PAWCATUCK RIVER, RHODE ISLAND
COASTAL STORM RISK MANAGEMENT
FEASIBILITY STUDY

APPENDIX C

COASTAL ENGINEERING

September 22, 2016

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1.0 Study Overview

The Pawcatuck River Coastal Study investigated the feasibility of various storm damage reduction measures along the southern coast of Rhode Island from the Connecticut border to the entrance of Narraganset Bay at Point Judith. The coastline was approximately 20 miles long when measured in a straight-line. The actual mileage of coastline due to the curved beaches and headlands as well as the several inlets and coastal lagoons was much greater. The location of the coastline can be seen in Figure 1-1. Within the study area are the towns of Westerly, Charlestown, South Kingstown, and Narragansett, all of which are within the southern half of Washington County, and often referred to as South County by local residents.

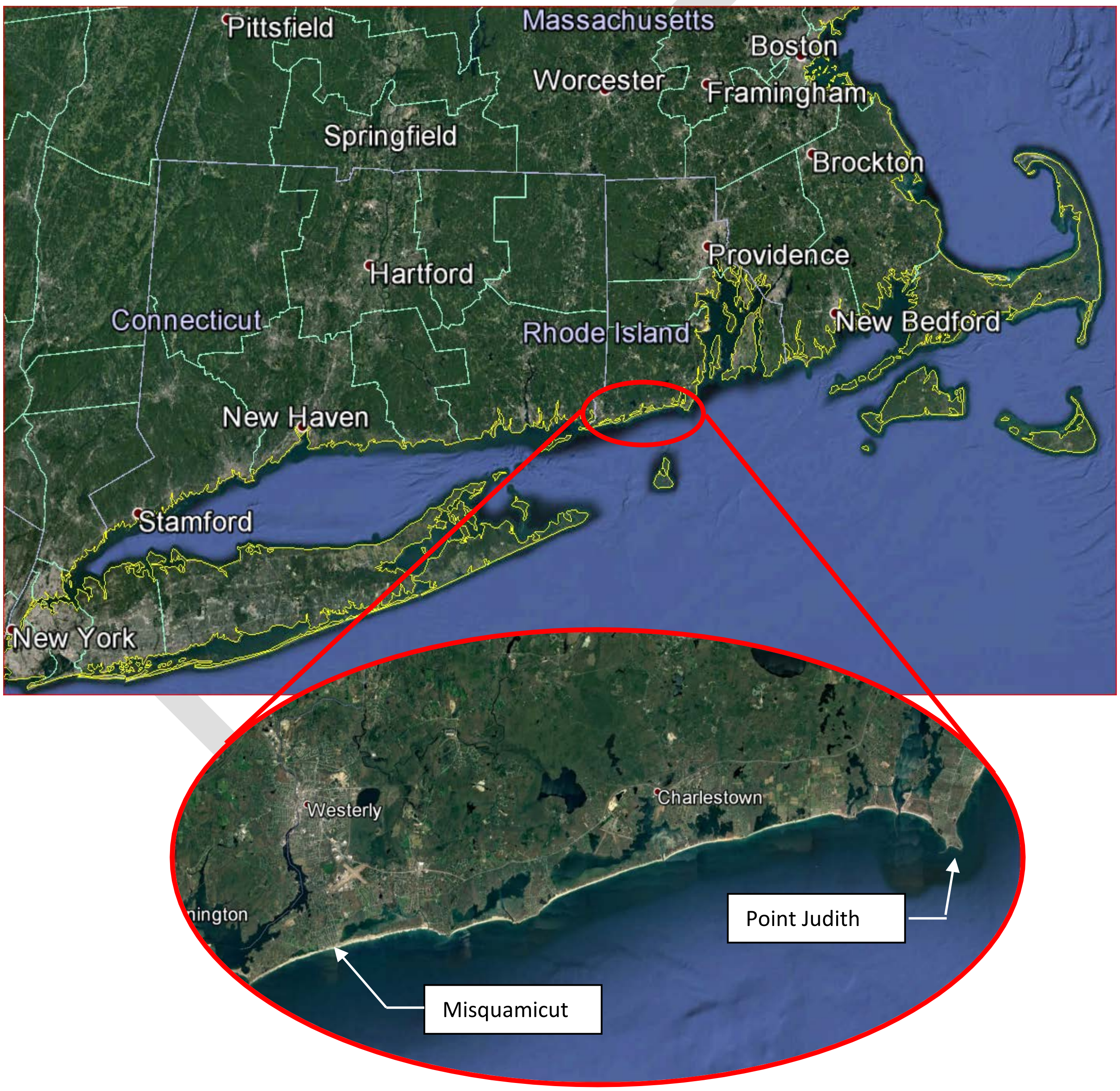

Figure 1-1. Study Location Map

Within the study area various forms of storm damage reduction were considered that included structural alternatives such as beach fill, sea walls, revetments, flood walls, dune improvements, and surge barriers/gates along with non- structural alternatives such as structure raising and relocation.

1.1 Study Areas

Within the approximately 20 mile study area there are several fairly defined areas that can be considered separately. Each area is provided below with a brief description of the housing/structure types, infrastructure and the storm forcing exposure. The areas are described from the west end of the study area to the east end starting at the Misquamicut Beach area and ending in the Point Judith Area. Those two areas are highlighted in Figure 1-1.

1.1.1 Misquamicut Beach Area and Winnapaug Pond (Westerly)

The Misquamicut area of Westerly, RI resides at the west end of the study area. The area is defined by an area of land fully connected (shown in Figure 1-2) and a barrier sand spit with a backing marsh that separates Winnapaug Pond from the Atlantic Ocean (Figure 1-3). Winnapaug pond is connected by a stabilized inlet at the east end of the Misquamicut area that is navigable and owned/maintained by the State of RI (Figure 1-4). There is a mix of housing and commercial properties along with the supporting infrastructure. The highest density of housing and commercial business are at the west end of Misquamicut Beach on the more fully connected land mass. Generally the shorefront is comprised of commercial properties in this area with residential areas landward. Moving east onto the barrier spit there are commercial business, a large state beach which is the most used in RI, town and pay for use private beaches, and beach front homes. The structure density on the spit is less than the western area and many of the homes are raised on piles. There is one road running east and west along the spit (Atlantic Avenue) for access. At the east end of the spit and along the inlet there is a large beach club area, both commercial properties and residential properties, and state access points to the inlet. Many of the homes have been raised in this area and sit on piles or have elevated foundations. Along the back bay shoreline (north shoreline) of Winnapaug Pond there are numerous private homes as well with these structures generally being higher in elevation due to the natural topography. Due to Winnapaug Pond the area is exposed to both direct beach front storm conditions which include water and wave overtopping and beach erosion and flooding of the back bay. The back bay or pond flooding occurs due to both the existing inlet and the potential for overtopping of the barrier spit or even breaching of the spit in severe storms. In addition to Winnapaug, the small pond to the west can be elevated due to overtopping or breaching. This results in the potential for the west end of Misquamicut, which is the more land based area to be inundated from the south and also from both the east and west sides, also known as flanking.

Figure 1-2. West end of Misquamicut Beach area

Figure 1-3. Middle area of Misquamicut constructed on barrier sand spit with backing marsh.

Figure 1-4. East end of Misquamicut and stabilized Winnapaug Pond inlet (Weekapaug Breachway)

1.1.2 Quonochontaug Pond

Quonochontaug Pond resides in both Westerly (west end) and Charlestown (east end). The east end of the pond and the inlet to the ocean is shown in Figure 1-5. Also shown in Figure 1-5 is the eastern end of the undeveloped barrier spit separating the pond from the Atlantic Ocean and the land mass/low elevation headland that separates Quonochontaug Pond from Ninigret Pond.

Figure 1-5. Quonochontaug Pond east end

The shoreline surrounding the pond is less developed than the shoreline surrounding Winnapaug Pond with the western barrier spit being fully undeveloped. The western spit area is fully used for recreation and wildlife habitat. The shoreline on the back bay and on the shorter eastern sand spit does have residential housing with the shoreward most houses being at the highest risk of flooding but much of the back bay shoreline rises fairly quickly in elevation which provides those residences natural protection against flooding. The land between Quonochontaug and Ninigret Pond is a lower headland feature. As with Winnapaug Pond the houses along the back bay shoreline can be impacted by water that comes directly through the inlet or over and through the spit breaches during severe storm events.

1.1.3 Ninigret and Green Hill Pond (Charlestown)

Ninigret Pond is the second largest pond along the South Coast of RI and resides fully in the town of Charlestown (Figure 1-6). As with the previously discussed coastal ponds, Ninigret is connected to the Atlantic Ocean by a relatively narrow inlet. To the west of the inlet is an uninhabited barrier sand spit that is used for recreation and for wildlife. To the east of the inlet there is a second barrier sand spit that contains a state beach and camp ground, numerous beach homes that are all elevated on piles to some, and a town beach facility. At the eastern most end of the pond is the inlet into Green Hill Pond (Figure 1-7). Separating Green Hill Pond and the Atlantic Ocean is the continuation of the eastern sand spit barrier of Ninigret Pond. There are elevated beach homes along this spit for approximately one half a mile and then the spit is uninhabited until the eastern end of Green Hill Pond is reached at he community of Green Hill Beach. Green Hill Beach is a low elevation headland feature previously similar to the aforementioned headlands between Winnapaug Pond, Quonochontaug Pond, and Ninigret Pond. The housing density is similar to those headland type features as well. The back bay shorelines of both Ninigret and Green Hill Ponds are moderately developed with the houses being generally being at lower elevations than along the back shore of Quonochontaug Pond. As with the other ponds water can enter by flowing through the inlets and also through and over the barrier spits during low frequency events.

Figure 1-6. Ninigret Pond Inlet and western 2/3rds of Ninigret Pond

Figure 1-7. Green Hill Pond Inlet, eastern end of Ninigret Pond, and western end of Green Hill Pond.

1.1.4 Matunuck and Potter Pond

The Matunuck area is comprised of a combination of a wider headland/fully connected land based area, sand beaches, a rocky/cobble type headland beach, and several rubblemound reveted stretches of shore front (Figure 1-8). At the eastern end of the Matunuck area a ribbon of land is backed by Potter Pond (Figures 1-8 and 1-9). The beach in this area is cobble and gravel based. The point feature that helps to define the Matunuck area is a popular surf area due to the “point break” feature. The structures consist of a large area of vacation cottages, a large area of mobile vacation homes, single family residences (both built on grade and on piles), and a stretch of commercial properties (restaurants and bars) on the immediate water front. The area has been experiencing the highest rate of shoreline erosion with recent erosion rates over 5 ft/year. In addition to the direct threat posed by the Atlantic Ocean flooding can reach the area on the back bay side through the Point Judith Inlet (discussed in the next section) and then through the inlet connecting Potter Pond to Point Judith Pond (Figures 1-10 and 1-11). These areas are vulnerable to flooding as well and comprise both residential housing and commercial properties (restaurants, bars, and marinas). Many of the building in this area are built on grade and are not elevated on piles. To the east of Matunuck is the East Matunuck State Beach (Figure 1-15), which is a sandy beach separating the Atlantic Ocean and Potter Pond. The area terminates at the west side of the Point Judith pond inlet.

Figure 1-8. Matunuck Beach area with the southwestern end of Potter Pond shown

Figure 1-9. Eastern end of Matunuck and Potter pond backing it.

Figure 1-10. The inlet connecting Point Judith Pond and Potter Pond

Figure 1-11. The inlet connecting Point Judith Pond and Potter Pond (zoomed in)

1.1.5 Point Judith Pond and Sand Hill Cove Area

Point Judith Pond is the largest coastal pond in Rhode Island and is connected to the ocean by a deep draft navigation channel/inlet to support the commercial fishing and ferry boat interests within Galilee (Figures 1-12 and 1-13). The inlet resides within the Point Judith Harbor of Refuge which is a large breakwater complex constructed to protect shipping interests during storms but now it mostly helps protect the inlet from ocean waves and the shoreline as well. The USCG has a station on the pond and the Narragansett Bay ship pilots use the pond as well. Just within the inlet, on both the east and west sides are numerous commercial business that support the fishing fleet, restaurants, bars, tourist shopping areas, etc. On the west side of the inlet the shoreline is sandy and includes the East Matunuck State Beach (Figure 1-15) and the East Matunuck area. This area is comprised of mostly residential building with many being constructed on grade. Moving north into the pond there is also a mix of commercial properties (marinas) and residential properties (Figure 1-14) with fairly dense clusters of housing/commercial properties. Due to the larger inlet and therefore a more direct connection to the ocean Point Judith Pond experiences a fuller tide range than the other ponds. To the east of the inlet the shoreline is sandy barrier spit that contains two state beaches and residential properties. On the back side of the barrier spit is the Galilee Bird Refuge which is separated from the rest of the pond by the access road causeway. There is an opening to the refuge under the causeway.

As discussed in the previous section, Point Judith Pond is connected to Potter Pond by an inlet through a marshy area (Figures 1-12 and 1-15). This inlet allows flow from Point Judith Pond into Potter Pond but also, if the barrier breaches, or is overtopped fronting Potter Pond, flow can to enter into Point Judith Pond.

Figure 1-12. Point Judith Pond inlet and southern end of pond

Figure 1-13. Point Judith Pond Inlet (Galilee, Rhode Island)

Figure 1-14. Point Judith Pond Inlet looking north

Figure 1-15. Marshy area connecting Point Judith Pond and Potter Pond with inlet

On the eastern side of the Harbor of Refuge, towards Point Judith point is an area called Sand Hill Cove (Figure 1-16). This area is a fairly dense residential area and while it resides on the shore within the Harbor of Refuge it does experience exposure to higher wave energy than the rest of the shoreline within the harbor due to the navigation opening in the breakwater system to the south of the Sand Hill Cove area

Figure 1-16. Point Judith Harbor of Refuge – Sand Hill Cover Area

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 is completed which limits and focuses the level of analysis associated with the project. As part of the reduced level of analysis there is a focus on using existing information. As such the coastal engineering analysis was somewhat limited as compared to previous studies of a similar scale. The work focused on providing annual recurrence interval water levels within the study area and performing a Beach-fx analysis for the Misquamicut Beach/Winnapaug Pond area. Both sets of analysis are detailed later in the report.

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 4.1, significant tropical storm events impacted 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 the 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. In addition to storm events, locally generated persistent summer sea breezes generate significant waves from the south west.

Both storm types and summer wave conditions can result in littoral transport along the coast and produce beach erosion and morphology change. Storms often cause coastal inundation leading to economic losses to improved property within the study area. Although economic losses are most often realized in the wake of major storm events, it is long-term chronic erosion that creates the vulnerability to major economic losses through volumetric depletion of beach material in the active profile, reduction in beach berm width and reduction in dune crest elevation and dune volume. Not all storms in the storm climatology produce measurable economic damages but they contribute to setting up vulnerability for economic losses. Long-term chronic erosion can be driven by gradients in the longshore sand transport rate, offshore losses and or volumetric losses associated with overwash processes and depend to some extent on sediment supply from updrift beaches.

The ocean exposed coast in the study area is predominately a sandy shoreline consisting of long barrier spits and several minor headland features. The headlands were called minor due to the relatively low elevation of these features and the relatively minor perturbation to the shoreline shape from these features. Behind the barrier spits are fairly large coastal lagoons, called salt ponds locally. The lagoons are connected to the ocean through stabilized inlets that allow tidal exchange with the ocean. The inlets are relatively narrow and shallow for the size of the ponds resulting in noticeably reduced tide ranges within the ponds with the exception of Point Judith Pond. As will be discussed in Section 2.2, the MHHW tide range of the ocean is 2.87 ft. The tidal range in the ponds, with the exception of Point Judith Pond, was from 1 to 2 feet.

2.1 Wave Climate

As shown in Figure 2-1 the Wave Information Study (WIS) data from station 63100 has been provided from the WIS web portal (http://wis.usace.army.mil/). The station is located approximately 24 miles south of the mainland Rhode Island coast. It can be seen in the Figure 2-1 that the predominant wave direction for larger waves is from the south-south west to south east with a definite skew to the southeasterly direction. These are the waves that are generated by both types of storms discussed. These waves do impact the southern shore of RI but are influenced by Block Island (both wave height and direction). In the study titled “Analysis of Extreme Wave Climates in Rhode Island Waters South of Block Island” by Annette R. Grilli, Taylor G. Asher, Stephan T. Grilli and Malcolm L. Spaulding as part of the RI Ocean Special Area Management Plan (SAMP) it was shown that Block Island does impact the wave climate fairly significantly on the western part of the shoreline of RI and to a lesser extent further east towards Point Judith. This trend makes sense based on the storm wave directions discussed with Block Island having a shadowing effect.

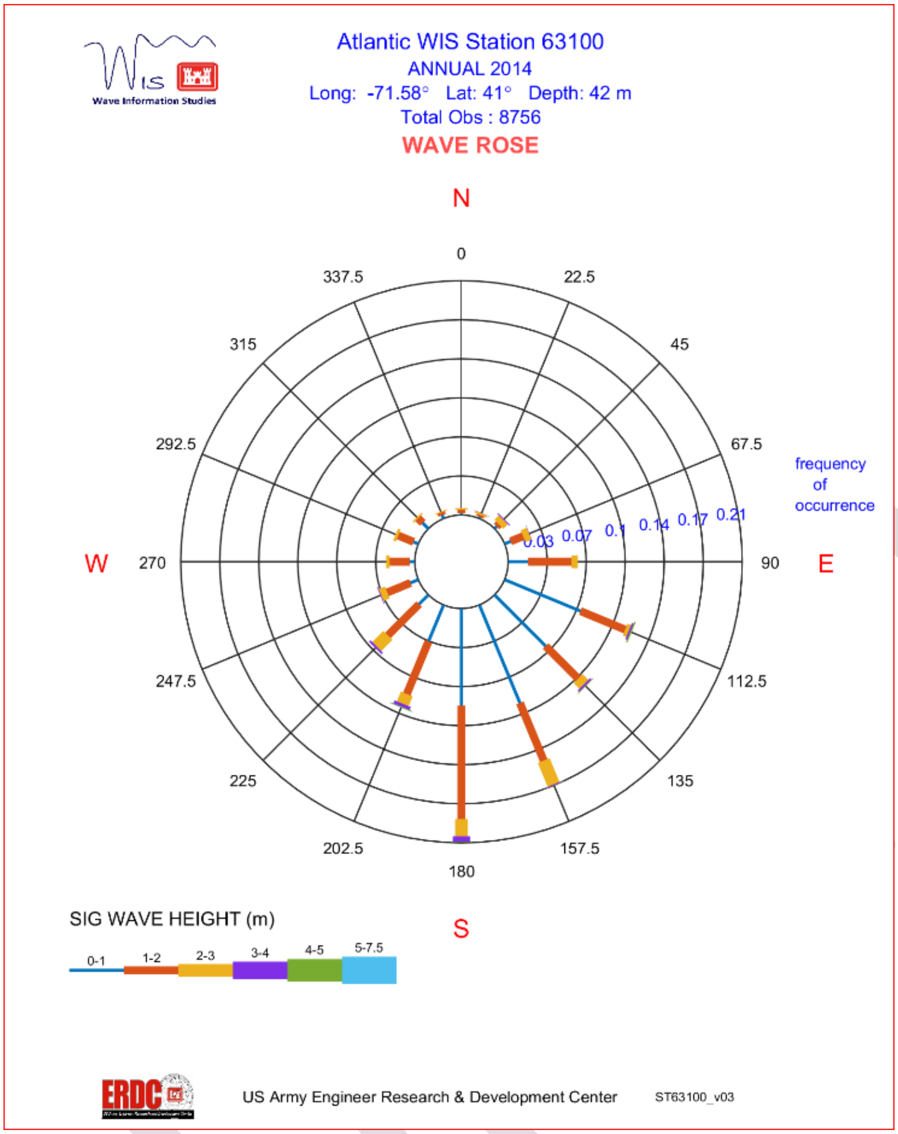

Figure 2-1. Wave rose south of Block Island, RI

2.2 Tidal Regime

The tides along the coast of Rhode Island are semi-diurnal (two low and two high tides per day) with one high and low tide typically of more magnitude than the other due to a slight diurnal shift. Tides along the coast are fairly uniform along the roughly 20 miles of open coastline along the project area with the range only varying a few tenths of a foot. The tide range along the ocean is provided in Table 2-1 and as shown has a Mean Lower Low Water (MLLW) to Mean Higher High Water (MHHW) tidal range of 2.87 ft.

The tide range within the coastal ponds (lagoons) ranges from being very restricted to minimally restricted with the controlling factor being the inlet size vs. the coastal pond size. The approximate tide ranges within the various coastal ponds is provided in Table 2-2 working from west to east. The tide ranges were not calculated but instead based on graphed information from the RI Regional Sediment

Management Study data collection effort completed and documented by the Woods Hole Group in 2011.

Table 2-1. Ocean Tide Range – taken from NOAA Tide Prediction Station Weekapaug Point, RI

Condition	**Elevation (feet, NAVD88*)**
Mean spring high water	+1.46
Mean higher high water	+1.13
Mean high water	+0.92
NAVD88	0.00
Mean tide level	-0.35
Mean low water	-1.61
Mean lower low water	-1.74
Mean spring low water	-1.94

*North American Vertical Datum of 1988 (NAVD88)

Table 2-2. Approximate tide ranges within southern RI coastal ponds.

Coastal Pond	**Approximate Tide Range**	**Notes**
Winnapaug Pond	≈ 2 ft	
Quonochontaug Pond	≈ 2 ft	
Ninigret Pond	≈ 1 ft	
Green Hill Pond	<1 ft	Restricted by a narrow inlet to Ninigret pond which also has a restricted tide range
Trustom Pond	0 ft	Trustom Pond is very often not connected to the ocean so the range varies
Cards Pond	0 ft	Cards Pond is very often not connected to the ocean so the range varies
Potter Pond	≈ 1.5 ft	Connected to Point Judith Pond

Point Judith Pond	≈ 3 ft	Full tide range – large inlet – federal navigation channel

2.3 Sediment Transport and Shoreline Change

Sediment transport along the study area is from west to east based on information provided by the RI CRMC and URI. It has been concluded that the persistent summer sea breeze generated waves control the net direction of sediment transport. The system can be characterized as sediment starved in that there is no or very little sand making it to the southern shore from riverine sources or offshore deposits. The sediment within the system comes from erosion of the shore itself.

Within the system sand is known to be lost to the flood shoal deltas within each coastal pond and evidence of the significant sediment sinks can be seen in aerial photos such as the one provided in Figure 2-2 which shows the flood shoal delta of Quonochontaug Pond. Just as sand is brought into the pond and sequestered from the beach system, sand is pushed offshore at these inlets as well due to the relatively high shore perpendicular currents that flow out from these inlets during an outgoing tide. Additionally sand is thought to be lost offshore during storm events due to shore perpendicular currents that develop due to various bathymetric features. Developing a detailed sediment budget and quantifying sediment losses by various modes and at various locations was not part of this effort. This information is provided for context.

Figure 2-2. Quonochontaug Pond inlet – flood shoal delta

The key piece of information that was used during this study related to sediment transport was shoreline change since this helped defined the future conditions of the beaches for both the with and without project conditions. With the shoreline change rates known, estimates of future shoreline position could be made as well as used for input into the Beach-fx modeling effort that will be discussed in Section 6.0. Shoreline erosion rates maps and position data were taken from the RI CRMC shoreline change web portal and analyzed by the Geographic Information Section within the New England District. The data from 1985 to 2004 was analyzed using the US Geological Survey's (USGS) digital Shoreline Analysis System (DSAS). A portion of the CRMC shoreline change rate map for the Misquamicut Beach region has been provided as Figure 2-3 as an example. The image is fairly large, requiring that only a small portion be included in this report. The legend/image information table was cut and pasted from the larger image and overlain to provide the reader with information related to the image. From the information contained on the maps and the accompanying study it was determined that the historic shoreline change rate was -1 ft/yr in the Misquamicut Beach area of this study.

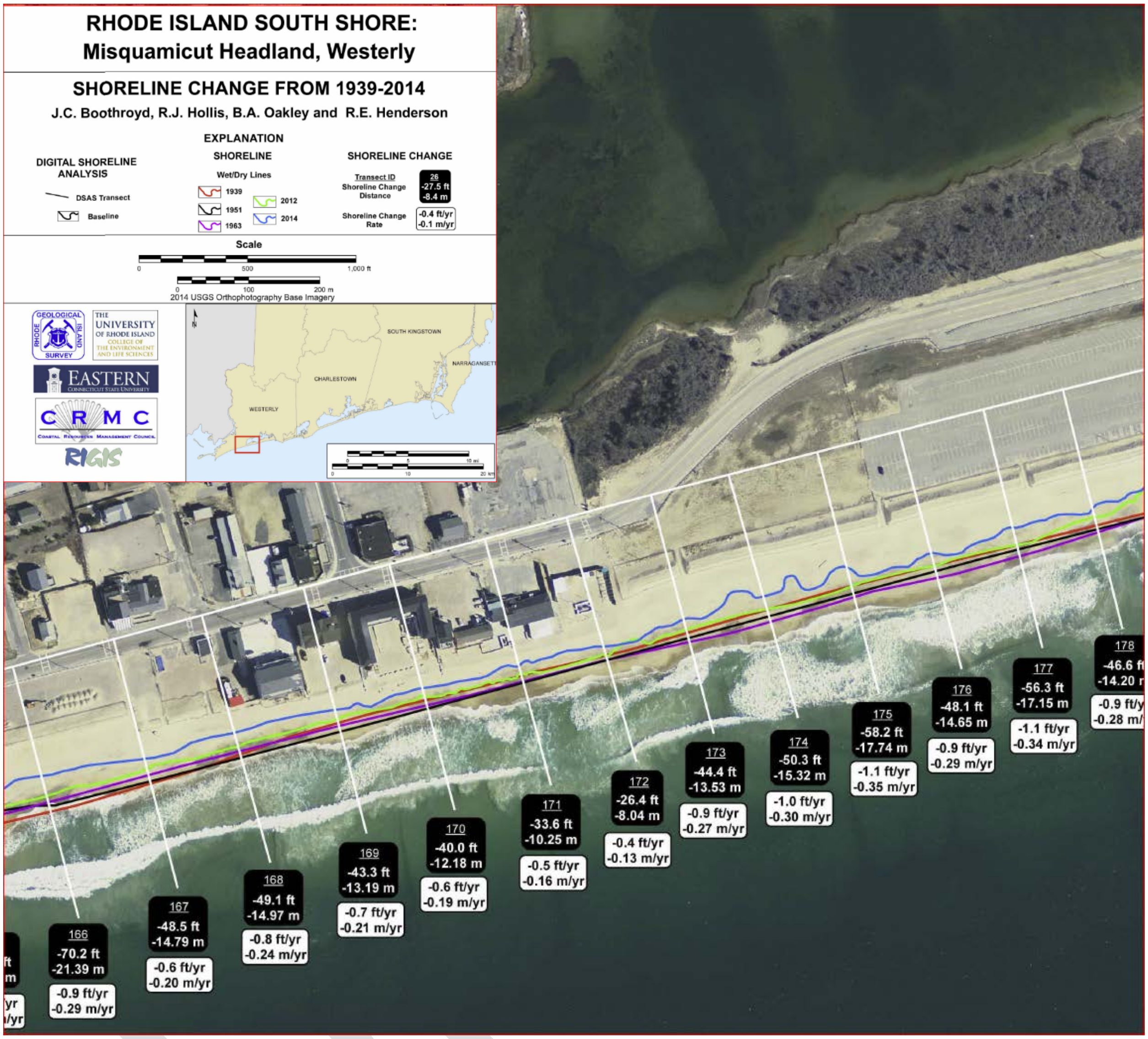

Figure 2-3. Example RI shoreline change map (west end of Misquamicut State Beach)

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 recurrence probability of storm conditions within the project area. Recurrence probability is the percent chance that an area experiences a particular level of storm conditions or greater. Often a key recurrence interval due to FEMA flood insurance requirements is the 1% annual chance storm. This is a storm water level that an area has a 1% chance of experiencing each year and every year. The 1% annual chance storm is often referred to as the 100 year storm. These two are the same since if one does the simple math a 100 year storm is a 1% annual chance storm. The representation of annual chance 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 wrong and as stated that level of storm or greater has a 1% chance of occurring each and every year even if it had just occurred the year previous. 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 occurring). Another way to consider the 1% annual chance storm is that during a 30 year period (length of a typical mortgage) a property in the 1% annual chance flood plain has at least a 26 % chance of experiencing the 1% 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% 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 level and wave height for some storms of note along the RI Coast.

