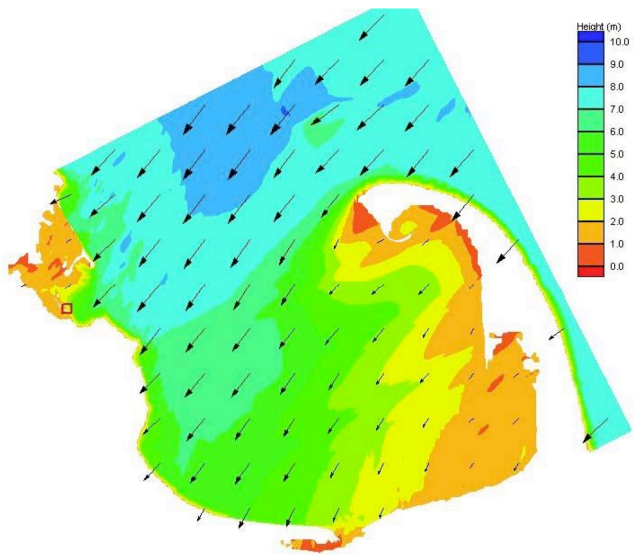
COASTAL ENGINEERING ANALYSIS
SUMMARY REPORT
LONG BEACH, PLYMOUTH, MA

March 29, 2004



OCC Project #203506.1

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EXECUTIVE SUMMARY

The U.S. Army Corps of Engineers (USACOE) and the Town of Plymouth are proposing a reconstruction of an existing dike in conjunction with large-scale beach nourishment at Long Beach in Plymouth, Massachusetts. The stated purpose of the proposed project is to increase storm damage protection for Plymouth Harbor and its navigation channel.

The Goldenrod Foundation have questioned the need for the reconstruction of the dike and beach nourishment and requested that Ocean and Coastal Consultants, Inc. (OCC) perform a Coastal Engineering Analysis of the existing site conditions ("no build alternative) in order to establish a baseline for evaluating alternatives to the proposed dike reconstruction and beach nourishment project. OCC employed a multi-phase modeling approach based upon the existing conditions at Long Beach. The analysis approach included:

- Wind and Wave Buoy Data and Analysis - collecting relevant archival wind and wave data and performing a statistical analysis to determine probable wind speeds and directions, and wave heights and directions for a range of return period storms;
- Wave Modeling – evaluating site specific wave characteristics using the numerical model STWAVE, developed by the USACOE. Offshore wind speeds and wave conditions were used as input into STWAVE to develop the local wave climate for a range of return period storms;
- Runup and Overtopping Analysis - evaluating wave runup and overtopping on the existing stone dike in accordance with methods described in the USACOE *Coastal Engineering Manual*. This analysis was conducted for a range of return period storm conditions; and
- Cross Beach Analysis - simulating beach profile response due to design storms using the numerical model SBEACH, developed by USACOE.

With respect to the "no build" alternative, the following conclusions can be drawn from the results of coastal engineering analysis:

- A massive stone barrier or beach fill is not necessary to limit overtopping rates during infrequent storm events because overtopping of the barrier beach is more a function of water elevation than wave height.
- Damage to the existing stone dike during infrequent storm events is not anticipated to significantly affect overtopping rates as the beach profile is stable and the overtopping is more related to water elevation than wave height.
- It is unlikely that sediment transport from wave overtopping is a significant source of sedimentation in the Plymouth Harbor navigation channel, given the indicated stability of the beach during infrequent storm events. In addition, the project location is south of the main navigation channel, so any overtopping that does occur is unlikely to impact the navigation channel.
- The existing beach is stable under infrequent return period storm conditions. Progressive damage of the dike with additional potential for increased overtopping and beach recession was not indicated by the coastal analysis.
- Breaching of the beach was not indicated given the stability of the beach and existing dike.

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1.0 INTRODUCTION

The U.S. Army Corps of Engineers (USACOE) and the Town of Plymouth are proposing a reconstruction of an existing dike in conjunction with large-scale beach nourishment at Long Beach in Plymouth, Massachusetts (see Photograph 1). The stated purpose of the proposed project is to increase storm damage protection for Plymouth Harbor and its navigation channel.

The Goldenrod Foundation has questioned the need for the reconstruction of the dike and beach nourishment. The Goldenrod Foundation requested that Ocean and Coastal Consultants, Inc. (OCC) perform a Coastal Engineering Analysis of the existing site conditions in order to establish a baseline for evaluating alternatives to the proposed dike reconstruction and beach nourishment project.

Photograph 1: Existing stone dike (view to the north).

1.1 Site Description

The subject site is located on a 2.8 mile long barrier beach that extends northwest from the mainland. Plymouth Long Beach separates Plymouth Harbor, to the west, from Plymouth Bay, to the east. This barrier beach provides essential habitat for state protected rare species, including Piping Plover and the Least Tern, public recreational opportunities and protection to Plymouth Harbor from storm-induced waves. A healthy dune system exists on Long Beach and is concentrated primarily in the northern end. Plymouth Harbor may primarily be characterized as extensive tidal flats, a shallow water environment and fringing salt marshes. Within the harbor is a deepwater boat basin and entrance channel, both of which require frequent maintenance dredging.

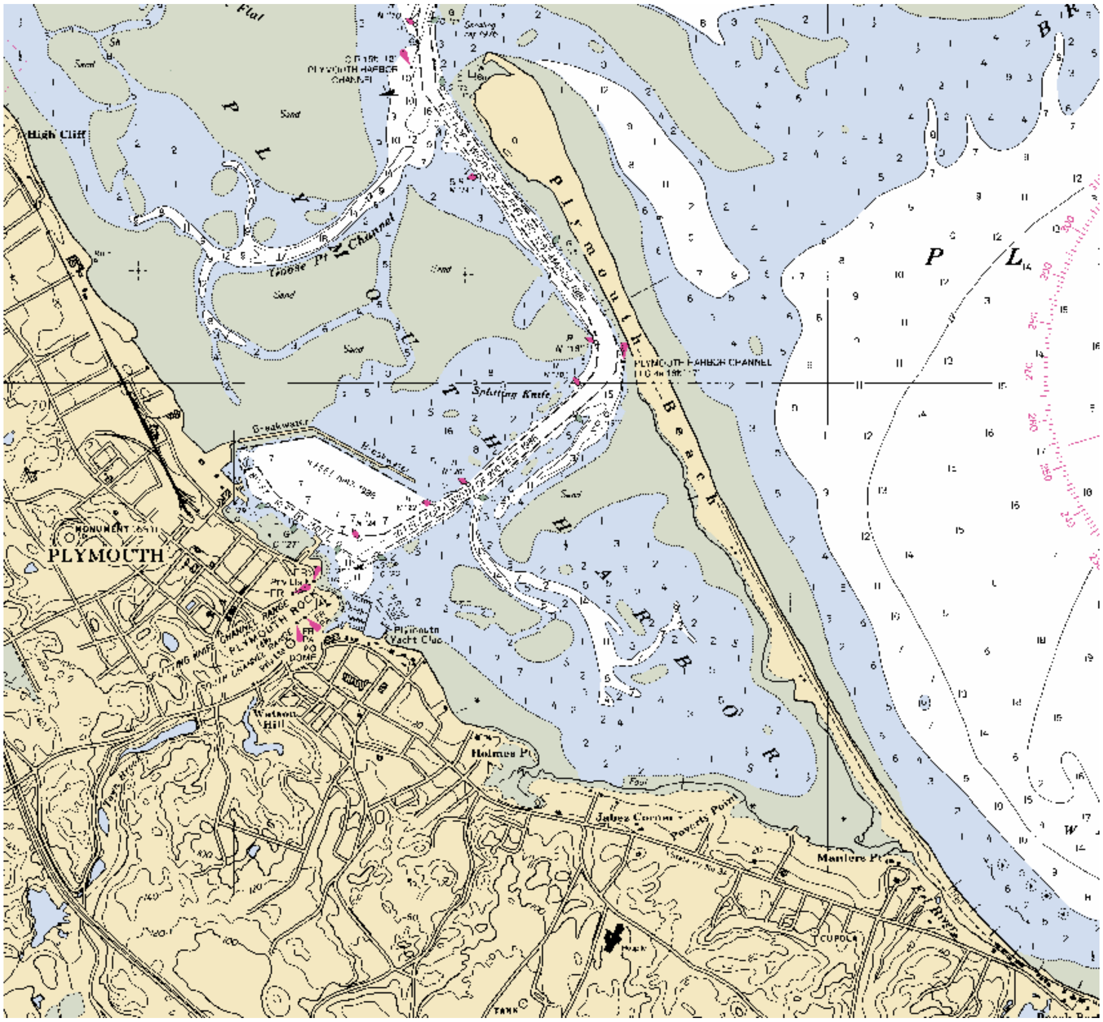

Figure 1.1: National Ocean Service Chart 11309, Plymouth Long Beach.

Historical efforts to protect Long Beach date back to the 1800s. The existing stone dike was built circa 1900 and extends the entire length of the beach. The dike effectively controls the shoreline position, limiting natural shoreline change/progression. It is theorized that the existing dike has contributed to the lowering of the beach seaward of the dike by limiting the availability of landward barrier beach and dune deposits to act as a source of beach material. It is also reported that the loss of the beach seaward of the dike has resulted in reduced storm protection, recreational area and habitat for rare species.

1.2 Proposed Project

The proposed USACOE project includes reconstruction of approximately 2,000 feet of the stone dike to a final elevation of 18 feet MLW and the addition of scour and splash aprons to improve the longevity of the structure. The stated intent of the dike reconstruction is to provide a "last line of defense" against storm waves. In addition, the Town of Plymouth is proposing to place approximately 300,000 cubic yards of sand along 4,500 feet of the beach. The project is expected to create additional beach frontage for recreational use, restore areas of suitable habitat for rare species, and reduce the impacts of storms on the dike.

1.3 Scope of Study

On behalf of the Goldenrod Foundation, Ocean and Coastal Consultants, Inc. (OCC) has performed a coastal engineering evaluation of the existing site conditions to establish a baseline to better assess alternatives for the proposed project. To this end, OCC employed a multi-phase modeling approach based upon the existing conditions at Long Beach. The analysis approach incorporates the latest coastal engineering tools and methods, along with proven modeling techniques and experience with similar sites in order to evaluate the existing beach function. The analysis included:

- Wind and Wave Buoy Data and Analysis - collecting relevant archival wind and wave data and performing a statistical analysis to determine probable wind speeds and directions, and wave heights and directions for a range of return period storms;
- Wave Modeling – evaluating site specific wave characteristics using the numerical model STWAVE, developed by the USACOE. Offshore wind speeds and wave conditions were used as input into STWAVE to develop the wave climate for a range of return period storms;
- Runup and Overtopping Analysis - evaluating wave runup and overtopping on the existing stone dike in accordance with methods described in the USACOE *Coastal Engineering Manual*. This analysis was conducted for a range of return period storm conditions; and
- Cross Beach Analysis - simulating beach profile response due to design storms using the numerical model SBEACH, developed by USACOE.

2.0 HISTORIC WIND AND WAVE DATA ANALYSIS

2.1 Introduction / Summary of Results

Wind generated waves are a major design consideration for coastal engineering structures and beach nourishment projects. The coastal engineer must evaluate site specific wave conditions in order to properly evaluate and design these structures. Statistical models allow the coastal engineer to accurately evaluate offshore wind and wave conditions that can be utilized in conjunction with numerical wave models to determine site specific wave conditions.

Coastal structure design is typically based on wave parameters from infrequent return period storm events, i.e. storm events with a return period of one year or greater. Traditional design has relied on establishing a design return period storm (for example, a 20, 50 or 100-year storm event) and determining the associated wave heights. Waves can arrive at the project site via a variety of mechanisms. For the purposes of this analysis we will consider two (2) mechanisms: swell, which is considered to be a wave that is generated at a distance and travels to the site; and wind waves that are locally generated.

