FAIRFIELD AND NEW HAVEN COUNTIES, CT
COASTAL STORM RISK MANAGEMENT
FEASIBILITY STUDY

APPENDIX C
COASTAL ENGINEERING

October 2020

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1.0 Study Overview

The Fairfield and New Haven Counties, Connecticut Coastal Storm Risk Management Study investigated the feasibility of various storm damage reduction measures along the southern coast of Connecticut within Fairfield and New Haven Counties from the New York border to Hammonasset Point in Madison, CT. The coastline is approximately 60 miles long when measured in a straight line. The actual mileage of coastline due to curved shorelines, headlands and embayments is much greater. The location of the study area can be seen in Figure 1-1. Within Fairfield County, the study area included the towns of Greenwich, Darien, Norwalk, Westport, Fairfield, and Stratford and the cities of Bridgeport and Stamford. Within New Haven County, the study area comprised the towns of East Haven, Branford, Guilford, and Madison and the cities of Milford, West Haven, and New Haven.

Of these municipalities, five primary damage areas (the towns of Stratford and Fairfield, and the cities of Milford, New Haven, and West Haven) were initially identified for assessment by the Regional Councils of Government in Connecticut. However, the Town of Fairfield and the City of New Haven areas were ultimately selected for further evaluation based on their potential to support a federally-constructed project. During the course of the study, alternative coastal storm risk management solutions were developed for both the Town of Fairfield and the City of New Haven. However, the alternatives developed for the Town of Fairfield required substantial costs for construction and real estate requirements that the Town was unable to commit to. Therefore, this report focuses solely on the New Haven study area, shown in Figure 1-2.

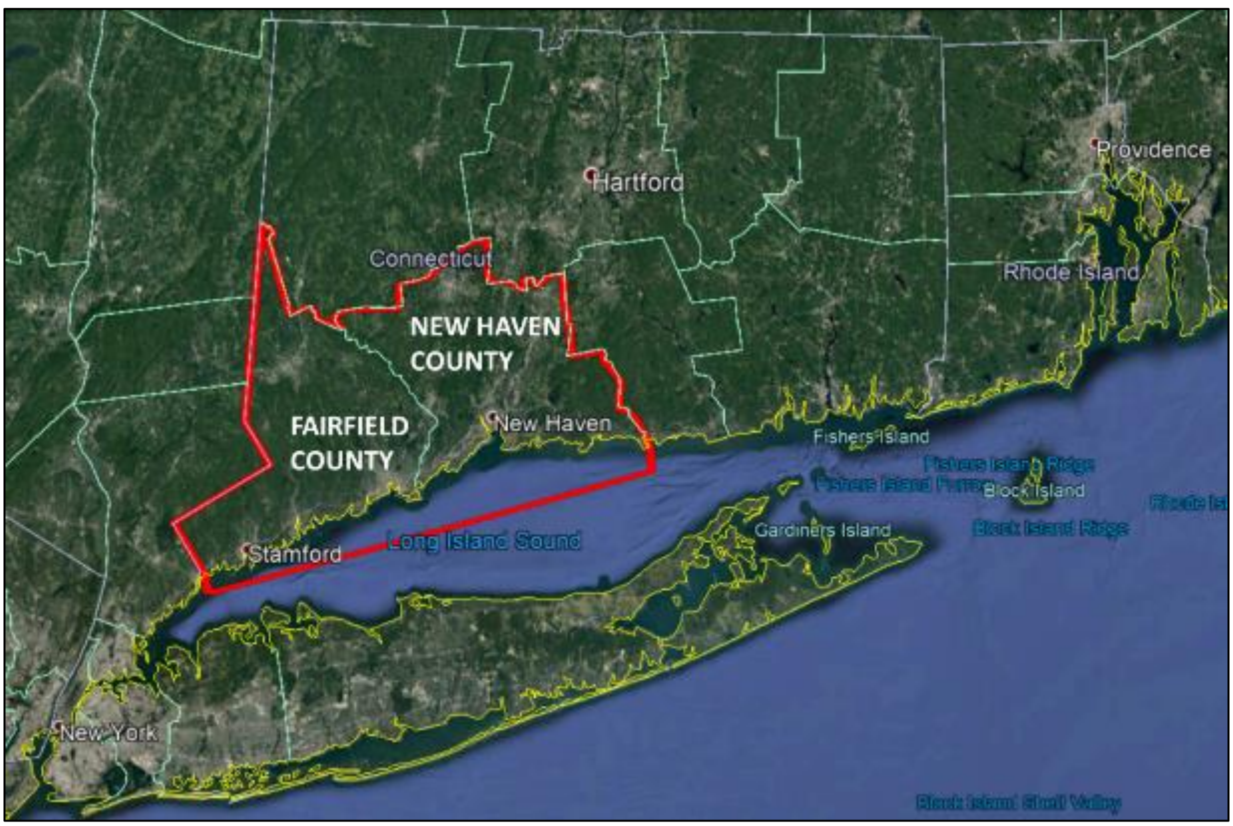

Figure 1-1. Study Area Map

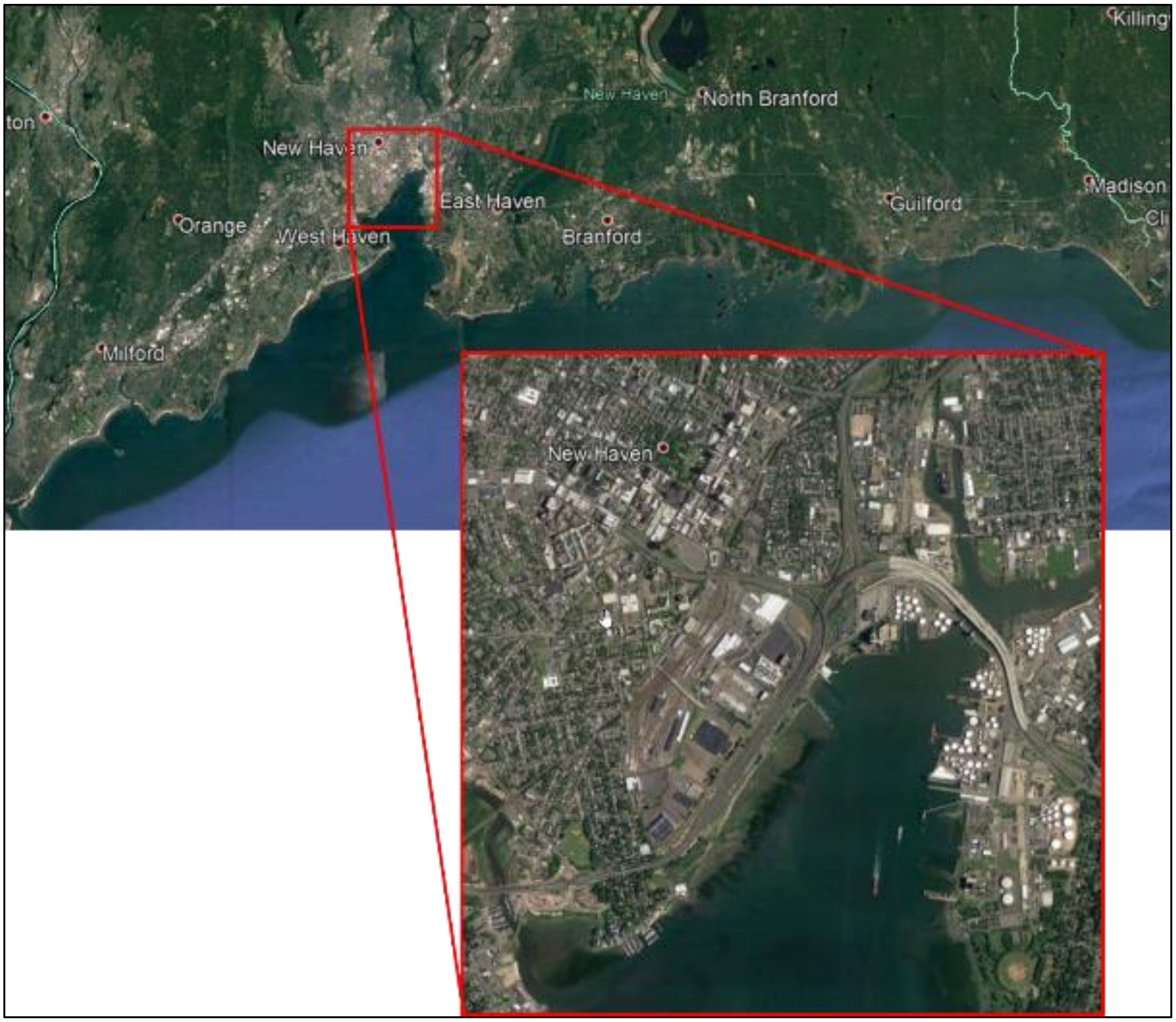

Figure 1-2. New Haven Study Area

Within the study area various forms of storm damage reduction were considered that included structural alternatives such as sea walls, flood walls, and surge barriers/gates along with non-structural alternatives such as floodproofing.

1.1 Study Area

New Haven is located in south central New Haven County on New Haven Harbor on the northern shore of Long Island Sound, approximately 75 miles northeast of metropolitan New York City. New Haven's best-known geographic features are its large, shallow harbor, and two reddish basalt trap rock ridges which rise to the northeast and northwest of the city core. The city is drained by three rivers—the West, Mill, and Quinnipiac, named in order from west to east. The West River discharges into New Haven Harbor on the west shore of the harbor, southwest of the city whereas the Mill and Quinnipiac Rivers discharge at the head of the harbor. New Haven is the second-most populous city in Connecticut after Bridgeport as well as a hub for transportation and industry.

The Long Wharf area, in particular, has been identified as area that is highly vulnerable to coastal flooding given its proximity to the harbor and relatively low elevation. This area, delineated in Figure 1-3, is an approximately 350-acre commercial and industrial district with several well-known companies (e.g., IKEA, Assa Abloy) as well as the Regional Water Authority, the New Haven Food Terminal, and the Long Wharf Maritime Center. The Long Wharf District also includes Union Station and the Connecticut Department of Transportation's (ConnDOT) largest railyard as well as Interstate Route 95. The Long Wharf shoreline also includes the Canal Dock Boathouse, the Long Wharf Nature Preserve and the Veteran's Memorial Park, a valuable cultural, recreational and ecological asset that provides the city with scenic views of New Haven Harbor. Aside from two highway underpasses, Route 95 bisects the Long Wharf District, separating much of the industrial and commercial structures from the shorefront park.

Figure 1-3. Long Wharf Study Area

Within the study area, ground elevations typically range between 5 to 10 feet NAVD88, with the exception of I-95. Most of I-95 within the project area is built on an earthen embankment. Elevations along I-95 range from 10 to 30 feet NAVD88, with lower ground elevations in the vicinity of the Church Street/Sargent Drive intersection and at the Hudson Street Bridge. Higher elevations exist closer to the northern and southern ends of the study area.

The land in the study area is an artificial feature, created from dredge and fill material placed along the west shore of New Haven Harbor during the mid-twentieth century. Much of the Long Wharf shoreline is developed with structures including quarry stone revetment, steel sheetpile bulkheads, piers, and seawalls. Other areas include an estuarine beach and tidal flat, sheltered wetlands and low marsh.

The northernmost shoreline of the study area is dominated by sheetpile and stone bulkheads and revetments. The shoreline to the south of Long Wharf Pier, along Long Wharf Park, widens, becomes less steep, and is more vegetated and natural than the shoreline areas to the north.

The Long Wharf Park area is generally flat, and is characterized by small changes in elevation. The mild sloping tidal flats significantly attenuate wave heights during normal tide conditions. Shoreline types and landcover for the study area are shown in Figure 1-4. In addition, there are seven stormwater outfalls within the study area, each fitted with tide gates. The locations of all utilities will be confirmed in Pre-Construction Engineering & Design (PED).

The vulnerability of Long Wharf to coastal flooding has been demonstrated in recent years by Hurricanes Irene and Sandy in August 2011 and October 2012, respectively. Hurricane Irene brought approximately 4.7 feet of storm surge to New Haven at high tide, bringing water levels to almost 8.0 feet NAVD88. Hurricane Sandy, which made landfall approximately 150 miles southwest along the New Jersey coast, brought 9.1 feet of storm surge to New Haven. Because peak storm surges occurred below mean tide level, water levels peaked at elevation 8.6 feet NAVD88, but were enough to cause 1 to 2 feet of inundation in the Long Wharf area. Erosion of the Long Wharf shoreline was an issue during both events. While Hurricanes Irene and Sandy were impactful, there is certainly potential for greater flood risk, especially future sea level rise is considered.

1.2 Coastal Engineering Scope of Work

Supporting the study, coastal analysis and engineering work was completed and provided to the Project Delivery Team (PDT). The information within this appendix describes this work and the information provided. As part of the Corps' SMART Planning process, earlier alternative screening was completed which limited and focused the level of analysis associated with the project. As part of the reduced level of analysis, an effort was made to use existing information where it remained applicable. This work focused on providing annual recurrence interval water levels within the study area for the design and evaluation of project alternatives, as well as hydrodynamic loads on proposed structural measures. These analyses are detailed in this report.

Figure 1-4. Shoreline types and landcover classifications (GZA)

1.3 Past Studies

1.3.1 North Atlantic Coast Comprehensive Study

The North Atlantic Coast Comprehensive Study (NACCS) (2015) report detailed the results of a two-year study by the U.S. Army Corps of Engineers which addressed coastal storm and flood risk to vulnerable populations, property, ecosystems, and infrastructure affected by Hurricane Sandy in the United States' North Atlantic Region. The purpose of the study was to identify flood risk and then plan and implement strategies to reduce the risk now and in the future. The study also determined the magnitude and uncertainty of existing and future forcing conditions. The study's conclusions included a recommendation to use its findings to assess coastal engineering projects for coastal storm risk management and resiliency for the areas in the region from Virginia to Maine.

The NACCS identified the New Haven shoreline as an area of high exposure that is densely populated and developed and would be subject to very significant damage if a Hurricane Sandy-like event were to hit. Within a reach beginning on the east side of New Haven Harbor at Morris Cove and terminating at Prospect Beach in West Haven, the NACCS identified several thousand residential, commercial, industrial, and municipal structures. The study also noted that New Haven Harbor is surrounded with many petroleum and bulk cargo based industries that rely heavily on the port for moving those products. In addition to many important rail lines, the area includes two major interstate highways, Routes 95 and 91, that are critical to the region for moving traffic.

In addition to the vulnerability and risk assessment components of the study, the NACCS included high-fidelity coastal numerical modeling of coastal hazards for the North Atlantic coast region. Storm surge and wave modeling results from these efforts in the New Haven area were used in this study and are discussed further in Section 3.1.

1.3.2 GZA Long Wharf Flood Protection Study

The Long Wharf Flood Protection Study was conducted by GZA Geo Environmental, Inc. (2017) under contract to the City of New Haven. The purpose of the study was to characterize Long Wharf's coastal flood hazards, evaluate the area's flood vulnerability, and identify and evaluate alternatives that would mitigate the coastal flood risk.

As part of their work, GZA performed flood simulations using numerical hydrodynamic models of tides and storm surge as well as wave models. These modeling efforts and their results will be discussed further in Section 3.2.

2.0 Coastal Climatology and Setting

Based on data from the North Atlantic Coast Comprehensive Study which will be discussed in Sections 2.4 and 3.1, significant tropical storm events impact this region of shoreline at a frequency of approximately once every 5.75 years. These tropical storms occur between June and November with 74 percent of them occurring in the months of August and September. Extratropical storms, known as Nor'easters, are a frequently occurring storm type that impacts this region annually with significant events occurring at a rate of approximately 0.96 storms per year. Extratropical storms typically occur at the project location between early fall through spring (October through March).

Tropical storm events are typically fast moving storms associated with elevated water levels and large waves whereas extratropical storms are slower moving with comparatively lower water level elevations but large wave conditions that can equal tropical storm events. Both storm types can cause coastal inundation leading to economic losses to improved property within the study area. In addition to storm events, locally generated persistent southerly breezes can generate significant wind setup across Long Island Sound.

2.1 Tidal Regime

New Haven Harbor experiences semi-diurnal tides (two low and two high tides per day) with one high and low tide typically of greater magnitude than the other due to a slight diurnal shift. NOAA installed a tide gage (Station 8465705) in August of 1999. The mean tide range in the Harbor is 6.2 feet and the diurnal range is 6.7 feet. The tides, which are created by the gravitational pull of the moon, the sun, and the earth's rotations are responsible for most of the water levels observed. Occasionally, abnormally high or low water levels occur as a result of changes in atmospheric pressure, storm surge, the magnitude and direction of wind and/or waves, and other meteorological anomalies. Table 2-1 provides the tidal datums for New Haven at Station 8465705. In New Haven the highest water level observed was 12.24 feet MLLW (8.62 feet NAVD88), which was during Hurricane Sandy on October 30, 2012.

Table 2-1. New Haven Harbor Tide Range – NOAA Station 8465705

Condition	Elevation (feet, MLLW)	Elevation (feet, NAVD88*)
Mean Higher High Water (MHHW)	6.71	3.09
Mean High Water (MHW)	6.39	2.77
NAVD88	3.62	0.00
Mean Sea Level (MSL)	3.32	-0.30
Mean Tide Level (MTL)	3.32	-0.30
Mean Low Water (MLW)	0.24	-3.38
Mean Lower Low Water	0.00	-3.62

***North American Vertical Datum of 1988**

2.2 Sediment Transport and Shoreline Change

A number of small rivers empty into New Haven Harbor, including the Mill, Quinnipiac, and West Rivers, and Morris Creek, which contribute silty shoal material to the harbor. Aside from the federal navigation channel, New Haven Harbor is generally shallow and at low tide there are large expanses of mud flats seaward of the Long Wharf area. The shoreline north of Long Wharf Pier is developed, dominated by sheetpile and stone bulkheads and revetments whereas the shoreline to the south is composed of a variety of shore types including sandy beach, tidal wetlands, and rock revetment.

GZA's Long Wharf study noted that natural processes have resulted in accretion of sandy beach and regularly flooded marsh along more natural shorelines. Shorelines protected by seawalls and bulkheads are stabilized by these structures. Shorelines hardened by quarry stone revetments are in a range of conditions, from good to severely damaged. Areas upland of the poor condition (or absent) revetment sections have experienced storm-related scour and erosion. The GZA study noted that future damage of the shoreline due to storm surge and waves is likely. For this reason, the City of New Haven is in the process of designing a living shoreline to be constructed along Long Wharf Park with grant funding from CT DEEP. The 3,600 linear foot project seeks to enhance the shoreline and nearshore environment and to improve resiliency to sea level rise and storm surge. The living shoreline is set to contain 8 acres of new tidal wetlands behind a rock sill, as well as 2 acres of new beach through the placement of sand between the new wetlands and the existing revetment. The living shoreline will act to reduce wave energy and scouring of the shoreline while providing enhanced natural resource and recreational value. As this project is funded for construction, the erosion protection it is planned to provide is included in the future conditions. For this reason, erosion of the Long Wharf shoreline was not a focus for this feasibility study.

2.3 Wind

Coastal wind data is collected in the vicinity of New Haven at Station NWHC3-8465705 within New Haven Harbor. All wind speeds were converted to knots at 10m equivalent height. New Haven Harbor is more sheltered than Long Island Sound, with an average wind speed of 7.8 knots. Wind speed magnitude and direction generally vary with season within the harbor. Winter winds average 8.7 knots from the North. In the summer, winds are lighter at 6.6 knots from the Southwest. Seasonal wind characteristics are presented as wind roses in Figure 2-1.

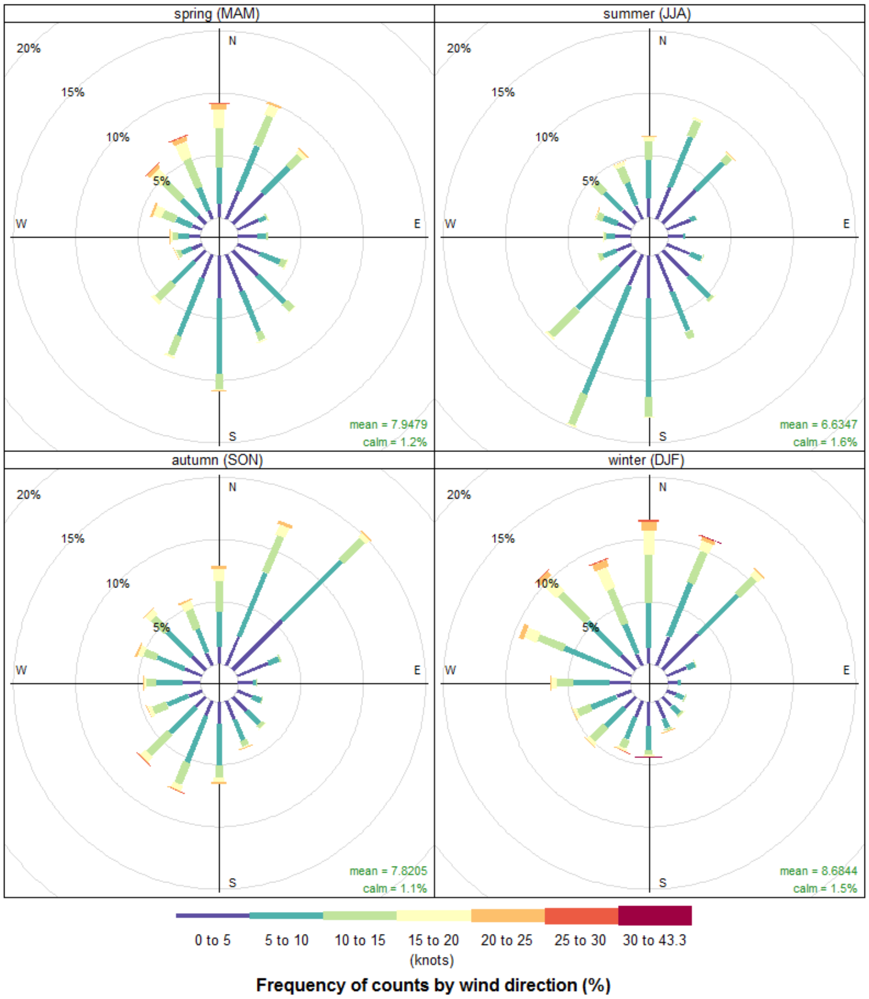

Figure 2-1. Station NWHC3-8465705, New Haven Harbor Seasonal Wind Roses

2.4 Annual Recurrence Probability (Storm Frequency) Information

The intent of this project is storm damage reduction and, as such, a vital piece of information for this study is the annual exceedance probability of storm conditions within the project area. Annual exceedance probability is the percent chance that an area experiences a particular level of storm conditions or greater in a given year. Often a key recurrence interval due to FEMA flood insurance requirements is the 1-percent annual chance storm. This is a storm water level that an area has a 1-percent chance of experiencing each year and every year. The 1-percent annual chance storm is often referred to as the 100-year storm. However, the representation of annual chance or annual exceedance probability is preferred since it more accurately describes the chances of an area experiencing such an event. Often people make the mistake that a 100-year storm only occurs once per 100 years and that once it occurs it will not happen again for 100 years. That is completely incorrect and as stated that level of storm or greater has a 1-percent chance of occurring each and every year, even if it had just occurred the year prior. Taking this concept further, there is a chance that multiple storms of this strength or greater will occur in the same year. This is all represented in the probabilities (percent chance of occurrence). Another way to consider the 1-percent annual chance storm is that during a 30 year period (length of a typical mortgage) a property in the 1-percent annual chance floodplain has at least a 26% chance of experiencing the 1-percent annual chance storm. That is fairly significant if one considers they have a 1 in 4 chance of experiencing an event during the life of their mortgage.

For studies such as this an understanding of probability of storm exposure is needed beyond the 1-percent annual chance storm since many properties and pieces of infrastructure are impacted by storms that occur more frequently and less frequently. To help frame the exposure along the study area, results from some of that work will be provided here as well as comparisons to actual recorded water levels along the Connecticut coast.

