North Shore of Long Island, Asharoken, New York Coastal Storm Risk Management Feasibility Study

Appendix A

Engineering

November 2015

APPENDIX A
ENGINEERING AND DESIGN

Beach Erosion Control and Storm Damage Reduction Feasibility
NORTH SHORE OF LONG ISLAND, ASHAROKEN, NEW YORK

1.0 INTRODUCTION

Location The Village of Asharoken is located in Suffolk County, NY within the Town of Huntington. The study area is located on the north shore of Long Island between Eaton's Neck Point to the west and National Grid (former LILCO) Power Plant to the east on Long Island Sound. The study location is shown on Figure A-1.

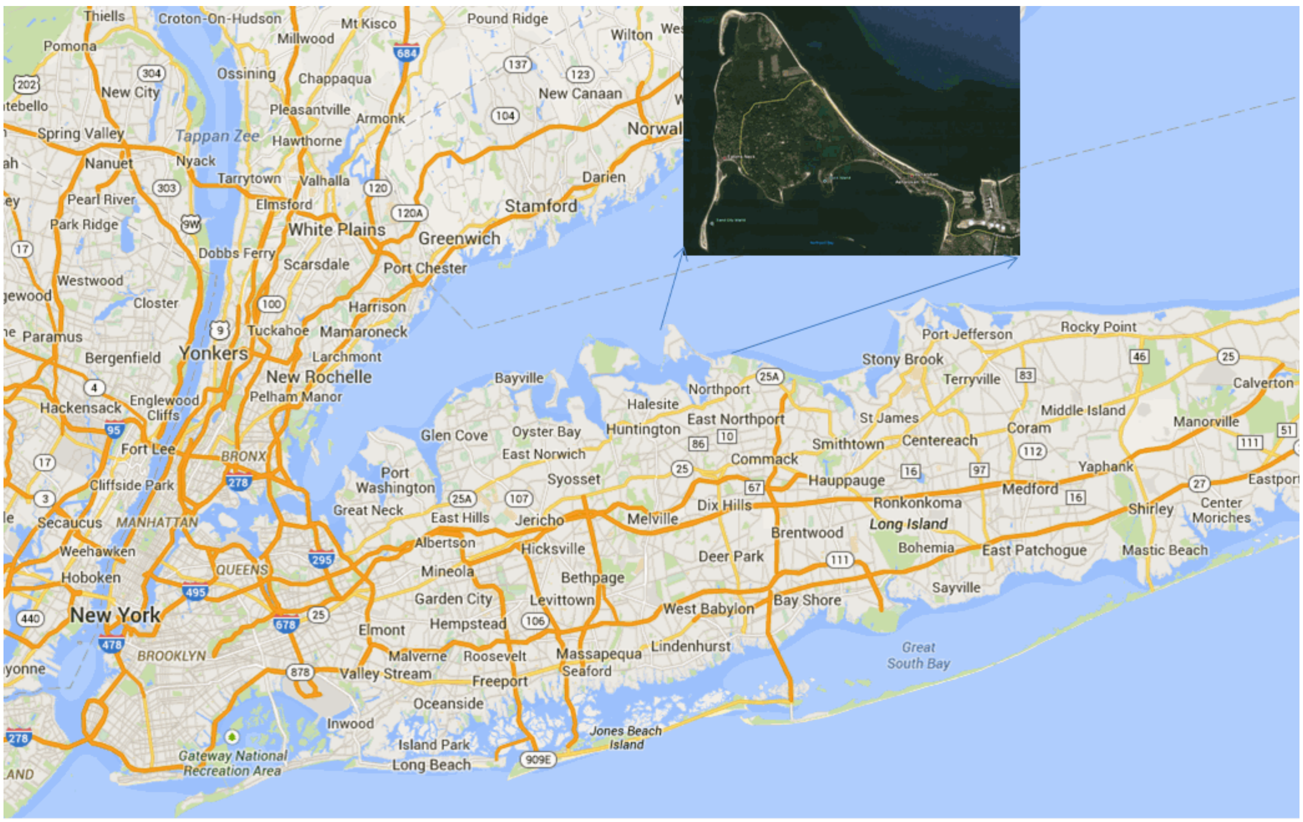


Figure A-1 Study Location

1.1 Study Area Description Asharoken Beach is a narrow section of land in the Town of Huntington on the north shore of Long Island, which connects Eaton's Neck and the Village of Asharoken with the Village of Northport. The length of Asharoken Beach is approximately 2.4 miles and the width varies from 100 feet at the northwestern section to 50 feet at the southeastern limit near the National Grid (formerly Long Island Lighting Company) power plant. The roadway along Asharoken Beach (Asharoken Avenue) provides the only access to Eaton's Neck, a community of approximately 1,400 residents based on 2010 census.

1.2 Project Site History In the late 18th century, a shoal began to form between Long Island and Eaton's Neck Island, gradually becoming navigable at high tide only. As a result of longshore transport from the east, accretion of the shoal continued, eventually joining Eaton's Neck with mainland Long Island (Figure A-2).

In the 1960's the former Long Island Lighting Company (LILCO) constructed the Northport power plant adjacent to the Northport basin. Fuel oil for the power plant is delivered by 50,000 DWT tankers to a platform located approximately 2 miles offshore and piped to the plant. Tugs and other vessels which service the platform are moored in Northport basin. The basin also provides a Town boat ramp which is frequented in the summer months by local residents.

As part of sand mining operations, two jetties were constructed and the Northport Basin Inlet channel dredged between them in early 1930's. As part of LILCO's plant construction, the existing barge jetty located east of the Northport Basin inlet was rehabilitated into a quarrystone and concrete riprap jetty and revetment. According to local officials the shoreline east and west of the basin originally exhibited no offset. However, since the jetty construction the west shoreline (Asharoken Beach) has experienced erosion.

During past storm events, municipal and residential structures have suffered minor damage. However, flooding of Asharoken Avenue has occurred at the northwestern portion of the study area. Additionally, during past storms the northwestern portion of the study area has experienced wave attacks which have caused overtopping of the dune system. This overtopping has deposited sand and debris and has created ponding of water on Asharoken Avenue causing the road to be impassible for more than 24 hours. Asharoken Avenue is the only access to the mainland for the approximately 1,500 residents of Eaton's Neck. Since the shoreline continues eroding, the roadway and properties will be subject to increasing storm damage without additional shore protection measures.

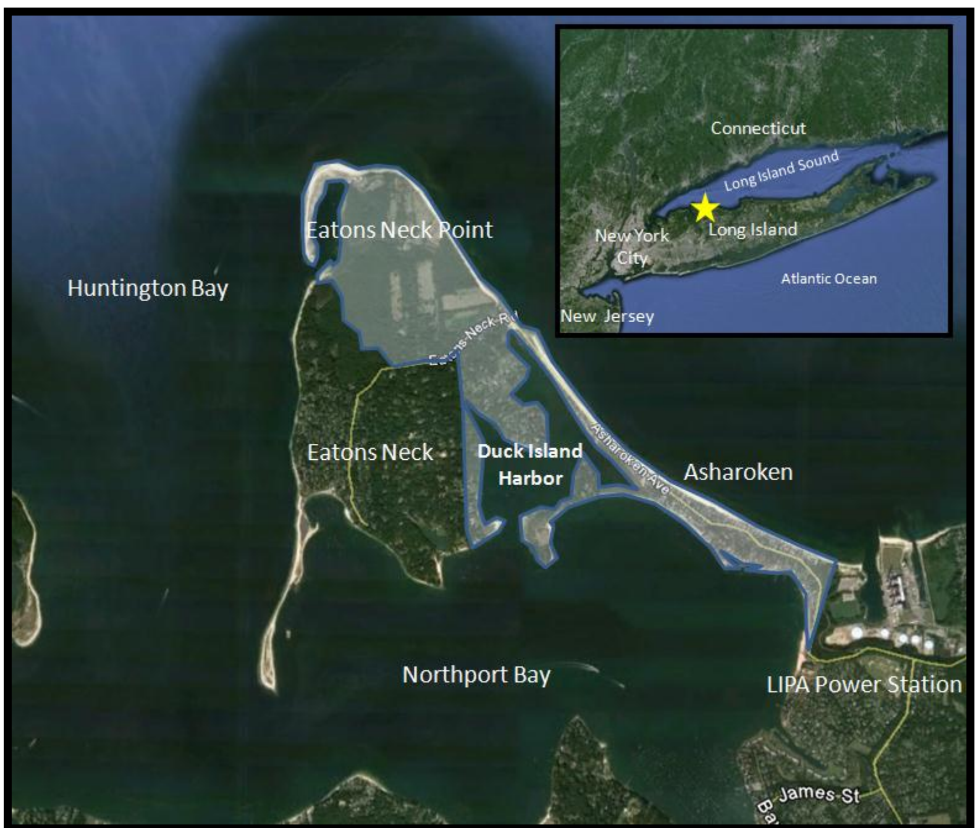


Figure A-2 Project Site

1.3 Problem Identification

a. Northwestern Critical Shoreline

Review of available aerial photography (Photo 1) strongly suggests that the existing stone groin located at the southeast end of the existing seawall (intersection of Asharoken Ave. & Bevin Rd., groin installed sometime between 1947 and 1962) is acting as a terminal structure which is trapping a significant quantity of longshore material and essentially holding the beach to the southeast in place in its current configuration. This beach protects a long section of the roadway. The removal or loss of functionality of this existing stone groin would likely result in the realignment of the long section of the updrift beach with an associated loss of width and elevation leading to a reduction in the level of protection the feature provides to the road.

In addition to the existing stone (terminal) groin, there are three non-engineered groins constructed to the northwest (downdrift) of the stone groin. Details of the three groins (identified as Groin #1, #2 and #3 in the 1994 aerial photo) design, material, date of construction are unknown, however, they are likely composed of concrete cube armor blocks during the same time period with the stone groin. Although partially damaged, the three groins still maintain a basic downdrift shoreline with dune and beach until damaged by the 1996 northeaster and several storms that follows. An emergency repair of the shoreline with steel sheetpile wall and rock toe protection was installed in 1996 – 1997.

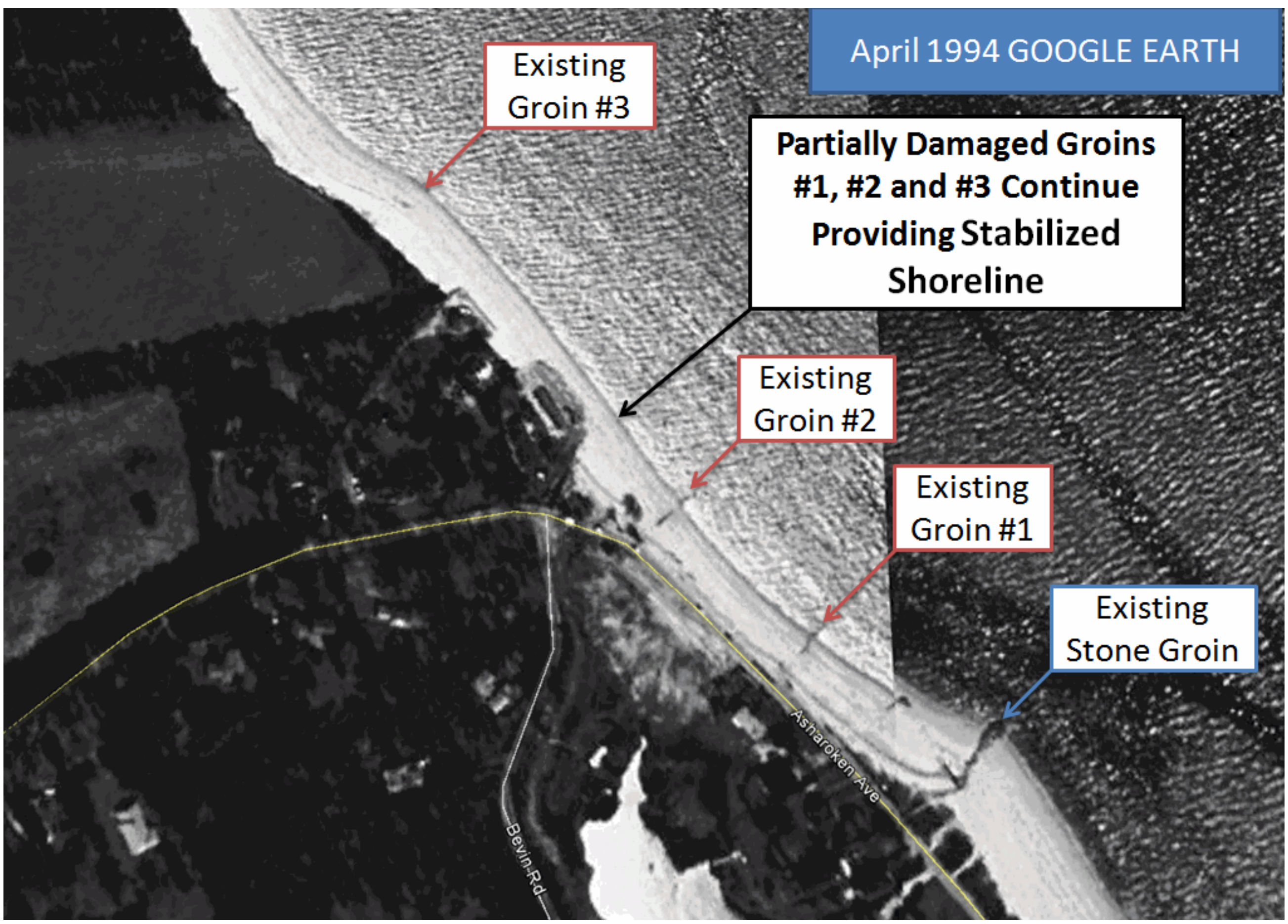


Figure A-3: 1994 Aerial Photo - Northwestern Critical Shoreline with Existing Groins

Since the storm damages of the three groins, the downdrift (northwest) side of the beach has been largely eroded away, due to a combination of the structure's shadow effect and wave induced scour of the exposed, northeast-facing shoreline without groins holding shoreline. The seawall has since been repaired three times in 2007, 2010, and 2012, all due to eroded shoreline resulting wave overtopping at the toe, as illustrated in the 2010 aerial photo after the 2009 northeaster "IDA".

Figure A-4: 2010 Aerial Photo – Post Northeaster "IDA" with Seawall Breach and Shoreline Erosion

The approximately 900 foot section of Asharoken Avenue extending from the existing stone groin northwest to where it starts uphill onto Eatons Neck is the only portion of the roadway which is really threatened by erosion. A seawall has been installed by the USACE to protect the road at this location. The three Sound front residential properties immediately northwest of the seawall have also installed stone riprap or revetments to protect their shorelines sometime during the 1990s. The ongoing erosion and loss of beach seaward of the wall since its installation has necessitated the addition of successively larger and heavier stone armor to protect the toe over the last 15 years or so.

The bathymetry of the beach and near shore area seaward of this wall has changed since its original installation. There was a beach seaward of the roadway when the seawall was first installed. Erosion steadily reduced the width and elevation of this beach until today the armor stone is at the water line at high tide over much of the structure's length. Under current conditions, waves break on the stone armor during some storm events, which sends large volumes of spray across the roadway and creates very dangerous conditions for motorists. Asharoken Avenue is the only road onto and off of Eatons Neck. In severe events the road has to be closed and vehicular access to Eatons Neck cut off due to unsafe driving conditions associated with heavy wave spray, standing water or deposited sand.

The reestablishment of a beach in front of the seawall would alleviate some of these impacts by rebuilding the natural protective feature of the beach and essentially pushing the place where storm waves break seaward, thereby absorbing / expending some of the energy before it gets to the wall and reducing the amount of spray and water which gets landward of the wall.

b. Southeastern Critical Shoreline

The littoral cell which encompasses the Asharoken shoreline has been substantially modified by people since at least the construction of the power plant in the early 1960s. The plant's jetty-stabilized intake lagoon and outfall lagoon have been demonstrated as interrupting the longshore movement of sand into the area seaward of Asharoken Avenue. This situation combined with the vulnerable northeast exposure of Asharoken Beach has resulted in episodes of erosion which have been significant enough to cause the majority of the Sound front property owners along the eastern half of the reach to install seawalls, bulkheads and similar hard shoreline stabilization structures, many of which were constructed in very seaward locations as illustrated in the 2014 aerial photo below. These shoreline stabilization structures have probably contributed to the accelerated rate of erosion of the fronting beach, just as a deepened area of Long Island Sound bottom located immediately off the subject beach from a previous dredging project to obtain renourishment sand probably is.

Figure A-5: 2014 Aerial Photo Showing Seaward Offset of Existing Bulkhead from Natural Shoreline (shown as orange line) at Southeast Critical Shoreline

2.0. COASTAL PROCESS

2.1 Historical Storms

Two types of storms are of primary significance along the North Shore: a) tropical storms, which typically impact the New York area from July to October, and b) extra tropical storms, which are primarily winter storms. Extra tropical storms (northeasters) are usually less intense than hurricanes; however, tend to have much longer durations. These storms often cause high water levels and intense wave conditions, and are responsible for significant damages and flooding throughout the coastal region of the project area. USACE (1969) states that 65 moderate to severe northeasters have impacted the New York coastal region over the 100-year period preceding 1965. More recently, a series of severe northeasters has impacted the New York coastal region in October 1991, December 1992, and March 1993. Table A-1 lists several severe hurricanes extra tropical storms that have had significant impacts in the New York area.

TABLE A-1 Historical Storms Impacting New York Area

Hurricane		Northeaster	
Date	Name	Date	Name
14 Sep 1904	-	03 Mar 1931	-
08 Sep 1934	-	17 Nov 1935	-
21 Sep 1938	-	25 Nov 1950	-
14 Sep 1944	-	06 Nov 1953	-
31 Aug 1954	Carol	11 Oct 1955	-
02 Sep 1954	Edna	25 Sep 1956	-
05 Oct 1954	Hazel	06 Mar 1962	-
03 Aug 1955	Connie	05 Nov 1977	-
12 Sep 1960	Donna	17 Jan 1978	-
10 Sep 1961	Esther	06 Feb 1978	-
20 Aug 1971	Doria	22 Jan 1979	-
14 Jun 1972	Agnes	22 Oct 1980	-
06 Aug 1976	Belle	28 Mar 1984	-
27 Sep 1985	Gloria	09 Feb 1985	-
19 Aug 1991	Bob	30 Oct 1991	-
08 Oct 1996	Josephine	01 Jan 1992	-
07 Sep 1999	Floyd	11 Dec 1992	-
01 Sep 2006	Ernesto	02 Mar 1993	-
28 Aug 2011	Irene	12 Mar 1993	-
20 Oct 2012	Sandy	28 Feb 1994	-
		21 Dec 1994	-
		05 Jan 1996	-
		06 Oct 1996	-
		02 Feb 1998	-
		14 Apr 2007	-
		15 Nov 2009	Nor'Ida
		13 Mar 2010	-
		17 Apr 2011	-

Notes:
1. Northeasters have no assigned names;
2. Hurricane Sandy affected the project area in late October, 2012;
3. This table lists only significant storms affecting the project area.

Damages incurred by hurricanes and northeasters are highly dependent on storm intensity and duration, however, the relative location of a storm with respect to the north shore of Long Island is particularly important for hurricanes. This dependency on storm location is linked to storm characteristics, which determine where, relative to the storm movement, the most severe conditions exist. Tropical cyclones are characterized as small, fast moving storms consisting of a counter-clockwise spiral about the center of the storm. Winds to the right of the eye are typically most severe because they are reinforced by the forward motion of the storm itself. This forward storm speed can exceed 20 mph among hurricanes in the North Atlantic. Therefore, consider hurricanes travel in a general northerly direction; the south-facing coastlines are usually subjected to the largest hurricane forces. On the other hand, north-facing coastlines are somewhat protected from the strongest hurricane impacts.

Extra tropical storms are similarly characterized by counter-clockwise spiral directed toward a central low-pressure center. The radius of rotation, however, is orders of magnitude greater than the hurricane. Wind direction and velocity at a given coastal location depend on the relative location of the storm track. Most critical to the north shore of Long Island is passage of a northeaster to the east of the Sound, where winds blow initially from the northeast. Wind direction eventually changes with storm movement, to the north/northeast. This storm scenario produces large waves and wind setup along the north shore. Historically, northeasters with northeasterly winds occurring through numerous tidal cycles have caused the worst damages along the study area.

2.2 Wind Condition

Wind data were recorded based on observations by the U.S. Weather Bureau at four weather stations vicinity of the project site. The four stations are:

- LaGuardia Airport, Long Island, New York;
- Bridgeport Airport, Connecticut;
- Avery Point, Connecticut;
- Groton, Connecticut

Locations of the four stations are shown in Figure A-6. Historical wind speed-direction readings were obtained for the period 1973 to 2001, approximately 30 years of record. In addition to the measured wind records, a file of continuous daily wind fields was available for the decade of the 1990's (1990 to 1999). This wind file includes all available historical marine surface data from buoys, ships, coastal stations, and scatterometer data, all adjusted to effective neutral 10-meter wind speeds. The wind data were validated using buoys 44025 and the CODAS data set. Quantile-quantile and exceedence plot comparisons of wind speed showed excellent agreement between the hindcasted winds and the measurements at NOAA Buoys 44025 and 44028 located off the south shore of Long Island and the south shore of Rhode Island Figure A-6. The hindcast wind field data was used for daily wave predictions and used as the basis for sediment transport estimates since this set of data reflects more recent weather conditions and is representative of over-water conditions at the project site.

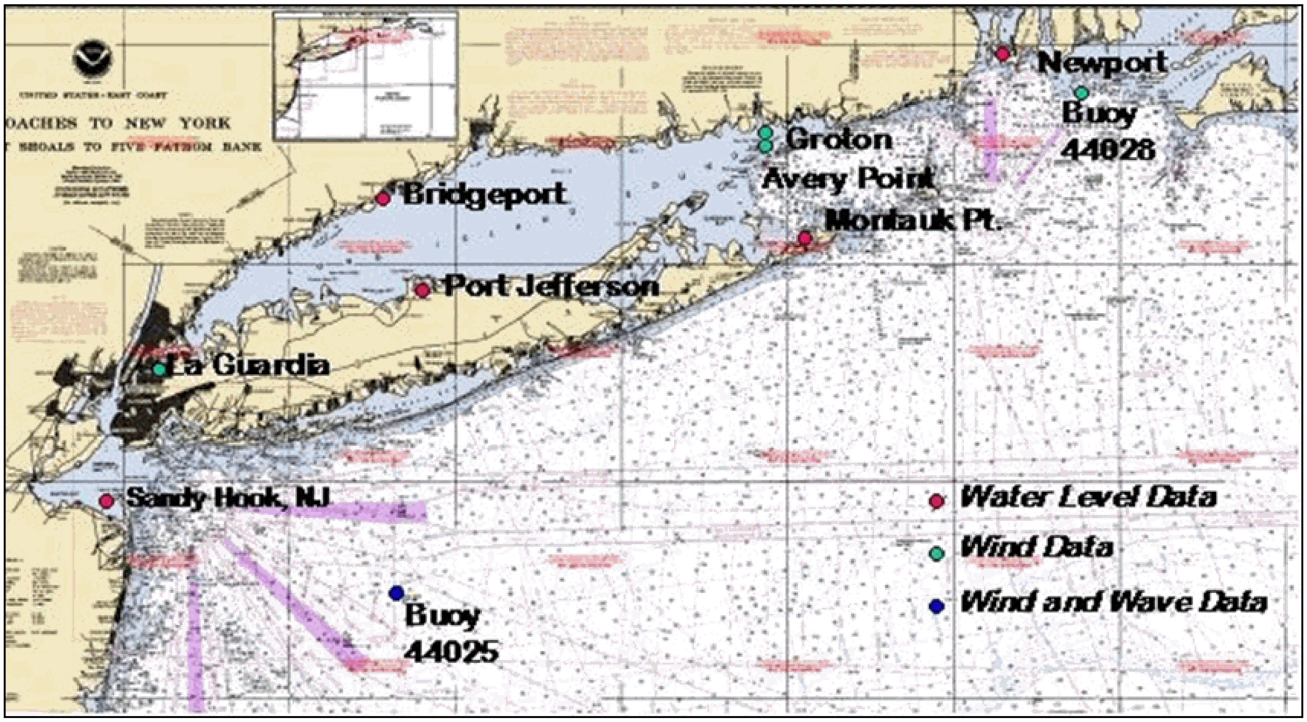


Figure A-6 Wind Stations

Long-Term Wind

The long-term wind speed percent-occurrence vs. directions for the study site was developed for the four measuring stations and for the hindcasted winds file. A plot of the hindcast wind rose is shown in Figure A-7. This figure indicates that the annual winds are predominantly from the south and west quadrant. The prevailing wind with speed range from 10 to 20 mph occur approximately 40% of the time.

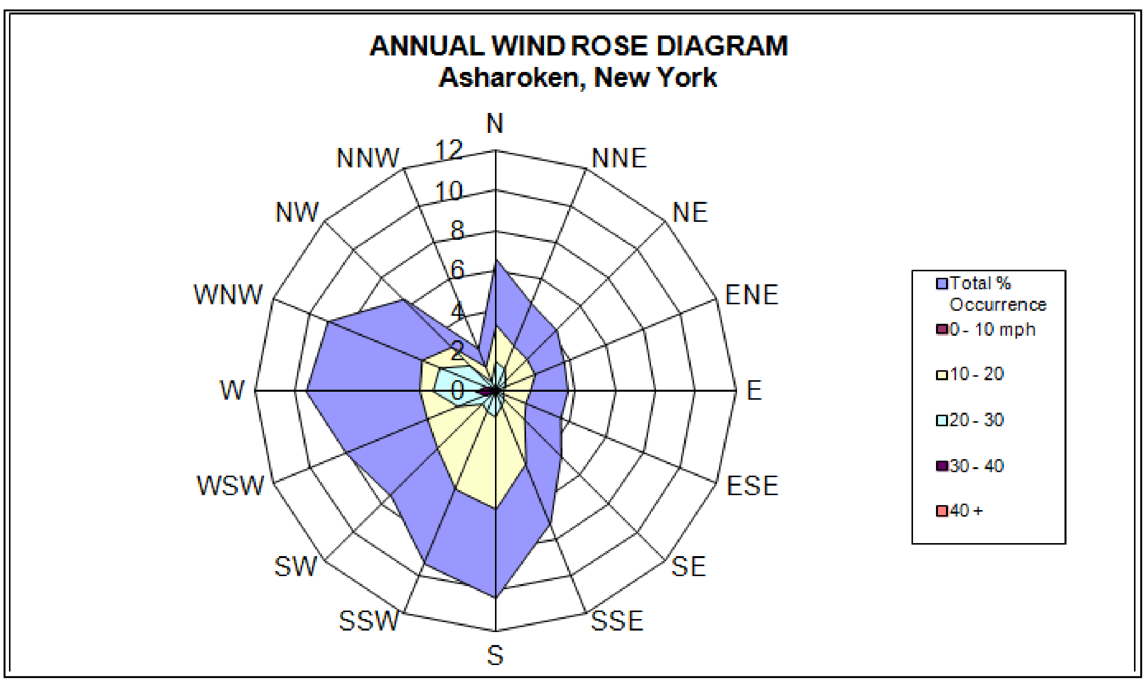


Figure A-7 Annual Wind Rose

Extreme Wind

Extreme wind speed-frequency statistics was developed based on the peak hindcasted wind speeds. Note that all peak storm winds during the events were from the east to northeast directions except for two events (the 1990 events where the peak wind speeds were from the NW and were below 40 mph). The extreme wind speeds (fastest mile) are presented in Table A-2 below and graphically in Figure A-8. Table A-2 indicates the 100-year wind speed is approximately 108 mph based on Gumbel distribution of all historical storms affecting the project site.

TABLE A-2 Storm Wind-Frequency Relationships
Asharoken, North Shore of Long Island, New York

Return Period (years)	All Storms (mph)	Extra Tropical (mph)	Tropical (mph)
5	62.0	55.0	50.0
10	73.7	59.0	64.7
25	88.0	71.5	85.3
50	98.3	80.2	98.7
100	108.0	88.6	111.5
200	118.5	96.9	124.1
500	132.8	107.7	140.6

Note: Wind speed in fastest mile based on Gumbel distribution

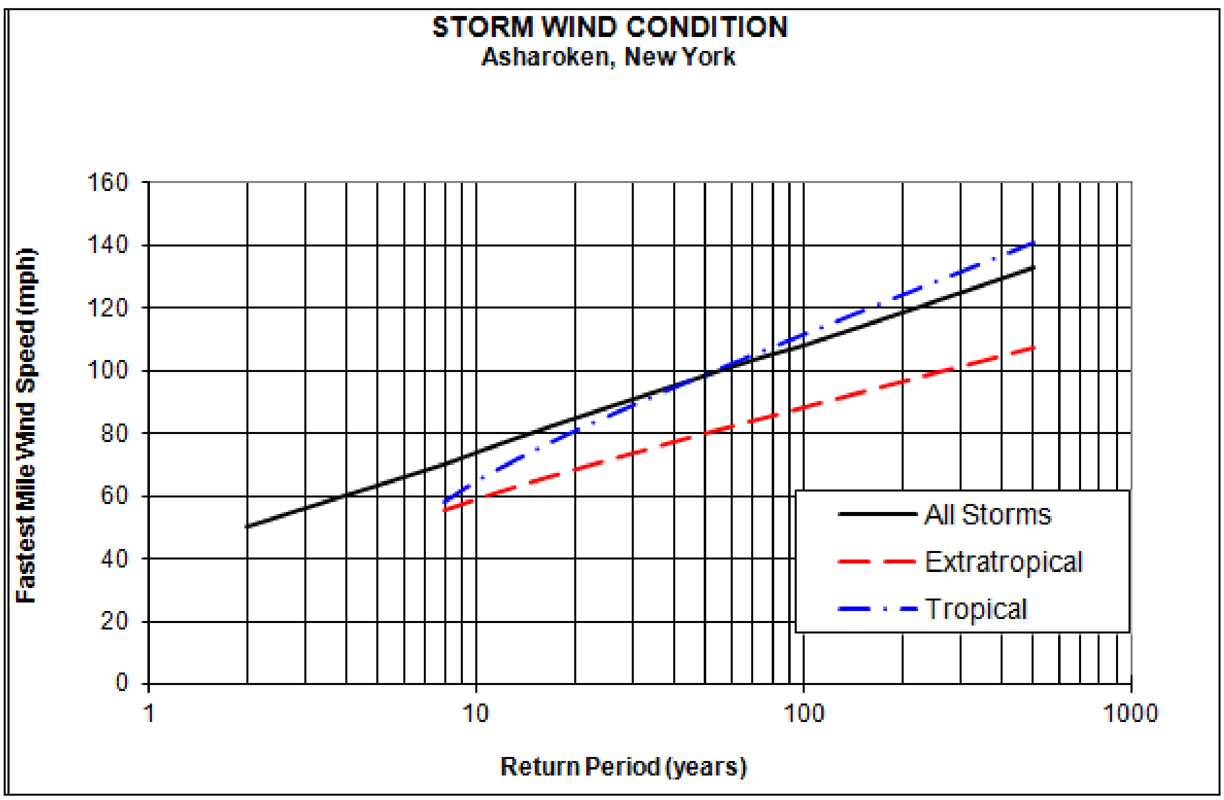


Figure A-8 Extreme Wind-Frequency

2.3 Wave Condition

The North Shore wave regime is dominated by wind-generated waves within Long Island Sound. For Asharoken Beach, the general shoreline is northwest-southeast oriented and the maximum exposure direction for critical wave generation is northeast.

A directional spectral time-stepping wave model WAVAD (also known as WISWAVE) was applied to determine extreme storm wave conditions for the same events modeled with ADCIRC, and to characterize long term wave climate by modeling ten-years of continuous waves during the 1990's. In addition, local wind wave hindcast model based on SPM 84 formula was used to predict the deep-water storm wave condition. Nearshore waves were calculated based on depth-limited wave shoaling at controlling depths of 0, 10, and 20 ft NGVD contours and compared with SBEACH nearshore wave model output.

Wave Hindcast Based on WISWAVE Modeling

Bathymetric data implemented for the WISWAVE modeling were obtained from several sources. Open ocean bathymetry was obtained from NOAA nautical charts and bathymetry for Long Island Sound was used from a USGS database (Signell, 1998). Three levels of nesting were used to generate the wave data at a resolution appropriate for the project. The wave model was driven by the 27 storm wind fields as well as by a 10-year continuous set of hindcasted winds. The storm wind fields were hindcasted on an hourly time increment so as to accurately resolve the time-varying characteristics of rapidly moving storm systems. The 10-years of winds were hindcasted on a 6-hourly time increment that is meant to define the long-term wave distribution in the area of interest.

Input parameters for the model include the model time step which was 60 seconds, the 15 wave frequencies over which to compute the wave spectrum (0.07 to 0.35 Hz with an increment of 0.02 Hz), and the discretization increment for the directional spectrum (22.5 degrees). Output includes bulk parameters (zero-moment wave height, peak wave period, peak wave direction) and full two-dimensional wave spectra that will serve as input to nearshore STWAVE modeling in very shallow water. Storm simulations were reported at output locations on an hourly basis. Continuous daily conditions during the ten-year hindcast were reported on a 6-hourly basis.

Although there are no available wave data sets to validate the wave modeling locally at project sites, the wave data produced by the hindcast model were assessed at NOAA buoy 44025 for a one-year period and for a significant storm event. The results indicate that over the year, the model reproduces the average zero moment wave height, Hmo, within 0.08m, the average peak spectral wave period, Tp, within 1.2 seconds and the average peak spectral wave direction within 5 degrees. These comparisons are considered acceptable for a long-term hindcast. The storm peak in early February (approximately record 120 in Attachment 4) is modeled to within 0.2 meters of the 5.6-meter peak wave height during the storm.

The results from the storm simulations and the 10-year continuous wave hindcast are archived at output locations shown in Table A-3 and Figure A-9. The locations are generally in water depths of 35 feet or greater relative to mean lower low water. During normal tide conditions, the average water depth is about 38 feet, and during storms at these locations the water depth is 43-45 feet deep or greater. These water depths, in combination with the rapidly dropping bottom seaward of the locations, are outside of depth-limited breaking conditions and are within the range of applicability of the wave model. For sediment transport assessment, conditions at these locations will be transformed landward using a model that is more appropriate for shallower conditions.

Long-term Wave Statistics

The wave model results from the 10-year (WISWAVE) continuous hindcast (1990-1991) were analyzed to estimate long-term wave statistics at the output location offshore of Asharoken. The bivariate (wave height-period) and trivariate (wave height-period-direction) frequency-of-occurrence tables for each site were calculated from the output. Wave heights are Hmo (zero moment wave height in feet), wave periods are peak spectral peak periods, Tp, in seconds, and wave directions are the direction from which waves travel in degrees clockwise from north. An annual wave rose diagram is shown in Figure A-10. Due to the orientation of project shoreline, only those waves from NW clockwise to ESE will reach the nearshore site and approximately 65% of the sea is calm during wind directions blowing offshore. As shown in Figure A-10, the long term annual waves are evenly distributed with larger waves from NNE and NE directions

Table A.3 Wave Model Output Location For Analysis

Location	Depth (ft MLLW)	Lattitude	Longitude
Offshore Asharoken	-35	40 deg 57 min N	73 deg 21 min W

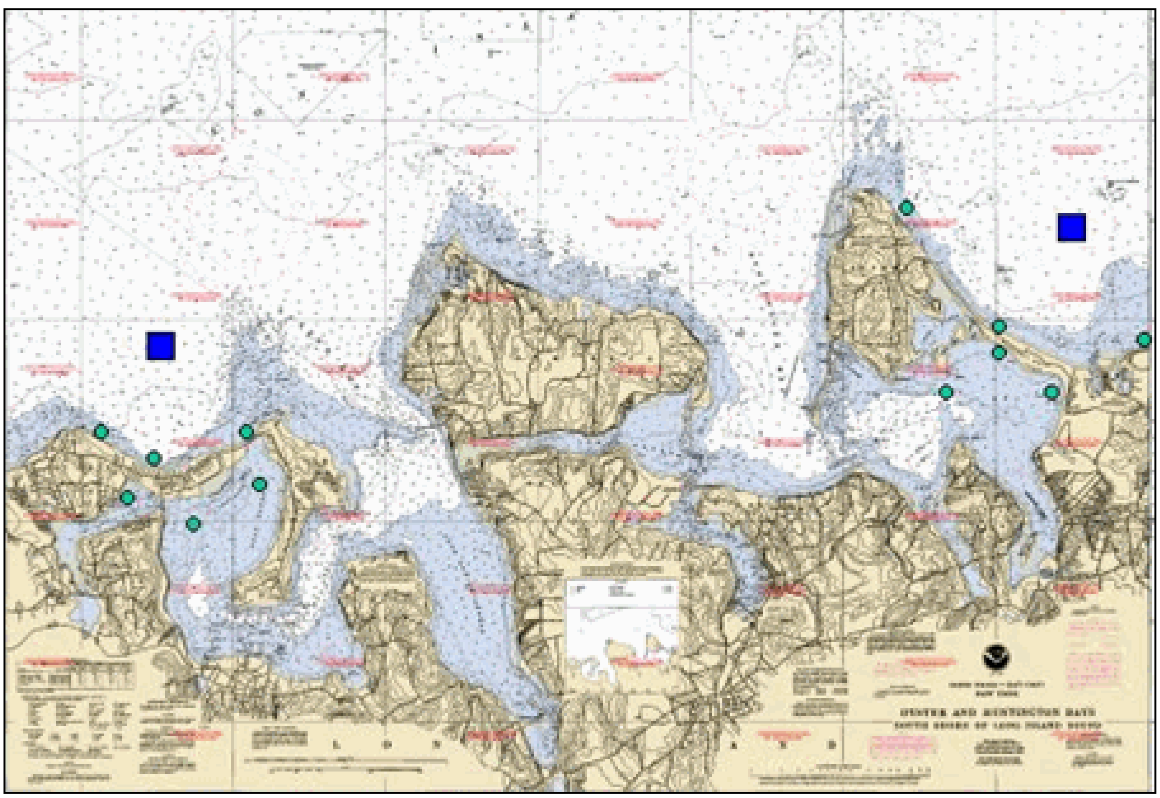


Figure A-9 Wave Model Output Stations

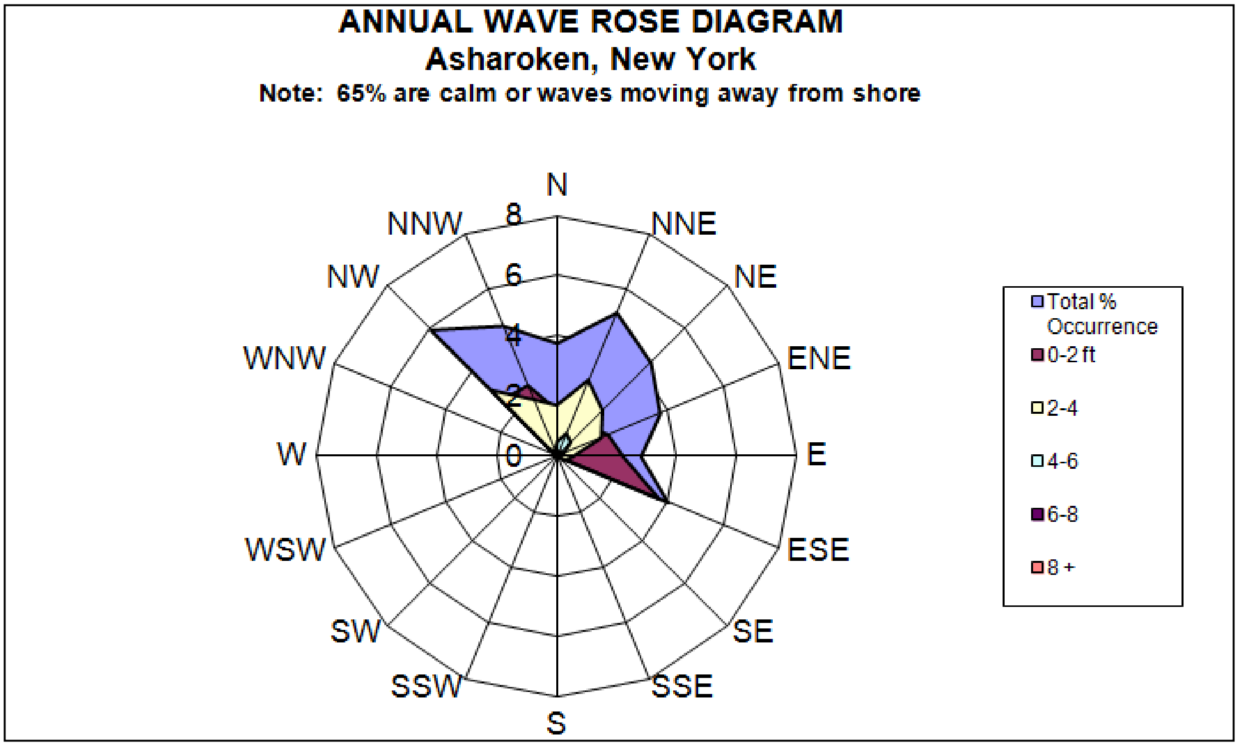


Figure A-10 Annual Wave Rose Diagram

Extreme Wave Statistics

Extreme wave statistics at the same offshore locations were determined by extracting peak wave heights for each storm event from the hindcasted time history. Those peaks were then analyzed with a best-fit Gumbel distribution using the ACES system to determine extreme wave heights for storm return periods. Results are based on the shallow water wave prediction curves in the Shore Protection Manual and a fetch to the northeast. The predicted extreme waves are presented in Table A-4.

In addition to the statistical prediction based on historical storms, offshore storm wave height-frequency was also predicted based on storm wind-frequency, fetch length, and average water depth along the critical wind direction from Northeast as shown in the last column of Table A-4. The significant wave height and corresponding peak wave period were calculated based on Equations 3-39 and 3-40 of SPM 1984. The average fetch used for hindcast along the critical wind direction (northeast) is 31.4 miles and the average water depth along the fetch is 75 ft NGVD. The hindcast waves are also shown graphically in Figure A-11. The hindcasted waves based on storm wind and fetch are be used for design calculation since it better represent the site condition, including limited fetch length, predominant storm wind direction, and water depths.

Table A-4 Asharoken Deep Water Storm Wave Condition

Return Period (years)	Based on Storm Events and Gumbel Distribution Tropical (ft)	Extratropical (ft)	All Storms (ft)	Hindcast Based on Winds and Fetch (ft)
2	3.1	5.2	7.8	8.4
5	8.4	9.2	12.3	10.5
10	12.0	11.8	15.3	12.4
25	16.4	15.2	17.8	14.8
50	18.4	17.7	18.4	16.4
100	18.4	18.4	18.4	18.0

Note: Wave heights shown are Hmo or Hs

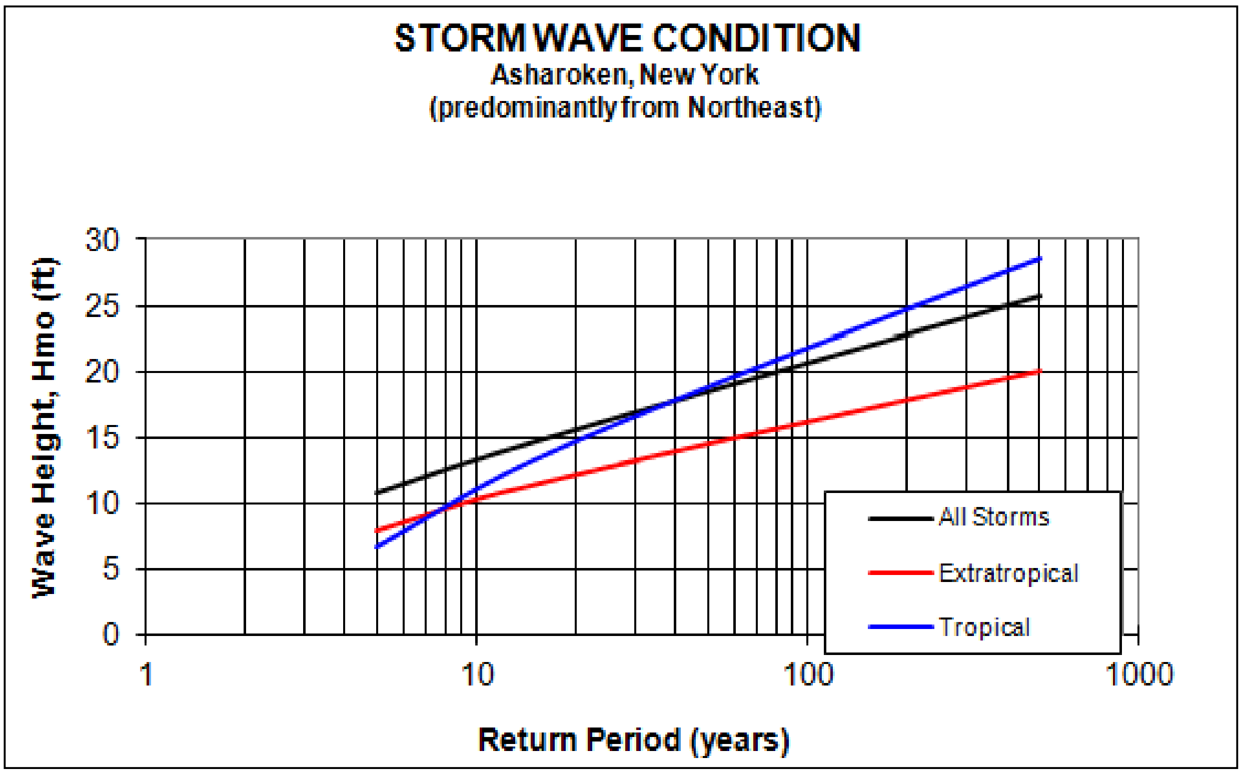


Figure A-11 Extreme Wave-Frequency

Nearshore Wave Condition

As deep water storm wave propagate onshore, it start refraction, shoaling, and breaking when feeling the bottom slope. Deep water waves continue to break as water depths become shallower, which is defined as depth-controlled wave. Nearshore wave heights are calculated based on shoaling of hindcasted deep water waves along a 1 vertical on 100 horizontal foreshore slope at 0, 10, and 20 ft contour depths as shown in Table A-5. The calculated wave heights are compared with peak storm wave (of all storms) using SBEACH transformation (Reference 1) with reasonable agreement.

Table A-5 Asharoken Nearshore Wave Condition

Return Period (years)	Based on SPM84 Shoaling Calculation of Hindcast Waves at Various Bottom Contours			Based on SBEACH Transformation of Peak Storm Waves
	0 ft NGVD	-10 ft NGVD	-20 ft NGVD	
2	2.5	10.2	16.8	7.3
5	6.1	13.6	20.3	13.4
10	7.2	14.7	21.5	17.9
25	8.3	15.8	22.7	23.3
50	9.1	16.6	23.6	23.8
100	10.0	17.5	24.5	24.2

Design Wave Condition

As discussed above, the storm wave-frequency based on limited fetch hindcast are used as design wave for existing condition and future with/without project calculations. The design deep water and nearshore (breaking) waves are summarized in Table A.6:

Table A-6 Design Wave Condition

Return Period (years)	Offshore (deep water) waves		Nearshore (shallow water) Waves (ft)		
	Hs (ft)	Tp (sec)	0 ft NGVD	-10 ft NGVD	-20 ft NGVD
2	8.4	5.9	2.5	10.2	16.8
5	10.5	6.4	6.1	13.6	20.3
10	12.4	6.9	7.2	14.7	21.5
25	14.8	7.3	8.3	15.8	22.7
50	16.4	7.7	9.1	16.6	23.6
100	18.0	7.9	10.0	17.5	24.5

2.4 Tides

Tides in the study area are semidiurnal with a mean tidal range of 7.1 ft and a spring range of 8.2 feet. Mean Lower Low Water is approximately 4.2 feet below the North American Vertical Datum of 1988 (NAVD) and Mean Higher High Water is approximately 2.9 feet above NAVD based on Tide Tables published by National Oceanic and Atmospheric Administration, National Ocean Service (NOAA, NOS). In the vicinity of the study area, the maximum observed tidal height was about 8.1 ft above 1988 NAVD as recorded on February 6, 1978 at Port Jefferson tide gage. The lowest tide was observed at the same tide gage at 7.6 feet below 1988 NAVD on January 10, 1978. The astronomical tide elevations are summarized in Table A.-7. The astronomical tidal range at Asharoken Back Bay shoreline is approximately 0.2 ft less.

Table A-7 Astronomical Tide Elevations
Asharoken, New York

Datum	Elevation (ft NGVD)	Elevation (ft NAVD)
Highest Observed (6 Feb. 1978)	+9.1	+8.1
Mean Higher High Water (MHHW)	+3.9	+2.9
Mean Tide Level (MTL)	+0.4	-0.6
Mean Lower Low Water (MLLW)	-3.2	-4.2
Mean Tide Range (ft)	7.1	7.1
Spring Tide Range (ft)	8.2	8.2
Lowest Observed (10 Jan. 1978)	-6.6	-7.6

Notes: 1. Highest and lowest observed elevations are based on Port Jefferson tide gage
2. NAVD Datum is 1.0 ft above NGVD Datum

2.5 Storm Surge

Extreme water elevations due to storm surges and wave setup in the study area can be generated by either large-scale extra tropical storms known as northeasters or tropical storms known as hurricanes. The impact of storm surge on total water level depends on the extent to which the storm coincides with the high astronomical tides. For the Village of Asharoken, the storm tide elevations were developed based on the finite element numerical Advanced Circulation (ADCIRC) model, which is part of the U.S. Army Corps of Engineers Surface water Modeling System (SMS). The extreme water levels were analyzed to estimate surge values corresponding to a range of return periods (2 to 500 years) using the Empirical Simulation Technique, or EST (Sheffner, et al., 1999). The following summarizes the ADCIRC model computation grid, input parameters, model validation, and estimates of extreme water levels based on EST technique. Details of ADCIRC and EST modeling study procedures are described in the OCTI Storm Surge Modeling Report (Reference 1).

Computational Grid

Bathymetric data implemented for the ADCIRC water level modeling and the wave modeling were obtained from several sources. The mesh was adapted from one developed by the Coastal and Hydraulics Laboratory for regional modeling of the Long Island area (Militello and Kraus, 2001). In that mesh, open ocean bathymetry was obtained from NOAA, traditional and SHOALS survey data were obtained for Shinnecock Bay and nearshore areas at Moriches Inlet and Jones Inlet. GEODAS data were obtained for Great Peconic Bay, Great South Bay, and Long Island Sound. For the present study, bathymetry for Long Island Sound was replaced with that from a USGS database (Signell, 1998), except for the local area around Asharoken and at Port Jefferson. Traditional and multibeam sonar surveys conducted by the U.S. Army Corps of Engineers, New York District, in 2001 were used to refine the bathymetry in the nearshore of the Asharoken study area. To facilitate model validation at a nearshore National Ocean Survey tide gauge location, bathymetric data for Port Jefferson was taken from NOAA nautical charts 12364-21 and 12362-1. The ADCIRC model mesh and bathymetry in the study area is shown in Figure A-12.

ADCIRC Model Input Parameters

A description of the method for configuring the ADCIRC model input parameters are described in complete detail by Westerink, et al. (1994). For this application, each storm simulation was forced by the wind fields and astronomical tides developed by the ADCIRC constituent database for the North Atlantic Ocean. Input coordinates of the bathymetric mesh are specified in degrees of longitude and latitude and the model solves the vertically-integrated form of the hydrodynamic equations. The standard quadratic parameterization for bottom friction is used with some refinement described in the Model Validation process. Convective acceleration terms, including both the spatial and time derivative components, are included in the computations. A spatially homogeneous nonlinear bottom friction and lateral viscosity coefficient (one value) is used throughout the computational domain. A spatially-varying Coriolis parameter is specified and a tidal potential forcing function is activated. The wind (and pressure, in the case of tropical storms) field was sufficiently long to allow for one 24-hour ramp at the beginning of each simulation. The generalized wave-continuity equation weight factor, TAU-0, is 0.01. The model tide step is three (3) seconds. The time weighting factors, A00, B00, and C00 are 0.35, 0.30 and 0.35, respectively. The bottom friction coefficient is 0.003 and the lateral eddy viscosity coefficient is 2.00. A total of eight tidal potential constituents and eight forcing frequencies on the open boundaries are employed including K1, O1, M2, N2, S2, K2, P1, and Q1.

ADCIRC Model Validation

Model verification was conducted in two parts. The first part was a tidal verification. A month-long run forced by only tidal constituents was conducted for the month of May 1988 and calculated water levels were compared with measurements at Port Jefferson. This location was selected for assessing the model performance because, of available historical gauging locations, it had the closest proximity to the study site. The month of May 1988 was selected after a careful review of historic data to identify a 30-day time period that was not affected significantly by storm surges. Figure A-13 shows the calculated and measured water level. The mean error over the time series for this month-long calculation is “minus” 0.02 m. Table A-8 presents a comparison of tidal constituents at Bridgeport, CT. The comparison indicates that the ADCIRC model over predicts tidal amplitudes by an average of 5% and tidal phases by an average of 2%. These comparisons are considered acceptable for a simulation over a limited time and given the complexities of the tidal propagation in Long Island Sound.

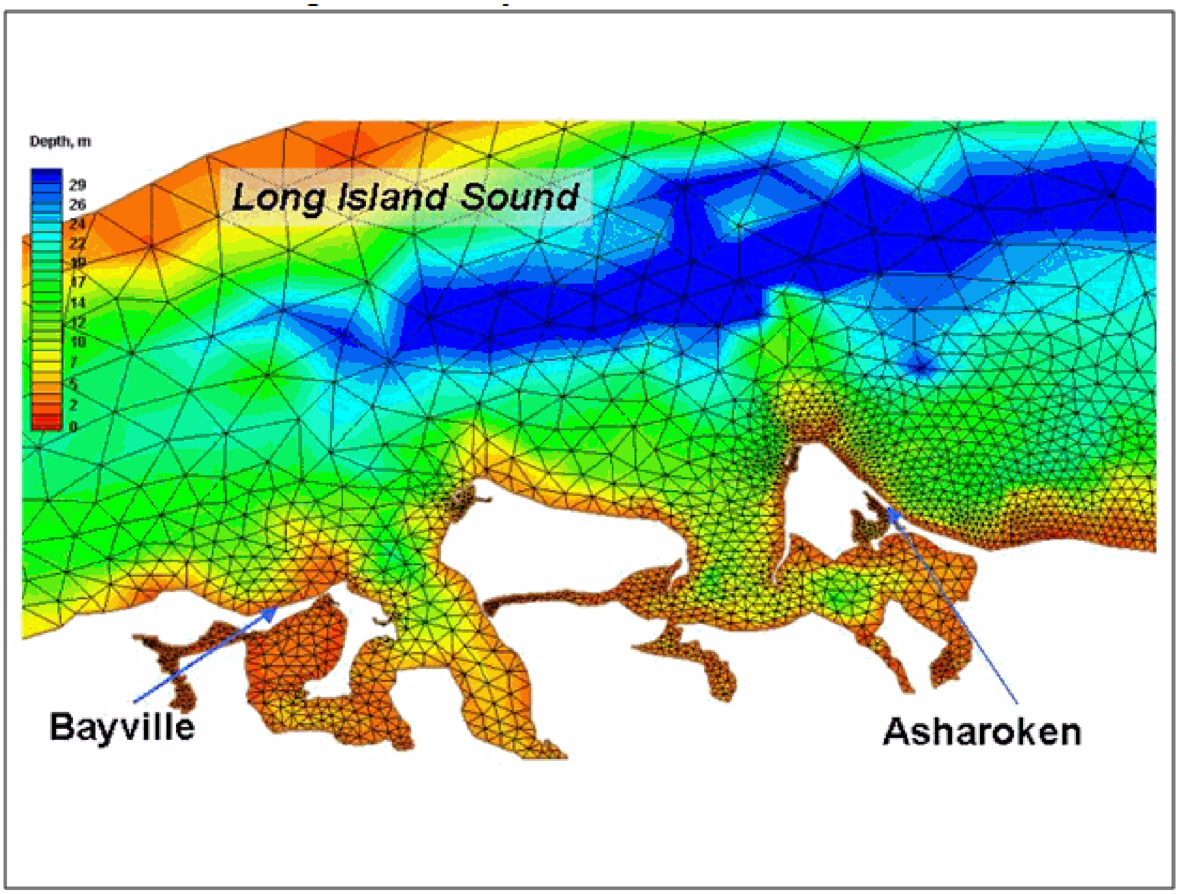


Figure A-12 ADCIRC Model Mesh

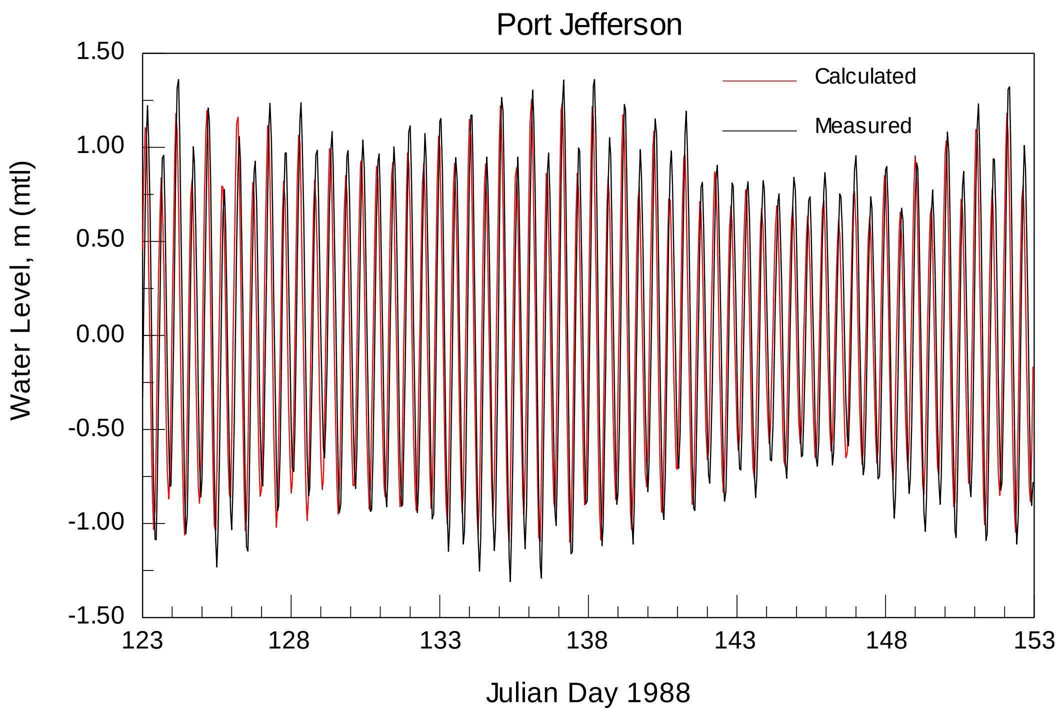


Figure A-13 ADCIRC Astronomical Tide Verification, May 1988

Table A-8 Comparison of NOS and ADCIRC tidal constituents at Bridgeport, CT.

Constituent	NOS Amp (m)	NOS Phase (deg)	ADCIRC Amp (m)	ADCIRC Phase (deg)	Amp Diff[1] (m)	Phase Diff[2] (deg)	Amp % Error[3]
K1	0.097	191.6	0.096	198.8	-0.001	7.2	1.0
O1	0.064	219.5	0.061	216.9	-0.003	-2.6	4.7
M2	0.991	109.6	0.918	107.7	-0.073	-1.9	7.4
N2	0.200	87.6	0.182	92.5	-0.018	4.9	9.0
S2	0.157	135.9	0.152	146.5	-0.005	10.6	3.2
K2	0.046	134.7	0.030	113.6	-0.016	-21.1	3.5
P1	0.030	204.1	0.028	204.3	-0.002	0.2	6.7
Q1	0.018	205.7	0.012	210.7	-0.006	5.0	3.3

1. Amp Difference = ADCIRC Amp – NOS Amp
2. Phase Difference = ADCIRC Phase – NOS Phase
3. Percent Error = abs((Amp Diff/NOS Amp)*100)

Validation for storms was also performed to establish the accuracy of the model for combined storm surge and astronomical tide. Modification to the wind fields (magnitude of the speed) was the initial method of attempted validation. Four storms were used to calibrate the model: February 1978, December 1990, December 1992, and February 1998. During this process, the small-scale (more accurate) winds were multiplied by one factor, while the large-scale (NCEP) winds were multiplied by another. In addition, the winds over Long Island Sound were scaled separately from the remaining small-scale winds. These scaling were done because the sources of information that went into the wind models were different, with some areas, such as Long Island Sound, having less certainty. With consistent scaling of the wind fields for each of the storms, response to wind forcing was not found to be consistent (i.e. good agreement for one or two storms was not met with good agreement on the remaining storms). Therefore, another approach was taken to validate the surge model.

