Charleston District

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

Charleston, South Carolina

ENGINEERING APPENDIX

April 2020

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Contents

SUB-APPENDICES

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.

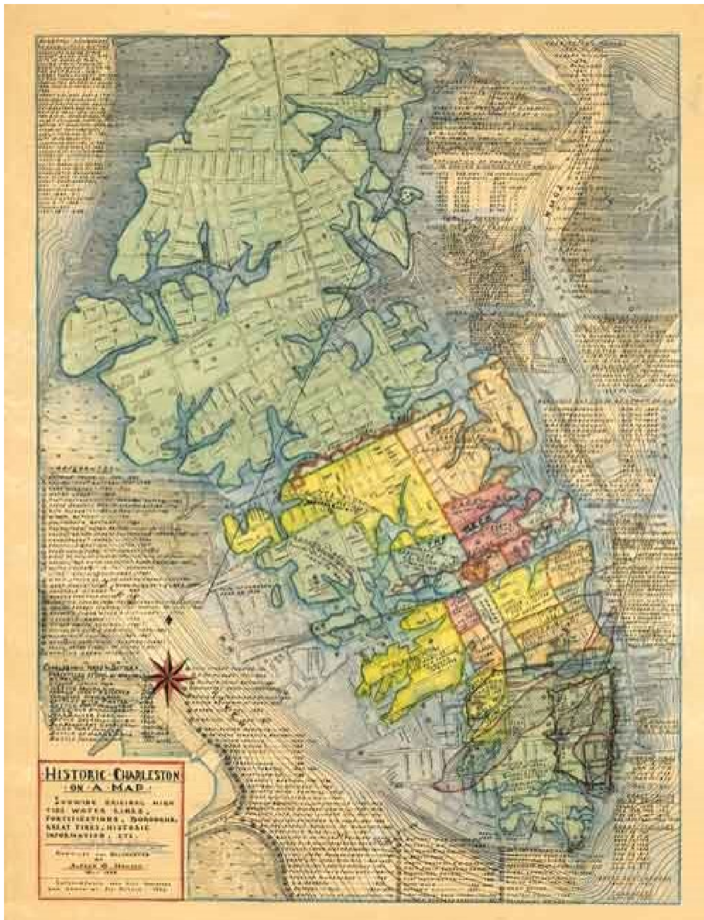

Figure 1.2: 1844 Map of Charleston

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 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, 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.

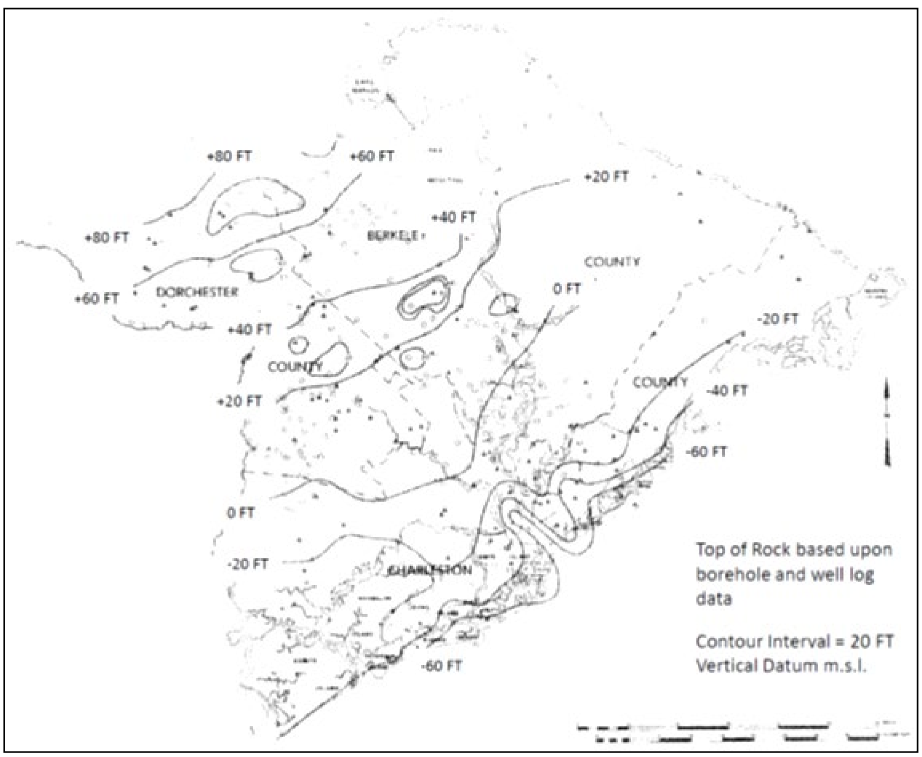

Figure 2.2.1: Structure contour map showing top of Cooper Formation, from Park (1985).

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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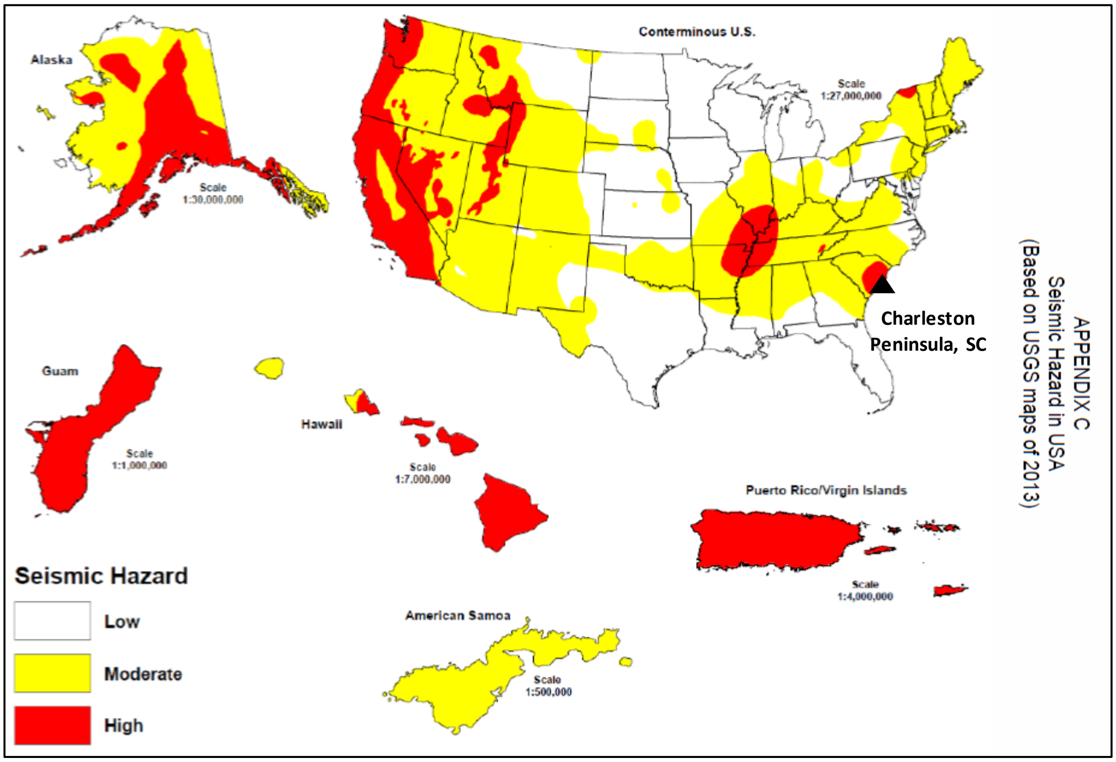


Figure 2.2.2.1: Project location shown on seismic hazard map of the USA, from ER-1110-2-1806.

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).

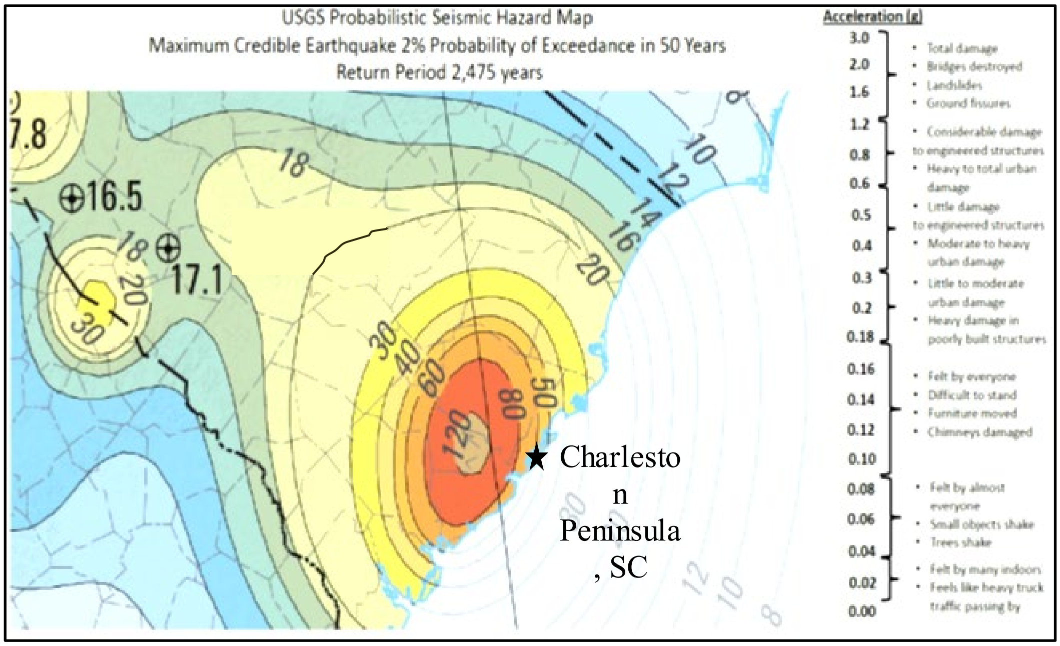


Figure 2.2.2.1.1: USGS Seismic Hazard Map, PGA, 2% Probability of Exceedance in 50 Years, from Peterson et al. (2015).

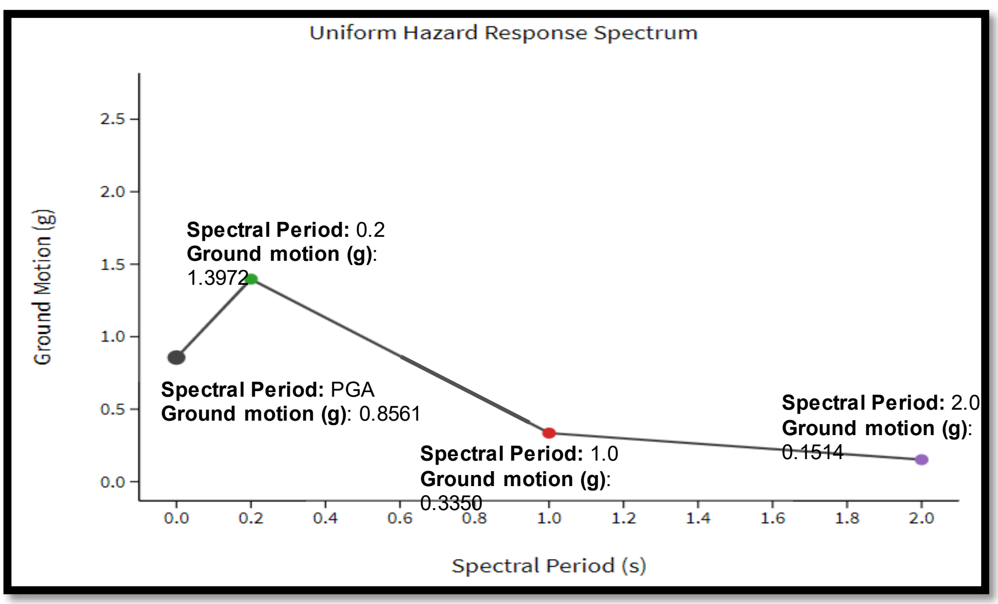


Figure 2.2.2.1.2: Uniform Hazard Response Spectrum predicted for the project site showing PGA with 2% in 50 years AEP (2,475 return period).

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.

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 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 DA but with additional lidar that could be added. CESAC obtained concurrence from the MSC that the change in flood risk of various barriers around the study area 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).

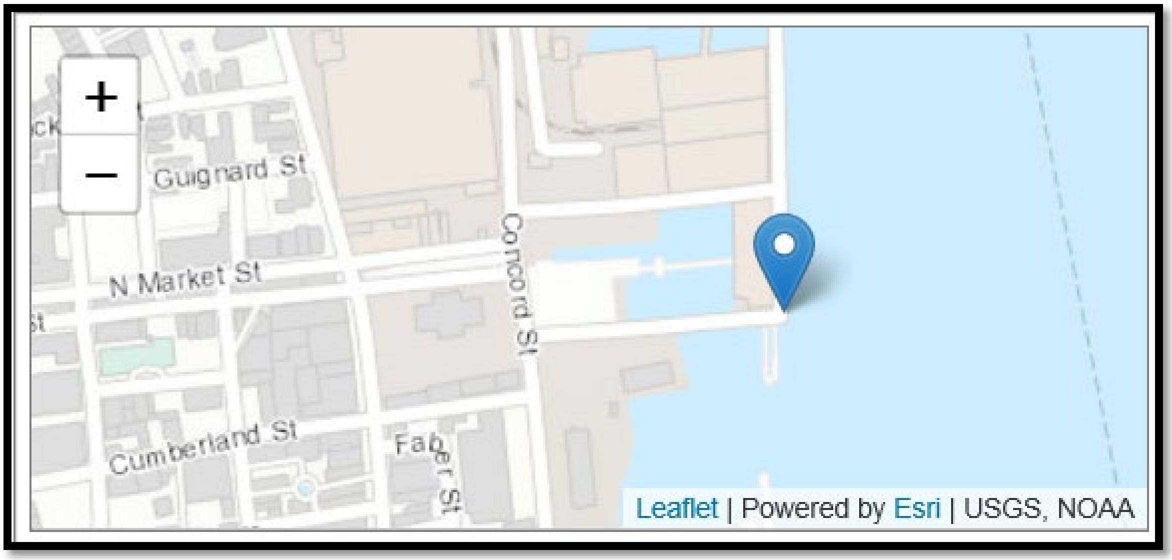


Figure 2.4.1 Location of NOAA Gage 8665530

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). (https://tidesandcurrents.noaa.gov/datums.html?id=8665530)

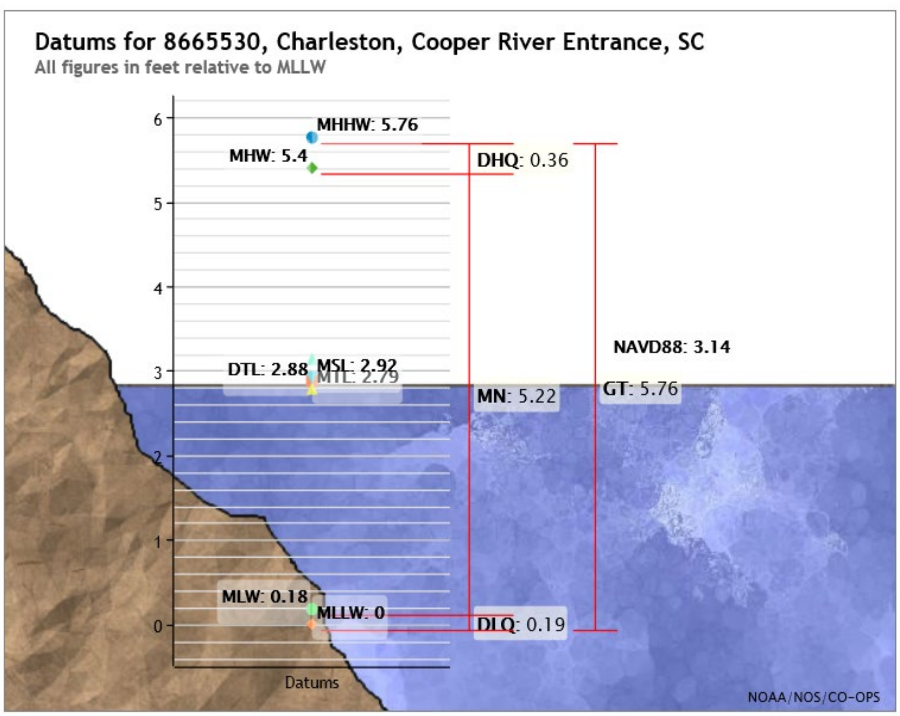


Figure 2.4.1 Tide Range Station 8665530

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 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 3.1.1 and Table 3.1.1.