Often it is mistakenly concluded that tropical based storm systems do not regularly impact the Connecticut coast. As shown in Figure 2-2, based on the historical tracks of tropical based systems between 1851 and 2018 (167 years), 45 tropical systems have come within 75 miles of New Haven. That is an average of one storm every four years which is similar to the frequency found in the NACCS modeling study. To help quantify the level of storm exposure along the coast, mean annual exceedance probability water levels for the study area from the NACCS are provided in Table 2-2. To put these water levels in context, the annual exceedance probabilities associated with the peak water levels recorded during Hurricanes Sandy and Irene are shown relative to these NACCS values in Figure 2-3. Hurricane Sandy, which certainly caused significant damage along the coast of Connecticut and was the impetus for performing this study, was slightly less than an 8% percent annual exceedance probability storm. In other words the study area has approximately a 1 in 12 chance each year of experiencing a Sandy level event.

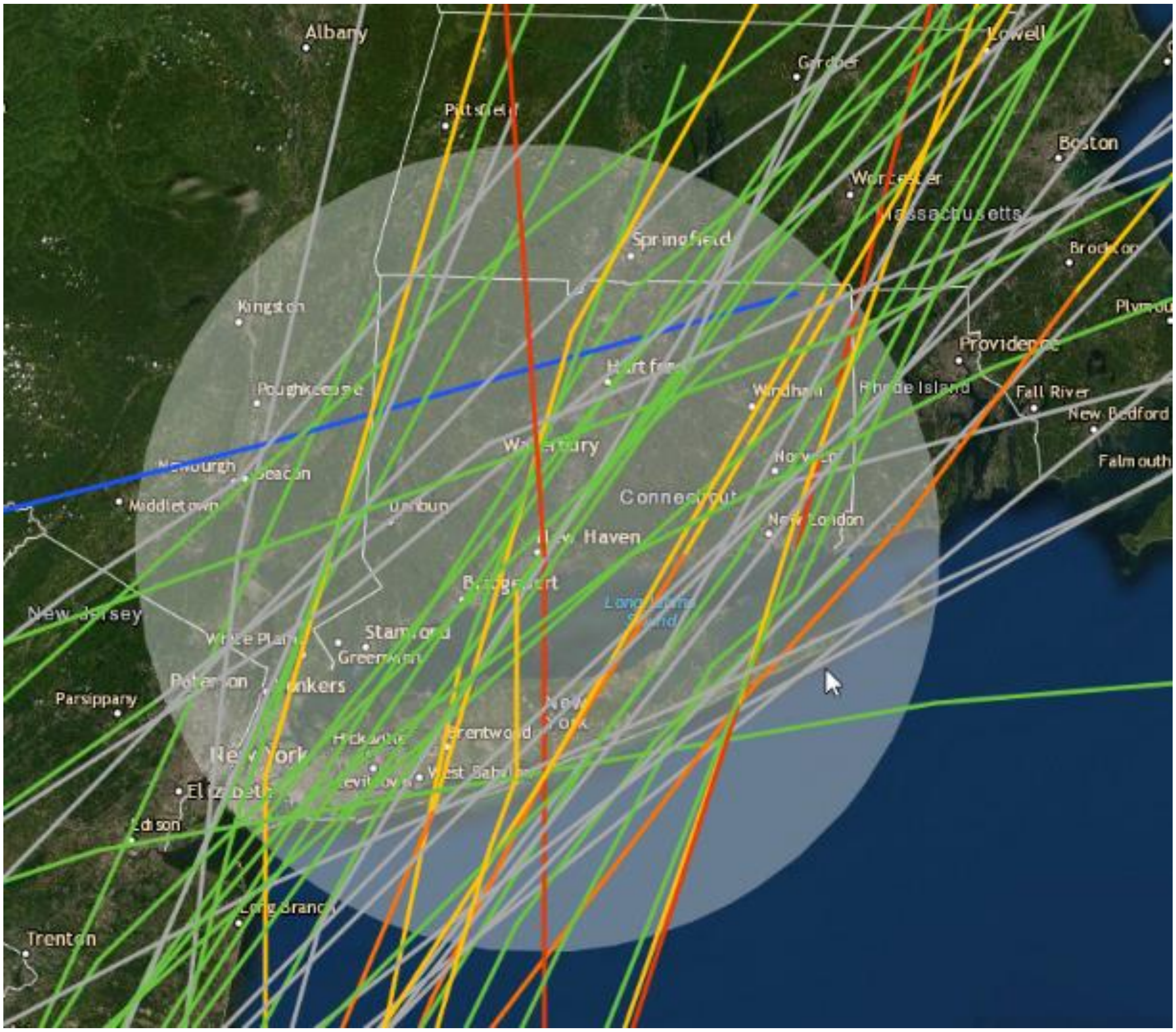

Figure 2-2. Tropical storm system paths from 1851 to 2018

Table 2-2. Mean Annual Exceedance Probability Water Levels for Long Wharf, New Haven

NACCS Save Point 8134	**Mean Annual Exceedance Probability**								
	99%	50%	20%	10%	5%	2%	1%	0.5%	0.2%
Water Levels (Feet, NAVD88)	5.35	6.26	7.46	8.33	9.20	10.46	11.65	13.10	15.10

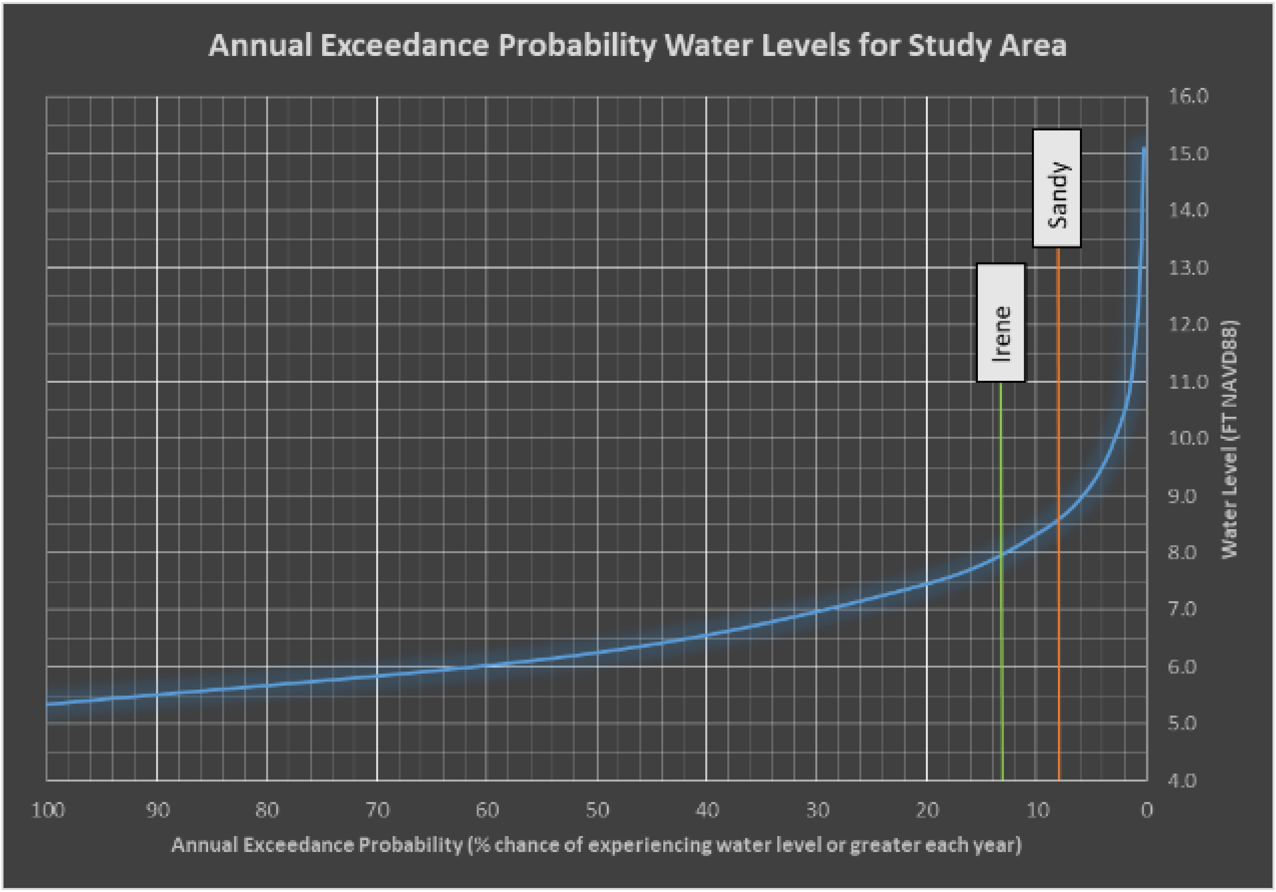

Figure 2-3. Mean annual exceedance probability water levels and recent storms of note

3.0 Water Levels and Wave Conditions (Storm Parameters)

As discussed in Sections 1 and 2, the study area is impacted by both tropical and extratropical storm systems, with the tropical systems generally being the most impactful due to the higher storm surges and total water levels associated with those systems. The frequencies of storm-based water levels was described in Section 2 which places the study area's storm exposure in context. That information was produced from the North Atlantic Coast Comprehensive Study (NACCS), which will be discussed further below in Section 3.1. Additional water levels used for plan formulation will be described in Section 3.2.

3.1 North Atlantic Coast Comprehensive Study (NACCS)

Water levels and wave heights were needed as input for the various types of coastal engineering and planning analyses performed in the study. The North Atlantic Coast Comprehensive Study (NACCS) was used as the primary source of water level information. The NACCS characterized the probabilistic tropical and extratropical storm climatology of the coastal areas defined by the extent of Hurricane Sandy's storm surge. This work, carried out by the Engineer Research and Development Center (ERDC) included rigorous regional statistical analysis and detailed high-fidelity numerical hydrodynamic modeling for the North Atlantic region to quantify coastal storm wave, wind, and storm-driven water level extremes. The NACCS modeling efforts included the latest atmospheric, wave, and storm surge modeling and extremal statistical analysis techniques. Products from this work were incorporated into the Coastal Hazards System (CHS) database, a data storage and mining system web tool, and include simulated winds, waves, and water levels for approximately 1,050 synthetic tropical events and 100 historical extratropical events computed at over 3 million computational locations. These storms span the range of practical storm probabilities for the region. For a detailed description of this modeling and the results, the reader is referred to the following USACE documents—"Coastal Storm Hazards from Virginia to Maine 2015" and "North Atlantic Coast Comprehensive Study (NACCS) Coastal Storm Model Simulations: Waves and Water Levels 2015"—which can be found at http://www.nad.usace.army.mil/CompStudy/. The CHS contains output at approximately 19,000 save points or data access points within the NACCS study area from Virginia to Maine. An example image of the save points that are provided in CHS is provided in Figure 3-1.

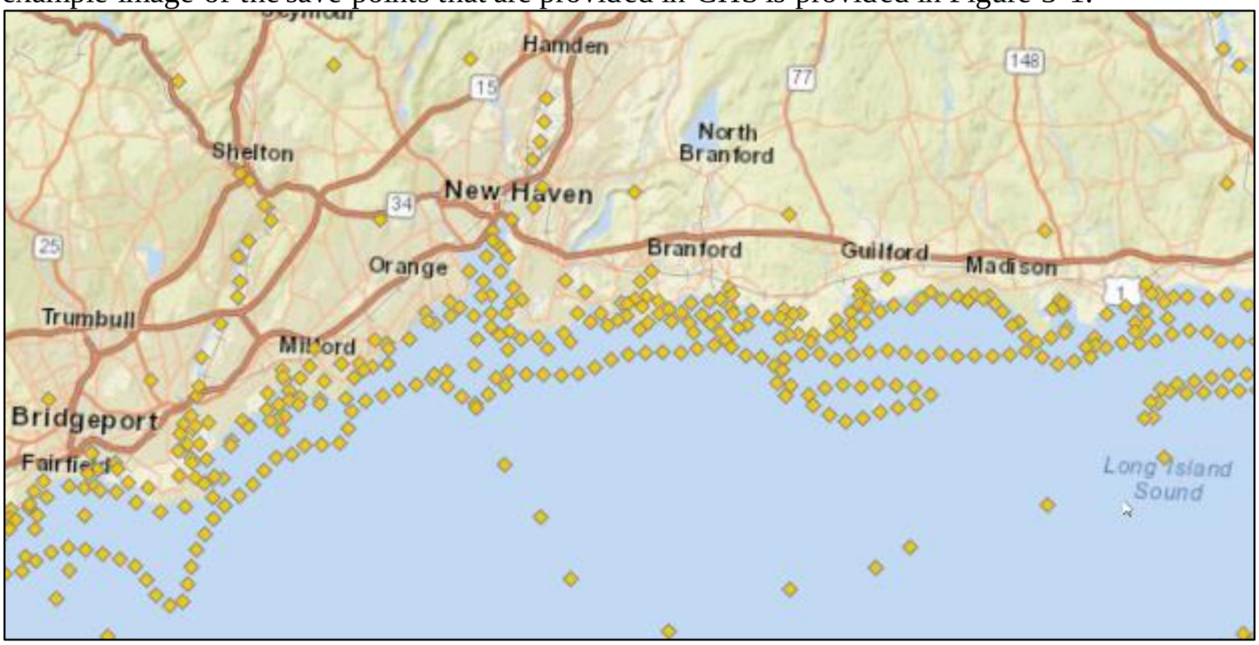

Figure 3-1. CHS model save points

3.1.1 Water Levels

NACCS water levels were used as input to the HEC-FDA economic model for evaluating damages in the future without- and with-project alternatives. Water levels and wave heights were used in designing the structural alternatives. For the Long Wharf area, water levels and wave heights were selected from save point 8134, shown in Figure 3-2. The NACCS model mesh is well-defined in this area and water level output was able to be applied directly to the study area without the need for transformation. This save point was considered most representative for the entirety of the study area. The CHS contains water levels in meters, relative to Mean Sea Level, at annual recurrence intervals from 1 year to 10,000 years at four confidence limits (CL). These water levels at save point 8134 are shown in Figure 3-3. The water levels were converted to feet, NAVD88 and are provided in Table 3-1.

Because economic analyses compute the National Economic Development (NED) Plan utilizing benefits at the mean level, the mean, or expected value, water levels from save point 8134 were used for evaluating damages in the study area. However, Figure 3-3 and Table 3-1 express the epistemic uncertainty of the water level response as confidence limits. As only the upper confidence limits are shown it is assumed that the distributions of annual exceedance probability are symmetrical. The annual exceedance probability water levels at higher confidence limits are presented to show the range of uncertainty. In accordance with ER 1105-2-101, Risk Assessment for Flood Risk Management Studies, the mean annual exceedance probability values have been used in the economic analyses while the 90-percent confidence limit values have been used to communicate project performance.

Figure 3-2. NACCS save point location

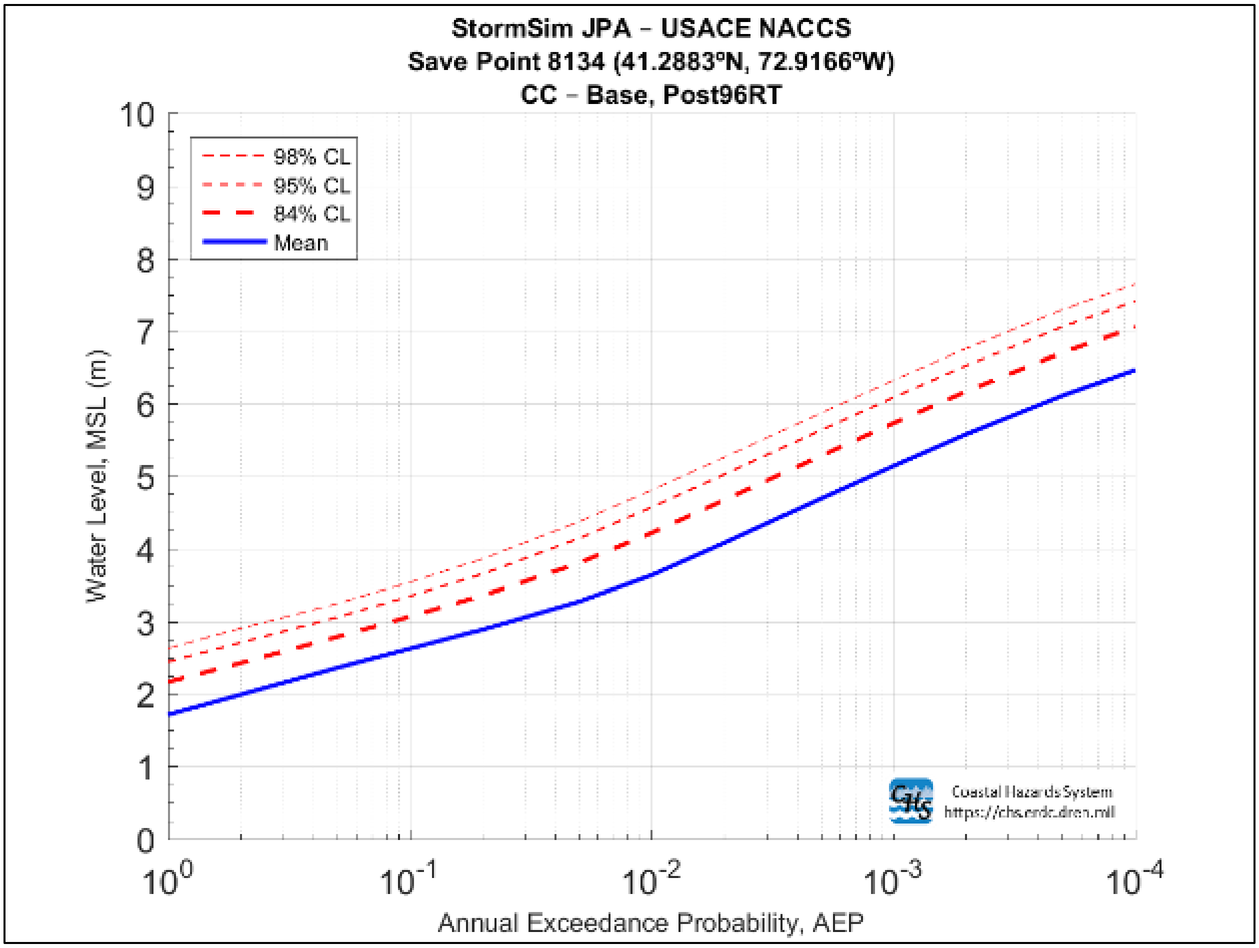

Figure 3-3. Annual exceedance probability water levels in meters, MSL

Table 3-1. Annual Exceedance Probability Water Levels from NACCS Study

NACCS Save Point 8134	Annual Exceedance Probability Water Level (feet, NAVD88)								
	99%	50%	20%	10%	5%	2%	1%	0.5%	0.2%
Mean, Expected Value	5.35	6.26	7.46	8.33	9.20	10.46	11.65	13.10	15.10
84% CL	6.83	7.69	8.87	9.78	10.73	12.20	13.56	15.04	17.04
90% CL	7.22	8.06	9.23	10.16	11.13	12.66	14.06	15.56	17.56
95% CL	7.74	8.61	9.75	10.70	11.71	13.31	14.72	16.20	18.20
98% CL	8.35	9.26	10.37	11.33	12.39	14.06	15.49	16.98	18.98

3.1.1 Wave Conditions

Long Island shelters the New Haven shoreline from long period waves from the Atlantic Ocean. Therefore, waves in the New Haven Harbor vicinity are fetch-limited only, driven by winds blowing over a length of the Sound. The breakwater system at the southern limits of the harbor provides protection within the harbor from waves approaching from southerly directions. Fetch and wave development are limited by topography in other directions. Although there are no wave records within New Haven Harbor, extreme wave conditions estimated through the NACCS modeling effort at save point 8134 are provided in Table 3.2.

Table 3-2. Mean Annual Exceedance Probability Wave Conditions from NACCS Study

	NACCS Save Point 8134, New Haven Harbor	
Annual Exceedance Probability	**Wave Height (feet)**	**Wave Period (seconds)**
99%	2.2	3.3
50%	2.7	3.6
20%	3.1	3.9
10%	3.5	4.0
5%	3.9	4.2
2%	4.4	4.4
1%	4.7	4.5
0.5%	5.0	4.7
0.2%	5.4	5.0

3.2 GZA Long Wharf Protection Study

GZA built off the NACCS effort to model storm surge and wave hazards specific to the Long Wharf study area. GZA modeled tidal flow, the 1-percent annual exceedance probability (100-year return period) coastal flood and the 0.2-percent annual exceedance probability (500-year return period) coastal flood events within the study area using the two-dimensional, hydrodynamic Advanced CIRCulation (ADCIRC) model. Waves were modeled using the Simulating Waves Nearshore (SWAN) model. The purpose of these modeling efforts was to evaluate flooding hydrodynamically and temporally, reflecting current topographic and shoreline conditions and to provide input for evaluating flood mitigation alternatives at Long Wharf.

The results of the GZA study were reviewed for their accuracy and assumptions and are considered to be adequate for use in this feasibility study for the evaluation and selection of the Tentatively Selected Plan (TSP). As the GZA modeling effort built off the NACCS, it provided more detailed and site-specific output than the regional modeling effort. USACE approved models and methods were used in the analyses.

3.2.1 Water Levels

GZA's ADCIRC storm surge flood simulation methodology used a robust, but simplified approach and included: 1) creation of a local area, high resolution model mesh; 2) development of synthetic hydrographs representative of storm types associated with the 1-percent and 0.2-percent annual exceedance probability coastal flood events; 3) utilization of the NACCS-predicted peak stillwater elevations at the model boundary to develop the peak hydrograph water level; and 4) stressing the model with the synthetic hydrograph and model domain wind field. This approach provided the benefits of numerical hydrodynamic models, approximating scenario-based simulations, but also tied the overall flood hazard definition (model boundary water levels) to those developed by the NACCS. The model was validated using tidal conditions and additional model checks were performed by comparing the ADCIRC modeling output to representative NACCS output for save points located within the model domain. Figures 3-4 and 3-5 show the detail of GZA's model mesh, developed specifically for the Long Wharf study area.

