstruction project of the iconic Low Battery Seawall to replace and raise the seawall to account for sea level rise projections. It was 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. The City's Design Division studied this site and used extensive stakeholder input and technical data to suggest general ideas and design concepts for the new seawall which are viewable online at www.charleston-sc.gov/SLR. 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*. " Results are in MSL and then converted for G2CRM to NAVD88. It does not include the sea level change as the G2CRM model has the three sea level curve formulas imbedded in the model. Only the historic rate of rise is needed for the G2CRM model to address sea level change. The G2CRM model also incorporates tide in the damage assessment.

- Future With a breakwater include the Future Without change to the low battery and a wave attenuator at the battery. The highest wave generation during storm events, based on past experiences, is at the battery, thus a wave attenuator was included in one alternative. ERDC ran the simulation of one size breakwater and the PDT Coastal Engineer ran two other sizes.

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. Statistical analysis using the JPM-OS determined that application of a Monte Carlo method to provide a random initial tidal level at the start of each production run would account for tidal variations in the storm surge analysis. Each production run began with a random tide phase in order to vary the phasing of the tide relative to the storm. The random phases were derived from a 60-day tide simulation from August 1 to September 30, 2010, which was preceded by a 15-day spin up period necessary for the model forcing to ramp up.

To account for steric effects, the project team calculated the seasonal water level change induced by the solar annual (SA) and solar semi-annual (SSA) tidal constituents during the 60-day period at Charleston Harbor. The amplitude, phase, and frequency of the constituents were obtained from the National Oceanic and Atmospheric Administration (NOAA) (NOAA, 2013). The project team determined the mean steric effect over the 60-day period of the simulations by integration (as sine waves with time = 0 on January 1 of each year) to obtain a total increase of 2.75 inches (7 cm) above mean sea level (MSL)." See the COASTAL MODELING SUB-APPENDIX and the ERDC modeling report for explanation of the ADCIRC and STWAVE modeling.

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 appendix Interior Hydrology. Because nonlinear residual (NLR) was proven to be very weak, effects shown by changes in sea level between the 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 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.

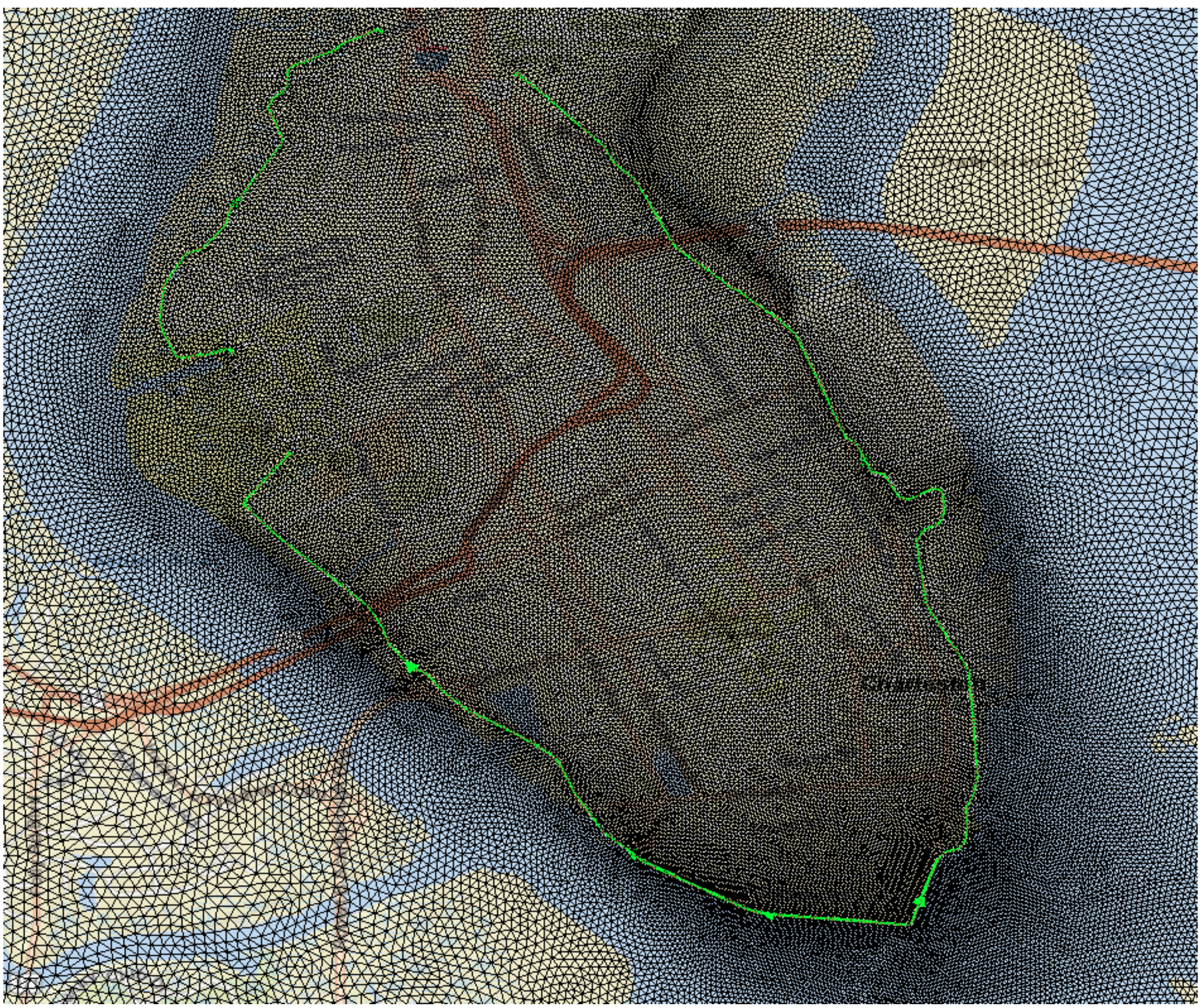


Figure 4.1. ADCIRC mesh used for FWP simulations with proposed 12 ft NAVD88 wall shown in light green.

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), so areas 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.

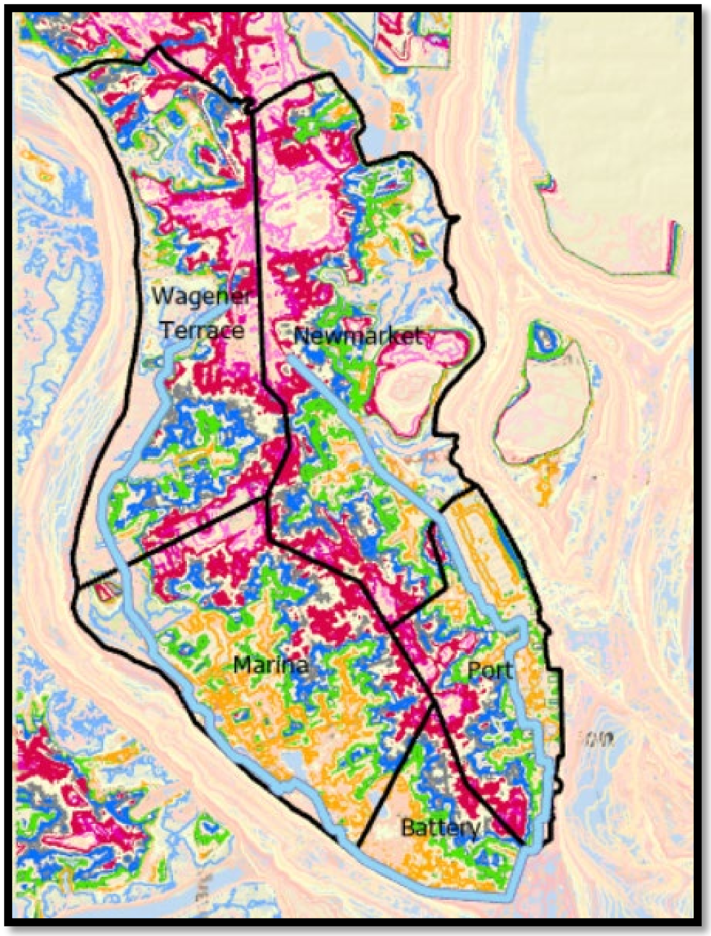


Figure 5.1.1 Map depicting Model Areas

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 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 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' properties based 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.

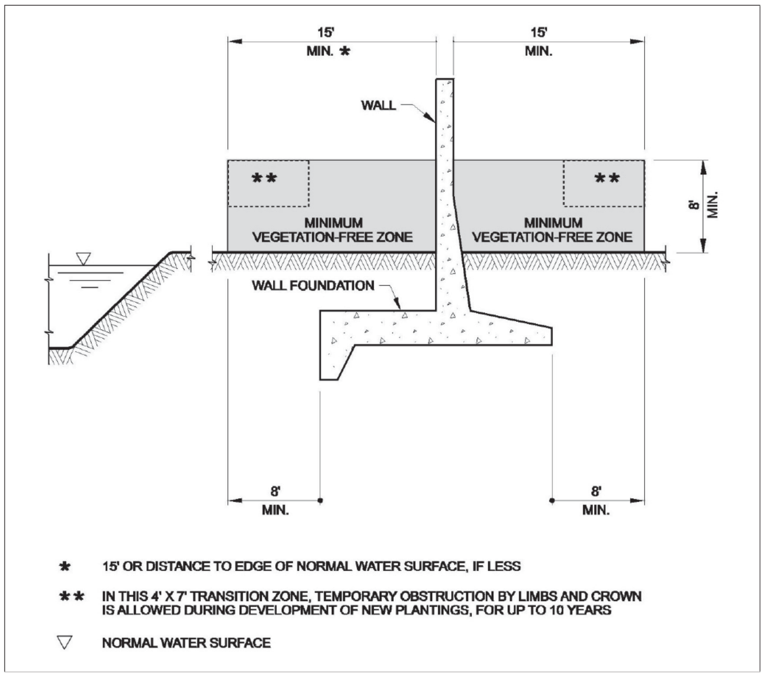


Figure A-16. Inverted-T Type Floodwall.

- 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. Effort 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. The remainder of the footprint has not changed since the draft report.

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.

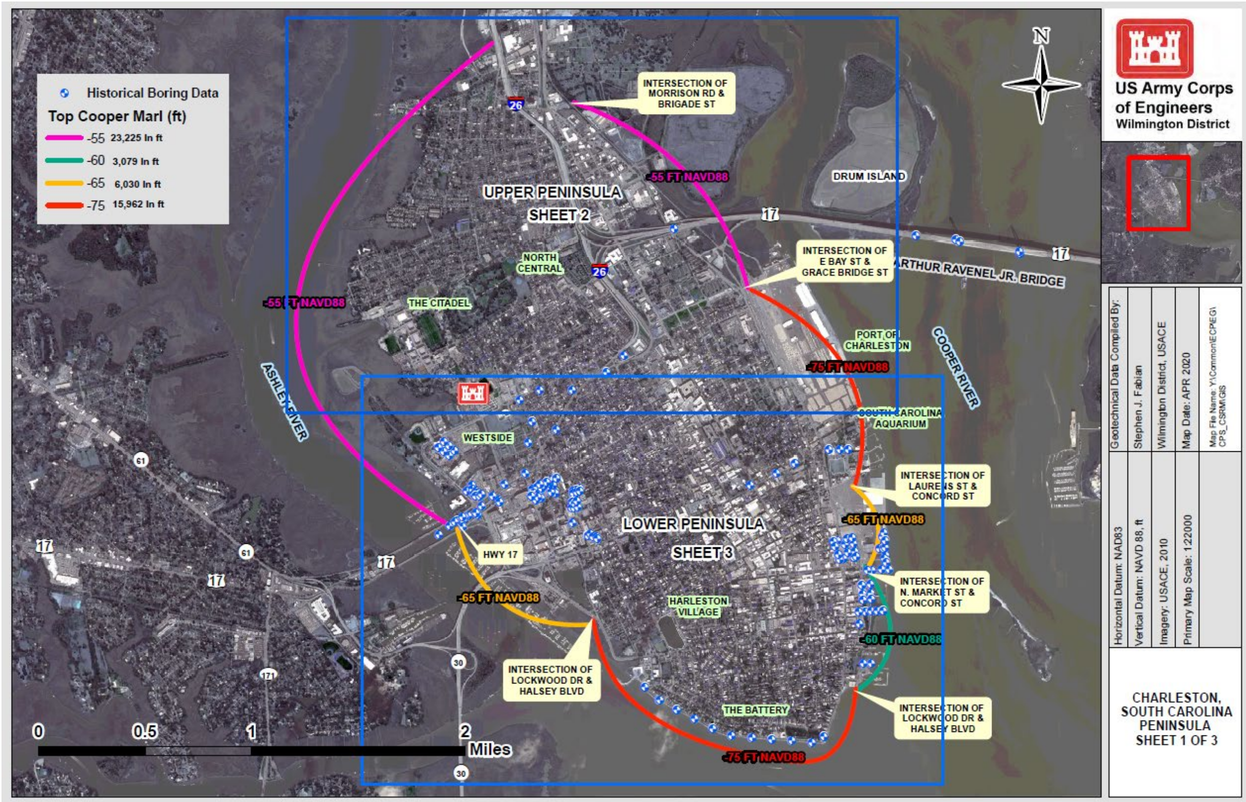


Figure 5.4.1: Assumed Top of Cooper Marl

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.

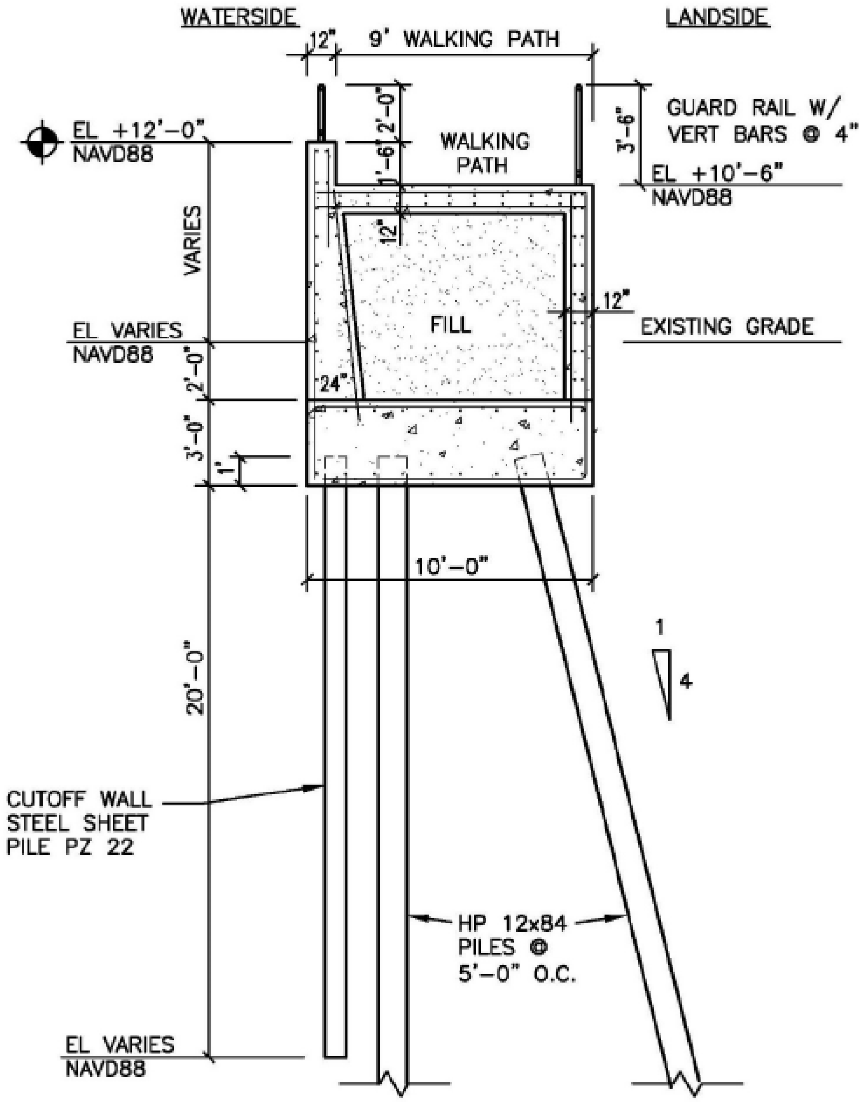


Figure 5.5.1 Typical T-Wall

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.

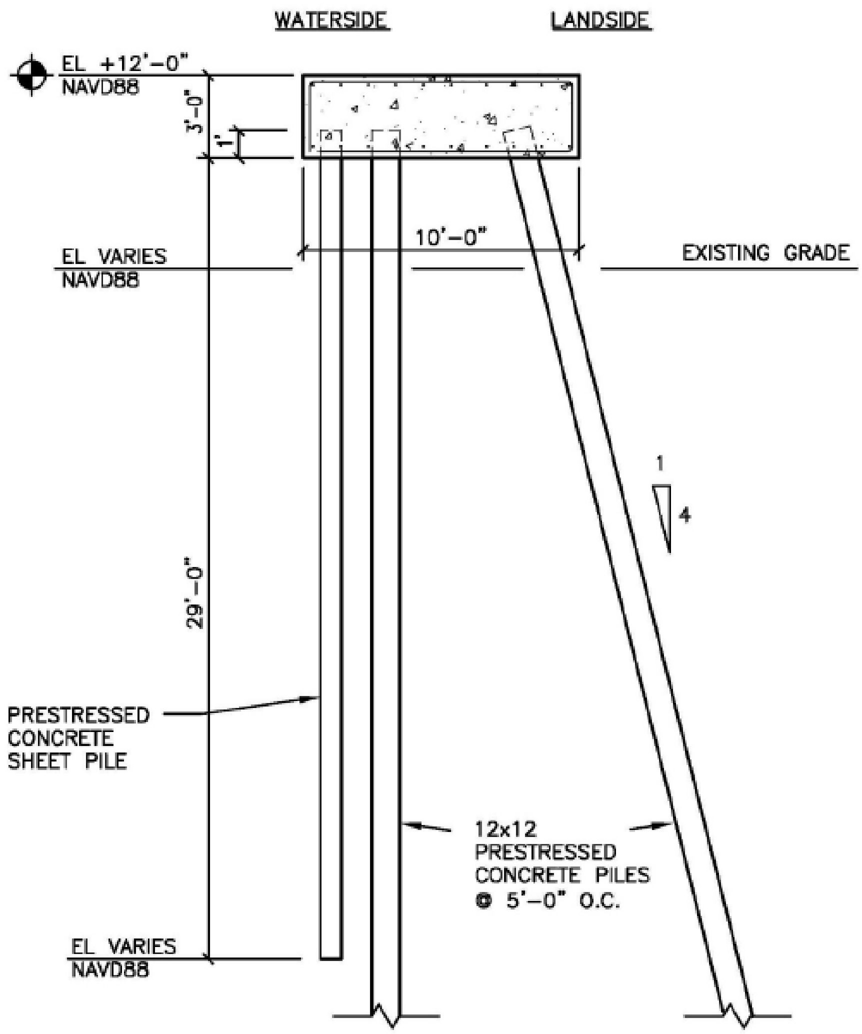


Figure 5.5.2 Typical Combo Wall

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

5.5.6 LOW BATTERY WALL

The Low Battery Wall is being renovated by the City of Charleston and provides a level of protectionto EL 9 ft NAVD 88. The designer of record stated that the new Low Battery Wall was designed to provide a level of protection of 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.

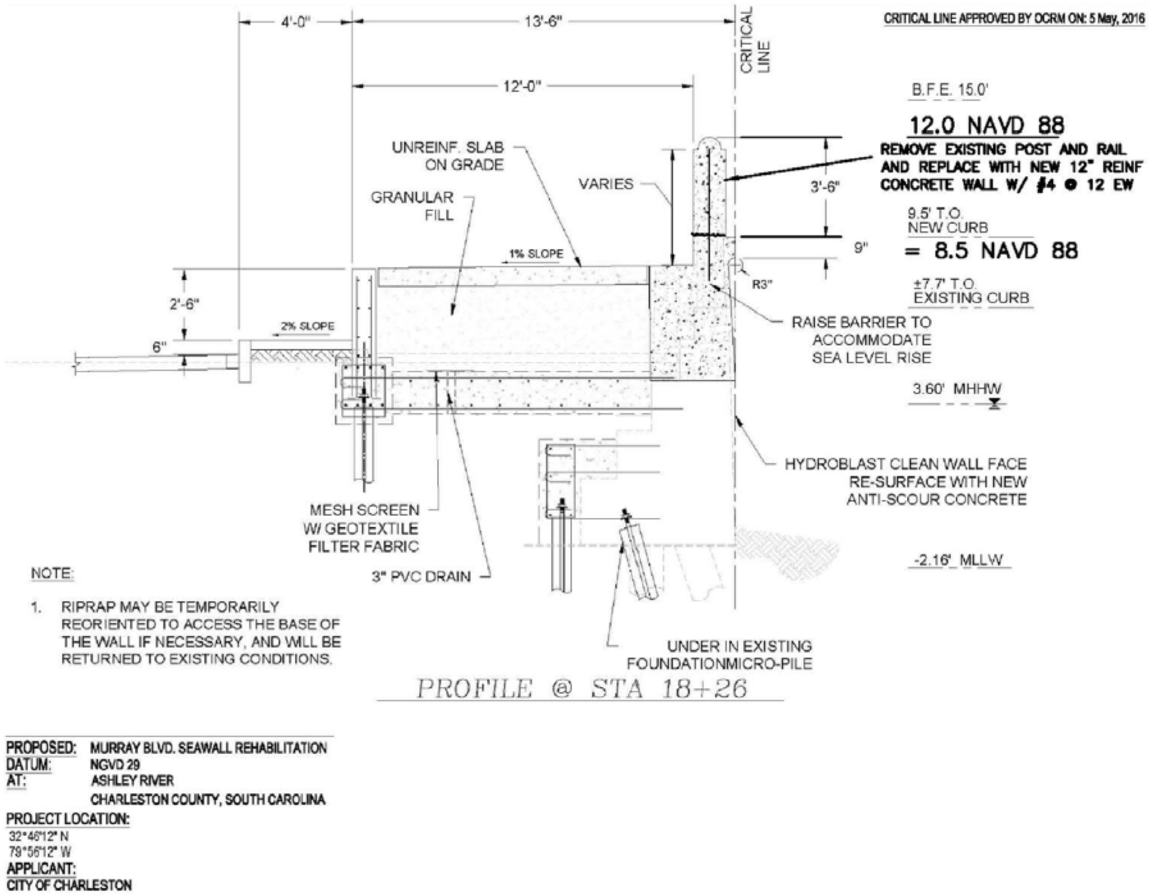


Figure 5.5.3 Typical section of Low Battery Wall Upgrade to EL 12.0 NAVD 88 flood protection

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.

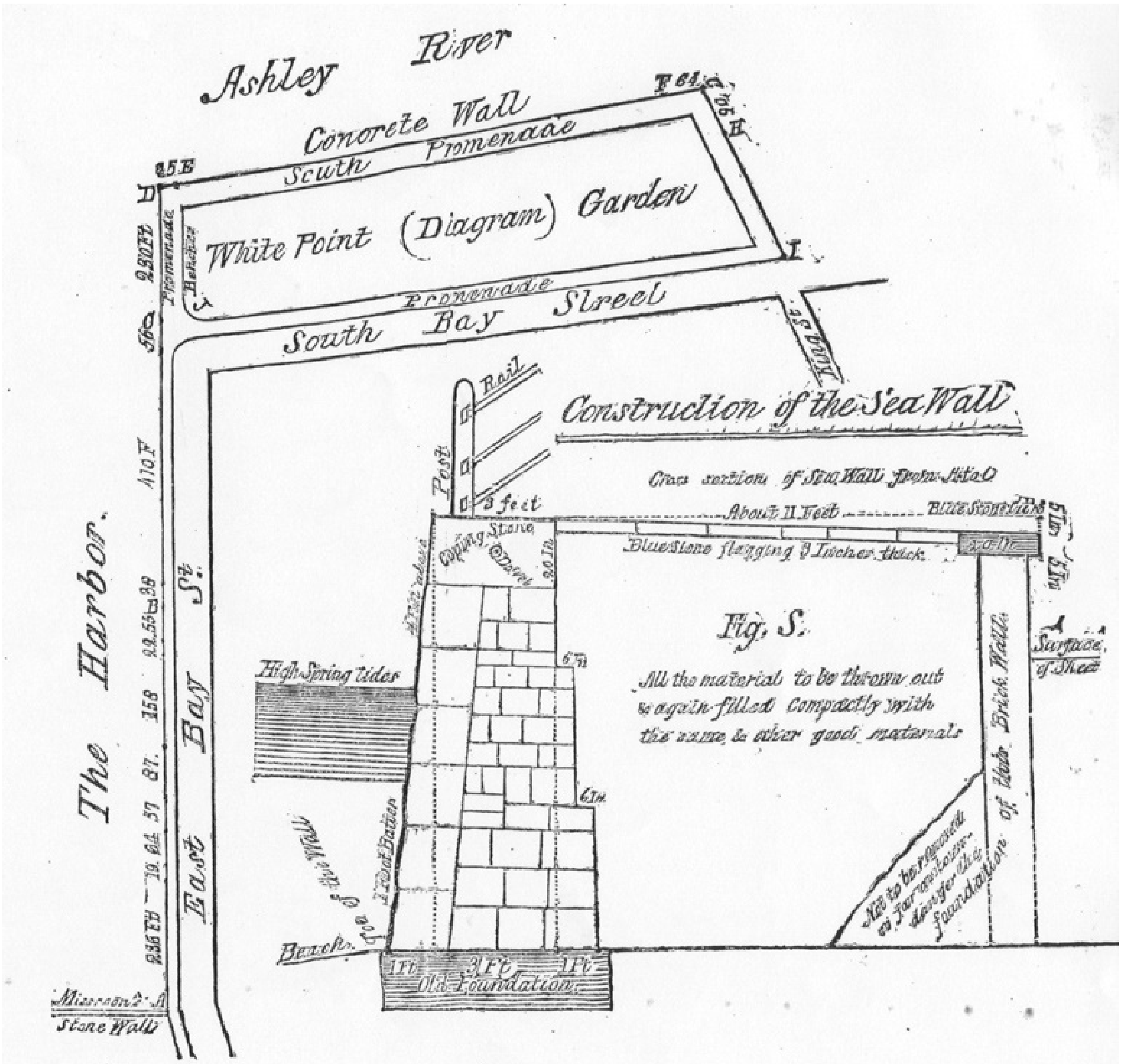


Figure 5.5.4 Existing High Battery Wall

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.

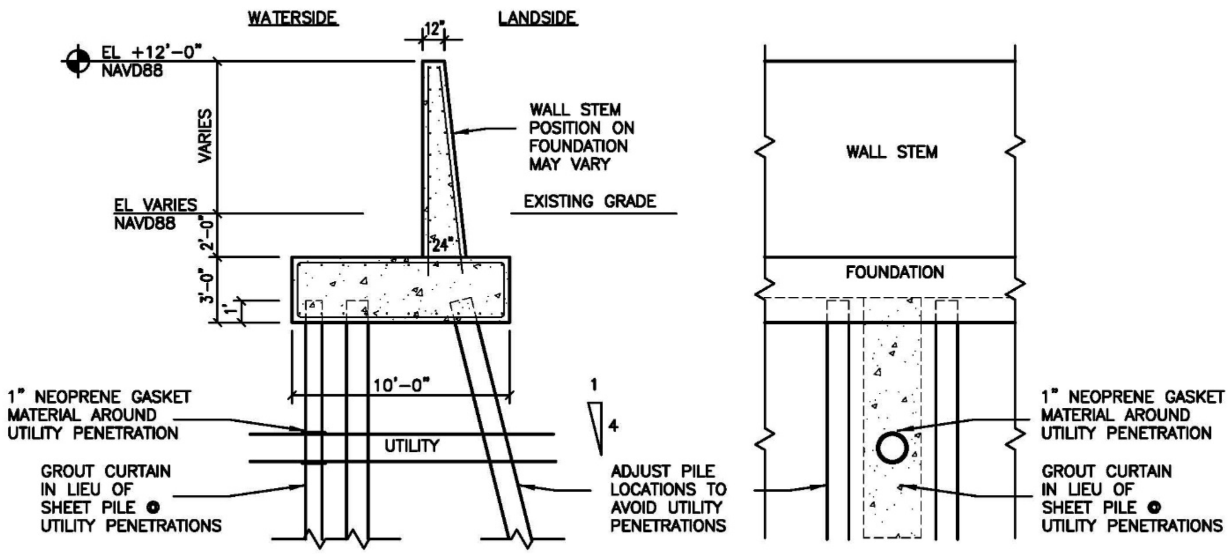


Figure 5.5.5 Grout Curtain Cutoff Wall Construction at Utility Crossings

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 water tight 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 51 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 water tight 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 water tight 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 water tight 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 2 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 backhoe may be required to lift the sections into place.

5.6.1.4 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.

5.6.1.5 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 restoredalong with adding a 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 5.6.4 Aerial View at Coast Guard Base

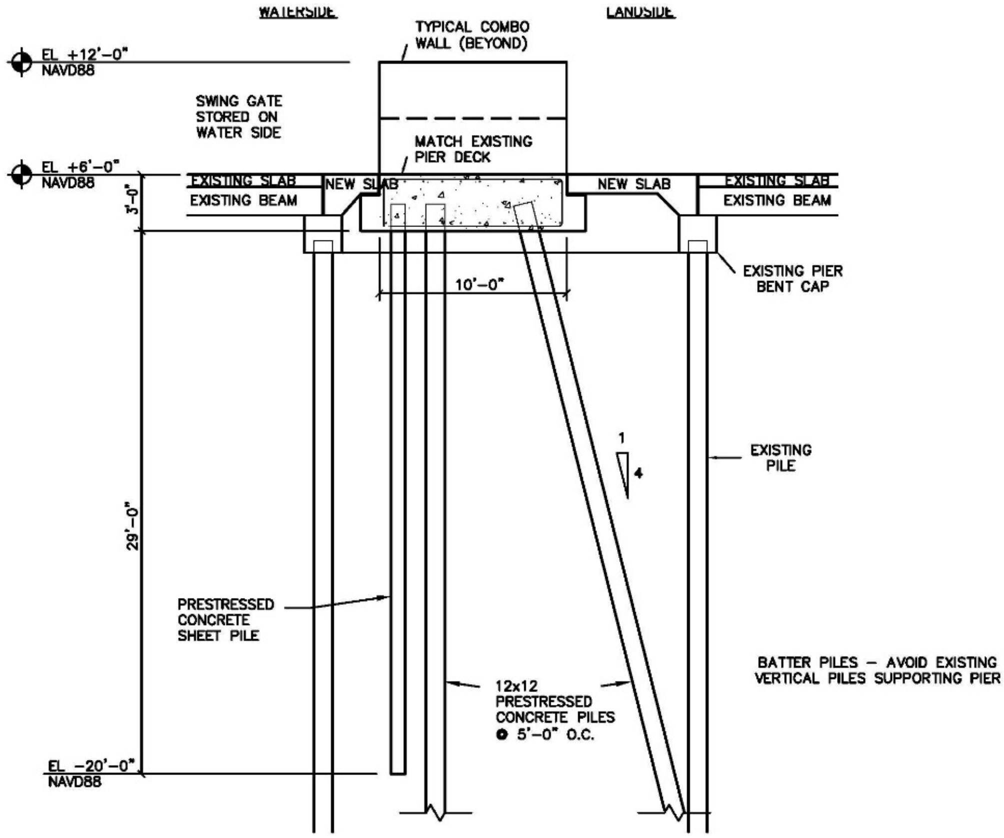


Figure5.6.5 Typical section of Combo Wall Crossing Coast Guard Dock

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, water tight pedestrian gates will be installed in these locations. There are approximately 20 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 water tight seal when closed. Typically, the gates will remain open and will only be closed during a coastal storm event.

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 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. Storms gates will primarily be sluice gate type for their simplicity and ease of operation. Note that stormwater outfalls owned by the city of Charleston are not included in this analysis as they already have or will have check valves installed by the city separate from this study that will serve to keep storm surge from directly entering the stormwater system. Figure 5.6.6 shows the Preliminary locations of Storm Gates. These are all located at tidal creeks, including creeks that are currently partially restricted by culverts.

Figure 5.6.6 Preliminary 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 5.6.7, 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 5.6.7: 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.

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. 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.

Gate closure procedure will be finalized during PED phase and dictated in the Operation and Maintenance Plan. Typically, all of the gates will remain open and will only be closed when required due to a coastal storm flooding 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. Closure of storm gates will also be dependent on minimizing impacts to aquatic resources. It is assumed at this time that as NOAA forecasts storm surges, which are issued 48 hours from the onset of storm impacts, that are equal or greater than major flooding, then storm gates will be closed on a low tide cycle, in order to keep the rising tide levels from taking storage needed for the associated rainfall. At present that elevation is identified as 8 MLLW or 4.86 NAVD88.

Table 5.6.1 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 assumes that all funding is provided and that concurrent construction will occur for the completion of the project.

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 FLOODING ANALYSIS

The interior flooding refers to the rainfall flooding that would occur due to the proposed wall prohibiting the rainfall to naturally runoff into the Ashley River, Cooper River, or Charleston Harbor, therefore, causing water to "pond" on the interior of the wall. Most of the rainfall is collected in a subsurface pipe network system with multiple outfalls but the City of Charleston does not have pipe network system coverage of the entire study area. The coverage they do have is in different models based on drainage area of various projects listed below. Ongoing storm drainage projects in the city, include:

- Calhoun Street East Drainage to the Concord Street Pump station is complete
- Market Street Drainage improvement project constructed 2 of the three-phase projects, connects 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 also in construction. Phase 5 is pending All be completed for future without condition.
- 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 (wetwell 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 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.

While the storm drainage system is not a CSRM responsibility, any impacts to the interior hydrology due to the proposed project must be evaluated and mitigated to the extent justified under USACE policy, if necessary. Creating and evaluating that system is outside the scope of the Feasibility study. It will be assessed in PED phase CESAC obtained concurrence from the MSC that the change in flood risk of various barriers around the study area be evaluated with the HECRAS 2D model only, including evaluating the existing and under construction pump systems.

The HEC-RAS 2D computational hydraulic modeling goal of the feasibility study discussed in the Hydraulics and Hydrology HEC-RAS 2D Modeling Sub-Appendix is to conduct an interior flooding analysis on the Charleston peninsula. HEC-RAS 2D is the software used to conduct this analysis to determine the change in interior water levels. A variety of different scenarios are being performed observing the change in the interior water levels which will give better designation of the solution of addressing residual and induced flooding, which at this time is assumed to be pumps and gates that will be needed to remove and drain interior flood waters. A rainfall suite consisting of the 50%, 20%, 10%, 4%, 2% and 1% Annual Exceedance Probability (AEP) is being evaluated through the HEC-RAS 2D model combined with different exterior tidal boundary conditions in a steady state condition.

5.10.1 GENERAL DESCRIPTION OF WORK

The purpose of the interior drainage analysis during the feasibility phase is to estimate the increase in interior rainfall flooding due to the impediment of the wall. HEC-RAS simulations were conducted for the future without-project condition and future with-project condition for the 50%, 20%, 10%, 4%, 2% and 1% Annual Exceedance Probability (AEP) precipitation events while being combined with different steady state tidal boundary conditions. The feasibility phase will compare interior rainfall drainage/flooding for the 12' (NAVD88) wall footprint for the with-project condition and no wall in place for the future without-project condition. The goal of the feasibility phase is to evaluate the rainfall flooding due to the wall being in place.

A qualitative assessment of Climate Change Impact to Inland Hydrology referenced 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, and stream flow trends and future findings. 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 local region. The report notes that projections of precipitation in the study area are less certain than those associated with air temperature however there is moderate consensus that future storm events in the region will be more intense and more frequent compared to the recent past.

The climate change effects in reference to changes in precipitation in terms of frequency, intensity, or duration of rainfall is not a focus of the study. However, precipitation of various frequencies are being analyzed to observe the hydraulic response of the overland interior drainage for a with-project condition with the goal of limiting the increase of interior water levels due to the wall in place prohibiting the natural overland flow. The precipitation data was provided via a City of Charleston Contracting group. This data is in the form of Direct Runoff Rain-on-Grid Precipitation Time Series Data based on SCS methodology and an average CN value. The interior hydrology is being modeled with HEC-RAS 2D analyzing various rainfall frequencies for the 2-, 5-, 10-, 25-, 50-, and 100- year frequencies. With this array of events, the HEC-RAS 2D model can show the potential of such large rainfall events.

The goal of this precipitation analysis is to size storm gates in the proposed wall that will allow interior storm water to drain out of the system efficiently via gravity flow. A closed system scenario is also being modeled to analyze the interior system when the gates are closed due to high tide. This closed system scenario has the goal of analyzing the feasibility of pumps and sizing those pumps if deemed necessary.

The figure 5.10.1 below is for informational purposes to note a +0.25 in/decade.

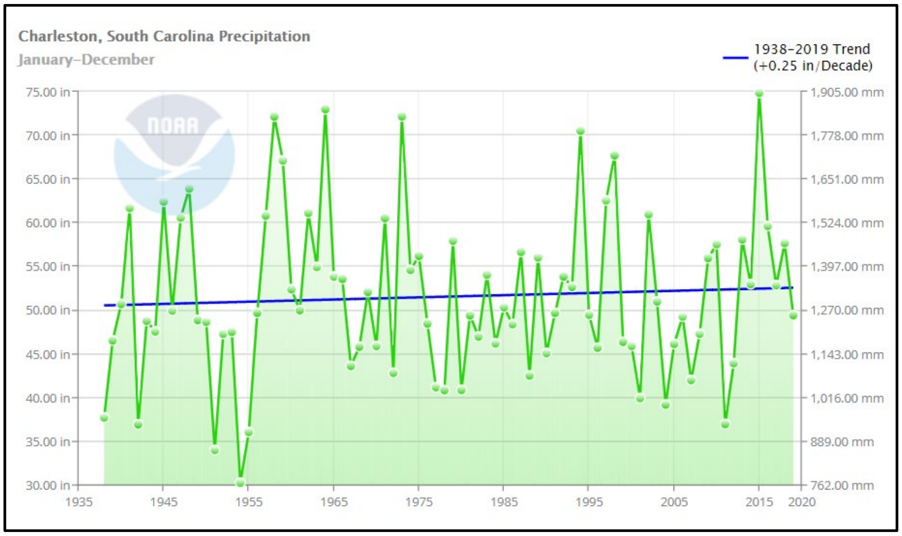


Figure 5.10.1 Precipitation Trend for Charleston, SC

The potential for future changes in rainfall are not considered in the feasibility phase. The study area is small and the changes in rainfall would be the same for future conditions whether with a project or without. The intent is to determine the impact of the wall not the changes in rainfall. If deemed necessary, further assessment of climate change impacts on the rainfall could be done in PED phase. See the Hydraulics and Hydrology HEC-RAS 2D Modeling Sub- Appendix for more details.

5.10.2.1. SOFTWARE

a. HEC-RAS 5.1 Alpha An alpha version of the Hydrologic Engineering Center's (CEIWR-HEC) River Analysis System (HEC-RAS) was used to model the complex flow of rainfall runoff within the interior and evaluate different hydraulic alternatives, such as storm gates and pumps. An alpha version was used because the latest officially released version, 5.0.7 does not have the ability to combine 2D areas with pump stations. The 5.1 version does have these features but has not been officially released. A meeting was held between PDT members from the Charleston District, South Atlantic Division (SAD), and the Modeling, Mapping, and Consequence Center (MMC) to approve the usage of the unreleased Alpha version. During PED Phase, the HEC-RAS modeling will be updated to the recently released HEC-RAS 6.0.

b. ESRI ArcMap 10.7 GIS software is being used to geo-reference different elements with the HEC-RAS 2D model such as the location of the 12' wall provided by the H&H team lead. A LIDAR dataset has been provided by the PDT GIS team member. This will be used as the terrain in the 2D Model.

c. 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 damages with the goal of capturing the estimated average annual damages for each future without-project scenario and each future with-project scenario.

5.10.2.2 HEC-RAS MODEL DEVELOPMENT

a. Original Model

The City of Charleston originally hired a contractor to perform HEC-RAS 2D modeling to assist them in 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. The terrain file used in their effort was based on the 2009 Charleston County LIDAR Data. The 2011 NLCD data was used during the original model development by the contractors. The current effort has updated the landcover layer to NLCD 2016.

b. Model Revision/Development

The model used in the Calhoun West effort was revised to perform the analysis for this effort. Revisions from the original model have primarily been restructuring of the 2D mesh and separating the 2D mesh into 2 different grids to represent the interior and exterior areas connected by a SA 2D connection. The original RAS model that was provided contained a road network shapefile that was being enforced in the 2D area as breaklines. That same breakline layout is being used in this 2D effort. Breaklines have also been applied to other appropriate locations to represent raised features in the model domain. 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”.

Peninsula outfall locations have been provided in GIS shapefile format to provide locations of the outfalls. The peninsula has numerous outfall discharges that drain the sub-surface pipe network and outfalls that drain overland flow through culverts which also allow for daily tidal fluctuations in tidal creeks. However, HEC-RAS is unable to compute subsurface flow therefore the outfalls connected to the sub-surface pipe network will not be utilized, and the model will assume no pipe flow capacity. Culvert data was provided by the City of Charleston and incorporated into the model. Figure 9 displays the culverts incorporated into the modeling. These culverts were assigned as SA/2D connections with culvert openings. As shown in Figure 9, several culverts are in line with the proposed wall alignment and these culverts are assumed to be equipped with storm gates in the design of the project. Some of the culverts that are in line with the wall are also considered peninsula outfalls. These include the culvert Near Joe Riley, Gadsden Creek, Lockwood Wetland, and Newmarket Creek. In a small number of cases, culvert dimensions and invert elevations had to be estimated or measured from Google Earth. Overall, these assumptions should have minor effects on the model results.

The exterior portion of the mesh is bounded by the Ashley River, Cooper River, and Charleston Harbor. The exterior portion of the mesh also includes areas of land that are outside of the 12’ wall alignment. The east side of the city will be walled internally and not walled out in the water, therefore there will be a substantial amount of land included in the exterior mesh. The interior portion includes everything that is inside of the 12’ wall alignment. The interior and exterior areas are connected by a storage area connection. This storage area connection represents the 12’ wall footprint. The weir profile within the storage area connection for the future without-project condition is set to the underlying terrain. In RAS2D, “terrain” includes the topography and bathymetry. The future with-project conditions storage area connection is set to a height of 12’ (NAVD88). This ensures the mesh alignment is the same for the future with-project and future without-project conditions, aside from the elevations in the storage area connection. Consistency in the geometry files allows for a better comparison in model results between with and without-project. As mentioned, the interior and exterior areas are connected by storage area connections. The connections within HEC-RAS must applied a weir coefficient to represent the hydraulic efficiency of the connection. Connections representing natural ground will have a lesser value than the connections representing an actual structure or wall. The connections representing natural ground are given a weir coefficient ranging from 0.2 to 1 depending on the elevation of the ground the connection is representing relative to typical water heights in the area. The value of natural ground weir coefficients were also decided during the iterative modeling process based on stability of the flow across the connection. The connections representing the proposed wall and the Battery are given a weir coefficient of 2 as this is typical guidance used for HEC-RAS modeling to characterize weir flow.

LiDAR provided by the South Carolina Department of Natural Resources is being utilized in this study. The figures on the following pages display the LiDAR terrain that is being used. The LiDAR is characterized as a single band raster with a 3ft x 3ft resolution that was collected in 2017. The dataset originally wasn't large 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 didn't capture. The LIDAR was resampled to 5ft x 5ft resolution when merged with the 2009 Charleston County raster. Buildings are not included in the LiDAR and are accounted for within the landcover layer.

A sensitivity analysis was performed for a terrain file with buildings included in the mesh and the terrain being used without the buildings included. The results using the terrain with buildings displayed slightly higher water levels than the results using the terrain without the buildings. On average the increase in water level was less than an inch. The penetration of flooding into buildings is captured using the landcover layer as opposed to including buildings in the terrain.

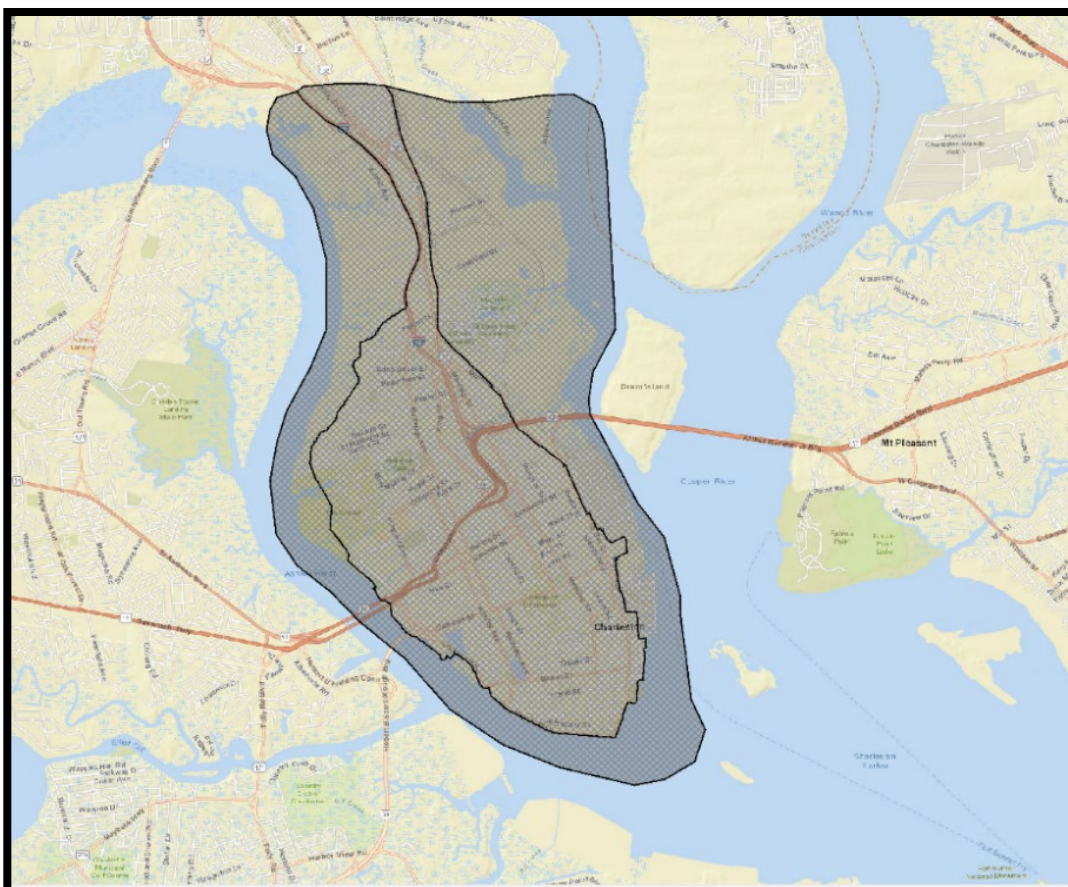


Figure 5.10.2. HEC-RAS 2D computational mesh

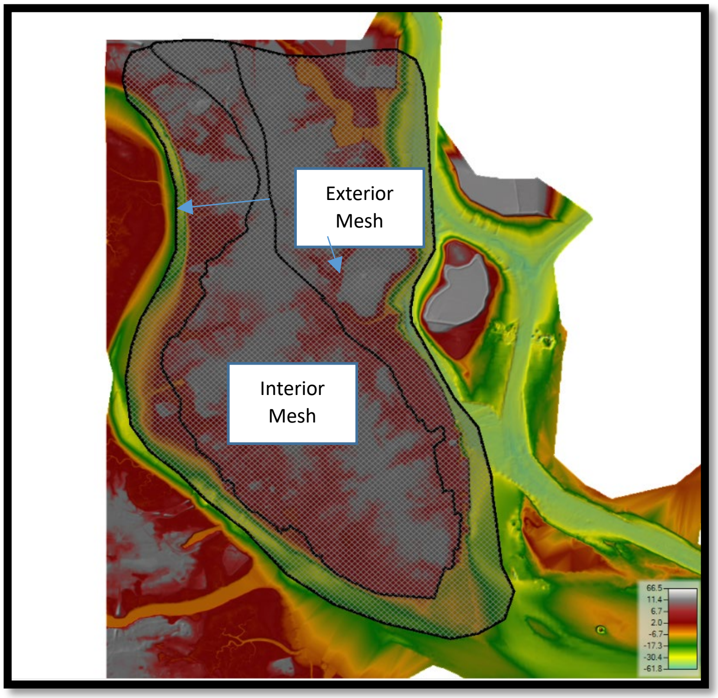


Figure 5.10.3 HEC-RAS 2D computational mesh and terrain (ft. NAVD88)

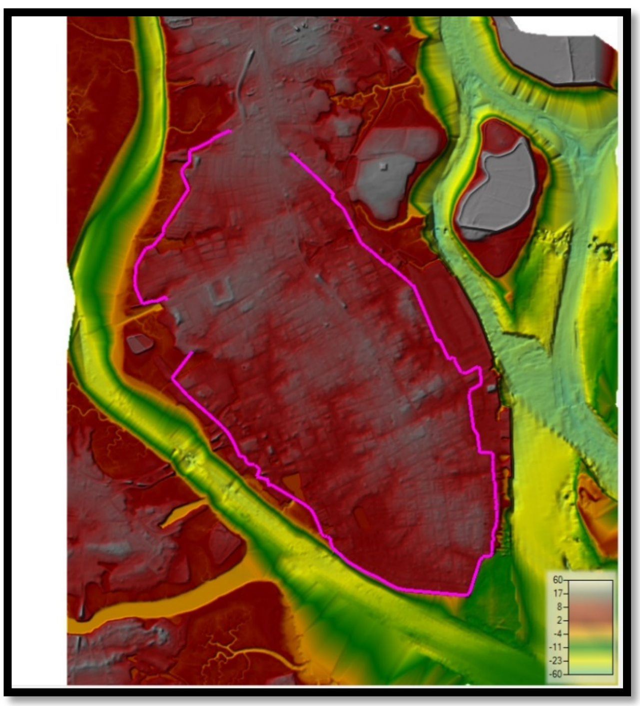


Figure 5.10.4. 12’ Wall Alignment

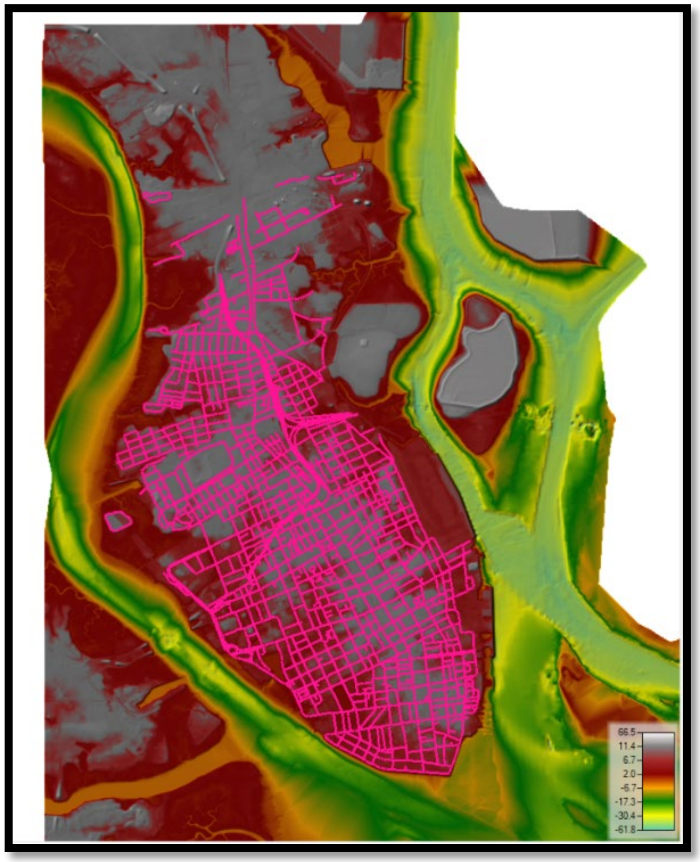


Figure 5.10.5 HEC-RAS Breaklines applied to 2D mesh

Rain-on-Grid Precipitation Time Series Data

Rainfall data was provided with the HEC-RAS model that was developed by the City of Charleston contractor working on the Calhoun West pumping station project. A runoff excel spreadsheet was used to develop the direct runoff based on SCS Type III methodology and an average CN Value of 88. The data was also provided in a HEC-DSSVue file with the direct runoff time series data which can be directly linked into the HEC-RAS unsteady flow files. Annual Exceedance probability rainfall data was provided for the 50%, 10%, 4%, 2% and 1% AEP rainfalls. The 20% AEP rainfall was estimated using the provided direct runoff data. The rainfall data is applied to the 2D mesh uniformly, however, the rainfall could vary spatially across the modeled area. More information on the rainfall data is in the HEC-RAS 2D stand-alone Sub-Appendix Section 3.4.

5.10.2.3 MODEL SCENARIOS

a. Existing Conditions with Known Flood Event

The existing conditions model scenario typically serves as a model validation or calibration event, however, there is little to no available gage data on land in Charleston to validate water levels in the interior. Verified interior water levels could be measured against the computed water levels if data was available. However, the HEC-RAS 2D model will serve its intended purpose to estimate the hydraulic response of the overall system by analyzing the various pumping capacities and storm gates.

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.

Highlighted values in Table 5.10.1 were used as exterior and interior water surface elevations within the pump and storm gate modeling analysis.

Table 5.10.1 Water Surface Elevations (WSEL) at NOAA Gage (8665530)

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

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. The figure below 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 rise value of +0.56 feet. Hurricane Irma peak water surface elevation was approximately 6.7 ft. NAVD88 so projecting this to the year 2032 assumes a peak water surface elevation of approximately 7.2 ft. NAVD88.

The purpose of Figure 8 is to provide visual representation of the potential increase in flooding for future storm events due to sea level rise. 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; therefore, the computed inundation is only a result of the stage hydrographs.

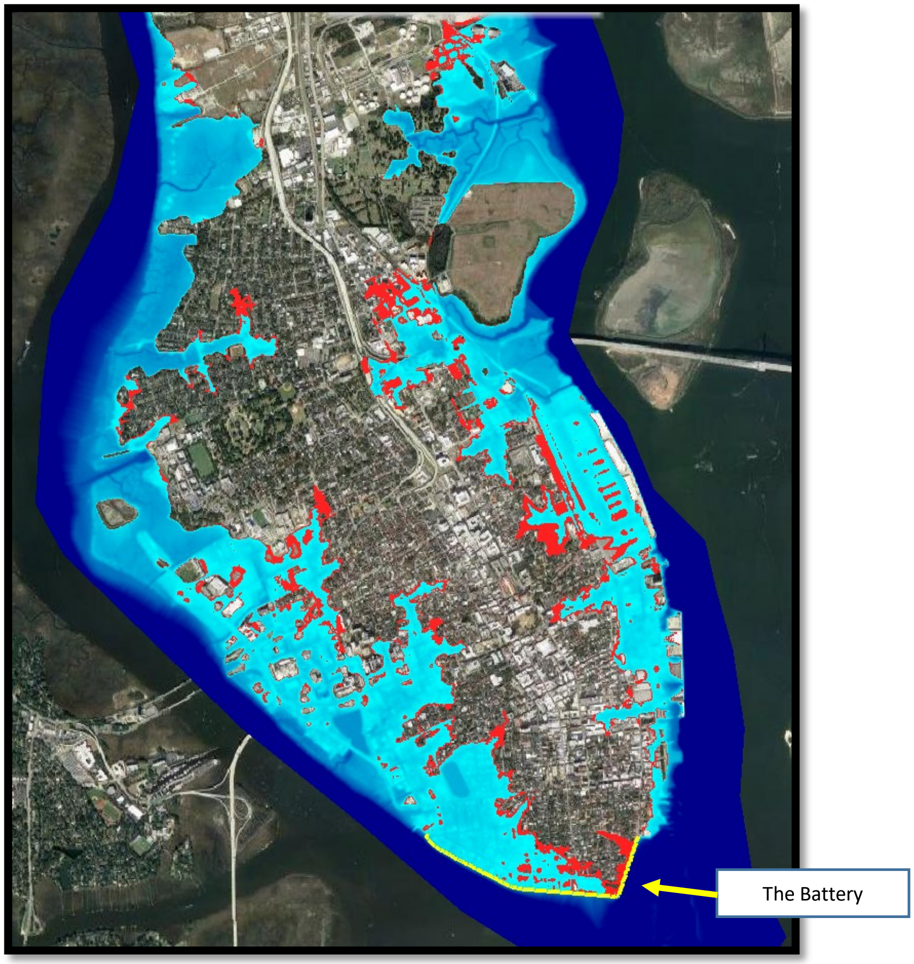


Figure 5.10.6 Hurricane Irma 2017 (Blue) vs Hurricane Irma Hypothetical 2032 (Red)

b. Future without Project Conditions

The goal of the future without-project condition is to run the entire rainfall suite onto the future without-project condition geometry in combination with different exterior water surface elevations and visualize flow paths and flood potential. Then, at the selected output locations the water surface elevations for the different scenarios will be documented and then compared against the water surface elevations at the same locations for the with-project conditions to observe the increase in interior rainfall flooding due to the wall. In this case, the feasibility study is analyzing the 12' (NAVD88) elevation wall footprint as the with-project condition. An overestimation of flooding could be assumed due to the inability of modeling subsurface drainage within HEC-RAS. HEC-RAS cannot not model the underground gravity driven storm water system that consists of a complex network of pipes and tunnels that discharge through outfall locations into exterior area.

The 50% AEP through 1% AEP rainfall suite were run through the future without-project conditions geometry associated with different steady state exterior water surface elevations to analyze how different exterior water surface elevations affect the drainage.

There are a numerous number of combinations that could occur when compounding storm surge and rainfall. Compound flooding in this case, being the joint probability that a given level (or greater) of interior drainage-rainfall will occur given that a given-probability storm surge event occurs. The goal is to address the potential issue of compound flooding with as simple approach as possible. The feasibility effort took a very simplistic approach in this compounding to achieve the goal designated for feasibility phase. The study can get more elaborate during the PED phase. Riverine flooding is not being considered for this study per PDT agreement as it is a very small component of the flooding. There has never been a peninsula flooding incident where the flooding was riverine based. All flooding incidences are related to tidal events or stormwater drainage system. Cooper River is controlled by upstream hydropower facility where the flood releases discharge into a different river. FEMA does not document any riverine flooding on the Ashley, Wando, or Cooper – it is all storm surge-based flood risk. This is further supported by these statements:

- "On October 6, 2015, a Major Disaster Declaration (FEMA-4241-DR) was declared for the State of South Carolina after an unprecedented rain event set rainfall records across the state and flooded entire communities. For some locations, the rainfall was historic and qualified as a 1,000-year rain event, resulting in deadly and disastrous flooding with damages that could top $1 billion (Beam and Kinnard, 2015). Rainfall was severe enough to close a 75-mile stretch of Interstate 95 between Interstates 20 and 26. In the City of Charleston, 6.40 inches of rain was received over a 12-hour period (Carolinas Integrated Sciences & Assessments [CISA], 2015) and shut down the city's historic district due to flooding. The hydrologic response was much longer than the four-day rainfall event. While flooding in coastal areas occurred as a result of intense and large amounts of precipitation from October 1 to 5, riverine floodwaters flowed downstream draining the state with some stream gages not recording peak flow until October 11, 2015 (CISA, 2015 (https://cisa.sc.edu/PDFs/October%202015%20Flood%20Event%204%20Pager.pdf).
- The most significant flooding occurred in areas along and near smaller creeks and streams, especially those that were tributaries to larger rivers such as the Edisto, Ashley, Cooper, and Santee (National Weather Service, 2015 (https://www.weather.gov/chs/HistoricFlooding-Oct2015)."

c. Future with Project Conditions

The goal of the future with-project condition for the feasibility phase is to run the entire rainfall suite onto the future with-project condition geometry in conjunction with a steady state high tide water level. The FWP for the feasibility phase is analyzing the conditions with the 12' (NAVD88) constant wall elevation. The footprint of this wall can be seen in Figure 5.10.2.3. The high tide water level 3.18' was chosen as the tidal boundary condition for the with-project scenarios. The tidal boundary condition is not as significant at this point in the analysis because the with-project condition is assuming to be a closed system, which means no flow in and no flow out. The analysis is simply looking at the increase in rainfall flooding on the interior due to the inability of rainfall to drain into the Ashley and Cooper Rivers because the wall is in place. RAS cannot model subsurface drainage; therefore, the peninsula outfalls and storm drainage network will not be modeled. It is the assumption that the check valve program on the outfalls will be complete which will prevent tidal backflow into the system which is defensible to the closed system assumption in that regard during high tide states.

In a scenario where the wall overtops, the interior area will be drained via gates after river/tide levels decrease. Any detailed assessment of the timing of an overtopping scenario versus the opening and draining via gates in the wall will be deferred to PED phase. If the wall were to overtop from storm surge then the city would be flooded for a future without-project condition, therefore there would not be an increase in flooding due to the project. The primary focus of the feasibility phase for FWP is to quantify the results so that the mitigation features such as pumps and storm gates can be sized and then implemented and analyzed in more detail during the PED phase.

d. Results at Selected Output Locations

Selected output locations were used to assess the increase in water levels for future with-project conditions versus future without-project conditions. Figure 5.10.7 displays the selected output locations for the RAS modeling. The locations were chosen to show various impacts around the peninsula.

Figure 5.10.7. Selected Output Locations for RAS model results

5.10.2.3 COMPARISON ANALYSIS

Pump stations and storm gates have been evaluated within the RAS model. Three alternatives have been evaluated for different size pump stations and storm gates. The storm gates have been placed at 15 different locations around the peninsula. The storm gates at each location have been evaluated with three alternatives of various dimensions. The storm gates will be placed in the wall at the selected sites or attached to existing culverts such as the creek behind Joe Riley Stadium, Gadsden Creek, Long pond, Lockwood Wetland, and Newmarket Creek. In supplement to storm gates, other gates have also been included in the modeling to account for the pedestrian and vehicular gates as part of the design of the wall. These gates are mostly placed at higher land elevations as opposed to the storm surge gates and should not play a significant role in the interior drainage analysis for gates open conditions.

The City of Charleston actively operates two pump stations, the Medical University of South Carolina station (MUSC), and Concord Street station. There are two other pump stations currently in construction phase or design phase, which are the Spring Fishburne station and King/Huger station. The MUSC, Concord Street, and Spring Fishburne stations have been incorporated into the modeling for both the future-without and future-with conditions. The King/Huger station, which is in design phase is not included in the modeling. The PDT pump station alternatives assumed similar or smaller capacities to that of the MUSC pump station as a starting reference.

The pump stations have been placed at ten different locations around the peninsula. Each location has been evaluated with three alternative pumping capacities. The PDT has evaluated five permanent pump stations and 5 temporary pump stations. Temporary meaning the pumps would only deploy during warranted storm events while portions of the infrastructure for the temporary pumps would permanently remain in place for quick deployment.

The system is analyzed with the goal of limiting the increase in interior stages for all precipitation events while applying a focus on the 10% AEP rainfall event for conceptual design purposes. As discussed in earlier sections of this report, the 10-YR precipitation event is the focus because the City of Charleston drainage infrastructure passes no more than the 10 % AEP event. Though the 10% AEP event is the focus of design, other storm frequencies as mentioned previously have been analyzed. The interior drainage analysis will look at the system in two different perspectives: an open system and a closed system.

i. The open system assumes non-storm conditions or typical tidal conditions; therefore, the storm gates remain open to allow daily tidal fluctuations. The precipitation applied to the 2D model will drain via gravity or overland flow through the storm gates.
ii. The closed system assumes storm conditions meaning a surge type event. In the event of a surge, the gates will be closed. However, for the purpose of properly sizing the pumps the modeling assumes the external stage condition to be at a high tide and not storm surge.

    Pre-storm water level drawdown is assumed in the modeling, meaning the gates are closed prior to the storm surge event arriving. In the model, the interior 2D area was assigned an initial interior elevation representing a low tide of -2.4 ft. NAVD88 for the year 2032 and -1.31 ft. NAVD88 for the year 2082.
iii. Additional closed system simulations were computed to assess rainfall plus overtopping. More information will be included in remaining sections of the report.

Table 5.10.2 describes the RAS geometries. FWO is future without-project and FW is future with-project. Future without-project made assumptions as provided by the City of Charleston. Future with-project analyzed the system as an open and closed system. During the open system analysis, the storm gates remained open. During the closed system, one geometry was setup with the gates closed and no PDT pumps to justify having pumps to complete the system while three other geometries were setup with three different pump alternatives to analyze the needed pump capacities.

It is known that high storm surge events will inundate the interior substantially and result in water levels that would be much higher than the water levels for a future with-project condition. In other words, the project will greatly reduce the water levels in the interior from a storm surge event. Therefore, analysis was focused on rainfall at high- tide with and without the wall. The difference is a comparison of gates open for a non-storm surge event and gates closed for a storm surge event. These simulations will provide adequate with-project and without project result comparisons for the proper sizing of pumps.

Table 5.10.2 RAS Geometries

Geometry Condition	Geometry Assumptions
FWO	Future without-project assumes Low Battery is raised to 9ft NAVD88 and three City of Charleston P.S. are active. No Subsurface pipes.
FW (gates open) alt 3	Future with-project assumes Low Battery is raised, three City P.S. are active, and PDT storm gates are placed and open throughout entire simulation. No Subsurface pipes.
FW (gates closed)	Future with-project assumes Low Battery is raised, three City P.S. are active, storm gates are closed but no PDT pumps active. No Subsurface pipes.
FW (gates closed) P. S. alt 1	Future with-project assumes Low Battery is raised, three City P.S. are active, storm gates are closed with P.S. alt 1 active. No Subsurface pipes.
FW (gates closed) P. S. alt 2	Future with-project assumes Low Battery is raised, three City P.S. are active, storm gates are closed with P.S. alt 2 active. No Subsurface pipes.
FW (gates closed) P. S. alt 3	Future with-project assumes Low Battery is raised, three City P.S. are active, storm gates are closed with P.S. alt 3 active. No Subsurface pipes.

The PDT pump station alternatives assumed similar or smaller capacities to that of the MUSC pump station as a starting reference. The storm gates were iteratively sized based on the height of the wall at each selected site. The storm gates were also sized based on dimensions of the existing culvert outlets that align. The storm gate and pump alternatives are seen in Figures 5.10.8 and 5.10.9. and listed in Table 5.10.3 and Table 5.10.4.

Table 5.10.3 Storm Gate Alternative Dimensions

Storm Gate Location	Open System. Flow passing by gravity through storm gates or removed by City of Charleston Pump Stations.		
	Storm gate alt. 1	Storm gate alt. 2	Storm gate alt. 3
	(ft. x ft.)	(ft. x ft.)	(ft. x ft.)
Wag Terr1	1 – 4’x4’	1 – 5’x5’	1 – 6’x6’
Wag Terr 2	1 – 4’x4’	1 – 5’x5’	1 – 6’x6’
Wag Terr 3	1 – 4’x4’	1 – 5’x5’	1 – 6’x6’
Wag Terr 4	1 – 4’x4’	1 – 5’x5’	1 – 6’x6’
Halsey Creek	2 – 6’x6’	2 – 8’x8’	2 – 10’x8’
Wag Terr 7	1 – 4’x4‘	1 – 6’x6’	1 – 7’x7’
Wag Terr 8	1 – 4’x4’	1 – 6’x6’	1 – 7’x7’
Wag Terr 9	1 – 4’x4’	1 – 6’x6’	1 – 7’x7’
The Citadel	2 – 4’x4’	2 – 6’x6’	2 – 8’x8’
CG Wetland	1 – 4’x4’	1 – 6’x6’	2 – 6’x6’
*Creek Behind Joe	1 – 12’x4’ box culvert	1 – 12’x4’ box culvert	1 – 12’x4’ box culvert
*Gadsden Creek	1 – 9’x4’ box culvert	1 – 9’x4’ box culvert	1 – 9’x4’ box culvert
*Longpond	1 – 4’ circular pipe	1 – 4’ circular pipe	1 – 4’ circular pipe
*Lockwood Wetland	1 – 3’ circular pipe	1 – 3’ circular pipe	1 – 3’ circular pipe
*Newmarket Creek	2 – 8’x3’ double box culvert	2 – 8’x3’ double box culvert	2 – 8’x3’ double box culvert
Totals	18 gates	18 gates	19 gates
*Existing culverts owned by the City of Charleston. During this phase of the study, the existing culverts are assumed to remain the same dimensions for each alternative. However, the City of Charleston has stated the possibility of upsizing these culverts in the future. These culverts are assumed to be equipped with storm gates as part of this study.			

Table 5.10.4 PDT Pump Station Alternatives

Pump Location	Closed System. Water removed by PDT pumps and City of Charleston Pumps.		
	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
Reid St. Basin (T)	10	20	40
Cooper St. Basin (T)	10	20	40
Newmarket Creek (P)	60	90	150
Totals	10 pump stations	10 pump stations	10 pump stations
	290 cfs	490 cfs	890 cfs
(P) Permanent (T) Temporary			

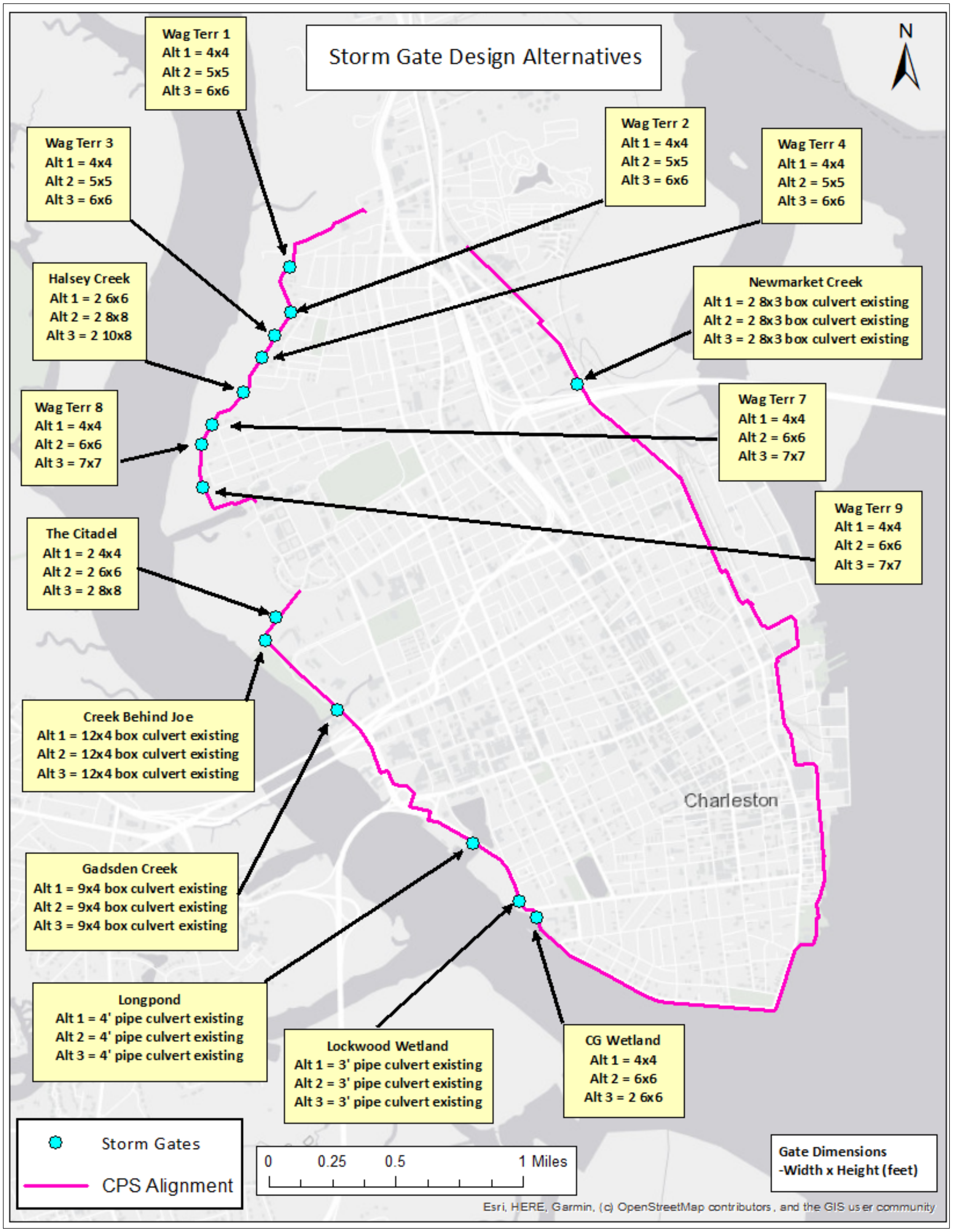


Figure 5.10.8 Storm gate alternatives

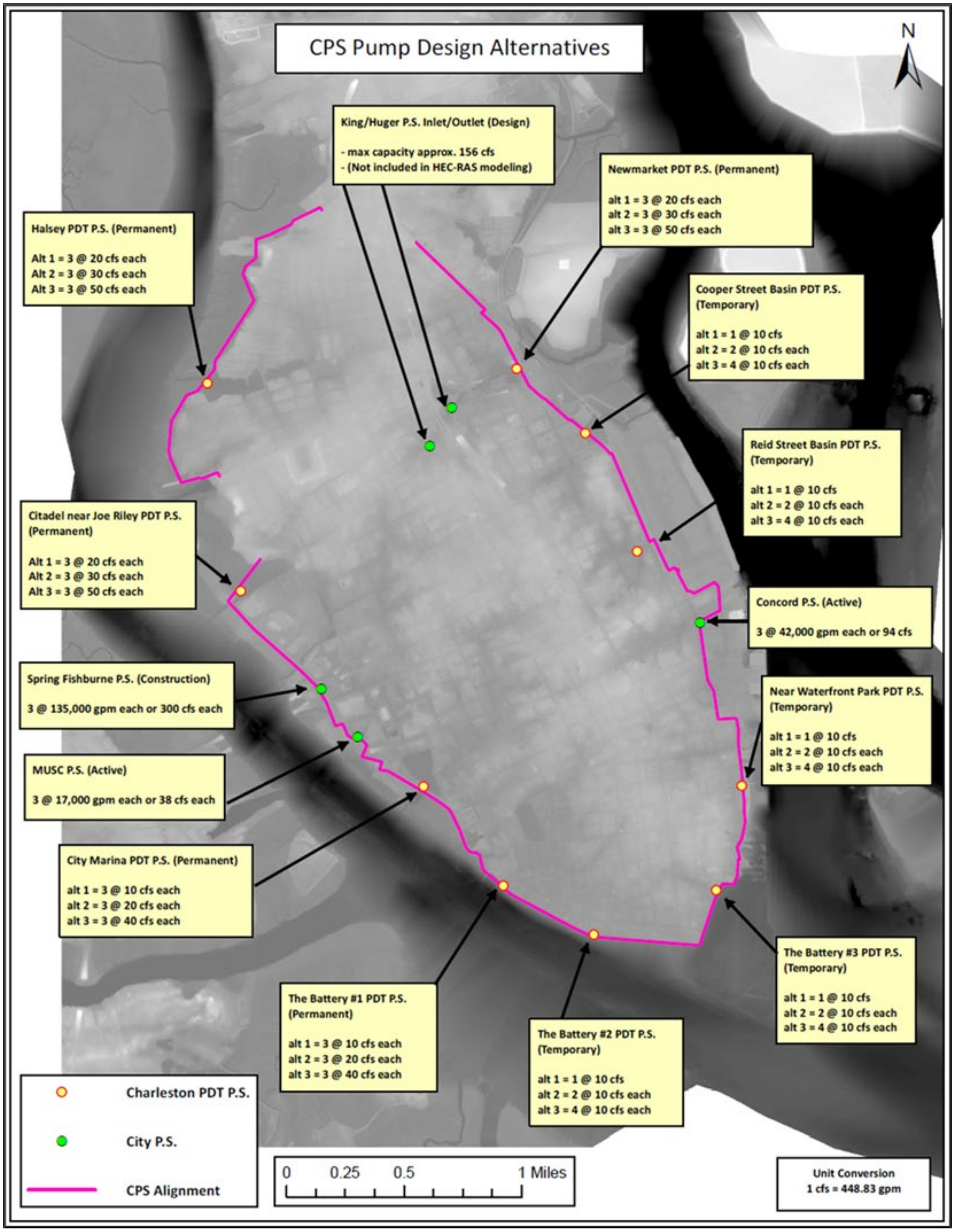


Figure 5.10.9 Pump station alternatives

To demonstrate the change over time and design pumps and gates for the worst impact, the scenarios will be evaluated in the year 2032 (first year after construction) and 2082 (end of economic project life).

5.10.2.3.1 Scenarios for the year 2032 (FWO vs FW)

Each event utilizes a stage boundary condition of high tide at 3.18 feet NAVD88. The results were provided at the selected location and input into the HEC-FDA analysis performed by Economist.

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 (gates open) alt 3		FW (gates closed)		FW (gates closed) P.S. alt 1		FW (gates closed) P.S. alt 2		FW (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

5.10.2.3.2 Scenarios for the year 2082 (FWO vs FW)

Each event utilizes a stage boundary condition of high tide at 4.27 feet NAVD88. The results were provided at the selected location and input into the HEC-FDA analysis performed by Economist.

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 2032									
FW (gates open) alt 3		FW (gates closed)		FW (gates closed) P.S. alt 1		FW (gates closed) P.S. alt 2		FW (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

5.10.2.3.3 Scenarios for the overtopping Analysis (FWO vs FW)

Each event utilizes a stage boundary condition of 10 feet NAVD88. Inputs were based on the data described in Section 5.11 Wave Overtopping.

Comparison matrix for OT	
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

Comparison matrix for OT							
FW (gates closed)		FW (gates closed) P.S. alt 1		FW (gates closed) P.S. alt 2		FW (gates closed) P.S. alt 3	
Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall
Elev 10.0 NAVD88	50% AEP rain +OT	Elev 10.0 NAVD88	50% AEP rain +OT	Elev 10.0 NAVD88	50% AEP rain +OT	Elev 10.0 NAVD88	50% AEP rain +OT
Elev 10.0 NAVD88	20% AEP rain + OT	Elev 10.0 NAVD88	20% AEP rain + OT	Elev 10.0 NAVD88	20% AEP rain + OT	Elev 10.0 NAVD88	20% AEP rain + OT
Elev 10.0 NAVD88	10% AEP rain + OT	Elev 10.0 NAVD88	10% AEP rain + OT	Elev 10.0 NAVD88	10% AEP rain + OT	Elev 10.0 NAVD88	10% AEP rain + OT
Elev 10.0 NAVD88	4 % AEP rain +OT	Elev 10.0 NAVD88	4 % AEP rain +OT	Elev 10.0 NAVD88	4 % AEP rain +OT	Elev 10.0 NAVD88	4 % AEP rain +OT
Elev 10.0 NAVD88	2% AEP rain +OT	Elev 10.0 NAVD88	2% AEP rain +OT	Elev 10.0 NAVD88	2% AEP rain +OT	Elev 10.0 NAVD88	2% AEP rain +OT
Elev 10.0 NAVD88	1 % AEP rain +OT	Elev 10.0 NAVD88	1 % AEP rain +OT	Elev 10.0 NAVD88	1 % AEP rain +OT	Elev 10.0 NAVD88	1 % AEP rain +OT

More information can be found in sub appendix 3 HYDRAULICS, HYDROLOGY HEC-RAS Modeling SUB-APPENDIX

5.10.2.4 CONCLUSIONS

The alternative which results in the smallest increase in interior water levels and economic damages for the storm gate alternative when analyzing the 10-YR rainfall event is alternative 3. Storm gate alternative 3 has the largest gate dimensions at each location. Storm gate alternative 3 was selected to be included within the project cost estimate.

Gate sizes were based on typical culvert sizes. Gate sizes may increase as environmental impacts are assessed.

The pump alternative analysis is highly complex. In comparison to future without-project conditions some locations may show reductions in water levels, some may show similar water levels, and some may show increases in water levels. Based on the iterative modeling process conducted so far, pump station alternative 2 was selected to be included within the project cost estimate.

The alternatives were not mixed and matched to analyze which pump alternative performed most efficiently at each location. For example, all pump stations within pump alternative 2 may not provide the adequate pumping capacity needed at each location but as a system as a whole pump alternative 2 provided a reduction in estimated average annual damages as compared to that of the future without-project. The mixing and matching of alternatives will assist in analyzing the alternatives on a site-by-site basis which will assist in conceptually design the system in PED phase.

5.10.2 PUMP LOCATIONS

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 5.10.1 below.

Figure 5.10.1 Locations of Permanent and Temporary Pump Stations

5.10.2.1 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.

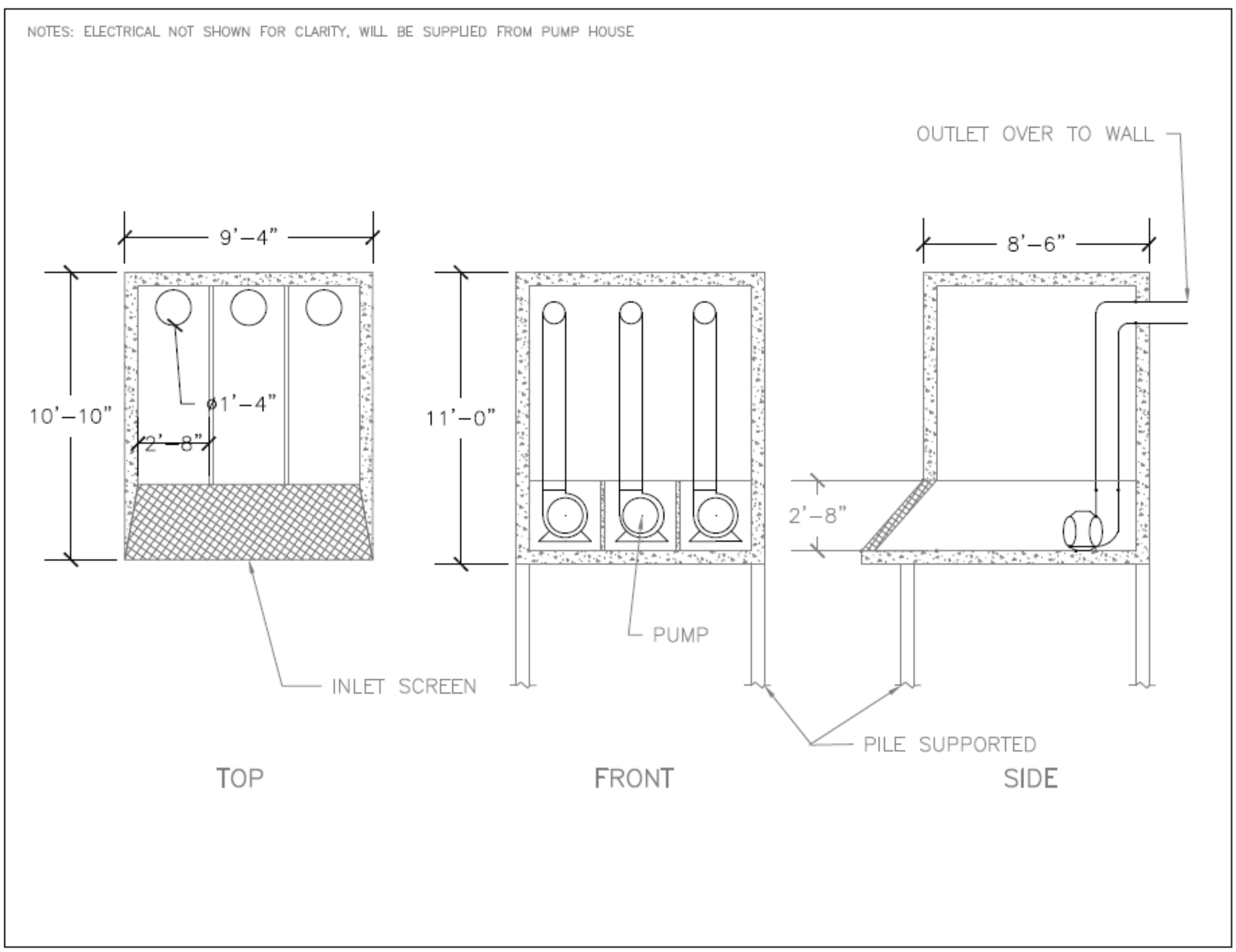


Figure 5.10.2.2 Pump Station Wet Well

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 actually located in the marsh or creek. 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 and avoid the bathtub effect created by the new wall.

5.10.2.2 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.

5.10.3. 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.

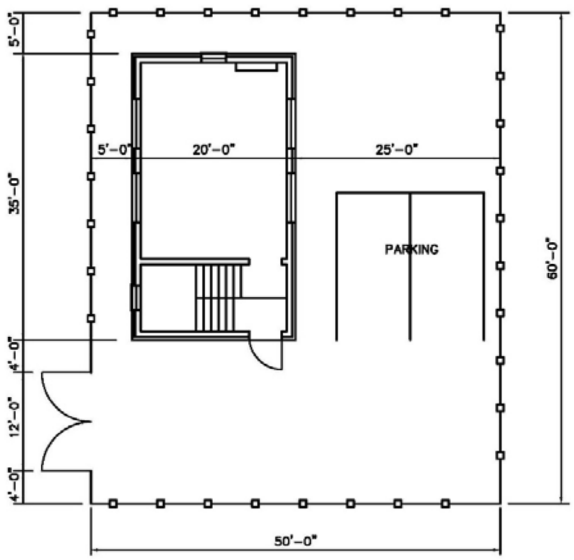


Figure 5.10.3.1. Site Plan of Pump Station

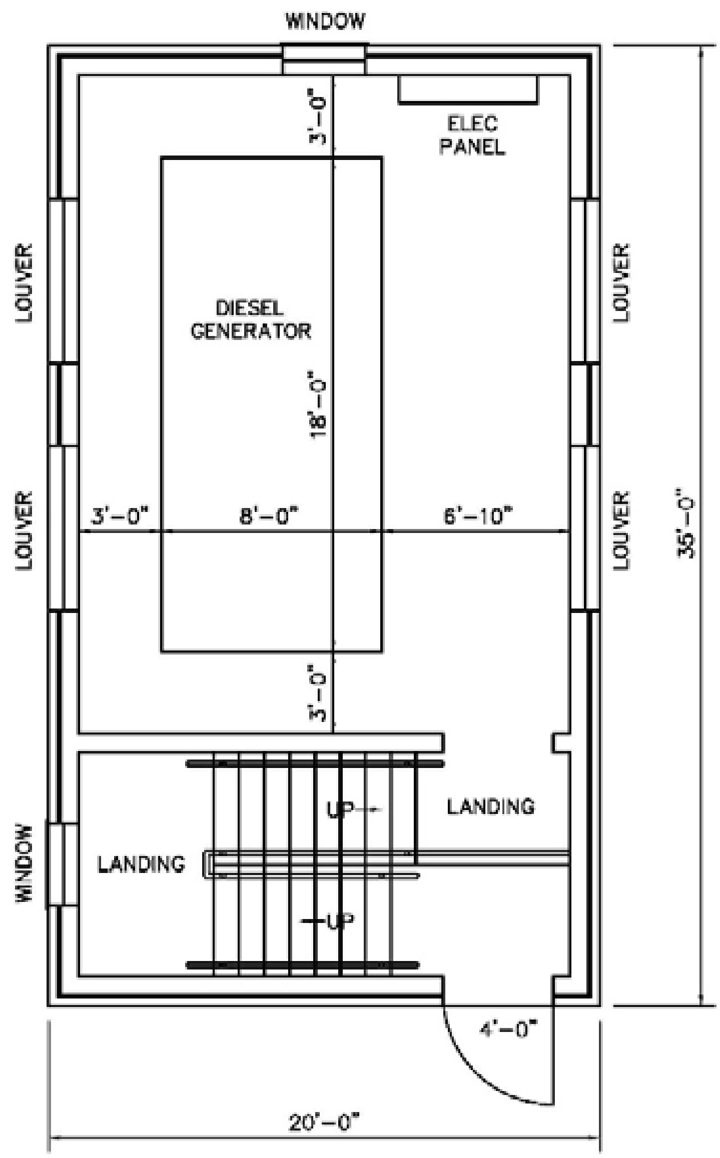


Figure 5.10.3.2 Floor Plan of Pump Station

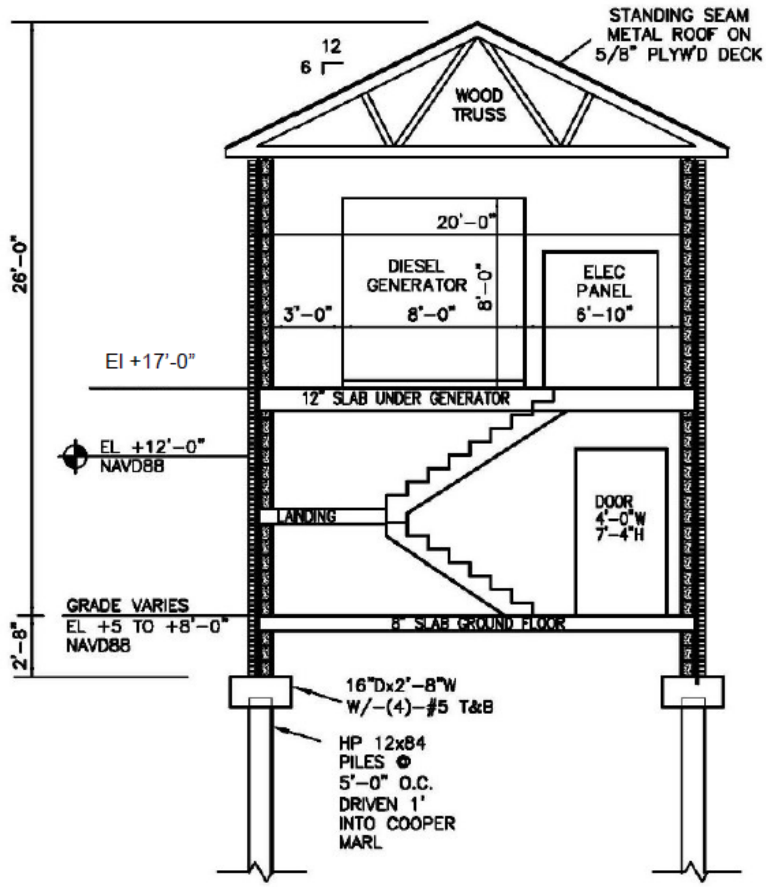


Figure 5.10.3.3 Typical Section of Pump Station

5.10.4. PUMP PREVENTATIVE MAINTENANCE

To ensure the new pumps are fully functional when needed, a regimen of preventative maintenance would be required. The details of the necessary steps for both the permanent and temporary pumps are outlined below.

5.10.4.1 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.

5.10.4.2 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

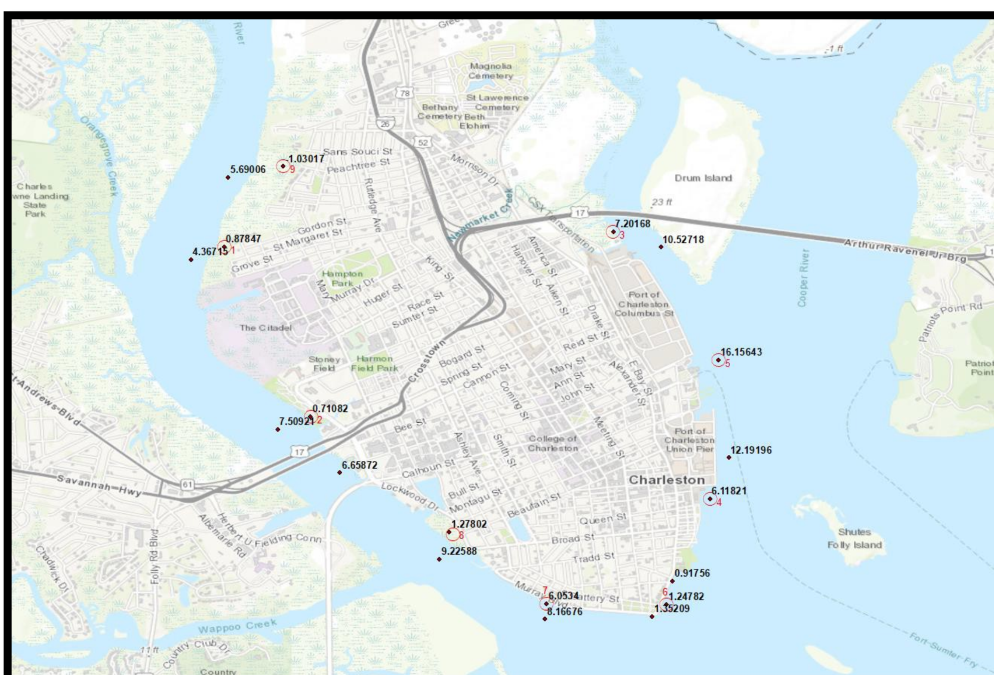
Structural design, discussed in the structural appendix, considered that the structure would be expected to withstand wind generated wave overtopping. Overtopping of the floodwall by the free flowing still water elevation is an indication of failure of defense, but not failure of the structure so long 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 with purple and pink lines showing 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. Methodology is 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, 0.01% AEP (100-year returnperiod) SWL (without considering sea level rise) is 3.1m (10.2 ft).

Figure 5.11.2: Representing Stations with Bathymetry Information

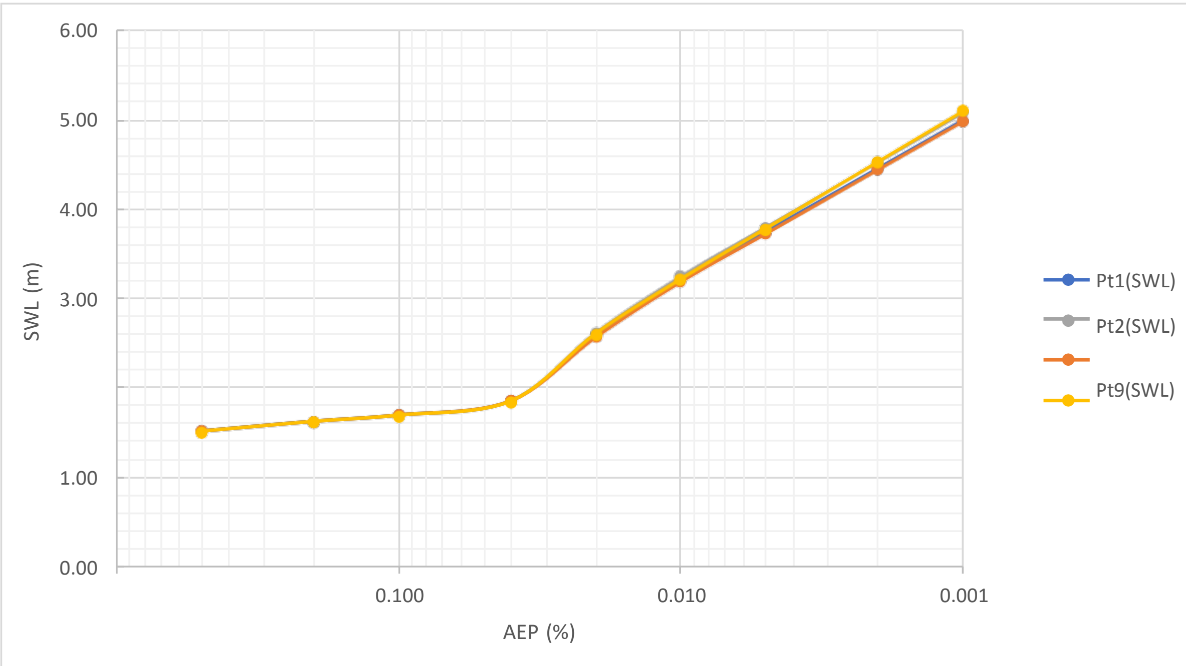


Figure 5.11.3: Still Water Level at Different Stations

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 0.01% 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 0.01% AEP wave height is between 0.8 to 1.4m. Figure 5.11.4(c) shows significant wave height alongthe Southern tip of harbor where 0.01% 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 waveenergy.

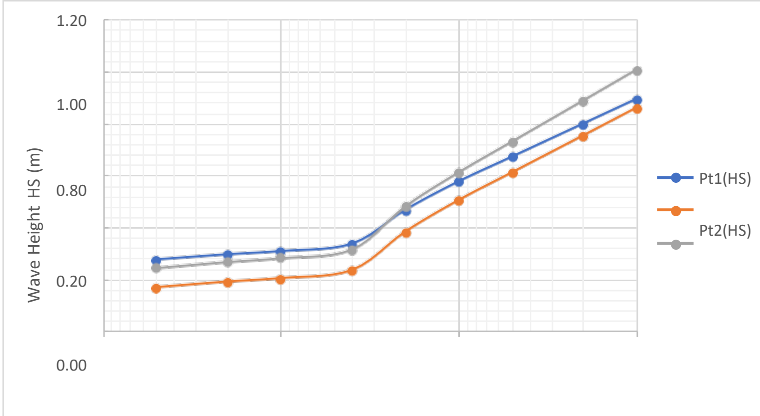


Figure 5.11.4(a): Significant Wave Height (HS) at Stations 1, 2, and 8

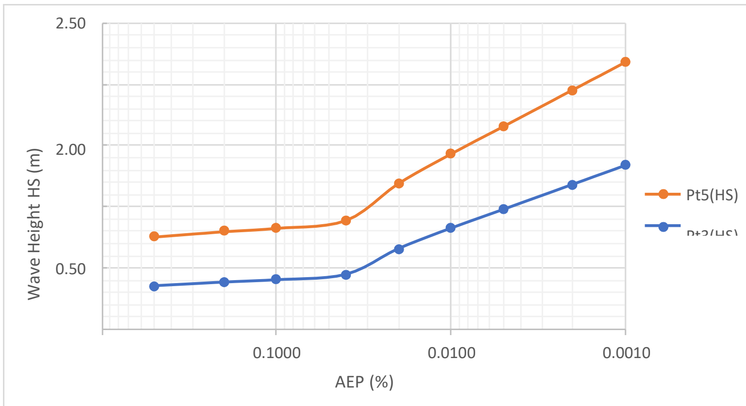


Figure 5.11.4(b): Significant Wave Height (HS) at Stations 3 and 5

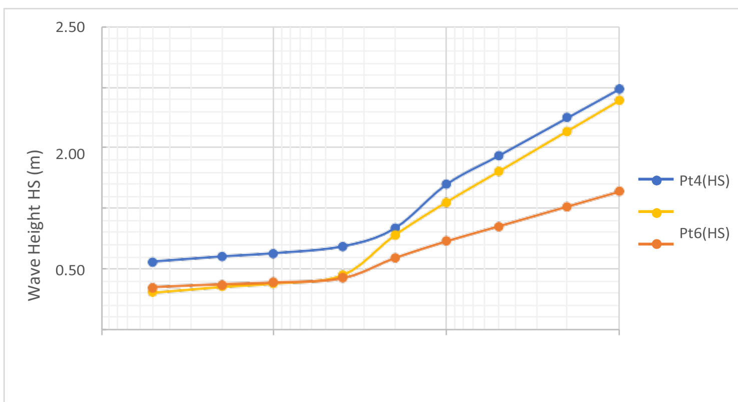


Figure 5.11.4(c): Significant Wave Height (HS) at Stations 4, 6, and 7

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 tobe calculated as broad crested weir flow.

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

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.

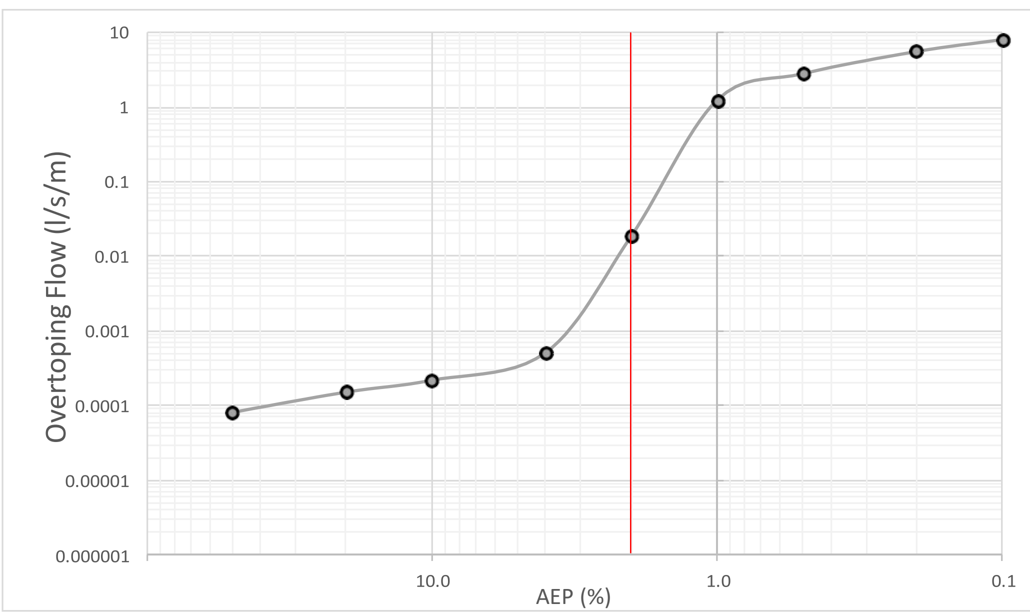


Figure 5.11.6: Overtopping Flow Calculated at Station 6.