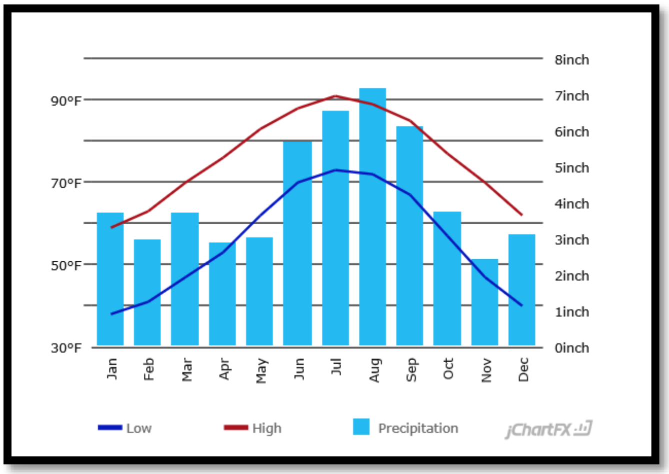


Figure 3.1.1 Charleston Temperature and Precipitation

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 & WAVES

The Post45 Harbor Deepening study documented the following information.

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

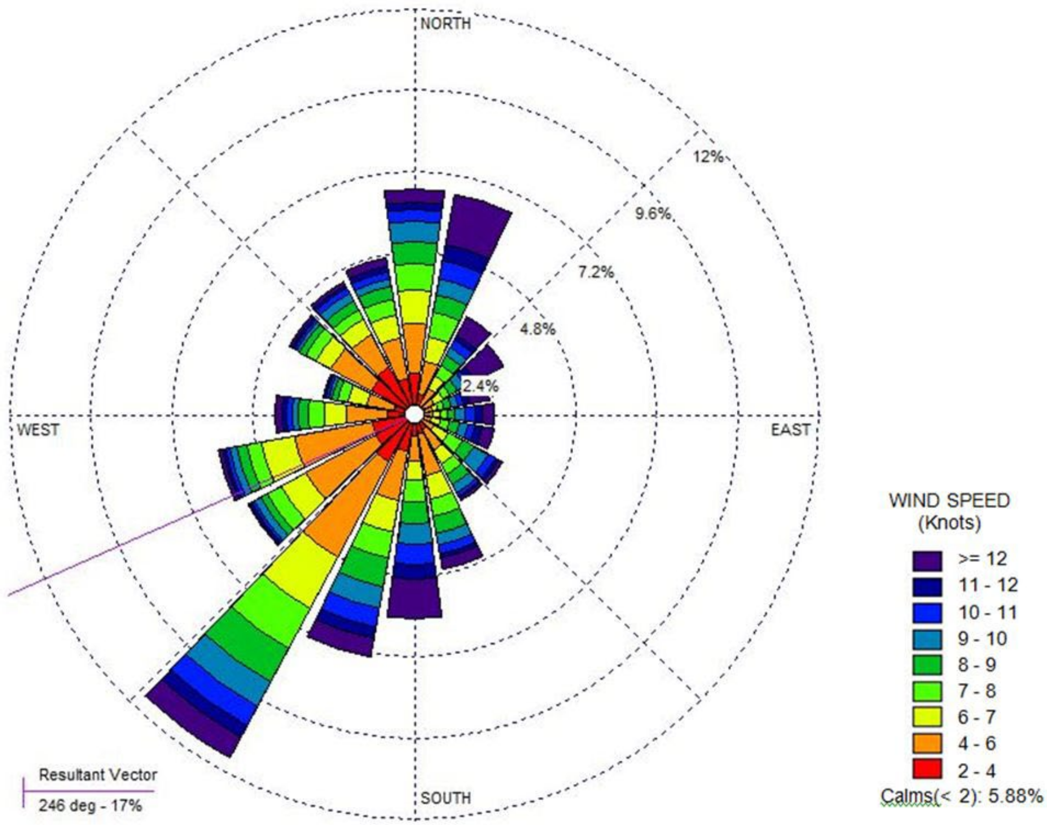


Figure 3.3.1. Wind Rose for Charleston Harbor Depicting Wind Direction and Speed Frequency

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

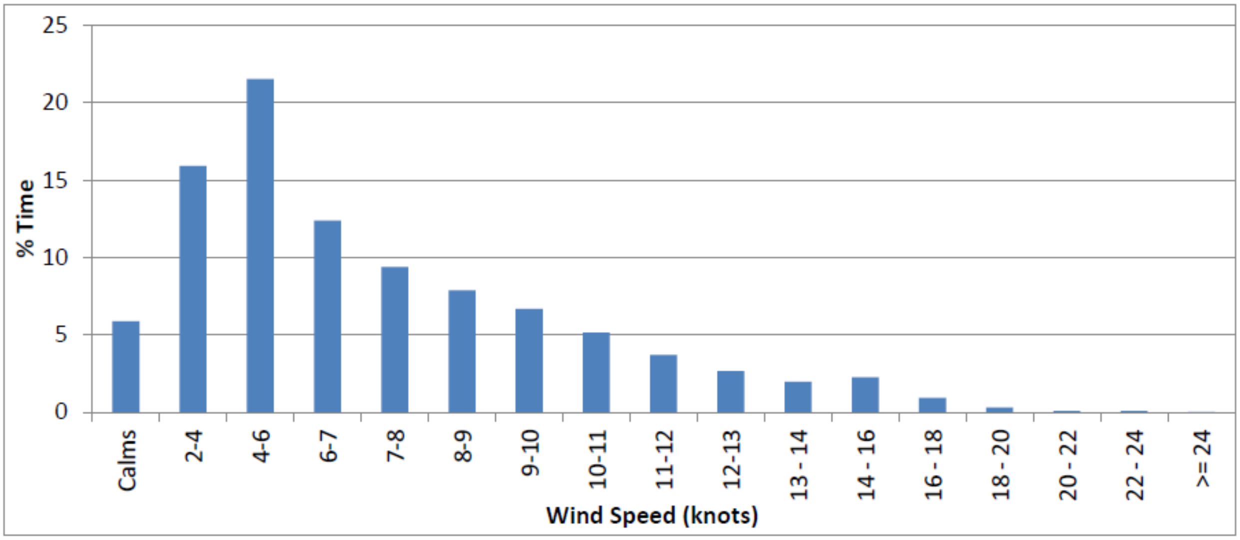


Figure 3.3.2 Wind Speed Frequency Distribution in Charleston Harbor from all directions

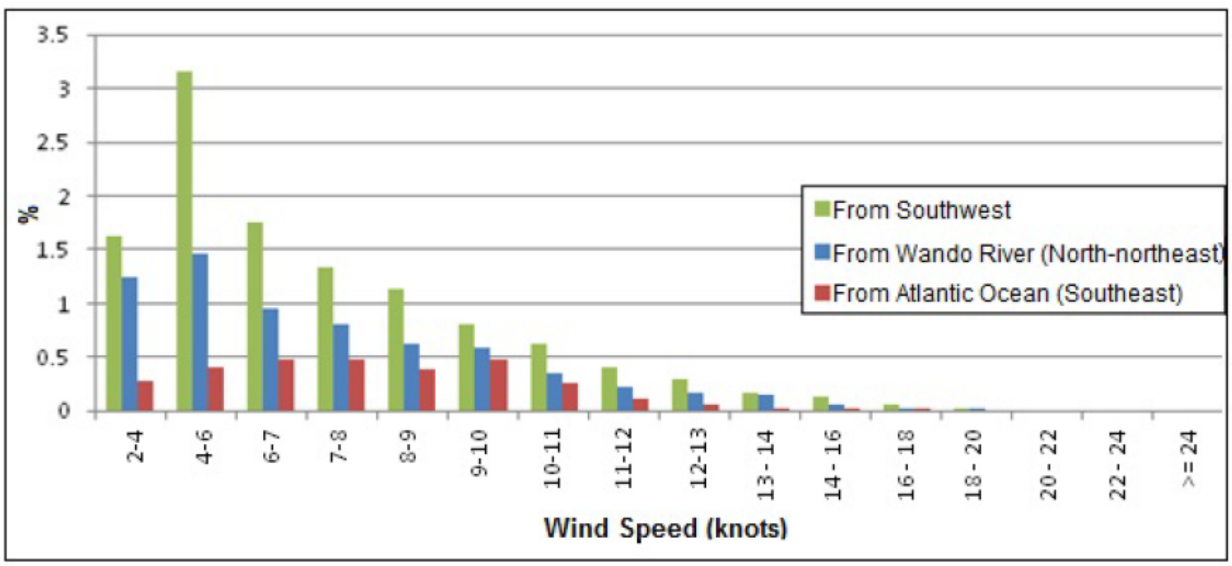


Figure 3.3.3 Wind Speed Frequency Distribution in Charleston Harbor comparing three key directions

3.4. ASTRONOMICAL TIDES & WATER LEVELS

3.4.1. ASTRONOMICAL TIDES

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

3.4.2. WATER LEVELS

The Charleston Harbor tide gauge was established in 1899. In that nearly 100-year time span, local sea level has risen 1.07 ft (Fig 3.4.2.1). 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).

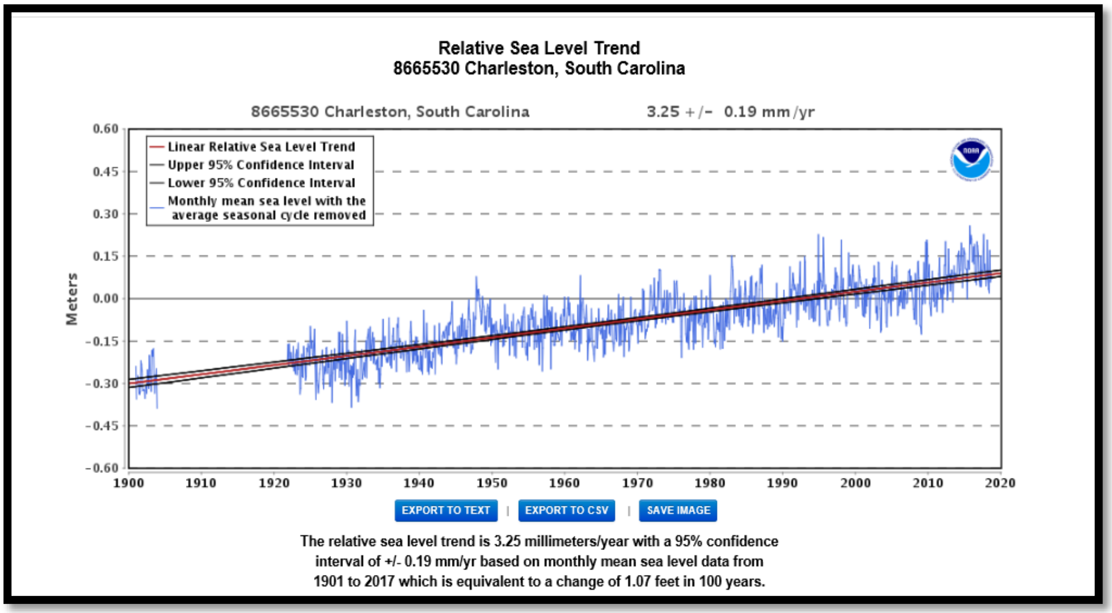


Figure 3.4.2.1 Observed Sea Level Rise at Charleston Harbor Gage

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.

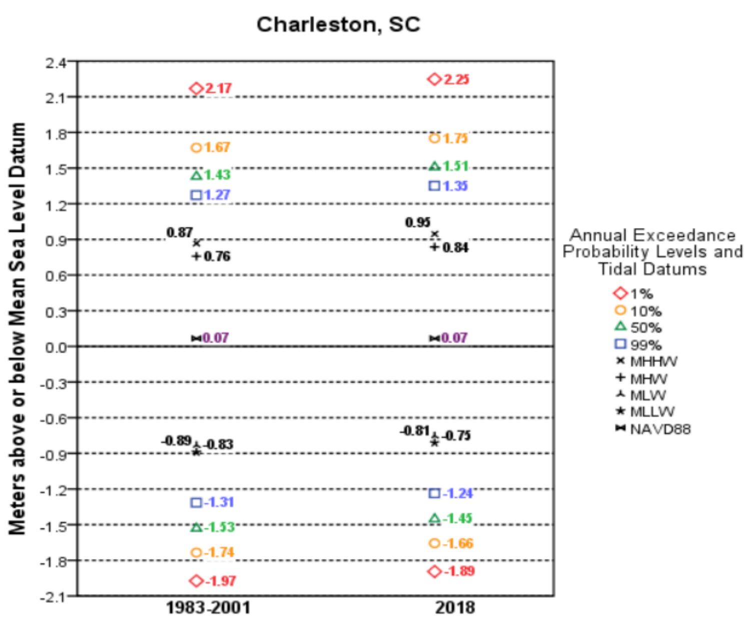
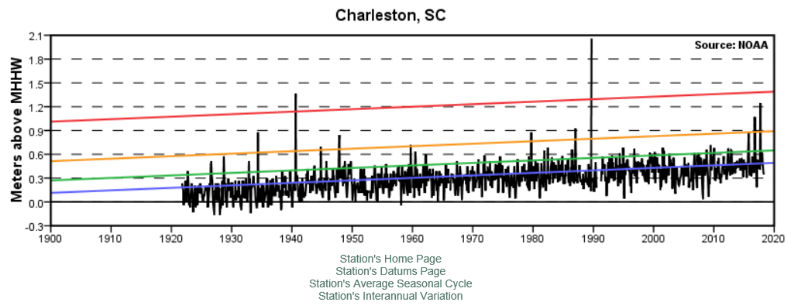


Figure 3.4.3.1 Exceedance Probability Levels and Tidal Datum of 8665530 Charleston, Cooper River Entrance, SC

As stated on NOAA website, on average, shown in Figure 3.4.3.2 the 1% level (red) will be exceeded in only one year per century, the 10% level (orange) will be exceeded in ten years per century, and the 50% level (green) will be exceeded in fifty years per century. The 99% level (blue) will be exceeded in all but one year per century, although it could be exceeded more than once in other years. The level of confidence in the exceedance probability decreases with longer return periods. Table 3.4.3.1 is tabulated in feet referenced to NAVD88. (source https://tidesandcurrents.noaa.gov/est/est_station.shtml?stnid=8665530)



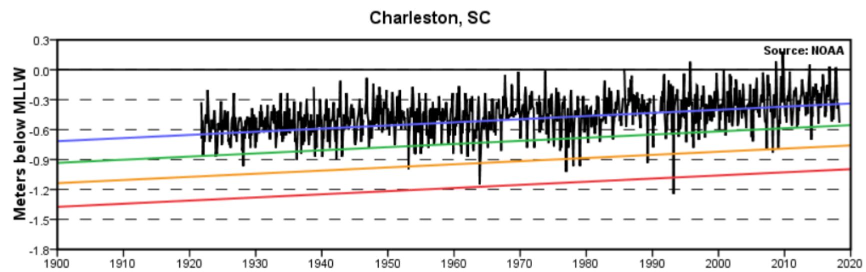


Figure 3.4.3.2 Seasonal and Interannual Variation of Gage 8665530 Extreme water Levels

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.	
*100 Yr:	7.18 (ft)
50 Yr:	6.59 (ft)
20 Yr:	5.95 (ft)
10 Yr:	5.54 (ft)
5 Yr:	5.18 (ft)
2 Yr:	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).

