SIASCONSET COASTAL BANK STABILIZATION AND BEACH PRESERVATION PROJECT

ALTERNATIVES ANALYSIS

SEPTEMBER 2010

OCC Project # 210019

Prepared for:

SIASCONSET BEACH PRESERVATION FUND, INC.

Prepared by:

OCEAN AND COASTAL CONSULTANTS

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Executive Summary

The coastal bank at the east end of Nantucket Island has experienced continued erosion resulting in continued decrease in the setback distance between the top of the coastal bank and numerous homes and public infrastructure on Baxter Road. Ocean and Coastal Consultants, Inc. (OCC) has investigated design alternatives for a coastal bank stabilization and beach preservation project that provides a cost-effective and environmentally sensitive means to prevent storm damage to these homes and public infrastructure from continued unabated erosion of the coastal bank while preserving the coastal beach.

The purpose of this alternatives analysis is to evaluate available options and develop a preferred alternative. OCC has prepared this document to provide alternative design options and present the underlying assumptions, parameters and design variables. The scope of work is based on available background data, including previous coastal engineering studies, site surveys and sediment sampling. The preferred alternative will be used in the permitting process and the basis for final design plans for construction.

A three cycle alternatives analysis has been initiated to identify the preferred design to stabilize the coastal bank at the project site while preserving the coastal beach. This document presents the results of all three cycles. Cycle 1 identifies the potential alternatives to be developed further in the concept design. Cycle 1 considers three alternatives including the no-action alternative, a geotextile tube+sand alternative and a marine mattress+sand alternative. For Cycle 2, conceptual design layouts were created using the geotextile tube+sand and the marine mattress+sand alternatives. The size and location of the new coastal engineering structures will need to co-exist with the existing conditions to provide the required level of stability to the coastal bank while promoting a stable functional beach. The concept design is intended to avoid a negative impact to the adjacent shorelines. Cycle 3 took the concept design selected in Cycle 2 and then considered design modifications required to upgrade the concept from a 50-year storm design to a design capable of withstanding the 100-year storm event.

Based on the Cycle 2 analysis results presented herein, the recommended design alternative is the stone-filled marine mattress with gabion toe protection and periodic sand nourishment (mattress+sand design). The purpose of this design is to stabilize the toe of the coastal bank from erosion and prevent storm damage to homes associated with a 50-year (i.e. 2% annual probability) design storm event. Differences in the geometry of the geotextile tube+sand alternative and mattress+sand alternative result in differences in the size of the project footprint and the amount of sand required for periodic nourishment. The mattress+sand configuration optimizes these parameters. Additionally, the mattress+sand design has flexibility in both the dimension (thickness) and installation approach thereby allowing confidence that the final design will result in a system that can survive the design storm conditions. The cost of imported sand adds to the cost of the geotextile tube+sand design when compared to the mattress+sand. It was also determined that the construction duration of the mattress+sand alternative would likely be shorter than the geotextile tubes+sand, and require less specialized skills. This is due to the fact that four rows of geotextile tube are required, and each row must be filled individually by personnel having experience pumping slurry into geotextile tubes. The mattress+sand structure can either be pre-fabricated and barged to the site for placement by a crane, or filled on site by the contractor. These details will be finalized during the final design phase.

The Cycle 3 analysis takes the recommended mattress+sand concept design selected in Cycle 2 and provides additional details and a more accurate cost estimate. Based on standard coastal design methodology, it was recommended that the 50-year design storm mattress+sand concept be upgraded to consider the effects of the 100-year design storm event (i.e. 1% annual probability). Estimated costs for the 100-year storm design versus the 50-year storm design will only increase marginally, however the enhanced design will provide considerably more protection against more severe storm conditions.

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ALTERNATIVES ANALYSIS

1. Project Overview

The coastal bank at the east end of Nantucket Island has experienced continued erosion resulting in minimal setback from the top of the coastal bank for numerous homes and public infrastructure on Baxter Road. The Siasconset Beach Preservation Fund (SBPF) has retained Ocean and Coastal Consultants, Inc. (OCC) to investigate design alternatives for a coastal bank stabilization project that would preserve the coastal beach while providing a cost-effective and environmentally sensitive means to prevent storm damage to these residential buildings (homes) from continued unabated erosion of the coastal bank.

The purpose of this alternatives analysis is to evaluate available options and develop a preferred alternative. OCC has prepared this document to provide alternative design options and present the underlying assumptions, parameters and design variables. The scope of work is based on available background data including previous coastal engineering studies, site surveys and sediment sampling. The preferred alternative will be used in the permitting process and the basis for final design plans for construction.

1.1. Planning Objectives

OCC has investigated a number of approaches to stabilize the eroding coastal bank within the project area while also preserving the adjacent coastal beach. Additional factors to consider are the safety of homeowners, initial and maintenance costs, and potential environmental impacts. For example, if a proposed alternative is likely to create a situation hazardous to the safety of homeowners or is likely to have high annual maintenance costs, that alternative will not be recommended. SBPF has previously investigated numerous alternatives to prevent or minimize the continued erosion of the coastal bank. Some of the alternatives that have been investigated including groins, seawalls, beach nourishment and other coastal engineering approaches were found to be not acceptable, while others were attempted and were not acceptable as they did not provide the desired storm damage prevention. This alternatives analysis investigates two available options to meet the project goals without compromising the stated planning objectives.

1.2. Planning Constraints

The purpose of the proposed project is to provide an engineered solution to stabilize the toe of the coastal bank within the project area, thus preventing storm damage associated with up to a 50-year (2% annual probability) design storm event for Cycle 2; and a 100-year (1% annual probability) design storm event for Cycle 3, while preserving the coastal beach. The proposed solution should be designed to conform to the regulatory standards of the Nantucket Conservation Commission, the Massachusetts Department of Environmental Protection and the US Army Corps of Engineers. An additional project constraint that will be addressed throughout the design process is the regulatory limitation that coastal engineering structures are not permissible except to protect pre-1978 constructed residential buildings that have not been substantially improved after August 1978.

2. Site Description

The project site is located on the eastern shore of Nantucket Island, Massachusetts. The proposed project is for coastal bank stabilization along an east facing portion of coast adjacent to fourteen (14) privately owned residential properties. Currently, the project plan is to stabilize the bottom of the coastal bank using either a geotextile tube alternative or marine mattresses with gabion toe protection alternative. The coastal area to be protected is bound on the north by Lot #49-34 and on the south by Lot #49-18 and is approximately 1,700 linear feet in length. There are a total of 14 lots in the project area.

Figure 1 - Aerial image of Nantucket Island and the project extents with property numbers and profile lines.

3. Site Conditions

The local conditions at the project site are unique and critical in determining the best alternative to achieve project goals. Through the analysis of available data and publications, historical trends can be identified and used as a basis for future predictions.

3.1. Climate

Nantucket Island is located in what is called a humid continental climate, which is a climate found over large areas of landmasses in the temperate regions of the mid-latitudes where there is a zone of conflict between polar and tropical air masses. The humid continental climate is marked by variable weather patterns and a large seasonal temperature variance. Summers are often warm and humid with frequent thunderstorms and winters can be very cold with frequent snowfall and persistent snow cover.

Nantucket, as is the entire coast of Massachusetts, is prone to nor'easters and to severe winter storms. Summers can bring thunderstorms, averaging around 30 days of thunderstorm activity per year. Nantucket Island, like the entire United States eastern seaboard, is vulnerable to hurricanes. Because its location is farther east in the Atlantic Ocean than states farther south, Massachusetts has suffered a direct hit from a major hurricane only three times since 1851. More often, hurricanes weaken to tropical storms as they pass near Massachusetts.

Because of the influence of the Atlantic Ocean, temperatures are typically a few degrees cooler in the summer and a few degrees warmer in the winter. A common misconception is that the climate is influenced largely by the warm Gulf Stream current, however that current turns eastward off the coast of Virginia and the local waters are more influenced by the cold Canadian Labrador Current. As a result, the ocean temperature rarely gets above 65 °F (18 °C).

Nantucket's climate is also known for a delayed spring season, being surrounded by an ocean which is still cold from the winter. However, it is also known for an exceptionally mild fall season due to the ocean remaining warm from the summer.

3.2. Wind

Wind data for the project site was obtained from the National Data Buoy Center Station NTKM3 - 8449130 - Nantucket Island, MA. Hourly data between January 1, 2009 and December 31, 2009 were analyzed. The results of the analysis are presented in the wind rose below. The primary wind directions and frequencies are northeast, 25%; northwest, 16%; southwest, 36%; southeast, 24%. The majority (75%) of winds are between 1 and 11 mph and originate from the southwest. Wind speeds greater than 7 mph generally originate from the southwest and, to a lesser extent, the northwest and northeast.

For determining wind loads, OCC used wind data provided in ASCE Standard ASCE/SEI 7-05 *Minimum Design Loads for Buildings and Other Structures.* For Nantucket Island, the one-hour duration design wind speeds for the 50-year and 100-year storm events are 79 mph (i.e. 120 mph, 3-second gust) and 85 mph (i.e. 128 mph, 3-second gust), respectively.

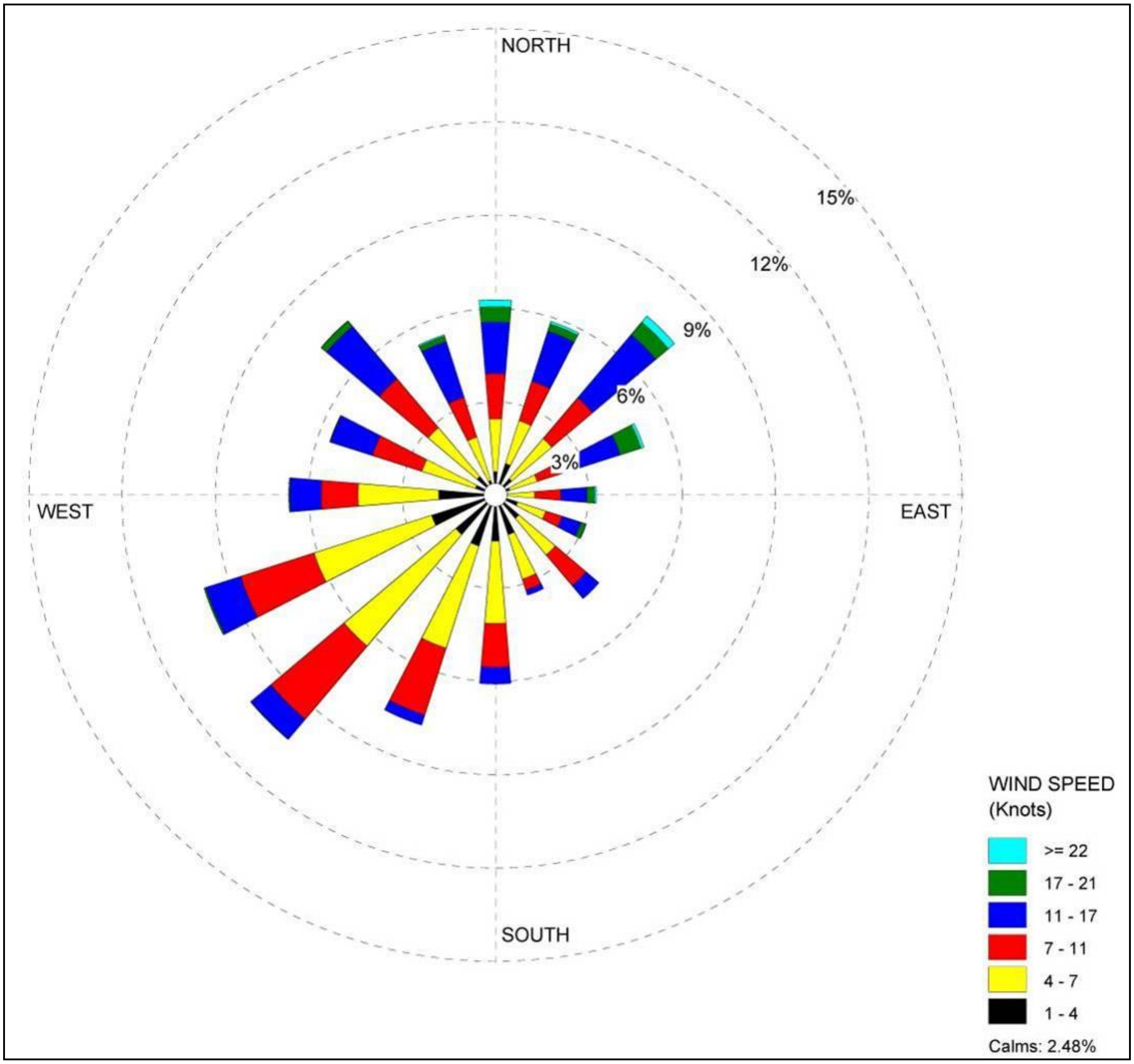


Figure 2 – Wind Rose for National Data Buoy Center Station NTKM3 - 8449130 - Nantucket Island, MA.

