Appendix 1

Coastal Engineering

Appendix 1. Coastal Engineering Appendix

<u>Montauk Point Shore Protection</u>

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Contents

1.0 Introduction

The Water Management Section was tasked with reviewing the coastal engineering work included within the New York District's most recent Montauk Point Hurricane and Storm Damage Feasibility Report and the associated coastal engineering appendix to confirm/update that work. This appendix summarizes that review and provides the recommended revisions to the previous work/results from the New York District.

2.0 Background

Montauk Point is the eastern most tip of Long Island as shown in Figure 1. As many coastal/shoreline point features, Montauk Point experiences heavy erosion due to a lack of sand source feeding the beach, wave energy exposure and focusing, and a relatively steep bathymetry. As such, the New York District has shown significant erosion of the point shoreline and backing bluffs through historic shoreline/bluff face analysis. The bluffs reach up to a height of approximately 60 feet. Atop the bluffs is the historical Montauk Point Light House (shown in cover figure). Due to the topography of the point, moving the light house landward was determined to be both destructive to historic resources and cost prohibitive. To prevent the loss of the lighthouse, shoreline erosion protection was deemed necessary. Numerous attempts have been made at protecting the point against shoreline erosion since the mid-1940's to various degrees of success. The New York District developed several alternatives to mitigate the shoreline erosion and ultimately recommended a revetment design that was approved by the North Atlantic Division.

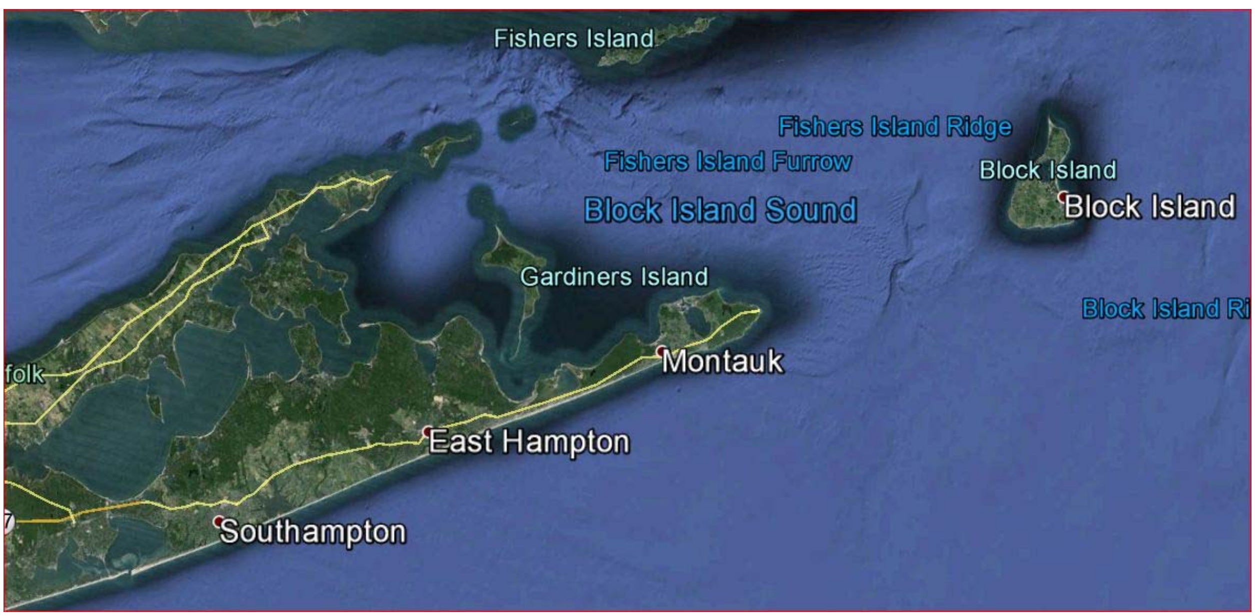

Figure 1. Montauk Point location map

3.0 Scope of Review

The Water Management Section developed a defined set of tasks as far as the review process and for any necessary design updates. The general scope of review has been provided below in a bulleted list.

a. Review the most recent Feasibility Report and Associated Appendices
b. Research the availability of the most recent survey work and LIDAR work
c. Review the return period water level analysis and compare to any more recent work
d. Review the wave analysis work and determine if more recent work is available
e. Investigate the impacts of sea level change (SLC)
f. Determine if the existing stone size is adequate based on revised forcing conditions
g. Determine if the existing revetment designs is adequate based on revised forcing conditions and constructability.

4.0 NOAA Benchmark Data

The Montauk Fort Pond Bay NOAA Benchmark is the nearest NOAA tidal benchmark and is located 5.5 miles west of the project site as shown in Figure 2. The benchmark also has a recording tide gage. The most recent benchmark data for Fort Pond has been provided below as Table 1. The data is from the 1983 to 2001 tidal epoch. The elevations have been converted to feet and the zero reference has been calculated for each of the datums. The work completed in the New District 2005 Feasibility report was referenced to NGVD29. The work done in this review effort was performed relative to the more recent NAVD88 datum. For this reason the conversion from NGVD29 to NAVD88 was included. The conversion was obtained from NOAA's Vertical Datum Transformation software package (version 3.2) and the conversion is included as Figure 3. The three datums of the most interest for this effort were highlighted in yellow in Table 1.

Figure 2. Montauk Fort Pond Bay NOAA Benchmark and Tide Gage Location

Table 1. NOAA Benchmark - Montauk, Fort Pond Bay

MONTAUK, FORT POND BAY	MLLW	MLW	MTL	MSL	NAVD88	NGVD29
	feet	feet	feet	feet	feet	feet
HIGHEST OBSERVED WATER LEVEL (08/31/1954)	8.44	8.27	7.23	7.20	6.87	7.81
MEAN HIGHER HIGH WATER (MHHW)	2.53	2.36	1.32	1.29	0.96	1.90
MEAN HIGH WATER (MHW)	2.24	2.07	1.03	1.00	0.67	1.62
NORTH AMERICAN VERTICAL DATUM-1988 (NAVD88)	1.57	1.40	0.36	0.33	0.00	0.94
MEAN SEA LEVEL (MSL)	1.24	1.07	0.03	0.00	-0.33	0.61
MEAN TIDE LEVEL (MTL)	1.21	1.04	0.00	-0.03	-0.36	0.58
NATIONAL GEODETIC VERTICAL DATAUM (NGVD29)	0.62	0.05	-0.58	-0.61	-0.94	0.00
MEAN LOW WATER (MLW)	0.17	0.00	-1.04	-1.07	-1.40	-0.45
MEAN LOWER LOW WATER (MLLW)	0.00	-0.17	-1.21	-1.24	-1.57	-0.62
LOWEST OBSERVED WATER LEVEL (02/02/1976)	-3.78	-3.95	-4.99	-5.02	-5.35	-4.41

LENGTH OF SERIES: 17 Years

TIME PERIOD: 1/1/1983-12/31/1992 ; 1/1/1994-12/31/2000

TIDAL EPOCH: 1983-2001

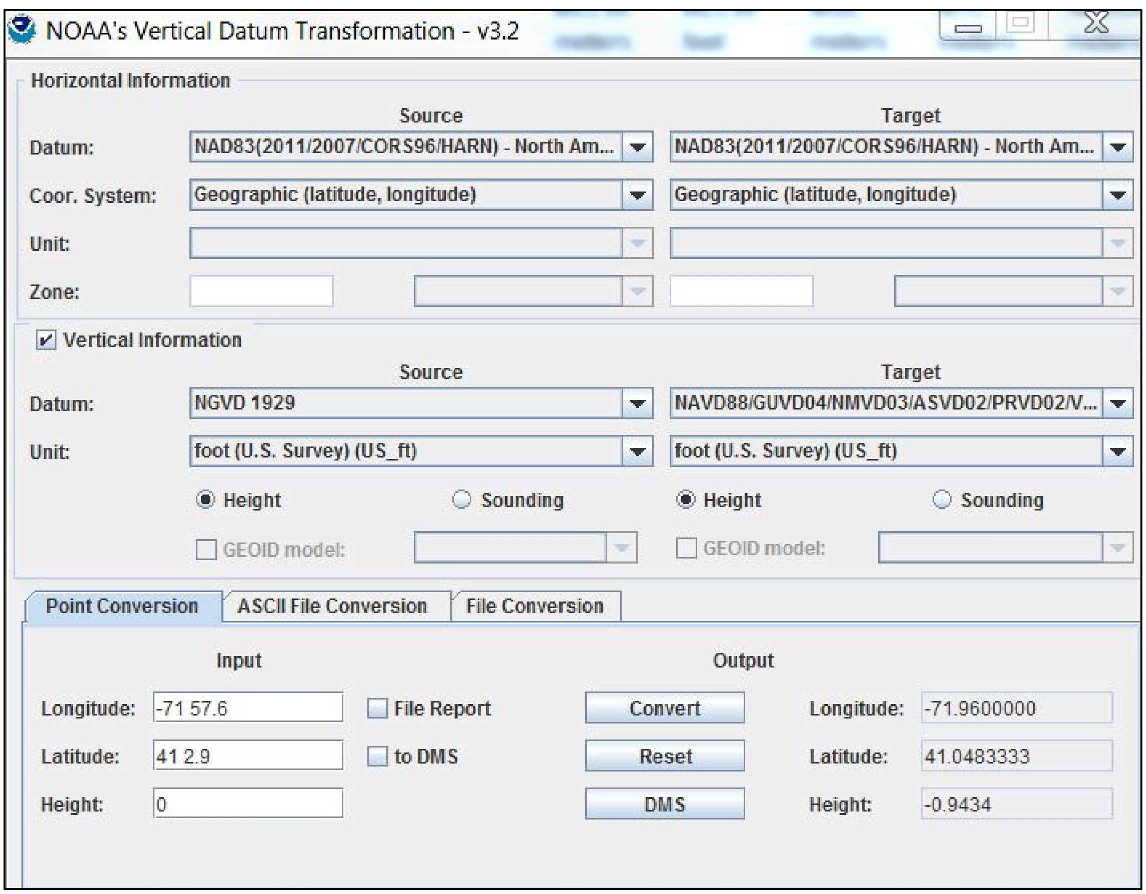

Figure 3. NOAA VDatum NGVD29 to NAVD88 worksheet

5.0 Design Storm Return Period Selection

The project is authorized at 73 year level of protection based on the economic and risk analysis performed in the 2005 Feasibility Study. The design guidance for designing the revetment alternatives used by the New York District in the 2005 Feasibility Study was EM 1110-2-1614 (30 June 1995) "Design of Coastal Revetments, Seawalls, and Bulkheads". Based on the EM guidance, a binomial distribution was used to establish a level of protection of 73 years. The Feasibility Study investigated designs with lower and higher return periods, and found that the 73-year storm protection level was identified as the NED Plan. This level of protection has been assumed as a constant for the Limited Re-Evaluation Report. For reference, the exact verbiage stated in the EM is provided below:

> "As a minimum, the design must successfully withstand conditions which have a 50 percent probability of being exceeded during the project's economic life. In addition, failure of the project during probable maximum conditions should not result in a catastrophe (i.e., loss of life or inordinate loss of money)."

6.0 Design Water Level

A critical component for coastal projects is quantifying the frequency of water surface elevations or more accurately the probability of experiencing a particular water level, or range of water levels, within given time period. Selecting the optimum water level for a design is often a complicated question since there are numerous factors that impact that selection process, especially when trying to incorporate risk. In the following sub sections of Section 5.0 the water level topic will be discussed.

6.1 New York District Water Level (2005 Feasibility Report)

The New York District developed the return period water levels at the Montauk Point Project site using the results from the Fire Island to Montauk Point Study (FIMP) and the assistance of the Coastal Hydraulics Lab (CHL) of the Engineering Research and Development Center (ERDC). The FIMP study was a multiple million dollar study conducted over many years that included a robust water level analysis. From this work, the NY District developed a table for the return period water levels at the project site. This table was included in the NY District 2005 feasibility study coastal engineering appendix as Table A6. It is provided below as Table 2.

Table 2. Return period water levels (reproduced from NY District)

Return Period (years)	Combined Storm Surge (Tropical plus Extratropical), NGVD feet	Combined Storm Surge + Astronomical MSL, NGVD feet	Wave Setup (from FIMP)	Storm Surge + Wave Setup + Astronomical MSL, NGVD feet	Utilized Storm Stage * + Wave Setup NGVD feet
5	4.76	5.20	2.72	7.92	8.10
10	5.34	5.78	2.88	8.66	8.69
25	6.14	6.58	3.19	9.77	9.52
50	6.73	7.17	3.42	10.59	10.34
100	7.33	7.77	3.57	11.34	11.51
500	10.29	10.73	3.88	14.61	14.51

In the following sections the validity of the water levels in Table 2 are checked given the occurrence of both Hurricanes Irene and Sandy since the feasibility report was completed.

6.2 Post Irene and Sandy Water Level Validity Check

As part of the review effort, the return period analysis for water level was reviewed as far a validity given the significant events that have occurred since the CHL work was completed. Since that time several significant storms have impacted the area including the most obvious one, Sandy. These events, when included in the probabilistic distribution, could skew the distribution which would then change the water level return period frequencies. This analysis was done at a cursory level due to the complication of project location vs. the location of the nearest long term recording tidal gage. As discussed in Section 4.0, the nearest long term NOAA tidal gage to Montauk, NY is the NOAA Montauk Fort Pond Bay Benchmark/Gage. The location of the Montauk gage is shown in Figure 2. The problem with the gage location is that it is not on the open coast, but instead is in relatively protected waters, away from the influences of wave set up, and from localized wind setup. What this means is that the return period statistics from the NOAA gage discussed below should be compared to the third column of Table 2 since that column excludes the localized wave setup.

Following Hurricane Sandy, as part of the North Atlantic Comprehensive Study, ERDC CHL developed revised return period statistics for all of the NOAA gages with historical data along the East Coast. This was summarized in the October 2013 draft report titled North Atlantic Coast Comprehensive Study Phase I: Statistical Analysis of Historical Extreme Water Levels with Sea Level Change written by Norberto C. Nadal-Caraballo and Jeffrey A. Melby. From this work a series of plots and tables were produced for return period water levels, surges, and SLR implications. The Plot showing the return period water levels for Montauk has been included below as Figure 4. As highlighted by the arrow, a return period water level of 73 years has been shown. For Montauk the value for a 73 year storm is 1.95 m-NAVD88 or 6.39 ft-NAVD88. A range of return periods has also been included for clarity as Table 3 which converts the m-NAVD88 values from the ERDC-CHL report to ft-NAVD88 and ft-NGVD29. When comparing the ft-NGVD29 values from Table 3 to the second column of Table 2, there is reasonable agreement between the 50 and 100 year storm events. The CHL work is 0.27 feet lower than the previous work for the 100 year storm and 0.39 feet lower than the previous work for the 50 year storm. This indicates that the work used in the NY District's 2005 feasibility study is still applicable and it could certainly be argued that the small difference between the two tables falls within the error of either analysis.

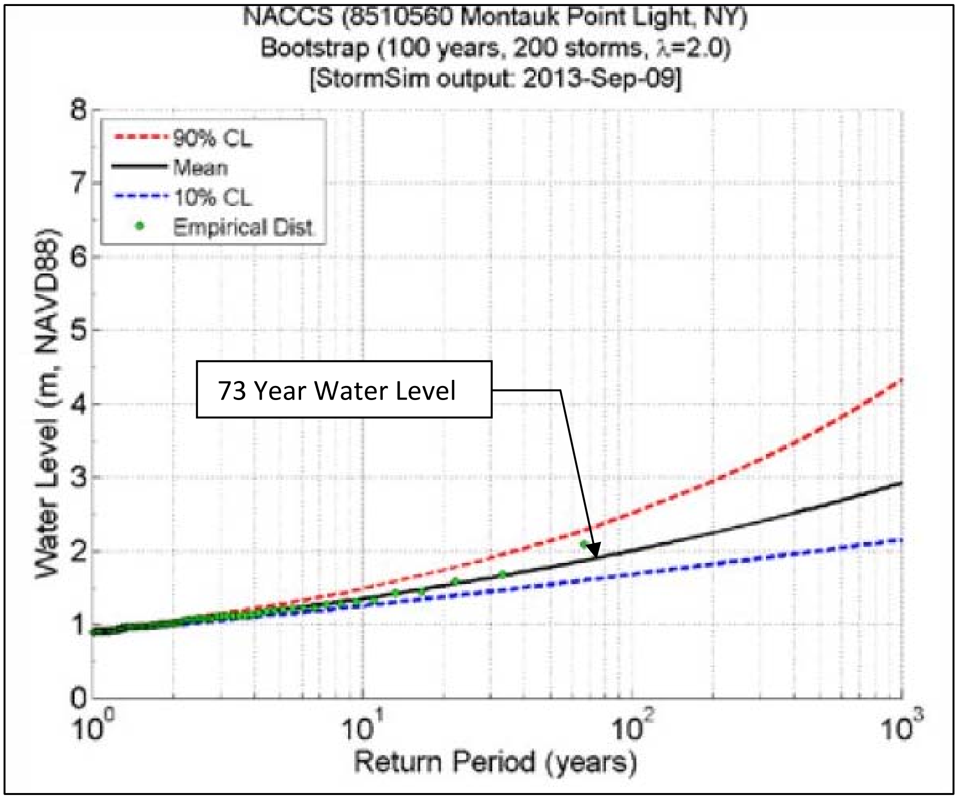

Figure 4. Montauk return period water levels

Table 3. Montauk Fort Pond Bay Benchmark CHL Return Periods

Montauk Return Period Water Levels (CHL Report Draft Report)			
Return Period (yrs)	Water Level (m-NAVD88)	Water Level (ft-NAVD88)	Water Level (ft-NGVD29)
1.00	0.89	2.92	3.86
10.00	1.35	4.43	5.37
25.00	1.58	5.18	6.12
50.00	1.78	5.84	6.78
100.00	2.00	6.56	7.50
500.00	2.60	8.53	9.47

Given that the underlying (surge and tides) water level return period frequencies developed in the earlier feasibility study were shown to still be valid and that the additional water elevation added to these elevations were derived through modeling, it was concluded that the overall water levels used in the previous study were likely valid. A check of the SBEACH modeling that was used to determine the localized wave setup was not performed in this effort since it was assumed this was done during the review process of the earlier feasibility study. To further validate the return period water levels in the sixth column of Table 2, for the outer coast/surf exposed coast, a look at water levels recorded during Sandy were used. Pre-Sandy, the USGS deployed numerous water level gages along the coast to record storm water levels at a fairly refined level along much of the Southern New England/Mid Atlantic Coast. As shown in Figure 5 there were numerous gages along the coast of Long Island. The nearest operable and recovered gage on Long Island to the project site was in East Hampton, NY with the location shown in Figure 6.