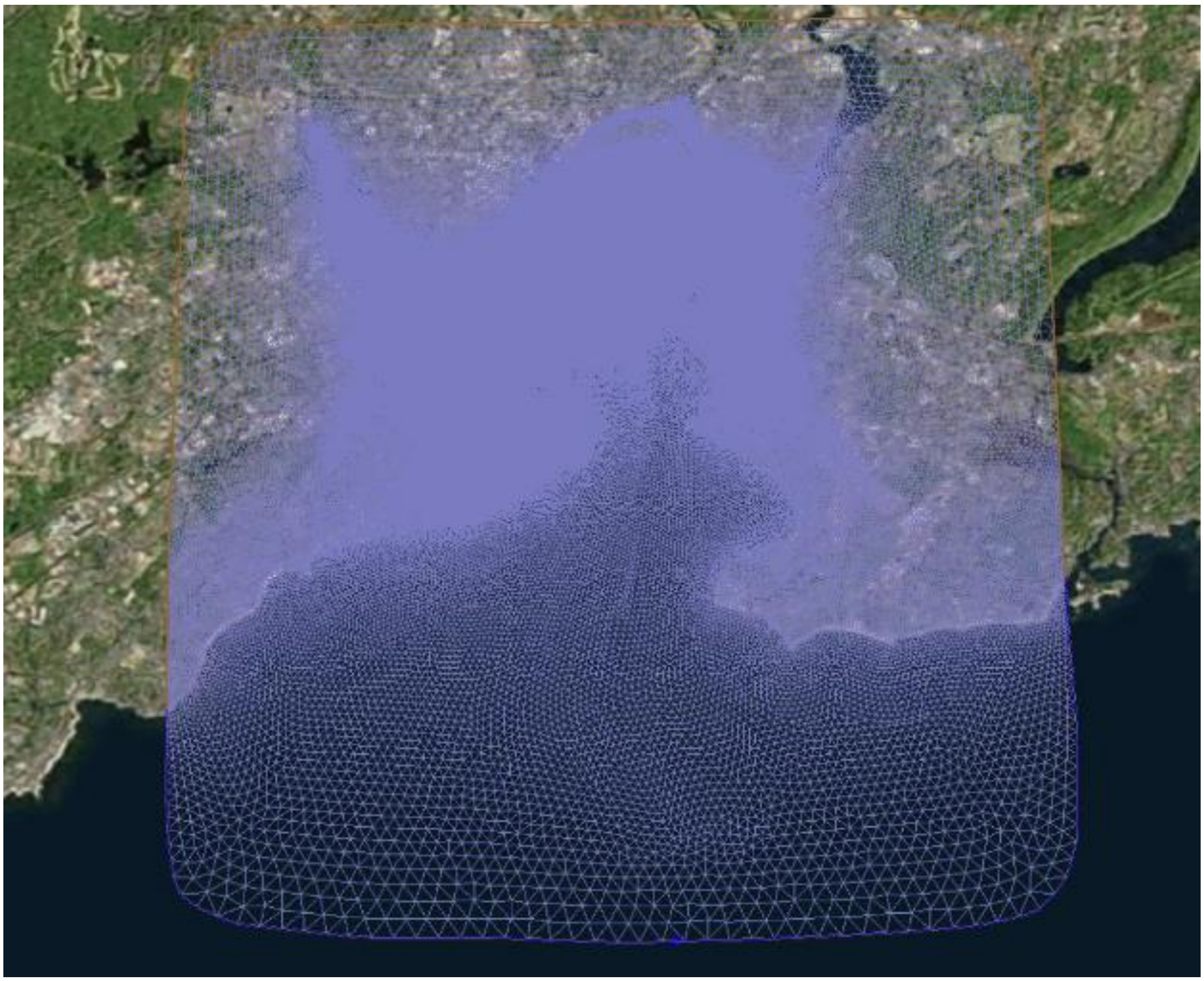

Figure 3-4. ADCIRC model mesh domain (GZA)

While the NACCS study provided peak water levels at a save point offshore of Long Wharf, the GZA modeling effort examined the propagation of storm surge throughout the study area over the course of an extreme storm event. By doing this, it could be determined which areas flood first and are most vulnerable, and at what rate flooding occurs. GZA modeled storms corresponding to the present day (2016) and future (2116) NACCS 1-percent and 0.2-percent annual exceedance probability (100 year and 500 year annual recurrence interval) peak flood elevations. The future model runs for the year 2116 added a sea level rise component corresponding to the USACE high sea level change scenario. Sea level change is discussed in detail in Section 4.0.

Figures 3-6 and 3-7 show the 1- and 0.2-percent annual exceedance probability stillwater extents, respectively. Water surface elevations are in feet relative to NAVD88.

Figure 3-5. High resolution ADCIRC model detail in the Long Wharf area (GZA)

In addition to determining annual exceedance probability flood extents, GZA determined the flooding pathways and sequence with which flooding occurs landward of the I-95 embankment. The flooding pathways, or water intrusion points, are identified in Figure 3-8 and numbered according to their vulnerability.

When storm surge water levels rise to levels greater than the Long Wharf shoreline, surge begins to propagate across Long Wharf Drive and beneath the Canal Dock Road I-95 underpass (Figure 3-8 water intrusion point 1). Canal Dock Road is a low point at about elevation 7 feet NAVD88.

As storm surge elevations rise to 10-11 feet NAVD88 (present 5%-1% NACCS mean AEP) at the shoreline, the inland areas to the west-northwest of the Canal Dock Road I-95 underpass begins to flood, including the area around the Pirelli Building and IKEA parking area to about elevation 8 feet NAVD88. At this point, the Long Wharf Drive I-95 underpass (water intrusion point 2) is also inundated as are the on and off ramps that link I-95 North to Long Wharf Drive.

Figure 3-6. 1-percent annual exceedance probability flood extents (modified from GZA, 2017)

Figure 3-7. 0.2-percent annual exceedance probability flood extents (modified from GZA, 2017)

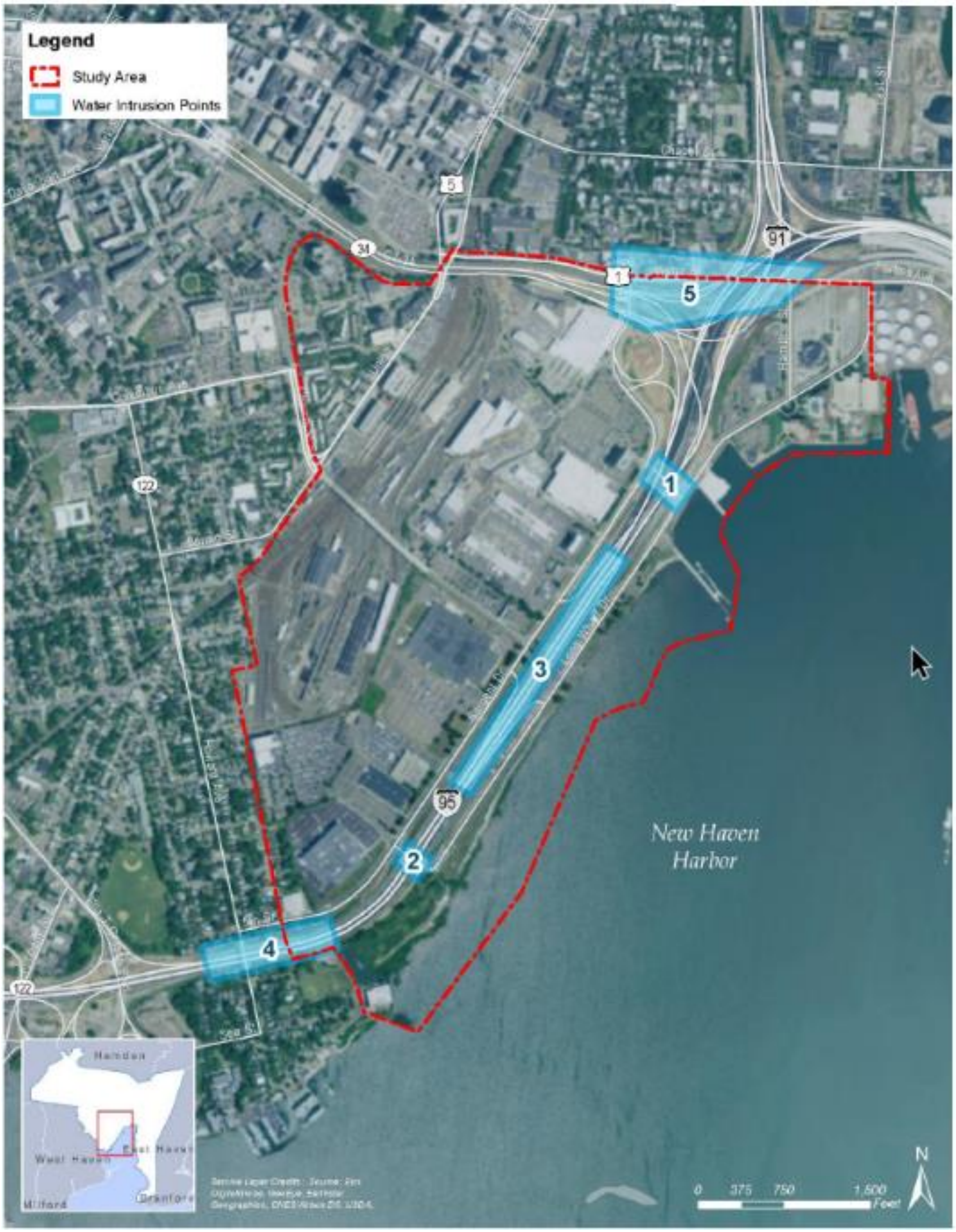

Figure 3-8. Study area water intrusion points (GZA)

If surge at the shoreline reaches elevations of 12-13 feet NAVD88 (present 1%-0.5% NACCS mean AEP), water levels inland of I-95 rise to about elevation 10 feet NAVD88. At this point, much of the commercial and industrial inland area of Long Wharf is flooded and the railyard is starting to experience significant flooding. The central, low-lying portion of I-95 (water intrusion point 3), which bottoms out between elevation 10 and elevation 11 feet NAVD88, is also flooded as well. In addition to potential roadway damages, this would have monumental transportation impacts.

Surges that rise to elevation 13-14 feet NAVD88 (present 0.5%-0.2% NACCS mean AEP) at the shoreline flood the area inland of I-95 to about elevation 12 feet NAVD88. All of the Long Wharf commercial and industrial area and the railyard are flooded. The flooded area of the low-lying portion of I-95 broadens and I-95 also floods beneath the Howard Avenue bridge (water intrusion point 4). At the north end of the study area, surge propagates to the west along Water Street/Route 1 and enters the Long Wharf inland area at the Brewery Street intersection (water intrusion point 5).

GZA noted that the depth and extent of flooding west of I-95 is partially dependent upon the shape and duration of the water level hydrograph since the I-95 underpasses constrict flow into the inner portion of the Long Wharf area. A sensitivity test was performed using two differently shaped hydrographs. The first was representative of an intense hurricane with a narrow, peaked hydrograph while the second was representative of a large nor'easter with a longer duration. While the hydrographs had the same peak flood elevation, the second hydrograph resulted in a greater amount of flooding west of I-95 since the duration of peak flooding (several tide cycles) and the total volume of flood water were greater.

3.2.2 Wave Conditions

As part of the Long Wharf Flood Protection Study, GZA performed computer simulations using the SWAN wave model for the present day (2016) 1-percent annual exceedance probability (100-year annual recurrence interval) flood. Wind and model boundary waves were applied from a southerly direction to maximize fetch within New Haven Harbor. Predicted significant wave heights are presented in Figure 3-9. Wave heights reach approximately 5 feet at the southern end of the project area and decrease moving north, reaching approximately 4 feet to the north of Long Wharf pier. These wave heights are comparable to the wave conditions from the NACCS presented in Table 3-2. I-95 prevents waves from propagating west of I-95. However, wind forces can locally generate waves landward of I-95 which reach up to approximately 1-2 feet.

In addition to modeling extreme wave conditions, GZA hindcasted nearshore wave conditions using wind-wave generation models recommended in the Shore Protection Manual (1984) from wind data gathered from Tweed Airport in New Haven, CT over a 68-year period (1948-2017). Wind data was split into 22.5-degree sectors based on wind direction. A separate wave fetch and average water depth was determined for each directional bin to calculate directional wave heights. Wave heights were then summed for all wave directions to determine the total number of occurrences of each wave height and the percentage of time each wave height was exceeded to create wave frequency curves. Wave statistics were computed for two locations, one north and one south of Long Wharf pier as shown in Figure 3-10.

Results from GZA's hindcasted nearshore wave conditions are shown in Figures 3-11 through 3-14. The 20-percent exceedance wave height ($H_{20\%}$) has been identified for use in the City's living shoreline design.

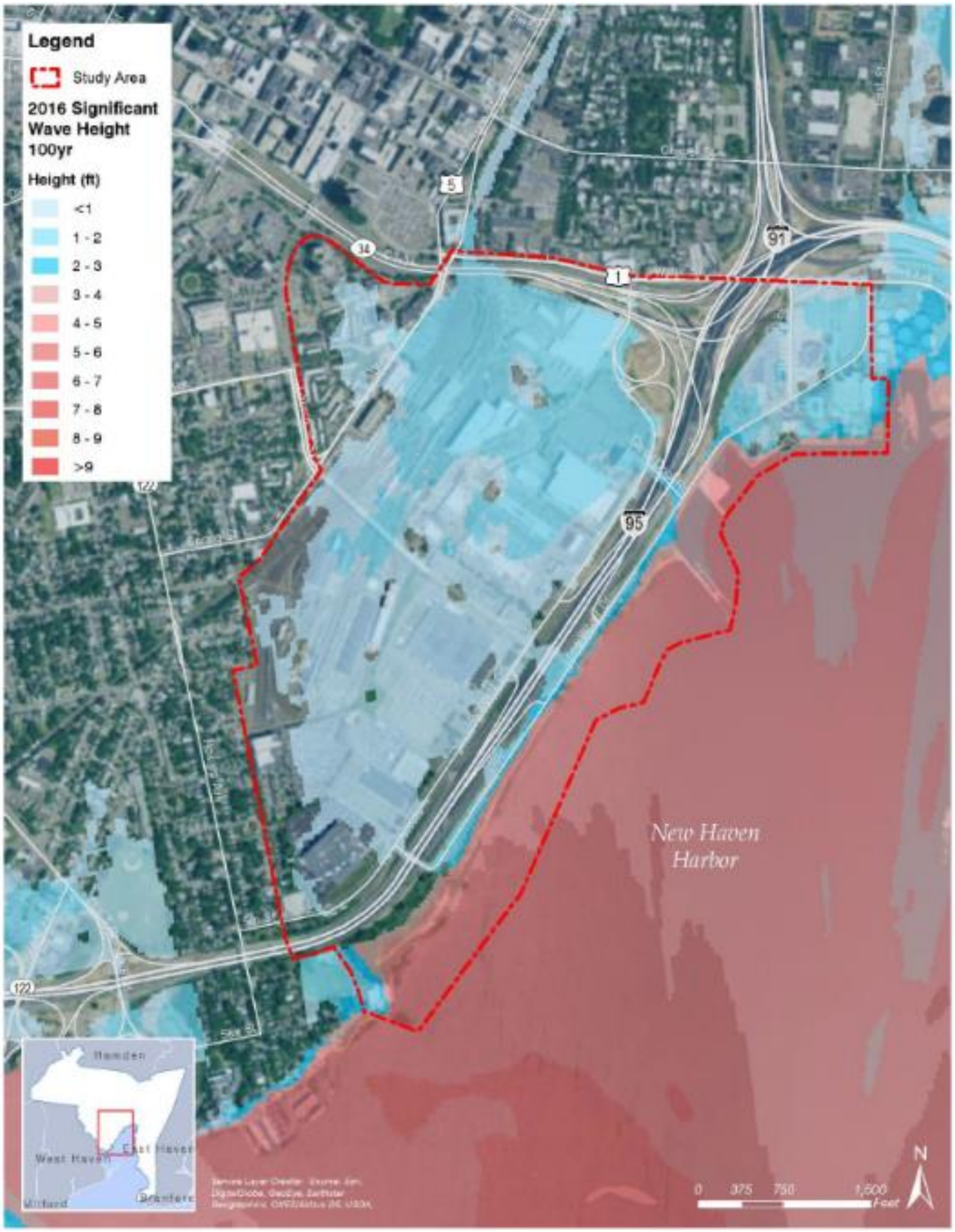

Figure 3-9. Modeled significant wave height, in feet (GZA)

Figure 3-10. Fetches for wind-wave growth hindcast for south (left) and north (right) of Long Wharf Pier (GZA)

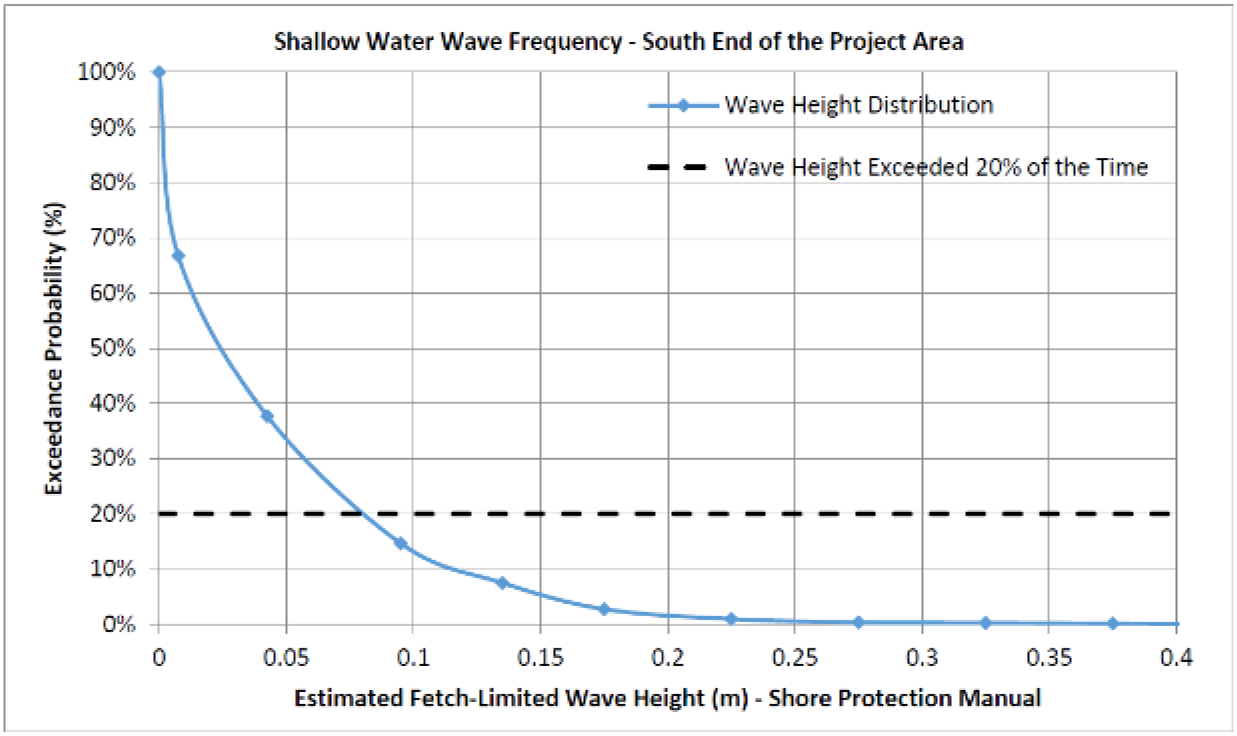

Figure 3-11. Nearshore wave frequency, south end of project area (GZA)

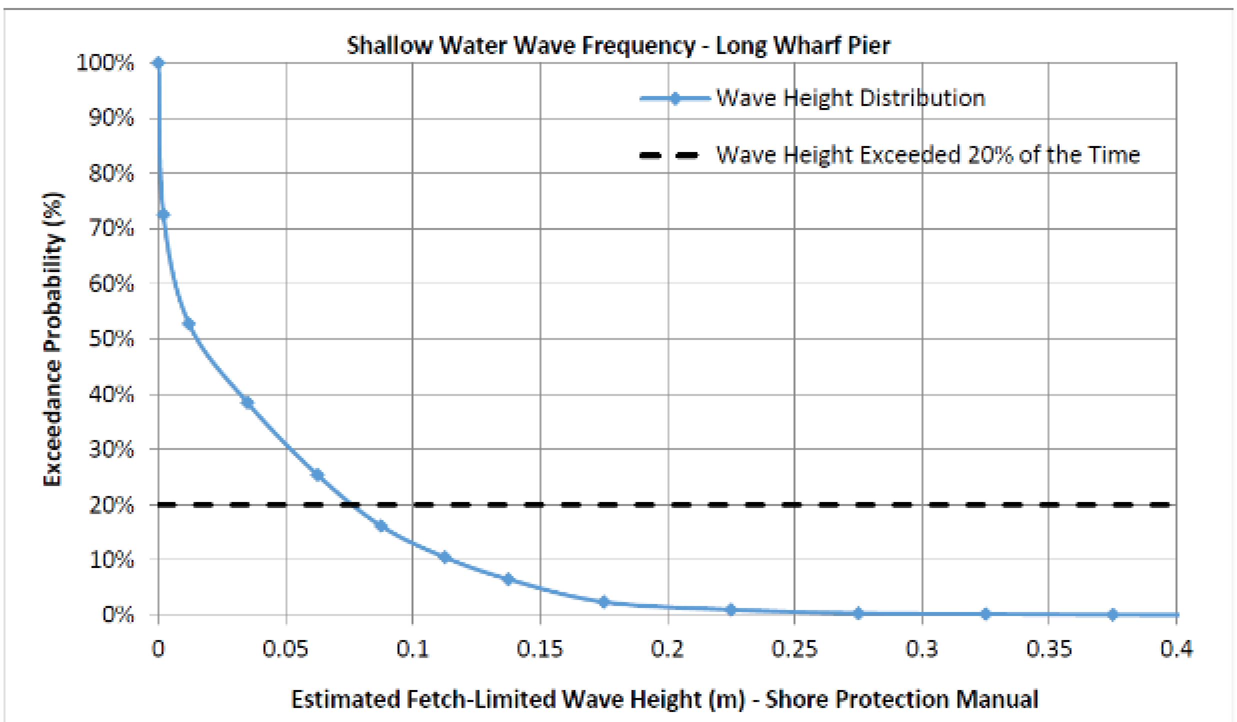

Figure 3-12. Nearshore wave frequency, Long Wharf Pier (GZA)

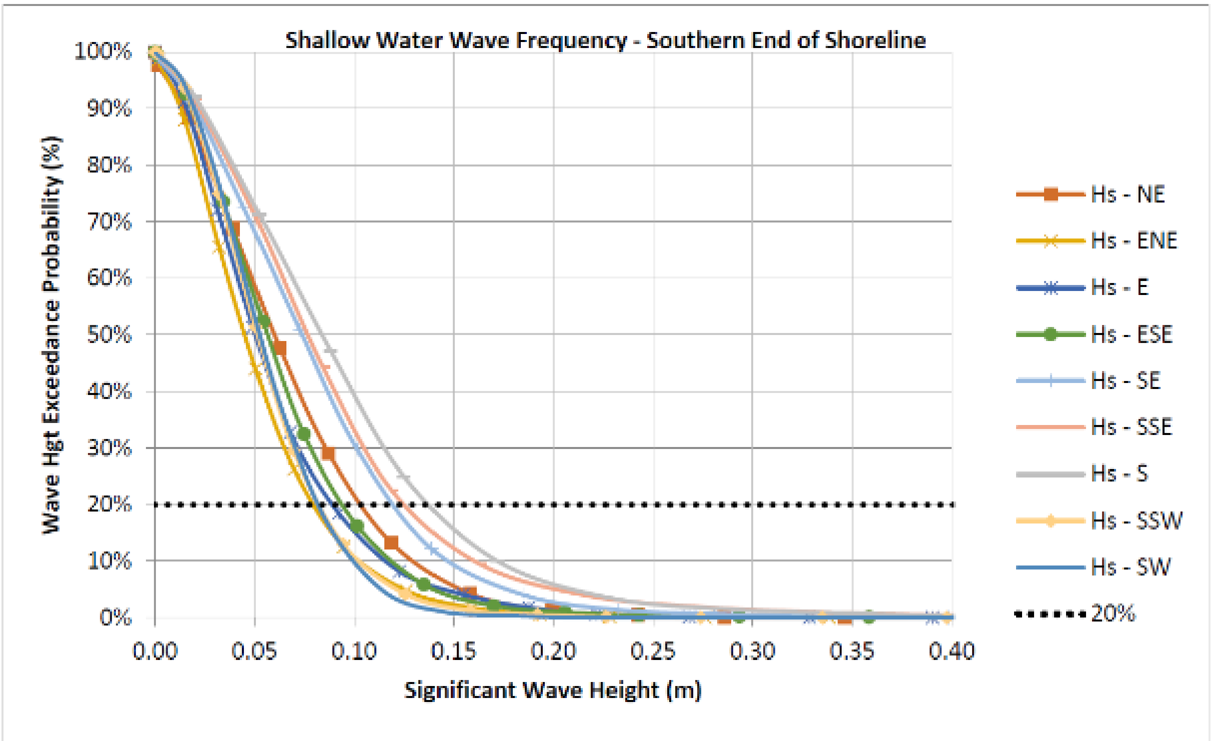

Figure 3-13. Nearshore wave frequency by direction, south end of project area (GZA)

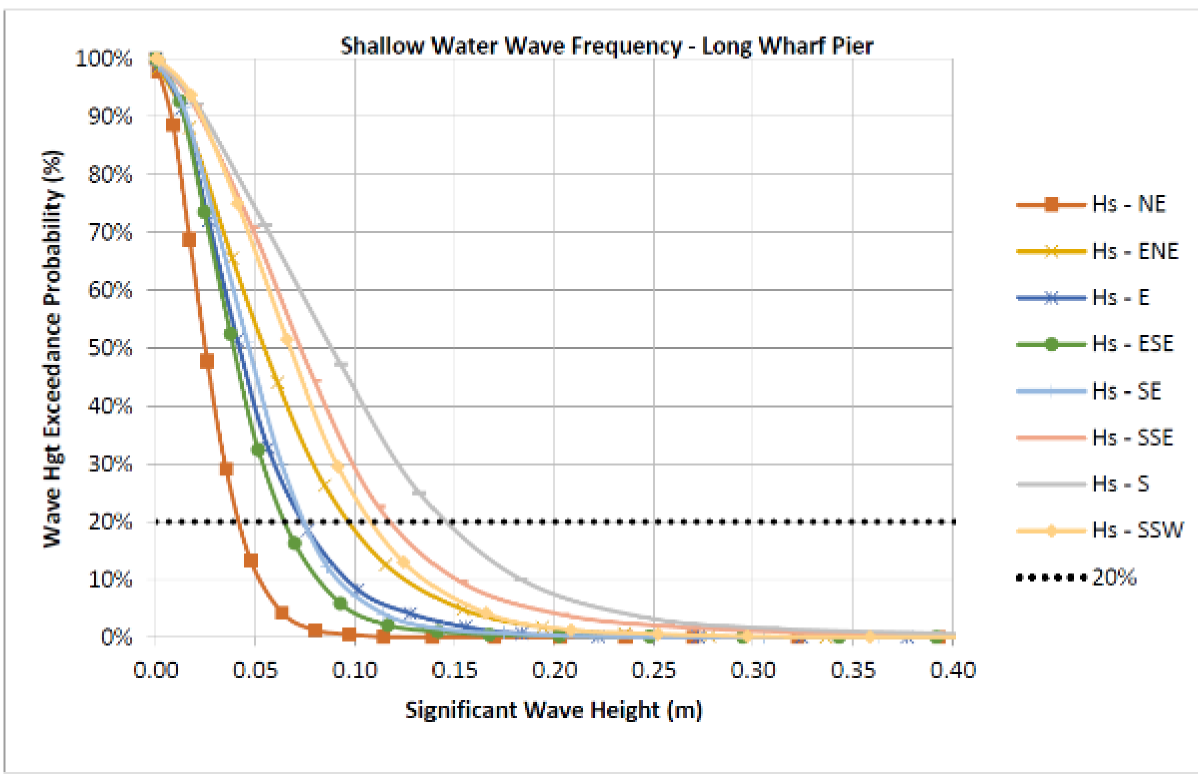

Figure 3-14. Nearshore wave frequency by direction, Long Wharf Pier (GZA)

3.3 FEMA

The FEMA Flood Insurance Study for New Haven County, effective May 2017, evaluated flood hazards at two shore-perpendicular coastal transects within the study area. The coastal water levels for this study were based on statistical analysis of regional tide gages through 2007 and, as such, did not include the impacts of Hurricanes Sandy and Irene. Transects 20 and 21 are shown in Figure 3-15 with the effective floodplain mapping. The flood extents of the 1-percent annual exceedance probability (100-year annual recurrence interval) event are depicted in blue. The floodplain is divided into polygons by Base Flood Elevation, calculated as the total stillwater elevation (stillwater elevation including storm surge plus wave setup) for the 1-percent annual exceedance probability storm plus the additional flood hazard from overland wave effects (overland wave propagation, wave runup and wave overtopping). In addition to the Base Flood Elevation, polygons in the study area are identified as being Zone VE, velocity wave zones with wave heights or runup depths greater than 3 feet, or Zone AE, areas with wave heights or runup depths less than 3 feet. The starting annual exceedance probability stillwater elevations for Transects 20 and 21 are provided in Table 3-3. Note that the FEMA water elevations are lower than those predicted by the NACCS. It was assumed that the 1992 mean sea level associated with the midpoint of the current National Tidal Datum Epoch was used in the FEMA analysis and mapping.