3.3. Waves

The project site is located along the open coast of the Atlantic Ocean and is exposed to varying daily swell as well as extreme storm waves caused by hurricanes and Nor'easters. In this location waves are the main component of sediment transport. Typical wave data is used to predict long term shoreline response. Extreme wave data is used to predict extreme erosion events and for developing design parameters for coastal structures.

Wave data for the project area is based on USACE WIS data from WIS Station 73, located approximately 15 miles east of the project site (41.25 N, 69.67 W). The station is located in a water depth of 89 feet. The average wave height is 4.2 feet with a period of 5.1 seconds and a direction of 208° (SSE). Approximately half of the waves propagate from the offshore direction. Of this portion of wave data, the average wave height is 4.4 feet with a period of 5.5 seconds.

Other information pertaining to waves:

- The largest onshore waves under typical conditions occur in December and January with average wave heights of 6.3 feet and maximum wave heights of 24.3 and 26.2 feet, respectively.
- The lowest onshore waves occur in July and August with average wave heights of 2.5 and 2.8 feet, respectively, and maximum wave heights of 15.4 feet.
- The largest and longest waves under typical conditions come from the northeast direction.
- The 50-year storm event has a maximum significant wave height of 27.7 feet and a period of 15.2 seconds. The 100-year storm event has a maximum significant wave height of 28.8 feet and a period of 15.2 seconds.

Table 1 - Hindcast wave data for onshore waves (0-180°) at WIS Station 73.

Month	Height (Hmo) (feet)		Period Tp (sec)	Dir (deg.)
	mean	max		
January	6.3	26.2	6.1	93.0
February	6.1	24.3	6.2	84.0
March	5.5	24.6	5.9	95.0
April	4.3	21.0	5.4	110.0
May	3.4	15.7	5.0	119.0
June	3.0	11.2	4.7	126.0
July	2.5	15.4	4.7	145.0
August	2.8	15.4	4.9	128.0
September	3.3	21.3	5.5	115.0
October	4.3	25.3	5.3	94.0
November	5.5	20.0	5.7	93.0
December	6.3	24.3	6.0	75.0
Mean	4.4	20.4	5.5	107.0

Table 2 - Return period for offshore storm conditions.

Return Period (years)	Height (Hmo) (feet)	Period Tp (sec)
1	18.8	11.1
2	21.4	12.3
3	22.4	12.8
4	23.1	13.1
5	23.5	13.3
6	23.9	13.5
7	24.2	13.6
8	24.5	13.7
9	24.7	13.8
10	24.9	13.9
15	25.7	14.3
20	26.2	14.5
25	**26.6**	**14.7**
30	26.9	14.8
35	27.1	14.9
40	27.4	15.0
45	27.5	15.1
50	**27.7**	**15.2**
100	**28.8***	**15.2***

* Based on Interpolated Values

3.4. Currents

Currents along the beach within the project site are generated by a combination of tidal, wave, and wind forcing. The direction and magnitude of the currents will vary based on changing weather patterns.

Data for water level and current velocity measurements were obtained from the document entitled Final Environmental Impact Report, Sconset Beach Nourishment Project, Nantucket, MA and dated November 30, 2006. The above referenced report summarizes data collected by the Woods Hole Group between October and November, 2005. The report states:

- Sontek current meters and RDI ADCP's were used to measure waves and currents.
- The ADCP was located in a water depth of approximately 24 feet.
- During flood tides, the (positive) current direction is from south to north (~360°) and that during the ebb tide, the (negative) current direction is from north to south (175°).
- Tides propagate from the south.

- Two peaks in the current speed occurred every tidal cycle, one during the flood and one during the ebb. Typical current speeds during the peak flood ranged from 2.5 to 3.0 feet per second. Typical current speeds during peak ebb ranged from 2.5 to 4.3 feet per second.
- The ebb-dominated current speeds were higher during spring tide and lower during neap tide.

There was no direct correlation between measured current velocities and observed waves.

3.5. Water Level

3.5.1. Tides

Tidal datum information for the project area was gathered from NOAA Station 8449130, located on Nantucket Island, Nantucket Sound, Massachusetts. This area is subject to mixed semi-diurnal tides. Information for this station is in the table below.

Table 3 - Tidal Datum data for NOAA Station 8449130.

Datum	Value	Description
MHHW	3.37	Mean Higher High Water
MHW	3.04	Mean High Water
MSL	1.57	Mean Sea Level
MTL	1.52	Mean Tide Level
MLW	0	Mean Low Water
MLLW	-0.2	Mean Lower Low Water
MN	3.03	Mean Range of Tide
Maximum	7.67	Highest Water Level on Station Datum
Max Date	10/30/1991	Date of Highest Water Level
Max Time	17:30	Time of Highest Water Level
Minimum	-2.34	Lowest Water Level on Station Datum
Min Date	2/12/1981	Date of Lowest Water Level
Min Time	12:30	Time of Lowest Water Level

3.5.2. High Tide Line

The high tide line is defined as the highest high tide of each month for the previous 12 months. The High Tide Line for the period of May 2009 to April 2010 is +4.97 feet MLW. The tidal station used to create this table was 8449130, Nantucket Island, MA.

Table 4 - High Tide Line for time period of May 2009 to April 2010.

Month	Highest High (MLW)
May-09	4.31
Jun-09	5.44
Jul-09	4.69
Aug-09	4.49
Sep-09	4.52
Oct-09	5.43
Nov-09	4.7
Dec-09	4.95
Jan-10	5.67
Feb-10	5.24
Mar-10	5.26
Apr-10	4.95
HTL:	**4.97**

3.5.3. Storm Surge

Storm surge elevations were obtained from the FEMA Flood Insurance Study for Nantucket (FEMA, 1996). Elevations in the table below are referenced to the Half Tide Line (HTL) and MLW. For the eastern side of Nantucket Island, HTL was defined as +3.0 feet MLW by FEMA in 1996. In the previous section, our analysis indicates that the High Tide Line is +4.97 feet MLW. This discrepancy is due to different definitions and uses for the term HTL and does not affect the predicted storm surge level.

Table 5 - Still water storm stage elevations (from FEMA 1996 and Epsilon 2006).

Storm Event	FEMA Elevations (feet above HTL)	MLW
10-year	4.8	7.8
50-year	**6.2**	**9.2**
100-year	7.2	10.2
500-year	9.5	12.5

3.5.4. Relative Sea Level Rise

Relative sea level rise data was obtained via the NOAA Tides and Currents website. Relative sea level rise (RSLS) is the apparent change in sea level compared to a fixed vertical datum. RSLS consists of two independent components, eustatic sea level change (the global change in ocean water level compared to a fixed vertical datum) and subsidence (the local change in land elevation compared to a fixed vertical datum). The mean sea level trend is 2.95 millimeters/year with a 95% confidence interval of +/- 0.46 mm/yr based on monthly mean sea level data from 1965 to 2006 which is equivalent to a change of 0.97 feet in 100 years.

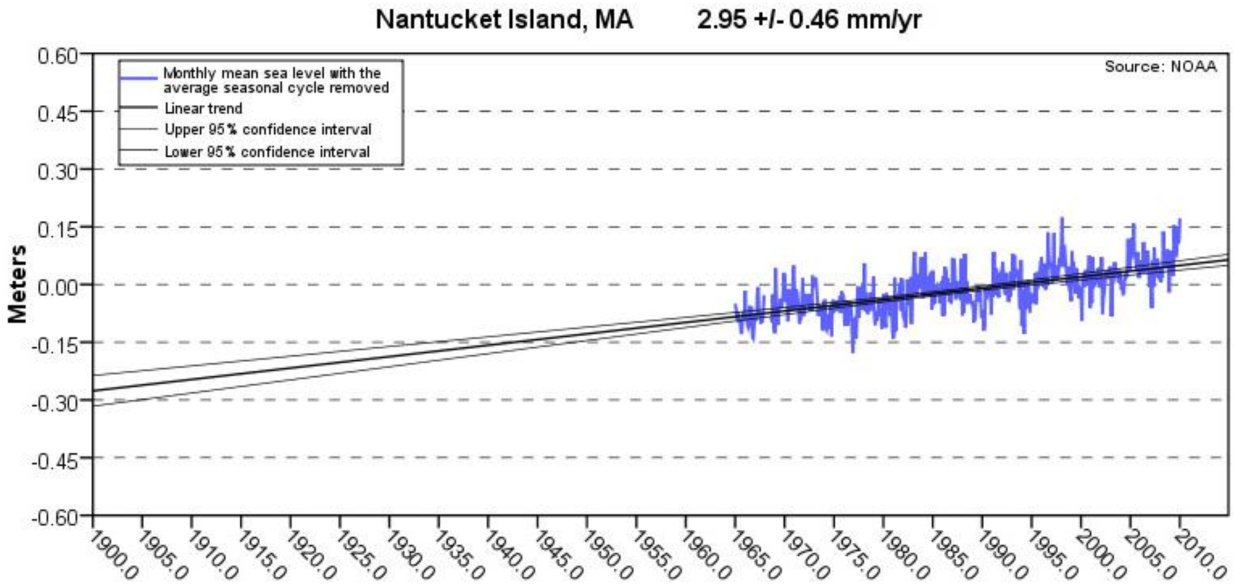


Figure 3 – Relative sea level rise trend for Nantucket, MA.

The plot shows the monthly mean sea level. The long-term linear trend is also shown, including its 95% confidence interval. The plotted values are relative to the most recent Mean Sea Level datum established by CO-OPS.

3.5.5. Scour

Waves impacting the beach and coastal bank during a storm can cause scour, particularly at the toe of a coastal engineering structure. Therefore, toe protection is required. Toe protection is especially important if the seaward end of the coastal engineering structure is in water depth less than 1.5 times the incident wave height. Waves at the coastal engineering structure will be limited by the depth at the structure, which is determined by combination of storm surge, additional setup due to the breaking wave and the depth at the structure's toe. The scour design goal is to determine the potential scour depth and width, and design the structure's toe accordingly. As a rule of thumb, the scour depth can be estimated as equal to the incident wave height, while the apron length can be up to 3.5 times the incident wave height.

3.6. Geology

Nantucket was formed by the outermost reach of the Laurentide Ice Sheet during the recent Wisconsin Glaciation, and shaped by the subsequent rise in sea level. The island's low ridge across the northern section was deposited as glacial moraine during a period of glacial standstill, a period during which till continued to arrive, but melted at a stationary front. The southern part of the island is an outwash plain, sloping away from the arc of moraine and shaped at its margins by the sorting actions and transport of alongshore drift. Nantucket became an island when rising sea levels re-flooded Nantucket Sound about 5,000–6,000 years ago.

3.7. Grain Size

A thorough sediment sampling effort was undertaken for an area on Nantucket approximately 3 miles north of the project site. Full results can be found in Final Environmental Impact Report, Sconset Beach Nourishment Project, Nantucket, MA (Epsilon, 2006). A total of 248 samples were collected at various cross-shore positions along each transect (including the coastal bank, coastal dune, beach, wave breaking zone and the offshore area). The composite mean grain size for all 248 samples (weighted by elevation) is 0.86 mm with an average silt content of 2.8%. A summary of grain size results for the coastal bank, coastal dune and coastal beach are presented in the table below.

Table 6 - Grain size summary.

	Coastal Bank		Coastal Dune		Coastal Beach	
	mean	% silt	mean	% silt	mean	% silt
Min.	0.31	2.9	0.6	0.1	0.6	0.2
Max.	0.51	28.9	0.8	1.3	0.9	0.4
Mean	0.4	13.6	0.7	0.3	0.7	0.2

3.8. Sediment Transport

Alongshore (or littoral) sediment transport is the movement of sand parallel to the coast primarily driven by waves that obliquely approach the shoreline. The amount of sand transport is a function of the size of the waves and the increasing angle that the waves make with the beach as they come in to break. This angle means that the water runs up the beach following the angle offset from perpendicular until the up-rush stops. Gravity then pulls the back swash directly down the beach slope so that this repetitive saw tooth pattern of water movement slowly moves the sand on the beach in the direction that the waves make the open angle to the beach.