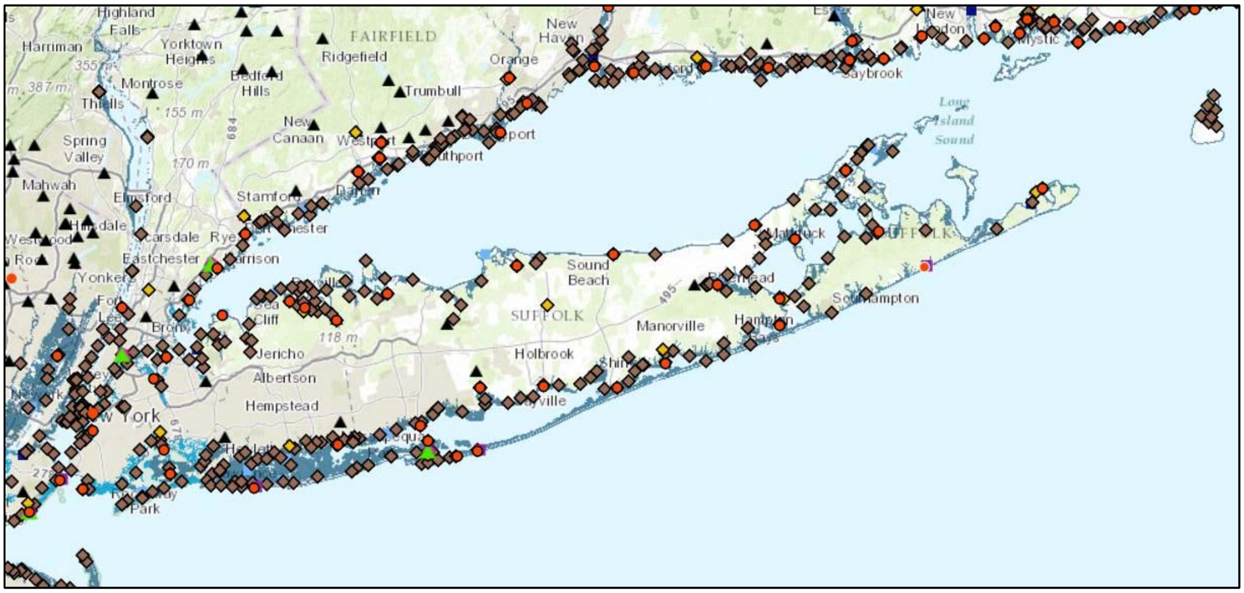

Figure 5. USGS temporary water level gages and high water marks

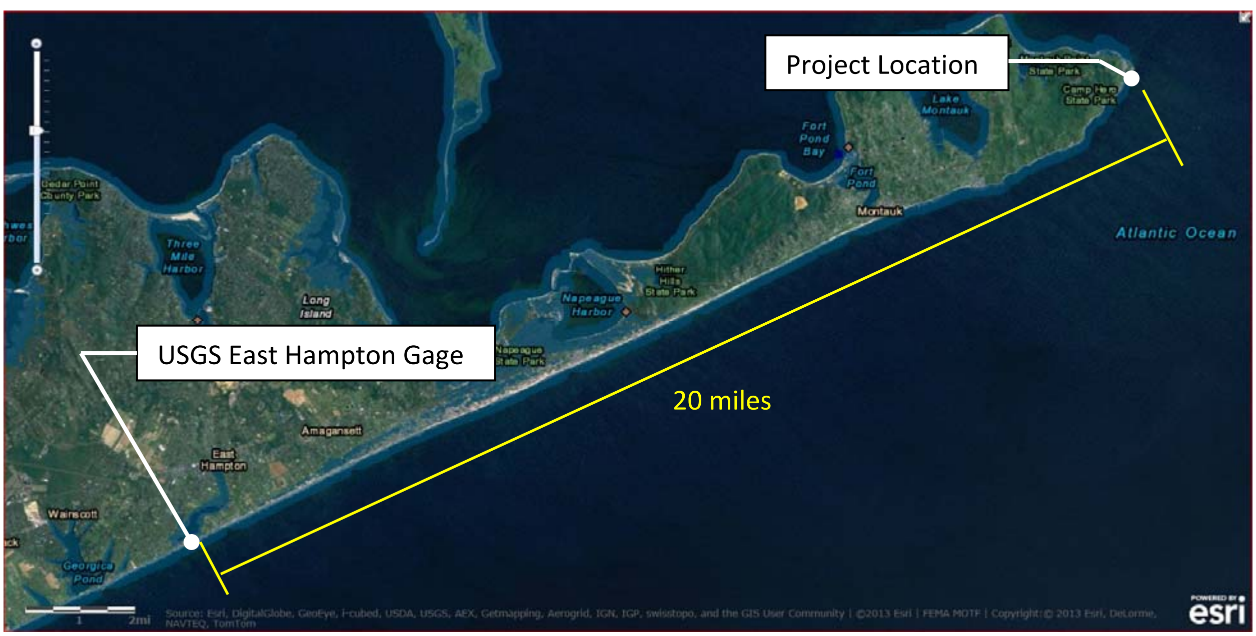

Figure 6. East Hampton, NY USGS temporary tide gage location

This gage was on the open coast and therefore experienced the full rise in water level (unlike the Montauk Fort Pond Bay NOAA tide gage). The recorded data from this gage is shown as Figure 7. As shown there are numerous, very short term/instantaneous spikes in the water level likely due to individual waves or wave sets. These spikes were ignored, and the more consistent upper bound of the recorded water level was used to determine the recorded water levels. This is shown as a solid line in Figure 7 which was drawn by eye with the peak of that solid line highlighted by a horizontal fine black line. The peak water level was 11.2 ft-NAVD88 or 12.14 ft-NGVD29.

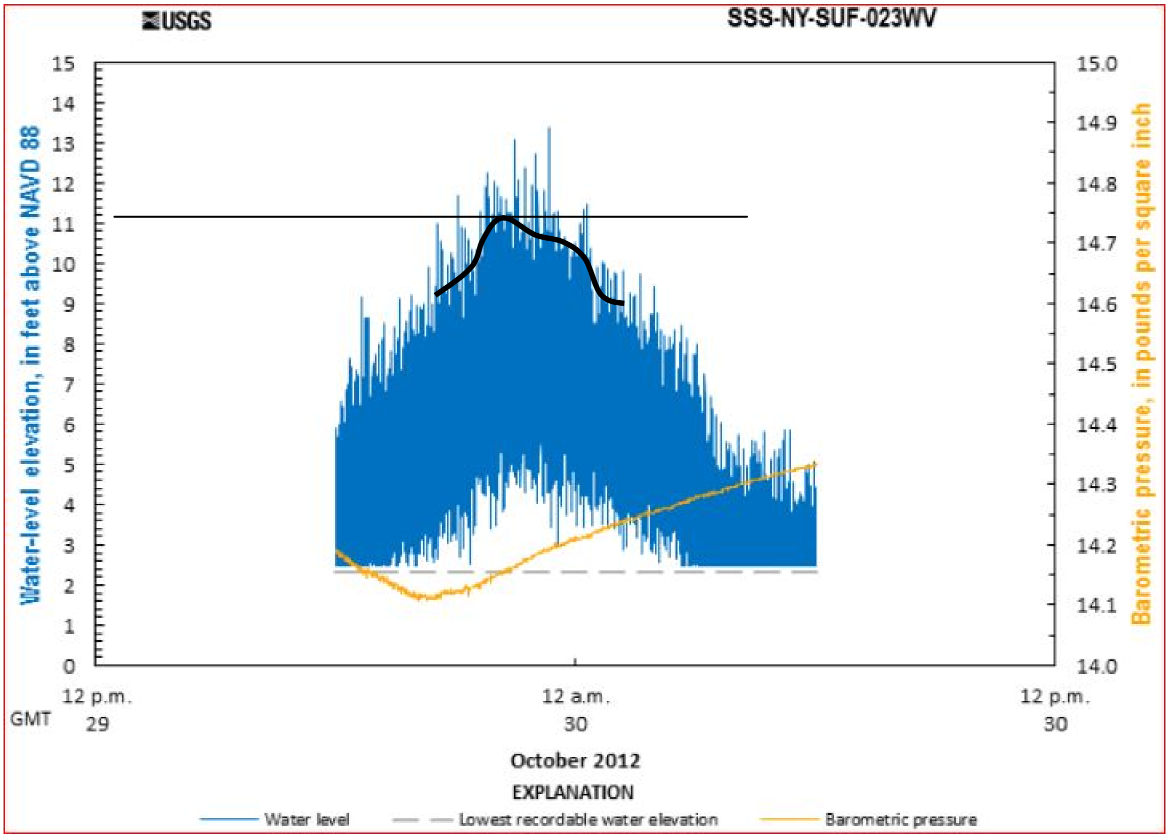

Figure 7. East Hampton, NY USGS Temporary Tide Gate - recorded data

When comparing this value to the values shown in Table 2, this equates to a return period of just over 100 years. Following Hurricane Sandy, Norberto C. Nadal-Caraballo and Jeffrey A. Melby performed an analysis similar to the previous draft study cited earlier with the intent of determining what return period storm Hurricane Sandy was for areas along the coast. As shown in Figures 8 and 9, Nadal and Melby determined that Hurricane Sandy was approximately an 80 year storm for total water level at the Montauk Fort Pond Bay NOAA tidal gage station and a 252 year storm for surge. Based upon that, one could classify Hurricane Sandy as a fairly low probability event with a return period greater than 80 years and for the open coast perhaps greater than a 100 level event for total water level given potential differences in the surge and astronomical tide phasing. This provides some level of confidence that the statistics used and the return period water levels used on the open coast are reasonable and are applicable to this project. Based on the previous discussion, it was concluded that at the time of this report development, the return period water levels used by the New York District in the 2005 feasibility report, and as provided in Table 2 of this appendix are still applicable. At the time of writing this review there was a significant modeling effort for the entire NAD region that will develop updated storm statistics along the Northeast Coast but that information will not be available for at least 1 year from the time of writing this report.