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 flowsto calculate total 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

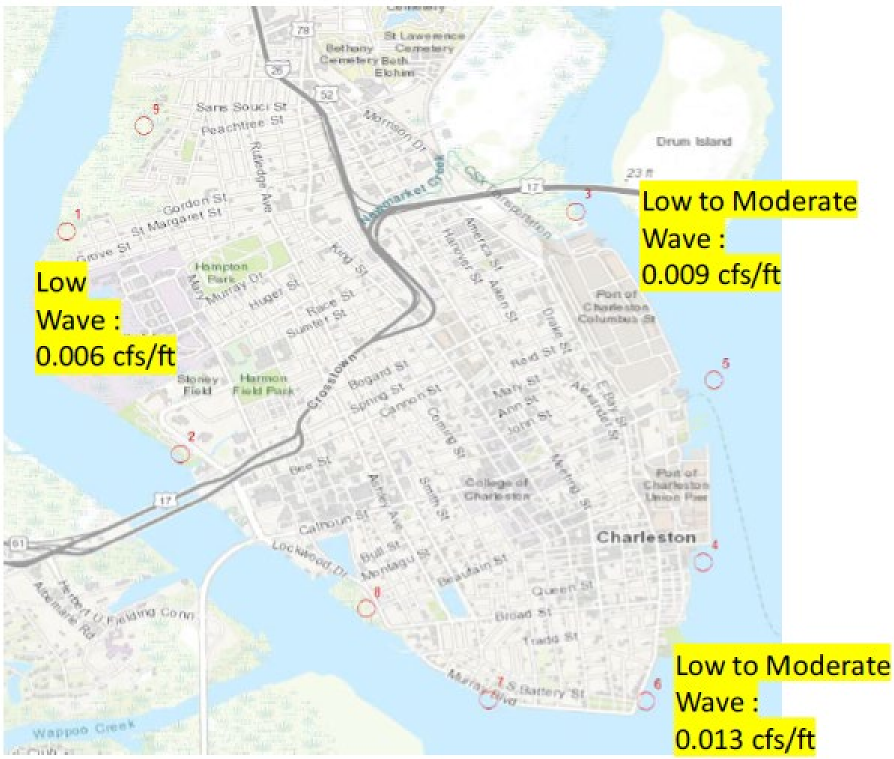


Figure 5.11.7: Overtopping Flow along Different Reaches

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.

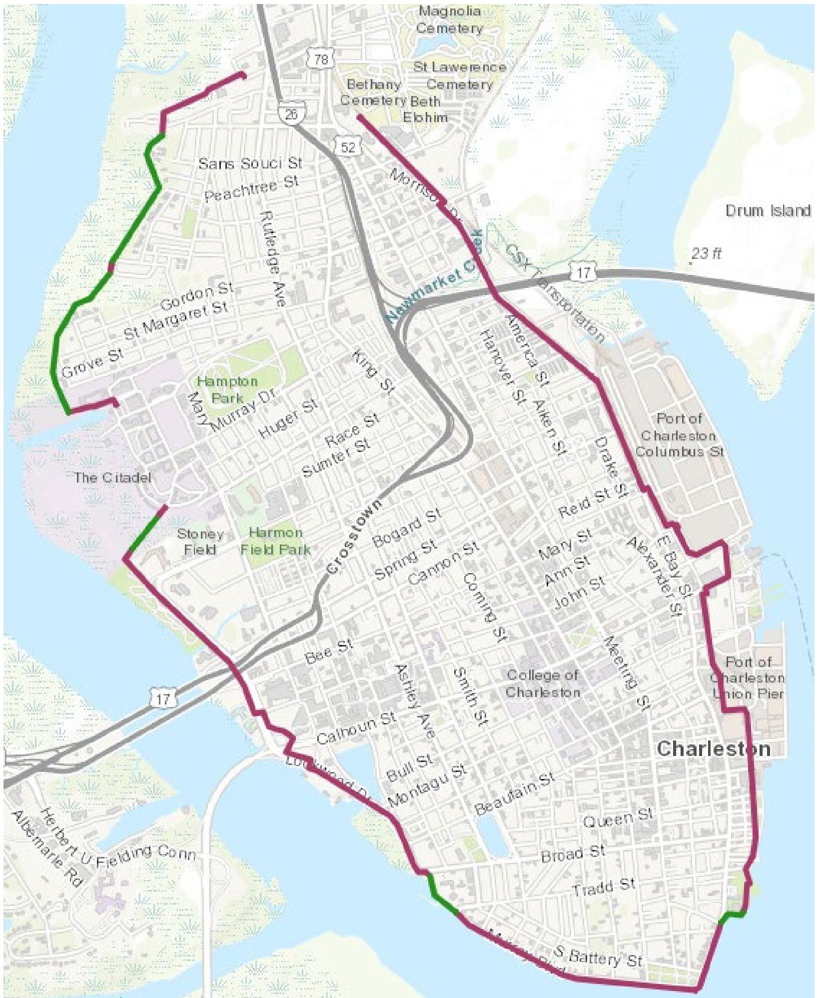


Figure 5.12.1 Wall types (red=Twall; Green =Combo Wall)

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/NEW										
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 T										
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:**	**1,392.50**	**1,392.50**	**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	3.5	-60	8.5	544	544	74630	1089	57061
201+32.29	to	298+29.68	Lowes Hotel -thru port	9,697.40	5.8	5	0.8	3.8	-75	8.2	1940	1940	326288	3881	203645
				12,414.60						**TOTALS:**	**2,485**	**2,485**	**400,919**	**4,970**	**260,707**
298+29.68	to	331+67.84	newmarket	3,338.16	5.3	5	0.3	3.3	-55	8.7	669	669	84594	1337	70101
331+67.84	to	363+66.2	newmarket	3,198.36	8.4	5	3.4	6.4	-55	5.6	641	641	85089	1281	67166
363+66.2	to	364+06.26	Upper new market	40.06	12.85	5	7.85	10.85	-55	1.15	9	9	1278	18	841
				6,576.58						**TOTALS:**	**1,318**	**1,318**	**170,961**	**2,637**	**138,108**
				18,991.18							**3,803**	**3,803**	**571,880**	**7,606**	**398,815**
Phase 4 - WAGNER TERRACE															
0+00	to	8+25.	Diesel Creek	825	9.8	5	4.8	7.8	-55	4.2	166	166	22519	332	17325
13+65	to	20+40.	owndes Poin	675	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	979.7	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 Quanitities

Stations			Description	Length (ft)	* Average Existing Grade Elev. (ft)	Top of Fdn Elev. (ft)	* Cooper Marl Elev (ft)	Wall Height (ft)	Excav Vol (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 (EA)	Total Sheet Pile Quantity (SF)
Phase 1 - MARINA																
77+60.	to	99+73.03	Marina to Lockwood/ Broad	2,213.03	4.9	2.9	-65	9	10,246	2459	1119	623	437	4546	887	46474
				2,213.03				**TOTALS:**	**10,246**	**2,459**	**1,119**	**623**	**437**	**4,546**	**887**	**46,474**
Phase 2 - BATTERY																
155+62.3	to	169+04.98	battery	1,342.68	4.5	2.5	-75	9.5	6216	1492	709	398	265	2758	539	28196
				1,342.68				**TOTALS:**	**6,216**	**1,492**	**709**	**398**	**265**	**2,758**	**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
- 17 storm surge gates (sluice gates)
- 51 vehicle gates
- 14 pedestrian gates
- 2 railroad gates

More details are in the Cost estimate.

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 risk assessment 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 (Interim Approach 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.

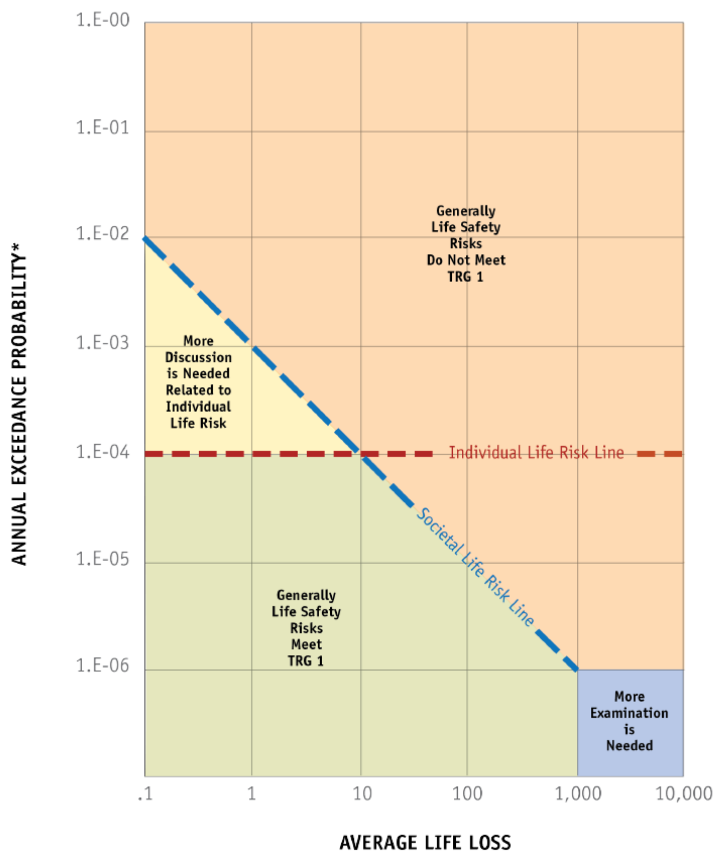


Figure 5.14.1 Standard Life Safety Risk Matrix

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

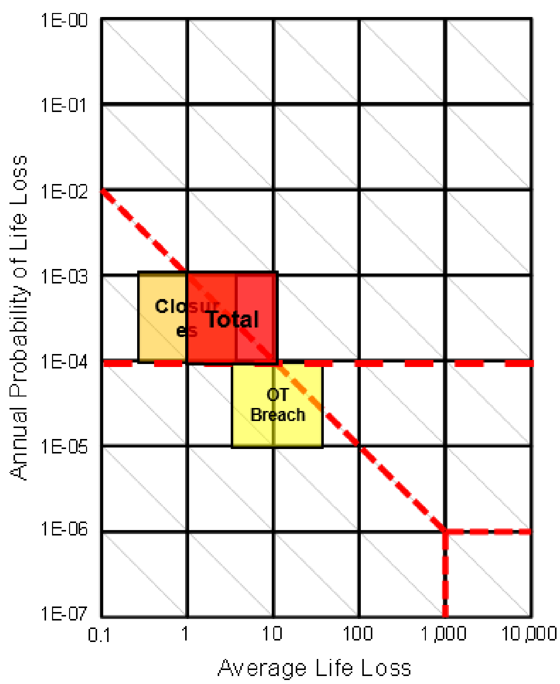


Figure 5.14.2 Societal Incremental Life Safety Risk Matrix

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. PFM 12 and 39 were also combined into a single PFM for gate(s) not being closed. More information can be found in the Charleston Planning Risk Assessment Sub-Appendix.

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 (SWL) from the most recent Flood Insurance Study, ERDC generated of Annual Exceedance Probability (AEP) for each of the save points requested. Still water level values in MSL were converted to NAVD88 and sea level rates 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 when communicating risk, a project performance, the AEP values at the 90% confidence level should be used. AECOM, contractor for FEMA, provided confidence limit formulas to apply. Table 5.14.1 lists the AEP with 90% confidence at the 5 locations selected for Model Areas for the year 2032 using the intermediate rate of Sea Level Change of .56 feet projected in 2032. Figure 5.14.4 is the same information plotted. Based on this the probability of annual exceedance for the wall at elevation 12 NAVD88 is approximately 2.6 % with a 90% confidence.

Table 5.14.1 Year 2032 Annual Exceedance with 90% confidence

	SWL	**SLR =**	**0.56**						
Area	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88
	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**
Wagner 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

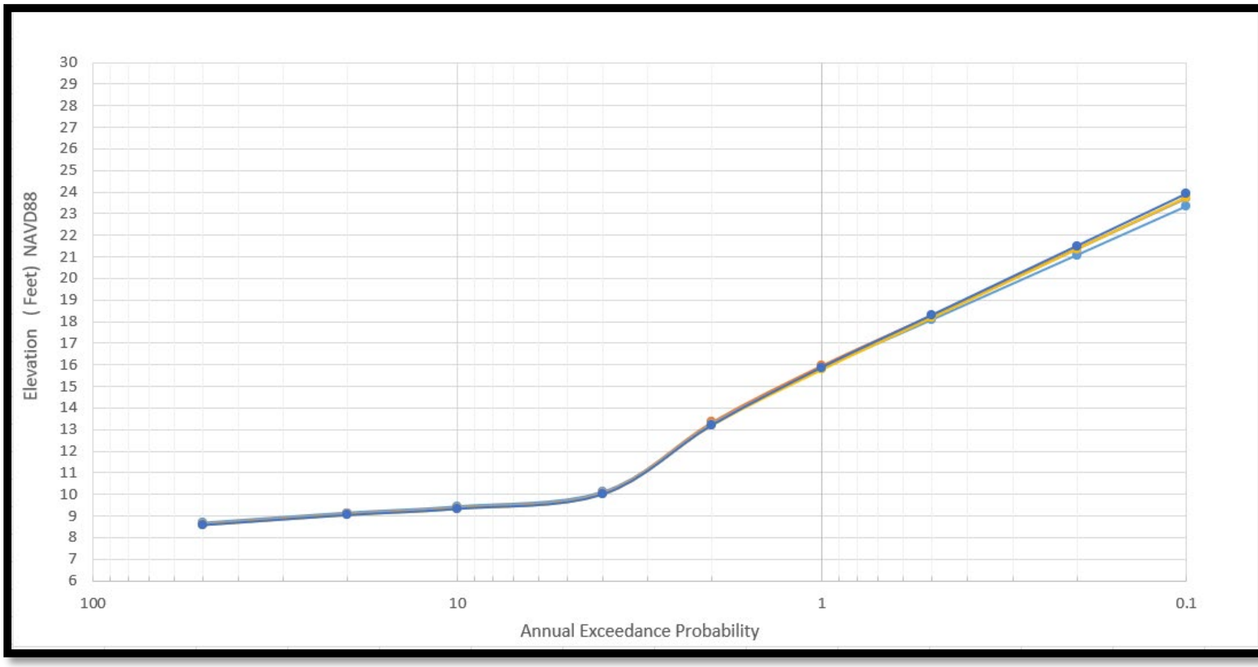


Figure 5.14.4 Year 2032 Annual Exceedance with 90% confidence

Using the intermediate rate of Sea Level Change of 1.65 feet projected in 2082, Table 5.14.2 lists the AEP with 90% confidence at the 5 locations selected for Model Areas. Figure 5.14.5 is the same information plotted. In the year 2082, the end of the economic project life, the probability of SWL annual exceedance of the 12 NAVD88 wall elevation is approximately 3.5 % with a 90% confidence.

Table 5.14.2 90% Confidence Annual Exceedance Probability Year 2082

	SWL	**SLR =**	**1.65**						
Area	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88
	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**
Wagner 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

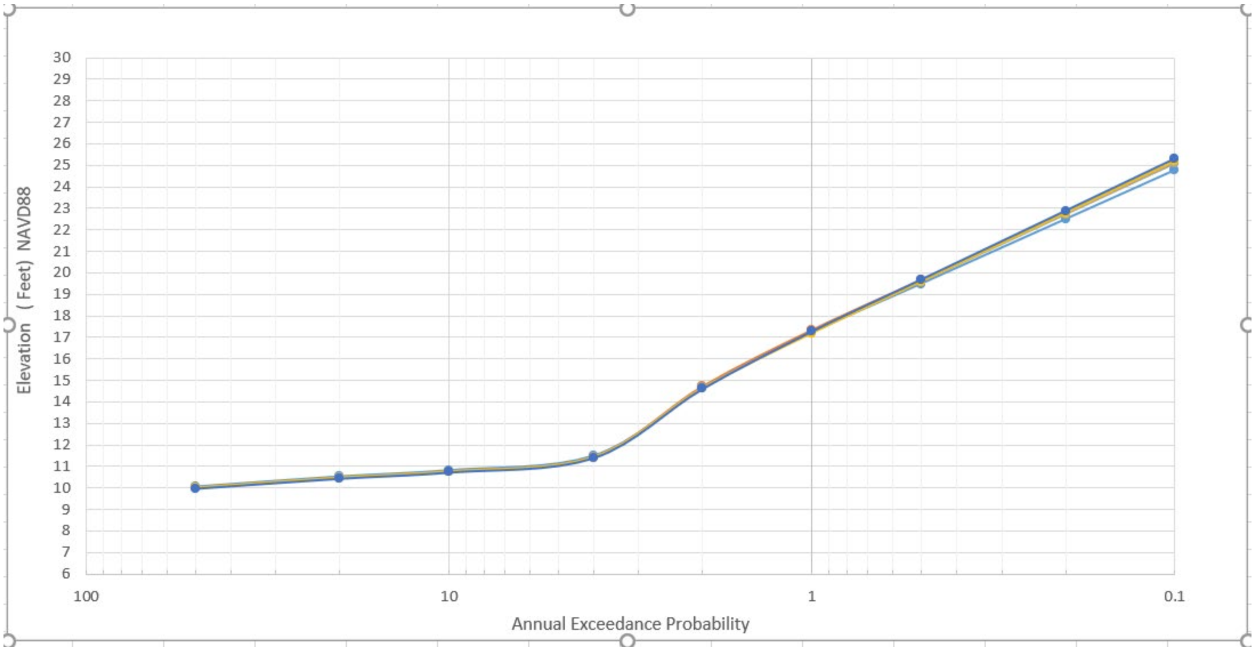


Figure 5.14.5 Year 2082 Annual Exceedance with 90% confidence

Also, 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. This was considered in G2CRM.

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 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 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.

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 PRECONSTRUCTION ENGINEERING AND DESIGN (PED) CONSIDERATIONS

6.1 FUTURE WORK REQUIRED IN 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.

6.1.1 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.

6.1.2 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.

6.1.3 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.

6.1.4 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.

6.1.5 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.

6.1.6 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.

6.1.7 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.

6.1.8 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.

6.1.9 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.

6.1.10 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.

6.1.11 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.

6.2 OPERATION and MAINTENANCE MANUAL

"Once a functional portion of the project has been constructed, the local sponsor will be notified, and USACE will provide an operations, maintenance, repair, replacement, and rehabilitation (OMRR&R) manual which will be written specifically for the local sponsor, the City of Charleston, who will have the primary responsibility for operating and maintaining the project. The intent of the document is to provide the local sponsor with some clear and comprehensive guidance on the operation and maintenance of floodwalls and other flood risk management structures. It will describe how to plan and prepare for high water and storm events and lays out steps to take during emergencies that will help reduce the threat of flooding. The manual will also explain the types of assistance that the U.S. Army Corps of Engineers can provide to a community before, during, and after a flood. Monitoring and inspections must occur to ensure that the project functions as designed and that the local sponsor confirms to all OMRR&R recommendations and requirements that will assist in functionality of the project. USACE will inspect the project each year with the City of Charleston (the local sponsor).

USACE conducts two types of levee and floodwall inspections: Routine Inspection and Periodic Inspection.

Routine Inspection is a visual inspection to verify and rate levee/floodwall system operation and maintenance. It is typically conducted each year for all levees/floodwalls in the USACE Levee Safety Program.

Periodic Inspection is a comprehensive inspection conducted by a USACE multidisciplinary team that includes the local sponsor and is led by a professional engineer. USACE typically conducts this inspection every five years on the federally authorized levees in the USACE Levee Safety Program. Periodic Inspections include three key steps:

(1) Data collection - A review of existing data on operation and maintenance, previous inspections, emergency action plans and flood fighting records.

(2) Field inspection - Similar to the visual inspection for a Routine Inspection, but with additional features.

(3) Final report development - A report including the data collected, field inspection findings, an evaluation of any changes in design criteria from the time the levee/floodwall was constructed, and additional recommendations as warranted, such as areas that need further evaluation.

Both Routine and Periodic Inspections result in a final inspection rating for operation and maintenance. The rating is based on the levee/floodwall inspection checklist, which includes 125 specific items dealing with the operation and maintenance of levee embankments, floodwalls, interior drainage, pump stations, channels, operation and trial erections of closure structures, and inspection/video inspection of pipes/conduits that pass through the project alignment. Each levee/floodwall segment receives an overall segment inspection rating of Acceptable, Minimally Acceptable, or Unacceptable. If a levee/floodwall system comprises one or more segments of the project, then the overall project system rating is the lowest of the segment ratings. The local sponsor must maintain the levee/floodwall to at least the minimally acceptable standard to remain eligible for federal rehabilitation assistance through the USACE Rehabilitation and Inspection Program (PL 84-99). USACE also shares the results with FEMA, to help inform decisions about levee accreditation for flood insurance purposes. The inspection ratings are available in the National Levee Database.

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 levees, riprap, and the areas adjacent to floodwalls, and from channels and waterways. Local sponsor will need to periodically install 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. To assist with monitoring, certain tools 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.

1. STRUCTURAL SUB-APPENDIX
2. GEOTECHNICAL SUB-APPENDIX
3. HYDRAULICS AND HYDROLOGY-HEC-RAS 2D Modeling SUB-APPENDIX
4. COASTAL SUB-APPENDIX
4A. ERDC Coastal Subappendix
5. COST ENGINEERING SUB-APPENDIX

US Army Corps of Engineers

Charleston District

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

Charleston, South Carolina

STRUCTURAL SUB-APPENDIX

MAY 2021
REVISED AUGUST 2021

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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)

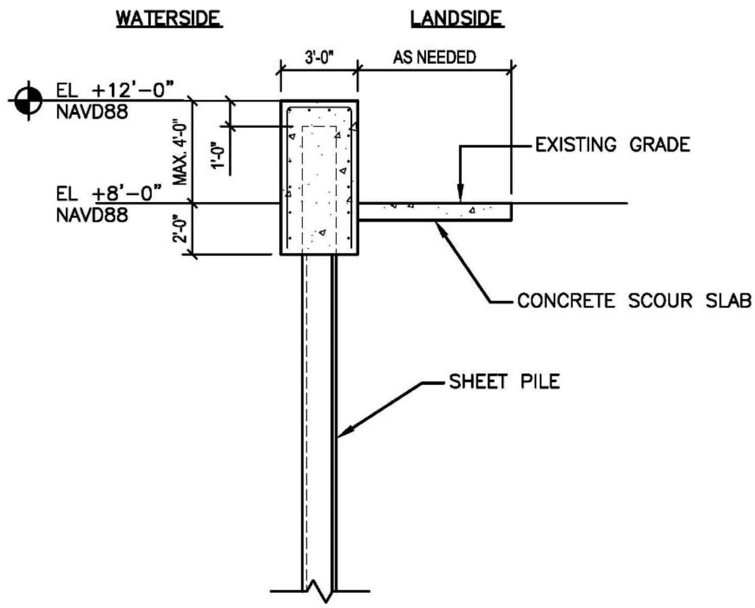
Earth Berms (Levees) were ruled out at a viable option due their large footprint requirement (i.e., 10 ft wide top, 3 to 1 slope, vegetative free zone on each side, etc.). The study is limited to the peninsula of Charleston, where the land has been heavily developed and available land is very scarce. 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

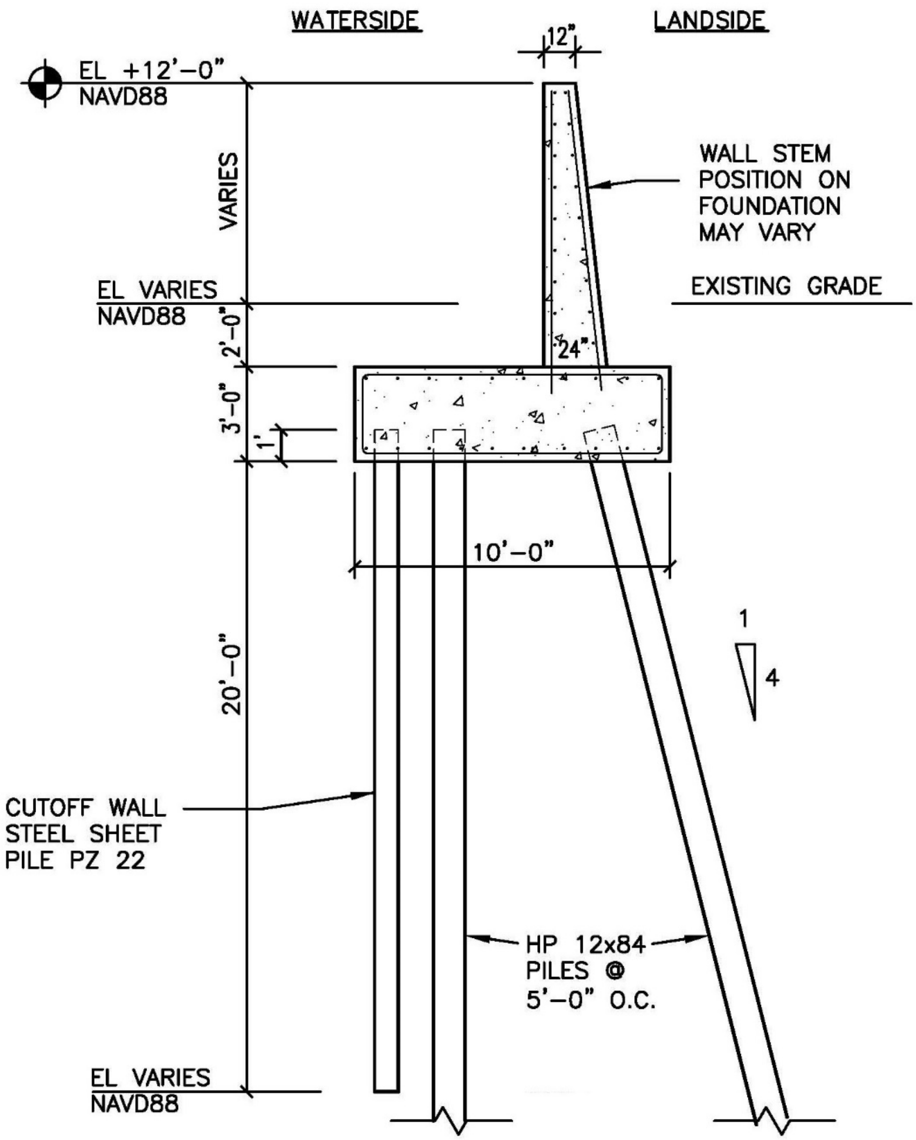
I - Walls were ruled out as a viable option for flood wall construction. This type of barrier would consist of driven sheet pile walls with a concrete cap. I - Walls occupy a small footprint and would be desirable in areas where space is limited. I – Walls did not perform well in New Orleans during Hurricane 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

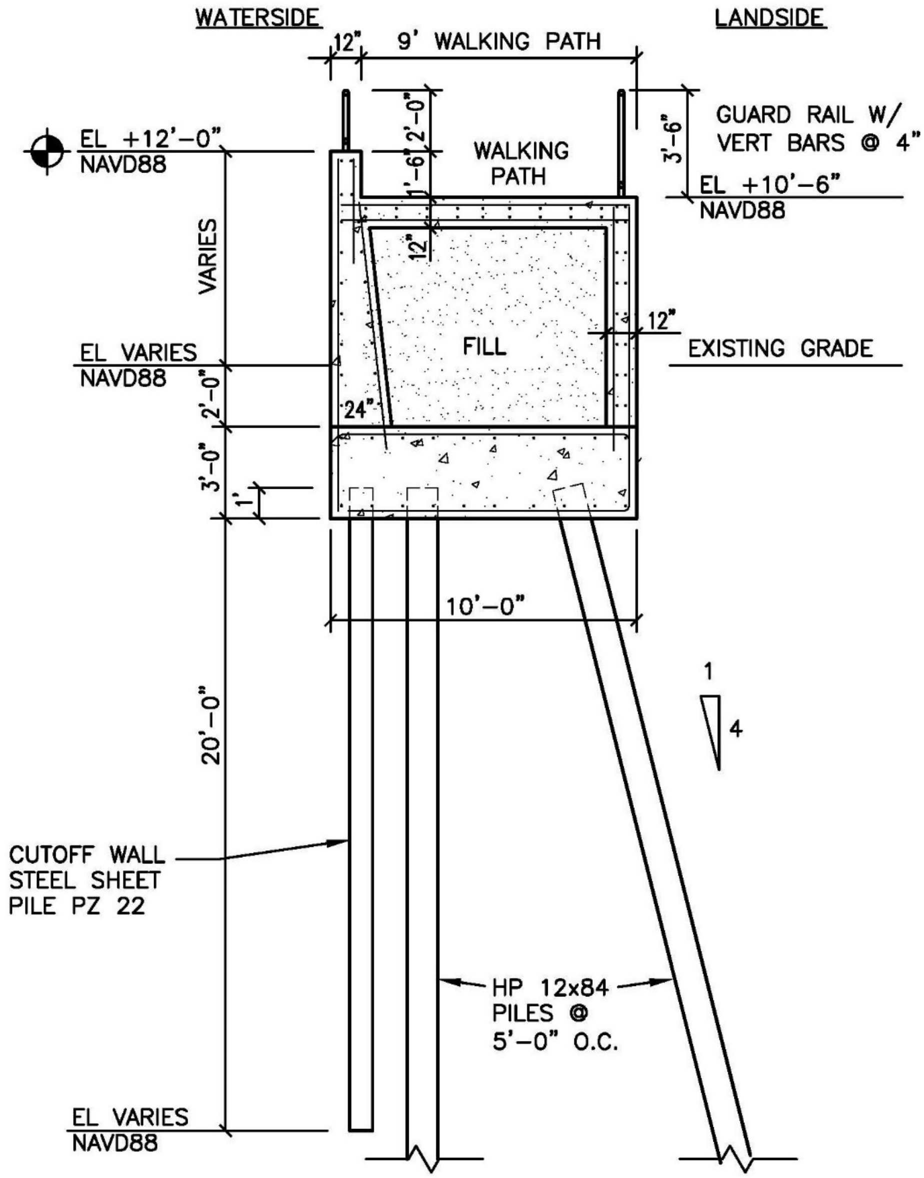
T - WALL

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

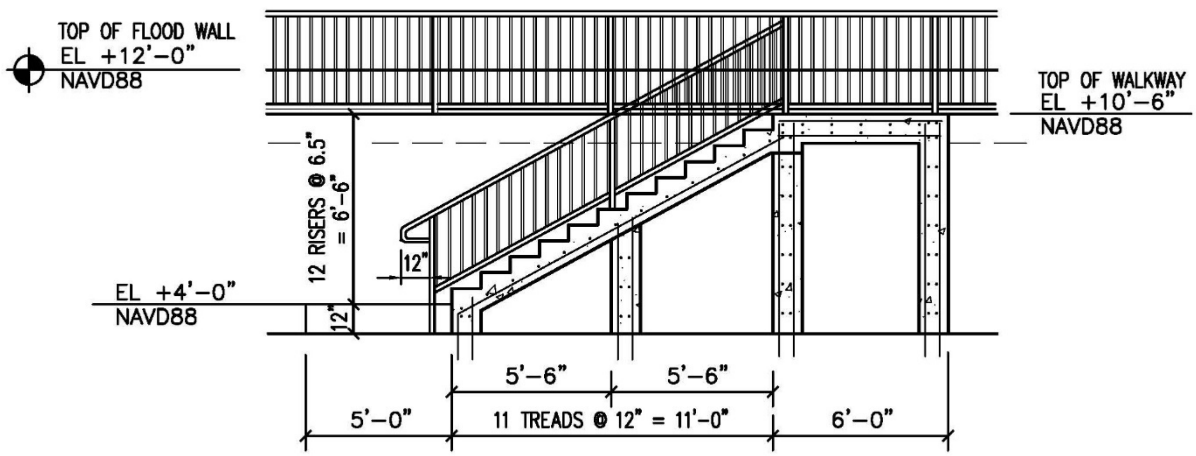
T – Walls with walkways are planned to be constructed in scenic areas such as along Lockwood Blvd and Brittlebank Park, and to replace the existing High Battery wall. The T – Wall with walkway section is similar to the T wall except that the stem is moved to the waterside and the walking path is constructed 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.



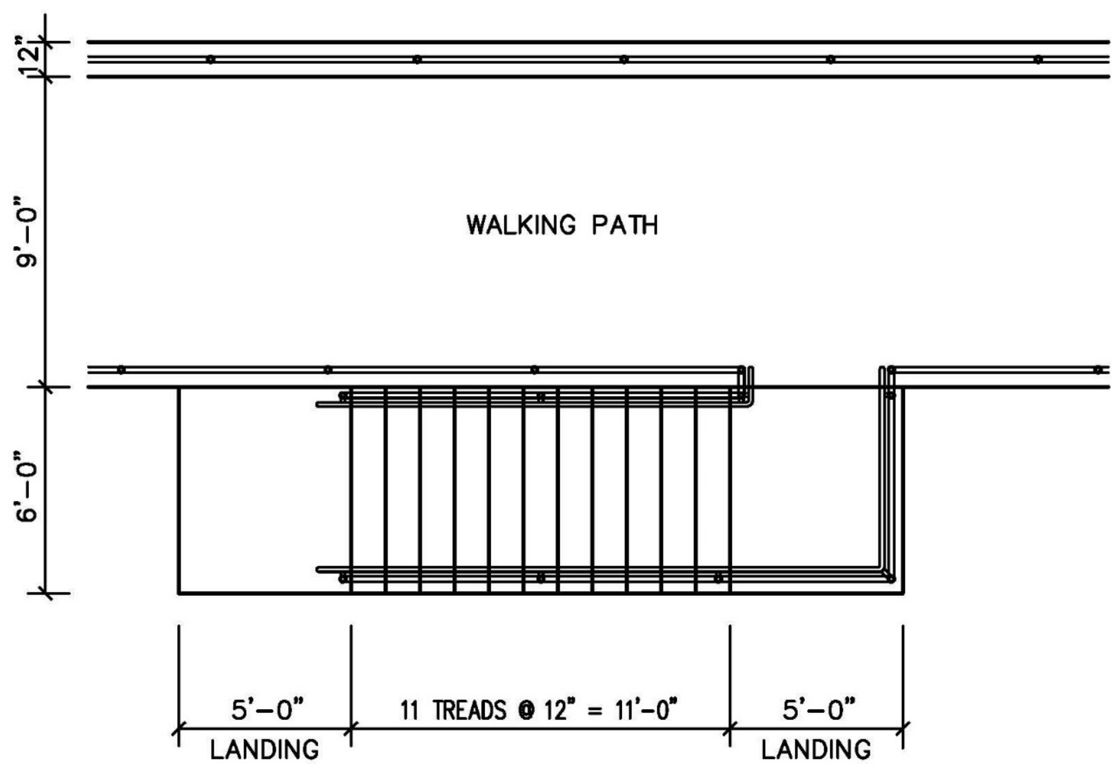
T - Wall with Walkway Typical Section

STAIRS and RAMPS

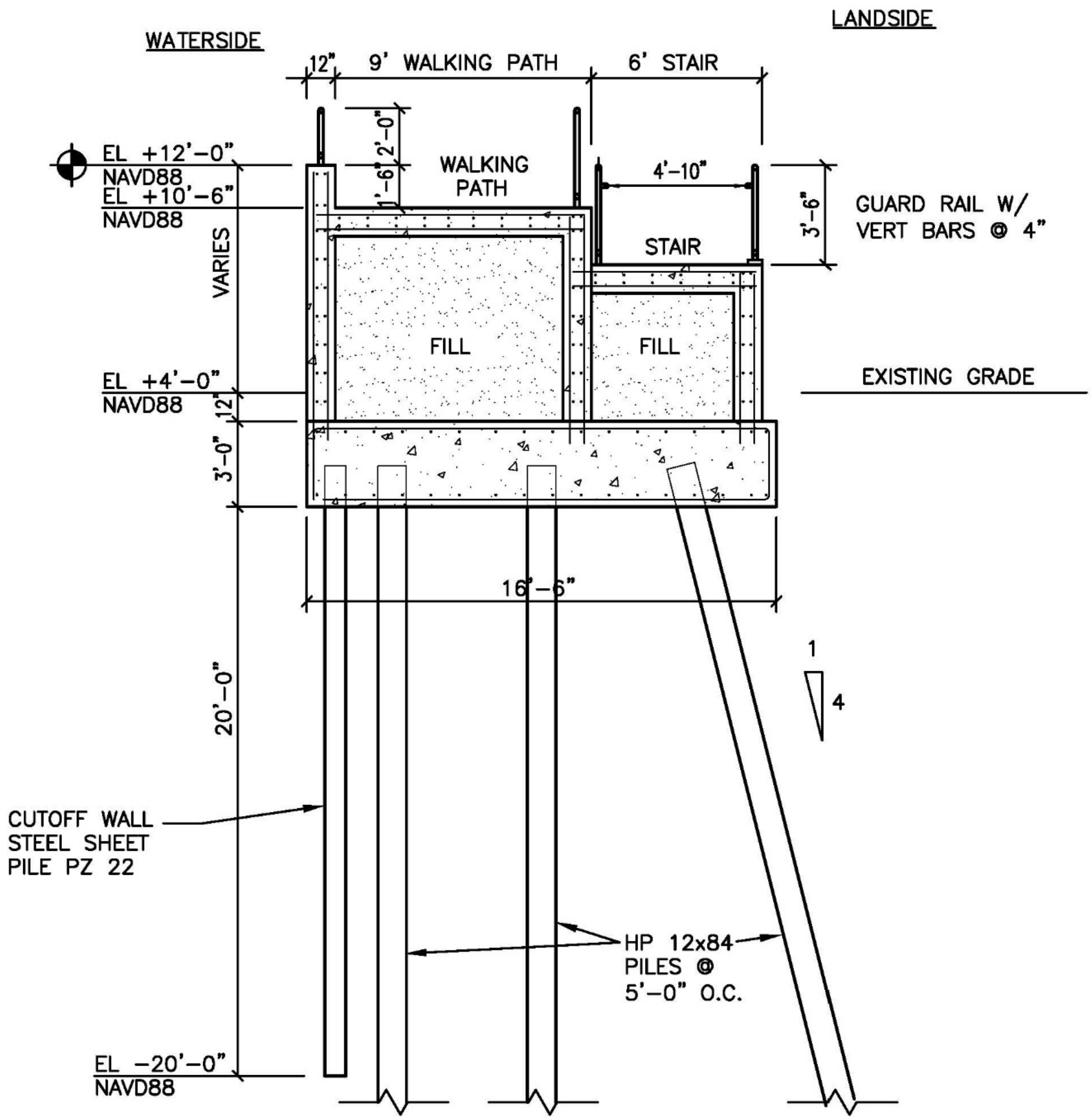
Stairs and Ramps will be required for pedestrian access to the T – Wall with walkway. Stairs and parallel Ramps would require a wider foundation than the typical T – Wall with walkway. "In Tandem" ramps would be identical to the typical T – Wall with walkway, sloping down to grade and would thus avoid 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.



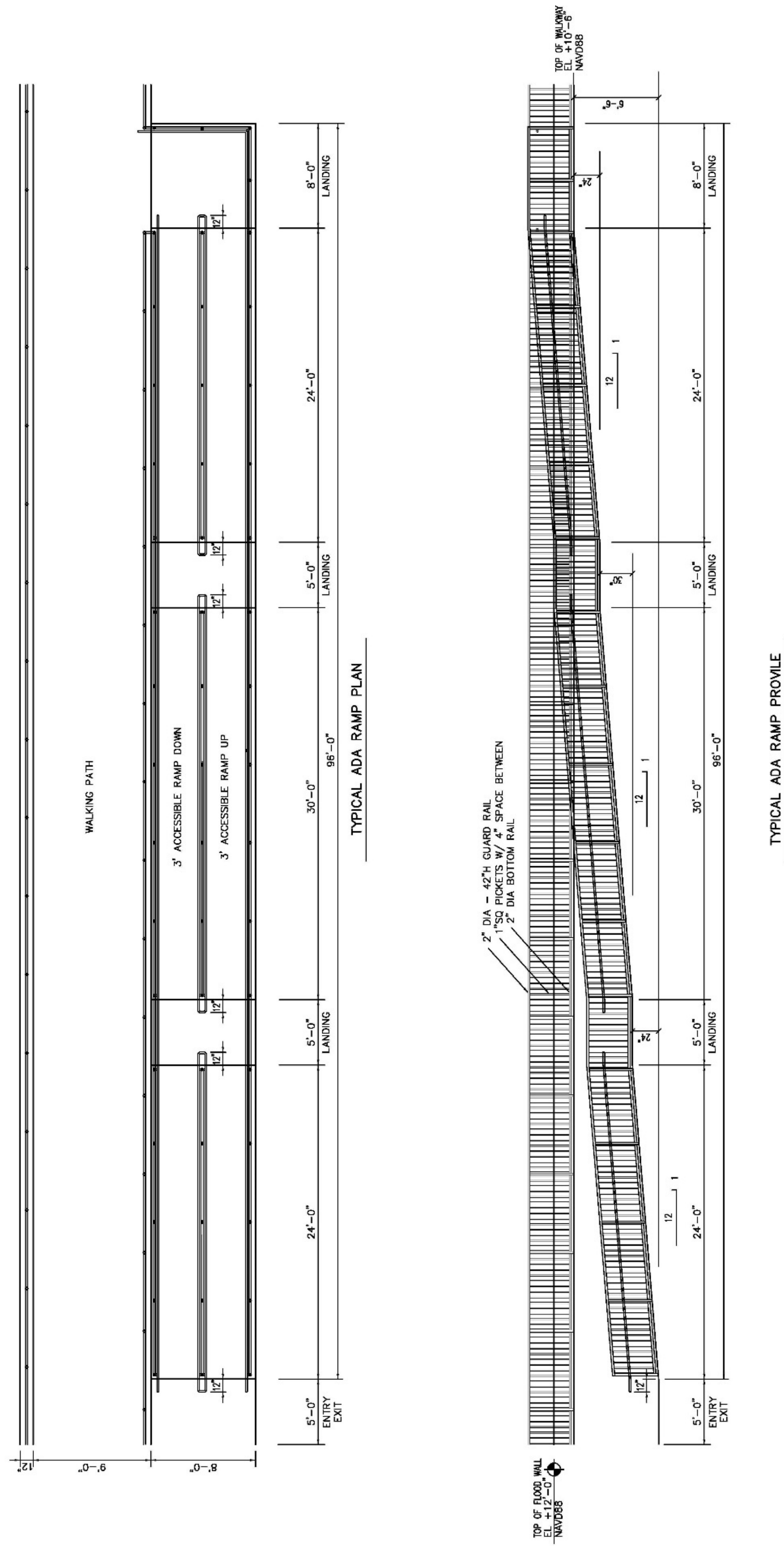
Stair Elevation



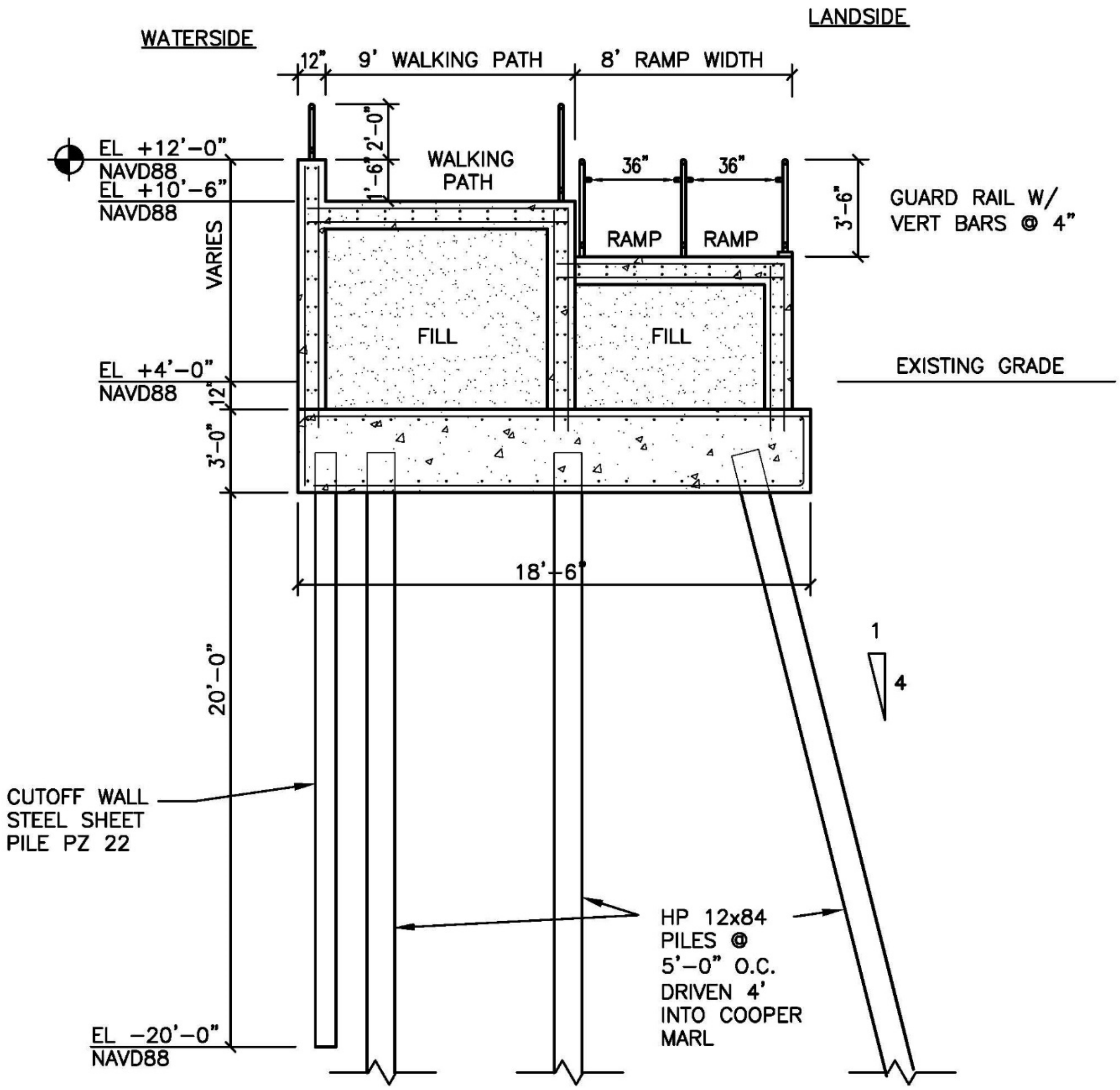
Stair Plan



T Wall with Walkway and Stair Typical Section



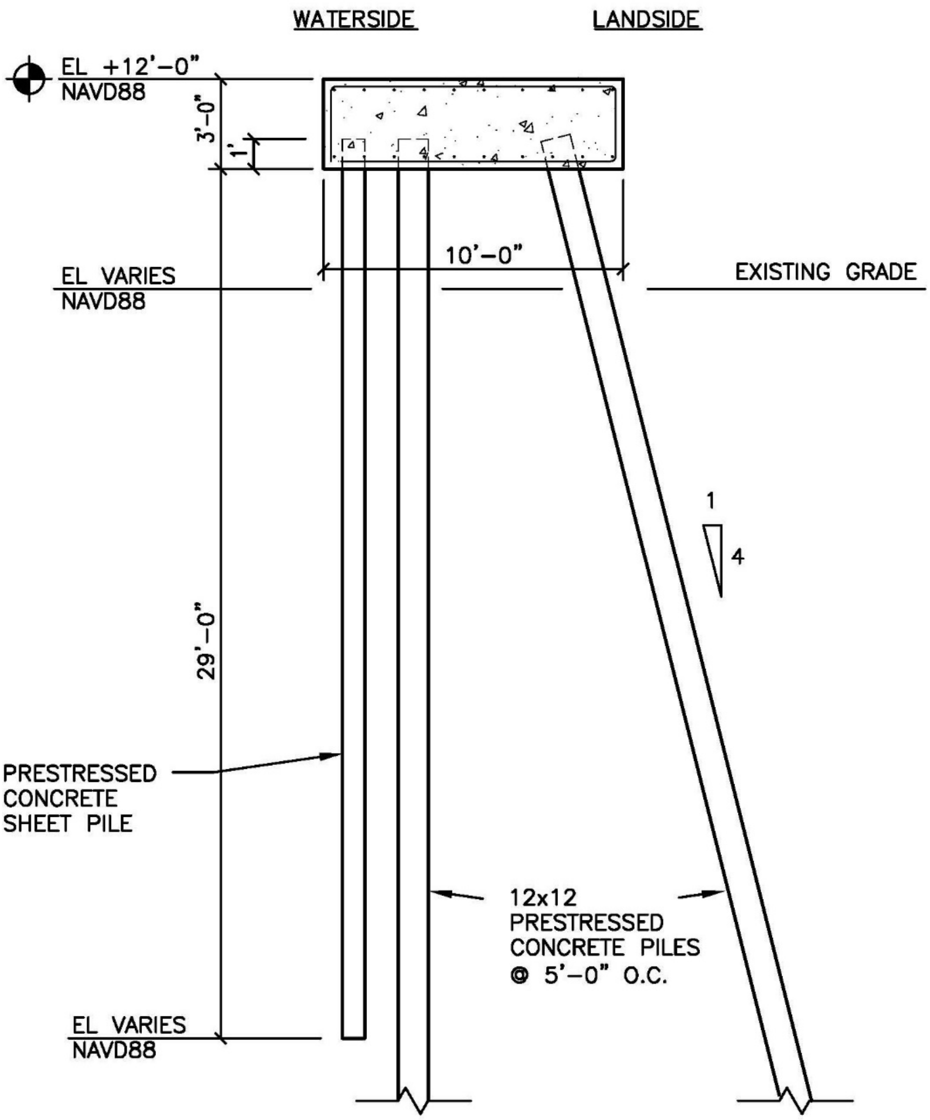
Parallel Ramp for Disabled Access



Parallel Ramp for Disabled Access Typical Section

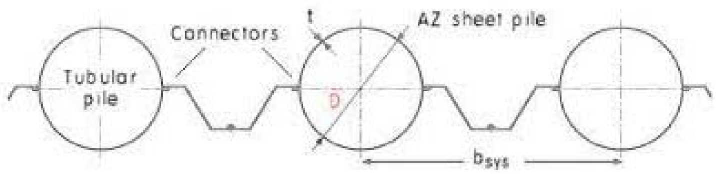
COMBO WALL

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.

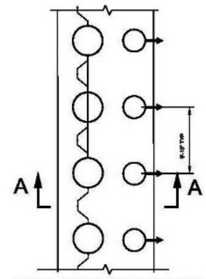


Typical section of Combo Wall

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.

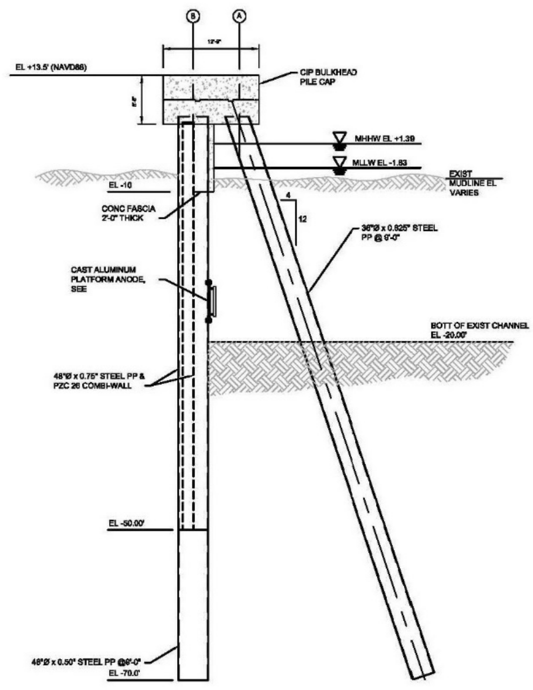


Plan of Tubular Pile with Sheet Pile Between



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.

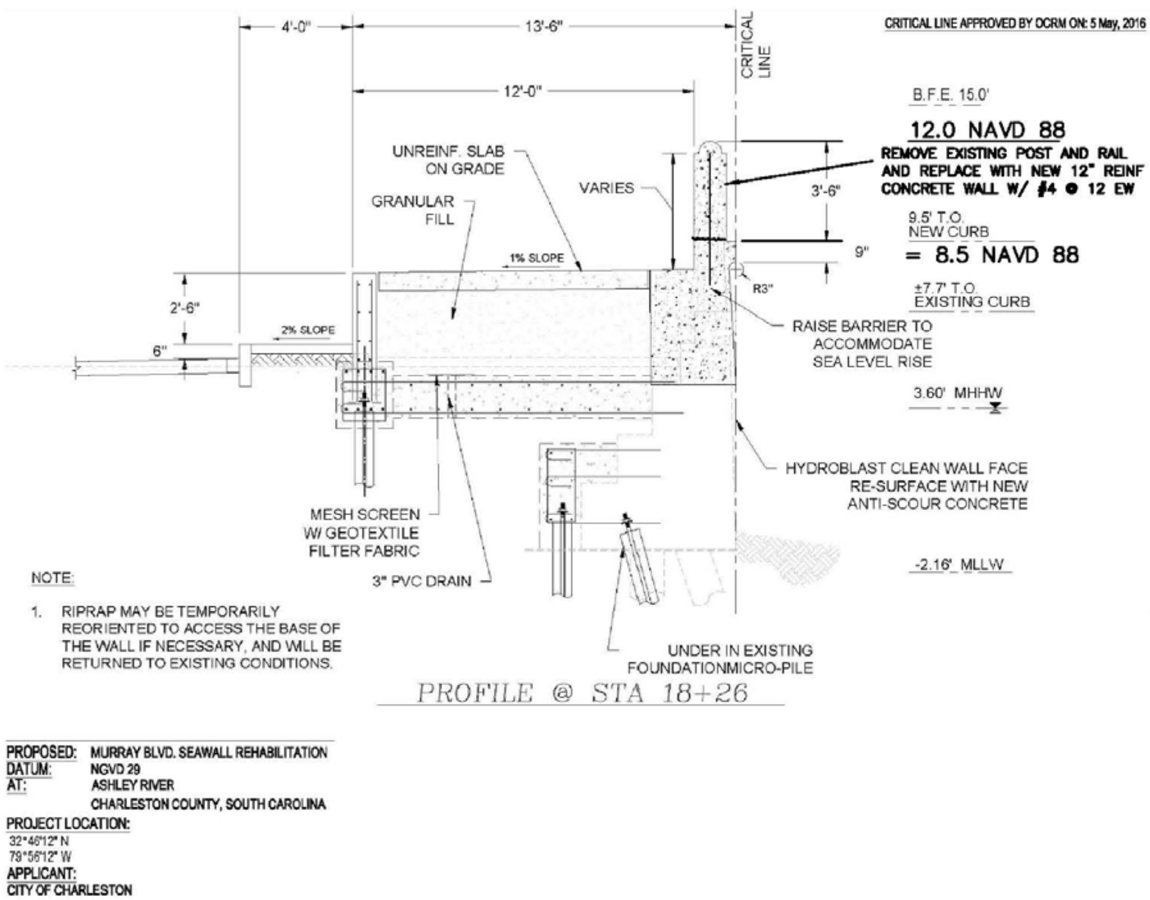
WORK TRESTLE

AERIAL VIEW OF WORK TRESTLE

VIEW FROM DECK OF WORK TRESTLE

LOW BATTERY WALL

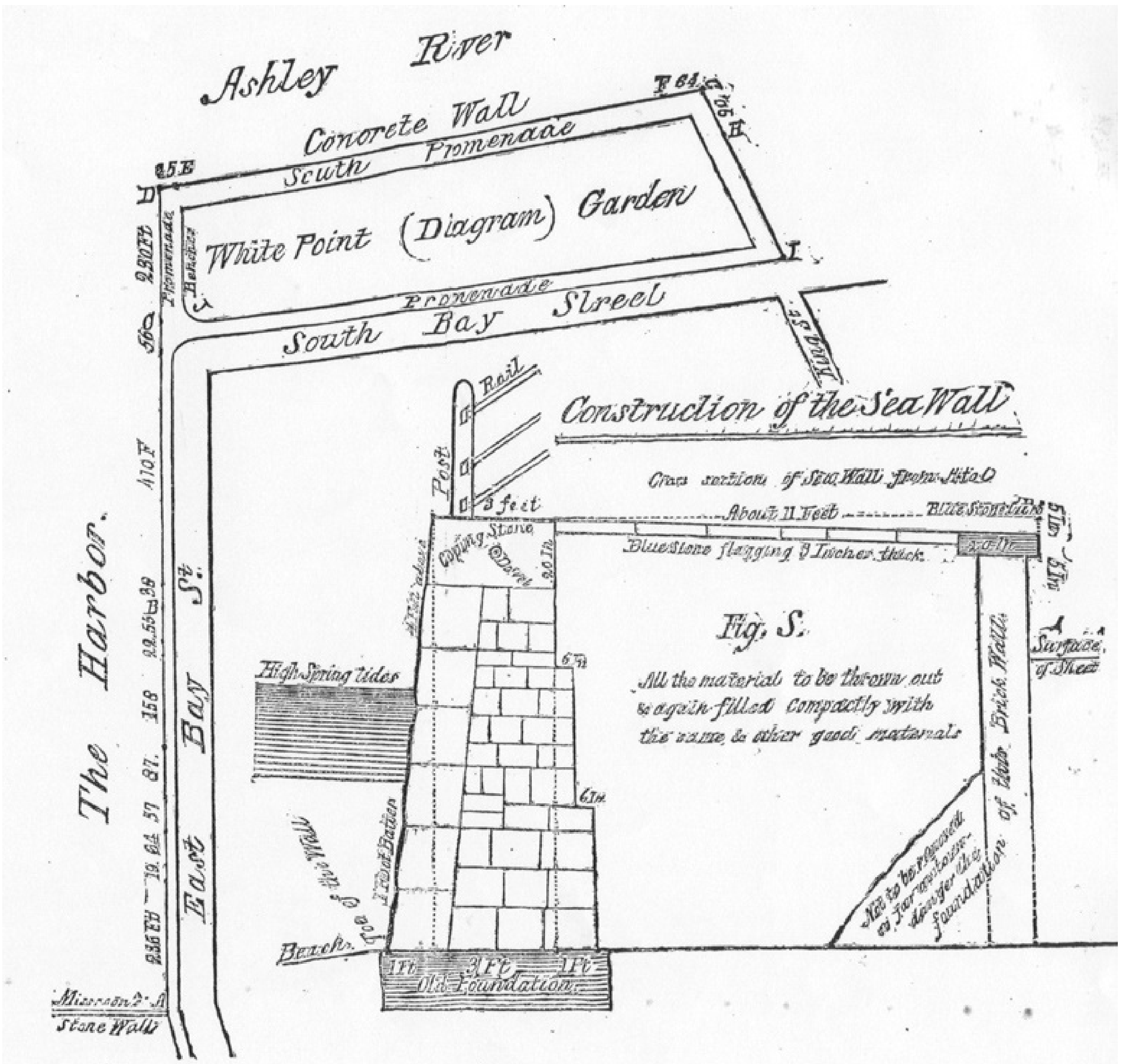
The Low Battery Wall was recently renovated by the City of Charleston and provides a level of protection to EL 9 ft NAVD 88. The designer of record stated that the new Low Battery Wall was designed to provide a level of protection of EL 12 ft NAVD 88. The wall can be retrofitted to provide a level of 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

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.



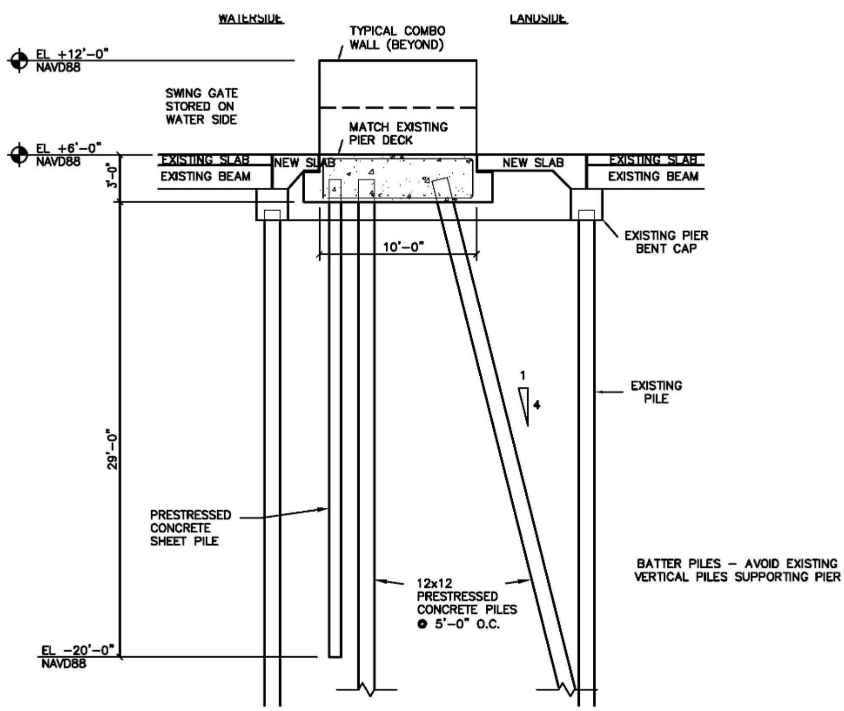
Existing High Battery Wall

GATES

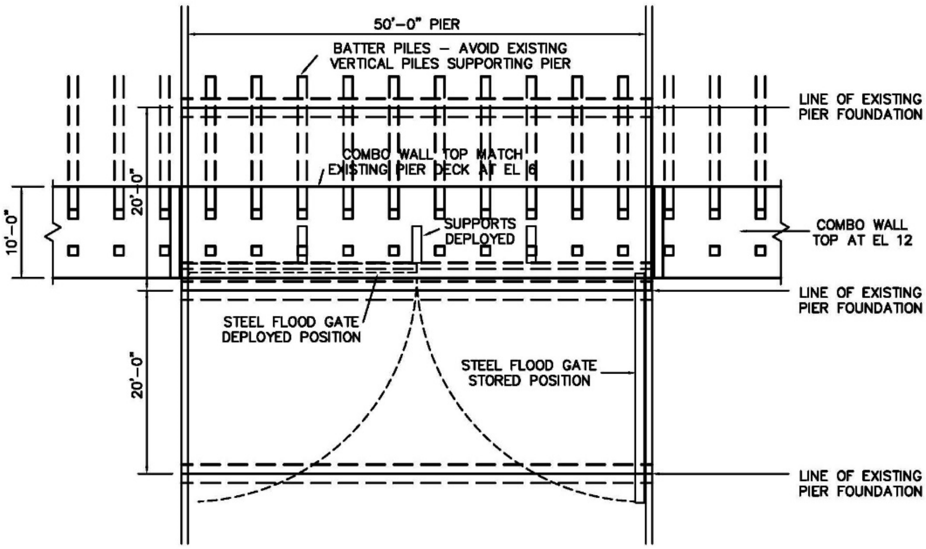
Preliminary structural analysis and design was performed for the gates. Many types of gates will be required ranging from very long gates across roadways to very short gates across pedestrian access routes.

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.

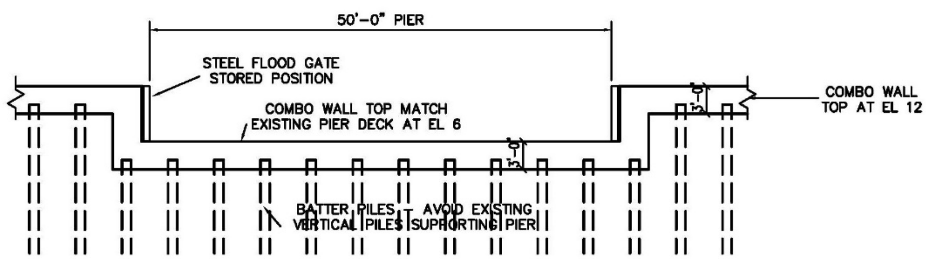
Aerial View at Coast Guard Base



Typical section of Combo Wall Crossing Coast Guard Dock



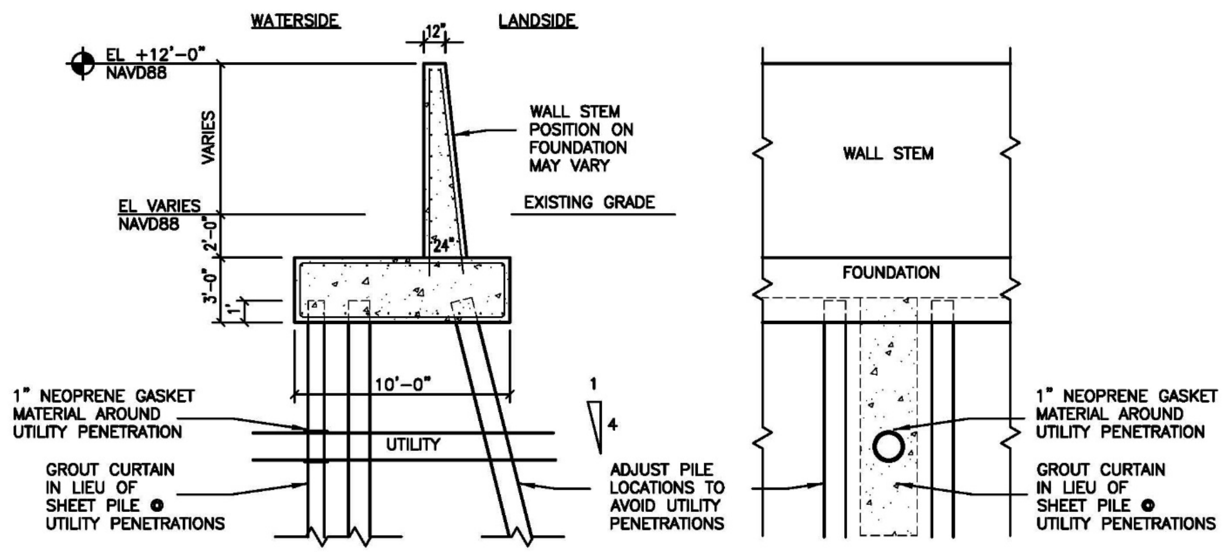
Plan of Combo Wall Crossing Coast Guard Dock



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.

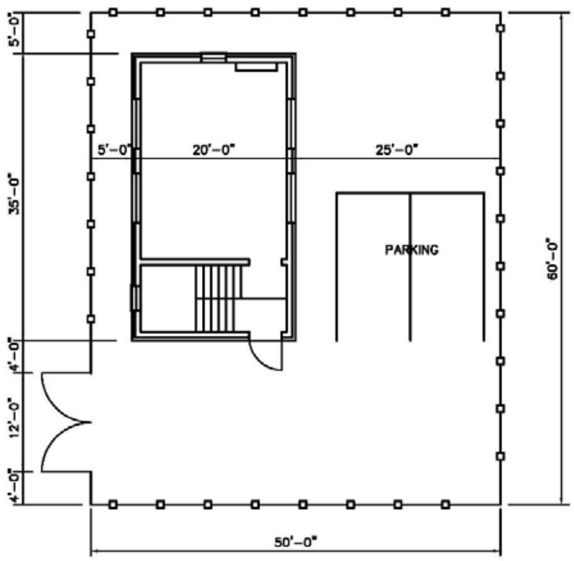
CORROSION PREVENTION

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:

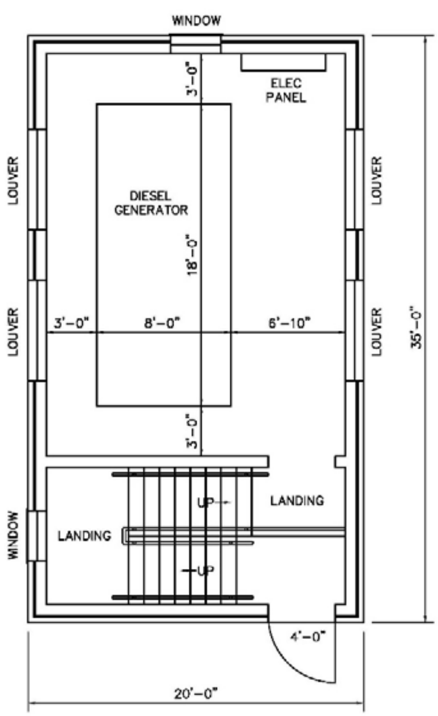
Noncorrosive rebar, such as galvanized, epoxy coated, or FRP composite
Noncorrosive sheet pile, such as prestressed concrete, vinyl, or FRP composite
Corrosion inhibiting admixtures for concrete
Stainless steel for railings and hardware

PUMP STATION

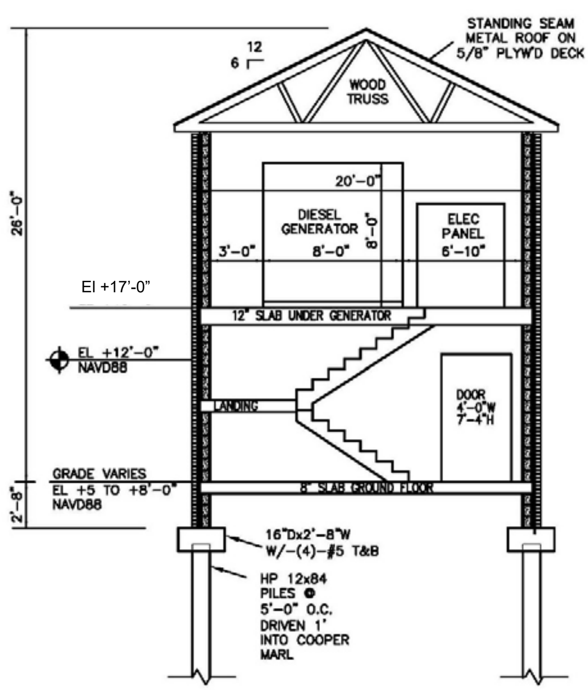
Pump stations for interior drainage will be required in several locations. Equipment such as electrical panels and controls will need to be elevated above the flood elevation and contained in a building for protection. The floor elevation is set at EL 17 to accommodate 3' future raising and an additional 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)

$\Rightarrow C_F = 1.3$

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

$\therefore$ WIND PRESSURE FOR DESIGN OF BARRIER SHALL BE $\underline{58.73}$ 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:

WATERSIDE **LANDSIDE**

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:

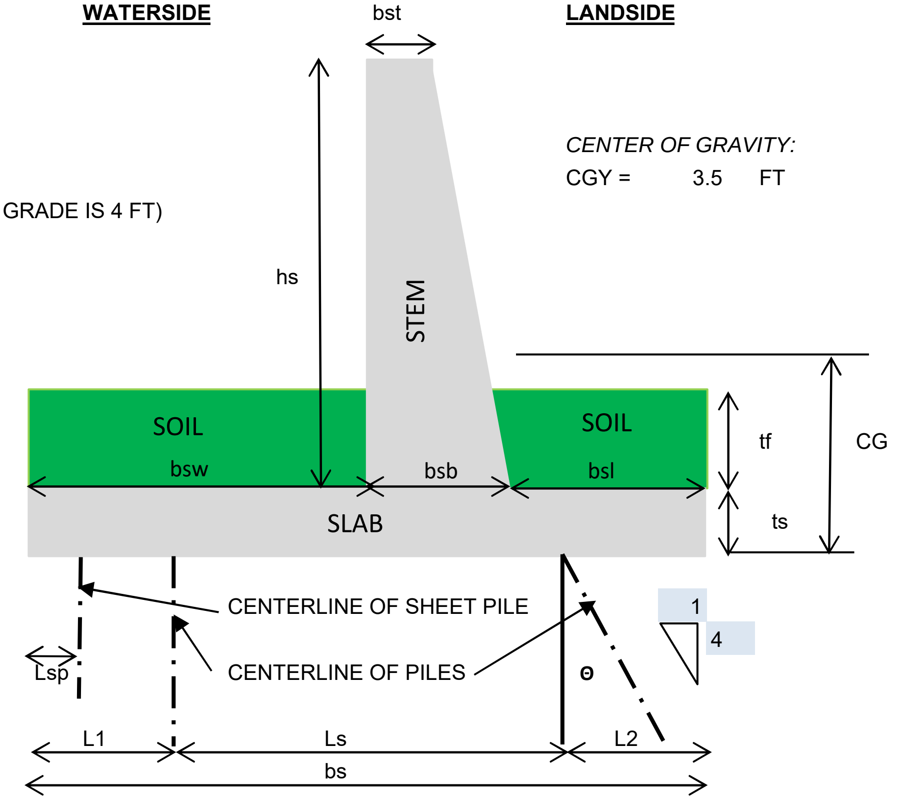
ϴ	14.04	DEG (4V:1H)	
L1	3	FT	
L2	3	FT	
Ls	4	FT	
s	5	FT	(spacing)
Lsp	2	FT	

SOIL:

tf	2	FT
ϕ	20	°
Ko	0.49	

CENTER OF GRAVITY:

CGY = 3.5 FT



LOADS

γc	150	PCF (WEIGHT OF CONCRETE)	Uw1	810	PSF (UPLIFT PRESSURE WATERSIDE OF SHEET PILE)
γw	64	PCF (WEIGHT OF WATER)	Uw2	315	PSF (UPLIFT PRESSURE LANDSIDE OF SHEET PILE)
γs	120	PCF (WEIGHT OF SOIL)			
Eh	0.36	x WEIGHT OF T-WALL			
W	58.73	PSF (WIND PRESSURE)			

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

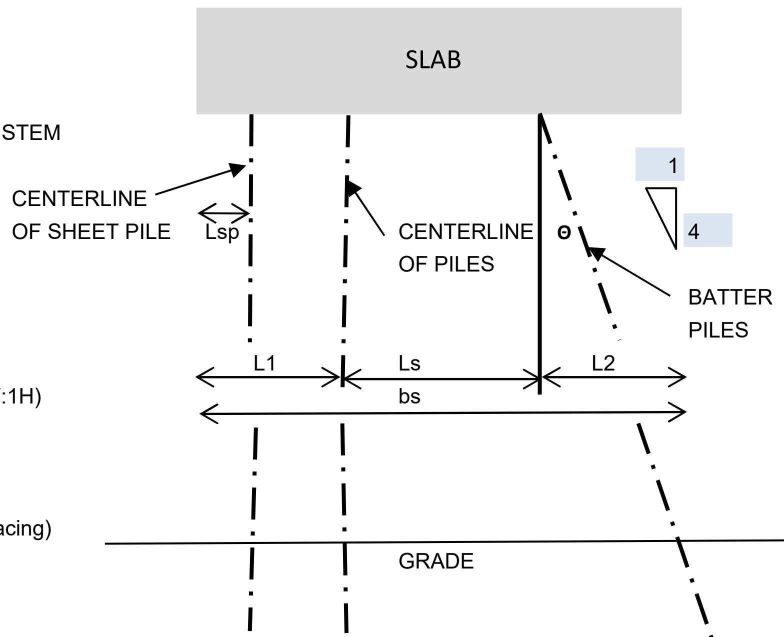
STEM:

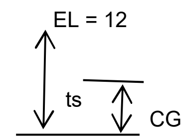
bst	1	FT
bsb	2	FT
hs	0	FT

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

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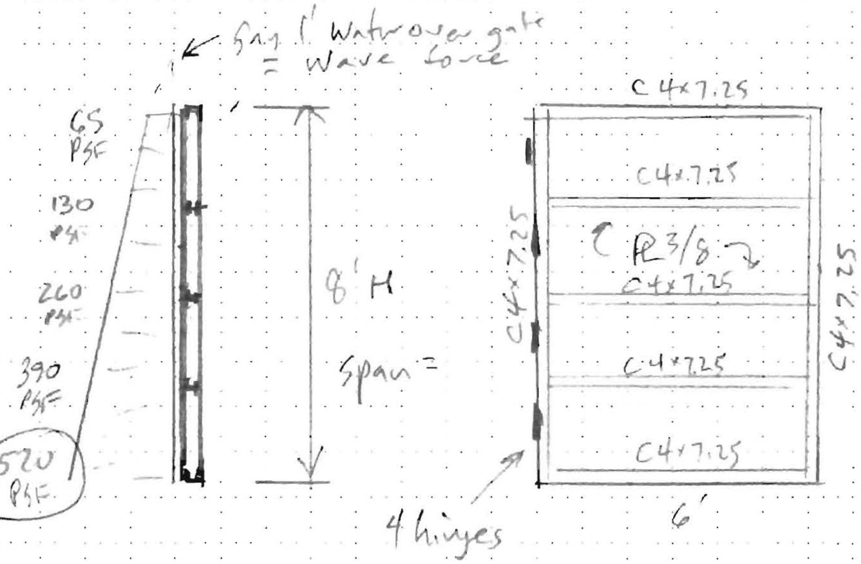
CHARLESTON DISTRICT

Project: Peninsula Study
Subject: Ped Gates
Design by: RDL Check by:
Date: 10/9/20 Sheet 1 of 3

Ped Gates: 6' W x 8' H

single swing

say 1' water over gate = wave force



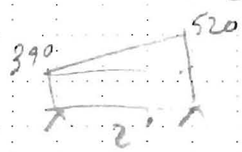
Revise during PED:
9x62.4 = 562 PSF

Bott channel
W=520 L=6 M=2340 R=1560
$S_{req'd} = \frac{2340 \times 12}{22000} = 1.27$ C 4x5.4 I=1.93

Use C4x7.25 for all framing

* 1st beam
W=390x2=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

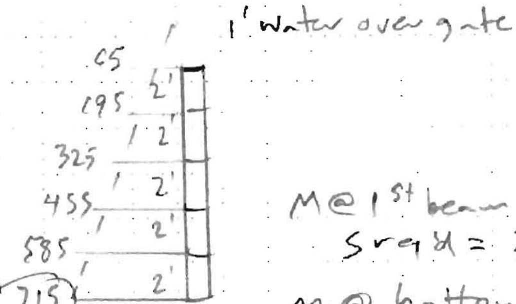
ESTIMATE: Total wt:
5x7.25x6 = 218
2x7.25x8 = 116
$\frac{3/8}{12} \times 490 =$ 15.3x6x8 = 735
Total wt = 1069#

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

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

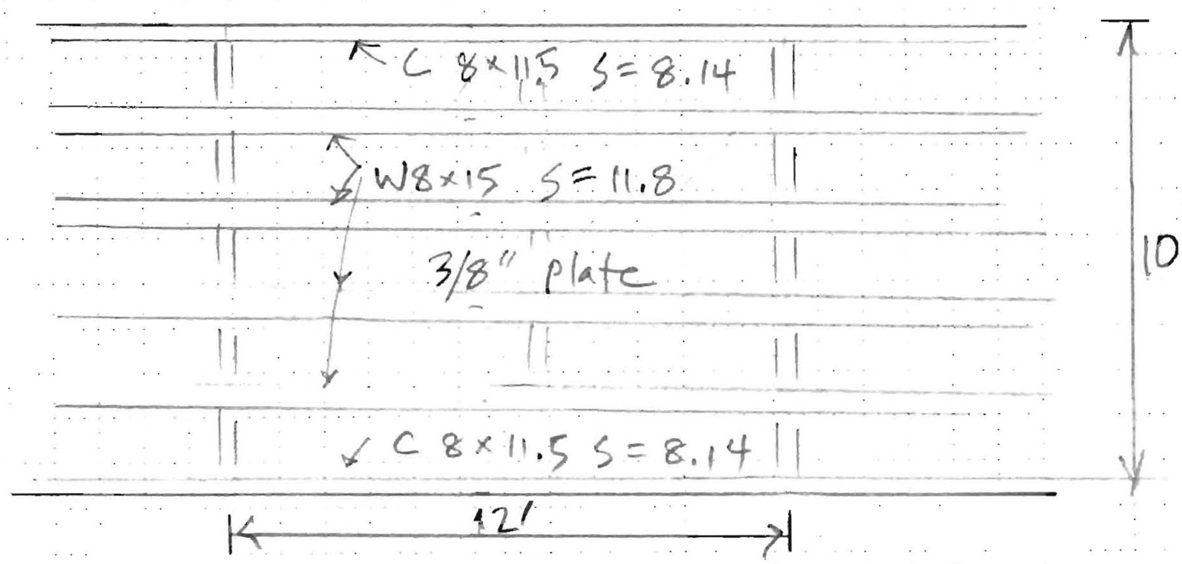


Revise during PED
11 × 62.4
= 686 PSF

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

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

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



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

US Army Corps of Engineers
CHARLESTON DISTRICT

Project Peninsula Study
Subject vehicle gates
Design by gpr Check by
Date 10/13/20 Sheet 3 of 3

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

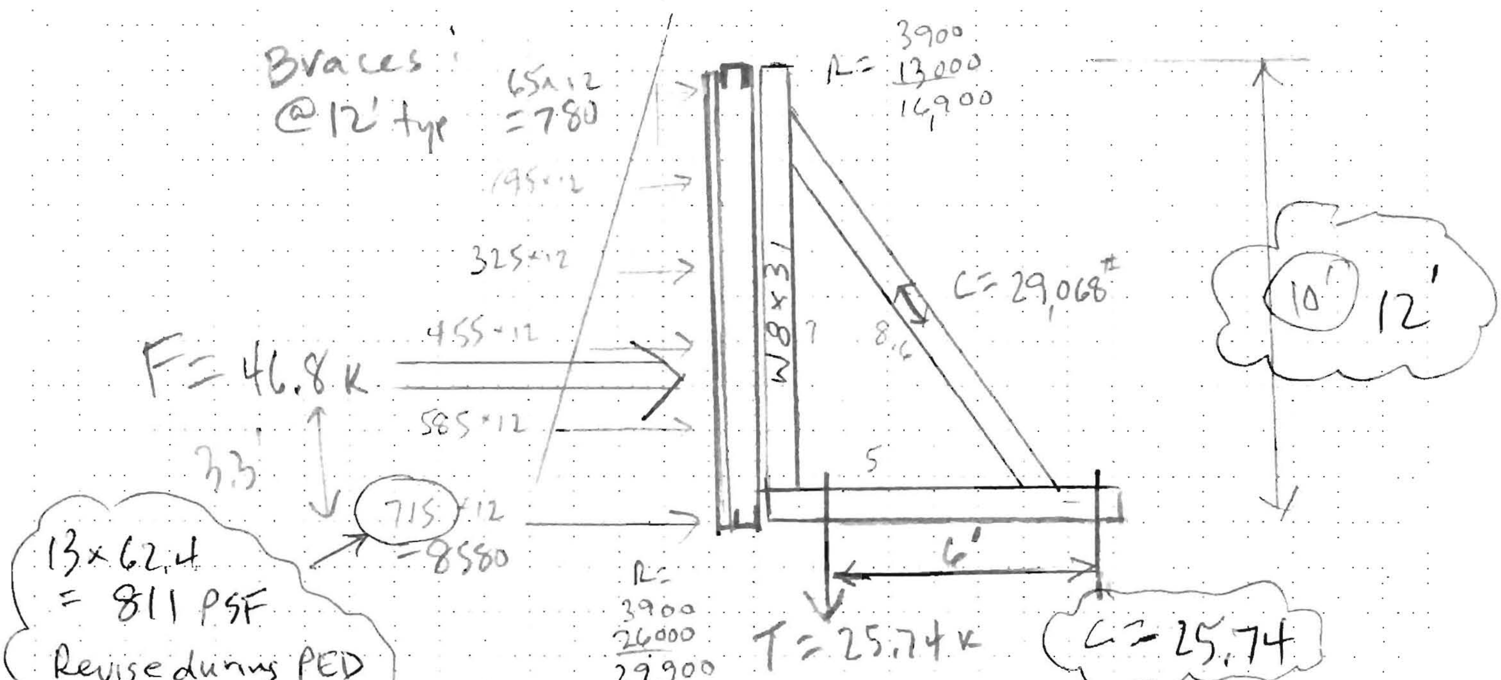
w @ 1st beam = 715

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 = $\frac{780\times10^2}{8}$ + (.1283×39,000×10) = 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.

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

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

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

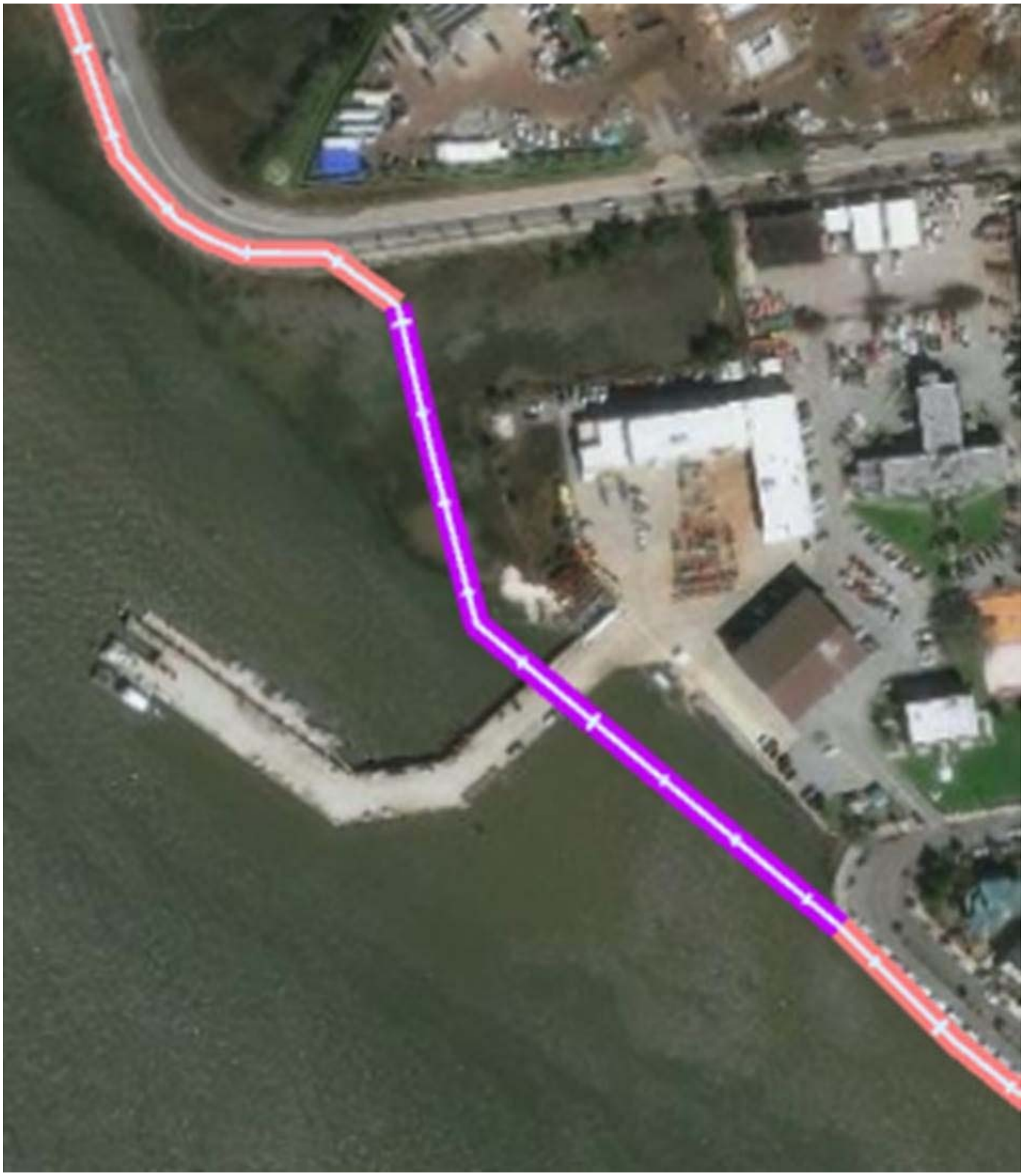
Currrent Condition:

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 Flood Risk Management Study

Charleston, South Carolina

GEOLOGIC AND GEOTECHNICAL SUB-APPENDIX

August 2021

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

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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.

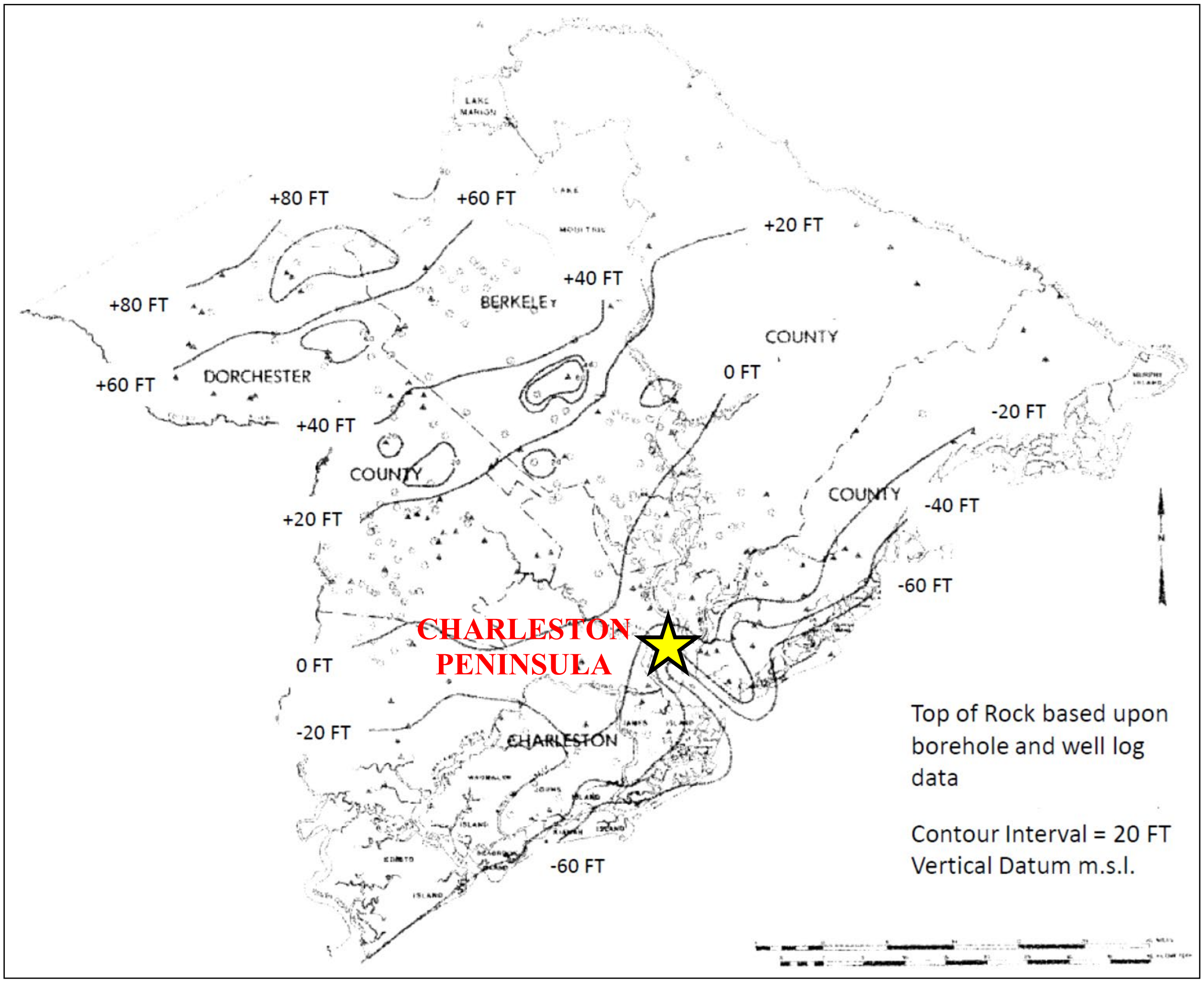

Figure 1: Structure contour map showing top of Cooper Formation, from Park (1985).

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.

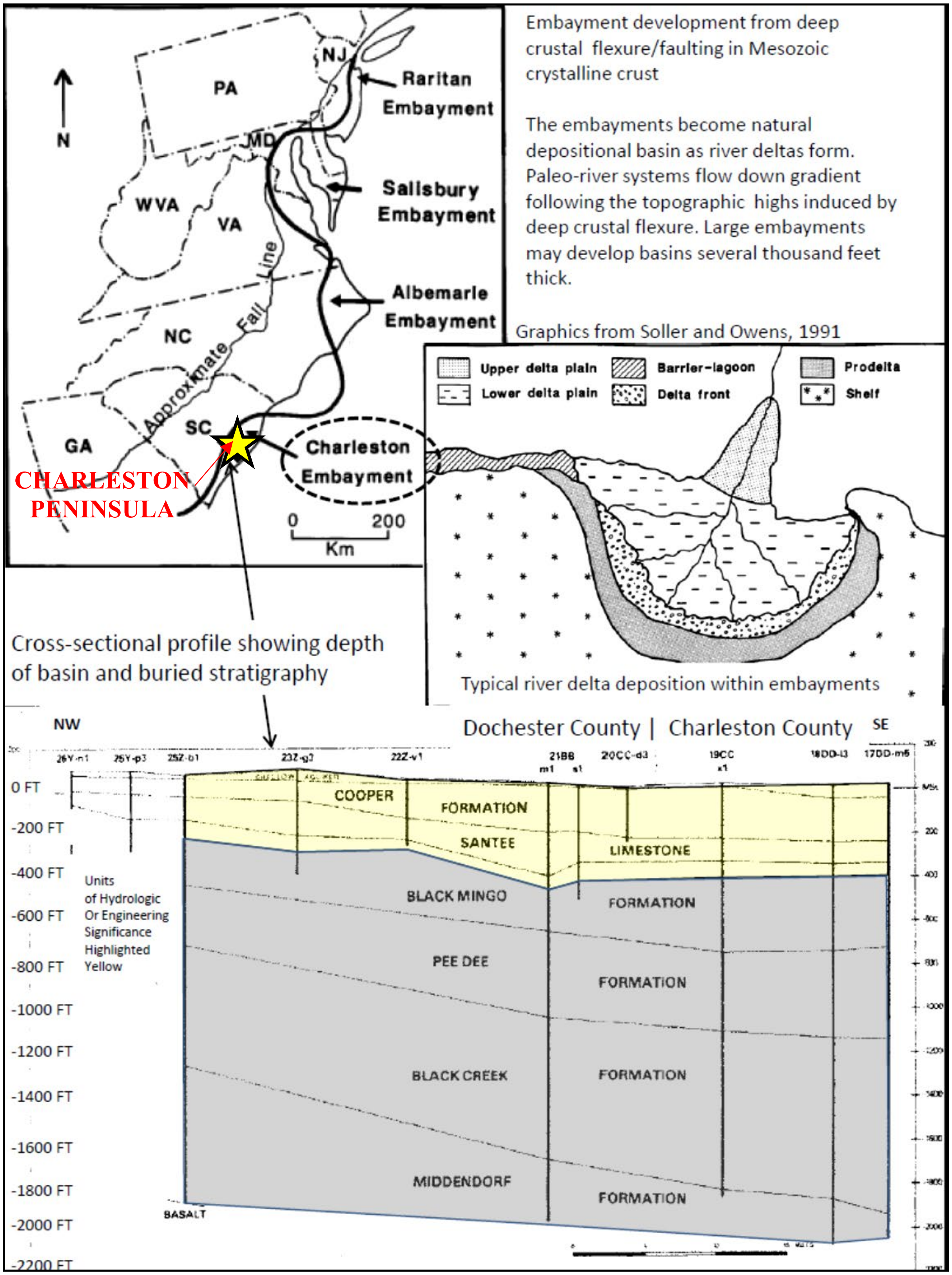

Figure 2: Regional geologic setting of the Charleston Embayment from Soller and Owens (1991) and Park (1985).

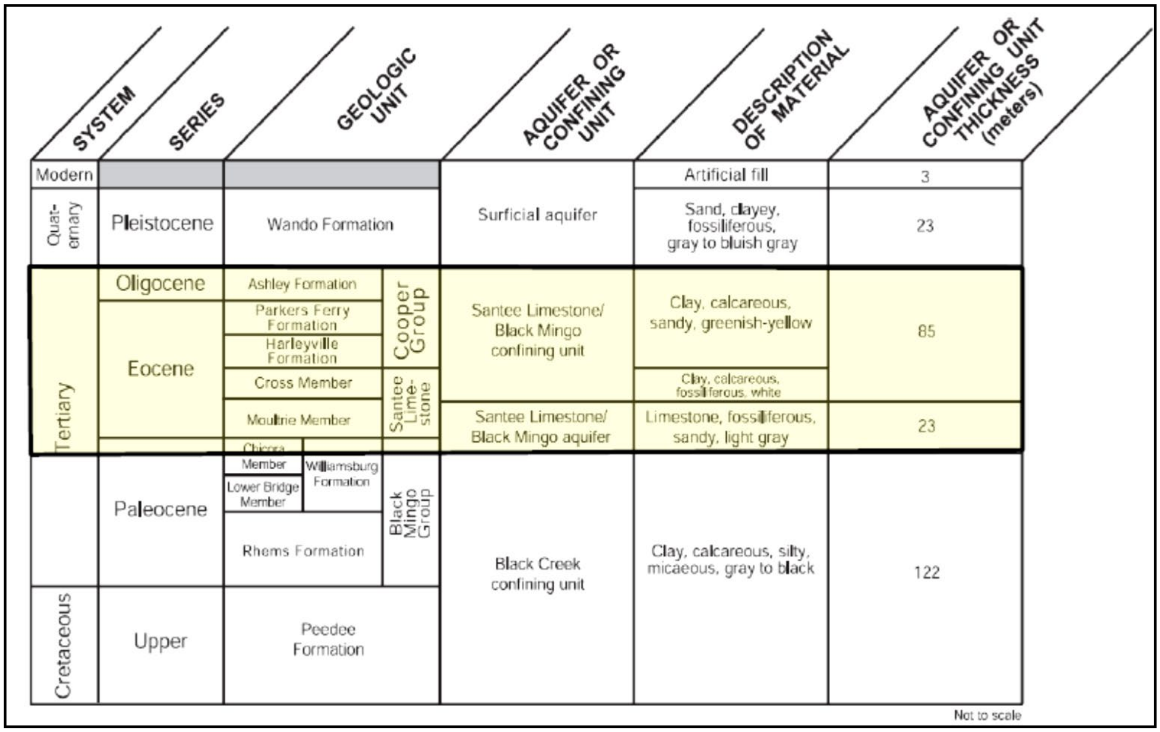

Figure 3: Project relevant stratigraphic & hydrogeologic units, from Petkewich et al. (2004).

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).