Based on the analysis described below, the following offshore design conditions were utilized in our evaluation of the existing conditions at Long Beach:

Table 2.1
Offshore Design Conditions
Long Beach, Plymouth, Massachusetts

Return Period (years)	Significant Wave Height (feet)		Peak Period (seconds)		Wind Speed (ft/sec)	
Quadrant	Northeast	Southeast	Northeast	Southeast	Northeast	Southeast
1	18.3	9.3	10.1	7.2	73.2	68.8
2	19.8	10.9	10.5	7.8	77.2	79.9
5	21.5	12.1	11.0	8.2	82.5	87.1
10	22.7	12.9	11.3	8.5	86.6	91.9
20	23.8	13.6	11.5	8.7	90.6	96.5
25	24.1	13.8	11.6	8.8	91.9	97.9
50	25.2	14.5	11.9	9.0	95.9	102.4
100	26.2	15.1	12.0	9.2	99.9	106.8

2.2 Technical Approach

Determining design wave conditions typically requires two primary inputs – the water depths where the waves will propagate (bathymetry) and the energy source inputs; wind and/or waves.

Extreme wave heights or wind speeds can be determined from historic wind or wave data by utilizing a statistical analysis such as the peaks-over-threshold method (Goda, 2000, Simiu, et al, 1996). The peaks-over-threshold method utilizes a sampling technique that only considers values in the data set over a certain threshold limit. This method can use a relatively large number of data in a sample, and thus can produce higher confidence in the results. In the peaks-over-threshold analysis, three theoretical distribution functions may provide the best fit to the sample data: Fisher-Tippett Type I (Gumbel) distribution, the Fisher-Tippett Type II (Frechet) distribution, and the Weibull distribution. Each distribution has its own characteristics for modes, means, and standard deviations. The methodology can be used to estimate an expected wave height or wind speed that would occur once in a specified return period.

The peaks-over-threshold methodology is not recommended for determining the wave period associated with the extreme wave height. Instead, wave periods associated with the wave height for a given return period can be calculated based on methods provided by Goda (2000), with the steepness of wind waves, defined as:

$$0.03 \leq H_s/[gT_s^2/2\pi] \leq 0.04 \qquad \text{[EQ. 2.1]}$$

The wave steepness gradually decreases from 0.04 to 0.03 as wind waves grow in the ocean. Based on this range of values, the peak period can be estimated based on an assumed value, or a range of periods can be analyzed.

2.3 Available Data Sources

Historic wind and wave data is available from a variety of public sources including NOAA buoy data and the National Climatic Data Center (NCDC). Specific wind and wave parameters, record lengths and data quality will vary by source and must be evaluated for the intended use on a case by case basis. One convenient source of both wind and wave information are the NOAA data buoys and the National Buoy Data Center (NBDC) – available at: http://www.ndbc.noaa.gov.

OCC researched available sources of wind and wave data and determined that best available wind and wave data source was NOAA WIS II Station 93. WIS II Station 93 provided the most comprehensive historical data that included the specific parameters of interest: wave direction, height, and period along with wind speed and direction. This buoy is located approximately 8 miles to the northeast of the project site (see Figure 2.1), and provides a 40 year record of data. The wave statistics are based on the Wave Information Study (WIS) hindcast data. WIS data is the result of a wave hindcast analysis using historical wind data.

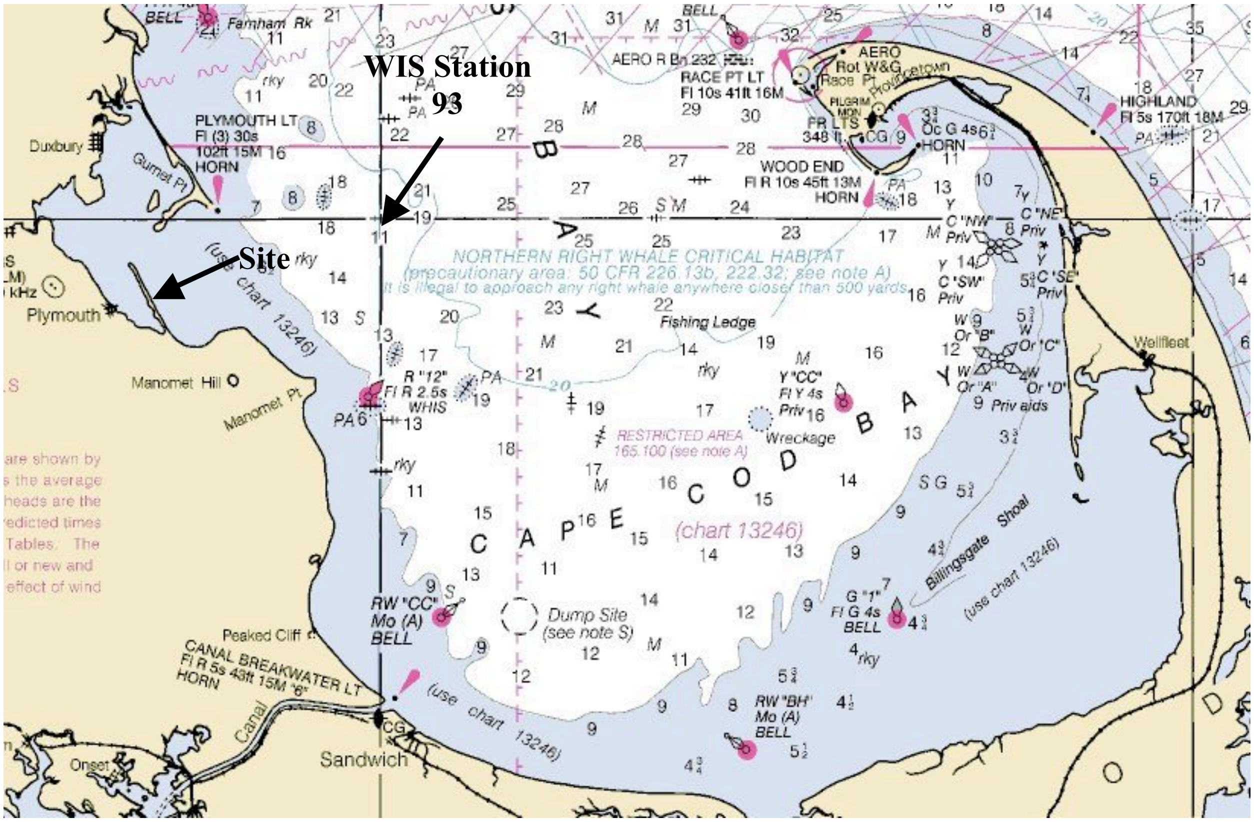

Figure 2.1: NOAA Chart 13200 depicting location of site and WIS Station 93.

The data set used for this analysis was for the 40 year period from 1956 to 1995 and included the effects of hurricanes. The wave data is provided as spectrally based significant wave height (in meters), corresponding peak period (in seconds) and direction (in degrees from true north). The data also provides wind speed (m/s) and direction. Data is provided as an averaged value for a 3 hour duration over the entire period of record. This results in a data set of 116880 values for each parameter.

2.4 Project Specific Analysis

For the purposes of this study, two (2) primary directions of wave approach were identified. The first is from the northeast quadrant (extending from 10 degrees to 70 degrees True North). Waves from this quadrant would generally be representative of winter storm and Nor'easter conditions. The second direction of wave approach is from the southeast quadrant. Waves for this quadrant (extending from 70 degrees to 130 degrees True North) would generally represent summer storms and potentially Hurricanes. Wind and wave data was sorted into three bins: one for the northeast quadrant; one for the southeast quadrant; and one for the remaining directions. Wind and waves from the northwest and southwest would not be traveling in the direction of the project site and were not considered in the statistical analysis.

For both the northeast and southeast quadrants, wind and wave data from WIS II Station 93 were analyzed using the "peaks-over-threshold" method. The following table summarizes the analysis:

Table 2.2
Wind and Wave Data Statistical Analysis
WIS II Buoy 93

Quadrant	Parameter	Threshold Value	Number of Samples	Best-fit Distribution	Correlation Coefficient
Northeast	Wave Height	16.4 ft	84	Weibull	0.990
	Wind Speed	72.2 ft/sec	55	Weibull	0.981
Southeast	Wave Height	9.8 ft	39	Weibull	0.984
	Wind Speed	72.2 ft/sec	38	Gumbel	0.970

Figure 2.2 shows a representative "peaks-over-threshold" calculation.

Once the best-fit distribution for each parameter was determined, parameters for a given return period can be calculated from the selected probability distribution.

As previously noted, the wave period was calculated based on Equation 2.1, with an assumed wave steepness of 0.035.

The results of the wind and wave statistical analysis are shown in Table 2.1.

2.5 References

Goda, Y. (2000). "Statistical Analysis of Extreme Waves", in Random Seas and the Design of Maritime Structures; Singapore: World Scientific Publishing C. Pte. Ltd.

Marrone, J.F. (2004). "Use of Numerical and Statistical Wave Models in the Structural Design and Operational Analysis of Harbor and Near-shore Structures", Ports 2004 Conference Proceedings, American Society of Civil Engineers (in publication).

Suniu, E., N. A. Heckert, T. Whalen. (1996). "Estimates of Hurricane Wind Speeds by the 'Peaks over Threshold' Method." National Institute of Standards and Technology Technical Note 1416.

U.S. Army Corps of Engineers. (1984). *Shore Protection Manual.*U.S. Army Corps of Engineers, Washington, D.C.

U.S. Army Corps of Engineers. (2002). *Coastal Engineering Manual*. Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. (in 6 volumes).

Weibull

$k := 1$ — Shape Parameter - Use 0.75, 1.0, 1.4 or 2.0 to obtain best correlation coeffecient. (Equation 11.15)

$\alpha := 0.20 + \frac{0.27}{\sqrt{k}}$ $\alpha = 0.47$ $\beta := 0.20 + \frac{0.23}{\sqrt{k}}$ $\beta = 0.43$ — Constants of unbiased plotting position formula. (Table 11.2)

$F := 1 - \frac{m - \alpha}{N_T - \beta}$ — Unbiased plotting position formula. (Equation 11.14)

$y := (-\ln(1 - F))^{\frac{1}{k}}$ — Reduced variate. (Equation 11.16)

$B := \mathrm{intercept}(y,x)$ $B = 21.758$ — Best-fit slope of Reduced variate vs. Extreme variate. (Scale Parameter)

$A := \mathrm{slope}(y,x)$ $A = 1.763$ — Best-fit intercept of Reduced variate vs. Extreme variate. (Shape Parameter).

$r := \mathrm{corr}(y,x)$ $r = 0.981$ $r^2 = 0.963$ — Correlation of best-fit line.

$\mathrm{Best\_fit} := B + A \cdot y$ — Equation of best-fit line. (Equation 11.17)

$R := \begin{pmatrix} 1 \\ 2 \\ 5 \\ 10 \\ 20 \\ 25 \\ 50 \\ 100 \end{pmatrix} \mathrm{yr}$ Return Period.

$y_R := (\ln(\lambda \cdot R))^{\frac{1}{k}}$ (Equation 11.24)

$x_R := B + A \cdot y_R$ Equation for Return Value (Equation 11.23)

$x_R = \begin{pmatrix} 22.3 \\ 23.5 \\ 25.2 \\ 26.4 \\ 27.6 \\ 28 \\ 29.2 \\ 30.4 \end{pmatrix}$ Return value (m/s).

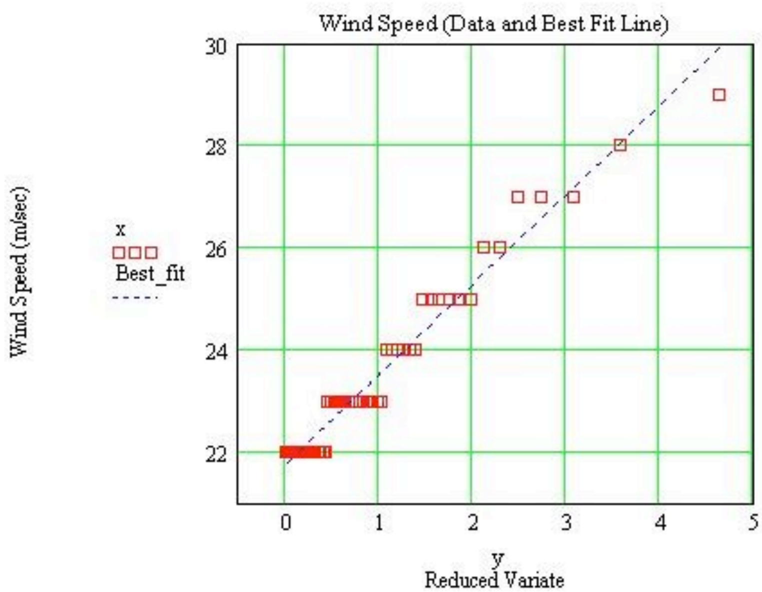

Figure 2.2: "Peaks–over-threshold" calculation for wind speed using Weibull method.