Net sediment transport rates determine the balance of sediment loss or gain within a system over a defined period of time. Net sediment transport for the specific project area within Sconset is from south to north. Since the project area is located at a nodal zone where the net transport shifts, transport at any particular time can be in either direction. Net sediment transport may vary within a given system or timeframe.

3.9. Shoreline Change

Shoreline change has been investigated from both a long term and short term perspective. Long term shoreline data was available from the Massachusetts Office of Geographic and

Environmental Information as a GIS layer. While the accuracy of this data layer is unknown, from the time period of 1846 to 1955, the shoreline advanced approximately 300 feet (approximately 2.75 feet per year). The current trend of erosion is evident in the shoreline data from 1955 to 1994. During this time period, the shoreline retreated at a rate of approximately 4.1 feet per year (see Figure 4).

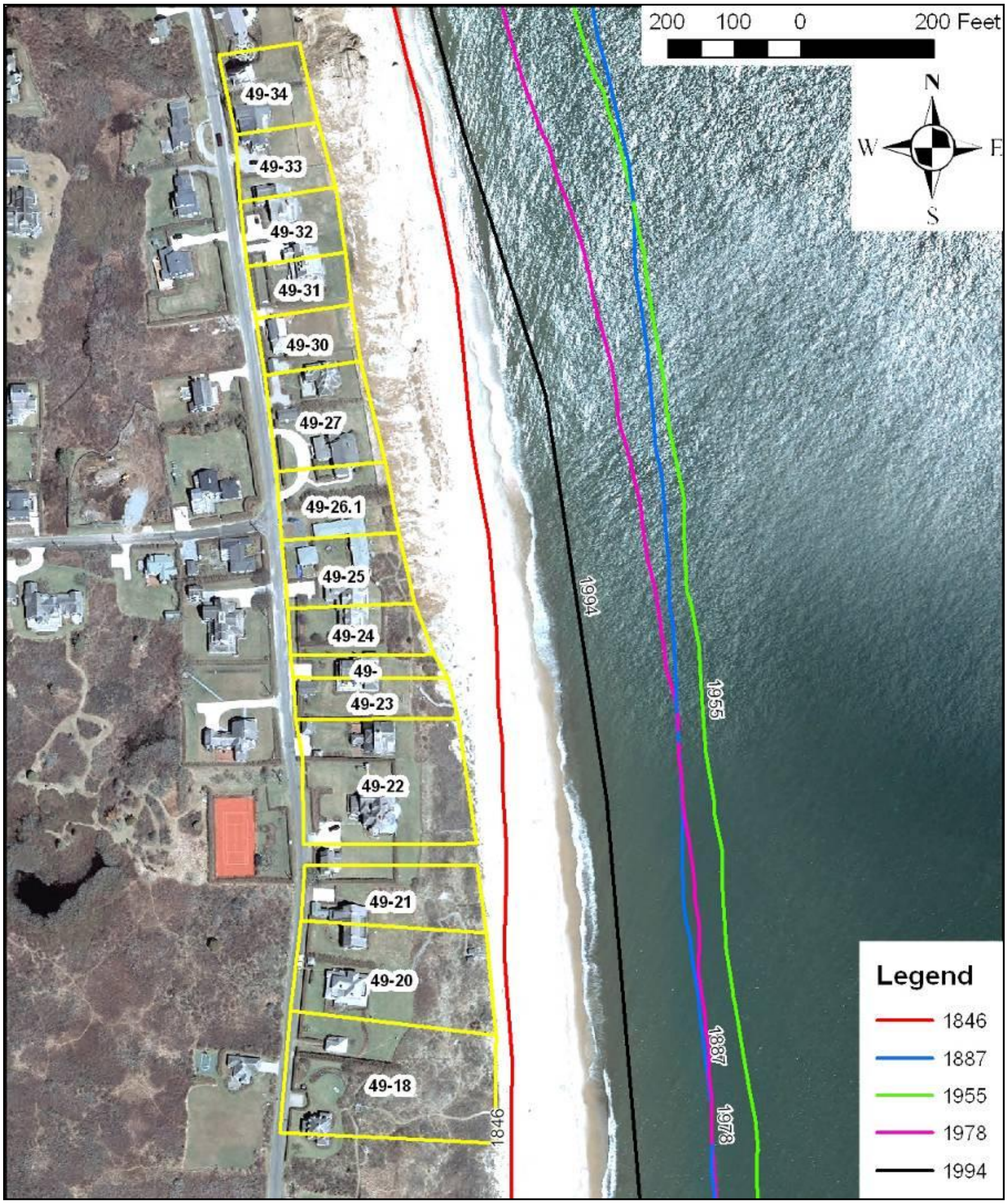


Figure 4 – Shoreline data for project area between 1846 and 1994 (MA Office of Geographic and Environmental Information).

More recent erosion rates were determined from a review of the Lighthouse Beach Dewatering Project, October 2009, 54th Quarterly Report with Comparisons to Baseline Erosion Rates prepared by the Woods Hole Group. Surveys have been performed within the project area quarterly since 1994. The report compared shoreline changes for three timeframes including November 1994 to October 2009, December 2001 to October 2009 and April 2009 to October 2009. For the time periods between November 1994 to October 2009 and December 2001 to October 2009, all profiles within the project area eroded at an average annual shoreline change rate of 8.2 feet per year and 3.0 feet per year, respectively. During the short 6 month period from April to October 2009, all profiles accreted with an average rate of 6.44 feet per year. This could be attributed to the large seasonal fluctuations of sediment during the summer months when beaches in the project area tend to accrete. A summary of the shoreline change data is presented in the table below.

Table 7 - Shoreline change data summary.

Profile	Shoreline Change (feet)					
	Nov-94 to Oct 09	Shoreline Change per Year	Dec-01 to Oct-09	Shoreline Change per Year	Apr-09 to Oct-09	Shoreline Change per Year
89.2	-124.98	-8.33	-26.68	-3.34	0.35	0.7
89.5	-124.22	-8.28	-24.82	-3.10	0.03	0.06
89.8	-128.01	-8.53	-20.91	-2.61	4.37	8.74
90	-128.25	-8.55	-20.45	-2.56	6.64	13.28
90.6	-109.92	-7.33	-28.02	-3.50	4.72	9.44
Avg (per year)	**-123.08**	**-8.21**	**-24.18**	**-3.02**	**3.22**	**6.444**

The coastal beach at the base of the coastal bank within the project site is subject to large seasonal changes in berm elevation. OCC reviewed cross sections of surveys from 1994 to 2009 from the above referenced Woods Hole Group report in order to estimate the lowest expected beach berm elevation and determine the ideal bottom elevation of the bank stabilization structure. Based on our review of the data, the lowest expected elevation, accounting for long term erosion and accretion as well as seasonal variations, is approximately +8.0 feet MLW. In addition to this seasonal fluctuation, OCC estimated the potential for up to 3.5 feet of scour during a 50-year storm event, and up to 8.0 feet of scour for a 100-year storm event.

3.10. Coastal Bank Retreat

It is important to understand that the coastal processes influencing the shoreline are different than those affecting the coastal bank. Beach erosion is primarily driven by waves and tidal currents within the surf zone. Bank erosion, however, is caused by slope instability. In the case of

Siasconset, this slope instability is caused by wave run-up during storms, which erodes the bank toe. Because the coastal erosion processes are different, the beach and the bank do not recede at the same rate.

The average annual retreat of the coastal bank is required to estimate the volume of sediment that will no longer be available to the coastal system as a result of the proposed bank stabilization. This volume is the most appropriate measure required to determine the amount of sand required to nourish the beach. This approach is also the method which has previously been accepted by the Massachusetts DEP and other agencies on other projects to determine the requirements for sand nourishment.

Epsilon Associates has performed an analysis of the coastal bank retreat along the Siasconset project area utilizing aerial photographs taken over the last 15 years; specifically 1994, 2003, and 2009. The photographs were obtained from the Massachusetts Office of Geographic and Environmental Information (i.e. MassGIS), and are geo-referenced to permit direct comparison of visible changes over time. Figure 5 provides a view of the 2009 aerial photograph, overlaid with the top of bank lines from 1994, 2003, and 2009. As summarized in Table 8, Epsilon's linear regression analysis of GIS transects at 10-foot spacing along the study area indicates that the coastal bank has retreated approximately 3 feet/year on average over the last 15 years. Assuming an average bank height of 68 feet, the annual volumetric loss of the bank per linear foot is approximately (68 ft) x (3 ft) x (1 cy/ 27 ft) = 7.6 cubic yards. Accounting for the fact that approximately 13% of the sediment in the bank is fines, the annual volumetric loss should be reduced to 6.6 cubic yards per linear foot.

Therefore, while the 1,700 linear foot beach system is losing a yearly average of 7,333 cubic yards of material, the bank is supplying approximately 11,200 cubic yards. This implies that the material in the bank is indeed supplying sand to not only the beaches in the project area, but to down drift beaches as well.

In order for the bank and beach system to remain relatively stable in the project area, and to replace the amount of sand that would ordinarily be contributed by the bank through erosion 6.6 cubic yards per year per linear foot of project length should be used as a nourishment target. This assumes that no material is supplied to the beach from the bank after stabilization of the entire 1,700 linear foot project length.

Table 8 - Coastal Bank Retreat Statistics at Siasconset from 1994 to 2009 (based on a linear regression analysis from Epsilon Associates).

	Top of Coastal Bank Recession Rate (ft/yr)
Maximum	-5.05
Minimum	-0.03
Average	-2.96

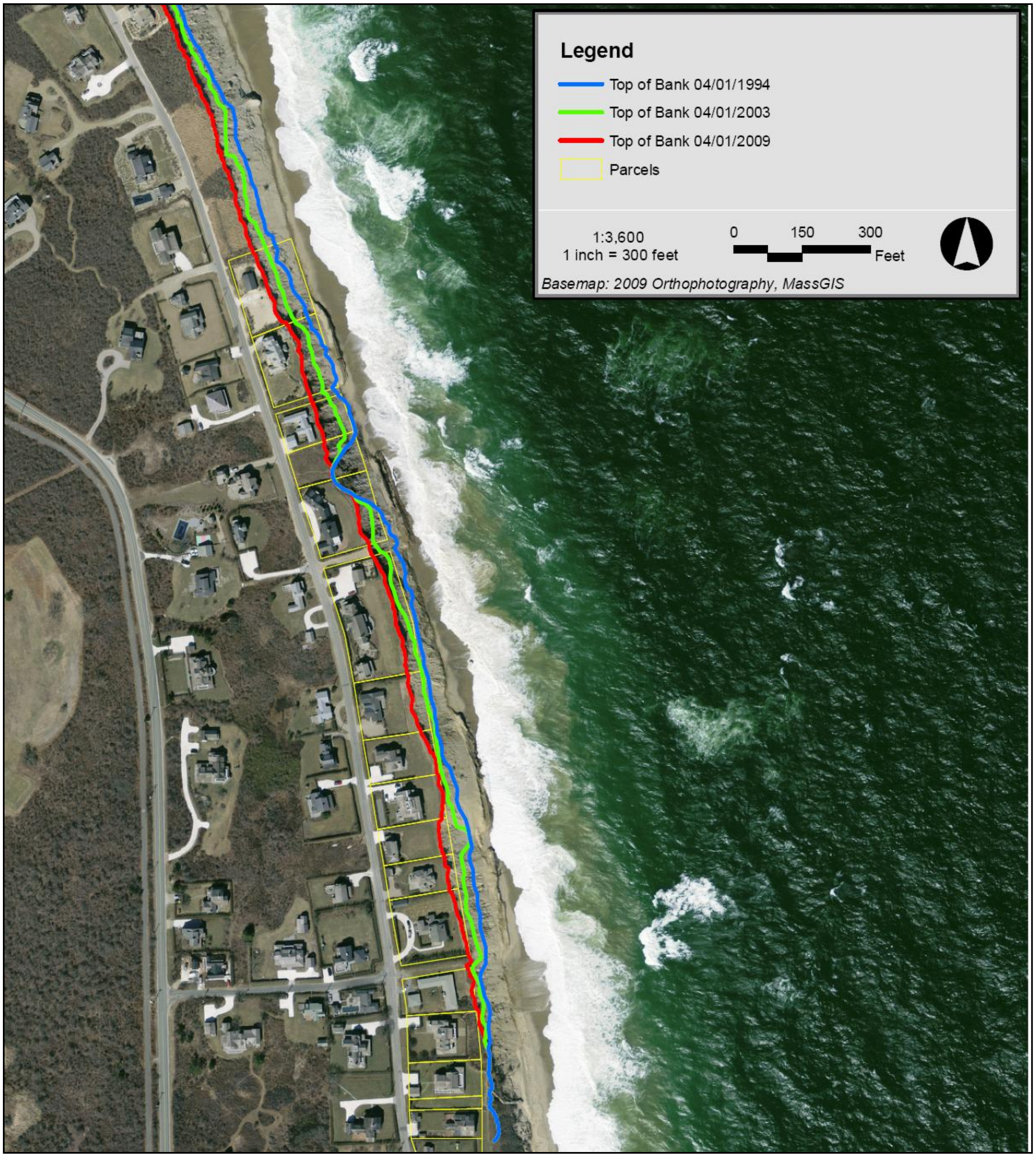


Figure 5 –Coastal Bank Retreat at Siasconset has been approximately 3 ft/yr since 1994, from GIS analysis by Epsilon Associates.

