DOWNTOWN MONTAUK

DOWNTOWN MONTAUK STABILIZATION PROJECT

HURRICANE SANDY LIMITED REEVALUATION REPORT

Evaluation of a Stabilization Plan for Coastal Storm Risk Management in Response to Hurricane Sandy
&
Public Law 113-2

BACK-UP CALCULATIONS APPENDIX

U.S. ARMY CORPS OF ENGINEERS

New York District

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1.0 INTRODUCTION

This appendix provides additional “back-up” documentation regarding the historical beach evolution at downtown Montauk, with-project erosion rates and beach replenishment volumes, feasibility level cost estimates for the five alternatives, details of the operations and maintenance cost estimate and construction schedule for the selected alternative.

2.0 BEACH EVOLUTION

2.1 Historical Sediment Budget at downtown Montauk

An Existing Conditions (c. 2001) sediment budget was developed for the entire FIMP study area (USACE-NAN 2007). Downtown Montauk is located at the eastern end of sediment budget cell M4, which includes Hither Hills State Park as well. The existing conditions sediment budget indicates that this cell is relatively stable from 1995 to 2001. However, the observed shoreline changes from 1979 to 1995 (Figure 1) indicate that within the Downtown Montauk Project Area the shoreline eroded on average by approximately 3 ft/yr (0.9 m/yr). In addition, subaerial morphological changes derived from LIDAR measurements collected in 2000 and Nov. 16 2012 indicate that downtown Beach experienced significant beach (-3.7 ft/yr) and dune erosion over this time period.

In light of these observations a background erosion rate of -3 ft/yr is selected for the Project Area.

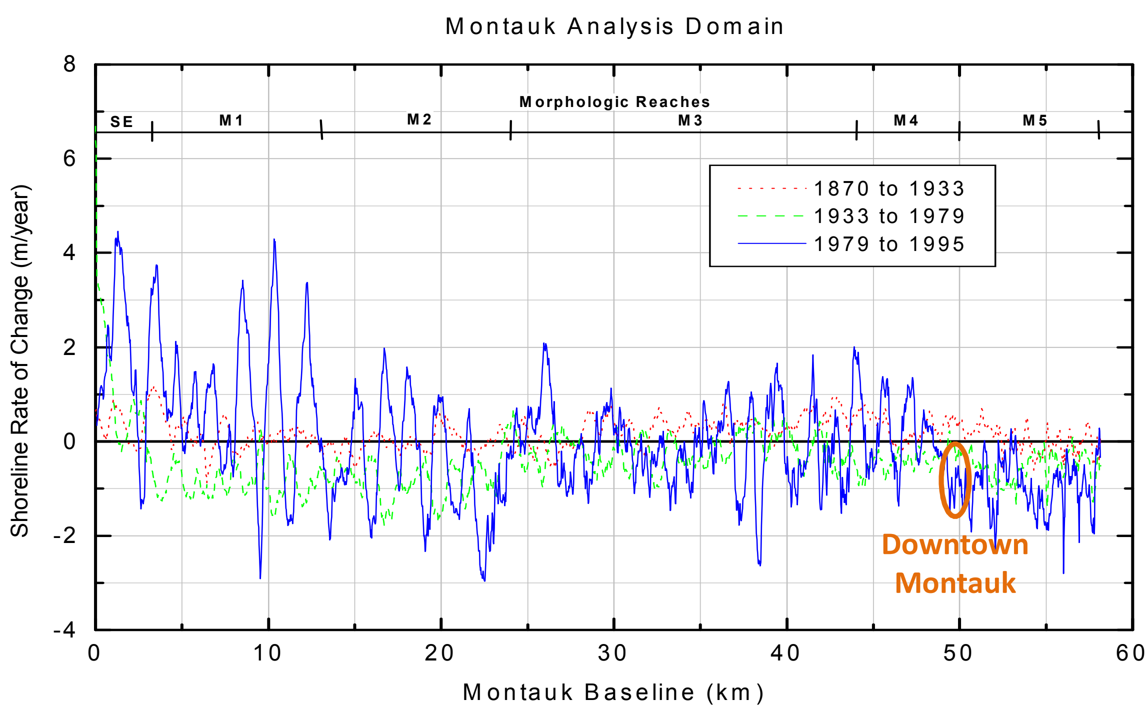


Figure 1: Historic Shoreline Change in Montauk (Gravens et al. 1999)

2.2 Profile Observations at downtown Montauk (1995-2012)

Initiated in 1995, the Atlantic Coast of New York Monitoring Program (ACNYMP), a cooperative effort of the New York State Department of State, U.S. Army Corps of Engineers' New York District and New York Sea Grant, has been collecting information and data on beach changes and coastal processes for the 135-mile stretch of shoreline between Coney Island and Montauk Point. One ACNYMP station, M-34, is located in downtown Montauk and captures the general profile evolution from 1995 to 2002 (Figure 2). Two additional profile lines were added to Figure 2 based extracted from LIDAR data (2012-11-14) and a beach profile survey conducted by Ocean Survey Inc. in August 2013 at the same profile origin (M-34).

The profile surveys show that significant dune erosion has occurred at M-34 since 1995. From 1995 to 2002 the crest elevation and location of the dune crest was relatively stable. Some dune scarping is captured by the 2002-03-09 ACNYMP profile survey (dark red). The two post-Sandy profile surveys (2012-11-14 and 2013-09-17) indicate that the crest elevation of the dune has been lowered and shifted landward. It is unclear from the available profile observations how much dune erosion occurred during Hurricane Sandy and much dune erosion occurred to more typical storm events between 2002 and 2012. The dune recovery observed in the OSI survey (2013-09-17) is attributed to dune repairs by local interest in response to Hurricane Sandy.

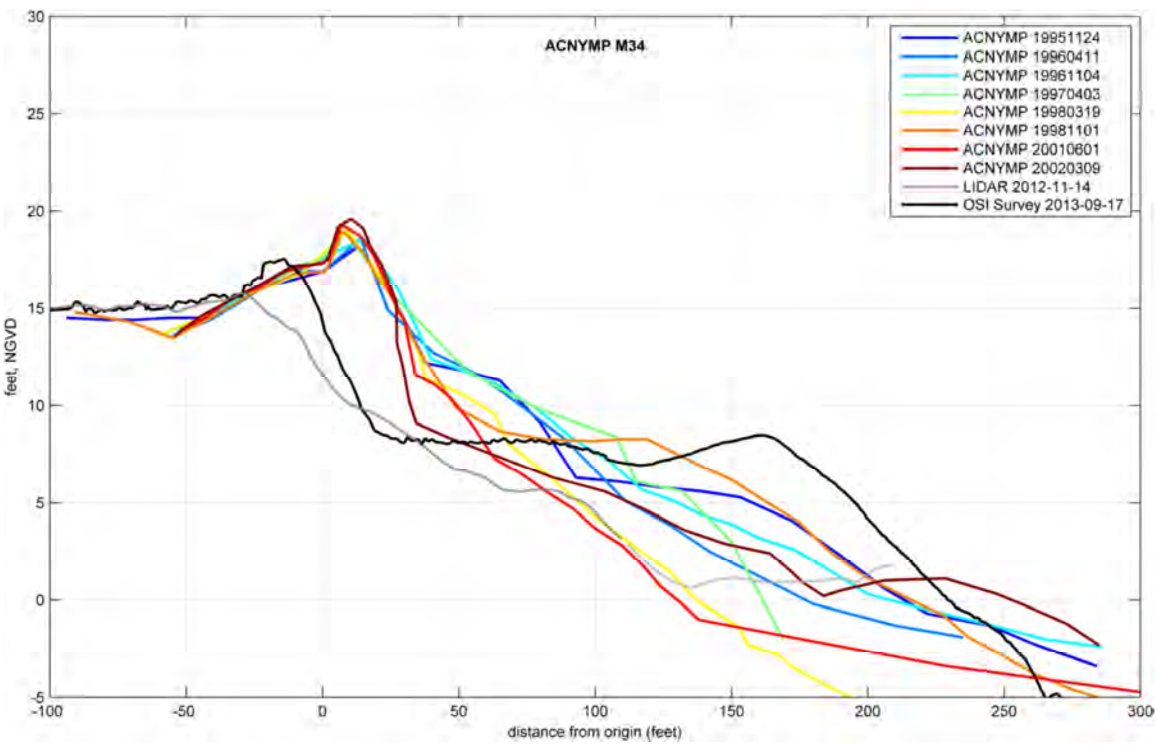


Figure 2: 18-Year Evolution of ACNYMP Profile M34

2.3 Hurricane Sandy

2.3.1 Water Levels & Waves

On 29 October 2012, Hurricane Sandy made landfall approximately five miles south of Atlantic City, NJ, where it collided with a blast of arctic air from the north, creating conditions for an extraordinary and historic storm along the East Coast with the worst coastal impacts centered on the northern New Jersey, New York City, and the Long Island coastline. Hurricane Sandy's unusual track and extraordinary size generated record storm surges and offshore wave heights in the New York Bight. The maximum water level at The Battery peaked at 12.4 feet NGVD29, exceeding the previous record by over 4 feet. Further east, at Montauk Point, the maximum water level reached 6.6 feet NGVD29, 1.4 feet less than the previous storm of record (Hurricane Carol in 1954). Coastal erosion and damages within the FIMP Study Area as a result of Hurricane Sandy were severe, substantial and devastating, particularly along Fire Island and in downtown Montauk. Following Hurricane Sandy, the protective beach in downtown Montauk has been largely eroded leaving many buildings vulnerable to additional damages from future storms.

2.3.2 Observed Subaerial Changes

Prior to Hurricane Sandy the beach at downtown Montauk was characterized by a relatively wide beach berm and sand dunes with heights between +16 and +25 feet NGVD. During Hurricane Sandy the wide beach berm was effectively removed and the dunes experienced severe erosion. The relatively high elevation of the dunes prevented significant overwash and overtopping from occurring in downtown Montauk during Hurricane Sandy except at the gaps in the dunes which provided public beach access. Figure 3 shows profile conditions at four profiles along downtown Montauk in 2000 and 2012 (post-sandy). The post-sandy conditions are characterized by a narrow beach berm and narrower dunes. Despite the dune erosion that occurred, the post-sandy dunes are still relatively high, between +16 and +25 feet NGVD, and provide protection against overwash and overtopping during future storm events. As previously discussed, it is unclear how much of the observed profile changes can be directly attributed to Hurricane Sandy versus other storm events and long-term coastal processes occurring between 2000 and 2012.

A quantitative analysis of the shoreline and dune migration was performed by analyzing the change in the +3 ft NGVD and +11 ft NGVD contours. These contours were selected to characterize the change in the beach and dune widths from 2000 to Nov. 2012. The top panel of Figure 4 shows the position of the contours in 2000 (blue) and Nov. 2012 (red). The bottom panel of Figure 4 shows the change in the horizontal position of the contours over this 12 year period (negative value represents erosion). It is clear that the entire project area experienced significant subaerial beach erosion, as both the shoreline and dune migrated 20 to 60 feet landward. In general the magnitude of shoreline recession is greater than dune recession. Within downtown Montauk (Reach M-1F) the shoreline and dune experience an average landward migration of 44 feet and 31 feet respectively

The beach conditions at downtown Montauk typically undergo a seasonal transformation from a narrower "winter" beach to a wide "summer" beach (Figure 3). During the fall and winter months, storm waves a more frequent and sand from the beach berm is transported offshore and deposited in a protective sand bar. During late spring and summer months, storm events are less frequent and smaller waves dominate, allowing sand to be transported landward restoring the wide summer berm. During particular sever storm events, such as Hurricane Sandy, sand may be transported offshore or downdrift and lost from the system. Beach surveys at Montauk were collected about once every two weeks in the year following Hurricane Sandy, capturing the seasonal variability in the beach conditions at Montauk Figure 6 illustrates the temporal evolution of the beach conditions at Montauk and transition from a winter beach profile to a summer beach profile and then beginning of the transition back to a winter beach profile. Profile measurements collected semi-monthly following Hurricane Sandy provide additional evidence of the seasonal transformation of the beach conditions (Figure 7 and Figure 8).

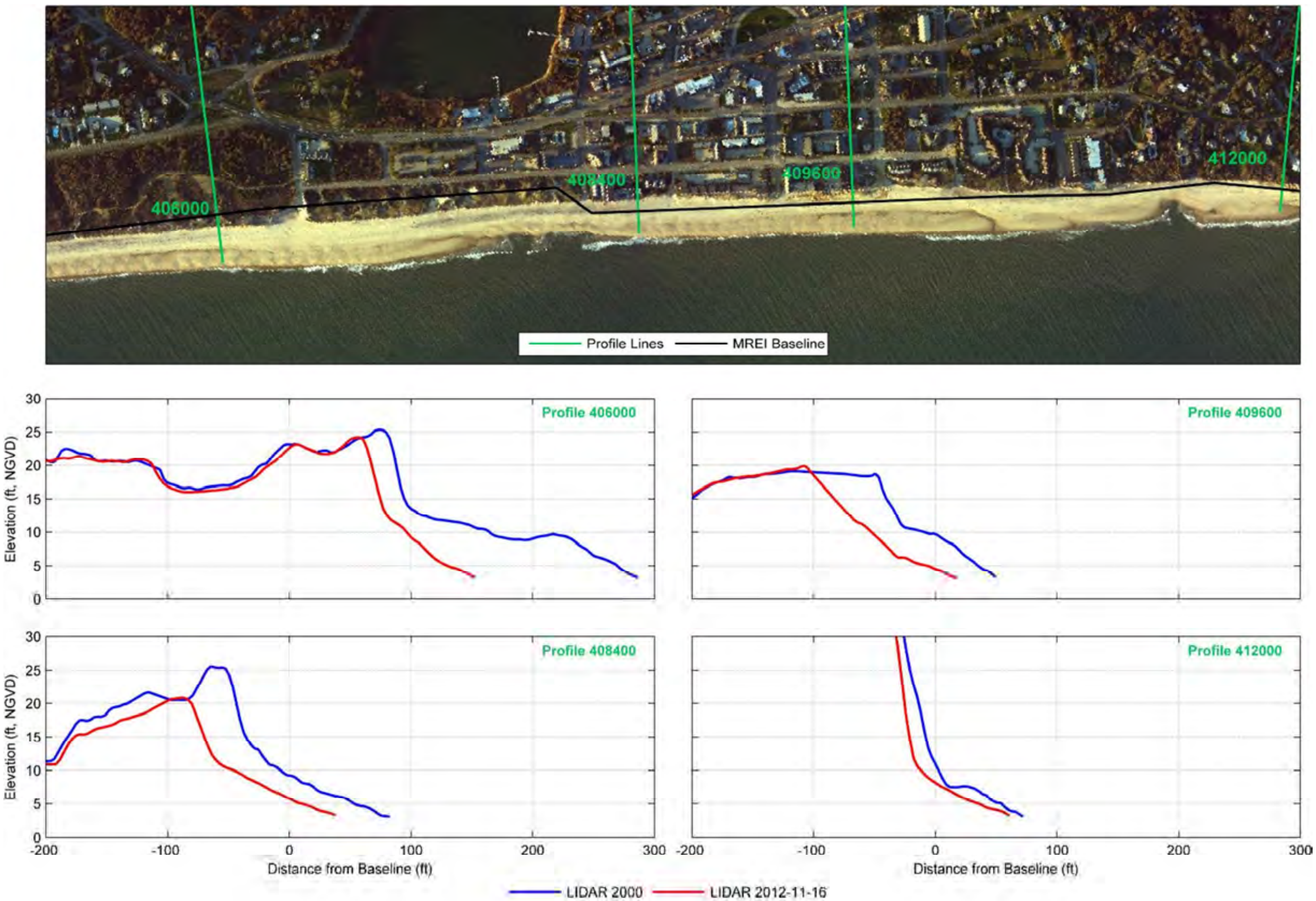


Figure 3: Observed Beach Profile Changes at downtown Montauk

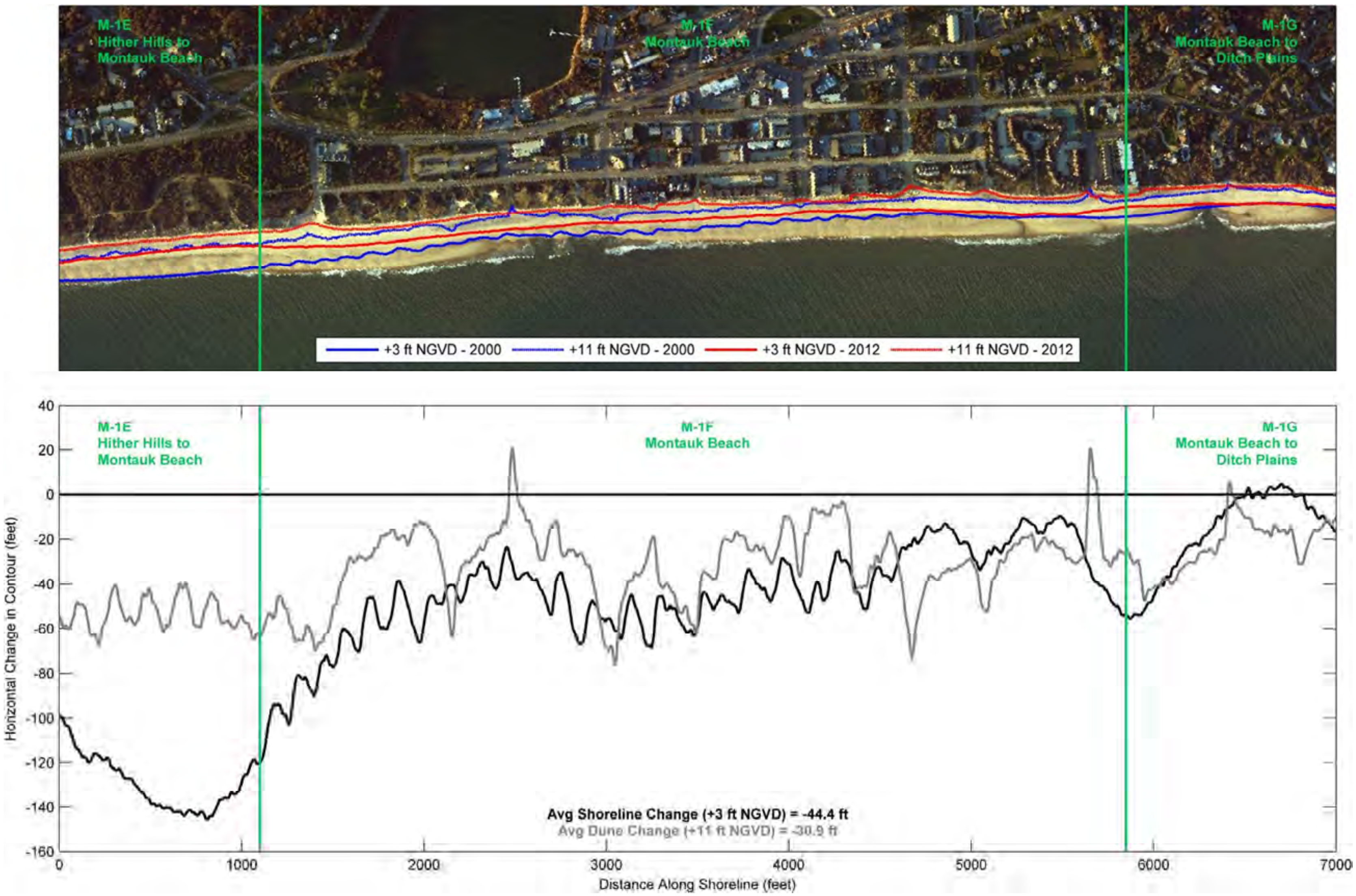


Figure 4: Observed Shoreline Changes at downtown Montauk

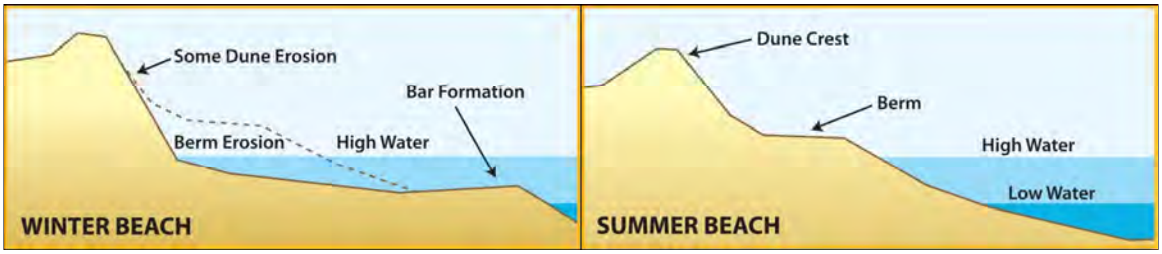


Figure 5: Schematic of Seasonal Changes in Beach Conditions (Maine Sea Grant)

Figure 6: Seasonal Changes in Beach Conditions at Montauk (Photos)

