SOUTH SHORE OF STATEN ISLAND

COASTAL STORM RISK MANAGEMENT

INTERIM FEASIBILITY STUDY
FOR
FORT WADSWORTH TO OAKWOOD BEACH

ENGINEERING & DESIGN APPENDIX

U.S. Army Corps of Engineers
New York District

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1.0 PROJECT DESCRIPTION

The US Army Corps of Engineers, New York District (CENAN) is conducting a comprehensive feasibility study to identify and evaluate Coastal Storm Risk management measures for the south shore of Staten Island, NY. The feasibility study is a multi-year and multi-task effort, involving project planning and engineering, economic analyses and environmental studies. This engineering and design appendix provides a descriptive review and discussion of field investigations and data collection, with and without-project coastal processes, and alternative plan development. The information contained within this engineering and design appendix serves as the basis for evaluating these measures in terms of federal interest. A separate appendix is being prepared that documents the development of an interior drainage plan in support of the feasibility study.

The appendix is divided into six primary sections. Section 1 identifies the Study Area and Project Area and summarizes the problem and study authorization. The history and physical conditions of the shoreline and meteorological and oceanographic conditions within the project area are provided in Section 2. The future without project coastal processes are evaluated in Section 3. Section 4 documents the development of structure type, geometry, and alternatives that compose the Tentatively Selected Plan. The National Economic Development (NED) plan is also presented in Section 4. Coastal processes associated with the with-project conditions are identified in Section 5. A monitoring plan for the NED plan is presented in Section 6.

1.1 Study Area

The Study Area comprises the entire southern shore of the Borough of Staten Island (Richmond County), New York City – a distance of approximately 13 miles (see Figure 1-1). It extends along Lower New York Bay and Raritan Bay from Fort Wadsworth at the Narrows to Tottenville at the mouth of Arthur Kill. Adjacent to Staten Island's western shore is the New Jersey shoreline of Raritan Bay, which extends from the community of South Amboy to the Sandy Hook peninsula. East of Staten Island are Brooklyn at the Narrows, Coney Island on Lower New York Bay, and Rockaway Point on the Atlantic Ocean. All of these lie on Long Island. The approach to Lower New York Bay from deep water in the ocean is through a 6-mile wide opening between Sandy Hook, New Jersey and Rockaway Point, New York. Figure 1-1 shows the limits of the Study Area.

The Study Area lies within the limits of the City of New York and consists of a series of neighborhoods. The principal neighborhoods along the southern shore of Staten Island from east to west are South Beach, Midland Beach, New Dorp Beach, Oakwood Beach, Great Kills, Annadale Beach, Huguenot Beach, Prince's Bay and Tottenville Beach.

The study area is divided into three similar segments to evaluate economic benefits of Coastal Storm Risk management measures:

- Fort Wadsworth to Oakwood Beach (Phase I)
- Great Kills Harbor to Crescent Beach (Phase II)
- Annadale to Tottenville (Phase II)

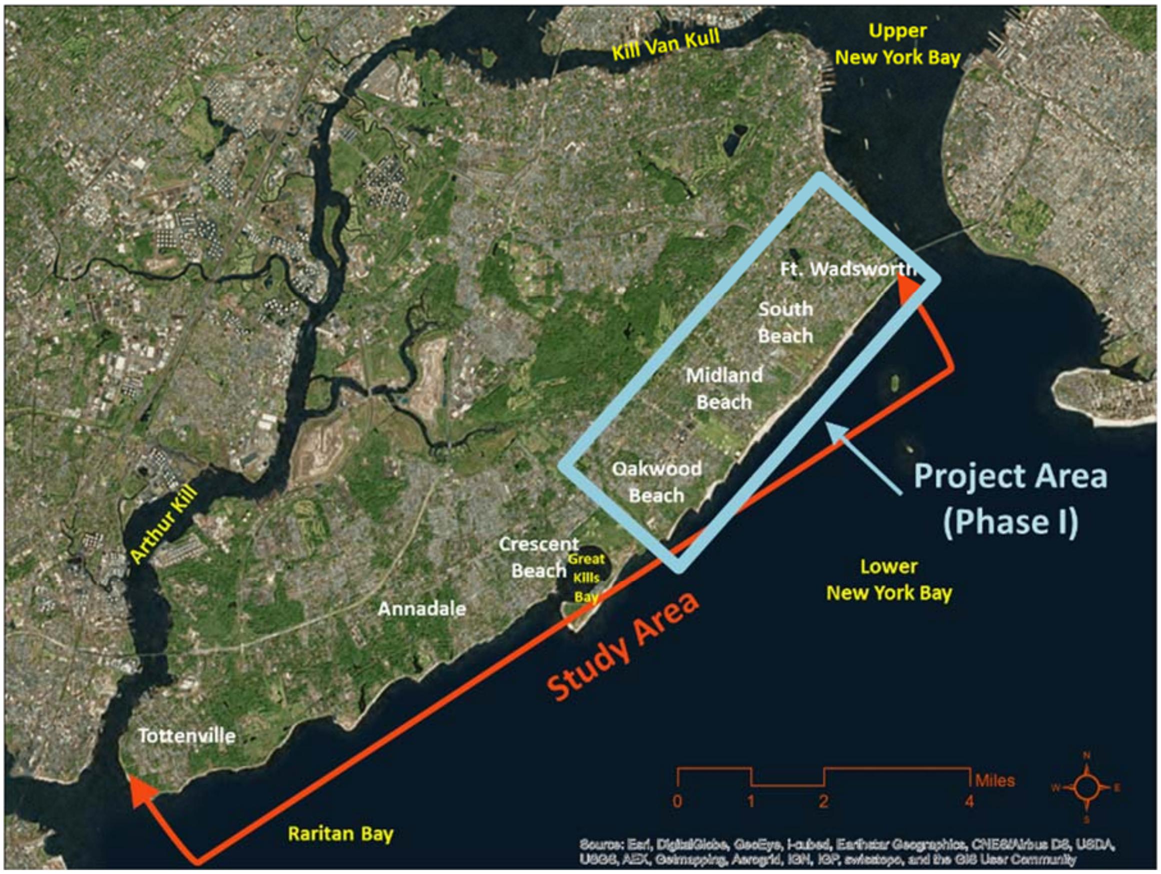


Figure 1-1: Study Area

In the aftermath of Hurricane Sandy the study area was split into two phases: Phase I – Fort Wadsworth to Oakwood Beach and Phase II Great Kills Harbor to Tottenville. The focus of this Engineering & Design Appendix is Phase I.

The Phase I Project Area is Fort Wadsworth to Oakwood Beach, as shown in Figure 1-2. The Project Area terrain ranges from high bluffs at the western and eastern end of the study area to low lying areas in much of the central section. The west end is fronted by low narrow beaches intersected by several creeks and lake outfalls. The east end generally has a wide low beach intersected by several drainage outfalls contained in groins. Behind the east end beaches are low-lying residential areas. The shoreline is irregular because of the downdrift offsets at groins and headlands. The Project Area was divided in four engineering descriptive "shoreline reaches" (A1 to A4) based on physical conditions of the shoreline, existing coastal and stormwater outfall structures, and existing infrastructure.

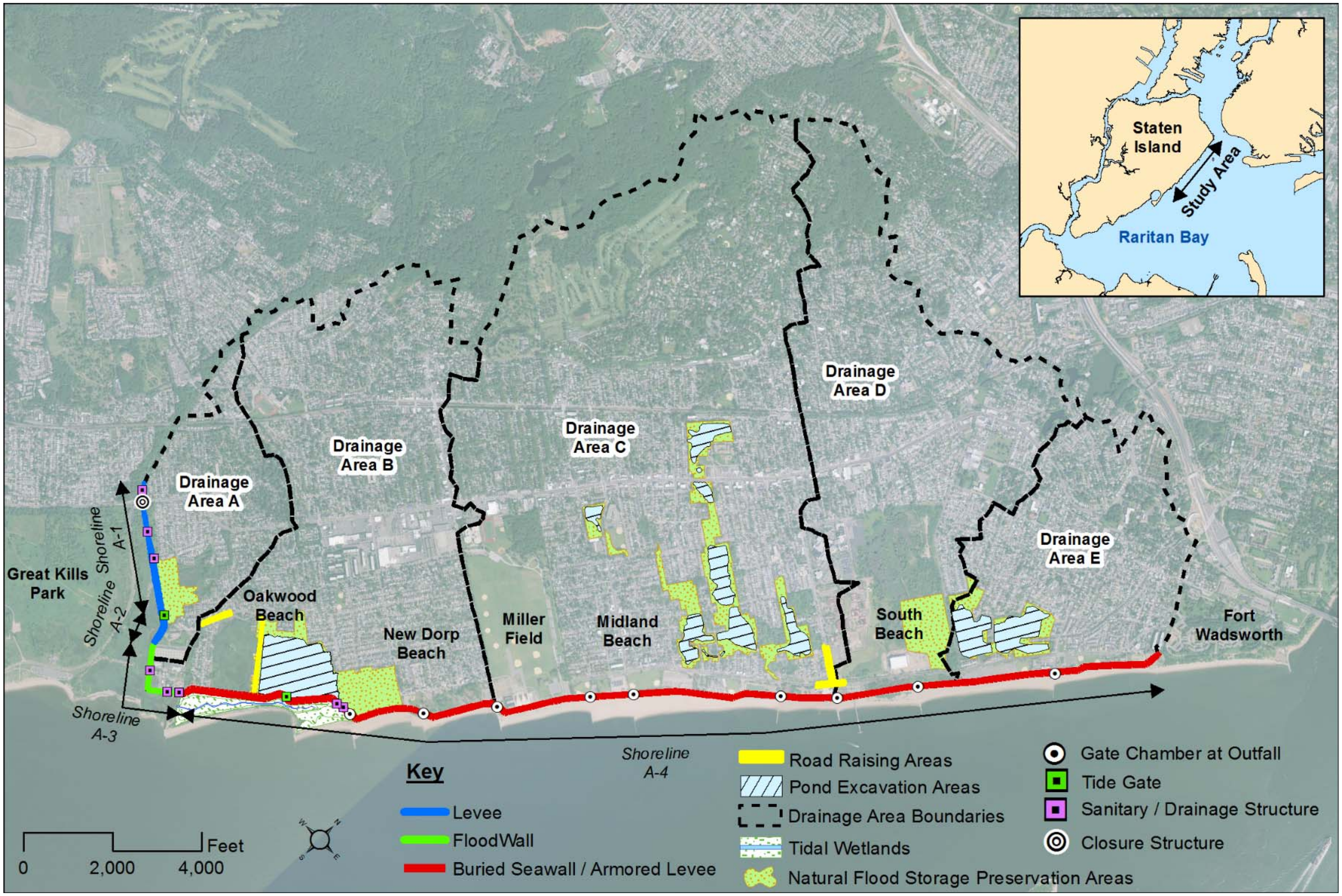


Figure 1-2: Project Area

1.2 Problem Identification

Coastal flooding along the south shore of Staten Island from hurricanes in August 1954, September 1954, and September 1960 (Hurricane Donna) caused extensive damage as low lying areas to the north of the Father Capodanno Boulevard (formerly Seaside Boulevard) were inundated by storm surge. In several locations near Fort Wadsworth, floodwaters approached three feet in depth. Large waves also resulted in extensive beach and dune erosion at Fort Wadsworth and Cedar Grove and New Dorp Beaches. The March 1962 Nor'easter resulted in additional flooding and damage.

A beach erosion study that included the south shore of Staten Island was authorized in 1955 and updated after the 1960 hurricane. The conclusion of the beach erosion control study lead to the authorization of a feasibility study in October 1965 to evaluate federal interest in a flood control program from Fort Wadsworth to Arthur Kill, Staten Island in October 1965. The feasibility study was extended to encompass Fort Wadsworth in 1969. The recommended protective works included beach fill with dunes, groins, levees, floodwalls, and interior drainage facilities including pumping stations and relocations. A draft EIS and Design Memorandum for this plan were completed in 1976 and 1977, respectively. Financial troubles that affected New York City in the late 1970's resulted in deferred construction of the project.

There was renewed interest in the project after extensive flooding occurred along the south shore of Staten Island from the December 1992 Nor'easter. During this storm, flood levels ranged from 3 to 5 feet above existing ground between Fort Wadsworth and Miller Field. Nearly 2,000 structures along this shoreline were affected during this event including a large number of cottages at Cedar Grove Beach. At Oakwood Beach, an artificial dune system constructed by City of New York was breached, inundating low-lying areas with 5 feet of water.

In 1995, a reconnaissance level investigation was authorized to evaluated federal interest in storm damage reduction along the shoreline from Fort Wadsworth to Oakwood Beach and Annadale Beach (USACE, 1995). Several flood control and shore protection alternatives were investigated based on local needs and preferences, comparative costs, and implementation constraints. The reconnaissance level analysis indicated that there was federal interest in continued study.

Following Hurricane Sandy the feasibility study was split into two phases:

- Phase I – Fort Wadsworth to Oakwood Beach
- Phase II – Great Kills to Tottenville

This engineering and design appendix covers Phase I.

1.3 Study Authority

A cooperative beach erosion and storm damage protection study was authorized by a resolution of the U.S. House of Representatives Committee on Public Works and Transportation and adopted May 13, 1993. The resolution states that:

> *"The Secretary of the Army, acting through the Chief of Engineers, is requested to review the report of the Chief of Engineers, on the Staten Island Coast from Fort Wadsworth to*

Arthur Kill, New York, published as House Document 181, Eighty-ninth Congress, First Session, and other pertinent reports, to determine whether modifications of the recommendations contained therein are advisable at the present time, in the interest of beach erosion control, storm damage reduction and related purposes on the South Shore of Staten Island, New York, particularly in and adjacent to the communities of New Dorp Beach, Oakwood Beach, and Annadale Beach, New York."

Formal requests for a new reconnaissance study were made by former Governor Mario Cuomo to the District Engineer in letters dated January 4, 1993 and June 24, 1993. The non-Federal sponsor for this study is the New York State Department of Environmental Conservation (NYSDEC) who subsequently entered into a partnering agreement with the New York City Department of Environmental Protection (NYCDEP) and the New York City Department of Parks and Recreation (NYCDPR).

2.0 EXISTING CONDITIONS

Hurricane Sandy occurred near the completion of the draft Feasibility Study for the south shore of Staten Island. Hurricane Sandy was the most devastating coastal storm event on record to impact the south shore of Staten Island. Consequently, some of the design conditions and technical analysis completed prior to Hurricane Sandy had to be updated to reflect the changed conditions. For example, the Federal Emergency Management Agency (FEMA) released a new Flood Insurance Study for the south shore identifying increased flood hazard areas and higher stillwater and base flood elevations. The new FEMA stillwater elevations were reviewed and incorporated into the analysis supporting the feasibility study. The revised stillwater elevations affect the design of the structures that compromise the Line of Protection for this project. The geometry and stability of the structures (height, armor stone size, etc.) were modified due to these changed condition. However, the analysis of coastal processes such as the long-term sediment budget and shoreline change were not updated since these processes are dominated by average wave and water level conditions and not a single large episodic event.

A list of updated baseline conditions and revisions to the technical analyses performed in support of the feasibility study are identified as follows:

- Stillwater Elevations – FEMA 2013 FIS Study for Staten Island,
- Storm History,
- Storm-Induced Shoreline Change (SBEACH Modeling),
- Recent Shoreline Change Analysis,
- Surfzone Wave Transformation (i.e. Depth-Limited Wave Conditions),
- Wave Overtopping and Structure Crest Elevations,
- Armor Stone Stability,
- Wave Forces on Vertical Walls,

- All of the above analysis incorporated the latest Post-Sandy LIDAR data in their respective analysis.

The following baseline conditions and technical analyses were not updated in the feasibility study:

- Tides, Sea Level Change, Currents, Winds, and Sediment Characteristics,
- Sediment Budget,
- Topographic Elevations shown on Plan View Drawing Set,
- Submerged Beach Profile Conditions,
- Offshore and Nearshore Wave Conditions.

2.1 Physical Characteristics

2.1.1 *Survey and Field Collection*

Topographic Survey

Topographic surveys conducted by Rogers Surveying in 2000 and 2001 are the most recent topographic survey data for the project area. Post-Hurricane Sandy LIDAR was collected by the USACE Joint Airborne LIDAR Bathymetry Technical Center of Expertise (JALBTCX) on November 16, 2012. In addition, beach profile data is available for the project area (described below). The project area was surveyed on the following dates:

- October 27, 2000 - Oakwood Beach,
- July 1, 2001 - Fort Wadsworth to Oakwood Beach,
- November 16, 2012 - LIDAR over entire Project Area (Fort Wadsworth to Oakwood Beach).

Bathymetric Survey

Beach profile surveys were performed in February 2000 for the entire Project Area. Table 2-1 shows a complete list of the beach profiles. Figure 2-1 shows the location of the beach profile surveys. Additional bathymetric data is available from NOAA Navigation Chart 12402 “New York Lower Bay” and is used to supplement the beach profile surveys and characterize the offshore bathymetry.

The profile data was grouped into four survey reaches along the Project Area. Reach A, representing Fort Wadsworth to South Beach, includes profile lines 1, 1A and 2. Reach B, representing Midland Beach, includes profile lines 2A, 3 and 3A. Reach C, representing New Dorp Beach, includes profile lines 4 and 4A. Reach D, representing Oakwood Beach, includes profile lines 5 and 5A.

Table 2-1: **Beach Profiles Surveyed in February 2000**

Profile	Survey Reach	Location
PL-1	A	Fort Wadsworth to South Beach
PL-1A	A	Fort Wadsworth to South Beach
PL-2	A	Fort Wadsworth to South Beach
PL-2A	B	Midland Beach
PL-3	B	Midland Beach
PL-3A	B	Midland Beach
PL-4	C	New Dorp Beach
PL-4A	C	New Dorp Beach
PL-5	D	Oakwood Beach
PL-5A	D	Oakwood Beach

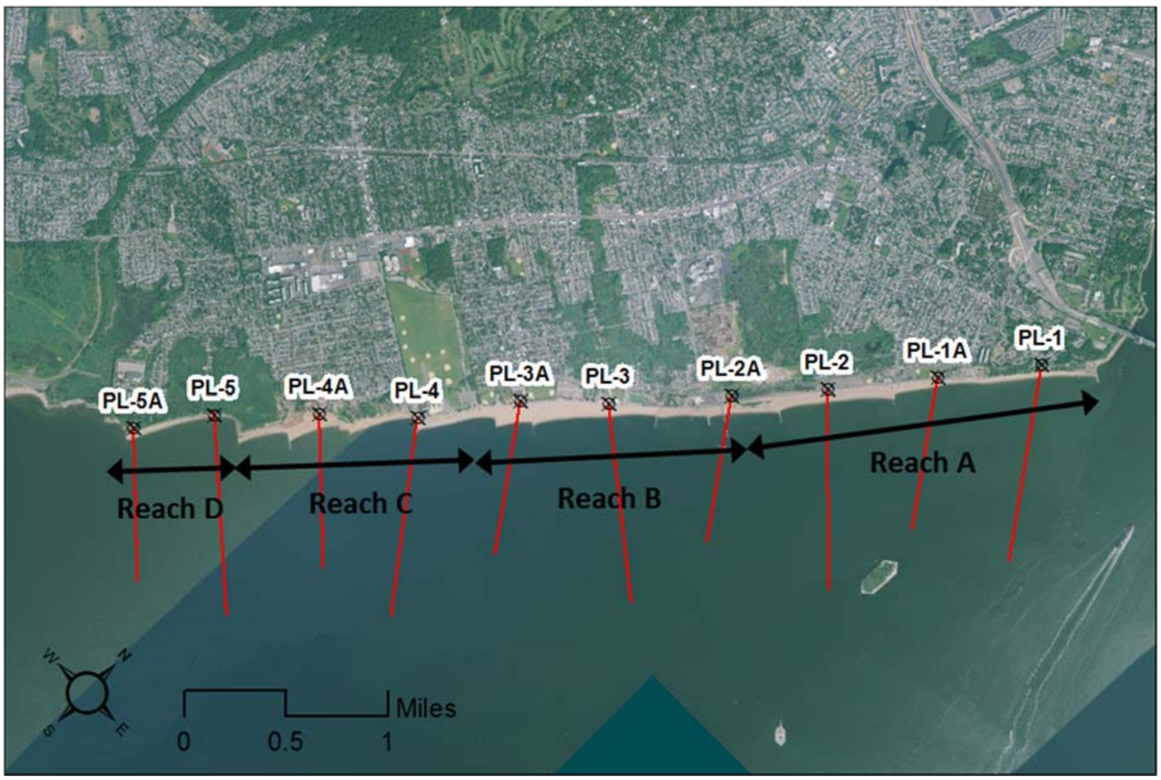


Figure 2-1: Beach Profile Survey Locations

Geotechnical Investigations

Fourteen (14) borings (designated as SS02-4 through SS02-17) were performed along the shoreline between Fort Wadsworth and Oakwood beach. In addition, in the vicinity of the water treatment plant, several test boring results provided by NYCDEP were utilized to evaluate the subsurface conditions along the Line of Protection in that area.

All borings were advanced using mud rotary drilling techniques. Soil samples were obtained using techniques and equipment in general accordance with the American Society for Testing and Materials (ASTM) Standard Specification D1586-Standard Penetration Test (SPT). Soils were classified using the Unified Soil Classification System (USCS) method and one to two samples per boring were chosen for laboratory analysis. Undisturbed Shelby tube samples were also obtained from relatively soft or organic fine-grained soils for laboratory testing. All test borings were advanced to final depths ranging from 24 to 30 ft below ground surface (bgs). Bedrock was not encountered in any of the test borings.

The laboratory testing program consisted of a variety of tests performed on selected soil samples obtained from the borings to verify the field classifications and to provide additional information for engineering evaluations. The tests included grain size, specific gravity, unit weight, and Atterberg Liquid and Plastic Limits. Triaxial compression strength, grain size, unit weight, and Atterberg limits tests were performed on undisturbed Shelby Tube samples. All tests were performed by SOR Testing Laboratories, Inc. of Cedar Grove, NJ.

Based on the results of the test borings, the primary soil type encountered at the project site was coarse to fine sand with varying amounts of silt and gravel. However, in the vicinity of the water treatment plant, soft compressible organic soils were encountered to depths of about 6 feet below the ground surface. The laboratory tests show that the majority of the sands at the site consist of trace to some amounts of silt and gravel. The borings also indicate the presence of some clay and silt lenses within this stratum that ranged from 1 to 9 ft in thickness, at various isolated locations. Generally, the SPT N-values within this stratum ranged from 10 blows per foot (bpf) to 30 bpf and with an average of about 18 bpf, indicative of a medium dense material. Since all borings were terminated within this stratum the thickness of this stratum is not defined at present.

<u>Sediment Sampling/Characteristics</u>

Beach sediment samples were obtained in January through March 1961, May 1962, February 1995 and February 2000. Generally the mechanical analyses of the samples indicate the littoral materials consist of fine to medium-grained sands. However, there is significant fluctuation in the mean grain size diameters and standard deviations of the acquired beach samples. Beach samples were taken within all the survey reaches listed above at the above tidal, intertidal and below tidal zones.

In previous stages of this study it was recommended that the 1995 data set best represents the native beach sediment for the south shore of Staten Island. Sediment characteristics were determined from mechanical analyses and from calculations of grain size distributions while omitting anomalous samples.

The February 2000 beach samples were collected along five of the surveyed profile lines, 1, 2, 3, 4, and 5. Samples were collected at elevations +12, +6, +2, 0, -2, -6 and -10 ft along each profile. Generally, samples collected below the tidal zone gave the best equilibrium profile (Y = Ax2/3). Median grain sizes from samples collected below the tidal zone closely match the 1995 sediment samples. Median grain sizes from the 1995 and 2000 data sets are shown in Table 2-2.

Table 2-2: Beach Profile Sediment Characteristics (1995 and 2000)

Profile	Survey Reach	Year	Location	D50 (phi)	D50 (mm)	Sample Elevation
PL-2	A	1995	Fort Wadsworth to South Beach	1.06	0.48	
PL-1	A	2000	Fort Wadsworth to South Beach	1.09	0.47	Below Tidal
PL-2	A	2000	Fort Wadsworth to South Beach	0.27	0.83	Below Tidal
PL-3	B	2000	Midland Beach	1.06	0.48	Below Tidal
PL-4	C	1995	New Dorp	1.03	0.49	
PL-4	C	2000	New Dorp	0.81	0.57	Intertidal
PL-5	D	2000	Oakwood Beach	0.81	0.57	Below Tidal

2.1.2 *Shoreline Characteristics*

Geology

The south shore of Staten Island lies within the Atlantic Coastal Plain province, which extends along the eastern margin of the United States. The surface of the plain slopes gently in a southeast direction toward the Atlantic Ocean and merges into the tidal marshes, shallow bays, and barrier beaches at the shore. The plain continues offshore beneath the waters of the ocean for about a distance of 100 miles to the edge of the continental shelf, where at a depth of approximately 100 fathoms, it is bounded by a steep escarpment. At the edge of the continental shelf the ocean bottom drops abruptly to far greater depths. A submarine valley of the Hudson River crosses the continental shelf in Lower New York Bay. The bed elevation of this valley is more than 100 feet below MSL and varies in width from 2 to 10 miles.

Beach Profile/Dimensions

The Project Area terrain ranges from high bluffs (Fort Wadsworth) to low-lying areas in much of the center. Most of the Project Area generally has 250-350 foot wide dune-less beach intersected by several outfall structures / groins. The shoreline is irregular because of the downdrift offsets at groins. Landward the beaches are low-lying residential areas containing many structures susceptible to flooding.

The average beach profile characteristics, including onshore, nearshore and offshore slopes, and berm heights and beach widths, are categorized by four representative reaches as measured and tabulated in Table 2-3. Figure 2-2 shows the surveyed beach profiles at all 10 locations.

The beach width in Reach A averages approximately 240 ft. The footprint of the boardwalk/promenade represents an additional beach width of approximately 40 ft. The wide beach berm provides protection against beach erosion and wave attack, but the low berm crest elevation (+10 ft NGVD) limits storm protection to the developed area of South Beach adjacent to Father Capodanno Boulevard. Ground elevations and associated foundations of residential and commercial buildings north of Father Capodanno Boulevard are lower, making these areas and buildings susceptible to flooding once water passes over the road.

Midland Beach, represented as Reach B, has the widest beach at approximately 360 ft fronting the 40-ft wide boardwalk/ promenade. The beach berm is at approximately elevation 10 ft NGVD. Similar to the conditions in South Beach, once floodwater pass over Father Capodanno Boulevard, low lying structures are subject to flooding.

Reach C (New Dorp Beach) has a progressively narrower beach compared to Midland Beach. The average beach width is 240 ft and the beach berm elevation is approximately 9 ft NGVD. There is no boardwalk or promenade in this area.

The beach widths in Reach D (Oakwood Beach) range 0 to 170 feet. Immediately downdrift of the bulkhead/groin at the eastern limit of the Oakwood Beach, the beach is very narrow (0 to 50 feet) and backed by an existing riprap revetment. Further west, the beach widens and a vegetated dune rises up to approximately 16 ft NGVD. The average berm height in the Oakwood Beach area is at elevation 8 ft NGVD, and the average beach width is 117 ft. Landward of the beach berm, the area consists of low-lying wetlands (elevation 3 to 4 feet) and a scattering of residential homes.

Table 2-3: Beach Profile Characteristics

Reach	Profile	Onshore Slope	Nearshore Slope	Offshore Slope	Beach/Bluff Elevation (ft, NGVD)	Beach Width (ft)
A	PL-1	1:9	1:15	1:98	10.1	254
	PL-1A	1:9	1:10	1:90	10.4	187
	PL-2	1:11	1:9	1:106	9.2	279
	Mean	**1:10**	**1:11**	**1:99**	**9.9**	**240**
B	PL-2A	1:10	1:11	1:104	9.2	339
	PL-3	1:9	1:10	1:81	10	319
	PL-3A	1:10	1:14	1:113	10	426
	Mean	**1:10**	**1:12**	**1:97**	**10.2**	**361**
C	PL-4	1:11	1:11.0	1:116	9.4	249
	PL-4A	1:9	1:11	1:413	8.3	234
	Mean	**1:10**	**1:11**	**1:182**	**8.9**	**242**
D	PL-5	1:11	1:9	1:458	6	63
	PL-5A	1:10	1:11	1:172	9.8	170
	Mean	**1:10**	**1:10**	**1:250**	**7.9**	**117**

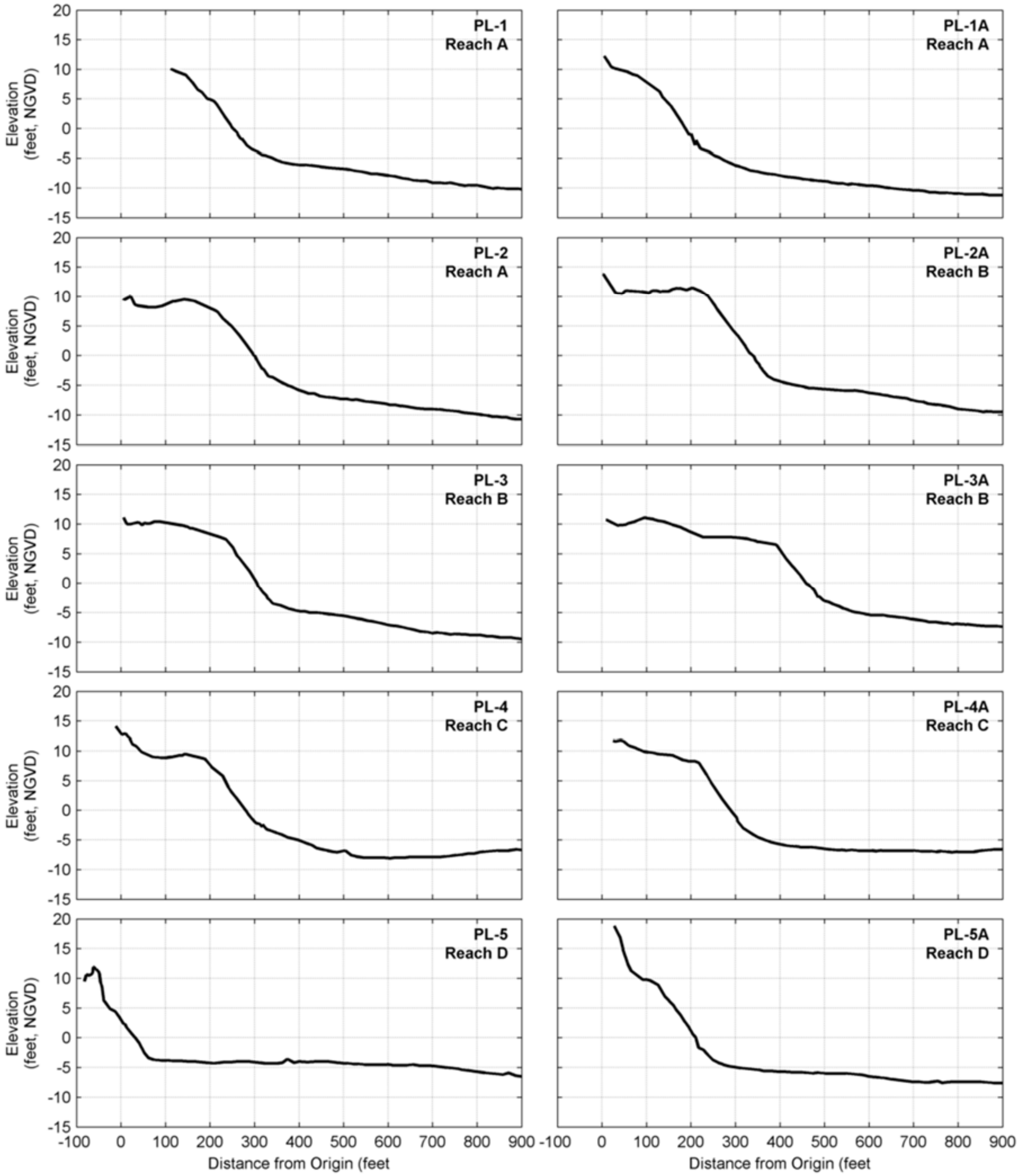


Figure 2-2: Beach Profile Surveys

<u>Historical Shoreline Change and Long-Term Erosion Rate</u>

Historical data on shoreline changes for the project area cover the time period 1836-1994 (Smith et al., 1995) based on topographic sheets and aerial photographs obtained from the National Atmospheric and Oceanic Administration (NOAA). Additional shoreline analysis was performed based on comparisons of beach profiles surveyed in March 1961, February 1995, and February

2000. Beach fill occurred between 1937 and 1960 but beach fill has not been placed between the beach profile survey dates (1961 and 1995). The results of the historical shoreline change analysis and recent shoreline change analysis are presented in Table 2-4 and Table 2-5.

The shorelines in the study area exhibit minimal recession and have generally been mildly erosional. Mechanically placed fill has resulted in major incidents of shoreline advance. The mean high water shoreline data from historic maps, aerial photographs, and surveys were used to conduct a shoreline analysis. The results indicated that the rate of erosion over most large areas of the shoreline is low. Most areas have averaged less than one foot of shoreline loss annually during the most recent period of analysis. Historic fill projects may have impacted shoreline loss rates in this area.

Despite the overall mild shoreline changes, certain areas have experienced dramatic change as the shoreline reaches equilibrium adjacent to newly constructed coastal structures. The effect has been the development of headland-like features, with dramatic embayments. An example is Oakwood Beach, where the shoreline immediately west of coastal structures is seriously offset. Areas such as Fort Wadsworth have experienced minimal change, as they lie adjacent to land masses featuring elevated headlands consisting of more rocky material, helping to naturally strengthen the land against erosional forces.

Table 2-4: Historical Shoreline Change (Based on Shorelines)

Time Period	Avg (ft/yr)	Max (ft/yr)	Reach (for Max)
1836-1855	-3.6	-9.3	Fort Wadsworth to New Creek
1855-1886	-3.2	-10.8	Oakwood Beach to Crookes point
1886-1935	-3.6	-18.8	Oakwood Beach to Crookes point
1935-1974	+2.3	+9.8	Fort Wadsworth to New Creek
1974-1994	-0.9	-4.9	Oakwood Beach to Crookes point

Table 2-5: Recent Shoreline Movement (Based on Beach Profiles)

<table>
<tr><th rowspan="2">Reach</th><th rowspan="2">Profile Line</th><th colspan="2">Avg (ft/yr)</th></tr>
<tr><th>1961-2000</th><th>1995-2000</th></tr>
<tr><td rowspan="4">A</td><td>1</td><td>0.0</td><td>-2.8</td></tr>
<tr><td>1A</td><td></td><td>+0.2</td></tr>
<tr><td>2</td><td></td><td>+1.2</td></tr>
<tr><td>AVERAGE</td><td>0.0</td><td>-0.5</td></tr>
<tr><td rowspan="4">B</td><td>2A</td><td></td><td>-0.2</td></tr>
<tr><td>3</td><td>+0.9</td><td>+1.8</td></tr>
<tr><td>3A</td><td></td><td>+3.6</td></tr>
<tr><td>AVERAGE</td><td>+0.9</td><td>+1.7</td></tr>
<tr><td rowspan="3">C</td><td>4</td><td>-0.3</td><td>+0.8</td></tr>
<tr><td>4A</td><td></td><td>-0.2</td></tr>
<tr><td>AVERAGE</td><td>-0.3</td><td>+0.3</td></tr>
<tr><td rowspan="3">D</td><td>5</td><td>-1.1</td><td>-1.3</td></tr>
<tr><td>5A</td><td></td><td>-1.6</td></tr>
<tr><td>AVERAGE</td><td>-1.1</td><td>-1.5</td></tr>
</table>

Recent Shoreline Change (2004-2014)

Recent shoreline changes were analyzed by comparing aerial imagery from the spring of 2004 and spring of 2014 that were published by Google Earth. This analysis was performed to reaffirm the historical shoreline trends described above and sediment budget described below. The wet/dry line on the aerial photography was selected as the baseline shoreline for 2004 and 2014. During this 10-year period the City of New York City Department of Parks and Recreation (NYC Parks) has performed miscellaneous beach management activities that include beach scraping and construction of artificial dunes. An exact accounting of sediment added to the active littoral system from these activities is not available, but the assumption is that these activities did not provide a significant contribution to the sediment budget or shoreline change.

The observed shoreline changes are shown in Figure 2-3. Reach average shoreline change rates along South Beach, Midland Beach, and New Dorp range between -1 ft/yr to -3.5 ft/yr. These shoreline change rates are similar to the historical sediment budget described below.

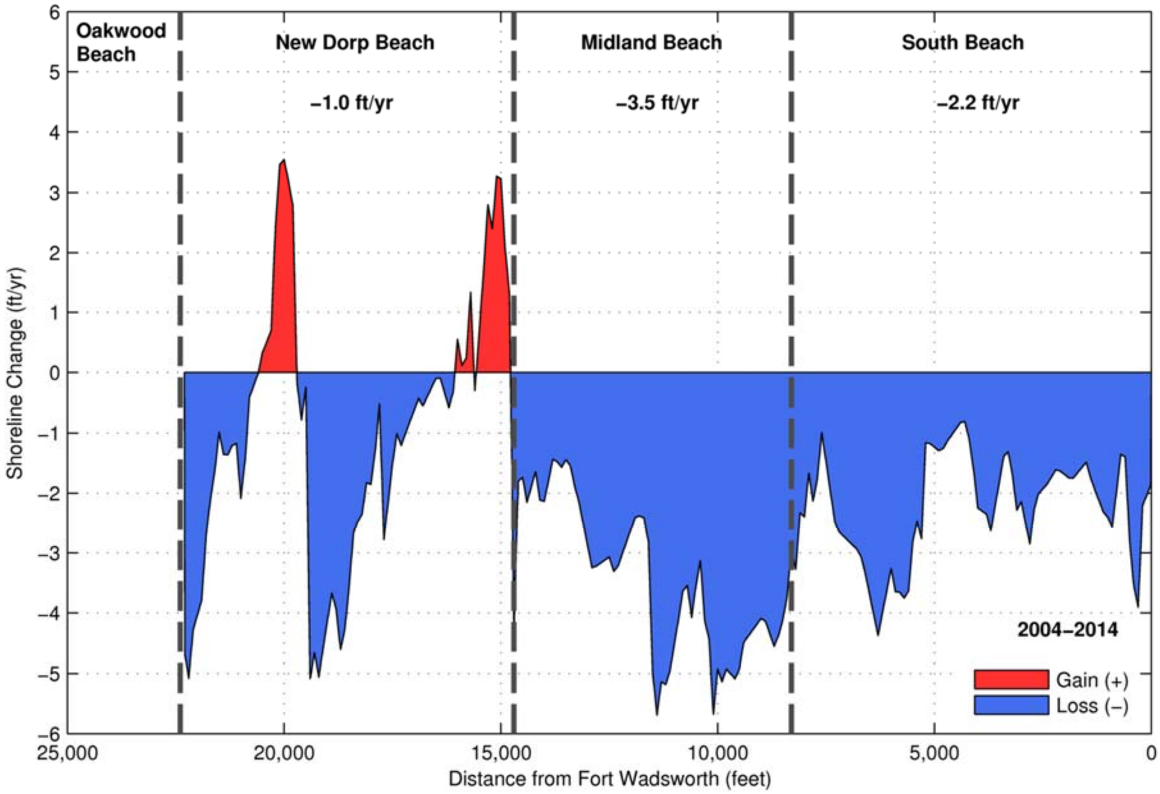


Figure 2-3: Recent Shoreline Changes

Long-term Volumetric Changes

Evaluation of volume changes for the project area was performed using the 1961 and 2000 profile surveys. Volume change computations show agreement with the shoreline location response. Within the 39 year period from 1961 to 2000, the beaches east of Great Kills Park showed mild erosion with the exception of Midland Beach which showed accretion.

Long-term erosion rates were calculated using the beach profile data characteristics at each location and a closure depth of -5 ft NGVD. Table 2-6 lists the volume changes and historic long term erosion rates for the Project Area shoreline. The erosion rates shown below are based on measured volumetric changes and the active beach, as opposed to the results shown in Table 2-5 above which refers to the MHW shoreline (+3 NGVD).

Table 2-6: Volumetric Changes 1961-2000

Location	Reach Length(ft)	(cy)	(cy/yr)	(cy/yr/ft)	Erosion (ft/yr)
Ft. Wadsworth to So. Beach	8,310	-699,020	-17,900	-2.2	-3.9
Midland Beach	5,365	193,140	5,000	0.9	1.6
New Dorp Beach	6,865	-10,805	-300	-0.04	-0.1
Oakwood Beach	3,135	-222,585	-5,700	-1.8	-3.2
Great Kills Park	3,000	-886,700	-26,100	-8.7	-15.7

Notes: Volume change data for these areas are based on shoreline change data for the period 1974 – 1994 (Reference 8). Erosion rates were calculated assuming a 10 ft high berm, using 1961 profile lines information. Depth of closure was assumed to be at -5 ft NGVD (USACE, 1995).

Sediment Budget

A sediment budget was developed for the south shore of Staten Island extending along the entire south shoreline of Staten Island from Fort Wadsworth to Tottenville. However, only sediment budget cells within the Project Area are presented here (Figure 2-4). A sediment budget is a tallying of sediment gains and losses, or sources and sinks, within a specified control volume (or cell). There are several valid approaches to developing sediment budgets. For the SSSI project, the sediment budget was derived from observed sediment gains and losses from 1961 & 2000 beach profiles (Table 2-6). This time period is well suited for evaluating the sediment budget since beachfill, coastal structures projects and/or dredging operations did not occur within the study area.

Sediment budgets are a critical tool in understanding how federally sponsored storm damage reduction projects with a shore protection component (beach nourishment, inlet management, etc.) affect the existing condition sediment budget and shoreline erosion rates within a study area. The Tentatively Selected Plan (TSP) for the SSSI does not include structures that affect or alter the sediment budget. Therefore, it is not anticipated that historical shoreline changes (i.e. 1961-2000) will change in response to the TSP.

The most important assumption in the sediment budget is that the observed volume changes from 1961-2000 accurately reflect historical, without- and with-project gains and losses within the cells. The other assumptions do not directly affect the predicted volume gains and losses in the sediment budget. To reduce the uncertainty in the predicted volume gains and losses in the sediment budget, more recent shoreline changes were evaluated, accounting for the impact of Hurricane Sandy. The recent shoreline change analysis was overall in agreement with the predicted gains and losses in the sediment budget, increasing confidence in the sediment budget.

Two other assumptions were made regarding the littoral dynamics of the area. Since the study area is bound at each end by deep channels, and beach fill projects did not occur during the study period, it is assumed that no sand volume enters the area from the east or west. Secondly, it is assumed that no significant offshore losses from the beach occur beyond the depth of closure. Longshore transport rates were calculated by balancing the sediment budget. Sediment sinks within the study area are limited to The Narrows (10,600 cy/yr) at the northern end of the study area and Great Kills Harbor (1,300 cy/yr) at the southern end. The estimated sediment losses at the Narrows and Great Kills Harbor are based on the Reconnaissance Report (1995).

Long-term sediment transport modeling to evaluate potential impacts to the sediment transport and shoreline change rates may not have been performed during the P7 study since the alternatives identified in P7 study were located landward of the dune and beach area (with the exception of the beach fill alternative). In addition, the calibration and validation data collected during the P7 study was limited and would have led to high level of uncertainty and low confidence levels in the modeling results.

Since the preferred alternative selected to be advanced in the feasibility study was located in the upland region where it would not affect the sediment transport and shoreline change rates, more extensive shoreline transport modeling was not required. Historical aerial photographs of the shoreline indicate that a nodal point in the longshore sediment transport exists at South Beach. East of the nodal point (Fort Wadsworth) the net sediment transport direction is east, west of the nodal point (Midland Beach to Great Kills Park) the net sediment transport direction is to the west.

Fort Wadsworth to South Beach, Cell R1, has an average sediment loss of 17,900 cy/yr (shoreline erosion). A total 10,600 cy/yr is estimated to be transported offshore and lost into the narrows at eastern end of the cell. An estimated 7,300 cy/yr is calculated to be transported westward towards Midland Beach.

Midland Beach, Cell R2, accumulates a total 5,000 cy/y (shoreline accretion). Based on aerial photos of the study area, the predominant sediment transport direction is to the west. With 7,300 cy/yr transported into the cell from the east, 2,300 cy/yr is calculated to be transported westward towards Miller Field.

New Dorp Beach, Cell R3, has an average sediment loss of 300 cy/yr (shoreline erosion). Adding this eroded sediment to the total sediment entering the cell from the east, a total 2,600 cy/yr is calculated to be transport westward towards Oakwood Beach.

Oakwood Beach, Cell R4, has an average sediment loss of 5,700 cy/yr (shoreline erosion). Adding this eroded amount to the total sediment entering the cell from the east, a total of 8,300 cy/yr is calculated to transported westwards towards Great Kills Park.

Great Kills Park, Cell R5, has an average sediment loss of 26,100 cy/yr (shoreline erosion). Interpreting the complex sediment transport system of the Great Kills Park required engineering judgment. Maintenance dredging records from 1945 to 1995 of the Great Kills entrance channel indicate about 1,600 cy/yr of sediment is transported into the channel from the east. With 26,100 cy/yr erosion and 8,300 cy/yr moving from the east, a total 34,400 cy/yr sediment movement must be transported west at the western end of GKP. Of the 34,400 cy/yr it is estimated that 1,600 cy/yr of the material is deposited in the entrance channel, 22,800 cy/yr is transported offshore and stored in ebb shoal, and the remaining 10,000 cy/yr are assumed to bypass the channel and transported westwards to the shoreline between Crescent Beach and Annadale Beach.

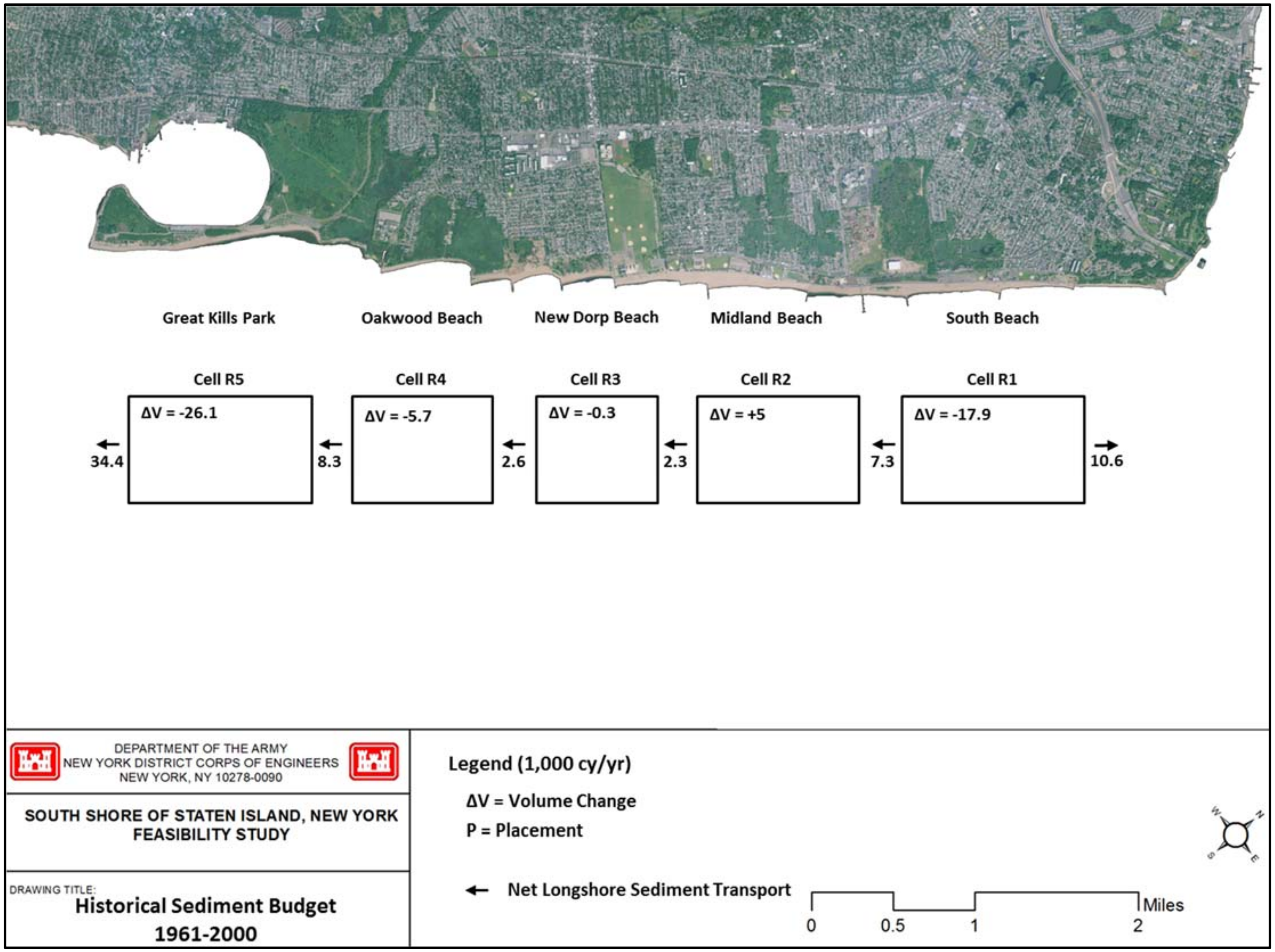


Figure 2-4: Historical Sediment Budget

Sea Level Change Impacts on Sediment Budget

The historic rate of sea level rise is incorporated in the historic shoreline change analysis. If the rate of sea level rise exceeds historical projections in the future, higher rates of shoreline erosion are anticipated. Cross-shore sediment losses due to sea level rise (SLR) are incorporated in the sediment budget based on Bruun (1962).

$$R = \frac{S}{\theta}$$

S = change in sea level

θ = average profile slope over active beach profile

R = horizontal recession of beach

The historic rate of SLR in the study area is +0.0128 ft/yr (NOAA Sandy Hook Tide Gauge). The average profile slope over the active beach profile, θ, was assumed as 1V:15H based on long profile surveys in the project area. Over the 20 year time period, commensurate with the time period used in the sediment budget, the sea level (S) will rise approximately 0.28 ft, which corresponds to a horizontal recession of the beach (R) of 4 ft. This 4-foot horizontal recession is equivalent to a volumetric loss of 2,800 cy/yr over the domain of the sediment budget. The impact of higher sea level rise rates is summarized below in Table 2-7. A detailed discussion of sea level rise is provided in Section 2.2.1.

Table 2-7: Sea Level Rise Impacts on Shoreline Changes and Sediment Budget

Sea Level Rise Scenario	SLR over 50-years (ft)	Shoreline Change (ft/yr)	Volume Loss (cy/yr)
USACE-Low (Historical)	0.7	-0.22	-2,800
USACE-Intermediate	1.1	-0.34	-4,400
USACE-High	2.4	-0.74	-9,500

Note: Historical SLR is included in historical sediment budget.

2.1.3 *Shoreline Stabilization and Stormwater Outfall Structures*

History

Since 1935, two federal and two state/city sponsored projects have been completed to stabilize and enhance Staten Island's south shore. Three of these were beach fill projects.

- In 1936-37, the federal government built 6 timber and rock groins, constructed a timber bulkhead, and placed an estimated 1,000,000 cubic yards of hydraulic fill at South Beach.
- The State and City placed about 1,880,000 cubic yards of fill between New Creek and Miller Field in 1955. The material, which consists of medium grained sand, was placed along the shore. Two concrete storm sewer outfalls that extend through the fill have acted as groins, helping to further stabilize the beach.

- The USACE constructed a project in 1999 to protect the Oakwood Beach area from Bay flooding. The project consists of two earthen levee segments, one tide gate structure, underground storm water storage, and road raising. The first levee segment, located south of the treatment plant and east of Oakwood Creek running parallel to the creek, has a top elevation of 10 feet NGVD. The second levee segment, located north of the treatment plant and running approximately northward and westward, is a raised road system with a top elevation varying between 7.9 ft NGVD to 8.4 ft NGVD. This project also consists of: (1) a new tide gate; (2) the raising of an access road at the northwestern area of the treatment plant property; and (3) underground storm runoff storage. The project is based on a 10 year economic life and protects against a 15-year storm (6.7% chance of occurring in any given year).

- After Hurricane Sandy (October 29-30, 2012) USACE awarded two repair contracts authorized under the Flood Control and Coastal Emergencies Act, PL 84-99 (USACE, 2003) that we completed in Fall 2013 to repair the levee and tide gate from damages inflicted by Hurricane Sandy.

- As part of other post-Sandy efforts, NYC initiated short term dune improvements as part of its Special Initiative for Rebuilding and Resiliency (SIRR) that included beach nourishment and dune construction along the study area in attempt to decrease future losses from coastal storm events. This program was completed in October 2013. Location and quantities of beach fill are pending.

In addition, the City of New York has constructed a significant number of outfalls structures to discharge stormwater runoff from streets and residential/commercial properties. Several of the outfall structures have been repaired and replaced over the past 50-years. The following structures have been identified by shoreline reach based on existing site photos and field surveys.