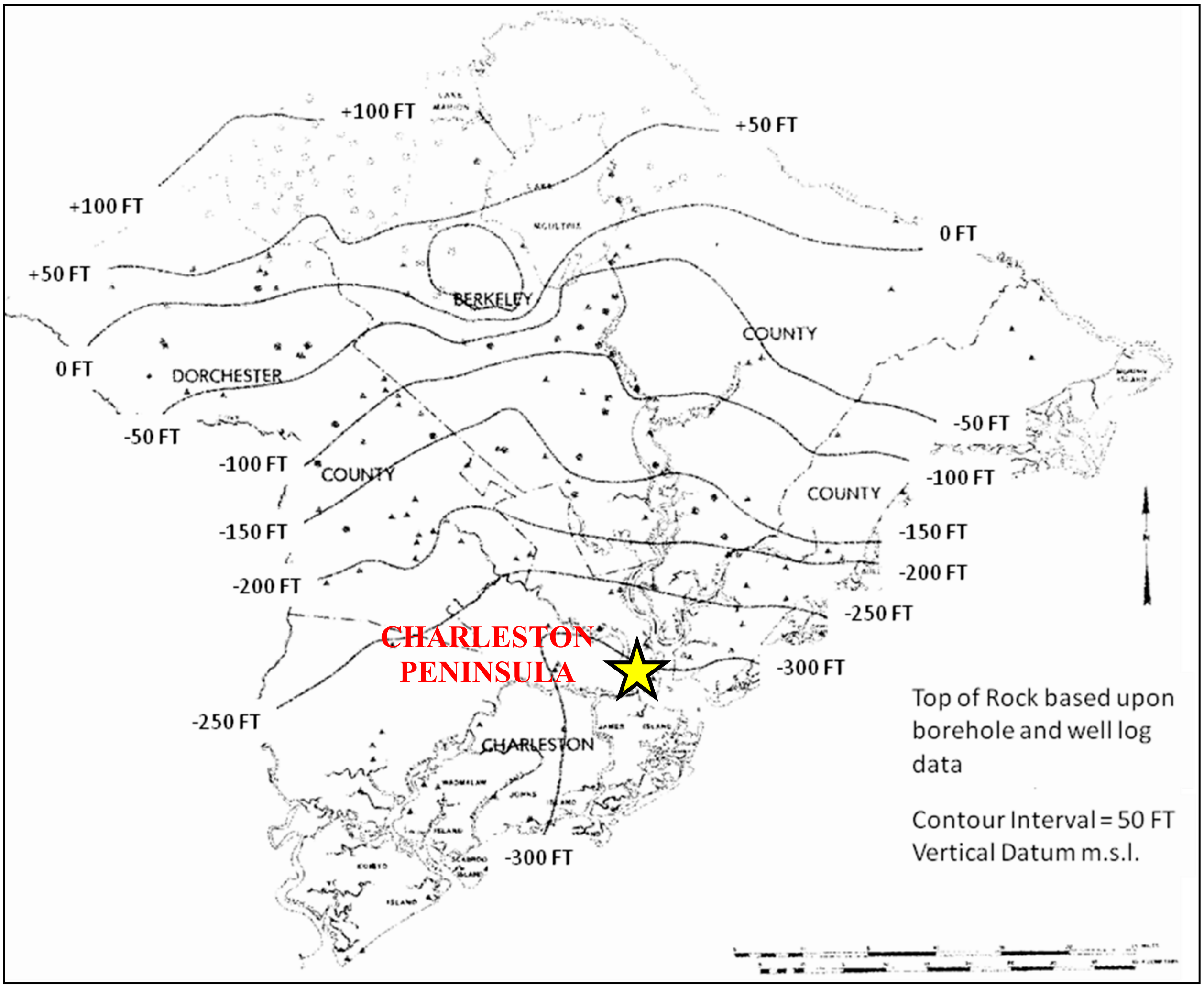

Figure 4: Structural contour map showing top of Santee Limestone, from Park (1985).

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).

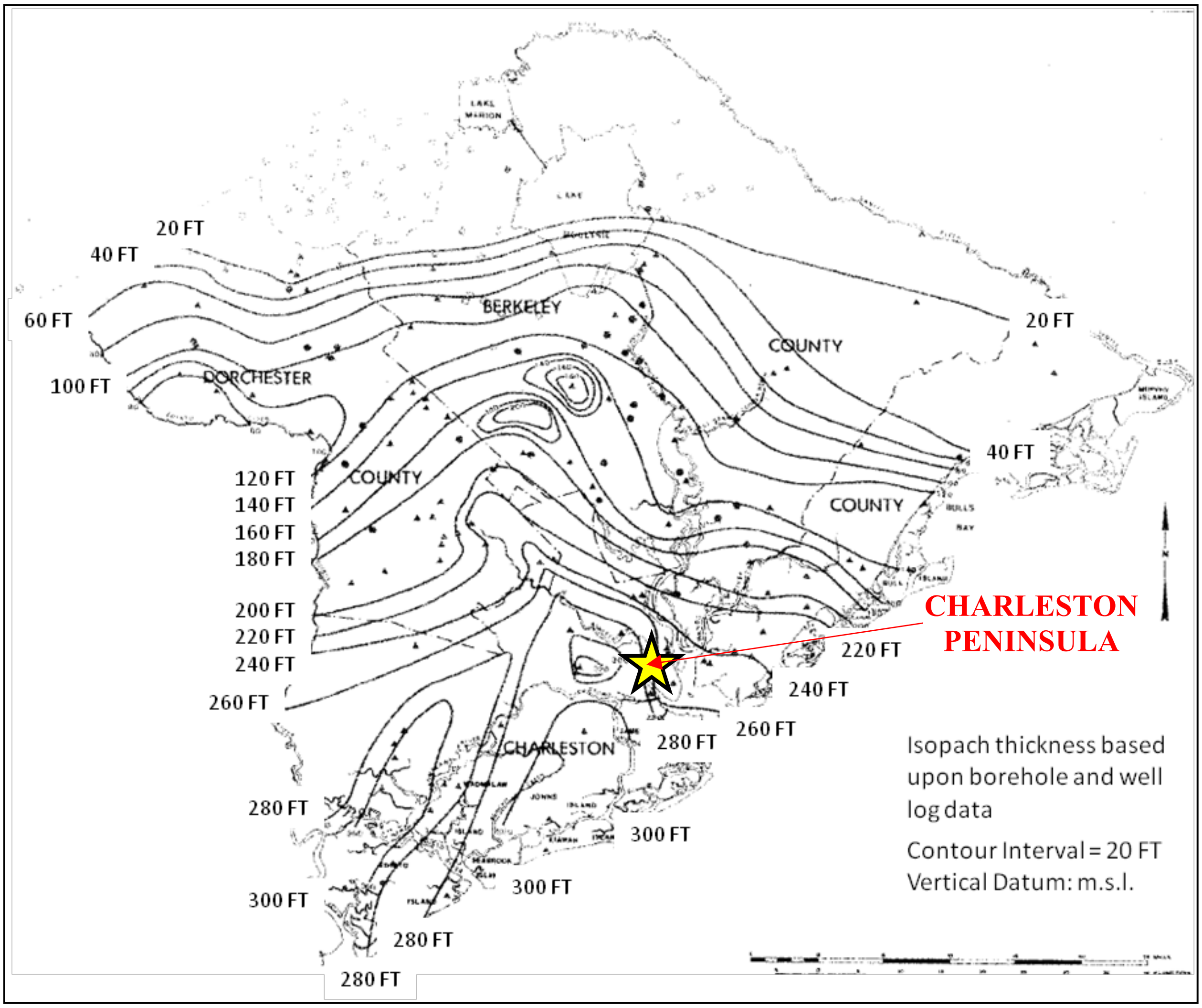

Figure 5: Isopach map showing thickness of the Cooper Formation, from Park (1985).

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).

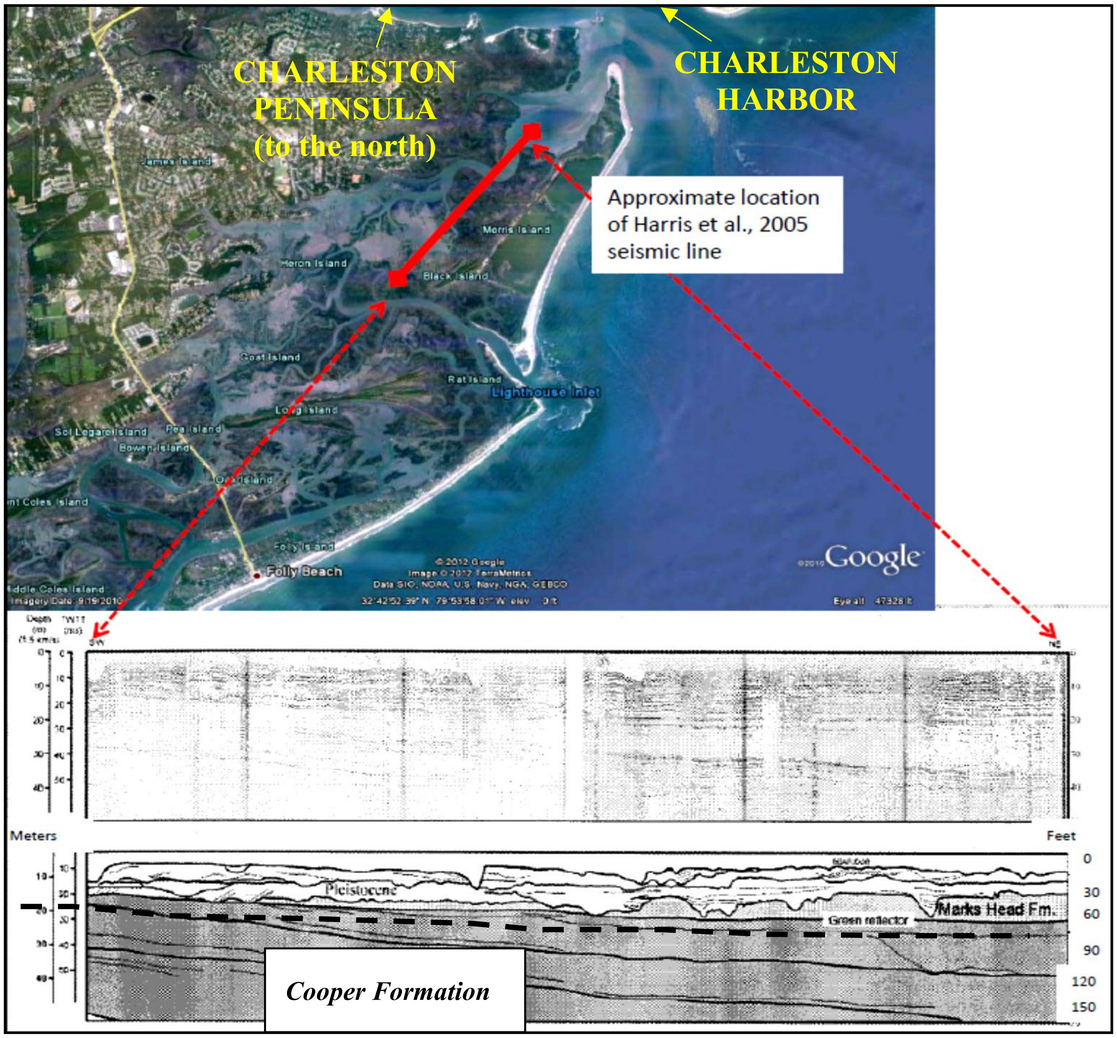

Figure 6: Seismic profile south of Charleston Harbor, from Harris et al. (2005).

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.

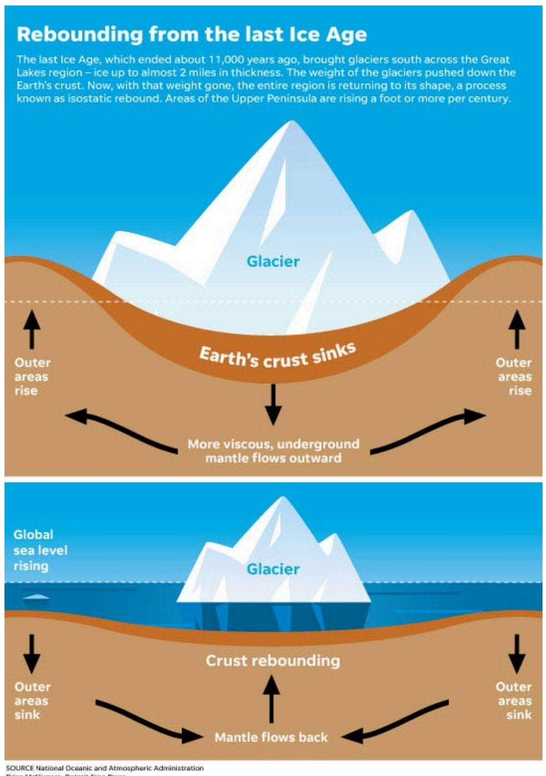

Figure 7: Crustal Deformation Caused be Glaciation (Matheny, Keith Source: National Oceanic and Atmospheric Administration, McNamara)

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. One 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 lower of 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 there 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. Additional, 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 deem to be within a high seismic hazard zone as indicated in Figure 8. This area is known as 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 on 1886.

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

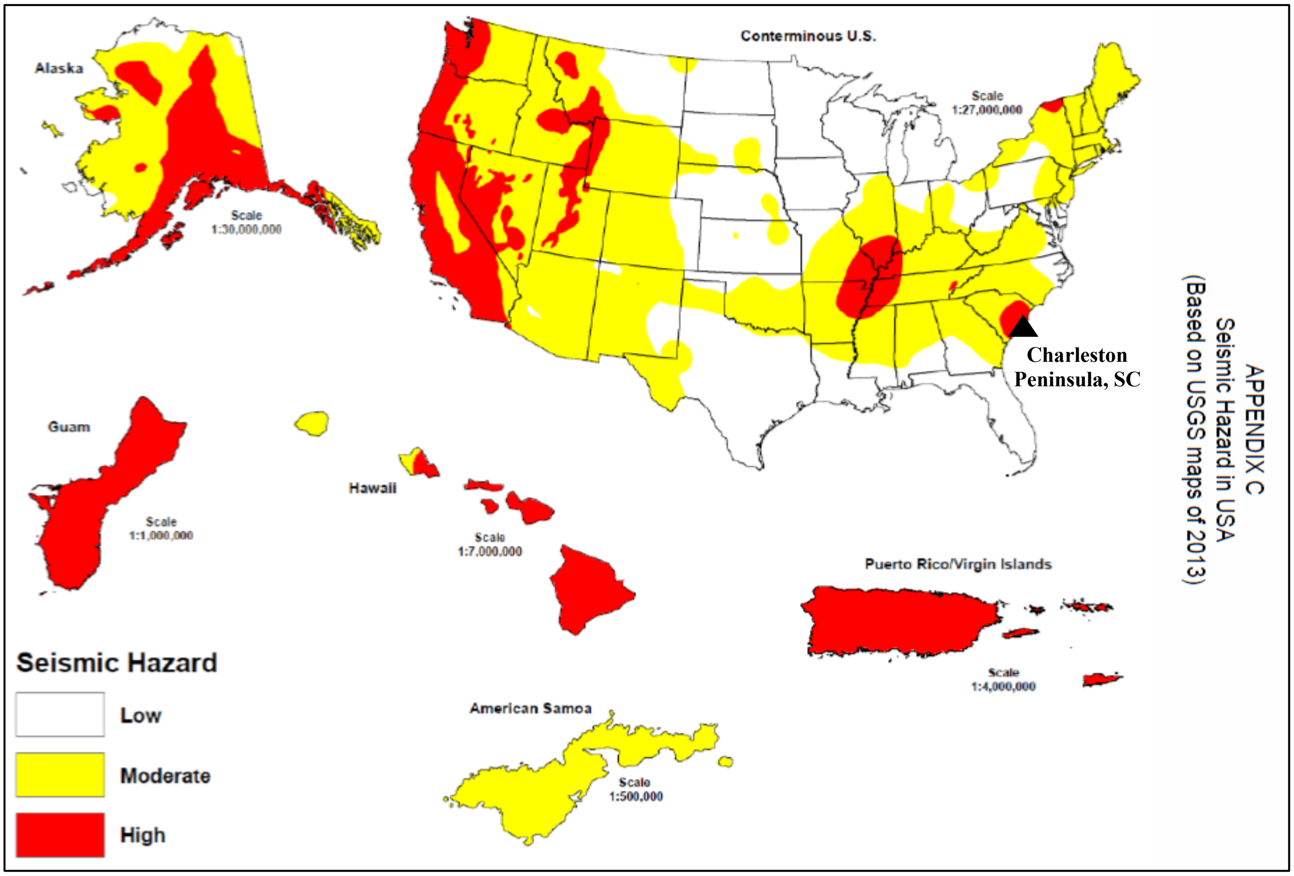

Figure 8: Project location shown on seismic hazard map of the USA, from ER-1110-2-1806.

6.1. Ground Motions

The seismic evaluate provided a range of ground motions for various events. A 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).

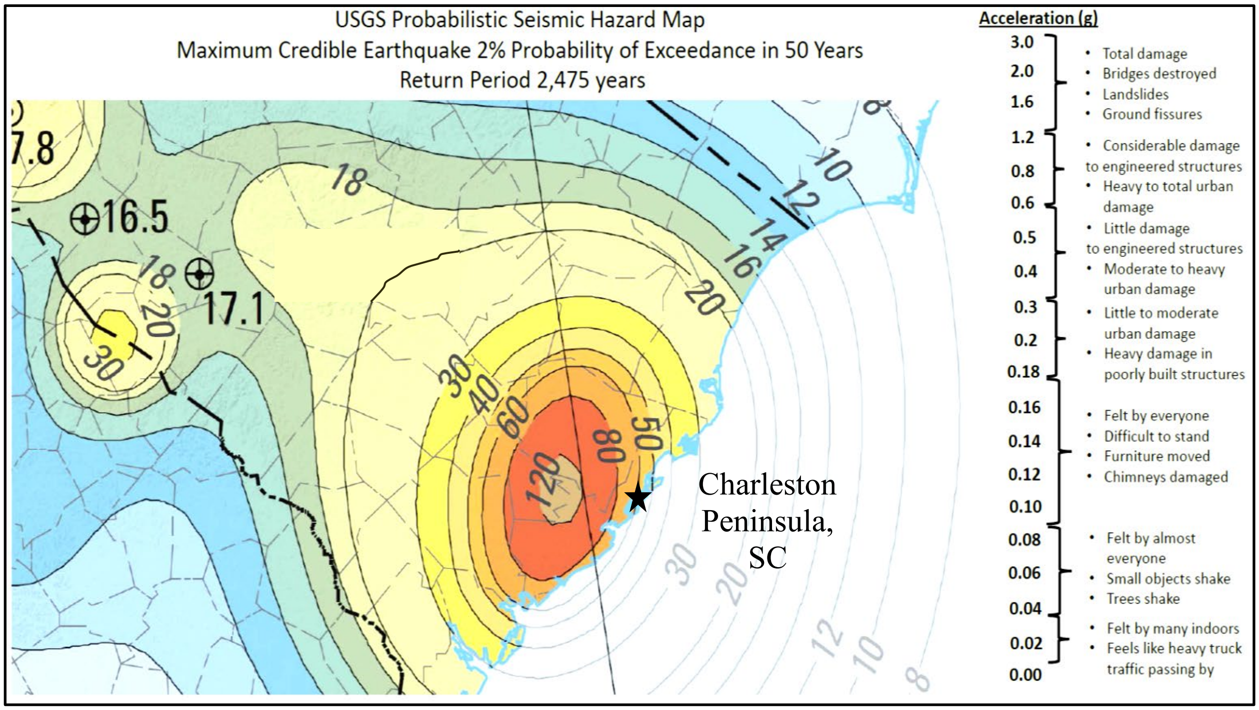

Figure 9: USGS Seismic Hazard Map, PGA, 2% Probability of Exceedance in 50 Years, from Peterson et al. (2015).

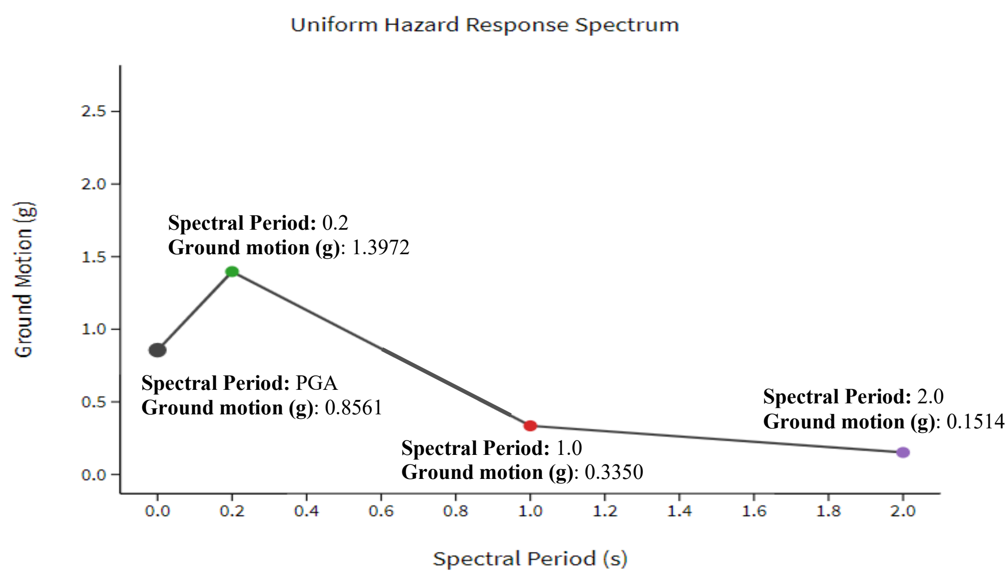

Figure 10: Uniform Hazard Response Spectrum predicted for the project site showing PGA with 2% in 50 years AEP (2,475 return period).

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 elevation of the Cooper Marl, which has been estimated using existing data received from various companies and other published documents. The variation in the foundation materials above the Cooper Marl are not expected to drastically effective the design of the walls and similarly the cost, with conservative assumptions being used. If the alignment was changing for the various alternatives, determining if there was a major change in foundation conditions would be warranted.

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.

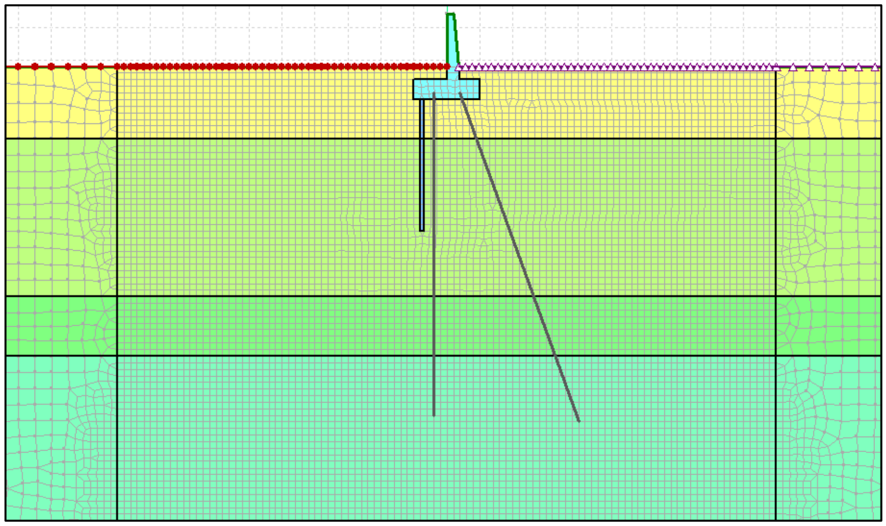

Figure 11: Example of Seep/W Model, Configuration and Mesh

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: 5×10^{-2} cm/s for the upper sand unit, 1×10^{-3} cm/s for the marsh unit, 1×10^{-2} cm/s for the lower sand unit, and 1×10^{-5} cm/s for the Cooper Marl. The sheet pile wall and the concrete seawall were modeled with a permeability of 3×10^{-11} cm/s.

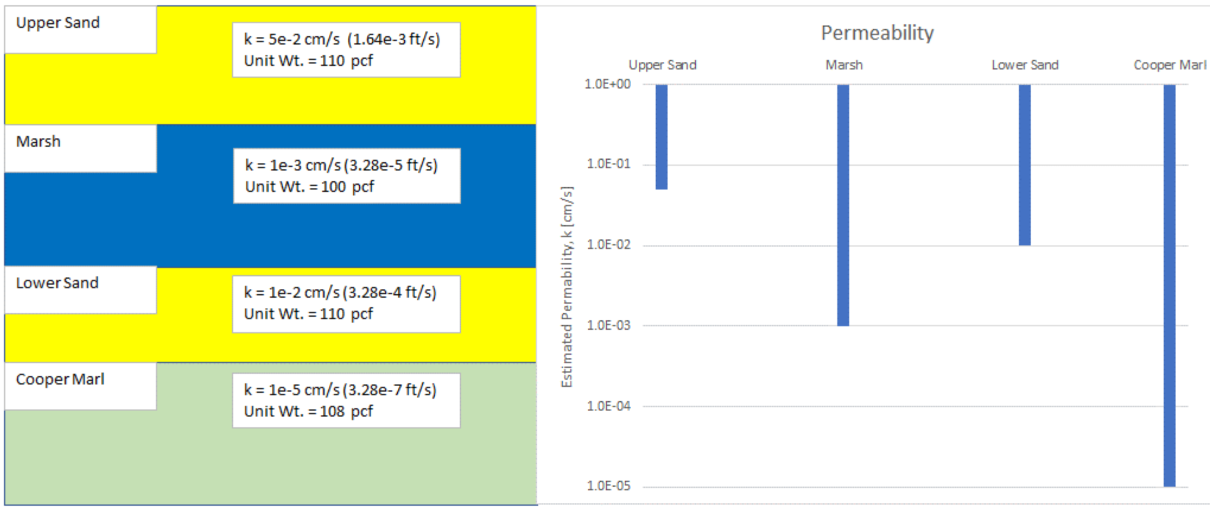

Figure 12. Permeability and Stratigraphy Model used in Analyses.

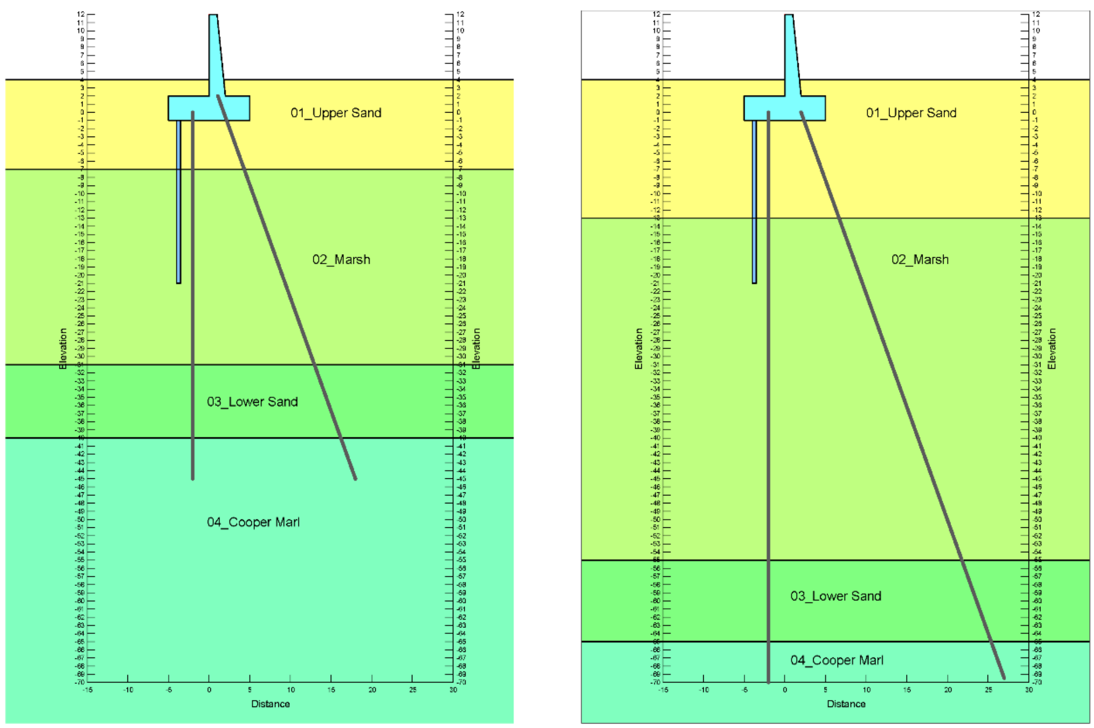

Figure 13. Seepage Model Scenarios. “Average” Thickness on left; “Lower” Copper Marl on Right

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:

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

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:

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

the gradient obtained from the model is:

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

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.

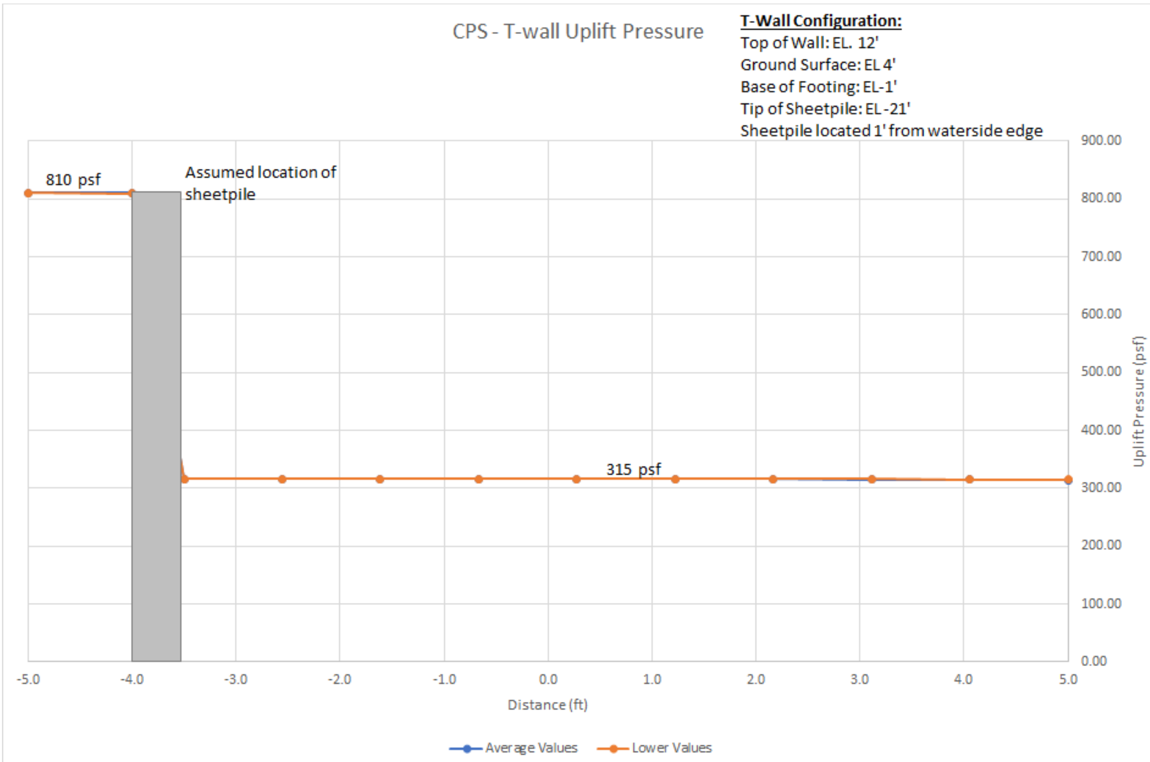

Figure 14: Uplift Pressures on T-Wall

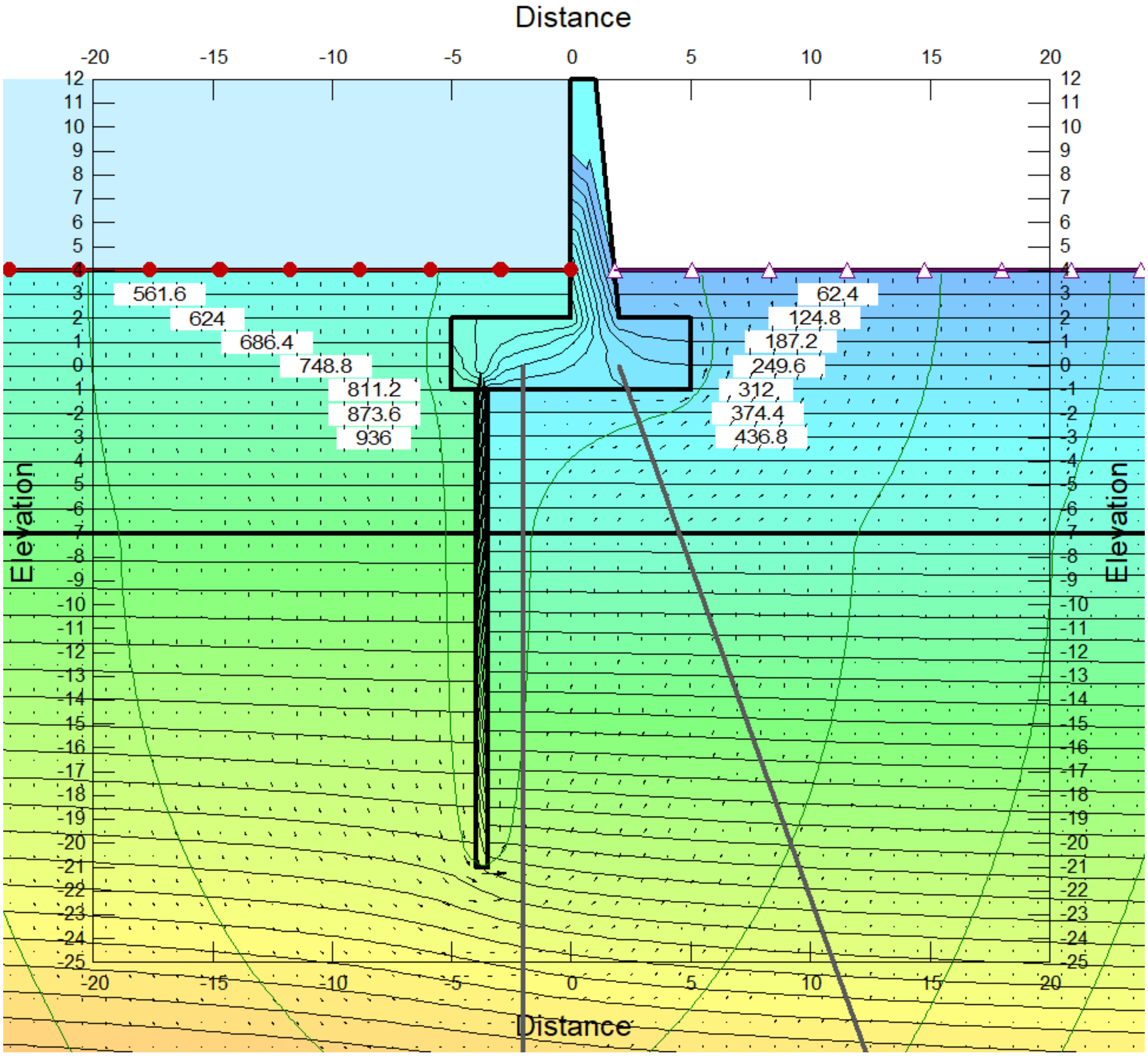

Figure 15. Seepage Analysis Results for "Average Model"

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.

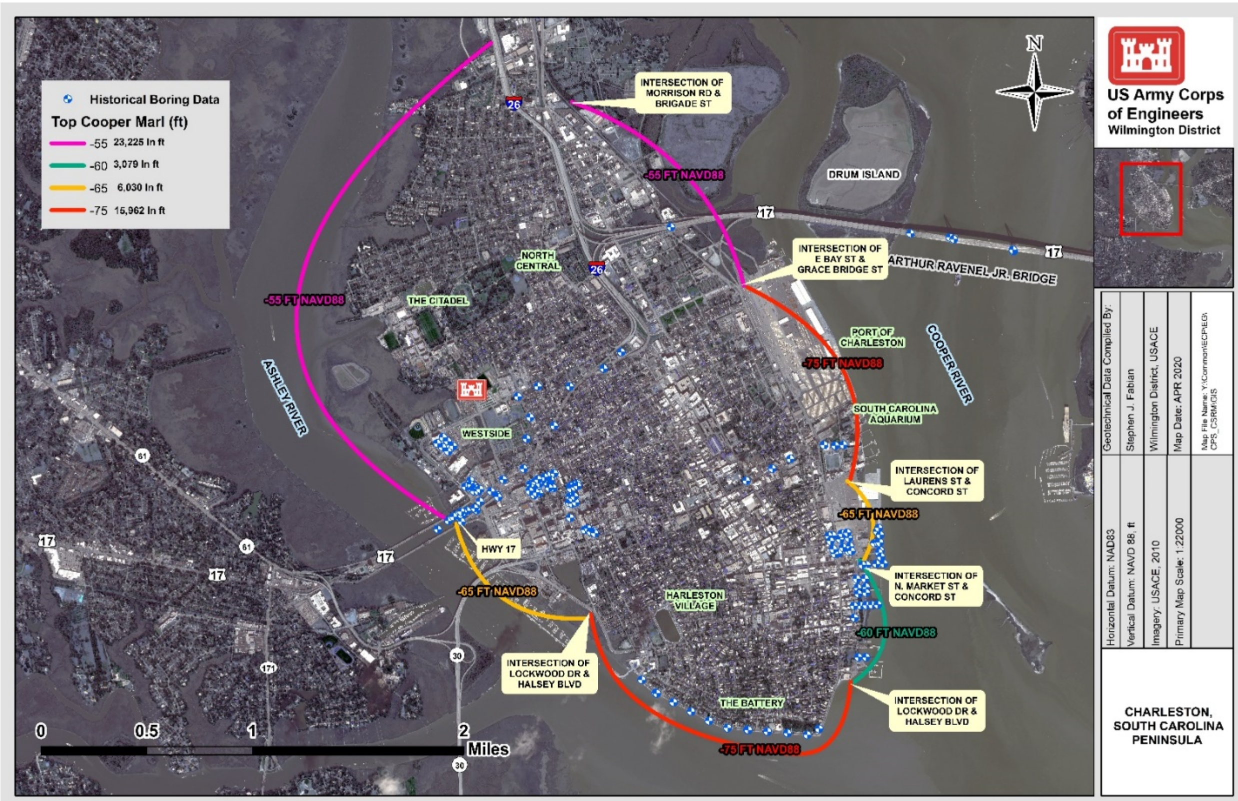

Figure 16: Top of Cooper Marl Within Various Reaches/Segments around the Charleston Peninsula

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.

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. 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.

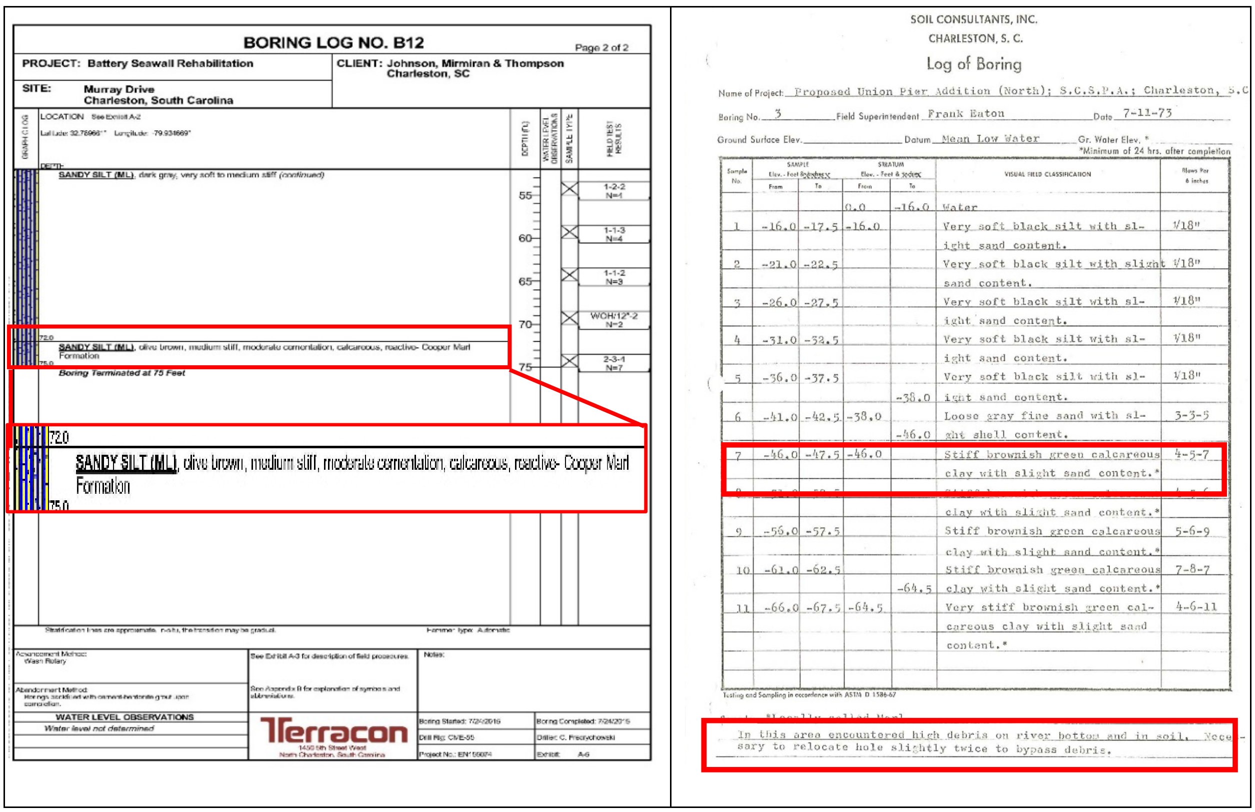

Figure 17. Two SPT logs from the Charleston Peninsula.

The log on the left was taken from 2015 along "The Battery" while the log on the left was taken north of "The Battery" at the Port of Charleston. 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

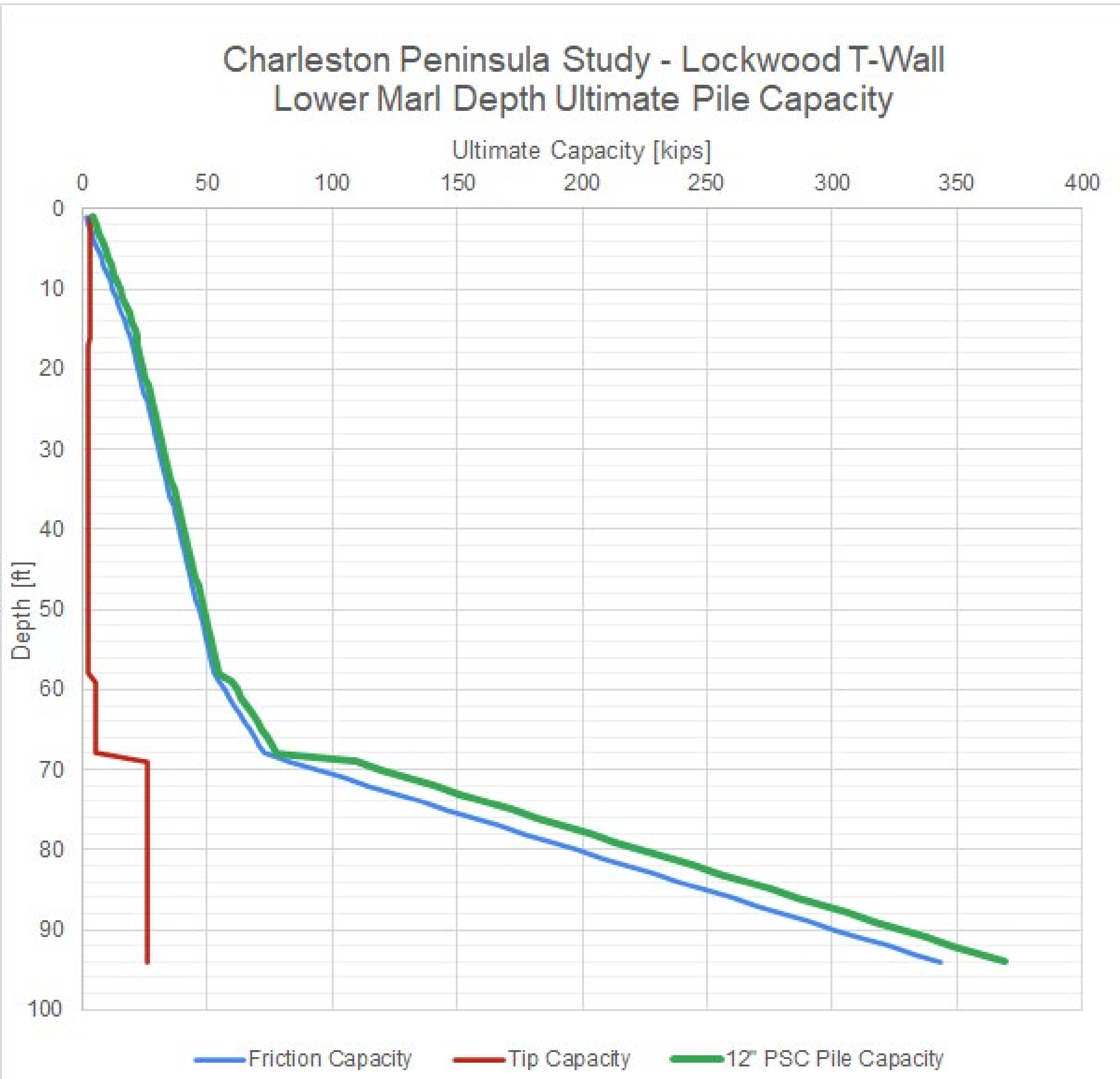

Figure 18. 12” Prestress Concrete Pile Ultimate Capacity Estimate for Cooper Marl at EL. -65 feet.

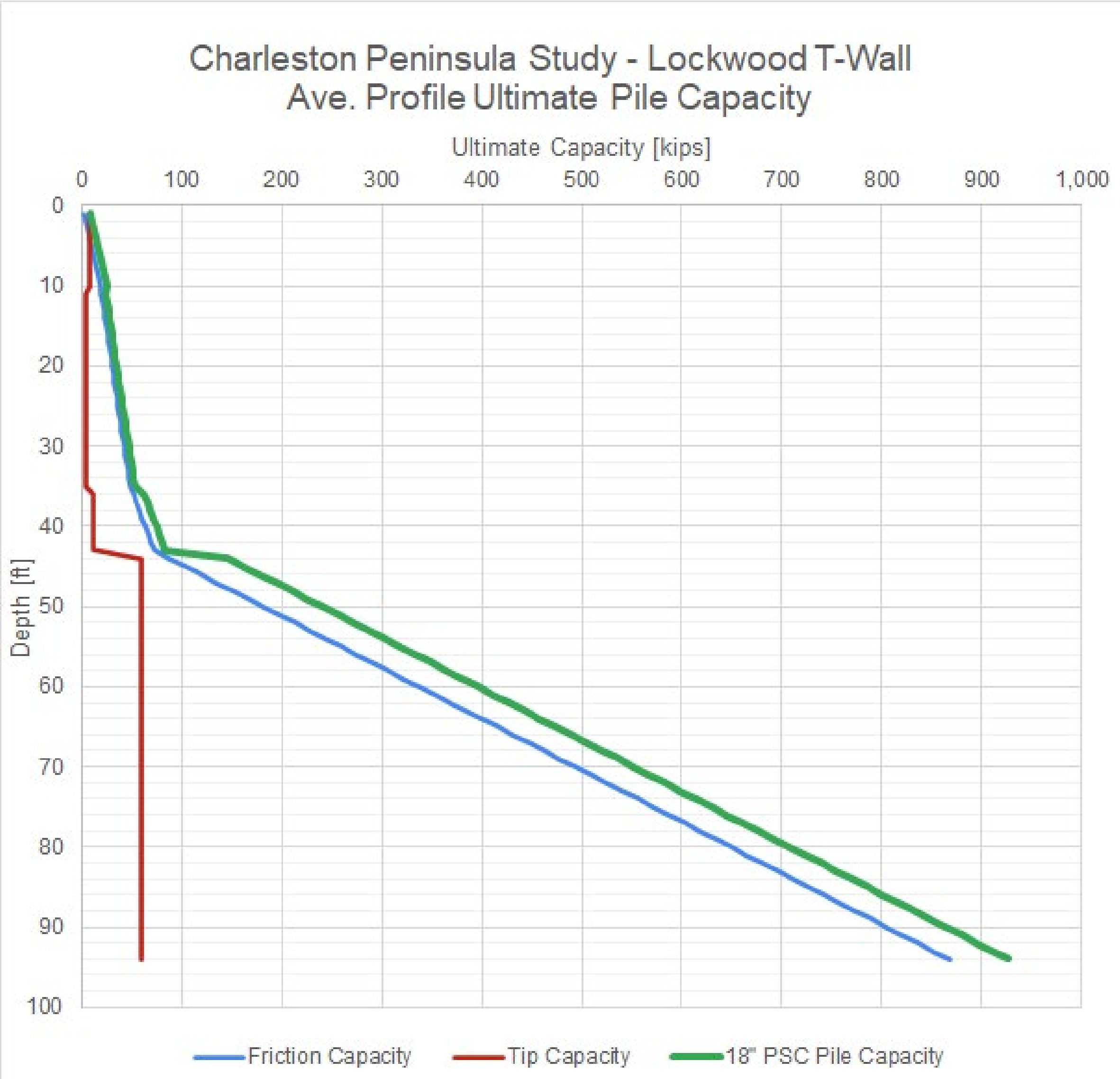

Figure 19. 18” Prestressed Concrete Pile Capacity Estimate for Cooper Marl at EL. -65 feet.

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.

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

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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

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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

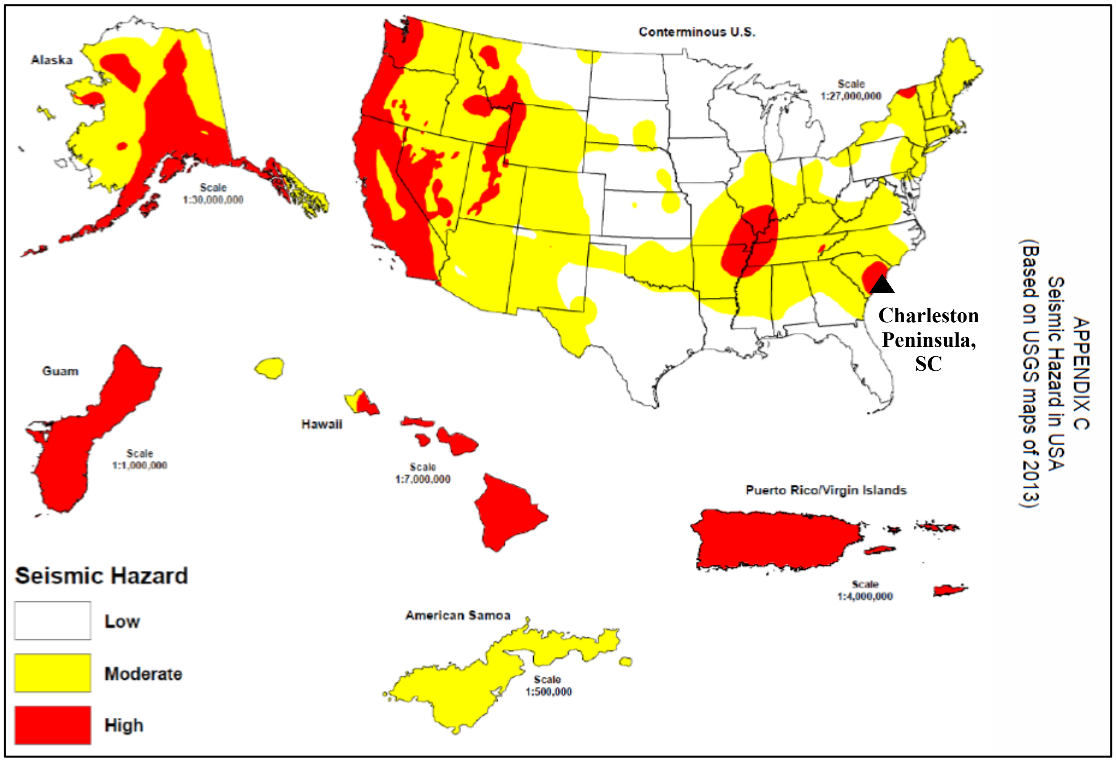

Figure 1: Project location shown on seismic hazard map of the USA, from ER-1110-2-1806.

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.

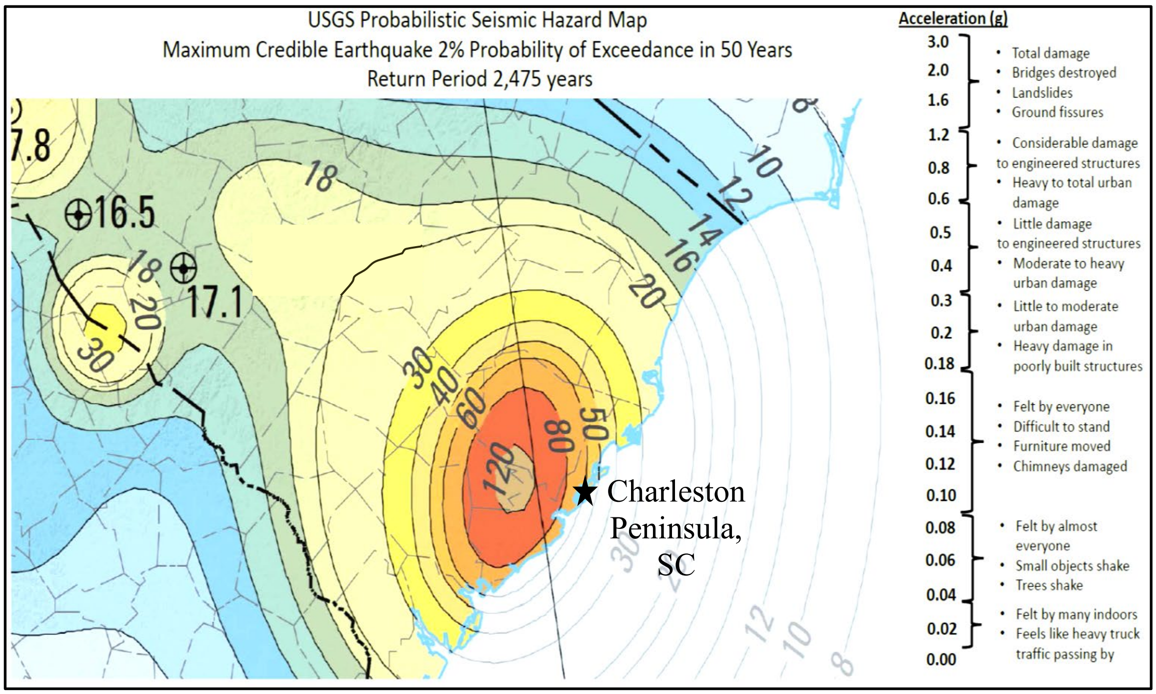

Figure 2: USGS Seismic Hazard Map, PGA, 2% Probability of Exceedance in 50 Years, from Peterson et al. (2015).

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 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.

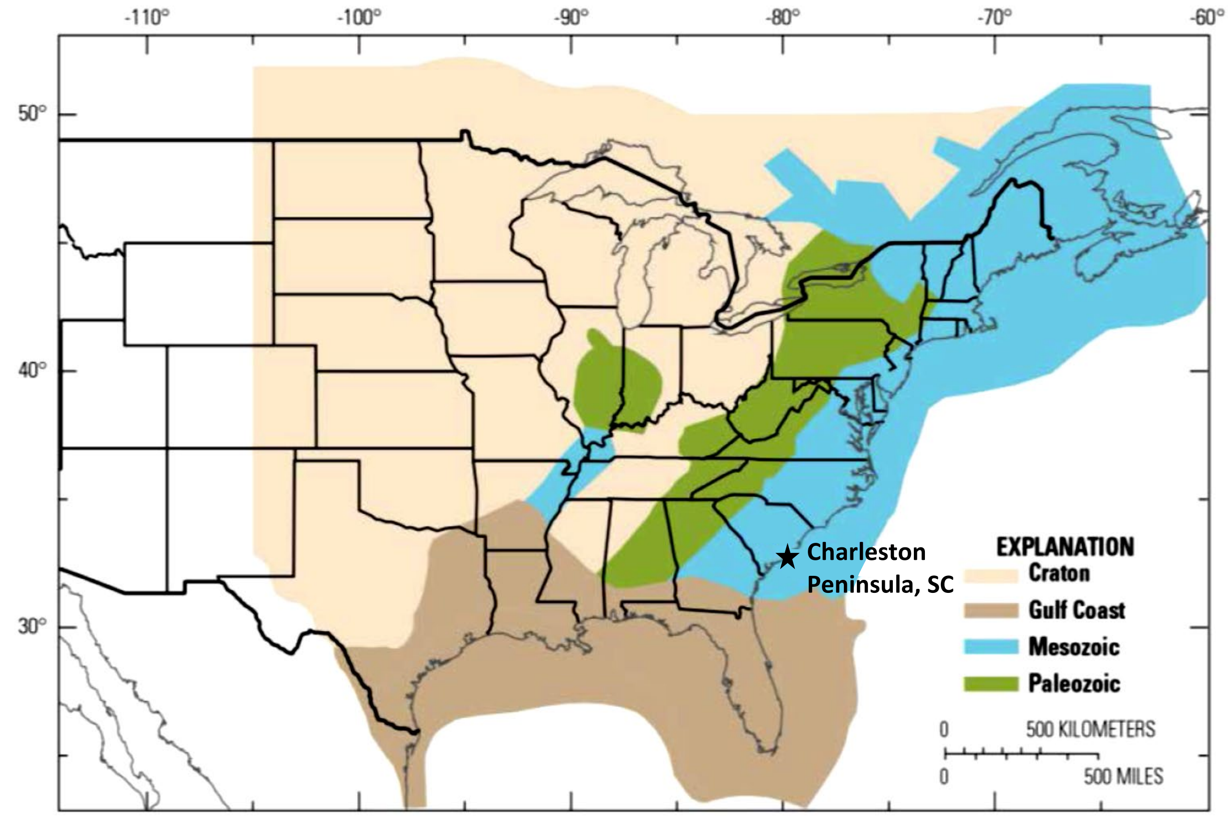

Figure 3: General geologic and seismic velocity structure of the CEUS seismotectonic zone.

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 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).

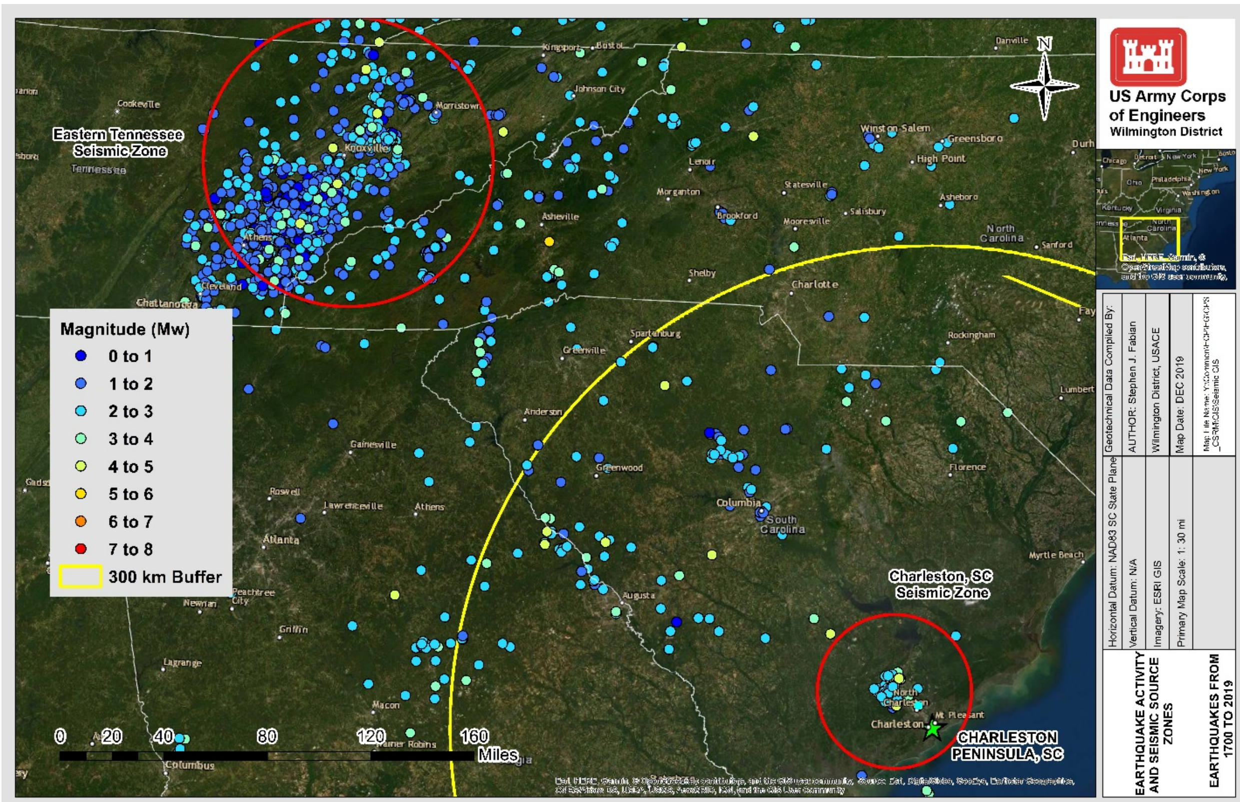

Figure 4: Map showing measured earthquake activity and seismic source zones within 300 km of the project site.

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 Charleston Seismic Zone or by local diffuse background sources.

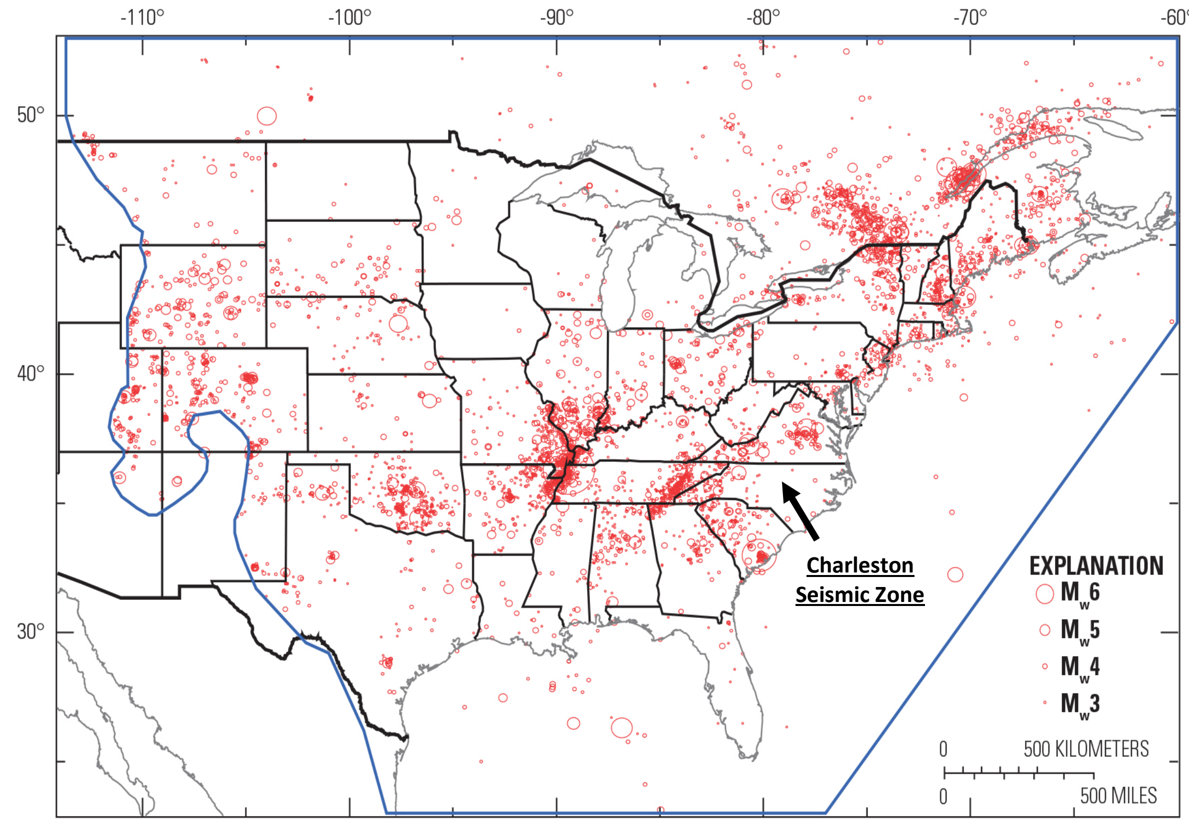

Figure 5: CEUS Earthquake data 1700 to 2012 used for USGS-sponsored seismic hazard mapping, from Peterson et al. (2014).

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 within 50km/31 miles of a project site, then far-field attenuation curves shall be used to evaluate

[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 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).

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.

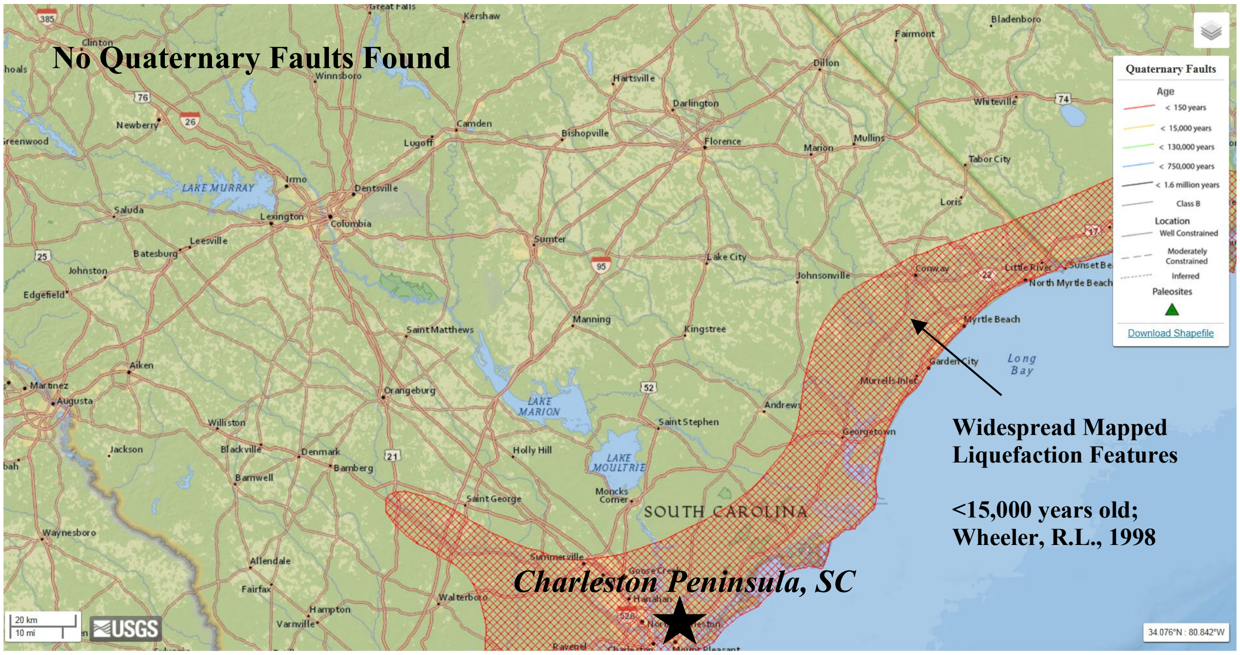

Figure 6: Location of paleoliquefaction features, from USGS Quaternary Fault and Fold Database of the U.S.

1.4.6. Charleston Seismic Zone Defined

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 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 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)

[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.

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 Charleston Seismic Zone.

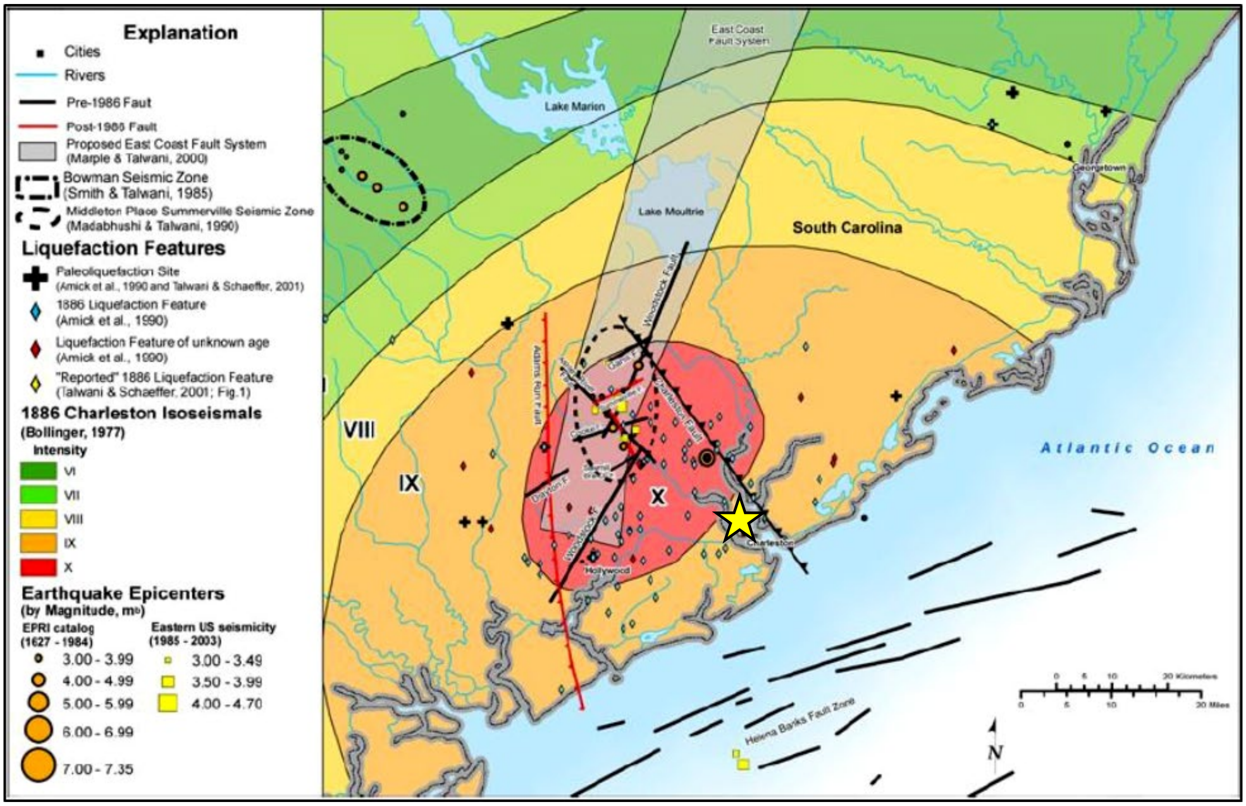

Figure 7: Map from ECB 1110-2-6000 showing seismicity, tectonic and paleoliquefaction, in the epicentral region of the 1886 Charleston earthquake, from Southern Nuclear Company (2007).

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.

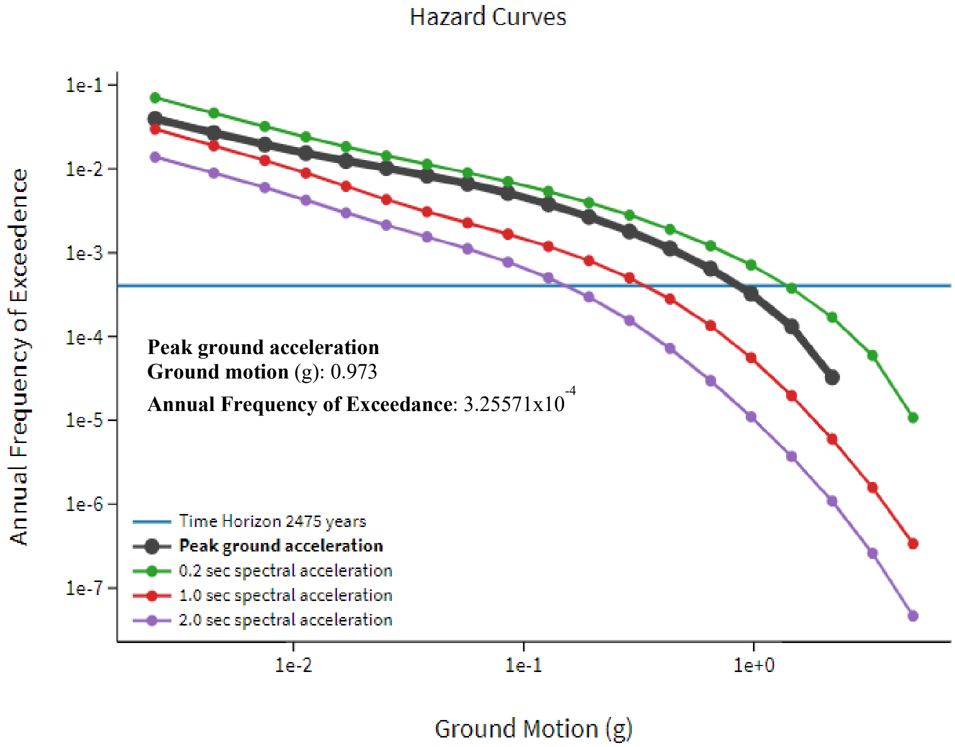

Figure 8: Probabilistic Seismic Hazard Curve showing PGA for 2% in 50 years AEP (2,475 year return period), using USGS 2014 Conterminous U.S. data.

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).

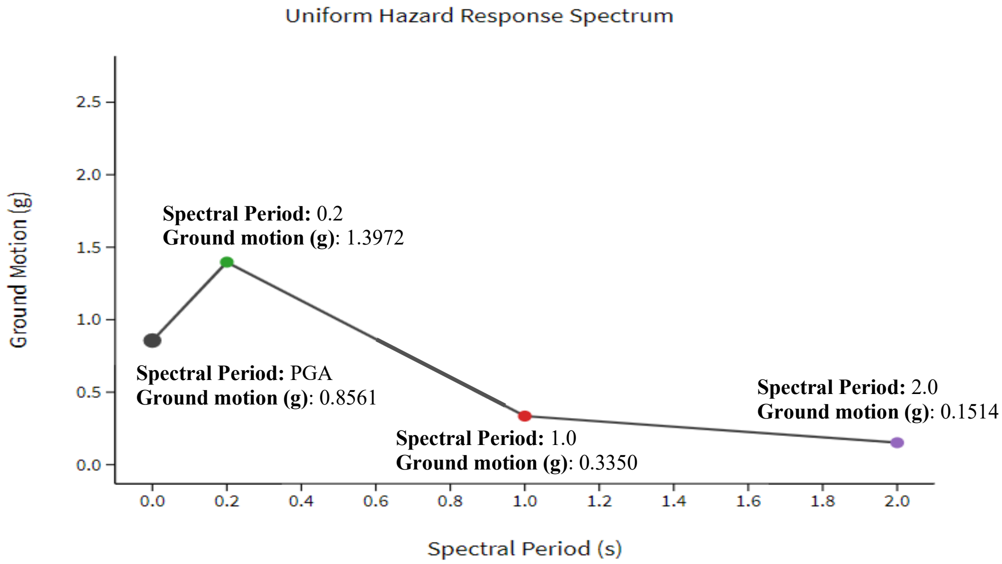

Figure 9. Uniform Hazard Response Spectrum predicted for the project site showing PGA with 2% in 50 years AEP (2,475 return period).

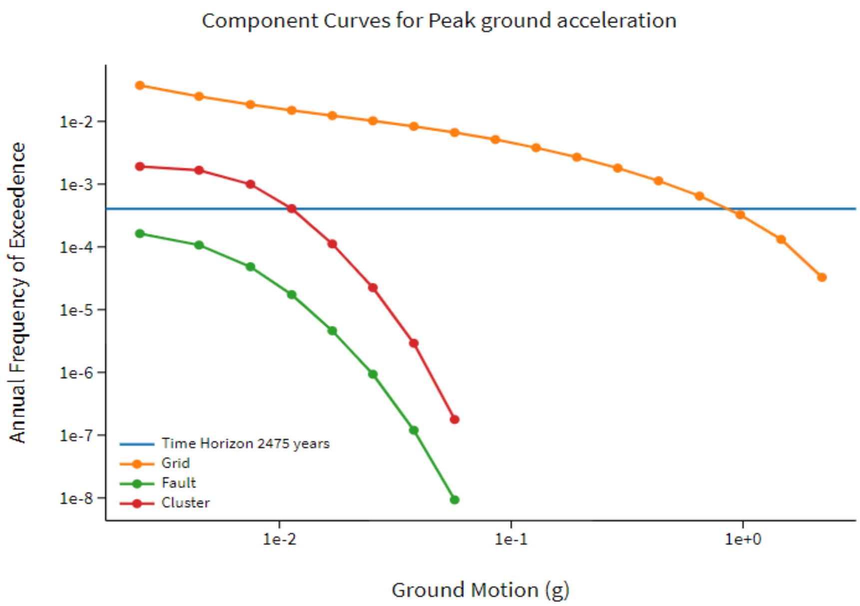

Figure 10: Component curves showing PGA differences based on different seismic sources as calculated in probabilistic hazard analysis.

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.

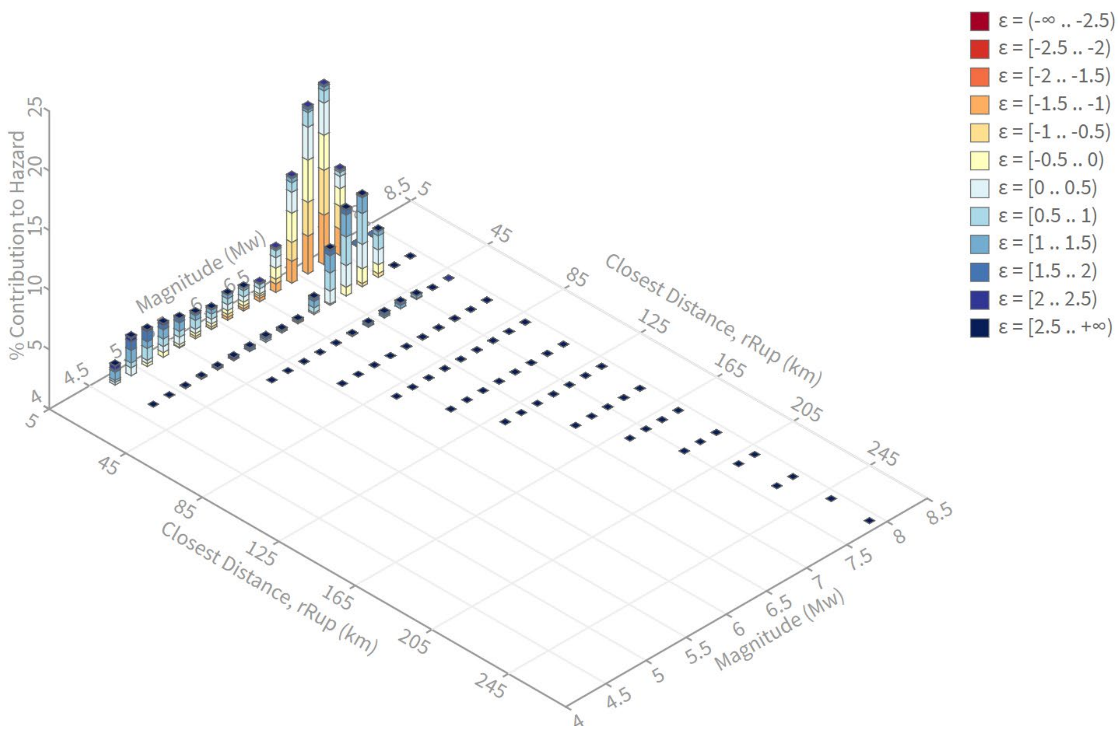

Figure 11: Deaggregation chart (USGS, 2014) showing itemized seismic hazard contribution to the Charleston Peninsula.

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 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

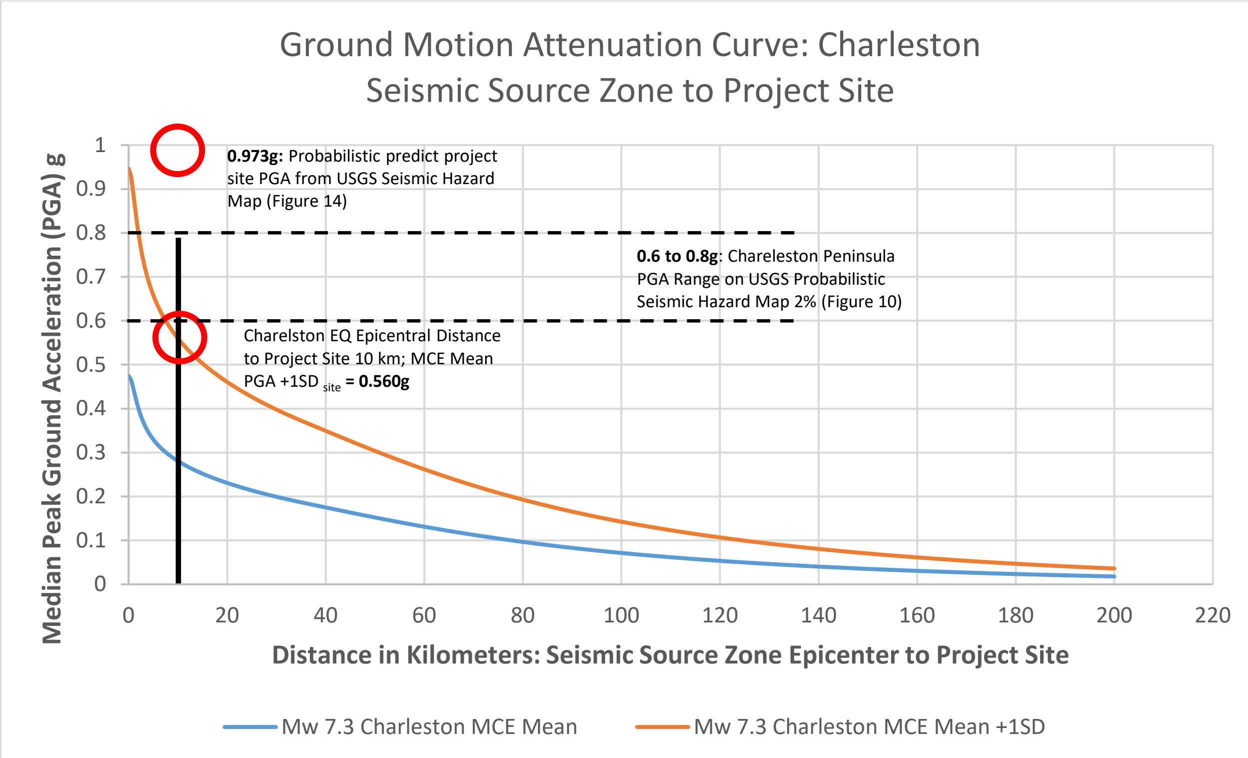

Figure 12: Attenuation curves for selected seismic source zones with respect to project site. Note: GMPE of Boore and Atkinson (2006) used to generate median PGA and median PGA $+1\sigma$. Curves generated using the attenuation relationship of Boore Atkinson (2006), with site Vs30 = 255 m/s. Mw Charleston 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 Charleston Seismic Zone and Background Earthquake.

Period (Seconds)	**Charleston Seismic Zone MCE Mw = 7.3**	**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.

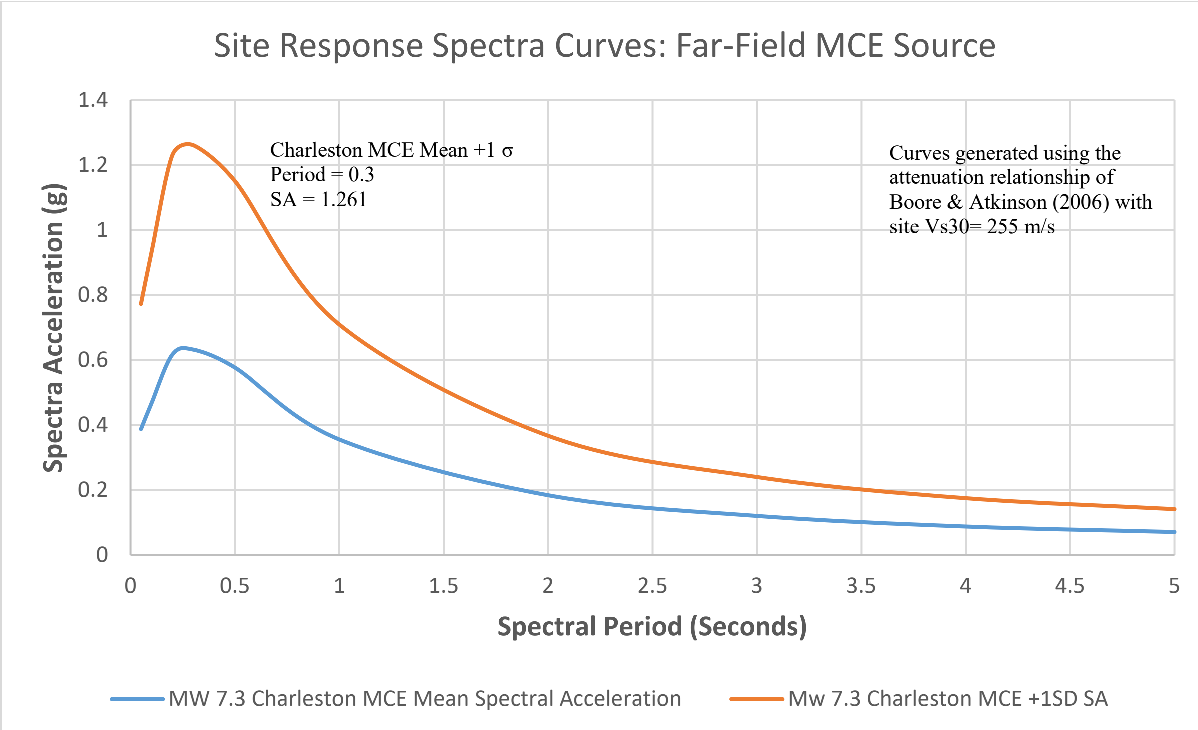

Figure 13: Deterministic acceleration response spectra curves for described ground motions.

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 Charleston Seismic Zone (Toro et al., 1997; USGS, 2003; Boore and Atkinson, 2006).

Period (Seconds)	**Charleston Seismic Zone MCE Mw = 7.3**	**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

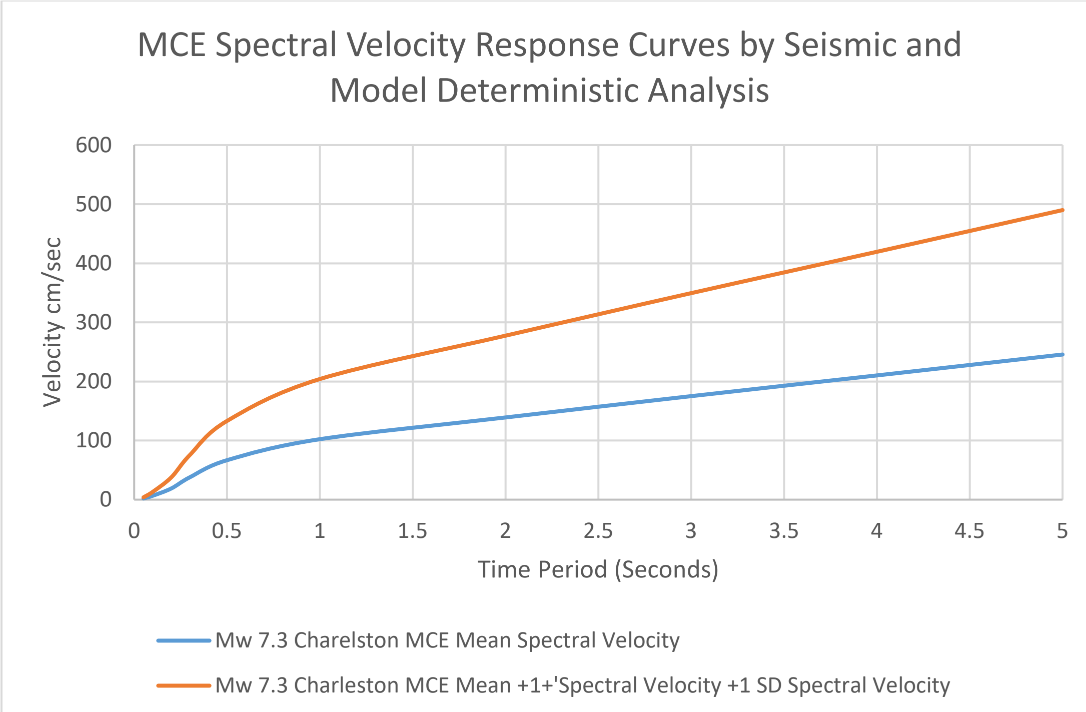

Figure 14: Deterministic velocity response spectra curves for described ground motions.

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.

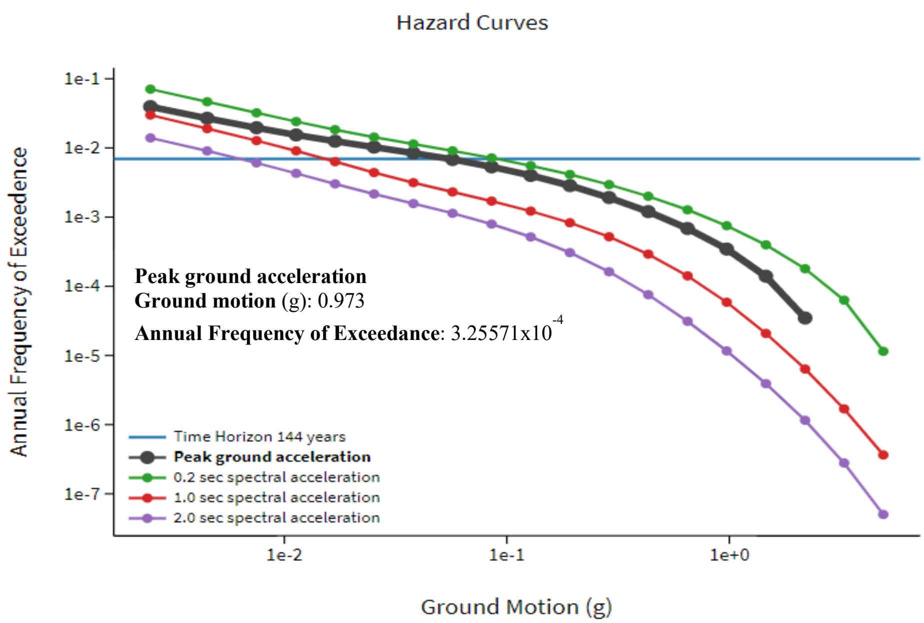

Figure 15: Site-specified seismic hazard curves showing ground motions for PGA and SA with 144-year return period.

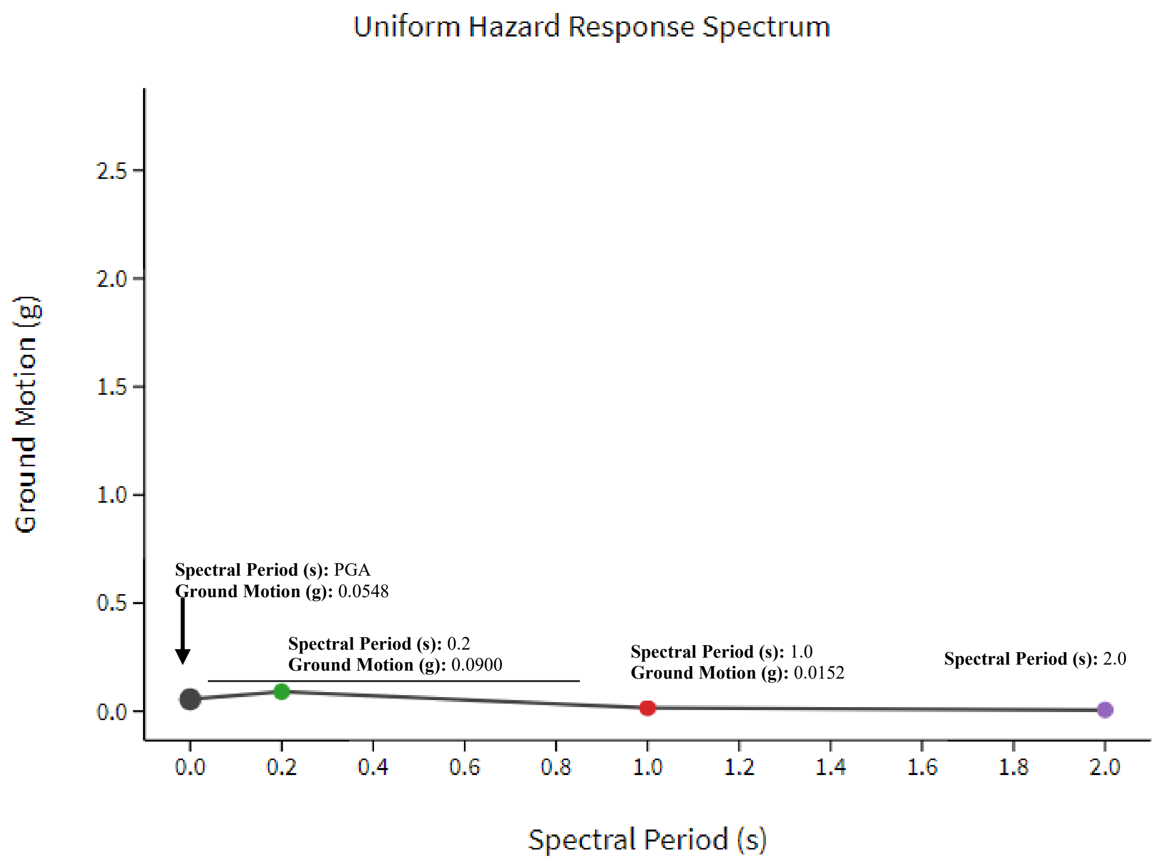

Figure 16: Site-specified uniform hazard response spectrum for the 144-year return period. PGA and SA Periods 0.2, 1.0, and 2.0 seconds shown.

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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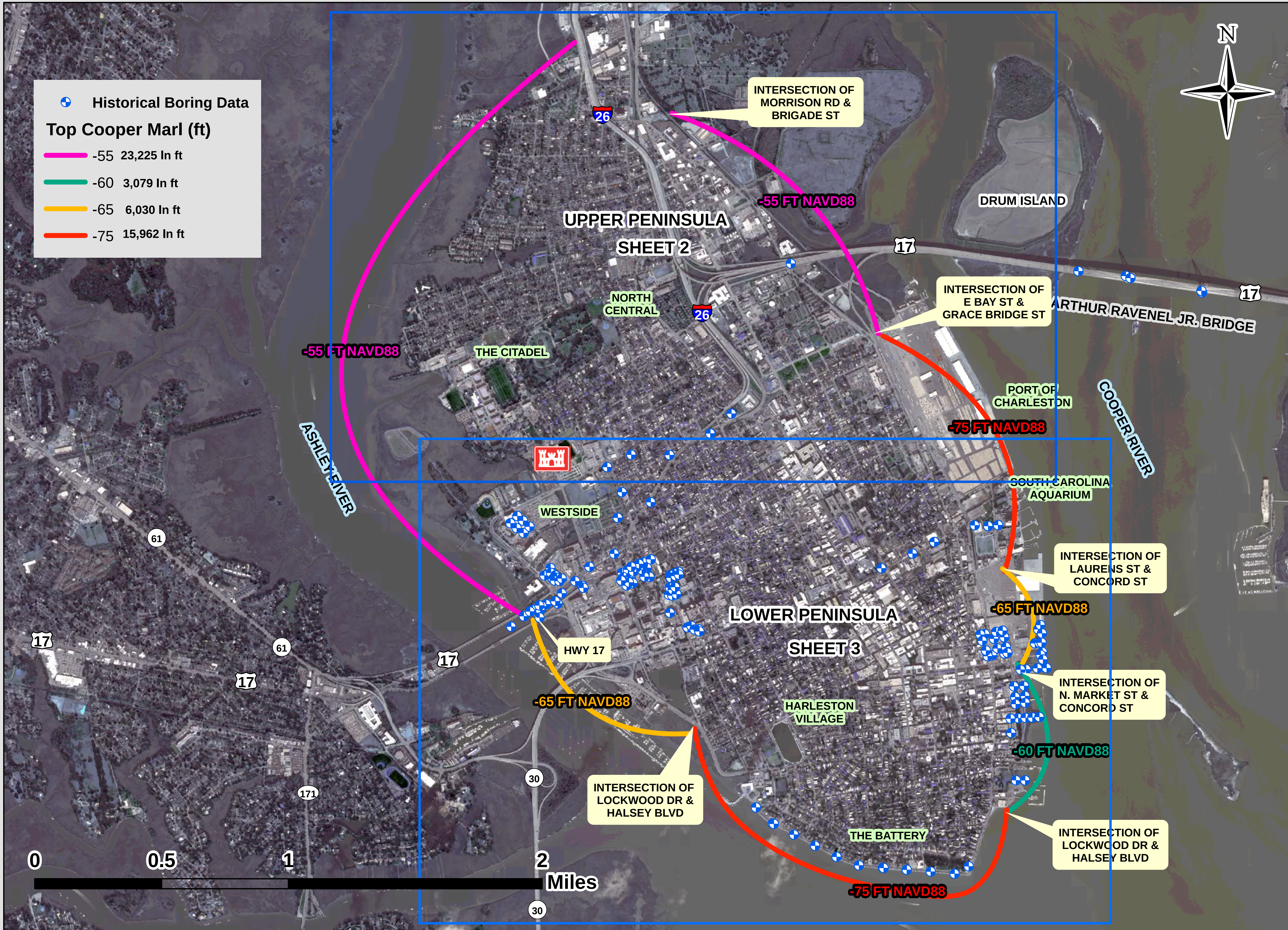

US Army Corps of Engineers
Wilmington District

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

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
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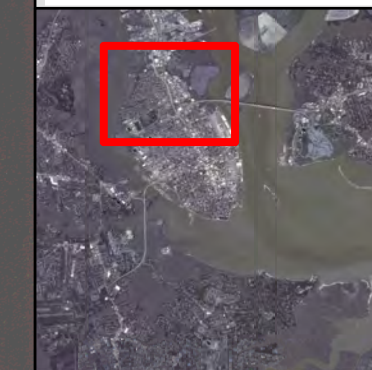
CHARLESTON, SOUTH CAROLINA
UPPER PENINSULA

SHEET 2 OF 3

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_CSRMGIS

Horizontal Datum: NAD83
Vertical Datum: NAVD 88, ft
Imagery: USACE, 2010
Primary Map Scale: 1:12500

CHARLESTON, SOUTH CAROLINA UPPER PENINSULA

SHEET 3 OF 3

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Attachment 3: T-wall Analyses

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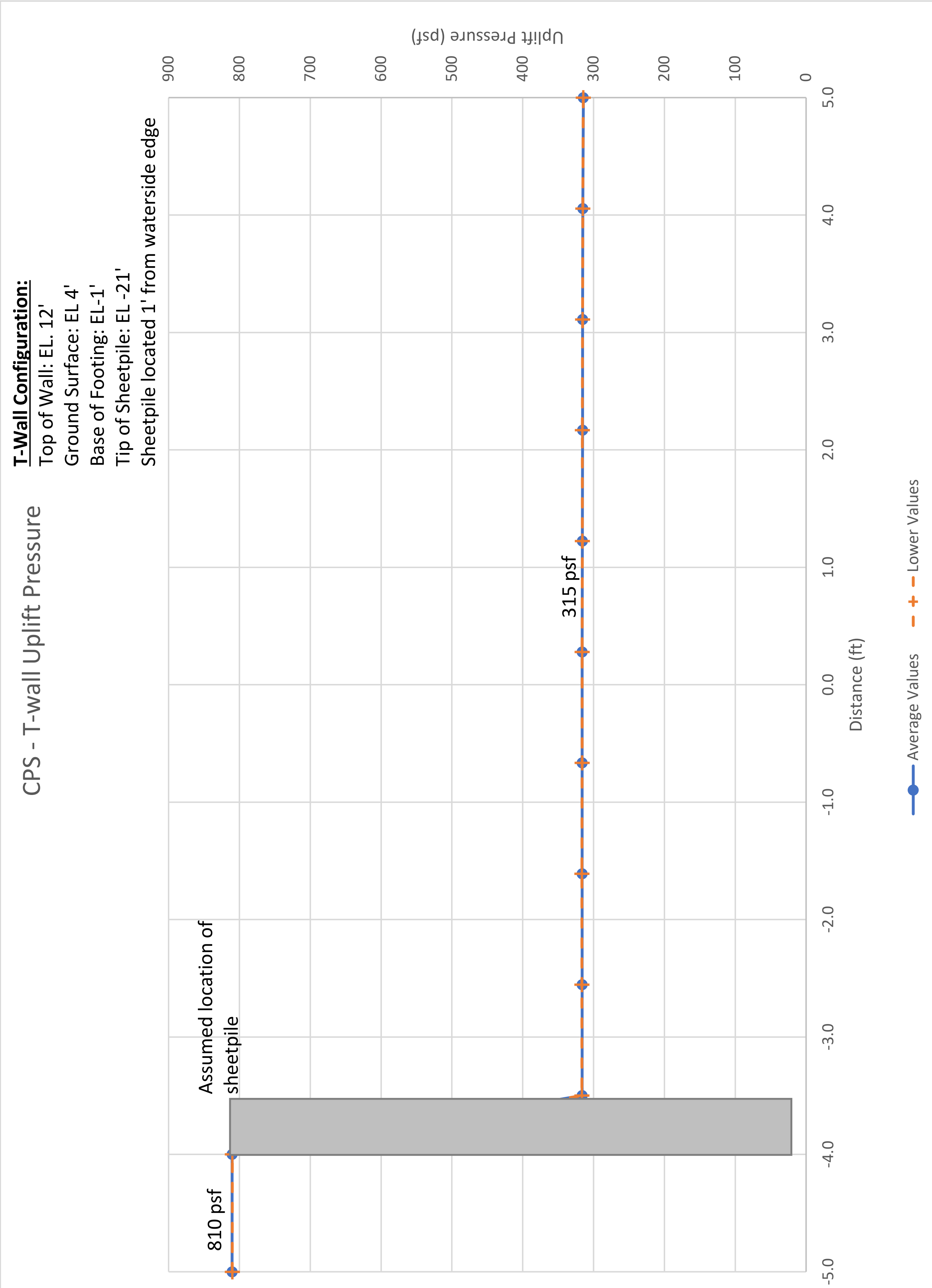

CPS - T-wall Uplift Pressure
T-Wall Configuration:
Top of Wall: EL. 12'
Ground Surface: EL 4'
Base of Footing: EL-1'
Tip of Sheetpile: EL -21'
Sheetpile located 1' from waterside edge
Uplift Pressure (psf)
0
100
200
300
400
500
600
700
800
900
Distance (ft)
5.0
4.0
3.0
2.0
1.0
0.0
-1.0
-2.0
-3.0
-4.0
-5.0
315 psf
810 psf
Assumed location of sheetpile
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					FOS	
Y (ft)	X (ft)	Water Total Head (ft)	Excess Head	Pore Pressure	Gradient	Gradient	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

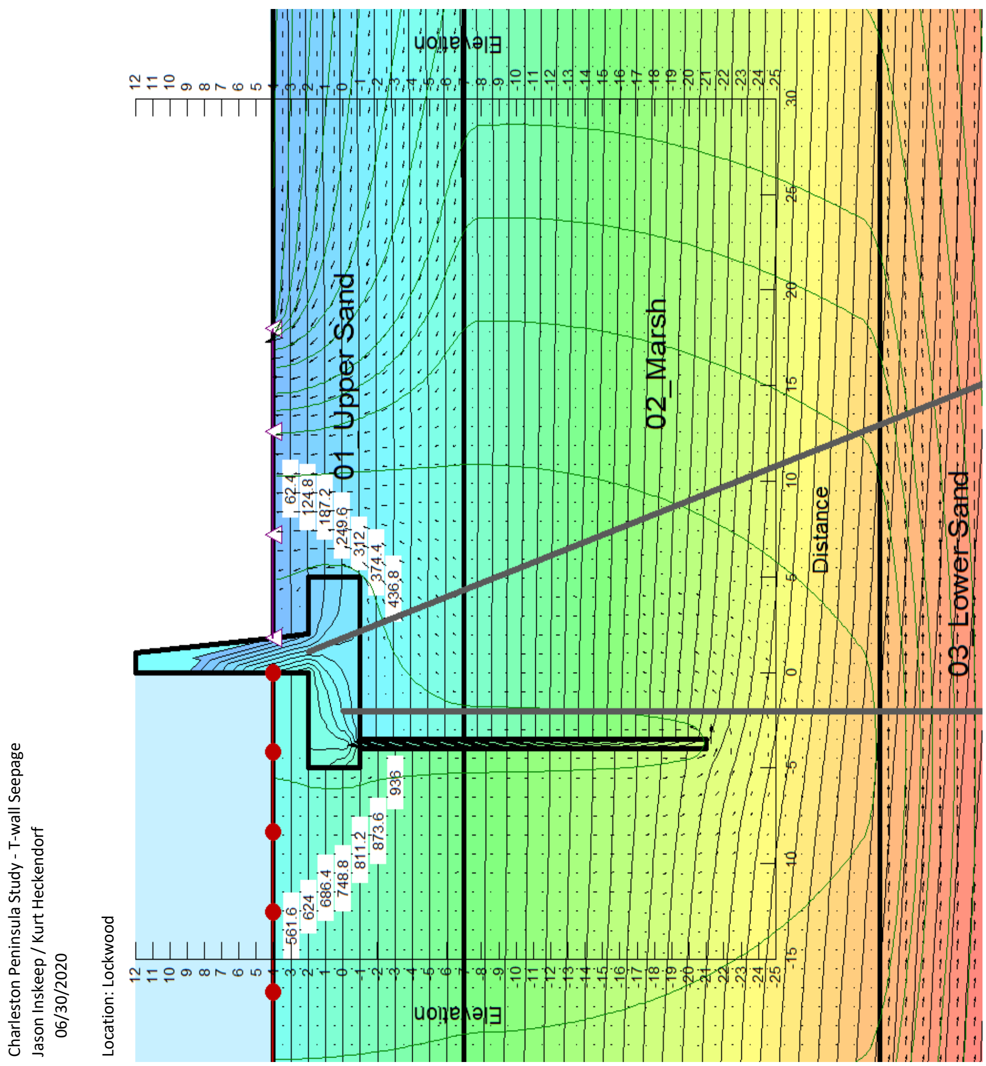
Blocked Seepage 13' beyond footing on dryside.