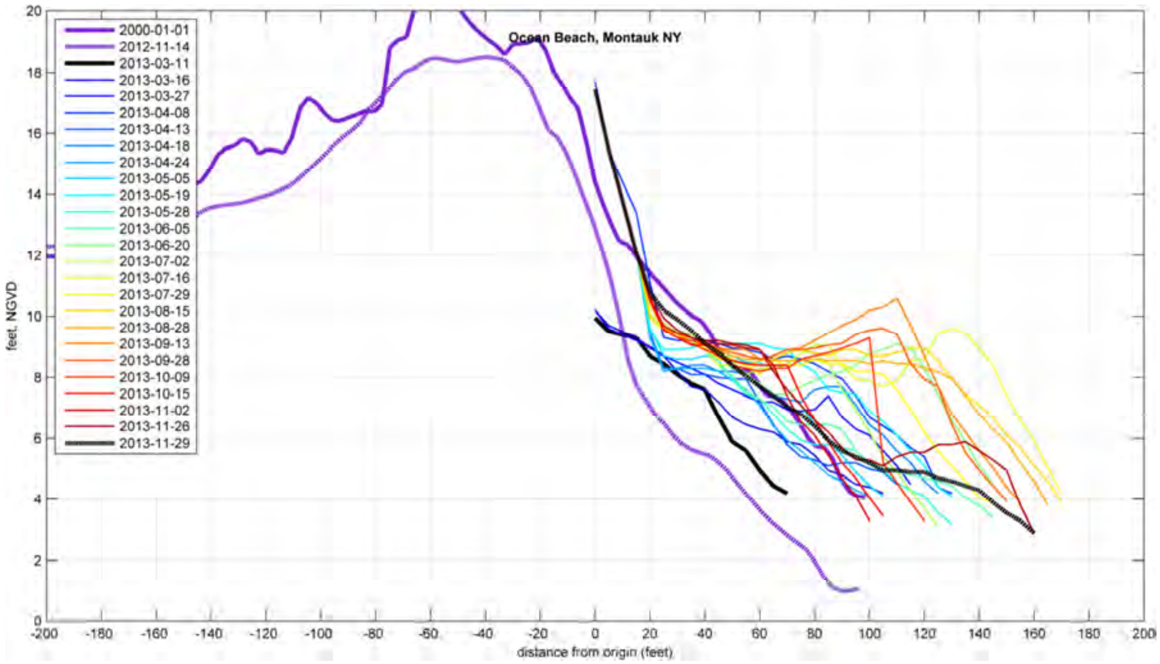


Figure 7: Post-Sandy Beach Evolution at Ocean Beach

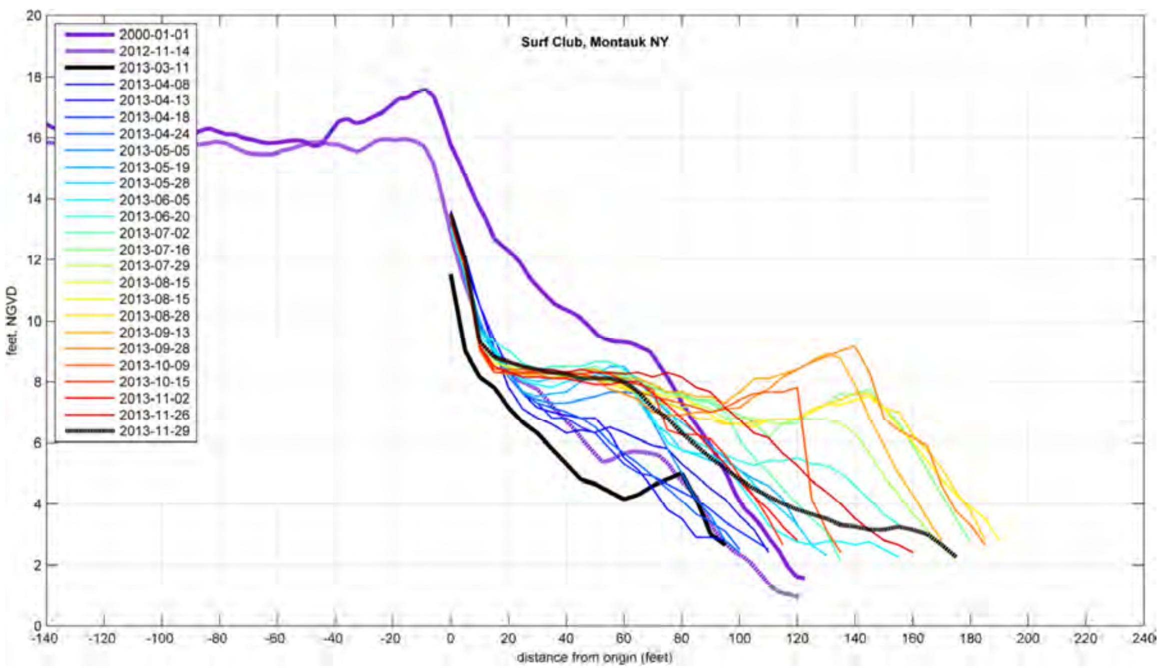


Figure 8: Post-Sandy Beach Evolution at Surf Club

3.0 ALTERNATIVE OVERVIEW

In the aftermath of Hurricane Sandy, it was recognized that there was a need to revisit the Tentatively Federally Selected Plan (TFSP) and determine if the eroded beach conditions and updated costs and benefits warranted selection of an alternative plan at downtown Montauk. A new evaluation of six conceptual alternatives was performed at downtown Montauk taking into consideration the eroded beach conditions following Sandy.

The six conceptual alternatives were narrowed down to five alternatives based on preliminary cost estimates and input from stakeholders:

- Alternative 1: Beach Restoration,
- Alternative 2: Beach Restoration and Buried Seawall,
- Alternative 3: Feeder Beach,
- Alternative 4: Dune Reinforcement,
- Alternative 5: Dune Reinforcement and Feeder Beach.

The five alternatives represent a range of measures providing different levels of protection and design project lives. Alternatives 1, 2, and 3 are designed to provide a 44 year level of protection and have a design project life of 50 years. The post-Sandy analysis also considered two lower cost alternatives that provided a lower level of protection, 25 years, and a shorter design life. A detailed description of the five alternatives is provided in the Main Report (Section 5.2.1).

4.0 WITH PROJECT EROSION RATES & RENOURISHMENT QUANTITIES

The advance fill berm width and renourishment volumes are determined based on the representative erosion rates for each design reach. The representative erosion rate accounts for:

1. "Spreading out" or diffusion of sand resulting from the shoreline anomaly or "bump" created by the beachfill;

2. Background shoreline erosion due to ongoing processes before the project was constructed.

Beachfill diffusion is a function of the longshore length of the beachfill, cross-shore width of the beachfill, and longshore diffusivity. The rate of beachfill diffusion is particularly sensitive to longshore length of the beachfill project. downtown Montauk is susceptible to relatively high rates of diffusion due to its short length. Analytical solutions to the diffusion equation (i.e. Pelnard Considere, 1956) are applied in Section 4.1.3 to determine the rate of beachfill diffusion at downtown Montauk.

Generally it is assumed that the background shoreline erosion will continue at the same rate as before the project. Background erosion rates were determined based on the sediment budget and recent measurements of shoreline change.

4.1 Beachfill Diffusion

A beach nourishment project constructed on a long beach represents a perturbation, which under wave action will spread out along the shoreline (Dean, 2005). If the wave action is small, than the rate at which the anomaly resulting from the beach nourishment is spread out from the placement area will likewise be small. It important to remember that beachfill diffusion is a separate process from background shoreline erosion, which is generally caused by gradients in the net longshore sediment transport.

4.1.1 Theoretical Background

The one-dimensional diffusion equation or Pelnard-Considere equation for planform evolution may be derived from combining the conservation of sediment equation with the total longshore sediment transport equation.

The conservation of sediment equation:

$$\frac{\partial Q}{\partial x} + (h_* + B)\frac{\partial y}{\partial t} = 0$$

Where Q is the total longshore sediment transport, y is the shoreline, and h_* and B are the depth of closure and berm height respectively.

The total longshore sediment transport, Q, equation or CERC formula is given by:

$$Q = C' H_b^{5/2} \sin 2\theta_b$$

$$C' = \frac{K\sqrt{g/\delta_b}}{8(S-1)(1-p)}$$

Where H_b is the breaking wave height, θ_b is breaking wave angle relative to shore normal, K sediment transport coefficient, g is acceleration of gravity, δ_b breaking wave index, S specific gravity of sand, and p is the porosity of sand.

For an undulating shoreline, with small values of $\partial y / \partial x$ the sediment transport equation may be re-written as follows

$$Q = C' H_b^{5/2} \sin 2\theta_b - G(h_* + B)\frac{\partial y}{\partial x}$$

The first term above represents the background sediment transport rate for shoreline parallel to the x-axis, and the second term represents the transport induced by the shoreline undulations ($\partial y / \partial x$). Parameter G is the longshore diffusivity and is equal to

$$G = \frac{2C' H_b^{5/2} \cos 2\theta_b}{\left(h_* + B\right)}$$

Taking the derivative of the sediment transport equation (assuming $\partial y / \partial x$ << 1) and combing with the conservation of sediment equation yields the final form of the Pelnard-Considere equation

$$\frac{\partial y}{\partial t} \cong G \frac{\partial^2 y}{\partial x^2}$$

There are many solutions to the equation, of interest here are the solutions for a rectangular and trapezoidal beachfill (e.g. with tapers) on a long straight beach. Consideration was given to solutions to the Pelnard-Considere equation for a barrier island with inlets; however, the distance between the inlets and limits of beachfill are sufficiently large to result in very small differences.

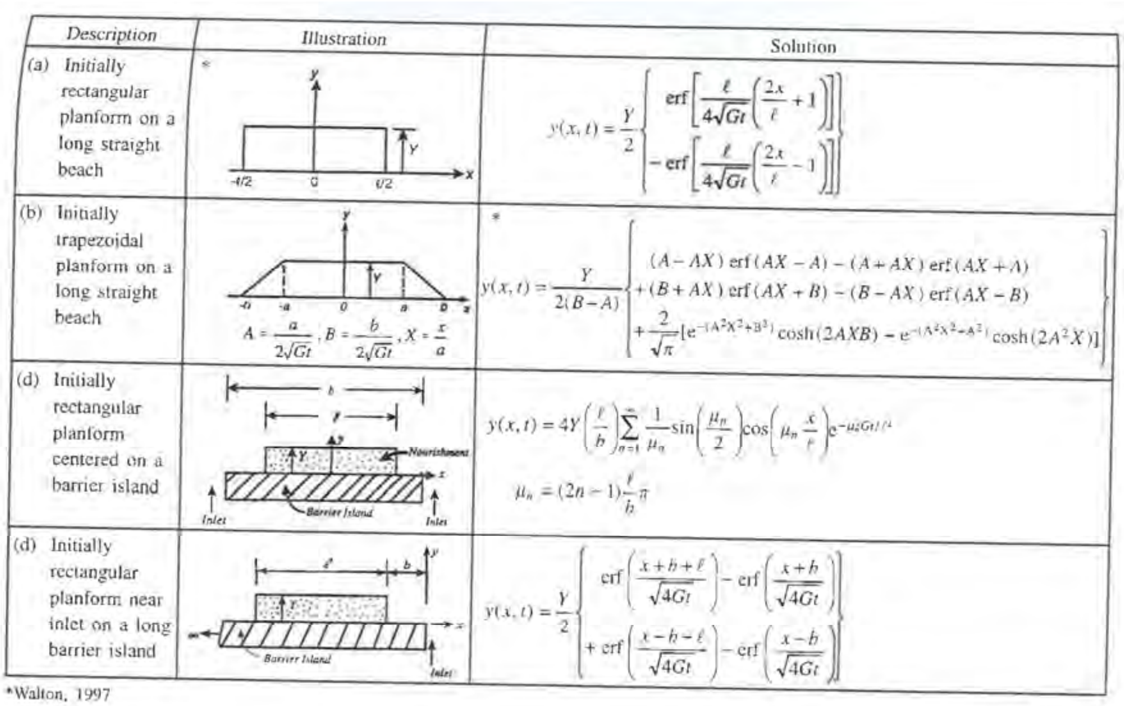
Rectangular Beachfill

The solution to the Pelnard-Considere equation for a rectangular beachfill project on a long straight beach is shown in panel "a" of Figure 9. The non-dimensional results for a rectangular beachfill project with alongshore length *l*, cross-shore width *Y*, and time *t* are shown in Figure 10 illustrating that the planform location after some time "*t*" is proportional to $1/l^2$. As a result, the performance of the beachfill is very sensitive to the alongshore length.

Figure 11 further demonstrates the sensitivity of the performance of a beachfill project to the alongshore length by plotting the fraction of volume remaining, M(t), versus non-dimensional time, $\sqrt{Gt}/l$. The solid black line shows the solution to the Pelnard-Considere equation, the dashed black line presents the results for exponential decay, and the four markers present the volume remaining after 4 years for beachfill projects at Western Fire Island (41,800 feet), Fire Island Pines (6,400 feet), Davis Park (4,200 feet), SPCP (19,400 feet), and downtown Montauk (6,600 feet). It is important to note, that the results in Figure 11 are in the absence of background erosion. The implications of Figure 11 are clear, shorter beachfill projects will experience a much higher rate of diffusion. Therefore, it is expected that the representative erosion rates at downtown Montauk will be much higher than at Western and Eastern Fire Island because the alongshore length of the beachfill project is significantly smaller.

Trapezoidal Beachfill

The solution to the Pelnard-Considere equation for a trapezoidal beachfill project on a long straight beach is shown in panel "b" of Figure 9. The results for a trapezoidal beachfill project a similar to the results for a rectangular beachfill project except that end losses are slightly lower due to the tapers. The trapezoidal beach solution is applied to Montauk Study since six (6) degree tapers are applied in this study.



Description	Illustration	Solution
(a) Initially rectangular planform on a long straight beach	*	$y(x,t)=\frac{Y}{2}\left\{\begin{array}{l}\mathrm{erf}\left[\frac{\ell}{4\sqrt{Gt}}\left(\frac{2x}{\ell}+1\right)\right]\\ -\,\mathrm{erf}\left[\frac{\ell}{4\sqrt{Gt}}\left(\frac{2x}{\ell}-1\right)\right]\end{array}\right\}$
(b) Initially trapezoidal planform on a long straight beach	$A=\frac{a}{2\sqrt{Gt}}, B=\frac{b}{2\sqrt{Gt}}, X=\frac{x}{a}$	* $y(x,t)=\frac{Y}{2(B-A)}\left\{\begin{array}{l}(A-AX)\,\mathrm{erf}(AX-A)-(A+AX)\,\mathrm{erf}(AX+A)\\ +(B+AX)\,\mathrm{erf}(AX+B)-(B-AX)\,\mathrm{erf}(AX-B)\\ +\frac{2}{\sqrt{\pi}}[e^{-(A^2X^2+B^2)}\cosh(2AXB)-e^{-(A^2X^2+A^2)}\cosh(2A^2X)]\end{array}\right\}$
(d) Initially rectangular planform centered on a barrier island		$y(x,t)=4Y\left(\frac{\ell}{b}\right)\sum_{n=1}^{\infty}\frac{1}{\mu_n}\sin\left(\frac{\mu_n}{2}\right)\cos\left(\mu_n\frac{x}{\ell}\right)e^{-\mu_n^2Gt/\ell^2}$ $\mu_n=(2n-1)\frac{\ell}{b}\pi$
(d) Initially rectangular planform near inlet on a long barrier island		$y(x,t)=\frac{Y}{2}\left\{\begin{array}{l}\mathrm{erf}\left(\frac{x+b+\ell}{\sqrt{4Gt}}\right)-\mathrm{erf}\left(\frac{x+b}{\sqrt{4Gt}}\right)\\ +\,\mathrm{erf}\left(\frac{x-b-\ell}{\sqrt{4Gt}}\right)-\mathrm{erf}\left(\frac{x-b}{\sqrt{4Gt}}\right)\end{array}\right\}$

*Walton, 1997

Figure 9: Solutions to Pelnard-Considere Equation (Dean, 2005)

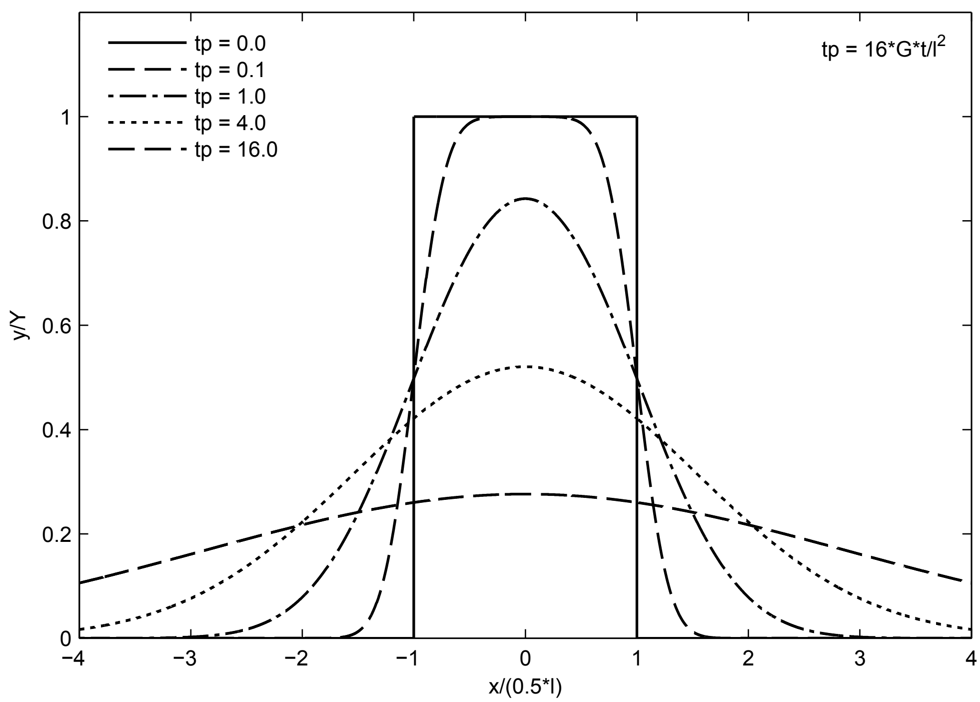


Figure 10: Non-dimensional Beachfill Evolution Based on Diffusion Equation

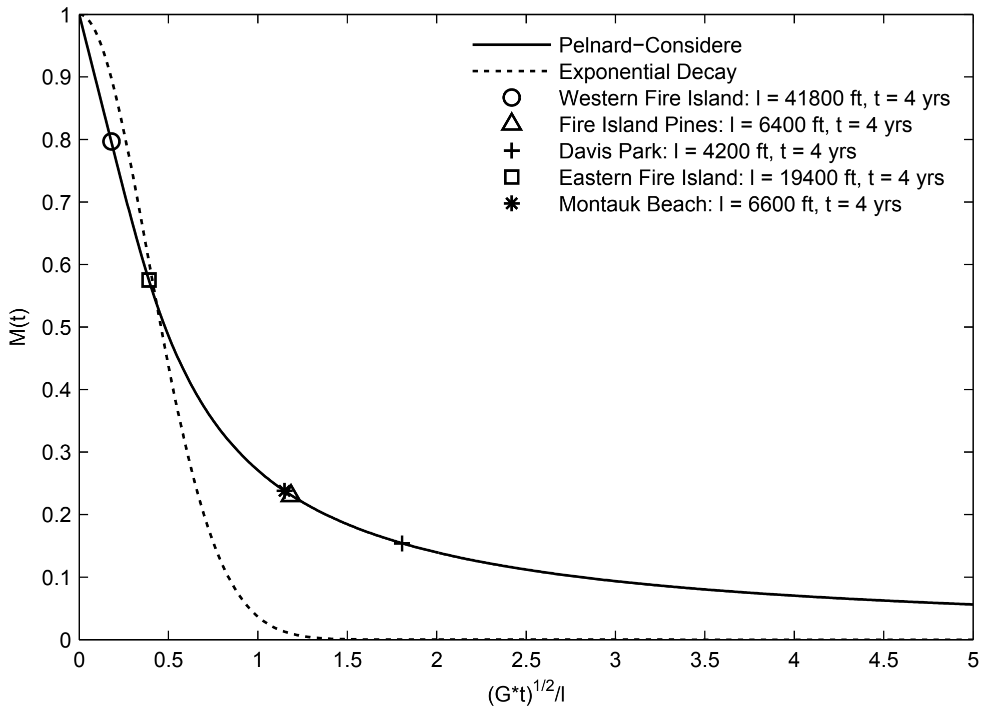


Figure 11: Theoretical Longevity of Beachfill (Excluding Background Erosion)

<u>Incorporating Background Erosion</u>

The combined effect of diffusion and background erosion, $\partial E / \partial t$, can be accounted for by adding an additional term to solutions for a rectangular or trapezoidal beachfill:

$$y(x,t) = \ldots - \frac{\partial E}{\partial t}$$

4.1.2 Alongshore Diffusivity

The alongshore diffusivity, G, controls the rate at which "spreading" or diffusion of the beachfill project occurs. The alongshore diffusivity is proportional to the breaking wave height raised to the 5/2 power. Since the wave conditions at a site vary over time, so too does the alongshore diffusivity. Therefore, the alongshore diffusivity can be determined by integrating G over time or by determining an effective wave breaking height.