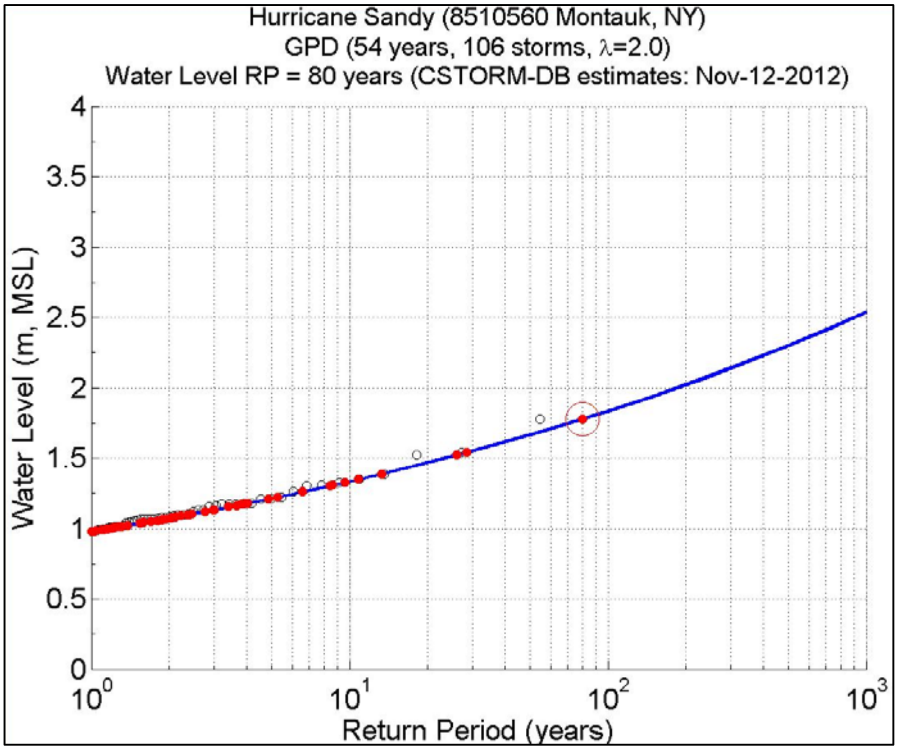

Figure 8. Hurricane Sandy Water level Return Period (Montauk Fort Pond Bay NOAA Tide Station)

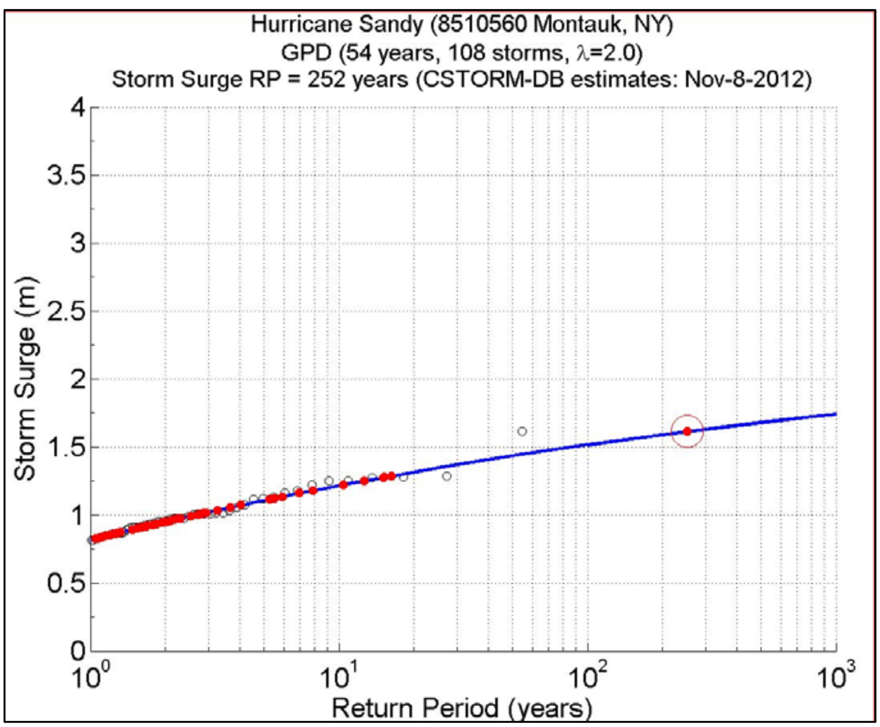

Figure 9. Montauk Fort Pond Bay NOAA Tide Station surge return period

6.3 Sea Level Change

In the 2005 analysis, sea level change (SLC) was included in the structure design analysis and operation and maintence analysis. The 2005 analysis only considered the historic rate of SLC and the impacts of SLC were essentially determined to be insignificant. The conclusion that the historic rate of SLC at the project location would not be significant is not surprising since over a 50 year project life, the sea level increase under the historic curve was only 0.70 feet. Since 2005, the inclusion of a more robust SLC analysis has become a closely scrutinized item for inclusion in all USACE coastal studies. As such the latest SLC guidance was included in this review and additional scrutiny was given to the impacts of the increased forcing conditions.

The latest guidance from the Corps for SLC is ER 1100-2-8162. Basically the guidance dictates that for all Corps projects three sea level change scenarios must be analyzed for impacts to the projects and the projects alternatives. The lowest curve is the historic rate of SLC at the site and the other two being higher rate curves designated by the NRC. The ER does not dictate which curve must be used for decision making but instead it is up to the PDT to determine the impacts and to try to mitigate the risk that the higher curves pose to the project in a practicable fashion. This analysis could range to the extreme end, where under the higher curves the project goals are not achievable and consideration of whether to move forward on a project must be given. Conversely, it could be determined that the project alternatives are flexible enough and adjustments to the project can be made to account for SLC through the regular O&M schedule i.e. adding more sand to a beach fill project or raising the elevation of a levee.

Since SLC varies along the coasts of the world and the USA, the rate of SLC for each general area must be determined. The ER specifies how this is to be done and it is fairly straightforward. The Corps' Institute of Water Resources (IWR) has developed an online worksheet and graph plotting website to make this determination even easier. The address of this web tool is http://www.corpsclimate.us/ccaceslcurves.cfm and the SLC results for the project area have been shown in Figure 10 and Table 4. A 50 year project economic life is shown with a construction time period of 2014. This may shift a year or two into the future but that will have a small impact on the SLC numbers presented.

As required by the ER, all three curves of SLC were investigated for impacts on the project site and the alternative design. The two key parameters impacted by increased water elevation for the project/alternatives are the stable stone size on the revetment due to larger wave heights impacting the revetment (depth limited waves) and increased run-up elevation/overtopping volume due to higher water levels and larger waves. As shown in Figure 10 and Table 4, the higher SLC curves result in noticeable higher sea levels over the 50 year project economic life when compared to the historic rate. The analysis related to SLC will be provided under the appropriate structure design sections (Sections 7 and 8).

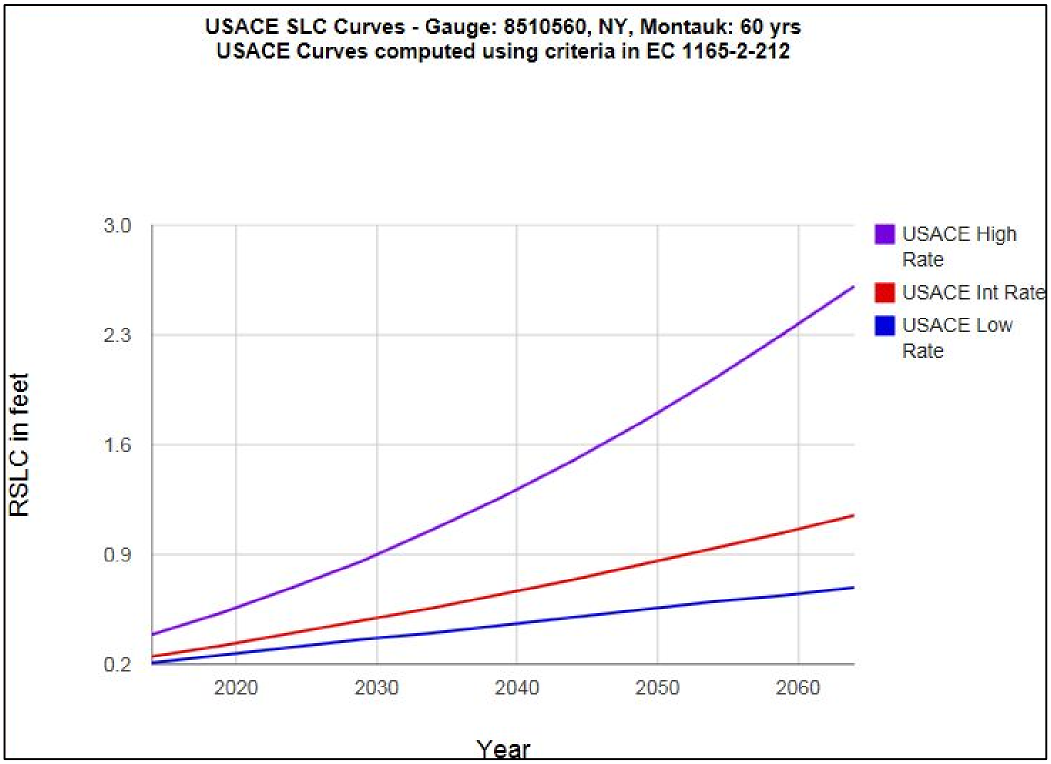

Figure 10. Sea Level Change Curves - Montauk, NY

Table 4. Sea Level Change values for Montauk, NY

Gauge NY, Montauk: 60 yrs All values are in feet			
Year	USACE Low	USACE Int	USACE High
2014	0.21	0.25	0.39
2019	0.26	0.32	0.53
2024	0.31	0.40	0.69
2029	0.36	0.48	0.86
2034	0.40	0.56	1.06
2039	0.45	0.65	1.27
2044	0.50	0.74	1.50
2049	0.55	0.84	1.75
2054	0.60	0.94	2.02
2059	0.64	1.04	2.31
2064	0.69	1.15	2.61