Fort Wadsworth to South Beach:

There are four groins in this reach. The structure (G1) is an approximately 220 ft long terminal groin approximately 220 ft long at the edge of the Verrazano Narrows. A revetment extends approximately 280 ft north and 300 ft west of the groin. A 400-ft long timber crib groin (G2) is located at the southern terminus of Lily Pond Avenue. A timber pile field (remnants of a recreational pier) and a submerged rubble structure are located on the updrift (west side) of G2. Structure G3 is a 10-ft wide by 6-ft high concrete box culvert that extends approximately 615 ft offshore of Sand Lane terminus. The downdrift (east) side of the structure is armored with large rock. The last structure within this reach, parallel drainage outfalls offshore seaward Ocean Breeze Park (G4), consists of two 10-ft wide by 6 ft-9" high concrete box culverts that extend 950 ft from Father Capodanno Boulevard.

Midland Beach:

Between Seaview Avenue and east side of Miller Field, there are four primary structures and two secondary structures that intersect the beach. The Seaview Avenue structure (G5) consists of two 800-ft long parallel 15-ft high by 6-ft high concrete box culverts encapsulated by armor stone. Just south of structure G5 are two small timber groins (approximately 150 and 200 ft, respectively) with armor terminal ends (G6 and G7). At the terminus of Naughton Avenue lie twin 10-ft wide by 6 ft-

6” high concrete box culverts that extend 875 ft from Father Capodanno Boulevard (G8). An approximately 1,370 ft long 8-ft wide by 4-ft high concrete box extends seaward from Father Capodanno Boulevard (G9) at Midland Avenue. Approximately 200-ft of the seaward end is protected by armor stone. The last structure (G10) within the reach is located on the eastern boundary of Miller Field and consists of twin 985-ft long, 15-ft high by 6 ft-3” high concrete box culverts. Armor stone is placed along approximately 350 ft of the seaward end.

New Dorp, Cedar Grove, and Oakwood Beaches:

This reach extends from the east side of Miller Field to the Oakwood Beach Wastewater Treatment Plant. Eight groin structures extend offshore of the shoreline; four of which contain stormwater outfall structures. In addition, the dune system fronting Oakwood has been enhanced with a riprap slope to protect this reach.

Structure G11, a single 450-ft long 13-ft wide by 5 ft-6” concrete box culvert, extends from the terminus of New Dorp Lane. The west side of the box culvert is protected by a wood bulkhead while armor stone has been placed on the seaward end. Structure G12 and Structure G13, located between New Dorp Lane and Cedar Grove Court, are non-emergent groins of 125 and 175 ft lengths, respectively. Structure G14, located at the terminus of Milbank Road, is an approximately 430-ft long groin structure with 125 ft of the seaward end submerged. A 10-ft by 5-ft concrete box culvert is incorporated into a 425-ft long groin structure (Structure G15) at Ebbitts Street. The outer end of G15 is protected by a pile field. Structure G16 bisects Cedar Grove Beach near the midpoint and consists of a 700-ft long rock groin with an internal 11-ft by 8-ft high concrete box culvert. Structure G17 is 790-ft long terminal groin consisting of a 590-ft section of timber training wall and a 200-ft section of submerged rock groin at the seaward end that effectively divides Cedar Grove Beach with Oakwood Beach. At the west end of Oakwood Beach is the last structure, G18, a combination 140-ft long concrete box culvert (140-ft long) and 165-ft long open top concrete flume that conveys return water from the Oakwood Beach Treatment Plant.

In addition to the structures, a dune system with a riprap revetment has been constructed along the Oakwood Beach segment. The reinforced dunes provide some local protection, but do not prevent storm surge inundation during large storms such as Hurricane Sandy.

Inundation and Flooding Areas

Flooding in the Project Area may occur from either storm surges or interior runoff. The high existing elevations along the shorefront in South Beach and Midland Beach provide protection from storm surge during small storms. Much of this reach is protected from storm surges until floodwaters rise above Father Capodanno Boulevard (+8 feet NGVD to +12 feet NGVD) or other areas of high ground. After the waters rise above that controlling elevation, large low-lying portions of inland areas become flooded, dramatically increasing flooding caused by rainfall runoff trapped landward of the high shoreline elevations.

Throughout the Project Area, more frequent localized flooding has been reported due to interior runoff, which becomes trapped by high tides or storm surges or is restricted by the capacity of the storm drainage system. The storm drainage system can convey flows only when the tides in Raritan and Lower New York Bay are below the interior flood elevations. When runoff and high tides occur at the same time, the runoff is unable to flow to the Bay. This situation results in flooding from the landward side of Father Capodanno Boulevard and is distinguished from storm surge flooding that results from elevated storm surges in Raritan and Lower New York Bays.

2.1.4 *Pedestrian/Vehicular Shoreline Access*

Boardwalk/Promenade

The Franklin D. Roosevelt (FDR) Boardwalk, which was originally constructed in 1935, stretches from Fort Wadsworth to Miller Field, a distance of approximately 2.5 miles. Approximately 1.5 miles of the boardwalk is a pile-supported wood boardwalk, with the last mile constructed at grade with an asphalt surface (promenade). The pile-supported section is approximately 40-ft wide with a deck elevation of 17 ft NGVD. The promenade section is also 40-feet wide with a 12-ft wide striped path. Recreational, concession, and restroom facilities are located on the north side of the boardwalk. The Ocean Breeze Pier, located just east of Seaview Avenue, is connected to the pile-supported section of the boardwalk.

Ramps/Dune Walkovers

Dedicated and maintained pedestrian and vehicular access to the beach is provided along the entire length of the FDR Boardwalk. Construction and maintenance vehicles access the beach on the east terminal end of the boardwalk using a 40-ft wide concrete ramp. A 15-ft wide concrete ramp is located on the west terminal end, where the wood boardwalk transitions to the at-grade promenade. The City of New York has a trash collection and maintenance facility at this location. Several vehicular access points are located along the at-grade promenade sections of the boardwalk including the City maintenance facility at Jefferson Avenue, the parking lot at Lincoln Avenue, and at the west loop section of Father Capodanno Boulevard. Vehicular access for maintenance is also provided on the west side of Miller Field at New Drop Lane, at Cedar Grove Lane, and Tarlton Street.

Pedestrian access, consisting of pile supported wood ramps, is provided along the entire length of the elevated boardwalk. These 8-ft wide ramps, spaced every 250 to 500 feet, also support ATV access for park and law enforcement personnel. Pedestrians can also access at-grade sections of the boardwalk at several City-owned parking lots (Iona Street, Jefferson Avenue, and Lincoln Avenue). The federal park at Miller Field provides pedestrian entry via parking lots at Cedar Grove Avenue and New Dorp Lane. Entrance to New Dorp Beach is via an asphalt and wooden boardwalk from the parking lot Cedar Grove Lane and Center Place.

At Cedar Grove Beach, the public can enter the beach by parking at terminal end of Cedar Grove Avenue and walking directly onto beach. At Kissam Avenue and Tarlton Street, makeshift paths have been created to gain entry to the beach.

2.2 Meteorological and Oceanographic Conditions

2.2.1 *Water Levels*

Astronomical Tides

Tides along the Project Area are semi-diurnal and have a mean range of 4.6 ft at Fort Wadsworth. Tidal datum relationships at Fort Wadsworth are presented in Table 2-8.

Table 2-8: Tidal Datum Relationships – Fort Wadsworth

Tidal Datum	ft, NGVD
Mean Higher High Water (MHHW)	3.5
Mean High Water (MHW)	3.2
North American Vertical Datum of 1988 (NAVD)	1.1
Mean Tide Level (MTL)	0.9
National Geodetic Vertical Datum of 1929 (NGVD)	0.0
Mean Low Water (MLW)	-1.4
Mean Lower Low Water (MLLW)	-1.6

Notes: Tidal datums based on NOAA's VDATUM 1983-2001 Epoch

Sea Level Change

By definition, sea level change (SLC) is a change (increase or decrease) in the mean level of the ocean. Eustatic sea level rise is an increase in global average sea level brought about by an increase to the volume of the world's oceans (thermal expansion). Relative sea level change takes into consideration the eustatic increases in sea level as well as local land movements of subsidence or lifting. Historic information and local MSL trends used for the Study Area are provided by the NOAA/NOS Center for Operational Oceanographic Products and Services (CO-OPS) using the tidal gauge at Sandy Hook, New Jersey. The historic sea level change rate (1935-2013) is approximately 0.0128 ft/year or about 1.3 ft/century.

Recent climate research has documented observed global warming for the 20th century and has predicted either continued or accelerated global warming for the 21st century and possibly beyond (IPCC 2013). One impact of continued or accelerated climate warming is continued or accelerated rise of eustatic sea level due to continued thermal expansion of ocean waters and increased volume due to the melting of the Greenland and Antarctic ice masses (IPCC, 2013). A significant increase in relative sea level could result in extensive shoreline erosion and dune erosion. Higher relative sea level elevates flood levels, which may result in smaller, more frequent storms that could result in dune erosion and flooding equivalent to larger, less frequent storms.

The current guidance (ETL 1100-2-1 dated 30 Jun 2014) from the Corps states that proposed alternatives should be formulated and evaluated for a range of possible future local relative sea level change rates. The relative sea level rates shall consider as a minimum a low rate based on an extrapolation of the historic rate, and intermediate and high rates which include future acceleration of the eustatic sea level change rate. These rates of rise correspond to 0.7 ft, 1.1 ft, and 2.4 ft over

50 years for the low, medium and high rates of relative sea level rise. The historic rate, 0.7 ft, is being used as the basis of design for the flood protection structures. However, a sensitivity analysis is performed to the medium and high SLC rates in the economic analysis.

Storm Surge

Two types of storms are of primary significance along the south shore of State Island: (1) tropical storms, which typically impact the New York area from July to October, and (2) extratropical storms, which are primarily winter storms occurring from October to March. These storms are often referred to as “nor’easters” due to the predominate direction from which the winds originate. Storm surge is water that is pushed toward the shore by the force of the winds, the decrease in astronomical area pressure during major storms, and other localized effects. Water levels rise at the shoreline when the motion of driven waters is arrested by the coastal landmass.

Stillwater elevations for the Project Area were obtained from preliminary FEMA Flood Insurance Study (FIS) results (FEMA, 2013) as shown in Table 2-9. The coastal study project area for the modeling study includes New Jersey, New York City, Westchester County, NY, and the banks of the tidal portion of the Hudson River. A region wide storm surge modeling study was performed by FEMA (2011) using the Advanced Circulation Model for Oceanic, Coastal and Estuarine Waters (ADCIRC) which was dynamically coupled to the unstructured numerical wave model Simulating Waves Nearshore (unSWAN). Synthetic tropical and extra-tropical storms were generated based on parametric models and historical data. The numerical modeling results from the synthetic storms are used to determine still water frequency of occurrence relationships. The model results were extracted offshore of the Project Area in the center of Lower New York Harbor (74°4’57.48”W, 40°30’9.74”) as shown in Table 2-9.

Table 2-9: Stillwater Elevations for Project Area (FEMA, 2013)1

Return Period (yr)	Still Water Level (ft NGVD)
2	5.3[2]
5	7.2
10	8.5
25	10.0
50	11.3
100	12.6
200	14.0
500	15.9

Notes: [1] Stillwater elevations obtained from FEMA (2013)
[2] Stillwater elevation for the 2-year event are obtained from Dredged Material Management Plan (DMMP) Study (WES, 1998)

Prior to the completion of the Interim Feasibility Study for Fort Wadsworth to Oakwood Beach, the USACE Coastal and Hydraulics Laboratory (CHL) released the storm surge modeling results associated with the North Atlantic Coast Comprehensive Study (NACCS). A comparison of the predicted stillwater elevations was performed to determine if there were any significant differences in the predicted stillwater elevations during extreme storm events. As shown in Figure 2-5, the FEMA FIS and NACCS stillwater levels are in agreement in the study area. Therefore, plan selection and project performance would not be affected by adopting the NACCS stillwater levels instead of the FEMA FIS stillwater levels.

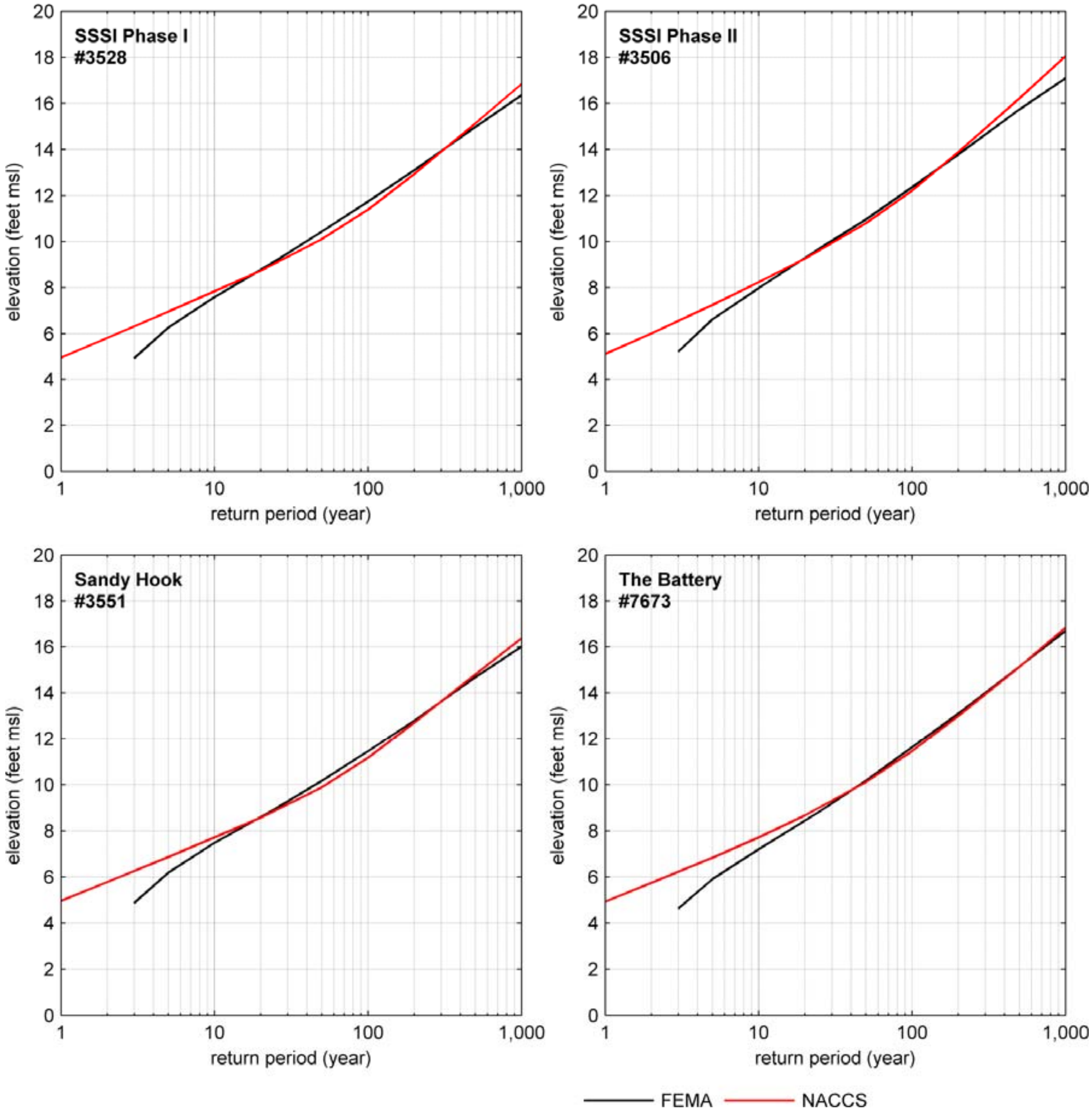


Figure 2-5: Comparison of FEMA and NACCS Stillwater Elevations

<u>Wave Setup</u>

Wave setup is an additional increase in the water elevation near the shoreline due to the transfer of wave momentum from the breaking waves to the water column. Wave setup for this study was calculated using small-amplitude wave formulas outlined in the CEM (1999) based on the breaking wave height (described below). The wave setup estimates represent the theoretical maximum wave setup near the instantaneous shoreline on a relatively steep beach (1V:10H). In reality water levels during the storm event may cause wave breaking to occur over a gentler slope reducing the breaking wave height and subsequent wave setup height. Therefore, the estimated wave setup values below are conservative estimates of the maximum possible wave setup. Table 2-10 shows the wave setup vs. frequency of occurrence relationships for Project Area.

Wave setup is not directly used anywhere in the design or analysis of the proposed Coastal Storm Risk management structures. Wave setup is indirectly included in the wave transformation calculations (i.e. Goda) and subsequent wave overtopping calculations. Wave setup is included here to provide an understanding of how the water levels increase along the shoreline due to wave breaking.

Table 2-10: Wave Setup Height for Project Area

Return Period (yr)	Wave Setup (ft)
2	1.7
5	2.1
10	2.3
25	2.4
50	2.6
100	2.7
200	3.0
500	3.1

2.2.2 *Currents*

Tidal currents in the study area are generally weak. Longshore wave driven currents are limited due to the moderate wave heights and orientation of the shoreline that is generally perpendicular to the dominant wave direction. Table 2-11 lists the tidal currents in the Project Area (NOAA, 2001) which are representative of conditions throughout the Project Area.

Table 2-11: Tidal Current Velocities (NOAA 2001)

Station	Peak Flood (knots)	Peak Ebb (knots)
Hoffman Island	0.9	0.8
Midland Beach	0.8	1.3
New Dorp Beach	0.5	0.5

2.2.3 *Wind*

Measured wind speeds and direction have been recorded at the Ambrose Light Station (ALSN6), which is located approximately 16 miles offshore of the South Shore of Staten Island and is well situated to measure wind speeds over open water. The annual wind rose at Ambrose Light House is presented in Figure 2-7.

2.2.4 *Waves*

The wave climate in Lower New York Harbor is comprised of a mixture of longer period swells that propagate from the New York Bight into the Harbor and locally wind-generated sea conditions. The complex wave conditions during large storm events are described by Coastal Engineering Research Center (CERC, 2001):

> *Wave energy concentrates over the shoals in the approach and entrance to Lower New York Harbor. Wave height drops steadily over the shoals as shallow water-induced wave breaking and energy dissipation continue to impact the waves entering Lower New York Harbor… Storm wave response in Lower New York Harbor involves several additional complications. The shoals at the entrance to Lower New York Harbor have controlling depths of 10 to 15 ft MLLW. The effect of this "gate" depends on the incident wave conditions, astronomical tide, and storm-generated water levels. Sites in Lower New York Harbor are also exposed to local fetches which, coupled with strong storm winds, can result in locally-generated waves of concern. Local storm wave conditions depend strongly on wind speed and direction. Local wave generation may restore some energy to incident ocean waves which have broken on entering Lower New York Harbor or, if wind direction differs significantly from incident wave direction, generate a new wave component from another direction.*

A wave hindcast and wave transformation study of the waves in New York Bight and Lower New York Harbor was performed by the CERC in support of the Dredged Material Management Plan for New York Harbor (CERC, 2001). The storm wave results were based on offshore wave hindcast data transformed to the bay using the STWAVE numerical wave model.

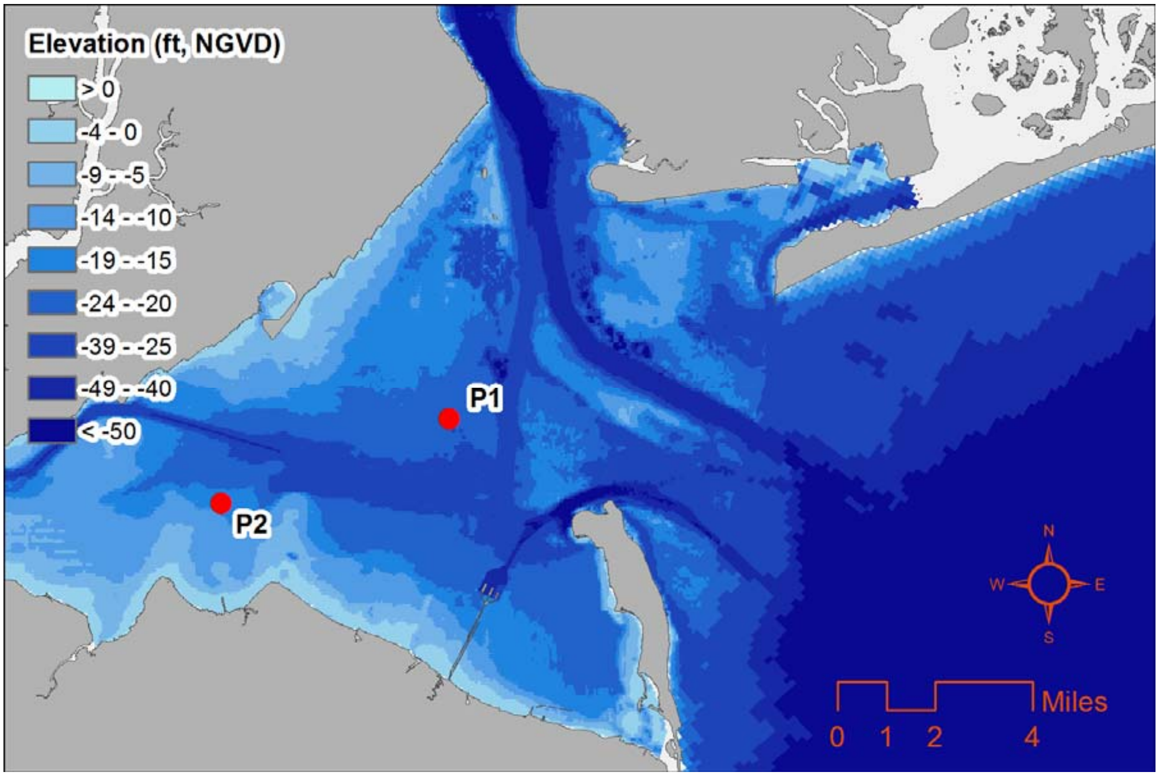


Figure 2-6: DMMP Stations

A total of 20 extratropical storms and 20 tropical storms were simulated in STWAVE. The extratropical storms were selected based on the data at WIS Station 73, near the STWAVE ocean boundary. Tropical storms were not sufficiently present in the WIS database, so special WIS model runs were made to generate wave information for 20 selected events. A detailed list of the storm events in the storm suite are contained in Attachment D "DMMP STWAVE Modeling". Extreme wave events were simulated using the Empirical Simulation Technique (EST). The input to the EST analysis was the results of the wave model storm simulations with variable water levels (tide and surge).

The results of the wave transformation study and EST analysis provide the basis for the design wave conditions. The spectral significant wave height, H_{m0}; peak wave period, T_p; and frequency of occurrence relationships were developed by CERC (2001) at several locations in the Study Area. The mean frequencies of occurrence relationships at Station P1 (or Station #8) define the wave characteristics for Project Area. The nominal depth at Station P1 is 17 ft (MSL).

Thompson & Vincent (1985) found that energy based wave height (H_{m0}) deviates from the statistical wave height ($H_{1/3}$) in shallow water prior to breaking. In deep water and inside the surf zone after wave breaking H_{m0} and $H_{1/3}$ are nearly equal. The model results for the study were extracted in relatively shallow water (17 ft) and are therefore converted to the statistical significant wave height, $H_{1/3}$, following Figure II-1-40 of the Coastal Engineering Manual (USACE, 1999) which was originally developed by Thompson & Vincent (1985). The significant wave height (H_s) is a general term that may be used to describe either H_{m0} or $H_{1/3}$. However, in this study significant wave height stands for $H_{1/3}$.

The equivalent un-refracted deep water wave height, H'_0, which had undergone wave refraction and diffraction, was calculated from H_s based on small amplitude wave theory.

The maximum possible breaking wave height and depth at which it breaks are estimated based on Figure 2-72 and Figure 2-73 in the Shore Protection Manual (USACE, 1984) based on a representative nearshore slope of 1 on 10. The offshore and breaking wave characteristics are representative of the nearshore wave conditions in Lower New York Harbor assuming a uniform nearshore slope. The nearshore slope along Project Area exhibits less uniformity and may be steeper or gentler, affecting the breaking wave conditions at these locations.

A summary of the nearshore, offshore, and wave breaking characteristics for Project Area are presented in Table 2-13.

Table 2-12: Nearshore Wave Conditions (CERC, 2001)

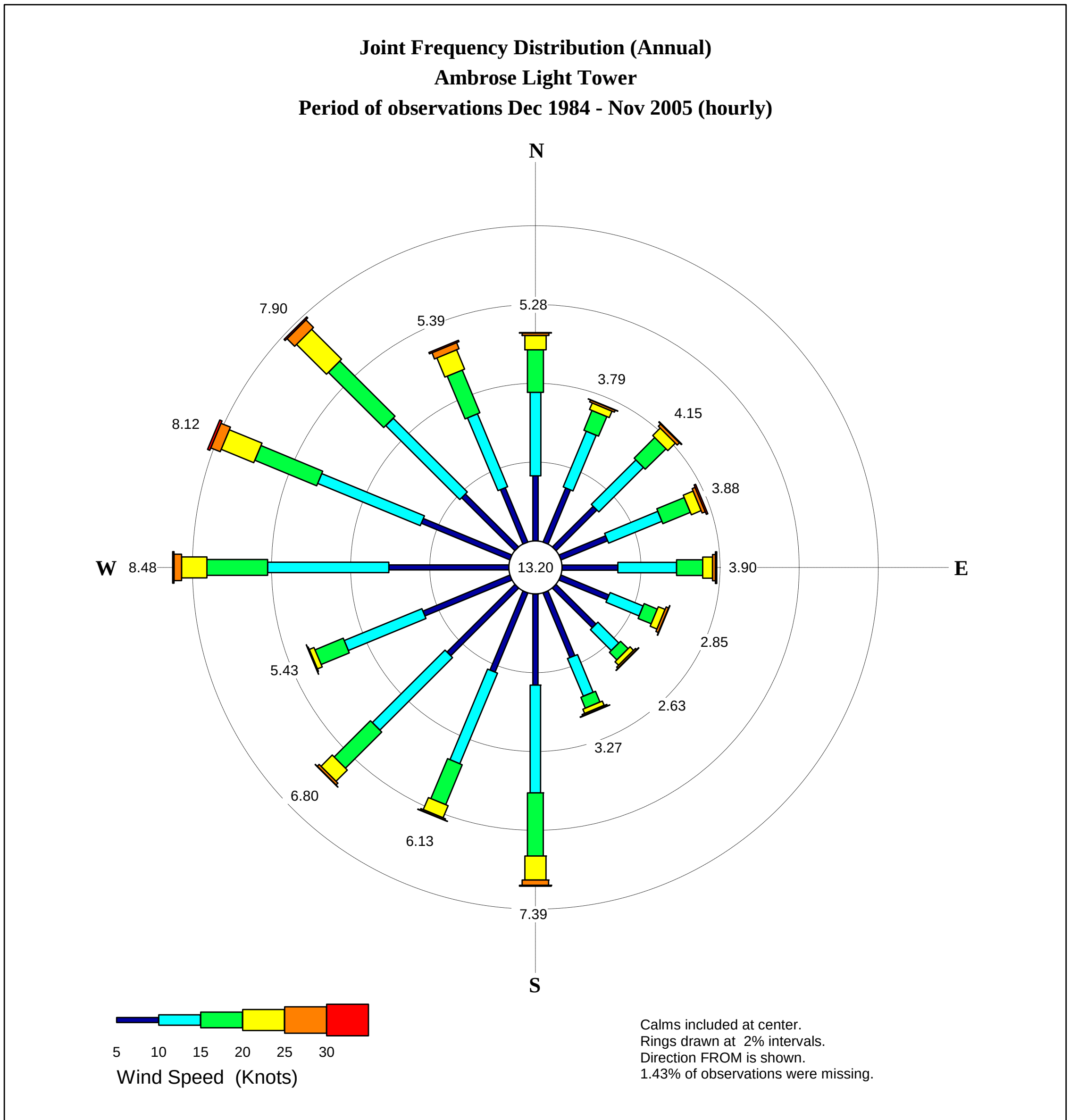
Return Period (yr)	Peak Wave Period (s)	H_{m0} (ft)	H_s (ft)
2	5.4	5.2	5.8
5	8.3	5.4	6.5
10	9.7	5.6	7.1
25	11.3	5.7	7.5
50	12.3	5.8	7.9
100	13.2	6.0	8.4
200	14.5	6.2	9.0
500	16.0	6.5	9.7

Notes: [1]Nominal depth of nearshore wave station is 17 feet (MSL)

Table 2-13: Offshore and Breaking Wave Characteristics

Return Period (yr)	H'0 (ft)	H_b (ft)	d_b (ft)
2	6.3	7.9	7.5
5	6.3	9.8	8.1
10	6.5	10.4	8.4
25	6.5	11.2	8.8
50	6.6	11.9	9.4
100	6.8	12.6	9.8
200	7.0	13.6	10.6
500	7.2	14.4	11.1

Note: H'0 is the equivalent un-refracted deep water wave height
H_b is the maximum possible breaking wave height
d_b is the water depth at which wave breaking occurs



PERCENT OCCURRENCE: Wind Speed (Knots)
LOWER BOUND OF CATEGORY

DIR	5	10	15	20	25	30
N	1.65	2.12	1.07	0.37	0.06	0.01
NNE	1.49	1.52	0.54	0.18	0.05	0.01
NE	1.46	1.59	0.74	0.26	0.08	0.02
ENE	1.27	1.45	0.75	0.27	0.08	0.05
E	1.41	1.48	0.66	0.25	0.07	0.04
ESE	1.31	0.91	0.36	0.19	0.07	0.01
SE	1.43	0.75	0.27	0.12	0.05	0.01
SSE	1.77	1.03	0.30	0.12	0.04	0.01

TOTAL OBS = 179219 MISSING OBS = 2554

PERCENT OCCURRENCE: Wind Speed (Knots)
LOWER BOUND OF CATEGORY

DIR	5	10	15	20	25	30
S	2.30	2.73	1.60	0.61	0.14	0.02
SSW	2.17	2.48	1.09	0.33	0.04	0.01
SW	2.43	2.60	1.30	0.40	0.07	0.01
WSW	2.38	2.12	0.77	0.13	0.02	0.01
W	3.03	3.06	1.53	0.64	0.19	0.03
WNW	2.42	2.80	1.69	0.89	0.26	0.05
NW	1.90	2.64	1.98	1.07	0.27	0.04
NNW	1.49	1.99	1.17	0.53	0.17	0.03

CALM OBS = 23657 PERCENT CALM = 13.20

Figure 2-7: Annual Wind Rose at Ambrose Light

2.2.5 *Interior Flooding*

Interior flooding from Fort Wadsworth to Oakwood Beach results from high Bay storm surges and interior runoff that cannot be conveyed to the Bay by the storm drainage system. Once flood waters rise above high ground along the shoreline, large low-lying inland areas are flooded, supplementing flooding caused by rainfall runoff trapped landward of the high shoreline elevations.

It is expected that future storms will continue to cause damage in this area. Since no major changes to the shorefront are expected, the magnitude and frequency of coastal flooding is expected to increase with sea level rise. It is also expected that continued development will occur in the floodplain as new construction is elevated above the base interior flood elevation. This fill would reduce storage of interior runoff and thereby exacerbate interior flooding conditions.

2.3 Storm History

<u>Hurricane Donna (September 1960)</u>

Prior to Hurricane Donna, an artificially filled beach and promenade was constructed between Miller Field and Fort Wadsworth. In addition, Seaside Boulevard (Father Capodanno Boulevard) was raised from Miller Field to the vicinity of Burgher Avenue (approximately half of the distance to Fort Wadsworth). During Hurricane Donna, these projects were very effective in protecting the many dwellings located inshore of the beach. Tidewaters and waves did, however, break through under the boardwalk and across the old road at the point where the new boulevard ends. Foam-capped breakers reportedly soared 50 ft or more in the air between South Beach and Midland Beach. The beach was also breached at Sand Lane to the east and around the end of the boardwalk near Fort Wadsworth, inundating Seaside Boulevard up to a depth of 3 ft.

In the community of Oakwood Beach, tide gates at a wastewater treatment plant flume at the south end of a protective sand dike failed to operate and tidewater began to flow into the streets. As the tide and wave action increased, the dike was flanked at the breach near the center. Twenty-five families were forced to leave the area when their homes were inundated.

In New Dorp Beach, the grounds of the Seaside Nursing Home were flooded up to the steps of the main building, but damages were confined to clean-up operations. The streets of the residential area were flooded about 500 ft inland. From the Ocean Edge Colony, along New Dorp Lane to Cedar Grove Beach, residents and Fire Department crews reportedly pumped water from the streets. Cedar Grove Avenue was impassable due to flooding.

Miller Field suffered damage when tidewater entered through the former New Dorp Avenue gate and flooded grounds, hangars and some buildings at the southeast end of the field.

<u>December 1992 Nor'easter</u>

During this storm, flood levels ranged from 8.4 to 10.6 ft NGVD between Fort Wadsworth and Miller Field. Nearly 2,000 structures within this area are at ground elevations at or below the average elevation of floodwaters recorded during this event.

The December 1992 storm caused the partial collapse of 22 bungalows at Cedar Grove Beach. Since that time, 26 bungalows at the western end of the beach have been demolished by New York City, and a dune was constructed in their place. The New York City Department of Parks and Recreation is reviewing the future uses for this area and the future of the remaining bungalows.

At Oakwood Beach the artificial dune system, located on New York City property, was breached in the 1992 storm. This occurred at Kissam Avenue, creating a breach in the dune up to 175 yards wide. In addition, prior to the completion of the USACE project in 1999, the Oakwood Beach area was open on its western flank to the low lands around the sewage treatment plant and Great Kills Park. Large areas along Fox Lane and Kissam Avenue were flooded with depths up to 5 ft. Remedial action has been planned and implemented by local authorities to remove debris in the watercourse, repair the sewer system and reconstruct the dune. As previously described, a short-term plan of protection was implemented under the Corps of Engineers Continuing Authority Program to protect Oakwood Beach residents from inundation from the western flanked area.

As a result of this storm, 225 flood claims totaling almost $2 million were paid out from the National Flood Insurance Program (NFIP).

Hurricane Sandy (October 2012)

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, NY peaked at 12.4 ft NGVD, exceeding the previous record by over 4 ft.

The south shore of Staten Island was one of the hardest hit areas by Hurricane Sandy. USGS deployed storm tide sensors and surveyed high water marks after the storm that indicated the maximum water levels during Sandy were likely between 12.5 and 13.6 ft, NGVD within the project area (USGS, 2013). Figure 2-8 shows a time series of the recorded water levels at Fort Wadsworth (RIC-001WL), The Battery, and surveyed USGS high water marks. It is noted that high water marks may be higher than still water levels just offshore of the shoreline since they may include an additional increase in the water level from wave setup or wave runup.

An overview of the extent of flooding in the project area is shown in Figure 2-9. Storm surge and waves devastated low-lying neighborhoods in the project area. At Kissam Avenue (Oakwood Beach) many homes were swept off of their foundations or flattened (Figure 2-10). Floodwaters rose rapidly in many neighborhoods in the Project Area once storm surge elevations exceeded the elevation of Father Capodanno Boulevard and other high spots creating a "bowl" that trapped water in some areas for several days. Figure 2-11 shows the damage to homes located along Cedar Grove Avenue (New Dorp Beach) that are located 700 ft landward of the shoreline.

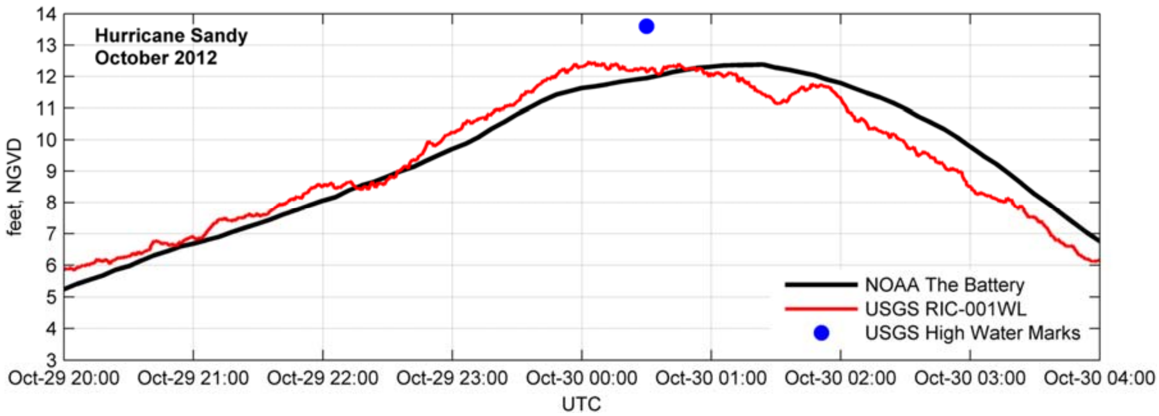


Figure 2-8: Hurricane Sandy Storm Tide and High Water Marks

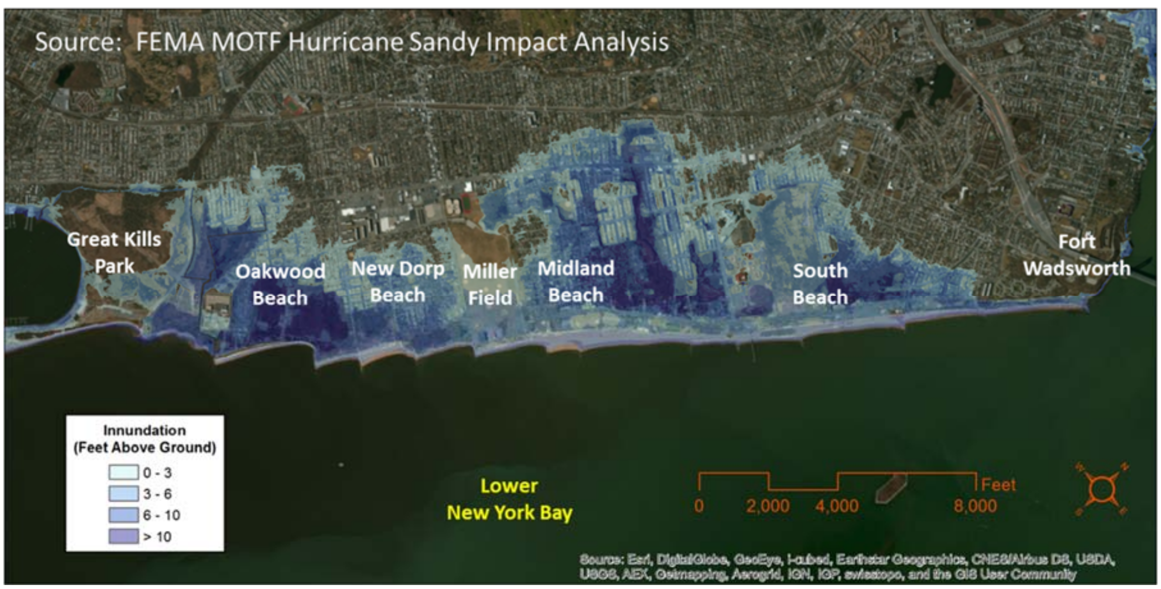


Figure 2-9: Hurricane Sandy Flood Inundation

Figure 2-10: Hurricane Sandy Damage at Kissam Avenue (Oakwood Beach)

Figure 2-11: Hurricane Sandy Damage at Cedar Grove Avenue (New Dorp Beach)

3.0 WITHOUT PROJECT COASTAL PROCESSES

3.1 Overview

The water level and wave setup vs. frequency of occurrence relationships provided in Table 2-9 and Table 2-10 were applied to assess the without project conditions. Future without project conditions included 0.7 ft of sea level rise.

The wave height vs. frequency of occurrence relationships presented in Table 2-12 and Table 2-13 were applied to assess the without project conditions. An appropriate wave height was selected from one of these tables depending on the application. A brief summary of the wave height selected for each coastal process application is given below:

- Wave Runup was calculated using un-refracted offshore wave height, H'_0;
- Mean Wave Overtopping was calculated using the nearshore significant wave height, H_s;

3.2 Storm Induced Shoreline Change

Storm induced shoreline changes were investigated for the project area using the Storm-Induced Beach Chance Model (SBEACH). SBEACH is a one-dimensional model, developed by the United States Army Corps of Engineers (USACE), which simulates cross-shore erosion of beaches, berms, and dunes under storm water levels and waves. A basic assumption of SBEACH is that all profile change is produced by cross-shore processes, with no net gain or loss of sediment. This is only true if longshore sediment transport processes are uniform, which is typical considered a reasonable assumption during storm events on open coasts away from inlets and structures. Long-term morphologic changes (e.g. shoreline erosion) are typically controlled by longshore sediment processes, which are not simulated by SBEACH. Shore-perpendicular structures (i.e. groins and outfalls) within the project area have historically had a large impact on the longshore coastal processes but the sediment budget the relative stability of the beach and shoreline positions over the last 50 years indicates that the shoreline has reached a dynamic equilibrium and the shore-perpendicular structures are no longer actively causing shoreline retreat or advancement.

The SBEACH model calculates beach profile change using an empirical morphologic approach with emphasis on beach and dune erosion. In model simulations, the beach profile progresses to an equilibrium state based on the initial profile, median grain size, and storm conditions (wave height, wave period, wave condition, wind speed and direction, and water level). The model also simulates overwash and dune lowering.

Six profile transects were selected to capture the variability in the beach conditions. The profile topography (above MHW) was obtained from the Post-Sandy 2012 LIDAR data. The submerged portion of the profile was based on the February 2000 beach profile surveys. Figure 3-1 shows the six SBEACH profile transects. The alongshore spacing of the SBEACH profile transects was selected based on the alongshore variability in the beach conditions and availability of submerged profile data. The beach conditions from Fort Wadsworth to Midland Beach are fairly uniform and two profiles are adequate to capture the range in beach conditions. More detail about the uniformity of the beach conditions in the study area is provided in (Figure 5-2, Section 5.3).

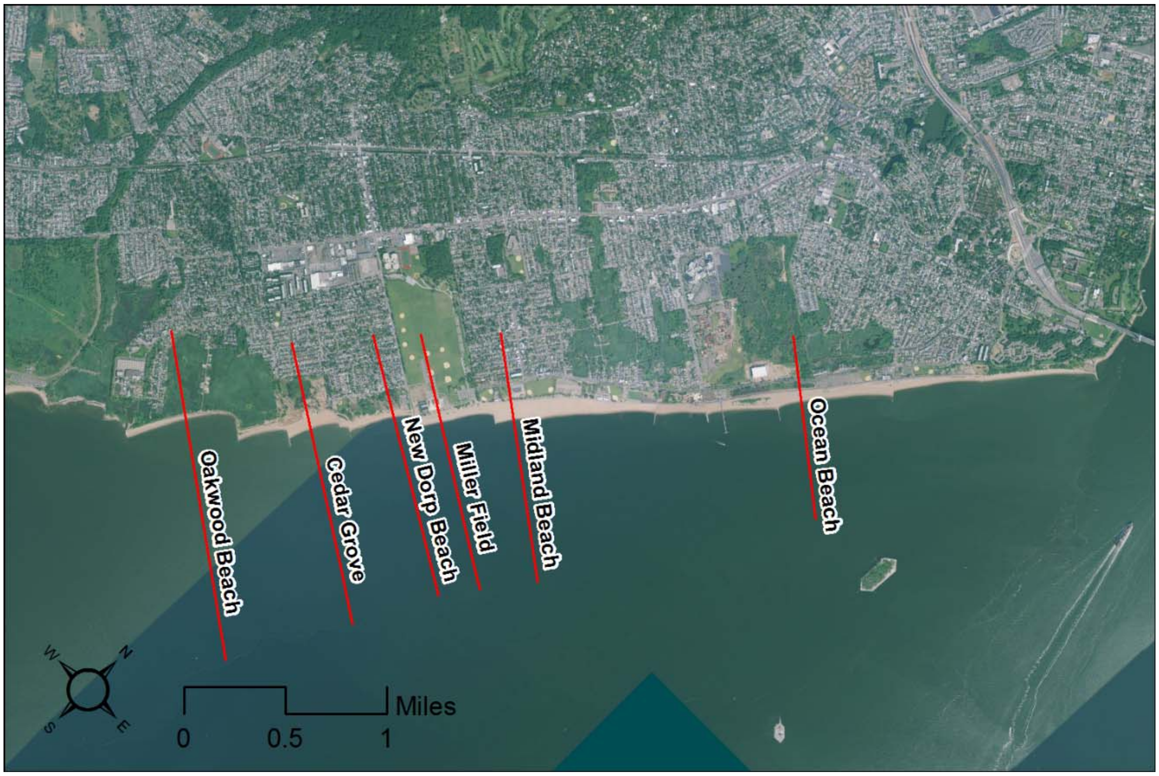


Figure 3-1: SBEACH Profile Lines

3.2.1 *Hurricane Sandy SBEACH Simulations*

The model has been verified based on field measurements at Duck, NC; Manasquan, NJ; Point Pleasant, NJ; and Torrey Pines, CA (Larson and Kraus 1989a, Larson et al. 1989b). However, it is still recommended that site specific model calibration be performed. No suitable data was available in the Project Area to calibrate the model. The available topographic data, 2001 and 2012, is too far apart to be used for model calibration. A sensitivity analysis and qualitative model validation were performed for Hurricane Sandy based on the available topographic data in 2001 and 2012. Figure 3-2 presents the results of the SBEACH simulations at New Dorp Beach, capturing the erosion of sandy dune that occurred during Hurricane Sandy (Figure 3-3). The model sensitivity analysis shows that model is able predict the erosion of the sandy dune. The SBEACH parameters applied in the modeling study are shown in Table 3-1. One of the most important parameters is the effective grain size, taken for this study as the mean grain size, 0.5 mm.

A comparison of the observed profile changes from 2001 to 2012 and modeled profile changes from Hurricane Sandy are presented in Figure 3-4. The observed results show in general very little beach change and in some cases accretion. This result is not unexpected since some areas of the beach may have experienced accretion over the 11-years, which may obscure the impact of Hurricane Sandy. The modeled changes during Hurricane Sandy overall show some erosion of the steep foreshore and erosion of any upland dunes (i.e. Miller Field and New Dorp Beach).

Table 3-1: SBEACH Parameters

SBEACH Parameter	Value
Landward surf zone depth (m)	0.3
Effective grain size (mm)	0.5
Maximum slope prior to avalanching (deg)	30
Transport rate coefficient (m^4/N)	1.75 e^{-6}
Overwash transport parameter (K_B)	5 e^{-3}
Coefficient for slope-dependent term (m^2/S)	2 e^{-3}
Transport rate decay coefficient multiplier	0.5

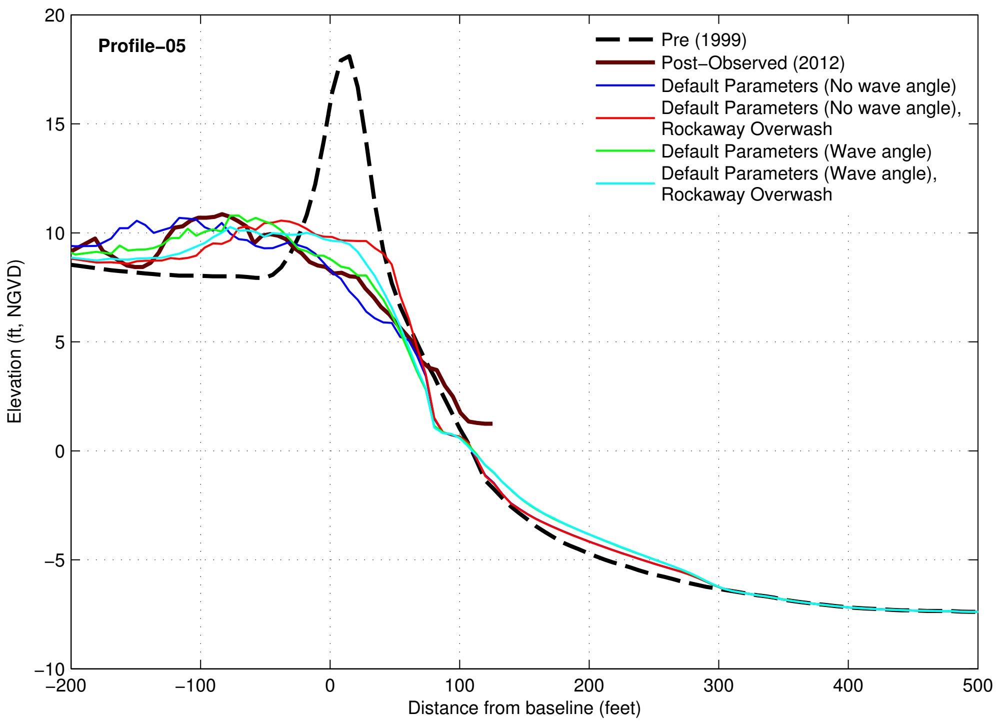


Figure 3-2: SBEACH results from Hurricane Sandy simulation (New Dorp Beach)

Figure 3-3: Pre- and Post-Hurricane Sandy Photos at New Dorp Beach

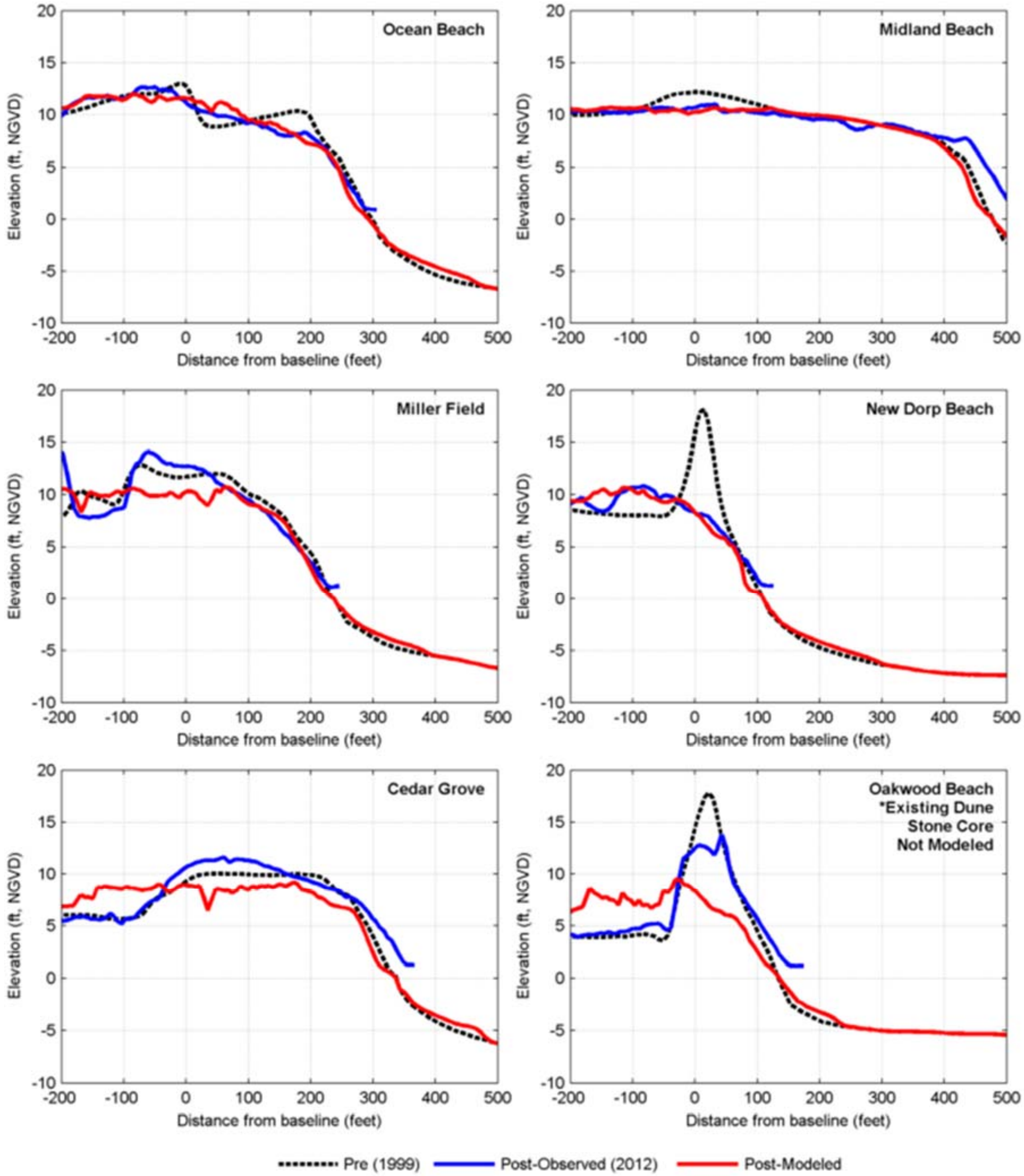


Figure 3-4: SBEACH results from Hurricane Sandy simulations

3.2.2 *Synthetic Storm SBEACH Simulations*

SBEACH simulations were performed for five synthetic storm events representing the 2, 10, 25, 100, and 500 year return periods. The synthetic storm events were developed following the same approach applied by Larson & Kraus (1989b) and utilized mathematical functions to represent the evolution of the surge during hurricanes and nor'easters. The basic premise is that Hurricanes have relatively short storm surge durations when compared to nor'easters. The 2, 10, and 25 year events were simulated as Nor'easters and the 100 and 500 year events were simulated as Hurricanes. Synthetic surge and wave boundary conditions were developed such that the maximum still water level and maximum wave height and wave period corresponded to the design conditions as described above and shown in Table 3-2. Figure 3-5 presents an example of the boundary conditions for a Nor'easter and Hurricane.