If the gross sediment transport rate at a site is known, than it is possible to back-calculate the effective breaking wave height, H_b, from the CERC sediment transport formula and use H_b to determine the alongshore diffusivity, G. It is important to use the gross sediment transport rates because it reflects the true diffusivity of project site. For example, if a study area had a very high gross sediment transport potential but virtually zero net sediment transport, one would still expect the alongshore diffusivity to be high.

Based on a gross sediment transport rate 2.25 million m^3/yr (2.94 MCY), at Montauk Point (Gravens et al, 1999), an effective breaking wave height of 3.65 feet (1.10 m), and alongshore diffusivity of 0.15 ft^2/s. The alongshore diffusivity was reduced by 60% to account for stabilizing effect of wave refraction around the beachfill project (Dean, 2005). Backup calculations for the alongshore diffusivity are provided in Appendix E.

4.1.3 Application to downtown Montauk

As previously discussed downtown Montauk is particularly vulnerable to losses from beachfill diffusion since the project length is relatively short (6,600 feet) and because the proposed design shorelines stick out from the existing shoreline. A simple analytical approach is applied here to determine the beachfill diffusion losses for the alternatives. The Beachfill and Beachfill & Buried Seawall were evaluated. The Dune Reinforcement Feeder Beach alternatives were not evaluated since these alternatives either don't have renourishment (Dune Reinforcement) or provide a fixed volume of sand for renourishment (Feeder Beach).

In order to apply the beachfill diffusion analysis the cross-shore width, *Y*, of the beachfill project must be known. In this application, the cross-shore width represents the distance that the design berm (plus advance nourishment) protrudes from the adjacent shoreline where no beachfill placement is planned. It is not a straightforward task to determine this cross-shore width. The cross-shore width, Y, can be further broken down into three components:

$$Y = Y_o + Y_a$$

Where Y_o is the initial cross-shore distance that the design shoreline protrudes from the adjacent shoreline and Y_a is the advance nourishment width. The alongshore diffusivity, and alongshore length of beachfill are the same for all three alternatives. A summary of the initial cross-shore widths for the three alternatives is provided in Table 1. The cross-shore widths were determined by comparing design MHW line to the existing MHW line. The required advance fill width was determined iteratively by calculating the solution to the diffusion analysis for different advance fill widths. As the advance fill width increases so does the beachfill losses.

The results of the diffusion analysis for the three alternatives are presented in Table 1. The theoretical evolution of the three alternatives at downtown Montauk is shown in Figure 12 and Figure 13.

The results from the beachfill diffusion analysis have been rounded off and adjusted based on engineering judgment to determine the final representative erosion rates to be used in the renourishment volume estimates (Table 2).

Table 1: Diffusion Results

Location	Length (ft)	Y_o (ft)	Y_a (ft)	Background Erosion (ft/yr)	Diffusive Erosion (ft/yr)	Representative Erosion (ft/yr)
Beachfill	6,600	60	91.2	3	19.8	22.8
Beachfill & Buried Seawall	6,600	5	28.9	3	4.2	7.2

Table 2: Representative Erosion Rates for Downtown Montauk Alternatives

Location	Representative Erosion (ft/yr)
MREI Beachfill	20
Beachfill & Buried Seawall	7

4.2 Renourishment Volumes

Future renourishment volumes over the project life (50 years) are calculated based on the representative erosion rates determined in Section 4.1.3. Similarly to the advance berm width, the renourishment volumes is equal to the representative erosion rate multiplied by the renourishment interval (e.g. 5 feet/year x 4 years = 20 feet). The relatively large representative erosion rate predicted for the Beachfill Alternative may warrant consideration of shorter renourishment interval. The renourishment extents are the same as the initial construction extents. Renourishment volumes for a single renourishment operation are presented in Table 3.

Table 3: Renourishment Beachfill Volumes

Item	Beach Restoration	Beach Restoration & Seawall	Feeder Beach	Dune Reinforcement	Feeder Beach & Dune Reinforcement
Length (ft)	6,600	6,000	3,100	3,100	3,100
Erosion Rate (ft/yr)	18	6			
Advance Fill (c.y.)	641,520	194,400	120,000	n/a	120,000
10% Overfill (c.y.)	64,152	19,440			
Subtotal (c.y.)	***705,672***	***213,840***			
15% Tolerance (c.y.)	105,851	32,076			
Total Fill (c.y.)	**812,000**	**246,000**	**120,000**	**n/a**	**120,000**

Note: Fill quantities are provided for each 4-year renourishment cycle.

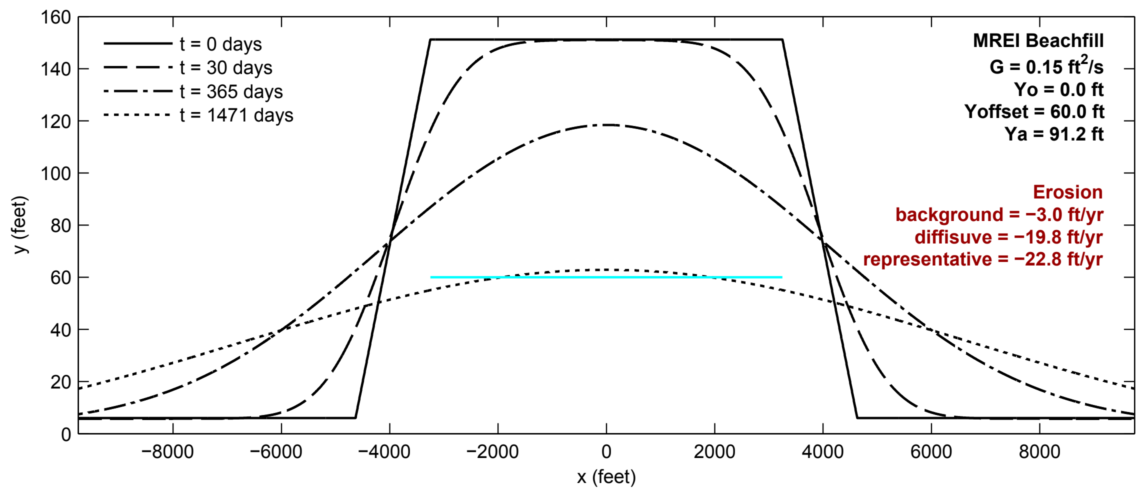


Figure 12: Beachfill Evolution at Downtown Montauk – Beachfill

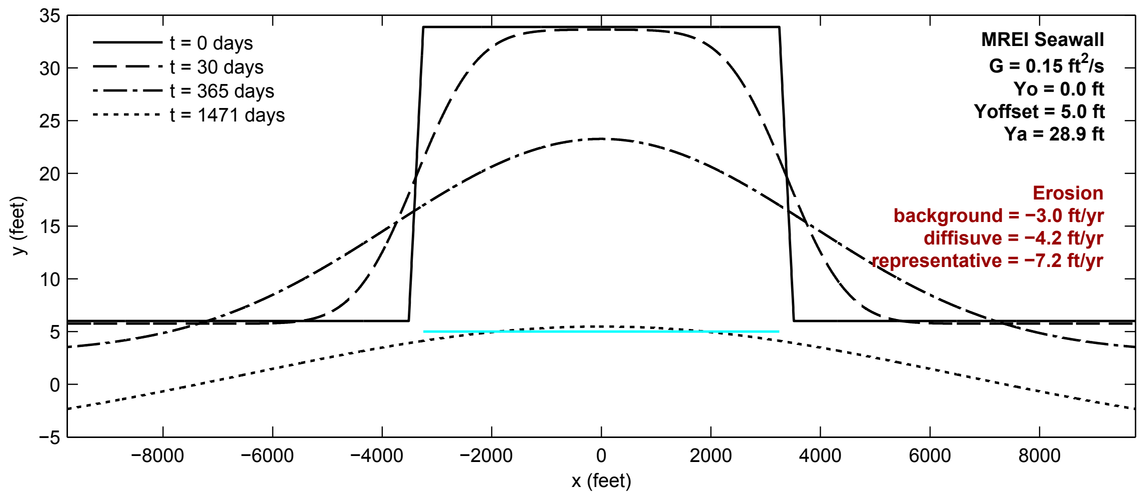


Figure 13: Beachfill Evolution at Downtown Montauk – Beachfill and Buried Seawall

5.0 INITIAL CONSTRUCTION QUANTITIES

5.1 Beachfill Quantities (Alternatives 1, 2, and 3)

5.1.1 Methodology

Initial construction beachfill quantities for Alternatives 1 and 2 were estimated from profile surveys conducted by OSI in September, 2013. Average end area calculations were performed based on the design section (Figure 14) and profile surveys. The fill volume at each survey location was calculated using an USACE product called RMAP (Regional Morphology Analysis Package). The Feeder Beach alternatives are not defined by a specific design profile and the alternative is not intended to provide and maintain a specific berm/dune width. Instead the alternative provides a source of sediment to the system that is intended to help alleviate background erosion. Therefore, a fixed quantity of sand, 120,000 cy, would be placed once every 4 years.

Advance fill is included in the initial beachfill quantities for Alternatives 1 and 2. Advance fill is a sacrificial quantity of sand that acts as an erosional buffer against long-term and storm-induced erosion as well as beachfill losses caused by "spreading out" or diffusion. The required advance berm width was computed based on representative erosion rates and expected renourishment interval (4 years). Since the Feeder Beach alternatives are not designed to maintain a specific berm width over time no advanced fill is added to these design quantities.

Below +3 ft NGVD, both the beach profile and design profile are set to the representative morphological profile. As a result, the berm fill volumes below +3 feet are equal to the offset in the +3 feet contour multiplied by 30 feet (depth of closure +3 feet). In general, the berm fill volumes are dominated by the subaqueous fill, which is directly related to the difference between the +3 feet contour in the design profile and survey data. Therefore, the beach fill volumes are very sensitive to the location of +3 feet NGVD.

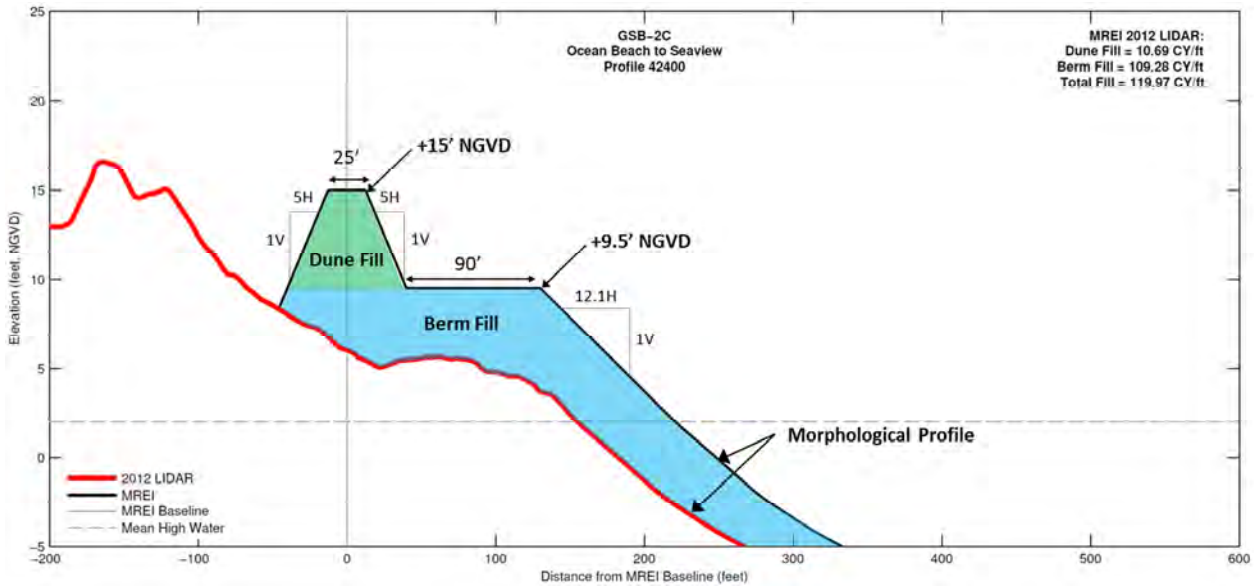


Figure 14: Beach Fill Design Section

5.1.2 Results

Table 4 presents the lengths in which dune and berm fill was considered for the five alternatives, the design volumes, advance fill volumes, and total initial fill volumes. The total initial fill volumes include a 15% contingency and 10% overfill.

Table 4: First Construction Beachfill Volumes (Alternatives 1, 2, and 3)

Item	Beach Restoration	Beach Restoration & Seawall	Feeder Beach
Length (ft)	6,600	6,000	3,100
Design fill (c.y.)	689,338	298,772	120,000
Advance Fill (c.y.)	591,514	140,873	
10% Overfill (c.y.)	128,085	43,865	
Subtotal (c.y.)	***1,408,937***	***482,510***	***120,000***
15% Tolerance (c.y.)	211,341	72,376	
Total Fill (c.y.)	**1,620,000**	**555,000**	**120,000**

5.2 Buried Seawall Stone Quantities

5.2.1 Methodology

In the FIMP Basis of Design Report (USACE-NAN, 2000) various combinations of beachfill berm width, seawall crest height, side slope, and toe elevation were evaluated for Montauk. The Storm-induced Beach Change (SBEACH) model was used to determine the required structure toe elevations (i.e. scour depth) for a range of berm widths. The procedure for determining the design seawall configuration is as follows:

1) Input Structural Constants, Design Water Level, Wave Height, and Wave Period

2) Estimate depth-limited wave conditions at site
   a) breaking wave condition
   b) maximum breaker height
   c) wave length at site

3) Compute maximum wave and significant wave heights based on random wave transformation by GODA

4) Perform runup calculations to determine seawall crest elevation
   a) based on Pilarcyzk (1990)
   b) based on van der Meer (1992)
   c) estimate mean runup assuming Rayleigh distribution
   d) estimate significant runup based on Pilarcyzk (1990)

5) Estimate overtopping
   a) based on van der Meer
   b) based on Pilarcyzk
   c) determine structure crest elevation based on tolerable overtopping limit

6) Seawall design
   a) determine armor size with Hudson formula and checked with van der Meer formulae
   b) determine armor and underlayer sizes and thicknesses
   c) determine scour toe berm width

7) Quantity Estimates

   a) determine quantity for 1 foot cross-section, include 1 foot of tolerance, and multiply by total structure length
   b) excavation volume is equal to the total stone volume plus 20%

5.2.2 Results

The optimum seawall configuration was identified for the 44 year level of protection is adopted for downtown Montauk. A typical section of the rubble mound seawall is provided in Figure 15. The proposed rubble-mound seawall has a crest elevation of +11 ft NGVD, toe elevation of +4.3 ft NGVD, a crest width of 7.7 ft, slope of 1V:1.5H, scour toe berm width of 13.1 ft, and armor stone size of 1.4 ton (USACE-NAN, 2000). Backup calculations are provided in Appendix C.

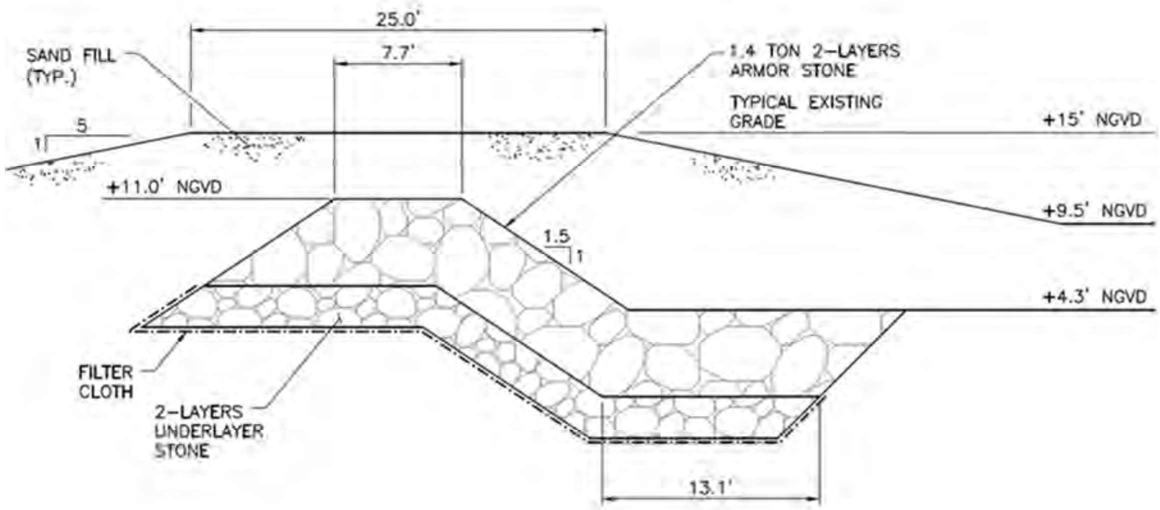


Figure 15: Typical Buried Seawall Section

Table 5 presents the total quantity of armor stone, underlayer stone, geotextile filter fabric, and excavation for the buried seawall alternative (3,150 feet).

Table 5: Buried Seawall Stone Quantities

Item	Quantity
Armor Stone (ton)	33,145
Underlayer / Core Stone (ton)	16,487
Geotextile (sq.yd.)	17,520
Excavation (c.y.)	41,193

5.3 Dune Reinforcement

5.3.1 Methodology

The beachfill quantities for the Dune Reinforcement (Alternative 4) were estimated from a profile survey conducted by First Coastal on November 24, 2013 at Ocean Beach. The quantities of excavation and sand fill were determined from a cut/fill calculation in CADD based on the typical section (Figure 16). The quantity of sand is required to fill the GSC, cover and build the dune, and build the berm cap were identified. The estimated quantities at the Ocean Beach profile were applied to the entire 3,100 feet length of the project to determine the total sand fill quantities.

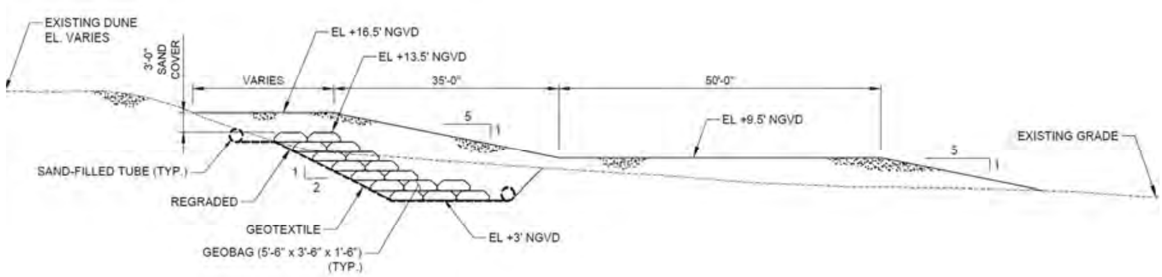


Figure 16: Reinforced Dune Typical Section

5.3.2 Results

The estimated quantities of excavation, sand fill, and GSC are shown in Table 6. Note that the excavated sand will be reused to construct the dune. A total of 51,000 cy of sand fill is required to construct the reinforced dune. Sixteen GSCs are required for each section with a total of 14,171 GSCs over the entire length of the project. The Feeder Beach and Dune Reinforcement Alternative includes the sand fill required to construct the dune as well as the 120,000 cy of sand fill to construct the feeder beach. A summary of the first construction beachfill volumes for Alternatives 4 and 5 is provided in Table 7.

Table 6: Dune Reinforcement Quantities

Item	Number	Unit
Excavation	20,283	cu.yd.
Sand Fill (Geobags)	15,146	cu.yd.
Sand Fill (Dune)	30,437[1]	cu.yd.
Sand Fill (Berm)	25,700	cu.yd.
Furnish Geobags	14,171	each
Fill & Place Geobags	14,171	each
Geotextile Filter Layer	24,357	sq.yd.

Notes: [1]20,283 cy of the required sand fill will be obtained from excavation.