Often it is mistakenly concluded that tropical based storm systems do not regularly impact the Rhode Island Coast. As shown in Figure 2-4, based on the historical tracks of tropical based systems between 1851 and 2013 (162 years), 55 tropical systems have come within 75 miles of Charlestown, RI which is the approximate geographic center of the study area. That is an average of one storm every 3 years which is similar to the frequency found in the NACCS modeling study. To help quantify the level of storm exposure along the coast water level annual exceedance probabilities for the study area have been provided in Table 2-3 and Figure 2-6. Annual exceedance water level probabilities have been provided from the center of the study area (Figure 2-5) and also at the nearest NOAA water level gages that are adjacent to end of the study. As shown in the table, the annual recurrence interval water levels are presented as compared to the mean sea level (MSL) which one can take for the purposes of this discussion as mid tide level.

To help put these water levels in context some of the more recent tropical based systems have been provided in Table 2-4 and added to Figure 2-6 showing the annual exceedance probability of those storms. The water levels in Table 2-4 have been provided in both mean sea level (MSL) to allow comparison to the NACCS annual exceedance water levels and to mean higher high water (MHHW) to allow for a better understanding of just how high the water levels are. MHHW is based on averaging the highest tides each month sot that the water levels reported in Table 2-4 relative to MHHW are how high the storm water levels would be above the highest non-storm tides in a month

To determine the annual exceedance probability of the storms actual recorded water levels from the Newport, RI and New London, CT NOAA water level gages for each of the historical storms listed in Table 2-4 were compared to the NACCS annual exceedance water levels. Most of the storms straddled a fairly tight range between the two water level stations except for Hurricane Bob. For that storm the annual exceedance probability between Newport and New London was considerable and therefore the storm label band was turned horizontal and stretched to cover the annual exceedance probability range. The most recent storm of record, Hurricane Sandy, which certainly caused significant damage along the coast of RI and was the impetus for performing this study was slightly more than 10% annual exceedance probability storm for the southern coast. In other words the study area has a 1 in 10 to 1 in 20 chance each year of experiencing a Sandy level event. A storm that is often looked at as a storm of record is the 1938 Hurricane and while it was known to be a storm that could happen again it was often thought to be a very rare occurrence. However based on the NACCS analysis it was found to be around a 1% annual chance storm—so rare, but not that rare. Prior to the NACCS study annual exceedance probability was determined based purely on NOAA water level station recorded data and extremal analysis. This approach skewed the probabilities of storms to the rarer level of occurrence. For example Sandy was originally thought to be around a 1.5% annual chance storm, so a much rarer event than a 10% to 20% event. Based on the state of the art numerical modeling and statistical analysis performed in the NACCS effort the actual storm exposure has been made clearer.

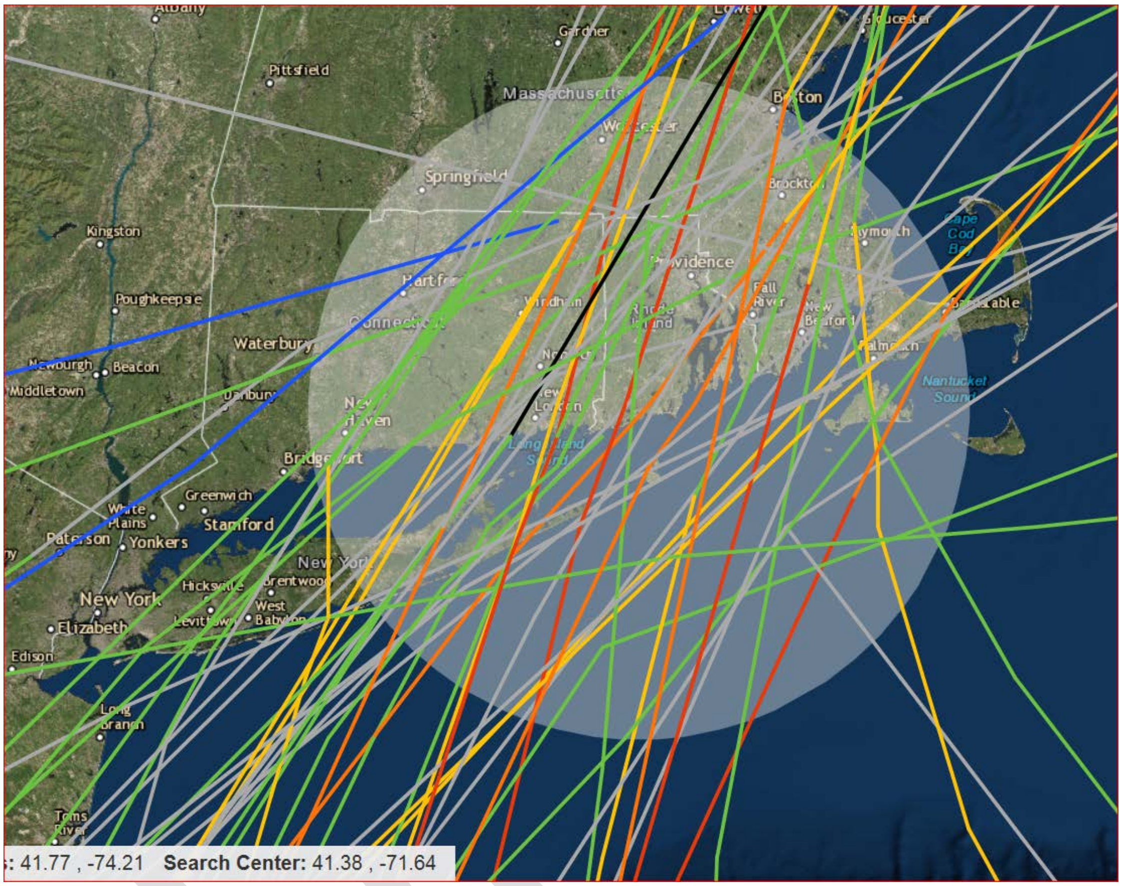

Figure 2-4. Tropical storm system paths from 1851 to 2013

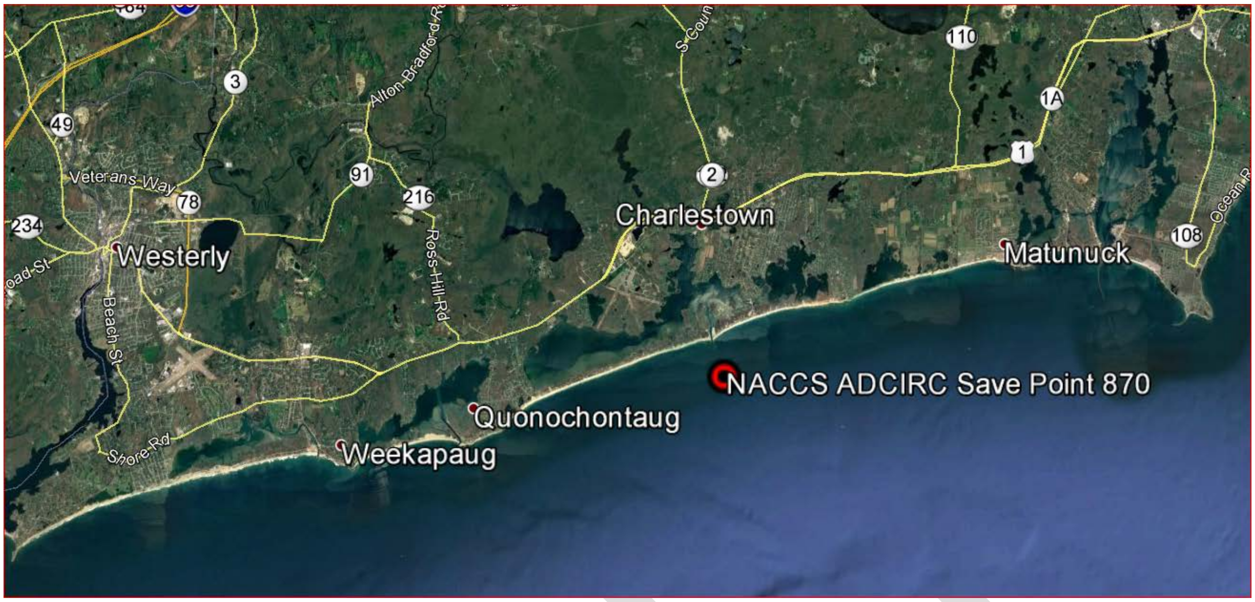

Figure 2-5. NACCS model data point used for study area annual exceedance probabilities

Table 2-3. Annual exceedance probability water levels from NACCS study

NACCS Save Point		Annual Exceedence Probability								
Location	**Number**	**100**	**50**	**20**	**10**	**5**	**2**	**1**	**0.5**	**0.2**
Neashore Charlestown	**870**	3.84	4.43	5.22	5.81	6.40	7.28	8.10	9.15	10.53
Newport, RI	**10278**	4.30	4.89	5.71	6.36	7.05	8.07	8.99	10.07	11.48
New London, CT	**899**	3.74	4.69	5.84	6.63	7.41	8.53	9.51	10.63	12.07
*** Water levels are in ft-MSL**										

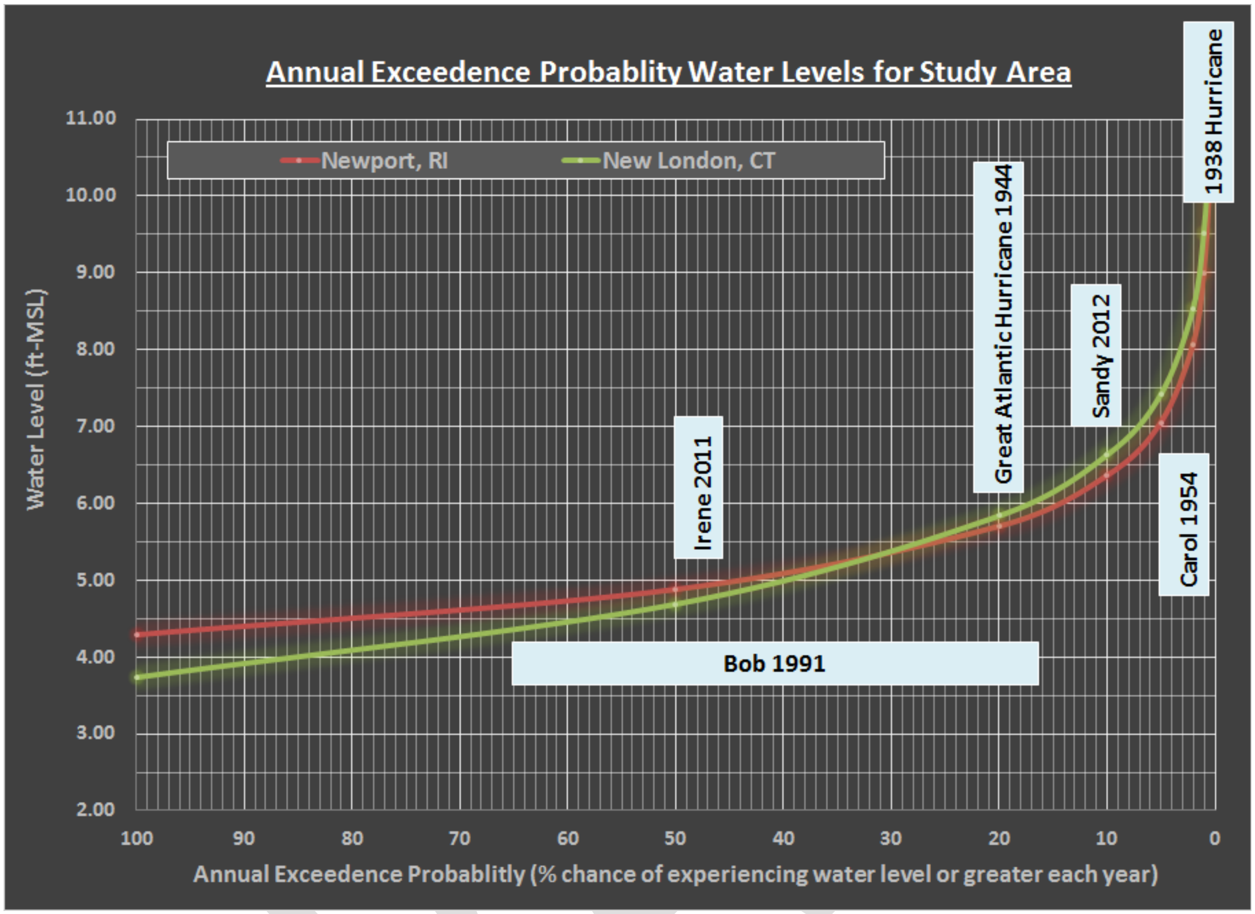

Figure 2-6. Annual exceedance water level probabilities and storms of note

Table 2-4. Hurricanes of note that impacted RI

Storm Name	Dates	Water Level (ft-MSL)		Water Level (ft-MHHW)	
		Newport, RI	New London, CT	Newport, RI	New London, CT
1938 Hurricane	21-Sep-38	11.58	9.04	9.46	7.52
Carol	Aug 31, 1954	8.87	7.34	6.75	5.82
Sandy	Oct 28-30, 2012	6.44	6.46	4.32	4.94
Great Atlantic Hurr.	Sep 14-15, 1944	6.07	5.44	3.95	3.92
Bob	Aug 19-20, 1991	5.85	4.22	3.73	2.70
Irene	Aug 28-29, 2011	4.75	4.92	2.63	3.40

3.0 Sea Level Change

Based on EC 1165-2-212, USACE studies must consider future rates of sea level change that are higher than the historical rates to account for the potential impacts of climate change. Due to the uncertainty associated with future sea level change the USACE policy is to look at three scenarios of sea level change and investigate the impact to project feasibility. These rates are the historical rate at the project site, an intermediate rate and a high rate of sea level rise. The intermediate and high rates are from the National Research Council (NRC) curves 1 and 3, respectively. While this study will follow USACE guidance, the State of Rhode Island uses a future sea level change rate equivalent to the NOAA high curve. In order to calculate these various rates for a project site USACE has developed an online calculator tool at the USACE climate change web portal (tool web address of http://corpsclimate.us/ccaceslcurves.cfm). The tool uses the closest NOAA tide station with an adequately long water level record to determine the historical trend. The tool then uses this historical trend along with a formulation provided in the EC to determine the intermediate and high rates of change. The NOAA curve values can also be provided.

For the historic mean sea level trend, the New London, CT NOAA station (NOAA 8461490) was used. The station is twenty two (22) miles west of the approximate center of the study area. The Newport, RI NOAA station, which is to the east of the project area, was also checked and there was a very minor difference between the New London station and the Newport station. Based on that comparison it was decided that using the New London station alone was adequate. The mean sea level trend is 0.00738 feet/year or 0.738 ft/century based on regionally corrected mean sea level data over 69 years. The mean trend is shown in Figure 3-1 which was taken from the NOAA Sea Level Trend web page https://tidesandcurrents.noaa.gov/sltrends/sltrends_station.shtml?stnid=8461490. As shown in the plot there are yearly and decadal cycles that cause the short term rate to vary. In addition to the long term trend plot the inter-annual variation in sea level between 1990 and 2016 has been provided as Figure 3-2 and the previous sea level trends/rates have been provided as Figure 3-3. In all three of these plots there is not a significant indication the rate of sea level change has increased within RI. Figure 3-3 shows that the annual rate of rise between 1939 and 2006 had a mean sea level rise rate of 2.25 mm/yr. As shown in the plot that rate increased to a high of 2.58 mm/yr between 1939 and 2014, but it dropped to 2.55 mm/yr when calculating the rate between 1939 and 2015. Looking at the shape of the plot in Figure 3-3 it appears the rate increased for a short time and is maybe dropping. If one was to take the increase from 2.25 mm/yr to 2.58 mm/yr over the 10 year period as a sign of the increase in sea level rise that would still only indicate an acceleration of 0.33 mm/yr/decade. Over 50 years or 5 decades that equates to 41.77 mm above the current trend or 0.14 ft. This is not to say that the rate of sea level rise will not increase in the future, or that it has not started happening already, just that the data does not presently indicate a statistically significant move from the historical trend.

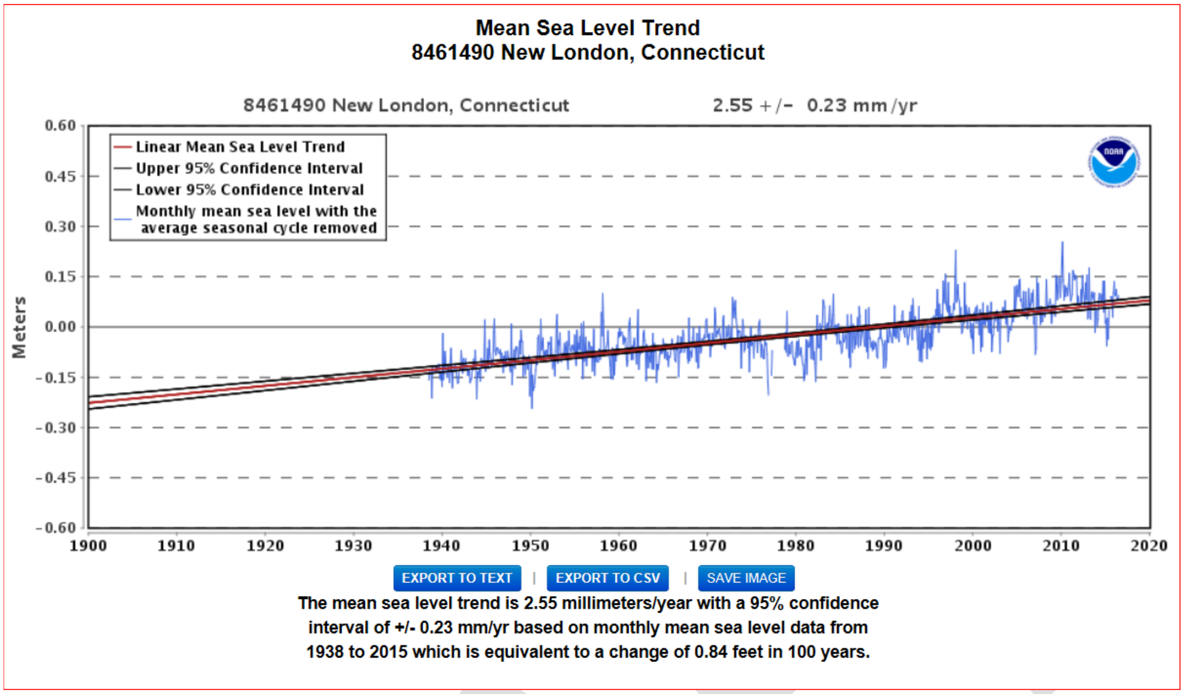

Figure 3-1. NOAA mean sea level trend plot

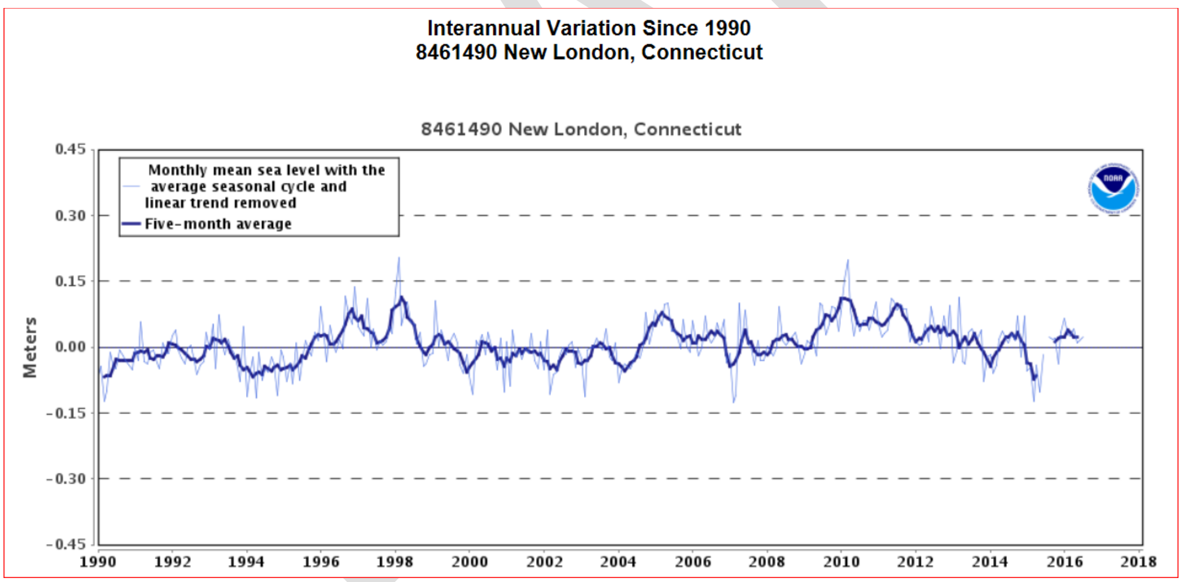

Figure 3-2. NOAA Interannual Sea Level Trend Variation at New London Station

Previous Mean Sea Level Trends
8461490 New London, Connecticut

As more data are collected at water level stations, the linear mean sea level trends can be recalculated each year. The figure compares linear mean sea level trends and 95% confidence intervals calculated from the beginning of the station record to recent years. The values do not indicate the trend in each year, but the trend of the entire data period up to that year.

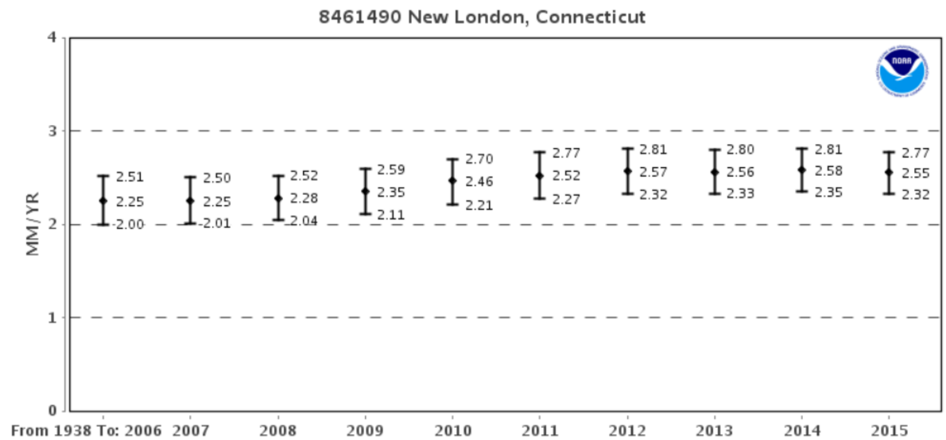

Figure 3-3. NOAA Mean Sea Level Previous Sea Level Trends

The aforementioned rates required by USACE for scenario based sensitivity analysis for future conditions and the NOAA high rate curve used by the State of RI have been provided in Figure 3-4 and Table 3-1. As shown the historic rate results in a 0.37 ft increase from the year 2020 to 2070 (anticipated project economic life) while the intermediate rate would cause an increase of 0.84 ft within that same 50 year period. For the USACE and NOAA high rate curves the increase would be 2.73 ft and 3.69 ft, respectively.

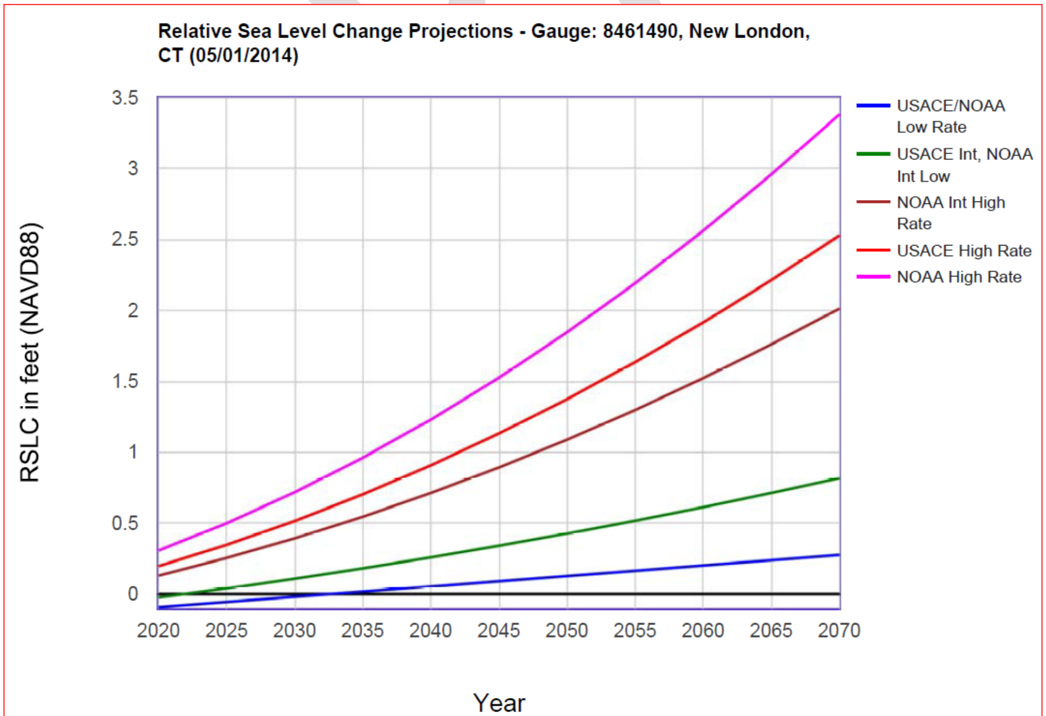

Figure 3-4. USACE and NOAA sea level change rates – future scenarios

Table 3-1. USACE and NOAA sea level change rates – future scenarios

Year	USACE Low NOAA Low	USACE Int NOAA Int Low	NOAA Int High	USACE High	NOAA High
2020	-0.09	-0.02	0.13	0.20	0.31
2025	-0.06	0.04	0.26	0.35	0.50
2030	-0.02	0.11	0.39	0.52	0.72
2035	0.02	0.18	0.55	0.70	0.96
2040	0.05	0.26	0.71	0.91	1.23
2045	0.09	0.34	0.89	1.13	1.53
2050	0.13	0.43	1.09	1.38	1.85
2055	0.17	0.52	1.30	1.64	2.19
2060	0.20	0.61	1.52	1.92	2.56
2065	0.24	0.71	1.76	2.22	2.96
2070	0.28	0.82	2.01	2.53	3.38

*Units are in ft-NAVD88 and the values are the projected increase in still water level above the 2020 still water level

4.0 Water Levels and Wave Conditions (Storm Parameters)

As discussed in Sections 1 and 2, the area is impacted by both extratropical and tropical 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 frequency of storm based water levels were described in Section 2 which places the study areas storm exposure in context. That information was produced from the North Atlantic Comprehensive Coastal Study (NACCS) which will be described below in Section 4.1. Additionally the water levels used for the study's Tentatively Selected Plan (TSP) will be described in Section 4.2, along with why the NACCS modeling results were not used.

4.1 North Atlantic Comprehensive Coastal Study (NACCS)

Water levels and wave heights were needed as input for the various types of coastal engineering/planning analysis performed in the study. The intent for this study was to use the results from the North Atlantic Comprehensive Coastal Study (NACCS) numerical modeling and statistical analysis. 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 coastal 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 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 the following web link http://www.nad.usace.army.mil/CompStudy/. An example image of the save points, or data access points that are provided in CHS is provided in Figure 4-1.

Figure 4-1. CHS model save points example

The NACCS data was used as a boundary condition for the Storm-induced BEAch CHange (SBEACH) cross sectional beach transformational model that was part of the Beach-fx effort. Beach-fx is a USACE planning level model that was used to analyze beach fill projects in the study area and details of the model and how the NACCS data was used are provided in Section 6.