6.4 Shoreline Erosion – Water Depth Impact

As shown in the NY District's Feasibility Report from 2005, Appendix A, Section 5, the beach system in front of the bluff has eroded over the recorded history of the site. The long term beach erosion average was shown to be 2.2 feet per year, while in the more recent 50 year time period the beach erosion rate was 1.8 feet per year. Given the level of protection being proposed for this project, it is assumed that the horizontal beach erosion rate will be slowed dramatically and most likely halted. However, littoral transport of beach material seaward of the proposed revetment will still occur in the long shore and cross shore direction. This will result in a continued vertical erosion of the beach/bathymetry seaward of the revetment. Based on the approximated beach slope shown in the NY District's Feasibility Report, Appendix A of the fronting beach has a slope of 1V:40H. Considering the erosion rate of 1.8 feet per year, over a 50 year project life the beach would be expected to erode 90 feet horizontally, which translates into 2.25 feet of anticipated vertical erosion of the bathymetry seaward of the proposed revetment when considering the 1V:40H beach slope. This vertical erosion may be overstated since it is well documented that there is a significant amount of cobble, gravel, and existing armor stone in the system that will likely help to reduce the vertical erosion rate directly in front of the proposed revetment. Over 25 years the vertical erosion would be one half of the 50 year change and would be 1.125 feet or with rounding 1 foot. This vertical erosion was considered in Sections 7 and 8 during the design of the revetment.

7.0 Design Wave Height

The design wave height selected during this effort was chosen by first reviewing the work included in the NY District Feasibility Report. Similar to the water level information in that report, the wave height development and process was fairly robust and was reasonable in both the approach and the results provided. An important factor highlighted in the NY Report was that the waves impacting the revetment are depth limited, which means the wave heights are controlled by the available water depth in front of the structure. Two basic design wave height conditions were utilized in the revetment design with the first being the design wave height for the stable armor stone size and the second for the overtopping volume calculations. The two wave heights are discussed in Sections 7.1 and 7.2 respectively.

7.1 Design Wave Height – Stable Stone Size

One of the most widely used stone stability formulas used for rubble mound structures is the Hudson Formula. The formula is a deterministic formula that utilizes armor type, slope, and a design wave height to determine the stable armor weight. The equation is provided below as Equation 1.

$$W_{50} = \frac{w_r H^3}{K_D (Sr-1)^3 \cot\alpha} \qquad \text{Equation 1}$$

where

W_{50} = median weight of armor stone
w_r = unit weight of rock
H^3 = design wave height
K_d = stability coefficient
S_r = specific gravity of armor unit
α = angle of structure slope

This formula was used in the NY District Feasibility Report along with the depth limited wave determination and the breaking wave formulation provided in the Corps 1984 Shore Protection Manual. After a review of this work it was concluded that the formulation of the design wave height was reasonable. The results of the design wave height versus return period storm/water level are provided in Table 5 which is Table A-8 from the NY District Feasibility Report. As mentioned in the 2005 report, and as back calculated from the table below, the design wave height is essentially a factor of 0.9 times the water depth in front of the structure. The water depth was determined by adding return period water elevation to the elevation of the bottom in front of the revetment. The bottom was measured in the available survey data as 4 feet of depth below the NGVD datum. This formulation is shown in detail in the NAN Feasibility Report. For a 73 year event the design water depth was 14.94 feet and therefore the design wave height was 13.4 feet.

Table 5. Return period breaking wave heights and periods (taken from NY Feasibility Report)

Return Period (yrs)	Offshore Significant Wave Height (ft)	Storm Stage (ft, NGVD)	Storm Stage plus Wave Setup (ft, NGVD)	Design Breaking Wave Height (ft) at Revetment Toe (ft) (-4' NGVD)	Wave Period (s)
2	17.13	4.53	7.07	10.1	13.00
5	20.57	5.38	8.10	10.9	13.15
10	21.03	5.81	8.69	11.4	14.48
25	21.56	6.33	9.52	12.2	16.13
44	21.99	6.77	10.16	12.8	17.10
50	22.11	6.92	10.34	12.9	17.37
73	22.49	7.42	10.94	13.4	18.11
100	22.83	7.94	11.51	13.9	18.66
150	23.26	8.63	12.31	14.6	19.44
200	23.62	9.12	12.86	15.1	20.04
500	24.70	10.63	14.51	16.5	22.23

As discussed in Section 6.3, the feasibility study did not consider the higher rates of SLC or potential deepening in front of the revetment due to long term erosion. Increased water depths due to accelerated SLC impacts the design wave heights since the wave heights for this project location are depth limited, which means as water depth increases, so does wave height, at factor 0.90. As shown in Equation 1, wave height is a cubed term in the equation so small changes in wave height will impact the stone weight significantly. With the possibility of accelerated SLC (Section 6.3) and deepening from erosion (Section 6.4) a very real possibility, it was necessary to test a range of increased water depths on the stable stone size. The results of increased water depths on the stable armor stone size are shown in Table 6. As shown, increases in design wave height significantly increase the required stable stone size. The selection process and discussion for the stable stone size is provided in Section 8.2.

Table 6. Wave height vs. Stable Stone Size

Design Wave Height (ft)	W_{50} (tons)
13.00	11.53
13.40	12.63
13.50	12.91
14.00	14.40
14.50	16.00
15.00	17.71
15.50	19.54
16.00	21.50
16.50	23.57
17.00	25.78
17.50	28.12
18.00	30.60
18.50	33.23

7.2 Design Wave Height – Overtopping Formulation

The design wave height for overtopping calculations was determined in a different fashion than for the stable armor stone size determination. Part of the reason is due to the formulas used for wave overtopping. Most overtopping formulas, including the ones used for this effort utilize H_{mo} or H_s at the toe of the revetment structure. This is a different parameter than for the stable stone size. The H_{mo} values were determined in the NY Feasibility report through the use of STWAVE and SBEACH. The results of those modeling efforts were provided in a summary table as Table A-3 in the NY Feasibility Report and is provided as Table 7 below. As shown the STWAVE model values are slightly larger than the SBEACH model values. For this effort a check was performed to determine if these values were realistic. As cited in the CEM, H_s is typically between 0.40 and 0.60 times the local water depth. The higher value is for areas with steeper shorelines where wave energy is not diminished through friction and breaking and the lower value is for areas that have shallow sloped, wide flat bottoms, where wave energy is dissipated through friction and wave breaking. Based on Figures of the shoreline from the NY District Report, an Hs/water depth factor of 0.50 selected. As discussed in Section 6.1 the 73 year return period water level is 10 ft-NAVD88. Based on drawings from the Civil Design section, a toe elevation of -4 ft-NAVD88 was assumed, resulting in a water depth of 14 feet. This water depth results in an H_s value of 7 feet at the toe of the revetment. This value is slightly larger than the NY Feasibility report values and was used in this effort since it was slightly more conservative.

Table 7. Significant wave heights (Hs) vs. return period (taken from NY 2005 Report)

Storm Return Period (years)	Wave Height at Toe (ft)	Wave Height at Toe (ft)	Local Wave Direction for Storms from E	Local Wave Direction for Storms from SSE
	SBEACH	STWAVE	Deg from due E	Deg from due E
2	4.36	3.27	+5	-22
5	4.82	4.75	+5	-27
10	5.05	5.19	+5	-25
25	5.41	5.86	+5	-26
50	5.77	6.35	+5	-27
72	6.05	6.55	+5	-27
100	6.40	6.80	+5	-27
500	7.87	8.77	+5	-29
(Note that wave directions are from STWAVE. At Turtle Cove, the wave directions are –12 deg for Easterly storms and –60 deg for South-Southeasterly storms)				

Similar to the stable stone size design wave height, increased water depth due to SLR and vertical erosion in front of the revetment structure was investigated. As with the stone size formulation a range of water depths were investigated with the first being a 2 foot increase in water depth through a maximum increase of 5 feet in water depth. This resulted in a range of future wave heights from 8 feet to 9.5 feet when using the 0.5 feet wave height depth ratio factor. The implications of these wave heights on overtopping will be discussed in Section 8.3.

8.0 Revetment Design

8.1 Cross Sectional Layout

The coastal engineering analysis considered numerous cross sectional layouts of the revetment structure. Working with the PDT, it was concluded that the best option would be to leave the existing structure in place and cover it with a new stone revetment which was comprised of larger armor stone installed at a shallower slope. Based on survey data the existing structure was fairly steep in numerous areas with the slope being close to 1V:1H.

Through several meetings with the PDT, numerous iterations were investigated. During the design process several design criteria and design parameters were developed and those have been included below.