Table 7: First Construction Beachfill Volumes (Alternatives 4 and 5)

Item	Dune Reinforcement	Feeder Beach & Dune Reinforcement
Length (ft)	3,100	3,100
Design fill (c.y.)	51,000	147,000
Advance Fill (c.y.)		
10% Overfill (c.y.)		
Subtotal (c.y.)	***51,000***	***147,000***
15% Tolerance (c.y.)		
Total Fill (c.y.)	**51,000**	**147,000**

6.0 GEOTEXTILE SAND CONTAINERS

Geotextile Sand Containers (GSCs) have been used in hydraulic and coastal applications in many parts of the world for the past 50 years. Over the past 20 years, advancements in container technology, as well as engineering design criteria, have established GSCs as a cost-effective, reversible, and versatile "soft" solution to a wide variety of projects. Successful projects using GSCs include those used in erosion control, bottom scour protection and scour fill, artificial reefs, groins, dams, seawalls, revetments and dune reinforcement (Saathoff, et al., 2007). Many coastal structures normally constructed using stone, concrete, or wood may alternatively be constructed with properly designed and maintained GSCs. GSC structures offer some advantages over traditional hard structures: GSCs may be constructed with in-situ sand/gravel, environmental friendly, user friendly, and easily reversible. However, there are also disadvantages to using GSCs in coastal applications that are primarily associated with the decreased stability, durability, and longevity of GSCs when compared to armor stone.

Early geotextile containers consisted predominately of relatively long "geotubes" manufactured predominately from woven geotextiles (Hornsey, et al., 2011). Although geotubes have proven to be cost effective as short term solutions, experience has shown that they do not often provide long-term engineering solutions as localized damage (e.g. differential settlement) or vandalism can cause large sections of the structure to fail (Saathoff, et al., 2007). Over the past 20 years, the use of geotubes has decreased in favor of structures consisting of smaller individually stacked GSCs constructed from woven and non-woven geotextiles (Hornsey, et al., 2011).

GSCs are a relatively new technology and consequently the geotextile materials and design guidance are still evolving. Nonetheless, there are already numerous case studies in the United States and rest of the world that highlight positive experiences and performance of structures constructed with GSCs. This memorandum outlines some of the important engineering (design and construction) and maintenance considerations and reviews several case studies.

Figure 17: Example Applications of Geotextile Sand Containers

6.1 Engineering Considerations

The engineering considerations of GSC coastal protection structures can be divided into three categories: wave stability, durability, and constructability. Other design considerations such as scour toe protection and required crest elevation are not unique to GSC structures and are not discussed herein.

6.1.1 Wave Stability

Similarly to stone structures the individual GSCs must be designed to be stable under the design wave conditions. GSCs have a lower specific gravity and are more susceptible to sliding and being pulled out than traditional stone. Studies have shown that the dislodgment and pullout of the slope containers by wave action, including the sliding and the overturning of crest containers, are strongly affected by the deformation of the sand containers (Dassanayake, et al., 2012). Established design formulae do not exist for GSC-structures; however, recent advances in understanding the hydraulic stability of the GSC under wave attack (Wouters, 1998; Pilarczyk, 2000; Oumeraci et al, 2003; and Dassanayake and Oumeraci, 2012) have led to several design formulae for GSC structures. Most of the design formulae relate the stability of the GSC to the surf similarity parameter and wave height. An increase in the wave height and wave period results in decreased stability of the GSCs and increases the required size / weight of the GSCs. Studies have shown that stability of GSCs is also affected by the amount of overlap between the GSCs, friction of geotextile material, sand fill ratio, and properties of fill material (Dassanayake, et al., 2012) (Saathoff, et al., 2007).

Generally, GSCs sizes range from 1 to 4 cubic yards. The selection of a bag size should assess: how large it needs to be to stable under design wave conditions; how small it should be such that one or multiple broken containers will not result in structure failure; and which bag size is appropriate for the preferred placement method given available equipment.

The aforementioned design guidance led to selection of GSCs with filled dimensions of approximately 5.5 ft long, 3.5 ft wide, and 1.5 ft tall and a weight of 1.7 ton. In order to increase the stability of the GSCs the long side of GSCs is laid out perpendicular to the shoreline with an overlap of 50% of the filled width. The selected GSCs are hydraulically stable under 25-year design conditions, and unstable under 50-year design considerations. The GSCs are expected to provide a 25-year level of protection. Hydraulic stability calculations for the GSCs under design conditions are available in Attachment B.

6.1.2 Durability

The longevity of GSC structure is often limited by the durability of the individual GSCs. The following characteristics of the GSCs affect its durability: UV resistance, seam strength, abrasion resistance, puncture resistance, fines retention, permeability, and elongation (Saathoff, et al., 2007).

Degradation due to UV radiation is a significant factor in long-term serviceability of the GSC (Saathoff, et al., 2007). The containers may also be exposed to abrasion due to water born sands, gravel, and shells carried by the currents and waves. Over time the abrasion may weaken the geotextile and lead to tearing. Puncture from vandalism or driftwood is often unavoidable.

The longevity and required durability of the GSCs may be reduced by limiting their exposure to UV, abrasion, and debris/vandalism. This may be accomplished by maintaining a protective cover of sand. Alternatively, stronger and thicker and more costly geotextile materials may be used for greater durability and increased longevity. Recent advances in geotextile materials have led to materials that have greater UV resistance and case studies that have withstood extreme UV exposure and abrasion for over 10 years (Saathoff, et al., 2007). Recent improvements have led to geotextile materials that are more resistant to puncture (Hornsey, et al., 2011).

GSCs are primarily made of two different types of fabrics: a woven polypropylene fabric, and a non-woven polyester fabric. The woven material typically has higher tensile strength than the non-woven material. However, non-woven may have better filtration, higher abrasive resistance. Within both classes of geotextile materials there are options available to select thicker, stronger, and more durable materials for increased longevity. The aforementioned design guidance led to selection of 1.7 ton GSCs with filled dimensions of approximately 5.5 ft long, 3.5 ft wide, and 1.5 ft tall. In order to increase the stability of the GSCs the long side of GSCs is laid out perpendicular to the shoreline with an overlap of 50% of the filled width. The selected GSC are hydraulically stable under 25-year design conditions, and unstable under 50-year design considerations. The GSC are expected to provide a 25-year level of protection. Hydraulic stability calculations for the GSC under design conditions are available in Attachment B.

Other design considerations that are not explicitly accounted for in the currently available design formulas are the sand fill ratio, friction between the GSCs, and incline angle of the GSCs (Dassanayake and Oumeraci, 2012). Most existing studies recommend a sand fill ratio of 80% for GSC which is believed to balance the advantages (higher stability) and disadvantages (elongation) of the sand fill ratio.

6.1.3 Constructability

Construction stages of GSC structures include preparation of the site, filling of the containers, and placement of the containers. Conventional heavy equipment may be used to prepare a smooth slope surface clear of all debris. A layer of geotextile fabric placed on the slope then must be either sewn at the ends, or provided with sufficient overlap. The GSCs may be mechanically or hydraulically filled with the available sediment (often locally available onshore or offshore). Care must be taken during construction to prevent damage and additional stresses (e.g. elongation) of the GSCs during placement. One advantage of hydraulically filling the GSCs is that the containers may be easily filled in place reducing the labor required to place the GSCs.

7.0 OPERATIONS & MAINTENANCE

Relatively high maintenance costs are associated with the Dune Reinforcement alternative at downtown Montauk for two reasons:

1. The GSCs should remain covered by a layer of sand to protect against UV degradation, vandalism, and debris.

2. Unlike typical beachfill projects, the dune is not protected by a wide design berm. As a result the dune is vulnerable to erosion during storm events.

Maintenance of the Dune Reinforcement alternative entails: a) trucking in sand in response to storm events which result in dune volume losses; and b) effort required to patch & fill or replace GSC damaged during a storm events. The required maintenance quantities were estimated based on Multivariate EST results, recession of the 3.0 m contour, for an eroded beach profile at downtown Montauk (Figure 18). The purpose of the reinforced dune core (GSC) is to prevent erosion landward the reinforced core during storm events. Therefore, the dune recession EST results were adjusted to capture the reduction dune recession and dune volume loss caused by the presence of the reinforced core (GSC).

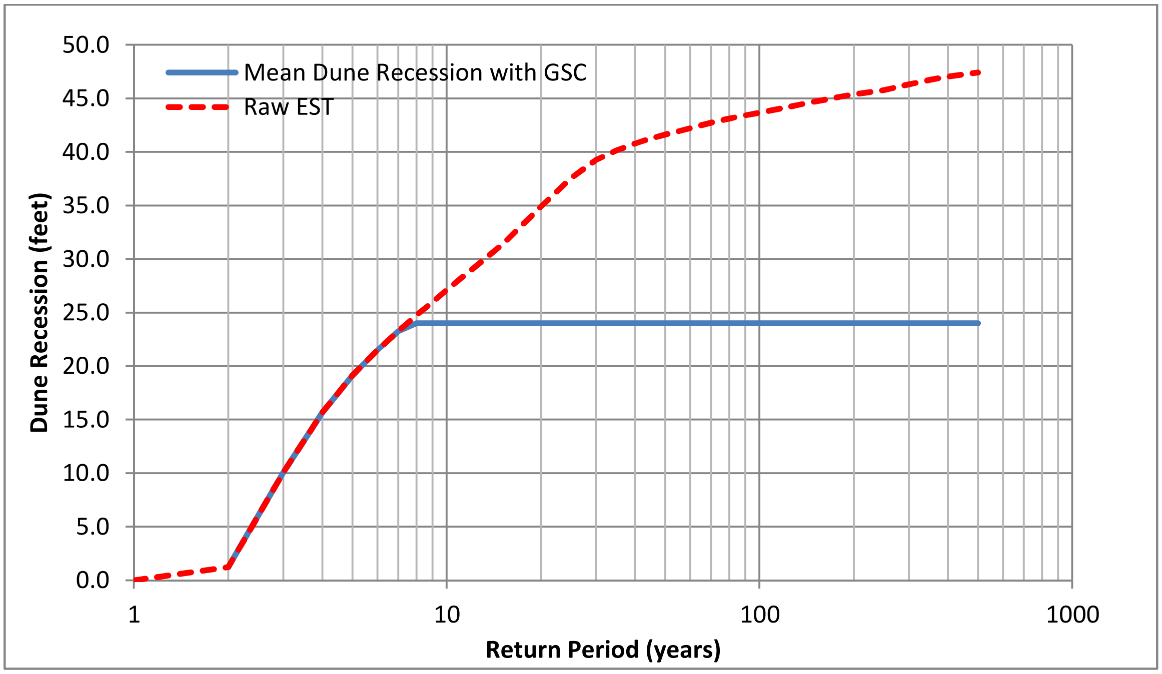


Figure 18: Storm Induced Dune Recession – EST Results

An estimate of the number of bags that would be damaged during storm events is estimated based on the likelihood that the GSC would be uncovered (roughly 5 year event) as well as the likelihood that the GSC would be subjected to large waves that have the potential to dislodge the GSC or carry debris up the GSC slope and puncture the containers.

One of the important variables applied in the estimate is the permanent loss factor. The permanent loss factor defines the percentage of sediment that is eroded from the beach and lost from the system. Typically a permanent loss factor between 10% and 30% is used in beachfill projects when estimating emergency rehabilitation volumes. However, a value of 50% was applied in this alternative because the eroded material is coming from the dune and not primarily from the berm. A value less than 100% was selected because the eroded dune material will not be completely lost from the system. A large percentage of the eroded dune material will likely be transported seaward and stored in a sand bar. During non-storm conditions the sediment in the sand bar will be gradually transported back to the berm. This process often takes days, weeks, or even a few months (e.g. summer/winter beach profiles). Longer time scales (e.g. months/years) are typically required for the dune to be naturally restored by aeolian transport. For this project it is assumed that a portion of the dune maintenance fill (50%) will be recovered from the system through naturally processes or beach scraping.

Table 8: Maintenance Costs (Dune Reinforcement)

Geobag Maintenance Costs				
Project Length	3,100	ft		
Project Life	15	Years		
Discount Rate	3.50%			
PVF (Maintenance)	11.517			
Annualized Maintenance Quantities & Costs				
Item	**Quantity**		**Parametric Estimate**	
	Number	**Unit**	**Unit Cost**	**Total Cost**
Rehab Dune Fill	2,754	cu.yd.	$35	$96,396
Patch & Fill Bags	78	each	$40	$3,118
Furnish Geobags	39	each	$70	$2,728
Mechanical Fill & Place Geobags	39	each	$300	$11,691
Patch Geotextile Roll (500 sq.yd.)	0.5	each	$1,350	$675
Subtotal				**$114,608**
Contingency	20%			$22,922
Total Construction				**$137,529**
E&D	7%			$9,627.06
S&A	7%			$9,627.06
Total Estimated Annualized Maintenance Cost				**$156,784**

8.0 REFERENCES

Dassanayake D., Oumeraci, H., 2012. “Hydraulic Stability of Coastal Structures Made of Geotextile Sand Containers (GSCS): Effect of Engineering Properties of GSCS”, Coastal Engineering.

Dean, R. G., 2005. “Beach Nourishment Theory and Practice,” World Scientific Publishing Co., Hackensak, NJ.

Gravens, M. B., Rosati, J. D., and Wise, R. A., 1999. “Fire Island Inlet to Montauk Point reformulation study (FIMP): Historical and existing condition coastal processes assessment,” prepared for the U.S. Army Engineer District, New York.

Oumeraci, H., Hinz, M., Bleck, M., Kortenhaus, A, 2003. “Sand Filled geotextile containers for shore protection”. COPEDEC VI, Colombo, Sri Lanka.

Pilarzyk K. W., 2000. “Geosynthetics and geosystems in hydraulic and coastal engineering.” A.A. Balkema, Rotterdam, the Netherlands

U.S. Army Corps of Engineers, New York District (USACE-NAN), DRAFT 2000. “Fire Island Inlet to Montauk Point, Long Island, New York: Basis of Design Report”, U.S. Army Corps of Engineers, New York District.

U.S. Army Corps of Engineers, New York District (USACE-NAN), 2007. “Fire Island Inlet to Montauk Point, Long Island, New York: Inlet Modifications Report”, U.S. Army Corps of Engineers, New York District.

Wouters, J. 1998. Open Taludbekledinger; Stabiliteit van Geosystems, Delft Hydraulics Report H1930, Delft, The Netherlands.

ATTACHMENT A

ALTERNATIVE PLAN LAYOUTS

PLAN DIMENSIONS:
BERM WIDTH = 90'
BERM ELEV = 9.5' NGVD
DUNE CREST WIDTH = 25'
DUNE CREST ELEV = 15' NGVD

CONCEPTUAL
SUBJECT TO CHANGE

DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, N.Y. 10278-0090

REFORMULATION STUDY
FIRE ISLAND INLET TO MONTAUK POINT, NEW YORK

DRAWING TITLE:
DOWNTOWN MONTAUK
BEACH FILL PLAN

SCALE:
250' 0' 250' 500'
SCALE: 1"=250' (34"x22" Sheet)
SCALE: 1"=500' (11"x17" Sheet)

DATE:	FILE NAME:	PLATE NO.
APRIL/08/2014	PLATE 1 – BEACH FILL 15-90.dwg	1

PLAN DIMENSIONS:
BERM WIDTH = 35'
BERM ELEV = 9.5' NGVD
DUNE CREST WIDTH = 25'
DUNE CREST ELEV = 15' NGVD
SEAWALL CREST ELEV=11' NGVD

CONCEPTUAL
SUBJECT TO CHANGE

DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, N.Y. 10278-0090

REFORMULATION STUDY
FIRE ISLAND INLET TO MONTAUK POINT, NEW YORK

DRAWING TITLE:
DOWNTOWN MONTAUK
BURIED SEAWALL PLAN

SCALE: 250' 0' 250' 500'
SCALE: 1"=250' (34"x22" Sheet)
SCALE: 1"=500' (11"x17" Sheet)

DATE: APRIL/08/2014
FILE NAME: PLATE 2 – BURIED SEAWALL 15-90.dwg
PLATE NO. 2

DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, N.Y. 10278-0090

REFORMULATION STUDY
FIRE ISLAND INLET TO MONTAUK POINT, NEW YORK

DRAWING TITLE:
DOWNTOWN MONTAUK
FEEDER BEACH

SCALE:
250' 0' 250' 500'
SCALE: 1"=250' (34"x22" Sheet)
SCALE: 1"=500' (11"x17" Sheet)

DATE:	FILE NAME:	PLATE NO.
APRIL/08/2014	PLATE 3 - FEEDER BEACH.dwg	3


FORT POND
MONTAUK
LIDAR NOV/2012
(FEET, NGVD)
REINFORCED DUNE
MREI
BASELINE
FINAL LOCATION OF REINFORCED
DUNE TO BE DETERMINED BASED
ON PRECONSTRUCTION SURVEY
CONCEPTUAL
SUBJECT TO CHANGE
LONG ISLAND
ATLANTIC OCEAN
KEY MAP
DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, N.Y. 10278-0090
REFORMULATION STUDY
FIRE ISLAND INLET TO MONTAUK POINT, NEW YORK
DRAWING TITLE:
DOWNTOWN MONTAUK
DUNE REINFORCEMENT
SCALE:
SCALE: 1"=250' (34"x22" Sheet)
SCALE: 1"=500' (11"x17" Sheet)
DATE:
JUNE/06/2014
FILE NAME:
PLATE 4 - DUNE REINFORCEMENT.dwg
PLATE NO.
4


FORT POND
MONTAUK
LIDAR NOV/2012
(FEET, NGVD)
REINFORCED DUNE
CONSTRUCTION MHW (+2 NGVD)
CONSTRUCTION SEAWARD BERM CREST
MREI
BASELINE
FINAL LOCATION OF REINFORCED
DUNE TO BE DETERMINED BASED
ON PRECONSTRUCTION SURVEY
CONCEPTUAL
SUBJECT TO CHANGE
LONG ISLAND
ATLANTIC OCEAN
KEY MAP
DEPARTMENT OF THE ARMY
NEW YORK DISTRICT CORPS OF ENGINEERS
NEW YORK, N.Y. 10278-0090
REFORMULATION STUDY
FIRE ISLAND INLET TO MONTAUK POINT, NEW YORK
DRAWING TITLE:
DOWNTOWN MONTAUK
FEEDER BEACH AND DUNE REINFORCEMENT
SCALE:
250' 0' 250' 500'
SCALE: 1"=250' (34"x22" Sheet)
SCALE: 1"=500' (11"x17" Sheet)
DATE:
JUNE/06/2014
FILE NAME:
PLATE 5 - FEEDER BEACH AND
DUNE REINFORCEMENT.dwg
PLATE NO.
5

ATTACHMENT B

GEOTEXTILE SAND CONTAINER DESIGN CALCULATIONS

Date: March 21, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: Montauk Beach - FIMP
Analysis: Geobag Stability - 25 year Return Period

Calculate stability of geobag revetment based on modified Hudso's formula

References: Krystian W. Pilarczyk, 1998. "Dikes and Revetments," A.A. Balkema, Rotterdam, Netherlands.
H. Oumeraci, M. Hinz, M. Bleck, and A. Kortenhaus, 2003. "Sand-filled Geotextile Containers for Shore Protection." Proceedings Coastal Structures 2003, Portland, Oregon.

Definitions

$\rho s := 165 \cdot \frac{lb}{ft^3}$	density of sand fill in geobags
$\rho w := 62.4 \cdot \frac{lb}{ft^3}$	density of water
$n := 0.45$	porosity of sand fill in geobags
$S := 0.5$	slope of geobags
$\theta := \mathrm{atan}(S) = 0.464$	angle of incline (radians)
$Ho := 20 \cdot ft$	deep water significant wave height
$Tp := 14 \cdot s$	peak wave period
$SWL := 6.6 \cdot ft$	still water level (NGVD)
$\eta s := 3.3 \cdot ft$	wave setup
$belev := 6.5 \cdot ft$	bed elevation during storm event (NGVD)
$L := 5.5 \cdot ft$	Length of geobags
$D := L \cdot \sin(\theta) = 2.46 \cdot ft$	Thickness of Cover Layer

Sediment Parameters

$\rho t := (1 - n) \cdot \rho s + n \cdot \rho w = 118.83 \cdot \frac{lb}{ft^3}$ density of top layer

$\Delta t := \frac{\rho t - \rho w}{\rho w} = 0.904$ relative mass under water of the top layer

Wave Parameters

$Lo := 1.56 \cdot Tp^2 \cdot \left(\frac{m}{s^2}\right) = 1.003 \times 10^3 \cdot ft$ deep water wave length

$\zeta op := \frac{S}{\left(\frac{Ho}{Lo}\right)^{0.5}} = 3.541$ surf similiarity parameter (Iribarren parameter)

$htoe := SWL + \eta s - belev = 3.4 \cdot ft$

$Htoe := 0.78 \cdot htoe = 2.652 \cdot ft$

Geobag Stability Criteria (derived from Hudson's Formula)

$$Dcr := \frac{Htoe \cdot (\zeta op)^{0.5}}{2.75 \Delta t} = 2.007 \cdot ft$$

D (2.46 ft) is > Dcr

GSC stable.

Date: March 21, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: Montauk Beach - FIMP
Analysis: Geobag Stability - 50 year Return Period

Calculate stability of geobag revetment based on modified Hudso's formula

References: Krystian W. Pilarczyk, 1998. "Dikes and Revetments," A.A. Balkema, Rotterdam, Netherlands.
H. Oumeraci, M. Hinz, M. Bleck, and A. Kortenhaus, 2003. "Sand-filled Geotextile Containers for Shore Protection." Proceedings Coastal Structures 2003, Portland, Oregon.