Table 3-2: SBEACH Boundary Conditions

Return Period (years)	Still Water Level (ft, NGVD)	Peak Wave Period (s)	Significant Wave Height (ft)
2	5.3	5.4	5.8
10	8.5	9.7	7.1
25	10.0	11.3	7.5
100	12.6	13.2	8.4
500	15.9	16.0	9.7

The Without-Project Conditions were defined by the post-Sandy (2012) ground elevations. A total of 6 profile locations were simulated in SBEACH.

The resulting eroded profiles for the "without project" conditions can be found in Attachment A. Table 3-3 and Table 3-4 present the maximum horizontal shoreline and dune/berm recession calculated in SBEACH, respectively. Shoreline recession is presented as the maximum horizontal recession of the +3.2 ft (NGVD) elevation contour which corresponds to Mean High Water (MHW). Dune/berm recession is presented relative to the 8 ft (NGVD) elevation contour. In general the model results indicate that shoreline recession is between 10 and 20 ft during storm events. Berm and dune recession is more dependent on the local profile conditions and varies along the Project Area accordingly. However, in general berm recession is between 10 and 60 ft during the largest of the storm events. Many of the profiles indicated that very little berm recession occurs during smaller storm events. As shown in previous shoreline evaluations, the South Shore of Staten Island shoreline is relatively stable due to its relatively coarse sand (0.5 mm) and relatively mild wave conditions in comparison to open ocean coastlines.

Table 3-3: Without-Project Shoreline Recession (ft)

Return Period (Years)	Oakwood Beach	Cedar Grove	New Dorp Beach	Miller Field	Midland Beach	South Beach
2	11	18	15	13	13	12
10	12	15	15	13	15	12
25	13	18	17	15	15	15
100	17	18	17	17	18	17
500	18	20	18	16	19	16

Table 3-4: Without-Project Dune/Berm Recession (ft)

Return Period (Years)	Oakwood Beach	Cedar Grove	New Dorp Beach	Miller Field	Midland Beach	South Beach
2	2	0	1	1	0	2
10	5	0	2	0	1	38
25	104	0	21	0	13	41
100	BC	1	23	5	16	45
500	BC	22	37	12	33	59

Notes: BC - Below contour. Entire profile was below the 8 ft contour at some point in the simulation.

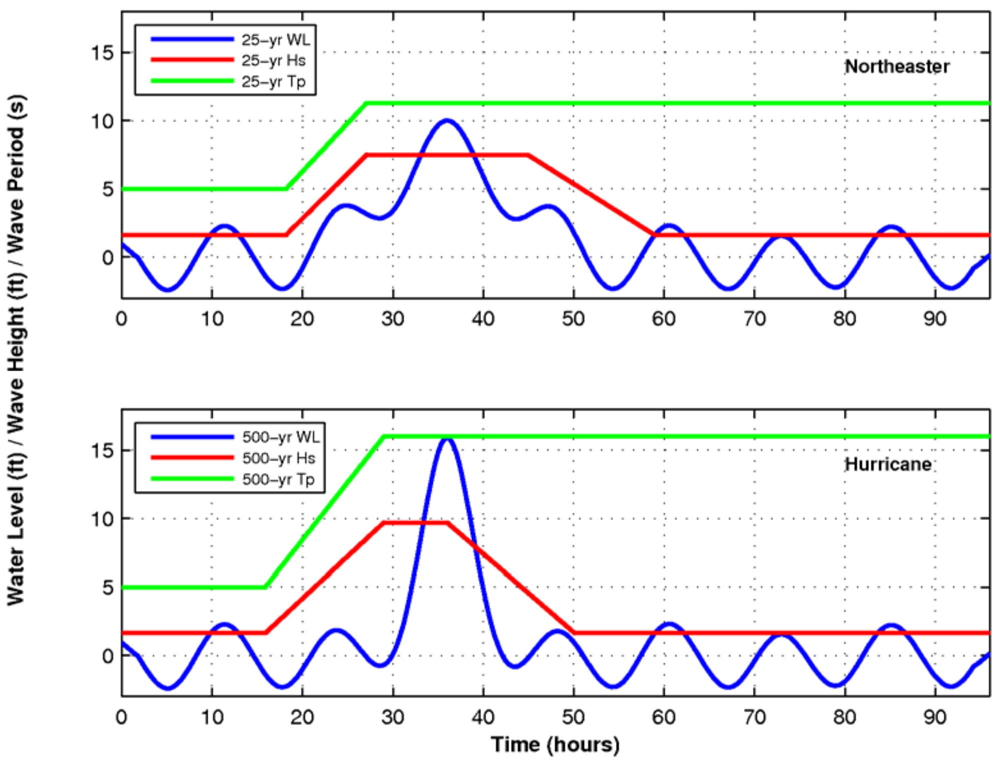


Figure 3-5: Synthetic Hurricane and Nor'easter Boundary Conditions

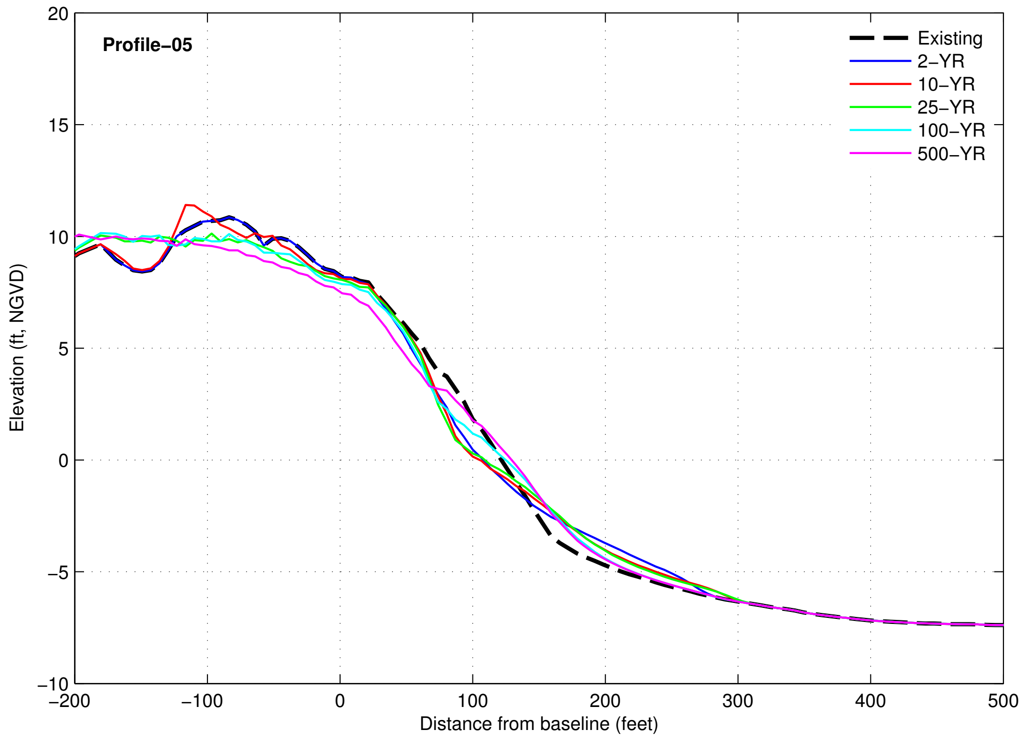


Figure 3-6: SBEACH Without-Project Results for New Dorp Beach

3.3 Wave Runup

Wave runup is the maximum elevation of wave uprush above the still water level. Wave runup consists of two components: wave setup and wave uprush (swash) fluctuations about that mean water level. The mean wave runup and 2% wave runup height were determined for the Project Area using the formulas given by USACE (1999) for irregular wave runup on plane, impermeable beaches.

Table 3-5: Wave Runup – Without Project Conditions

Return Period (Years)	Mean Wave Runup (ft)	2% Wave Runup (ft)
2	3.4	7.0
5	4.5	9.5
10	5.2	10.9
25	5.7	12.1
50	6.1	13.0
100	6.6	13.9
200	7.2	15.2
500	7.8	16.6

Notes: Wave runup values are heights and reported relative to the still water level.

3.4 Wave Overtopping

The wave overtopping rate, q, reported in this study is the mean overtopping discharge (liters/s/m). In actuality, wave overtopping occurs in sporadic short pulses and is not constant over time. It is coastal engineering practice to use mean wave overtopping rates in engineering applications due to the ability to easily measure the rates in laboratory studies.

Wave overtopping is generally classified into two types: “green water,” where complete sheets of water run up the face of dune/structure and over the crest of the dune, and “white water,” where spray from wave breaking is carried over the structure. The first type of overtopping, “green,” will only occur when wave runup exceeds the crest elevation of the dune/structure. The second type of overtopping, “white,” may occur even if the runup elevations are not greater than the crest elevation.

Due to the topography of the project area, wave overtopping is only relevant at one location along the project area, Oakwood Beach. At this location a dune and riprap slope provides limited protection to the low-lying marsh area farther inland. Elsewhere along the project area the beach gradually transitions to inland areas without any dunes. During Hurricane Sandy the dune/riprap slope at Oakwood Beach was inundated and the homes landward in the low-lying marsh were largely destroyed (Kissam Ave). The dunes and rip rap slope experienced some erosion, but were largely intact and only suffered mild erosion. For this study, it is assumed the Post-Sandy elevations at Oakwood Beach, with a dune crest elevation of +12 ft NGVD, represent the Without Project Condition.

Wave overtopping on the dune at Oakwood Beach was calculated based on a modified version of the Van der Meer (1995, 1998) methodology, which was originally intended for estimating wave overtopping on coastal structures. Kobayashi et al. (1996) extended the Van der Meer overtopping formula to sandy dunes based on an equivalent uniform beach slope parameter, and the Alfageme (2001) empirical coefficient adjustment based on large scale tests performed by Delft Hydraulics Laboratory (1983). Note that the wave overtopping analysis applied to the coastal structures is based on the Van der Meer (1995, 1998) methodology as described in Section 5.4.

Table 3-6: Wave Overtopping at Oakwood Beach– Without Project Conditions

Return Period (Years)	Overtopping (Liters/s/m)
2	0.1
5	10
10	51
25	176
50	389
100	Submerged
200	Submerged
500	Submerged

4.0 ALTERNATIVES DEVELOPMENT

4.1 Alternatives Development Overview

An iterative planning process occurring over many years has been applied in the development, evaluation, and selection of the Line of Protection plan. The following analyses have been performed:

- Previously Authorized Federal Project (1965)
- Reconnaissance Study (1995)
- Formulation and Evaluation of Risk Management Measures (2002)
- Comparison of Alternative Plans and Tentative Plan Selection (2005)
- Optimization and NED Plan (2014)

A detailed description of the alternative plan development is provided in the Main Report.

4.1.1 *Optimization and NED Plan*

During the last phase of the study, Optimization and NED Plan, Alternative #4 (floodwall, levees and a buried seawall/armored levee with a raised promenade) was refined and evaluated at three different design levels to establish the NED plan. The NED plan is the alternative that reasonably maximizes net benefits and is the baseline against which other alternatives are compared. Normally, the Federal share of the NED plan is the limit of Federal expenditures on any more costly plan.

Although the NED plan forms the basis for establishing the Federal share of a project cost, the planning process recognizes that the non-Federal partners may have additional desires for coastal storm risk management and erosion control that may differ from that provided by the NED plan. A locally-preferred plan may be recommended provided the non-Federal partner agrees to pay any difference in cost and the plan is economically feasible with a benefit-to-cost ratio greater than unity.

The Tentatively Selected Plan for the Line of Protection Alternative was originally identified prior to Hurricane Sandy (October 29-30, 2012). The optimization process to identify the NED Plan, however, incorporates some post-Hurricane Sandy analyses and design changes. They are:

- Use of updated stage frequency curves from FEMA's forthcoming coastal Flood Insurance Study for New York City,
- Changes in plan alignment and design section types based post-Sandy site conditions, and
- A recent update in technical guidance related to I-Type floodwall design

Prior to Hurricane Sandy, Alternative #4 was optimized to four still water levels: 10.6, 11.6, 12.5, and 14.3 ft NGVD. During this pre-Sandy evaluation, the plan optimized to the highest still water level, 14.3 ft NGVD. Based on the results of the pre-Sandy evaluation, the plan was re-evaluated at the following four still water surface elevations:

- 13.3 ft NGVD – 100-year still water level (plus 0.7 ft allowance for SLC);
- 14.3 ft NGVD – Pre-Sandy optimized still water level;
- 15.6 ft NGVD – 100-year still water level (plus 3 ft allowance for SLC).
- 16.6 ft NGVD – 500-year still water level (plus 0.7 ft allowance for SLC).

Feasibility level design, quantities, costs, and economic benefits were calculated for the four plans to determine the optimal plan. The sections below provide an overview of the design criteria used to refine the plans including a description of the plans alignment, structure types, structural design considerations, geotechnical design considerations, and plan to provide pedestrian and vehicular access.

4.2 Optimized NED Plan

4.2.1 *Alignment*

The alignment of structures was initially defined as part of the reconnaissance level study and subsequent meetings with the sponsor (City of New York), the State of New York, and the USACE. Following Hurricane Sandy and additional meetings with the sponsor, the alignment was shifted landward in some areas to increase protective buffer between the ocean and structures. These changes allowed for a more homogenous structure geometry, lower structure crest elevations, and potentially lower maintenance costs.

The NED plan provides coastal storm risk management to several neighborhoods along the south shore of Staten Island. Figure 2-9 shows the maximum inundation depths during Hurricane Sandy highlighting the vulnerability of the entire Project Area. In order to provide Coastal Storm Risk management to severe storms, such as Hurricane Sandy, it is necessary to limit coastal flooding throughout the entire project shoreline. If one location floods, it is likely that the flood waters will spread to other low-lying areas in the Project Area.

The NED plan consists of four shoreline reaches and three typical structures:

- Shoreline Reach A-1: Levee
- Shoreline Reach A-2: Levee
- Shoreline Reach A-3: Vertical Floodwall
- Shoreline Reach A-4: Buried Seawall

An overview of the alignment is provided in Figure 4-1.

Figure 4-1: Overview of Line of Protection

Starting in Oakwood Beach in Shoreline Reach A-1, the earthen levee with a 10-ft wide crest ties into high ground on the northwest side of Hylan Boulevard. A closure structure, consisting of H-shaped posts that accommodate the stacking of metal panels, is proposed at Hylan Boulevard to prevent floodwaters from flanking the levees during rare high water events. The earthen levee continues southeast through Oakwood Beach parallel to Oakwood Creek and Buffalo Street until the levee crosses over Oakwood Creek. A tide gate structure is proposed at this location. The total length of Shoreline Reach A-1 is 2,800 ft.

Shoreline Reach A-2 is a 600 ft long earthen levee section with a wider 15-ft crest to accommodate maintenance vehicles accessing the tide gate structure. This wider levee section begins on the south side of the tide gate and terminates at the northwest corner of the Oakwood Beach Waste Water Treatment Plant.

In Shoreline Reach A-3 the structures transitions from an earthen levee to a vertical concrete T-shaped floodwall due to the limited area between Oakwood Creek and the Oakwood Beach Waste Water Treatment Plant (WWTP). The 1,800 ft long vertical floodwall protects the west and south sides of the WWTP.

Shoreline Reach A-4 extends 22,700 ft from the southeast corner of the WWTP to Fort Wadsworth. In previous alternatives Shoreline Reach A-4 consisted of a mixture of exposed armor stone revetments, buried seawalls, and vertical steel sheet pile flood walls. The structure was revised to a continuous buried seawall. The alignment of the buried seawall through Oakwood Beach deviates from previously developed alternatives, extending across a portion of the Fox Beach neighborhood that is being environmental restored as part of the State of New York's Blue-Belt Plan. The alignment continues across the marshes of Oakwood Beach and past Kissam Ave. The alignment in this marshy area is landward of New York City's sanitary sewer trunk line to the WWTP. A service access road is proposed along the seaward edge of the buried seawall to facilitate access to the trunk line. A bend in the alignment occurs at the eastern end of Oakwood Beach to accommodate a second proposed tide gate structure.

Within Cedar Grove Beach and New Dorp Beach, the alignment was shifted landward from previous alternatives to reduce the impacts of wave overtopping on the structure, resulting in a reduction of the structure crest elevations, footprint, and maintenance costs. At the eastern end of New Dorp Beach, the alignment incorporates a 45 degree bend before continuing eastward along the alignment of the existing dunes fronting Miller Field. The alignment of the buried seawall fronting Miller Field was coordinated with the National Park Service.

From Midland Beach to Fort Wadsworth the alignment generally follows the footprint of the existing promenade and FDR Boardwalk. There are a few exceptions where the alignment was shifted landward to maximize a protective beach buffer between the shoreline and structures. This is most noticeable at the eastern end of the project area where the beach narrows. The optimized NED plan ties-in to high ground at Fort Wadsworth.

4.3 Structural Design Considerations

4.3.1 *Water Levels, Waves, and Coastal Processes*

As described above, alternative plans were designed based on still water elevations of 13.3, 14.3, 15.6, and 16.6 ft NGVD. These still water elevations are roughly equivalent to a future conditions 100, 150, 300, and 500 year storm event based on the frequency of occurrence relationships for the Project Area and a sea level rise allowance of 0.7 ft (Section 2.2.1). In addition, the alternative plans were designed to withstand wave forces, wave overtopping, local scour, and coastal erosion. A detailed description of these coastal processes is provided in Section 5.0.

4.3.2 *Armor Stone Stability*

The required weight of the armor stone that comprises the core structure of the buried seawall in Reach 4 was determined based on armor stone stability methodologies developed by Van der Meer and Hudson. Both methodologies relate the stability of the armor stone to the weight of the stone and transformed wave height at the toe of the structure. The required nominal armor stone weight was calculated based on both formulas and the maximum weight was selected for use in the study.

The recommended stability coefficient and wave height characterization (e.g. $H_{1/3}$ vs. H_{max}) for Hudson's equation have evolved over time. The Rock Manual (2006) recommends using $H_{1/10}$ and a stability coefficient (K_d) of 4.0 for permeable structures. These values result in nominal weights that are in between the values determined if the 1977 Shore Protection Manual (USACE, 1977) and 1984 Shore Protection Manual (USACE, 1984) are applied. Table 4-1 presents a summary of the recommended armor stone weights for the Buried Seawall.

Table 4-1: Armor Stone Weight – Buried Seawall

Return Period years	SWL ft NGVD	T_p (s)	$H_{1/10}$ (ft)	H_{max} (ft)	Van der Meer (Tons)[1]	Hudson (Tons)[1]	Recommended (Tons)[1]
100	13.3	13.2	5.1	5.9	0.5	0.9	1
150	14.3	14.1	6.1	6.9	0.8	1.5	1.5
300	15.6	15.2	7.4	8.2	1.4	2.7	3

500	16.6	16.0	8.4	9.2	2.0	3.9	4

Notes: [1] Nominal weight of armor stone (W_{50})

During Plans, Engineering, and Design (PED), the armor stone specifications for the buried seawall will be finalized. In general, the stone shall consist of angular, fresh, sound, hard, dense, close-grained, durable stone of crystalline igneous or metamorphic rock, which will be separated from bedrock by quarrying. Armor stone shall be furnished in blocky and angular shapes, with its greatest dimension not greater than three times its least dimension. Flat stones, slabs, boulders and parts of boulders will be rejected. Bedding stone shall not be from silt, clay, organic material, debris or other unsuitable material. Typically the bedding stone is obtained from the same quarry as the armor stone, and the smaller remaining stone is broken into smaller stone suitable for under layer and bedding stone.

4.3.3 *Structure Type*

This section describes the type of structures used to form the optimized NED plan. Since many of the structures are located along sections of the shoreline with similar topographic and bathymetric conditions and are exposed to similar wave and water level design criteria, the design of structures throughout the Project Area are the same. The alternative plan is being optimized to four different design levels based on the 13.3, 14.3, 15.6, and 16.6-ft NGVD still water levels. Attachment A consists of plan and typical cross section sheets of the optimized NED plan.

The crest elevations for the structures were determined based on wave overtopping and a maximum allowable wave overtopping rate for each structure type: 2 liters/m/s for levee, and 50 liters/m/s vertical floodwall and buried seawall. A detailed discussion of the wave overtopping analysis and determination of structure crest elevations is provided in Section 5.0.

<u>Earthen Levee</u>

An earthen levee is proposed in Shoreline Reaches A-1 and A-2 (Station 10+25 to Station 47+14.81) to terminate the structures in the optimized NED plan into high ground northwest of Hylan Boulevard, thereby creating a closed system that protects the project area from floodwaters. The termination point of the earthen levee on the northwest side of Hylan Boulevard will be finalized once updated topographic information is collected and coordination with NYC Parks on the trail system integration is complete. The proposed levee in Shoreline Reaches A-1 and A-2 has crest elevations of 16, 17, 18, and 19-ft NGVD (corresponding to four different design still water levels). The proposed levee is a trapezoidal core section consisting of compacted impervious fill placed at 2.5H:1V side slopes. An inspection/seepage trench, created by excavating native soil a minimum of 6 ft below the existing ground surface and replacing it with compacted impervious fill, is incorporated into the design to prevent seepage. A high performance turf reinforcement mat will be placed on the exterior side slopes and levee crest to minimize scour and erosion during storm events. The levee along Shoreline Reach A-1 has a crest width of 10 ft, which is widened to 15 ft in Shoreline Reach A-2 to accommodate maintenance vehicle access to the tide gate.

Vertical Floodwall

A reinforced concrete floodwall is proposed for Shoreline Reach A-3 (Station 47+14.87 to Station 65+00) where a confined footprint is necessary to minimize impacts to the Oakwood Beach WWTP. The floodwall design consists of an H-pile supported T-wall with top of wall elevations of 16, 18, 20.5, and 22.5-ft corresponding to the four still water levels, respectively.

The structure footing was designed to accommodate localized wave induced and overtopping jet scour by defining a 4-ft thick base set 2-ft below grade. In addition, a rock blanket extends 15-ft seaward side of the wall to address wave scour and a rock splash apron extends 10 to 15 ft landward from the concrete footing to provide adequate overtopping jet scour protection. A vertical steel sheet pile wall has been added beneath the wall to prevent seepage below the footing.

Buried Seawall

A buried seawall proposed for Shoreline Reach A-4 (Station 65+00 to Station 292+44.67) is the structure type that is used for the majority of the optimized NED plan Four crest elevations, 16, 18, 20.5, and 22.5 ft NGVD corresponding to the four still water levels, respectively.

The buried seawall comprises a trapezoidal shaped core structure with a 10 to 18-ft wide crest and 1.5:1 (horizontal: vertical) side slopes. The core is constructed with two-stone thickness armor stone and bedding stone layers. A 10 to 18-ft wide scour apron is incorporated into the seaside structure toe. The seaward face or the landward and seaward faces of the above-grade portions of the structure are covered with material excavated to accommodate the structure foundation. This material, primarily sand with some clay, silts, and topsoil, will be placed on 2:1 side slope to support native beach vegetation. The 2 to 3-ft material cover is used to visually integrate the buried seawall with surrounding topography and to protect the public from climbing and/or falling on the uneven rock surface. Geotextile fabric is placed underneath the bedding layer to reduce settlement and around the core structure to minimize loss of fill through the voids. A vertical steel sheet pile wall will be installed in the interior of the structure to prevent seepage.

Station 65+00 and Station 158+00

This reach of the buried seawall incorporates a 17 ft wide raised promenade at elevation 22.5 ft NGVD. The raised promenade is constructed with reinforced cast-in-place concrete with an asphalt or paver surface finish to support maintenance vehicles. Seaward and landward faces of the buried seawall are covered with the excavated material and planted with native dune vegetation. Phragmites control will be conducted on the seaward faces between Station 65+40 and Station 102+00 within the Oakwood Beach corridor.

The two sanitary sewer interceptor lines (30-inch and 60-inch diameter) that convey wastewater from the eastern communities of Staten Island to the Oakwood Beach Wastewater Treatment Plant generally follow an alignment that is landward of the Line of Protection (LOP) except within the Oakwood Beach Corridor. The two interceptor lines cross underneath the LOP on the south side of Cedar Grove Beach and generally follow a parallel alignment to that of the LOP on the seaward side. As a means to provide the City of New York with access to the interceptor lines for maintenance purposes and to minimize the risk of flooding to the sanitary system during more frequent storm events, a service access corridor has been provided.

The service access corridor consists of raising the grade above the two interceptor lines to elevation +10 feet NGVD, installing concrete junctions boxes with sealed manhole covers, and adding a 20-foot paved surface to facilitate vehicle movements. The seaward face of the raised grade will be stabilized with armor stone to minimize erosion during storm events. The landward face of the service access corridor will be integrated with the seaward face of the LOP except where it crosses drainage flow paths associated with the City's Bluebelt plan. In these locations, the landward face will not extend to the LOP but will be sloped to meet existing grade and stabilized with armor stone. Vehicular ramps to provide entry to the service access corridor will be incorporated into the LOP at Cedar Grove Beach, Oakwood Beach WTTP, and Kissam Avenue. The integration of the Bluebelt plan and the final location and alignment of the vehicular ramps will be coordinated with the City during the Preliminary Engineering and Design (PED) phase.

<u>Station 158+00 to Station 268+00</u>

The buried seawall incorporates a 2.4 mile long, 38-ft wide pile supported promenade as a functionally replacement of the existing 1.0 mile long 38-ft wide at-grade paved and existing 1.4 mile long 40-ft wide pile supported promenade of the FDR Boardwalk and esplanade that currently extends between Fort Wadsworth and Miller Field. The width of the new promenade, 38 feet, is the maximum allowable width for replacing the "at-grade" foot-print of the existing promenade.

A new pile supported boardwalk integrated into the buried seawall will have a deck elevation of 22.5 ft NGVD. Reinforced concrete grade beams located on the crest of the buried seawall will support the waterside section of the boardwalk. Landward of the grade beams, piles with concrete spread footings support the remainder of the boardwalk. The concrete spread footings will be poured to form an integrated section within the armor stone layer of the buried seawall. The piles are connected by longitudinal pile caps and cross-bracing. Design, materials, and finishes of the pile supported boardwalk will developed in collaboration with NYCDPR.

Several recreational facilities operated by NYC Parks as well as concessions along the existing at-grade paved esplanade and pile support sections of the FDR Boardwalk have first floor elevations lower than the deck elevation of the timber boardwalk at the 15.6 and 16.6 ft WSEL. The buried seawall design was modified to provide access to these facilities. Landward of the structure crest, the rock slope was replaced by a combination wall comprised of steel H-piles and steel sheet pile. This vertical element accommodates two boardwalks, one with a width of 25 ft at elevation 22.5 ft NGVD and a 13-ft wide section that may be ramped down to meet building first floor elevations. The 13-ft section is ADA compliant. The ramp maintains a minimum 12-ft clear distance between railings for two way pedestrian and bicycle traffic.

Only the seaward face of the buried seawall is covered with the excavated material and planted with native dune vegetation. The landward face of the buried seawall lies underneath the pile-supported boardwalk where the placement of cover material and native beach vegetation is challenging to implement and maintain. Phragmites control will not be required for this reach.

<u>Station 268+00 to Station 288+00</u>

The buried seawall in this 2,000 foot section also incorporates a 38-ft wide-foot pile supported promenade as described above for Station 158+00 to Station 268+00. In this 2,000 foot section, from Sand Lane to Ocean Ave, the width of the armored crest of the buried seawall is increased to

18 ft to accommodate the larger design waves and reduce wave overtopping. The weight of the armor stone and depth of scour protection are also increased to handle the larger design waves.

STA 288 + 00 to STA 292 + 44.67

The section of the buried seawall ties into high ground adjacent the Seaside Plaza Apartments and the south boundary of Fort Wadsworth, the former military installation that is now operated by NPS as part of the Gateway National Recreational Area. This approximately 400-ft section has a 2-to 3-ft layer of excavated material covering the landward and seaward faces and the structure crest. Native dune vegetation will be planted along the seaward face of the structure adjacent to boardwalk), transition to upland grasses and planting along the remaining areas. A promenade is not incorporated into this tie-in section.

4.3.4 *Vertical Floodwall Design*

Three failure modes were evaluated for the concrete T-wall: (1) structural performance, (2) global stability, and (3) seepage beneath the wall. Design development and supporting calculations were based on USACE design guidance (EM 1110-21-2502 and the USACE New Orleans District Hurricane and Storm Damage Risk Reduction System Design Guidelines) for floodwalls, including the use of dead and live load factors of 1.7 and a hydraulic load factor of 1.3 on all shear and bending calculations. A safety factor of 1.5, which is consistent with structural engineering practice for retaining/floodwall design, was used to resist both overturning and sliding. The design of the concrete reinforced walls was performed using LRFD load reduction factors.

Wave loads were calculated as pressure distributions along the wall; however, they have been reduced to resultant forces at the heights above ground level provided in the table below. The wave forces were applied to the wall at their respective heights above the existing ground, resulting in a large bending moment at the base of wall. Due the maximum load occurring at the base of the wall, the top of wall was set at a minimum 18-inch dimension and tapered to the required thickness at the base, where necessary.

Table 4-2: Floodwall Design Criteria

Reach	SWL (ft, NGVD)	Crest Elevation (ft, NGVD)	Ground Elevation (ft, NGVD)	Wave Force (kip/ft)	Height of Moment Arm (ft)
A3	13.3	16	10	2.9	4.4
A3	14.3	18	10	4.2	5.4
A3	15.6	20.5	10	5.9	6.6
A3	16.6	22.5	10	7.5	7.5

The overturning and sliding components consist of the resultant wave force, the flood water level and soil pressure on active side of the wall only. The wall is designed for overtopping jet-induced scour from the ground elevation to the base of the footing on the passive (landward) side, leaving the concrete self-weight as the single resisting component. The overturning and sliding analyses of the wall with a spread footing resulted in factors of safety significantly less than 1.5; therefore, the structure is designed to resist these forces in the piles. Two piles, HP14x89 H-piles of lengths of

60, 65, 80, and 95-feet for the 13.3, 14.3, 15.6, and 16.6-ft stillwater levels, respectively are designed to handle two force components: 1) the axial (tension and compression) forces resulting from the moments in the overturning analysis and 2) the shear forces from the sliding analysis. Bearing loads are not analyzed due to the structure being pile supported. During the Preconstruction Engineering and Design phase (PED), additional options will be evaluated for the foundation such as modifying the structure to utilize plumb piles in lieu of battered piles and/or increasing the width of the footing to incorporate scour/splash protection.

The footing was designed to be 4-ft thick with a 2-ft overburden to allow for up to 6-ft of scour without comprising the integrity of the wall. A 15-ft wide scour blanket on the seaward side of the wall and 10 to 15-ft wide splash apron on the landside are provided to accommodate localized wave induced and overtopping jet scour. The footprint of the floodwall may be reduced between Stations 55+00 and 58+00 due to the close proximity of the Oakwood Creek and the WWTP. A narrower concrete footing at this location with a reduced pile spacing and/or increased pile lengths would be incorporated.

A PZ 22 sheet pile wall beneath the footing has been sized to handle full hydrostatic head (from SWL on one side of the wall to MLLW on the opposite side) and the seepage analysis dictated a required sheet pile tip elevation of -10-ft NGVD.

4.4 Geotechnical Design Considerations

The engineering evaluations and recommendations presented herein are based on the subsurface investigation results as well as our experience on other similar projects and the requirements for this project. As per the project requirements, engineering evaluations were primarily performed using the USACE design manuals, EM 1110-2-1913 "Design and Construction of Levees", EM 1110-2-2502 "Retaining and Flood Walls" and EM 1110-2-1901 "Seepage Analysis and Control for Dams". It should also be noted that engineering evaluations presented herein were performed for shoreline segment FWOB for the 300 year storm event (WSEL 15.6 ft NGVD).

Seepage and stability analyses were performed for each type of structure using conservatively selected representative sections.

The following sections provide descriptions and results of analysis performed to evaluate soil behavior under seismic conditions, seepage conditions, and slope stability.

4.4.1 *Generalized Subsurface Conditions*

Based on the results of the 2002 subsurface exploration program, the primary soil type encountered at the project site was coarse to fine sand with varying amounts of silt and gravel. However, in the vicinity of the water treatment plant, soft compressible organic soils were encountered to depths of about 6 feet below the ground surface. The laboratory tests show that the majority of the sands at the site consist of trace to some amounts of silt and gravel. The borings also indicate the presence of some clay and silt lenses within this stratum at isolated locations. Generally, the SPT N-values within this stratum ranged from 10 bpf to 30 bpf with an average of about 18 bpf, indicative of a medium dense material. Since all borings were terminated within this stratum at depths ranging from 25 to 30 ft, the thickness of this stratum is not defined at present. Considering that soils encountered at the project site are predominantly medium dense sandy soils, the subsurface conditions at the project site are generally suitable for the construction of the storm damage reduction structures. However, additional test borings should be performed during the final design stage at locations where pockets of soft clayey/silty soils and loose sandy soils were encountered during the 2002 exploration program to verify the extent of such soils and in the vicinity of the water treatment plant. Further, it should be noted that engineering evaluations presented in the following sections are based on the assumption that there are no continuous layer of soft clayey/silty soils and/or loose sandy soils within the limits of the project site.

4.4.2 *Seismic Considerations*

In accordance with EM 1110-2-1913, slope stability analyses should also be performed for the seismic loading case as presented below in "Slope Stability Analysis". The seismic loading condition was evaluated using the pseudo-static method of analysis. The effects of the seismic motion were simulated by applying a pseudo-static coefficient in the horizontal direction. The pseudo-static coefficient was assumed 2/3 of the peak ground acceleration (PGA) at the foundation (ground surface) level for the 2,500-year seismic event. Considering that the depth to bedrock at the project site appears to be greater than 100 ft, and the soils within the top 100 ft are likely to be generally medium dense to dense in compactness, the soil profile type will very likely to be seismic site class 'D' (S_D). Based on 2008 Probabilistic Hazard Curves from the U.S. Geological Survey (USGS, 2008), the PGA at the bedrock level is approximately 0.16g for a 2,500-year seismic event at the project site. Therefore, as per NEHRP (2009) provisions, the PGA at the ground surface for seismic site class D (S_D) soil profile was estimated to be about 0.24g. Hence, the pseudo-static coefficient of 0.16g (i.e., 0.67 x 0.24) was assumed for the seismic loading case.

Since the sandy soils below the groundwater level at the project site are generally medium dense to dense in compactness, it appears that seismic induced liquefaction at the project site will not likely occur and therefore should not be a concern. However, it should be noted that at a few isolated locations, pockets of loose sandy soils were encountered and additional investigations will be required to verify the extent of such loose sandy soils.

4.4.3 *Seepage Analyses*

Seepage analyses for all types of storm damage reduction structures were performed in order to estimate the seepage quantity through and/or underneath the structures, exit hydraulic gradients on the downstream side of the structures and the pore pressures within the embankments (used for Case III of slope stability analyses).

Typically, it is standard practice to conservatively use the fully developed phreatic surface obtained from a steady-state seepage analysis to perform the slope stability analysis under a long-term condition. This condition occurs when the water remains at or near flood stage for a sufficient period of time to result in full embankment saturation and a condition of steady seepage. However, considering the relatively short duration (about 6 hours to 24 hours) of anticipated storms, this condition will most likely not occur during the anticipated storms. Therefore, for buried seawalls, both transient and steady seepage analyses were performed. For all other structures, only steady seepage analyses were conservatively performed.

The 300 year storm still water elevation of 15.6 ft (NGVD 29) was used in the seepage analyses. The storm hydrographs used in the transient seepage analyses are based on November 1950 nor'easter in New York City and hurricane hydrograph for the New Bedford Harbor area in Massachusetts. The two hydrographs were chosen to represent shorter and longer durations in peak water levels.

The seepage analyses were performed using the commercially available finite element method (FEM) software program SEEP/W©. In order to perform the seepage analyses a representative cross section was selected for each type of structure. As indicated above, these representative sections were conservatively selected at maximum height locations. The "maximum height" refers to the difference between the lowest ground surface elevation and highest structure elevation. One of the important parameters required to perform the seepage analyses is the hydraulic conductivity of storm damage reduction structure materials and foundation materials.

The saturated hydraulic conductivity of porous materials varies typically by one or two orders of magnitude (e.g. silty sand, 10^{-3} to 10^{-5} cm/sec). Therefore, seepage analyses were performed for a range of hydraulic conductivity values. Based on the results of these analyses, conservative values were selected and are presented in this section.

The phreatic surfaces for the stability analyses were developed from the seepage analyses. In order to develop the phreatic surfaces, the materials within the embankments were modeled as saturated/unsaturated materials with hydraulic conductivity as function of the pore pressure. However, considering that the results of the seepage analyses are not sensitive to hydraulic conductivity as function of the pore pressure, only saturated hydraulic conductivity values are presented.

1. The foundation soils generally consist of coarse to fine sands with varying amounts of clay, silt and gravel. Considering this, the hydraulic conductivity (k) for the foundation soils was assumed to be 1×10^{-4} cm/sec.

2. Compacted fill will be used for core and shell material for levee structures, and as earth cover material on the water side and impervious fill on the landside for the buried seawalls. Considering that the compacted fill should be relatively impervious, it is anticipated that silty sand (SM) and/or clay sand (SC) with a hydraulic conductivity less than 1×10^{-5} cm/sec will be used as compacted fill. Therefore, for the compacted fill, a hydraulic conductivity (k) of 1×10^{-5} cm/sec was assumed.

3. Armor and bedding stones will be used for the construction of buried seawalls. Considering that these materials will have a significant amount of voids, a hydraulic conductivity (k) of 10 cm/sec was assumed for these materials.

The results of both transient and steady-state seepage analyses were presented in URS memorandum dated July 22, 2011 (URS, 2011), for buried seawalls (see Attachment D). Based on those analyses, steady-state seepage conditions are not expected to develop during the anticipated storms. Therefore, for buried seawalls, the results of transient seepage analyses are presented in Table 4-3. However, for the levee and vertical wall, the steady-state seepage (conservative) analyses results are presented in Table 4-3.

Table 4-3: Summary of Seepage Analyses Results

Type of Structure	Seepage Quantity per 1000 ft		Exit Hydraulic Gradient
	ft^3/sec (cfs)	Gallons/min (gpm)	
Buried Seawall	< 1	5	0.25
Vertical Floodwall	< 1	20	0.05
Levee	< 1	10	0.25

4.4.4 *Slope Stability Analyses*

As per EM 1110-2-1913, slope stability analyses were performed for four loading conditions as follows:

- Case I, end of construction (downstream slope);
- Case II, sudden drawdown (upstream slope);
- Case III, steady-state seepage from full flood stage (downstream slope);
- Case IV, earthquake (downstream slope).

A commercially available computer program, SLOPE/W©, was used to perform the slope stability analyses. SLOPE/W© is a general purpose slope stability program that uses limit equilibrium methods to compute the factor of safety (FOS) for a given slope geometry and loading conditions. Spencer's Procedure for the method of slices for circular failure was used to evaluate the slope stability as this procedure satisfies the complete static equilibrium for each slice. SLOPE/W© automatically searches for the circular shear surface associated with the minimum FOS, which is considered the critical or controlling shear surface. As mentioned in "Seepage Analyses", the pore pressures within the embankments for the Case III loading condition were obtained from the phreatic surfaces developed using the transient and/or steady state seepage analyses using SEEP/W©. For Case II (sudden drawdown) loading condition, because of the instantaneous drawdown, it was assumed that pore pressures within the embankment remain the same before and after the drawdown.

Because of the low probability of earthquakes coinciding with severe storm events, stability analyses for the Case IV (earthquake) loading condition was performed assuming no water above the ground surface. As described in "Seismic Considerations", pseudo-static coefficient of 0.16g was assumed for the earthquake loading case.

Besides knowledge of the pore pressure distribution within the embankment, the shear strength parameter values of the embankment materials and foundation soils are important for the slope stability analyses.

The material parameters required for the stability analyses are the shear strength and unit weight properties of the embankment fill and foundation soils. Considering that sandy soil and stones will be used as embankment fill materials, and since the foundations soils generally consist of sandy materials, effective stress shear strength parameter values were used in the stability analyses for all conditions as follows:

1. Foundation soils are generally medium dense to dense sandy soils. Based on the SPT N-values obtained within the foundation soils and widely used empirical correlations, a conservative effective stress friction angle of 30 degrees was used in the current analysis for the foundation soils. However, as previously mentioned, pockets of soft clayey/silty soils and loose sandy soils were encountered at isolated locations. But, currently it was assumed that there are no continuous layers of soft clayey/silty soils and/or loose sandy soils within the project limits.

2. Sandy fill will be compacted to a density corresponding to 95% of the maximum dry density. Therefore, a conservative effective stress friction angle of 32 degrees was used in the current analysis for the compacted fill.

3. Bedding stone and armor stone friction angle values are typically greater than 36 degrees. Therefore, conservative effective stress friction angle values of 36 degrees and 38 degrees were used in the current analyses for bedding stone and armor stone, respectively.

Table 4-4 below summarizes the material shear strength and unit weight parameter values used in the stability analyses.

Table 4-4: Summary of Material Parameters for Stability Analyses

Materials	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Foundation Soils	120	30	0
Compacted Fill	125	32	0
Bedding Stone	140	36	0
Armor Stone	145	38	0

As presented previously, the slope stability analyses of buried seawalls for the Case III loading condition was performed using pore pressures obtained from transient seepage analyses. However, for the earth embankment levees the slope stability analyses for the Case III loading condition were performed using conservative pore pressures obtained from the steady-state seepage analyses. The slope stability analyses results are presented in Table 4-5, along with the corresponding minimum acceptable factors of safety.

Table 4-5: Summary of Slope Stability Analyses Results

Slope	Design Condition	Factor of Safety		
		Minimum Acceptable	Buried Seawall	Levee
Downstream	Case I: End of Construction	1.3	1.4	1.7
Upstream	Case II: Sudden drawdown	1.0	1.2	1.2
Downstream	Case III: Seepage from maximum flood level	1.4	1.4	1.5
Downstream	Case IV: Earthquake	1.0	1.0	1.2

4.4.5 *Groundwater*

No groundwater observation well was installed during the 2002 investigation. In addition groundwater levels were not measured inside the test borings. Considering the proximity of the site to the Lower New York Bay and the topography of the site, it is anticipated that the groundwater is likely to be encountered at about +2 ft NGVD 29.

4.5 Closure Structure

In order to tie-off the optimized NED plan at Drainage Area A, the alignment extended to the north of Hylan Boulevard by approximately 300 linear ft. The grades on Hylan Boulevard are not high enough, at elevation 13 ft NGVD, to prevent floodwaters from affecting areas in Oakwood Beach. Raising of the road would affect existing residential and commercial buildings and existing intersection at Buffalo Road. In order to prevent water from passing through the 110 ft wide opening, three alternatives were considered: 1) a stop log gate structure which would utilize removable columns installed between each lane, at the median and adjacent to each curb line; 2) a roller gate which would require an open area for storage monolith on the south side of Hylan Boulevard along with a gate monolith and track extending across the road; and 3) a swing gate with a removable center column.

Comparing the three alternatives indicates that the stop log gate would have limited impact on utilities and road closures. However the closure itself will take several hours to gather, deliver and install the removable columns, stop logs, and sand bags. The roller gate would require extensive road closures for installation of the gate monolith and track, utility installation and modification of the roadway but would also provide a faster method of closing the roadway. The swing gate would require storage monoliths on both sides of the roadway to store the gate when its open, would have limited utility impacts, and the time to close the roadway would be similar to the roller gate except for the placement of sandbags along the base of the gate.

A comparison of the cost indicates that the stop log gate would be the least expensive at approximately $740,000. The swing gate would be nearly double this cost. The cost of the roller gate would exceed two million dollars assuming extensive roadway and utility relocation cost. Since the proposed crossing at Hylan Boulevard is higher than the 100-year stillwater stage (12.6 ft NGVD 1929) and therefore the anticipated number of gate closures is infrequent the stop log gate structure was chosen for closing off Hylan Boulevard for this Feasibility Study. However, during PED design refinements will be conducted for all plan elements based on new field investigations and analyses. The appropriate closure structure will be evaluated based on these new field investigations and analyses.

4.6 Tide Gates, Stormwater Outfalls, and Drainage/ Sanitary Sewer Structures

Existing stormwater outfalls, consisting of single and double concrete box culverts, pass beneath the Shoreline Reach A-4 buried seawall at nine locations. At these locations, the sheet pile seepage wall terminates either side of the existing culverts and the buried seawall rock structure will be constructed around these existing culverts. A drainage control structure that incorporates tide and secondary sluice gates will be integrated with the stormwater pipe to prevent elevated storm-induced water levels from flooding interior areas behind the LOP structures.

Tide gate structures with reinforced concrete wing walls and concrete channel bottom are proposed at two locations in Shoreline Reaches A-2 and A-4 where the alignment crosses existing creeks. Aside from increases in wall height and thickness, the basic design of the proposed tide gate structures is consistent with the design of the existing tide gate structure located to the east of the Oakwood Beach Water Treatment Plant. The 10 inch thick slab atop the tide gate structure was designed to handle a 60 pound per square foot (PSF) pedestrian live load. The structure has not been designed for vehicular loading. The wing walls were designed to handle the soil pressure on the backside of the wall. Wing wall thickness varies linearly from 1ft-3” at the top to 2 ft-8” at the base. A 12-inch thick concrete slab will line the bottom of the channel.

There are eight existing drainage and sanitary sewer lines that cross the earthen levee and buried seawall in Shoreline Reaches A-1 through A-4 in Oakwood Beach area. Control structures with integrated sluice gates are installed where the pipes cross the earthen levee, concrete floodwall, and buried seawall to prevent floodwaters from entering these pipes and inundating low-lying areas behind the LOP structures. It is noted that utility crossings through the LOP also provide risks for seepage. Mitigation measures to minimize seepage include collars and cutoff walls. Appropriate measures will be considered during PED and incorporated into the final design.

4.7 Pedestrian and Vehicular Access

Three types of access points are provided along the LOP: Maintenance vehicle access (MVA), combined truck and pedestrian access (DTP), and pedestrian access (PA). One vehicle access is provided at Shoreline Reach A-2; however, the remainder of the access points are dispersed along the buried seawall (Shoreline Reach A-4), approximately every 500-ft.

Earthen ramps are proposed to provide vehicular access to the tide gate and stormwater outfall structures. These ramp sections are designed to handle HS-20 loading to allow maintenance vehicles to access the sluice gates in the drainage structures from above. Maintenance vehicle access is provided at one location on Shoreline Reach A-2 and at four locations along Shoreline Reach A-4 between New Dorp Beach and Oakwood Beach.

An additional nine earthen ramps are proposed between Oakwood Beach and the east end of South Beach. These ramps are designed for both pedestrian and HS-20 vehicular access and meet the 1:12 maximum slope required by ADA guidelines. The ramps have been strategically located to provide beach access from existing roads and access paths.

Pedestrian access points are located along Shoreline Reach A-4 between Midland Beach and South Beach. Each access point comprises 10-ft wide reinforced concrete stairs on both the landward and seaward sides of the buried seawall, providing access to the promenade and the beach.

The buried seawall crest elevation exceeds the existing deck elevation for the Ocean Breeze fishing pier. The pier segments nearest to the promenade will need to be reconstructed to ramp up to the promenade at a 1:12 maximum slope required by ADA guidelines.

4.8 Adaptability

The low relative sea level was used in the evaluation of the structures based on current guidance (ETL 1100-2-1 dated 30 Jun 2014). However, immediate or high rates of sea level change may affect the performance of the optimized NED Plan. The ability of the structures to adapt to higher rates of sea level change by raising their crest and/or top of wall height, without the need to rebuild the structures, was evaluated during the optimization phase. The intent in developing the adaptability measures was to minimize enlarging the structure footprint; therefore, the measures were developed to raise a structure's height within the existing structure footprint where possible.

A reinforced concrete parapet wall and base constructed atop the crest of the buried seawall would raise the crest height of the structure by up to 3 feet to provide overtopping protection as shown in Figure 4-2. The parapet wall and base may be aligned with the landward or seaward crest edge of the buried seawall (Figure 4-2 shows the latter alignment). A concrete base integrated with the armor layer of the buried seawall is designed to prevent overtopping and sliding of the parapet wall due to wave-induces horizontal and vertical forces.

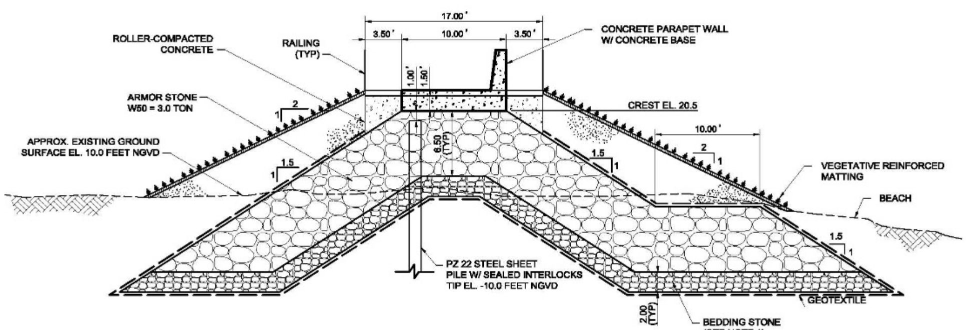


Figure 4-2: Concrete Parapet Wall atop Buried Seawall

The concrete vertical floodwall may accommodate sea level change by raising the top of wall height. By designing the foundation of the concrete floodwall during the initial construction to counteract future hydrostatic and wave forces, the reinforcing steel matrix is arranged to accept doweling of the future cast-in-place concrete wall addition as shown in Figure 4-3.

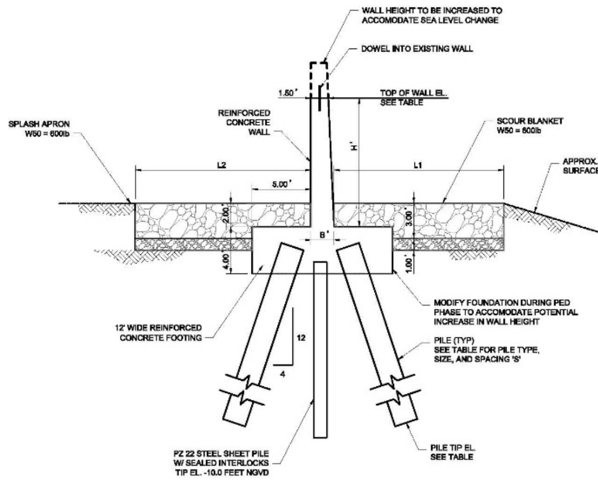


Figure 4-3: Raising of Concrete Floodwall

Raising of the earthen levee by up to 3 feet may be accomplished by adding imperious and selected backfill to the same lines and grades of the initial construction as shown in Figure 4-4. This raising will increase the footprint of the structure but would fall within the 15-ft wide flood protection easement. If additional height is required, a concrete parapet wall, similar to that shown for the buried seawall, could be added to the levee crest.

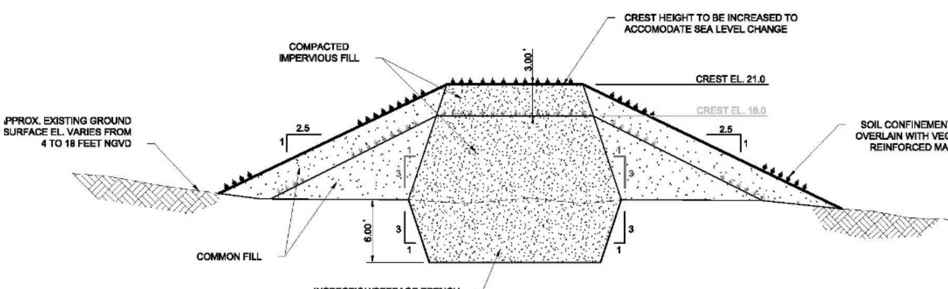


Figure 4-4: Raising of Earthen Levee

The cost to adapt the line of protection to the high sea level change scenario is estimated to be $58 million. The cost estimate includes 1) mob/demob; 2) adaption of the structures (i.e. increase levee height, upfront costs of larger H-piles, increase floodwall height, add concrete parapet wall, and demo timber boardwalk); 3) S&A (20%); 4) Contingency (30%).

The beach along the South Shoreline of Staten Island is a buffer between the Line of Protection (LOP) structures (earthen levee, concrete vertical floodwall, and buried seawall) and Raritan Bay, dissipating wave energy and insulating the LOP structures from short and long-term changes in shoreline position. The alignment of the LOP structures was selected so the structures are set back and elevated, minimizing their exposure to storm induced water levels and waves except during infrequent extreme events (i.e. 25-year event and greater).

Beach erosion is not anticipated to affect the performance of the structures or the sediment transport processes that may affect the stability of beaches in or adjacent to the project area until it reaches a minimum beach width. A minimum beach width threshold of 75 feet (measured from MHW) was determined based on analysis of the impact of LOP structures on storm induced beach change using the SBEACH model.