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

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

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	Lockwood Top of Lower Sand	Lockwood Bottom of Lower Sand	Lockwood Top of Cooper Marl	Pumpstation Bottom of Upper Sand	Pumpstation Top of Lower Sand	Pumpstation Bottom of Lower Sand	Pumpstation Top of Cooper Marl	22 Westedge Bottom of Upper Sand	22 Westedge Top of Lower Sand	22 Westedge Bottom of Lower Sand	22 Westedge 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	

		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**

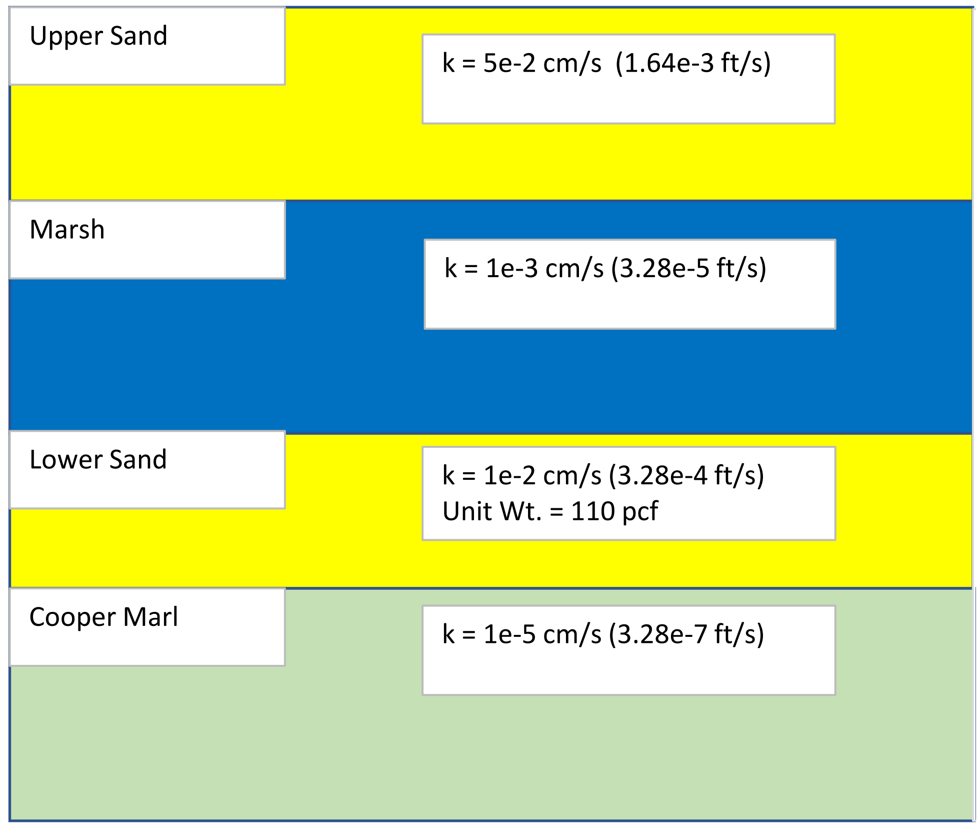
*refers to base elevation of formations

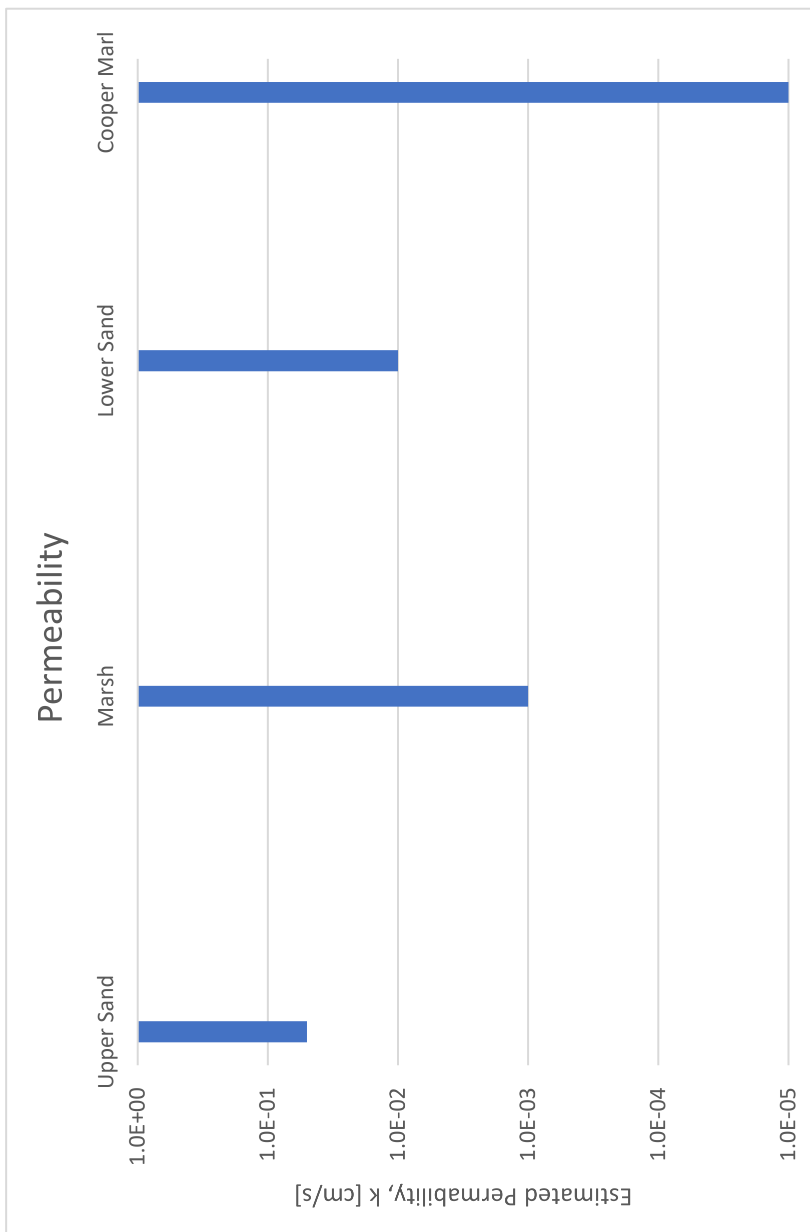
Charleston Peninsula Study - Lockwood Soil Profiles
06/17/2020
Jason Inskeep

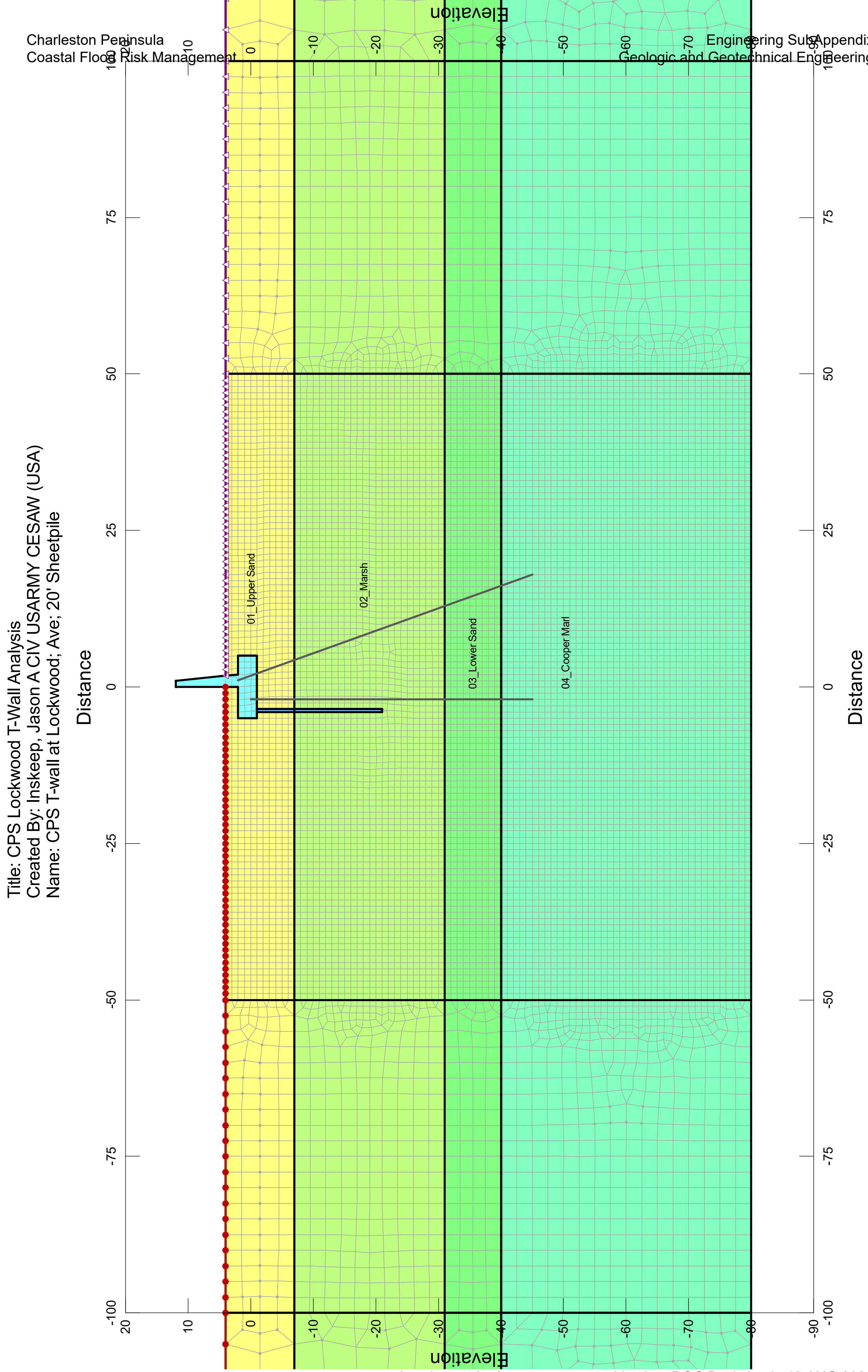
Permeability Values from USBR	cm/sec		m/day		m/sec		ft/sec
Fine to Medium Sand	1.15E-03	1.15E-02	1.00E+00	1.00E+01	1.15E-05	1.15E-04	3.77E-04
Clean Sand and Gravel			1.00E+02		1.15E-03		3.77E-03
Silt and Clay	1.15E-04	1.15E-05	1.00E-01	1.00E-02	1.15E-06	1.15E-07	3.77E-07
Cooper Marl	1.15E-07	1.15E-08	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						

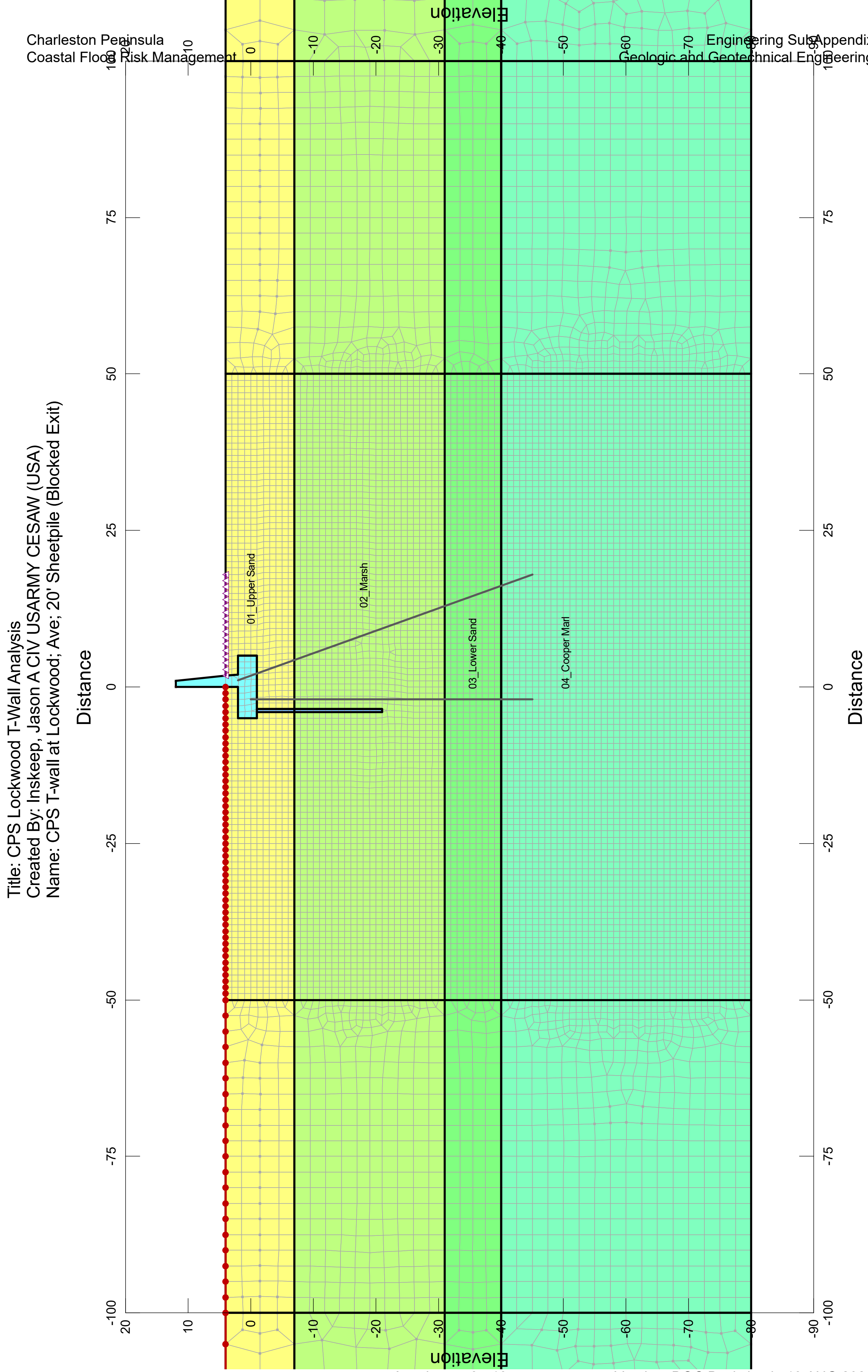
Permeability

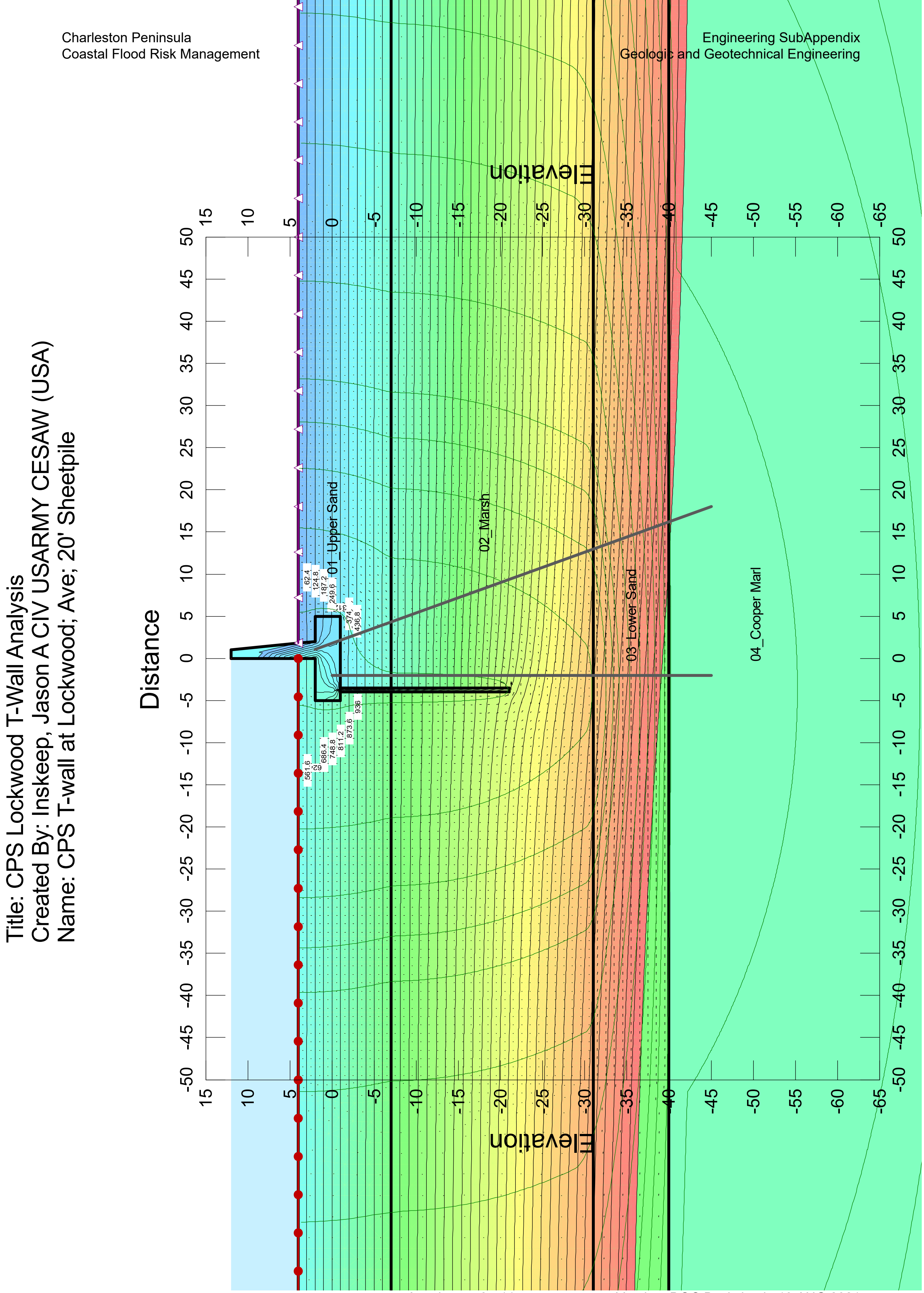
	m/s	cm/s	ft/s
Upper Sand	5.0E-04	5.0E-02	1.64E-03
Marsh	1.0E-05	1.0E-03	3.28E-05
Lower Sand	1.0E-04	1.0E-02	3.28E-04
Cooper Marl	1.0E-07	1.0E-05	3.28E-07



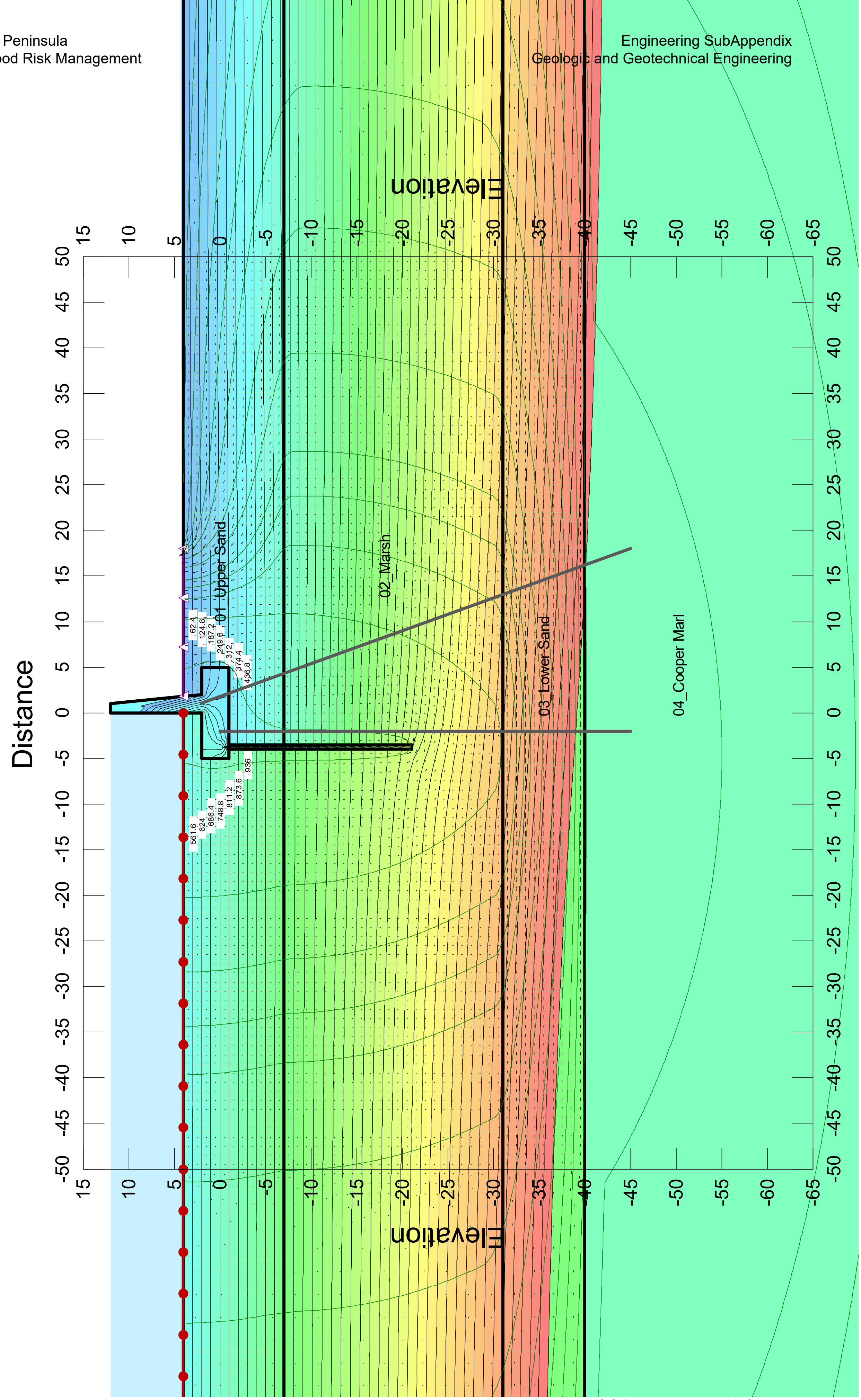

Permeability
Upper Sand
Marsh
Lower Sand
Cooper Marl
1.0E+00
1.0E-01
1.0E-02
1.0E-03
1.0E-04
1.0E-05
Estimated Permability, k [cm/s]

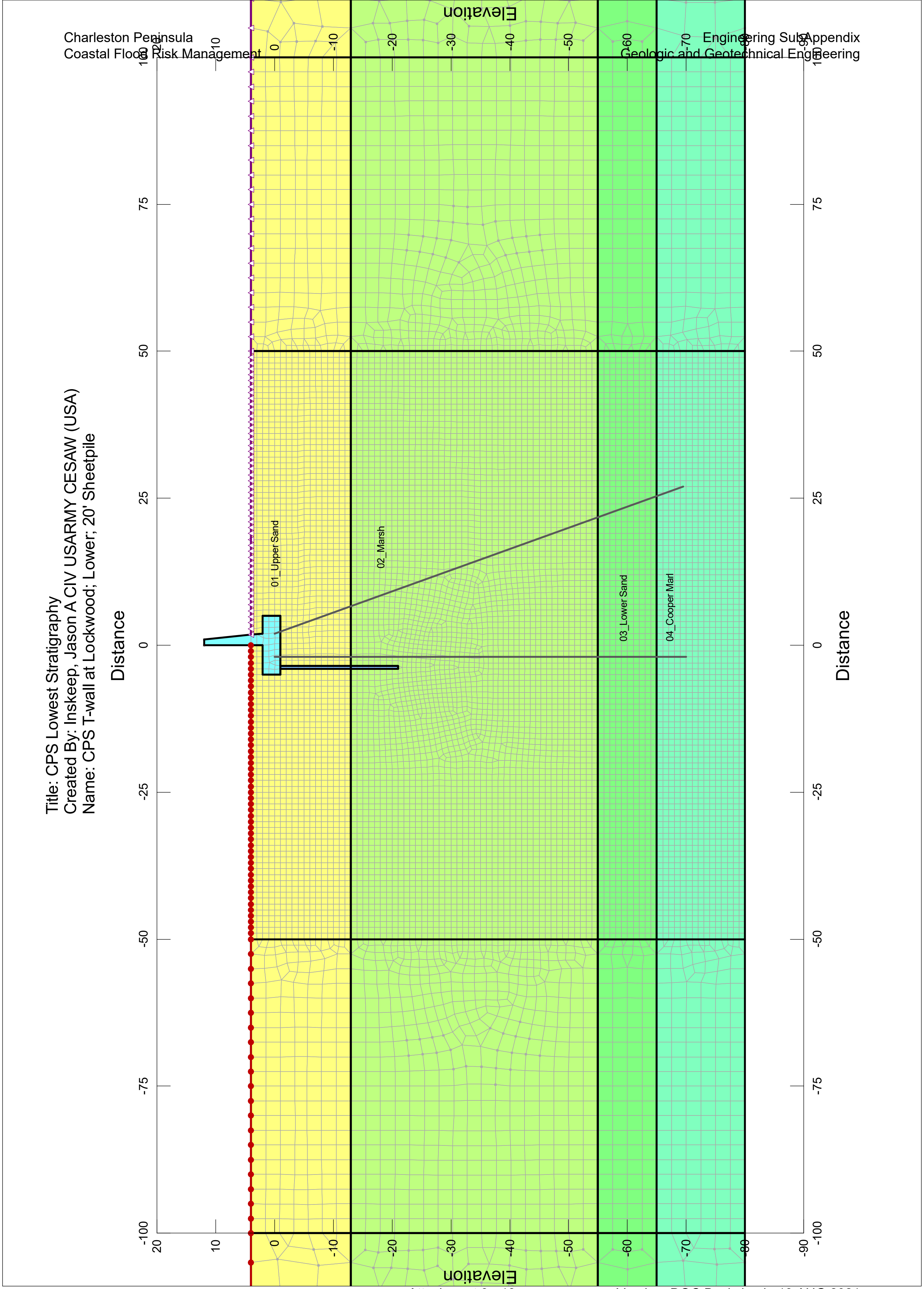

Title: CPS Lockwood T-Wall Analysis
Created By: Inskeep, Jason A CIV USARMY CESAW (USA)
Name: CPS T-wall at Lockwood; Ave; 20' Sheetpile
01_Upper Sand
02_Marsh
03_Lower Sand
04_Cooper Marl
Distance
Elevation

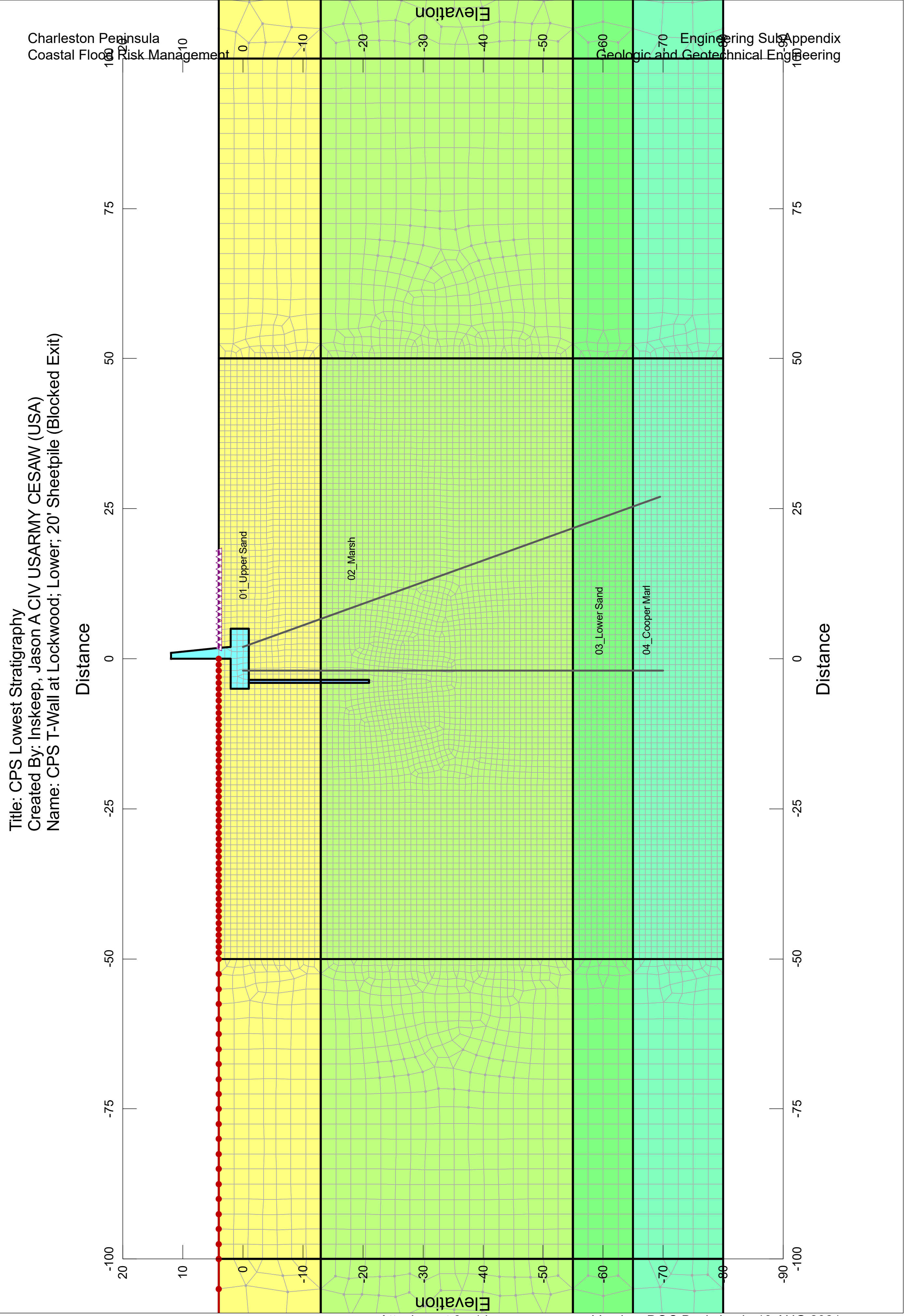

Title: CPS Lockwood T-Wall Analysis
Created By: Inskeep, Jason A CIV USARMY CESAW (USA)
Name: CPS T-wall at Lockwood; Ave; 20' Sheetpile (Blocked Exit)
Distance
Elevation
01_Upper Sand
02_Marsh
03_Lower Sand
04_Cooper Marl

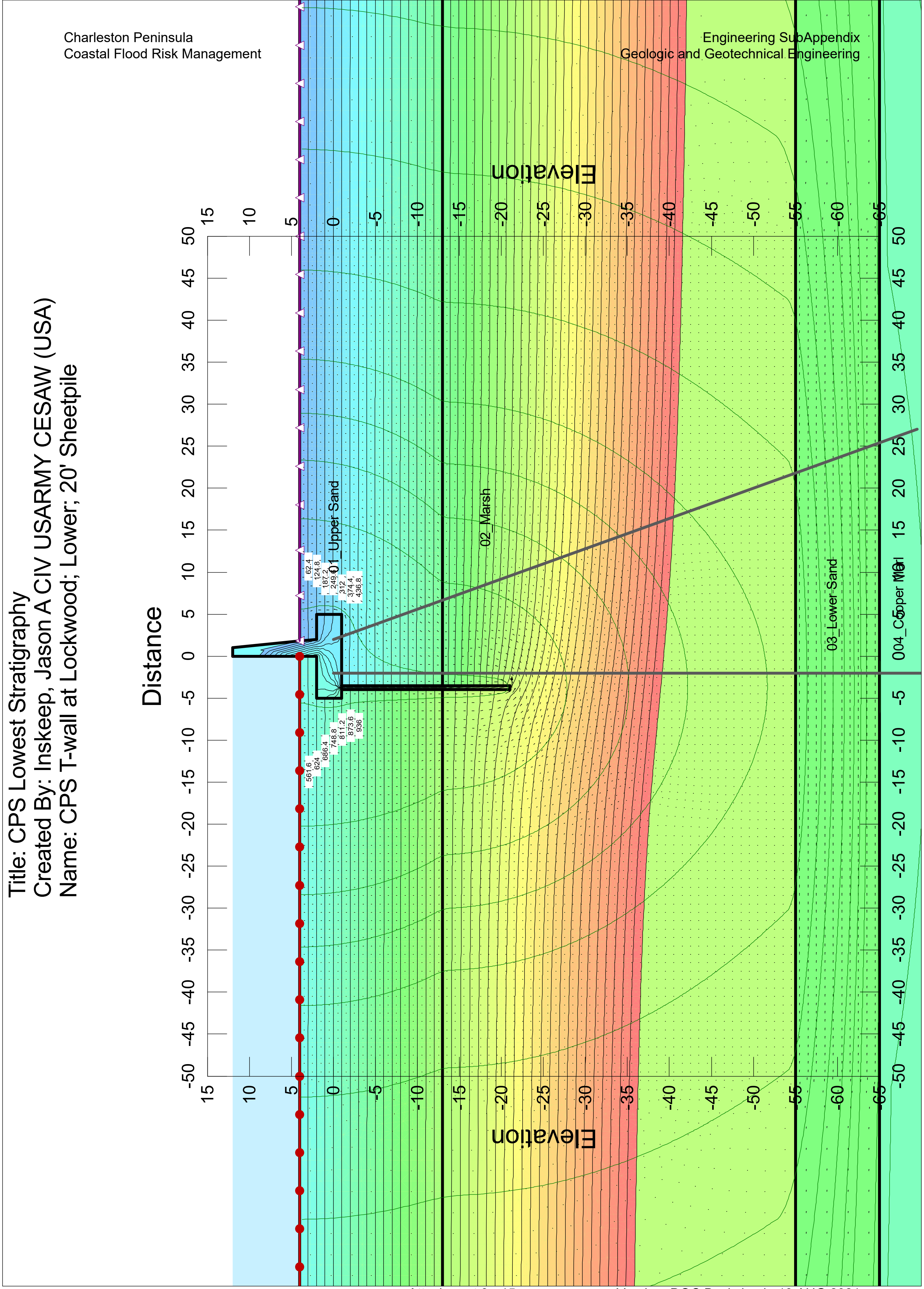

Title: CPS Lockwood T-Wall Analysis
Created By: Inskeep, Jason A CIV USARMY CESAW (USA)
Name: CPS T-wall at Lockwood; Ave; 20' Sheetpile
Distance
Elevation
01_Upper Sand
02_Marsh
03_Lower Sand
04_Cooper Marl
62.4
124.8
187.2
249.6
374
436.8
561.6
686.4
748.8
811.2
873.6
936

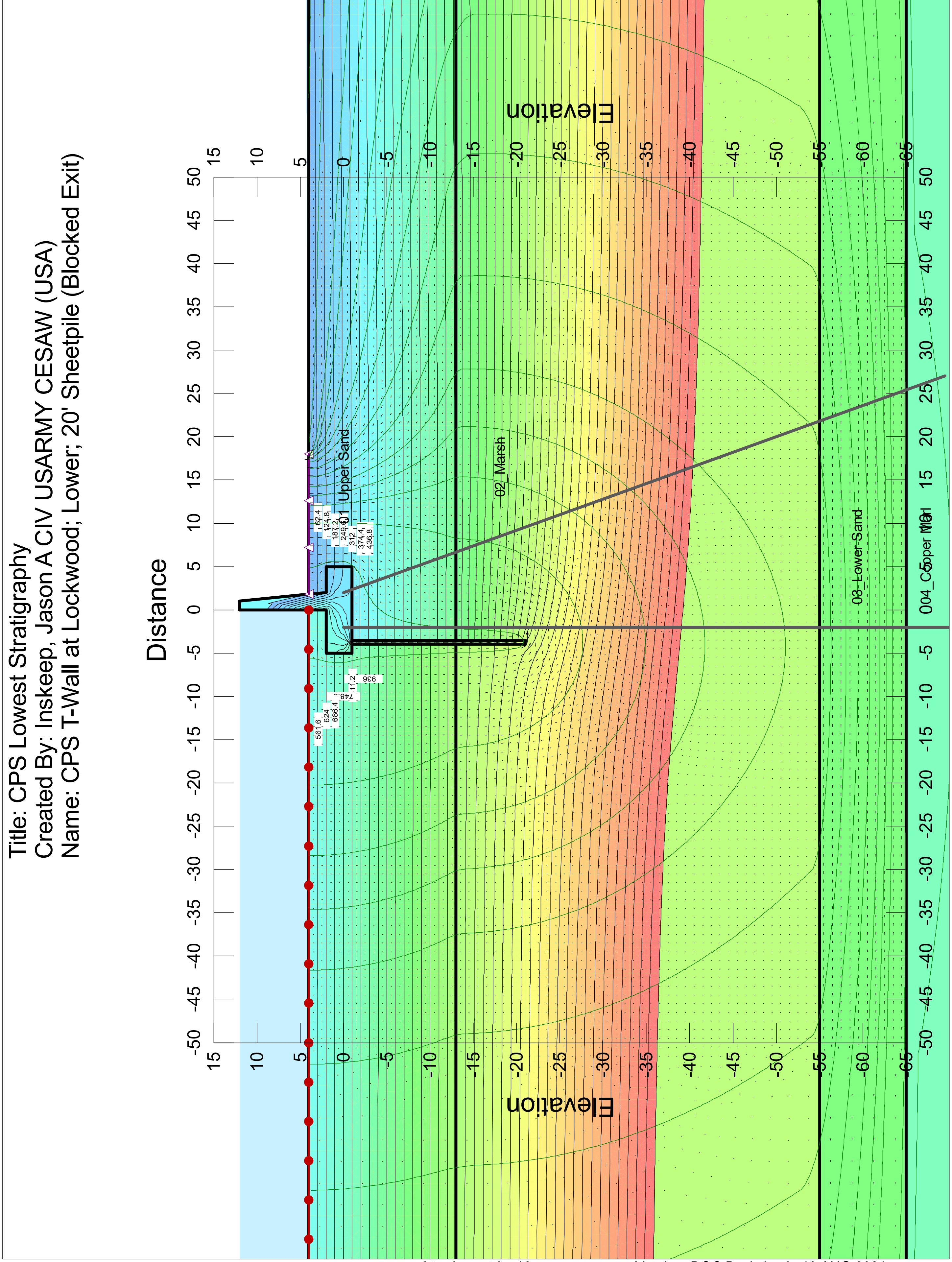
Title: CPS Lockwood T-Wall Analysis
Created By: Inskeep, Jason A CIV USARMY CESAW (USA)
Name: CPS T-wall at Lockwood; Ave; 20' Sheetpile (Blocked Exit)




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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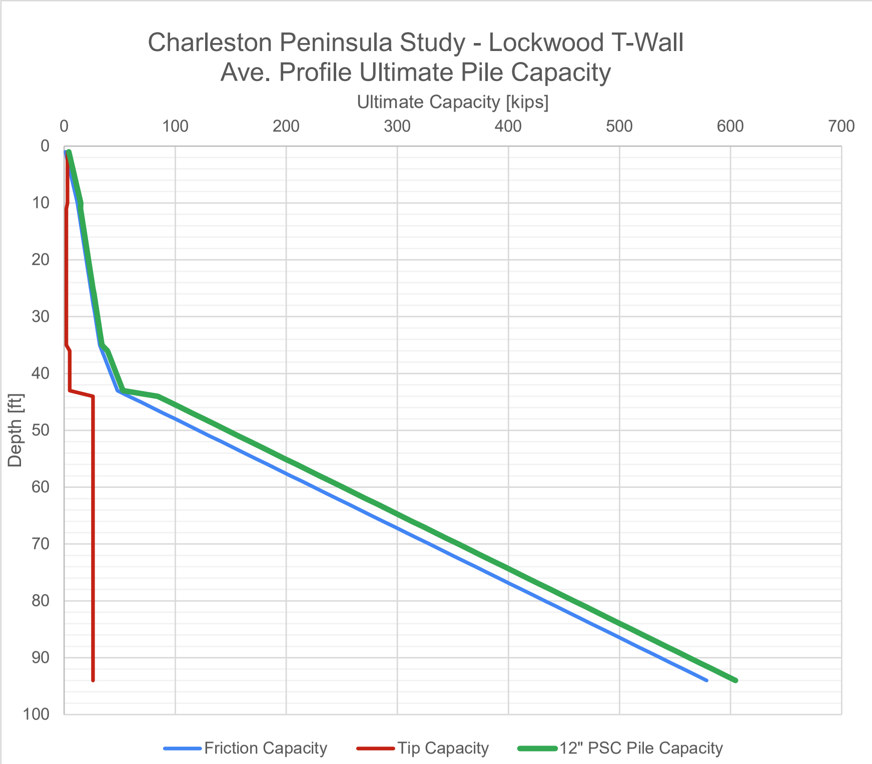
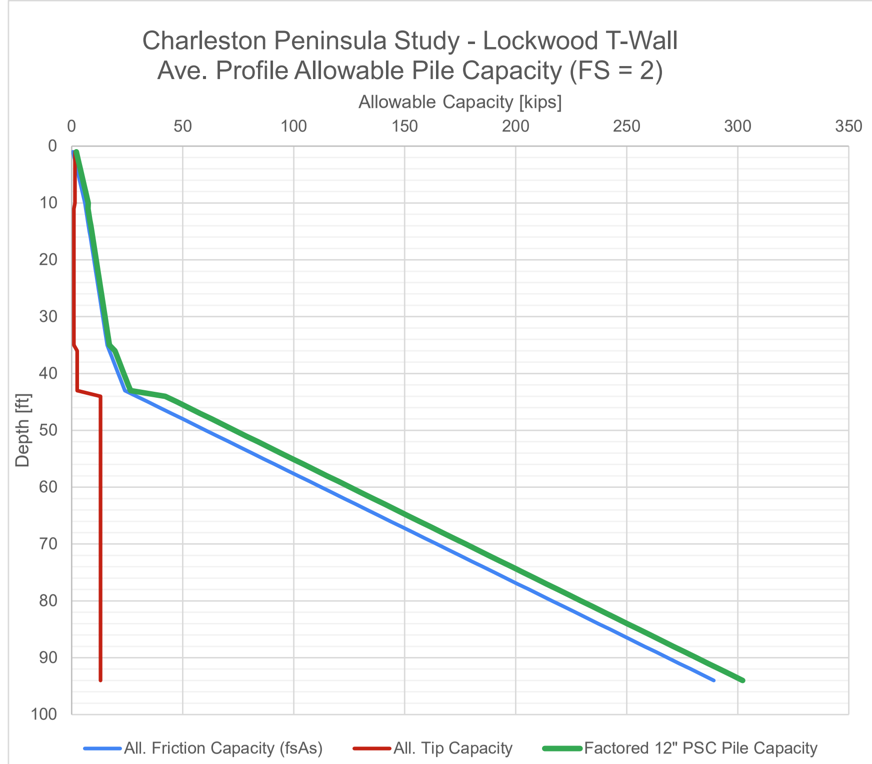
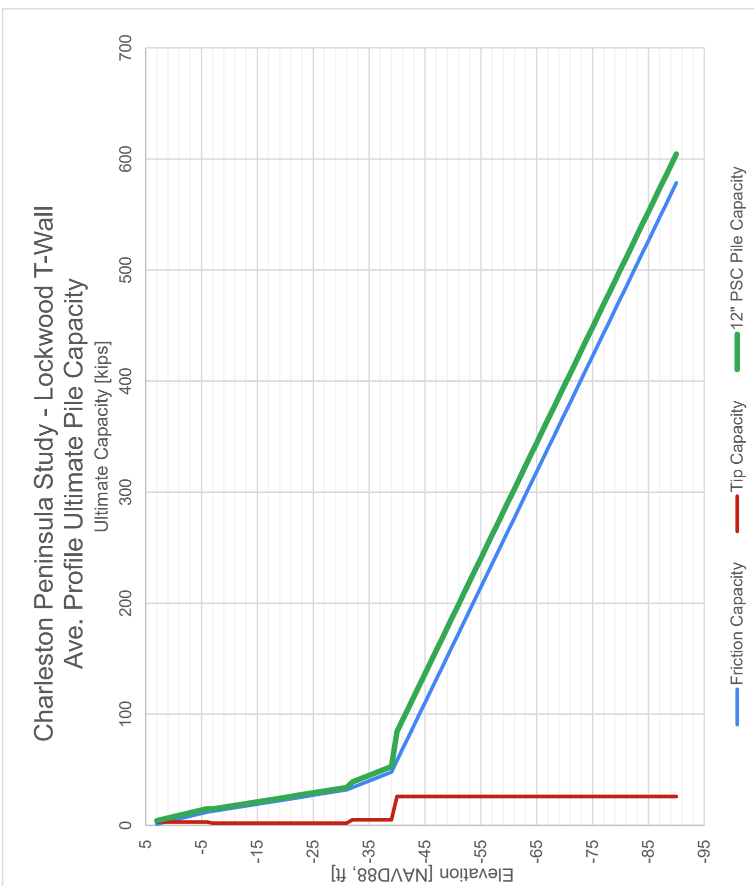
Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Average Marl Depth, Top of Marl at EL. -40 FT
Computed By: JAI Revised By: JAI
Date: 07/02/2020 Date Revised: 07/06/2020

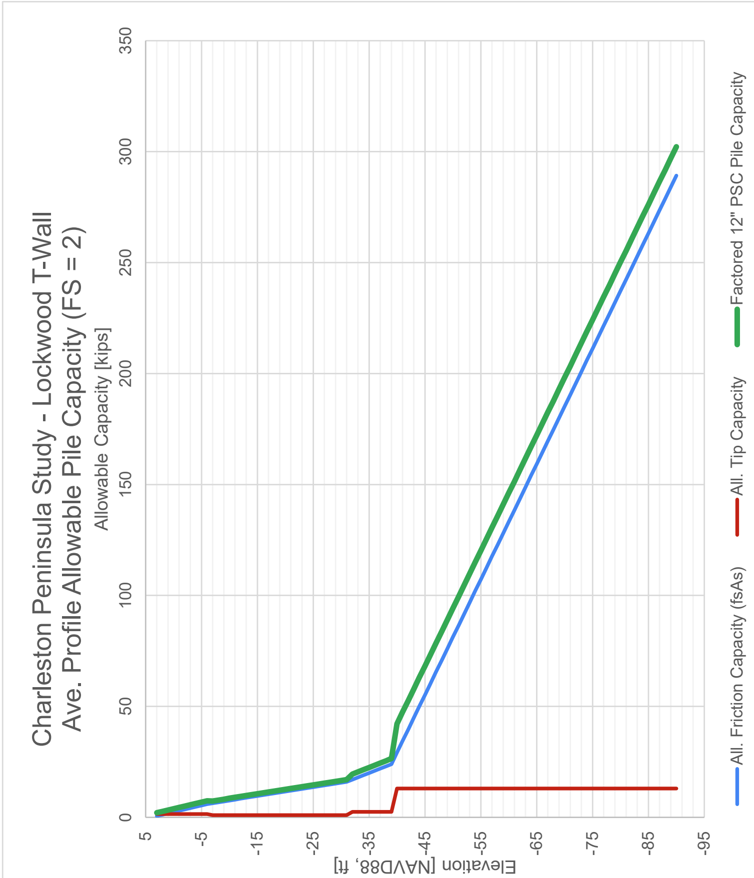
Reviewed By: KAH
Date Reviewed: 07/02/2020





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
Revised By: JAI
Reviewed By: KAH
eference 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

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
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)***	***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
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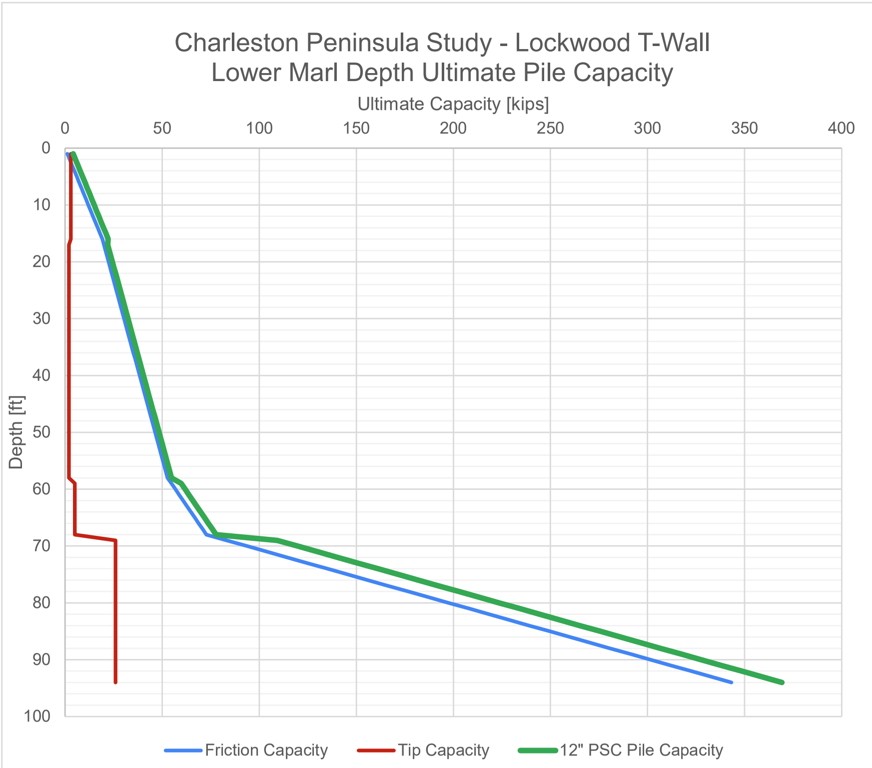
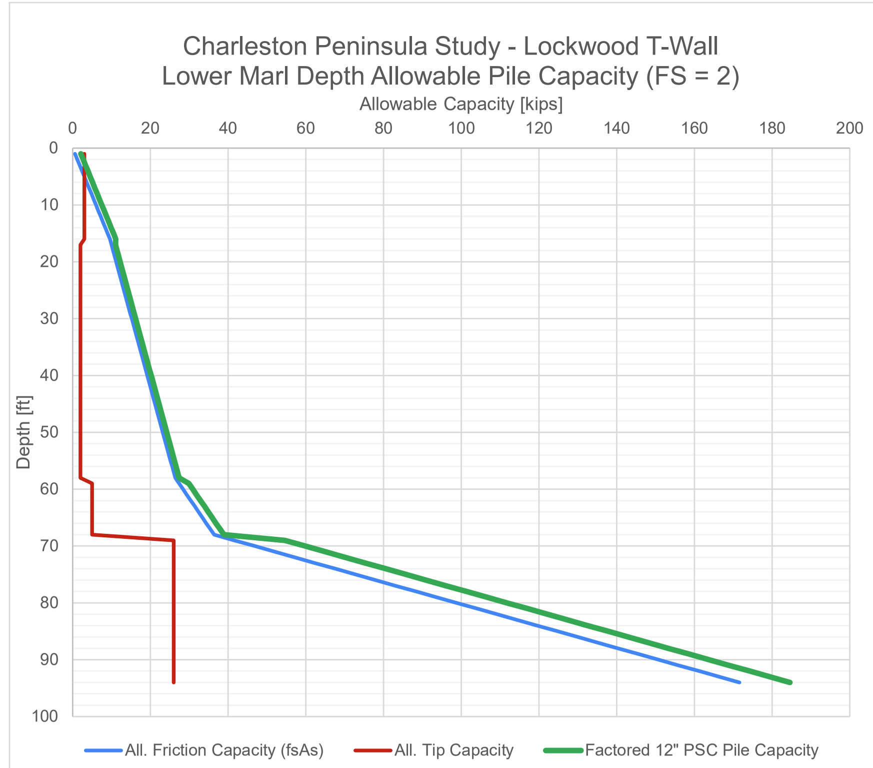
Date Revised: 07/06/2020
Reviewed By: KAH
Date Reviewed: 07/02/2020





Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Lower Marl Depth, Top of Marl at EL. -65 FT
Computed By: JAI
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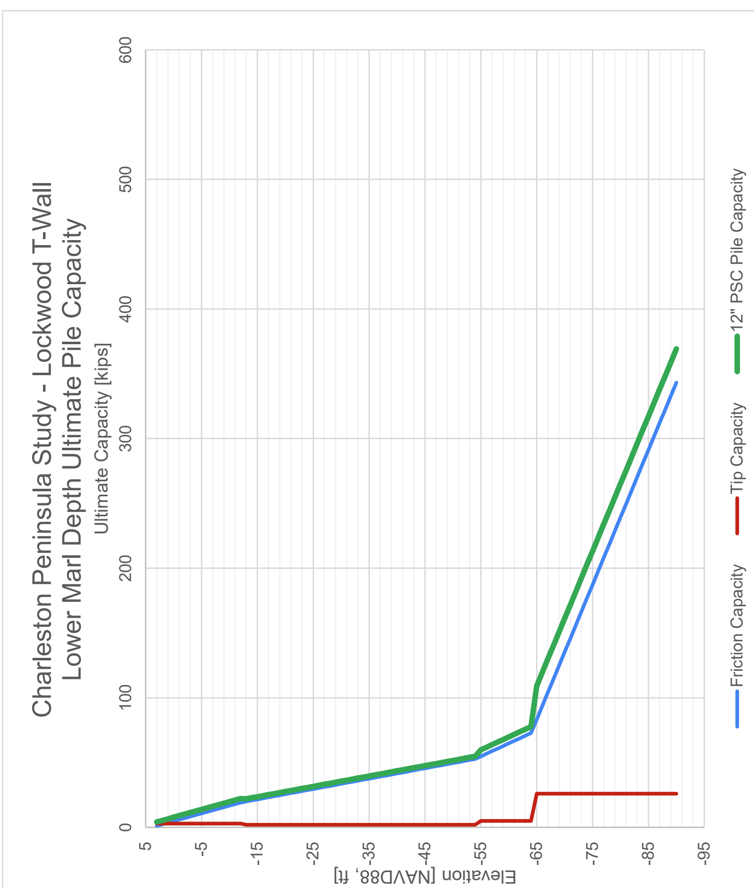
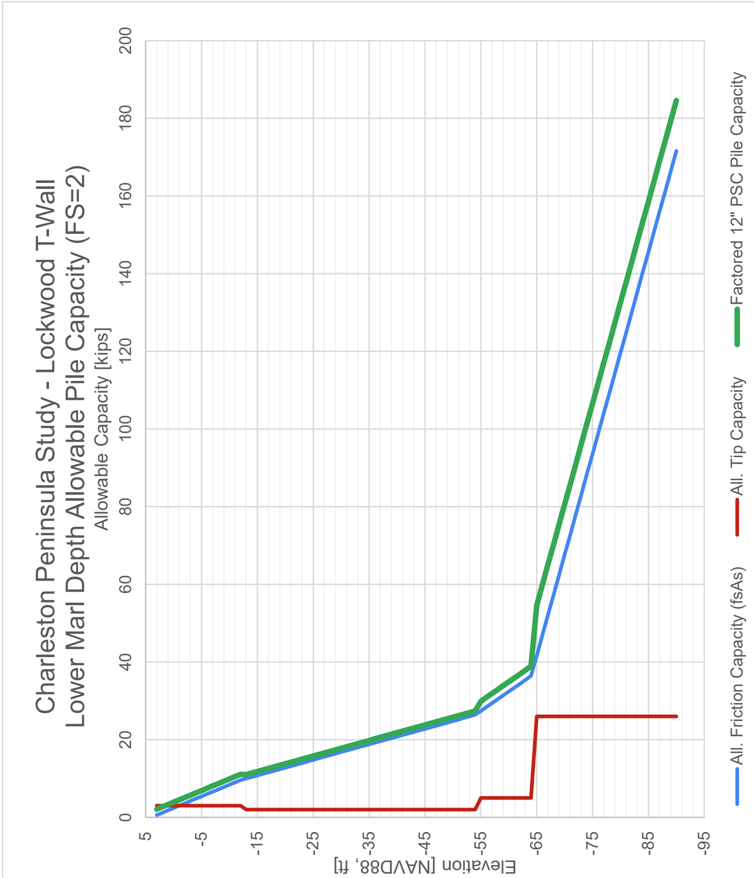




Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Lower Marl Depth, Top of Marl at EL. -65 FT
Computed By: JAI
Date: 07/02/2020
Revised By: JAI
Date Revised: 07/06/2020
Reviewed By: KAH
Date Reviewed: 07/02/2020
eference Project 22 Westedge

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
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

	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
Date Revised: 07/06/2020
Reviewed By: KAH
Date Reviewed: 07/02/2020
ference Project 22 Westedge

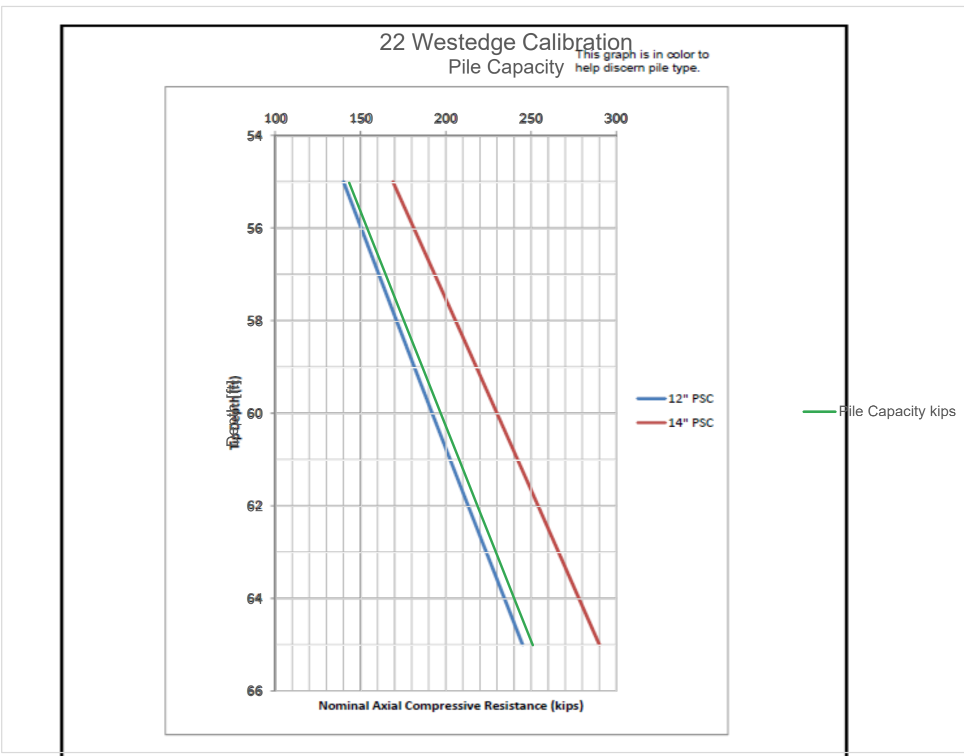
Boring		
Drill Rig	CPT	
Depth to Water	2	
Su/sigmaz' NC	0.22	
Factor of Safety	2	
Nc*	10	
Ground Surface	4	ft NAVD88

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 efficeintlcy 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			Pile Section	12-in. PSC			Su	Depth to Bottom of Layer
Drill Rig	CPT		Pile area	144	in^2	Upper Sand	400	10
Depth to Water	2			1.000	ft^2	Marsh/Muck	200	39
Su/sigmaz' NC	0.22		Depth of Section	12	ft	Sand	500	56
Factor of Safety	2		Flange Width, bf	12	ft	Silty Sand/Marl	2700	
Nc*	10		Pile Perimeter	4.00	ft			
Ground Surface	4	ft NAVD88						

Used greatest values of depths to change in material type as reported in the original report.

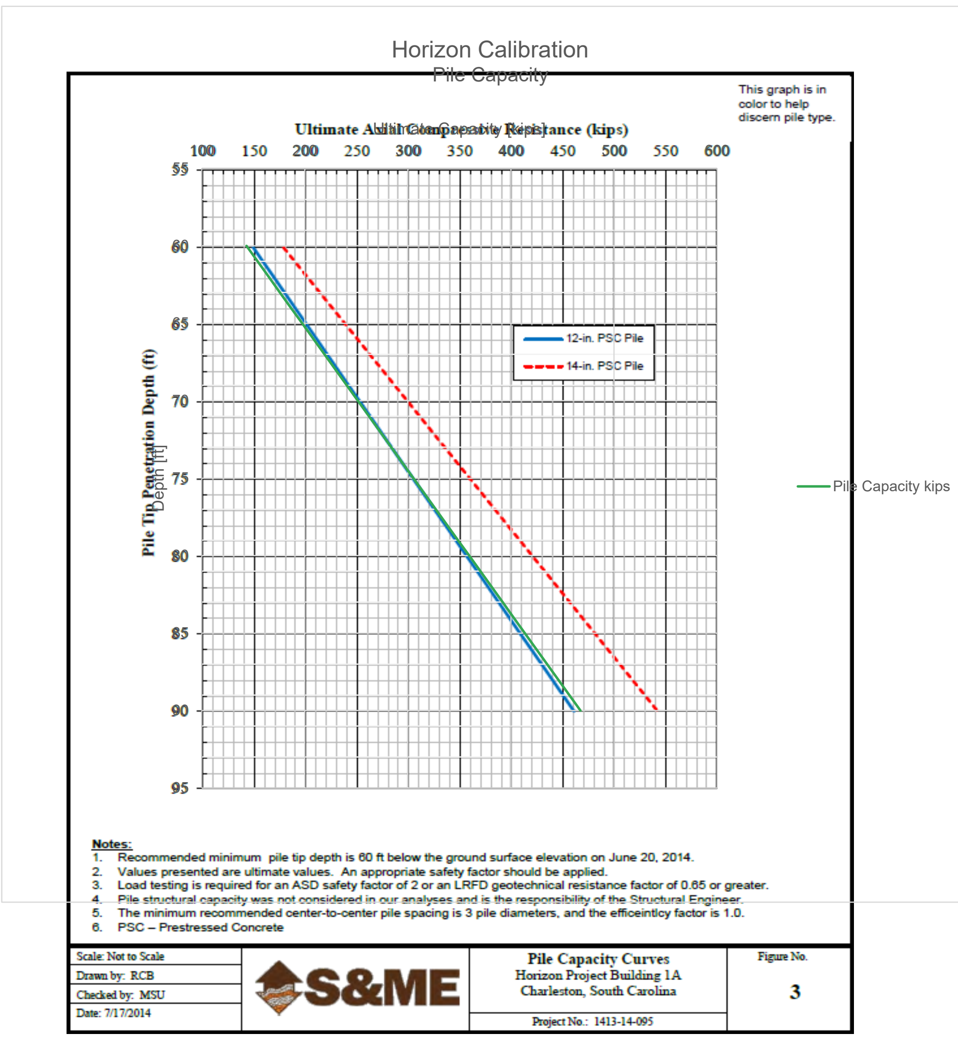
Elevation	Depth	Material	Su	alpha	Nominal Friction Cap. (fs)	Friction Capacity (fsAs)	All. Friction Capacity	Nominal Tip Capacity (qn)	Tip Capacity (qnAt)	All. Tip Capacity (qnAt)	Pile Capacity	Factored Pile Capacity
Feet NAVD88	ft	USCS Class	psf	-	psf	kips	kips	psf	kips	kips	kips	kips
4	0	Upper Sand										
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			Pile Section	12-in. PSC			Su	Depth to Bottom of Layer
Drill Rig	CPT		Pile area	144	in^2	Upper Sand	400	10
Depth to Water	2			1.000	ft^2	Marsh/Muck	200	39
Su/sigmaz' NC	0.22		Depth of Section	12	ft	Sand	500	56
Factor of Safety	2		Flange Width, bf	12	ft	Silty Sand/Marl	2700	
Nc*	10		Pile Perimeter	4.00	ft			
Ground Surface	4	ft NAVD88						

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 Compression Resistance (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

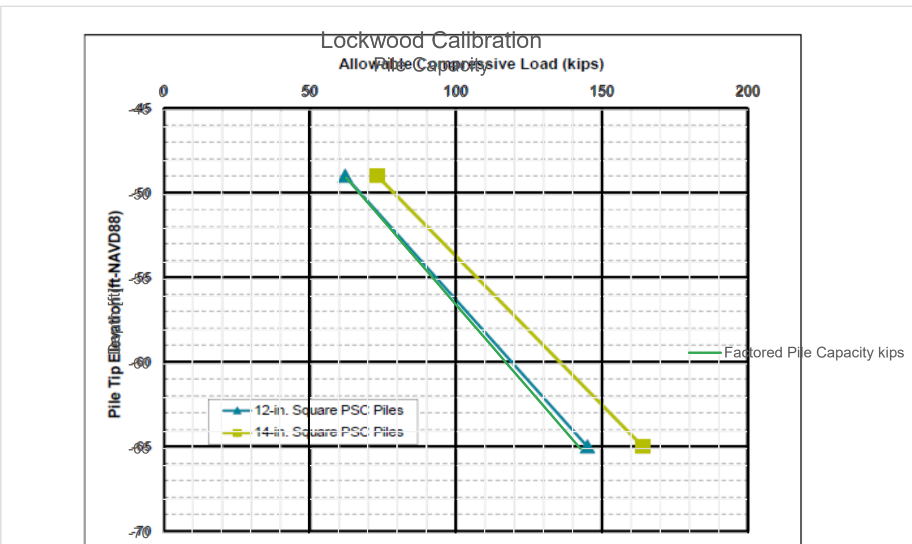
	Su
Upper Sand	200
Marsh/Muck	200
Sand	500
Silty Sand/Marl	2500

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

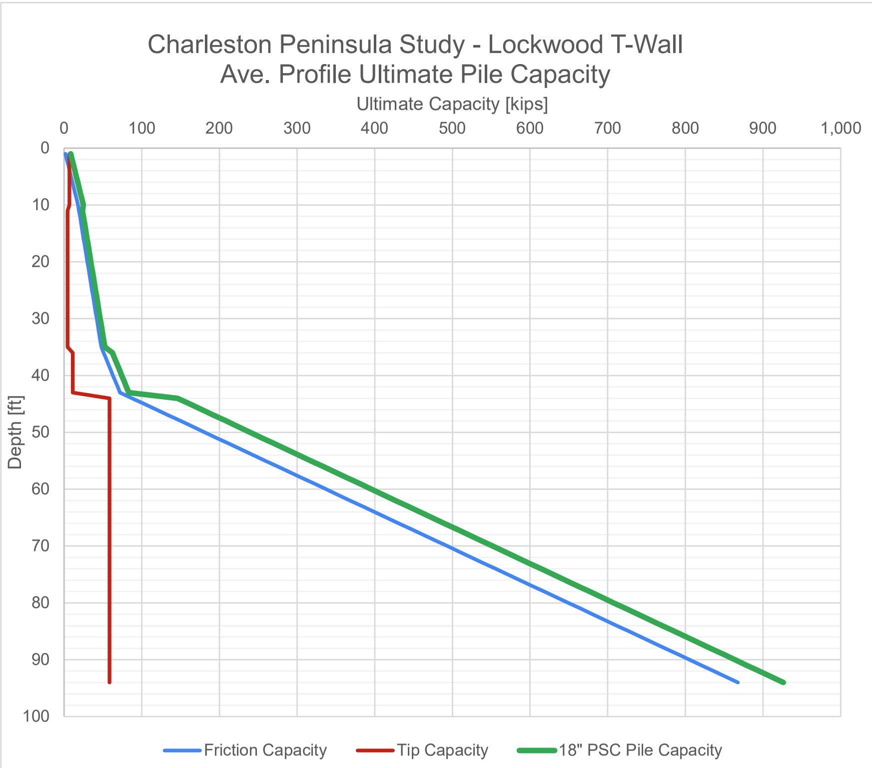


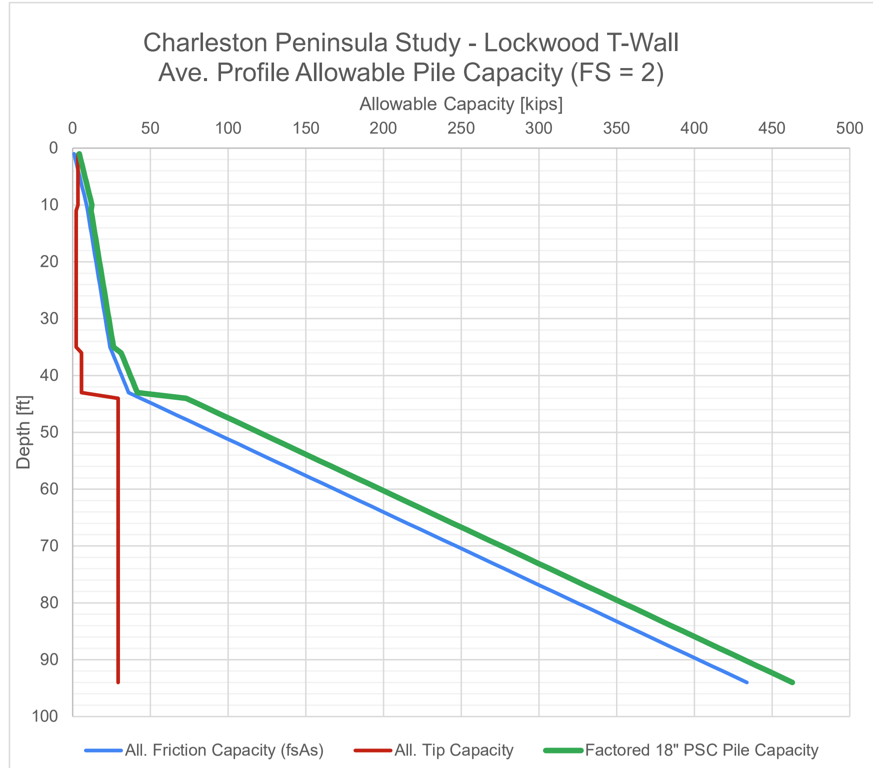
NOTES:

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)

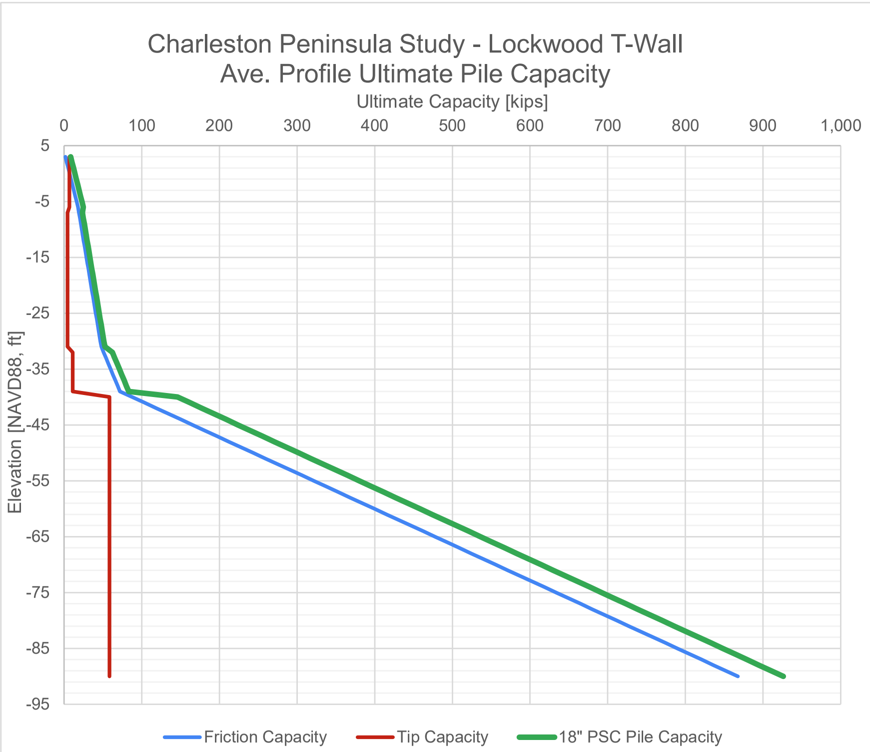
Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Average Marl Depth, Top of Marl at EL. -40 FT
Computed By: JAI
Date: 07/02/2020
Revised By: JAI
Date Revised: 07/06/2020
Reviewed By: KAH
Date Reviewed: 07/02/2020

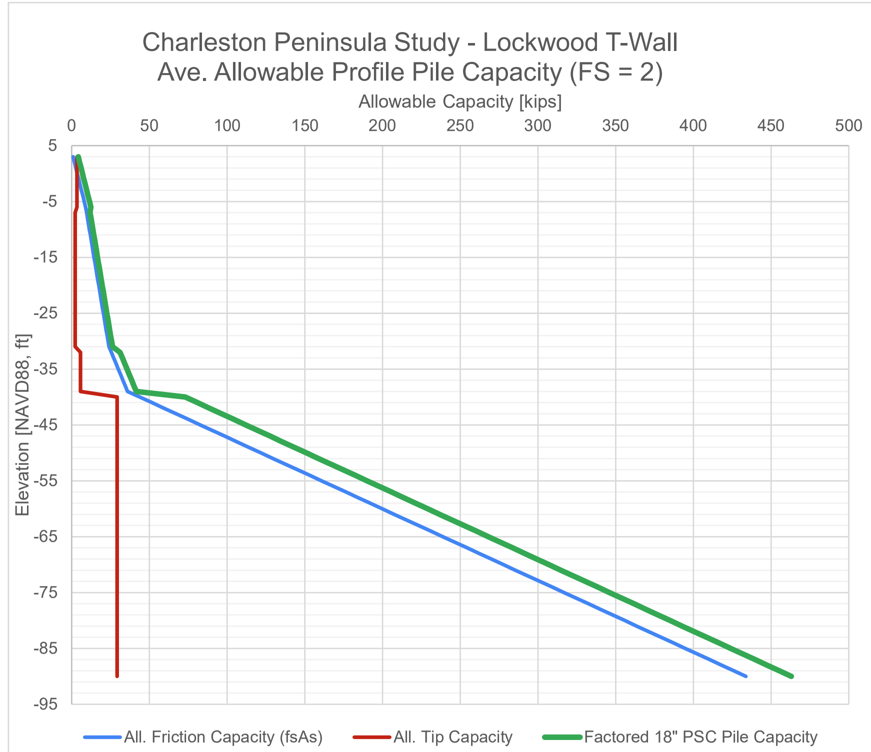




Project: Charleston Peninsula Study - Lockwood T-Wall
Subject: Pile Capacity for Average Marl Depth, Top of Marl at EL. -40 FT
Computed By: JAI | Revised By: JAI
Date: 07/02/2020 | Date Revised: 07/06/2020

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
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

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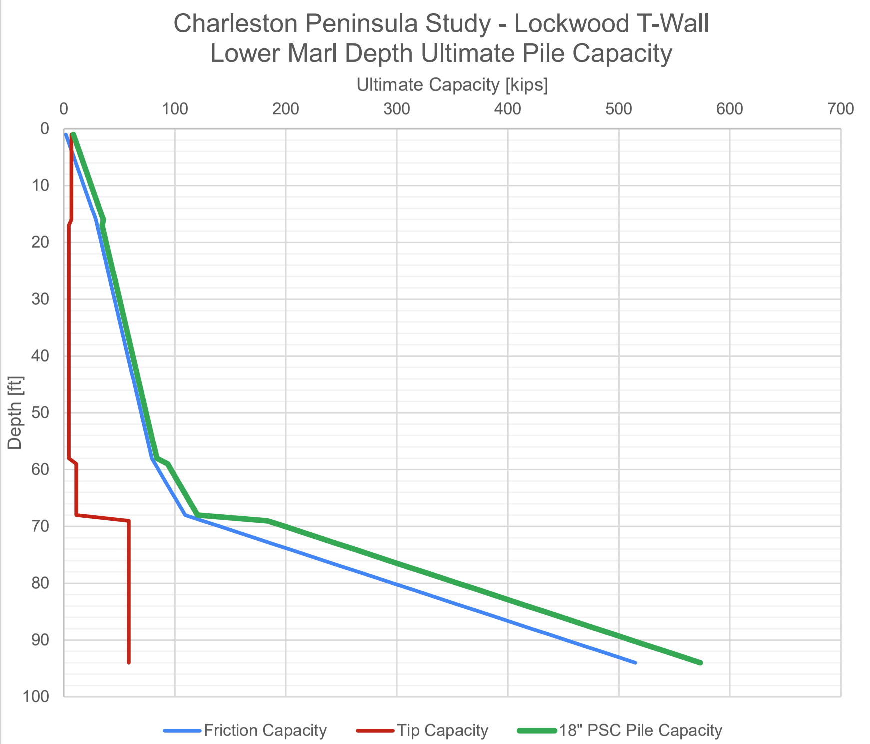
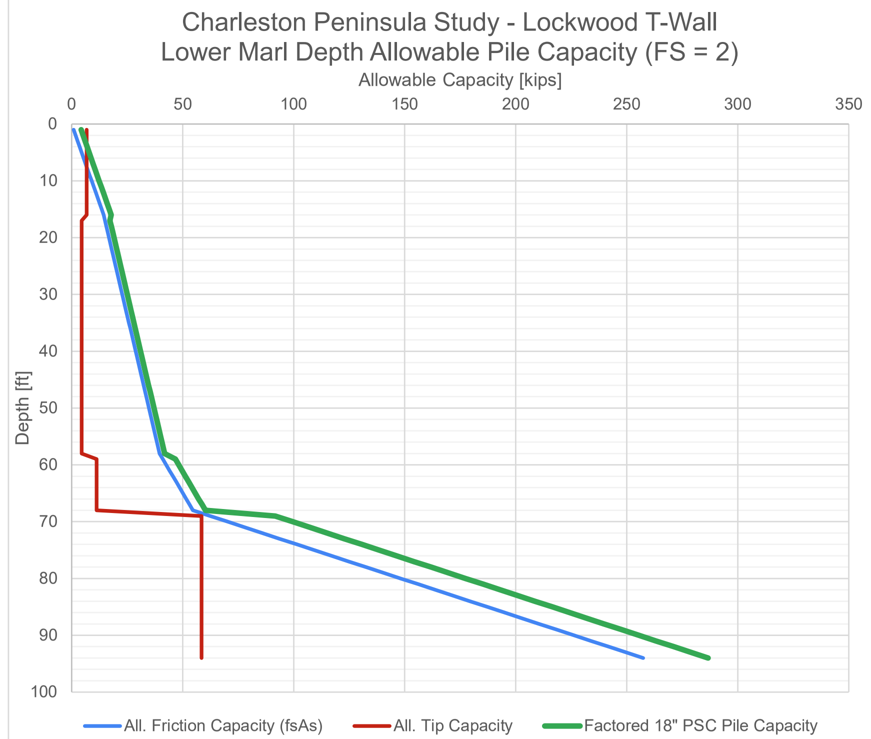
Date: 07/02/2020   Date Revised: 07/06/2020   Date Reviewed: 07/02/2020





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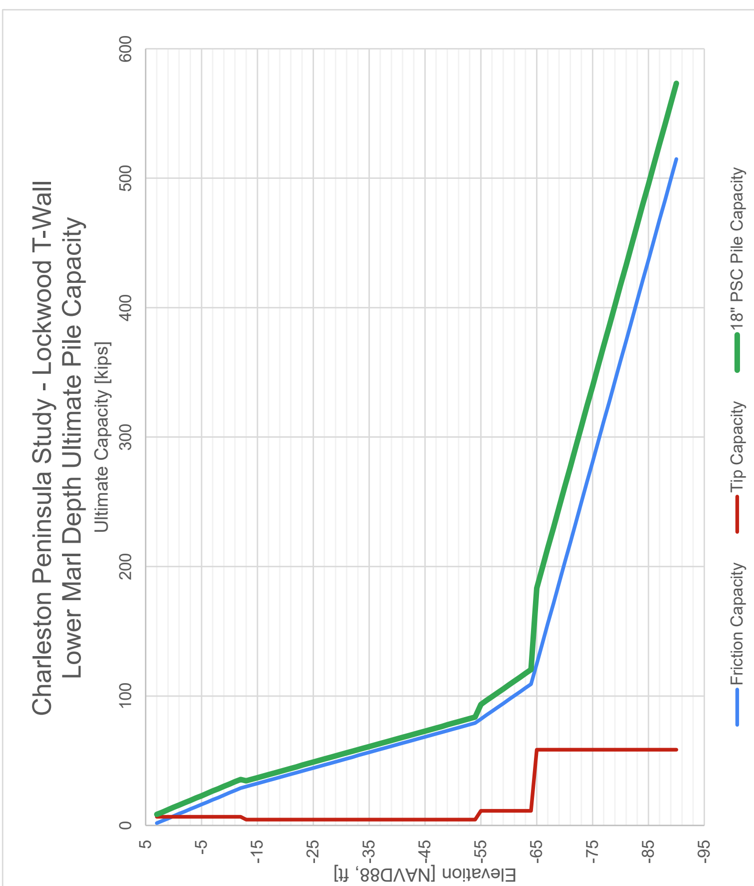

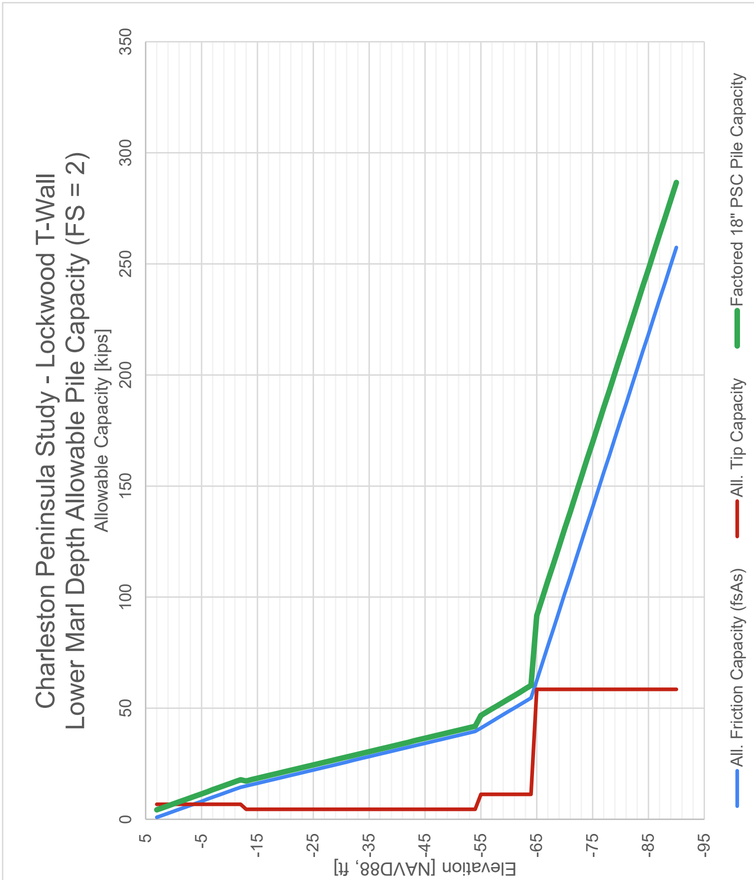

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ɛference Project 22 Westedge

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

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Boring	
Drill Rig	CPT
Depth to Water	2
Su/sigmaz' NC	0.22
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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
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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

US Army Corps of Engineers®

Charleston District

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

Charleston, South Carolina

Hydraulics, Hydrology, and Coastal SUB-APPENDIX B-3
(Interior Hydrology HEC-RAS 2D Modeling)

August 2021

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Introduction

The purpose of the interior drainage analysis to estimate the hydraulic response of the proposed system for various storm gate alternatives and pump station alternatives using the Hydrologic Engineering Center's (CEIWR-HEC) River Analysis System (HEC-RAS) software. This analysis will assist in analyzing the impact to rainfall runoff induced by the wall as it impedes the runoff to naturally drain as it does without a wall.

A variety of scenarios were computed using mean high tide levels projected to the years 2032 and 2082, which are the years for end of construction and 50-year project life. These tide levels were used as stage boundary conditions and computed in combination with a rainfall suite consisting of the 50%, 20%, 10%, 4%, 2%, and 1% Annual Exceedance Probability (AEP) rainfall events. In supplement to analyzing rainfall at high tide, a variety of scenarios were also computed to analyze rainfall during storm surge and the wave wash overtopping stemming from storm surge.

Utilizing the hydrologic and meteorologic conditions detailed in this report, future without-project and future with-project conditions were modeled. The future without-project conditions were computed with geometry assumptions detailed in this report. The future with-project conditions were computed to analyze the performance of various storm gate and pump station alternatives. The storm gate simulations were computed as an open system and the pumps were computed as a closed system. The storm gates scenarios are assumed to be open to allow for daily tidal fluctuations and to drain the interior rainfall runoff. The pumps scenarios were computed with storm gates closed; therefore, a closed system, and pumps are utilized as the rainfall mitigation feature to remove the excess interior rainfall runoff. The City of Charleston drainage infrastructure, as stated by the City of Charleston, does not accommodate events more than a 10% AEP rainfall. The focus of this study is not to increase or enhance the City's drainage infrastructure but to ensure the Charleston PDT's proposed system does not induce damages that would otherwise not occur without the system in place. The system is analyzed with the goal of limiting the increase in interior stages for all rainfall events while applying a focus on the 10% AEP rainfall event for conceptual design purposes.

The outputs from the HEC-RAS modeling were utilized as inputs for economics modeling within the Hydrologic Engineering Center's (CEIWR-HEC) Flood Damage Reduction Analysis (HEC-FDA) software. The economics team member computed damages with the goal of capturing the Expected Average Annual Damages for each without-project scenario and each with-project scenario. The damages for each without-project event was then compared to the damages for its respective with-project event. The damage estimates were used to select the storm gate and pump station alternatives to be included within the project cost estimate. Peak water surface elevations and hydrographs from the RAS modeling are provided at 24 output locations to provide a general sense of the impact to water level change at various locations. Water level impacts at other areas may differ. Selected output locations and HEC-FDA modeling provide insight into the systems performance, but site-specific analysis will be further analyzed in PED phase.

Coastal modeling was performed by a Galveston District H&H engineer. This effort was performed using EUROTOP methodology to provide overtopping flow rates to be utilized within the HEC-RAS modeling effort. More information on this effort is found in Section 4.5.3 of this report.

The Relative Sea Level Change (RSLC) within this effort assume NOAA's 2006 published intermediate rates of +0.56 feet for the year 2032 and +1.65 feet for the year 2082. These values were used to adjust the stage boundary conditions within the HEC-RAS modeling.

*All elevations used in the modeling were converted to the North American Vertical Datum of 1988 (NAVD88).

1. General Description of Work

The purpose of the interior drainage analysis is to estimate the hydraulic response of the proposed system for various storm gate and pump station alternatives. This analysis will assist in analyzing the impact to rainfall runoff induced by the wall as it impedes the runoff to naturally drain as it does without a wall.

A variety of scenarios were computed using mean high tide levels projected to the years 2032 and 2082, which are the years for end of construction and 50-year project life. These tide levels were used as stage boundary conditions and computed in combination with a rainfall suite consisting of the 50%, 20%, 10%, 4%, 2%, and 1% AEP rainfall events. In supplement to analyzing rainfall at high tide, a variety of scenarios were computed to analyze storm surge with rainfall and the wave wash overtopping stemming from the storm surge. During feasibility phase, the interior drainage for future without-project conditions versus future with-project conditions using the 12 ft. NAVD88 wall alignment provided by the PDT will be analyzed against one another.

The focus of this study is not to increase or enhance the City's drainage infrastructure but to ensure the Charleston PDT's proposed system does not induce damages that would otherwise not occur without the system in place. The system is analyzed with the goal of limiting the increase in interior stages for all rainfall events while applying a focus on the 10% AEP rainfall event for conceptual design purposes. If the system 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.

The Relative Sea Level Change (RSLC) within this effort assume NOAA's 2006 published intermediate rates of +0.56 feet for the year 2032 and +1.65 feet for the year 2082.

2. Software

2.1 HEC-RAS 5.1 Alpha

An alpha version of the Hydrologic Engineering Center's (CEIWR-HEC) River Analysis System (HEC-RAS) was used to model the complex flow of rainfall runoff within the interior and evaluate different hydraulic alternatives, such as storm gates and pumps. An alpha version was used because the latest officially released version, 5.0.7 does not have the ability to combine 2D areas with pump stations. The 5.1 version does have these features but has not been officially released. A meeting was held between PDT members from the Charleston District, South Atlantic Division (SAD), and the Modeling, Mapping, and Consequence Center (MMC) to approve the usage of the unreleased Alpha version. During PED Phase, the HEC-RAS modeling will be updated to the recently released HEC-RAS 6.0.

2.2 ESRI ArcMap 10.7

GIS software is being used to geo-reference different elements with the HEC-RAS 2D model such as the location of the 12' wall provided by the H&H team lead, Sara Brown. A LiDAR dataset has been provided by the PDT GIS team member, Jennifer Kist. This will be used as the terrain in the 2D Model.

2.3 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 damages with the goal of capturing the estimated average annual damages for each future without-project scenario and each future with-project scenario. More discussion on this in section 4.2.

3. HEC-RAS MODEL DEVELOPMENT

3.1 Original Model

The City of Charleston originally hired a contractor to perform HEC-RAS 2D modeling to assist them in 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. The terrain file used in their effort was based on the 2009 Charleston County LIDAR Data. The 2011 NLCD data was used to generate a Manning's roughness layer.

3.2 Model Revision/Development

The model used in the Calhoun West effort was revised to perform the analysis for this effort. Revisions from the original model have primarily been restructuring of the 2D mesh and separating the 2D mesh into 2 different grids to represent the interior and exterior areas connected by a SA 2D connection. The original RAS model that was provided contained a road network shapefile that was being enforced in the 2D area as breaklines. That same breakline layout is being used in this 2D effort. Breaklines have also been applied to other appropriate locations to represent raised features in the model domain.

Peninsula outfall locations have been provided in GIS shapefile format to provide locations of the outfalls. The peninsula has numerous outfall discharges that drain the sub-surface pipe network and outfalls that drain overland flow through culverts which also allow for daily tidal fluctuations in tidal creeks. However, HEC-RAS is unable to compute subsurface flow therefore the outfalls connected to the sub-surface pipe network will not be utilized, and the model will assume no pipe flow capacity. Culvert data was provided by the City of Charleston and incorporated into the model. Figure 9 displays the culverts incorporated into the modeling. These culverts were assigned as SA/2D connections with culvert openings. As shown in Figure 9, several culverts are in line with the proposed wall alignment and these culverts are assumed to be equipped with storm gates in the design of the project. Some of the culverts that are in line with the wall are also considered peninsula outfalls. These include the culvert Near Joe Riley, Gadsden Creek, Lockwood Wetland, and Newmarket Creek. Figure 73 displays a map of the peninsula outfalls. In a small number of cases, culvert dimensions and invert elevations had to be estimated or measured from Google Earth. Overall, these assumptions should have minor effects on the model results.

The exterior portion of the mesh is bounded by the Ashley River, Cooper River, and Charleston Harbor. The exterior portion of the mesh also includes areas of land that are outside of the 12' wall alignment. The east side of the city will be walled internally and not walled out in the water, therefore there will be a substantial amount of land included in the exterior mesh. The interior portion includes everything that is inside of the 12' wall alignment. The interior and exterior areas are connected by a storage area connection. This storage area connection represents the 12' wall footprint. The weir profile within the storage area connection for the future without-project condition is set to the underlying terrain. In RAS2D, "terrain" includes the topography and bathymetry. The future with-project conditions storage area connection is set to a height of 12' (NAVD88). This ensures the mesh alignment is the same for the future with-project and future without-project conditions, aside from the elevations in the storage area connection. Consistency in the geometry files allows for a better comparison in model results between with and without-project. As mentioned, the interior and exterior areas are connected by storage area connections. The connections within HEC-RAS must applied a weir coefficient to represent the hydraulic efficiency of the connection. Connections representing natural ground will have a lesser value than the connections representing an actual structure or wall. The connections representing natural ground are given a weir coefficient ranging from 0.2 to 1 depending on the elevation of the ground the connection is representing relative to typical water heights in the area. The value of natural ground weir coefficients were also decided during the iterative modeling process based on stability of the flow across the connection. The connections representing the proposed wall and the Battery are given a weir coefficient of 2 as this is typical guidance used for HEC-RAS modeling to characterize weir flow.

LiDAR provided by the South Carolina Department of Natural Resources is being utilized in this study. The figures on the following pages display the LiDAR terrain that is being used. The LiDAR is characterized as a single band raster with a 3ft x 3ft resolution that was collected in 2017. The dataset originally wasn't large 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 didn’t capture. The LIDAR was resampled to 5ft x 5ft resolution when merged with the 2009 Charleston County raster. Buildings are not included in the LiDAR and are accounted for within the landcover layer.

A sensitivity analysis was performed for a terrain file with buildings included in the mesh and the terrain being used without the buildings included. The results using the terrain with buildings displayed slightly higher water levels than the results using the terrain without the buildings. On average the increase in water level was less than an inch. The penetration of flooding into buildings is captured using the landcover layer as discussed in section 3.3.

