Raritan Bay and Sandy Hook Bay
Highlands, New Jersey
Coastal Storm Risk Management
Feasibility Study

Feasibility Report

May 2020

Appendix B2:

Coastal Engineering

Raritan Bay and Sandy Hook Bay, Highlands, New Jersey

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1 Chapter 1: Existing Conditions

1.1 Tidal Datums

NOAA tidal station #8531680 is located in Sandy Hook Bay. Mean tide range for Sandy Hook is 4.7 ft. spring range is 5.2 feet. Mean sea level of the epoch between 1983 and 2001 is 2.6 ft. above MLLW. The NAVD88 (North American Vertical Datum of 1988) is located 0.24 ft. above mean sea level of the epoch between 1983 and 2001. NGVD29 (National Geodetic Vertical Datum of 1929) is located 0.86 feet below this MSL. Table B2- 1 contains the datum information. Figure B2- 1 shows the location.

Table B2- 1: NOAA Station 8531680 Tidal Datums (epoch 1983-2001)

Datums	Feet above NAVD88
MHHW	2.40
MHW	2.08
NAVD88	0.00
MSL	-0.24
NGVD29	-1.10
MLW	-2.62
MLLW	-2.82

1.2 Tidal Currents

Tidal currents along the shore of the study area are generally weak except at the entrances to Raritan and Shrewsbury Rivers, where the average velocity at strength of the current is 1.8 and 2.6 knots, respectively. A large part of the tidal circulation in the bay occurs in relatively deep water along an east-west axis approximately 2 miles off shore from the study area.

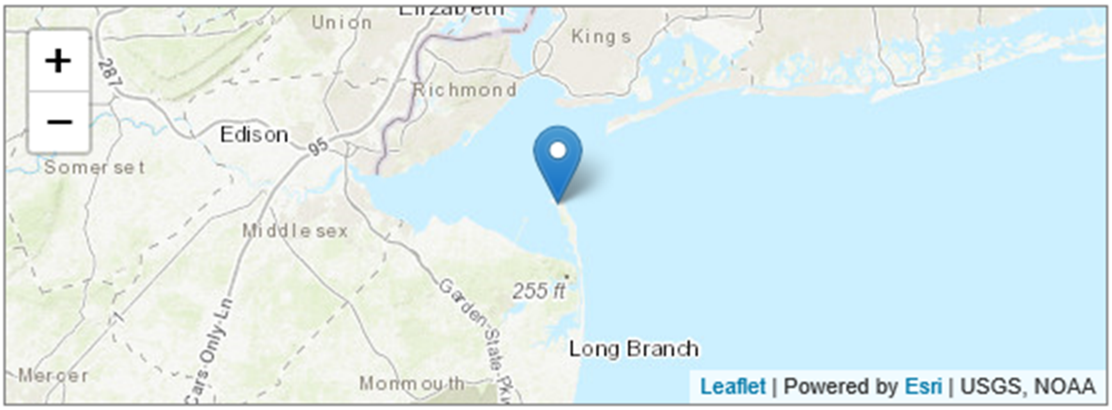

Figure B2- 1: Location of NOAA Tidal Station 8531680

1.3 Offshore Bathymetric Features

1.3.1 Offshore Channels

The dominant bathymetric feature offshore of Highlands is the presence of two channels, one of which is the natural Shrewsbury River Channel which runs immediately adjacent to the shoreline, and the secondary channel branches off from the river channel and heads north. The dominant river channel is approximately 1000 ft. wide, and has an approximate bottom elevation of – 21 ft. NAVD88. Along many of the profiles, this channel's side slope begins immediately at the toe of the shoreline bulkhead. The secondary channel branches off this primary channel in the approximately location of the central municipal bulkhead region. This secondary channel heads perpendicularly away from the shoreline (roughly north), then reaches the west shoreline of Sandy Hook and veers northwest, and has an approximate bottom elevation of –16 ft. NAVD88.

1.3.2 Offshore Contours

West of Highlands, the offshore bathymetric contours run parallel to the shoreline, running roughly east-west. However, due to the impact of the landmass of Sandy Hook peninsula and the effect of offshore channels, the offshore contours turn sharply to northeast-southwest at the west terminus of Highlands.

1.3.3 Bathymetry Effects on Wave Development

The effect of these channel features on wave development in Highlands is significant due to the channel bringing relatively deep water close to the shoreline, and in addition minimizing the effects of refraction and shoaling close to the shoreline. Wave refraction and shoaling effects are therefore assumed to be negligible. Near shore waves at bulkhead are predominantly non-breaking except those at pocket beaches.

1.4 Reach Delineation

Due to the geographic orientation of the study area, the Highlands shoreline is sheltered from ocean-generated waves coming into the bay through the opening between Sandy Hook Point and Rockaway Point. The western 75% of the Highlands shoreline faces Sandy Hook Bay, and is exposed to locally generated wind waves. The eastern 25% of the shoreline faces Shrewsbury River, and is sheltered from locally wind-generated waves. The shoreline was divided into four reaches, based on shoreline characteristics and orientation; three bay fronting reaches, and one river fronting reach. The engineering reaches are shown on Figure B2- 2.

1.5 Climate

The climate of the Raritan Bay and Sandy Hook Bay vicinity is temperate with an average annual temperature of 52 degrees Fahrenheit. The extreme temperatures observed were 31 degrees below zero and 110 degrees above. The average growing season is about 180 days and the relative humidity averages approximately 70 percent. The average annual precipitation is approximately 44 inches; the observed annual extreme values at an individual station were 61.70 and 29.94 inches at Sandy Hook, N.J. in 1878 and 1955, respectively. The distribution of precipitation throughout the year is rather uniform, with a slightly higher amount during the summer months. Maximum precipitation during a hurricane was recorded as 24 inches at Ewan, N.J. for the storm of 1 September 1940 which passed far off the coast.

The study area is subject to damage from hurricanes and extratropical cyclones, which are also known as northeasters.

Figure B2- 2: Location of Engineering Reaches

Hurricanes affects the project area most severely with high winds, waves, rainfall, and tidal flooding. A hurricane is defined as a cyclonic storm with winds in excess of 75 mph and a central barometric pressure of 29.0 inches or less, which originates in the tropical or subtropical latitudes of the Atlantic Ocean and moves erratically in a curved path, changing from an initial northwest to a final northeast direction. Hurricanes may affect localities along the entire Atlantic Coast of the United States. The hurricanes that most severely affect the study area usually approach from the south-southwest direction after recurving around eastern Florida and skirting the Middle Atlantic States.

The hurricanes originate principally during the months of August, September, and October. In the northern hemisphere, the revolving winds blow in a counter-clockwise direction about an eye or calm center. The diameters of the storms vary from 50 to over 500 miles, the velocity of the circular air movement being greatest near the center of the calm, and decreasing the relatively lighter winds at the outer periphery. The storm translates forward at a moderate speed typically 25 to 30 mph when approaching the study area, but at times reaching 60 mph. In most cases, tropical storms have moderated considerably from their peak intensity before reaching the study area. However, a number of notable exceptions have occurred, and hurricanes of devastating intensity have struck the area. The most severe hurricane on record for the study area is Hurricane Donna, which occurred on 12 September 1960.

In a northeaster, wind speeds are generally not as great and the central pressure is not as low as they are in a severe hurricane. The wind field of a northeaster is less symmetrical than that of a hurricane and covers a much greater area, and the forward motion of the storm is more likely to slow down. Thus, it may produce periods of onshore winds resulting in longer periods of flooding. Northeasters typically occur in the fall-winter season. The most severe northeasters on record for the study area occurred on 25 November 1950, 6-7 November 1953, 6-8March 1962, and 11-13 December 1992.

A summary of storms that occurred near the project area can be found in Engineering Sub-Appendix B4.

1.6 Water Surface Elevations

1.6.1 Water Surface Elevations for Design of Alternatives 1 through 5 and 5a through 5e

The U.S. Army Corps of Engineers has conducted an evaluation of storm-induced water levels using the state of the art ADCIRC model for the Fire Island to Montauk Point (FIMP) Reformulation Study in 2005. The 95% confidence interval predictions matched the 75 years of rank-ordered historic data the most closely. Therefore, this 95% confidence interval, combined tropical and extra tropical stage frequency was used for this Highlands study. The resulting water surface elevations for Highlands are shown in Table B2- 2, below. Wave setup was negligible due to the waves arriving in an unbroken state. See the Waves Section.

Table B2- 2: Water Surface Elevation in ft. NAVD88 from 2005 Model (Assumes Negligible WAVE SETUP)

Annual Chance of Exceedance	Water Surface Elevation in ft. NAVD88
50%	3.7
20%	5.4
10%	6.5
4%	7.5
2%	8.1
1%	8.8
0.5%	9.5
0.2%	10.1

1.6.2 Water Surface Elevations for Optimization of the Tentatively Selected Plan (5e Small, Medium, and Large)

The USACE North Atlantic Coast Comprehensive Study (NACCS) sought to quantify existing and future forcing for use in assessing potential engineering projects that would reduce flooding risk and increase resiliency. In the NACCS, rigorous regional statistical analyses and detailed high-fidelity numerical hydrodynamic modeling were conducted for the northeast Atlantic coastal region from Virginia to Maine in order to quantify coastal storm wave, wind and water level extremal statistics.

One node was selected to represent the entire study area for the optimization of the Tentatively Selected Plan (5e Small, Medium, and Large). The location of the selected node (Node 3555) is shown in Figure B2- 3. The expected values (50% confidence interval) were used for this analysis. The midpoint of the NOAA Tidal Epoch, 1992 water surface elevations are shown in Table B2- 3. For the optimization of the Tentatively Selected Plan (5e Small, Medium, and Large) the 1992 values shall be projected to 2026 and 2076 conditions using historic sea level change. 2026 will represent the without-project conditions, 2076 the future with and without-project conditions.

The near shore wave model Steady State spectral WAVE (STWAVE) was applied for the NACCS. During a two-way coupling process, a single instance of ADCIRC passes water elevations and wind fields to multiple instances of STWAVE. Upon completion, STWAVE passes wave radiation stress gradients to ADCIRC to drive wave-induced water level changes (e.g., wave set-up and set down). The resulting NACCS water surface elevation-frequency results include wave setup.

Figure B2- 3: Location of NACCS Node 3555

Table B2- 3: Water Surface Elevation in ft. NAVD88 INCLUDING WAVE SETUP for NACCS Node 3555

Annual Chance of Exceedance	Surface Elevation in m MSL (of 1983 to 2001 epoch)	Water Surface Elevation in ft. MSL (of 1983 to 2001 epoch)	Water Surface Elevation in ft. NAVD88 (of 1983 to 2001 epoch)
100%	1.5	5.0	4.7
50%	1.8	5.8	5.6
20%	2.1	7.0	6.7
10%	2.4	7.8	7.5
5%	2.6	8.6	8.4
2%	3.0	9.9	9.7
1%	3.4	11.3	11.0
0.50%	3.9	12.8	12.6
0.20%	4.5	14.8	14.6

1.6.3 Comparison of the Water Surface Elevations used for Alternatives 1 through 5 and 5a through 5e and those used for Optimization of the Tentatively Selected Plan (5e Small, Medium, and Large)

The NACCS model was coupled with STWAVE, and the resulting wave setup is **included** in the water surface elevation results, whereas the 2005 model discussed above had no wave setup component (the linear wave theory wave analysis shown below resulted in nonbreaking waves which create no setup). The NACCS model updated the wave climate using state-of-the-art wave models, whereas linear wave theory is a gross simplification of a very complex process. It is likely that the NACCS water surface elevations which include wave setup are more accurate. Nonetheless, the evaluation of Alternatives 1 through 5 and 5a through 5e occurred prior to the availability of the NACCS results. The NACCS expected value (50% Confidence) Water Surface Elevations are used. The comparison of the two simulated water surface elevations is shown in Table B2- 4.

Table B2- 4: Comparison of Water Surface Elevations from Different Models

Annual Chance of Exceedence	Water Surface Elevation from the FIMP (2005) model at the midpoint of the NOAA Epoch in ft. NAVD88	Water Surface Elevation from the NACCS model at the midpoint of the NOAA Epoch in ft. NAVD88
50%	3.7	5.6
20%	5.4	6.7
10%	6.5	7.5
4%	7.5	8.4
2%	8.1	9.7
1%	8.8	11.0
0.50%	9.5	12.6
0.20%	10.1	14.6

At this point in time and due to study funding and scheduling constraints, it was infeasible to tease out how much of the change was due to wave setup and how much was model improvement. Nonetheless, it was decided to stick with the 2005 model results guiding the analysis of Alternatives 1 through 5, and 5a through 5e, and then using the NACCS data for optimization (5e Small, Medium, and Large). It is unlikely that using the NACCS data in the analysis of Alternatives 1 through 5, and 5a through 5e would have resulted in a different Tentatively Selected Plan (5e).

The NACCS water surface elevations are modestly higher than those from the 2005 modeling. One ft. higher at the 10% Annual Chance of Exceedance, and 2.2 ft. at the 1%. Or in other words, the water surface elevation of 6.5 ft. NAVD88 expected to have an Annual Chance of Exceedance of 10% as per the 2005 modeling, now has a slightly more than 20% Annual Chance of Exceedance. At the more extreme water surface elevation of 10.1 ft. NAVD88 which had an Annual Chance of Exceedance of 0.2% now has slightly less than 2% annual chance of occurrence with the newer modeling. To use more dramatic terms, a water surface that used to register as a 500-year average return interval with new modeling now has an approximate 50-year return interval. Both models used the very near Sandy Hook NOAA Station 8531680 to calibrate/verify. Datums were converted to ft. NAVD88 in the 1983 to 2001 epoch for both model results. One factor that the chance may be attributed to is the difference in wave setup for both models. This is discussed in the sections below. But for now, the linear wave theory estimated non-breaking waves for

Highlands which wave setup is negligible; versus the tightly coupled ADCIRC and STWAVE of the NACCS model.

1.7 Wave Heights and Periods

1.7.1 Waves at Shoreline for Alternatives 1 through 5 and 5a through 5e.

The 1984 SPM fetch-generated wind wave model was utilized to develop wave heights at average fetch depths for each reach. Then linear wave transformation was used to estimate the wave heights at the shoreline. Several historic wave and wind roses in the area were available, and show predominant wind directions from NW clockwise to NE. Directional wind speed data is available from NOAA National Data Buoy Center Station ALSN6, Ambrose Light for 1983 to 1994, and from Wave Information Study database (WIS 1993), with locations shown in Figure B2- 4. Ambrose winds are averaged over an 8-minute period, whereas WIS winds are averaged over an hour. Ambrose wind speed data (measured 10 m above ground) was chosen. These 8-minute wind speeds were converted to mean hourly wind speeds to use as input to the wave-forecasting model.