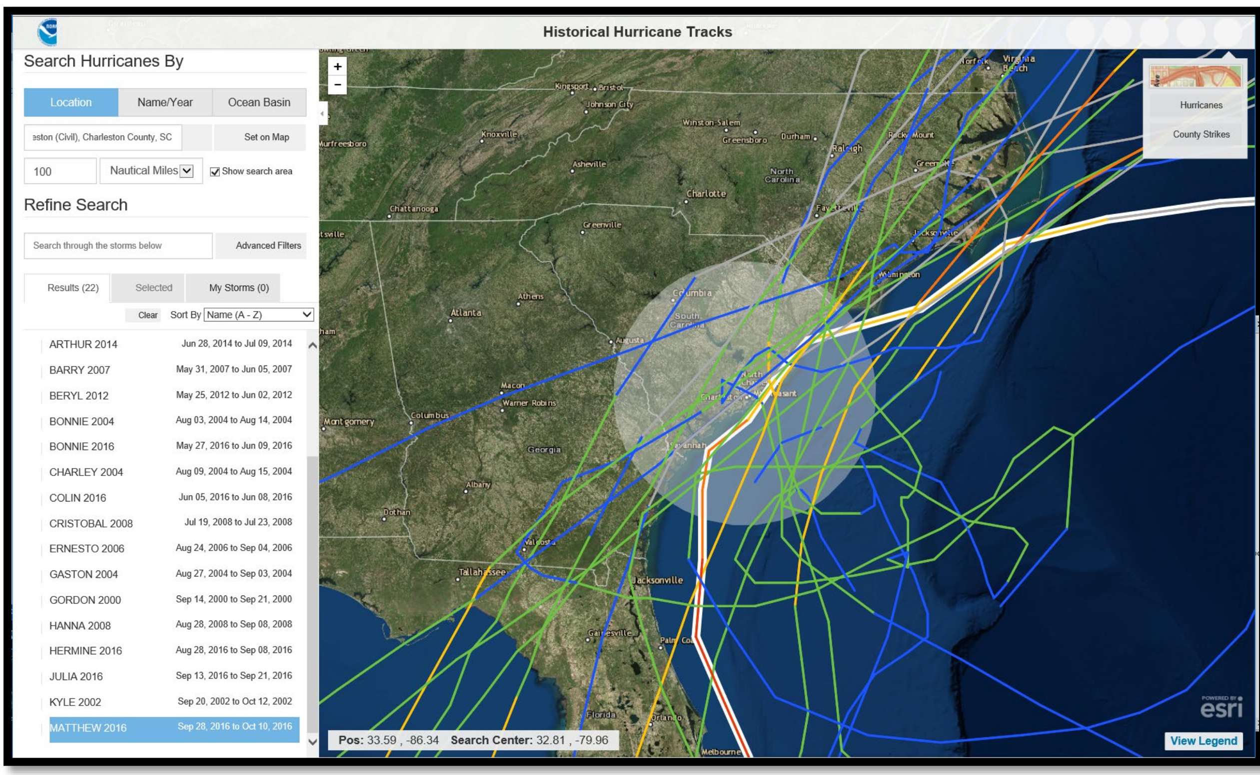


Figure 3.5.1 Twenty two storms passed within 100 nautical miles of Charleston between 2000 and 2019.

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, 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 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 6.14 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. SEA LEVEL RISE, VULNERABILITY

3.6.1. RELATIVE SEA LEVEL RISE (RSLR)

Using the USACE Sea-Level Change Curve Calculator (Version 2017.55) for the Charleston Gage 8665530 shown in figure 3.6.1.

Figure 3.6.1 Location Charleston Gage 8665530

The historic rate of future RSLR (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. According to National Oceanographic and Atmospheric Administration (NOAA) for the Charleston Gage 8665530, NOAA's 2006 Published Rate is 0.01033 feet/yr. Shown in Figure 3.6.2. 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. ETL 1100-2-1 (*pdf, 9.87 MB*), Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation.

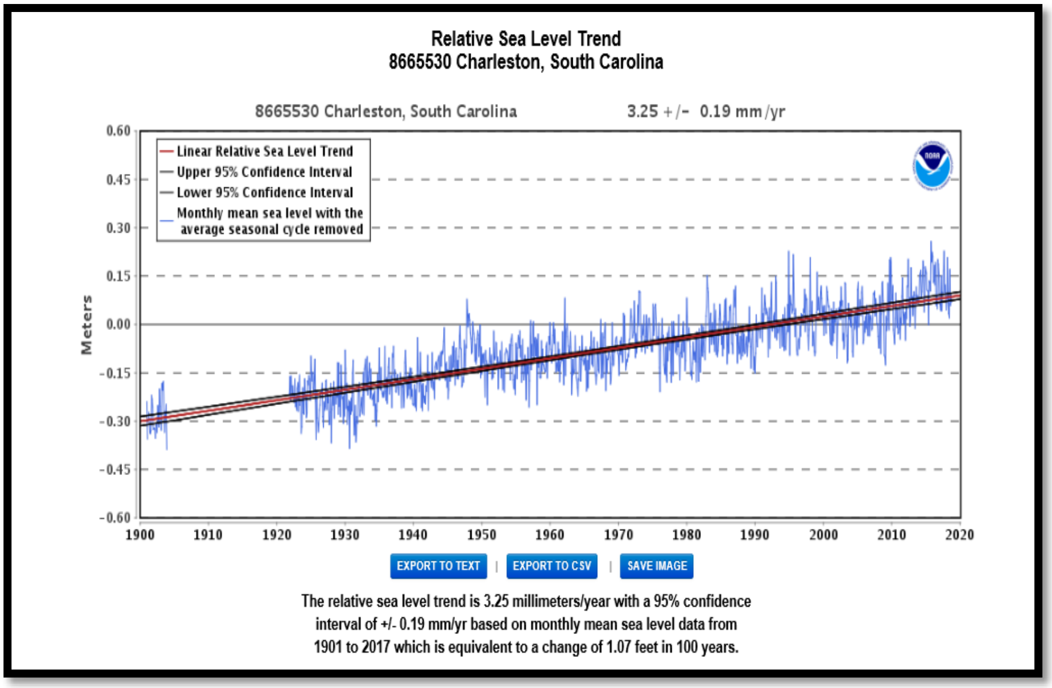


Figure 3.6.2 Relative Sea Level Trend

The future condition 50 years after construction assumed to be 2025. Alternatives were evaluated using the most likely SLR of the intermediate rate. Historic rate has already been shown to be changing, based on recent trends. The low rate would be 0.53 feet in 50 years, but the recent trend in Charleston is a rise of about 1/8 of an inch each year according to https://www.charleston-sc.gov/index.aspx?NID=1577.

Therefore, it was agreed to use the intermediate rate for alternative evaluation, which would result in 1.13 by 2075, once 0.12 is subtracted for year 2019 to reflect Relative Sea Level Rise. Table 3.6.1 indicates the incremental rate of sea level rise for the 50 year project life as well as the 100 year project into the future. Figure 3.6.3 plots the same information.

Table 3.6.1 Estimate Relative Sea Level Change

Estimated Relative Sea Level Change from 2019 To 2150 Charleston Peninsula
8665530, Charleston, SC
NOAA's 2006 Published Rate: 0.01033 feet/yr
All values are expressed in feet

Year	USACE Low	USACE Int	USACE High
2019	0.00	0.00	0.00
2020	0.01	0.02	0.03
2025	0.06	0.09	0.20
2030	0.11	0.18	0.38
2035	0.17	0.27	0.58
2040	0.22	0.36	0.80
2045	0.27	0.45	1.04
2050	0.32	0.56	1.30
2055	0.37	0.66	1.57
2060	0.42	0.77	1.87
2065	0.48	0.88	2.18
2070	0.53	1.00	2.51
2075	0.58	1.13	2.86
2080	0.63	1.25	3.23
2085	0.68	1.39	3.62
2090	0.73	1.52	4.02
2095	0.79	1.66	4.45
2100	0.84	1.81	4.89
2105	0.89	1.96	5.35
2110	0.94	2.11	5.83
2115	0.99	2.27	6.33
2120	1.04	2.44	6.85
2125	1.10	2.60	7.38
2130	1.15	2.78	7.94
2135	1.20	2.95	8.51
2140	1.25	3.13	9.10
2145	1.30	3.32	9.71
2150	1.35	3.51	10.34

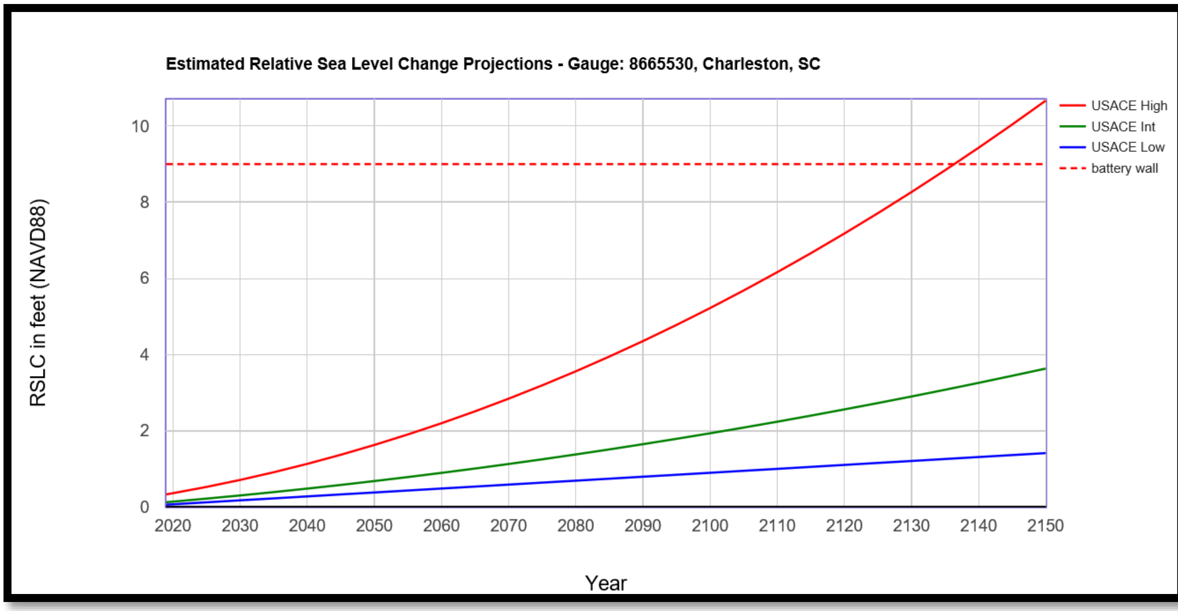


Figure 3.6.3 Estimated Relative Sea Level Change

Estimating a 50 year project life ending in 2075, the "low" rate of change is 0.58 feet, the "intermediate" is 1.13 feet and the "high" is 2.86 feet. At 2125, the changes are 1.104, 2.60 and 7.38 feet, respectively, for the low, intermediate, and high rate scenarios.

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, 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 a 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. The three scenarios were: existing, future without and future with a breakwater as a wave attenuator. 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. The highest wave generation during storm events, based on past experiences, is at the battery, thus a wave attenuator was included in one alternative. See discussion of wave attenuation and the breakwater design in the Engineering Appendix.

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). "

Since G2CRM includes tide and sea level rise, the Stillwater elevations are generated in meters at MSL and were then converted to feet MSL. The G2CRM model was then used to evaluate wall footprint and elevations as a stand-alone option (Alternative 2) and in conjunction with a breakwater wave attenuator (Alternative 3).

See the Sub-Appendix 5 COASTAL MODELING SUB-APPENDIX for the ERDC modeling report that includes the STWAVE modeling and the ADCIRC modeling.

The final recommended structures will be incorporated into the ADCIRC and STWAVE models and evaluated for impacts outside the project area under the three sea level rise scenarios outlines in ER 1100-2-8162, INCORPORATING SEA LEVEL CHANGE IN CIVIL WORKS PROGRAMS.

4.2 RESULTS

Comparison of the future with breakwater to the future without condition, indicated expected changes at the breakwater. While changes in maximum water elevation at the save points were found in remote areas where changes would not be expected (i.e. distant tributaries, single points inland) due to the addition of the breakwater in the model, it has been concluded that these anomalies are due to poorly- resolved tributaries when using the available bathymetry. This discontinuity results in erroneous changes in water levels on single storm simulations, and for one tributary it occurred on three of the storm simulations when the breakwater was added to the model. It is important to note that the majority of the shorelines along the surrounding areas (West Ashley, James Island, Mt. Pleasant...) had zero changes in water level, and none had changes greater than 1 inch, which is within the accuracy of Coastal modeling

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.

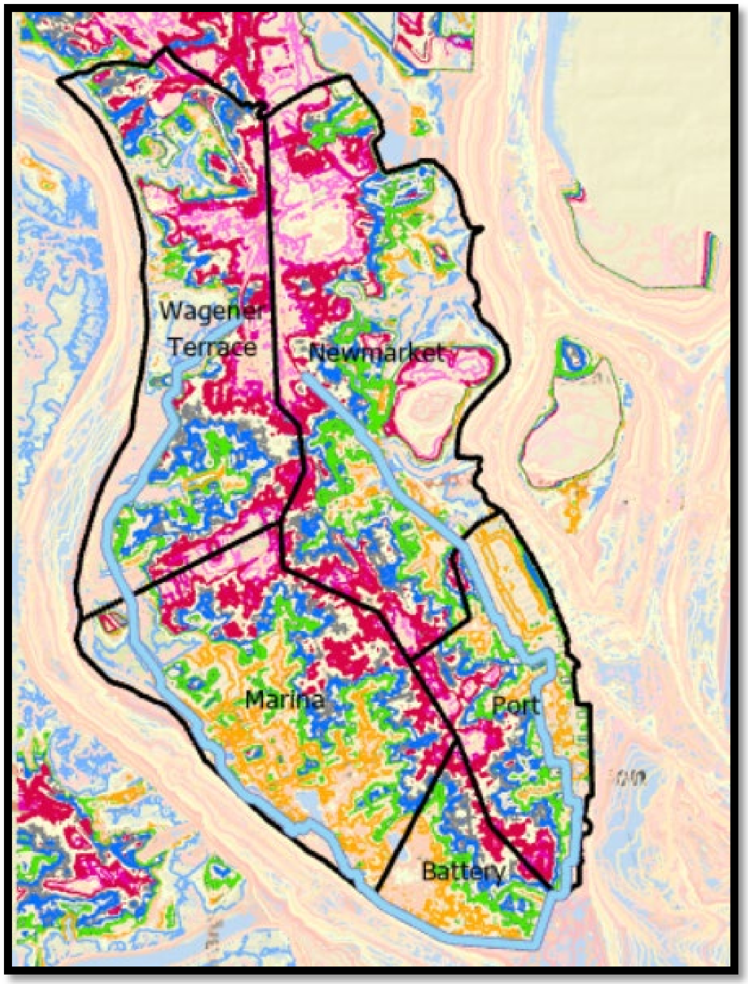


Figure 5.1.1 Map depicting Model Areas

5.2. ADCIRC WATER LEVELS

Using the FEMA analysis of still water elevation levels, ERDC generated Annual Exceedance Probability (AEP) for each of the save points submitted by the District. From that dataset of over 1000 points, 5 were selected to represent the Model Areas used for G2CRM (figure 5.2.1). To estimate the future condition 1.13 feet was added for SLR (table 5.2.1).

Table 5.2.1 Annual Exceedance Probability at the 5 Model Area save points

Model Area	**AEP% 50**	**AEP% 20**	**AEP% 10**	**AEP% 4**	**AEP% 2**	**AEP% 1**	**AEP% 0.5**	**AEP% 0.2**	**AEP% 0.1**
	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88
Wagener Terrace	5.9	6.2	6.5	7.0	9.4	11.5	13.2	15.6	17.3
Marina	5.8	6.2	6.4	7.0	9.5	11.5	13.4	15.8	17.6
Newmarket	5.8	6.2	6.4	6.9	9.5	11.5	13.3	15.7	17.6
Port	5.8	6.2	6.4	6.9	9.4	11.4	13.3	15.8	17.7
Battery	5.8	6.2	6.4	6.9	9.4	11.5	13.4	15.9	17.8

Figure 5.2.1 Location of Save points for the Model Areas

5.3. PROJECT ALIGNMENT

The primary criteria was to avoid personal property for footprint, avoid taking houses/businesses unless there is no other option. Additional criteria was to take advantage of existing topography, such as the abutment of the Highway 17 bridges over the Ashely River, 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 has not been finalized. Further evaluation of the optimum elevation will be evaluated under optimization and submitted as the final recommendation in the final report. Assessments of impacts are based on a wall at elevation 12' NAVD88.

Table 5.3.1 Permanent and Construction Easements.

Feature	Permanent Easement		Construction Easement		Buffer (just to shift alignment)
	Riverside of Centerline	Interior Side, from Centerline	Riverside of Centerline	Interior Side, from Centerline	Interior Side, from Centerline
T-Wall (2-ft thick stem)	16 feet	~20 feet (will depend on location in footing and length of battered piles)	35 feet	25 feet (10 feet for excavation and additional 15 feet beyond excavation)	Additional 15 feet between tip of battered pile to existing structure foundations
Combo Wall (10-ft wide cap)	20 feet	~25 feet (will depend on location in footing and length of battered piles)	35 feet	35 feet	Additional 15 feet between tip of battered pile to existing structure foundations

This criteria resulted in the following eliminations and assumptions:

1. Storm Surge Protection structure type: An earthen levee embankment was eliminated as 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. 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.