4. Preliminary Design Alternatives

A three cycle alternatives analysis can be used to identify the preferred design to stabilize the coastal bank at the project site. This document presents the results of Cycles 1 and 2. Cycle 1 identifies potential alternatives to be used in the concept design. The project team evaluated each alternative individually based on its ability to work within the project specific coastal system and to meet the project's design parameters. Concepts that meet most of these criteria were then evaluated in the Cycle 2 analysis.

In Cycle 2, concept design layouts were developed using multiple combinations of the selected Cycle 1 alternatives. These layouts were evaluated by the team with more detail using engineering and scientific judgment and experience to identify a preferred conceptual design.

In Cycle 3, the performance of the preferred conceptual design will be evaluated in more detail and modified as necessary. The design that has the best performance and meets the project's design parameters was selected as the preferred design layout for future evaluation in Cycle 3. Final design details of the structures will be determined later to create a final design plan.

4.1. Design Parameters

The following is a list of the design parameters identified by OCC for the Siasconset Coastal Bank Stabilization and Beach Preservation Project:

Project Objectives:

- Protect residential buildings from coastal bank erosion and storm damage.
- Provide a cost-effective, "softer" coastal engineering structure to protect the toe of the coastal bank from further erosion due to wave attack from the 50-year design storm (2% annual probability) conditions, or back-to-back 25 year events.
- Maintain an appropriate supply of sand to the coastal beach and nearshore environment based on an application of historical coastal bank retreat rates.

Project Constraints:

- Permanent or "hard" coastal engineering structures (i.e. seawalls, bulkheads or revetments) have been previously evaluated and have been found to be unacceptable.
- Proposed alternatives must be the “Best Available Measures” defined at 310 CMR 10.04 as “the most up-to-date technology or the best designs, measures or engineering practices that have been developed and that are commercially available.”.
- Proposed alternatives should meet the current performance standards of the Town of Nantucket Conservation Commission Wetlands Protection Regulations.

Coastal Engineering Criteria:

- Top of Coastal Bank Stabilization = Elev. +23.5 feet MLW (accounts for wave run-up during 50-yr storm conditions)

- 50-Year Design Storm Still Water Level (SWL) = Elev. +9.2 feet MLW
- Toe of Coastal Bank = Elev. +8.0 feet MLW
- High Tide Line (HTL) = Elev. +4.97 feet MLW
- Bottom of Coastal Bank Stabilization = Elev. +4.5 feet MLW

4.2. Cycle 1 - Alternatives

Three alternatives, including a no-action alternative, were identified and evaluated. The evaluation criteria focused on the ability to satisfy the project goals, constructability, required maintenance, and relative costs. Each alternative was evaluated using available documentation, engineering judgment, and previous experience.

4.2.1. No-Action Alternative

Under the no-action alternative, the project area would remain "as is." Without action it is expected that the coastal bank will continue to erode. Continued erosion would further endanger homes and public infrastructure.

With no action taken, this alternative does not require construction nor have a direct cost associated with the project. However there may be indirect costs associated with this alternative when coastal storms cause further erosion and ultimately damage the landward building and infrastructure. This alternative does not meet the project objective of protecting landward buildings or infrastructure.

An advantage of this option would be that there would not be any immediate costs for construction. The disadvantage of this option is that the coastal bank will continue its erosional trend and result in storm damage losses of homes in the very near future. The protective function of the coastal bank is likely to deteriorate further and create an inherent storm damage risk to the residential buildings and infrastructure. Based on our evaluation, the no-action alternative was not considered for further analysis in Cycle 2.

4.2.2. Alternative #1: Geotextile Tubes+Sand

Coastal engineering structures can be built in the coastal environment using a variety of materials, typically stone, concrete or timber. A somewhat “softer” and relatively recent innovation is the use of sand-filled geotextile tubes; OCC has extensive experience using geotextiles in the coastal environment. High strength woven geotextiles are sewn into an empty tube and deployed. The tube is then hydraulically filled with sand to form a dense, elliptical shaped structure. As flexible, gravity structures, geotextile tubes have some advantages over traditional structures. Tubes have some inherent flexibility to accommodate foundation changes, however if the subsurface changes too much due to scour, the tube can become unstable and can fail.

Geotextile tubes are fabricated from high strength, woven polyester or polypropylene. Sheets of the geotextile are sewn together in the factory with fill sleeves (or ports) on top. Fill port spacing is somewhat dependant on the grain size of the fill material. Since tube fabrics are woven in 15 foot wide sheets, typical tube sizes are 15, 30 and 45 foot circumference. Tubes are most stable when the height to width ratio is 0.5 or less. This also corresponds to the natural shape the tube assumes when properly filled. A 30 foot circumference tube will be filled to its optimum shape when the width is 12 feet and the height is 6 feet.

Polypropylene, usually black, is the less expensive of the fabric types, but has a lower tensile strength (400 lb/in. x 600 lb/in.) and can suffer from creep elongation during filling. Stress during filling and the fact that the fabric is inherently weaker means that the filled tube will have a lower factor of safety. Polyester, usually white, is more expensive but considerably stronger (1,000 lb/in. x 1,000 lb/in.) and exhibits much less elongation. This means that seams are stronger and the resulting installed tube has a high factor of safety.

Both fabrics are susceptible to UV degradation and debris damage. For this reason and aesthetics, an armor layer is often recommended in coastal applications. The best armor layer fabric is a durable, woven vinyl-coated polyester sewn to the tube in the factory. Armor layers can match the color of the sand and offer protection to the main tube.

Geotextile tubes, like any coastal engineering structure, need toe protection. This is often provided with a polypropylene scour apron. The scour apron protects the tube's foundation to prevent excessive movement. Scour can occur due to wave impact and currents.

Similar Installations of Geotextile Tubes

Geotextile tubes have been used extensively for shoreline protection projects. One example was the installation of three, 30-foot circumference tubes along an eroding stretch of beach in Sea Isle City, New Jersey in December of 1997 to protect an 18-unit apartment complex. The construction sequence included a one foot deep trench onto which a scour apron was placed, filling the tubes to a height of 5.5 to 6.0 feet (with a width of 12-13 feet) and covering with a one to two foot layer of sand cover. The resulting dune was approximately 8 feet high. Three months after the installation, the project area was hit with a Nor'easter and Cape May County was declared a national disaster area. The condominium complex protected by the geotextile tubes was not damaged.

Figure 6 – Aerial view of geotextile tubes in Sea Isle City, NJ, 1998.

Another example of geotextile tubes used for coastal protection is the installation of 40-foot circumference tubes to protect the toe of a coastal bluff at Ashkelon Cliffs, Israel subject to the full force of storm generated waves in the Mediterranean Sea. The tubes were installed at the base of sand bluffs approximately 40 to 50 feet tall to protect upland infrastructure. An image of the installed tubes is below in Figure 7 .

Figure 7 –Geotextile tubes at the base of the Ashkelon Cliffs, Israel.

4.2.3. Alternative #2: Marine Mattress+Sand

The marine mattress+sand alternative consists of rock-filled containers constructed of high-strength geogrid. Figure 8 illustrates a marine mattress structure called the "Triton Marine Mattress System" which was developed by the Tensar Corporation, manufacturer of the geogrid panels used to form the marine mattress. These mattresses are approximately 6.5 feet wide and are available in various lengths and thicknesses.

Similar Installations of Stone-filled Marine Mattress

Both marine mattresses and gabions are used to create sloped protective structures along the coast or waterways. The coastal engineering structure being considered for the Siasconset project is designed to protect the lower portion of the coastal bank from erosion due to direct wave attack. This alternative uses gabion baskets at the toe of the coastal bank to stabilize the coastal bank by preventing scour while the marine mattress constructed on the slope of the coastal bank stabilizes the coastal bank landform by preventing storm damage erosion due to breaking waves and run up.

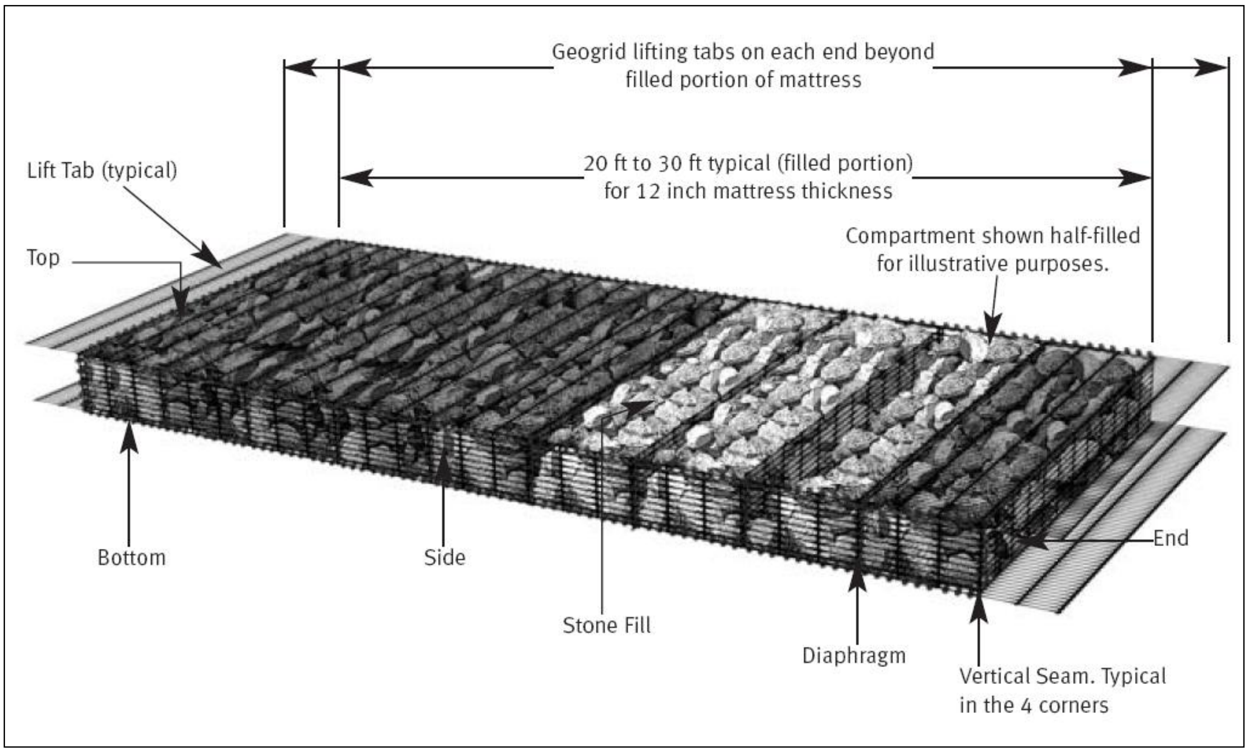


Figure 8 – Typical configuration of a Tensar Marine Mattress.

