US Army Corps of Engineers®

Charleston District

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

Charleston, South Carolina

HYDRAULICS, HYDROLOGY & COASTAL SUB-APPENDIX

April 2020

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CHAPTER 1 – INTRODUCTION

1.1. DESCRIPTION OF PROJECT AREA AND VICINITY

Centrally located along the coast of South Carolina, the Charleston Peninsula project area is approximately 8 square miles, located between the Ashley and Cooper Rivers (Figure 1.1.1). Charleston Harbor is formed by the confluence of the Cooper, Ashley and Wando Rivers before discharging into the Atlantic Ocean. It includes the tidal estuary of the lower 12 miles of the Cooper River and the four miles of open bay between the confluence of the Ashley and Cooper Rivers and the Atlantic Ocean. The Cooper River contributes most of the freshwater inflow to the system and is the largest of the estuaries, extending about 57 miles from the harbor entrance to the Jefferies Hydroelectric Station at Lake Moultrie dam in Pinopolis, SC. The Cooper River flows are controlled under a contractual agreement with USACE to reduce shoaling in Charleston Harbor federal navigation channel. They are limited to a 4500 cfs average by week.

The Charleston Harbor is sheltered by barrier islands at the entrance.

Figure 1.1.1 Charleston Peninsula Study Boundary

The first European settlers arrived in Charleston around 1670. Since that time, the peninsula city has undergone dramatic shoreline changes, predominantly by landfilling of the intertidal zone. Early maps show that over one-third of the peninsula has been "reclaimed." Much of the landfilling occurred on the southern tip of Charleston, behind a seawall and promenade, known as the Battery and along the western shoreline. Figure 1.1.2 shows the Halsey Map of 1844 which depicts the original shoreline of the Charleston Peninsula.

Figure 1.1.2 Halsey Map of 1844

1.2. NOAA COOPER RIVER ENTRANCE TIDAL GAGE RECORD

The Cooper River Entrance Tidal Gage is Station 8665530 and is locally referred to as the Charleston Harbor or Custom's House gage. It was established September 13, 1899. It is located downtown on the peninsula in the vicinity of U.S. Custom House, along East Bay Street, and along Broad Street. The tide gage and staff are on the south end of the dock. Shown in Figure 1.2.1.

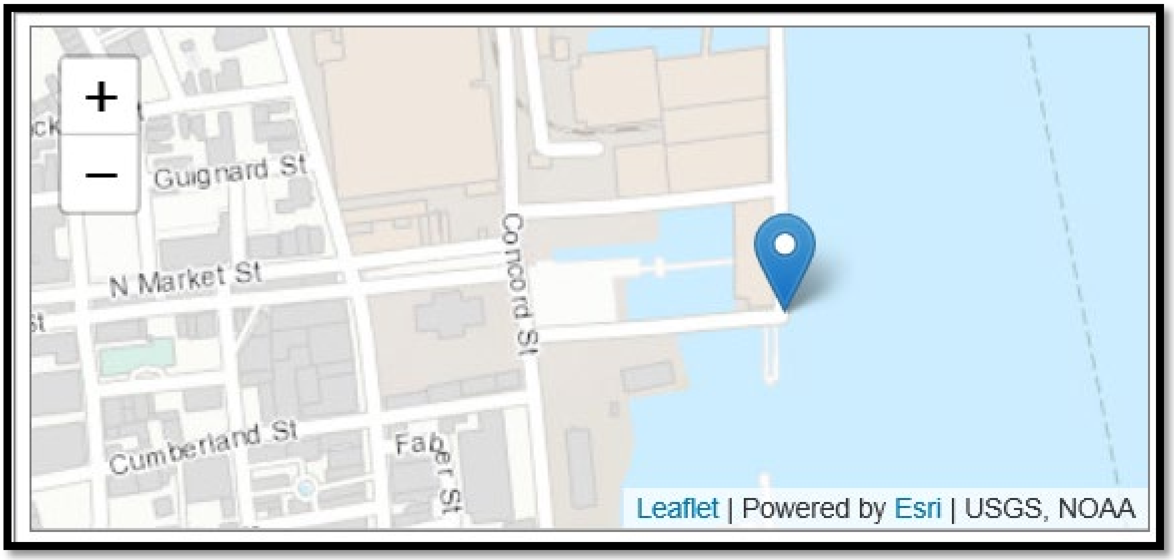


Figure 1.2.1 Location of NOAA Gage 8665530

Datum information provided by NOAA on their Tides and Currents website indicate a tide range of 5.76 feet (https://tidesandcurrents.noaa.gov/datums.html?id=8665530). Shown in Figure 1.2.2 and Table 1.2.1.

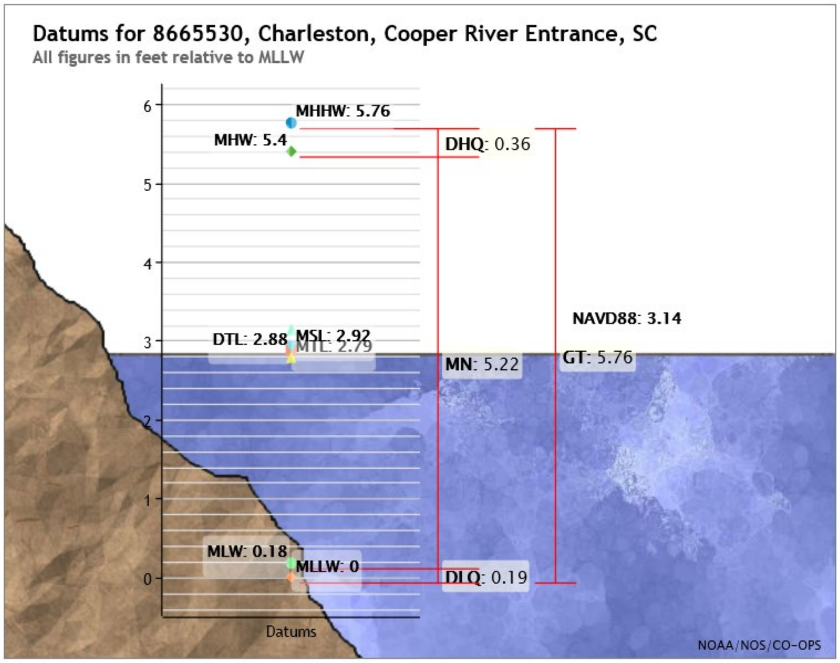


Figure 1.2.2 Tide Range Station 8665530

Table 1.2.1 Elevations on Mean Lower Low Water

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

Tidal Datum information provided from the NOAA website:
https://tidesandcurrents.noaa.gov/datums.html?id=8665530

1.3. CLIMATE

Charleston SC has hot humid summers and fairly mild winters. Average Annual high temperatures is approximately 75 degrees F and average annual low temperatures are approximately 53 degree F. Average annual precipitation is 44.29 inches with an average of 102 days of precipitation per year. Shown in Figure 1.3.1 and Table 1.3.1.

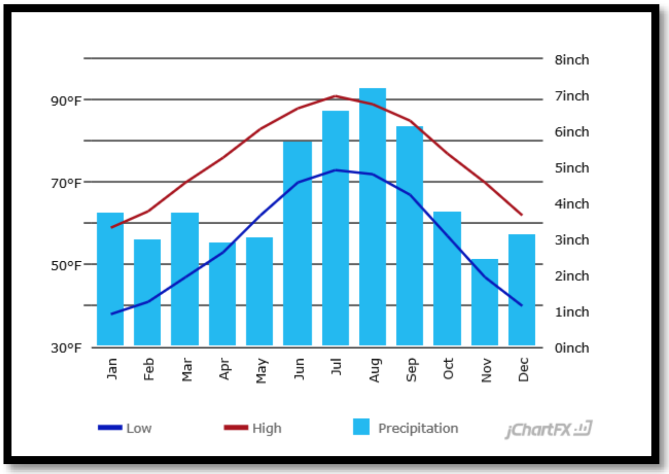


Figure 1.3.1 Charleston Temperature and Precipitation

Table 1.3.1 Charleston Temperature and Precipitation

Climate Charleston AFB - South Carolina

°C | °F

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

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

1.4 HORIZONTAL AND VERTICAL DATUMS

Horizontal datum for this study is tied to the State Plan Coordinate System using North American Datum of 1983(NAD83, South Carolina 2900). Distances are I feet by horizontal measurement. The vertical datum for this study is tied to the North American Vertical Datum of 1988 (NAVD88), a requirement of ER 1110-2-8160. Elevations are in feet.

1.5 WINDS

The Post45 Harbor Deepening study documented the following information.

1.5.1 Winds in Charleston Harbor

Winds can be described by their speed, direction, and duration. The National Oceanic and Atmospheric Administration (NOAA) operates a weather station in Charleston Harbor which collect 6-minute wind data. This station records wind speed and direction at the shore. A wind rose was generated using the hourly averaged data recorded between January 2010 and December 2011 to visualize the distribution of winds which pass over Charleston Harbor (See Figure 1.5.1).

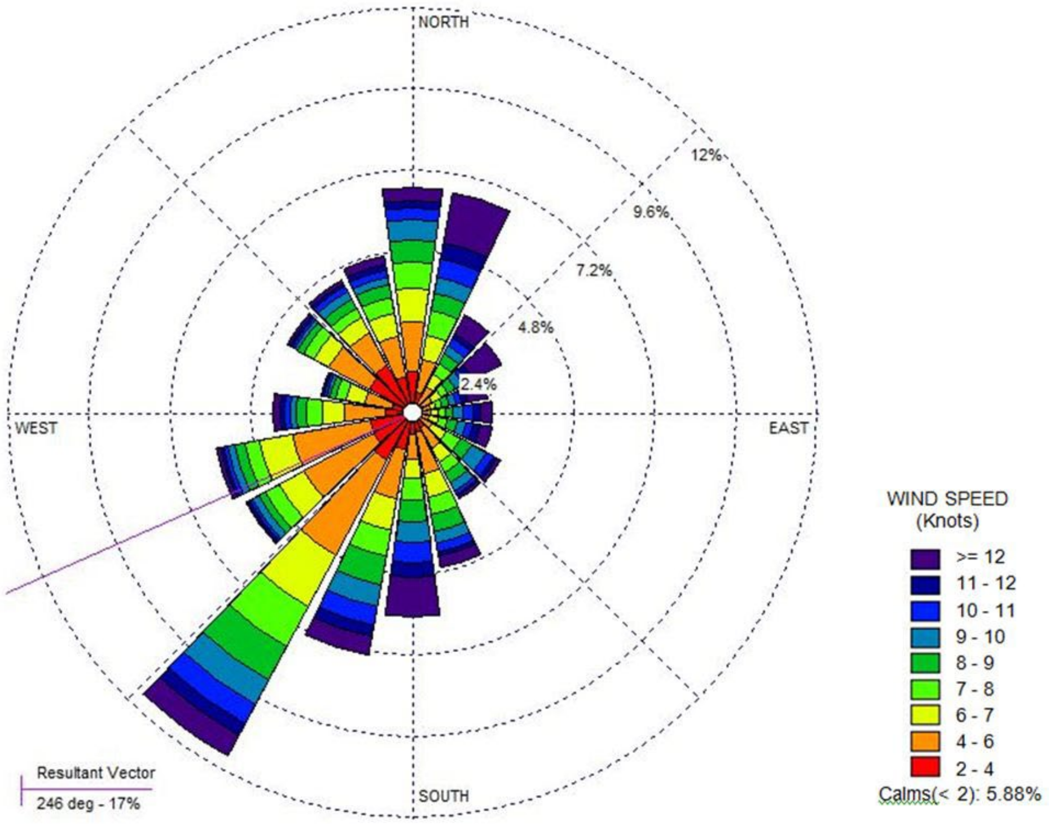


Figure 1.5.1. Wind Rose for Charleston Harbor Depicting Wind Direction and Speed Frequency

The distribution of wind speeds varies by direction (Refer to Figure 1.5.1. This figure is known as a wind rose). The total winds over Charleston Harbor, regardless of angle of approach, have the distribution by wind speed class shown in Figure 1.5.2. Three petals of the wind rose from Figure 1.5.1 are shown as frequency distributions in Figure 1.5.3. The petals selected reflect the three key directions: the largest number of winds, the highest speed winds and those with longest fetch (distance to travel). The largest number of winds in Charleston Harbor come from the southwest, while the most high-speed winds (fastest 10% of winds) come from the north-northeast direction (Wando River). Winds entering the harbor from open ocean (south-east) have the potential to travel the furthest distance before reaching a shoreline.

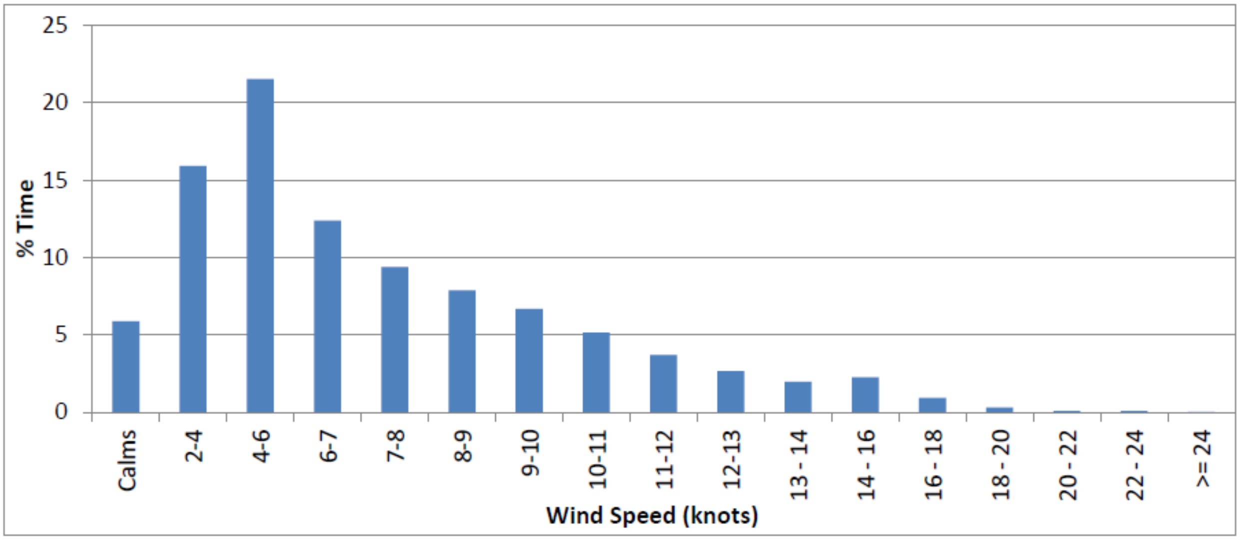


Figure 1.5.2 Wind Speed Frequency Distribution in Charleston Harbor from all directions

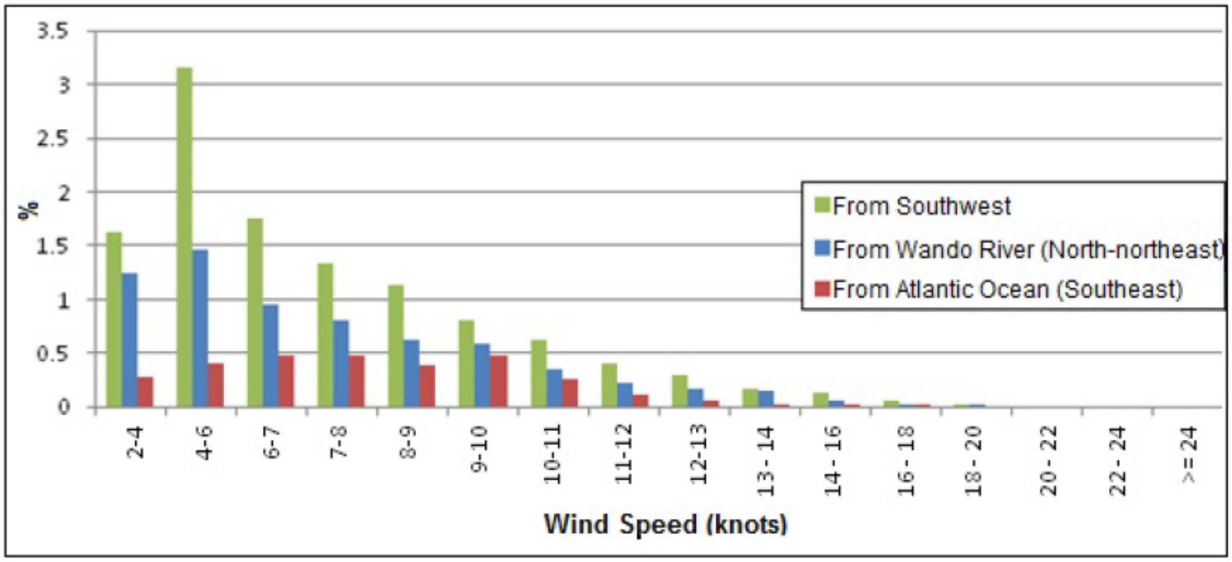


Figure 1.5.3 Wind Speed Frequency Distribution in Charleston Harbor comparing three key directions