The resulting offshore wave-frequency relationships for Reaches 1, 2, 3, and 4 (for waves located at the average fetch depth) are shown in Table B2- 5. These results confirm the assumption of increasing wave sheltering towards the east, varying (for a 2% Annual Chance of Exceedance) from significant wave heights of 6.8 ft. in Reach 1, to 4.1 ft. in Reach 2, to 3.0 ft. in Reach 3, to 1.5 ft. in Reach 4. These significant wave heights at the average fetch depths were reverse transformed hypothetically to deep water using linear wave theory. Table B2- 5 also shows the resulting deep-water wave heights for each of the reaches.

The wave heights at the average fetch depths were transformed to the shoreline using linear wave theory and were compared to depth limited wave heights estimated at the shoreline. The resulting significant (Hs) and 5% Exceedance wave heights (H5%) and the depth-limited wave heights at the shoreline for the seaward line of protection are shown in**Error! Reference source not found.**

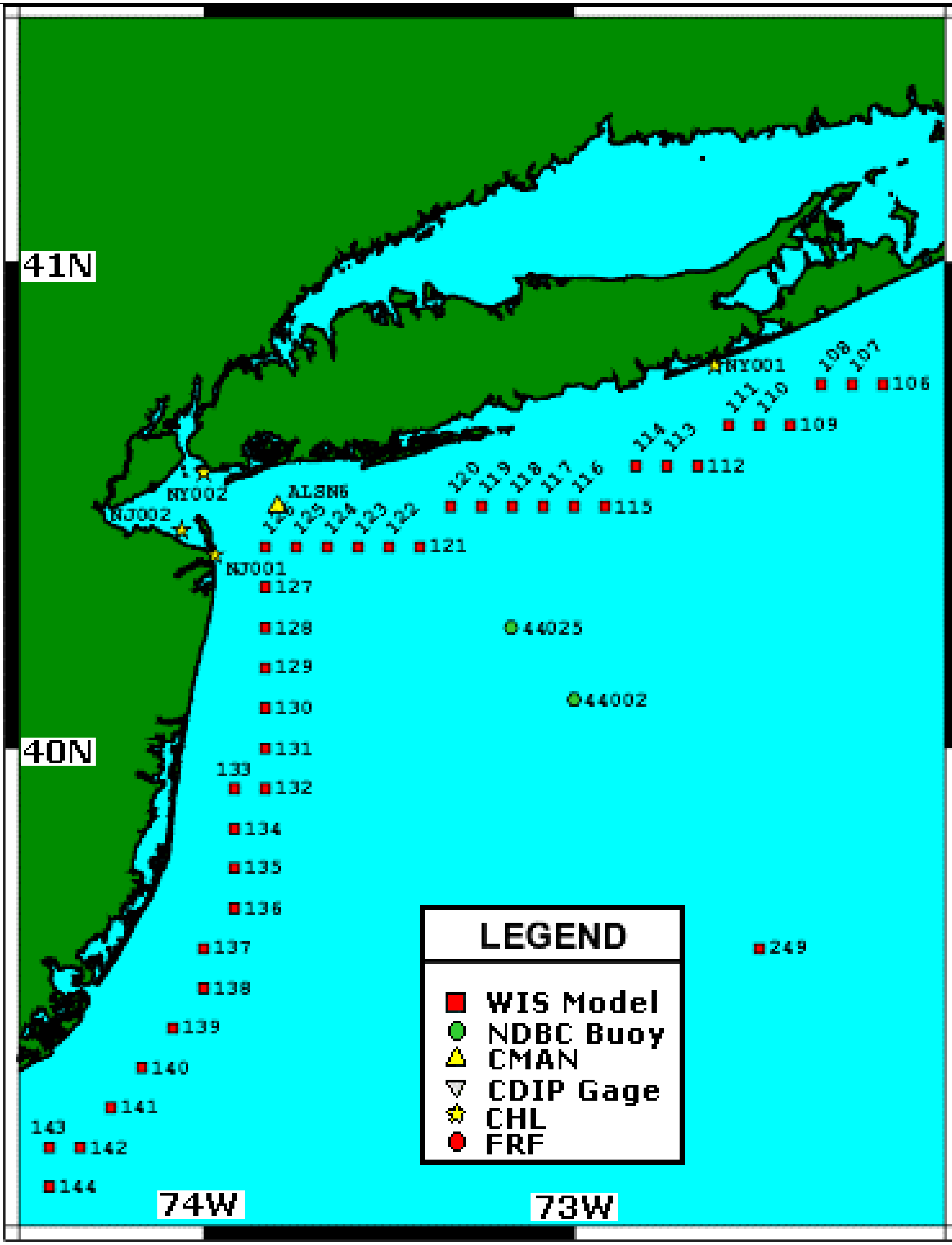

Figure B2- 4: Location of Station ALSN6 and WIS Model Nodes

Table B2- 5: Offshore Wave Heights for Alternatives 1 through 5 and 5a through 5e

Reach	Annual Chance of Exceedence in %	Water Surface Elevation from 2005 model in ft. NAVD88	Significant Wave Height, Hs, at the Average Fetch depth in ft.	Significant Wave Period, Ts, in seconds	Deep Water Wave Height, H'o, in ft.
Reach 1	20%	5.4	5.2	4	5.4
	10%	6.5	5.6	4.1	5.9
	4%	7.5	6.3	4.3	6.6
	2%	8.1	6.8	4.4	7.2
	1%	8.8	7.3	4.5	7.7
	0.5%	9.5	7.9	4.7	8.4
Reach 2	20%	5.4	3.1	2.8	3.1
	10%	6.5	3.3	2.8	3.3
	4%	7.5	3.7	3	3.7
	2%	8.1	4.1	3	4.1
	1%	8.8	4.4	3.1	4.4
	0.5%	9.5	4.8	3.2	4.9
Reach 3	20%	5.4	2.2	2.3	2.2
	10%	6.5	2.5	2.3	2.5
	4%	7.5	2.8	2.4	2.8
	2%	8.1	3	2.5	3.0
	1%	8.8	3.3	2.6	3.4
	0.5%	9.5	3.6	2.7	3.7
Reach 4	20%	5.4	1.1	1.4	1.1
	10%	6.5	1.2	1.5	1.2
	4%	7.5	1.4	1.5	1.4
	2%	8.1	1.5	1.6	1.5
	1%	8.8	1.7	1.6	1.7
	0.5%	9.5	1.8	1.7	1.8

As per EM 1110-2-1614, the minimum of the 5% Exceedance waves and the depth-limited waves are selected as the design wave conditions. In all cases, the 5% waves are the controlling wave conditions, and the waves arrive at the wall in a non-breaking condition. The resulting design wave conditions are shown in **Error! Reference source not found.** and was used for wall design and without-project conditions at the shoreline location.

Table B2- 6: Design Wave Heights at the Structure Toe for Alternatives 1 through 5 and 5a through 5e

Reach	Annual Chance of Exceedence in %	Water Surface Elevation from 2005 model in ft. NAVD88	Significant Wave Period, Ts, in seconds	Estimated Water depth, ds, at Structure Toe	Average Height of Waves where only 5% of the wave spectrum heights exceed this height, H5%, at the Structure Toe in ft.	Depth-Limited Breaking Wave Height, Hb, in ft.	Design Wave Height,Hd, at the Structure Toe in ft.
Reach 1	5	6.5	4	9.1	7.0	8.6	7.0
	10	7.6	4.1	10	7.6	9.3	7.6
	25	8.6	4.3	11.2	8.6	10.2	8.6
	50	9.2	4.4	11.9	9.2	10.8	9.2
	100	9.9	4.5	12.5	10.0	11.3	10.0
	200	10.6	4.7	13.1	10.9	12.2	10.9
Reach 2	5	6.5	2.8	9.1	4.0	8.2	4.0
	10	7.6	2.8	10	4.3	9.0	4.3
	25	8.6	3	11.2	4.9	10.1	4.9
	50	9.2	3	11.9	5.4	10.7	5.4
	100	9.9	3.1	12.5	5.8	11.3	5.8
	200	10.6	3.2	13.1	6.3	11.8	6.3
Reach 3	5	6.5	2.3	4.2	2.9	3.8	2.9
	10	7.6	2.3	5.1	3.2	4.6	3.2
	25	8.6	2.4	6.3	3.7	5.7	3.7
	50	9.2	2.5	7	3.9	6.3	3.9
	100	9.9	2.6	7.6	4.3	6.8	4.3
	200	10.6	2.7	8.2	4.7	7.4	4.7
Reach 4	5	6.5	1.4	8.5	1.1	6.6	1.1
	10	7.6	1.5	9.4	1.2	7.3	1.2
	25	8.6	1.5	10.6	1.4	8.3	1.4
	50	9.2	1.6	11.3	1.5	8.8	1.5
	100	9.9	1.6	11.9	1.7	9.3	1.7
	200	10.6	1.7	12.5	1.8	9.8	1.8

1.7.2 Wave Setup at the Shoreline for Alternatives 1 through 5 and 5a through 5e

Wave setup is defined as the super-elevation of the mean water level caused by wave breaking action. Typically wave set down occurs at the wave breaking point, and then wave setup increases from the depth of breaking (db) to the intersection of the mean water level with the shoreline. For Highlands, the majority of design waves are non-breaking hence wave setup is insignificant.

1.7.3 Wave Heights for Optimization (5e Small, Medium, and Large)

At the time of the analysis, wave heights for the North Atlantic Coastal Comprehensive Study were not available. FEMA developed simulations using ADCIRC (ADvanced CIRCulation)+SWAN (Simulating WAves Nearshore) models. The output included peak surge elevation and associated significant wave heights and mean wave periods. These were used as forcing factors for a more localized ADCIRC model which simulated peak surge elevation and associated significant wave heights and mean wave periods at thirty nodes in the Highlands nearshore region. The nodes with wave data output are numbered 1 through 18, and are shown in Figure B2- 5.

The waves at four of the nodes (Node 18 for Reach 1, Node 11 for Reach 2, Node 9 for Reach 3, Node 8 for Reach 4) were transformed to the shoreline (to 2 or 3 different contour locations in each reach to better represent the reach conditions) using linear wave theory. The resulting wave characteristics are shown in Table B2- 7 for Reach 1, Table B2- 8 for Reach 2, Table B2- 9 for Reach 3, and Table B2- 10 for Reach 4.

1.7.4 Comparison of the Wave Heights used for Alternatives 1 through 5 and 5a through 5e and those used for Optimization of the Tentatively Selected Plan (5e Small, Medium, and Large)

Linear wave theory wave analysis resulted in nonbreaking waves used for design of Alternatives 1 through 5 and 5a through 5e. The New Orleans District wave model using FEMA water surface elevations was the wave climate used for evaluation of the optimization phase design of Alternatives 5e Small, Medium, and Large. The comparison of the two simulated water surface elevations is shown in **Error! Reference source not found.**.

At this point in time and due to study funding and scheduling constraints, it was infeasible to tease out how much of the change was due to wave setup and how much was model improvement. Nonetheless, it was decided to stick with the

2005 model results guiding the analysis of Alternatives 1 through 5, and 5a through 5e, and then using the NACCS data for optimization (5e Small, Medium, and Large). It is unlikely that using the NACCS data in the analysis of Alternatives 1 through 5, and 5a through 5e would have resulted in a different Tentatively Selected Plan (5e).

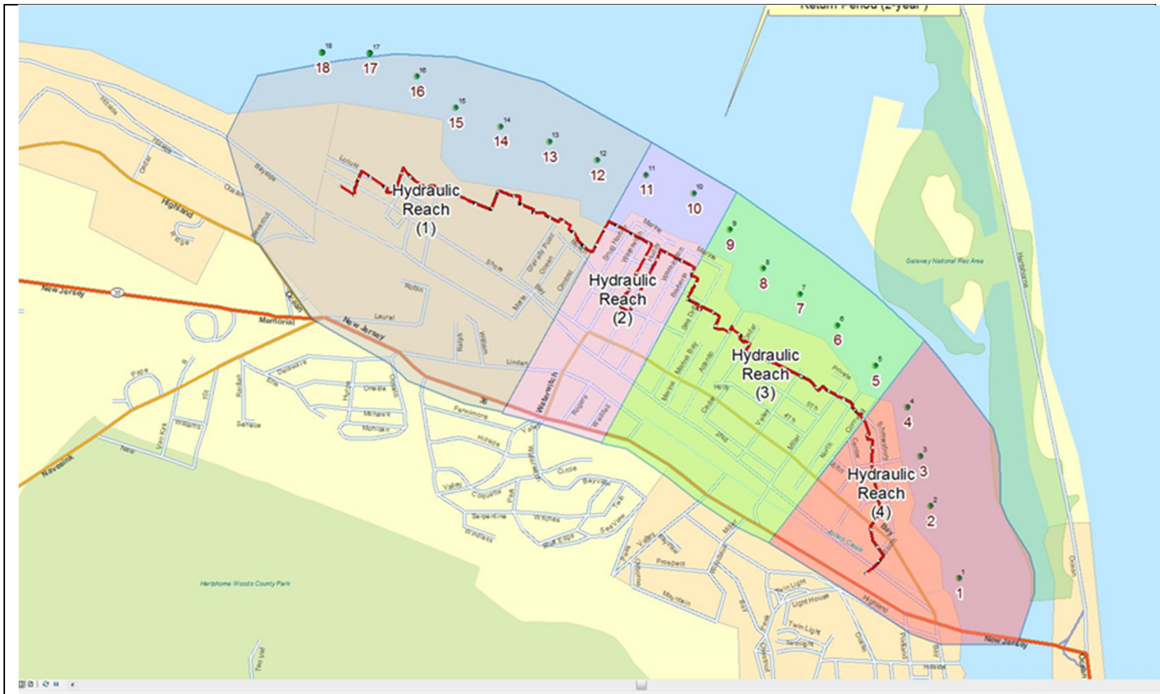

Figure B2- 5: Location of FEMA Modeling Nodes 1 through 18

Table B2- 7: Reach 1 Design Wave Characteristics for 5e Small, Medium, and Large

Reach	Node	Elevation Contour in ft. NAVD	Average Annual Exeedence Probability in %	Return Period in years	Significant Wave Height, Hs in ft	Peak Wave Period, Tp in sec.
1	18	0	100%	1	1.1	2.7
		0	50%	2	1.3	3.0
		0	20%	5	1.7	3.5
		0	10%	10	1.9	3.9
		0	5%	20	2.2	4.2
		0	2%	50	2.5	4.7
		0	1%	100	2.7	5.1
		0	1%	200	3.0	5.5
		0	0%	500	3.3	5.9
1	18	2	100%	1	0.8	2.6
		2	50%	2	1.0	3.0
		2	20%	5	1.4	3.4
		2	10%	10	1.6	3.8
		2	5%	20	1.9	4.1
		2	2%	50	2.2	4.6
		2	1%	100	2.5	5.0
		2	1%	200	2.7	5.3
		2	0%	500	3.0	5.8