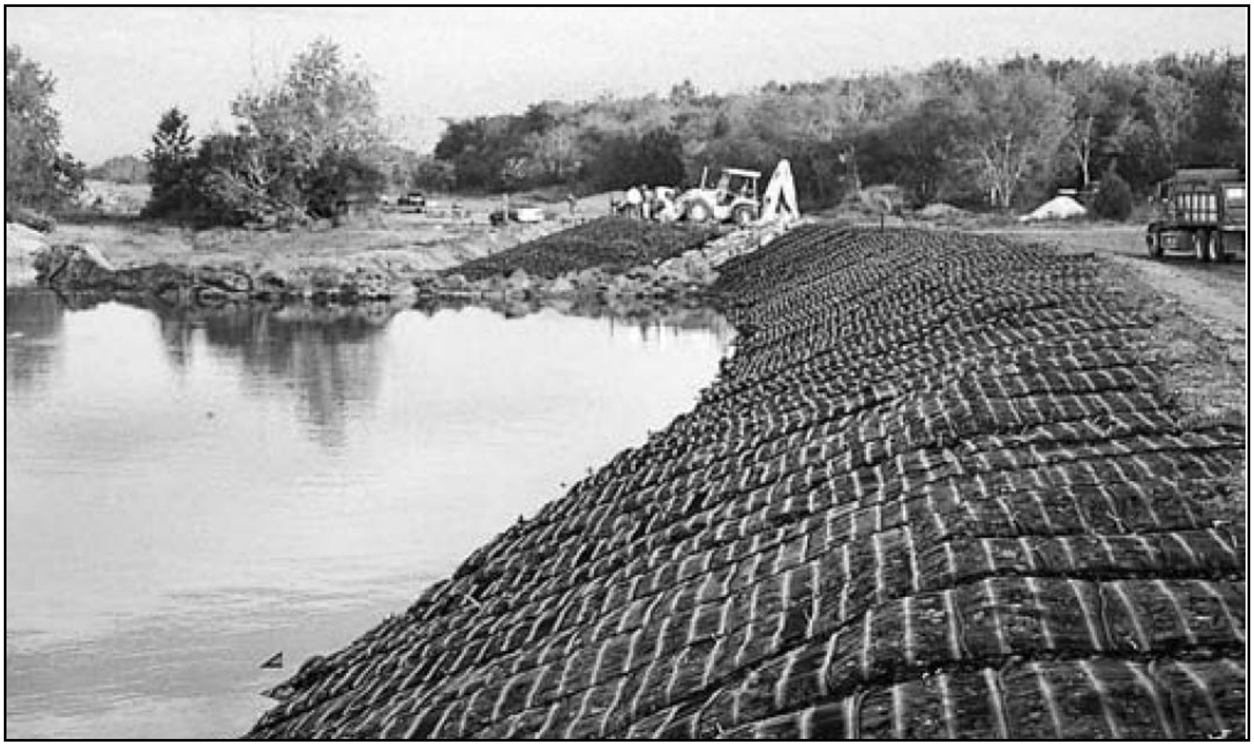


Figure 9 – Completed example of marine mattresses used for riverbank stabilization.

Similar coastal engineering structures have been installed in various locations around the world. One successful structure is located on the north shore of Nantucket Island at 28 Hinckley Lane, which was installed in 2005 and is performing very well. This design consisted of a row of 18 foot long buried marine mattresses placed along the toe of the coastal bank to prevent erosion from wave action. A vegetated dune was constructed over the marine mattresses to preserve the natural look of the property. This installation has been tested by several coastal storms. As shown in Figure 10, the storm waves erode the dune, which acts as a sacrificial sediment source to nourish the beach. The marine mattresses prevent the erosion from progressing any further, which would undermine the toe of the bank and cause subsequent slope failure of the upper bank. At the end of each winter season, the sacrificial dune is reconstructed over the marine mattresses.

Figure 10 – View of a marine mattress with gabion toe coastal engineering structure permitted and installed at 28 Hinckley Lane, Nantucket (a) during construction, (b) after construction, (c) following coastal storm erosion, and (d) after annual nourishment to re-cover the marine mattress and gabion toe structure.

A 450 linear foot marine mattress in a similar wave environment is located in Cape May, NJ (see Figure 11). This marine mattress was installed in 1995 by the New Jersey Department of Environmental Protection. The 50-year design wave height for Cape May, NJ is listed as 22.6 feet by the USACE (1997). The toe of the structure is covered by 2-4 ton stone for scour protection in contrast to the gabion baskets at Hinckley Lane. Figure 11 shows three remnant concrete footers of a structure lost to erosion in the foreground. Also shown is the end of the structure which has been undermined due to flanking. This underscores the importance of designing returns in a marine mattress structure.

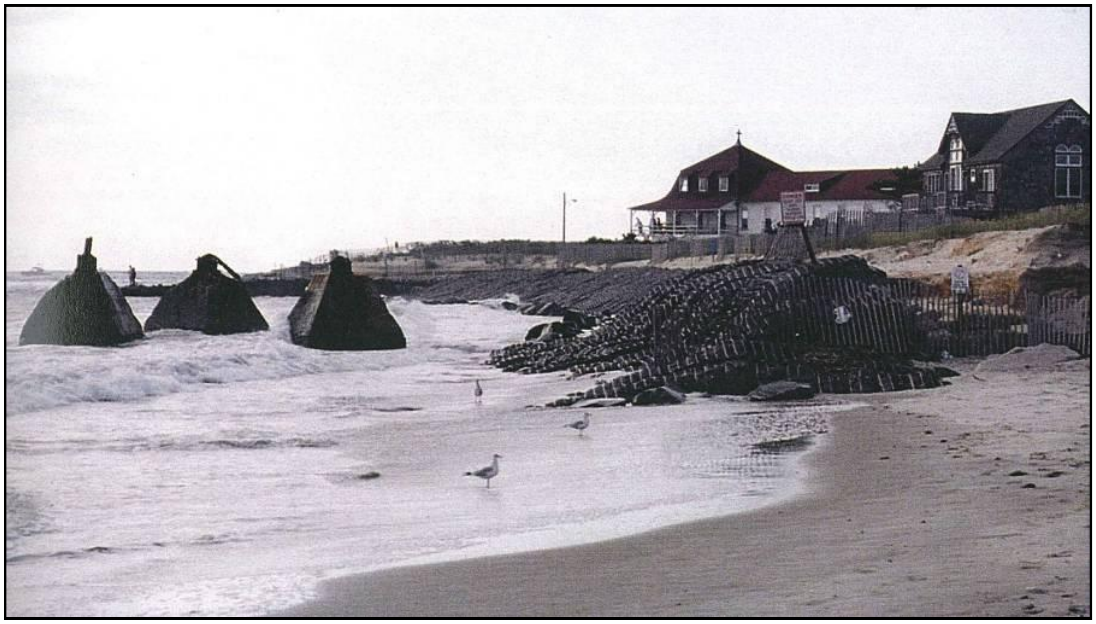


Figure 11 – Marine mattresses installed along the shoreline in Cape May, NJ.

The marine mattress at Cape May has been in place for 15 years. Figure 12 and Figure 13 are Google Earth images showing the exposed condition of the structure in 2002 and post USACE beachfill. The storm season during the winter of 1997 and 1998 was particularly energetic, resulting in a Presidential Disaster Declaration for New Jersey's coastal counties.

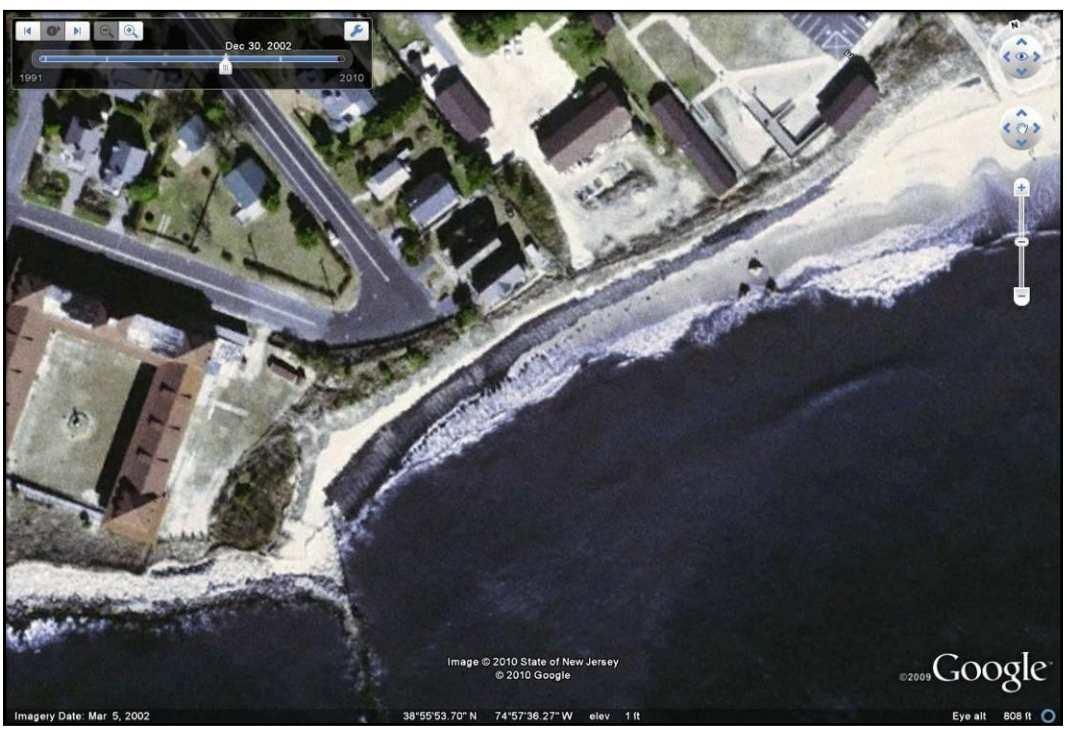


Figure 12 – Google Earth image of marine mattresses installed along the shoreline in Cape May, NJ.

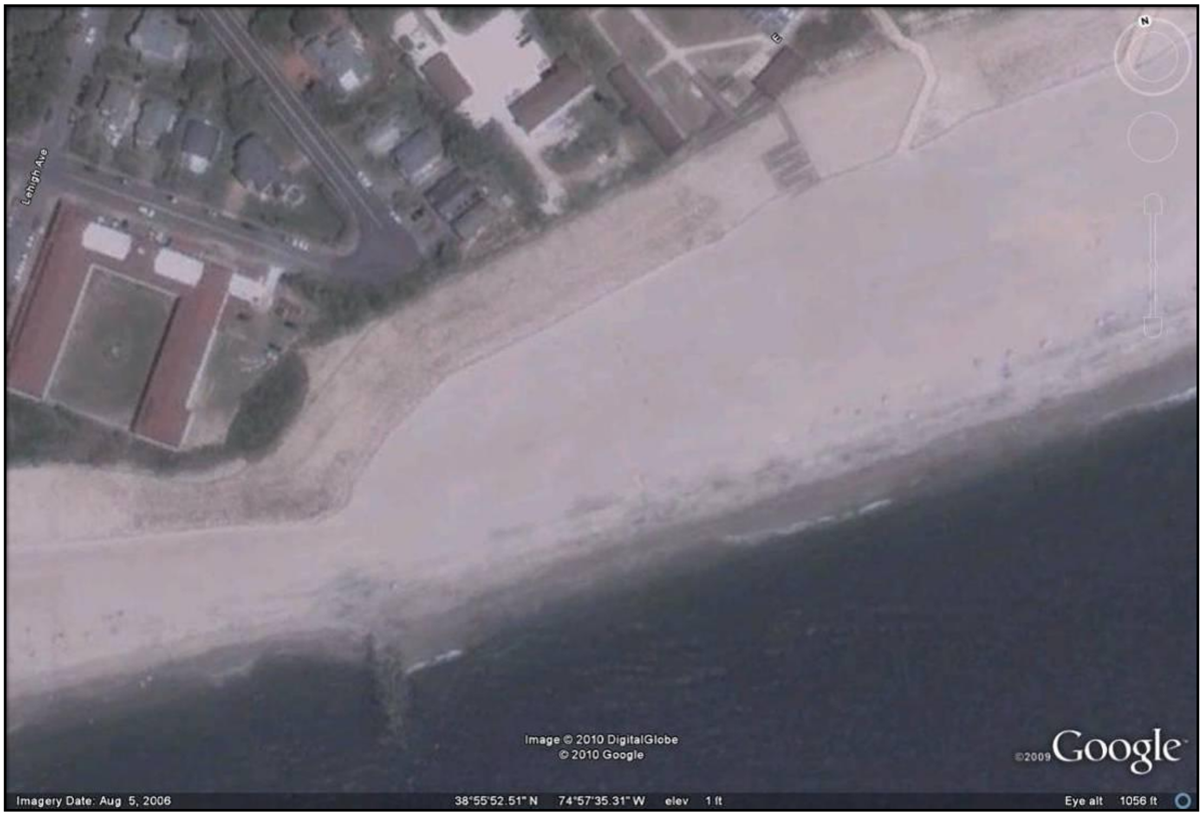


Figure 13 – Google Earth image of marine mattresses installed along the shoreline in Cape May, NJ after a 2006 USACE beachfill.

4.2.4. Cycle 1 Preferred Alternatives

A summary of the available alternatives considered at the project location are shown in the table below.

Table 9 - Cycle 1 alternatives matrix.