Since the long-term sediment budget for the project area indicates that the beach is relatively stable, it is not anticipated over the project period of analysis (50-years) for the beach to erode below the minimum 75-ft threshold. A project cost to maintain the beach was not included for this reason.

The long-term beach erosion rate may be affected by climate variability, including increasing sea level rise and frequency/duration of coastal storm events. If the long-term beach erosion accelerated such that the minimum beach width of 75 ft were reached, beach maintenance/restoration activities may be evaluated. The implementation of beach maintenance/restoration as a future project adaptation would be based on a future decision document that would evaluate and record the changed metrological and oceanographic conditions.

4.9 Cost Estimate

Feasibility level quantities and costs were calculated for the four plans to aid in the selection of the optimal plan. The cost estimates are based on July 2014 price levels, a period of analysis of 50 years, 3.375% interest rate, 48 month construction period, and 30% contingency. The cost estimate only includes the components of the overall plan related to the Line of Protection. It also includes the vehicle and pedestrian access, as well as demolition of the existing boardwalk. A summary of the cost estimates for the four plans is provided in Table 4-6.

Table 4-6: Line of Protection Cost Estimate

Item Description	13.3' SWL	14.3' SWL	15.6' SWL	16.6' SWL
Mob/Demob.	$6,219,000	$7,033,000	$8,526,000	$9,705,000
Clearing/Grubbing &Stripping of Topsoil	$1,268,000	$1,268,000	$1,268,000	$1,268,000
Demolition of Timber Boardwalk & Asphalt Walkway	$4,480,000	$4,480,000	$4,480,000	$4,480,000
Tidal Wetlands Mitigation	$5,598,000	$5,598,000	$5,598,000	$5,598,000
Cultural Mitigation	$3,000,000	$3,000,000	$3,000,000	$3,000,000
A-1 Levee	$3,042,000	$3,294,000	$3,541,000	$3,612,000
A-2 Levee	$589,000	$655,000	$700,000	$750,000
A-3 Vertical Floodwall	$10,720,000	$11,955,000	$13,281,000	$15,133,000
A-4 Buried Seawall	$126,788,000	$145,577,000	$181,290,000	$208,834,000
Subtotal	$161,704,000	$182,860,000	$221,684,000	$252,382,000
Contingency (30%)	$42,043,000	$54,858,000	$66,505,000	$75,714,000
Subtotal	$210,215,000	$237,718,000	$288,187,000	$328,096,000
Engineering and Design, and S&A (20%)	$42,043,000	$47,544,000	$57,637,000	$65,619,000
Total Project Cost	$252,258,000	$285,262,000	$345,824,000	$393,715,000
IDC (3.375%, 48 months)	$17,415,000	$19,694,000	$23,875,000	$27,181,000
Total Investment Cost	**$269,673,000**	**$304,956,000**	**$369,699,000**	**$420,895,000**
Annualized Investment Cost (3.375%, 50 years)	$11,239,000	$12,710,000	$15,408,000	$17,542,000
O&M Cost	$178,000	$178,000	$178,000	$178,000
Total Annual Cost	**$11,417,000**	**$12,888,000**	**$15,586,000**	**$17,720,000**

5.0 WITH PROJECT COASTAL PROCESSES

5.1 Overview

This section describes the coastal engineering analyses applied to determine the crest elevations of the structures that comprise the line of protection. The primary purpose of the line of protection is to manage the risk of flooding and wave attack along the Project Area.

The principal criteria used to determine the structure crest elevations is wave overtopping. Floodwalls that are exposed to heavy wave overtopping for many hours are susceptible to structural failure (Goda, 2000). Therefore, floodwalls are often designed to limit wave overtopping below a certain threshold depending on the structure type and desired level of risk management. The Coastal Engineering Manual (USACE, 1999) provides guidelines for maximum allowable mean wave overtopping rates for various structures before the structure begins to exhibit damage which may eventually lead to structural failure. Based on available literature including European and United States reference documents including Table 5-1, the following overtopping thresholds for specific structures types have been applied to determine the structure crest elevations:

- 2 liters/m/s for levees;
- 50 liters/m/s vertical floodwall and buried seawall.

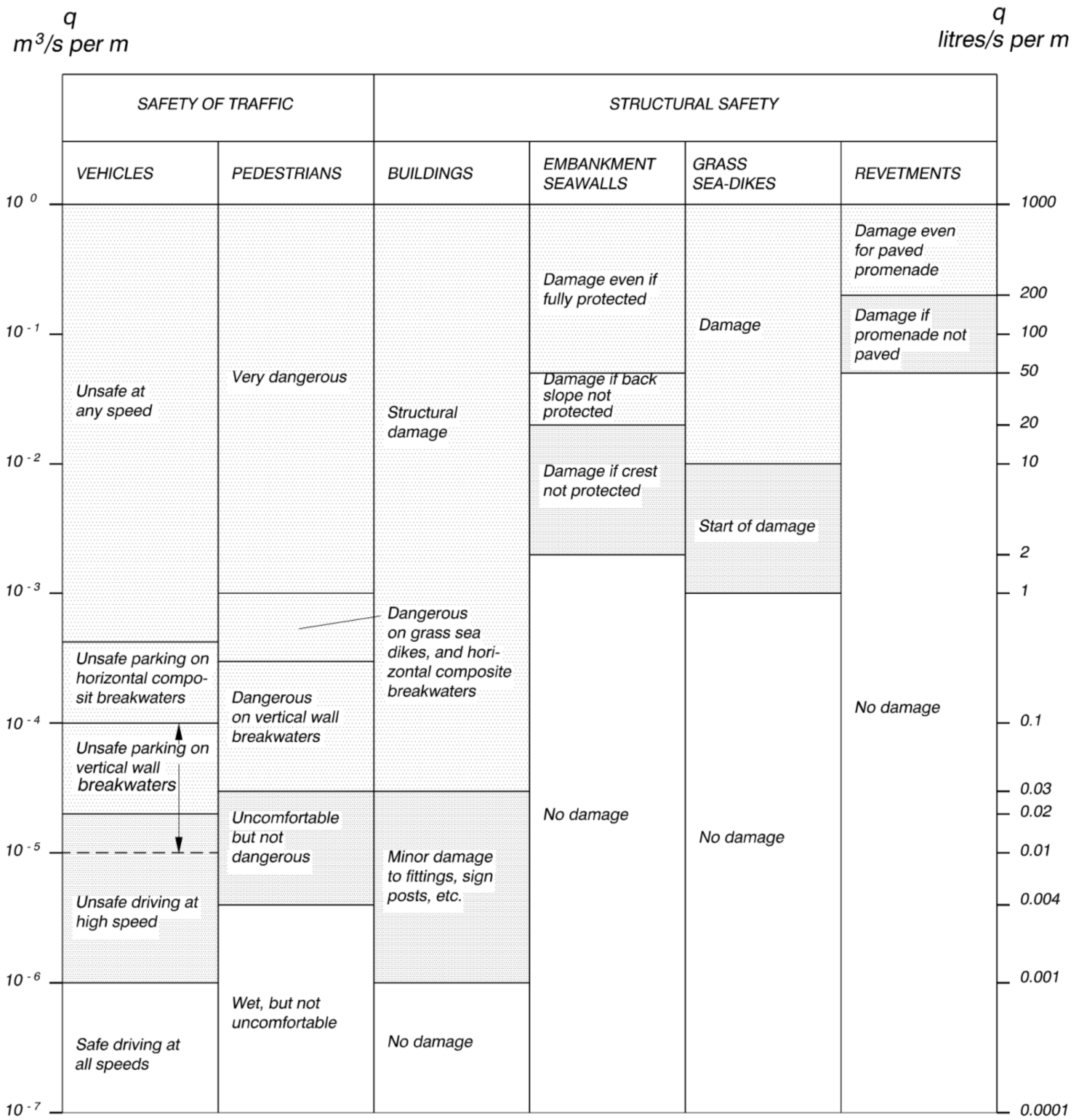
Four different plans are being evaluated at this phase of the study which are characterized by four still water levels (sea level rise, 0.7 ft over 50 years, is already included in the still water levels):

- 13.3 ft NGVD (1.00 % annual exceedance probability, 100 year return period)
- 14.3 ft NGVD (0.67% annual exceedance probability, 150 year return period)
- 15.6 ft NGVD (0.33% annual exceedance probability, 300 year return period)
- 16.6 ft NGVD (0.20% annual exceedance probability, 500 year return period)

As discussed above, the structure crest elevations required for the four still water levels (100, 150, 300, and 500 year return periods) were determined based on the maximum allowable wave overtopping. The return periods associated with these four still water levels were determined based on the FEMA stage frequency curve applied for the Project Area (Table 2-9).

Several other coastal processes must be evaluated prior to calculating the mean wave overtopping rate: storm-induced shoreline change and wave transformation across the surf zone. SBEACH simulations were performed to evaluate the expected profile change and possible storm-induced erosion with the structures in place. Wave conditions at the toe of the structures, which are depth limited, were determined using Goda's (2000) model of random wave breaking.

Table 5-1: Critical Values of Mean Wave Overtopping (Table V1-5-6, CEM)



5.2 Storm-Induced Shoreline Change

With Project SBEACH simulations were performed at the same six profiles as in the Without Project modeling (Figure 3-1). Seawalls were added to the profiles based on the preliminary location of the toe of the structure. A failure threshold for the seawalls was not added to SBEACH since the primary goal is to evaluate how the structure impacts profile change during storm events. Therefore the structure is treated as an infinitely high vertical wall. SBEACH does not model the detailed physics at the structure: wave runup, wave reflection, and local scour. However, SBEACH does capture the larger scale effect of the structures on the profile change such as preventing wave overwash and starving the area immediately seaward of the structure of sand.

The effect of the structures storm induced profile change to the location of the structure is presented in Figure 5-1. The top panel shows an example of storm-induced erosion if the structure is located on the berm. In this example the structure does not have a significant impact on the profile erosion. The bottom panel of Figure 5-1 shows an example of storm induced erosion when the structure is located in the foreshore. In this example (bottom panel) the structure has an impact on the profile change and causes an increase in erosion immediately seaward of the structure.

The line of protection was selected to minimize the risk of storm-induced profile lowering at the toe of the structure, which could lead to structural instability. Local scour may occur at the structures due to wave breaking, turbulence, and wave reflection, which is not accounted for in SBEACH.

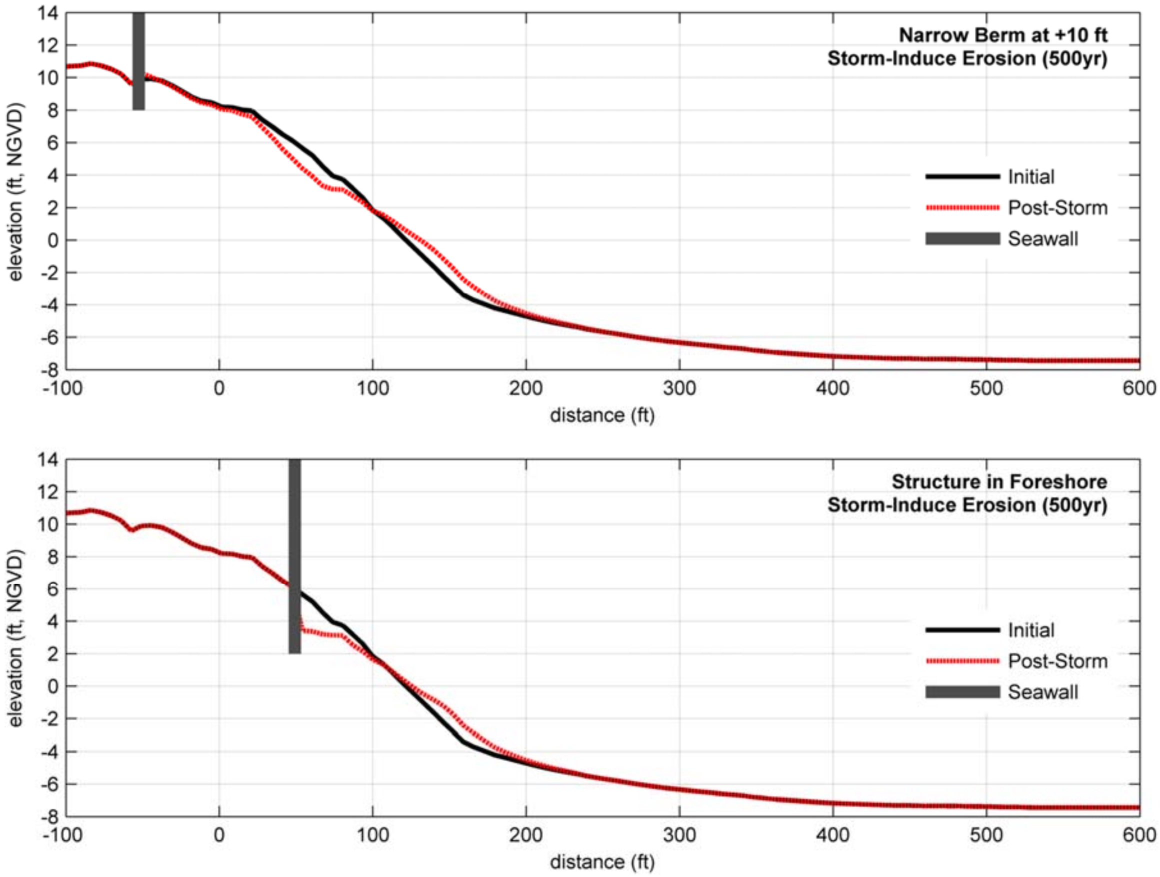


Figure 5-1: Sensitivity of Storm Induced Erosion to Structure Alignment

5.3 Surf Zone Wave Conditions

Goda (2000) developed a computational model of wave transformation in the surf zone which accounts for wave shoaling, random wave breaking, wave setup, and surf beat. The model relates the wave height inside the surf zone to the wave steepness and nearshore slope. As the wave steepness decreases (longer waves relative) and profile becomes steeper, wave shoaling increases and the wave heights increase. The impacts of the profile slope and wave steepness are most pronounced near the breaker line. Further inside the surf zone (e.g. depth limited waves) the impacts of the profile slope and wave steepness are smaller.

An analysis of the beach profile characteristics in the study area (Section 2.1.2) indicates that the nearshore and onshore beach slope is fairly consistent throughout the project area. The shoreline may generally be characterized by a gentle offshore slope (e.g. 1:100) and steep nearshore and onshore beach slope (e.g. 1:10). Most of the shoreline is characterized by a gently sloping beach berm between 8 and 10 ft NGVD. Figure 5-2 shows the beach profiles at four locations along the Project Area aligned at MHW, highlighting the similarity in the offshore, nearshore, and onshore beach slopes.

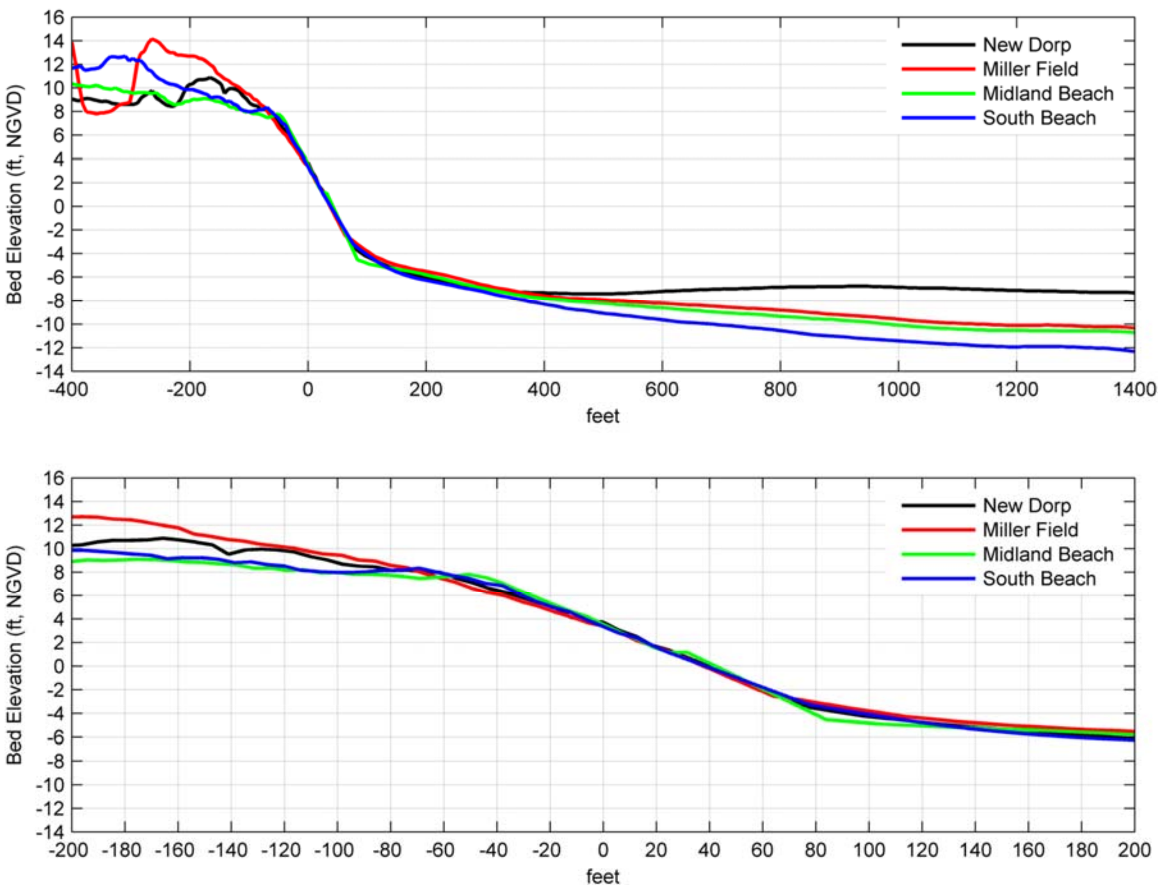


Figure 5-2: Aligned Beach Profiles in Project Area

As a result of the alongshore uniformity in the beach profiles the most distinguishing factor in the wave conditions at the structures is the alignment of the structures that form the Line of Protection. If a structure is set back from the shoreline and located near the back of the berm than the wave conditions (depth limited) will be relatively small due to the high berm elevation (+10 NGVD) and gentle beach slope immediately seaward of the structure. In contrast, if the structure is located along the steep foreshore, than the wave heights will be considerably larger since the water depth and beach slope will be greater. Consequently, a considerable reduction in wave height at the structure may be achieved by setting the alignment as far landward as possible.

The entire line of protection may be characterized by four scenarios that describe the beach conditions on the seaward side of the structures:

- Scenario 1a - Narrow Berm at +10 ft NGVD (Buried Seawall), 1:15 beach slope;
- Scenario 1b – Narrow Beach at +8 ft NGVD (Buried Seawall), 1:11 beach slope;
- Scenario 2 - Gently sloping upland area at + 10 ft NGVD (Floodwall), 1:100 beach slope;
- Scenario 3 – Wide upland area with fetch-limited wave conditions (Levee).

The wave heights at the toe of the structures for the four scenarios are presented below in Table 5-2. Scenario 1a applies to the majority of the buried seawall where existing beach is relatively wide and the shoreline is stable.

The existing beach is relatively narrow along the last 2,000 feet of the buried seawall near Fort Wadsworth (Station 268+00 to Station 288+00). In addition, the historical shoreline change data shows that this area has experienced shoreline erosion at an average rate of approximately 2 ft/yr. The beach conditions at the end of the 50-year project period of in this 2,000 foot section are characterized by Scenario 1b, which results in larger design wave heights.

The wave conditions along the west tie-off earthen levee are protected from long period ocean swells by an area of high ground. However, the levee may be exposed to locally generated wind waves (fetch limited) during storm events. The wave heights and wave periods for the fetch limited wave conditions were determined using ACES (Table 5-3).

Table 5-2: Depth Limited Wave Heights (Goda, 2000)

Return Period (yr)	SWL (ft, NGVD)	T_p (s)	H'_0 (ft)	Scenario 1a H_{toe} (ft)	Scenario 1b H_{toe} (ft)	Scenario 2 H_{toe} (ft)
100	13.3	13.2	6.78	4.0	6.1	3.0
150	14.3	14.1	6.9	4.8	7.0	3.6
300	15.6	15.2	7.05	5.8	8.2	4.4
500	16.6	16.0	7.2	6.6	9.1	5.1

Table 5-3: Fetch Limited Wave Heights (Scenario 3)

Return Period (yr)	SWL (ft, NGVD)	Wind Speed (mph)	Fetch (ft)	Avg. Depth (ft)	T_p (s)	H_{toe} (ft)
100	13.3	75	3,000	3.3	2.03	1.7
150	14.3	75	3,000	4.3	1.99	1.75
300	15.6	75	3,000	5.6	2.09	2.06
500	16.6	75	3,000	6.6	2.10	2.14

5.4 Wave Runup and Wave Overtopping

Wave runup and mean overtopping rates at structures are calculated based on the EurOtop manual (Pullen et. al. 2007). All the overtopping formulas applied herein require the wave height at the toe of the structure as an input. Other key parameters are the freeboard, structure slope, and wave period. The wave runup and overtopping presented herein are based on the deterministic EurOtop formulas. The deterministic formulas are recommended for deterministic design, which include one standard deviation from the mean to account for the scatter in the empirical data.

The wave runup height is given by R2%. This is the wave runup level, measured vertically from the still water line, which is exceeded by 2% of the number of incident waves.

The overtopping rate, Q, is given as the mean overtopping discharge (liters/s/m). In reality there is no constant discharge over the crest of a structure, rather periodic events caused by the largest waves. Wave overtopping is generally classified into two types: "green water," where complete sheets of water run up the face of structure and over the crest, and "white water," where spray from wave breaking/dissipation is transported over the structure. The first type of overtopping, "green water," will only occur if the greatest wave runup elevations exceed the crest elevation (i.e. Rmax exceeds crest elevation). The second type, "white water," may occur even if the runup elevations are not greater than the crest elevation.

The calculated wave overtopping rates are very sensitive to the crest elevation or freeboard height of the structure. Required crest elevations were calculated to the nearest tenth of a foot (e.g. 18.1 feet) and then rounded to nearest half-foot increment (e.g. 18 feet) based on the accuracy of the overtopping formulas and precision in construction. The maximum allowable overtopping thresholds were selected based on the overtopping value at which the structure begins to exhibit damage. The structures are designed to handle a considerable amount of wave overtopping and have additional measures incorporated into the design such as splash aprons and scour toe on the leeward of the structures to further mitigate overtopping. Therefore, the structures may be subjected to wave overtopping rates in excess of the threshold before failure.

Buried Seawall

Wave runup and wave overtopping along the buried seawall were calculated using formulas in EurOtop for armored rubble slopes and mounds. The wave conditions at the toe of the structure were set to the depth limited wave conditions for Scenario 1a. The buried seawall has an impermeable core, two-layers of armor stone, and a 1V:1.5H slope. Crest elevations were set based on a maximum allowable overtopping of 50 liters/s/m. The wave runup and overtopping results are presented in Table 5-4.

Table 5-4: Wave Runup and Overtopping Results – Buried Seawall

Return Period (yr)	Crest Elv. (ft, NGVD)	SWL (ft, NGVD)	Tp (s)	Htoe (ft)	R2% (ft)	Qd (liters/s/m)
100	16	13.3	13.2	4.0	14.3	50.0
150	18	14.3	14.1	4.8	16.9	44.1
300	20.5	15.6	15.2	5.8	20.2	43.0
500	22.5	16.6	16.0	6.6	22.8	42.5

Rather than increasing the elevation of the buried seawall along the 2,000 foot section near Fort Wadsworth to accommodate the larger design wave conditions (Scenario 1b), the armored crest width is increased to reduce wave overtopping. The EurOtop manual indicates that wave overtopping is lower for structures with a wide crest (more than 3 armor stones) due to the extra wave dissipation over the crest. The width of the structure crest was increased to 4 armor stones in this section to reduce wave overtopping below the maximum allowable overtopping threshold.

Vertical Wall

Wave overtopping along the vertical wall was calculated using formulas in EurOtop for vertical and steep walls. The wave conditions at the toe of the structure were set to the depth limited wave conditions for Scenario 2. Crest elevations were set based on a maximum allowable overtopping of 50 liters/s/m. The wave overtopping results are presented in Table 5-5.

Table 5-5: Wave Overtopping Results – Vertical Wall

Return Period (yr)	Crest Elv. (ft, NGVD)	SWL (ft, NGVD)	Tp (s)	Htoe (ft)	Qd (liters/s/m)
100	16	13.3	13.2	3.0	53.3[1]
150	18	14.3	14.1	3.6	47.7
300	20.5	15.6	15.2	4.4	50.4[1]
500	22.5	16.6	16.0	5.1	55.1[1]

Notes: [1]Crest Elevation rounded to nearest half-foot, resulting in wave overtopping rate slightly above target threshold.

Earthen Levee

Wave runup and wave overtopping along the levee were calculated using formulas in EurOtop for coastal dikes and embankment seawalls. The wave conditions at the toe of the structure were set to the fetch limited wave conditions for Scenario 3. The Levee has a 1V:2.5H slope and is assumed to have roughness factor of 1.0 (i.e. no reduction due to surface roughness). Crest elevations were set based on a maximum allowable overtopping of 2 liters/s/m. The wave runup and overtopping results are presented in Table 5-6.

Table 5-6: Wave Runup and Overtopping Results – Levee

Return Period (yr)	Crest Elv. (ft, NGVD)	SWL (ft, NGVD)	Tp (s)	Htoe (ft)	R2% (ft)	Qd (liters/s/m)
100	16	13.3	2.0	2.0	4.1	1.3
150	17	14.3	2.0	2.0	4.1	1.3
300	18	15.6	2.0	2.0	4.1	2.2[1]
500	19	16.6	2.0	2.0	4.1	2.2[1]

Notes: [1]Crest Elevation rounded to nearest half-foot, resulting in wave overtopping rate slightly above target threshold.

5.5 With Project Coastal Impacts

The beach along the South Shoreline of Staten Island is a buffer between the Line of Protection (LOP) structures (earthen levee, concrete vertical floodwall, and buried seawall) and Raritan Bay, dissipating wave energy and insulating the LOP structures from short and long-term changes in shoreline position. The alignment of the LOP structures was selected so the structures are set back and elevated, minimizing their exposure to storm induced water levels and waves except during infrequent extreme events (i.e. 25-year event and greater). The with-project coastal impacts are expected to be minor for the LOP structures.

5.5.1 *Erosional Impacts from Line of Protection*

Hall and Pilkey (1991) categorized the possible mechanisms for beach degradation from shore parallel seawalls as (1) placement loss, (2) passive erosion, and (3) active erosion.

Placement loss is described as the loss of useable recreational beach due to the construction of a seawall seaward of the mean high water line. In these instances the available beach width has narrowed and the sand supply to the beach from the berm is cut off. Placement loss also characterizes the loss of sediment supply from eroding dunes or bluffs behind the seawall.

Passive erosion may occur along eroding shorelines and is described as gradual narrowing of a beach due to the fixed landward boundary of the beach (i.e. seawall). Eventually if erosion is severe the entire beach fronting the seawall may disappear.

Active erosion describes any process that accelerates beach erosion due to the presence of the structure. There is not a consensus among the scientific community about the role and importance of active erosion. Kraus (1988) reviewed over 100 articles on the effects of seawalls on beaches and concluded that the impact of seawalls on cross-shore processes is relatively minor and are only potentially damaging when longshore processes are interrupted (e.g. seawall sticking out from the shoreline acting as a headland or groin).

In general the impact of all three mechanisms for this project are expected to be minimized by the selected alignment of the structures comprising the Line of Protection and relatively stable shoreline positions in the project area. Placement losses are minimized by positioning the buried seawall at the landward edge of the beach. Since the majority of the South Beach, Midland Beach, New Dorp Beach, and Cedar Grove Beach shorelines lack dunes or bluffs to supply sediment to the littoral system, the storm induced modeling results indicate that the buried seawall location is positioned landward of the active littoral zone to avoid placement losses (e.g. cutting off supply of sand from berm/dune). In some instances, the buried seawall may actually increase sediment in the system by blocking overwash and wind transport. The sand cover on the buried seawall will also provide a layer of erodible material that will help supply sediment to the beach.

Similarly, passive erosion is expected to be minor since the shoreline positions are relatively stable in the project area. If there is a significant acceleration in sea level rise in the future than the beach widths may narrow.

Active erosion is also expected to be minor due to the setback location of the structure. The structure will only be exposed during storm events with a return of 25 years or more. It is expected that during these storm events there will some additional local scour near the toe of the structure. However, the scoured sediment is not lost from the system and may recover naturally following the storm. The low probability of occurrence for storm events exposing the seawall are therefore unlikely to result in any significant impacts in the sediment budget, which is dominated by more frequent storms and longshore sediment processes.

An analysis was undertaken to identify a minimum beach width that shall be maintained over the project life to provide a protective beach buffer if accelerated sea level rise would occur. This minimum beach width shall also maintain the performance of the project. The performance of the Line of Protection is tied to the beach conditions which dissipate wave energy and prevent the structures from being undermined. Identification of a minimum beach width for the project is based on analysis of the impact of the structures on storm induced beach change. The location of the structures relative to a typical beach profile is evaluated at several locations across the beach profile to identify at what location the structures begins to affect storm induce beach change. The analysis is based on simulations with the storm induced beach model (SBEACH). A typical beach profile developed for the storm-induced shoreline change modeling discussed in Section 5.2 was used.

The location of the structures are treated as a vertical wall and represents their seaward toe. Simulations of the structures at a location 150, 125, 100, 75, and 50 feet landward of MHW are evaluated, as well as a simulation without any structures. Simulations are performed for a range of storm events as described in Section 3.2.2. The results as shown in Figure 5-3, indicate the structures begin to have a significant impact on the storm induce beach change when it is located within the steep foreshore (i.e. 50 feet landward of MHW). In instances where the structures are located 75 feet or more landward of MHW, the structures do not significantly affect the storm induced profile change. The model results show that in some instances the structures may actually help keep sediment within the active beach by preventing overwash.

Since the long-term sediment budget for the project area indicates that the beach is relatively stable, it is not anticipated over the project period of analysis (50-years) for the beach to erode below the minimum 75-foot threshold. A project cost to maintain the beach was not included for this reason.

The long-term beach erosion rate may be affected by climate variability, including increasing sea level rise and frequency/duration of coastal storm events. If the long-term beach erosion accelerated such that the minimum beach width of 75 feet were reached, beach maintenance/restoration activities may be evaluated. The implementation of beach maintenance/restoration as a future project adaptation would be based on a future decision document that would evaluate and record the changed metrological and oceanographic conditions.

5.5.2 *Erosional Impacts of Any New Drainage Structures*

Any new drainage structures will be built adjacent to existing drainage structures and extend the same distance seaward. Since the existing drainage structures are already impermeable, increasing the width of these structures would have little impact on longshore sediment transport or the sediment budget.

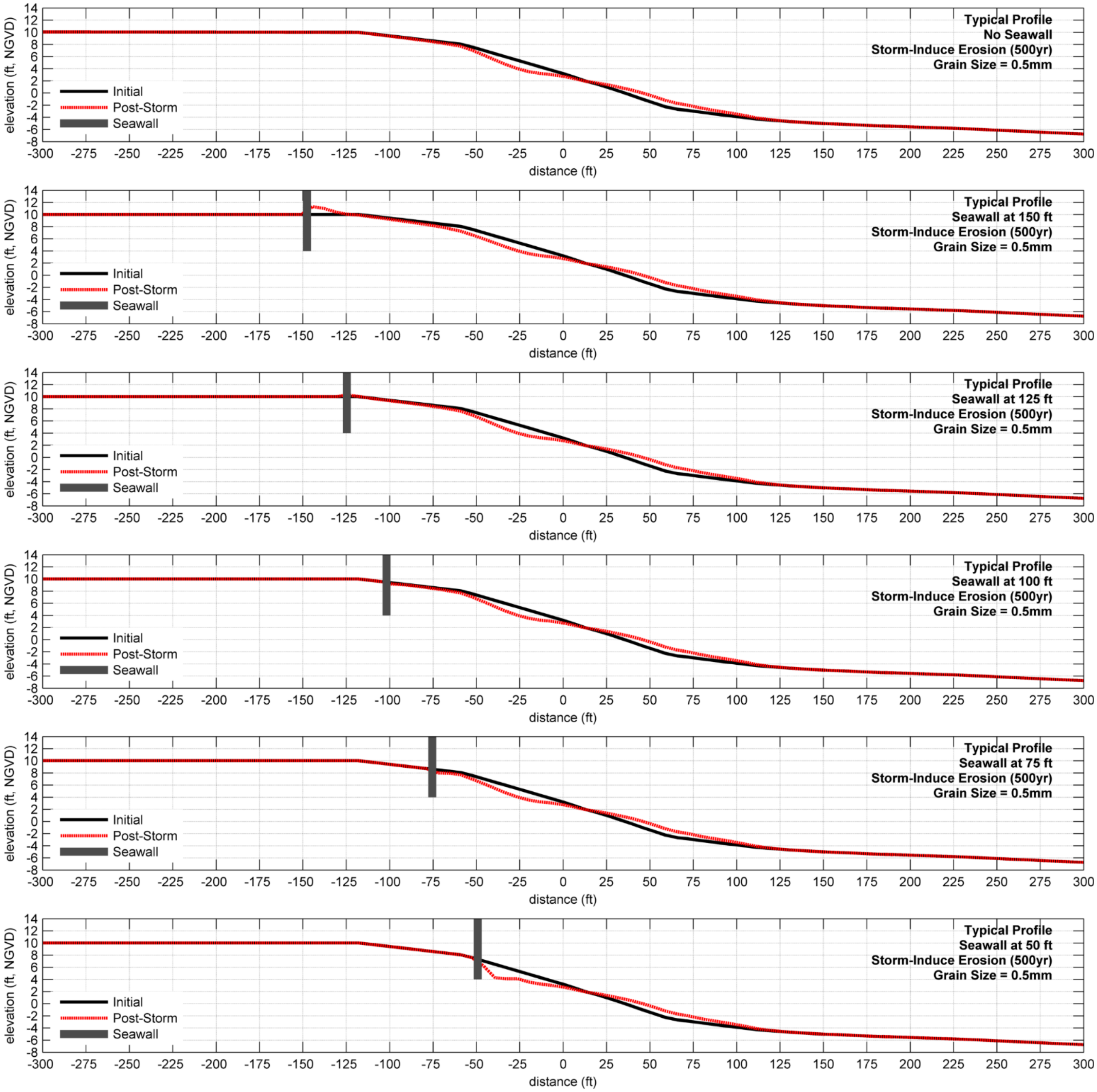


Figure 5-3: SBEACH Model Results

6.0 MONITORING PLAN

This monitoring plan is the basis to evaluate the structural condition and performance of the Line of Protection (LOP) once implemented along the south shore of Staten Island. The LOP consists of three primary flood and wave protection structures along the 4.6 mile shoreline; 1) a buried seawall, 2) vertical concrete floodwall, and 3) earthen levee. These structures will be constructed to minimize potential damage to existing and future infrastructure landward of the proposed structure due to storm surge and waves. This monitoring plan outlines the requirements to document the original condition, location, outline and elevations as well as provide a detailed inspection procedure for each of the structures. In addition, this plan utilizes a tiered inspection approach that adjusts the required frequency and complexity of the inspection components depending on the recorded performance of the structures. This monitoring plan also provides requirements for weather-triggered and special event inspections and evaluate impacts of structures on coastal processes, environment, and recreational resources.

6.1 Pre-Construction Monitoring

Topographic surveys shall be performed to document baseline conditions of the shoreline and beach and existing site infrastructure that will be impacted by the structures prior to the construction. In addition, geo-rectified aerial photography of the project site will be taken. This baseline assessment will serve as the basis for comparison with future monitoring events.

6.1.1 *Topographic Survey*

Topographic surveys shall consist of a collection of three dimensional coordinate data points measured with survey grade instrumentation. Profiles or transects will be surveyed every 500 feet along approximately 24,500 linear feet of shoreline. In addition, transects shall be surveyed on the updrift and downdrift sides of existing outfall structures. The transect will extend from 15 feet landward of the proposed structure (coinciding with levee easement line) to the mean high water line (MHW) line, with a sufficient number of survey points along the transect to identify existing structures (elevated and at-grade boardwalks/trails), the landward and seaward toes and crests of structure and cover material (where appropriate), the beach berm and beach slope to the MHW line.

6.1.2 *Geo-Rectified Aerial Photography*

Geo-rectified digital images of pre-construction conditions will be produced at a scale of 1:500 from aerial photography prior to construction to identify shoreline position, vegetation coverage, beach plan shape, tidal marsh morphology, and land use to compare future conditions of these shoreline features.

6.2 Post-Construction Monitoring

The monitoring plan (Plan) will assess the structural performance of the structures using a three tiered inspections that include the following elements: topographic surveys, geo-rectified aerial photographs, and visual inspections. The Plan establishes a schedule of inspections and surveys. The Plan allows for the adjustment of the schedule after a sequence of favorable inspections occurs. The Plan also includes provisions for inspections of the system after the occurrence of severe weather and special condition events. The program elements are described in detail in the following sections.

6.2.1 *Visual Inspection*

The buried seawalls, vertical floodwall, earthen levees, tide gates, and outfall structures will be visually inspected for general damage caused by natural and man-made activities including severe weather. Inspectors should note, at a minimum, any of the following anomalies:

<u>Buried Seawall</u>

1. Overall structural stability of the buried seawalls
2. Formation of voids in armor stone layer (missing armor stones)
3. Displaced armor stones
4. Loss of fines from between armor stones
5. Exposure of the underlayer of stone
6. Change in elevation of the crest
7. Appearance of scouring patterns at the toe of structure
8. Exposure of filter fabric layers
9. Closure structures
10. Sand and/or fill material coverage
11. Vegetation coverage

<u>Vertical Floodwall</u>

1. Overall structural stability of concrete wall including sliding/overturning.
2. Concrete integrity including spalling, cracking, and exposed reinforcing steel.
3. Active erosion or scouring at toe of structure
4. Displaced stones and exposure of underlay stone or filter fabric on scour blanket and splash pad
5. Seepage
6. Location of vegetation in proximity to structure

Levee

1. Embankment and foundation seepage including:
    a. Settlement
    b. Cracking
    c. Seepage
    d. Sod/Vegetation coverage
    e. Unwanted vegetation growth
    f. Animal Control
2. Embankment Stability/erosion including:
    a. Slope stability
    b. Settlement
    c. Depressions/rutting
    d. Cracking
    e. Erosion/bank caving
    f. Toe erosion/scour

Tide Gate and Outfall Structures

1. Overall structural stability of foundation and concrete chamber of outfall structure.
2. Overall structural stability of foundation and wing walls of tide gate.
3. Concrete integrity including spalling, cracking, and exposed reinforcing steel for outfall and tide gate structures.
4. Integrity and condition of pre-engineered bridge spanning tide gate.
5. Movement of gate actuator and sluice gates at the tide gate.
6. Condition of sluice gates and bar screens at the tide gate.
7. Movement of flap and sluice gates at outfall structures including manual and electric actuation systems for sluice gates.

6.2.2 *Topographic Survey*

Topographic survey of the buried seawall and earthen levee will be performed based on survey profile spacing of 500 feet. The number of survey points along the profile shall be sufficient to identify the landward and seaward toes, the landward and seaward crest or cap limits, the side slopes, the horizontal extent of the splash scour blankets. The survey shall extend from 15 feet landward of the structure toe (coincides with easement) to the Mean High Water line.

Long profile surveys will be performed at an additional ten locations along the project area. The profile locations will be consistent with the 2000 beach profile survey (Table 2-1). These surveys will capture the entire subaerial and submerged profile from 15 feet landward of the structure toe to 4,000 ft to 6,000 feet offshore, well beyond the depth of closure.

6.2.3 *Geo-Rectified Aerial Photography*

Geo-rectified digital images of post-construction conditions will be produced as discussed in Section 6.1.3

6.3 Weather Triggered Inspection

A weather triggered inspection shall take place following a severe weather event. A weather triggered inspection shall follow all of the inspection and reporting requirements of a visual inspection as specified in Section 6.2.1 Visual Inspection.

As a minimum, a visual inspection shall be completed when National Weather Service (NWS) defined severe weather conditions occur at or within ten miles of the site.

1. Sustained winds of 58 mph or greater are measured by any available anemometer at Newark International Airport or Ambrose Light Station.

2. NOAA predicted elevated water levels of 5 feet above Mean High Water.

6.4 Special Inspection

A special inspection is a visual inspection conducted in accordance with the procedures and requirements outlined in Section 6.2.1 Visual Inspection. A special inspection is required when an unusual and unanticipated incident occurs that has the potential to cause damage to a structure. Vehicular damage to the earthen levee is an example of a special condition that would require a visual inspection. A special inspection can be localized to location along the Line of Protection if the conditions warrant it. Special inspections are case-by-case events and have no scheduled frequency, follow-up inspection requirements and do not affect the normal visual inspection schedules.

6.5 Monitoring Plan Schedule

6.5.1 *General Description*

There shall be three inspections of the structures as follows:

- Baseline survey
- Annual Survey
- Five (5) Year Survey

6.5.2 *Inspection Schedule*

<u>Baseline Survey</u>

A baseline survey shall be performed as soon as practical after construction has been completed. This baseline inspection shall serve the purpose to set the original as-built conditions of the structures and the surrounding land.

<u>Annual Inspection</u>

All structured shall be visually inspected) each year and after severe weather and special incidents. If Watch List items from previous visual inspections are determined to be degrading or if significant anomalies are identified during the inspection, operation and maintenance activities will commence.

<u>Five (5) Year Inspection</u>

A regularly scheduled comprehensive visual inspection of all structures and topographic survey of the earthen levee and buried seawall shall be performed at 5-year intervals. After the second 5-year inspection, the interval shall increase to 10-years.

If the analysis of two subsequent surveys shows that no substantial change in the structures, the beach berm, and MHW line during the survey interval, then regularly scheduled 5-year inspections surveys may be suspended.

6.6 Survey Report Requirements

6.6.1 *Topographic survey*

The following shall be provided:

1. Fully contoured plan view drawings of each surveyed structure.
2. Cross section drawing of each surveyed structure at 500 foot intervals.

3. Analysis of the 3-D data sets of the baseline, preceding and current surveys to produce plots of relative elevation changes between the surveys.

4. Plan and cross section drawing comparing the present survey with both the most recent past survey and the baseline survey.

5. Written report including: analysis of any change in shape or elevations of LOP structures found during the survey comparison and recommendations for any necessary repairs.

6. An electronic copy of the topographic survey data.

6.6.2 *Visual Inspection*

Comprehensive Written Report

The inspecting firm shall submit a detailed, comprehensive report identifying all observed conditions that were noted during the visual inspection. The established stations shall be used to locate these observed conditions. The report shall include a Watch List of all observed minor anomalies and document their conditions in sufficient detail to allow the determination in subsequent inspections if the conditions are stable or deteriorating. If either deteriorating Watch List items or significant anomalies are observed the report shall recommend appropriate Level 3 surveying. If the visual inspection is weather triggered or special condition triggered, a detailed summary of the weather event or special condition shall be included in the inspection report.

Observed Conditions Drawing

A scaled drawing in plan view shall be provided showing any observed conditions and their location (station) on drawings.

Digital Photographs

Digital photographs of both overall typical views of structures and specific observed conditions shall be provided. All photographs shall include a ruler or similar measuring device to provide a graphic scale. A minimum of three (3) photos per structure shall be provided. Photo documentation consisting of the name of the photographer, the date, time, location, and description of the photograph shall be provided for each photograph submitted. A disc containing all of these photographs shall also be provided.

6.7 Survey Record Maintenance

Copies of survey reports and records of maintenance/repair activities for LOP structures shall be maintained by the New York District, US Army Corps of Engineers.

7.0 MAINTENANCE PLAN

The maintenance plan applies to the Line of Protection (LOP) structures along the South Shore of Staten Island. The Line of Protection structures are intended, in conjunction, with interior drainage features, to reduce coastal flooding from severe winter storms and hurricane events. Major project components implemented to reduce flooding along 5.5 miles of shoreline include a buried seawall, vertical concrete floodwall, and earthen levees.

The performance of the LOP plan will continue to meet its design intent if it is properly maintained during normal (non-storm conditions) and properly operated during times of nor'easters and hurricane flooding events. The need for proper maintenance of the LOP is critical given the potential damages to infrastructure in this urban area if deterioration or damage to structures due to lack of maintenance fail during the storm event. Maintenance and proper operation require that the personnel overseeing the LOP structures understand the functionally aspects of the individual structure and the best means of maintaining the project during non-storm events as well as operating the system during a storm event.

This O&M will provide an overview of operational considerations during storm events, specific maintenance works to be performed, and the frequency or timing of work. The evaluation and need for maintenance is based on the needs assessment conducted during the monitoring period and outlined in the Monitoring Report.

7.1 Maintenance

A summary of maintenance requirements for each Line of Protection structure is provided. Maintenance is defined as the upkeep and repair of structures to maintain the function of the structure after construction is complete.

7.1.1 *Buried Seawall*

The primary maintenance of this structure is the repositioning of armor and bedding stones that may be displaced during storm event. Additional maintenance on the buried seawall will also include repair and/or replacements the protective material cover, vegetation and associated reinforcing matting. Specific maintenance activities include:

<u>Displaced/Dislodged Stones</u>

Repair of displaced/dislodged stones is initiated once damage exceeds thresholds based on visual operations and surveys taken during the monitoring period. The basis of the damage evaluation for these two structures is the non-dimensional damage level variable, S, defined as:

$$S = \frac{A_e}{D_{n50}^2}$$

, where

$A_e =$ Area of eroded section

D_{n50}= median rock diameter

Damage classifications are those outlined in the Coastal Engineering Manual[1] are:

- Initial damage – few stones are displaced in spot locations. This corresponds to the no damage condition in relation to the Hudson formula stability coefficient where no damage level is defined as 0-5% displaced units. This corresponds to an S value of 2 for a two layer armor design.

- Intermediate damage – Units are displaced but without causing exposure of the under or filter layer to direct wave attack. This corresponds to an S value of 3-5 for a two layer armor design.

Repairs options shall vary depending on the damage level. If the monitoring report indicates initial damage to the armor layer, this layer shall be repaired by replacing the dislodged armor stones with stones of similar type and size. The reuse of displaced armor stones, supplemented with new units, is acceptable providing the old armor stones are still sound and have not broken into pieces.

If the monitoring report indicates damage level corresponds to a value of S of 3 or more, then repair will consist of two options: 1) two-layer stone overlay or 2) replacement of the cross section. The overlay option may be applied at transitions between adjoining LOP structures where interlocking with adjacent armor layers is possible to create a cohesive structure. At all other locations, the cross section shall be repaired to the origin design intent. The reuse of displaced armor stones, supplemented with new units, is acceptable providing the old armor stones are still sound and have not broken into pieces.

Material Cover and Vegetation

The material cover on the landward and seaward slopes of the buried seawall may be eroded due to wind-borne transport, high water and wave events, animal burrows, and human activity. Similarly vegetative cover may fail to establish or die-off. If the thickness of the material cover is less than 6-inches, these erosive areas should be immediately repaired by replacing the lost material. The repaired area should then be stabilized using and reinforcing mat or fabric and replanting to reestablish the vegetative cover. The vegetative cover shall be replanted and monitored for a period of 3 months to ensure establishment.

If the material cover is sufficient but vegetative cover has not established or died off within a 100 sq. foot area, the vegetative cover shall be replanted and monitored for a period of 3 months to ensure establishment.

Recreational Trial and Access Ramps

The crown of the buried seawall and the access ramps should be properly maintained and kept serviceable. This work involves repairing the roller compacted concrete or asphalt top coat.

[1] U.S. Army Corps of Engineers, draft. Coastal Engineering Manual, EM 1110-2-1100 (Part VI), pageVI-5-60. URL: http://chl.erdc.usace.army.mil/cem

7.1.2 *Floodwall*

Maintenance of the concrete T-shaped floodwall is based on maintaining the integrity of the structure, which may be reduced due to loss of material at the toe of the structure and/or liquefaction of soil due to poor drainage. In addition, repair of the concrete shall be performed to minimize corrosion of the reinforcing steel within the concrete.

Repair of the scour and splash blankets protecting structure toe is similar to the procedures outlined for the buried seawall. When damage levels are identified as initial, displaced or dislodged stones should be repositioned or replaced with new stones of similar type and size. If damage levels are considered immediate, the damaged section should be removed and replaced by a cross section that contains stones of similar size and type to original design.

The geotechnical conditions in the coastal environment have been identified as primarily permeable materials that would not result in long-duration ponding (>2 days) on the land and/or seaward side of the structure. If ponding does occur, an investigation should be undertaken by a qualified engineer to determine the cause and extent of the ponding. Remediation measures shall be undertaken if required.

Repairs to concrete spalls should be initiated if concrete coverage over reinforcing steel is less than one (1) inch. Repair work shall consist of removing the deteriorated concrete and installing and patching with a cementitious concrete product.

7.1.3 *Earthen Levee*

Earthen levees shall be maintained to remedy any adverse conditions threatening the integrity of the structure. The following sections identify the activities and repair recommendations for this structure.

Crown Roadway and Access Ramps

The levee crown should be maintained and all crown roadways, ramps, and access roads should be properly maintained and kept serviceable. This work involves periodically grading and gravelling road surfaces.

Rodent Activity

Squirrels and other burrowing rodents can threaten the structural integrity of levees by loosening soil, increasing the risk or erosion and sloughing, and increasing the likelihood of piping-type erosion failures. Therefore, a rodent control program should be implemented year-round for the levees.

Vegetation Management

Mowing, burning, spraying, and other vegetation management procedures should be implemented on an annual basis. Broadleaf weeds growing among desirable grasses should be controlled by selective herbicides. Ground cover should be maintained at 12 inches in height or less. Trees and shrubs are not permitted to grow on levee slopes or crown. Any plant that obscures the view from the crown of the levee to the toe where boils and leaks would be most likely to occur should be removed. All vegetation over 2 inches in diameter should be removed from an area that extends for fifteen feet from the waterside and landside toes of the levee.

In general, vegetation within any existing access easements landward and seaward of the levee toe shall be limited to groundcovers to allow unimpeded maintenance activities, inspections and flood fighting. Vegetation should be maintained in such a manner as to allow for unimpaired passage and operation of maintenance equipment and flood fight efforts.

Erosion Control and Repair

Dragging of the levee slopes to repair minor surface erosion or irregularities and prevent serious erosion should be performed annually. Areas of significant erosion, as determined by a qualified Engineer, should be over-excavated and filled with compacted backfill. The material properties and compaction requirements for the backfill should be the same as specified for the original project construction. The repaired area should then be stabilized using an erosion mat or fabric, as approved by the Engineer, and reseeded to reestablish the ground cover.

Seepage

Areas of heavy seepage and/or boils should be immediately reported to and evaluated by a qualified engineer and remedial measures implemented, as determined necessary.

Cracking, Settlement and Slips

All cracks in the levee crown or slopes should be repaired using the following procedure: 1) remove and salvage the gravel surfacing material on the levee crown if applicable; 2) excavate the levee crown and/or slope along the crack to the full depth of the crack; 3) backfill with compacted clayey material placed in thin lifts and meeting the material property and compaction requirements for the original levee construction; 4) replace and compact the gravel surfacing over the levee crown; and 5) stabilize the repaired area on the levee slope using an erosion mat or fabric and reseed it to reestablish the ground cover.

All slips in the levee crown or slopes should be repaired using the following procedure: 1) Remove and salvage the gravel surfacing material on the levee crown; 2) excavate and remove the entire slip or crack surface to ensure that the failure plane and all failed materials (since these materials would thereafter only obtain residual strength) are completely removed and; 3) backfill with compacted clayey material placed in thin lifts and meeting the material property and compaction requirements for the original levee construction; 4) replace and compact the gravel surfacing over the levee crown; and 5) stabilize the repaired area on the levee slope using an erosion mat or fabric and reseed it to reestablish the ground cover.

8.0 REFERENCES

Alfageme, S., 2001. “Selection of Alternative Design Beach Profiles.” Draft Memorandum prepared for the U.S. Army Engineer District, NY.

CERC, 2001. “Wave Statistics for Lower New York Bay – Dredged Material Management Plan”.

FEMA, 2011. “Draft Report, Redefinition of the Coastal Flood Hazard Zones in FEMA Region II: Analysis of the Coastal Storm Surge Flood Frequencies,” Federal Emergency Management Agency, Department of Homeland Security, 500 C Street, SW, Washington, DC.

FEMA, 2013. “Flood Insurance Study, City of New York, NY,” Preliminary: December 5, 2013

Goda, Y., 2000. “Random Seas and Design of Maritime Structures, Advance Series on Ocean Engineering, - Volume 15,” World Scientific, P O Box 128, Farrer Road, Singapore 912805.

Hall, M. J., Pilkey, O. H., 1999. “Effects of Hard Stabilization on Dry Beach Width for New Jersey,” Journal of Coastal Research, V0l 7, No 3, pp. 771-785.