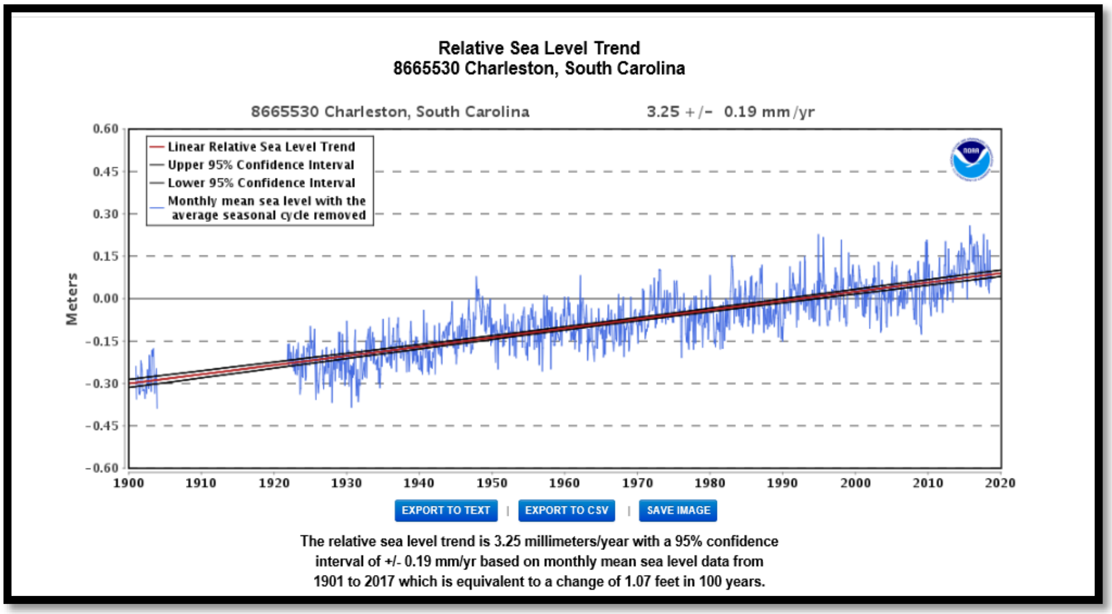
1.6 ASTRONOMICAL TIDES & WATER LEVELS

1.6.1 Astronomical Tides

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

1.6.2. Water Levels

The Charleston Harbor tide gauge was established in 1899. In that nearly 100-year time span, local sea level has risen 1.07 ft (Figure 1.6.2.1). One way to track local impacts from sea level 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).



The relative sea level trend is 3.25 millimeters/year with a 95% confidence interval of +/- 0.19 mm/yr based on monthly mean sea level data from 1901 to 2017 which is equivalent to a change of 1.07 feet in 100 years.

Figure 1.6.2.1 Observed Sea Level Rise at Charleston Harbor Gage

1.6.3 Extreme Water Levels

According to NOAA Tides and Currents explanation of Extreme Water Levels: Extremely high or low water levels at coastal locations are an important public concern and a factor in coastal hazard assessment, navigational safety, and ecosystem management. Exceedance probability, the likelihood that water levels will exceed a given elevation, is based on a statistical analysis of historic values. This product provides annual and monthly exceedance probability levels for select Center for Operational Oceanographic Products and Services (CO-OPS) water level stations with at least 30 years of data. When used in conjunction with real time station data, exceedance probability levels can be used to evaluate current conditions and determine whether a rare event is occurring. This information may also be instrumental in planning for the possibility of dangerously high or low water events at a local level. Because these levels are station specific, their use for evaluating surrounding areas may be limited. A NOAA Technical Report, "Extreme Water Levels of the United States 1893-2010" describes the methods and data used in the calculation of the exceedance probability levels.

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

High and low annual exceedance probability levels are shown relative to the tidal datum and the geodetic North American Vertical Datum (NAVD88), if available. The levels are in meters relative to the National Tidal Datum Epoch (1983-2001) Mean Sea Level datum at most stations or a recent 5-year modified epoch MSL datum at stations with rapid sea level rates in Louisiana, Texas, and Alaska. On the left of Figure 1.6.2.2 are the exceedance probability levels for the mid-year of the tidal epoch currently in effect for the station. Figure On the right are projected exceedance probability levels and tidal datum assuming continuation of the linear historic trend.

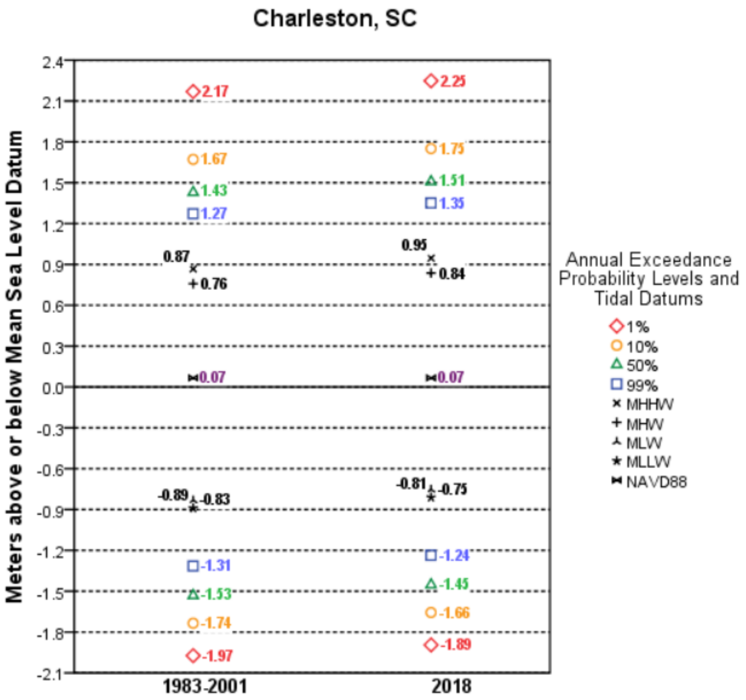


Figure 1.6.2.2 Exceedance Probability Levels and Tidal datum of 8665530 Charleston, Cooper River Entrance, SC

As stated on the NOAA Tides and Currents website, on average, shown in Figure 1.6.2.3 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 1.6.2.1 is tabulated in feet referenced to NAVD88.

(source https://tidesandcurrents.noaa.gov/est/est_station.shtml?stnid=8665530)

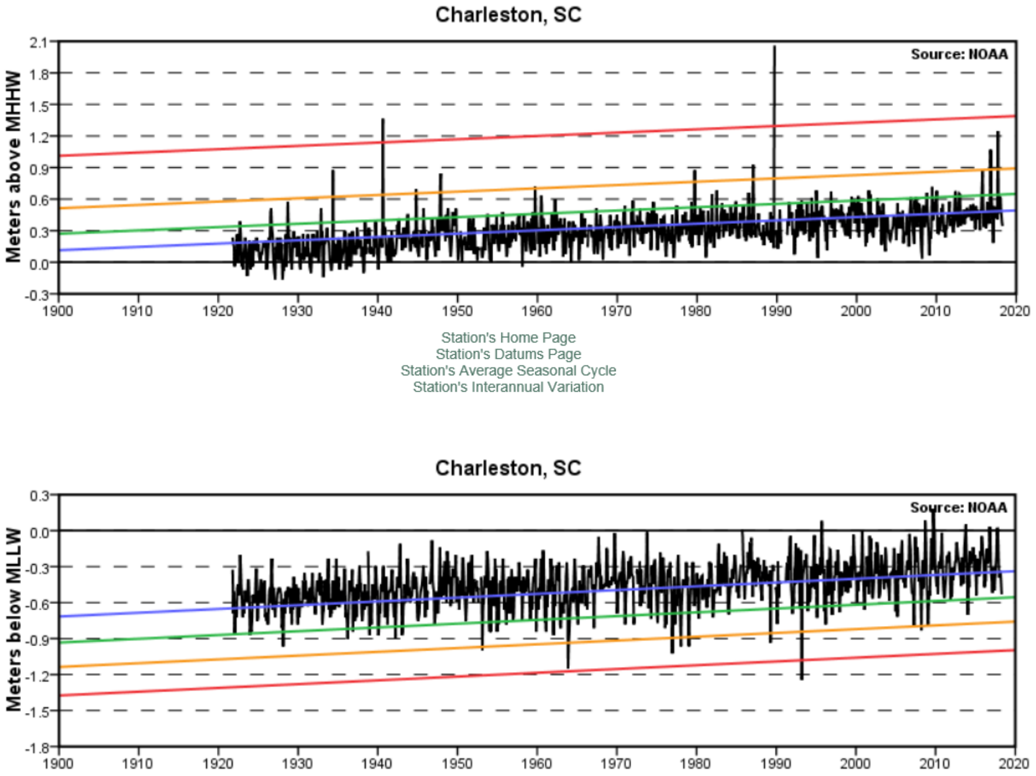


Figure 1.6.2.3 Seasonal and Interannual Variation of Gage 8665530 Extreme water Levels

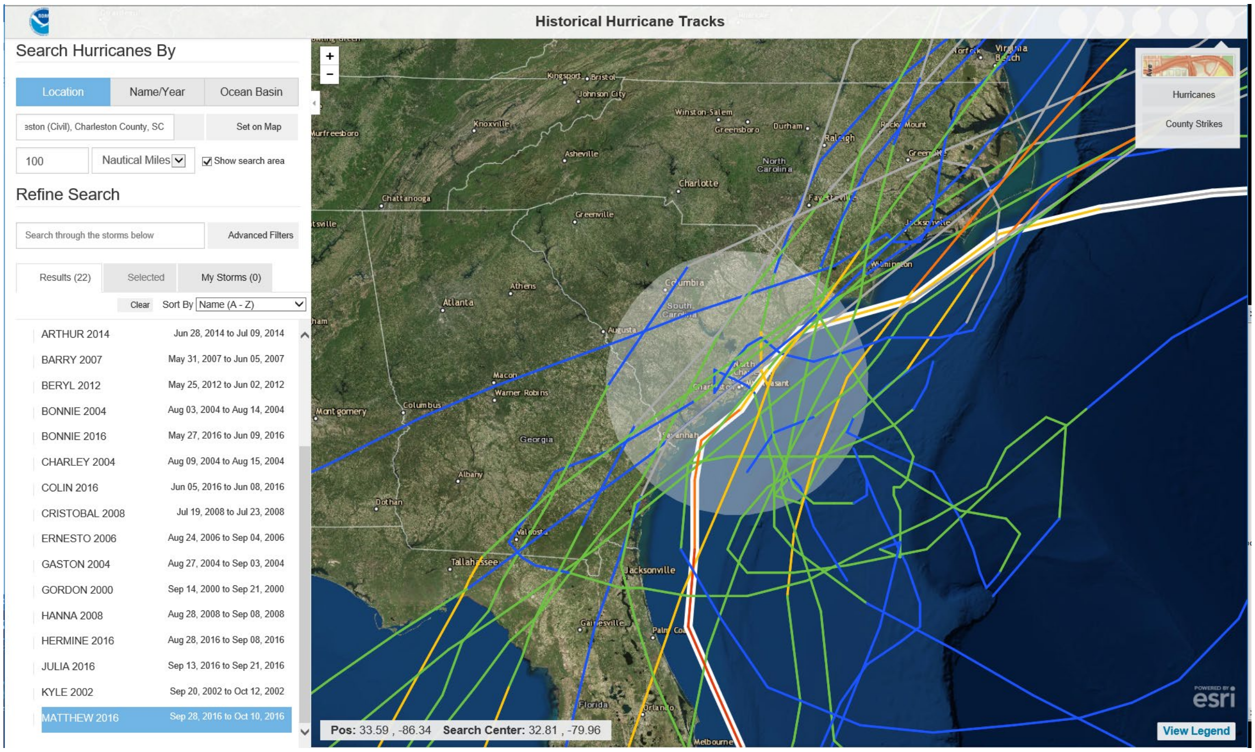
Table 1.6.2.1 Extreme Water levels and Tidal datum of 8665530 Charleston, Cooper River Entrance, SC

Version of Data :	05/17/2017
ID:	8665530
Reference Datum:	NAVD88
Name:	Charleston, SC
HAT:	4.12 (ft)
MHHW:	2.62 (ft)
MHW:	2.27 (ft)
MSL:	-0.22 (ft)
MLW:	-2.95 (ft)
MLLW:	-3.14 (ft)
NAVD88:	0.00 (ft)
EWL Type:	NOAA GEV (NAVD88)
EWLs adjusted to 2019 using the historic rate.	
*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

1.7 STORMS

1.7.1. Tropical Cyclones

Storms do not have to make landfall to have a flooding impact. Charleston experiences flooding from all three types of tropical cyclones: hurricanes, tropical storms and tropical depressions. 22 storms passed within 100 nautical miles of Charleston between 2000 and present Figure 1.7.1.1). The number of storms in the entire period of record will also be given, but an image would likely be too busy (156 storms passed the same area shown in the image).



ɘston (Civil), Charleston County, SC

100 Nautical Miles ☑ Show search area

Search through the storms below

Sort By Name (A - Z)

ARTHUR 2014	Jun 28, 2014 to Jul 09, 2014
BARRY 2007	May 31, 2007 to Jun 05, 2007
BERYL 2012	May 25, 2012 to Jun 02, 2012
BONNIE 2004	Aug 03, 2004 to Aug 14, 2004
BONNIE 2016	May 27, 2016 to Jun 09, 2016
CHARLEY 2004	Aug 09, 2004 to Aug 15, 2004
COLIN 2016	Jun 05, 2016 to Jun 08, 2016
CRISTOBAL 2008	Jul 19, 2008 to Jul 23, 2008
ERNESTO 2006	Aug 24, 2006 to Sep 04, 2006
GASTON 2004	Aug 27, 2004 to Sep 03, 2004
GORDON 2000	Sep 14, 2000 to Sep 21, 2000
HANNA 2008	Aug 28, 2008 to Sep 08, 2008
HERMINE 2016	Aug 28, 2016 to Sep 08, 2016
JULIA 2016	Sep 13, 2016 to Sep 21, 2016
KYLE 2002	Sep 20, 2002 to Oct 12, 2002
MATTHEW 2016	Sep 28, 2016 to Oct 10, 2016

Pos: 33.59 , -86.34 Search Center: 32.81 , -79.96

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

1.7.2. Hurricanes

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

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

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

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

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

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

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

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

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

Mathew (October 2016) moved north and then northwest through the Caribbean Sea and then through the Bahamas while strengthening to a Category 4 hurricane. Tracked just off the east coast of FL and GA while weakening to a Category 1 storm before making landfall near McClellanville, SC with winds near 85 mph. Produced hurricane force wind gusts along the entire coast, significant coastal flooding from high storm tides (including a record level at Fort Pulaski), and very heavy rainfall (widespread 6 to 12 inches with locally higher amounts near 17 inches) which led to significant freshwater flooding. Tide level reached 6.06 feet NAVD88.

Irma (Sep 2017) made landfall in the Florida Keys as a Category 4 hurricane and then moved along the southwest coast of Florida as a Category 3 hurricane. The storm then moved north near the west coast of Florida while weakening to a tropical storm before moving into southwest Georgia and continuing to weaken. Produced significant coastal flooding, wind gusts near hurricane-force along with 4 tornadoes, flooding rainfall and river flooding across southeast SC/GA. NOAA tide level reached elevation 6.61 feet NAVD88.

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

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

1.7.3. Historical Storms

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

CHAPTER 2 – PAST STUDIES

There have been no past USACE Coastal Storm Risk Management Studies performed for the Charleston, Berkeley, Dorchester area, where city of Charleston Peninsula resides.

There have been numerous navigation studies done on the federal navigation project in Charleston Harbor.