4.2 Federal Emergency Management Agency (FEMA) Base Flood Elevation (BFE) Data

4.2.1 FEMA Flood Insurance Map (FIRM) Use Decision

As impressive as the NACCS modeling effort and results are and as dense as the grid was for such a large regional model (cell sizes as small as 100 ft), even at these relatively small cell sizes, the model’s applicability for direct use very nearshore and on shore must be evaluated for each localized case. For this study area, due to the fairly complex bathymetry and topography, including the aforementioned narrow inlets (several of which were approximately 100 feet wide) the model was determined not to be adequate to accurately portray very near shore and onshore storm water conditions in the study area. The bathymetry and topography was basically smoothed and simplified in the model due to the model grid cell size for these areas which would likely skew the various storm parameters modeled and output. Additionally, the model assumed a static bathymetry and topography which is not correct during any storm and certainly not correct during hurricanes where the barrier spits could be overtopped and potentially breached.

To address the model limitations at the very near shore a plan was developed to refine the NACCS model grid to an appropriate resolution and then rerun a select set of storms to determine the total water levels and wave heights within the study area. This work was not done by USACE since USACE was informed this type of work would be done by others outside of USACE as part of a FEMA flood insurance map revision. However, during the course of this study it was determined that the model refinement would not be completed for the FEMA map update. Unfortunately, at the time USACE determined that work had not been completed and would not proceed it was too late in the USACE study process to allow adequate time for the detailed modeling to be completed by USACE. After significant discussion within the project team it was concluded that the best option when considering the time available in the study schedule was to use the existing FEMA Flood Insurance Study (FIS) information and the associated Flood Insurance Rate Maps (FIRMs) from 2013.

4.2.2 FEMA FIRM and BFE Data Description

The information within the FIS and on the FIRMs includes a wide array of information related to the FEMA study but most importantly includes the Base Flood Elevation (BFE) information in both table form and as maps. Base Flood Elevation essentially means the elevation one would have to have the bottom of their house or structure of interest at to be above the water hazards associated with a 1% annual exceedance probability storm (also known as a 100 year storm or Base Flood). During the FEMA FIS and FIRM development numerous transects which start in the ocean and progress overland are used in a 1-D model to determine water and wave movement from the ocean across the shore to determine water levels and wave heights overland. For each transect select input and output information is provided in a table that can be read directly from within the FIS report. Using these transects and tables along with topographic maps, the information is translated into FIRMs which are maps of the BFE. Within the study area the FIRMs contain two types of flood zones, the AE zones and the VE zones. The AE zone BFE is defined as the water elevation that results from storm surge on top of the normal tides and the localized increase in water level caused by wave setup. Wave setup is the localized increase in water elevation along the shore caused by breaking waves and the associated momentum and energy within those breaking waves. The AE BFE is one where wave heights were determined to be less than three feet and less impactful than the waves in the VE zone. The VE zone includes the same water level factors as the AE zone, but also includes an additional wave height component that exceeds three feet. In these areas the height of the waves adds to the elevation a structure must be above to avoid both flooding damage and wave impact damage. VE zones are typically on more exposed coastlines or in areas where broken wave energy can redevelop, or where locally wind generated waves can be a factor. An example of a FIRM has been provided in Figure 4-2 which shows the various AE and VE BFE zones and elevations as well as the transects used to model the water levels and wave heights. Table 4-1 has been provided to show the information provided for each transect. The map and table are for the Charlestown, RI area.

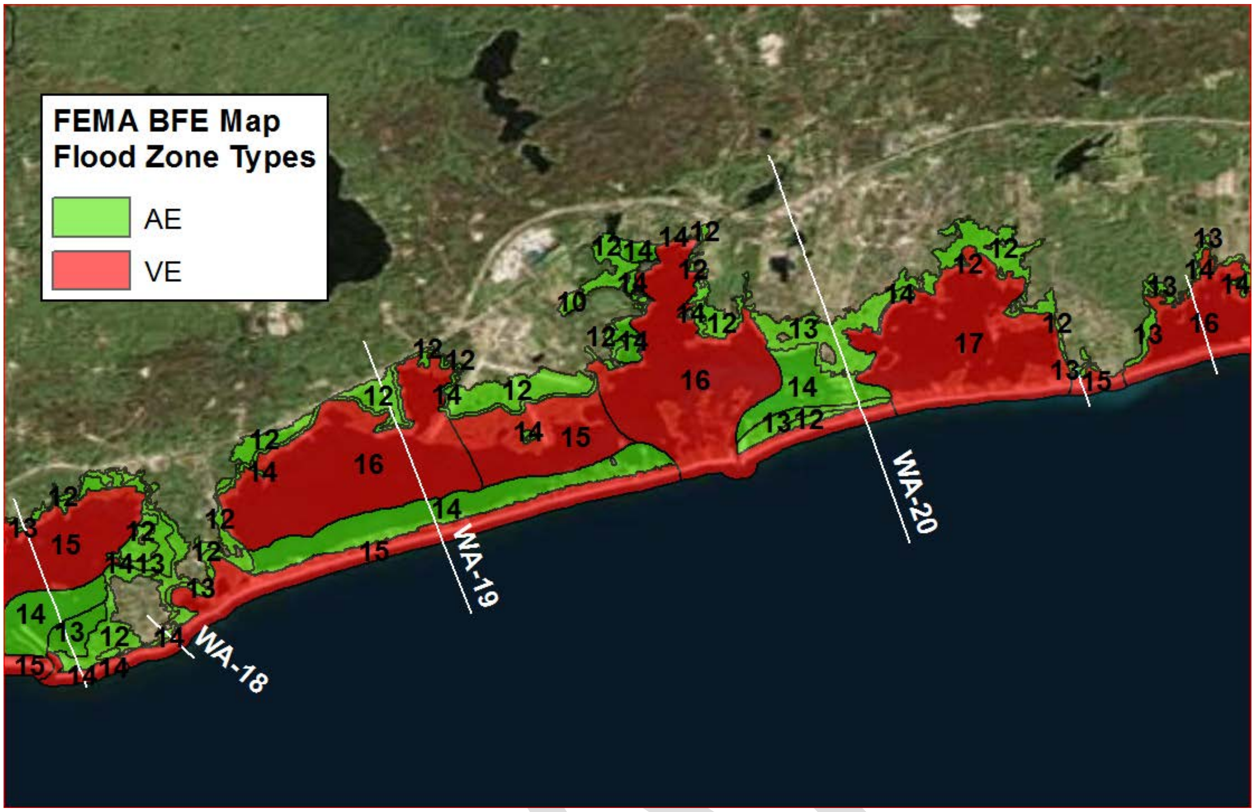

Figure 4-2. FEMA FIRM – Charlestown, RI area

Table 4-1. FIS transect table

Flooding Source and Transect Number	Stillwater Elevation				Total Water Level[1]	Zone	Base Flood Elevation
	10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance	1-percent-annual-chance		(Feet NAVD 88)[2]
BLOCK ISLAND SOUND - continued							
Transect 17	5.1	8.0	10.1	19.0	12.1	VE	15
						AE	12
Transect 18	5.1	8.0	10.1	19.1	12.7	VE	15
						AE	13
Transect 19	5.1	8.0	10.1	19.1	12.3	VE	16
						AE	12
Entire shoreline within South Kingstown							
Transect 20	5.2	8.1	10.2	19.3	12.5	VE	16
						AE	12
Transect 21	5.2	8.1	10.3	19.3	12.4	VE	15
						AE	12
Transect 22	5.2	8.1	10.3	19.4	12.7	VE	16
						AE	14
Transect 23	5.2	8.1	10.3	19.4	12.6	VE	16
						AE	13

4.2.3 FEMA FIRM and BFE Data Manipulation

As discussed in Section 1.2 and will be discussed in Section 5 the key information that ended up being needed was the total water surface elevation, comprising the still water elevation, wave setup, and wave height contribution, within the study area for a range of storm recurrence intervals.

In order for the planning process to conduct an economics benefit analysis through the comparison of without and with project conditions for various alternatives a table of total water surface elevations (essentially BFEs for various annual occurrence probabilities other than the 1% or 100 yr storm) was needed. This meant data manipulation was necessary, because as discussed, and as shown in Table 4-1, for each transect by FEMA definition the BFE is specific to the 1% annual chance storm. For the other annual chance water levels shown in Table 4-1, only the still water elevation was included with no wave set up component nor wave height component. As detailed in Appendix B the HEC-FDA model was utilized for the economic analysis. The HEC-FDA model requires that eight (8) return frequency storms be provided as input. To add to the existing four (4) shown in Table 4-1, four more annual chance water levels needed to be developed.

As shown in Figure 4-2 there were numerous BFE AE/VE zones. When looking at the maps for the full study area the range of AE Zone BFEs was from 11 ft to 14 ft and the range of VE Zone BFEs was from 14 ft to 17 ft. For each one of these “elevation” zones the full range of recurrence interval elevations needed to be developed. When considering there was a total of eight (8) elevation zones and eight (8) recurrence intervals that were needed a total of sixty four (64) values were needed. Out of those sixty four (64), only eight (8) were directly provided by the FEMA FIS report which meant fifty six (56) values needed to be interpolated, extrapolated, curve fit, or in some other way manipulated from the FIS report. The process of completing that has been provided below in Tables 4-2 and 4-3 for the AE 13 zone and the VE 15 zone. The details and methodology of developing the numbers are provided in the footnotes and descriptions below the table. The process used to generate the values for each zone was essentially repeated for each of the other zones. It was considered that for each AE and VE zone there were minor differences in still water level and wave height for each transect but the differences for each transect within each zone along the south shore were less than 0.5 feet in elevation. As noted in Tables 4-2 and 4-3 the NACCS results were used to help set the water levels and wave height contributions in several instances. As an example for the 0.2% chance event, it was made clear during the NACCS study that the FEMA water levels for the rare events (such as the 0.2% chance event) were considerably higher and based on the methodology used by FEMA much more likely to be incorrect. In order to help keep the 0.2% storm event water levels more reasonable the relation between the 1% chance storm and the 0.2% chance storm in the NACCS was used to extrapolate the 0.2% chance event for this effort instead of taking the FEMA FIS 0.2% chance event water level and adding a wave contribution. Both are known to be incorrect and in reality it does not significantly influence the results of the study since the very rare events do not impact the HEC-FDA modeling/optimization effort significantly (as discussed in Appendix B).

It must be remembered that the annual chance water levels generated for each zone are not BFEs and should not be referred to as such since they are for annual chance probabilities that are less than or greater than the 1% annual chance storm. Once again by definition base flood elevation (BFE) is specifically for the 1% annual chance storm.

Numerous tables and plots similar to Tables 4-2 and 4-3 and Figure 4-3 were developed and those results were entered into the summary table below (Table 4-4). Once all the values were entered into the table these values were used to generate plots for both the AE and VE based zones. The VE zone plot has been provided as Figure 4-4. From these plots slight adjustments were made to some of the elevations due to comparisons between the various zones. Table 4-4 is the final product of the FEMA based water level vs. recurrence interval analysis performed. This was used during the economics analysis and details of that effort can be found in Appendix B.

Table 4-2. AE 13 Zone Recurrence Interval Flood Elevation worksheet

	Percent Annual Chance of Exceedance (equivalent return period)							
Water Elevation Definition	**50% (2 yr)**	**10% (10 yr)**	**4% (25 yr)**	**2% (50 yr)**	**1.33% (75 yr)**	**1% (100 yr)**	**0.4% (250 yr)**	**0.2% (500 yr)**
SWL (without wave set up)	3.1	5.1		7.9		10		12.5
Recurrence Int. Flood Elev.	5	7	8.5	10.5	12	13	15	16

* Values highlighted in gray were taken direclty from FIS transect tabels (example table provided as Table 4-1.
* Values in yellow were used as the reccurence interval flood elevations
* Elevation data is ft-NAVD88. Wave height is provided in feet (not related to a vertical datum.
* Developed values are desrcibed below in the order they were generated. The order assisted in interpolation, extrapolation, and comparison to NACCS

1% (100 year storm)	Taken directly from FEMA. The SWL without wave set up averaged from transects in study area.
2% (50 year storm)	Wave set up assumed to be same as 1% annual chance storm based on NACCS wave information at near shore save points.
10% (10 year storm)	Wave set up reduced to 2 feet due to significant but lesser waves in NACCS results.
0.2% (500 year storm)	SWL with waves determined by percantage increase in NACCS value of 0.2% annual chance storm from 1% annual chance storm. Wave set up only 0.5 ft more than 1% annual chance storm since NACCS waves only marginally more than 1% annual chance storm.
50% (2 year storm)	SWL reduced from 10% annual chance storm SWL by 2 feet based on difference between a 10% annual chance storm and 50% annual chance storm NACCS results. Wave set up overland assumed to be the same as a 10% annaul chance storm. Likely conservatively high.
4% (25 year storm), 1.33% (75 year storm), and 0.4% (250 year storm)	These values were dermined by plotting the previoulsy described values and curve fitting a line to them. These values were then taken from that plot Figure 4-3).

Table 4-3. VE 15 Zone Recurrence Interval Flood Elevation worksheet

Water Elevation Definition	Percent Annual Chance of Exceedance (equivalent return period)							
	50% (2 yr)	10% (10 yr)	4% (25 yr)	2% (50 yr)	1.33% (75 yr)	1% (100 yr)	0.4% (250 yr)	0.2% (500 yr)
SWL (without wave set up)	3.1	5.1		7.9		10		12.5
SWL (with wave set up)	5.1	7.1		10.4		12.5		15.5
Recurrence Int. Flood Elev.	7	9	11	13	14.3	15	17	19
Wave Height Contribution	2	2		2.5		2.5		3.5

* Values highlighted in gray were taken direclty from FIS transect tabels (example table provided as Table 4-1.
* Values in yellow were used as the reccurence interval flood elevations
* Elevation data is ft-NAVD88.
* Developed values are desrcibed below in the order they were generated. Order assisted in interpolation, extrapolation, and comparison to NACCS

1% (100 year storm)	Taken directly from FEMA. The SWL with wave set up averaged from transects in study area. Wave height calculated by subracting SWL with wave set up from BFE.
2% (50 year storm)	Wave set up assumed to be same as 1% annual chance storm based on NACCS wave information at near shore save points. Wave heights assumed to be the same overland.
10% (10 year storm)	Wave set up reduced to 2 feet due to significant but lesser waves in NACCS results. The overland wave heights were also reduced to 2 feet due to lesser energy.
0.2% (500 year storm)	SWL with waves determined by percantage increase in NACCS value of 0.2% storm from 1% storm. Wave set up only 0.5 ft more than 1% annual chance storm since NACCS waves only marginally more than 1% annual chance storm. Wave height kept the same as 1% annual chance storm.
50% (2 year storm)	SWL reduced from 10% annual chance storm SWL by 2 feet based on difference between a 10% annual chance storm and 50% annual chance storm in the NACCS results. Wave set up and wave height overland assumed to be the same as a 10% annual chance storm. Likely conservatively high.
4% (25 year storm), 1.33% (75 year storm), and 0.4% (250 year storm)	These values were dermined by plotting the previoulsy described values and curve fitting a line to them. These values were then taken from that plot Figure 4-3).

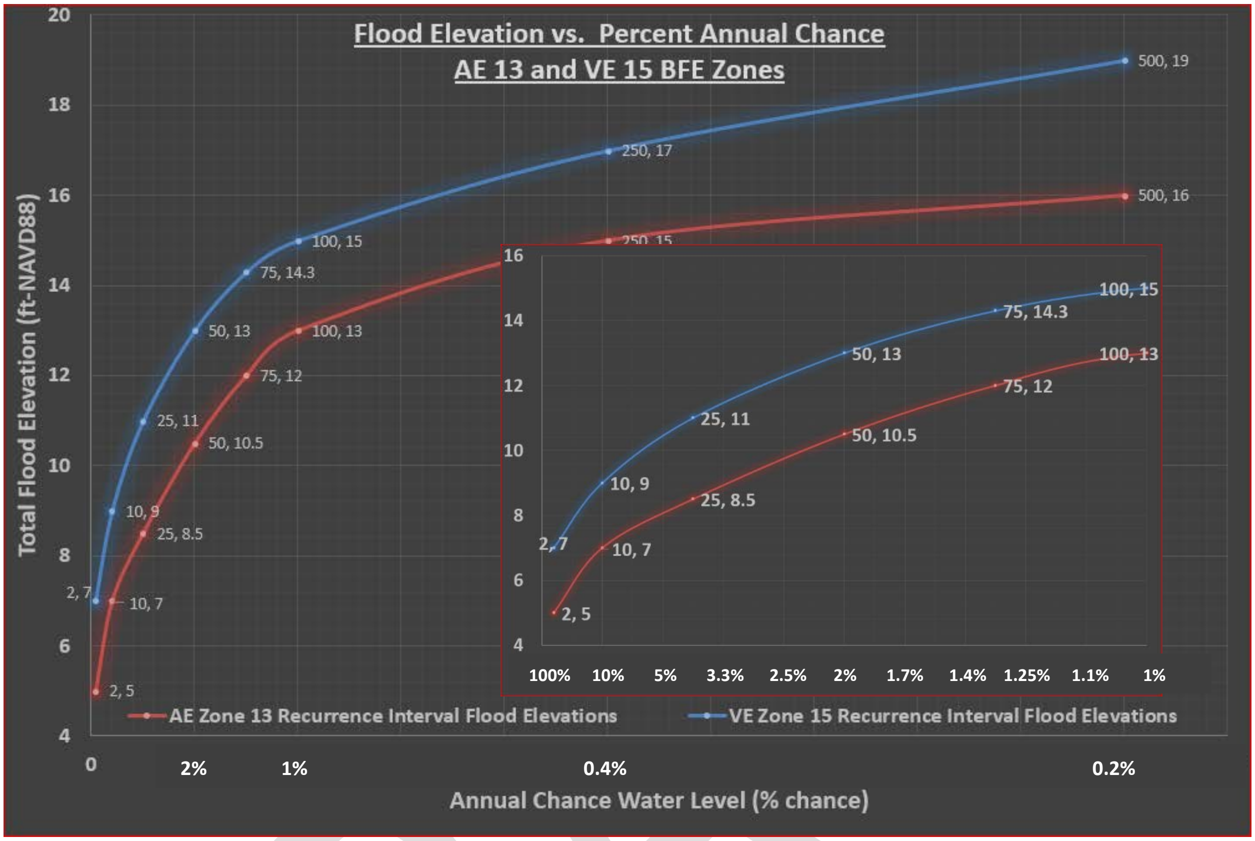

Figure 4-3. Plot of AE Zone 13 and VE Zone 15 recurrence interval used to curve fit intermediate recurrence interval storm water levels.

Table 4-4. Recurrence interval water levels vs. AE/VE zones

Annual Chance (%)	Water Elevevations (ft-NAVD88) for Each Zone							
	VE 14	VE 15	VE 16	VE 17	AE 11	AE 12	AE 13	AE 14
50	7	7	7	8	5	5	5	5
10	8	9	9	10	5	6	7	8
4	10	11	12	13	6.5	7.5	8.5	9.5
2	12	13	14	14.5	8.5	9.5	10.5	11.5
1.33	13.3	14.3	15.3	16	10	11	12	13
1	14	15	16	17	11	12	13	14
0.4	16	17	18	19	13	14	15	16
0.2	18	19	20	21	14	15	16	17

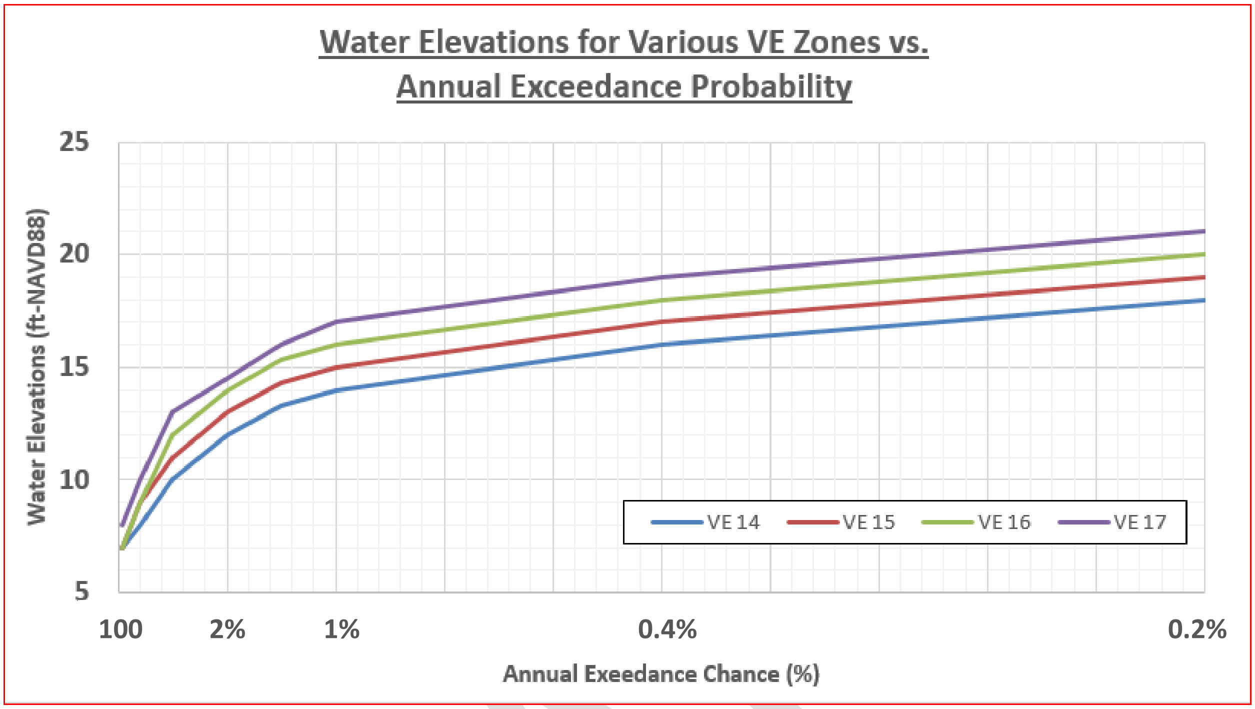

Figure 4-4. Water level vs. annual exceedance probabilities for the various VE zones in the study area

5.0 Coastal Storm Damage Reduction Measures Considered

Considering the information from the previous sections various storm damage reductions measures were considered as part of the planning process. Each of the measures is discussed in the below sections. 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 the relatively low population density along the Rhode Island coast and a fairly robust highway road system, and the relatively short distance to high ground, evacuation is very viable.

5.1 Beach Fill

Along a sandy shoreline such as the south shore of Rhode Island beach fill is usually considered as an alternative since it addresses long term erosion impacts to the shoreline and reduces flood/wave damage during storm events. Beach fill is exactly what the name describes in that sand is brought to the beach either from an upland sand source or an offshore sand deposit and placed on the beach in an engineered cross sectional template. The template design is based on existing beach elevations and slopes, dune configuration, and storm erosion potential. An example beach fill is shown in Figure 5-1.

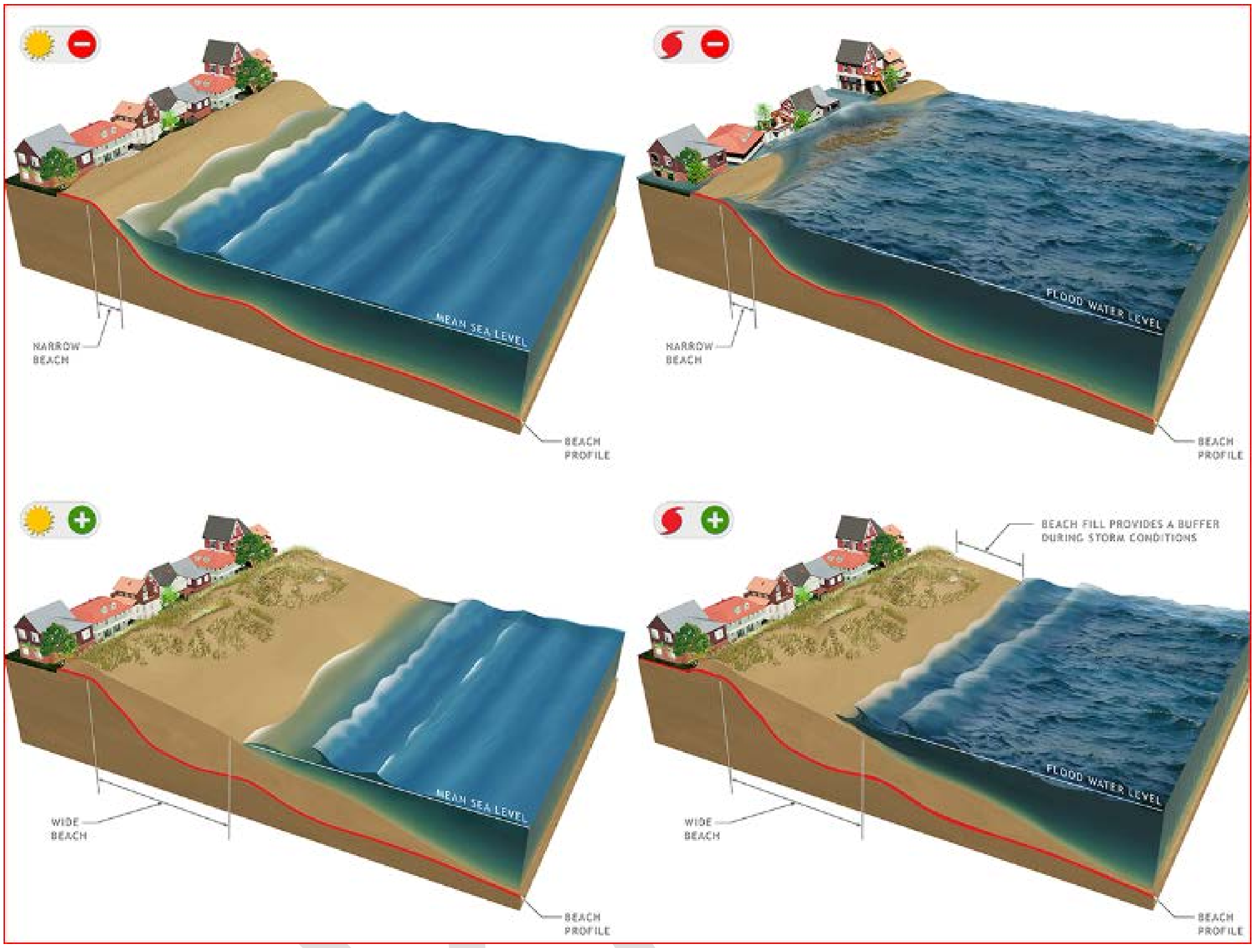

Figure 5-1. Beach fill cross section template

Beach fill alternatives typically require periodic renourishment to replace sand that is eroded from the beach system. The volume of renourishment is dependent upon the rate of sand loss and the minimum required beach to be in place to optimize economic return. Beach fill is typically seen as the least impactful storm damage reduction measure.

5.2 Revetment

A second option that was considered to prevent further shoreline retreat and to reduce wave impact damage along the sandy shorelines of the study area was revetments. A revetment is a sloped structure that is constructed of some type of placed stone or specially designed concrete unit shape (Figure 5-2). The term “rubble mound structure” is often used since the slope is constructed of stone rubble.   This is not to imply that the design and construction of such a structure is simply to dump random stone on slope. These structures are in fact engineered with structure slope, crest elevation and width, toe depth/configuration, and stone size specifically selected based on environmental forcing conditions such as water level, wave height and period, shoreline type, etc. These structures can be very effective at eliminating landward shoreline migration if properly designed and maintained. Revetments are often considered preferable solution to concrete or steel based seawalls since revetments reflect less wave energy due to the surface roughness of the structure. Increased wave reflection can impact erosion rates at the toe of structure, navigation interests, beach use adjacent to the project, etc. The downside to these structures is that due to the sloped structure front the overall width of the structure can be significant. At constructed slope of 1V:2H a 10 foot tall revetment as a slope width of 20 feet with additional width of the structure crest. Also, the beach in front of revetments is often lost since the existing erosion condition that prompted the need for such a solution still exists. The beach continues to erode from in front of the revetment and if it is not supplemented through natural sand migration or with a beach fill the beach is lost and eventually the revetment toe ends up being underwater full time. Additionally, the revetment sequesters the earthen material behind it and often that material was a source of natural beach nourishment to the system. This impacts the beach directly seaward of the revetment but also beaches down drift of the revetment.