The wind stress coefficient in ADCIRC was modified from Garratt's 1977 formulation to Hsu's 1988 formulation. Garratt's formulation gives the wind stress coefficient C_d as

$$C_d = 0.001(0.75 + 0.67W)$$

where *W* is the wind speed at a height of 10 m above the water surface. This formulation gives a linear relationship between the wind speed and wind drag coefficient. Hsu's formulation for the wind stress coefficient is given by

$$C_d = \left(\frac{0.4}{14.56 - 2\ln W}\right)^2$$

which implicitly includes the effect of roughness height owing to waves and assumes fully-developed seas. Hsu's formulation gives a nonlinear relationship between the wind speed and wind-stress coefficient. Garratt's and Hsu's formulations give the same value for C_d at a wind speed of approximately 30 m/sec. Below wind speed of 30 m/sec, Hsu's formulation gives a greater value of the wind drag coefficient, and above 30 m/sec, Garratt's formulation gives a greater value. Applying Hsu's formulation, surge results became consistent from storm to storm. The final wind scaling that was conducted for the verification was no modification to the small-scale winds, and 25% increase in the NCEP winds. Upon agreement with surge results for the initial storms, additional northeasters and tropical storms were simulated for validation. In general, agreement was good between measured and calculated water levels, particularly at the Bridgeport, CT and Port Jefferson, NY stations.

Including the four calibration storms used for wind field and wind stress examination, a total of nine validation storms were used to assess, or validate, the performance of the model. Four were tropical storms and five were extra tropical events. The results are shown in the validation plots in Attachment 3 of OCTI Report, and summarized in Tables A-9 and A-10. Measurements were used from NOS tide gauges at Bridgeport or Port Jefferson, depending upon availability. Port Jefferson data were used for this comparison unless no data exists, in which case Bridgeport data were used. The agreement between the numerical model and the measurements is considered acceptable as compared to other published surge model applications (Committee on Tidal Hydraulics, 1980).

Table A-10 illustrates the model performance by storm type and by measurement location. The average deviation at Port Jefferson of –0.41 feet could potentially be rectified by obtaining improved bathymetry at that site; however, if the 1990 storm (by far the poorest validation) is eliminated as an outlier, the average deviation at Port Jefferson becomes –0.22 feet. With or without the 1990 storm, these deviations are considered acceptable for model performance but should be considered as a possible bias when using estimated storm extremes that are estimated later in this report

Table A-9 Comparison of Model Results to Measurements

STORM	GAUGE LOCATION	HINDCAST WATER LEVEL (FT, MTL)	MEASURED WATER LEVEL (FT, MTL)	DEVIATION (FT)
February 1998	Bridgeport	5.73	5.60	+0.13
March 1993	Bridgeport	5.67	5.60	+0.07
December 1992	Bridgeport	8.29	8.57	-0.28
December 1990	Port Jefferson	4.88	6.08	-1.20
February 1978	Port Jefferson	8.11	8.29	-0.18
July 1976	Bridgeport	4.02	3.58	+0.44
August 1991	Port Jefferson	3.60	3.50	+0.10
September 1985	Port Jefferson	5.69	5.90	-0.21
August 1976	Port Jefferson	5.47	6.07	-0.60
			Mean Deviation	-0.19

Table A-10 Water Level Model Validation Statistics

	DEVIATION	COMMENT
Mean Deviation All Storms	-0.19 ft	Mean deviation is –0.06 ft if Dec 1990 storm is excluded
Mean Deviation Extra tropical	-0.22 ft	Mean deviation is –0.05 if Dec 1990 storm is excluded
Standard Deviation Extra tropical	0.54 ft	Standard Deviation is 0.20 ft if Dec 1990 storm is excluded
RMS Error Extra tropical	0.56 ft	RMS Error is 0.18 ft if Dec 1990 storm is excluded
Mean Deviation Tropical	-0.06 ft	Worst deviation is –0.60 ft
Standard Deviation Tropical	0.44 ft	
RMS Error Tropical	0.39 ft	
Mean Deviation Bridgeport	+0.10 ft	Worst deviation is +0.44 ft
Standard Deviation Bridgeport	0.30 ft	
RMS Error Bridgeport	0.27 ft	
Mean Deviation Port Jefferson	-0.41 ft	Nearly all storms under predicted indicating possible need for improved bathymetry to resolve gauge
Standard Deviation Port Jefferson	0.50 ft	0.28 ft if 1990 Storm is excluded
RMS Error Port Jefferson	0.61 ft	0.33 ft if 1990 Storm is excluded

Estimates of Extreme Water Levels

Extreme storm water levels are analyzed to estimate values corresponding to a range of return periods using the Empirical Simulation Technique, or EST (Scheffner, et al., 1999).

As mentioned earlier, the ADCIRC model was used to develop tropical and extra-tropical storm surge hydrographs for the Long Island Sound area. The storms that were simulated are listed in Table A-11. The ADCIRC model was also used to simulate a 30-day period of constituent-driven astronomical tide conditions that allowed the computation of tidal constituents at 6 analysis locations within the Asharoken project areas (listed in Table A-12 and illustrated in Figure A-14). This astronomical tide information provided a basis for estimating the tidal component of the water level during each storm event and, based on the difference between the storm water level and the tide, the storm surge component was singled out for EST analysis.

Table A-11 List of Hindcasted Storm Events

Storm Date (Year, Month,Day)	Storm Type	Duration (days)
19380920	Tropical	3.74
19440913	Tropical	3.99
19540830	Tropical	2.99
19540910	Tropical	3.74
19600912	Tropical	3.24
19720621	Tropical	3.99
19760809	Tropical	3.49
19850926	Tropical	3.46
19910819	Tropical	3.47
19960712	Tropical	3.47
19501121	Extratropical	7.89
19531102	Extratropical	7.89
19551010	Extratropical	7.89
19601208	Extratropical	7.89
19620302	Extratropical	7.89
19670424	Extratropical	6.87
19730319	Extratropical	7.89
19780204	Extratropical	6.99
19800112	Extratropical	7.89
19801021	Extratropical	6.80
19840323	Extratropical	7.99
19901108	Extratropical	7.89
19901201	Extratropical	6.99
19911027	Extratropical	7.89
19921208	Extratropical	6.99
19930309	Extratropical	5.83
19980201	Extratropical	7.89

Table A-12 Locations for Estimation of Extreme Water Levels

LOCATION	ADCIRC NODE	LATITUDE	LONGITUDE	EST ANALYSIS
Northport Bay (East End)	19462	40.9233	73.3542	Surge+Tide
Asharoken (Back Bay)	19425	40.9292	73.3783	Surge+Tide
Northport Bay (Entrance)	19299	40.9208	73.3783	Surge+Tide
East of Keyspan (LI Sound	18381	40.9300	73.3333	Surge+Tide
Asharoken Beach (LI Sound)	18499	40.9333	73.3676	Surge+Tide; Surge+Tide+Wave Setup
Eaton’s Neck North End	18063	40.9542	73.3867	Surge+Tide

The required total water level (surge plus tide) time history for each storm event in the suite of plausible storms is compiled by combining the ADCIRC generated storm surge hydrograph with a time history of the astronomical tide. Each storm surge hydrograph corresponding to a historical storm event that impacted the project area is combined with 12 different representations of the astronomical tide. The 12 representations of the astronomical tide were developed by considering three tidal ranges corresponding to Mean, Spring, and Neap tidal ranges and aligning the tidal time history with the storm surge hydrograph at four phases in the tidal curve such that the maximum storm surge occurs at high tide, mean tide falling, low tide and mean tide rising. Based on the tidal constituents calculated at each of the 6 analysis locations, Spring, Mean, and Neap tidal ranges were determined. The time series of tidal elevations was constructed based on a cosine curve with amplitudes corresponding to the computed Spring, Mean, and Neap tidal amplitudes.

The input vectors for these surge plus tide analyses were peak storm surge (in feet) and peak astronomical tide level relative to NGVD29. The response vector was the sum of the storm surge and the astronomical tide (in feet relative to NGVD29) during the storm time history after the astronomical tide was phased with the peak storm surge as described above. The mean tide permutations were given a relative probably of occurrence of twice that given to the spring and neap tide permutations.

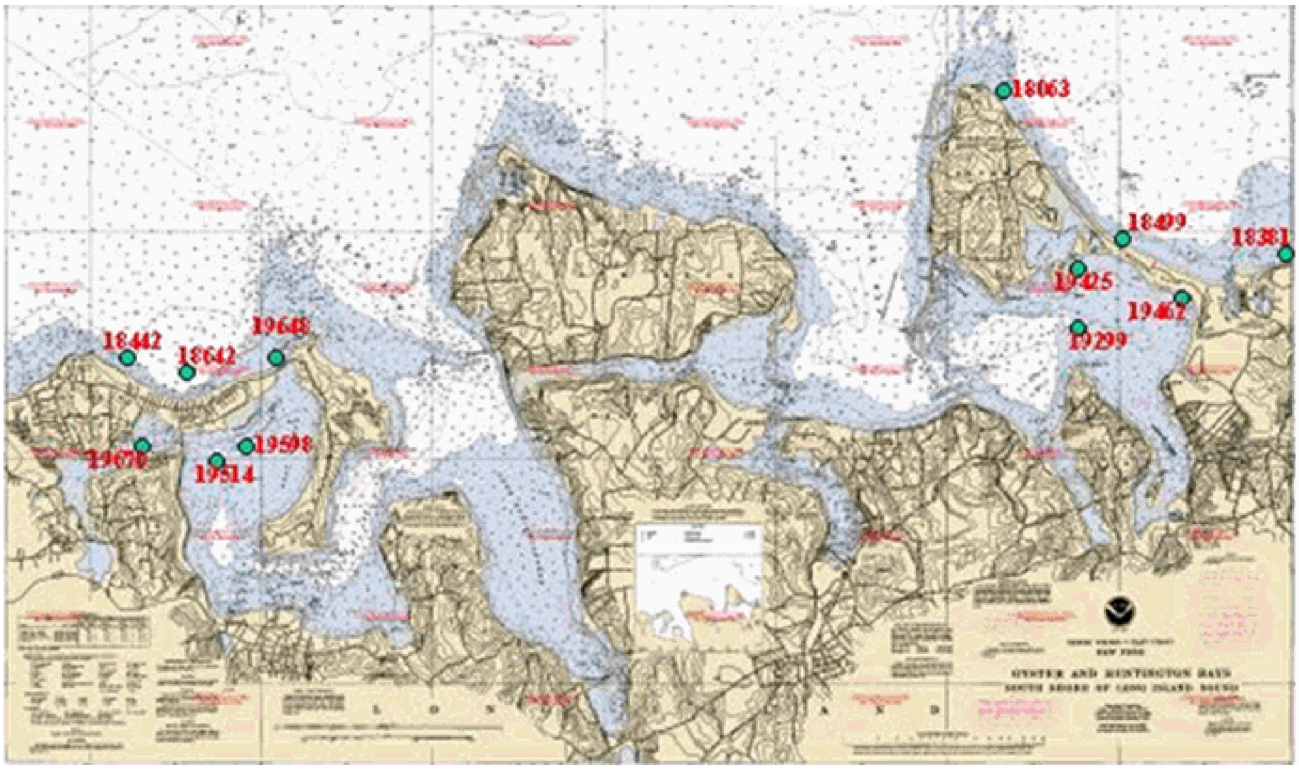


Figure A-14 Locations for Extreme Water Level Analysis

The results of the EST were compared to the previous estimate for surge plus astronomical tide water level made in Reconnaissance Report (U.S. Army Corps of Engineers, 1995). The Reconnaissance Report assumed that the back bay water levels were the same as those occurring on Long Island Sound during extreme events, so there was no differentiation between back bays and open Sound water levels in that document. At Asharoken Sound side, wave setup was estimated using typical beach profiles and all the permutations of water level required by the EST analysis. Wave setup was estimated using the SBEACH model that employs the Goda method across the beach profile. The wave setup is used at the location along the profile where the still water depth (surge plus tide) is about 0.5 feet.

The input vectors for these surge plus tide plus wave setup analyses were the peak storm offshore wave height (Hmo in feet), the peak surge (in feet NGVD29), and the peak total water level during the storm time history after the astronomical tide was phased with the peak storm surge as described above. The response vectors were the peak storm surge (in feet relative to NGVD29), the peak wave setup at the shoreline (in feet), and the total water level (TWL, the sum of tide plus surge plus the wave setup in feet relative to NGVD29). The mean tide permutations were given a relative probably of occurrence of twice that given to the spring and neap tide permutations.

The results of the EST analysis of storm surge plus astronomical tide with and without wave setup are presented in Table A-13 and Figure A-15. Table A-13 also presents a comparison of the present estimates with the Reconnaissance estimates for the combined storm population (tropical and extra tropical together). As shown in the table and figure, the recent surge frequency results indicates lower surge level for lower return period while the surge level is higher than the reconnaissance estimates for longer return periods. This is likely due to more advanced modeling technique and model input of high intensity storms affecting the project area in the 1990's. The average wave setup is approximately 1.5 ft above the storm surge elevation.

Comparison of North Atlantic Comprehensive Study (NACCS)

The North Atlantic Comprehensive Study (NACCS) was recently completed, which provides the most recent stage-frequency computation for the Asharoken study area. This effort involved the application of a suite of high fidelity numerical models within the Coastal Storm Modeling System (CSTORM-MS, with WAM, STWAVE, and ADCIRC). CSTORM-MS modeling produced nearshore wind, wave, and water level estimates and the associated marginal and joint probabilities from Virginia to Maine. Both tropical and extratropical storms were strategically selected to characterize the regional storm hazard. Results computed off of the Asharoken study area are tabulated below and shown in Figures A-15a and A-15b. Generally, the NACCS stage-frequencies lie between the Asharoken Sound Side with and without wave set up curves.

Consideration was given to determine which of the available stage-frequency data sets is most robust and applicable to the project area. The NACCS modeling effort covers a large-region. In contrast, ADCIRC and other modeling done specifically for the Asharoken Study used site-specific wind, storm surge and wave hindcasting modeling, which were calibrated to local winds, waves, and water levels. Nodes close to the shoreline were utilized for the Feasibility Study. Actual beach profile surveys were utilized to transform the modeled waves onto the shoreline in order to estimate the wave effects on the water levels, which were quite significant, for some cases in Asharoken increasing the water surface elevation over 25%. With the importance and relative magnitude of the wave effects, it was decided to use the project specific modeling, which uses actual measured bathymetry data all the way to the shoreline.

Extreme Water Levels Used for Design

ADCIRC and other modeling done specifically for the Asharoken Study area were utilized for design. These are the Asharoken with and without wave setup values shown below.

Table A-13a Stage-Frequency (Return Period) at Asharoken
(Elevation in ft NGVD; entire storm population included)

Return Period (Years)	Asharoken Sound Side (w/o wave setup)	Asharoken Sound Side (w/wave setup)	Northport Bay/Asharoken Back Bay	Recon. Report (for Reference Only)	North Atlantic Comprehensive Study (NACCS) (Mean Values)
5	7.47	9.16	7.23	8.4	9.40
10	8.79	10.81	8.57	9	10.25
15	9.42	11.59	9.18		
20	9.84	12.04	9.57		11.01
25	10.12	12.36	9.87	9.5	
44	10.9	13.1	10.7		
50	11.09	13.24	10.93	10.4	12.02
73	11.7	13.77	11.6		
100	12.21	14.25	12.16	11.5	12.91
150	12.92	14.94	13.05		
200	13.44	15.45	13.63	12.6	14.16
500	14.99	17.04	15.53		15.93

Note: NAVD datum is 1.0 ft above NGVD datum

Table A-13b Stage-Frequency (Return Period) at Asharoken
(Elevation in ft NAVD; entire storm population included)

Return Period (Years)	Asharoken Sound Side (w/o wave setup)	Asharoken Sound Side (w/wave setup)	Northport Bay/Asharoken Back Bay	Recon. Report (for Reference Only)	North Atlantic Comprehensive Study (NACCS) (Mean Values)
5	6.47	8.16	6.23	7.4	8.40
10	7.79	9.81	7.57	8	9.25
15	8.42	10.59	8.18		
20	8.84	11.04	8.57		10.01
25	9.12	11.36	8.87	8.5	
44	9.9	12.1	9.7		
50	10.09	12.24	9.93	9.4	11.02
73	10.7	12.77	10.6		
100	11.21	13.25	11.16	10.5	11.91
150	11.92	13.94	12.05		
200	12.44	14.45	12.63	11.6	13.16
500	13.99	16.04	14.53		14.93

Note: NAVD datum is 1.0 ft above NGVD datum

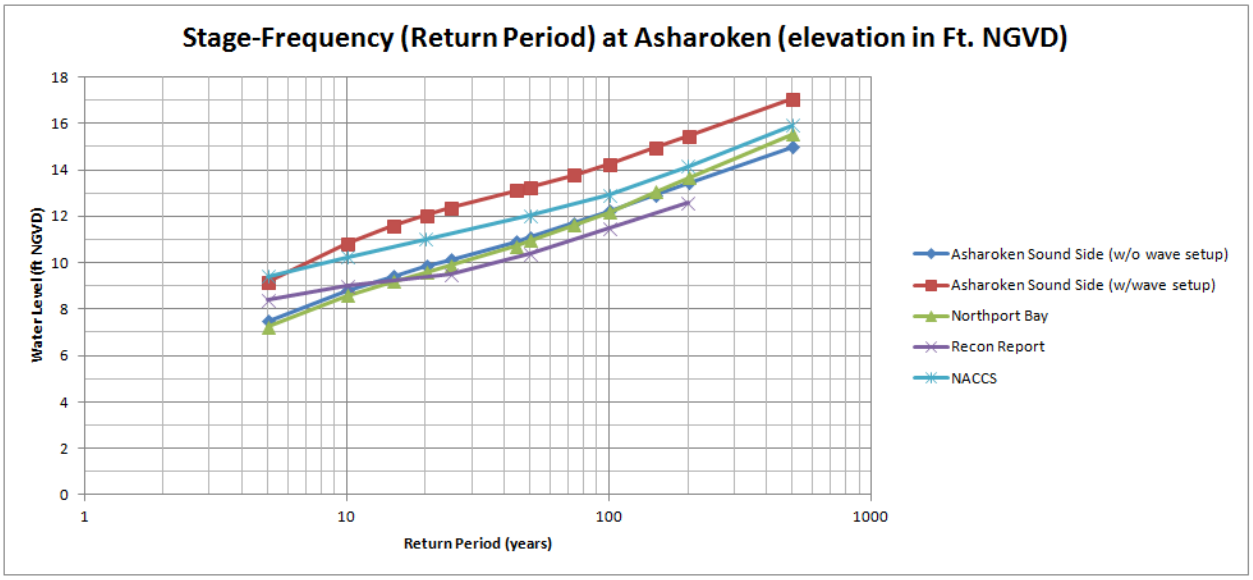


Figure A-15a: Storm Surge Frequency Curve, ft NGVD

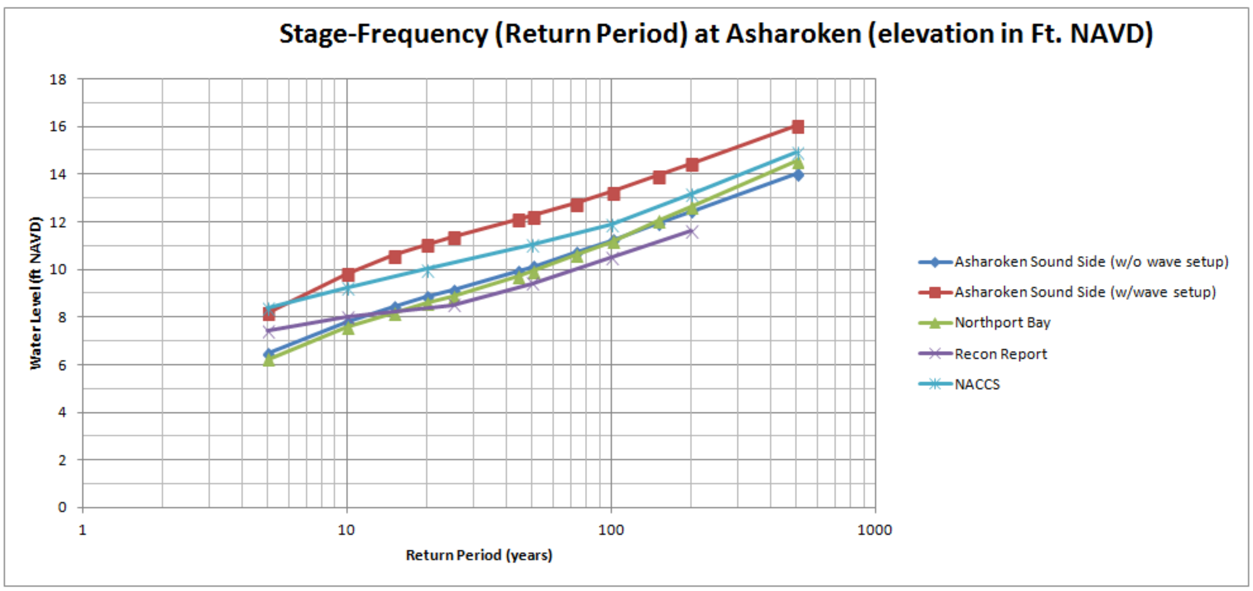


Figure A-15b: Storm Surge Frequency Curve, ft. NAVD

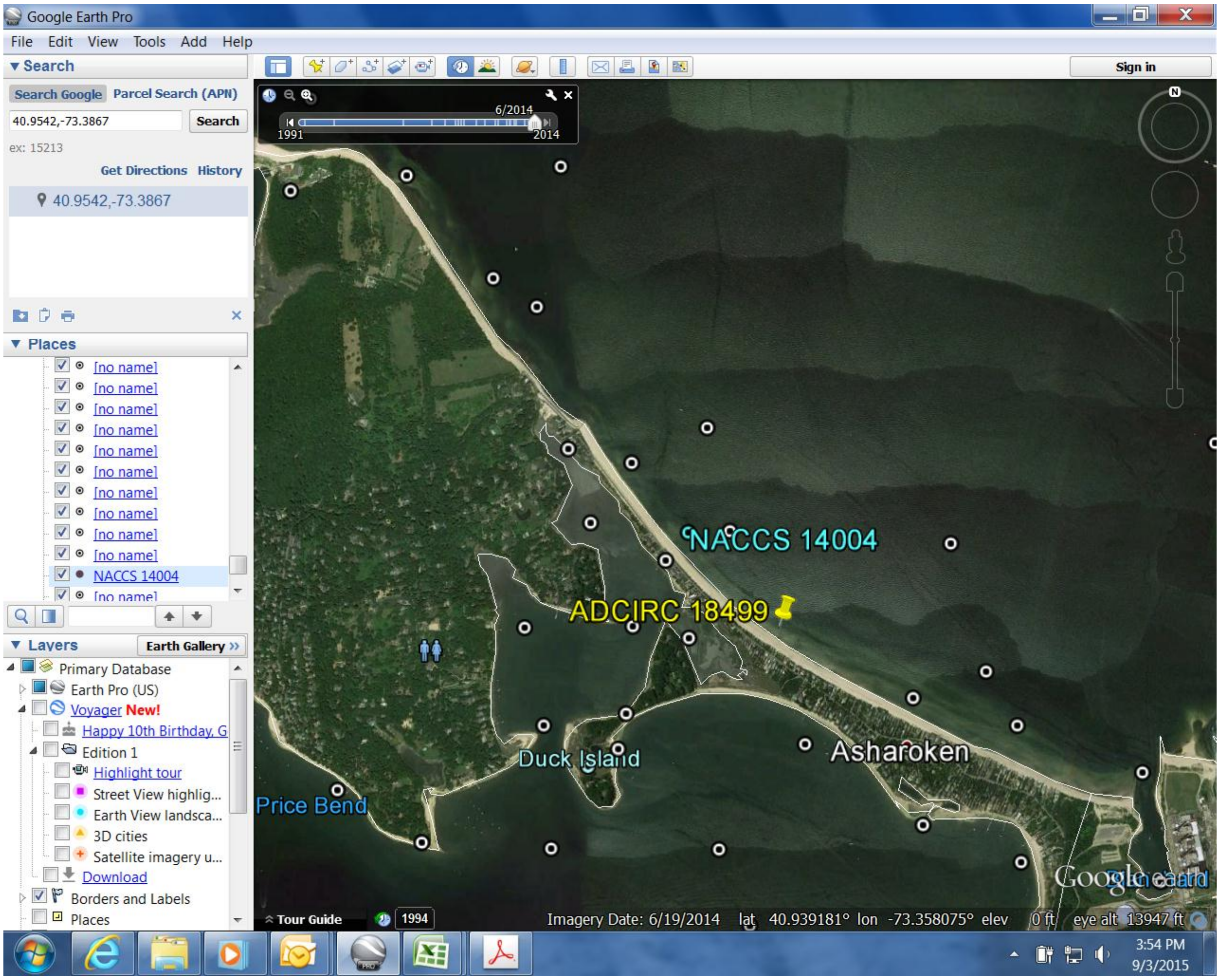


Figure A-16: North Atlantic Comprehensive Study and ADCIRC nodes in Asharoken study area.

Hurricane Sandy Damage

Hurricane Sandy passed the general project area in the period from late October to early November, 2012 for duration of three days. The storm track traveled through the New York Bight and inflicted heavy damage to the northern New Jersey and western Long Island Atlantic shoreline with combined storm surge and wave forces. Many locations experienced significant damage from combined wave and water level conditions of 200-year return period storm intensity, or greater. However, the Asharoken shoreline was not affected as significantly. Based on USGS data, the high water marks along both LI Sound and Bay shoreline are approximately +10 ft NAVD, not including wave setup. Assuming a 20% wave setup, this storm is equivalent to approximately 50 year intensity along the project shoreline. Damage at the Section 103 seawall consisted of overtopping leading to road flooding and scour of the road embankment behind the wall. Utility poles along Asharoken Avenue were downed. Dunes were lost, and overwashed sand covered bayside wetlands. Inundation depths of up to 2.5 feet were recorded in nearby Huntington. Some inundation of homes and businesses occured within Asharoken, including the Asharoken Village Hall. USGS high water marks are shown in Figure A-17 below, as well as the extent of inundation from Hurricane Sandy in the project area.

Figure A-17: Hurricane Sandy USGS High Water Marks and Inundation Limits

<u>2.6 Tidal Currents</u>

The direction and velocity of the tidal currents near Asharoken Beach is based on Tidal Current Tables published by NOAA, NOS. At Eaton's Neck, the tidal currents flow northwest to southwestward during flood periods (depends on location of current meter), and northeast to eastward during ebb periods. The average maximum speed (or strength of current) during flood current is predicted to be 2.4 feet per second (fps) and the average maximum ebb current is predicted to be 2.4 fps based on the current meter located approximately 1.3 miles north of Eaton's Neck. Tidal current velocity offshore of Asharoken Beach is expected to be weaker and flow in northwest-southeast directions. A field measurement of current speed and directions was conducted vicinity of the National Grid facility as part of the sediment transport study. The results of measured current velocity indicate a tidal current strength ranging from 0.3 to 0.8 fps along the study shoreline.

<u>2.7 Geomorphology</u>. Long Island belongs to the inner part of the Atlantic Coastal Plain. Part of the deposits of the island are true coastal plain deposits, whereas, the greater portion of both the surficial and underlying materials are of Pleistocene age and represent moraine and outwash accumulations associated with the continental glaciers. Cretaceous forms underlying those of Pleistocene age are exposed at several locations within the study area. The extensive unconsolidated sediments underlying the study area are of Cretaceous, Pleistocene and Recent origin, ranging from fine silts and clays to sands and coarse gravel.

The North Shore of Long Island consists of features that include beaches, bluffs, dunes, wetlands, and barrier landforms. Topographic character and sediment composition of the area determines the manner in which these landforms interact with the marine environment and directly impact coastal erosion and flooding.

The study shoreline vicinity is highly irregular, indented by several deep harbors and bays. Peninsulas or necks extending into Long Island Sound separate these bays and harbors. The narrow beaches of the necks are backed mostly by bluffs composed of Manhasset formation, till and outwash deposit. Bluff heights are generally low (approximately 30 feet) along the westernmost portion of the study area. Heights generally increase in an easterly direction, ranging from 75 to 110 feet in the vicinity of Lloyd Point, Eatons Neck, and Nissequogue. Eroded bluff material has formed small pocket beaches in many locations between the projecting points of the necks. Additionally, material eroded from the necks and offshore islands has been deposited as spits, bay mouth bar, and tombolos.

2.8 Climate Change Adaptation (Sea Level Change)

Comprehensive Evaluation of Projects with Respect to Sea Level Change
Sea Level Change (SLC) is the combined effect of the eustatic (i.e. global average) sea level increase due to global warming trend and the land movement in the region. The future SLC for the project area is estimated based on the National Research Council (NRC) and Intergovernmental Panel for Climate Change (IPCC) estimates of eustatic SLC and corrected to include the local land subsidence. Both the historic SLC trend and the future accelerated rate are identified and used for planning, design, sensitivity and risk & uncertainty analysis if required.

SLC Guidance
In October 2011, USACE published guidance to incorporate sea-level change for project planning and design (EC1165-2-212). This SLC guidance has since been expired and replaced with ER 1100-2-8162 (Feb.2014) and ETL 1100-2-1 (Dec.2014). The most recent guidance recommends both the National Research Council report (NRC, 1987) and the Intergovernmental Panel for Climate Change report (IPCC, 2007) findings for prediction of future sea level change. The recommendations are summarized as follows:

1) An extrapolation of the historic rate of local mean-sea-level rise shall be used as the low rate of sea level change for analysis, design, and evaluation;

2) Estimate the intermediate rate of local mean sea-level change using the modified NRC Curve I and NRC equations 2 and 3, and add those to the local rate of vertical land movement.

$$E(t) = 0.0017t + bt_2 \qquad \text{(NRC Equation 2)}$$
$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^2 - t_1^2) \qquad \text{(NRC Equation 3)}$$

3) An upper (high) rate of local sea level change shall be estimated by considering the modified NRC Curve III value, and combining these numbers with the local rate of vertical land movement. This scenario of high rate of local mean sea level rise exceeds the upper bounds of the IPCC estimates from both the 2001 and 2007 and also includes additional sea-level rise to accommodate the potential for rapid loss of ice from Antarctica and Greenland;

4) The sensitivity, risk, and uncertainty analysis were not conducted since it is not required for this designed and authorized project.
Sea Level Change Calculator
The local SLC chart and curve are calculated based on the online calculator provided by USACE. Both the USACE and NOAA curves and charts are calculated and presented in this report. The link to the online calculator is shown below:

http://www.corpsclimate.us/ccaceslcurves.cfm

EC 1165-2-212 and its successor ER 1100-2-8162 were developed with the assistance of coastal scientists from the NOAA National Ocean Service and the US Geological Survey. Their participation on the USACE team allows rapid infusion of science into engineering guidance. ETL 1100-2-1, Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation.

a) The rate for the "USACE Low Curve" is based on EC 1165-2-212 and its successor ER 1100-2-8162 . Use the historic rate of sea-level change as. ETL 1100-2-1, Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation. The historic rate used for this project is shown in the Figure A-18 below:

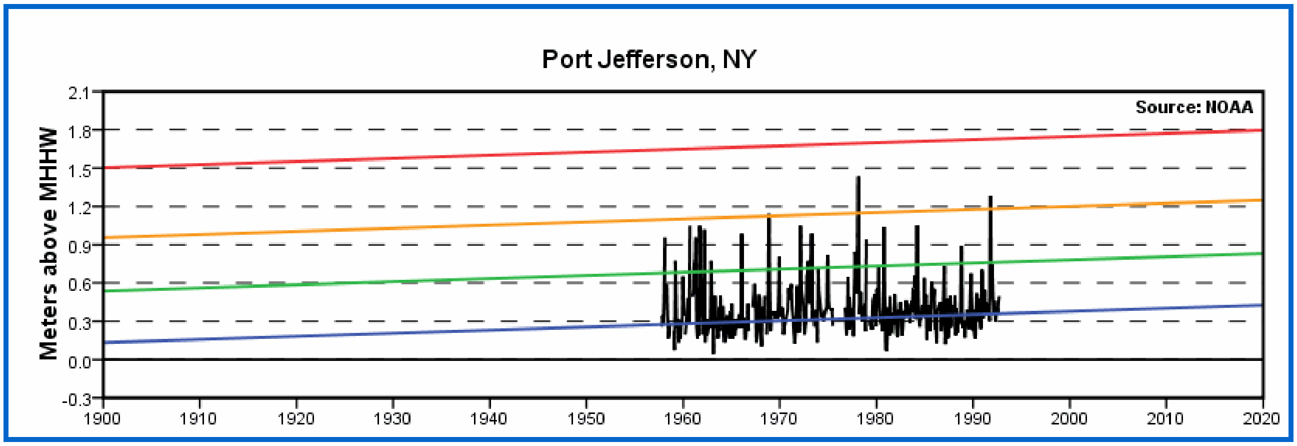


Figure A-18: Historic SLC Rate at Nearest NOAA Tide Gage Station

b) The rate for the "USACE Intermediate Curve" is computed from the modified NRC Curve I considering both the most recent IPCC projections and modified NRC projections with the local rate of vertical land movement added.

c) The rate for the "USACE High Curve" is computed from the modified NRC Curve III considering both the most recent IPCC projections and modified NRC projections with the local rate of vertical land movement added.

The three scenarios proposed by the NRC result in global eustatic sea-level rise values, by the year 2100, of 0.5 meters, 1.0 meters, and 1.5 meters. Adjusting the equation to include the historic GMSL change rate of 1.7 mm/year and the start date of 1992 (which corresponds to the midpoint of the current National Tidal Datum Epoch of 1983-2001), instead of 1986 (the start date used by the NRC), results in updated values for the coefficients (b) being equal to 2.71E-5 for modified NRC Curve I, 7.00E-5 for modified NRC Curve II, and 1.13E-4 for modified NRC Curve III.

The three local relative sea level change scenarios updated from EC 1165-2-212 (and its successor ER 1100-2-8162, Equation 2) are depicted in the Figure to the right of the table in the SLC calculator. A link to an Excel version of the calculator is below the table. The Excel version has a drop-down menu to select tide gauges. Below that is a direct link to the NOAA Tides and

Currents web site for the selected tide gauge. ETL 1100-2-1, Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation.

EC 1165-2-212, Equation 2: $E(t) = 0.0017t + bt^2$

This on-line Sea Level Change Calculator has several added features which are detailed in the User's Manual. You can plot both the USACE and NOAA curves in feet or meters relative to either NAVD88 or LMSL. The NPCC2013 projections for New York City are also available when the NOAA gauge, "The Battery" is selected. This calculator also develops the SLC curves between the user entered dates using equation #3 in ER 1100-2-8162

Local Calculated SLC Results

The local SLC chart and curves for both USACE and NOAA rates for year 2016 to 2116 in 5-year interval are estimated based on the on-line calculator and shown in the following table and chart.

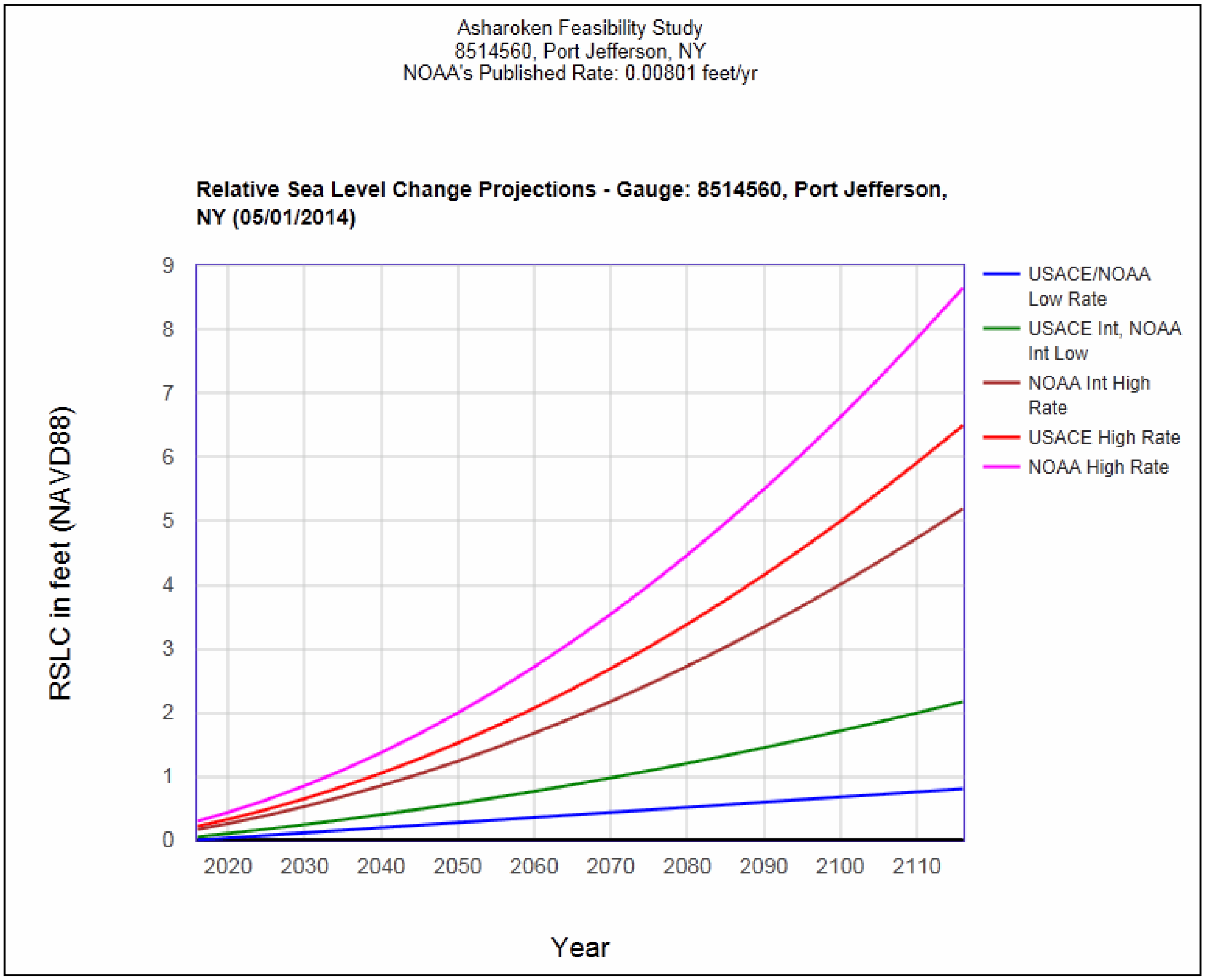


Figure A-19: Relative Sea Level Change Projections – Port Jefferson, NY

Estimated Relative Sea Level Change from 2016 To 2116 - Asharoken							
Based on NOAA Tide Gage 8514560, Port Jefferson, NY							
NOAA's Published Rate: 0.00801 feet/yr							
All values are expressed in feet relative to NAVD88							
Year	**NOAA Low**	**USACE Low**	**NOAA Int Low**	**USACE Int**	**NOAA Int High**	**USACE High**	**NOAA High**
2016	0	0	0	0	0	0	0
2020	0.03	0.03	0.05	0.05	0.09	0.11	0.14
2025	0.07	0.07	0.12	0.12	0.22	0.26	0.33
2030	0.11	0.11	0.19	0.19	0.36	0.43	0.56
2035	0.15	0.15	0.27	0.27	0.52	0.62	0.8
2040	0.19	0.19	0.35	0.35	0.69	0.83	1.07
2045	0.23	0.23	0.43	0.43	0.87	1.06	1.37
2050	0.27	0.27	0.52	0.52	1.07	1.31	1.7
2055	0.31	0.31	0.61	0.61	1.28	1.57	2.05
2060	0.35	0.35	0.71	0.71	1.51	1.85	2.42
2065	0.39	0.39	0.81	0.81	1.75	2.15	2.82
2070	0.43	0.43	0.92	0.92	2.01	2.47	3.25
2075	0.47	0.47	1.03	1.03	2.28	2.81	3.7
2080	0.51	0.51	1.15	1.15	2.56	3.17	4.17
2085	0.55	0.55	1.27	1.27	2.86	3.55	4.68
2090	0.59	0.59	1.4	1.4	3.17	3.94	5.2
2095	0.63	0.63	1.52	1.52	3.5	4.35	5.76
2100	0.67	0.67	1.66	1.66	3.84	4.78	6.34
2105	0.71	0.71	1.8	1.8	4.2	5.23	6.94
2110	0.75	0.75	1.94	1.94	4.57	5.7	7.57
2115	0.79	0.79	2.09	2.09	4.95	6.19	8.23
2116	0.8	0.8	2.12	2.12	5.03	6.29	8.36

Recommendations

The following local Sea Level Change (SLC) rates are recommended for use:

The extrapolation of historical rate of +0.4 ft/50 years or 0.8 ft/100 years with 95% confidence is used for project planning, design, and analysis. Sensitivity, Risk and Uncertainty analyses will be conducted to determine how sensitive the recommended designs are to these various rates of future local mean SLC, how this sensitivity affects calculated risk, and what design of operations and maintenance measures can be implemented to minimize adverse consequences while maximizing benefits. Both the USACE intermediate and high rates in future 50 and 100 years will be used for sensitivity, risk & uncertainty analysis.

3.0. EXISTING SHORELINE CONDITION

3.1 Existing Shoreline Characteristics and Typical Beach Profiles
the project shoreline is facing northeast on Long Island Sound located between Eatons Neck Point to the northwest and National Grid power plant cooling water intake lagoon to the southeast. The majority of the project shoreline is populated with residential development with year-round houses built to the edge of water. Typical beach profiles range from residential homes built behind dunes, to buildings sitting on bulkheaded foundations next to the beach berm. An emergency erosion control structure approximately 900 ft long is located within the northwest boundary of the project shoreline. The structure was built in 1997 and is composed of steel sheetplie seawall with riprap toe revetment.

The study shoreline is approximately 2.4 miles in length with relatively mild offshore slope, steep foreshore slope and low sloping berm. Average foreshore beach widths range from 50 to 100 ft backed with dune or bulkhead. The project shoreline is divided into four typical reaches based on beach profile types and the waterfront structural characteristics. The four reaches are analyzed individually for the with and without project coastal processes and for both existing and future conditions and for economic analysis of the alternatives considered. Reaches are shown in Figure A-20. Typical beach profiles for the four reaches are shown in Figures A-21 to A-24 and described as follows:

Reach 1a. This reach starts from the western border of project shoreline near the intersection of Bevin Road and Asharoken Avenue, extending east approximately 900 ft along the waterfront shoreline. A stone groin is located to the eastern limit of this reach. This shoreline was washed over during the 1992 northeaster and was since rebuilt to a 15 year design life erosion control structure under the authority of Section 103 Small Shore Protection Projects. The Section 103 design includes a steel sheet pile at +11.5 NAVD (+12.5 ft NGVD) crest elevation, a riprap toe protection and an approximately 20 ft wide backfill. The road elevation behind this reach is approximately 8 ft above NAVD (9 ft above NGVD). Beach profile of this stretch of shoreline is characterized by a relatively steep foreshore slope and a narrow berm averaging +5 ft NAVD (+6 ft NGVD) elevation in front of the steel bulkhead. The foreshore slope is approximately 1 vertical on 8 horizontal down to elevation -7 ft NAVD (–6 ft NGVD). Asharoken Avenue is located landward of the backfill. Beach widths in this reach range from 0 ft to 20 ft measured from the MHHW (+2.9 ft NAVD, +3.9 ft. NGVD) shoreline to bulkhead toe line with riprap protection. The offshore slope is approximately 1 vertical on 100 horizontal. The typical beach profile for this reach is described in Figure A-21.

Reach 1b. This reach extends approximately 5,300 ft along the shoreline from the stone groin east to Duck Island Lane. The waterfront along this stretch of shoreline is a typical dune and beach formation with approximately +14.5 ft NAVD (+15.5 ft NGVD) dune crest, sloping berm, steep foreshore slope, and mild offshore slope. Asharoken Avenue is located landward of the dune with private properties located further landward of the road. The average ground elevation (of Asharoken Ave.) behind the dune is approximately 11 ft above NAVD (12 ft above NGVD). The 50 ft wide sloping berm changes from+9 ft NAVD (+10 ft NGVD) at the toe of dune down to +3 ft NAVD (+4 ft NGVD). Foreshore slope along this reach is approximately 1 vertical on 8 horizontal. The average beach width from base of dune to -1 ft NAVD (0 ft. NGVD) shoreline is approximately 100 ft. Offshore slope is approximately 1 vertical on 100 horizontal. The typical Beach profile within this reach is described in Figure A-22.

Reach 2a. This 5,000 ft reach, extending from Duck Island Lane (located at approximately the first house on the waterfront) east to the last house on waterfront (approximately 1,200 ft west of west jetty) is characterized by waterfront properties protected with timber bulkheads at an average of approximately +13 ft NAVD (+14 ft NGVD) crest elevation. The average ground elevation behind the bulkhead is approximately +12 ft NAVD (+13 ft NGVD). There is a stretch of approximately 800 ft shoreline without bulkhead but protected with dune crest at an average of +14 ft NAVD (+15 ft NGVD). Typical beach profile in this reach is comprised of a relatively low berm and a steep forshore slope at 1 vertical on 8 horizontal. The average seaward berm elevation stands at +3 ft NAVD (+4 ft NGVD) and gently slopes up to +11 ft NAVD (+12 ft NGVD). Riprap toe protection fronting the bulkhead is scattered along the entire length of the reach. The average beach width from bulkhead to MHHW (+2.9 ft NAVD, +3.9 ft. NGVD) shoreline ranges between approximately 0 to 120 ft. Offshore slope is approximately 1 vertical on 100 horizontal. The typical Beach profile in this reach is described in Figure A-23.

Reach 2b. This reach extends approximately 1,200 ft along the shoreline from eastern limit of private houses (approximately 1,200 ft west of the west jetty) to the west jetty of the intake lagoon. This stretch of shoreline is a typical dune and beach formation with approximately +16 ft NAVD (+17 ft NGVD) dune crest, sloping berm continuous with a steep foreshore slope to elevation -3 ft NAVD (–2 ft NGVD) and a mild offshore slope. The average ground elevation behind the dune is approximately 13 ft above NAVD (14 ft above NGVD). The foreshore beach slope is approximately 1 vertical on 8 horizontal. Offshore slope is approximately 1 vertical on 100 horizontal. Seaward dune slope is approximately 1 vertical on 3 horizontal. The average beach width from base of dune to MHHW (+2.9 ft NAVD, +3.9 ft. NGVD) shoreline is approximately 40 to 60 ft. The typical Beach profile within this reach is described in Figure A-24. The typical beach profile characteristics and landside conditions of the four reaches are summarized in Tables A-14 and A-15.

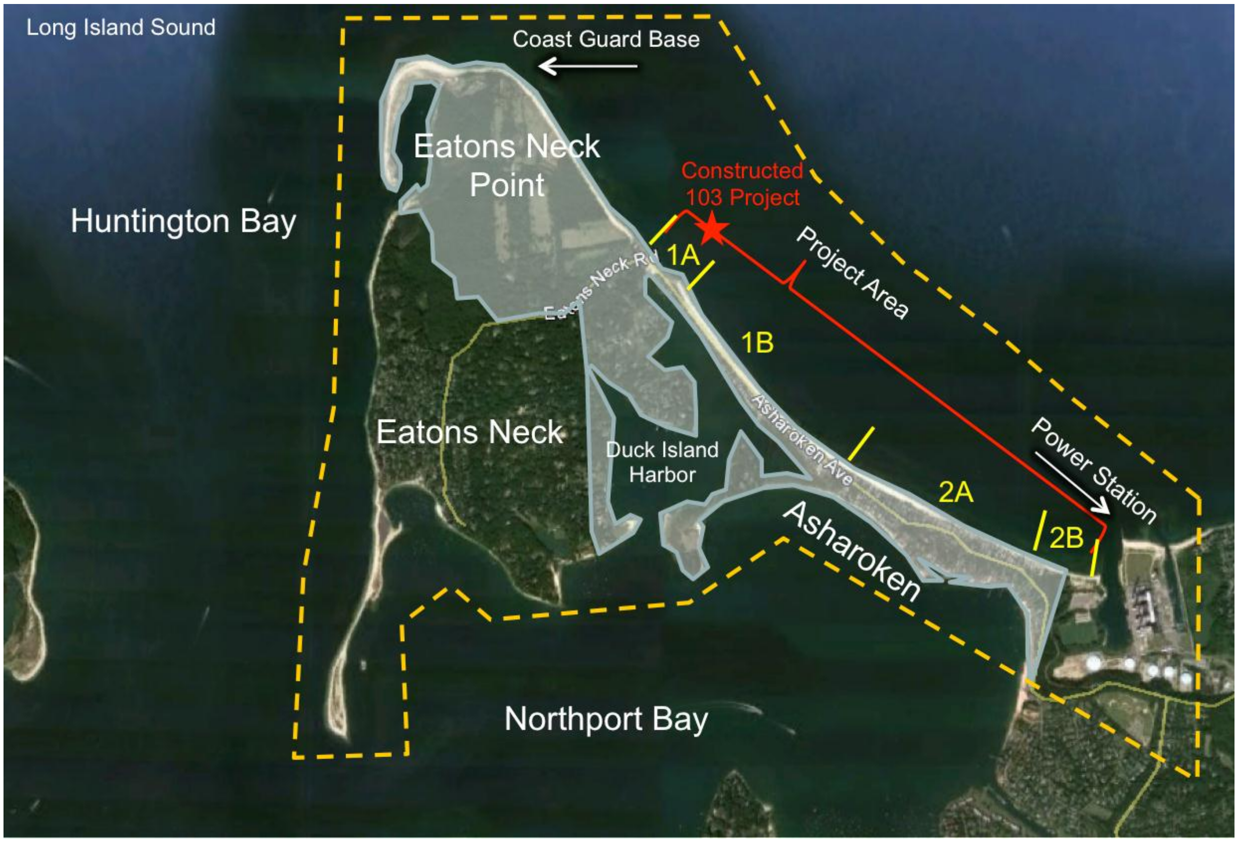


Figure A-20: Project Reaches

Table A-14 Beach Profile Characteristics

Reach No.	General Location	Approx. Length (ft)	Dune Elevation (ft NGVD)	Berm Elevation (ft NGVD)	Dry Beach Width to MHHW (ft)	Foreshore Slope (x V on y H)	Offshore Slope (x V on y H)
1a (2006)	Bevin Road (0+00) to Rock Groin (9+00)	900	12.5 (Bulkhead)	+6	0 to 20	1 on 8	1 on 100
1b (2001)	Rock Groin (9+00) to Duck Island Lane (62+00)	5,300	15.5	+4 to +12	80	1 on 8	1 on 100
2a (2001)	Duck Island Lane (62+00) to 1,200' West of West Jetty (112+00)	5,000	+14 (Bulkhead) +15 (Dune app.1,000 ft)	+4 to +12	0 to 120	1 on 8	1 on 100
2b (2001)	1,200' West of West Jetty (112+00) to West Jetty (124+00)	1,200	+17	+8	40 to 60	1 on 8	1 on 100

Table A-15 Landside Condition

Reach No.	General Location	Approx. Length (ft)	Structure	Average Dune Crest Widths (ft)	Avg. Elevaton At Crest of Dune/Str. (ft NGVD)	Avg. Elevation Behind Dune/Str. (ft NGVD)
1a	Bevin Road (0+00) to Rock Groin (9+00)	900	Dune fronted with Bulkhead	15	+12.5	+9
1b	Rock Groin (9+00) to Duck Island Lane (62+00)	5,300	Dunes	0-5	+15.5	+12
2a	Duck Island Lane (62+00) to 1,200' West of West Jetty (112+00)	5,000	Bulkhead/Dune (approx. 1,000')	0-5	0.93	13
2b	1,200' West of West Jetty (112+00) to West Jetty (124+00)	1,200	High Dunes	0-5	17	14

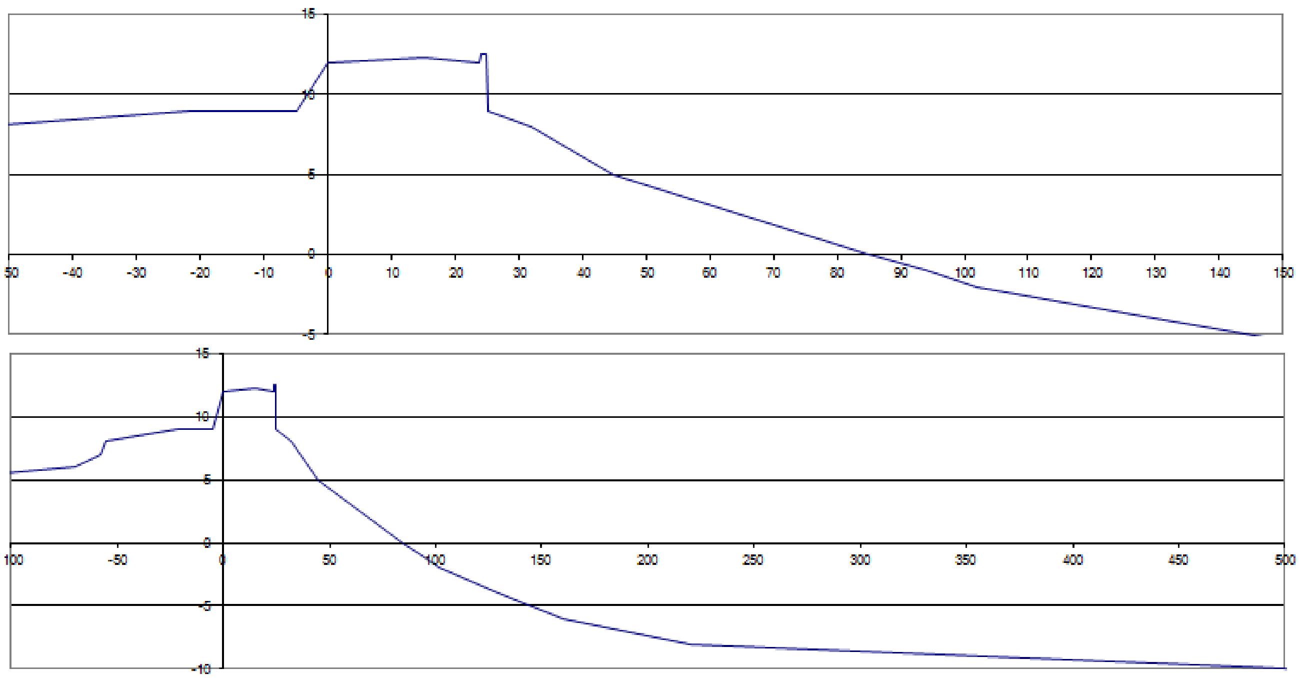


Figure A-21 Typical Profile for Reach 1a Shoreline

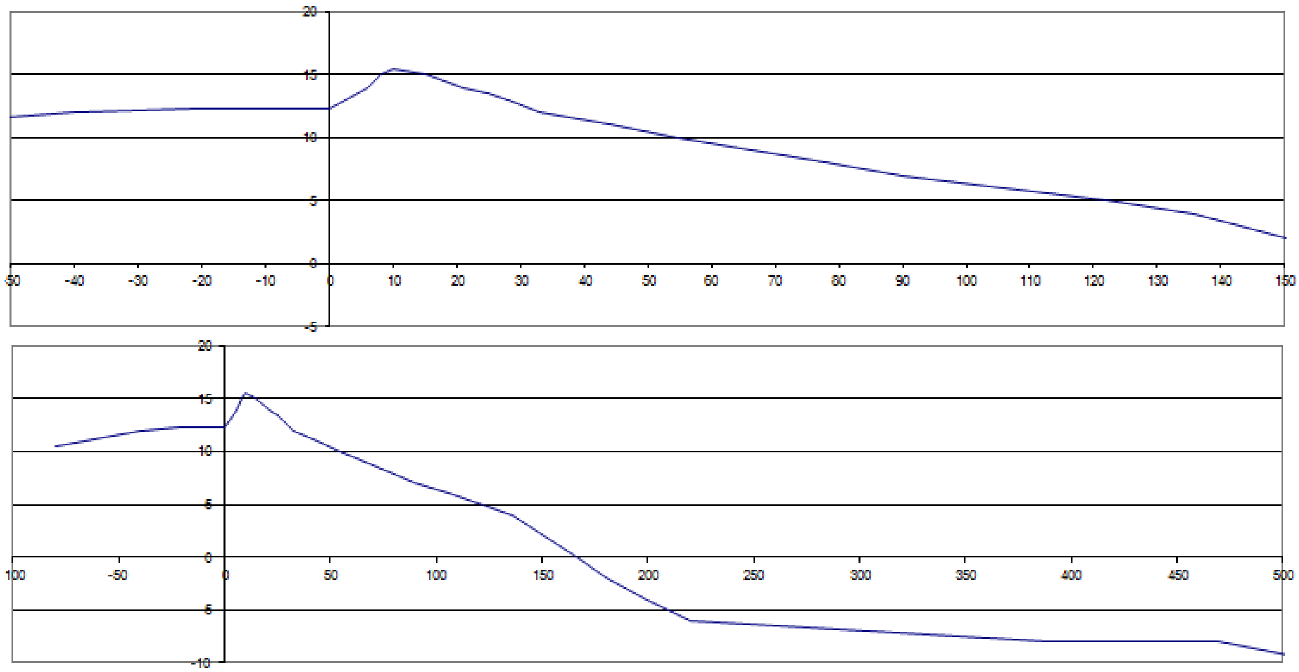


Figure A-22 Typical Profile for Reach 1b Shoreline

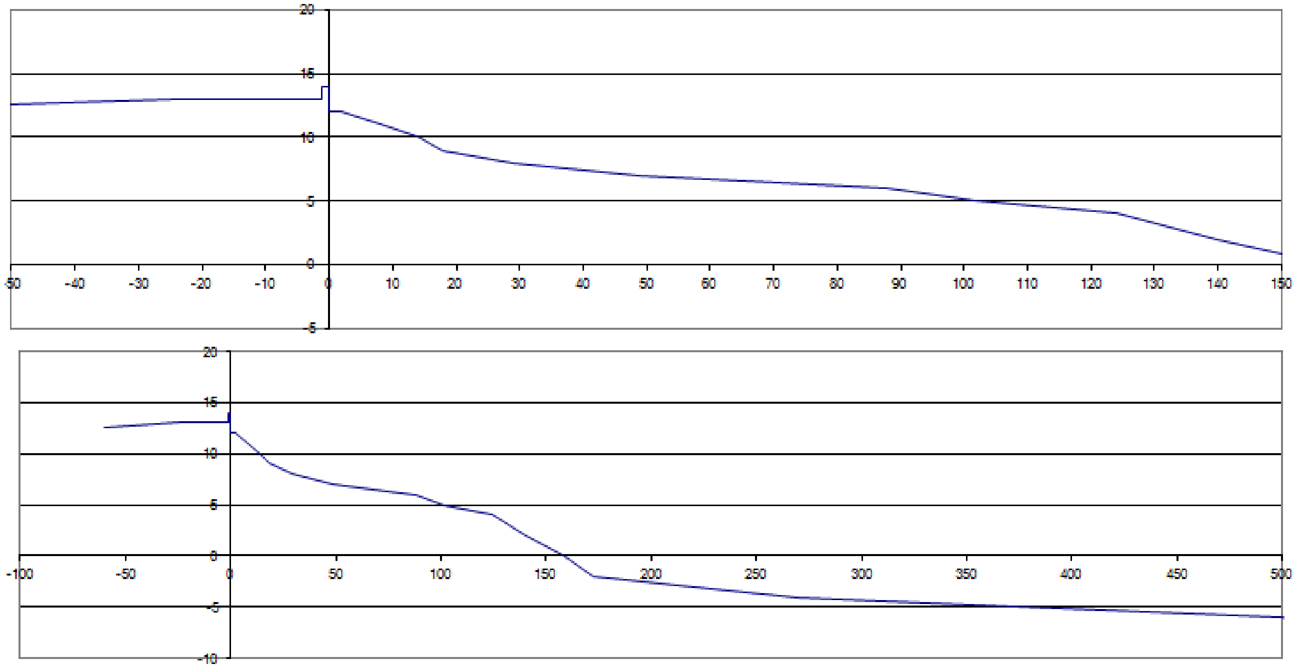


Figure A-23 Typical Profile for Reach 2a Shoreline

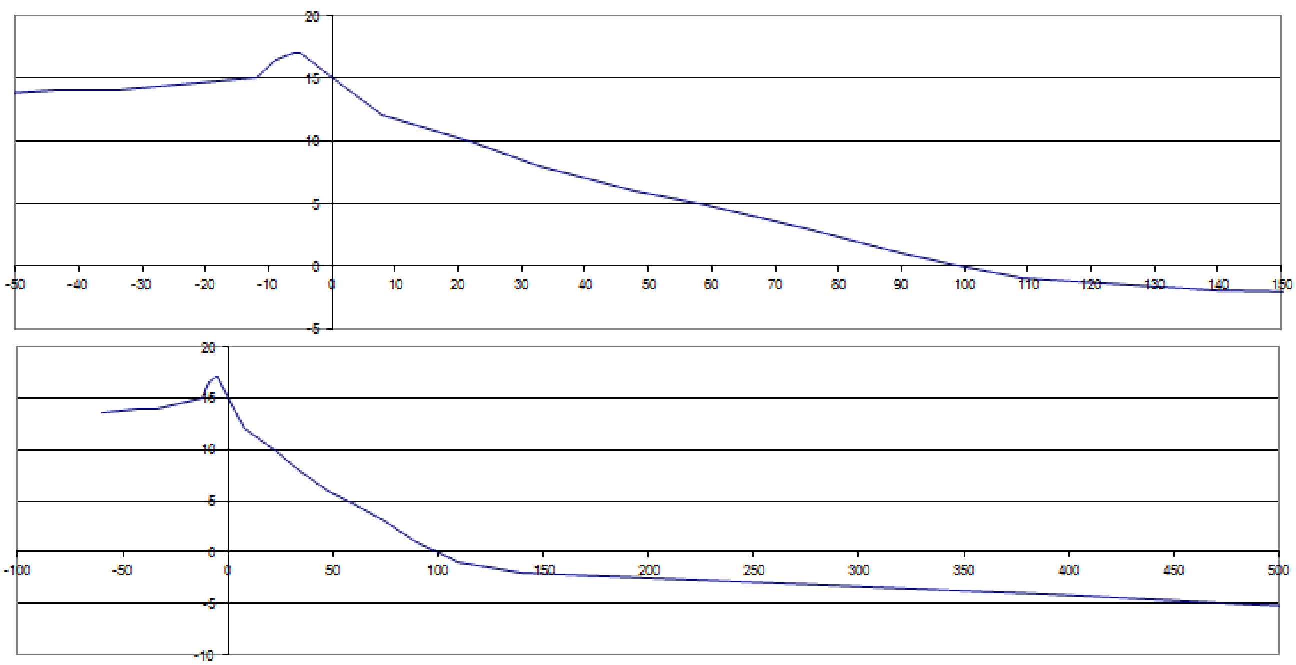


Figure A-24 Typical Profile for Reach 2b Shoreline

4.0. SEDIMENT TRANSPORT ANALYSIS

4.1 General

Asharoken Beach is a narrow section of land in the Town of Huntington on the north shore of Long Island, connecting Eaton's Neck and the Village of Asharoken with the Village of Northport. In 1929 Metropolitan Sand and Gravel Co. filed with Corps of Engineers for a permit to construct two jetties into Long Island Sound at the western corner of their property located east of the Village border. The jetties were constructed between 1931 and 1932 with a lagoon (a.k.a. Northport Basin) and inlet channel dredged shortly thereafter. In 1935, Metropolitan requested and received a 3-year extension for the use and maintenance of the facility and was extended periodically until March 1968 when LILCO requested a change in the permit to construct a power plant adjacent to Northport Basin. As part of LILCO's plant construction, the existing barge jetties were rehabilitated into permanent quarrystone and concrete riprap jetties. In addition, a water-cooling pond was constructed to the east of the power plant with cooling water effluent discharged directly into the Sound via a weir structure on the beach.