CHAPTER 3 – IMPACTS OF CLIMATE CHANGE

GUIDANCE

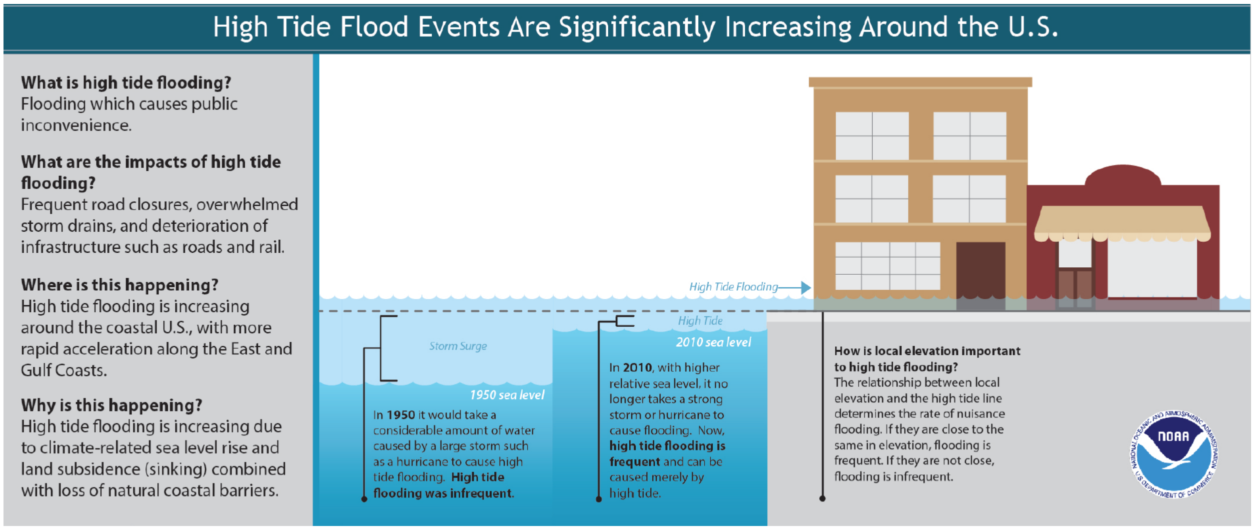
Climate change is defined as a change in global or regional climate patterns. Climate change has already been observed globally and in the United States. These included increases and changes in air and water temperatures, reduced frost days, increased frequency and intensity of heavy downpours, a rise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

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

3.1 OBSERVED IMPACTS

The effects of Climate change are already observed in the study area with the increase in "nuisance" flooding. According to NOAA's Ocean Service: high tide flooding, sometimes referred to as "nuisance" flooding, is flooding that leads to public inconveniences such as road closures (Figure 3.1.1). It is increasingly common as coastal sea levels rise. As relative sea level increases, it no longer takes a strong storm or a hurricane to cause coastal flooding. Flooding now occurs with high tides in many locations due to climate-related sea level rise, land subsidence, and the loss of natural barriers.

High tide flooding—which causes such public inconveniences as frequent road closures, overwhelmed storm drains and compromised infrastructure—has increased in the U.S. on average by about 50 percent since 20 years ago and 100 percent since 30 years ago.



High Tide Flood Events Are Significantly Increasing Around the U.S.

What is high tide flooding?
Flooding which causes public inconvenience.

What are the impacts of high tide flooding?
Frequent road closures, overwhelmed storm drains, and deterioration of infrastructure such as roads and rail.

Where is this happening?
High tide flooding is increasing around the coastal U.S., with more rapid acceleration along the East and Gulf Coasts.

Why is this happening?
High tide flooding is increasing due to climate-related sea level rise and land subsidence (sinking) combined with loss of natural coastal barriers.

In **1950** it would take a considerable amount of water caused by a large storm such as a hurricane to cause high tide flooding. **High tide flooding was infrequent.**

In **2010**, with higher relative sea level, it no longer takes a strong storm or hurricane to cause flooding. Now, **high tide flooding is frequent** and can be caused merely by high tide.

How is local elevation important to high tide flooding?
The relationship between local elevation and the high tide line determines the rate of nuisance flooding. If they are close to the same in elevation, flooding is frequent. If they are not close, flooding is infrequent.

Figure 3.1.1. High Tide Flooding

NOAA Ocean Service further explains: A King Tide is a non-scientific term people often use to describe exceptionally high tides. Tides are long-period waves that roll around the planet as the ocean is "pulled" back and forth by the gravitational pull of the moon and the sun as these bodies interact with the Earth in their monthly and yearly orbits. Higher than normal tides typically occur during a new or full moon and when the Moon is at its perigee, or during specific seasons around the country.

SCDHEC is leading the South Carolina **King Tides** initiative to document the effect that extreme **tide** events have on our state's beaches, coastal waterways, private properties and public infrastructure on their MyCoast website. The effects of individual King Tides may vary considerably. King Tides may result in coastal erosion, flooding of low-lying areas, and road closures which may disrupt normal daily routines. This is particularly true when a King Tide coincides with significant precipitation because water drainage and runoff is impeded.

As an example: DHEC issues King Tide notifications to MyCoast members when water levels are predicted to reach 6.6 feet above mean lower low water (MLLW) (or 3.46 ft NAVD88) or higher at the Charleston Harbor Tide Station. NOAA’s National Weather Service (NWS) Forecast Office in Charleston has established thresholds for minor (7.0 ft. MLLW), moderate (7.5 ft. MLLW), and major (8.0 ft. MLLW) flooding in the Charleston area. NOAA has also established a threshold for high tide flooding (HTF) in Charleston (7.6 ft. MLLW). Thresholds established for the Charleston area and terminology descriptions are provided in Table 3.1.1 below.

Table 3.1.1 Flooding Thresholds for Charleston, SC

Water Level Thresholds Established (Feet above MLLW)		**Feet above NAVD88**
Action Stage (NOAA NWS)	6.5	3.36
King Tide (SCDHEC)	6.6	3.46
Minor Flooding (NOAA NWS) Minor flooding on roadways around Downtown Charleston occurs, possibly including Lockwood Drive, Wentworth and Barre, Fishburne and Hagood, and Morrison Drive. As the tide height approaches 7.5 ft MLLW, roads can become impassable and closed	7.0	3.86
Moderate Flooding (NOAA NWS) In Downtown Charleston, additional impacted roads include HW-17 at HW-61, Market Street, East Bay, Rutledge, and areas around MUSC.	7.5	4.36
Major Flooding (NOAA NWS) Widespread flooding occurs in Downtown Charleston with numerous roads flooded and impassable and some impact to structures	8.0	4.86

Terminology

Action Stage: *The stage or level where the NWS or a partner/user needs to take action in preparation for possible significant hydrologic activity (NOAA NWS).*

King Tide: *A non-scientific term often used to describe exceptionally high tides (NOAA National Ocean Service).*

Minor Flooding: *Minimal or no property damage, but possibly some public threat (NOAA NWS).*

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

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

High Tide Flooding (HTF): *Heights ranging from about 0.5 to 0.65 meters above mean higher high water and varying regionally with tide range. HTF height thresholds are based upon the minor-flood thresholds set by NWS Weather Forecasting Offices (WFOs) and on-the-ground local emergency managers who prepare for response to impending conditions (NOAA National Ocean Service).*

Further information on nuisance flooding can be found at https://oceanservice.noaa.gov/facts/nuisance-flooding.html).

High tides affect drainage systems by filling the stormwater collection systems, such that rainfall will pool and the runoff water will drain slower than if the systems were open. As Sea Level Rises in South Carolina, the occurrence of flooding associated with King Tides will also increase. Adapted from: Sweet, W. V., and J. Park, 2014: From the extreme to the mean: Acceleration and tipping points of coastal inundation from sea level rise, City of Charleston plotted “Observed and Predicted “Minor Coastal Flooding” in Charleston” (Figure 3.1.2) in their Sea Level Rise Strategy, 2019. Charleston SC expects a significant increase based on trend and even more if sea level rise rate increases. Increases are expected along the entire South Carolina coast.

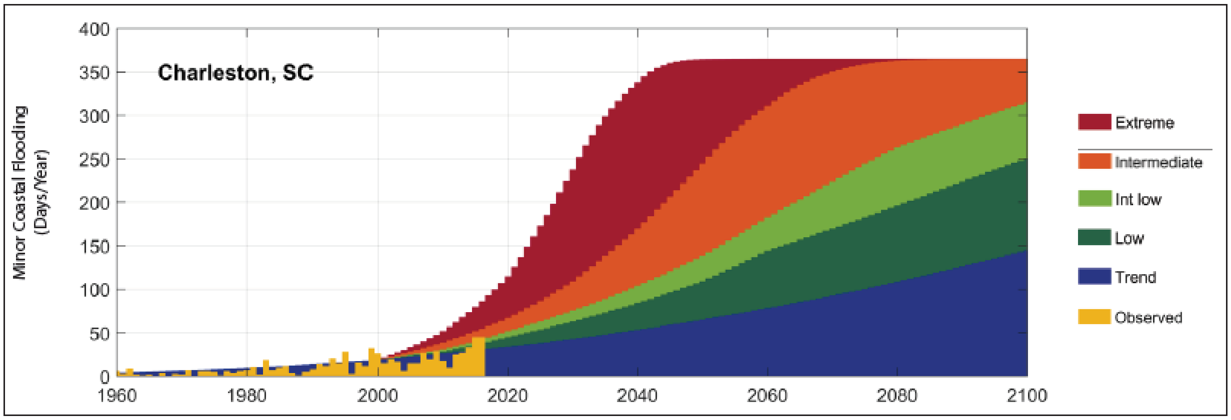


Figure 3.1.2. Observed and Predicted “Minor Coastal Flooding” in Charleston

The City of Charleston has already taken steps to address the tidal filling of storm drains by adding check valves on many of the cities storm drainage pipelines and plans to continue. A check valve prevents seawater from backing up into drainage infrastructure to mitigate tidal flooding, while still allowing the outfall to drain stormwater as usual when the tide recedes. Overland flooding in areas such as Lockwood Boulevard are due to low-lying areas adjacent to the river and harbor which have a direct shoreline to increasing water levels.

3.2 COMPONENTS OF RELATIVE SEA LEVEL RISE

Sea Level Rise is an increase in the volume of water in the world’s ocean, resulting in an increase in sea level called global sea level rise. The sea level rise local to a specific area is called relative sea level rise. Sea level rise at specific locations (relative sea level rise) may be more or less than the global average (global sea level rise). Sea level rise is attributed to global climate change by the added water from melting ice sheets and glaciers. Melting of floating ice shelves or icebergs at sea raises sea levels only slightly. Local factors such as subsidence of the land also impact local communities. Subsidence is the motion of the land surface as it shifts downward relative to a vertical datum.

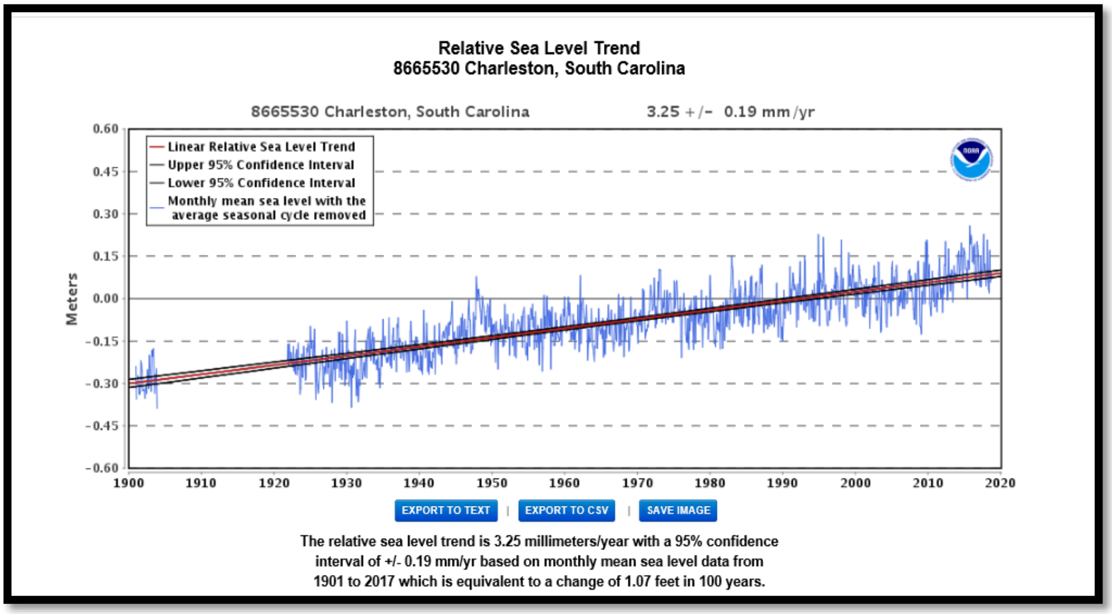
3.3 RATES OF RELATIVE SEA LEVEL RISE

RSLR considers the effects of (1) the eustatic, or global, average of the annual increase in water surface elevation due to the global warming trend, and (2) the “regional” rate of vertical land movement (VLM) that can result from localized geological processes, including the shifting of tectonic plates, the rebounding of the Earth’s crust in locations previously covered by glaciers, the compaction of sedimentary strata and the withdrawal of subsurface fluids (USGS 2013).

Figure 3.3.1 Location of Charleston Gage 8665530

3.3.1 Historic Rate

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 3.3.1.1. 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.



The relative sea level trend is 3.25 millimeters/year with a 95% confidence interval of +/- 0.19 mm/yr based on monthly mean sea level data from 1901 to 2017 which is equivalent to a change of 1.07 feet in 100 years.

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

3.3.2 Intermediate and High Rate

The rate for the "USACE Intermediate Curve" is computed from the modified NRC Curve I considering both the most recent IPCC projections and modified NRC projections with the local rate of vertical land movement added.

The rate for the "USACE High Curve" is computed from the modified NRC Curve III considering both the most recent IPCC projections and modified NRC projections with the local rate of vertical land movement added.

According to National Oceanographic and Atmospheric Administration (NOAA) and using the USACE Sea-Level Change Curve Calculator (Version 2017.55) for the Charleston Gage 8665530, the sea level rise in 2100 for the intermediate rate is 1.81 feet and for high rate is 4.89. Using the SLC tool to project the RSLC scenarios past 2100, for the purpose of evaluating PLANNING Horizon project adaptation strategies results in historic (low) rate projection of 1.10 feet, intermediate rate projection of 2.60 feet and a high rate project of 7.38 feet for the year 2125. (Table 3.3.3.1)

Table 3.3.3.1 Estimates Sea Level Change 2019 to 2150

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

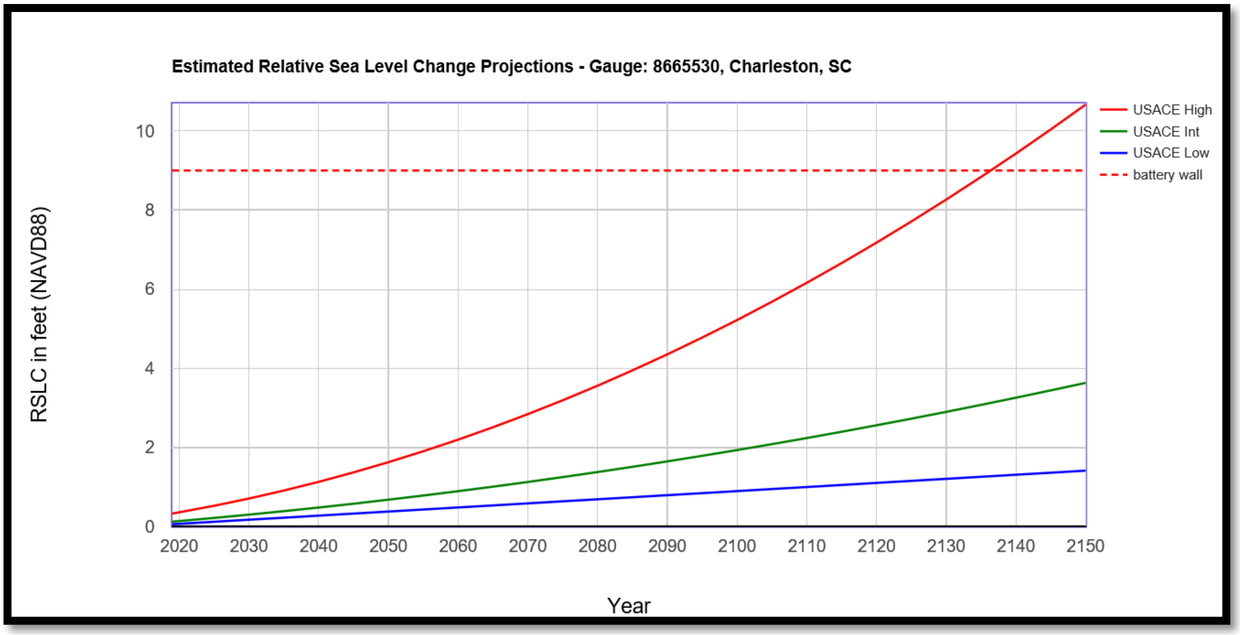


Figure 3.3.3.1 Low, Intermediate and High Sea Level Projection Gage 8665530

3.3.4 Rate used for this study

Historic rate has already been shown to be changing, based on recent trends. The historic or 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.