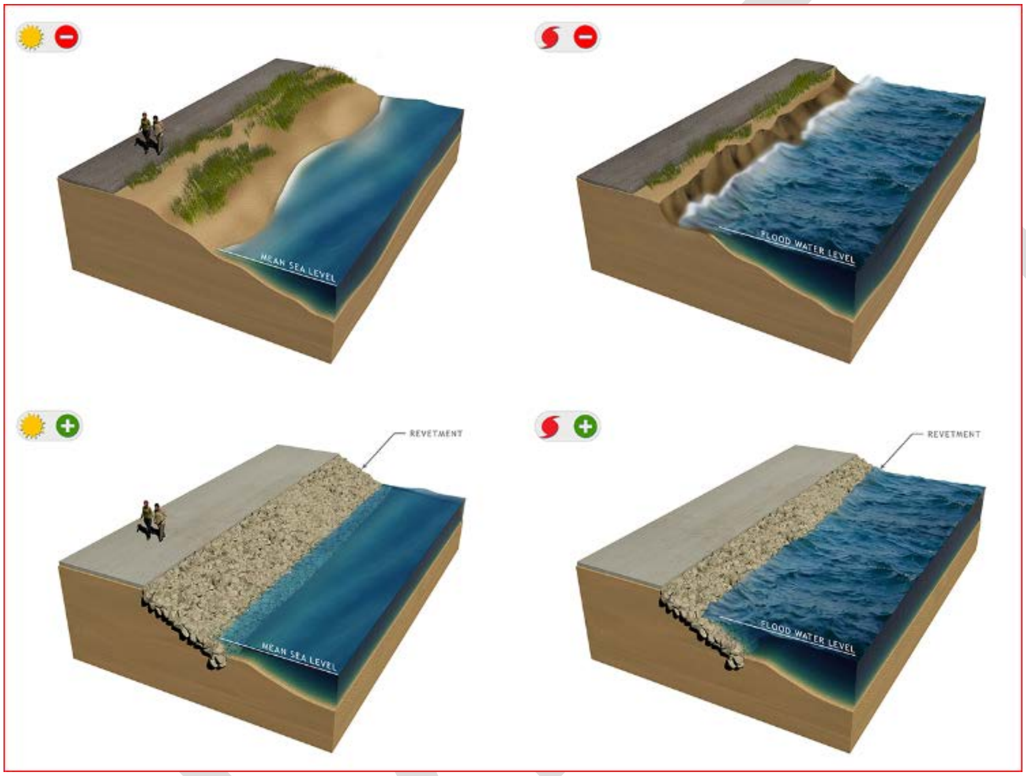

Figure 5-2. Revetment example image

5.3 Seawalls

Seawalls are structures that are typically built using concrete or steel and have same intent of a revetment. These are built to withstand the very large forces developed by breaking waves during elevated storm water conditions (Figure 5-3). Perhaps the number one reason seawalls are constructed vs. revetments is the lesser footprint these structures require for the same level of damage reduction potential. As with revetments they are very effective at maintaining a fixed shoreline position if properly designed and maintained but do have the same downsides of a revetment regarding the existing beach and to down drift beaches. As discussed in the revetment discussion, seawalls are often more reflective to wave energy than revetments and reflected wave energy, depending on the particular local environment, can have significant negative impacts.

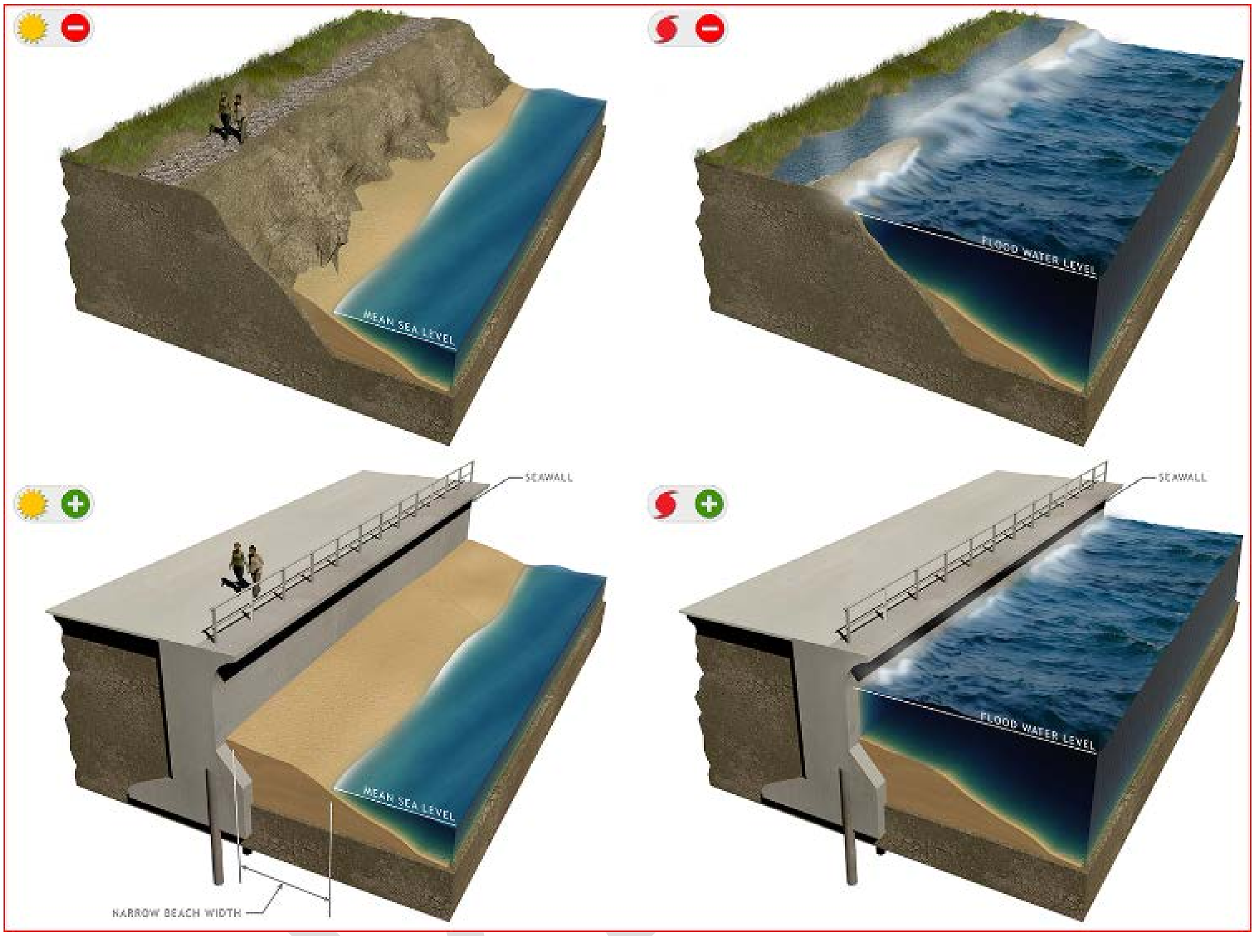

Figure 5-3. Seawall example

5.4 Flood Wall

Similar to seawalls, floodwalls are typically constructed of concrete or steel and are vertical or nearly vertical (Figure 5-4). While the purpose of both structures can overlap, typically seawalls are constructed to halt shoreline migration and to protect against wave damage while 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/maintence 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.

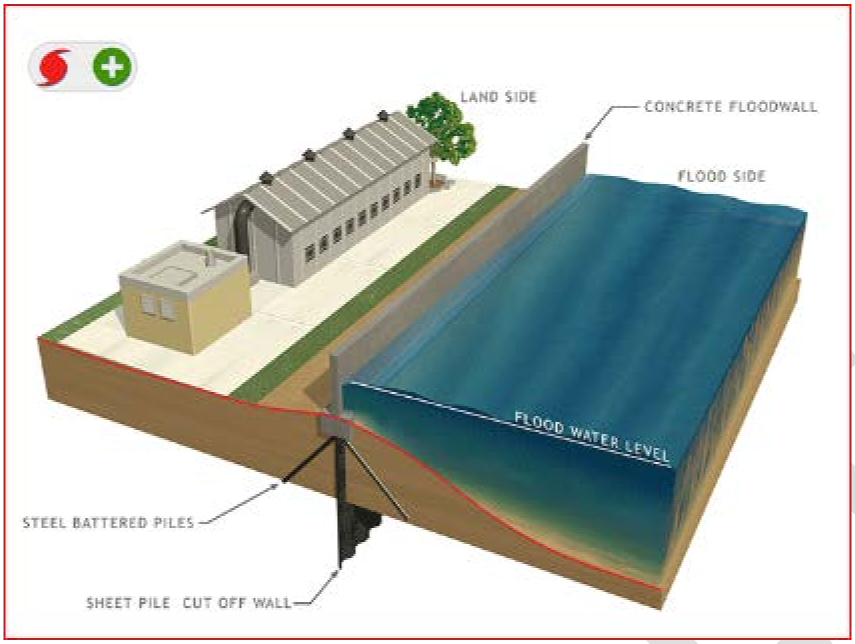

Figure 5.4. Floodwall example image

5.5 Surge Barrier (Tide Gate)

As discussed in Section 1, each of the coastal pond back bay areas could be flooded by storm water flowing over the sand barrier or through storm generated breaches in those barriers. Also the storm induced increased water levels could reach the back bay areas through the inlets that connect each pond to the ocean. As such, an alternative considered for each pond was a surge barrier, or a gate(s) that would close across each inlet to block the storm surge from entering the pond. These types of structures have been used in the US and in numerous locations around the world. These gates can vary in size from controlling the flow into small tidal creeks to massive structures blocking flow into very large rivers, navigation channels, estuaries, etc. There are many types of gate that can be used and selection is often made on cost, predicted surge elevations, navigation, bottom type, habitat considerations, etc. An example image of surge barriers has been provided in Figure 5-5. Within RI there is a hurricane surge barrier at Fox Point to reduce flood damage potential for the city of Providence.

Figure 5-5. Storm surge barrier example (sector gate type)

5.6 Structure Elevation

As discussed in the main report, the tentatively selected plan (TSP) of this study is to elevate structures in place (Figure 5-6). Basically the structures first floor living area is lifted to an elevation above the aforementioned FEMA BFE and place on piles of some type. During the cost/benefit optimization the elevation required to maximize federal investment maybe lower than that elevation, due to local zoning codes and policies structures that are elevated are done to at least the BFE. To elevate a structure the existing structure is placed on a temporary wood or steel frame, lifted off of the existing foundation or grade, moved to the side, piles are then driven into the ground, cut to a uniform elevation, and then the house is placed on top of those piles and secured. A key consideration when elevating a structure is to ensure it will not be lost due to long term erosion or short term storm erosion. The structure must be built far enough back away from the shoreline for the long term consideration and the piles must be driven deep enough and supported enough to survive the short term storm induced impacts.

Figure 5-6. Structure elevation example

5.7 Relocation

Relocation is exactly as the name suggests and includes the physical movement of an area's residents to a new area and the removal of the existing structures and infrastructure. Under this alternative the movement of the residents alone is not acceptable since the abandoned structures would pose numerous issues and dangers to the nearby communities and to the environment. The removal and disposal of the existing buildings/infrastructure must be factored into the cost of relocation. Relocation has been considered by the Corps in most studies but historically it has almost always been determined to be less cost effective than reducing the flood/damage potential for a particular area to the aforementioned considerations. As sea level continues to rise and perhaps accelerate, relocation may become more cost effective and necessary as the number of areas requiring storm damage reduction measures rise and the available funding for such efforts remains uncertain.

6.0 Beach-fx Analysis – Misquamicut/Winnapaug Pond Area

During the coastal storm damage reduction measures evaluation discussed in Section 5 and in greater detail in the main report it was determined through a screening process that the Misquamicut Beach area in Westerly, RI was the area most likely to be a candidate for a beach fill project due to the number of properties and infrastructure that could be benefited from such an alternative. The Pawcatuck River discharges at the western edge of Westerly, adjacent to the study area as shown in Figure 6-1 and includes approximately 3 miles of coastal extent and upland reaches subject only to back-bay flooding as shown in Figure 6-2. The analyzed area is vulnerable to coastal erosion, ocean-side flooding and direct wave impact as well as back-bay flooding, thus the protective measures considered include beach fill and beach fill in combination with floodwalls and a tide gate at the mouth of the Weekapaug Breachway (Winnapaug Pond inlet). Life cycle project costs and storm-induced damages were evaluated for a 50 year period with the project in place and producing benefits in 2020 (anticipated construction completion date). The goal of this study is to identify the combination of protective measures that maximize net economic benefits and are feasible from both an environmental and constructability standpoint.

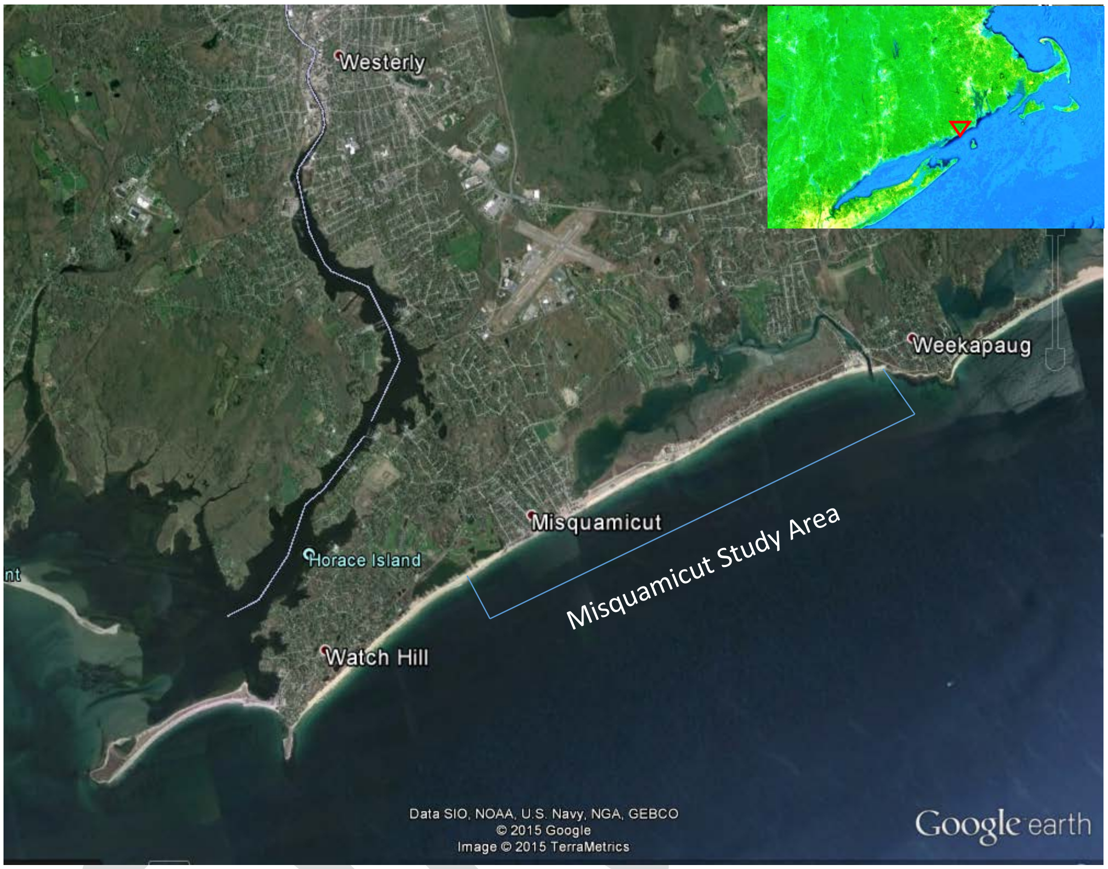

Figure 6-1. Town of Westerly, Rhode Island – Misquamicut Study Area.

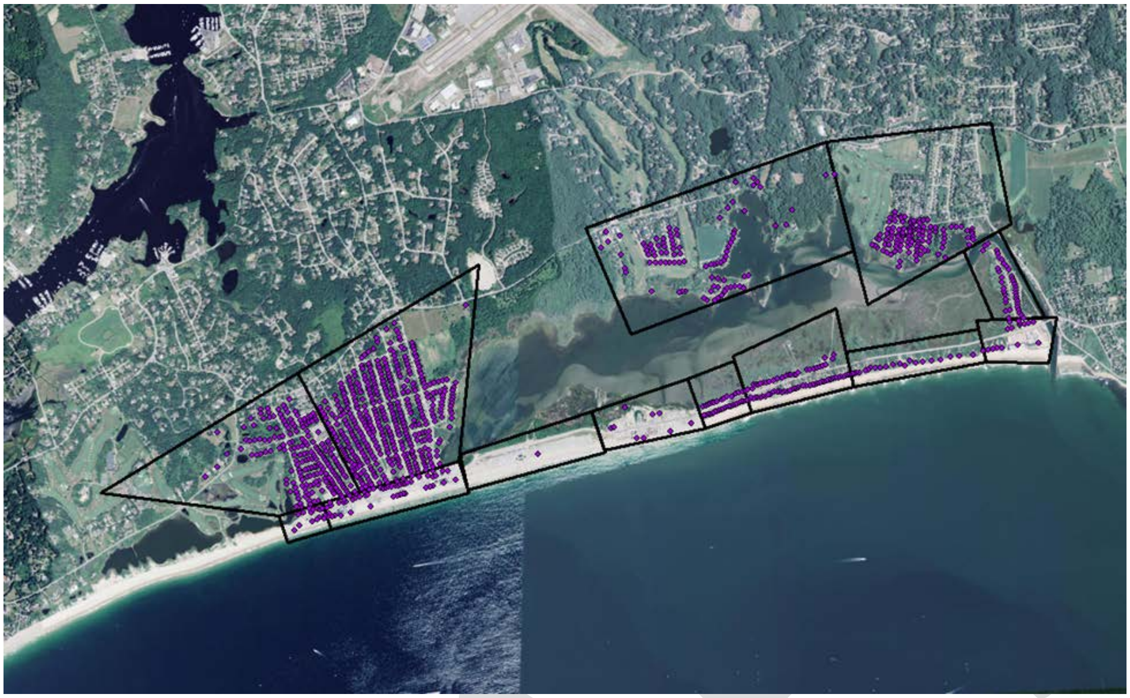

Figure 6-2. Extent of Misquamicut study area.

Beach-fx was used to analyze the physical performance and economic benefits and costs of storm damage reduction alternatives in the Pawcatuck study area. Beach-fx is an event-based, Monte Carlo life cycle simulation tool capable of estimating storm damage along coastal zones caused by erosion, flooding, and wave impact. Required inputs include meteorology, coastal morphology, economics, and management processes. Within Beach-fx, data elements are stored in relational databases where rules for applying the data elements are inherent in the program (Gravens et. al. 2007). The input data necessary to run a Beach-fx project provide a full description of the coastal area under study. The software requires an inventory of assets susceptible to damage, a set of possible storms which span the storm climatology probability space and can impact the area, the estimated morphology response of the beach to each storm in the storm set, and damage-driving parameters for estimating inundation, erosion, and wave impact damages on the structures.

Beach-fx relies on a Shore Response Database (SRD) to evolve the project area beach profiles throughout the simulated life cycle. The SRD is populated with beach profile responses to each storm in the plausible storm suite and includes cross-shore profiles of damage-driving parameters required to estimate damages at specified assets. The beach profile response model employed in this study was SBEACH, (Storm-induced BEAch CHange; Larson and Kraus, 1990), a storm-induced beach profile response program within the CEDAS (Coastal Engineering Design & Analysis System) software package. SBEACH simulates beach profile response to storm conditions and provides estimates of the cross-shore profile of wave height, water surface elevation, and erosion. These data are stored in the SRD by post-processing the SBEACH results with an import routine that is a component of Beach-fx. Input requirements for SBEACH include an initial beach profile or profiles for the study location as well as environmental forcing conditions including a time series of waves, and total water levels (tides plus surge) for each storm modeled. Within the Beach-fx framework, the project is divided into reaches, which are defined as contiguous, morphologically homogeneous areas. Reaches are defined and grouped by a representative profile, or cross sections of the beach which characterize the beach morphology. Each reach contains lots and each lot contains one or more damage elements, such as a residential home or nonresidential structure.

The purpose of Section 6 is to describe, in detail, the Coastal Engineering input employed in the Beach-fx application, and to summarize the calibration, without-project results, and with-project findings for the Pawcatuck Beach study area. This includes developing the representative reaches for the study area, a plausible storm suite, historic shoreline change conditions, and profile response to the array of storm events.

6.2 Sea Level Rise for Beach-fx Model

As discussed in Section 3.0, USACE requires several potential sea level change scenarios during planning studies. The information from Section 3.0 was used in this Beach-fx analysis accordingly and was coded within Beach-fx. Sea level change is internally computed continuously throughout the simulated project lifecycle.

6.3 Development of Beach-fx Storm Suite

As discussed in Section 4.0, state of the art numerical modeling results were available for use for the study from the NACCS modeling effort. As also discussed in Section 4.0 the Beach-fx effort and associated support models were able to use the NACCS information at the seaward boundary. In the following sections the storm parameter data and manipulation of that data for use in the Beach-fx effort is provided.

6.3.1 Analysis of Stage Frequency Curves for Selection of the Number of Forcing Locations

Figure 6-3 shows the location of the storm surge (ADCIRC) and wave (STWAVE) save points in the Misquamicut project area from the NACCS study discussed in Section 4.0. The potential project area spans about 3.5 miles and an analysis was conducted to determine if one set of driving conditions could be applied to the entire extent of the project site. Stage frequency information for five save stations in the nearshore (around 1500' offshore) region of the site was extracted from the CHS to determine the similarity/difference in water level response in this region (Figure 6-4; Table 6-1). The locations were selected offshore of the project site in approximately -15 to -45 ft-NAVD88 water depth. From these curves, a negligible difference in the water levels is observed. CHS ADCIRC water level station 227 at a water depth of 25 ft-NAVD88 and location of 41°19' 28.49" N and 71°46' 17.36" E (see Figure 6-3) was selected for extracting water level forcing conditions for input to the SBEACH model. Wave data is extracted from the STWAVE save point collocated at this ADCIRC save point (wave save station 12).

Figure 6-3. ADCIRC save point stations from the NACCS study (accessed through the CHS)

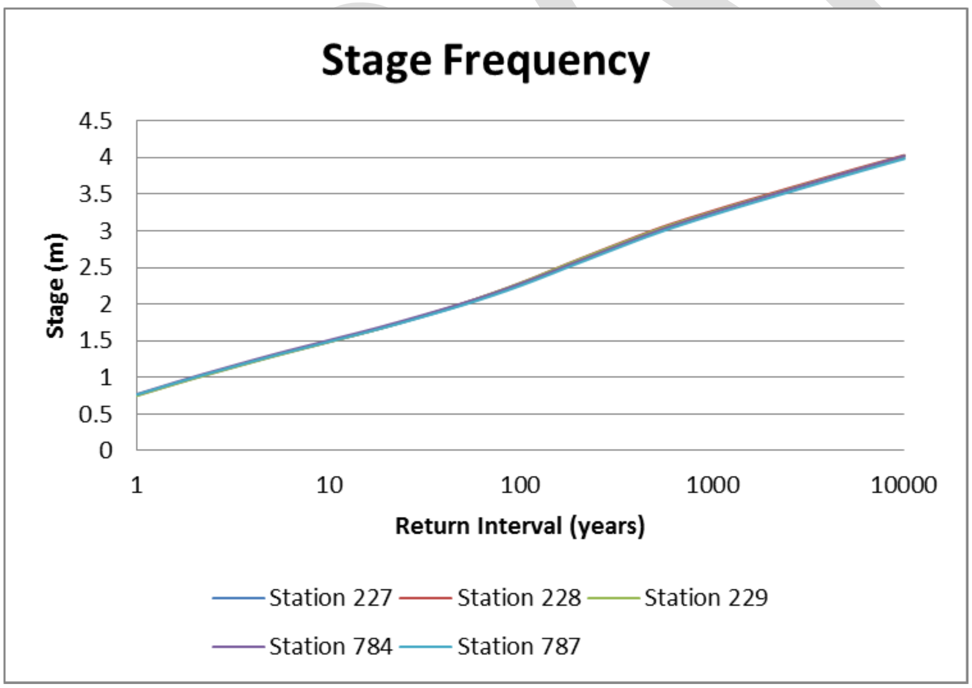

Figure 6-4. Stage frequency curves at NACCS stations near Misquamicut, RI.
Datum is meters-MSL and taken from base conditions results (surge only, no tides)

Table 6-1. CHS Water level (m) for tropical storms at select return periods for Stations near Misquamicut, RI					
Station	227	228	229	784	787
Return Period					
1	0.76	0.75	0.75	0.77	0.77
2	1.00	0.99	0.99	1.01	1.00
5	1.28	1.28	1.28	1.30	1.29
10	1.49	1.49	1.48	1.51	1.49
20	1.69	1.69	1.69	1.71	1.69
50	1.99	2.01	2.01	2.01	1.99
100	2.27	2.29	2.29	2.28	2.25
200	2.59	2.61	2.61	2.59	2.56
500	2.99	3.02	3.00	2.99	2.96
1000	3.25	3.27	3.26	3.24	3.22
2000	3.49	3.51	3.49	3.49	3.45
5000	3.79	3.81	3.80	3.80	3.76
10000	4.01	4.03	4.02	4.02	3.98

Datum is meters-MSL and taken from base conditions results (surge only, no tides)

6.3.2 Time Series Analysis for Tidal Amplitudes

Future storms can occur at any time during the spring-neap tidal cycle, therefore, the inclusion of the range of tidal amplitudes and the timing of occurrence of the tide (tide phase) must be accounted for and combined with the storm surge hydrograph. This is accomplished by combining a statistically defensible simplified (cosine) tidal signal representing the expected tidal amplitude (high, mean, and low) with the surge hydrographs at four phases of the tide; high tide, mean tide falling, low tide, and mean tide rising. To accomplish this, the high, mean, and low tide amplitudes must be computed at the SBEACH model forcing location (save point 227).

First an equilibrium harmonic tide signal for a full tidal epoch of 19 years at a 10 minute time step is computed. The computed equilibrium harmonic tide signal was based on application of the East coast 2001 Database of Tidal Constituents (Mukai et al. 2002). Then a CDF (cumulative distribution function) of the tide elevation is created from the 19 year harmonic tide signal and analyzed to estimate statistically representative tidal ranges. Because tidal elevations are provided relative to mean sea level and a simplified (cosine) tidal signal is assumed, the tidal range is twice the tidal amplitude. A high tide range was estimated (representing Spring tides) by computing the mean value of the largest 25 percent of all tidal ranges. The mean tide range was estimated by computing the mean value of the central 50 percent of all tidal ranges and a low tide range was estimated (representing Neap tides) by computing the mean value of the lowest 25 percent of all tidal ranges.

The ranges are estimated by partitioning the 19-year tidal time series into water level bins, each with a water level increment of 0.0005-m. The cumulative distribution function at this location is shown in Figure 6-5. The extreme 12.5% water level values in the CDF represent the spring tide range, therefore the extreme *highest* 6.25% water levels and the extreme *lowest* 6.25% water levels, were used to compute the high tide amplitude. From this analysis, the high tide amplitude was estimated at 0.539 m. The next most extreme 25% (representing the CDF curve from 0.0625 to 0.1875 and from 0.8125 to 0.9375) were used to compute the mean tide amplitude. From this analysis, the mean tide amplitude was estimated at 0.393 m. Lastly, the low tide range was computed based on the next 12.5% water levels (0.1875 to 0.25 and 0.75 to 0.8125). From this analysis, the low tide amplitude was estimated at 0.302 m.