Since the construction of jetties and navigation channel, the shoreline east of the basin has accreted while the shoreline west of the jetties recessed from its natural position. Approximately 6,000 feet shoreline west of the jetties has experienced increased beach erosion as a combined effect of disrupted upstream sediment supply and storm erosion activities. Construction of groins and seawall along the upstream Crab Meadow shoreline in the 1950's and early 1960's further reduced natural sediment source. To counter the erosion, timber bulkheads were constructed along the eroding stretch by homeowners and five interlocking groins were placed along the eroding stretch by New York State DPW in 1956. A concrete and stone groin was also constructed at the northwest portion of the shoreline in 1952. An approximately 840,000 cubic yards of sand nourishment was placed on the beach from a borrow area located approximately 1,000 feet directly offshore in the 1960's. Since the construction of the power plant facility, LILCO (now National Grid) continued bypassing shoaling material dredged from the boat channel and basin depositing it on the beach west of the jetties. Based on the dredging records, the average bypassing rate in the period from 1962 to 2001 is approximately 10,000 cubic yards/year.

Despite the various beach erosion control activities, Asharoken beach shoreline continued to erode. The causes of erosion are continuously debated between experts representing Village of Asharoken and the Power Plant and a court decision is pending. Understanding the sensitivity of this issue and as a neutral party, the US Army Corps of Engineers (USACE) has taken this study independently with all analyses based on existing public and established data including USGS shoreline positions, NOS bathymetric contours, rectified and geo-referenced aerial photos, historical dredging and beachfill records, shore improvement histories, and records of daily cooling water discharge, channel dredging and beachfill records. All available data were analyzed with USACE developed Coastal Engineering Design Modeling System (CEDAS) and Surface Modeling System (SMS). Models applied include SBAS for sediment budget analysis, ADCIRC, STWAVE for nearshore wave-current effect of sediment transport, and GENESIS for shoreline responses in the vicinity of the jetties. Numerical model outputs were interpreted with engineering judgment. A detailed sediment transport study was performed by Offshore & Coastal Technologies, Inc. (Reference 2). Study methodologies and results were reviewed by independent local and academic specialists and consensus conclusions were made and summarized below.

Study Methodology.

A sediment budget was developed for the Asharoken coast between Crab Meadow (east) and Eatons Neck (west) for the periods 1962 to 2001 and 1976 to 2001. Ten sediment budget cells were established at coastal structure boundaries and where shoreline orientation changes are significant as shown in Figure A-25.

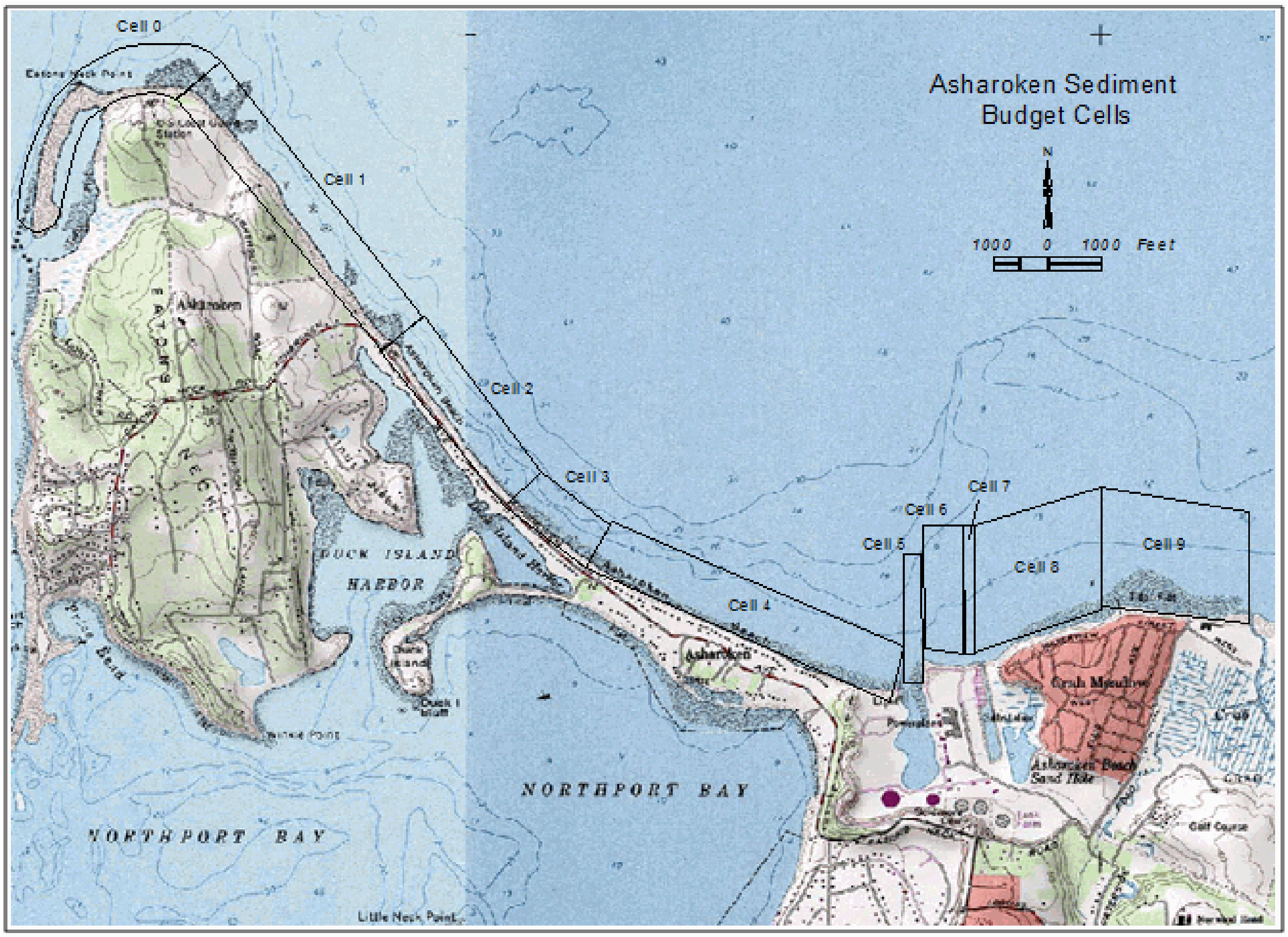


Figure A-25 Sediment Budget Cells

Historical shoreline positions were established with U.S. Coast and Geodetic Survey (USGS) topographic maps (1885/86, 1917, and 1931) and rectified aerial photography (1961, 1962, 1972, 1976, 1983, 1988, 1994, and 2001). Bathymetry data for the periods 1967 (acquired from the National Ocean Service) and 2001 (collected by the New York District through Gahagan and Bryant) were used to quantify sand volume changes from the high-water line offshore to the 20-ft (NGVD) depth contour. Sand volume change at Eatons Neck spit (south of Eatons Neck Point) was determined using bathymetry data for the periods 1931 and 1990. Sand volume change in the offshore region north of Eatons Neck Point was determined using bathymetry data from 1967 and 1990. Information from a 10-yr wave hindcast simulation was used to estimate potential longshore sand transport rates throughout the study area. Permanent offshore sand losses resulting from high-energy events were determined by quantifying sand accumulation seaward of the 20-ft (NGVD) depth contour from the bathymetric change surface.

An investigation of sediment transport processes local to the power plant outfall, a nearshore finite element hydrodynamic model and sedimentation analysis was employed. This analysis was performed to indicate any fine scale processes that impact the sediment budget sources and sinks, including the effects of cooling water discharge, the stone jetties and the lagoon. A shoreline change model, GENESIS, validated for the time period 1994-2001, was used to simulate a hypothetical future condition with the Key span plant jetties removed.

4.2 Historical Shoreline Evolution. Shoreline changes prior to the completion of Northport Basin and jetties is defined with the comparison of 1885/1886 and 1931 aerial photos. During this period the net westward sediment transport on Asharoken Beach was uninterrupted and the study shoreline was stable with minor beach erosion. The post-jetty construction effect is illustrated by comparing aerial photos in the period of 1932 to 2001. A shoreline differential developed since the completion of jetties, with shoreline accretion to the east and erosion to the west. The western shoreline experienced continued beach erosion on the order of 2 ft/year even after receiving periodical bypassing at an average rate of 10,000 cy/year in the period 1962-2001 and a one-time beachfill of 840,000 cubic yards in the 1960’s placed in the middle portion of Asharoken Beach. The approximately 6,000 ft shoreline immediately west of the jetties experienced higher erosion rates due to combined effect of sediment supply deficit and storm activities; while the western 6,000 ft shoreline experienced less erosion due to continued but reduced supply of littoral material from the eastern Asharoken Beach shoreline.

4.3 Sediment Budget. Sediment budget analysis was performed to quantify historic and existing transport rates. The 1976-2001 sediment budget, shown in Table A-16 and Figure A-26, represents recent sediment transport pattern at the project shoreline and is used for transport rate estimates. This sediment budget excludes the effect of 840,000 cy beachfill (considered to be in-compatible to native beach sand) placed in the mid-1960’s, but includes the current and ongoing sand bypassing by the power plant. This sediment budget provides several useful key erosion and transport rates summarized as follows (note that all rates are rounded to thousands to reflect the degree of confidence):

- Based on the 1976-2001 sediment budget, the erosion rate on the eastern shoreline immediately west of the jetties (Cell 4) is eroding at approximately 10,000 cy/year after the 10,000 cy/year bypassed from upstream by Keyspan, a total erosion rate of 20,000 cy/year;
- The shoreline in the middle of Asharoken Beach (Cells 3 and 2) are relatively stable, experiencing minor shore erosion at approximately 4,000 cy/year;
- Beach erosion increases along the western shoreline (Cell 1) at approximately 18,000 cy/year. The 900 ft Section 103 shoreline experienced higher erosion due to the interruption of sediment supply by a concrete/stone groin located just east of this section;
- The sand spit just west of Eaton’s Neck Point (Cell 0) is growing at a rate of 16,000 cy/year, representing net sediment transport into this cell less sediment lost offshore;
- The sediment supply from upstream shoreline is approximately 15,000 cy/year (Cell 8 to Cell6), with 10,000 cy/year being bypassed downstream (Cell 4) and approximately 5,000 cy/year retained in Cell 6 or lost offshore;

Table A-16 Sediment Budget 1976-2001

Cell Number	1	2	3	4	5	6	7	8	9
Input (W-E) (+)	3,000	21,000	21,000	22,000	2000	0	20,000	20,000	23,100
Output (W-E)(-)	-21,000	-21,000	-22,000	-2,000	0	-20,000	-20,000	-23,100	-23,800
Input (E-W) (+)	39,000	37,500	39,100	13,100	11,100	34,700	34,700	37,500	38,800
Output (E-W)(-)	-46,400	-39,000	-37,500	-39,100	-13,100	-11,100	-34,700	-34,700	-37,500
Offshore	-5,000	-1,000	-1,300	-4,800	0	0	0	0	
Onshore	12,500	0	0	0	0	0	0	0	
Placement									
Residual	**-17,900**	**-2,500**	**-700**	**-10,800**	**0**	**3,600**	**0**	**-300**	**600**

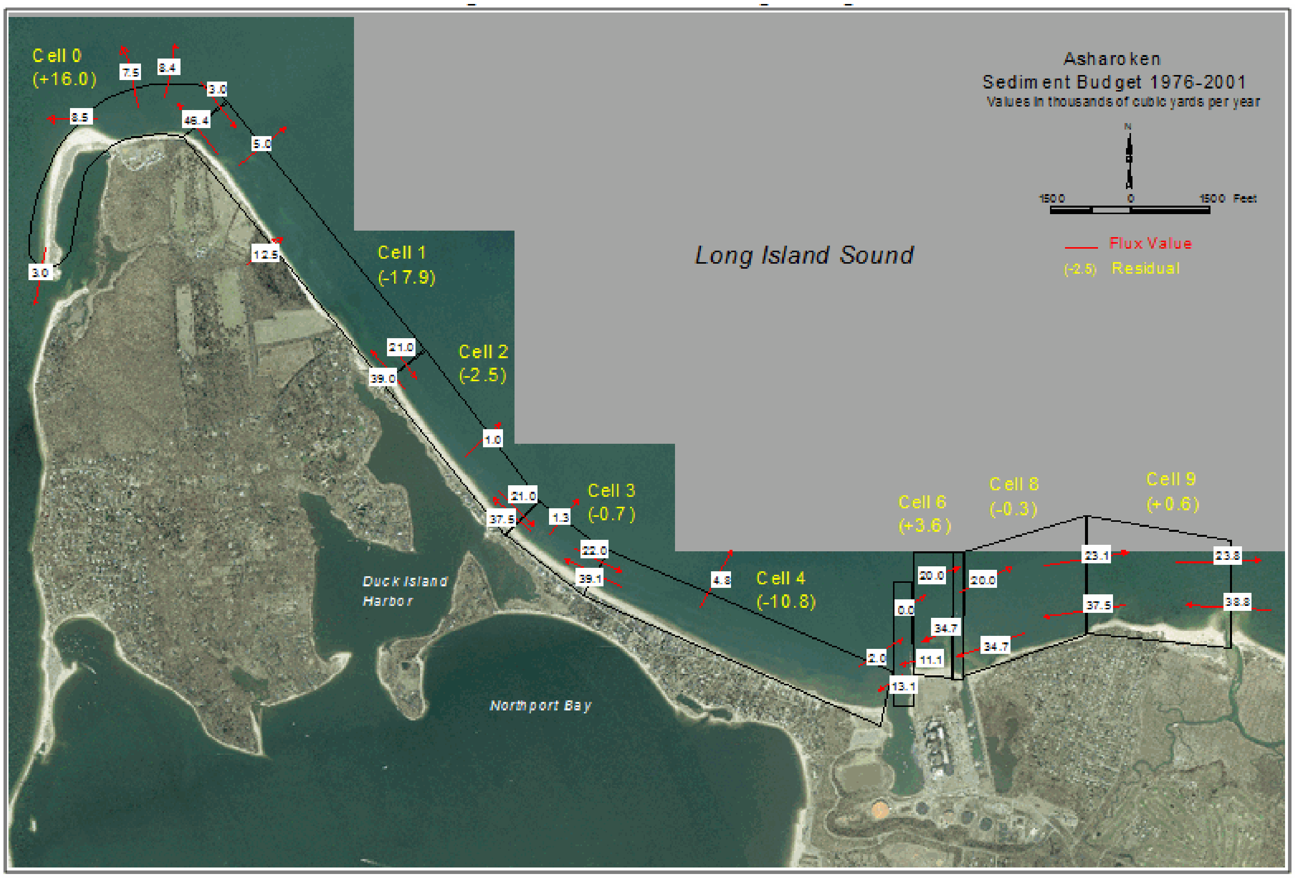


Figure A-26 Sediment Budget Diagram

<u>4.4 Future Renourishment.</u> Periodic nourishment is necessary to stabilize the Asharoken Beach during the 50-year project evaluation period. The nourishment (deficit) volume is estimated at 20,000 cubic yards per year based on the existing condition beach erosion rate on the eastern half of the Asharoken Beach shoreline. The estimated volume includes 10,000 cy/year being bypassed by the power plant periodically plus the 10,000 cy/year annual erosion rate. Beach nourishment will be performed in a 5-year interval and can be synchronized with ongoing power plant bypassing operations. The nourishment volume should be placed evenly on the eastern half shoreline and is expected to transport naturally downstream, feeding the downstream shoreline. Note that historically the available upstream source is approximately 15,000 cy/year, with approximately 5,000 cy/year being retained east of the jetties, therefore, the balance of the nourishment volume (10,000 cy/yr) will be need to be supplemented from a borrow source. Of

the 15,000 cy/year upstream source, 10,000 cy/year is being bypassed and 5,000 cy/year could potentially be tapped from the sediment fillet accumulated at east of east jetty (Cell 6), however the fillet material is far from ideal as nourishment material (too fine); therefore it is recommended that the full 10,000 cy/yr be obtained from an alternate borrow source.

4.5 Effect of Cooling Water Effluent Flow. The sediment transport pattern in the vicinity of the cooling water outfall was simulated with the application of an advanced circulation model (ADCIRC) combined with a fine-scale nearshore wave model (STWAVE). Long-term depth changes were calculated to determine the direction and distance of sediment movement. Results of model analysis indicates that sediment in the surf zone near the outfall would be carried offshore to a maximum 600 ft distance during normal operating conditions and approximately a maximum 1,800 ft during storm conditions, which might be lost permanently. The material carried up to 600 ft offshore within the active surf zone would continue its alongshore transport pattern and be deposited in the boat channel or carried inside the boat basin, to be eventually bypassed onto Asharoken Beach via dredge/disposal operation. Littoral material naturally bypassing the jetties and returned to the Asharoken Beach is minimal as evidenced by a lack of offshore bar formation across the jetties (Figure A-27). As a result, installation of an effluent flow diffusion pipe (not thermal diffusion pipe) to divert flow offshore would be ineffective to enhance natural bypassing or to significantly reduce sediment loss offshore.

4.6 Effect of Jetties. The shoreline response model GENESIS was applied to investigate the linear extent of the jetties influence on the study shoreline. The GENESIS model was validated for the time period 1995-2001 based on wave records in this period and measured shoreline positions. A hypothetical jetty removal was configured using the 2001 shoreline as a base to predict the future without jetties shoreline for a 10-year period. Model results indicate a restored shoreline similar to the pre-1932 condition. The downdrift shoreline affected by the jetties show an impact over and approximately 6,000 ft distance. However, it requires a complete removal of jetties and closing of Northport Basin for restoration to a straight shoreline.

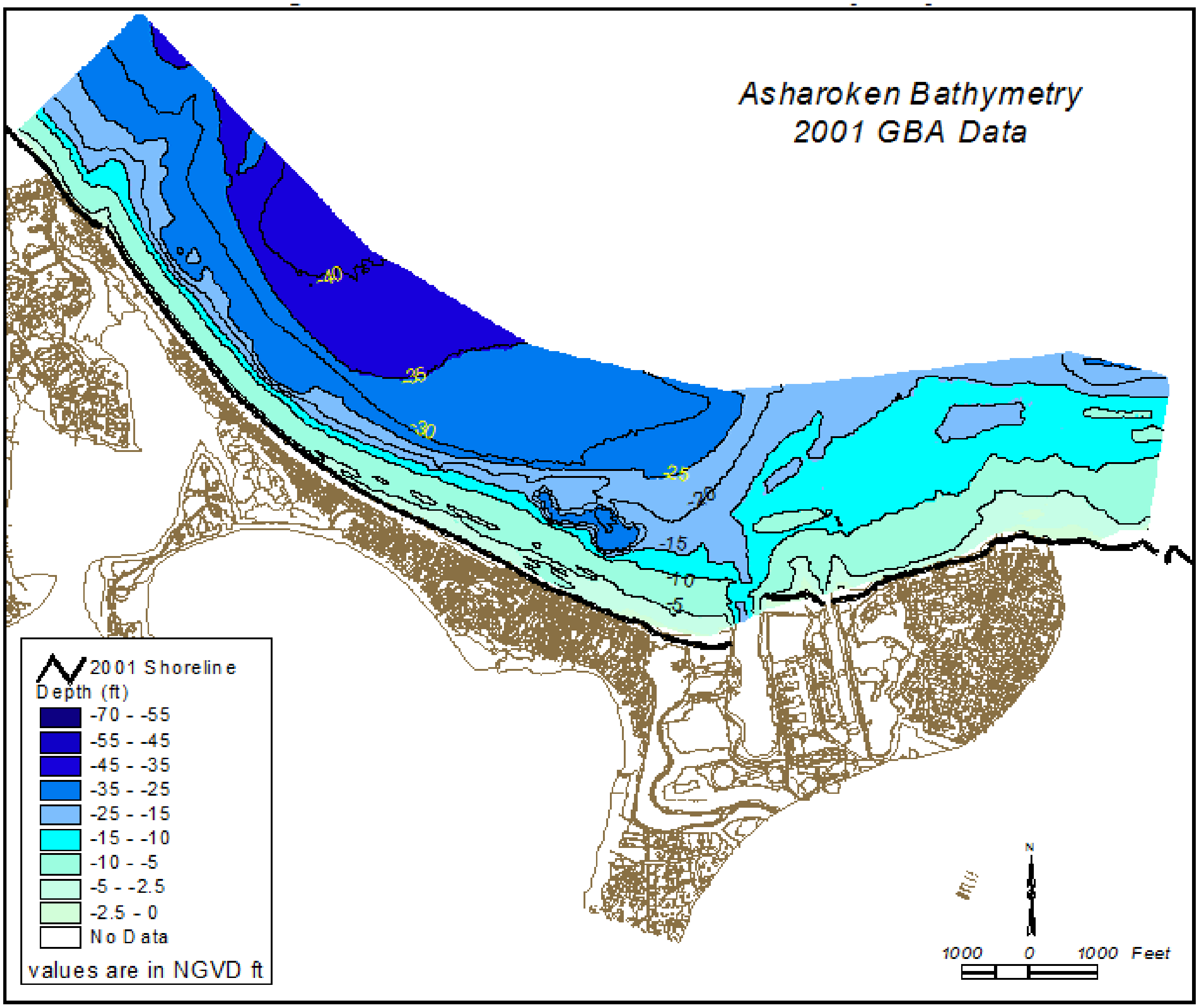


Figure A-27 Asharoken Beach offshore Bathymetry

5.0 BORROW SOURCE

5.1 General

The objective of the borrow source investigation was to identify and delineate sources of sand borrow material for use as design fill and nourishment for Asharoken beach erosion control project. Beachfill sediments of suitable grain size and distribution were sought and present in sufficient volume, within a reasonable distance from the project shoreline.

Borrow source investigations included both offshore and upland sources. The grain size distributions and available volumes of the potential borrow sources were obtained from samples collected at the upland stockpile and offshore vibracore samples collected for this study. The grain sizes were compared with typical native beach sand size distribution taken from the project site to determine the compatibility of the borrow material. Those suitable borrow sources were checked to determine if volume at the borrow site would be sufficient for the beachfill project. This section summarizes the methodologies and procedure to identify potential borrow sources and to recommend a best cost/efficient source. Appendix B – Borrow Source Investigations presents detailed information of the study.

5.2 Existing Beach Sand Evaluation

Native beach sediments must be matched with similar grain size of borrow material so that the beach fill (initial and renourishment quantities) will reasonably endure over the required project life by being similar to more stable grain size distribution. In order to determine this representative sediment, samples of native beach were collected and analyzed for grain size distribution. Beach sample parameters derived from the grain size distribution (GSD) curves are then compared mathematically using the methodology from the 1984 Shore Protection Manual with the GSD curves of the borrow area sediments to determine the adjusted fill factor (Ra) and stability factor (Rj) of potential borrow sediments. Beach sediment samples were taken in June 2002 along six selected profile lines evenly (approximately 2,000 ft) distributed along the entire project shoreline. A total of nine samples were taken along each profile line:

- at three locations along the berm at back-berm (BB), mid-berm (MB) and berm-crest (BC) (locations 1, 2, and 3, respectively);
- at three locations in the inter-tidal zone at Mean High Water (MHW), Mean Tide Level (MTL), and Mean Low Water (MLW) (locations 4, 5, and 6, respectively);
- and at three offshore locations along the profile lines at water depths -3.0 ft. MLW, -9.0 ft MLW and -15.0 ft. MLW (locations 7, 8, and 9, respectively.

The mean grain size diameter, M_{phi}, is defined by the following formula:

$$M_\phi = \frac{\phi_{84} + \phi_{50} + \phi_{16}}{3} \qquad \text{(SPM 84 Equation 4-3)}$$

$\phi =$ grain size defined in “phi” units

Where phi_{84} is the phi transformation of the percentile at which 84 percent of the particles on the grain size distribution curve have larger diameters, and 16 percent have diameters finer than the diameter of the 84th percentile. Whereas, phi_{16} and phi_{50} are the phi value of the 16^{th} and 50^{th} percentile, similarly determined. The mean diameter is used to categorize the beach material into its appropriate component. The standard deviation, $Sigma_{phi}$, is a measure of the natural sorting of the sample. It is defined by:

$$\sigma_{\phi} = \frac{\phi_{84} - \phi_{16}}{2} \qquad \text{(SPM 84 Equation 4-4)}$$

$\sigma_{\phi} = Sigma_{\ phi}$ = Standard Deviation in phi units

Where phi_{84} and phi_{16} are the associated percentile phi values as defined above. The mean and standard deviation of the native beach and bottom samples taken at the six profile stations are summarized in Table A-17. In general, the bottom (sub aqueous) samples are finer than the samples on beach. The average value of all samples (excluding samples at MLW-15) is shown in Table A-18. As shown in Table A-18, the mean diameter for the existing beach material is found to be 0.9 mm, classified as "coarse sand" (SPM 1984). The standard deviation for the existing beach material is found to be 1.94 in phi units. A value of 0.5 indicates uniform sorting of beach materials. The existing beach sand at Asharoken Beach therefore is classified as well mixed material. The native beach sand grain size distribution curve representing a mathematically mixed composite grain size distribution is shown in Figure A-28.

Table A-17

Average Values of the Asharoken Beach Samples by Profile Line

(Sample 9, -15Ft. excluded from average, assuming beach fill is above this depth)

Line No	**phi 16 (ϕ_{16})**	**phi 50 (ϕ_{50})**	**Phi84 (ϕ_{84})**	**Mean (ϕ)**	**Mean (mm)**	**Standard Deviation (ϕ)**
1	-1.13	0.73	1.63	0.41	0.75	1.38
2	-2.45	0.46	1.31	-0.23	1.17	1.88
3	-2.23	-0.10	1.41	-0.30	1.23	1.82
4	-2.05	0.94	1.94	0.28	0.83	1.99
5	-1.68	1.11	2.23	0.56	0.68	1.96
6	-1.96	0.62	1.95	0.20	0.87	1.95

Table A-18

Average Beach Sample GSD

Mean (ϕ)	0.15
Mean (mm)	0.90mm
Standard Deviation (ϕ)	1.94

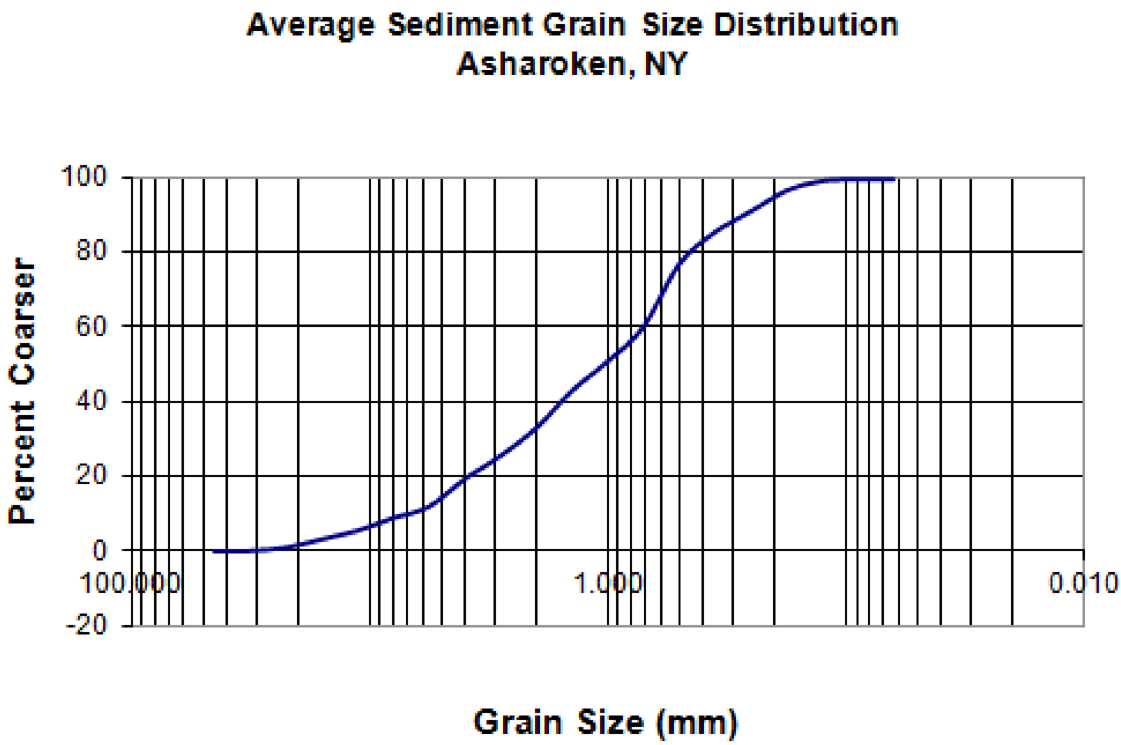


Figure A-28 Existing Beach Grain Size Distribution

5.3 Borrow Source Investigations

Several potential borrow sources were investigated to determine the suitability of the borrow material for Asharoken. The suitability of borrow source includes compatibility to native beach grain size, amount of volume available, environmental/permit considerations, and distance to the project site. Several potential borrow sites located both offshore and upland were investigated based on the existing and recent collection of boring logs, seismic maps, and samples collected at various upland sites. A detailed discussion of the procedures, methodology, and data analysis is included in Appendix B and the Borrow Source Investigations Report by Alpine (Reference 3). The following offshore and upland sites, which are reasonably close to the project site, were short-listed and investigated with more details.

Potential Upland Borrow Sources

Seven primary suppliers were identified and contacted as possible upland source sites for a beach nourishment project at Asharoken Beach on the north shore of Long Island. These sources are summarized in Table A-19:

Table A-19 Upland Sources

Potential Upland Sources	Location	Discussion	Comments
Empire Sand & Stone	Westbury, NY	Not sufficient Quantity Available	
European Mason Supply, Inc.	Kings Park, NY	No longer accepts large contracts	Previously Used but no longer available
Horan Sand & Gravel Corp.	Syosset, NY	Excavating area near Central Islip Not Sufficient Quantity	Collected 5 samples & took 3 pictures
Hubbard Sand & Gravel, Inc.	Bay Shore, NY	Excavating at Mt. Sinai for Golf Course – has 1 M cu. yd. available.	Collected 5 samples & took 3 pictures
Ranco Sand & Stone	Manorville, NY	Quarry in business for 30 years has sufficient quantity	Collected 5 samples & took 3 pictures
Sand, Stone, Soil & Rock	Lindenhurst, NY	Not sufficient quantity available	
American Sand & Gravel, Inc.	Deer Park, NY	Not sufficient quantity available	

The potential upland sites were narrowed down to two sites: Horan Sand & Gravel Corp near Central Islip (20 miles) and Ranco Sand & Stone at Manorville (40 miles) based on screening of volume availability and grain size.

Table A-20 Characteristics of Potential Upland Sand Sources

Name of Quarry	Mean Size (ø)	Mean Size (mm)	Standard Deviation (ø)
Horan Sand & Gravel Corp	0.60	0.66	1.26
Ranco Sand & Stone	0.68	0.63	1.43

Potential Offshore Borrow Sources

The existing geologic maps and boring samples in Long Island Sound and within reasonable distance from the project site were examined. Details of the literature reviews and results are summarized in Appendix B. Three potential sites were short-listed based on their available size, suitability, and environmental considerations. The three sites are summarized as follows and shown in Figure A-29:

- An Offshore source in Long Island Sound, borrow area “A” located north of Asharoken, approximately 1 to 2 miles from the beachfill site, ½ miles offshore;
- An Offshore source in Long Island Sound, borrow area “B” located northeast of Asharoken, approximately 5 miles from the beachfill site;
- An Offshore source in Long Island Sound, borrow area “C” located north of Bayville, approximately 10 miles from the beachfill site;

Characterization of the Geologic Strata in Areas A, B and C

The geologic sequence was updated based on interpretation of the acoustic reflection record for each area. Various zones of stratification were identified and assigned nomenclature similar to the work performed in Long Island Sound by the USGS and Connecticut Department of Environmental Protection and others. In order to establish the suitability of an area as a borrow source the distribution of the various deposits are mapped and characteristics of the sediments are determined by their grain size distributions. A final suitability analysis is determined by a calculation of the R_a (Overfill Factor) and R_j (Renourishment Factor) factors using the native beach model derived from the Asharoken Beach samples.

<u>5.4 Characteristics of Core Samples</u>
The mean grain size and sorting ratios of all core samples were derived from graphically interpolated quartile values taken from the grain size distribution curves and applied to the formulas for Mean and Standard Deviation contained in Chapter 4 of the 1984 version of The Shore Protection Manual (referred as SPM 84).

Characteristics of Stratigraphic Zones Sampled in Asharoken Area A
Four (4) surficial stratigraphic zones were observed in Area A. These are:

- Beach Face Deposits
- Recent Marine Deposits
- Transitional Zone Deposits
- Glacial Contact Deposits

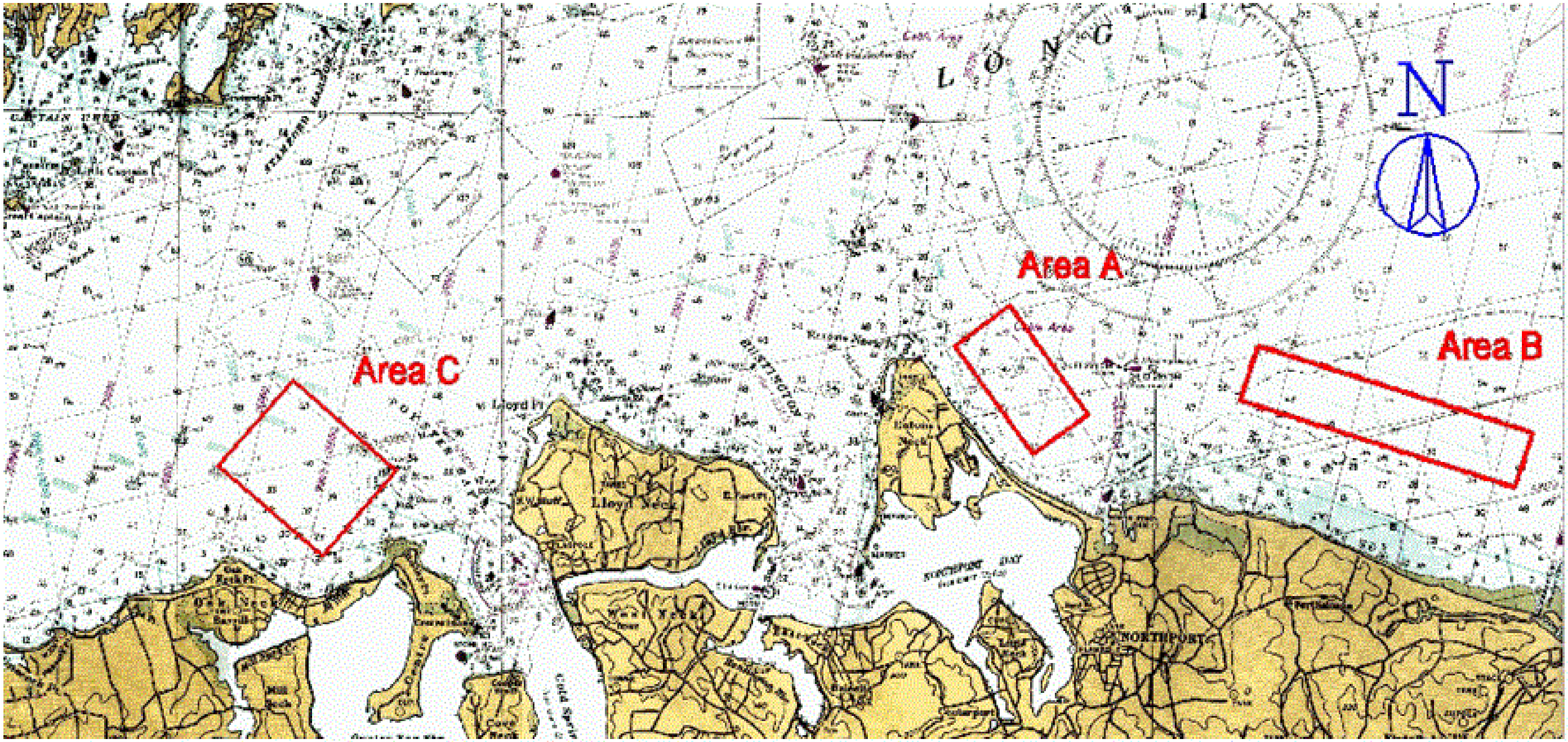


Figure A-29 Potential Offshore Borrow Area

The Beach Face Deposits are in shallow water along the southeastern margin of block. They are presumed to be composed of the sands currently being eroded off of the present Asharoken Beach and are probably comparable to the offshore beach samples collected along the -15 ft MLLW bottom contour. These deposits have not been sampled because of their fineness and are not considered a borrow resource. The recent Marine Deposits were cored at location A-03 and A-04. At these locations the Vibracore penetrated and sampled both the very soft silty clay of these deposits as well as the Transition Zone and Glacial Contact deposits found below. Transition Zone Deposits were encountered and sampled at core locations A-02, A-04 and A-07, while Glacial Contact Deposits make up the entire sampled cores at locations A-01, A-03, A-05, and A-06. The weighted averages of all twenty-five (25) **Glacial Contact** sediment samples representing 89.6 feet of sediment in Area A and Eleven (11) **Transitional Zone** sediment samples representing 37.6 feet of sediment provides the following result:

Glacial Contact Deposits (25 Samples) of 89.6 feet	
Mean (ϕ)	-1.04
Mean (mm)	2.06
Standard Deviation (ϕ)	2.50

Transition Zone Deposits (11 Samples) of 37.6 feet	
Mean (ϕ)	1.27
Mean (mm)	0.42
Standard Deviation (ϕ)	1.13

Characteristics of Stratigraphic Zones Sampled in Asharoken Area B

The stratigraphic zones interpreted from the acoustic reflection data shows four zones. These are listed from the youngest to the oldest in the sequence:

- Recent Marine Deposit (shoal)
- Recent Marine Deposit (not shoal)
- Transition Zone Deposit
- Glacial Contact Deposits

The weighted averages of mean grain size and sorting ratios for all samples in Area B provide the following result:

Glacial Contact Deposits (only 13 Sample) of 3.2 feet	
Mean (ϕ)	-1.08
Mean (mm)	2.11
Standard Deviation (ϕ)	2.44

Transition Zone Deposits (10 Samples) of 34.5 feet	
Mean (ϕ)	0.39
Mean (mm)	0.76
Standard Deviation (ϕ)	2.44

Recent Marine (Shoal) Deposits (23 Samples) of 98.9 feet	
Mean (ϕ)	1.61
Mean (mm)	0.33
Standard Deviation (ϕ)	0.66

Recent Marine (Non-Shoal) Deposits (6 Samples) of 22.0 feet

Mean (ϕ)	n/a
Mean (mm)	n/a
Standard Deviation (ϕ)	n/a

Characteristics of Stratigraphic Zones Sampled in Bayville Area C

Three (3) stratigraphic zones are evident in the subsurface of Area C. Recent marine deposits form the upper most layer and vary in thickness from 4 feet in the southwest corner of the block to over 36 feet near the northeast corner. Also discernible are Transition Zone Deposits, which are principally found as channel fill and below the Transition Zone are the Glacial Contact Deposits. Due to the obvious unsuitability of Area C as a potential borrow area, sampling was curtailed and samples which were taken in three (3) cores, C-03, C-06 and C-07 were not analyzed as they contain soft silty clay.

The exception was a location at core C-03 where the core sample penetrated the recent marine clays at 16.6 feet and encountered fine grain sands. These sands are in the Transition Zone and are some 25 feet thick covering the Glacial Contact sediment below. The sample between 16.6 feet and 19.2 feet has a Mean ϕ of 1.83 (0.28mm) and a Standard deviation of 0.85 ϕ. Another fine sandy layer was observed within the recent marine deposits at 6.8 feet to 9.2 feet with similar characteristics. No further analytical work was done in this area.

5.5 Suitibility Analysis

Suitability Criteria

The suitability of sediments from potential borrow sites considered as a source of supply for beachfill were evaluated by use of the techniques and mathematical equations presented and discussed by James (1975) and Hobson (1977) in SPM 1984. The publications provided the source for the development of computer program to evaluate two numbers, the Overfill Factor, Ra, and the Renourishment Ratio, Rj. New York District suitability criteria divide sediment into three categories: suitable, marginal and unsuitable. The Ra and Rj ranges for these criteria are listed in Table A-21 below.

TABLE A-21 SEDIMENT STABILITY CRITERIA

Ra	Classification	Rj
1.00 - 1.20	Suitable	0.00 - 1.00
1.20 - 1.30	Marginal	1.00 - 1.10
1.30 - ++	Unsuitable	1.10 - ++

The Overfill Factor, Ra, predicts the amount of overdredge of a given borrow material which will be required to produce after natural sorting. Losses due to the dredging processes are in addition to those natural sorting losses. The more desirable Ra factors are those closest to 1.00. An Ra factor of 1.0 to 1.1 is considered as representing the most suitable material. An extra fill volume of ten percent or less produces the desired sediment volume on the beach for Ra values between 1.0 to 1.1. A Ra factor of 1.1 to 1.3 means that an extra fill volume of up to thirty percent would be required to produce the post sorting loss design beachfill volume. For this project, the limits for suitability based on Ra factor are between 1.0 and 1.2.

The Renourishment Ratio, Rj, is a measure of the stability of the placed borrows material relative to the native sands. The more desirable Rj factors are those closest to or less than 1.0. An Rj ratio of 1.0 means the native and borrow sediments are of equal stability, having very similar grain size distributions. A renourishment factor of one-third (Rj = 0.33) means in theory that the borrow material is three times as stable as the natural beach sands, or that the renourishment with this borrow material would be required one-third as often as the native-like sediments. Beach nourishments are based on Rj of 1.0 to be conservative even if their Rj may be less than 1.0. For this project, the limits for suitability based on Rj ratio are between 0.0 and 1.00.

Each sand source was compared to the "native" beach sand sample, by use of the mean and the standard deviation of the grain size distribution of the beach model. Wave and surge action at the beach will wash away some of the beach fill from the borrow sources (sorting loss) as it adjusts to the coastal process action. To compensate for this loss of beach after placement, extra sand (based on the overfill factor, 1.0<Ra<1.2) from the borrow sources is needed for the initial placement of fill and renourishment.

Upland Sources Sample Analysis

Five (5) samples from each of the three quarries visited were sent to Johnson Soil Laboratories for grain size analysis. Grain size distribution curves were constructed from the tabulated grain size data and the mean grain size values both in Phi units and millimeters along with the sorting ratios (standard deviations) were computed for each sample. The results of grain size distribution data, grain size distribution curves and tables of mean grain size statistics and sorting ratios are contained in Appendix B of Alpine Report. Note that there are no statistics calculated for samples from the Hubbard Quarry because the material is too fine. Hubbard samples had over 38% of the grain sizes passing the #230 sieve (0.063 mm). Table A-22 presents the average mean grain size and sorting ratio for both the Horan and Ranco sites. The same table shows the calculated R_a and Rj factors based on a comparison with the native Asharoken Beach Model.

Table A-22 Characteristics of Long Island Upland Sand Sources
and Compatibility with the Asharoken Native Beach Model

Name of Quarry	Mean Size (ø)	Mean Size (mm)	Standard Deviation (ø)	Ra	Rj	Suitability
Horan Sand & Gravel Corp	0.60	0.66	1.26	3.0	1.85	Unsuitable
Ranco Sand & Stone	0.68	0.63	1.43	2.5	1.75	Unsuitable

The Ra (overfill factor) and Rj (renourishment factor) were computed graphically using methods outlined in the Shore Protection Manual (1984) and in the Engineering and Design Manual EM1110-2-3301, 1995, “Design of Beach Fills.” The upland sand source samples are considered “unsuitable” for the Asharoken project based on the suitability criteria. However, since the upland sources are close to the project site and the mean grain sizes are medium to coarse, the sources could be considered for initial construction and renourishment if mixed with coarser sand and more economical than utilization of an offshore borrow area.

Offshore Borrow Area Suitability Analysis

The suitability of the sediment in Areas A and B were determined by computing their Ra (overfill) and Rj (renourishment) factors using the native Asharoken Beach Model. Two different approaches were used in calculations. The first method is to compute the Ra and Rj factors for the various strata in each of the Areas using the weighted average characteristics of the strata as derived in the previous section. The second method is to calculate for the first ten feet of sediment at each core location the compatibility of the sediment at that location.

Suitability Evaluation Based on Weighted Average of the Geologic Strata in Areas A & B
Table A-23 present the results of calculating the Ra and Rj factors based on weighted average of the geologic strata in Areas A and B. Results of the sediment strata-based suitability analysis show that only glacial sediments in Areas A and B are suitable. Therefore, in the second stage of the suitability analysis, suitability will be determined for the glacial cores in Area A only, assuming dredging to a depth of 10 ft. The glacial material in Area B is closest to the surface in Core B-9. The glacial material at this location is overlain by transitional material. The second stage suitability analysis for Area B consisted of determining the suitability for the uppermost 10 feet at this location. A mathematically mixed composite was developed for this core of the upper 3 feet of transitional material, and the underlying 7 feet of glacial material.

Suitability Evaluation Based on Dredging to a Depth of 10 ft. For the uppermost 10 feet of glacial Cores A-1, A-5, A-6 show a resulting overfill factor of 1.03, and a renourishment factor of 0.39, which are suitable for use as beach fill for the Asharoken Beach fill area. Suitability of Core B-9 in Area B (consisting of a mathematical composite of the 3 feet of transitional material overlying 7 feet of glacial material) has an average Ra of 1.26, and an average Rj of 0.68, which is considered suitable for use as beach fill for Asharoken. These results are shown in Table A.24.

Table A-23 Ra and Rj Strata Results

Location	**Strata**	**Ra**	**Rj**	**Suitability**	**Applicable Cores**
A	Glacial	1.03	0.39	suitable	A-1, A-5, and A-6
A	Transitional	10.00	2.48	unsuitable	A-2, A-4, and A-7
B	Glacial	1.03	0.40	suitable	B-9
B	Transitional	1.81	1.36	unsuitable	B-6 and B-8
B	Marine Shoal	10.00	3.30	unsuitable	B-1 through B-5, B-7, and B-10 through B-12

Note: * Beach Composite of all samples except -15 ft. NGVD

Table A-24 Weighted Ra and Rj Results

Compatible Core	% of Length Glacial*	% of Length Transitional*	Glacial Ra	Transition Ra	Weighted Ra	Glacial Rj	Transition Rj	Weighted Rj	Average Suitability
A-1	100%	0%	1.03	10.00	1.03	0.39	2.48	0.39	**suitable**
A-5	100%	0%	1.03	10.00	1.03	0.39	2.48	0.39	**suitable**
A-6	100%	0%	1.03	10.00	1.03	0.39	2.48	0.39	**suitable**
B-9	70%	30%	1.03	1.81	1.26	0.40	1.36	0.68	**marginal**

Notes: * to a dredging depth of 10 feet.

5.6 Discussion of Results and Estimates of Volumes

Asharoken Area A

Of all the deposits sampled in Area A only the Glacial Contact Deposits were found suitable using a strict application of the Ra and Rj factors based on mean grain size and sorting ratios compared to the existing sediments on Asharoken Beach. The total estimated available volume of suitable Glacial Contact deposits in the A block surveyed is approximately 3,750,000 cubic yards. However, much of that volume is contained in areas with sparse coring data, and would need further data collection (coring) to verify suitability. Approximately 1,400,000 cubic yards is available immediately in the vicinity of Cores A-1, A-5, and A-6. In computing the volume, only the first ten (10) feet of sediments are included and only those areas deeper than thirty (30) feet

MLLW enter into the estimates. The volume calculation includes full dredging to the footprint of the area plus the addition of the side slopes around the perimeter at a slope of 1V:3H.

Table A-25 Corner Coordinates of All Potential Area A Borrow Sites (including those that need further coring to verify)

Corner	Easting	Northing	Corner	Easting	Northing
I	1,156,500	290,010	**III**	1,157,900	283,010
II	1,158,900	286,710	**IV**	1,154,700	288,710

Table A-26 Corner Coordinates of Immediately Available Potential Borrow Area A

Corner	Easting	Northing	Corner	Easting	Northing
A	1,155,200	289,100	**D**	1,157,330	285,650
B	1,155,850	288,000	**E**	1,156,550	285,300
C	1,156,900	287,620	**F**	1,154,700	288,700

Note: Coordinates shown in New York State Plane Coordinate System

Asharoken Area B

Approximately 83% of the seabed in Area B is composed of an active shoal deposit of very fine sands. These shoal deposits have been shown to be unsuitable for beach fill use because they are very well sorted with sorting ratios of around 0.5-0.6. The remaining deposits which outcrop on the seabed and which have been classified as Transition Zone deposits due to their placement in the stratigraphic sequence, account for remaining 17% of the sediments. Based on all the core samples taken and analyzed from the Transition Zone it is established that the material with a Mean (ϕ) 0.29, Mean (mm) 0.82, and a Standard deviation (ϕ) of 2.40 is a marginal borrow source.

However, the sediments around location Core B-9, which have a weighted average Mean (ϕ) of -0.45, Mean (mm) 1.37 and Standard Deviation (ϕ) of 1.89, are quite suitable for beach fill. Based on this assumption, and the assumption of 10 feet of dredging depth, it is estimated that there are approximately 1,700,000 cubic yards of potential borrow material in the blocks surrounding Core B-9. However, only the northern portion of the area has sufficient data to be used as a borrow source immediately. This immediately available area contains 900,000 cy. able A-27 shows the coordinates of the entire area (including immediately available sediments, and those needing further coring), which represents the boundary for environmental monitoring. And table A-28 shows the coordinates of the immediately available area.

Table A-27 Corner Coordinates of All Potential Area B Borrow Sites (including those that need further coring to verify)

Corner	Easting	Northing	Corner	Easting	Northing
I	1,183,550	283,310	**III**	1,184,870	280,590
II	1,185,640	282,260	**IV**	1,182,760	280,590

Table A-28 Corner Coordinates of Immediately Available Potential Borrow Area B

Corner	Easting	Northing	Corner	Easting	Northing
A	1,183,550	283,310	**C**	1,185,150	281,460
B	1,185,640	282,260	**D**	1,183,100	282,460

5.7 Constructability of Offshore Borrow Material

Constructability refers to the methods employed in the extraction and delivery of suitable offshore sources of borrows materials to the project beaches at Asharoken. The areas with potential borrow sources, Areas A and B are located as close to the project beaches as possible and dimensioned to avoid encroachment on both the pipeline and cable emplacement areas, and the leased shellfish beds (Figure A-30). The borrow sources are located at –30 ft NGVD depth contour or deeper, which is beyond the active surf zone limit at approximately -24 ft depth contour. Therefore, there would be minimal coastal process impact due to dredging. The material in any or all of these designated potential borrow sites are suitable and there should be no serious impediments to dredging due to environmental or location factors. However, there may be some constraints imposed by the presence of cobbles and boulders in the borrow areas, particularly in Area A. The Glacial Contact deposits in Area A have a larger content of gravels, some cobbles and there are signs of occasional boulders. This content may affect the choice of dredging methods used, dredging operations and production rates.

Cutterhead dredge would be the preferred method of dredging in Area A since it combines the qualities of high productivity in difficult soils with the ability to transport via pipeline to shore over distances of up to 15,000 feet and 30,000 feet with a booster pump. However, the Cutter Suction is unsuitable for large production in cobbles. Bars may be installed at the input end of the suction tube to reject large cobbles at their source. Where the cutterhead dredge cannot efficiently work, the backhoe or clamshell dredge would be utilized. Cutterhead with a booster pump (mounted on a spud barge) or hopper dredge may be used in Area B where the material is finer with apparently lower levels of cobbles and boulders.

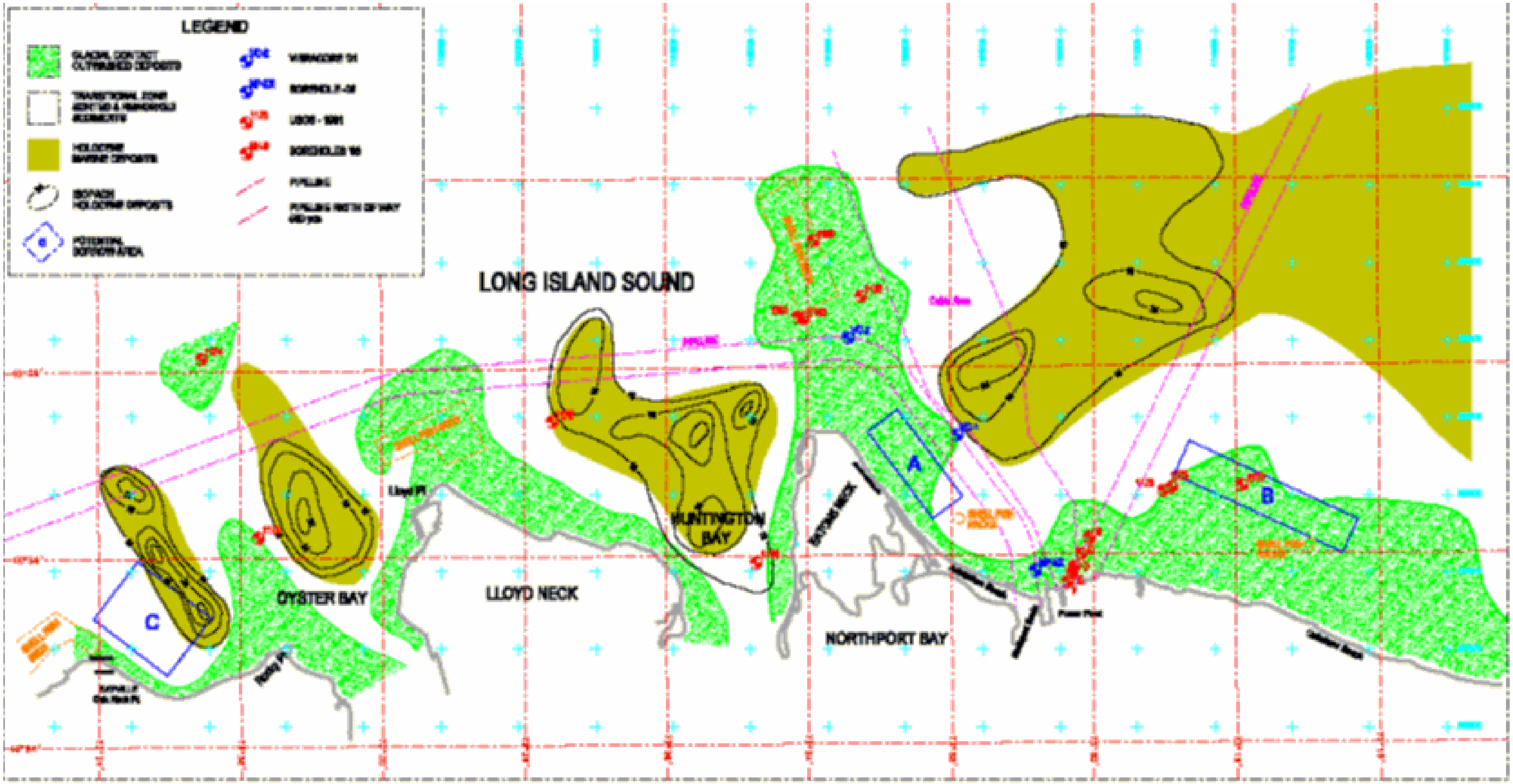


Figure A-30 Locations of Submarine Pipeline, Cable, and Leased Shellfish Beds

5.8 Selection of Borrow Source

Selection of the best feasible borrow source was based on the criteria of suitability, available volume, distance to project site, and permit considerations. The beach fill material at all potential upland sites are unsuitable. However, if this sand is mixed with coarser material to obtain suitability, it can be considered if more economical than offshore borrow area utilization. The upland sources modified with added coarser sand may also be considered as a backup to offshore source, which will require dredging permits.

Offshore Borrow Area A contains the larger volume of suitable material and is closer to the project site, therefore, is recommended as primary borrow source. Offshore Area B is an alternative site. Further environmental investigations are necessary to obtain dredging permit.

5.9 Construction Methods

Several alternative schemes were developed based on the location of potential borrow sources and methods of construction as shown in Table A-29 below.

