Nantasket Beach
Hull, Massachusetts

Coastal Engineering Appendix

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1.0 Introduction

As part of the Nantasket Beach Section 103 Hurricane and Storm Damage Reduction Project, the Water Management Section performed a coastal engineering analysis related to beach erosion, beach fill design, and seawall failure. This effort was completed in conjunction with the Woods Hole Group (WHG) in Falmouth, MA since they were performing beach related analysis for the Department of Conservation and Recreation (DCR) as well. WHG invested a much larger set of alternatives than the ones being considered by USACE. The details of the information used from the WHG effort will be discussed in the appropriate sections of this report. A total of three different beach fill berm elevations were modeled in this report with varying berm widths. For the beach fill alternatives two different mean grain sizes were used (D_{n50} = 0.25 mm and D_{n50} = 0.45 mm). Additionally, protection to the sea wall and the study area afforded by two revetment designs was evaluated. The impacts of sea level rise (SLR) were considered for the no project condition, the revetment alternatives and the various beach fill alternatives.

2.0 Beach Geological History and Land-Ocean Interface Dynamics

2.1 Topography

Nantasket Beach is located in the Town of Hull, Plymouth County, Massachusetts, about 4 miles southeast of the main entrance to Boston Harbor and approximately 12 miles east-southeast of Boston on the southeast shore of Massachusetts Bay (Figure 1). It’s a 3-1/2-mile pocket beach positioned between the headlands known as Atlantic Hill on the southeastern mainland end of the beach and Allerton Hill, a former drumlin island, to the northwest. In geological terms Nantasket is known as a complex tombolo, which unites several former drumlin islands and the mainland (Johnson and Reed, 1910), although it is sometimes referred to as a barrier spit. The tombolo ranges between 500 ft and 2,800 ft in width, and has a beach facing the Atlantic Ocean to the northeast and the backside facing Hull Bay.

Several drumlin hills, including Hampton Hill, Sagamore Head, White Head, Strawberry Hill, Allerton Hill, Nantasket Hill at Hull also known as Telegraph Hill, and Thornbush Hill rise above the surrounding area, reaching elevations up to 100 ft or more above North American Vertical Datum of 1988 (NAVD88). The remainder of the Nantasket area is relatively flat and low-lying with elevations ranging from sea level to approximately 10 to 20 feet, NAVD88. Glaciation, coastal processes, and human development have been the primary influences on the area’s topography.

2.2 Coastal Geology

Coastal processes have played a major role in the formation of Nantasket Beach. Erosional forces exerted by tidal fluctuations and wave action together with a slowly varying sea level acted to erode exposed drumlins and provide littoral material (sand and gravel) that Johnson and Reed (1910) indicated formed a series of connecting beaches between adjacent drumlins or drumlins and the mainland. As such, the Nantasket barrier form likely evolved around a series of drumlins that served as anchor points. As the drumlins eroded, sand was contributed to the barrier system and eventually, when the glacial headland disappeared, the barrier moved rapidly onshore to another drumlin that could act as a pinning point. The work of Johnson and Reed (1910) suggest that Nantasket Beach has been an accretionary feature for most of its existence. The present shoreline is seaward of the wave-cut cliffs in several of the drumlins, further evidence that the Nantasket Beach essentially accreted in-place due to the erosion of former

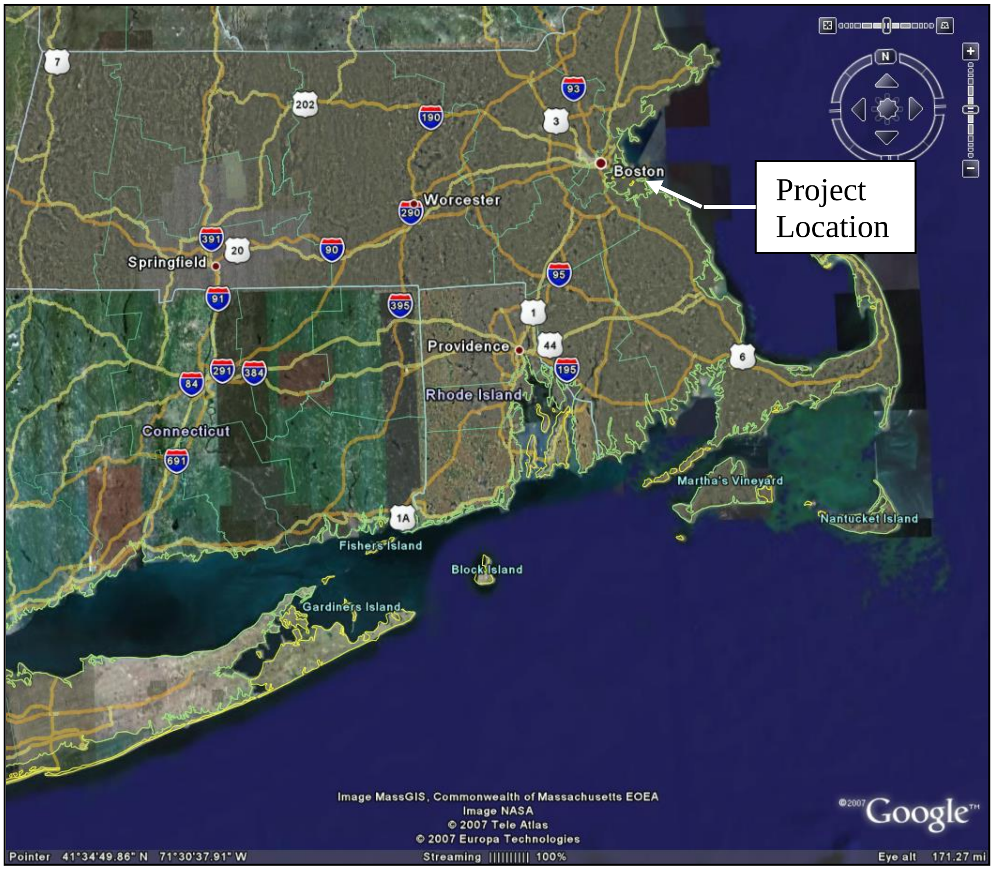


Figure 1. Study Location

drumlins to the east of the present beach. However, FitzGerald et. al. (1994) pointed out that the amount of sand available from drumlins is insufficient to account for the volume of the Nantasket barrier. They also note that the poorly sorted, immature sediment of the drumlins is also quite different from the fine, well-sorted sand that comprises much of the beach's material at Nantasket Beach. Consequently, they suggest that the sediment for the Nantasket barrier was probably derived from a number of intercepted drumlins and other glacial deposits, and reworked onshore late during the Holocene transgression. FitzGerald et. al. (1994) further noted that Nantasket Beach is adjacent to a major offshore sand deposit (FitzGerald et. al., 1990).

Nantasket Beach, therefore owes its existence to the erosion and redistribution of sediment from the existing drumlins (Allerton Hill, etc.) as well as the drumlin remnants offshore. There is good evidence that there were other drumlins present offshore to the east of Nantasket, based on the orientation of old beach ridges and wave cut cliffs on the remaining drumlins west of the present shoreline, and the existence of rocky ledges offshore of the beach. The geologic literature notes a remarkable lack of change in shoreline position at Nantasket Beach over the past 300 years, which may be partially attributed to the presence of the remnant drumlins offshore (Hayes et.al., 1973; and Brenninkmeyer, 1976).

Figure 2. Key Feature Location Map

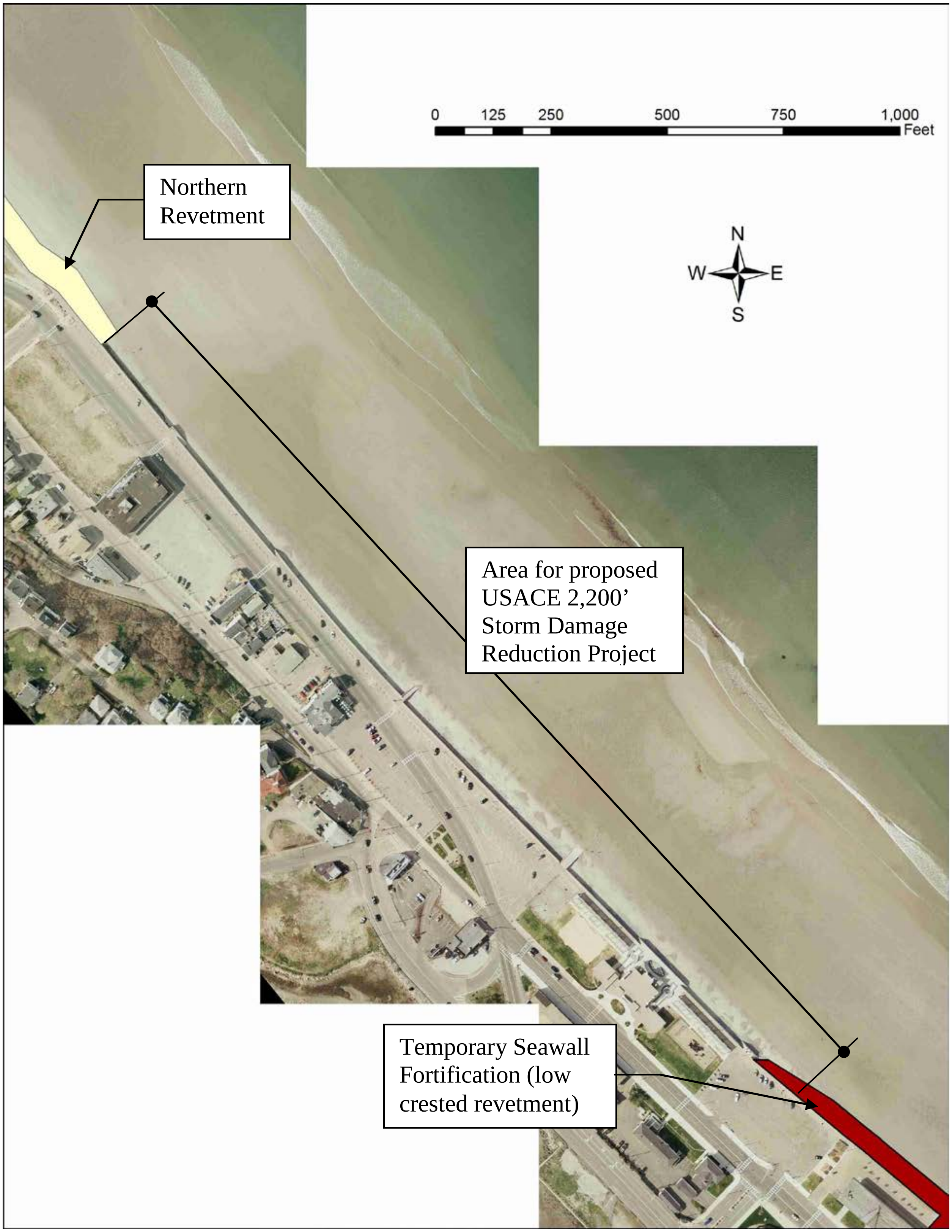


Figure 3. Project map

2.3 Tidal Regime

This area of the Atlantic Ocean experiences semi diurnal tides with a Mean Lower Low Water (MLLW) to Mean Higher High Water (MHHW) tide range of 10+ feet. The tidal benchmark data for Boston Harbor has been provided in Table 1. Due to the close proximity of Nantasket Beach to Boston Harbor the data is applicable to Nantasket Beach. A very slight correction exists between the two locations, but is very minimal and for the purposes of this study not worth considering.

Table 1. Tidal Regime Boston Harbor (Nantasket Beach)

Table 1. Boston, MA Bench Mark Data				
	MLLW	MTL	NGVD29	NAVD88
Datum	feet	feet	feet	feet
100-Year Return Period Water Level (adjusted to 2006)[1]			10.47	9.66
50-Year Return Period Water Level (adjusted to 2006)[1]			10.17	9.36
25-Year Return Period Water Level (adjusted to 2006)[1]			9.71	8.90
15-Year Return Period Water Level (adjusted to 2006)[1]			9.41	8.60
10-Year Return Period Water Level (adjusted to 2006)[1]			9.17	8.36
5-Year Return Period Water Level (adjusted to 2006)[1]			8.76	7.95
2-Year Return Period Water Level (adjusted to 2006)[1]			7.71	6.90
1-Year Return Period Water Level (adjusted to 2006)[1]			7.57	6.76
Max. Annual Predicted Tide (2005 to 2023)[2]	12.50	7.41	7.80	6.99
Max. Annual Predicted Tide (average)[3]	12.23	7.14	7.53	6.72
MEAN HIGHER HIGH WATER (MHHW)	10.27	5.18	5.57	4.77
MEAN HIGH WATER (MHW)	9.83	4.74	5.13	4.32
NORTH AMERICAN VERTICAL DATUM-1988 (NAVD)	5.51	0.42	0.81	0.00
Mean Sea Level (MSL)	5.20	0.11	0.50	-0.31
MEAN TIDE LEVEL (MTL)	5.09	0.00	0.39	-0.42
NGVD29	4.70	-0.39	0.00	-0.81
Mean Low Water (MLW)	0.45	-4.64	-4.25	-5.06
Mean Lower Low Water (MLLW)	0.00	-5.09	-4.70	-5.51
LENGTH OF SERIES: 19 Years				
TIME PERIOD: January 1983 - December 2001				
TIDAL EPOCH: 1983-2001				
[1]The elevations were adjusted using the sea level rise rate provided by NOAA for Boston Harbor.				
The elevations were corrected from 1988 (study completion date) to 2006 by applying the				
0.87 feet/century rise rate over the 19 year time period or a correction of 0.17 feet.				
[2]The elevation was determined using Tides and Currents Pro software to find the maximum				
annual predicted tide and then taking the maximum from that list (19 years of tidal predictions used)				
[3]The elevation was determined using Tides and Currents Pro software to find the maximum				
annual predicted tide and then the average was taken (19 years of tidal predictions used)				

3.0 Data Collection

In order to conduct a study of this type a fair amount of data is necessary to achieve results that are useful. In 2005 the Nantasket Beach Characterization Study (NBCS) was performed which collected beach profile data from the DCR seawall or back of dunes out to – 35 ft NAVD88. There were a total of eight profiles collected. Additionally, sediment samples were taken and analyzed to determine the sediment characteristics of the beach. The report has been included electronically as Appendix 1. The study found that the beach is comprised mostly of fine sand with a D_{n50} around 0.22 mm to 0.25mm with cobble and gravel mixed in. The cobble and gravel is evident where it has built up along the seawall and revetments within the DCR reservation. Additionally, in the study it was found through comparisons to profile data taken in the early 1960's by the USACE that the beach within the DCR reservation has not eroded nearly as much as believed prior to the completion of the study.

In addition to beach characterization data, time series storm data was needed. Time series water level data is available at the Boston Harbor tide station and wind and wave data is available offshore for the last 20 years from the Wave Information Study (WIS). The combination of the two data sets allowed for storm time series data to be developed.

4.0 Without Project and Beach Fill Alternative Analysis Methodology

Among the alternatives being considered by the Corps and DCR as part of the Section 103 investigation were the no action alternative and beach fill alternatives. In order to evaluate these alternatives the USACE model SBEACH was used. SBEACH is an empirically based numerical model used for simulating two-dimensional cross-shore beach change. The model was initially formulated using data from prototype-scale laboratory experiments and further developed and verified based on field measurements (Larson and Kraus 1989; Larson, Kraus, and Byrnes 1990). Prior to this effort SBEACH was most recently used for modeling Nantasket Beach in the Nantasket Beach Alternatives Analysis Study which was completed by USACE for the DCR in 2003. This study has been included as Appendix 2.

In order to run the SBEACH model two basic pieces of information are needed and those are beach profiles for the beach of interest and time series storm data (water level, wave height, wave period, and wind direction). This data was available for Nantasket Beach from the Nantasket Beach Characterization Study completed by USACE and the DCR in 2005 and from the USACE WIS Hind Cast Data (As discussed in Section 3.0). The WIS Hind Cast Data was taken from the following web page – Wave Hindcasts (http://frf.usace.army.mil/cgi-bin/wis/atl/atl_main.html)

4.1 Design Storm Conditions

With the level of study being considered for this project, developing a set of storms with a range of return periods was kept relatively simple. Thoughts were first given to using a Joint Probability Method (JPM), Empirical Simulation Technique (EST), or a Monte Carlo type simulation, but the associated effort for each of these, and the level of numerical modeling needed to make these worthwhile were beyond the scope of work for this effort. Fortunately for the New England District a study was completed in the late 1980's that provided return period tide elevation for the entire New England Coast. The title of the study is “Tidal Profiles of New England” and the tidal profile curves for the study area have been provided as Figure 4. With this information it was possible to rank various historical storm events based on return period water level.

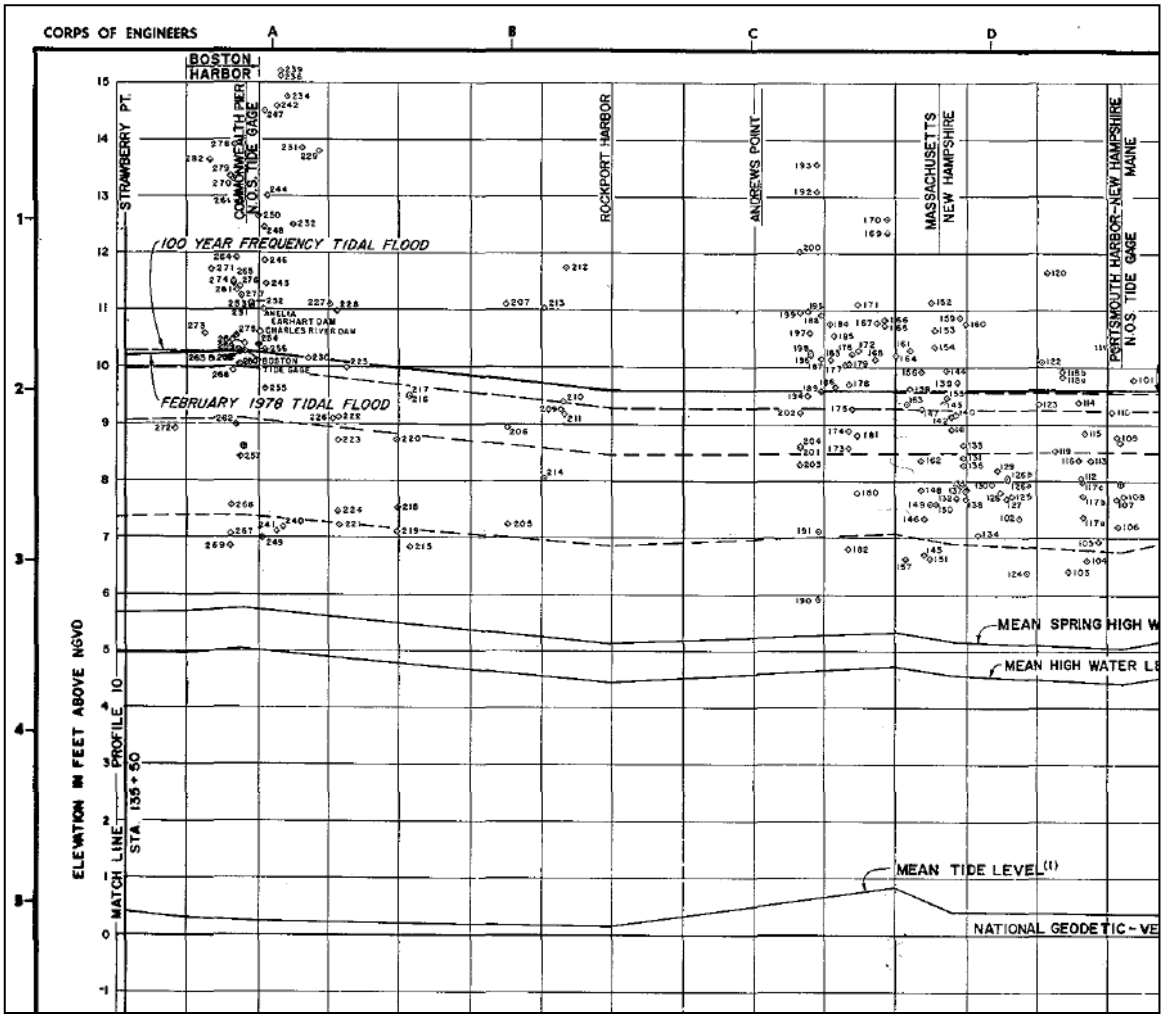


Figure 4. Tidal profile plot for Nantasket Beach (Boston Harbor, MA)

For this site it was reasoned that a ranking based upon water level was a valid methodology since the project area is a shallow sloped beach and for the most part experiences depth limited wave breaking conditions during storms. Depth limited wave breaking means that water depth controls the wave height impacting the site. Waves become unstable in water that is too shallow for a given wave height and wave period. Generally depth limited breaking occurs when the wave height exceeds 0.78 times the water depth or for random waves when significant wave height (H_s) exceeds 0.55 to 0.60 times the water depth. However, this ranking scheme was far from perfect and this was shown in the storm modeling results. Factors that control a storms severity or level of impact also include duration and wind direction. Each of these factors can strongly impact a storms impact on a specific site. Duration is perhaps the most significant factor. Consider two storms that generate identical maximum tidal elevations and produce similar waves; If one is moving at half the speed as the other it will impact an area twice as long and will be a much more damaging event even though based on a return period water level ranking they would be the same. However, ranking on duration would have inherent issues as well. With the multitude of dimensions that create a storm ranking is difficult.

Given the ranking criteria chosen the first step was to download the entire set of tide data from Boston Harbor back to the year 1975. The year 1975 was chosen since that was furthest back WIS Wave Hind Cast data was acquired to. This data was then screened in an excel spread sheet to determine when return period water level events occurred. From this screening high water event dates were used to screen the USACE WIS Hind Cast data to obtain corresponding wave height, wave period, wave direction, and wind direction data for those dates. For many of the low level water elevation events occurred only from astronomically high tide elevations, or during events that did not cause significant wave energy to impact Nantasket Beach. Once all of the data was filtered a total of 14 storms were chosen with return periods ranging from 2 to 50 years.

Ideally storms meeting the ranking criteria of 20 and 25 years would have been present within the data, but they were not when using the chosen ranking criteria.

Once the storms were chosen the WIS wave data were transitioned from the WIS data location using WISPH3 in CEDAS. This is a fairly simplistic but fast way to transfer waves from deep water to the shallow/intermediate water depths at the seaward edge of the project site (-35 ft contour). The plots of the 14 storms at the WIS station and after being transformed to the offshore study boundary can be seen in Appendix 3.

4.2 Beach Profile Selection

During the NBCS, a total of eight profiles were taken along Nantasket Beach, with four of them within the DCR Reservation. These northern profiles contained natural beach foreshore slopes, berms, and dunes, which made them valuable for comparison to the more highly impacted beach profiles in front of the DCR seawall and revetments. Fortunately, profile number 5, from the BCS was very nearly in the middle of the 2,000 foot section of beach being focused upon during the analysis for the project and therefore this profile was chosen to use in the SBEACH modeling analysis. The cross section is shown in Figure 5. Based on discussions with other users of SBEACH, the more typical practice would be to develop an average profile from multiple profiles for use in this type of study, but this was not done for several reasons. First the adjacent profiles to the north and south were in known "hot spot" areas that were identified in the AAS report and where revetment structures already had been constructed to protect the seawall. It was decided that the use of profiles from this area would provide an inaccurate representation of the study area. Secondly, using the northern most profiles that did not have a seawall feature would be very misrepresentative.

4.3 Beach Profile Adjustments

As discussed, profile # 5 was used to model without project conditions and as a basis for the numerous beach fill alternatives investigated during this study. The first without project runs were completed using the exact profile data collected in September 2005 and the cross section can be seen in Figure 5.

While using profile #5 for this study was reasonable it was decided that since the profile was collected during September 2005 it was representative of a summer beach condition and the performance during the various storm conditions would likely be better than a beach that was already adjusted to a winter profile. Essentially by modeling the summer profile with the various storms, the results would represent a design level storm hitting the beach early in the storm season. It was decided that a more realistic scenario would be to run the design level storms on a winter beach profile. A winter profile was developed by running the January 1998 northeaster (2 year storm) on profile #5 and then using the final resultant profile as a starting profile for the rest of the storm suite. The winter adjusted profile can be seen in Figure 5. It can be seen that the lower profile below -10 ft-NAVD88 remains unchanged, but above that elevation the level of sand against the wall drops approximately 3 ft which inherently starts the wall off in a lower stability condition.

The third without project condition modeled was one considering long term erosion and the resulting reduction in beach elevation in front of the wall. As discussed in Section 4.0 and the NBCS, the beach within the DCR reservation has lost very little sand during the 42 year span between USACE beach surveys. As discussed in the NBCS, when the volume lost over those 42 years is converted to a vertical change, on average, the beach only lowered 0.27 feet across the beach profile. It could certainly be argued very strongly that the future conditions 50 years out could be represented by the profiles from 2005 since it easily falls within the error of this modeling effort. However, based on the long shore modeling work performed in the AAS and the more recent work completed by WHG, the net transport in this area was calculated to be 10,000 to 15,000 yds^3/yr. Furthermore WHG determined the transport is from south to north, or out of the DCR Reservation to North Nantasket. Since new material is likely not being introduced to the system a conservative approach of assuming the 10,000 yds^3/yr would be taken from the beach within the DCR reservation and transported to the north. If the 10,000 yds^3/yr is averaged over the area within the DCR reservation the resulting beach lowering will be 1 foot over 50 years. In order to gage this condition, model runs for the suite of 14 storms were performed after first lowering the Summer 2005 beach profile by 1 foot and then "winterizing" it with the January 1998 storm. The lowered winterized profile can be seen in Figure 5.