Figure 3-15. FEMA floodplain mapping and transects in the study area

Table 3-3. FEMA Annual Exceedance Probability Stillwater Levels

Transect	Annual Exceedance Probability Stillwater Elevation (feet, NAVD88)				
	10%	2%	1%	1% + wave setup	0.2%
20	6.8	8.3	8.9	10.9	10.5
21	6.8	8.3	8.9	12.2	10.6

4.0 Sea Level Change

The USACE Sea Level Change Curve Calculator (2019.21) was used to predict three local relative sea level change (SLC) scenarios per ER 1100-2-8162: Incorporating Sea Level Change in Civil Works Programs and EP 1100-2-1: Procedures to Evaluate Sea Level Change: Impacts, Response, and Adaptation. The purpose of the ER is to incorporate relative sea level changes into the project alternatives and design. The three SLC scenarios are illustrated by curves representing the low (historic) rate of SLC at the project area, an intermediate rate (modified NRC Curve I), and a high rate of SLC (modified NRC Curve III). All three local SLC curves include the global (eustatic) sea level rise rate (approximately 1.7 mm/year according to IPCC 2007) as well as local vertical land movement.

The length of tide station record is important to consider when estimating historic relative SLC because inter-annual, decadal, and multi-decadal variations in sea level are sufficiently large that misleading or erroneous sea level trends can be derived from periods of record that are too short. A minimum record length of 40 years is recommended to determine reasonable trends. The nearest long-term NOAA tide gage is located in Bridgeport, CT (Station 8467150, 85 year record). The historic mean sea level trend at Bridgeport from 1964 to 2017 is 0.00942 feet/year (2.87 mm/year) or 0.94 feet per century. The mean trend is shown in Figure 4-1 which was taken from the NOAA Sea Level Trend web page https://tidesandcurrents.noaa.gov/sltrends/sltrends_station.shtml?stnid=8467150. As shown in the plot there are yearly and decadal cycles that cause the short term rate to vary. These observations illustrate that water levels are rising, but that the variations in the data are large, making it difficult to discern a statistically significant change from the historic rate or any of the future sea level rise scenarios at this time. By the end of the 50-year economic period of analysis (2074), sea level at New Haven is projected to rise 0.77 feet, 1.37 feet, and 3.27 feet under the USACE low, intermediate, and high scenarios, respectively, from the base year of 1992 which corresponds to the midpoint of the current National Tidal Datum Epoch of 1983-2001. Projections through 2124 are provided in Figure 4-2 and Table 4-1.

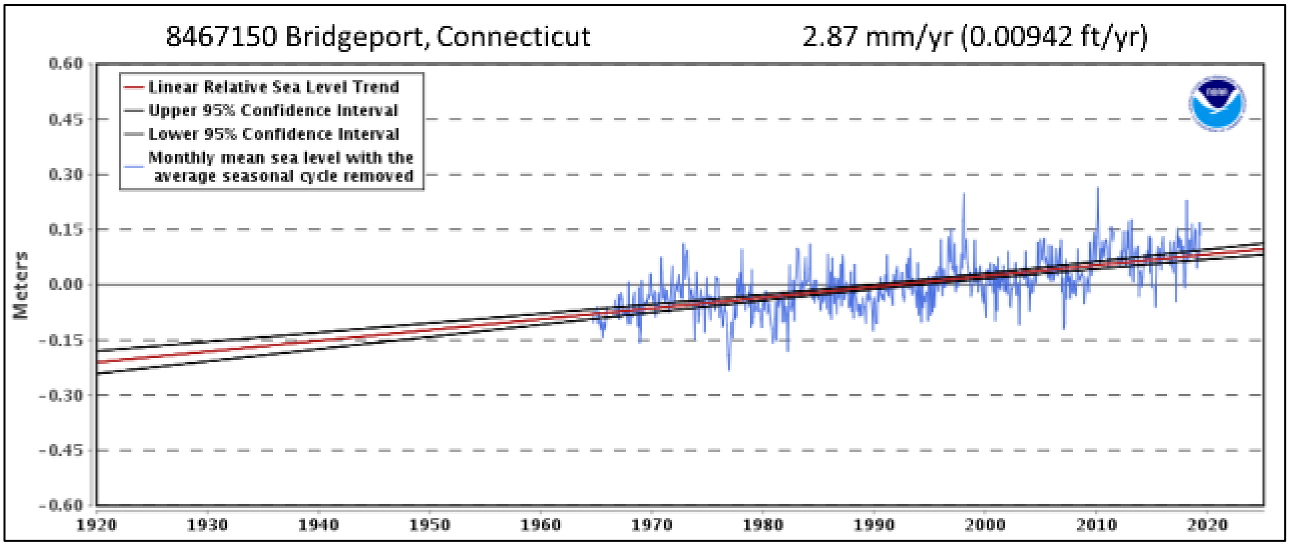

Figure 4-1. Historic sea level change at Bridgeport 1964-2017 (from NOAA/NOS CO-OPS)

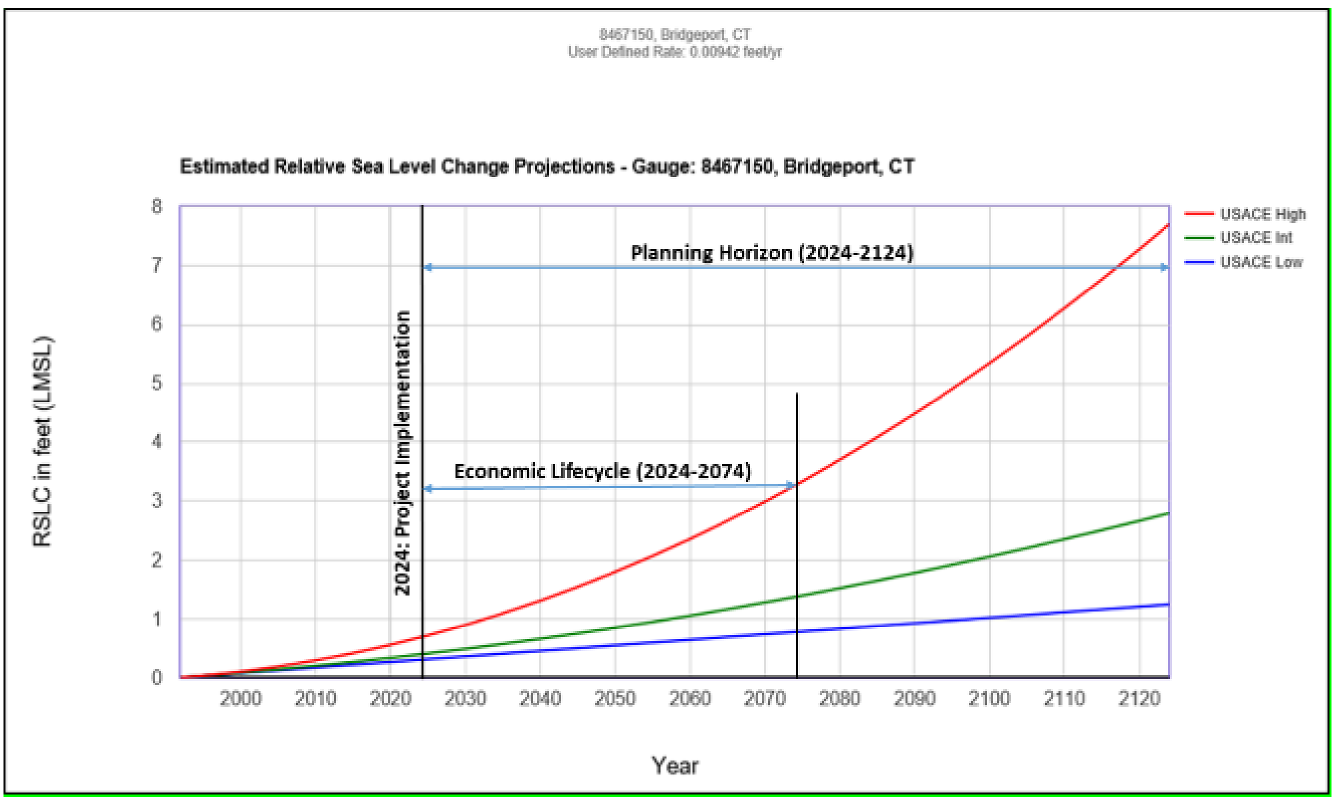

Figure 4-2. Relative Sea Level Change Projections at Bridgeport

Table 4-1. USACE sea level change rates – future scenarios

Year	Low RSLC (FT)	Intermediate RSLC (FT)	High RSLC (FT)
2074	0.77	1.37	3.27
2124	1.24	2.79	7.70

Note: Sea level change values are relative to the base year of 1992 which corresponds to the midpoint of the current National Tidal Datum Epoch of 1983-2001

In June 2018, Connecticut adopted Public Act 18-82—An Act Concerning Climate Change Planning and Resiliency. The act includes updating current statutory references to sea level rise to reflect the most recent sea level change scenario based upon the sea level change scenarios published by the NOAA in Technical Report OAR CPO-1, "Global Sea Level Rise Scenarios for the United States National Climate Assessment," and other available scientific data necessary to create a scenario applicable to the state coastline. The NOAA report bases global sea level rise by 2100 on four estimates that reflect different degrees of ocean warming and ice sheet loss resulting in four scenarios: lowest, intermediate-low, intermediate-high, and highest. Projected sea level rise worldwide ranges across these scenarios from 0.66 to 6.6 feet by 2100.

To narrow this estimate, the University of Connecticut's Department of Marine Science is charged with updating the NOAA projections every 10 years, and, specifically the Connecticut Institute for Resilience and Climate Adaptation (CIRCA) with determining sea level rise statistics for the State of Connecticut. CIRCA provided specific projections for several sea level rise scenarios along with recommendations for specific scenarios in their October 2018 report "Sea Level Rise and Coastal Flood Risk in Connecticut: An Overview." CIRCA utilized projections from other sources and adjusted the projections based on local oceanographic and land motion conditions. CIRCA's projections for the four NOAA scenarios range approximately from 1.9 to 6.6 feet in 2100.

The CIRCA analysis also recommends that planning anticipates that sea level will be 0.5 m (1.6 feet) higher than the national tidal datum by 2050. Further, it recommends planning for an increase of 1.0 m (3.3 feet) by 2100. These recommendations are slightly higher than the rates given by the USACE intermediate scenario. Given this recommendation, the USACE intermediate sea level rise scenario was used to estimate future conditions for the feasibility study. The sensitivity of the Tentatively Selected Plan to the low and high sea level change scenarios was also evaluated. Table 4-2 provides mean annual exceedance probability water levels, adjusted for the three sea level change scenarios, for the start and end years of the project's economic and planning horizons. Table 4-3 presents the same water level information at the 90-percent confidence limit.

Table 4-2. NACCS Mean Annual Exceedance Probability Water Levels Adjusted for SLC Scenarios

NACCS Save Point 8134	Mean Annual Exceedance Probability Water Levels (feet, NAVD88)								
Low SLC									
	99%	50%	20%	10%	5%	2%	1%	0.5%	0.2%
2024	5.65	6.56	7.76	8.63	9.50	10.76	11.95	13.40	15.40
2074	6.12	7.03	8.23	9.10	9.97	11.23	12.42	13.87	15.87
2124	6.59	7.50	8.70	9.57	10.44	11.70	12.89	14.34	16.34
Intermediate SLC									
	99%	50%	20%	10%	5%	2%	1%	0.5%	0.2%
2024	5.74	6.65	7.85	8.72	9.59	10.85	12.04	13.49	15.49
2074	6.72	7.63	8.83	9.70	10.57	11.83	13.02	14.47	16.47
2124	8.14	9.05	10.25	11.12	11.99	13.25	14.44	15.89	17.89
High SLC									
	99%	50%	20%	10%	5%	2%	1%	0.5%	0.2%
2024	6.03	6.94	8.14	9.01	9.88	11.14	12.33	13.78	15.78
2074	8.62	9.53	10.73	11.60	12.47	13.73	14.92	16.37	18.37
2124	13.05	13.96	15.16	16.03	16.90	18.16	19.35	20.80	22.80

Table 4-3. NACCS 90% Confidence Limit Annual Exceedance Probability Water Levels Adjusted for SLC Scenarios

NACCS Save Point 8134	90% Confidence Limit Annual Exceedance Probability Water Levels (feet, NAVD88)								
Low SLC									
	99%	50%	20%	10%	5%	2%	1%	0.5%	0.2%
2024	7.52	8.36	9.53	10.46	11.43	12.96	14.36	15.86	17.86
2074	7.99	8.83	10.00	10.93	11.90	13.43	14.83	16.33	18.33
2124	8.46	9.30	10.47	11.40	12.37	13.90	15.30	16.80	18.80
Intermediate SLC									
	99%	50%	20%	10%	5%	2%	1%	0.5%	0.2%
2024	7.61	8.45	9.62	10.55	11.52	13.05	14.45	15.95	17.95
2074	8.59	9.43	10.60	11.53	12.50	14.03	15.43	16.93	18.93
2124	10.01	10.85	12.02	12.95	13.92	15.45	16.85	18.35	20.35
High SLC									
	99%	50%	20%	10%	5%	2%	1%	0.5%	0.2%
2024	7.90	8.74	9.91	10.84	11.81	13.34	14.74	16.24	18.24
2074	10.49	11.33	12.50	13.43	14.40	15.93	17.33	18.83	20.83
2124	14.92	15.76	16.93	17.86	18.83	20.36	21.76	23.26	25.26

5.0 Climate Hydrology

Connecticut is characterized by cold, snowy winters and warm, humid summers. The polar jet stream is often located near the state leading to highly variable weather patterns and generally abundant precipitation throughout the year. Temperatures along the coast are moderated by close proximity to the Atlantic Ocean with warmer winters than inland areas. The temperature averages 52 degrees Fahrenheit (°F) annually along the coast, ranging from a low monthly average of 31 °F in February to a high monthly average of 74 °F in July. Temperatures above 90 °F are rather infrequent, with an average of up to 2 days in New Haven annually. Extreme cold (below 0 °F) occurs on average 0.3 days per year. Precipitation throughout the state is abundant, but highly variable from year to year; amounts range from 31 to 63 inches per year. In the winter months, average accumulated snowfall ranges between 30 and 35 inches along the coast (Runkle et al, 2017). Climate varies throughout the year. Flooding from inland sources has historically happened during all seasons but the largest recent flood (Tropical Storm Irene) occurred in the summer of 2011.

A climate assessment for New Haven was developed to address the requirements contained within ECB 2018-14, Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil

Works Studies, Designs, and Projects. The assessment included a regional literature synthesis and relied on the rainfall analysis completed by CDM Smith for the City of New Haven's Downtown Stormwater Modeling Project Final Report in 2017. The main climate variables that have been identified to affect inland hydrology within the study area include temperature, precipitation intensity, and precipitation volume.

5.1 Literature Synthesis

2018 National Climate Assessment

According to the 2018 National Climate Assessment (NCA), hydrologic changes are occurring within the New Haven area (Dupigny-Giroux et al. 2018). The following discussion was largely developed based upon the content summarized in the 2018 NCA. The NCA content and the knowledge gained through the quantitative modeling were translated to develop conclusions on how inland hydrology affecting the study area would be impacted.

The dominant trend in precipitation throughout the Northeast has been towards increases in rainfall intensity. Increases in precipitation are expected during the winter and spring but little change is expected during the summer with monthly precipitation projected to be about 1 inch greater for December through April by the end of the century (2070-2100) under the higher scenario. Over the period 1958 to 2012, the amount of precipitation falling in the heaviest (1% annual chance exceedance) precipitation events has increased 55% in the Northeastern U.S. Moderate flooding events are reportedly expected to become more frequent in the Northeast during the 21st century because of more intense precipitation related to climate change.

The hydrologic changes are most evident in the winter and spring seasons, where temperature increases of approximately 1.67 °F over the period of 1940 to 2014 have led to an advance in the timing of snowmelt and spring runoff by more than 10 days. Winters have warmed three times faster than summers. Warmer winter temperatures have increased the fraction of precipitation that falls as rain instead of snow. Correspondingly, the freeze free period is expected to expand. In New Haven County, under the lower climate scenario (RCP4.5) the last spring freeze is expected to be 10-14 days earlier and the first fall freeze is expected to be 0-6 days later by 2069. At the same location, under the higher climate scenario (CP8.5), the last spring freeze is expected to be 18-22 days earlier and the first fall freeze is expected to be 22-26 days later by the year 2069. This suggests that under the lower scenario there will be 10-20 additional frost free days and under the higher scenario there will be 40-48 additional frost free days. These projected higher temperatures and frost free days would lead to less early winter snowfall and earlier snowmelt.

NOAA State Climate Summary

NOAA has published a set of individual state climate summaries containing information on historic climate variations and trends, future climate model predictions, and past and future conditions of sea level and coastal flooding. NOAA reported the following key messages for Connecticut:

- Temperatures in Connecticut have increased about 3 °F since the beginning of the 20th century. Under a higher emissions pathway, historically unprecedented warming is projected by the end of the 21st century, with associated increases in heat wave intensity and decreases in cold wave intensity.

- Precipitation has increased during the last century, with the highest number of extreme events occurring over the last decade. Increases in the frequency and intensity of extreme precipitation events are projected, as well as increases in winter and spring precipitation.
- Sea level has risen at a rate of 10-11 inches per century along the Connecticut coast, faster than the global rate. Global sea level is projected to rise another 1 to 4 feet by 2100, with even greater possible rises for Connecticut.

Similar to the NCA, NOAA reported that the average annual temperature has increased approximately 3 °F in Connecticut since the early 20th century. Future warming predictions are presented in Figure 5-1.

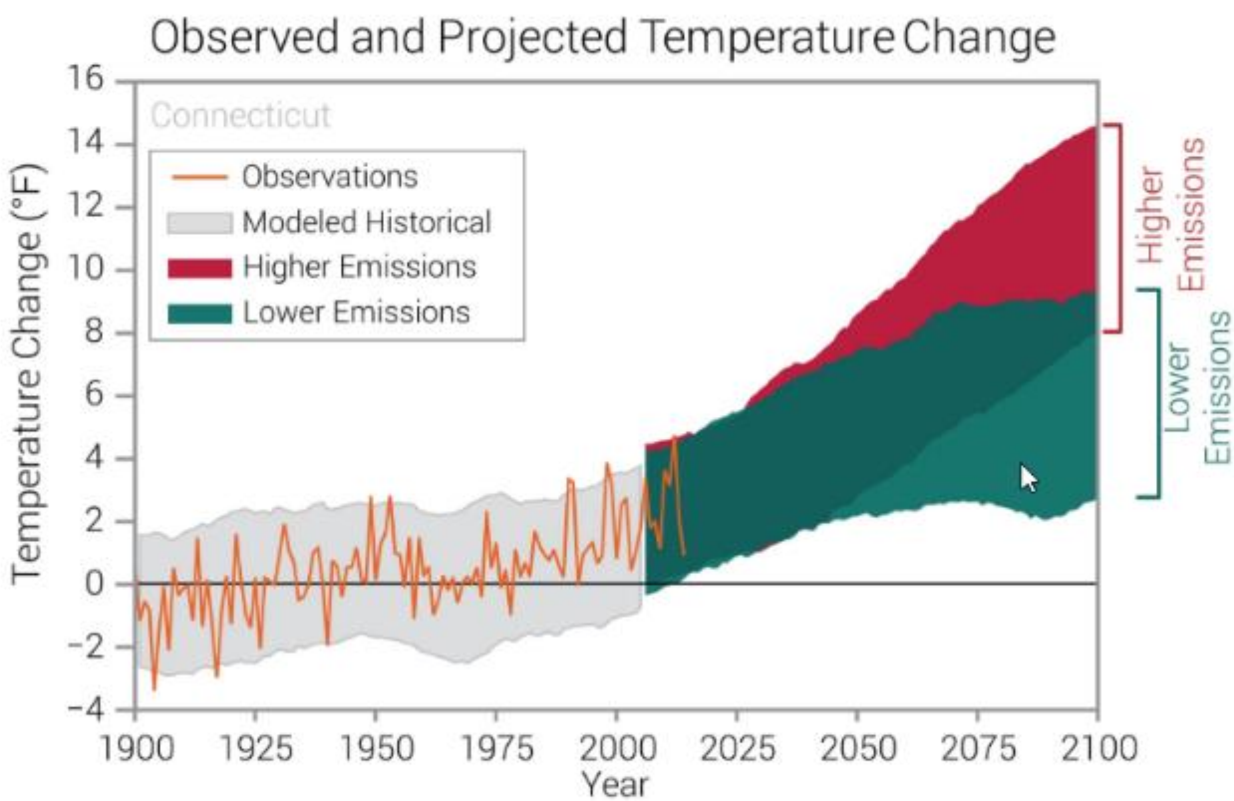

Figure 5-1. Observed and projected temperature change in Connecticut (Source: NOAA State Climate Summary 149-CT)

NOAA reported that precipitation is abundant but highly variable from year to year in Connecticut. Generally, however, above average precipitation has occurred since the 1970s. Annual average precipitation is projected to increase, with increases most likely occurring in the spring and winter. Increases in total precipitation and in the number of extreme precipitation events (storms) may also result in increased coastal and inland flood risks.