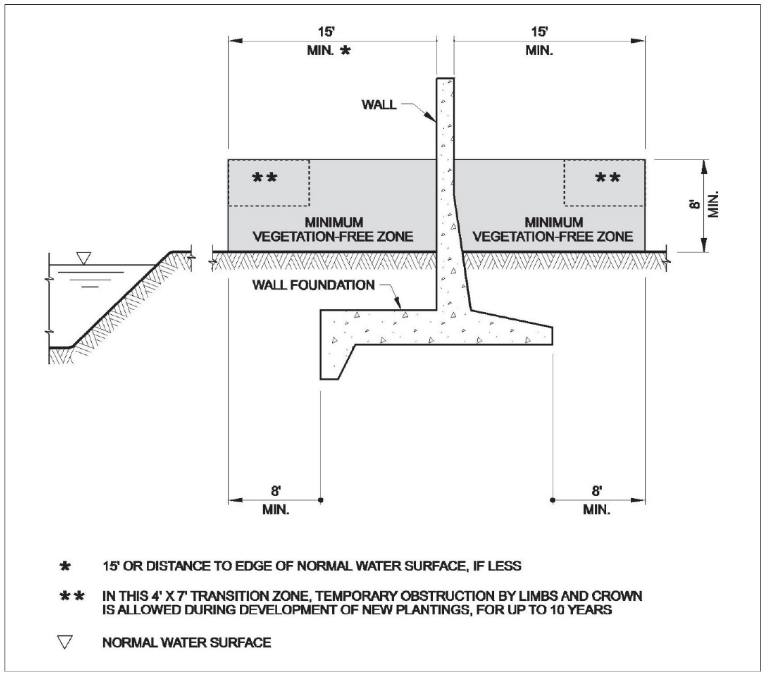


Figure A-16. Inverted-T Type Floodwall.

- 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 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 steel pipe 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 doesn'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 - ~35 ft clearance from MHW (Combo Wall)
- Ravenel Bridge - ~25 ft clearance from existing grade in the parking lot (T-Wall)
- The wall connects to high ground at the abutments of US 17, as there is no clearance under the bridge. This elevation is limited to 12 NAVD88. Anything higher will require gates across the bridge which will limit access and evacuation.

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.

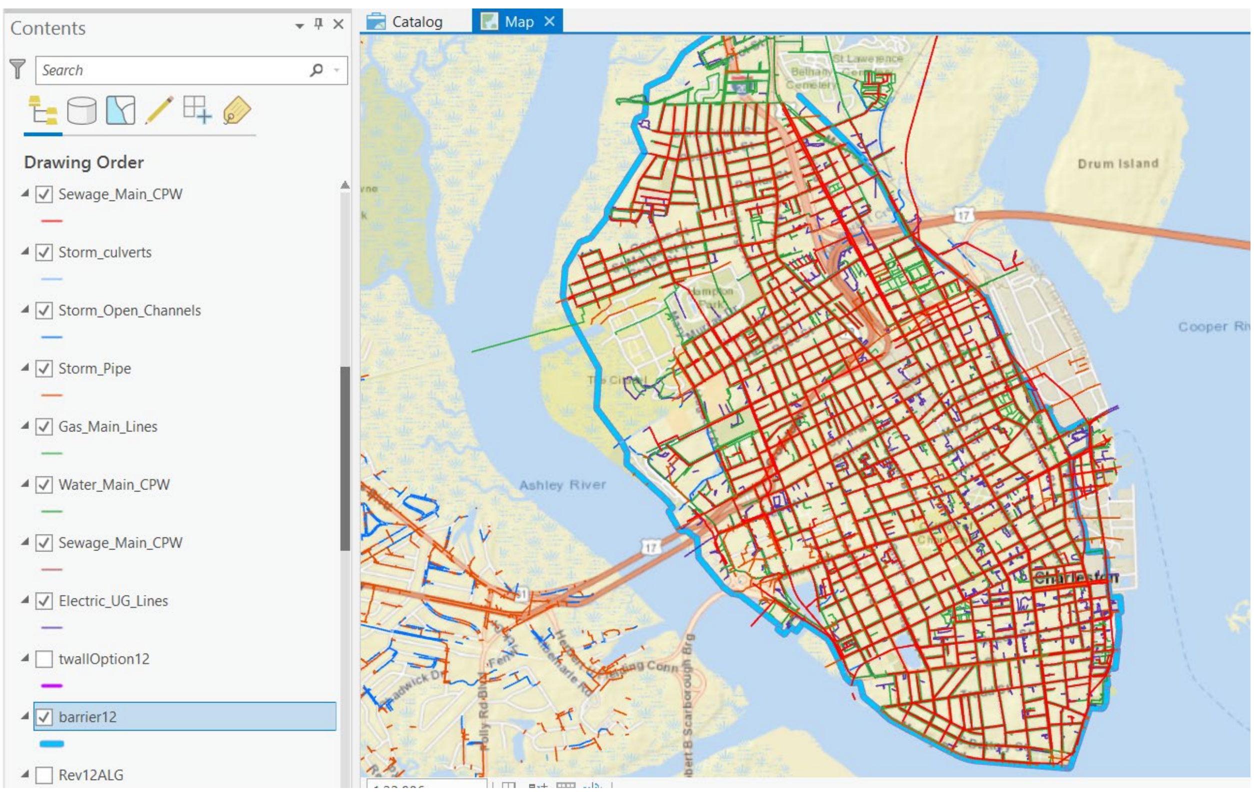


Figure 5.3.2. Utility Dataset Used in Project Alignment Optimization and Cost Considerations

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.

Small areas of development within the study area are excluded from being inside the wall, but those are addressed by nonstructural solutions.

Figure 5.3.3 shows the recommended footprint evaluated. Further refinement of footprint will occur during optimization during post TSP.

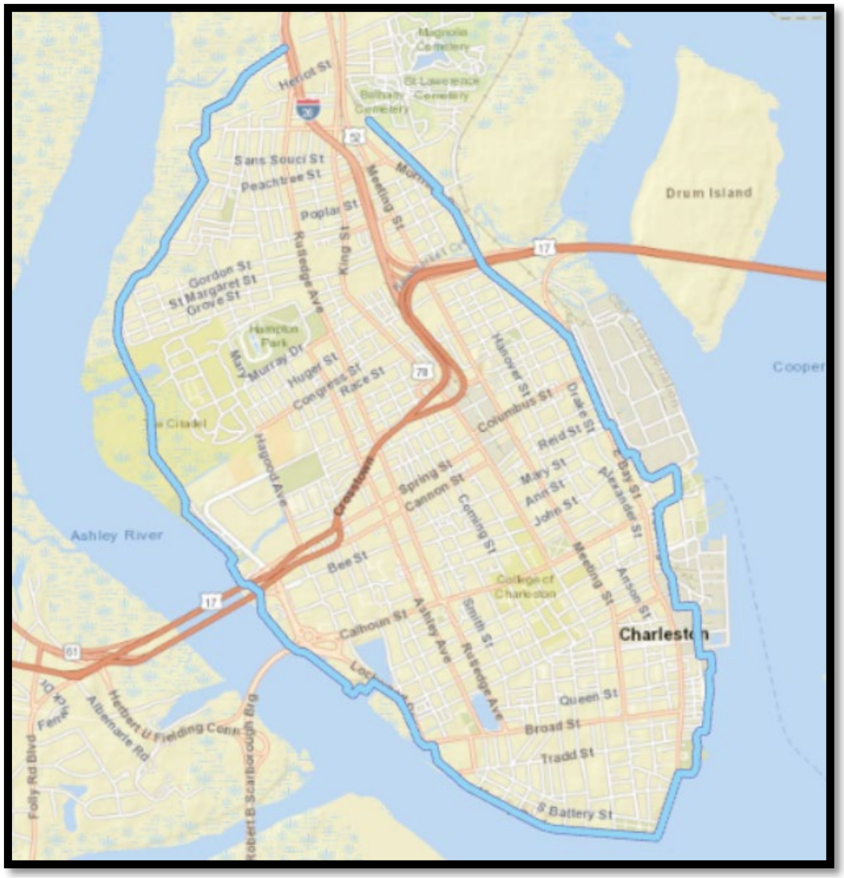


Figure 5.3.3 Alignment of the Perimeter Storm Surge Wall

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 was assumed to be installed to reduce underseepage and uplift on the wall. It was assumed that the sheetpile would be 20 feet long for the EL. 12 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.

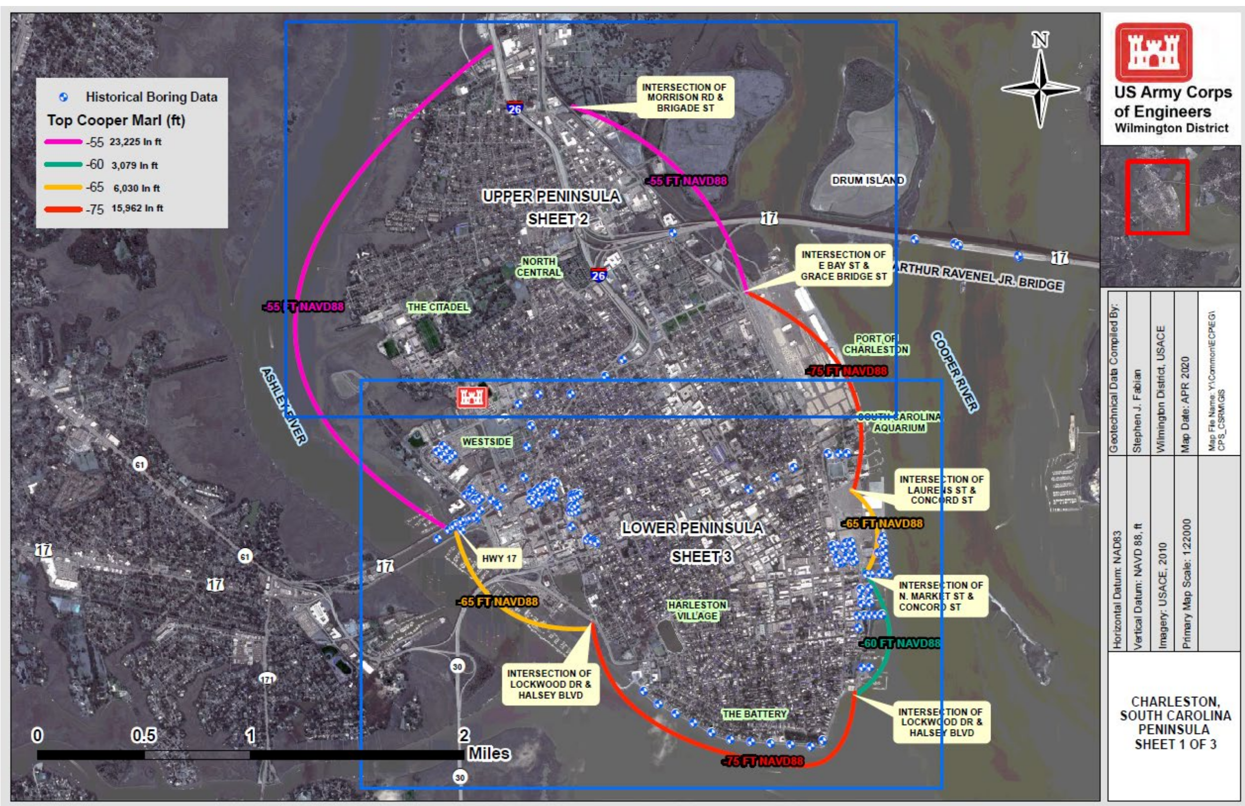


Figure 5.4.1: Assumed Top of Cooper Marl

5.5 STRUCTURAL ASPECTS OF THE FEASIBILITY MEASURES

5.5.1 I-WALL

Due to barrier height uncertainties and unknowns at this stage of the study, I-Walls were not utilized when determining location of the barrier. I-Walls will consist of driven sheet pile walls with a concrete cap. For the purposes of this study and due to soil conditions, I-Walls will only be considered for barriers whose top elevations were 4 feet or less above the current finish grade. Due to the small footprint requirements of the I-Wall, it is more viable where space and land is limited, as well as locations where the existing land elevation is higher and shorter wall heights are needed. As stated earlier, I-Walls were not utilized at this stage of the study, but will be considered during optimization of this study.

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. During optimization it is anticipated that the T-Wall will be used where existing grades are at the lowest elevations. These lower elevations will primarily occur where the barrier is anticipated to be constructed within the marsh, or near the MHW line. 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. 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.

5.5.3 COMBO WALLS

For purposes of this study, a Combo Wall was assumed to be used where the barrier is located in the march. A Combo Wall is a combination of a large-diameter steel pipe piles with sheet piles installed in between to form a surge barrier structure. Due to soil conditions and required loads, the Combo Wall will require battered piles to provide sufficient lateral support.

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. 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 2 locations where head clearance is a concern.

- James Island Connector - ~35 ft clearance from MHW (Combo Wall)
- Ravenel Bridge - ~25 ft clearance from existing grade in the parking lot (T-Wall)

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 City of Charleston is currently completing a multi-phased construction project that consists of raising the low battery to the same elevation as the existing High Battery Wall. Upon speaking to the designer of record for this design, it was determined that the new Low Battery Wall will provide a level of protection to elevation 9 ft NAVD88, and has been designed to provide a level of protection of elevation 12 ft NAVD88 upon retrofitting the structure. Therefore, if the barrier is to be above elevation 9 ft NAVD88, the Low Battery Wall will need minor construction work done, but no structural upgrades will be required. However, if a level of protection of greater than 12 ft NAVD88 will require more detailed analysis, and may require major structural upgrades to the foundation system and/or structural diaphragm and framing.

5.5.7 HIGH BATTERY WALL

After reaching to multiple sources with the City of Charleston and Public Library, it was determined that we do not have accurate data about the construction of the high battery wall. In addition, given its age and assumed construction techniques used for the time period of which it was constructed; it is a safe assumption that the high battery wall will not meet the criteria to be part of the Federal project.

5.6 GATES

No analysis was performed to date; however, existing gate information from the NAO feasibility study and gate information obtained during a site visit to New Orleans, were used as a go-by at this stage. 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 40 vehicle gates required. Due to the simple design and lower maintenance costs, swing gates and slide gates will be prioritized. More information about each gate type is shown below:

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. In the event that the gate needs to be closed, the gate is simply swung around by the hinges and into place, then secured to the wall on the opposite side of the hinges. Compressible seals along the bottom and sides of the gate provide a water tight seal. See an example photo of a swing gate from New Orleans, LA in Figure 5.6.1.1 below. However, swing gates require large clearances to be able to close them, and they remain exposed to the elements even when not in use.

Figure 5.6.1.1: Swing Gate Example (Photo by USACE Charleston District)

5.6.1.2 SLIDE GATES

Slide gates are 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. Slide gates do not have the same clearance issues as swing gates, and can potentially be stored within the wall itself keeping 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 5.6.1.2.1 and 5.6.1.2.2 below.

Figure 5.6.1.2.1: Slide Gate Example (Photo courtesy of USACE Norfolk District)

Figure 5.6.1.2.2: 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 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.

5.6.2 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 such as storm water outfalls, tidal creeks, and marsh areas within the study area. The gates will remain open the majority of the time to allow normal passage of storm water, 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. The exact type of gate installed depends on the individual location and area it is protecting. Figure 5.6.3.1 shows the Preliminary locations of Storm Gates. These are located at all tidal creeks. Further evaluation of wetland viability will determine additional locations of storm gates.

Figure 5.6.3.1 Preliminary locations of Storm Gates

Gate types include miter gates and sluice gates. More detail is provided in the individual gate descriptions below:

5.6.3.1 MITER GATES

There is one location where the wall crosses a tidal inlet that has a boat launch, which means that an opening large enough for boat traffic will have to be maintained. A miter gate was chosen as they are heavily used in similar flood control systems all over the world and can easily accommodate the width requirements of the boat launch area. As shown in the example photo below, a miter gate consists of two metal gate sections orientated into a shallow vee shape. The point of the vee is positioned on the high water side. Then it only takes a small difference in water height and the resulting hydrostatic pressure helps push the two gate sections together and provide a water tight seal. The below picture (Figure 5.6.3.1) is an example of a miter gate from a flood mitigation project in Virginia.

Figure 5.6.3.1: Miter Gate Example (Photo courtesy of USACE Norfolk District)

5.6.3.2 SLUICE GATES

Other areas within the study where the wall crosses tidal creeks and marshes that do not need to accommodate boat traffic, sluice gates shall be utilized. Sluice gates were chosen 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, 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 shall 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 approximately 18 sluice gates required in the study area.