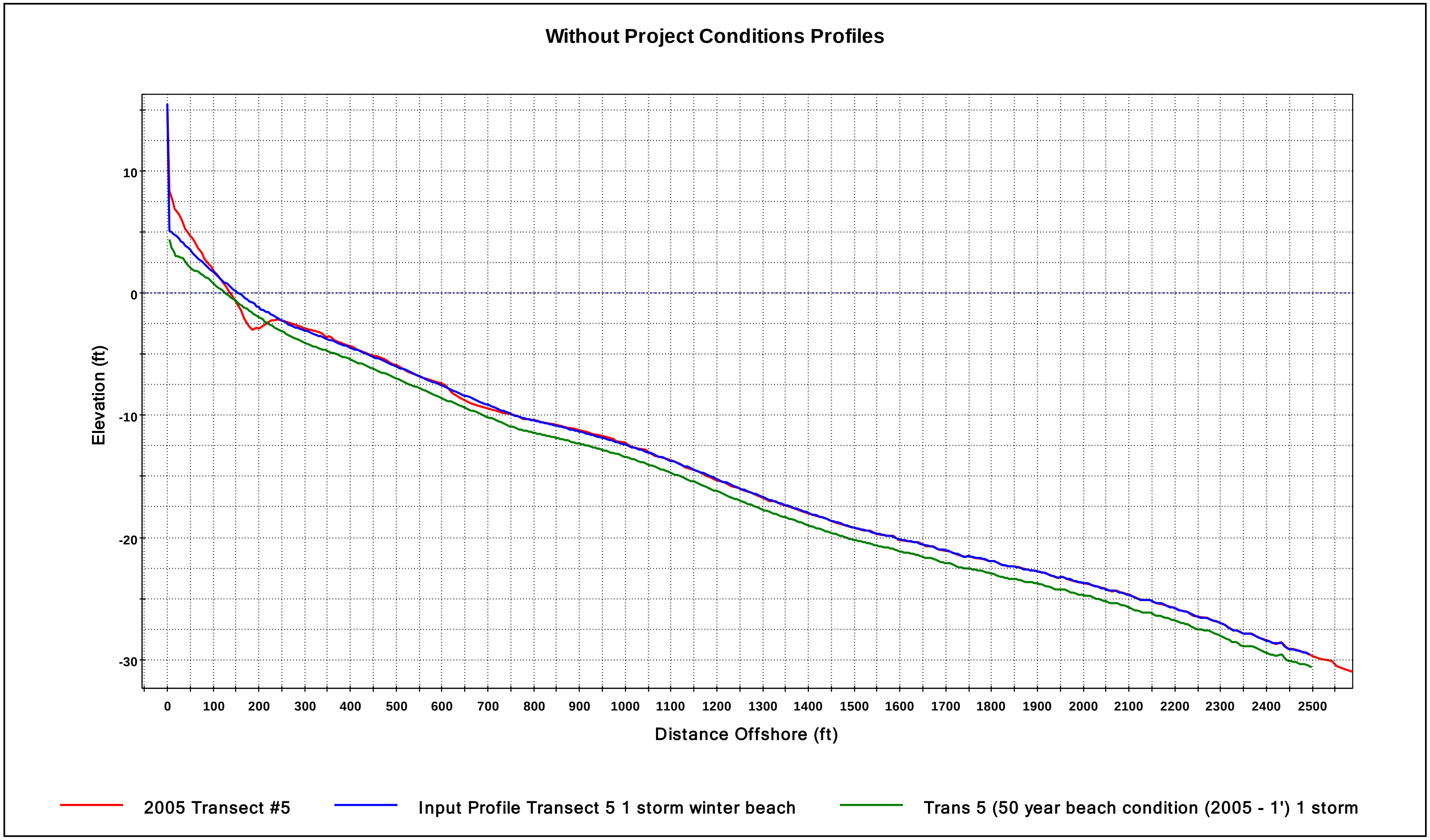


Figure 5. Profile #5 in Summer, Winter, and Winter Long Term Erosion Condition

4.4 Sea Level Rise

Given the uncertainty associated with this study it could once again be argued that sea level rise (SLR) over the next 50 years would fall within the accuracy of the study. However to look at this issue in a relative manner and to perform a sensitivity analysis, three scenarios were looked at for the without project condition and for several of the more likely beach fill alternatives. The three scenarios were to keep sea level at the present day value, the 50 year condition with the historic SLR reported by NOAA at Boston (0.435 ft/50 years), and doubling this rate to examine sensitivity to a SLR rate increase (0.87 ft/50 years).

4.5 Beach Performance and Seawall Stability Evaluation

When evaluating the performance of the various profiles, wall stability criteria for the seawall was used to gage the performance of the beach and its protection of the seawall. Through conversations with the projects structural engineer it was concluded that a berm at elevation 8 ft-NGVD and with a width of 5 to 10 ft would provide adequate support to keep the wall stable and to stay within the Corps safety margins. Because the project was working primarily in NAVD88 the berm elevation was approximated at elevation 7.25 ft-NAVD88 and for simplicity a berm width of 7.25 feet was used. This provided a clear cutoff for evaluating the performance of the beach as far as protecting the seawall from failure. If during a model storm run the minimum profile berm was reduced to a width narrower than 7.25 feet at elevation 7.25 feet-NAVD88 the wall stability would be considered unacceptable. Greater than that number it was concluded the beach was adequate to protect against that storm.

5.0 Without Project Condition Analysis

In order to properly represent without project conditions a total of five without project scenarios were looked at. Upon first consideration one may think there should only be one without project condition, but as discussed in Sections 4.3 and 4.4, once sea level rise and variations in beach erosion rates are considered the need for multiple without project future conditions needed to be considered. While it is difficult to predict the future, the analysis will provide a valuable sensitivity analysis and insight into the risk associated with various factors. When combined with the 14 storm events being used for evaluation a total of 84 model runs were completed for the without project condition. The without project condition model scenarios are listed below.

1. Without Project – 2005 Beach Profile #5
2. Without Project – 2005 Beach Profile #5 (adjusted with January 1998 storm)
3. Without Project – 2005 Beach Profile #5 with Historic Sea Level Rise
4. Without Project – 2005 Beach Profile #5 with Historic Sea Level Rise x 2
5. Without Project – 2005 Beach Profile #5 with Historic SLR minus 1 Foot
6. Without Project – 2005 Beach Profile #5 with Historic SLRx2 minus 1 Foot

As discussed in Section 4.5 the modeling and wall stability analysis was simplified down to measuring the berm width at elevation 7.25 feet-NAVD88 to determine if the wall was safe from failure during a particular storm. A graphical summary of how each without project scenario performed when exposed to the suite of design storms has been provided in Figure 6. It can be seen that none of the without project conditions provide adequate stability for the seawall. That does not necessarily mean the wall is currently unstable, but rather the current beach does not provide the level of protection to meet Corps safety requirements.

Additionally, it can be seen that for without project conditions, SLR and even doubling SLR over the next 50 years will have a minimal impact on the performance of the beach as it relates to wall stability since the difference in minimum elevations at the wall face drops on average by only 0.58 feet with a STD of 0.22 feet (for SLR doubled).

While model runs were not performed for only lowering the beach the impacts can be readily seen by looking at the combined SLR and beach lowering cases. Looking at the historical SLR cases it can be seen that by lowering the beach one foot the minimum beach elevation experienced during storm conditions drops on average by 1.90 ft with a STD of 0.24 ft. This was not unexpected since the one foot beach lowering results in a significant loss of beach volume per linear foot of beach.

These results demonstrate that perhaps the impacts of SLR are not that significant for the project site for the without project condition when considering seawall stability but long term erosion would be if the rates in the future are higher than the rates measured over the last 42 years.

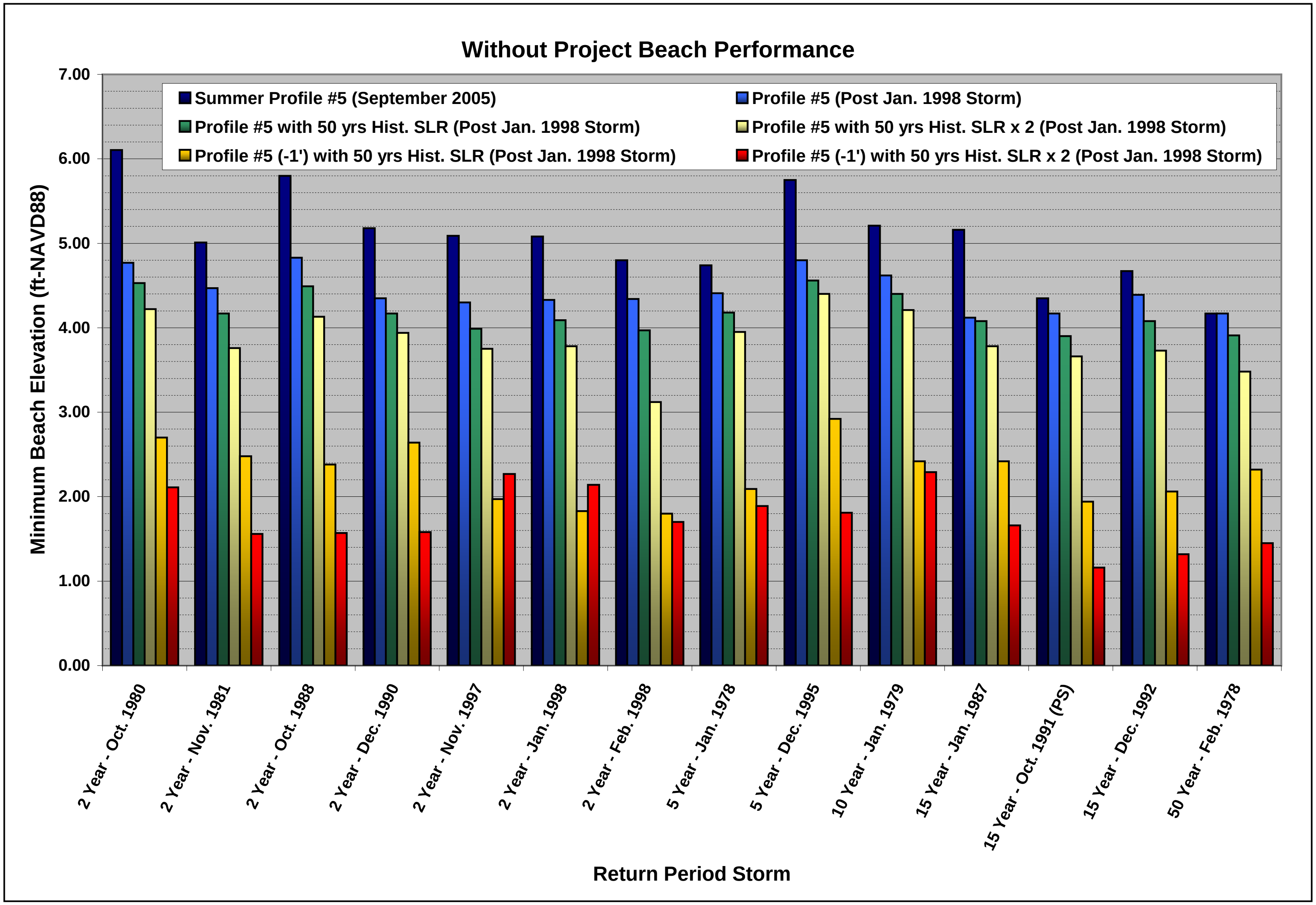


Figure 6. Without project beach performance during a storm

6.0 With Beach Fill Project Alternative Results

Beach fill design was completed using the Regional Morphology Analysis Package (RMAP) software which is part of the Coastal Engineering Design and Analysis System (CEDAS) software package. The first set of alternatives modeled used a D_{n50} grain size of 0.25 mm, which was considered compatible sand to the existing beach sand based on the NBCS Report from 2005. Three different berm elevations were modeled to determine performance sensitivity to berm elevation. Based upon the more complete beach profiles to the north of the DCR reservation it was determined the natural berm elevation for the system was approximately 9 ft-NAVD88. For the modeling alternatives the elevations of 9.25, 10.25 and 11.25 ft-NAVD88 were chosen. The lower elevation of 9.25 ft was chosen to match the upper end of the natural berm elevations, 10.25 was modeled to investigate if storm performance would be improved by a higher berm elevation, and 11.25 was chosen to match the upper elevation being studied in the WHG effort. The 0.25 foot elevations were done to match the WHG effort. WHG was using the NGVD datum which is approximately 0.75 ft higher than NAVD88. Hence the alternatives were equivalent to 10, 11, and 12 ft-NGVD29.

For each of the berm elevations a range of berm widths were modeled. Berm widths for each berm elevation were started at 0 feet and then widened to a 5 foot berm, and then in 10 foot increments following that. Berm widths were increased until the minimum beach profile during a storm model run stayed above the 7.25 foot berm width at elevation 7.25 feet-NAVD88 wall stability criteria.

As with the without project condition runs, SLR impact was modeled by using historic SLR and a rate double the historic rate. However, the impacts of SLR were only tested for the 9.25 and 10.25 NAVD88 since they were the more likely alternatives to be chosen.

Each of the alternatives were modeled using the full set of return period storms. Given that there were over 45 beach fill/SLR scenarios modeled for 14 storms the total number of model runs exceeded 700 for the final analysis. The SLR model runs will be discussed in Section 7.5.

6.1 With Project – 9.25 ft-NAVD88 Berm Elevation with D_{n50} of 0.25 mm

For the 9.25 ft berm elevation it can be seen in Figure 7 that the necessary beach berm width needed to be present at the beginning of 2 year storm to ensure seawall safety (by Corps criteria) is between 10 ft and 20 ft. With the 10 foot berm in place the berm width falls below the minimum 7.25 ft width criteria for most of the two year storms but for a 20 foot berm the beach stays above the 7.25 ft width criteria for all 2 year storms. For the higher level storms (5 to 50 yr events) the results are a little less conclusive. For example when looking at the two 15 year events a 20 ft wide berm is more than adequate to provide the necessary protection during the January 1987 storm but not the October 1991 storm, which is known as the Halloween Storm or the Perfect Storm. This returns the discussion back to points raised in Section 4.1 related to storm ranking criteria. The

Perfect Storm was a much longer duration storm as can be seen when comparing Figures A3-11 and A3-12. Not much can be done to resolve this issue at the level of this study and the results will have to be used in the context given. It can be seen that in general for the 5 to 50 year events a starting berm width of 20 ft will provide adequate protection against many of the storms. If a 30 ft berm is present the wall is well protected and the berm width exceeds 7.25 ft even for the 50 year storm (Blizzard of 78). It must also be remembered that even during the storms where the berm width falls below the minimum criteria the wall would likely not fail since the seawall has already survived significant events with less protection.

6.2 With Project – 10.25 ft-NAVD88 Berm Elevation with Dn50 of 0.25 mm

Looking at the results for the 10.25 ft berm elevation in Figure 8 it can be seen that the necessary beach width at elevation 10.25 ft-NAVD88 required to be present at the beginning of 2 year storms to ensure seawall safety by Corps criteria is 0 ft. Due to the higher elevation of the fill the width of the sand fill at elevation 7.25 ft-NAVD88 is wide enough to withstand two year events. What this would look like is a natural sloping beach up to the wall with no plateau or berm. For the higher level events the same issues seen in for the 9.25 ft berm alternative were present in that there was higher variability in the minimum beach width at elevation 7.25 ft-NAVD88. With the 5 ft berm width present the only storms that cause the beach width at 7.25 ft-NAVD88 to fall below the 7.25 ft width were two of the 15 year storms and the 50 year storm (Blizzard of 1978). With the 10 ft wide berm in place the beach is adequate to protect against all of the storms tested.

6.3 With Project – 11.25 ft-NAVD88 Berm Elevation with Dn50 of 0.25 mm

Similar to the 10.25 ft-NAVD88 beach fill, as shown in Figure 9, the 11.25 ft fill does not require an actual level berm at elevation 11.25 ft-NAVD88 to provide storm protection. Due to the width of the fill at elevation 7.25 ft-NAVD88 this fill will protect against all of the modeled storms and will have over double the minimum width required at elevation 7.25 ft-NAVD88 even during the 50 year storm (Blizzard of 1978).

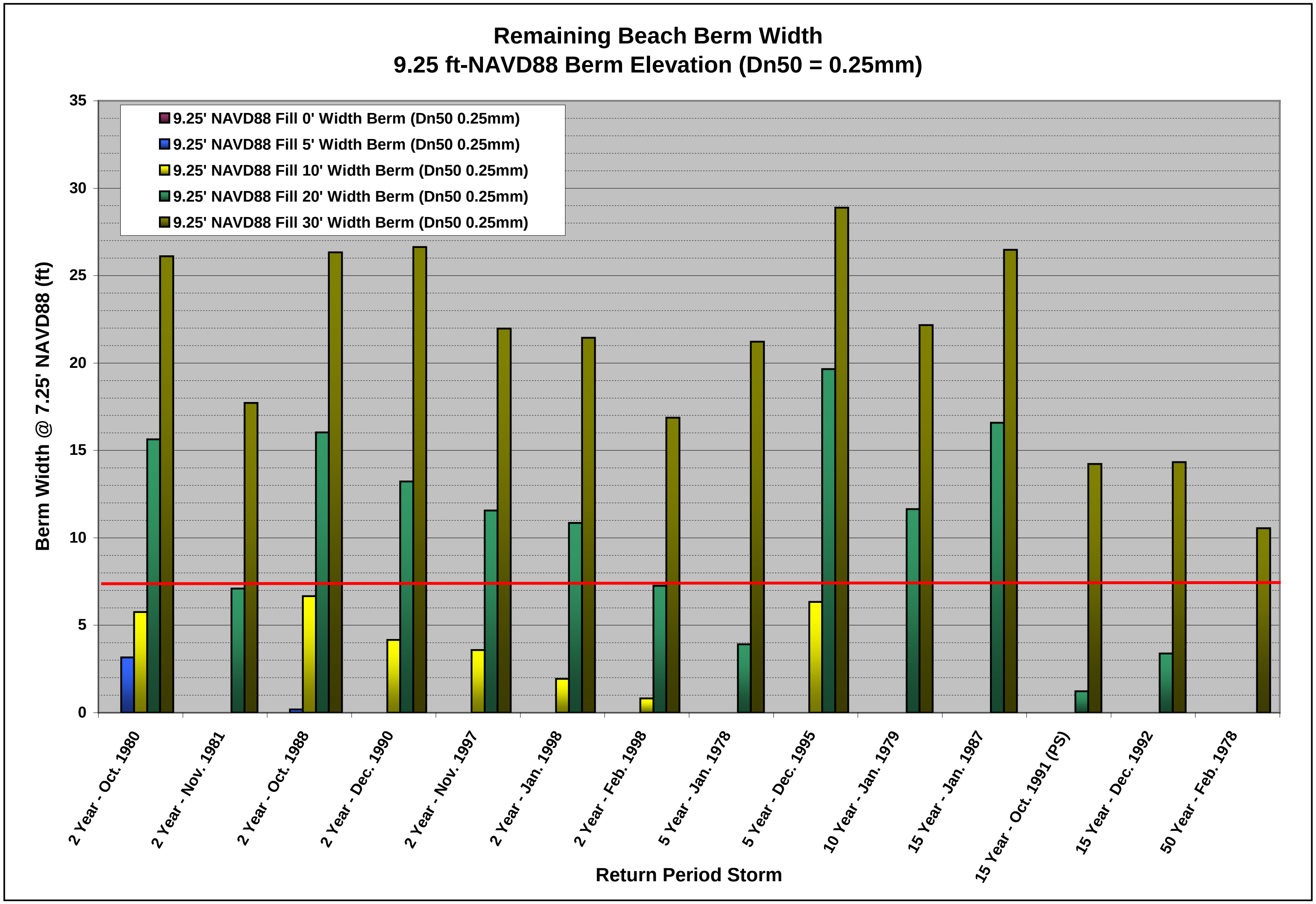


Figure 7. Beach fill performance 9.25' NAVD88 berm elevations

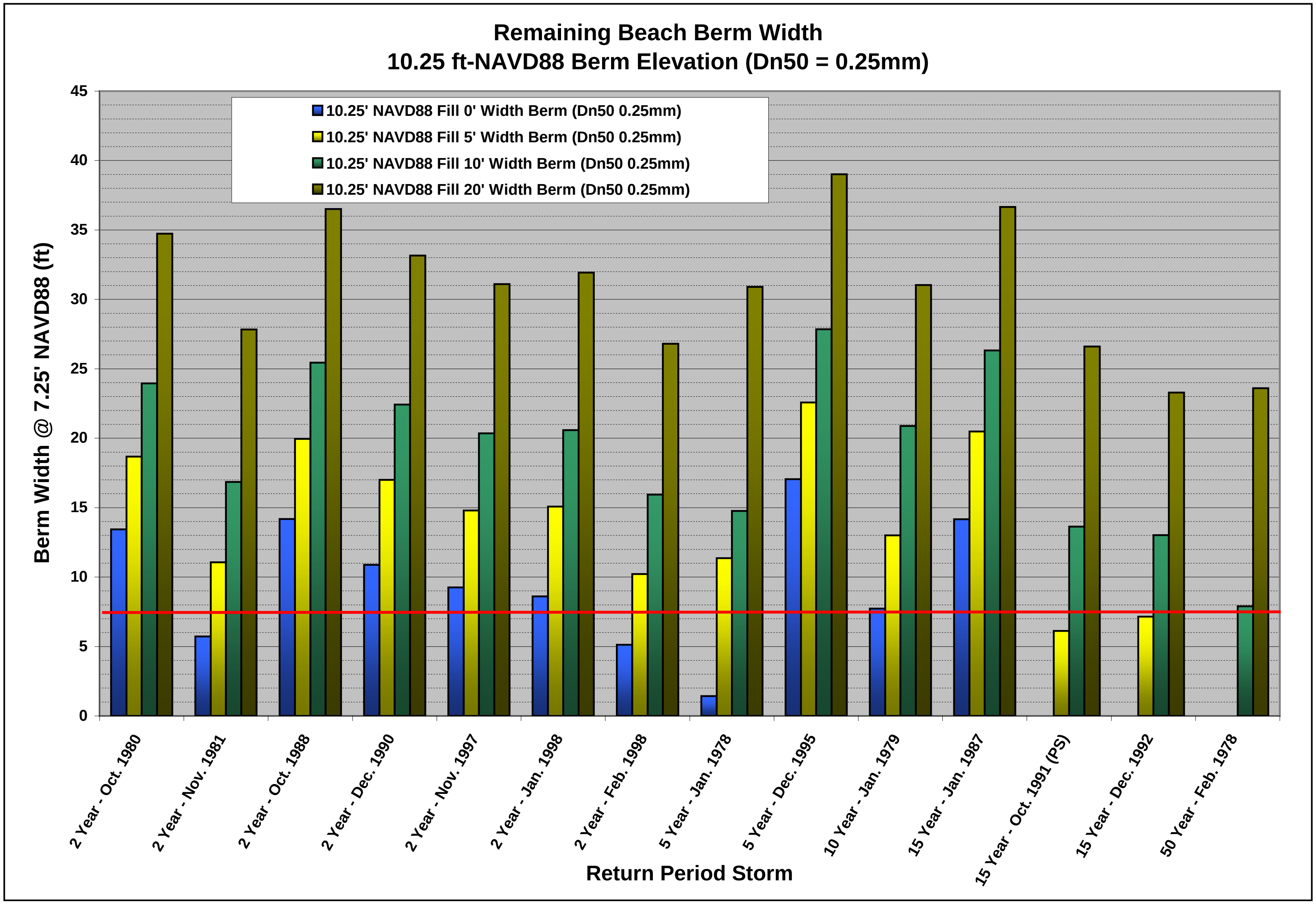


Figure 8. Beach fill performance 10.25' NAVD88 berm elevations

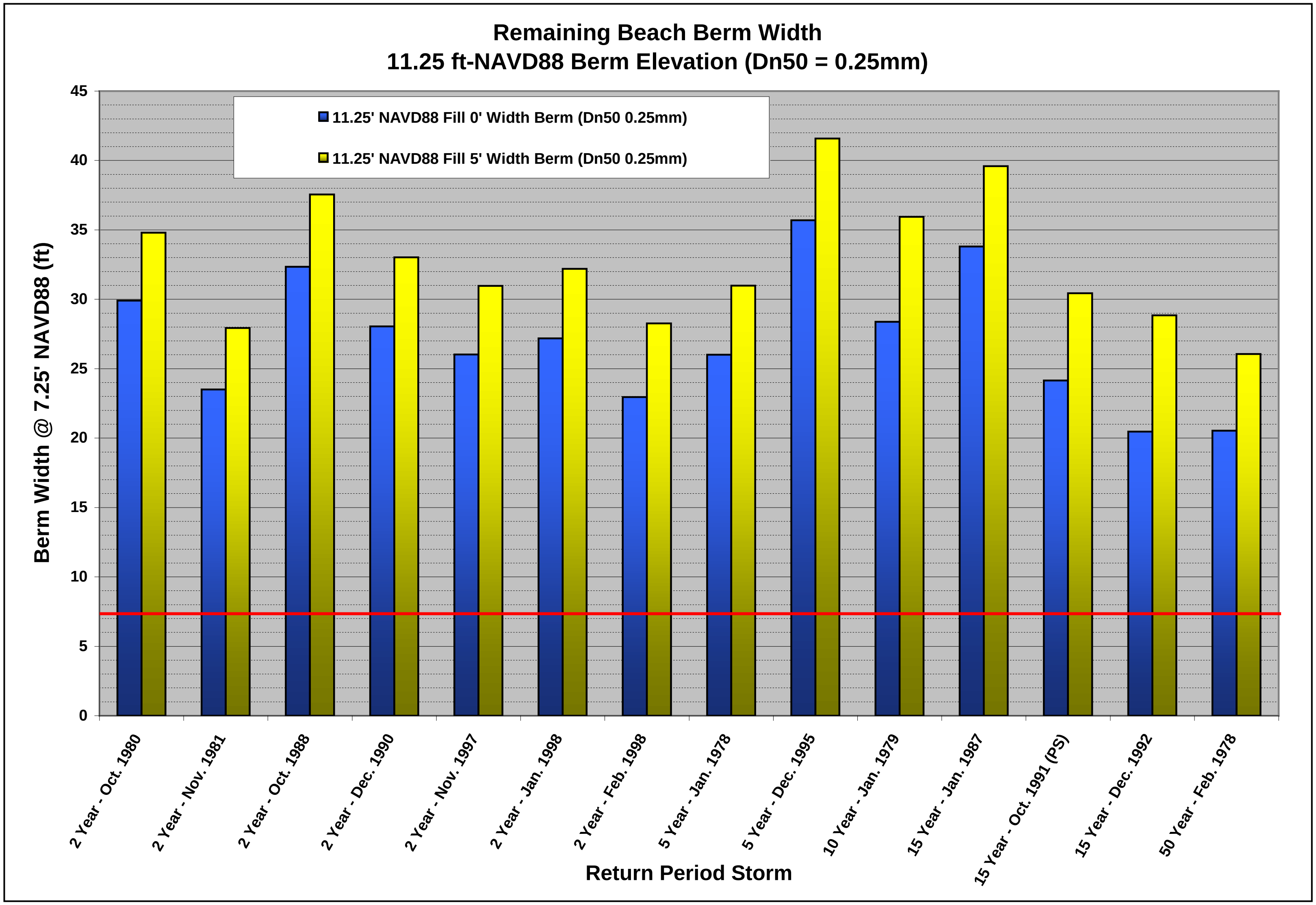


Figure 9. Beach fill performance 11.25' NAVD88 berm elevations

6.4 Beach Fill Alternative Performance Volume Comparison

As seen in Sections 6.2 to 6.3, for the higher beach berm elevation alternatives, less berm width at the design berm elevation is needed to meet the minimum protection criteria for each of the storms. At first glance this may seem to indicate less beach fill is needed with the higher beach fill alternatives. However, the reason the higher berm alternatives perform better with narrower beach berm widths is that the higher beach fill alternatives contain more beach fill volume. To better look at storm performance versus the beach fill alternatives the fill volume associated with each alternative elevation was looked at (Table 2). In order to compare alternatives the berm width at each elevation that provided the minimum criteria level of protection against the 50 year storm was highlighted in red. Based on this it can be seen in Table 2 the volume of fill needed to provide the minimum protection level is essentially the same and does not depend on berm elevation. The difference between the three alternatives is only 10 cubic yards per linear foot of fill or about 20,000 cubic yards for the entire fill length of 2,000 feet. In actuality these volumes would be even closer if the level of protection was made exactly equal. The storm performance vs. volume relationship also holds true for the lower level storms.