3.0 WAVE MODELING

3.1 Introduction / Summary of Results

OCC developed the site specific wave climate at Long Beach in order to perform a coastal engineering analysis and determine the response of the beach and existing structures in the study area assuming existing, or no-build, conditions. The site specific wave characteristics were determined using the numerical model STWAVE, developed by the United States Army Corps of Engineers (USACOE). STWAVE is a nearshore wave transformation model that simulates depth-induced refraction and shoaling, depth and steepness-induced wave breaking and wind-wave growth. The purpose of applying nearshore wave transformation models is to describe the change in wave parameters (wave height, period, direction, and spectral shape) between the offshore and the nearshore environments. Deep water wave fields are fairly uniform, but in the nearshore, waves are strongly influenced by variations in bathymetry, water level, currents, and coastal structures. The design wind speeds and wave heights from the WIS data, as described in Section 2.0, were used as input into STWAVE to develop the site specific wave climate for a range of return period storms. The results of the STWAVE modeling are shown in Table 3.1 below.

Table 3.1
STWAVE Model Results

	Northeast Quadrant			Southeast Quadrant		
Storm Recurrence Interval (years)	Significant Wave Height (feet)	Peak Period (seconds)	Direction*	Significant Wave Height (feet)	Peak Period (seconds)	Direction*
1	4.8	11.1	61	4.5	9.1	73
2	5.1	11.1	61	4.8	10.0	74
5	5.4	11.1	61	5.2	10.0	74
10	5.8	12.5	61	5.5	10.0	74
20	6.0	12.5	61	5.7	11.1	74
25	6.1	12.5	61	5.8	10.0	75
50	6.4	12.5	61	6.1	11.1	75
100	6.6	12.5	61	6.3	11.1	75

* - direction of wave approach relative to true north

The results indicate that the northeast is the dominant direction and that the significant wave height has a narrow range from 4.8 to 6.6 feet (1.5 to 2.0 meters) for the storm recurrence intervals of interest. The results of the STWAVE analysis were used as the basis for the overtopping analysis of the existing stone dike at Long Beach, which is further described in Section 4.0 and the cross beach analysis described in Section 5.0.

3.2 Technical Approach

The STeady-state Spectral WAVE model (STWAVE) was developed by the United States Army Corps of Engineers to predict nearshore wind-wave growth and propagation. STWAVE is a finite difference model based on the wave action balance equation. STWAVE is typically applied to determine the change in wave parameters (wave height, period, direction, and spectral shape) between the offshore and the nearshore environments. Deep water wave fields are fairly uniform, but in the nearshore, waves are strongly influenced by variations in bathymetry, water level, currents, and coastal structures. STWAVE simulates depth and current-induced wave refraction and shoaling, depth- and steepness-induced wave breaking, diffraction, wind-wave growth, and wave-wave interaction that redistributes and dissipates energy in a growing wave field. The Surface-Water Modeling System (SMS) provides a user interface for grid generation, generation of input spectra, and visualization of model output.

Basic Equation

The governing wave dispersion relationship is given in the moving reference frame (relative to the current) as

$$\varpi_r^{\,2} = gk \tanh kd \qquad \text{[EQ. 3.1]}$$

where:

ω = angular frequency
g = gravitational acceleration
k = wave number
d = water depth.

In the absolute frame of reference, the dispersion equation is

$$\omega_a = \omega_r + kU \cos(\delta - \alpha) \qquad \text{[EQ. 3.2]}$$

where:

U = current speed
δ = direction of current relative to a reference frame
α = wave orthogonal direction (normal to wave crest)

The wave number is solved for by substituting equation 2 into equation 3 and iteratively solving for k.

Solutions for refraction and shoaling also require wave celerities, C, and group celerities, Cg, in both reference frames. After several substitutions, the governing equation for steady-state conservation of spectral wave action along a wave ray (direction of energy propagation) is given by:

$$\left(C_{ga}\right)_i \frac{\partial}{\partial x_i} \frac{C_a C_{ga} \cos(\mu - \alpha) E}{\omega_r} = \Sigma \frac{S}{\omega_r} \quad \text{[EQ. 3.3]}$$

where:

E = wave energy density spectrum (which is a function of absolute angular frequency ω_a and direction θ)
S = energy source and sink terms.

Additional parameters and substitutions are implemented into the dispersion and conservation of spectral energy equations to account for refraction and shoaling, diffraction, surf-zone wave breaking, wind input, wave-wave interaction and white capping.

Boundary Conditions

The STWAVE model is formulated on a Cartesian grid. Grid cells are square and variable grid resolution can be obtained by nesting model runs. STWAVE operates on a local coordinate system, with the x-axis oriented in the cross-shore direction and the y-axis located along shore. The orientation of the x-axis defines the half plane that is represented in the model. The y-axis is usually aligned with the bottom contours. Lateral boundaries in the model can be specified as land or water. If the boundaries are specified as water, a zero-gradient type of boundary is applied that allows energy to propagate into or out of the domain along the lateral boundary.

Assumptions

Mild bottom slope and negligible wave reflection
STWAVE assumes a mild bottom slope and negligible wave reflection. Wave energy can propagate only from the offshore toward the nearshore and waves reflected from the shoreline travel in directions outside the model boundary and thus are neglected.

Spatially homogeneous offshore wave conditions
The input spectrum is constant along the offshore boundary.

Steady-state waves, currents, and winds
Waves and currents remain constant over the length of the domain. Winds are assumed to have remained steady sufficiently long for the waves to attain fetch-limited or fully developed conditions.

Linear refraction and shoaling.
STWAVE incorporates only linear wave refraction and shoaling, thus does not represent wave irregularity.

Depth-uniform current
The wave-current interaction in the model is based on a current that is constant through the water column.

Bottom friction is neglected
Propagation distances in nearshore models are relatively short, so the cumulative bottom friction dissipation is small.

Input

STWAVE has four input files. These files specify model parameters, bathymetry, incident wave spectra, and current fields (optional). The input files are generated using the SMS interface. Incident two-dimensional wave spectra are specified as energy density as a function of frequency and direction. A single input spectrum is applied along the entire offshore boundary of the STWAVE grid. The offshore incident wave height, period, and direction are input into the model and the spectrum is generated by the number of frequency bins (typically 20 -30 bins with the peak frequency within the lower one-third of the range). The wind speed can also be specified in combination with the source terms or alone for wind wave generation only. A water-elevation correction for tides or storm surge can also be added to the input file.

Output

STWAVE has three output files containing wave spectra at selected output points, wave height, period, and direction at the selected monitoring stations; and fields of wave height, period, and direction over the entire modeling domain. The results are given as deepwater significant wave height, peak periods, and mean direction. The energy densities are given in units of meters/hertz/radian. The results can be viewed graphically in SMS.

3.3 Available Data Sources

STWAVE requires two primary inputs – the water depths where the waves will propagate (bathymetry) and the energy source inputs (wind and/or waves). The bathymetry was obtained from the National Geophysical Data Center (NGDC) Hydrographic Survey Database. The elevations found in this database are in meters relative to the National Geodetic Vertical Datum of 1929 (NGVD 29). Water depths were modified to account for the still water surface elevations for the range of storm return periods. The water surface elevations for 1, 10, 50 and 100-year storms were obtained from the U.S. Army Corps of Engineers New England Coastline Tidal Flood Survey for Plymouth Bay, MA. Additional water levels were calculated using the equation for the trendline of the plotted data. Table 3.2 presents the additional water depth that was added to the NGDC bathymetry for each storm.

Table 3.2
Storm Still Water Elevations

Storm Recurrence Interval (years)	Still Water Level (ft NGVD)
1	7.4
2	7.9
5	8.5
10	9.0
20	9.3
25	9.5
50	9.9
100	10.3

Both wind-generated and propagated waves are of concern for the project site. The wind and wave data obtained from the statistical analysis of WIS Station 93 were used as the input at the offshore boundary of the STWAVE model to determine the wind-wave growth in Plymouth Bay (see Section 2.0 of this report).

3.4 Project Specific Analysis

One (1) grid was developed to model the project site in Plymouth Bay for each of the storm recurrence intervals of interest. Models were run for two primary directions: waves traveling from the northeast (Atlantic Ocean); and waves traveling from the southeast (within Cape Cod Bay). As previously discussed, the bathymetry was obtained from the National Geophysical Data Center (NGDC) Hydrographic Survey Database. The shorelines were traced into the model using NOAA charts for Plymouth and Cape Cod Bay.

The wind data from the statistical analysis provided a 3-hour wind speed duration, which was converted into the 1-hour wind speed duration using methods from the USACOE *Coastal Engineering Manual* in order to simulate the wind blowing from approximately the same direction and at approximately the same velocity over a 1-hour period. It should be noted that short duration gales will not generate the design wave conditions. Table 3.3 provides the design wind speeds, converted to the one-hour duration, and the significant wave heights and peak periods obtained from the statistical analysis of the WIS data which were used as the input into the models to simulate wind-wave growth and propagation from the offshore boundary. Monitoring stations were set at the project site in the model to record the site specific significant wave height, peak period, and mean wave direction.

Table 3.3
STWAVE Input Parameters

	Northeast Quadrant			Southeast Quadrant		
Storm Recurrence Interval (years)	Wind Speed 1-Hr Duration (m/s)	Significant Wave Height (meters)	Peak Period (seconds)	Wind Speed 1-Hr Duration (m/s)	Significant Wave Height (meters)	Peak Period (seconds)
1	24.0	5.6	10.1	22.8	2.8	7.2
2	25.3	6.0	10.5	26.1	3.3	7.8
5	27.2	6.6	11.0	28.7	3.7	8.2
10	28.5	6.9	11.3	30.4	3.9	8.5
20	29.7	7.3	11.5	31.9	4.1	8.7
25	30.2	7.3	11.6	32.3	4.2	8.8
50	31.5	7.7	11.9	33.7	4.4	9.0
100	32.8	8.0	12.1	35.1	4.6	9.2

3.5 Results

The results from the numerical model simulations are shown graphically on the attached plots and summarized in Table 3.1. It was determined that the wave heights from the southeast direction would not govern the coastal engineering analysis. As such, the results from this direction were not considered for further evaluation. Figures 3.1 and 3.2 below show typical results for the northeast wave approach during the 50-year storm return period for wave height and period, respectively. Figures 3.3 and 3.4 show typical results for the southeast wave approach during the 50-year storm return period for wave height and period, respectively.

The STWAVE results are presented in terms of deepwater significant wave height (Hs) and spectral peak period (Tp). The peak period is the wave period that corresponds to the maximum energy of the wave spectrum.