Figure 5.6.3.2: An Example of a Pair of Sluice Gates (Photo courtesy of USACE Norfolk District)

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 sea weed, algae and 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. Note that this replacement does not include the miter gate, which due to its' larger size will be built robust enough to withstand the elements and last the full life of the project.

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. More detail is provided on each category and subcategory in the Structural sub appendix.

Gate closure procedure will be finalized during PED phase. The plan is that as NOAA predicts storm surges equal or greater than major flooding, the storm gates will be closed at low tide, 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).*

There are a variety of gates required in the study area to ensure water tightness of the barrier during storm events. 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.

5.8 CONSTRUCTION PHASING

Footprint broken out by MA for costs will be considered in construction phasing which will consider separate or combinations of MA that will provide protection - taking into consideration the high ground of the city.

5.9. WAVE ATTENUATION

Due to observed high wave action observed during past storm events as the fetch of the harbor generates waves, a wave attenuator was considered near the battery to reduce wave action. The exact type of wave attenuator will be designed in the PED phase of the study. The numerical model used cannot evaluate most wave attenuators, or changes to the wall exterior, but it can evaluate a breakwater. Therefore, an assessment will be made assuming a breakwater. Exposed breakwaters act to reduce waves, while submerged breakwaters or sills act as barriers to shore normal sediment motion, and they have less wave attenuation. Therefore, an exposed breakwater was evaluated.

Guidance indicated that one source of water levels could be the FEMA FIS. The VE area around the battery is indicated at 15 NAVD88 for the 1% exceedance. Adding 1.13 ft for RLSC, results in 16.2 feet NAVD88 for the elevation of breakwater.

Alignment was selected to be parallel to shoreline. Guidance indicates water depths are usually limited to 30 feet or less due to the high cost of construction in deeper waters.

NACCS generic breakwater indicated 1V:1.5H side slope, as shown in the figure 5.9.1, but our estimate assumed 1V:2H and top width of 18 feet.

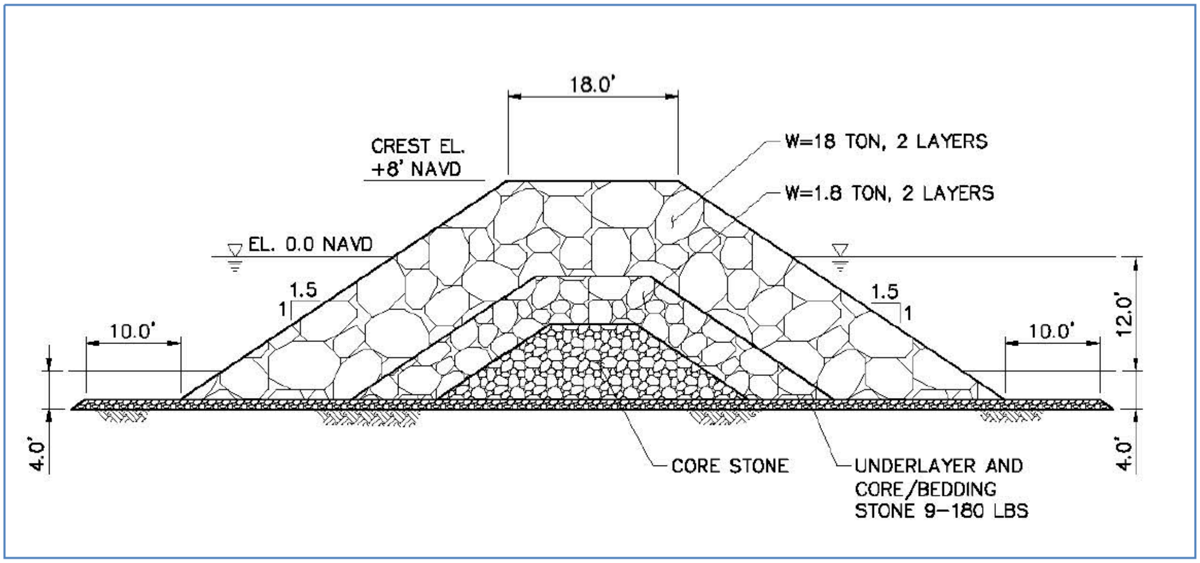


Figure 5.9.1 typical detail of breakwater from NACCS

The placement had to avoid encroachment into federal channels, which are shown in figure 5.9.2. as well as not block any water access features along the perimeter (i.e. marinas, yacht clubs…) The footprint terminated 100 feet from the top of the cutoff template of the Ashley River Federal channel.

Figure 5.9.2 Depiction of the federal channels near the study area.

There was a lack of detailed bathymetry in the area, based on available contours it was assumed that the bottom ranged from 16 to 20 feet. Therefore the footprint should be approximately 160 feet wide.

Summary of break water projects from 1993 report indicates that they can be close to the shoreline note Chesapeake is 39 to 44 meters. The breakwater was place with the centerline 310 feet off battery wall, with a toe is 230 feet off battery wall, as shown in Figure 5.9.3.

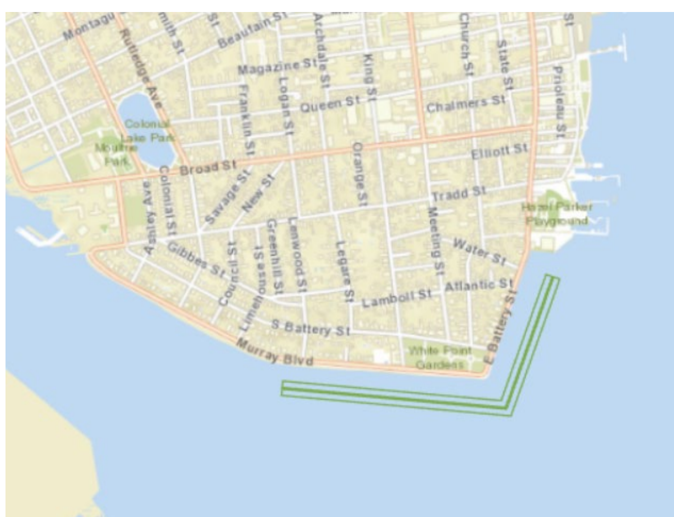


Figure 5.9.3 Location of Breakwater

5.10. INTERIOR FLOODING ANALYSIS

The HEC-RAS 2D computational hydraulic modeling goal of the feasibility study is to conduct an interior flooding analysis on the Charleston peninsula. 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. 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%, 5%, 4%, 2% and 1% Annual Exceedance Probability (AEP) is being evaluated through the HEC-RAS model combined with different exterior tidal boundary conditions in a steady state.

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. 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 project, 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 areas 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.

The City of Charleston contractor 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 the above projects. 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, not evaluating the existing or proposed pump systems.

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%, 5%, 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. Further assessment over the wall overtopping will be conducted during the PED phase.

5.10.1.1. SOFTWARE

a. HEC-RAS 5.0.7. The latest version of the Hydraulic Engineering Center's (CEIWR-HEC) River Analysis System (HEC-RAS) is being utilized to model the complex flow of rainfall runoff within the interior and will eventually be used to evaluate different hydraulic alternatives to remove interior flooding such as gates within the wall and pump stations.

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.

5.10.1.2 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 to generate a Manning's roughness layer.

b. Model Revision

The model used in that effort has been obtained and revised to perform the analysis for this current effort. (Figure 5.10.1.2.1) Revisions from the original model have primarily been in the resampling of the 2D mesh, separating the 2D mesh into 2 different grids to represent the interior and exterior areas connected by a Storage Area/Two Dimensional (SA/2D) connection. (Figure 5.10.1.2.2) At this point in the modeling, that SA/2D connection is geo-referenced using the 12' wall elevation footprint. To make sure the with-project and without-project geometries are as identical as possible, the 12' wall SA/2D connection is being used in the without-project geometry and the station/elevation data is utilizing the underlying terrain data where the with-project condition will have a constant elevation of 12' (Figure 5.10.1.2.3). 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 (Figure 5.10.1.2.4). Breaklines have also been applied to other appropriate locations to represent raised features in the model domain. Peninsula outfall locations have been provided in a GIS shapefile format. However, HEC-RAS is unable to compute subsurface flow therefore the outfalls will not be utilized and the model will assume no pipe flow capacity. Culverts in the interior model area that convey overland flow have been included into the model and were estimated in size and placement using google earth imagery and the underlying terrain data used in the 2D model.

The exterior portion of the mesh includes the bounding bodies of water named 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 protected landscape. 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 landscape. The interior and exterior areas are connected with 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 is exactly 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 the different conditions.

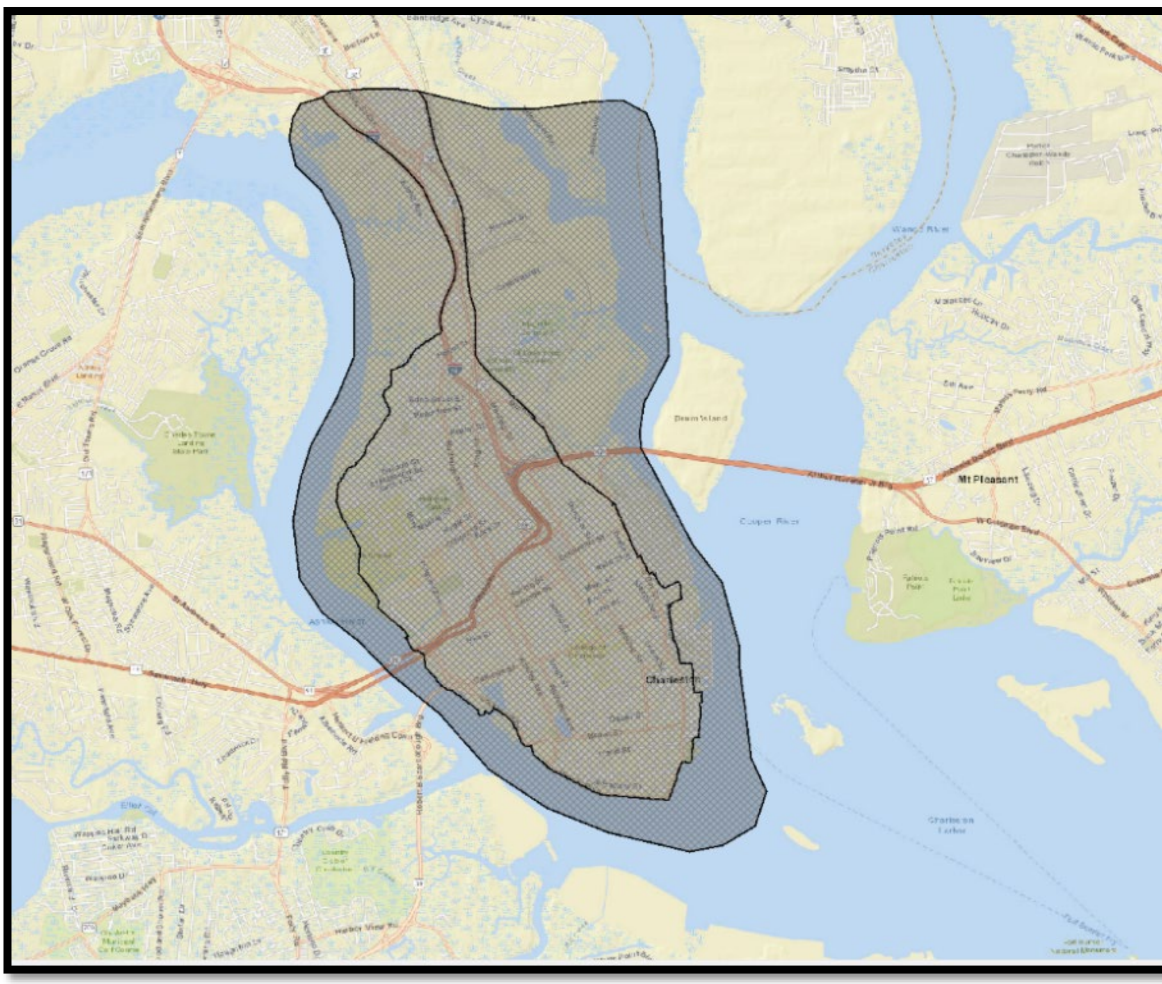


Figure 5.10.1.2.1. HEC-RAS 2D computational mesh

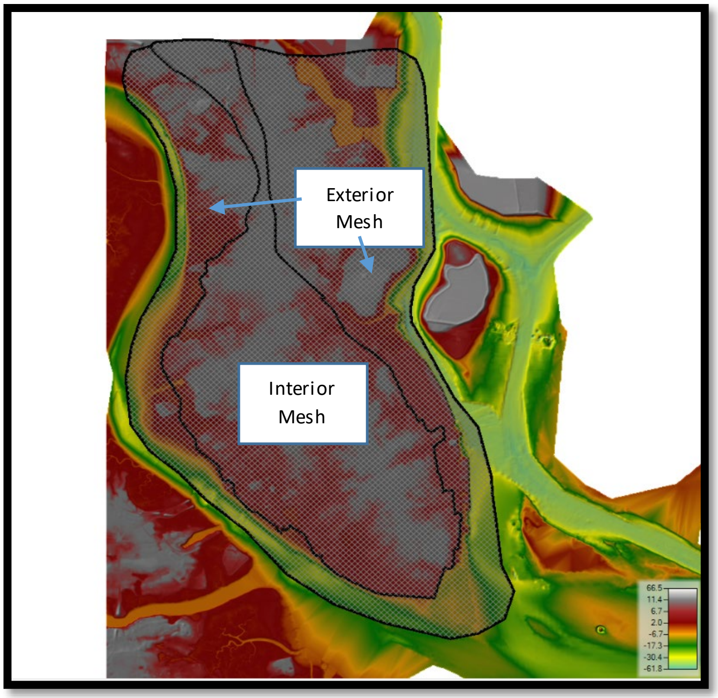


Figure 5.10.1.2.2. HEC-RAS 2D computational mesh and terrain (ft. NAVD88)

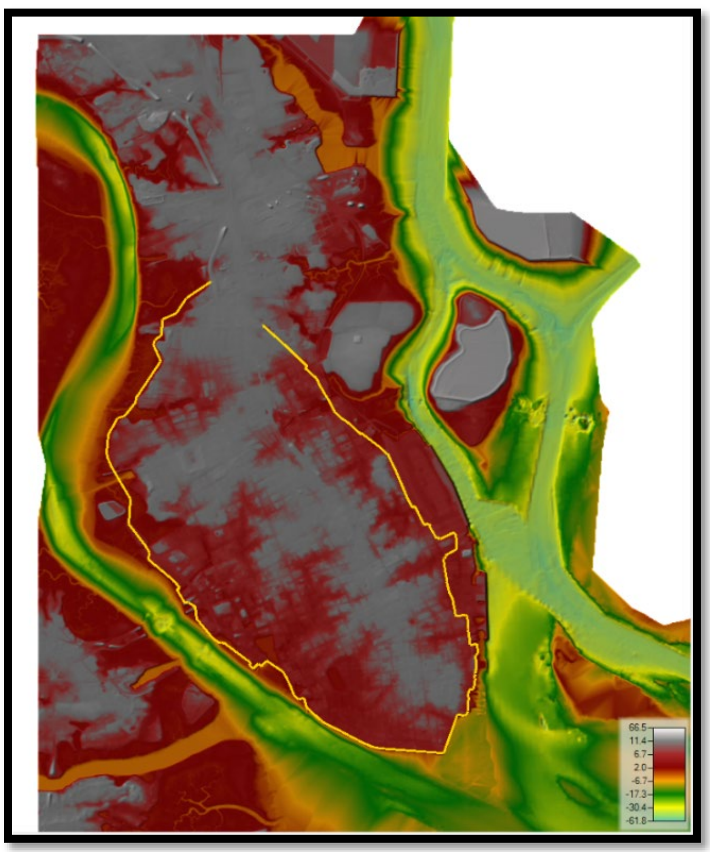


Figure 5.10.1.2.3. 12' Wall Alignment

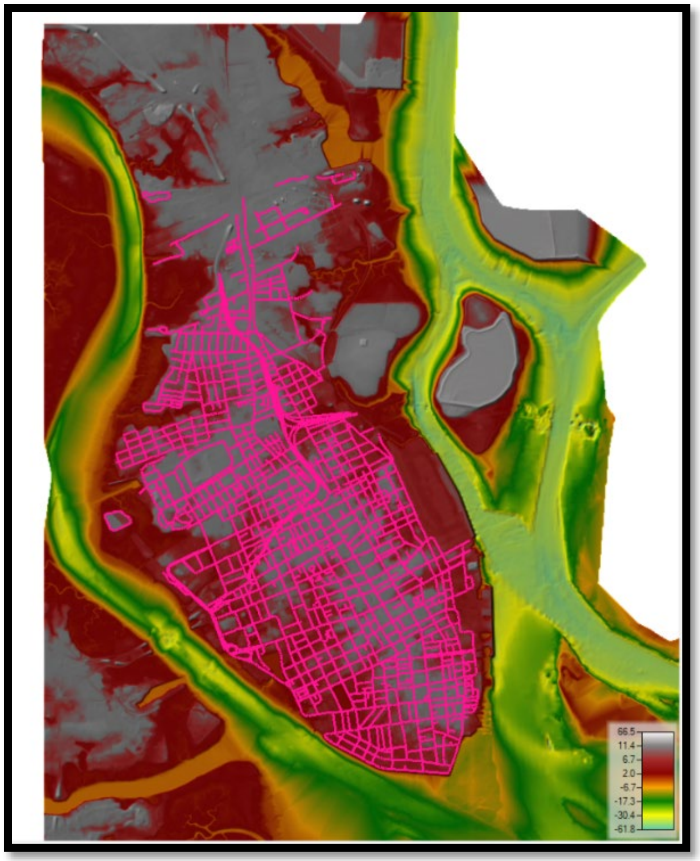


Figure 5.10.1.2.4. HEC-RAS Breaklines applied to 2D mesh

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 Figure 8.1.3.6). 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 5% and 20% AEP rainfalls were interpolated from the direct runoff data. The rainfall data is applied to the 2D mesh uniformly, where in reality the rainfall will vary spatially across the modeled area.