Table 2. Beach fill volumes

Beach Fill Volumes - Winter Profile Present (Berm Elevation of 9.25' NAVD88 with Compatible Sand Dn_{50} 0.25mm)

Profile	Beach Fill Volume/ft of beach (yds^3/ft)	Beach Fill Volume/2,000 ft fill (yds^3)	Xon (ft)	Xoff (ft)	Volume (yds^3/ft)	Contour Location (-29' NAVD 88) (ft)
2005 Trans #5 (post Jan 1998 storm)	0	0	3.81	2,450	1,292.95	2,443.4
Trans #5 (0' berm @ 9.25' NAVD, 0.25mm Dn50)	38.73	77,458	3.81	2,450	1,331.68	2,472.3
Trans #5 (5' berm @ 9.25' NAVD, 0.25mm Dn50)	45.75	91,498	3.81	2,450	1,338.70	2,477.3
Trans #5 (10' berm @ 9.25' NAVD, 0.25mm Dn50)	52.76	105,520	3.81	2,450	1,345.71	2,482.3
Trans #5 (20' berm @ 9.25' NAVD, 0.25mm Dn50)	66.75	133,500	3.81	2,450	1,359.70	2,492.3
Trans #5 (30' berm @ 9.25' NAVD, 0.25mm Dn50)	80.69	161,372	3.81	2,450	1,373.64	2,502.3
Trans #5 (40' berm @ 9.25' NAVD, 0.25mm Dn50)	94.56	189,126	3.81	2,450	1,387.52	2,512.3
Trans #5 (50' berm @ 9.25' NAVD, 0.25mm Dn50)	108.38	216,758	3.81	2,450	1,401.33	2,522.3
Trans #5 (60' berm @ 9.25' NAVD, 0.25mm Dn50)	122.15	244,294	3.81	2,450	1,415.10	2,532.3
Trans #5 (70' berm @ 9.25' NAVD, 0.25mm Dn50)	135.91	271,822	3.81	2,450	1,428.86	2,542.3

Beach Fill Volumes - Winter Profile Present (Berm Elevation of 10.25' NAVD88 with Compatible Sand Dn_{50} 0.25mm)

Profile	Beach Fill Volume/ft of beach (yds^3/ft)	Beach Fill Volume/2,000 ft fill (yds^3)	Xon (ft)	Xoff (ft)	Volume (yds^3/ft)	Contour Location (-29' NAVD 88) (ft)
2005 Trans #5 (post Jan 1998 storm)	0	0	3.81	2450	1292.95	2443.44
Trans #5 (0' berm @ 10.25' NAVD, 0.25mm Dn50)	61.60	123,194	3.81	2450	1354.55	2488.39
Trans #5 (5' berm @ 10.25' NAVD, 0.25mm Dn50)	68.77	137,538	3.81	2450	1361.72	2493.39
Trans #5 (10' berm @ 10.25' NAVD, 0.25mm Dn50)	75.93	151,852	3.81	2450	1368.88	2498.39
Trans #5 (20' berm @ 10.25' NAVD, 0.25mm Dn50)	90.20	180,394	3.81	2450	1383.15	2508.39
Trans #5 (30' berm @ 10.25' NAVD, 0.25mm Dn50)	104.41	208,812	3.81	2450	1397.36	2518.39
Trans #5 (40' berm @ 10.25' NAVD, 0.25mm Dn50)	118.56	237,118	3.81	2450	1411.51	2528.39

Beach Fill Volumes - Winter Profile Present (Berm Elevation of 11.25' NAVD88 with Compatible Sand Dn_{50} 0.25mm)

Profile	Beach Fill Volume/ft of beach (yds^3/ft)	Beach Fill Volume/2,000 ft fill (yds^3)	Xon (ft)	Xoff (ft)	Volume (yds^3/ft)	Contour Location (-29' NAVD 88) (ft)
2005 Trans #5 (post Jan 1998 storm)	0	0	3.81	2450	1292.95	2443.44
Trans #5 (0' berm @ 11.25' NAVD, 0.25mm Dn50)	85.08	170,158	3.81	2450	1378.03	2504.59
Trans #5 (5' berm @ 11.25' NAVD, 0.25mm Dn50)	92.39	184,776	3.81	2450	1385.34	2509.59
Trans #5 (10' berm @ 11.25' NAVD, 0.25mm Dn50)	99.68	199,364	3.81	2450	1392.64	2514.59
Trans #5 (20' berm @ 11.25' NAVD, 0.25mm Dn50)	114.23	228,452	3.81	2450	1407.18	2524.59
Trans #5 (30' berm @ 11.25' NAVD, 0.25mm Dn50)	128.73	257,462	3.81	2450	1421.68	2534.59
Trans #5 (40' berm @ 11.25' NAVD, 0.25mm Dn50)	143.23	286,468	3.81	2450	1436.19	2544.59

To further demonstrate the relationship between beach fill volume and performance the storm modeling data was reorganized into a single table which has been provided as Table 3. To create this table each of the alternatives (9.25, 10.25 and 11.25 ft berm elevations) were ranked according to beach fill volume and the minimum beach width was provided for each storm for each beach fill volume (beach fill alternative). From this table a series of plots were created and provided as Figures 10 to 12. In the first plot the resulting minimum beach width at the elevation of 7.25 ft-NAVD88 has been provided for each storm and is plotted against the beach fill volume (cubic yards per linear foot of beach). Notice once again that no attention was paid to the beach fill elevation and only to the fill volume associated with each alternative.

Table 3. Beach fill volume and storm performance (berm width at 7.25 ft-NAVD88)

Beach Fill Volume	Modeled Storms							
yds^3/ft of beach	2 Year-Oct. 1980	2 Year-Nov. 1981	5 Year-Jan. 1978	5 Year-Dec. 1995	10 Year-Jan. 1979	15 Year-Oct. 1991 (PS)	15 Year-Dec. 1992	50 Year-Feb. 1978
38.729	0	0	0	0	0	0	0	0
45.749	3.16	0	0	0	0	0	0	0
52.76	5.75	0	0	6.33	0	0	0	0
61.597	13.43	5.73	1.42	17.05	7.73	0	0	0
66.75	15.63	7.1	3.91	19.65	11.64	1.23	3.39	0
68.769	18.68	11.07	11.36	22.57	13	6.12	7.15	0
75.926	23.95	16.86	14.77	27.85	20.89	13.64	13.02	7.89
80.686	26.11	17.72	21.22	28.89	22.17	14.23	14.33	10.55
85.079	29.9	23.51	26	35.7	28.38	24.15	20.47	20.54
90.197	34.73	27.83	30.91	39.01	31.04	26.61	23.29	23.61
92.388	34.8	27.94	30.99	41.58	35.94	30.43	28.85	26.05

Note several of the 2 year storms and one of the 15 year storms were not included so the table would fit in the report format. The entire table has been provided in Appendix 2.

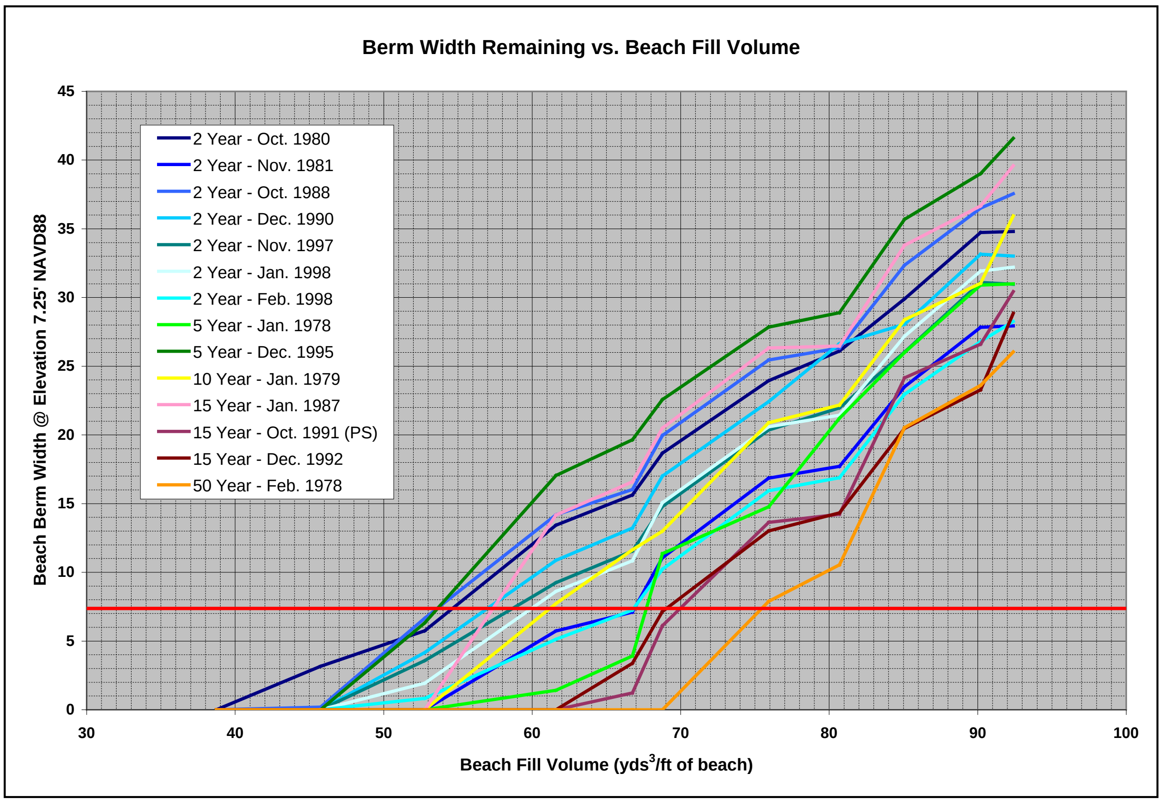


Figure 10. Minimum berm width vs. fill volume for each storm event

It can be seen clearly that for each storm that the relationship of beach fill volume to storm performance (beach width remaining at elevation 7.25 ft-NAVD88) is very nearly linear with only slight variation. This once again shows that the performance of the beach fill projects is controlled by fill volume and not by beach fill berm elevation.

Additionally it can be seen that for the storms tested that the range of beach fill volume that needs to be in place to protect the seawall ranges from 54.5 yds^3/ft to 75.5 yds^3/ft. If these numbers are converted to a 2,000 foot long beach fill the range is 109,000 yds^3 to 151,000 yds^3.

Looking at Figure 10, one will see that the variation in the beach fill volume necessary to ensure the minimum protection level for the seawall width for the various return periods is fairly considerable. Each return period was categorized by color. To help demonstrate this more clearly Figure 11 has been provided. In this figure all of the 2 year storms were designated as blue lines, the 5 to 10 year storms were designated as green lines, the 15 year storms were designated as orange lines, and the sole 50 year storm was designated as a red line. What Figure 11 shows is that there is a wide variation in beach performance within each return period storm range. This helps to demonstrates some of the points made in Section 4.1, in that although the storm's return periods were defined by water level (which was reasonable), the influence of wave period, direction, and storm duration are also significant factors in the performance of the beach.

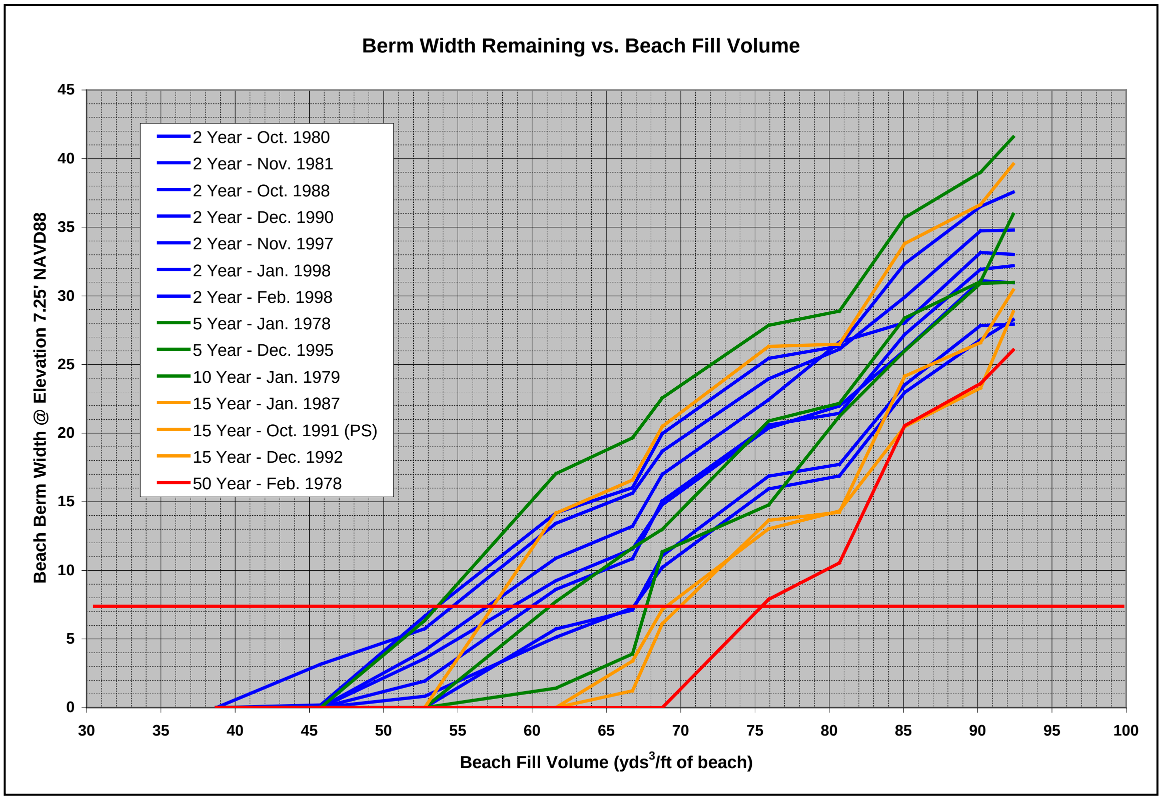


Figure 11. Min. berm width vs. fill volume (2 year, 5 to 10, 15, and 50 year storm)

Given the wide scatter of the 2 year events, the 5 and 10 year events, and the 15 year events average plots were created for each of these return period ranges and plotted in Figure 12. As shown in Figure 12, the difference between the averaged return period ranges is not very significant with the difference in fill volume being 9.5 yds^3/foot of beach. That equates to a 19,000 yds^3 difference for the entire 2,000 foot long fill. It can also be seen that even with the limited number of storms used in each category there appears to be a noticeable trend that makes sense in that the lower level events require less beach fill in place than higher level events. In addition to the averaged performance curves, the maximum and minimum curves for each return period level were included as dotted lines. These are basically the best and worst storms plotted for each of the storm levels. If the worst case scenario or most damaging storms are looked at instead of the averages for each return period category it can be seen that 67.0 yds^3/foot of beach would be needed for the 2 year storms to ensure the minimum protection level is not exceeded, for the 5 to 10 year events 67.5 yds^3/foot of beach fill needs to be in place, and for the 15 year storms 70.0 yds^3/foot of beach would need to be in place. The difference between the three is less than 3 yds^3/foot of beach or 6,000 yds^3 for the entire 2,000 foot long fill.

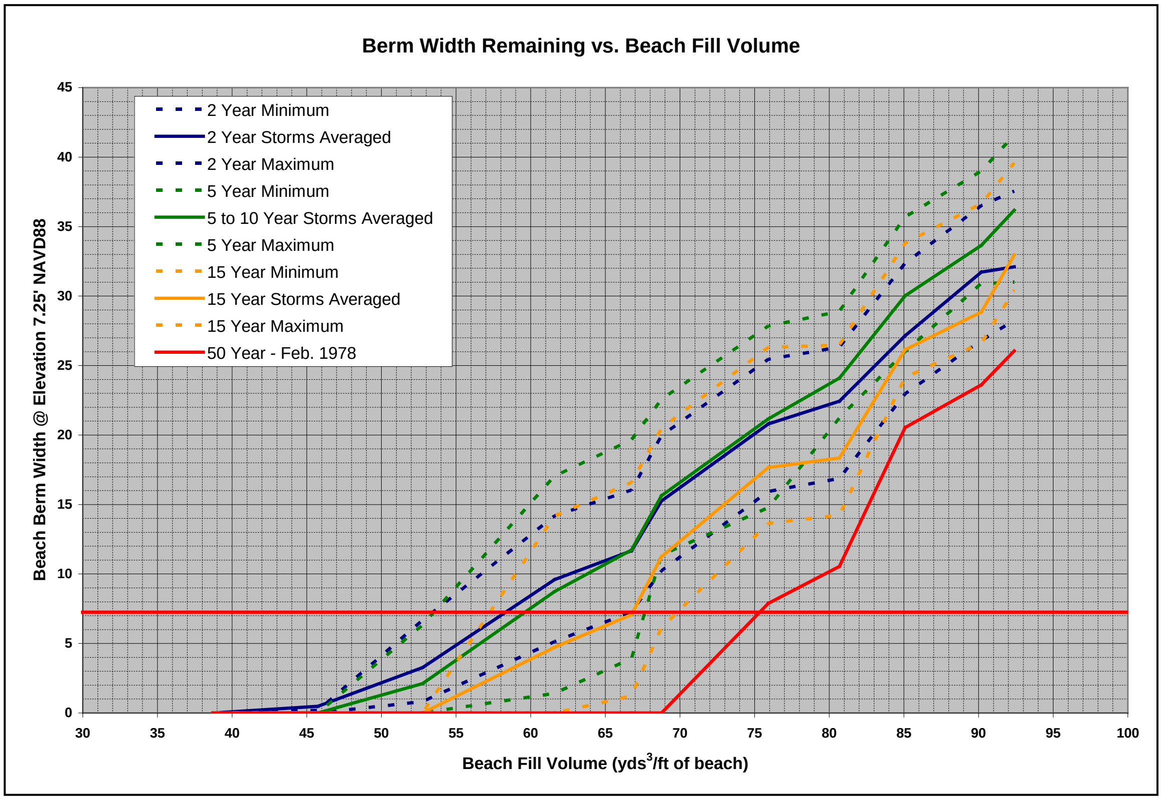


Figure 12. Min berm width vs. fill volume (2, 5 to 10, and 15 year storm averages)

As expected and shown, the 50 year storm would require the most significant fill volume to be in place to withstand the storm and not fall below the Corps' minimum protection criteria. It can be seen in Figure 12, that this storm would require 75 yds^3/ft of beach fill, which is 17 yds^3/ft more than the average 2 year storm, 15.5 yds^3/ft more than the average 5 to 10 year storm, and 8.5 yds^3/ft than the average 15 year storm. However, when looking at the maximum or worst case storms for each return period interval the difference is smaller with 8.5 yds^3/ft more than the maximum 2 year storm, 8 yds^3/ft for the maximum 5 to 10 year storm, and only 5.5 yds^3/ft more than the maximum15 year storm. The differences are 17,000 yds^3, 16,000 yds^3, and 11,000 yds^3 of fill respectively for the entire 2,000 ft long beach fill.

Once again what this analysis has shown is that the actual berm elevation is not really the critical issue for storm performance, but instead the fill volume is. Beach fill berm elevation is important for beach use and placing the berm too high or too low results in usability and safety concerns.

6.5 D_{n50} 0.25 mm Beach Fill Alternative SLR Impacts

As with the without project condition, SLR was looked at for the 9.25 ft-NAVD88 and the 10.25 ft-NAVD88 alternatives. Both the historical rate of SLR was considered and a rate double the historic SLR was considered. The alternatives were modeled with the same suite of storms as before, but the water levels were adjusted to 50 years in the future with these SLR rates. The results for both berm elevations can be seen in Figures 13 and 14. Looking at the impacts of SLR on the alternatives it can be seen, that unlike the without project condition, SLR does have a noticeable impact on beach fill performance. For the 9.25 ft-NAVD88 alternative it can be seen that for many of the storms that the historic SLR scenarios reduces the minimum beach berm width at elevation 7.25 ft-NAVD88 by 5 ft to 8 ft. The impacts of doubling the rate of sea level rise are more significant with reductions in minimum berm width often exceeding 10 ft vs. the exiting condition minimum berm widths. The reductions in berm widths for the 10.25 ft alternative were not as great due to the higher beach fill volume associated with these alternatives. For the 10.25 ft alternative the berm widths for the historic SLR scenarios were reduced by 5 ft or less and for the double the historic SLR rate the berm widths were reduced by between 5 ft and 10 ft.

When looking at the alternatives based on beach fill volume instead of beach berm width and berm elevation, the results are similar to the results seen in the with no SLR analysis (Section 6.4). Shown in Figures 15 through 18 are the volume based comparisons for the alternatives for historical SLR and SLRx2. Just as for the without project condition, two plots were shown for each SLR scenario. The first showing the minimum berm width for each beach fill volume for each storm, with the 2 year, 5 to 10 year, 15 year, and 50 year storms grouped. The second plot shows the average for each storm grouping and the maximum and minimum condition. As the figures show, the relationship of minimum berm width to beach fill volume is nearly linear, with slightly more deviations when compared to the no SLR scenario in Section 6.4. As seen with the no SLR condition the variation among each storm classification or grouping are high. To address this scatter the averaging analysis performed for the without project condition was performed for the SLR alternatives. For the SLR scenario analysis it can be seen that the averaged minimum berm widths versus beach fill volume did rank as expected and the lower level storms required less beach fill than the stronger or higher return period storms.

6.6 Beach Fill Volume Analysis Summary

To summarize the beach fill volume discussion and compare the no sea level rise and sea level rise scenarios, Figure 19 was developed. In the figure the volumes associated with the averaged minimum berm widths at the 7.25’ NAVD88 elevation for each of the return period categories (2 year, 5 to 10 years, 15 years, and the 50 year storms) were taken from Figures 12, 16, and 18 and plotted against return period. The 5 to 10 year category was plotted as 7.5 years to allow for plotting. Along with the averaged storms beach fill volume the maximum and minimum volumes for each return period category were plotted to provide a range and to keep the plotted information in perspective. From this plot the impacts on the beach fill from SLR are more clearly shown. It must be remembered though this plot was generated with only four return periods and a limited number of storms per category. Many more storms would be needed for this plot to have statistical significance. However, this plot does summarize the available information for this study. Looking at the plot it is shown that for an “average” 2 year storm that 58.5 yds^3/ft of beach is needed to protect the seawall if SLR is not considered. With historic SLR factored in, in 50 years 66 yds^3/foot of beach would be needed to protect against the same “average” 2 year storm and 70.5 yds^3/ foot of beach would be needed if the rate of SLR is double the historic rate over the next 50 years. If the rate of SLR doubles the fill volume will increase almost 20 yds^3/foot of beach to protect the seawall against an “average” 2 year storm. Looking at Figure 16 it is shown that for an “average” 5 to 10 year storm the volume of sand needed to protect the wall is 60 yds^3/foot of beach if there is no SLR. With historic SLR factored in, in 50 years 68 yds^3/foot of beach would be needed to protect against the same “average” 5 to 10 year storm and 72.5 yds^3/foot of beach would be needed if the rate of SLR is double the historic rate over the next 50 years. If the rate of SLR doubles the fill volume will increase almost 13 yds^3/foot of beach to protect the seawall against an “average” 5 to 10 year storm.

Sea level rise must be kept in context though and especially the predictions of SLR rate increase. Considering the project life is 50 years the analysis performed for SLR was out at year 50. This basically provided the worst case scenario for the project since in year 20 the impacts of SLR will be less. Also consider if the rate of SLR is increasing which it has not definitively shown to be yet, early in the project life the SLR rate will very likely be the same or similar to the historic rate. This means that even if the rate of SLR is increasing it will only impact the project later into the project life.

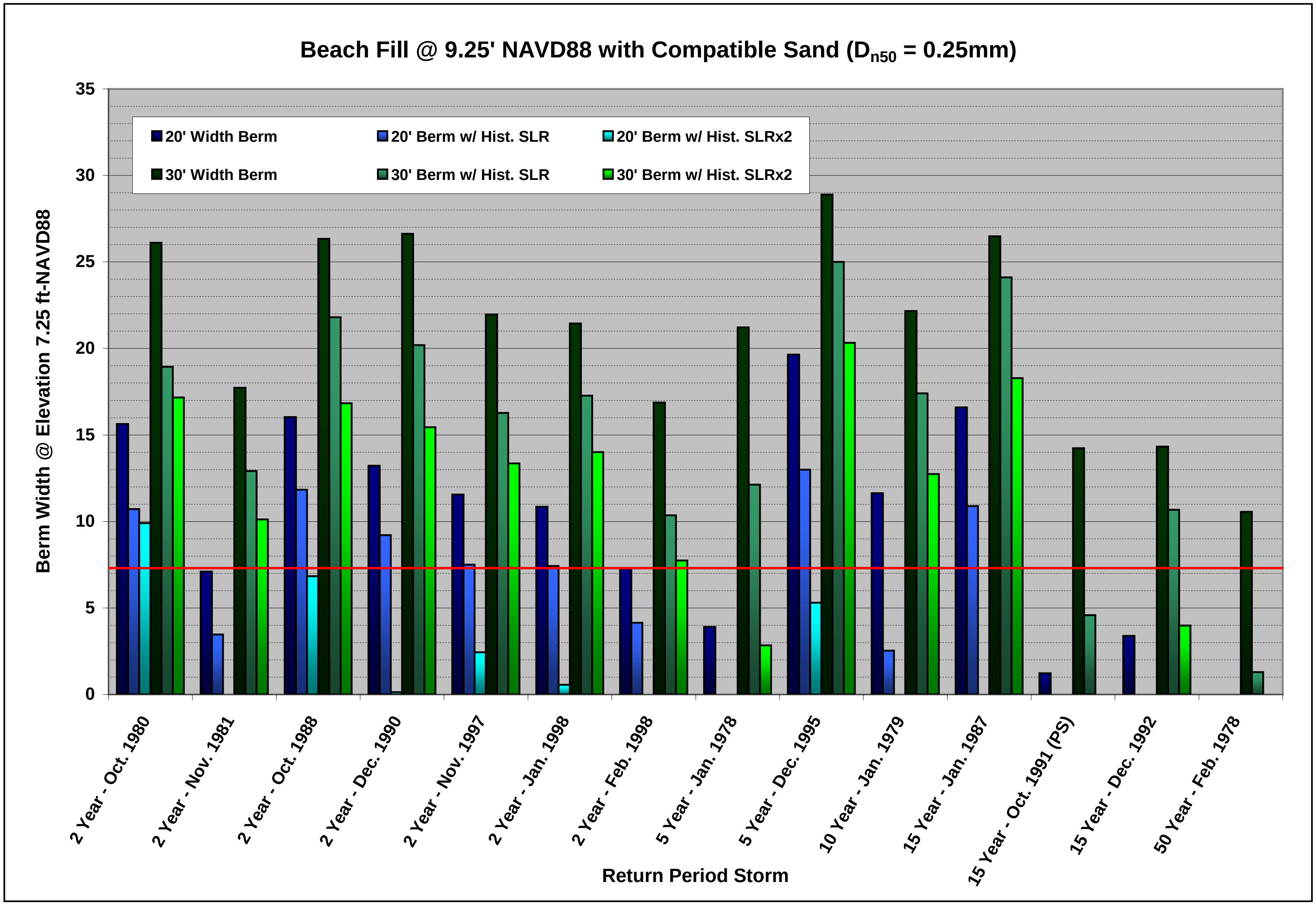


Figure 13. SLR impacts (9.25 ft-NAVD88 berm elevation and compatible sand)