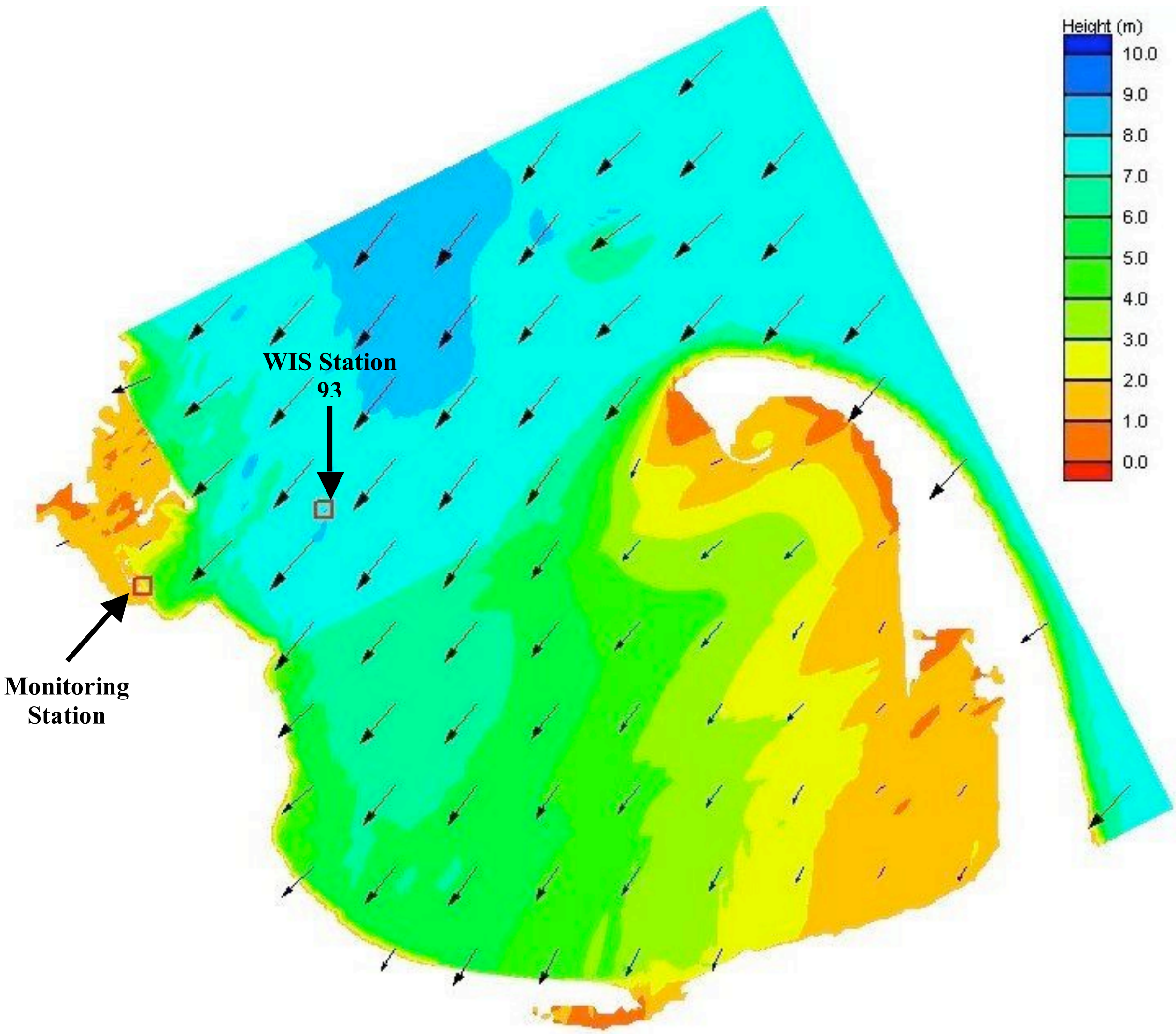

Figure 3.1: STWAVE output for wave height: 50-year storm from northeast direction.

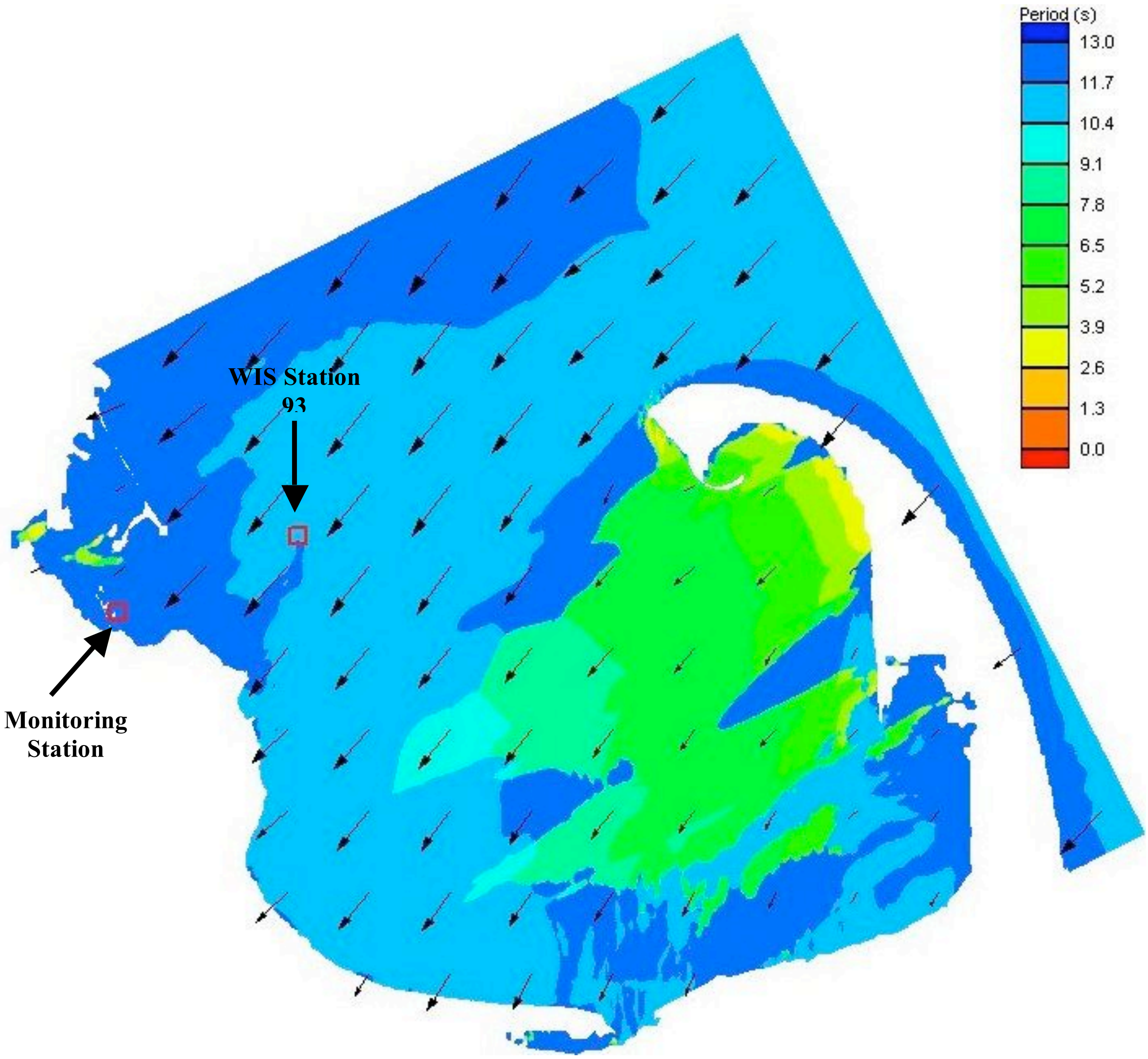

Figure 3.2: STWAVE output for period: 50-year storm from northeast direction.

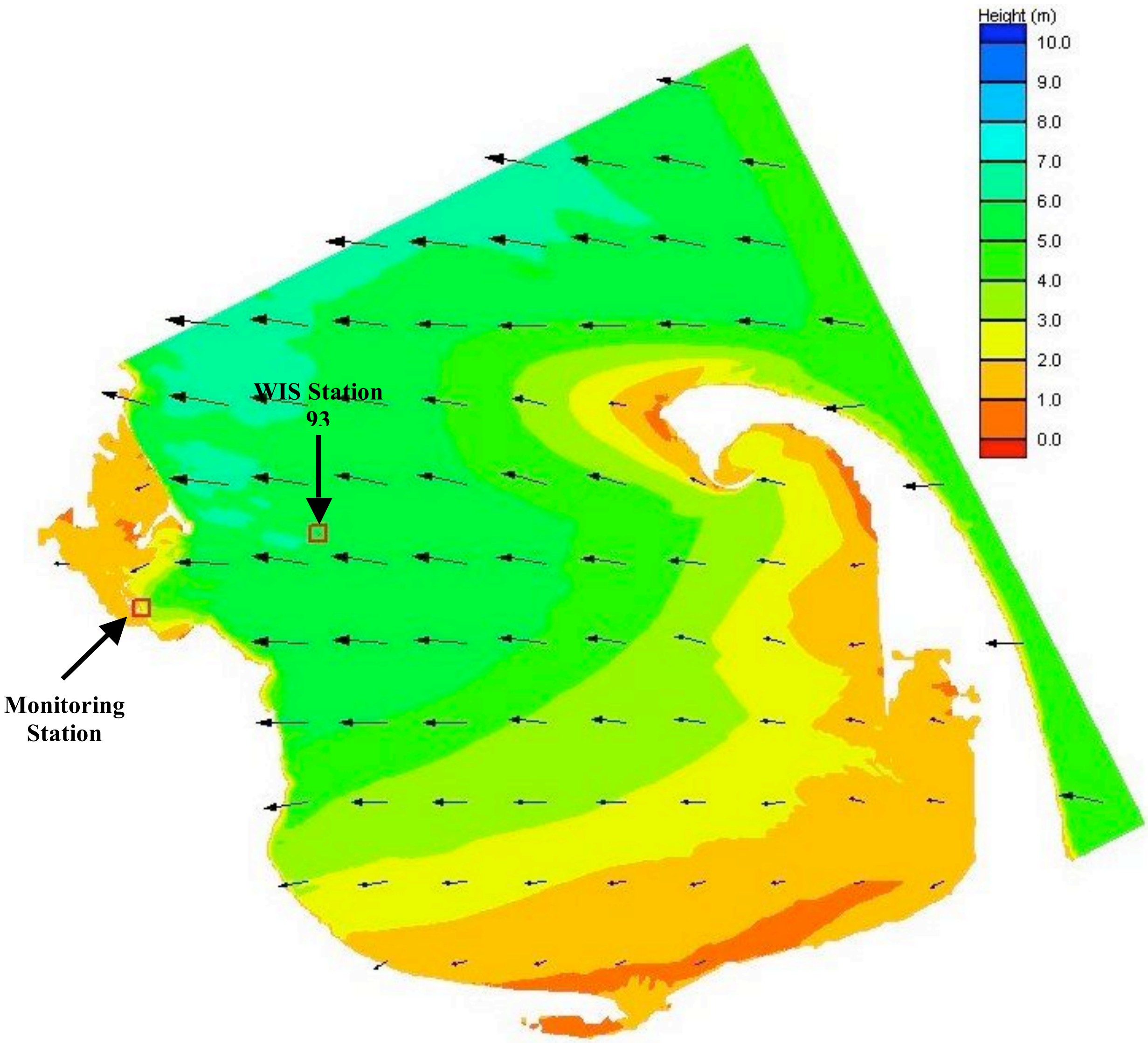

Figure 3.3: STWAVE output for wave height: 50-year storm from southeast direction.

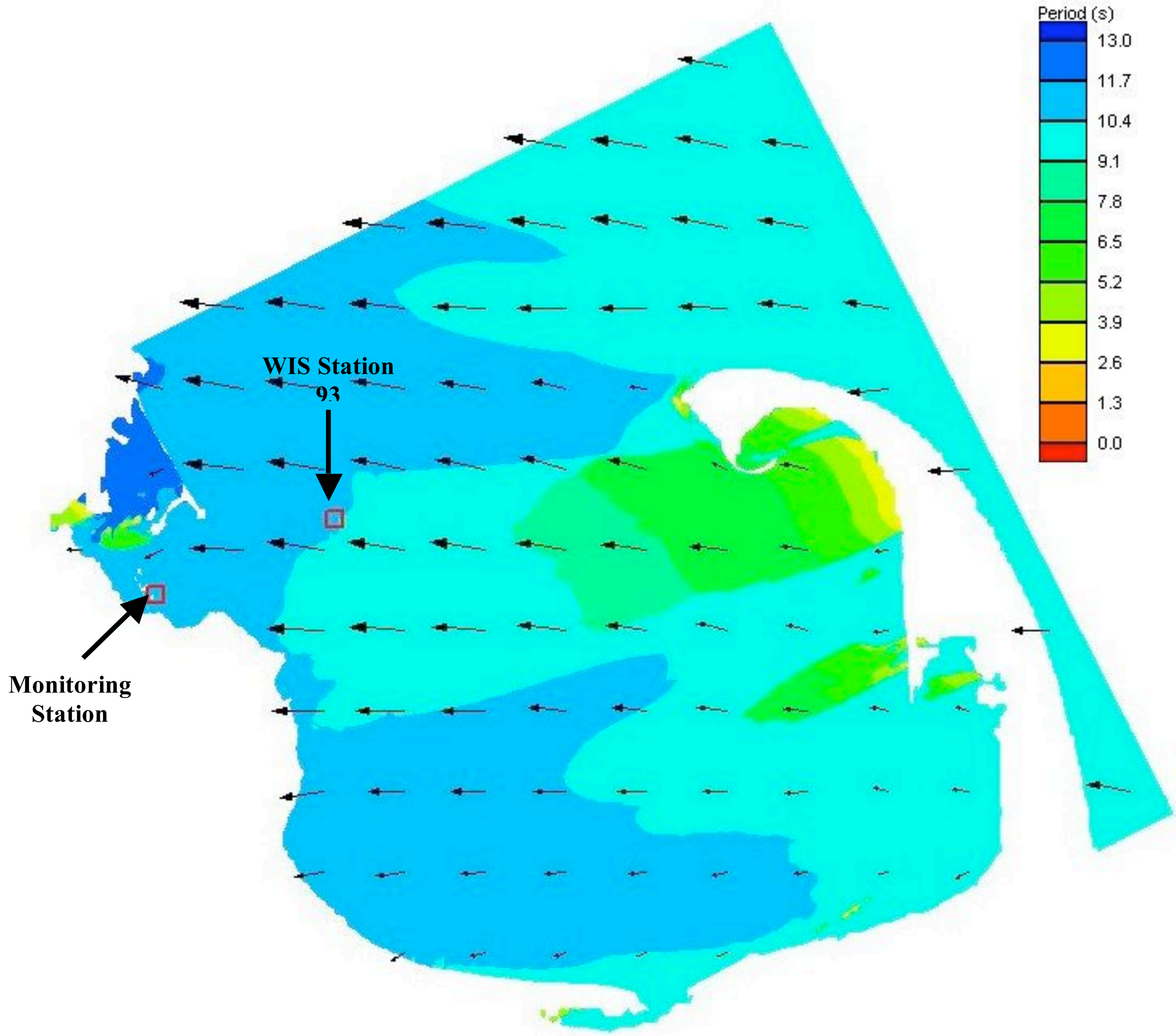

Figure 3.4: STWAVE output for wave period: 50-year storm from southeast direction.