1. The incorporation of a larger stone size than currently exists and as originally designed in the feasibility report. Stone size will be discussed in Section 8.2.
2. A shallower slope was investigated vs. the existing steep slope of 1V:1H to 1V:1.5H (discussed further in Section 8.2).
3. Construction considerations and equipment requirements related to larger armor stone size were an important consideration. Also the construction issues related to a buried toe, as in the original design, were considered.
4. Area of sub-tidal habit impacted was a major consideration and was an important factor in selecting the revetment slope.

5. Crest elevation of the revetment and the resulting overtopping volumes related to structure stability and upper bluff impact.

The resulting revetment developed is shown in Figure 11.

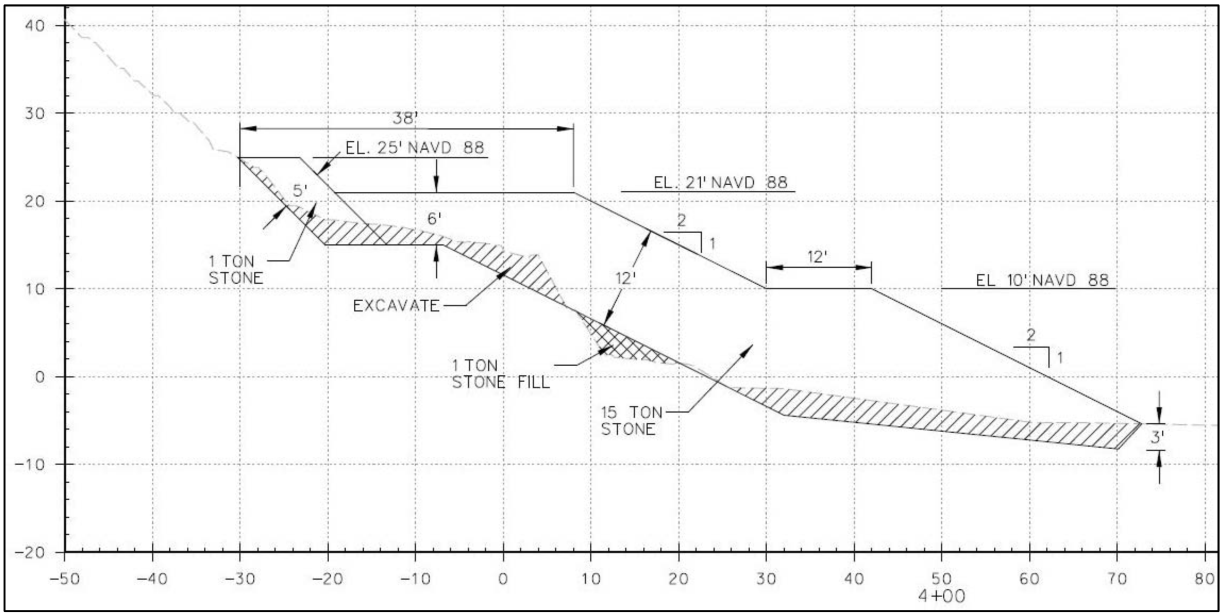

Figure 11. Revetment cross-section

The reasons behind the cross section selection are included in list form below.

1. The stone size was selected based on calculations and design consideration discussed in Section 8.2.
2. The crest elevation of 21 ft-NAVD88 was selected since it is the minimum elevation that could be achieved if the entire existing revetment was to be covered by the larger armor stone. The existing crest elevation is at elevation 15 ft-NAV88 and the selected armor stone size has a nominal diameter of 6 feet. Placing a 6 foot stone layer on top of the existing revetment crest results in a 21 ft-NAVD88 crest elevation.
3. The crest elevation of 21 ft-NAVD88 was found to reduce overtopping volume rates to an allowable level with that discussion provided in Section 8.3.
4. The toe berm at elevation 10 ft-NAVD88 was selected as a necessary feature for construction as well as providing additional protection of the structure toe. Due to the large stone size, it would have been difficult to construct the revetment from the top of the structure (elevation 21 ft) and to reach the toe. That would have been a reach of over 55 feet for land based construction. Through construction sequencing, starting with the toe berm at elevation 10 ft-NAVD88, the structure could be built with reasonable reach requirements for a crane. It was anticipated that the toe berm would be built first and then the upper part of the revetment will be built on top of the toe berm, partially covering the construction platform. The elevation of 10 ft-NAVD88 was selected due to the layer thickness of two 15 ton stones (12 feet) and the existing bottom elevation of -2 ft-NAVD88. It is understood that the toe berm crest elevation is only slightly

above the design water level elevation and therefore may be susceptible to damage due to wave attack. However, the toe berm is armored with an additional two layers of 15 ton stone to preclude damage. The toe berm elevation of 10 ft-NAVD88 was also considered acceptable for construction since there will be over 8 feet of freeboard between the construction (toe berm) platform and the MHHW tide level. This will provide reasonable protection against waves during construction.

5. A non buried toe was chosen since excavation of the existing armor stone relic structure, existing large natural stones, cobble and gravel, etc. was deemed to be very difficult. This likely would have required dewatering of the site and that too was deemed to be very difficult and costly. Given the large amount of natural rock at the project location downward scour and flanking of the revetment toe was considered to be a minimal risk.
6. Minimal excavation of the existing bottom is indicated since it is comprised largely of cobble, gravel, earlier revetment stone, glacial till, etc.
7. The revetment slope of 1V:2H was selected since it allowed the revised revetment design to nearly fit within the original NAN revetment footprint, allowed for a more stable slope, and reduced the stone size from that required for a steeper 1V:1.5H slope. A shallower slope would have been more stable but would have increased the structure footprint and buried more sub-tidal bottom. Covering more sub-tidal habitat would likely have required increased environmental coordination. The details of stone sizing and slope selection are provided in Section 8.2.

8.2 Stone Size Determination

As discussed in Section 7.0, the controlling factor for the design wave height was water depth in front of the structure. Under existing conditions the design water depth was determined in the NY District Report to be 14.94 feet which resulted in a design wave height 13.4 feet, which then resulted in a stable design stone weight of 12.6 tons. However, as discussed in Section 6, the water depths in front of the structure are anticipated to increase through the project life and therefore the design wave height is expected to increase through the project life. This in turn will result in larger stable stone weights for the same return period storm. Essentially there is not one design water level or wave height for the project, and the “design condition” is likely an ever increasing moving target through the project life. As discussed, in Section 6, there is not necessarily a right answer when selecting the design conditions. Should the design conditions be today’s conditions, at the mid-point of the project lift, at the end of the project life, etc? These decisions can result in structures being under designed for times later in a project life or being conservative for much or all of a projects life. Once again making the right decision is not typically a clear cut one and the decision is often reached through discussions with the PDT, local sponsor, giving consideration to residual risk, future cost for adjusting to SLR, what is being protected, the project purpose, etc. From a risk standpoint it does not necessarily make sense to design a revetment for the 50 year condition. As discussed that is a conservative design. If this project was protecting a large population center or a piece of critical infrastructure then a design to the very conservative limit may make sense, but for this project it was concluded by the PDT and the NY District that it does not.

Looking at Table 6 does reveal that as water depth, and therefore wave height increase, the stable armor stone size increases considerably. This means that picking the high curve of SLR rate and using the SLR increase at the end of the 50 year project life, would result in a very conservative and large stone size. Looking at Table 4 the SLC increase would be 2.61 feet over 50 years. If there was an additional 1 foot of depth from erosion, the increase in water depth would be 3.61 feet, and therefore a wave height increase of 3.25 feet. Adding that to the existing design wave height of 13.4 feet, results in a design wave height of 16.65 feet. From Table 6 that results in a stable stone size of nearly 25 tons. That stone size is nearly double the originally recommended stone size of 12.6 tons by the New District. This stone size would be conservative since the extreme water depth increases would not occur until the end of the project's economic life and therefore for all the years up to the last year, the design would be conservative relative to the 73 year design storm conditions. Conversely, using the stone size of 12.6 tons that was developed for the existing conditions would cause the project to be under designed for future years if the higher SLC rates occur as well as the vertical erosion of the bathymetry in front of the revetment.

An important consideration for selecting the stone size, that essentially became a controlling factor, was the stone size availability and the equipment necessary to transport and place the stone. Through conversation with the Geology and Cost Engineer PDT members it became apparent at the beginning of this analysis that anything over 18 tons likely would result in stone size availability issues and equipment/placement issues. This is not to say stones larger than this could not be acquired and placed, but the number of suppliers would become limited and the costs to the project would go up markedly due to higher stone production costs, transportation costs, and then placement costs.

Keeping the 18 ton stone threshold in mind, based on Table 6, that equates to just over a 15 foot design wave height, or essentially a water depth increase of 2 feet. Given the discussion related to vertical erosion (deepening due to erosion) and SLC in Section 6, 2 feet of water depth increase was considered likely. Vertical erosion in front of coastal structures in the surf zone is a well documented occurrence. Also, it has been measured at this project site for 150 years. SLC is also a process that is present and will happen at least to the historic rate and possibly, if not probably, at a higher rate over the project life. For a quick analysis the 2 feet of vertical change was split evenly between vertical erosion and SLC meaning 1 foot of change was attributed to vertical erosion and 1 foot of change was attributed to SLC. Based on the discussion in Section 6.4 1 foot of erosion would occur around year 25 of the project's economic life or the mid way point. At year 25, based on Table 4, due to the historic rate, the intermediate rate, or the high rate of SLC, water depth will have increased 0.45, 0.65, or 1.27 feet, respectively. That means for the first 25 years of the projects economic life (2039), for the design level 73 year storm, the project will be conservative if the historic or intermediate level of SLC occurs. This is because the structure will be designed for 2 feet of extra water depth, but only 1.65 feet will have occurred. For the high rate of SLR, at year 25, the increase of water depth is 1.27 ft, so total of 2.27 feet, which means under the high curve the design, is slightly below conservative by year 2039. Considering that information, a stable stone size of 18 tons was recommended from the coastal engineering analysis. It was concluded that it included adequate overbuild/conservatism to address erosion and SLR but not too much to where the structure was overbuilt for its intended purpose.