Table B2- 8: Reach 2 Design Wave Characteristics for 5e Small, Medium, and Large

Reach	Node	Elevation Contour in ft. NAVD	Average Annual Exeedence Probability in %	Return Period in years	Significant Wave Height, Hs in ft	Peak Wave Period, Tp in sec.
2	11	0	100%	1	0.8	2.6
		0	50%	2	1.0	3.0
		0	20%	5	1.4	3.4
		0	10%	10	1.6	3.8
		0	5%	20	1.8	4.1
		0	2%	50	2.2	4.6
		0	1%	100	2.4	5.0
		0	1%	200	2.7	5.3
		0	0%	500	3.0	5.8
2	11	2	100%	1	1.1	2.6
		2	50%	2	1.3	3.0
		2	20%	5	1.7	3.5
		2	10%	10	1.9	3.8
		2	5%	20	2.2	4.2
		2	2%	50	2.5	4.7
		2	1%	100	2.8	5.0
		2	1%	200	3.0	5.4
		2	0%	500	3.4	5.8

Table B2- 9: Reach 3 Design Wave Characteristics for 5e Small, Medium, and Large

Reach	Node	Elevation Contour in ft. NAVD	Average Annual Exeedence Probability in %	Return Period in years	Significant Wave Height, Hs in ft	Peak Wave Period, Tp in sec.
3	9	0	100%	1	0.9	2.6
		0	50%	2	1.1	3.0
		0	20%	5	1.4	3.4
		0	10%	10	1.7	3.8
		0	5%	20	1.9	4.1
		0	2%	50	2.3	4.6
		0	1%	100	2.5	5.0
		0	1%	200	2.8	5.3
		0	0%	500	3.1	5.8
3	9	2	100%	1	0.8	2.6
		2	50%	2	1.0	3.0
		2	20%	5	1.4	3.4
		2	10%	10	1.6	3.8
		2	5%	20	1.9	4.1
		2	2%	50	2.2	4.6
		2	1%	100	2.5	5.0
		2	1%	200	2.7	5.3
		2	0%	500	3.0	5.8
3	9	3	100%	1	0.9	2.6
		3	50%	2	1.1	3.0
		3	20%	5	1.5	3.5
		3	10%	10	1.7	3.8
		3	5%	20	2.0	4.2
		3	2%	50	2.3	4.7
		3	1%	100	2.6	5.0
		3	1%	200	2.8	5.4
		3	0%	500	3.2	5.8

Table B2- 10: Reach 4 Design Wave Characteristics for 5e Small, Medium, and Large

Reach	Node	Elevation Contour in ft. NAVD	Average Annual Exeedence Probability in %	Return Period in years	Significant Wave Height, Hs in ft	Peak Wave Period, Tp in sec.
4	4	0.5	100%	1	0.8	2.6
		0.5	50%	2	1.0	3.0
		0.5	20%	5	1.4	3.4
		0.5	10%	10	1.6	3.8
		0.5	5%	20	1.9	4.1
		0.5	2%	50	2.2	4.6
		0.5	1%	100	2.4	5.0
		0.5	1%	200	2.7	5.3
		0.5	0%	500	3.0	5.8
4	4	2	100%	1	0.9	2.6
		2	50%	2	1.1	3.0
		2	20%	5	1.5	3.5
		2	10%	10	1.7	3.8
		2	5%	20	2.0	4.2
		2	2%	50	2.3	4.7
		2	1%	100	2.5	5.0
		2	1%	200	2.8	5.4
		2	0%	500	3.1	5.8
4	4	8	100%	1	0.0	2.7
		8	50%	2	0.7	3.0
		8	20%	5	1.6	3.5
		8	10%	10	2.2	3.9
		8	5%	20	2.5	4.2
		8	2%	50	2.8	4.7
		8	1%	100	3.1	5.1
		8	1%	200	3.3	5.5
		8	0%	500	3.6	5.9

2 Sea Level Change

2.1 Three USACE Scenarios

The Department of the Army Engineering Circular ER1100-2-8162 (31 Dec 2013) requires that future sea level change (SLC) projections must be incorporated into the planning, engineering design, construction and operation of all civil works projects. The project team should evaluate structural and non-structural components of the proposed alternatives in consideration of the “low,” “intermediate” and “high” potential rates of future SLC for both “with” and “without project” conditions.

Typically, foundation design is the least adaptable component of the line of protection. Overbuilt and future upgrade-type foundation designs suitable for intermediate and high potential rates of sea level change will be considered. SLC considers the effects of (1) the “regional” rate of vertical land movement (VLM) that can result from localized geological processes, including the shifting of tectonic plates, the rebounding of the Earth’s crust in locations previously covered by glaciers, the compaction of sedimentary strata and the withdrawal of subsurface fluids, and (2) the eustatic, or global, average of the annual increase in water surface elevation due to the global warming trend.

2.1.1 Vertical Land Movement

Highlands, New Jersey is located in an area that experiences positive land subsidence due to geological processes; therefore, the net relative sea level rise at Highlands is greater than the eustatic SLC. Said differently, when land in Highlands subsides as water surface elevation increases, the net local SLC is greater in Highlands than at a location experiencing an increase in water surface elevation only. When calculating the intermediate and high rates of sea level rise, the local rate of VLM must first be determined.

The local rate of VLM, which is considered to be constant through time, is determined by subtracting the NRC/IPCC eustatic SLC value (1.7 mm/yr) from the local mean sea level trend. Recall that the two components figuring into the local mean sea level include the eustatic SLC value and the local rate of VLM. The mean rate of SLC at the Sandy Hook station is +3.9 mm/year.

The local rate of VLM at Sandy Hook is calculated from the relationship: VLMSandy Hook = [local rate of SLC] – [eustatic rate of SLC], or VLMSandy Hook = 3.9 mm/yr – 1.7 mm/yr = 2.2 mm/yr.

This local rate of VLM is added back into the sea level rise computations after the eustatic portion has been determined from NRC curves I and III.

At Sandy Hook, the local rate of VLM accounts for a total of 0.61 ft. in year 2076 (the 50th year of the evaluation period).

2.1.2 Historic (or Low) Rate of Sea Level Change

The historic rate of future sea-level rise is determined directly from gauge data gathered in the vicinity of the project area. The nearest NOAA tide gauges from which tide data can be evaluated include: The Battery and Montauk Point gauges in New York, and the Sandy Hook gauge in New Jersey. Of these three locations, tide conditions at Sandy Hook (NOAA Station #8531680, as shown on Figure B2-1) best represent the conditions experienced in Highlands. A 75-year record (1932 to 2006) of tide data gathered at Sandy Hook, NJ indicates a mean sea level trend (eustatic SLR + the local rate of VLM) of +3.9 mm/year (Figure B2- 6).

At Sandy Hook, the Historic (or Low) Rate of SLC, including VLM, accounts for a total of 1.07 ft. in year 2076 (the 50th year of the evaluation period) since 1992, and is shown in Table B2- 11 and Figure B2- 7.

2.1.3 Intermediate Rate of Sea Level Change

The intermediate rate of local mean SLC is estimated by considering the modified NRC projections and adding the appropriate value to the local rate of vertical land movement. The intermediate rate of local sea level rise is based on the modified NRC Curve I since its value is comparable to that of the IPCC projection. The intermediate rate of sea level rise is computed using the equation
$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^2 - t_1^2) + \text{local VLM}$

where t_1 and t_2 represent the start and end dates of the projected time horizon in years, relative to 1992 (for both the intermediate and high rates of SLR, the NRC curves accelerate upward over time beginning in the year 1992 when the curves were developed; therefore, it is necessary to estimate SLR for a particular time horizon relative to 1992), and b is a constant value of $2.71E^{-5}$ for the intermediate rate.

At Sandy Hook, the Intermediate Rate of SLC, including VLM, accounts for a total of 1.70 ft. in year 2076 (the 50th year of the evaluation period) since 1992, and is shown in Table B2- 11 and Figure B2- 7.

2.1.4 High Rate of Sea Level Change

The high rate of local mean SLR is estimated by determining the modified NRC Curve III value and adding it to the local rate of vertical land movement. This high rate scenario exceeds the 2001 and 2007 IPCC projections and considers the potential rapid loss of ice from Antarctica and Greenland. The NRC Curve III is also based on the general equation $E(t) = 0.0017t + bt2$; however, the constant b changes to $b = 1.13E-4$, and has the same initial date of 1992.

At Sandy Hook, the High Rate of SLC, including VLM, accounts for a total of 3.69 ft. in year 2076 (the 50^{th} year of the evaluation period) since 1992, and is shown in Table B2- 11 and Figure B2- 7.

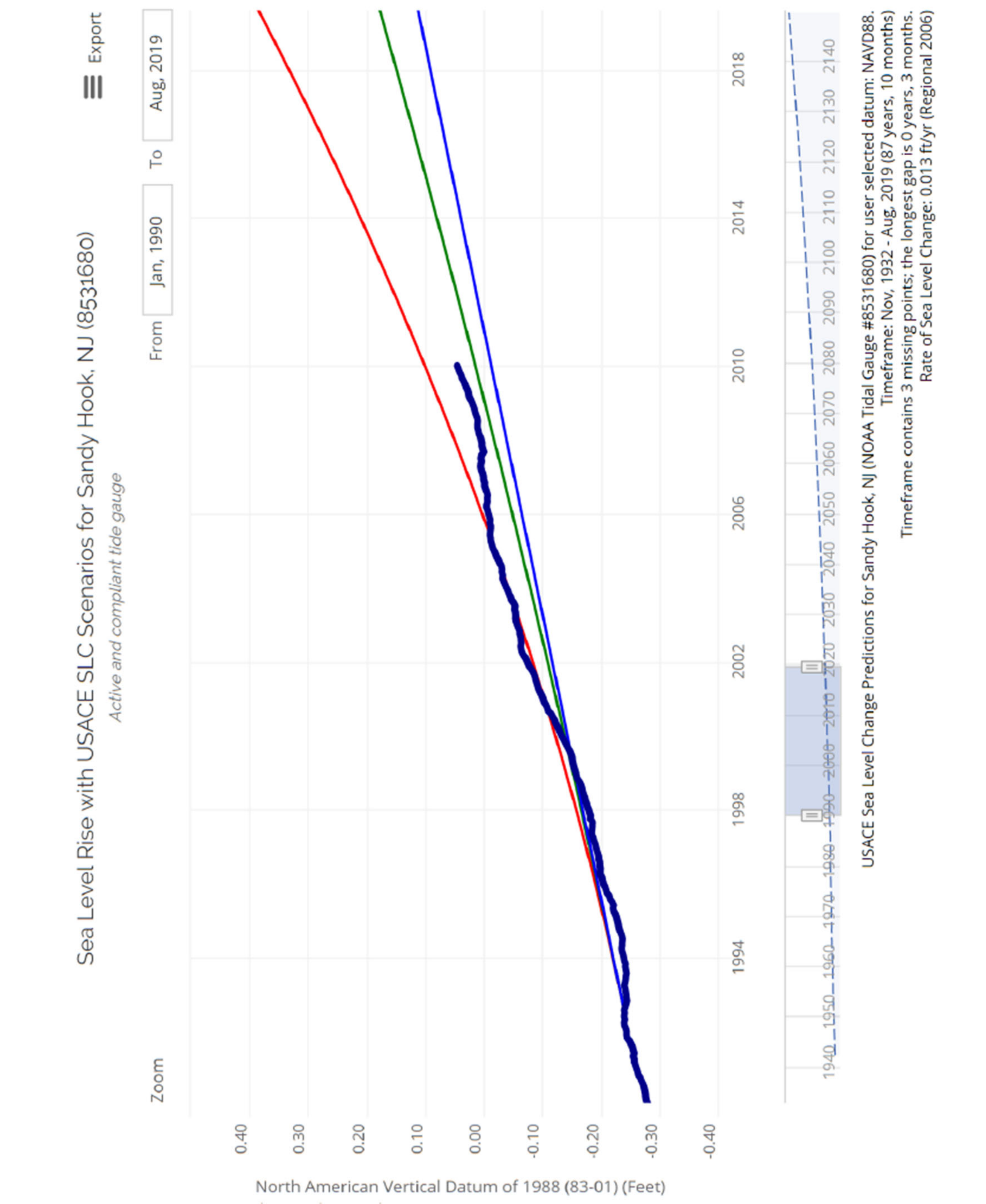

Figure B2- 6: Sea Level Tracker for Sandy Hook, NJ (NOAA Station #8531680)

Table B2- 11: Sea Level Change Estimates for Highlands, NJ			
USACE Curves computed using criteria in ER 1100-2-8162, 31 Dec 13			
Gauge: 8531680, NJ, Sandy Hook: 75 yrs. All values are in feet.			
Year	USACE Low	USACE Intermediate	USACE High
1992	0	0	0
2026	0.44	0.54	0.86
2036	0.56	0.74	1.28
2046	0.69	0.95	1.77
2056	0.82	1.18	2.34
2066	0.95	1.43	2.98
2076	1.07	1.70	3.69
2086	1.20	1.99	4.48
2096	1.33	2.29	5.34
2106	1.46	2.61	6.28
2116	1.59	2.95	7.29
2126	1.71	3.31	8.37

Note: Values provided past 2100 are speculative and should not be relied upon (see https://climate.sec.usace.army.mil).

2.2 Sea Level Change Tracker

The Climate Community of Practice created a Sea Level Tracker tool to show actual sea level vs. the projected sea level change curves plainly and to answer the question, "What rate of sea level change is currently being observed at the selected gauge?" The tool and its information may be located at https://climate.sec.usace.army.mil/slr_app/. The Sandy Hook location was selected for our study comparisons, and the plot of actual sea levels at this location between 1992 and 2019 is shown in Figure B2- 6. The actual sea level between 1992 and 2000 appears to approximately follow the low/historic rate. But following 2000, the recorded sea level varies between the intermediate and high rates. This may be an indication that the low/historic rate of sea level change is too low, and the future may hold sea levels between the intermediate and high rates. Adaptability is built into this project, as in others, such that if higher rates than the low/historic rate occurs, adaptive measures may easily and efficiently be applied.

2.3 Critical Thresholds

The values of sea level change in the above paragraphs are in feet. A significant portion of the streets in Highlands have average elevations lower than 3 ft. NAVD88 (0.6 ft. MHHW from 1983 to 2001 epoch). Figure B2- 8 shows the special extent of inundation from a water surface elevation of 3 ft. NAVD88. **Error! Reference source not found.** compares the road elevation of 0.6 ft. MHHW from 1983 to 2001 epoch to the Mean Sea Level in ft. MHHW over time from the Low, Intermediate, and High sea level change scenarios. Future Mean Higher High Water elevations equal ground elevations of 0.6 ft. MHHW from 1983 to 2001 epoch in 2020 for the High scenario, in 2030 for the Intermediate scenario, and in 2040 for the Low/Historic scenario. The Mean Sea Level from the High scenario is equivalent to 0.6 ft. MHHW from 1983 to 2001 epoch in 2060. This same occurs past 2100 for the Intermediate scenario. This indicates that in 2060 under the High scenario all ground elevations lower than 0.6 ft. MHHW from 1983 to 2001 epoch including roads will be inundated two times per tide cycle. This is without any storms.