5.10.1.1 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, even without validation data the HEC-RAS model will provide enough information needed by the PDT to make an informed decision on the total pumping capacity and drainage structure feasibility at various locations on the peninsula due to rainfall flooding or flooding incurred by the wall overtopping which will be analyzed during the PED phase. The HEC-RAS model will serve its intended purpose to estimate the hydraulic response of the overall system for various pumping and drainage structures capabilities once these features are put into place for model testing during the PED phase.

b. Future without Project Conditions

The goal of the future without-project condition for the feasibility phase 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. The Ocean and Coastal Resource Management Office (OCRM) currently indicates that King tide is 6.6 feet (MLLW) which equates to 3.46 feet (NAVD88). Once you add the 1.13 feet in sea level rise, King Tide equals 4.59 feet (NAVD88). It is known that areas on the Charleston Peninsula at low lying elevations would flood at such a water surface elevation. Therefore, elevations for the exterior boundary condition will need to be levels that are low enough to allow for the rainfall to drain with little to no resistance to flow out of the proposed protected area. This will allow for a better comparison when comparing the rainfall flooding on the interior when running the rainfall suite on the future with-project conditions geometry. The 6 feet NAVD88 exterior water level for approximately a present day 4 percent annual exceedance probability Stillwater elevation surge and a future intermediate rate of SLR 33% AEP still water elevation surge. Four different tidal boundary conditions were run against the entire rainfall suite. These exterior water surface elevations were applied to the 2D mesh as a boundary condition line to the outer perimeter of the exterior mesh. The four water surface elevations are as follows in NAVD88: 1’, 2’, 4.59’ and 6’. A higher variance of storm surge levels can be run through the model during the optimization and further evaluated during PED phase to analyze different pump alternatives and drainage capacities.

The without-project simulation does not include a high storm surge event at this point. 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. These simulations will provide adequate with-project and without project result comparisons for the proper sizing of pumps and storm gates.

There are a numerous amount 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.

FWO inundations produced by the 10% AEP Rainfall

The figure 5.10.1.2.5 in the top right displays the peak water surface elevation produced by the 10% AEP rainfall for the FWO condition combined with a steady state 2’ tide. The figure 5.10.1.2.6 in the bottom left displays the peak depth produced by the 10% AEP rainfall for the FWO condition. Although difficult to tell at the scale in these figures, the model captures highly detailed water surface elevations and flood depths throughout the study area.

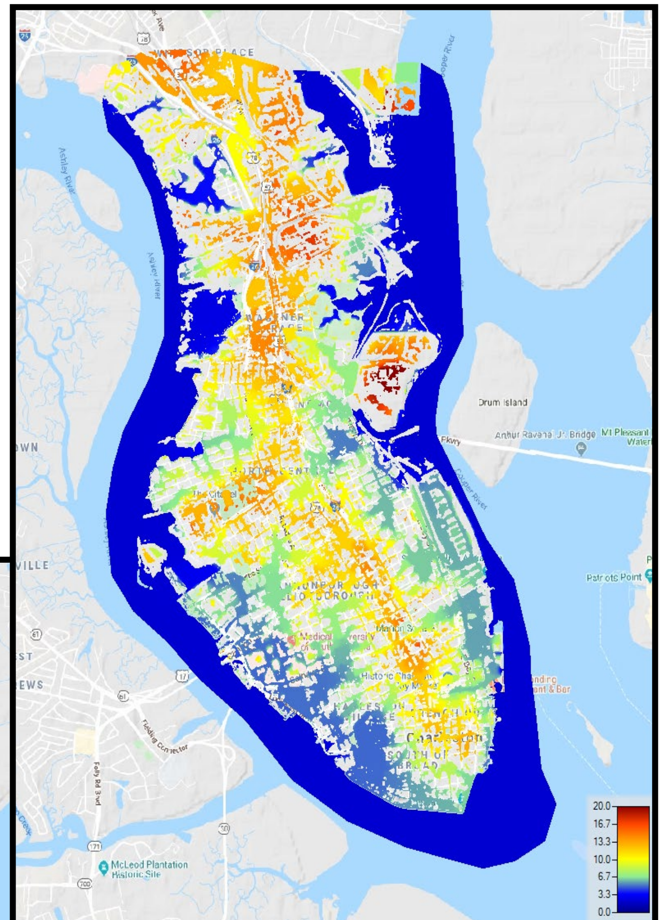


Figure5.10.1.2.5 10% AEP FWO Peak

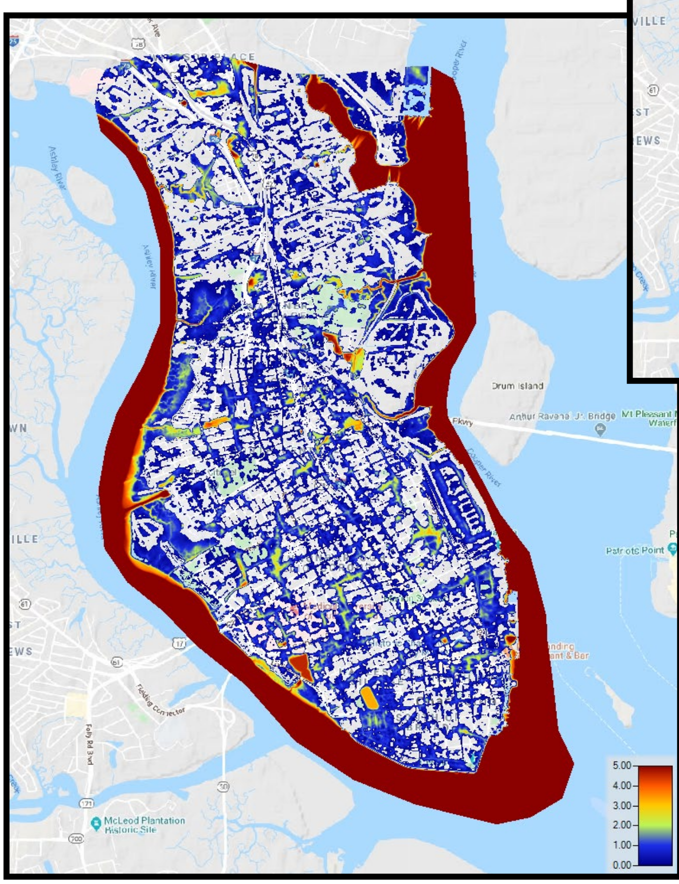


Figure 5.10.1.2.6 10% AEP FWO Peak

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 5.10.1.2.7. The Level 4.59' 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 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 during the PED phase.

Selected output locations were used to assess the increase in water levels for future with-project conditions versus future without-project conditions. Figure 5.10.1.2.7 displays the selected output locations for the RAS modeling. The locations were chosen to show various impacts around the peninsula.

d. Results at Selected Output Locations

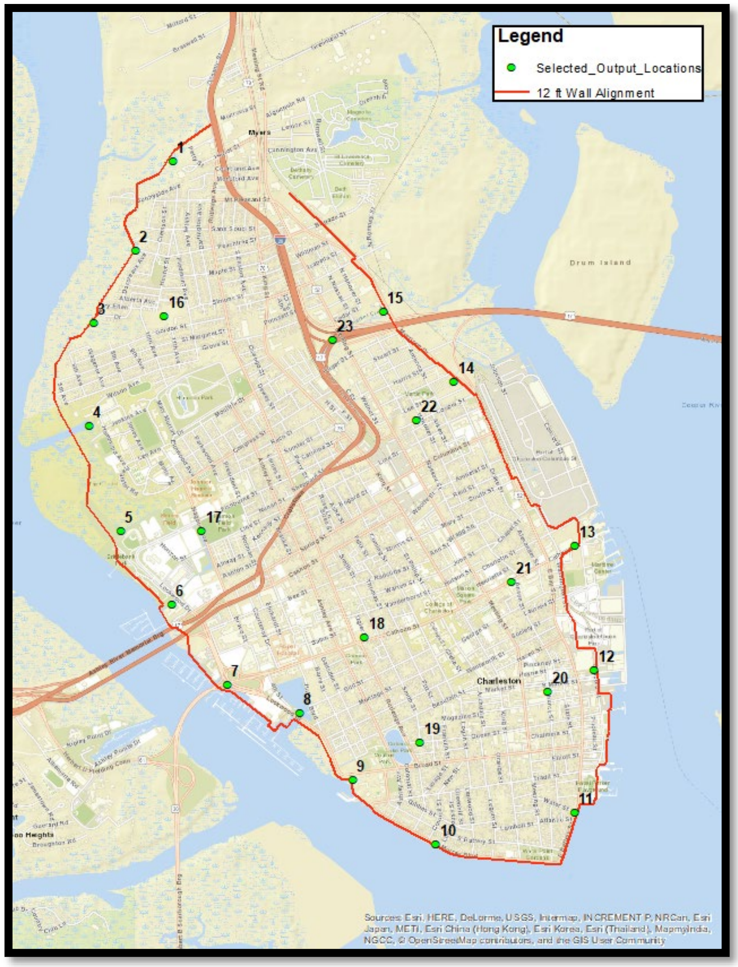


Figure 5.10.1.2.7. Selected Output Locations for RAS model results

Table 5.10.1.1 and Table 5.10.1.2 are examples of the comparison of Future-With Project to the Future Without Project condition for a 20% annual exceedance and a 10% annual exceedance. Future-With Project assuming a closed system at this point (meaning no flow in or out). The Future Without Project condition assumes an open system with tide impacts on the marshes. Looking at the 1 foot and 2 feet exterior water surface elevation (WSE) it can be seen that the With project condition rises above the Without Project condition for almost every location. This is not a condition where the gates would actually be closed, because this is not a storm surge condition. It is intended to show impacts of tidal marshes on storage when there is a storm surge. This is demonstrated by the exterior elevation of the 6' NAVD88 exterior WSE. From these scenarios it can be seen that during a storm surge event with gates closed there would be a reduction in interior water levels for much of the output locations. This reduction could be significant for much larger storm surge events. In other words, the project could greatly reduce the water levels in the interior for a storm surge event, regardless of pump capacity.

Table 5.10.1.1 20% Annual Exceedance Comparison

20 %	Future with-Project	FWO @ 1' exterior WSE (NAVD88)		FWO @ 2' exterior WSE (NAVD88)		FWO @ 4.59' exterior WSE (NAVD88)		FWO @ 6' exterior WSE (NAVD88)	
	Peak Water Surface Elevation (ft. NAVD88)	Peak Water Surface Elevation (ft. NAVD88)	Difference from with-project condition (ft)	Peak Water Surface Elevation (ft. NAVD88)	Difference from with-project condition (ft)	Peak Water Surface Elevation (ft. NAVD88)	Difference from with-project condition (ft)	Peak Water Surface Elevation (ft. NAVD88)	Difference from with-project condition (ft)
1	5.43	3.61	1.82	3.61	1.82	4.61	0.82	6.01	-0.58
2	5.26	2.59	2.67	2.59	2.67	4.60	0.66	6.01	-0.75
3	5.93	1.96	3.97	2.28	3.65	4.62	1.31	6.03	-0.10
4	3.19	1.03	2.16	2.02	1.17	4.61	-1.42	6.02	-2.83
5	3.19	2.68	0.51	2.86	0.33	4.62	-1.43	6.02	-2.83
6	5.40	2.17	3.23	2.36	3.04	4.61	0.79	6.02	-0.62
7	5.03	1.01	4.02	2.01	3.02	4.59	0.44	6.02	-0.99
8	5.03	4.40	0.63	4.61	0.42	4.94	0.09	6.04	-1.01
9	5.03	2.25	2.78	2.30	2.73	4.60	0.43	6.02	-0.99
10	5.03	5.12	-0.09	5.13	-0.10	5.17	-0.14	6.1	-1.07
11	6.19	6.19	0.00	6.20	-0.01	6.20	-0.01	6.37	-0.18
12	6.90	5.84	1.06	5.85	1.05	5.78	1.12	6.2	0.70
13	6.61	5.52	1.09	5.52	1.09	5.53	1.08	6.15	0.46
14	6.47	4.10	2.37	4.10	2.37	4.61	1.86	6.02	0.45
15	6.64	4.43	2.21	4.43	2.21	5.44	1.20	6.47	0.17
16	6.13	6.11	0.02	6.10	0.03	6.17	-0.04	6.33	-0.20
17	5.44	4.78	0.66	4.79	0.65	5.20	0.24	6.09	-0.65
18	5.68	5.67	0.01	5.67	0.01	5.68	0.00	6.12	-0.44
19	5.02	4.62	0.40	4.62	0.40	4.90	0.12	6.09	-1.07
20	7.32	7.07	0.25	7.07	0.25	7.07	0.25	7.19	0.13
21	6.67	6.62	0.05	6.64	0.03	6.69	-0.02	6.77	-0.10
22	6.47	5.77	0.70	5.77	0.70	5.77	0.70	6.29	0.18
23	6.64	6.28	0.36	6.28	0.36	6.30	0.34	6.55	0.09

Table 5.10.1.2 10% Annual Exceedance Comparison

10 %	Future with-Project	FWO @ 1' exterior WSE (NAVD88)		FWO @ 2' exterior WSE (NAVD88)		FWO @ 4.59' exterior WSE (NAVD88)		FWO @ 6' exterior WSE (NAVD88)	
	Peak Water Surface Elevation (ft. NAVD88)	Peak Water Surface Elevation (ft. NAVD88)	Difference from with-project condition (ft)	Peak Water Surface Elevation (ft. NAVD88)	Difference from with-project condition (ft)	Peak Water Surface Elevation (ft. NAVD88)	Difference from with-project condition (ft)	Peak Water Surface Elevation (ft. NAVD88)	Difference from with-project condition (ft)
1	5.90	3.67	2.23	3.67	2.23	4.62	1.28	6.01	-0.11
2	6.42	2.61	3.81	2.61	3.81	4.60	1.82	6.01	0.41
3	6.43	2.25	4.18	2.45	3.98	4.64	1.79	6.03	0.40
4	4.13	1.04	3.10	2.03	2.10	4.61	-0.48	6.02	-1.89
5	4.13	2.82	1.31	2.95	1.18	4.63	-0.50	6.03	-1.90
6	5.50	2.29	3.21	2.44	3.06	4.61	0.89	6.02	-0.52
7	5.32	1.01	4.31	2.01	3.31	4.59	0.73	6.02	-0.70
8	5.32	4.80	0.52	4.88	0.44	5.05	0.27	6.04	-0.72
9	5.32	2.32	3.00	2.37	2.95	4.60	0.72	6.02	-0.70
10	5.32	5.21	0.11	5.23	0.09	5.28	0.04	6.14	-0.82
11	6.35	6.34	0.01	6.35	0.00	6.35	0.00	6.51	-0.16
12	7.25	6.00	1.25	6.00	1.25	5.89	1.36	6.25	1.00
13	6.94	5.64	1.30	5.64	1.30	5.65	1.29	6.2	0.74
14	6.77	4.17	2.60	4.17	2.60	4.62	2.15	6.03	0.74
15	7.02	5.09	1.93	5.09	1.93	5.92	1.10	6.61	0.41
16	6.42	6.32	0.10	6.33	0.09	6.35	0.07	6.44	-0.02
17	5.56	0.17	5.39	5.18	0.38	5.38	0.18	6.12	-0.56
18	5.84	5.84	0.00	5.84	0.00	5.85	-0.01	6.21	-0.37
19	5.32	4.78	0.54	4.72	0.60	5.02	0.30	6.12	-0.80
20	7.62	7.36	0.26	7.36	0.26	7.36	0.26	7.39	0.23
21	6.94	6.79	0.15	6.80	0.14	6.86	0.08	6.89	0.05
22	6.77	5.94	0.83	5.94	0.83	5.94	0.83	6.38	0.39
23	7.03	6.46	0.57	6.46	0.57	6.47	0.56	6.7	0.33

Post TSP Evaluations

- Consider coincident or compound flooding that occurs with rainfall and tidal events.
- Assess the flooding that will occur interior due to rainfall with an open gate system for tide events that do not warrant gate closure (i.e. high tides).
- Assess the flooding that will occur interior due to rainfall with a closed gate system for storm surge events.
- Assess the flooding that will occur with overwash due to wave action over the wall (after final selection of wall elevation) in combination with rainfall.
- Provide water elevations to Economics for computation of residual damages (versus what would have occurred without federal project)

- Evaluate options to resolve residual damages ((i.e. storage, nonstructural, collection system and pumps…)
- Select option and include as part of plan

More information can be found in sub appendix 3 HYDRAULICS, HYDROLOGY & COASTAL SUB-APPENDIX

5.10.2 PUMP LOCATIONS

Because the new wall will restrict the normal flow of rainwater and storm water runoff, especially when all of the gates have been secured for a storm event, pump stations may be required to mitigate this rainwater accumulation. There are two types of pumps that are being considered, permanent pump stations and temporary pumps. More detail about each type and the potential locations for each is shown below.