The USACE report titled *Recent US Climate Change and Hydrology Literature Applicable to the US Army Corps of Engineers Missions—New England Region* summarizes observed and projected climate and hydrometeorologic patterns cited in reputable peer-reviewed literature and authoritative national and regional reports. The review was performed at the HUC-4 level and extended beyond the study area limits to include the entire USACE New England Region. It was noted that USACE judged that the regional, sub-continental climate signals projected by the driving climate models are coherent and useful at the HUC-2 scale and that the confidence in the driving climate model outputs declines for areas smaller than the watershed scale of the 4-digit HUC.

The review found that most studies agree that there has been an overall increase in average temperatures over the past century. However, some indicate that there may be a seasonal or localized cooling trends occurring. Some studies also indicate a greater temperature increase occurring during the winter months. Minimum temperatures were also deemed to appear to be increasing but it was reported that there was no clear trend in high temperature extremes. Based on the review, a strong consensus exists in the literature that projected temperatures show an increasing trend through the next century in both average temperatures and high temperature extremes. Although no literature was reviewed studying projected extreme low temperatures, some studies indicate that seasonal winter temperatures are expected to rise at a faster rate than the annual average.

The review concluded that most studies identified an increase in both average and extreme precipitation. Snowfall was reported to appear to be decreasing as winter rainfall increases. Based on the review, average precipitation volumes are generally expected to increase along with the frequency and total precipitation volume of extreme events. However, the review found low consensus in the literature as some studies show no trend or variability by season or by location within the region, while others noted that projected precipitation trends vary between different model output datasets.

5.2 Nonstationarity Assessment

USACE ETL 1100-2-3, Guidance for Detection of Nonstationarities in Annual Maximum Discharges, requires the stationarity of all streamflow records analyzed in support of hydrologic analysis carried out for planning and engineering decision-making purposes be assessed. However, as a stream gage was not located within the study area and streamflow does not contribute to the flooding in the Long Wharf area, the precipitation record was examined.

Precipitation frequency statistics for New Haven were published by NOAA in October 2015 in Atlas 14, Volume 10. This publication formally replaced the 1961 National Weather Bureau TP-40 report, and superseded the 2013 Northeast Regional Climate Center (NRCC) atlas. Table 5-1 presents the average recurrence interval (ARI) extreme rainfall depths and Table 5-2 presents the ARI extreme rainfall intensities for New Haven as published in Atlas 14. In each table the average estimates are shown in bold with the lower and upper bounds of the 90% confidence intervals provided in parentheses. For example, the 10-year 1-day rainfall depth for New Haven is 5.30 inches and the 10-year 1-day precipitation intensity is 0.221 inches per hour.

A nonstationarity analysis of extreme precipitation data was completed by CDM Smith as part of their 2017 Downtown Stormwater Modeling Project Final Report, prepared for the City of New Haven. This assessment plotted the annual series of one-day precipitation maxima and the annual series of hourly precipitation maxima for New Haven's Tweed Airport. The annual series of one-day precipitation maxima for New Haven are shown in red in Figure 5-2. These are the same data used to develop the frequency statistics presented in Atlas 14. Typical annual maximum daily precipitation has remained consistent, except for the cluster of very high maxima in the 1870s, with about 10 percent of years exceeding 4.6 inches. As the New Haven data has some gaps, CDM Smith also looked at comparable datasets from Bridgeport's Sikorsky Airport, 13 miles to the southwest, which has a nearly complete record beginning in 1894. Also shown is data from New York City's Central Park, which is slightly wetter, but has a complete dataset since the mid-1800s.

Table 5-1. ARI Depth Estimates for New Haven

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)[1]										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**0.345** (0.278-0.424)	**0.415** (0.334-0.512)	**0.530** (0.424-0.655)	**0.626** (0.499-0.779)	**0.758** (0.581-0.986)	**0.857** (0.641-1.14)	**0.961** (0.695-1.33)	**1.08** (0.734-1.52)	**1.25** (0.811-1.82)	**1.39** (0.877-2.06)
10-min	**0.488** (0.393-0.601)	**0.588** (0.473-0.725)	**0.752** (0.603-0.929)	**0.887** (0.707-1.10)	**1.07** (0.823-1.40)	**1.22** (0.908-1.61)	**1.36** (0.985-1.88)	**1.53** (1.04-2.15)	**1.77** (1.15-2.58)	**1.96** (1.24-2.92)
15-min	**0.574** (0.463-0.707)	**0.692** (0.557-0.853)	**0.885** (0.710-1.10)	**1.04** (0.831-1.30)	**1.26** (0.969-1.64)	**1.43** (1.07-1.90)	**1.60** (1.16-2.21)	**1.80** (1.22-2.53)	**2.08** (1.35-3.03)	**2.31** (1.46-3.44)
30-min	**0.792** (0.638-0.975)	**0.954** (0.768-1.18)	**1.22** (0.977-1.51)	**1.44** (1.15-1.79)	**1.74** (1.34-2.26)	**1.97** (1.47-2.61)	**2.21** (1.60-3.04)	**2.48** (1.68-3.49)	**2.86** (1.86-4.18)	**3.18** (2.01-4.74)
60-min	**1.01** (0.813-1.24)	**1.22** (0.978-1.50)	**1.55** (1.25-1.92)	**1.83** (1.46-2.28)	**2.22** (1.70-2.88)	**2.51** (1.88-3.33)	**2.81** (2.03-3.88)	**3.16** (2.15-4.45)	**3.65** (2.37-5.32)	**4.05** (2.57-6.04)
2-hr	**1.31** (1.07-1.61)	**1.58** (1.28-1.94)	**2.02** (1.63-2.49)	**2.39** (1.92-2.95)	**2.89** (2.23-3.74)	**3.27** (2.46-4.32)	**3.67** (2.67-5.04)	**4.13** (2.82-5.78)	**4.81** (3.14-6.97)	**5.37** (3.41-7.94)
3-hr	**1.52** (1.24-1.86)	**1.84** (1.49-2.24)	**2.35** (1.90-2.87)	**2.77** (2.23-3.41)	**3.35** (2.60-4.32)	**3.79** (2.87-4.99)	**4.25** (3.11-5.83)	**4.79** (3.28-6.69)	**5.59** (3.66-8.08)	**6.26** (3.98-9.22)
6-hr	**1.94** (1.59-2.35)	**2.34** (1.91-2.83)	**2.99** (2.43-3.63)	**3.52** (2.85-4.31)	**4.26** (3.32-5.46)	**4.81** (3.66-6.30)	**5.40** (3.97-7.36)	**6.09** (4.18-8.44)	**7.12** (4.67-10.2)	**7.98** (5.09-11.7)
12-hr	**2.42** (2.00-2.91)	**2.91** (2.40-3.51)	**3.72** (3.05-4.49)	**4.39** (3.57-5.33)	**5.31** (4.16-6.76)	**6.00** (4.59-7.81)	**6.73** (4.98-9.12)	**7.60** (5.24-10.5)	**8.88** (5.85-12.7)	**9.97** (6.38-14.5)
24-hr	**2.85** (2.36-3.40)	**3.46** (2.87-4.14)	**4.47** (3.69-5.36)	**5.30** (4.34-6.39)	**6.44** (5.08-8.16)	**7.29** (5.62-9.45)	**8.21** (6.12-11.1)	**9.32** (6.44-12.7)	**11.0** (7.26-15.6)	**12.4** (7.98-17.9)
2-day	**3.19** (2.66-3.78)	**3.94** (3.29-4.68)	**5.17** (4.30-6.16)	**6.19** (5.11-7.42)	**7.60** (6.04-9.59)	**8.64** (6.71-11.2)	**9.77** (7.35-13.2)	**11.2** (7.76-15.2)	**13.4** (8.87-18.9)	**15.3** (9.88-22.0)
3-day	**3.46** (2.90-4.08)	**4.28** (3.59-5.06)	**5.64** (4.70-6.69)	**6.76** (5.60-8.07)	**8.31** (6.63-10.4)	**9.44** (7.36-12.2)	**10.7** (8.08-14.4)	**12.3** (8.52-16.6)	**14.7** (9.76-20.6)	**16.9** (10.9-24.1)
4-day	**3.70** (3.12-4.36)	**4.58** (3.85-5.40)	**6.01** (5.02-7.11)	**7.20** (5.97-8.56)	**8.83** (7.06-11.1)	**10.0** (7.83-12.9)	**11.3** (8.58-15.2)	**13.0** (9.05-17.5)	**15.6** (10.3-21.8)	**17.8** (11.5-25.4)
7-day	**4.40** (3.72-5.16)	**5.35** (4.52-6.27)	**6.91** (5.80-8.12)	**8.19** (6.84-9.69)	**9.97** (8.00-12.4)	**11.3** (8.83-14.4)	**12.7** (9.60-16.9)	**14.4** (10.1-19.4)	**17.1** (11.4-23.7)	**19.4** (12.5-27.4)
10-day	**5.09** (4.32-5.94)	**6.08** (5.15-7.10)	**7.70** (6.50-9.03)	**9.05** (7.58-10.7)	**10.9** (8.76-13.5)	**12.3** (9.62-15.5)	**13.8** (10.4-18.1)	**15.5** (10.9-20.7)	**18.1** (12.1-25.1)	**20.4** (13.2-28.7)
20-day	**7.19** (6.14-8.33)	**8.26** (7.05-9.58)	**10.0** (8.50-11.7)	**11.5** (9.67-13.4)	**13.5** (10.9-16.4)	**15.0** (11.8-18.7)	**16.6** (12.5-21.4)	**18.3** (12.9-24.3)	**20.8** (14.0-28.6)	**22.8** (14.9-32.0)
30-day	**8.95** (7.67-10.3)	**10.1** (8.62-11.6)	**11.9** (10.1-13.8)	**13.4** (11.4-15.6)	**15.5** (12.6-18.8)	**17.1** (13.5-21.1)	**18.7** (14.1-23.9)	**20.5** (14.5-27.0)	**22.8** (15.4-31.2)	**24.7** (16.1-34.4)
45-day	**11.1** (9.59-12.8)	**12.3** (10.6-14.2)	**14.2** (12.2-16.4)	**15.8** (13.4-18.3)	**18.0** (14.6-21.6)	**19.7** (15.5-24.1)	**21.3** (16.0-27.0)	**23.0** (16.3-30.1)	**25.1** (17.0-34.2)	**26.7** (17.5-37.2)
60-day	**13.0** (11.2-14.9)	**14.2** (12.2-16.3)	**16.1** (13.8-18.6)	**17.7** (15.1-20.5)	**20.0** (16.3-23.9)	**21.7** (17.1-26.5)	**23.4** (17.6-29.4)	**25.0** (17.8-32.7)	**27.0** (18.3-36.6)	**28.4** (18.6-39.3)

Table 5-2. ARI Intensity Estimates for New Haven

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches/hour)[1]										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**4.14** (3.34-5.09)	**4.98** (4.01-6.14)	**6.36** (5.09-7.86)	**7.51** (5.99-9.35)	**9.10** (6.97-11.8)	**10.3** (7.69-13.7)	**11.5** (8.34-15.9)	**12.9** (8.81-18.2)	**15.0** (9.73-21.8)	**16.6** (10.5-24.8)
10-min	**2.93** (2.36-3.61)	**3.53** (2.84-4.35)	**4.51** (3.62-5.57)	**5.32** (4.24-6.62)	**6.44** (4.94-8.38)	**7.29** (5.45-9.68)	**8.17** (5.91-11.3)	**9.17** (6.23-12.9)	**10.6** (6.89-15.5)	**11.8** (7.46-17.5)
15-min	**2.30** (1.85-2.83)	**2.77** (2.23-3.41)	**3.54** (2.84-4.38)	**4.18** (3.32-5.19)	**5.06** (3.88-6.57)	**5.72** (4.28-7.59)	**6.41** (4.63-8.84)	**7.19** (4.89-10.1)	**8.32** (5.41-12.1)	**9.24** (5.85-13.8)
30-min	**1.58** (1.28-1.95)	**1.91** (1.54-2.35)	**2.44** (1.95-3.01)	**2.88** (2.29-3.58)	**3.48** (2.67-4.53)	**3.94** (2.95-5.23)	**4.42** (3.19-6.09)	**4.95** (3.37-6.98)	**5.73** (3.73-8.36)	**6.36** (4.03-9.48)
60-min	**1.01** (0.813-1.24)	**1.22** (0.978-1.50)	**1.55** (1.25-1.92)	**1.83** (1.46-2.28)	**2.22** (1.70-2.88)	**2.51** (1.88-3.33)	**2.81** (2.03-3.88)	**3.16** (2.15-4.45)	**3.65** (2.37-5.32)	**4.05** (2.57-6.04)
2-hr	**0.657** (0.532-0.803)	**0.792** (0.641-0.969)	**1.01** (0.816-1.24)	**1.20** (0.958-1.48)	**1.45** (1.12-1.87)	**1.64** (1.23-2.16)	**1.84** (1.34-2.52)	**2.07** (1.41-2.89)	**2.40** (1.57-3.48)	**2.68** (1.70-3.97)
3-hr	**0.507** (0.413-0.618)	**0.611** (0.497-0.746)	**0.782** (0.633-0.956)	**0.922** (0.742-1.14)	**1.12** (0.865-1.44)	**1.26** (0.954-1.66)	**1.42** (1.04-1.94)	**1.60** (1.09-2.23)	**1.86** (1.22-2.69)	**2.09** (1.33-3.07)
6-hr	**0.324** (0.266-0.392)	**0.390** (0.319-0.473)	**0.498** (0.406-0.606)	**0.588** (0.476-0.719)	**0.712** (0.554-0.911)	**0.804** (0.611-1.05)	**0.902** (0.663-1.23)	**1.02** (0.698-1.41)	**1.19** (0.779-1.71)	**1.33** (0.850-1.95)
12-hr	**0.201** (0.166-0.241)	**0.242** (0.199-0.291)	**0.309** (0.253-0.373)	**0.364** (0.297-0.442)	**0.441** (0.345-0.561)	**0.498** (0.381-0.648)	**0.559** (0.413-0.757)	**0.631** (0.435-0.868)	**0.737** (0.485-1.05)	**0.827** (0.529-1.20)
24-hr	**0.119** (0.099-0.142)	**0.144** (0.120-0.172)	**0.186** (0.154-0.223)	**0.221** (0.181-0.266)	**0.268** (0.212-0.340)	**0.304** (0.234-0.394)	**0.342** (0.255-0.462)	**0.388** (0.268-0.531)	**0.458** (0.302-0.649)	**0.518** (0.332-0.748)
2-day	**0.066** (0.055-0.079)	**0.082** (0.068-0.097)	**0.108** (0.090-0.128)	**0.129** (0.106-0.155)	**0.158** (0.126-0.200)	**0.180** (0.140-0.233)	**0.204** (0.153-0.275)	**0.233** (0.162-0.317)	**0.279** (0.185-0.393)	**0.320** (0.206-0.458)
3-day	**0.048** (0.040-0.057)	**0.059** (0.050-0.070)	**0.078** (0.065-0.093)	**0.094** (0.078-0.112)	**0.115** (0.092-0.145)	**0.131** (0.102-0.169)	**0.148** (0.112-0.200)	**0.170** (0.118-0.231)	**0.205** (0.136-0.287)	**0.234** (0.151-0.335)
4-day	**0.039** (0.032-0.045)	**0.048** (0.040-0.056)	**0.063** (0.052-0.074)	**0.075** (0.062-0.089)	**0.092** (0.074-0.115)	**0.104** (0.082-0.134)	**0.118** (0.089-0.159)	**0.135** (0.094-0.183)	**0.162** (0.108-0.227)	**0.186** (0.120-0.264)
7-day	**0.026** (0.022-0.031)	**0.032** (0.027-0.037)	**0.041** (0.035-0.048)	**0.049** (0.041-0.058)	**0.059** (0.048-0.074)	**0.067** (0.053-0.085)	**0.076** (0.057-0.100)	**0.086** (0.060-0.115)	**0.102** (0.068-0.141)	**0.115** (0.075-0.163)
10-day	**0.021** (0.018-0.025)	**0.025** (0.021-0.030)	**0.032** (0.027-0.038)	**0.038** (0.032-0.044)	**0.045** (0.037-0.056)	**0.051** (0.040-0.065)	**0.057** (0.043-0.075)	**0.065** (0.045-0.086)	**0.076** (0.050-0.105)	**0.085** (0.055-0.120)
20-day	**0.015** (0.013-0.017)	**0.017** (0.015-0.020)	**0.021** (0.018-0.024)	**0.024** (0.020-0.028)	**0.028** (0.023-0.034)	**0.031** (0.025-0.039)	**0.035** (0.026-0.045)	**0.038** (0.027-0.051)	**0.043** (0.029-0.060)	**0.048** (0.031-0.067)
30-day	**0.012** (0.011-0.014)	**0.014** (0.012-0.016)	**0.017** (0.014-0.019)	**0.019** (0.016-0.022)	**0.022** (0.017-0.026)	**0.024** (0.019-0.029)	**0.026** (0.020-0.033)	**0.028** (0.020-0.037)	**0.032** (0.021-0.043)	**0.034** (0.022-0.048)
45-day	**0.010** (0.009-0.012)	**0.011** (0.010-0.013)	**0.013** (0.011-0.015)	**0.015** (0.012-0.017)	**0.017** (0.014-0.020)	**0.018** (0.014-0.022)	**0.020** (0.015-0.025)	**0.021** (0.015-0.028)	**0.023** (0.016-0.032)	**0.025** (0.016-0.034)
60-day	**0.009** (0.008-0.010)	**0.010** (0.008-0.011)	**0.011** (0.010-0.013)	**0.012** (0.010-0.014)	**0.014** (0.011-0.017)	**0.015** (0.012-0.018)	**0.016** (0.012-0.020)	**0.017** (0.012-0.023)	**0.019** (0.013-0.025)	**0.020** (0.013-0.027)

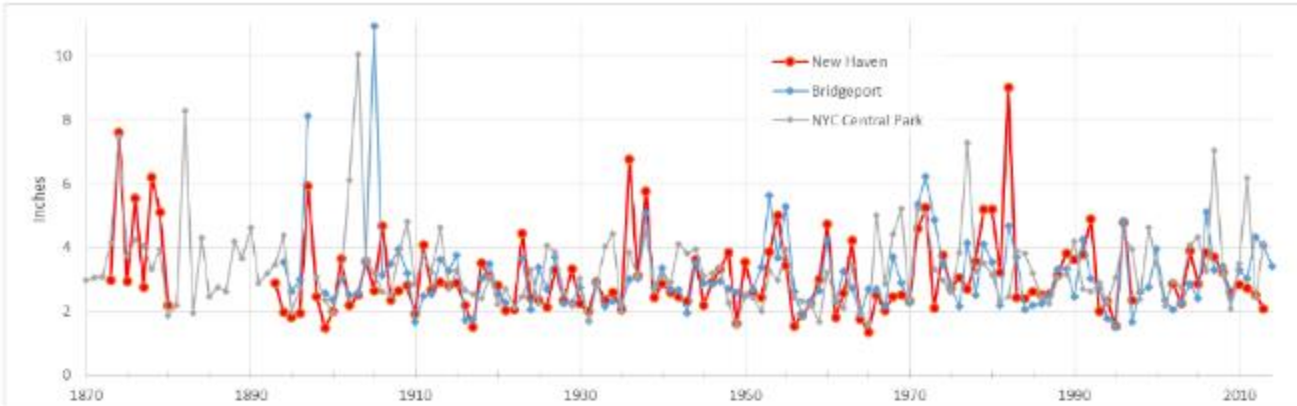

Figure 5-2. Annual series of one-day precipitation maxima

Figure 5-3 shows analogous hourly datasets for the same stations. While this figure shows record rainfall in Bridgeport in 2012, as well as an extreme event in New Haven that year, there is similarly no evidence of trends in the magnitude.

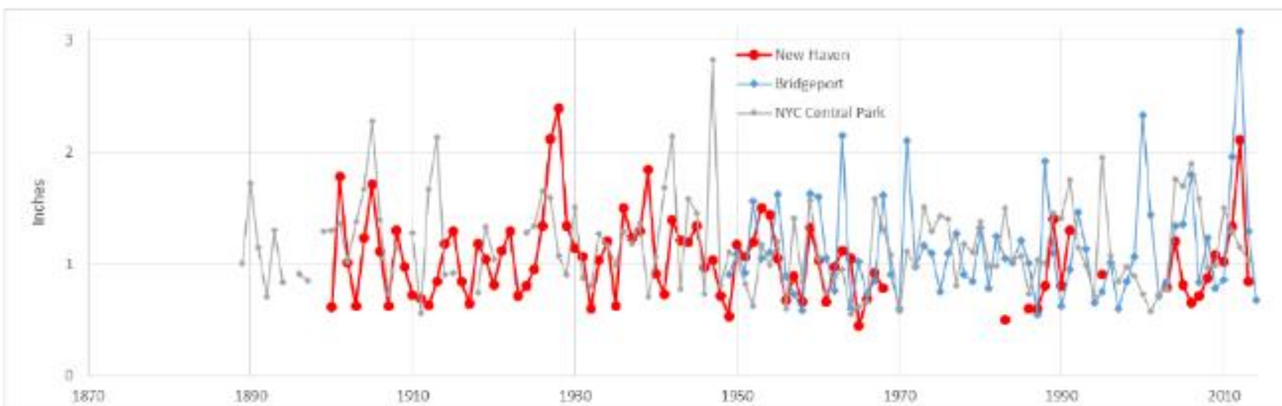

Figure 5-3. Annual series of hourly precipitation maxima

While these annual series of extreme precipitation for the region do not exhibit significant trends and support Atlas 14's assumption of stationarity of the historic data, the CDM Smith report notes others have recognized New England's climate has become wetter in recent decades (Horton et al, 2012). And, while Atlas 14 makes no projections to account for future climate change, the CDM Smith report estimated changes in extreme rainfall statistics using EPA's CREAT (Climate Resilience Evaluation and Awareness Tool) and SWMM-CAT (Storm Water Management Model Climate Adjustment Tool) software. Table 5-3 shows the projected percent increases in 24-hour New Haven rainfall estimates for the 2045-2074 period for three climate change scenarios. The table shows, for instance, that the 10-year 24-hour rainfall is estimated to increase by 7.5 percent under the Hot/Dry scenario. Thus, the Atlas 14 10-year 24-hour depth of 5.30 inches is projected to be 5.70 inches by 2060 (the midpoint of the forecast period).

Table 5-3. Projected Percent Increases in 24-Hour Extreme Rainfall Estimates Under Different Climate Change Scenarios (CDM Smith, 2017)

Scenario	5-Year	10-Year	50-Year
Warm/Wet	10.6	10.4	10.8
Hot/Dry	7.3	7.5	8.3
Median	3.8	4.0	6.1

For proposed drainage improvements, the City of New Haven has selected the 10-year 24-hour rainfall depth as their design storm, assuming the Hot/Dry climate change scenario identified above.

5.3 Watershed Vulnerability Assessment

The USACE Watershed Vulnerability Assessment (VA) Tool provides a nationwide, screening-level assessment of climate change vulnerability relating to the USACE mission, operations, programs and projects. Indicators are used to develop the vulnerability scores specific to each of the 200 watersheds within the contiguous United States and to each of the USACE business lines. The Weighted Order Weighted Average (WOWA) method is used to aggregate individual vulnerability indicators and their associated datasets into the watershed-scale vulnerability scores. The WOWA score combines indicators using a weighting technique to control how much an indicator with a small value can average out an indicator with a large value, thereby affecting perceived vulnerability. The VA tool is based on downscaled climate information and hydrology aggregated at the HUC 4 watershed level for selected indicator variables. The tool supports a qualitative identification of potential vulnerabilities.