When storms are included, Figure B2- 10 shows that in 2026 all ground elevations lower than 4.0 ft. MHHW are inundated under a High scenario, elevations lower than 3.7 ft. MHHW are inundated under a Medium scenario, and elevations lower than 3.6 ft. MHHW in the Low scenario at the peak of a 50% Annual Chance of Exceedance water surface elevation. And in 2076 all ground elevations lower than 6.9 ft. MHHW are inundated under a High scenario, elevations lower than 4.9 ft. MHHW are inundated under a Medium scenario, and elevations lower than 4.2 ft. MHHW in the Low scenario at the peak of a 50% Annual Chance of Exceedance water surface elevation.

2.4 Non-Linearity of Sea Level Change

Linear superposition of sea level change was assumed for the purposes of the analysis in this study. This means simply that sea level change experiences negligible nonlinear relationships which could cause sea level change damping or escalation. The NACCS website, https://chswebtool.erdc.dren.mil/ , was consulted for Global Sea Level Change Bias for Node 3555, which revealed negligible nonlinear effects.

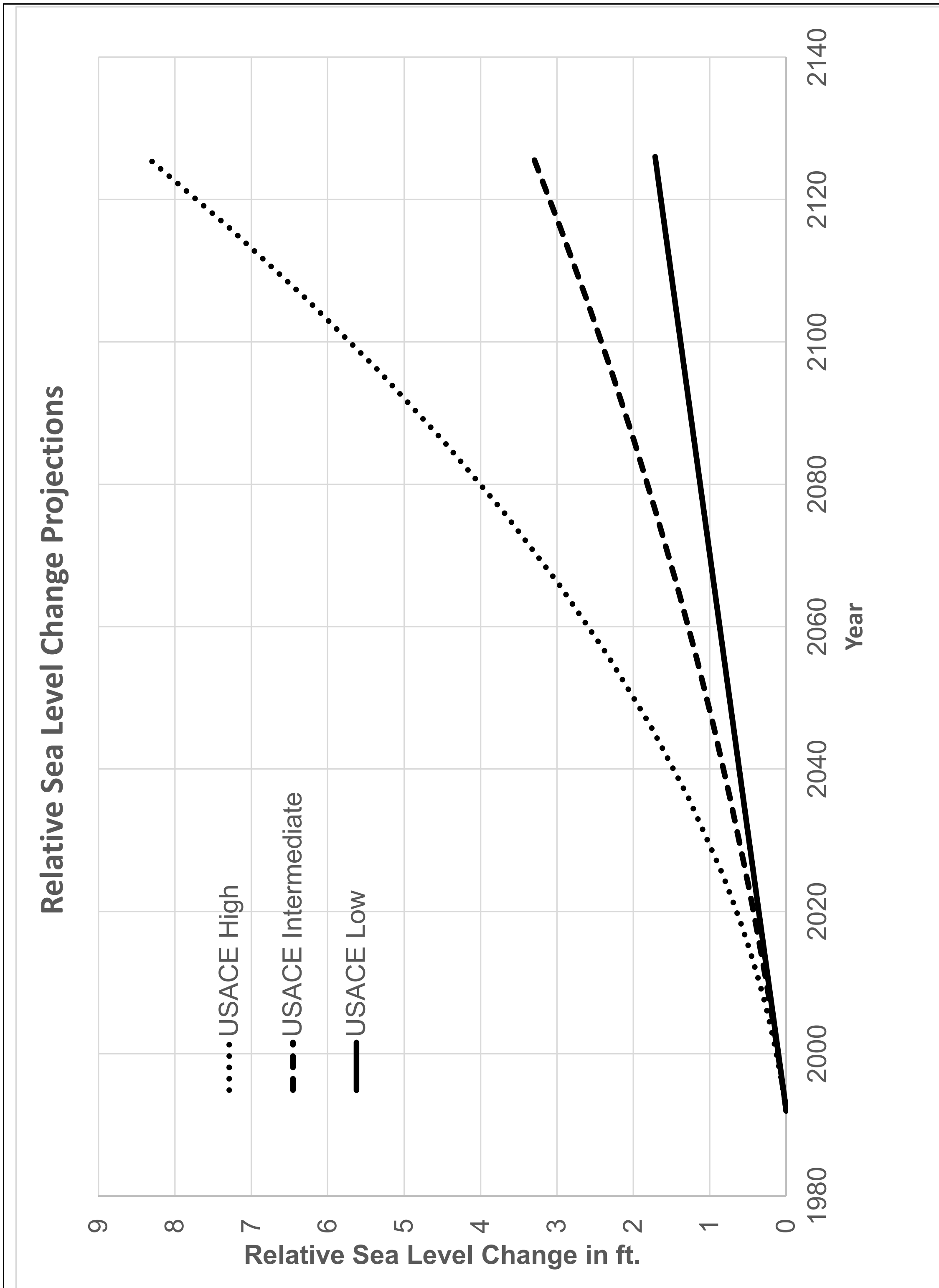

Figure B2- 7: Relative Sea Level Rise Projections in ft. at Sandy Hook

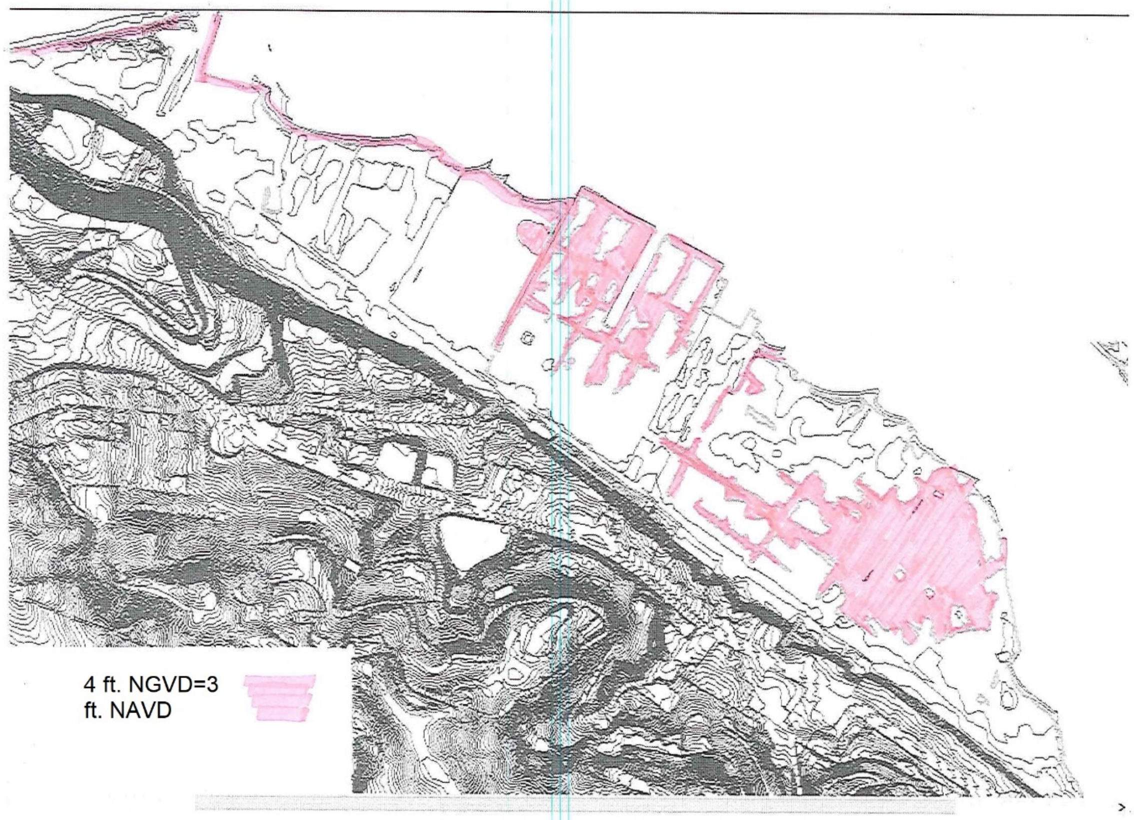

Figure B2- 8: Limits of Inundation for Water Surface Elevation of 3 ft. NAVD88

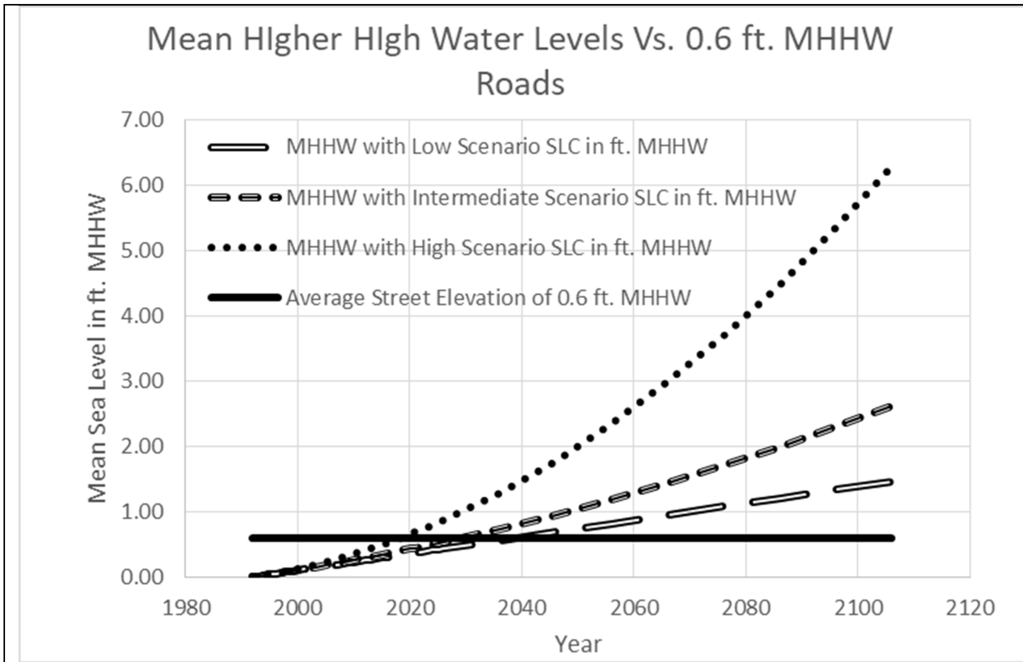

Figure B2- 9: Mean Sea Level and Street Elevation in ft. MHHW under the 3 Sea Level Change Scenarios

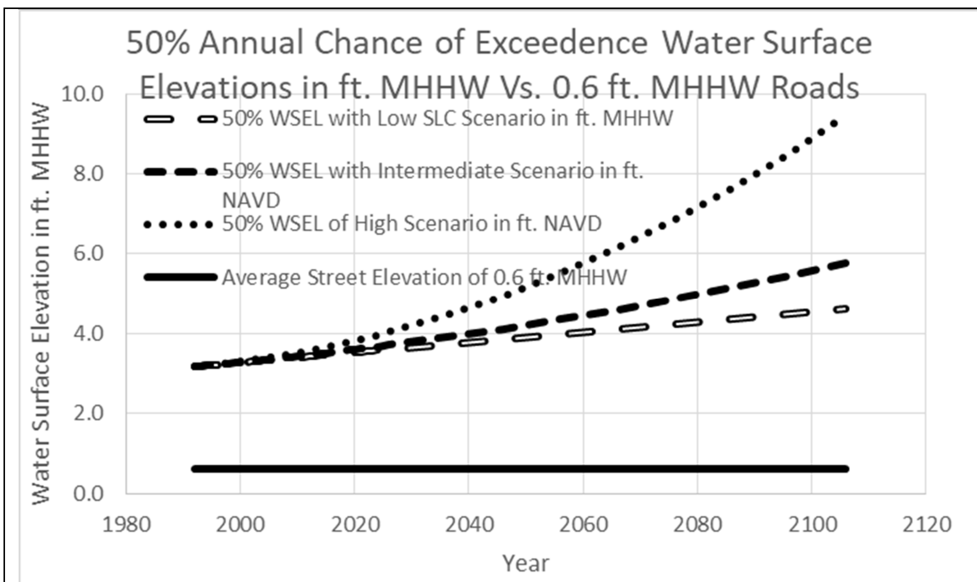

Figure B2- 10: 50% Annual Chance of Exceedance Water Surface Elevations in ft. MHHW Vs. 0.6 ft. MHHW Roads

3 Design Input for Evaluating Alternatives 1 through 5 and 5a through 5e

A 2% Annual Chance of Exceedance event was selected to compare each of the alternatives 1 through 5 to each other, as it was achievable yet wouldn't require a long tie back on the west side. Alternative 5 was later shown to have the largest net benefits. Alternatives 1 through 5 are listed below. (Details are given in the Civil Sub Appendix.)

- Alternative 1: Updated USACE Plan identified in the Pre-Feasibility Study
- Alternative 2: Non-Structural Plan
- Alternative 3: Offshore Closure Plan
- Alternative 4: Beach and Dune Fill Plan
- Alternative 5: Hybrid Plan

The Alternative 5 was varied in different ways, substituting different measures (bulkhead around marina instead of tide gate at mouth, permanent floodwall instead of temporary, etc.). Similarly, the resulting alternatives were formulated using a 2% Annual Chance of Exceedance. A few different bulkhead crest elevations were tested with the 2% event, the difference being the level of splash reinforcement required in the design cross-section. The resulting alternatives 5a through 5e are listed below, and described further in detail in the Civil Sub Appendix.

- Alternative 5a: Alternative 5 with Perimeter Bulkhead in lieu of Buoyant Swing Gate
- Alternative 5b: Alternative 5 with Raised Bulkhead and Non-Structural Measures in lieu of Removable Flood Wall, Target Elevation of 12 feet NGVD
- Alternative 5c: Alternative 5 with Raised Bulkheads and Non-Structural Measures in lieu of Removable Flood Wall, Target Elevation of 13.2 feet NGVD
- Alternative 5d: Alternative 5 with Raised Bulkheads in lieu of Removable Flood Wall, Target Elevation of 13.9 feet NGVD
- Alternative 5e: Alternative 5A and Alternative 5D Combined

3.1 Water Surface Elevation for Evaluating Alternatives 1 through 5 and 5a through 5e

The plans were formulated to provide for a 2% Annual Chance of Exceedance event, which had an expected water surface elevation of 8.8 ft. NAVD88 (see Table B2- 2).

3.2 Wave Heights for Evaluating Alternatives 1 through 5 and 5a through 5e

The relevant wave heights used were 9.2, 5.4, 3.9, and 1.5 feet for Reaches 1, 2, 3, and 4, respectively (see Table B2- 6).