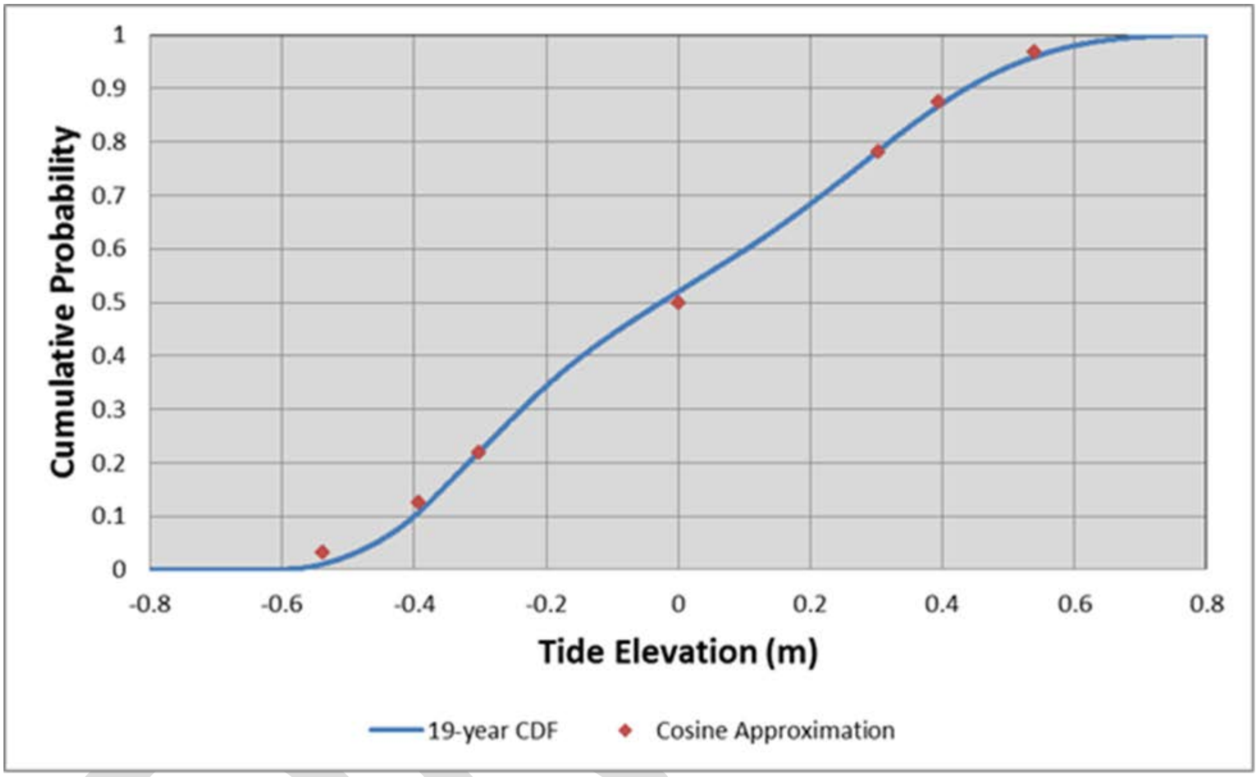

Figure 6-5. Cumulative Probability Distribution Curve for Tides at Pawcatuck with Representative Points Indicating High and Low Tide Elevations for Statistically Representative High, Mean, and Low Tide Ranges.

6.3.3 Selection of Storms

Step 1: Minimum Water Level Threshold

The selection of storms to use as part of the forcing conditions for the SBEACH model simulations required an examination of all potential NACCS storms and then the selection of storms that best represent the water level responses at CHS Station 227. The first criterion for selecting storms is that a given storm peak water level at Station 227, including the low tide amplitude computed in the previous section (0.302 m), exceeds the NACCS 1-year return period water level of 1.17 m given in Table 6-2 for Tropical Base+Tide conditions. Any storm producing a peak water level less than the 1-yr return period water level is considered inconsequential and was eliminated from further consideration. Employing this criterion reduced the number of potential tropical storms from 1050 to 481. The same analysis was performed for the extra tropical storm events resulting in 72 extra tropical storms exceeding the minimum threshold in Table 6-2 (0.68 m) (over a 75 year time period).

Table 6-2. Tropical and Extratropical CHS Water level (m) at select return periods for Station 227 in Pawcatuck, RI				
Station	**Tropical**		**Extra tropical**	
Return	**Base Only**	**Base + Tide**	**Base Only**	**Base + Tide**
1	0.77	**1.17**	0.68	0.8957
2	1.07	1.38	0.89	1.12
5	1.35	1.65	1.15	1.3786
10	1.51	1.83	1.32	1.549
20	1.64	2.01	1.49	1.7004
50	2.04	2.26	1.68	1.8736
100	2.43	2.48	1.82	1.9905
200	2.76	2.77	1.9385	2.0928
500	3.15	3.18	2.09	2.2108
1000	3.44	3.46	2.19	2.2886
2000	3.73	3.72	2.28	2.3577
5000	4.11	4.03	2.39	2.4374
10000	4.37	4.26	2.47	2.4899

Datum is meters-MSL

Step 2: Elimination of storms outside a 200 km radius

The NACCS synthetic tropical storms are separated into Region1, Region2, and Region3 and included bypassing storm tracks and landfalling storm tracks (Melby and Green, 2015). The 1050 NACCS synthetic tropical storms propagated along 130 defined storm tracks shown in Figure 6-6. As a second criterion for tropical storm selection, only storms along those tracks that passed within a 200 km radius of the study area were considered as potential forcing conditions to the model. (The 200-km radius zone is considered the area of influence for each NACCS synthetic storm event (Nadal-Caraballo and Melby, 2014).) The storms that passed within a 200-km radius of the study site were cross-referenced with the storms selected in Step 1, resulting in the inclusion of 235 storms to be used in the next step of the analysis.

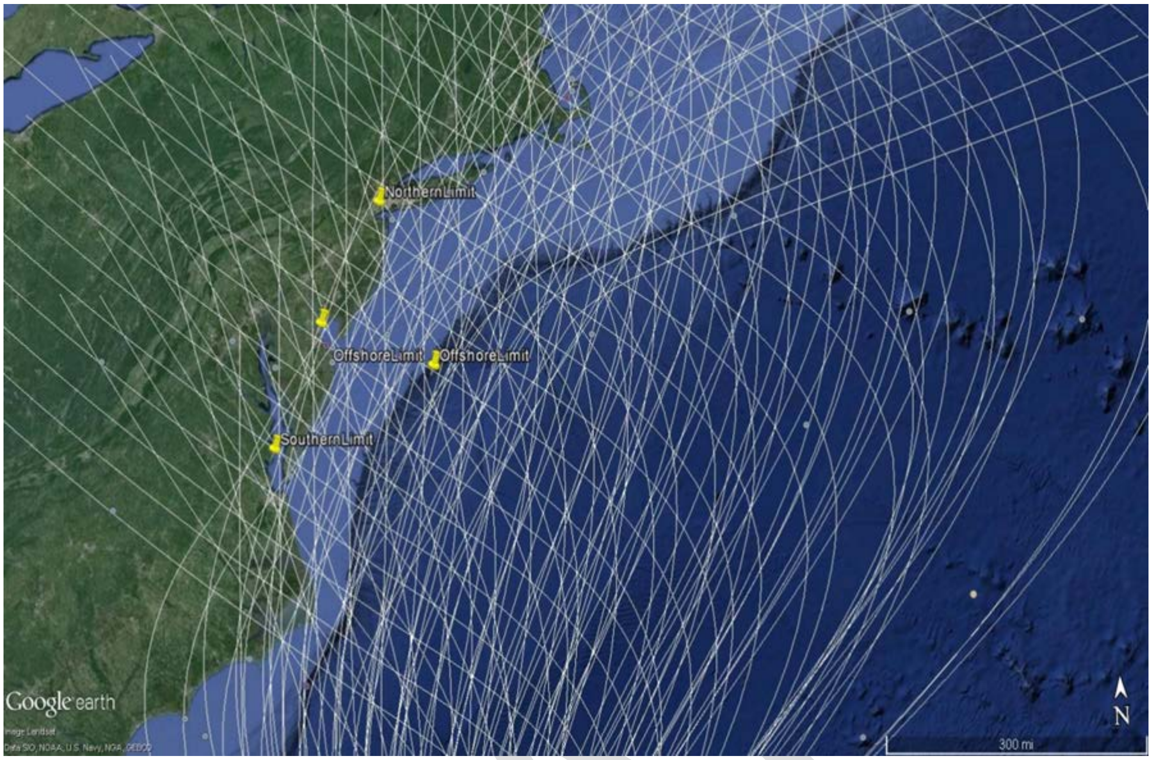

Figure 6-6. Tropical storm tracks from the NACCS Study.

Step 3: Binning of Storms

To further reduce the number of unique storms in the plausible storm suite the storm surge hydrograph time series, within each range shown in Table 6-2, were examined and representative storms were selected to characterize storms within that bin. For the 307 (235 tropical storms and 72 extra tropical storms) remaining storms, the tropical storms that met the Step 1 and Step 2 criteria were binned into 10 groups based on peak water level including the mean tide amplitude of 0.393 m to represent Base+Tide return periods of 1, 2, 5, 10, 20, 50, 100, 200, 500, and 1000-yr or less frequent in Table 6-2. Storm hydrographs in each of the 10 groups were then plotted together to identify similarities or differences in the time history of water level to achieve a given peak water level. If all storms in a given group have water level time histories that looked similar (hydrographs of similar shape), then one storm was selected to represent that group. If the time histories had markedly different hydrographs, then several storms were selected to represent that portion of the stage-frequency curve. Storm duration was key in determining if one or more storms would be selected from a given storm bin.

As an example, all storms with peak surge+mean tide that fall into the 10-year base+tide return period bin were grouped or binned together. The 10-year bin is defined from the mid-point between the 5yr- and 10-yr return period values ((1.74 m) to the mid-point of the 10- and 20-year return periods (1.92 m). The tropical storms that met these criteria were selected and base hydrographs are illustrated in Figure 6-7. Note that the plots do not include the mean tide, therefore the peaks are in the 1.53 to 1.75 m range. These storms were analyzed to determine a representative storm or storms for that bin. The same procedure was done for the 72 remaining extratropical storms. Following the procedure described in Steps 1 through 3, representative storms were selected and combined with wave and tidal forcing for input to SBEACH.

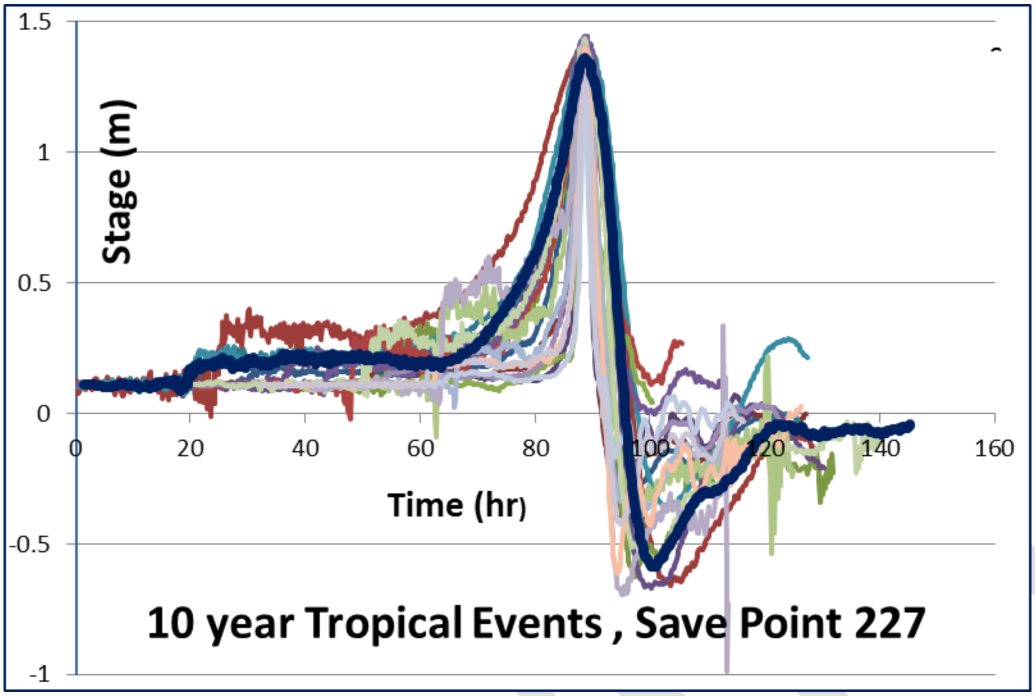

Figure 6-7. All tropical storm with peak water levels (m-MSL) within the 200-yr return period bin.

The number of storms occurring within each cluster and the selected representative storm ID numbers is documented in Table 6-3. The 100 Extratropical storm events were reduced to 6 representative storm events and the 1050 tropical storm event were reduced to 13 representative storm events.

Table 6-3: Bin thresholds and selected storms

Storm Return Period (Yr)	Stage of Tropical storms, Base +tide (m) MSL	Mid Points (m) MSL	No. of storms (in each cluster)	Selected storm ID's , Tropicals	Stage of extra-tropical storms, Base +tide (m) MSL	Mid Point s (m) MSL	No. of storms (in each cluster)	Selected storm ID's Extra-tropicals
1	1.17			419	0.90		2	53
2	1.38	1.28	72	428	1.12	1.01	42	68
5	1.65	1.52	39	511, 460, 466	1.38	1.25	15	50
10	1.83	1.74	35	569	1.55	1.46	7	22
20	2.01	1.92	32	207	1.70	1.62	5	37

50	2.26	2.14	19	446, 235	1.87	1.79		
100	2.48	2.37	12	367	1.99	1.93		
200	2.77	2.63	13	546	2.09	2.04		
500	3.18	2.98	4	503	2.21	2.15		
1000	3.46	3.32	3	458	2.29	2.25		
2000	3.72	3.59			2.36	2.32		
5000	4.03	3.88			2.44	2.40	1	7
10000	4.26	4.15			2.49	2.46		

6.3.4 Aligning Surge and Wave Time series

Once storms were selected based on the surge hydrographs, the associated wave height and period time series were extracted from the NACCS database. The NACCS wave data from STWAVE are provided at 15 minute, 30 minute or 60 minute time steps, depending on the forward speed of the modeled storm. All ADCIRC outputs are provided at 10 minute intervals. The time series of waves and water level were appropriately aligned for input to SBEACH.

6.3.5 Development of SBEACH input files.

Each storm surge hydrograph (extratropical and synthetic tropical) is combined with a cosine representation of the astronomical tide to generate a plausible total water level elevation. The cosine tidal signal representing the three tidal ranges (high, mean, and low) and the peak surge elevation was aligned with four tidal phases (high tide, mid-tide falling, low tide, and mid-tide rising) to create suite of 12 plausible representations of each historical storm surge hydrograph. The high, mean and low tidal amplitudes (1.77, 1.29 and 0.99 ft) were obtained from an analysis of a 19-year-long equilibrium tide as discussed in section 6.3.2. An example of the results of this procedure is provided in Figure 6-8.The 6 extra tropical storms were expanded to a plausible storm suite consisting of 72 events and the 13 tropical storms were expanded to a plausible storm suite consisting of 156 events.

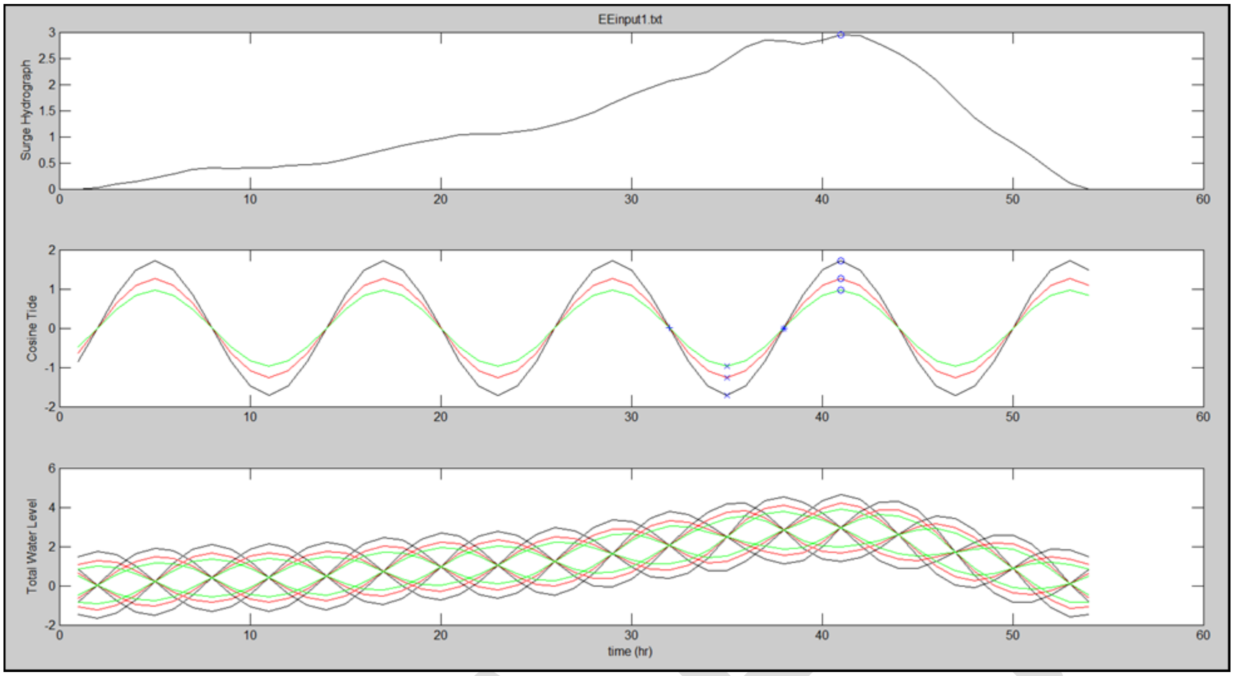

Figure 6-8. Storm surge hydrograph, cosine tide and 12 total water levels an extratropical storm event.

The water level information to this point in the analysis has been referenced to Mean Sea Level (MSL). Datum conversion to NAVD88 was performed for compatibility with the beach profile input to SBEACH and the datum associated with the asset inventory attributes. Datum conversion from MSL to NAVD88 was calculated by subtracting 0.37 ft from the NACCS MSL water level information based on the datum conversion for save station 227 provided in CHS.

6.4 Representative Beach Profiles and Reach Designation

As mentioned, Beach-fx was employed to analyze the physical performance and economic benefits and costs of alternatives. For a general description of the principles upon which Beach-*fx* operates the reader is directed to Gravens, et al. (2007). An overview of the general hierarchical data structure employed in Beach-*fx* is provided in Figure 6-9. Within Beach-*fx,* the overall unit of analysis is the "project," a shoreline area for which the analysis is to be performed. The project is divided into reaches characterized by a representative beach profile or cross section of the beach morphology. The structures within a reach are referred to as Damage Elements (DEs), and are located within lots. All locations are geospatially referenced using a cartographic coordinate system such as state plane coordinates. This project definition scheme is shown schematically in Figure 6-10, in which the shoreline is linearized into reaches. Each reach is associated with a representative beach profile that describes the shape of the cross-shore profile and beach composition.

A reach is defined as contiguous, morphologically homogeneous stretch of beach; however morphologic features of the existing beach, such as dune height, berm width, and offshore profile shape, typically vary along the project study domain. Therefore to accurately estimate storm erosion response for the existing condition, a set of representative morphologic reaches are developed to describe variations in profile shape along the project domain. Morphology analysis software applications such as BMAP or RMAP can be used to define morphologic reaches by analyzing profiles, grouping similar profiles, and calculating an average representative profile for each reach. According to the CEM, the profile characteristics that should be considered when developing morphologic reaches include dune height and width, berm width, nearshore and offshore profile slopes, sand grain size, presence of seawalls or other structures, and proximity to inlets.

The profile is the basic unit of beach response. Natural beach profiles are complex; for the modeling, a simplified or idealized beach profile, representing key morphological features defined by points, is used as shown in Figure 6-11. The idealized profile represents a single trapezoidal dune with a horizontal berm and a horizontal upland landward of the dune feature. The submerged portion of the profile is represented by a detailed series of distance-elevation points that are determined through an analysis of available beach profile information. For the Pawcatuck project, the detailed submerged beach profile was developed by averaging across multiple surveyed beach transects containing similar offshore slopes.

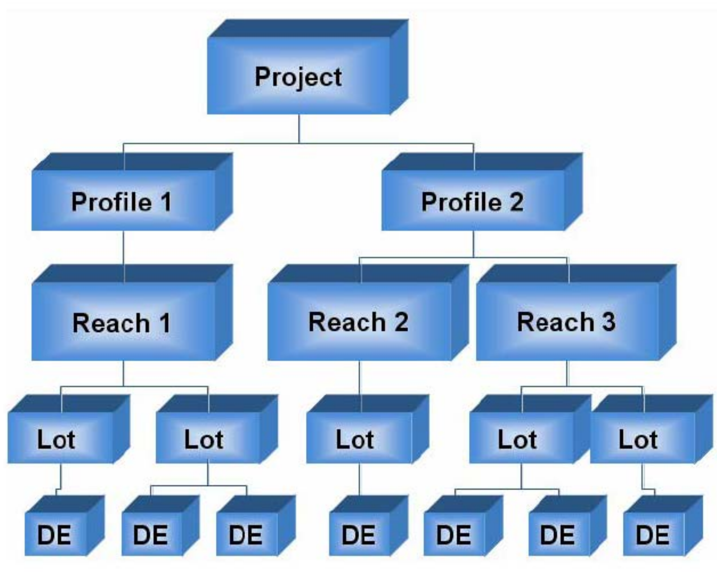

Figure 6-9. Hierarchical representation of Beach-fx data elements (taken from Beach-fx Users Manual, Version 1.0).

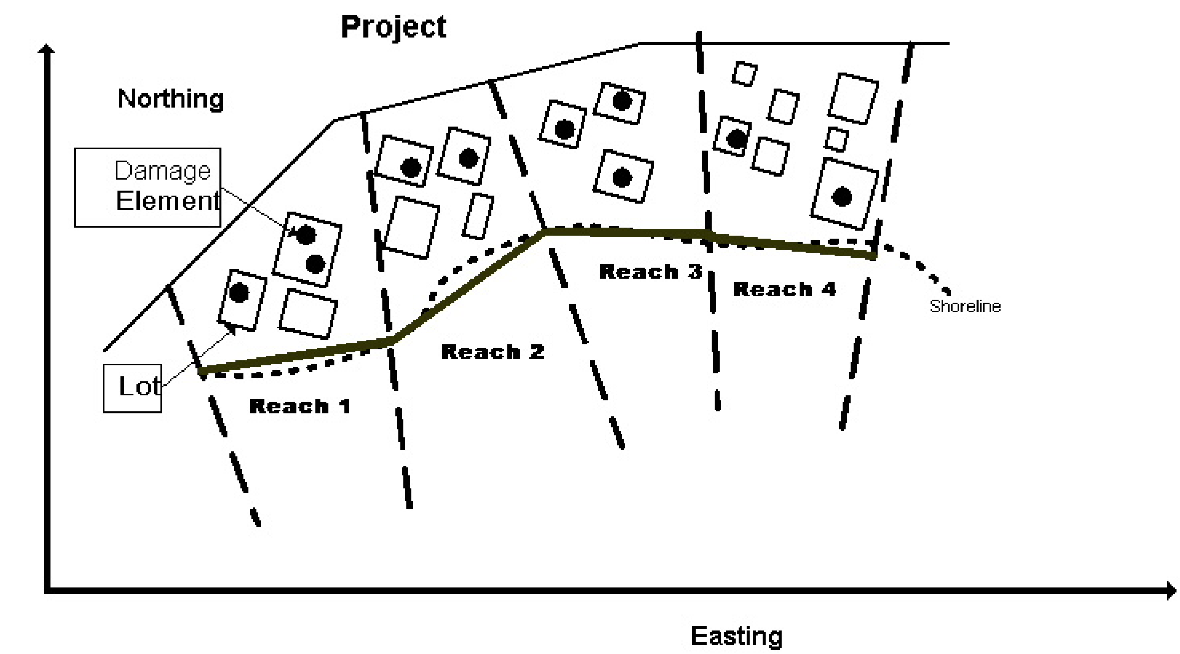

Figure 6-10. Beach-fx schematization of the project study area.

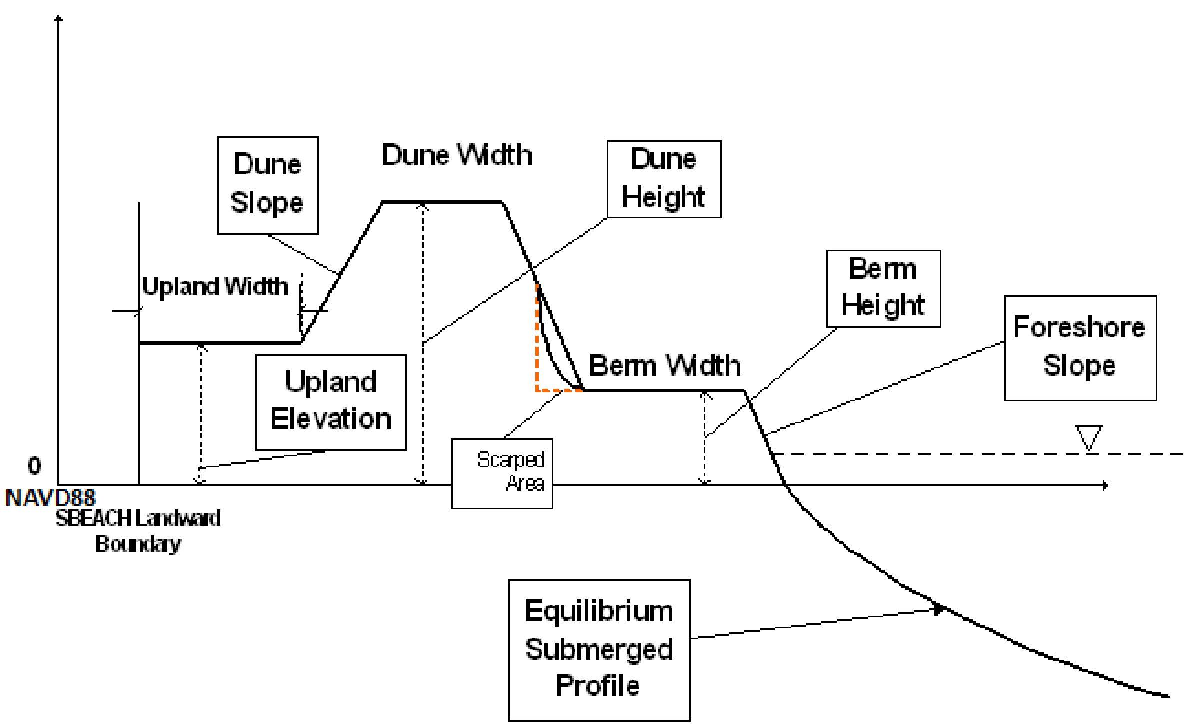

Figure 6-11. Beach-fx idealized beach profile.

6.4.1 Geospatial Data

CENAE provided 18 transects of the beach profile from 2007 and 2010 Lidar based topography and bathymetry (Figures 6-12 through 6-14). Further, 2015 surveys were provided by CENAE at each of the 18 transects as seen in Figure 6-15. These 2015 data were used to verify the characterization of the dry beach portion of the representative profiles derived from the 2010 Lidar transects and captured the recent morphological changes.