	Alternative	Satisfy Design Parameters	Ease of Construction*	Required Maintenance*	Cost*	Considered for Cycle 2
1	No Action	no	N/A	high **	none	no
2	Geotextile Tubes	Yes	moderate	moderate	high	Yes
3	Marine Mattress with Gabion Toe	Yes	simple	less	moderate to high	Yes

* - relative to other alternatives

** - damage to upland infrastructure expected to be high

4.3. Cycle 2 Analysis

Conceptual design layouts were created using the geotextile tube alternative (Tube+Sand) and the marine mattress with gabion toe alternative (Mattress+Sand). The size and location of the new coastal engineering structures were analyzed on their ability to provide the required level of stabilization of the coastal bank while preserving the coastal beach. Additionally, using "best available measures," the concept design must minimize adverse effects on the adjacent coastal beaches.[1] The two alternatives analyzed during this cycle (the Tube+Sand and Mattress+Sand alternatives) are commonly used, well established techniques for providing this type and level of toe protection. Examples of previous applications of both of these design alternatives are described in section 4.2.2 and 4.2.3. Each layout was evaluated using engineering and scientific judgment and experience with an overall project goal of stabilizing the coastal bank in accordance with the design parameters.

4.3.1. Concept Design-1: Geotextile Tubes and Sand Cover (Tube+Sand)

Design

Concept Design 1 consists of four, 30-foot circumference geotextiles tubes installed in a terraced alignment, with clean sand fill cover. Construction requires excavating the existing profile to +4.5 feet MLW and installing a 3-foot circumference anchor tube and scour apron. Tubes will then be installed and filled on the excavated terraces to approximately 5 feet tall and 11 feet wide. After tubes have been filled, a clean sand fill will be placed to a top elevation of approximately +23.5 feet MLW. The sand fill will be placed on a 1V:2.5H slope to meet existing grade while maintaining a continuous one foot thick sand cover over the filled tubes. A schematic cross section is presented below in Figure 14.

Toe protection is accomplished in this option by use of a scour apron and anchor tube. The bottom tube is installed with a base elevation of +4.5 feet MLW. This elevation is determined to be adequate for scour resulting from a 50-year storm. The scour apron extends for ten (10) feet seaward and ends with a three (3) foot circumference sand tube anchor. Where the seabed is subject to further erosion or scour at seaside edge of the revetment, the extended scour apron will dynamically adjust by lowering down; under the weight of the anchor tube weight; and will protect against further erosion and scour stabilization will ultimately be achieved. This will account for the back-to-back 25-year storm scenario.

[1] *See* Coastal Bank performance standard at 310 CMR 10.30 (3)(a). "Best Available Measures" is defined at 310 CMR 10.04 as "the most up-to-date technology or the best designs, measures or engineering practices that have been developed and that are commercially available."

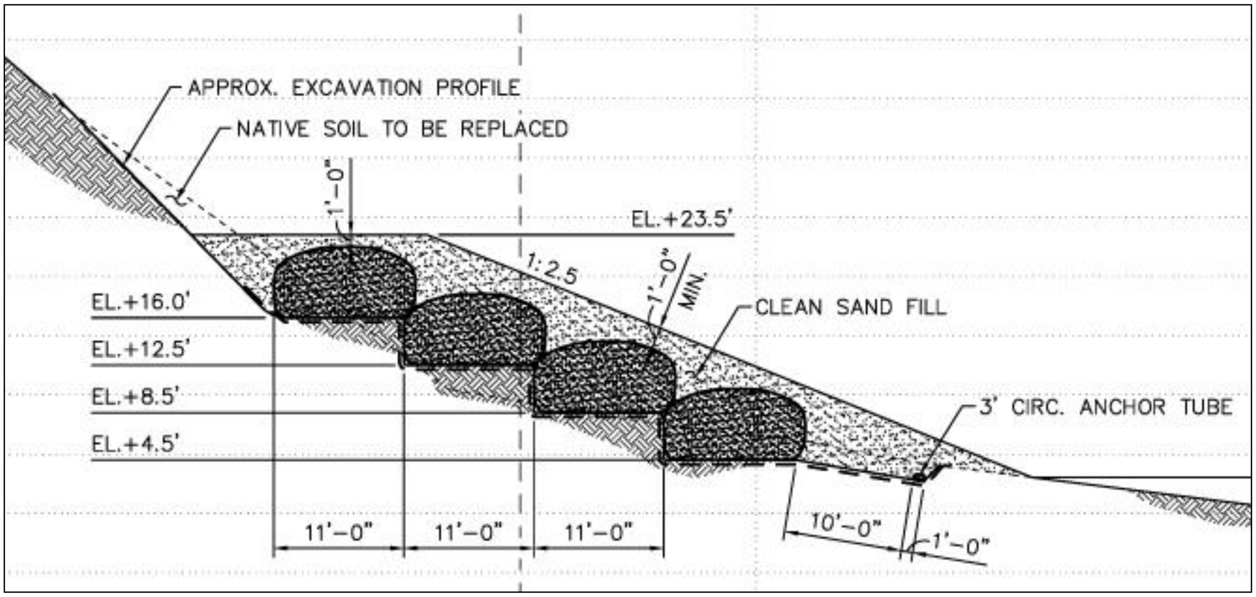


Figure 14 – Schematic cross section of Concept Design 1.

Maintenance

Maintenance of the Tube+Sand system consists of replacing the sand covering once per year. In the event of a storm, the system may become uncovered. An inspection of the uncovered tubes should be undertaken when visible to check for holes or any obvious signs of movement or settling. The project is being designed such that major damage is not expected except for major storm events or combinations of storms having exceptional duration. During a severe storm event, the geotextile tube with scour apron and anchor tube may experience complete or partial loss of sand covering due to wave attack and scour at the toe of the structure. This is expected and will not compromise a properly maintained system. The system was designed with a scour apron and anchor tube to prevent undermining and destabilizing of the main geotextile tube's foundation. The scour apron prevents the system from scouring between the main geotextile tube and the anchor tube. Any depression that develops due to scour beneath and seaward of the free end of the anchor tube will cause the anchor tube to adjust downward into it. This adjustment will therefore hinder the progression of future scour that could potentially undermine and destabilize the main geotextile tube.

In an event where the geotextile tube exposed and is damaged, the damaged section should be repaired. Holes smaller than 3 inches in diameter can be closed by the use of cable ties if there is slack in the geotextile, a piece of nonwoven geotextile inserted in the hole, or the "sock method" which is to put a sock or other small flexible container in the hole and fill with expanding foam to form a seal. Damage including holes larger than 3 inches in diameter but not larger than 8 feet by 2 feet (axial length by circumferential width) should be repaired by a competent contractor utilizing a plywood repair or a patch attached by hog rings or sewing with heavy duty polyester thread. An alternative repair uses a neoprene gasket, backing ring and firm plastic plate in the shape of the repair placed over the hole and bolted into place.

For the purposes of the Alternatives Analysis level of design, the yearly maintenance costs are estimated to be $16,750. This estimated cost includes two components: 1) the mobilization of a contractor once every year to repair holes caused by debris, and 2) the mobilization of a contractor every ten years to replace failed tubes. Other actions such as post-storm inspections and minor repairs (holes less than three inches) are considered within the ability of the homeowners or SBPF to accomplish. The cost of sand replacement has been accounted for in a separate yearly cost.

Removal of the System

The removal of the Tube+Sand system, if required by the regulatory agencies, would incur the following costs; mobilization of a barge with crane, slicing open the sand-filled tubes, disposing of the geotextiles in a landfill. Re-grading of the excavated sand is anticipated. For the purposes of the Alternatives Analysis level of design, the costs of removal are expected to be 60% of the initial construction mobilization costs (due to no pumps needed), and 15% of the costs of the geotextile tubes and scour aprons. The anticipated estimate for removal is $440,000.

Costs

It has been assumed elsewhere in the report that the average annual amount of sand lost from the coastal beach system is 6.6 cubic yards per linear foot. At a cost of $50 per cubic yard, the annual sand nourishment costs for this project area, regardless of which design option is chosen, is $561,000. Additionally, the face of the bank is to be vegetated above the protective structure. This vegetation would be to counteract surface erosion, not deep-seated geotechnical adjustments. Therefore, a cost effective way to vegetate the 139,400 square feet of exposed bank face would be planting beach grass. The cost estimate is approximately $118,490, and is applicable to both design options.

An order of magnitude cost estimate for the initial construction of this concept is provided in the table below.

Table 10 - Preliminary cost estimate for Concept Design 1.

Option 1		
Geotextile tubes		
Project length is 1,700 linear feet		
Item	**Unit**	**Cost**
Geotextile tubes	6,800 l.f	$ 1,020,000
Scour aprons	130,000 sf	$ 650,000
Sand for tubes	16,000 cy	$ 800,000
Sand for cover	13,250 cy	$ 662,500
vegegtation	139,400 sf	$ 118,490
Subtotal		$ 3,250,990
Mobilization (10%)		$ 325,099
Contingency (25%)		$ 812,748
	TOTAL	**$ 4,388,837**
Expected annual maintenance for tubes and sand		$577,750

With the tube configuration, the slope of the tube structure is 1V:2.5H. This means that the toe of the sand fill is further out from the existing toe of the bluff. Based on Concept Design 1, both the minimum and maximum amount of sand to be placed on the tube structure is the same, and is presently estimated to be 7.8 cubic yards per linear foot. Over the 1,700 linear foot project length, the total quantity of material required is 13,250 cubic yards. At $50 per cubic yard, the cost is approximately $662,000.

In the case of a project smaller than the full 1,700 linear feet, the costs will be similar on a per linear foot basis, with an increase in the mobilization percentage. For example, a 300 linear foot tube project, with 50-foot returns, may have mobilization costs of 20%, resulting in an estimated project cost of $1,080,675.

Advantages vs. Disadvantages

An advantage to the Tube+Sand option is that, if a failure does occur or the tubes need to be removed, the sand filling the tubes us simply introduced into the littoral system and the only disposal requirements is the fabric. On the other hand, if marine mattresses needed to be removed, disposing of the stone fill may become an issue.

There are several disadvantages associated with this design concept. First, due to the amount of sand required for filling the tubes, the costs are much higher. Also, mobilizing equipment to the beach face may present coordination issues during construction. Another disadvantage is the construction time and project duration associated with filling four 1,700 foot long geotextile tubes. Experience has shown that marine mattresses can be installed at a much quicker rate than filled geotextile tubes.

4.3.2. Concept Design-2a: Marine Mattress with Gabion Toe (Mattress+Sand)

Design

Concept Design 2 consists of stone filled marine mattress placed at the base of the coastal bank on a 1V:1.5H slope with gabions at the toe. Sand fill will be required to create this grade and the placement of a scour apron.

The gabion toe protection at the seaward end of the marine mattresses will consist of two rows of 6 ft x 4 ft x 6 ft geogrid baskets filled with 4- to 6-inch stone, which will provide a total width of twelve feet of scour protection in front of the marine mattresses. The scour elevation at the seaward row will be +2.5' MLW to account for two 25-yr storms in succession; the inner row will be placed two (2) feet higher to provide the elevation transition.

After the marine mattress and gabion structure is installed, a clean sand fill will be placed to cover the system and provide a sacrificial berm to nourish the beach. The berm will extend 10 feet seaward of the landward edge of the Mattress+Sand structure and then be graded on a 1V:2H slope to meet existing grade, maintaining a consistent cover of one foot over the structure.