3.3 Wave Overtopping Analysis (used for Crest Elevation Design) for Alternatives 1 through 5 and 5a through 5e

The crest elevations were determined using the threshold wave overtopping method, whereby 50 liters/sec/m threshold overtopping flowrate was selected to represent a no damage condition (refer to Department of the Army 1995, "Design of Coastal Revetments, Seawalls, and Bulkheads", U.S. Army Corps of Engineers, CECW-EH-D, Washington DC, Figure VI-5-6), and crest elevations are input to the equations iteratively. The lowest crest elevation that resulted in an overtopping flowrate less than 50 l/s/m was selected as the design crest elevation. The following overtopping methods were used:

- Ward, D.L., and J.P. Ahrens (1992), "Overtopping Rates for Seawalls", Miscellaneous Paper CERC-92-3, Coastal Engineering Research Center, U.S. Army Waterways Experiment Station, Vicksburg, MS
- Bradbury, A. P., and Allsop, N. W. 1988. "Hydraulic Effects of Breakwater Crown Walls, "Proceedings of the Breakwaters '88 Conference, Institution of Civil Engineers, Thomas Telford Publishing, London, UK, pp 385-396.
- Owen, M. W. 1982. "The Hydraulic Design of Seawall Profiles," Proceedings of the Coastal Protection Conference, Institution of Civil Engineers, Thomas Telford Publishing, London, UK, pp 185-192.
- van der Meer, J. W., and Janssen, W. 1995. "Wave Run-Up and Wave Overtopping at Dikes," In Wave Forces on Inclined and Vertical Wall Structures, Kobayashi and Demirbilek, eds., American Society of Civil Engineers, pp 1-27.

The first item was references in Department of the Army 1995, “Design of Coastal Revetments, Seawalls, and Bulkheads”, U.S. Army Corps of Engineers, CECW-EH-D, Washington DC. The last 3 of the list above were found in Department of the Army 2003, Coastal Engineering Manual”, EM 1110-2-1100, U.S. Army Corps of Engineers, CECW-EH-Y, Washington DC. Averages of the crest elevations from the four methods became the design crest elevations for Alternatives 1 through 5 and 5a through 5e.

4 Without- and With-Project Conditions for Alternatives 1 through 5 and 5a through 5e

The Borough of Highlands damage mechanisms have been identified as:

- inundation
- wave impacts to buildings

It is expected that storms will continue to occur into the future, causing damage in this area. Tidal inundation is expected to increase gradually over time, in direct relation to the anticipated rise in relative sea level. In future years this will result in more frequent and higher stages of flooding (see Section 6.6.4). And as waves were assumed to be depth limited, future wave damages were assumed to increase as sea levels increase.

4.1 Without-Project Conditions Input for Inundation Damages used for Evaluating Alternatives 1 through 5 and 5a through 5e.

To estimate the future condition state-frequency relationships, the incremental SLC rates were superimposed upon the existing condition stage-frequency relationships.

The water surface elevations from the 2005 model were assumed to represent 2005 conditions. And the change in water surface elevation between 2005 and 2026 was assumed to be negligible. Table B2- 12 contains the FEMA stage-frequency and wave-frequency relationships for the preliminary design for the 2076 condition with all three sea level change scenarios. (Note economic modeling utilizes the Low rate for this preliminary portion of the analysis. These alternatives would perform similarly to one another under each sea level change scenario for preliminary comparison. The next stage of evaluation simulates all three scenarios.) The comparison of Alternatives 1 through 5 and 5a through 5e was performed in the vicinity of 2006. The assumption of 2005 model results representing 2005 water surface elevations has since been proven false-in fact, 2005 model results represent water surface elevations of the midpoint of the most recent tidal epoch (1983 to 2001, SO 1992). And the assumption of the negligible change between the model results and 2026 has also since been proven false. Based on the types of measures used in the alternatives, should the erroneous assumptions been corrected, the selection of the tentatively selected plan is likely to have remained unchanged.

Table B2- 12: Without-Project Existing and Future Water Surface Elevations for Evaluation of Alternatives 1 through 5 and 5a through 5e

Annual Chance of Exceedance	1992 Water Surface Elevation (from FIMP (2005 model) in ft. NAVD88	2026 Water Surface Elevation in ft. NAVD88	2076 Water Surface Elevation under Low SLC in ft. NAVD88	2076 Water Surface Elevation under Intermediate SLC in ft. NAVD88	2076 Water Surface Elevation under High SLC in ft. NAVD88
50%	3.7	3.7	4.3	4.9	6.5
20%	5.4	5.4	6.0	6.6	8.2
10%	6.5	6.5	7.1	7.7	9.3
4%	7.5	7.5	8.1	8.7	10.3
2%	8.1	8.1	8.7	9.3	10.9
1%	8.8	8.8	9.4	10.0	11.6
0.50%	9.5	9.5	10.1	10.7	12.3
0.20%	10.1	10.1	10.7	11.3	12.9

4.2 Without-Project Conditions Input for Wave Impacts Used for Evaluating Alternatives 1 through 5 and 5a through 5e.

Shorefront areas in the Borough of Highlands are exposed to waves which can break against some buildings with enough force to destroy the structure.

Buildings is the operative word. The buildings are further back from the shoreline than a line of protection would be, and hence the waves developed above for the line of protection are not appropriate. The wave heights developed for wave attack on the buildings were all assumed to be depth limited. This means that the wave generation (or wave height) is limited by water depth. Depth-limited waves were used to estimate damage to the first row of buildings. Thereafter, FEMA's "Ways of Estimating Wave Heights in Coastal Hazard Areas" (April 1981) was used to determine the wave height for buildings landward of the first row. This method uses transmission factors to apply to the depth limited wave height fronting the structures, to account for the impedance of waves by the rows of buildings. The calculation of these waves was internal to the HEC-FDA modeling performed for economic calculations, once it was decided by coastal engineers to use depth-limited waves.

Wave heights used for estimating without-project existing and future conditions wave damages to buildings were computed internally in HEC-FDA modeling, assuming depth-limited wave processes. The water surfaces elevations shown in Table B2- 12 for the without-project existing and future conditions were compared to the building ground elevations. The resulting depths were multiplied by 0.78 to estimate the depth limited wave height at each building (this factor is the linear wave theory depth-limited wave limit for approximately flat slopes).

4.3 With-Project Conditions Input for Inundation Damages used for Evaluating Alternatives 1 through 5 and 5a through 5e.

The water surface elevations shown in Table B2- 12 for the existing and future conditions were compared to the crest elevations of Alternatives 1 through 5 and 5a through 5e. Where the water surface elevation is lower than the design crest elevation, the water surface elevation behind the line of protection at the building locations is assumed to be negligible. Where the water surface elevations exceed the crest elevations of the alternatives, the conservative assumption was made that the water surface elevations at the building locations was equivalent to the water surfaces elevations of the without-project existing and future conditions (as if the line of protection wasn't present, or rather as if the water surfaces behind the wall would rise to equal that in front of the wall, in an instant).

4.4 With-Project Conditions Input for Wave Impacts used for Evaluating Alternatives 1 through 5 and 5a through 5e.

Wave heights used for estimating with-project existing and future conditions wave damages to buildings were computed internally in HEC-FDA modeling, assuming depth-limited wave processes. The water surfaces for the with-project existing and future conditions water surface elevations described in the above paragraph were compared to the building ground elevations. The resulting depths were multiplied by 0.78 to estimate the depth limited wave height at each building (this factor is the linear wave theory depth-limited wave limit for approximately flat slopes). Where the water surface elevations are lower than the crest elevation of the alternative, water depth at the buildings is assumed to be negligible, hence wave impacts would also be negligible. Where the water surface elevations exceed the crest elevations of the alternative, it was assumed that the water surface elevations at the buildings was equivalent to those for the without-project existing and future conditions water surface elevations, with the wave heights being computed at 0.78 times the water depth.

5 Design Input for Optimizing (5e Small, Medium, and Large)

Three crest elevations of 11, 13, and 14 ft. NAVD88 were selected for the Small, Medium, and Large plans, respectively. These guide the crest elevations not subject to wave energy, such as tie-back elevations. Crest elevations accessible to waves shall be adjusted to prevent wave overtopping greater than 50 l/s/m. Still the Small, Medium, and Large versions of Alternative 5e are referred to by the crest elevations 11, 13, and 14 ft. NAVD88.

The small plan is designed to the height of the existing NJ state bulkhead in Reach 2 (+11 ft. NAVD88) along the entire alignment. The alternative assumes replacement of the existing bulkhead upfront rather than waiting to midway through the period of analysis to minimize uncertainty about project performance.

The medium plan is the updated TSP. The design heights (which vary throughout the alignment in response to wave action) are unchanged, but the project components have been changed from raised or capped bulkheads to a combination of I-type and T-type floodwalls.

The large plan is designed to height of the planned floodwall/bulkhead under construction at the condominium development at the western end of the alignment +14 ft. NAVD88. The developer chose the height after consulting the FEMA FIRMs. The elevation of +14ft ft. NAVD88 is along the entire alignment, and is achieved mostly through the construction of T-type floodwalls and I-type floodwalls where possible.

5.1 Water Surface Elevations for Optimizing (5e Small, Medium, and Large)

The water surface elevations used for the evaluation of the 5e Small, Medium, and Large plans are shown in Table B2- 13.

5.2 Wave Heights for Optimizing (5e Small, Medium, and Large)

The design wave characteristics are shown in Table B2- 7 for Reach 1, Table B2- 8 for Reach 2, Table B2- 9 for Reach 3, and Table B2- 10 for Reach 4.

5.3 Crest Elevations for Optimization (5e Small, Medium, and Large)

The Tentatively Selected Plan (5e) became the medium plan. One smaller and one larger plan was tested. The medium plan maximum crest elevations varied slightly alongshore (12.5 ft. NAVD88 in Reach 1, +12 ft. NAVD88 in Reaches 2 and 3, and +13 ft. NAVD88 in Reach 4). The small and large plans had consistent crest elevations alongshore (+11 ft. NAVD88 for the small plan, and +14 ft. NAVD88 for the large plan). These are elaborated below.

5.3.1 Reach 1 Crest Elevations for Optimization (5e Small, Medium, and Large)

For features in Reach 1, the small, medium, and large design elevation are set at 11.0, 12.5, and 14.0 ft. NAVD88, respectively. Raised bulkheads are proposed throughout Reach 1 and will include toe stone and concrete splash pad along the entire length. The seaside toe stone will provide toe reinforcement against erosion and will act as a rubble toe to reduce wave action. The rubble toe berm will consist of 3T, 4T, and 7T armor in a triangular configuration of elevation 10.9 ft., 11 ft., and 11 ft. NAVD88, and fronting slope of 1V:2H overlying a 12 in. layer of bedding material on geotextile for the small, medium and large design plans, respectively. The concrete splash pad will be placed on the landside to protect against erosion from overtopping. The splash pad is 10 ft. wide and 2 ft. thick and will be placed on top of a 1 ft. layer of bedding material on geotextile (for all plans).

5.3.2 Reach 2 Crest Elevations for Optimization (5e Small, Medium, and Large)

In Reach 2, the small, medium and large plans will consist for the 1,415 linear feet of the existing state bulkhead will fronted by a new sheetpile wall of top crest elevation 11, 12, and 14 ft. NAVD88, respectively. A wave deflection feature of approximately 10 to 15 degrees will be applied to the cap for the medium and large plans to reduce wave overtopping impacts. The landward side of the capped bulkhead (above grade) will need to be structurally reinforced to avoid the potential of exceeding the design loads of the existing bulkhead with the added loads intercepted by the capping for all plans. This reinforcement will include a 1.5-foot thick (average) monolithic section of reinforced concrete along the landside of the existing bulkhead, continuing with a 2-foot thick, 10-foot wide monolithic reinforced concrete slab at grade.

In the center of Reach 2 is a canal containing Captain's Cove Marina. A setback raised bulkhead on the landward side of the existing perimeter bulkhead is proposed with crest elevations of 11, 12, and 14 ft. NAVD88 for the small, medium, and large plans, respectively. No rubble toe is proposed for the setback wall as wave action is reduced within the marina and toe reinforcement will be provided by the existing wall that is left in place. A concrete splash pad is included on the landside of the setback wall to protect against erosion from overtopping for all plans (same design as for Reach 1). The Reach 2 capped bulkhead resumes at the end of the canal and will connect to a raised bulkhead in Reach 3.

5.3.3 Reach 3 Crest Elevations for Optimization (5e Small, Medium, and Large)

For features in Reach 3, the small, medium and large design elevations are set at 11, 12, and 14 ft. NAVD88, respectively. Raised bulkheads are proposed throughout Reach 3 and will include toe stone (1T armor for all plans) triangular in shape with a crest elevation of 5 ft. NAVD88 (for all plans) and fronting slope of 1V:2H, with the same concrete splash pad configuration as for Reach 1 along the entire length.

5.3.4 Reach 4 Crest Elevations for Optimization (5e Small, Medium, and Large)

In Reach 4 the floodwall will transition to elevations 11, 13, and 14 ft. NAVD88, and then transition down to be encompassed by the Veterans Memorial Park, which ground is to be elevated to 9, 10, and 11 ft. NAVD88 to cover the sheetpile for the small, medium, and large plans, respectively. The raised surface will duplicate the existing park features and surfacing, including the raising of a monument at the entrance to the park. The raised ground area will be capped with 6 inches of topsoil and planted with native vegetation. At the southeastern end of this area, the crest elevation of the raised ground will continue at elevations 9, 10, and 11 ft. NAVD88 and meet a section of concrete I-Wall Closure Structure with structural steel hydraulic gates crossing the existing Bay Avenue and tying into the bluff for the small, medium, and large plans, respectively.

6 Without- and With-Project Conditions for Optimizing (5e Small, Medium, and Large)

The same damage mechanisms of inundation and wave impacts to buildings were estimated. In addition, wave overtopping damage was estimated by reach to existing and proposed shoreline structures.

6.1 Without-Project Conditions Input for Inundation Damages used for Optimization (5e Small, Medium, and Large).

To estimate the future condition state-frequency relationships, the incremental SLC rates were superimposed upon the existing condition stage-frequency relationships.

Table B2- 13 contains the NACCS water surface elevations for 2026 and 2076 with three rates of sea level change (Low, Intermediate, and High). Low sea level change was assumed between 1992 (the midpoint of the NOAA Tidal Datum Epoch) and 2026. (Note economic modeling utilizes all years of sea level change, not only 2026 and 2076. Year 2076 is just shown for comparison here.)

Table B2- 13: Without-Project Existing and Future Condition Water Surface Elevations for Optimization (5e Small, Medium, and Large)

Annual Chance of Exceedence in %	Water Surface Elevation (1983 to 2001 epoch) in ft. NAVD88	Change in Water Surface Elevation between 1992 and 2026 in feet	Existing Condition Water Surface Elevation (in Project Base Year 2026) in ft. NAVD88	Future Condition (2076) Water Surface Elevation with **Low Rate of SLC** in ft. NAVD88	Future Condition (2076) Water Surface Elevation with **Intermediate Rate of SLC** in ft. NAVD88	Future Condition (2076) Water Surface Elevation with **High Rate of SLC** in ft. NAVD88
100%	4.7	0.44	5.2	5.8	6.4	8.4
50%	5.6	0.44	6.0	6.6	7.3	9.3
20%	6.7	0.44	7.2	7.8	8.4	10.4
10%	7.5	0.44	8.0	8.6	9.2	11.2
5%	8.4	0.44	8.8	9.5	10.1	12.1
2%	9.7	0.44	10.1	10.7	11.4	13.4
1.00%	11.0	0.44	11.4	12.1	12.7	14.7
0.50%	12.6	0.44	13.0	13.7	14.3	16.3
0.20%	14.6	0.44	15.0	15.7	16.3	18.3
USACE Curves computed using criteria in ER 1100-2-8162, 31 Dec 13						
Gauge: 8531680, NJ, Sandy Hook: 75 yrs. All values are in feet.						