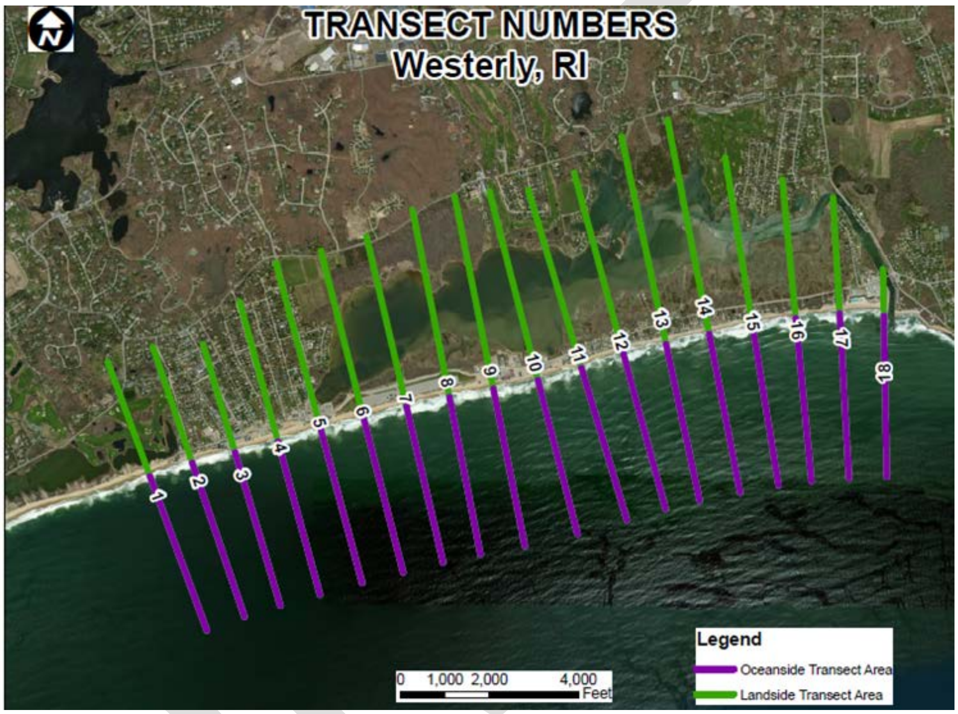

Figure 6-12. Location of Lidar transects.

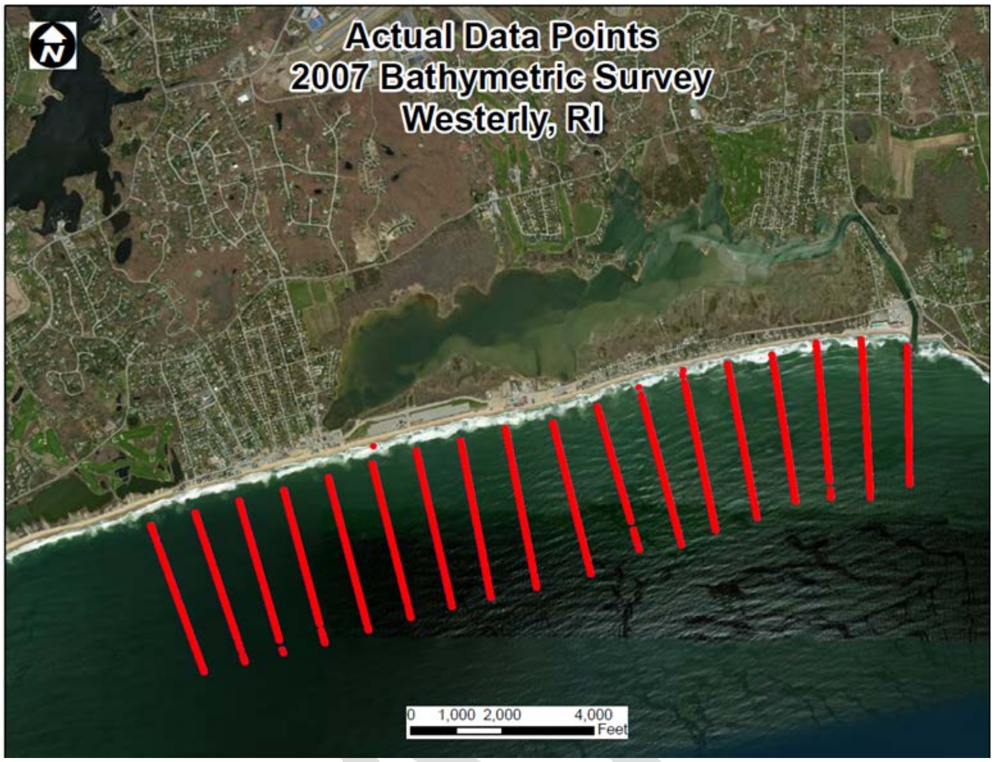

Figure 6-13. Location of 2007 bathymetry transect data.

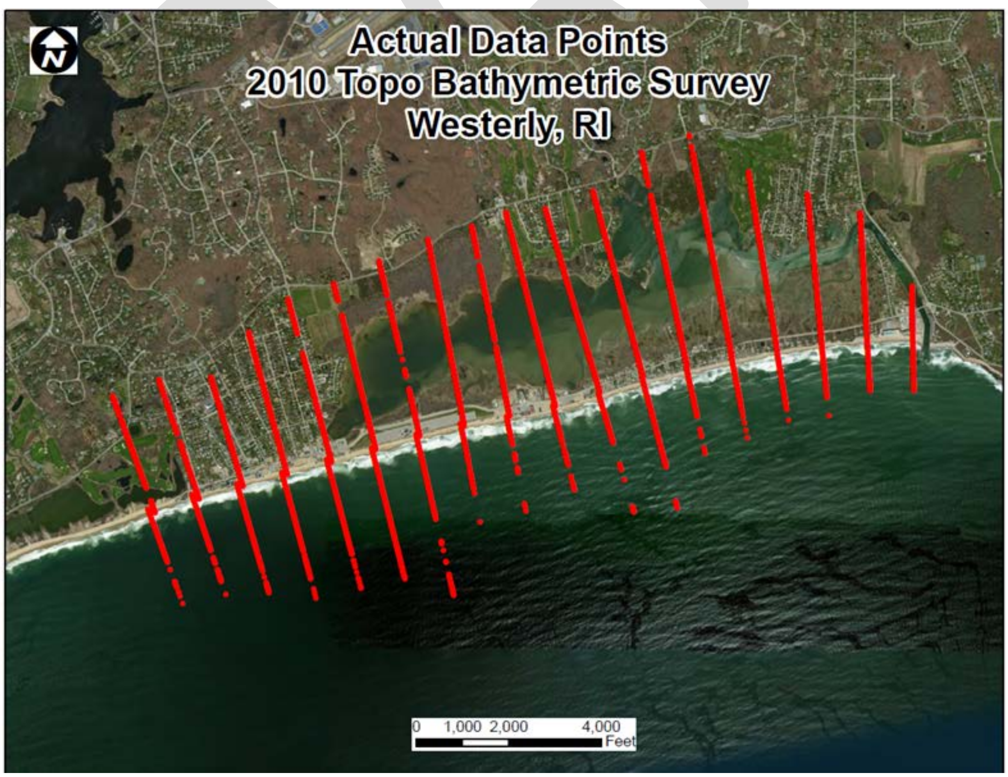

Figure 6-14. Location of 2010 topographic and bathymetry transect data.

Figure 6-15. Location of 2015 survey data provided at 18 transects.

The transects from the 2010 Lidar extractions were grouped and averaged to result in four representative existing condition profiles. These groupings align comparable profiles to preserve the morphological features of each profile.

Figure 6-16. Extents of the 4 profiles developed from the 18 original lidar transects.

6.4.2 Idealized profile parameters

Based on the 4 representative profiles, the upper beach profile was simplified and idealized for input into Beach-fx. The idealized Beach-fx profile (Figure 6-11) requires the seaward and landwared dune slope to be equal, which may result in a wider idealized dune width to account for the typically milder landward dune slope that was observed in the measured beach profiles. The berm elevation is constant across the project study area and was determined from an analysis of the measured beach profiles. Figures 6-17, 6-19, 6-21, and 6-23 show the averaged representative profiles, while Figures 6-18, 6-20, 6-22 and 6-24 show the idealized version of those representative profiles. The veritcal datum is ft-NAVD88 for those figures. Table 6-4 records the morphological parameters of each of the developed idealized profiles.

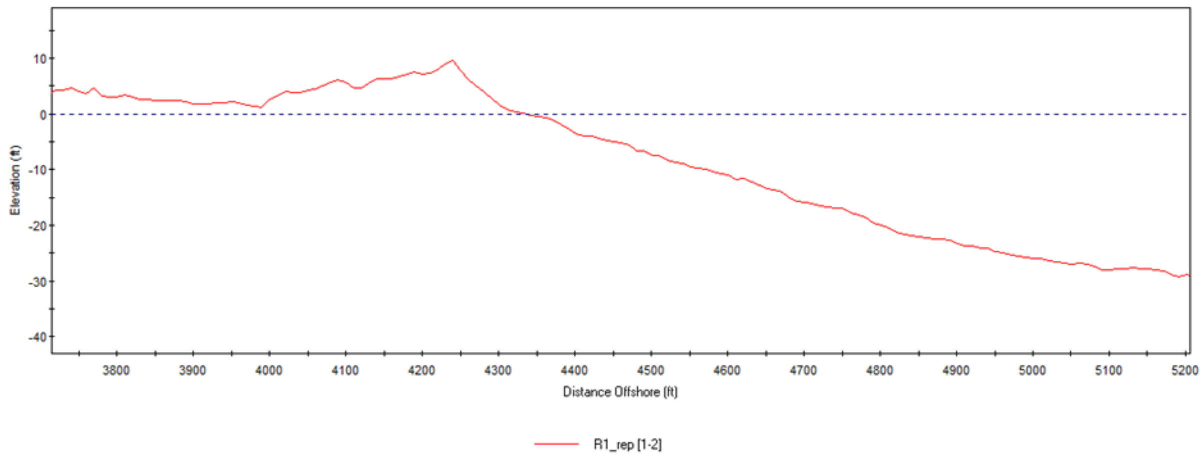

Figure 6-17. 2010 Representative profile for transects 1 and 2, Profile 1.

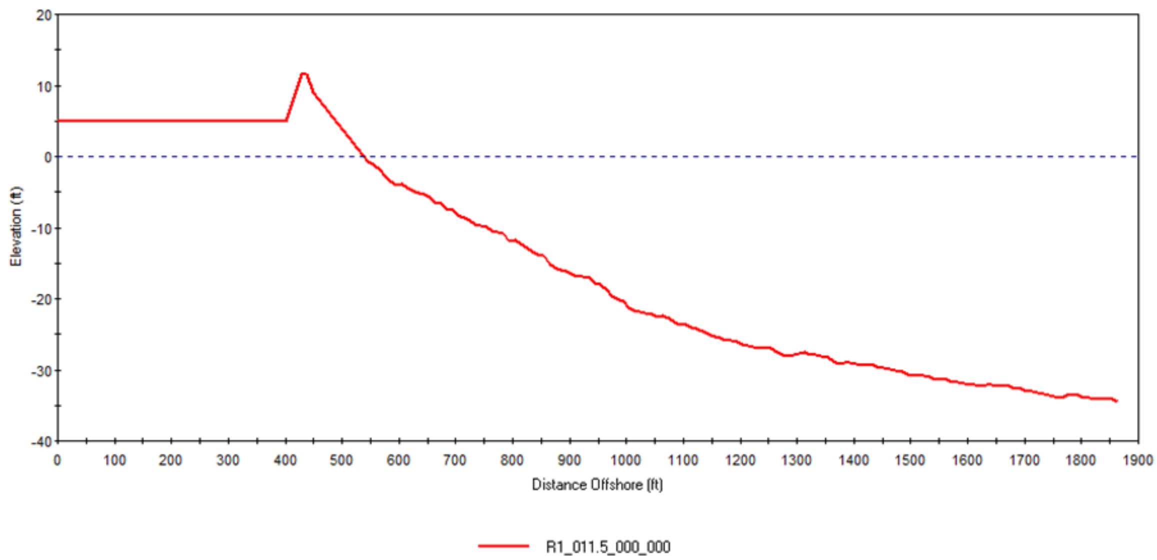

Figure 6-18. Idealized profile for transects 1 and 2, Profile 1.

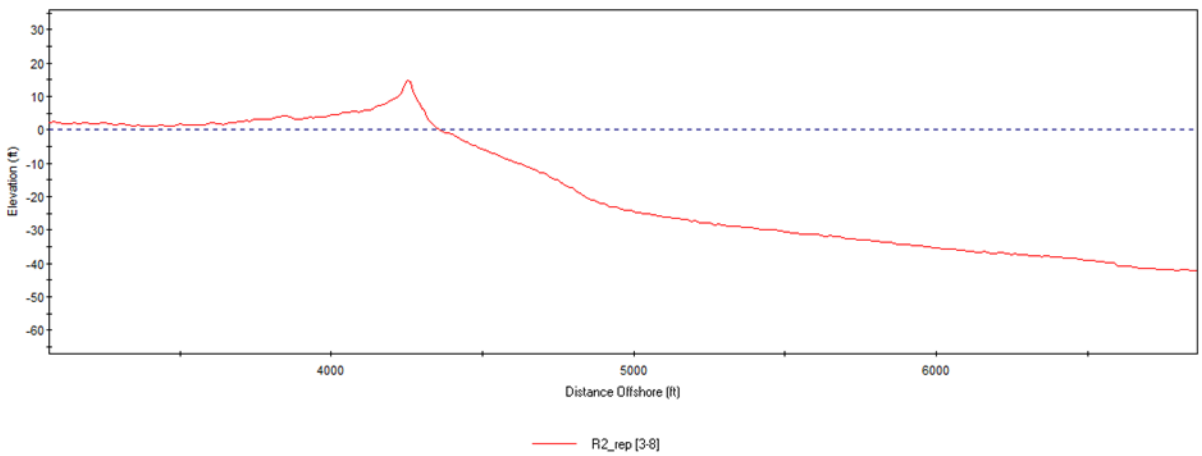

Figure 6-19. 2010 Representative profile for transects 3 through 8, Profile 2.

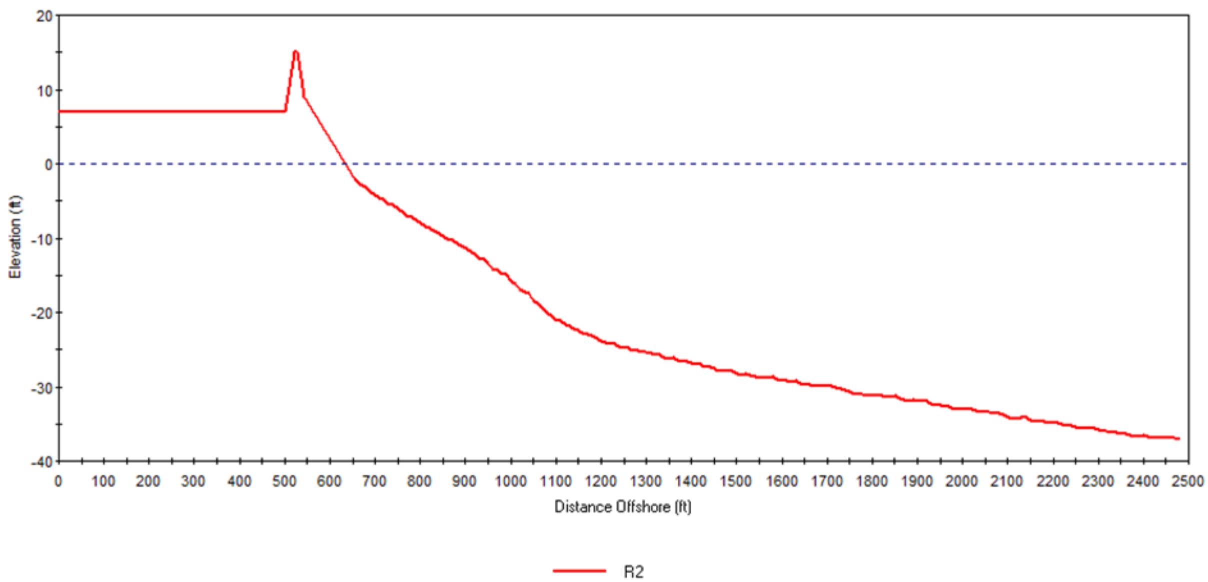

Figure 6-20. Idealized profile for transects 3 through 8, Profile 2.

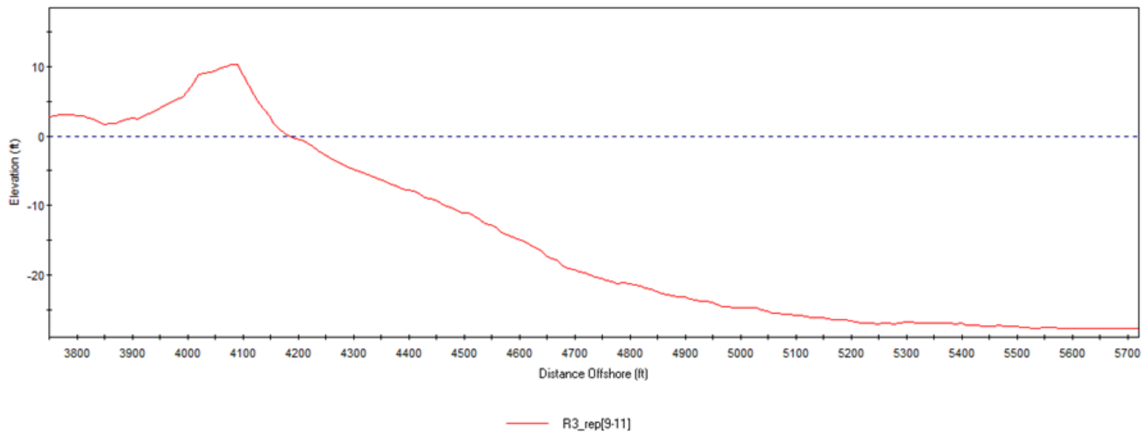

Figure 6-21. 2010 Representative profile for transects 9 through 11, Profile 3.

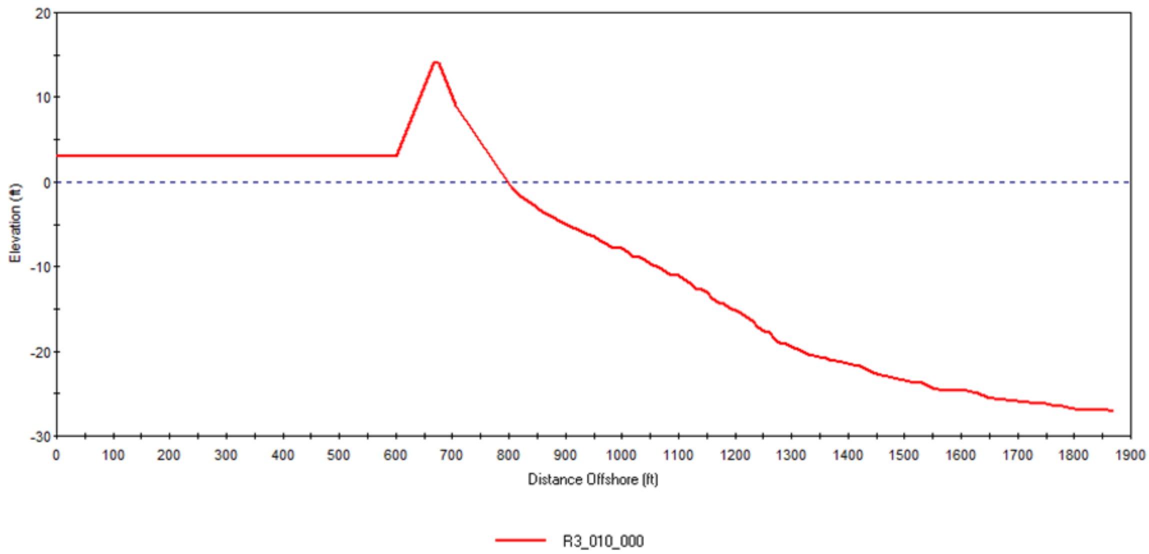

Figure 6-22. Idealized profile for transects 9 through 11, Profile 3.

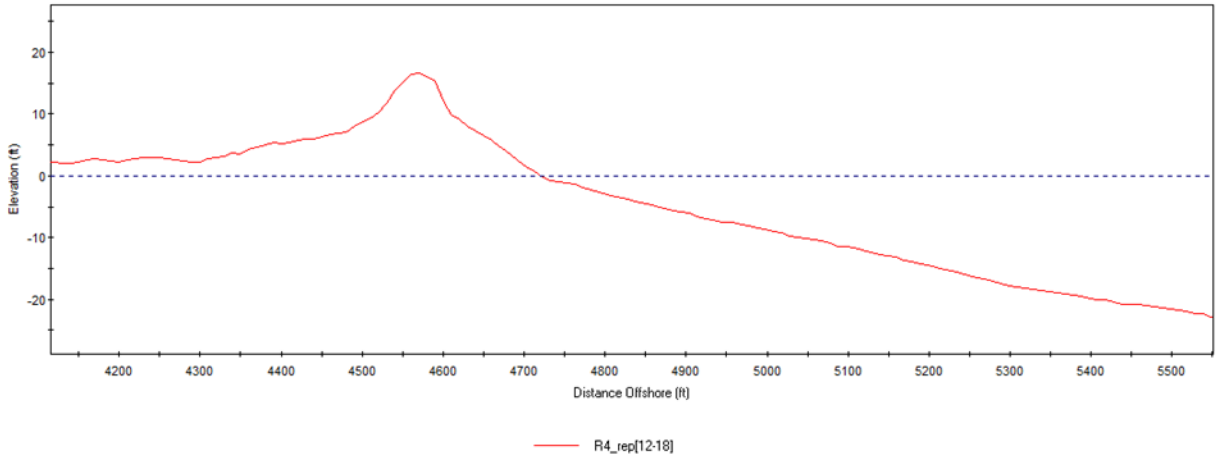

Figure 6-23. 2010 Representative profile for transects 12 through 18, Profile 4.

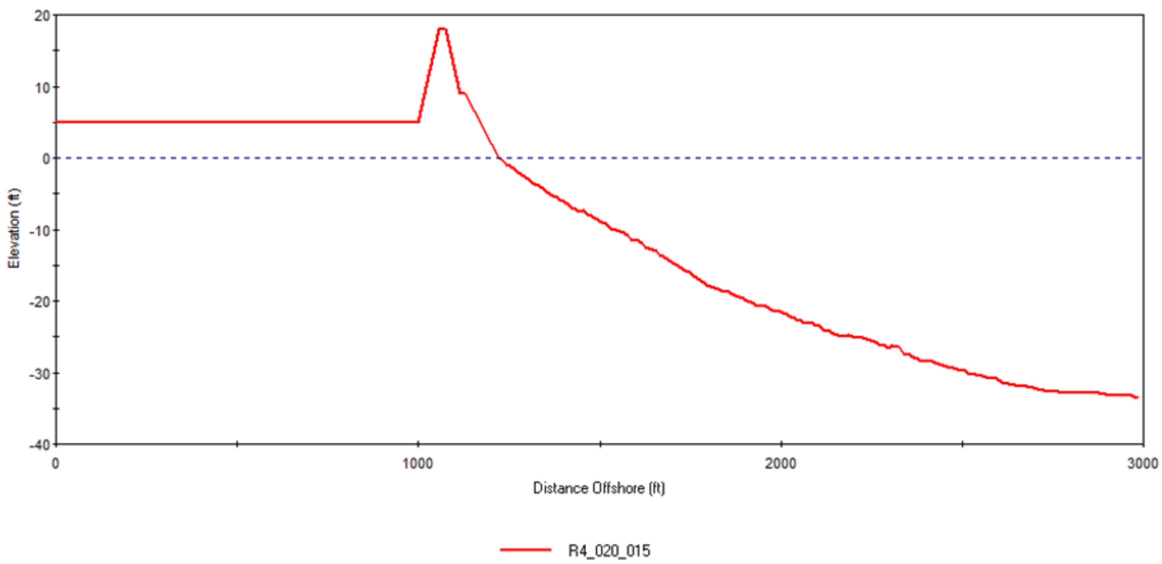

Figure 6-24. Idealized profile for transects 12 through 18, Profile 4.

Table 6-4. Idealized profile parameters for each representative profile (ft)

Input Parameter	Profile 1	Profile 2	Profile3	Profile 4
Dune height	11.5	15	14	18
Dune width	10	5	10	20
Berm elevation	9	9	9	9
Berm width	0	0	0	15

Dune slope	0.23	0.357	0.166	0.23
Foreshore Slope	0.104	0.104	0.104	0.104
Upland width	400	500	600	1000
Upland elevation	5	7	3	5

6.4.3 Beach-fx Reaches

The Pawcatuck study area was characterized within Beach-fx with 13 Beach-fx Reaches as illustrated in Figure 6-25. Reaches R1 through R7 and R13 are open coast Reaches for which storm-induced beach profile responses were computed using SBEACH. Reaches R8 through R12 are all back-bay Reaches which had static beach profiles but were subject to back-bay flooding based on water levels at the NACCS stations indicated in Figure 6-25.

The NACCS save stations used for back-bay flooding estimates are believed to be sufficiently reliable for the use in the Beach-fx portion of the study as explained in Section 4. As discussed later the elevations from within the pond from the NACCS data were used to design flood wall elevations and tide gates for protecting the back bay areas. These options were shown in the Beach-fx effort not to be cost effective and adjustments in water surface elevation would not change that outcome. Higher water levels would require more robust flood wall designs (higher cost) and tide gates (higher cost) negating the potential greater benefit pool.

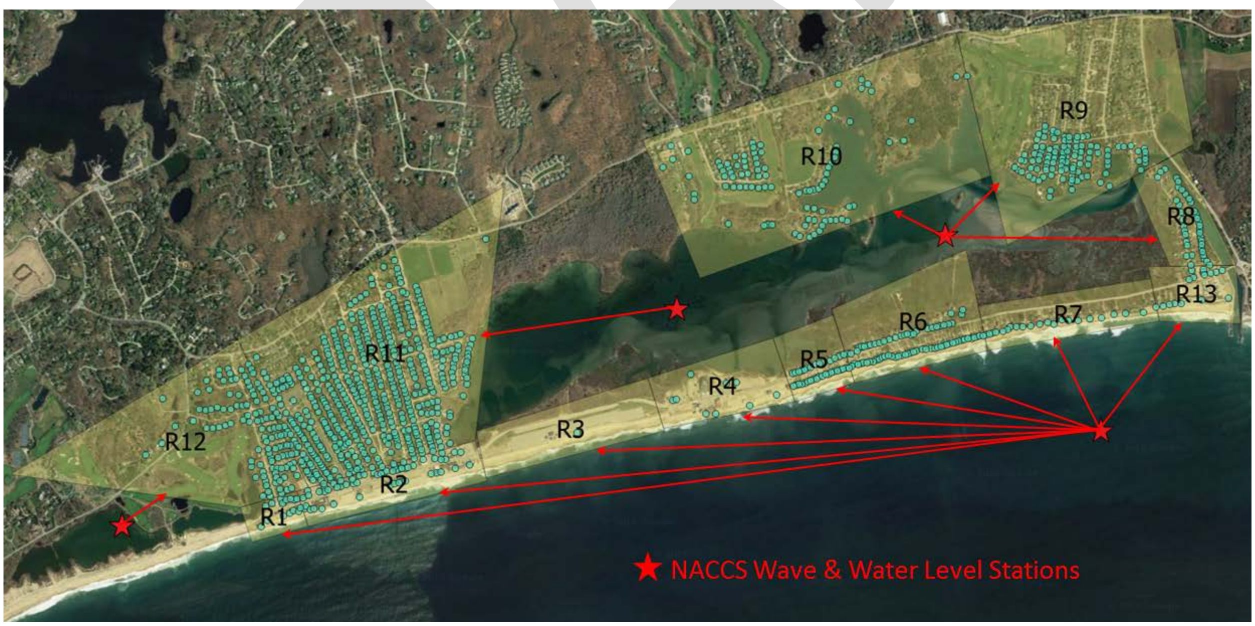

Figure 6-25. Beach-fx Reaches and NACCS environmental forcing stations.