Definitions

$\rho s := 165 \cdot \frac{lb}{ft^3}$ density of sand fill in geobags

$\rho w := 62.4 \cdot \frac{lb}{ft^3}$ density of water

$n := 0.45$ porosity of sand fill in geobags

$S := 0.5$ slope of geobags

$\theta := \mathrm{atan}(S) = 0.464$ angle of incline (radians)

$Ho := 20 \cdot ft$ deep water significant wave height

$Tp := 17 \cdot s$ peak wave period

$SWL := 7.8 \cdot ft$ still water level (NGVD)

$\eta s := 3.6 \cdot ft$ wave setup

$belev := 6.5 \cdot ft$ bed elevation during storm event (NGVD)

$L := 5.5 \cdot ft$ Length of geobags

$D := L \cdot \sin(\theta) = 2.46 \cdot ft$ Thickness of Cover Layer

Sediment Parameters

$\rho t := (1 - n) \cdot \rho s + n \cdot \rho w = 118.83 \cdot \frac{lb}{ft^3}$ density of top layer

$\Delta t := \frac{\rho t - \rho w}{\rho w} = 0.904$ relative mass under water of the top layer

Wave Parameters

$Lo := 1.56 \cdot Tp^2 \cdot \left(\frac{m}{s^2}\right) = 1.479 \times 10^3 \cdot ft$ deep water wave length

$\zeta op := \frac{S}{\left(\frac{Ho}{Lo}\right)^{0.5}} = 4.3$ surf similiarity parameter (Iribarren parameter)

$htoe := SWL + \eta s - belev = 4.9 \cdot ft$

$Htoe := 0.78 \cdot htoe = 3.822 \cdot ft$

Geobag Stability Criteria (derived from Hudson's Formula)

$$Dcr := \frac{Htoe \cdot (\zeta op)^{0.5}}{2.75\Delta t} = 3.187 \cdot ft$$

D (2.46 ft) is < Dcr

GSCUnstable

ATTACHMENT C

BURIED SEAWALL DESIGN BACKUP

Date: October 8, 2013
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: Downtown Montauk Stabilization
Analysis: Buried Seawall Design

Note: This analysis was originally performed by Moffatt & Nichol in 2000 for the FIMP Basis of Design Report.

PROCEDURE:

1. Input Structural Constants, Design Water Level, Wave Height, and Wave Period

2. Estimate depth-limited wave conditions at site
   a. breaking wave condition
   b. maximum breaker height
   c. wave length at site

3. Compute maximum wave and significant wave heights based on random wave transformation by GODA

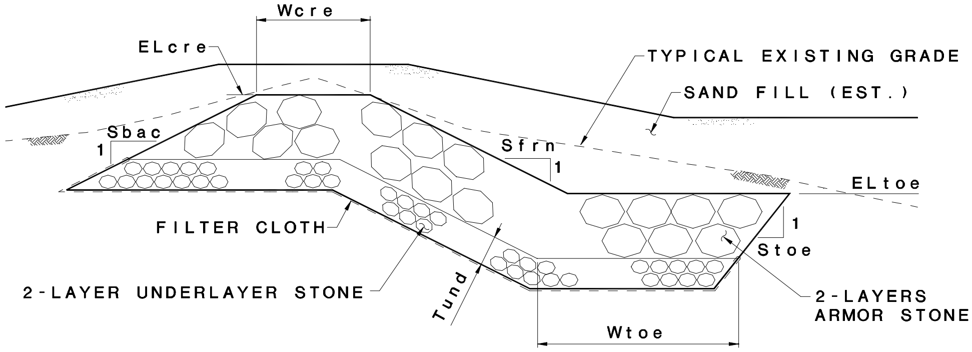
4. Perform runup calculations to determine seawall crest elevation
   a. based on Pilarcyzk (1990)
   b. based on van der Meer (1992)
   c. estimate mean runup assuming Rayleigh distribution
   d. estimate significant runup based on Pilarcyzk (1990)

5. Estimate overtopping
   a. based on van der Meer
   b. based on Pilarcyzk
   c. determine structure crest elevation based on tolerable overtopping limit

6. Seawall design
   a. determine armor size with Hudson formula and checked with van der Meer formulae
   b. determine armor and underlayer sizes and thicknesses
   c. determine berm width

7. Quantity Estimates

TYPICAL SECTION:



Structure Constants:
Elevation=11' NGVD
Slope=1(v):1.5(h)
Structure Variables:
Return Period=44, 73 and 150 years
Scour Design Toe Elevation= 5.6, 5.0, 4.3, 2.4 ft NGVD for typical profile MR1
Corresponding to Berm Widths=30, 20, 10, 0 meter

$\mathbf{ecrest} := 11.0\cdot\mathbf{ft}$

$\mathbf{Sslope} := \dfrac{1}{1.5}$ structure slope

CALCULATION

Water level, wave heights & wave periods for Eastern domain as obtained from hindcasts & CHL data:

$$\mathbf{RP} := \begin{pmatrix} 44 \\ 73 \\ 150 \end{pmatrix}\cdot\mathbf{yr} \quad \mathbf{wl} := \begin{pmatrix} 10.2 \\ 10.9 \\ 12.3 \end{pmatrix}\cdot\mathbf{ft} \quad \mathbf{Hs} := \begin{pmatrix} 17.2 \\ 17.4 \\ 18.0 \end{pmatrix}\cdot\mathbf{ft} \quad \mathbf{Tp} := \begin{pmatrix} 17.1 \\ 18.1 \\ 19.4 \end{pmatrix}\cdot\mathbf{sec}$$

$\mathbf{j} := 0..2$

DEPTH LIMITED WAVE CONDITIONS:

$$\mathbf{etoe} := \begin{pmatrix} 5.6 \\ 5.0 \\ 4.3 \\ 2.4 \end{pmatrix}\cdot\mathbf{ft}$$

$\mathbf{k} := 0..3$

Estimated scour elevation for berm widths =

$$\mathbf{bermw} := \begin{pmatrix} 30 \\ 20 \\ 10 \\ 0 \end{pmatrix}\cdot\mathbf{m}$$

$\mathbf{d}_{\mathbf{j},\mathbf{k}} := \mathbf{wl}_{\mathbf{j}} - \mathbf{etoe}_{\mathbf{k}}$ Compute limiting depth

$\mathbf{depth}_{\mathbf{j},\mathbf{k}} := \mathbf{if}\left(\mathbf{d}_{\mathbf{j},\mathbf{k}} \le 0\cdot\mathbf{ft}, 0.1\cdot\mathbf{ft}, \mathbf{d}_{\mathbf{j},\mathbf{k}}\right)$ Set minimum depth to 0.1 ft.

$$\mathbf{depth} = \begin{pmatrix} 4.6 & 5.2 & 5.9 & 7.8 \\ 5.3 & 5.9 & 6.6 & 8.5 \\ 6.7 & 7.3 & 8 & 9.9 \end{pmatrix}\cdot\mathbf{ft}$$

Depth-limited breaking wave conditions:

$\mathbf{slope} := \dfrac{1}{20}$ Breaker zone slope:

$\mathbf{Aw} := 43.8\cdot(1 - \mathbf{exp}(-19\cdot\mathbf{slope}))$ $\mathbf{Aw} = 26.9$

$\mathbf{Bw} := \dfrac{1.56}{(1 + \mathbf{exp}(-19.5\cdot\mathbf{slope}))}$ $\mathbf{Bw} = 1.1$

$\mathbf{Tm}_{\mathbf{j}} := 0.9\cdot\mathbf{Tp}_{\mathbf{j}}$ From EM 1110-2-1614

Initial guess at the max breaker height: $\mathbf{Hb} := 5 \cdot \mathbf{ft}$

Given

$$\mathbf{depth} = \frac{\mathbf{Hb}}{\mathbf{Bw} - \mathbf{Aw} \cdot \dfrac{\mathbf{Hb}}{\mathbf{g} \cdot \left(\mathbf{Tp}^2\right)}}$$

$$\mathbf{HBF}(\mathbf{depth}, \mathbf{Aw}, \mathbf{Bw}, \mathbf{Tp}) := \mathbf{Find}(\mathbf{Hb})$$

$$\mathbf{Hb}_{j,k} := \mathbf{HBF}\left(\mathbf{depth}_{j,k}, \mathbf{Aw}, \mathbf{Bw}, \mathbf{Tp}_j\right)$$

$$\kappa_{j,k} := \frac{\mathbf{Hb}_{j,k}}{\mathbf{depth}_{j,k}}$$ Wave breaking coefficient

$$\mathbf{RP} = \begin{pmatrix} 44 \\ 73 \\ 150 \end{pmatrix} \cdot \mathbf{yr} \qquad \mathbf{Hb} = \begin{pmatrix} 5.1 & 5.8 & 6.6 & 8.6 \\ 5.9 & 6.6 & 7.4 & 9.4 \\ 7.5 & 8.1 & 8.9 & 11 \end{pmatrix} \cdot \mathbf{ft} \qquad \kappa = \begin{pmatrix} 1.12 & 1.12 & 1.11 & 1.11 \\ 1.12 & 1.12 & 1.11 & 1.11 \\ 1.12 & 1.11 & 1.11 & 1.11 \end{pmatrix}$$

Wave length at structure toe used to estimate deepwater conditions

$\mathbf{L} := 100 \cdot \mathbf{ft}$ Initial Guess for Wavelength Calculation

Given

$$\mathbf{L} = \frac{\mathbf{g}}{2 \cdot \pi} \cdot \mathbf{T}^2 \cdot \mathbf{tanh}\left(2 \cdot \pi \cdot \frac{\mathbf{depth}}{\mathbf{L}}\right)$$

$$\mathbf{Wavel}(\mathbf{T}, \mathbf{depth}) := \mathbf{Find}(\mathbf{L})$$

$$\mathbf{L}_{j,k} := \mathbf{Wavel}\left(\mathbf{Tp}_j, \mathbf{depth}_{j,k}\right)$$

$$\mathbf{L} = \begin{pmatrix} 207.4 & 220.4 & 234.6 & 269.4 \\ 235.6 & 248.5 & 262.7 & 297.7 \\ 283.8 & 296.1 & 309.9 & 344.4 \end{pmatrix} \cdot \mathbf{ft}$$ Wavelength at structure toe

GODA Wave Calculation Method: random wave transformation to structure toe

$$Ks_{j,k} := \sqrt{\frac{1}{\tanh\left(2\cdot\pi\cdot\frac{depth_{j,k}}{L_{j,k}}\right)}\cdot\frac{1}{\left(1+4\cdot\pi\cdot\frac{\frac{depth_{j,k}}{L_{j,k}}}{\sinh\left(4\cdot\pi\cdot\frac{depth_{j,k}}{L_{j,k}}\right)}\right)}}$$

$$Ks = \begin{pmatrix} 1.91 & 1.85 & 1.79 & 1.68 \\ 1.89 & 1.84 & 1.79 & 1.69 \\ 1.85 & 1.81 & 1.77 & 1.68 \end{pmatrix}$$

Shoaling coefficient estimate

$$Hop_{j,k} := \frac{Hb_{j,k}}{1.8\cdot Ks_{j,k}}$$

Deepwater significant wave height from Goda

$$Hop = \begin{pmatrix} 1.5 & 1.7 & 2 & 2.9 \\ 1.7 & 2 & 2.3 & 3.1 \\ 2.2 & 2.5 & 2.8 & 3.6 \end{pmatrix}\cdot ft$$

Equivalent Deepwater Wave Height:

$$slopoff := \frac{1}{20} \qquad \theta b := atan(slopoff)$$

Bottom angle based on slope

$$Lp_j := \frac{g}{2\cdot\pi}\cdot\left(Tp_j\right)^2 \qquad Lp = \begin{pmatrix} 1497.3 \\ 1677.6 \\ 1927.2 \end{pmatrix}\cdot ft$$

Deep water wave length

Compute the Hmax=H1/250 wave height from Goda's Theory

First re-compute the maximum breaker height according to Goda

$$\mathbf{Hbg}_{j,k} := \mathbf{Lp}_j \cdot .18 \cdot \left[1 - e^{\left[-1.5\cdot\pi\cdot\frac{\mathbf{depth}_{j,k}}{\mathbf{Lp}_j}\cdot\left(1+15\cdot\tan(\theta b)^{\frac{4}{3}}\right)\right]}\right]$$

Goda Formula for Upper Limit Irregular Waves

Goda upper limit of irregular waves vs. depth-limited estimate

$$\mathbf{Hbg} = \begin{pmatrix} 4.9 & 5.6 & 6.3 & 8.3 \\ 5.7 & 6.3 & 7.1 & 9.1 \\ 7.2 & 7.8 & 8.6 & 10.6 \end{pmatrix}\cdot \mathbf{ft} \qquad \mathbf{Hb} = \begin{pmatrix} 5.1 & 5.8 & 6.6 & 8.6 \\ 5.9 & 6.6 & 7.4 & 9.4 \\ 7.5 & 8.1 & 8.9 & 11 \end{pmatrix}\cdot \mathbf{ft}$$

GODA'S COEFFS FOR MAX WAVES

$$\beta os_{j,k} := 0.052\cdot\left(\frac{\mathbf{Hop}_{j,k}}{\mathbf{Lp}_j}\right)^{-.38}\cdot e^{\left(20\cdot\tan(\theta b)^{1.5}\right)}$$

$$\beta 1s_j := 0.63\cdot e^{(3.8\cdot\tan(\theta b))}$$

$$\beta maxs_{j,k} := 0.53\cdot\left(\frac{\mathbf{Hop}_{j,k}}{\mathbf{Lp}_j}\right)^{-.29}\cdot e^{(2.4\cdot\tan(\theta b))}$$

$$\beta maxs_{j,k} := \text{if}\left(\beta maxs_{j,k} > 1.65, \beta maxs_{j,k}, 1.65\right)$$

$$\mathbf{Hmax1}_{j,k} := \beta os_{j,k}\cdot\mathbf{Hop}_{j,k} + \beta 1s_j\cdot\mathbf{depth}_{j,k}$$

$$\mathbf{Hmax2}_{j,k} := \beta maxs_{j,k}\cdot\mathbf{Hop}_{j,k}$$

$$\mathbf{Hmax3}_{j,k} := 1.8\cdot\mathbf{Ks}_{j,k}\cdot\mathbf{Hop}_{j,k}$$

$$\mathbf{H2}_{j,k} := \text{if}\left(\mathbf{Hmax1}_{j,k} < \mathbf{Hmax2}_{j,k}, \mathbf{Hmax1}_{j,k}, \mathbf{Hmax2}_{j,k}\right)$$

$$\mathbf{H2}_{j,k} := \text{if}\left(\mathbf{H2}_{j,k} < \mathbf{Hmax3}_{j,k}, \mathbf{H2}_{j,k}, \mathbf{Hmax3}_{j,k}\right)$$

GODA'S COEFFS FOR SIG WAVES

$$\beta o_{j,k} := 0.028\cdot\left(\frac{\mathbf{Hop}_{j,k}}{\mathbf{Lp}_j}\right)^{-.38}\cdot e^{\left(20\cdot\tan(\theta b)^{1.5}\right)}$$

$$\beta 1_j := 0.52\cdot e^{(4.2\cdot\tan(\theta b))}$$

$$\beta max_{j,k} := 0.32\cdot\left(\frac{\mathbf{Hop}_{j,k}}{\mathbf{Lp}_j}\right)^{-.29}\cdot e^{(2.4\cdot\tan(\theta b))}$$

$$\beta max_{j,k} := \text{if}\left(\beta max_{j,k} > .92, \beta max_{j,k}, .92\right)$$

$$\mathbf{Hs1}_{j,k} := \beta o_{j,k}\cdot\mathbf{Hop}_{j,k} + \beta 1_j\cdot\mathbf{depth}_{j,k}$$

$$\mathbf{Hs2}_{j,k} := \beta max_{j,k}\cdot\mathbf{Hop}_{j,k}$$

$$\mathbf{Hs3}_{j,k} := \mathbf{Ks}_{j,k}\cdot\mathbf{Hop}_{j,k}$$

$$\mathbf{Hsa}_{j,k} := \text{if}\left(\mathbf{Hs1}_{j,k} < \mathbf{Hs2}_{j,k}, \mathbf{Hs1}_{j,k}, \mathbf{Hs2}_{j,k}\right)$$

$$\mathbf{Hsa}_{j,k} := \text{if}\left(\mathbf{Hsa}_{j,k} < \mathbf{Hs3}_{j,k}, \mathbf{Hsa}_{j,k}, \mathbf{Hs3}_{j,k}\right)$$

$$\mathbf{H2}_{j,k} := \text{if}\left(\mathbf{H2}_{j,k} > \mathbf{Hbg}_{j,k}, \mathbf{Hbg}_{j,k}, \mathbf{H2}_{j,k}\right)$$

Set max value to upper limit

Depth

$$\mathbf{depth} = \begin{pmatrix} 4.6 & 5.2 & 5.9 & 7.8 \\ 5.3 & 5.9 & 6.6 & 8.5 \\ 6.7 & 7.3 & 8 & 9.9 \end{pmatrix} \cdot \mathbf{ft}$$

GODA
Hb

$$\mathbf{Hbg} = \begin{pmatrix} 4.9 & 5.6 & 6.3 & 8.3 \\ 5.7 & 6.3 & 7.1 & 9.1 \\ 7.2 & 7.8 & 8.6 & 10.6 \end{pmatrix} \cdot \mathbf{ft}$$

GODA
H2% approx. =
H1/250

$$\mathbf{H2} = \begin{pmatrix} 4.8 & 5.4 & 6.1 & 8 \\ 5.6 & 6.2 & 6.8 & 8.7 \\ 7 & 7.6 & 8.3 & 10.1 \end{pmatrix} \cdot \mathbf{ft}$$

GODA
Hsa (significant)

$$\mathbf{Hsa} = \begin{pmatrix} 2.9 & 3.2 & 3.7 & 4.8 \\ 3.3 & 3.7 & 4.1 & 5.2 \\ 4.2 & 4.5 & 4.9 & 6.1 \end{pmatrix} \cdot \mathbf{ft}$$

$$\mathbf{K}_{j,k} := \frac{\mathbf{H2}_{j,k}}{\mathbf{Hsa}_{j,k}}$$

$$\mathbf{K} = \begin{pmatrix} 1.7 & 1.69 & 1.68 & 1.66 \\ 1.69 & 1.69 & 1.68 & 1.66 \\ 1.69 & 1.68 & 1.67 & 1.66 \end{pmatrix}$$

Preliminary Revetment Design, RUNUP CALCULATIONS:

from Pilarcyzk (1990)

Equivalent surf similarity parameter

$$\xi\mathbf{p}_{j,k} := \frac{\mathbf{Sslope}}{\sqrt{\dfrac{\mathbf{Hsa}_{j,k}}{\mathbf{Lp}_j}}}$$

$$\gamma\mathbf{r} := 0.55$$

Runup roughness reduction coefficient

$$\mathbf{Fact}_{j,k} := 1.75 \cdot \xi\mathbf{p}_{j,k} \qquad \mathbf{Fact}_{j,k} := \mathbf{if}\left(\xi\mathbf{p}_{j,k} > 2.5, 3.5, \xi\mathbf{p}_{j,k}\right)$$

$$\mathbf{Fact} = \begin{pmatrix} 3.5 & 3.5 & 3.5 & 3.5 \\ 3.5 & 3.5 & 3.5 & 3.5 \\ 3.5 & 3.5 & 3.5 & 3.5 \end{pmatrix}$$

Runup from Pilarczyk

$$\mathbf{R2p}_{j,k} := \mathbf{Fact}_{j,k} \cdot \mathbf{Hsa}_{j,k} \cdot \gamma\mathbf{r}$$

$$\mathbf{R2p} = \begin{pmatrix} 5.5 & 6.2 & 7 & 9.2 \\ 6.3 & 7 & 7.9 & 10.1 \\ 8 & 8.7 & 9.5 & 11.7 \end{pmatrix} \cdot \mathbf{ft}$$

de Waal and Van der Meer (1992)

Depth reduction factor

$$\gamma h_{j,k} := 1 - 0.03 \cdot \left(4 - \frac{depth_{j,k}}{Hsa_{j,k}}\right)^2 \qquad \gamma h = \begin{pmatrix} 0.83 & 0.83 & 0.83 & 0.83 \\ 0.83 & 0.83 & 0.83 & 0.83 \\ 0.83 & 0.83 & 0.83 & 0.83 \end{pmatrix}$$

$$\gamma h_{j,k} := \text{if}\left(\frac{depth_{j,k}}{Hsa_{j,k}} > 4, 1, \gamma h_{j,k}\right) \qquad Fact_{j,k} := 1.5 \cdot \xi p_{j,k}$$