Figure 1. HEC-RAS 2D computational mesh

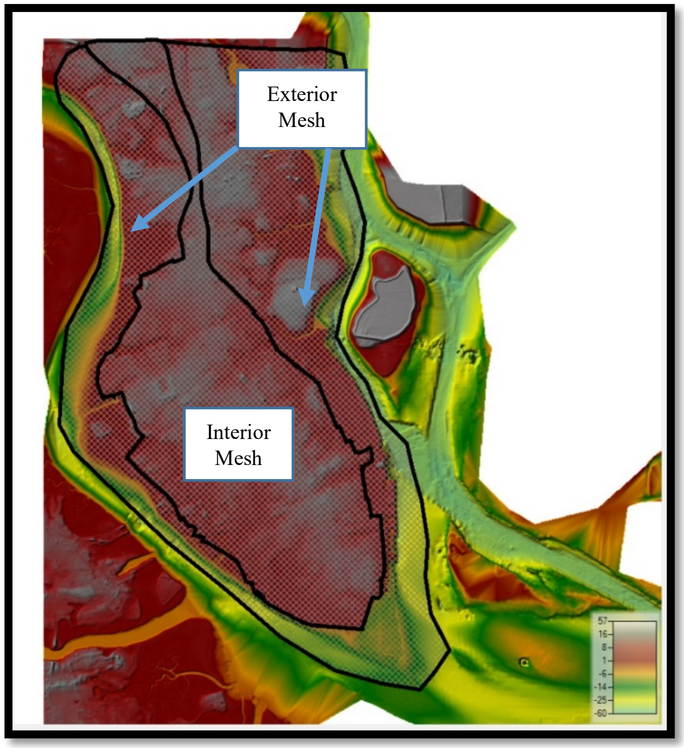


Figure 2. HEC-RAS 2D computational mesh and terrain (ft. NAVD88)

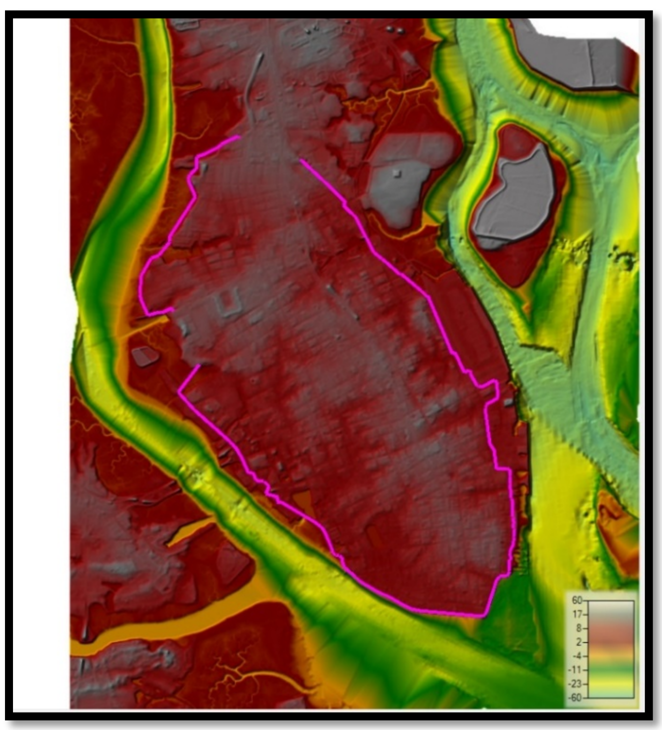


Figure 3. 12’ Wall Alignment

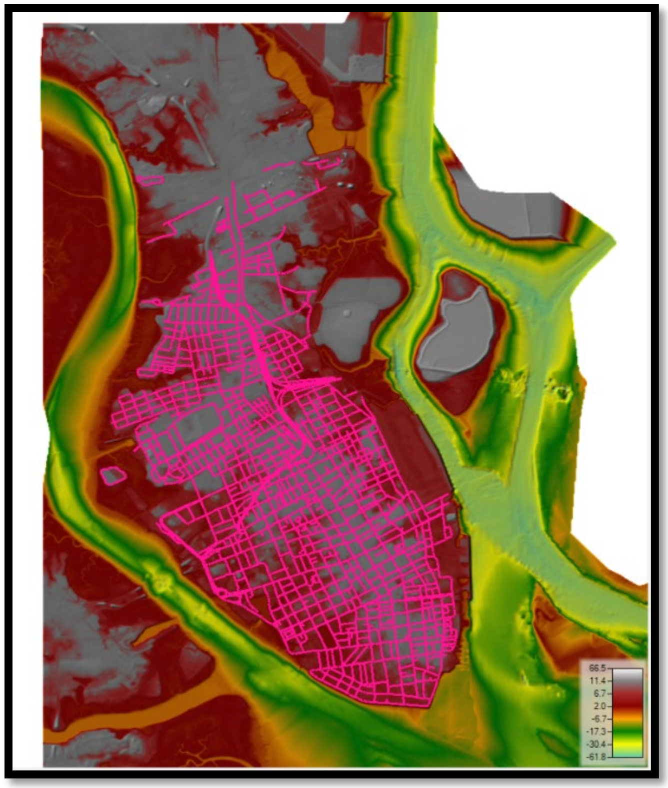
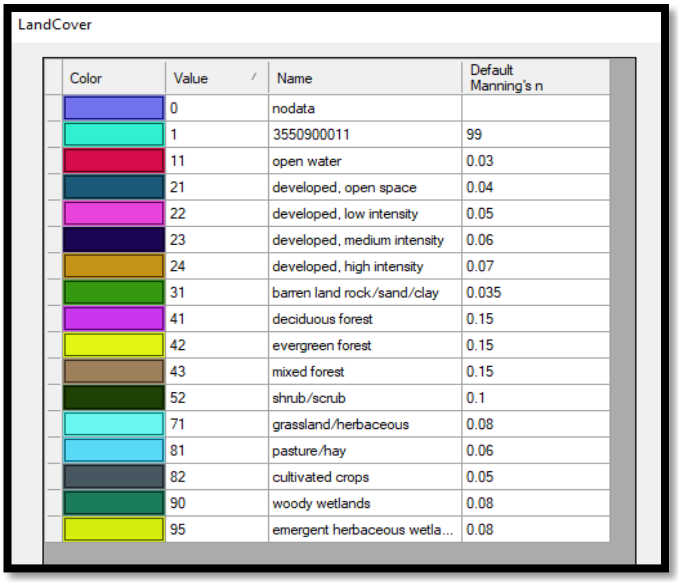


Figure 4. HEC-RAS Breaklines applied to 2D mesh

3.3 Manning's n values applied to the HEC-RAS 2D Mesh

Figure 5 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 are 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 5. Manning's n values applied to the HEC-RAS 2D model

3.4 Rain-on-Grid Precipitation Time Series Data

The City of Charleston contractor 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. The data was provided in HEC-DSSVue format with the direct runoff time series data which can be directly linked into the HEC-RAS unsteady flow files. Rainfall data was provided for the 50%, 10%, 4%, 2%, and 1% AEP rainfall events. The 20% AEP rainfall was estimated using the provided direct runoff data. 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 1.

AEP	24-HR Depth (in)	Q_{cn} (in)	Q_{cn} (rate)
50%	4.5	3.20	0.013
10%	6.5	5.11	0.021
4%	7.9	6.47	0.027
2%	9	7.55	0.031
1%	10.3	8.83	0.037

Table 1. 24-HR Rainfall Time-Series Data

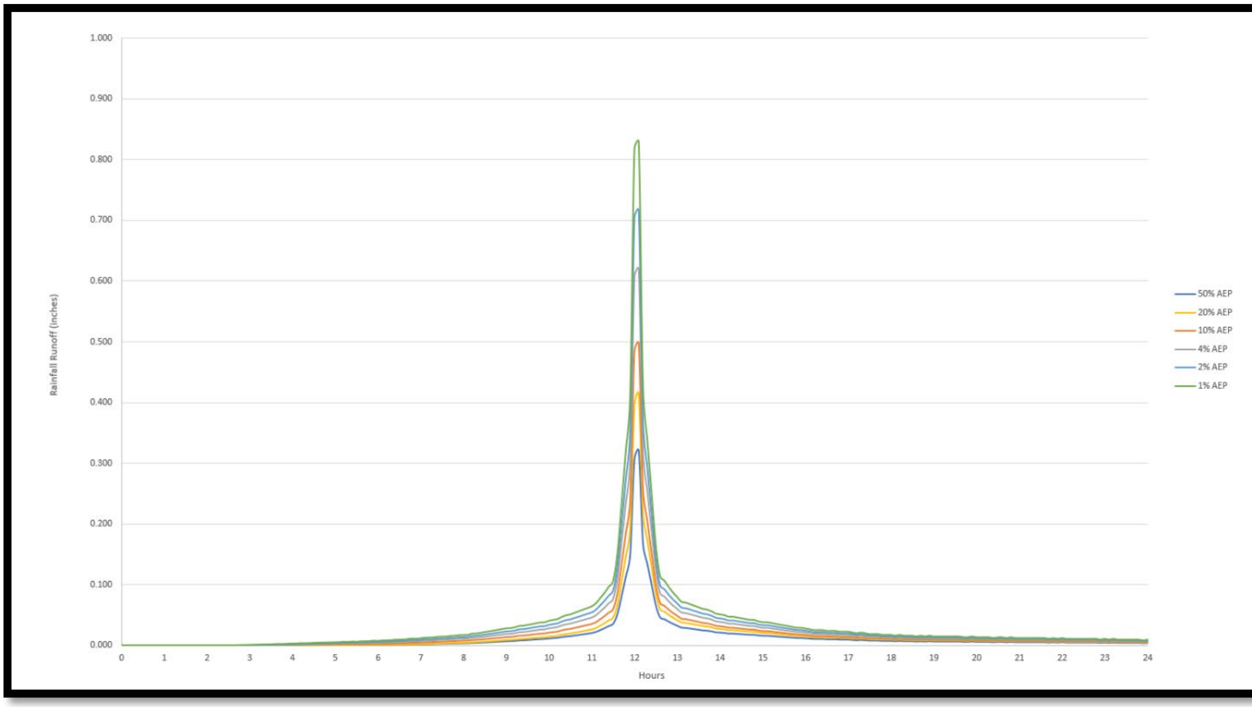


Figure 6. Direct Rainfall Runoff Time-Series Data for the 50% AEP through 1% AEP over a 24-hr period

3.5 Boundary Conditions

A stage tidal boundary condition is applied to the exterior 2D grid. This boundary condition is used to project the tidal stages into the peninsula.

- For the existing conditions with known flood event scenario, 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 analysis in the year 2032, the stage hydrograph is set to the Mean Higher High-Water surface elevation of 3.18 ft. NAVD88. Currently the MHHW is 2.62 ft., and we increase that value by the intermediate sea level rise of 0.56 ft. for the year 2032.

- For the analysis 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 rise of 1.65 feet for the year 2082.

- For the analysis 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 modeler 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 analysis can be found in section 4.5.3 of this report.

3.6 Existing Conditions with Known Flood Event

The existing conditions scenario typically serves as a model validation or calibration event, however, there is little to no available gage data or provided high water marks in Charleston to validate water levels in the interior. Verified interior water levels could be measured against the computed water levels if the data was available. However, the HEC-RAS 2D model will serve its intended purpose to estimate the hydraulic response of the overall system by analyzing the various pumping capacities and storm gates.

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 7 displays the Hurricane Irma stage hydrograph (NAVD88).

Highlighted values in Table 2 were used as exterior and interior water surface elevations within the pump and storm gate modeling analysis.

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

Table 2. Water Surface Elevations (WSEL)

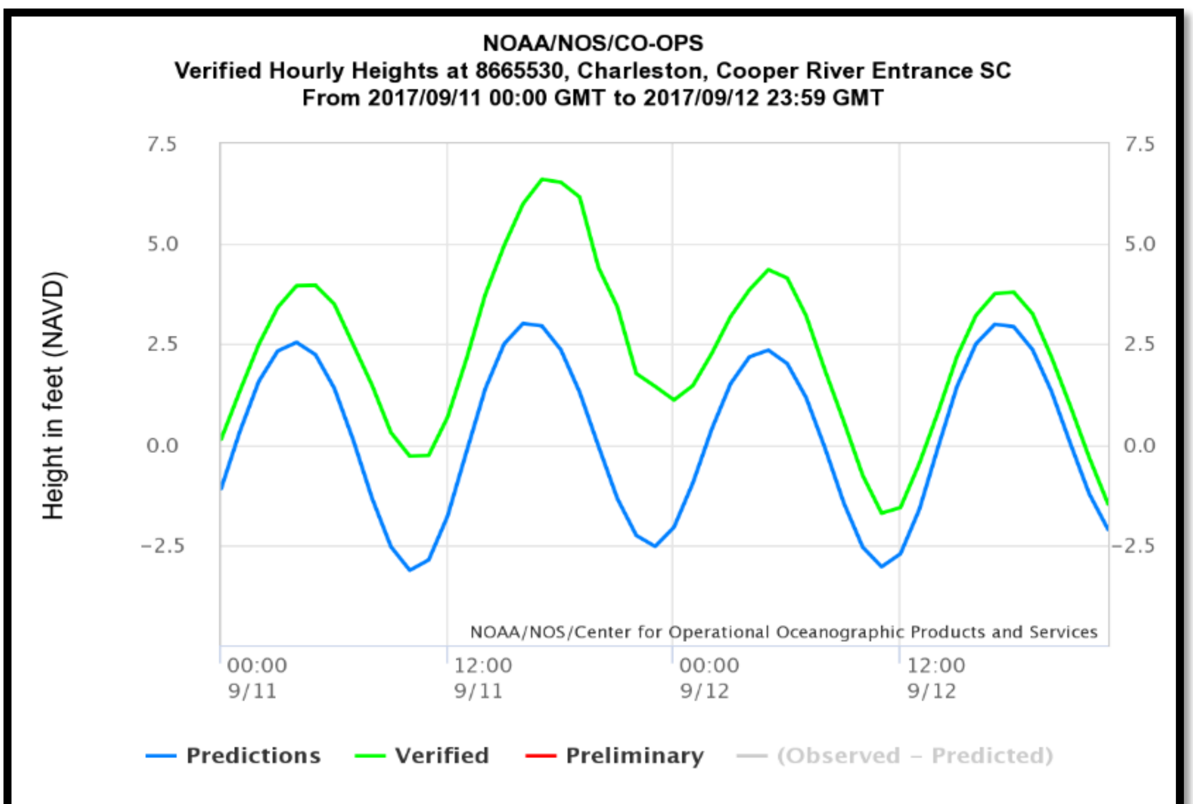


Figure 7. Charleston, Cooper River Entrance SC Tidal Gage (Hurricane Irma)

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. The figure below 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 rise value of +0.56 feet. Hurricane Irma peak water surface elevation was approximately 6.7 ft. NAVD88 so projecting this to the year 2032 assumes a peak water surface elevation of approximately 7.2 ft. NAVD88.

The purpose of Figure 8 is to provide visual representation of the potential increase in flooding for future storm events due to sea level rise. 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; therefore, the computed inundation is only a result of the stage hydrographs.

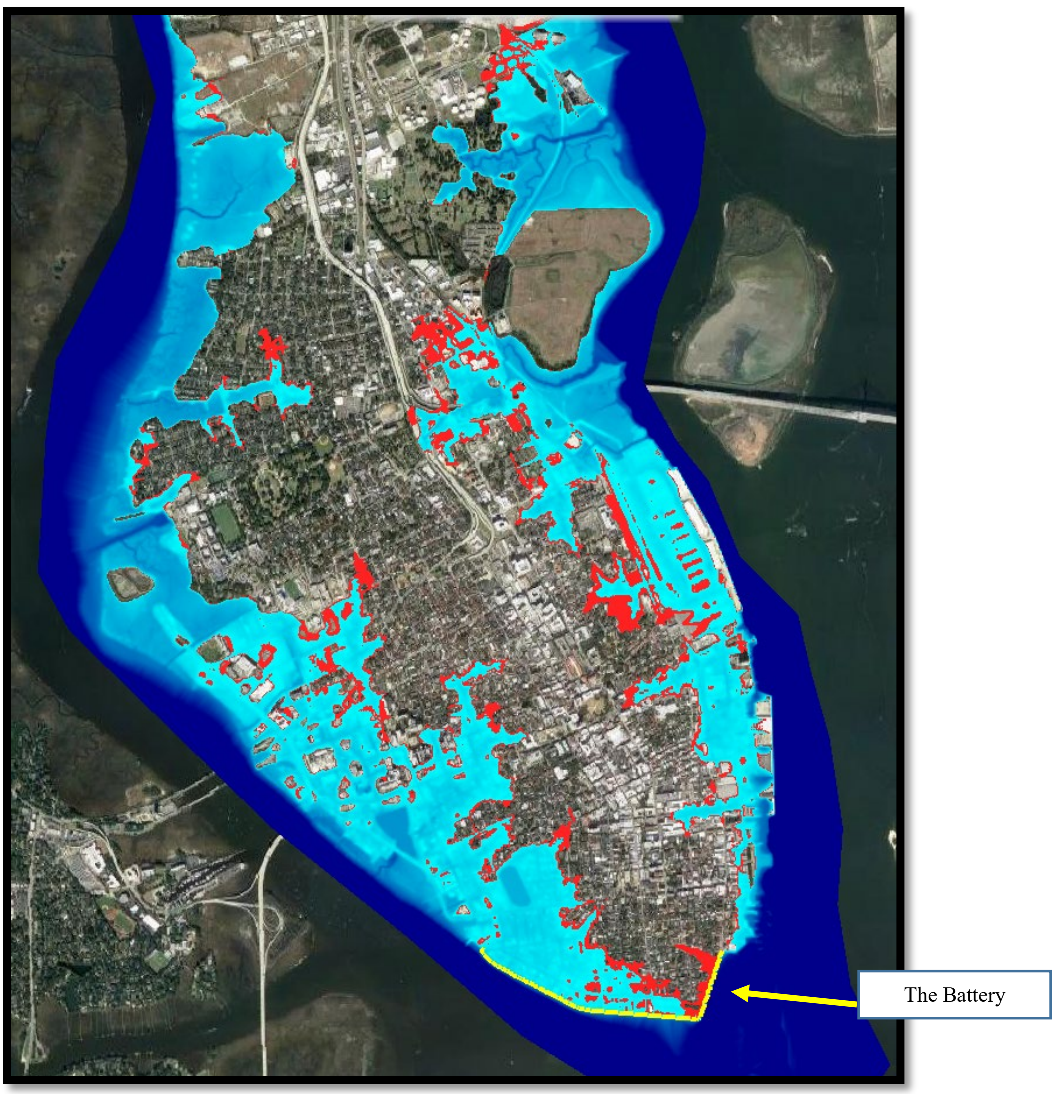


Figure 8. Hurricane Irma 2017 (Blue) vs Hurricane Irma Hypothetical 2032 (Red)

4. HEC-RAS Model Application for Pump Station and Storm Gate Alternatives

4.1 Modeling Alternatives and Assumptions

Pump stations and storm gates have been evaluated within the RAS model. Three alternatives have been evaluated for both the pump stations and storm gates. The storm gates have been placed at 15 different locations around the peninsula. The storm gates at each location have been evaluated with three alternatives of various dimensions. The storm gates will be placed in the wall at the selected sites or attached to existing culverts such as the creek behind Joe Riley Stadium, Gadsden Creek, Longpond, Lockwood Wetland, and Newmarket Creek. In supplement to storm gates, other gates have also been included in the modeling to account for the pedestrian and vehicular gates as part of the design of the wall. These gates are mostly placed at higher land elevations as opposed to the storm surge gates and should not play a significant role in the interior drainage analysis for gates open conditions.

The pump stations have been placed at ten different locations around the peninsula. Each location has been evaluated with three alternative pumping capacities. The PDT has evaluated five permanent pump stations and 5 temporary pump stations. Temporary meaning the pumps would only deploy during warranted storm events while portions of the infrastructure for the temporary pumps would permanently remain in place for quick deployment.

The system is analyzed with the goal of limiting the increase in interior stages for all rainfall events while applying a focus on the 10% AEP rainfall event for conceptual design purposes. As discussed in earlier sections of this report, the 10% AEP rainfall event is the focus because the City of Charleston drainage infrastructure passes no more than the 10% AEP event. Though the 10% AEP event is the focus of design, other storm frequencies as mentioned previously have been analyzed. The interior drainage analysis will look at the system in two different perspectives: an open system and a closed system.

i. The open system assumes non-storm conditions or typical tidal conditions; therefore, the storm gates remain open to allow daily tidal fluctuations. The rainfall applied to the 2D model will drain via gravity or overland flow through the storm gates.

ii. The closed system assumes storm conditions meaning a surge type event. In the event of a surge, the gates will be closed. However, for the purpose of properly sizing the pumps the modeling assumes the external stage condition to be at a high tide and not storm surge. More detailed information explaining this method of analysis can be found in section 4.5 of this report.
    - Pre-storm water level drawdown is assumed in the modeling, meaning the gates are closed prior to the storm surge event arriving. In the model, the interior 2D area was assigned an initial interior elevation representing a low tide of -2.4 ft. NAVD88 for the year 2032 and -1.31 ft. NAVD88 for the year 2082.

iii. Additional closed system simulations were computed to assess rainfall plus overtopping. More information will be included in remaining sections of the report.

The City of Charleston actively operates two pump stations, the Medical University of South Carolina station, and Concord Street station. There are two other pump stations currently in construction phase or design phase, which are the Spring Fishburne station and King/Huger station. The MUSC, Concord Street, and Spring Fishburne stations have been incorporated into the modeling for both the future-without and future-with conditions. The King/Huger station, which is in design phase is not included in the modeling.

The PDT pump station alternatives assumed similar or smaller capacities to that of the MUSC pump station as a starting reference. The storm gates were iteratively sized based on the height of the wall at each selected site. The storm gates were also sized based on dimensions of the existing culvert outlets that align. These culverts are highlighted with an asterisk in Table 4. The storm gate and pump alternatives can be seen in Figures 10 and 11. Also, included in the RAS geometries is a “dummy” culvert. This is labeled as CG Dummy in the 2D connection labeled Ash River 12. This dummy culvert is modeled here to allow for flow underneath a pier that is not captured within the terrain data.

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 3. Charleston Existing Culverts

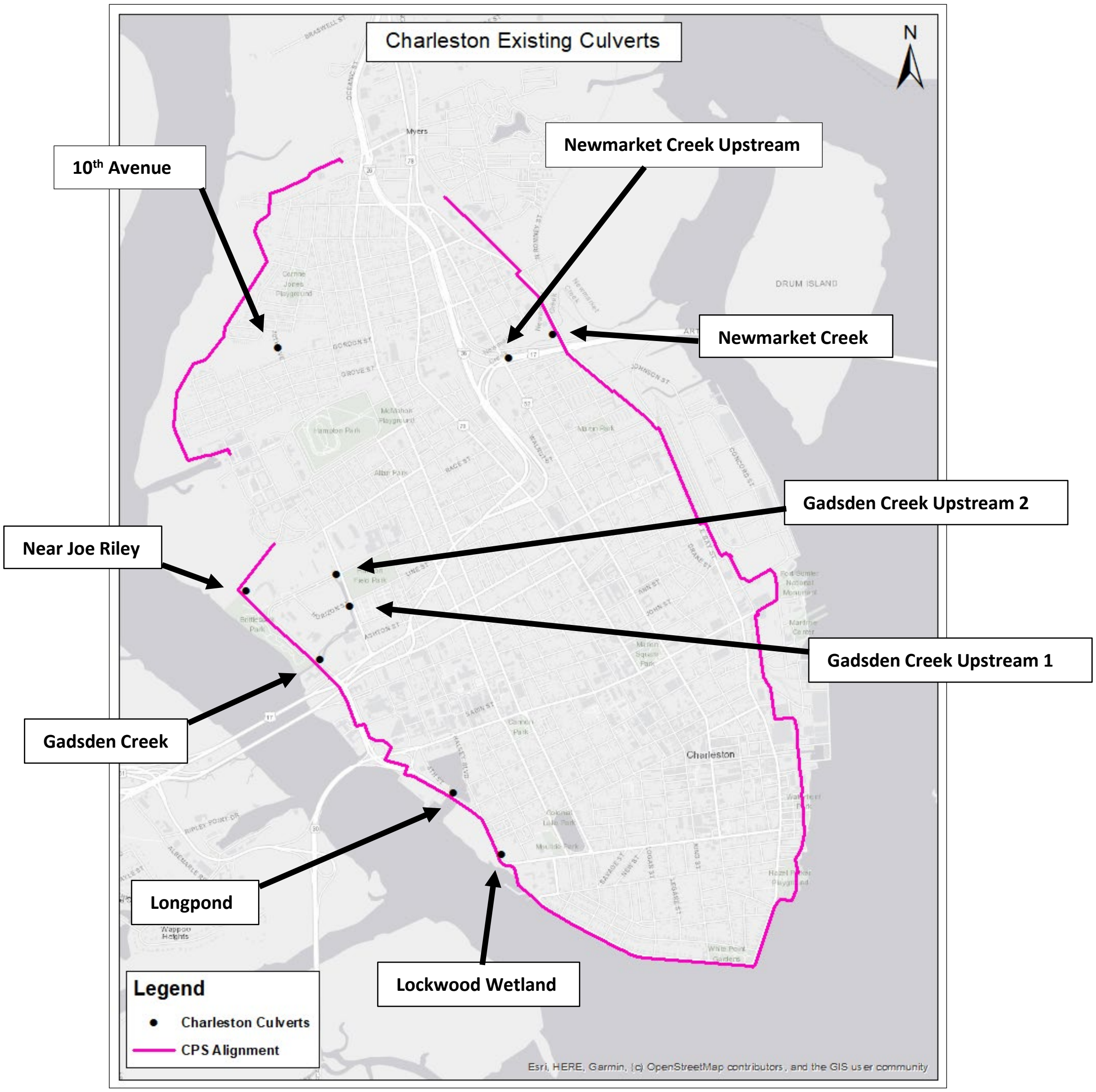


Figure 9. Charleston Culverts as modeled

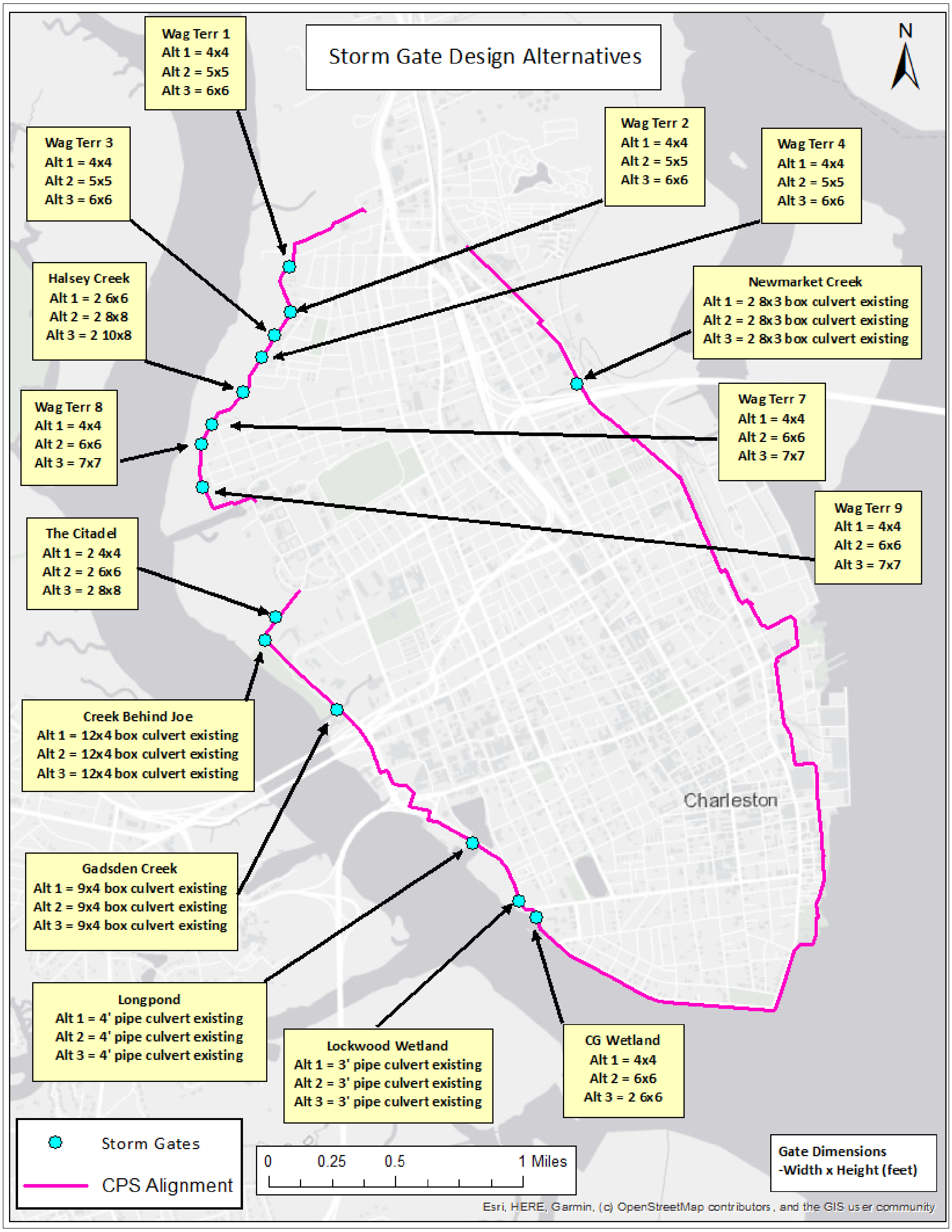


Figure 10. Storm gate alternatives

Storm Gate Location	Open System. Flow passing by gravity through storm gates/culverts or removed by City of Charleston Pump Stations.		
	Storm gate alt. 1	Storm gate alt. 2	Storm gate alt. 3
	(ft. x ft.)	(ft. x ft.)	(ft. x ft.)
Wag Terr1	1 – 4’x4’	1 – 5’x5’	1 – 6’x6’
Wag Terr 2	1 – 4’x4’	1 – 5’x5’	1 – 6’x6’
Wag Terr 3	1 – 4’x4’	1 – 5’x5’	1 – 6’x6’
Wag Terr 4	1 – 4’x4’	1 – 5’x5’	1 – 6’x6’
Halsey Creek	2 – 6’x6’	2 – 8’x8’	2 – 10’x8’
Wag Terr 7	1 – 4’x4‘	1 – 6’x6’	1 – 7’x7’
Wag Terr 8	1 – 4’x4’	1 – 6’x6’	1 – 7’x7’
Wag Terr 9	1 – 4’x4’	1 – 6’x6’	1 – 7’x7’
The Citadel	2 – 4’x4’	2 – 6’x6’	2 – 8’x8’
CG Wetland	1 – 4’x4’	1 – 6’x6’	2 – 6’x6’
*Creek Behind Joe	1 – 12’x4’ box culvert	1 – 12’x4’ box culvert	1 – 12’x4’ box culvert
*Gadsden Creek	1 – 9’x4’ box culvert	1 – 9’x4’ box culvert	1 – 9’x4’ box culvert
*Longpond	1 – 4’ circular pipe	1 – 4’ circular pipe	1 – 4’ circular pipe
*Lockwood Wetland	1 – 3’ circular pipe	1 – 3’ circular pipe	1 – 3’ circular pipe
*Newmarket Creek	2 – 8’x3’ double box culvert	2 – 8’x3’ double box culvert	2 – 8’x3’ double box culvert
Totals	18 gates	18 gates	19 gates
*Existing culverts owned by the City of Charleston. During this phase of the study, the existing culverts are assumed to remain the same dimensions for each alternative. However, the City of Charleston has stated the possibility of upsizing these culverts in the future. These culverts are assumed to be equipped with storm gates as part of this study.			

Table 4. Storm Gate Alternative Dimension

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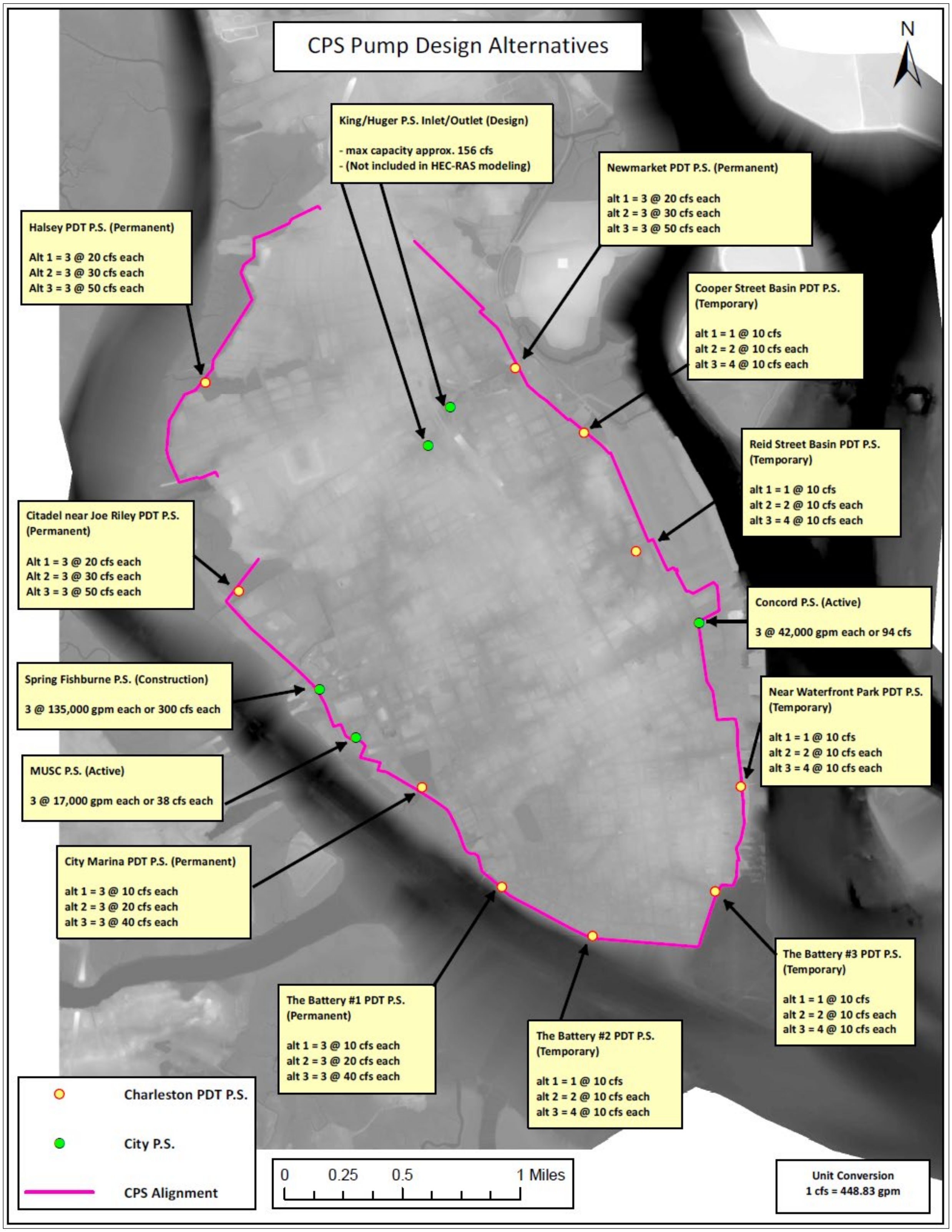

Figure 11. Pump station alternatives

Pump Location	Closed System. Water removed by PDT pumps and City of Charleston Pumps.		
	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
Reid St. Basin (T)	10	20	40
Cooper St. Basin (T)	10	20	40
Newmarket Creek (P)	60	90	150
Totals	10 pump stations	10 pump stations	10 pump stations
	290 cfs	490 cfs	890 cfs
(P) Permanent (T) Temporary			

Table 5. PDT Pump Station Alternatives

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

Table 6. City of Charleston Pump Stations

The delineation shown in Figure 12 was expanded upon based on the City of Charleston's delineation. The provided delineation can be found in the Appendix section of this report. It is assumed the City's delineation is part of their plan for future drainage basins and was split out based on their plans to modify and repair each section. The delineation in Figure 12 implemented new sub basins for informational purposes for the purpose of this study. These delineations were estimated based upon the topography and terrain files used in the hydraulic modeling. The additional basins are Halsey Creek, Newmarket Creek, Citadel, and others in the lower downtown area.

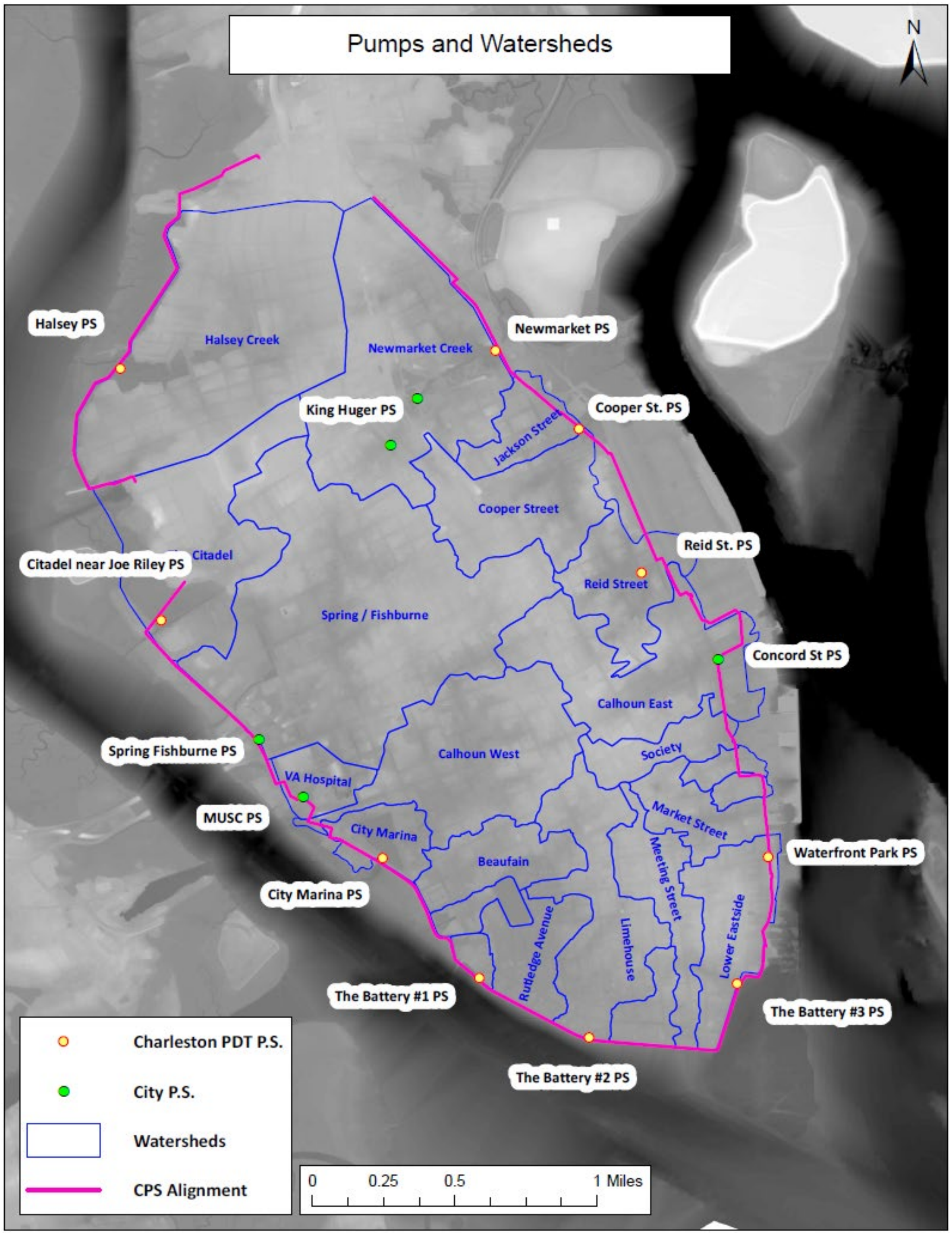

Figure 12. Pump stations and watersheds

Figure 13 displays the City's pump stations and PDT proposed pump stations to provide a visual representation of the locations of the pump stations and the layout of the storm pipe network. The storm pipe network shapefile was provided within the City's GIS catalogue.

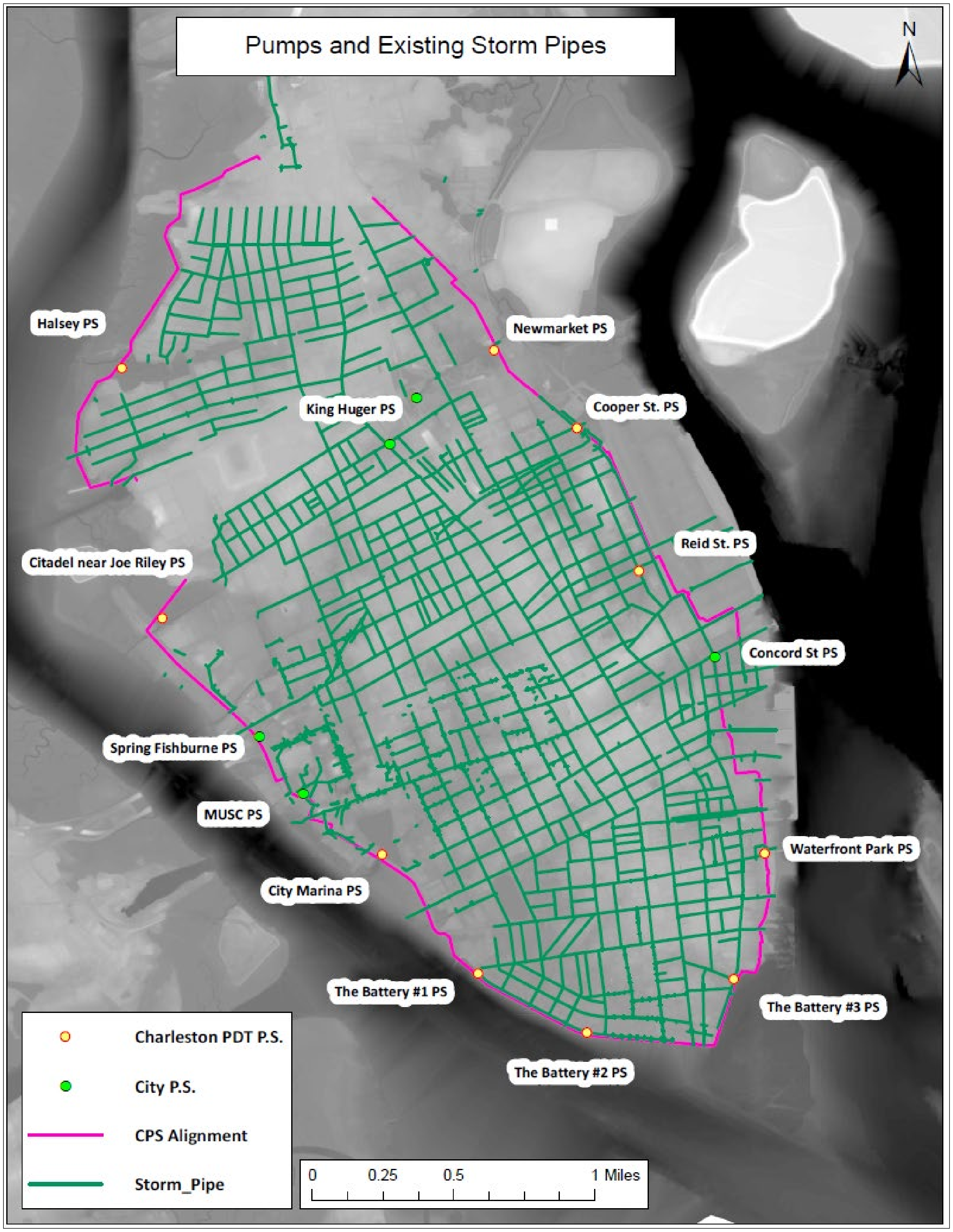

Figure 13. Pump stations and existing storm pipes

4.2 HEC-RAS Simulations for Flood Damage Analysis

HEC-RAS simulations for three storm gate alternatives and 3 pump station alternatives were computed using the 50%, 20%, 10%, 4%, 2%, and 1% AEP rainfall events combined with tidal boundary conditions. The primary focus for the hydraulic design is the 10% AEP rainfall event as mentioned in the previous section.

The following tables display the HEC-RAS scenarios that were selected for the economic analysis. Scenarios were simulated for the years 2032 and 2082, which are the end of construction and 50-yr project life. An iterative process was performed by computing numerous simulations within RAS and using those outputs as inputs for FDA. The FDA model provided expected average annual damages for each rainfall event (50% to 1% AEP) within each alternative. These average annual damages for each rainfall event within each alternative were then summed to provide the total expected average annual damage for that alternative. By comparing the total average annual damage of each future with-project alternative versus the future without-project, an informed decision can be made on the selection of the appropriate future with-project alternative.

Based on these results, storm gate alternative 3 and pump station alternative 2 are the selected alternatives to be included within the project cost estimate. Storm gate alternative 3 is the alternative equipped with the largest dimensions of storm gates at each location and this alternative provided the most comparable average annual damage estimates to the future without-project damages. However, storm gate alternative 3 does slightly increase the average annual damages and this is likely due to the entrapment of water in areas on the interior that would drain into the storm pipe network. Based on the FDA results, pump station alternative 2 shows a reduction in total average annual damages as compared to the future without-project meaning it provides net benefits. Pump alternative 3 also shows a reduction as well but a much higher reduction could lead to over-design which leads to an inflated project cost.

At this phase of the study, the alternatives have not been mixed and matched to analyze which alternative performs most efficiently at each location. The selections of storm gate alternative 3 and pump station alternative 2 have been made by observing the performance of the overall system and the total average annual damages produced by those alternatives as one functioning system. For example, all pump stations within alternative 2 may not provide adequate pumping capacity needed at each location but as a system as a whole pump alternative 2 provides that reduction in estimated average annual damages.

While the system has been analyzed holistically, the FDA model does provide damage estimates on a per model area basis. The peninsula was delineated into 5 model areas and this delineation can be found in Appendix C: Economics. Damage estimates provided per model area will assist in making informed decisions during PED phase when the alternatives are analyzed on a site-by-site basis. Therefore, these selected alternatives are subject to change during PED phase.

*******All hydrographs displayed in this report will show the x-axis of time in the year 2082. This is for modeling simplicity purposes only for keeping all model simulation times in the same year, therefore do not be confused in the following sections when a hydrograph referencing the year 2032 contains an x-axis of time in the year 2082. Keeping model times within the same time frame allows for overlaying results within the model during such an iterative effort.

Table 7 describes the RAS geometries. FWO is future without-project and FW is future with-project. Future without-project made assumptions as provided by the City of Charleston. Future with-project analyzed the system as an open and closed system. During the open system analysis, the storm gates remained open. During the closed system, one geometry was setup with the gates closed and no PDT pumps to justify having pumps to complete the system while three other geometries were setup with three different pump alternatives to analyze the needed pump capacities.

Geometry Condition	Geometry Assumptions
FWO	Future without-project assumes Low Battery is raised to 9ft NAVD88 and three City of Charleston P.S. are active. No Subsurface pipes.
FW (gates open) alt 3	Future with-project assumes Low Battery is raised, three City P.S. are active, and PDT storm gates are placed and open throughout entire simulation. All gates open. No Subsurface pipes.
FW (gates closed)	Future with-project assumes Low Battery is raised, three City P.S. are active, storm gates are closed but no PDT pumps active. All gates closed. No Subsurface pipes.
FW (gates closed) P. S. alt 1	Future with-project assumes Low Battery is raised, three City P.S. are active, storm gates are closed with P.S. alt 1 active. All gates closed. No Subsurface pipes.
FW (gates closed) P. S. alt 2	Future with-project assumes Low Battery is raised, three City P.S. are active, storm gates are closed with P.S. alt 2 active. All gates closed. No Subsurface pipes.
FW (gates closed) P. S. alt 3	Future with-project assumes Low Battery is raised, three City P.S. are active, storm gates are closed with P.S. alt 3 active. All gates closed. No Subsurface pipes.

Table 7. Geometry Conditions per Scenario

<u>Note:</u> An explanation for understanding the following scenario tables. The results of the events listed in the tables labeled FWO will be compared to each of its respective events listed in the tables labeled FW. For example, FWO at high tide with the 10% AEP rain will be compared to FW (gates open) alt 3 at high tide with the 10% AEP rain.

4.2.1 Scenarios for the year 2032 (FWO vs FW)

Each event utilizes a stage boundary condition of high tide at 3.18 feet NAVD88.

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 (gates open) alt 3		**FW (gates closed)**		**FW (gates closed) P.S. alt 1**		**FW (gates closed) P.S. alt 2**		**FW (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

4.2.2 Scenarios for the year 2082 (FWO vs FW)

Each event utilizes a stage boundary condition of high tide at 4.27 feet NAVD88.

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 (gates open) alt 3		FW (gates closed)		FW (gates closed) P.S. alt 1		FW (gates closed) P.S. alt 2		FW (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

4.2.3 Scenarios for the overtopping Analysis (FWO vs FW)

Each event utilizes a stage boundary condition of 10 feet NAVD88.

Comparison matrix for OT	
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

Comparison matrix for OT							
FW (gates closed)		FW (gates closed) P.S. alt 1		FW (gates closed) P.S. alt 2		FW (gates closed) P.S. alt 3	
Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall	Exterior Elev	Interior rainfall
Elev 10.0 NAVD88	50% AEP rain +OT	Elev 10.0 NAVD88	50% AEP rain +OT	Elev 10.0 NAVD88	50% AEP rain +OT	Elev 10.0 NAVD88	50% AEP rain +OT
Elev 10.0 NAVD88	20% AEP rain + OT	Elev 10.0 NAVD88	20% AEP rain + OT	Elev 10.0 NAVD88	20% AEP rain + OT	Elev 10.0 NAVD88	20% AEP rain + OT
Elev 10.0 NAVD88	10% AEP rain + OT	Elev 10.0 NAVD88	10% AEP rain + OT	Elev 10.0 NAVD88	10% AEP rain + OT	Elev 10.0 NAVD88	10% AEP rain + OT
Elev 10.0 NAVD88	4 % AEP rain +OT	Elev 10.0 NAVD88	4 % AEP rain +OT	Elev 10.0 NAVD88	4 % AEP rain +OT	Elev 10.0 NAVD88	4 % AEP rain +OT
Elev 10.0 NAVD88	2% AEP rain +OT	Elev 10.0 NAVD88	2% AEP rain +OT	Elev 10.0 NAVD88	2% AEP rain +OT	Elev 10.0 NAVD88	2% AEP rain +OT
Elev 10.0 NAVD88	1 % AEP rain +OT	Elev 10.0 NAVD88	1 % AEP rain +OT	Elev 10.0 NAVD88	1 % AEP rain +OT	Elev 10.0 NAVD88	1 % AEP rain +OT

4.3 Selected Output Locations

Selected output locations were used to assess the impacts to interior water levels for each storm gate alternative and pump station alternative. Peak water surface elevations were extracted from the 24 output locations which provides a general sense of the overall impacts at these various locations. Figure 14 displays the selected output locations for the RAS modeling.

Figure 14. Selected Output Locations for RAS model results

4.4 Storm Gate Alternative Evaluation

The tables in this section will display the peak water surface elevations collected from the selected output locations. The future without-project tables will only include the future without-project water surface elevations. The future with-project tables will include the with-project water surface elevations and an additional column labeled "Difference from without project condition (ft.). This additional column will show if the project alternative is increasing (+) or decreasing (-) the peak water surface elevation at the selected output locations.

At this phase of the study, storm gate alternative 3 was selected as the alternative to be calculated within the project cost estimate. The selected alternative is subject to change during PED phase. The following tables will display results for peak water surface elevations, however, to condense the amount of results displayed in this section of the report only storm gate alternative 3 will be included. Storm gate alternative 3, as seen in figure 10, contains the largest dimensional sizing of gates.

The figures in this section will display hydrographs from selected output locations. The only figures shown in this section will be hydrographs produced by the 10% AEP rainfall event for storm gate alternative 3.

4.4.1 Results for the year 2032

Selected Output Locations	FWO 2032 @ 3.18ft WSEL					
	50% AEP	20% AEP	10% AEP	4% AEP	2% AEP	1% AEP
	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)
1	3.70	3.78	3.85	3.93	4.00	4.07
2	3.19	3.19	3.19	3.19	3.19	3.19
3	3.20	3.23	3.28	3.36	3.39	3.73
4	6.15	6.27	6.41	6.58	6.70	6.82
5	3.22	3.26	3.33	3.41	3.48	3.56
6	3.51	3.84	4.42	4.95	5.20	5.43
7	4.53	4.66	4.82	4.97	5.05	5.17
8	4.29	4.75	5.00	5.20	5.31	5.40
9	3.25	3.33	3.47	4.02	4.38	4.54
10	4.84	5.05	5.20	5.38	5.51	5.64
11	4.98	5.16	5.38	5.59	5.73	5.87
12	6.18	6.36	6.56	6.78	6.94	7.11
13	6.05	6.13	6.21	6.30	6.37	6.44
14	5.35	5.50	5.63	5.78	5.88	6.02
15	4.41	4.58	4.76	5.00	5.14	5.29
16	3.68	3.93	4.30	4.97	5.46	6.01
17	5.97	6.26	6.47	6.69	6.83	6.97
18	4.49	4.79	5.11	5.46	5.66	5.84
19	5.70	5.85	6.00	6.18	6.30	6.45
20	11.07	11.17	11.26	11.34	11.39	11.44
21	7.72	7.82	7.89	7.99	8.06	8.15
22	5.86	6.05	6.17	6.32	6.42	6.54
23	5.77	5.99	6.08	6.31	6.47	6.64
24	5.21	5.98	6.26	6.58	6.75	6.91

Table 8. Peak water surface elevations for future without-project in the year 2032

Table 9 displays the peak water surface elevations for the future with-project and the difference in elevation compared to the future without-project. The 10% event has been highlighted and will be the focus of discussion.

West Side Output Locations:

Output locations 1 through 9 display slight increases in elevations due to the project, except for output location 1. Output location 1 displays an increase of greater than 1 foot and this is caused by the wall spanning across the marsh area at this location entrapping rainfall on the interior. There is currently no storm gate or pump proposed at output location 1. Locations 1 through 9 are on the west side of the peninsula and most are in low lying, tidal creeks, or marsh areas. Most of these increases are between 1-5 inches. While there are slight increases at these locations, these increases would likely impact the stages within the tidal creeks nearest the wall but cause minimal increase further away from the wall at higher land elevations. Output locations 17 through 19 on the west side of the peninsula at higher elevations show no change in water surface due to the project.

The Battery:

Output locations 10 through 12 represent the area of the Battery. Locations 10 through 12 show no change in water surface due to the project.

East Side Output Locations:

The east side of the peninsula is at naturally higher land elevations than the west side except for areas such as Newmarket Creek. The highest increase on the east side of the peninsula is 0.4 feet at output location 15 which is within the Cooper Street basin.

Selected Output Locations	FW 2032 (gates open) alt 3 @ 3.18ft WSEL											
	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	4.65	0.95	5.03	1.25	5.44	1.59	5.93	2.00	6.27	2.27	6.59	2.52
2	3.24	0.05	3.26	0.07	3.28	0.09	3.31	0.12	3.34	0.15	3.38	0.19
3	3.34	0.14	3.47	0.24	3.66	0.38	3.96	0.60	4.26	0.87	4.64	0.91
4	6.22	0.07	6.40	0.13	6.57	0.16	6.78	0.20	6.92	0.22	7.07	0.25
5	3.30	0.08	3.38	0.12	3.50	0.17	3.67	0.26	3.81	0.33	3.99	0.43
6	3.49	-0.02	3.82	-0.02	4.39	-0.03	4.93	-0.02	5.18	-0.02	5.41	-0.02
7	4.65	0.12	4.74	0.08	4.88	0.06	5.04	0.07	5.17	0.12	5.28	0.11
8	4.30	0.01	4.77	0.02	5.05	0.05	5.30	0.10	5.43	0.12	5.56	0.16
9	3.26	0.01	3.40	0.07	3.63	0.16	4.42	0.40	4.68	0.30	5.12	0.58
10	4.84	0.00	5.05	0.00	5.20	0.00	5.38	0.00	5.51	0.00	5.64	0.00
11	4.98	0.00	5.16	0.00	5.38	0.00	5.59	0.00	5.73	0.00	5.87	0.00
12	6.18	0.00	6.36	0.00	6.55	-0.01	6.78	0.00	6.94	0.00	7.10	-0.01
13	6.19	0.14	6.28	0.15	6.40	0.19	6.55	0.25	6.64	0.27	6.74	0.30
14	5.42	0.07	5.56	0.06	5.69	0.06	5.84	0.06	5.95	0.07	6.09	0.07
15	4.59	0.18	4.85	0.27	5.16	0.40	5.55	0.55	5.85	0.71	6.09	0.80
16	3.67	-0.01	3.90	-0.03	4.26	-0.04	4.94	-0.03	5.43	-0.03	6.00	-0.01
17	5.97	0.00	6.27	0.01	6.48	0.01	6.69	0.00	6.82	-0.01	6.97	0.00
18	4.49	0.00	4.79	0.00	5.11	0.00	5.47	0.01	5.66	0.00	5.84	0.00
19	5.71	0.01	5.86	0.01	6.00	0.00	6.18	0.00	6.31	0.01	6.45	0.00
20	11.07	0.00	11.16	-0.01	11.26	0.00	11.33	-0.01	11.38	-0.01	11.43	-0.01
21	7.81	0.09	7.90	0.08	7.98	0.09	8.08	0.09	8.16	0.10	8.25	0.10
22	5.76	-0.10	6.07	0.02	6.30	0.13	6.47	0.15	6.58	0.16	6.72	0.18
23	5.76	-0.01	5.91	-0.08	6.08	0.00	6.31	0.00	6.48	0.01	6.65	0.01
24	5.21	0.00	5.98	0.00	6.27	0.01	6.58	0.00	6.75	0.00	6.91	0.00

Table 9. Peak water surface elevations for future with-project (gates open) alt 3 in the year 2032

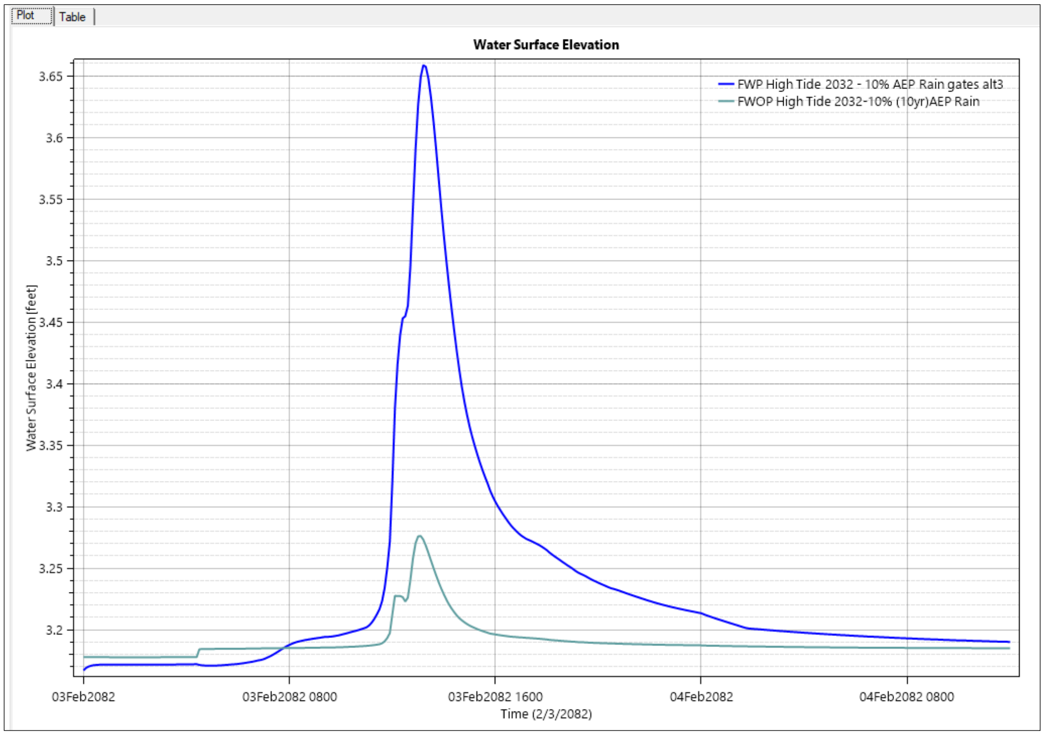

Figure 15. Storm gate hydrograph at output location #3 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

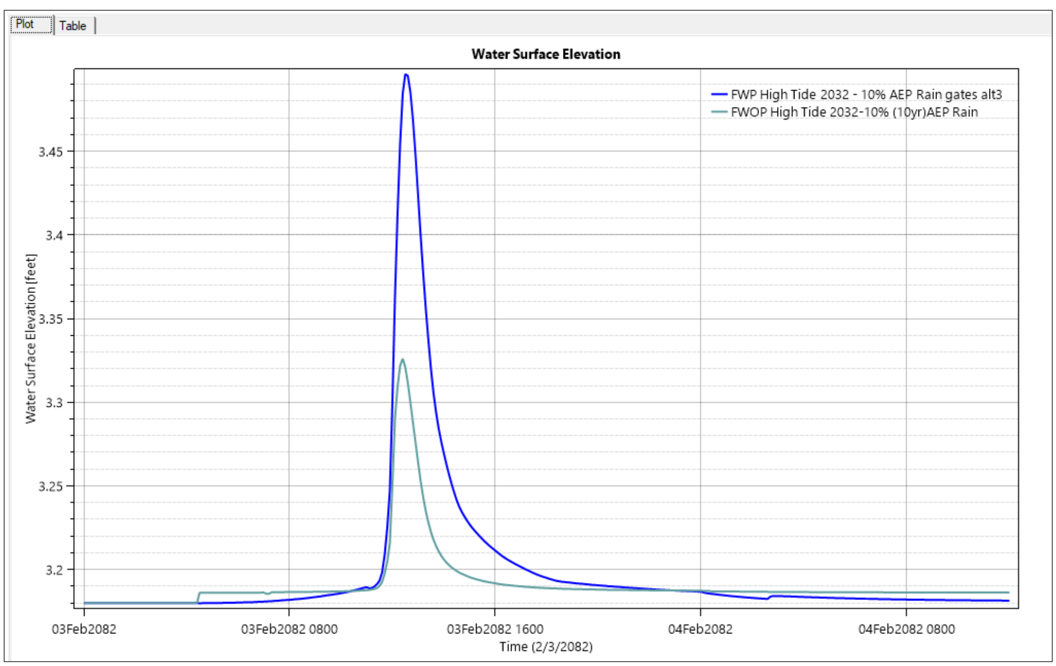

Figure 16. Storm Gate hydrograph at output location #5 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

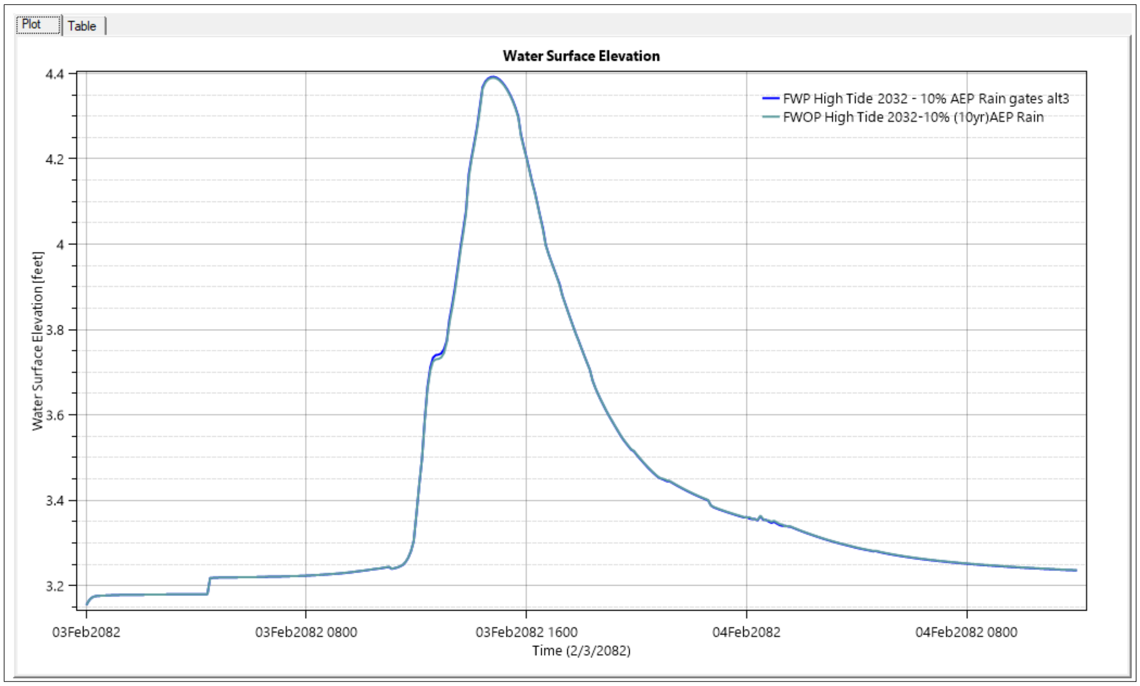

Figure 17. Storm Gate hydrograph at output location #6 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

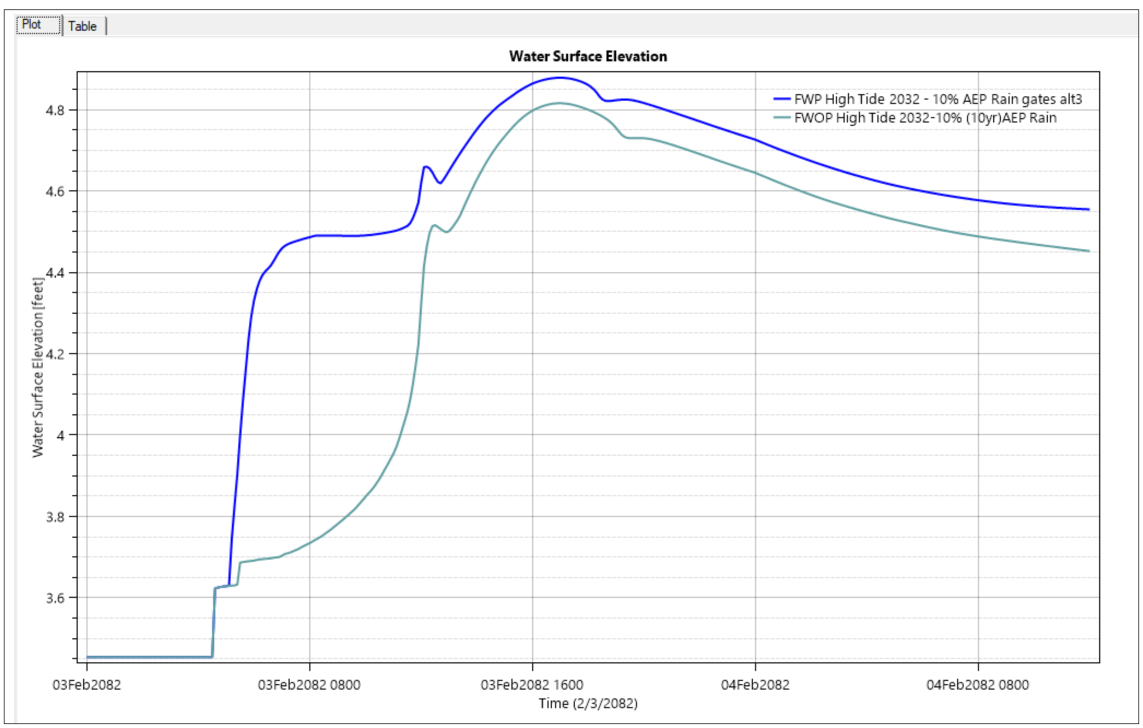

Figure 18. Storm Gate hydrograph at output location #7 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

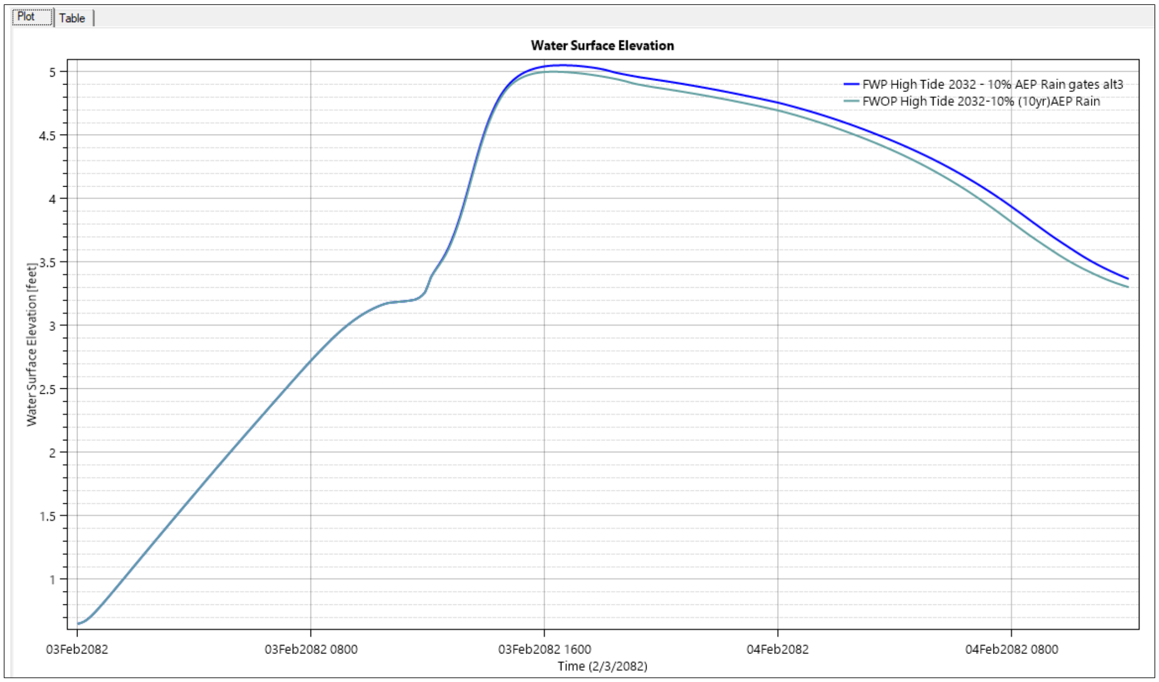

Figure 19. Storm Gate hydrograph at output location #8 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

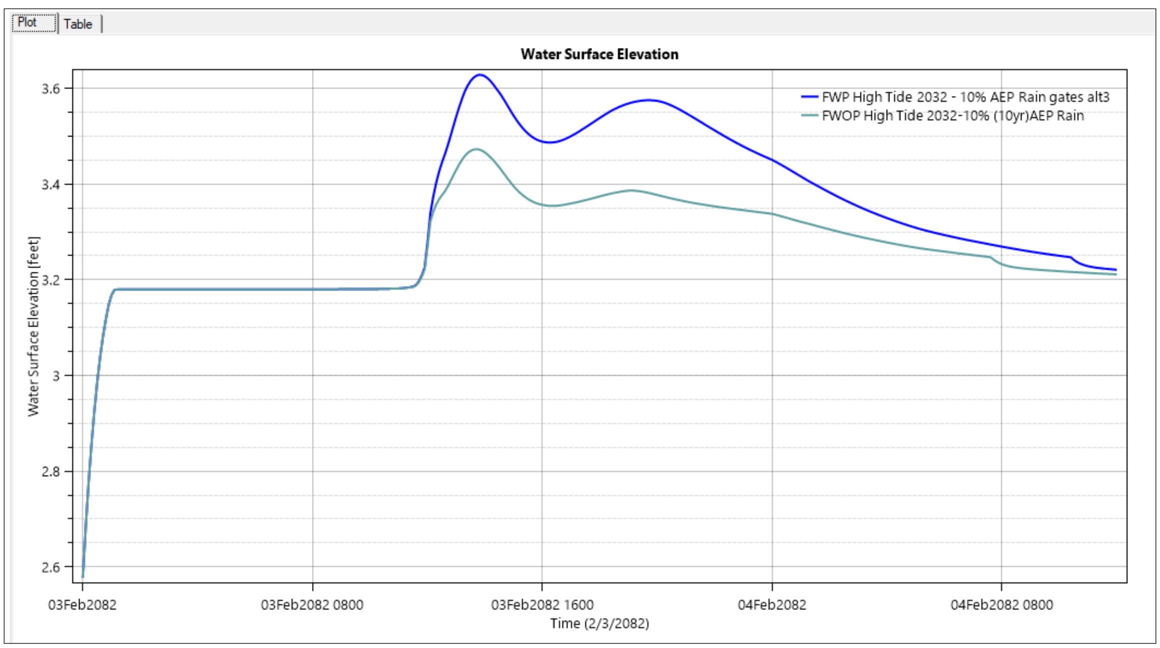

Figure 20. Storm Gate hydrograph at output location #9 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

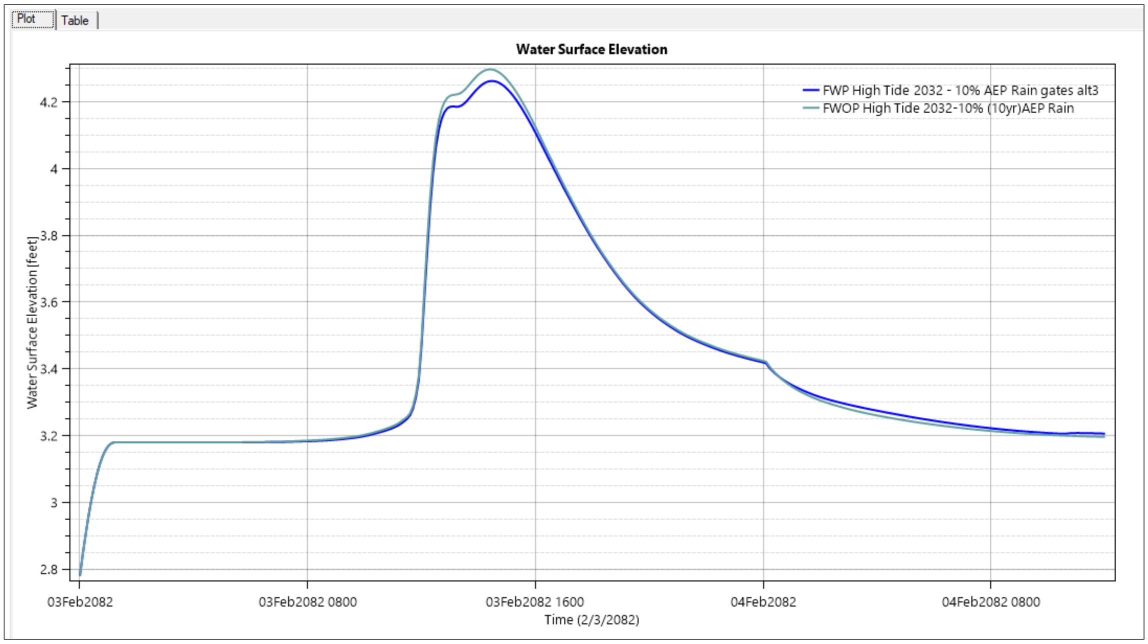

Figure 21. Storm Gate hydrograph at output location #16 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

4.4.2 Results for the year 2082

Selected Output Locations	FWO 2082 @ 4.27ft WSEL					
	50% AEP	20% AEP	10% AEP	4% AEP	2% AEP	1% AEP
	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)
1	4.30	4.32	4.34	4.37	4.39	4.43
2	4.28	4.28	4.28	4.28	4.28	4.28
3	4.27	4.30	4.31	4.35	4.38	4.42
4	6.16	6.27	6.41	6.58	6.70	6.81
5	4.29	4.30	4.32	4.34	4.36	4.40
6	4.39	4.57	4.79	5.10	5.29	5.48
7	4.58	4.73	4.87	5.00	5.09	5.20
8	4.79	4.97	5.12	5.26	5.34	5.42
9	4.30	4.32	4.37	4.45	4.53	4.61
10	4.84	5.05	5.20	5.38	5.50	5.64
11	4.98	5.16	5.38	5.59	5.73	5.86
12	6.18	6.36	6.56	6.78	6.94	7.11
13	6.08	6.17	6.25	6.34	6.40	6.47
14	5.34	5.49	5.62	5.77	5.88	6.02
15	4.46	4.61	4.77	5.00	5.15	5.31
16	4.48	4.60	4.76	5.37	5.80	6.23
17	6.06	6.31	6.50	6.69	6.82	6.96
18	4.75	4.96	5.24	5.52	5.70	5.87
19	5.71	5.85	6.01	6.18	6.31	6.45
20	11.07	11.17	11.26	11.34	11.39	11.44
21	7.72	7.81	7.89	7.98	8.05	8.13
22	5.85	6.05	6.17	6.32	6.42	6.54
23	5.76	5.91	6.08	6.31	6.70	6.64
24	5.29	6.00	6.28	6.59	6.77	6.92

Table 10. Peak water surface elevations for future without-project in the year 2082

Table 11 displays the peak water surface elevations for the future with-project and the difference in elevation compared to the future without-project. The 10% event has been highlighted and will be the focus of discussion.

<u>West Side Output Locations:</u>

Output locations 1 through 9 display slight increases in elevations due to the project. These are locations on the west side of the peninsula, and most are in low lying, tidal creeks, or marsh areas. Most of these increases are between 1-5 inches. While there are slight increases at these locations, these increases would likely impact the stages within the tidal creeks nearest the wall but cause minimal increase further away from the wall at higher elevations. Output location 1 displays an increase of greater than 1 foot and this is caused by the wall spanning across the marsh area at this location entrapping rainfall on the interior. There is currently no storm gate or pump proposed at output location 1. Output locations 17 through 19 on the west side of the peninsula at higher elevations show no change in water surface due to the project.

<u>The Battery:</u>

Output locations 10 through 12 represent the area of the Battery. Locations 10 through 12 show no change in water surface due to the project.

<u>East Side Output Locations:</u>

The east side of the peninsula is at naturally higher land elevations than the west side except for areas such as Newmarket Creek. The highest increase on the east side of the peninsula is 0.37 feet at output location 15 which is within the Cooper Street basin.

Selected Output Locations	FW 2082 (gates open) alt 3 @ 4.27ft WSEL											
	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	4.90	0.60	5.24	0.92	5.57	1.23	6.00	1.63	6.32	1.93	6.61	2.18
2	4.32	0.04	4.33	0.05	4.35	0.07	4.37	0.09	4.39	0.11	4.40	0.12
3	4.40	0.13	4.49	0.19	4.61	0.30	4.80	0.45	4.97	0.59	5.21	0.79
4	6.23	0.07	6.40	0.13	6.58	0.17	6.78	0.20	6.92	0.22	7.07	0.26
5	4.34	0.05	4.38	0.08	4.45	0.13	4.54	0.20	4.63	0.27	4.74	0.34
6	4.38	-0.01	4.57	0.00	4.80	0.01	5.10	0.00	5.29	0.00	5.48	0.00
7	4.68	0.10	4.81	0.08	4.94	0.07	5.10	0.10	5.22	0.13	5.34	0.14
8	4.80	0.01	5.01	0.04	5.20	0.08	5.39	0.13	5.49	0.15	5.61	0.19
9	4.32	0.02	4.37	0.05	4.45	0.08	4.71	0.26	4.95	0.42	5.25	0.64
10	4.84	0.00	5.05	0.00	5.20	0.00	5.38	0.00	5.51	0.01	5.65	0.01
11	4.98	0.00	5.16	0.00	5.38	0.00	5.59	0.00	5.73	0.00	5.87	0.01
12	6.18	0.00	6.36	0.00	6.55	-0.01	6.78	0.00	6.94	0.00	7.10	-0.01
13	6.19	0.11	6.28	0.11	6.40	0.15	6.55	0.21	6.64	0.24	6.73	0.26
14	5.42	0.08	5.57	0.08	5.70	0.08	5.85	0.08	5.96	0.08	6.10	0.08
15	4.57	0.11	4.83	0.22	5.14	0.37	5.54	0.54	5.80	0.65	6.08	0.77
16	4.47	-0.01	4.59	-0.01	4.75	-0.01	5.36	-0.01	5.79	-0.01	6.26	0.03
17	6.08	0.02	6.32	0.01	6.50	0.00	6.68	-0.01	6.80	-0.02	6.91	-0.05
18	4.76	0.01	4.96	0.00	5.24	0.00	5.53	0.01	5.70	0.00	5.87	0.00
19	5.70	-0.01	5.86	0.01	6.01	0.00	6.18	0.00	6.31	0.00	6.45	0.00
20	11.07	0.00	11.17	0.00	11.27	0.01	11.35	0.01	11.39	0.00	11.44	0.00
21	7.79	0.07	7.89	0.08	7.97	0.08	8.07	0.09	8.14	0.09	8.23	0.10
22	5.74	-0.11	6.08	0.03	6.30	0.13	6.47	0.15	6.58	0.16	6.72	0.18
23	5.76	0.00	5.91	0.00	6.09	0.01	6.31	0.00	6.48	-0.22	6.65	0.01
24	5.29	0.00	6.00	0.00	6.29	0.01	6.60	0.01	6.76	-0.01	6.92	0.00

Table 11. Peak water surface elevations for future with-project (gates open) alt 3 in the year 2082

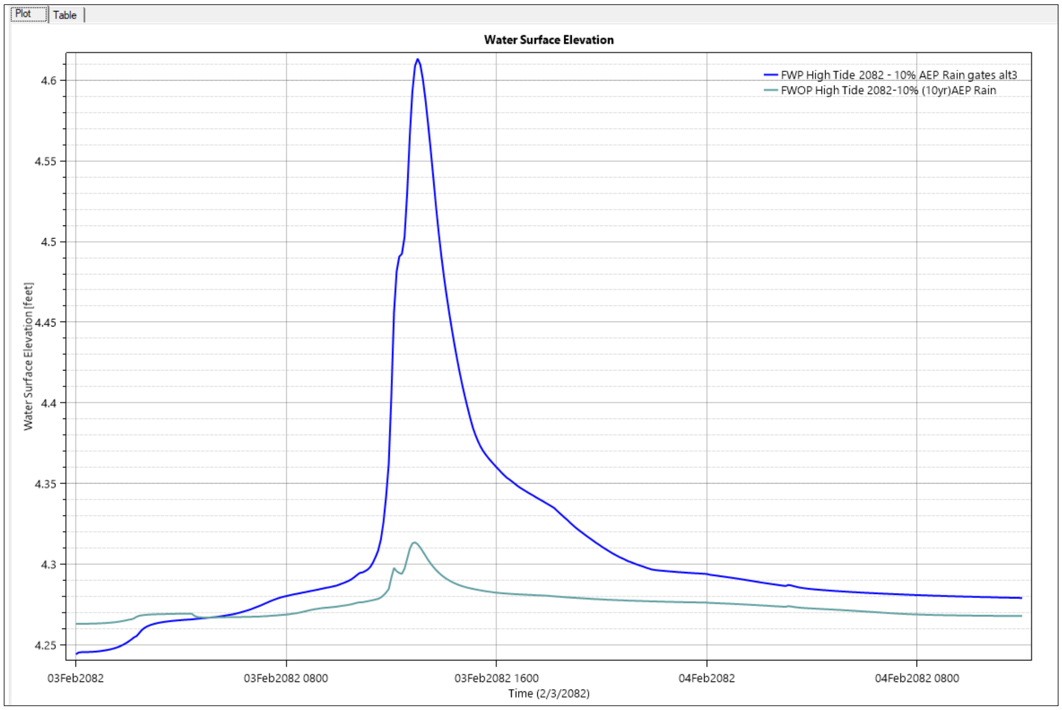

Figure 22. Storm gate hydrographs at output location #3 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

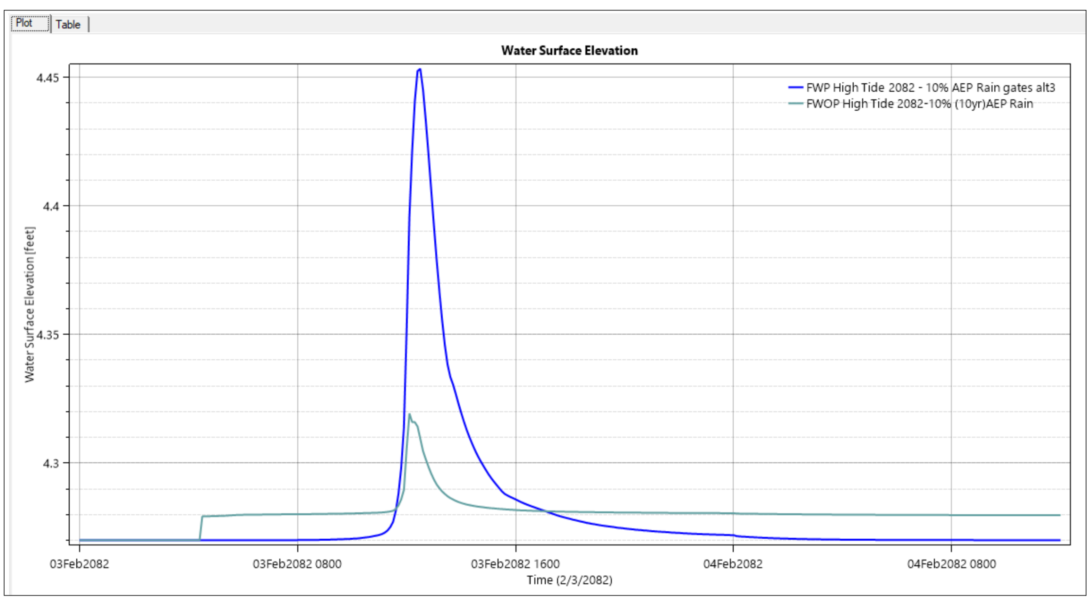

Figure 23. Storm gate hydrographs at output location #5 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

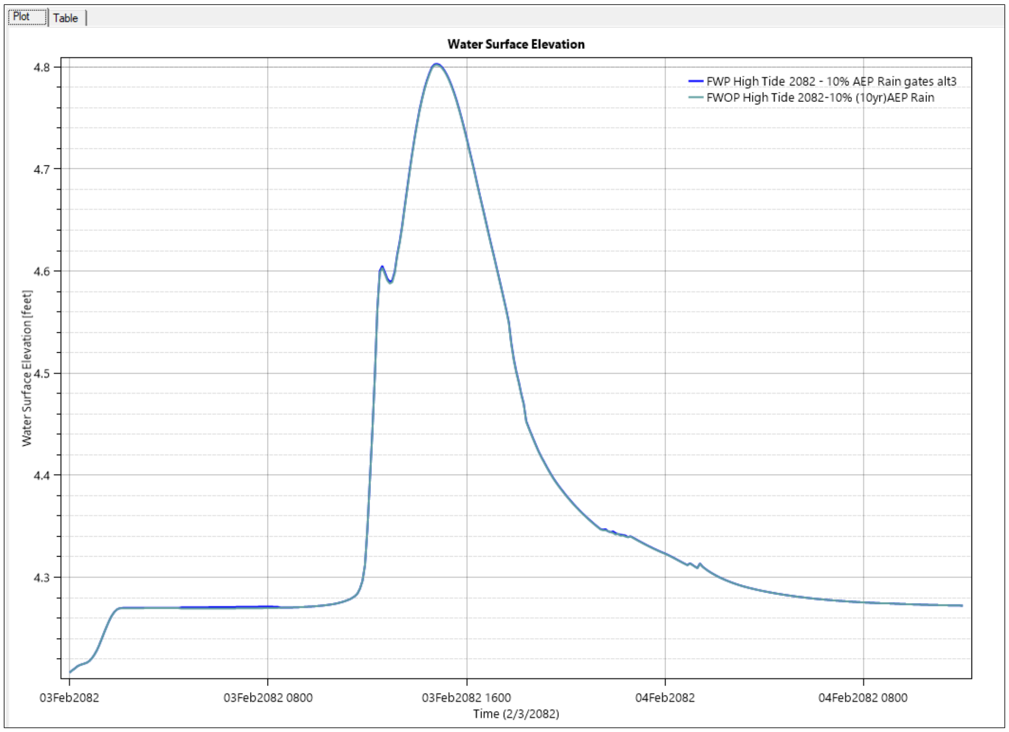

Figure 24. Storm gate hydrographs at output location #6 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

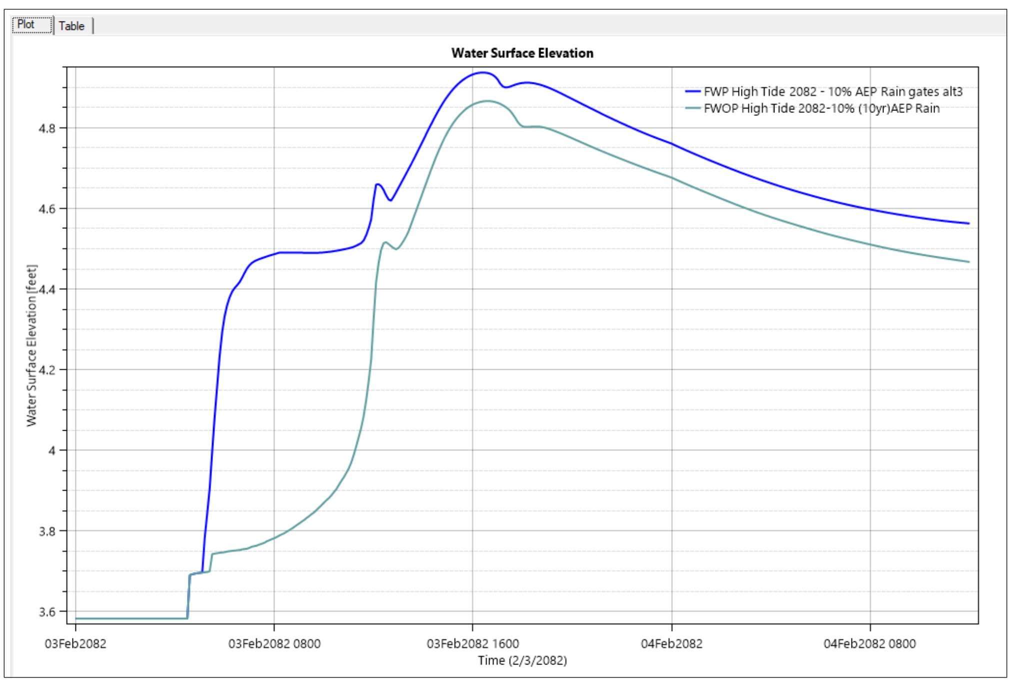

Figure 25. Storm gate hydrographs at output location #7 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

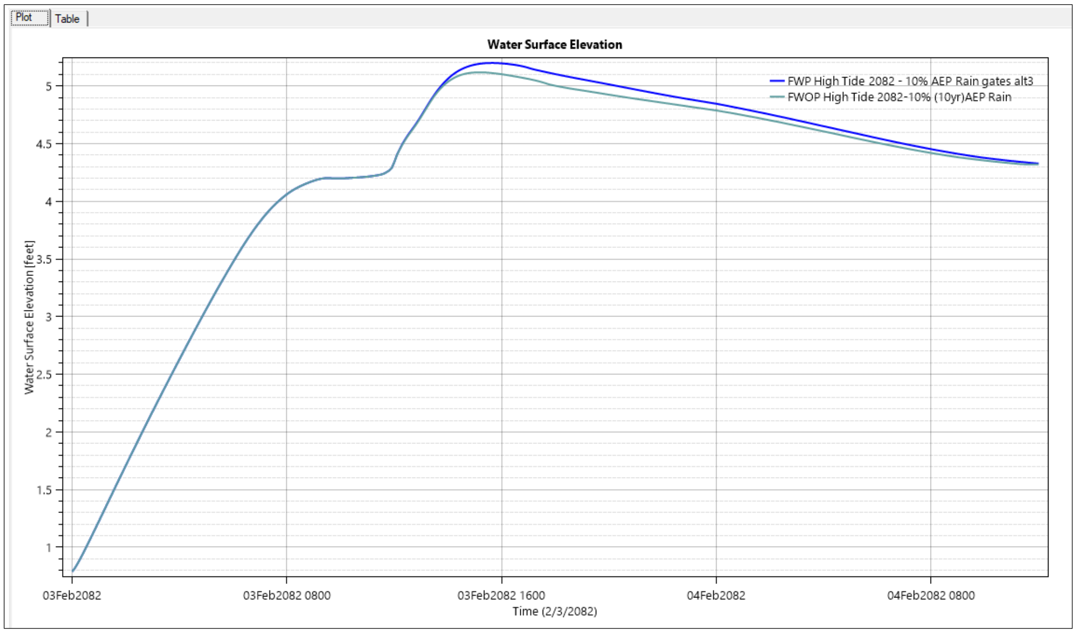

Figure 26. Storm gate hydrographs at output location #8 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

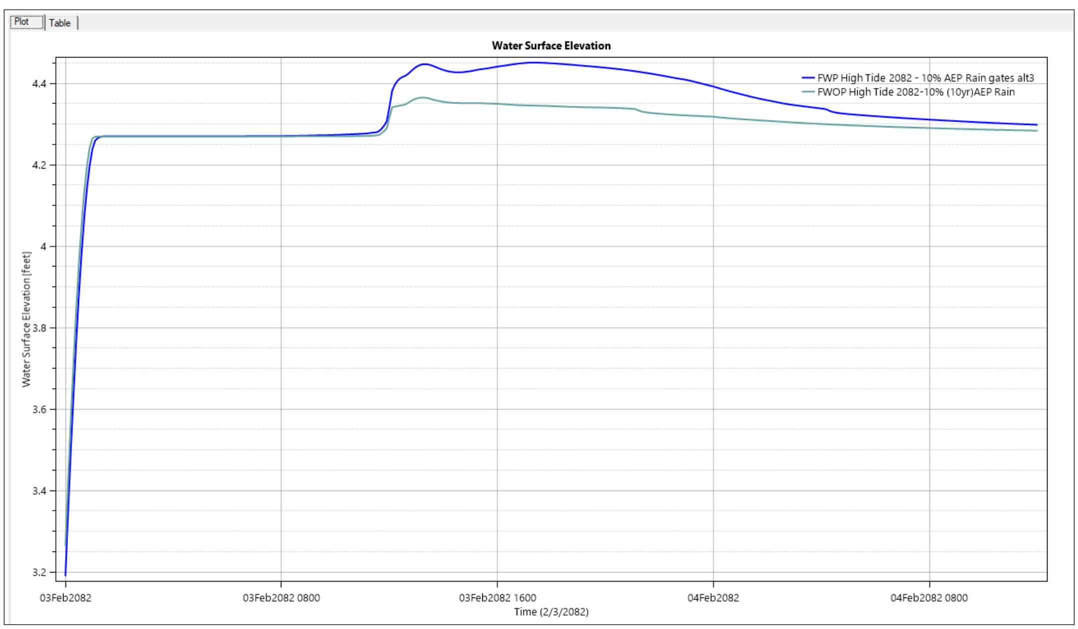

Figure 27. Storm gate hydrographs at output location #9 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

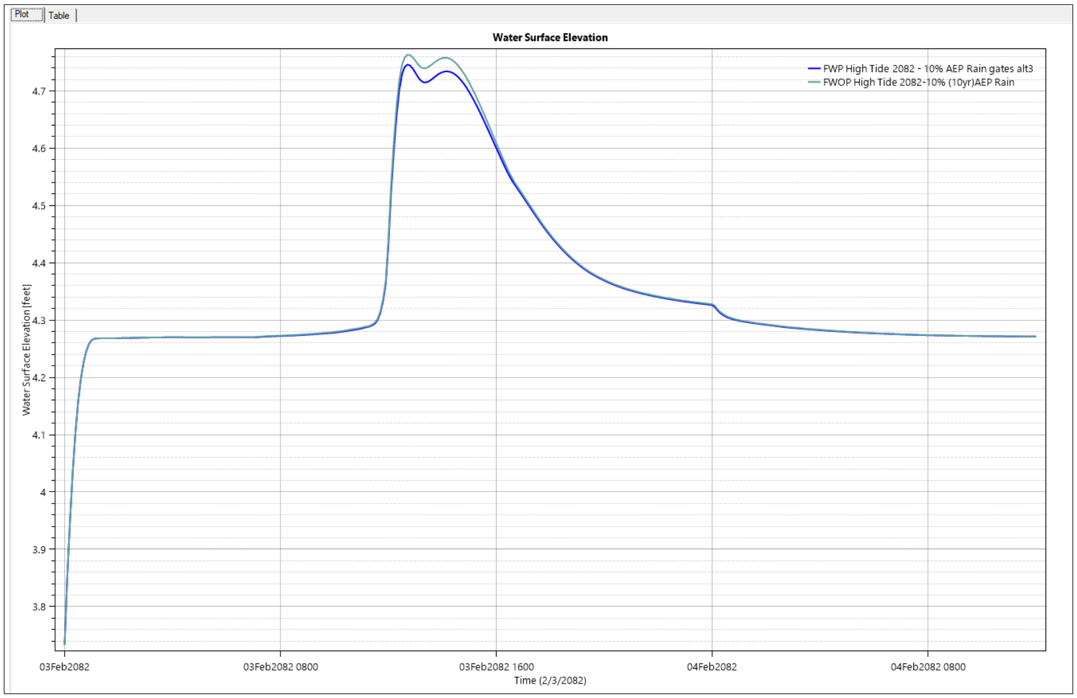

Figure 28. Storm gate hydrographs at output location #16 - FWO (Turquoise) vs FW (Blue) – 10% AEP Rainfall

4.5 Pump Station Alternative Evaluation

The tables in this section will display the peak water surface elevations collected from the selected output locations. The future with-project tables will include the with-project water surface elevations and an additional column labeled "Difference from without project condition (ft.). This additional column will show if the project alternative is increasing (+) or decreasing (-) the peak water surface elevation at the selected output locations.

The figures in this section will display hydrographs from selected output locations. The only figures shown in this section will be hydrographs produced by the 10% rainfall event.

At this phase of the study, pump station alternative 2 was selected as the alternative to be calculated within the project cost estimate. The selected alternative is subject to change during PED phase as the pumps as the pumps are analyzed on a site-by-site basis.

At this phase of the study, it is unknown the exact operability of the City's pumping operations and complex storm pipe network that collects and routes drainage to these pump stations and outfalls.

Storm gates are assumed to close at low tide prior to a forecasted storm surge event. Starting water surface elevations were applied to the interior 2D for these pumping events which were -2.4 ft. for 2032 and -1.31 ft. for 2082.

The gates closed conditions are analyzed assuming a high tide but in actual operation the gates are not planned to be closed at a typical high tide. The gates closed conditions were analyzed versus the future without conditions at high tide. In other words, the project will greatly reduce the water levels on the interior for a storm surge event up to the level of design of the wall, regardless of pump capacity. The purpose of the pumps is to remove rainfall accumulated during a storm event. Evaluating the performance of each alternative versus the future without-project condition at high tide allows the proper sizing of pumps to analyze the ponding effect.

West Side Output Locations:

Selected output locations 3 and 17 will represent the drainage area for the proposed Halsey Creek pump station. Location 5 represents the Citadel near the Joe pump station. These two pumps are in tidal creek, marshy areas. Locations 6 and 18 more so represent the drainage area for the City of Charleston's Spring Fishburne pump station. Location 7 represents the area near the City of Charleston's MUSC pump station. Location 8 represents the area of drainage at Longpond. Location 9 represents the area near the Lockwood Boulevard culvert.

The Battery Output Locations:

Locations 10, 11 and 12 represent the areas of drainage near the Battery. The Battery is proposed to have 1 permanent pump station and 2 temporary pump stations. Output location 12 incurs stability errors at lower elevations during the pump simulations due to the higher water levels receding to lower levels or dry conditions. Once the pumps are triggered to turn on, they are assumed to remain on throughout the remaining simulation time. Once the storm passes, the water levels at output location 12 go back to dry type conditions which means the pumps in the model are pumping air causing oscillations. However, this is not assumed to affect the results as this is hours after the peak water levels have receded.East Side Output Locations:

Output location 13 represents the area near the temporary pump station at Waterfront Park. Output locations 14 represents the area of drainage for the City of Charleston's Concord Street pump station. Locations 15 and 23 represent the area near the temporary Cooper Street pump stations. Location 22 represents the area near the temporary Reid Street pump station. Location 16 represents the area the near Newmarket Creek pump station.