The future condition 50 years after construction assumed to be 2025. Alternatives were evaluated using the most likely SLR of the intermediate rate. The following Figure 3.3.4.1 shows the increase in flooding from Hurricane Irma that would occur assuming the 1.13 feet of sea level rise as depicted with the HECRAS modeling.

Table 3.3.4.1 Project SLR over Study period and Planning Horizon.

Projected SLR Increase (ft) 2019 to 2075 using 2019 sea level trend of 0.01033 ft/yr	
USACELow	0.58
USACE Intermediate	1.13
USACE High	2.86
PLANNING Horizon project adaptation strategies results for the year 2125:	
Historic (low)	1.10 feet
Intermediate	2.60 feet
High	7.38 feet

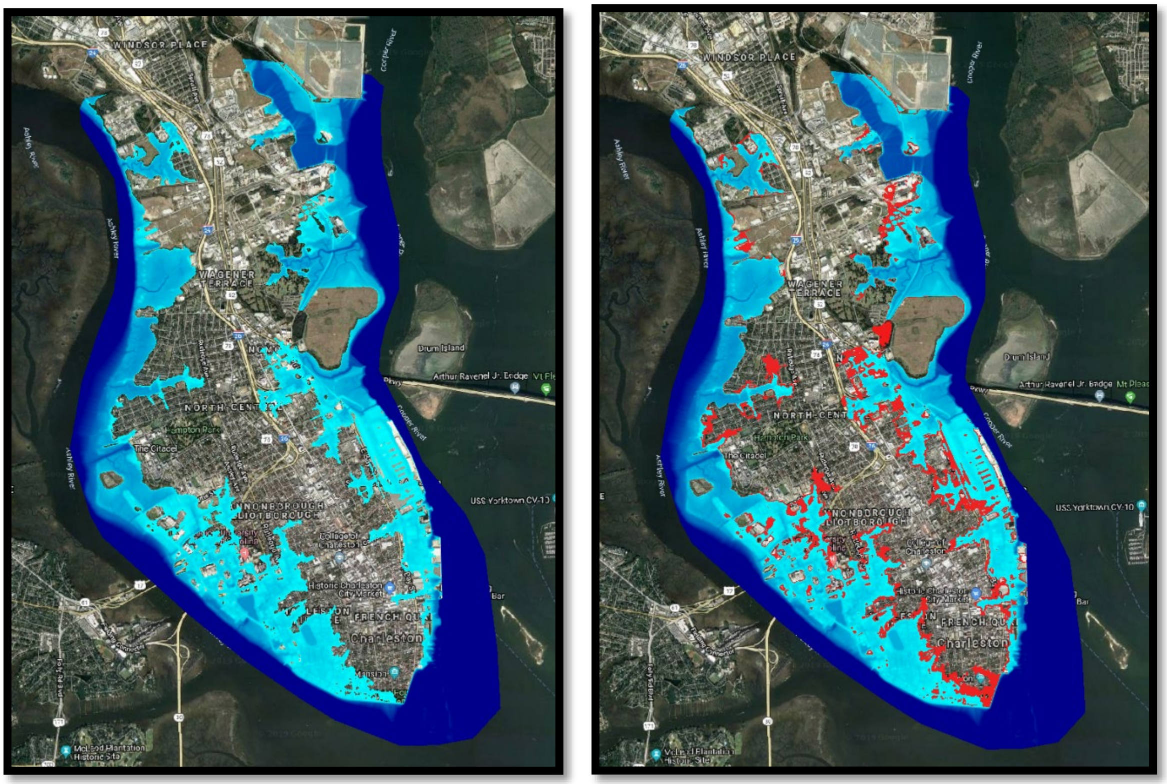


Figure 3.3.4.1 a. Hurricane Irma surge flooding – blue (without rainfall) and b. with 1.13 feet of SLR-red (without rainfall).

3.4 EXTREME WATER LEVEL PROJECT WITH SEA LEVEL RISE

The 5% AEP Extreme water level indicates an elevation of 6.9 ft NAVD88 in the year 2075, when using the intermediate rate of sea level change.

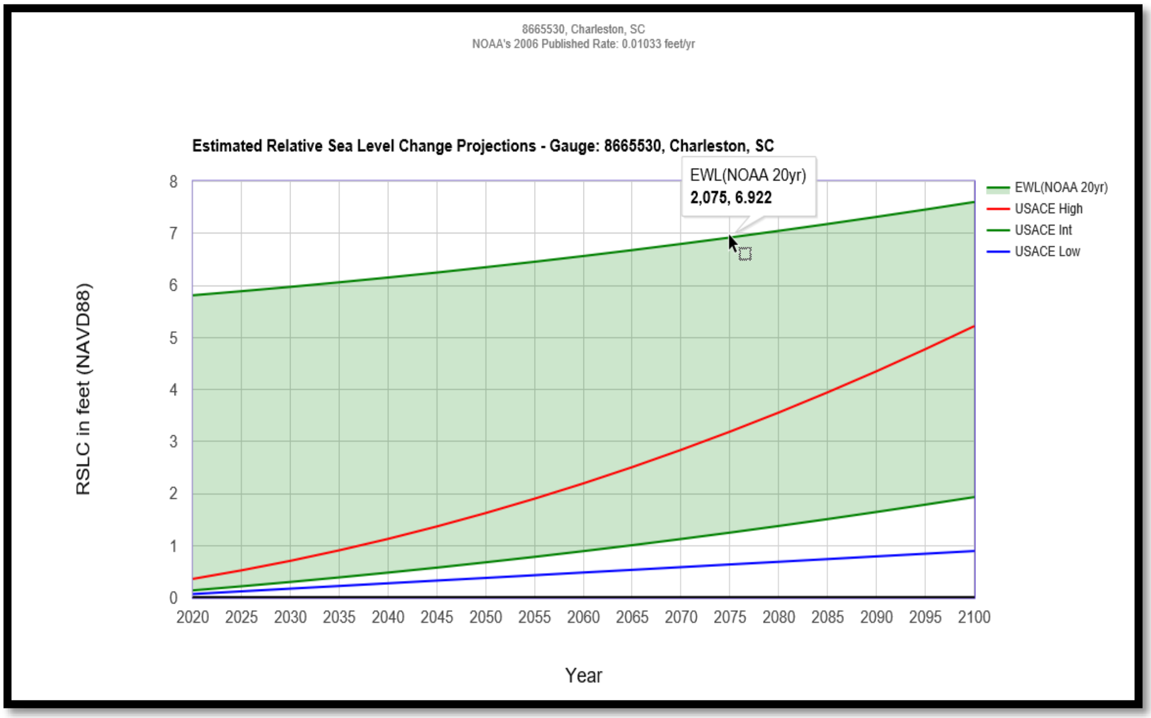


Figure 3.4.1 Estimate Intermediate RSLC with 5% AEP EWL

3.5 SPONSOR SEA LEVEL RISE STRATEGY

The City of Charleston has indicated their intent to use the latest NOAA 2017 projections for their future considerations (shown in Figure 3.5.1 and Table 3.5.1). "**In** the 2015 Sea Level Rise Strategy, the City recommended a 1.5 to 2.5 foot elevation increase for new facilities and infrastructure to account for sea level rise over 50 years. Considering the latest sea level rise projections, the City is increasing the recommendation to 2 to 3 feet. The range accounts for varying types of investments: a 2-foot increase is intended for less vulnerable infrastructure such as parking lots, while a 3-foot increase is for more critical long term infrastructure, such as medical facilities."

(Source: https://www.charleston-sc.gov/DocumentCenter/View/20299/Flooding-and-Sea-Level-Rise-Strategy-2019-web-viewing?bidId=)

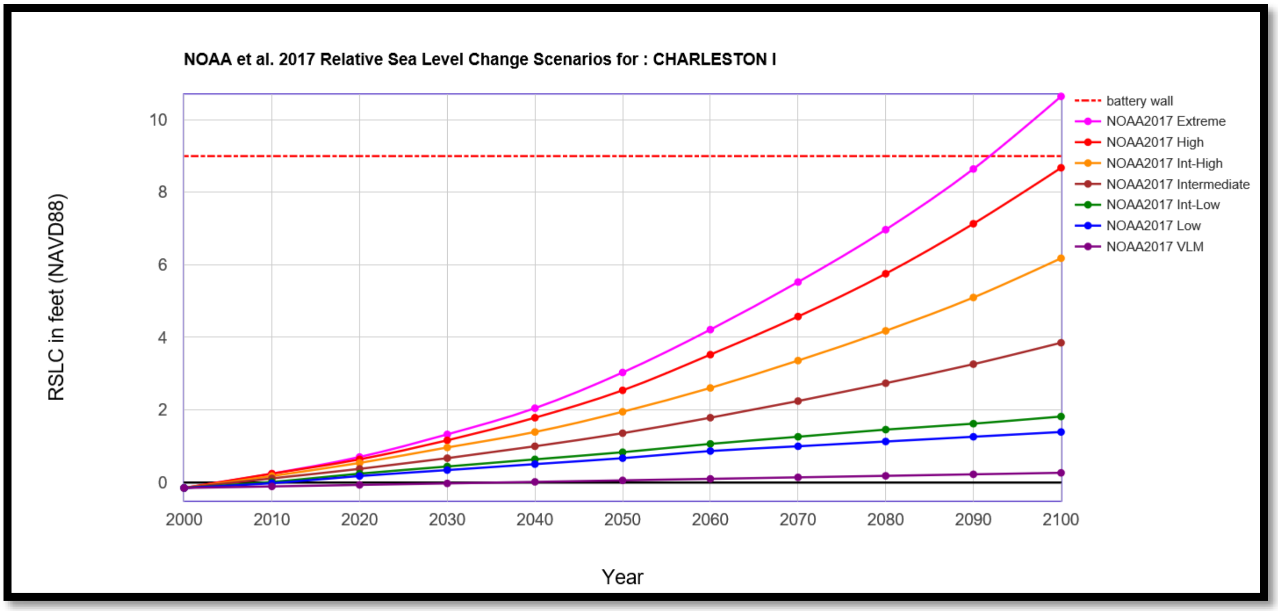


Figure 3.5.1 NOAA Relative Sea Level Change for Charleston

Table 3.5.1 NOAA Relative Sea Level Change for Charleston

Charleston Peninsula
Scenarios for CHARLESTON I
NOAA2017 VLM: 0.00417 feet/yr
All values are expressed in feet

Year	NOAA2017 VLM	NOAA2017 Low	NOAA2017 Int-Low	NOAA2017 Intermediate	NOAA2017 Int-High	NOAA2017 High	NOAA2017 Extreme
2000	-0.15	-0.15	-0.15	-0.15	-0.15	-0.15	-0.15
2010	-0.11	-0.02	0.02	0.11	0.18	0.25	0.25
2020	-0.06	0.18	0.25	0.38	0.54	0.64	0.70
2030	-0.02	0.34	0.44	0.67	0.97	1.16	1.33
2040	0.02	0.51	0.64	1.00	1.39	1.79	2.05
2050	0.06	0.67	0.84	1.36	1.95	2.54	3.03
2060	0.10	0.87	1.07	1.79	2.61	3.53	4.22
2070	0.14	1.00	1.26	2.25	3.36	4.58	5.53
2080	0.19	1.13	1.46	2.74	4.18	5.76	6.97
2090	0.23	1.26	1.62	3.26	5.10	7.14	8.64
2100	0.27	1.39	1.82	3.85	6.18	8.68	10.65

3.6 PROJECTED WATER SURFACE ELEVATION WITH ANNUAL EXCEEDANCE PROBABILITY

Using FEMA still water elevation levels from the most recent Flood Insurance Study (still preliminary), ERDC generated of Annual Exceedance Probability (AEP) for each of the save points requested (Table 3.6.1) . 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. NWS indicates major flooding occurs at 4.86 ft NAVD88 – which now has approximately a 50 percent annual exceedance probability (AEP) Stillwater elevation. It is important to note, however, this differs from the base flood elevation because the Stillwater level does not include wave regeneration that occurs over a large body of water before it reaches the shoreline. Also the tide range in Charleston is up to 6 feet, suggesting that the tide phase at the time of landfall may significantly influence surge levels produced by a given storm.

Table 3.6.1 Annual Exceedance Probability Stillwater Elevations

	AEP(%)	**AEP(%)**	**AEP(%)**	**AEP(%)**	**AEP(%)**	**AEP(%)**	**AEP(%)**	**AEP(%)**	**AEP(%)**
Save Point	**50**	**20**	**10**	**4**	**2**	**1**	**0.5**	**0.2**	**0.1**
	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88
Wagener terrace	4.7	5.1	5.3	5.9	8.3	10.3	12.1	14.4	16.2
Marina	4.7	5.1	5.3	5.8	8.4	10.4	12.2	14.6	16.5
Newmarket	4.7	5.1	5.3	5.8	8.3	10.3	12.2	14.6	16.5
Port	4.7	5.0	5.3	5.8	8.3	10.3	12.2	14.7	16.5
Battery	4.7	5.0	5.3	5.8	8.3	10.4	12.3	14.7	16.6

To project water surface elevations into the future for the AEP the intermediate rate of sea level rise of 1.13 feet was added to the existing still water elevations (Table 3.6.2).

Table 3.6.2 Projected Water Levels associated with Annual Exceedance Probability

	AEP(%)	**AEP(%)**	**AEP(%)**	**AEP(%)**	**AEP(%)**	**AEP(%)**	**AEP(%)**	**AEP(%)**	**AEP(%)**
Save Point	**50**	**20**	**10**	**4**	**2**	**1**	**0.5**	**0.2**	**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

During optimization, several wall elevations will be evaluated. G2CRM will be the primary tool for assessing the damages and benefits derived from various wall elevations. After the final selected plan is chosen, further evaluation of the risk associated with the selected wall will be performed.

CHAPTER 4 -WAVE DATA, MODELING, AND RESULTS

4-1 Modeling

As previously stated, there were no existing USACE studies addressing Coastal Storm Risk Management. USACE reached out to SCDNR, the FEMA POC for Flood Insurance Studies (FIS) in the state of SC, for available coastal models to minimize costs and improve efficiencies of the study. FEMA/SCDNR contractor, AECOM, provided ADCIRC models, storm sets, SWAN 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 (Figure 4.1 and Figure 4.2). ERDC evaluated the suite of storm provided by AECOM and selected a subset of storms. The goal of storm selection was to find the optimal combination of storms given a predetermined number of storms to be sampled (e.g., 20 TCs), referred to as reduced storm set (RSS). In the process of selecting 20 TCs, it was determined that a RSS of this size adequately captured the storm surge hazard for the range of probabilities covered by the FSS (122 TCs). In order to also include high frequency events, five (5) additional storms were selected from the range of probabilities determined from EVA of water level measurements. Details are found in ERDC report located in Sub-Appendix 5 COASTAL MODELING SUB-APPENDIX.

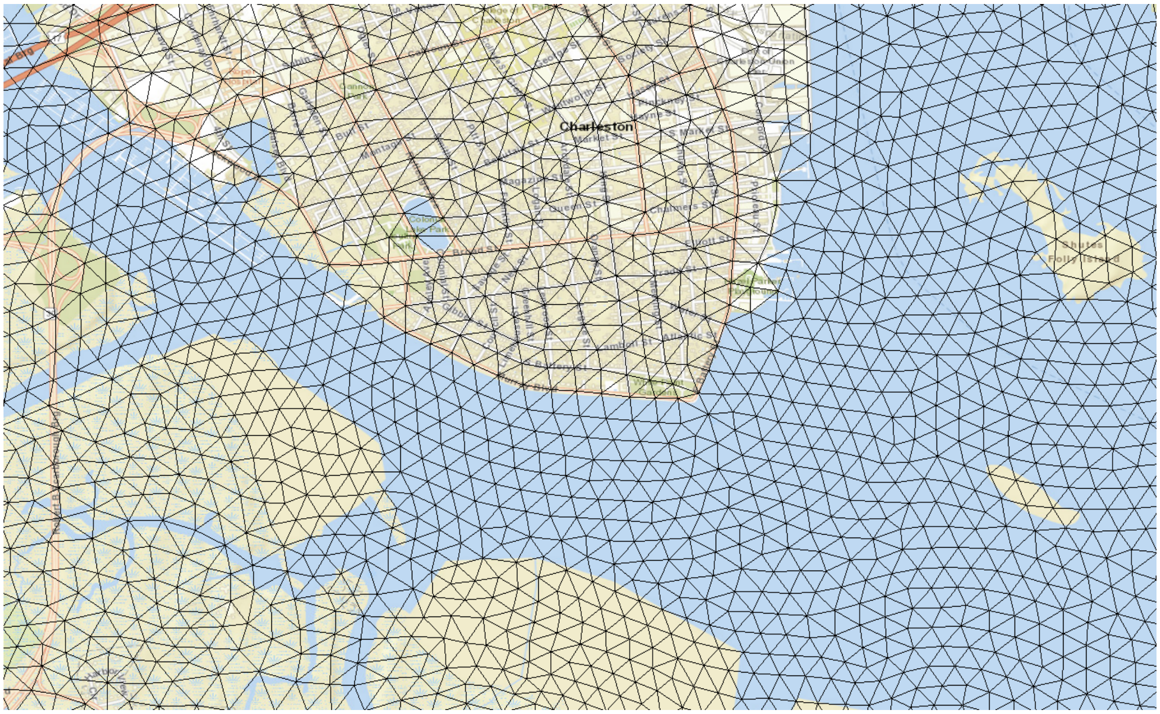


Figure 4.1 Zoom in to the Charleston Peninsula (before the grid refinement).