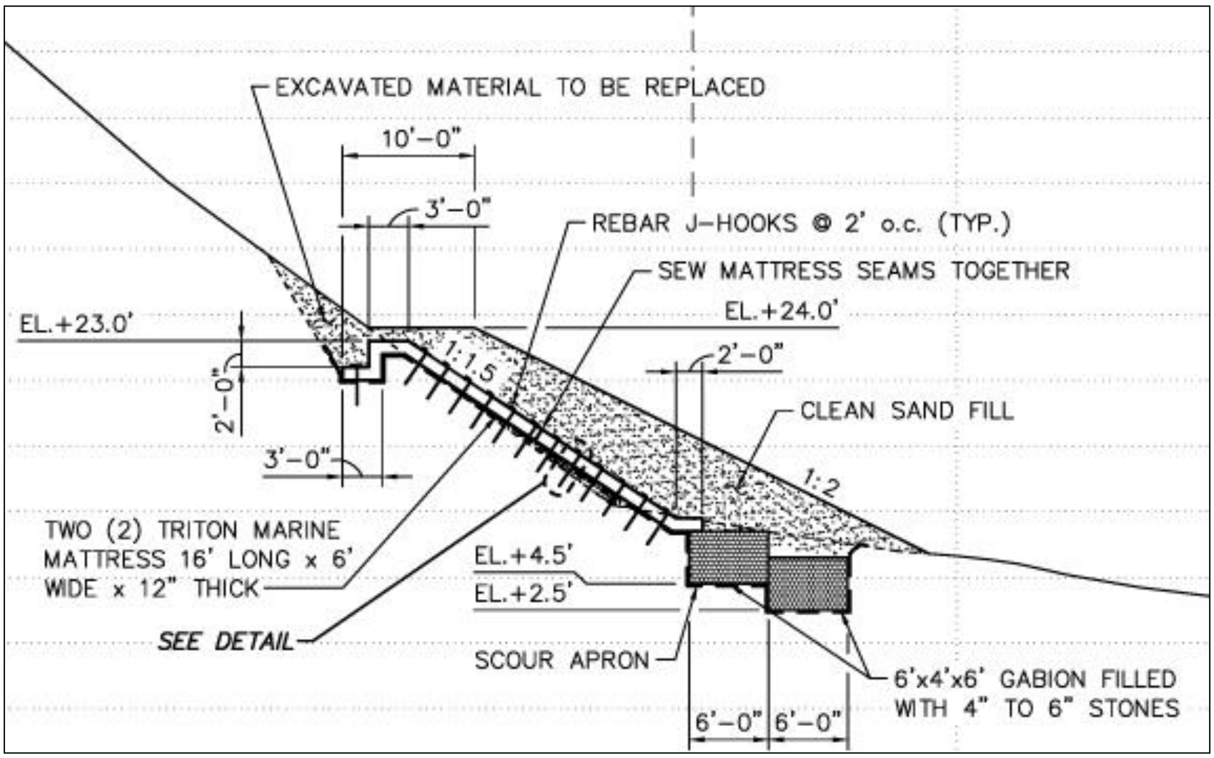


Figure 15 – Schematic cross section of Concept Design 2.

Maintenance

Maintenance of the Mattress+Sand system consists of replacing the sand covering once per year. In the event of a storm, the system may become uncovered. An inspection of the uncovered mattress should be undertaken when visible to check for broken ribs or any obvious signs of movement or settling. The project is design accounts for the anticipated wave forces and scour, such that the main concern would be damage to exposed geogrids from any heavy debris in the surf zone. In the event of damage, the geogrids may be repaired in place with 30 pound tensile strength polyester or polyethylene cable ties (not nylon). If any individual compartments were to break open, the stone should be replaced, and the section of failed geogrid patched.

Degradation of the geogrid from exposure to the sun and marine environment is not a concern for the geogrids because all of the components are made from either HDPE or copolymer (HDPE and PP) with carbon black to prevent UV degradation. One of the oldest marine mattress projects installed by Tensar is located at the end of the runway at Logan Airport in Boston, which was installed in 1994. The expected life span of the geogrids is expected to be up to 40 years, unless subjected to physical damage from large debris.

For the purposes of the Alternatives Analysis level of design, the yearly maintenance costs are estimated to be $3,000. This estimated cost includes the mobilization of a contractor once every five years to replace stone and sections of geogrid. Other actions such as post-storm inspections and minor repairs are considered within the ability of the homeowners or SBPF to accomplish. The cost of sand replacement has been accounted for in a separate yearly cost.

Removal

The removal of the Mattress+Sand system, if required by the regulatory agencies, would incur the following costs; mobilization of a barge with crane, removal of the stone-filled mattresses, opening the mattress compartments, disposal of the geogrid in a landfill, recycling of the rebar and re-use of the stone. No re-grading of the bank is anticipated. For the purposes of the Alternatives Analysis level of design, the costs of removal are expected to be 90% of the initial construction mobilization costs, and 10% of the costs of the marine mattresses and gabion baskets. The anticipated estimate for removal is therefore $300,000.

Costs

It has been assumed elsewhere in the report that the average annual amount of sand lost from the coastal beach system is 6.6 cubic yards per linear foot. At a cost of $50 per cubic yard, the annual sand nourishment costs for this project area, regardless of which design option is chosen, is $561,000. Additionally, the face of the bank is to be vegetated above the protective structure. This vegetation would be to counteract surface erosion, not deep-seated geotechnical adjustments. Therefore, a cost effective way to vegetate the 139,400 square feet of exposed bank face would be to plant beach grass. The cost estimate is approximately $118,490, and is applicable to both design options.

An order of magnitude cost estimate for the initial construction of this design concept is provided in the table below.

Table 11 - Preliminary cost estimate for Design Concept 2.

Option 2			
Marine Mattress w/ gabion basket			
Project length is 1,700 linear feet			
Item	**Unit**	**Cost A**	**Cost B**
gabion baskets	568 units	$ 17,000	$ 17,000
marine mattresses	68,000 SF	$ 1,156,000	$ 1,156,000
sand			
small profile	9,450 cy	$ 472,300	N/A
large profile	30,700 cy	N/A	$ 1,543,800
vegegtation	17 days	$ 118,490	$ 118,490
stone for baskets	3,030 cy	$ 151,500	$ 151,500
rebar		$ 200,000	$ 200,000
Subtotal		$ 2,115,290	$ 3,186,790
Mobilization (10%)		$ 211,529	$ 318,679
Contingency (25%)		$ 528,823	$ 796,698
	TOTAL	**$ 2,855,642**	**$ 4,302,167**
Expected annual maintenance for matresses and sand			$561,300

In the case of a project smaller than the full 1,700 linear feet, the costs will be similar on a per linear foot basis, with an increase in the mobilization percentage. For example, a 300 linear foot marine mattress project, with 50 ft returns, may have mobilization costs of 15%, resulting in an estimated project cost of $669,300.

Advantages vs. Disadvantages

With the Mattress+Sand configuration, the slope of the structure can be steeper at 1V:1.5H. This means that the toe of the sand fill can be to the existing toe of the bluff allowing more usable beach. Conversely, the sand fill can be designed to be greater, extending seaward to a similar position on the beach as the tube configuration. Therefore, there is a range of sand fill quantities. Based on Design Concept 2, the maximum amount of sand to be placed on the marine mattress revetment is presently estimated to be 18 cubic yards per linear foot. Over the 1,700 linear foot project length, the total quantity of material required would be 30,700 cubic yards. At $50 per cubic yard, the cost is $1,534,800. The minimum amount of sand that could be placed and still provide adequate covering of the marine mattress structure is 5.5 cubic yards per linear foot totaling 9,450 cubic yards at a cost of approximately $472,300.

4.3.3. Cycle 2 Alternatives Matrix

A summary of alternatives is presented in the table below:

Table 12 - Alternatives Comparison Matrix

Alternative	Benefits	Disadvantages	Achieve Goals	COSTS			Permitting Issues	Expected Periodic Renourishment
				Construction	Maintenance	Removal		
(1) No Action	No required construction No direct costs associated with this alternative	The beach and bluff face will continue to erode. Potential for further endangerment of landward infrastructure including homes. There is a potential for indirect costs related to damage incurred during storm events.	No, this alternative does not achieve the project goals of stabilizing the coastal bank and preserving the beach.	None	None	None	None	1 year and after a major storm event
(2) Geotextile Tubes + Sand	Considered a "softer" solution than stone and concrete. Tubes have an inherent flexibility to accommodate foundation changes. Less material to dispose of if removal is required.	Although tubes have flexibility, too much scour at the toe could potentially lead to structural failure. More susceptible to damage from vandalism and debris. Susceptible to UV degradation. Mobilization of equipment to island could increase project costs. Long duration of construction time to fill 1,700 linear feet of geotextile tube (times four).	Achieves project goals of protecting the toe of the bluff and preserving the beach. This alternative is designed for a 50-year storm.	$4.4 Million	$16,750	$440,000	Moderate to Difficult. May be seen in a slightly more favorable light by regulators because of the "soft" nature of the tubes as well as the ease of removal.	11,000 cubic yards annually
(3) Marine Mattresses + Sand	Steeper design slope of bank requires less fill material, although more sand could be added for a larger beach. Marine mattresses can be installed quicker than filling of geotextile tubes resulting in shorter construction timelines. Less susceptible to vandalism and damage from debris.	Disposal of mattresses and stone fill could be problematic if removal is required. Less flexibility than geotextile tubes. Considered a "harder" solution.	Achieves project goals of protecting the toe of the bluff and preserving the beach. This alternative is designed for a 50-year storm.	$2.8 Million to $4.3 Million	$3,000	$300,000	Difficult - Placing hard structures on a beach and bluff is usually discouraged by regulators.	11,000 cubic yards annually

5. Recommended Design

Based on the foregoing analyses, the recommended design that meets the regulatory design standard of “best available measures” is the marine mattress+sand alternative. Both options were designed to provide similar stabilization to the coastal bank to provide storm damage prevention for homes from a 50-year probability storm event. Therefore both options have similar crest elevations. Differences in the geometry of the tubes and marine mattresses result in differences in the footprint and amount of initial sand required. Additionally, the marine mattress+sand design has flexibility in both the dimension (thickness) and installation approach thereby allowing confidence that the final design will result in a robust coastal engineering structure.

The cost of imported sand adds to the cost of the geotextile tube+sand design when compared to the marine mattress+sand design. It was also determined that the construction duration of the marine mattresses is likely shorter than the tubes, and require less specialized skills. This is due to the fact that four rows of geotubes are required, and each row must be filled individually by personnel having experience pumping slurry into tubes. The marine mattresses however, can be pre-fabricated and either pre-filled with stone, or filled on site by the contractor. These details will be finalized during the final design phase.

It has been determined that the average annual amount of sand lost from the coastal beach system is 6.6 cubic yards per linear foot. At a cost of $50 per cubic yard for sand delivered in-place, the annual nourishment costs for the project area, regardless of which design option is chosen, is approximately $561,000.

Additionally, the face of the bank is to be vegetated above the protective structure. The purpose of the vegetation is to stabilize the middle and upper slopes of the coastal bank from surface erosion, not deep-seated geotechnical adjustments. Several options exist for vegetating the bank including hydroseeding, planting beach grass or planting woody vegetation. It is recommended that beach grass be used to vegetate the slope because this type of vegetation has a stronger root system and has previously been permitted on Nantucket Island. The cost for this option is $0.85 per square foot, compared to woody vegetation that costs up to $10 per square foot.

The beach grass plantings include a jute erosion control net that is installed until the root system fully develops. If the coastal bank is vegetated from the top of the marine mattress to the top of the bluff, this results in a total area of 139,400 square feet over the project area. The cost for this is $118,490, and is applicable to both design options. If a more robust design is required to address stormwater drainage, ground water seepage or to stop future adjustment of the existing bank slopes, then a cost for this will need to be factored in.

6. Cycle 3 Final Design Analysis

During the Cycle 2 Analysis, preliminary designs were developed for the comparison of the Tube+Sand concept to the Mattress+Sand concept for the 50-year storm event. Factors such as impacts to the environment, overall functionality of the system and construction and maintenance costs were compared, resulting in the conclusion that the most suitable alternative for the project was the Mattress+Sand option.

The Mattress+Sand concept was further refined during the Cycle 3 analysis. Because standard coastal engineering practice generally considers a 100-year storm event as the most appropriate level for design, OCC increased the assumed wave, wind and storm surge parameters to upgrade the 50-year storm design to the more severe storm conditions produced by the 100-year event.

While the upgrading to a 100-year storm parameters results in a more substantial design, the associated labor costs will only increase slightly, since the overall construction process remains unchanged. Thus, because the 100-year design offers greater protection for only a relatively marginal cost increase, the 100-year design for the Mattress+Sand concept is the recommended final design alternative.

6.1. Coastal Engineering Criteria

The following design criteria were used to provide protection from the anticipated 100-year storm conditions:

- High Tide Line (HTL) Elevation = +4.97 feet MLW
- Beach elevation at toe of coastal bank = +8.0 feet MLW
- Estimated depth of scour at the bank toe = 8.0 feet.
- 100-year design storm Still Water Level (SWL) Elevation = +10.2 feet MLW
- To protect the lower slope from erosion due to wave run-up during 100-year storm conditions, the top of the proposed coastal engineering structure should be extended to Elevation +25.0 feet MLW;

6.2. Final Concept Design for 100-Year Storm

Design

As described previously for the 50-year storm design concept, the lower portions of the bank slope will be prepared initially by grading the existing lower slope as required along a 1,700-foot-long stretch of coastal bank area to provide a maximum stable slope of 1V:1.5H prior to placement of the marine mattresses.