TABLE A-29 ALTERNATIVE BORROW SCHEMES

Borrow Location/ Potential Volume	**Distance from Project Site**	**Method of Construction for Dredging/Transportation**	**Borrow Site Restrictions**
Asharoken Offshore Borrow Area A 3.75 million cubic yard **(Recommended)**	2 miles	Cutterhead pipeline dredge	Require dredging permit based on future environmental testing
Asharoken Offshore Borrow Area B 0.8 million cubic yard **(Alternative Site)**	5 miles	3,000 cy hopper dredge with pump-out facility	Require dredging permit based on future environmental testing
Upland Long Island Horan Sand and Gravel **(Backup Site)**	20 miles	Trucked to site	See Note Limited available volume Median grain size=0.6mm
Upland Long Island Ranco Sand and Stone **(Backup Site)**	40 miles	Trucked to Site	See Note Limited available volume Median grain size=0.6mm

Note:

1. All upland sources require additions of coarser grain sand to become suitable;
2. Upland sources may be used for future nourishment.

6.0 WITHOUT PROJECT CONDITION

The expected without project future condition include:

- Long term shoreline recession;
- Storm wave damage;
- Wave runup and overtopping induced dune and coastal structure failure and property damage;
- Inundation (from both L.I.Sound and Back Bay).

In addition to the general shoreline recession, there are two critical erosion areas that requires attention in order to provide a comprehensive protection due to storm waves. The two critical erosion areas are located near the northwestern and southeastern boundaries of the project shoreline as shown in Figure A-31 .

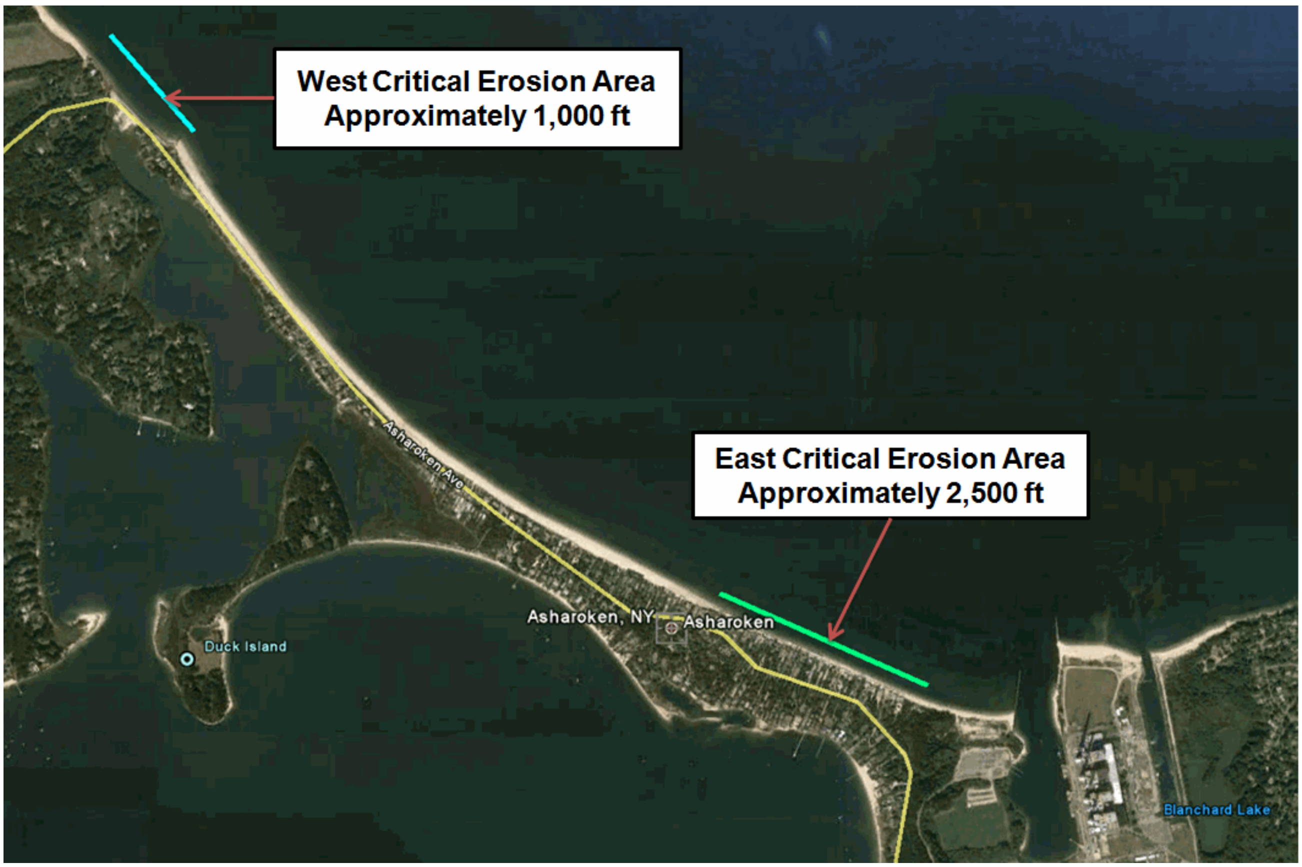


Figure A-31 Critical Erosion Areas

The existing and future without project conditions at the two critical areas are discussed as follows:

<u>6.1 West Critical Erosion Area</u>. This critical area is illustrated in Figure A-32. As shown in the figure, the sediment supply for this stretch of shoreline is blocked by the existing rock groin located at the eastern border. Sediment flow are accumulated updrift (east) of the groin, causing sediment deficit with downdrift erosion and destabilize the toe of the steel sheetpile seawall (Figures A-32, 33, and34). The beach fronting the seawall has eroded to the toe of the structure and exposed to direct storm wave attack. The shoreline erosion accelerated after two downdrift groins were

damaged in the period 1996-2006. The following summarize without project condition at this critical area:

- Continued toe erosion, de-stabilized existig riprap toe protection at seawall;
- Frequent seawall toe failure, overtopping and roadway damage;
- Worsened downdrift (west) bluff erosion and shoreline retreat (Figure A-35).

Figure A-32 West Critical Erosion Area

Figure A-33 Seawall west of the rock groin, Looking East

Figure A-34 Rock Armor protection at Toe of Sheetpile Seawall, Looking East

Figure A-35 Rock Armor protection at Toe of Seawall Looking West, Note Downdrift Bluff Erosion

<u>6.2 East Critical Erosion Area</u>. This critical area is illustrated in Figure A-36. As shown in the figure, a row of timber sheetpile bulkhead is constructed approximately 50 ft further offshore from the natural shoreline alignment. This forward alignment promotes accelerated erosion, especially during storm when shoreline re-adjusts to its natural alignment. Figure A-37 illustrates a 50-ft beach nourishment was quickly eroded away (in 6 months) after a 45,000 cubic yare, 50 ft berm nourishment. The cause of the rapid erosion in this area are likely due to the following:

- ➢ Sediment deficit due to blockage of updrift (east) jetties;
- ➢ Un-natural shoreline alignment of timber bulkheads;
- ➢ Deep water hole located just offshore alternating the offshore bathymetry and creating a "Nodal Point" at the center of critical shoreline, worsening the erosion condition (Fig. A-38);
- ➢ Storm waves amplify at the toe of vertial bulkhead wall causing accelerated erosion, overtopping and failure of seawall (Figures A-39, 40, 41).

A-36 East Critical Erosion Area

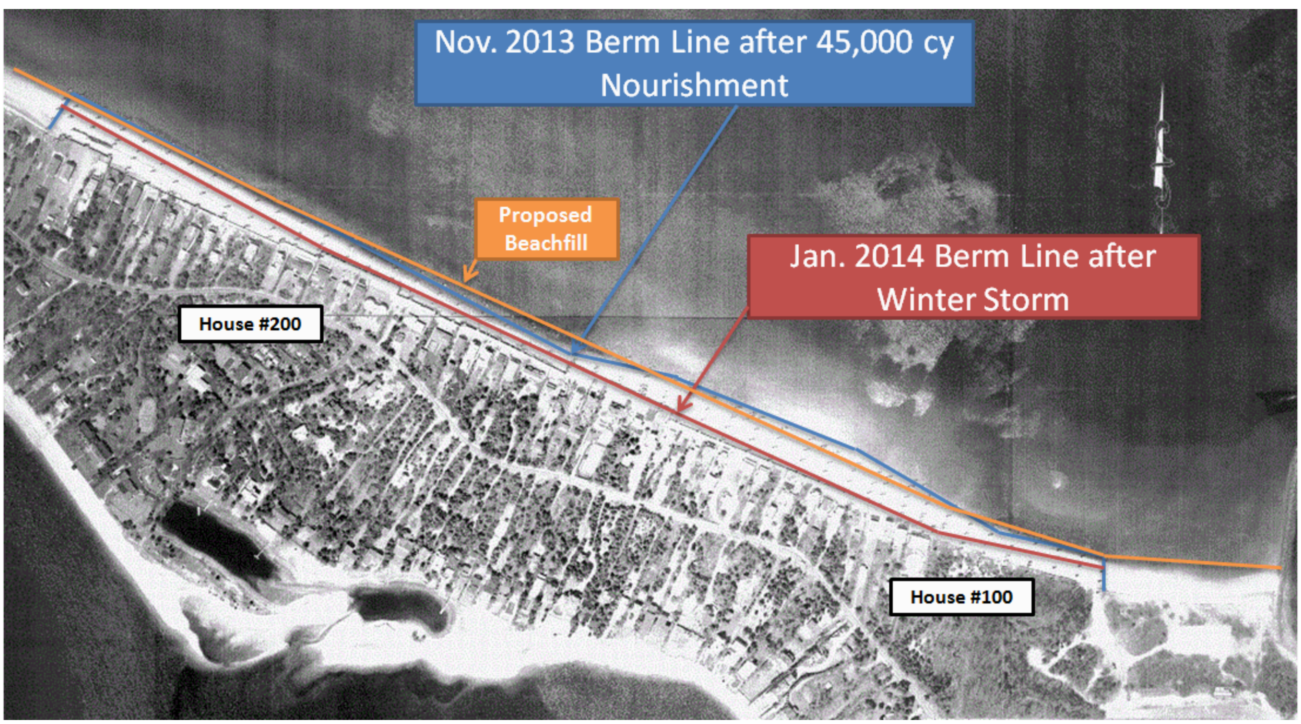


Figure A-37 Post-Nourishment Storm Erosion

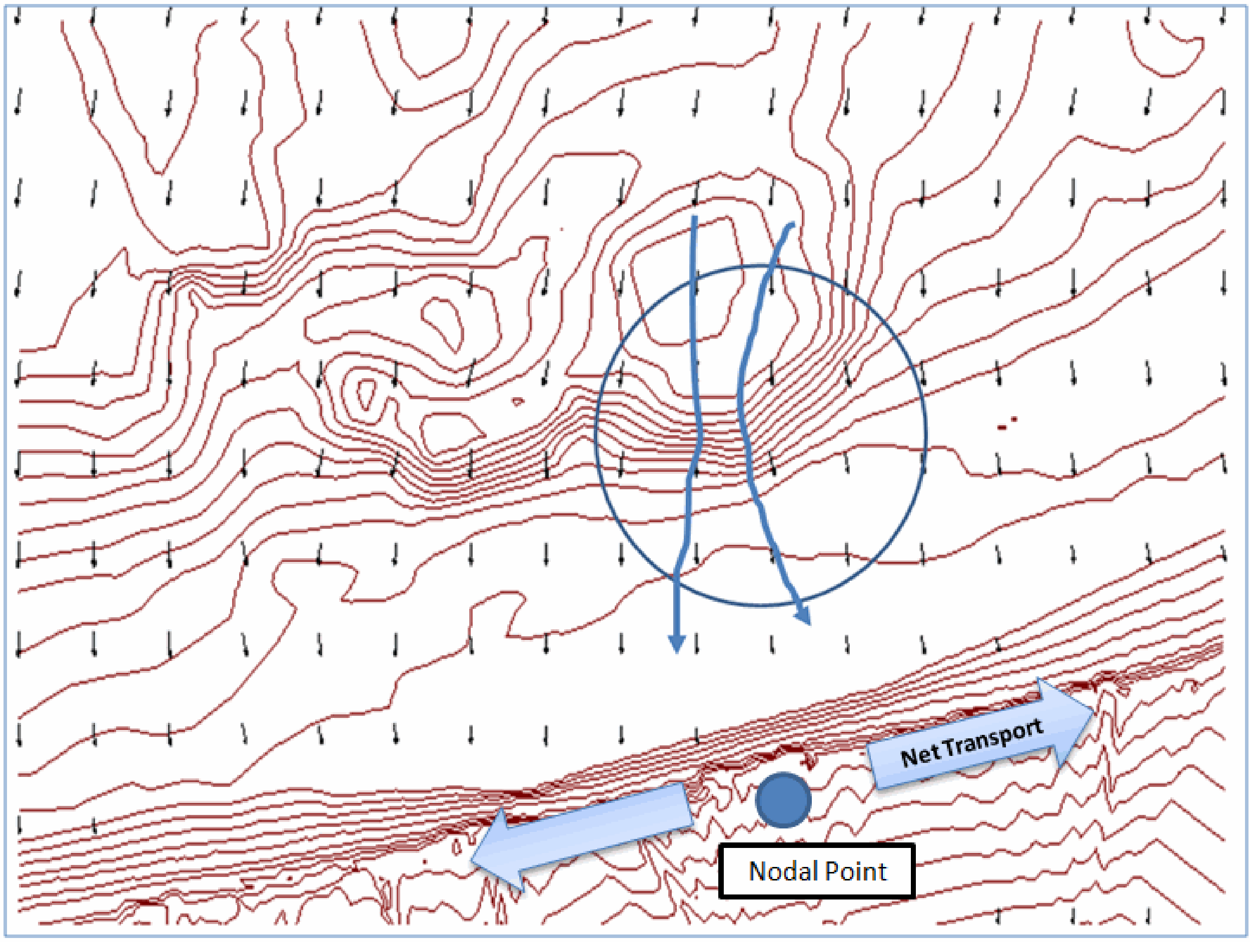


Figure A-38 Illustration of Nodal Point

Figure A-39 Beach and Toe Erosion near House #100, Looking East

Figure A-40 Beach and Toe Erosion at East Critical Area, Looking West

Figure A-41 Beach and Toe Erosion near House #200, Looking West

7.0 DAMAGE ESTIMATES

7.1 Wave Overtopping

Based on the observation of failure modes, it was judged that the primary dune and bulkhead seawall failure mechanism at Asharoken is due to wave overtopping, causing erosion of backfill material landward of the structure and an eventual foundation failure. Overtopping rates at bulkhead and dunes were estimated based on the crest elevations at the four typical reaches of Asharoken shoreline. Overtopping estimates were compared with the allowable overtopping rates determined for various failure conditions of interest to determine the level of protection provided by the existing shore conditions. Bulkhead failure due to overtopping was assumed to occur when the backfill material behind the wall has been eroded. Dune failure or breaching would occur after overwashing waves erode the backslope. It should be noted that the threshold overtopping rate varies widely due to various lab testing results and the actual site conditions. The critical values of average overtopping discharges for Asharoken are based on CEM2001 Table VI-5-6. The threshold overtopping rates used to determine bulkhead and dune failure are summarized as follows:

- Threshhold overtopping rate for protected crest: 0.20 cfs/ft
- Threshhold overtopping rate for unprotected crest: 0.05 cfs/ft

The overtopping discharge rate at the bulkhead wall can be estimated based on formula by van der Meer and Janssen (1995) and van der Meer (1998) as discussed in the Coastal Engineering Manual (CEM2001). The proposed formula for estimating the average wave overtopping on coastal structures subject to random waves can be expressed as:

$$\frac{q}{\sqrt{gH_s^{\;3}}} = 8.10^{-5} \exp\left[3.1 \frac{R_{u2\%} - R_c}{H_s}\right]$$

Maximum: $$\frac{q}{\sqrt{gH_s^{\;3}}} = 0.2 \exp\left[-2.3 \frac{R_c}{H_s} \frac{1}{\gamma_f \gamma_\beta}\right]$$

Where: q = mean wave overtopping discharge per unit width

ξ_p =breaker parameter

H_s =significant wave height

R_c = revetment crest freeboard (height of structure above still water)

γ_b = reduction factor for a berm

γ_f = reduction factor for slope roughness

γ_β = reduction factor for oblique wave attack

γ_v = reduction factor due to a vertical wall on a slope

For prediction of wave overtopping rates over dunes, Kobayashi et al. (1996) conducted seven small-scale tests to measure wave reflection, overtopping, and overwash of dunes. He compared measured overtopping rates with the empirical formula from van der Meer (which was developed for coastal structures) and through the use of an equivalent uniform slope (m_o) and showed that this formula can predict the order of magnitude of the measured overtopping rates. The equivalent uniform slope (m_o) is assumed to be the overall slope between the dune crest and the point where the water depth equals the significant wave height. Kobayashi's analysis, however, relies on small-scale tests, which generally do not accurately extrapolate to prototype conditions. Therefore, van der Meer's expression was further adapted through the use of results from a limited number of large scale tests performed by Delft Hydraulics Laboratory (1983). A similar approach was used in the analysis of overtopping, wave run-up, and wave impacts analysis for the Westhampton Interim Project (Moffatt and Nichol, 1993) and the Breach Contingency Plan (Moffatt & Nichol Engineers, 1995). Note, however, that those two previous analyses were based on an older overtopping formulation (Pilarczyk, 1990) which does not readily allow for a berm reduction factor and has not been directly applied to other dune overtopping measurements like the new van der Meer formula.

Overtopping data from the Delft experiments were used to derive new empirical coefficients for van der Meer's formula (i.e., 0.06 and –4.7). The following overtopping formula, which includes the new coefficients (0.013 and –2.33), is considered to be a better estimate of dune overtopping under prototype conditions.

$$\frac{q}{\sqrt{gH_s^{\;3}}}=\frac{0.013}{\sqrt{\tan\alpha}}\gamma_b\xi_p\exp\left[-2.33\frac{R_c}{H_s}\frac{1}{\xi_p\gamma_b\gamma_f\gamma_\beta\gamma_v}\right]$$

The above formula was developed using the equivalent slope assumption proposed by Kobayashi, which in the case of the Delft tests was computed to be approximately 1 on 15. The overtopping rates were estimated for existing bulkhead elevations at +11.5 ft NAVD (+12.5 ft NGVD) (Reach 1b) and +13 ft NAVD (+14.0 ft NGVD) (Reach 2b); and for existing dune elevations at+14 to +14.5 ft NAVD (+15 to +15.5 ft NGVD) (Reach 1d), and +16 ft NAVD (+17 ft NGVD) (Reach 2d). The following is a summary of parameters used :

- Offshore Slope: 1 vertical on 100 horizontal
- Toe Slope: 1 vertical on 3 horizontal
- Berm Height: +7 ft NAVD (+8.0 ft NGVD)
- Oblique Wave Angle: 0 degree (no reduction)

Wave Heights and Surge Levels used are summarized in Table A.30 and the estimated overtopping rates are summarized in Table A-31 and shown graphically in Figures A-42 (bulkhead) and A-43 (dune). Overtopping calculations are shown in Attachment A1.

Table A-30 Wave and Surge Input for Overtopping Estimates

Return Period (years)	Deep Water Wave Height Hs (ft)	Storm Surge Elevation (ft NAVD)	Storm Surge Elevation (ft NGVD)
2	8.4	4.9	5.9
5	10.5	6.5	7.5
10	12.4	7.8	8.8
25	14.8	9.1	10.1
50	16.4	10.1	11.1
100	18.0	11.2	12.2
200	19.6	12.4	13.4

Table A-31 Estimated Overtopping Rates at Existing Bulkhead and Dune

Return Period (years)	**Bulkhead Elevation (ft NAVD/NGVD)**		**Dune Elevation (ft NGVD)**		
	+11.5/+12.5	**+13.0/+14.0**	**+14.0/+15.0**	**+14.5/+15.5**	**+16.0/+17.0**
2	0.0000	0.0000	0.0000	0.0000	0.0000
5	0.0005	0.0001	0.0007	0.0004	0.0001
10	0.0205	0.0019	0.0126	0.0075	0.0016
25	0.3611	0.0041	0.1206	0.0766	0.0196
50	1.9024	0.2779	0.4503	0.2972	0.0855
100	9.4092	1.6209	1.5872	1.0856	0.3473
200	32.4501	8.2553	5.0900	3.5958	1.2678

Note: Estimated Overtopping Rates in cfs/ft

As shown in Figure A-43, the level of protection for bulkhead in Reach 1b (elevation +11.5 ft NAVD/+12.5 ft NGVD) ranges from 15 to 20 years depending on existing bulkhead condition. Since the bulkhead was designed for a 15 year life, it is judged that the existing level of protection is approximately 15 years. For existing bulkhead in Reach 2b (elevation +13.0 ft NAVD/+14.0 ft NGVD), the level of protection ranges from 25 to 40 years depending on the existing structural condition. Since the retaining walls in this reach are of timber material without tie-back structures, and with sporatic toe protection or splash guard, it is judged that the level of protection in this reach is at the low end of 25 years.

Based on Figure A-44, the level of protection of the existing dunes in Reaches 1d (elevations +14.0 to +14.5 ft NAVD/+15.0 to +15.5 ft NGVD) range from 20 to 40 years. Since the existing dune crest and backslope are in fair condition and well vegetated, it is judged that dune failure would only occur for storm waves with 25 year or higher intensity. The level of protection for Reach 2d (elevation +16.0 ft NAVD/+17.0 ft NGVD) ranges from 40 to 75 years. Based on the existing well vegetated dune condition in this reach, it is estimated that overtopping damage would be minimal in the 50 year project life.

The estimated level of protection for the four reaches are summarized in Table A.32 below.

Table A-32 Estimated Level of Protection - Existing Condition

Reach	Shore Protection Type	Crest Elevation (ft NAVD)	Crest Elevation (ft NGVD)	Level of Protection (years)
1a	Steel Bulkhead	11.5	12.5	10
1b	Vegetated Dune	14	15	25
2a	Timber Bulkhead	13	14	25
2b	Vegetated Dune	16	17	50

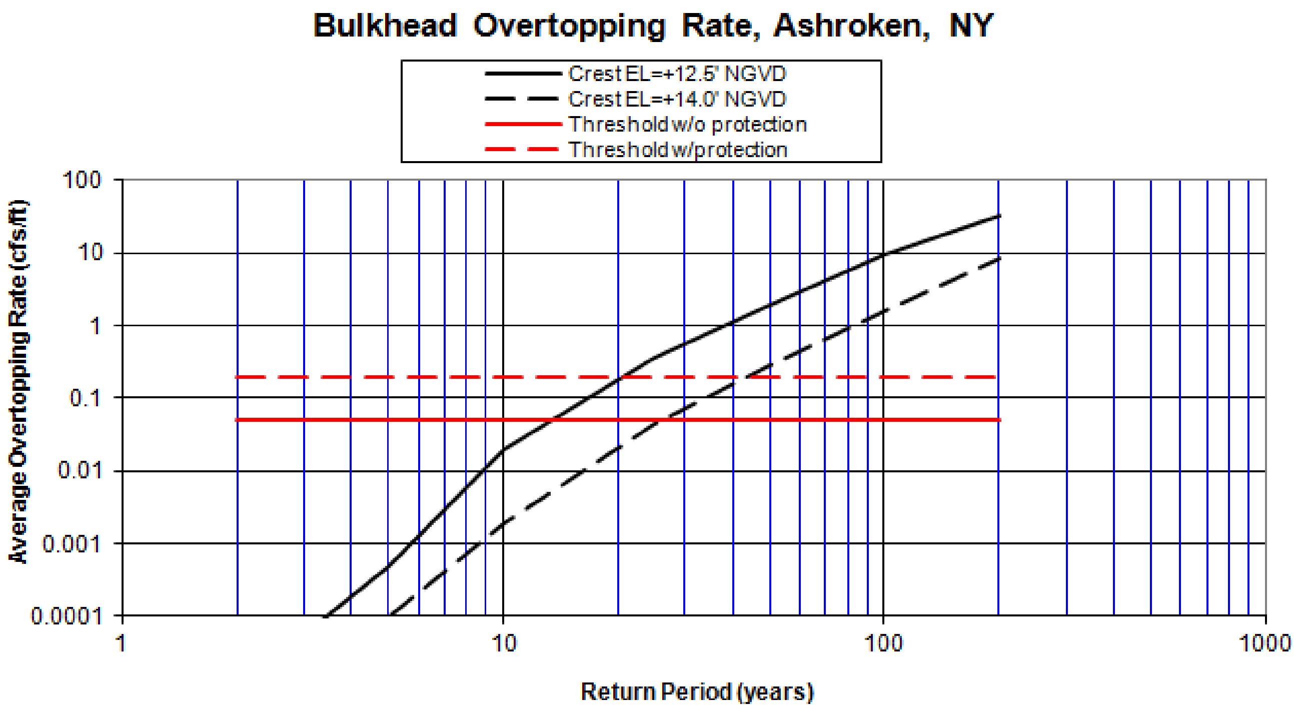


Figure A-42 Estimated Overtopping Rate at Existing Bulkhead

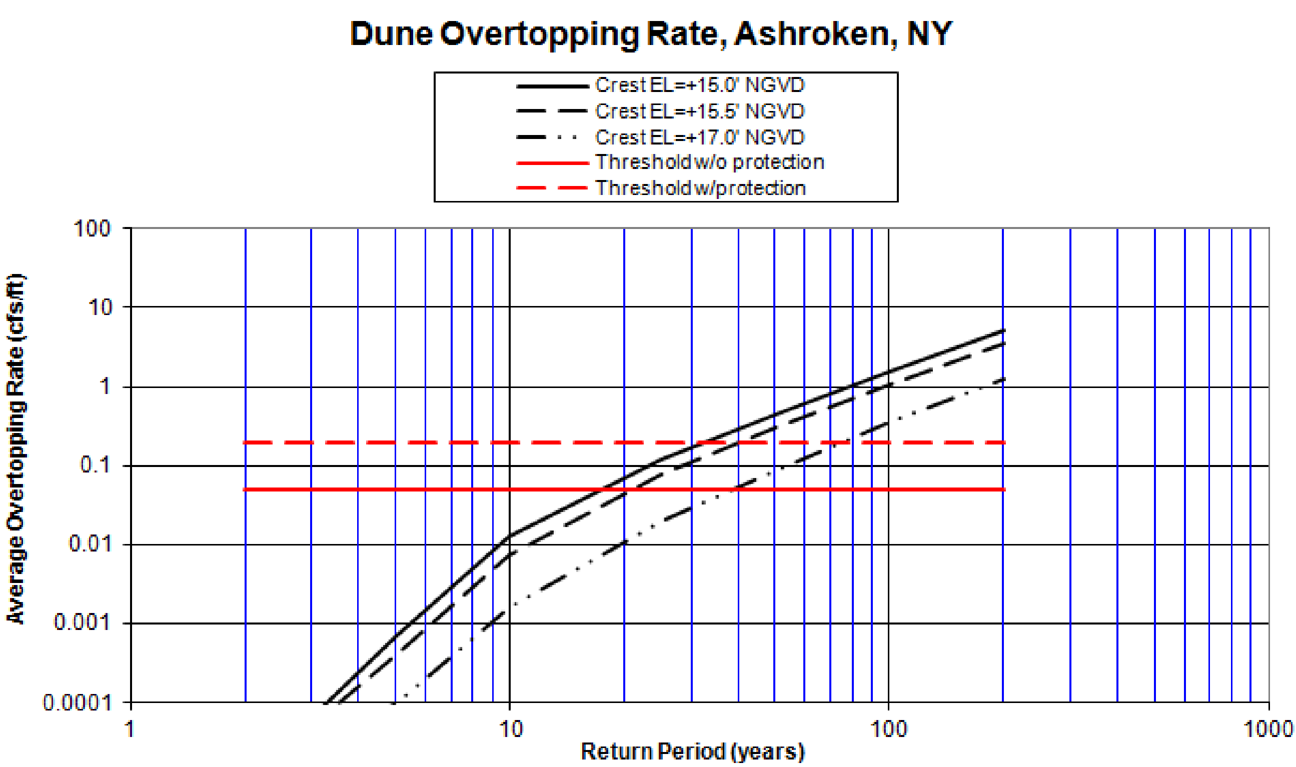


Figure A-43 Estimated Overtopping Rate at Existing Dune

7.2 Long Term Erosion Rates

Based on the sediment transport study and sediment budget analysis discussed in this report, the long term shoreline erosion rates along L.I.Sound front are:

- Reach 1a - 2 ft/year
- Reach 1b - 1 ft/year
- Reach 2a - 1 ft/year
- Reach 2b - 5 ft/year

The seawall in Reach 1a is steel bulkhead at crest elevation +11.5 ft NAVD (+12.5 ft NGVD) with penetration depth at +1.5 ft NAVD (+2.5 ft NGVD). It is judged that the bulkhead will be subject to failure when 90% of the sheetpile is exposed, i.e. fronting grade is ~+0.5 ft NAVD (+1.5 ft. NGVD). Assuming the foreshore beach profile maintains the same slope as the shoreline retreats, it would take approximately 20 ft of shoreline recession, or a 10-year event to trigger a bulkhead failure. After bulkhead failure, this reach will be treated as a dune and beach profile with reduced dune crest. For Reach 2a, approximately 4,000 ft of timber sheet pile was constructed seaward of the properties. It is estimated that it would take approximately 25 years of long term erosion (from 2001) to endanger the toe and trigger bulkhead failure

7.3 Short Term (Storm) Erosion Rates

The numerical model EDUNE was used to estimate the short term storm (storm) erosion rates. The following erosion distances (measured from the dune or bulkhead line) for the existing dune reaches, as well as for bulkheaded reaches after bulkhead failure are summarized in Table A.33. The erosion rate following bulkhead failure is similar to the pre-bulkhead failure rate- initially there is a jump in recession, but then the beach slope restores itself over time and the background erosion rate resumes. The surge levels and wave conditions used for EDUNE analysis are summarized in Table A.34. The resulting maximum erosion distances from the dune or bulkhead line are shown graphically in Figure A-44 and A-45.

Table A.33 Storm Erosion Distance (from Bulkhead Line)

Return Period (year)	Max. Dune Erosion Distance (ft)
2	17
10	42
25	54
50	66
100	80
500	85

Table A.34 Input Condition for EDUNE Model

Return Period (year)	Hb at -11 ft NAVD/ -10 ft NGVD	Hb(rms) at -11 ft NAVD/ –10 ft NGVD contour	Surge Elevation (ft NAVD/NGVD)
2	10.2	7.2	4.9/5.9
5	13.6	9.6	6.5/7.5
10	14.7	10.4	7.8/8.8
25	15.8	11.2	9.1/10.1
50	16.6	11.7	10.1/11.1
100	17.5	12.4	11.2/12.2
200	18.5	13.1	12.4/13.4

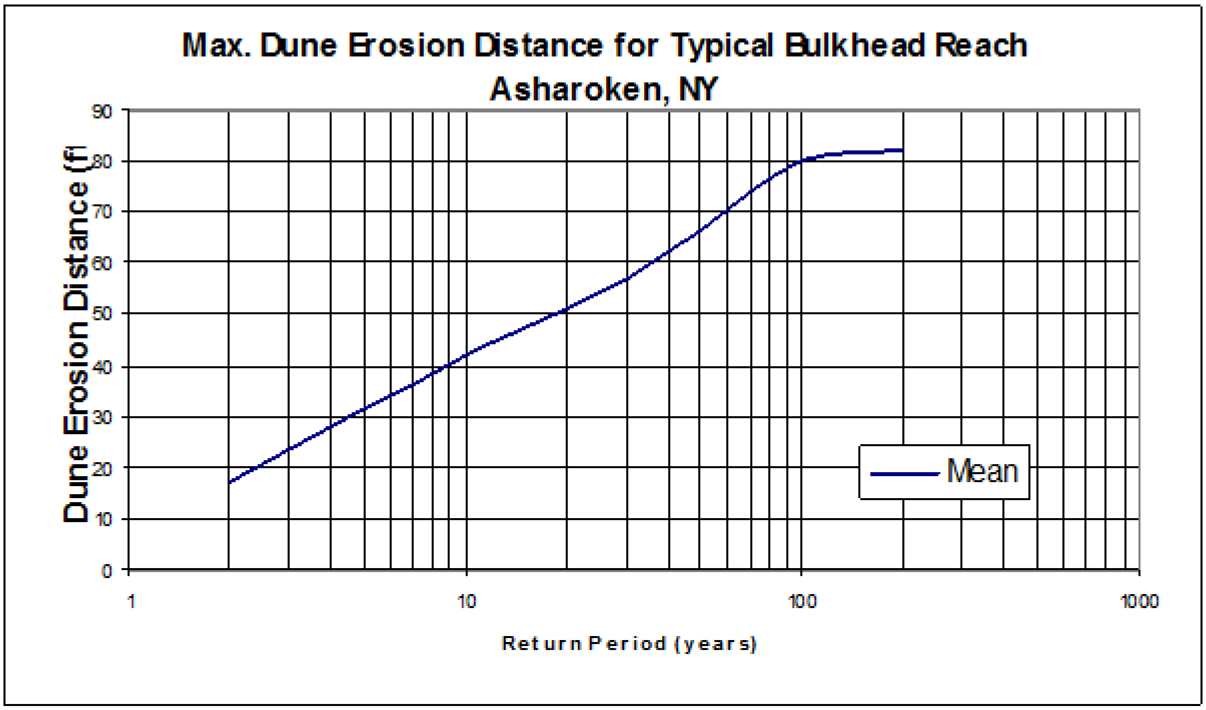


Figure A-44 Estimated Max. Erosion Distance from Baseline

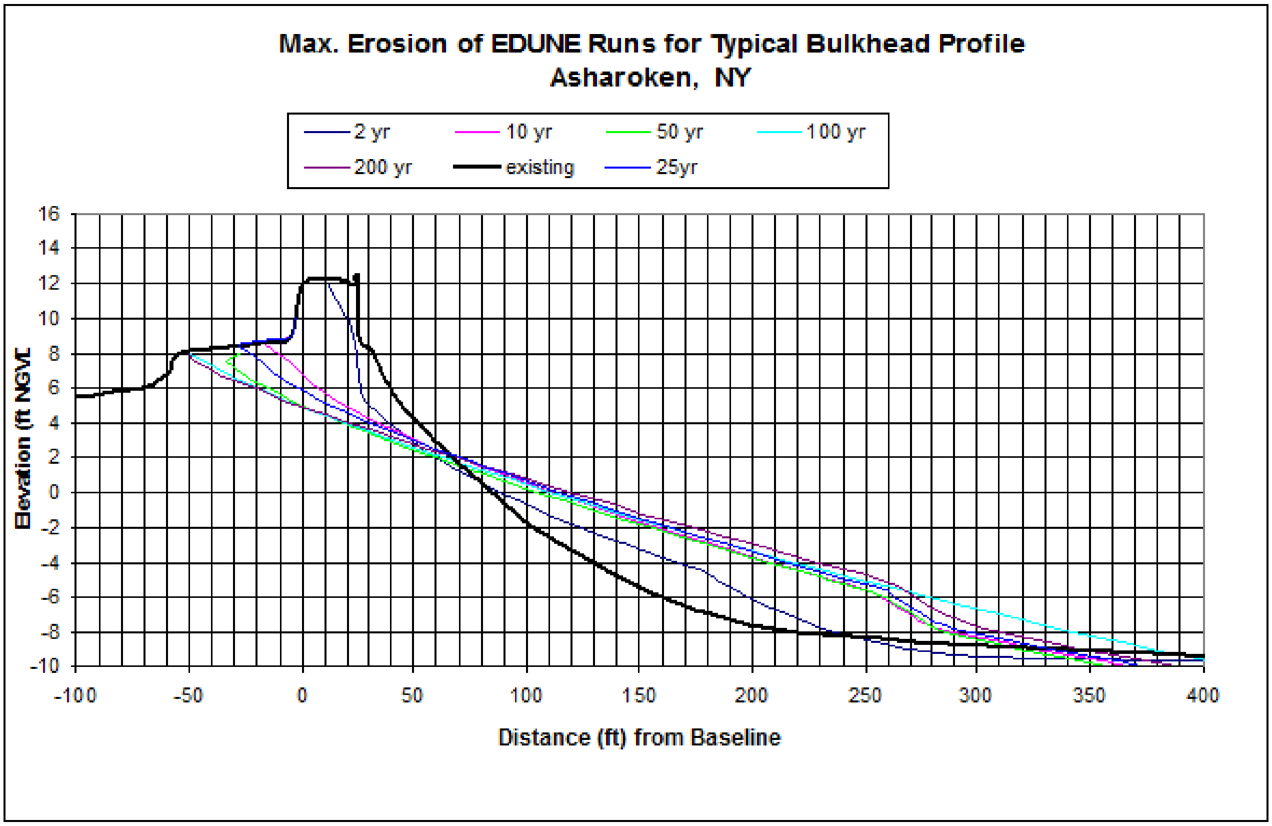


Figure A-45 Max. Erosion Distance

7.4 Future W/O Project Level of Protection

The project shoreline would continue to erode without any advanced or periodic beach nourishment. The existing condition level of protection would deteriorate due to increased wave runup and overtopping at the eroded dunes and bulkheads. The future W/O project level of protection is estimated based on EDUNE and runup/overtopping predictions of the eroded beach widths within the project life as shown in Table A.35.

Table A.35 Estimated Level of Protection

Future W/O Project Condition

Reach	Shore Protection Type	Crest Elevation (ft NAVD)	Crest Elevation (ft NGVD)	Level of Protection (years)
1a	Steel Bulkhead	11.5	12.5	10
1b	Vegetated Dune	14	15	5 to 15
2a	Timber Bulkhead	13	14	5 to 15
2b	Vegetated Dune	16	17	30 to 50

7.5 Future W/O Project Condition at the Section 103 Emergency Shore Erosion Protection Site
This project consists of a 900 ft shoreline overwashed during the 1992 storm. This shoreline was repaired with steel sheetpile (at +11.5 ft NAVD/+12.5 ft NGVD) and armor toe protection under Section 103 authority. This stretch of shoreline was designed to provide 22-year return period level of protection. The current condition of the structure provides protection against a 10-year event. The protective bulkhead could be damaged in the future due to excessive wave overtopping of storm intensity beyond 10-year return period or due to toe scour as a result of long-term erosion and storm wave erosion. After failure of the Section 103 bulkhead, it is assumed that the Village would perform repair to provide basic protection for Asharoken Avenue. The repair would include a 5 year level of protection consisting of a steel sheetpile wall at +9.5 ft NAVD/+10.5 ft NGVD with rock armor toe protection and sand backfill to +9.0 ft NAVD/+10 ft NGVD and a 6" layer of splash apron landward of the sheetpile. The estimated overtopping rate at the temporary wall is shown in Table A.36 below. The allowable overtopping rate at the new wall is 0.20 cfs/ft.

Table A.36
Estimated Overtopping Rates at the Sec.14 Bulkhead

Return Period (years)	Overtopping Rate (cfs/ft)
2	0.01
5	0.2
10	1.4
25	7.2
50	18.9
100	45.0
200	87.0

7.6 Future W/O and WITH Project Condition Damage Estimates
The future with and without project shoreline damage estimates are based on EDUNE beah storm erosion model. Details of the model methodology and results are presented in Attachment 1. The model results are used to determine with and without project damage avoided benefits.

8. 0 ALTERNATIVES SCREENING

Alternative plans have been formulated for provision of storm damage reduction for the Long Island Sound shoreline along the Village of Asharoken. The majority of shoreline erosion and storm damage problems occur over a stretch of developed shoreline extending approximately 2.4 miles (12,400 feet) from the westerly limit of Asharoken Avenue near Bevin Road, southeast to the west stone jetty of the LILCO cooling water intake lagoon. Possible alternative plans have been formulated through screening of protection features and evaluation of potential planning alternatives. The potential economically feasible plans, which would not appear to have significant adverse effects on environmental and cultural resources, have been identified for further evaluation. Those plans that do not meet the goals of the feasibility study and would not be implementable, are identified and will not proceed further for more detailed evaluation.

Alternatives for reducing damages from bayside flooding (tidal inundation) have also been investigated. Due to the general nature of study area, it is not deemed practical at this juncture to investigate structural tidal flood damage reduction measures since tidal flood damages are not major compared to the damages caused by wave attack and erosion. Non structural measures are included where appropriate to reduce flood damages.

The following sections briefly describe the criteria for and the evaluation of potential planning alternative features.

8.1 No Action (Continue Current Practice)

This plan means no additional Federal actions would be taken to provide for erosion control and storm damage protection. This plan fails to meet the objectives or needs for the majority of the project area. It will, however, provide the base against which the project benefits are measured. Additionally, if project costs exceed project benefits, thus indicating that protection measures are not in the Federal interest under current NED guidelines, no further Federal action would be recommended through the Corps of Engineers.

8.2 Non-Structural Measures

Buy-out Plan. This plan includes permanent evacuation of existing areas subject to erosion and/or inundation, by acquisition of this land and its structures, either by purchase through a willing seller or by exercising the powers of eminent domain. Following this action, all development in these areas is either demolished or relocated. With an anticipated high-depreciated replacement cost of structures in the 50-year design frequency floodplain, including land and relocation, this plan would appear to be prohibitively expensive and was thus dropped from consideration as a comprehensive solution. Due to the high value (Mean house value of $500 K) and number of structures (50-100 for houses fronting the beach), a buy-out plan would more than likely be cost prohibitive and not reduce the threat to Asharoken Ave. Limited buy-outs may be an effective means to enhance or supplement protection provided by other alternatives.

Zoning. Through proper land use regulation, floodplains can be managed to insure that their use is compatible with the severity of a flood hazard. Several means of regulation are available, including zoning ordinances, subdivision regulations, and building and housing codes. Zoning regulations at the New York State level under the Costal Erosion Hazard Act (CEHA) and at the Federal level under the Federal Emergency Management Agency (FEMA) flood insurance program exists to deal with structures that are more than 50% damaged in a storm. Under both programs, no repair is permitted if there were more than 50% damage to the structure after a storm event. It should be noted that the CEHA has not yet been implemented in this area, limiting the present effectiveness of this alternative. This could gradually mitigate the damage in high-risk areas over time by removing severely damaged structures from the hazard zone. However, it should also be noted that the magnitude of 50% damage to a structure would not likely occur for some period of time, i.e. probably not less than 10 years and therefore would not be implementable currently. The Corps would have no regulatory authority to implement a zoning plan, although the Corps could suggest zoning changes to be implemented by the non-Federal Sponsor.

Retrofitting. Retrofitting, by definition, is a body of techniques for preventing damages due to floods, and requires adjustments both to structures and to building contents. It involves keeping water out of structures, as well as reducing the effects of water entry. Such adjustments can be applied by an individual or as part of a collective action, either when buildings are under construction or as part of a remodeling or retrofitting of existing structures. From the 1992 storm (estimated 100 year storm event by the National Weather Service), several houses that front the Long Island Sound and Huntington Bay experienced first floor flooding of 1 to 2 feet. Flood proofing typically includes elevating the structure but could also consist of a ring levee to protect the house, or the structures. The applicability of these measures will be investigated further.

Relocation. Relocation includes the effort necessary to move an existing structure from the erosion prone area further inland, south of the CEHA line. This can include relocation of a structure to an adjacent empty lot, or relocation further back within the existing lot. Relocation of a building to an adjacent lot is not cost effective or implementable due to the high cost of acquiring property, and the limited availability of vacant lands for relocation. This measure will reduce the threat of erosion to the buildings, but will not address the threat of long term erosion and wave attack to Asharoken Ave. Opportunities for relocation of houses away from the beach, but still in the property will be investigated further as alternatives that can be included with other alternatives.

8.3 Structural Measures

The following sections briefly describe various structural protection techniques considered as elements of a comprehensive erosion control and flood damage prevention solution.

***Beach Nourishment*.** Beach nourishment involves the placement of sand on an eroding shoreline to restore its form and to provide adequate protection. A beach fill typically includes a berm backed by a dune or existing bulkhead; these elements combine to prevent erosion, wave impact and inundation damages to leeward areas. Beach nourishment represents a near natural method for reducing damages on the open coast. Beach renourishment on 5 to 10 year cycles are required over the life of the project to counteract long term and storm induced erosion and additional erosion from sea level rise. A typical beach nourishment section with berm and dune fill is shown in Figure A-46.

For an average dune elevation of 15 to 17 ft., a 25 ft dune crest width, 1V:5H dune slopes, and a 50 to 100 ft. berm crest at el. +8 to 9 ft (the historically most stable berm crest elevation to prevent scarping) with a historically stable 1V:15H foreshore slope and sand fence and beach grass for dune enhancement, the cost of the "beachfill only" option is approximately $1,000 to 1,200/l.f. of shoreline plus approximately $500 to $700/ l.f..for renourishment at 5 year to 10 cycles. This includes all operation and maintenance of the project. Based on a reach of 12,400 ft, the total annual cost of a "beachfill only" project would be approximately $1,800/LF (per Linear Foot) or $22,320,000 total cost for the entire project reach, including the initial construction cost and the present worth of 9 renourishment operations.

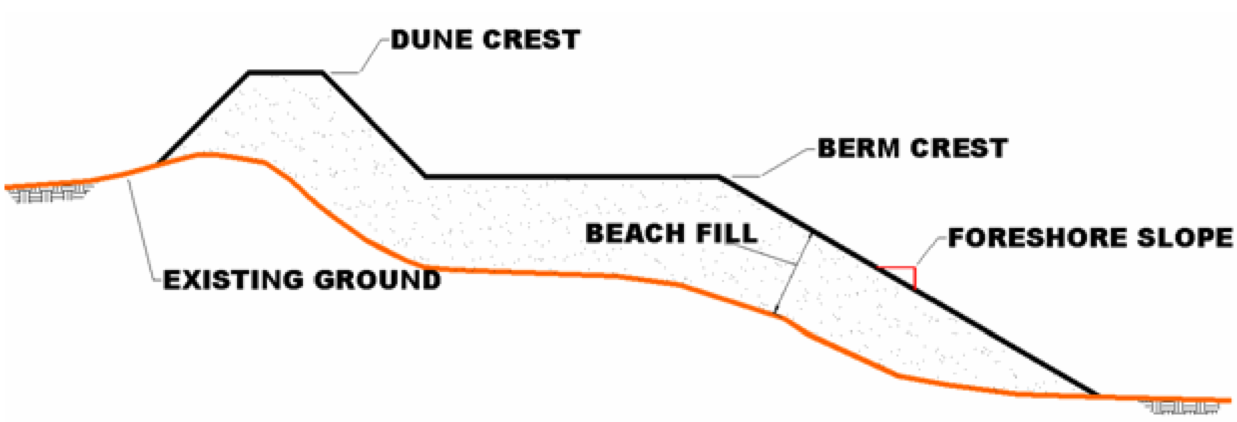


Figure A-46 Typical Beachfill Alternative

Floodwalls and Levees. Floodwalls and levees are intended to provide protection against coastal and riverine flooding. These structures can be cost-effective measures against tidal flooding when placed landward of direct wave exposure. Used in this manner, floodwalls and levees provide flood protection to interior structures. While these structures may provide a cost-effective means to prevent flooding of low-lying areas, runoff trapped behind the structure may affect the hydrology and drainage of interior areas. This may alter tidal wetlands and require additional drainage facilities. A typical concrete "L" shape vertical floodwall with 15-inch thickness and 14 ft crest elevation and a cut-off wall cost approximately $2,500 per linear foot of shoreline. Floodwalls and levees would not be applicable for this project since the predominant damages to the project area are due to shore erosion and storm wave damages which would not be addressed by a levee or floodwall. The cost of protecting these structures from undermining would be prohibitive. In general the use of floodwalls and levees would not be consistent with the community focus on waterfront access for fishing, boating and other uses and will not be considered further.

Reinforced Dune with Beachfill. A reinforced dune is similar to beach nourishment, however, the core of the dune is strengthened with structural elements such as a buried rock seawall. The reinforced dune will prevent the potential for breaching of the dune line exposed to severe events and limit the overtopping from wave action due to the permanent nature of the dune reinforcement vs. the erodable nature of a sand dune. Less frequent renourishment is required for the reinforced dune option since the sand losses are less for the narrower berm required for the reinforced dune due to reduced storm induced damage losses. A typical reinforced dune and beach fill section is illustrated in Figure A-47. The cost for a typical reinforced dune with sand cover is approximately $2,500 to $3,000 /l.f. including a 15 ft dune crest width, 1V:3H dune slope encapsulating a trapezoidally shaped stone seawall with a crest elevation of 12 to 12.5 ft NAVD (13 to 13.5 ft. NGVD) for a 10 ft. width and 1V:1.5H side slopes, 100 ft.berm width at el. 6 to 7 ft NAVD (7 to 8 ft. NGVD), a 1V: 15 H foreshore slope and a $400 to $600/l.f. beach fill reourishment at 10 year cycles throughout the life of the project. Based on a reach of 12,400 ft, the cost of a beach fill with dune reinforcement project would be approximately $ 43,200,000 dollars for the first cost of construction and the present worth of 5 renourishment operations.

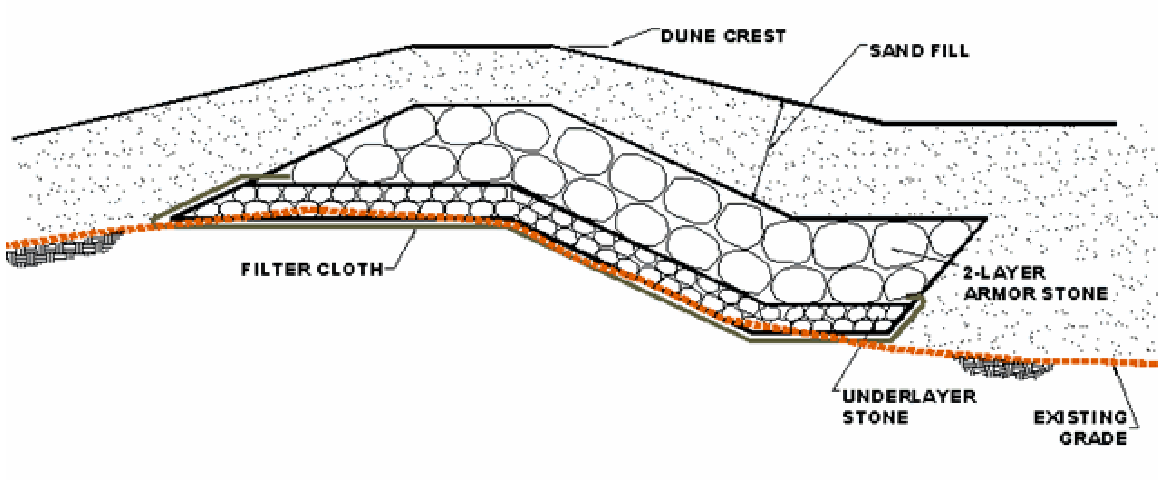


Figure A-47 Typical Reinforced Dune

Bulkhead or Bulkhead with Rock Toe Bulkhead shore stabilization measures offer both erosion and wave impact protection for shorefront structures. Bulkhead material may be steel, or composite high density plastic with tie-backs, stone splash blanket, and toe-protections. Bulkhead stabilization measures help to reduce effects of wave action, minimize overtopping floodwaters and limit landward movement of the shoreline. Rock toe and splash blanket protection is required to preclude undermining of the bulkhead foundation. However, this plan does not prevent beach erosion from encroaching on the bulkhead which will require costly maintenance and replacement of sand and toe armor. A typical bulkhead section is shown in Figure A-48. The cost of a bulkhead with rock toe can cost over $2,500/ l.f., but is not recommended due to frequent maintenance requirements.

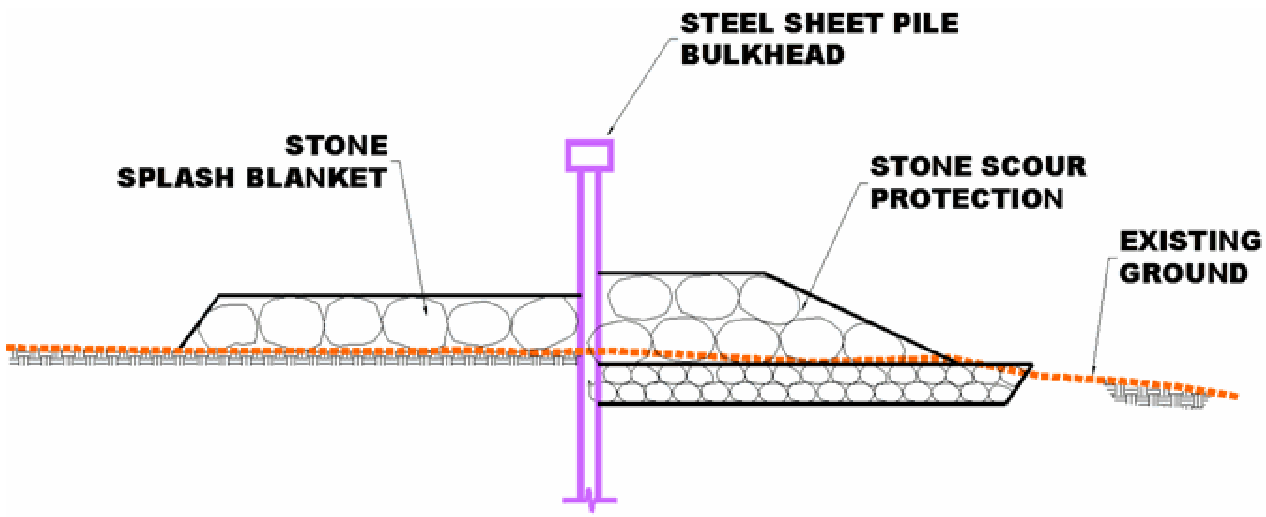


Figure A-48 Typical Bulkhead with Raised Dune

Groins with Beachfill. Groins are rubblemound or timber/steel sheet piles constructed perpendicular to shoreline. By properly setting the groin length and the space between groins, the existing and new beachfill material will be retained to reduce long-term erosion rates and retain renourishment fill. However, groins may not be effective in reducing offshore movement of beach material during storms and can create adverse impact downdrift effects. In order to retain material moving offshore, a T-groin with shore-parallel section attached to the groin head may be considered if a large portion of offshore sand movement is confirmed. Assuming 500 ft spacing between groins and a 150 ft length, approximately 24 groins would be necessary with total cost of approximately $5,000/LF plus $300 to $400/ LF for renourishment. This relates to a $45,000,000 present value for initial construction cost, with renourishment. If T-Groins were used, the total number of groins may be reduced to 12 with approximately the same initial construction and renourishment cost. A typical groin field plan and profile is illustrated in Figure A-49 and the T-Groin plan and profile is illustrated in Figure A-50.

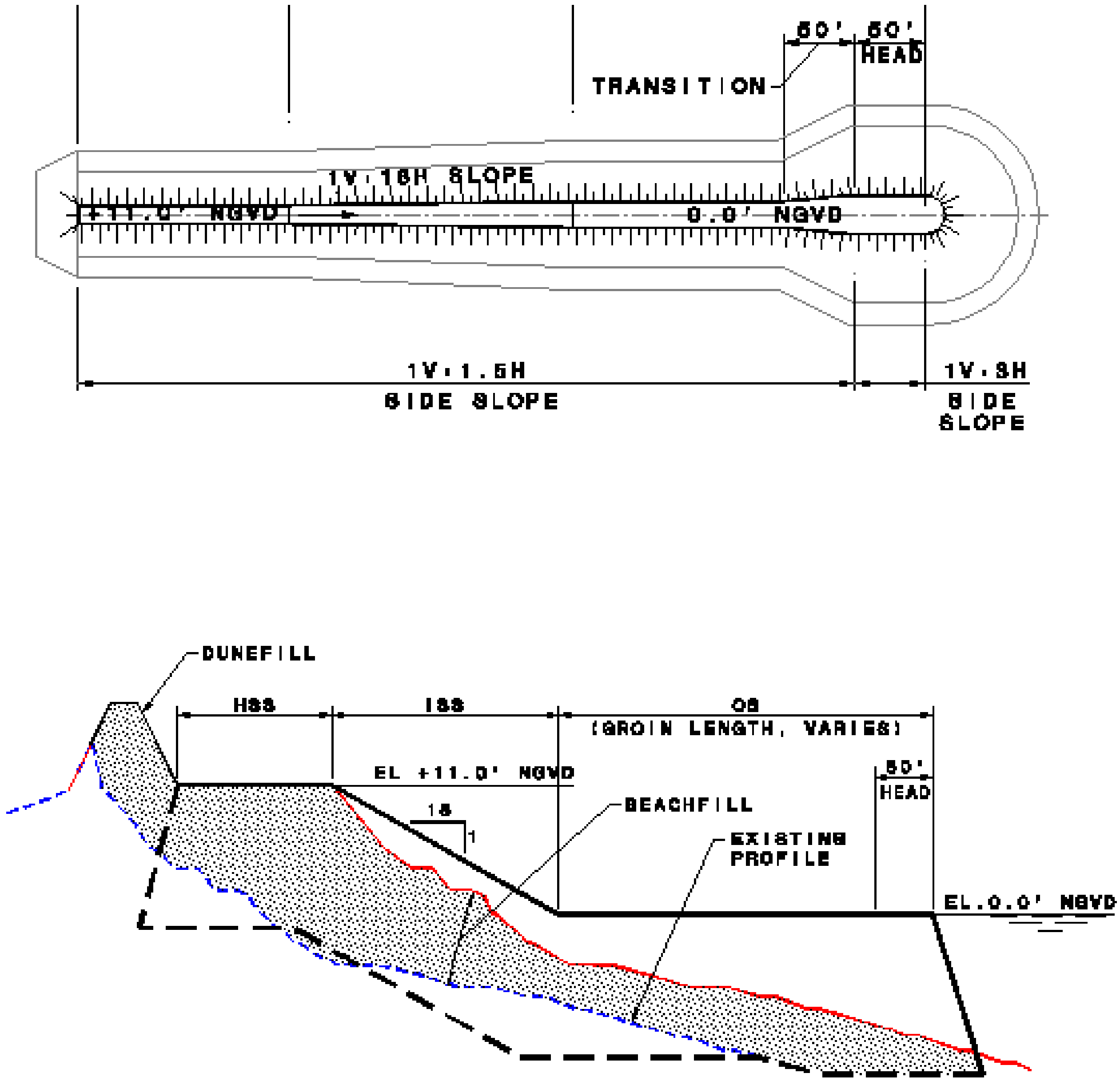


Figure A-49 Typical Groin Plan and Profile

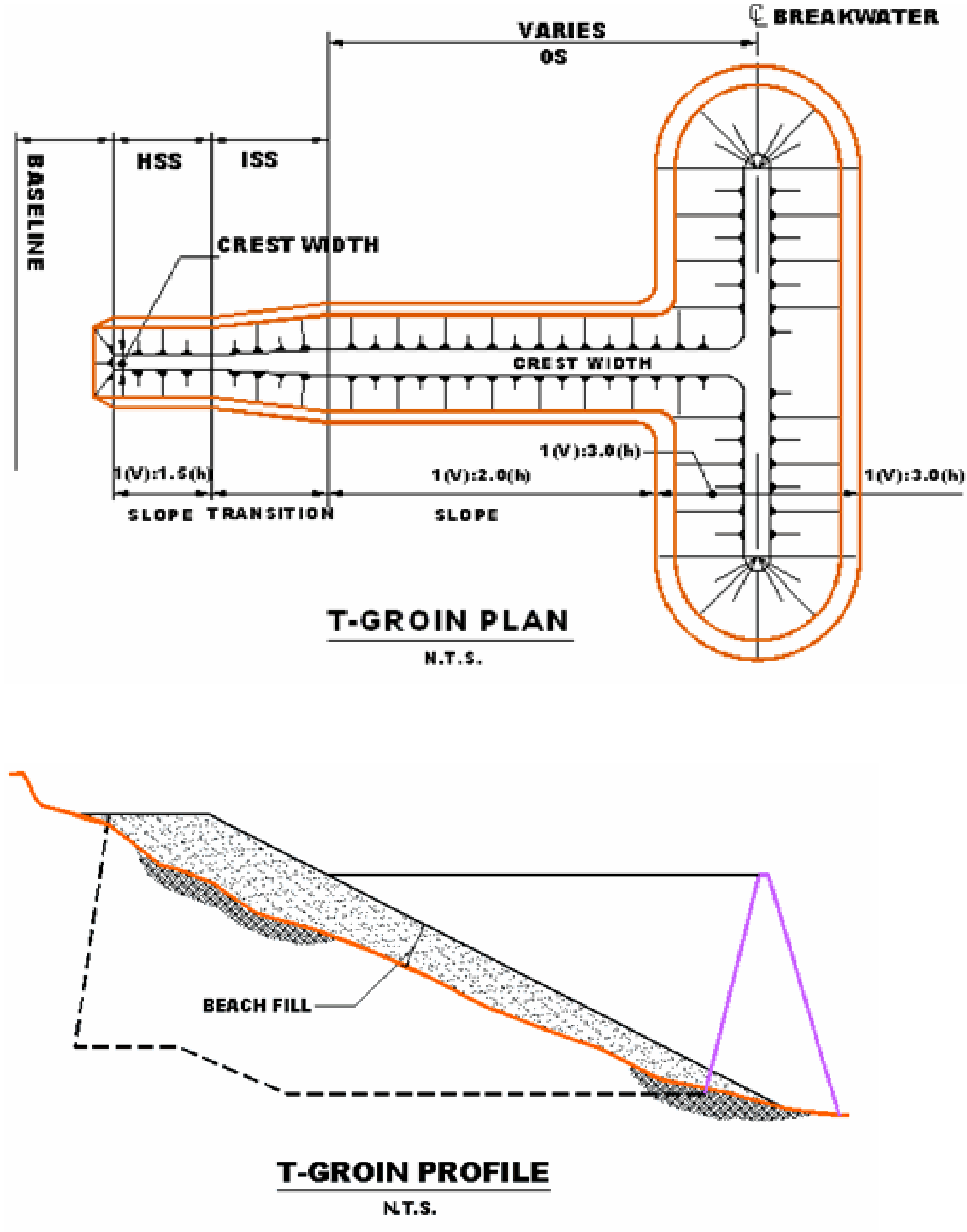


Figure A-50 TypicalT-Groin Plan and Profile

Reduced Groins with Beachfill. Instead of a 12 to 24 structure groin field, only a limited number of groins will be provided to shoreline locations with historical high erosion rates and at locations experiencing high storm damages. As discussed in the sediment transport analysis, there are two reaches identified:

- Reach 1a – approximately 900 ft of shoreline fronting the Steel bulkhead seawall are experiencing severe toe erosion due to a rock groin located just upstream (east) of the seawall, depriving littoral flow downstream. A small groin field constructed in this reach would stabilize the seawall toe and reduce future storm damage repairs;
- Reach 2b – up to 2 groins should be constructed at the downstream (west) end of the existing bulkheads. In general, the existing bulkhead is situated approximately 50 ft seaward of the general shoreline and requires frequent renourishment to retain a minimum design berm width for the design level of protection. Groins would aid in retaining fill fronting the bulkheads.