3.6 References

Smith, Jane McKee, Resion, Donald T., Zundel, Alan K. (1999). *STWAVE: Steady-state spectral wave model. Report 1, User's Manual for STWAVE Version 2.0*, Instruction Report CHL-99-1. U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, MS.

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4.0 RUNUP / OVERTOPPING ANALYSIS

4.1 Introduction / Summary of Results

The Long Beach Stone Dike Reconstruction and Beach Nourishment project includes the substantial reconstruction of the existing stone dike. In general, design of coastal structures such as dikes, requires an evaluation of acceptable levels of wave runup and overtopping. According to the USACOE *Coastal Engineering Manual*, wave runup level is one of the most important factors affecting the design of coastal structures since it ultimately determines the crest elevation of the structure in cases where no (or only marginal) overtopping is acceptable. In order to compare project alternatives, it is useful to determine the amount of overtopping occurring on the existing structure.

Briefly stated, wave runup is the uprush of water from wave action on a shore barrier measured from the stillwater level. Wave overtopping occurs when the structure crest is below the runup level. Allowable amounts of overtopping, or overtopping discharge, is a valuable parameter since it allows the engineer to define the required height of a coastal structure. Design levels of overtopping discharges vary depending on the anticipated function of the particular structure. Certain functions put restrictions on the allowable overtopping discharge. For example, a structure fronting access roads and buildings should allow for less overtopping than a structure designed to dissipate wave energy.

The rate of overtopping along the dike at Plymouth Long Beach was evaluated using methods described by van der Meer and Janssen (1995). Four (4) cross-sections of the existing structure were evaluated for overtopping discharge during return interval storms of interest.

The results of the analysis indicate that overtopping of the existing dike will occur for all design storms over the entire length of the existing jetty. However, the runup and overtopping analysis generally indicates that the levels of overtopping, occurring during any given return period, are less along the northern portion of Plymouth Long Beach than along the southern extent. This trend is attributable to the increased structural crest elevation and gentler structural slope along the northern reaches of the dike. Some of the overtopping rates exceed empirical thresholds for structural damage to the dike. There is a rapid increase in overtopping rates with design storm level. However, as noted in Section 3.0, there is a limited increase in storm wave height with increasing design storm level. As such, it is concluded that the increase in overtopping rate with design storm return interval is strongly influenced by the return period water elevation. Results of the runup and overtopping analysis are shown in Figures 4.1 through 4.4.

4.2 Technical Approach

Formulas for determining the average overtopping are generally based upon hydraulic model test results for specific breakwater geometries. Due to the empirical nature of these methods, it is essential to select the method which most accurately reflects the existing conditions. The existing dike on Plymouth Long Beach is a straight-sloped structure with little to no observable berm. With this geometry in mind, OCC selected the van der Meer and Janssen (1995) formula since it is a result of overtopping studies conducted on straight and bermed slopes. It should be noted that the study conducted to develop this method also considered slopes which were impermeable. The existing dike on Plymouth Long Beach is a permeable structure. Although the existing structure does not completely match the van der Meer and Janssen methodology, the results will still be applicable for comparative purposes.

Basic Equation

The van der Meer and Janssen method for determining average overtopping is dependent upon the surf similarity parameter, ξ_{op}. The surf similarity parameter can be determined as:

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{s_{op}}} \qquad \text{[EQ. 4.1]}$$

where:

α = structure slope
s_{op} = deepwater wave steepness

For cases where $\xi_{op} > 2$, van der Meer and Janssen formula is:

$$\frac{q}{\sqrt{gH_s^{\,3}}}\sqrt{\frac{s_{op}}{\tan\alpha}} = 0.006\exp\left(-5.2\frac{R_c}{H_s}\frac{\sqrt{s_{op}}}{\tan\alpha}\frac{1}{\gamma_r\gamma_b\gamma_h\gamma_b}\right) \qquad \text{[EQ. 4.2]}$$

where:

q = average overtopping rate
g = gravitational constant
H_s = significant wave height
R_c = freeboard
γ_r = surface roughness reduction factor (Table VI-5-3, *Coastal Engineering Manual*)
γ_b = berm influence reduction factor (Eq. VI-5-8, *Coastal Engineering Manual*)
γ_h = shallow-water reduction factor (Eq. VI-5-10, *Coastal Engineering Manual*)
γ_β = wave angle influence reduction factor (Eq. VI-5-26, *Coastal Engineering Manual*)

For cases where $\xi_{op} < 2$, the formula is:

$$\frac{q}{\sqrt{gH_s^{\,3}}} = 0.2\exp\left(-2.6\frac{R_c}{H_s}\frac{1}{\gamma_r\gamma_b\gamma_h\gamma_\beta}\right) \qquad \text{[EQ. 4.3]}$$

As can be seen from the above equations, the van der Meer and Janssen method takes into account the reduction in overtopping due to slope surface roughness, presence of a berm, influence of shallow water, and the angle of wave incidence by incorporating the respective reduction coefficients. This method requires that any combination of the reduction factors may not be less than 0.5.

4.3 Available Data Sources

The van der Meer and Janssen method for evaluating overtopping of a structure requires input parameters based upon two primary factors – the incoming wave and the subject coastal structure. Information regarding the incoming wave climate was collected from the STWAVE model described previously. Wave data for the 1-, 2-, 5-, 10-, 20-, 25-, 50-, and 100-year return periods were considered during this investigation.

Structural information was collected from the Plymouth Long Beach Nourishment Notice of Intent (NOI), prepared by Applied Coastal Research and Engineering, Inc., dated September 3, 2003. This report contains plans prepared by Sullivan Engineering, Inc., dated August 14, 2003, which reflect nine cross-sections along the existing dike. For the purposes of this investigation, OCC selected four (4) cross-sections labeled in the NOI as 50+00, 60+00, 67+50, and 71+00.

4.4 Project Specific Analysis

Using van der Meer and Janssen's method, OCC determined the average overtopping rate, q, at each of the four (4) cross-sections along Plymouth Long Beach for each of the storm recurrence intervals of interest.

In addition to the data described above, OCC made a few assumptions regarding the overtopping reduction factors at the subject site. Presented below is a summary of those assumptions.

<u>Surface Roughness Reduction Factor</u>

The values for surface roughness reduction described in the *Coastal Engineering Manual*, are valid when $1 < \xi_{op} < 3\text{-}4$. This assumption is valid for all cases considered in this investigation. Assumed reduction factors are presented in Table VI-5-3 of the *Coastal Engineering Manual*, for several types of structures. It is assumed that the existing dike on Plymouth Long Beach consists of one (1) layer of rock and the ratio of rock diameter to significant wave height is between 1.5 and 3.0. From Table VI-5-3, the roughness reduction factor for this type of structure ranges from 0.55 – 0.60. To be conservative, a surface roughness reduction factor was applied for all cases.

<u>Berm Influence Reduction Factor</u>

Delft Hydraulic has conducted testing to determine the effects of a berm on runup and overtopping. From this testing de Waal and van der Meer were able to develop a formulation for the reduction of overtopping due to the berm. This equation is based in part on the ratio of the equivalent slope for the berm and the slope of the structure. For cases where a berm is not present, this ratio will go to unity resulting in a reduction factor of 1. Since the existing dike on Plymouth Long Beach is not fronted by a berm, the berm influence reduction factor is assumed to be 1.

Wave-Angle Influence Reduction Factor

The angle of incidence affects the amount of runup and overtopping at a structure. The influence of wave-angle on overtopping can be determined from Equation VI-5-11 in the *Coastal Engineering Manual*. Since the current investigation considers waves approaching at an incidence angle of -2º (as determined by STWAVE) for all return intervals, the wave angle reduction factor is assumed to be 1.

4.5 Results

The average overtopping rates during each of the storm recurrence intervals at the four (4) cross-sections were determined by OCC. These results are presented in Figures 4.1 through 4.4.

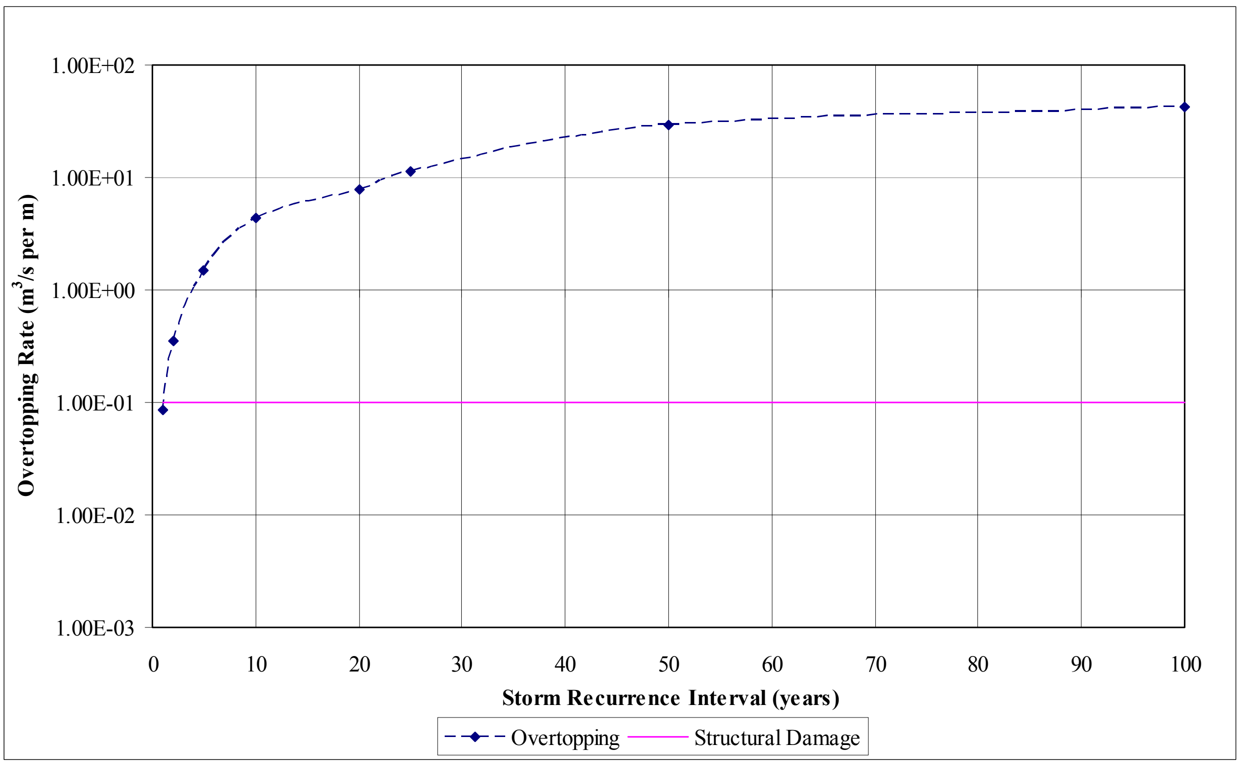

Figure 4.1: Overtopping Rate vs. Storm Recurrence Interval at Station 50+00.