Following slope preparation a scour apron, consisting of three rows of 4 ft x 5 ft x 6 ft gabion baskets will be buried along the toe of the prepared coastal bank. Each gabion basket will be filled with 12- to 22-inch diameter stones. These toe gabions will provide a 15-foot wide scour apron seaward of the marine mattress array. The base elevation of the seaward row of gabions will lie at +0.0 feet MLW, the base of the middle row will lie at +2.0 feet MLW and the landward row will lie at +4.0 feet. This design will provide the intended level of scour protection from a 100-year storm event

After the gabion baskets are constructed, an array of stone-filled marine mattresses will be installed. The fill for the mattresses will consist of angular crushed stone approximately 3- to 6-inches in diameter. The bottom of the mattress array will be located at the top of the landward toe gabions and will then extend up the bank face to an elevation of approximately +25 feet MLW. The mattresses will be placed for a collective dimension of 38-feet long x 6-feet wide x 18-inches thick. The end-to-end splicing of the two adjacent mattresses will be accomplished using an HDPE bar to form a bodkin connection (as recommended by the manufacturer). Mattresses will be anchored to the existing bank slope using J-Hooks fabricated from #4 steel reinforcing bars, spaced approximately 4 feet on center.

In addition to the Mattress+Sand system that runs parallel to the shoreline, return walls consisting of stone filled gabions will also be constructed at either end of the system to protect against flanking erosion. The object of the return walls will be to provide protection for exposed faces of the slope that run perpendicular to the shoreline. The return walls will consist of a total of two layers of gabion baskets. The bottom row will consist of three adjacent 4 ft x 5 ft x 6 ft gabion baskets filled with 3- to 6-inch diameter stone and the top row will contain two adjacent 4 ft x 4 ft x 6 ft gabion baskets filled with 3 to 6-inch diameter stone.

The total number of return walls required will be dependant on the number of residents that elect to participate in the project. A return wall will be required each time the system is terminated. Specifically, if the system runs continuous for multiple properties, return walls will only be required at the start and end, however if the system is installed at every other property, return walls will be required at each property line.

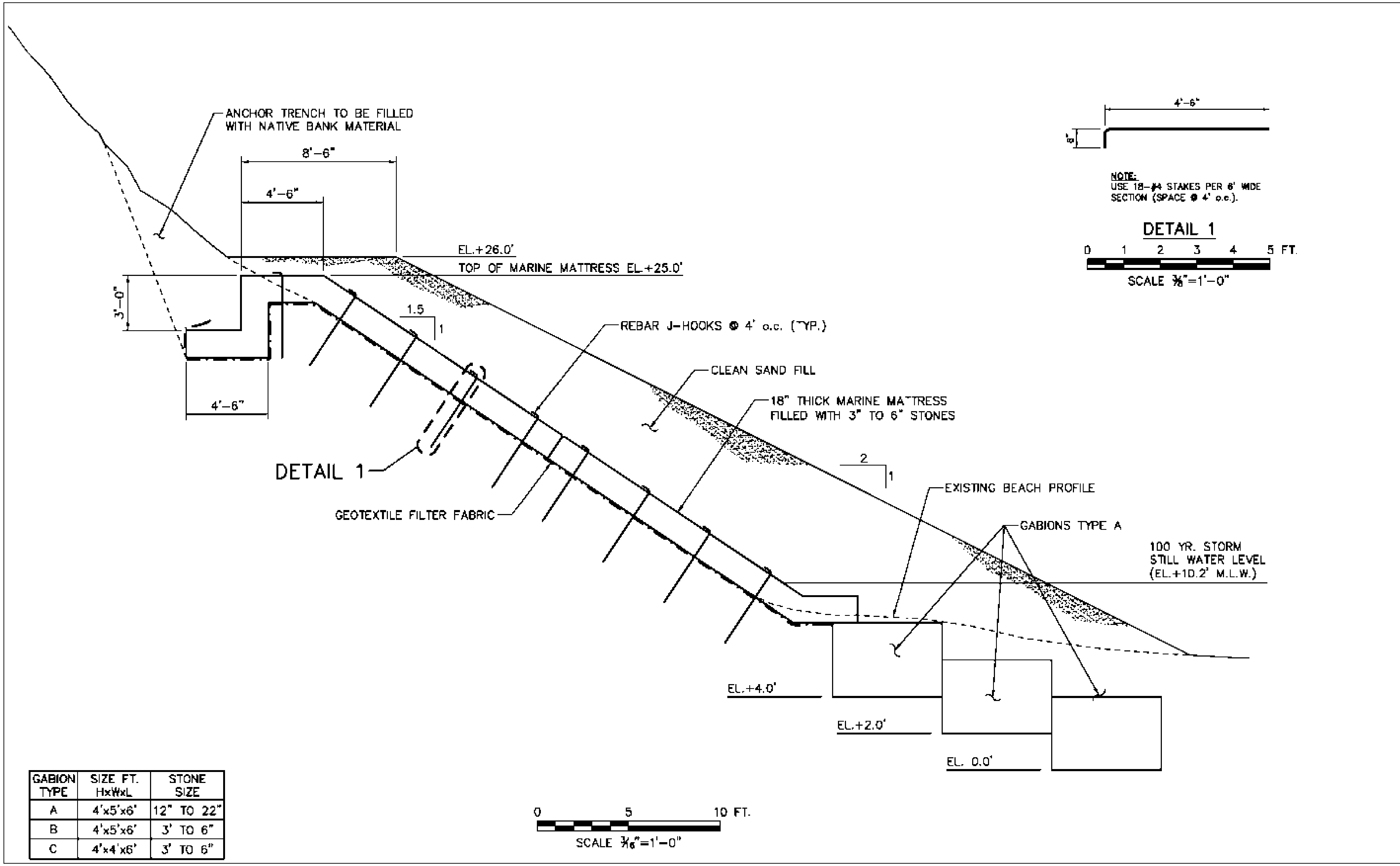
The final construction element will consists of placing a minimum 12-inch-thick layer of clean, beach compatible sand over the marine mattress array and toe gabions such that the berm will extend 15 feet seaward of the landward edge of the structure and will form a 1H:2V grade.

Maintenance

Maintenance requirements for the structure will the same as required for the 50-year storm design concept and will consist primarily of replenishing the sacrificial sand berm covering the mattresses and toe gabions as necessary to maintain the contribution of beach-compatible sediment from the Project area. Additional maintenance may be required on occasion to repair or patch any damaged geogrid or to replace any associated lost fill stone.

Removal of the System

Costs for removal of the mattresses and gabions, if required by the regulatory agencies, are similar to those projected for removal of the 50-year design concept, as the labor required would be similar.



GABION TYPE	SIZE FT. HxWxL	STONE SIZE
A	4'x5'x6'	12" TO 22"
B	4'x5'x6'	3" TO 6"
C	4'x4'x6'	3" TO 6"

Figure 16 - Schematic cross section of Final Design Concept for 100-Year Storm Event

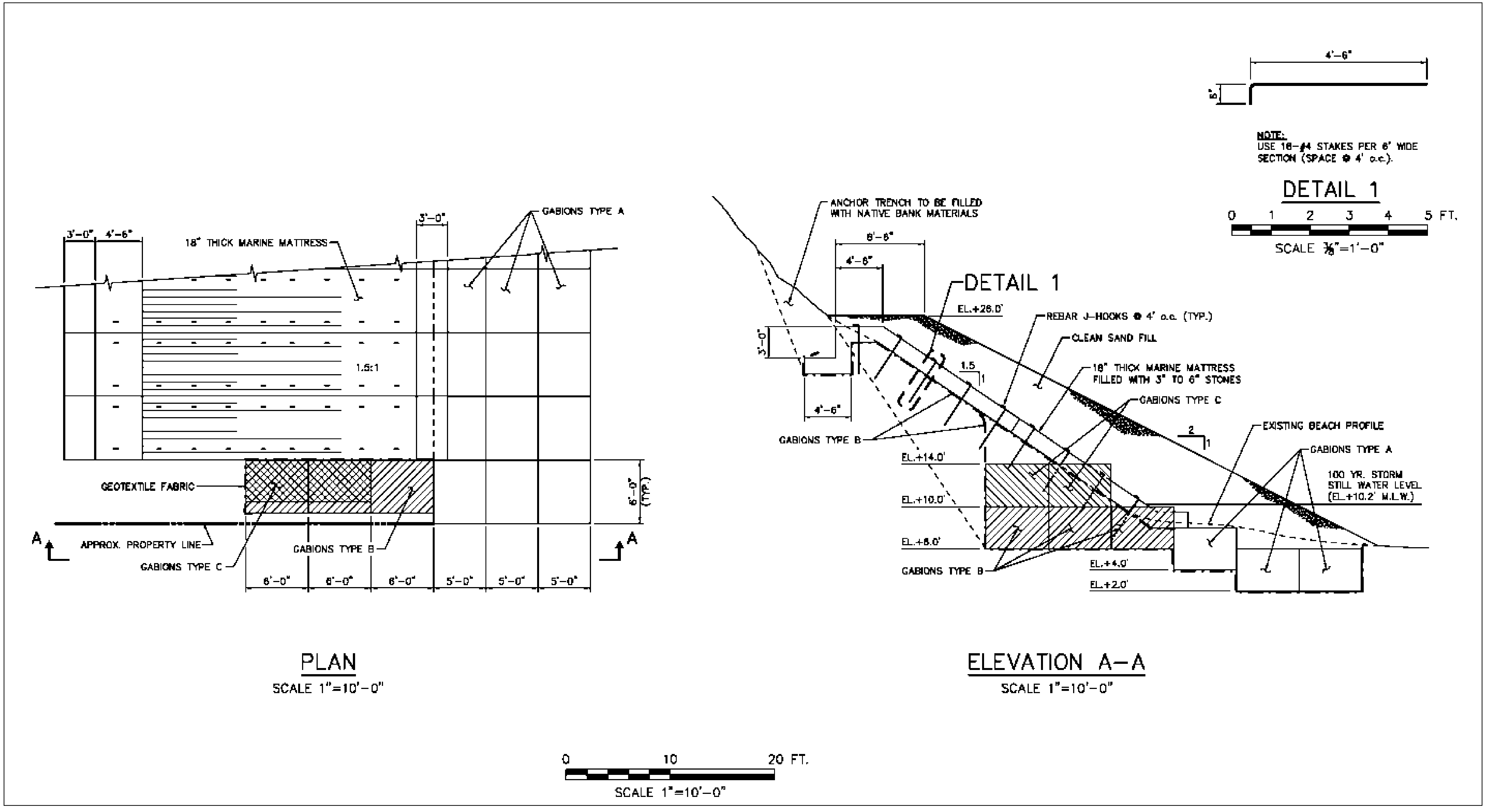


Figure 17 - Schematic of Return Walls for Final Design Concept for 100-Year Storm Event.

Costs

Based on the more comprehensive design concept obtained in the Cycle 3 Analysis, an Opinion of Probable Cost (OPC) estimate was completed for the final design concept. Compared to the Order of Magnitude estimate performed for the Cycle 2 analyses, the OPC estimate provides a much more detailed approximation of potential construction costs. The OPC utilizes labor rates based on prevailing wage or Davis-Bacon rates for the geographic location of the site and equipment rates based on those of contractors in the area. Durations of tasks are developed in consideration of the type of activity and any site constraints, material quantities are taken off the drawing or sketches available and priced out using current material costs or actual vendor quotes whenever possible. The amount of contingency is developed as a function of the level of design and the degree of uncertainty in portions of the work. The contingency is generally developed to give the client a realistic budget number that will generally reflect the total construction cost due to any unknowns or potential material cost variations. A summary of the OPC developed for the 100-year storm final design concept is included below. For estimating purposes, a total of eight (8) returns walls were assumed to be required.

Table 13 - Summary of OPC for Final Design Concept for 100-Year Storm Event.

ITEM NO.	WORK ITEM DESCRIPTION	OPC PRICE (LUMP SUM)
1	MOBILIZATION	$50,000
2	MARINE MATTRESS CONSTRUCTION	$6,948,000
3	DE-MOBILIZATION	$37,000
4	GABION RETURN WALLS (8 ASSUMED)	$120,000
	TOTAL	**$7,155,000**
OPINIONS OF PROBABLE COST INCLUDE THE FOLLOWING MARK-UPS:		
	GENERAL CONDITIONS:	**5%**
	OVERHEAD:	**10%**
	PROFIT:	**10%**
	SALES TAX:	**0%**
	ESCALATION:	**0%**
	CONTINGENCY:	**20%**

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