Offshore Breakwaters with Beachfill. Offshore breakwaters are rock mounds constructed along the shoreline in approximately -5 ft MLW depth. The reef structures are effective for retaining beach material lost due to long term erosion and reducing sand movement offshore during storms. In addition, wave runup and overtopping would be reduced due to pre-breaking of storm waves. Beachfill maintenance (renourishment) may be reduced because of tombolo (sand trapping) formation. Each reef segment would be 800 ft length with a 200 ft gap between segments, requiring approximately 10 offshore reefs. These would cost approximately $5,500 /l.f. and have a total present value construction cost of $50,000,000 including a small amount of renourishment at 10 year cycles. A typical offshore breakwater plan and section is illustrated in Figures A-51 and 52. This alternative is not considered further due to high initial construction cost.

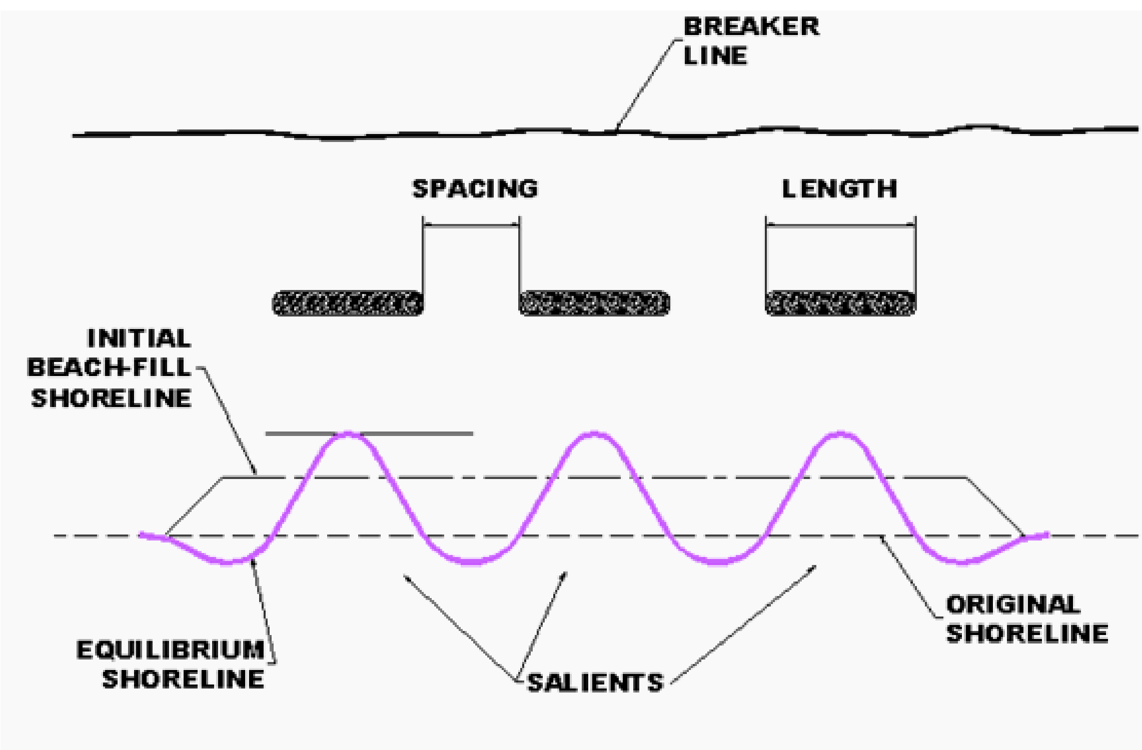


Figure A-51 Typical Offshore Breakwater plan

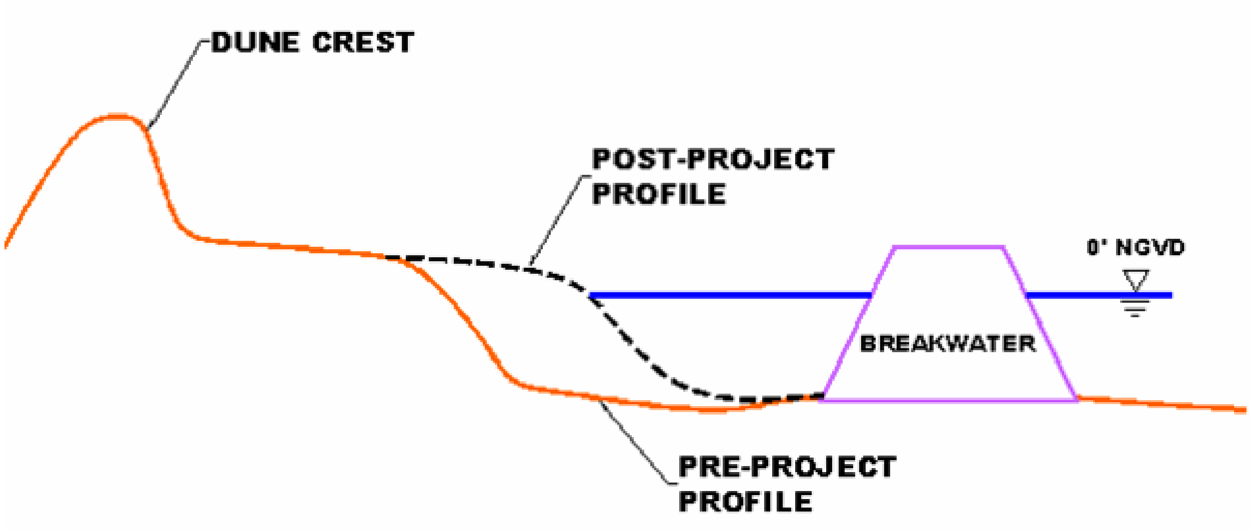


Figure A-52 Typical Offshore Breakwater Section

Regional Sediment Management Alternatives

Sand Bypassing. The power plant has historically been by-passing sand to the down drift (west) side of the jetties. The source of the sand has been the dredging of the entrance channel to the cool water intake lagoon. Based on the historical bypassed volumes of sand that were dredged (10,000 cy/year) and placed down drift, and looking at historical rates of erosion, additional bypassing of 10,000 cy/year may be required, resulting a total of 20,000 cy/year. Assuming unit cost of $15/cy, annual bypassing cost is approximately $300,000/yr. Future rate of sand bypassing should be adjusted based on channel silting rate and east jetty sediment fillet accumulation monitoring results. However, the bypassing option does not address storm damage reduction of the downdrift beach to the west. It will not directly impact damages from significant storm events where higher beach grade elevations are required.

Installation of a Diffusion Pipe. Based on data provided by Keyspan (Keyspan, 2003), the power plant is discharging approximately 1,100 cfs (494,000 gpm) of cooling water to L.I.Sound via an overflow weir located at the mouth of the cooling water retention pond. The discharge flow plume centerline velocity is up to 7 fps at peak flow (Taylor Engineering, July 2001). Littoral materials are carried by the effluent flow and re-deposited to a distance not more than 600 ft. offshore where some of it moves into the inlet channel to the west, based on preliminary modeling results. In order to position the exit of a diffusion pipe beyond the active surf zone at approximately 24 ft depth contour, a 6,000 ft diffusion pipe would be installed at the cost approximately $6,000,000. This measure would have to be evaluated in conjunction with other measures to determine its cost effectiveness and the benefits it actually provides. In addition, like sand bypassing, a diffusion pipe from the plant may improve the littoral transport into the system, but will not solve the storm damage problems occurring to the north and west of the power plant.

Modifications to Jetties. The east and west jetties were renovated with rock material after the cooling water intake channel and lagoon were dredged in 1966 and Unit #1 of the power plant was on line in 1967. Littoral material has continued to accumulate in the intake channel and east of the east jetty. Recently the fillet growth has stabilized with little additional sand being trapped by the jetty believed to contain not more than several tens of thousands of cubic yards of sand. Dredging records indicate that approximately 10,000 cy/year of sand had been dredged from the intake channel and basin and placed on downdrift beach periodically. In order to increase the amount of material bypassing the inlet, the rock jetties could be significantly shortened or removed to achieve this goal. However, littoral material will continue to be trapped in the dredged channel and Northport basin unless the jetties are removed and channels closed. The option of removal of the jetties will not be pursued, because it will eliminate the access necessary for the operation of the LILCO plant, and as such would not be implementable. The benefits and costs of modifying the jetties will be pursued further.

Modification to the Roadway and Utilities

Causeway. The only way to protect Asharoken Ave. through modification of the roadway would be to elevate the road on concrete pilings and build a causeway. This would protect the road by elevating and allowing overwashes to go under the road. This option would not be considered further because it would not satisfy one of the study criteria which is to protect the houses on the project shoreline.

Road Raising. This alternative would utilize the roadway as the primary protective element. The road would be raised and the protective element would be placed immediately adjacent to the roadway and be fronted by beachfill. This plan is similar to the reinforced dune plan with the dune located as far landward as possible or as part of the road to minimize cost and impacts. This option will not be evaluated further due to the same reason as causeway alternative.

The potential alternatives and the estimated conceptual costs are aummarized in Table A.37 for initial screening.

Table A-37 Cost Table for Alternative Screening Elements

Potential Alternative Solution		Unit Cost	Unit	Construction Cost	Remarks
A. Non-Structural Features					
	Buyout			$50,000,000	Based on 100 Houses
	Zoning			Not Costed	
	Flood Proofing			Not Costed	
B. Structural Features					
	Beach Nourishment (Initial + Renourish)	$1,800	LF	$22,320,000	Assume 12,400 Shore Length
	Floodwalls without Beachfill	$2,500	LF	$31,000,000	**Not Applicable**
	Rock Reinforced Dune w/Beachfill	$3,000	LF	$43,200,000	
	Bulkhead Seawall w/Toe Protection	$2,500	LF	$31,000,000	**Not Recommended**
	Groin or T-Groin Field w/Beachfill			$45,000,000	Assume 24 Groins or 12 T-Groins
	Offshore Breakwaters w/Beachfill			$50,000,000	Assume 10 Offshore Breakwaters
	Reduced Groins w/Advance Fill			$20,000,000	Assume up to 5 Groins
C. Regional Sediment Management Features					
	Sand Bypassing			$300,000/year	Assume 20,000 cy/year
	Installation of a Diffusion Pipe			$6,000,000	Assume 6,000 LF pipe length
	Modification to Jetties			$3,000,000	Assume jetty length=1,000 ft

Note: All Costs are preliminary and subject to change due to actual site condition

9.0 GENESIS MODEL EVALUATION OF GROIN ALTERNATIVES

In addition to initial beachfill and periodic nourishment, limited groin field or other shore protection structures constructed at the critical erosion shorelines can reduce future beach nourishment quantity and frequcecy, and post-storm beach repair. The statigically placed groin field also maintain a berm width required to protect the existing sheet pile bulkhead and seawall from storm damage due to due to combined storm surge and wave overtopping. The limited initial construction cost would be compensated with reduced future renourishment costs. The critical area identified for evaluation are illustrated in Figure A-31 and re-shown below as Figure A53 :

- West Critical Area – Approximately 1,000 ft shoreline fronting the steel bulkhead seawall west of the existing rock groin;
- East Critical Area – Approximately 2,500 ft shoreline fronting timber bulkhead seawall between house #100 and #200;+

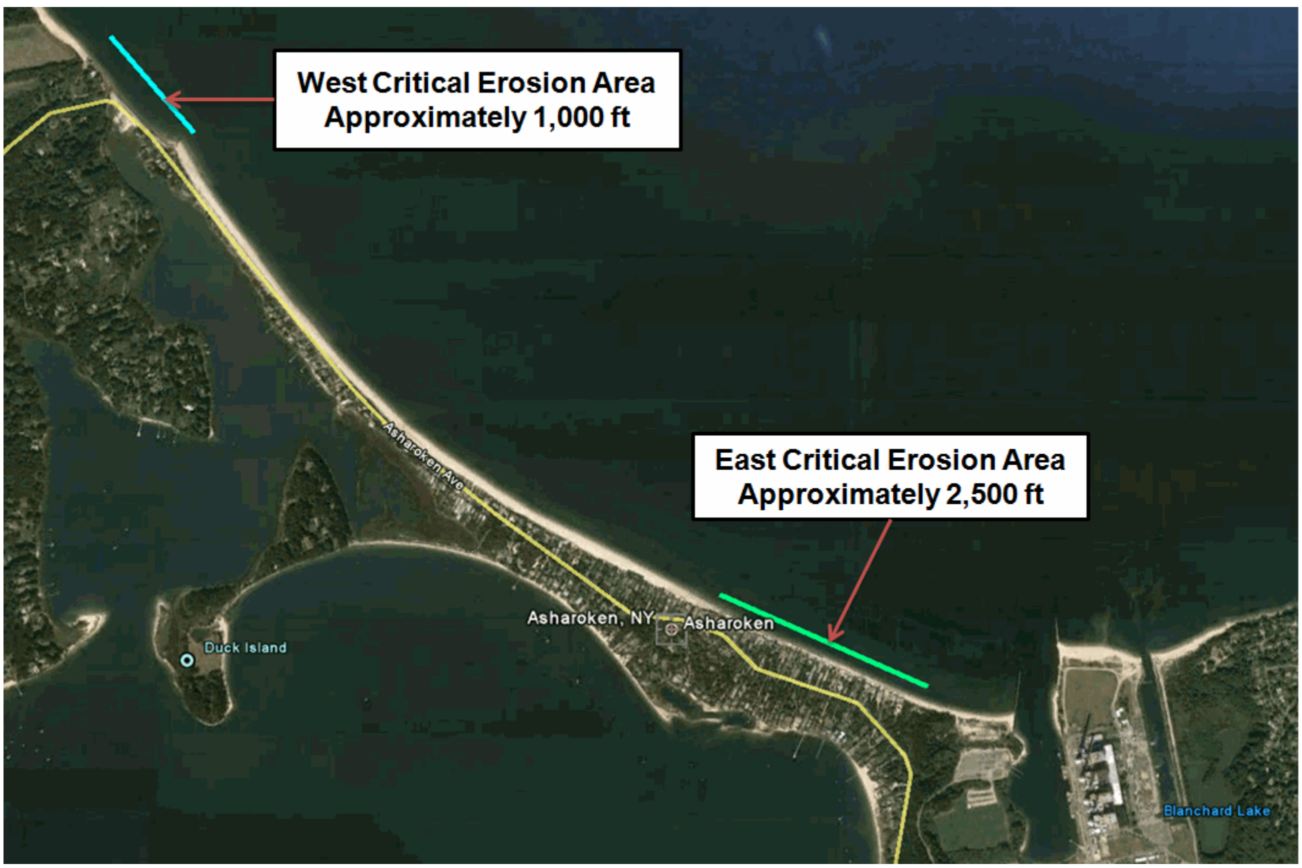


Figure A-53: Critical Erosion Areas

9.1 Model Evaluation

Various structural features are evaluated to determine the effectiveness of sediment retention and shoreline stabilization. The GENESIS model was used to evaluate the performances of each structural alternative including groins, breakwaters, and removal of existing groin. The model is capable of predicting shoreline responses with and without coastal structures. The model was validated with historical shoreline changes and the wave condition within the same period. Model simulations were carried out with alternative structural features. Details of model capability description, calibration, and simulation runs are presented in Attachment A2. The following discuss evaluation results for each critical area studied.

9.2 West Critical Area Alternatives Evaluation

The high erosion rate and frequent post-storm repairs in this critical area are attributed primarily to the existing rock groin located east of the critical shoreline. This groin retains most of the sediment supply from upstream (east), maintaining a healthy dune and beach profile east of the groin while causing downdrift (west) beach erosion. Three potential shore protection alternatives are evaluated in order to provide sediment retention in front of the bulkhead seawall and reducing future nourishment.

- Alternative W1 – Remove 50% Existing Groin;
- Alternative W2 – Remove 100% Existing Groin;
- Alternative W3 – Construct Tapered Groin Field;

Alternatives W1 and W2, modification of the existing groin, would enhance the sediment supply downdrift, while alternative W3 is to retain the design beach profile while keeping the existing groin intact. The calibrated GENESIS model was used to simulate the shoreline responses after the initial beachfill and compared with the beachfill only shoreline. The model simulation period is 10 years. Model simulation results are shown graphically in Figure A-54 (groin removal alternatives) and Figure A-55 (groin field alternative). Shoreline responses and estimated future nourishment requirements are listed in Table A-38. The model results are summarized as follows:

- Removal 50% of existing groin would provide minimal improvement to the critical shoreline;
- Total removal of existing groin would retain 15 ft wider beach (71% retention), and approximately 15,000 cubic yards nourishment reduction in 10 years;
- The new groin field would provide retention of 40 ft wider beach (total 96% retention) and approximately 40,000 cubic yards nourishment reduction in 10 years;
- Advantages of Alternative W2 is lower initial cost, however, it would cause a significant retreat of the updrift (up to 3,000 ft east) shoreline due to groin removal;
- Advantages of Alternative W3 is reduced future nourishment savings vs. initial groin construction cost;
- the adverse effect of downdrift erosion can be mitigated with shoreline tapering and providing feeder beach;

Alternative W3, tapered 3-groin field is considered for further evaluation in the alternative evaluations based on reduced frequency of nourishment and cost and ability to retain the design beach width.

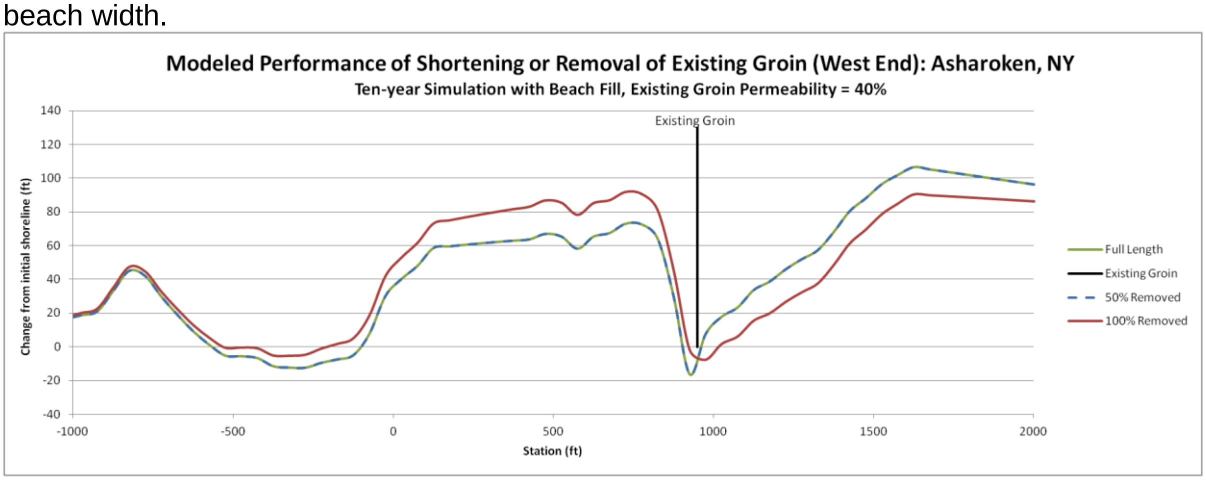


Figure A-54 GENESIS Simulation on Groin Removal

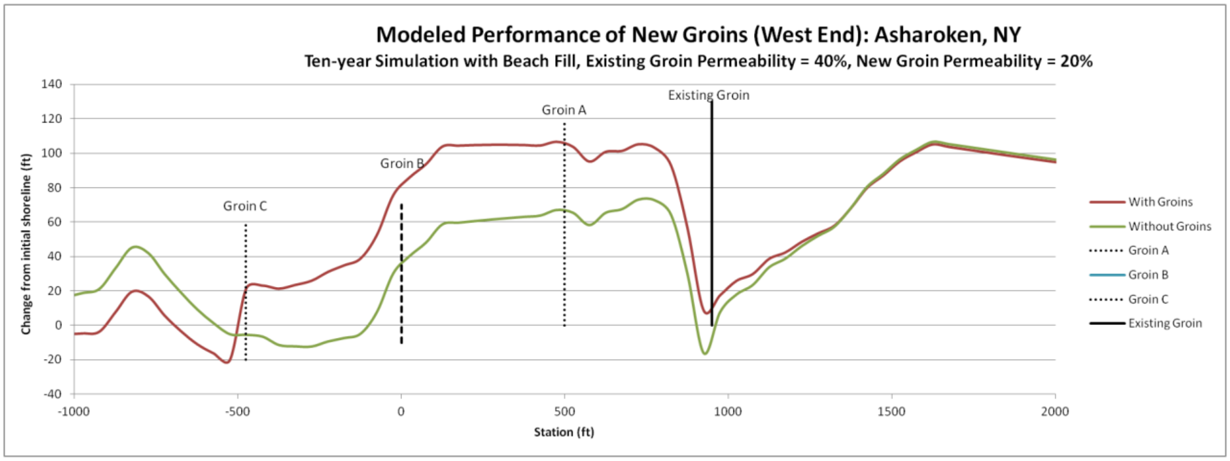


Figure A-55 GENESIS Simulation on Beachfill with 3-Groin Field

Table A-38 GENESIS Modeling Results Summary-West Shoreline Alternatives

West Shoreline Alternatives, Station 0+00 to 10+00, 1,000 ft Shoreline Length				
	Beachfill Only Baseline Condition	Alternative W1 Remove 50% Existing Groin	Alternative W2 Remove Existing Groin	**Alternative W3 Construct Tapered 3-Groin Field**
Design Beach Width (ft)	100	100	100	**100**
Beach Width after 10 years	56	56	71	**96**
10-year Nourishment Volume (CY)	44,000	44,000	29,000	**4,000**
10-year Nourishment Savings (CY)	-	0	15,000	**40,000**
Advantages	-	none	Continuous Littoral Transport	**Retain 95% Shoreline Width**
Disadvantages	-	Will De-stabilize Updrift Shoreline	Will De-stabilize Updrift Shoreline	**Need Mitigate Downdrift Erosion**

9.3 East Critical Area Alternatives Evaluation

Four sediment retention alternatives are evaluated using the calibrated GENESIS model:

- Alternative E1 – two terminal groins or offshore breakwaters;
- Alternative E2 – sediment pocket with two terminal groins and one breakwater;
- Alternative E3 – three low crest, short groins;
- Alternative E4 – eitght to twelve low crest, short groins constructed with even spacing including tapers;

The alternatives are simulated with GENESIS model for a ten-year period. Graphic shoreline responses are presented in Figures A-56 and A-57. The beach width changes with estimated nourishment volument requirements are shown in Table A-39. The shoreline response graphics indicates that the there is no difference in the performance of breakwater or groin, therefore, groins are used for comparison due to lower initial cost. The simulation results are summarized as follows:

- The performances of groin and breakwater are similar, therefore, breakwater structure is not considered further due to higher initial cost;
- The sediment pocket containment system is slightly more effective in sediment retention, however, is not enough to justify the higher construction cost;
- All alternatives except Alternative 4 are not justified if considering the sediment retention capability or the nourishment reduction;
- Alternative E4 is justified due to better retention capability (70% vs. 47 to 49%);
- Groin field would be justified if the advantages of reduced nourishment savings tops the initial groin construction cost;

Alternative E4 is recommended based on the highest reduction of frequency and quantity of renourishment and is more efficient to retain the design beach width. The downdrift effect of Alternative E4 can be mitigated with advance fill taper and feeder beach west of the groin field;

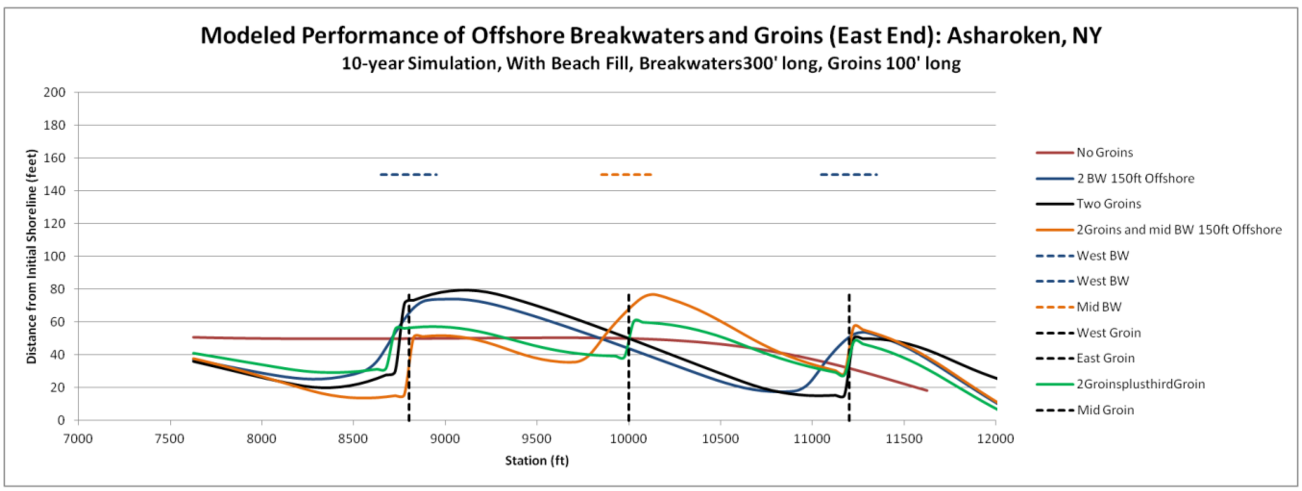


Figure A-56 GENESIS Simulation on Beachfill with Breakwater, 3-Groin Field, and Combination

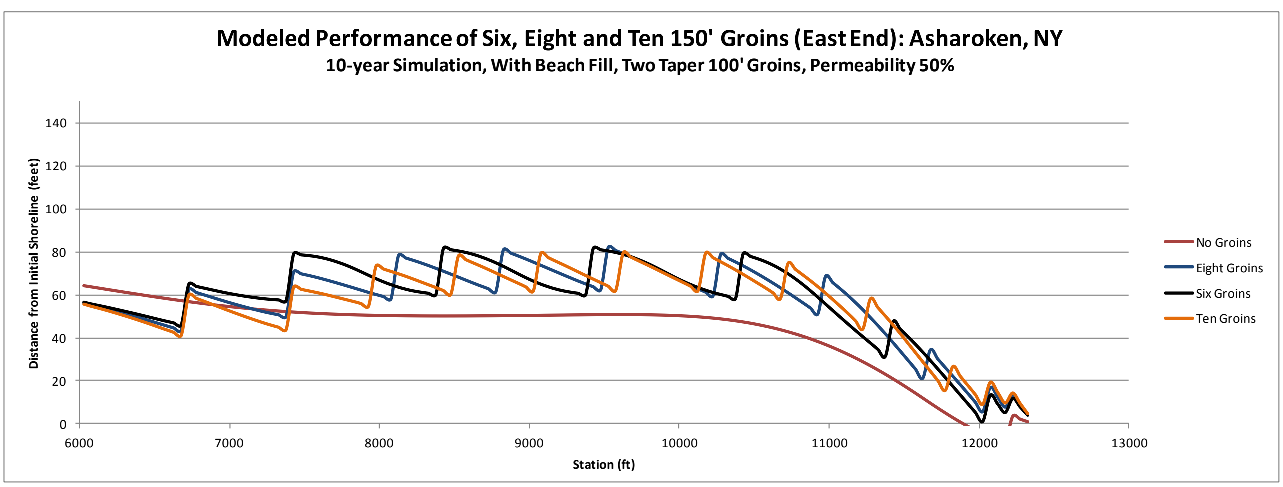


Figure A-57 GENESIS Simulation on Beachfill with 6, 8,and 10 plus 2 tapered groins for each Groin Field Alternbative

Table A-39 GENESIS Modeling Results Summary-East Shoreline Alternatives

East Shoreline Alternatives, Station 87+50 to 112+50, 2,500 ft Shoreline Length					
	Beachfill Only Baseline Condition	Alternative E1 Two Terminal Groins or Breakwaters	Alternative E2 Two Terminal Groins and One Breakwater	Alternative E3 Three Low Crest, Short l Groins	Alternative E4 Eight Low Crest, Short Groins
Design Beach Width (ft)	100	100	100	100	100
Beach Width after 10 years	47	49	49.5	49	70
10-year Nourishment Volume (CY)	132,750	127,250	126,250	127,750	75,000
10-year Nourishment Savings (CY)	0	5,500	6,500	5,000	60,000
Advantages	-	Retain Design Beach Updrift of Structure	Retain Design Beach Updrift of Structure	Downdrift Effect is Minimized and Retains Design Berm Width	Downdrift Effect is Minimized and Retains Design Berm Width
Disadvantages	-	May Cause Downdrift Erosion	May Cause Downdrift Erosion	-	High Initial Cost

Notes:

1. Beach width changes are based on GENESIS model results, East critical shoreline includes 50 ft advance fill and periodic nourishment from power plant;
2. Volume estimate is based on the assumption 1 ft shore change = 1 cubic yard volume change;
3. Required nourishment = volume necessary to restore to design berm width;
4. Ten year nourishment savings = Baseline Condition Volume – 10-year with project volume

9.4 Modeling of the Effect of Offshore Hole

Additional model simulation of the effect of offshore hole located north of the eastern critical shoreline (figure 5) was performed to determine the response of shoreline changes assuming the hole was filled with sand. Both the without and with beachfill shoreline conditions were simulated. The model results indicated a very limited changes of shoreline responses with or without the offshore hole. The reason for the limited changes is that the entire offshore bathymetry has been adjusted in response to the hole since the creation of the offshore hole in the 1960's. Therefore, it would take a similar long period of time (on the order of 50 years) to re-adjust the offshore bathymetry back to normal condition.

10.0 SHORT LISTED ALTERNATIVES

Based on the alternatives screening and GENESIS modeling results, the more cost effective alternative measures that meet the project objectives are the Beachfill Only Plan, the Reinforced Dune (buried stone seawall) with Beachfill Plan, and the Reduced Groin Field with Beachfill Plan. These measures are developed further over varying areas of coverage with conceptual plans for comparison and selection. The alternative plans considered below will address the problems for the entire project shoreline with alternative improvements. The structural alternatives below will potentially accomplish storm damage reduction to economically justify a project recommendation.

<u>Alternative 1 – Beach Fill Only Plan</u>

This alternative includes 12,400 linear feet of beach berm and dune fill, from intersection of Bevin Road and Asharoken Avenue south, east to the west jetty of the power facility's inlet basin. The beach design template includes a dune height at elevation +15 ft NAVD with a 15 ft dune crest width, landward and seaward dune slopes of 1V:3H, a 50 ft berm width at elevation +8 ft NAVD and a foreshore slope of 1V:15H to the existing bottom. The dune includes beach grass on the dune crest and landside slope, and sand fence on the dune seaward slope for dune enhancement and long-term performance. Figure A-58 and A-59 show typical dune and beachfill and typical berm fill only sections. Note that the berm fill only section in front of the existing bulkhead at the southeastern 6,200 ft shoreline as shown in Figure 59 does not include advance nourishment.

The dune alignment fronting the Section 103 project will be shifted slightly seaward with 300 ft to 500 ft transitions at each end, to be able to wrap around the steel bulkhead seawall. This alternative will require approximately 20,000 cy/year of re-nourishment with 3-year renourishment period and 50 ft advanced berm fill in Reach 2. The estimated renourishment quantity and frequency are based on historical beachfill and seawall repair records and are further discused in the Alternatives Comparison section.

The borrow area for initial construction will be from an offshore source, identified as Borrow Area A, in the Long Island Sound approximately 3 miles north of the project area. Future renourishment source will be from upland, which will be more expensive than the offshore source.

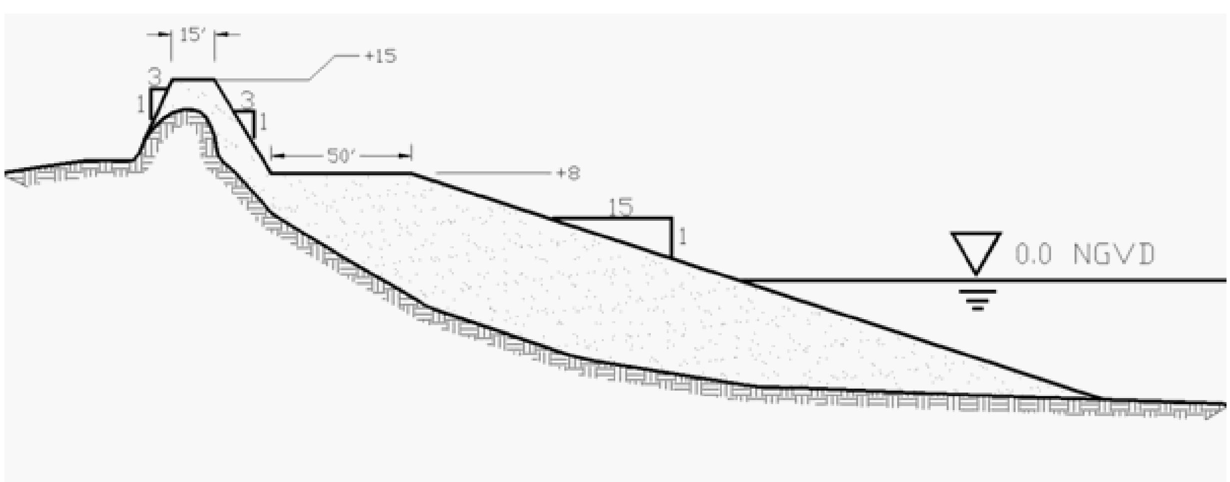


Figure A-58 Typical Dune and Beachfill Section

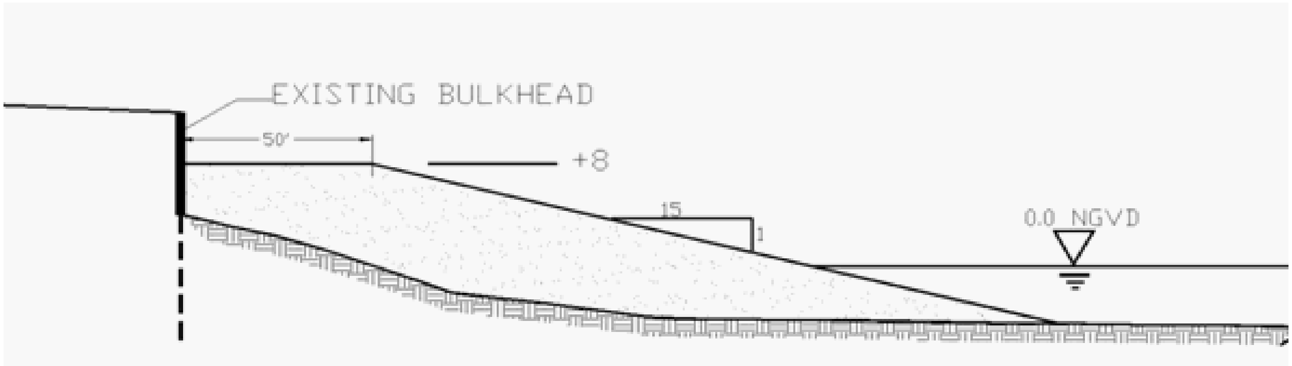
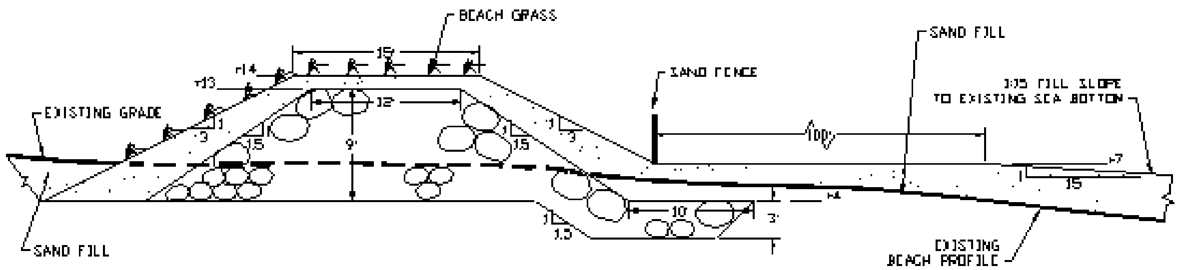


Figure A-59 Typical Berm Fill Section in front existing bulkhead (Advance Fill not included)

Alternative 2 – Reinforced Dune (Buried Stone Seawall) with Beachfill Plan

This alternative includes 12,400 linear feet of beach berm, reinforced dune and dune fill cover for the same project length as Alternative 1. The sand dune design template has a crest width of 15 ft at elevation +13 ft NAVD and both the seaward and landward slopes of 1V:3H that completely encapsulate a trapezoidal shaped stone seawall of crest width 10 ft at elevation +11.5 to +12 ft NAVD with 1V:1.5H side slopes. The seawall has a crest cover of minimum 1 ft of sand and a 0.5 ft. of topsoil for dune grass planting at the crest and backslope. Sand fence is included to keep the sand cover intact as best as possible. The dune alignment, as with Alternative 1, is shifted seaward with 500 ft transition in the area of the Section 103 project to be able to front the project's bulkhead. For the 6,200 ft southeastern shoreline, a low-profile dune will be designed to accommodate with the existing bulkhead elevation. The renourishment of this alternative will include approximately 200,000 cubic yards of fill every 10 years through the 50-year project life. Due to reinforcement, the renourishment frquency is reduced from once every 5 year to 10 year, and the post-storm repair will not be necessary. The borrow area for initial construction is the same offshore area used for Alternative 1. The reduction in beach nourishment frequency is due to reduced risk of storm damage by dune reinforcement.

The additional advantage of Alternative 2 is the lower required dune crest elevation for the same level of protection. This is because Alternative 2 can tolerate more overtopping (crest can be set lower) due to its buried splash blanket protection, landside of the dune, to protect undermining of the structure. Alternative 1, which has no splash blanket protection and is subject to dune lowering, therefore permits smaller threshold overtopping (with a higher required dune elevation) to preclude damage to the deformable dune. This lower dune elevation allows for less obstructed views (by approximately 2 feet), which may be more favored by the community. A typical reinforced dune section is shown in Figure A-60.



BURIED SEAWALL SECTION
WITH BEACH FILL
(NOT TO SCALE)

Figure A-60 Typical Reinforced Dune Section

<u>Alternative 3a – Combination Reinforced Dune at Eastern 6,200 ft and Beachfill for the Rest Plan</u>
This plan includes 6,200 ft of beach and dune fill from Bevin's Rd. south (same as Alternative 1) and 6,200 ft of beach fill with reinforced dune (same as Alternative 2) from the southern border of non-reinforced dune to the west jetty of the power plant facility. Renourishment is the same for each 6,200 ft. reach as their associated alternatives. Post-storm damage repairs will not be necessary assuming both critical areas will be reinforced.

<u>Alternative 3b – Combination Reinforced Dune at Western 6,200 ft and Beachfill for the Rest Plan</u>
This plan includes 6,200 ft of reinforced dune with beach fill from Bevin's Rd. south (same as Alternative 2) and 6,200 ft of beach fill only (with advance fill, same as Alternative 1) from the southern end of reinforced dune with beach fill to the west jetty of the Keyspan power plant facility. Renourishment is the same for each 6,200 ft. reach as their associated alternatives. Post-storm damage repairs will not be necessary assuming both critical areas will be reinforced.

<u>Alternative 4 – Beachfill with West Groin Field Only Plan</u>
In addition to the initial beachfill, this alternative provides groin field protection at the west critical erosion area to protect against storm damage and reduce renourishment frequency and quantity. A groin field identified as Alternative W3 in Table A-38 will be constructed along the existing Section 103 steel bulkhead west of the existing (150 ft) rock groin in Reach 1a (Figure A-31). This groin field consists of three rock groins with lengths at 120 ft, 100 ft, and 80 ft tapering from east to west. The layout of the 120 ft and 100 ft groins will be on top of the footprint of the remains of pre-existing rock structure. The purpose of this re-establishment of groin field is to retain a design beachfill width necessary to provide toe protection of the steel bulkhead seawall and to reduce wave height and runup, overtopping at this critical shoreline.

Historically, this section of bulkhead seawall experienced frequent storm damage due to disruption of sediment supply caused by the existing rock groin and storm damage of the existing groins in front of the seawall. An option of shortening or removal of the existing 150 ft groin was considered but discarded due to concerns of potential de-stabilization of the upstream (eastern) dune and beach shoreline. In addition, the shoreline response of groin shortening is not as effective as the groin field as demonstrated by the GENESIS model results. The advantages of re-establishing the west groin fields are:

- Stabilize existing Section 103 steel bulkhead seawall and reduce frequent storm damage repair costs. The estimated nourishment volume reduction is a total 200,000 cy for the 50 year project life and less frequency of renourishment ;
- Maintain the integrity and stability of the updrift dune and beach shoreline without shorting or removal of the existing rock groin;
- Eliminate the frequent post storm seawall repair;
- As an added benefit, the added and stabilized beachfront with dune cross-over structure can be used as one of the required public access location required by the project.

The potential downdrift erosion will be mitigated with the following measures:

- Provide advanced fill tapers at the downdrift shoreline, approximately 500 to 1,000 ft in shoreline length;
- Provide feeder beach in the form of higher dune and wider berm to compensate possible downdrift erosion;
- Conduct frequent post-construction monitoring and place future nourishments at the high-erosion area;

<u>Alternative 5 – Beachfill with both West and East Groin Field Plan</u>
In addition to the initial beachfill, this alternative provides groin field protection at both the west and east critical erosion areas to protect against storm damage and reduce both renourishment frequency and quantity. In addition to Alternative 4, a groin field layout identified as Alternative E4 in Table A-39 will be constructed along the east critical erosion area and tapering along the entire Reach 2 shoreline (Figure A-31). This additional east groin field consists of eight rock groins with lengths ranging from 80 to 120 ft, and with average spacing of 800 ft.

The purpose of this east groin field is to retain a design beachfill width necessary to provide toe protection of the critical erosion area seaward of the existing timber bulkhead. Historically, this section of bulkhead experienced frequent storm damage due to disruption of sediment supply by the Power Plant jetties and an existence of Nodal Point which accelerate the erosion rate. Based on the sediment transport analysis, the shoreline erosion rate at the east critical area range from 10 to 15 ft/year by comparing historical shoreline changes and the frequency of sediment bypassed and placed in this area.

The advantages of adding the east groin field are:

- Stabilize the east critical erosion shoreline and reduce frequent storm damage repair costs;
- The estimated volume reduction base on GENESIS model result is approximately 960,000 cy in 10 years with longer renourishment period;

The potential downdrift erosion will be mitigated with the following measures:

- Provide advanced fill tapers and taper groins at the downdrift shoreline, approximately 1,000 ft in shoreline length;
- Provide feeder beach in the form of higher and wider dune and berm widths to compensate the erosion;
- Conduct frequent post-construction monitoring and place future nourishments at the high-erosion area;

11.0 ALTERNATIVE COMPARISON AND SELECTION

The preliminary project costs for each of the alternatives are annualized over the project life and compared on total annual cost basis. Total annual costs include annualized Initial construction cost, monitoring, and annual renourishment costs. The preliminary cost estimates are based on the Cost Appendix. All costs shown are for comparison only. These costs and the estimated effectiveness are used to rank each alternative plan. The effectiveness of each alternative are measured on how much storm or erosion damages are reduced or avoided which are classified as benefits. In addition, the success of the alternatives will depend on future renourishment requirements and potential adverse impacts which requires mitigation.

11.1 Renourishment Estimates

The renourishment quantity were estimated based on sediment budget and the GENESIS model results. The following summarize assumptions used for the alternative renourishment estimates. Estimated renourishment quantities and frequency are summarized in Table A-40.

- Coastal structures reduce the renourishment frequency and/or quantity;
- Post-storm nourishments are required for (east and west) critical erosion areas;
- Quantity estimates are based on GENESIS modeling results with assumption of 1 ft of beach erosion being equivalent to 1 cy/ft of shoreline;
- Alternatives 2 and 3 will not need frequent nourishment assuming the buried seawall would maintain its design function for toe protection between scheduled nourishments;

Table A-40 Alternatives Renourishment Quantity Estimates

	Scheduled Renourishment			Total Volume in 50 years (cy)
Alternative	Renourish per Cycle (years)	Annual (cy/yr)	Quantity/Cycle (cy)	
1-Beachfill Only	3	20,000	60,000	1,000,000
2-Full Buried Seawll	10	20,000	200,000	1,000,000
3-Half Buried Seawall	10	20,000	200,000	1,000,000
4-West Groin Field Only	5	16,000	80,000	800,000
5-West & East Grin Field	10	10,000	100,000	500,000

Notes:
1. Sediment bypassing from power plant is accounted for and included in the scheduled nourishment;
2. The scheduled renourishment quantity is based on sediment budget analysis;
3. Volume reduction quantities are based on GENESIS model results.

As shown in Table A-40, more frequent renourishment cycles are required if the critical erosion shoreline were not reinforced or beachfill retained with groins. Based on historical shoreline and sediment budget analysis (figures A-61), both the west and east critical areas experienced up to 10 ft/year erosion rate, therefore, a 3-year renourishment cycle is required. The erosion rate is greatly reduced with groin field in place (Figure A-62). As illustrated in the with-project sediment budget diagram, mitigations will be needed downdrift of the proposed groin fields.

The downdrift mitigation volumes maybe provided as advanced fill during initial construction. Additional mitigation will be provided during each renourishment cycle.

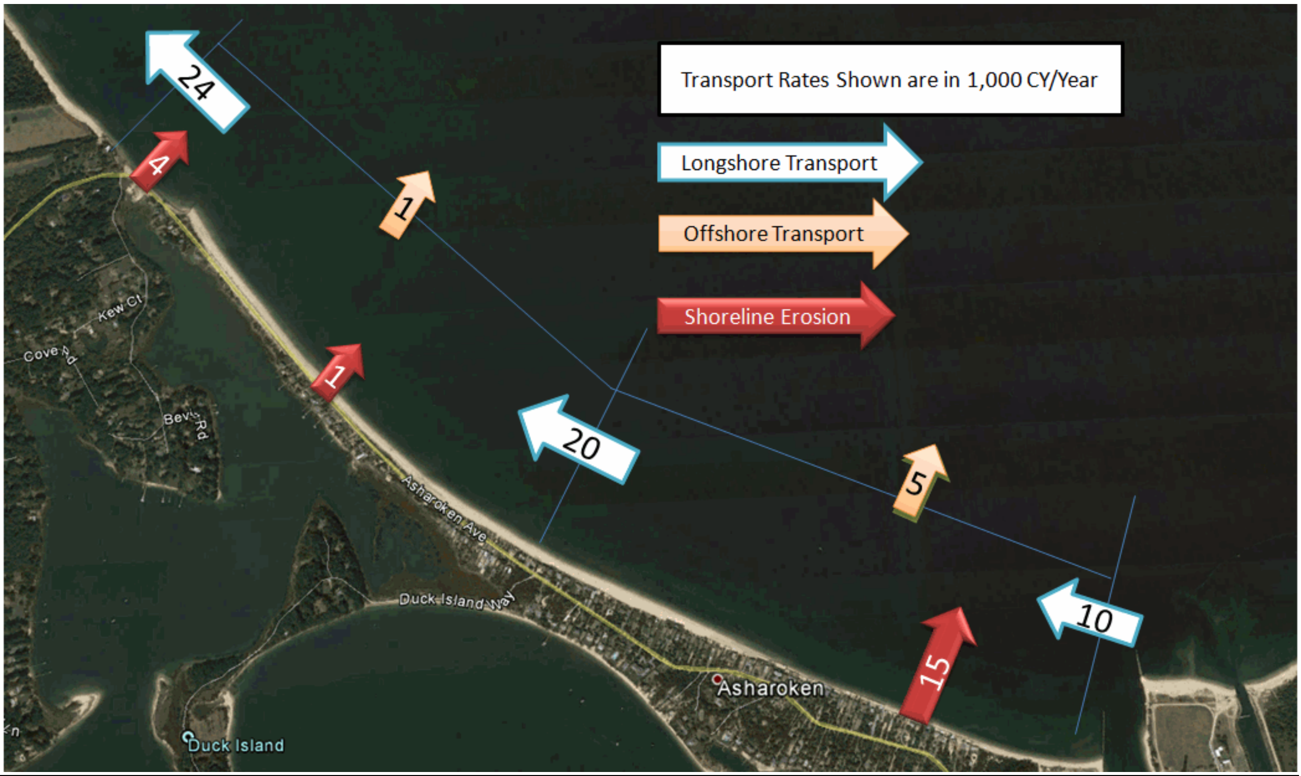


Figure A-61 Sediment Budget without Groin Field

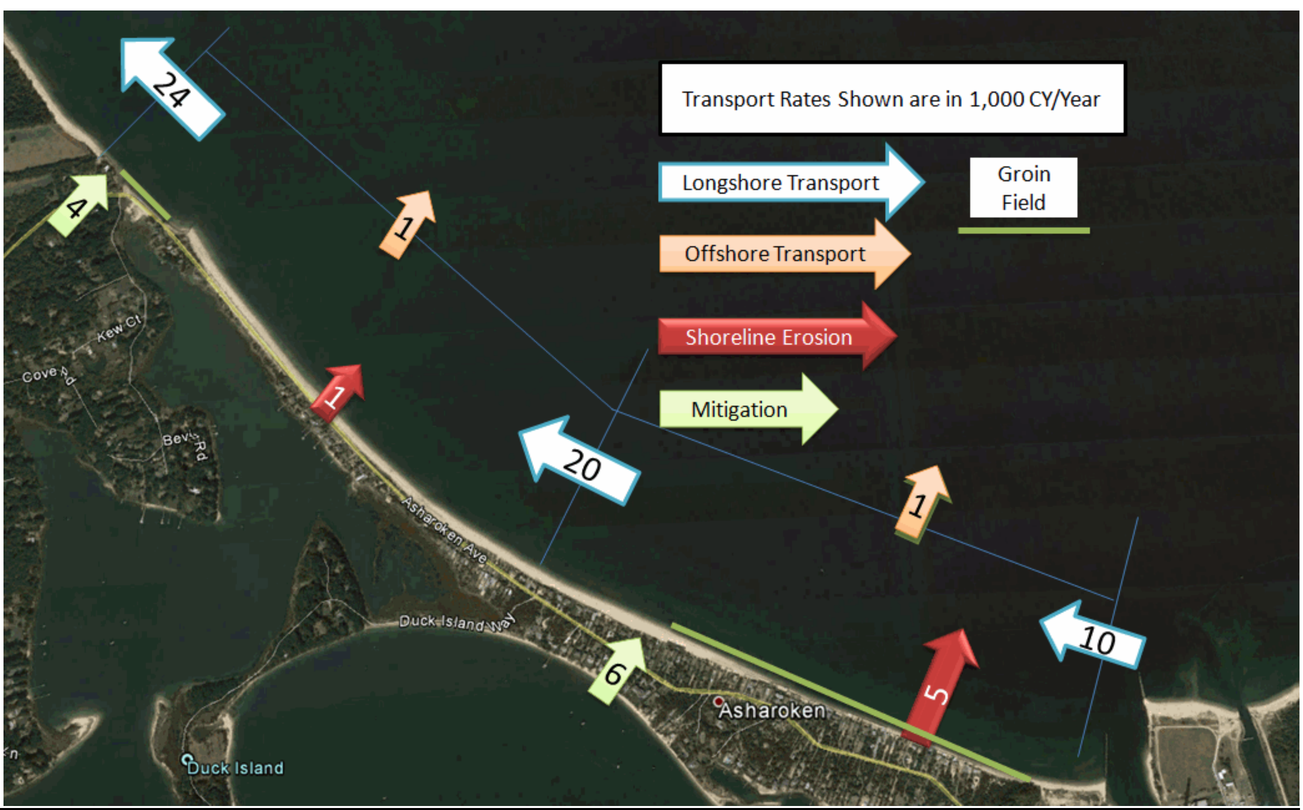


Figure A62 Sediment Budget with Groin Fields

11.2 Initial Construction Costs

The initial construction costs are based on the conceptual design quantity estimates and MCACES costs. Details of the of the design plan and typical sections are discussed in Section A12. Details of the cost estimates are shown in Cost Appendix.

11.3 Monitoring Costs

Project monitoring costs include beach profile monitoring and environmental monitoring/mitigation. Monitoring items, frequency, and estimated costs are summarized below and costs are annualized for cost/effectiveness comparison:

Environmental Monitoring/Mitigation:

- Habitat/finfish and benthic monitoring;
- Plover monitoring and:
- In-house labor;
- The 4 years mentioned above consists of: 1 year pre-construction (during P&S) and 3 consecutive years post-construction.

Beach Profile Monitoring/Downdrift Mitigation (Mitigation Monitoring Only):

- Post-construction monitoring twice per year for 5 years;
- Post-storm monitoring and analysis for 5 years;
- Mitigation survey, analysis and report;

Offshore Bathymetry and Post-Borrow Site Monitoring:

- Pre- and Post-construction offshore bathymetric monitoring at year 1, 3, and 5;
- Survey, analysis and report.

11.4 Operations, Maintenance, Repair, Replacement and Rehabilitation (OMRRR)

The local partner will be responsible for the Operations and Maintenance (O&M) of the Asharoken Project. The O&M Responsibilities will be provided in greater specificity in the OMRR&R Plan (Operations, Maintenance, Repair, Replacement and Rehabilitation Plan), which is provided to the partner after completion of initial construction and describes the specific requirements of the non-Federal partner. Anticipated OMRRR activities for this project are listed below:

Administrative and Operational Responsibilities:

- Maintain public ownership and public use of the Project Area which are the basis of the Federal participation in the project. This includes preventing trespass or encroachment by private interests by the placement, onto these shores or within the seaward portion of the project, of any temporary or permanent structures, except as specifically permitted by the District Engineer or authorized representative.
- Prohibit any excavation of or construction on, over, under, or through the dune, beach berm, groins or other project elements without prior written approval of the District Engineer or authorized representative
- Prohibit alterations in any feature of the beach fill that may affect its functional performance unless prior written approval has been obtained from the District Engineer
- Prohibit unauthorized vehicular traffic on the beach and restrict authorized vehicle access to authorized access ways.

- Assure that no drains discharge onto the beach.
- Perform day to day operations of the facilities.
- Permit the District Engineer or authorized representative access to the project at all times.
- Maintain organized records of activities and costs covering maintenance, operation, inspection, repair and replacement of protective works
- Participate in a yearly joint inspection of the project with personnel from the New York District.
- Ensure that safe operation of recreational activities continues during construction and maintenance operations.

Maintenance Responsibilities:
- Grade and reshape the design berm and beach to original elevations to repair erosion caused by wind or wave action, or loss of elevation caused by human activities
- Prevent sand from blowing off the dune and berm onto nearby streets and into adjacent properties, including deploying and keeping sand fences in an upright position and in serviceable condition.
- Maintain dune crossovers or other accessways in good repair.
- Undertake Quarterly Inspections of all project elements including beach width measurements as well as before and after each tropical and extratropical storm.

Reporting Responsibilities:
- Provide semi-annual Inspection Reports
- Provide organized records of activities and costs covering maintenance, operation, inspection, repair and replacement of protective works.
- Contact the District Engineer if at any time storm or other erosion reduces the berm to below the minimum beach fill cross-section width and maintenance measures to move sand from accreted areas to eroded areas prove inadequate to restore the design section.

OMRRR Costs:
Operation, maintenance, repair, replacement, and rehabilitation of the alternatives include costs for routine beach profile maintenance and repair, and rehabilitation of the rock groins. Beach profile maintenance costs are part of periodic beachfill monitoring and renourishment. The annual rock groin maintenance costs are estimated as 0.5% of the initial construction cost plus mobilization/demobilization of equipment and maintenance personnel.

11.5 Alternatives Comparison

The alternatives costs are compared based on estimated initial beachfill quantity, coastal structures, monitoring, OMRRR costs, and estimated future nourishment quantity and frequency. The preliminary costs are based on conceptual design and estimated quantities. The initial construction cost and future nourishment costs are annualized with 3.375% annual interest rate for 50 years of project life. The estimated quantities and costs are summarized in Table A-41. In addition to the cost/effective comparison, alternatives are compared based on considerations including solutions to the existing problems, frequency of repair, potential impacts and mitigation requirements.

11.6 Alternatives Selection

The initial alternative selection is based on estimated annual costs, future nourishment requirements, monitoring, maintenance costs and estimated benefits summarized in Tables A-41**.** As shown in this table, Alternatives 1, 4 and 5 are within the benefit/cost estimate tolerance range and are narrowed down for final TSP consideration. Each of the three has its own advantages and disadvantages. In addition, the three alternatives provide same quantity of initial fill and level of protection, with or without groin fields. The final selections are based on further considerations of local preferences, downdrift shoreline effects and mitigation requirements, frequency of renourishment, and the frequency of storm damage repairs. Applicable NYSDEC water quality permit are also included in the consideration. Table A-42 provides a list of advantages/disadvantages of the three alternative to facilitate final selection:

Table A-41 Alternatives Comparison

Asharoken, Long Island, New York					
	Alternative 1	**Alternative 2**	**Alternative 3**	**Alternative 4**	**Alternative 5**
	Beachfill Only	Beachfill and Buried Seawall-full shoreline	Beachfill and Buried Seawall-half shoreline	Beachfill with West Groins	Beachfill with West and East Groins
Initial Fill Volume (CY)	600,000	375,000	450,000	600,000	600,000
Coastal Structures	n/a	buried seawall	partial buried seawall	3 rock groins	11 rock groins
Nourisment (cy/period)	60,000 cy/3 yrs	200,000 cy/10 yrs	200,000 cy/10 yrs	80,000 cy/5 yrs	100,000 cy/10 yrs
Total Nourishment in 50yrs	1,000,000 cy	1,000,000 cy	1,000,000 cy	800,000 cy	500,000 cy
COSTS					
Initial Construction Cost	$21,552,000	$66,931,000	$45,940,000	$23,665,000	$32,426,000
Annualized Initial Constr.	$734,000	$2,310,000	$1,579,000	$806,000	$1,114,000
Annual Nourishment Cost	$1,143,000	$997,000	$997,000	$883,000	$504,000
Annualized Monitoring Cost	$50,000	$50,000	$50,000	$50,000	$93,000
Annual OMRR Cost	$147,000	$353,000	$259,000	$156,000	$196,000
Total Annual Cost	$2,074,000	$3,710,000	$2,885,000	$1,895,000	$1,907,000
Annual Damage Benefits	$2,570,900	$2,570,900	$2,570,900	$2,570,900	$2,570,900
Net Benefit:	$496,900	-$1,139,100	-$314,100	$675,900	$663,900
Benefit/Cost Ratio:	**1.24**	**0.69**	**0.89**	**1.36**	**1.35**

Notes
1. All quantities and costs shown are conceptual and are for comparison only;
2. Nourishment quantities are preliminary and are based on sediment budget study and GENESIS Modeling estimates;
3. Annual costs are developed based on 50 year project life;
4. Cost estimates are based on M2 estimates as shown in the Cost Appendix;

Table A-42 List of Advantages/Disadvantages of Selected Alternatives for Consideration

	Alternative 1	**Alternative 4**	**Alternative 5**
	Beachfill Only	Beachfill+3 West Groins	Beachfill+11 Groins
Initial Fill Volume (CY)	600,000	600,000	600,000
Coastal Structures	n/a	3 rock groins	11 rock groins
Nourisment (cy/period)	60,000 cy/3 yrs	80,000 cy/5 yrs	100,000 cy/10 yrs
Total Nourishment in 50yrs	1,000,000 cy	800,000 cy	500,000 cy
Advantages	Low Initial Cost	Reduced Erosion Rate	Reduced Erosion Rate
		Reduced Nourishment	Reduced Nourishment
		Volume and Frequency	Volume and Frequency
		Stabilized West	Stabilized both East and
		Shoreline	West Shoreline
		Reduced Seawall Damage	Reduced both Seawall and
			Timber Bulkhead Damages
Disadvantages	Frequent Nourishment	Need Downdrift Mitigation	Need Downdrift Mitigation
	Frequent Seawall and		
	Bulkhead Damage Repair		

12.0 Recommendation and Description of Tentatively Selected Plan (TSP)

Alternatives 1, 4 and 5 are in the same cost-benefit tolerance range with net annual benefits approximately $0.5 to 0.7 million and B/C ratio at 1.24 to 1.36. As a result, all three alternatives are included for consideration towards final TSP selection.