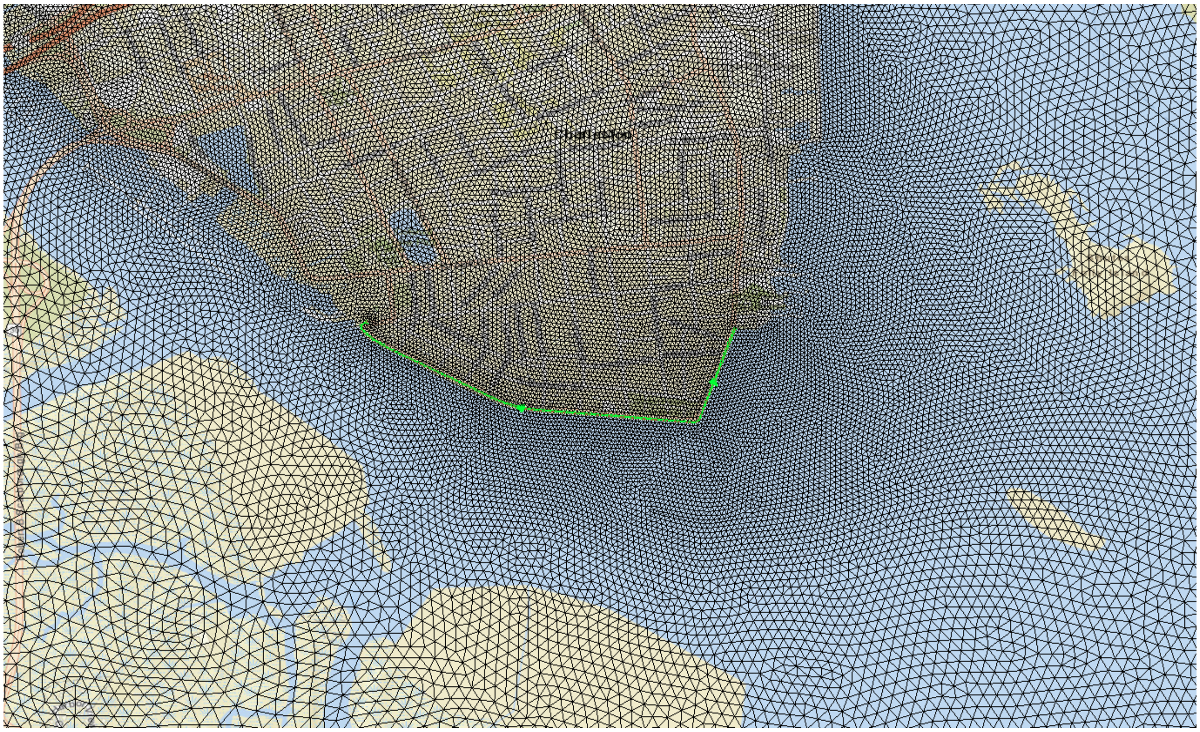


Figure 4.2 Zoom in to Charleston Peninsula after grid refinement and FWO condition of battery wall

ERDC was asked to run STWAVE and ADCIRC to generate time series still water elevations for input into the G2CRM model. Figure 4.3 shows the STWAVE domain for the analysis. 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.

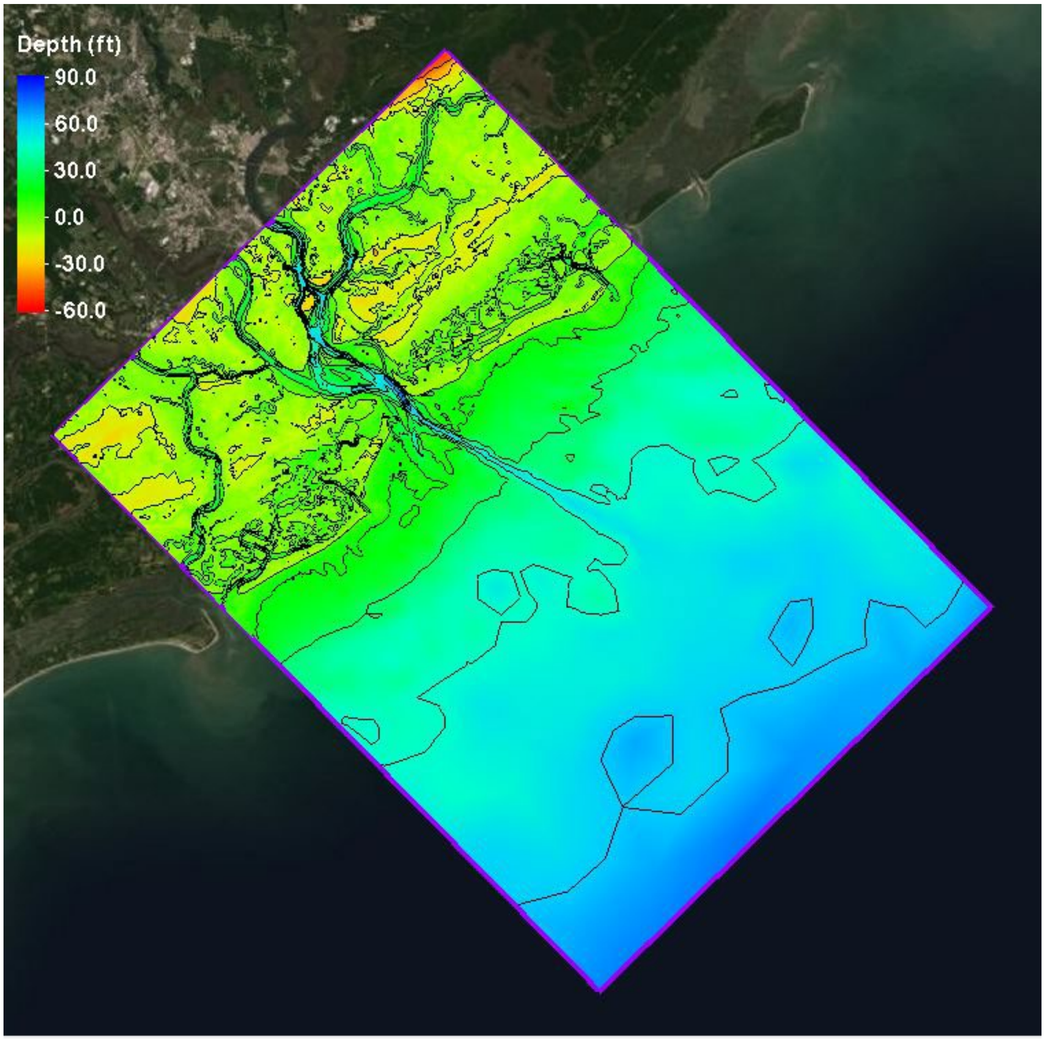


Figure 4.3 STWAVE Domain

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

As explained in the SOUTH CAROLINA STORM SURGE PROJECT DELIVERABLE 3: PRODUCTION RUNS, FINAL STATISTICS, AND RESULTS ANALYSIS report generated by URS for FEMA/SCDNR. "The tide range in South Carolina is up to 6 feet (ft), suggesting that the tide phase at the time of landfall may significantly influence the surge levels produced by a given storm. Statistical analysis using the JPM-OS determined that application of a Monte Carlo method to provide a random initial tidal level at the start of each production run would account for tidal variations in the storm surge analysis. Each production run began with a random tide phase in order to vary the phasing of the tide relative to the storm. The

random phases were derived from a 60-day tide simulation from August 1 to September 30, 2010, which was preceded by a 15-day spin up period necessary for the model forcing to ramp up.
To account for steric effects, the project team calculated the seasonal water level change induced by the solar annual (SA) and solar semi-annual (SSA) tidal constituents during the 60-day period at Charleston Harbor. The amplitude, phase, and frequency of the constituents were obtained from the National Oceanic and Atmospheric Administration (NOAA) (NOAA, 2013). The project team determined the mean steric effect over the 60-day period of the simulations by integration (as sine waves with time = 0 on January 1 of each year) to obtain a total increase of 2.75 inches (7 cm) above mean sea level (MSL). "

Since G2CRM includes tide and sea level rise, the Stillwater elevations are generated in meters at MSL and were then converted to feet MSL. Plots are shown in Figure 4.4. 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.

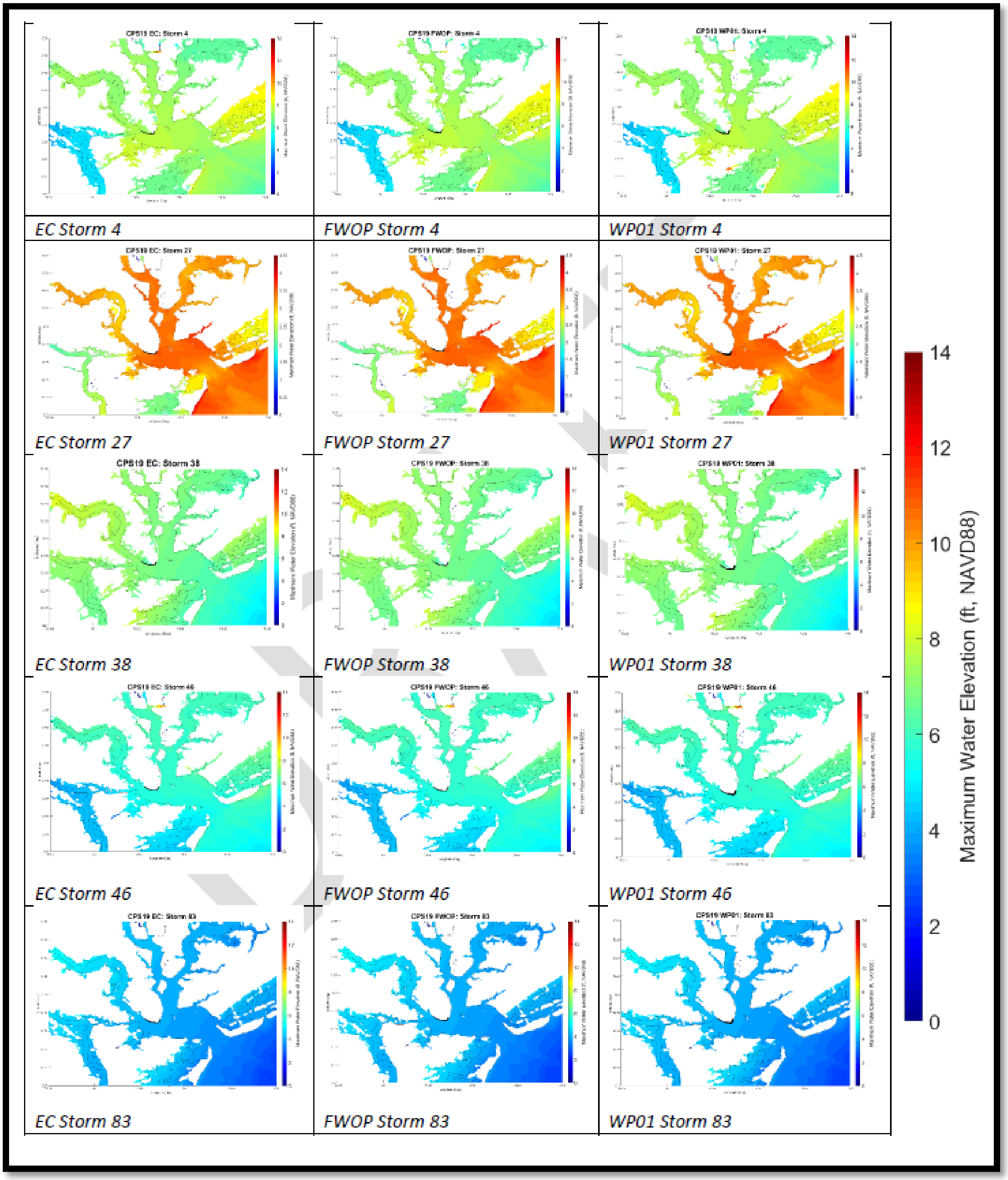


Figure 4.4 Plots of Still Water Elevations

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 a no more than 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 the model.

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.

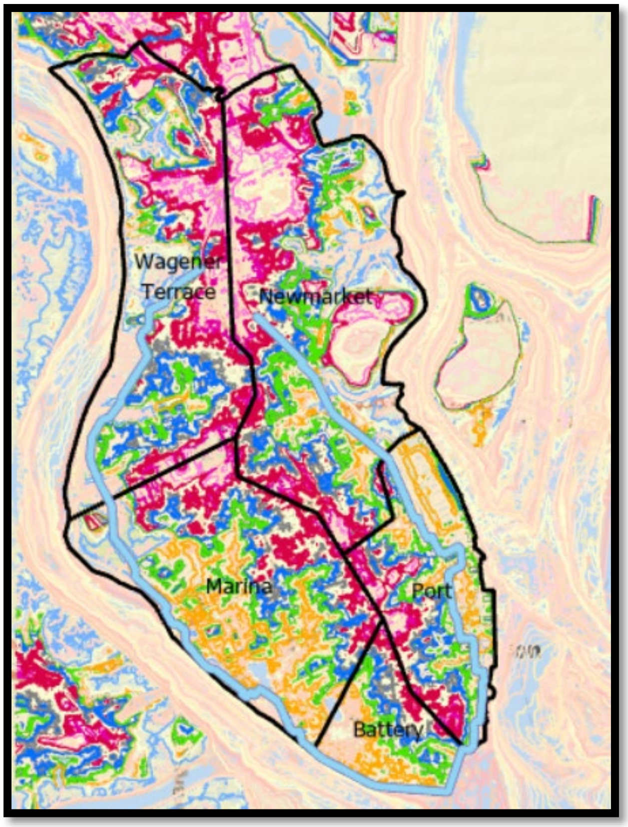


Figure 5.1.1 Map depicting Model Areas

5.2. ADCIRC Water Levels

Using the FEMA analysis of still water elevation levels, ERDC generated of AEP for each of the save points submitted by the SAC. 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

	AEP%	**AEP%**	**AEP%**	**AEP%**	**AEP%**	**AEP%**	**AEP%**	**AEP%**	**AEP%**
Model Area	**50**	**20**	**10**	**4**	**2**	**1**	**0.5**	**0.2**	**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 - don't take houses/businesses unless there is no other option. Earthen embankments were eliminated due to the large real estate requirements needed, so vertical walls are recommended. 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. Figure 5.3.1 shows the recommended footprint evaluated. Further refinement of footprint will occur during optimization during post TSP.