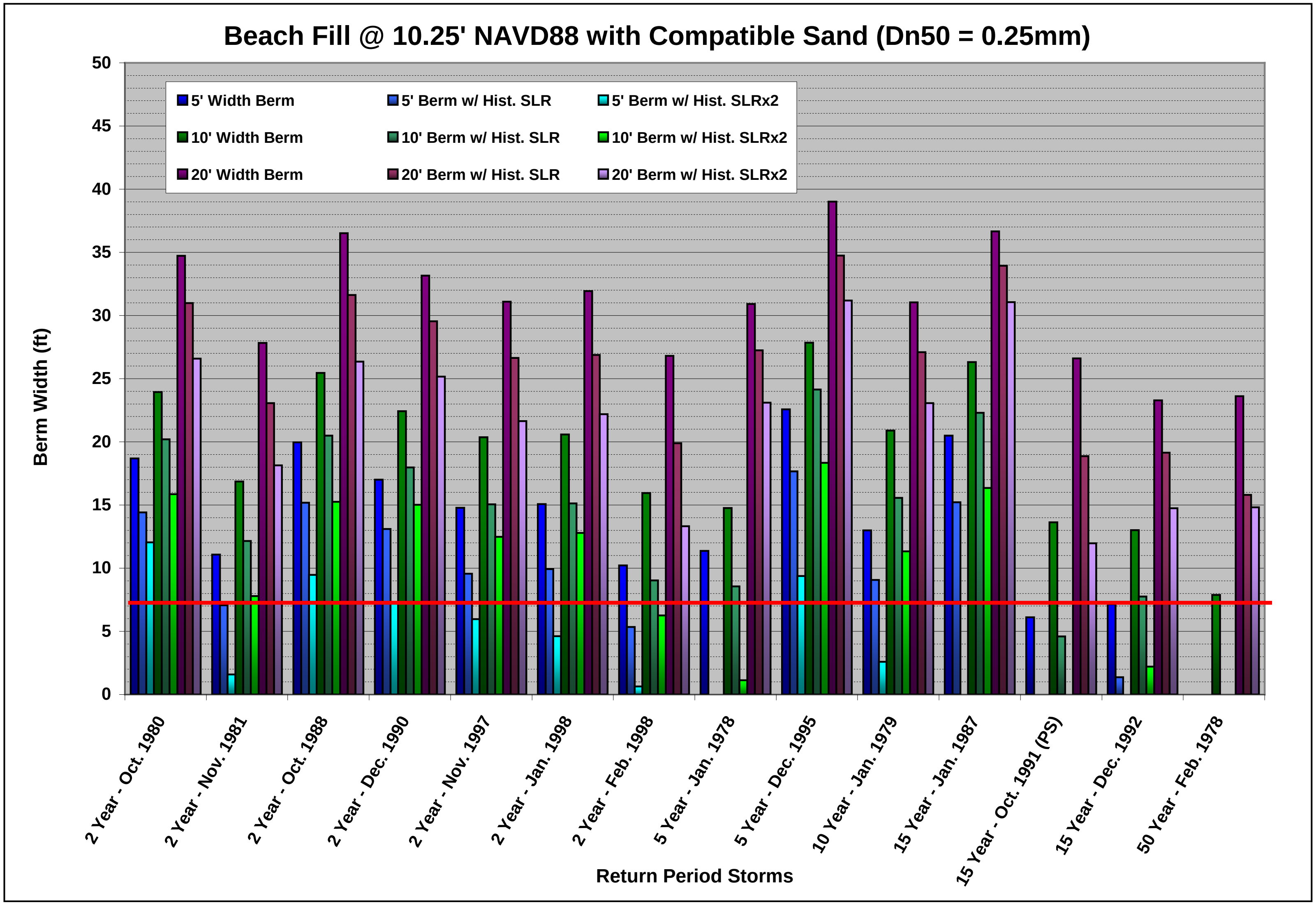


Figure 14. SLR impacts (10.25 ft-NAVD88 berm elevation and compatible sand)

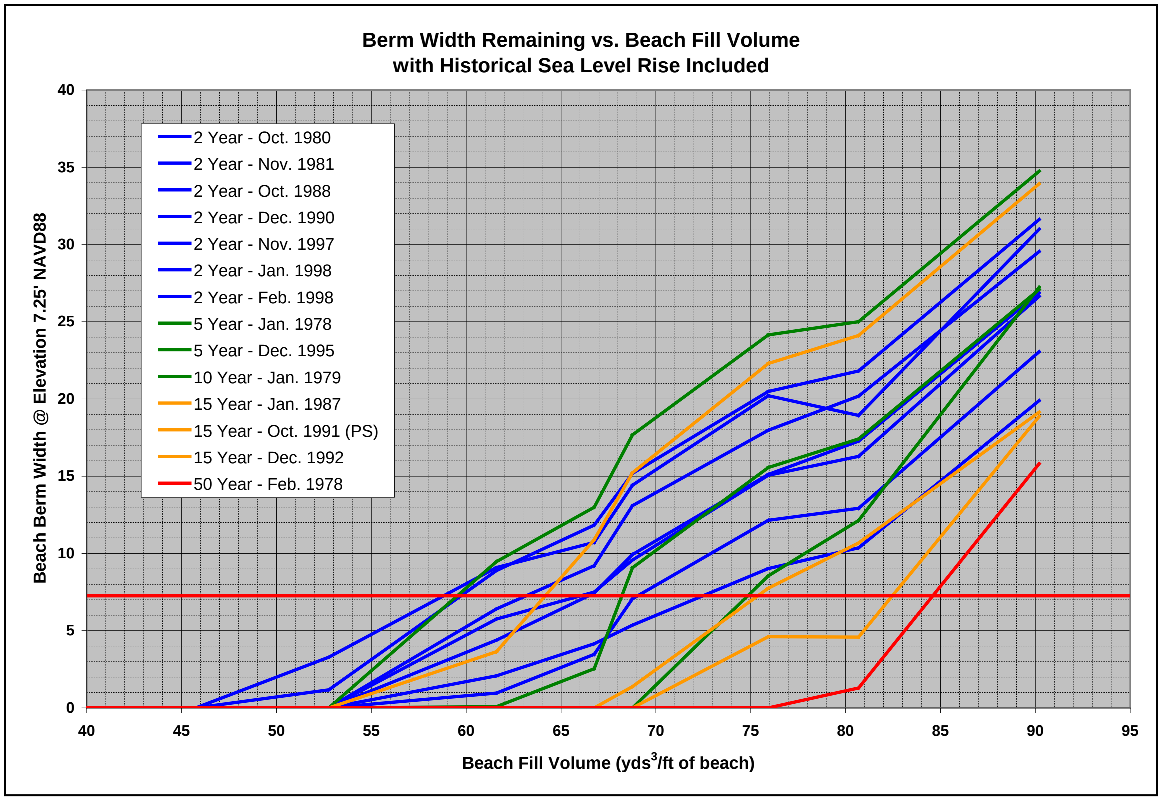


Figure 15. Min. berm width vs. fill volume with SLR (2, 5 to 10, 15, and 50 yr storm)

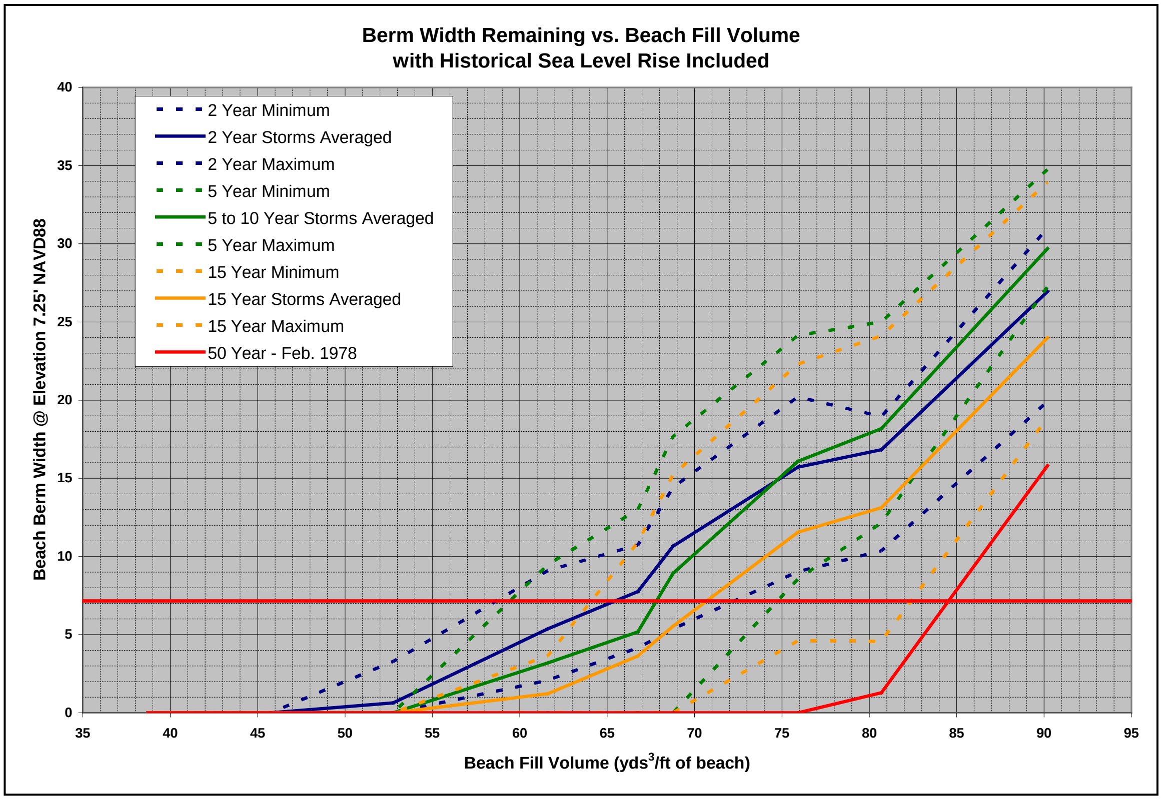


Figure 16. Min berm width vs. fill vol. with SLR (2, 5 to 10, and 15 year storm avg.)

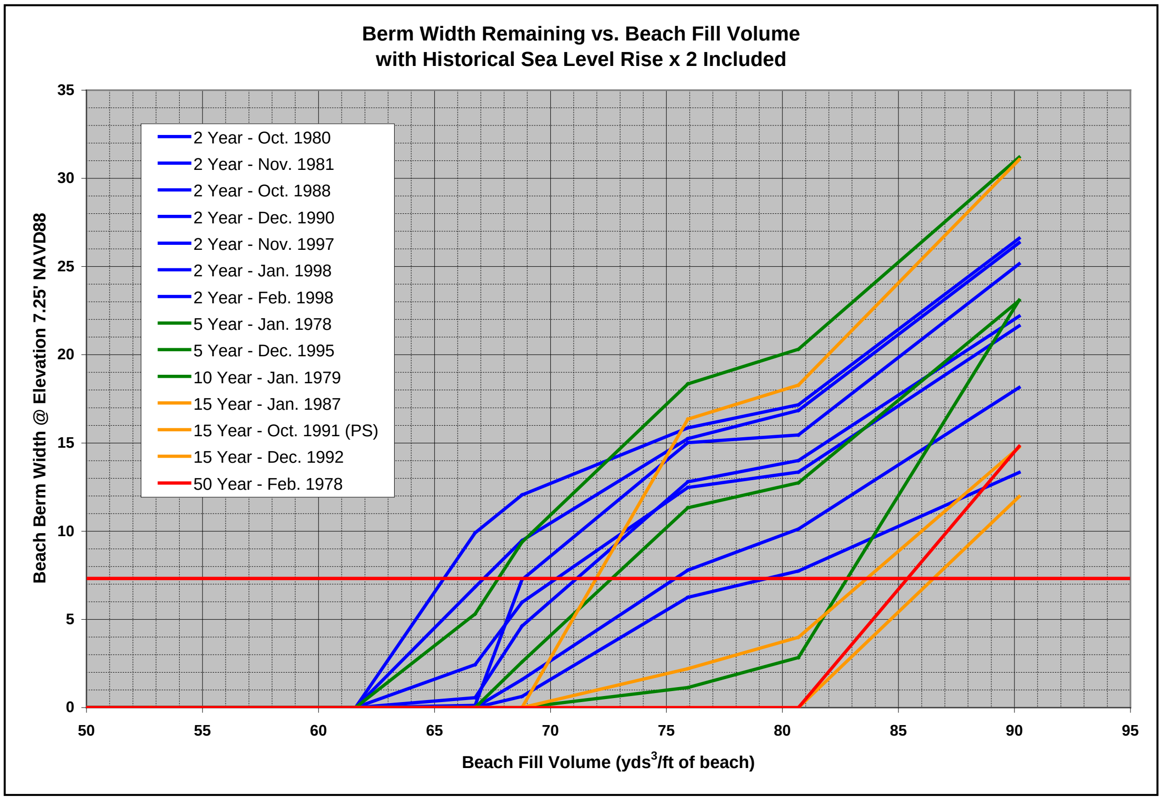


Figure 17. Min. berm width vs. fill vol. with SLRx2 (2, 5 to 10, 15, and 50 yr storm)

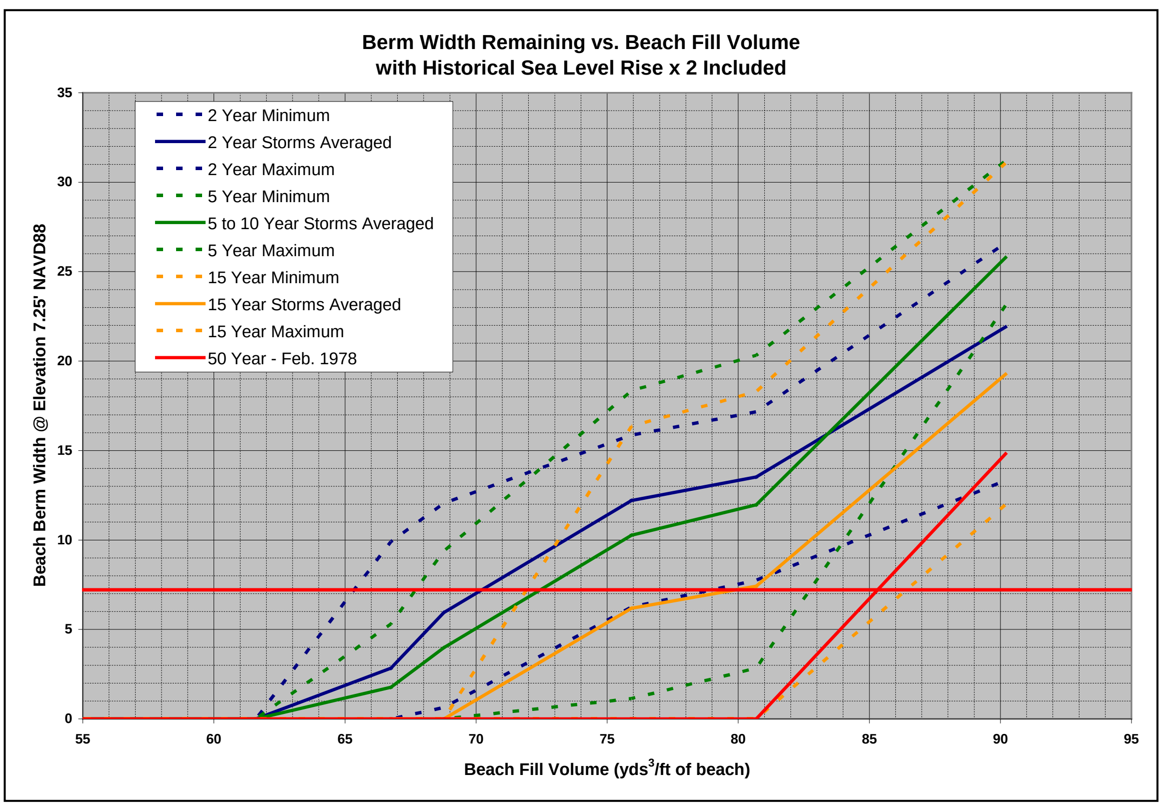


Figure 18. Min berm width vs. fill vol. with SLRx2 (2, 5 to 10, and 15 yr storm avg.)

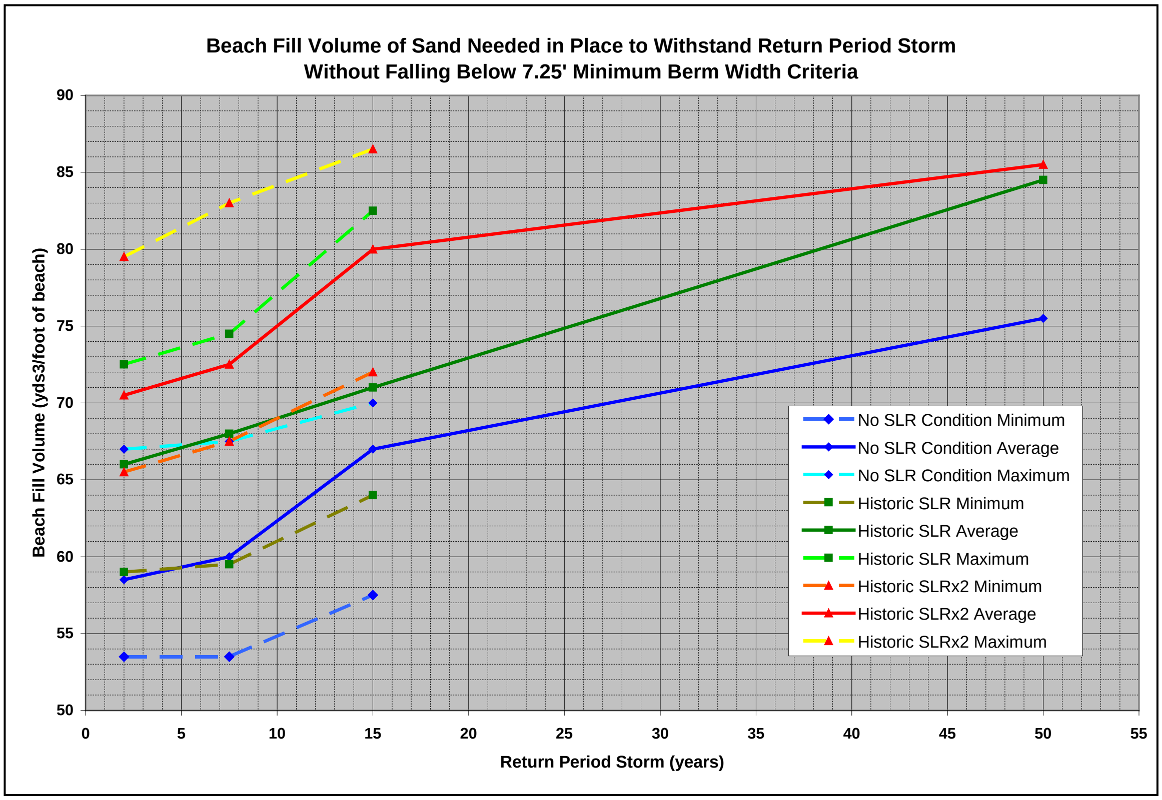


Figure 19. Summary of SLR impact on beach fill performance

6.7 <u>Grain Size Impact (use of 0.45 mm D_{n50} sand)</u>

As highlighted and discussed in the Nantasket Seawall Alternatives Analysis study, using beach fill sand that is coarser than the native beach sand is beneficial. Coarser fill results in using less sand to achieve the same dry beach berm widths as the native sand. Also, coarser sand typically performs better during storm conditions and provides better protection. Basically the coarser sand holds a steeper slope during regular conditions and it is transported less easily offshore during storm conditions. The two benefits result in requiring less fill for the same or improved storm protection which results in a lower cost for the same or improved storm protection.

Due to the potential benefits, coarser fill was examined for this study. It was decided based upon potential upland sources that a 0.45 mm D_{n50} would be used to represent a coarser beach fill. The first examination was the benefits of reducing the amount of beach fill required to achieve particular beach berm widths at both the 9.25 and 10.25 feet NAVD88. Shown in Figure 20 is a comparison for the two grains sizes and berm elevations. It can be seen that for the coarser sand the required volume of sand is significantly less to achieve the same berms widths than with the finer (native) sand. For the 9.25' berm the volume of sand needed is approximately four times less and for the 10.25' berm the volume required is almost six times less.

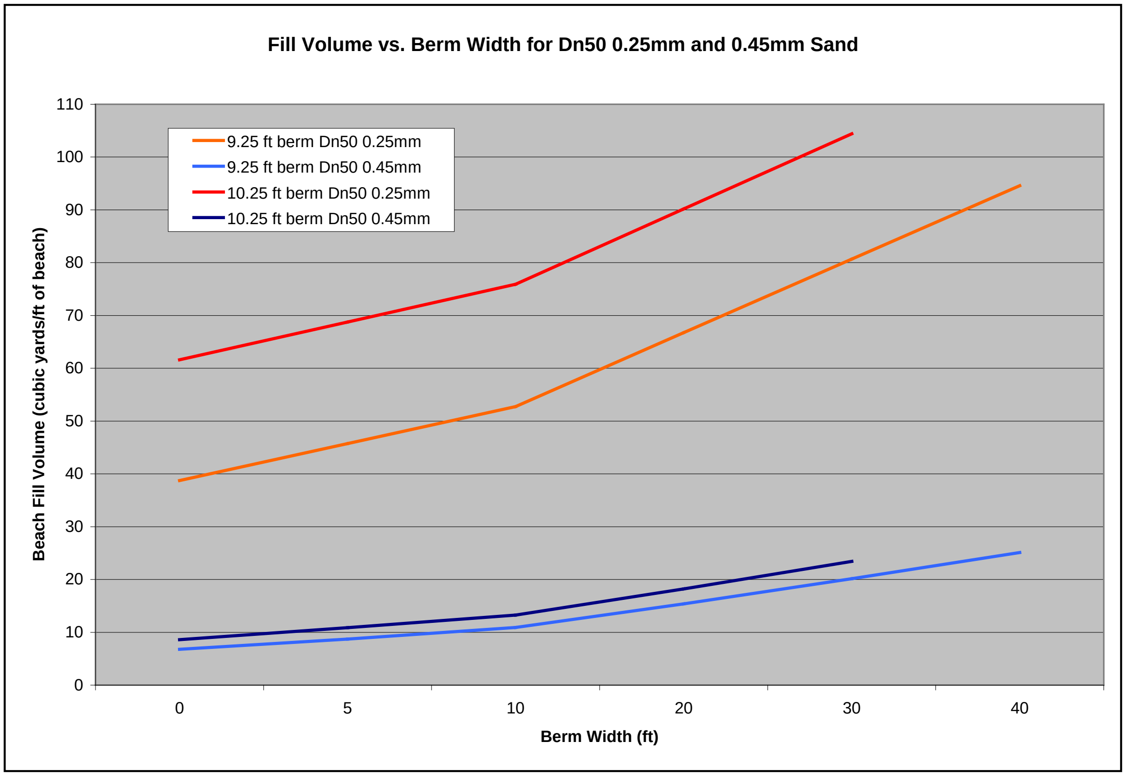


Figure 20. Beach fill volume comparison between 0.25 and 0.45mm sand

To demonstrate the improved storm performance of using a coarser grained beach fill a similar analysis was performed for the 0.45 mm sand that was performed for the 0.25 mm sand. A range of beach fill widths for the two berm elevations were run through SBEACH using the same set of storms used earlier for the 0.25 mm sand. The results of the coarser beach fill performance are shown in Figures 21 and 22. The results are plotted alongside the results of the 0.25 mm beach fill results. It can be seen that for the same beach berm widths the coarser fill performs as well or better in almost all cases. There are a few cases for the 10.25' berm where the performance is less, but it is important to keep in mind that the fill volumes are much less than for the finer grained native sand beach fill. A cautionary note must be given related to the storm performance of the coarser grained beach fill. SBEACH can model only one grain size during a given storm model run. This means that with a coarser grain size beach fill that holds a steeper slope a portion of the profile exposed during the model run will be the finer native beach that has a much shallower slope. However, SBEACH will see this sand as coarser sand which results in a model profile that contains both coarse grain and a shallow slope. This may improve the storm performance of the coarser grain size beach in the SBEACH model and overstate the performance. If a coarser grain size fill becomes a possibility for this project a closer look at the performance must be taken to better understand the true performance of a coarser beach fill. However, the results shown here and it's a known fact that using coarser grain beach fill has definite benefits for the volume of sand needed and the performance of the beach fill during a storm.