Alternative 4 is identified as the Tentatively Selected Plan based on the following justifications:

a. Alternative 4 is the best cost-efficient project based on NED criteria;
b. Alternative 4 provides a system-wide erosion control approach of the entire project shoreline, including initial fill of project shoreline, advance fill at eastern shoreline which provides advanced nourishment for the downdrift bech, and providing short groins downdrift of the existing stone terminal groin to mitigate downdrift erosion;
c. Reduces potential downsrift impact and necessary mitigation of alternative 5 at the eastern critical area by keeping 3 west groins only;
d. More applicable NYSDEC WQC permit with proposed groins constructed over the existing rock footprints (Figure A-63);
e. Less populated downdrift beach and shoreline, providing ample space for beachfill taper, construction of taper groin, and stockpile (in the form of dune and high berm) for downdrift mitigation;

Figure A-63 Proposed West Groin Field Layout

<u>Description of TSP</u>

The Tentatively Selected Plan and the typical sections are described below. The proposed plan includes approximately 2.4 miles of beachfill, 3 rock groins, and periodic nourishment. For design layout, a design baseline is set up along the existing permanent features including the existing steel bulkhead seawall at the western border located at east of Eatons Neck Road, the northern border of Asharoken Avenue, and the timber bulkhead along the eastern project shoreline. Details of the Tentatively Selected Plan are described below.

Beachfill at East Critical Area (House #100 to 200, Reach 2)

- Average Added MHW Beach Width: 110 ft;
- Average Beachfill Volume per (100 ft) House Lot: 4,500 Cubic Yards;
- Volume Estimates are based on 2001 survey and Post-SANDY LIDAR;

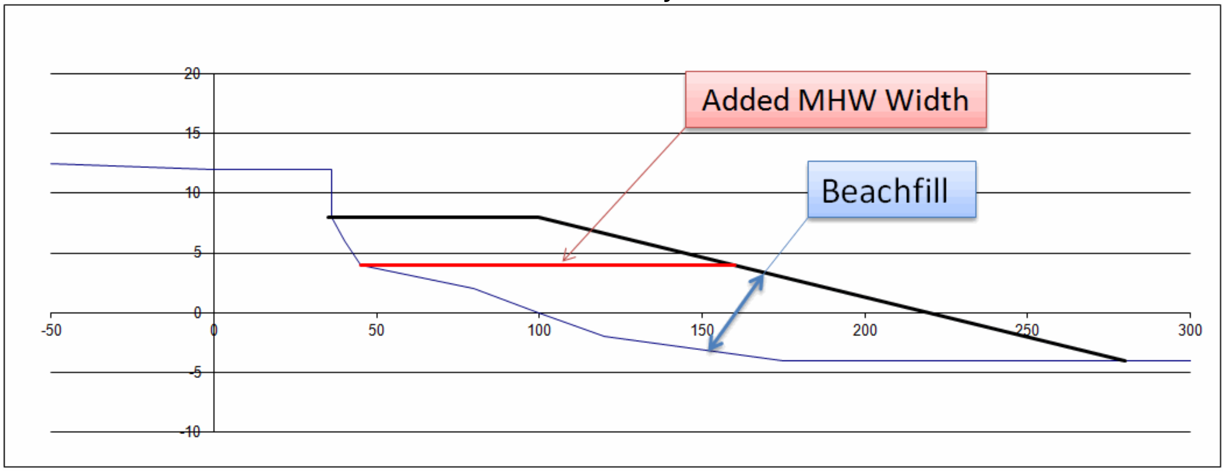


Figure A-64: Typical Beachfill at East Critical Area (House #100 to 200, Reach 2)

Beachfill at Private Beach Lot Area (Reach 1b)

- Average Added MHW Beach Width: 50 ft;
- Average Beachfill Volume per (20 ft) Beach Lot: 1,000 Cubic Yards;
- Volume Estimates are based on 2001 survey and Post-SANDY LIDAR;

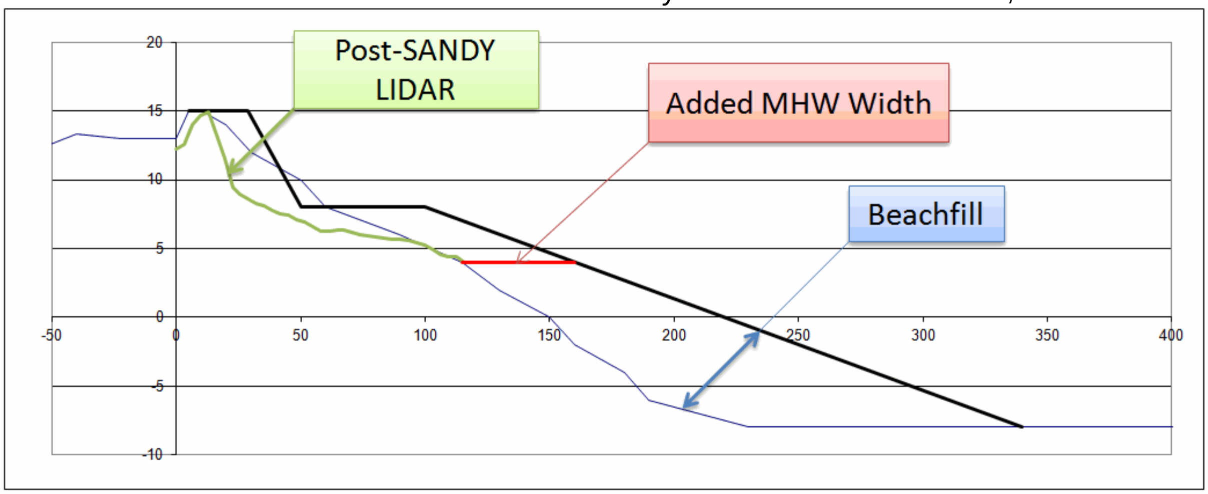


Figure A-65: Typical Beachfill at Private Beach Lot Area (Reach 1b)

<u>Station 0+ 00 to 9+00 – Initial 100 ft Width Composite Beachfill with Three Rock Groins and 500 ft Beachfill Tapers</u>

Beachfill with combination of high berm at +12 ft NAVD and low berm at +8 ft will be provided at this 900 ft shoreline fronting the existing steel bulkhead seawall. The crest width of the +12 ft berm is 50 ft, with 1 vertical on 5 horizontal seaward slope. The +8 ft berm is 30 ft wide with a 1 vertical on 15 horizontal foreshore slope (Figure A-66). The composite beachfill will provide storm wave protection to the existing bulkhead seawall. The 100 ft wide composite berm width will be retained with three new rock groins located at stations -5+00, 0+00 and 5+00. The groin lengths are 120 ft, 100 ft, and 80 ft in length respectively tapering from southeast to northwest, with crest elevation at +9 NAVD. The typical section and profile are shown in figure A-67. Note that the terminal groin at station -5+00 may require longer trunk section in order to tie into the existing toe of bluff to avoid structural flanking.

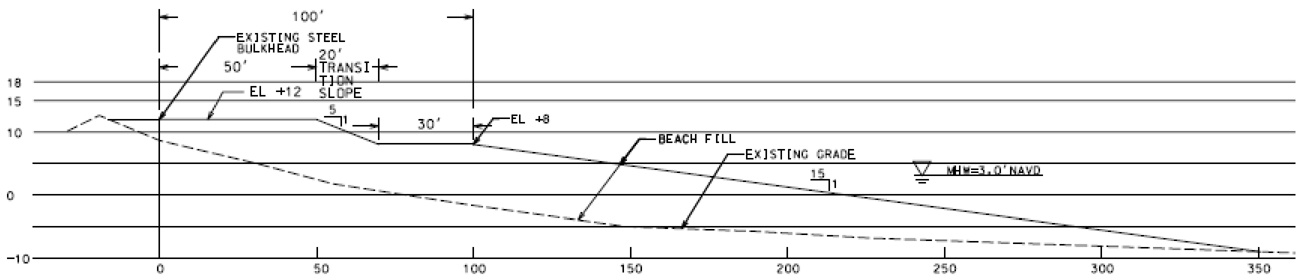
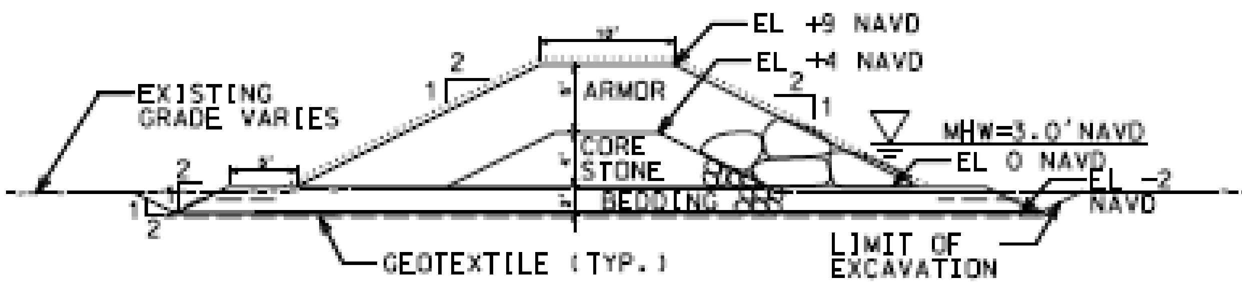


Figure A-66 Typical Beachfill Section Station 0+00 to 9+00



TYPICAL SECTION

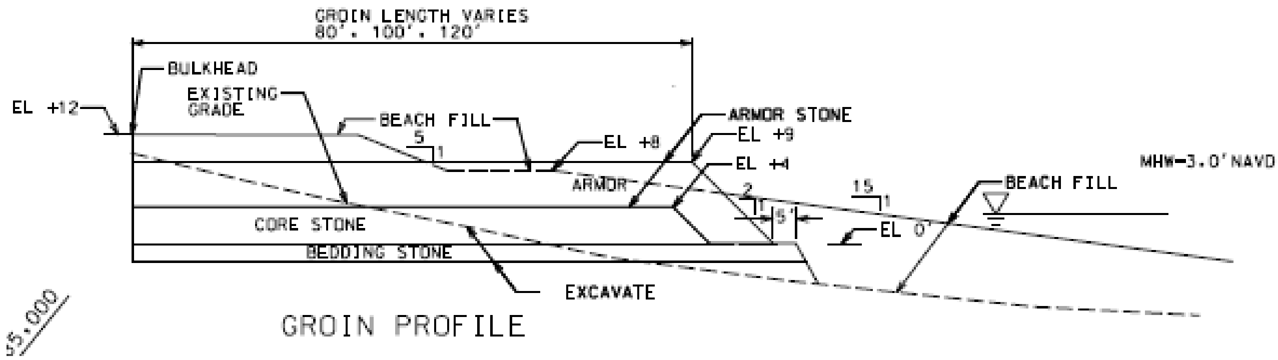


Figure A-67 Typical groin section and profile

<u>Station 11+ 00 to 61+00 – 100 ft Width Composite Dune and Beachfill (no advance fill)</u>
A composite 100 ft wide dune and beachfill will be provided in this stretch of shoreline (Figure A-68). The dune feature is a +15 ft crest width with 1 vertical on 3 horizontal side slopes on both landward and seaward sides. The berm is 50 ft wide with 1 vertical on 15 horizontal foreshore slopes. The proposed dune and beachfill will provide a total of 100 ft wide beach and a higher dune elevation of +15 ft NAVD. Details of the 200 ft beachfill transition from Sta.9+00 to 11+00 (berm fill to dune and berm fill) will be provided during final planning and design.

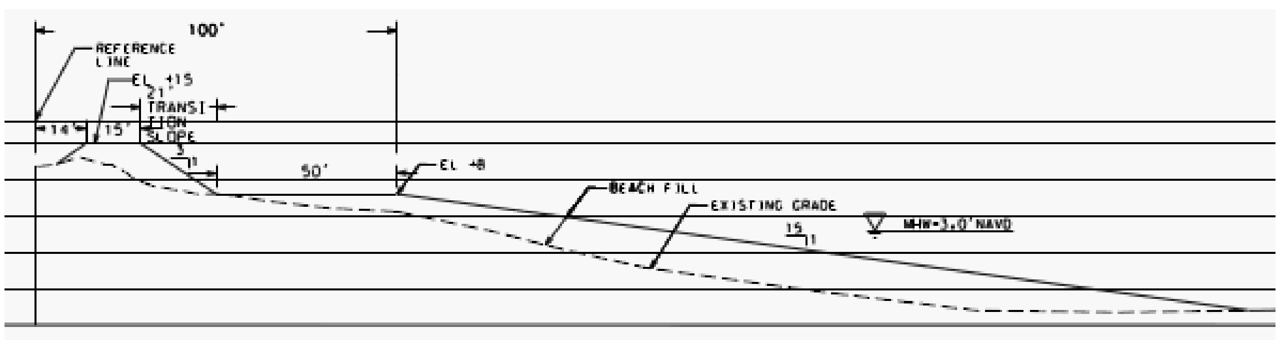


Figure A-68 Typical Dune and Beachfill Sta.11+00 to Sta.61+00

<u>Station 63+ 00 to 124+00 – 100 ft Width Beachfill Including 50 ft Advanced Nourishment</u>
A total of 100 ft berm and beachfill will be provided along this stretch of shoreline fronting the existing timber bulkhead. The proposed beachfill will include a 50 ft wide berm at +8 ft NAVD and 1 vertical on 15 foreshore slopes, plus an additional 50 ft berm width equivalent to 5 years of advance nourishment volume, including contingency due to outdated offshore bathymetry (Figure A-69).

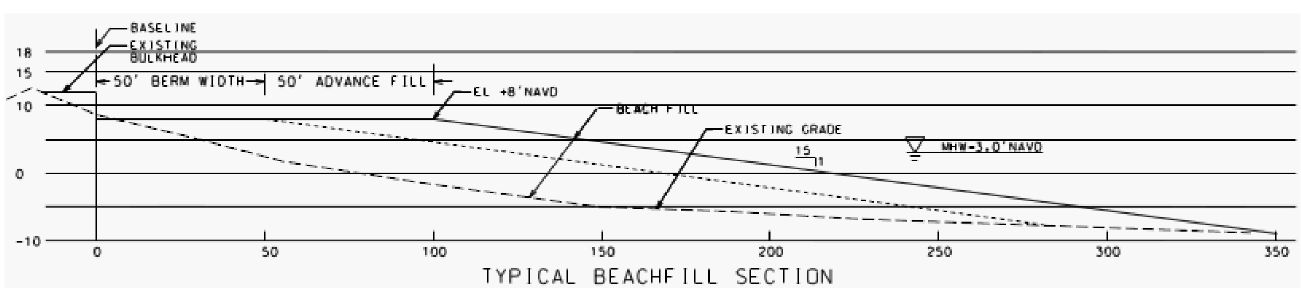


Figure A-69 Typical Beachfill against Existing Bulkhead

Tapering and Transition Beachfills

A 500 ft beachfill taper will be provided from Station -5+00 to -10+00. This transition will provide a continuous beachfill shoreline and stability west of the proposed taper groin at station -5+00. Two 200 ft beachfill transitions will be provided at station ranges from 9+00 to 11+00 and from 61+00 to 63+00 to maintain a continuous shoreline.

Periodic Nourishments

Periodic sand nourishments will be provided within the 50 year project life. The estimated quantities and frequency of nourishment requirements and assumptions are summarized as follows:

- Sediment bypassing from power plant is accounted for and included in the scheduled nourishment;
- The scheduled renourishment quantity is based on sediment budget analysis;
- Volume reduction quantities are based on GENESIS model results.

	Scheduled Renourishment			Total Volume in 50 years (cy)
Alternative	Renourish per Cycle (years)	Quantity/Cycle (cy)	Annual (cy/yr)	
4-West Groin Field Only	5	80,000	16,000	800,000

Borrow Source

The initial and advance fill source will be from Borrow Area A as shown in Figure A-70. The average distance to beachfill sites is approximately 2.5 miles including offshore and onshore pipelines. The average water depths at the borrow site range from 15 to 20 ft and the average dredging depth is 3 to 5 feet from bottom. Upland borrow sources will be used for future nourishment.

Mitigation to Preclude Potential Downdrift Erosion at Groins

Mitigation features included in the plan to address the potential for downdrift erosion at the recommended groins includes groin tapering, placement of advance fill, and beach monitoring.

Preliminary Layout Plan and Typical Sections

The general beachfill layout of the draft selected plan is shown in Figure A-71. Detailed layout of the west rock groin field is shown in Figure A-72.

Beachfill, Groin fields, and Borrow Area Footprint

Figure A-73 presents the existing and proposed beachfill Mean High Water (MHW) lines and toe of slope limit plotted on the most recent GIS aerial map. As shown in the figure, the MHW beachfill footprint width range from 350 to 500 feet. The proposed groin field lengths range from 112 to 182 feet at the offshore toe of the rock slope. As a result, the footprint of the groins are within the beachfill. The approximate footprints of beachfill area, groins, and borrow area are listed in Table T-1 and are summarized as follows:

- Beachfill: 74.04 acre;
- Groin Field: 0.58 acre (within beachfill footprint);
- Borrow Area: 55.0 acre out of 370 acre in Borrow Area A;

Table T-1 Estimated Footprint of the Proposed Plan

Project: Asharoken, New York							
TSP Estimate of Approximate Footprint and Quantities							
Description	**Dimension**	**Estimated Footprint**		**Dredged**	**Placed**	**Avg Dredge**	**Remarks**
		Sq. ft	Acre	Cubic Yard	Cubic Yard	Depth (ft)	
Beachfill Area	MHW to Toe of Fill	3,225,000	74.04		600,000		Include 500' Taper
	250'Wx(12,400+500)L						Use Avg. Width
West Groin Field							
Groin A-	152' L x 64' W	9,728	0.22				On Existing Damaged Groin #1
Groin B-	132' L x 64' W	8,448	0.19				On Existing Damaged Groin #2
Groin C-	112' L x 64' W	7,168	0.16				Layout to be Determined at PED
	Total:	**25,344**	**0.58**				
Borrow Area A	App. 1,200'x2,000'	2,400,000	55.00	800,000	600,000	10	1.4 mcy available
	Dredging Area						Approx. 2,000'x8,000'
							370 Acre in Area A
Note:							
1. Proposed TSP is Preliminary and Quantities are Approximate							
2. Assume 20% sand loss during construction							
3. PED stands for Pre-construction Engineering and Design							

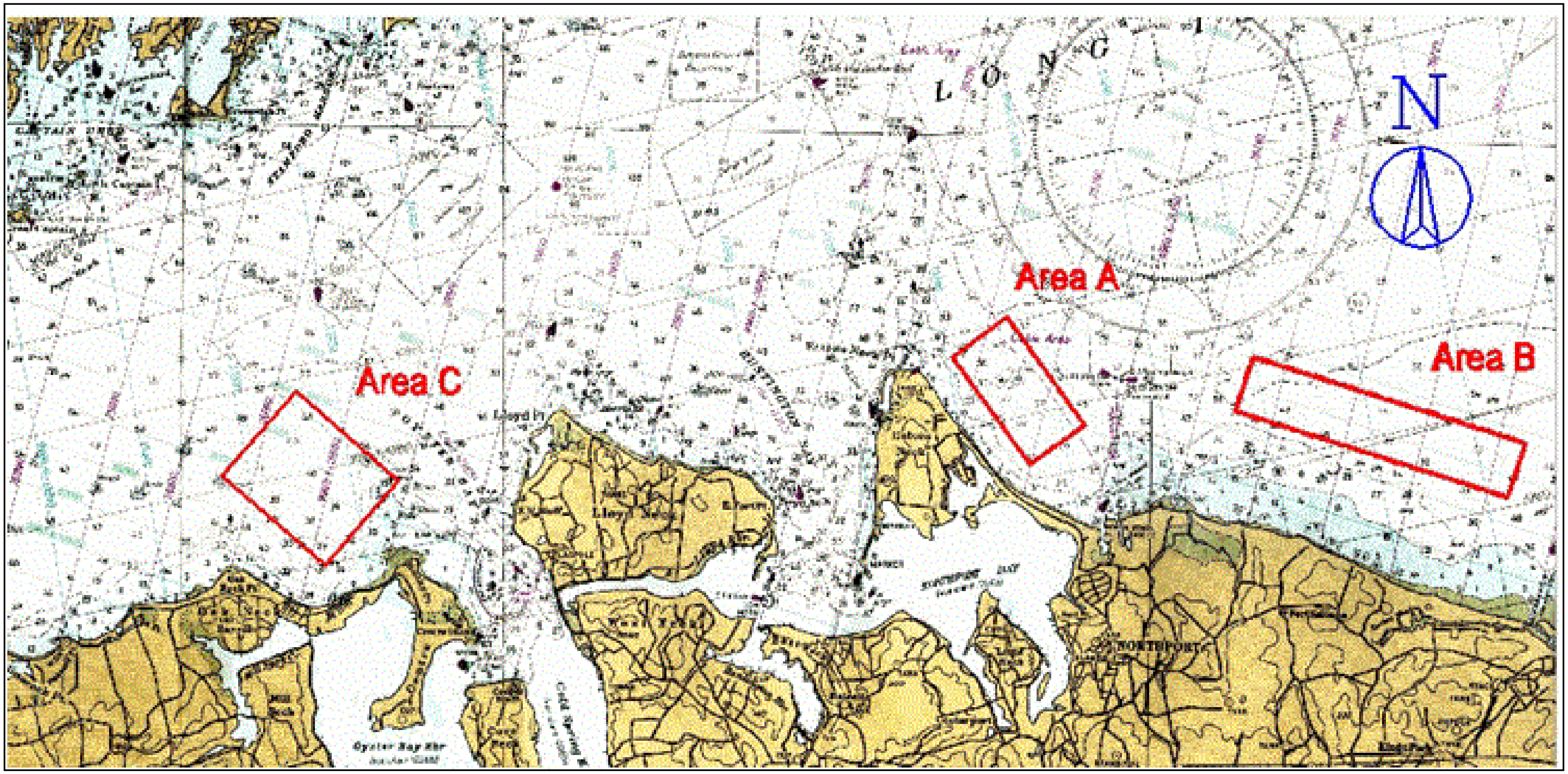


Figure A-70 Borrow Source (Area A) Location Map

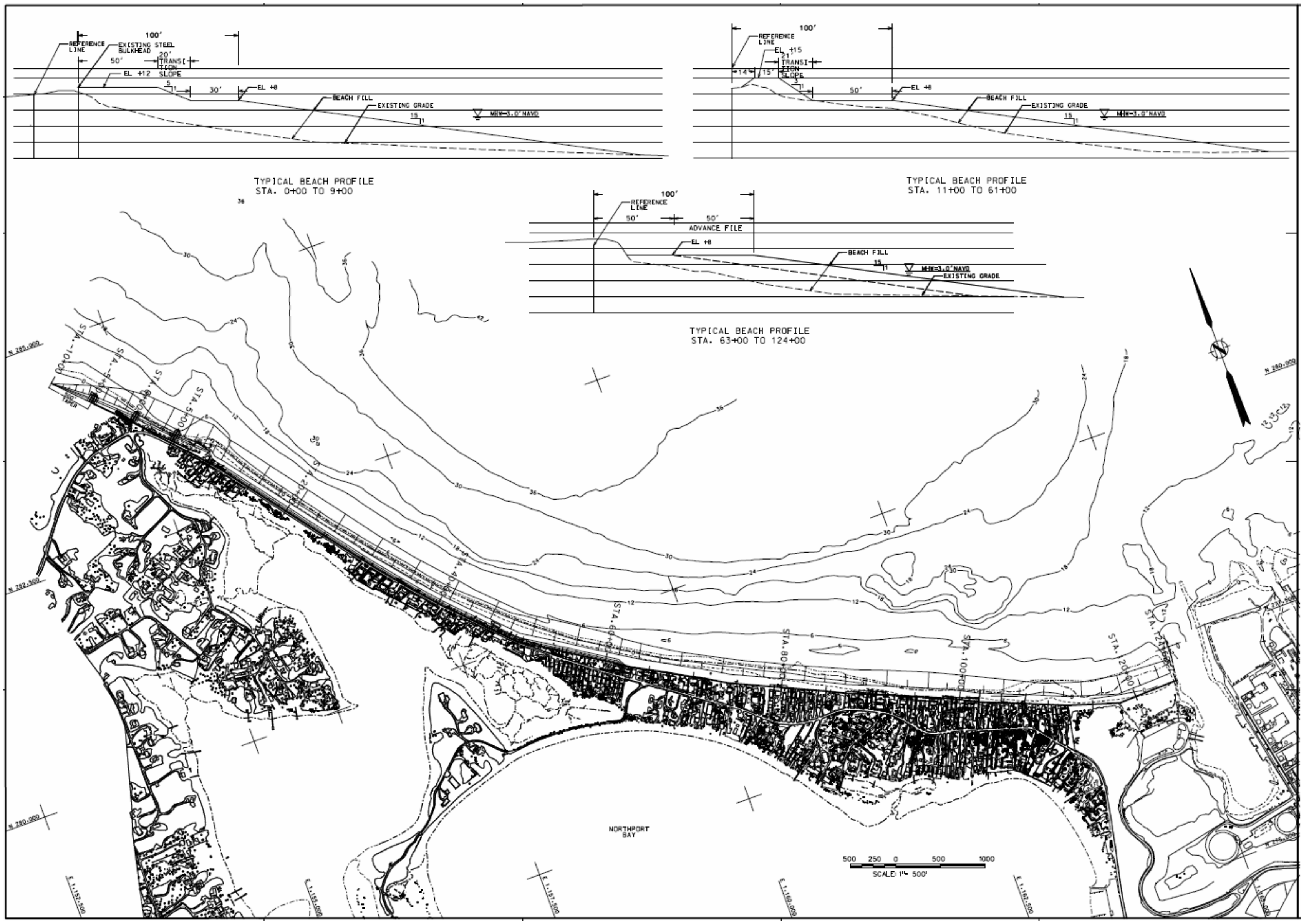


Figure A-71 TSP General Layout Plan

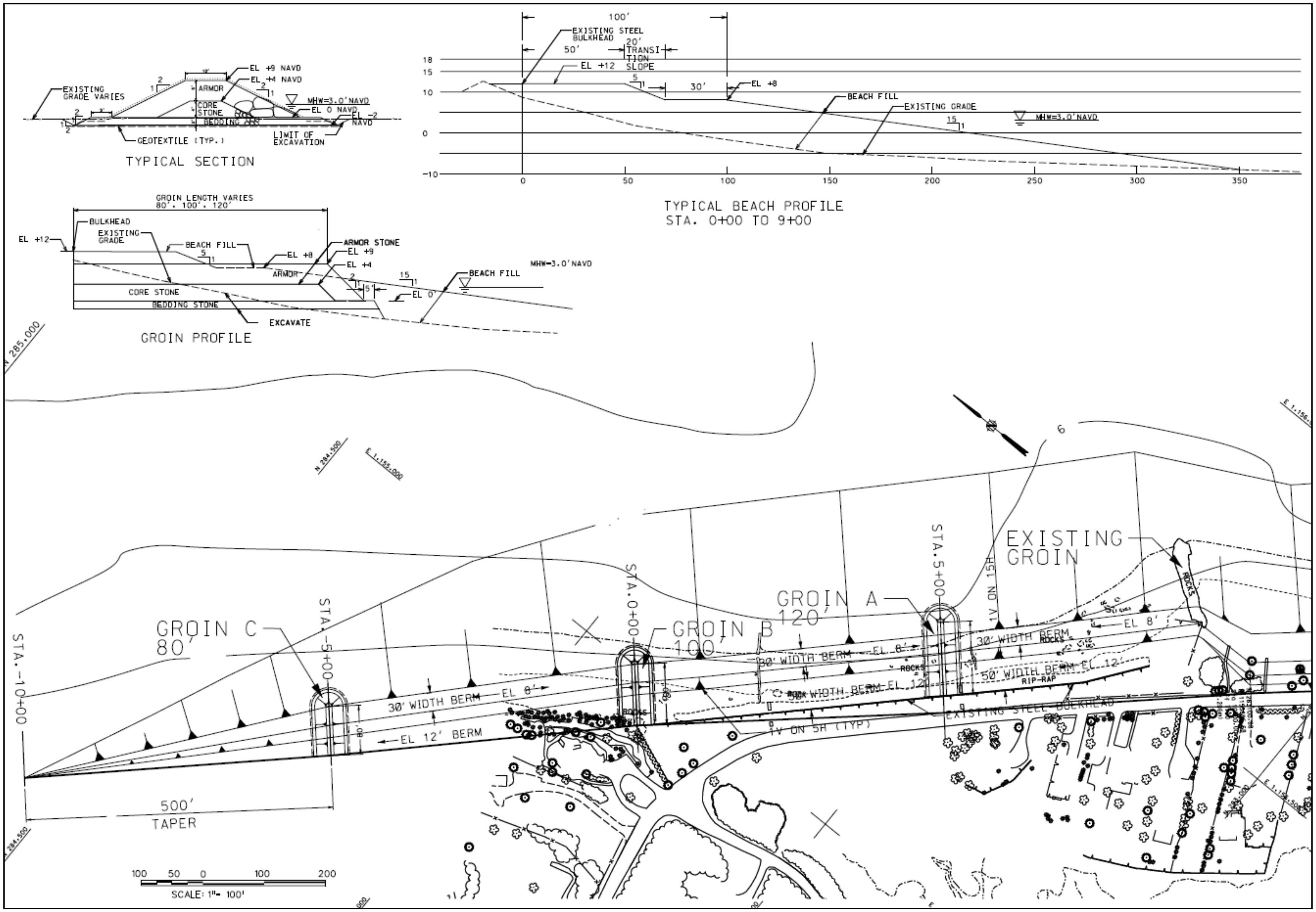


Figure A-72 West Groin Field Layout Plan

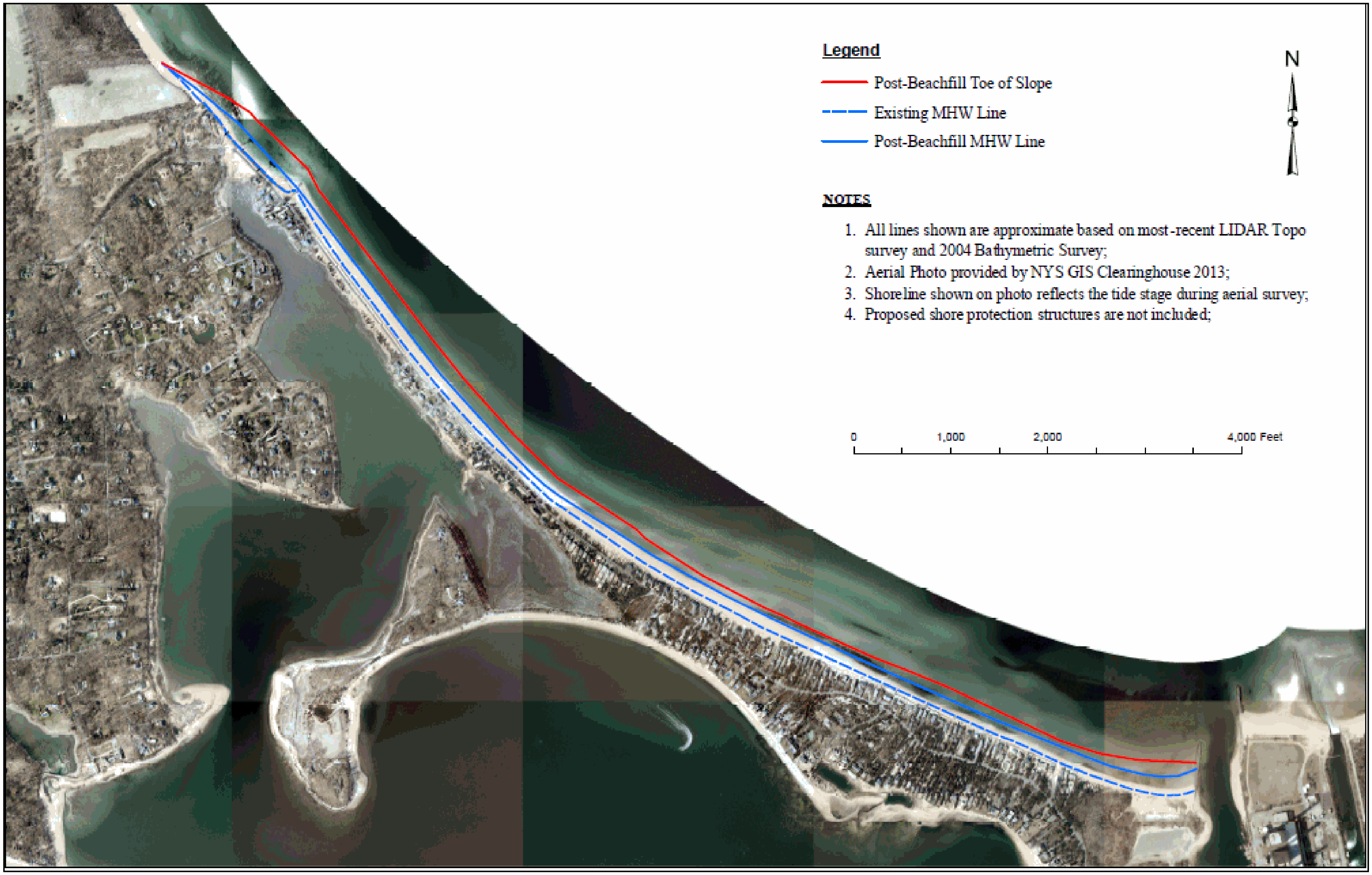


Figure A-73 Beachfill MHW and Toe of Slope Limits, All Proposed Groins are within the Toe of Slope

REFERENCES

1. Wind, Storm Surge and Wave Hindcast Modeling for Asharoken/Bayville, Long Island, New York, by OCTI, January 13, 2002;

2. Sediment Transport Analysis, Village of Asharoken, New York, Combined Erosion Control and Storm Damage Protection Feasibility Study by OCTI, August, 2004;

3. Borrow Area Identification for the Feasibility Study of the North Shore of Long Island Beach Erosion Control and Storm Damage Protection, Village of Asharoken and Bayville, New York; Prepared by Alpine Ocean Seismic Survey, Inc., January 2003;

4. US Army Corps of Engineers, EC 1165-2-212, *Sea Level Change Considerations for Civil Works Programs*, 1 Oct 2011;

5. US Army Corps of Engineers, ER 1100-2-8162, *Incorporating Sea Level Change in Civil Works Programs*, 31 December 2013;

6. Intergovernmental Panel for Climate Change (IPCC) Report, *Contribution of Working Groups I, II and III to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change Core Writing Team*, Pachauri, R.K. and Reisinger, A. (Eds.) IPCC, Geneva, Switzerland. pp 104;

7. NRC 1987, National Research Council (1987) *Responding to Changes in Sea Level: Engineering, Implications*. Washington, DC: National Academy Press.

ATTACHMENT A1

DAMAGE ESTIMATES
BASED ON EDUNE MODELING

DAMAGE ESTIMATES
BASED ON EDUNE MODEL

A1. Economic modeling uses coastal processes inputs to predict what damage may occur to damageable elements in each reach both now and up to 50 years into the future. The Asharoken study area is broken into four economic reaches; 1A with the public seawall fronting a public road, Reach 1B where a dune and beach front the public road; 2A with private bulkheads front residential structures; and 2B where a wider beach is adjacent to the timber jetty. In Reach 1A, wave induced scour and its effect on the public seawall is evaluated, and wave overtopping flowrates are evaluated for the effect on the seawall and the road. In Reach 1B, storm induced erosion and wave overtopping flowrates are evaluated for effects on the road behind the dunes. In Reach 2A wave overtopping flowrates are evaluated for damage to the private bulkheads, and storm-induced erosion was evaluated for undermining of the structures located landward of the bulkhead. No evaluations were performed for Reach 2B due to the limited damageable elements in the reach.

A2. Typical profiles for each reach were developed as follows: Reach 1A used gross profile shape estimates from recent site visits, Reach 1B used 2001 topographic and bathymetric data from station 35+00, and Reach 2A used approximated an average of all the 2001 data from station 62+00 to 110+00. Reach 1A was assumed to represent 2014 conditions; Reach 1B and 2A had the average long-term erosion of 1 ft/yr applied to bring the profiles to 2014 conditions. 2014 profile data was collected onshore however these profiles were too out of equilibrium for use in models based on equilibrium profile theory. Typical profile features are as follows: Reach 1A is shown in Figure 1, Reach 1B is shown in Figure 2, and Reach 2A is shown in Figure 3.

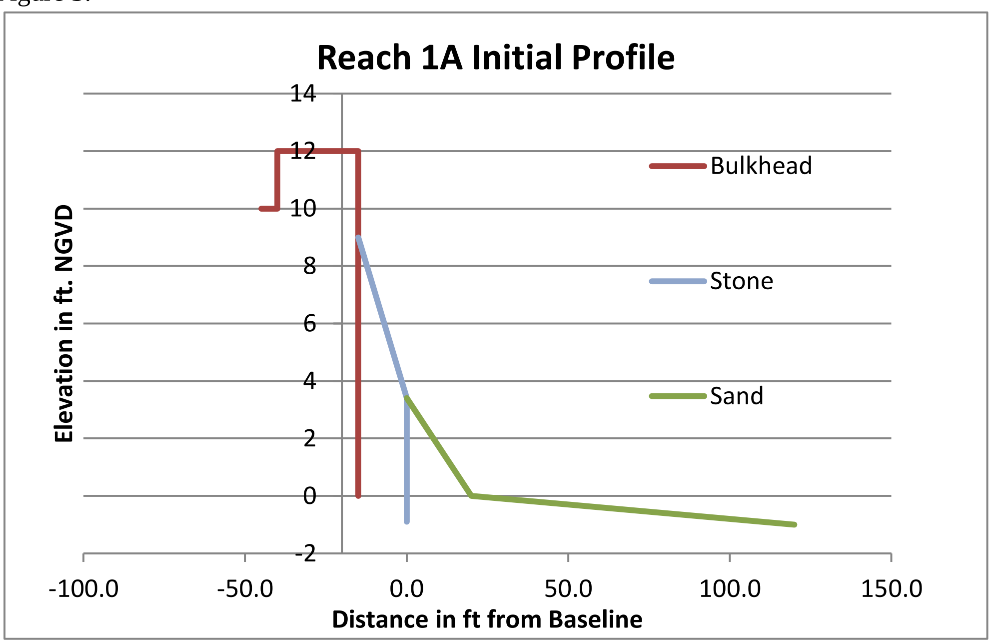
i. 

Figure 1: Reach 1A Initial Profile

ii.

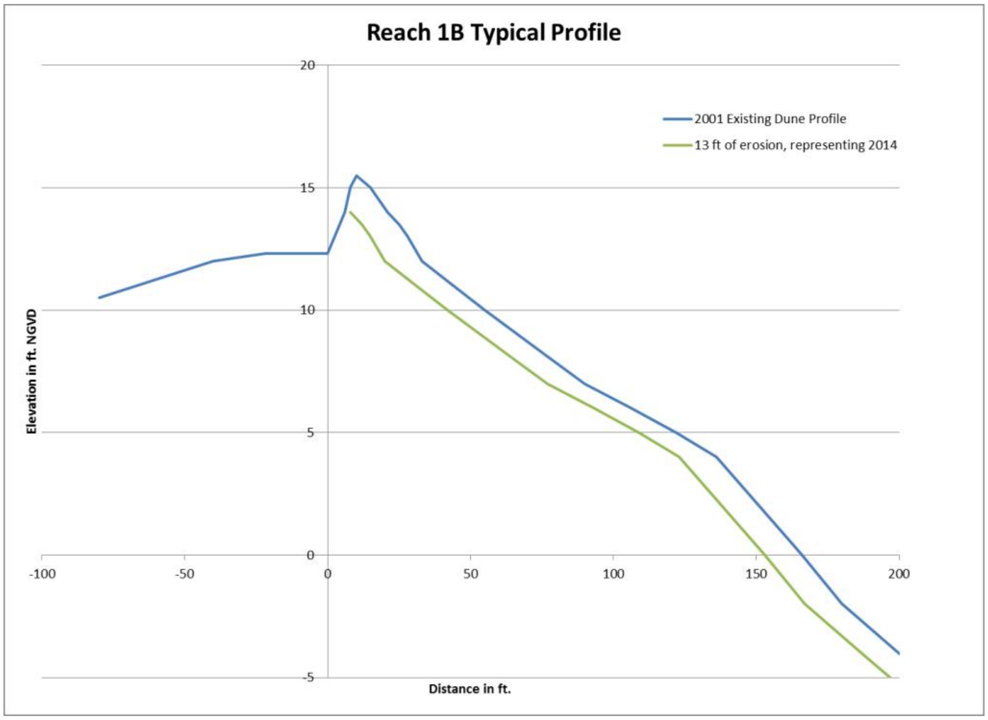


Figure 2: Reach 1B Typical Profile

iii.

b.

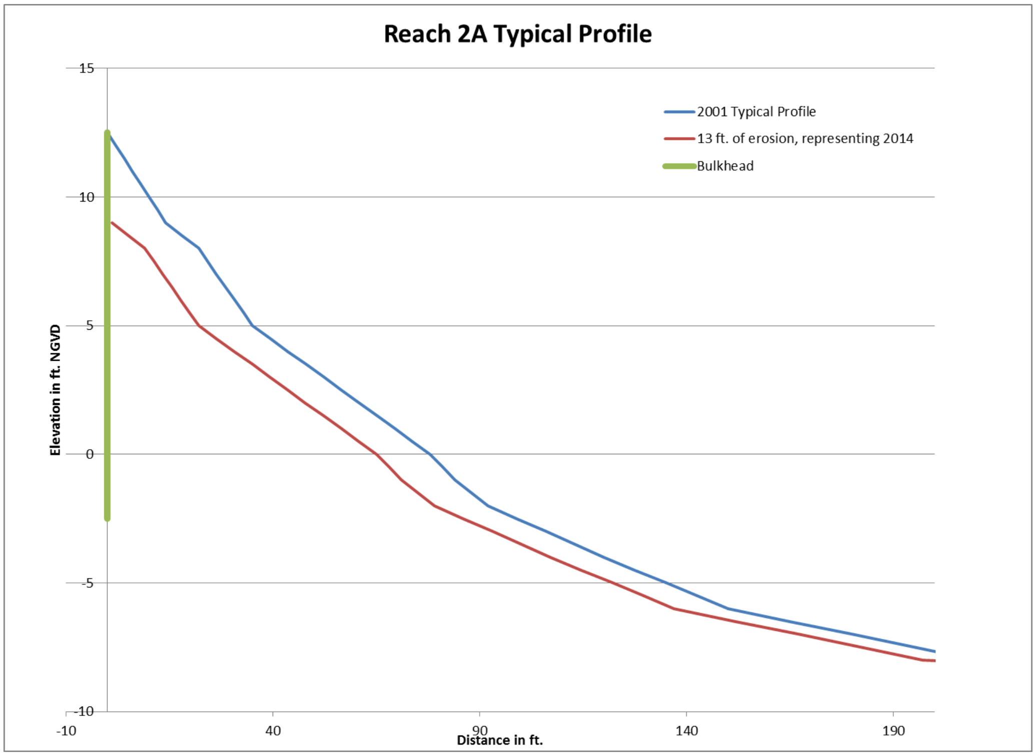


Figure 3: Reach 2A Typical Profile

Future conditions were estimated via the evolving condition of the typical profiles from long-term erosion. This was assumed to act upon the profiles in a parallel translation manner, i.e., the profile shape was assumed to stay constant, but migrate landward. The following erosion rates were assumed: 0.5 ft/yr for Reach 1A; and 0.65 ft/year for Reaches 1B and 2A. The evolution for Reach 1A is shown in Figure 4, 1B is shown in Figure 5, and 2A is shown in Figure 6. The relevant features that change due to long-term erosion are distance between the baseline and the 0 ft. NGVD contour for all the reaches: specifically the elevation of the toe in Reach 1A, maximum dune elevation in Reach 1B, and toe elevation in Reach 2A. These are shown in

A3. Table 1.

A4. Future conditions for all the reaches also included the inclusion of different sea level change heights; 0.5, 1.0, 1.5, and 2.0 feet. These heights were superimposed on the storm water surface elevations and are shown in Table 2.

i.

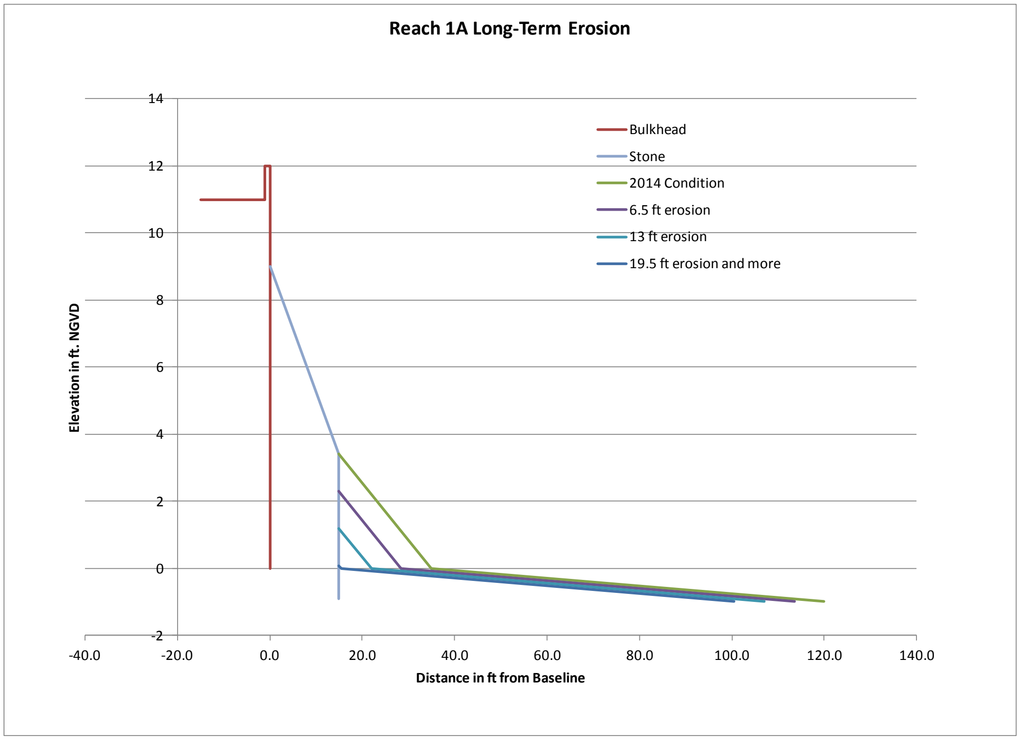


Figure 4: Reach 1A Future Conditions

ii.

iii.

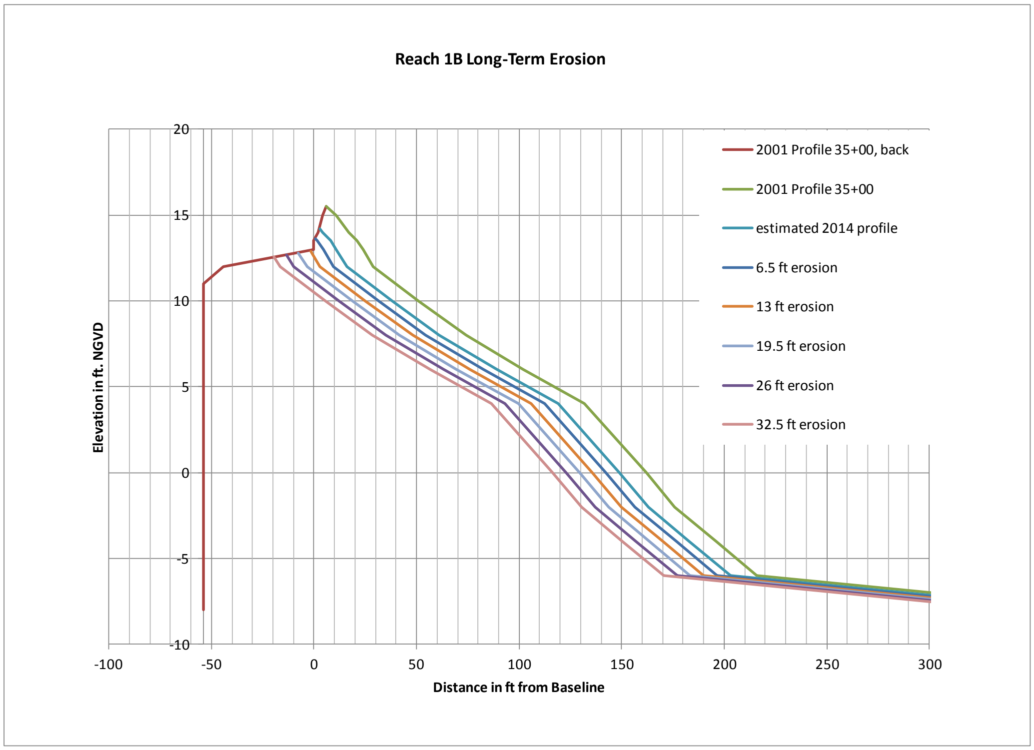


Figure 5: Reach 1B Future Conditions

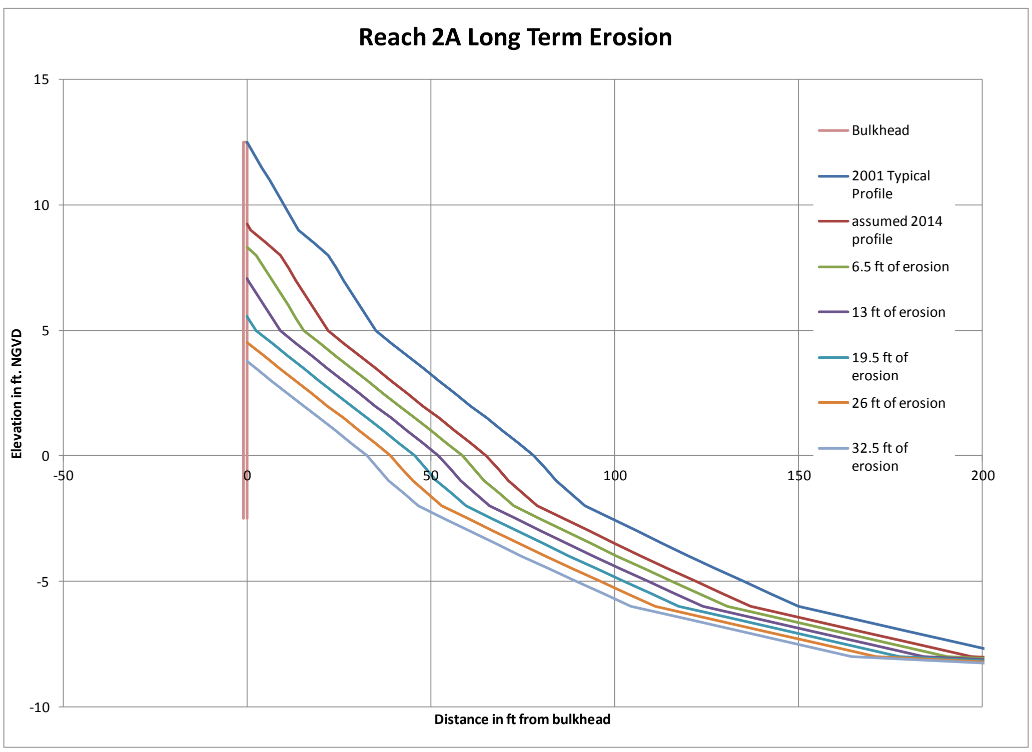


Figure 6: Reach 2A Long Term Erosion

iv.

Table 1: Relevent Profile Features

Relevent Profile Features Changed by Long-Term Erosion						
	Reach 1A		Reach 1B		Reach 2A	
Time	Toe Elevation in ft. NGVD	Distance between the Baseline and the 0 ft. NGVD Contour in ft.	Maximum Dune Elevation in ft. NGVD	Distance between the Baseline and the 8 ft. NGVD Contour in ft.	Toe Elevation in ft. NGVD	Distance between the Baseline and the 0 ft. NGVD Contour in ft.
2014 Condition	3.4	35.0	14.2	61.0	9.3	65.0
6.5 ft of erosion	2.3	28.5	13.6	54.5	8.3	58.5
13 ft of erosion	1.2	22	12.9	48.0	7.1	52.0
19.5 ft of erosion	0.1	15.5	12.8	41.5	5.6	45.5
26 ft of erosion	0.1	15.5	12.7	35.0	4.5	39.0
32.5 ft of erosion	0.1	15.5	12.6	28.5	3.8	32.5

b.

Table 2: Coastal Forcing Factors

Still Water Surface Elevation (ft NGVD) and Root Mean Square Breaking Wave Height (ft) for different Sea Level Rise Heights								
Recurrence Intervals in years	Root Mean Square Breaking Wave Height in ft	Still Water Surface Elevation with Sea Level Rise of 0 ft.	Still Water Surface Elevation with Sea Level Rise of 0.5 ft.	Still Water Surface Elevation with Sea Level Rise of 1.0 ft.	Still Water Surface Elevation with Sea Level Rise of 1.5 ft.	Still Water Surface Elevation with Sea Level Rise of 2.0 ft.	Still Water Surface Elevation with Sea Level Rise of 2.5 ft.	Still Water Surface Elevation with Sea Level Rise of 3.0 ft.
2	7.1	5.9	6.4	6.9	7.4	7.9	8.4	8.9
5	8.9	7.5	8	8.5	9	9.5	10	10.5
10	11	8.8	9.3	9.8	10.3	10.8	11.3	11.8
25	13.2	10.1	10.6	11.1	11.6	12.1	12.6	13.1
50	15	11.1	11.6	12.1	12.6	13.1	13.6	14.1
100	16.5	12.2	12.7	13.2	13.7	14.2	14.7	15.2
200	18.2	13.4	13.9	14.4	14.9	15.4	15.9	16.4
500	20.4	14.9	15.4	15.9	16.4	16.9	17.4	17.9

c.

A5. The Reach 1A without-project evaluations included wave scour as a damage mechanism for the public seawall, and wave overtopping flowrates as a damage mechanism for the seawall and the public road. These evaluations are described below.

A6. For Reach 1A, a simplified and commonly-accepted method of estimating scour; assuming the scour depth is equivalent to the wave height at the toe of the structure; was used. The wave height at the toe of the structure was computed using two linear wave theory methods; depth limited wave heights, and deep water wave transformation. The smaller of these was designated the scouring wave height. The resulting scour depths for Reach 1A are shown in Table 3. The assumption is made that the seawall would experience considerable damage when the scour elevation reaches -2 ft. NGVD. This damage occurs with a 25-yr event for the 2014 condition with no sea level rise; with a 10-yr event for the 6.5 ft. of long-term erosion case for up to one ft. of sea level rise; and with the 2-yr event for the 13 ft of long-term erosion case with no sea level rise and for the 6.5 ft of long-term erosion case with sea level rise of 1.5 ft.

Table 3: Reach 1A Scour Elevations

Reach 1A Scour Elevations in ft. NGVD							
Return Period	Sea Level	2014	6.5 ft erosion	13 ft erosion	19.5 ft erosion	26 ft erosion	32.5 ft erosion
2	0	0.9	-1.2	-3.1	-5.1	-5.1	-5.1
5	0	-0.6	-2.5	-4.5	-6.0	-6.0	-6.0
10	0	-1.8	-3.7	-5.7	-6.0	-6.0	-6.0
25	0	-2.9	-4.9	-6.0	-6.0	-6.0	-6.0
50	0	-3.8	-5.9	-6.0	-6.0	-6.0	-6.0
100	0	-4.8	-6.0	-6.0	-6.0	-6.0	-6.0
200	0	-5.9	-6.0	-6.0	-6.0	-6.0	-6.0
500	0	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
2	0.5	0.5	-1.6	-3.5	-5.6	-5.6	-5.6
5	0.5	-1.0	-2.9	-5.0	-6.0	-6.0	-6.0
10	0.5	-2.1	-4.1	-6.0	-6.0	-6.0	-6.0
25	0.5	-3.3	-5.3	-6.0	-6.0	-6.0	-6.0
50	0.5	-4.2	-6.0	-6.0	-6.0	-6.0	-6.0
100	0.5	-5.2	-6.0	-6.0	-6.0	-6.0	-6.0
200	0.5	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
500	0.5	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
2	1	0.0	-1.9	-3.9	-6.0	-6.0	-6.0
5	1	-1.3	-3.3	-5.4	-6.0	-6.0	-6.0
10	1	-2.5	-4.5	-6.0	-6.0	-6.0	-6.0
25	1	-3.7	-5.8	-6.0	-6.0	-6.0	-6.0
50	1	-4.6	-6.0	-6.0	-6.0	-6.0	-6.0
100	1	-5.7	-6.0	-6.0	-6.0	-6.0	-6.0
200	1	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
500	1	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
2	1.5	3.0	-2.3	-4.4	-6.0	-6.0	-6.0
5	1.5	-1.7	-3.8	-5.8	-6.0	-6.0	-6.0
10	1.5	-2.9	-5.0	-6.0	-6.0	-6.0	-6.0
25	1.5	-4.1	-6.0	-6.0	-6.0	-6.0	-6.0
50	1.5	-5.1	-6.0	-6.0	-6.0	-6.0	-6.0
100	1.5	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
200	1.5	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
500	1.5	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
2	2	-0.7	-2.8	-4.8	-6.0	-6.0	-6.0
5	2	-2.2	-4.2	-6.0	-6.0	-6.0	-6.0
10	2	-3.3	-5.4	-6.0	-6.0	-6.0	-6.0
25	2	-4.6	-6.0	-6.0	-6.0	-6.0	-6.0
50	2	-5.5	-6.0	-6.0	-6.0	-6.0	-6.0
100	2	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
200	2	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
500	2	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
2	2.5	-1.1	-3.2	-5.2	-6.0	-6.0	-6.0
5	2.5	-2.6	-4.6	-6.0	-6.0	-6.0	-6.0
10	2.5	-3.8	-5.8	-6.0	-6.0	-6.0	-6.0
25	2.5	-5.0	-6.0	-6.0	-6.0	-6.0	-6.0
50	2.5	-5.9	-6.0	-6.0	-6.0	-6.0	-6.0
100	2.5	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
200	2.5	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
500	2.5	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
2	3	-1.6	-3.6	-5.6	-6.0	-6.0	-6.0
5	3	-3.0	-5.0	-6.0	-6.0	-6.0	-6.0
10	3	-4.2	-6.0	-6.0	-6.0	-6.0	-6.0
25	3	-5.4	-6.0	-6.0	-6.0	-6.0	-6.0
50	3	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
100	3	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
200	3	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0
500	3	-6.0	-6.0	-6.0	-6.0	-6.0	-6.0

A7. For Reach 1A, overtopping flowrates were estimated using Ward and Ahrens (1992) model, as recommended in EM 1110-2-1614. The same incident wave heights used for the Reach 1A scour analysis were used in the overtopping model. When the water surface elevation exceeds the seawall crest elevation (i.e., the structure is submerged), failure of the structure is assumed. Reach 1A's vertical wall is most similar to Ward and Ahrens Group 1 seawalls. The basic equation for overtopping used was:

$$Q = Qo \exp(C_1 F') \tag{1}$$

where Q_0 and C_1 are constants (71.952 and -13.586, respectively) and the dimensionless freeboard, F1 is defined as follows:

$$F' = F/\sqrt[3]{(H_{mo}^2 \, L_{p)}} \tag{2}$$

where F is freeboard (vertical distance from the still-water level to top of structure), H_{mo} is the zeroth moment wave height measured near the toe and L_p is the local wavelength at the toe of the structure.