The VA tool examines the vulnerability of projects within all USACE business lines using data for two scenarios and three epochs. The epochs include the current time period as the base period and the two future 30-year periods centered about the years 2050 (2035-2065) and 2085 (2070-2100). Within each future epoch, Global Climate Models are sorted by cumulative runoff projections and divided into two equal-sized groups that represent a Dry scenario and a Wet scenario. All results are given for each combination of scenario and future epoch: Dry-2050, Dry-2085, Wet-2050, and Wet-2085. The VA tool allows the user to explore dominant indicators and summarize vulnerability in several different ways for each scenario-epoch combination. For this study, the VA tool was used to assess the vulnerability of the Connecticut Coastal (HUC 0110) watershed with emphasis on the vulnerability indicators for the flood risk management business line.

Table 5-4 provides the names of selected indicators for the flood risk management business line and their importance weights within the VA tool's National Standard View, along with a brief description of each.

Table 5-4. Importance weight, name, and description of VA indicators for the flood risk management business line

Importance Weight	Name	Description
1.25	175C ANNUAL COV	Annual CV of unregulated runoff (cumulative). Long-term variability in hydrology: ratio of the standard deviation of annual runoff to the annual runoff mean. Includes upstream freshwater inputs (cumulative)
1.0	277 RUNOFF PRECIP	Percent change in runoff divided by percent change in precipitation. Median of: deviation of runoff from monthly mean times average monthly runoff divided by deviation of precipitation from monthly mean times average monthly precipitation
1.8	568C FLOOD MAGNIFICATION	Flood magnification factor (cumulative). Change in flood runoff: ratio of indicator 571C (monthly runoff exceeded 10% of the time, including upstream freshwater inputs) to 571C in the base period
1.4	568L FLOOD MAGNIFICATION	Flood magnification factor (local). Change in flood runoff: ratio of indicator 571l (monthly runoff exceeded 10% of the time, including upstream freshwater inputs) to 571C in the base period
1.75	590 URBAN 500 YR FLOODPLAIN AREA	Acres of urban area within the 500-year floodplain

The Vulnerability Assessment tool results indicate that the project is not relatively vulnerable to the impacts of climate change for the flood risk management business line, nor the seven other business lines evaluated in the VA tool. Table 5-5 lists the vulnerability scores of the flood risk management business line for HUC 0110, Connecticut Coastal, as well as the ranges of scores nationally and for the North Atlantic Division (NAD) and New England District (NAD) for all scenario-epoch combinations.

Table 5-5. Vulnerability Scores for HUC 0110 (Column 3) for the Flood Risk Management business line for each scenario-epoch combination nationally, NAD, and NAE

Business Line	Scenario - Epoch	WOWA Score	Range Nationally	Range in NAD	Range in NAE
Flood Risk Management	Base	41.29	37.30 - 70.89	39.08 - 49.57	39.08 - 42.35
	Dry – 2050	45.02	35.15 - 70.08	40.04 - 52.58	40.04 - 46.30
	Dry – 2085	46.19	35.66 - 69.10	40.01 - 53.37	40.01 - 48.77
	Wet – 2050	49.35	39.80 - 92.85	43.13 - 54.82	43.13 - 50.42
	Wet – 2085	52.86	40.86 - 86.71	43.12 - 56.91	43.12 - 53.79

The vulnerability scores for the flood risk management business line show that the study area is somewhat sensitive or has some change in the WOWA score by epoch and wet/dry scenario with a range of 45.02 to 52.86 for the scenario-epoch combinations in the VA tool. The WOWA scores increase with future epochs and in wetter climate scenarios. Figure 5-4 illustrates the changes in WOWA score by scenario-epoch combination for the NAD watersheds. The indicator contributions for HUC 0110 are shown to the right. The dominant indicator for all scenario-epoch combinations is flood magnification (568C FLOOD MAGNIFICATION), accounting for approximately half of each WOWA score.

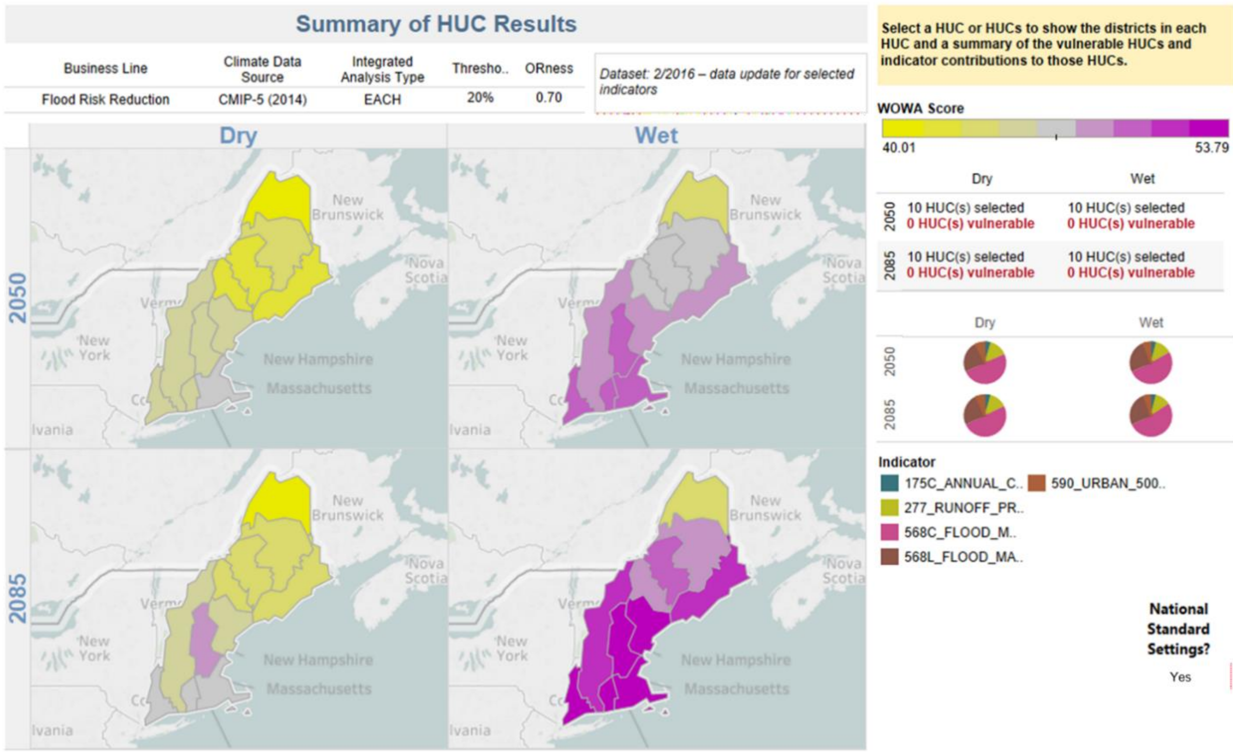

Figure 5-4. VA tool results summary for the flood risk management score of the Connecticut Coastal watershed compared to NAD

6.0 Coastal Storm Damage Reduction Measures Considered

Considering the information from the previous sections, various storm damage reduction measures were considered as part of the planning process. Each of the measures is discussed in the sections below. It must be understood that the measures discussed were for storm damage reduction for reducing economic impacts of storms and were not considered for life safety. Evacuation is the measure that must be used for life safety ahead of a significant storm event. The National Weather Service typically gives several days of storm warning and forecasts allowing the appropriate state and federal governmental agencies to set evacuation requirements. Due to a robust highway road system and short distance to high ground, evacuation is very viable.

6.1 Flood Wall

Floodwalls are typically constructed of concrete or steel and are vertical or nearly vertical (Figure 6-1). Floodwalls are constructed to reduce the frequency of flooding to the areas behind the wall. Floodwalls are effective at reducing flood potential but do not eliminate it since it is typically not cost effective to build flood walls to such an elevation that they will never get overtopped or overtopped only during the rarest events. The floodwall crest elevation in USACE projects is almost always selected based on an optimized construction/maintenance cost vs. benefits analysis. In addition to the cost of building such a structure the real world engineering considerations must be factored in and also the quality of life for the nearby residents. Floodwalls often block views, shade private property, separate communities, impact local hydrology, reduce wildlife mobility, etc.

6.2 Deployable Closure Structures

Deployable closure structures such as street gates are measures added to a line of coastal storm damage risk reduction across a road or driveway, which allows for unimpeded access across the alignment during normal day-to-day conditions. Operable floodgates can be either manually or automatically operated. Manually operated gates require the mobilization of personnel to physically go to the location of the gate and close it for storm conditions. With the gate in place, access to the flood side of the line of protection is impeded. Types of closure structures vary considerably. Examples include stoplogs, swing gates, miter gates, rolling gates, and trolley gates.

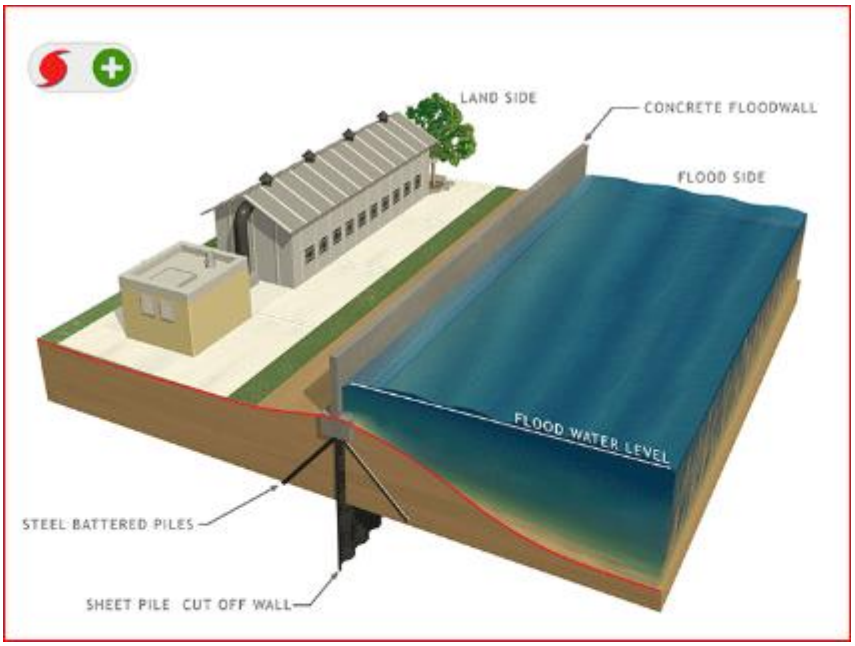

Figure 6-1. Floodwall example image

6.3 Floodproofing

Commercial structures that cannot be elevated may be considered for floodproofing techniques. There are two types of floodproofing—wet and dry. Wet floodproofing allows for water to enter and exit a structure at the same rate as the flood waters outside and the focus is on protecting the structure's service equipment and relocating materials stored below the flood risk management elevation. Wet floodproofing may be accomplished by installing openings for water passage, using flood-resistant construction and finishing materials in areas below the flood risk management elevation, and protecting service equipment. In contrast, dry floodproofing seals the exterior of a building below flood level to prevent the entry of flood waters. Because the walls are exposed to flood waters and the pressures they exert, dry floodproofing is practical for structures with walls constructed of flood-resistant materials and only where flood depths are low (typically no more than 2 to 3 feet). Dry floodproofing can be accomplished through the use of sealants and shields, the installation of a drainage system, and protection of service equipment.

7.0 Alternatives Evaluation

A final array of alternative plans for the study area was developed from the measures discussed in the previous section. Both structural and non-structural alternatives were investigated. Each of the alternatives is summarized briefly here. Please refer to the Main Report for a complete description of all alternatives.

Alternative 1: No Action

This alternative assumes no measures will be implemented and makes no changes to the current floodplain conditions. The No Action alternative serves as the future without project condition and the base condition to use as a comparison against all the other alternatives. Under this alternative it is assumed that present coastal storm risk will increase over time due to sea level change.

Alternative 2: Non-Structural Floodproofing

This alternative consists of making non-structural improvements to buildings within the study area, but does not include any measures to reduce the flood hazard. Thus, while damages to individual properties may be reduced, flooding is still expected to impact the rail, highway, and street infrastructure and corridors within the study area.

Alternative 3A: Existing Embankment

This alternative considers the use of deployable closure structures beneath the I-95 underpasses at Long Wharf Drive and Canal Dock Road to prevent floodwaters from propagating inland west of I-95. By sealing off these two water intrusion points, I-95 becomes a line of protection, limited by the elevation of its lowest section. Thus, this alternative manages and reduces risk up to a flood elevation of approximately 10 feet NAVD88, above which it is assumed that water would begin flooding across I-95. Assets east of I-95 were considered for floodproofing.

Alternative 3B: Enhanced Embankment

This alternative combines the deployable closure structures at the I-95 underpasses at Long Wharf Drive and Canal Dock Road in Alternative 3A with a 5,800 linear foot floodwall seaward of I-95 which extends from near the Howard Avenue overpass to 600 feet north of Canal Dock Road. The floodwall addition mitigates for the low elevation section of I-95, but will require two additional deployable closure structures at the I-95 North on and off ramps at Long Wharf Drive. For the feasibility study, the top of wall elevation for the floodwall and closure structures was assumed to be at elevation 15 feet NAVD88. This elevation was initially selected considering available tie-ins to high ground and future mean annual exceedance probability water levels under the intermediate and high sea level change scenarios. An elevation of 15 feet NAVD88 aligned well with both the 2074 1-percent mean annual exceedance probability water level under the intermediate sea level rise scenario (13.02 feet NAVD88), with some allowance for wave action, and the 2074 1-percent mean annual exceedance probability water level under the high sea level change scenario (14.92 feet NAVD88) alone. The elevation of 15 feet NAVD88 was confirmed through economic optimization. To manage and reduce risk to elevation 15 feet NAVD88, a closure structure at Brewery Street is also proposed to prevent against flanking. Assets east of I-95 were considered for floodproofing.

Alternative 4A: Shoreline Floodwall

Rather than make use of the I-95 embankment in Alternative 3B, this alternative consists of a 6,850 linear foot floodwall that reduces coastal storm risk within the same general area, but shifts the location of the wall away from I-95 and toward the shoreline. Instead of the deployable closure structures at the I-95 underpasses and off ramps, this alternative would require similar closure structures at the Long Wharf Park parking area, the Canal Dock Boathouse access, and crossing Long Wharf Drive to tie back into high ground. Constructed to an elevation of 15 feet NAVD88, a closure structure at Brewery Street would still be needed to prevent against flanking. Assets east of I-95 were considered for floodproofing.

Alternative 4B: Extended Shoreline Floodwall

This alternative would continue the shoreline floodwall in Alternative 4A northeast, extending it approximately 3,000 linear feet around properties within the Long Wharf Maritime Center. While the closure structure crossing Long Wharf Drive would no longer be needed, additional closure structures would have to cross East Street and Water Street. Top of wall elevations were again assumed to be at elevation 15 feet NAVD88 and a closure structure at Brewery Street would also be proposed to prevent against flanking.

In accordance with ER 1105-2-101, Risk Assessment for Flood Risk Management Studies, the flood risk management performance of each alternative was estimated as its ability to manage the flood hazard for the full range of possible events. The flood hazard was defined using the 2024 90-percent confidence limit annual exceedance probability water levels presented in Table 8-1. This flood hazard does not include wave effects such as runup and overtopping. The performance of each alternative is reported using two metrics, Annual Exceedance Probability (AEP) and Long-Term Exceedance Probability (LTEP). AEP represents the probability of any event equaling or exceeding the level of protection provided by each alternative in any given year. LTEP describes the probability of flooding over a specified period. LTEP accounts for the repeated annual exposure to flood risk over time. Table 7-1 presents the AEPs and the LTEPs over 10, 30, and 50 years for each alternative.

Table 7-1. Performance of alternatives described by 90% Confidence Limit AEP and LTEP

Alternative (Elevation, ft NAVD)	AEP	LTEP (Probability of Exceedance Over Indicated Time)		
		10 Years	30 Years	50 Years
Without Project (El. 8)	0.744	1.00	1.00	1.00
Non-Structural Floodproofing (El. 8)	0.744	1.00	1.00	1.00
Existing Embankment (El. 10.5)	0.104	0.67	0.96	1.00
Enhanced Embankment (El. 15)	0.008	0.08	0.21	0.33
Shoreline Floodwall (El. 15)	0.008	0.08	0.21	0.33
Extended Shoreline Floodwall (El. 15)	0.008	0.08	0.21	0.33

Note: AEP's correspond to 2024 90% confidence limit water levels. The Intermediate SLC scenario was used to approximate 2024 water levels.

8.0 Performance of the Selected Plan

Following selection of the Tentatively Selected Plan (TSP), Alternative 3B—Enhanced Embankment was evaluated further to define its performance, sensitivity to alternate sea level change scenarios, and detail residual risks.

Currently, the Long Wharf area landward of I-95 is vulnerable to coastal flooding when water levels exceed approximately elevation 8 feet NAVD88. The proposed floodwall and closure structure system would manage and reduce risk up to elevation 15 feet NAVD88. With the proposed project, the Long Wharf area would be subject to a 1 in 125 chance of being flooded by storm surge alone in any year but a 1 in 3 chance in 50 years. This likelihood of flooding does not include the effects of wave overtopping or interior flooding that might occur with the flood barriers closed. Wave overtopping and runoff from rainfall are discussed in Sections 11 and 12 and any increases in interior water levels are proposed to be mitigated through pumping.

Similarly, the same level of risk reduction will be afforded to critical and transportation infrastructure, including I-95 and the rail corridor, as these are situated landward of the floodwall and closure structure system.

It is recognized that potential flooding from greater events, with water levels exceeding 15 feet NAVD88, will remain. However, a taller floodwall would require adding considerable length to tie into higher ground. The probability of water levels exceeding the floodwall crest at elevation 15 feet NAVD88 will increase over time due to sea level rise. By the end of the project's 50 year period of economic analysis in 2074, the floodwall will have a 0.8-percent annual exceedance probability under the low sea level change scenario, a 1.2-percent annual exceedance probability under the intermediate sea level change scenario and a 3.5-percent annual exceedance probability under the high sea level change scenario. These levels of residual risk are considered to be low and tolerable.

Over the 100 year adaptation horizon, the likelihood of water levels exceeding the floodwall remains low under the low and intermediate sea level change scenarios. By 2124, the floodwall and closure structure system will have a 1.1-percent annual exceedance probability and a 2.7-percent annual exceedance probability under the low and intermediate sea level change scenarios, respectively. The risk increases markedly under the high sea level change scenario. By 2124, the structure crest at elevation 15 feet NAVD88 has approximately a 95-percent annual chance exceedance. While the floodwall and closure structure system will still manage and reduce risk, its probability of being overtopped is far greater.

The following figures illustrate how the 1-percent and 2-percent AEP water levels are projected to increase with sea level rise over the 50 year period of economic analysis and the 100 year adaptation horizon. Figure 8-1 shows the projected change in the 1-percent AEP (50-percent assurance) water level relative to the crest elevation of the proposed floodwall system. Elevation 15 ft NAVD88 is exceeded by the 1-percent AEP (50-percent assurance) water level under the high sea level change scenario in the year 2076 and is not exceeded under the low and intermediate scenarios within the planning horizon.

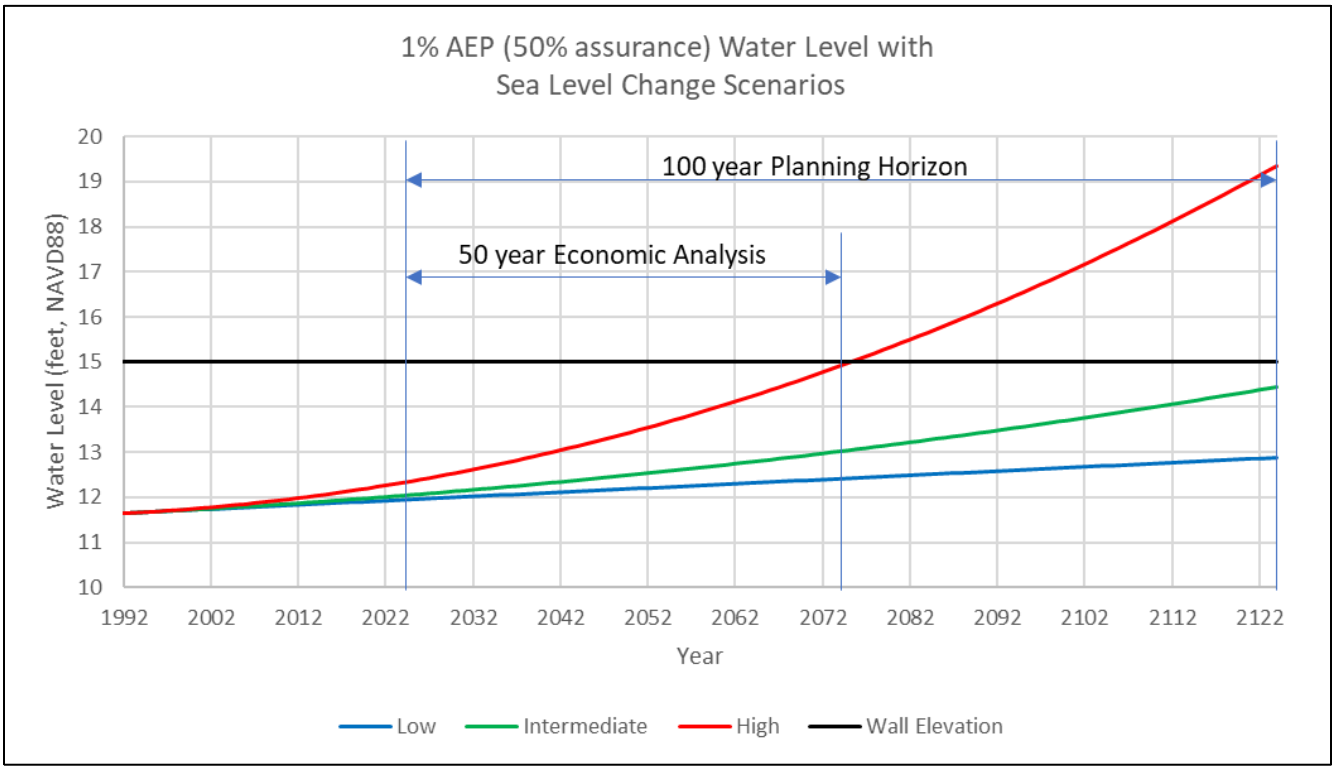

Figure 8-1. 1-percent AEP (50-percent assurance) water level with sea level change

Figure 8-2 shows the projected change in the 2-percent AEP (50-percent assurance) water level. In this case, the floodwall crest is exceeded by the 2-percent AEP (50-percent assurance) water level under the high sea level change scenario in the year 2091 and is not exceeded under the low and intermediate scenarios.