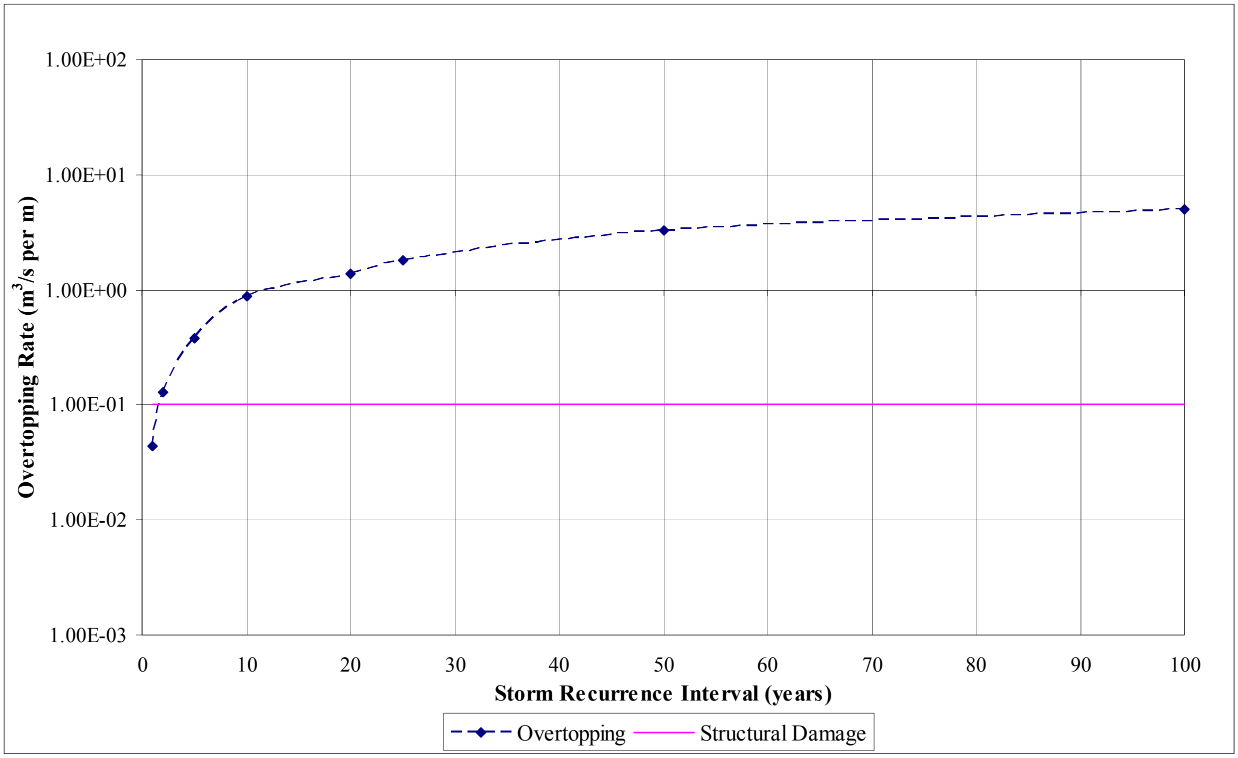

Figure 4.2: Overtopping Rate vs. Storm Recurrence Interval at Station 60+00.

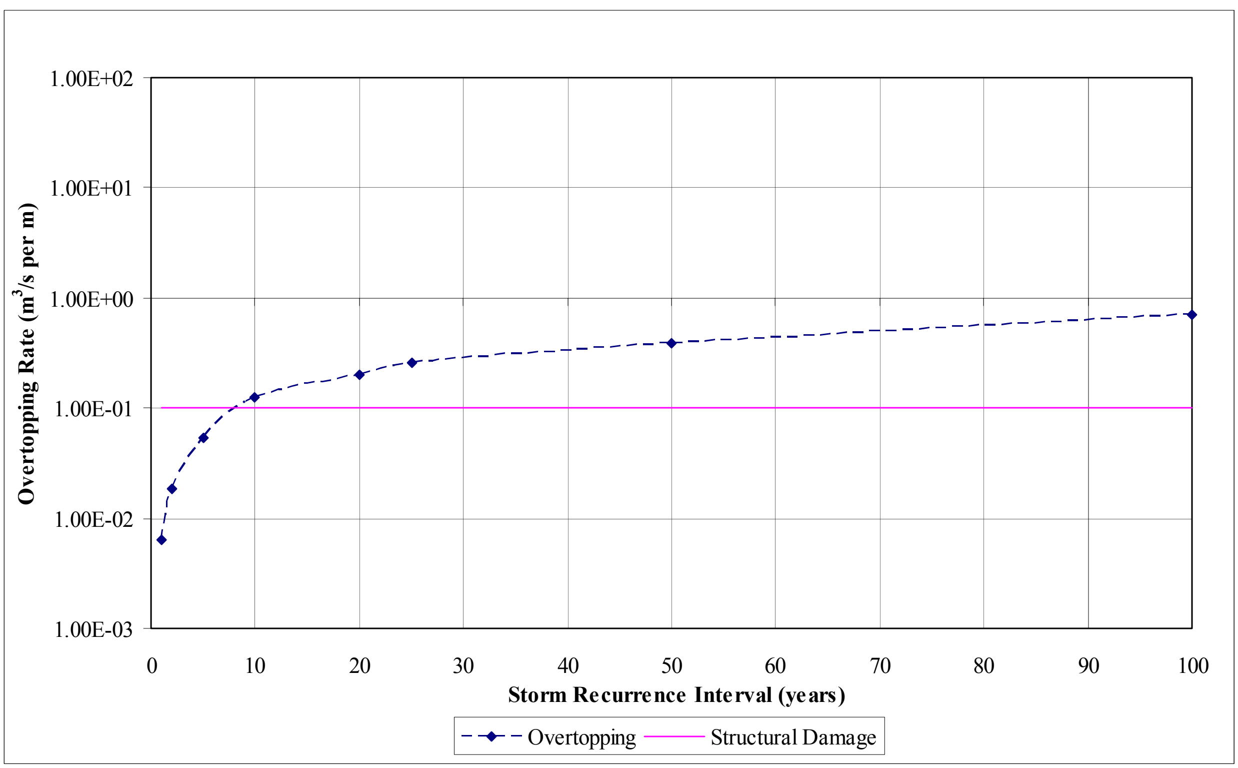

Figure 4.3: Overtopping Rate vs. Storm Recurrence Interval at Station 67+50.

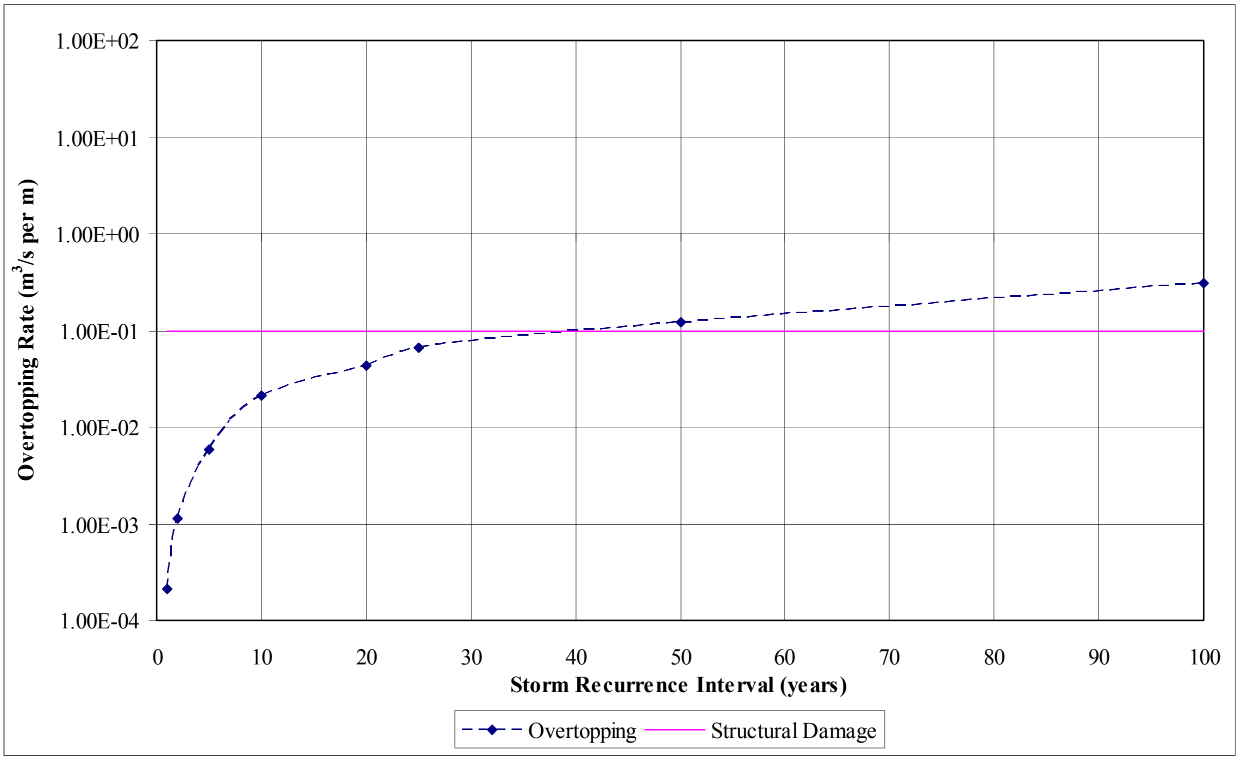

Figure 4.4: Overtopping Rates vs. Storm Recurrence Interval at Station 71+00.

It is important to note that the overtopping discharge from wind-generated waves is intermittent and isolated, being confined to some portion of occasional wave crests at scattered locations. Several field studies have been conducted to define critical values of q for considerations such as pedestrian and vehicular traffic, and structural safety. The results of these studies are presented in Table VI-5-6 of the *Coastal Engineering Manual.* The critical values shown in that table vary from 10^{-7} $m^3/s/m$ to 10 $m^3/s/m$. These values are regarded as rough guidelines since the overtopping discharge varies depending upon time and space. In addition, the intensity of water hitting a specific location is dependent on the geometry of the structure and the distance from the front of the structure. The maximum intensities might locally be up to two orders of magnitude larger than the overall value of q. Further, the determination of an acceptable amount of overtopping is subjective to personal opinion and experience. For the purposes of this investigation, two (2) critical overtopping rates were identified: allowable overtopping for vehicular traffic (10^{-5} $m^3/s/m$) and allowable overtopping before damage to the coastal structure occurs (10^{-1} $m^3/s/m$). For comparison, the allowable overtopping for stability of a coastal structure is included on each of the figures.

Generally, the levels of overtopping, occurring during any given return period, are less along the northern portion of Plymouth Long Beach than along the southern extent. This trend is attributable to the increased structural crest elevation and gentler structural slope along the northern reaches of the dike. However, results of the analysis indicate that the aforementioned overtopping limits for safety of vehicular traffic are exceeded during all design storms for the entire length of the existing jetty.

At the most southern point of the existing dike considered, cross-section 50+00, the overtopping rate during the 1-year storm event is acceptable with regard to structural safety. More significant storms, those with greater return periods, will result in overtopping that can result in failure of the dike in this area. Portions of the dike in the vicinity of cross-section 60+00 have overtopping rates at levels considered safe for structures during both the 1- and 2-year storms. At cross-section 67+50, the dike will have tolerable overtopping rates up to the 10-year storm. The northern most section of the dike evaluated, cross-section 71+00, is able to reduce overtopping to levels deemed safe for structures up to the 50-year storm.

4.6 References

Applied Coastal Research and Engineering, Inc. (2003). "Plymouth Long Beach Nourishment Notice of Intent ."