IPCC, 2013: Summary for Policymakers. In: Climate Change 2013: The Physical Science Basis. Contribution of Working Group I to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change [Stocker, T.F., D. Qin, G.-K. Plattner, M. Tignor, S.K. Allen, J. Boschung, A. Nauels, Y. Xia, V. Bex and P.M. Midgley (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA.

Kobayashi, N., Tega, Y., and Hancock, M. 1996. “Wave reflection and Overwash of Dunes,” Journal of waterway, Port, Coastal, and Ocean Engineering. May/June 1996, Vol 122, No. 3.

Kraus, N. C., 1988. “The effects of seawalls on the beach: An extended literature view. In: Kraus, N. C. and Pilkey, O.H.” Journal of Coastal Research, Special Issue No. 4, pp. 1-29.

Larson, M., Kraus, N., C., 1989a. “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., 1989b. “Prediction of Beach Fill Response to Varying Waves and Water Level,” Proceedings of Coastal Zone ’89, American Society of Civil Engineers, pp. 607-621, 1989.

NYC, 2013. “A Stronger More Resilient New York.”

Pullen, T., Allsop, N.W.H., Bruce, T., Kortenhaus, A., Schuttrum,pf, H., Van der Meer, J. W., 2007. *EurOtop: “Wave overtopping of sea defenses and related structures: Assessment Manual,”* http://www.overtopping-manual.com

Smith, J. J., Leatherman, S. P., Gregory, T. F., 1995. "Historical Shoreline Mapping of Staten Island and Jones Beach, New York," Prepared for: The State of New York, Department of State, Laboratory for Coastal Research, 1113 Lefrak Hall, College Park, Maryland.

U.S Army Corps of Engineers (USACE), New York District, 1995, "South Shore of Staten Island, New York, Beach Erosion Control and Storm Damage Reduction Reconnaissance Study," June 1995.

U.S. Army Corps of Engineers (USACE), 1977. Shore Protection Manual.

U.S. Army Corps of Engineers (USACE), 1984. Shore Protection Manual.

U.S. Army Corps of Engineers (USACE), 1999. EM-1110-2-1100, Coastal Engineering Manual.

Van der Meer, J.W. and Janssen, J.P.F.M. 1995. "Wave run-up and wave overtopping at dikes." Wave forces on inclined and vertical wall structures, N. Kobayashi and Z. Demirbilek, eds. ASCE, New York, N.Y., 1-27

Van der Meer, J.W. 1998. A Code for Dike Height Design and Examination. Proceedings, Coastlines, Structures and Breakwaters'98, Thomas Telford, London.

ATTACHMENT A

PLAN SHEETS

(See Section 14 in Main Report)

ATTACHMENT B

COST ESTIMATE

(See Appendix IV)

ATTACHMENT C

COASTAL ENGINEERING CALCULATONS

Date: July 14, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: Without Project Coastal Processes
Design: Deepwater & Wave Breaking Characteristics

Solve for deepwater wave height and breaking wave characteristics

References: Shore Protection Manual (1984) and Coastal Engineering Manual (1999)

Definitions

H_s - significant wave height (general term may be either H_{m0} or $H_{1/3}$).

H_{m0} - spectral significant wave height (4σ)

$H_{1/3}$ - statistical significant wave height (derived from zero-upcrossing or zero down crossing)

H'_0 - equivalent deepwater wave height (hypothetical wave that has undergone wave refraction and diffraction)

T_p - peak wave period

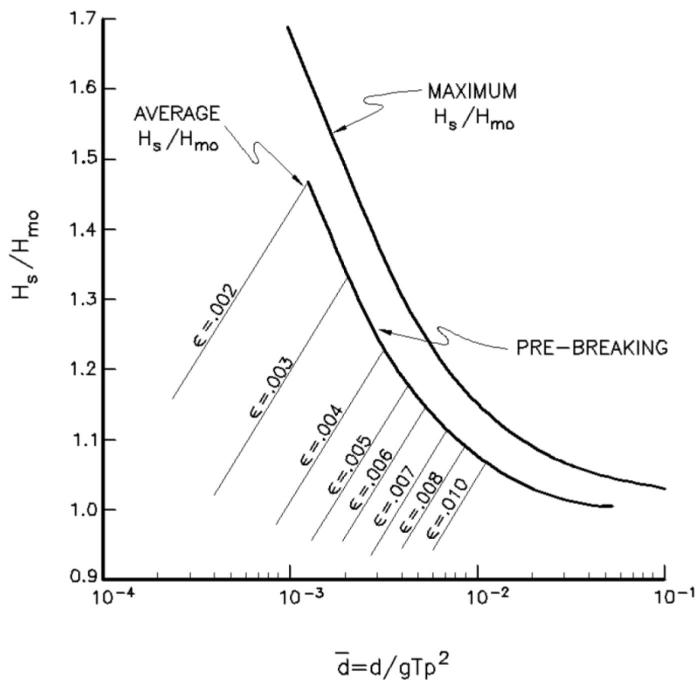
Wave Inputs - Lower New York Harbor Wave Modeling Study (CERC, 1988)

Mean frequency of occurence relationships at Station 8, nominal depth of 17 ft MSL

$$RP := \begin{pmatrix} 2 \\ 5 \\ 10 \\ 25 \\ 50 \\ 100 \\ 200 \\ 500 \end{pmatrix} \cdot yr \qquad Hm0 := \begin{pmatrix} 5.2 \\ 5.4 \\ 5.6 \\ 5.7 \\ 5.8 \\ 6.0 \\ 6.2 \\ 6.5 \end{pmatrix} \cdot ft \qquad Tp := \begin{pmatrix} 5.4 \\ 8.3 \\ 9.7 \\ 11.3 \\ 12.3 \\ 13.2 \\ 14.5 \\ 16.0 \end{pmatrix} \cdot s \qquad d := \begin{pmatrix} 17 \\ 17 \\ 17 \\ 17 \\ 17 \\ 17 \\ 17 \\ 17 \end{pmatrix} \cdot ft$$

Significant Wave Height, $H_{1/3}$

Thompson & Vincent (1985) found that energy based wave height (H_{m0}) deviates from the statistical wave height ($H_{1/3}$) in shallow water prior to breaking. In deep water $H_{m0} \approx H_{1/3}$. After wave breaking $H_{m0} \approx H_{1/3}$. The maximum curve represents the upper limit of the relationship observed in laboratory data (narrow frequency spectrum) and is conservative. Design waves are depth limited, conversion has minor impact on project.



$$db := \frac{d}{g \cdot Tp^2} = \begin{pmatrix} 0.0181 \\ 0.0077 \\ 0.0056 \\ 0.0041 \\ 0.0035 \\ 0.003 \\ 0.0025 \\ 0.0021 \end{pmatrix} \qquad val := \begin{pmatrix} 1.11 \\ 1.2 \\ 1.26 \\ 1.32 \\ 1.36 \\ 1.40 \\ 1.45 \\ 1.49 \end{pmatrix} \qquad Hs := \overrightarrow{(Hm0 \cdot val)} = \begin{pmatrix} 5.77 \\ 6.48 \\ 7.06 \\ 7.52 \\ 7.89 \\ 8.4 \\ 8.99 \\ 9.69 \end{pmatrix} \cdot ft$$

Equivalent Deep water wave height , H'_0

An equiavalent deep water wave height that has undergone wave refraction & diffraction may be calculated from small amptidue wave theory.

$H_s = H'_0 * Ks$, where $Ks = (C_{g0}/C_g)^{1/2}$

$L := 150 \cdot m$ Initial guess

Given

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

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

$$L := \overrightarrow{Wavel(Tp, d)} = \begin{pmatrix} 111.2 \\ 184.3 \\ 218.4 \\ 257.1 \\ 281 \\ 302.5 \\ 333.5 \\ 369.1 \end{pmatrix} \cdot ft$$

$$Ks := \overrightarrow{\sqrt{\frac{1}{\tanh\left(2\cdot\pi\cdot\frac{d}{L}\right)}\cdot\frac{1}{\left(1 + 4\cdot\pi\cdot\frac{\frac{d}{L}}{\sinh\left(4\cdot\pi\cdot\frac{d}{L}\right)}\right)}}} = \begin{pmatrix} 0.92 \\ 1.03 \\ 1.09 \\ 1.16 \\ 1.2 \\ 1.24 \\ 1.29 \\ 1.35 \end{pmatrix}$$

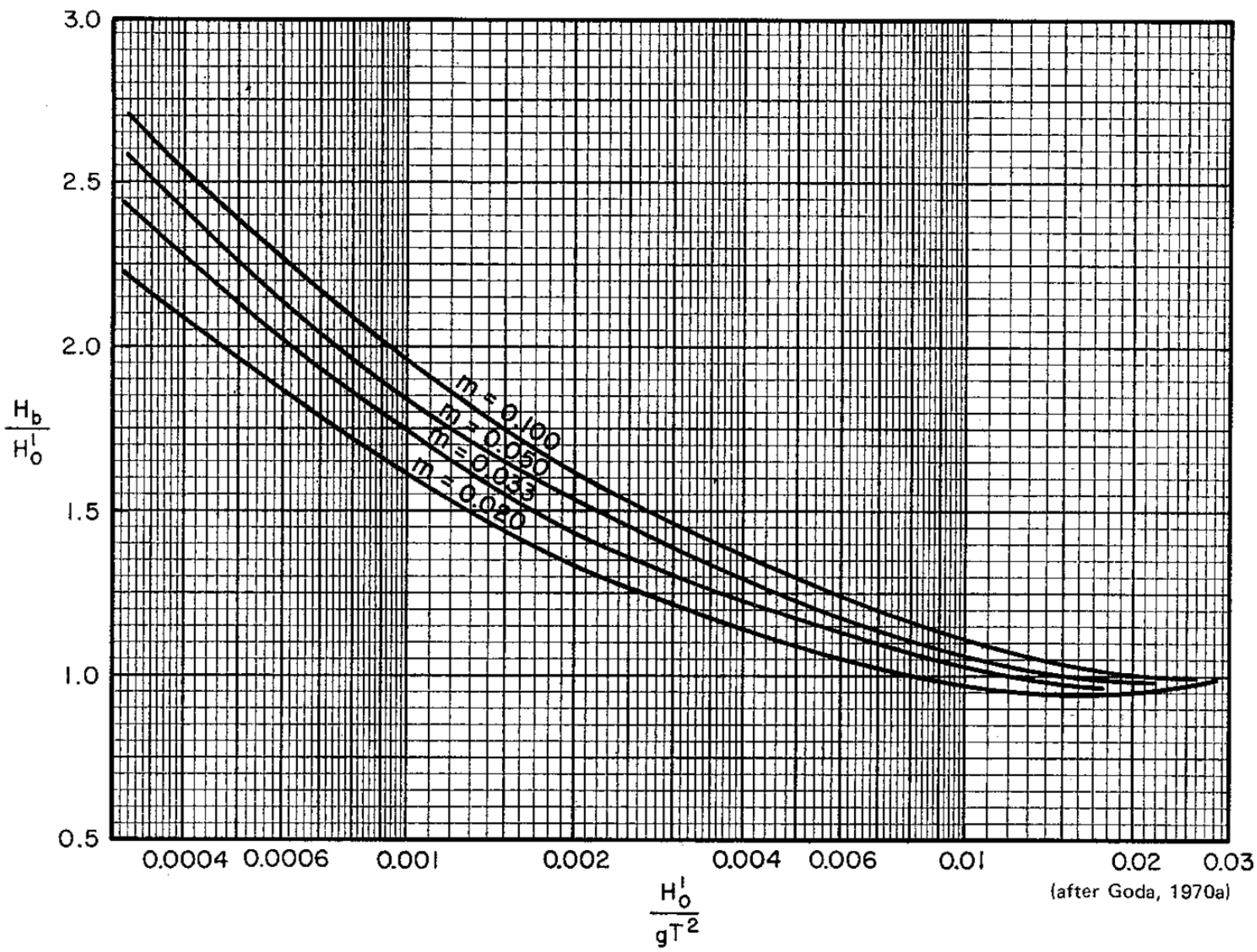
$$(H0) := \overrightarrow{\left(\frac{Hs}{Ks}\right)} = \begin{pmatrix} 6.25 \\ 6.3 \\ 6.47 \\ 6.49 \\ 6.57 \\ 6.78 \\ 6.96 \\ 7.17 \end{pmatrix} \cdot ft$$

Breaking Wave Characteristics

Maximum possible breaking wave height and depth at which it breaks are estimated based on Figure 2-72 and Figure 2-73 in the Shore Protection Manual (SPM-1984).

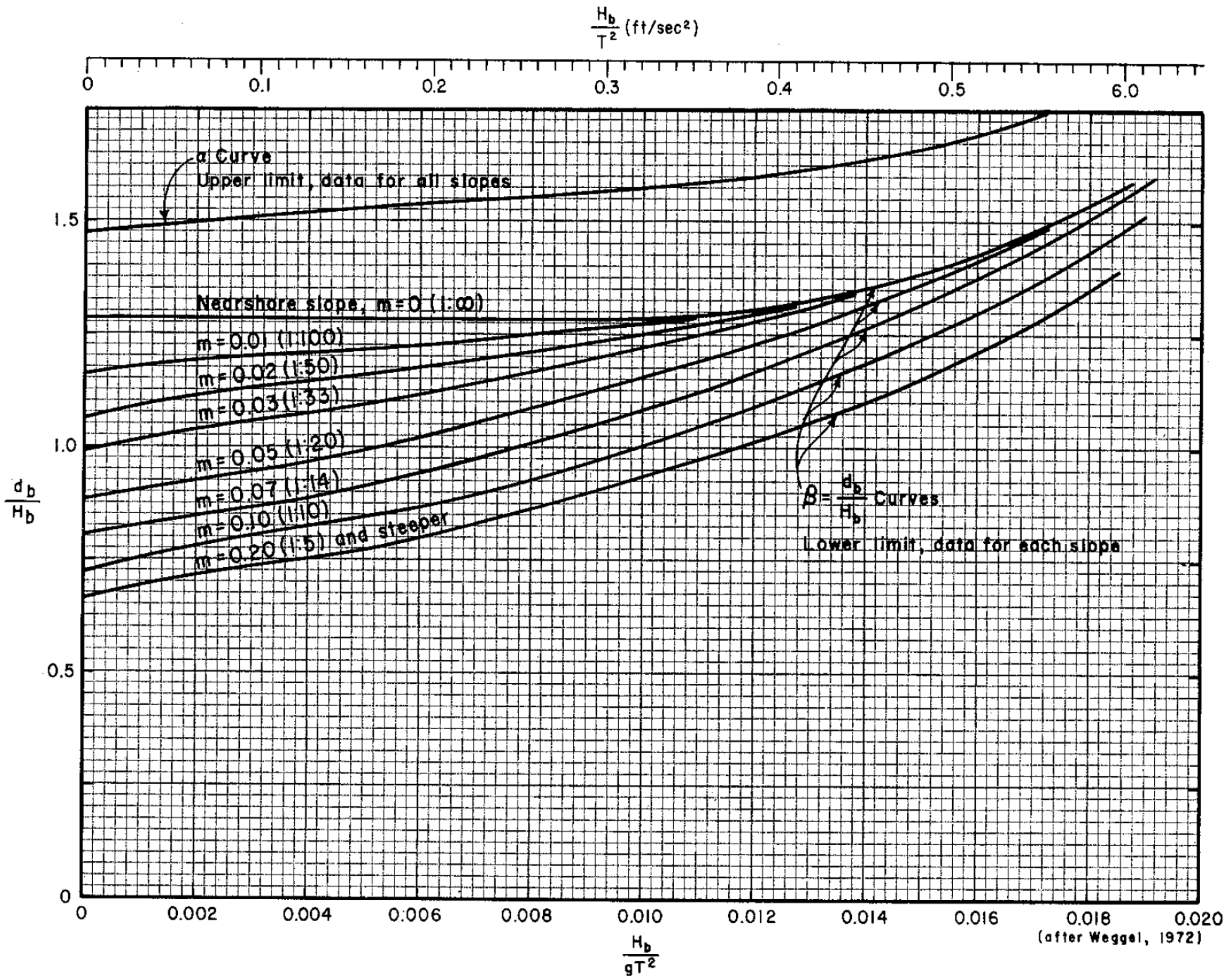
Beach Slope, mn $\quad mn := \frac{1}{10}$

Figure 2-72 (SPM, 1984)



$$xaxis := \frac{H0}{g \cdot Tp^2} = \begin{pmatrix} 0.0067 \\ 0.0028 \\ 0.0021 \\ 0.0016 \\ 0.0013 \\ 0.0012 \\ 0.001 \\ 0.0009 \end{pmatrix} \qquad yaxis := \begin{pmatrix} 1.26 \\ 1.55 \\ 1.6 \\ 1.72 \\ 1.82 \\ 1.86 \\ 1.96 \\ 2.01 \end{pmatrix} \qquad Hb := \overrightarrow{(H0 \cdot yaxis)} = \begin{pmatrix} 7.88 \\ 9.76 \\ 10.36 \\ 11.17 \\ 11.95 \\ 12.61 \\ 13.64 \\ 14.41 \end{pmatrix} \cdot ft$$

Figure 2-73 (SPM, 1984)



The curves in Figure 2-73 are given by:

$$a := 43.75\cdot(1-\exp(-19.5\cdot mn)) = 37.526 \qquad b := \frac{1.56}{(1+\exp(-19.5\cdot mn))} = 1.366$$

$$yaxis := \frac{1}{b-\left(a\cdot\frac{Hb}{g\cdot Tp^2}\right)} = \begin{pmatrix} 0.95 \\ 0.83 \\ 0.81 \\ 0.79 \\ 0.79 \\ 0.78 \\ 0.78 \\ 0.77 \end{pmatrix} \qquad db := \overrightarrow{(Hb\cdot yaxis)} = \begin{pmatrix} 7.5 \\ 8.13 \\ 8.37 \\ 8.84 \\ 9.38 \\ 9.84 \\ 10.57 \\ 11.08 \end{pmatrix}\cdot ft$$

Summary of Wave Characteristics

$$RP = \begin{pmatrix} 2 \\ 5 \\ 10 \\ 25 \\ 50 \\ 100 \\ 200 \\ 500 \end{pmatrix} \cdot yr \quad Tp = \begin{pmatrix} 5.4 \\ 8.3 \\ 9.7 \\ 11.3 \\ 12.3 \\ 13.2 \\ 14.5 \\ 16 \end{pmatrix} s \quad Hm0 = \begin{pmatrix} 5.2 \\ 5.4 \\ 5.6 \\ 5.7 \\ 5.8 \\ 6 \\ 6.2 \\ 6.5 \end{pmatrix} \cdot ft$$

$$Hs = \begin{pmatrix} 5.8 \\ 6.5 \\ 7.1 \\ 7.5 \\ 7.9 \\ 8.4 \\ 9 \\ 9.7 \end{pmatrix} \cdot ft \quad H0 = \begin{pmatrix} 6.3 \\ 6.3 \\ 6.5 \\ 6.5 \\ 6.6 \\ 6.8 \\ 7 \\ 7.2 \end{pmatrix} \cdot ft \quad Hb = \begin{pmatrix} 7.9 \\ 9.8 \\ 10.4 \\ 11.2 \\ 11.9 \\ 12.6 \\ 13.6 \\ 14.4 \end{pmatrix} \cdot ft \quad db = \begin{pmatrix} 7.5 \\ 8.1 \\ 8.4 \\ 8.8 \\ 9.4 \\ 9.8 \\ 10.6 \\ 11.1 \end{pmatrix} \cdot ft$$

Date: July 14, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: With Project Coastal Processes
Design: Surfzone Wave Transformation - Scenario 1a

Solve for wave height at the toe of the structure

References: Goda (2000)

Definitions

H_{toe} - significant wave height ($H_{1/3}$) at toe of structure.

H'_0 - equivalent deepwater wave height (hypothetical wave that has undergone wave refraction and diffraction)

$T_{1/3}$ - average wave period of the highest 1/3 of waves, use peak wave period

Wave Inputs - Derived from Lower New York Harbor Wave Modeling Study (CERC, 1988)

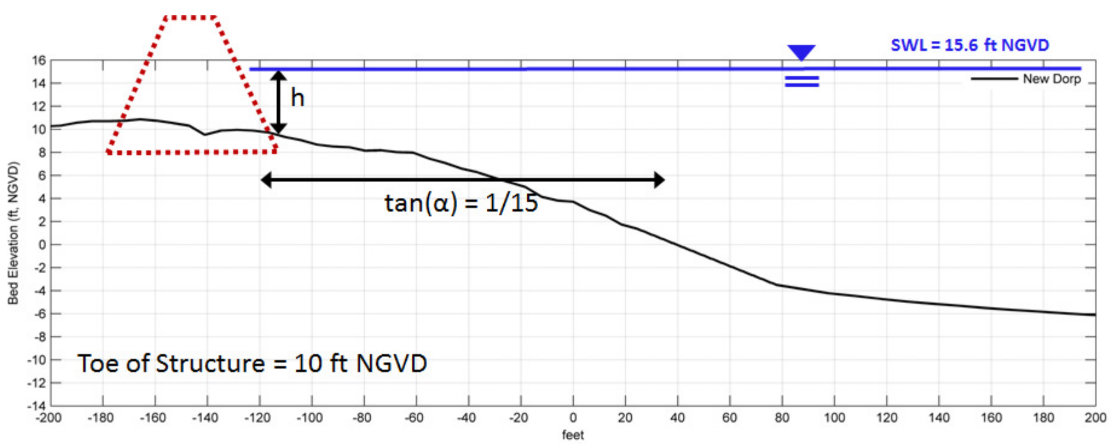
Mean frequency of occurence relationships at Station 8, nominal depth of 17 ft MSL

$$RP := \begin{pmatrix} 100 \\ 164 \\ 309 \\ 500 \end{pmatrix} \cdot yr \qquad swl := \begin{pmatrix} 13.3 \\ 14.3 \\ 15.6 \\ 16.6 \end{pmatrix} \cdot ft \qquad H0 := \begin{pmatrix} 6.78 \\ 6.9 \\ 7.05 \\ 7.2 \end{pmatrix} \cdot ft \qquad Tp := \begin{pmatrix} 13.2 \\ 14.1 \\ 15.2 \\ 16 \end{pmatrix} \cdot s$$

Scenario 1a - Narrow Berm at +10 ft NGVD

$mn := \frac{1}{15}$ Nearshore Slope, m

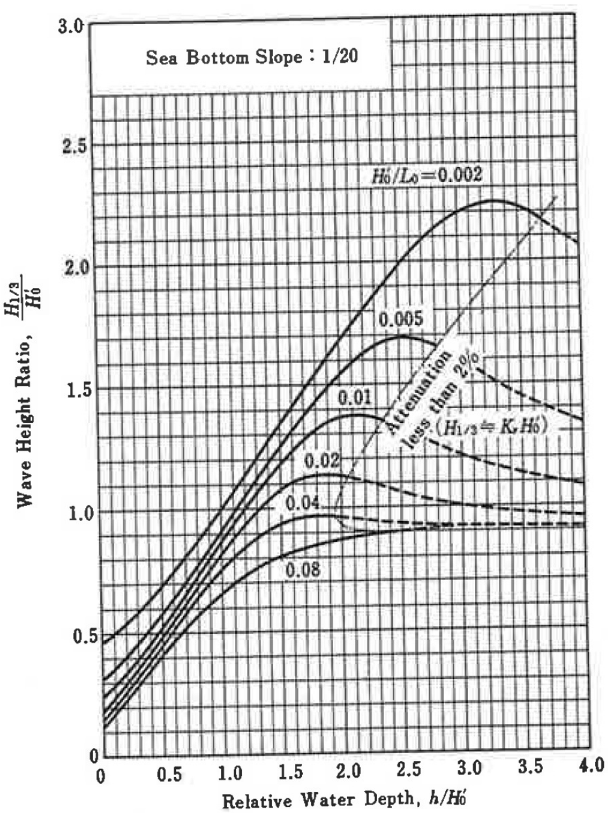
$etoe := 10 \cdot ft$



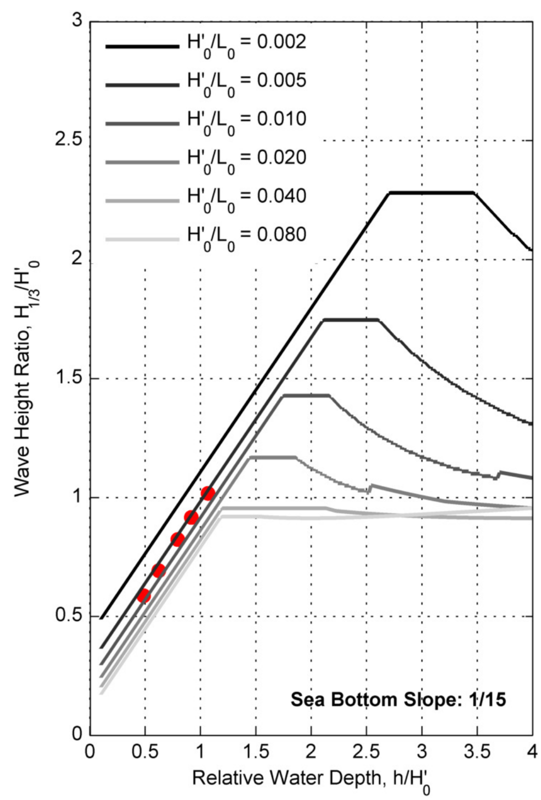
Goda (2000) Computational Model of Wave Transformation in the Surf Zone

Goda (2000) developed a computation model of random wave breaking in the surf zone that provides a tool for estimating wave heights accross the surf zone. The computational model accounts for wave shoaling, random wave breaking, wave setup, and surf beat. The inputs to the model are: offshore wave height (H'_0), wave period ($T_{1/3}$), water depth (h), and nearshore slope (mn).

Goda Diagram (1:20 slope)



Goda Mathematical Model (1:15 slope)



deep water wave length

$$L0 := \frac{g \cdot Tp^2}{2 \cdot \pi} = \begin{pmatrix} 892.2 \\ 1018 \\ 1183.1 \\ 1310.9 \end{pmatrix} \cdot ft$$

water depth (not including wave setup)

$$d := swl - etoe = \begin{pmatrix} 3.3 \\ 4.3 \\ 5.6 \\ 6.6 \end{pmatrix} \cdot ft$$

wave steepness

$$S0 := \frac{H0}{L0} = \begin{pmatrix} 0.0076 \\ 0.0068 \\ 0.006 \\ 0.0055 \end{pmatrix}$$

$$xaxis := \frac{(d)}{H0} = \begin{pmatrix} 0.487 \\ 0.623 \\ 0.794 \\ 0.917 \end{pmatrix} \qquad yaxis := \begin{pmatrix} 0.587 \\ 0.692 \\ 0.823 \\ 0.916 \end{pmatrix}$$

Depth limited wave height (see red dots above in Goda Mathematical Model)

$$Htoe := \overrightarrow{(H0 \cdot yaxis)} = \begin{pmatrix} 3.98 \\ 4.77 \\ 5.8 \\ 6.6 \end{pmatrix} \cdot ft$$

Date: July 14, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: With Project Coastal Processes
Design: Surfzone Wave Transformation - Scenario 1b

Solve for wave height at the toe of the structure

References: Goda (2000)

Definitions

H_{toe} - significant wave height ($H_{1/3}$) at toe of structure.

H'_0 - equivalent deepwater wave height (hypothetical wave that has undergone wave refraction and diffraction)

$T_{1/3}$ - average wave period of the highest 1/3 of waves, use peak wave period

Wave Inputs - Derived from Lower New York Harbor Wave Modeling Study (CERC, 1988)

Mean frequency of occurence relationships at Station 8, nominal depth of 17 ft MSL

$$RP := \begin{pmatrix} 100 \\ 164 \\ 309 \\ 500 \end{pmatrix} \cdot yr \qquad swl := \begin{pmatrix} 13.3 \\ 14.3 \\ 15.6 \\ 16.6 \end{pmatrix} \cdot ft \qquad H0 := \begin{pmatrix} 6.78 \\ 6.9 \\ 7.05 \\ 7.2 \end{pmatrix} \cdot ft \qquad Tp := \begin{pmatrix} 13.2 \\ 14.1 \\ 15.2 \\ 16 \end{pmatrix} \cdot s$$

Scenario 1b - No Berm at +8 ft NGVD

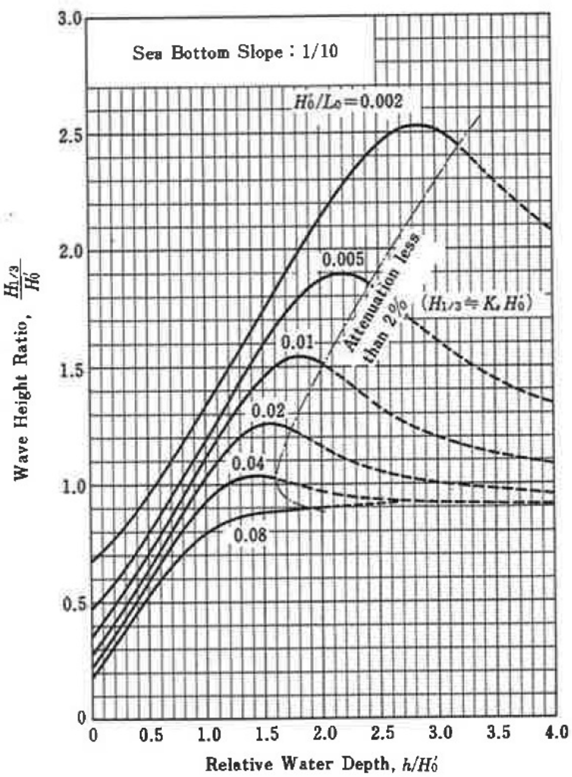
$mn := \frac{1}{11}$ Nearshore Slope, m

$etoe := 8 \cdot ft$

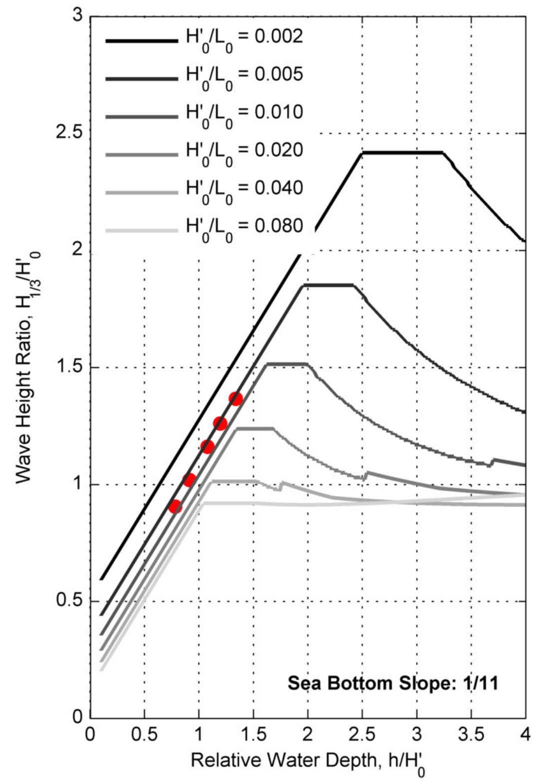
Goda (2000) Computational Model of Wave Transformation in the Surf Zone

Goda (2000) developed a computation model of random wave breaking in the surf zone that provides a tool for estimating wave heights accross the surf zone. The computational model accounts for wave shoaling, random wave breaking, wave setup, and surf beat. The inputs to the model are: offshore wave height (H'_0), wave period ($T_{1/3}$), water depth (h), and nearshore slope (mn).

Goda Diagram (1:10 slope)



Goda Mathematical Model (1:11 slope)



deep water wave length

$$L0 := \frac{g \cdot Tp^2}{2 \cdot \pi} = \begin{pmatrix} 892.2 \\ 1018 \\ 1183.1 \\ 1310.9 \end{pmatrix} \cdot ft$$

water depth (not including wave setup)

$$d := swl - etoe = \begin{pmatrix} 5.3 \\ 6.3 \\ 7.6 \\ 8.6 \end{pmatrix} \cdot ft$$

wave steepness

$$S0 := \frac{H0}{L0} = \begin{pmatrix} 0.0076 \\ 0.0068 \\ 0.006 \\ 0.0055 \end{pmatrix}$$

$$xaxis := \frac{(d)}{H0} = \begin{pmatrix} 0.782 \\ 0.913 \\ 1.078 \\ 1.194 \end{pmatrix} \qquad yaxis := \begin{pmatrix} 0.905 \\ 1.019 \\ 1.161 \\ 1.260 \end{pmatrix}$$

Depth limited wave height (see red dots above in Goda Mathematical Model)

$$Htoe := \overrightarrow{(H0 \cdot yaxis)} = \begin{pmatrix} 6.14 \\ 7.03 \\ 8.19 \\ 9.07 \end{pmatrix} \cdot ft$$

Date: July 14, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: With Project Coastal Processes
Design: Surfzone Wave Transformation - Scenario 2

Solve for wave height at the toe of the structure

References: Goda (2000)

Definitions

H_{toe} - significant wave height ($H_{1/3}$) at toe of structure.

H'_0 - equivalent deepwater wave height (hypothetical wave that has undergone wave refraction and diffraction)

$T_{1/3}$ - average wave period of the highest 1/3 of waves, use peak wave period

Wave Inputs - Derived from Lower New York Harbor Wave Modeling Study (CERC, 1988)

Mean frequency of occurence relationships at Station 8, nominal depth of 17 ft MSL

$$RP := \begin{pmatrix} 100 \\ 164 \\ 309 \\ 500 \end{pmatrix} \cdot yr \qquad swl := \begin{pmatrix} 13.3 \\ 14.3 \\ 15.6 \\ 16.6 \end{pmatrix} \cdot ft \qquad H0 := \begin{pmatrix} 6.78 \\ 6.9 \\ 7.05 \\ 7.2 \end{pmatrix} \cdot ft \qquad Tp := \begin{pmatrix} 13.2 \\ 14.1 \\ 15.2 \\ 16 \end{pmatrix} \cdot s$$

Scenario 2 - Upland Area at +10 ft NGVD

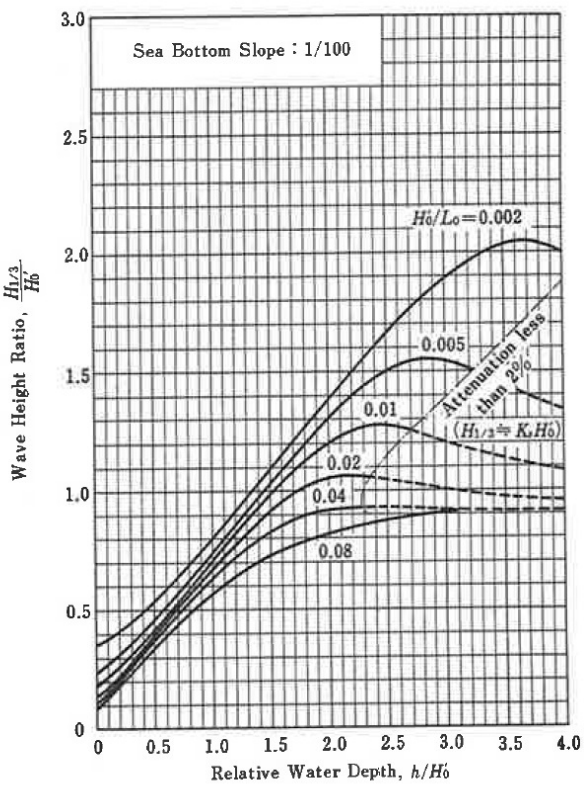
$mn := \frac{1}{100}$ Nearshore Slope, m

$etoe := 10 \cdot ft$

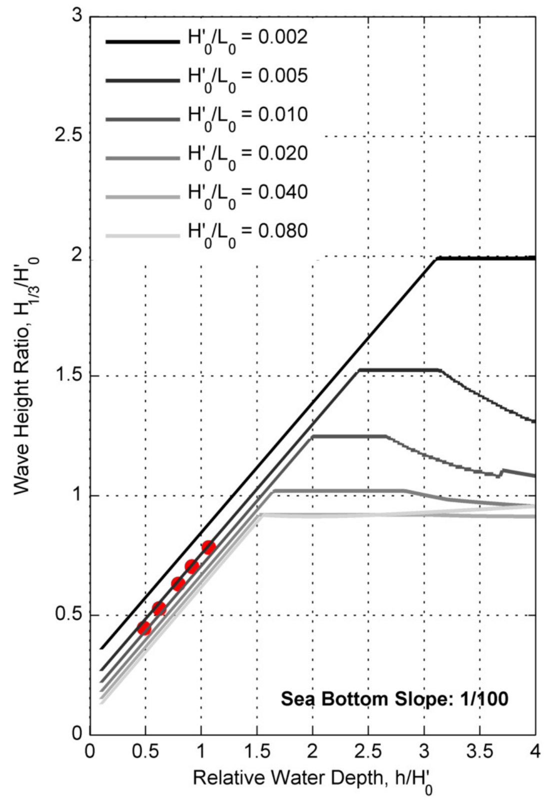
Goda (2000) Computational Model of Wave Transformation in the Surf Zone

Goda (2000) developed a computation model of random wave breaking in the surf zone that provides a tool for estimating wave heights accross the surf zone. The computational model accounts for wave shoaling, random wave breaking, wave setup, and surf beat. The inputs to the model are: offshore wave height (H'_0), wave period ($T_{1/3}$), water depth (h), and nearshore slope (mn).

Goda Diagram (1:100 slope)



Goda Mathematical Model (1:100 slope)



deep water wave length

$$L0 := \frac{g \cdot Tp^2}{2 \cdot \pi} = \begin{pmatrix} 892.2 \\ 1018 \\ 1183.1 \\ 1310.9 \end{pmatrix} \cdot ft$$

water depth (not including wave setup)

$$d := swl - etoe = \begin{pmatrix} 3.3 \\ 4.3 \\ 5.6 \\ 6.6 \end{pmatrix} \cdot ft$$

wave steepness

$$S0 := \frac{H0}{L0} = \begin{pmatrix} 0.0076 \\ 0.0068 \\ 0.006 \\ 0.0055 \end{pmatrix}$$

$$xaxis := \frac{(d)}{H0} = \begin{pmatrix} 0.487 \\ 0.623 \\ 0.794 \\ 0.917 \end{pmatrix} \qquad yaxis := \begin{pmatrix} 0.446 \\ 0.529 \\ 0.631 \\ 0.704 \end{pmatrix}$$

Depth limited wave height (see red dots above in Goda Mathematical Model)

$$Htoe := \overrightarrow{(H0 \cdot yaxis)} = \begin{pmatrix} 3.02 \\ 3.65 \\ 4.45 \\ 5.07 \end{pmatrix} \cdot ft$$

Date: July 14, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: Without Project Coastal Processes
Design: Wave Setup & Wave Runup

Solve for wave setup and wave runup at typical beach profile

References: Shore Protection Manual (1984) and Coastal Engineering Manual (1999)

Definitions

H_s - significant wave height ($H_{1/3}$).

H'_0 - equivalent deepwater wave height (hypothetical wave that has undergone wave refraction and diffraction)

H_b - breaking wave height

d_b - water depth at break point

T_p - peak wave period

$mn := \frac{1}{10}$ Nearshore Slope, m

Wave Inputs - Derived from Lower New York Harbor Wave Modeling Study (CERC, 1988)

$$RP := \begin{pmatrix} 2 \\ 5 \\ 10 \\ 25 \\ 50 \\ 100 \\ 200 \\ 500 \end{pmatrix} \cdot yr \quad Tp := \begin{pmatrix} 5.4 \\ 8.3 \\ 9.7 \\ 11.3 \\ 12.3 \\ 13.2 \\ 14.5 \\ 16.0 \end{pmatrix} \cdot s \quad Hs := \begin{pmatrix} 5.8 \\ 6.5 \\ 7.1 \\ 7.5 \\ 7.9 \\ 8.4 \\ 9.0 \\ 9.7 \end{pmatrix} \cdot ft \quad H0 := \begin{pmatrix} 6.3 \\ 6.3 \\ 6.5 \\ 6.5 \\ 6.6 \\ 6.8 \\ 7.0 \\ 7.2 \end{pmatrix} \cdot ft \quad Hb := \begin{pmatrix} 7.9 \\ 9.8 \\ 10.4 \\ 11.2 \\ 11.9 \\ 12.6 \\ 13.6 \\ 14.4 \end{pmatrix} \cdot ft \quad db := \begin{pmatrix} 7.5 \\ 8.1 \\ 8.4 \\ 8.8 \\ 9.4 \\ 9.8 \\ 10.6 \\ 11.1 \end{pmatrix} \cdot ft$$

Wave Steepness & Breaker Parameter

deep water wavelength

$$L0 := \frac{g}{2\cdot\pi}\cdot Tp^2 = \begin{pmatrix} 149.3 \\ 352.8 \\ 481.8 \\ 653.9 \\ 774.7 \\ 892.2 \\ 1076.6 \\ 1310.9 \end{pmatrix}\cdot ft$$

wave steepness

$$S0 := \frac{H0}{L0} = \begin{pmatrix} 0.0422 \\ 0.0179 \\ 0.0135 \\ 0.0099 \\ 0.0085 \\ 0.0076 \\ 0.0065 \\ 0.0055 \end{pmatrix} \qquad S0s := \frac{Hs}{L0} = \begin{pmatrix} 0.0388 \\ 0.0184 \\ 0.0147 \\ 0.0115 \\ 0.0102 \\ 0.0094 \\ 0.0084 \\ 0.0074 \end{pmatrix}$$

breaker parameter, surf similarity parameter, irrabaren number

Surging Waves $\xi o > 3.3$
Plunging Waves $3.3 > \xi o > 0.5$
Spilling Waves $0.5 > \xi o$

$$\xi o := \frac{mn}{\sqrt{S0}} = \begin{pmatrix} 0.487 \\ 0.748 \\ 0.861 \\ 1.003 \\ 1.083 \\ 1.145 \\ 1.24 \\ 1.349 \end{pmatrix} \qquad \xi os := \frac{mn}{\sqrt{S0s}} = \begin{pmatrix} 0.507 \\ 0.737 \\ 0.824 \\ 0.934 \\ 0.99 \\ 1.031 \\ 1.094 \\ 1.163 \end{pmatrix}$$

wave conditions are primarily plunging breakers, not surprising considering relatively steep slope, and large wave periods

Wave Setup

Wave setup is the superelevation of the mean water level inside the surf zone caused by wave breaking. Calculations base on small amplitude wave theory (CEM, 1999)

breaker depth index, δb

$$\delta b := \frac{Hb}{db} = \begin{pmatrix} 1.05 \\ 1.21 \\ 1.24 \\ 1.27 \\ 1.27 \\ 1.29 \\ 1.28 \\ 1.3 \end{pmatrix}$$

wave setdown at break point, ηb

$$\eta b := \frac{-Hb^2}{16 \cdot db} = \begin{pmatrix} -0.52 \\ -0.74 \\ -0.8 \\ -0.89 \\ -0.94 \\ -1.01 \\ -1.09 \\ -1.17 \end{pmatrix} \cdot ft$$

wave setup at shoreline, ηs

$$\eta s := \left[\overrightarrow{\eta b + \left(\overrightarrow{\frac{1}{1 + \frac{8}{3 \cdot \delta b^2}} \cdot db}\right)}\right] = \begin{pmatrix} 1.68 \\ 2.13 \\ 2.26 \\ 2.43 \\ 2.59 \\ 2.74 \\ 2.96 \\ 3.13 \end{pmatrix} \cdot ft$$

Wave Runup Height

Mean wave runup and 2% wave runup for irregular waves on beaches computed based on Coastal Engineering Manual (1999). Note that the wave runup height includes wave setup height.

Irregular Mean Wave Runup on Beaches (CEM)

$$R\mu CEM := 0.88\,\overrightarrow{\left(H0 \cdot \xi o^{0.69}\right)} = \begin{pmatrix} 3.37 \\ 4.54 \\ 5.16 \\ 5.73 \\ 6.14 \\ 6.57 \\ 7.15 \\ 7.79 \end{pmatrix} \cdot ft$$

Irregular 2% Wave Runup on Beaches (CEM)

$$R2\%CEM := 1.86\,\overrightarrow{\left(H0 \cdot \xi o^{0.71}\right)} = \begin{pmatrix} 7.03 \\ 9.54 \\ 10.87 \\ 12.12 \\ 12.99 \\ 13.93 \\ 15.17 \\ 16.57 \end{pmatrix} \cdot ft$$

Date: July 14, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: Without Project Coastal Processes
Design: Wave Overtopping on Dune at Oakwood Beach

Solve for wave overtopping at Oakwood Beach

References: Kobayashi, N., Tega, Y., and Hancock, M. 1996. “Wave reflection and Overwash of Dunes”

Definitions

H_s - significant wave height ($H_{1/3}$).

T_p - peak wave period

$mn := \frac{1}{15}$ effective slope

$zcrest := 12 \cdot ft$ dune crest elevation (ft, NGVD)

$slr := 0.7 \cdot ft$ sea level rise (50 years)

Wave Inputs - Derived from Lower New York Harbor Wave Modeling Study (CERC, 1988)

$$RP := \begin{pmatrix} 2 \\ 5 \\ 10 \\ 25 \\ 50 \\ 100 \\ 200 \\ 500 \end{pmatrix} \cdot yr \qquad Tp := \begin{pmatrix} 5.4 \\ 8.3 \\ 9.7 \\ 11.3 \\ 12.3 \\ 13.2 \\ 14.5 \\ 16.0 \end{pmatrix} \cdot s \qquad Hs := \begin{pmatrix} 5.8 \\ 6.5 \\ 7.1 \\ 7.5 \\ 7.9 \\ 8.4 \\ 9.0 \\ 9.7 \end{pmatrix} \cdot ft \qquad swl := \begin{pmatrix} 5.3 \\ 7.2 \\ 8.5 \\ 10.0 \\ 11.3 \\ 12.6 \\ 14.0 \\ 15.9 \end{pmatrix} \cdot ft$$

Wave Steepness & Breaker Parameter

deep water wavelength

$$L0 := \frac{g}{2\cdot\pi}\cdot Tp^2 = \begin{pmatrix} 149.3 \\ 352.8 \\ 481.8 \\ 653.9 \\ 774.7 \\ 892.2 \\ 1076.6 \\ 1310.9 \end{pmatrix}\cdot ft$$

wave steepness

$$S0s := \frac{Hs}{L0} = \begin{pmatrix} 0.0388 \\ 0.0184 \\ 0.0147 \\ 0.0115 \\ 0.0102 \\ 0.0094 \\ 0.0084 \\ 0.0074 \end{pmatrix}$$

breaker parameter, surf similarity parameter, irrabaren number

$$\xi os := \frac{mn}{\sqrt{S0s}} = \begin{pmatrix} 0.338 \\ 0.491 \\ 0.549 \\ 0.622 \\ 0.66 \\ 0.687 \\ 0.729 \\ 0.775 \end{pmatrix}$$

Surging Waves $\xi o > 3.3$
Plunging Waves $3.3 > \xi o > 0.5$
Spilling Waves $0.5 > \xi o$

wave conditions are primarily plunging breakers, not surprising considering relatively steep slope, and large wave periods

Compute Dune Overtopping Rate - M&N 2001 (based on VDM 1998)

$$\gamma h := 1 - .03\left(4 - \frac{30}{9}\right)^2 = 0.987$$ water depth coefficient

$$Rc := zcrest - swl - slr = \begin{pmatrix} 6 \\ 4.1 \\ 2.8 \\ 1.3 \\ 0 \\ -1.3 \\ -2.7 \\ -4.6 \end{pmatrix} \cdot ft$$

$$q := \overrightarrow{\left[\overrightarrow{\left[\sqrt{g \cdot Hs^3}\left(\frac{0.013}{\sqrt{mn}} \cdot \xi os\right)\right]} \cdot \exp\left[-2.33\,\overrightarrow{\left(\frac{Rc}{Hs} \cdot \frac{1}{\xi os \cdot \gamma h}\right)}\right]\right]} = \begin{pmatrix} 0.1 \\ 10.4 \\ 50.6 \\ 175.8 \\ 388.9 \\ 755.4 \\ 1380.1 \\ 2635 \end{pmatrix} \cdot \frac{liter}{s \cdot m}$$

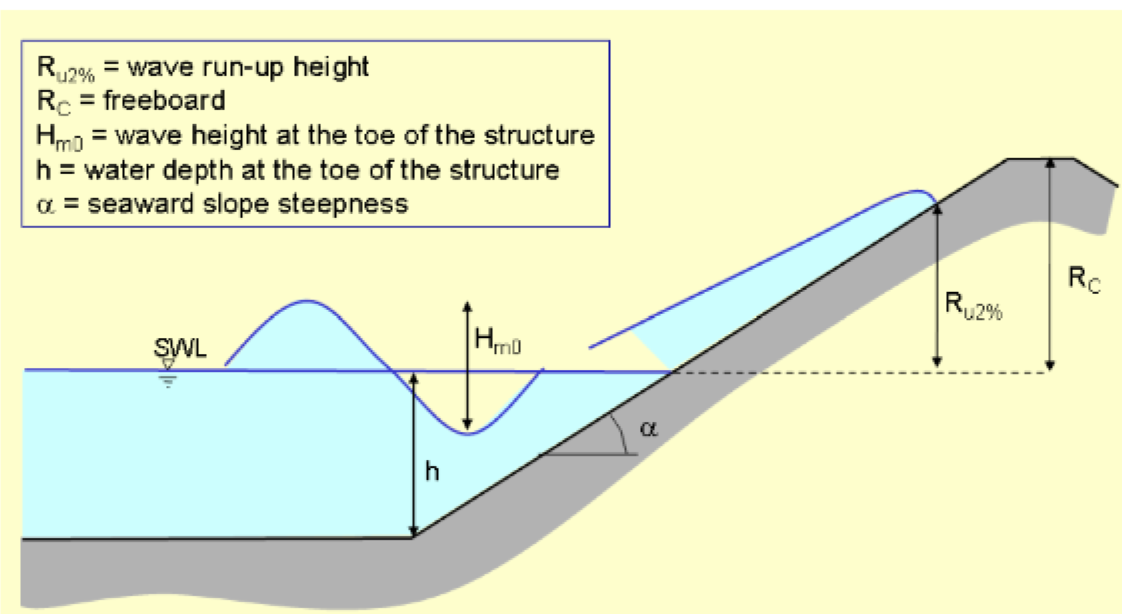
Date: July 14, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: With Project Coastal Processes
Design: Wave Runup & Overtopping - Levee - Scenario 3 - 309yr

Solving for the average wave overtopping rate of levee during 309-year conditions

Reference: Pullen et. al, 2007. "EurOtop - Wave Overtopping of Sea Defences and Related Structures: Assessment Manual"

Definitions

$Htoe := 2 \cdot ft$	significant wave height
$Tp := 2 \cdot sec$	peak wave period
$swl := 15.6 \cdot ft$	still water level
$\alpha := atan\left(\frac{1}{2.5}\right)$	levee slope
$\alpha n := atan\left(\frac{1}{50}\right)$	nearshore slope fronting structure
$Zcrest := 18.1ft$	crest elevation above datum
$Ztoe := 10 \cdot ft$	toe elevation above datum
$\gamma b := 1$	berm coefficient
$\gamma f := 1$	roughness of front slope
$\gamma \beta := 1$	wave angle coefficient
$\gamma v := 1$	coefficient for a vertical wall



Calculations

$Rc := Zcrest - swl = 2.5 \cdot ft$ freeboard at dune crest

$htoe := swl - Ztoe = 5.6 \cdot ft$ water depth at toe of structure

$mn := \tan(\alpha n) = 0.02$ nearshore slope

$Tm1 := \frac{Tp}{1.1} = 1.82\,s$ mean spectral wave period

$Lo := g \cdot \frac{Tp^2}{2 \cdot \pi} = 20.5 \cdot ft$ deep water wave length

$Lp := \sqrt{g \cdot htoe}\,Tp = 26.8 \cdot ft$ wave length at toe of structure

$Lm1 := g \cdot \frac{Tm1^2}{2\pi} = 16.9 \cdot ft$ deep water wave length based on Tm1

$Som1 := \frac{Htoe}{Lm1} = 0.118$ spectral wave steepness

$\xi m1 := \frac{\tan(\alpha)}{\sqrt{Som1}} = 1.164$ breaker parameter, surf similarity, irabarren number