5.10.2.1 PERMANENT PUMPS

Permanent pump stations would consist of a wet well installed in a low lying area where water will likely collect. All of the supporting infrastructure such as power and/or fuel source would be located on higher ground to avoid flooding. The outlet of the pump would be piped either over the wall or through it with a check valve to prevent back flow. There are 5 locations that have been suggested to receive permanent pump stations. As shown in the below Figure 5.10.2.1, the target areas are low lying marsh areas, as rain water from the surrounding area will naturally flow to these areas. Four of the locations are on the Ashley River side of the study area, with the fifth being on the Cooper River side. The intent would be to construct the pump stations with as minimal an impact to the marsh as possible. Therefore either the pumps stations would be built as part of the wall where it passes through the marsh, or will be built on dry land with only a wet well or intake pipe actually located in the marsh. Pumps will be electric powered with a built in diesel powered generator, which will maximize the flexibility and ensure the pumps can run even if the storm has disrupted the electricity supply.

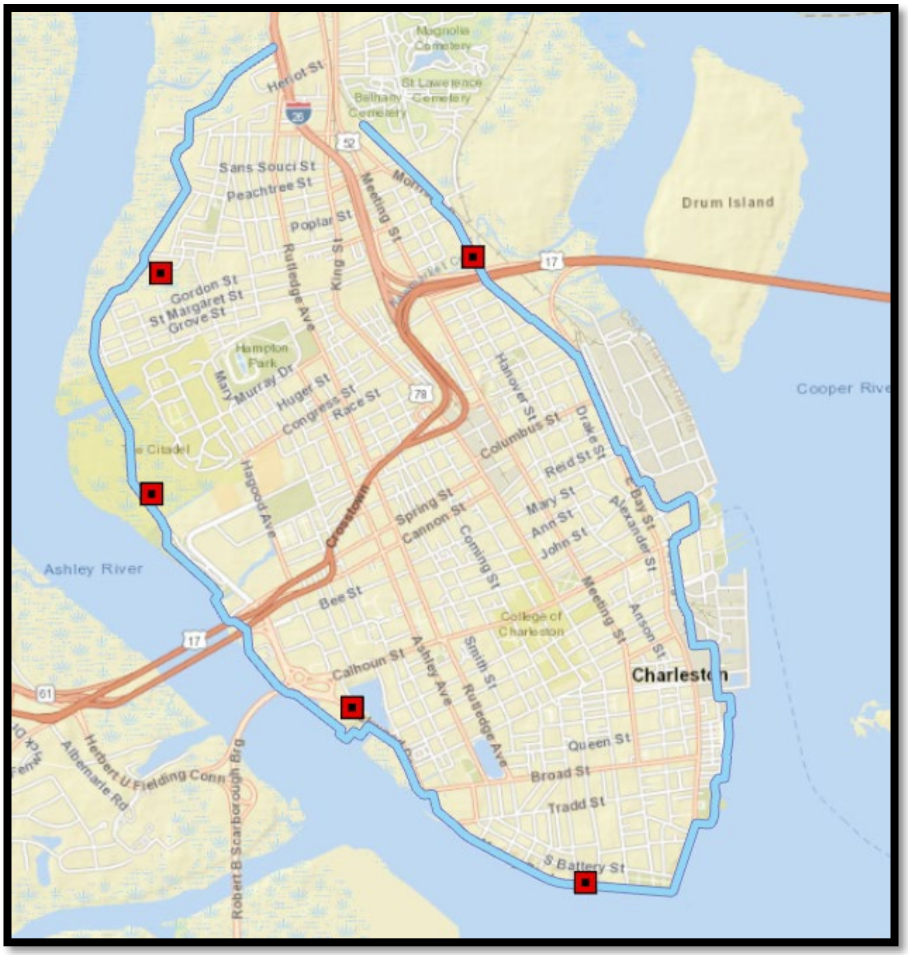


Figure 5.10.2.1 Preliminary permanent pump locations

5.10.2.2 PORTABLE PUMPS

To maximize flexibility, five portable pumps are also considered to be utilized. The pumps would be trailer mounted, diesel powered units capable of being moved and deployed where needed to handle flooding due to the normal storm water runoffs being blocked by the wall. To aid in deployment of the pumps, several pre-determined locations would have a concrete pad complete with anchors, also with a pre-piped wet well and outlet through the new wall. This will allow the portable pumps to be rapidly and safely deployed where needed and also be moved around during the storm if necessary. Having the pre-piped well and outlet minimizes issues that portable pumps have, as the pump itself cannot be submerged and it can be difficult to get intake and outlet pipes/hoses to ideal locations due to flooding. Plus having the concrete pad and anchors mitigates the risk of high winds affecting the pump. There are ten locations that are suggested to have the premade pad and piping installed for the temporary pumps. The target areas are mostly on the Cooper River side as well as along the battery wall at the end of the peninsula. Specific locations have not been identified.

5.10.3. 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.

5.10.3.1 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, sea weed, 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.

5.10.3.2 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 FORCES ON VERTICAL WALL

On the final selected wall, to determine the wave forces on the vertical wall due to wave action an evaluation will be done using Eurotop: Wave Overtopping of Sea Defenses and Related Structures..

5.12. WAVE OVERTOPPING

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 identified as a Dynamic Still Water Level (DSWL). 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 graphic in Figure 5.12.1 depicts typical surge profile as it approaches land with a vertical wall.

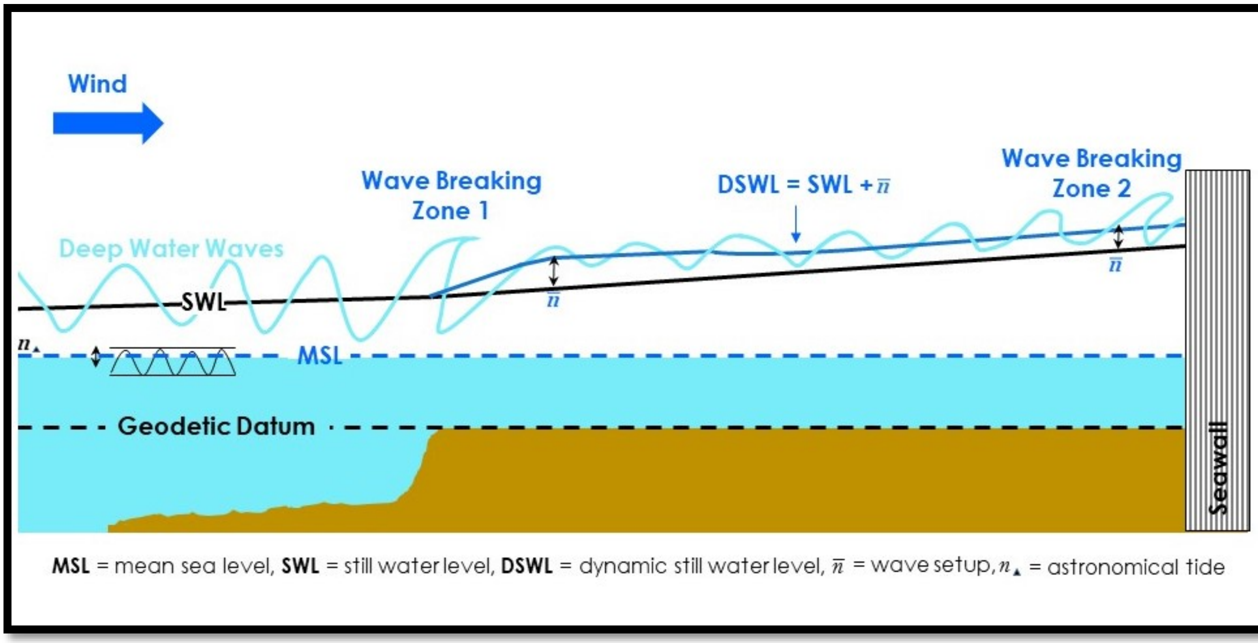


Figure 5.12.1 Dynamic Still Water Level

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 defense but not failure of the structure so long as the structure is designed for overtopping without structural failure. This analysis will be performed on the final elevation and footprint of the proposed structure. The following sections discuss overtopping by still water elevation, dynamic still water level and overwash due to wave action.
.

5.12.1 OVERTOPPING FLOOD WALL ANALYSIS

Using the Stillwater elevation annual exceedance probability , based on the FEMA analysis and adding sea level rise of 1.13 feet resulted in the following probability curve at a save point near the NOAA gage, shown in figure 5.12.1.1. The 1 percent Stillwater Annual Exceedance Probability (AEP) is estimated to be 11.4 feet NAVD88. The 10 percent AEP Stillwater elevation is 6.4 ft NAVD88.

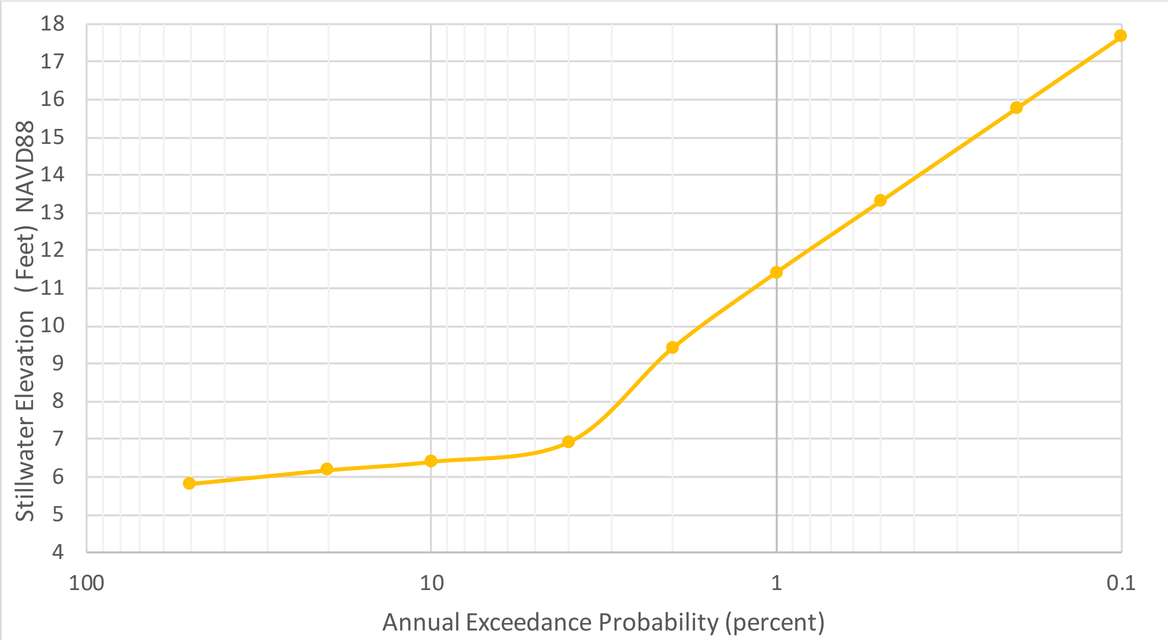


Figure 5.12.1.1 Still Water Elevation Annual Exceedance probability with addition of 1.13 feet of sea level rise.

Stillwater 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, but the still water elevation should not be considered the total probability of risk so as to not mislead the public. The Stillwater 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 5.12.1.2, obtained from FEMA contractor.

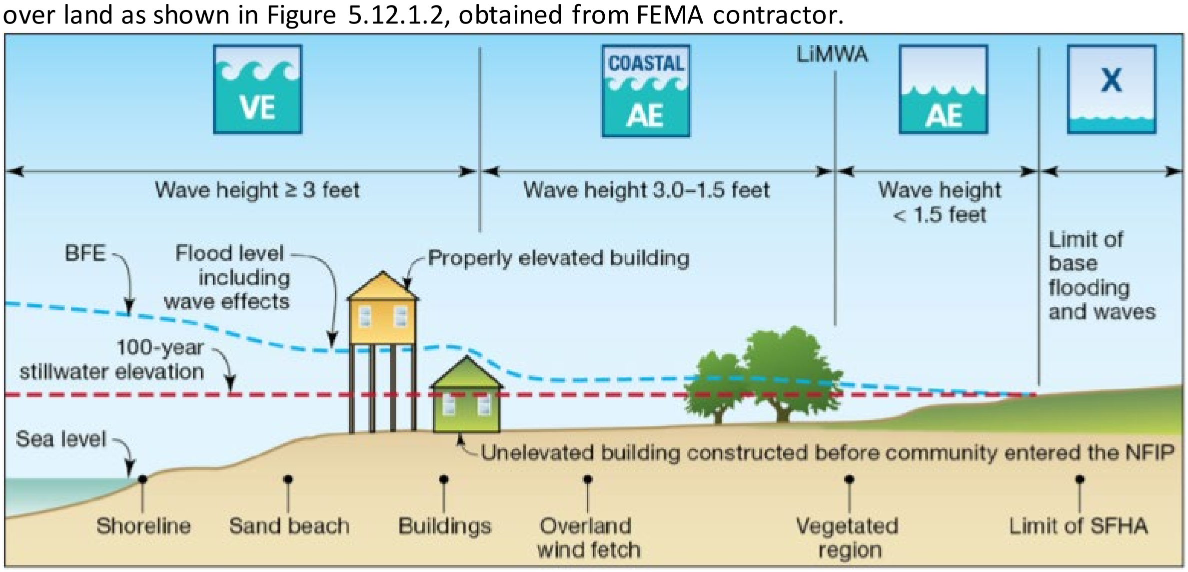


Figure 5.12.1.2 Demonstration of Stillwater elevation, BFE and various Special Flood Hazard Areas. (Source FEMA)

Considering the risk at high tide (a dynamic still water level) the AEP graph changes to Figure 5.12.1.3. This would result in 1 percent AEP of 13.7 feet NAVD88 and a 10 percent AEP of 8.7 ft NAVD88.