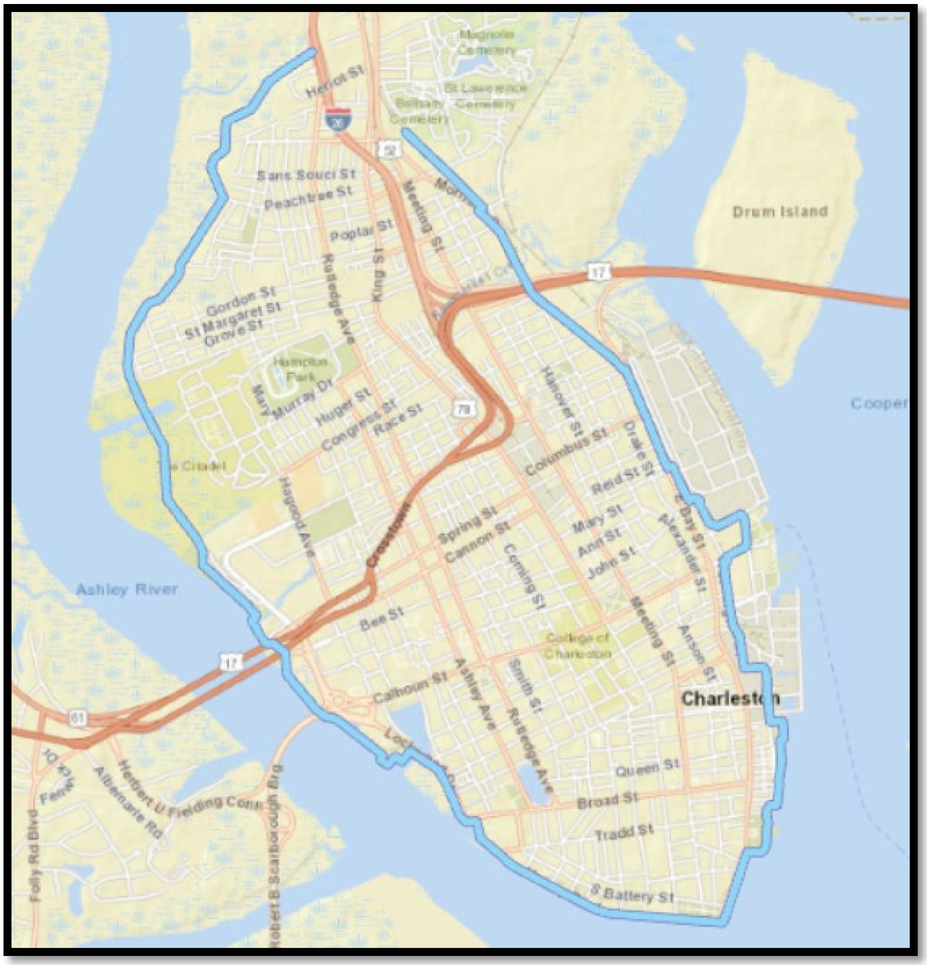


Figure 5.3.1 Alignment of the Perimeter Storm Surge Wall

CHAPTER 6 - WAVE OVERTOPPING ANALYSIS

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 6.1 depicts typical surge profile as it approaches land with a vertical wall.

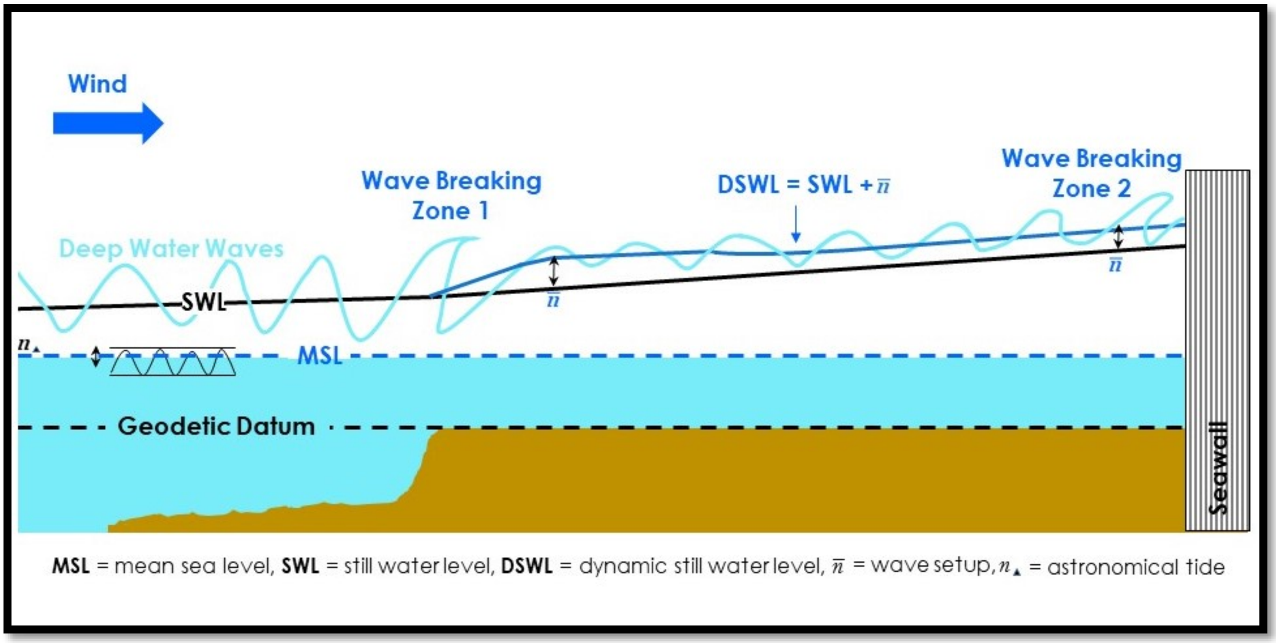


Figure 6.1 Dynamic Still Water Level

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.

6.1 Overtopping Floodwall Analysis

Using the still water elevation annual exceedance probability, based on the FEMA analysis and adding sea level rise of 1.13 feet resulted in the following frequency curve at a save point near the NOAA gage, shown in figure 6.1.1. and Table 6.1.1. . The 1 percent Stillwater AEP is estimated to be 11.5 feet NAVD88. The 10 percent AEP Stillwater elevation is 6.4 ft NAVD88.

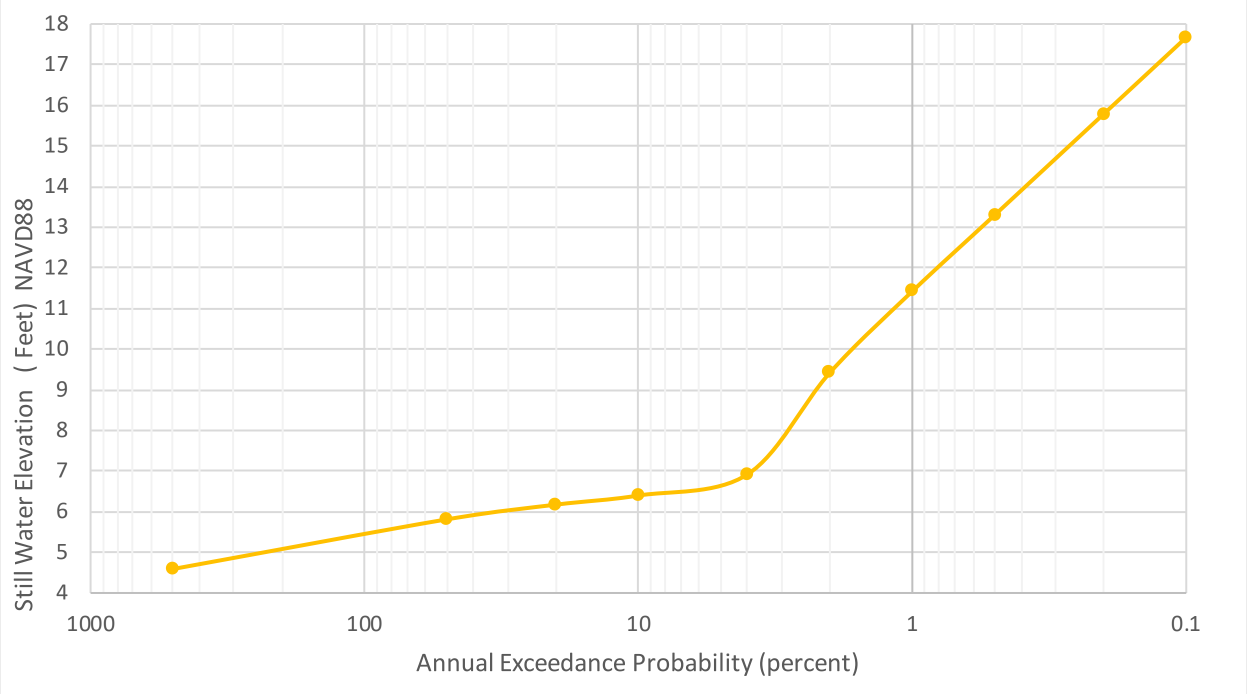


Figure 6.1.1 Stillwater Elevation Annual Exceedance probability with addition of 1.13 feet of sea level rise at save point nearest NOAA gage.

Table 6.1.1 Stillwater Elevation Annual Exceedance probability with addition of 1.13 feet of sea level rise at save point nearest NOAA gage.

		SLR =	**1.13**	**tide**					
Save Point	**50**	**20**	**10**	**4**	**2**	**1**	**0.5**	**0.2**	**0.1**
	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88
Port	5.8	6.2	6.4	6.9	9.4	11.4	13.3	15.8	17.7

Still water elevations were computed at MSL, therefore the risk of flooding at high tide has to be considered when assessing risk and potential damages. This was considered in the G2CRM analysis of damages, but the still water elevation should not be considered the total probability of risk, so as to not mislead the public. The still water elevation is documented in the FIS but it is not the Base Flood Elevation that is considered a better estimate of the flood hazard. To obtain the final Base Flood Elevations (BFEs), FEMA then uses WHAFIS, for the overland wave height analysis. The WHAFIS model can also cause wave regeneration if it goes over a sizable body of water. It can then dissipate as it passes over land as shown in Figure 6.1.2, obtained from FEMA contractor.

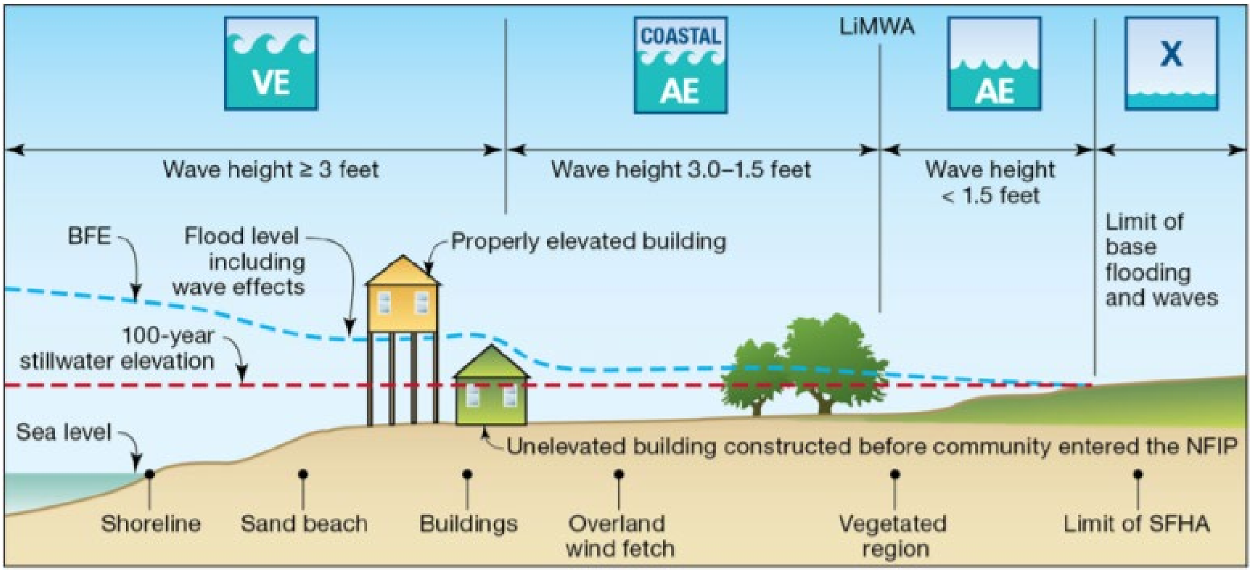


Figure 6.1.2 Demonstration of Stillwater elevation, BFE and various Special Flood Hazard Areas. (Source FEMA)

Considering the risk at high tide (a dynamic still water level) the AEP graph changes to Figure 6.1.3 and table 6.1.2. This would result in 1 percent AEP of 13.7 feet NAVD88 and a 10 percent AEP of 8.7 ft NAVD88.

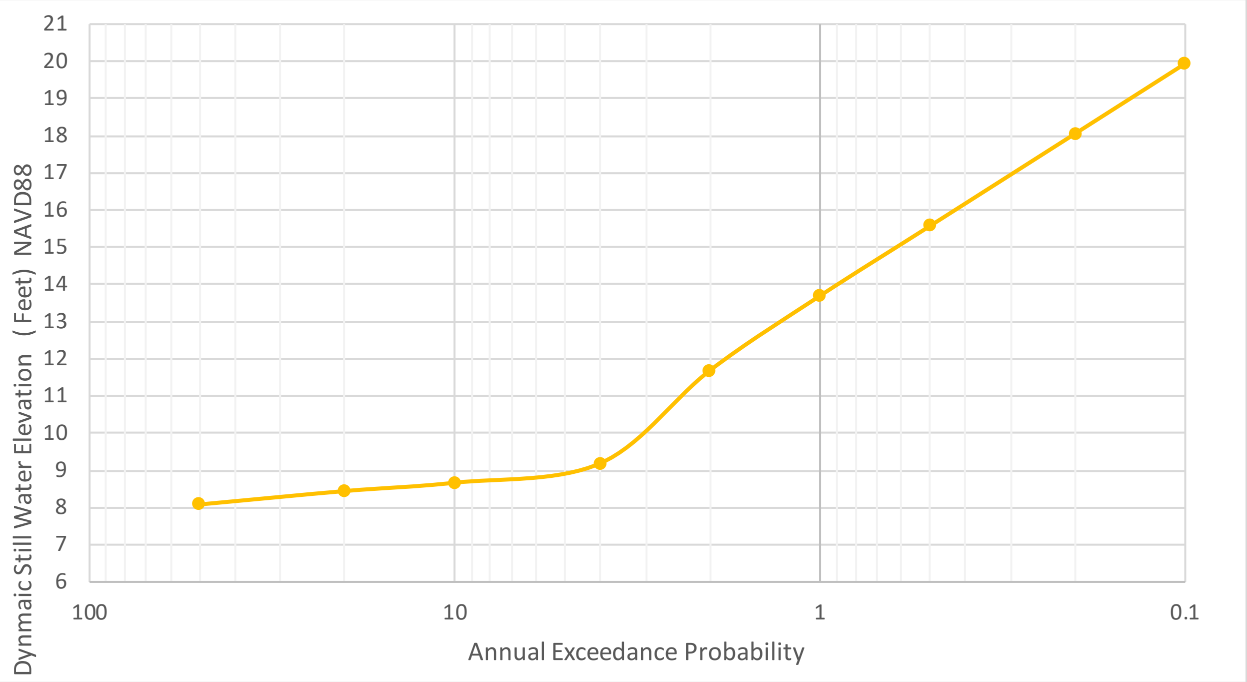


Figure 6.1.3 Dynamic Still Water Elevation Annual Exceedance probability (with addition of 2.27 feet of high tide) at save point nearest NOAA gage

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, a wall at 12 feet NAVD88 would equate to approximately 1.8 percent AEP.

Table 6.1.2 Dynamic Still Water Elevation Annual Exceedance probability (with addition of 2.27 feet of high tide) at save point nearest NOAA gage.

		SLR =	1.13	tide	2.27				
Save Point	AEP % 50	AEP % 20	AEP % 0	AEP % 4	AEP % 2	AEP % 1	AEP % 0.5	AEP % 0.2	AEP % 0.1
	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88	NAVD88
Port	8.1	8.4	8.7	9.2	11.7	13.7	15.6	18.1	19.9

6.2. 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.

CHAPTER 7 – WAVE FORCES ON A VERTICAL WALL

On the final selected wall elevation and footprint, to determine the wave forces on the vertical wall due to wave action, an evaluation will be done using Eurotop.

CHAPTER 8 – INTERIOR DRAINAGE ANALYSIS

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

While the storm drainage system is not a CSRM responsibility, any impacts to the interior hydrology due to the proposed project have to be evaluated and mitigated to the extent justified under USACE policy, if necessary. 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. Originally it was discussed if they would provide an existing model of the entire pipe network system in one model - PCSWMM. This was not doable in the time frame of the study based on conversations with the city on their contract approval procedures.

City of Charleston Contractor indicated they had majority of the study area in HEC RAS 2D, which they use the HEC RAS for rainfall and flow to the inlets for the drainage system and then pipe network model for conveyance to river or to the drywell/pump system depending upon drainage area). They have provided the HEC RAS model. The provided RAS model did not cover the entire DA (Figure 8.1) but additional LIDAR was obtained and the entire DA is now captured within the revised model for this effort (Figure 8.2).