4.5.1 Results for the year 2032

Selected Output Locations	FW 2032 (gates closed) @ 3.18ft Exterior WSEL/-2.4ft Initial Interior WSEL											
	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	7.22	3.52	7.86	4.08	8.10	4.25	8.40	4.47	8.53	4.53	8.72	4.65
2	2.62	-0.57	5.91	2.72	6.64	3.45	7.39	4.20	7.84	4.65	8.28	5.09
3	5.52	2.32	5.92	2.69	6.64	3.36	7.40	4.04	7.85	4.46	8.29	4.56
4	6.22	0.07	6.40	0.13	6.64	0.23	7.40	0.82	7.85	1.15	8.28	1.46
5	5.34	2.12	5.66	2.40	5.91	2.58	6.19	2.78	6.36	2.88	6.55	2.99
6	5.48	1.97	5.66	1.82	5.90	1.48	6.16	1.21	6.33	1.13	6.51	1.08
7	4.58	0.05	4.77	0.11	4.98	0.16	5.19	0.22	5.56	0.51	5.83	0.66
8	4.75	0.46	4.98	0.23	5.24	0.24	5.51	0.31	5.71	0.40	5.93	0.53
9	4.25	1.00	5.03	1.70	5.28	1.81	5.56	1.54	5.75	1.37	5.97	1.43
10	4.84	0.00	5.06	0.01	5.29	0.09	5.57	0.19	5.76	0.25	5.97	0.33
11	4.98	0.00	5.16	0.00	5.37	-0.01	5.58	-0.01	5.72	-0.01	5.98	0.11
12	6.18	0.00	6.35	-0.01	6.55	-0.01	6.77	-0.01	6.93	-0.01	7.09	-0.02
13	7.77	1.72	7.77	1.64	7.85	1.64	7.97	1.67	8.08	1.71	8.22	1.78
14	5.38	0.03	5.52	0.02	5.65	0.02	5.80	0.02	5.91	0.03	6.06	0.04
15	6.16	1.75	6.50	1.92	6.81	2.05	7.19	2.19	7.50	2.36	7.79	2.50
16	5.86	2.18	6.47	2.54	6.90	2.60	7.37	2.40	7.58	2.12	7.82	1.81
17	5.95	-0.02	6.25	-0.01	6.64	0.17	7.40	0.71	7.85	1.02	8.28	1.31
18	5.48	0.99	5.67	0.88	5.91	0.80	6.18	0.72	6.36	0.70	6.55	0.71
19	5.71	0.01	5.85	0.00	6.00	0.00	6.18	0.00	6.31	0.01	6.45	0.00
20	11.07	0.00	11.17	0.00	11.26	0.00	11.34	0.00	11.39	0.00	11.43	-0.01
21	7.82	0.10	7.92	0.10	8.00	0.11	8.10	0.11	8.18	0.12	8.27	0.12
22	5.74	-0.12	6.09	0.04	6.45	0.28	6.89	0.57	7.32	0.90	7.72	1.18
23	6.16	0.39	6.50	0.51	6.81	0.73	7.18	0.87	7.50	1.03	7.79	1.15
24	5.86	0.65	6.47	0.49	6.90	0.64	7.37	0.79	7.58	0.83	7.82	0.91

Table 12. Peak water surface elevations for future with-project (gates closed) No PDT Pumps in the year 2032

Selected Output Locations	FW 2032 (gates closed) P.S. alt 1 @ 3.18ft Exterior WSEL/-2.4ft Initial Interior WSEL											
	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	7.22	3.52	7.86	4.08	8.09	4.24	8.35	4.42	8.52	4.52	8.72	4.65
2	2.64	-0.55	2.83	-0.36	4.10	0.91	6.52	3.33	7.14	3.95	7.72	4.53
3	3.80	0.60	4.74	1.51	5.84	2.56	6.52	3.16	7.14	3.75	7.72	3.99
4	6.22	0.07	6.40	0.13	6.57	0.16	6.78	0.20	7.14	0.44	7.72	0.90
5	3.38	0.16	3.62	0.36	5.09	1.76	5.94	2.53	6.20	2.72	6.44	2.88
6	5.48	1.97	5.61	1.77	5.75	1.33	5.94	0.99	6.19	0.99	6.41	0.98
7	4.57	0.04	4.70	0.04	4.84	0.02	5.07	0.10	5.24	0.19	5.69	0.52
8	4.08	-0.21	4.76	0.01	5.05	0.05	5.32	0.12	5.54	0.23	5.79	0.39
9	4.11	0.86	4.61	1.28	5.00	1.53	5.33	1.31	5.56	1.18	5.81	1.27
10	4.43	-0.41	4.66	-0.39	4.95	-0.25	5.33	-0.05	5.57	0.06	5.81	0.17
11	4.69	-0.29	4.85	-0.31	5.11	-0.27	5.44	-0.15	5.60	-0.13	5.82	-0.05
12	5.98	-0.20	6.21	-0.15	6.42	-0.14	6.66	-0.12	6.82	-0.12	7.00	-0.11
13	7.65	1.60	7.68	1.55	7.76	1.55	7.87	1.57	7.99	1.62	8.14	1.70
14	5.38	0.03	5.53	0.03	5.66	0.03	5.81	0.03	5.92	0.04	6.07	0.05
15	5.72	1.31	6.12	1.54	6.47	1.71	6.89	1.89	7.18	2.04	7.44	2.15
16	3.42	-0.26	4.13	0.20	5.53	1.23	6.53	1.56	6.94	1.48	7.32	1.31
17	5.95	-0.02	6.25	-0.01	6.47	0.00	6.68	-0.01	7.14	0.31	7.71	0.74
18	5.48	0.99	5.61	0.82	5.75	0.64	5.95	0.49	6.20	0.54	6.44	0.60
19	5.71	0.01	5.85	0.00	6.00	0.00	6.18	0.00	6.31	0.01	6.45	0.00
20	11.07	0.00	11.16	-0.01	11.26	0.00	11.33	-0.01	11.38	-0.01	11.43	-0.01
21	7.80	0.08	7.90	0.08	7.98	0.09	8.08	0.09	8.16	0.10	8.25	0.10
22	5.75	-0.11	5.84	-0.21	6.00	-0.17	6.48	0.16	6.80	0.38	7.20	0.66
23	5.76	-0.01	6.12	0.13	6.47	0.39	6.89	0.58	7.18	0.71	7.44	0.80
24	5.19	-0.02	5.97	-0.01	6.27	0.01	6.58	0.00	6.94	0.19	7.32	0.41

Table 13. Peak water surface elevations for future with-project (gates closed) P.S. alt 1 in the year 2032

Selected Output Locations	FW 2032 (gates closed) P.S. alt 2 @ 3.18ft Exterior WSEL/-2.4ft Initial Interior WSEL											
	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	7.23	3.53	7.86	4.08	8.09	4.24	8.34	4.41	8.52	4.52	8.72	4.65
2	2.67	-0.52	2.81	-0.38	3.46	0.27	6.22	3.03	6.88	3.69	7.50	4.31
3	3.46	0.26	4.21	0.98	5.45	2.17	6.22	2.86	6.87	3.48	7.50	3.77
4	6.22	0.07	6.40	0.13	6.57	0.16	6.78	0.20	6.92	0.22	7.50	0.68
5	3.37	0.15	3.43	0.17	4.21	0.88	5.73	2.32	6.10	2.62	6.38	2.82
6	5.48	1.97	5.61	1.77	5.75	1.33	5.91	0.96	6.10	0.90	6.35	0.92
7	4.57	0.04	4.70	0.04	4.82	0.00	5.01	0.04	5.18	0.13	5.56	0.39
8	3.50	-0.79	4.36	-0.39	4.92	-0.08	5.23	0.03	5.44	0.13	5.68	0.28
9	4.11	0.86	4.56	1.23	4.83	1.36	5.18	1.16	5.41	1.03	5.68	1.14
10	4.41	-0.43	4.58	-0.47	4.76	-0.44	5.11	-0.27	5.40	-0.11	5.68	0.04
11	4.63	-0.35	4.70	-0.46	4.81	-0.57	5.27	-0.32	5.49	-0.24	5.69	-0.18
12	5.69	-0.49	6.05	-0.31	6.31	-0.25	6.57	-0.21	6.74	-0.20	6.92	-0.19
13	7.47	1.42	7.53	1.40	7.64	1.43	7.78	1.48	7.90	1.53	8.07	1.63
14	5.43	0.08	5.57	0.07	5.71	0.08	5.86	0.08	5.97	0.09	6.11	0.09
15	4.43	0.02	5.89	1.31	6.26	1.50	6.69	1.69	6.99	1.85	7.30	2.01
16	3.40	-0.28	3.51	-0.42	4.86	0.56	6.17	1.20	6.72	1.26	7.13	1.12
17	5.95	-0.02	6.25	-0.01	6.47	0.00	6.68	-0.01	6.88	0.05	7.50	0.53
18	5.48	0.99	5.62	0.83	5.75	0.64	5.91	0.45	6.11	0.45	6.38	0.54
19	5.71	0.01	5.85	0.00	6.01	0.01	6.18	0.00	6.31	0.01	6.45	0.00
20	11.06	-0.01	11.16	-0.01	11.25	-0.01	11.33	-0.01	11.38	-0.01	11.43	-0.01
21	7.73	0.01	7.82	0.00	7.90	0.01	7.99	0.00	8.07	0.01	8.15	0.00
22	5.74	-0.12	5.84	-0.21	5.93	-0.24	6.22	-0.10	6.54	0.12	6.91	0.37
23	5.76	-0.01	5.92	-0.07	6.27	0.19	6.69	0.38	6.99	0.52	7.30	0.66
24	5.19	-0.02	5.97	-0.01	6.27	0.01	6.58	0.00	6.76	0.01	7.13	0.22

Table 14. Peak water surface elevations for future with-project (gates closed) P.S. alt 2 in the year 2032

Selected Output Locations	FW 2032 (gates closed) P.S. alt 3 @ 3.18ft Exterior WSEL/-2.4ft Initial Interior WSEL											
	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	7.22	3.52	7.86	4.08	8.09	4.24	8.33	4.40	8.51	4.51	8.72	4.65
2	2.61	-0.58	2.81	-0.38	3.46	0.27	5.65	2.46	6.48	3.29	7.16	3.97
3	3.39	0.19	3.64	0.41	4.60	1.32	6.00	2.64	6.48	3.09	7.16	3.43
4	6.22	0.07	6.40	0.13	6.57	0.16	6.78	0.20	6.92	0.22	7.16	0.34
5	3.36	0.14	3.39	0.13	3.53	0.20	4.71	1.30	5.74	2.26	6.21	2.65
6	5.48	1.97	5.61	1.77	5.75	1.33	5.91	0.96	6.03	0.83	6.23	0.80
7	4.57	0.04	4.70	0.04	4.81	-0.01	4.96	-0.01	5.09	0.04	5.29	0.12
8	3.39	-0.90	3.58	-1.17	4.44	-0.56	5.06	-0.14	5.30	-0.01	5.53	0.13
9	4.11	0.86	4.56	1.23	4.72	1.25	5.01	0.99	5.22	0.84	5.47	0.93
10	4.41	-0.43	4.57	-0.48	4.72	-0.48	4.89	-0.49	5.08	-0.43	5.42	-0.22
11	4.61	-0.37	4.68	-0.48	4.73	-0.65	4.83	-0.76	5.19	-0.54	5.50	-0.37
12	4.91	-1.27	5.58	-0.78	6.05	-0.51	6.38	-0.40	6.57	-0.37	6.77	-0.34
13	5.12	-0.93	6.89	0.76	7.33	1.12	7.59	1.29	7.75	1.38	7.93	1.49
14	5.37	0.02	5.53	0.03	5.66	0.03	5.81	0.03	5.91	0.03	6.07	0.05
15	4.36	-0.05	4.59	0.01	5.98	1.22	6.45	1.45	6.76	1.62	7.08	1.79
16	3.38	-0.30	3.43	-0.50	3.58	-0.72	5.34	0.37	6.20	0.74	6.79	0.78
17	5.95	-0.02	6.25	-0.01	6.47	0.00	6.68	-0.01	6.82	-0.01	7.16	0.19
18	5.48	0.99	5.61	0.82	5.75	0.64	5.91	0.45	6.03	0.37	6.24	0.40
19	5.71	0.01	5.85	0.00	6.00	0.00	6.18	0.00	6.30	0.00	6.44	-0.01
20	11.06	-0.01	11.16	-0.01	11.25	-0.01	11.33	-0.01	11.38	-0.01	11.43	-0.01
21	7.83	0.11	7.93	0.11	8.01	0.12	8.11	0.12	8.19	0.13	8.28	0.13
22	5.74	-0.12	5.84	-0.21	5.93	-0.24	6.06	-0.26	6.24	-0.18	6.59	0.05
23	5.76	-0.01	5.91	-0.08	6.09	0.01	6.46	0.15	6.76	0.29	7.08	0.44
24	5.19	-0.02	5.97	-0.01	6.27	0.01	6.58	0.00	6.75	0.00	6.91	0.00

Table 15. Peak water surface elevations for future with-project (gates closed) P.S. alt 3 in the year 2032

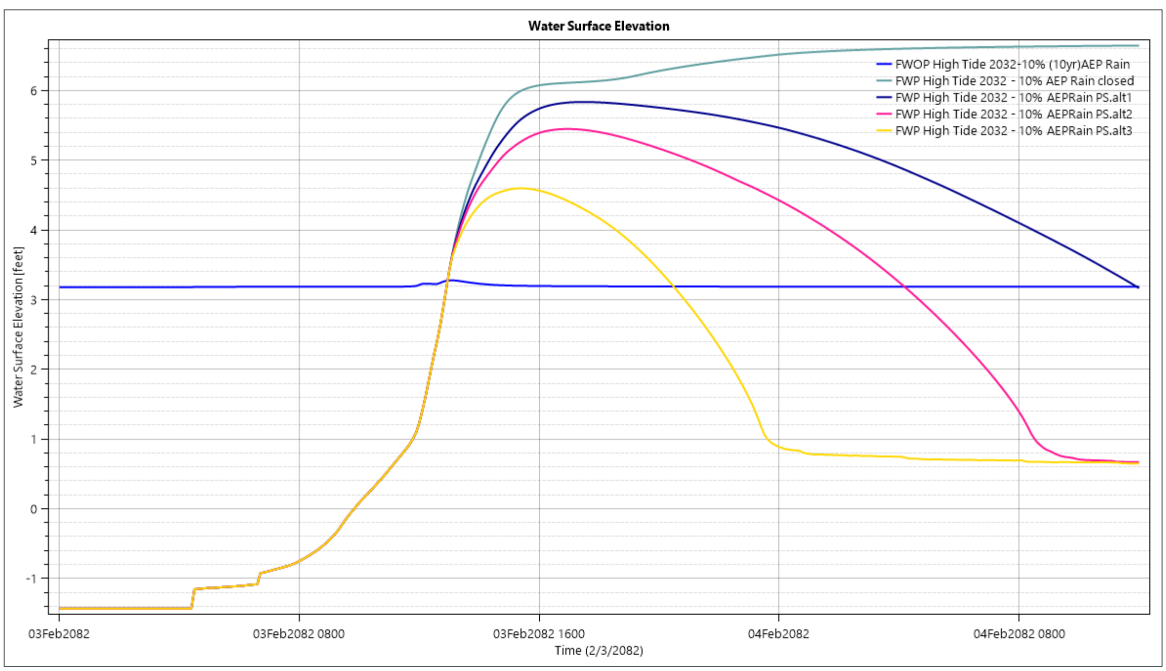

Figure 29. Hydrographs at output location #3 for pump station alternatives in the year 2032

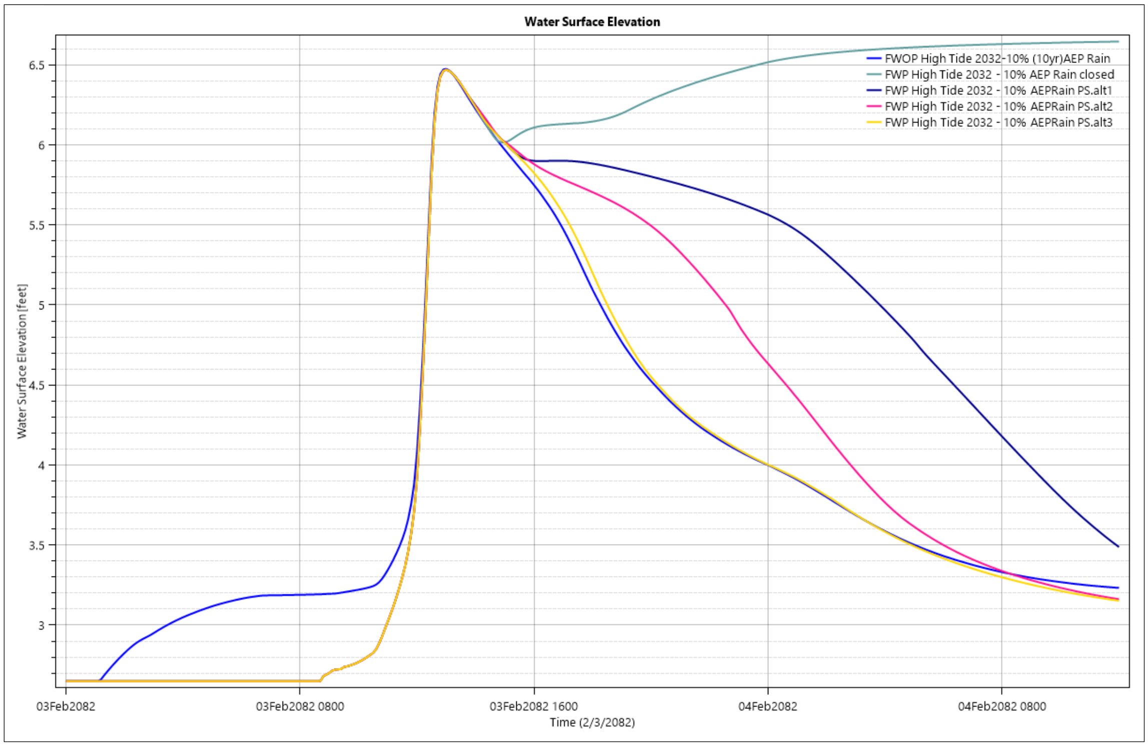

Figure 30. Hydrographs at output location #17 for pump station alternatives in the year 2032

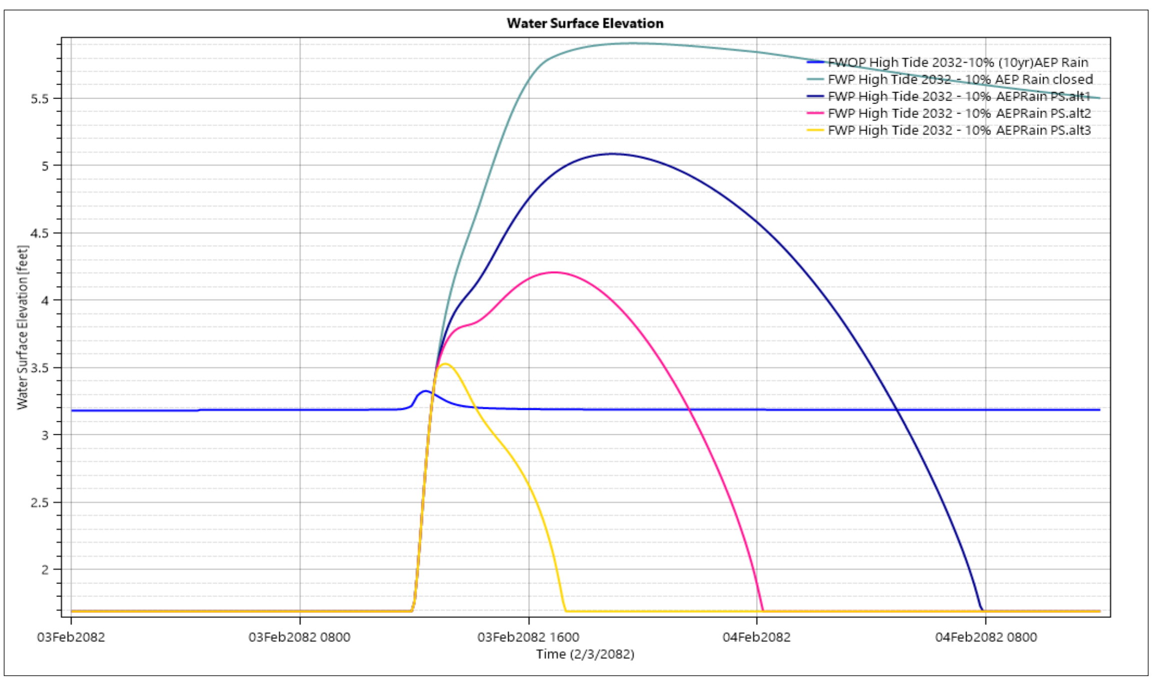

Figure 31. Hydrographs at output location #5 for pump station alternatives in the year 2032

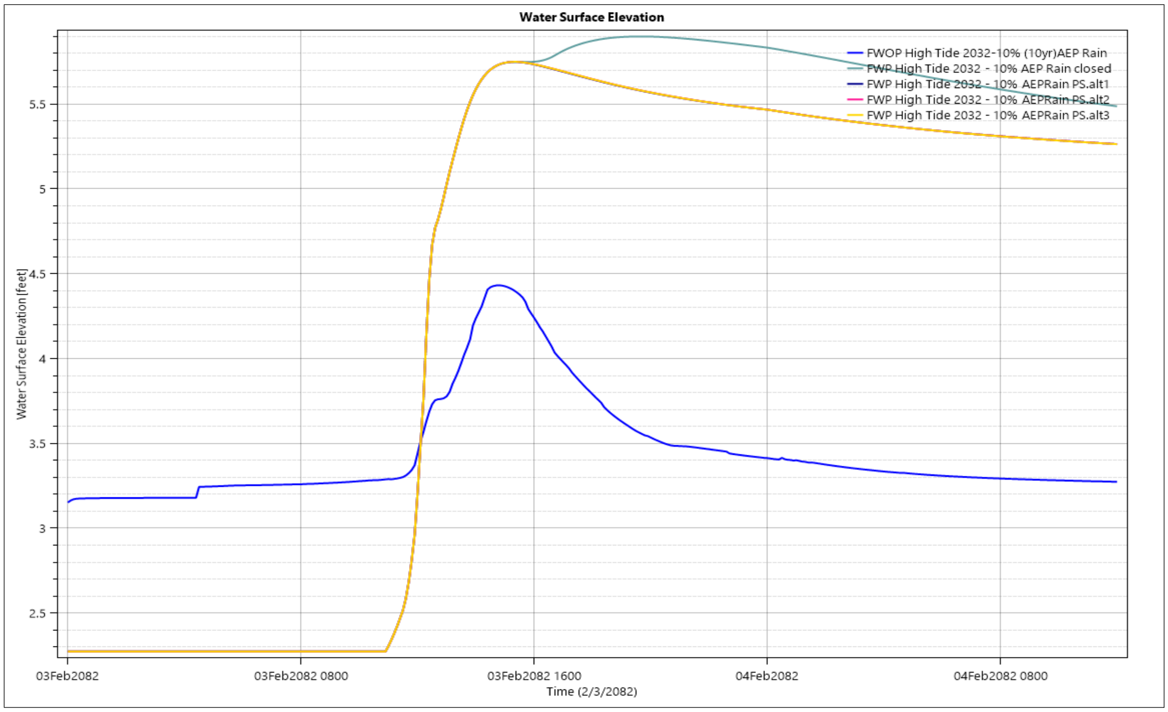

Figure 32. Hydrographs at output location #6 for pump station alternatives in the year 2032

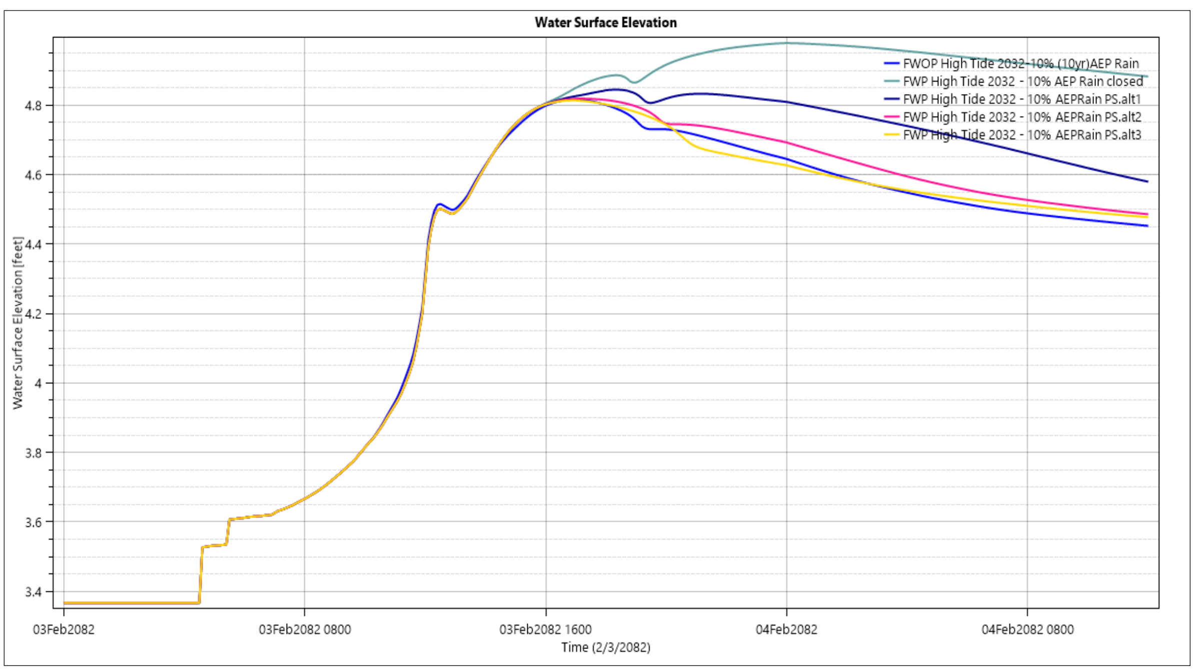

Figure 33. Hydrographs at output location #7 for pump station alternatives in the year 2032

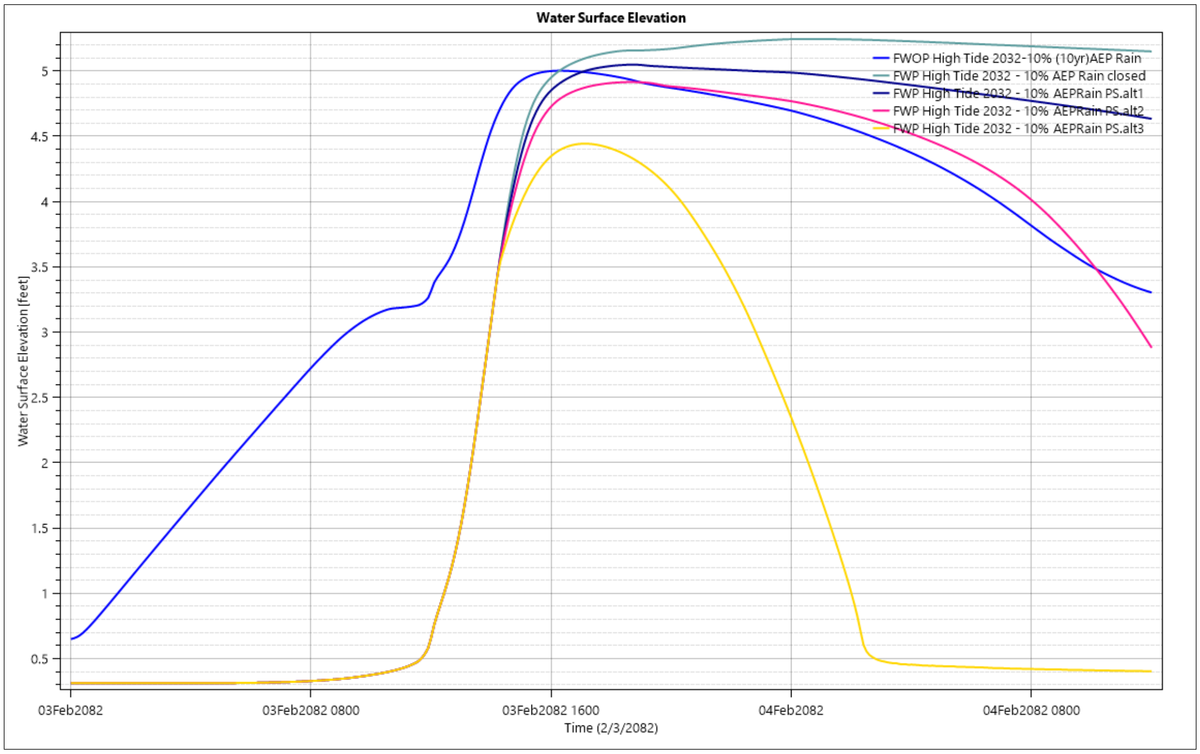

Figure 34. Hydrographs at output location #8 for pump station alternatives in the year 2032

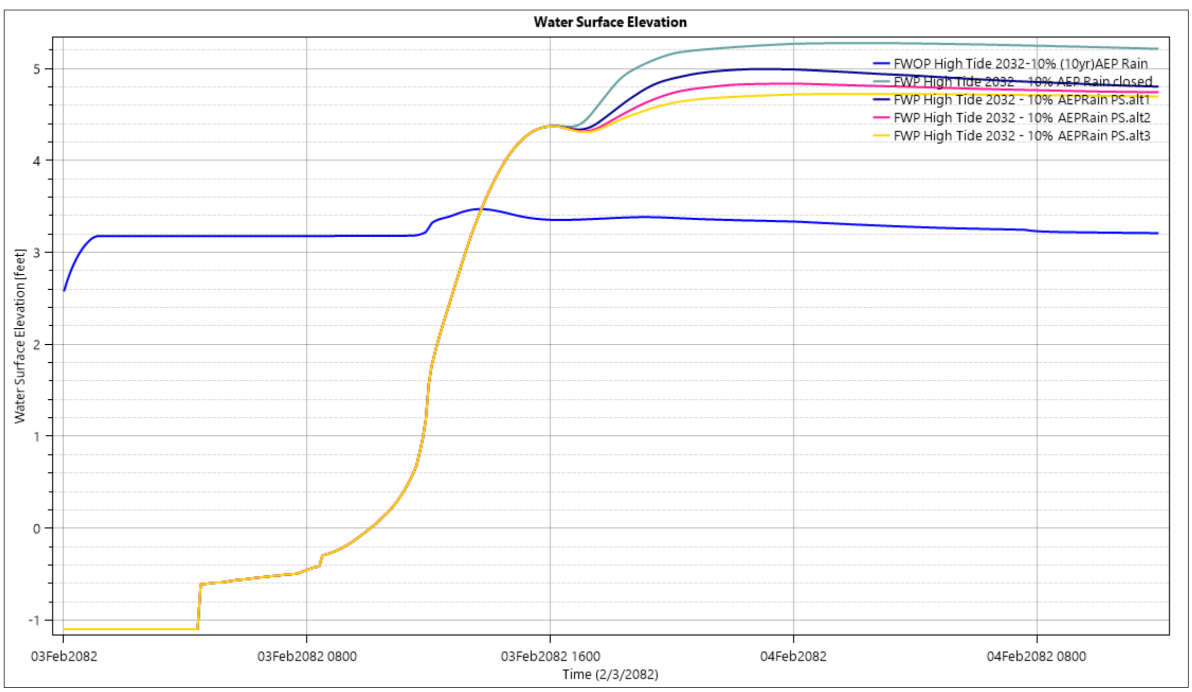

Figure 35. Hydrographs at output location #9 for pump station alternatives in the year 2032

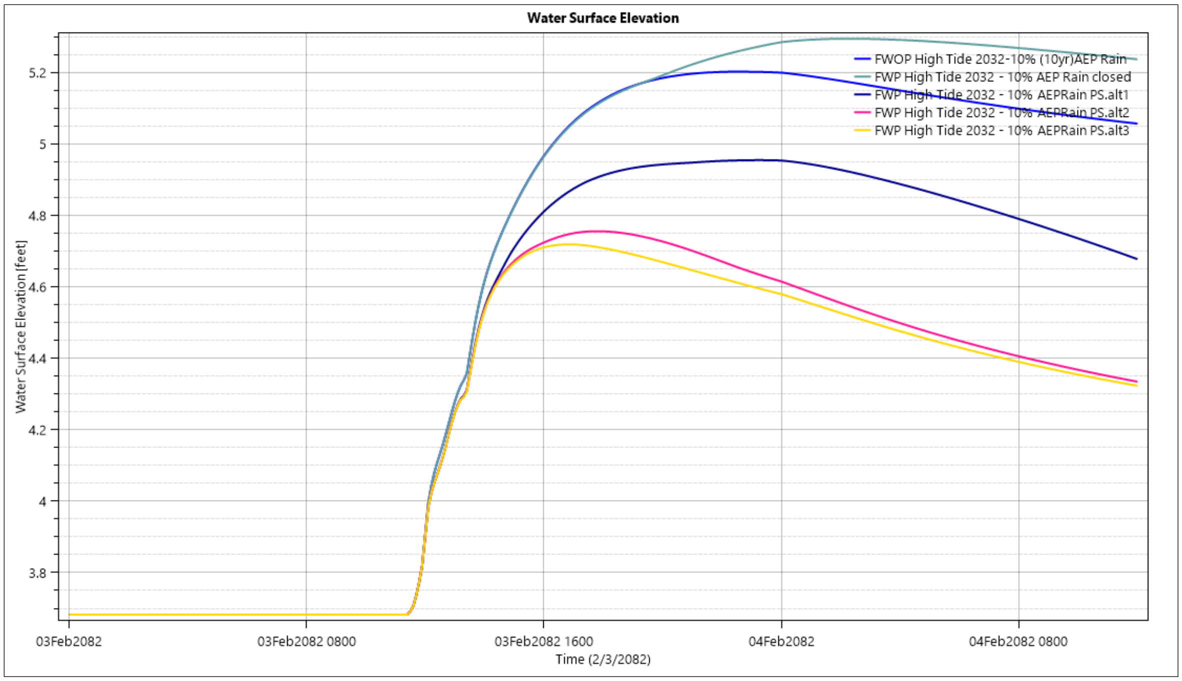

Figure 36. Hydrographs at output location #10 for pump station alternatives in the year 2032

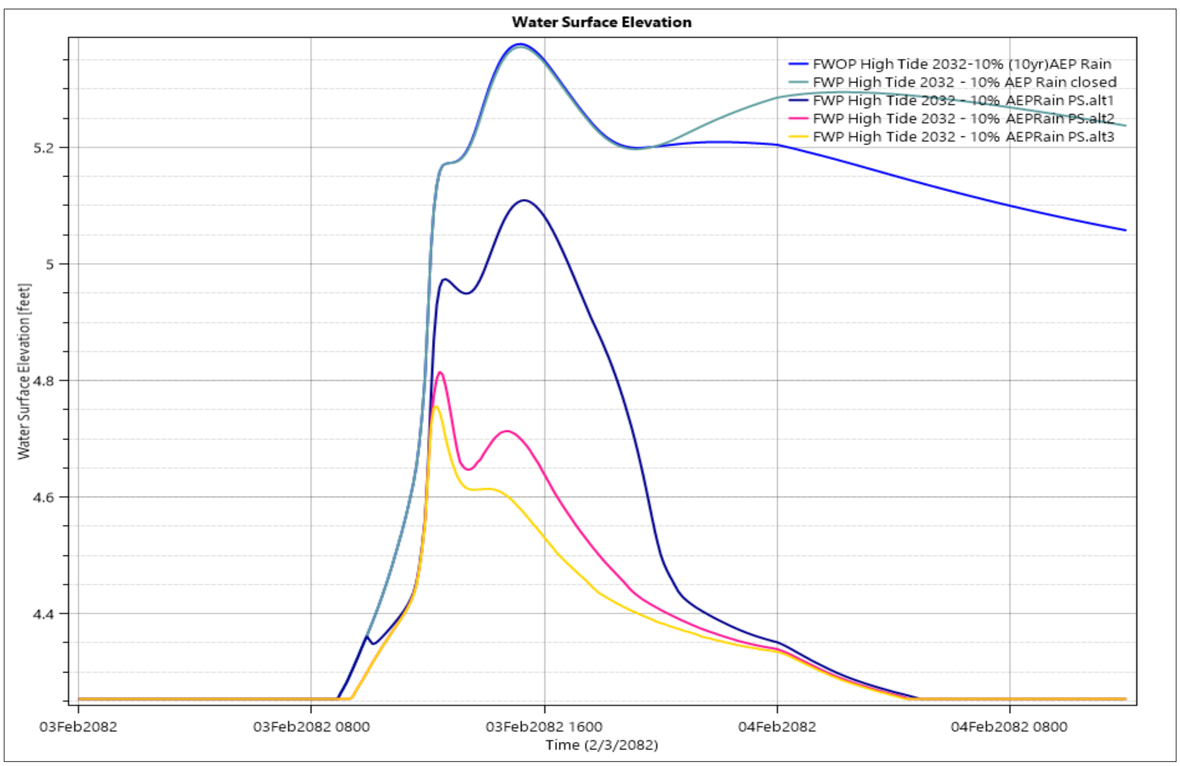

Figure 37. Hydrographs at output location #11 for pump station alternatives in the year 2032

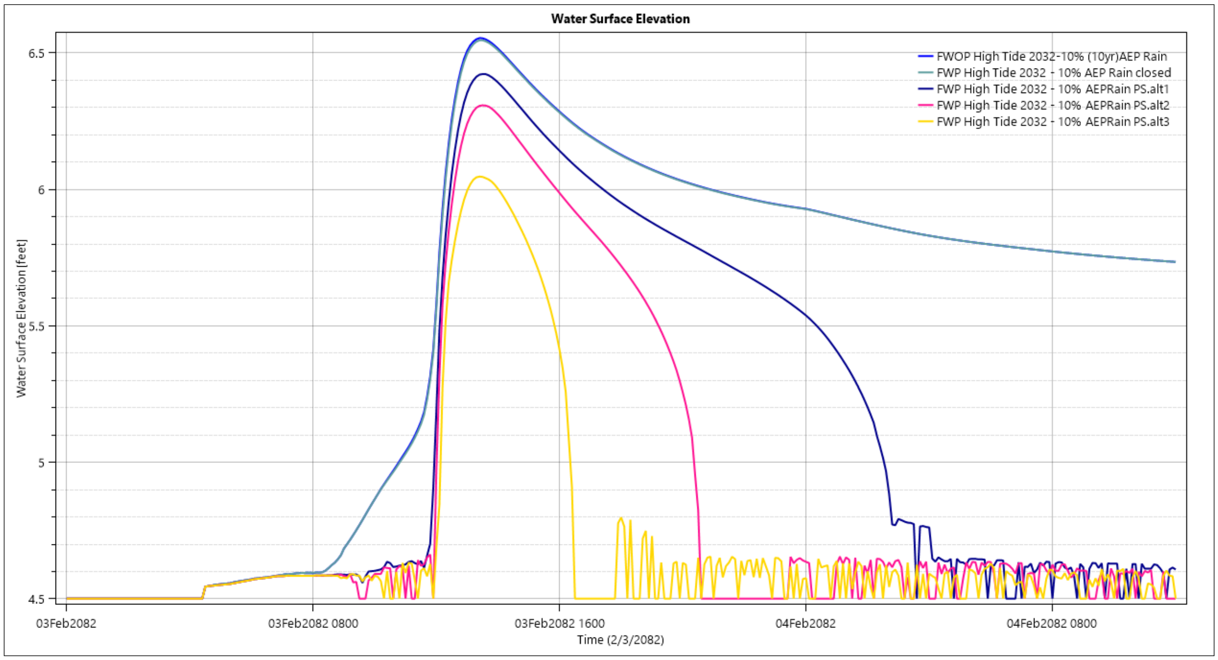

Figure 38. Hydrographs at output location #12 for pump station alternatives in the year 2032

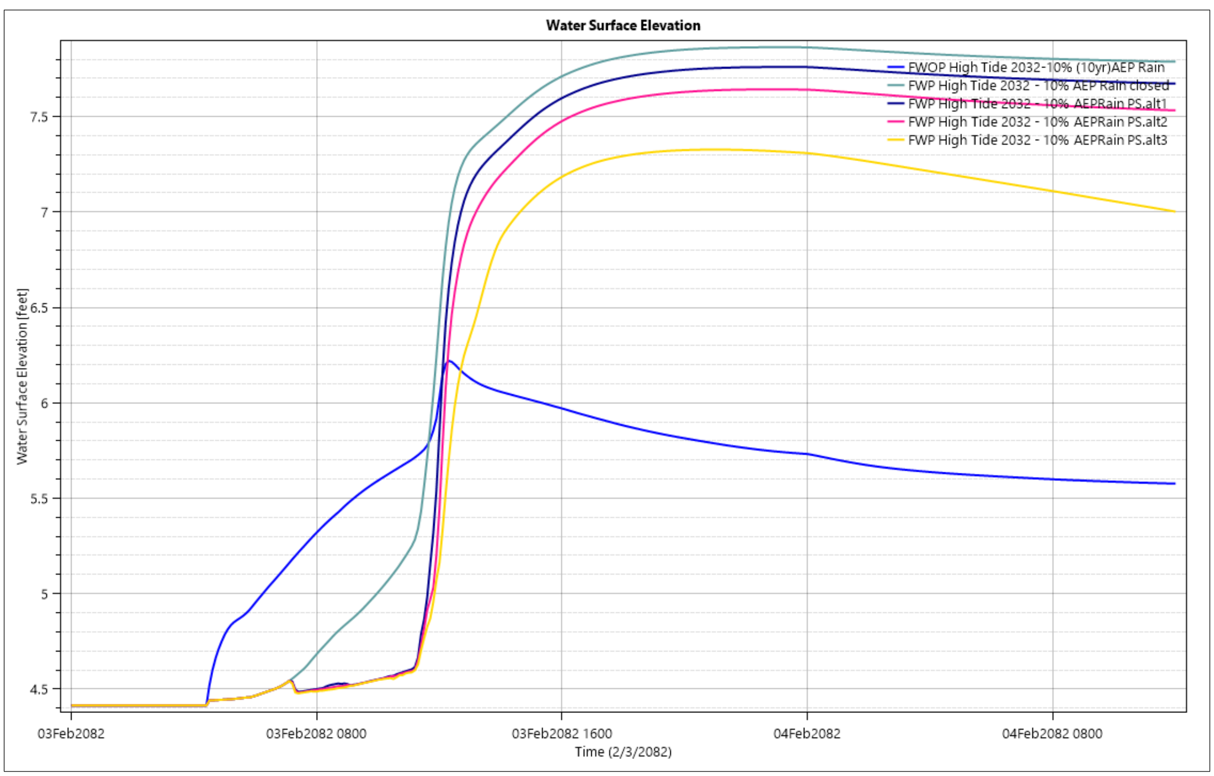

Figure 39. Hydrographs at output location #13 for pump station alternatives in the year 2032

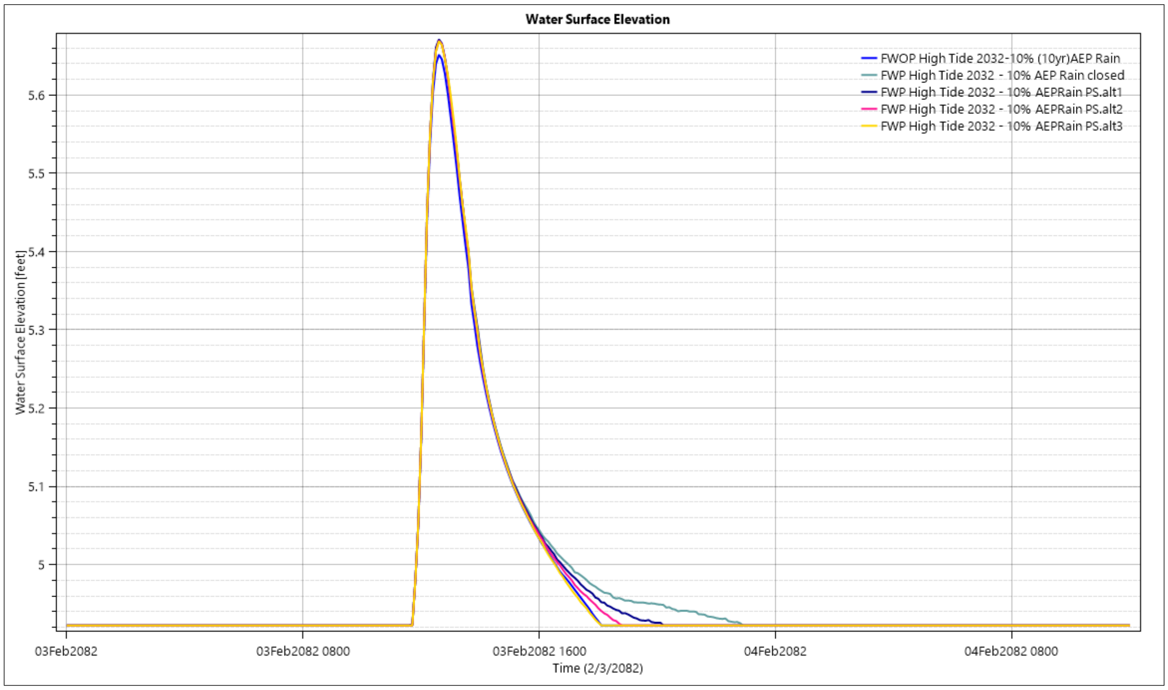

Figure 40. Hydrographs at output location #14 for pump station alternatives in the year 2032

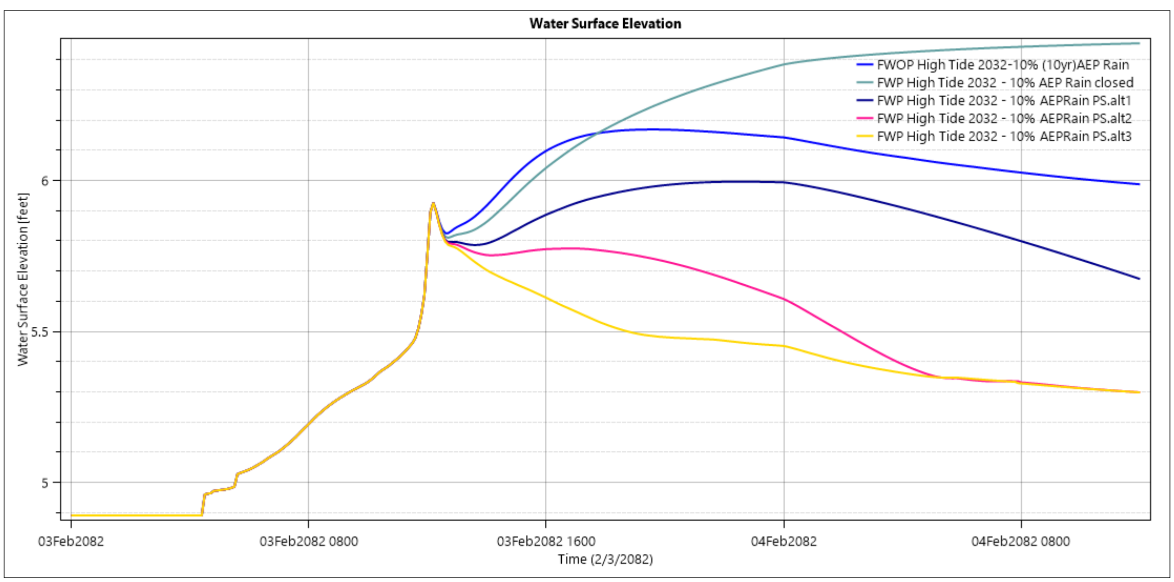

Figure 41. Hydrographs at output location #22 for pump station alternatives in the year 2032

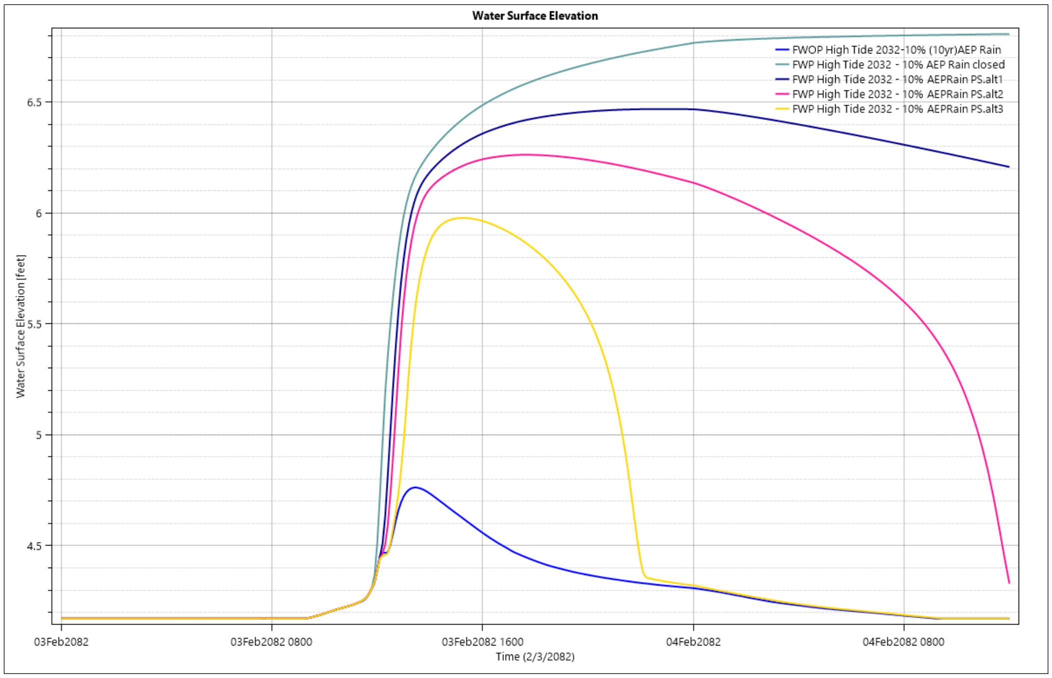

Figure 42. Hydrographs at output location #15 for pump station alternatives in the year 2032

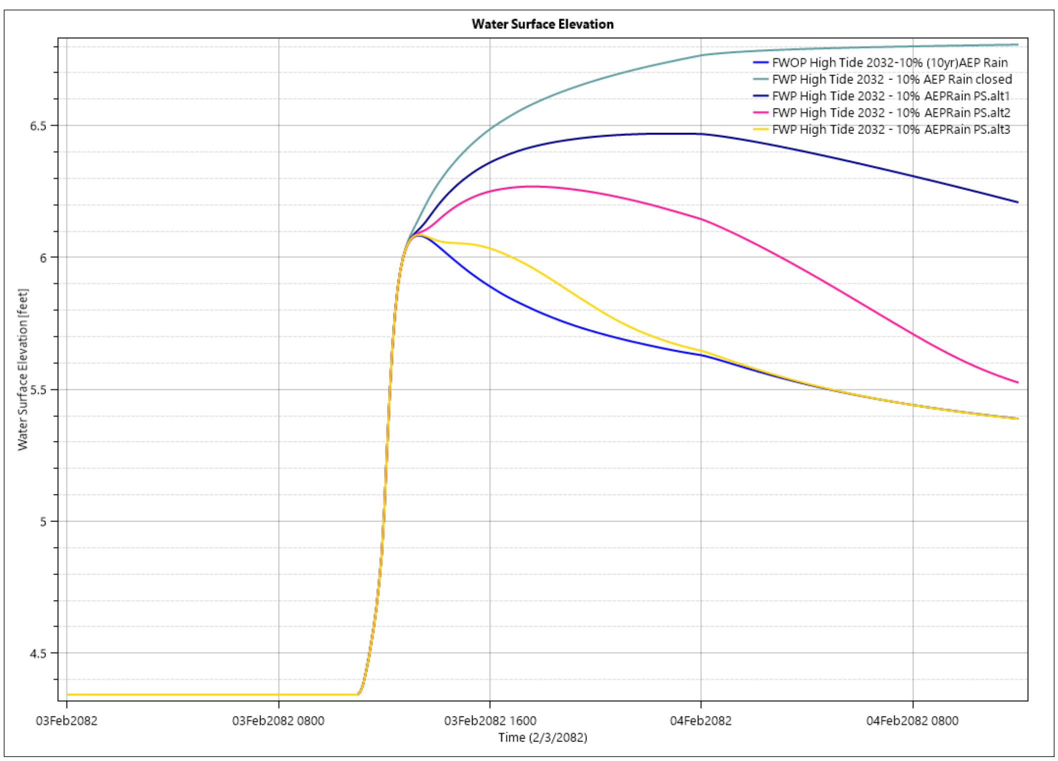

Figure 43. Hydrographs at output location #23 for pump station alternatives in the year 2032

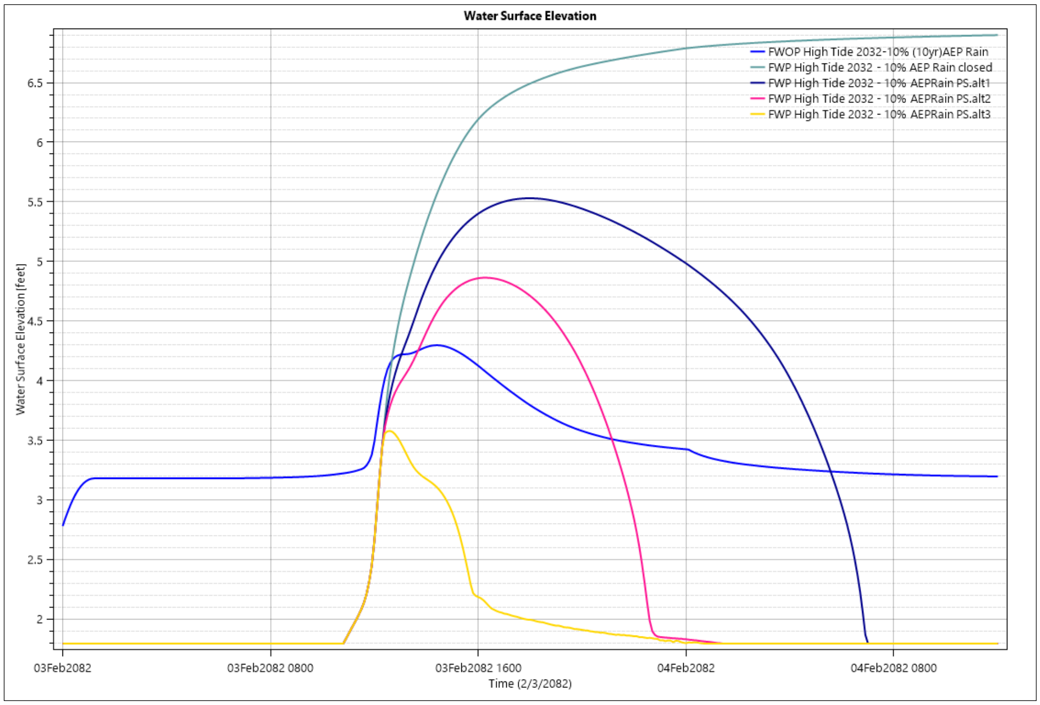

Figure 44. Hydrographs at output location #16 for pump station alternatives in the year 2032

4.5.2 Results for the year 2082

Selected Output Locations	FW 2082 (gates closed) @ 4.27ft Exterior WSEL/-1.31ft Initial Interior WSEL											
	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	7.22	2.92	7.86	3.54	8.10	3.76	8.40	4.03	8.53	4.14	8.72	4.29
2	2.62	-1.66	5.91	1.63	6.64	2.36	7.39	3.11	7.84	3.56	8.28	4.00
3	5.52	1.25	5.92	1.62	6.64	2.33	7.40	3.05	7.85	3.47	8.29	3.87
4	6.22	0.06	6.40	0.13	6.64	0.23	7.40	0.82	7.85	1.15	8.28	1.47
5	5.34	1.05	5.66	1.36	5.91	1.59	6.19	1.85	6.36	2.00	6.55	2.15
6	5.48	1.09	5.66	1.09	5.90	1.11	6.16	1.06	6.33	1.04	6.51	1.03
7	4.58	0.00	4.77	0.04	4.98	0.11	5.19	0.19	5.56	0.47	5.83	0.63
8	4.75	-0.04	4.98	0.01	5.24	0.12	5.51	0.25	5.71	0.37	5.93	0.51
9	4.25	-0.05	5.03	0.71	5.28	0.91	5.56	1.11	5.75	1.22	5.97	1.36
10	4.84	0.00	5.06	0.01	5.29	0.09	5.57	0.19	5.76	0.26	5.97	0.33
11	4.98	0.00	5.16	0.00	5.37	-0.01	5.58	-0.01	5.72	-0.01	5.98	0.12
12	6.18	0.00	6.35	-0.01	6.55	-0.01	6.77	-0.01	6.93	-0.01	7.09	-0.02
13	7.77	1.69	7.77	1.60	7.85	1.60	7.97	1.63	8.08	1.68	8.22	1.75
14	5.38	0.04	5.52	0.03	5.65	0.03	5.80	0.03	5.91	0.03	6.06	0.04
15	6.16	1.70	6.50	1.89	6.81	2.04	7.19	2.19	7.50	2.35	7.79	2.48
16	5.86	1.38	6.47	1.87	6.90	2.14	7.37	2.00	7.58	1.78	7.82	1.59
17	5.95	-0.11	6.25	-0.06	6.64	0.14	7.40	0.71	7.85	1.03	8.28	1.32
18	5.48	0.73	5.67	0.71	5.91	0.67	6.18	0.66	6.36	0.66	6.55	0.68
19	5.71	0.00	5.85	0.00	6.00	-0.01	6.18	0.00	6.31	0.00	6.45	0.00
20	11.07	0.00	11.17	0.00	11.26	0.00	11.34	0.00	11.39	0.00	11.43	-0.01
21	7.82	0.10	7.92	0.11	8.00	0.11	8.10	0.12	8.18	0.13	8.27	0.14
22	5.74	-0.11	6.09	0.04	6.45	0.28	6.89	0.57	7.32	0.90	7.72	1.18
23	6.16	0.40	6.50	0.59	6.81	0.73	7.18	0.87	7.50	0.80	7.79	1.15
24	5.86	0.57	6.47	0.47	6.90	0.62	7.37	0.78	7.58	0.81	7.82	0.90

Table 16. Peak water surface elevations for future with-project (gates closed) in the year 2082

Selected Output Locations	FW 2082 (gates closed) P.S. alt 1 @ 4.27ft Exterior WSEL/-1.31ft Initial Interior WSEL											
	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	7.22	2.92	7.86	3.54	8.09	3.75	8.35	3.98	8.52	4.13	8.72	4.29
2	2.64	-1.64	2.83	-1.45	4.10	-0.18	6.52	2.24	7.14	2.86	7.72	3.44
3	3.80	-0.47	4.74	0.44	5.84	1.53	6.52	2.17	7.14	2.76	7.72	3.30
4	6.22	0.06	6.40	0.13	6.57	0.16	6.78	0.20	7.14	0.44	7.72	0.91
5	3.38	-0.91	3.62	-0.68	5.09	0.77	5.94	1.60	6.20	1.84	6.44	2.04
6	5.48	1.09	5.61	1.04	5.75	0.96	5.94	0.84	6.19	0.90	6.41	0.93
7	4.57	-0.01	4.70	-0.03	4.84	-0.03	5.07	0.07	5.24	0.15	5.69	0.49
8	4.08	-0.71	4.76	-0.21	5.05	-0.07	5.32	0.06	5.54	0.20	5.79	0.37
9	4.11	-0.19	4.61	0.29	5.00	0.63	5.33	0.88	5.56	1.03	5.81	1.20
10	4.43	-0.41	4.66	-0.39	4.95	-0.25	5.33	-0.05	5.57	0.07	5.82	0.18
11	4.69	-0.29	4.85	-0.31	5.11	-0.27	5.44	-0.15	5.60	-0.13	5.82	-0.04
12	5.98	-0.20	6.21	-0.15	6.42	-0.14	6.66	-0.12	6.82	-0.12	7.00	-0.11
13	7.65	1.57	7.68	1.51	7.76	1.51	7.87	1.53	7.99	1.59	8.14	1.67
14	5.38	0.04	5.53	0.04	5.66	0.04	5.81	0.04	5.92	0.04	6.07	0.05
15	5.72	1.26	6.12	1.51	6.47	1.70	6.89	1.89	7.18	2.03	7.44	2.13
16	3.42	-1.06	4.13	-0.47	5.53	0.77	6.53	1.16	6.94	1.14	7.32	1.09
17	5.95	-0.11	6.25	-0.06	6.47	-0.03	6.68	-0.01	7.14	0.32	7.71	0.75
18	5.48	0.73	5.61	0.65	5.75	0.51	5.95	0.43	6.20	0.50	6.44	0.57
19	5.71	0.00	5.85	0.00	6.00	-0.01	6.18	0.00	6.31	0.00	6.45	0.00
20	11.07	0.00	11.16	-0.01	11.26	0.00	11.33	-0.01	11.38	-0.01	11.43	-0.01
21	7.80	0.08	7.90	0.09	7.98	0.09	8.08	0.10	8.16	0.11	8.25	0.12
22	5.75	-0.10	5.84	-0.21	6.00	-0.17	6.48	0.16	6.80	0.38	7.20	0.66
23	5.76	0.00	6.12	0.21	6.47	0.39	6.89	0.58	7.18	0.48	7.44	0.80
24	5.19	-0.10	5.97	-0.03	6.27	-0.01	6.58	-0.01	6.94	0.17	7.32	0.40

Table 17. Peak water surface elevations for future with-project (gates closed) P.S. alt 1 in the year 2082

Selected Output Locations	FW 2082 (gates closed) P.S. alt 2 @ 4.27ft Exterior WSEL/-1.31ft Initial Interior WSEL											
	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	7.23	2.93	7.86	3.54	8.09	3.75	8.34	3.97	8.52	4.13	8.72	4.29
2	2.67	-1.61	2.81	-1.47	3.46	-0.82	6.22	1.94	6.88	2.60	7.50	3.22
3	3.46	-0.81	4.21	-0.09	5.45	1.14	6.22	1.87	6.87	2.49	7.50	3.08
4	6.22	0.06	6.40	0.13	6.57	0.16	6.78	0.20	6.92	0.22	7.50	0.69
5	3.37	-0.92	3.43	-0.87	4.21	-0.11	5.73	1.39	6.10	1.74	6.38	1.98
6	5.48	1.09	5.61	1.04	5.75	0.96	5.91	0.81	6.10	0.81	6.35	0.87
7	4.57	-0.01	4.70	-0.03	4.82	-0.05	5.01	0.01	5.18	0.09	5.56	0.36
8	3.50	-1.29	4.36	-0.61	4.92	-0.20	5.23	-0.03	5.44	0.10	5.68	0.26
9	4.11	-0.19	4.56	0.24	4.83	0.46	5.18	0.73	5.41	0.88	5.68	1.07
10	4.41	-0.43	4.58	-0.47	4.76	-0.44	5.11	-0.27	5.40	-0.10	5.68	0.04
11	4.63	-0.35	4.70	-0.46	4.81	-0.57	5.27	-0.32	5.49	-0.24	5.69	-0.17
12	5.69	-0.49	6.05	-0.31	6.31	-0.25	6.57	-0.21	6.74	-0.20	6.92	-0.19
13	7.47	1.39	7.53	1.36	7.64	1.39	7.78	1.44	7.90	1.50	8.07	1.60
14	5.43	0.09	5.57	0.08	5.71	0.09	5.86	0.09	5.97	0.09	6.11	0.09
15	4.43	-0.03	5.89	1.28	6.26	1.49	6.69	1.69	6.99	1.84	7.30	1.99
16	3.40	-1.08	3.51	-1.09	4.86	0.10	6.17	0.80	6.72	0.92	7.13	0.90
17	5.95	-0.11	6.25	-0.06	6.47	-0.03	6.68	-0.01	6.88	0.06	7.50	0.54
18	5.48	0.73	5.62	0.66	5.75	0.51	5.91	0.39	6.11	0.41	6.38	0.51
19	5.71	0.00	5.85	0.00	6.01	0.00	6.18	0.00	6.31	0.00	6.45	0.00
20	11.06	-0.01	11.16	-0.01	11.25	-0.01	11.33	-0.01	11.38	-0.01	11.43	-0.01
21	7.73	0.01	7.82	0.01	7.90	0.01	7.99	0.01	8.07	0.02	8.15	0.02
22	5.74	-0.11	5.84	-0.21	5.93	-0.24	6.22	-0.10	6.54	0.12	6.91	0.37
23	5.76	0.00	5.92	0.01	6.27	0.19	6.69	0.38	6.99	0.29	7.30	0.66
24	5.19	-0.10	5.97	-0.03	6.27	-0.01	6.58	-0.01	6.76	-0.01	7.13	0.21

Table 18. Peak water surface elevations for future with-project (gates closed) P.S. alt 2 in the year 2082

Selected Output Locations	FW 2082 (gates closed) P.S. alt 3 @ 4.27ft Exterior WSEL/-1.31ft Initial Interior WSEL											
	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	7.22	2.92	7.86	3.54	8.09	3.75	8.33	3.96	8.51	4.12	8.72	4.29
2	2.61	-1.67	2.81	-1.47	3.46	-0.82	5.65	1.37	6.48	2.20	7.16	2.88
3	3.39	-0.88	3.64	-0.66	4.60	0.29	6.00	1.65	6.48	2.10	7.16	2.74
4	6.22	0.06	6.40	0.13	6.57	0.16	6.78	0.20	6.92	0.22	7.16	0.35
5	3.36	-0.93	3.39	-0.91	3.53	-0.79	4.71	0.37	5.74	1.38	6.21	1.81
6	5.48	1.09	5.61	1.04	5.75	0.96	5.91	0.81	6.03	0.74	6.23	0.75
7	4.57	-0.01	4.70	-0.03	4.81	-0.06	4.96	-0.04	5.09	0.00	5.29	0.09
8	3.39	-1.40	3.58	-1.39	4.44	-0.68	5.06	-0.20	5.30	-0.04	5.53	0.11
9	4.11	-0.19	4.56	0.24	4.72	0.35	5.01	0.56	5.22	0.69	5.47	0.86
10	4.41	-0.43	4.57	-0.48	4.72	-0.48	4.89	-0.49	5.08	-0.42	5.42	-0.22
11	4.61	-0.37	4.68	-0.48	4.73	-0.65	4.83	-0.76	5.19	-0.54	5.50	-0.36
12	4.91	-1.27	5.58	-0.78	6.05	-0.51	6.38	-0.40	6.57	-0.37	6.77	-0.34
13	5.12	-0.96	6.89	0.72	7.33	1.08	7.59	1.25	7.75	1.35	7.93	1.46
14	5.37	0.03	5.53	0.04	5.66	0.04	5.81	0.04	5.91	0.03	6.07	0.05
15	4.36	-0.10	4.59	-0.02	5.98	1.21	6.45	1.45	6.76	1.61	7.08	1.77
16	3.38	-1.10	3.43	-1.17	3.58	-1.18	5.34	-0.03	6.20	0.40	6.79	0.56
17	5.95	-0.11	6.25	-0.06	6.47	-0.03	6.68	-0.01	6.82	0.00	7.16	0.20
18	5.48	0.73	5.61	0.65	5.75	0.51	5.91	0.39	6.03	0.33	6.24	0.37
19	5.71	0.00	5.85	0.00	6.00	-0.01	6.18	0.00	6.30	-0.01	6.44	-0.01
20	11.06	-0.01	11.16	-0.01	11.25	-0.01	11.33	-0.01	11.38	-0.01	11.43	-0.01
21	7.83	0.11	7.93	0.12	8.01	0.12	8.11	0.13	8.19	0.14	8.28	0.15
22	5.74	-0.11	5.84	-0.21	5.93	-0.24	6.06	-0.26	6.24	-0.18	6.59	0.05
23	5.76	0.00	5.91	0.00	6.09	0.01	6.46	0.15	6.76	0.06	7.08	0.44
24	5.19	-0.10	5.97	-0.03	6.27	-0.01	6.58	-0.01	6.75	-0.02	6.91	-0.01

Table 19. Peak water surface elevations for future with-project (gates closed) P.S. alt 3 in the year 2082

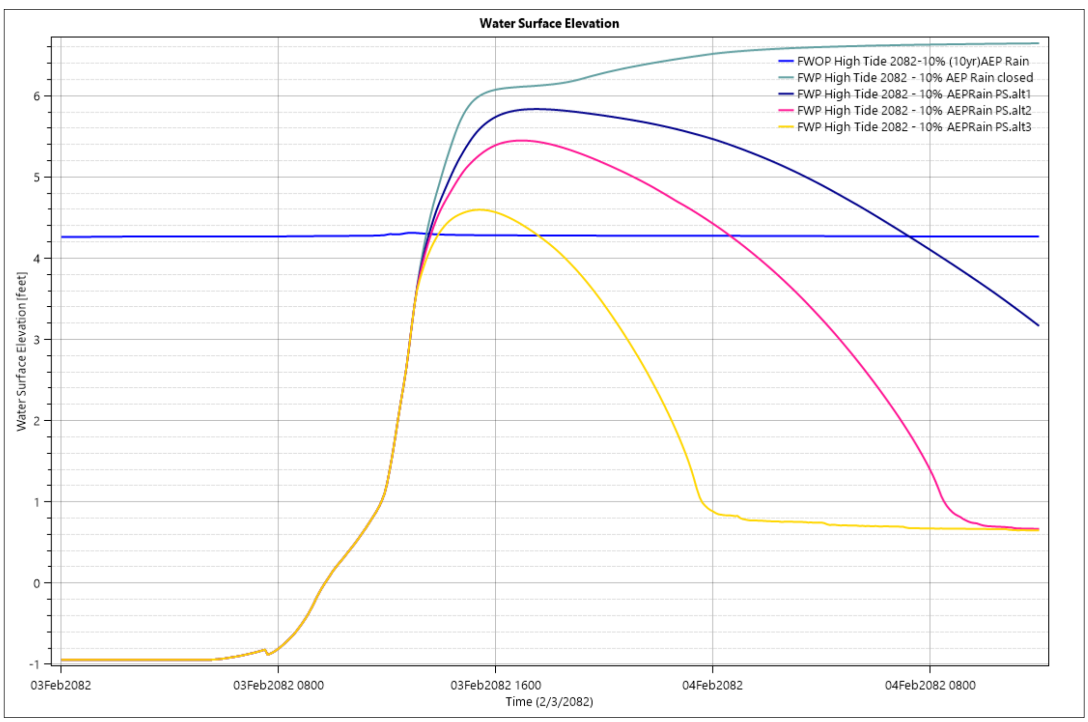

Figure 45. Hydrographs at output location #3 for pump station alternatives in the year 2082

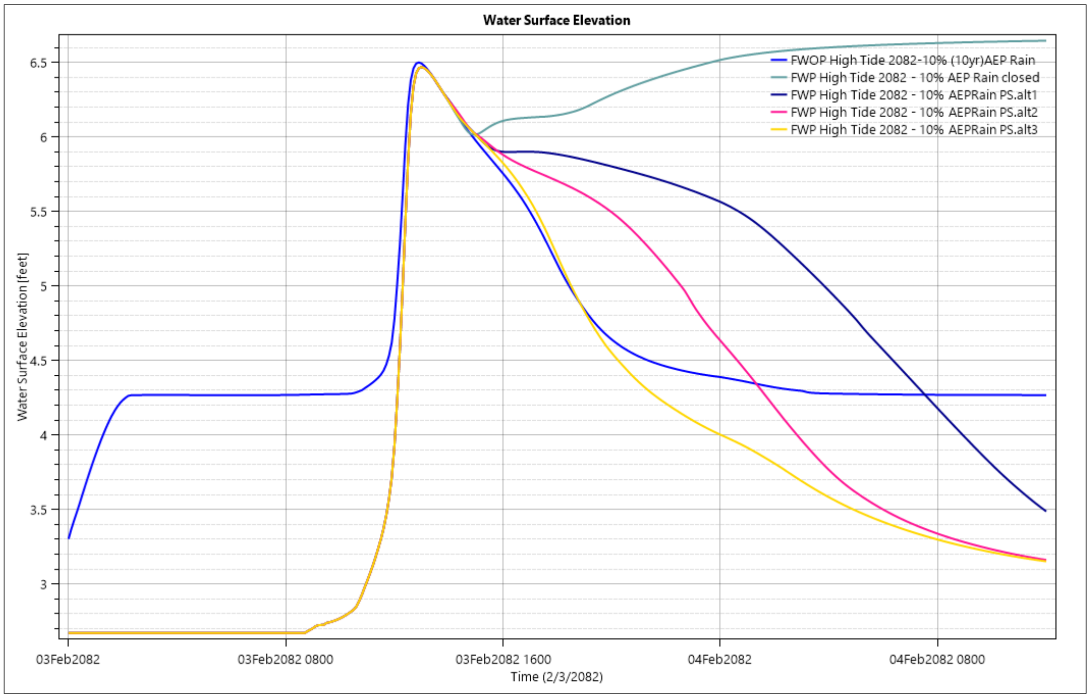

Figure 46. Hydrographs at output location #17 for pump station alternatives in the year 2082

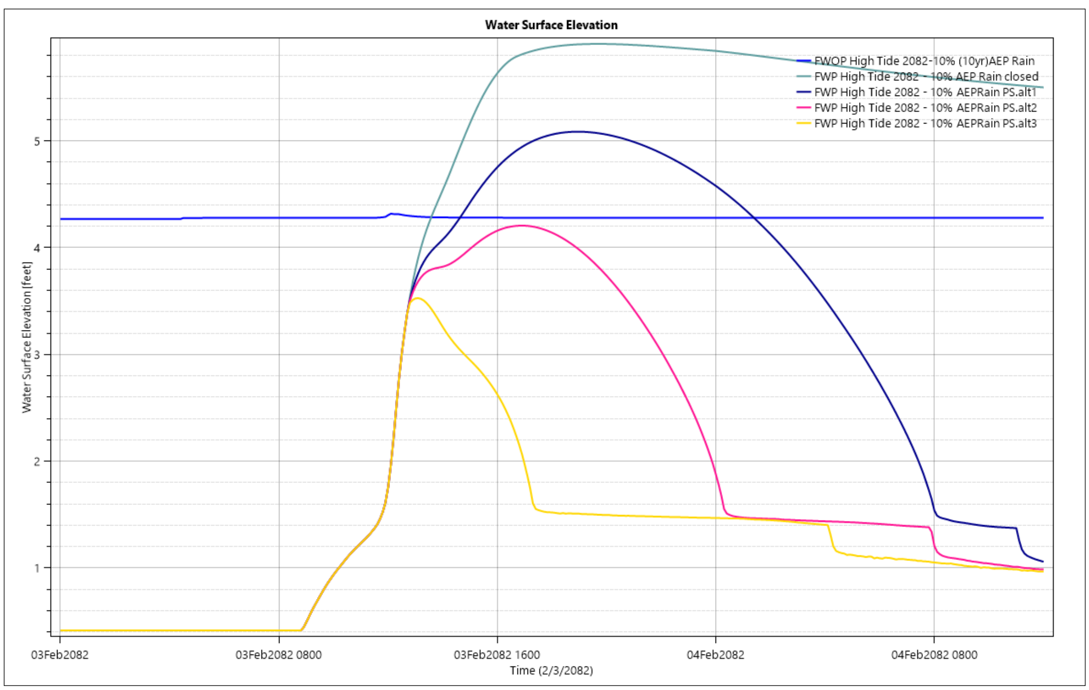

Figure 47. Hydrographs at output location #5 for pump station alternatives in the year 2082

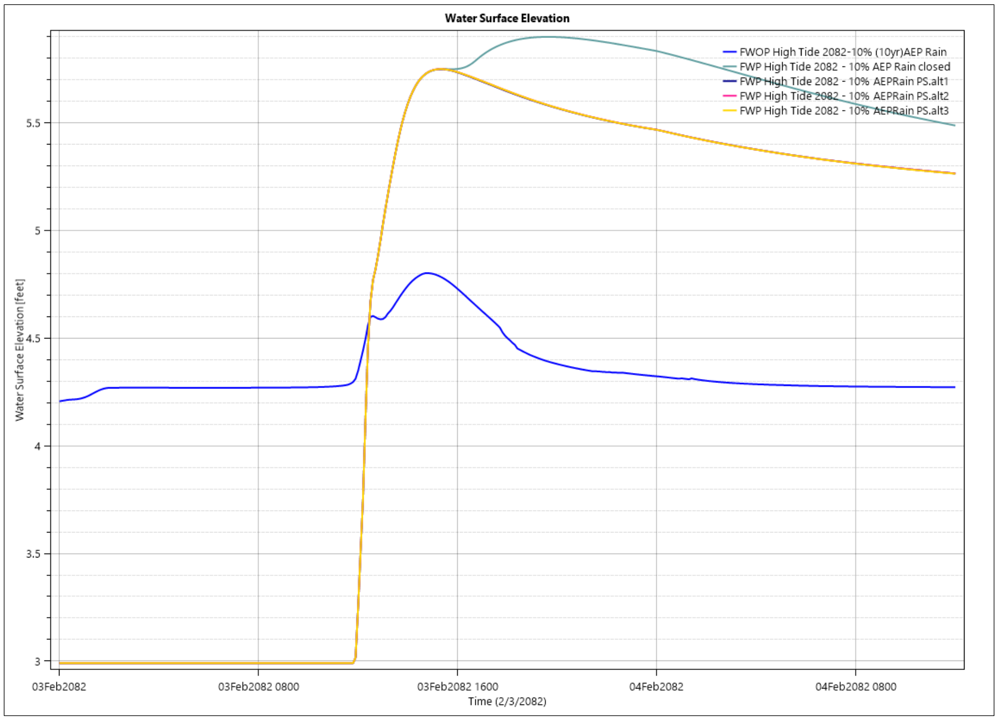

Figure 48. Hydrographs at output location #6 for pump station alternatives in the year 2082

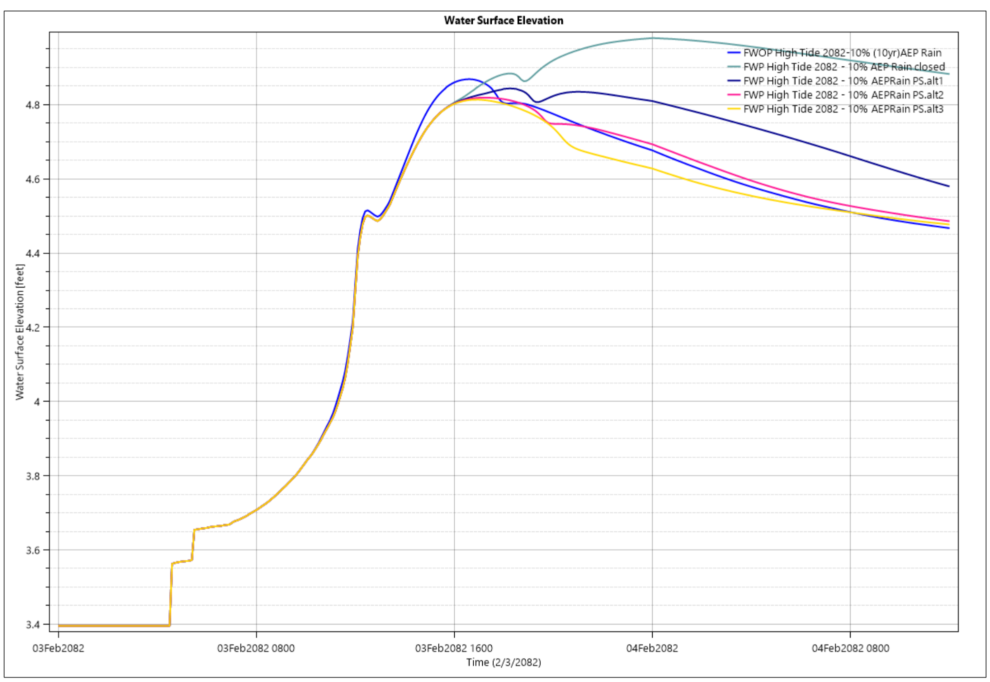

Figure 49. Hydrographs at output location #7 for pump station alternatives in the year 2082

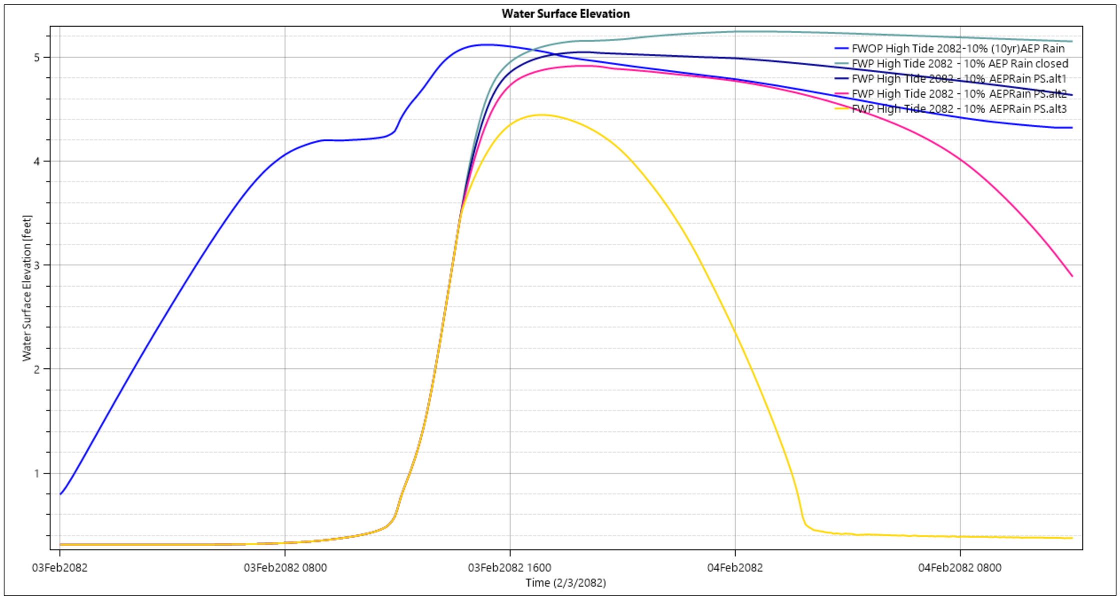

Figure 50. Hydrographs at output location #8 for pump station alternatives in the year 2082

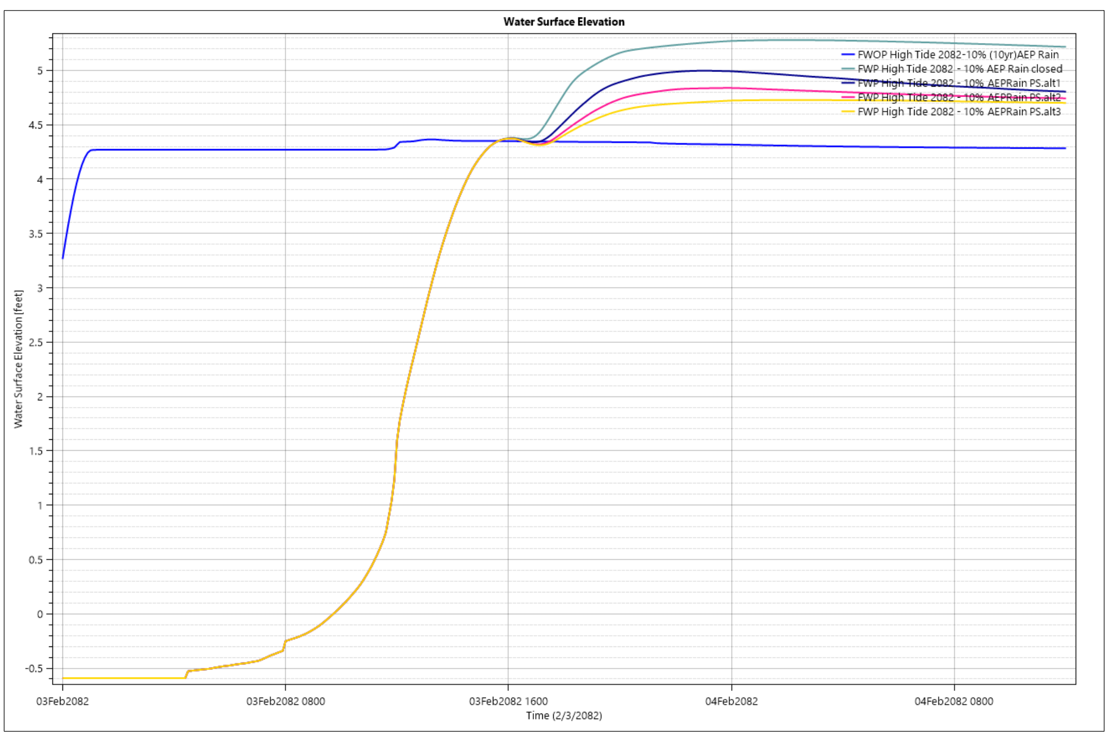

Figure 51. Hydrographs at output location #9 for pump station alternatives in the year 2082

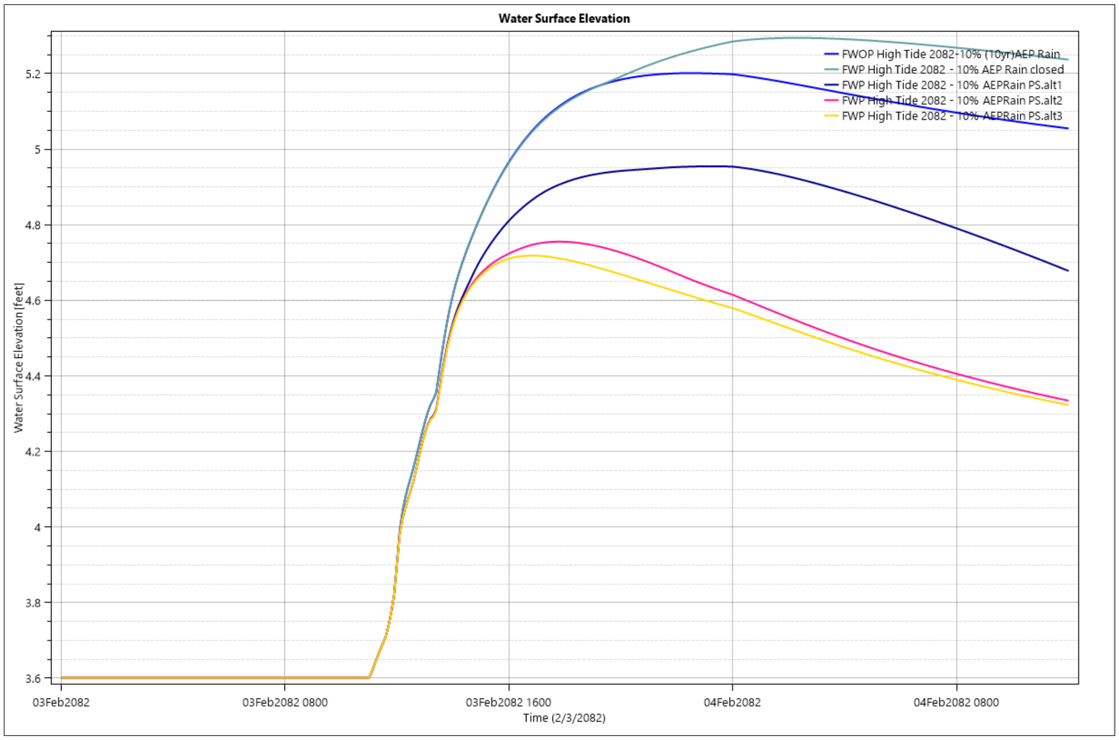

Figure 52. Hydrographs at output location #10 for pump station alternatives in the year 2082

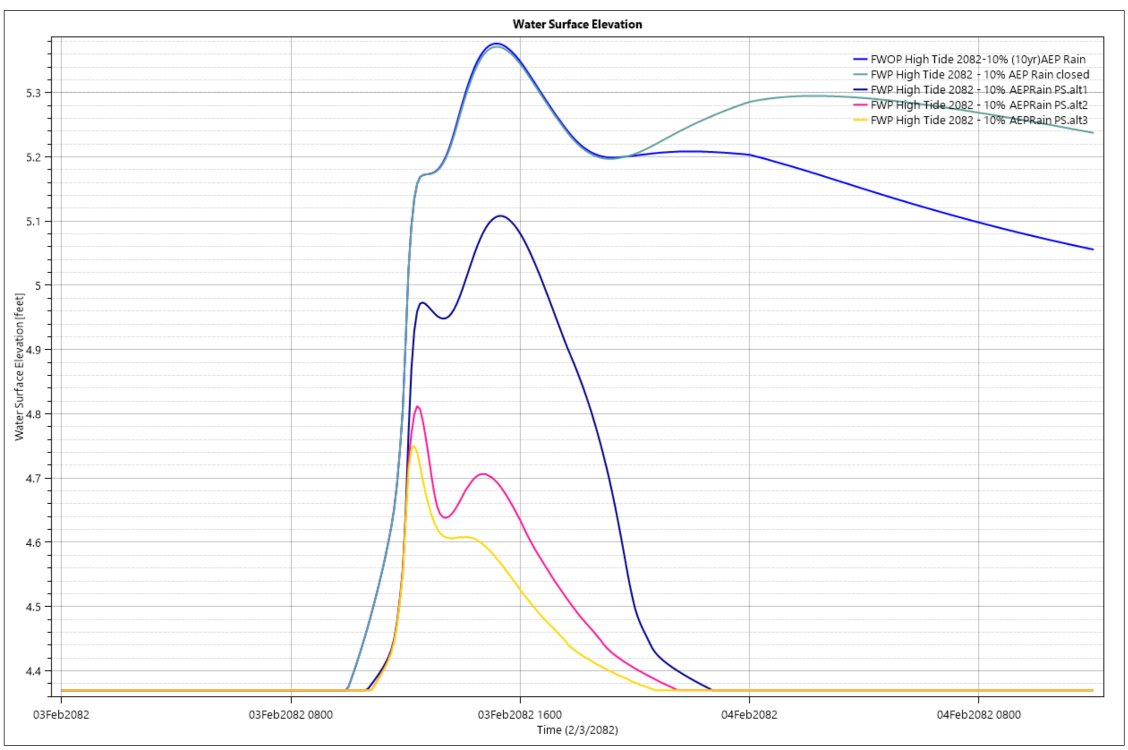

Figure 53. Hydrographs at output location #11 for pump station alternatives in the year 2082

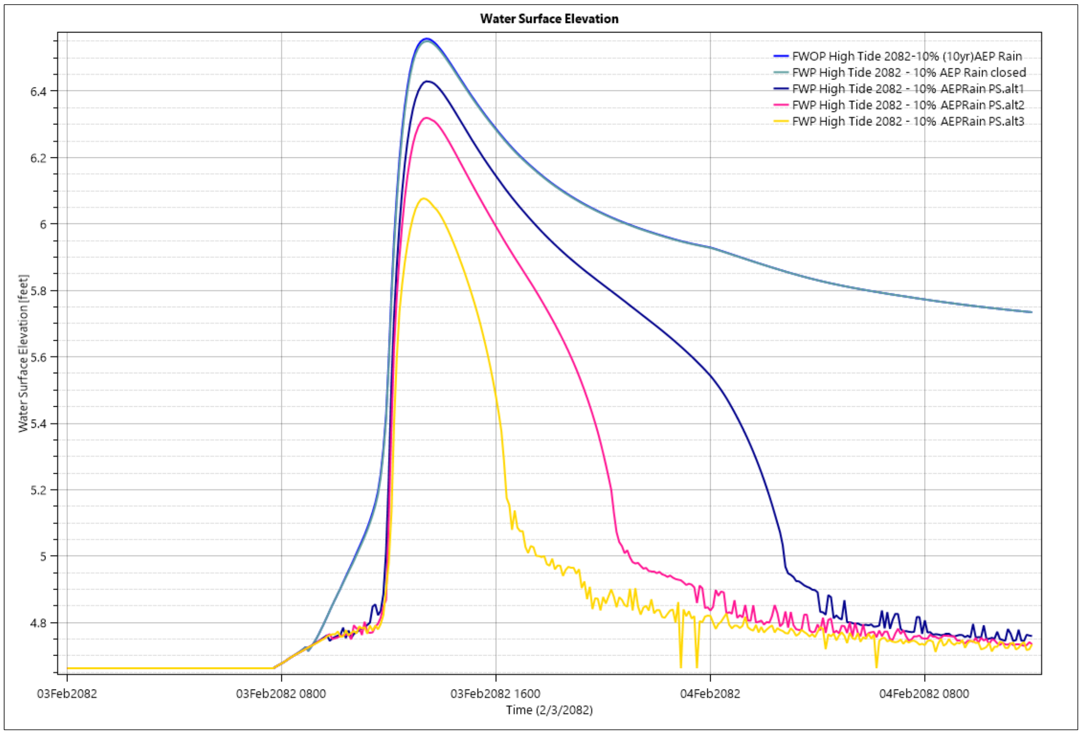

Figure 54. Hydrographs at output location #12 for pump station alternatives in the year 2082

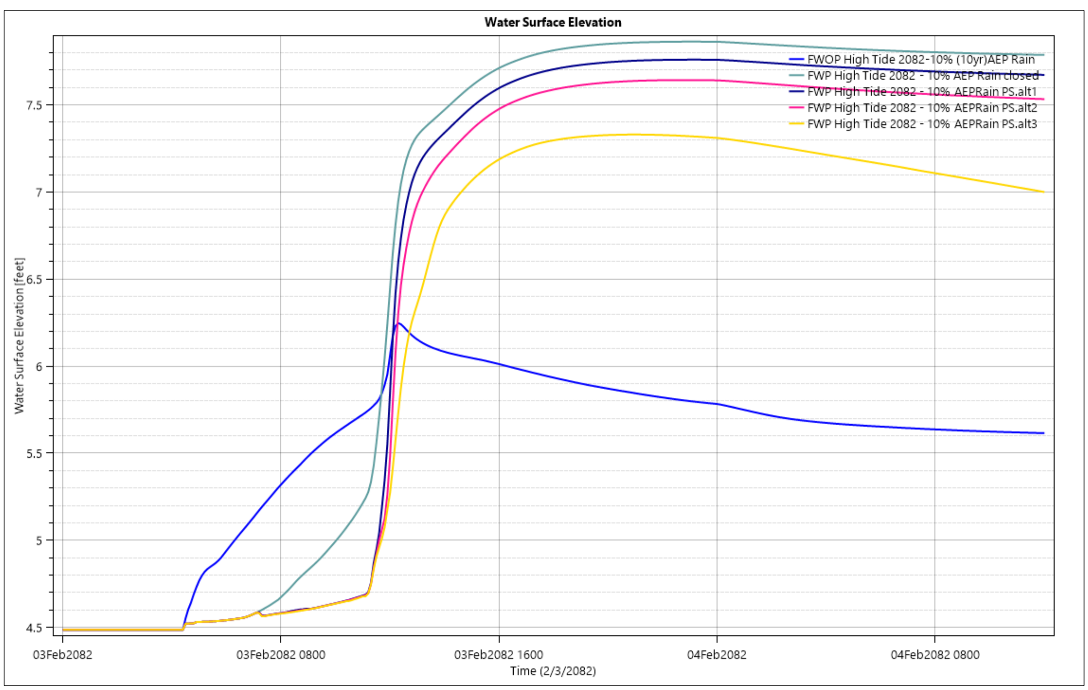

Figure 55. Hydrographs at output location #13 for pump station alternatives in the year 2082

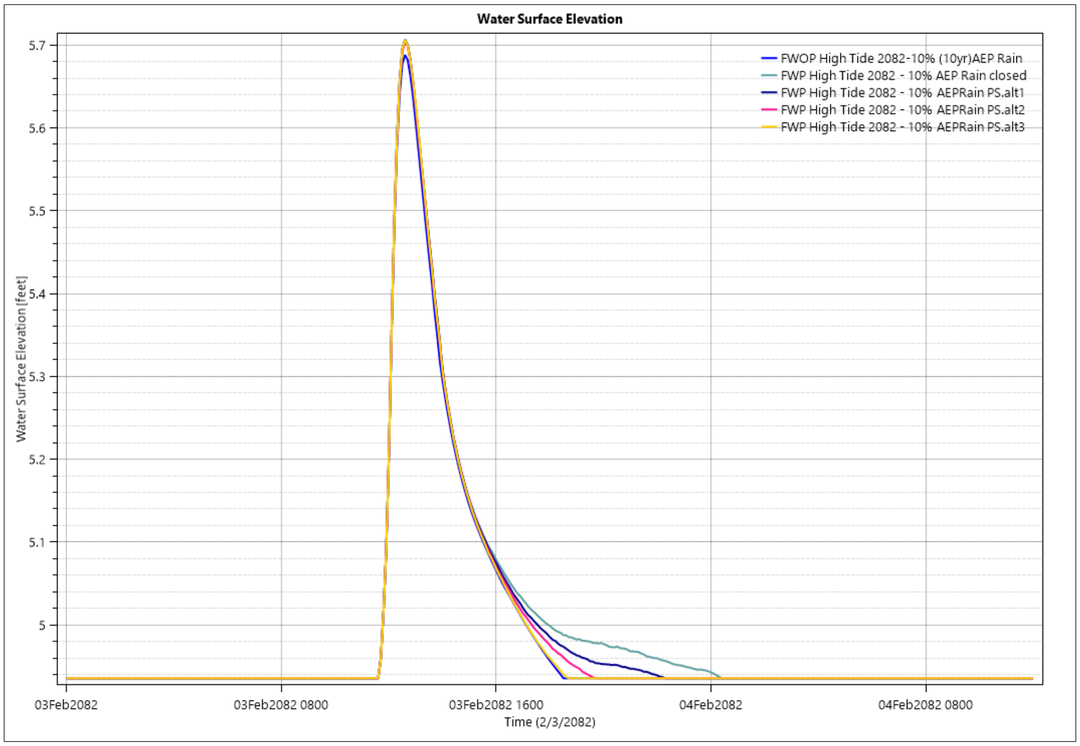

Figure 56. Hydrographs at output location #14 for pump station alternatives in the year 2082

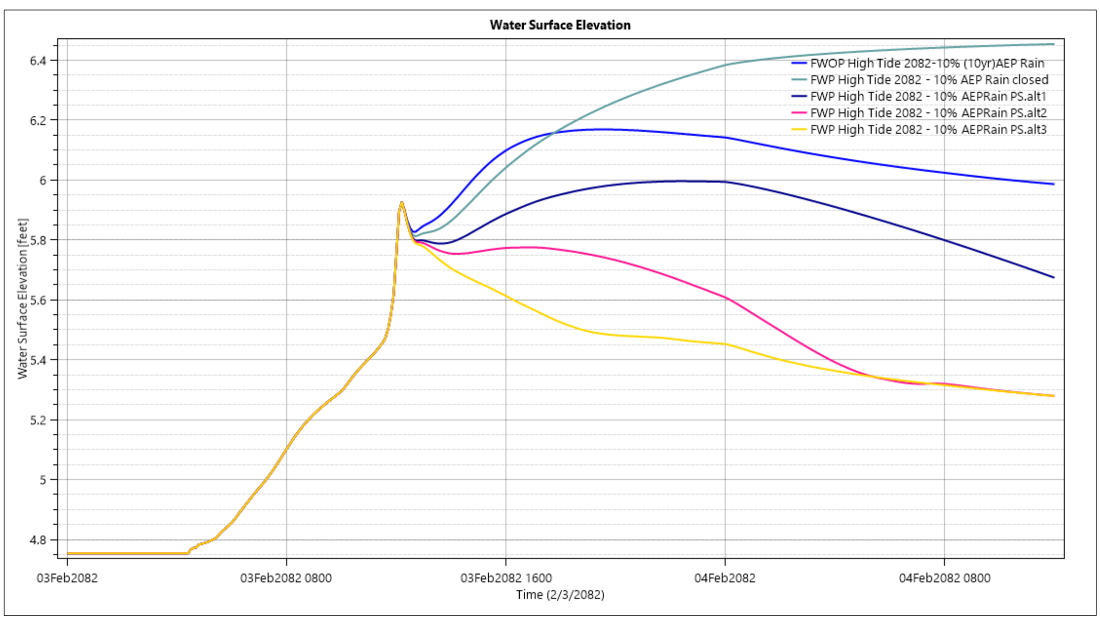

Figure 57. Hydrographs at output location #22 for pump station alternatives in the year 2082

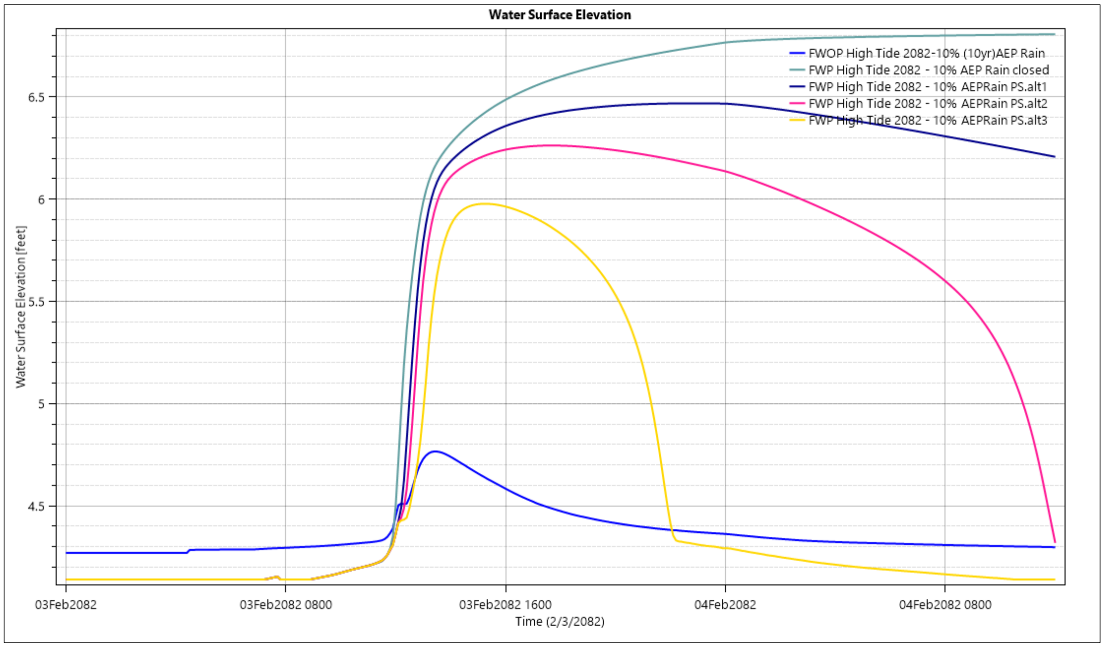

Figure 58. Hydrographs at output location #15 for pump station alternatives in the year 2082

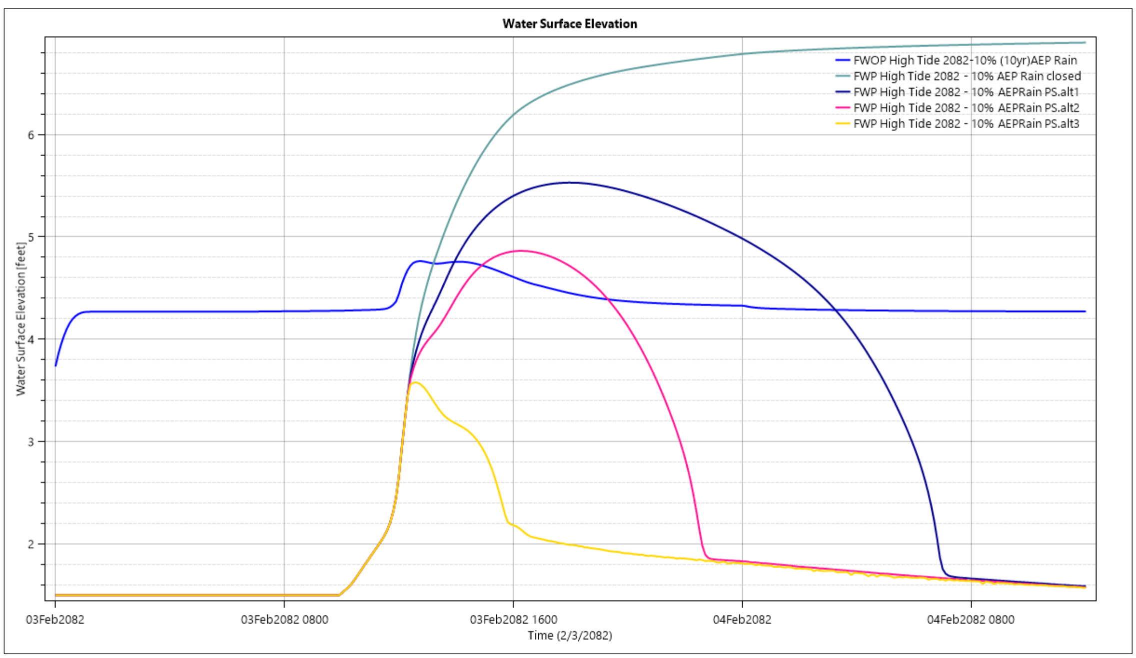

Figure 59. Hydrographs at output location #16 for pump station alternatives in the year 2082

4.5.3 Inputs for the 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.

Performing the analysis within RAS 2D provides insight into the performance of the wall during a storm surge event as opposed to a storm surge event without a wall. This analysis also provides insight into the performance of pumps during a wave wash overtopping event. One aspect of the analysis is to demonstrate that a wall that experiences wave overtopping plus rainfall is less damaging than a storm surge event without a wall.