A8. Reach 1A overtopping flowrate results are shown in Table 4. A damage threshold of 0.5 cfs/ft is assumed, meaning when the flowrate exceeds this threshold, damage to the structure and the road are initiated. The results predict wall damage for a 10-year event in the 2014 condition with no further long-term erosion and with up to one ft. of sea level rise. Damage from a 5-yr event is predicted to occur for the 19.5 ft of long-term erosion case with no sea level rise, the 13 ft of long-term erosion case with 0.5 ft. of sea level rise, the 6.5 ft. of long-term erosion case with one ft of sea level rise. Damage to the seawall from a 2-yr event is predicted for the 19.5 ft of long-term erosion case with 1.5 ft. of sea level rise, the 13 ft. of long-term erosion case with 2 ft. of sea level rise, and for the 6.5 ft. of long-term erosion case with 2.5 ft. of sea level rise.

Table 4

Reach 1A Wave Overtopping Flowrates in cfs/ft.							
Return Period in years	Sea Level Rise Height in ft..	2014	6.5 ft erosion	13 ft erosion	19.5 ft erosion	26 ft erosion	32.5 ft erosion
2	0	0.0	0.0	0.0	0.0	0.0	0.0
5	0	0.0	0.1	0.4	0.8	0.8	0.8
10	0	0.6	1.3	2.5	4.2	4.2	4.2
25	0	4.8	7.1	10.4	14.5	14.5	14.5
50	0	14.9	20.0	24.9	31.6	31.6	31.6
100	0	submerged	submerged	submerged	submerged	submerged	submerged
200	0	submerged	submerged	submerged	submerged	submerged	submerged
500	0	submerged	submerged	submerged	submerged	submerged	submerged
2	0.5	0.0	0.0	0.0	0.1	0.1	0.1
5	0.5	0.1	0.3	0.8	1.6	1.6	1.6
10	0.5	1.5	2.6	4.4	6.9	6.9	6.9
25	0.5	8.5	12.4	16.2	21.4	21.4	21.4
50	0.5	23.6	30.0	35.9	43.9	43.9	43.9
100	0.5	submerged	submerged	submerged	submerged	submerged	submerged
200	0.5	submerged	submerged	submerged	submerged	submerged	submerged
500	0.5	submerged	submerged	submerged	submerged	submerged	submerged
2	1	0.0	0.0	0.1	0.3	0.3	0.3
5	1	0.3	0.8	1.6	2.8	2.8	2.8
10	1	3.0	5.1	7.4	10.8	10.8	10.8
25	1	14.4	19.5	25.4	30.8	30.8	30.8
50	1	submerged	submerged	submerged	submerged	submerged	submerged
100	1	submerged	submerged	submerged	submerged	submerged	submerged
200	1	submerged	submerged	submerged	submerged	submerged	submerged
500	1	submerged	submerged	submerged	submerged	submerged	submerged
2	1.5	0.0	0.1	0.3	0.6	0.6	0.6
5	1.5	0.8	1.7	3.1	5.0	5.0	5.0
10	1.5	5.7	8.8	12.6	17.0	17.0	17.0
25	1.5	24.1	29.5	36.8	44.5	44.5	44.5
50	1.5	submerged	submerged	submerged	submerged	submerged	submerged
100	1.5	submerged	submerged	submerged	submerged	submerged	submerged
200	1.5	submerged	submerged	submerged	submerged	submerged	submerged
500	1.5	submerged	submerged	submerged	submerged	submerged	submerged
2	2	0.1	0.3	0.6	1.3	1.3	1.3
5	2	1.8	3.3	5.5	8.1	8.1	8.1
10	2	10.6	14.3	19.3	24.9	24.9	24.9
25	2	submerged	submerged	submerged	submerged	submerged	submerged
50	2	submerged	submerged	submerged	submerged	submerged	submerged
100	2	submerged	submerged	submerged	submerged	submerged	submerged
200	2	submerged	submerged	submerged	submerged	submerged	submerged
500	2	submerged	submerged	submerged	submerged	submerged	submerged
2	2.5	0.2	0.6	1.4	2.5	2.5	2.5
5	2.5	3.9	6.4	9.1	12.7	12.7	12.7
10	2.5	17.6	23.4	28.8	35.5	35.5	35.5
25	2.5	submerged	submerged	submerged	submerged	submerged	submerged
50	2.5	submerged	submerged	submerged	submerged	submerged	submerged
100	2.5	submerged	submerged	submerged	submerged	submerged	submerged
200	2.5	submerged	submerged	submerged	submerged	submerged	submerged
500	2.5	submerged	submerged	submerged	submerged	submerged	submerged
2	3	0.6	1.4	2.6	4.3	4.3	4.3
5	3	7.3	10.8	14.9	19.5	19.5	19.5
10	3	28.0	35.1	42.7	49.6	49.6	49.6
25	3	submerged	submerged	submerged	submerged	submerged	submerged
50	3	submerged	submerged	submerged	submerged	submerged	submerged
100	3	submerged	submerged	submerged	submerged	submerged	submerged
200	3	submerged	submerged	submerged	submerged	submerged	submerged
500	3	submerged	submerged	submerged	submerged	submerged	submerged

A9. In Reach 1B, storm induced erosion and wave overtopping flowrates are evaluated for effects on the road behind the dunes. These are described in further detail below.

A10. For Reach 1B, the simplified beach erosion model EDUNE, developed in 1989 by Dr. David Kriebel was utilized instead of the more current Corps model, SBEACH. Based on the following facts: calibration data is not available, and equilibrium profile shape and grain size correlation does not hold, it was assumed that EDUNE could predict storm-erosion as well as SBEACH. EDUNE and SBEACH models are based on the equilibrium beach profile theory of Dean (1977, 1984), where a power law was observed to fit beach profiles following:

$$h = A\, x^{2/3} \tag{3}$$

where h is the water depth at a distance x seaward of the still water shoreline, while A is a scaling parameter governing the steepness of the profile. A may be determined either from mean sand grain size based on an available relationship of Moore (1982) or may be determined by a least squares fit of a measured profile.

A11. The following assumptions were made for EDUNE: an A factor of 0.32; dune erosion slopes of 1H:1V; berm erosion slope of 1V:8H; storm durations of 18 hours. The water surface elevations and root mean square breaking wave heights associated with these events were used as direct input, and are shown in Table 2. The EDUNE-predicted erosion for the 2014 condition with sea level rise and the 13 ft. of long-term erosion case with 2 ft. of sea level rise are shown in Figure 7 and Figure 8. Undermining is assumed to begin when erosion reaches 2 ft or more landward of the baseline. This occurs with a 50-yr event in the 2014 condition with no sea level rise; with a 25-yr event for the 19.5 ft. of long-term erosion case with no sea level rise and the 6.5 ft. of long-term erosion case with 0.5 ft. of sea level rise.

A12. The maximum remnant elevation predicted by EDUNE for Reach 1B is used in wave overtopping flowrate analyses to determine road failure by wave action. These are shown in Table 5. The wave overtopping flowrate model used is adapted from Pilarczyk (1990). The form of the equation is:

$$y = e^{-2.5x} \tag{4}$$

where

$$x = F_c \cot(a) \Big/ \sqrt{H_m\, L_0} \tag{5}$$

and

$$y = q\, T_m \sqrt{\cot(a)} \Big/ 0.1\, H_m\, L_0 \tag{6}$$

Where
F_c = height of structure crest above design water level
H_m = mean wave height; value exceeded by 50% of the wave heights
T_m = mean wave period
L_o = deep water wave length
a = slope angle with respect to horizontal
q = average overtopping discharge per unit width

The above formulas were developed for relatively steep coastal structures. In USACE (1997), Moffat & Nichol Engineers derived new empirical coefficients based on Delft overtopping data on eroded dune profiles. The revised (and utilized for Reach 1B) equations are:

$$y = e^{-2.024x} \tag{7}$$

where

$$x = \frac{F_c \cot(a)}{\sqrt{H_m L_0}} \tag{8}$$

and

$$y = \frac{q\, T_m \sqrt{\cot(a)}}{0.0584\, H_m L_0} \tag{9}$$

A13. The resulting wave overtopping flowrates for Reach 1B are shown in Table 6. Overtopping rates of 0.2 cfs/ft are assumed to cause damage to the pavement from direct impact and from removing the supporting sand underneath. Road undermining damage is predicted for a 25-yr event in the 2014 condition with no sea level rise; a 25-yr event with the 13 ft. of long-term erosion case with 0.5 ft. of sea level rise; and a 10-yr event with the 32.5 ft. of long-term erosion case with 1.5 ft. of sea level rise.

A14. In Reach 2A wave overtopping flowrates are evaluated for damage to the private bulkheads, and storm-induced erosion was evaluated for undermining of the structures located landward of the bulkhead. Equations (1) and (2) were used for overtopping modeling Reach 2B. These are discussed in further detail below.

A15. For Reach 2A, the simplified beach erosion model EDUNE was run with the following assumptions were made for EDUNE: an *A* factor of 0.27; dune erosion slopes of 1H:1V; berm erosion slope of 1V:8H; and a storm duration of 9 hours (half the entire storm duration as bulkheads were assumed if they fail at all to fail midway through the storm). The output of maximum profile elevation at the location of the bulkhead (i.e., the baseline) was utilized in overtopping modeling to determine if the bulkhead fails. The 2014 condition with no sea level rise and 19.5 ft. of long-term erosion case with 1 ft. of sea level rise post-storm recession EDUNE results are shown in Figure 9 and Figure 10. The resulting wave overtopping flowrates (using the same model as in Reach 1A) are shown in Table 7. Bulkhead failure is predicted for a 25-yr event with the 2014 condition and no sea level rise; for the 10-yr event with the 6.5 ft. of long-term erosion case and no sea level rise; and for the 5-yr. event with the 6.5 ft. of long-term erosion case with 3 ft. of sea level rise.

A16. The maximum extent of horizontal erosion is used to determine undermining of the structures located behind the bulkhead in Reach 2A, which may occur after bulkhead failure. These eroded distances are compared to the distances between the damageable element and the residential structures, and when the eroded distance is greater than the element distance, undermining of the element is initiated. The EDUNE-predicted eroded distances are shown in Table 8. Each damageable element has a unique distance from the bulkhead, so generalization of the table of eroded distances isn’t helpful; however it can be noted that erosion does not occur the return period of bulkhead failure i.

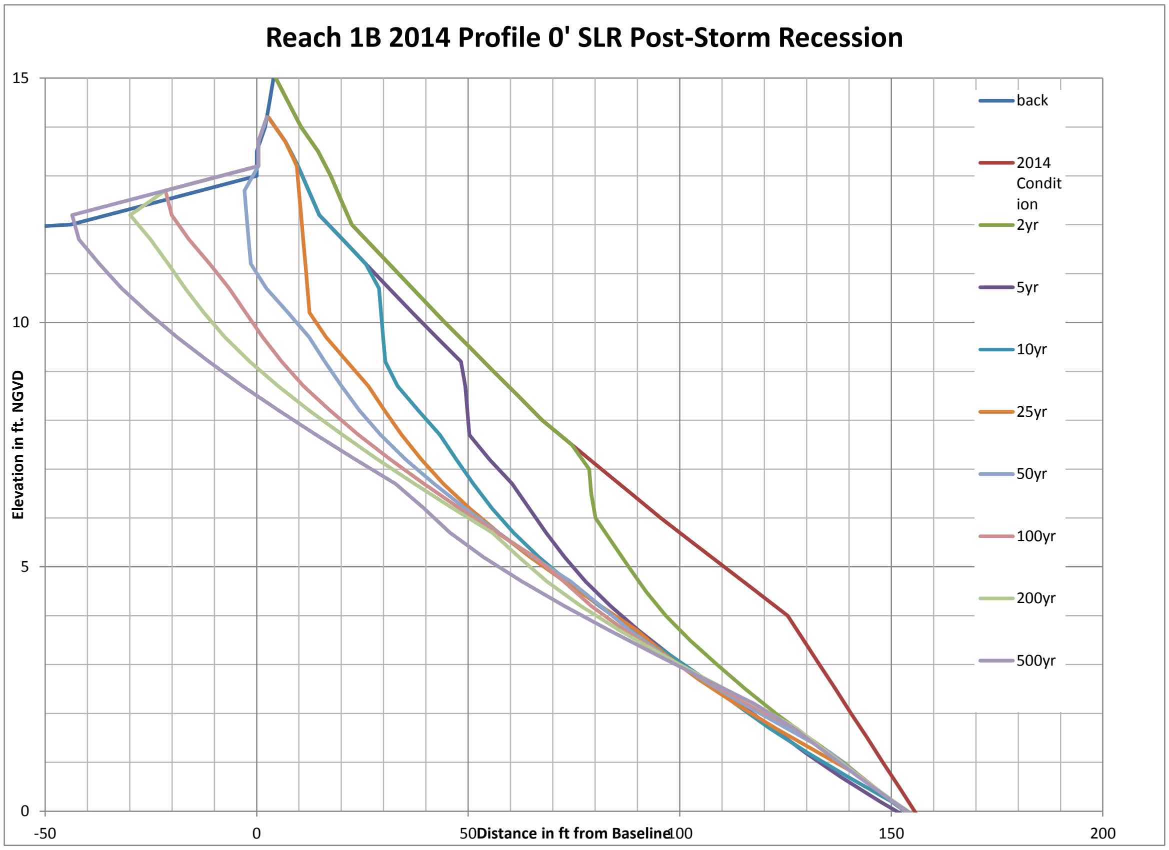

Reach 1B 2014 Profile 0' SLR Post-Storm Recession
Elevation in ft. NGVD
Distance in ft from Baseline
15
10
5
0
-50
0
50
100
150
200
back
2014 Condition
2yr
5yr
10yr
25yr
50yr
100yr
200yr
500yr

Figure 7

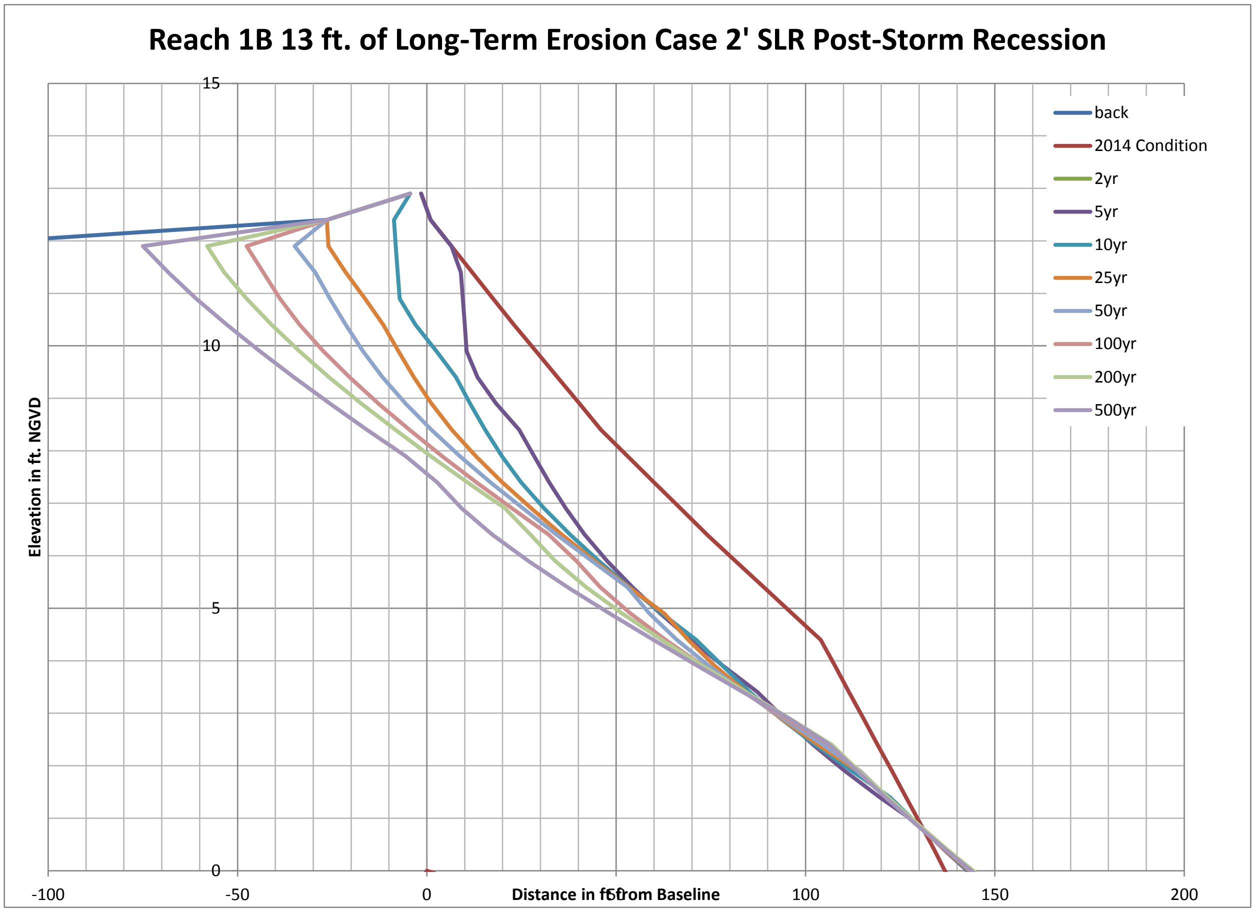

Reach 1B 13 ft. of Long-Term Erosion Case 2' SLR Post-Storm Recession
Elevation in ft. NGVD
Distance in ft. from Baseline
15
10
5
0
-100
-50
0
50
100
150
200
back
2014 Condition
2yr
5yr
10yr
25yr
50yr
100yr
200yr
500yr

Figure 8

Table 5

Reach 1B Distance Between Baseline and Maximum Erosion Extent in ft. (- is landward of baseline)

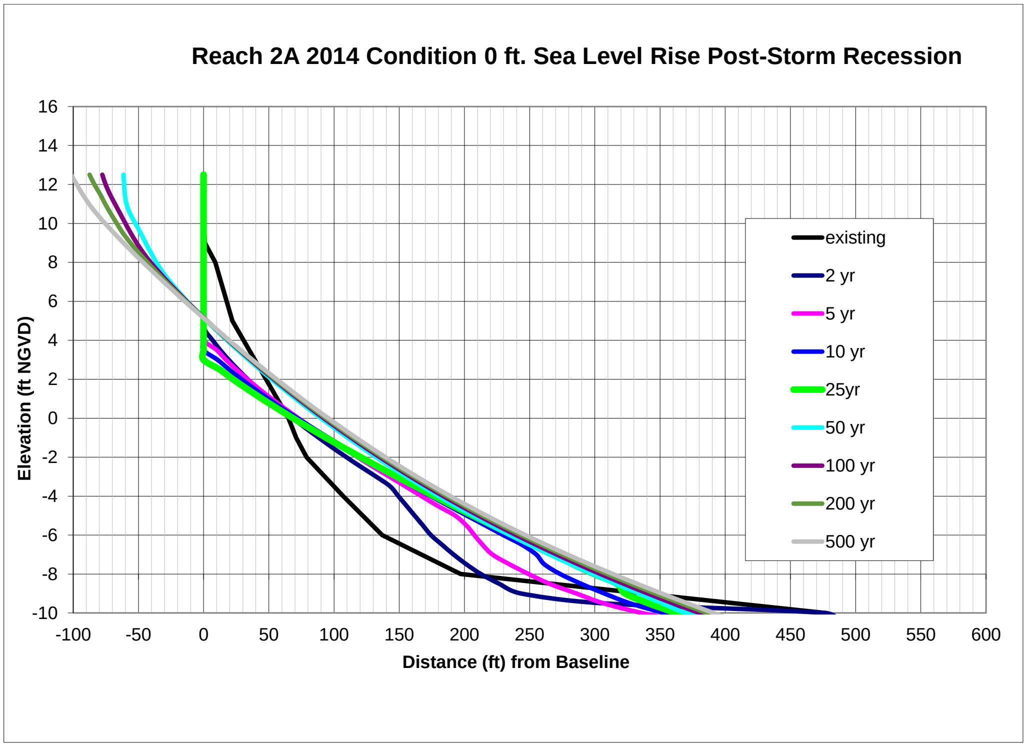
Return Period in years	Sea Level Rise Height in ft.	2014 Condition	6.5 ft erosion	13 ft erosion	19.5 ft erosion	26 ft erosion	32.5 ft erosion
2	0	0.0	0	0	0	0	0
5	0	0.0	0	0	0	0	0
10	0	0.0	0	0	0	0	0
25	0	0.0	0	0	-9	-10	-11
50	0	-2.9	-11	-14	-22	-25	-28
100	0	-21.6	-28	-30	-38	-41	-43
200	0	-29.9	-38	-40	-49	-52	-56
500	0	-43.6	-50	-56	-63	-66	-70
2	0.5	0	0	0	0	0	0
5	0.5	0	0	0	0	0	0
10	0.5	0	0	0	0	0	0
25	0.5	0	-3	-8	-15	-17	-20
50	0.5	-12	-18	-21	-29	-32	-35
100	0.5	-29	-33	-35	-42	-44	-47
200	0.5	-40	-44	-44	-51	-53	-55
500	0.5	-48	-55	-61	-69	-72	-75
2	1	0	0	0	0	0	0
5	1	0	0	0	0	0	0
10	1	0	0	0	0	0	0
25	1	-2	-10	-14	-22	-25	-28
50	1	-17	-24	-27	-34	-37	-39
100	1	-29	-36	-38	-45	-48	-50
200	1	-38	-44	-50	-56	-59	-62
500	1	-52	-60	-66	-73	-77	-80
2	1.5	0	0	0	-8	-9	-10
5	1.5	0	0	0	-8	-9	-10
10	1.5	0	0	-3	-9	-11	-12
25	1.5	-9	-17	-21	-29	-32	-35
50	1.5	-22	-29	-31	-38	-41	-43
100	1.5	-32	-44	-43	-50	-53	-56
200	1.5	-44	-47	-53	-61	-64	-67
500	1.5	-56	-64	-64	-77	-80	-84
2	2	0	0	0	0	0	0
5	2	0	0	0	0	0	0
10	2	0	-6	-9	-16	-18	-21
25	2	-16	-21	-26	-33	-36	-39
50	2	-25	-33	-35	-41	-44	-47
100	2	-37	-44	-48	-55	-58	-61
200	2	-44	-52	-58	-65	-69	-73
500	2	-60	-68	-75	-82	-85	-89
2	2.5	0	0	0	0	0	0
5	2.5	0	0	0	0	0	0
10	2.5	-4	-13	-15	-23	-26	-29
25	2.5	-22	-28	-31	-37	-40	-42
50	2.5	-33	-37	-39	-46	-49	-51
100	2.5	-44	-45	-52	-59	-62	-65
200	2.5	-45	-56	-63	-70	-74	-78
500	2.5	-63	-71	-79	-86	-89	-93
2	3	0.0	0.0	0.0	0.0	0.0	0.0
5	3	0.0	0.0	0.0	0.0	0.0	0.0
10	3	-4.8	-14.7	-17.9	-26.5	-30.1	-33.7
25	3	-25.2	-32.8	-35.6	-41.8	-44.5	-47.3
50	3	-38.0	-41.0	-43.2	-50.5	-52.5	-54.6
100	3	-47.3	-48.0	-55.6	-62.9	-65.5	-68.1
200	3	-47.2	-58.9	-66.3	-73.2	-77.6	-81.9
500	3	-66.1	-75.0	-82.3	-89.3	-93.2	-97.1

b.

Table 6

Reach 1B Wave Overtopping Flowrates in cfs/ft.

Return Period in years	Sea Level Rise Height in ft..	2014	6.5 ft erosion	13 ft erosion	19.5 ft erosion	26 ft erosion	32.5 ft erosion
2	0	0.0	0.0	0.0	0.0	0.0	0.0
5	0	0.0	0.0	0.0	0.0	0.0	0.0
10	0	0.0	0.0	0.1	0.1	0.1	0.1
25	0	0.3	0.6	1.1	1.2	1.3	1.4
50	0	2.8	3.3	3.3	3.3	3.3	3.3
100	0	7.2	7.8	7.8	7.8	7.8	7.8
200	0	submerged	submerged	submerged	submerged	submerged	submerged
500	0	submerged	submerged	submerged	submerged	submerged	submerged
2	0.5	0.0	0.0	0.0	0.0	0.0	0.0
5	0.5	0.0	0.0	0.0	0.0	0.0	0.0
10	0.5	0.0	0.0	0.2	0.2	0.2	0.2
25	0.5	0.9	1.2	1.8	1.8	1.8	1.8
50	0.5	3.9	4.6	5.0	5.3	5.9	6.5
100	0.5	submerged	submerged	submerged	submerged	submerged	submerged
200	0.5	submerged	submerged	submerged	submerged	submerged	submerged
500	0.5	submerged	submerged	submerged	submerged	submerged	submerged
2	1	0.0	0.0	0.0	0.0	0.0	0.0
5	1	0.0	0.0	0.0	0.0	0.0	0.0
10	1	0.0	0.1	0.4	0.4	0.4	0.4
25	1	1.9	2.3	2.4	3.0	3.5	4.2
50	1	5.3	6.2	6.5	6.5	6.5	6.5
100	1	submerged	submerged	submerged	submerged	submerged	submerged
200	1	submerged	submerged	submerged	submerged	submerged	submerged
500	1	submerged	submerged	submerged	submerged	submerged	submerged
2	1.5	0.0	0.0	0.0	0.0	0.0	0.0
5	1.5	0.0	0.0	0.0	0.0	0.1	0.3
10	1.5	0.4	0.1	0.6	0.8	1.1	1.4
25	1.5	2.8	2.8	3.5	4.4	5.3	submerged
50	1.5	7.1	submerged	submerged	submerged	submerged	submerged
100	1.5	submerged	submerged	submerged	submerged	submerged	submerged
200	1.5	submerged	submerged	submerged	submerged	submerged	submerged
500	1.5	submerged	submerged	submerged	submerged	submerged	submerged
2	2	0.0	0.0	0.0	0.0	0.0	0.0
5	2	0.0	0.0	0.1	0.1	0.1	0.1
10	2	0.5	1.1	1.4	1.9	2.5	3.2
25	2	3.9	3.9	5.3	5.9	submerged	submerged
50	2	submerged	submerged	submerged	submerged	submerged	submerged
100	2	submerged	submerged	submerged	submerged	submerged	submerged
200	2	submerged	submerged	submerged	submerged	submerged	submerged
500	2	submerged	submerged	submerged	submerged	submerged	submerged
2	2.5	0.0	0.0	0.0	0.0	0.0	0.0
5	2.5	0.0	0.0	0.2	0.4	0.7	1.2
10	2.5	1.4	2.1	2.1	2.7	3.5	4.5
25	2.5	5.9	submerged	submerged	submerged	submerged	submerged
50	2.5	submerged	submerged	submerged	submerged	submerged	submerged
100	2.5	submerged	submerged	submerged	submerged	submerged	submerged
200	2.5	submerged	submerged	submerged	submerged	submerged	submerged
500	2.5	submerged	submerged	submerged	submerged	submerged	submerged
2	3	0.0	0.0	0.0	0.0	0.0	0.1
5	3	0.0	0.1	0.4	0.8	1.4	2.2
10	3	2.3	3.1	3.0	3.9	submerged	submerged
25	3	submerged	submerged	submerged	submerged	submerged	submerged
50	3	submerged	submerged	submerged	submerged	submerged	submerged
100	3	submerged	submerged	submerged	submerged	submerged	submerged
200	3	submerged	submerged	submerged	submerged	submerged	submerged
500	3	submerged	submerged	submerged	submerged	submerged	submerged


Reach 2A 2014 Condition 0 ft. Sea Level Rise Post-Storm Recession
Elevation (ft NGVD)
16
14
12
10
8
6
4
2
0
-2
-4
-6
-8
-10
-100
-50
0
50
100
150
200
250
300
350
400
450
500
550
600
Distance (ft) from Baseline
existing
2 yr
5 yr
10 yr
25yr
50 yr
100 yr
200 yr
500 yr

c.

Figure 9

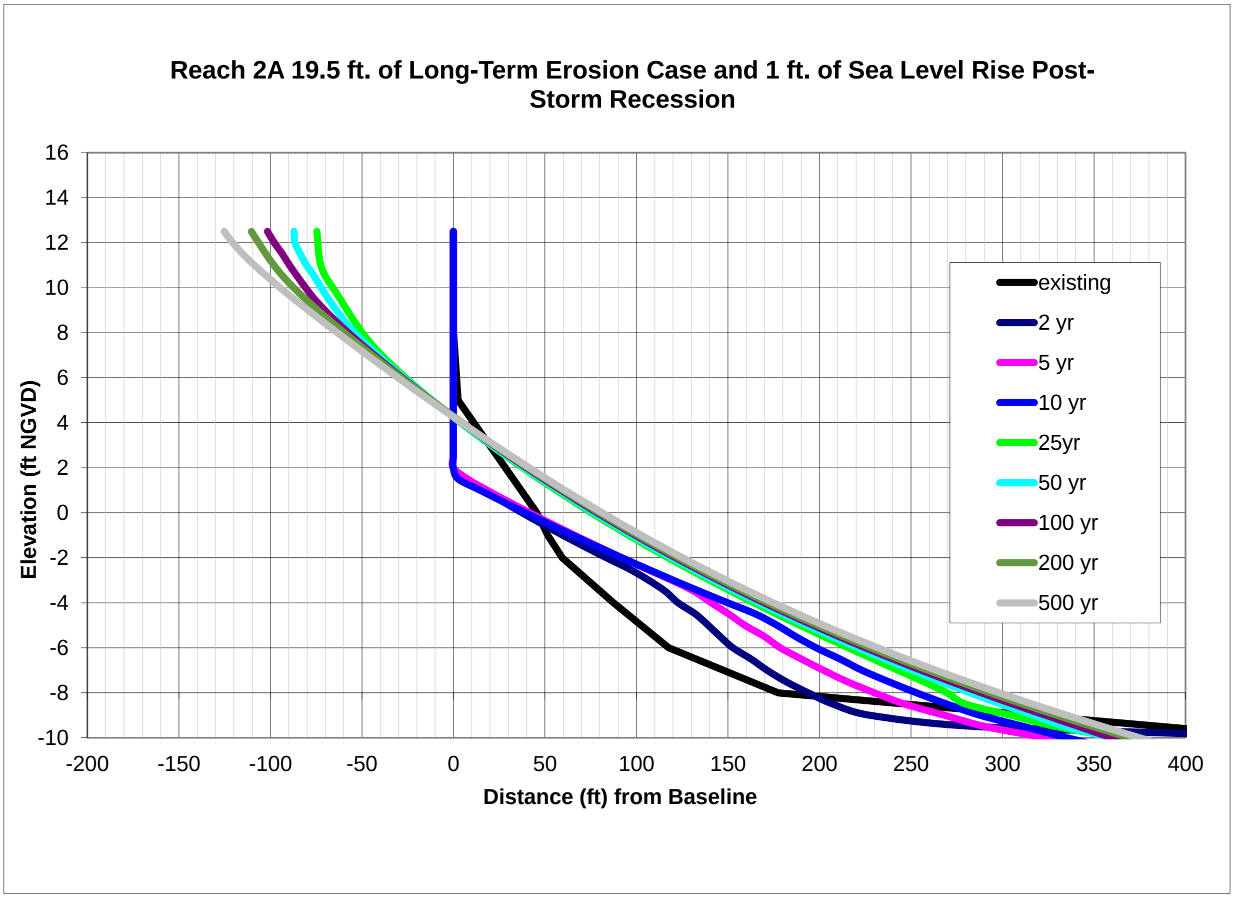

Reach 2A 19.5 ft. of Long-Term Erosion Case and 1 ft. of Sea Level Rise Post-Storm Recession
Elevation (ft NGVD)
16
14
12
10
8
6
4
2
0
-2
-4
-6
-8
-10
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
Distance (ft) from Baseline
existing
2 yr
5 yr
10 yr
25yr
50 yr
100 yr
200 yr
500 yr

Figure 10

Table 7

Reach 2A Without-Project Flowrates at the Bulkhead in cfs/ft.							
Return Period in years	Sea Level Rise Height in ft.	2014	6.5 ft erosion	13 ft erosion	19.5 ft erosion	26 ft erosion	32.5 ft erosion
2	0	0.0	0.0	0.0	0.0	0.0	0.0
5	0	0.0	0.0	0.0	0.0	0.0	0.0
10	0	0.0	0.0	0.0	0.0	0.0	0.0
25	0	1.0	1.1	1.1	1.1	1.1	1.2
50	0	3.1	3.2	3.2	3.2	3.2	3.3
100	0	6.5	6.5	6.5	6.5	6.5	6.5
200	0	submerged	submerged	submerged	submerged	submerged	submerged
500	0	submerged	submerged	submerged	submerged	submerged	submerged
2	0.5	0.0	0.0	0.0	0.0	0.0	0.0
5	0.5	0.0	0.0	0.0	0.0	0.0	0.0
10	0.5	0.1	0.1	0.1	0.2	0.2	0.2
25	0.5	1.7	1.7	1.8	1.8	1.8	1.8
50	0.5	4.4	4.4	4.5	4.5	4.5	4.5
100	0.5	submerged	submerged	submerged	submerged	submerged	submerged
200	0.5	submerged	submerged	submerged	submerged	submerged	submerged
500	0.5	submerged	submerged	submerged	submerged	submerged	submerged
2	1	0.0	0.0	0.0	0.0	0.0	0.0
5	1	0.0	0.0	0.0	0.0	0.0	0.0
10	1	0.4	0.4	0.4	0.4	0.4	0.4
25	1	2.7	2.8	2.8	2.8	2.9	2.9
50	1	5.9	5.9	6.0	6.0	6.0	6.0
100	1	submerged	submerged	submerged	submerged	submerged	submerged
200	1	submerged	submerged	submerged	submerged	submerged	submerged
500	1	submerged	submerged	submerged	submerged	submerged	submerged
2	1.5	0.0	0.0	0.0	0.0	0.0	0.0
5	1.5	0.0	0.0	0.0	0.0	0.0	0.0
10	1.5	0.9	0.9	1.0	1.0	1.0	1.0
25	1.5	4.0	4.0	4.0	4.0	4.1	4.1
50	1.5	submerged	submerged	submerged	submerged	submerged	submerged
100	1.5	submerged	submerged	submerged	submerged	submerged	submerged
200	1.5	submerged	submerged	submerged	submerged	submerged	submerged
500	1.5	submerged	submerged	submerged	submerged	submerged	submerged
2	2	0.0	0.0	0.0	0.0	0.0	0.0
5	2	0.0	0.0	0.0	0.0	0.1	0.1
10	2	1.6	1.6	1.7	1.7	1.7	1.7
25	2	5.5	5.5	5.5	5.5	5.6	5.6
50	2	submerged	submerged	submerged	submerged	submerged	submerged
100	2	submerged	submerged	submerged	submerged	submerged	submerged
200	2	submerged	submerged	submerged	submerged	submerged	submerged
500	2	submerged	submerged	submerged	submerged	submerged	submerged
2	2.5	0.0	0.0	0.0	0.0	0.0	0.0
5	2.5	0.2	0.2	0.2	0.2	0.2	0.2
10	2.5	2.6	2.7	2.7	2.7	2.8	2.8
25	2.5	submerged	submerged	submerged	submerged	submerged	submerged
50	2.5	submerged	submerged	submerged	submerged	submerged	submerged
100	2.5	submerged	submerged	submerged	submerged	submerged	submerged
200	2.5	submerged	submerged	submerged	submerged	submerged	submerged
500	2.5	submerged	submerged	submerged	submerged	submerged	submerged
2	3	0.0	0.0	0.0	0.0	0.0	0.0
5	3	0.6	0.6	0.6	0.6	0.7	0.7
10	3	4.0	4.1	4.1	4.1	4.2	4.2
25	3	submerged	submerged	submerged	submerged	submerged	submerged
50	3	submerged	submerged	submerged	submerged	submerged	submerged
100	3	submerged	submerged	submerged	submerged	submerged	submerged
200	3	submerged	submerged	submerged	submerged	submerged	submerged
500	3	submerged	submerged	submerged	submerged	submerged	submerged

Table 8

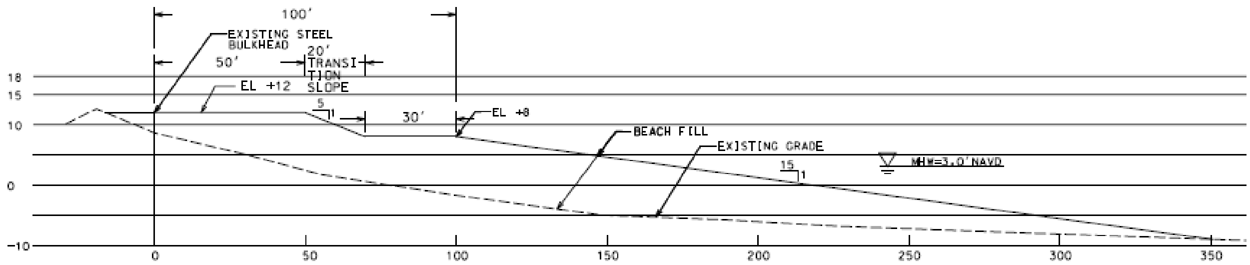
Reach 2A Distance Between Baseline and Maximum Erosion Extent in ft. (+ is landward of baseline)							
Return Period in years	Sea Level Rise Height in ft.	2014	6.5 ft erosion	13 ft erosion	19.5 ft erosion	26 ft erosion	32.5 ft erosion
2	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0
25	0	0	0	0	0	0	0
50	0	62	67	72	76	81	85
100	0	78	83	88	92	97	102
200	0	88	93	98	102	107	112
500	0	102	107	112	117	122	127
2	0.5	0	0	0	0	0	0
5	0.5	0	0	0	0	0	0
10	0.5	0	0	0	0	0	0
25	0.5	0	0	0	0	0	0
50	0.5	67	72	77	82	86	91
100	0.5	83	88	93	97	102	107
200	0.5	92	97	102	107	111	116
500	0.5	106	111	116	121	126	131
2	1	0	0	0	0	0	0
5	1	0	0	0	0	0	0
10	1	0	0	0	0	0	0
25	1	60	66	70	75	79	83
50	1	73	78	83	87	92	96
100	1	87	92	97	102	106	111
200	1	96	101	106	110	115	120
500	1	110	116	121	125	130	135
2	1.5	0	0	0	0	0	0
5	1.5	0	0	0	0	0	0
10	1.5	54	59	64	68	73	77
25	1.5	66	71	76	80	85	89
50	1.5	79	84	89	94	98	103
100	1.5	91	96	101	106	110	115
200	1.5	100	106	110	115	120	125
500	1.5	114	120	125	130	134	139
2	2	0	0	0	0	0	0
5	2	0	0	0	0	0	0
10	2	60	65	70	74	78	83
25	2	72	77	82	86	90	95
50	2	84	89	94	98	103	107
100	2	95	100	105	110	114	119
200	2	104	109	114	119	124	129
500	2	118	124	129	134	138	143
2	2.5	0	0	0	0	0	0
5	2.5	0	0	0	0	0	0
10	2.5	67	72	76	81	85	89
25	2.5	78	83	88	92	96	101
50	2.5	88	93	98	103	107	112
100	2.5	98	104	108	113	118	123
200	2.5	107	113	118	122	127	132
500	2.5	122	127	132	137	142	147
2	3	0	0	0	0	0	0
5	3	63	68	73	77	81	85
10	3	73	78	83	87	92	96
25	3	84	89	94	98	102	107
50	3	93	98	103	107	112	116
100	3	102	107	112	117	121	126
200	3	111	116	121	126	131	135
500	3	125	130	135	140	145	150

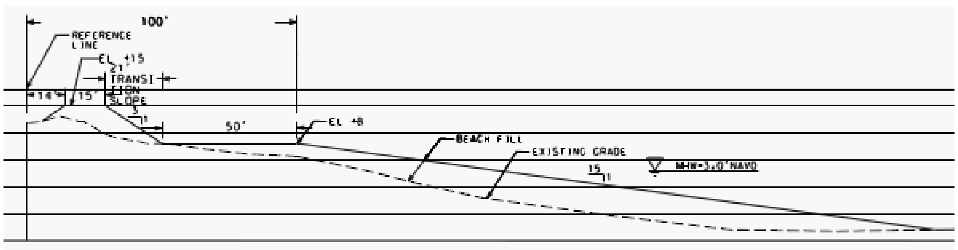
d.

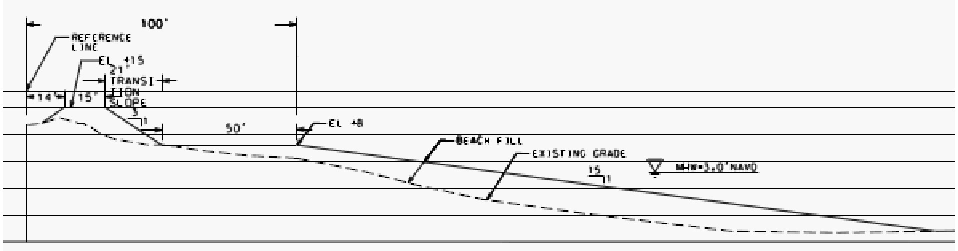
EXISTING AND FUTURE WITH-PROJECT CONDITION DAMAGE ESTIMATES

A17. The design plans evaluated included: a composite berm in Reach 1A with a 50 ft. wide crest at elevation +12.5 ft. NGVD (flush with the existing public seawall), sloping down to a berm at elevation +8 ft. NGVD of 30, 50, and 70 ft. comparative widths; a classic dune and berm cross section in Reach 1B with a 15 ft. wide dune at elevation +15 ft. NGVD, sloping to a berm at elevation +8 ft. NGVD of 30, 50, and 70 ft. comparative widths; and a classic berm only cross section in Reach 2A with crest elevation +8 ft. NGVD of 30, 50, and 70 ft. comparative widths.

A18. With-project evaluations for Reach 1A included wave overtopping flowrates as a damage mechanism for the seawall and the public road. These evaluations are described below.







A19. Storm-induced erosion modeling was performed. The output reported is the remaining distance between the eroded upper berm crest (at elevation +12.5 ft. NGVD) and the public seawall. The purpose of this output was to use for development of the with-project wave flowrate model to determine the wave overtopping if the waves reach the public seawall. A standard splash blanket design width was calculated in order to provide a gross estimate of how far the overtopping splash can intrude. This value was then compared to the remaining distance between the crest and the seawall to determine whether the overtopping splash reaches the seawall. The resulting flow rate modeling results are shown in Table 10 for the 30, 50, and 70 ft. berm width plans, respectively, and represent the flowrate at the public seawall location if the wave splash reaches the wall and hence are relevant to be used to determine failure of the seawall and road due to wave overtopping. Submergence is assumed to cause failure of the seawall and road. According to the results in Table 9 the 25-yr return period is predicted to cause failure in all the Alternative berm widths in the 3 ft. sea level rise cases; 50-yr fails all berm width alternatives for the 1.5, 2.0, and 2.5 ft. sea level rise cases; 100-yr fails all berm width alternatives for the 1.0 and 0.5 ft. sea level rise cases and for the 0 ft. sea level rise with the 30 ft. Berm Width Alternative; and the 200-yr fails the 50 and 50 ft. berm width alternatives for the 0 ft. sea level rise cases.

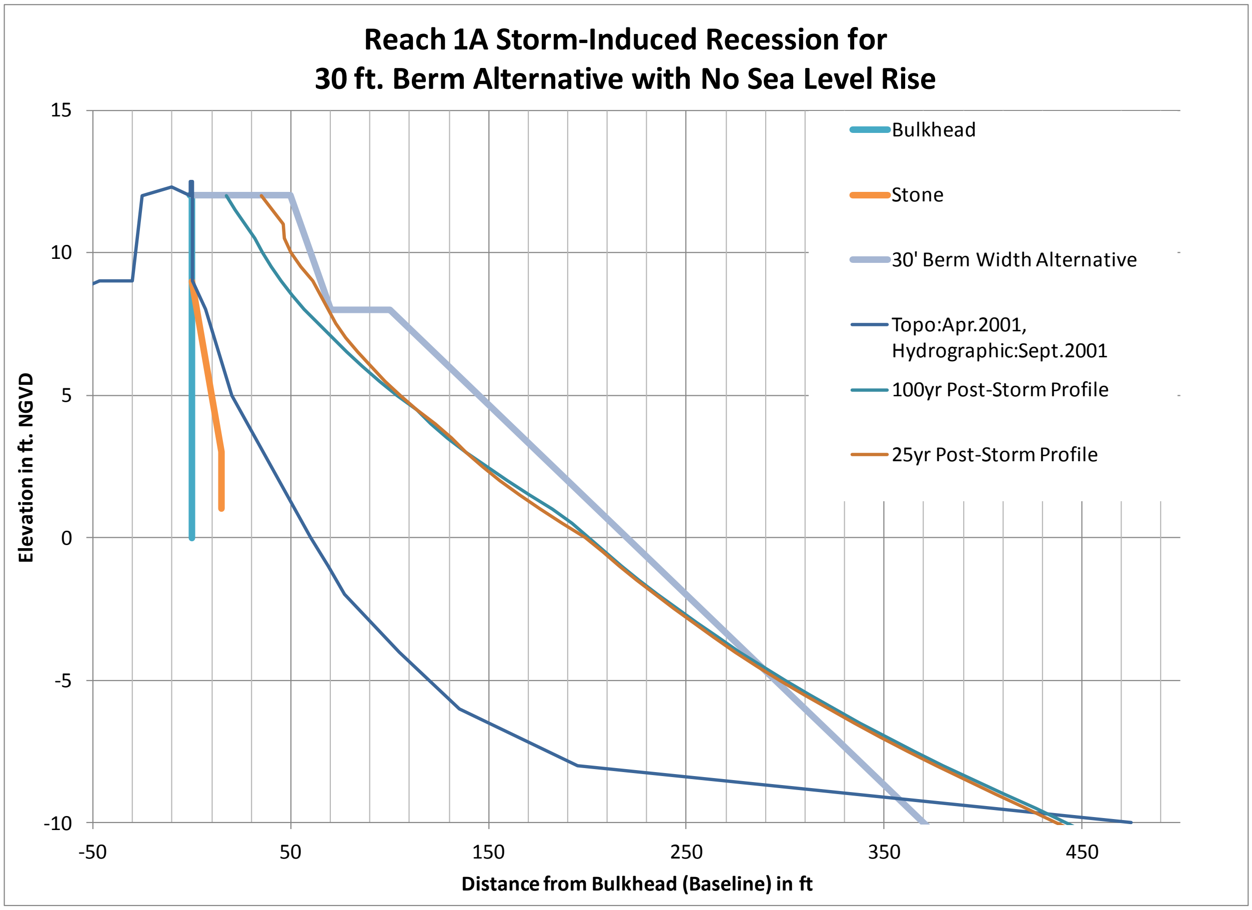

Reach 1A Storm-Induced Recession for
30 ft. Berm Alternative with No Sea Level Rise
Bulkhead
Stone
30' Berm Width Alternative
Topo:Apr.2001, Hydrographic:Sept.2001
100yr Post-Storm Profile
25yr Post-Storm Profile
Elevation in ft. NGVD
Distance from Bulkhead (Baseline) in ft
15
10
5
0
-5
-10
-50
50
150
250
350
450

Figure 11

Table 9

Reach 1A With-Project Wave Overtopping Flowrates in cfs/ft.				
Return Period in years	Sea Level Rise Height in ft.	30 ft. Alternative Berm Width Plan	50 ft. Alternative Berm Width Plan	70 ft. Alternative Berm Width Plan
2	0	0.0	0.0	0.0
5	0	0.0	0.0	0.0
10	0	0.0	0.0	0.0
25	0	0.0	0.0	0.0
50	0	0.0	0.0	0.0
100	0	1.2	0.0	0.0
200	0	submerged	submerged	submerged
500	0	submerged	submerged	submerged
2	0.5	0.0	0.0	0.0
5	0.5	0.0	0.0	0.0
10	0.5	0.0	0.0	0.0
25	0.5	0.0	0.0	0.0
50	0.5	0.0	0.0	0.0
100	0.5	submerged	submerged	submerged
200	0.5	submerged	submerged	submerged
500	0.5	submerged	submerged	submerged
2	1	0.0	0.0	0.0
5	1	0.0	0.0	0.0
10	1	0.0	0.0	0.0
25	1	0.0	0.0	0.0
50	1	0.0	0.0	0.0
100	1	submerged	submerged	submerged
200	1	submerged	submerged	submerged
500	1	submerged	submerged	submerged
2	1.5	0.0	0.0	0.0
5	1.5	0.0	0.0	0.0
10	1.5	0.0	0.0	0.0
25	1.5	0.0	0.0	0.0
50	1.5	submerged	submerged	submerged
100	1.5	submerged	submerged	submerged
200	1.5	submerged	submerged	submerged
500	1.5	submerged	submerged	submerged
2	2	0.0	0.0	0.0
5	2	0.0	0.0	0.0
10	2	0.0	0.0	0.0
25	2	0.0	0.0	0.0
50	2	submerged	submerged	submerged
100	2	submerged	submerged	submerged
200	2	submerged	submerged	submerged
500	2	submerged	submerged	submerged
2	2.5	0.0	0.0	0.0
5	2.5	0.0	0.0	0.0
10	2.5	0.0	0.0	0.0
25	2.5	0.0	0.0	0.0
50	2.5	submerged	submerged	submerged
100	2.5	submerged	submerged	submerged
200	2.5	submerged	submerged	submerged
500	2.5	submerged	submerged	submerged
2	3	0.0	0.0	0.0
5	3	0.0	0.0	0.0
10	3	0.0	0.0	0.0
25	3	submerged	submerged	submerged
50	3	submerged	submerged	submerged
100	3	submerged	submerged	submerged
200	3	submerged	submerged	submerged
500	3	submerged	submerged	submerged

A20. The design plan for Reach 1B of the classic dune and berm cross section with a 15 ft. wide dune at elevation +15 ft. NGVD, sloping to a berm at elevation +8 ft. NGVD of 30, 50, and 70 ft. comparative widths were evaluated for storm induced erosion undermining of the road located landward of the dune, and damage of the road due to overtopping at the dune location.

A21. As in the previous Reach, splash blanket calculations were used to grossly estimate whether the wave splash has potential to reach the road. The resulting beach wave overtopping flowrates if the flow can reach the road are shown in Table 10. According to the results in Table 10, damage is initiated on the road with a 25-yr event for the 30 ft. Berm Alternative with the 2.5 and 3 ft. of sea level cases and for the 50 ft. Berm Alternative with the 3 ft. of sea level rise case; the 50-yr event for the 30 ft. Berm Alternative with the 1.0, 1.5, and 2.0 ft. of sea level rise cases, the 50 ft. Berm Alternative for the 1.5, 2.0 and 2.5 ft. of sea level rise cases, and the 70 ft. Berm Alternative with the 2, 2.5, and 3 ft. of sea level rise cases; the 100-yr event for the 30 ft. Berm Alternative with the 0 and 0.5 ft. sea level rise cases, the 50 ft. Berm Alternative with the 0.5 and 1 ft. sea level rise cases, and the 70 ft. Berm Alternative with the 0.5, 1, and 1.5 ft. of sea level rise cases; and the 200-yr event for the 50 and 70 ft. Berm Alternatives with no sea level rise.

A22. In Reach 2A with the classic berm only cross section in Reach 2A with crest elevation +8 ft. NGVD of 30, 50, and 70 ft. comparative widths was evaluated for damage to the private bulkheads and potential intrusion of erosion under the residential structures.

A23. The with-project wave overtopping flow rates for post-storm bulkhead and berm alternatives for Reach 2A are shown in Table 11. The flowrate results show that damage to the bulkhead is initiated with a 10-yr event for all the berm width alternatives for the 2.5 and 3 ft. sea level rise cases and the 30 ft. Berm Alternative for the 2 ft. sea level rise case; with a 25-yr event for the 50 and 70 ft. Berm Alternatives with 1, 1.5, and 2 ft. seal level rise cases and the 30 ft. Berm Alternative with 0.5, 1, and 1.5 ft. sea level rise cases; and with a 50-yr event for the 50 and 70 ft. Berm Alternatives with 0 and 0.5 ft. sea level rise cases and for the 30 ft. Berm Alternative with no sea level rise case.

A24. As in the previous two reaches, wave splash blanket distances were estimated grossly using splash blanket width calculations in Reach 2A. The resulting post-storm erosion intrusion distances, for the cases where the bulkhead fails midway through the storm, are shown in Table 12. These represent the potential undermining extents of the residential structures located behind the bulkhead. As an example, if a structure will experience damage when the erosion extent reaches 60 ft. then a 25-yr event would cause damage with the 30 ft. Berm Alternative with 3 ft. sea level rise case; a 50-yr event would cause damage with the 30 ft. Berm Alternative with 2.5 ft. sea level rise case; 100-yr event with the 30 ft. Berm Alternative with 1, 1.5, and 2 ft. sea level rise cases and the 50 ft. Berm Alternative with 2.5 and 3 ft. sea level rise cases; the 200-yr event with the 30 ft. Berm Alternative with 0 and 0.5 ft. sea level rise case and the 50 ft. Berm Alternatives with 1.5 and 2 ft. sea level rise cases; and the 500-yr event with 0, 0.5, and 1 ft. sea level rise cases, and the 70 ft. Berm Alternatives with 2, 2.5 and 3 ft. sea level rise cases.

A25. As a recap, Reach 1A uses damage to the public seawall and road. The potential failure mechanisms include wave scour and wave overtopping flowrates. Table 3 and Table 4 show the results of these analyses, respectively. Table 9 shows the resulting flowrates directly correlated to damage of the public seawall.

A26. Recapping damages to the road behind the dune in Reach 1B, **Error! Reference source not found.** shows the without-project erosion extents landward of the baseline (which relate directly to road undermining), Table 5 shows the without-project flowrates, which directly impact the road. Table 10 shows the with-project wave overtopping flowrates which directly impact the road.

A27. For Reach 2A recap, the damages evaluated included damage to the private bulkheads by overtopping, and then undermining of the residential structures located behind the bulkhead. Here the bulkhead is the baseline. Table 8 shows the results of the without-project wave overtopping flowrates at the bulkheads, which directly correlate to damage to the bulkheads. Table 9 shows the without-project storm erosion intrusion extents landward of the bulkhead in the cases of bulkhead failure. Table 11 shows the with-project post-storm overtopping flowrate results. Table 12 shows the post-storm with-project erosion intrusion extents landward of the bulkhead for the cases where the bulkhead failed.

Table 10

Reach 1B With-Project Wave Overtopping Flowrates in cfs/ft.				
Return Period in years	Sea Level Rise Height in ft..	30 ft. Alternative Berm Width Plan	50 ft. Alternative Berm Width Plan	70 ft. Alternative Berm Width Plan
2	0	0.0	0.0	0.0
5	0	0.0	0.0	0.0
10	0	0.0	0.0	0.0
25	0	0.0	0.0	0.0
50	0	0.0	0.0	0.0
100	0	0.5	0.1	0.1
200	0	submerged	submerged	3.7
500	0	submerged	submerged	submerged
2	0.5	0.0	0.0	0.0
5	0.5	0.0	0.0	0.0
10	0.5	0.0	0.0	0.0
25	0.5	0.0	0.0	0.0
50	0.5	0.0	0.0	0.0
100	0.5	submerged	2.6	0.9
200	0.5	submerged	submerged	submerged
500	0.5	submerged	submerged	submerged
2	1	0.0	0.0	0.0
5	1	0.0	0.0	0.0
10	1	0.0	0.0	0.0
25	1	0.0	0.0	0.0
50	1	1.1	0.0	0.0
100	1	submerged	submerged	1.9
200	1	submerged	submerged	submerged
500	1	submerged	submerged	submerged
2	1.5	0.0	0.0	0.0
5	1.5	0.0	0.0	0.0
10	1.5	0.0	0.0	0.0
25	1.5	0.0	0.0	0.0
50	1.5	2.0	0.6	0.1
100	1.5	submerged	submerged	submerged
200	1.5	submerged	submerged	submerged
500	1.5	submerged	submerged	submerged
2	2	0.0	0.0	0.0
5	2	0.0	0.0	0.0
10	2	0.0	0.0	0.0
25	2	0.2	0.0	0.0
50	2	submerged	submerged	1.1
100	2	submerged	submerged	submerged
200	2	submerged	submerged	submerged
500	2	submerged	submerged	submerged
2	2.5	0.0	0.0	0.0
5	2.5	0.0	0.0	0.0
10	2.5	0.0	0.0	0.0
25	2.5	1.6	0.4	0.0
50	2.5	submerged	submerged	2.5
100	2.5	submerged	submerged	submerged
200	2.5	submerged	submerged	submerged
500	2.5	submerged	submerged	submerged
2	3	0.0	0.0	0.0
5	3	0.0	0.0	0.0
10	3	0.0	0.0	0.0
25	3	submerged	1.1	0.2
50	3	submerged	submerged	submerged
100	3	submerged	submerged	submerged
200	3	submerged	submerged	submerged
500	3	submerged	submerged	submerged

Table 11

Reach 2A With-Project Wave Overtopping Flowrates in cfs/ft.				
Return Period in years	Sea Level Rise Height in ft.	30 ft. berm width	50 ft. berm width	70 ft. berm width
2	0	0.0	0.0	0.0
5	0	0.0	0.0	0.0
10	0	0.0	0.0	0.0
25	0	0.2	0.0	0.0
50	0	2.1	1.3	1.3
100	0	8.1	7.9	7.6
200	0	submerged	submerged	submerged
500	0	submerged	submerged	submerged
2	0.5	0.0	0.0	0.0
5	0.5	0.0	0.0	0.0
10	0.5	0.0	0.0	0.0
25	0.5	0.6	0.3	0.3
50	0.5	4.2	3.4	3.4
100	0.5	submerged	submerged	submerged
200	0.5	submerged	submerged	submerged
500	0.5	submerged	submerged	submerged
2	1	0.0	0.0	0.0
5	1	0.0	0.0	0.0
10	1	0.0	0.0	0.0
25	1	1.6	1.1	1.1
50	1	7.1	6.5	6.5
100	1	submerged	submerged	submerged
200	1	submerged	submerged	submerged
500	1	submerged	submerged	submerged
2	1.5	0.0	0.0	0.0
5	1.5	0.0	0.0	0.0
10	1.5	0.1	0.0	0.0
25	1.5	3.5	2.8	2.8
50	1.5	submerged	submerged	submerged
100	1.5	submerged	submerged	submerged
200	1.5	submerged	submerged	submerged
500	1.5	submerged	submerged	submerged
2	2	0.0	0.0	0.0
5	2	0.0	0.0	0.0
10	2	0.5	0.3	0.3
25	2	6.5	6.0	6.0
50	2	submerged	submerged	submerged
100	2	submerged	submerged	submerged
200	2	submerged	submerged	submerged
500	2	submerged	submerged	submerged
2	2.5	0.0	0.0	0.0
5	2.5	0.0	0.0	0.0
10	2.5	1.6	1.2	1.2
25	2.5	submerged	submerged	submerged
50	2.5	submerged	submerged	submerged
100	2.5	submerged	submerged	submerged
200	2.5	submerged	submerged	submerged
500	2.5	submerged	submerged	submerged
2	3	0.0	0.0	0.0
5	3	0.0	0.0	0.0
10	3	3.7	3.2	3.2
25	3	submerged	submerged	submerged
50	3	submerged	submerged	submerged
100	3	submerged	submerged	submerged
200	3	submerged	submerged	submerged
500	3	submerged	submerged	submerged

Table 12

Reach 2A With-Project Maximum Erosion Extent Landward of Bulhead (Baseline) in ft.				
Return Period in years	Sea Level Rise Height in ft.	30 ft. Alternative Berm Width	50 ft. Alternative Berm Width	70 ft. Alternative Berm Width
2	0	0	0	0
5	0	0	0	0
10	0	0	0	0
25	0	0	0	0
50	0	37	24	4
100	0	53	40	21
200	0	63	50	30
500	0	77	64	44
2	0.5	0	0	0
5	0.5	0	0	0
10	0.5	0	0	0
25	0.5	22	0	0
50	0.5	43	30	10
100	0.5	58	45	25
200	0.5	67	54	34
500	0.5	81	68	48
2	1	0	0	0
5	1	0	0	0
10	1	0	0	0
25	1	36	23	3
50	1	48	35	16
100	1	62	49	30
200	1	71	58	38
500	1	85	72	53
2	1.5	0	0	0
5	1.5	0	0	0
10	1.5	0	0	0
25	1.5	42	29	9
50	1.5	54	41	22
100	1.5	66	53	34
200	1.5	75	62	43
500	1.5	90	77	57
2	2	0	0	0
5	2	0	0	0
10	2	24	0	0
25	2	47	34	15
50	2	59	46	27
100	2	70	57	38
200	2	79	66	47
500	2	94	81	61
2	2.5	0	0	0
5	2.5	0	0	0
10	2.5	30	0	0
25	2.5	59	43	18
50	2.5	65	52	32
100	2.5	74	61	42
200	2.5	83	70	51
500	2.5	98	85	65
2	3	0	0	0
5	3	0	0	0
10	3	32	0	0
25	3	63	45	19
50	3	67	54	34
100	3	76	63	43
200	3	85	72	52
500	3	99	86	67

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ATTACHMENT A2

GENESIS MODELING

Shoreline Change Modeling, Asharoken, New York

Evaluation of Long Term Effects of Shoreline Stabilization Alternatives

Final Report

December 2014

Prepared For

U.S. Army Corps of Engineers
New York, NY

Prepared By

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<u>Table of Contents</u>

Shoreline Change Modeling, Asharoken, New York

List of Figures

Shoreline Change Modeling, Asharoken, NY

1 Introduction

This report provides a numerical model analysis of longshore sediment transport responses to groins constructed at critical erosion areas to retain the beachfill for the Feasibility Study at Asharoken, Long Island, New York. The following modeling procedures are performed using GENESIS model:

- Historic shoreline data have been assembled from the years 1994, 2001 and 2013 and converted to an x-y coordinate relative to the numerical model baseline,
- A nearshore wave climate is defined based on a prior hindcast for Long Island Sound that is used to drive the movement of sediment in the numerical model,
- Offshore wave data are transformed to the nearshore area of Asharoken using a directional spectral model, STWAVE,
- The numerical model is calibrated using shoreline changes defined by the historic shorelines,
- The calibrated model is applied to evaluate the performance of beach fill alone along the entire project area and in comparison to structural alternatives. At the western end of the study area, the model is used to assess the use of new groin alternatives and potential changes if the existing large rock groin is shortened or removed. The model is also applied to evaluate the performance of new groin and offshore breakwater alternatives at the eastern end of the study area.
- A summary of the changes in the shoreline position for the various alternatives is provided, including estimates in the change in renourishment requirements to maintain the design beach width.