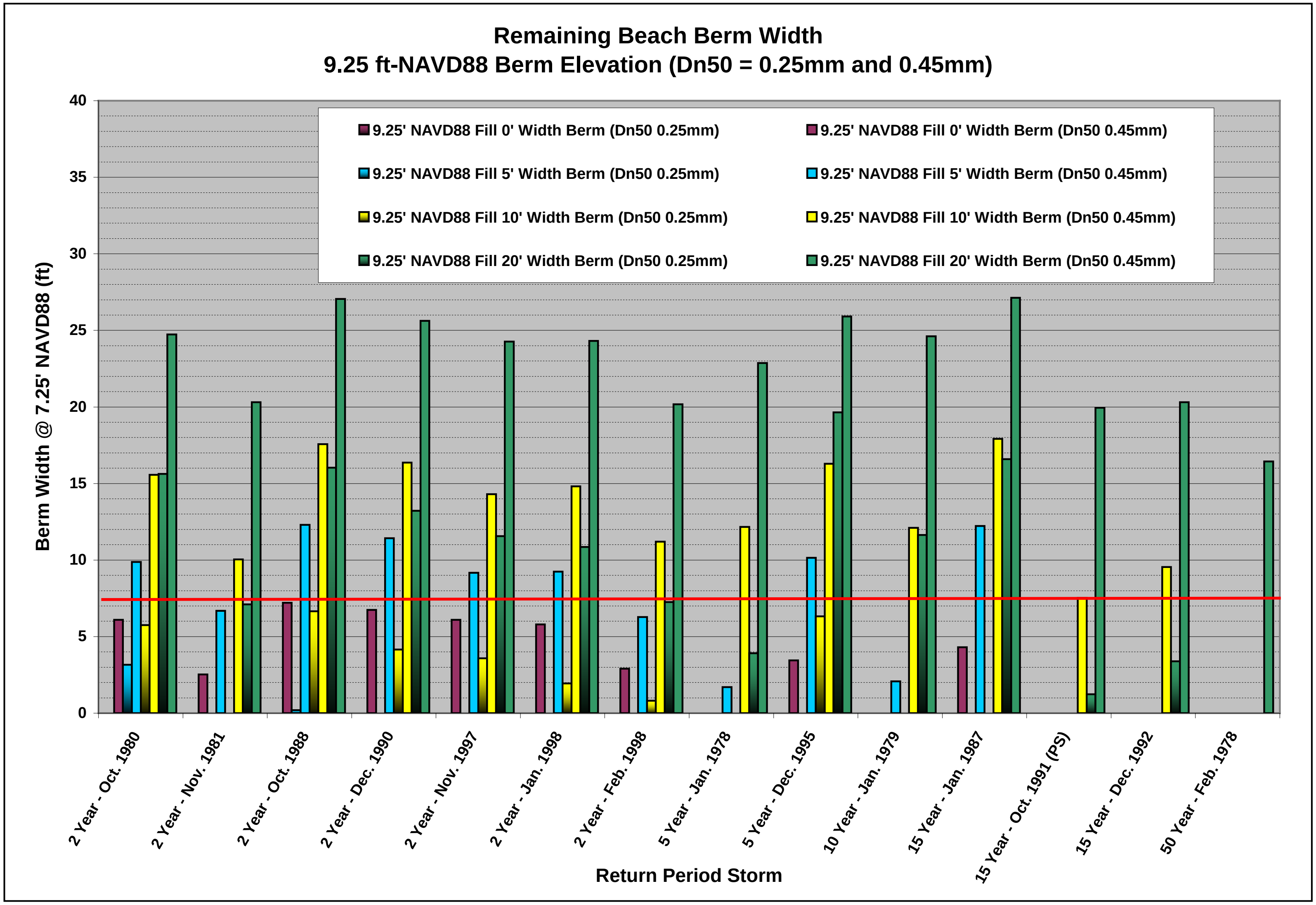


Figure 21. Beach fill storm performance comparison of 0.25 and 0.45 mm sand (9.25 ft-NAVD88 berm)

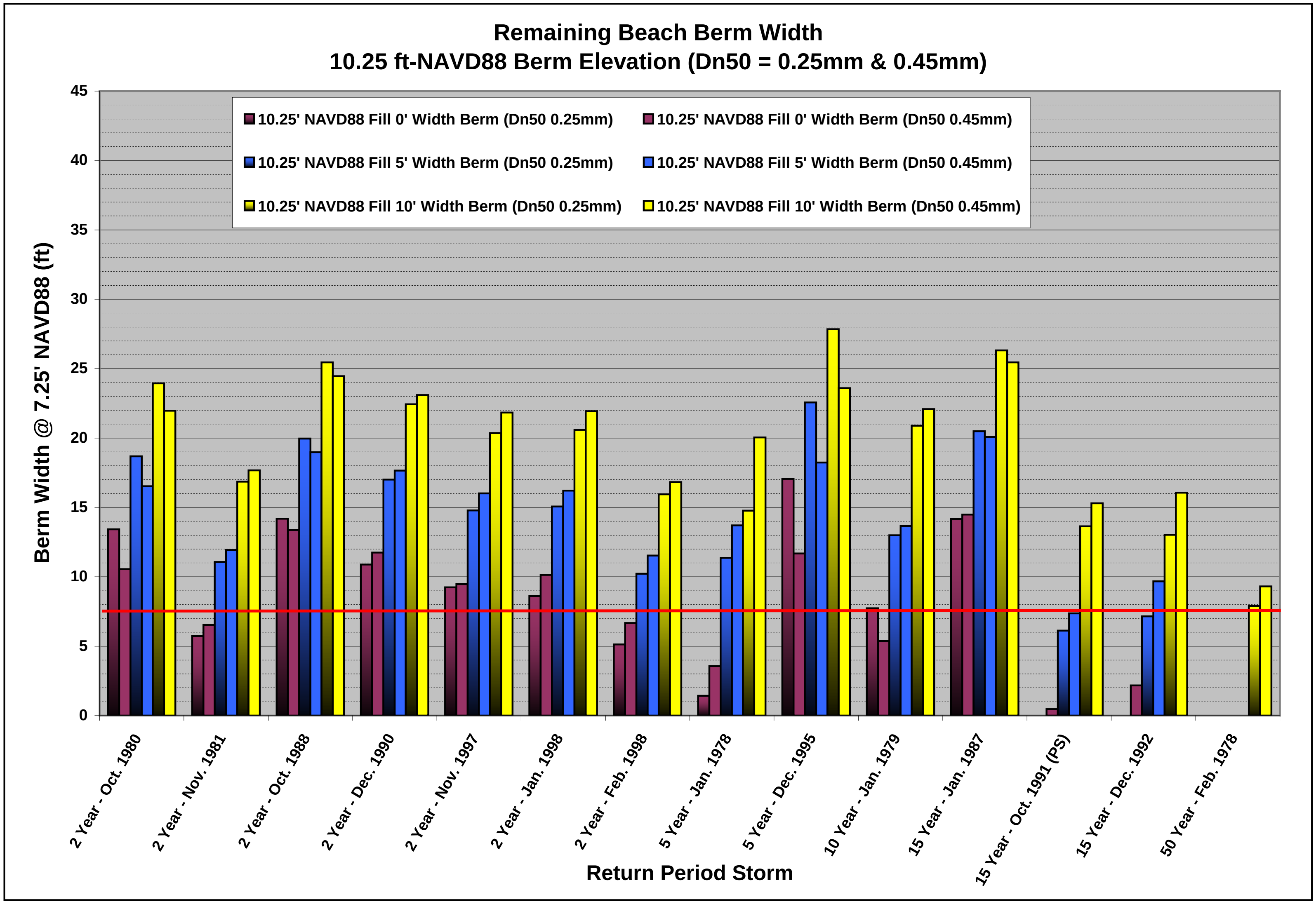


Figure 22. Beach fill storm performance comparison of 0.25 and 0.45 mm sand (10.25 ft-NAVD88 berm)

7.0 Beach Fill Longevity

Beach fill longevity depends on numerous factors and requires a fairly significant level of analysis to determine. The original plan for this study was to continue the study performed by the Engineering Research and Development Center – Coastal Hydraulic Lab (CHL) during the North Nantasket Alternatives Study. However, following the submission of the study plan a parallel contracting effort was undertaken by the Woods Hole Group from Falmouth, MA at the direction of the DCR. The study purpose was to look at many alternatives outside of the purview of the USACE study in order to address the beach erosion/seawall issues within the DRC reservation. Examples of the alternatives were complete removal of the seawall and replacing it with dunes or a stone revetment. The WHG study was commissioned following the approval processes and construction of the temporary seawall fortification at the southern end of the DCR reservation in 2004. Through close coordination with the DCR and WHG it was concluded that the WHG would be performing a detailed long shore transport model and beach fill longevity analysis for numerous alternatives and the resulting information would be suitable for use in this study. The WHG report has been included as an electronic Appendix 4 in this report.

WHG revised their beach fill longevity analysis for the Corps due to differences in the assumed fill cross section at the beginning of the beach fill longevity analysis timeline. WHG assumed the timeline began from the end of construction or when the steeper constructed slope was still present. Using monitoring data from some of their previous studies and the long shore transport model WHG developed tables similar to Table 3. The analysis performed for the Corps was very similar with the exception that the starting point of the beach's longevity analysis was after the beach transitioned to an equilibrium slope or natural slope. This meant a significant portion of the sand that was above water near the wall was already moved in the cross shore direction filling out the natural beach slope. Basically the Corps timeline started with much less dry beach in front of the wall. The Corps typically assumes that because beach fills often transition from the over steepened construction slope to an equilibrium slope fairly quickly (1 to 2 years) that for analysis purposes it is reasonable to assume the equilibrium slope as a starting point. The Corps approach is more conservative than the WHG approach and both are valid as long as the reader/user understands the implications of both starting point assumptions.

As noted the WHG beach fill longevity results that were modified for the Corps are shown in Table 3. As indicated on the table the longevity results are for a beach fill with a berm elevation of 11.25 ft-NAVD88 using 0.25 mm D_{n50} sand (compatible sand). It is understood that beach fill longevity is impacted by beach berm width and elevation and this issue was discussed with WHG. In their analysis it was found that using beach berm elevations of 10.25 ft-NAVD88 in the longevity analysis resulted in only a 1 to 2 percent difference. Therefore it was determined that the analysis for the beach berms at elevation 11.25 ft was suitable for the three berm elevations used in this study. Also, as discussed in Section 6.4 it was found in the SBEACH model that beach fill volume was the most important measurement and not berm elevation/width as far as storm performance.

To help present the results shown in Table 3, three plots were generated that include the reduction in beach fill volume with time along with the minimum beach fill volume thresholds required to protect against various return period storms (Figures 23 to 25). The three plots cover the three SLR scenarios (no SLR, historical SLR, and historical SLRx2). It must also be remembered that the thresholds provided on the plots for each return period are only the average threshold volumes taken from plots 12, 16, and 18 and as shown on those plots there are significant variations in fill volumes between maximum and minimum impact storms for each return period. It also must be remembered that only a limited number of storms were used for each return period so the statistical significance of the averages and ranges is certainly far less than ideal. What these plots show, as expected, is that following the initial fill there is a sharp decrease in fill volume within the first several years. This is largely due to the unnatural perturbation the fill causes to the shape of the beach. Material from the fill will be lost most significantly through the northern end of the project due to “end losses”. To the south this will not be as a significant issue due to the exiting headland which acts as a natural groin. (sand retention feature).

The three figures for beach fill longevity were summarized in Table 4. It can be seen that for the historical SLR case and for a beach fill density of 128.73 yds^3/ft (740,000 yds^3 beach fill for 5,750 ft of beach), the beach fill project will protect against 2 year storms (without re-nourishment) for nearly 5.5 years and against less frequent 15 year storms between 4.5 and 5 years. For the maximum fill size considered, and once again for the historical SLR case the project would protect against 2 year storms and even 15 year storms for 12+ years. Once again the sensitivity of a beach fill project to SLR was provided in Table 4. It can be seen that a higher SLR rate shortens the period of protection against the less frequent storms by a little over a year and for the higher frequency storms by one to two years.

Table 3. Beach fill longevity for various beach fill volumes

Beach Fill Longevity and Storm Protection

Grain Size = 0.25 mm, Berm Height 11.25 ft-NAVD88, 5,750 foot fill length

128.73 yds^3/ft or 257,462 yds^3 for 2,000 ft fill

Time	Material Left in DCR Res.	Voume Remaining	
Years	%	ft^3/ft of beach	ft^3/2,000 ft of beach
0	100.0%	128.73	257,462
1	84.3%	108.50	217,004
3	66.1%	85.06	170,113
5	53.6%	68.95	137,907
7	43.2%	55.64	111,283

148.23 yds^3/ft or 286,468 yds^3 for 2,000 ft fill

Time	Material Left in DCR Res.	Voume Remaining	
Years	%	ft^3/ft of beach	ft^3/2,000 ft of beach
0	100.0%	148.23	286,468
1	85.1%	126.17	243,839
3	68.6%	101.65	196,440
5	57.7%	85.57	165,380
7	49.1%	72.72	140,531
9	45.2%	66.95	129,384
10	43.0%	63.71	123,131

157.69 yds3/ft or 315,378 yds3 for 2,000 ft fill

Time	Material Left in DCR Res.	Voume Remaining	
Years	%	ft^3/ft of beach	ft^3/2,000 ft of beach
0	100.0%	157.69	315,378
1	85.6%	135.01	270,024
3	70.1%	110.50	220,995
5	60.2%	94.98	189,954
7	52.6%	82.88	165,752
9	46.0%	72.50	144,995
11	42.9%	67.71	135,429
12	42.5%	66.99	133,980

172.08 yds3/ft or 344,154 yds3 for 2,000 ft fill

Time	Material Left in DCR Res.	Voume Remaining	
Years	%	ft^3/ft of beach	ft^3/2,000 ft of beach
0	100.0%	172.08	344,154
1	85.3%	146.76	293,515
3	69.7%	120.01	240,013
5	59.9%	103.07	206,139
7	52.2%	89.87	179,728
9	45.6%	78.54	157,078
11	42.6%	73.32	146,639
12	42.1%	72.53	145,058

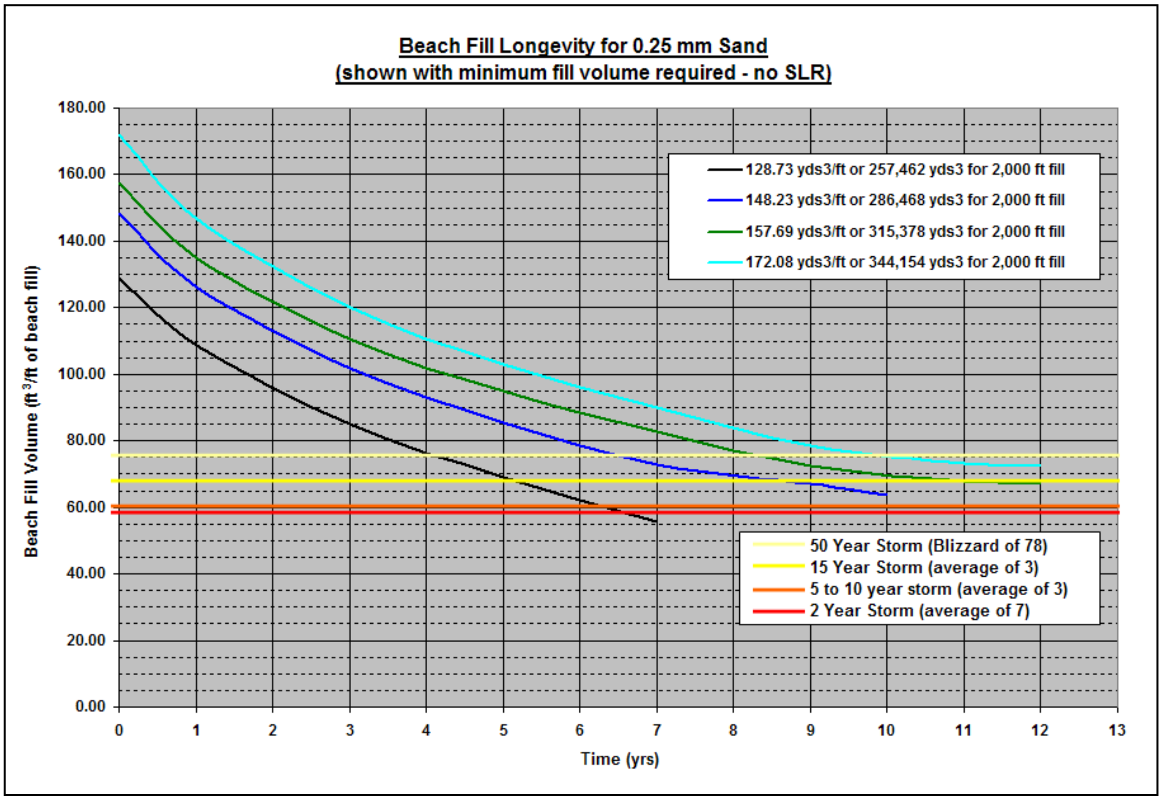


Figure 23. Beach fill longevity vs. beach fill volume (no SLR)

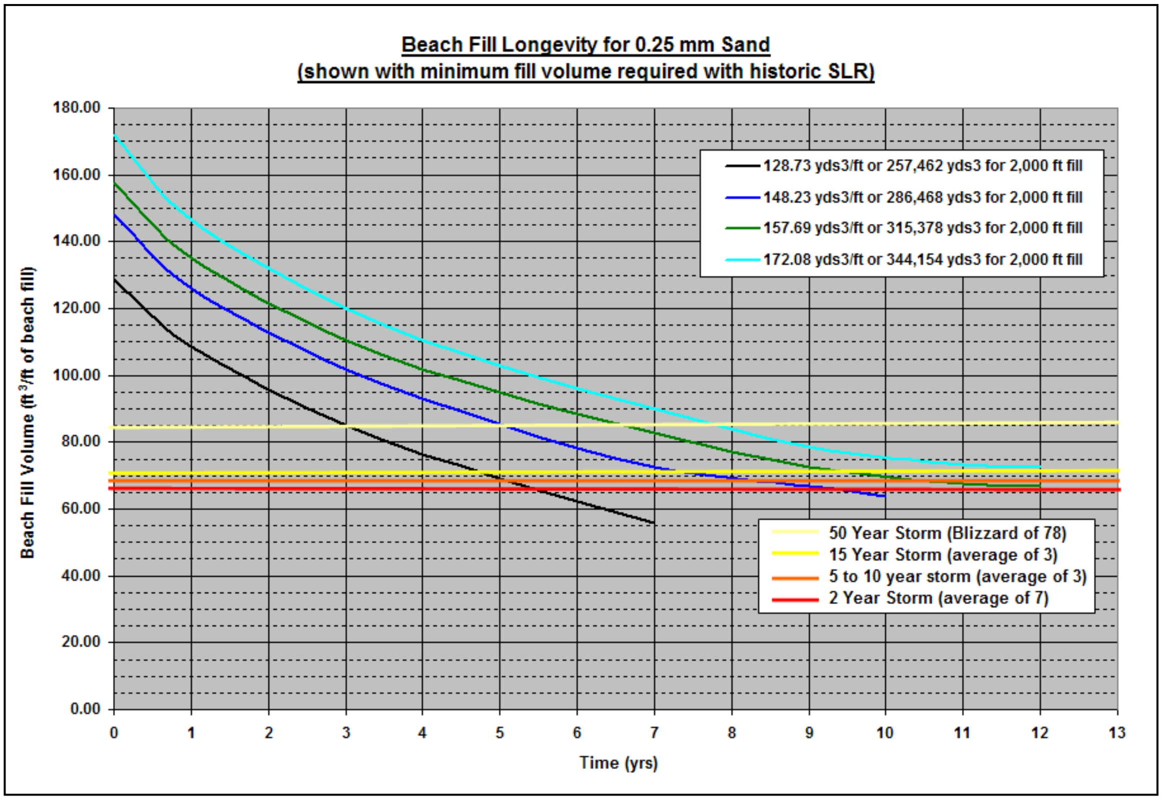


Figure 24. Beach fill longevity vs. beach fill volume (hist. SLR)

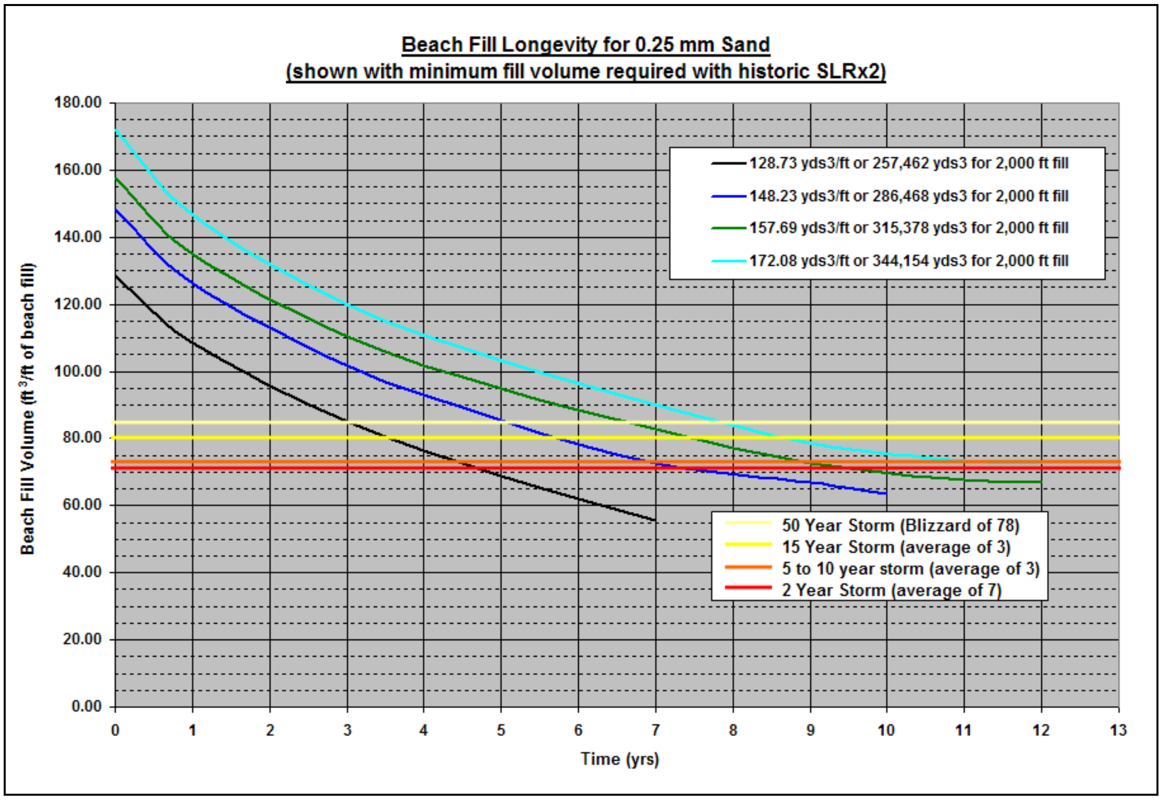


Figure 25. Beach fill longevity vs. beach fill volume (hist. SLRx2)

Table 4. Duration of protection for various beach fill volumes (summary)

Beach Fill	No SLR (Return Event - Years)				Hist. SLR (Return Event - Years)				2 X Hist. SLR (Return Event - Years)			
yds³/ft	2	5 to 10	15	50	2	5 to 10	15	50	2	5 to 10	15	50
128.73	6.5	6.3	5.2	4.1	5.4	5.1	4.7	3	4.7	4.4	3.5	3
148.23	10+	10+	8.5	6.5	9.4	8.5	7.3	5.1	7.4	6.9	5.7	5.2
157.69	12+	12+	10.8	8.4	12+	10.5	9.3	6.6	9.5	8.8	7.5	6.7
172.08	12+	12+	12+	10	12+	12+	12+	7.7	12+	11	8.6	7.9

8.0 Revetment Alternative

In lieu of beach fill placement a stone revetment was proposed to afford protection to the seawall and its footing, provide rotational and tilting stability to the seawall, and to reduce overtopping by absorbing and dissipating wave energy. The cross sectional design of the revetment alternatives are shown in Figure 26 and were developed by the Geotechnical Section. The design details can be found in Appendix E of the Main Report. Return period water level and depth limited wave height information was taken from the Coastal Appendix and from previous studies. Two similar concept designs were developed that feature a buried toe, a 1V:3H slope and an 8 foot wide crest between the top of the slope and the seawall face. The two plans were designed using the 10 and 25 year return period wave heights for the stable stone size calculation. Using the 25 year return period wave height to calculate the stable stone size resulted in a heavier stone which would be expected to reduce the frequency of maintenance required over the project life. The differences in crest width and armor layer thickness are minimal between the two stone size alternatives so only one cross section has been provided.

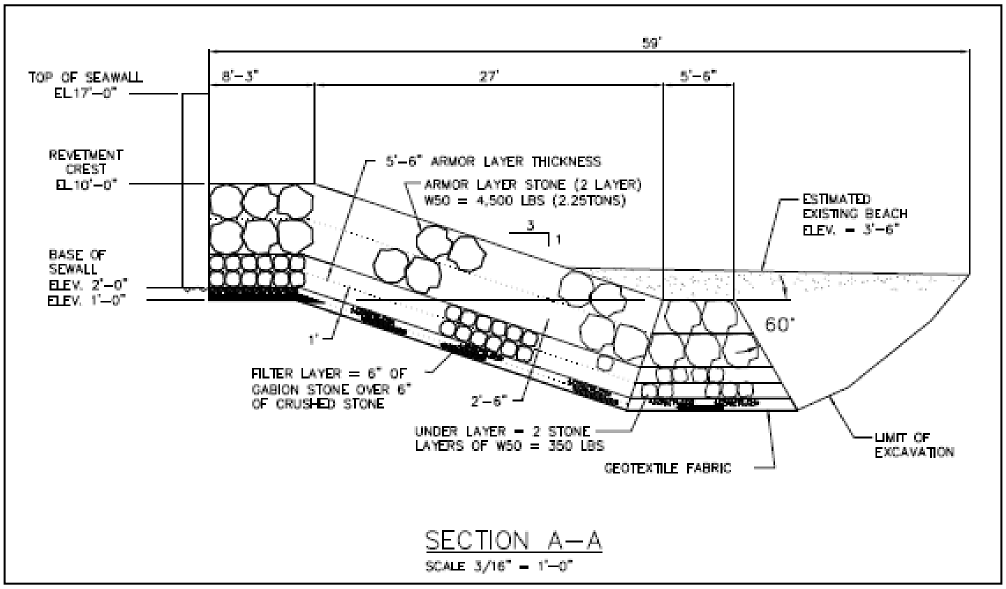


Figure 26. Revetment cross section

The revetment alternatives will provide similar benefits as the beach fill alternatives as far as protecting/stabilizing the wall but the overtopping benefits were found to be less as discussed in Sections 9.3 and 9.4. Also, there would be no recreational benefits for the revetment alternative.

9.0 Overtopping Analysis

Overtopping was analyzed using information from both the previous Feasibility Report (96 and 2001) as well as new information developed in this update effort. The information from the previous feasibility report was used for the existing conditions overtopping and subsequent flood levels behind the wall. The overtopping formulation used was the USACE program ACES and these overtopping volumes were then related to interior flood elevations. The interior flood elevations were developed using FEMA cross sections, field validation, and a surface flow analysis. Exact details of the flow analysis could not be located in the project file and were not provided in the Feasibility Report. For this reason, and the magnitude of trying to reproduce the previous work, the existing conditions overtopping/flooding analysis was used. Once again this was done due to the complexities of the analysis and the inability to locate/reproduce the information to a degree that would allow the newer overtopping formulations available today (and used for with project conditions) to be easily incorporated, or in manner that was cohesive.

The with-project conditions utilized a different overtopping formulation than the previous Feasibility Report. This formulation was taken from the much newer EurOtop Manual and the Corps' Coastal Engineering Manual (CEM) which contained overtopping formulations that better represented the site conditions of the project. Utilizing the different overtopping formulations is less than ideal but was necessary. However, as will be discussed in following Sections this issue is most likely insignificant in the context of this analysis.

9.1 Overtopping - Existing Conditions

The discussion related to overtopping for existing conditions will be kept relatively brief. As mentioned the analysis comprised of developing peak overtopping rates using the ACES software package for various idealized return period storms and then relating those peak overtopping rates to the interior flood levels. In the project area of interest the topography is relatively flat with water flowing generally from east to west to Hingham Bay through surface flow and surface drainage. The flood level elevations were referenced to water surface elevation above street level. The draft report prepared in January 2000 discussed only briefly a surface flow analysis of some type that utilized FEMA cross sections, assumed Manning's roughness coefficients, and field data verification during the 1992 Nor'easter. The result was a series of tables showing peak overtopping rates for various storms and several plots showing return period storm frequency vs. street flood water elevation. The analysis was conducted for three hydrologic zones that apparently were defined by surface flow/drainage pattern. For this study only Zone 2 was looked at since this is the zone directly behind the roughly 2,200 foot section of beach being considered in this effort. A map showing the hydrologic zones and the study area has been provided as Figure 27.

A significant purpose of a storm damage reduction project is to prevent the failure of the existing seawall. Given that the wall was found to be unstable versus USACE criteria and could not be certified, it was concluded after much consideration by the PDT and the New England District Levee Safety Officer that the without project condition would be a failed wall condition. If the seawall fails, significant damage will be caused by the direct impacts of erosion and also due to increased flooding during the storm. To take wall failure into consideration, overtopping rates were determined for a failed wall condition. This analysis was provided in the Feasibility Report and, based on the discussion in the report, it was done using a similar formulation as with the wall standing. Provided in Tables 5a and 5b are the overtopping volumes associated with the 2,485 feet of Zone 2 and the resulting street level flood elevations for both the wall standing and failed wall case without-project conditions. A failure of the wall along the complete length of Zone 2 is unlikely, but it is a simplifying assumption for a very complex and difficult analysis.

The data provided in Tables 5a and 5b was also plotted for easier viewing and is provided as Figure 28. Both flood elevation and overtopping volumes were plotted against return period storm.