The provided overtopping rates in Table 20 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 assumed to occur for approximately 4 hours in hydrograph form. This duration assumption comes from the provided hydrograph shown in Figure 63.

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 20 and Figure 60 represent the estimated (1% AEP) overtopping flow rates.

Reaches & Stations	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

Table 20. Overtopping Flow Rates at Different Regions

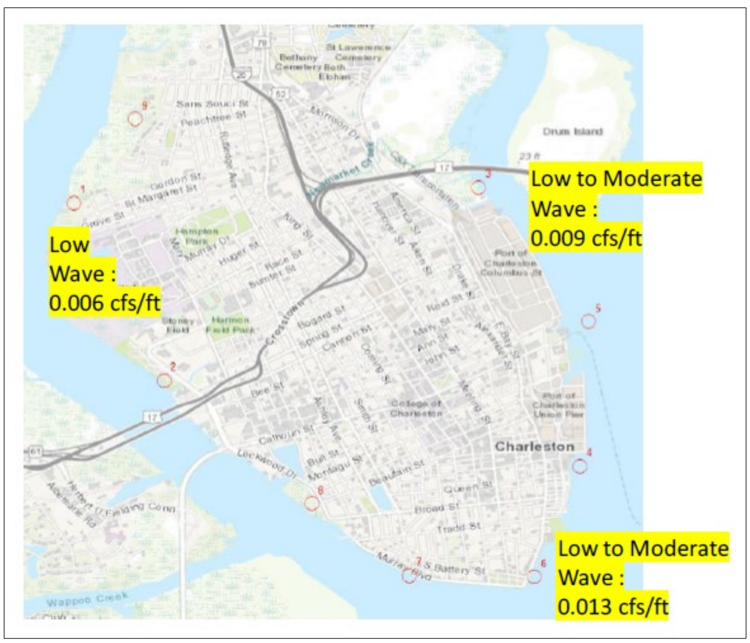

Figure 60. Overtopping flow along different reaches

The coastal modeler 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 62 shows the wave overtopping flow calculated at Station 6 (2% AEP). Station 6 is located near the Battery and is shown in Figure 60. 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 modeler 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 20, 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.

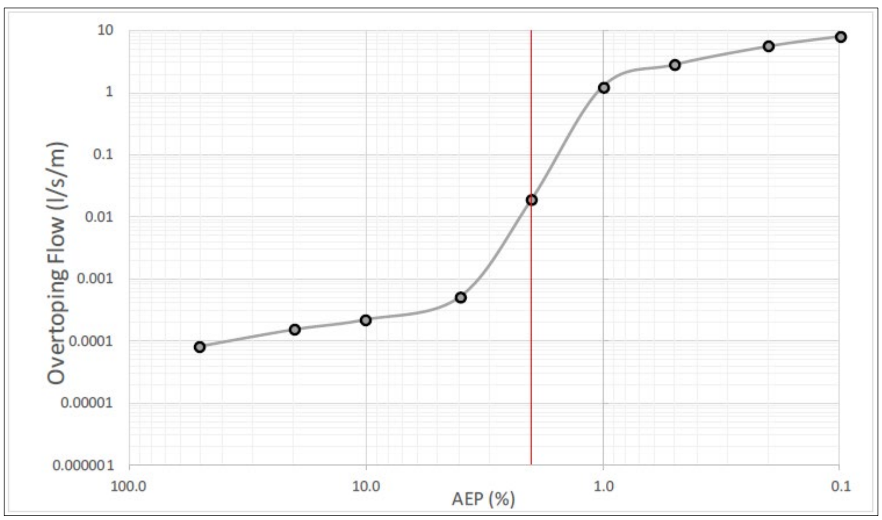

Figure 61. Overtopping Flow Calculated at Station 6

Figure 63 displays the duration of overtopping calculated at Station 6 as provided by the coastal modeler.

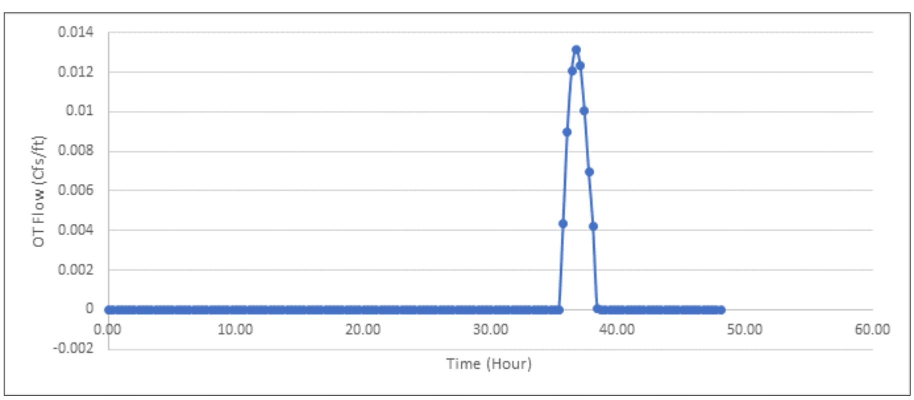

Figure 62. Duration of Overtopping Calculated at Station 6

4.5.4 Results for the Overtopping Analysis

Table 21 displays the WSEL for a future without-project condition during a storm surge event at the 10ft NAVD88 SWL. As seen in Table 21, most output locations do not see increases in WSEL much higher than 10ft NAVD88. This shows the rainfall has little to no effect on peak stages during a surge of this magnitude.

Selected Output Locations	FWO 10ft Exterior WSEL	
	10% AEP	1% AEP
	Peak Water Surface Elevation (ft. NAVD88)	Peak Water Surface Elevation (ft. NAVD88)
1	10.02	10.03
2	10.03	10.04
3	10.03	10.04
4	10.03	10.04
5	10.03	10.04
6	10.03	10.04
7	10.03	10.03
8	10.03	10.04
9	10.02	10.04
10	10.02	10.03
11	10.02	10.03
12	10.01	10.02
13	10.02	10.03
14	10.03	10.04
15	10.03	10.04
16	10.04	10.08
17	10.05	10.08
18	10.03	10.05
19	10.04	10.07
20	11.27	11.44
21	10.04	10.10
22	10.03	10.05
23	10.03	10.05
24	10.04	10.08

Table 21. FWO 10ft NAVD88 SWL with 10% AEP and 100% AEP Rainfall

Table 22 displays the results for the overtopping analysis for the future with-project during the event of a 10ft NAVD88 SWL with rainfall plus overtopping. The tables will show the with-project condition greatly decreases the interior water surface due to storm surge by approximately 1 to 5 feet depending on the output location for both a 10% AEP and 1% AEP rainfall events. The gates closed condition in this table assumes no PDT pumps.

Selected Output Locations	FW (gates closed) @ 10ft Exterior WSEL/-1.31ft Initial Interior WSEL			
	10% AEP Rainfall plus OT		1% AEP Rainfall plus OT	
	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	8.22	-1.80	8.89	-1.14
2	6.77	-3.26	8.35	-1.69
3	6.77	-3.26	8.35	-1.69
4	6.77	-3.26	8.35	-1.69
5	5.93	-4.10	6.57	-3.47
6	5.92	-4.11	6.52	-3.52
7	5.01	-5.02	5.87	-4.16
8	5.30	-4.73	5.97	-4.07
9	5.34	-4.68	6.00	-4.04
10	5.35	-4.67	6.01	-4.02
11	5.50	-4.52	6.01	-4.02
12	6.63	-3.38	7.14	-2.88
13	7.91	-2.11	8.32	-1.71
14	5.68	-4.35	6.17	-3.87
15	6.88	-3.15	7.84	-2.20
16	6.98	-3.06	7.86	-2.22
17	6.77	-3.28	8.35	-1.73
18	5.93	-4.10	6.56	-3.49
19	6.01	-4.03	6.45	-3.62
20	11.27	0.00	11.41	-0.03
21	7.95	-2.09	8.28	-1.82
22	6.67	-3.36	7.77	-2.28
23	6.88	-3.15	7.84	-2.21
24	6.98	-3.06	7.86	-2.22

Table 22. FW (gates closed) 10ft NAVD88 SWL with 10% AEP and 100% AEP Rainfall

Table 23 displays the results for the overtopping analysis for the future with-project during the event of a 10ft NAVD88 SWL with rainfall plus overtopping. The tables will show the with-project condition greatly decreases the interior water surface due to storm surge by approximately 1 to 6 feet depending on the output location for both a 10% AEP and 100% AEP rainfall. The gates closed condition in this table shows results for pump station alternative 2.

Pump alternative 2 is the focused alternative at this point in the study, therefore, only results for pump alternative 2 are displayed in this section of the report.

Selected Output Locations	FW (gates closed) P.S. alt 2 @ 10ft Exterior WSEL/-1.31ft Initial Interior WSEL			
	10% AEP Rainfall plus OT		1% AEP Rainfall plus OT	
	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	8.21	-1.81	8.72	-1.31
2	4.24	-5.79	7.50	-2.54
3	5.60	-4.43	7.50	-2.54
4	6.60	-3.43	7.50	-2.54
5	4.35	-5.68	6.38	-3.66
6	5.76	-4.27	6.35	-3.69
7	4.82	-5.21	5.56	-4.47
8	4.93	-5.10	5.68	-4.36
9	4.89	-5.13	5.68	-4.36
10	4.81	-5.21	5.68	-4.35
11	5.25	-4.77	5.69	-4.34
12	6.39	-3.62	6.92	-3.10
13	7.71	-2.31	8.07	-1.96
14	5.69	-4.34	6.08	-3.96
15	6.33	-3.70	7.30	-2.74
16	5.14	-4.90	7.13	-2.95
17	6.47	-3.58	7.50	-2.58
18	5.76	-4.27	6.38	-3.67
19	6.00	-4.04	6.45	-3.62
20	11.26	-0.01	11.44	0.00
21	7.95	-2.09	8.28	-1.82
22	5.93	-4.10	6.91	-3.14
23	6.33	-3.70	7.30	-2.75
24	6.27	-3.77	7.13	-2.95

Table 23. FW (gates closed) P.S. alt 2 for a 10ft NAVD88 SWL with 10% AEP and 100% AEP Rainfall

The following figures will display the hydrographs for two simulations: future with-project pump alternative 2 with a 10% AEP rainfall versus future with-project pump alternative 2 with a 10% AEP rainfall plus the wave wash overtopping. This additional analysis will provide insight into the performance of the pumps during rainfall events versus pumps during rainfall plus wave wash overtopping events. In other words, the wall greatly decreases the water levels on the interior due to a storm surge event up to the level of design of the wall, regardless of pump capacity. The primary goal is to assess the performance of the pump alternatives for various rainfall frequencies and then to assess those pump alternatives for rainfall plus wave wash overtopping. Therefore, the following figures will not show future without-project conditions because as shown in the previous tables in this section the wall does greatly decrease the interior water levels during such a surge event and the focus now is analyzing pump performance against wave wash overtopping.

The following hydrographs will show the overtopping does not significantly increase the peak water surface but does significantly increase the duration of flooding when we compare these two situations of rainfall only versus rainfall plus overtopping.

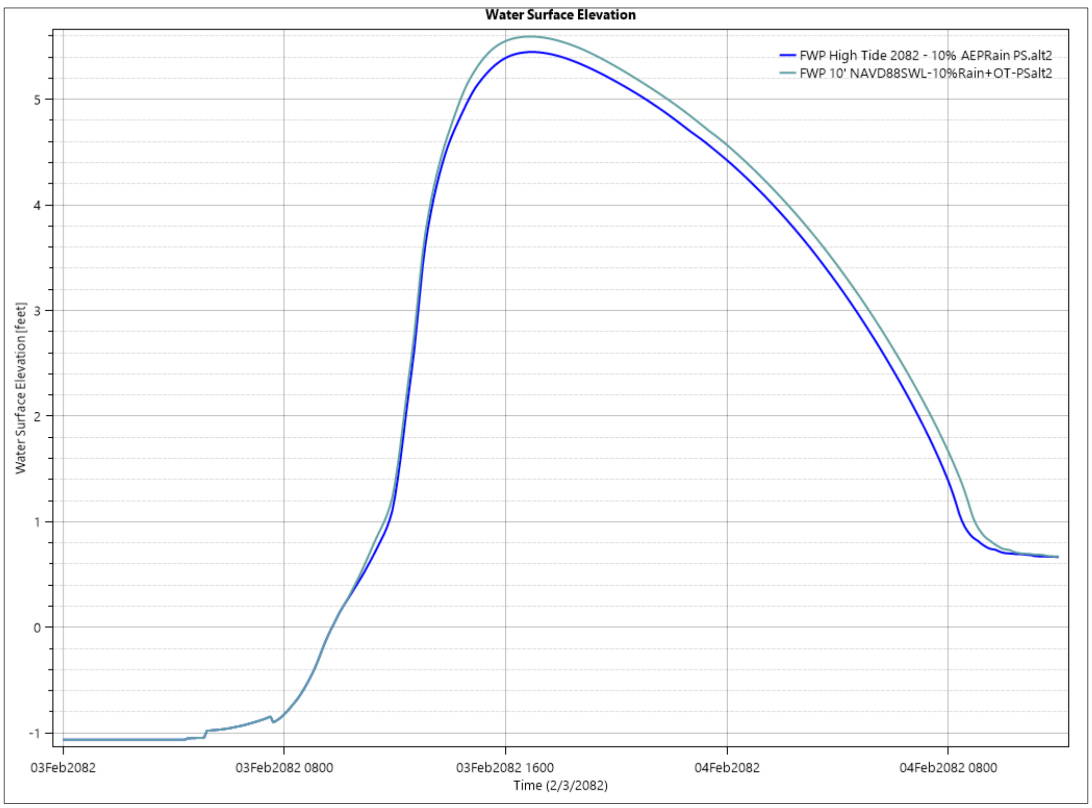

Figure 63. 10% AEP Rainfall versus 10% AEP Rainfall plus overtopping at output location #3 for pump station alternative 2

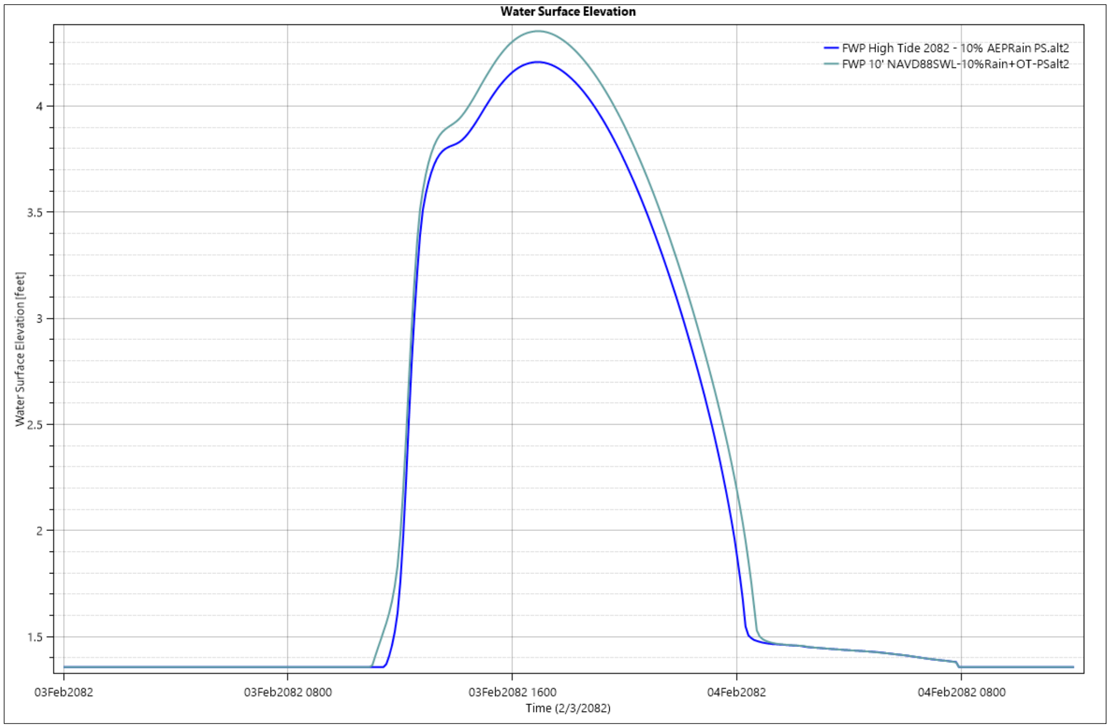

Figure 64. 10% AEP Rainfall versus 10% AEP Rainfall plus overtopping at output location #5 for pump station alternative 2

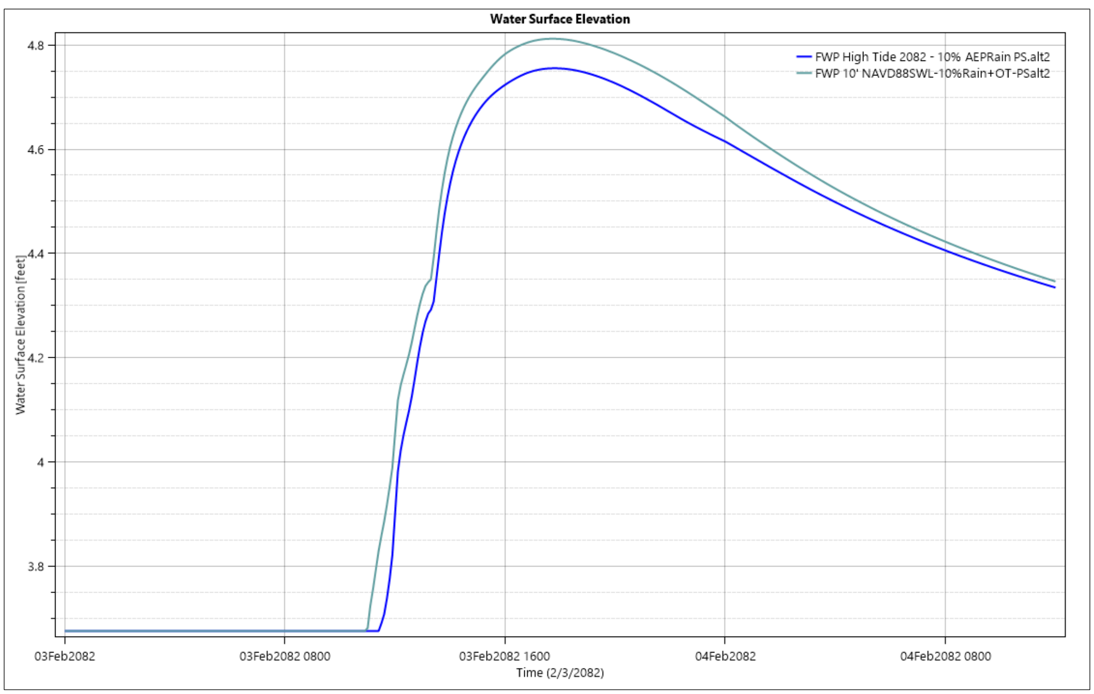

Figure 65. 10% AEP Rainfall versus 10% AEP Rainfall plus overtopping at output location #10 for pump station alternative 2

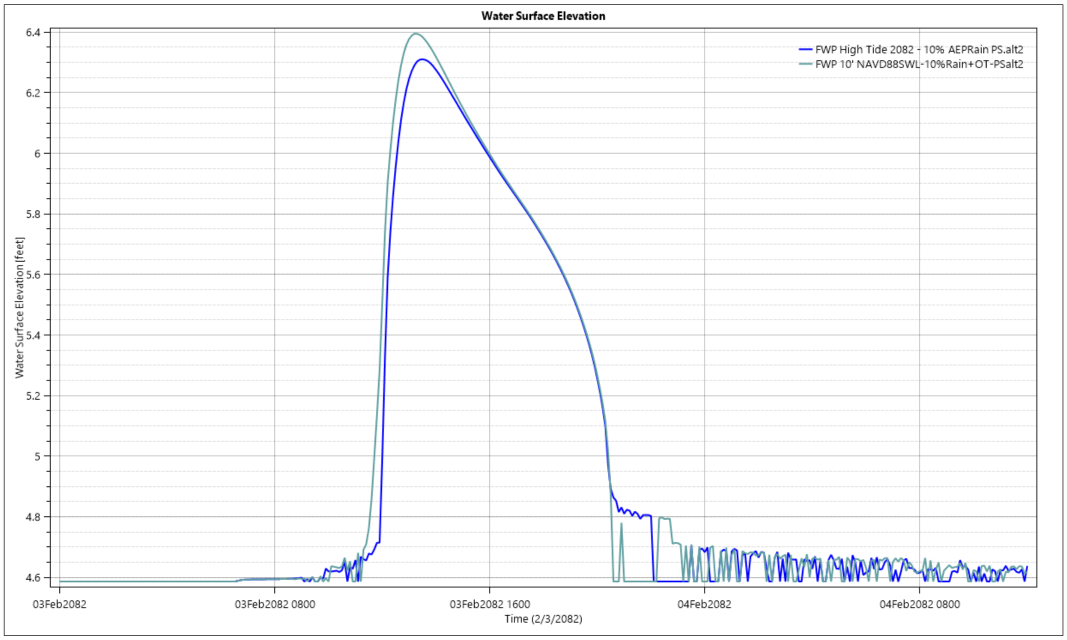

Figure 66. 10% AEP Rainfall versus 10% AEP Rainfall plus overtopping at output location #12 for pump station alternative 2

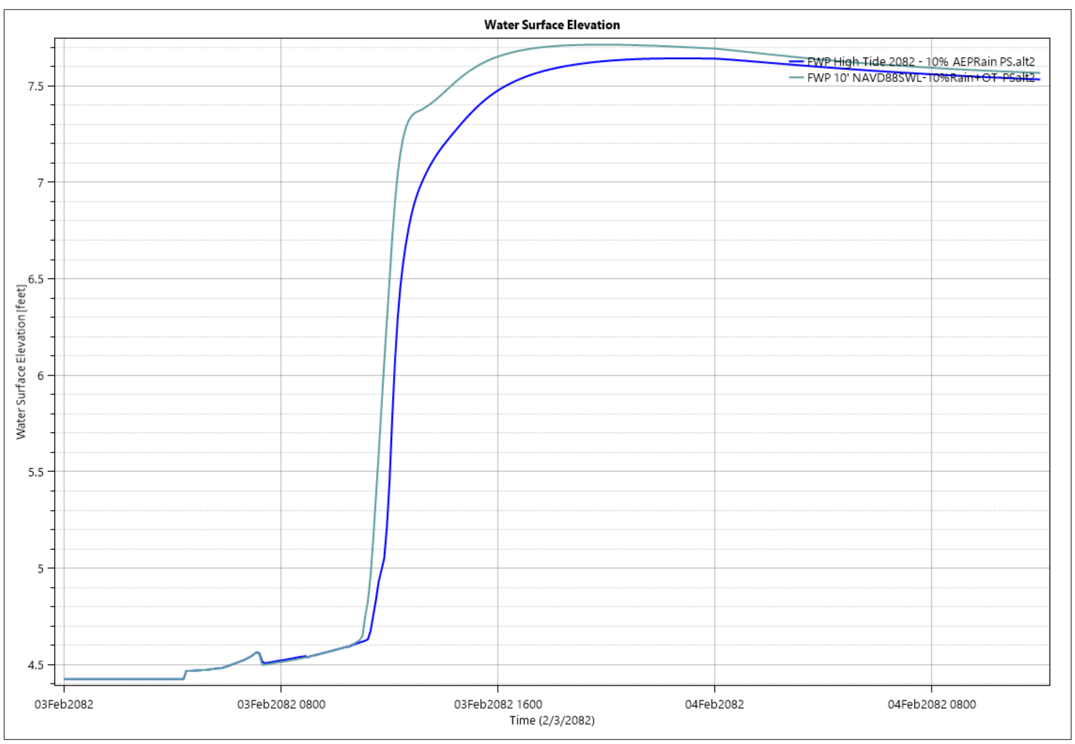

Figure 67. 10% AEP Rainfall versus 10% AEP Rainfall plus overtopping at output location #13 for pump station alternative 2

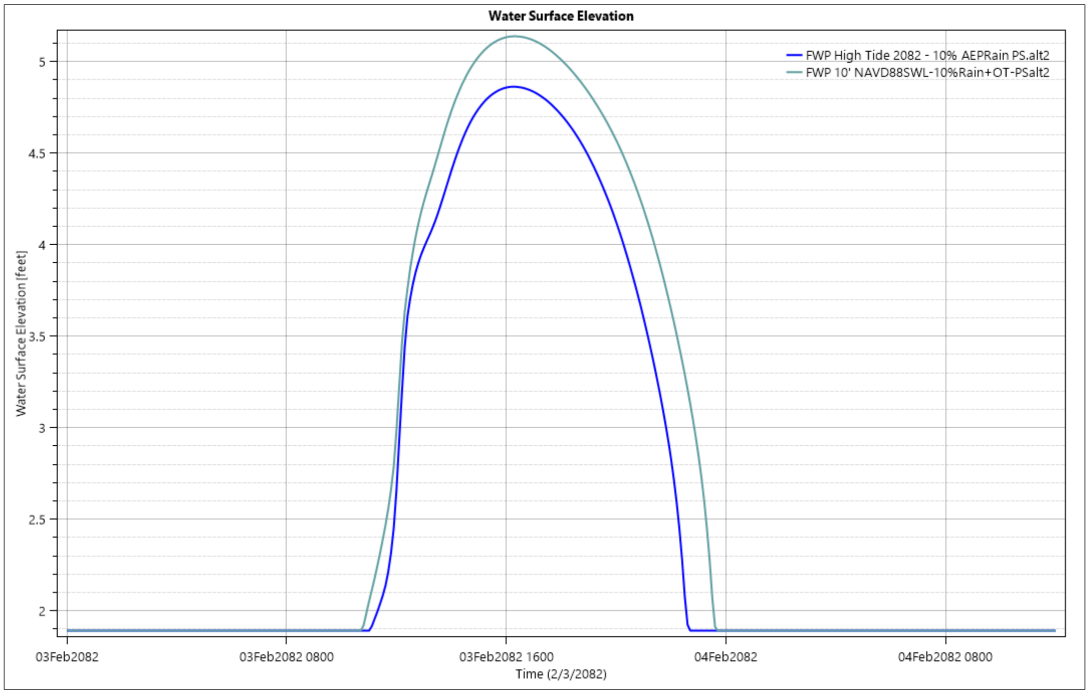

Figure 68. 10% AEP Rainfall versus 10% AEP Rainfall plus overtopping at output location #16 for pump station alternative 2

5. Additional Modeling (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 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.

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

Table 24. National Weather Service Flood Categories

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 70 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.

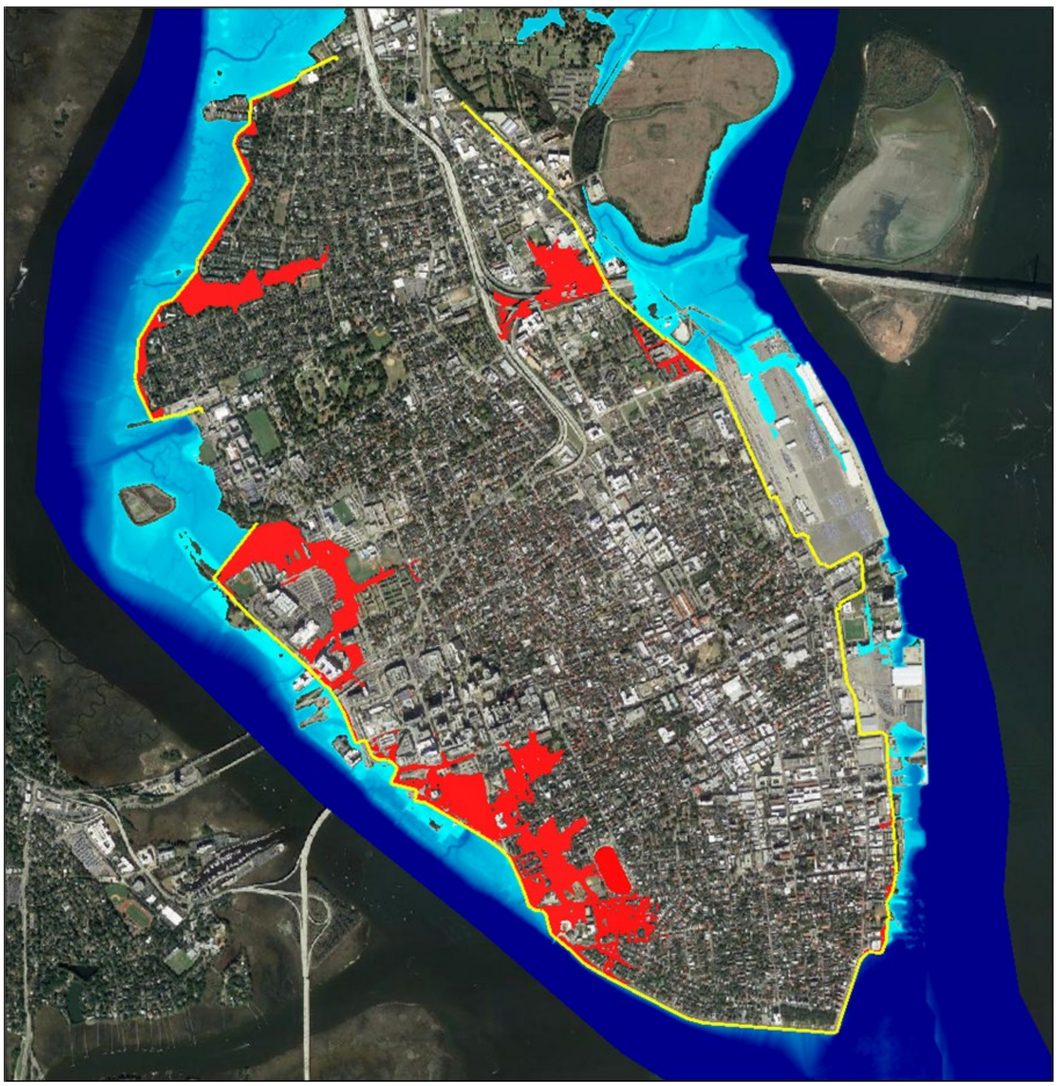

Figure 69. NWS Major Flooding Event FWO (Red) vs FW (Blue) Gates Closed

6. Assumptions and Limitations

- Vertical Datum used for modeling – NAVD88
- Relative Sea Level Change +0.56 feet for 2032 and +1.65 feet for 2082 (NOAA's 2006 Published Intermediate Rates)
- The City of Charleston will raise the Low Battery to match the High Battery. This elevation is assumed to be 9 feet NAVD88 which is approximately the mean elevation of the higher side of the wall currently.
- Storm surge is the primary concern of the peninsula while rainfall is mitigated via drainage infrastructure and pump stations. The interior hydrology analysis using HEC-RAS analyzes both impacts from storm surge and rainfall. Various alternatives of storm gates and pump stations were analyzed as mitigation measures for the conceptual design of the system.
- The purpose of the HEC-RAS modeling is to estimate hydraulic response of the overall system for various storm gate and pumping alternatives. Highly detailed inundation grids were extracted from the RAS results and used as inputs for the HEC-FDA modeling to determine the economic damage estimates for future without-project and future with-project conditions. Peak water surface elevations for FWO and FW conditions are provided at 24 selected output locations to provide a general sense of the overall impacts
- Subsurface drainage is a major component in designing urbanized drainage systems. RAS is unable to model subsurface conditions within the 2D modeling that was performed therefore the losses and storm drainage routing to pump stations were not accounted. Overland flow is the basis of modeling. Due to this aspect, water surface elevations as results of the RAS modeling may appear to be higher than expected or more conservative. This would be the case for both without project and with project conditions, therefore the scale of conservativeness would be the same for both with and without project conditions.
- The primary focus of design is the 10% AEP rainfall event. The City of Charleston has shared that most of their drainage infrastructure passes no more than a 10% AEP rainfall event and, in some cases, less than the 10% AEP rainfall event. This means the sub-surface pipe network becomes a smaller component of drainage during events of a 10% AEP and events greater than the 10% AEP. Therefore, the HEC-RAS surface flow only model becomes more representative of the stormwater movement within the system during these larger events
- The RAS model was computed for a known flood event, but there is essentially no gage data or high-water marks available on the interior of the peninsula to calibrate or validate the model. However, the RAS model will provide a relative comparison of impacts to water surface elevations between without project and with project conditions.
- Rainfall time-series data was provided by a City of Charleston contractor who developed a model for a locally proposed pump station called the Calhoun West Pump Station.
- Rainfall time-series data was applied uniformly across the entire 2D mesh. Rainfall intensity could vary spatially across the model area or peninsula.
- It is assumed the City of Charleston will complete the peninsula outfall check valve program which will prevent tidal flow into the storm drainage outfalls.
- An unofficially released version of HEC-RAS was used in this modeling effort. HEC-RAS 5.1 Alpha was used to operate pump stations within 2D modeling. The pumps were modeled with a 10-minute Startup Time and to operate maximum capacity throughout the simulation once the 10-minute Startup Time was achieved. The total operation is more complex and would depend on factors such as the head differential between interior and exterior, pump efficiency curves, and other complicated factors. In reality, the pump

would not operate at maximum capacity throughout a storm because the head differential would not remain constant which would impact the pump efficiency curve.

- Pre-storm water-level drawdown was assumed in the modeling. This means the storm gates were closed at low tide, prior to the arrival of the storm. In many ways, this operation could improve the performance of the system as a storm arrives and reduce the likelihood of adverse impacts within the interior. However, this pre-storm drawdown contains uncertainty for different reasons but primarily the absent of subsurface modeling.

- The pre-drawdown assumption may impact pump performance in low lying, tidal creek areas more so than pumps in higher land elevation areas such as on the east side of the peninsula. A RAS 2D model that does not include the drainage pipe infrastructure could potentially under predict the flow rate of water to low points on the peninsula where some of the pump stations will be sited. An under-prediction in drainage flow rates could lead to an under-designed pump station that may face increased demand, in reality, as to that of the computed flow rates of a purely surface flow model to maintain a design level of service within these low-lying areas. At this time, it is assumed the pumps will tie into the City's pipe network and further analysis will be conducted in PED phase.

7. Conclusions at Feasibility Phase

- HEC-RAS simulations were computed for existing conditions, future without-project, and future with-project conditions for various storm gate and pumping alternatives for the 50%, 20%, 10%, 4%, 2%, and 1% AEP rainfall events.

- Three storm gate alternatives were evaluated with various gate dimensions. Three pump station alternatives with various total capacities were evaluated.

- The alternative which results in the smallest increase in interior water levels and economic damages for the storm gate alternative when analyzing the 10% AEP rainfall event is alternative 3. Storm gate alternative 3 has the largest gate dimensions at each location. Storm gate alternative 3 was selected to be included within the project cost estimate.

- The pump alternative analysis is highly complex. In comparison to future without-project conditions some locations may show reductions in water levels, some may show similar water levels, and some may show increases in water levels. Based on the iterative modeling process conducted so far, pump station alternative 2 was selected to be included within the project cost estimate.

- As mentioned in section 4.2 while discussing FDA, the alternatives were not mixed and matched to analyze which alternative performed most efficiently at each location. For example, all pump stations within pump alternative 2 may not provide the adequate pumping capacity needed at each location but as a system as a whole pump alternative 2 provided a reduction in estimated average annual damages as compared to that of the future without-project. The mixing and matching of alternatives will assist in analyzing the alternatives on a site-by-site basis which will assist in conceptually design the system in PED phase.

- The HEC-RAS modeling has provided insightful information to the PDT to make informed decisions in the conceptual design of the system both for storm gate locations and dimensions and pump station locations and capacities.

References

Engineering Manual

- EM_1110-2-1413_Hydrologic Analysis of Interior Areas

Webpages

- NOAA Tides and Currents. https://tidesandcurrents.noaa.gov/stationhome.html?id=8665530.

- National Weather Service – AHPS. https://water.weather.gov/ahps2/hydrograph.php?wfo=chs&gage=chts1

Appendix

The appendix section of the report will display other relevant information to the study.

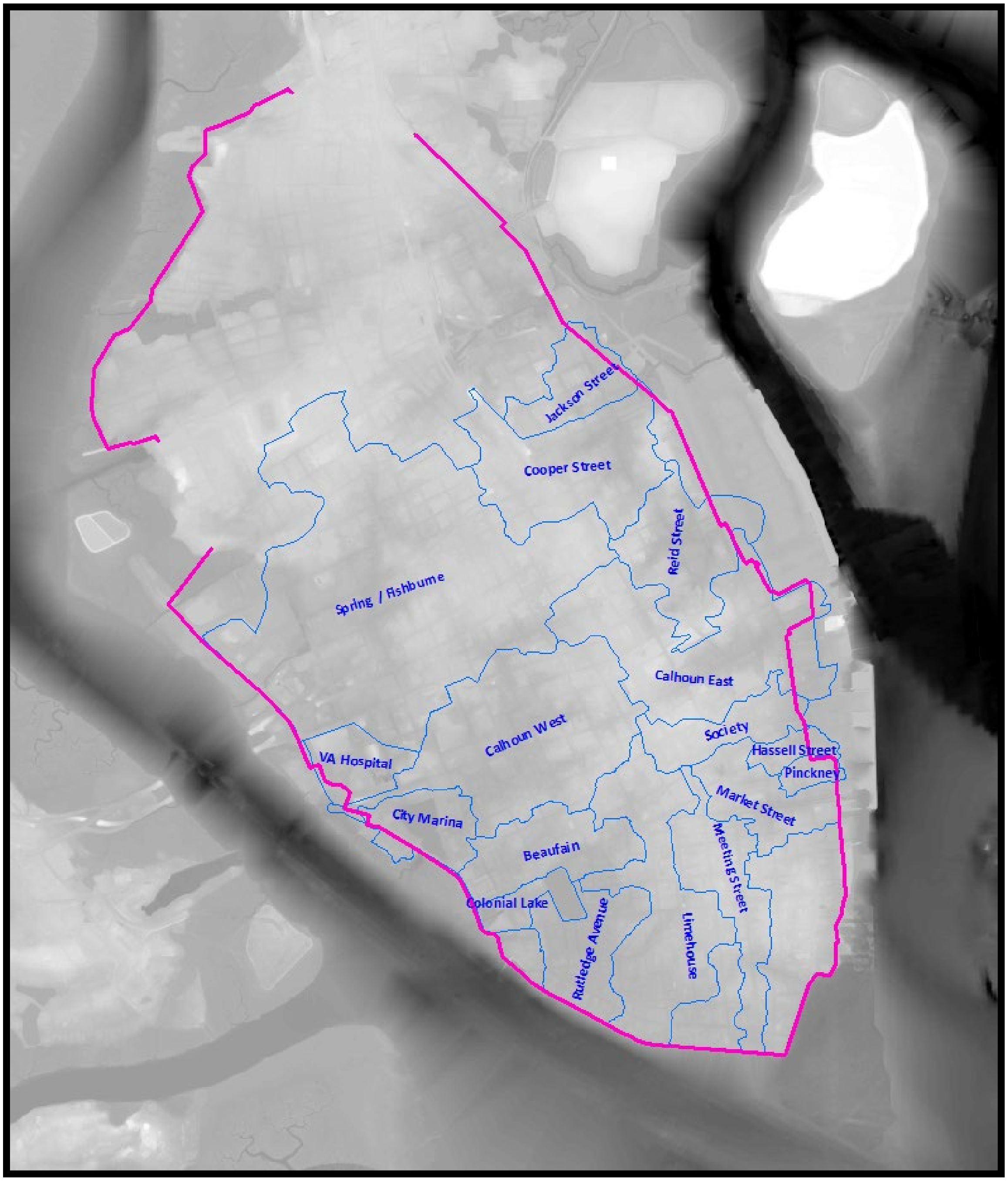

Figure 70. Original Delineation provided within the City of Charleston GIS Catalogue

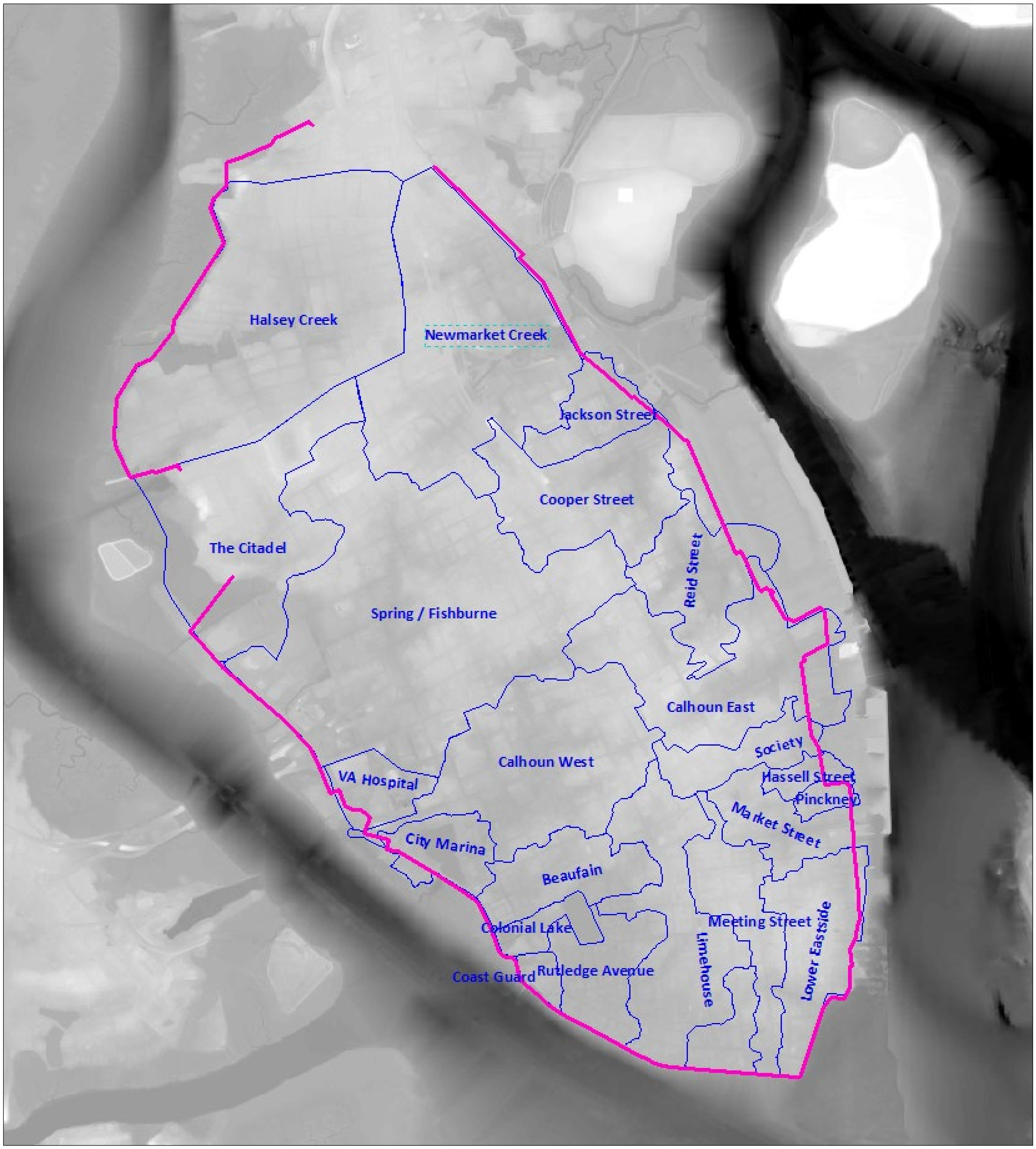

Figure 71. Revised Delineation for PDT Study for Informational Purposes

Drainage area	Area (Acres)
Beaufain	72.14
Calhoun East	156.22
Calhoun West	210.15
City Marina	46.16
Coast Guard*	11.96
Colonial Lake	23.34
Cooper Street	130.88
Halsey Creek*	380.44
Hassell Street	15.98
Jackson Street	43.83
Limehouse*	176.42
Lower Eastside *	94.89
Market Street	57.04
Meeting Street	76.28
New Market Creek*	300.19
Pinckney	11.68
Reid Street	91.78
Rutledge Avenue	65.62
Society	47.98
Spring/Fishburne	521.69
The Citadel*	209.89
VA Hospital	51.08

Table 25. Drainage Areas by Size for Revised Delineation

The Drainage areas marked with an asterisk are the ones that were added/revised from the original shapefile layer.

Figure 78 is the same as Figure 10 shown earlier in the report. This figure is placed here for a quick reference to the following tables and figures in this section. Table 26 and Figures 79 through 84.

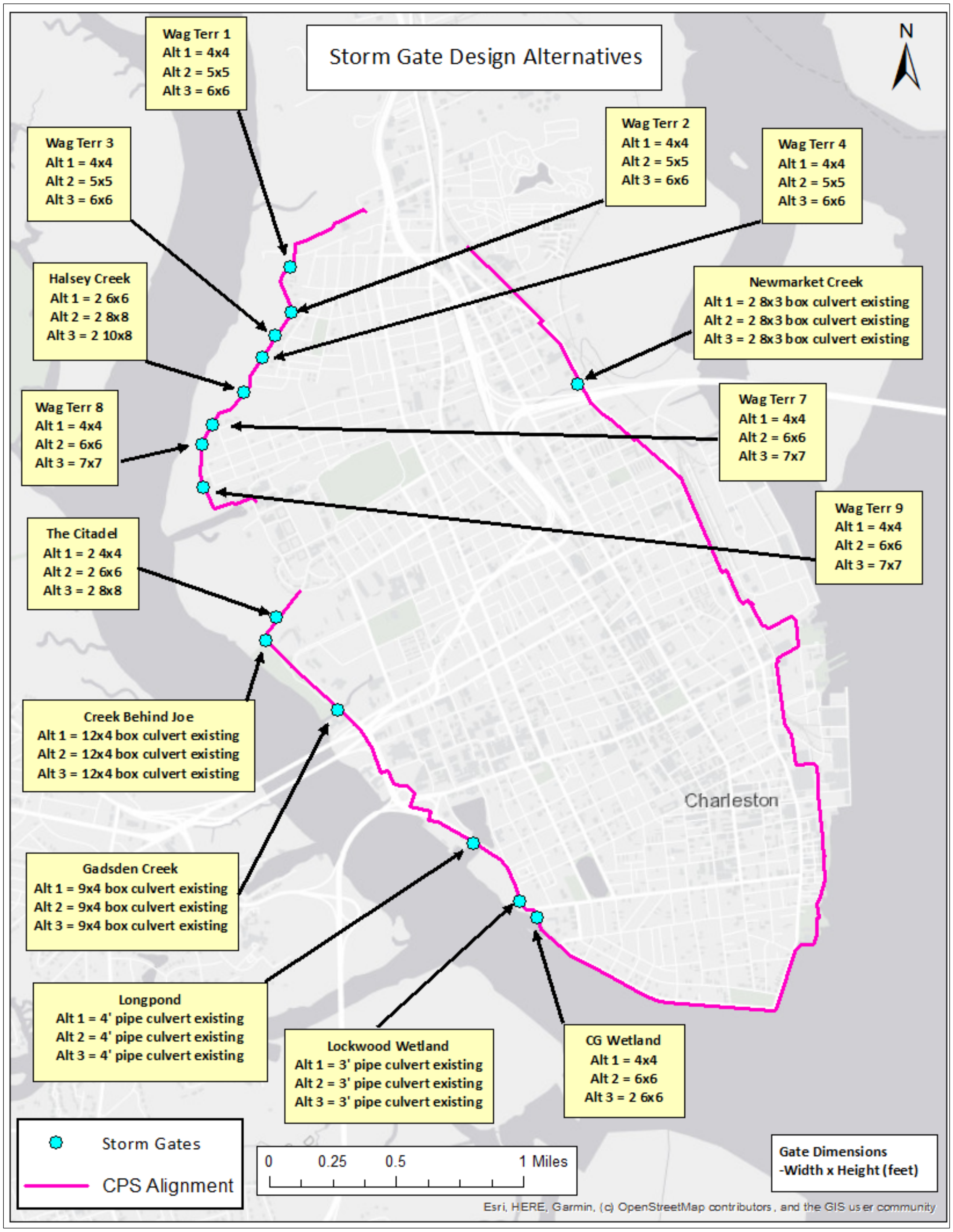

Figure 72. Storm gate alternatives

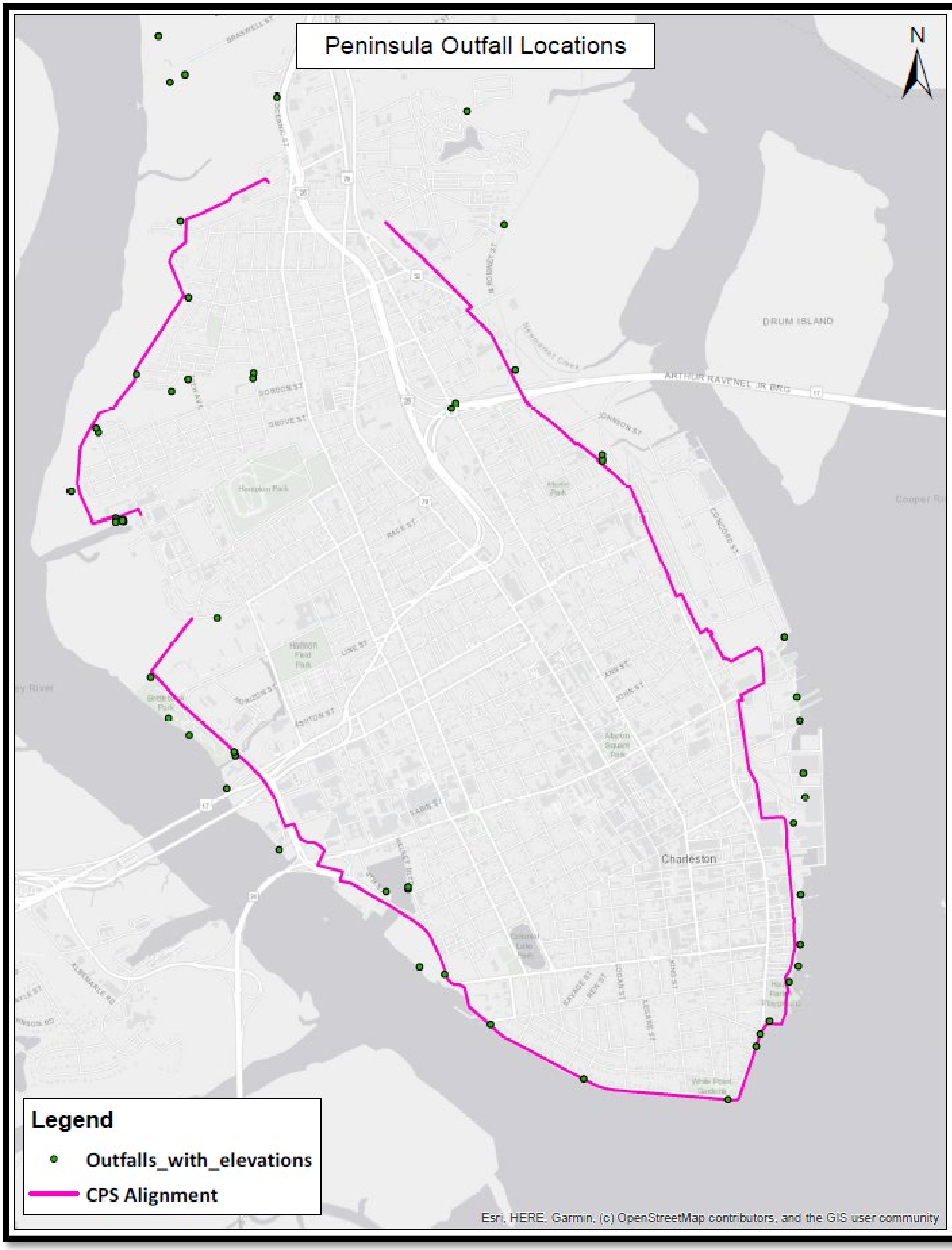

Figure 73. Peninsula Outfall Locations

Table 27 displays the peak flow rate (cfs) through each storm gate and culvert with the conditions at high tide (3.18 ft. NAVD88) in the year 2032 for a 10% AEP rainfall event. This will display peak flow rates for storm gate alternative 3.

Storm Gate	Storm Gate Alternative 3 (cfs)
Wag Terr 1	20
Wag Terr 2	13
Wag Terr 3	19
Wag Terr 4	19
Halsey Creek	325
Wag Terr 7	39
Wag Terr 8	56
Wag Terr 9	27
The Citadel	131
Creek Behind Joe	102
Gadsden Creek	144
Longpond	76
Lockwood Wetland	23
CG Wetland	26
Newmarket Creek	122

Table 26. Peak flow rate through storm gate drainage structures for 10% AEP Rainfall event in 2032

***All hydrographs displayed in this report will show the x-axis of time in the year 2082. This is for modeling simplicity purposes only by keeping all model simulation times in the same year, therefore do not be confused in the following sections when a hydrograph referencing the year 2032 contains an x-axis of time in the year 2082.

The following figures will display hydrographs through the storm gate drainage structures for the 10% AEP rainfall occurring in the year 2032 with a high tide of 3.18 ft. NAVD88.

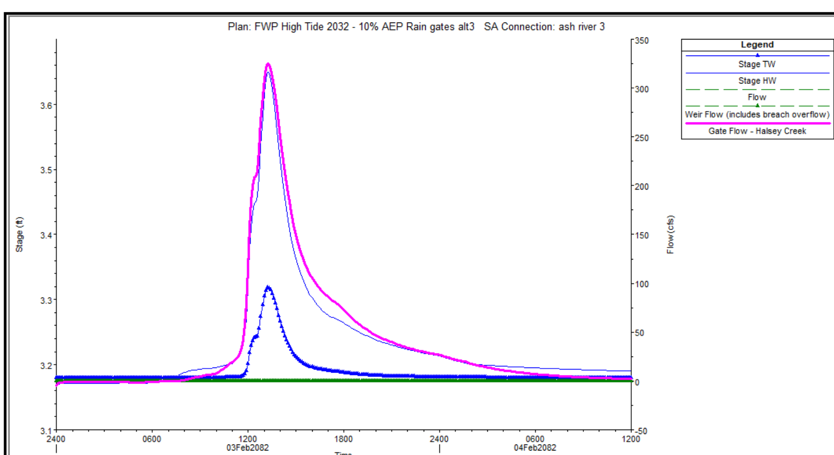

Figure 74. Flow rate through the Halsey Creek storm gate for 10% AEP Rainfall

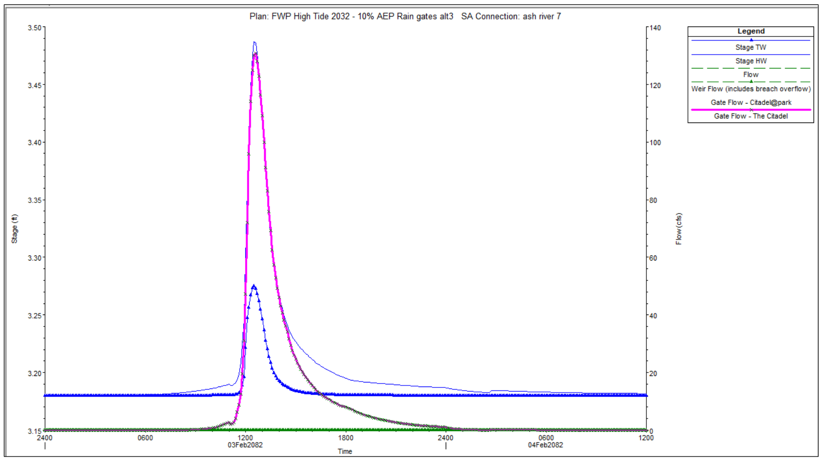

Figure 75. Flow rate through the Citadel storm gate for 10% AEP Rainfall

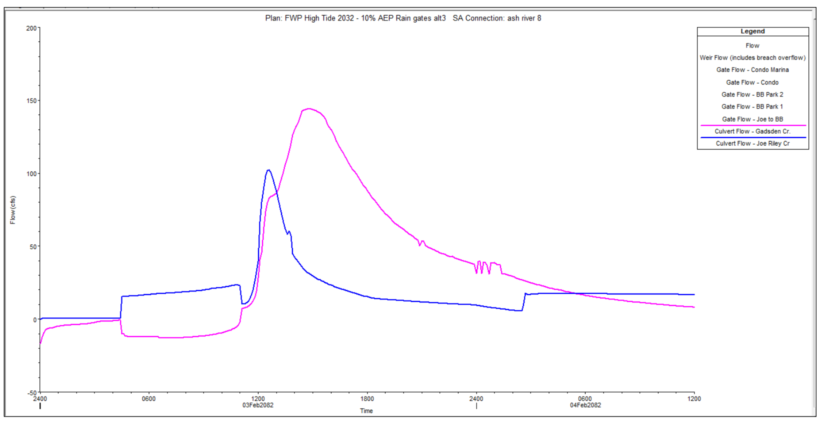

Figure 76. Flow rate through the Gadsden Creek and Joe Riley culverts for 10% AEP Rainfall

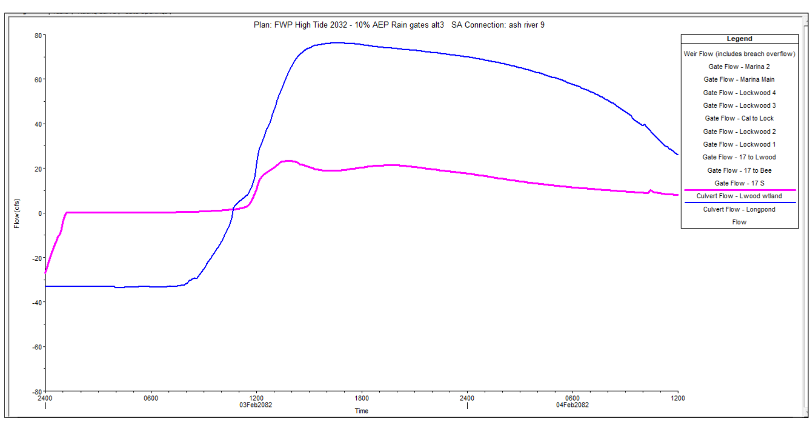

Figure 77. Flow rate through Longpond and Lockwood Wetland culverts for 10% AEP Rainfall

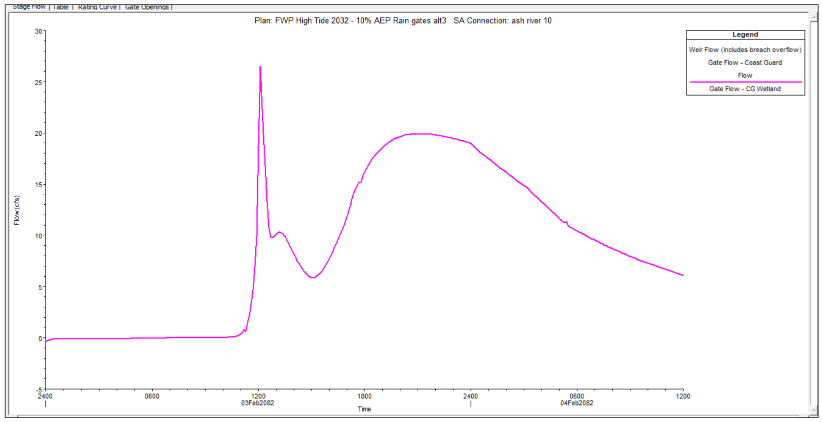

Figure 78. Flow rate through CG Wetland storm gate for 10% AEP Rainfall

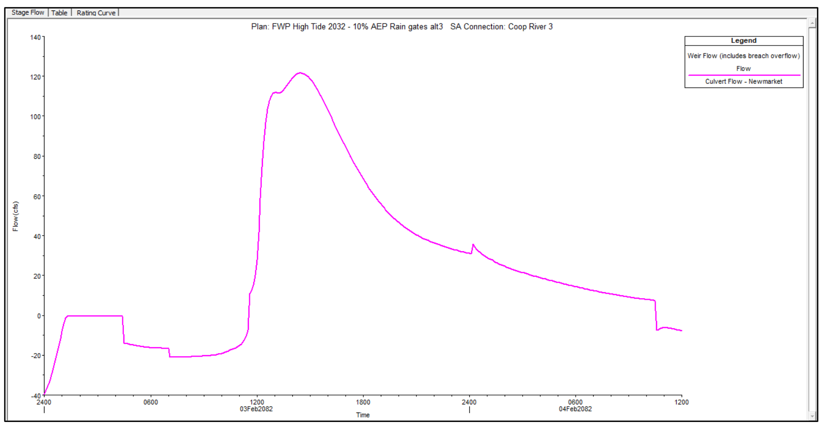

Figure 79. Flow rate through Newmarket Creek Culvert for 10% AEP Rainfall

US Army Corps
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Charleston District

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

Charleston, South Carolina

COASTAL SUBAPPENDIX B- 4

AUGUST 2021

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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 1.1.1). 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.

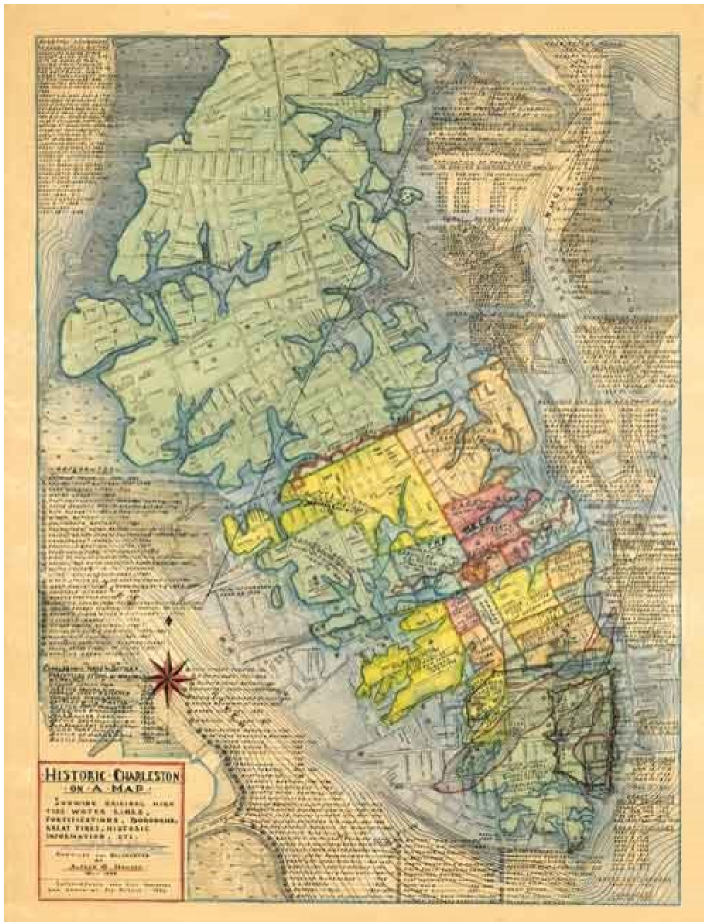

Figure 1.1.2 Halsey Map of 1844

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.

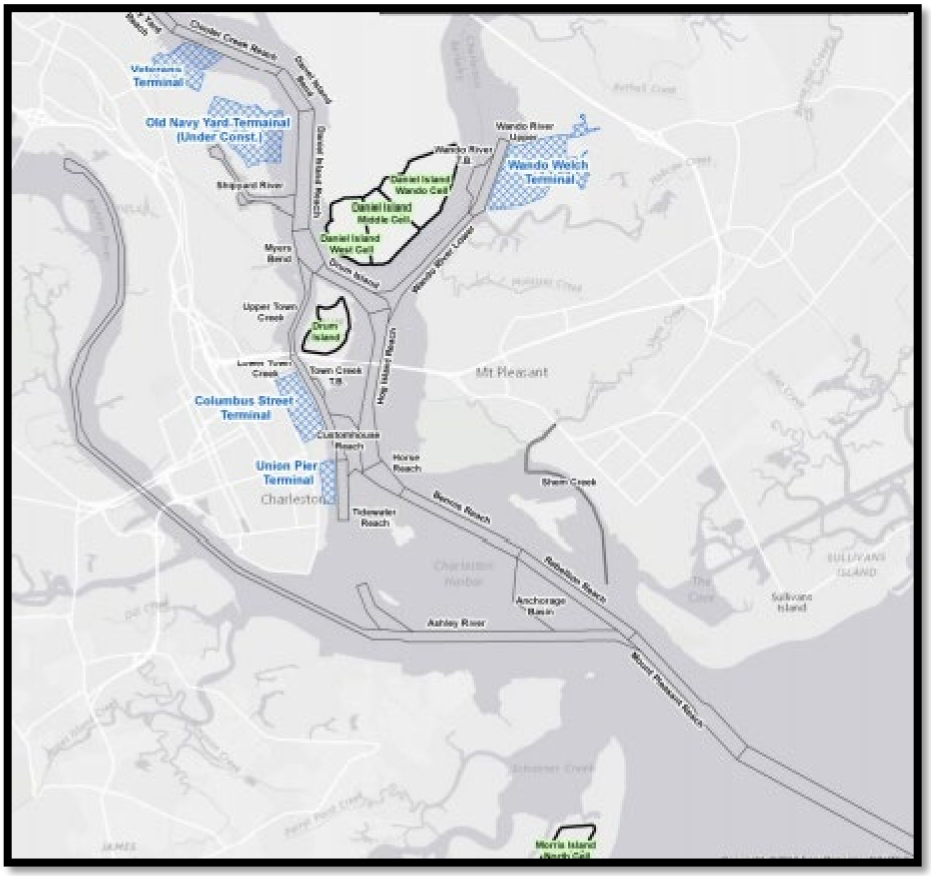

Figure 1.1.3 Charleston Harbor Navigation Channel

1.2. NOAA COOPER RIVER ENTRANCE TIDAL 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. Shown in Figure 1.2.1.

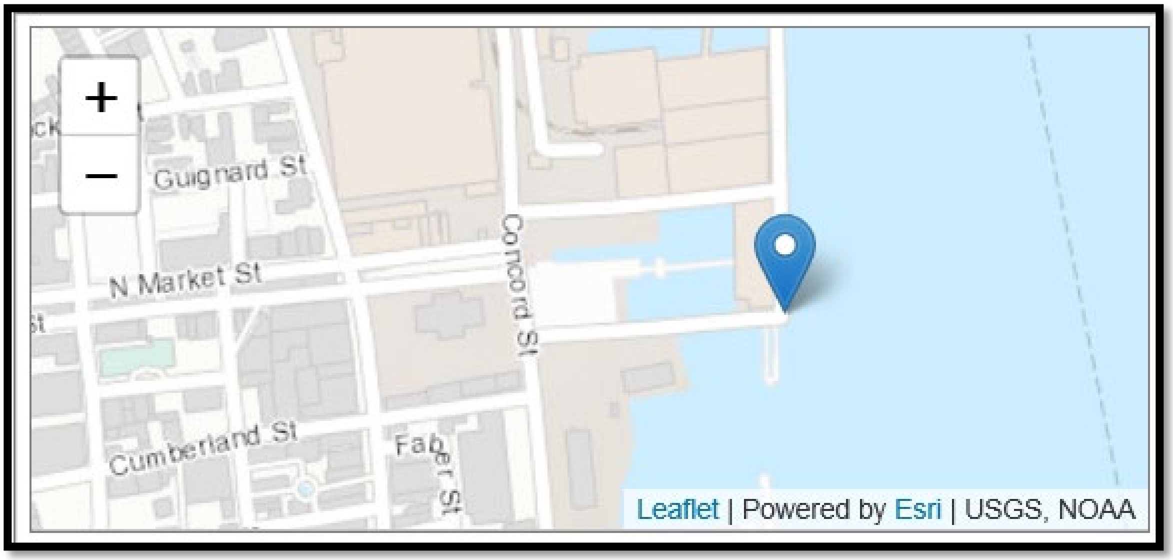

Figure 1.2.1 Location of NOAA Gage 8665530

Datum information provided by NOAA on their Tides and Currents website indicate a tide range of 5.76 feet (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.

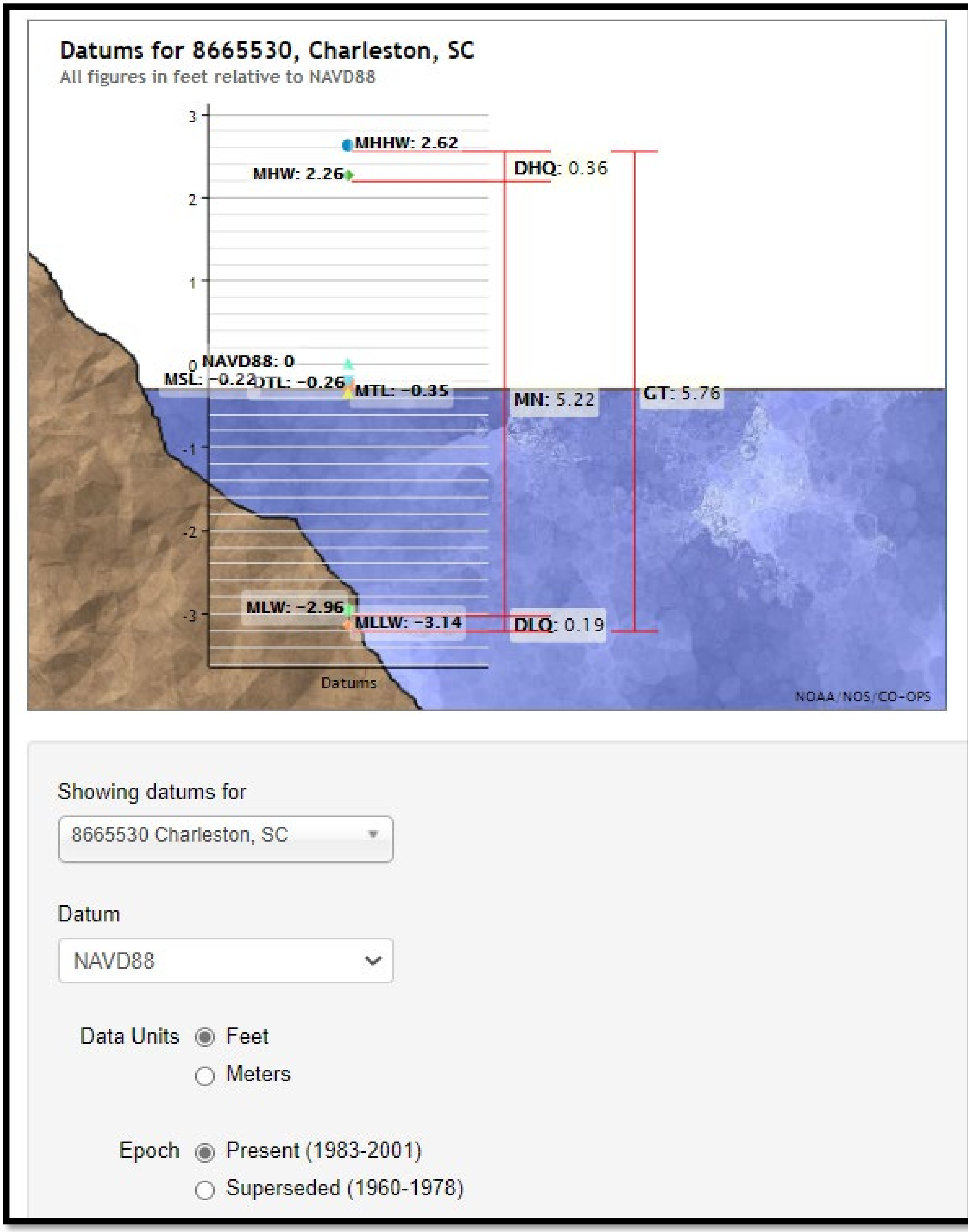

Figure 1.2.2 Tide Range Station 8665530 (Epoch 1983-2001)

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.

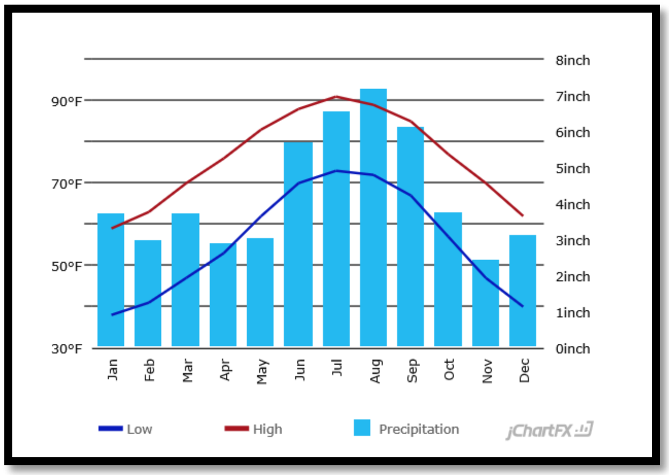

Figure 1.3.1 Charleston Temperature and Precipitation

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 2900). Distances are I 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).

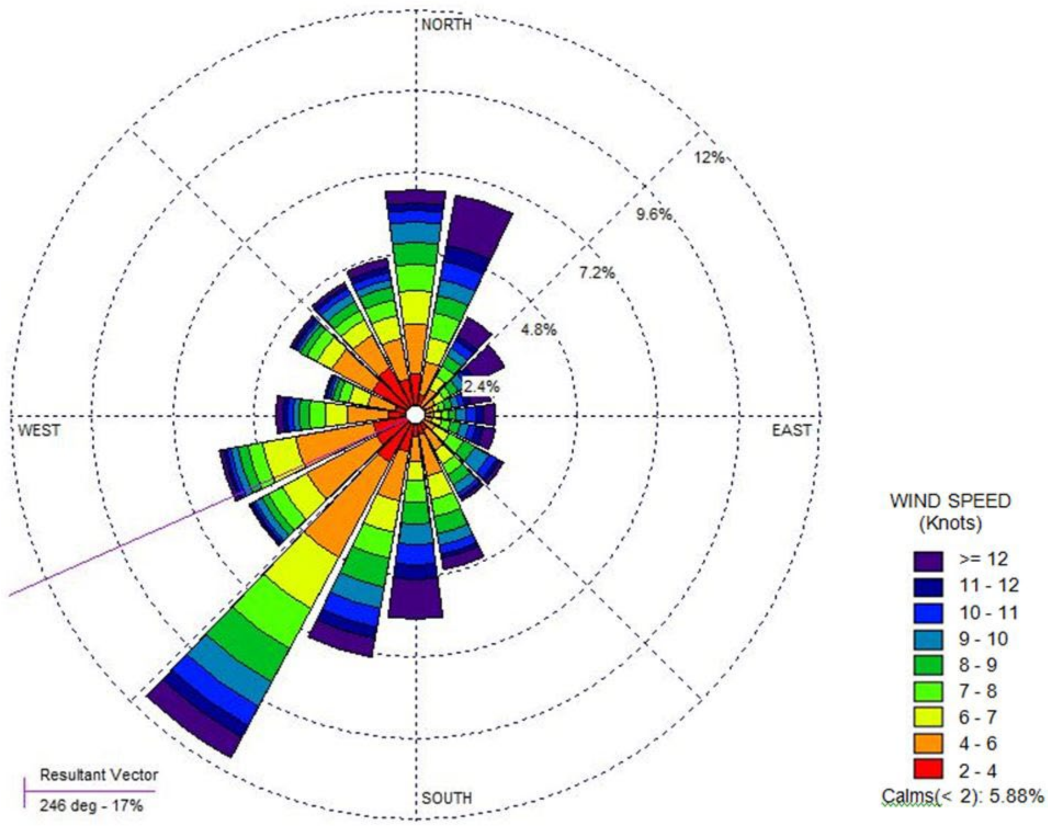

Figure 1.5.1. Wind Rose for Charleston Harbor Depicting Wind Direction and Speed Frequency

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.

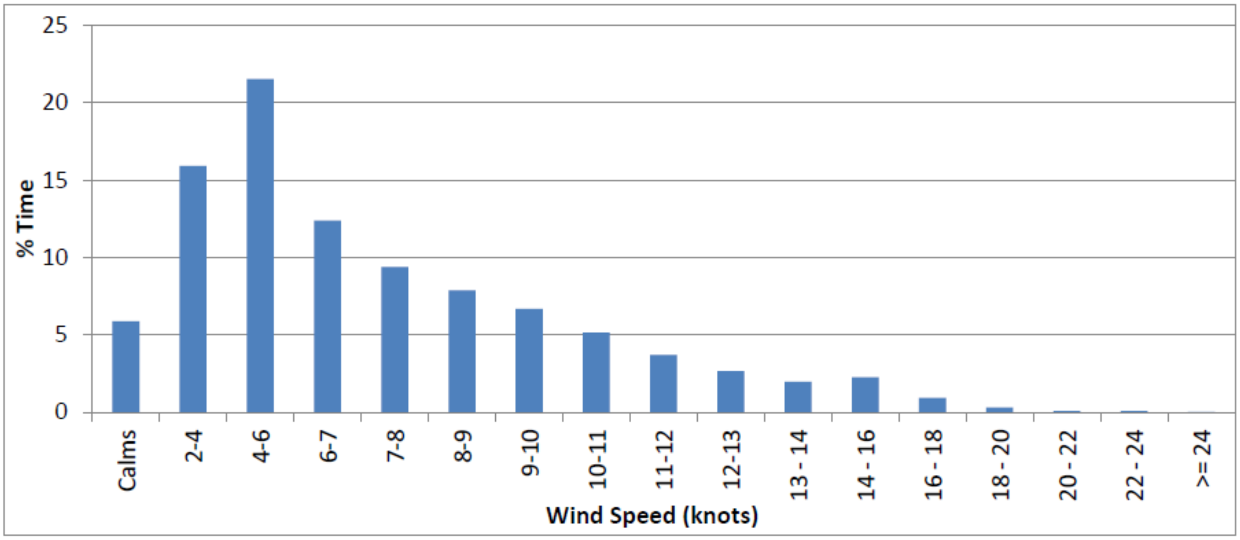

Figure 1.5.2 Wind Speed Frequency Distribution in Charleston Harbor from all directions

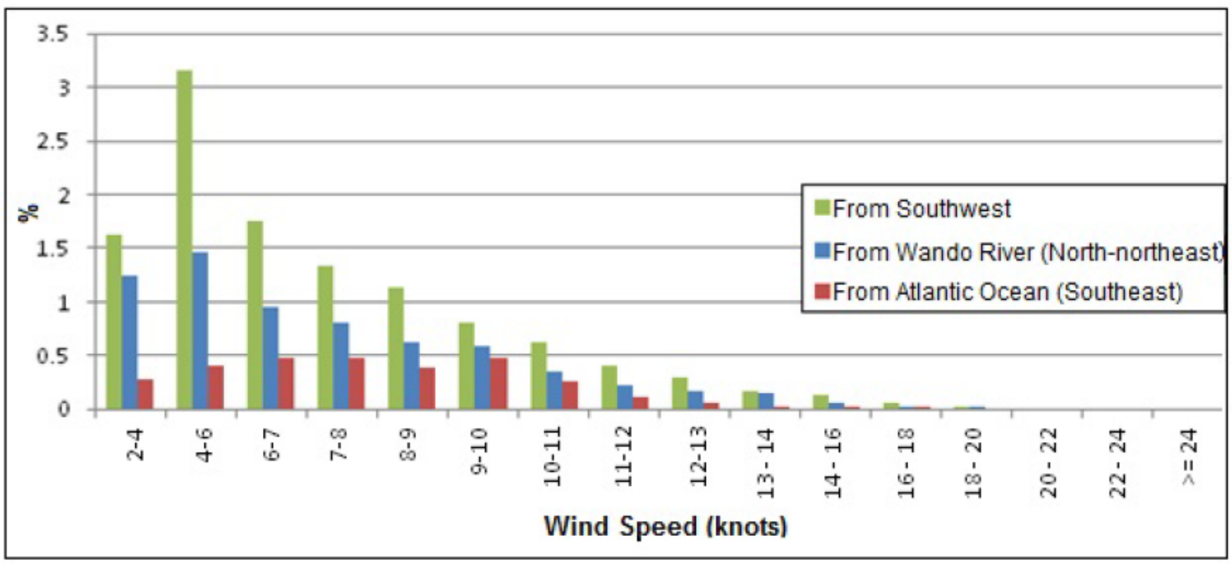

Figure 1.5.3 Wind Speed Frequency Distribution in Charleston Harbor comparing three key directions

1.6 ASTRONOMICAL TIDES & WATER LEVELS

1.6.1 Astronomical Tides

The Cooper River Entrance Tidal Gage (8665530), or the Charleston Harbor or Custom's House gage is the most 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).

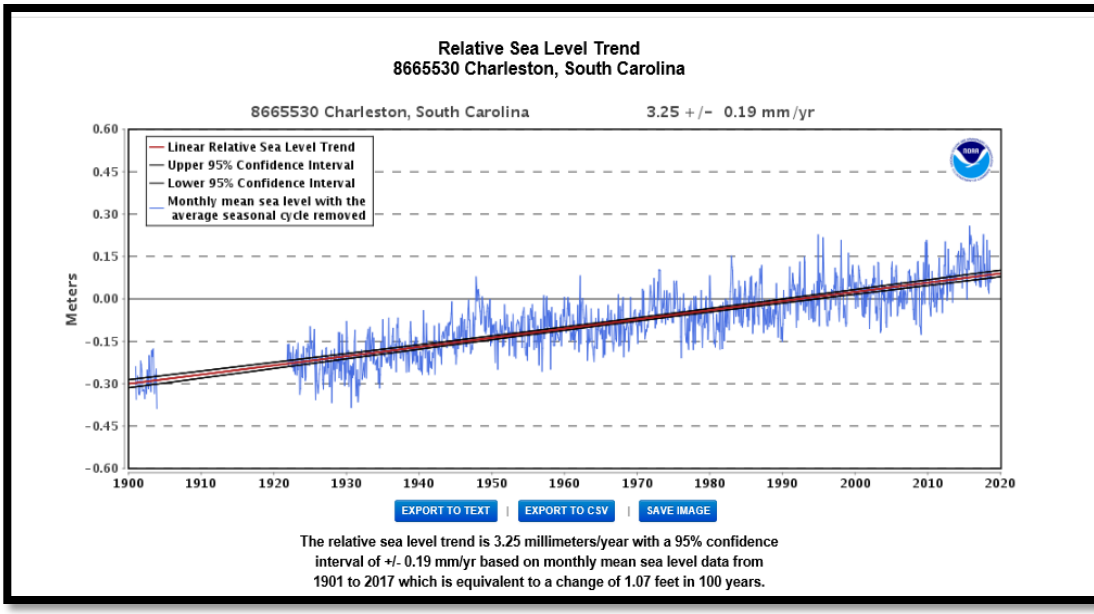

Figure 1.6.2.1 Observed Sea Level Change at Charleston Harbor Gage

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. Figure On the right are projected exceedance probability levels and tidal datum assuming continuation of the linear historic trend.

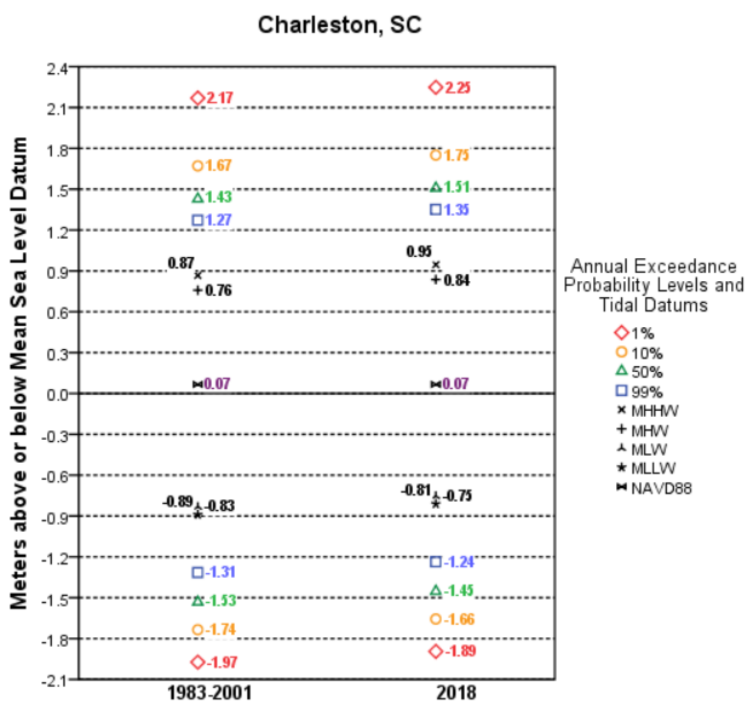

Figure 1.6.2.2 Exceedance Probability Levels and Tidal datum of 8665530 Charleston, Cooper River Entrance, SC

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)

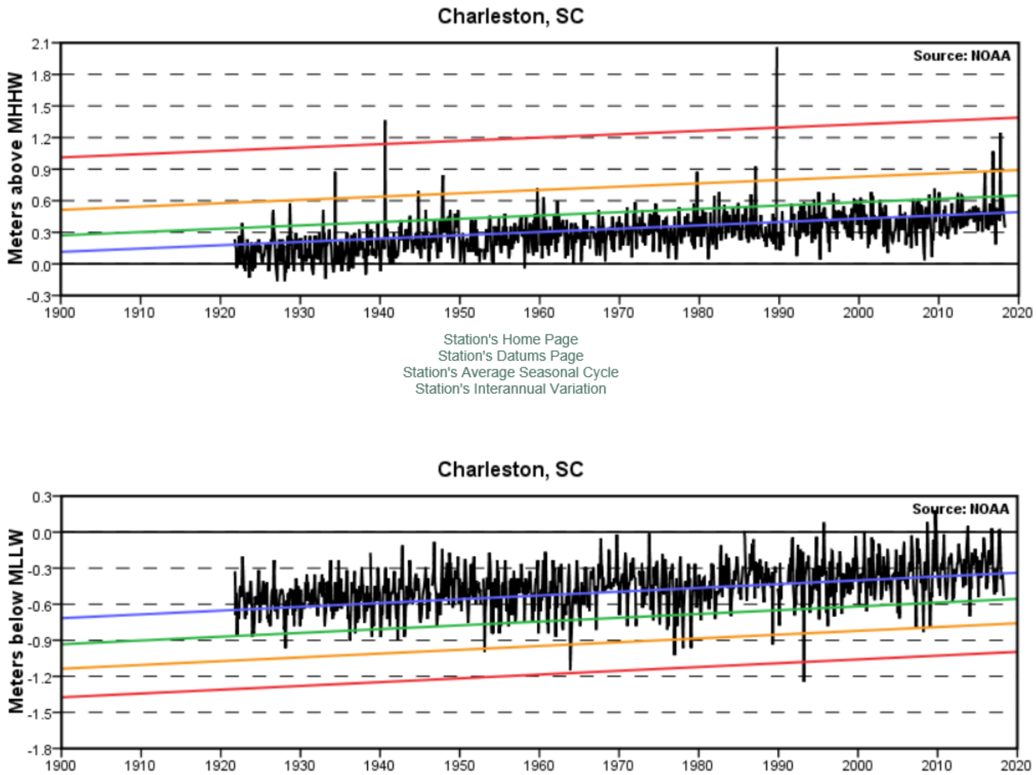

Figure 1.6.2.3 Seasonal and Interannual Variation of Gage 8665530 Extreme water Levels

Table 1.6.2.1 Extreme Water levels and Tidal datum of 8665530 Charleston, Cooper River Entrance, SC

Version of Data :	05/17/2017
ID:	8665530
Reference Datum:	NAVD88
Name:	Charleston, SC
HAT:	4.12 (ft)
MHHW:	2.62 (ft)
MHW:	2.27 (ft)
MSL:	-0.22 (ft)
MLW:	-2.95 (ft)
MLLW:	-3.14 (ft)
NAVD88:	0.00 (ft)
EWL Type:	NOAA GEV (NAVD88)
EWLs adjusted to 2019 using the historic rate.	
*1% :	7.18 (ft)
2%:	6.59 (ft)
5%:	5.95 (ft)
10%:	5.54 (ft)
20%:	5.18 (ft)
50%:	4.75 (ft)
Yearly:	4.23 (ft)
Monthly:	NaN (ft)
From:	1921
To:	2007
Years of Record:	86

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).

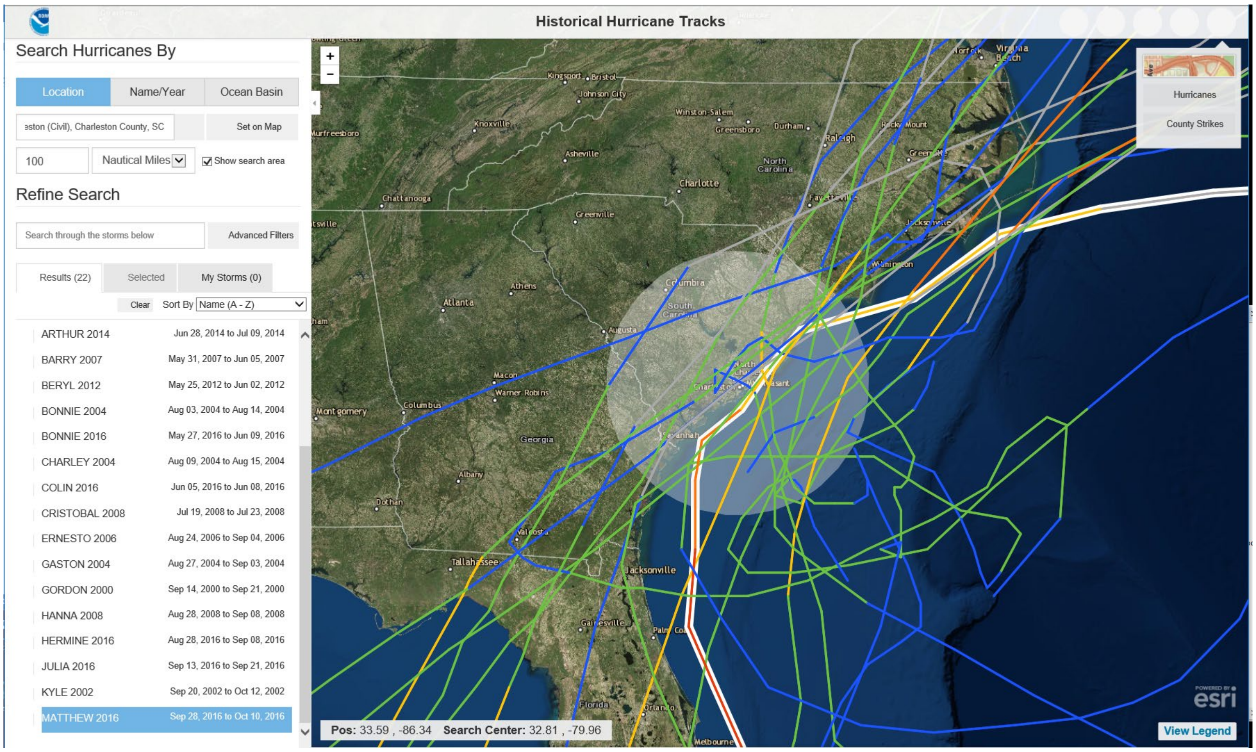

Figure 1.7.1.1 Twenty two storms passed within 100 nautical miles of Charleston between 2000 and 2019.

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 led to many fallen trees and some power outages.

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 regards 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.

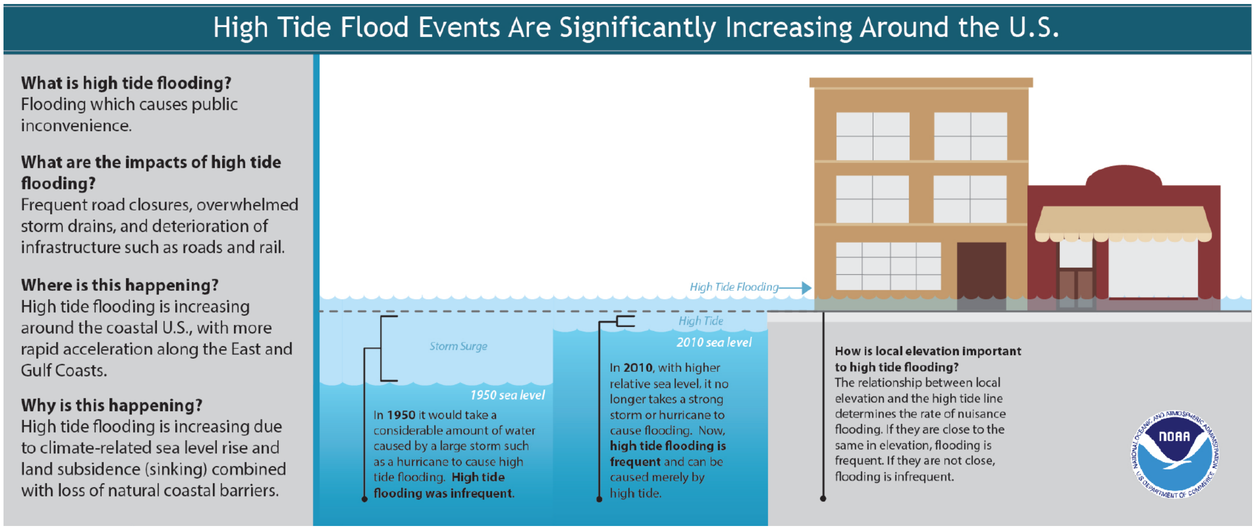

Figure 3.1.1. High Tide Flooding

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, 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.

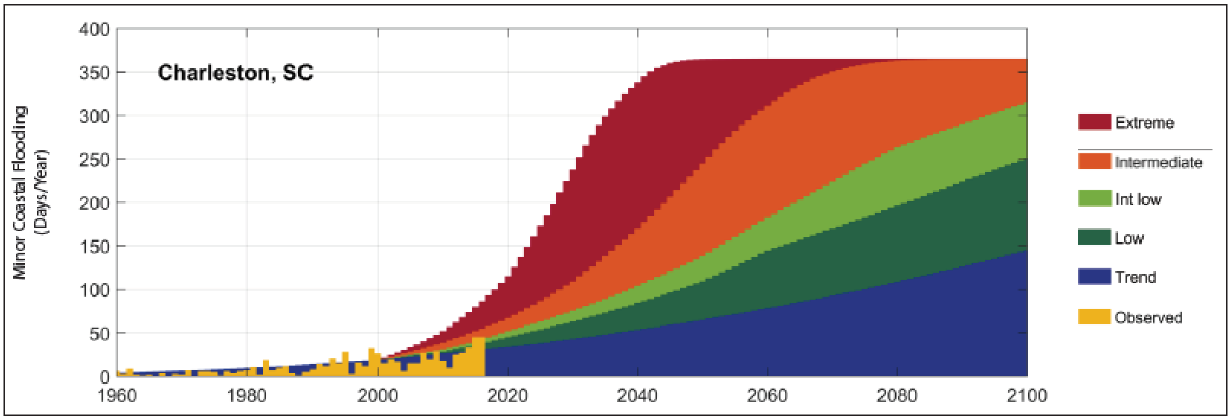

Figure 3.1.2. Observed and Predicted "Minor Coastal Flooding" in Charleston

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 Gage 8665530

3.3.1 Historic Rate

The historic rate of future RSLC (or USACE Low Curve) is determined directly from gage 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 Gage 8665530, 2006 Published Rate of 0.01033 feet/yr. However, more recent updates to the National Oceanographic and Atmospheric Administration (NOAA) for the Charleston Gage 8665530 are shown in 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.

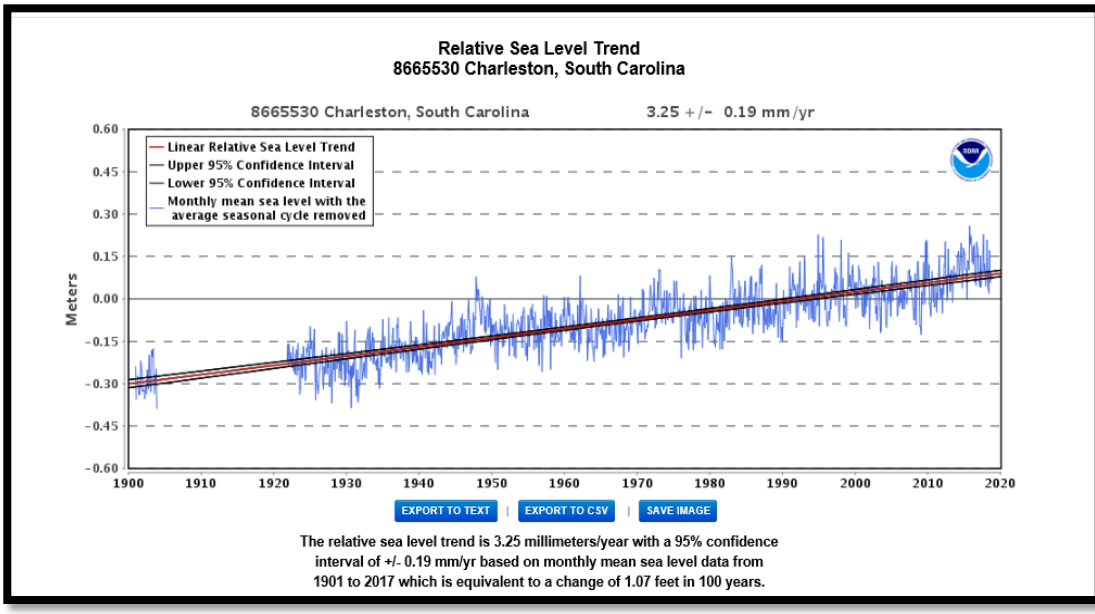

Figure 3.3.1.1 Mean Sea Level trend in Charleston 8665530 (source NOAA Tides and currents)

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

$$E(t2) – E(t1) = 0.0017(t2 – t1) + b(t22 – t12) + \text{local VLM}$$

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 + bt2; 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 Gage 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

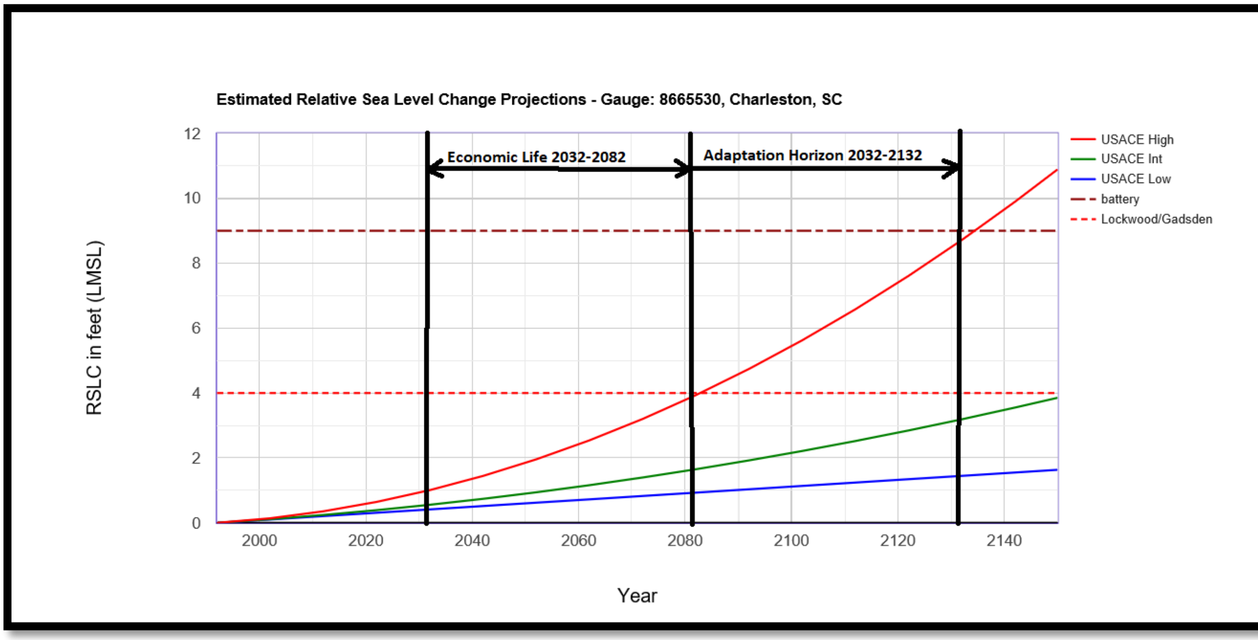

Figure 3.3.2.1 Low, Intermediate and High Sea Level Projection Gauge 8665530

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.