The final selected deign stone size however was selected by further discussions with the PDT and the NY District and it was decided that pushing the stone size to 18 tons would be too close to the previously discussed production and construction capability limits. Instead of the 18 ton design stone W_{50}, a 15 ton design W_{50} was selected. It was concluded by the PDT that it offered conservatism, but was less risk adverse from a constructability standpoint. The conclusion this was still conservative was based on the fact the current design assumes essentially a no-damage criteria, or 0-5% damage, per Table 7-9, pg 7-211 in the SPM and) site observations since the late 1990's that indicate toe scour has been a lesser contributor to damage to the existing structure than undersized armor, slope steepness, or overtopping.

8.3 Overtopping Rate

The overtopping rates were calculated in this effort using a different formulation than used during the NAN feasibility report effort. The overtopping formula and information used in this effort came from the Wave Overtopping of Sea Defenses and Related Structures: Assessment Manual, also known as the Eurotop Manual, which was a unified effort by various European research universities and agencies to develop a comprehensive manual for determining wave run up elevations and overtopping volumes/rates. The formulas and analysis included in this manual is the state of the art and considered the best available information. Many of the formulas used in the Eurotop manual were cited in the CEM or are early versions of equations and work cited in the CEM. Consideration was given to using a RANS type 1-D numerical model but budget and time limitations prevented that approach. The Corps CSHORE model was also considered but it has yet to be released in and was not on the approved model list for USACE use.

Within the Eurotop Manual there are several automated tools that allow for the calculations to be performed through a GUI/web based interface. This aided in the calculations of the overtopping rates. Also included in the Eurotop Manual are a neural network calculation option and a PCOvertopping option. Both allow for more accurate overtopping determinations than the deterministic empirical equation used but the neural network tool parameters did not include the range of this project's parameters so the tool was not applicable for this project design. The PC overtopping tool, was a web based tool and it was not functioning properly during the time period this analysis was being used.

Given the inability to use the more advance overtopping analyses methods, the more direct empirical/deterministic methods were used. To further simplify the analysis a simple uniform slope and crest was used and no refinement or reduction in overtopping associated with the toe berm was incorporated. This was done because during a design level event the water level is nearly at the top of the toe berm and it was concluded the reduction in overtopping would be minimal. This also added conservatism into the overtopping values determined. The equation used from the Eurotop Manual has been included below as Equation 2 (Equation 5.9 from Eurotop Manual) and an example worksheet from the online tool has been provided as Figure 12.

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}} \gamma_b \cdot \xi_{m-1,0} \cdot \exp\left(-4.3 \frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right)$$ Equation 2

$$\text{with a maximum of: } \frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp\left(-2.3 \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right)$$

Where:

q = overtopping rate
H_{mo} = equals wave height at structure toe
α = structure slope
ξ_{om} = breaker parameter based on s_{Om}
ϒ = correction factors
g = gravity

*Taken form Eurotop Manual

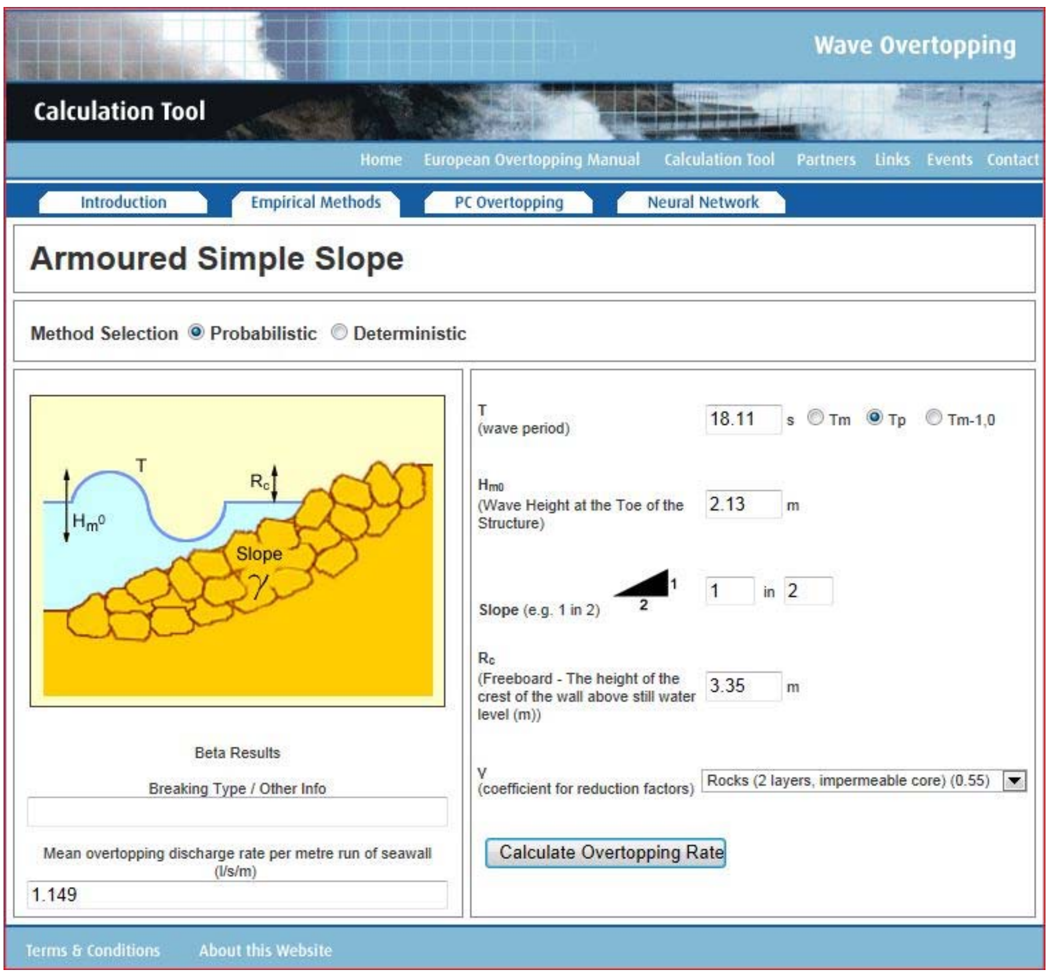

Figure 12. Eurotop calculation tool - simple rock revetment

Two overtopping reduction factors were considered during this analysis. The first was the increased roughness/porosity factor likely to be present in the proposed structure. Typically for a 2 layer rubble mound revetment, an impermeable/low permeability core is assumed and the resulting roughness/permeability factor used in the overtopping formula is 0.55 (taken from Table 6.2 of the Eurotop Manual). The factor is multiplied times the overtopping rate so the smaller the factor the greater the reduction for overtopping. For this structure, the new revetment will be placed over an existing, large armor stone, revetment. It was therefore reasoned the roughness/porosity would be increased and more significant reduction factor would be justified. Looking at Table 6.2 of the Eurotop Manual the reduction factor was 0.40. As stated this reduces the overtopping volume. A second factor that is expected to reduce overtopping is the wide crest width of the proposed revetment. With the new revetment being built over the existing revetment, an over widened crest results. Typically a revetment crest is 2 to 3 armor stones wide, and the overtopping formulas have that typical crest width built into them due to the physical model study parameters tested. For the armor stone selected the D_{n50} is approximately 6 feet, which results in a 3 stone crest width of 18 feet. The proposed crest width is 33 feet without the extra bluff protection (Figure 11) which is nearly double the typical width. Based on information in Section 6.32 of the Eurotop Manual, this should result in a significant reduction in overtopping. Based on the information provided in the manual a reduction level of 75% was selected or a reduction factor of 0.25. It is still believed that this is conservative based on the information provided in the manual. A more conservative number was selected since the manual uses a correlation of H_{mo} (H_s) and D_{n50} that is somewhat out of range for the proposed structure. Based purely on the relationships provided in the manual a reduction of 85% to 97% could have been used. It was concluded that this was likely too drastic a reduction and therefore a lesser crest width reduction was selected.

The overtopping rates are provided for three water level/depth conditions and for several combinations of the reduction factors discussed above. This was done so that the reader can see what the implications are for the various factors. The water level/depth increases presented include existing conditions (water level of 10' NAVD88, bottom at elevation of -4 ft-NAVD88, 7 ft H_s), the future water level condition used to recommend the stable stone size (water level of 11 ft-NAVD88, a bottom elevation of -5 ft-NAVD88, 8 ft Hs), and finally a more severe future conditions (water level of 12 ft-NAVD88, a bottom elevation of -6 ft-NAVD88, 9 ft Hs). For all cases the peak wave period used was 18.11 seconds which was taken from the New York District Report and Table 5 of this report. The results for each case and reduction factor are provided in Table () below.