Upper runup limit:

$$Fact_{j,k} := \text{if}\left(Fact_{j,k} > 3, 3, Fact_{j,k}\right)$$

$$R2v_{j,k} := Fact_{j,k} \cdot \gamma r \cdot \gamma h_{j,k} \cdot Hsa_{j,k}$$

$$R2p = \begin{pmatrix} 5.5 & 6.2 & 7 & 9.2 \\ 6.3 & 7 & 7.9 & 10.1 \\ 8 & 8.7 & 9.5 & 11.7 \end{pmatrix} \cdot ft \qquad R2v = \begin{pmatrix} 3.9 & 4.4 & 5 & 6.6 \\ 4.5 & 5 & 5.6 & 7.2 \\ 5.7 & 6.2 & 6.8 & 8.4 \end{pmatrix} \cdot ft$$

$$trup_{j,k} := wl_j + R2p_{j,k}$$

$$truv_{j,k} := wl_j + R2v_{j,k}$$

Runup elevations from van der Meer and Pilarczyk (ft, NGVD)

$$truv = \begin{pmatrix} 14.1 & 14.6 & 15.2 & 16.8 \\ 15.4 & 15.9 & 16.5 & 18.1 \\ 18 & 18.5 & 19.1 & 20.7 \end{pmatrix} \cdot ft \qquad trup = \begin{pmatrix} 15.7 & 16.4 & 17.2 & 19.4 \\ 17.2 & 17.9 & 18.8 & 21 \\ 20.3 & 21 & 21.8 & 24 \end{pmatrix} \cdot ft$$

Overtopping Calculations:

$$\mathbf{Hm}_{j,k} := \frac{\mathbf{Hsa}_{j,k}}{1.6} \qquad \mathbf{Tm}_j := 0.9\cdot\mathbf{Tp}_j$$

$$\mathbf{Lo}_j := \frac{\mathbf{g}\cdot\left(\mathbf{Tm}_j\right)^2}{2\cdot\pi}$$

$$\mathbf{Hm} = \begin{pmatrix} 1.8 & 2 & 2.3 & 3 \\ 2.1 & 2.3 & 2.6 & 3.3 \\ 2.6 & 2.8 & 3.1 & 3.8 \end{pmatrix}\cdot\mathbf{ft} \qquad \mathbf{Tm} = \begin{pmatrix} 15.4 \\ 16.3 \\ 17.5 \end{pmatrix}\mathbf{s} \qquad \mathbf{wl} = \begin{pmatrix} 10.2 \\ 10.9 \\ 12.3 \end{pmatrix}\cdot\mathbf{ft}$$

$$\mathbf{Fc}_j := \mathbf{ecrest} - \mathbf{wl}_j \qquad \mathbf{Fc} = \begin{pmatrix} 0.8 \\ 0.1 \\ -1.3 \end{pmatrix}\cdot\mathbf{ft}$$

Structure freeboard calculations

$$\mathbf{FB}_{j,k} := \frac{\mathbf{R2v}_{j,k} - \mathbf{Fc}_j}{\mathbf{Hsa}_{j,k}} \qquad \mathbf{FB} = \begin{pmatrix} 1.1 & 1.1 & 1.1 & 1.2 \\ 1.3 & 1.3 & 1.3 & 1.4 \\ 1.7 & 1.7 & 1.6 & 1.6 \end{pmatrix}$$

Wave Overtopping By Van der Meer:

$$\mathbf{qv}_{j,k} := \sqrt{\mathbf{g}\cdot\left(\mathbf{Hsa}_{j,k}\right)^3}\cdot\left(8\cdot 10^{-5}\right)\cdot\mathbf{e}^{\left(3.1\cdot\frac{\mathbf{R2v}_{j,k}-\mathbf{Fc}_j}{\mathbf{Hsa}_{j,k}}\right)} \qquad \mathbf{qv}_{j,k} := \mathbf{if}\left(\mathbf{qv}_{j,k} < 0.01\cdot\frac{\mathbf{ft}^3}{\mathbf{ft}\cdot\mathbf{sec}},\ 0.01\cdot\frac{\mathbf{ft}^3}{\mathbf{ft}\cdot\mathbf{sec}},\ \mathbf{qv}_{j,k}\right)$$

$$\mathbf{qv} = \begin{pmatrix} 0.06 & 0.08 & 0.11 & 0.2 \\ 0.17 & 0.2 & 0.24 & 0.36 \\ 0.7 & 0.74 & 0.79 & 0.93 \end{pmatrix}\cdot\frac{\mathbf{ft}^3}{\mathbf{ft}\cdot\mathbf{sec}}$$

Estimate overtopping confidence bands

$$x_{j,k} := \log\left[\frac{qv_{j,k}}{\sqrt{g\cdot\left(Hsa_{j,k}\right)^3}}\right]$$

$$xu_{j,k} := x_{j,k} - 1.645\cdot\left(\frac{0.11\cdot x_{j,k}}{\sqrt{1}}\right) \qquad qvmax_{j,k} := 10^{xu_{j,k}}\cdot\sqrt{g\cdot\left(Hsa_{j,k}\right)^3}$$

$$xl_{j,k} := x_{j,k} + 1.645\cdot\left(\frac{0.11\cdot x_{j,k}}{\sqrt{1}}\right) \qquad qvmin_{j,k} := 10^{xl_{j,k}}\cdot\sqrt{g\cdot\left(Hsa_{j,k}\right)^3}$$

Overtopping calculation summary:

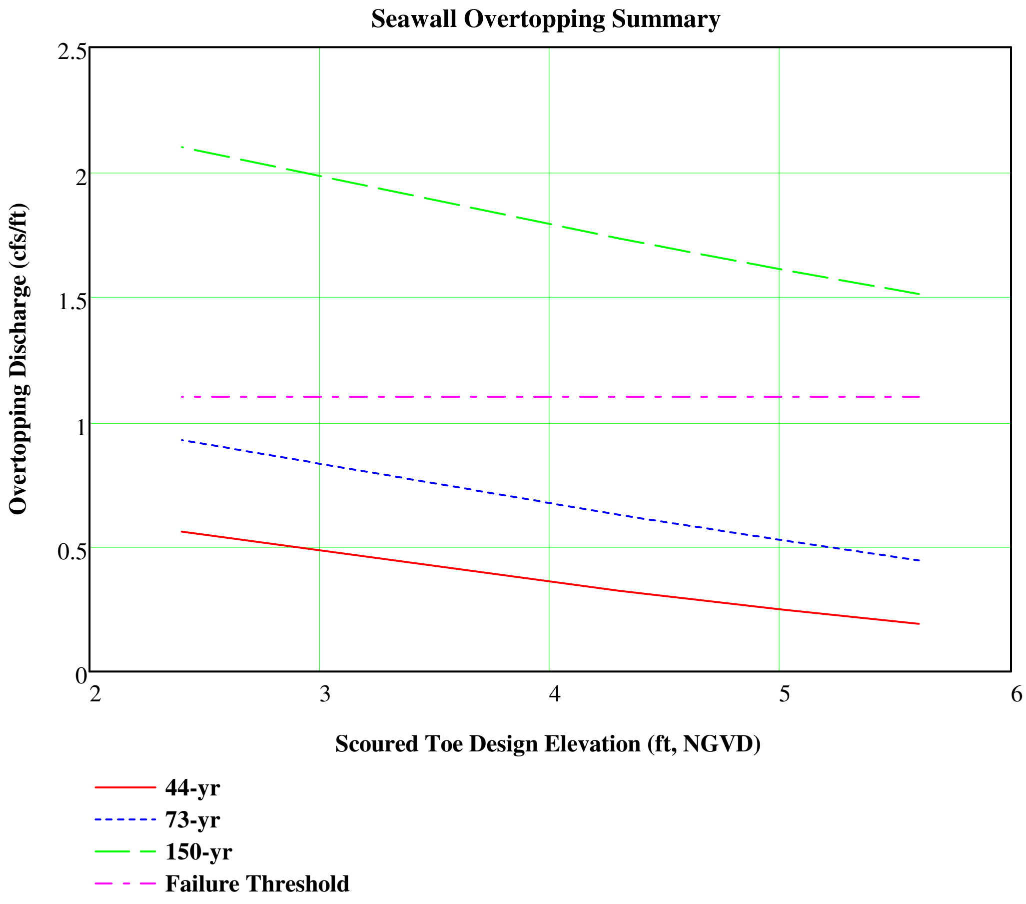
$$ecrest = 11\cdot ft$$

$$qv = \begin{pmatrix} 0.1 & 0.1 & 0.1 & 0.2 \\ 0.2 & 0.2 & 0.2 & 0.4 \\ 0.7 & 0.7 & 0.8 & 0.9 \end{pmatrix}\cdot\frac{ft^3}{ft\cdot sec} \qquad qvmax = \begin{pmatrix} 0.2 & 0.2 & 0.3 & 0.6 \\ 0.4 & 0.5 & 0.6 & 0.9 \\ 1.5 & 1.6 & 1.7 & 2.1 \end{pmatrix}\cdot\frac{ft^3}{ft\cdot sec}$$

Tolerable overtopping limit

$$tol := 1.1\cdot\frac{ft^3}{ft\cdot sec} \qquad RP = \begin{pmatrix} 44 \\ 73 \\ 150 \end{pmatrix}\cdot yr \qquad etoe = \begin{pmatrix} 5.6 \\ 5 \\ 4.3 \\ 2.4 \end{pmatrix}\cdot ft \qquad bermw = \begin{pmatrix} 30 \\ 20 \\ 10 \\ 0 \end{pmatrix} m$$

Seawall stone.xmcd



Revetment Armor Calculations:

$$\mathbf{wl} = \begin{pmatrix} 10.2 \\ 10.9 \\ 12.3 \end{pmatrix} \cdot \mathbf{ft} \qquad \mathbf{Hb} = \begin{pmatrix} 5.1 & 5.8 & 6.6 & 8.6 \\ 5.9 & 6.6 & 7.4 & 9.4 \\ 7.5 & 8.1 & 8.9 & 11 \end{pmatrix} \cdot \mathbf{ft} \qquad \mathbf{Hs} = \begin{pmatrix} 17.2 \\ 17.4 \\ 18 \end{pmatrix} \cdot \mathbf{ft} \qquad \mathbf{etoe} = \begin{pmatrix} 5.6 \\ 5 \\ 4.3 \\ 2.4 \end{pmatrix} \cdot \mathbf{ft}$$

$$\mathbf{depth} = \begin{pmatrix} 4.6 & 5.2 & 5.9 & 7.8 \\ 5.3 & 5.9 & 6.6 & 8.5 \\ 6.7 & 7.3 & 8 & 9.9 \end{pmatrix} \cdot \mathbf{ft} \qquad \mathbf{Tp} = \begin{pmatrix} 17.1 \\ 18.1 \\ 19.4 \end{pmatrix} \mathbf{s}$$

$$\mathbf{L} = \begin{pmatrix} 207.4 & 220.4 & 234.6 & 269.4 \\ 235.6 & 248.5 & 262.7 & 297.7 \\ 283.8 & 296.1 & 309.9 & 344.4 \end{pmatrix} \cdot \mathbf{ft} \qquad \mathbf{Lo} = \begin{pmatrix} 1.2 \times 10^3 \\ 1.4 \times 10^3 \\ 1.6 \times 10^3 \end{pmatrix} \cdot \mathbf{ft} \qquad \mathbf{Lp} = \begin{pmatrix} 1497.3 \\ 1677.6 \\ 1927.2 \end{pmatrix} \cdot \mathbf{ft}$$

$$\mathbf{Kd} := 2.0$$

$$\mathbf{cot\theta} := \frac{1}{\mathbf{Sslope}} \qquad \mathbf{\gamma r} := \frac{170 \cdot \mathbf{lb}}{\mathbf{ft}^3} \qquad \mathbf{\gamma w} := \frac{64 \cdot \mathbf{lb}}{\mathbf{ft}^3}$$

Structure characteristics

$$\mathbf{ton} := 2000 \cdot \mathbf{lb}$$

Hudson formula:

$$\mathbf{W}_{\mathbf{j},\mathbf{k}} := \frac{\mathbf{\gamma r} \cdot \left(\mathbf{H2}_{\mathbf{j},\mathbf{k}}\right)^3}{\mathbf{Kd} \cdot \left(\frac{\mathbf{\gamma r}}{\mathbf{\gamma w}} - 1\right)^3 \cdot \mathbf{cot\theta}} \qquad \mathbf{W} = \begin{pmatrix} 0.7 & 1 & 1.4 & 3.1 \\ 1.1 & 1.5 & 2 & 4.1 \\ 2.1 & 2.7 & 3.5 & 6.4 \end{pmatrix} \cdot \mathbf{ton}$$

van der Meer:

$$\theta := \mathrm{atan}\left(\frac{1}{\mathrm{Sslope}}\right)$$

Check required rock sizes using Van der Meer's Formulae

$\mathbf{P} := .4$ Structure Permeability factor

$\mathbf{N} := 7000$ Number of Waves

$\mathbf{Sd} := 2$ Damage Level, about 0 to 5 %

$$\Delta := \left(\frac{\gamma r}{\gamma w} - 1\right) \qquad \Delta = 1.7$$

$$\xi mc := \left[6.2 \cdot P^{.31} \cdot \sqrt{\left(\frac{1}{\cot\theta}\right)}\right]^{\frac{1}{(P+.5)}} \qquad \xi mc = 4.4 \qquad \xi m_{j,k} := \frac{\frac{1}{\cot\theta}}{\sqrt{\frac{Hsa_{j,k}}{Lo_j}}} \quad Sm_{j,k} := \frac{Hsa_{j,k}}{Lo_j}$$

$$\xi mc = 4.4$$

$\alpha 1_{j,k} := \xi m_{j,k} < \xi mc$ Factors to determine if plunging or surging waves

$\alpha 2_{j,k} := \xi m_{j,k} > \xi mc$

$$\xi m = \begin{pmatrix} 13.7 & 12.9 & 12.2 & 10.6 \\ 13.5 & 12.8 & 12.2 & 10.7 \\ 12.9 & 12.4 & 11.8 & 10.7 \end{pmatrix} \qquad \alpha 1 = \begin{pmatrix} 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \end{pmatrix} \qquad \alpha 2 = \begin{pmatrix} 1 & 1 & 1 & 1 \\ 1 & 1 & 1 & 1 \\ 1 & 1 & 1 & 1 \end{pmatrix}$$

Plunging Waves:

$$\mathbf{Dn50p}_{\mathbf{j},\mathbf{k}} := \frac{\mathbf{Hsa}_{\mathbf{j},\mathbf{k}}}{\left[6.2\cdot\mathbf{P}^{0.18}\cdot\left(\frac{\mathbf{Sd}}{\sqrt{\mathbf{N}}}\right)^{0.2}\cdot\left(\xi\mathbf{m}_{\mathbf{j},\mathbf{k}}\right)^{-0.5}\right]\cdot\Delta}\cdot\alpha\mathbf{1}_{\mathbf{j},\mathbf{k}} \qquad \mathbf{W50p}_{\mathbf{j},\mathbf{k}} := \left(\mathbf{Dn50p}_{\mathbf{j},\mathbf{k}}\right)^{3}\cdot\gamma\mathbf{r}$$

$$\mathbf{Dn50p} = \begin{pmatrix} 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \end{pmatrix}\cdot\mathbf{ft} \qquad \mathbf{W50p} = \begin{pmatrix} 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \end{pmatrix}\cdot\mathbf{ton}$$

Surging Waves:

$$\mathbf{Dn50s}_{\mathbf{j},\mathbf{k}} := \frac{\mathbf{Hsa}_{\mathbf{j},\mathbf{k}}}{\left[1.0\cdot\mathbf{P}^{-0.13}\cdot\left(\frac{\mathbf{Sd}}{\sqrt{\mathbf{N}}}\right)^{.2}\cdot\sqrt{\mathbf{cot\theta}}\cdot\left(\xi\mathbf{m}_{\mathbf{j},\mathbf{k}}\right)^{\mathbf{P}}\right]\cdot\Delta}\cdot\alpha\mathbf{2}_{\mathbf{j},\mathbf{k}} \qquad \mathbf{W50s}_{\mathbf{j},\mathbf{k}} := \left(\mathbf{Dn50s}_{\mathbf{j},\mathbf{k}}\right)^{3}\cdot\gamma\mathbf{r}$$

$$\mathbf{Dn50s} = \begin{pmatrix} 0.9 & 1.1 & 1.2 & 1.7 \\ 1.1 & 1.2 & 1.4 & 1.9 \\ 1.4 & 1.5 & 1.7 & 2.2 \end{pmatrix}\cdot\mathbf{ft} \qquad \mathbf{W50s} = \begin{pmatrix} 0.1 & 0.1 & 0.2 & 0.4 \\ 0.1 & 0.2 & 0.2 & 0.6 \\ 0.2 & 0.3 & 0.4 & 0.9 \end{pmatrix}\cdot\mathbf{ton}$$

$$\mathbf{W50} := \mathbf{W50p} + \mathbf{W50s} \qquad \mathbf{Dn50} := \mathbf{Dn50p} + \mathbf{Dn50s}$$

Compare van der Meer
and Hudson

$$\mathbf{W50} = \begin{pmatrix} 0.1 & 0.1 & 0.2 & 0.4 \\ 0.1 & 0.2 & 0.2 & 0.6 \\ 0.2 & 0.3 & 0.4 & 0.9 \end{pmatrix}\cdot\mathbf{ton} \qquad \mathbf{Dn50} = \begin{pmatrix} 0.9 & 1.1 & 1.2 & 1.7 \\ 1.1 & 1.2 & 1.4 & 1.9 \\ 1.4 & 1.5 & 1.7 & 2.2 \end{pmatrix}\cdot\mathbf{ft}$$

$$\mathbf{W} = \begin{pmatrix} 0.7 & 1 & 1.4 & 3.1 \\ 1.1 & 1.5 & 2 & 4.1 \\ 2.1 & 2.7 & 3.5 & 6.4 \end{pmatrix}\cdot\mathbf{ton}$$

Armor Layer Thickness:

Crest Width:

n

$$r_{j,k} := nl \cdot \left(\frac{W_{j,k}}{\gamma r}\right)^{\frac{1}{\,}} \qquad r = \begin{pmatrix} 4.1 & 4.6 & 5.1 & 6.7 \\ 4.7 & 5.2 & 5.7 & 7.3 \\ 5.9 & 6.4 & 6.9 & 8.5 \end{pmatrix} \cdot ft$$

$$rc_{j,k} := \frac{3}{2} \cdot r_{j,k}$$

$$rc = \begin{pmatrix} 6.1 & 6.8 & 7.7 & 10 \\ 7 & 7.7 & 8.6 & 10.9 \\ 8.8 & 9.5 & 10.4 & 12.7 \end{pmatrix} \cdot ft$$

Burial depth at bottom of primary toe (a=Hs)
Width of primary toe (b=2a)

$$a_{j,k} := Hb_{j,k} \qquad b_{j,k} := 2 \cdot a_{j,k}$$

$$a = \begin{pmatrix} 5.1 & 5.8 & 6.6 & 8.6 \\ 5.9 & 6.6 & 7.4 & 9.4 \\ 7.5 & 8.1 & 8.9 & 11 \end{pmatrix} \cdot ft \qquad b = \begin{pmatrix} 10.3 & 11.6 & 13.1 & 17.3 \\ 11.8 & 13.2 & 14.7 & 18.8 \\ 15 & 16.3 & 17.8 & 21.9 \end{pmatrix} \cdot ft$$

Underlayer:

$$Wu := \frac{W}{10}$$

$$ru_{j,k} := 2 \cdot \left(\frac{Wu_{j,k}}{\gamma r}\right)^{\frac{1}{3}} \qquad Wu = \begin{pmatrix} 142.2 & 200.6 & 286 & 627.1 \\ 216.4 & 292.3 & 400.5 & 815.8 \\ 429 & 545.8 & 706 & 1285.6 \end{pmatrix} \cdot lb$$

$$Wu = \begin{pmatrix} 0.1 & 0.1 & 0.1 & 0.3 \\ 0.1 & 0.1 & 0.2 & 0.4 \\ 0.2 & 0.3 & 0.4 & 0.6 \end{pmatrix} \cdot ton \qquad ru = \begin{pmatrix} 1.9 & 2.1 & 2.4 & 3.1 \\ 2.2 & 2.4 & 2.7 & 3.4 \\ 2.7 & 3 & 3.2 & 3.9 \end{pmatrix} \cdot ft$$

$$W = \begin{pmatrix} 0.7 & 1 & 1.4 & 3.1 \\ 1.1 & 1.5 & 2 & 4.1 \\ 2.1 & 2.7 & 3.5 & 6.4 \end{pmatrix} \cdot ton$$

QUANTITY ESTIMATES:

Design Feature for 1.0 ft shore parallel section:

$\mathbf{j} := 0 .. 2$

$\mathbf{Tarm} := \mathbf{r}$ $\mathbf{Tund} := \mathbf{ru}$ $\mathbf{Wcre} := \frac{3}{2}\cdot\mathbf{r}$ $\mathbf{Wtoe} := \mathbf{b}$ $\mathbf{ELcre} := \mathbf{ecrest}$

$$\mathbf{RP} := \begin{pmatrix} 44 \\ 73 \\ 150 \end{pmatrix}\cdot\mathbf{yr} \quad \mathbf{Tarm} = \begin{pmatrix} 4.1 & 4.6 & 5.1 & 6.7 \\ 4.7 & 5.2 & 5.7 & 7.3 \\ 5.9 & 6.4 & 6.9 & 8.5 \end{pmatrix}\cdot\mathbf{ft} \quad \mathbf{Tund} = \begin{pmatrix} 1.9 & 2.1 & 2.4 & 3.1 \\ 2.2 & 2.4 & 2.7 & 3.4 \\ 2.7 & 3 & 3.2 & 3.9 \end{pmatrix}\cdot\mathbf{ft}$$

$\mathbf{ELcre} = 11\cdot\mathbf{ft}$

$$\mathbf{Wcre} = \begin{pmatrix} 6.1 & 6.8 & 7.7 & 10 \\ 7 & 7.7 & 8.6 & 10.9 \\ 8.8 & 9.5 & 10.4 & 12.7 \end{pmatrix}\cdot\mathbf{ft} \quad \mathbf{Wtoe} = \begin{pmatrix} 10.3 & 11.6 & 13.1 & 17.3 \\ 11.8 & 13.2 & 14.7 & 18.8 \\ 15 & 16.3 & 17.8 & 21.9 \end{pmatrix}\cdot\mathbf{ft}$$

$$\mathbf{ELtoe}_{\mathbf{j},\mathbf{k}} := \mathbf{etoe}_{\mathbf{k}} + \frac{\mathbf{r}_{\mathbf{j},\mathbf{k}}}{2}$$

$\mathbf{Sfrn} := \frac{1}{\mathbf{Sslope}}$

$\mathbf{Sbac} := \mathbf{Sfrn}$

$\mathbf{Stoe} := 1.0$

$\boldsymbol{\theta}\mathbf{1} := \mathbf{atan}\left(\frac{1}{\mathbf{Sfrn}}\right)$ $\boldsymbol{\theta}\mathbf{1} = 0.6$ $\sin(\boldsymbol{\theta}\mathbf{1}) = 0.55$

$\boldsymbol{\theta}\mathbf{2} := \mathbf{atan}\left(\frac{1}{\mathbf{Sbac}}\right)$ $\boldsymbol{\theta}\mathbf{2} = 0.6$

$\boldsymbol{\theta}\mathbf{3} := \mathbf{atan}\left(\frac{1}{\mathbf{Stoe}}\right)$ $\boldsymbol{\theta}\mathbf{3} = 0.8$

Area of Armor: A1+A2+A3+Atolerance

$$\mathbf{A1}_{\mathbf{j},\mathbf{k}} := 0.5\cdot\mathbf{Tarm}_{\mathbf{j},\mathbf{k}}\cdot\left[2\cdot\mathbf{Wcre}_{\mathbf{j},\mathbf{k}} + \mathbf{Tarm}_{\mathbf{j},\mathbf{k}}\cdot(\mathbf{Sbac} + \mathbf{Sfrn})\right]$$

$$\mathbf{A2}_{\mathbf{j},\mathbf{k}} := 2\cdot\frac{\mathbf{Tarm}_{\mathbf{j},\mathbf{k}}}{\sin(\boldsymbol{\theta}\mathbf{1})}\cdot\left(\mathbf{ELcre} - \mathbf{ELtoe}_{\mathbf{j},\mathbf{k}} - \mathbf{Tarm}_{\mathbf{j},\mathbf{k}}\right)$$

$$\mathbf{A3}_{\mathbf{j},\mathbf{k}} := 0.5\cdot\mathbf{Tarm}_{\mathbf{j},\mathbf{k}}\cdot\left[2.0\cdot\mathbf{Wtoe}_{\mathbf{j},\mathbf{k}} + \mathbf{Tarm}_{\mathbf{j},\mathbf{k}}\cdot(\mathbf{Sfrn} + \mathbf{Stoe})\right]$$

$$\mathbf{Atol}_{\mathbf{j},\mathbf{k}} := \left[\mathbf{Wcre}_{\mathbf{j},\mathbf{k}} + \mathbf{Wtoe}_{\mathbf{j},\mathbf{k}} + \left(\mathbf{ELcre} - \mathbf{ELtoe}_{\mathbf{j},\mathbf{k}}\right)\cdot\left(\mathbf{sin}(\theta\mathbf{1})^{-1} + \mathbf{sin}(\theta\mathbf{2})^{-1}\right)\right]\cdot 1\cdot\mathbf{ft}$$

$$\mathbf{Aarmor} := \mathbf{A1} + \mathbf{A2} + \mathbf{A3} + \mathbf{Atol}$$

Cross-sectional feature:

$$\mathbf{Wcre1}_{\mathbf{j},\mathbf{k}} := \mathbf{Wcre}_{\mathbf{j},\mathbf{k}} + \mathbf{Tarm}_{\mathbf{j},\mathbf{k}}\cdot\left(\mathbf{Sbac} + \mathbf{Sfrn} - \frac{1}{\mathbf{sin}(\theta\mathbf{1})}\right)$$

$$\mathbf{Wtoe1}_{\mathbf{j},\mathbf{k}} := \mathbf{Wtoe}_{\mathbf{j},\mathbf{k}} + \mathbf{Tund}_{\mathbf{j},\mathbf{k}}\cdot\left(\frac{1}{\mathbf{sin}(\theta\mathbf{1})} - \mathbf{Sfrn} - \mathbf{Stoe}\right)$$

Area of Underlayer: C1+C2+C3

$$\mathbf{C1}_{\mathbf{j},\mathbf{k}} := 0.5\cdot\mathbf{Tund}_{\mathbf{j},\mathbf{k}}\cdot\left[2\cdot\mathbf{Wcre1}_{\mathbf{j},\mathbf{k}} + \mathbf{Tund}_{\mathbf{j},\mathbf{k}}\cdot(\mathbf{Sbac} + \mathbf{Sfrn})\right]$$

$$\mathbf{C2}_{\mathbf{j},\mathbf{k}} := \frac{\mathbf{Tarm}_{\mathbf{j},\mathbf{k}}}{\mathbf{sin}(\theta\mathbf{1})}\cdot\mathbf{Tund}_{\mathbf{j},\mathbf{k}}$$

$$\mathbf{C3}_{\mathbf{j},\mathbf{k}} := 0.5\cdot\mathbf{Tund}_{\mathbf{j},\mathbf{k}}\cdot\left[2.0\cdot\mathbf{Wtoe1}_{\mathbf{j},\mathbf{k}} + \mathbf{Tund}_{\mathbf{j},\mathbf{k}}\cdot(\mathbf{Sfrn} + \mathbf{Stoe})\right]$$

$$\mathbf{Aund} := \mathbf{C1} + \mathbf{C2} + \mathbf{C3}$$

Filter Cloth Area:

$$\mathbf{Lfilter} := \frac{\mathbf{Tund}}{\mathbf{sin}(\theta\mathbf{3})} + \mathbf{Wtoe1} + \frac{\mathbf{Tarm} + \mathbf{Tund}}{\mathbf{sin}(\theta\mathbf{1})} + \mathbf{Wcre1} + \mathbf{Tund}\cdot\mathbf{Sbac} + \frac{\mathbf{Tund}}{\mathbf{sin}(\theta\mathbf{2})}$$

$$\mathbf{Lfilter} = \begin{pmatrix} 39.5 & 44.4 & 50.1 & 65.2 \\ 45.5 & 50.3 & 56 & 71.2 \\ 57.2 & 62.1 & 67.7 & 82.9 \end{pmatrix}\cdot\mathbf{ft}$$

Armor Quantity:

$\gamma w := 170 \cdot \frac{lb}{ft^3}$ **Porosity** $:= 0.37$ $cy := 27 \cdot ft^3$ $sy := 9 \cdot ft^2$ $ton := 2000 \cdot lb$

$Conv := \gamma w \cdot (1 - Porosity)$ $Conv = 1.446 \cdot \frac{ton}{cy}$

Total Length:

$L1 := 1.0 \cdot ft$

$Varmor := Aarmor \cdot L1$

$Warmor := Varmor \cdot Conv$

$$Varmor = \begin{pmatrix} 4.8 & 5.9 & 7.3 & 11.7 \\ 5.6 & 6.7 & 8.2 & 12.9 \\ 7.2 & 8.5 & 10.2 & 15.4 \end{pmatrix} \cdot cy$$

$$Warmor = \begin{pmatrix} 7 & 8.5 & 10.5 & 16.9 \\ 8.1 & 9.7 & 11.9 & 18.7 \\ 10.4 & 12.3 & 14.7 & 22.2 \end{pmatrix} \cdot ton$$

Underlayer Quantity

$Vund := Aund \cdot L1$

$$Vund = \begin{pmatrix} 2.3 & 2.9 & 3.6 & 6.1 \\ 3 & 3.7 & 4.5 & 7.3 \\ 4.7 & 5.6 & 6.6 & 9.9 \end{pmatrix} \cdot cy$$

$Wund := Vund \cdot Conv$

$$Wund = \begin{pmatrix} 3.3 & 4.1 & 5.2 & 8.9 \\ 4.3 & 5.3 & 6.6 & 10.6 \\ 6.8 & 8 & 9.6 & 14.3 \end{pmatrix} \cdot ton$$

Filter Cloth Quantity:

$Afilter := Lfilter \cdot L1$

$$Afilter = \begin{pmatrix} 4.4 & 4.9 & 5.6 & 7.2 \\ 5.1 & 5.6 & 6.2 & 7.9 \\ 6.4 & 6.9 & 7.5 & 9.2 \end{pmatrix} \cdot sy$$

Excavation for Revetment:

Assume that excavation volume is approximately the total stone volume plus 20%

$a := 0.2$

$Vexca := (Varmor + Vund) \cdot (1 + a)$

$$Vexca = \begin{pmatrix} 8.5 & 10.5 & 13.1 & 21.4 \\ 10.3 & 12.5 & 15.3 & 24.3 \\ 14.3 & 16.9 & 20.2 & 30.3 \end{pmatrix} \cdot cy$$

Summary of Design Configuration

Use Index (0,2) or 44 year return period & 4.3 foot toe elevation which corresponds to a 10 m berm width to calculate quantities

$\mathbf{ELcre} = 11 \cdot \mathbf{ft}$ crest elevation

$\mathbf{etoe}_2 = 4.3 \cdot \mathbf{ft}$ toe elevation

$\frac{1}{\mathbf{Sslope}} = 1.5$ structure slope

$\mathbf{qv}_{0,2} = 0.1 \cdot \frac{\mathbf{ft}^3}{\mathbf{ft} \cdot \mathbf{sec}}$ mean overtopping rate

$\mathbf{tol} = 1.1 \cdot \frac{\mathbf{ft}^3}{\mathbf{ft} \cdot \mathbf{sec}}$ mean overtopping rate threshold

$\mathbf{W}_{0,2} = 1.4 \cdot \mathbf{ton}$ armor stone size

$\mathbf{Tarm}_{0,2} = 5.1 \cdot \mathbf{ft}$ armor stone layer thickness

$\mathbf{Wcre}_{0,2} = 7.7 \cdot \mathbf{ft}$ armor stone crest width

$\mathbf{Wu}_{0,2} = 0.1 \cdot \mathbf{ton}$ under layer stone size

$\mathbf{Tund}_{0,2} = 2.4 \cdot \mathbf{ft}$ under layer stone layer thickness

$\mathbf{Wtoe}_{0,2} = 13.1 \cdot \mathbf{ft}$ scour toe berm width

Total Quantities

Use Index (0,2) or 44 year return period & 4.3 foot toe elevation which corresponds to a 10 m berm width to calculate quantities

$\mathbf{PL} := 3150 \cdot \mathbf{ft}$ Project Length

$\mathbf{Warmor\_PL} := \mathbf{Warmor}_{0,2} \cdot \frac{\mathbf{PL}}{\mathbf{ft}} = 33145 \cdot \mathbf{ton}$ Total Armor Stone Quantity

$\mathbf{Wund\_PL} := \mathbf{Wund}_{0,2} \cdot \frac{\mathbf{PL}}{\mathbf{ft}} = 16487 \cdot \mathbf{ton}$ Total Underlayer Stone Quantity

$\mathbf{Afilter\_PL} := \mathbf{Afilter}_{0,2} \cdot \frac{\mathbf{PL}}{\mathbf{ft}} = 17520 \cdot \mathbf{sy}$ Total Area of Filter

$\mathbf{Vexca\_PL} := \mathbf{Vexca}_{0,2} \cdot \frac{\mathbf{PL}}{\mathbf{ft}} = 41193 \cdot \mathbf{cy}$ Total volume of excavation

ATTACHMENT D

UPLAND SEDIMENT ANALYSIS

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Upland Sediment Sources - Montauk

By: Dornhelm, Esther

Date: 03/18/2014

Abstract: Summary of potential upland sand distributors, their prices, sand properties, and overfill required.

There are five principal upland source of sand on Long Island, with two located in Montauk with the capability to supply the entire volume of sand required (45,000 CY). There is also potential for the sand distributor to deliver and place the sand. A list of the two sand distributors being considered, their information, sand properties, and transportation capacity is shown in Table 1. Figure 1 shows the location of the sand stockpiles. The upland suppliers were contacted to provide price quotes for the cost of the raw material as well as the cost including transportation to downtown Montauk

A comparison of grain size distribution was completed to determine overfill required for each option. Overfill (placing over 1 cubic yard a fill for every 1 cubic yard of beach sand required at the site) is required to compensate for the finer sands in the placed sand that is lost when subjected to the native beach's sediment transport environment. For this reason, it is preferred to place sand with similar or larger grain size to the native beach. The overfill factor (R_A) was determined following the methodology presented in *Shore Protection Manual (1984)* . Grain size distributions and calculations are shown in Table 2. The graph that was used to determine R_A is shown in Figure 2.

The compatibility of the color of the sediment is illustrated by Figure 4 which compares sediment samples from the two upland sediment sources. The "white" sand on the left of Figure 4 is the East Coast Coarse Washed and the "yellow" sample on right is from Bistrian.

Table 1: Summary of Sand Distributors

Potential Upland Source	Phone Number	Location	Miles to	Sand Type	Median Grain Size	Overfill Factor	Color	Cost per CY*		Availability	Transportation
								Material	Material & Transport		
Bistrian	631-324-1123	225 Spring Fireplace Road	14.2	N/A	0.5 mm	1.12	More yellow than native beach sand.	$14.00	$21.00	Sufficient. 55-60,000 CY in stock pile currently.	Fleet available to transport 1000 CY per day using a combination of 20 CY dump trucks and 35 CY trailers.
East Coast	631-653-5445	585 Middle Line Hwy	24.6	Coarse Washed Sand	0.91 mm	1.0	More white than native beach sand.	$13.23	$18.98	Sufficient.	Fleet of six-40 ton trailers and three-25 ton tri-axles.
				Fine Dry Sand	0.34 mm	1.75	More white than native beach sand.	$12.08	$17.83		
				Fine Washed Sand	0.44 mm	1.25%	More white than native beach sand.	$13.23	$18.98		

* Assuming 1.15 tons per CY.

Table 2: Determining Overfill Factor using Grain Size Distribution

	D50* (mm)	D16* (mm)	D84* (mm)	D50* (phi)	D16* (phi)	D84* (phi)	Std Dev (phi)	Mean (phi)	σϕb/σϕn	(Mϕb-Mϕn)/σϕn	RA from Chart
Native Sand	0.42	0.30	0.77	1.25	1.72	0.38	0.67	1.05	1.00	0.00	
Bistrian	0.51	0.81	0.27	0.97	0.30	1.89	0.79	1.10	1.18	0.07	1.12
East Coast Coarse (Washed)	0.905	1.7	0.43	0.14	-0.77	1.22	0.99	0.23	1.48	-1.23	1
East Coast Fine (Dry)	0.34	0.64	0.202	1.56	0.64	2.31	0.83	1.48	1.24	0.64	1.75
East Coast Fine (Washed)	0.44	1.15	0.22	1.18	-0.20	2.18	1.19	0.99	1.78	-0.09	1.25

*Percentiles represent "percent coarser"

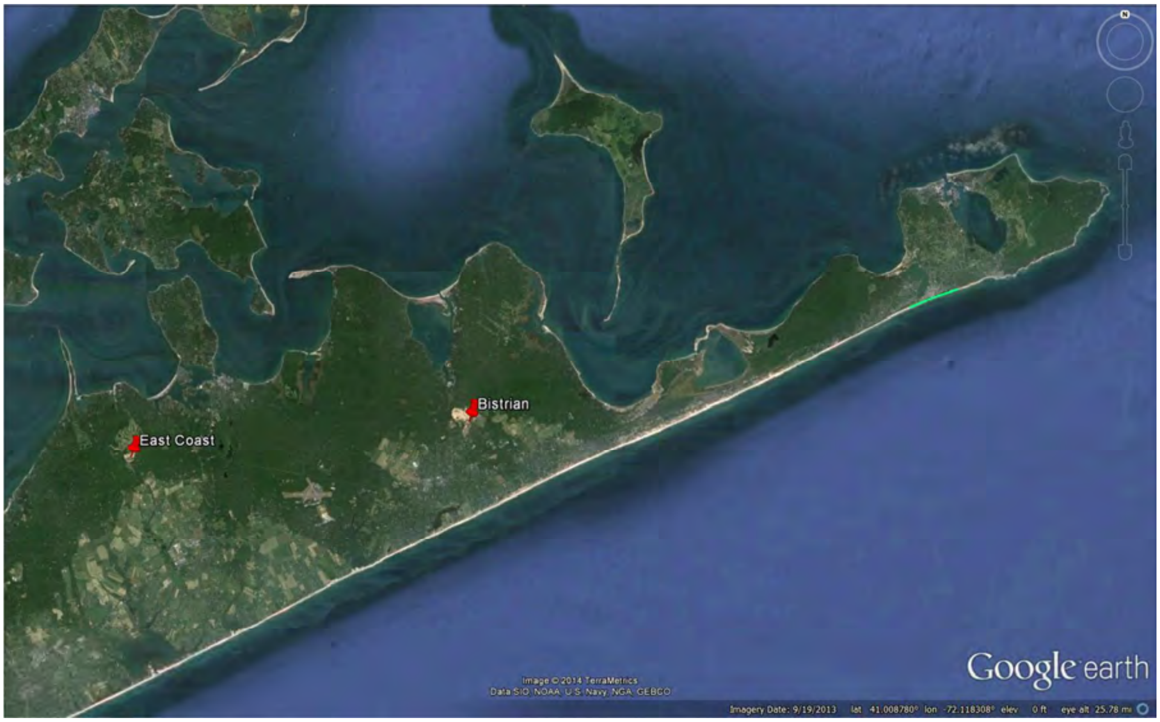

Figure 1: Sand Distributor Locations

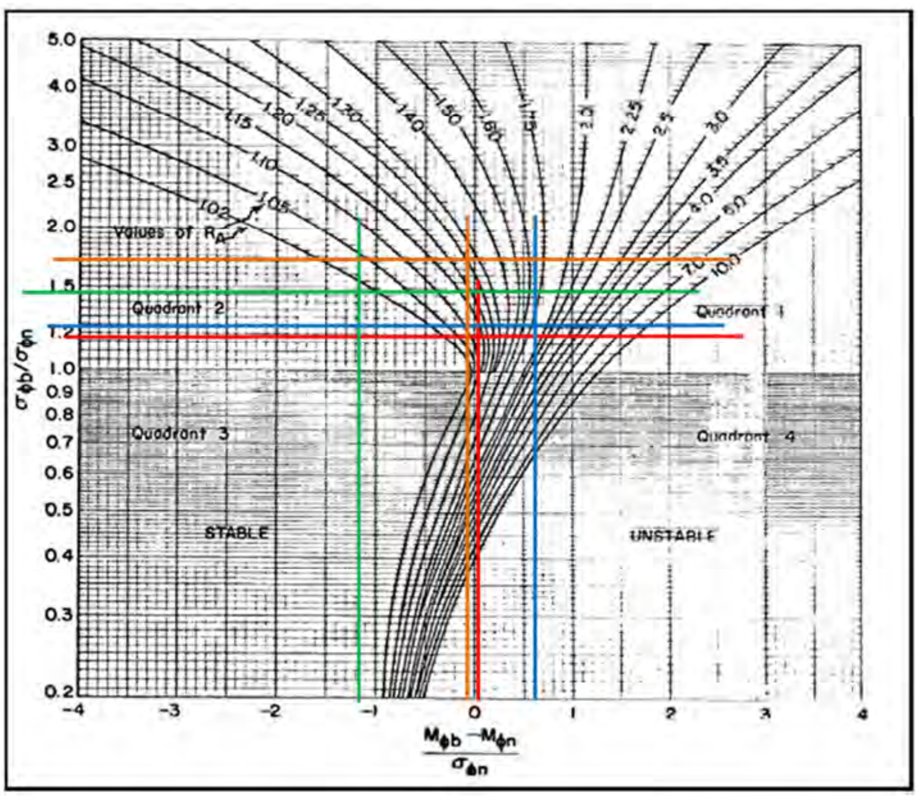

Figure 2: Overfill Factor Isolines

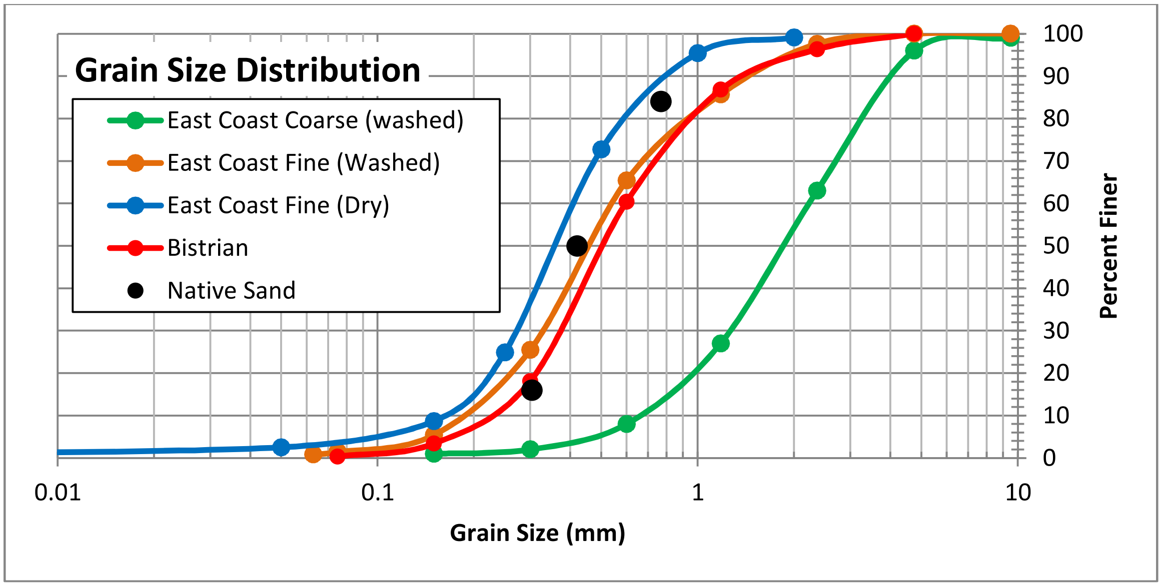

Figure 3: Grain Size Distribution of Upland Sediment Sources

Figure 4: Upland Sediment Samples

ATTACHMENT E

BEACHFILL DIFUSSION ANALYSIS

Date: July 18, 2013
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: Fire Island Interim
Analysis: Alongshore Diffusivity

Solving for the Alongshore Diffusivity along Fire Island based on predicted Gross Sediment Transport Rate

Reference: Dean R. G., Dalyrmple R. A., 2002. "Coastal Processes with Engineering Applications," Cambridge University Press, New York, NY.)