6.2 *Without-Project Conditions Input for Wave Damage used for Optimization (5e Small, Medium, and Large).*

Wave heights used for estimating without-project existing and future conditions wave damages to buildings were computed internally in HEC-FDA modeling, assuming depth-limited wave processes. The water surfaces shown in Table B2-13 for the without-project existing and future conditions water surface elevations were compared to the building ground elevations. The resulting depths were multiplied by 0.78 to estimate the depth limited wave height at each building (this factor is the linear wave theory depth-limited wave limit for approximately flat slopes).

6.3 Without-Project Conditions Input for Wave Overtopping Damage to Shoreline Structures used for Optimization (5e Small, Medium, and Large).

The dominant wave-induced coastal processes affecting the project shoreline is wave overtopping. The Ward and Ahrens method of overtopping (Ward and Ahrens, 1992) was employed to compare the without and the with-project damages for varied levels of projected sea-level change. A non-failure overtopping flow rate threshold of 50 liter/s/m and a full failure point of 200 l/s/m were adopted. The results are described by reach below. In the figures, a green circle indicates flow rate is below the non-failure threshold, yellow indicates it’s between the non-failure and the failure thresholds, and the red circle that the flow rate exceeds the failure threshold.

6.3.1 Reach 1 Without-Project Conditions Input for Wave Overtopping Damage to Shoreline Structures used for Optimization (5e Small, Medium, and Large).

The Reach 1 without-project wave overtopping flow rates are shown in Table B2-14 below. The results show that the existing condition grade can withstand a 100% average annual exceedance event for 3 ft. or less of sea level change without failure and a 10% AAEP event for no sea level change. The results also predict failure of the existing grades for a 20% AAEP event with 2 ft. of sea level change; and for a 2% AAEP event for current levels of sea level.

Table B2- 14: Reach 1 Optimization Without-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 0' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 0.5' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 1' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 1.5' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 2' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 2.5' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 3' SLC
100%	1	0	0	0	0	1	4	12
50%	2	0	0	1	3	18	46	115
20%	5	4	11	27	76	233	484	995
10%	10	37	79	166	375	909	1692	3132
5%	20	194	370	698	1368	2820	4845	8243
2%	50	1256	2136	3605	6185	11040	17371	27182
1%	100	5541	8792	13871	21974	35284	52873	78927
1%	200	23106	34566	51493	76538	113797	162544	234003
0%	500	103781	147451	209024	287674	401990	551741	757212

6.3.2 Reach 2 Without-Project Conditions Input for Wave Overtopping Damage to Shoreline Structures used for Optimization (5e Small, Medium, and Large).

The Reach 2 without-project wave overtopping flow rates are shown in Table B2-15 below. The results show that the existing condition grade landward of the existing State bulkhead can withstand a 50% average annual exceedance event for 3 ft. or less of sea level change without failure; and a 5% AAEP event for 0.5 ft. of sea level change or less. The results also predict failure of the existing landward grades for a 10% AAEP event with 3 ft. or more of sea level change; and for a 1% AAEP event for current levels of sea level.

Table B2- 15: Reach 2 Optimization Without-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 0' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 0.5' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 1' SLC	Without-Project Condition Overtopping Flowrate at 2' NAVD in l/s/m for 1.5' SLC	Without-Project Condition Overtopping Flowrate at 2' NAVD in l/s/m for 2' SLC	Without-Project Condition Overtopping Flowrate at 2' NAVD in l/s/m for 2.5' SLC	Without-Project Condition Overtopping Flowrate at 2' NAVD in l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	1	3
20%	5	0	0	1	2	11	25	53
10%	10	2	4	9	22	72	138	261
5%	20	15	29	56	122	317	556	967
2%	50	145	251	429	840	1772	2826	4488
1%	100	819	1317	2103	3809	7033	10621	15992
0.5%	200	4187	6338	9548	16188	27006	38848	55811
0.2%	500	23295	33341	47590	75237	114543	157844	217243

6.3.3 Reach 3 Without-Project Conditions Input for Wave Overtopping Damage to Shoreline Structures used for Optimization (5e Small, Medium, and Large).

The Reach 3 without-project wave overtopping flow rates are shown in Table B2-16 below. The results show that the existing condition grade can withstand a 100% average annual exceedance event for up to 3 ft. of sea level change without failure and a 10% AAEP event for current sea levels. The results also predict failure of the existing grades for a 20% AAEP event with 3 ft. or more of sea level change; and for a 2% AAEP event for current sea levels.

Table B2- 16: Reach 3 Optimization Without-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 0' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 0.5' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 1' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 1.5' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 2' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 2.5' SLC	Without-Project Condition Overtopping Flowrate at 0' NAVD in l/s/m for 3' SLC
100%	1	0	0	0	0	1	4	12
50%	2	0	0	1	3	18	46	115
20%	5	4	11	27	76	233	484	995
10%	10	37	79	166	375	909	1692	3132
5%	20	194	370	698	1368	2820	4845	8243
2%	50	1256	2136	3605	6185	11040	17371	27182
1%	100	5541	8792	13871	21974	35284	52873	78927
0.5%	200	23106	34566	51493	76538	113797	162544	234003
0.2%	500	103781	147451	209024	287674	401990	551741	757212

6.3.4 Reach 4 Without-Project Conditions Input for Wave Overtopping Damage to Shoreline Structures used for Optimization (5e Small, Medium, and Large).

Reach 4 has two distinct portions: the standard water fronting bulkhead with the toe between 0 and 2 ft. NAVD88, and the tie-back region on land with the land elevations approximately 8 ft. NAVD88. These regions were modeled separately and called Reach 4 Waterfront and Reach 4 Tie-Back, respectively.
The Reach 4 without-project waterfront wave overtopping flow rates are shown in Table B2- 17 below. The results show that the existing condition grade landward of the existing local bulkheads can withstand a 100% average annual

exceedance event for 3 ft. or less of sea level change; and a 10% AAEP event for current sea levels without failure. The results also predict failure of the existing landward grades for a 20% AAEP with 3 ft. or more of sea level change; and for a 2% AAEP event for current sea levels.

Table B2- 17: Reach 4 Optimization Waterfront Without-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Without-Project Condition Overtopping Flowrate at 0.5' NAVD in l/s/m for 0' SLC	Without-Project Condition Overtopping Flowrate at 0.5' NAVD in l/s/m for 0.5' SLC	Without-Project Condition Overtopping Flowrate at 0.5' NAVD in l/s/m for 1' SLC	Without-Project Condition Overtopping Flowrate at 2' NAVD in l/s/m for 1.5' SLC	Without-Project Condition Overtopping Flowrate at 2' NAVD in l/s/m for 2' SLC	Without-Project Condition Overtopping Flowrate at 2' NAVD in l/s/m for 2.5' SLC	Without-Project Condition Overtopping Flowrate at 2' NAVD in l/s/m for 3' SLC
100%	1	0	0	0	0	1	3	10
50%	2	0	0	1	3	16	43	111
20%	5	4	11	27	74	236	498	1038
10%	10	37	79	166	389	958	1799	3356
5%	20	196	374	706	1465	3054	5253	8965
2%	50	1278	2175	3672	6786	12153	19107	30112
1%	100	5665	8987	14186	24266	39255	58714	87674
0.5%	200	23606	35523	52923	84820	127201	181210	257888
0.2%	500	106546	151137	214126	322212	449228	615750	843124

The Reach 4 Tie-Back without-project wave overtopping flow rates are shown in Table B2- 18 below. The results show that the existing condition grade landward of the existing local bulkheads can withstand a 20% average annual exceedance event for 3 ft. or less of sea level change; and a 10% AAEP for current sea levels without failure. The results also predict failure of the existing landward grades for a 10% AAEP with 3 ft. or more of sea level change; and for a 2% AAEP event for current sea levels.

Table B2- 18: Reach 4 Optimization Tie-Back Without-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Without-Project Condition Overtopping Flowrate at 0.5' NAVD in l/s/m for 0' SLC	Without-Project Condition Overtopping Flowrate at 0.5' NAVD in l/s/m for 0.5' SLC	Without-Project Condition Overtopping Flowrate at 0.5' NAVD in l/s/m for 1' SLC	Without-Project Condition Overtopping Flowrate at 8' NAVD in l/s/m for 1.5' SLC	Without-Project Condition Overtopping Flowrate at 8' NAVD in l/s/m for 2' SLC	Without-Project Condition Overtopping Flowrate at 8' NAVD in l/s/m for 2.5' SLC	Without-Project Condition Overtopping Flowrate at 8' NAVD in l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	1	0	0	0	0
20%	5	4	11	27	27	27	27	27
10%	10	37	79	166	167	167	167	231
5%	20	196	374	706	1816	1816	1816	1816
2%	50	1278	2175	3672	12349	12349	12349	12349
1%	100	5665	8987	14186	14186	14186	17999	26357
0.5%	200	23606	35523	52923	155599	155599	155599	155599
0.2%	500	106546	151137	214126	555841	555841	555841	555841

6.4 With-Project Conditions Input for Inundation Damages used for Optimization (5e Small, Medium, and Large).

A non-failure overtopping flow rate threshold of 50 liter/s/m and a full failure point of 200 l/s/m were adopted. With-project inundation damages to the buildings landward of the line of protection are zero, until the point that the overtopping exceeds 200 l/s/m. At that point, the water surface elevations landward of the failed line of protection equal the water surface elevations seaward of the line of protection, which are shown above in Table B2- 13.

6.5 With-Project Conditions Input for Wave Damage used for Optimizing (5e Small, Medium, and Large).

Similar to the inundation damages above, wave damage to buildings inside the line of protection occur only after failure of the line of protection. If an event creates a flowrate of greater than 200 l/s/m, then the water surface elevation at the buildings is equal to that experienced with the line of protection. The HEC-FDA computes a depth limited wave height at the buildings, with an associated wave transmission factor landward of the first row.

6.6 With-Project Conditions Input for Wave Overtopping Damage to the Line of Protection used for Optimizing (5e Small, Medium, and Large)

6.6.1 5e Small Plan With-Project Conditions Input for Wave Overtopping Damage

6.6.1.1 5e Small Plan Reach 1 With-Project Conditions Input for Wave Overtopping Damage

The Reach 1 small plan with-project wave overtopping flow rates are shown in Table B2- 19 below. The results show that the splash pad and supporting grade landward of the seawall can withstand a 50% AAEP event for 3 ft. of sea level change or less; and a 5% AAEP event for today's sea level. The results also predict failure of the splash pad and supporting grade landward of the seawall, potentially causing complete failure of the wall for a 10% AAEP event with 3 ft. or more of sea level change; and for a 1% AAEP event for current levels of sea level.

Table B2- 19: 5e Small Plan Reach 1 With-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Small Plan Overtopping Flowrate at 2' NAVD in l/s/m for 0' SLC	Small Plan Overtopping Flowrate at 2' NAVD in l/s/m for 0.5' SLC	Small Plan Overtopping Flowrate at 2' NAVD in l/s/m for 1' SLC	Small Plan Overtopping Flowrate at 0' NAVD in l/s/m for 1.5' SLC	Small Plan Overtopping Flowrate at 0' NAVD in l/s/m for 2' SLC	Small Plan Overtopping Flowrate at 0' NAVD in l/s/m for 2.5' SLC	Small Plan Overtopping Flowrate in at 0' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	1	3
20%	5	0	0	1	3	12	26	55
10%	10	2	4	8	23	72	138	259
5%	20	15	29	56	121	308	537	927
2%	50	151	261	452	794	1658	2643	4187
1%	100	879	1413	2259	3535	6477	9789	14736
1%	200	4561	6899	10412	14875	24634	35489	51341
0%	500	25632	36597	52153	68294	103883	143348	197667

6.6.1.2 5e Small Plan Reach 2 With-Project Conditions Input for Wave Overtopping Damage

The Reach 2 small plan with-project wave overtopping flow rates are shown in Table B2- 20 below. The results show exactly the same flow rates as for the existing condition, which makes sense since the small plan elevations are equivalent to that of the existing condition.

Table B2- 20: 5e Small Plan Reach 2 With-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Small Plan Overtopping Flowrate at 0' NAVD in l/s/m for 0' SLC	Small Plan Overtopping Flowrate at 0' NAVD in l/s/m for 0.5' SLC	Small Plan Overtopping Flowrate at 0' NAVD in l/s/m for 1' SLC	Small Plan Overtopping Flowrate at 2' NAVD in l/s/m for 1.5' SLC	Small Plan Overtopping Flowrate at 2' NAVD in l/s/m for 2' SLC	Small Plan Overtopping Flowrate at 2' NAVD in l/s/m for 2.5' SLC	Small Plan Overtopping Flowrate in at 2' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	1	3
20%	5	0	0	1	2	11	25	53
10%	10	2	4	9	22	72	138	261
5%	20	15	29	56	122	317	556	967
2%	50	145	251	429	840	1772	2826	4488
1%	100	819	1317	2103	3809	7033	10621	15992
0.5%	200	4187	6338	9548	16188	27006	38848	55811
0.2%	500	23295	33341	47590	75237	114543	157844	217243

6.6.1.3 5e Small Plan Reach 3 With-Project Conditions Input for Wave Overtopping Damage

The Reach 3 small plan with-project wave overtopping flow rates are shown in Table B2- 21 below. The results show that the splash pad and supporting grade landward of the seawall can withstand a 50% AAEP event for up to 3 ft. of sea level change; and a 5% AAEP event for current sea levels. The results also predict failure of the splash pad and supporting grade landward of the seawall, potentially causing complete failure of the wall for a 10% AAEP event with 3 ft. or more of sea level change; and for a 1% AAEP event for current levels of sea level.

Table B2- 21: 5e Small Plan Reach 3 With-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Small Plan Overtopping Flowrate at 2' NAVD in l/s/m for 0' SLC	Small Plan Overtopping Flowrate at 2' NAVD in l/s/m for 0.5' SLC	Small Plan Overtopping Flowrate at 2' NAVD in l/s/m for 1' SLC	Small Plan Overtopping Flowrate at 3' NAVD in l/s/m for 1.5' SLC	Small Plan Overtopping Flowrate at 3' NAVD in l/s/m for 2' SLC	Small Plan Overtopping Flowrate at 3' NAVD in l/s/m for 2.5' SLC	Small Plan Overtopping Flowrate in at 3' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	1	2
20%	5	0	0	1	2	11	24	52
10%	10	2	4	8	22	72	139	265
5%	20	15	29	56	124	326	573	996
2%	50	151	261	452	874	1856	2956	4693
1%	100	879	1413	2259	4001	7416	11200	16799
0.5%	200	4561	6899	10412	17061	28493	41156	59123
0.2%	500	25632	36597	52153	79400	121781	166437	228612

6.6.1.4 5e Small Plan Reach 4 With-Project Conditions Input for Wave Overtopping Damage

Reach 4 has two distinct portions: the standard water fronting bulkhead with the toe between 0 and 2 ft. NAVD88, and the tie-back region on land with the land elevations approximately 8 ft. NAVD88. These regions were modeled separately and called Reach 4 Waterfront and Reach 4 Tie-Back, respectively.