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5.0 CROSS BEACH ANALYSIS

5.1 Introduction / Summary of Results

It is often convenient to separate nearshore sediment movement into two components, longshore sediment transport and cross-shore sediment transport. For beaches located away from structures (that are perpendicular to the shoreline), inlets, and river mouths, it may be appropriate to neglect longshore transport as a first approximation, i.e., assume the gradient of the longshore transport rate is negligibly small at the site. In this case, cross-shore transport will determine the change in beach profile contours. Therefore, in this investigation we will assume that the change in longshore sediment transport across the study area is negligible and profile change will be produced solely by cross-shore sediment transport.

Beaches erode and accrete in response to varying waves and water levels in the nearshore zone. During storms, significant beach and dune erosion may occur in a matter of hours, resulting in considerable shoreline recession and damage to structures. In other cases, beaches can remain stable during storms and continue to provide storm protection for inshore properties and structures. In order to estimate beach profile changes and beach stability quantitatively under storm conditions, numerical modeling can be employed. Numerical modeling of beach evolution is a powerful technique that can be applied to assist in project design. Numerical models provide a framework for predicting project response, objectively evaluating the effectiveness of design alternatives, and analyzing data to develop an understanding of coastal processes.

The beach profile changes under various storm conditions at the project site were determined using a numerical model SBEACH (Storm-induced BEAch CHange), developed by the U.S. Army Corps of Engineers. SBEACH simulates cross-shore beach, berm, and dune erosion produced by storm waves and water levels.

The cross beach analysis indicated that the storms with return periods up to 100 years will cause minor erosion and beach recession in the beach crest and intertidal area with accretion in the higher berm area between +8' and +11' NGVD and below mean low water between -3' and -7' NGVD. Overall, comparison between the pre- and post-storm beach profile shows that the beach is stable. Sample pre and post-storm profiles are shown in Figures 5.1, 5.2 and 5.3.

5.2 Technical Approach

SBEACH is an empirically based numerical model for simulating two-dimensional cross-shore beach change. The model was initially formulated using data from prototype-scale laboratory experiments and has been further developed and verified with laboratory and field data. SBEACH calculates meso-scale beach profile change with emphasis on beach and dune erosion and bar formation and movement. The model is intended for predicting short-term profile response to storms. A fundamental assumption of the SBEACH model is that profile change is produced solely by cross-shore processes, resulting in a redistribution of sediment across the profile with no net gain or loss of material. Longshore processes are considered to be uniform and are neglected in calculating profile change. This assumption is expected to be valid for short-term storm-induced profile response on open coasts away from tidal inlets and coastal structures.

Recent model enhancements include a random wave model and refined sediment transport relationships to improve calculation of beach response under random waves, and an algorithm to simulate beach and dune erosion produced by overwash.

SBEACH consists of two sub-models: the Wave Model and the Profile Change Model.

Wave Model

The wave model allows wave reformation to take place and the possibility of multiple break points across the nearshore zone. Wave reformation occurs if the local wave height reaches the stable wave height, implying no wave energy dissipation. After energy dissipation ceases, shoaling and refraction become dominant until wave breaking occurs and dissipation is initiated once more.

The one-dimensional equation for conservation of energy flux incorporating energy dissipation associated with wave breaking is given as follows:

$$\frac{\partial}{\partial x}\left(F\cos\theta\right)=\frac{\kappa}{d}\left(F-F_S\right) \qquad \text{[EQ. 5.1]}$$

where:

x = cross-shore coordinate, positive directed seaware
F = wave energy flux
θ = wave angle with respect to the bottom contours
κ = empirical wave decay coefficient (SBEACH uses an average value of 0.15)
F_S = stable wave energy flux
d = total water depth

$$d = h + \eta \qquad \text{[EQ. 5.2]}$$

where:

h = still water depth
η = the mean water surface elevation (setup or setdown) produced by wave motion.

The wave energy flux is given by:

$$F=\frac{1}{8}\rho g H^2 C_g \qquad \text{[EQ. 5.3]}$$

where:

ρ = density of water
H = wave height

C_g = wave group speed, which relates to wave period and total water depth.

The stable wave energy flux is given by:

$$F_S = \frac{1}{8}\rho g (\Gamma d)^2 C_g \qquad \text{[EQ. 5.4]}$$

where:

Γ = the empirical stable wave height coefficient (SBEACH uses an average value of 0.40)

<u>Profile Change Model</u>

Profile change is calculated from the mass conservation equation using the net transport rate distribution. If the sand sources or sinks are absent, the mass conservation equation is:

$$\frac{\partial h}{\partial t} = \frac{\partial q}{\partial x} \qquad \text{[EQ. 5.5]}$$

where:

t = time
q = the net sediment transport rate determined by the appropriate equation below.

For pre-breaking zone ($x_b < x$):

$$q = q_b e^{-\lambda_1 (x - x_b)} \qquad \text{[EQ. 5.6]}$$

For transition zone ($x_p \le x \le x_b$):

$$q = q_p e^{-\lambda_2 (x - x_p)} \qquad \text{[EQ. 5.7]}$$

For broken wave zone ($x_z \le x \le x_p$):

$$\text{For } D > \left(D_{eq} - \frac{\varepsilon}{K}\frac{dh}{dx} \right): \qquad q = K\left(D - D_{eq} + \frac{\varepsilon}{K}\frac{dh}{dx} \right)$$

[EQ. 5.8]

$$\text{For } D \le \left(D_{eq} - \frac{\varepsilon}{K}\frac{dh}{dx} \right): \qquad q = 0$$

For swash zone ($x_r < x < x_z$):

$$q = q_z\left[\frac{x - x_r}{x_z - x_r}\right] \qquad \text{[EQ. 5.9]}$$

where:

λ_1 = spatial decay coefficients in pre-breaking zone (empirically related to median grain size and breaking wave height)

λ_2 = spatial decay coefficients in transition zone (SBEACH uses a value of $0.2\lambda_1$)

K = sand transport rate coefficient, varies in the range of $0.3x10^6 \longrightarrow 2.5x10^6 \frac{m^4}{N}$

($2.5x10^6 \frac{m^4}{N}$ was used in the analysis to be conservative)

ε = slope-related sand transport rate coefficient (a value of $0.002 \frac{m^2}{\text{sec}}$ was used in this analysis)

D = wave energy dissipation per unit water volume

$$D = \frac{\kappa}{d^2}(F - F_S) \qquad \text{[EQ. 5.10]}$$

D_{eq} = equilibrium wave energy dissipation per unit water volume

$$D_{eq} = \frac{5}{24}\rho g^{1.5}\gamma^2 A^{1.5} \qquad \text{[EQ. 5.11]}$$

In which, γ is the ratio between wave height and water depth at breaking. A is the beach profile shape parameter and is mainly a function of median grain size.

The subscripts b, p, z, and r represent quantities evaluated at the break point, plunge point, end of the surf zone, and runup limit, respectively. Different spatial decay coefficients are used in the prebreaking and transition zones, denoted by the subscripts 1 and 2, to describe the decrease in sand transport rate with distance.

5.3 Available Data Sources

SBEACH requires three primary inputs – the initial beach profile, median grain size diameter, and storm source inputs (wind, waves, and water levels). Existing beach profiles for four (4) transects at the Long Beach site were available in the project Environmental Notification Form (ENF). Storm data typically includes time-histories of storm wave height, period (direction is optional) and water levels. Sampling intervals of input wave and water level time-histories usually range from 1 to 4 hours. However, time series data for the particular storm recurrence intervals of interest were not available. The results of the STWAVE modeling, as described in Section 3.0, provided the input parameters for wave height and period. A time series based on a

24-hour tidal cycle, adjusted for storm surge, provided the input water levels. The wind speeds used were those from the WIS Station 93 data, adjusted to account for a longer duration storm.

Data on median grain size diameter was not available for the project site. Project data in the ENF describes the beach material as “medium to coarse grained sand”. Using the Unified Soils Classification system, medium to coarse sand has a range of grain sizes from 0.42 mm to 4.76 mm. With the Wentworth Classification System, medium to coarse sand has a range of grain sized from 0.25 mm to 1.0 mm. Based on this information, a median grain size diameter of 0.5 mm was selected for this analysis.

5.4 Project Specific Analysis

As previously discussed, existing beach profile data was available in the Plymouth Long Beach ENF. Data for four (4) transects (STA 50+00, STA 60+00, STA 67+50, STA 71+00) were entered into SBEACH to model the existing conditions of the beach, which varies in slope, berm and structure crest height along the project site. The profiles were extended further offshore with data obtained from NOAA chart 13253. A median grain size of 0.5 mm was used to correspond to “medium to coarse grained sand”. A typical 24 hour tidal cycle for Plymouth was adjusted to the maximum water level for each storm recurrence interval. Additional input required for model configuration includes parameters such as grid size, time-step, and calibration coefficients (default values are defined). Typical values of model grid size and time-step are 3 meters and 5 minutes, respectively were used in this analysis.

The STWAVE results for deepwater significant wave heights and peak periods (from the northeast quadrant), as listed in Table 3.1, were input as the wave properties at the offshore boundary. The wave height and periods were held constant as time series data for the different storm return periods was not available. This assumption is conservative as the wave heights used as input to the STWAVE model were based on 3 hour averages, and the 24 hour average wave height would be lower. The wave heights were set to vary 20% to account for irregular waves. The results of the statistical analysis of the WIS Station wind data were adjusted to longer duration storm using methods from the *Coastal Engineering Manual*.

An SBEACH model was run for each beach profile at each storm recurrence interval, for a total of 32 models. The models simulated storm durations of 24 hours. The existing stone dike structure was also included in the SBEACH model. The structure was “allowed to fail” at the wave and water levels that contribute to structural damage in the overtopping analysis, as described in Section 4.0.

5.5 Results

Representative SBEACH results for profile STA 67+50 are provided in Tables 5.1 and 5.2. Table 5.1 provides the volume of sand (cubic yard per linear foot) eroded or accreted at the beach/structure crest, the mean tide elevation, and the mean low water elevation. Table 5.2 provides the maximum recession in feet at the beach/structure crest, the mean tide elevation, and the mean low water elevation that can be expected for each storm recurrence interval. Figures 5.1 through 5.3 also depict representative profile changes for this transect for the 1-year, 20-year and

100-year storm recurrence intervals. Most storm levels will cause minor erosion and beach recession in the intertidal area with accretion in the higher berm area between +8’ and +11’ NGVD and below mean low water between -3’ and -7’ NGVD. Overall, comparison between the pre- and post-storm beach profile shows that the beach is stable.

It should be noted that damage to the dike from infrequent storm events does not seem to create additional erosion landward of the structure or scour at the toe.

Table 5.1
Beach Volume Change

	Beach Sand Volume Change (yd^3/ft)							
Elevation (ft NGVD)	**100-year**	**50-year**	**25-year**	**20-year**	**10-year**	**5-year**	**2-year**	**1-year**
Above 15.0 (Crest)	-0.39	-0.34	-0.34	-0.35	-0.35	-0.35	0.00	0.00
Above 5.12 (MHW)	-0.40	-0.39	-0.39	-0.39	-0.39	0.169	0.004	0.127
Above 0.21 (MTL)	-0.45	-0.22	-0.06	-0.02	0.00	-0.01	0.00	0.03
Above -4.7 (MLW)	-1.84	-1.52	-1.18	-1.06	-0.80	-0.66	-0.29	-0.14

Note: "+" equals accretion and "-" equals erosion.

Table 5.2
Maximum Beach Recession/Advance

	Maximum Recession/Advance (feet)							
Elevation (ft NGVD)	**100-year**	**50-year**	**25-year**	**20-year**	**10-year**	**5-year**	**2-year**	**1-year**
15.0 (Crest)	-11.4	-9.5	-9.5	-9.5	-9.5	-9.5	0.0	0.0
5.12 (MHW)	4.5	3.5	3.5	3.5	3.6	7.8	1.2	8.5
0.21 (MTL)	-16.0	-9.6	-5.0	-3.8	-1.0	-1.5	-0.3	-3.6
-4.7 (MLW)	25.2	22.8	20.0	18.8	15.9	14.0	10.3	6.9

Note: "+" equals advance and "-" equals recession.