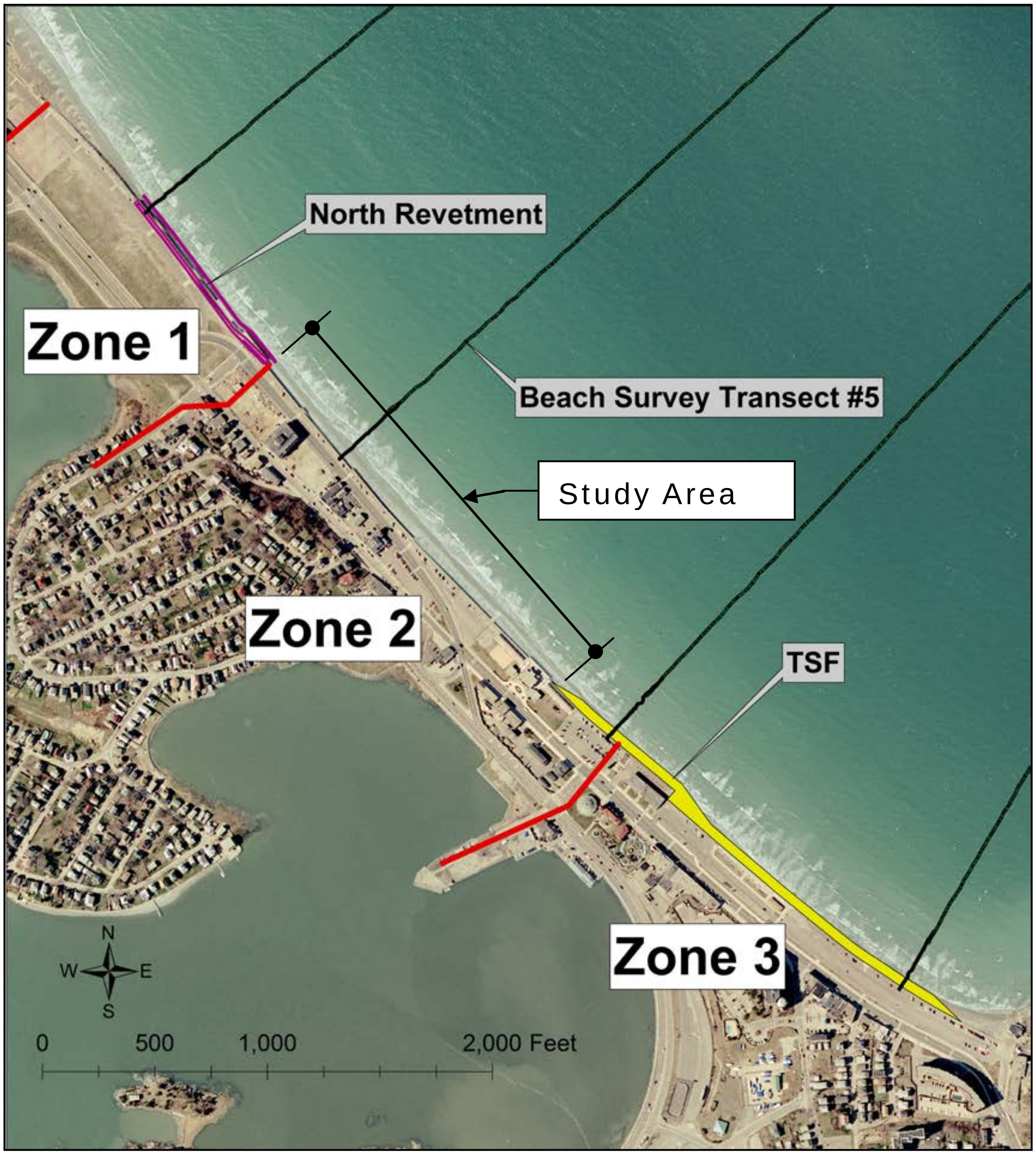


Figure 27. Hydrologic flood zones

Tables 5a and 5b. Water elevation and peak overtopping vs. return period storm

Zone 2 - Existing Conditions

Return Period	Water Level		Peak Overtopping Rate (cfs)				Water Elevation
year	ft-NGVD	ft-NAVD88	Region 4 (1450 ft)	Region 5 (50 ft)	Region 6 (1385)[1]	Total	above curb
100	10.3	9.5	6,490	280	3,349	10,119	1.65
50	10.0	9.2	5,680	250	2,744	8,674	1.50
25	9.6	8.8	4,440	210	2,048	6,698	1.35
10	9.1	8.3	2,890	150	1,315	4,355	1.05
5	8.7	7.9	1,830	100	683	2,613	0.80
2	8.2	7.4	590	50	156	796	0.35

[1] 985 feet of Region 6 are in Zone 2. The overtopping volumes from the previous work were reduced proportionately

Zone 2 - Failed Wall Condition

Return Period	Water Level		Peak Overtopping Rate (cfs)				Water Elevation
year	ft-NGVD	ft-NAVD88	Region 4 (1450 ft)	Region 5 (50 ft)	Region 6 (1385)[1]	Total	above curb
100	10.3	9.5	8,510	280	4,558	13,348	1.90
50	10.0	9.2	7,910	250	4,053	12,213	1.80
25	9.6	8.8	7,070	210	3,534	10,814	1.70
10	9.1	8.3	5,640	150	2,823	8,613	1.55
5	8.7	7.9	4,710	100	2,147	6,957	1.40
2	8.2	7.4	3,330	50	1,465	4,845	1.20

[1] 985 feet of Region 6 are in Zone 2. The overtopping volumes from the previous work were reduced proportionately

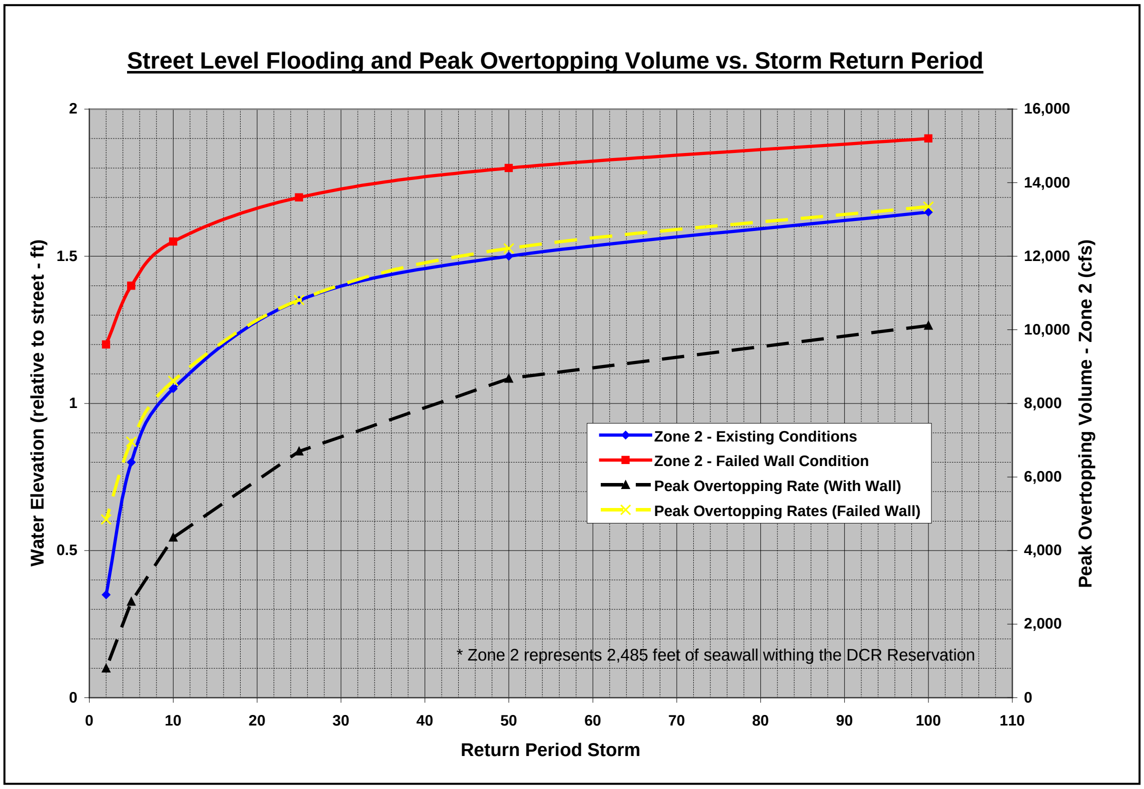


Figure 28. Street flooding and peak overtopping volumes vs. storm return period

9.2 <u>Overtopping - With Project Beach Fill Alternatives</u>

The overtopping rates were calculated for the with-project beach fill conditions using the more recent overtopping formulations provided in the EurOtop Overtopping Manual. This is appropriate because there are significant differences in the alternatives considered in this report versus previous studies. The equations and formulations used are shown below. Based on the project cross section the overtopping formulas related to the cross section shown in Figure 29, which is Figure 7.6 in the EurOtop Manual, were chosen for this effort.

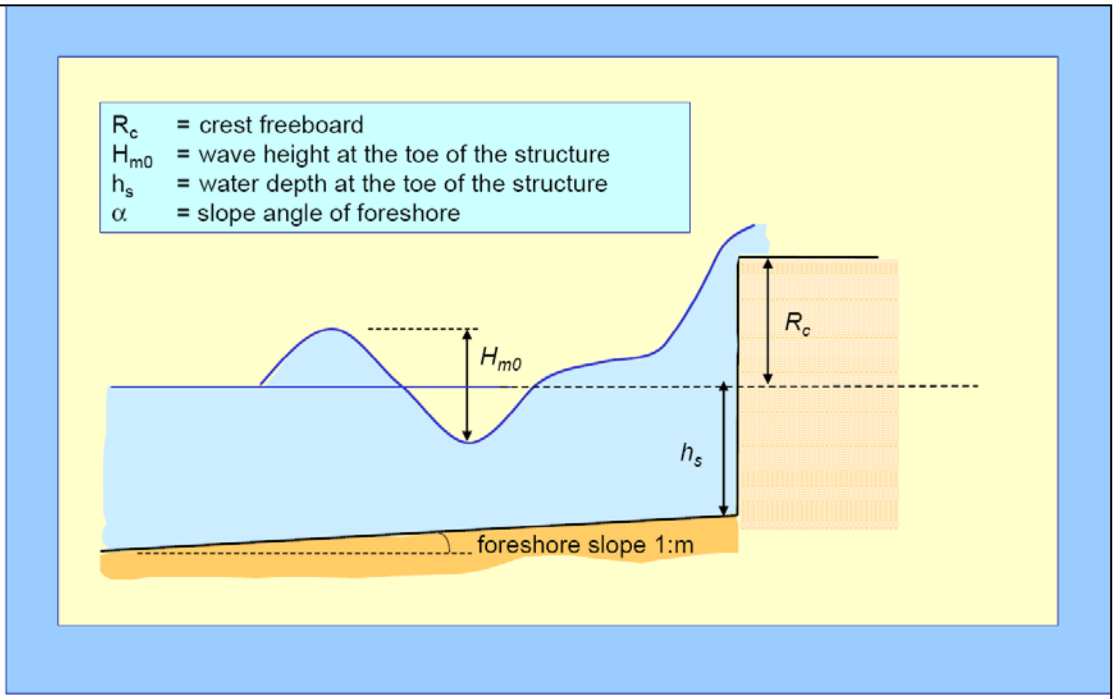


Figure 7.6: Definition sketch for assessment of overtopping at plain vertical walls

Figure 29. Cross section for overtopping of vertical seawalls from EurOtop Manual

A critical determination for the overtopping analysis was to determine if the waves breaking against the wall were Non-Impulsive or Impulsive. This criteria was important in deciding which specific overtopping formula should be used for a vertical seawall overtopping situation. The formula for calculating Non-Impulsive vs. Impulsive waves has been provided below as Figure 30, and once again this is a snap shot taken directly from the EurOtop Manual.

This method is for distinguishing between impulsive and non-impulsive conditions at a vertical wall where the toe of the wall is submerged (h_s> 0; Figure 7.6). When the toe of the wall is emergent (h_s < 0) only broken waves reach the wall.

For submerged toes (h_s> 0), a wave breaking or "impulsiveness" parameter, h_* is defined based on depth at the toe of the wall, h_s, and incident wave conditions inshore:

$$h_* = 1.35 \frac{h_s}{H_{m0}} \frac{2\pi h_s}{gT^2_{m-1,0}} \qquad 7.1$$

Non-impulsive (pulsating) conditions dominate at the wall when h_* > 0.3, and impulsive conditions occur when h_* < 0.2. The transition between conditions for which the overtopping response is dominated by breaking and non-breaking waves lies over 0.2 ≤ h_* ≤ 0.3. In this region, overtopping should be predicted for both non-impulsive and impulsive conditions, and the larger value assumed.

Figure 30. Formula for calculating Impulsive vs. Non-Impulsive waves (EurOtop Manual)

Once the wave breaking/overtopping conditions were determined using the above calculation the actual overtopping rate was calculated. For the project area the waves were determined to be impulsive and this was due to the relatively shallow beach and breaking wave conditions in front of and at the wall. The formula used for calculating the overtopping rates has been provided below as Figure 31 and it was taken as a snapshot from the EurOtop Manual.

Deterministic design or safety assessment, impulsive conditions (h_* ≤ 0.2): For deterministic design or safety assessment, the following equation incorporates a factor of safety of one standard deviation above the mean prediction:

$$\frac{q}{h_*^2 \sqrt{g h_s^3}} = 2.8 \times 10^{-4} \left(h_* \frac{R_c}{H_{m0}} \right)^{-3.1} \quad \text{valid over } 0.03 < h_* \frac{R_c}{H_{m0}} < 1.0 \qquad 7.7$$

Figure 31. Overtopping formula for Impulsive wave conditions (EurOtop Manual)

The actual calculations were done using software provided on the HR Wallingford web page (http://www.overtopping-manual.com/) that automates the calculations. This allows the user to choose the most closely fitting cross section to their project and then simply enter the project specific variables. As always the user must be familiar with the internal formulations to ensure proper application of the tool. Screen shots of the web page at the critical selection points have been provided for further documentation as Figures 32 and 33. The software was used for efficiency and to reduce the chance of calculation error.

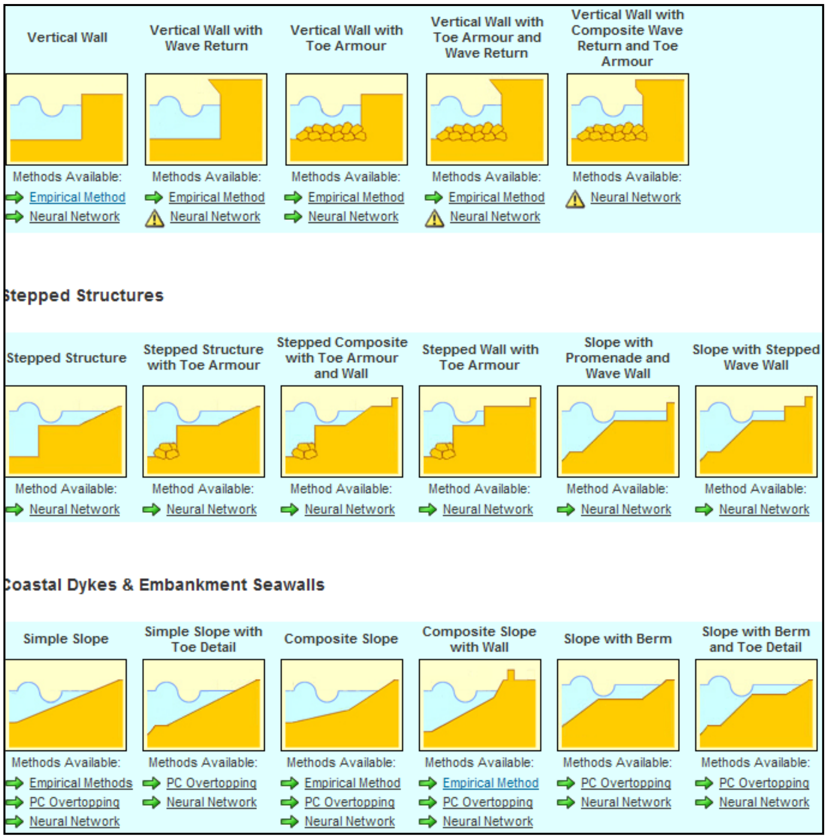


Figure 32. Cross section condition selection (EurOtop Manual - Calculation Tool)

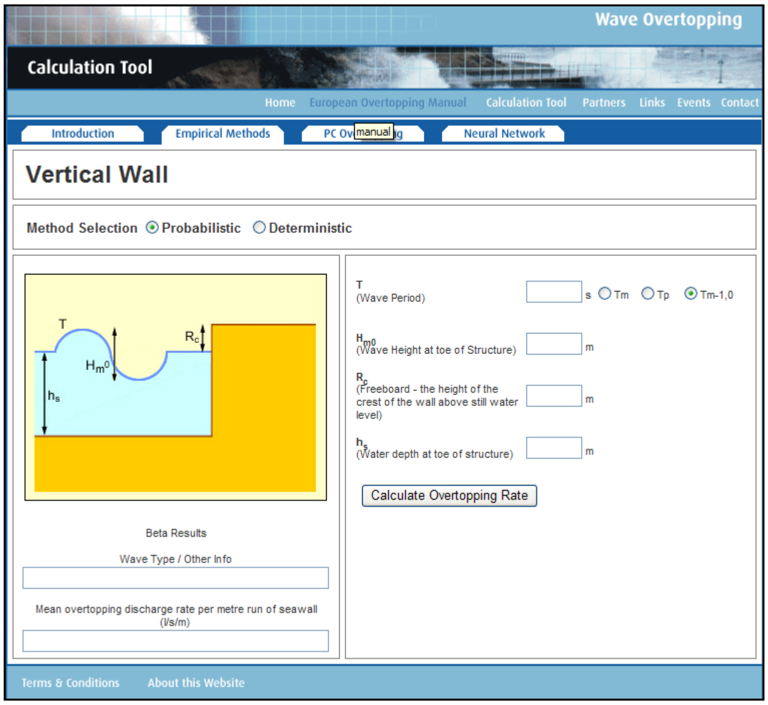


Figure 33. Project specific data entry (EurOtop Manual - Calculation Tool)

Given that a beach fill project is a dynamic form of protection that will decrease in effectiveness with time due to erosion, determining when in the project life to calculate overtopping damages was a major factor that was considered. The decision process for when to calculate overtopping is provided in the following discussion. As discussed in previous sections a significant reason the beach fill project is needed is to prevent the failure of the seawall. As shown in Figures 27 the project area is the only portion of the seawall not protected by a stone revetment. Additionally, the beach fill analysis and life cycle analysis was based on the need to have at least the minimum volume of sand in front of the wall so that it would not collapse during a design event. When the beach fill is initially placed the level of protection far exceeds the calculated minimum level, but as time progresses the beach fill loses sediment (as shown in Figures 23 to 25) and the beach fill level of protection moves closer to the minimal level of protection. If the beach fill project is allowed to erode beyond that point the seawall will be at risk for failure if the design storm hits. These key levels of protection for various levels of return period storm are shown in Figures 23 to 25. Once the beach fill project reaches the minimum acceptable level it is assumed a beach fill re-nourishment project would be completed. Given that the beach fill in front of the wall should not be allowed to pass this minimum level it was reasoned that this should be the worst case scenario for overtopping (smallest allowable beach in front of the seawall). Basically the beach fill project will be at its lowest and most susceptible level of protection against overtopping. Using this reasoning, overtopping was calculated for the return period storms analyzed in the beach fill design analysis with the beach cross section representing the minimum beach necessary to protect against those return period storms. A summary of the analysis has been provided in Table 6. In Table 6 the necessary beach and storm parameters for the overtopping formulation discussed above were summarized from the numerous SBEACH runs used for the beach fill design. The information from Table 6 was used in the automated EurOtop formulations. As can be seen in Table 6 an overtopping analysis was done for each return period case. To help simplify the effort, averages of the storm parameters and the fronting beach parameters were taken and then this info was used in the overtopping calculation.

As can be seen in Table 6 and presented in Figure 34, the overtopping rate for the with-project conditions for the various return periods are all low when compared to the without-project conditions and the failed wall conditions. All but the 50 year storm experienced overtopping volumes that were inconsequential and that would result in no interior flooding. The 50 year storm overtopping was certainly more significant but still much smaller than the without project conditions. As shown in Figure 34 and Tables 5a and 6, the overtopping volume from a 50 year storm of 1,763 cfs for Zone 2 would only cause water to be a couple of tenths of a foot above street level.

A question the reader may ask, and an important point, is what about the overtopping in a situation where the beach is at a 2 year level of protection against seawall failure and a 10 year return period storm hits? The answer is obvious that the overtopping and therefore the flooding would be greater, but the overtopping for this type of scenario was not calculated since the wall would be expected to collapse. Remember the beach is at the minimum level necessary to protect against the wall from failure during a 2 year return period storm. In this situation the overtopping/flooding information from the previous Feasibility Report, which was discussed in Section 9.1, for a failed seawall condition would be used since the wall would be expected to fail. This would be the same case if the beach in place was only adequate to prevent wall failure during a 5 year storm, and a 15 year storm hits, or any situation where a storm of greater magnitude hits than the beach is able to protect the seawall against. The end result is that as along as the beach is maintained to protect against a chosen return period storm flooding due to overtopping will not be an issue. If the beach is not maintained wall failure will be the issue and the overtopping/flooding from this condition will be experienced.

Table 6. Beach and Storm Parameters for overtopping and overtopping results

2 year Storm Results (9.25' NAVD88 Beach Fill 10' Berm (Dn50 0.25mm))						
	Min. Beach Elev.	**Max Water Elevation**	**Free Board**	**Water depth @ Wall**	**Max Wave Height**	**Mean Wave Period**
Storm	@ seawall (ft-NAVD88)	@ seawall (ft-NAVD88)	wall crest-water surf. (ft)	water surf.-beach elev. @ wall (ft)	@ wall (ft)	seconds
2 Year - Nov. 1981	7.13	8.18	7.32	1.05	0.70	7.89
2 Year - Oct. 1988	7.88	8.96	6.54	1.08	0.76	8.11
2 Year - Dec. 1990	7.88	8.46	7.04	0.58	0.95	7.88
2 Year - Nov. 1997	7.80	8.34	7.16	0.54	0.80	7.91
2 Year - Jan. 1998	7.42	8.41	7.09	0.99	0.63	8.29
2 Year - Feb. 1998	7.34	9.10	6.40	1.76	0.83	8.41
Average	7.58	8.58	6.93	1.00	0.78	8.08
Average Meters			2.11	0.30	0.24	
Overtopping						
0.044	l/s/m					
0.01	l/s/ft					
0.00	cfs/ft					
1	**cfs (for length of Zone 2 @ 2,485 ft)**					
5 year Storm Results (9.25' NAVD88 Beach Fill 10' Berm (Dn50 0.25mm))						
	Min. Beach Elev.	**Max Water Elevation**	**Free Board**	**Water depth @ Wall**	**Max Wave Height**	**Mean Wave Period**
Storm	@ seawall (ft-NAVD88)	@ seawall (ft-NAVD88)	wall crest-water surf. (ft)	water surf.-beach elev. @ wall (ft)	@ wall (ft)	seconds
5 Year - Jan. 1978	6.71	10.56	4.94	3.85	2.21	12
5 Year - Dec. 1995	7.66	9.59	5.91	1.93	1.23	8.46
Average	7.185	10.075	5.425	2.89	1.72	10.23
Average (meters)			1.65	0.88	0.52	
Overtopping						
1.98	l/s/m					
0.60	l/s/ft					
0.02	cfs/ft					
53	**cfs (for length of Zone 2 @ 2,485 ft)**					
10 year Storm Results (10.25' NAVD88 Beach Fill 0' Berm (Dn50 0.25mm))						
	Min. Beach Elev.	**Max Water Elevation**	**Free Board**	**Water depth @ Wall**	**Max Wave Height**	**Mean Wave Period**
Storm	@ seawall (ft-NAVD88)	@ seawall (ft-NAVD88)	wall crest-water surf. (ft)	water surf.-beach elev. @ wall (ft)	@ wall (ft)	seconds
10 Year - Jan. 1979	7.68	11.84	3.76	4.16	2.47	11
in meters			1.15	1.27	0.75	
Overtopping						
8.84	l/s/m					
2.69	l/s/ft					
0.10	cfs/ft					
236	**cfs (for length of Zone 2 @ 2,485 ft)**					
15 year Storm Results (10.25' NAVD88 Beach Fill 5' Berm (Dn50 0.25mm))						
	Min. Beach Elev.	**Max Water Elevation**	**Free Board**	**Water depth @ Wall**	**Max Wave Height**	**Mean Wave Period**
Storm	@ seawall (ft-NAVD88)	@ seawall (ft-NAVD88)	wall crest-water surf. (ft)	water surf.-beach elev. @ wall (ft)	@ wall (ft)	seconds
15 Year - Jan. 1987	8.52	10.09	5.51	1.57	0.80	7.40
15 Year - Oct. 1991 (PS)	7.57	10.70	4.90	3.13	1.79	9.52
15 Year - Dec. 1992	7.60	10.68	4.92	3.08	1.85	10.60
Average	7.90	10.49	5.11	2.59	1.48	9.17
Average (meters)			1.56	0.79	0.45	
Overtopping						
1.06	l/s/m					
0.32	l/s/ft					
0.01	cfs/ft					
28	**cfs (for length of Zone 2 @ 2,485 ft)**					
50 year Storm Results (10.25' NAVD88 Beach Fill 10' Berm (Dn50 0.25mm))						
	Min. Beach Elev.	**Max Water Elevation**	**Free Board**	**Water depth @ Wall**	**Max Wave Height**	**Mean Wave Period**
Storm	@ seawall (ft-NAVD88)	@ seawall (ft-NAVD88)	wall crest-water surf. (ft)	water surf.-beach elev. @ wall (ft)	@ wall (ft)	seconds
50 Year - Feb. 1978	7.5	12.97	2.63	5.47	3.1	13
in meters			0.80	1.67	0.94	
Overtopping						
65.91	l/s/m					
20.09	l/s/ft					
0.71	cfs/ft					
1,763	**cfs (for length of Zone 2 @ 2,485 ft)**					

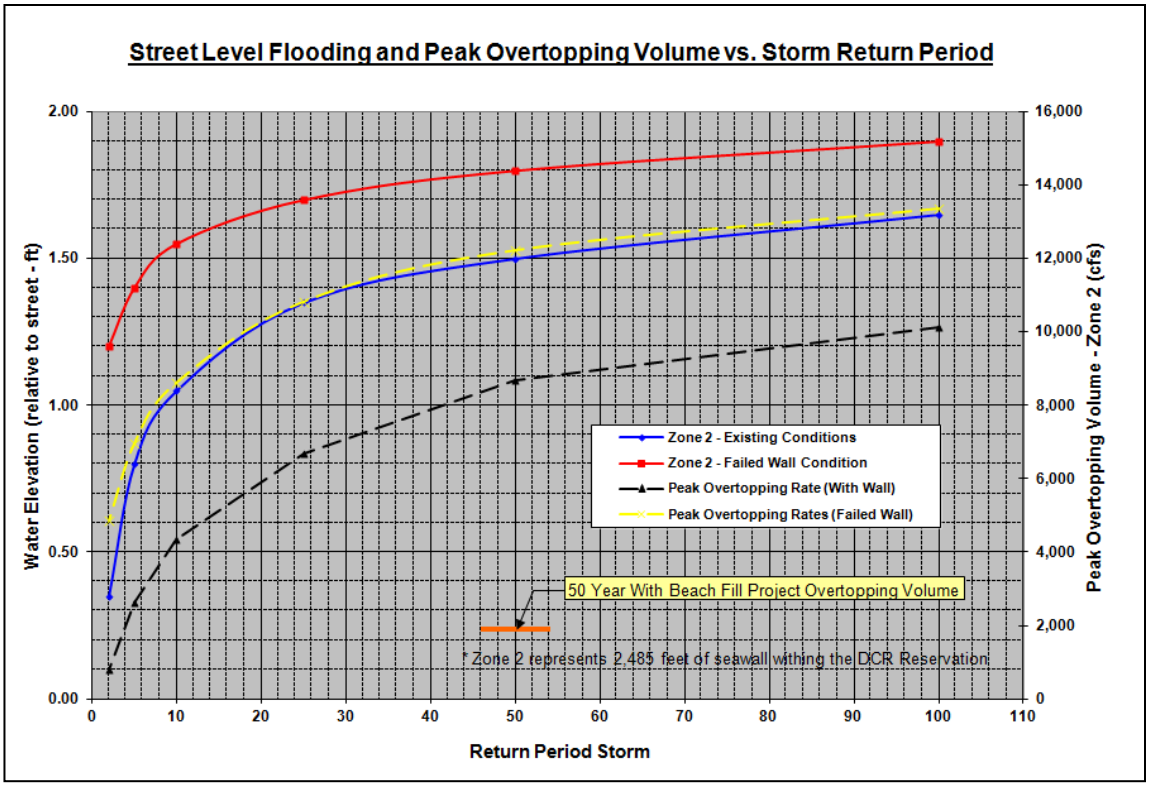


Figure 34. Street flooding and peak overtopping volumes vs. storm return period with project condition added