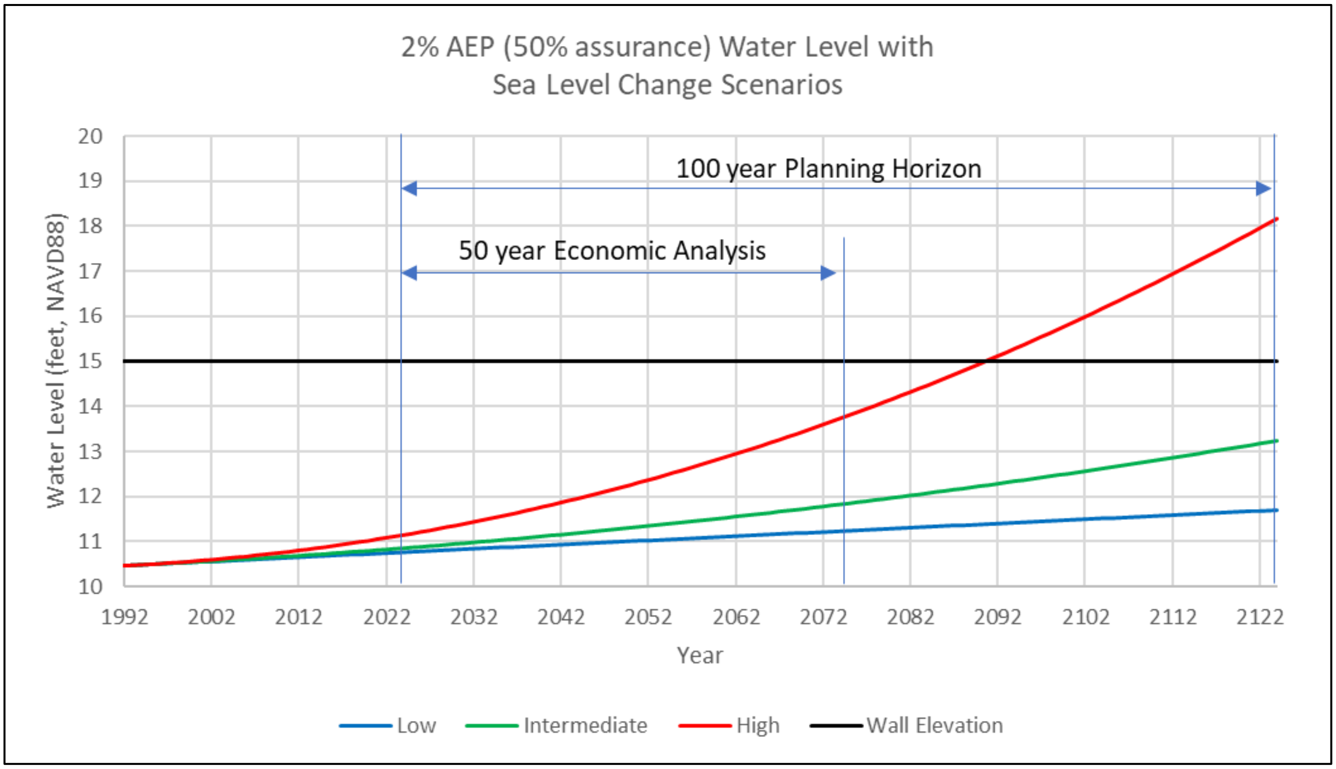

Figure 8-2. 2-percent AEP (50-percent assurance) water level with sea level change

A similar pair of curves are provided in Figures 8-3 and 8-4 for the 1- and 2-percent AEP water levels with 90-perecent assurance, respectively. Figure 8-3 shows the projected change in the 1-percent AEP (90-percent assurance) water level relative to the crest elevation of the proposed floodwall system. Elevation 15 ft NAVD88 is exceeded by the 1-percent AEP (90-percent assurance) water level under the high sea level change scenario in the year 2032, in the year 2055 in the intermediate scenario, and in the year 2092 under the low sea level change scenario. Figure 8-4 shows the projected change in the 2-percent AEP (90-percent assurance) water level. In this case, the floodwall crest is exceeded by the 2-percent AEP (90-percent assurance) water level under the high sea level change scenario in the year 2060 and the intermediate sea level change scenario in the year 2110. Elevation 15 ft NAVD88 is not exceeded by the 2-percent AEP (90-percent assurance) water level under the low sea level change scenario within the planning horizon.

Per ER 1105-2-101, the 90-percent assurance values were used for communicating project performance. The mean (50-percent assurance) values were used in the economic analysis.

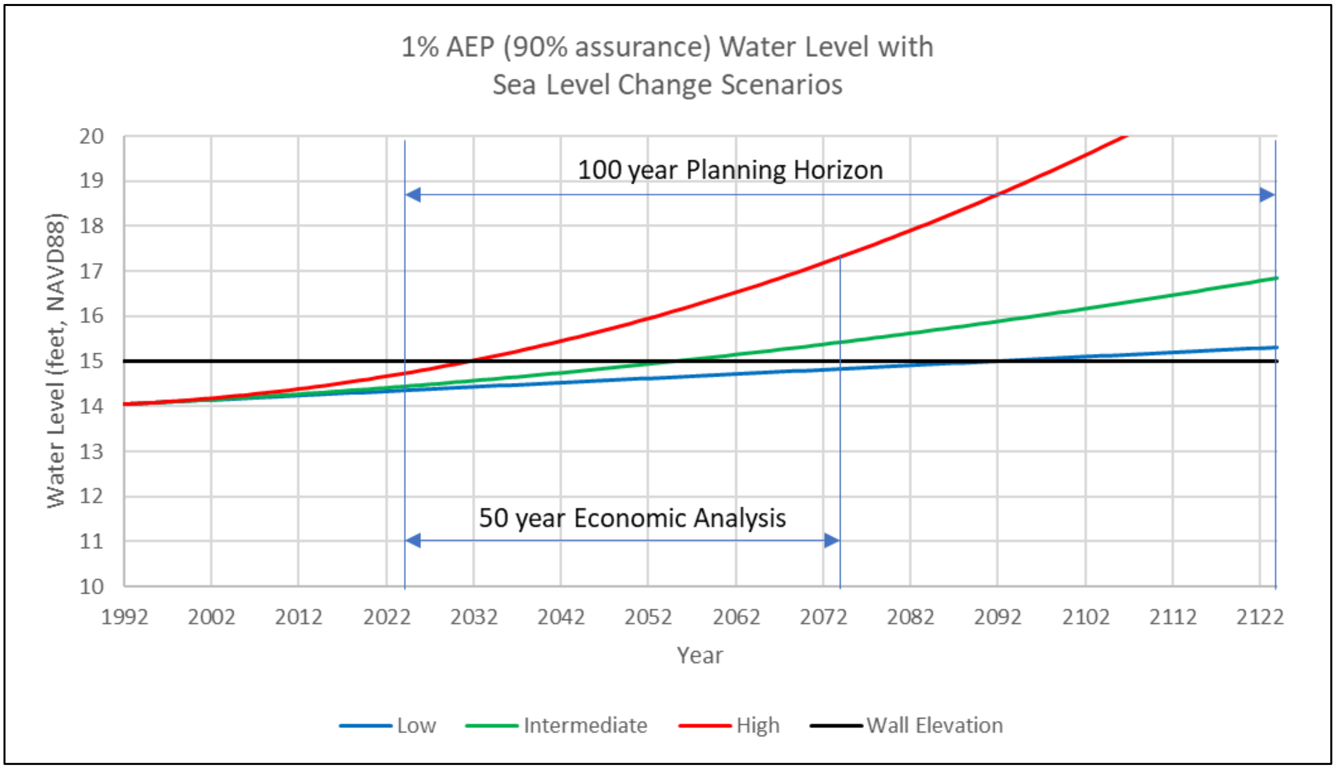

Figure 8-3. 1-percent AEP (90-percent assurance) water level with sea level change

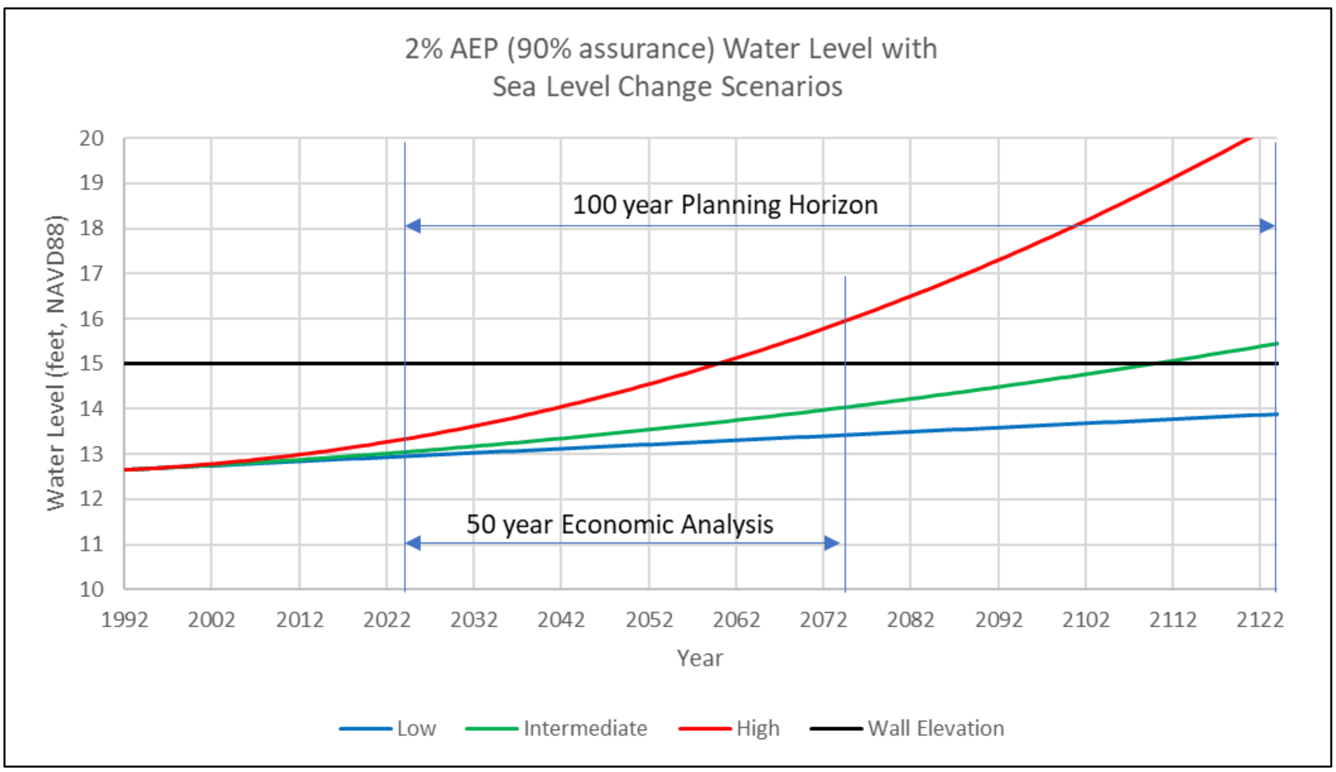

Figure 8-4. 2-percent AEP (90-percent assurance) water level with sea level change

9.0 Wave Forces on Vertical Walls

Several structural alternatives include vertical floodwall and closure structure measures. A characteristic of vertical structures like these is that the kinetic energy of a wave is stopped suddenly at the wall face. The energy is then reflected or translated by vertical motions of the water along the wall face. The upward component of this can result in the wave crests to rise and/or to double their deepwater wave height (non-breaking case). The downward component causes very high velocities at the base of the wall and horizontally away from the wall for half of a wavelength, thus causing erosion and scour. The forces exerted on vertical walls by reflected water waves were calculated for the vertical floodwall and closure structures proposed for this study.

The Goda method was selected for computing wave forces on vertical walls for this study. The Goda method assumes a trapezoidal shape for pressure distribution along the front of a vertical wall (Figure 9-1). The pressures at the top of the wall (labeled p_2), at the stillwater level (p_1), and at the toe of the wall (p_3) define the pressure distribution for the force calculation. For simplicity and conservatism it was assumed that all waves approached the structure normal (perpendicular) to it. This removed any considerations for wave obliqueness. Waves at the structure were also assumed to be depth-limited based on the stillwater elevation and local topography seaward of the wall, up to 4 feet in height. This upper bound on wave height was based on the NACCS statistical wave conditions and the results of GZA's nearshore wave modeling.

Wave loads were calculated for each of the floodwall and closure structure measures. The top of wall elevation for each measure was initially assumed to be two feet higher than the effective FEMA Base Flood Elevation (BFE). The water depth at the toe of the structure (h_w) and seaward of the structure (h_s) varied based on the local topography. Wave load calculations for each floodwall measure and closure structure are provided as an attachment to this appendix.

The hydrostatic and hydrodynamic (wave) forces computed using the Goda method were provided to the structural and geotechnical engineering disciplines to inform their designs of the floodwall and closure structures and their foundations. The need for scour protection at the toe of the floodwall will be evaluated during Pre-Construction Engineering and Design when the location and alignment of the wall is finalized.

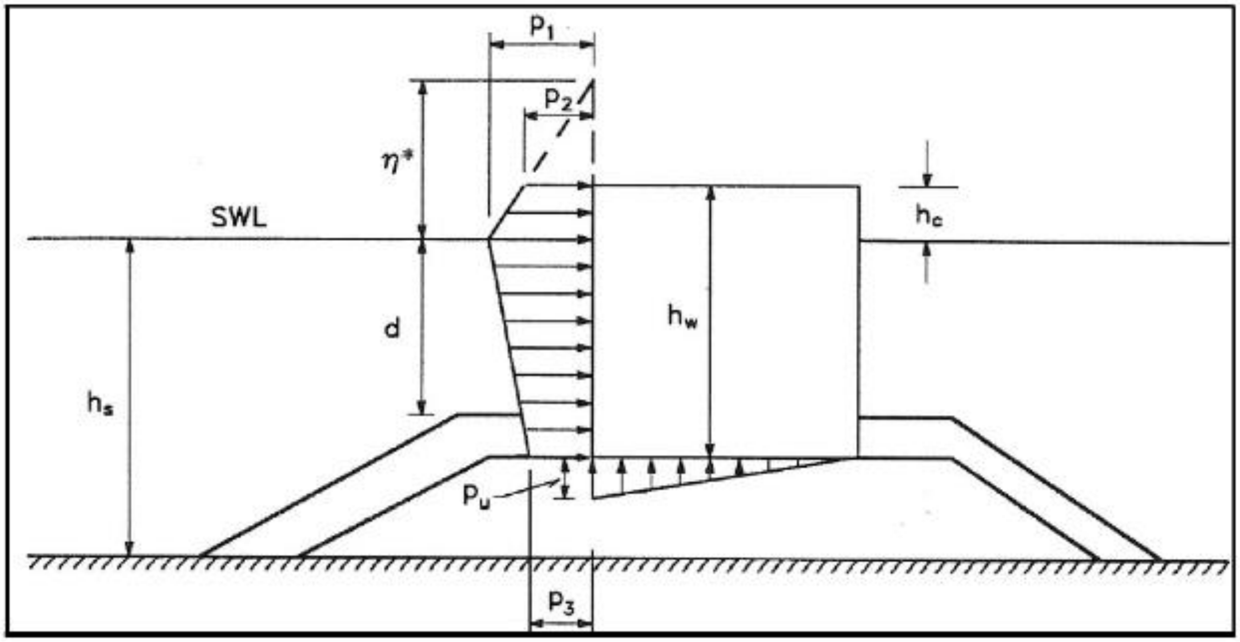

Figure 9-1. Definition sketch and pressure distribution for Goda formula

10.0 Wave Overtopping

There are two types of overtopping—flood overtopping and wave overtopping. Flood overtopping is when a continuous flow of a water elevation exceeds the floodwall crest. For overtopping by waves, or wave overtopping, the stillwater elevation approaches but does not exceed the crest elevation. Instead, waves approaching the structure run up its profile and overtop the crest. The wave action can form an equivalent discharge per linear distance of the structure and can lead to erosion, potential failure of the structure, and can create ponding areas on the land side of a project alignment if pump stations are not considered. Wave overtopping is a function of the stillwater elevation, wave height, period and direction, and structure slope, freeboard, and roughness. Following the Tentatively Selected Plan milestone, wave overtopping rates were estimated at the two closure structures at Canal Dock Road and Long Wharf Drive.

Wave overtopping was analyzed using the 90-percent confidence limit NACCS water levels. Waves at the structure were assumed to be depth-limited based on the water level and local topography seaward of the wall, up to 4 feet in height. This upper bound on wave height was based on the NACCS statistical wave conditions and the results of GZA's nearshore wave modeling. Overtopping rates were calculated across a range of annual exceedance probability water levels for the years 2024 and 2074, representing the start and end years of the project economic period of analysis, and the three sea level change scenarios.

Overtopping rates were calculated for the closure structures using the formulations provided in the European Overtopping (EurOtop) Manual for overtopping at plain vertical walls under impulsive conditions. The following two equations were used to estimate the wave overtopping rate, q, given on the non-dimensional freeboard, R_c/H_{m0}:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.0155\,(\frac{H_{m0}}{h\,s_{m-1,0}})^{0.5} exp(-2.2\,\frac{R_c}{H_{m0}}) \qquad \text{valid for } 0.1 < R_c/H_{m0} < 1.35 \qquad 7.9$$

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.0020\,(\frac{H_{m0}}{h\,s_{m-1,0}})^{0.5} (\frac{R_c}{H_{m0}})^{-3} \qquad \text{valid for } R_c/H_{m0} \geq 1.35 \qquad 7.10$$

where R_c is the structure freeboard, H_{m0} is the wave height at the structure, h is the water depth at the structure and $s_{m-1.0}$ is the wave steepness.

If the water level is higher than the structure crest, large overtopping quantities overflow the structure via flood, or overflow, overtopping. In this situation, the amount of water flowing to the land side of the structure is composed by a part which can be attributed to flood (overflow) overtopping and a part which can be attributed to wave overtopping, as illustrated in Figure 10-1.

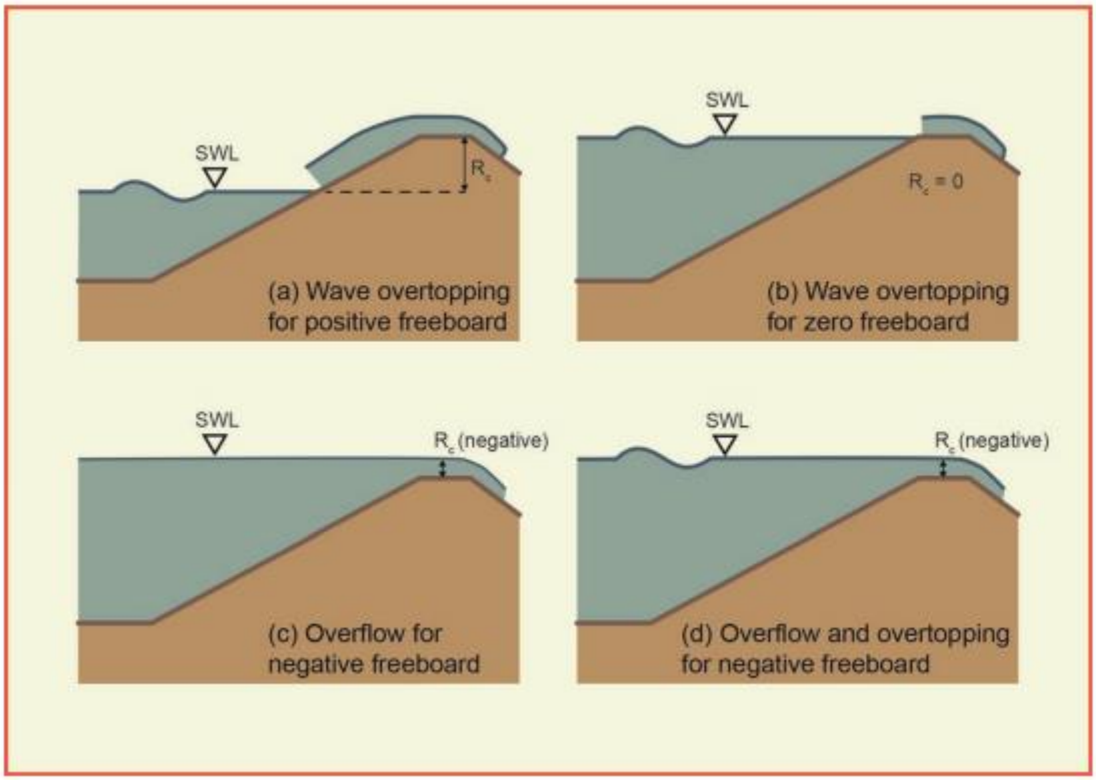

Figure 10-1. Wave overtopping and flood overtopping (overflow) for positive, zero, and negative freeboard (EurOtop)

The flood overtopping contribution was calculated using the weir formula:

$$q_{overflow} = 0.54 \cdot \sqrt{g \cdot |-R_c^3|} \qquad 5.20$$

In this case, the wave overtopping contribution is calculated assuming a zero freeboard condition exists. This wave overtopping component combined with the flood overtopping contribution total the overtopping hazard for a negative freeboard condition.

Results of the overtopping analysis for the Long Wharf Drive and Canal Dock Road closure structures are reported in Tables 10-1 and 10-2, respectively, for the 10-, 5-, 2-, and 1-percent annual exceedance probability water levels for the three sea level change scenarios for the years 2024 and 2074. Results are presented as overtopping rates per unit length of structure in cubic feet per second/foot and as overtopping flows, in cubic feet per second, equal to the overtopping rate multiplied by the structure length. Closure structure lengths were estimated at 60 feet and 190 feet for Long Wharf Drive and Canal Dock Road, respectively.

As illustrated in Figure 10-2, overtopping rates at both closure structures increase sharply when the stillwater elevation exceeds the crest at elevation 15 feet NAVD88. However, this is predicted to occur for only the most extreme storm events within the project economic lifecycle under the intermediate (1-percent AEP event) and high (1-percent and 2-percent AEP events) sea level change scenarios.