Definitions

$GST := 2250000 \cdot \frac{m^3}{yr}$	gross sediment transport at Montauk (Gravens et al., 1999)
$S := 2.65$	specific gravity of sand
$p := 0.35$	porosity of sand
$\gamma b := 0.78$	breaking wave index
$K := 0.77$	sediment transport coefficient for medium sand (e.g. 0.3 mm) (Komar & Inman 1970)
$\theta b := 10 \cdot deg$	effective breaking wave angle
$hc := 27 \cdot ft$	depth of closure (NGVD)
$B := 9.5 \cdot ft$	berm elevation (NGVD)
$Tp := 8 \cdot s$	assumed effective wave period

CERC Sediment Transport Equation

$$GST = \blacksquare \cdot Cp \cdot Hb^{\frac{5}{2}} \cdot \sin(2 \cdot \theta b)$$ CERC Equation

$$Cp := K \cdot \frac{\sqrt{\frac{g}{\gamma b}}}{16 \cdot (S - 1) \cdot (1 - p)} = 0.159\, m^{0.5} \cdot s^{-1}$$

$$Hb := \left(\frac{GST}{Cp \cdot \sin(2 \cdot \theta b)}\right)^{\frac{2}{5}} = 1.114\, m$$ effective breaking wave height

Alongshore Diffusivity, G

$$G := 2 \cdot Cp \cdot Hb^{\frac{5}{2}} \cdot \frac{\cos(2 \cdot \theta b)}{hc + B} = 0.035\, m^2 \cdot s^{-1}$$

Effect of Wave Refraction on Alongshore Diffusivity, G

Reference: Dean R. G., 2005. "Advanced Series on Ocean Engineering - Volume 18: Beach Nourishment Theory and Practice," World Scientific Publishing Co., Hackensack, NJ.)

Dean showed that wave refraction at a beachfill project can reduce the alongshore diffusivity by the ratio Cb/Cc where Cb and Cc are the wave celerity at breaking and depth of closure respectively

Wave Length and Celerity at depth of closure

$L := 150 \cdot m$ Initial value

Given

$$L = \frac{g}{2 \cdot \pi} \cdot T^2 \cdot \tanh\left(2 \cdot \pi \cdot \frac{depth}{L}\right)$$

$Wavel(T, depth) := Find(L)$

$Lc := Wavel(Tp, hc) = 65.641\,m$

$Cc := \frac{Lc}{Tp} = 8.205\,m \cdot s^{-1}$

Wave Length and Celerity at break point

$L := 150 \cdot m$ Initial value

Given

$$L = \frac{g}{2 \cdot \pi} \cdot T^2 \cdot \tanh\left(2 \cdot \pi \cdot \frac{depth}{L}\right)$$

$Wavel(T, depth) := Find(L)$

$Lb := Wavel\left(Tp, \frac{Hb}{\gamma b}\right)$ $Lb = 29.492\,m$

$Cb := \frac{Lb}{Tp} = 3.687\,m \cdot s^{-1}$

$\frac{Cb}{Cc} = 0.449$

$ref := 0.4$ set reduction factor to 0.4

Adjusted Alongshore Diffusivity, G

$G := G \cdot ref = 0.0141\,m^2 \cdot s^{-1}$

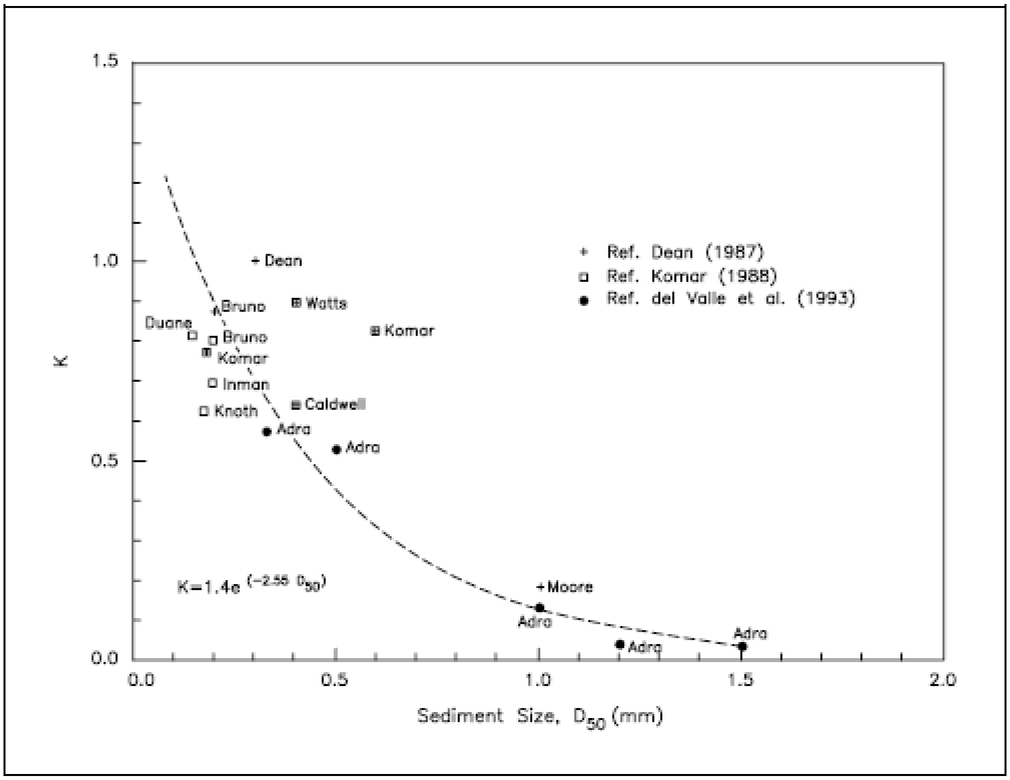

Figure III-2-6. Coefficient *K* versus median grain size D_{50} (del Valle, Medina, and Losada 1993)

Reproduced from CEM, Page III-2-15

Date: October 8, 2013
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: Montauk Beach - FIMP
Analysis: Beachfill Diffusion - MREI Beachfill

Calculate fraction of beachfill volume remaining with Pelnard-Considere Equation for Rectangular Beachfill

Reference: Dean R. G., Dalyrmple R. A., 2002. "Coastal Processes with Engineering Applications," Cambridge University Press, New York, NY.)

Note: This analysis does not include beachfill tapers, the final analysis for the Montauk Beach applied 6 degree berm tapers, which requires the trapazoidal beachfill solution to the Pelnard-Considere Equation. The trapzoidal beachfill solution was solved numerically in Matlab.

Definitions

$hc := 27 \cdot ft$ depth of closure (NGVD)

$B := 9.5 \cdot ft$ berm elevation (NGVD)

$G := 0.014 \cdot \frac{m^2}{s}$ alongshore diffusivity

$Yo := 60 \cdot ft$ initial cross-shore distance between design shoreline and natural shoreline

$Ya := 91.2 \cdot ft$ advance nourishment width

$l := 6600 \cdot ft$ alongshore length of beachfill

$t := 4yr$ time after initial placement

$be := 3 \cdot \frac{ft}{yr}$ background erosion rate

$Yb := 6 \cdot ft$ addditional distance natural shoreline sticks out relative to baseline

Analytical Solution to Pelnard-Considere for Rectangular Beachfill Project

$$M := \left[\frac{\sqrt{4 \cdot G \cdot t}}{l \cdot \sqrt{\pi}} \cdot \left[e^{-\left(\frac{l}{\sqrt{4 \cdot G \cdot t}}\right)^2} - 1\right] + \operatorname{erf}\left(\frac{l}{\sqrt{4G \cdot t}}\right)\right] - \frac{be \cdot t}{(Yo + Ya - Yb)}$$ fraction of initial fill remaining after time t

$M = 0.308$

$re := (1 - M) \cdot \frac{(Yo + Ya - Yb)}{t} = 25.129 \cdot \frac{ft}{yr}$ representative erosion rate

$de := re - be = 22.129 \cdot \frac{ft}{yr}$ diffusive erosion rate

Date: October 8, 2013
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: Montauk Beach - FIMP
Analysis: Beachfill Diffusion - MREI Beachfill and Buried Seawall

Calculate fraction of beachfill volume remaining with Pelnard-Considere Equation for Rectangular Beachfill

Reference: Dean R. G., Dalyrmple R. A., 2002. "Coastal Processes with Engineering Applications," Cambridge University Press, New York, NY.)

Note: This analysis does not include beachfill tapers, the final analysis for the Montauk Beach applied 6 degree berm tapers, which requires the trapazoidal beachfill solution to the Pelnard-Considere Equation. The trapzoidal beachfill solution was solved numerically in Matlab.

Definitions

$hc := 27 \cdot ft$	depth of closure (NGVD)
$B := 9.5 \cdot ft$	berm elevation (NGVD)
$G := 0.014 \cdot \frac{m^2}{s}$	alongshore diffusivity
$Yo := 5 \cdot ft$	initial cross-shore distance between design shoreline and natural shoreline
$Ya := 28.9 \cdot ft$	advance nourishment width
$l := 6600 \cdot ft$	alongshore length of beachfill
$t := 4yr$	time after initial placement
$be := 3 \cdot \frac{ft}{yr}$	background erosion rate
$Yb := 6 \cdot ft$	addditional distance natural shoreline sticks out relative to baseline

Analytical Solution to Pelnard-Considere for Rectangular Beachfill Project

$$M := \left[\frac{\sqrt{4 \cdot G \cdot t}}{l \cdot \sqrt{\pi}} \cdot \left[e^{-\left(\frac{l}{\sqrt{4 \cdot G \cdot t}}\right)^2} - 1\right] + \mathrm{erf}\left(\frac{l}{\sqrt{4G \cdot t}}\right)\right] - \frac{be \cdot t}{(Yo + Ya - Yb)}$$

fraction of initial fill remaining after time t

$M = -0.04$

$re := (1 - M) \cdot \frac{(Yo + Ya - Yb)}{t} = 7.252 \cdot \frac{ft}{yr}$ representative erosion rate

$de := re - be = 4.252 \cdot \frac{ft}{yr}$ diffusive erosion rate

ATTACHMENT F

CONSTRUCTION SCHEDULE

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www.moffattnichol.com

MEMORANDUM

To: Santiago Alfageme, Rob Hampson

From: Adam Isaacson

Date: May 23, 2014

Subject: Downtown Montauk Stabilization Project – Construction Feasibility Schedule

M&N Job No.: 7190-14

Copy: Jack Fink

This memorandum summarizes the assumptions applied in developing a feasibility level construction schedule for the Downtown Montauk Stabilization Project. The project is expected to be given a Notice to Proceed (NTP) on January 2, 2014 and it is the local stakeholders desire to have construction completed in time for the Memorial Day holiday weekend May 22, 2015. The schedule is based on a ten hour work day, seven days a week with the exception of the upland sediment supplier, which is only open for a half day on Saturday and closed on Sundays.

Pumps

Based on discussions with a representative from Maccaferri, each pump and crew is assumed to have a production rate of 8 bags per hour. As stated previously, it is assumed that the construction crew and pumps would be operated 10 hours a day, 7 days a week. Each pump and crew is expected to be able to fill and place 80 geobags per day and 560 geobags per week.

Truck Trips

The stabilization project will require approximately 2,762 truck round trips to deliver the 45,000 cubic yards of sand needed to complete the project. This trip quantity is based on an average sand load per truck of 22 tons or 16.3 cubic yards. The required frequency of truck trips depends on the duration of the job, which is controlled by the number of pumps used. The number of truck trips per ten hour work day needed to complete the job in the three-pump scenario is 5 trucks per hour. The number of truck needed to complete the job in the two-pump scenario is 3.4 trucks per hour.

Upland Sediment Supplier

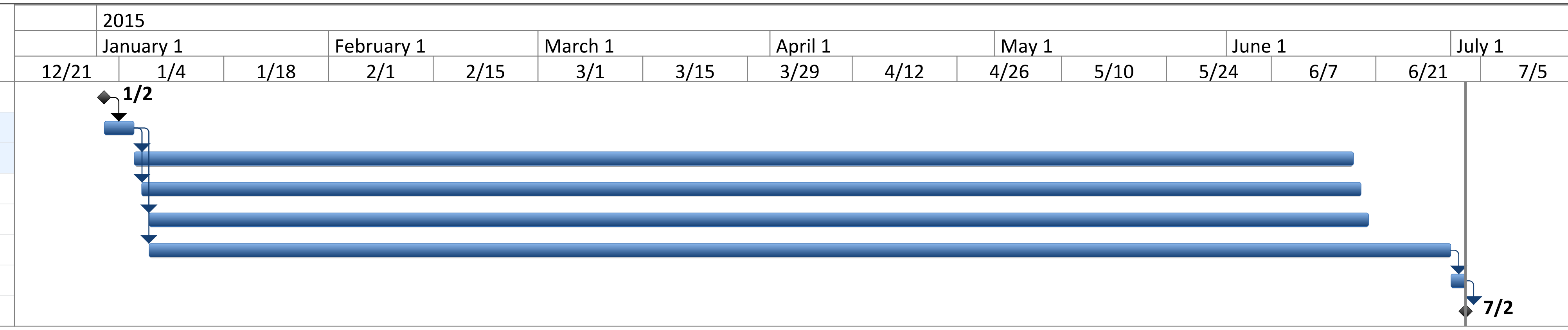
Based on discussions with two local upland suppliers, it is assumed that the upland sediment supplier has the ability to fulfill the needs of this project. It is also assumed that the business hours of the supplier are 7 AM – 4:30 PM Monday-Friday and 7 AM – 12 PM Saturday.

Excavation and Grading

The necessary sequence for the excavators and bulldozers assumed for this schedule are as follows. Excavation will begin two days after sand delivery begins and one day prior to the start of filling and placing geobags. This sequencing will allow a sufficient stockpile to build up and ensure a sufficient trench is ready once geobags begin to be filled and placed. It is assumed that excavated materials will be placed over top of geobags shortly after the placement of the geobags to limit their exposure. Final grading of the dunes will take place intermittently following the replacement of excavated materials. It will also extend one week after the placement of the final geobags to finish the final beach segment and to touch up other areas of the project if necessary.

Montauk Beach Rehabilitation
Construction Feasibility Schedule - 2 Pumps
May 29, 2014

ID	Task Mode	Task Name	Duration	Start	Finish
1		Notice to Proceed (NTP)	0 days	Fri 1/2/15	Fri 1/2/15
2		Mobilization	2 days	Fri 1/2/15	Mon 1/5/15
3		Sand Delivery	117 days	Tue 1/6/15	Wed 6/17/15
4		Trench Excavation	117 days	Wed 1/7/15	Thu 6/18/15
5		Fill and Place Geobags	117 days	Thu 1/8/15	Fri 6/19/15
6		Grading of Sand Over Bags	124 days	Thu 1/8/15	Tue 6/30/15
7		Demobilization	2 days	Wed 7/1/15	Thu 7/2/15
8		Completion	0 days	Thu 7/2/15	Thu 7/2/15

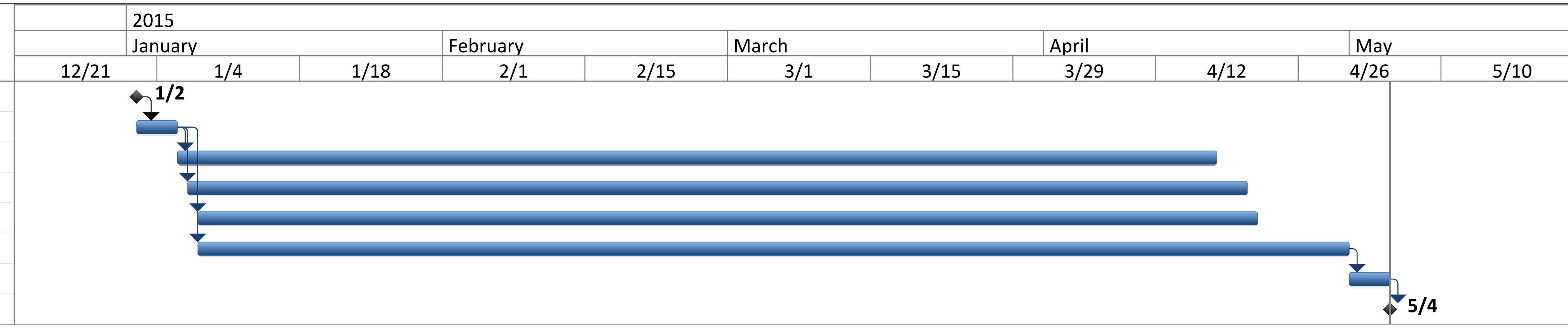


Project: Montauk Feasibility Sche
Date: Thu 5/29/14

Task	Project Summary	Inactive Milestone	Manual Summary Rollup	Deadline
Split	External Tasks	Inactive Summary	Manual Summary	Progress
Milestone	External Milestone	Manual Task	Start-only	
Summary	Inactive Task	Duration-only	Finish-only	

Montauk Beach Rehabilitation
Construction Feasibility Schedule - 3 Pumps
May 29, 2014

ID	Task Mode	Task Name	Duration	Start	Finish
1		Notice to Proceed (NTP)	0 days	Fri 1/2/15	Fri 1/2/15
2		Mobilization	2 days	Fri 1/2/15	Mon 1/5/15
3		Sand Delivery	74 days	Tue 1/6/15	Fri 4/17/15
4		Trench Excavation	74 days	Wed 1/7/15	Mon 4/20/15
5		Fill and Place Geobags	74 days	Thu 1/8/15	Tue 4/21/15
6		Grading of Sand Over Bags	81 days	Thu 1/8/15	Thu 4/30/15
7		Demobilization	2 days	Fri 5/1/15	Mon 5/4/15
8		Completion	0 days	Mon 5/4/15	Mon 5/4/15



Project: Montauk Feasibility Sche
Date: Thu 5/29/14

Task	Project Summary	Inactive Milestone	Manual Summary Rollup	Deadline
Split	External Tasks	Inactive Summary	Manual Summary	Progress
Milestone	External Milestone	Manual Task	Start-only	
Summary	Inactive Task	Duration-only	Finish-only	