9.3 Overtopping – With Project Revetment Alternative

Overtopping of the revetment was performed using Equation VI-5-27 from the CEM since it was determined to be the closest match in cross section to the revetment/seawall combination in the revetment alternatives for this project. The equation is provided below and the structure cross section/test parameters used to develop the equation have been provided in Figure 35. However, due to the close proximity of the revetment crest elevation to the return period water levels, none of the overtopping equations, including the one used, were truly applicable since the design cross sectional parameters were out of the model test parameter ranges used to develop the formulas. Given no other reasonable option and the lack of funds/time to run a series of Reynolds-averaged Navier Stokes equations (RANS) models, Equation VI-5-7 was used understanding the results would be less reliable. Along these lines even Equation VI-5-7 could only be used for several of the lesser return period storm water levels since the 10 year and higher storm return periods exceeded the revetment crest elevation and fully put the equation and cross section out of the applicable range. Even if one wanted to use the equation for this condition, it could not be done since the negative value of Ac, which is the freeboard of the revetment crest to the water surface is negative and the results from the equation switch sign. For this reason only the 2 and 5 year return period storm water levels were used to determine overtopping.

$$\frac{q T_{om}}{L_{om}^2} = 3.2 \cdot 10^{-5} \left(\frac{H_s}{R_c}\right)^3 \frac{H_s^2}{A_c \, B \, \cot \alpha}$$

(VI-5-27)

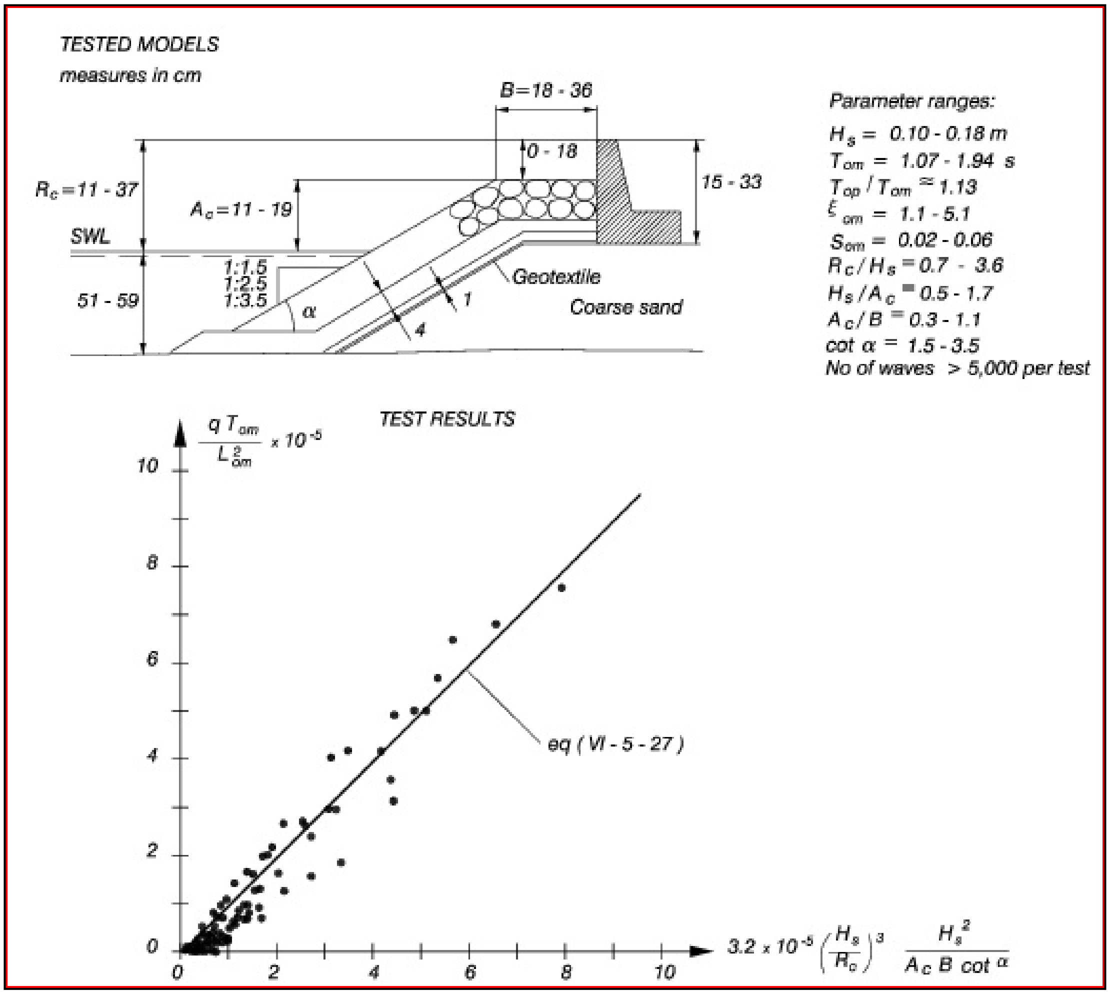


Figure 35. Revetment/seawall overtopping formula equation test parameters

Provided in Table 7 are the input conditions and resulting overtopping rates for the revetment alternative and the 2 and 5 year storm water levels. The rates are provided for the full 2,485 feet of the study/wall length as with the without project conditions and the with beach fill project conditions.

Table 7. Overtopping rates for revetment alternative (2 and 5 year storms)

2 year Storm Results Revetment Alternative

Storm	Min. Beach Elev. @ revet. (ft-NAVD88)	Max Water Elevation @ seawall (ft-NAVD88)	Free Board (Revetment) revet. crest-water surf. (ft)	Free Board (Seawall) seawall crest-water surf. (ft)	Water depth @ Wall (ft)	Max Wave Height @ wall (ft)	Mean Wave Period seconds
2 Year - Nov. 1981	1.50	7.49	2.51	8.01	5.99	4.10	7.89
2 Year - Oct. 1988	1.50	8.09	1.91	7.41	6.59	4.23	8.11
2 Year - Dec. 1990	1.50	7.55	2.45	7.95	6.05	4.43	7.88
2 Year - Nov. 1997	2.20	7.53	2.47	7.97	5.33	4.07	7.91
2 Year - Jan. 1998	2.10	7.42	2.58	8.08	5.32	4.29	8.29
2 Year - Feb. 1998	1.70	8.27	1.73	7.23	6.57	4.33	8.41
Average	1.75	7.73	2.28	7.78	5.98	4.24	8.08
Average Meters			0.69	2.37	1.82	1.29	

Overtopping

0.0021845	m^3/sec/m
0.08	ft^3/s/m
0.02	cfs/ft
58	**cfs (for length of Zone 2 @ 2,485 ft)**

5 year Storm Results (9.25' NAVD88 Beach Fill 10' Berm (Dn50 0.25mm))

Storm	Min. Beach Elev. @ revet. (ft-NAVD88)	Max Water Elevation @ seawall (ft-NAVD88)	Free Board revet. crest-water surf. (ft)	Free Board (Seawall) seawall crest-water surf. (ft)	Water depth @ Wall (ft)	Max Wave Height @ wall (ft)	Mean Wave Period seconds
5 Year - Jan. 1978	1.8	10.31	-0.31	5.19	8.51	5.07	12
5 Year - Dec. 1995	1.8	9.21	0.79	6.29	7.41	4.81	8.46
Average	1.8	9.76	0.24	5.74	7.96	4.94	10.23
Average (meters)			0.07	1.75	2.43	1.51	

Overtopping

0.24	m^3/sec/m
8.42	ft^3/s/m
2.57	cfs/ft
6,381	**cfs (for length of Zone 2 @ 2,485 ft)**

To help put the results in context, a comparison table between the three project condition overtopping results has been provided in Table 7.

Table 7. Overtopping rate comparison (Peak OT Rate CFS)

	Without Project		With Project	
Return Period Storm	**With Wall**	**Failed Wall**	**Beach Fill**	**Revetment**
2	796	4,845	1	58
5	2,613	6,957	53	6,381
10	4,355	8,613	256	
15			28	
25	6,698	10,814		
50	8,674	12,213	1,763	
100	10,119	13,348		

As shown in Table 7, the results are not very reliable nor defendable for the revetment overtopping since it shows that for the with revetment alternative the overtopping rate exceeds the without project wall standing condition. Intuitively overtopping would be reduced by the revetment since wave energy would be absorbed by the revetment. This is especially true since the revetment alternatives have a 1V:3H slope with an 8 foot wide crest. The discrepancy most likely results from the use of different overtopping rate formulations/equations for the various project conditions. While the actual numerical results are questionable qualitative conclusions can be formed. Based on the numbers provided, it appears that the revetment alternative may provide limited overtopping reduction. At first this is perhaps surprising, but given the relative proximity of the water levels to the revetment crest and the exceedence of the crest during larger storm events the qualitative results make some sense. From this information it could be generally concluded that the revetment alternative as designed will provide limited reduction in overtopping and therefore flood protection behind the wall. The real benefits of the revetment will be ensuring the seawall stays standing and therefore it will continue to provide erosion and flood protection benefits.

9.4 Sea Level Rise Impacts (SLR) on Overtopping

As ocean level rises, overtopping will increase for without-project conditions due to less freeboard or distance between the wall crest and water surface elevation. Also, larger waves will be able to make it to the seawall due to increased water depth over the existing beach. SLR was not analyzed in the previous Feasibility Report effort and was not updated in this effort due to the issues discussed in Section 8.1.

For with project conditions the overtopping would increase if the beach fill is not maintained to a prescribed level of protection against sea wall failure. Provided in Sections 6.5 and 7 are the impacts of SLR on beach fill protection levels against wall failure and beach fill longevity. Once again, as long as SLR is addressed during the project life by maintaining the beach fill project, SLR will have minimum impacts to overtopping and the resulting interior flooding.

For the revetment alternatives, if the elevation of the revetment is maintained constant along with the sea wall crest elevation, overtopping will increase with SLR. As shown in Section 9.3, overtopping rates with the revetment alternatives will be similar to the without project conditions. If the revetment is maintained at the design elevation, overtopping rates will worsen since the revetment will be further submerged during storm events. This will further reduce the limited overtopping/flood reduction benefits of the revetment. If the revetment crest and seawall crest elevations are raised to keep up with SLR then the overtopping rates will remain similar to the conditions that exist today.

10.0 Summary and Conclusions

As part of the feasibility study the Water Management Section of the New England District USACE performed a beach fill performance analysis using historical storm data along with the SBEACH storm performance model. During the study the without project conditions were investigated which included the impacts of future beach erosion and sea level rise. With project conditions were analyzed as well and included numerous beach fill alternatives along with the influences of sea level rise and beach fill sand grain size. The large quantity of information was distilled into several tables and graphs for the various future scenarios (no SLR, historical SLR, and 2x historical SLR) that demonstrated the performance in both storm protection and beach fill longevity. The results indicate that a beach fill project will provide adequate protection to the seawall and depending on the initial beach fill volume and future SLR will not have to be re-nourished for 5 to 12+ years. These results demonstrate that perhaps the impacts of SLR are not that significant for the project site for the without project condition when considering seawall stability but long term erosion would be if the rates in the future are higher than the rates measured over the last 42 years.

In addition to the beach fill alternatives, two revetment alternatives designed by the Geotechnical Section were reviewed/and analyzed. The revetment alternatives would be as effective and perhaps more effective at providing stability and protection to the existing seawall as the beach fill alternatives. The revetment plan calling for 25-year wave height stable stone size would likely require less maintence over the project life than the plan calling for 10-year wave height stable stone size.

Overtopping was also addressed in this study update. It was decided that the analyses and overtopping/flooding results provided in the previous Feasibility Report were the best to use for the without-project scenario. However, for the with-project scenario it was decided a fresh overtopping analysis was the best option. The newer overtopping formulations in the EurOtop manual and CEM were used for various with-project return period conditions and it was found that overtopping of the seawall and interior flooding will not be an issue if the beach fill project is maintained. For the revetment alternatives, overtopping will likely not be reduced significantly from the wall still standing condition due to the relatively low crest elevation of the revetment and the resulting limited freeboard between the revetment crest and the storm water surface. The real benefits of the revetment will be to keep the seawall standing and for both the revetment and seawall to provide protection levels similar to existing conditions.

Appendix 1
Nantasket Beach Characterization Study

Nantasket Beach
Characterization Stuc

Nantasket Beach Characterization Study 2005

Prepared for:

The Commonwealth of Massachusetts
Department of Conservation and Recreation (DCR)
Engineering and Construction

Prepared by:

U.S. Army Corps of Engineers
New England District

04/03/2006

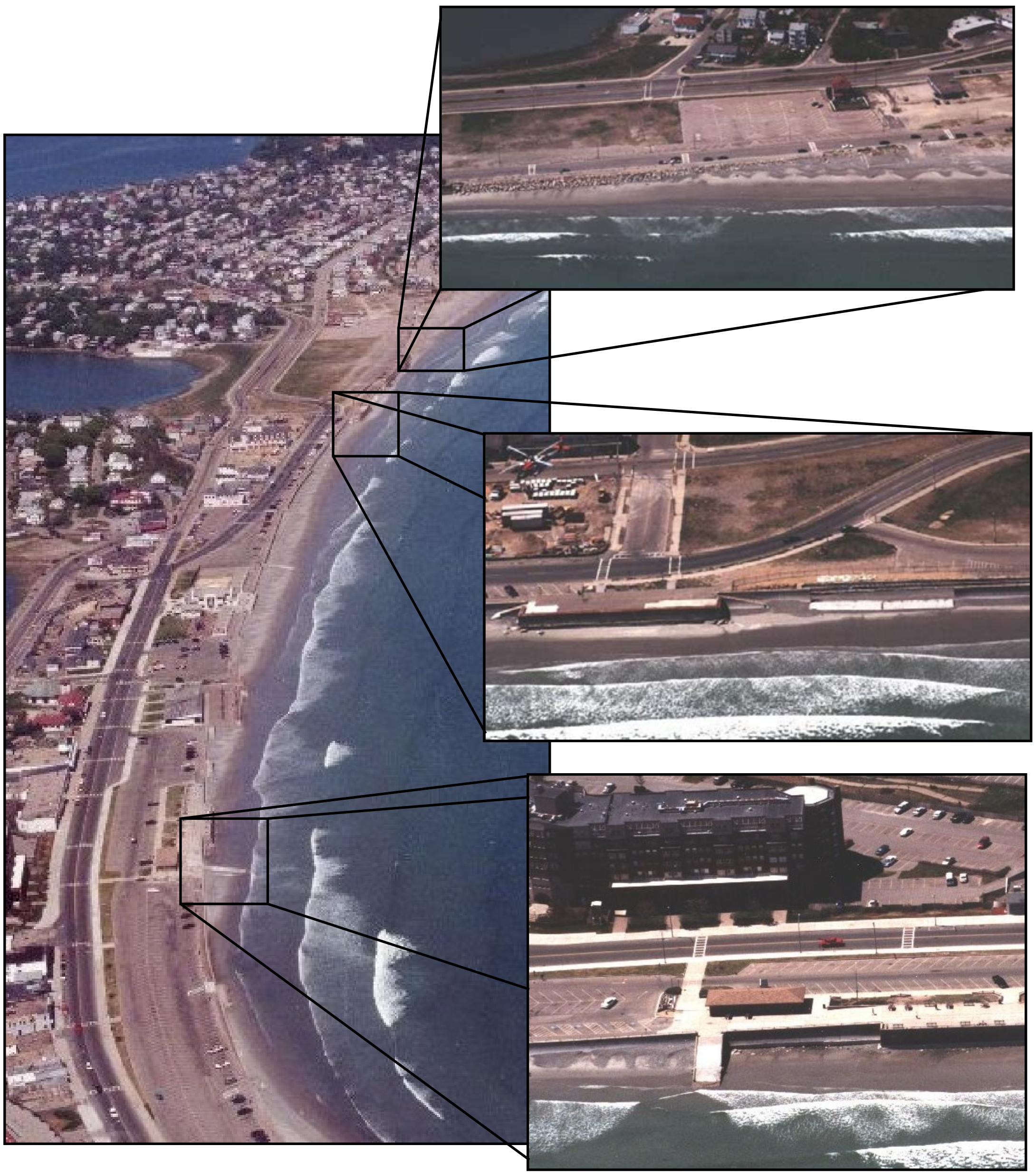
Appendix 2
Nantasket Beach Alternatives Analysis Study – Coastal Engineering Appendix

US Army Corps of Engineers®
New England District

Nantasket Beach
AAS - Coastal Engine

Nantasket Beach Alternatives/Analysis Study
Appendix A – Coastal Engineering and Processes

September 2003



Appendix 3
Storm Data

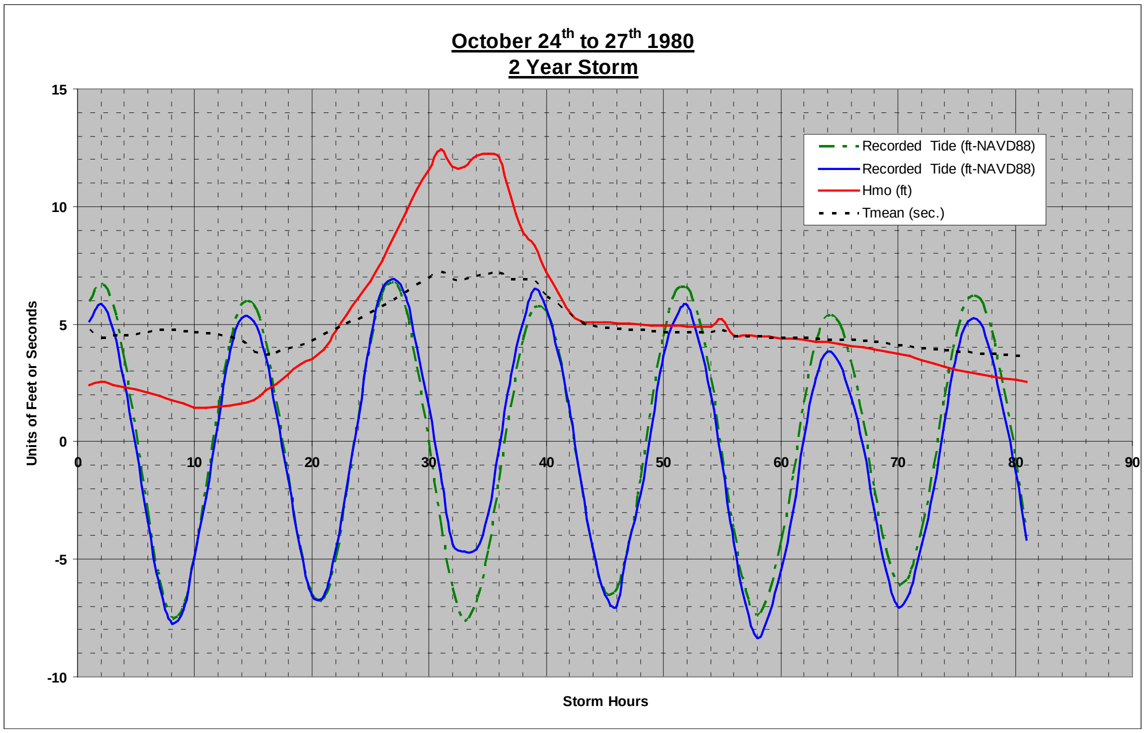


Figure A1. Storm Data 2-year storm (Oct. 24 to 27, 1980)

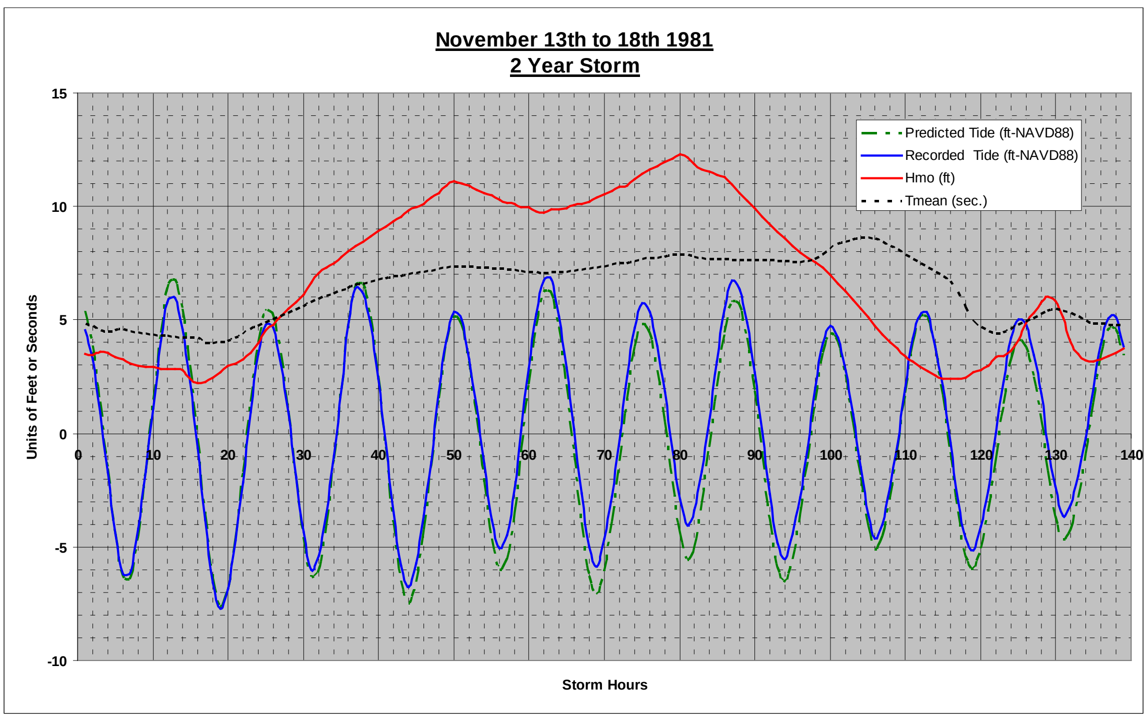


Figure A2. Storm Data 2-year storm (Nov. 13 to 18, 1981)

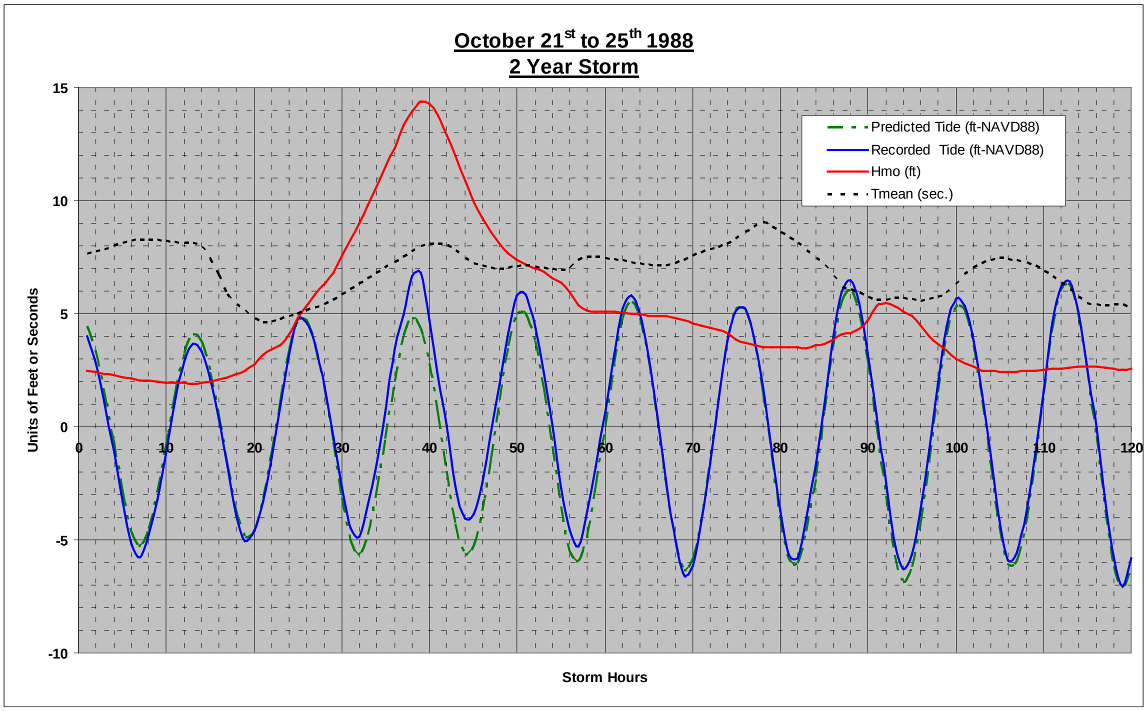


Figure A3. Storm Data 2-year storm (Oct. 21 to 25, 1988)

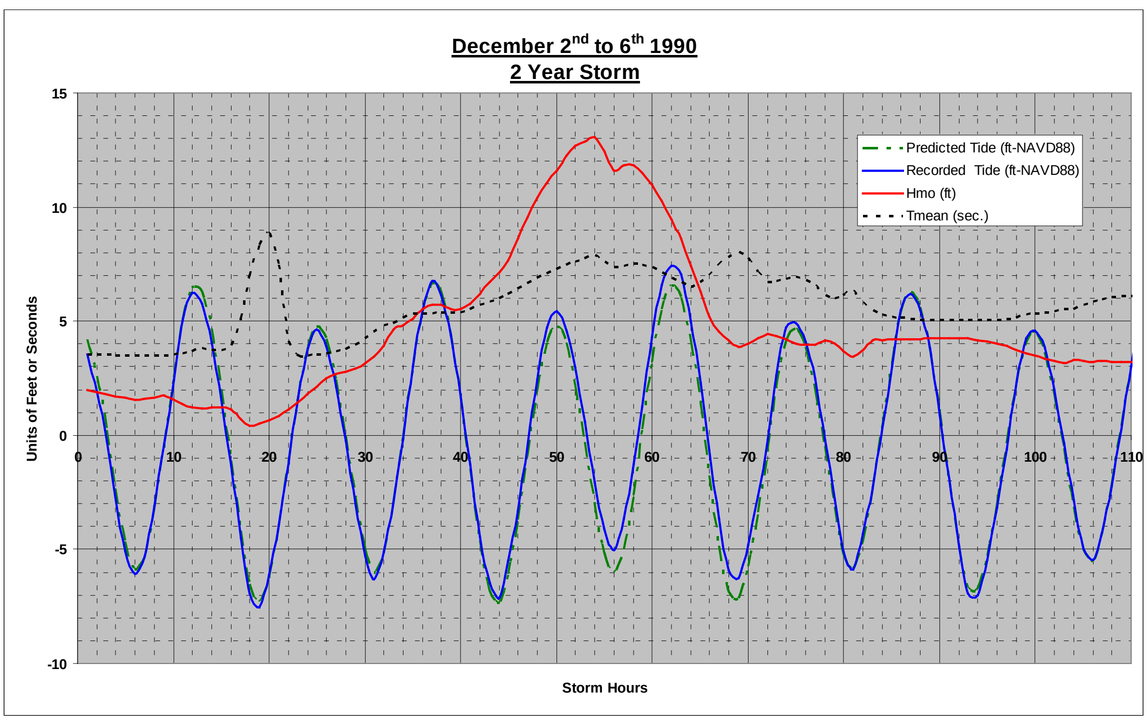


Figure A4. Storm Data 2-year storm (Dec. 2 to 6, 1990)