6.5 Beach-fx Coastal Processes Input Data Development

6.5.1 Storm Season Specification

Beach-fx requires the specification of storm seasons and assignment of the expected average number of storms occurring in each season. For the Pawcatuck study the storm climatology includes both extratropical and tropical storms. Extratropical storms occur between October and March and a total of 72 extratropical storms of significance to the Pawcatuck study area were identified from a 75 year period yielding an expected annual extratropical storm occurrence rate of 0.96 storms per year on average. A uniform distribution of extratropical storm occurrence was assumed across the 6-month extratropical storm season. As such, the expected average number of extratropical storms was specified at 0.16 in each of the 6 months in which extratropical storms occur. Tropical storms occur between June and November and the NACCS statistical analysis reports an annual tropical storm occurrence rate of 0.1736 tropical storms per year (or approximately 1 storm every 5.75 years). Cialone et al. 2015 reports that the distribution of tropical storm occurrence across the tropical storm season (June through November) is as follows: June (0.04), July (0.04), August (0.26), September (0.48), October (0.12), and November (0.06). Consequently the expected average number of tropical storms was specified as follows: June (0.006944), July (0.006944), August (0.045136), September (0.083328), October (0.020832) and November (0.010416). Note that the sum of the monthly storm rates is equal to the expected annual storm rate. The months of April and May are specified as a “calm” season with no storms occurring in these months. During the months of October and November both extratropical and tropical storms can occur.

6.5.2 Relative Storm Probability Specification

Beach-fx allows specification of the relative probability of storms comprising the plausible storm suite. Plausible storms associated with the mean tidal range are double weighted as compared to plausible storms associated with the high or low tidal ranges. Likewise, because this study employed selected representative storms as discussed in section 6.3.3 each selected storm was weighted according to the cumulative weight of those storms it represents. For the extratropical storms the relative storm probability assigned to the selected representative storm was equal to the number of storms that selected storm represents. For example, there were 15 storms in the 5-year return period cluster (Table 6-3) and the selected storm representing this cluster was assigned a relative probability of 15. Likewise, there were 5 storms in the 20-year return period cluster and the selected storm for this cluster was assigned a relative probability of 5. Because the extratropical storms are historically based and were identified through a simple peak over threshold analysis technique all storms are equally weighted under the assumption that the historical record analyzed encompasses the full probability space. The NACCS tropical storms on the other hand are probabilistic and therefore are not equally weighted. Consequently, the relative probability assigned to the selected representative tropical storms was determined by summing the relative probabilities of all the storms that the selected storm represents. For example, the cumulative relative probability of the 35 storms comprising the 10-year return period cluster (Table 6-3) was 0.0104822 and the selected representative storm (storm 569) was initially assigned a relative probability of 0.0104822. Ultimately, the initially assigned relative probabilities of the selected storms were normalized by dividing all relative probabilities by the least relative probability (that associated with the 1000-year return period cluster). In this way the relative probability (or relative weight) of each of the selected storms was greater than or equal to one.

6.5.3 Storm-Induced Beach Profile Responses

The availability of a large database of beach profile responses to the each storm in the plausible storm suite is central to the operation of Beach-fx. Two kinds of data are stored in the SRD for each storm/profile simulation: changes in berm width, dune width, dune height and upland width, and cross-shore profiles of erosion, maximum wave height, and total water elevation. The morphology changes (berm width, dune width, dune height and upland width) are used to modify the pre-storm beach profile to obtain the post-storm profile. The damage driving parameters (cross-shore profile of erosion, maximum wave height, and total water elevation) are used to estimate damages to assets within reaches associated with that representative profile. The SRD is a pre-generated set of beach profile responses to storms comprising the plausible storm suite, for a range of beach profile configurations that are expected to exist for different sequences of storm events and management action scenarios. The numerical model for simulating storm-induced beach change (SBEACH), (Larson and Kraus, 1990) was used to estimate beach profile responses to each of the storms contained in the plausible storm suite. As discussed in section 6.3 the storm suite used to generate the SRD includes 13 synthetic tropical storm events and 6 extratropical storm events. When these hypothetical storms were combined with a statistical representation of astronomical tides the number of plausible storms was increased to 156 tropical and 72 extratropical events. A companion range of beach profile configurations were developed to encompass all expected beach configurations encountered under each of the evaluated without-project scenarios. Profiles were developed at 10 ft increments on berm width, 5 ft increments on dune width, and 1.5 ft increments on dune height between the most robust and most vulnerable beach profiles. This procedure generated a total of 1540 unique upper beach profile configurations across the 4 representative profiles. The response of each of these beach profiles to the entire storm suite consisting of 228 plausible storm events was simulated using the SBEACH model. A total of 351120 SBEACH simulations were performed and the results populated the SRD used as input to Beach-fx.

6.5.4 Historical Rate of Shoreline Change

The next step required to fully implement the project in Beach-*fx* is calibration. The goal of the calibration procedure is to cause Beach-*fx* to reproduce, on average over multiple lifecycle simulations, the historical shoreline rate of change. To do this one must first develop an estimate of the historical shoreline rate of change. For this region, estimates of annual shoreline erosion have been developed by both the USGS and the State of Rhode Island based on analyses of aerial photos. The New England District provided the modeling team with a target historical shoreline rate of change estimate of -1 ft/year based on the USGS and State analyses.

6.6 Beach-fx Calibration

The calibration procedure for Beach-*fx* involves specification and tuning of a reach-level attribute known as the “applied erosion rate”. The applied erosion rate accounts for long-term shoreline change not attributed to storm-induced shoreline changes which are captured within the model by the random sampling of storm events as the model progresses through the life cycle simulation. The concept employed here is that there are two essentially separable components of beach evolution, the first is cross-shore transport dominated shoreline change due to storm events which is mostly recoverable due to post-storm berm width recovery and the second is longshore transport dominated shoreline change that is driven by longshore sediment transport gradients, the underlying geological setting and other factors such as relative sea level change. This second component of beach evolution is considered nonrecoverable. The Beach-*fx* calibration concept is that the combination of these two drivers of beach evolution should, on average, over multiple simulated project life cycles return the long-term average rate of shoreline change. Because the Beach-*fx* simulated life cycle iteration employs a random sequence of storm events the returned shoreline change rate differs in each life cycle simulated. The Beach-*fx* calibration task is to determine an appropriate applied erosion rate for each reach such that the computed average rate of shoreline change on a reach-by-reach basis is equal to the estimated target historical shoreline change rate over multiple life cycle simulations.

For the Pawcatuck Beach project, Beach-fx was calibrated across 300 iterations of a 56-year life cycle using an assigned depth of closure specification of -27 ft NAVD88. The depth of closure estimate was developed based on an analysis of the available beach profile data. The 56-year life cycle duration stems from the use of the January 2015 beach profile survey to define the initial condition leading to a start year specification of 2015 and the specification of year 2020 as the base year for calculating the economics and an economic analysis horizon corresponding to a 50-year project life. After a number of calibration iterations Beach-*fx* was calibrated to precisely reproduce the target historical SRC on average over 300 56-year project life cycles.

The Beach-fx calibration procedure involves assigning a "background" shoreline change rate such that when combined with the storm-induced shoreline changes the model, on average over multiple lifecycles, returns the target historical rate of shoreline change. In the Pawcatuck study area the target historical shoreline change rate was given at -1 ft/year (Section 2.3). The storm-induced shoreline change derived from random sampling of the SBEACH-computed morphology responses generated shoreline change rates ranging from approximately -2.0 to -3.5 ft/year. In order to adjust the storm-induced only shoreline change, calibrated applied shoreline change rates for each reach were necessary. These calibration or adjustment values ranged between approximately +1.2 and +2.6 ft/year (Table 6-5). When these values were added to the storm-induced only shoreline change rates the model then returned a shoreline change rate of -1.0 ft/year (erosion) across the study area on average over multiple lifecycles.

Table 6-5. Calibrated applied erosion rates for each profile (and reach)

Profile	Reach	ft/yr
1	1	2.566
2	2	2.431
2	3	2.431
3	4	1.864
3	5	1.864
4	6	1.172

4	7	1.172
4	13	1.172

6.7 Future Without-Project Beach-fx Simulations

Future without-project simulations were performed to estimate expected future damages within the Pawcatuck study area in the absence of a Federal hurricane and storm damage reduction project. The analysis involved the simulation of 300 unique 56-year duration life cycles beginning in 2015 and concluding at the end of 2070. All economic calculations are expressed in 2020 dollars (base year 2020) using a discount rate of 3.125%. The estimated future without-project damages to structures and contents average $65.196 million across the 300 unique life cycles simulated. The distribution of without-project damages across the 300 life cycles is illustrated in Figure 6-26 along with a box-and-whisker. Here it is seen that although 42 percent of the simulated life cycles returned damages between $55 and $75 million, 10 life cycles returned damages less than $35 million and at the other end of the distribution 10 life cycles returned damages greater than $105 million. The box-and-whisker plot shows that none of the lifecycles returned damages that are statistical outliers, the upper quartile (highest 25%, represented by upper whisker) ranges from $77.762 to $114.264 million. The lower quartile (lowest 25%, represented by lower whisker) ranges from $27.172 to $51.811 million. Figure 6-27 provides box-and-whisker plots representing without-project damages to structures and contents for each of the 13 Pawcatuck Beach-fx reaches. As seen in Figure 6-27 the reaches in which the greatest damages are realized are Reaches 2 and 11 accounting for $12.5 and $14.7 million in damages, respectively. Damages in these two reaches alone make up approximately 42% of the damages across the entire Pawcatuck study area.

In addition to damages to structures and contents the future without-project condition is expected to incur damages and/or costs associated with armoring of properties fronting Block Island Sound. Armoring costs were estimated at $12.586 million. These without-project costs are incurred from the rebuilding of existing armoring that fails over the course of the life cycle and from the expected construction of new armoring at lots where erosion threatens the integrity of assets located in those lots.

In total, damages and costs for the future the without-project were estimated at $77.782 million on average over the 300 unique life cycles simulated. However, because the Beach-fx simulations began in 2015 not all of these damages and benefits are available for benefits. That is, damages and costs that occurred between 2015 and the end of 2019 are not available for benefits because those costs will be incurred in both with- and without-project conditions. Damages available for benefits were estimated by removing the pre-base year damages and cost (common to both the with- and without-project condition). Damages to structures and contents available for benefits were estimated at $45.299 million. Armoring costs available for benefits were estimated at $9.419 million. As a result, the average potential benefit pool for the Pawcatuck study area is estimated at $54.718 million.

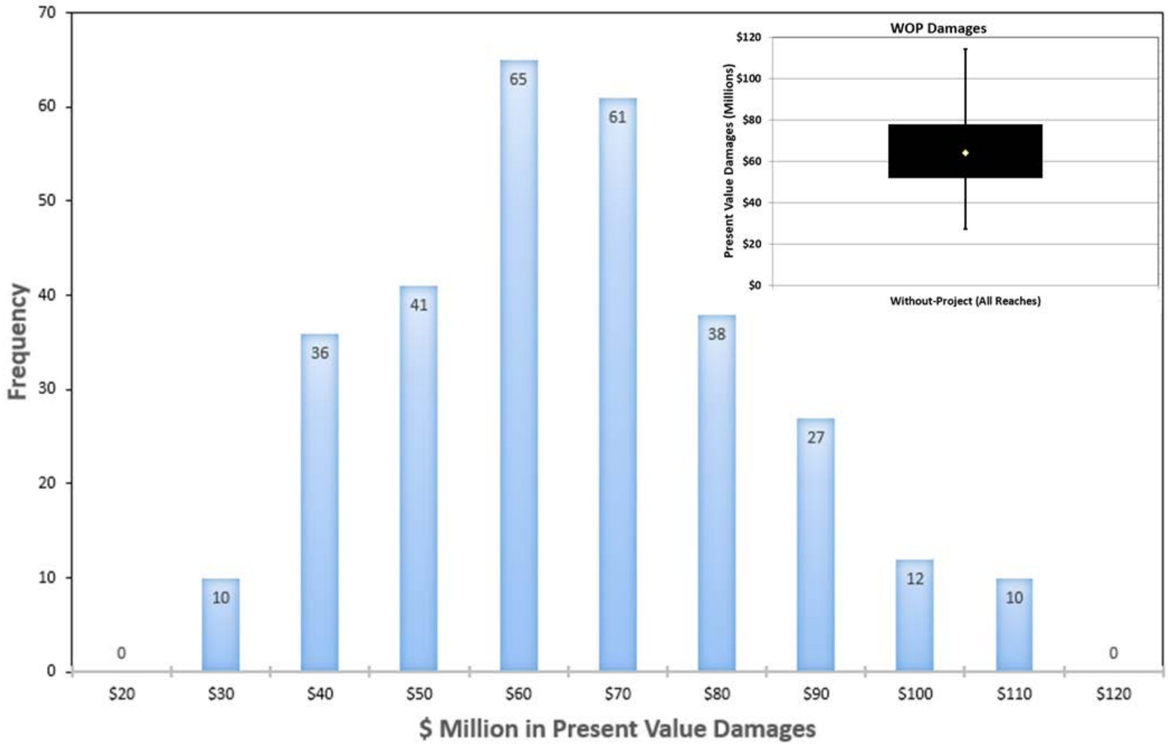

Figure 6-26. Distribution of future without-project damages.

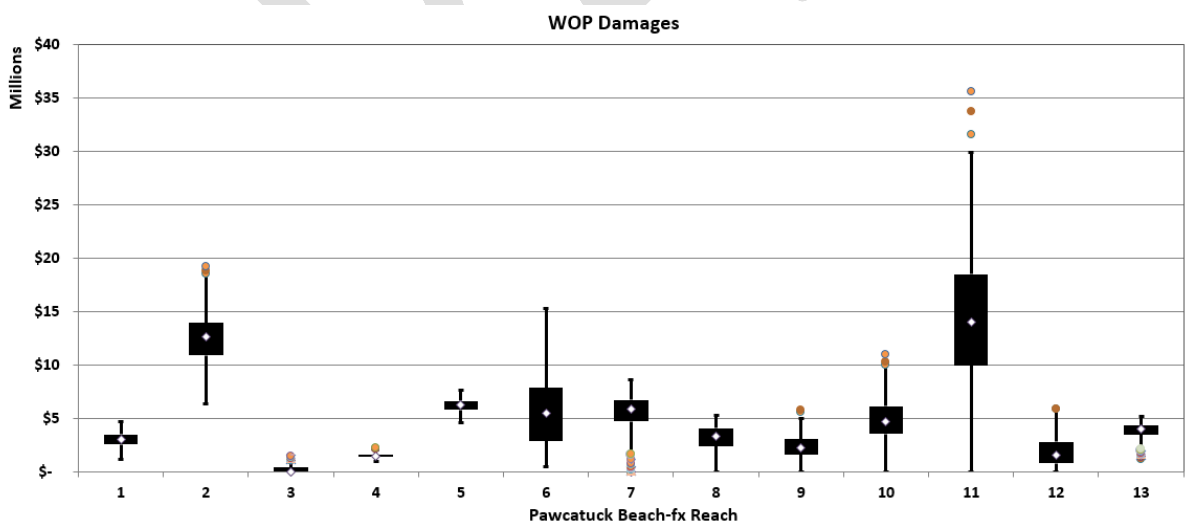

Figure 6-27. Without-Project present value damages by Beach-fx reach.

6.8 With-Project Beach-fx Simulations

The analysis of the with-project condition involved estimating the economic performance of 13 alternatives that employed 3 different protective measures including beach nourishment, flood walls (with closures), and a tide gate located in the Weekapaug Breachway (Winnapaug Pond inlet). There are a total of 8 alternatives involving the beach nourishment protective measure which included two spatial extents; a 4000 linear foot beach nourishment that covered Beach-fx Reaches R1 and R2 and a 9000 linear foot beach nourishment that covered Beach-fx Reaches R1, R2, R3, and R4. For both the 4000 and 9000 linear foot beach nourishment alternatives two different design cross sections were evaluated referred herein as the minimum and maximum design cross sections. The minimum design cross section involved a 12 ft NAVD design dune crest elevation with a 10 ft dune crest width and a 50 ft design berm width at elevation 9 ft NAVD. The maximum design cross section involved a 14 ft NAVD design dune crest elevation with a 10 ft dune crest width and a 100 ft design berm width at elevation 9 ft NAVD. The final variant in the beach nourishment alternatives relates to the source and cost of the beach nourishment fill material. Each alternative was evaluated assuming first an offshore source of material that would be hydraulically placed on the beach at a cost of approximately $23.50 per cubic yard, and re-evaluated assuming an upland source of material that would be truck hauled to the project at a cost of approximately $41.00 per cubic yard. Because a suitable offshore source of fill material had yet to be identified, the results involving the more costly truck haul scenarios were given more weight. Table 6-6 provides a summary overview of the beach nourishment alternatives evaluated.

Back-bay flooding issues were addressed by considering two flood walls and a tide gate located in Weekapaug Breachway and involved a western flood wall providing a measure of protection for Beach-fx Reach R11 and an eastern flood wall providing a measure of protection for Beach-fx Reach R12. These protective measures were evaluated though a series of 5 alternatives. The western and eastern flood walls were evaluated separately in two alternatives and together in a third alternative. The tide gate located in the Weekapaug Breachway was evaluated alone in one alternative and together with the western flood wall in another alternative. The crest elevation of all flood walls and the tide gate was set at the elevation of 10.5 ft NAVD which was provided by the New England District to the Coastal Hydraulics Lab. The 10.5 ft-NAVD elevation was provided as the wall elevation to protect against the 100-year flood. This elevation is higher than the elevations provided in Table 6-2 due to the source. Early in the study and the preliminary analysis, the elevations were pulled directly from the NACCS model data save points within the coastal ponds to start the design process for the flood wall features. As explained in Section 4, it is realized the NACCS save point data in the ponds is likely faulty for the aforementioned reasons. However, this was determined not to be of significance for the study since it was shown during the Beach-fx analysis (discussed in the following sections) that beach fill was not cost effective and therefore adding flood walls to protect against flanking was also not cost effective. If flood walls were constructed without a beach fill the back door would be closed and protected while leaving the front door open. If the beach fill and flood wall option had shown to be cost effective a more detailed look at back bay water elevations and wall height would have been completed. Table 6-7 provides a summary overview of the back-bay flooding alternatives and the Beach-fx reaches that potentially benefit from those alternatives.

6.8.1 Beach Nourishment Alternatives

4000 foot Beach Nourishment Alternatives: 4K_min, 4K_min TH, 4K_max, and 4K-max TH

The coverage of the 4000 linear foot beach nourishment alternative is illustrated in Figure 6-28. As seen these alternatives are intended to provide a measure of hurricane and storm damage reduction for

Beach-fx Reaches R1 and R2. The “minimum” design cross section as compared to the existing (2015) condition is modest in that the design dune crest elevation is just 0.5 ft higher than the Reach R1 dune crest and three feet lower than the Reach R2 dune crest elevation. The design dune width is approximately the same as the existing condition dune width. The design berm width however, is notably wider than the existing condition at 50 ft whereas the existing condition berm width is effectively non-existent. The “maximum” design cross section on the other hand is 2.5 ft higher than the Reach R1 existing condition profile and 1 ft lower than the Reach R2 existing condition profile. The design berm width is rather robust compared to the existing condition berm widths which are mostly non-existent. These design conditions will for the most part maintain the existing condition level of protection for the design life of the project through re-nourishments whereas in the without project scenario the beach and dune system is expected to continue to erode and degrade rendering upland infrastructure vulnerable to the destructive forces of coastal storms.

Table 6-6. Beach nourishment alternatives				
Alternative Name	Coverage (Reaches)	Dune Crest Elevation (ft, NAVD)	Berm Width (ft)	Fill Material Placement Method
4K_min	R1→R2	12	50	Hydraulic
4K_min TH	R1→R2	12	50	Truck Haul
4K_max	R1→R2	14	100	Hydraulic
4K_max TH	R1→R2	14	100	Truck Haul
9K_min	R1→R4	12	50	Hydraulic
9K_min TH	R1→R4	12	50	Truck Haul
9K_max	R1→R4	14	100	Hydraulic
9K_max TH	R1→R4	14	100	Truck Haul

Table 6-7. Back-bay flooding alternatives		
Alternative Name	Protective Measure	Protected Reaches
FW-West	Flood Wall	R12
FW-East	Flood Wall	R11
FW-West-East	Flood Wall	R11-R12
Tide Gate	Tide Gate	R8→R11
Tide Gate + FW-West	Tide Gate Flood Wall	R8→R12

Figure 6-28. 4000 linear foot beach nourishment alternative lay-out.

The Beach-fx simulations for the with-project alternatives were the same as those performed for the without-project condition and involved the simulation of 300 unique 56-year duration life cycles beginning in 2015 and concluding at the end of 2070.

Economic input related to the 4000 linear foot beach nourishment alternatives involving hydraulic placement of the fill material included mobilization costs of $2.974 million per nourishment operation and a unit placement cost of $23.50 per cubic yard of fill material. For truck haul placement of fill material the economic input included mobilization costs of $0.544 million per nourishment operation and a unit placement cost of $41.00 per cubic yard of fill material. Beach-fx results related to the costs of the 4000 linear foot beach nourishment are illustrated in the form of box-and-whisker plots in Figure 30. The average expected costs for the minimum design cross-section ranges from $10.662 million for hydraulic placement to $12.109 million for truck haul placement. For the maximum design cross section the average expected costs ranges from $19.667 million for hydraulic placement to $27.555 for truck haul placement. These planned nourishment project costs far exceed the average benefit pool (without-project damages to structures and contents plus armor costs available for benefits) for Reaches R1 ($2.132 million) and R2 ($6.884 million). Potential benefit cost ratios for the 4000 linear foot beach nourishment alternatives are illustrated in Figure 6-30 and show that for the “minimum” design cross section only about 25% of the life cycles simulated return a benefit to cost ratio that exceeds a value of one. For the “maximum” design cross section, none of the simulated life cycles returned a benefit to cost ratio exceed a value of one. The primary reason estimated potential benefit to costs ratios are for the most part less than one are the costs associated with the construction of beach nourishment within the Pawcatuck study area. Projects where benefit to costs ratios exceed a value of one usually involve fill material placement cost less than about $13.00 per cubic yard. The unit volume placement costs in the Pawcatuck project area exceed this high-end unit volume placement cost by between 45 and 62% for hydraulic and truck haul placement respectively. Consequently, the finding of an unfavorable benefit to cost ratio, for Federal participation, in a beach nourishment protective measure for the Pawcatuck study is not entirely unexpected.

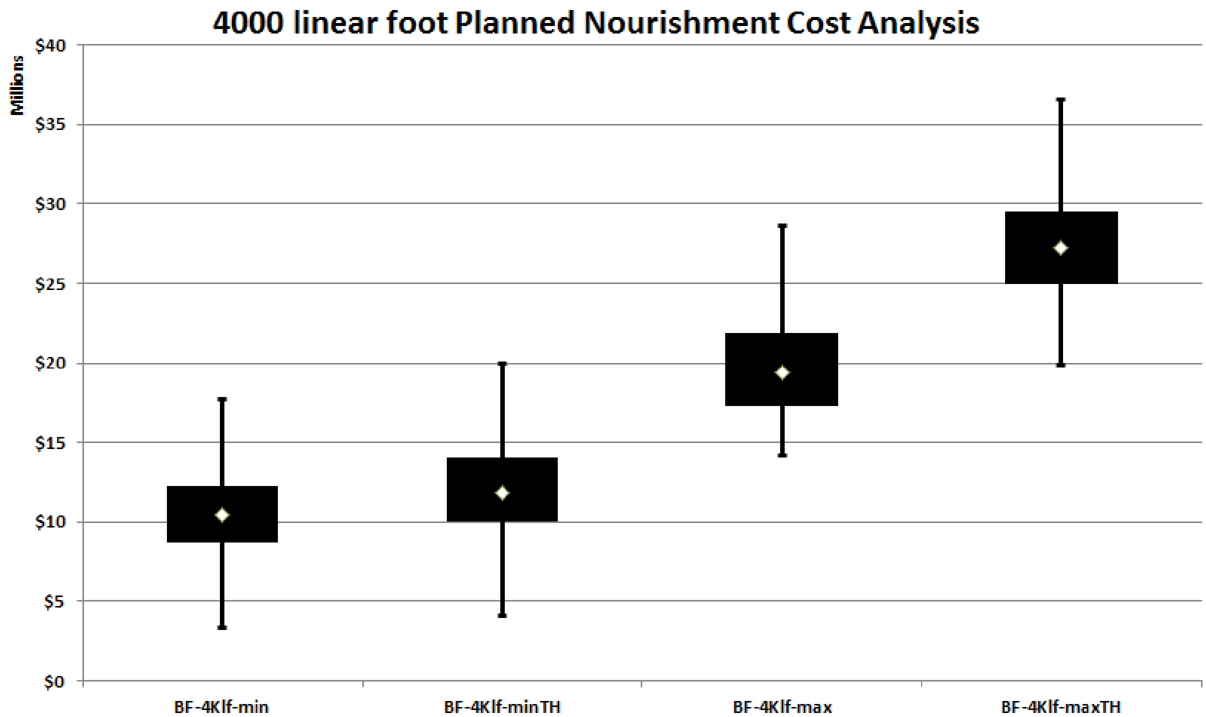

Figure 6-29. 4000 linear foot beach nourishment costs.

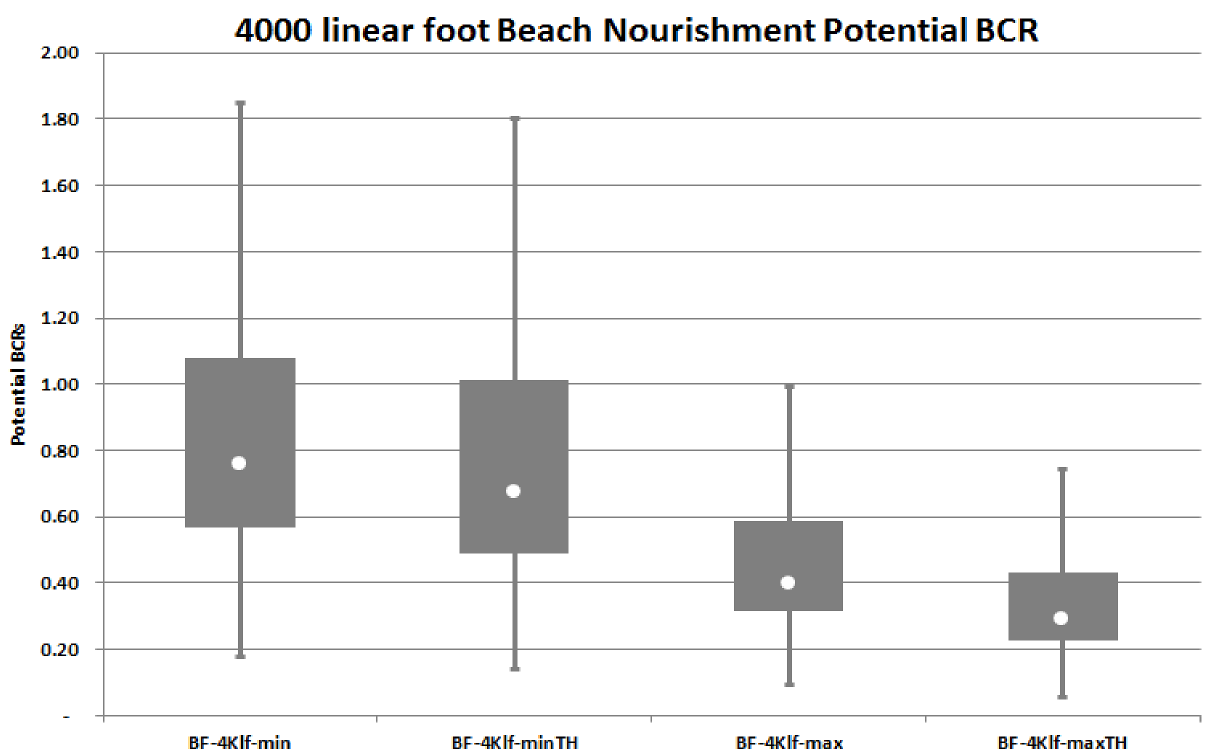

Figure 6-30. 4000 linear foot beach nourishment costs.