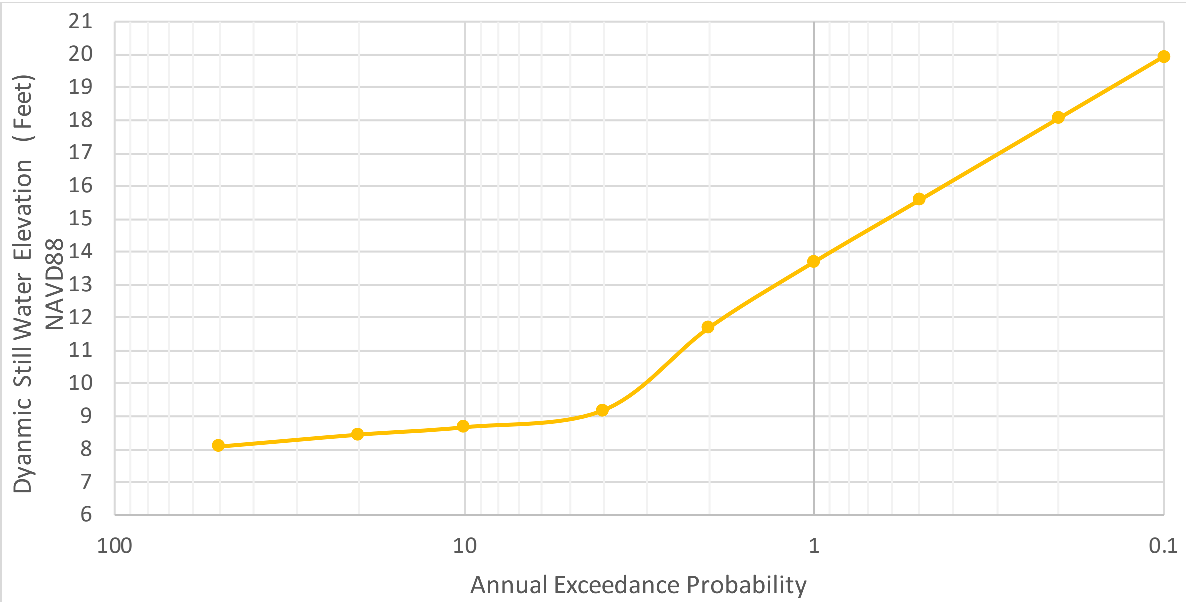


Figure 5.12.1.3 Dynamic Still Water Elevation Annual Exceedance probability with addition of 2.27 feet of high tide

The elevation of the wall has not been finalized. Further evaluation of the optimum elevation will be evaluated and submitted as the final recommendation in the final report. Assessments of impacts are based on a wall at elevation 12’ NAVD88. Based on the Stillwater Elevation Annual Exceedance probability with addition of 2.27 feet of high tide, the wall at 12 feet NAVD88 would equate to approximately 1.8 percent AEP.

5.12.2. WAVE OVERWASH/OVERTOPPING

On the final selected wall, to determine the potential risk of overwash/overtopping due to wave action an evaluation will be done to compute wave run-up and wave overtopping. Wave Run-up is the dynamic water component that is added to the static dynamic still water level to define Total Water Level (TWL). The analysis will be done using Eurotop: Wave Overtopping of Sea Defenses and Related Structures.

5.13. QUANTITY ESTIMATES

The following sections provide the quantities for each structural measure that were determined for the 12’ NAVD88 elevation along the proposed alignment. Alignment is to be optimized after TSP milestone and quantities will change before final recommended plan.

5.13.1 COMBO WALL QUANTITY CALCULATIONS

Pile Spacing: 9 ft
Pile Embedment: 1 ft
Battered Pile Slope: 3V: 1H
Cap Thickness: 6ft
Cap Width: 12 ft
Quantities used for construction cost estimate are detailed in Table 5.13.1.

Table 5.13.1 Combo Wall Quantities

12 FT ELEV (NAVD88) WALL						Length (ft)	Cooper Marl Elev (ft)	Concrete Qty (CY)	Vertical Pile Qty (EA)	Vertical Pile Length (LF)	Total Length of Vertical Piles (LF)	Battered Pile Qty (EA)	Battered Pile Length (LF)	Total Length of Battered Piles (LF)
REACH 1 - WAGENER TERRACE														
2	+	50	to	23	+ 50	2100	-55	5600	234	62	14529	234	65	15315
28	+	0	to	56	+ 70	2870	-55	7653	320	62	19833	320	65	20906
57	+	40	to	87	+ 40	3000	-55	8000	334	62	20729	334	65	21850
88	+	10	to	95	+ 0	690	-55	1840	78	62	4815	78	65	5076
						8660		**23093**	**966**		**59906**	**966**		**63146**
REACH 2 - MARINA (W/ COMBO WALL ALONG LOCKWOOD BLVD)														
95	+	0	to	107	+ 50	1250	-55	3333	140	62	8673	140	65	9142
139	+	20	to	145	+ 20	600	-55	1600	68	62	4195	68	65	4422
147	+	40	to	148	+ 90	150	-55	400	18	62	1095	18	65	1155
151	+	20	to	218	+ 70	6750	-65	18000	751	72	54072	751	76	56997
						8750		**23333**	**976**		**68036**	**976**		**71716**
REACH 2 - MARINA (W/ T-WALL ALONG LOCKWOOD BLVD) - NOT CALCULATED														
0	+	0	to	0	+ 0	0	-55	0	1	62	62	1	65	65
0	+	0	to	0	+ 0	0	-55	0	1	62	62	1	65	65
0	+	0	to	0	+ 0	0	-65	0	1	72	72	1	76	76
0	+	0	to	0	+ 0	0	-75	0	1	82	82	1	86	86
						0		**0**	**4**		**278**	**4**		**293**
REACH 3 - BATTERY - NO COMBO WALL PROPOSED FOR THIS REACH														
REACH 4 - PORT														
296	+	40	to	300	+ 20	380	-75	1013	43	82	3544	43	86	3736
309	+	20	to	314	+ 20	500	-60	1333	57	67	3789	57	71	3994
						880		**2347**	**100**		**7333**	**100**		**7730**
REACH 5 - NEWMARKET - NO COMBO WALL PROPOSED FOR THIS REACH														

5.13.2 T WALL QUANTITY CALCULATIONS

<u>T-WALL SLAB DIMENSIONS</u>			
Width	10	FT	
Thickness	3	FT	
Soil Overfill	2	FT	
Min. Depth	5	FT	(below grade)
<u>T-WALL STEM DIMENSIONS</u>			
Thicknesses:			
Base	2	FT	
Top	1	FT	
<u>H-PILE DIMENSIONS</u>			
12 x 84 H-Piles			
Embed Depth	1	FT	(embedment depth in Cooper Marl)
Embed Depth	5	FT	(embedment depth in Slab)
Θ	15	degrees	
Spacing	5	FT	
Spacing	8	FT	
<u>EXCAVATION CALCULATIONS</u>			
Excavation Width:	20	FT	(assume 5 ft on each side)
Assume a 1V:2H side slope			

Figure 5.13.2.1 depicts the cross section of the T-wall and Table 5.14.2.1 details the quantities used in the construction cost estimate.

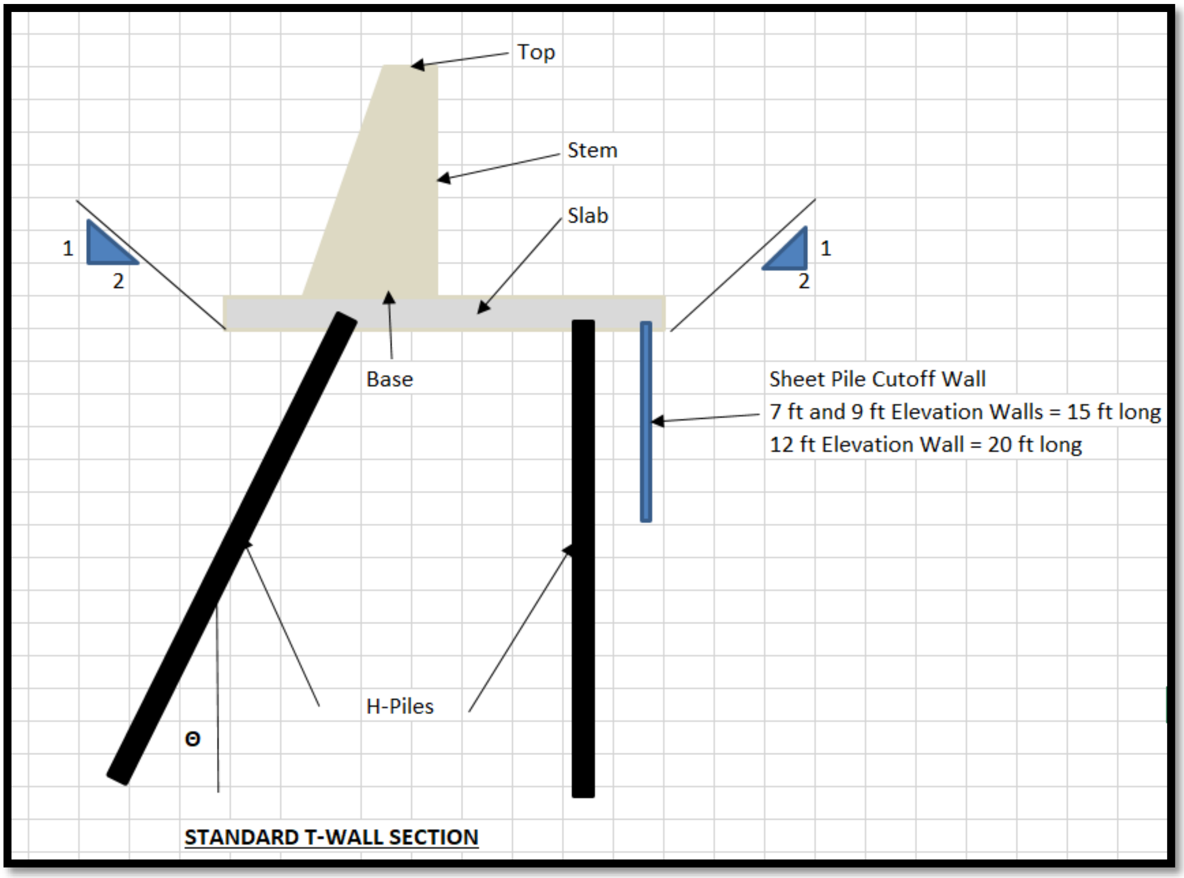


Figure 5.13.2.1 T wall graphic

Table 5.13.2.1 T wall Quantities

12 FT ELEVATION (NAVD88) WALL

Stations	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)	Excavation Volume (CY)	Concrete Volume - Slab (CY)	Concrete Volume - Stem (CY)	Total H-Pile Quantity (LF)	Total H-Pile Quantity (EA)
REACH 1 - MA FOR WAGENER TERRACE												
0+00 to 1+50	150	8	7	1	4	-55	8	894	167	67	4432	62
1+50 to 2+50	100	4	6	-2	1	-55	11	511	111	61	2857	42
23+50 to 25+50	200	4	5	-1	2	-55	10	852	222	111	5673	82
25+50 to 28+00	250	8	5	3	6	-55	6	1,065	278	83	7526	102
56+70 to 57+40	70	4	7	-3	0	-55	12	417	78	47	2006	30
87+40 to 88+10	70	8	11	-3	0	-55	12	656	78	47	2006	30
							TOTAL:	**4,396**	**933**	**416**	**24,500**	**348**
REACH 2 - MA FOR MARINA												
107+50 to 114+50	700	5	6	-1	2	-55	10	3,578	778	389	19508	282
114+50 to 118+50	400	7	5	2	5	-55	7	1,704	444	156	11767	162
118+50 to 129+50	1100	5	4	1	4	-55	8	3,748	1222	489	31596	442
129+50 to 139+20	970	5	5	0	3	-55	9	4,131	1078	485	27429	390
145+20 to 145+70	50	12	7	5	8	-55	4	298	56	11	1674	22
146+70 to 147+40	70	13	8	5	8	-55	4	477	78	16	2283	30
148+90 to 149+20	30	8	9	-1	2	-55	10	230	33	17	968	14
149+80 to 150+40	60	8	5	3	6	-55	6	256	67	20	1919	26
150+70 to 151+21	50	8	7	1	4	-55	8	298	56	22	1573	22
							TOTAL:	**14,720**	**3,811**	**1,604**	**98,716**	**1,390**
REACH 4 - MA FOR PORT												
285+45 to 289+45	400	5	4	1	4	-65	8	1,363	444	178	13448	162
289+45 to 296+40	695	6	4	2	5	-65	7	2,072	772	270	23566	280
314+20 to 316+45	225	6	5	1	4	-65	8	958	250	100	7637	92
317+45 to 336+20	1875	6	5	1	4	-65	8	7,986	2083	833	62426	752
336+20 to 364+20	2800	5	4	1	4	-65	8	9,541	3111	1244	93140	1122
364+20 to 389+20	2500	7	6	1	4	-65	8	12,778	2778	1111	83179	1002
389+20 to 396+20	700	5	5	0	3	-65	9	2,981	778	350	23084	282
396+20 to 414+00	1780	7	5	2	5	-65	7	7,581	1978	692	60094	714
							TOTAL:	**45,261**	**12,194**	**4,779**	**366,575**	**4,406**
REACH 5 - MA FOR NEWMARKET												
414+00 to 432+00	1800	5	5	0	3	-65	9	7,667	2000	900	59103	722
432+00 to 464+00	3200	6	5	1	4	-65	8	13,630	3556	1422	106422	1282
464+00 to 478+00	1475	11	6	5	8	-65	4	7,539	1639	328	51874	592
							TOTAL:	**28,835**	**7,194**	**2,650**	**217,399**	**2,596**

5.14. 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. Thus the contingency reflects the uncertainty associated with the data available. 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 an Abbreviated Risk Analysis (ARA) 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 ARA performed on this project, refer to the Cost Engineering Sub-Appendix.

5.15. 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.

Generally speaking, one factor of uncertainty is the confidence of the ADCIRC/STWAVE still water levels. Another factor is the projection of Sea Level Rise. These are based on guidance and science, but there are always inherent risks in using them for a protection design of future unknown conditions.

For this study, there are still factors to be evaluated such as wave overtopping, and assumptions that were made that will need more data and analysis to reduce the risk, but there will always be risk. These include subsurface information.

Risk associated with the cost is taken into consideration when the Abbreviated Risk Analysis (ARA) and Cost and Schedule Risk Analysis (CSRA) were performed on the project. Refer to the Cost Engineering Sub-Appendix for more detailed discussion on risk considered in the project cost.

5.16. 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 require 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 Work Days 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

5.17. 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 will be considered during optimization of this study.

5.17.1 INCREASING BARRIER HEIGHT

Since the I-Wall does not have any battered piles or major lateral resisting elements, the I-Wall will 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.

In addition, the T-Wall and Combo Wall have battered piles which are currently assumed to be driven to the Cooper Marl stratum providing more lateral resistance. This will allow for easier retrofitting of the barrier to provide an increased level of protection without requiring structural or foundation upgrades.

There are some topographical constraints that would require more than just raising the wall. The low battery wall raising being done the city puts a constraint on going any higher than elevation 12 due to the foundation design. Additionally, the roads into and out of the city on the Ashley River have a maximum tie-into the abutment at elevation 12 NAVD88, which would result in a requirement for a gate across US17. The interstate I26 leading inland is a primary evacuation route and there are limited opportunities to connect to the abutment at any elevation higher than12NAVD88, thus requiring a longer wall inland parallel to the interstate, gates across the interstate or raising the road.

5.17.2 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. These measures will consist of cathodic protection, use of galvanized or epoxy coated rebar, or use of fiberglass rebar. In addition, where material strengths are sufficient, vinyl sheet piles should be considered.

5.18. 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 alignment. Along with determining stratigraphy, 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). 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.

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.

6.1.9 BOUSSINESQ WAVE MODEL FOR WAVE RUN-UP

Rough estimates of wave overtopping will be done in the Feasibility study, however, more accurate Boussinesq wave modeling should be done to determine the wave run-up along the final barrier wall.

6.1.10 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.

6.2 OPERATION and MAINTENANCE MANUAL

Once the project has been constructed and turned over, 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.

CHAPTER 7 REFERENCES

(not referenced in Sub Appendices)

EM 1110-2-1617 Coastal Groins and Nearshore Breakwaters

EM 1110-2-1615 HYDRAULIC DESIGN OF SMALL BOAT HARBORS

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1. STRUCTURAL SUB-APPENDIX
2. GEOTECHNICAL SUB-APPENDIX
3. HYDRAULICS, HYDROLOGY & COASTAL SUB-APPENDIX
4. COST ENGINEERING SUB-APPENDIX
5. COASTAL MODELING SUB-APPENDIX
6. INTERIOR FLOODING -HEC-RAS 2D MODLING SUB-APPENDIX
7. SELECTED PLAN DRAWINGS SUB-APPENDIX