The Reach 4 small plan waterfront with-project wave overtopping flow rates are shown in Table B2- 22 below. The results show that the splash pad and supporting grade landward of the seawall can withstand a 50% average annual exceedance event for 3 ft. or less of sea level change; and a 5% AAEP event for current sea level without failure. The results also predict failure of the existing landward grades for a 10% AAEP with 3 ft. or more of sea level change; and for a 1% AAEP event for current sea levels.

Average Annual Exceedence Probability in %	Return Period in Years	Small Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 0' SLC	Small Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 0.5' SLC	Small Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 1' SLC	Small Plan Overtopping Flowrate at 2' NAVD in l/s/m for 1.5' SLC	Small Plan Overtopping Flowrate at 2' NAVD in l/s/m for 2' SLC	Small Plan Overtopping Flowrate at 2' NAVD in l/s/m for 2.5' SLC	Small Plan Overtopping Flowrate in at 2' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	1	3
20%	5	0	0	1	2	11	25	53
10%	10	2	4	9	22	72	138	261
5%	20	15	29	56	122	317	556	967
2%	50	146	253	434	840	1772	2826	4488
1%	100	831	1337	2136	3809	7033	10621	15992
0.5%	200	4266	6463	9741	16188	27006	38848	55811
0.2%	500	23814	34054	48587	75237	114543	157844	217243

Table B2- 22: 5e Small Plan Reach 4 Waterfront With-Project Flowrates

The Reach 4 small plan tie-back with-project wave overtopping flow rates are shown in Table B2- 23 below. The results show that the splash pad and supporting grade landward of the seawall can withstand a 20% average annual exceedance event for 3 ft. or less of sea level change; and a 10% AAEP event for current sea level without failure. The results also predict failure of the existing landward grades for a 10% AAEP with 3 ft. or more of sea level change; and for a 2% AAEP event for current sea levels.

Table B2- 23: 5e Small Plan Reach 4 Tie-Back With-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Small Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 0' SLC	Small Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 0.5' SLC	Small Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 1' SLC	Small Plan Overtopping Flowrate at 8' NAVD in l/s/m for 1.5' SLC	Small Plan Overtopping Flowrate at 8' NAVD in l/s/m for 2' SLC	Small Plan Overtopping Flowrate at 8' NAVD in l/s/m for 2.5' SLC	Small Plan Overtopping Flowrate in at 8' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	0	0
20%	5	0	0	1	0	0	0	11
10%	10	2	4	9	9	9	47	231
5%	20	15	29	56	28	158	569	1527
2%	50	146	253	434	1049	2773	4839	7447
1%	100	831	1337	2136	6542	12334	17999	26357
0.5%	200	4266	6463	9741	27129	45723	64158	90147
0.2%	500	23814	34054	48587	122392	187871	253889	339564

6.6.2 5e Medium Plan With-Project Conditions Input for Wave Overtopping Damage

6.6.2.1 5e Medium Plan Reach 1 With-Project Conditions Input for Wave Overtopping Damage

The Reach 1 medium plan with-project wave overtopping flow rates are shown in Table B2- 24 below. The results show that the splash pad and supporting grade landward of the seawall can withstand a 10% average annual exceedance event for any sea level change without failure; and a 2% AAEP event for current sea levels. The results also predict failure of the splash pad and supporting grade landward of the seawall, potentially causing complete failure of the wall for a 2% AAEP event for 3 ft. or more of sea level change; and for a 1% AAEP event with current sea levels.

Table B2- 24: 5e Medium Plan Reach 1 With-Project Flowrates								
Average Annual Exceedence Probability in %	Return Period in Years	Medium Plan Overtopping Flowrate at 2' NAVD in l/s/m for 0' SLC	Medium Plan Overtopping Flowrate at 2' NAVD in l/s/m for 0.5' SLC	Medium Plan Overtopping Flowrate at 2' NAVD in l/s/m for 1' SLC	Medium Plan Overtopping Flowrate at 0' NAVD in l/s/m for 1.5' SLC	Medium Plan Overtopping Flowrate at 0' NAVD in l/s/m for 2' SLC	Medium Plan Overtopping Flowrate at 0' NAVD in l/s/m for 2.5' SLC	Medium Plan Overtopping Flowrate in at 0' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	0	0
20%	5	0	0	0	0	1	3	6
10%	10	0	0	1	3	11	21	40
5%	20	2	4	8	20	59	103	180
2%	50	29	51	89	170	400	644	1030
1%	100	205	333	537	898	1816	2763	4186
1%	200	1245	1902	2885	4354	7818	11336	16459
0%	500	8263	11870	17005	23227	37653	52165	72189

6.6.2.2 5e Medium Plan Reach 2 With-Project Conditions Input for Wave Overtopping Damage

The Reach 2 medium plan with-project wave overtopping flow rates are shown in Table B2- 25 below. The results show that the splash pad and supporting grade landward of the seawall can withstand a 20% average annual exceedance event for up to 3 ft. of sea level change without failure; and a 2% AAEP event for current sea levels. The results also predict failure of the splash pad and supporting grade landward of the seawall, potentially causing complete failure of the wall for a 5% AAEP event for 3 ft. or more of sea level change; and for a 1% AAEP event with current sea levels.

Table B2- 25: 5e Medium Plan Reach 2 With-Project Flowrates								
Average Annual Exceedence Probability in %	Return Period in Years	Medium Plan Overtopping Flowrate at 0' NAVD in l/s/m for 0' SLC	Medium Plan Overtopping Flowrate at 0' NAVD in l/s/m for 0.5' SLC	Medium Plan Overtopping Flowrate at 0' NAVD in l/s/m for 1' SLC	Medium Plan Overtopping Flowrate at 2' NAVD in l/s/m for 1.5' SLC	Medium Plan Overtopping Flowrate at 2' NAVD in l/s/m for 2' SLC	Medium Plan Overtopping Flowrate at 2' NAVD in l/s/m for 2.5' SLC	Medium Plan Overtopping Flowrate in at 2' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	0	0
20%	5	0	0	0	0	3	6	12
10%	10	0	1	2	5	20	38	73
5%	20	4	8	16	35	102	181	317
2%	50	49	86	148	295	676	1087	1733
1%	100	315	510	819	1509	2977	4518	6830
0.5%	200	1782	2714	4111	7072	12444	17987	25964
0.2%	500	11037	15854	22708	36356	57839	79917	110274

6.6.2.3 5e Medium Plan Reach 3 With-Project Conditions Input for Wave Overtopping Damage

The Reach 3 medium plan with-project wave overtopping flow rates are shown in Table B2- 26 below. The results show that the splash pad and supporting grade landward of the seawall can withstand a 20% average annual exceedance event for any sea level change without failure; and a 5% AAEP event for current sea levels. The results also predict failure of the splash pad and supporting grade landward of the seawall, potentially causing complete failure of the wall for a 5% AAEP event for 3 ft. or more of sea level change; and for a 1% AAEP event with current levels of sea level.

Table B2- 26: 5e Medium Plan Reach 3 With-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Medium Plan Overtopping Flowrate at 2' NAVD in l/s/m for 0' SLC	Medium Plan Overtopping Flowrate at 2' NAVD in l/s/m for 0.5' SLC	Medium Plan Overtopping Flowrate at 2' NAVD in l/s/m for 1' SLC	Medium Plan Overtopping Flowrate at 3' NAVD in l/s/m for 1.5' SLC	Medium Plan Overtopping Flowrate at 3' NAVD in l/s/m for 2' SLC	Medium Plan Overtopping Flowrate at 3' NAVD in l/s/m for 2.5' SLC	Medium Plan Overtopping Flowrate in at 3' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	0	0
20%	5	0	0	0	0	2	5	12
10%	10	0	1	2	5	19	38	73
5%	20	4	8	15	35	104	184	322
2%	50	50	88	153	304	703	1125	1799
1%	100	333	539	867	1574	3114	4725	7138
0.5%	200	1919	2923	4425	7417	13084	18935	27332
0.2%	500	12051	17276	24706	38244	61195	84057	115766

6.6.2.4 5e Medium Plan Reach 4 With-Project Conditions Input for Wave Overtopping Damage

The Reach 4 medium plan waterfront with-project wave overtopping flow rates are shown in Table B2- 27 below. The results show that the splash pad and supporting grade landward of the seawall can withstand a 10% average annual exceedance event for 3 ft. of sea level change or less; and a 2% AAEP event for current sea levels without failure. The results also predict failure of the splash pad and supporting grade landward of the seawall, potentially causing complete failure of the wall for a 2% AAEP event for 2 ft. or more of sea level change; and for a 0.5% AAEP event with up to 0.5 ft. of sea level change.

Table B2- 27: 5e Medium Plan Reach 4 Waterfront With-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Medium Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 0' SLC	Medium Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 0.5' SLC	Medium Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 1' SLC	Medium Plan Overtopping Flowrate at 2' NAVD in l/s/m for 1.5' SLC	Medium Plan Overtopping Flowrate at 2' NAVD in l/s/m for 2' SLC	Medium Plan Overtopping Flowrate at 2' NAVD in l/s/m for 2.5' SLC	Medium Plan Overtopping Flowrate in at 2' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	0	0
20%	5	0	0	0	0	1	1	3
10%	10	0	0	0	1	5	11	20
5%	20	1	2	4	10	33	59	104
2%	50	17	30	51	104	258	418	669
1%	100	122	199	322	598	1260	1921	2917
0.5%	200	771	1176	1793	3089	5734	8328	12078
0.2%	500	5323	7673	11025	17568	29206	40463	55976

The Reach 4 medium plan tie-back with-project wave overtopping flow rates are shown in Table B2- 28 below. The results show that the splash pad and supporting grade landward of the seawall can withstand a 10% average annual exceedance event for 3 ft. of sea level change or less; and a 2% AAEP event for up to 0.5 ft. of sea level without failure. The results also predict failure of the splash pad and supporting grade landward of the seawall, potentially causing complete failure of the wall for a 2% AAEP event for 3 ft. or more of sea level change; and for a 0.5% AAEP event with current levels of sea level.

Table B2- 28: 5e Medium Plan Reach 4 Tie-Back With-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Medium Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 0' SLC	Medium Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 0.5' SLC	Medium Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 1' SLC	Medium Plan Overtopping Flowrate at 8' NAVD in l/s/m for 1.5' SLC	Medium Plan Overtopping Flowrate at 8' NAVD in l/s/m for 2' SLC	Medium Plan Overtopping Flowrate at 8' NAVD in l/s/m for 2.5' SLC	Medium Plan Overtopping Flowrate in at 8' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	0	0
20%	5	0	0	0	0	0	0	0
10%	10	0	0	0	0	0	1	9
5%	20	1	2	4	4	6	35	132
2%	50	17	30	51	89	321	616	976
1%	100	122	199	322	897	1990	2944	4367
0.5%	200	771	1176	1793	4730	8988	12797	18235
0.2%	500	5323	7673	11025	26950	45544	61920	83977

6.6.3 5e Large Plan With-Project Conditions Input for Wave Overtopping Damage

6.6.3.1 5e Large Plan Reach 1 With-Project Conditions Input for Wave Overtopping Damage

The Reach 1 large plan with-project wave overtopping flow rates are shown in Table B2- 29 below. The results show that the splash pad and existing grade landward of the seawall can withstand a 5% average annual exceedance event for any sea level change without failure; and a 1% AAEP event for current sea levels. The results also predict failure of the splash pad and supporting grade landward of the seawall, potentially causing complete failure of the wall for a 2% AAEP event for 3 ft. or more of sea level change; and for a 0.5% AAEP event with current sea levels.

Table B2- 29: 5e Large Plan Reach 1 With-Project Flowrates								
Average Annual Exceedence Probability in %	Return Period in Years	Large Plan Overtopping Flowrate at 2' NAVD in l/s/m for 0' SLC	Large Plan Overtopping Flowrate at 2' NAVD in l/s/m for 0.5' SLC	Large Plan Overtopping Flowrate at 2' NAVD in l/s/m for 1' SLC	Large Plan Overtopping Flowrate at 0' NAVD in l/s/m for 1.5' SLC	Large Plan Overtopping Flowrate at 0' NAVD in l/s/m for 2' SLC	Large Plan Overtopping Flowrate at 0' NAVD in l/s/m for 2.5' SLC	Large Plan Overtopping Flowrate in at 0' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	0	0
20%	5	0	0	0	0	0	0	1
10%	10	0	0	0	0	2	3	6
5%	20	0	1	1	3	11	20	35
2%	50	6	10	18	37	96	157	253
1%	100	48	78	128	228	509	780	1189
0.5%	200	340	525	799	1274	2481	3621	5276
0.2%	500	2664	3850	5544	7900	13647	18983	26364

6.6.3.2 5e Large Plan Reach 2 With-Project Conditions Input for Wave Overtopping Damage

The Reach 2 large plan with-project wave overtopping flow rates are shown in Table B2- 30 below. The results show that the splash pad and existing grade landward of the seawall can withstand a 5% average annual exceedance event for up to 3 ft. of sea level change without failure; and a 1% AAEP event for current sea levels. The results also predict failure of the splash pad and supporting grade landward of the seawall, potentially causing complete failure of the wall for a 2% AAEP event for 3 ft. or more of sea level change; and for a 0.5% AAEP event with current sea levels.

Table B2- 30: 5e Large Plan Reach 2 With-Project Flowrates								
Average Annual Exceedence Probability in %	Return Period in Years	Large Plan Overtopping Flowrate at 0' NAVD in l/s/m for 0' SLC	Large Plan Overtopping Flowrate at 0' NAVD in l/s/m for 0.5' SLC	Large Plan Overtopping Flowrate at 0' NAVD in l/s/m for 1' SLC	Large Plan Overtopping Flowrate at 2' NAVD in l/s/m for 1.5' SLC	Large Plan Overtopping Flowrate at 2' NAVD in l/s/m for 2' SLC	Large Plan Overtopping Flowrate at 2' NAVD in l/s/m for 2.5' SLC	Large Plan Overtopping Flowrate in at 2' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	0	0
20%	5	0	0	0	0	0	0	1
10%	10	0	0	0	0	1	3	6
5%	20	0	1	1	3	11	19	34
2%	50	6	10	18	37	99	161	258
1%	100	47	76	124	237	533	817	1246
0.5%	200	323	498	762	1350	2642	3856	5619
0.2%	500	2477	3585	5170	8489	14748	20486	28414

6.6.3.3 5e Large Plan Reach 3 With-Project Conditions Input for Wave Overtopping Damage

The Reach 3 large plan with-project wave overtopping flow rates are shown in Table B2- 31 below. The results show that the splash pad and existing grade landward of the seawall can withstand a 5% average annual exceedance event up to 3 ft. of sea level change without failure; and a 1% AAEP event for current sea levels. The results also predict failure of the splash pad and supporting grade landward of the seawall, potentially causing complete failure of the wall for a 2% AAEP event for 3 ft. or more of sea level change; and for a 0.5% AAEP event with current levels of sea level.