Compute Probabilistic Wave Runup and Overtopping - EurOtop 2007

Wave Runup

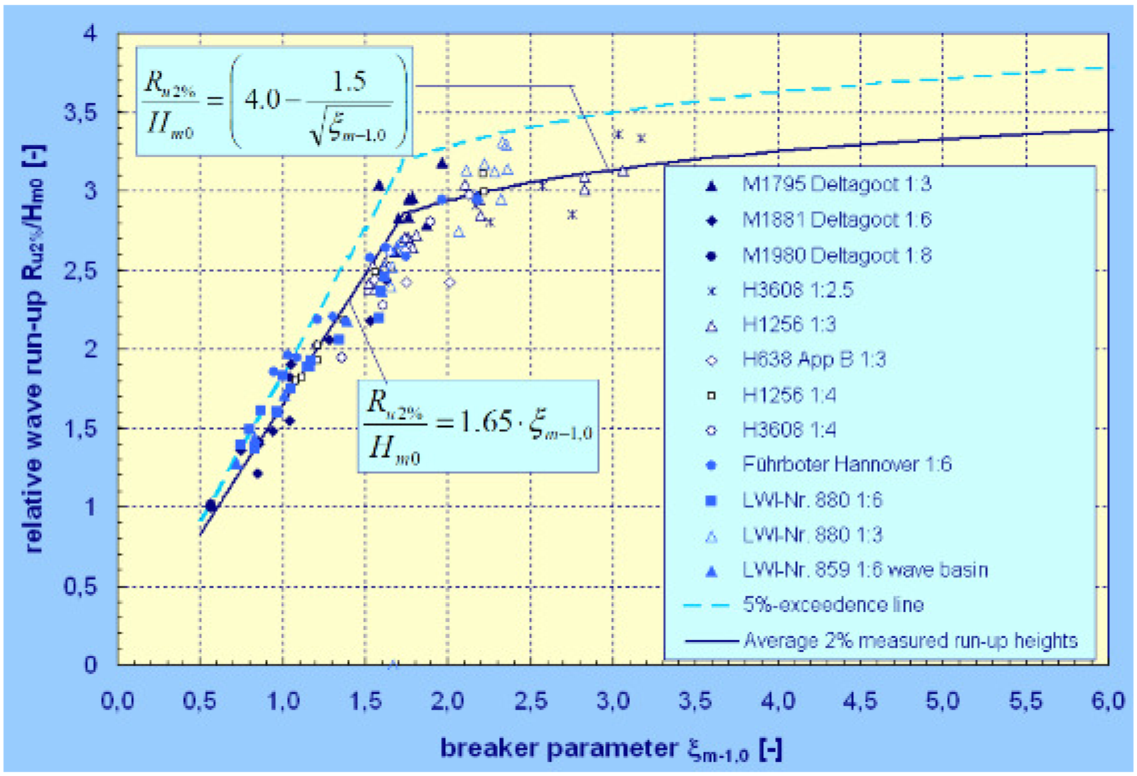
$R2a := Htoe \cdot 1.65 \cdot \gamma b \cdot \gamma f \cdot \gamma \beta \cdot \xi m1 = 3.84 \cdot ft$

$R2b := Htoe \cdot \gamma b \cdot \gamma f \cdot \gamma \beta \cdot \left(4 - \frac{1.5}{\sqrt{\xi m1}}\right) = 5.219 \cdot ft$

$R2p := if(R2a \le R2b, R2a, R2b) = 3.84 \cdot ft$

Relative Wave Runup

$\xi m1 = 1.164$ $\frac{R2p}{Htoe} = 1.92$



Wave Overtopping

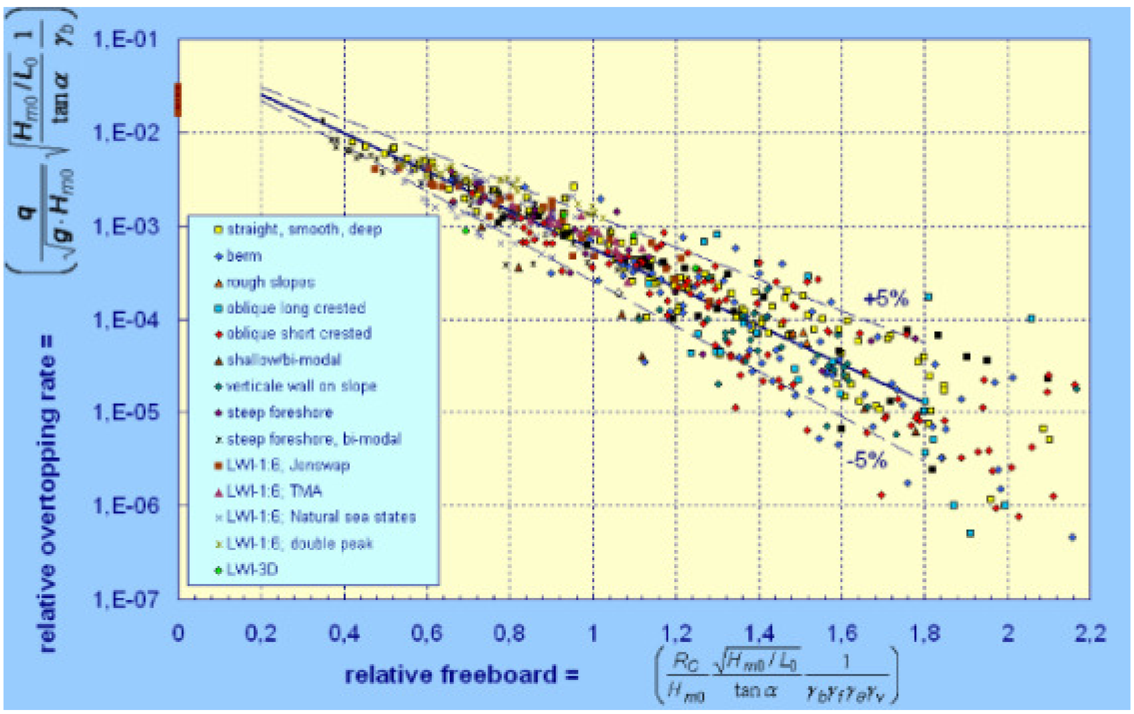
$$Qa := \sqrt{g \cdot Htoe^3} \cdot \frac{.067}{\sqrt{\tan(\alpha)}} \cdot \xi m1 \cdot \gamma b \cdot \exp\left(-4.75 \cdot \frac{Rc}{Htoe} \cdot \frac{1}{\xi m1 \cdot \gamma b \cdot \gamma f \cdot \gamma\beta \cdot \gamma v}\right) = 1.118 \cdot \frac{liter}{s \cdot m}$$

$$Qb := \sqrt{g \cdot Htoe^3} \cdot .2 \cdot \exp\left(-2.6 \cdot \frac{Rc}{Htoe \cdot \gamma f \cdot \gamma\beta}\right) = 11.558 \cdot \frac{liter}{s \cdot m}$$

$$Qp := \text{if}(Qa \le Qb, Qa, Qb) = 1.118 \cdot \frac{liter}{s \cdot m}$$

Relative Wave Overtopping - Breaking Waves

$$\frac{Qp}{(g \cdot Htoe)^{0.5}} \cdot \left(\frac{Som1}{\tan(\alpha)}\right)^{0.5} \cdot \frac{1}{\gamma b} = 2.485 \times 10^{-4} m \qquad \frac{Rc}{Htoe} \cdot \frac{Som1^{0.5}}{\tan(\alpha)} \cdot \frac{1}{\gamma b \cdot \gamma f \cdot \gamma\beta \cdot \gamma v} = 1.074$$



Compute Deterministic Wave Runup and Overtopping - EurOtop 2007

Wave Runup

$$R2a := Htoe \cdot 1.75 \cdot \gamma b \cdot \gamma f \cdot \gamma\beta \cdot \xi m1 = 4.073 \cdot ft$$

$$R2b := Htoe \cdot \gamma b \cdot \gamma f \cdot \gamma\beta \cdot \left(4.3 - \frac{1.6}{\sqrt{\xi m1}}\right) = 5.634 \cdot ft$$

$$R2d := if(R2a \le R2b, R2a, R2b) = 4.073 \cdot ft$$

Wave Overtopping

$$Qa := \sqrt{g \cdot Htoe^3} \cdot \frac{.067}{\sqrt{\tan(\alpha)}} \cdot \xi m1 \cdot \gamma b \cdot exp\left(-4.3 \cdot \frac{Rc}{Htoe} \cdot \frac{1}{\xi m1 \cdot \gamma b \cdot \gamma f \cdot \gamma\beta \cdot \gamma v}\right) = 1.813 \cdot \frac{liter}{s \cdot m}$$

$$Qb := \sqrt{g \cdot Htoe^3} \cdot .2 \cdot exp\left(-2.3 \cdot \frac{Rc}{Htoe \cdot \gamma f \cdot \gamma\beta}\right) = 16.817 \cdot \frac{liter}{s \cdot m}$$

$$Qd := if(Qa \le Qb, Qa, Qb) = 1.813 \cdot \frac{liter}{s \cdot m}$$

Results

Deterministic2% Exceedance Wave Runup - EurOtop 2007

R2p $= 3.84 \cdot \text{ft}$ Qp $= 1.118 \cdot \frac{\text{liter}}{\text{s} \cdot \text{m}}$ Qp $= 0.012 \cdot \frac{\text{ft}^3}{\text{ft} \cdot \text{s}}$

Deterministic Runup and Mean Wave Overtopping - EurOtop 2007

R2d $= 4.073 \cdot \text{ft}$ Qd $= 1.813 \cdot \frac{\text{liter}}{\text{s} \cdot \text{m}}$ Qd $= 0.02 \cdot \frac{\text{ft}^3}{\text{ft} \cdot \text{s}}$

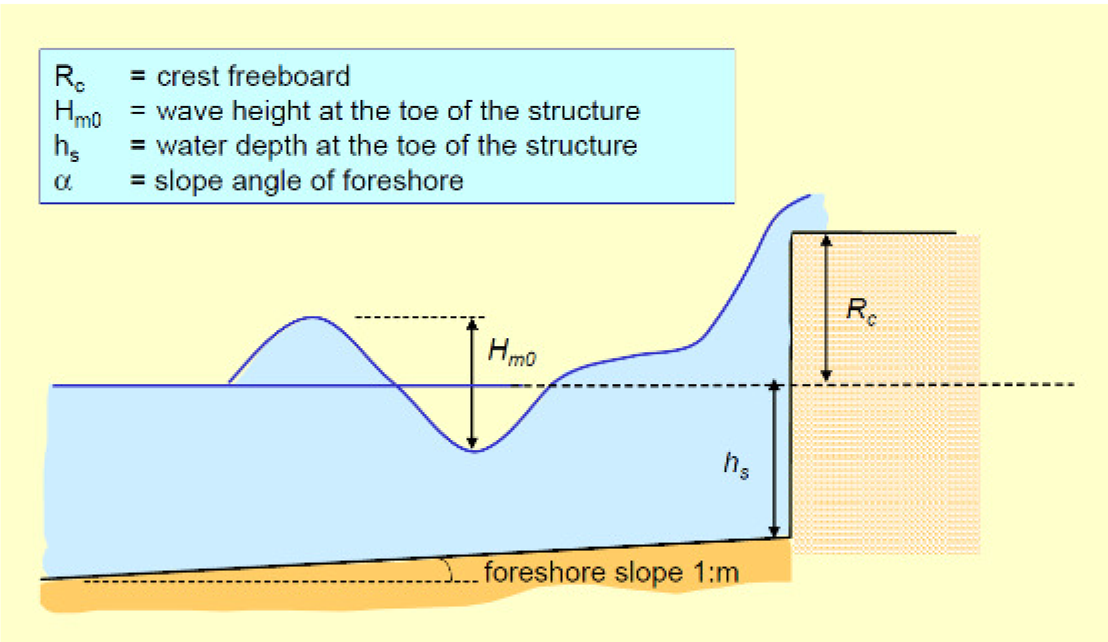
Date: July 14, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: With Project Coastal Processes
Design: Wave Runup & Overtopping - Vertical Wall - Scenario 2 - 309 yr

Solving for the average wave overtopping rate at Vertical Wall during 309-year conditions

Reference: Pullen et. al, 2007. "EurOtop - Wave Overtopping of Sea Defences and Related Structures: Assessment Manual"

Definitions

$Htoe := 4.4 \cdot ft$	significant wave height
$Tp := 15.2 \cdot sec$	peak wave period
$swl := 15.6 \cdot ft$	water level
$\alpha n := atan\left(\frac{1}{100}\right)$	nearshore slope fronting structure
$Zcrest := 20.5ft$	crest elevation above datum
$Ztoe := 10 \cdot ft$	toe elevation above datum



Calculations

$Rc := Zcrest - swl = 4.9 \cdot ft$ freeboard at dune crest

$htoe := swl - Ztoe = 5.6 \cdot ft$ water depth at toe of structure

$mn := \tan(\alpha n) = 0.01$ nearshore slope

$Tm1 := \frac{Tp}{1.1} = 13.82\,s$ mean spectral wave period

$Lo := g \cdot \frac{Tp^2}{2 \cdot \pi} = 1183.1 \cdot ft$ deep water wave length

$Lp := \sqrt{g \cdot htoe}\,Tp = 204 \cdot ft$ wave length at toe of structure

$Lm1 := g \cdot \frac{Tm1^2}{2\pi} = 977.7 \cdot ft$ deep water wave length based on Tm1

$Som1 := \frac{Htoe}{Lm1} = 0.0045$ spectral wave steepness

Compute Probabilistic Wave Overtopping - EurOtop 2007

Impulsiveness parameter

$$hst := 1.35 \cdot \frac{htoe^2 \cdot 2 \cdot \pi}{Htoe \cdot g \cdot Tm1^2} = 0.0098$$

hst > 0.3 non-impulsive conditions
hst < 0.2 impulsive conditions

Non Impulsive Conditions

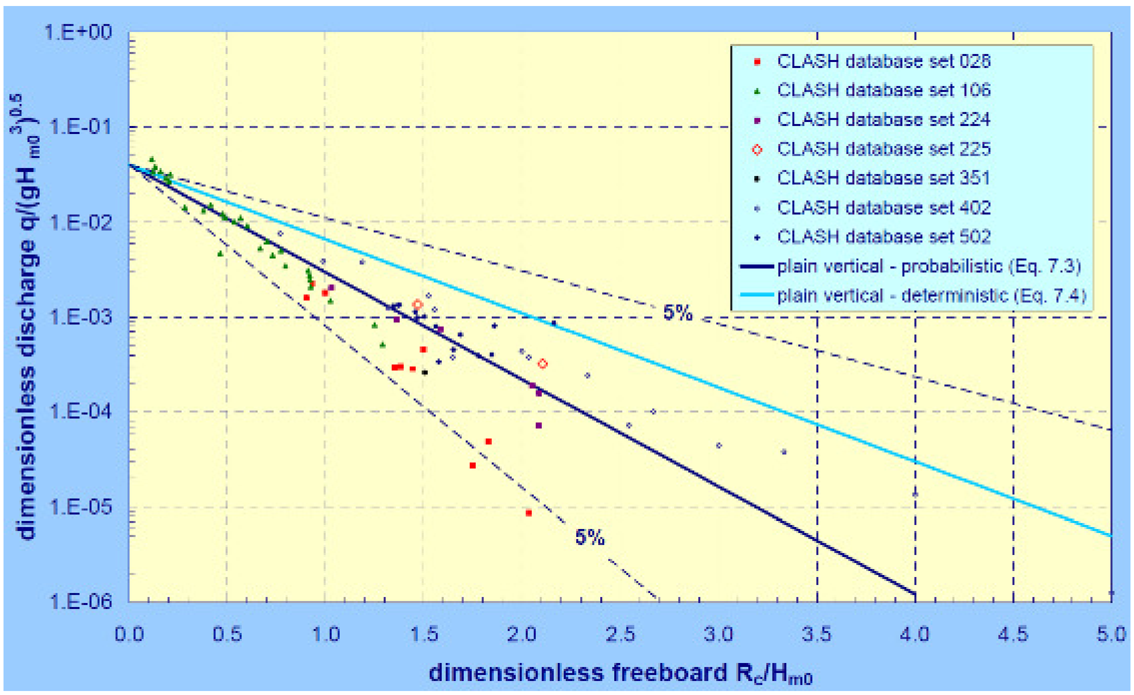
$$Qeuna := \sqrt{g \cdot Htoe^3} \cdot .04 \cdot \exp\left(-2.6 \cdot \frac{Rc}{Htoe}\right) = 10.753 \cdot \frac{liter}{s \cdot m}$$

$$Qeunb := \sqrt{g \cdot Htoe^3} \cdot (.062 + .0062) = 331.701 \cdot \frac{liter}{s \cdot m}$$

$$Qeun := if\left(\frac{Rc}{Htoe} \geq .1, Qeuna, Qeunb\right) = 10.753 \cdot \frac{liter}{s \cdot m}$$

$$\frac{Qeun}{\left(g \cdot Htoe^3\right)^{0.5}} = 2.211 \times 10^{-3}$$

$$\frac{Rc}{Htoe} = 1.11$$



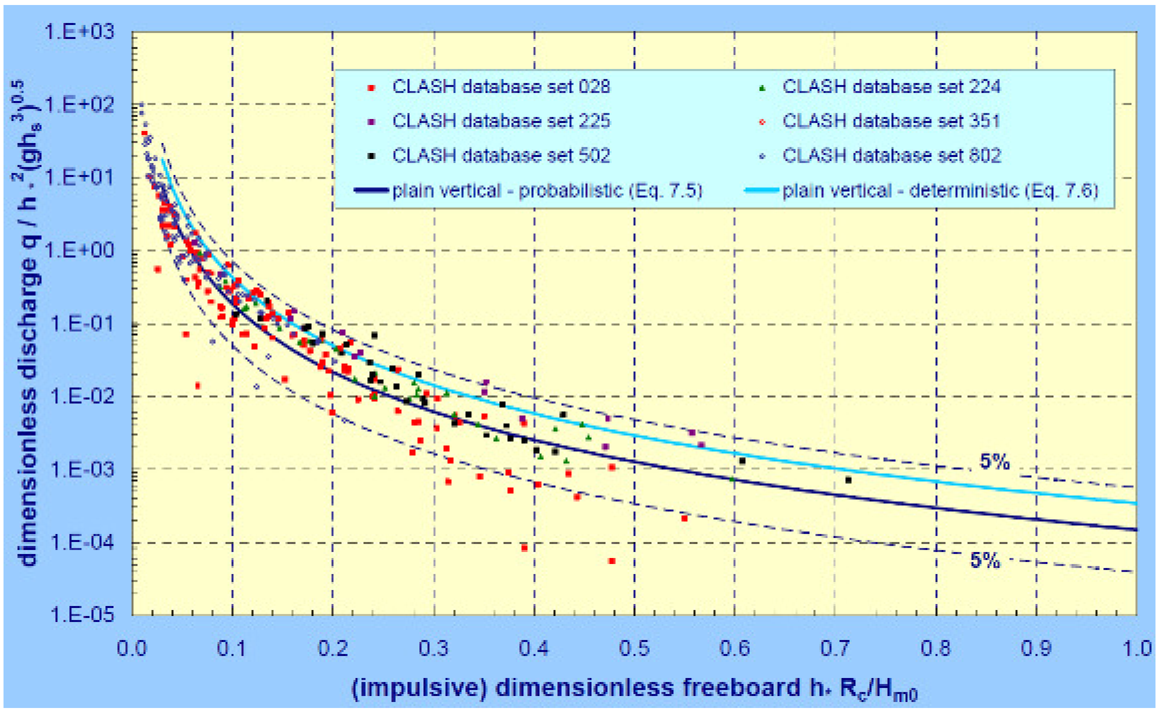
Impulsive Conditions

$$Qeuia := hst^2 \cdot \sqrt{g \cdot htoe^3} \cdot .00015 \cdot \left(hst \cdot \frac{Rc}{Htoe}\right)^{-3.1} = 121.037 \cdot \frac{liter}{s \cdot m}$$

$$Qeuib := hst^2 \cdot \sqrt{g \cdot htoe^3} \cdot .00027 \cdot \left(hst \cdot \frac{Rc}{Htoe}\right)^{-2.7} = 35.818 \cdot \frac{liter}{s \cdot m}$$

$$Qeui := if\left(hst \cdot \frac{Rc}{Htoe} \geq .02, Qeuia, Qeuib\right) = 35.818 \cdot \frac{liter}{s \cdot m}$$

$$\frac{Qeui}{hst^2 \left(g \cdot Htoe^3\right)^{0.5}} = 7.6 \times 10^1 \qquad \frac{hst \cdot Rc}{Htoe} = 0.011$$



Probalisitic Overtopping Value

$$Qeup := if(hst \leq .2, Qeui, Qeun) = 35.818 \cdot \frac{liter}{s \cdot m}$$

Compute Deterministic Wave Overtopping - EurOtop 2007

Non Impulsive Conditions

$$Qeuna := \sqrt{g \cdot Htoe^3} \cdot .04 \cdot \exp\left(-1.8 \cdot \frac{Rc}{Htoe}\right) = 26.21 \cdot \frac{liter}{s \cdot m}$$

$$Qeunb := \sqrt{g \cdot Htoe^3} \cdot (.062 + .0062) \cdot 1.5 = 497.551 \cdot \frac{liter}{s \cdot m}$$

Add one standard deviation for deterministic design (Assume 50% for now)

$$Qeun := \text{if}\left(\frac{Rc}{Htoe} \geq .1, Qeuna, Qeunb\right) = 26.21 \cdot \frac{liter}{s \cdot m}$$

Impulsive Conditions

$$Qeuia := hst^2 \cdot \sqrt{g \cdot htoe^3} \cdot .00028 \cdot \left(hst \cdot \frac{Rc}{Htoe}\right)^{-3.1} = 225.936 \cdot \frac{liter}{s \cdot m}$$

$$Qeuib := hst^2 \cdot \sqrt{g \cdot htoe^3} \cdot .00038 \cdot \left(hst \cdot \frac{Rc}{Htoe}\right)^{-2.7} = 50.41 \cdot \frac{liter}{s \cdot m}$$

$$Qeui := \text{if}\left(hst \cdot \frac{Rc}{Htoe} \geq .02, Qeuia, Qeuib\right) = 50.41 \cdot \frac{liter}{s \cdot m}$$

Deterministic Overtopping Value

$$Qeud := \text{if}(hst \leq .2, Qeui, Qeun) = 50.41 \cdot \frac{liter}{s \cdot m}$$

Compute Wave Overtopping - Ward and Ahren, 1992

$C1 := -7.385$ Group #1 Parameter

$C2 := -2.178$ Group #1 Parameter

$Q0p := .338$ Group #1 Parameter

$Fp := \dfrac{Rc}{\left(Htoe^{2} \cdot Lp\right)^{\frac{1}{3}}} = 0.31$ Relative Freeboard

$X := \dfrac{Rc}{htoe} = 0.875$ Group #1 Parameter

$Qp := Q0p \cdot \exp(C1 \cdot Fp + C2 \cdot X) = 0.0051$ Dimensionless Overtopping

Average Wave Overtopping

$Qaw := Qp \cdot \sqrt{g \cdot Htoe^{3}} = 24.78 \cdot \dfrac{liter}{s \cdot m}$

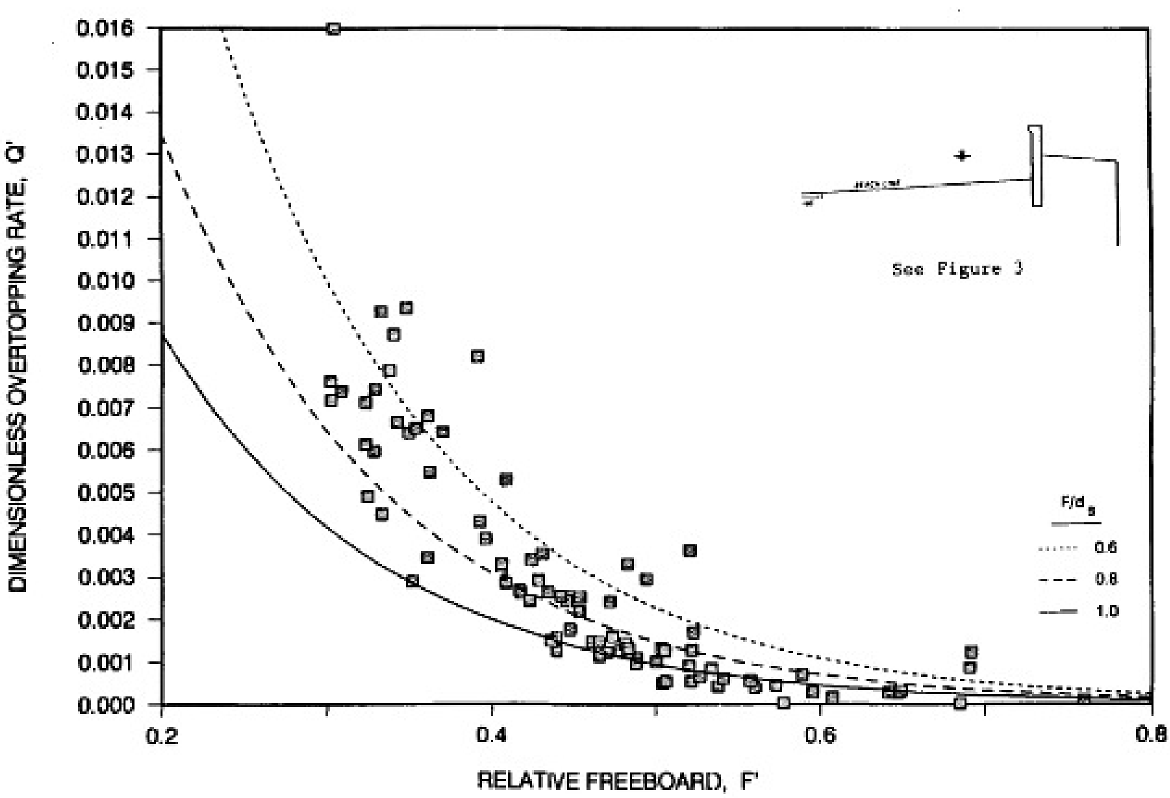


Figure 20. Group 1 measured overtopping and regression curves for improved overtopping model

Results

Probablistic Mean Wave Overtopping - EurOtop 2007

$\text{Qeup} = 35.818 \cdot \frac{\text{liter}}{\text{s} \cdot \text{m}}$ $\text{Qeup} = 0.386 \cdot \frac{\text{ft}^3}{\text{s} \cdot \text{ft}}$

Deterministic Mean Wave Overtopping - EurOtop 2007

$\text{Qeud} = 50.41 \cdot \frac{\text{liter}}{\text{s} \cdot \text{m}}$ $\text{Qeud} = 0.543 \cdot \frac{\text{ft}^3}{\text{s} \cdot \text{ft}}$

Mean Wave Overtopping - Ward & Ahren, 1992

$\text{Qaw} = 24.777 \cdot \frac{\text{liter}}{\text{s} \cdot \text{m}}$ $\text{Qaw} = 0.267 \cdot \frac{\text{ft}^3}{\text{s} \cdot \text{ft}}$

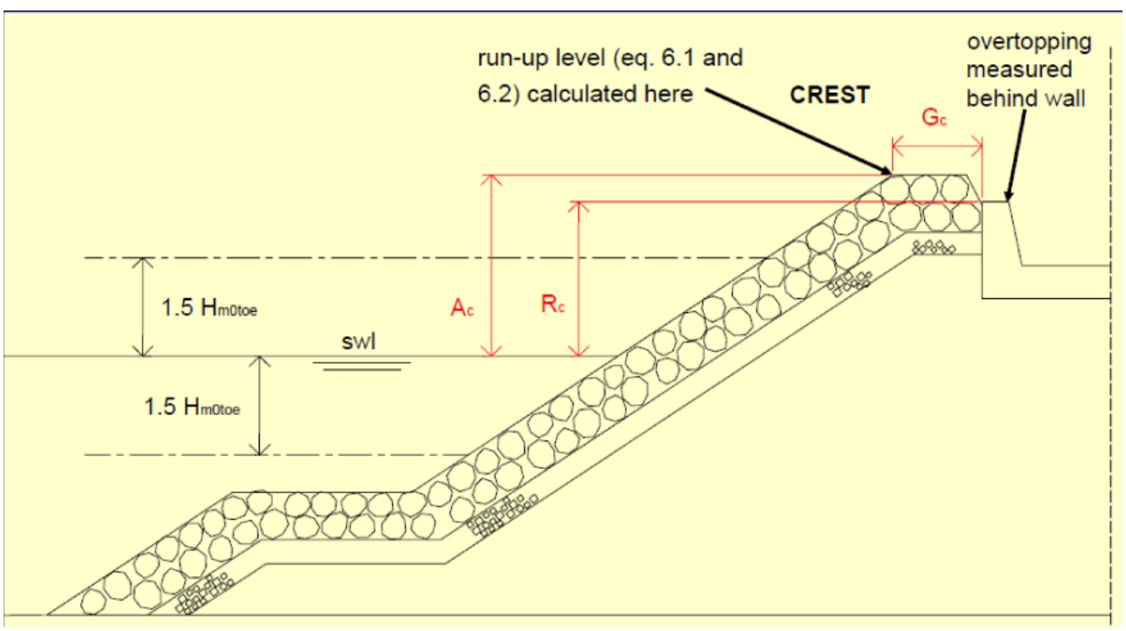
Date: July 14, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: With Project Coastal Processes
Design: Wave Runup & Overtopping - Scenario 1a - 309yr

Solving for the average wave overtopping rate of rubble mound embankment during 309-year conditions

Reference: Pullen et. al, 2007. "EurOtop - Wave Overtopping of Sea Defences and Related Structures: Assessment Manual"

Definitions

$Htoe := 5.8 \cdot ft$	significant wave height
$Tp := 15.2 \cdot sec$	peak wave period
$swl := 15.6 \cdot ft$	still water level
$\alpha := atan\left(\frac{1}{1.5}\right)$	structure slope
$\alpha n := atan\left(\frac{1}{15}\right)$	nearshore slope fronting structure
$Zcrest := 20.5ft$	crest elevation above datum
$Ztoe := 10 \cdot ft$	toe elevation above datum
$\gamma b := 1$	berm coefficient
$\gamma f := 0.55$	roughness of front slope
$\gamma\beta := 1$	wave angle coefficient
$\gamma v := 1$	coefficient for a vertical wall



Calculations

$Rc := Zcrest - swl = 4.9 \cdot ft$	freeboard at dune crest
$htoe := swl - Ztoe = 5.6 \cdot ft$	water depth at toe of structure
$mn := \tan(\alpha n) = 0.067$	nearshore slope
$Tm1 := \frac{Tp}{1.1} = 13.82\,s$	mean spectral wave period
$Lo := g \cdot \frac{Tp^2}{2 \cdot \pi} = 1183.1 \cdot ft$	deep water wave length
$Lp := \sqrt{g \cdot htoe}\,Tp = 204 \cdot ft$	wave length at toe of structure
$Lm1 := g \cdot \frac{Tm1^2}{2\pi} = 977.7 \cdot ft$	deep water wave length based on Tm1
$Som1 := \frac{Htoe}{Lm1} = 0.0059$	spectral wave steepness
$\xi m1 := \frac{\tan(\alpha)}{\sqrt{Som1}} = 8.656$	breaker parameter, surf similarity, irabarren number

Compute Probabilistic Wave Runup and Overtopping - EurOtop 2007

$R2a := Htoe \cdot 1.65 \cdot \gamma b \cdot \gamma f \cdot \gamma\beta \cdot \xi m1 = 45.56 \cdot ft$

$\gamma fsa := \gamma f + (\xi m1 - 1.8) \cdot \dfrac{(1 - \gamma f)}{8.2} = 0.926$

$\gamma fsb := 1$

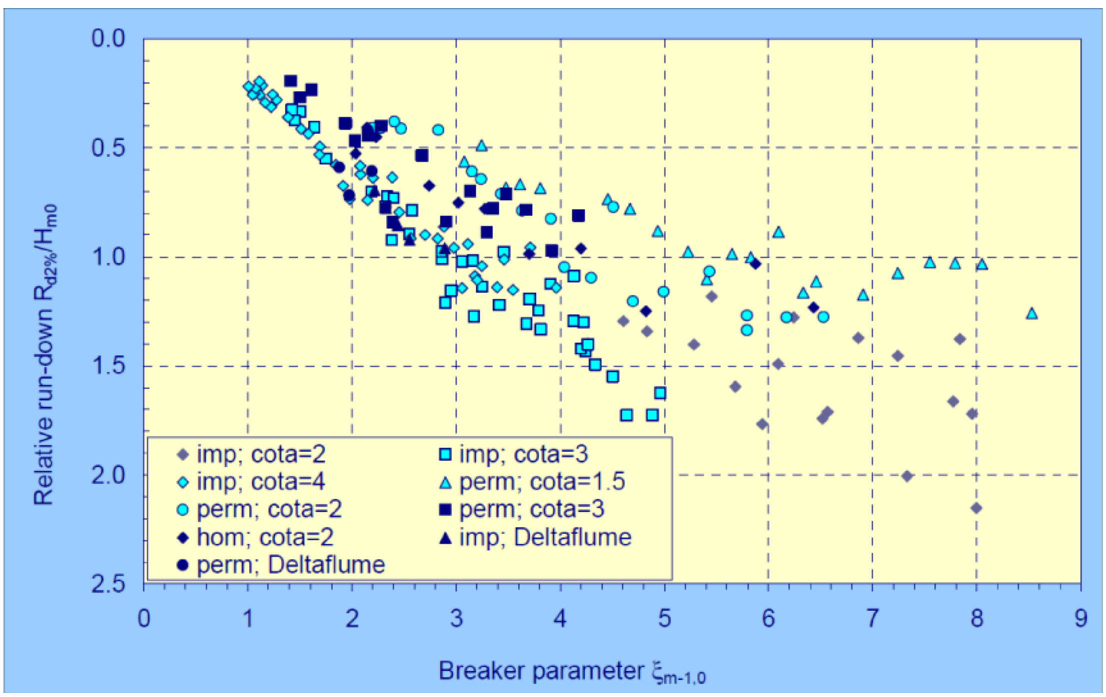
$\gamma fs := if(\xi m1 \le 10, \gamma fsa, \gamma fsb) = 0.926$ adjusted roughness factor for surging

$R2b := Htoe \cdot \gamma b \cdot \gamma fs \cdot \gamma\beta \cdot \left(4 - \dfrac{1.5}{\sqrt{\xi m1}}\right) = 18.75 \cdot ft$

$R2p := if(R2a \le R2b, R2a, R2b) = 18.75 \cdot ft$

Relative Wave Runup

$\xi m1 = 8.656$ $\dfrac{R2p}{Htoe} = 3.233$ *breaker parameter is outside normal conditions, results should be used with caution*

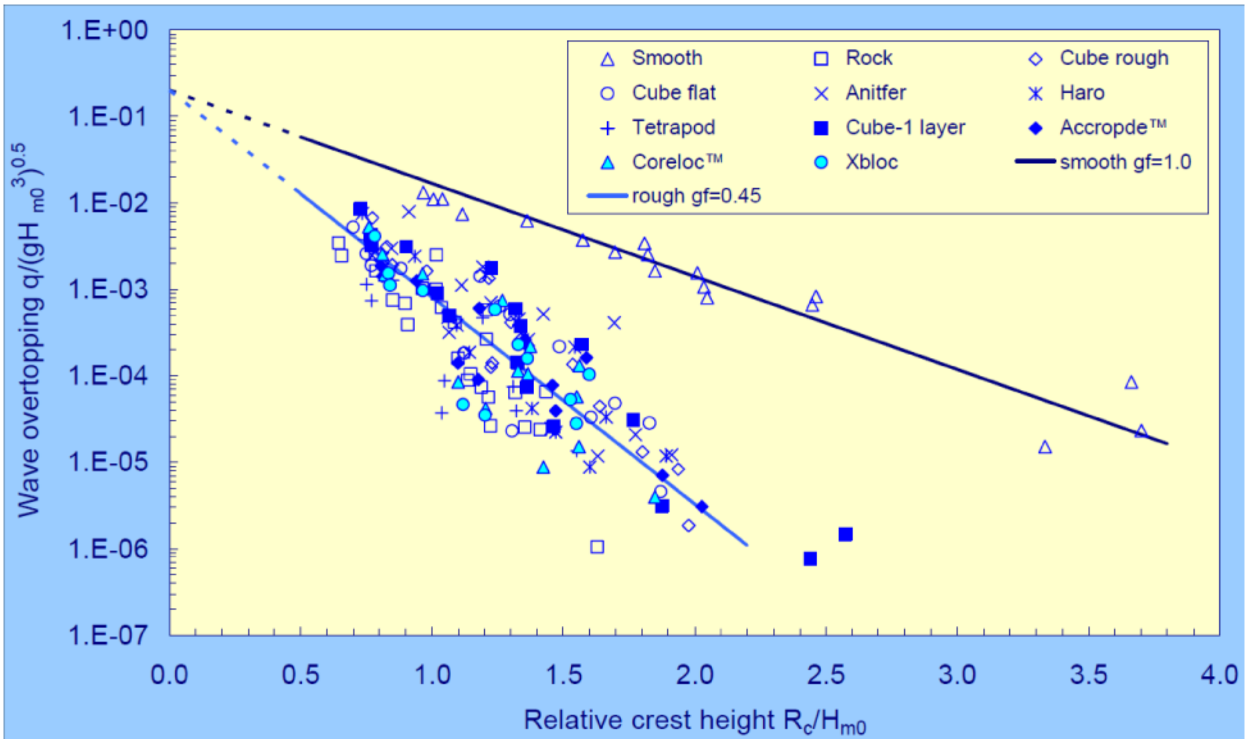


Probalisitic Overtopping Value

$$Qp := \sqrt{g \cdot Htoe^3} \cdot .2 \cdot \exp\left(-2.6 \cdot \frac{Rc}{Htoe \cdot \gamma f \cdot \gamma\beta}\right) = 27.133 \cdot \frac{liter}{s \cdot m}$$

Relative Wave Overtopping

$$\frac{Qp}{\left(g \cdot Htoe^3\right)^{0.5}} = 3.686 \times 10^{-3} \qquad \frac{Rc}{Htoe} = 0.845$$



Compute Deterministic Wave Runup and Overtopping - EurOtop 2007

$$R2a := Htoe \cdot 1.75 \cdot \gamma b \cdot \gamma f \cdot \gamma\beta \cdot \xi m1 = 48.32 \cdot ft$$

$$R2b := Htoe \cdot \gamma b \cdot \gamma fs \cdot \gamma\beta \cdot \left(4.3 - \frac{1.6}{\sqrt{\xi m1}}\right) = 20.18 \cdot ft$$

$$R2d := \mathrm{if}(R2a \le R2b, R2a, R2b) = 20.18 \cdot ft$$

$$Qa := \sqrt{g \cdot Htoe^3} \cdot \frac{.067}{\sqrt{\tan(\alpha)}} \cdot \xi m1 \cdot \gamma b \cdot \exp\left(-4.3 \cdot \frac{Rc}{Htoe} \cdot \frac{1}{\xi m1 \cdot \gamma b \cdot \gamma f \cdot \gamma\beta \cdot \gamma v}\right) = 2.438 \times 10^3 \cdot \frac{liter}{s \cdot m}$$

$$Qb := \sqrt{g \cdot Htoe^3} \cdot .2 \cdot \exp\left(-2.3 \cdot \frac{Rc}{Htoe \cdot \gamma f \cdot \gamma\beta}\right) = 43.016 \cdot \frac{liter}{s \cdot m}$$

Deterministic Overtopping Value

$$Qd := \mathrm{if}(Qa \le Qb, Qa, Qb) = 43.016 \cdot \frac{liter}{s \cdot m}$$

Compute Wave Overtopping - Ward and Ahren, 1992

$C1 := -10.732$ Group #2 Parameter

$C2 := -6.629$ Group #2 Parameter

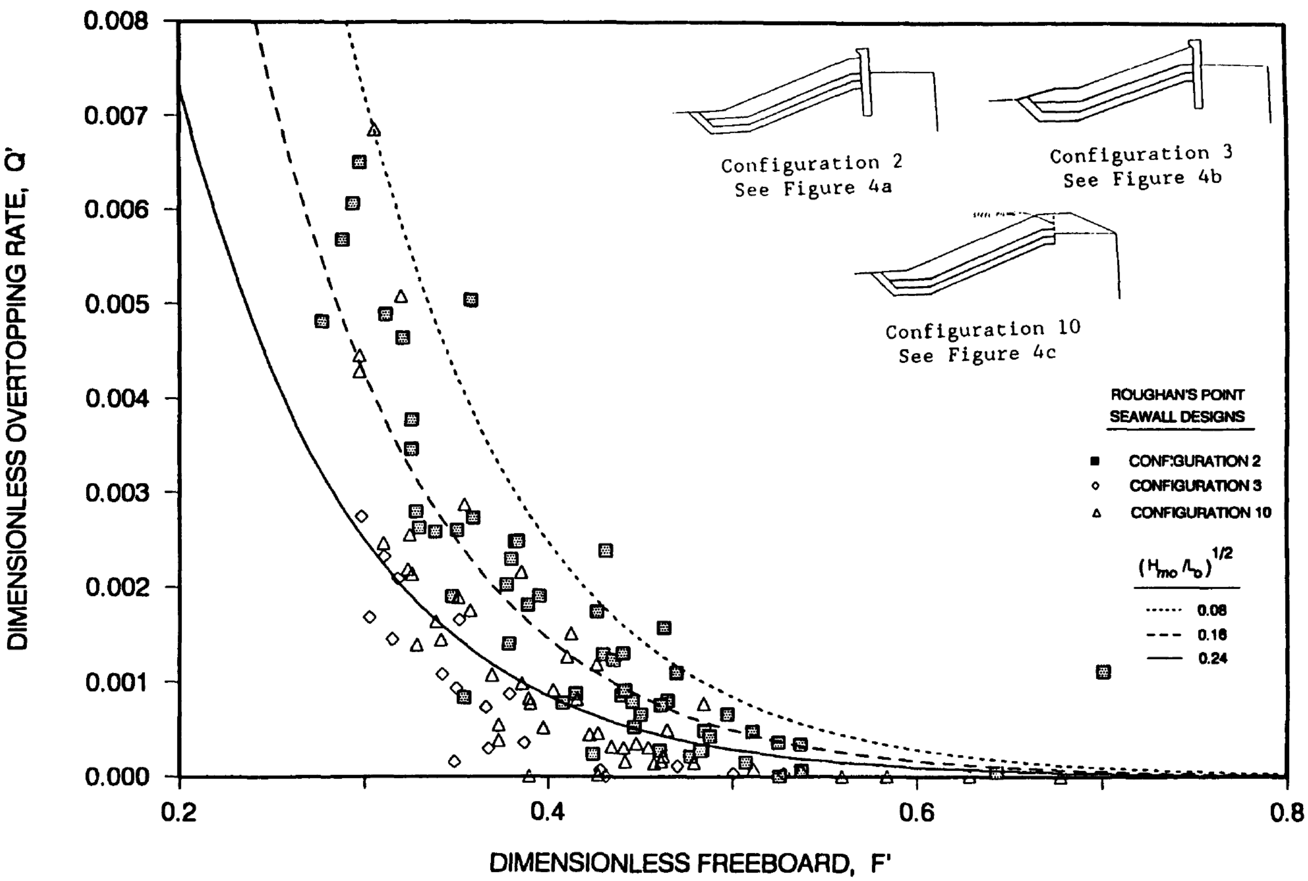
$Q0p := .308$ Group #2 Parameter

$Fprime := \dfrac{Rc}{\left(Htoe^{2} \cdot Lp\right)^{\frac{1}{3}}} = 0.258$ Dimensionless Freeboard

$X := \sqrt{\dfrac{Htoe}{Lo}} = 0.07$ Group #2 Parameter

$Qprime := Q0p \cdot \exp(C1 \cdot Fprime + C2 \cdot X) = 0.0122$ Dimensionless Overtopping

$Qaw := Qprime \cdot \sqrt{g \cdot Htoe^{3}} = 89.568 \cdot \dfrac{liter}{s \cdot m}$ Actual Overtopping



Data points are outside the range of conditions tested due to the relative low dimensionless freeboard which is caused by the relatively long waves.

Results

Probabilistic Runup and Mean Wave Overtopping - EurOtop 2007

$R2p = 18.7 \cdot ft$ $\quad Qp = 27.1 \cdot \frac{liter}{s \cdot m}$ $\quad Qp = 0.292 \cdot \frac{ft^3}{ft \cdot s}$

Deterministic Runup and Mean Wave Overtopping - EurOtop 2007

$R2d = 20.2 \cdot ft$ $\quad Qd = 43 \cdot \frac{liter}{s \cdot m}$ $\quad Qd = 0.463 \cdot \frac{ft^3}{ft \cdot s}$

Mean Wave Overtopping - Ward & Ahrens

$Qaw = 89.6 \cdot \frac{liter}{s \cdot m}$ $\quad Qaw = 0.964 \cdot \frac{ft^3}{ft \cdot s}$

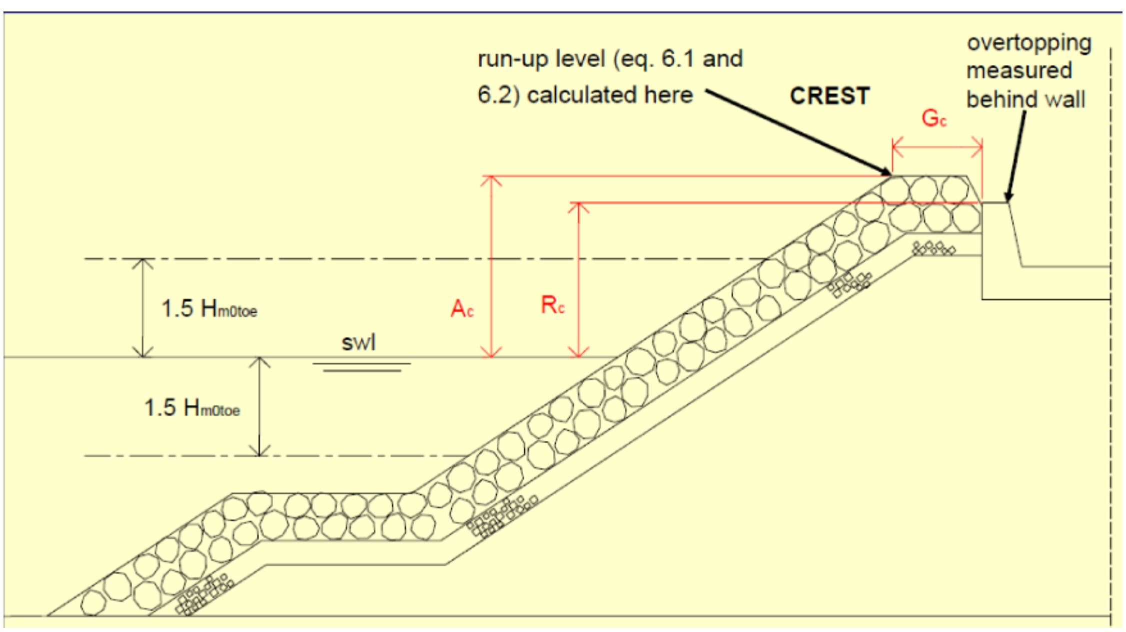
Date: July 14, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: With Project Coastal Processes
Design: Wave Runup & Overtopping - Scenario 1b - 309yr

Solving for the average wave overtopping rate of rubble mound embankment during 309-year conditions

Reference: Pullen et. al, 2007. "EurOtop - Wave Overtopping of Sea Defences and Related Structures: Assessment Manual"

Definitions

$Htoe := 8.2 \cdot ft$	significant wave height
$Tp := 15.2 \cdot sec$	peak wave period
$swl := 15.6 \cdot ft$	still water level
$\alpha := atan\left(\frac{1}{1.5}\right)$	structure slope
$\alpha n := atan\left(\frac{1}{11}\right)$	nearshore slope fronting structure
$Zcrest := 20.5ft$	crest elevation above datum
$Ztoe := 8 \cdot ft$	toe elevation above datum
$\gamma b := 1$	berm coefficient
$\gamma f := 0.55$	roughness of front slope
$\gamma \beta := 1$	wave angle coefficient
$\gamma v := 1$	coefficient for a vertical wall
$Gc := 18 \cdot ft$	crest width



Calculations

$Rc := Zcrest - swl = 4.9 \cdot ft$ freeboard at dune crest

$htoe := swl - Ztoe = 7.6 \cdot ft$ water depth at toe of structure

$mn := \tan(\alpha n) = 0.091$ nearshore slope

$Tm1 := \dfrac{Tp}{1.1} = 13.82\,s$ mean spectral wave period

$Lo := g \cdot \dfrac{Tp^2}{2 \cdot \pi} = 1183.1 \cdot ft$ deep water wave length

$Lp := \sqrt{g \cdot htoe}\,Tp = 237.7 \cdot ft$ wave length at toe of structure

$Lm1 := g \cdot \dfrac{Tm1^2}{2\pi} = 977.7 \cdot ft$ deep water wave length based on Tm1

$Som1 := \dfrac{Htoe}{Lm1} = 0.0084$ spectral wave steepness

$\xi m1 := \dfrac{\tan(\alpha)}{\sqrt{Som1}} = 7.28$ breaker parameter, surf similarity, irabarren number

Compute Probabilistic Wave Runup and Overtopping - EurOtop 2007

$R2a := Htoe \cdot 1.65 \cdot \gamma b \cdot \gamma f \cdot \gamma\beta \cdot \xi m1 = 54.17 \cdot ft$

$\gamma fsa := \gamma f + (\xi m1 - 1.8) \cdot \dfrac{(1 - \gamma f)}{8.2} = 0.851$

$\gamma fsb := 1$

$\gamma fs := \text{if}(\xi m1 \le 10, \gamma fsa, \gamma fsb) = 0.851$ adjusted roughness factor for surging

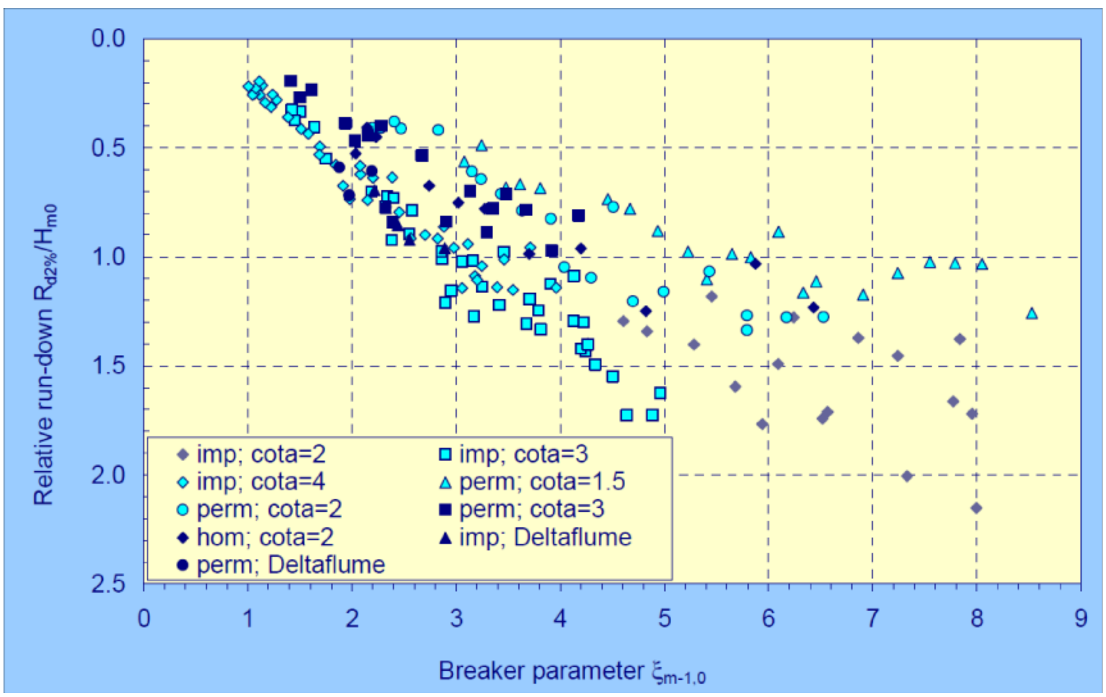
$R2b := Htoe \cdot \gamma b \cdot \gamma fs \cdot \gamma\beta \cdot \left(4 - \dfrac{1.5}{\sqrt{\xi m1}}\right) = 24.03 \cdot ft$

$R2p := \text{if}(R2a \le R2b, R2a, R2b) = 24.03 \cdot ft$

Relative Wave Runup

$\xi m1 = 7.28$ $\dfrac{R2p}{Htoe} = 2.93$

breaker parameter is outside normal conditions, results should be used with caution

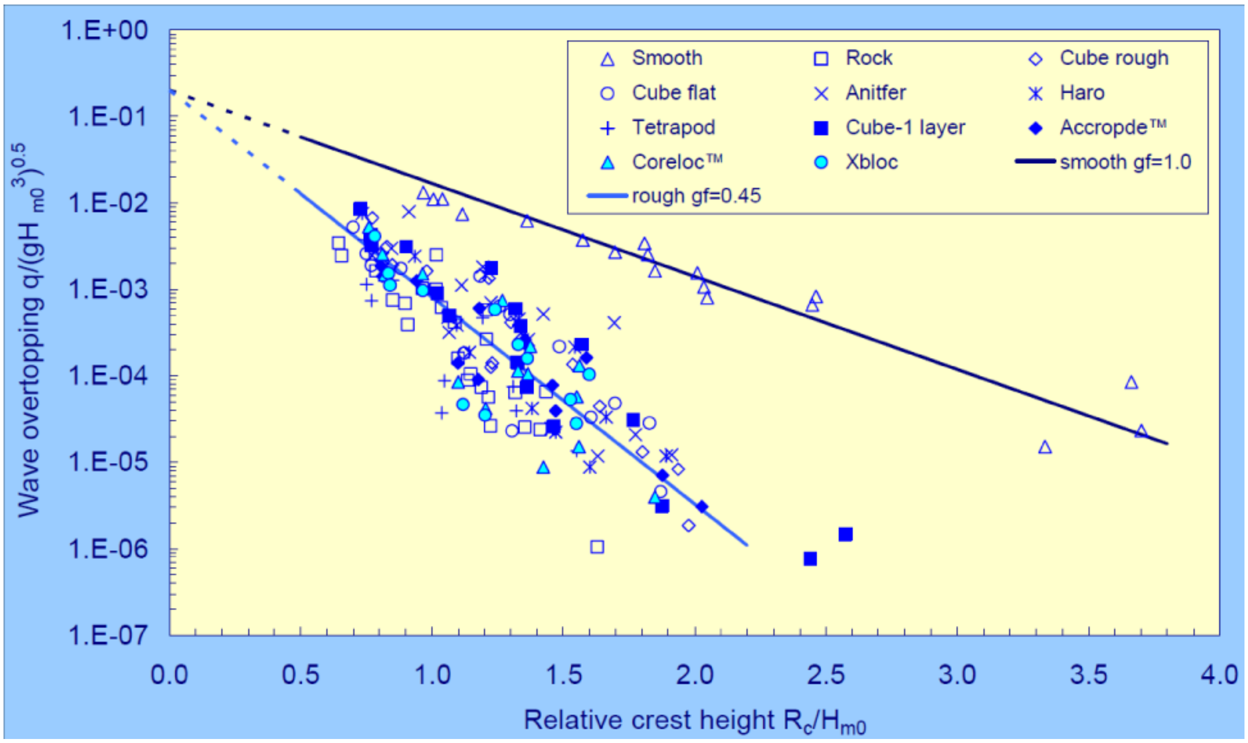


Probalisitic Overtopping Value

$$Qp := \sqrt{g \cdot Htoe^3} \cdot .2 \cdot \exp\left(-2.6 \cdot \frac{Rc}{Htoe \cdot \gamma f \cdot \gamma \beta}\right) = 146.799 \cdot \frac{liter}{s \cdot m}$$

Relative Wave Overtopping

$$\frac{Qp}{\left(g \cdot Htoe^3\right)^{0.5}} = 0.012 \qquad \frac{Rc}{Htoe} = 0.598$$



Compute Deterministic Wave Runup and Overtopping - EurOtop 2007

$$R2a := Htoe \cdot 1.75 \cdot \gamma b \cdot \gamma f \cdot \gamma\beta \cdot \xi m1 = 57.46 \cdot ft$$

$$R2b := Htoe \cdot \gamma b \cdot \gamma fs \cdot \gamma\beta \cdot \left(4.3 - \frac{1.6}{\sqrt{\xi m1}}\right) = 25.86 \cdot ft$$

$$R2d := if(R2a \le R2b, R2a, R2b) = 25.86 \cdot ft$$

$$Qa := \sqrt{g \cdot Htoe^3} \cdot \frac{.067}{\sqrt{\tan(\alpha)}} \cdot \xi m1 \cdot \gamma b \cdot \exp\left(-4.3 \cdot \frac{Rc}{Htoe} \cdot \frac{1}{\xi m1 \cdot \gamma b \cdot \gamma f \cdot \gamma\beta \cdot \gamma v}\right) = 3.891 \times 10^3 \cdot \frac{liter}{s \cdot m}$$

$$Qb := \sqrt{g \cdot Htoe^3} \cdot .2 \cdot \exp\left(-2.3 \cdot \frac{Rc}{Htoe \cdot \gamma f \cdot \gamma\beta}\right) = 203.366 \cdot \frac{liter}{s \cdot m}$$

Deterministic Overtopping Value

$$Qd := if(Qa \le Qb, Qa, Qb) = 203.366 \cdot \frac{liter}{s \cdot m}$$

Effect of Armored Crest wider than 3xD50

$$Cr := 3.06 \cdot \exp\left(-1.5 \cdot \frac{Gc}{Htoe}\right) = 0.114$$

$$Qd := Qd \cdot Cr = 23.121 \cdot \frac{liter}{s \cdot m}$$

$$Qp := Qp \cdot Cr = 16.69 \cdot \frac{liter}{s \cdot m}$$

Results

Probabilistic Runup and Mean Wave Overtopping - EurOtop 2007

$R2p = 24 \cdot ft$ $\quad Qp = 16.7 \cdot \frac{liter}{s \cdot m}$ $\quad Qp = 0.18 \cdot \frac{ft^3}{ft \cdot s}$

Deterministic Runup and Mean Wave Overtopping - EurOtop 2007

$R2d = 25.9 \cdot ft$ $\quad Qd = 23.1 \cdot \frac{liter}{s \cdot m}$ $\quad Qd = 0.249 \cdot \frac{ft^3}{ft \cdot s}$

Date: August 4, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: Armor Stone Stability
Design: Buried Seawall - Scenario 1a

Solve for required armor stone size based on wave conditions at toe of structure

References: Hudson's Formula (1977 SPM), Hudson's Formula (1984 SPM), Van der Meer's Formula (1988), Rock Manual (2006)

Definitions

H_s - significant wave height ($H_{1/3}$) at toe of structure.