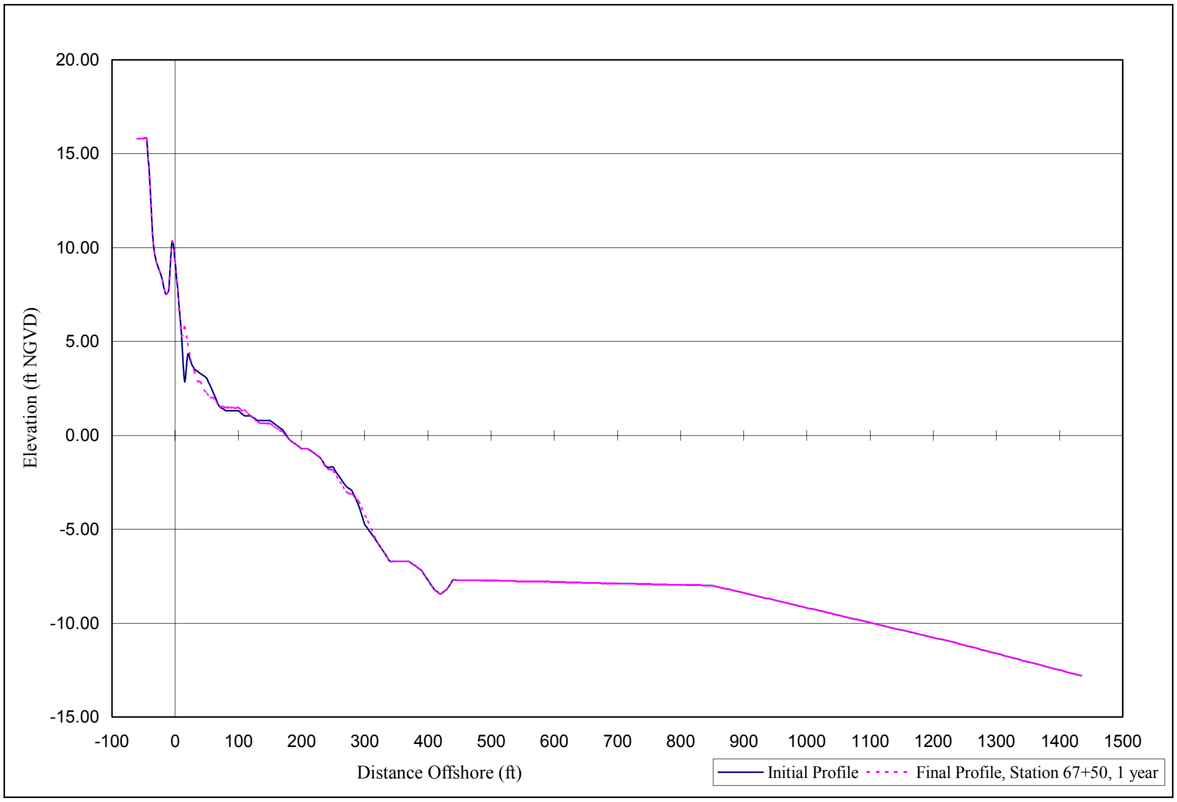

Figure 5.1: Beach profiles for Station 67+50: Existing condition and 1-year storm.

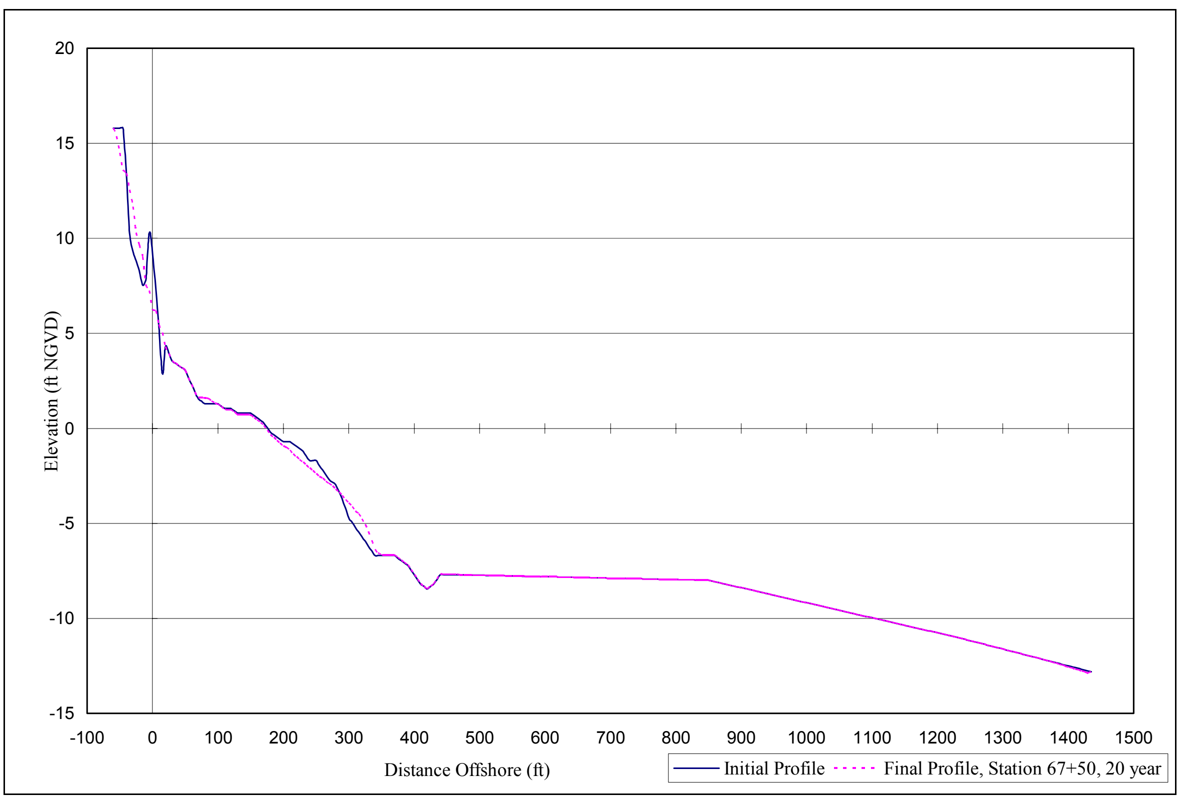

Figure 5.2: Beach profiles for Station 67+50: Existing condition and 20-year storm.

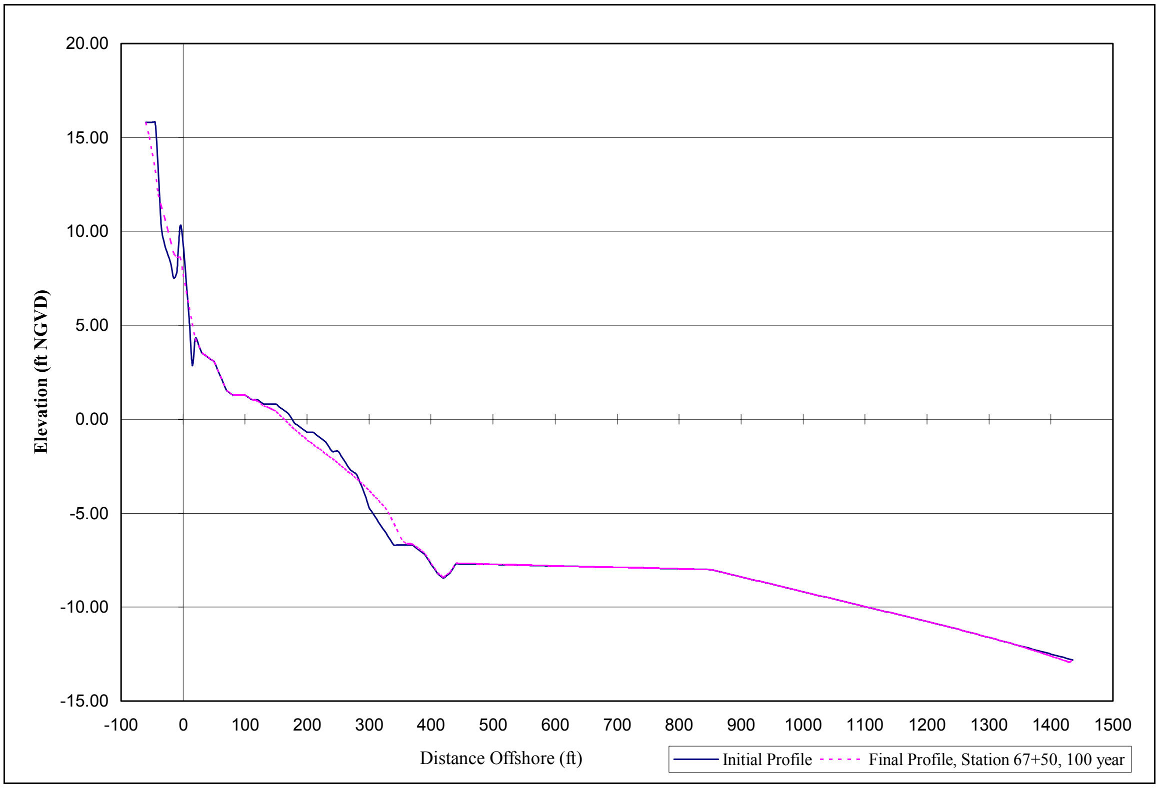

Figure 5.3: Beach profiles for Station 67+50: Existing condition and 100-year storm.

5.6 References

Larson, M., and Kraus, N. C. (1989). SBEACH: Numerical model for simulating storm-induced beach change; Report 1, Empirical foundation and model development, Technical Report CERC-89-9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Larson, M., Kraus, N. C., and Byrnes, M. R. (1990). SBEACH: Numerical model for simulating storm-induced beach change; Report 2, Numerical formulation and model tests, Technical Report CERC-89-9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

6.0 CONCLUSIONS

The Coastal Engineering Analysis completed to date by OCC demonstrates the following regarding the existing conditions at Long Beach, Plymouth, MA:

- WIS Station 93 provides a good record of offshore wind and wave data for use in analyzing the wave climate at Long Beach.
- Waves approaching Long Beach from the northeast govern the design wave conditions for infrequent return period storm events.
- The shallow nearshore waters of Plymouth Bay limit storm wave propagation to the site (the -6 foot contour (MLLW datum) is located approximately 1200 feet offshore and the -12 contour is located approximately 3400 feet offshore). This is reflected in the site specific wave heights from the STWAVE model.
- There is no significant focusing of wave energy at the site.
- The existing stone dike will be overtopped by waves during infrequent storm events. The rate of overtopping is significantly more on the southern portion of the project area.
- Infrequent return period water elevations have a more significant effect on overtopping at the site than infrequent return period wave heights.
- The existing beach is very stable for infrequent return period storm conditions.

With respect to the "no build" alternative, the following can be interpreted from the coastal engineering analysis:

- A massive stone barrier or beach fill is not necessary to limit overtopping rates during infrequent storm events because overtopping of the barrier beach is more a function of water elevation than wave height.
- Damage to the existing stone dike during infrequent storm events is not anticipated to significantly affect overtopping rates as the beach profile is stable and the overtopping is more related to water elevation than wave height.
- It is unlikely that sediment transport from wave overtopping is a significant source of sedimentation in the Plymouth Harbor navigation channel given the indicated stability of the beach during infrequent storm events. In addition, the project location is south of the main navigation channel, so any overtopping that does occur is unlikely to impact the navigation channel.
- The existing beach is stable under infrequent return period storm conditions. Progressive damage of the dike with additional potential for increased overtopping and beach recession was not indicated by the coastal analysis.
- Breaching of the beach was not indicated due to the stability of the beach and existing dike.

The above conclusions are based on the analyses described in this report and the data made available at the time of the analysis. Therefore, OCC reserves the right to revisit some or all of the conclusions reached if new or additional information becomes available in the future.

GLOSSARY

overtopping	Passing of water over a the top of a structure as a result of wave runup or surge action.
refraction	The process by which the direction of a wave moving in shallow water at an angle to the contours is changed: the part of the wave advancing in shallower water moves more slowly than that part still advancing in deeper water, causing the wave to crest to bend toward alignment with the underwater contours.
runup	The rush of water up a structure or beach on the breaking of a wave.
scour	Removal of underwater material by waves and currents, especially at the base or toe of a shore structure.
shoal	To become shallow gradually, to cause to become shallow or to proceed from a greater to a lesser depth of water.
significant wave height	The average height of the one-third highest waves of a given wave group. Note that the composition of the highest waves depends upon the extent to which the lower waves are considered. In wave record analysis, the average height of the highest one-third of a selected number of waves, this number being determined by dividing the time of record by the significant period.

Refererences

U.S. Army Corps of Engineers. (2002). *Coastal Engineering Manual.* Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. (in 6 volumes).