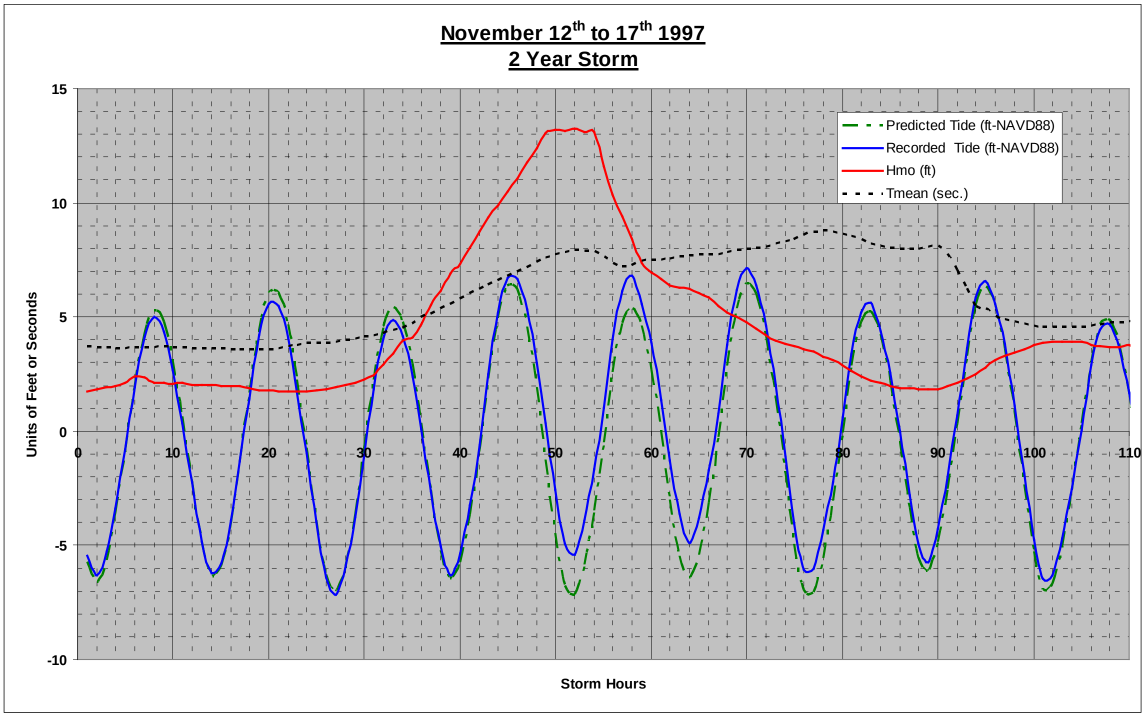


Figure A5. Storm Data 2-year storm (Nov. 12 to 17, 1997)

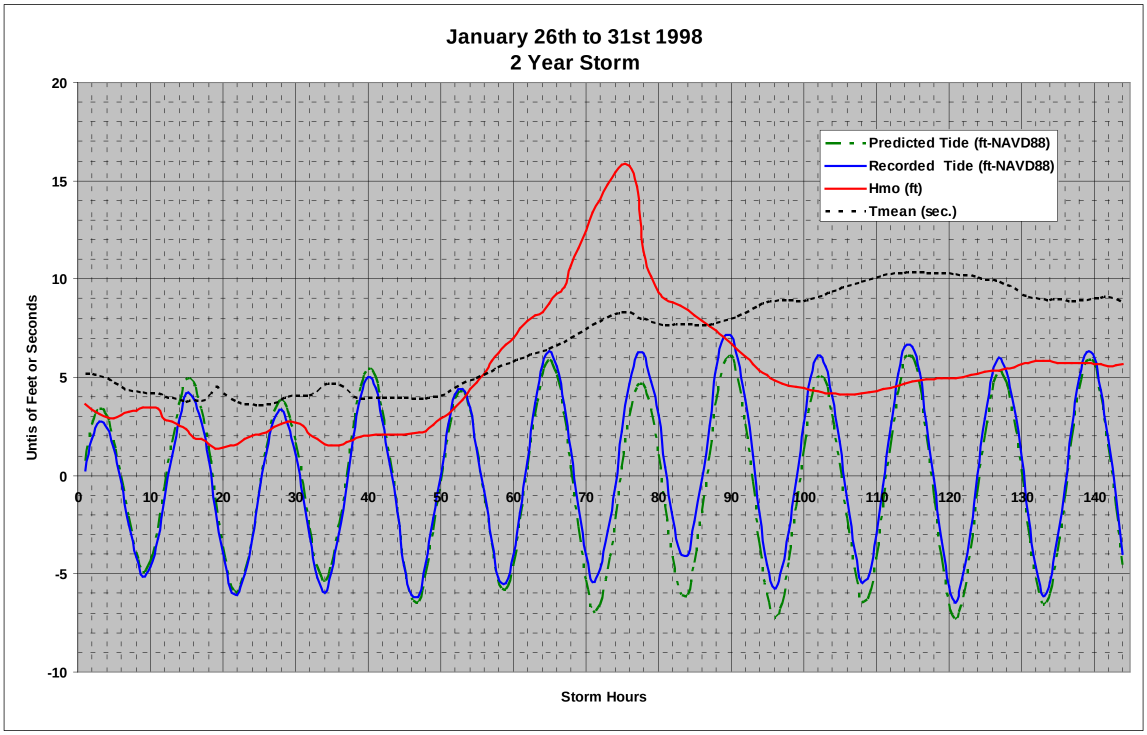


Figure A6. Storm Data 2-year storm (Jan. 26 to 31, 1998)

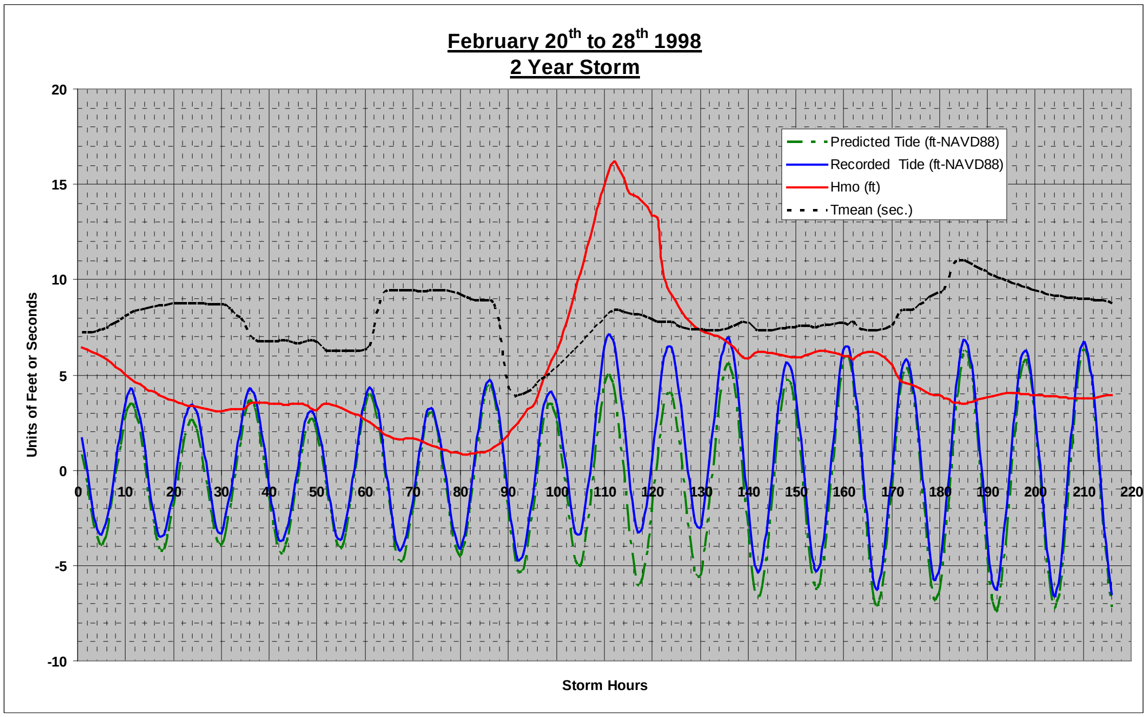


Figure A7. Storm Data 2-year storm (Feb. 20 to 28, 1998)

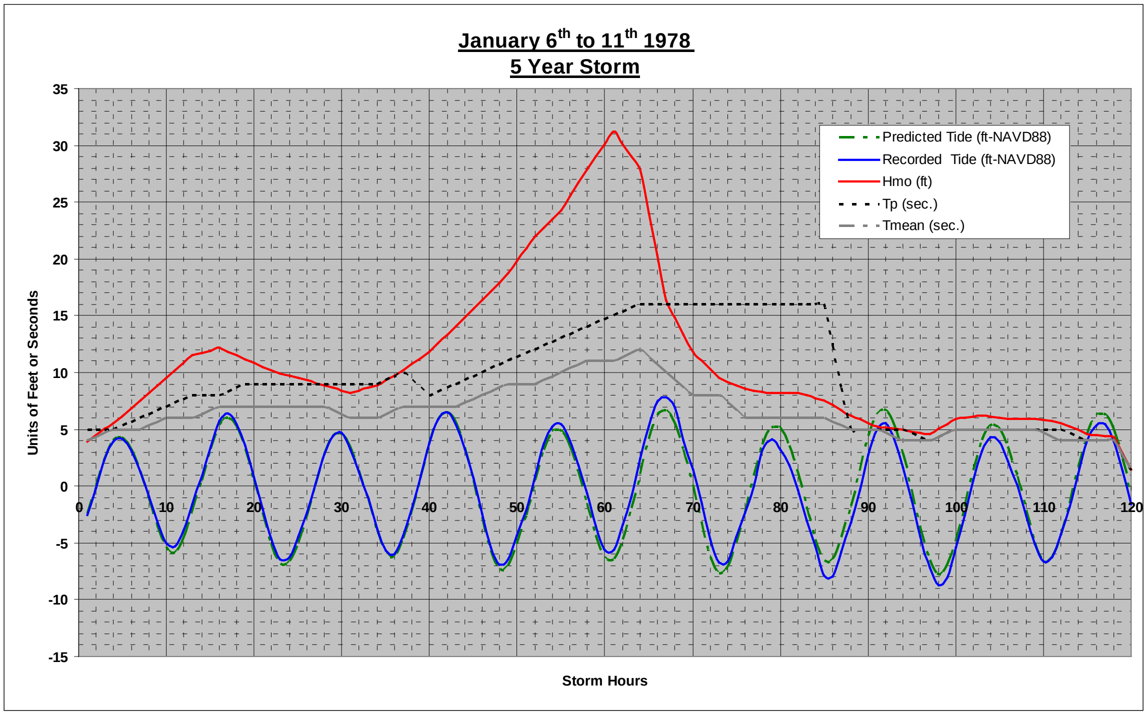


Figure A8. Storm Data 5-year storm (Jan. 6 to 11, 1978)

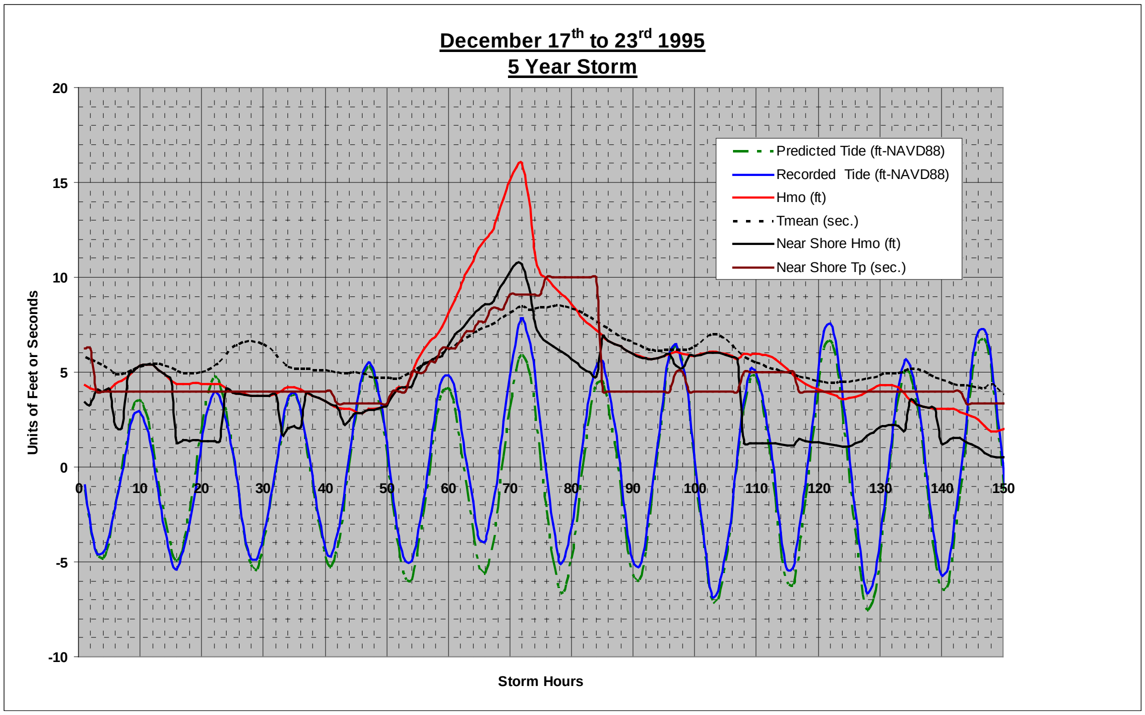


Figure A9. Storm Data 5-year storm (Dec. 17 to 23, 1995)

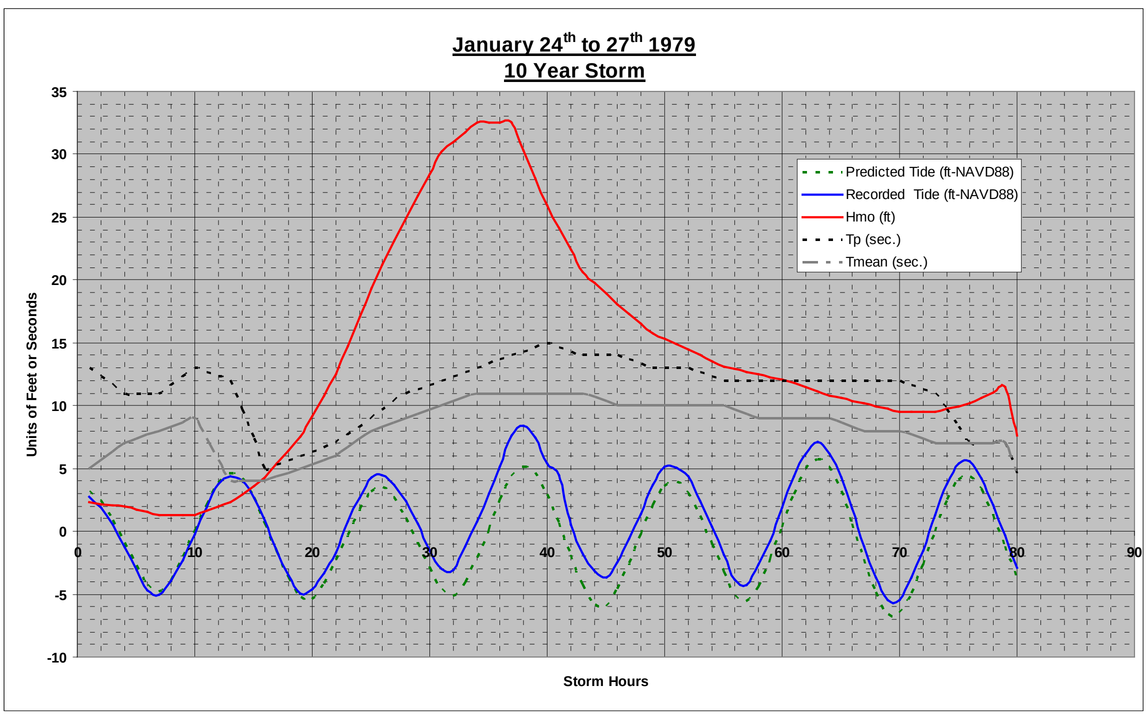


Figure A10. Storm Data 10-year storm (Jan. 24 to 27, 1979)

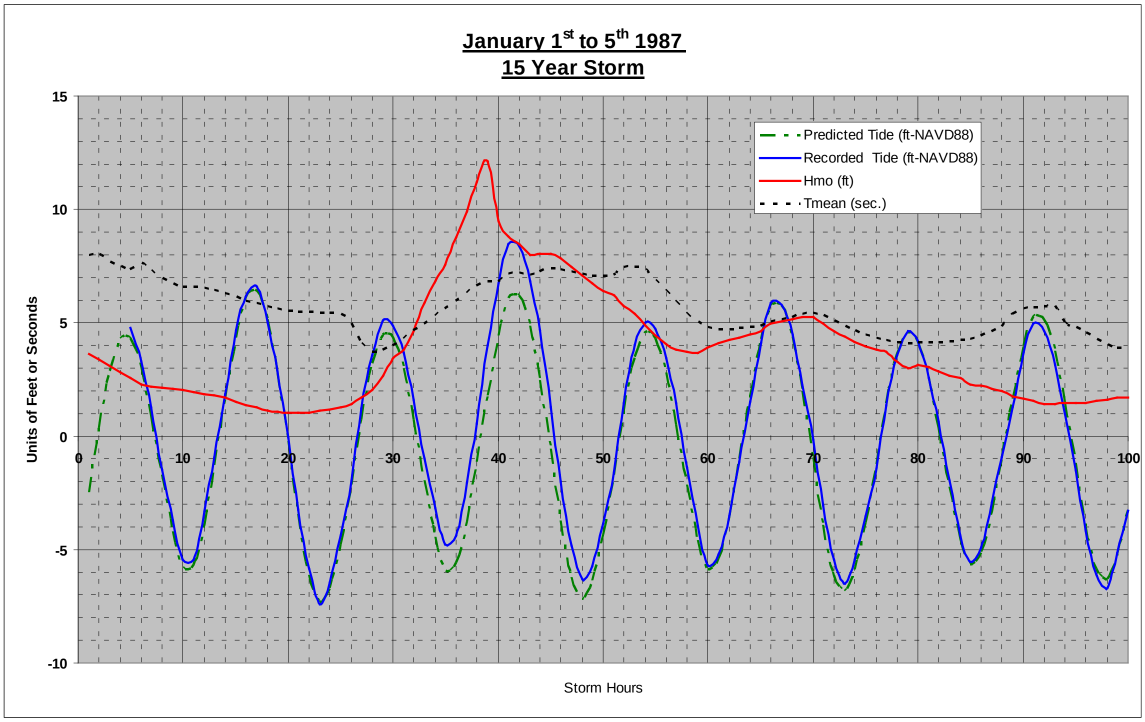


Figure A11. Storm Data 15-year storm (Jan. 1 to 5, 1987)

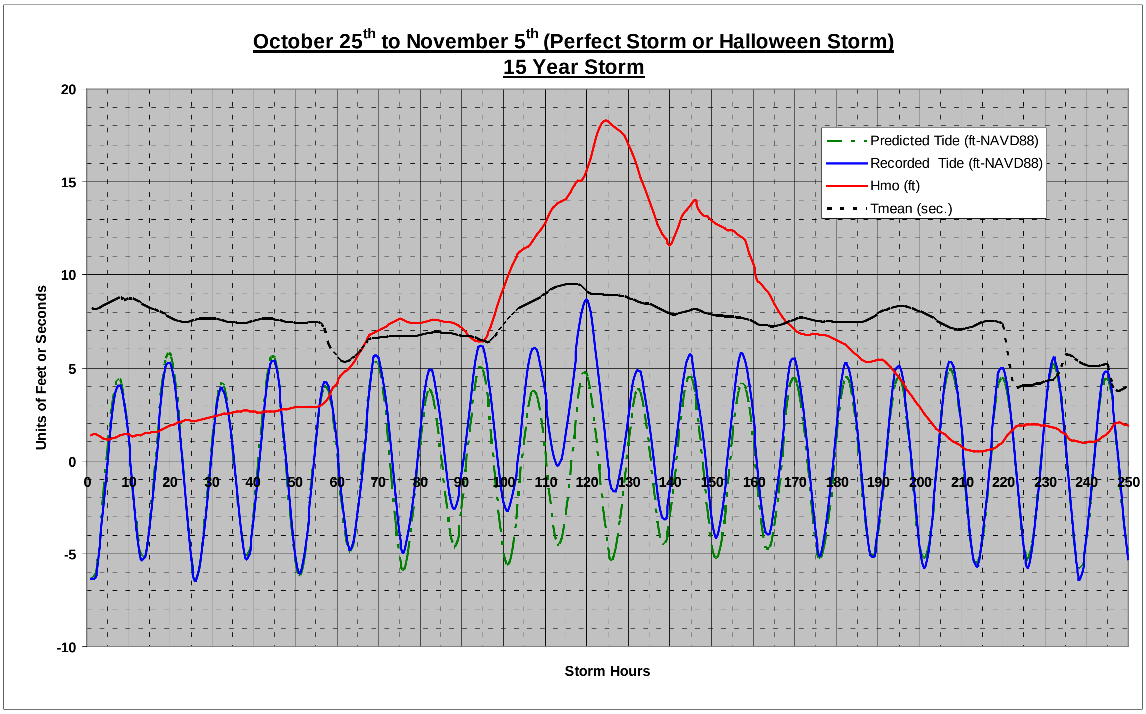


Figure A12. Storm Data 15-year storm (Oct. 25 to Nov. 5, 1991)

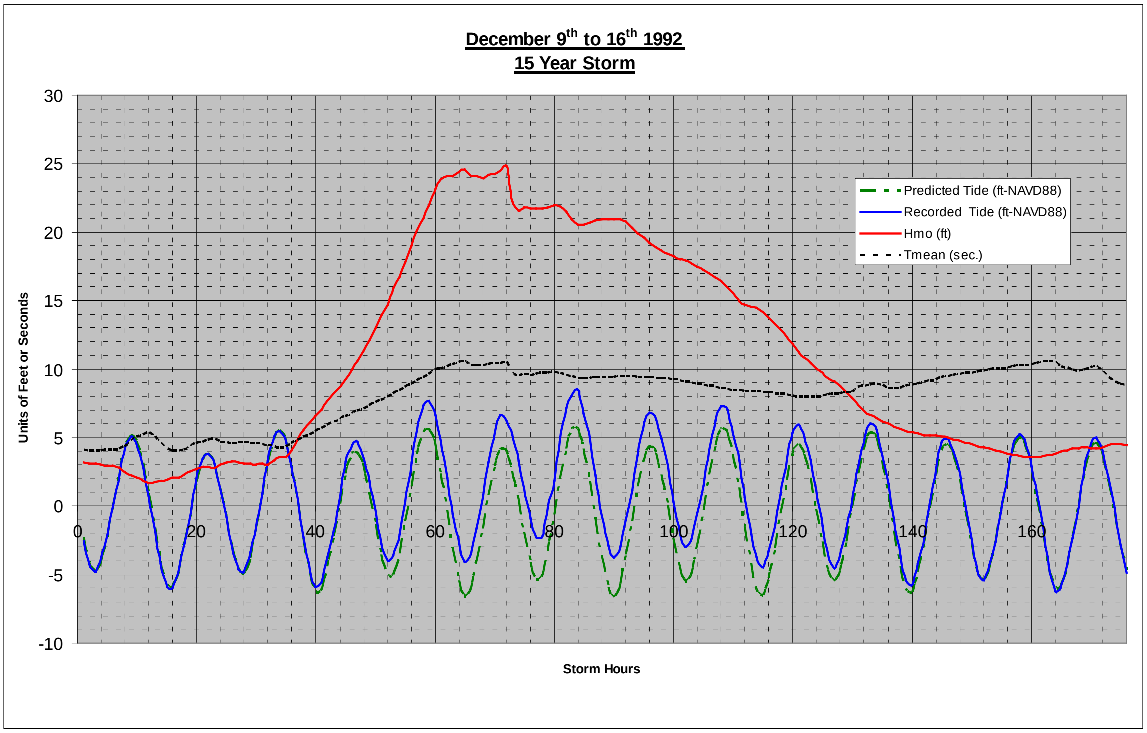


Figure A13. Storm Data 15-year storm (Dec. 9 to 16, 1992)

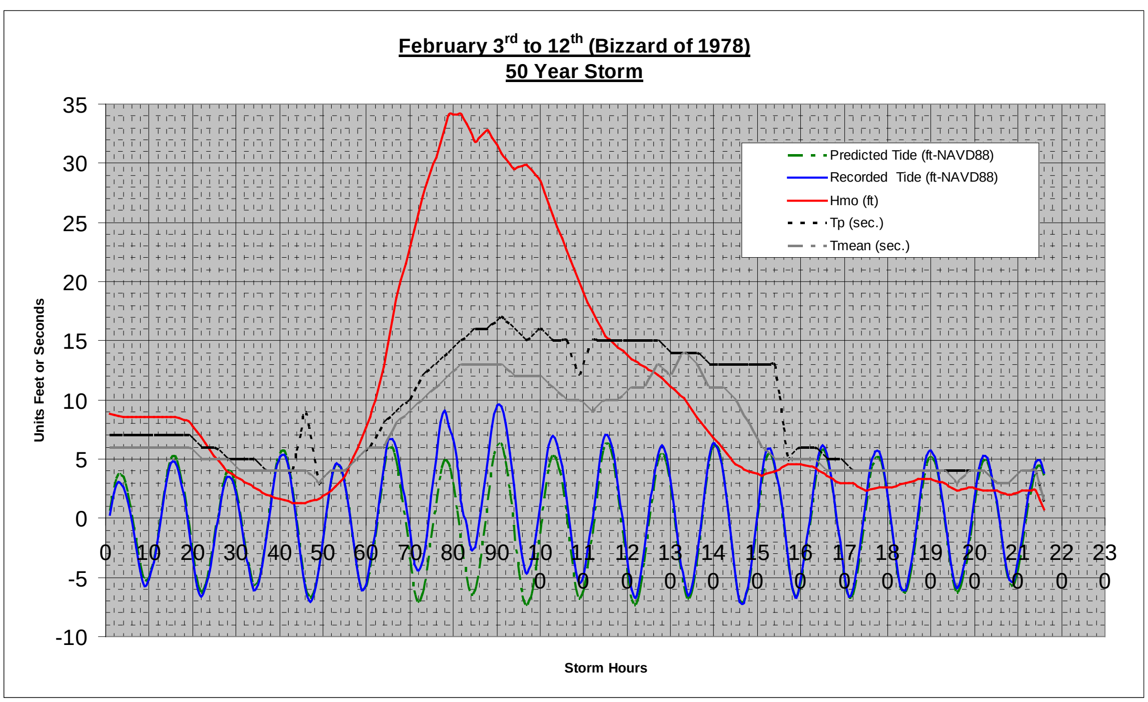


Figure A14. Storm Data 50-year storm (Feb. 3 to 12, 1978)

Appendix 4
Woods Hole Group Beach Alternatives Study