H_{10} - average wave height of the largest 10% of waves.

H_{max} - maximum wave height (calculated from Goda)

T_p - peak wave period

T_m - mean wave period

swl - still water level (ft, NGVD)

Wave Inputs - Derived from Lower New York Harbor Wave Modeling Study (CERC, 1988)

Mean frequency of occurence relationships at Station 8, nominal depth of 17 ft MSL

$$RP := \begin{pmatrix} 100 \\ 164 \\ 309 \\ 500 \end{pmatrix} \cdot yr \qquad swl := \begin{pmatrix} 13.3 \\ 14.3 \\ 15.6 \\ 16.6 \end{pmatrix} \cdot ft \qquad Tp := \begin{pmatrix} 13.2 \\ 14.1 \\ 15.2 \\ 16 \end{pmatrix} \cdot s \qquad Hs := \begin{pmatrix} 4 \\ 4.8 \\ 5.8 \\ 6.6 \end{pmatrix} \cdot ft$$

$$H_{10} := 1.27Hs = \begin{pmatrix} 5.08 \\ 6.096 \\ 7.366 \\ 8.382 \end{pmatrix} \cdot ft \qquad Hmax := \begin{pmatrix} 5.9 \\ 6.9 \\ 8.2 \\ 9.2 \end{pmatrix} \cdot ft \qquad Tm := \frac{Tp}{1.25} = \begin{pmatrix} 10.56 \\ 11.28 \\ 12.16 \\ 12.8 \end{pmatrix} s$$

Structure Inputs

$mn := \frac{1}{15}$ rearshore slope, m

$etoe := 10 \cdot ft$ toe elevation of structure

$\gamma w := 64 \cdot \frac{lbf}{ft^3}$ unit weight of water

$\gamma r := 172.5 \cdot \frac{lbf}{ft^3}$ unit weight of stone

$Sr := \frac{\gamma r}{\gamma w} = 2.695$ specific gravity of stone

$n := 2$ number of layers

$mr := \frac{1}{1.5}$ structure slope, m

$k_{\Delta} := 1.0$ layer coefficient (DM 26.2) for 2 layers of rough quarrystone

$\Delta := \left(\frac{\gamma r}{\gamma w} - 1\right)$ $\Delta = 1.695$

$tonf := 2000 \cdot lbf$ unit definition

$zelv := \begin{pmatrix} 16 \\ 18 \\ 20.5 \\ 22.5 \end{pmatrix} \cdot ft$ structure crest elevation

$Rc := zelv - swl = \begin{pmatrix} 2.7 \\ 3.7 \\ 4.9 \\ 5.9 \end{pmatrix} \cdot ft$ structure freeboard

Hudson's Formula (1977 SPM)

1977 SPM reccomended using Hs and a Kd of 3.5 for permeable structures.

$Kd_{77} := 3.5$ stability coefficient based on two-layer, trunk stability for breaking waves, random placement

$$Wt_{77} := \overrightarrow{\frac{\gamma r \cdot Hs^3}{Kd_{77} \cdot (Sr - 1)^3 \cdot mr}} = \begin{pmatrix} 0.49 \\ 0.84 \\ 1.48 \\ 2.18 \end{pmatrix} \cdot tonf$$ weight of stone (SPM 1977)

$$D50_{77} := \overrightarrow{\left(\frac{Wt_{77}}{\gamma r}\right)^{\frac{1}{3}}} = \begin{pmatrix} 1.78 \\ 2.13 \\ 2.58 \\ 2.94 \end{pmatrix} \cdot ft$$ median diameter of stone (SPM 1977)

Hudson's Formula (1984 SPM)

1984 SPM reccomended using H_{10} and a Kd of 2.0 for permeable structures. This added a considerable amount of conservativeness to the original 1977 SPM.

$Kd_{84} := 2$ stability coefficient based on two-layer, trunk stability for breaking waves, random placement

$$Wt_{84} := \overrightarrow{\frac{\gamma r \cdot (H_{10})^3}{Kd_{84} \cdot (Sr - 1)^3 \cdot mr}} = \begin{pmatrix} 1.74 \\ 3.01 \\ 5.31 \\ 7.82 \end{pmatrix} \cdot tonf$$ weight of stone (SPM 1984)

$$D50_{84} := \overrightarrow{\left(\frac{Wt_{84}}{\gamma r}\right)^{\frac{1}{3}}} = \begin{pmatrix} 2.72 \\ 3.27 \\ 3.95 \\ 4.49 \end{pmatrix} \cdot ft$$ median diameter of stone (SPM 984)

Hudson's Formula (Rock Manual)

The Rock Manual reccomends using H_{10} and a Kd of 4.0 for permeable structures. The basis for these reccomendations comes from data gathere by Van der Meer and Van Gent et al (2004).

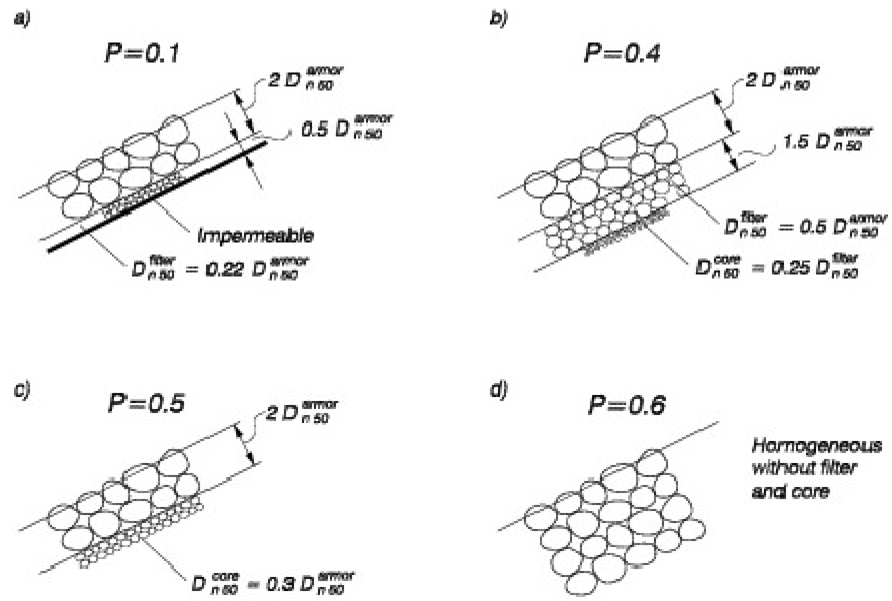
$Kd_{rm} := 4$ stability coefficient based on two-layer, trunk stability for breaking waves, random placement

$$Wt_{rm} := \overrightarrow{\frac{\gamma r \cdot (H_{10})^3}{Kd_{rm} \cdot (Sr - 1)^3 \cdot mr}} = \begin{pmatrix} 0.87 \\ 1.5 \\ 2.65 \\ 3.91 \end{pmatrix} \cdot tonf$$ weight of stone (Rock Manual)

$$D50_{rm} := \overrightarrow{\left(\frac{Wt_{rm}}{\gamma r}\right)^{\frac{1}{3}}} = \begin{pmatrix} 2.16 \\ 2.59 \\ 3.13 \\ 3.57 \end{pmatrix} \cdot ft$$ median diameter of stone (Rock Manual)

Van der Meer (1988)

If conditions are controlled by depth-limited waves then Hs is replaced by $H_{2\%}/1.4$. van der Meer suggests that Goda's H_{max} be used to approximate $H_{2\%}$. However, the Van der Meer formula is based on deep water (non depth-limited waves) & unbroken waves. A few test were run with breaking waves but were primarily spilling waves. Therefore, the Hudson formula is often preferred in depth-limited, breaking wave conditions.



$P_B := 0.4$ Structure Permeability factor

Damage level by S for two-layer armor *(CEM Tbl VI-5-21)*

Unit	Slope	Initial Damage	Intermediate Damage	Failure
Rock	1 : 1.5	2	3-5	8
Rock	1 : 2	2	4-6	8
Rock	1 : 3	2	6-9	12
Rock	1 : 4 - 1 : 6	3	8-12	17

$Sd := 2$ Damage Level, about 0 to 5%

$N := 4000$ Number of Waves

$$\mathrm{StormLength} := N \cdot Tm = \begin{pmatrix} 11.733 \\ 12.533 \\ 13.511 \\ 14.222 \end{pmatrix} \cdot hr$$

Wave Parameters (Van der Meer 1988)

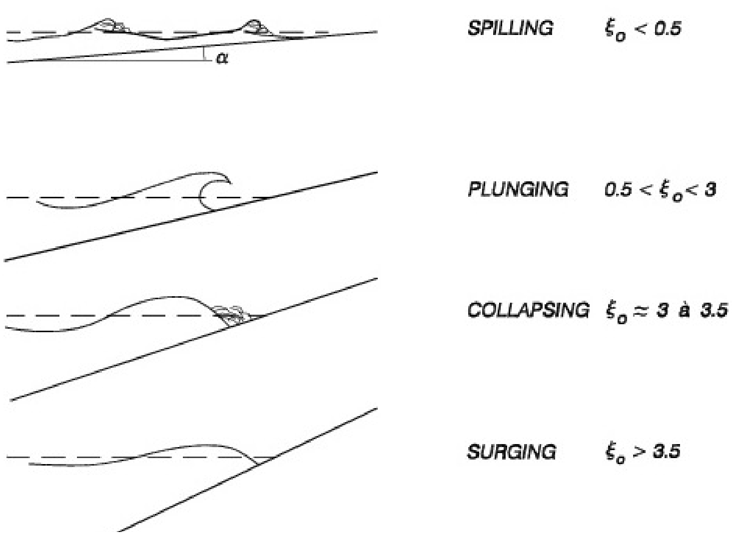
$$Lop := g\cdot\frac{Tp^2}{2\cdot\pi} = \begin{pmatrix}892.2\\1018\\1183.1\\1310.9\end{pmatrix}\cdot ft$$ deepwater wavelength - peak wave period

$$Sp := \frac{Hs}{Lop} = \begin{pmatrix}0.00448\\0.00471\\0.0049\\0.00503\end{pmatrix}$$ wave steepness - peak wave period

$$Lom := g\cdot\frac{Tm^2}{2\cdot\pi} = \begin{pmatrix}571\\651.5\\757.2\\839\end{pmatrix}\cdot ft$$ deepwater wavelength - mean wave period

$$Sm := \frac{Hs}{Lom} = \begin{pmatrix}0.007\\0.00737\\0.00766\\0.00787\end{pmatrix}$$ wave steepness - mean wave period

$$\varepsilon m := Sm^{-.5}\cdot mr = \begin{pmatrix}7.965\\7.767\\7.617\\7.516\end{pmatrix}$$ surf similarity, irrabaren number, and wave breaker parameter



$$\varepsilon mc := \left(6.2\cdot P_B^{0.31}\cdot mr^{0.5}\right)^{\frac{1}{P_B+0.5}}\cdot\begin{pmatrix}1\\1\\1\\1\end{pmatrix} = \begin{pmatrix}4.421\\4.421\\4.421\\4.421\end{pmatrix}$$

Armor Size (Van der Meer 1988)

$$fi := \overrightarrow{\left[1.25 - 4.8\cdot\frac{Rc}{Hs}\cdot\left(\frac{Sp}{2\cdot\pi}\right)^{0.5}\right]^{-1}} = \begin{pmatrix} 0.86 \\ 0.871 \\ 0.88 \\ 0.886 \end{pmatrix}$$

suggested reduction factor for overtoped rock slopes (CEM)

$$D50_p := \overrightarrow{\left(\frac{Hmax}{1.4}\cdot\frac{N^{.1}\cdot\varepsilon m^{0.5}}{\Delta\cdot 6.2\cdot Sd^{0.2}\cdot P_B^{\;0.18}}\cdot\frac{1}{fi}\right)} = \begin{pmatrix} 3.1 \\ 3.53 \\ 4.11 \\ 4.55 \end{pmatrix}\cdot ft$$

stone size for plunging waves (εm < εmc)

$$D50_s := \overrightarrow{\left[\frac{Hmax}{1.4}\cdot\frac{P_B^{\;0.13}\cdot N^{0.1}}{\Delta\cdot Sd^{0.2}\cdot\left(\frac{1}{mr}\right)^{0.5}\cdot\varepsilon m^{P_B}}\cdot\frac{1}{fi}\right]} = \begin{pmatrix} 1.82 \\ 2.13 \\ 2.52 \\ 2.82 \end{pmatrix}\cdot ft$$

stone size for surging waves (εm > εmc)

$$\alpha := \varepsilon m \le \varepsilon mc = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix}$$

$$D50_{vm} := \overrightarrow{\left[\alpha\cdot D50_p + (1-\alpha)\cdot D50_s\right]} = \begin{pmatrix} 1.82 \\ 2.13 \\ 2.52 \\ 2.82 \end{pmatrix}\cdot ft$$

identify stone size based on wave breaker criteria

$$Wt_{vm} := D50_{vm}^{\;3}\cdot\gamma r = \begin{pmatrix} 0.52 \\ 0.83 \\ 1.38 \\ 1.94 \end{pmatrix}\cdot tonf$$

calculate weight of armor stone

Summary

$$Wt_{77} = \begin{pmatrix} 0.49 \\ 0.84 \\ 1.48 \\ 2.18 \end{pmatrix} \cdot tonf \qquad D50_{77} = \begin{pmatrix} 1.8 \\ 2.1 \\ 2.6 \\ 2.9 \end{pmatrix} \cdot ft$$

Hudson (SPM 1977)

$$Wt_{84} = \begin{pmatrix} 1.74 \\ 3.01 \\ 5.31 \\ 7.82 \end{pmatrix} \cdot tonf \qquad D50_{84} = \begin{pmatrix} 2.7 \\ 3.3 \\ 3.9 \\ 4.5 \end{pmatrix} \cdot ft$$

Hudson (SPM 1984)

$$Wt_{rm} = \begin{pmatrix} 0.87 \\ 1.5 \\ 2.65 \\ 3.91 \end{pmatrix} \cdot tonf \qquad D50_{rm} = \begin{pmatrix} 2.2 \\ 2.6 \\ 3.1 \\ 3.6 \end{pmatrix} \cdot ft$$

Hudson (Rock Manual)

$$Wt_{vm} = \begin{pmatrix} 0.52 \\ 0.83 \\ 1.38 \\ 1.94 \end{pmatrix} \cdot tonf \qquad D50_{vm} = \begin{pmatrix} 1.8 \\ 2.1 \\ 2.5 \\ 2.8 \end{pmatrix} \cdot ft$$

Van der Meer (1988)

Use Hudson (Rock Manual) Results:

$$Wt := \begin{pmatrix} 1 \\ 1.5 \\ 3 \\ 4 \end{pmatrix} \cdot tonf \qquad D50 := \left(\frac{Wt}{\gamma r}\right)^{\frac{1}{3}} = \begin{pmatrix} 2.3 \\ 2.6 \\ 3.3 \\ 3.6 \end{pmatrix} \cdot ft$$

Date: August 4, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: Armor Stone Stability
Design: Buried Seawall - Scenario 1b

Solve for required armor stone size based on wave conditions at toe of structure

References: Hudson's Formula (1977 SPM), Hudson's Formula (1984 SPM), Van der Meer's Formula (1988), Rock Manual (2006)

Definitions

H_s - significant wave height ($H_{1/3}$) at toe of structure.

H_{10} - average wave height of the largest 10% of waves.

H_{max} - maximum wave height (calculated from Goda)

T_p - peak wave period

T_m - mean wave period

swl - still water level (ft, NGVD)

Wave Inputs - Derived from Lower New York Harbor Wave Modeling Study (CERC, 1988)

Mean frequency of occurence relationships at Station 8, nominal depth of 17 ft MSL

$$RP := \begin{pmatrix} 100 \\ 164 \\ 309 \\ 500 \end{pmatrix} \cdot yr \qquad swl := \begin{pmatrix} 13.3 \\ 14.3 \\ 15.6 \\ 16.6 \end{pmatrix} \cdot ft \qquad Tp := \begin{pmatrix} 13.2 \\ 14.1 \\ 15.2 \\ 16 \end{pmatrix} \cdot s \qquad Hs := \begin{pmatrix} 6.1 \\ 7 \\ 8.2 \\ 9.1 \end{pmatrix} \cdot ft$$

$$H_{10} := 1.27Hs = \begin{pmatrix} 7.747 \\ 8.89 \\ 10.414 \\ 11.557 \end{pmatrix} \cdot ft \qquad Hmax := \begin{pmatrix} 8.6 \\ 9.7 \\ 11.2 \\ 12.3 \end{pmatrix} \cdot ft \qquad Tm := \frac{Tp}{1.25} = \begin{pmatrix} 10.56 \\ 11.28 \\ 12.16 \\ 12.8 \end{pmatrix} s$$

Structure Inputs

$mn := \frac{1}{11}$ rearshore slope, m

$etoe := 8 \cdot ft$ toe elevation of structure

$\gamma w := 64 \cdot \frac{lbf}{ft^3}$ unit weight of water

$\gamma r := 172.5 \cdot \frac{lbf}{ft^3}$ unit weight of stone

$Sr := \frac{\gamma r}{\gamma w} = 2.695$ specific gravity of stone

$n := 2$ number of layers

$mr := \frac{1}{1.5}$ structure slope, m

$k_{\Delta} := 1.0$ layer coefficient (DM 26.2) for 2 layers of rough quarrystone

$\Delta := \left(\frac{\gamma r}{\gamma w} - 1\right)$ $\Delta = 1.695$

$tonf := 2000 \cdot lbf$ unit definition

$zelv := \begin{pmatrix} 16 \\ 18 \\ 20.5 \\ 22.5 \end{pmatrix} \cdot ft$ structure crest elevation

$Rc := zelv - swl = \begin{pmatrix} 2.7 \\ 3.7 \\ 4.9 \\ 5.9 \end{pmatrix} \cdot ft$ structure freeboard

Hudson's Formula (1977 SPM)

1977 SPM reccomended using Hs and a Kd of 3.5 for permeable structures.

$Kd_{77} := 3.5$ stability coefficient based on two-layer, trunk stability for breaking waves, random placement

$$Wt_{77} := \overrightarrow{\frac{\gamma r \cdot Hs^3}{Kd_{77} \cdot (Sr - 1)^3 \cdot mr}} = \begin{pmatrix} 1.72 \\ 2.6 \\ 4.18 \\ 5.72 \end{pmatrix} \cdot tonf$$ weight of stone (SPM 1977)

$$D50_{77} := \overrightarrow{\left(\frac{Wt_{77}}{\gamma r}\right)^{\frac{1}{3}}} = \begin{pmatrix} 2.71 \\ 3.11 \\ 3.65 \\ 4.05 \end{pmatrix} \cdot ft$$ median diameter of stone (SPM 1977)

Hudson's Formula (1984 SPM)

1984 SPM reccomended using H_{10} and a Kd of 2.0 for permeable structures. This added a considerable amount of conservativeness to the original 1977 SPM.

$Kd_{84} := 2$ stability coefficient based on two-layer, trunk stability for breaking waves, random placement

$$Wt_{84} := \overrightarrow{\frac{\gamma r \cdot (H_{10})^3}{Kd_{84} \cdot (Sr - 1)^3 \cdot mr}} = \begin{pmatrix} 6.17 \\ 9.33 \\ 14.99 \\ 20.49 \end{pmatrix} \cdot tonf$$ weight of stone (SPM 1984)

$$D50_{84} := \overrightarrow{\left(\frac{Wt_{84}}{\gamma r}\right)^{\frac{1}{3}}} = \begin{pmatrix} 4.15 \\ 4.76 \\ 5.58 \\ 6.19 \end{pmatrix} \cdot ft$$ median diameter of stone (SPM 984)

Hudson's Formula (Rock Manual)

The Rock Manual reccomends using H_{10} and a Kd of 4.0 for permeable structures. The basis for these reccomendations comes from data gathere by Van der Meer and Van Gent et al (2004).

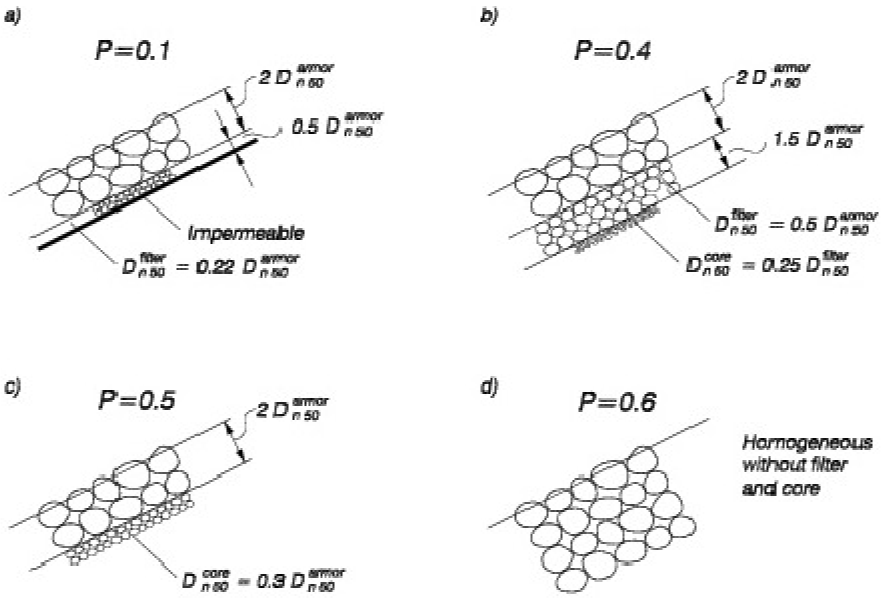
$Kd_{rm} := 4$ stability coefficient based on two-layer, trunk stability for breaking waves, random placement

$$Wt_{rm} := \overrightarrow{\frac{\gamma r \cdot (H_{10})^3}{Kd_{rm} \cdot (Sr - 1)^3 \cdot mr}} = \begin{pmatrix} 3.09 \\ 4.66 \\ 7.5 \\ 10.25 \end{pmatrix} \cdot tonf$$ weight of stone (Rock Manual)

$$D50_{rm} := \overrightarrow{\left(\frac{Wt_{rm}}{\gamma r}\right)^{\frac{1}{3}}} = \begin{pmatrix} 3.3 \\ 3.78 \\ 4.43 \\ 4.92 \end{pmatrix} \cdot ft$$ median diameter of stone (Rock Manual)

Van der Meer (1988)

If conditions are controlled by depth-limited waves then Hs is replaced by $H_{2\%}/1.4$. van der Meer suggests that Goda's H_{max} be used to approximate $H_{2\%}$. However, the Van der Meer formula is based on deep water (non depth-limited waves) & unbroken waves. A few test were run with breaking waves but were primarily spilling waves. Therefore, the Hudson formula is often preferred in depth-limited, breaking wave conditions.



$P_B := 0.4$ Structure Permeability factor

Damage level by S for two-layer armor *(CEM Tbl VI-5-21)*

Unit	Slope	Initial Damage	Intermediate Damage	Failure
Rock	1 : 1.5	2	3-5	8
Rock	1 : 2	2	4-6	8
Rock	1 : 3	2	6-9	12
Rock	1 : 4 - 1 : 6	3	8-12	17

$Sd := 2$ Damage Level, about 0 to 5%

$N := 4000$ Number of Waves

$$\text{StormLength} := N \cdot Tm = \begin{pmatrix} 11.733 \\ 12.533 \\ 13.511 \\ 14.222 \end{pmatrix} \cdot hr$$

Wave Parameters (Van der Meer 1988)

$$\mathrm{Lop} := g\cdot\frac{\mathrm{Tp}^2}{2\cdot\pi} = \begin{pmatrix}892.2\\1018\\1183.1\\1310.9\end{pmatrix}\cdot\mathrm{ft}$$

deepwater wavelength - peak wave period

$$\mathrm{Sp} := \frac{\mathrm{Hs}}{\mathrm{Lop}} = \begin{pmatrix}0.00684\\0.00688\\0.00693\\0.00694\end{pmatrix}$$

wave steepness - peak wave period

$$\mathrm{Lom} := g\cdot\frac{\mathrm{Tm}^2}{2\cdot\pi} = \begin{pmatrix}571\\651.5\\757.2\\839\end{pmatrix}\cdot\mathrm{ft}$$

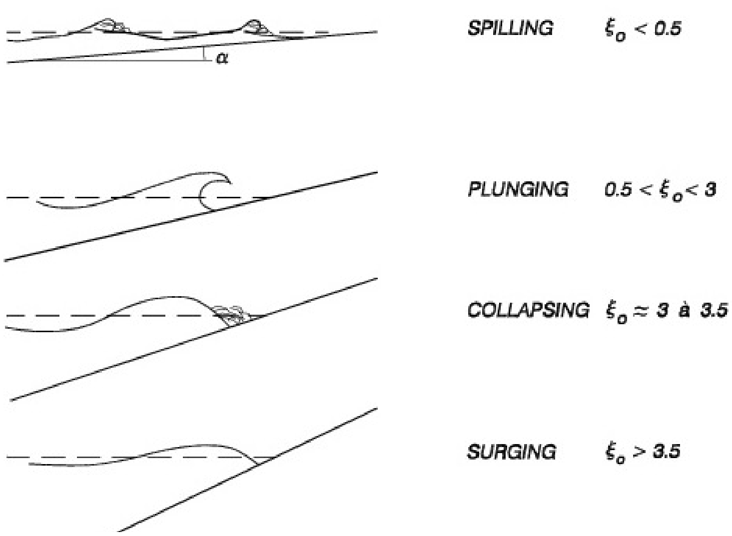
deepwater wavelength - mean wave period

$$\mathrm{Sm} := \frac{\mathrm{Hs}}{\mathrm{Lom}} = \begin{pmatrix}0.01068\\0.01074\\0.01083\\0.01085\end{pmatrix}$$

wave steepness - mean wave period

$$\varepsilon\mathrm{m} := \mathrm{Sm}^{-.5}\cdot\mathrm{mr} = \begin{pmatrix}6.45\\6.432\\6.406\\6.401\end{pmatrix}$$

surf similarity, irrabaren number, and wave breaker parameter



$$\varepsilon\mathrm{mc} := \left(6.2\cdot P_B^{\,0.31}\cdot\mathrm{mr}^{0.5}\right)^{\frac{1}{P_B+0.5}}\cdot\begin{pmatrix}1\\1\\1\\1\end{pmatrix} = \begin{pmatrix}4.421\\4.421\\4.421\\4.421\end{pmatrix}$$

Armor Size (Van der Meer 1988)

$$fi := \overrightarrow{\left[1.25 - 4.8 \cdot \frac{Rc}{Hs} \cdot \left(\frac{Sp}{2 \cdot \pi}\right)^{0.5}\right]^{-1}} = \begin{pmatrix} 0.848 \\ 0.858 \\ 0.866 \\ 0.872 \end{pmatrix}$$

suggested reduction factor for overtoped rock slopes (CEM)

$$D50_p := \overrightarrow{\left(\frac{Hmax}{1.4} \cdot \frac{N^{.1} \cdot \varepsilon m^{0.5}}{\Delta \cdot 6.2 \cdot Sd^{0.2} \cdot P_B^{0.18}} \cdot \frac{1}{fi}\right)} = \begin{pmatrix} 4.12 \\ 4.59 \\ 5.23 \\ 5.71 \end{pmatrix} \cdot ft$$

stone size for plunging waves (εm < εmc)

$$D50_s := \overrightarrow{\left[\frac{Hmax}{1.4} \cdot \frac{P_B^{0.13} \cdot N^{0.1}}{\Delta \cdot Sd^{0.2} \cdot \left(\frac{1}{mr}\right)^{0.5} \cdot \varepsilon m^{P_B}} \cdot \frac{1}{fi}\right]} = \begin{pmatrix} 2.93 \\ 3.27 \\ 3.75 \\ 4.09 \end{pmatrix} \cdot ft$$

stone size for surging waves (εm > εmc)

$$\alpha := \varepsilon m \le \varepsilon mc = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix}$$

$$D50_{vm} := \overrightarrow{\left[\alpha \cdot D50_p + (1 - \alpha) \cdot D50_s\right]} = \begin{pmatrix} 2.93 \\ 3.27 \\ 3.75 \\ 4.09 \end{pmatrix} \cdot ft$$

identify stone size based on wave breaker criteria

$$Wt_{vm} := D50_{vm}^{\ 3} \cdot \gamma r = \begin{pmatrix} 2.18 \\ 3.03 \\ 4.54 \\ 5.9 \end{pmatrix} \cdot tonf$$

calculate weight of armor stone

Summary

$$Wt_{77} = \begin{pmatrix} 1.72 \\ 2.6 \\ 4.18 \\ 5.72 \end{pmatrix} \cdot tonf \qquad D50_{77} = \begin{pmatrix} 2.7 \\ 3.1 \\ 3.6 \\ 4 \end{pmatrix} \cdot ft$$

Hudson (SPM 1977)

$$Wt_{84} = \begin{pmatrix} 6.17 \\ 9.33 \\ 14.99 \\ 20.49 \end{pmatrix} \cdot tonf \qquad D50_{84} = \begin{pmatrix} 4.2 \\ 4.8 \\ 5.6 \\ 6.2 \end{pmatrix} \cdot ft$$

Hudson (SPM 1984)

$$Wt_{rm} = \begin{pmatrix} 3.09 \\ 4.66 \\ 7.5 \\ 10.25 \end{pmatrix} \cdot tonf \qquad D50_{rm} = \begin{pmatrix} 3.3 \\ 3.8 \\ 4.4 \\ 4.9 \end{pmatrix} \cdot ft$$

Hudson (Rock Manual)

$$Wt_{vm} = \begin{pmatrix} 2.18 \\ 3.03 \\ 4.54 \\ 5.9 \end{pmatrix} \cdot tonf \qquad D50_{vm} = \begin{pmatrix} 2.9 \\ 3.3 \\ 3.7 \\ 4.1 \end{pmatrix} \cdot ft$$

Van der Meer (1988)

Use Hudson (Rock Manual) Results:

$$Wt := \begin{pmatrix} 3 \\ 5 \\ 8 \\ 10 \end{pmatrix} \cdot tonf \qquad D50 := \left(\frac{Wt}{\gamma r}\right)^{\frac{1}{3}} = \begin{pmatrix} 3.3 \\ 3.9 \\ 4.5 \\ 4.9 \end{pmatrix} \cdot ft$$

Date: August 11, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: Wave Forces
Design: Vertical Wall - Scenario 2

Solve for wave forces following Goda's design formula

References: Goda (2000)

Definitions

H_s - signficant wave height

H_{max} - maximum wave height

T_p - peak wave period

swl - still water level (ft, NGVD)

Wave Inputs - Derived from Lower New York Harbor Wave Modeling Study (CERC, 1988)

Wave conditions at toe of structure obtained with Goda's numerical model of random wave breaking

$$RP := \begin{pmatrix} 100 \\ 164 \\ 309 \end{pmatrix} \cdot yr \qquad swl := \begin{pmatrix} 13.3 \\ 14.3 \\ 15.6 \end{pmatrix} \cdot ft \qquad Tp := \begin{pmatrix} 13.2 \\ 14.1 \\ 15.2 \end{pmatrix} \cdot s \qquad Hmax := \begin{pmatrix} 4.5 \\ 5.3 \\ 6.3 \end{pmatrix} \cdot ft \qquad Hs := \begin{pmatrix} 3.0 \\ 3.6 \\ 4.4 \end{pmatrix} \cdot ft$$

Structure Inputs

$mn := \frac{1}{100}$ rearshore slope, m

$\beta := 0 \cdot deg$ angle of wave attack

$etoe := 10 \cdot ft$ toe elevation of structure

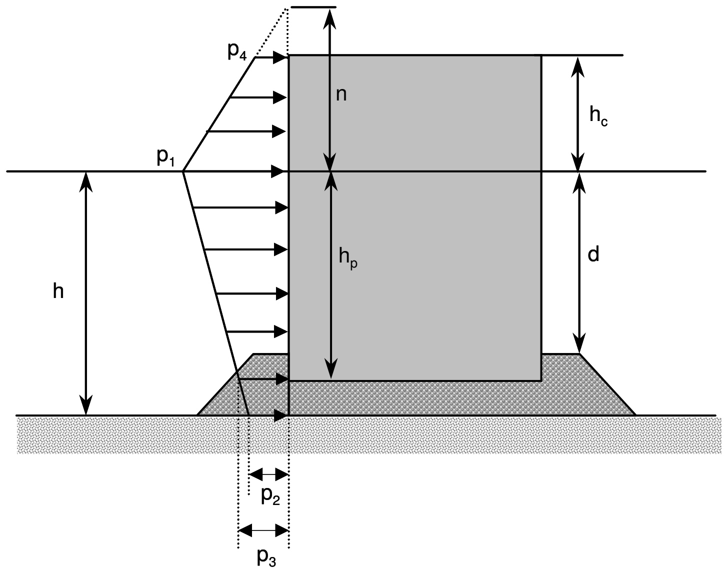
$Kip := 1000 \cdot lbf$ unit definition

$\rho := 1030 \frac{kg}{m^3}$ density of water

$zelv := \begin{pmatrix} 16 \\ 18 \\ 20.5 \end{pmatrix} \cdot ft$ structure crest elevation

Goda (2000) Formula of Wave Pressure

H_{max} at the location at a distance $5H_{1/3}$ seaward of structure (if inside the surfzone). However, in this case the ground is relatively flat and the wave height at the toe of structure is sufficient.



$$h := swl - etoe = \begin{pmatrix} 3.3 \\ 4.3 \\ 5.6 \end{pmatrix} \cdot ft$$ water depth

$$hc := zelv - swl = \begin{pmatrix} 2.7 \\ 3.7 \\ 4.9 \end{pmatrix} \cdot ft$$ structure freeboard

$$hp := h + 2 \cdot ft = \begin{pmatrix} 5.3 \\ 6.3 \\ 7.6 \end{pmatrix} \cdot ft$$ depth including rubble scour protection

Additional Wave Definitions

$L := 150 \cdot m$ Initial guess

$$h_b := \overrightarrow{(h + 5 \cdot Hs \cdot mn)} = \begin{pmatrix} 3.45 \\ 4.48 \\ 5.82 \end{pmatrix} \cdot ft$$ breaking wave depth

Given

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

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

$$L := \overrightarrow{Wavel(Tp, h)} = \begin{pmatrix} 135.5 \\ 165.1 \\ 203 \end{pmatrix} \cdot ft$$

B) Elevation to which the wave pressure is exerted

$$n := 0.75\cdot(1 + \cos(\beta))\cdot Hmax = \begin{pmatrix} 6.75 \\ 7.95 \\ 9.45 \end{pmatrix}\cdot ft$$

maximum elevation of wave pressure:

C) Wave pressure on the front of a vertical wall

$$\alpha 1 := 0.6 + 0.5\cdot\left(\frac{4\cdot\pi\cdot\frac{h}{L}}{\sinh\left(4\cdot\pi\cdot\frac{h}{L}\right)}\right)^2 = \begin{pmatrix} 1.085 \\ 1.083 \\ 1.08 \end{pmatrix}$$

$$arg1 := \overrightarrow{\left[\frac{h_b - h}{3\cdot h_b}\cdot\left(\frac{Hmax}{h}\right)^2\right]} = \begin{pmatrix} 0.027 \\ 0.02 \\ 0.016 \end{pmatrix} \qquad arg2 := \frac{2\cdot h}{Hmax} = \begin{pmatrix} 1.467 \\ 1.623 \\ 1.778 \end{pmatrix} \qquad \psi := arg1 \le arg2 = \begin{pmatrix} 1 \\ 1 \\ 1 \end{pmatrix}$$

$$\alpha 2 := \overrightarrow{[arg1\cdot\psi + arg2\cdot(1 - \psi)]} = \begin{pmatrix} 0.027 \\ 0.02 \\ 0.016 \end{pmatrix}$$

$$\alpha 3 := \overrightarrow{\left[1 - \left[\frac{hp}{h}\cdot\left(1 - \frac{1}{\cosh\left(2\cdot\pi\cdot\frac{h}{L}\right)}\right)\right]\right]} = \begin{pmatrix} 0.981 \\ 0.981 \\ 0.98 \end{pmatrix}$$

$$p1 := \overrightarrow{\left[0.5\cdot(1 + \cos(\beta))\cdot\left(\alpha 1 + \alpha 2\cdot\cos(\beta)^2\right)\cdot\rho\cdot g\cdot Hmax\right]} = \begin{pmatrix} 0.322 \\ 0.376 \\ 0.444 \end{pmatrix}\cdot\frac{Kip}{ft^2}$$

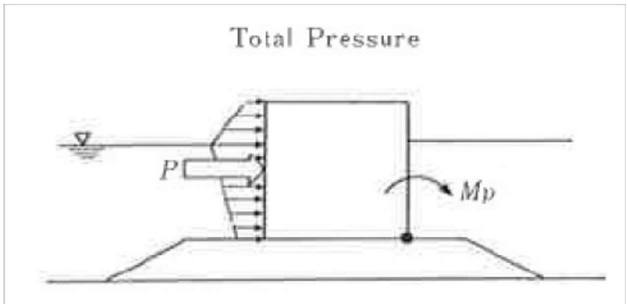
$$p2 := \frac{p1}{\cosh\left(2\cdot\pi\cdot\frac{h}{L}\right)} = \begin{pmatrix} 0.318 \\ 0.371 \\ 0.438 \end{pmatrix}\cdot\frac{Kip}{ft^2}$$

$$p3 := \overrightarrow{(\alpha 3\cdot p1)} = \begin{pmatrix} 0.316 \\ 0.369 \\ 0.435 \end{pmatrix}\cdot\frac{Kip}{ft^2}$$

$$arg1 := \overrightarrow{\left[p1\cdot\left(1 - \frac{hc}{n}\right)\right]} = \begin{pmatrix} 0.193 \\ 0.201 \\ 0.214 \end{pmatrix}\cdot\frac{Kip}{ft^2} \qquad arg2 := 0\cdot\frac{Kip}{ft^2} \qquad \psi := n > hc = \begin{pmatrix} 1 \\ 1 \\ 1 \end{pmatrix}$$

$$p4 := arg1\cdot\psi + arg2\cdot(1 - \psi) = \begin{pmatrix} 0.608 \\ 0.608 \\ 0.608 \end{pmatrix}\cdot\frac{Kip}{ft^2}$$

Total Horizontal Wave Pressure and Overturning Moment



$$\psi := n < hc = \begin{pmatrix} 0 \\ 0 \\ 0 \end{pmatrix} \qquad hh := \overrightarrow{[n \cdot \psi + hc \cdot (1 - \psi)]} = \begin{pmatrix} 2.7 \\ 3.7 \\ 4.9 \end{pmatrix} \cdot ft$$

$$PF := \overrightarrow{[0.5 \cdot (p1 + p3) \cdot hp + 0.5 \cdot (p1 + p4) \cdot hh]} = \begin{pmatrix} 2.94 \\ 4.16 \\ 5.92 \end{pmatrix} \cdot \frac{Kip}{ft}$$

Total Horizontal Force

$$Mp := \overrightarrow{\left[\frac{1}{6} \cdot (2 \cdot p1 + p3) \cdot hp^2 + \frac{1}{2}(p1 + p4) hp \cdot hh + \frac{1}{6}(p1 + 2 \cdot p4) \cdot hh^2\right]} = \begin{pmatrix} 13 \\ 22.5 \\ 39 \end{pmatrix} \cdot Kip \cdot \frac{ft}{ft}$$

Overturning Moment

$$L_{arm} := \frac{Mp}{PF} = \begin{pmatrix} 4.42 \\ 5.4 \\ 6.58 \end{pmatrix} \cdot ft$$

Length of Moment Arm

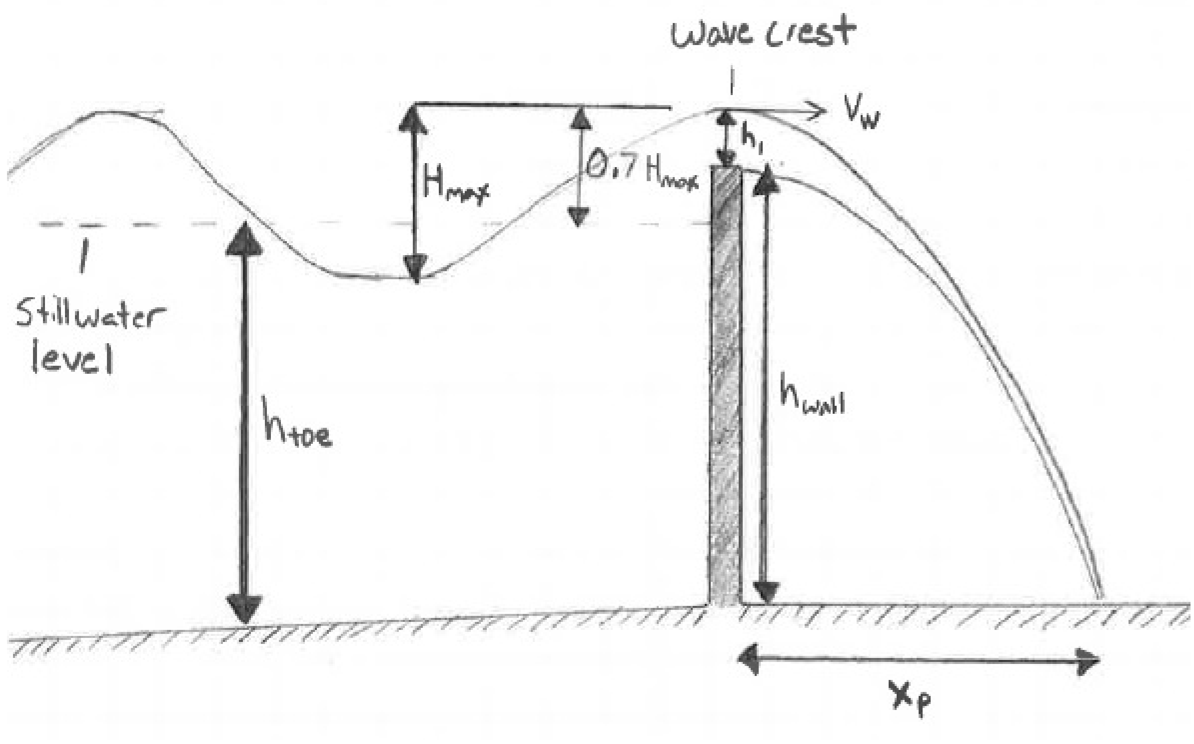
Date: October 2, 2014
Analyst: Rob Hampson, Moffatt & Nichol
Client: U.S. Army Corps of Engineers
Project: South Shore of Staten Island, NY
Analysis: With Project Coastal Processes
Design: Leeside Scour Protection - Vertical Wall - Scenario 2 - 309 yr

Solve for overtopping jet characteristics and stone size.

Reference: Schiereck, G. J., 2001. "Introduction to Bed, bank and shore protection", Delft University Press

Definitions

$Htoe := 4.4 \cdot ft$	significant wave height
$Tp := 15.2 \cdot sec$	peak wave period
$swl := 15.6 \cdot ft$	water level
$\alpha n := atan\left(\frac{1}{100}\right)$	nearshore slope fronting structure
$Zcrest := 20.5ft$	crest elevation above datum
$Ztoe := 10 \cdot ft$	toe elevation above datum
$Zback := 6 \cdot ft$	elevation on back side of structure (excluding armor stone)



Calculations

$Rc := Zcrest - swl = 4.9 \cdot ft$ freeboard at dune crest

$htoe := swl - Ztoe = 5.6 \cdot ft$ water depth at toe of structure

$mn := \tan(\alpha n) = 0.01$ nearshore slope

$Tm1 := \frac{Tp}{1.1} = 13.82\,s$ mean spectral wave period

$Lo := g \cdot \frac{Tp^2}{2 \cdot \pi} = 1183.1 \cdot ft$ deep water wave length

$Lp := \sqrt{g \cdot htoe}\,Tp = 204 \cdot ft$ wave length at toe of structure

$Lm1 := g \cdot \frac{Tm1^2}{2\pi} = 977.7 \cdot ft$ deep water wave length based on Tm1

$Som1 := \frac{Htoe}{Lm1} = 0.0045$ spectral wave steepness

$Hmax := Htoe \cdot 1.8 = 7.92 \cdot ft$ maximum wave height at toe of structure (statistical max)

$\omega := 2 \cdot \frac{\pi}{Tp} = 0.413 \frac{1}{s}$ angular wave period

Overtopping Jet

$Vw := \frac{g}{\omega} \cdot \tanh\left(\frac{2\pi}{Lp} \cdot htoe\right) = 13.291 \cdot \frac{ft}{s}$ phase velocity of wave

$Zjet := 0.7 \cdot Hmax + swl = 21.144 \cdot ft$ elevation of jet passing over the wall

$hjet := Zjet - Zback = 15.144 \cdot ft$ height of jet above landside

$hwall := Zcrest - Zback = 14.5 \cdot ft$ height of wall above landside

$h1 := hjet - hwall = 0.644 \cdot ft$ thickness of jet

$q1 := h1 \cdot Vw = 8.56 \cdot \frac{ft^3}{s \cdot ft}$ flow rate of jet

$q1 = 795.219 \cdot \frac{liter}{s \cdot m}$ flow of jet in liters/s/m (note this is maximum jet, not average)

$tf := \left(2 \cdot \frac{hwall}{g}\right)^{\frac{1}{2}} = 0.949\,s$ time before jet reaches bed

$Xp := Vw \cdot tf = 12.619 \cdot ft$ horizontal distance of jet

$Xpnet := Xp - 1.5 \cdot ft = 11.119 \cdot ft$ horizontal distance of jet including thickness of T-wall

$Vg := (2 \cdot g \cdot hwall)^{\frac{1}{2}} = 30.546 \cdot \frac{ft}{s}$ vertical velocity of jet upon impact

$\theta := \tan\left(\frac{Vw}{Vg}\right) = 26.634 \cdot deg$ angle of jet upon impact

Rock Size

$\gamma r := 172.5 \cdot \dfrac{lb}{ft^3}$ density of rock

$\gamma w := 64 \cdot \dfrac{lb}{ft^3}$ density of water

$\Delta := \dfrac{\gamma r}{\gamma w} - 1 = 1.695$ specific density

$Dn50 := 0.7 \cdot \dfrac{Vw^2}{(2 \cdot g \cdot \Delta)} = 1.134 \cdot ft$ median stone size (Izbash)

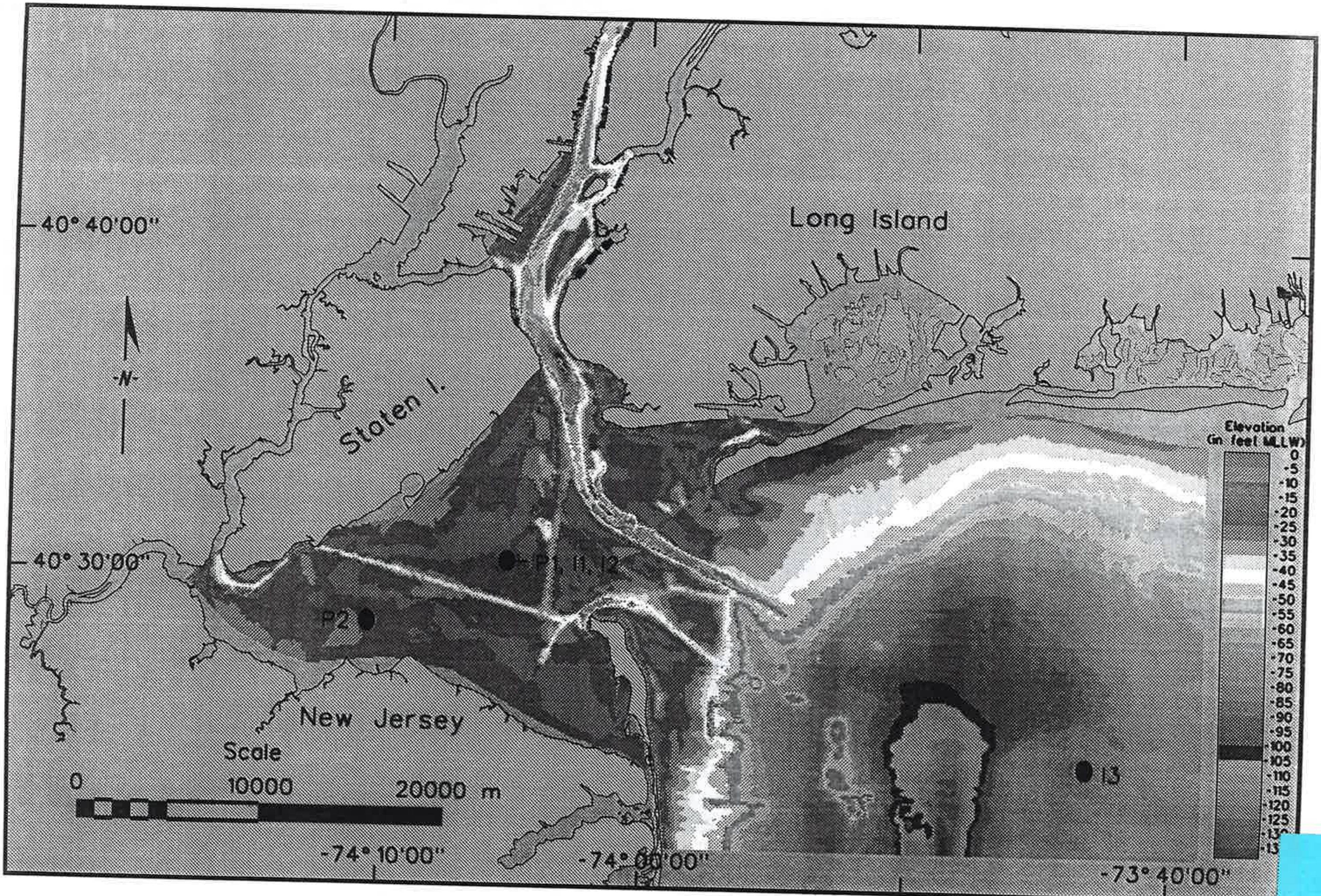
$Wn50 := Dn50^3 \cdot \gamma r = 251.277 \cdot lb$ median weight of stone

ATTACHMENT D

DMMP STWAVE MODELING

ATTACHMENT D

DMMP STWAVE MODELING


Long Island
Staten I.
New Jersey
-N-
40° 40'00"
40° 30'00"
-74° 10'00"
-74° 00'00"
-73° 40'00"
P1, I1, I2
P2
I3
Scale
0
10000
20000 m
Elevation
(in feet MLLW)
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
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-115
-120
-125

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REMARKS Per Conversation — Updated portion of DMMP Chapter 2 Wave Climate and some draft supplementary polar plots which help visualize tabulated peak storm conditions. Call if you have further questions.