Table B2- 31: 5e Large Plan Reach 3 With-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Large Plan Overtopping Flowrate at 2' NAVD in l/s/m for 0' SLC	Large Plan Overtopping Flowrate at 2' NAVD in l/s/m for 0.5' SLC	Large Plan Overtopping Flowrate at 2' NAVD in l/s/m for 1' SLC	Large Plan Overtopping Flowrate at 3' NAVD in l/s/m for 1.5' SLC	Large Plan Overtopping Flowrate at 3' NAVD in l/s/m for 2' SLC	Large Plan Overtopping Flowrate at 3' NAVD in l/s/m for 2.5' SLC	Large Plan Overtopping Flowrate in at 3' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	0	0
20%	5	0	0	0	0	0	0	1
10%	10	0	0	0	0	1	3	5
5%	20	0	1	1	3	10	19	34
2%	50	6	10	18	37	101	163	264
1%	100	48	78	128	244	549	841	1289
0.5%	200	340	525	799	1402	2759	4008	5841
0.2%	500	2664	3850	5544	8873	15453	21440	29686

6.6.3.4 5e Large Plan Reach 4 With-Project Conditions Input for Wave Overtopping Damage

The Reach 4 large plan waterfront with-project wave overtopping flow rates are shown in Table B2- 32 below. The results show that the splash pad and existing grade landward of the seawall can withstand a 5% average annual exceedance event for up to 3 ft. of sea level change without failure; and a 1% AAEP event for current sea levels without failure. The results also predict failure of the splash pad and supporting grade landward of the seawall, potentially causing complete failure of the wall for a 2% AAEP event for 3 ft. or more of sea level change; and for a 0.5% AAEP event with current sea levels.

Table B2- 32: 5e Large Plan Reach 4 Waterfront With-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Large Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 0' SLC	Large Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 0.5' SLC	Large Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 1' SLC	Large Plan Overtopping Flowrate at 2' NAVD in l/s/m for 1.5' SLC	Large Plan Overtopping Flowrate at 2' NAVD in l/s/m for 2' SLC	Large Plan Overtopping Flowrate at 2' NAVD in l/s/m for 2.5' SLC	Large Plan Overtopping Flowrate in at 2' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	0	0
20%	5	0	0	0	0	0	0	1
10%	10	0	0	0	0	1	3	6
5%	20	0	1	1	3	11	19	34
2%	50	6	10	18	37	99	161	258
1%	100	47	77	125	237	533	817	1246
0.5%	200	328	502	769	1350	2642	3856	5619
0.2%	500	2516	3642	5252	8489	14748	20486	28414

The Reach 4 large plan tie-back with-project wave overtopping flow rates are shown in Table B2- 33 below. The results show that the splash pad and existing grade landward of the seawall can withstand a 5% AAEP event for 3 ft. of sea level change or less; and a 1% AAEP event for current sea levels without failure. The results also predict failure of the splash pad and supporting grade landward of the seawall, potentially causing complete failure of the wall for a 2% AAEP event for 2.5 ft. or more of sea level change; and for a 0.5% AAEP event with current levels of sea level.

Table B2- 33: 5e Large Plan Reach 4 Tie-Back With-Project Flowrates

Average Annual Exceedence Probability in %	Return Period in Years	Large Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 0' SLC	Large Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 0.5' SLC	Large Plan Overtopping Flowrate at 0.5' NAVD in l/s/m for 1' SLC	Large Plan Overtopping Flowrate at 8' NAVD in l/s/m for 1.5' SLC	Large Plan Overtopping Flowrate at 8' NAVD in l/s/m for 2' SLC	Large Plan Overtopping Flowrate at 8' NAVD in l/s/m for 2.5' SLC	Large Plan Overtopping Flowrate in at 8' NAVD l/s/m for 3' SLC
100%	1	0	0	0	0	0	0	0
50%	2	0	0	0	0	0	0	0
20%	5	0	0	0	0	0	0	0
10%	10	0	0	0	0	0	0	2
5%	20	0	1	1	0	1	9	39
2%	50	6	10	18	26	109	220	353
1%	100	47	77	125	332	799	1191	1778
0.5%	200	328	502	769	1975	3985	5715	8201
0.2%	500	2516	3642	5252	12646	22424	30579	41762

6.6.4 Critical Thresholds

Using 14.0 ft. NAVD88 as a typical line of protection elevation for illustration purposes, overtopping analyses have shown that a water surface elevation higher than 11.0 ft. NAVD88 results in unacceptable wave overtopping flowrates. Figure B2- 11 shows the water surface elevations under the three sea level change scenarios, and the 14.0 ft. NAVD88 crest elevation and the limit water surface elevation of 10.7 ft. NAVD88.

The figure shows that under the low sea level change scenario, the limit water surface elevation is reached in 2076. With the intermediate scenario, it is reached in 2051, and in the high scenario it is reached in 2031. At these points in time, the measures would require adaptation in order to perform as expected. Adaptation measures are discussed below.

Looking at it in a different way, a threshold water surface elevation of 11.0 ft. NAVD in 2026 has an annual chance of exceedence of 1.3%. This same 11.0 ft. NAVD88 water surface elevation, should it occur in 2076, would have a 1.8% annual chance of occurrence with under the Low sea level change scenario. This means that this water surface elevation would happen almost 1.5x as frequently in 2076 as it was in 2026. If Intermediate sea level change occurs, this same 11.0 ft. NAVD 88 water surface elevation would have an annual chance of exceedence of 2.9%, which is over 2x as frequently in 2076 than in 2026. Finally, if High sea level change occurs, a 11.0 ft. NAVD88 water surface elevation has an annual chance of exceedence of 12.8%. This is almost 10x more frequent in 2076 than in 2026. These relationships are shown in Figure B2-12.

6.6.5 Adaptation for Sea Level Change

The crest elevations and configurations were evaluated and designed using wave overtopping as the most significant coastal damage mechanism. In the example above, 11.4 ft. NAVD88 was the limit of water surface elevation for the +14.0 ft. NAVD88 (5e Large) Plan with zero feet of sea level change. This would be equivalent to 10.7 ft. NAVD88 in year 2076. Water surface elevations higher than this threshold result in unacceptable damage from wave overtopping on the landward side of the structure. The years in which this water surface is exceeded under the 3 sea level change scenarios is as follows: 2076 for low scenario, 2050 for intermediate scenario, and 2030 for high scenario. Slightly prior to these points in time, adaptation measures are to be undertaken, such that overtopping damage to landward side will be prevented. The most common adaptation is elevation of the crest elevations of the measures. At times, this may require a larger foundation or footing. This larger footing is anticipated, and included in the initial construction. And the measures selected in the proposed plan are able to be elevated. Elevation could be adding a certain extra height to a concrete wall with more concrete, keying it in to make a unified cross-section. It could be adding fill to a levee crest and side slope, such that the intended side slope is maintained with the higher crest elevation. More real estate may be required to contain the larger footprint. This is anticipated and obtained prior to initial construction in 2026, not at the time of the adaptation action.

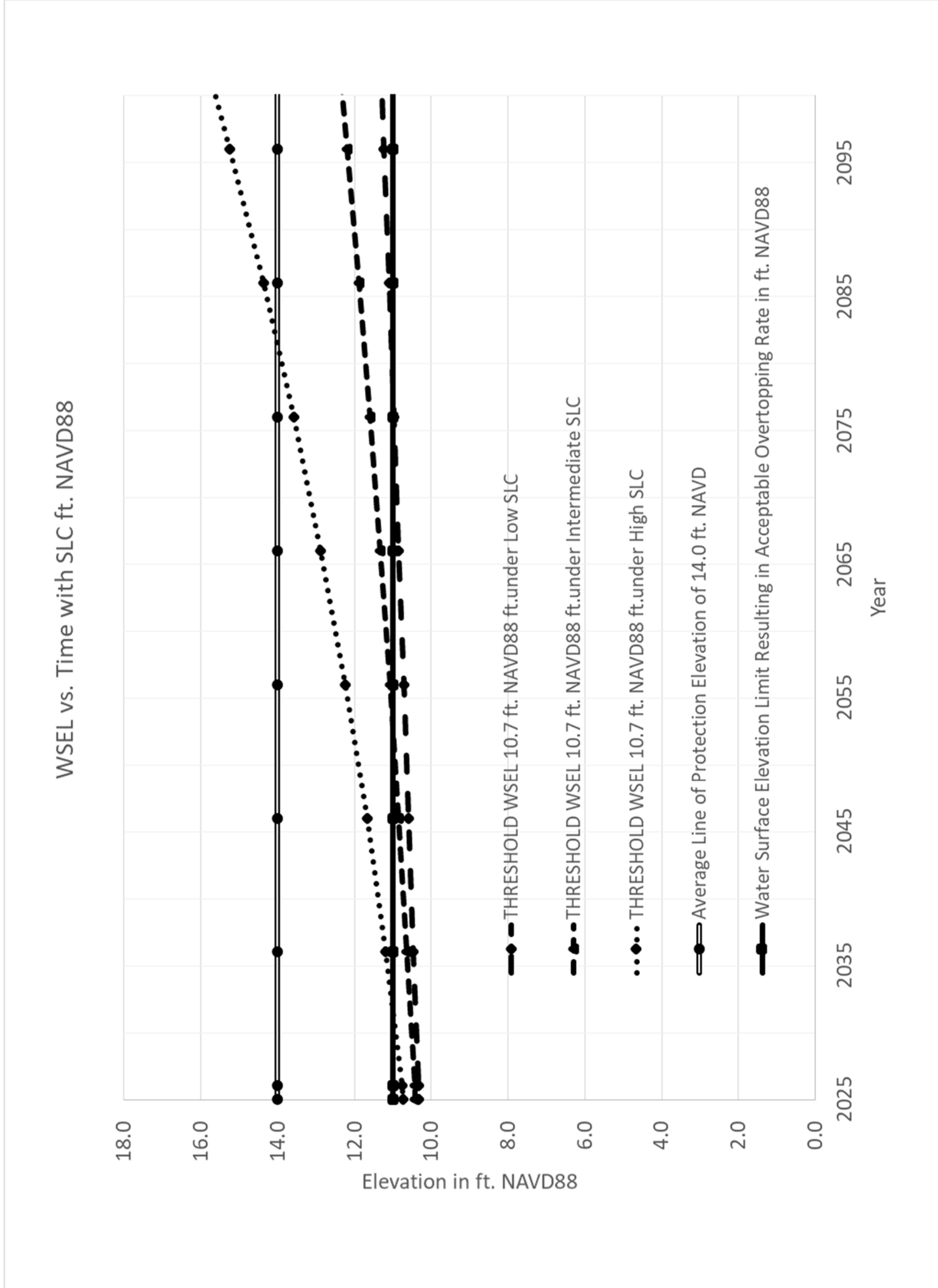

Figure B2- 11: Water Surface Elevation versus Time with Sea Level Change compared to Critical Threshold Elevation and Line of Protection Crest Elevation in ft. NAVD88

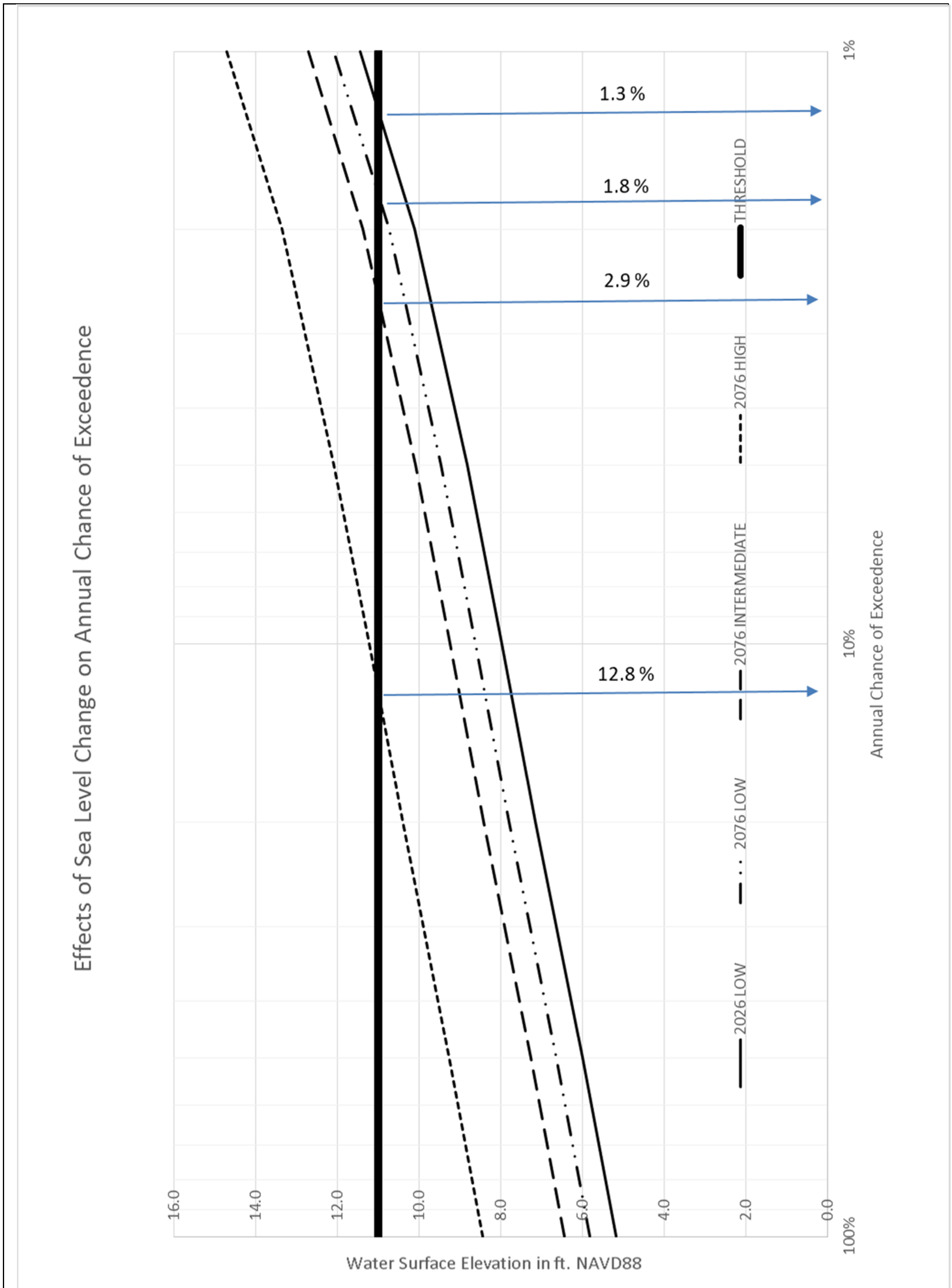

Figure B2- 12: Effects of Sea Level Change on Annual Chance of Exceedence