9000 foot Beach Nourishment Alternatives: 9K_min, 9K_min TH, 9K_max, and 9K-max TH

The coverage of the 9000 linear foot beach nourishment alternative is illustrated in Figure 6-31. As seen these alternatives are intended to provide a measure of hurricane and storm damage reduction for Beach-fx Reaches R1, R2, R3, and R4. These alternatives involved the same "minimum" and "maximum" design cross sections described above for the 4000 linear foot beach nourishment alternatives. The existing condition beach profile for Reach R3 is the same as that described previously for Reach R2. The existing condition beach profile for Reach R4 involves a 14 ft dune crest elevation with a 10 dune crest width and like the other reaches has no effective berm width. As such, the "minimum" design cross section is less robust compared to the existing condition profile (with the exception of the notable berm width in the design cross section) and the "maximum" design cross section is the equivalent to the existing condition except that the design cross section involves a 100-ft design berm width.

Figure 6-31. 9000 linear foot beach nourishment alternative lay-out.

Economic input related to the 9000 linear foot beach nourishment alternatives is the same as that described for the 4000 linear foot beach nourishment alternatives except in mobilization costs. In the 9000 linear foot beach nourishment alternatives the mobilization cost for hydraulic placement was $3.521 million per nourishment operation and for truck haul placement $0.544 million per nourishment operation. The unit placement cost was the same as for the 4000 linear foot beach nourishment alternatives. Beach-fx results related to the costs of the 9000 linear foot beach nourishment are illustrated in the form of box-and-whisker plots in Figure 6-32. The average expected costs for the minimum design cross-section ranges from $18.630 million for hydraulic placement to $23.641 million for truck haul placement. For the maximum design cross section the average expected costs ranges from $42.018 million for hydraulic placement to $68.831 for truck haul placement. These planned nourishment project costs far exceed the average benefit pool (without-project damages to structures and contents plus armor costs available for benefits) for Reaches R1 ($2.132 million), R2 ($6.884 million), R3 ($0.198 million) and R4 ($3.422 million). Potential benefit cost ratios for the 9000 linear foot beach nourishment alternatives are illustrated in Figure 6-33 and show that for the "minimum" design cross section less than 25% of the life cycles simulated return a benefit to cost ratio that exceeds a value of one. For the "maximum" design cross section, none of the simulated life cycles returned a benefit to cost ratio exceed a value of one. This finding is the same as for the 4000 linear foot beach nourishment alternatives for the same reasons. The conclusion of these analyses is that hurricane and storm damage protection for the Pawcatuck study area through implementation of beach nourishment protective measures is not expected to produce sufficient net benefits to justify Federal participation in such a project.

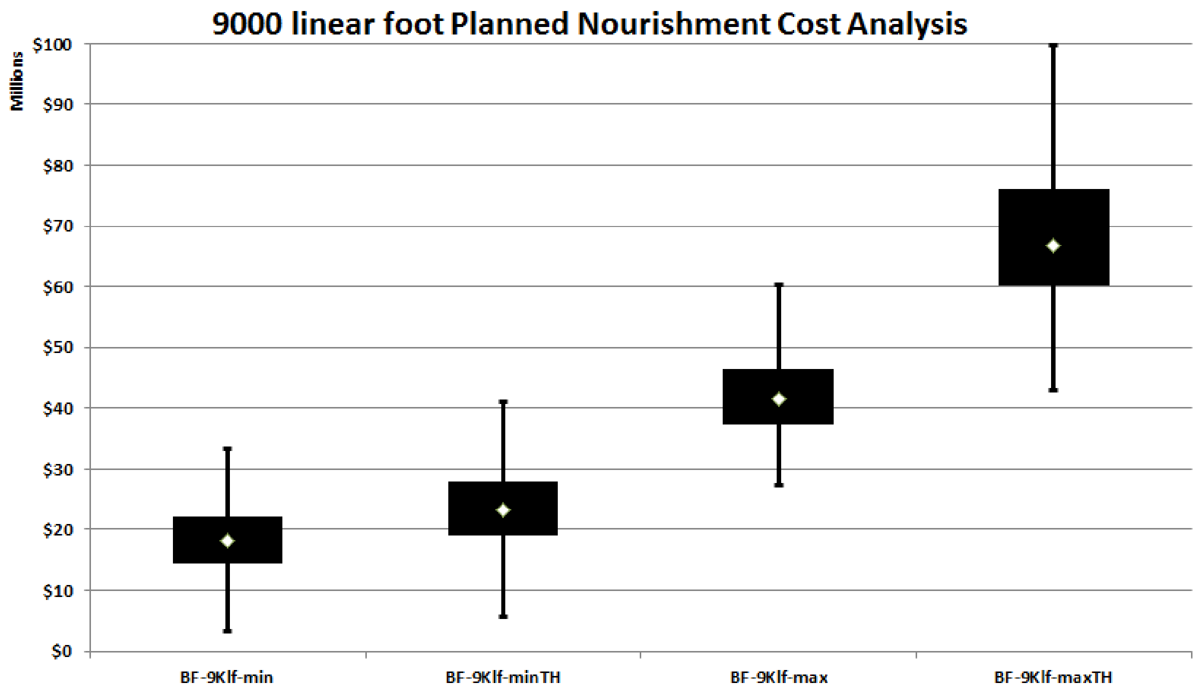

Figure 6-32. 9000 linear foot beach nourishment costs.

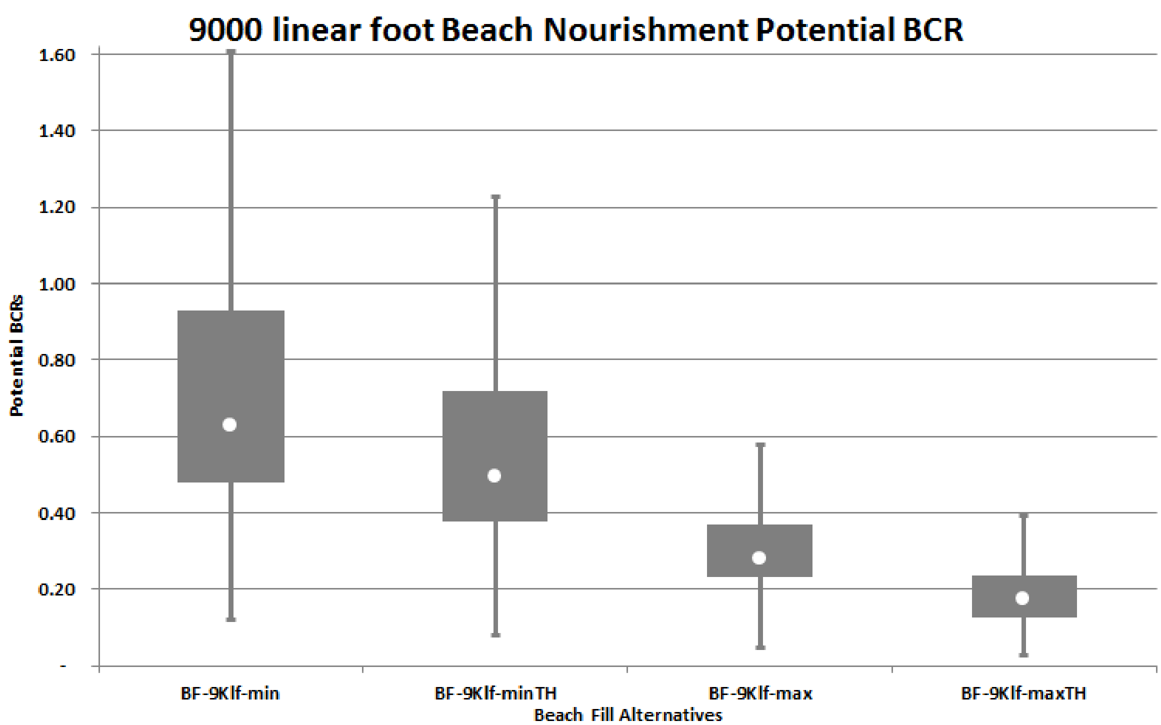

Figure 6-33. 9000 linear foot beach nourishment costs.

6.8.2 Back-bay Flooding Alternatives

East and West Flood Alternatives

The proposed flood wall protective measures are illustrated in Figure 6-34. These protective measures provide protection from back-bay flooding up to the flood wall crest elevation specified at 10.5 ft NAVD. Because Flood Wall West provides back-bay flood protection from Little Maschaug Pond which has no direct connection to Block Island Sound water levels in Little Maschaug Pond (from the NACCS station) must exceed 9.0 ft NAVD (the natural berm elevation) before assets located in Beach-fx Reach R12 begin to incur inundation damages. Consequently, the benefits generated by Flood Wall West are equivalent to the without-project damages associated with flood water levels in Little Maschaug Pond between 9.0 and 10.5 ft NAVD. Flood Wall East on the other hand, provides back-bay flood protection from Winnapaug Pond which has a direct connection to Block Island Sound water levels (through the Weekapaug Breachway) and therefore, Flood Wall East provides benefits equivalent to the without-project damages associated with all flood water levels in Winnapaug Pond up to 10.5 ft NAVD.

Figure 6-34. 9000 linear foot beach nourishment costs.

The Beach-fx results related to the Flood Wall West alternative are illustrated in the form of box-and-whisker plots in Figure 6-35. The average without-project damages for the influenced Beach-fx reach

(Reach R12) are estimated at $1.527 million whereas the estimate with-project (FW-West alternative) average damages are estimated at $0.533 million. The average FW-West benefits are estimated at $0.995 million. However, as indicated in Figure 6-35 the total costs associated with the implementation of Flood Wall West are estimated at $4.808 million which exceeds the benefits associated with this alternative. The results of this analysis indicated that the estimated benefits generated through implementation of the Flood Wall West protective measure are insufficient to justify Federal participation in this protective measure.

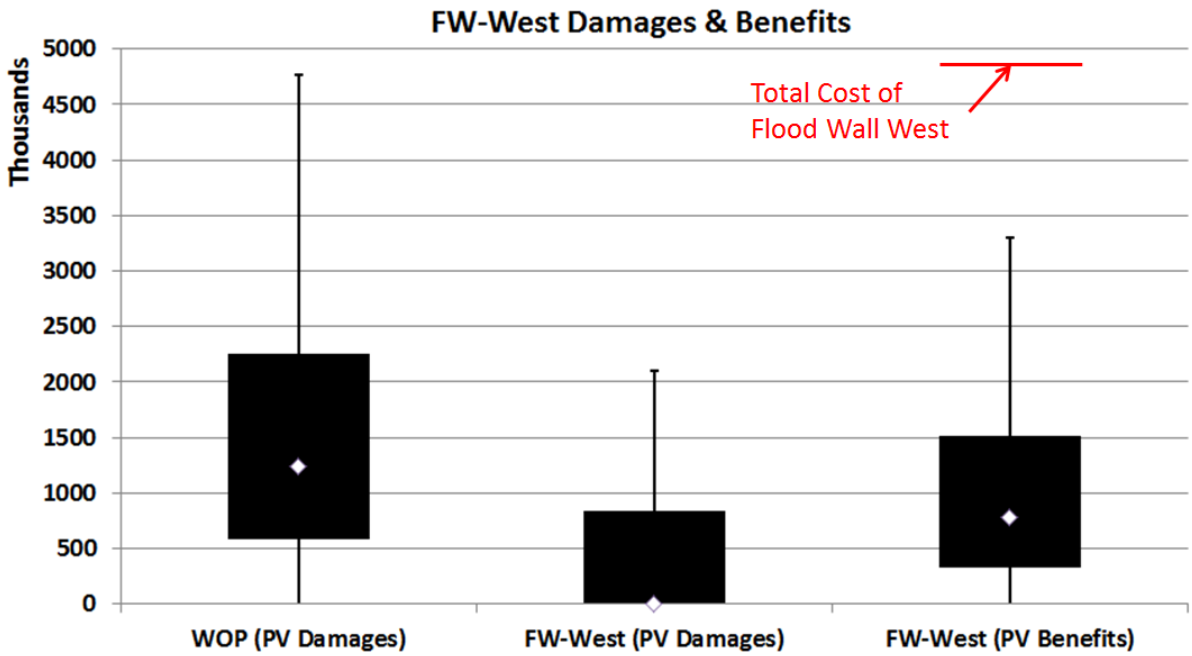

Figure 6-35. Flood Wall West, present value damages and benefits.

The Beach-fx results related to the Flood Wall East alternative are illustrated in the form of box-and-whisker plots in Figure 6-36. The average without-project damages for the influenced Beach-fx reach (Reach R11) are estimated at $11.895 million whereas the estimated with-project (FW-East alternative) average damages are estimated at $2.411 million. The average FW-East benefits are estimated at $9.484 million. As indicated in Figure 6-36 the total costs associated with the implementation of Flood Wall East are estimated at $9.311 million which is less than the average benefits (but greater than the median benefits of $8.853 million) associated with this alternative. The results of this analysis indicated that although the benefit to cost ratio (based on average values) is expected to be greater than 1.0, nearly half of the simulated life cycles indicate benefit to cost ratios less than 1.0. The estimated net benefits generated through implementation of the Flood Wall East protective measure is not expected to maximize net benefits and consequently is not expected to be the selected alternative or plan.

The Beach-fx results related to the Flood Wall East-West alternative are illustrated in the form of box-and-whisker plots in Figure 6-37. The average without-project damages for the influenced Beach-fx reaches (Reaches R11 and R12) are estimated at $13.422 million whereas the estimated with-project (FW East-West alternative) average damages are estimated at $2.944 million. The average FW East-

West benefits are estimated at $10.479 million. As shown in Figure 6-37 the total costs associated with the implementation of Flood Wall East-West are estimated at $14.651 million which, is less than both the average and median benefits associated with this alternative. Figure 6-37 shows that for more than 75% of the simulated life cycles the benefit to cost ratio will be less than 1.0. As such, the results of this analysis indicated that the estimated benefits generated through implementation of the Flood Wall East-West protective measure are insufficient to justify Federal participation in this protective measure.

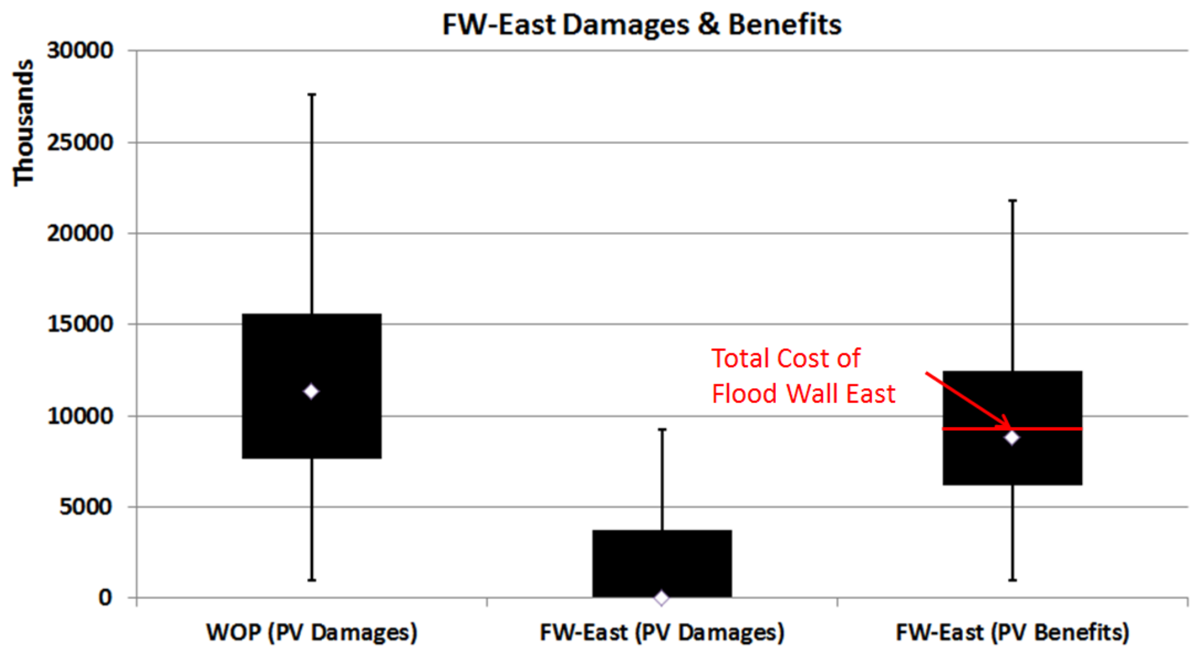

Figure 6-36. Flood Wall East, present value damages and benefits.

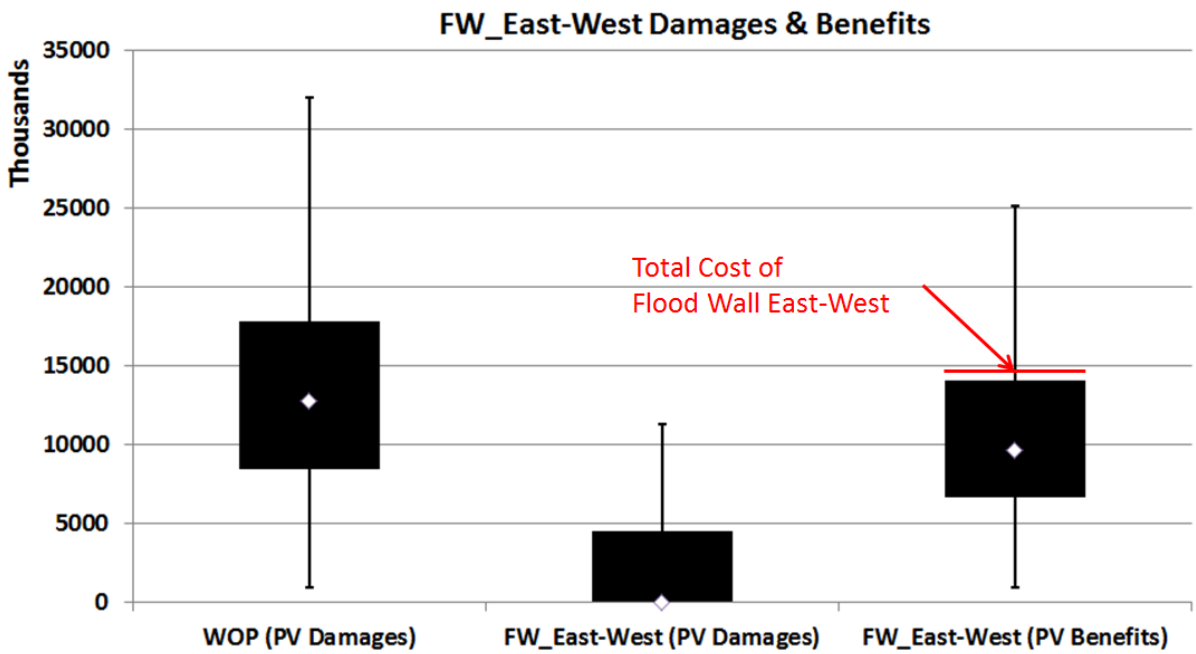

Figure 6-37. Flood Wall East-West, present value damages and benefits.

Tide Gate Alternatives

The proposed tide gate protective measure is illustrated in Figure 6-38. This protective measure provides protection from back-bay flooding up to the tide gate crest elevation specified at 10.5 ft NAVD. Alternatives involving the tide gate provide protection from back-bay flooding at Beach-fx Reaches R8, R9, R10, and R11; and R12 when combined with Flood Wall West.

The Beach-fx results related to the Tide Gate alternative are illustrated in the form of box-and-whisker plots in Figure 6-39. The average without-project damages for the influenced Beach-fx reaches (Reaches R8, R9, R10, and R11) are estimated at $20.384 million whereas the estimated with-project (Tide Gate alternative) average damages are estimated at $3.814 million. The average Tide Gate benefits are estimated at $16.570 million. However, as indicated in Figure 6-39 the total costs associated with the implementation of the Tide Gate are estimated at $14.143 million. Although the cost of implementing the Tide Gate alternative is less than both the mean and median benefits generated by this alternative approximately 45% of the simulated life cycles returned benefits less than the project implementation costs. The estimated net benefits generated through implementation of the Tide Gate protective measure is not expected to maximize net benefits and consequently is not expected to be the selected alternative or plan.

Figure 6-38. Tide Gate in Weekapaug Breachway.

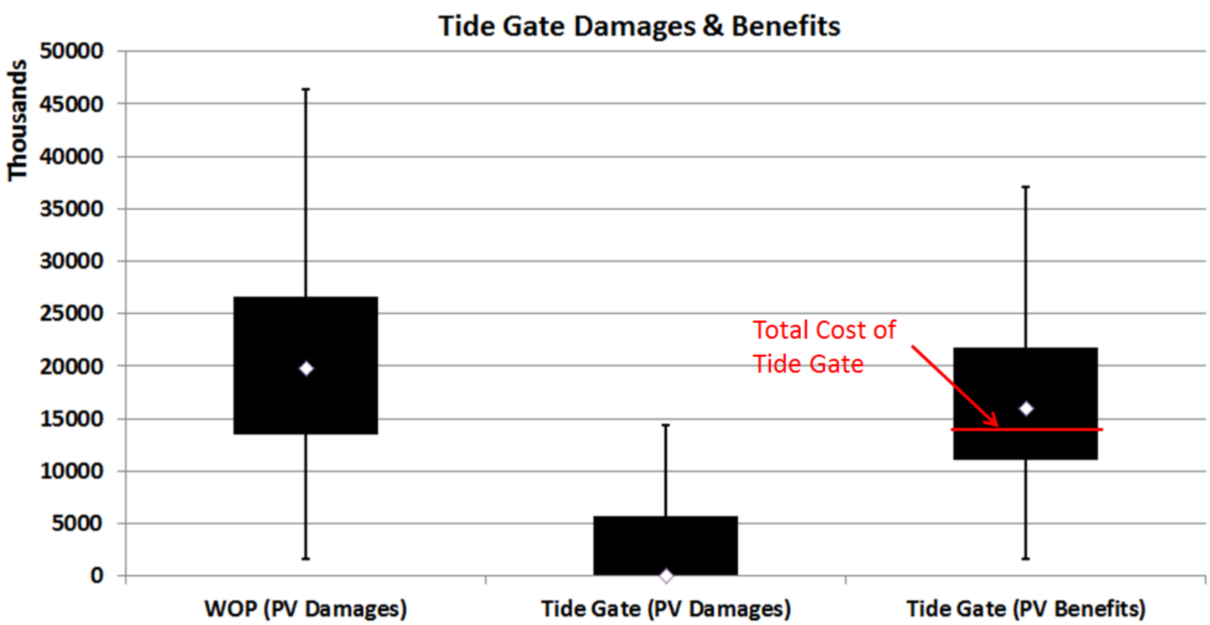

Figure 6-39. Tide Gate, present value damages and benefits.

The Beach-fx results related to the Tide Gate-FW-West alternative are illustrated in the form of box-and-whisker plots in Figure 6-40. The average without-project damages for the influenced Beach-fx reaches (Reaches R8, R9, R10, R11, and R12) are estimated at $21.911 million whereas the estimated with-project (Tide Gate-FW-West alternative) average damages are estimated at $4.347 million. The average Tide Gate-FW-West benefits are estimated at $17.564 million. However, as indicated in Figure 6-40 the total costs associated with the implementation of the Tide Gate-FW-West alternative are estimated at $19.596 million. The cost of implementing the Tide Gate alternative is greater than both the mean and median benefits generated by this alternative and approximately 60% of the simulated life cycles returned benefits less than the project implementation costs. The results of this analysis indicated that the estimated benefits generated through implementation of the Tide Gate-FW-West protective measure are insufficient to justify Federal participation in this protective measure.

6.8.3 Summary of With-Project Beach-fx Simulations

The engineering-economic model Beach-fx was employed to evaluate the economic performance of a total of 13 with-project alternatives. The costs associated with implementing the 8 alternatives involving the beach nourishment protective measure were shown to exceed the Without-project damages (potential benefit pool) due to the high unit placement cost of fill material (section 6.8.1). The analysis of the three alternatives involving the Flood Wall protective measure produced mixed results (section 6.8.2). The Flood Wall West alternative produced negative net benefits. The Flood Wall East alternative produced positive net benefits in about 48% of the simulated life cycles. The Flood Wall East-West alternative produced positive net benefits in just 20% of the simulated life cycles. The analysis of the two alternatives involving the Tide Gate protective measure also produced mixed results (section 6.8.2). The Tide Gate alone alternative produced positive net benefits in 61% of the simulated life cycles. The Tide Gate combined with Flood West alternative produced positive net benefits in 40% of the simulated life cycles. Of the alternatives evaluated using Beach-fx the Tide Gate alternative is expected to produce the greatest net benefits followed by Flood Wall East, Tide Gate and Flood Wall West, and finally Flood Wall East-West. Alternatives Flood Wall West and all the beach nourishment alternatives are expected to produce negative net benefits.

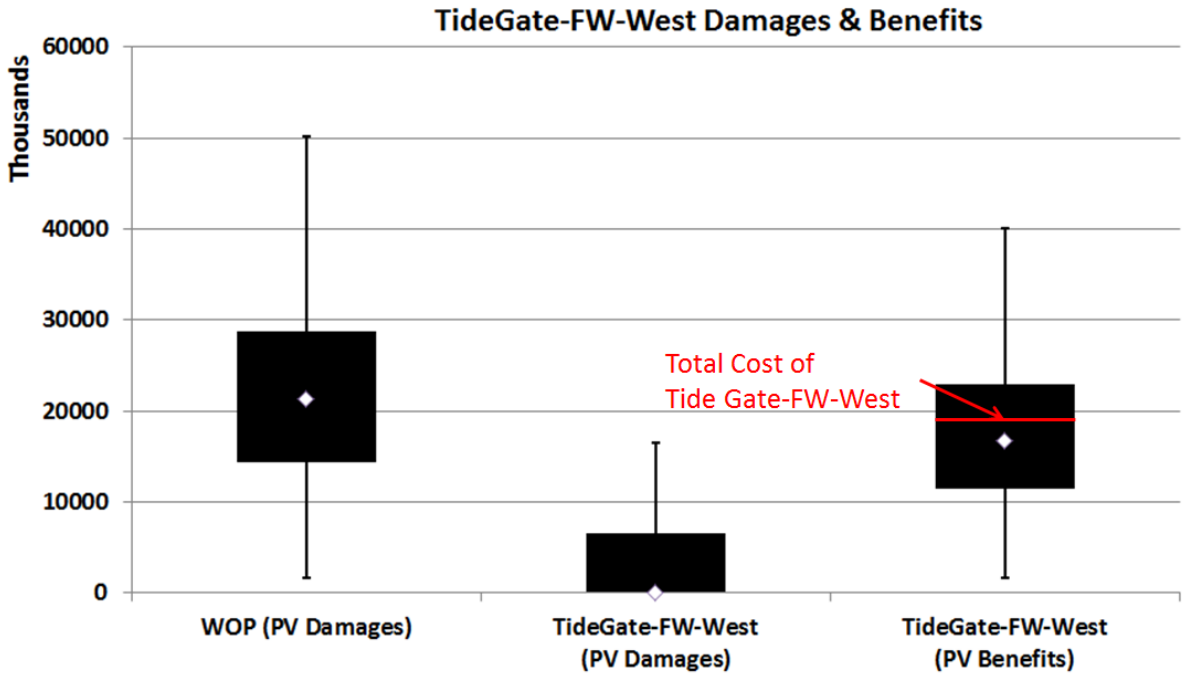

Figure 6-40. Tide Gate-FW-West, present value damages and benefits.

7.0 Summary and Conclusions

Both the Water Management Section of the New England District and the Coastal Hydraulics Lab of ERDC provided analysis and input for the study. The key product from the New England District was water levels that were ultimately used during the economics analysis for the Tentatively Selected Plan of structure elevation. The water levels provided were extracted from the FEMA FIS and FIRMs that were released in 2013. While this was not an ideal solution it was necessary to meet project schedule and to fill in for work that was anticipated to be completed by others.

The work completed by the CHL was for a detailed analysis of various beach fill alternatives for the Misquamicut Beach area as well as flood walls and a tide gate to protect the more dense housing and commercial area just to the west of the state beach. The effort utilized the Beach-fx model and through the use of that combined engineering and planning model it was shown that beach fill was not an economically justified alternative. Partially based on the Beach-fx analysis it was concluded that beach fill was not a viable alternative for any of the project areas.

8.0 References

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