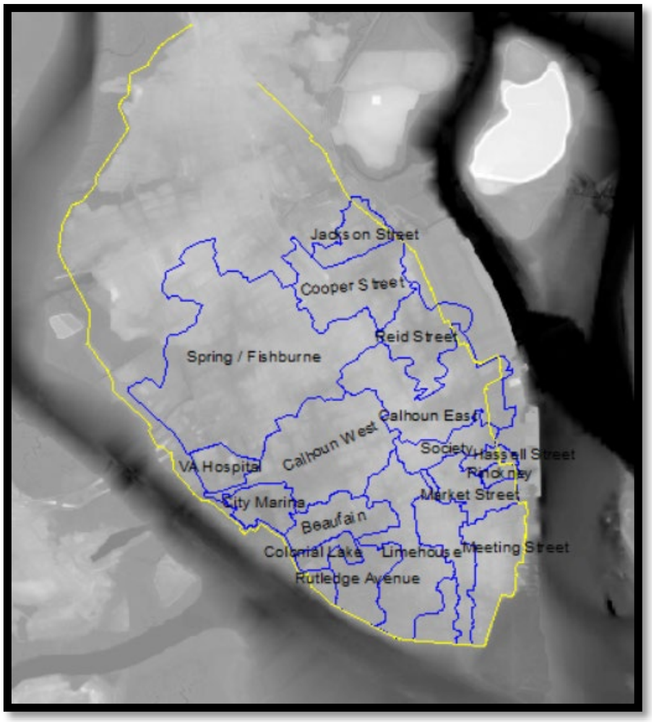


Figure 8.1 **Original Delineation found in the project folder GIS catalogue**

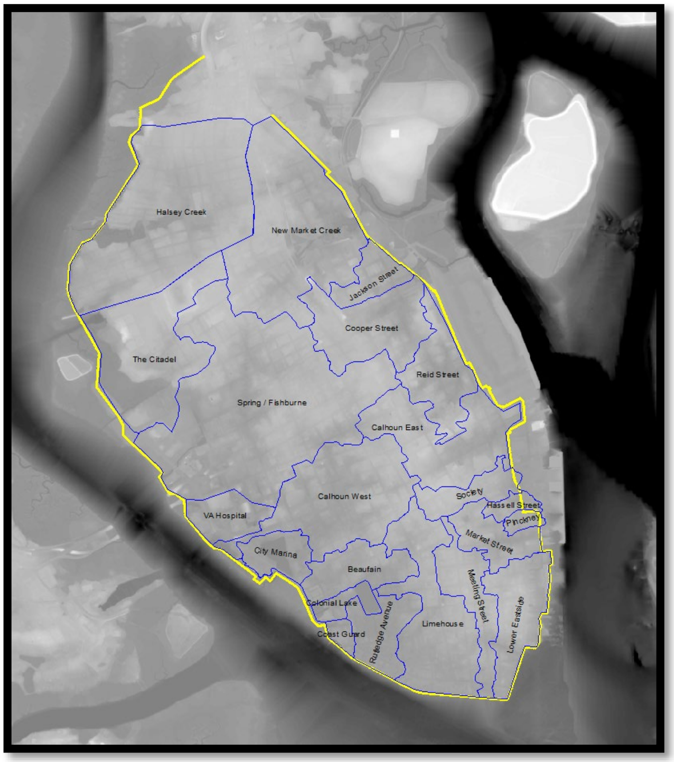


Figure 8.2 **Revised Delineation for the HEC-RAS Analysis**

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. The RAS model is used to observe and assess overland flow and the resulting water surface elevations at various locations around the pensinula for FWO and FWP conditions during different rainfall and tidal events. FWO condtions allow for overland flow back into the bounding rivers and FWP conditions will alter or prohibit the overland flow thus any increase in WSE would have to be mitigated with additional storage or pumps. This would result in an appropriate level of the change in flood elevations for the interior area.

8.1 INTRODUCTION

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 be performed observing the increase in the interior water levels which will give better designation of the types of 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 Probabilities (AEP) is being evaluated through the HEC-RAS model combined with different exterior tidal boundary conditions in a steady state.

8.1.1. 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% 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.

8.1.2. 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.

8.1.3. 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 8.1.3.1) 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 8.1.3.2) 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 8.1.3.3). 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 8.1.3.4). 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 to provide locations of the outfalls. 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.

LIDAR provided by the South Carolina Department of Natural Resources is being utilized in this study. The figures on the following pages display the LIDAR terrain that is being used. The LIDAR is characterized as a single band raster with a 10ft x 10ft resolution that was collected in 2008. The dataset originally wasn't large enough to capture the entire study area so the LIDAR was merged with the 2009 Charleston County raster data and tinned by the GIS team member to extract and "smooth" out the data at the merging boundary. The 2009 Charleston County raster provided terrain values into the

Ashley/Cooper Rivers and into the Charleston Harbor which the SCDNR LIDAR didn't capture. The LIDAR was resampled to 5ft x 5ft resolution when merged with the 2009 Charleston County raster.

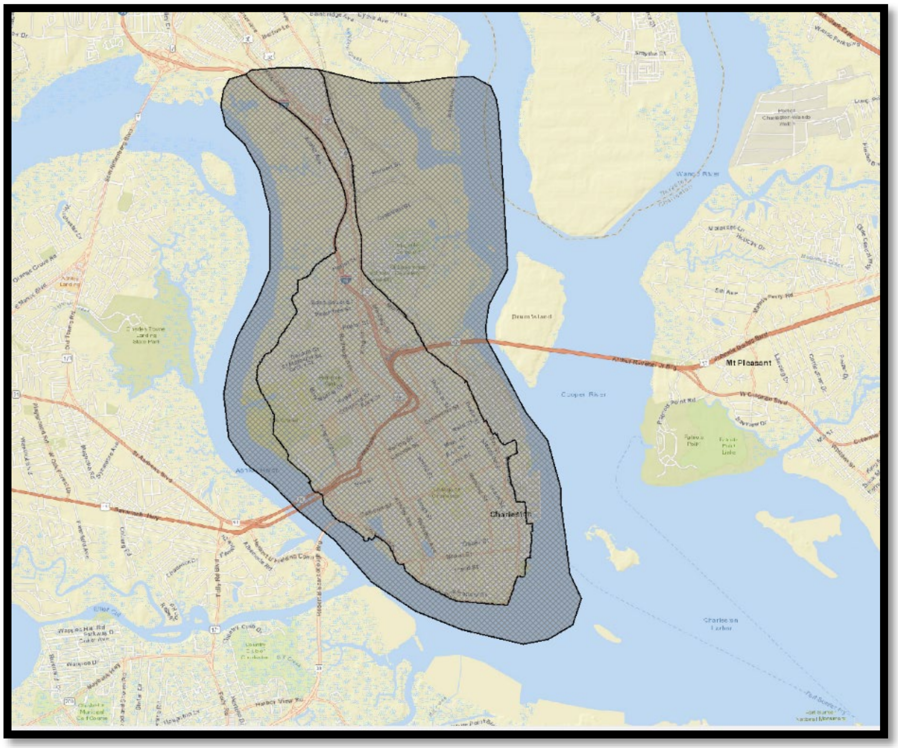


Figure 8.1.3.1. HEC-RAS 2D computational mesh

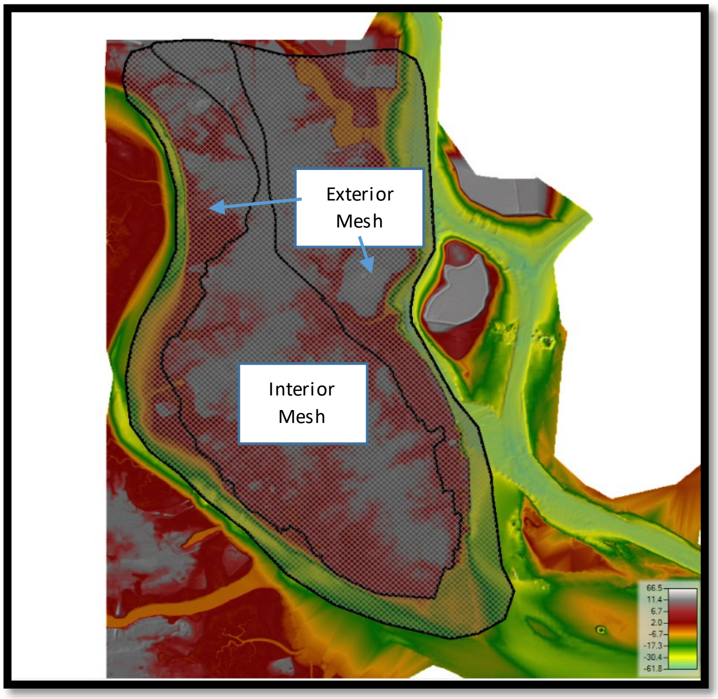


Figure 8.1.3.2. HEC-RAS 2D computational mesh and terrain (ft. NAVD88)

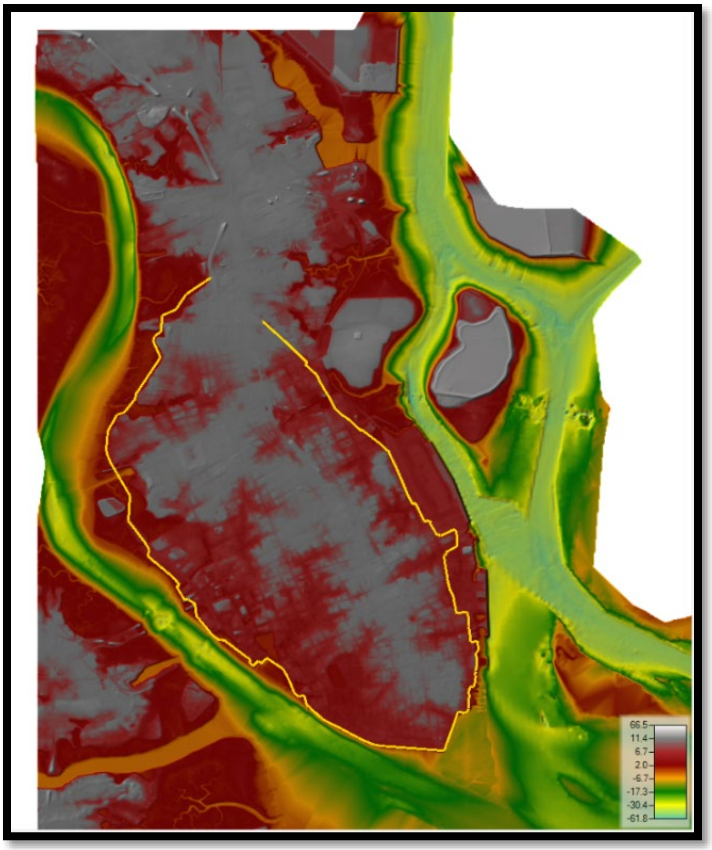


Figure 8.1.3.3. 12’ Wall Alignment

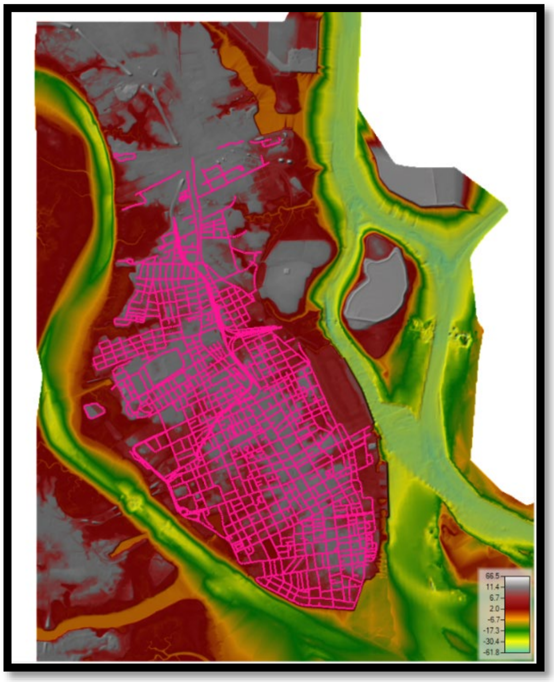
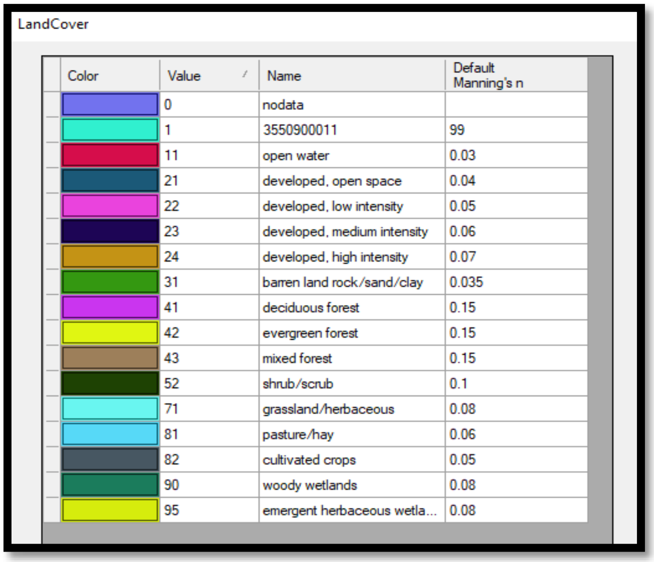


Figure 8.1.3.4. HEC-RAS Breaklines applied to 2D mesh

c. Manning’s n values applied to the HEC-RAS 2D Mesh

Figure 5 displays the Manning’s n values applied to the HEC-RAS 2D mesh. The 2011 National Land Cover is being used in this modeling effort. More information on this dataset is provided at http://www.mrlc.gov/. Manning’s n values were assigned to the various land coverage types.

The type of land displaying the Manning’s n value of 99 represents areas that are buildings. (Figure 8.1.3.5) A GIS shapefile layer of the buildings on the Charleston Peninsula was provided by the City of Charleston. This layer was merged with the Manning’s n layer in order to simulate the hydraulic effects of water penetrating a building and becoming stagnant with little to no velocity. The value of 99 represents an extremely high n value which will be successful in stagnating the flow that penetrates a building.



LandCover

Color	Value	Name	Default Manning's n
🟪	0	nodata	
🟦	1	3550900011	99
🟥	11	open water	0.03
🟦	21	developed, open space	0.04
🟪	22	developed, low intensity	0.05
⬛	23	developed, medium intensity	0.06
🟧	24	developed, high intensity	0.07
🟩	31	barren land rock/sand/clay	0.035
🟪	41	deciduous forest	0.15
🟨	42	evergreen forest	0.15
🟫	43	mixed forest	0.15
🟩	52	shrub/scrub	0.1
🟦	71	grassland/herbaceous	0.08
🟦	81	pasture/hay	0.06
⬛	82	cultivated crops	0.05
🟩	90	woody wetlands	0.08
🟨	95	emergent herbaceous wetla...	0.08

Figure 8.1.3.5. Manning’s n values applied to the HEC-RAS 2D model

d. 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 20% and 5 % 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. The rainfall data was developed as shown in the following Table 8.1.3.1.

Table 8.1.3.1. 24-HR Rainfall Time-Series Data

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

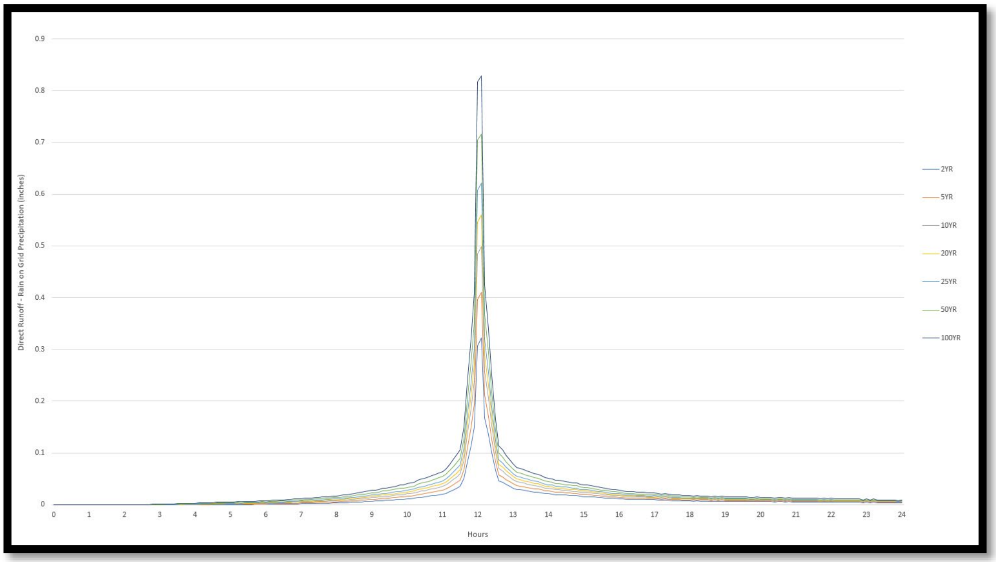


Figure 8.1.3.6. Direct Rainfall Runoff Time-Series Data for the 50%-1% AEP Return Periods over a 24-hr period

8.1.4. 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.