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Storms

As identified in Chapter 1, a suite of major storms impacting New York Bight was selected for modeling. The WIS database provided incident wave information for 20 selected extratropical storms at WIS Station 73, near the STWAVE detailed ocean grid boundary. Storm wave height, period, and direction parameters and wind speed and direction parameters were derived at 3-hr intervals over a period of 3 days spanning the storm peak. These parameters were used to recreate incident directional wave spectra for the STWAVE detailed ocean grid. All spectra were treated as actively growing sea conditions. Water level information in Lower Bay at 3-hr intervals during each storm was derived from ADCIRC numerical modeling (see Chapter 1) and included in STWAVE input. Water depth for wave modeling was based on time variation of water surface elevation. Maximum significant wave height and wind conditions from each storm are listed in Table 5.

Tropical storms were not sufficiently represented in the WIS database. Special WIS model runs were made to generate wave information for 20 selected events (Chapter 1). Storm wave parameters were saved at 1-hr intervals over a 2-day period spanning the storm peak. Because tropical storm winds can change significantly over short distances, wind parameters were saved at a WIS point nearer the entrance to Lower Bay, at 40.5 deg N, 74 deg W. Water level information from ADCIRC modeling was at 2-hr intervals. Maximum significant wave height and wind conditions from each storm are listed in Table 6.

STWAVE was run for each of the selected storms. Information saved includes incident wave spectra, wave information over the grid for each condition run (significant height, peak period, and peak direction at each grid point), and special summary files for use in the sediment modeling described in Chapters 5 and 6. Wave field plots for the extratropical and tropical storms giving the highest wave height at the P1/I1/I2 site are shown in Figures 20 and 21. Wave patterns are quite similar for the two storms. Wave energy concentrates over shoal areas in the approach and entrance to Lower Bay. Wave height drops steadily over the shoals as shallow water-induced breaking and energy dissipation continue to impact waves entering Lower Bay. Wave height in the outer portion of the two navigation channels is visibly lower than over the surrounding shoals. The P1/I1/I2 location is sufficiently far inside Lower Bay that it is protected from much of the storm wave energy. Significant wave height from both storms at the P1/I1/I2 location is 1.8 m, less than one-third of the incident storm wave height.

Storm wave response in Lower Bay involves several additional complications. The shoals at the entrance to Lower Bay, between Sandy Hook and Ambrose Channel, have controlling depths of 3-4 m MLLW. The effect of this "gate" depends on incident ocean wave conditions, astronomical tide, and storm-generated water levels. Sites in Lower Bay are also exposed to local fetches which, coupled with strong storm winds, can result in locally-generated waves of concern. Local storm wave conditions depend strongly on wind speed and direction. Local wave generation may restore some energy to incident ocean waves which have broken on entering Lower Bay or, if wind direction differs significantly from incident wave direction, generate a new wave component from another direction. Water depth limitations may also affect extreme storm waves inside Lower

Bay, especially at the P2 location, so that water level further influences extreme wave conditions.

Locally-generated storm waves in Lower Bay were quantified with some additional STWAVE runs. Extratropical and tropical storm events were selectively modeled to include at least the peak wind speed and fetch combinations in each storm relative to the P1/I1/I2 and P2 locations. Model grids for this study task were developed to accommodate all possible wind directions as part of the New York New Jersey Channel Deepening Study underway at WES, funded by the New York District. Fetches for winds from easterly directions are determined by the seaward grid boundary location rather than by land boundaries. The seaward boundary for this grid is a north/south line just east of Ambrose Light.

Peak wave conditions in each storm at the P1/I1/I2 location due to incident ocean waves and locally-generated waves are summarized in Tables 7 and 8. The date-time (year/month/day/hour) and wind conditions for each peak event are given. For some storms, the peak ocean and local wave conditions occurred at the same time, but in most cases they were a few hours apart. Extratropical and tropical storms are summarized in separate tables.

The tables indicate that ocean waves arrive at P1 from a very narrow direction range, 91-97 deg azimuth. Maximum significant wave height at P1 due to ocean waves is 2.46 m, at 1700 hr UTC on 27 September 1985. The corresponding wave period is 9 sec. Wave periods for peak ocean conditions range from 5 sec to 14 sec, with extratropical storms typically giving longer periods than tropical storms. Maximum significant wave height at P1 due to locally-generated waves is 2.48 m, at 1500 hr UTC on 27 September 1985. The corresponding wave period and direction are 6.8 sec and 73 deg azimuth. Wave periods for peak locally-generated waves range from 3.5 sec to 6.8 sec. Wave directions are varied, generally between 90 deg and 270 deg for extratropical storms and between 0 deg and 110 deg for tropical storms. The storm events indicate that significant wave heights greater than 1 m can approach P1 from any direction except northwest.

Peak wave conditions in each storm at the P2 location are summarized similarly (Tables 9 and 10). Ocean waves arrive at this sheltered location from directions of 68-72 deg. Maximum significant height at P2 due to ocean waves is 0.9 m, due to a variation of the 27 September 1985 tropical storm. The corresponding wave period is 9 sec. Maximum significant wave height at P2 due to locally-generated waves is 2.18 m, also from a variation of the 27 September 1985 tropical storm. The corresponding wave period and direction are 6 sec and 50 deg azimuth. Wave periods for peak locally-generated waves range from 2.6 sec to 6 sec. Wave directions are varied over similar ranges as at the P1 location. Significant wave heights greater than 1 m occurred from approach directions between 341 deg and 104 deg azimuth.

This analysis of storm wave conditions indicates that both ocean waves propagating into Lower Bay and waves locally-generated inside Lower Bay are important considerations at P1/I1/I2 and P2 locations. The combined effect of these two wave components during peak storm conditions depends strongly on relative wave directions and water level. If the ocean wave and locally-generated wave components differ significantly in direction, they may act as independent wave systems. Significant wave height due to combined ocean and local wave components may be approximated by taking the root-mean-square of component significant heights. Shallow depths in various parts of Lower Bay may impose limits on combined significant height.

Extreme Waves

Extreme wave events were simulated using the Empirical Simulation Technique (EST) described in Chapter 1. The input to EST analysis was results of wave model storm simulations with variable water levels (tide and surge) using existing bathymetry. Maximum wave height (storm peak) and a standard deviation are given for the locations of pits P1 and P2, and islands I1, I2, and I3. Pit and island bathymetries were not used in modeling the storm waves.

Tables 11 and 12 provide extreme ocean wave estimates for extratropical and tropical storms, respectively. Table 13 provides extreme locally-generated tropical storm wave estimates for pit/island sites in Lower Bay. Locally-generated extratropical storm waves were omitted because they often came from southerly directions with relatively short fetches and are probably not representative of true extreme local conditions.

Although the tables give H_s values up to a return period of 200 years, results for the longer return periods are speculative. Typically, extremes at return periods of up to three times the time period covered by storm information are considered realistic. Depth-induced limits on H_s are very evident. Extreme H_s values at P2 are well below those at P1/I1/I2, and both are much lower than at I3.

With the complexities introduced at Lower Bay sites by water level controls on H_s, simultaneous ocean and locally-generated wave components, and variable directions, the extreme wave tables should only be used with caution. For any design applications where the combined ocean and locally-generated wave components could approach depth-limited values, a depth-limited wave height based on design water level would be preferable to the tables given here.

Table 5. Extratropical Storm Peak Incident Wave & Wind Conditions from WIS Modeling

Date	Max. Wind[1]		Maximum Incident Wave		
	Speed m/s	Dir., deg az.	Hs, m	Tp, sec	θp, deg az.
77110721	20	110	5.3	12	122
78010900 78011000	19 22	170 285	5.0	11	151
78012815 78012812	21 22	210 190	5.7	12	166
79012600 79012418	25 30	130 125	6.4	13	133
79032421 79032418	21 22	150 160	5.7	12	151
80102521 80102812	16 24	175 270	6.4	13	137
81020212	21	185	5.7	12	166
81021112 81021106	26 27	175 165	6.9	14	151
81033021 81033018	17 20	210 210	4.2	11	173
83121218 83121206	19 20	125 110	5.8	13	126
84032912	23	90	6.3	13	119
85021218	25	130	5.8	11	137
85092715	36	60	7.0	12	115
85110503 85110500	20 21	110 110	6.8	13	130
86031918 86031912	23 25	225 200	5.7	11	176
86120303 86120300	20 22	140 135	7.3	14	137
87033115 87033106	17 19	180 160	5.3	13	151
92121109 92121106	20 24	110 115	5.5	12	130
93112812 93112806	17 19	155 145	5.5	12	140
95111203 95111200	17 19	210 175	5.4	11	156

[1] If max. wind speed occurs at different time during storm than max incident wave height, both are listed. Max. wind based on WIS Station 73.

Table 6. Tropical Storm Peak Incident Wave & Wind Conditions from WIS Modeling

Date	Max. Wind[1]		Maximum Incident Wave		
	Speed m/s	Dir., deg az.	Hs, m	Tp, sec	θp, deg az.
85092717 85092716	24 32	345 35	5.2	9	86
85092718a 85092716a	18 35	310 40	6.8	10	117
85092716b	35	45	6.0	10	74
76081008 76081005	11 18	300 10	3.3	8	132
72062222 72062219	12 25	260 20	4.8	11	221
71082812	24	215	3.6	7	171
67091621 67091608	7 9	40 65	2.2	9	89
60091223 60091219	8 21	335 20	3.5	10	116
55081314 55081313	17 18	115 110	4.7	11	109
54083113	19	5	2.9	6	38
54101602 54101601	19 21	160 135	3.2	7	133
44091506 44091601	13 23	310 25	3.7	10	127
38092121	29	340	4.1	7	6
38092120a	34	25	4.8	8	44
38092120b	23	5	3.2	7	25
36091908 36091905	12 18	355 30	3.9	10	102
35090618 35090617	12 13	50 55	3.5	9	87
33082406	19	140	4.4	10	126
33091711	13	45	4.6	12	91
29100307 29100306	23 24	190 170	4.9	10	165

[1] If max. wind speed occurs at different time during storm than max incident wave height, both are listed. Max. wind based on WIS point at 40.5 deg N, 74 deg W.

Table 7. Extratropical Storm Peak Wave Conditions at Pit P1 from STWAVE Modeling

Date	Wind		Incident[1] Hs, m	Waves at P1, Ocean			Waves at P1, Local Wind		
	Speed m/s	Dir., deg az.		Hs, m	Tp, sec	θp, deg az.	Hs, m	Tp, sec	θp, deg az.
77110803 77110721	19 20	115 110	5.2	1.84	13.0	95	1.36	5.2	102
78010821 78011000	19 22	165 285	4.8	1.54	11.0	96	1.16	4.3	281
78012609 78012706	21 20	175 255	4.9	1.38	10.0	98	1.14	4.2	257
79012421 79012418	27 30	125 125	6.3	2.05	12.0	95	1.73	6.4	114
79032500 79032418	21 22	155 150	5.7	1.56	13.0	98	1.17	3.9	153
80102615 80102612	19 24	130 270	5.9	1.80	12.0	98	1.40	4.5	268
81020212	21	185	5.7	1.35	12.0	97	1.00	3.6	185
81021106	27	165	6.1	1.66	12.0	96	1.46	4.1	168
81033021 81032918	17 20	210 245	4.2	1.13	11.0	97	1.09	4.3	251
83121215 83121206	20 20	120 110	5.6	1.86	13.0	95	1.35	5.2	102
84032912	23	90	6.3	1.99	13.0	95	1.78	5.4	90
85021215	25	125	5.2	1.91	10.0	95	1.40	5.9	114
85092712 85092715	23 36	90 60	6.4	1.81	10.0	96	2.48	6.8	73
85110508 85110500	19 21	105 110	5.3	1.63	14.0	95	1.42	5.3	102
86031818 86031912	23 25	225 200	5.7	1.26	11.0	97	1.21	4.0	200
86120300 86120218	22 20	135 115	6.9	1.84	13.0	96	1.27	5.2	105
87033109 87033106	19 19	165 160	5.2	1.52	12.0	96	0.94	3.5	161
92121106	24	115	5.3	1.95	10.0	95	1.56	5.7	106
93112809 93112806	18 19	160 145	5.2	1.63	12.0	96	0.99	3.7	149
95111203 95111200	17 19	210 175	5.4	1.41	11.0	97	0.91	3.6	175

[1] Incident wave height corresponding to max. significant wave height at P1 due to ocean waves.

Table 8. Tropical Storm Peak Wave Conditions at Pit P1 from STWAVE Modeling

Date	Wind[1]		Incident[2] H_s, m	Waves at P1, Ocean			Waves at P1, Local Wind		
	Speed m/s	Dir., deg az.		Hs, m	Tp, sec	θp, deg az.	Hs, m	Tp, sec	θp, deg az.
85092717 85092716	24 32	345 35	5.2	2.48	9.0	94	1.97	4.8	33
85092716a	35	40	5.6	2.37	9.0	94	2.21	5.2	38
85092716b	35	45	6.0	2.44	10.0	94	2.09	6.1	49
76081004 76081005	16 18	40 10	2.6	1.32	6.0	92	0.83	3.5	10
72062218 72062219	22 26	25 20	3.5	1.64	7.0	92	1.44	4.2	19
71082810 71082811	14 23	85 80	1.9	1.09	6.0	92	1.73	5.5	84
67091620 67091608	7 9	40 65	2.2	1.10	8.0	91	0.52	3.8	70
60081301 60091219	4 21	20 20	3.1	1.33	10.0	95	1.18	3.9	19
55081312 55081313	17 18	110 110	4.5	1.78	10.0	95	1.20	5.0	102
54083121 54083113	2 19	80 5	2.5	1.17	10.0	91	1.00	3.6	5
54101600	20	115	2.7	1.48	8.0	94	1.27	5.2	105
44091500 44091501	20 23	45 25	2.8	1.42	5.0	91	1.32	4.1	23
38092119 38092120	22 28	30 10	2.9	1.05	6.0	92	1.61	4.4	10
38092122a 38092120a	20 34	270 25	4.3	1.86	8.0	95	2.00	4.9	25
38092118b 38092120b	18 23	40 5	2.1	0.94	5.0	91	1.26	4.0	5
36091906 36091902	12 13	355 75	3.9	1.55	10.0	95	0.96	4.4	80
35090818 35090814	12 13	50 85	3.5	1.57	9.0	94	0.98	4.3	86
33082403 33082323	17 15	120 110	4.4	1.73	10.0	95	0.99	4.6	101
33091709 33091705	13 13	55 70	4.4	1.75	11.0	95	0.93	4.4	78
29100303 29100306	20 24	135 170	3.9	1.59	8.0	95	1.23	3.9	171

[1] Wind based on WIS point at 40.5 deg N, 74 deg W.
[2] Incident wave height corresponding to max. significant wave height at P1 due to ocean waves.

Table 9. Extratropical Storm Peak Wave Conditions at Pit P2 from STWAVE Modeling

Date	Wind		Incident[1] Hs, m	Waves at P2, Ocean			Waves at P2, Local Wind		
	Speed m/s	Dir., deg az.		Hs, m	Tp, sec	θp, deg az.	Hs, m	Tp, sec	θp, deg az.
77110803 77110721	19 20	115 110	5.2	0.48	13.0	68	1.09	4.3	98
78010821 78011000	19 22	165 285	4.8	0.38	11.0	68	0.86	3.9	293
78012609 78012706	21 20	175 255	4.9	0.35	10.0	69	0.73	3.7	266
79012421 79012418	27 30	125 125	6.8	0.54	12.0	68	1.58	5.3	104
79032500 79032418	21 22	155 150	5.7	0.38	13.0	69	0.56	2.9	151
80102512 80102612	21 24	125 270	5.2	0.47	10.0	68	0.91	4.0	280
81020212	21	185	5.7	0.33	12.0	70	0.46	2.7	184
81021109 81021106	27 27	170 165	6.6	0.41	13.0	69	0.78	3.1	165
81033021 81032918	17 20	210 245	4.2	0.27	11.0	70	0.70	3.7	257
83121206	20	110	4.7	0.51	10.0	69	1.09	4.3	98
84032903 84032912	22 23	100 90	6.1	0.57	10.0	69	1.32	4.5	86
85021215	25	125 100	5.2	0.51	10.0	68	1.25	4.8	104
85092712 85092715	23 18 36	90 100 or 320 60	5.4	0.50	10.0	69	1.84	5.8	62
85110512 85110500	18 21	105 110	5.3	0.47	14.0	68	1.16	4.4	98
86031918 86031912	23 18 25	225 200	5.7	0.31	11.0	70	0.62	3.0	200
86120218	20	115	5.3	0.50	10.0	69	1.07	4.3	100
87033109 87033106	19 19	165 210 160	5.2	0.37	12.0	68	0.42	2.6	159
92121106	24 16	115 60	5.3	0.54	10.0	69	1.33	4.7	100
93112808	19 12	145 171	4.8	0.41	10.0	68	0.44	2.7	146
95111203 95111200	17 19 3	210 140 175 260	5.4	0.35	11.0	70	0.39	2.6	174

[1] Incident wave height corresponding to max. significant wave height at P2 due to ocean waves.

Table 10. Tropical Storm Peak Wave Conditions at Pit P2 from STWAVE Modeling

Date	Wind[1]		Incident[2] Hs, m	Waves at P2, Ocean			Waves at P2, Local Wind		
	Speed m/s	Dir., deg az.		Hs, m	Tp, sec	θp, deg az.	Hs, m	Tp, sec	θp, deg az.
85092717 85092716	24 32	345 35	5.2	0.89	9.0	71	1.52	4.3	33
85092716a	35	40	5.6	0.90	9.0	71	1.76	4.8	37
85092716b	35	45	8.0	0.87	10.0	71	2.18	6.0	60
76081004 76081005	16 18	40 10	2.6	0.53	6.0	71	0.67	3.1	10
72062219	25	20	4.1	0.65	7.0	71	1.08	3.7	19
71082810 71082811	14 23	85 80	1.9	0.35	6.0	68	1.25	4.6	77
67091608	9	65	1.6	0.37	5.0	71	0.36	3.1	65
60091219	21	20	3.2	0.54	7.0	71	0.87	3.4	19
55081312 55081313	17 18	110 110	4.5	0.50	10.0	69	0.96	4.1	98
54083113 54083111	19 15	5 45	2.9	0.41	6.0	72	0.80	4.0	60
54101600	20	115	2.7	0.47	6.0	69	1.07	4.3	100
44091500	20	45	2.8	0.58	6.0	71	1.14	4.6	50
38092119 38092121	22 29	30 340	2.9	0.45	6.0	71	1.21	3.9	341
38092120a	34	25	4.6	0.56	8.0	71	1.49	4.3	24
38092118b 38092120b	18 23	40 5	2.1	0.38	6.0	71	0.93	3.5	6
36091904	15	45	3.0	0.54	7.0	70	0.82	4.0	50
35090616 35090614	13 13	70 85	3.2	0.49	8.0	69	0.67	3.5	83
33082401 33082403	16 17	115 120	4.1	0.49	10.0	69	0.83	4.0	103
33091707 33091711	13 13	65 45	4.2	0.52	10.0	69	0.69	3.8	50
29100303 29100301	20 17	135 125	3.9	0.43	8.0	68	0.75	4.1	104

[1] Wind based on WIS point at 40.5 deg N, 74 deg W.
[2] Incident wave height corresponding to max. significant wave height at P2 due to ocean waves.

Table 11
Extratropical Storm Wave Frequency Analysis, Ocean Waves

Return Period, yr	Pit P1, Islands I1 & I2		Pit P2		Island I3	
	Max H_s, m	Std. Dev., m	Max H_s, m	Std. Dev., m	Max H_s, m	Std. Dev., m
5	1.91	0.02	0.52	0.01	6.49	0.11
10	1.99	0.02	0.56	0.01	6.99	0.15
25	2.07	0.03	0.59	0.01	7.51	0.17
50	2.12	0.04	0.61	0.01	7.84	0.22
100	2.18	0.05	0.63	0.01	8.19	0.29
125	2.20	0.05	0.63	0.01	8.32	0.30
150	2.21	0.05	0.64	0.02	8.40	0.33
175	2.22	0.06	0.64	0.02	8.47	0.36
200	2.23	0.06	0.64	0.02	8.51	0.38

Table 12
Tropical Storm Wave Frequency Analysis, Ocean Waves

Return Period, yr	Pit P1, Islands I1 & I2		Pit P2		Island I3	
	Max H_s, m	Std. Dev., m	Max H_s, m	Std. Dev., m	Max H_s, m	Std. Dev., m
10	1.24	0.32	0.42	0.10	2.95	0.82
25	1.79	0.13	0.59	0.05	4.57	0.43
50	2.08	0.20	0.71	0.10	5.37	0.46
100	2.48	0.35	0.88	0.15	6.10	0.60
125	2.65	0.41	0.95	0.16	6.38	0.67
150	2.77	0.47	1.00	0.18	6.57	0.78
175	2.86	0.53	1.03	0.20	6.70	0.87
200	2.92	0.57	1.06	0.21	6.80	0.94

Table 13
Tropical Storm Wave Frequency Analysis, Locally-Generated Waves

Return Period, yr	Pit P1, Islands I1 & I2		Pit P2	
	Max H_s, m	Std. Dev., m	Max H_s, m	Std. Dev., m
10	0.91	0.27	0.66	0.22
25	1.51	0.19	1.21	0.16
50	1.85	0.20	1.62	0.21
100	2.13	0.23	1.87	0.30
125	2.21	0.23	2.03	0.34
150	2.27	0.24	2.13	0.39
175	2.32	0.26	2.21	0.43
200	2.35	0.27	2.26	0.47

Information Supplied To Other Tasks

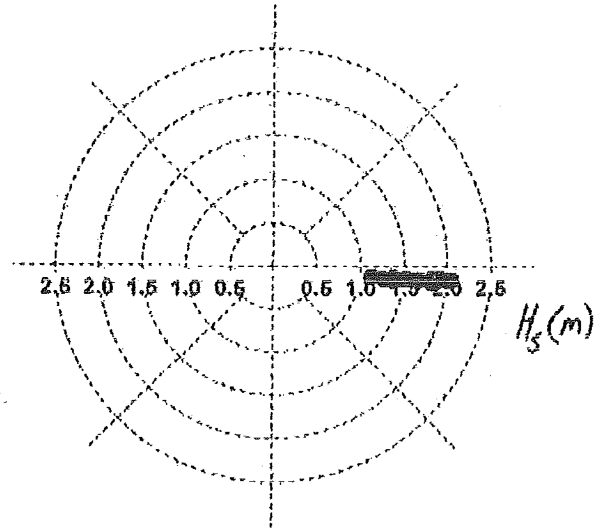
Sediment Transport Modeling

Two types of wave information at proposed CAD pit locations were supplied to the sediment modeling activities: storm waves, and a representative 1-yr time history of wave conditions. Storm wave information was provided for each storm as a time history file of significant wave height, peak period, and peak direction *(HTθ)* at selected STWAVE grid locations. The time history included all dates and times modeled with STWAVE, typically 3-hr intervals over a 3-day time period. Grid locations included the center of proposed pits P1 and P2.

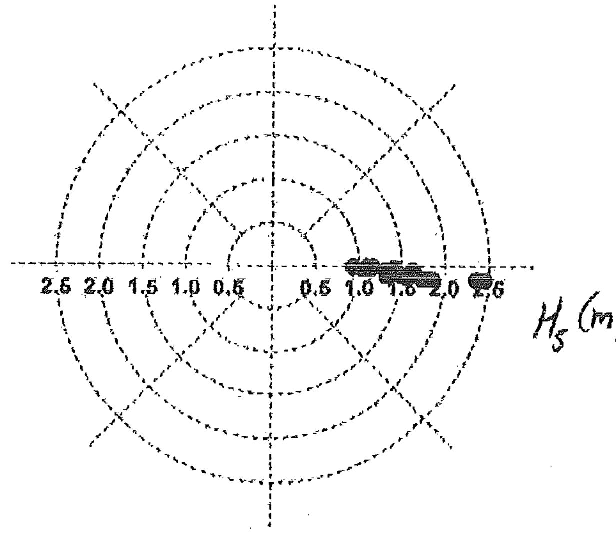
The 1-yr time history covered the period from Aug 96 through Jul 97. It included local and ocean waves with existing bathymetry. Hourly sea and swell *HTθ* information was provided at P1 and P2 center locations. Winds for local wave growth were taken from Ambrose Light, corrected to 10-m elevation, and incident ocean waves were taken from NDBC buoy 44025. Since the field data time histories had occasional short gaps, complete time histories were generated by interpolating across the gaps. The STWAVE wave field output files from climate runs described previously were used to create a table-lookup key relating local wind and incident ocean waves to local and ocean waves at pit locations. At each hour of the 1-yr time history, the appropriate keys were computed for wind and incident ocean waves and the table-lookup provided sea and swell at the pit location. If wind or incident ocean waves were from a direction outside those modeled, the sea or swell were assumed to be calm and zero values were assigned.

PIT P1 Peak Storm H_s + Direction
(see Tables 7+8)

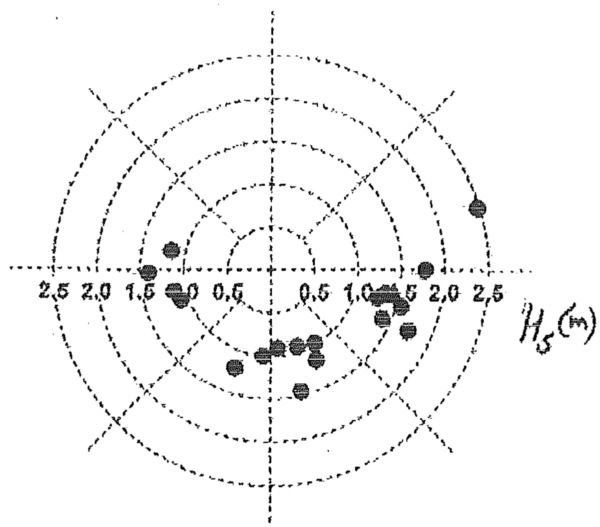
Northeaster - ocean



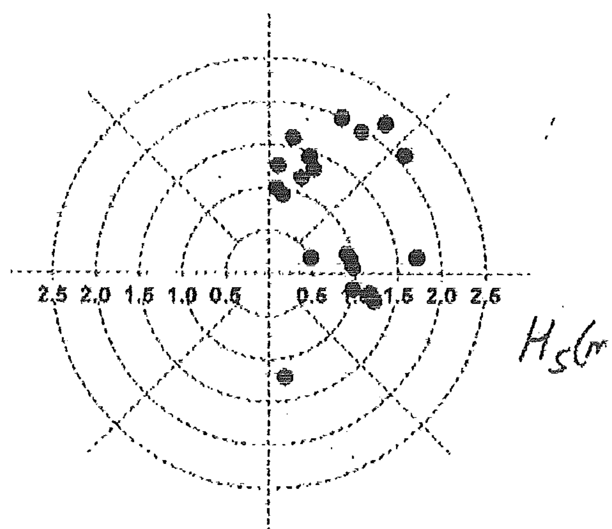
Hurricane - ocean



Northeaster - local

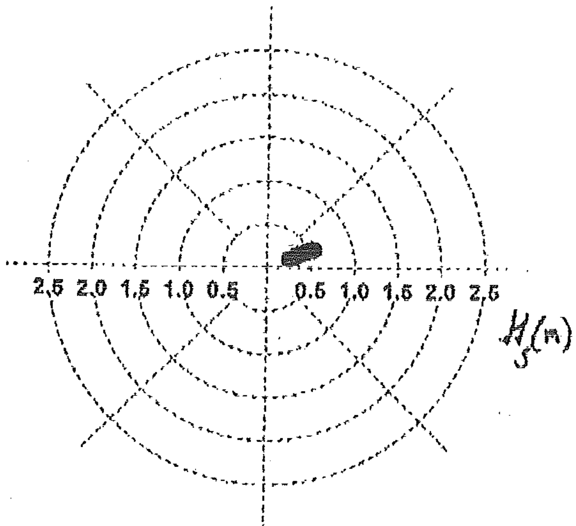


Hurricane - local

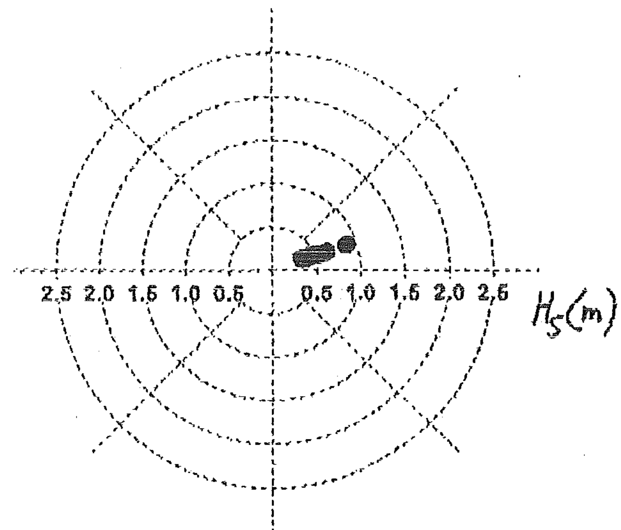


$\boxed{\text{PIT P2}}$ Peak Storm H_s + Direction (see Tables 9+10)

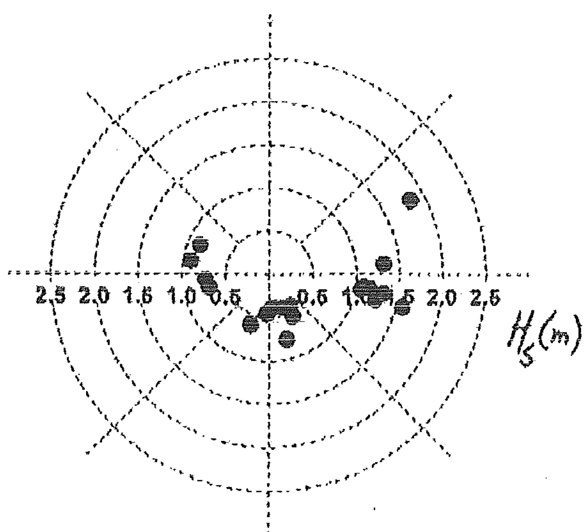
Northeaster-ocean



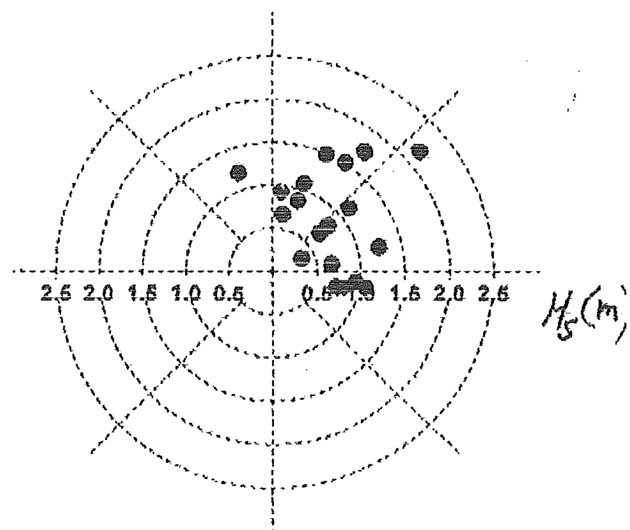
Hurricane-ocean



Northeaster-local



Hurricane-local



Training set of events

The advantage of the EST over the JPM is that the input/response vector space describes events which can or have occurred at the location of interest. This is assured by creating the vector space by numerically simulating a set of storm events which have impacted or could impact the area of interest. Each event of the training set must be a realistic event for the area, either a historic event or a hypothetical event based on a historic event with a slightly altered path or radius to maximum wind. Site specificity is then assured because the joint probabilities among the various input/response vectors reflects the joint probabilities inherent in parameters descriptive of actual events (or some slight variation thereof) which are site specific. The following sections describe the construction of the training set of storms.

Tropical Events. A tropical storm database (Scheffner et al. 1994) was generated during the DRP through simulation of 134 historically based storm events along the east coast, Gulf of Mexico, and Caribbean Sea via ADCIRC. The events were selected from the 1989 tabulation of the National Oceanic and Atmospheric Administration (NOAA) National Hurricane Center's database of tropical storm events (Jarvinen, Neumann, and Davis 1988). This data base (called the HURDAT data base) contains all hurricane, tropical storm and severe depression data which impacted the east coast, Gulf of Mexico, and Caribbean Sea from 1896 to present. This indexed database indicated that 16 tropical storm events impacted the study area during the 104-year period from 1886 to 1989.

Ideally, historical events represent the full range of possible event intensities. If this occurs, the historical events can be used directly to develop the full training set of storms. For extratropical events, this is generally the case because extratropical events occur often, cover extremely large areas, and persist for long periods of time (i.e. days). However, with tropical events this is often not the case. At most locations, the worst case tropical event scenario may be a historic event with a slightly shifted path or larger/smaller radius to maximum wind. Because the accuracy of the EST is dependent upon a full training set, some augmentation of the historic events is often necessary. This was found to be necessary for the FIMP reformulation study because station locations of interest spanned over 50 miles, from Fire Island Inlet to east of Shinnecock Inlet and typical radius to maximum winds are of this magnitude. Therefore, a storm of record in the eastern portion of the study area may not severely impact the western regions. An example follows.

Although severe events have occurred in the study area, i.e., the 1938 storm and Hurricane Gloria in 1985, they do not impact all stations equally. For example, the 1938 hurricane made landfall at the eastern side of Great South Bay, generating a 5.0-ft surge (without tide) in the vicinity of Fire Island Inlet. If, however, the 1938 event had made landfall 40 miles to the west, near Jones Inlet, the maximum storm surge near Fire Island Inlet would have been on the order of 11.0 ft. Therefore, although the 1938 event may represent a storm of record for some locations, it does not for other locations within the study area.

Table 5
Tropical Storm Events Impacting the Study Area

HURDAT Storm No.		Given Name	Date (mo/dy/yr)
1.	296	NOT NAMED	9/22/1929
2.	327	NOT NAMED	8/17/1933
3.	332	NOT NAMED	9/8/1933
4.	353	NOT NAMED	8/29/1935
5.	370	NOT NAMED	9/8/1936
6.	386	NOT NAMED	9/10/1938
7.	436	NOT NAMED	9/9/1944
8.	535	CAROL	8/25/1954
9.	541	HAZEL	10/5/1954
10.	545	CONNIE	8/3/1955
11.	597	DONNA	8/29/1960
12.	657	DORIA	9/8/1967
13.	702	DORIA	8/20/1971
14.	712	AGNES	6/14/1972
15.	748	BELLE	8/6/1976
16.	835	GLORIA	9/16/1985
17.	386a	1938_a	-
18.	386b	1938_b	-
19.	835a	GLORIA_a	-
20.	835b	GLORIA_b	-

In order to supplement the training set so that all stations within the study experience a (hopefully) maximum intensity event, and thereby fill the vector space with events ranging from nominal to intense, four additional storm events were added to the initial training set. These events were developed as perturbations of the two most intense events of record, the 1938 event and Hurricane Gloria. The hypothetical events were created as follows:. The 1938 event was assumed to (a) make landfall 0.6 deg longitude (40 miles) to the west and (b) 0.6 deg to the east. Hurricane Gloria had a reported radius to maximum winds of approximately 27 statute miles at landfall (Jarvinen and Gebert 1985). Two perturbations of this event were specified, one with a radius to maximum winds of 35 miles and one with a radius of 50 miles. These four combinations produced maximum surges throughout the study area. The final training set consisted of 20 events, 16 historical, and four representing perturbations of the two most severe events of record. The full training set is shown in Table 5.

Results obtained for the FIMP study were considered very good, therefore, the identical training set of events were used for the present analysis.

The HURDAT storm number designation in Table 5 refers to the storm identification number of the events in the National Hurricane Center data base of historic tropical events.

Extratropical Events. Extratropical storms to be modeled were selected using peak wave height as the criterion for selection. Time series of waves hindcast by the Wave Information Study (WIS) at Phase II Station #76 (40.50 N, 73.0 W) were examined for the years 1976-95. This particular station offshore of the entrance to the NY Harbor was selected because it is representative of the project area and did not reflect shallow-water transformation effects (water depth at the WIS station is approximately 30m). A computer program was used to screen the time series for the 20-year period, select the top 250 wave heights and order them by rank. For each of the selected events, the wave heights preceding and following the event were examined to ensure the event represented a true storm. This also prevented duplicate simulation of the same event. Out of these storms, the top twenty (shown in Table 6) were selected for simulation with the ADCIRC model.

Storm Parameterization

Once the training set of tropical and extratropical storms have been selected from historical and historically based events, each event must be parameterized to quantify the input vectors describing the physical attributes of the event with respect to each location, or station, of interest and the response of each event. The following sections describe input and response vector space creation.

Input vectors. The EST requires specifying a set of parameters which describe the dynamics of some physical system - tropical and extratropical storms in this case. These parameters, which must be descriptive of the process being modeled, are defined as a N-dimensional vector space.

$$\underline{\nu} = (\nu_1 , \nu_2 , \nu_3 , \ldots\ldots , \nu_N)$$

Input vectors are defined separately for tropical and extratropical events due to basic differences in storm type characteristics. The following subsections define the input vectors defined for each storm type.

Tropical events can be reasonable well parameterized according to several criteria. For the present study, the following five input vectors were defined for each event at each station:

TABLE 6
Extratropical Storms

No.	Storm No.	Simulation Start, GMT		Nominal Duration (days)
		(mm/dd/yy)	(hr/min)	
1	1177(11/04/77)	11/04/77	1200	6.0
2	0178_A	01/23/78	0000	6.0
3	0178_B	01/06/78	0000	6.0
4	0179	01/22/79	0000	6.0
5	0379	03/20/79	0000	6.0
6	1080	10/22/80	1200	6.0
7	0281_A	02/08/81	0000	6.0
8	0281_B	01/30/81	0000	6.0
9	0381	03/27/81	0000	6.0
10	1283	12/10/83	0000	8.0
11	0384	03/26/84	0000	6.0
12	0285	02/10/85	0000	6.0
13	0985	09/24/85	1200	6.0
14	1185	11/02/85	0000	6.0
15	0386	03/19/86	1200	6.0
16	1286	11/30/86	0000	6.0
17	0387	03/28/87	0000	6.0
18	1292	12/08/92	1200	6.0
19	1193	11/25/93	0000	6.0
20	1195	11/09/95	0000	6.0

Note 4 digit # for easy id. First two digits denote the month and the next two the year

(a) Tidal phase during the event, with 1.0 corresponding to high water slack, 0.0 for Mean Sea Level (MSL) at maximum ebb, -1.0 low water slack, and 0.0 MSL at maximum flood. Because the study area is characterized by semi-diurnal tides, a representative tide was selected as the Root Mean Square (RMS) value (0.707) of a single sinusoidal tide represented as the sum of the amplitudes of the five primary tidal constituents.

(b) Minimum distance from the eye of the storm to the location of interest in statute miles.

(c) The central pressure deficit of the hurricane eye at the minimum distance, in mb.

(d) Maximum winds in the hurricane at the minimum distance, measured in knots.

(e) Forward speed of the eye of the hurricane at the minimum distance, measured in statute miles per hour.

Extratropical event events are difficult to parameterize due to their large temporal and spatial scales. Therefore, a 1-D formulation of the EST was used in which input vectors were not specifically defined. However, response vectors were defined corresponding to some storm response under spring, mean, and neap astronomical tide conditions.

Response vectors. The second class of vectors involve some selected response resulting from the input-vector parameterized storm, i.e.,

$$\underline{r} = (r_1 \ , r_2 \ , r_3 \ , \ ...... \ , r_M)$$

For the DMMP study,.the following responses: storm-induced water level, wave height, and vertical erosion within a pit, are computed via the ADCIRC, STWAVE, and LTFATE models, respectively.

EST Implementation

Although response vectors are related to input vectors

$$v \Rightarrow r \ ,$$

the interrelationship is highly nonlinear and involves correlation relationships which can not be directly defined, i.e., a non-parametric relationship. For example, in addition to the storm input parameters, storm surge is a function of local bathymetry, shoreline position and curvature, ocean currents, temperature, etc. as well as their spatial and temporal gradients. It is assumed that these combined effects are reflected in the response vector.

The historical data for storms can thus be characterized as

$$[v_i \ ; \ i = 1, \ ... \ , I]$$

where I is the number of historical storm events. For example let v_i have d_v-components

$$v_i \in \Re^{d_v}$$

where $\Re^{dv}$ denotes a d_v-dimensional space.

We will now introduce a separate set of storm events

$$[\ \nu_j^*\ ,j = 1,\ \dots\ ,J\ ]$$

which we call the training set that will be used as input for appropriate numerical models which compute the desired response vectors. The set of v^*_j usually includes the historical events but may include storms which could have occurred. For example, a historical storm with a slightly altered path.

Once the training set has been defined with each event/station represented by an appropriate input/response vector, life cycle simulations via the EST can be generated. The goal of the EST can be summarized as:

(a) Given the following:

(1) The historical data $[\mathbf{v}_i \in \Re^{dv}; i=1,\dots,I]$.

(2) The "training set" data $[\mathbf{v}^*_j \in \Re^{dv}; j=1,\dots,J]$.

(3) The response vectors calculated from the training set $[\mathbf{r}^*_j \in \Re^{dr}; j=1,\dots,J]$

(b) Produce N simulations of a T-year sequence of events, each with their associated input vectors $\mathbf{v} \in \Re^{dv}$ and response vectors $\mathbf{r} \in \Re^{dr}$.

Two criteria are required of the T-year sequence of events. The first is that the individual events must be similar in behavior and magnitude to historical events, i.e., the inter-relationships among the input and response vectors must be realistic. The second criteria is that the frequency of storm events in the future will remain the same as in the past. The following sections describe how these two criteria are preserved.

Storm event consistency. The first major assumption in the EST is that future events will be similar to past events. This criteria is maintained by insuring that the input vectors for simulated events have similar joint probabilities to those of the training set. For example, a hurricane with a large central pressure deficit and low maximum winds is not a realistic event - the two parameters are not independent although their precise dependency is unknown. The simulation of realistic events is accounted for in the nearest-neighbor interpolation, bootstrap, resampling technique developed by Borgman (Borgman, et al. 1992).

The basic technique can be described in two dimensions as follows. Let X_1, X_2, X_3, ..., X_n be n independent, identically distributed random vectors (storm events), each having two components $[X_i = \{\underline{x}_i(1), \underline{x}_i(2)\}; I=1,n]$.

Each event X_i has a probability p_i as $1/n$, therefore, a cumulative probability relationship can be developed in which each storm event is assigned a segment of the total probability of 0.0 to 1.0. If each event has an equal probability, then each event is assigned a segment s_j such that $s_j \dashrightarrow X_j$. Therefore each event occupies a fixed portion of the 0.0 to 1.0 probability space according to the total number of events in the training set.

$$[\ 0 < s_1 \leq \frac{1}{n}\ ]$$

$$[\ \frac{1}{n} < s_2 \leq \frac{2}{n}\ ]$$

$$[\ \frac{2}{n} < s_3 \leq \frac{3}{n}\ ]$$

$$\vdots$$

$$[\ \frac{n-1}{n} < s_n \leq .\ 1]$$

A random number from 0 to 1 is selected to identify a storm event from the total storm population. The procedure is equivalent to drawing and replacing random samples from the full storm event population.

The EST is not simply a resampling of historical events technique, but rather an approach intended to simulate the vector distribution contained in the training set data base population. The EST approach is to select a sample storm based on a random number selection from 0 to 1 and then perform a random walk from the event X_i with x_1 and x_2 response vectors to the nearest neighbor vectors. The walk is based on independent uniform random numbers on (-1,1) and has the effect of simulating responses which are not identical to the historical events but are similar to events which have historically occurred.

Storm event frequency. The second criteria to be satisfied is that the total number of storm events selected per year must be statistically similar to the number of historical events which have occurred at the area of concern. Given the mean frequency of storm events for a particular region, a Poisson distribution is used to determine the average number of expected events in a given year. For example, the Poisson distribution can be written in the following form:

$$Pr(s;\lambda) = \frac{\lambda^s e^{-\lambda}}{s!}$$

for s=0,1,2,3,... The probability Pr(s;λ) defines the probability of having s events per year where λ is a measure of the historically-based number of events per year.

In the DMMP study, historical data were used to define λ as follows:

Tropical events: $\lambda = 0.15385$ (16 events/104 years) 20 yrs of data
Extratropical events: $\lambda = 0.742857$ (26 events/35 years)
I thought there were 20?

A 10,000 element array is initialized to the above Poisson distribution. For example, the number corresponding to $\lambda = 0.32$ and for s=0 storms per year is 0.7261. Thus if a random number selection is less than or equal to 0.7261 on an interval of 0.0 to 1.0, then no hurricanes would occur during that year of simulation. If the random number is between 0.7261 and 0.7261 + P[N=1] = 0.7261 + 0.2324 = 0.9585, one event is selected. Two events for 0.9585 + 0.0372 = 0.9957, etc. When one or more storms are indicated for a given year, they are randomly selected from the nearest neighbor interpolation technique described above.

The storm events of Tables 5 and 6 represent the range of intensities of tropical and extratropical events which impact the study area. In the following section, the approach adopted for using these storms to develop frequency-of-occurrence relationships is given.

EST frequency computation

The set of input/response vectors described in the previous sections are input to the EST to generate 100 separate simulations of 200 years of storm event activity for each of the disposal alternative locations being considered. Because tropical and extratropical events can be considered independent, EST life cycle simulations are performed separately for each storm type. Estimates of frequency-of-occurrence for both tropical and extratropical storms requires post-processing of each of the 100 simulations.

These computations begin with calculating a cumulative distribution function (cdf) for the response vector of interest, for example, the maximum storm-induced water level. Let $X_1, X_2, X_3, \ldots, X_n$ be n identically distributed random response variables with a cumulative cdf

$$F_X(x) = Pr[X \leq x]$$

where Pr[ ] represents the probability that the random variable X is less than or equal to some value x and $F_X(x)$ is the cumulative probability distribution function ranging from 0.0 to 1.0. The problem is to estimate the value of F_X without introducing some parametric relationship for probability. The following procedure is adopted because it makes use of the probability laws defined by the data and does not incorporate any prior assumptions concerning the probability relationship.

Assume that we have a set of n observations of data. The n values of x are first ranked in order of increasing size such that, where the parentheses surrounding the subscript indicates that the data have been rank-ordered. The value $x_{(1)}$ is the smallest in the series and $x_{(n)}$ represents largest. Let r denote the rank of the value $x_{(r)}$ such that rank 1 is the smallest and rank $r = n$ is the largest.

An empirical estimate of $F_X(x_{(r)})$, denoted by $\hat{F}_X(x_{(r)})$, is given by Gumbel (1954) (see also Borgman and Scheffner, 1991 or Scheffner and Borgman 1992).

$$\hat{F}_x(x_{(r)}) = \frac{r}{(n+1)} \tag{5}$$

for $\{x_{(r)}, r = 1, 2, 3, \ldots, n\}$. This form of estimate allows for future values of x to be less than the smallest observation $x_{(1)}$ with probability of $1/(n+1)$, and to be larger than the largest value $x_{(n)}$ also with probability $1/(n+1)$. In the implementation of the EST, tail functions (Borgman and Scheffner 1991) are used to define the cdf for events larger than the largest or smaller than the smallest observed event so that there is no discontinuity in the cdf.

The cdf, as defined by Equation 5, is used to develop stage-frequency relationships in the following manner. Consider that the cdf for some storm impact corresponding to an n-year return period event can be determined from:

$$F(x) = 1 - \frac{1}{n} \tag{6}$$

where F(x) is the simulated cdf of the n-year impact. Frequency-of occurrence relationships are obtained by linearly interpolating a stage from Equation 5 corresponding to the cfd associated with the return period specified in Equation 6.

For each return period year, a standard deviation, defined as:

$$\sigma = \sqrt{\left[(1/N)\sum_{n=1}^{n=N}(x_n - \bar{x})\right]}$$

(where bar x is the mean value), is computed to define an error band of +/- one standard deviation corresponding to each mean value curve. The standard deviation is a measure of the error bands associated with each mean value, and is a measure of the uncertainty associated with the mean value.

Development of frequency-of-occurrence relationships for storm-induced water level (surge plus tide) are presented in this chapter. Results for waves and vertical erosion are presented in Chapters 2 and 6, respectively.

Results

Potential sites for CAD pits were identified at two locations (P1 and P2) based on the results of a GIS-based site screening process. Potential island CDF sites were defined at two locations, one of which was the location of P1. Island CDFs of several sizes were considered at location P1 (these are designated as islands I1 and I2). The other potential island CDF site is located offshore, and is designated as I3. Locations of these sites are listed in Table 7 and shown in Figure 5. where,

Figure 5. Locations for potential disposal sites

Table 7 Disposal Alternative Locations		
Pit Site	East Longitude, deg	North Latitude, deg
P2	-74.16792754	40.47429751
P1/I1/I2	-74.08263438	40.50270556
I3	-73.84213115	40.46502782

The response vectors (in this case peak water level) computed from ADCIRC simulations of the entire set of tropical and extratropical storms were used along with the EST procedures to compute frequencies of occurrence for each response vector at the desired locations. Table 8 summarizes water level results for sites P1/I1/I2, P2, and I3, for both tropical and extratropical storms.

Table 8
Total Elevation Frequency Analysis

Return Period yrs	P2 Trop/extrop surge ft,msl	P1/I1/I2 Trop/extrop surge ft,msl	I3 Trop/extrop surge ft,msl
5	0.00/7.10	0.00/6.70	0.00/5.47
10	1.79/8.19	1.63/7.78	1.33/6.44
25	5.26/9.39	4.97/8.81	4.65/7.29
50	7.35/10.21	6.92/9.64	7.16/7.71
100	9.49/~~11.08~~	8.98/~~10.50~~	9.17/8.02
200	11.55/~~11.91~~	11.02/~~11.32~~	11.19/8.28

MTL !!

~~add 0.76~~ add 0.76

USC NGVD

check [illegible] 5/[illegible]

per [illegible] cons.

← these seem avg 2' lower than 1995 F.MP Sta 44.

is it expected that extrat → trop @ higher [illegible]?

For the offshore island site, extratropical storms are the primary factor determining extreme water levels at return periods of up to about 10-20 years. At that return interval and beyond, tropical storms become increasing more important as a cause of extreme water levels. Also note that the data base used to develop the training set of extratropical storms represents a duration of about 20 years. The assumptions inherent in the EST approach concerning statistical similarity of past events to future events, suggest that caution be used in interpreting the results for extratropical storms at return periods greater than 50 years. Since the tropical storms included in the training set reflect a much longer period of time, over 100 years, estimates of extreme water levels associated with tropical storms and having return periods of 100 and 200 years can be made with greater confidence. The same