Table 10-1. Long Wharf Drive Closure Structure Overtopping Summary

Long Wharf Drive Closure Structure Overtopping Summary				
Low Sea Level Rise Scenario				
Year	**AEP**	**Water Level (FT, NAVD88)**	**Overtopping Rate (CFS/FT)**	**Overtopping Flow (CFS)**
2024	10%	10.46	0.01	1
	5%	11.43	0.1	5
	2%	12.96	0.8	49
	1%	14.36	1.8	109
2074	10%	10.93	0.03	2
	5%	11.90	0.2	12
	2%	13.43	1.1	68
	1%	14.83	2.3	139
Intermediate Sea Level Rise Scenario				
Year	**AEP**	**Water Level (FT, NAVD88)**	**Overtopping Rate (CFS/FT)**	**Overtopping Flow (CFS)**
2024	10%	10.55	0.01	1
	5%	11.52	0.1	6
	2%	13.05	0.9	54
	1%	14.45	1.9	114
2074	10%	11.53	0.1	6
	5%	12.50	0.5	29
	2%	14.03	1.5	92
	1%	15.43	3.4	203
High Sea Level Rise Scenario				
Year	**AEP**	**Water Level (FT, NAVD88)**	**Overtopping Rate (CFS/FT)**	**Overtopping Flow (CFS)**
2024	10%	10.84	0.03	2
	5%	11.81	0.2	11
	2%	13.34	1.1	65
	1%	14.74	2.2	133
2074	10%	13.43	1.1	68
	5%	14.40	1.9	111
	2%	15.93	5.3	316
	1%	17.33	13.4	805

Table 10-2. Canal Dock Road Closure Structure Overtopping Summary

Canal Dock Road Closure Structure Overtopping Summary				
Low Sea Level Rise Scenario				
Year	**AEP**	**Water Level (FT, NAVD88)**	**Overtopping Rate (CFS/FT)**	**Overtopping Flow (CFS)**
2024	10%	10.46	0.05	9
	5%	11.43	0.2	45
	2%	12.96	0.9	163
	1%	14.36	1.8	333
2074	10%	10.93	0.1	20
	5%	11.90	0.4	83
	2%	13.43	1.1	207
	1%	14.83	2.2	425
Intermediate Sea Level Rise Scenario				
Year	**AEP**	**Water Level (FT, NAVD88)**	**Overtopping Rate (CFS/FT)**	**Overtopping Flow (CFS)**
2024	10%	10.55	0.05	10
	5%	11.52	0.3	51
	2%	13.05	0.9	170
	1%	14.45	1.8	349
2074	10%	11.53	0.3	51
	5%	12.50	0.7	129
	2%	14.03	1.5	281
	1%	15.43	3.3	628
High Sea Level Rise Scenario				
Year	**AEP**	**Water Level (FT, NAVD88)**	**Overtopping Rate (CFS/FT)**	**Overtopping Flow (CFS)**
2024	10%	10.84	0.09	17
	5%	11.81	0.4	74
	2%	13.34	1.0	197
	1%	14.74	2.1	405
2074	10%	13.43	1.1	207
	5%	14.40	1.8	340
	2%	15.93	5.2	986
	1%	17.33	13.3	2535

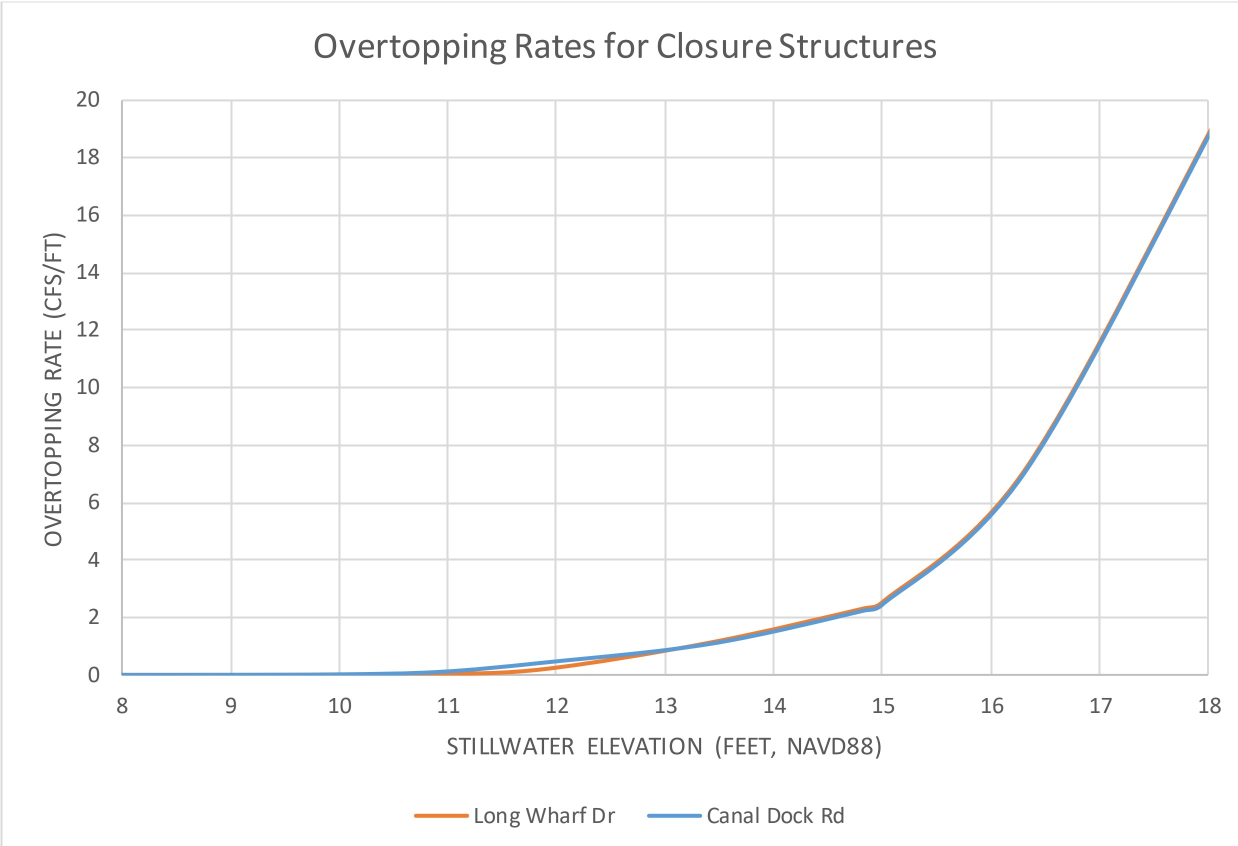

Figure 10-2. Wave overtopping rates by stillwater elevation for Long Wharf Drive and Canal Dock Road closure structures

11.0 Interior Drainage

Since the Tentatively Selected Plan (TSP) recommended a system of closure structures and floodwalls to manage flood risk from coastal storms and sea level rise, a limited interior drainage analysis was performed. Existing stormwater modeling conducted using the City of New Haven's SWMM (Storm Water Management Model) model was primarily reviewed. The SWMM model is approved for use in the USACE engineering community. The SWMM software typically analyzes urban stormwater hydrology and can route stormwater through a system of pipes, pumps, and outfalls. The primary focus for the feasibility phase was to quantify the need for mitigation features such as pumps which can be designed in detail during the Pre-Construction Engineering and Design (PED) phase.

EM 1110-2-1413 Hydrologic Analysis of Interior Areas references that if flooding within the interior area increases beyond what has occurred naturally, a relief system, such as pumps, should be recommended to mitigate for any increases in water level within the interior area. For the TSP, the line-of-protection was defined as the closure structure and floodwall system at elevation 15 feet NAVD88, which excludes coastal flood water originating from the exterior, but does not alleviate flooding that may subsequently occur from interior runoff. The interior area was defined as the interior watershed behind the line-of-protection, and included not only the Long Wharf study area but much of the Downtown watershed. This approximately 800-acre drainage area is shown in Figure 11-1 and generally coincides with the Downtown/Long Wharf storm sewershed studied in the Downtown Stormwater Modeling Project Final Report, prepared in 2017 by CDM Smith for the City of New Haven.

Figure 11-1. Downtown/Long Wharf sewershed

For this study, an order of magnitude discharge for the watershed was initially estimated using the rational method. This calculation conservatively assumes that the full 800-acre watershed area is contributing flow at the same time and does not account for storage or conveyance via the stormwater sewers. Rainfall intensity was obtained from Atlas 14. A few times of concentration and runoff coefficients were tested for sensitivity. These calculations are presented in Table 11-1. Peak discharges range from approximately 650 to 1400 cfs.

Table 11-1. Rational Method Discharge Estimates

Return Period (yr)	Time of Concentration (hr)	C	I (in)	I (in/hr)	A (acres)	Q (cfs)
10	1	0.95	1.85	1.85	800	1406
10	2	0.95	1.85	0.925	800	703
10	1	0.9	1.85	1.85	800	1332
10	2	0.9	1.85	0.925	800	666
5	1	0.95	1.57	1.57	800	1193

The 2017 Downtown Stormwater Modeling Project noted that intense, short duration rainfall is the principal cause of flooding in New Haven's drainage system, as the time of concentration is well under an hour for most local streets, and about an hour for the complete Downtown/Long Wharf system. Runoff from intense rainfall can also overwhelm catch basin inlet capacity, and can carry debris that obstructs catch basin inlets. Flow entering the drainage system can exceed its conveyance capacity due to inadequately sized infrastructure. The CDM Smith report provided a great level of detail for the 10-year frequency 24-hour rainfall event, the design rainfall event for the City of New Haven, and while it did not evaluate other frequency rainfall events, it did highlight that the sewer infrastructure alone cannot handle intense rainfall events. For instance, it estimated that the simulated 10-year peak discharges are one and one-half to more than twice the full pipe flow capacities for key pipes within the study area. While the model's downstream tidal hydrograph was adjusted for 0.9 feet of sea level rise, approximating 2066 tidal conditions under the intermediate sea level change scenario, the modeled rainfall event was not paired with a coastal storm event.

In 2020, USACE coordinated with the City of New Haven to run a historic nor'easter, adjusted for sea level rise, with their design 10-year frequency 24-hour rainfall event in the SWMM model. This was done to assess the pumping needs that might be expected with a coincident coastal storm and rainfall event. The December 1992 nor'easter was selected as it was a long duration event, with storm surge exceeding 2 feet over 6 tidal cycles, and a peak water level of 8.4 feet NAVD88. While higher water levels have occurred in New Haven Harbor, it was thought that the longer duration of elevated tidal conditions at the stormwater outfalls would have a greater effect on inland flooding and pumping needs. Tidal conditions were adjusted further to account for future changes due to sea level rise. A sea level rise component of 1.37 feet, the projected rise by 2074 under the intermediate sea level change scenario, was added to the 1992 storm hydrograph. As a conservative assumption, the peak rainfall and peak storm surge coincided in the SWMM model run. Results showed that a 900 cfs pump station would be capable of removing stormwater from the sewershed without flooding low elevation areas that are known to be problematic under coincident rainfall-surge events.

While this analysis was limited to specific rainfall and surge events and a more comprehensive evaluation of coincident storm surge and rainfall frequency events will be needed in the PED phase, this analysis demonstrated a need for pumping to provide relief to the interior area. A detailed assessment of rainfall runoff and pump size is anticipated in PED, as is a closer look at storm coincidence. Higher intensity, shorter duration events should be studied. In a scenario where the floodwall and closure structure system overtops, a detailed assessment of the timing of an overtopping event versus the opening and draining of the area will also be examined. In addition to pumping alone, storage in combination with a smaller pump station should be evaluated for cost effectiveness.

12.0 Climate Risk

As discussed in Sections 4 and 5, the study area is most vulnerable to sea level rise, increases in precipitation frequency and intensity, and increases in air temperature. Per guidance in ECB 2018-14, Table 12-1 identifies risks resulting from changing climate conditions in the future. The table shows the major project feature, the trigger event (climate variable that causes the risk), the hazard (resulting dangerous environmental condition), the harms (potential damage to the project or changed project output), and a qualitative assessment of the likelihood and uncertainty of this harm. Note that not all impacts of climate change will result in increased risk.

Project benefits may change as a result of climate change due to sea level change. In addition, project benefits may be impacted by climate change due to inland hydrology. Changes to benefits due to climate change may occur due to increases in flooding produced by sea level rise, or flooding produced by a combination of precipitation and sea level rise. There may be positive impacts to the project from increased air temperatures.

Table 12-1. Climate Risk Register

Feature or Measure	Trigger	Hazard	Harm	Qualitative Likelihood
Floodwall	Increased sea level	Increased water levels and wave heights seaward of the floodwall	Increased SLR may increase frequency and magnitude of water level and wave loading on floodwall. Risk reduction level decreases while residual risk increases.	Likely
Closure Structures	Increased sea level	Increased water levels and wave heights seaward of closure structures	Increased SLR may increase frequency of structure closure, increasing operational costs. Frequency and magnitude of water level and wave loading may increase. Risk reduction level decreases while residual risk increases.	Likely

Feature or Measure	Trigger	Hazard	Harm	Qualitative Likelihood
Pump Station	Increased sea level	Increased water levels to pump against	Increased O&M costs associated with running pumps for a longer duration and with higher head differentials.	Likely
Pump Station, Elevated Gravity Inlet Piping	Increased extreme precipitation	Future flood volumes may be larger than present	Larger flood volumes may not be adequately captured by elevated gravity inlet piping and pumps. Water that cannot be pumped from interior may reduce project benefits or cause nuisance flooding. Current pump size may be able to handle increased water levels at a higher energy cost (longer pumping duration)	Somewhat Likely
Pump Station, Elevated Gravity Inlet Piping	Increased air temperatures	Increased evapotranspiration or drought	Decrease in flow volumes entering the elevated gravity inlet piping and through the pump station	Likely

13.0 Selected Water Levels

This section summarizes the water levels that were selected for use in the feasibility study. Water levels from the NACCS study were adjusted for the intermediate sea level change scenario for use in the economic analysis. Annual exceedance probability water levels for the start and end years of the period of analysis are provided in Table 13-1.

Table 13-1. Mean Annual Exceedance Probability Water Levels

NACCS Save Point 8134	Mean Annual Exceedance Probability Water Level (feet, NAVD88)								
Intermediate SLC									
	99%	50%	20%	10%	5%	2%	1%	0.5%	0.2%
2024	5.74	6.65	7.85	8.72	9.59	10.85	12.04	13.49	15.49
2074	6.72	7.63	8.83	9.70	10.57	11.83	13.02	14.47	16.47

When reporting performance, annual exceedance probability water levels are to be reported to the 90-percent confidence limit, per ER 1105-2-101. Table 13-2 contains the 90-percent confidence limit annual exceedance probability water levels to be used for design and communicating performance.

Table 13-2. 90% Confidence Limit Annual Exceedance Probability Water Levels

Annual Exceedance Probability	99%	50%	20%	10%	5%	2%	1%	0.5%	0.2%
Water Level (feet, NAVD88)	7.61	8.45	9.62	10.55	11.52	13.05	14.45	15.95	17.95

14.0 Summary and Conclusions

The Water Management Section’s coastal assessment reviewed available water level and wave data and recommended water levels to be used for the formulation and design of plan alternatives and as input to the economic analysis for the Tentatively Selected Plan. The water levels provided were extracted from the NACCS study and adjusted for anticipated changes due to sea level rise. Wave heights from the NACCS and the GZA Long Wharf study were used to determine the hydrodynamic wave loads for alternatives with vertical floodwalls and closure structures. These wave forces were provided to the structural and geotechnical engineering disciplines as input to the designs of the floodwalls and closure structures and their foundations. Overtopping discharges of the proposed closure structures were calculated and a stormwater evaluation by the City of New Haven was reviewed to inform interior drainage needs and pump sizing.

15.0 References

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ATTACHMENT A

WAVE LOADS ON VERTICAL WALLS

Shoreline Floodwall, south of Long Wharf Pier
Existing BFE: 13 FT NAVD88
Proposed Crest Elevation: 15 FT NAVD88

2074 0.5% AEP WSEL: 14.39 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
7	8	7.39	243.45	218.70	213.47	1829.26
8	7	6.39	248.05	222.83	221.55	1644.02
9	6	5.39	252.83	227.12	229.97	1447.53
10	5	4.39	220.66	194.46	204.37	1059.55
11	4	3.39	173.78	147.05	163.84	670.12
12	3	2.39	124.99	97.72	119.93	360.60

2074 1% AEP WSEL: 12.94 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
7	8	5.94	250.18	164.29	225.30	1839.10
8	7	4.94	245.67	158.11	225.29	1579.16
9	6	3.94	199.80	110.51	186.54	1080.69
10	5	2.94	181.50	90.53	173.61	655.42
11	4	1.94	102.38	9.46	99.01	310.54
12	3	0.94	50.62	0.00	49.81	99.35

2074 2% AEP WSEL: 11.75 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
7	8	7.39	237.09	98.44	218.17	1626.49
8	7	6.39	190.87	49.49	178.81	1083.74
9	6	5.39	142.78	0.00	136.14	615.53
10	5	4.39	92.70	0.00	89.95	310.47
11	4	3.39	40.55	0.00	40.03	96.11
12	3	2.39				

2074 5% AEP WSEL: 10.49 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
7	8	7.39	178.55	0	168.04	1007.45
8	7	6.39	129.95	0	124.48	609.81
9	6	5.39	79.35	0	77.34	295.66
10	5	4.39	26.83	0	26.76	73.62
11	4	3.39				
12	3	2.39				

Shoreline Floodwall, north of Long Wharf Pier
Existing BFE: 13 FT NAVD88
Proposed Crest Elevation: 15 FT NAVD88

2074 0.5% AEP WSEL: 14.39 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
8	7	16.39	495.24	467.26	444.16	3294.94
8	7	13.39	496.43	468.39	444.65	3301.04
8	7	11.39	493.55	465.67	441.70	3280.68
8	7	6.39	309.09	283.87	276.07	2050.42
7	8	17.39	454.53	428.85	400.53	3428.86
7	8	9.39	453.56	427.94	398.08	3415.65
7	8	8.39	404.07	378.96	354.48	3041.68
6	9	17.39	429.23	404.99	371.34	3612.82
5	10	17.39	412.23	388.95	350.01	3823.08

2074 1% AEP WSEL: 12.94 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
8	7	14.94	603.15	488.11	554.80	3984.13
8	7	11.94	595.40	481.84	547.14	3931.65
8	7	9.94	582.50	471.39	534.96	3845.63
8	7	4.94	245.67	158.11	225.29	1579.16
7	8	15.94	520.42	421.15	470.44	3912.68
7	8	7.94	409.29	318.53	368.85	3060.73
7	8	6.94	346.05	258.26	311.75	2576.11
6	9	15.94	472.19	382.13	419.21	3973.14
5	10	15.94	442.24	357.89	385.47	4110.17

2074 2% AEP WSEL: 11.75 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
8	7	13.75	803.93	562.01	754.79	5142.26
8	7	10.75	781.14	546.08	732.88	4995.53
8	7	8.75	680.70	464.60	638.35	4334.32
8	7	3.75	190.87	49.49	178.81	1083.74
7	8	14.75	625.18	437.05	576.95	4581.19
7	8	6.75	365.17	214.90	336.21	2608.40
7	8	5.75	296.37	153.19	272.78	2082.26
6	9	14.75	532.88	372.52	483.12	4392.29
5	10	14.75	480.73	336.06	428.03	4394.32

2074 5% AEP WSEL: 10.49 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
8	7	12.49	719.09	418.81	689.77	4320.00
8	7	9.49	740.43	431.23	709.92	4447.77
8	7	7.49	673.58	326.92	645.64	3898.57
8	7	2.49	129.95	0.00	124.48	609.81
7	8	13.49	840.51	489.52	792.65	5849.06
7	8	5.49	320.24	95.39	301.50	2022.18
7	8	4.49	242.00	34.24	227.80	1442.75
6	9	13.49	657.32	382.83	609.17	5188.82
5	10	13.49	550.50	320.62	501.19	4851.27

Shoreline Floodwall, north of Long Wharf Pier
Existing BFE: 16 FT NAVD88
Proposed Crest Elevation: 18 FT NAVD88

2074 0.5% AEP WSEL: 14.39 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
5	13	17.39	412.23	274.44	350.01	4818.17
6	12	17.39	429.23	285.76	371.34	4648.93
7	11	12.39	459.92	306.19	404.25	4575.94
7	11	10.39	457.18	304.37	401.45	4547.25
8	10	15.39	496.12	330.29	444.76	4497.76
10	8	15.39	677.36	450.95	629.19	4904.48
10	8	14.39	675.25	449.54	627.04	4888.76
10	8	9.39	642.48	427.73	595.79	4649.73
11	7	13.39	828.35	551.47	782.52	5221.02
11	7	9.39	856.37	570.12	808.31	5396.42
12	6	13.39	700.37	466.27	673.05	3747.03
12	6	9.39	728.38	484.91	699.57	3896.41
12	6	4.39	288.97	85.87	277.35	1353.33

2074 1% AEP WSEL: 12.94 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
5	13	15.94	442.24	235.04	385.47	4999.57
6	12	15.94	472.19	250.96	419.21	4922.77
7	11	10.94	514.26	273.32	463.96	4897.90
7	11	8.94	481.31	248.47	433.91	4564.55
8	10	13.94	601.61	319.75	553.20	5183.44
10	8	13.94	767.25	407.78	730.51	5174.52
10	8	12.94	773.70	411.21	736.51	5217.81
10	8	7.94	742.26	337.96	705.96	4861.85
11	7	11.94	652.43	346.75	631.66	3773.51
11	7	7.94	621.40	282.93	601.35	3474.03
12	6	11.94	524.45	278.74	516.36	2521.24
12	6	7.94	493.42	224.66	485.70	2276.93
12	6	2.94	272.37	0.00	268.04	943.08

2074 2% AEP WSEL: 11.75 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
5	13	14.75	480.73	202.53	428.03	5202.21
6	12	14.75	532.88	224.50	483.12	5287.83
7	11	9.75	600.76	253.10	553.58	5409.87
7	11	7.75	446.30	138.68	411.02	3864.21
8	10	12.75	798.32	336.33	749.35	6447.67
10	8	12.75	622.66	262.32	604.83	3839.61
10	8	11.75	629.42	265.17	611.34	3881.26
10	8	6.75	548.26	114.37	532.24	3016.16
11	7	10.75	508.48	214.22	502.19	2637.42
11	7	6.75	420.28	87.67	415.01	1900.59
12	6	10.75				
12	6	6.75				
12	6	1.75				

2074 5% AEP WSEL: 10.49 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
5	13	13.49	550.50	167.70	501.1913	5583.744
6	12	13.49	657.32	200.24	609.1687	6063.424
7	11	8.49	694.48	169.42	654.2361	5597.454
7	11	6.49	418.34	4.59	393.943	3005.551
8	10	11.49	725.93	221.14	696.2256	5326.831
10	8	11.49	469.97	143.17	466.1834	2531.675
10	8	10.49	477.08	145.33	473.2217	2569.967
10	8	5.49				
11	7	9.49				
11	7	5.49				
12	6	9.49				
12	6	5.49				
12	6	0.49				

I-95 Embankment Floodwall
Existing BFE: 11 FT NAVD88
Proposed Crest Elevation: 13 FT NAVD88

2074 1% AEP WSEL: 12.94 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
9	4	4.94	261.58	258.87	244.27	1012.15
9	4	3.94	199.80	197.20	186.54	772.99
10	3	4.94	302.83	299.68	287.87	886.41
10	3	2.94	152.07	149.41	144.51	445.01
10	3	1.94	97.97	95.38	93.09	286.65
11	2	4.94	442.57	437.98	428.15	871.02
11	2	2.94	175.31	172.25	169.57	344.96
11	2	1.94	102.38	99.67	99.01	201.40
12	1	4.94	365.97	362.17	360.19	363.14
12	1	1.94	145.49	141.64	143.17	144.29

2074 2% AEP WSEL: 11.75 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
9	4	3.75	209.70	149.95	199.98	788.08
9	4	2.75	142.78	87.31	136.14	527.32
10	3	3.75	283.83	202.97	275.46	793.64
10	3	1.75	92.70	36.11	89.95	240.33
10	3	0.75	38.69	0.00	37.54	90.88
11	2	3.75	286.86	205.13	283.23	521.28
11	2	1.75	147.81	57.57	145.93	238.52
11	2	0.75	40.55	0.00	40.03	55.56
12	1	3.75				
12	1	0.75				

2074 5% AEP WSEL: 10.49 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
9	4	2.49	158.22	21.90	154.23	458.84
9	4	1.49	79.35	0.00	77.34	216.32
10	3	2.49	192.66	26.67	191.07	369.28
10	3	0.49	26.63	0.00	26.41	46.42
10	3	-0.51				
11	2	2.49				
11	2	0.49				
11	2	-0.51				
12	1	2.49				
12	1	-0.51				

Long Wharf Drive Closure
Existing BFE: 11 FT NAVD88
Proposed Crest Elevation: 13 FT NAVD88

2074 1% AEP WSEL: 12.94 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
8	5	4.94	245.67	243.12	225.92	1177.93

2074 2% AEP WSEL: 11.75 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
8	5	3.75	190.87	136.49	178.81	897.76

2074 5% AEP WSEL: 10.49 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
8	5	2.49	129.95	17.99	124.48	502.44

Canal Dock Road Closure
Existing BFE: 11 FT NAVD88
Proposed Crest Elevation: 13 FT NAVD88

2074 1% AEP WSEL: 12.94 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
7	6	5.94	250.18	247.68	225.30	1427.11

2074 2% AEP WSEL: 11.75 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
7	6	4.75	237.09	183.77	218.17	1344.28

2074 5% AEP WSEL: 10.49 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
7	6	3.49	178.55	68.80	168.04	915.23

Brewery Street Closure
Existing BFE: Currently outside FEMA floodplain, assumed 11 FT NAVD88
Proposed Crest Elevation: 13 FT NAVD88

2074 1% AEP WSEL: 12.94 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
12	1	0.94	50.62	47.86	49.81	50.16

2074 2% AEP WSEL: 11.75 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
12	0	0	0	0	0	0

2074 5% AEP WSEL: 10.49 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
12	0	0	0	0	0	0

Water Street Closure
Existing BFE: 13 FT NAVD88
Proposed Crest Elevation: 15 FT NAVD88

2074 1% AEP WSEL: 12.94 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
10	5	2.94	152.07	61.00	144.51	655.42

2074 2% AEP WSEL: 11.75 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
10	5	1.75	192.7090.87	0	89.95	310.47

2074 5% AEP WSEL: 10.49 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
10	5	0.49	26.63	0	26.41	73.05

East Street Closure
Existing BFE: 13 FT NAVD88
Proposed Crest Elevation: 15 FT NAVD88

2074 1% AEP WSEL: 12.94 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
9	6	3.94	199.80	110.51	186.54	1080.70

2074 2% AEP WSEL: 11.75 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
9	6	2.75	142.78	0	136.14	615.53

2074 5% AEP WSEL: 10.49 FT NAVD88

Ground Elevation (ft, NAVD88)	hw (ft)	hs (ft)	p1 (lf/sf)	p2 (lf/sf)	p3 (lb/sf)	F (lb/ft)
9	6	1.49	79.35	0	77.34	295.66