The existing conditions scenario was computed using verified water levels produced by Hurricane Irma on September 11, 2017. These water levels were extracted from NOAA Tides & Currents webpage from the Charleston, Cooper River Entrance SC gage. The Station ID is listed as 8665530.

Figure 8.1.4.1 displays the water levels during Hurricane Irma in NAVD88 and Figure 8.1.4.2 shows the inundation due to storm surge (not rainfall).

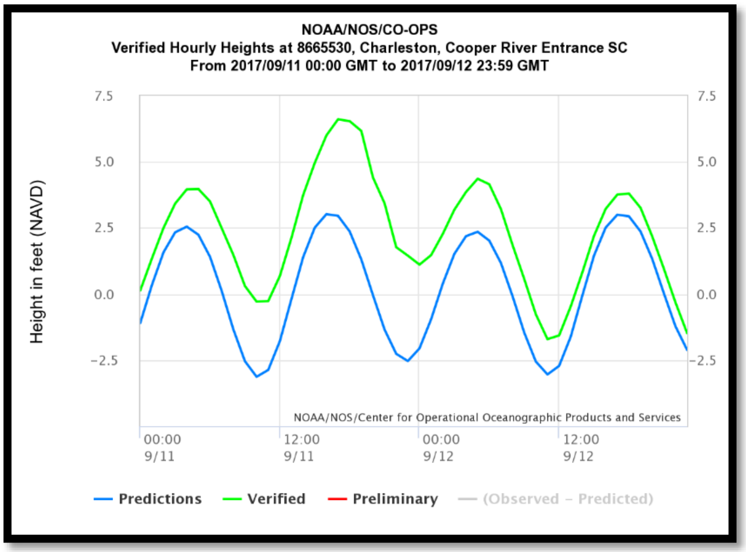


Figure 8.1.4.1. Charleston, Cooper River Entrance SC Tidal Gage (Hurricane Irma)

The conversions to NAVD88 are as followed:

- Highest Water Level (+9.39) (Hurricane Hugo September1989)
- MHHW (+2.63)
- MLLW (-3.14)
- Lowest Water Level (–7.22) (Storm of the Century March 1993)

The existing conditions scenario displays many areas being inundated along the east and west side of the perimeter of the peninsula. The low side or west side of the Battery was overtopped by storm surge during this event. The Low Battery near the U.S. Coast Guard property is flanked by flooding well before the wall Low Battery overtops. Precipitation data was not run through the model for the existing conditions.

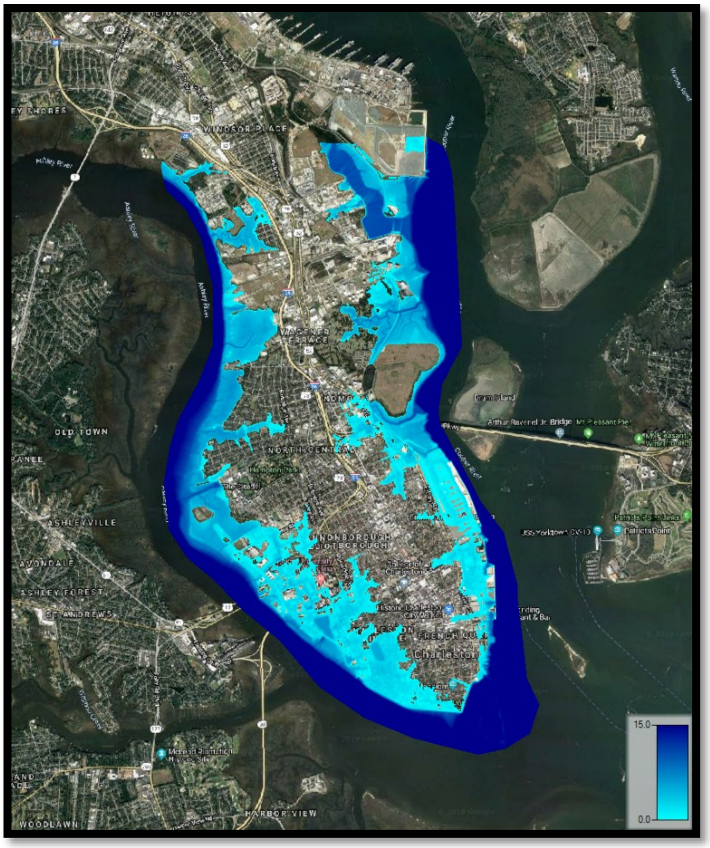


Figure 8.1.4.2. Existing Conditions Inundation (Hurricane Irma)

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% AEPr 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% 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 8.1.4.3 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 8.1.4.4 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.

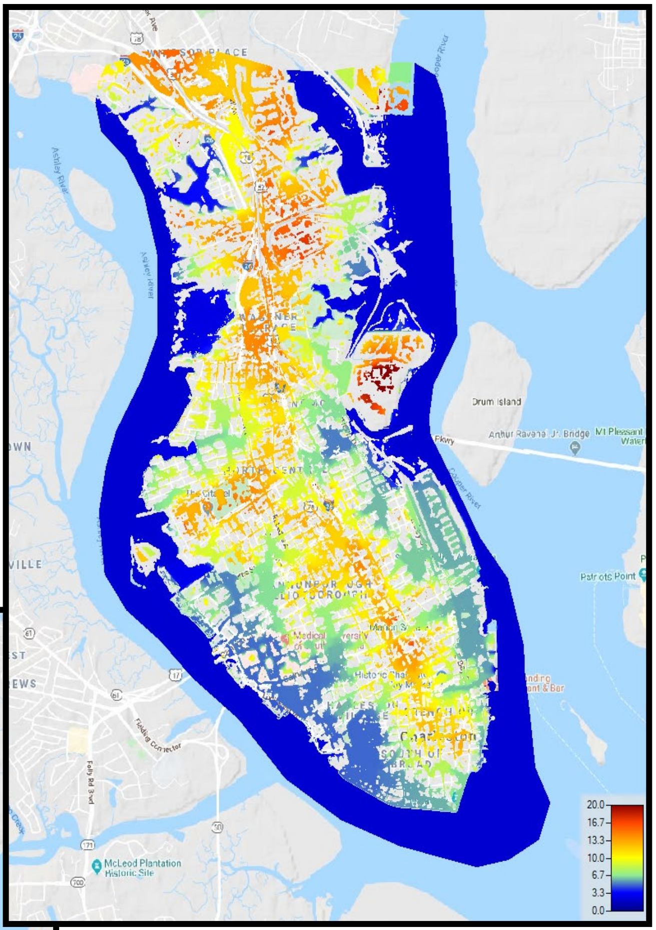


Figure 8.1.4.3. 10% AEP FWO Peak

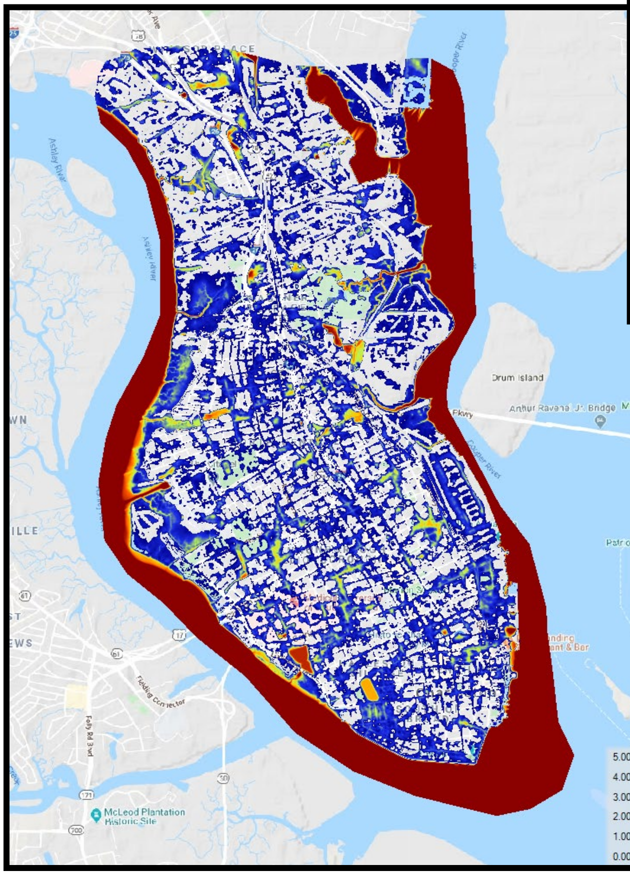


Figure 8.1.4.4. 10% AEP FWO Peak

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 8.1.4.5. 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 8.1.4.5 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

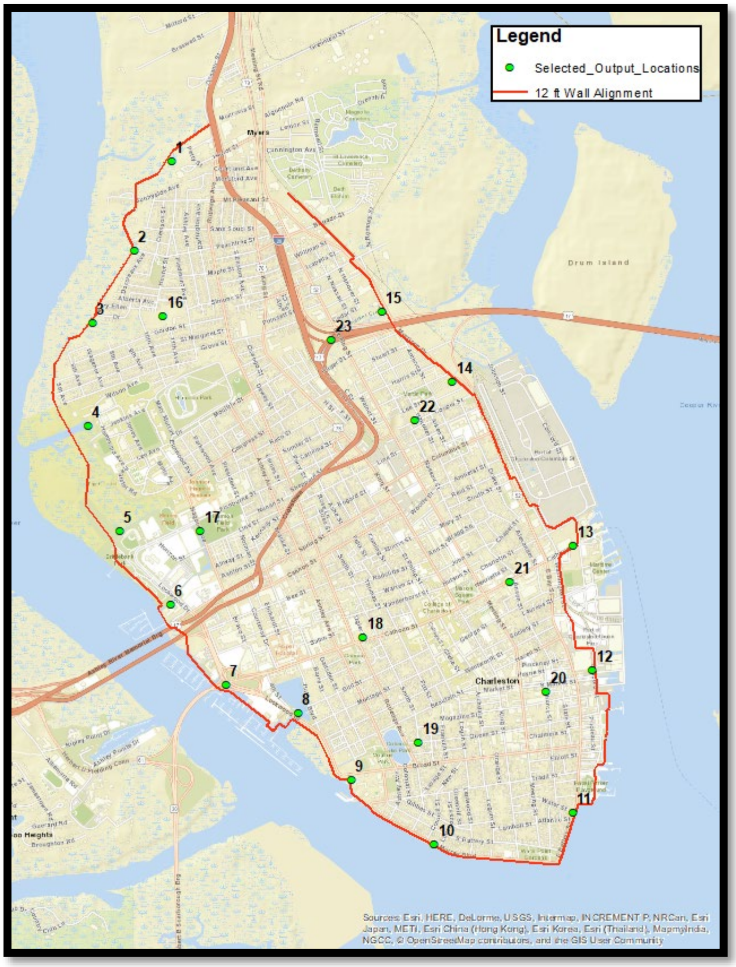


Figure 8.1.4.5. Selected Output Locations for RAS model results

Table 8.1.4.1 and Table 8.1.4.2 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 8.1.4.1 20% Annual Exceedance Comparison

20% AEP	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 8.1.4.2 10% Annual Exceedance Comparison

10% AEP	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

8.1.5. Results by RAS Model Screenshots

The 12’ (NAVD88) wall is placed internally (on-land) on the east side of the peninsula so much of the area near the Port would be inundated during a 6’ (NAVD88) tide event. The National Weather Service indicates major flooding at 8’ (MLLW) which equals 4.86’ (NAVD88). Adding SLR of 1.13’ equates to 5.99’ (NAVD88). So, a future with-project condition considering the 12’ (NAVD88) wall shows much of the Port being inundated at such a water level. Precipitation was not input into this scenario. This scenarios is provided to show the flooding on the east side of the city outside of the wall due to a 6’ (NAVD88) water surface elevation.

Figure 8.1.5.1. 12' NAVD88 wall versus a 6' NAVD88 exterior WSE with no rainfall

Note: This was an additional scenario simulation for the future with-project condition to observe inundation on the east side of the peninsula near the port which is outside the wall.

ASSUMPTIONS AND LIMITATIONS

- Vertical Datum used for modeling – NAVD88
- Sea Level Rise – 1.13 feet from present day or 1.25 feet from 1992.
- The City of Charleston will raise the Low Battery to match the High Battery. This elevation is assumed to be 9 feet NAVD88 which is approximately the mean elevation of the higher side of the wall currently.
- Storm surge is the primary focus of the study while rainfall is used as a mitigation measure for the conceptual design and placement of drainage structures and pumping stations. Overtopping of the wall

will also serve as a measure of drainage and pump sizing. Separating storm surge from rainfall when assessing rainfall flooding as a mitigation measure.

- The assessment of the wall overtopping will be deferred to the PED phase. A scenario where the wall overtops would mean the city would be flooded for a scenario without a wall, therefore flooding would not be increased due to the project being proposed during the feasibility phase.
- The HEC-RAS model was computed with a known flood event, but there is little to no gage data available on land or on the interior to calibrate or validate the model. This leaves uncertainty in the modeled water levels. However, the RAS model will provide a relative comparison between with project and without project alternatives. 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.
- Rainfall data was provided by the City of Charleston contractor that was contracted for the RAS modeling of the Calhoun West pumping station. The precipitation time-series data is applied uniformly across the entire 2D mesh. In reality, the precipitation will vary spatially across the entire model area. There can be uncertainty with precipitation data in terms of the volume.
- Peak water levels for FWO and FWP are provided at 23 sample locations and provide a general sense of the overall impacts at various locations. Some of these locations are just on the interior of the proposed 12’ wall footprint to assess the increase in interior rainfall flooding due to the wall. Some of these locations are at higher elevations near the central area of the peninsula and show increase in water surface elevations.
- The inability of HEC-RAS to model subsurface drainage. The losses due to urban storm water drainage systems cannot be accounted for within RAS.
- Assumption that the City of Charleston will complete the peninsula outfall check valve program which will prevent tidal backflows into the storm drainage outfalls. Tide will not be able to back up into the drainage system.

8.1.6 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…) using HEC-RAS 5.1.
- Select option and include as part of plan

Future Considerations for PED Phase HEC-RAS Modeling

Based on the present assumption of a pump solution to address residual damages:

- The HEC-RAS modeling will provide information needed by the PDT to make an informed decision on the total pumping capacity and drainage system. Once the pumps and gates are implemented, the RAS simulations will estimate the hydraulic response of the overall system for various pumping and storm gates alternatives.

- The pump stations will most likely be modeled to deliver the required flow extraction against a constant exterior water surface elevation. In reality the head will not be constant over a storm.
- What is the design level for pumps?? How fast can the city storm water drainage system route interior drainage to the pumps? What is the limit of the storm water drainage system?
- Refinement of the Analysis of the wall overtopping during PED phase.
- Analysis of storm gates opening and closing during PED phase. Alternative sizing of gates to compare within HEC-RAS simulations during PED phase.
- Compound flooding and timing versus pump stations and storm gates during PED phase. How to couple rainfall and storm surge for analyzing operation of the pumps and gates.
- Assess the gates at different alternative sizes to drain at low tide and pumps at different capacities to pump at a max tide or design event.

CHAPTER 9 – GATE CLOSURE ANALYSIS

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 9.1 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.

CHAPTER 10 – REFERENCES

o **ER 1105-2-100** (Appendix K): *Planning Guidance Notebook* (April 2000).
o **ER 1100-2-8162**: *Incorporating Sea Level Changes in Civil Works Programs* (December 2013).
o **ECB 2016-5**: *Using Non-NOAA Tide Gauge Records for Computing Relative Sea Level Change* (Jan 2016).
o **ETL 1100-2-1**: *Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation* (June 2014).
o **ECB 2018-3**: *Using Non-NOAA Tide Gauge Records for Computing Relative Sea Level Change* (Feb 2018).

Additional important guidance is provided within the following documents:
o **ER 1110-2-8160**: *Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums* (March 2009).
o **EM 1110-2-6056**: *Standards and Procedures for Referencing Project Elevation Grades to Nationwide Vertical Datums* (December 2010).
o **ER 1110-2-8159**: *Life Cycle Design and Performance* (October 1997).
o **ER 1110-2-1150**: *Engineering and Design for Civil Works Projects* ((August 1999).
o **ECB 2018-2:** *Implementation of Resilience Principles in the Engineering & Construction Community of Practice* (Jan 2018).
o **EP 1100-1-3:** *USACE Sustainability: Definition and Concepts Guide* (July 2018).
o EM 1110-2-1413 Hydrologic Analysis of Interior Areas

Webpage

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