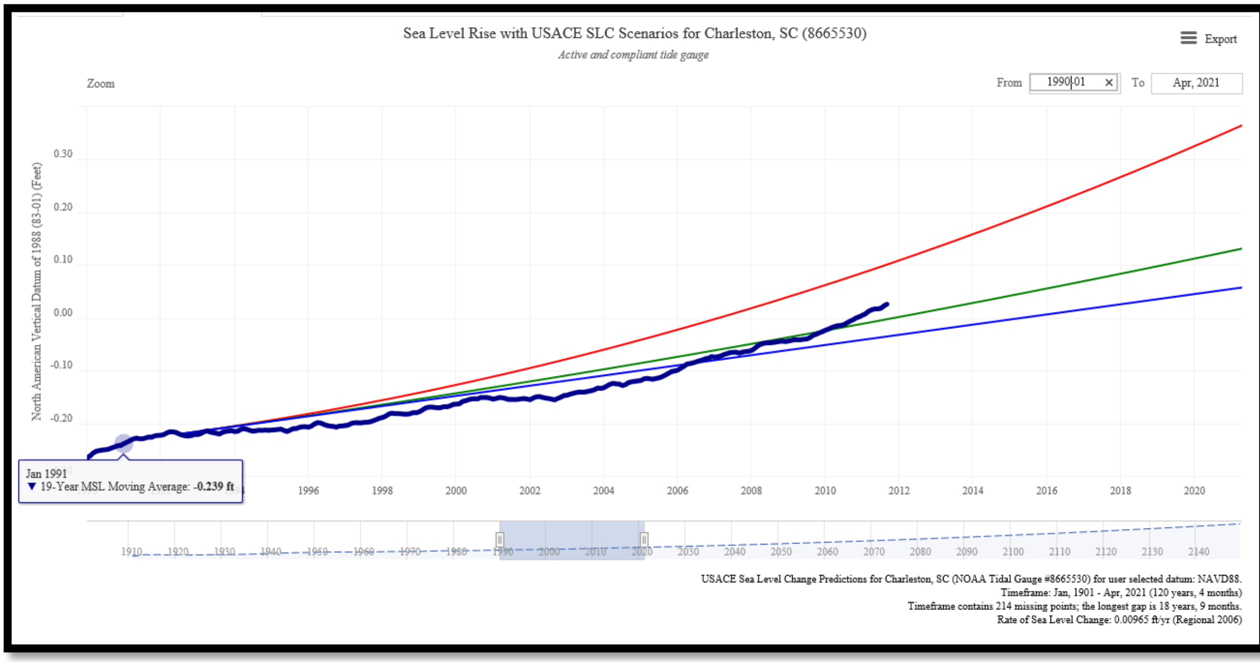

Figure 3.4.1 Sea Level Tracker Charleston SC (NOAA Station 8665530)

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 most likely SLC 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. 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 Section XX.

The future condition for the economic considerations was performed using the intermediate rate of sea level rise for the 50 year project life ending in 2082 as 1.65 feet. The 100 year adaptation range for the project into the future (year 2132) would be 3.19 feet for the intermediate rate of RSLC.

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.

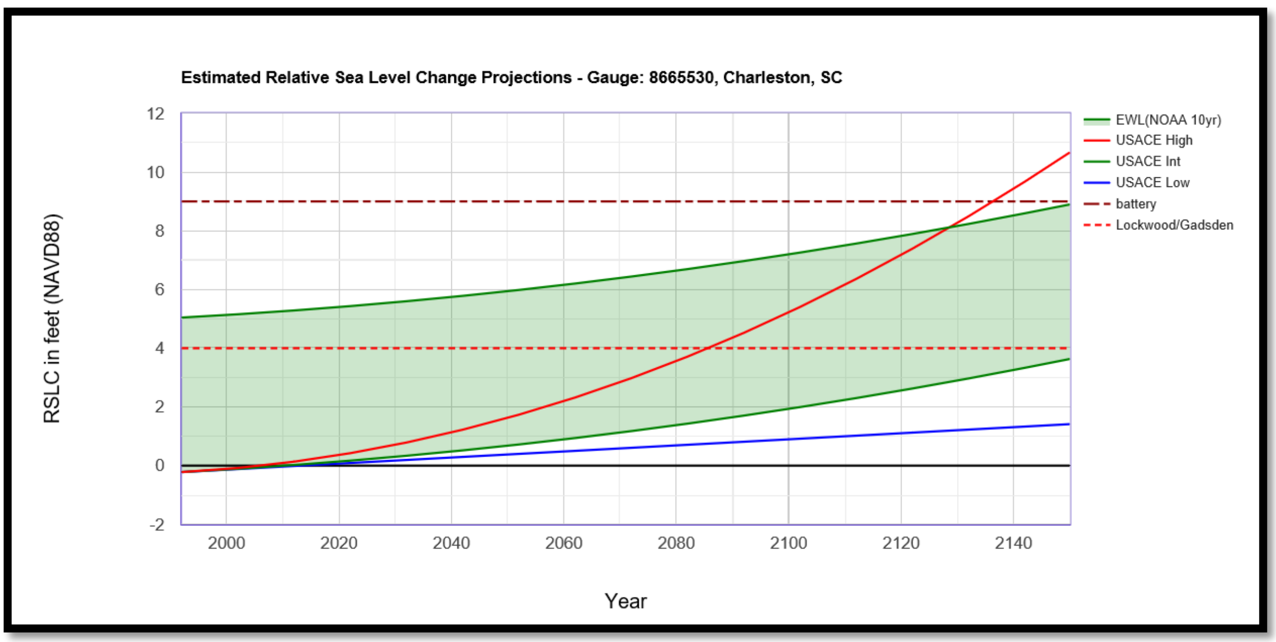

Figure 3.4.2 10% EWL on Intermediate rate of SLC.

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.

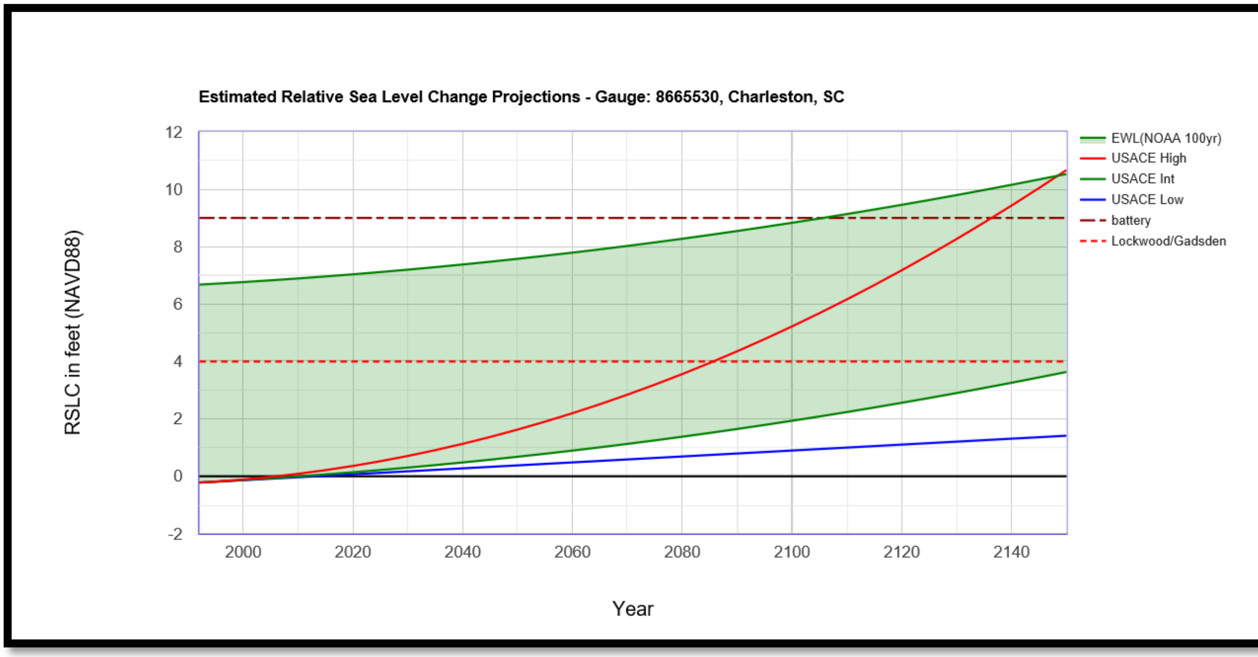

Figure 3.4.3 1% EWL with Intermediate Rate of SLC

3.5 SPONSOR SEA LEVEL CHANGE STRATEGY

As indicated in the City of Charleston's " Flooding and Sea Level Rise Strategy", revised in 2019, the goal is to address flooding and promote a more resilient and sustainable future in response to recurrent flooding, rising seas and more frequent extreme weather. They have indicated their intent to use the latest NOAA 2017 projections for their future considerations (shown in Figure 3.5.1 and Table 3.5.1). The projection of 2-3 feet of rise in 50 years is higher than the USACE rate of 1.4 feet on the intermediate scale. The city is using this information to address stormwater management, response to King tide flooding and building permit compliance with the National Flood Insurance Program. The Strategy also identifies the goal for critical infrastructure, such as hospitals, fire stations, police substations and transportation corridors to remain accessible in order to protect its citizens. The strategy identifies ongoing activities such as installation of check valves to keep tidal water out of the storm drainage system, upgrades to storm water systems, new tunnels and pump stations for peninsula drainage system. It also documents the Governance, Land Use opportunities, Collaboration and Outreach efforts to meet the goals of the Strategy.

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

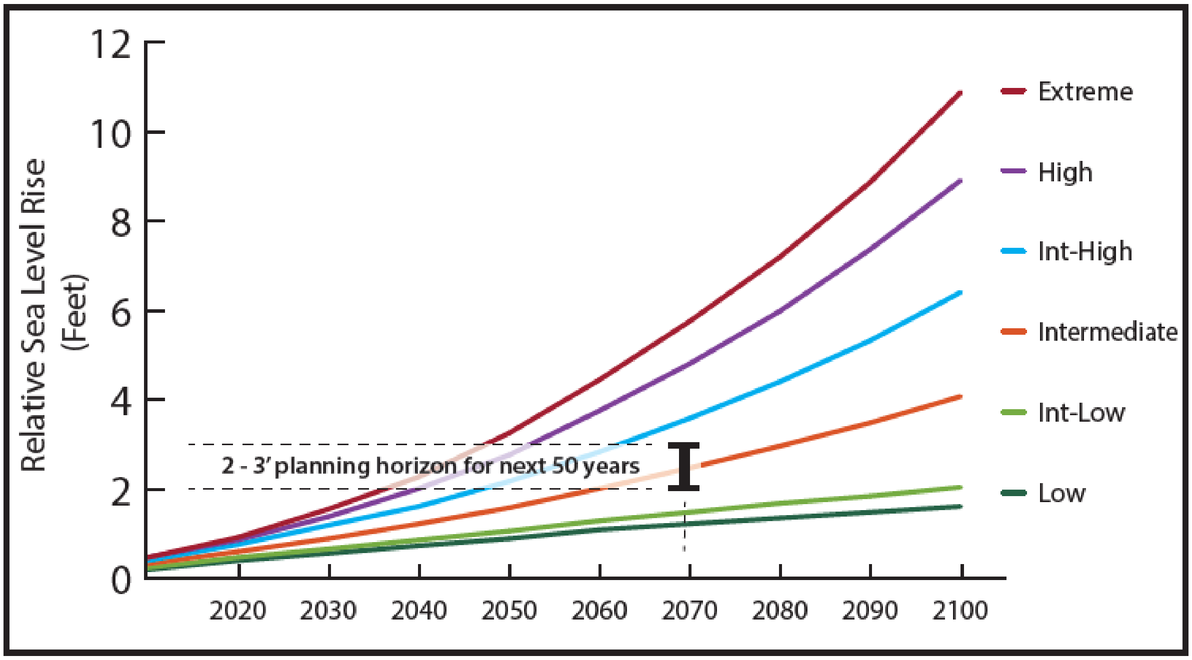

Figure 3.5.1 City of Charleston Planning Horizon for Relative Sea Level Change

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

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 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.

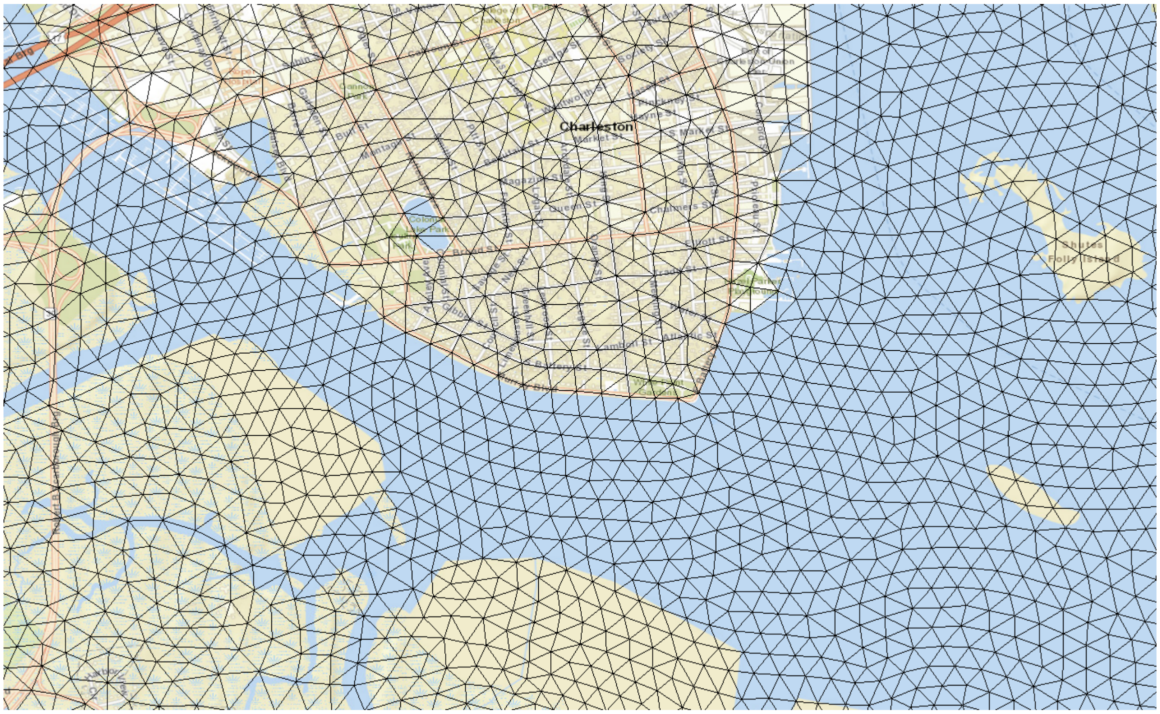

Figure 4.1 Zoom in to the Charleston Peninsula (before the grid refinement).

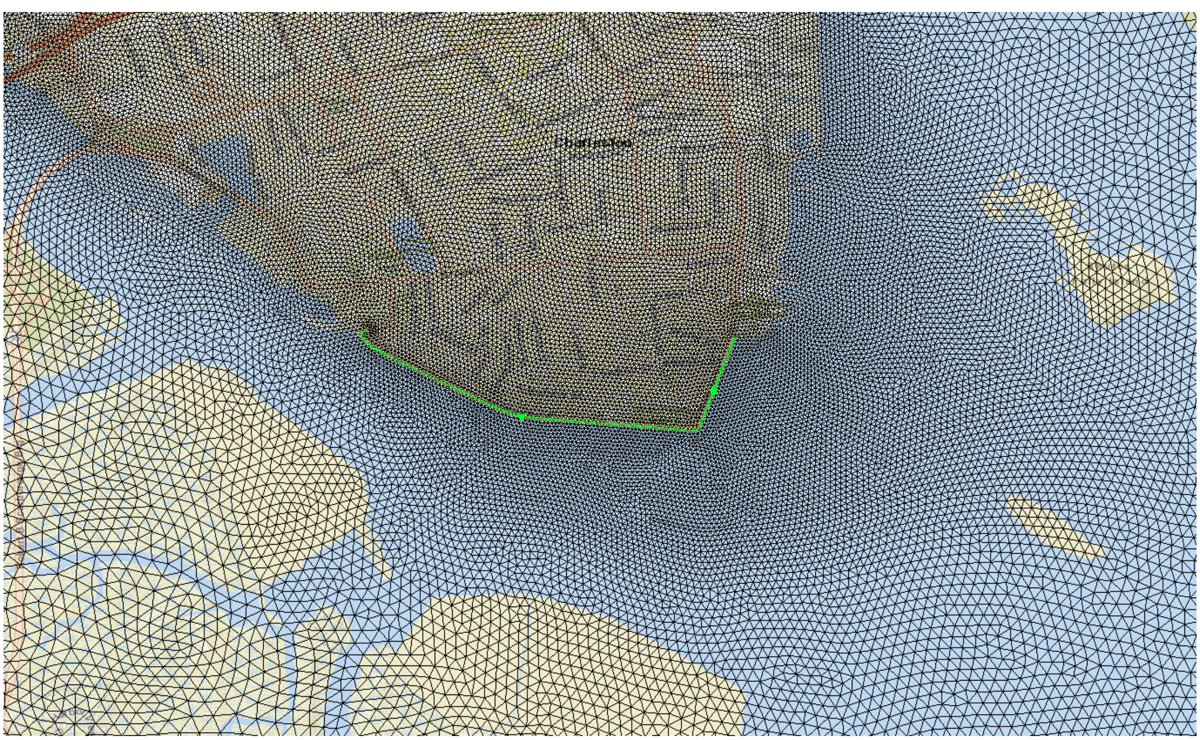

Figure 4.2 Zoom in to Charleston Peninsula after grid refinement and FWO condition of battery wall

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.

- Existing Condition is the topography and bathymetry that was provided by FEMA for the latest Flood Insurance Study. The only modification was the mesh in the area of the peninsula (Figure 4.1 and 4.2). Water level results are mean sea level (MSL) and the converted to NAVD88 for submission to G2CRM.

- Future Without condition only included the raising of the existing low battery wall to the same elevation (9 feet NAVD88) as the existing high battery wall. From the city of Charleston Flooding and Sea Level Rise Strategy : "In early 2019, the City will begin an extensive reconstruction project of the iconic Low Battery Seawall to replace and raise the seawall to account for sea level rise projections. It was 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. The City's Design Division studied this site and used extensive stakeholder input and technical data to suggest general ideas and design concepts for the new seawall which are viewable online at www.charleston-sc.gov/SLR. 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." Water level results are in MSL and the converted to NAVD88 for submission to G2CRM.
- . It does not include the sea level change as the G2CRM model has the three sea level curve formulas imbedded in the model. Only the historic rate of rise is needed for the G2CRM model to address sea level change. The G2CRM model also incorporates tide in the damage assessment.

- Future With a breakwater includes the Future Without change to the low battery and a wave attenuator at the battery. The highest wave generation during storm events, based on past experiences, is at the battery, thus a wave attenuator was included in one alternative. ERDC ran the simulation of one size breakwater and the district ran two other sizes before the breakwater was eliminated after economic analysis determined there were not sufficient benefits.

Figure 4.3 shows the STWAVE domain for the analysis.

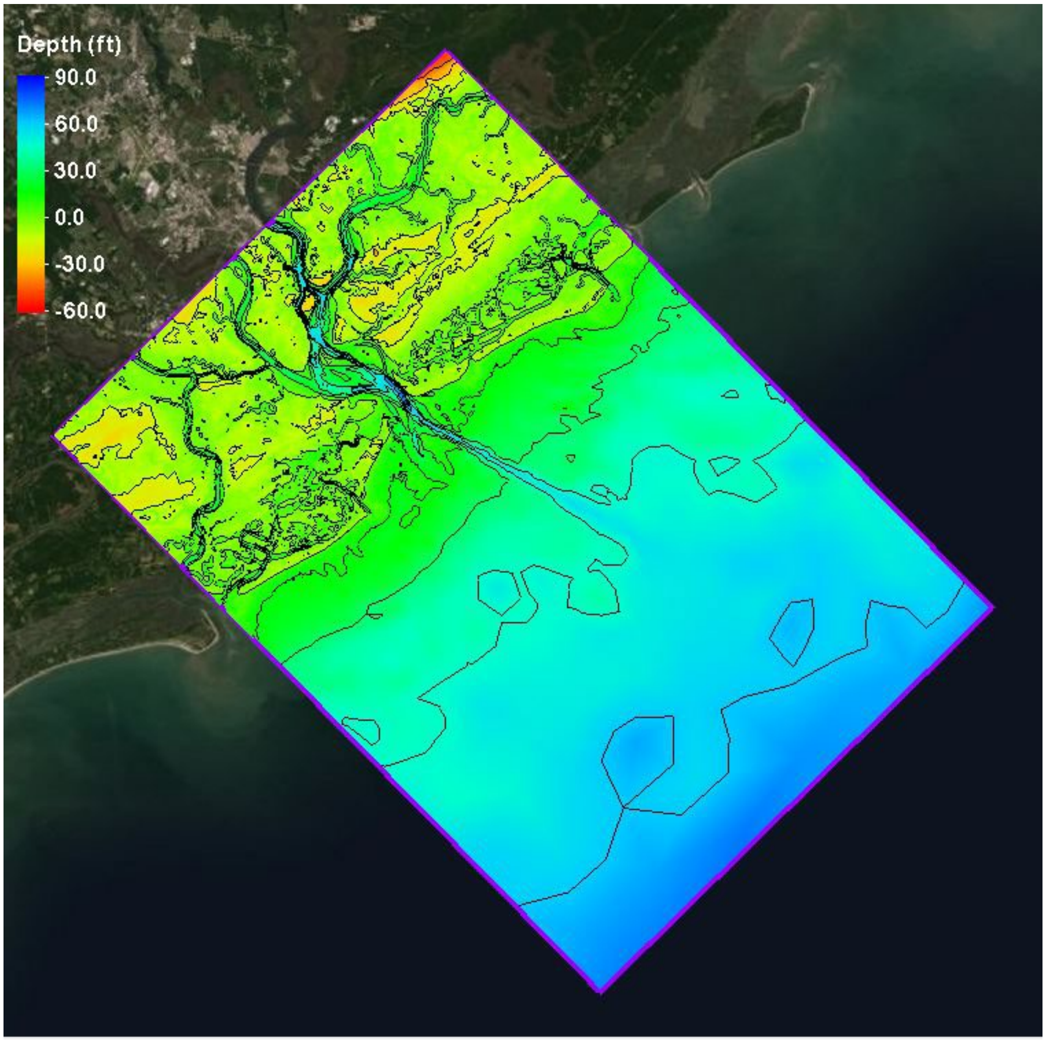

Figure 4.3 STWAVE Domain

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. Statistical analysis using the JPM-OS determined that application of a Monte Carlo method to provide a random initial tidal level at the start of each production run would account for tidal variations in the storm surge analysis. Each production run began with a random tide phase in order to vary the phasing of the tide relative to the storm. The random phases were derived from a 60-day tide simulation from August 1 to September 30, 2010, which was preceded by a 15-day spin up period necessary for the model forcing to ramp up.

To account for steric effects, the project team calculated the seasonal water level change induced by the solar annual (SA) and solar semi-annual (SSA) tidal constituents during the 60-day period at Charleston Harbor. The amplitude, phase, and frequency of the constituents were obtained from the National Oceanic and Atmospheric Administration (NOAA) (NOAA, 2013). The project team determined the mean steric effect over the 60-day period of the simulations by integration (as sine waves with time = 0 on January 1 of each year) to obtain a total increase of 2.75 inches (7 cm) above mean sea level (MSL). "

See the Sub-Appendix COASTAL MODELING SUB-APPENDIX for the ERDC modeling report that includes the STWAVE modeling and the ADCIRC modeling.

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.

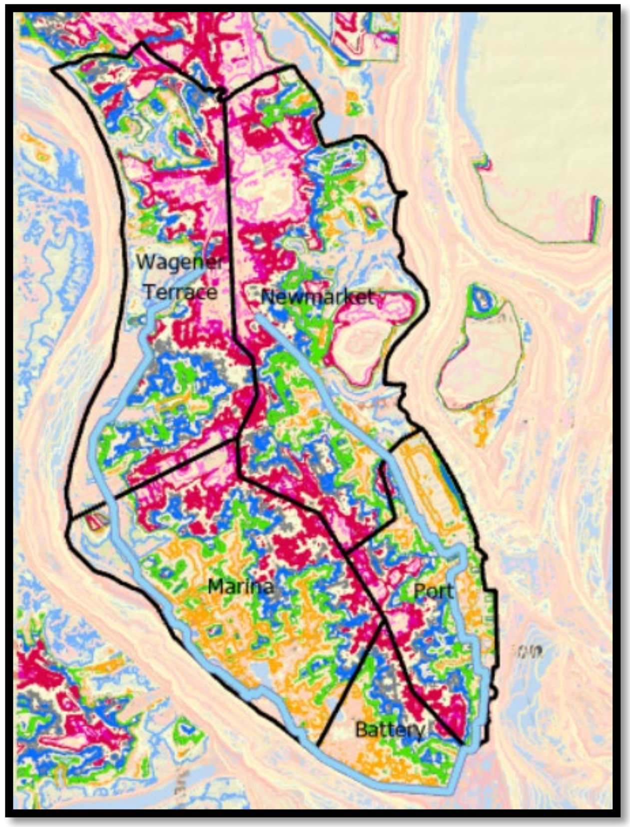

Figure 4.4 Map depicting Model Areas

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, 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 report on page 38, G2CRM applies 0.7 to the given wave height to estimate wave setup. Wave height data was calculated in the models so 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 Economic Appendix (SECTION???) 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 MSL. Plots are shown in Figure 4.6. ADCIRC/STWAVE output was then converted to NAVD88 for input into G2CRM. 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 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 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.

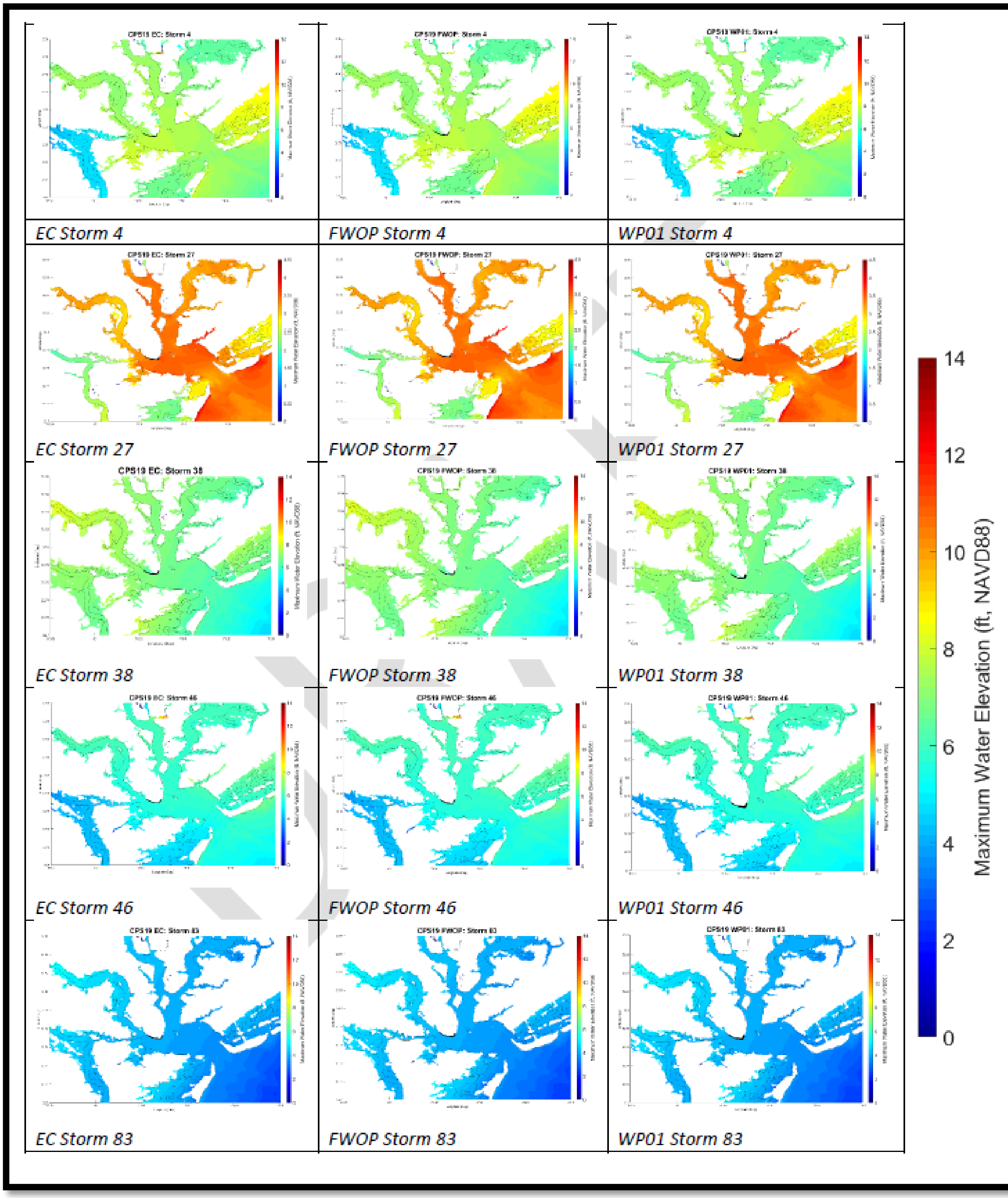

Figure 4.6 Plots of Still Water Elevations

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 returnperiod) 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 0.01% AEP wave height is between 0.5 and 0.6 m. Figure 5.4(b) shows significant wave height along the Eastern side of the harbor where 0.01% 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 0.01% 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

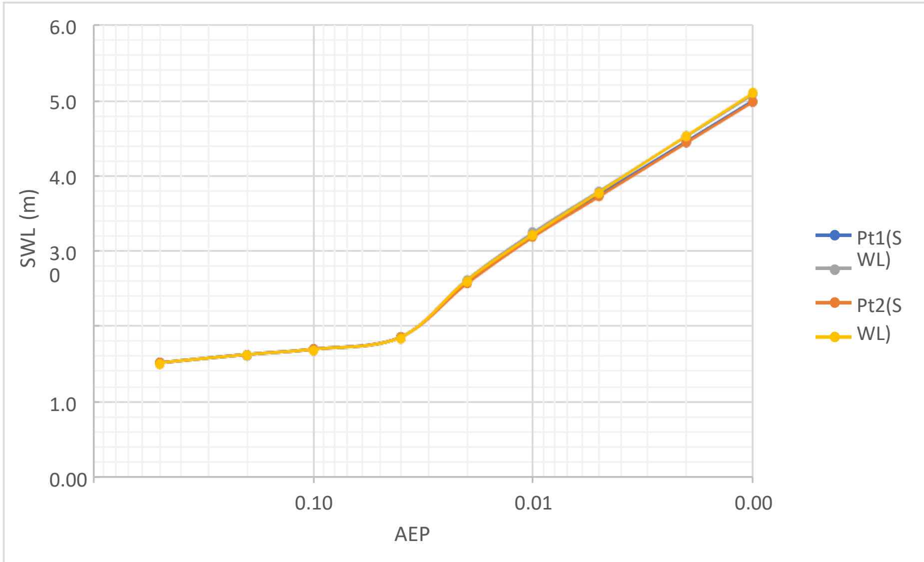

Figure 5.3: Still Water Level at Different Stations

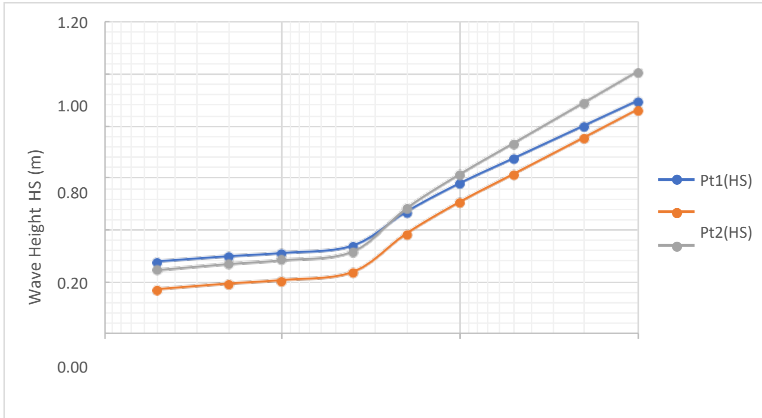

Figure 5.4(a): Significant Wave Height (HS) at Stations 1, 2, and 8

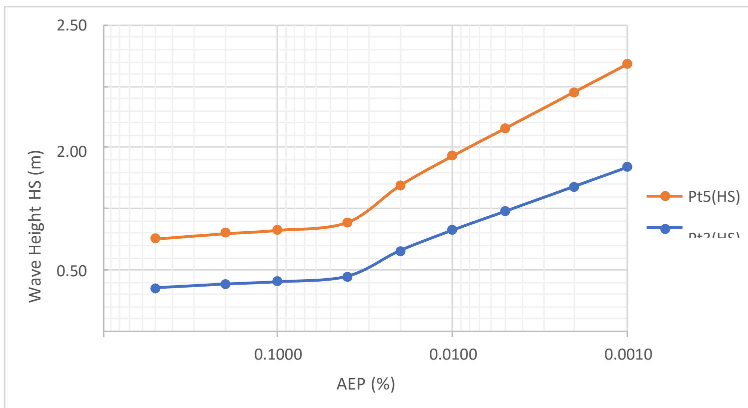

Figure 5.4(b): Significant Wave Height (HS) at Stations 3 and 5

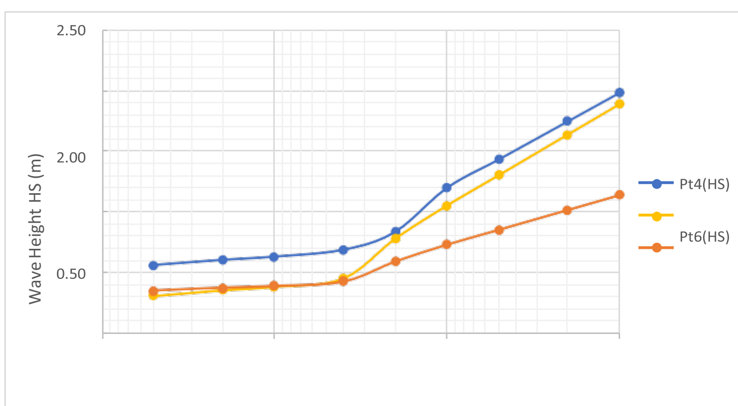

Figure 5.4(c): Significant Wave Height (HS) at Stations 4, 6, and 7

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 analyses, NLR has been calculating 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.

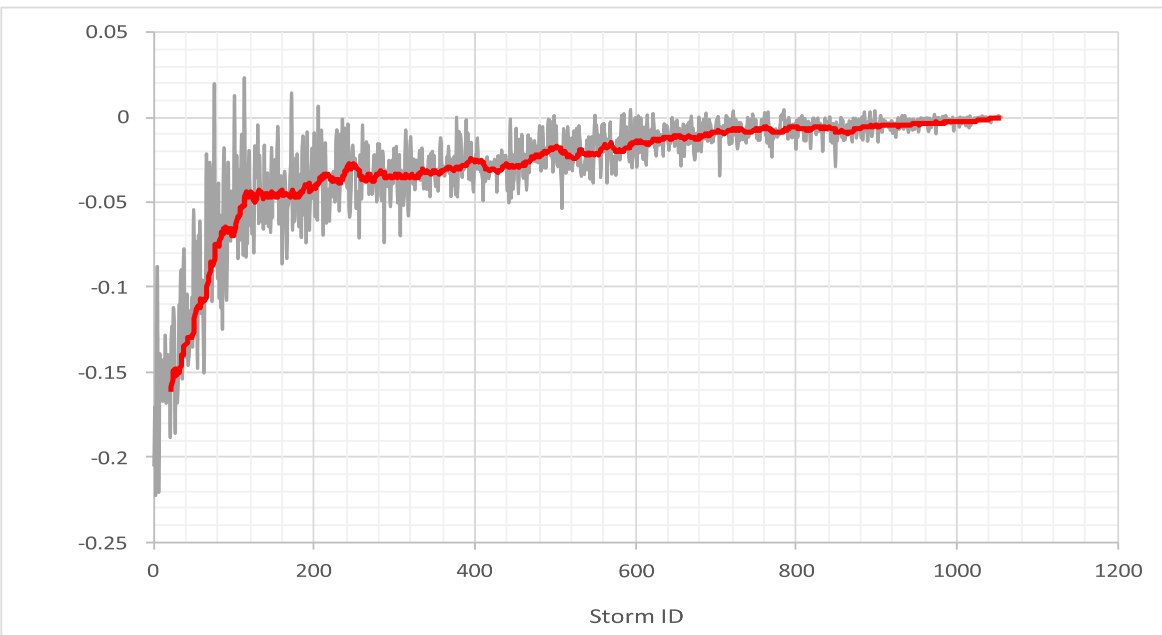

Figure 5.5(a): Distribution of NLR for Station 6

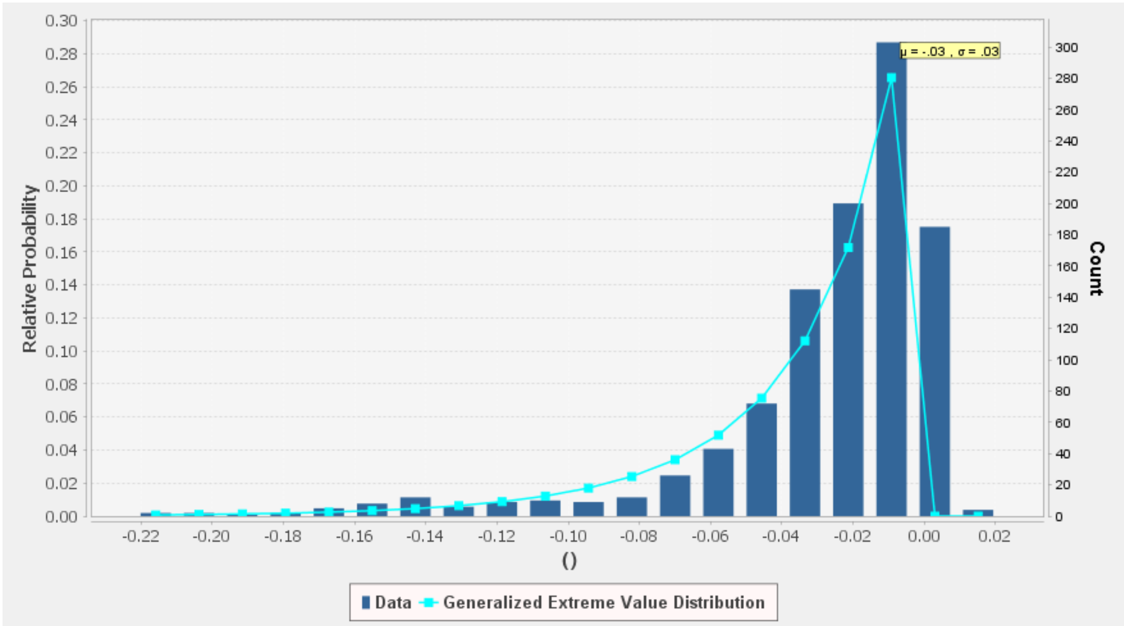

Figure 5.5(b): Frequency Distribution of NLR for Station 6

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.

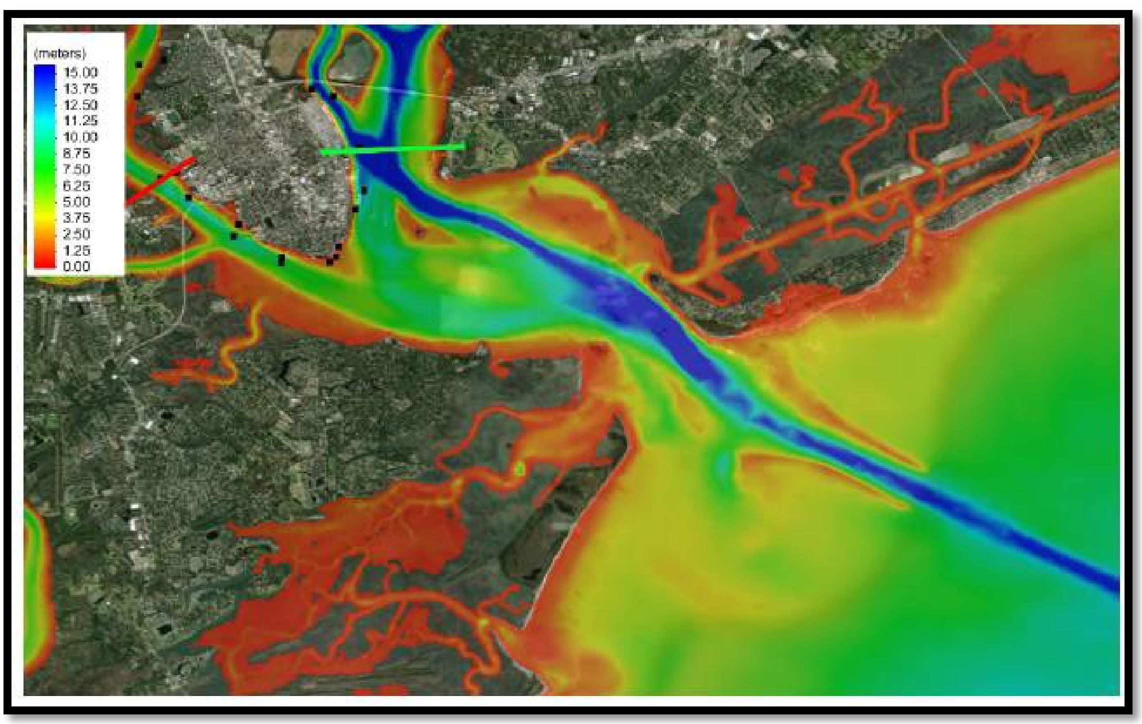

Figure 5.6(a): Water Column Height (SLC0 Case, SLR=0)

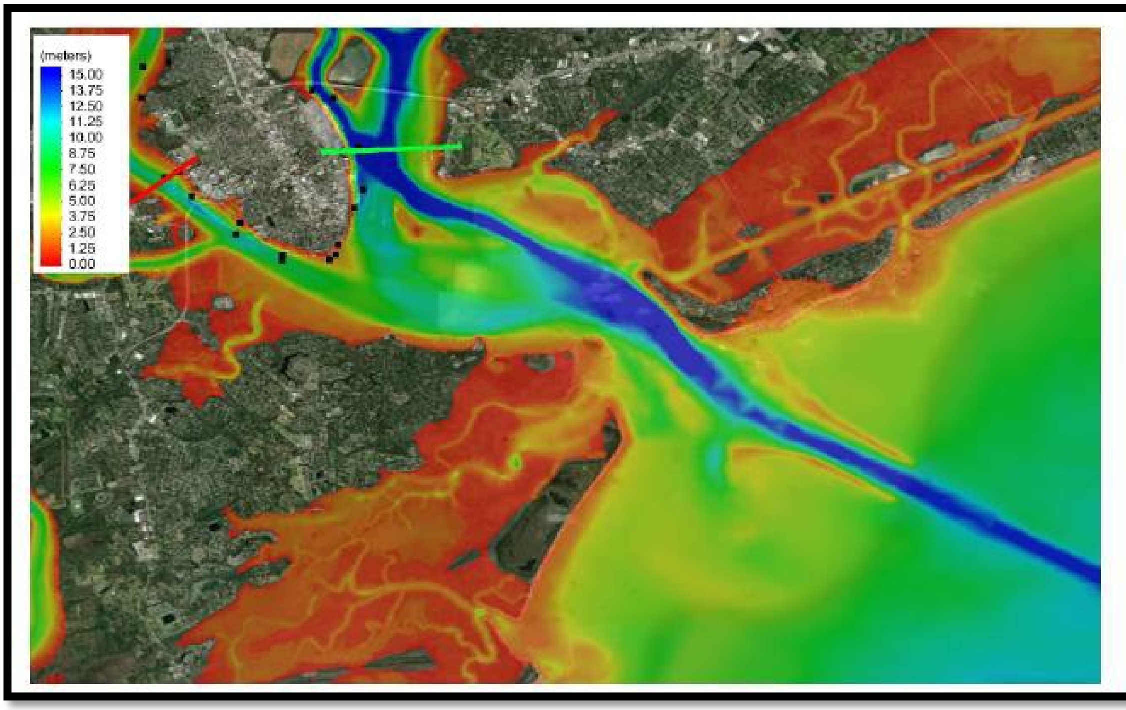

Figure 5.6(b): Water Column Height (SLC1 Case, SLR=2.82 ft)

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.

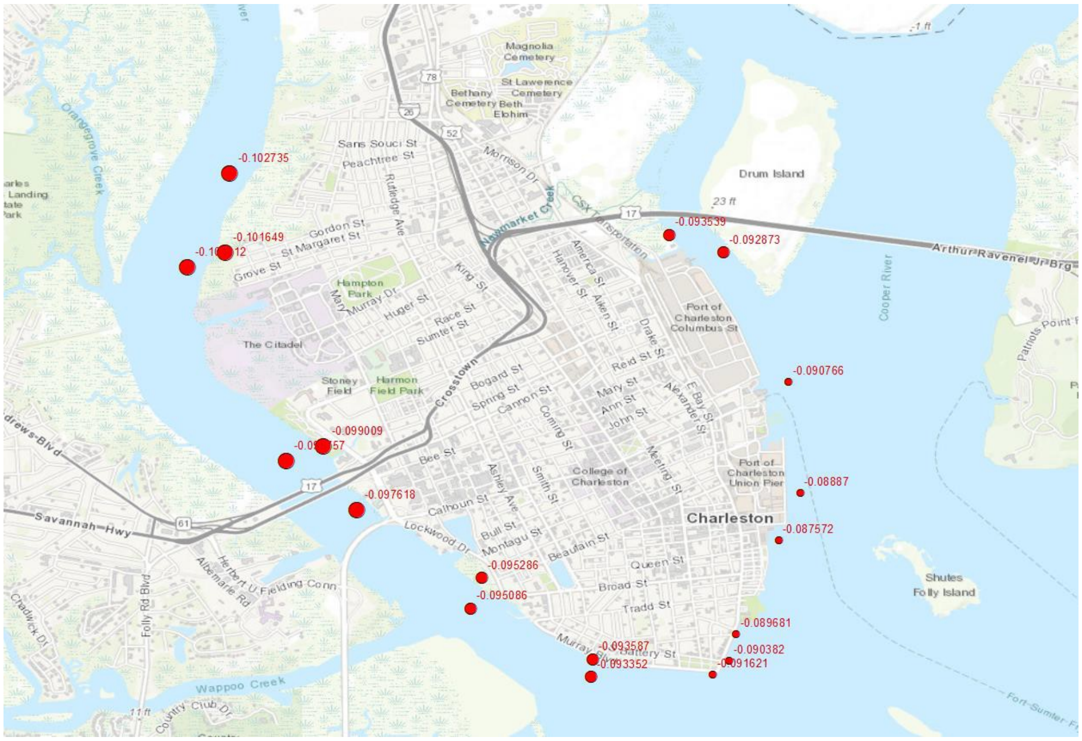

Figure 5.7: Distribution of NLR (In Ft) across Different Stations

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.

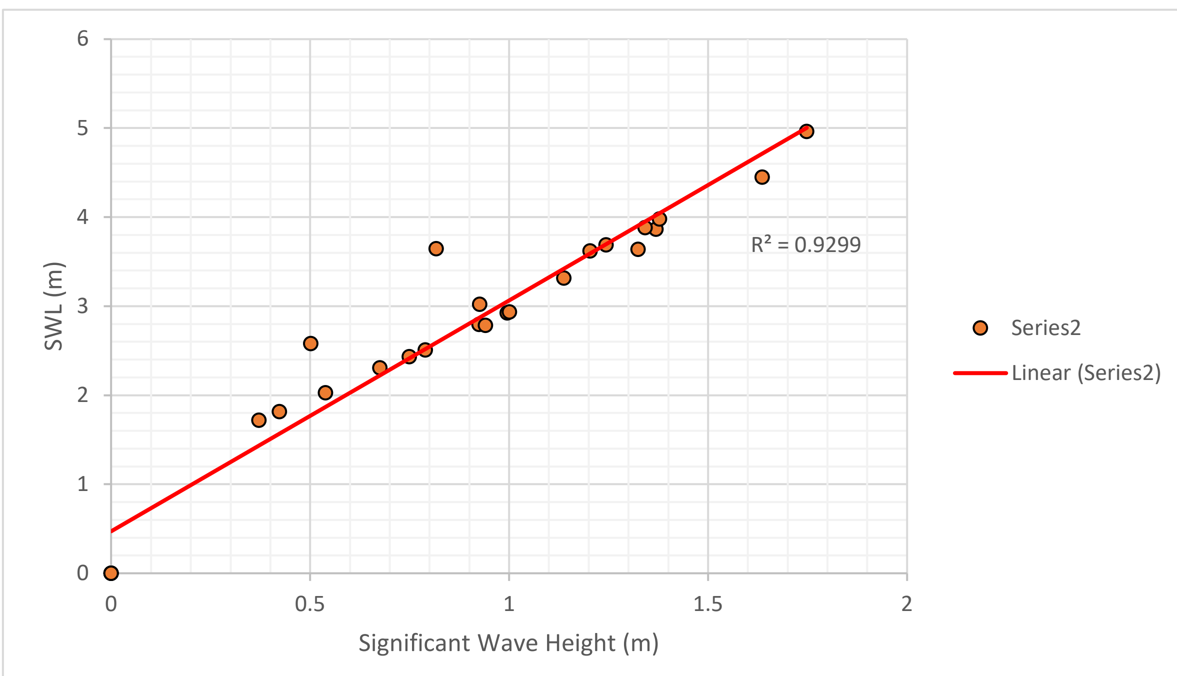

Figure 5.8(b) Simulated SWL and significant wave height Point 6

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.

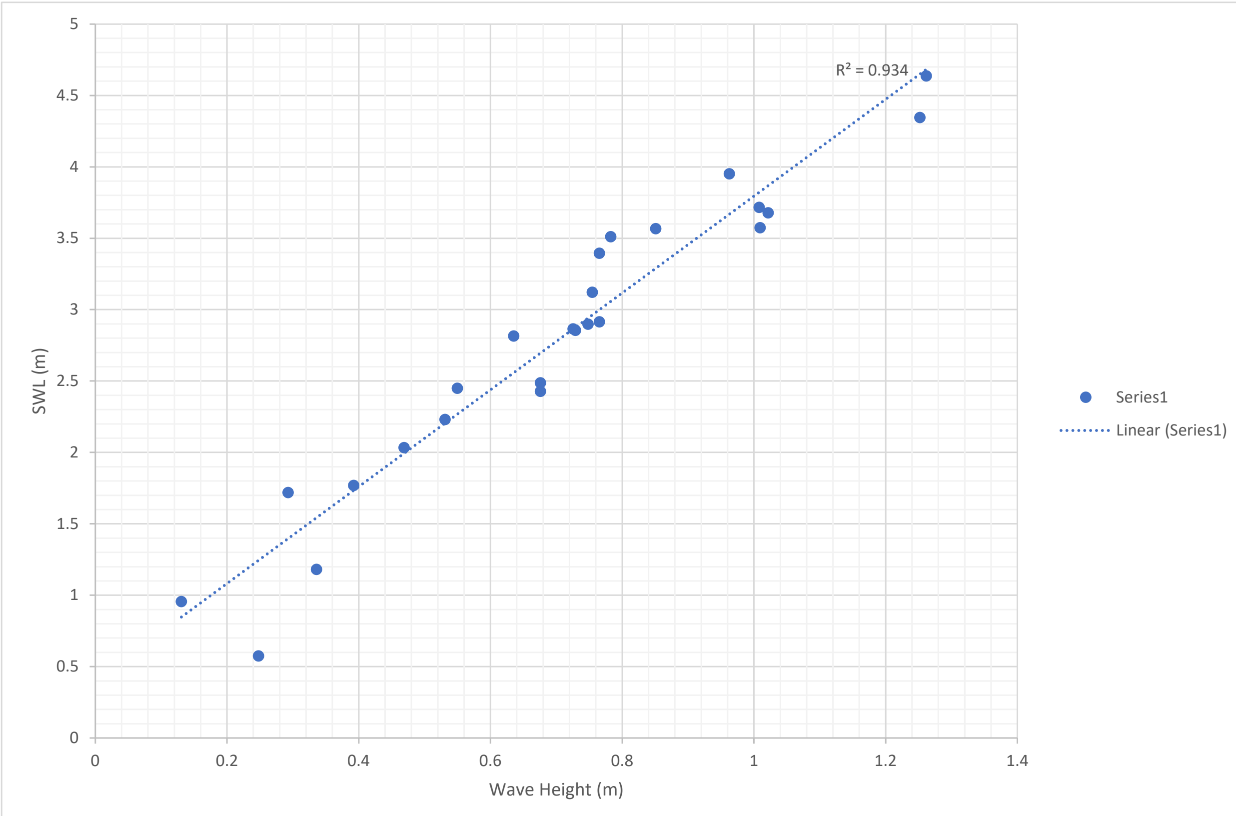

Figure 5.8(c) Simulated SWL and significant wave height point 3

(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.

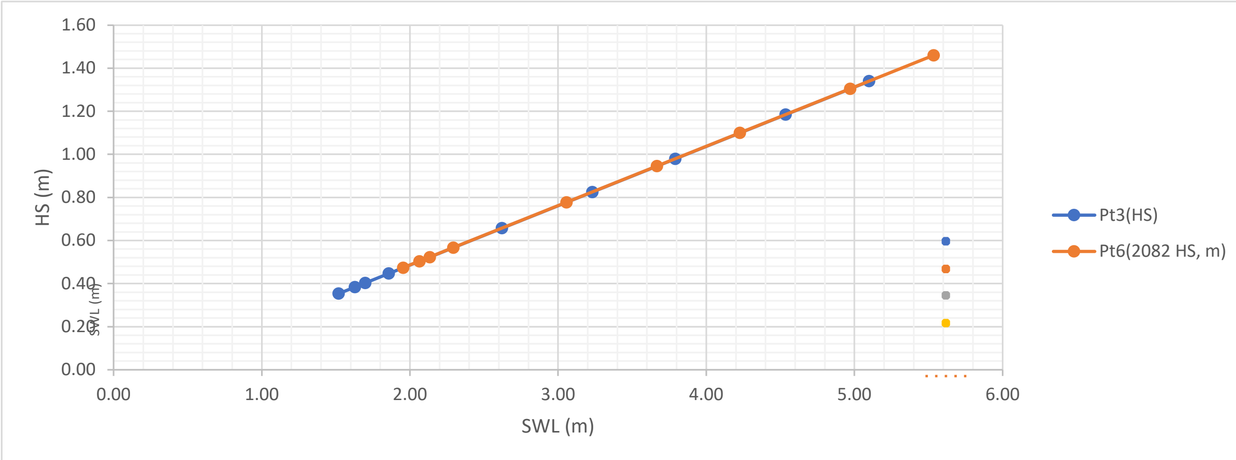

Figure 5.8 (d) Extraction Mechanism Point 3

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.

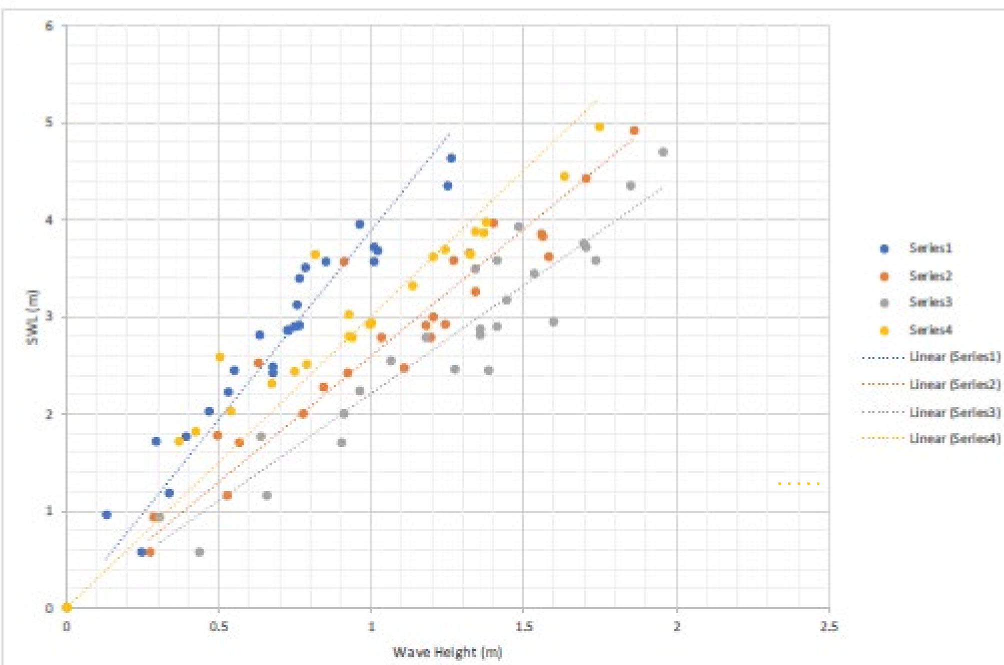

Figure 5.9: Correlation Between SWL and Wave Height

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.

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.05\exp\left(-2.78\frac{R_c}{H_{m0}}\right)$$ non-impulsive		8.50
Impulsive conditions:		
$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.011\left(\frac{H_{m0}}{hs_{m-1,0}}\right)^{0.5}\exp\left(-2.2\frac{R_c}{H_{m0}}\right)$$	valid for $0 < R_c/H_{m0} < 1.35$	8.51
$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.0014\left(\frac{H_{m0}}{hs_{m-1,0}}\right)^{0.5}\left(\frac{R_c}{H_{m0}}\right)^{-3}$$	valid for $R_c/H_{m0} \geq 1.35$	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 NAVD. This happens roughly at a 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.

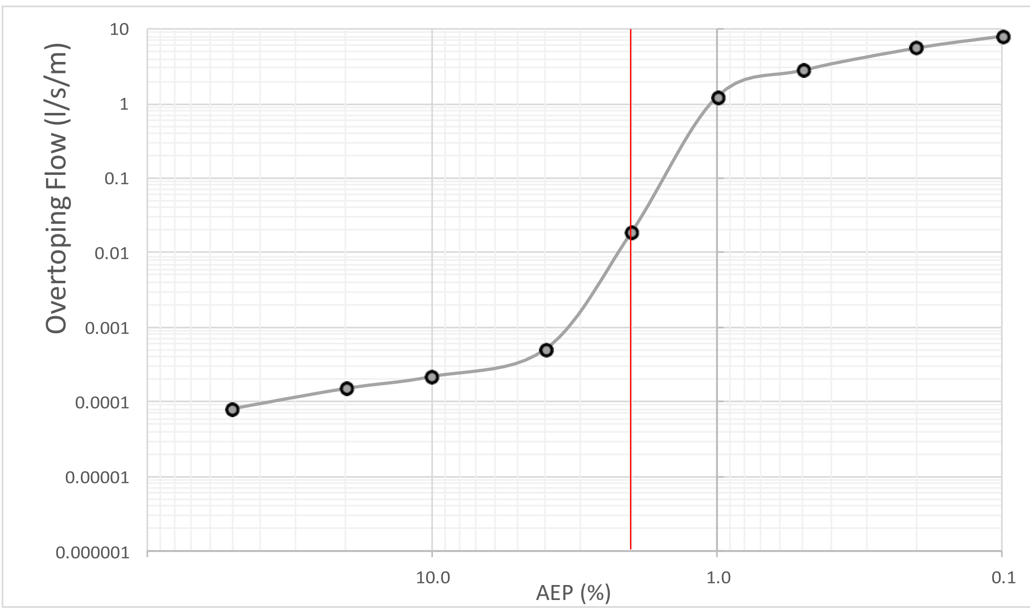

Figure 5.11: Overtopping Flow Calculated at Station 6.

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 flowsto 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

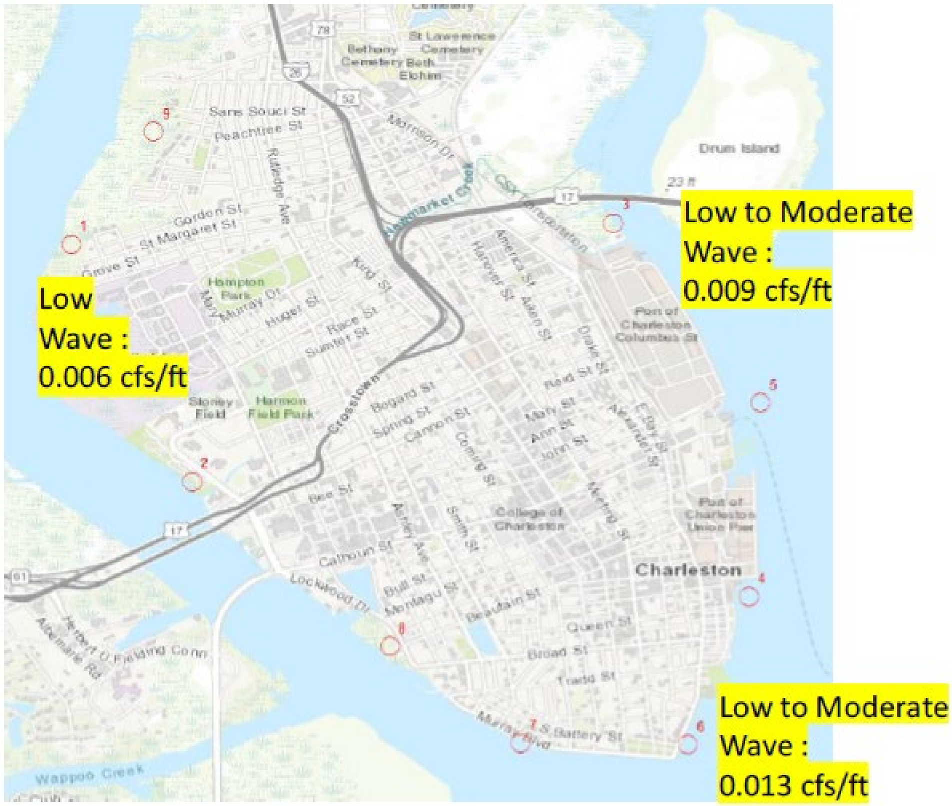

Figure 5.12: Overtopping Flow along Different Reaches

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 (SWL) from the most recent Flood Insurance Study, ERDC generated an Annual Exceedance Probability (AEP) for each of the save points requested. Still water level values in MSL were converted to NAVD88 and sea level rates 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. Table 5.2 lists the AEP with 90% confidence at the 5 locations selected for Model Areas for the year 2032 using the intermediate rate of Sea Level Change of 0.56 feet projected in 2032. Figure 5.13 is the same information plotted. Based on this the probability of annual exceedance for the wall at elevation 12 ft NAVD88 is approximately 2.6% with a 90% confidence.

Table 5.2 Year 2032 Annual Exceedance with 90% confidence

	SWL	**SLR =**	**0.56**						
Area	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88
	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**
Wagner 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

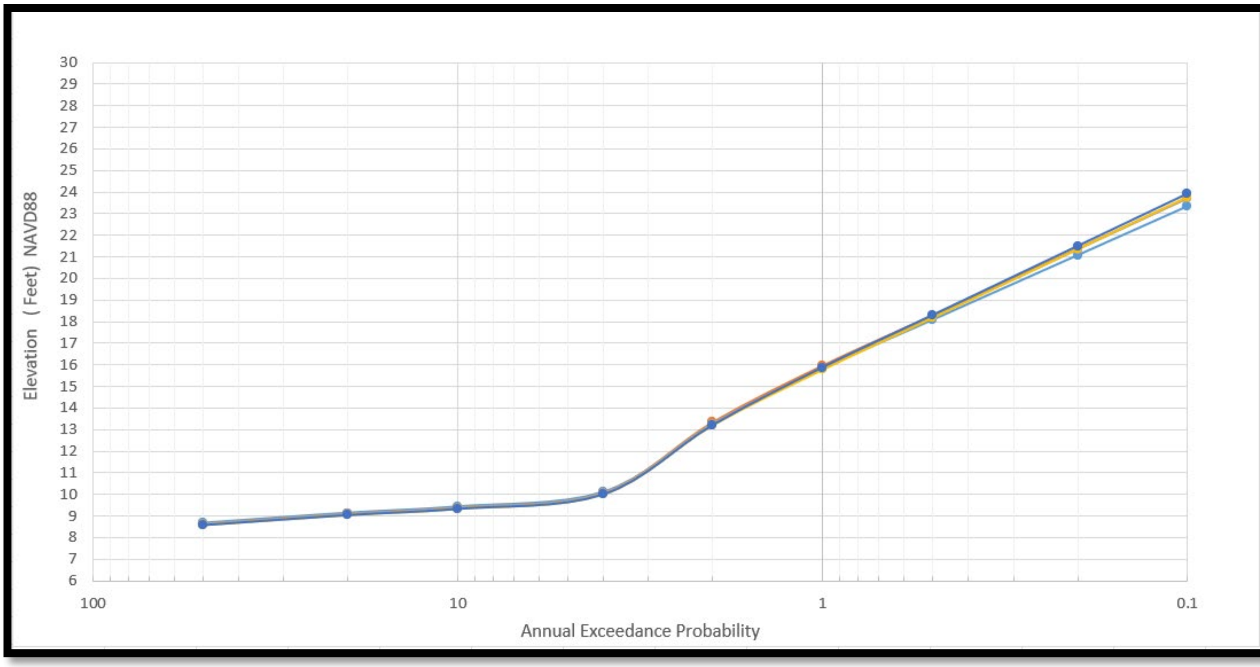

Figure 5.13 Year 2032 Annual Exceedance with 90% confidence

Using the intermediate rate of Sea Level Change of 1.65 feet projected in 2082, Table 5.3 lists the AEP with 90% confidence at the 5 locations selected for Model Areas. Figure 5.14 is the same information plotted. In the year 2082, the end of the economic project life, the probability of SWL annual exceedance of the 12 ft NAVD88 wall elevation is approximately 3.5% with a 90% confidence.

Table 5.3 90% Confidence Annual Exceedance Probability Year 2082

	SWL	**SLR =**	**1.65**						
Area	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88
	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

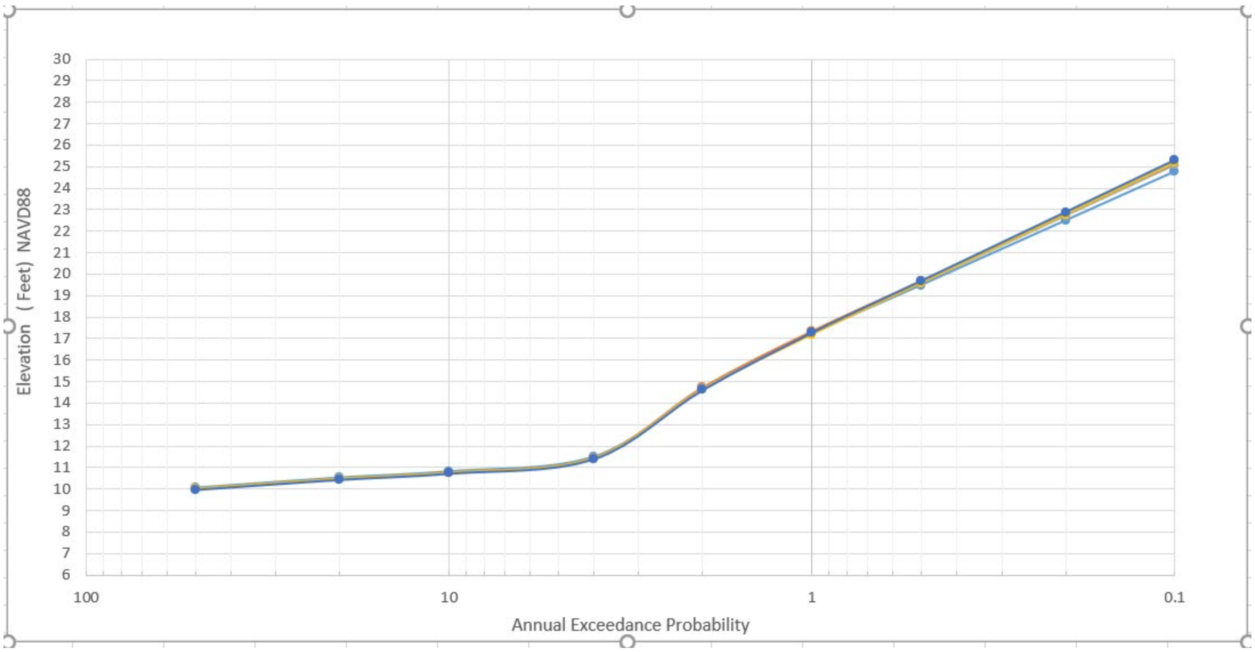

Figure 5.14 Year 2082 Annual Exceedance with 90% confidence

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 has to be considered when assessing risk and potential damages. This was considered in the G2CRM analysis of damages.

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.

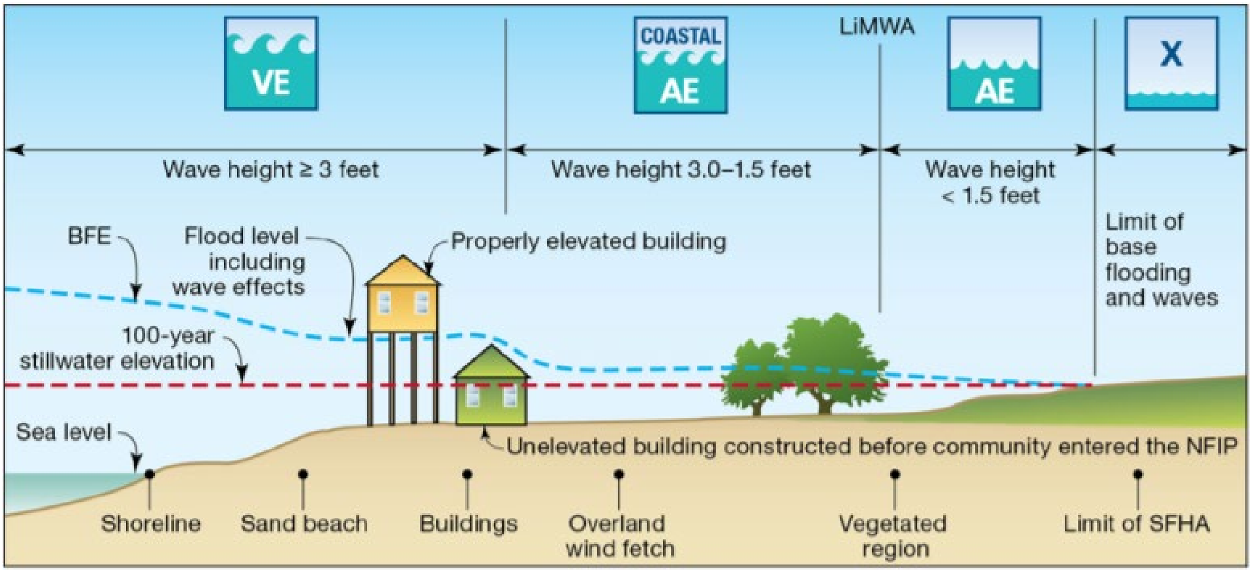

Figure 5.15 Demonstration of Stillwater elevation, BFE and various Special Flood Hazard Areas. (Source FEMA)

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 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 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 between the 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 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 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 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 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
NOAA Tides and Currents. https://tidesandcurrents.noaa.gov/stationhome.html?id=8665530.
NASA/JPL https://sideshow.jpl.nasa.gov/post/series.html

US Army Corps of Engineers®

Charleston District

Charleston Peninsula Coastal Flood Risk Management (CFRM) Study

Charleston, South Carolina

COST ENGINEERING SUB - APPENDIX

July 2021

Summary of Scope of Work:

Alternative #2 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 "RE Total Costs 18JUN21" in the "CPS 12' Optimized Basis of Estimate 01-JUL-2021" Excel file.
- Account 02 - Relocations. It is anticipated that at a minimum, three types of utilities will 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 tab called "Wall Alignment" in the "CPS 12' Optimized Basis of Estimate 01-JUL-2021" 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 "Natural Resources Mitigation" in the "CPS 12' Optimized Basis of Estimate 01-JUL-2021" 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 and Combo wall were used. Combo wall design includes 36" and 48" diameter steel pipe piles and PZC 26 steel sheet piles with pile cap and concrete fascia. 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 "Combo Wall Quantities" and "T-Wall Quantities" tabs in the "CPS 12' Optimized Basis of Estimate 01-JUL-2021" 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 "Cultural Resources Assumptions" tab in the "CPS 12' Optimized Basis of Estimate 01-JUL-2021" Excel file.
- Account 19 – Buildings, Grounds & Utilities. This account includes non-structural cost provided by PM based on the "Non-structural" and "Storage Facility Cost" tabs in the "CPS 12' Optimized Basis of Estimate 01-JUL-2021" 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

Alternative screening costs within this study have been prepared to an Estimate Class 4 Concept Study/Feasibility 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 to inform Alternative selection and early 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 Coastal Storm Risk Management Feasibility Project:

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 3rd Quarter of FY2021.

[1] Source: www.aacei.org.

d. Construction costs developed by Estimating and Specifications Section, Engineering Division, 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 National Labor Library.

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 Abbreviated Risk Analysis.

g. Contract Acquisition Strategy: Acquisition strategy is not yet determined at this point. However to reflect the historical market condition for this type of work, Prime Contractor is assumed to perform minimal earth work and will sub-contract out all remaining work.

h. Labor Shortages: It is assumed that there will be a normal labor market

i. 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. Since fuels fluctuate irrationally, an average was used.

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 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 Abbreviated Risk Analysis for the two alternatives.

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**
PROJECT NO: P2 474899
LOCATION: Charleston, South Carolina

DISTRICT: SAC District **PREPARED: 7/1/2021**
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	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	**TOTAL ($K)**	Spent Thru: **1-Oct-21** ($K)	**TOTAL FIRST COST** ($K)	INFLATED (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*		*K*	*L*	*M*	*N*	*O*
02	RELOCATIONS	$10,730	$3,648	34.0%	$14,379	2.6%	$11,012	$3,744	$14,756	$0	$14,756	19.1%	$13,110	$4,458	$17,568
06	FISH & WILDLIFE FACILITIES	$19,852	$6,750	34.0%	$26,601	2.6%	$20,373	$6,927	$27,300	$0	$27,300	19.1%	$24,255	$8,247	$32,502
11	LEVEES & FLOODWALLS	$452,199	$153,748	34.0%	$605,947	2.6%	$464,072	$157,784	$621,856	$0	$621,856	19.1%	$552,503	$187,851	$740,354
13	PUMPING PLANT	$32,289	$10,978	34.0%	$43,267	2.6%	$33,136	$11,266	$44,403	$0	$44,403	19.1%	$39,451	$13,413	$52,864
18	CULTURAL RESOURCE PRESERVATION	$62,198	$21,147	34.0%	$83,346	2.6%	$63,831	$21,703	$85,534	$0	$85,534	19.1%	$75,995	$25,838	$101,833
19	BUILDINGS, GROUNDS & UTILITIES	$47,620	$16,191	34.0%	$63,810	2.6%	$48,870	$16,616	$65,486	$0	$65,486	19.1%	$58,182	$19,782	$77,964
	#N/A	$0	$0	-	$0	-	$0	$0	$0	$0	$0	-	$0	$0	$0
	#N/A	$0	$0	-	$0	-	$0	$0	$0	$0	$0	-	$0	$0	$0
	CONSTRUCTION ESTIMATE TOTALS:	$624,888	$212,462		$837,350	2.6%	$641,294	$218,040	$859,334	$0	$859,334	19.1%	$763,496	$259,589	$1,023,085
01	LANDS AND DAMAGES	$91,454	$39,846	43.6%	$131,300	2.6%	$93,855	$40,892	$134,747	$0	$134,747	3.8%	$97,415	$42,444	$139,859
30	PLANNING, ENGINEERING & DESIGN	$43,742	$4,902	11.2%	$48,644	3.1%	$45,098	$5,054	$50,152	$0	$50,152	4.7%	$47,217	$5,291	$52,508
31	CONSTRUCTION MANAGEMENT	$43,742	$4,902	11.2%	$48,644	2.5%	$44,836	$5,025	$49,860	$0	$49,860	19.1%	$53,387	$5,983	$59,370
	PROJECT COST TOTALS:	$803,826	$262,112	32.6%	$1,065,938		$825,083	$269,011	**$1,094,094**	$0	**$1,094,094**	16.5%	$961,515	$313,307	$1,274,822

ESTIMATED TOTAL PROJECT COST: $1,274,822

______________________ **CHIEF, COST ENGINEERING, Lance Mahar**

______________________ **PROJECT MANAGER, Wesley Wilson**

______________________ **CHIEF, REAL ESTATE, John Hinely**

______________________ **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**

Filename: CPS 12' Optimized TPCS 20-JUL-2021.xlsx
TPCS

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Charleston Peninsula Study- Optimized Plan
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: 7/1/2021

Estimated Cost — Estimate Prepared: **1-Jul-21**; Effective Price Level: 1-Oct-21

Project First Cost (Constant Dollar Basis) — Program Year (Budget EC): 2023; Effective Price Level Date: 1 OCT 22

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
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	$678	$230	34.0%	$908	2.6%	$695	$236	$932	2030Q1	19.1%	$828	$281	$1,109
06	FISH & WILDLIFE FACILITIES	$7,810	$2,655	34.0%	$10,466	2.6%	$8,015	$2,725	$10,740	2030Q1	19.1%	$9,543	$3,244	$12,787
11	LEVEES & FLOODWALLS	$122,003	$41,481	34.0%	$163,484	2.6%	$125,207	$42,570	$167,777	2030Q1	19.1%	$149,065	$50,682	$199,748
13	PUMPING PLANT	$9,500	$3,230	34.0%	$12,730	2.6%	$9,749	$3,315	$13,064	2030Q1	19.1%	$11,607	$3,946	$15,554
18	CULTURAL RESOURCE PRESERVATION	$17,398	$5,915	34.0%	$23,314	2.6%	$17,855	$6,071	$23,926	2030Q1	19.1%	$21,258	$7,228	$28,485
19	BUILDINGS, GROUNDS & UTILITIES	$645	$219	34.0%	$864	2.6%	$662	$225	$887	2030Q1	19.1%	$788	$268	$1,056
	#N/A	$0	$0	0.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
	CONSTRUCTION ESTIMATE TOTALS:	$158,034	$53,732	34.0%	**$211,766**		$162,183	$55,142	$217,326			$193,088	$65,650	$258,738
01	LANDS AND DAMAGES	$91,454	$39,846	43.6%	$131,300	2.6%	$93,855	$40,892	$134,747	2024Q3	3.8%	$97,415	$42,444	$139,859
30	PLANNING, ENGINEERING & DESIGN													
7.0%	of Construction Estimate Totals	$11,062	$1,240	11.2%	$12,302	3.1%	$11,405	$1,278	$12,683	2024Q3	4.7%	$11,941	$1,338	$13,279
31	CONSTRUCTION MANAGEMENT													
7.0%	of Construction Estimate Totals	$11,062	$1,240	11.2%	$12,302	2.5%	$11,339	$1,271	$12,610	2030Q1	19.1%	$13,502	$1,513	$15,015
	CONTRACT COST TOTALS:	$271,613	$96,057		$367,670		$278,783	$98,584	**$377,366**			$315,946	$110,945	**$426,891**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Charleston Peninsula Study- Optimized Plan
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: 7/1/2021

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared: 1-Jul-21 Effective Price Level: 1-Oct-21				Program Year (Budget EC): 2023 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	$142	$48	34.0%	$191	2.6%	$146	$50	$196	2030Q1	19.1%	$174	$59	$233
06	FISH & WILDLIFE FACILITIES	$164	$56	34.0%	$219	2.6%	$168	$57	$225	2030Q1	19.1%	$200	$68	$268
11	LEVEES & FLOODWALLS	$17,048	$5,796	34.0%	$22,845	2.6%	$17,496	$5,949	$23,444	2030Q1	19.1%	$20,830	$7,082	$27,912
13	PUMPING PLANT	$6,835	$2,324	34.0%	$9,159	2.6%	$7,015	$2,385	$9,400	2030Q1	19.1%	$8,352	$2,840	$11,191
18	CULTURAL RESOURCE PRESERVATION	$17,919	$6,093	34.0%	$24,012	2.6%	$18,390	$6,252	$24,642	2030Q1	19.1%	$21,894	$7,444	$29,338
	#N/A	$0	$0	0.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
	#N/A	$0	$0	0.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
	CONSTRUCTION ESTIMATE TOTALS:	$42,109	$14,317	34.0%	**$56,426**		$43,214	$14,693	$57,907			$51,449	$17,493	$68,942
01	LANDS AND DAMAGES	$0	$0	0.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
30	PLANNING, ENGINEERING & DESIGN													
7.0%	of Construction Estimate Totals	$2,948	$330	11.2%	$3,278	3.1%	$3,039	$341	$3,380	2024Q3	4.7%	$3,182	$357	$3,538
31	CONSTRUCTION MANAGEMENT													
7.0%	of Construction Estimate Totals	$2,948	$330	11.2%	$3,278	2.5%	$3,021	$339	$3,360	2030Q1	19.1%	$3,598	$403	$4,001
	CONTRACT COST TOTALS:	$48,004	$14,978		$62,982		$49,275	$15,372	**$64,647**			$58,228	$18,252	**$76,481**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Charleston Peninsula Study- Optimized Plan
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: 7/1/2021

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared: 1-Jul-21 Effective Price Level: 1-Oct-21				Program Year (Budget EC): 2023 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	$5,566	$1,892	34.0%	$7,459	2.6%	$5,712	$1,942	$7,654	2030Q1	19.1%	$6,801	$2,312	$9,113
11	LEVEES & FLOODWALLS	$116,182	$39,502	34.0%	$155,684	2.6%	$119,232	$40,539	$159,771	2030Q1	19.1%	$141,953	$48,264	$190,217
13	PUMPING PLANT	$1,518	$516	34.0%	$2,034	2.6%	$1,558	$530	$2,087	2030Q1	19.1%	$1,854	$630	$2,485
18	CULTURAL RESOURCE PRESERVATION	$14,070	$4,784	34.0%	$18,854	2.6%	$14,439	$4,909	$19,349	2030Q1	19.1%	$17,191	$5,845	$23,036
	#N/A	$0	$0	0.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
	#N/A	$0	$0	0.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
	#N/A	$0	$0	0.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
	CONSTRUCTION ESTIMATE TOTALS:	$137,336	$46,694	34.0%	**$184,030**		$140,942	$47,920	$188,862			$167,799	$57,052	$224,850
01	LANDS AND DAMAGES	$0	$0	0.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
30	PLANNING, ENGINEERING & DESIGN													
7.0%	of Construction Estimate Totals	$9,614	$1,077	11.2%	$10,691	3.1%	$9,912	$1,111	$11,022	2024Q3	4.7%	$10,377	$1,163	$11,540
31	CONSTRUCTION MANAGEMENT													
7.0%	of Construction Estimate Totals	$9,614	$1,077	11.2%	$10,691	2.5%	$9,854	$1,104	$10,958	2030Q1	19.1%	$11,733	$1,315	$13,048
	CONTRACT COST TOTALS:	$156,563	$48,849		**$205,412**		$160,707	$50,135	**$210,842**			$189,909	$59,529	**$249,438**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Charleston Peninsula Study- Optimized Plan
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POC: CHIEF, COST ENGINEERING, Lance Mahar
PREPARED: 7/1/2021

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared:			**1-Jul-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 3 - MA - Newmarket													
02	RELOCATIONS	$3,640	$1,238	34.0%	$4,878	2.6%	$3,736	$1,270	$5,006	2030Q1	19.1%	$4,448	$1,512	$5,960
06	FISH & WILDLIFE FACILITIES	$28	$9	34.0%	$37	2.6%	$28	$10	$38	2030Q1	19.1%	$34	$11	$45
11	LEVEES & FLOODWALLS	$60,795	$20,670	34.0%	$81,465	2.6%	$62,391	$21,213	$83,604	2030Q1	19.1%	$74,280	$25,255	$99,536
13	PUMPING PLANT	$8,735	$2,970	34.0%	$11,705	2.6%	$8,965	$3,048	$12,013	2030Q1	19.1%	$10,673	$3,629	$14,302
18	CULTURAL RESOURCE PRESERVATION	$5,223	$1,776	34.0%	$6,999	2.6%	$5,360	$1,822	$7,183	2030Q1	19.1%	$6,382	$2,170	$8,551
19	BUILDINGS, GROUNDS & UTILITIES	$24,877	$8,458	34.0%	$33,335	2.6%	$25,530	$8,680	$34,210	2030Q1	19.1%	$30,395	$10,334	$40,729
	#N/A	$0	$0	0.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
	CONSTRUCTION ESTIMATE TOTALS:	$103,298	$35,121	34.0%	**$138,419**		$106,010	$36,043	$142,053			$126,211	$42,912	$169,122
01	LANDS AND DAMAGES	$0	$0	0.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
30	PLANNING, ENGINEERING & DESIGN													
7.0%	of Construction Estimate Totals	$7,231	$810	11.2%	$8,041	3.1%	$7,455	$835	$8,290	2024Q3	4.7%	$7,805	$875	$8,680
31	CONSTRUCTION MANAGEMENT													
7.0%	of Construction Estimate Totals	$7,231	$810	11.2%	$8,041	2.5%	$7,412	$831	$8,242	2030Q1	19.1%	$8,825	$989	$9,814
	CONTRACT COST TOTALS:	$117,760	$36,742		**$154,501**		$120,877	$37,709	**$158,586**			$142,841	$44,775	**$187,616**

Filename: CPS 12' Optimized TPCS 20-JUL-2021.xlsx
TPCS

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: Charleston Peninsula Study- Optimized Plan
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: 7/1/2021

Civil Works Work Breakdown Structure		**ESTIMATED COST**				**PROJECT FIRST COST (Constant Dollar Basis)**				**TOTAL PROJECT COST (FULLY FUNDED)**				
		Estimate Prepared:			**1-Jul-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	$704	$239	34.0%	$944	2.6%	$723	$246	$969	2030Q1	19.1%	$861	$293	$1,153
06	FISH & WILDLIFE FACILITIES	$11,850	$4,029	34.0%	$15,879	2.6%	$12,161	$4,135	$16,296	2030Q1	19.1%	$14,479	$4,923	$19,402
11	LEVEES & FLOODWALLS	$136,170	$46,298	34.0%	$182,468	2.6%	$139,745	$47,513	$187,259	2030Q1	19.1%	$166,375	$56,567	$222,942
13	PUMPING PLANT	$5,700	$1,938	34.0%	$7,638	2.6%	$5,850	$1,989	$7,839	2030Q1	19.1%	$6,964	$2,368	$9,332
18	CULTURAL RESOURCE PRESERVATION	$7,588	$2,580	34.0%	$10,168	2.6%	$7,787	$2,648	$10,435	2030Q1	19.1%	$9,271	$3,152	$12,423
19	BUILDINGS, GROUNDS & UTILITIES	$22,098	$7,513	34.0%	$29,612	2.6%	$22,678	$7,711	$30,389	2030Q1	19.1%	$27,000	$9,180	$36,180
	#N/A	$0	$0	0.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
	CONSTRUCTION ESTIMATE TOTALS:	$184,111	$62,598	34.0%	**$246,709**		$188,945	$64,241	$253,186			$224,950	$76,483	$301,432
01	LANDS AND DAMAGES	$0	$0	0.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
30	PLANNING, ENGINEERING & DESIGN													
7.0%	of Construction Estimate Totals	$12,888	$1,444	11.2%	$14,332	3.1%	$13,287	$1,489	$14,776	2024Q3	4.7%	$13,912	$1,559	$15,471
31	CONSTRUCTION MANAGEMENT													
7.0%	of Construction Estimate Totals	$12,888	$1,444	11.2%	$14,332	2.5%	$13,210	$1,480	$14,690	2030Q1	19.1%	$15,729	$1,763	$17,492
	CONTRACT COST TOTALS:	$209,887	$65,486		**$275,373**		$215,442	$67,211	**$282,653**			$254,590	$79,805	**$334,395**

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.

July 14, 2021

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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 $625M, excluding contingency and expressed in FY 2021 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 $213M or approximately 34% 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 ->	$712,372,000	
Confidence Level	**Contingency Value**	**Contingency**
0%	128,226,960	18%
10%	185,216,720	26%
20%	192,340,440	27%
30%	206,587,880	29%
40%	213,711,600	30%
50%	220,835,320	31%
60%	227,959,040	32%
70%	235,082,760	33%
80%	**242,206,480**	**34%**
90%	263,577,640	37%
100%	334,814,840	47%

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 $242.2M and schedule risks adding a potential 32 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 very preliminary. Variable quantities and additional details are likely to cause costs to increase.

- 8 Scope Refinement – 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.

- 20 Vibration Impacts on Historic Structures – 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.

- 40 Utility Crossings – There is information missing and some inaccurate information is also currently being utilized. Impacts concerning utilities may alter dimensions and alignment of the wall structures. Cost increases could potentially be significant.

- 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). An imported workforce is likely needed 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 National Environmental Policy Act (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 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 ->	$712,372,000	
Confidence Level	**Contingency Value**	**Contingency**
0%	128,226,960	18%
10%	185,216,720	26%
20%	192,340,440	27%
30%	206,587,880	29%
40%	213,711,600	30%
50%	220,835,320	31%
60%	227,959,040	32%
70%	235,082,760	33%
80%	**242,206,480**	**34%**
90%	263,577,640	37%
100%	334,814,840	47%

Contingency on Base Estimate	80% Confidence Project Cost	
Base Estimate ->	**$712,372,000**	
Estimate Contingency ->	**$242,206,480**	**34%**
Base Estimate w/Contingency (80% Confidence) ->	**$954,578,480**	

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.

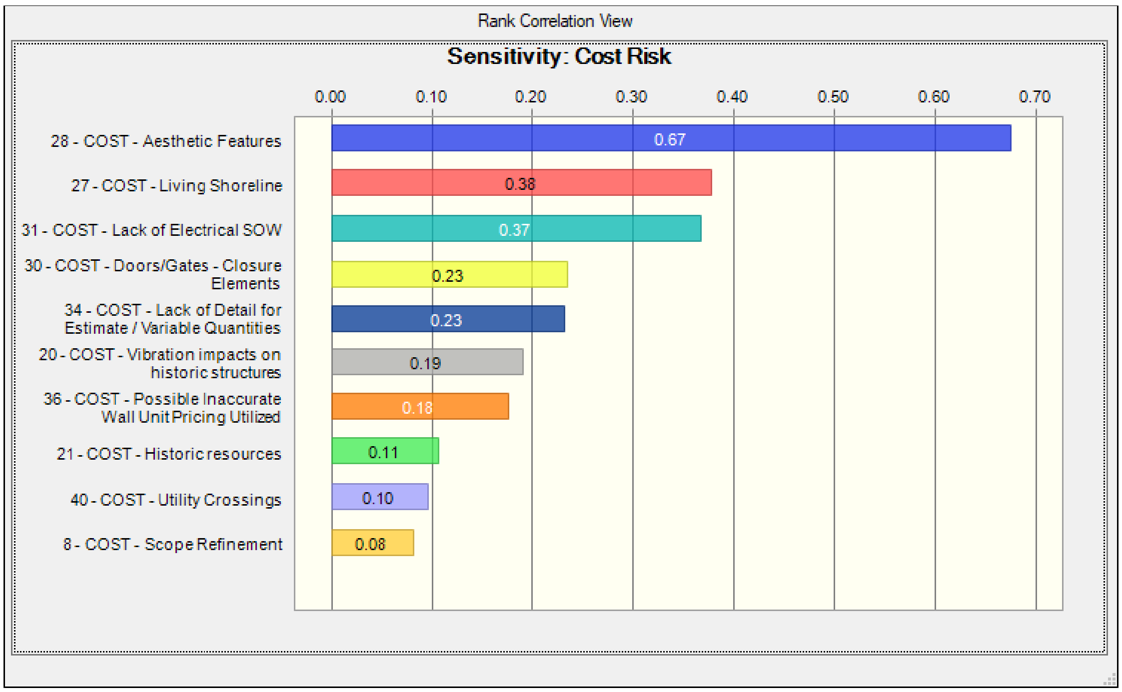
Key cost drivers identified in the sensitivity analysis can be used to support development of a risk management plan that will facilitate control of risk factors and their potential impacts throughout the project lifecycle. Together with the risk register, sensitivity analysis results can also be used 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.

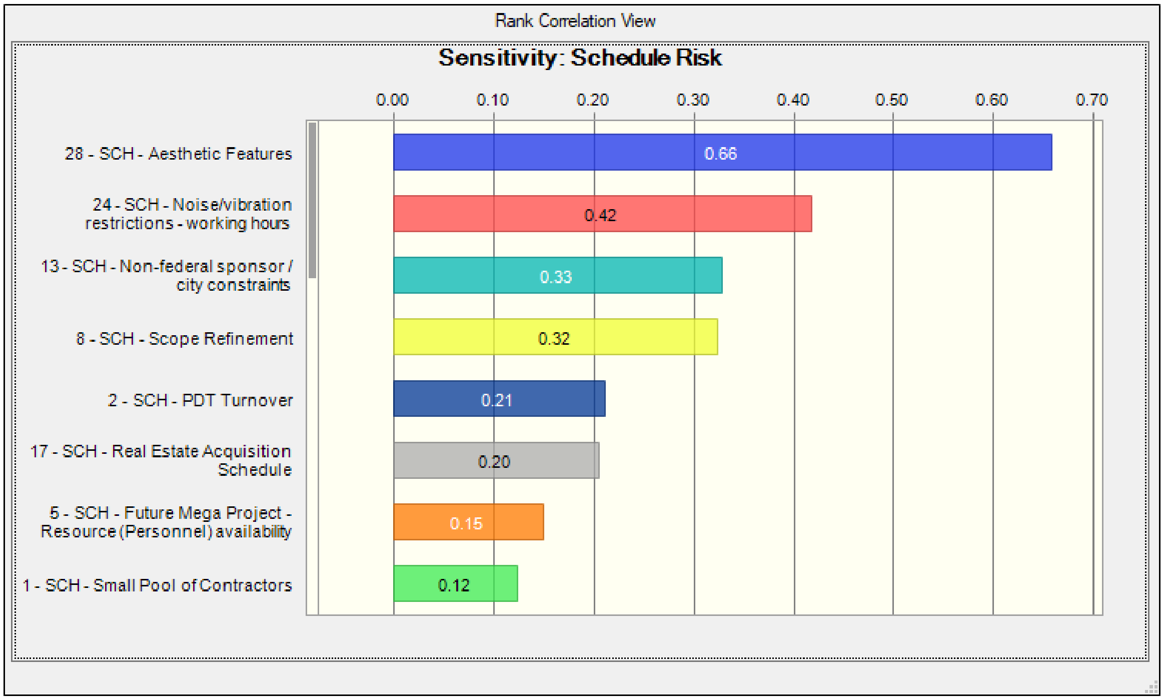
The schedule was not resource loaded and contained open-ended tasks and non-zero lags (gaps in the logic between tasks) that limit the overall utility of the schedule risk analysis. These issues should be considered as limitations in the utility of the schedule contingency data presented. Schedule contingency impacts presented in this analysis are based solely on projected residual fixed costs.

Table 2. Schedule Duration Contingency Summary

Base Schedule Duration ->	132.0 Months	
Confidence Level	**Contingency Value**	**Contingency**
0%	19.8 Months	15%
10%	26.4 Months	20%
20%	27.7 Months	21%
30%	29.0 Months	22%
40%	29.0 Months	22%
50%	30.4 Months	23%
60%	30.4 Months	23%
70%	31.7 Months	24%
80%	**31.7 Months**	**24%**
90%	33.0 Months	25%
100%	38.3 Months	29%

Contingency on Base Schedule	80% Confidence Project Schedule	
Base Schedule Start Date ->	July 1, 2021	
Base Schedule Finish Date ->	July 1, 2032	
Base Schedule Duration ->	132.0 Months	
Schedule Contingency Duration ->	31.7 Months	24%
Base Schedule w/Contingency (80% Confidence) ->	163.7 Months	
Base Finish Date w/Contingency (80% Confidence) ->	February 21, 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 $242.2M and schedule risks adding a potential 31.7 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.

- 20 Vibration Impacts on Historic Structures – 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. If damages are realized, they could cause significant cost impacts.

- 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 PED could help mitigate.

- 40 Utility Crossings – There is information missing and some inaccurate information is also currently being utilized. Impacts concerning utilities may alter dimensions and alignment of the wall structures. Cost increases could potentially be significant.

- 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 ->	$712,372,000	
Confidence Level	Contingency Value	Contingency
0%	128,226,960	18%
10%	185,216,720	26%
20%	192,340,440	27%
30%	206,587,880	29%
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70%	235,082,760	33%
80%	**242,206,480**	**34%**
90%	263,577,640	37%
100%	334,814,840	47%

Contingency on Base Estimate	80% Confidence Project Cost	
Base Estimate ->	$712,372,000	
Estimate Contingency ->	$242,206,480	34%
Base Estimate w/Contingency (80% Confidence) ->	$954,578,480	

Table 4. Construction Schedule Comparison Summary (Uncertainty Analysis)

Base Schedule Duration ->	132.0 Months	
Confidence Level	**Contingency Value**	**Contingency**
0%	19.8 Months	15%
10%	26.4 Months	20%
20%	27.7 Months	21%
30%	29.0 Months	22%
40%	29.0 Months	22%
50%	30.4 Months	23%
60%	30.4 Months	23%
70%	31.7 Months	24%
80%	**31.7 Months**	**24%**
90%	33.0 Months	25%
100%	38.3 Months	29%

Contingency on Base Schedule	80% Confidence Project Schedule	
Base Schedule Start Date ->	July 1, 2021	
Base Schedule Finish Date ->	July 1, 2032	
Base Schedule Duration ->	132.0 Months	
Schedule Contingency Duration ->	31.7 Months	24%
Base Schedule w/Contingency (80% Confidence) ->	163.7 Months	
Base Finish Date w/Contingency (80% Confidence) ->	February 21, 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

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. Not modeled as cost from schedule - Assumed related to PDT turnover and already captured above. • 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.	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).	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**

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)
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.	Unrated	Negligible	**#N/A**	Unrated	Negligible	**#N/A**
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**

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)
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 is 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**

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)
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 fairly 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**

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)
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 day-time) 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, I think the main risk will be associated also with limited work day 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**

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)
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 augering 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.	Unrated	Negligible	**#N/A**	Unrated	Negligible	**#N/A**
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**
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**

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)
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**

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)
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**
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.	Unrated	Negligible	**#N/A**	Unrated	Negligible	**#N/A**

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)
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.	Unrated	Negligible	**#N/A**	Unrated	Negligible	**#N/A**
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**
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 assume 3-years of PED. LV: Assume 3-years of PED = 0 Mo. Variance ML: Assume 6 Mo. Delay HV: Assume 12-Mo. delay which may be mitigated by the real estate schedule duration as well.	Likely	Moderate	**Medium**	Likely	Significant	**High**