Table 8. Overtopping rates for various water levels and reduction factors

	Overtopping Rates (liters/sec/meter)			
	Simple Slope	Crest Width	Extra Roughness	Extra Roughness and Crest Width
Existing Condtions	1.2	0.3	0.1	0.0
Future Condtions (reccomended SLR/vertical erosion)	6.5	1.6	0.7	0.2
Future Condtions (Max SLR/vertical erosion)	25.1	6.3	4.3	1.1

To place the above values into context Table VI-5-6 from the CEM was used as well as Table 3.5 from the Eurotop Manual. Those tables are provided below as Tables 9 and 10, respectively. Based on Table 8 above and Tables 9 and 10, under existing conditions the overtopping rates during a design level event, without any of the beneficial adjustment factors, are at the low end of the start of damage. The values in Table 9 used were under the categories of Embankment Seawalls and Revetments. When considering either or both of the adjustment factors there is no indication that overtopping will be a problem under existing conditions. For the future conditions with SLC and vertical erosion the overtopping does become problematic for the bluff face, especially for the "worst case" SLC/vertical erosion condition. However, with the beneficial adjustment factors applied individually, and especially together, the overtopping levels are below the start of damage threshold. Given this analysis it was concluded that overtopping would not be an issue to the bluff above the proposed 21 ft-NAVD88 crest elevation. However, based on discussion with the PDT and the NY District, it was decided that a safety factor against potential bluff impacts was desired. As a result of those discussions and a decision was made to include a short vertical bluff face stone protection component to elevation 25 ft-NAVD88. This is shown in Figure 11 and comprised of 5 foot thick layer of 1 ton stone.

Table 9. Allowable overtopping rates (taken from CEM Table VI-5-6)

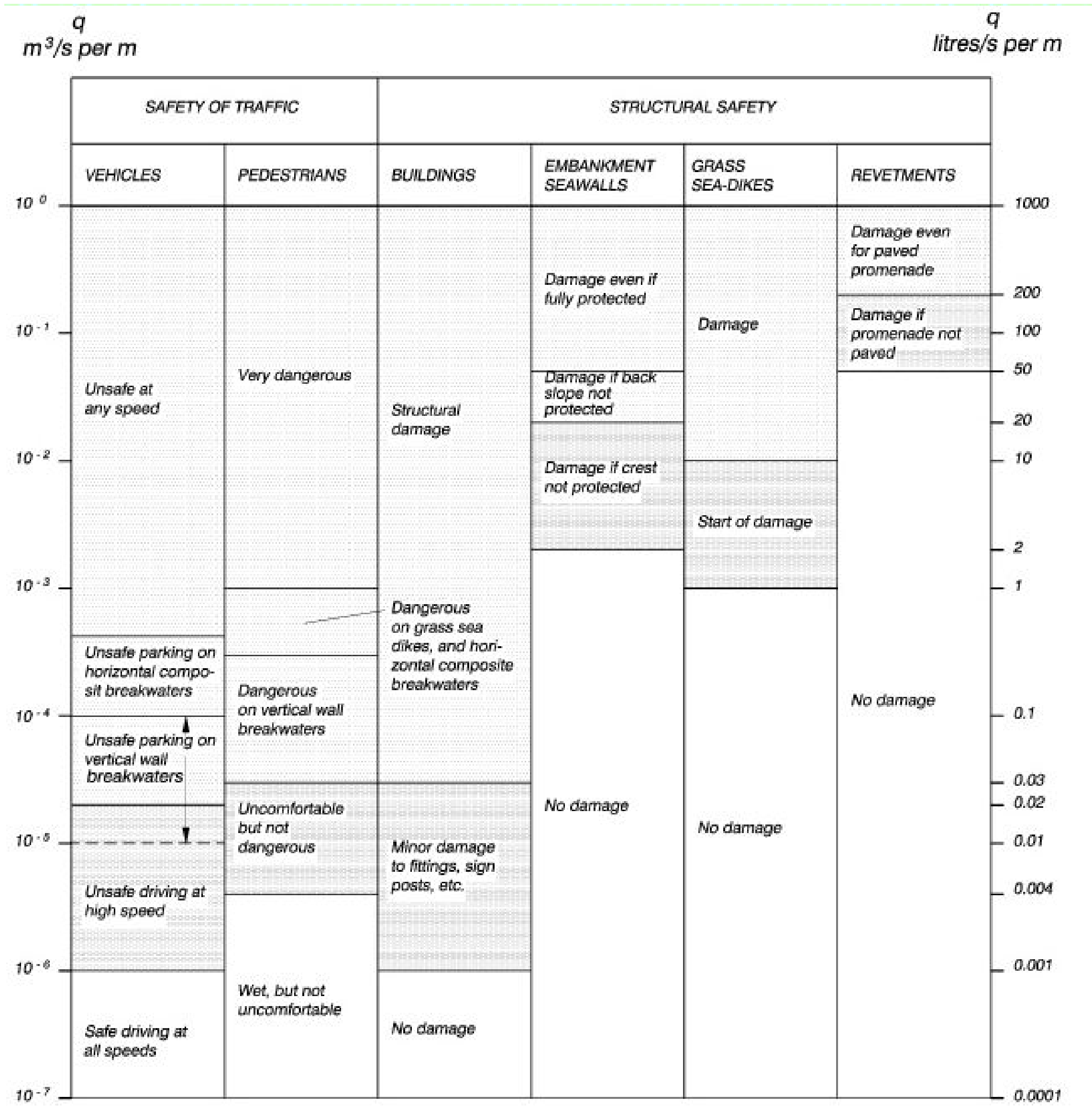

Table 10. Allowable overtopping rates (Eurotop Manual Table 3.5)

Hazard type and reason	Mean discharge q (l/s/m)
Embankment seawalls/sea dikes	
No damage if crest and rear slope are well protected	50–200
No damage to crest and rear face of grass covered embankment of clay	1–10
No damage to crest and rear face of embankment if not protected	0.1
Promenade or revetment seawalls	
Damage to paved or armoured promenade behind seawall	200
Damage to grassed or lightly protected promenade or reclamation cover	50

9.0 Surfing Impacts

In the New York District Feasibility study, the Surfrider Foundation, Eastern Long Island Chapter, raised concerns regarding the impact of the proposed project on recreational surfing. In response to the Surfrider Foundation's concerns, the Corps performed an analysis to determine the potential effect of the proposed project in the Feasibility Report on near shore breaking waves. The results of this analysis determined that the reflection coefficient for the existing revetment ranged from 0.30 to 0.33, whereas the reflection coefficient for the proposed revetment would range from 0.25 to 0.28, an approximate 15 percent reduction from that of the existing revetment.

This reduction was due to the milder front slope and the greater porosity of the thick layers of randomly placed stone of the proposed revetment. Based upon the modeling results, the Corps concluded that implementation of the 2005 proposed project would have little to no impact on the quality or surfability of the waves in the offshore waters of Montauk Point, and may, in fact, have less impact than the existing structure. The proposed structure in designed in this effort has the same slope and a similar foot print as the Feasibility Report structure, thus no impact is anticipated to surfing. A comparison of the currently proposed structure to the structure proposed in the Feasibility report is provided in as Figure 13.

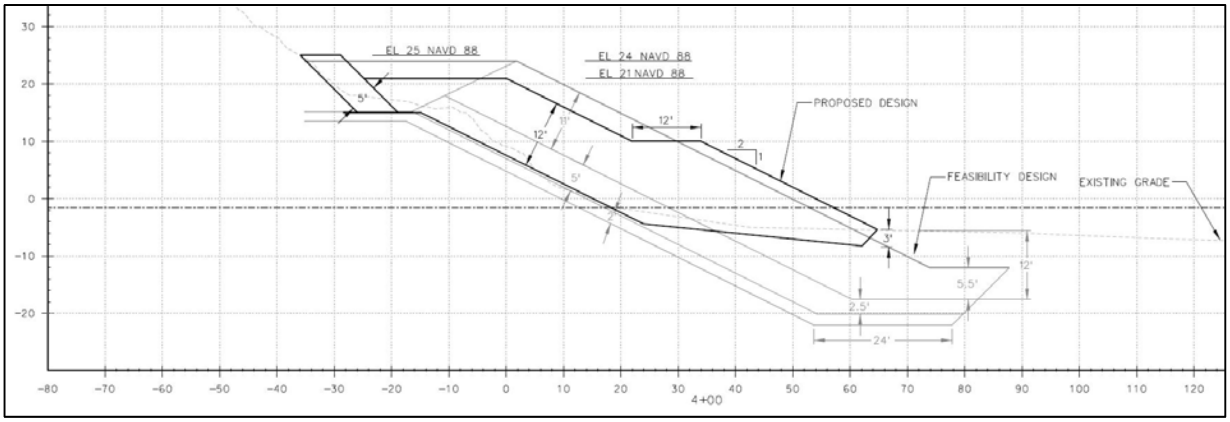

Figure 13. Comparison of proposed revetment to NY District Feasibility Report revetment

11.0 Summary and Conclusions

During the LRR effort, the proposed design from the New York District Feasibility Report was reviewed and updated where necessary. For the most part the original study was found to be adequate with the most significant changes resulting from a more robust SLC analysis and from constructability considerations. These factors resulted in a recommended larger stone size for the revetment and for a toe berm feature vs. a buried toe. As shown in Figure 13, the overall foot print of the revised structure is similar to the 2005 FS design, except for the large amount of excavation that would have been necessary for a buried toe. At Mean Low Low Water, the HSLRR design is approximately four feet further seaward than the FS design.

It was shown using the latest overtopping formulas that overtopping rates during a design level event should not be a problem for the bluff above the revetment. The bluff is very well vegetated and should be resistant to erosion from the overtopping that was calculated to reach the bluff. It is recommended that vegetation on slopes above the revetment be maintained throughout the project life. As a precaution, an extra four feet of structure elevation (21’-25’ NAVD88) was added on the bluff face to address any potential run up/overtopping issues.