2 Modeling System

Modeling of longshore sediment transport and shoreline change was conducted by application of the U.S. Army Corps of Engineers numerical model GENESIS (Generalized Model for Simulating Shoreline Change) that is operated from within the Coastal Engineering Design and Analysis System (CEDAS). The model is well-documented and is considered a standard tool for evaluating long term coastal project response (Hanson and Kraus, 1989; Gravens, Kraus and Hanson, 1991). The incorporation of the capability to model tombolo formation in the lee of offshore breakwaters was incorporated into a version called GENESIS by Hanson, Larson, Kraus and Gravens (2006) using validation data from Gravens and Wang (2007).

GENESIS is a model that calculates shoreline change caused primarily by wave action and can be applied to a diverse variety of situations involving almost arbitrary numbers, locations and combinations of groins, jetties, detached breakwaters, seawalls and beach fills. The model assumes that the beach profile is unchanged, thereby describing beach changes as a shift in shoreline position. Two solution schemes can be used: an implicit model for general use (called GENESIS), and an explicit model (called GENESIS) to simulate tombolo formation in the lee of offshore breakwaters. Both models are applied appropriately for this project.

3 Model Domain

The model domain includes the Asharoken shoreline along Long Island Sound and the coastal and offshore area extending seaward to a depth of approximately -40 ft NGVD. The longshore extent of domain is from approximately 2000 feet west of the new beachfill taper to the west jetty at the power plant cooling water inlet.

4 Bathymetric Data

Topographic data for the Asharoken area was derived via LIDAR collected in 2013 by the New York District. This data set provided the location of the +5 ft NGVD contour, which was defined as the mean high water shoreline in prior aerial photograph and shoreline change analyses (NAN, 2004). The mean high water shoreline in 2013 is thus defined consistently with two other historic digital shorelines that were based on aerial photography taken in 1994 and 2001. Bathymetric data for the coastal and offshore area was derived from the NOS Navigation Chart digital database (NOAA, 2014).

5 Model Development

Offshore Wave Data

Wave data were taken from a wave hindcast that covers the time period 1990-1999, performed for a prior study of sediment transport at Asharoken (2004). Although data are available from a wave buoy located about 40 miles east of the project site in central Long Island Sound since 2004, the buoy does not measure or record wave directional information which is crucial for sediment transport analysis.

The closest archived hindcast wave data is at 40.95 degrees N and 73.35 degrees W in a water depth of approximate 55 feet relative to Mean Sea Level (MSL). The location is depicted in Figure 12. Typical mean daily conditions at this location include waves primarily from the northeast and west-northwest with a height of about 2 feet and a wave period of about 3 seconds. The largest heights and longest wave periods occur from the northeast, as do the largest percentage of waves that are capable of transporting sediment along the coast. The wave characteristics at the offshore hindcast node are presented in Figure 13 and Figure 14.

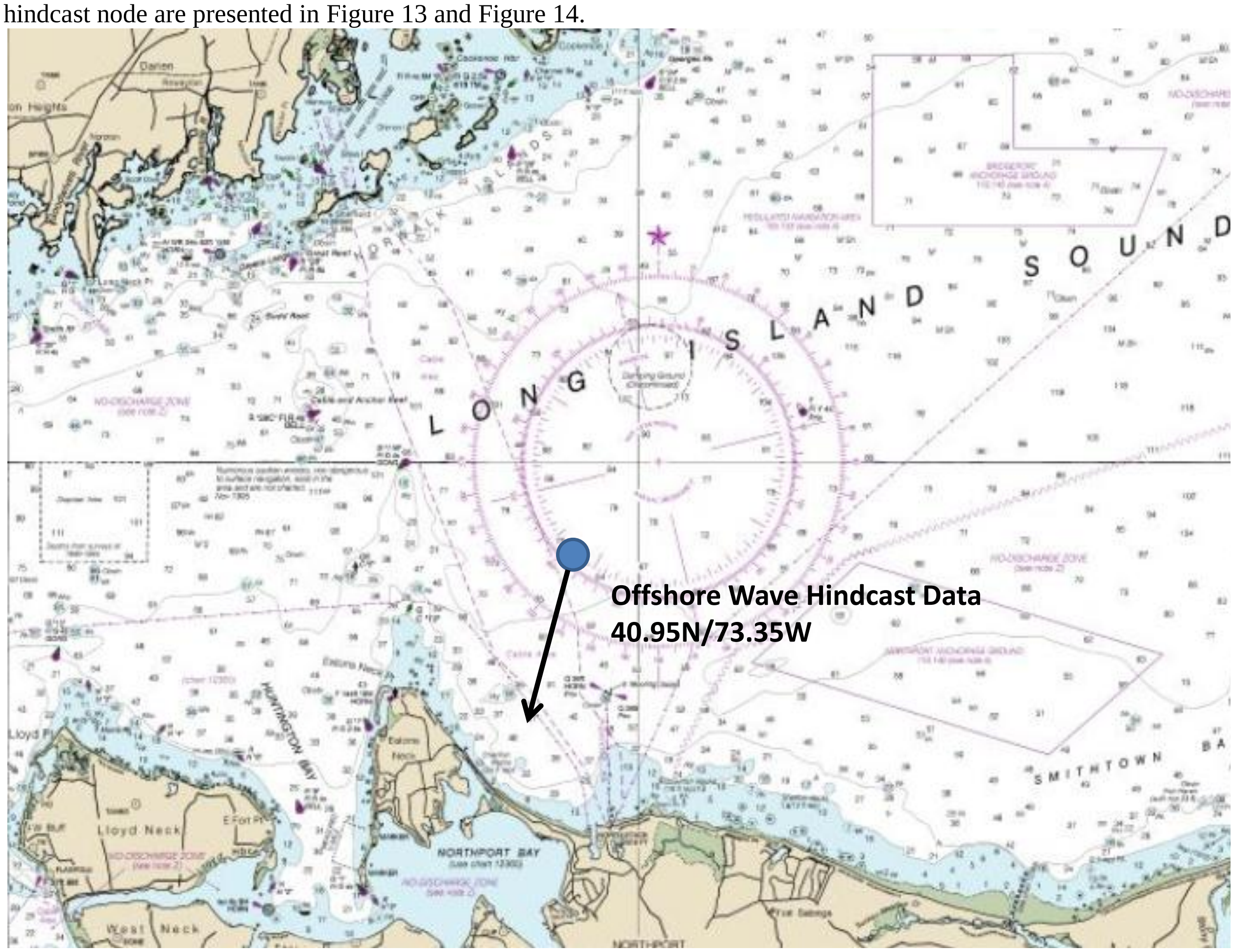

Figure 12. Location of Offshore Wave Hindcast Data Relative to the Study Area and the Direction of Wave Transformation to the Project Site

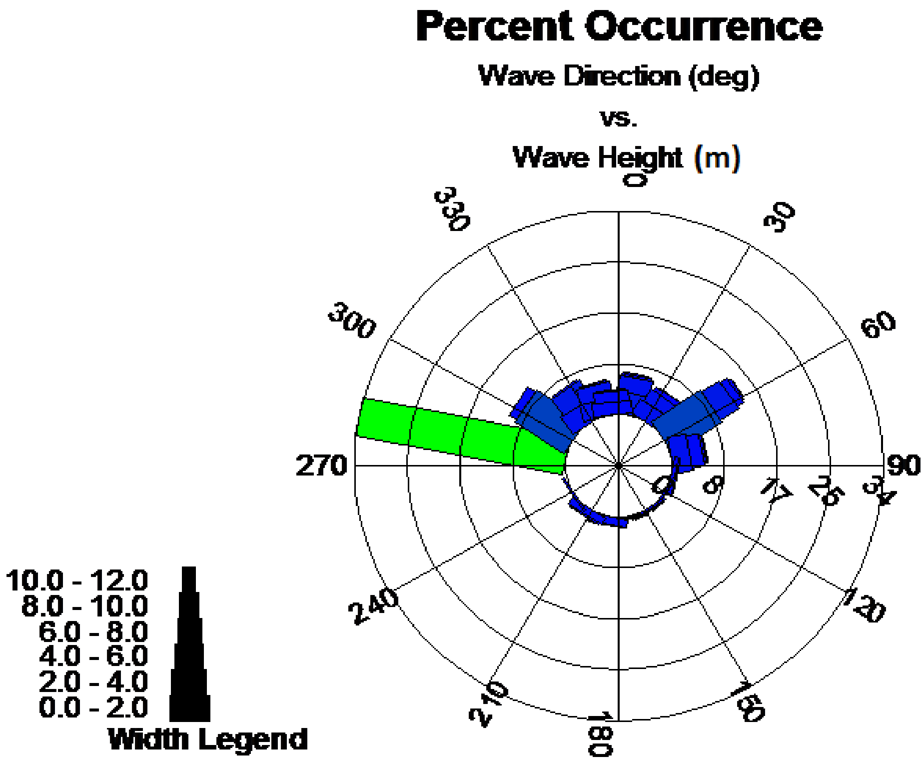

Figure 13. Offshore Wave Characteristics: Wave Height by Direction (Green Indicates Most Frequently Occurring Condition)

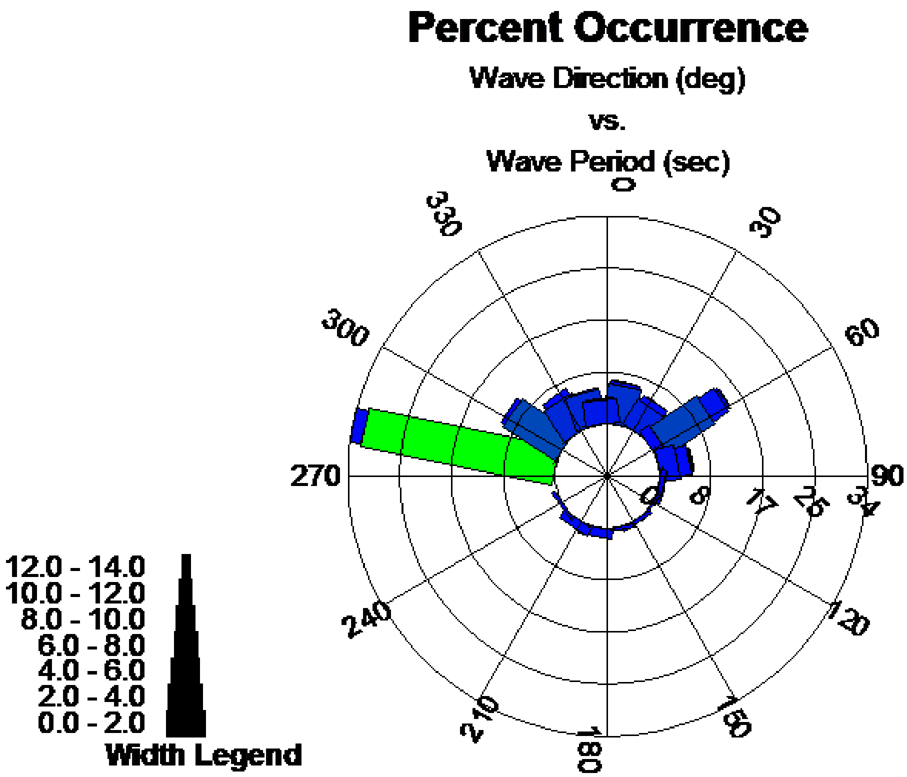

Figure 14. Offshore Wave Characteristics: Wave Period by Direction (Green Indicates Most Frequently Occurring Condition)

Nearshore Waves

The offshore wave information was transformed to the outer boundary of the nearshore Asharoken area where more detailed wave modeling was performed. The transformation to the nearshore area was performed by WISPH3, which is a simplified point to point steady-state spectral transformation model.

Along the outer boundary of the nearshore area, called the STWAVE grid domain (Figure 15), directional spectra were generated for each unique combination of wave height, period and direction in the nearshore wave time history. These waves were then transformed to wave "stations" located every 50 feet along the shoreline on the -15 ft MSL contour using the steady-state directional spectral model STWAVE. The -15 ft MSL contour was selected because it is shoreward of bathymetric features that govern wave transformations that, in turn, govern longshore sediment transport. The transformed wave information along the -15 ft MSL contour is used by the sediment transport and shoreline change model GENESIS.

The importance of applying STWAVE is shown in Figure 16, which shows wave vectors passing over the approximately 30-foot deep hole located at the western end of the study area. During longer wave period events, refractive effects cause a divergence of wave energy that contributes to a gradient in longshore transport along the shoreline. At the center of the gradient is a "Nodal Point" with sediment transport away from the point, creating a "Critical Erosion" shoreline.

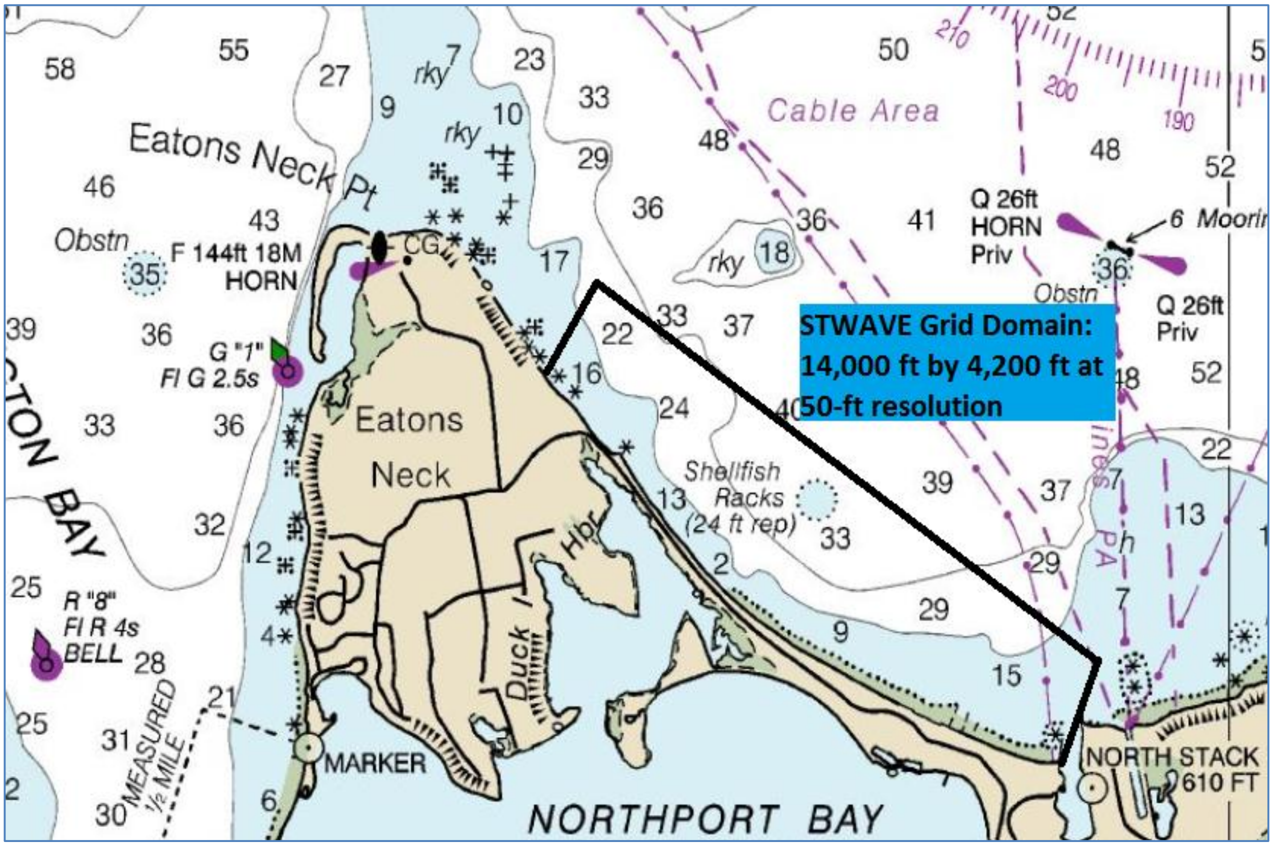

Figure 15. Nearshore Wave Model Grid Domain

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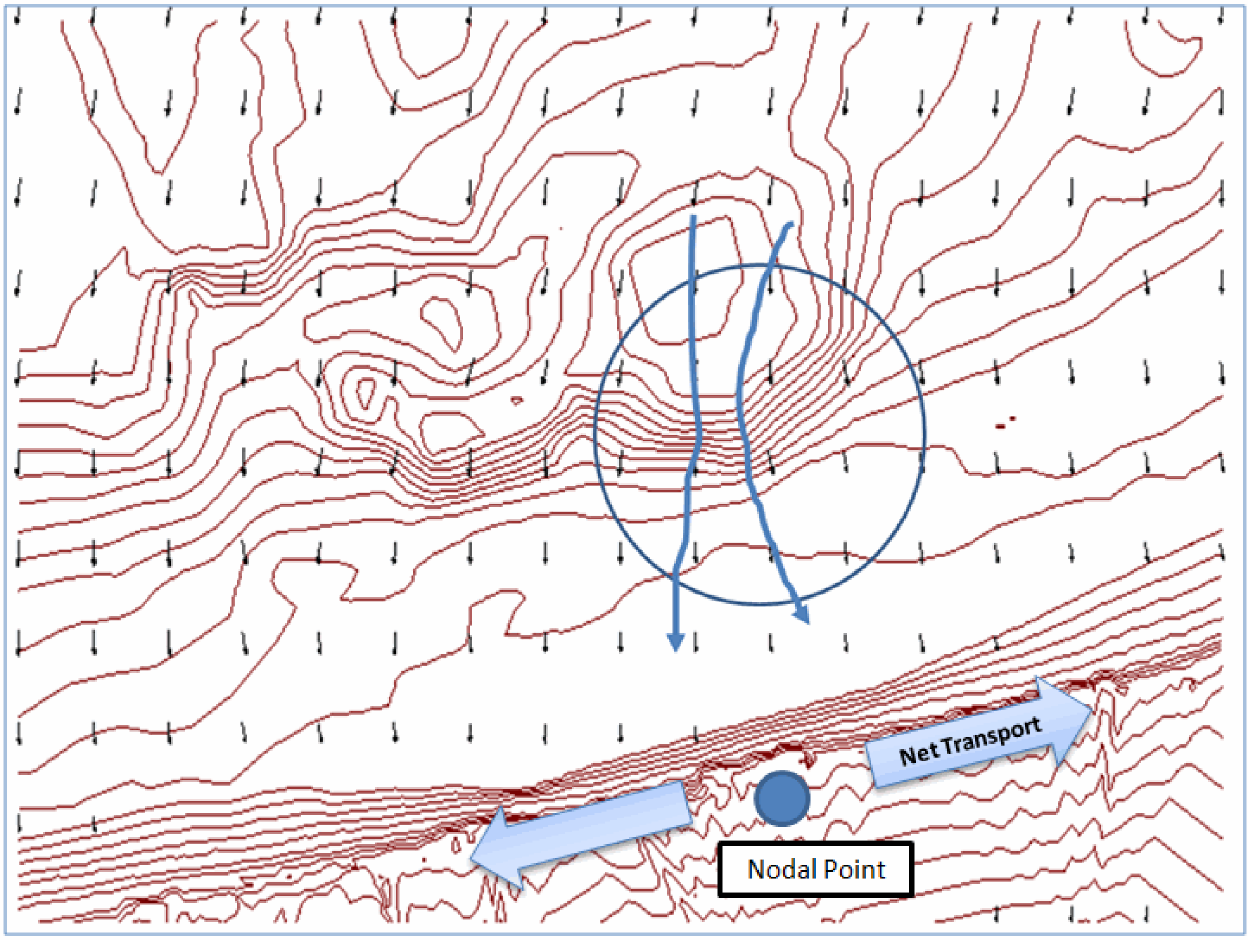


Figure 16. Wave Transformation Over the Deep Hole at East End of Asharoken Area (Wave Vectors are Shown Approaching the Shoreline).

GENESIS Implementation

Shoreline changes due to beach fills and structures are simulated using the model GENESIS. As mentioned earlier, this version of the model was implemented because one of the alternatives under consideration includes an offshore breakwater that is expected to induce salient formation, possibly evolving into a tombolo.

The shoreline change model origin is established at Baseline 1,152,975.1 ft E/283,457.2 ft N. The baseline (+x direction) extends from the origin toward an azimuth of 125 degrees clockwise from north. The shoreline and modeled changes are modeled at an x-increment of 50 feet. Project station 0+00 corresponds to GENESIS coordinate of x=1375 feet.

6 Model Calibration

A. Shoreline change 2001-2013

A broad-scale validation was performed for GENESIS for the time period 2001-2013 to optimize the model performance over the entire project area with focus on the western area (west of GENESIS x-coordinate 4000 feet) and the eastern area (from GENESIS x-coordinate 10000 to 12500). Figure 17 illustrates the overall shoreline calibration. The model is implemented using a 50-ft resolution along its baseline with a model time step of 3 hours. Wave input is provided to the model from the wave hindcast at a 3-hour interval. The mean sediment size D50 is specified as 0.95mm, the average berm height is at +10 ft NAVD and the closure depth is specified as -20 feet NAVD. It should be noted that the waves used to simulate sediment transport extend from 1990 through 1999 (with 1990-1991 repeated), whereas the measured shorelines span 2001-2013. The assumption here, because directional wave data are not available for the 2001 to 2013 time period, is that long term wave statistics that drive sediment transport are similar. Note, however, that the measured shoreline from 2013 includes the effects of Hurricane Sandy, whereas the wave hindcast data include the major northeasters of the early 1990s. These differences cause some localized differences in the model calibration, along with the recognition that the measured shoreline data are snapshots in time with perturbations that GENESIS tends to smooth out.

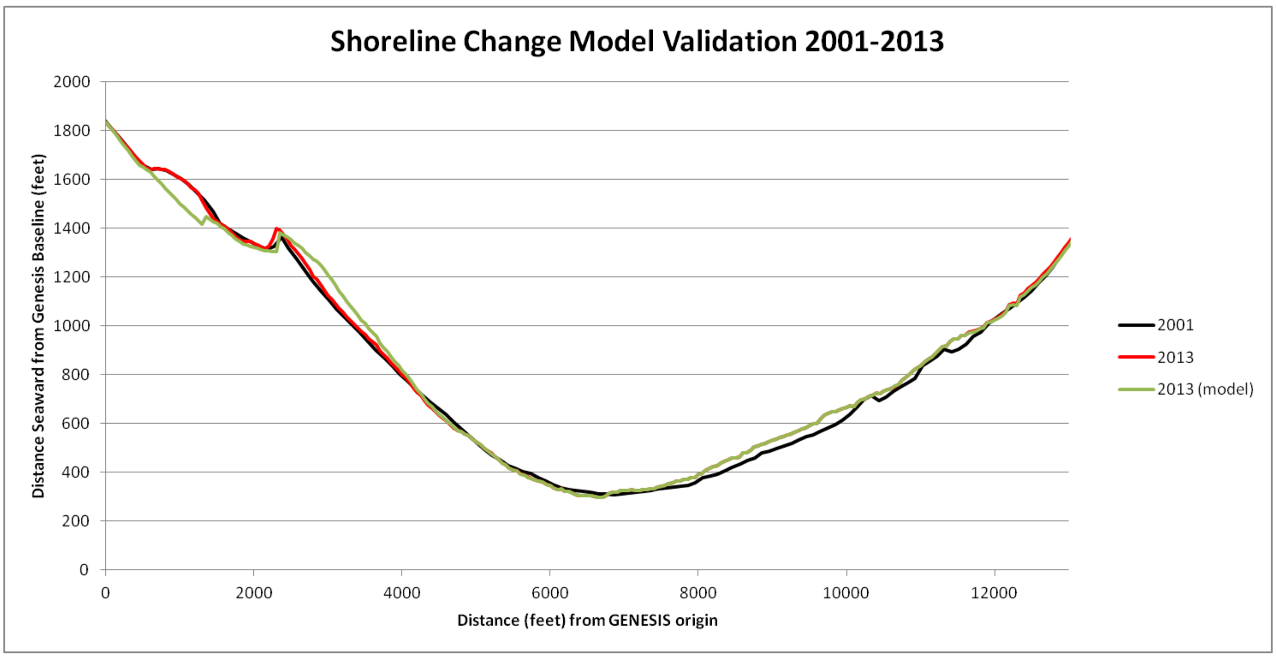


Figure 17. Model Calibration for the Broad Scale Asharoken Area

B. Fillet formation adjacent to old groin

Two areas of concern in the overall calibration figure are adjacent to the existing rock groin at GENESIS x-coordinate 2300 ft. The overprediction of shoreline recession to the west of the groin is attributed to a smoothing of the shoreline bulge by GENESIS. Based on the historic shorelines, the area actually appears quite stable, so GENESIS results in that area will be conservative. The rock groin permeability was varied to optimize the bypassing results to achieve the best possible calibration in the area immediately west of the groin. The results are shown in Figure 18 using a groin permeability of 40% and K1 and K2 parameters equal to 0.3. Note that GENESIS x-coordinate 1350 equates to project station 0+00.

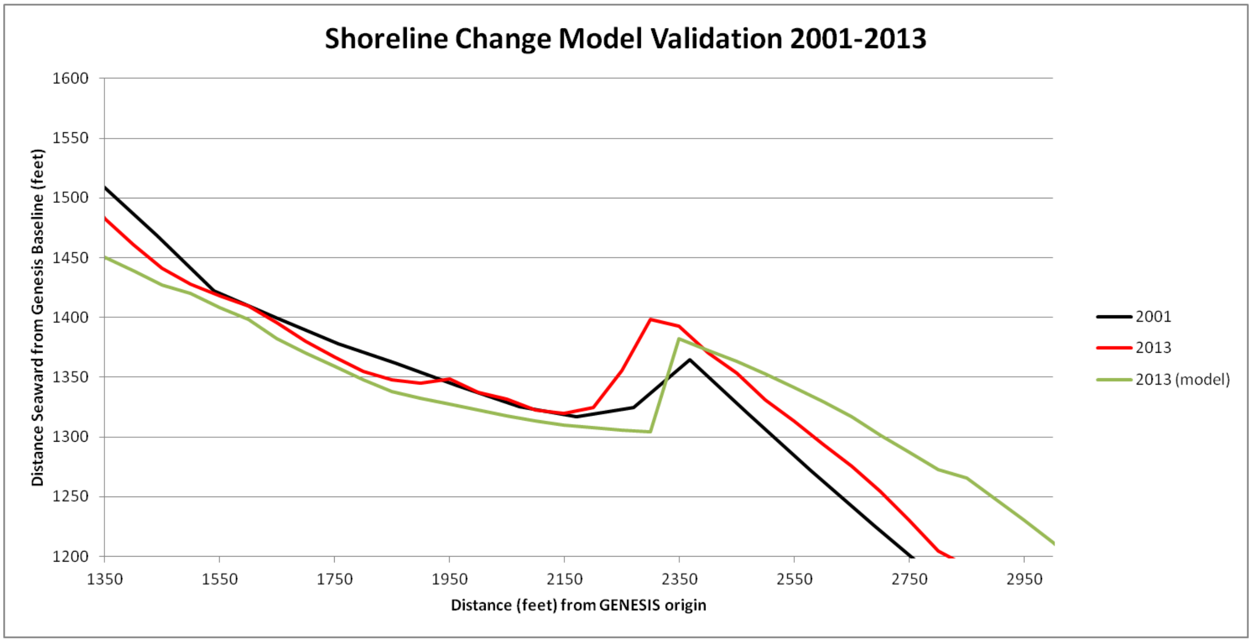


Figure 18. Model Validation Locally to the Existing Groin

7 Modeling of Proposed Alternatives

Shoreline configuration files were developed for each of the proposed alternatives. The GENESIS model was applied to the entire model domain described earlier; however, the assessment of proposed alternatives will be separated into alternatives at the west end and east end, respectively, of the project area.

A. West Critical Shoreline

Four alternatives are considered for the west end of the project area. In all cases the existing 118 ft rock groin is included unless stated otherwise. The west end alternatives include beach fill only, beach fill with 50% of the existing groin removed, beach fill with 100% of the existing groin removed, and beach fill with a set of 3 groins tapering in length toward the west.

1. Beach fill only

The beach fill that is included in all of the alternatives consists of a 100 ft wide berm from station -5+00 to station 61+00 and a 50 ft wide berm from station 61+00 to station 124+00. From station 0+00 to station 9+00 and from station 61+00 to station 124+00, the beach is backed by a bulkhead. The beach fill is tapered from the 100 ft berm width to zero from station -5+00 to station -8+00. From station 9+00 to station 61+00, a dune is located on the landward 50 feet of the 100 ft wide berm. The design profiles include a stepped berm elevation; an average berm elevation of 10 ft is used in the model. An existing 118 ft long groin is located approximately at project station 9+00.

The model results for this alternative are presented in **Figure 19**. With the existing groin and the design beach fill, the net transport over the ten-year simulation causes sand to bypass the groin and the added fill supplies the area to the west even beyond the limits of the fill taper at station -8+00.

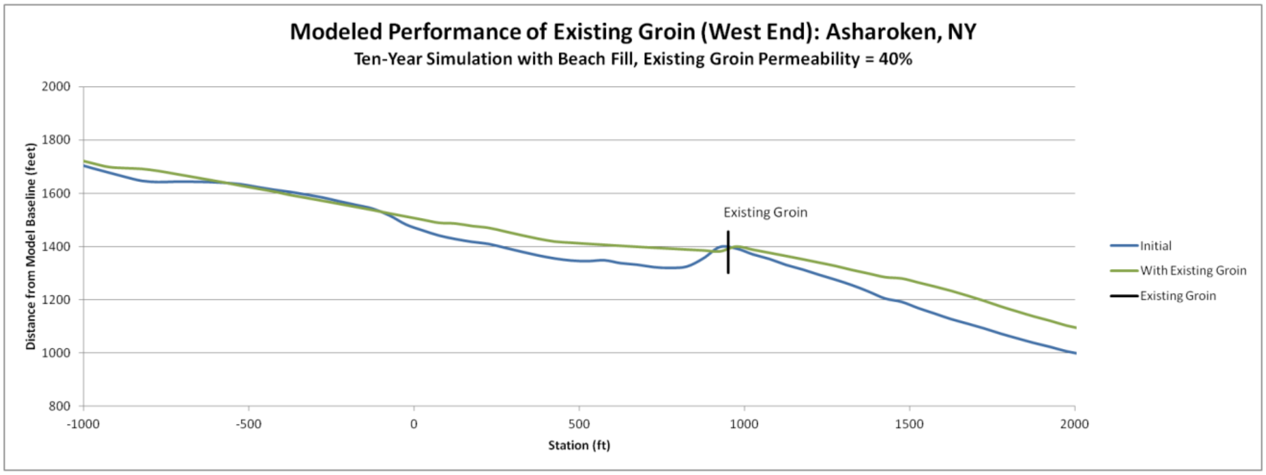


Figure 19. Ten-Year Change in Shoreline at West End of Asharoken With Beach Fill and Existing Groin

2. Beachfill with existing groin shortened by 50%

The beach fill described in the first west end alternative is again considered but with the existing 118 ft long groin (located at the approximate project station 9+00) shortened to a length of 59 ft. The results are presented in **Figure 20**. As in the first alternative, the beach fill bypasses the existing groin and the associated bulge in the shoreline, tending to smooth out the shoreline alignment while supplying sediment several hundred feet to the west.

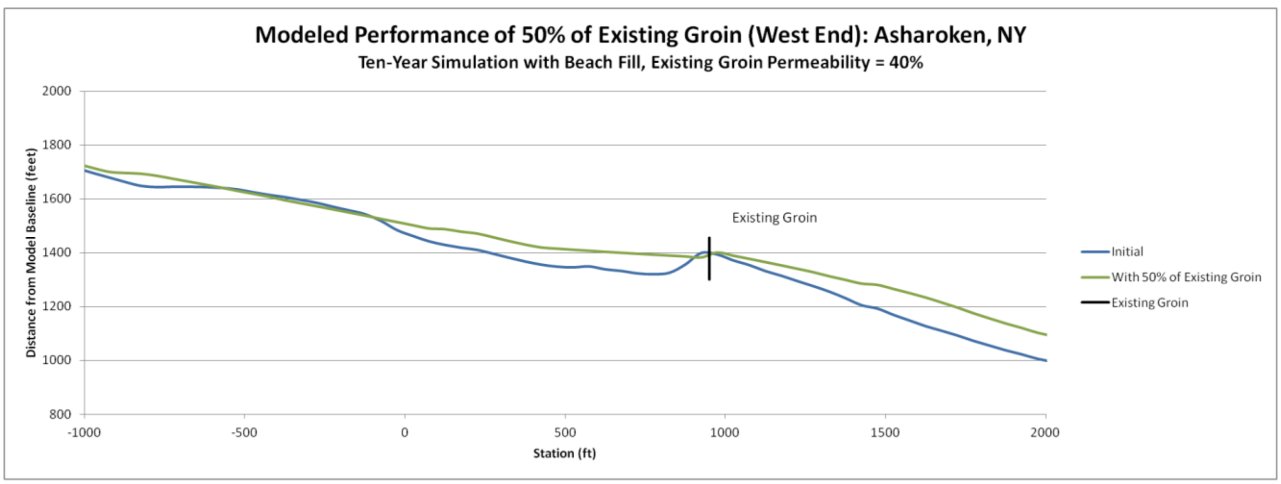


Figure 20. Ten-Year Change in Shoreline at West End of Asharoken With Beach Fill and 50% of Existing Groin Removed

3. Beach fill with existing groin removed

The beach fill described in the first west end alternative is again considered but with100% of the existing groin (located at the approximate project station 9+00) removed. The results, depicted in **Figure 21**, show similar trends to the full-length and 50% length cases; however, there is slightly more bypassing of sand to the downdrift (western) shoreline with associated erosion updrift of the former groin.

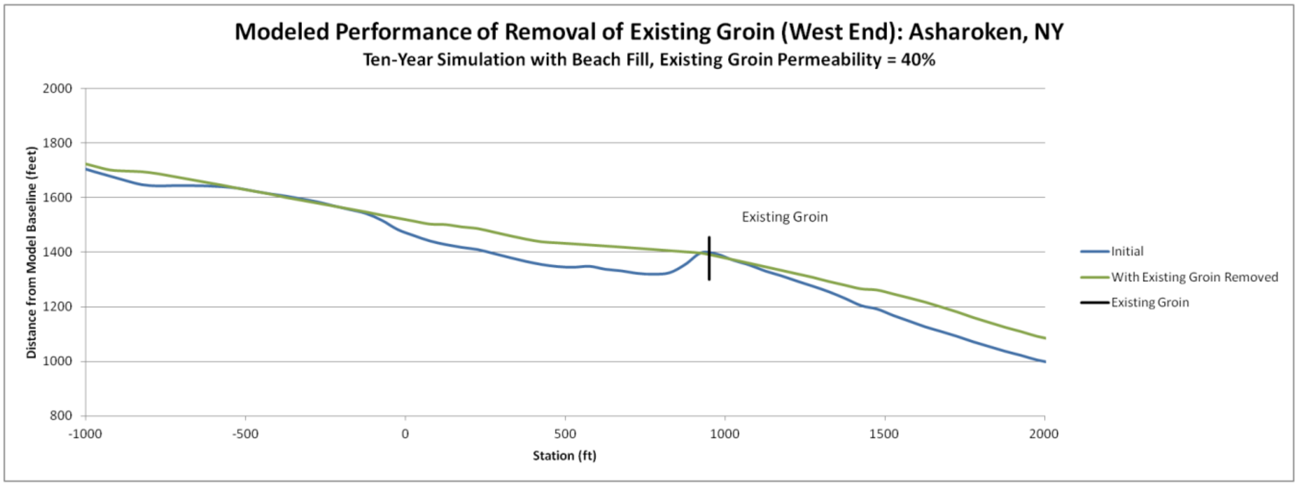


Figure 21. Ten-Year Change in Shoreline at West End of Asharoken With Beach Fill and 100% of Existing Groin Removed

4. Beachfill with proposed groin field (3 groins)

The beach fill described in the first west end alternative is again considered but with three groins added. An 80 ft long groin is added at station -5+00, a 100 ft long groin is added at station 0+00, and a 120 ft long groin is added at station 5+00.

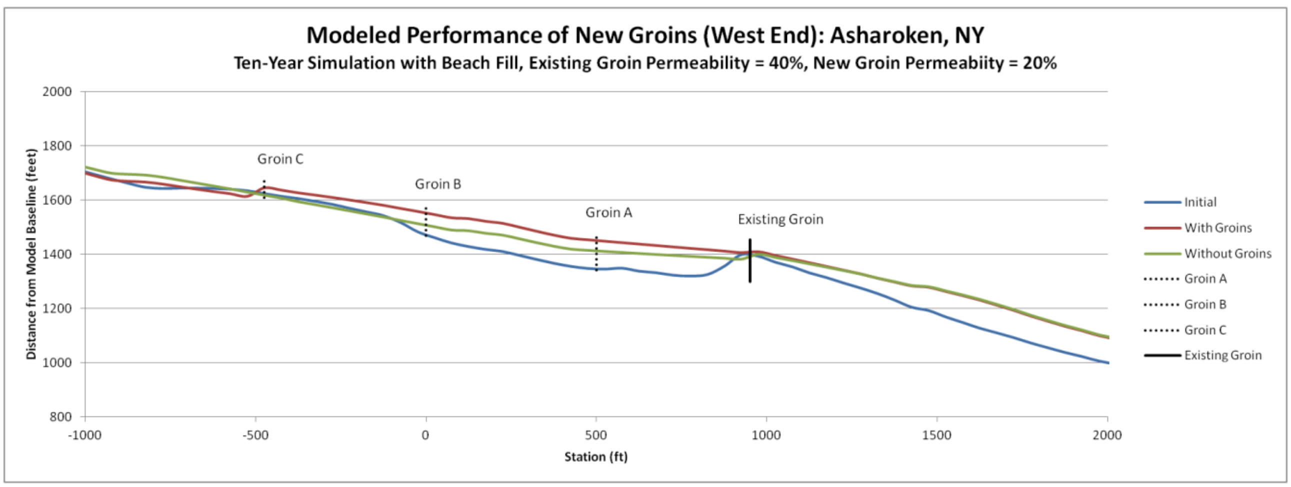


Figure 22. Ten-Year Change in Shoreline at West End of Asharoken With Beach fill and Tapered Groin Field

5. Comparison of West End Alternatives

Three alternatives were examined for treatment of the existing stone groin at approximately station 9+00: leaving it as-is, removing 50% of its length, and removing it entirely. In all the simulations, the design beach fill was included in the ten-year simulation of shoreline change with the groin alternatives. **Figure 23** presents a comparison of the effects of the three alternatives in terms of shoreline change from the pre-beach fill position.

The figure shows that there is no difference in sand retention or bypassing between the full-length groin and 50% of the groin left in place. In both cases, there is similar beach recession downdrift (west) of the groin over the ten-year period after the fill is placed; but the model shows stability of the filled shoreline west of the groin to about station 0+00 and very little net loss west of that point beyond the beach fill placed at the beginning of the simulation.

Updrift (east) of the groin, again there is no difference between results of the existing groin and 50% removal. However, for the entire removal, the updrift beach recession is about 2 ft/yr greater than the full length groin and the 50% removal cases.

Figure 23. Comparison of Shoreline Changes at West End of Asharoken For 50% and 100% Removal of Existing Groin

A similar comparison of shoreline change relative to the initial pre-beach fill shoreline is shown in **Figure 24** for the tapered three-groin field (and existing groin) and for the existing groin. Both alternatives include beach fill at the outset of the 10-year simulation. The figure indicates that the tapered groin field retains the beach fill throughout the area west of the existing groin. West of the existing groin, there is little effect of the three groins as compared to the effect of the existing groin alone.

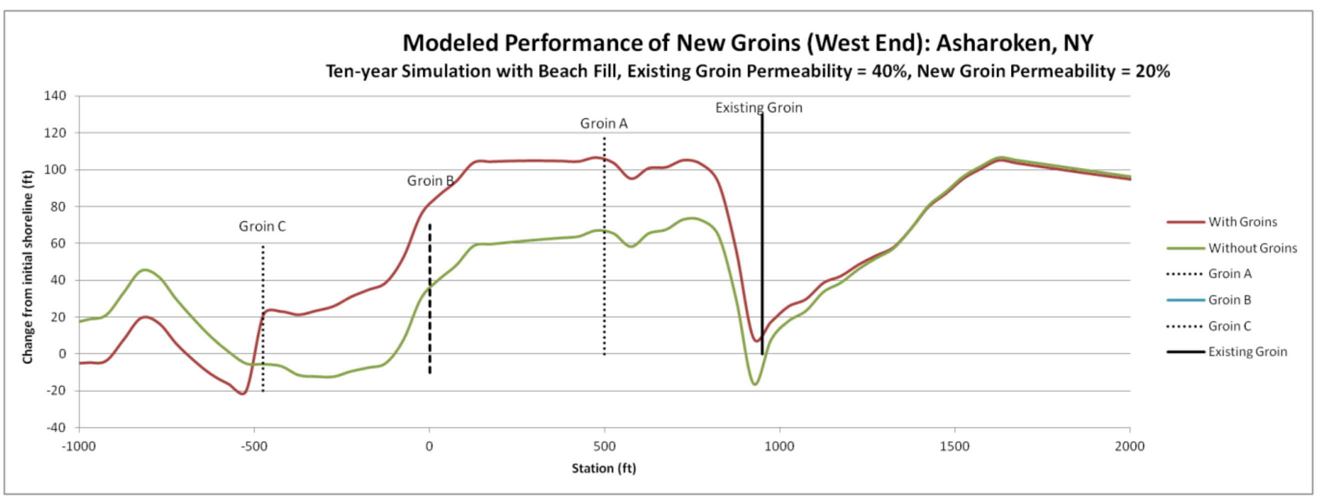


Figure 24. Comparison of Shoreline Changes at West End of Asharoken for Beach Fill Only (with Existing Groin) and Tapered Groin Field

B. Modeling of **East Critical Shoreline**

Four alternatives are considered for the east end of the project area. The east end alternatives include beach fill only, beach fill plus two groins added, beach fill plus two offshore breakwaters, and beach fill with two groins and one offshore breakwater.

1. Beachfill only

This alternative is the same as the first alternative described for the west end of the project area. The beach fill that is included in all of the alternatives consists of a 100 ft wide berm from station -5+00 to station 61+00 and a 50 ft wide berm from station 61+00 to station 124+00. From station 0+00 to station 9+00 and from station 61+00 to station 124+00, the beach is backed by a bulkhead. The beach fill is tapered from the 100 ft berm width to zero from station -5+00 to station -8+00. From station 9+00 to station 61+00, a dune is located on the landward 50 feet of the 100 ft wide berm. The design profiles include a stepped berm elevation; an average berm elevation of 10 ft is used in the model.

2. Two groins

The beach fill described in the first east end alternative is again considered but with two groins added, each 100 ft long from the existing shoreline, at station 88+00 and station 112+00. **Figure 25** illustrates the effect of the two groins and indicates that the structures are effective in stabilizing the beach area to approximately 1200 feet east of each structure. There is some beach recession on the east (downdrift) side of each structure relative to the fill-only alternative.

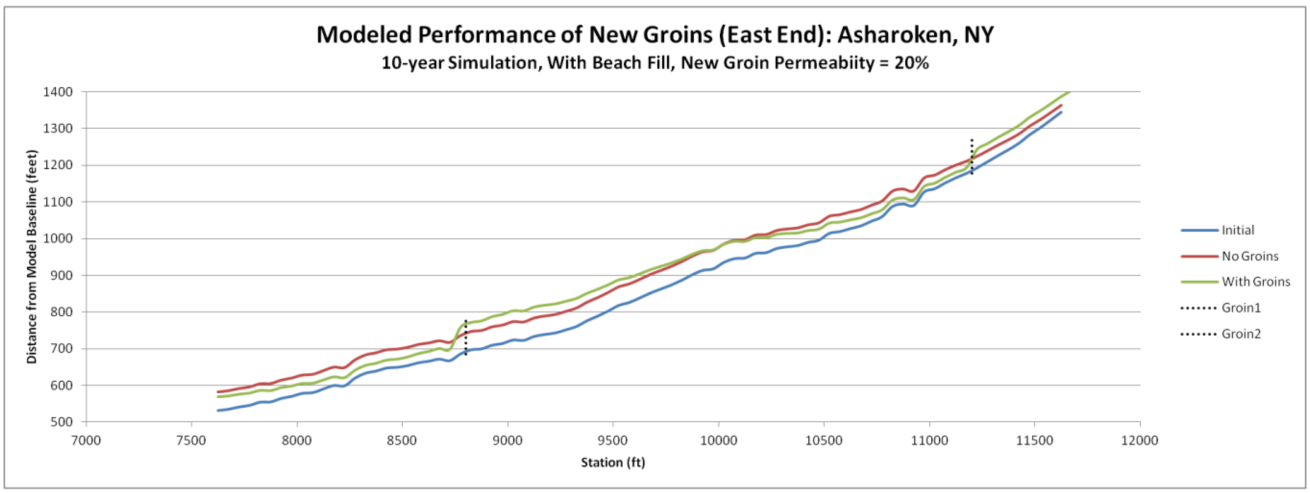


Figure 25. Ten-Year Shoreline Change Due to Fill Only and Two 100' Groins at East End of Asharoken, NY

3. Two breakwaters, 300 ft long, 150 ft offshore

The beach fill described in the first east end alternative is again considered but with two offshore breakwaters added, each 300 ft long and 150 ft offshore, and centered at stations 88+00 and 112+00. This alternative performs in a similar manner to the east end alternative 2, as shown in **Figure 26**, with an increase in stabilization of the beach fill in the lee and to the east of each structure but with downdrift recession to the west of each structure.

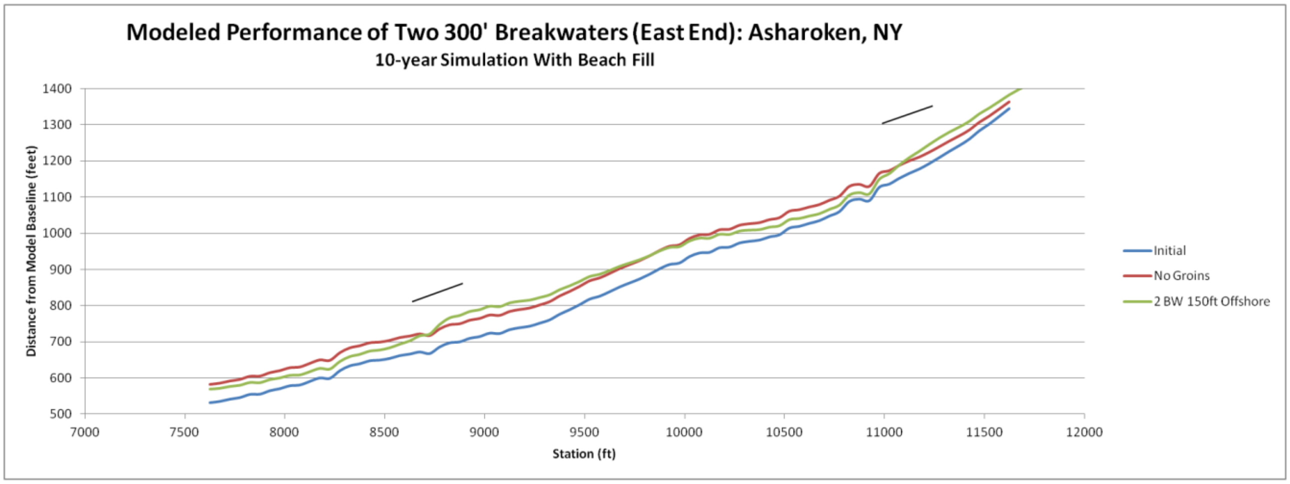


Figure 26. Ten-Year Shoreline Change Due to Fill Only and Two Offshore Breakwaters 300 ft at East End of Asharoken, NY

4. Two groins (as in alt 2) and one 300 ft breakwater at midpoint 150 ft offshore

The beach fill described in the first east end alternative is again considered but with two groins and an offshore breakwater added (**Figure 27**). Each groin is 100 ft long from the existing shoreline, at station 88+00 and station 112+00. The breakwater is 300 ft long, positioned parallel to the existing shoreline 150 ft offshore and centered between the two groins. The offshore breakwater eliminates most of the downdrift recession between the groins that is observed in the east end alternatives 2 and 3; however, the downdrift recession associated with the westernmost groin is more pronounced.

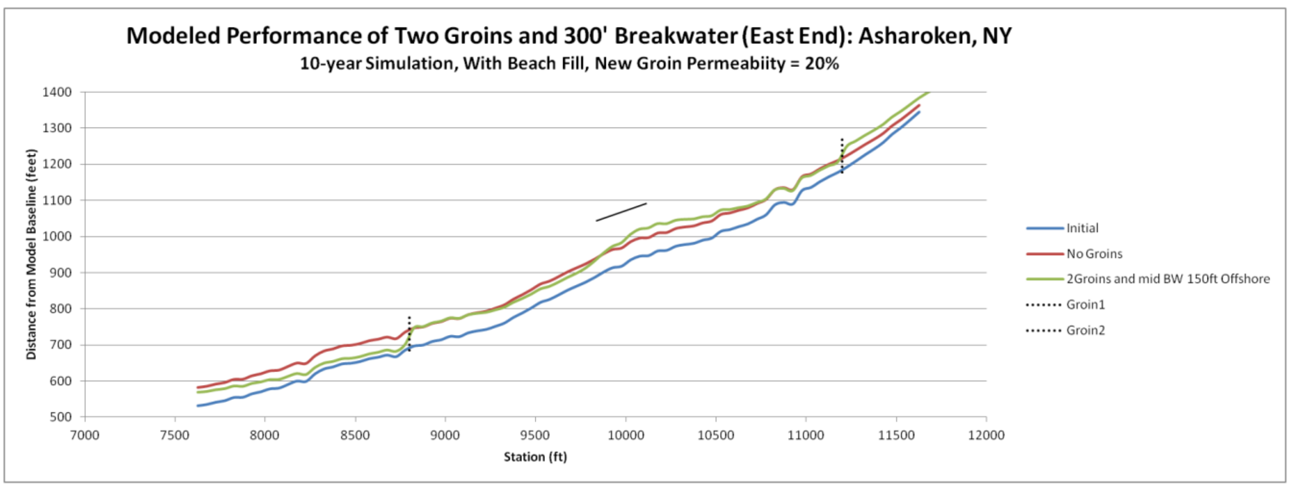


Figure 27. Ten-Year Shoreline Change Due to Fill Only and Two Groins with 300' Breakwater at East End of Asharoken, NY

5. Two groins (as in alt 2) and third at midpoint

The beach fill described in the first east end alternative is again considered but three groins added (**Figure 28**). Each groin is 100 ft long from the existing shoreline, at station 87+50, station 100+00 and station 112+50. The three groins, represented with 50% permeability that is approximately equivalent to a low crested design, stabilize the shoreline response throughout the groin field while lessening the downdrift recession west of the westernmost groin.

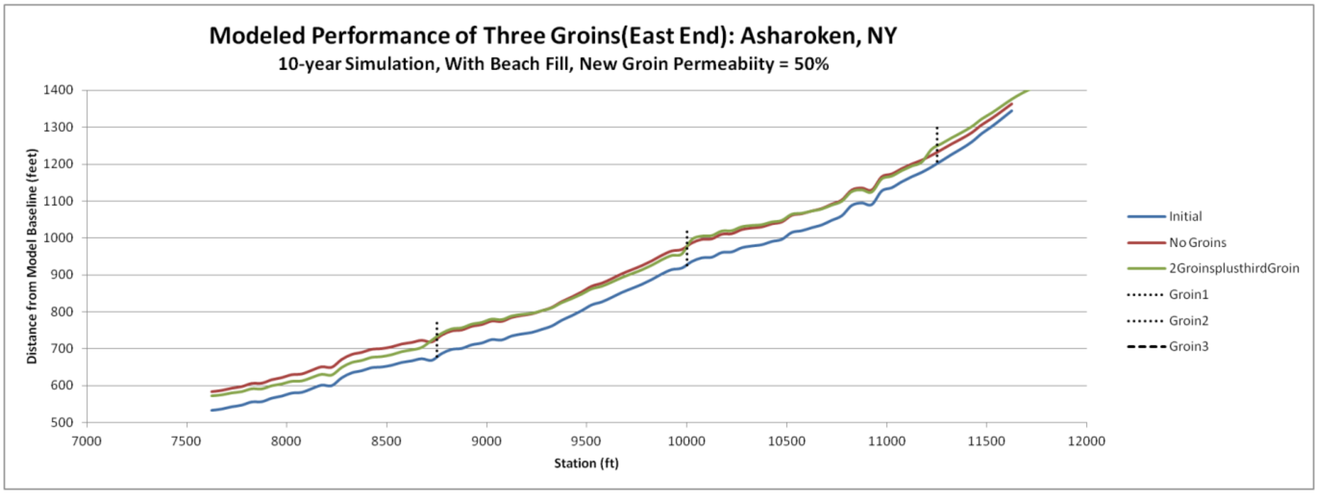


Figure 28. Ten-Year Shoreline Change Due to Fill Only and Two Groins with 300' Breakwater at East End of Asharoken, NY

Figure 29 presents a comparison of the five east end alternatives. The figure compares the difference of the shoreline at the end of the 10-year simulations with the initial shoreline position (pre fill). The two-groin alternative and the two-breakwater alternative are very similar in performance, with similar updrift (east side) accretion and downdrift (west side) erosion relative to the fill-only (no groin) alternative. When an offshore breakwater is added in between the two groins, virtually the entire area between the groins and updrift of the easternmost groin is stabilized; however, the recession on the west side of the westernmost groin is slightly more pronounced. By using a third groin (the fifth alternative) rather than an offshore breakwater, the amount of downdrift recession is lessened at the west end of the groin field and the variations in beach width within the groin field are lessened as well.

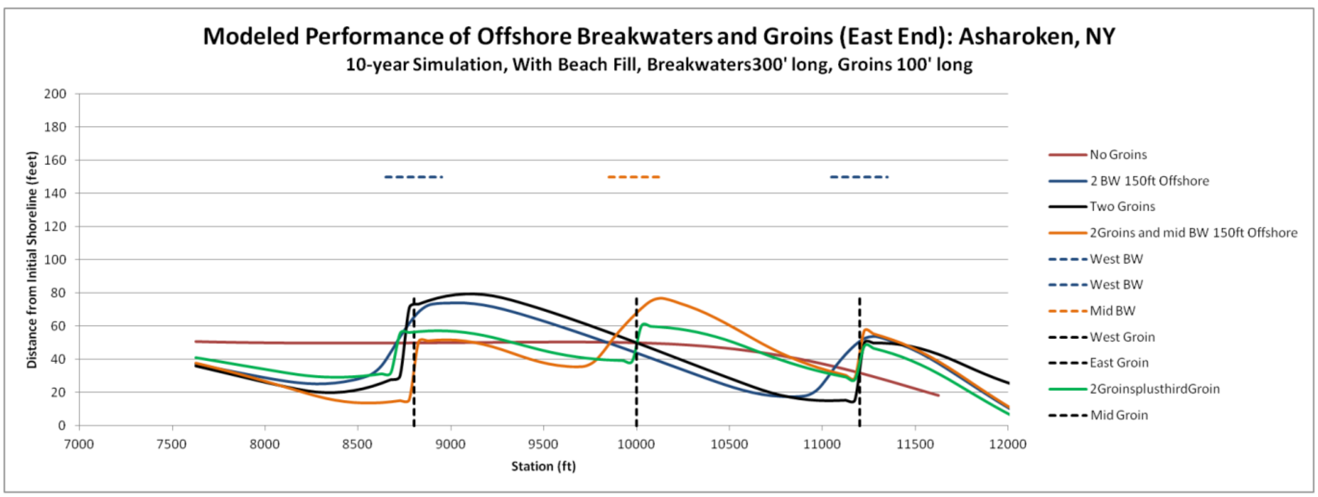


Figure 29. Comparison of the Ten-Year Shoreline Change Due to Five East End Alternatives.

6. Eight to twelve groin field

Three additional alternatives are considered for the east end of the project area. The east end alternatives include beach fill, two taper groins 100 ft long at the western end of the treatment area, plus a number of 150 ft long groins as described below.

Beachfill

The beach fill that is included in all of the alternatives described below consists of a 100 ft wide berm from station -5+00 to station 61+00 and a 50 ft wide berm from station 61+00 to station 124+00. From station 0+00 to station 9+00 and from station 61+00 to station 124+00, the beach is backed by a bulkhead. The beach fill is tapered from the 100 ft berm width to zero from station -5+00 to station -8+00. From station 9+00 to station 61+00, a dune is located on the landward 50 feet of the 100 ft wide berm. The design profiles include a stepped berm elevation; an average berm elevation of 10 ft is used in the model.

Additionally, 45,000 cubic yards of sand is added over a 5,000 ft length of east end beach from station 74+00 to station 124+00 every 3 years. This is represented by an added berm width of 9 feet over that area every 3 years.

a. Six groins plus taper

Two 100 ft long groins are placed at station 60+00 and 67+00. Six 150 ft long groins are spaced at 100 ft intervals, placed at station 74+00, 84+00, and 124+00.

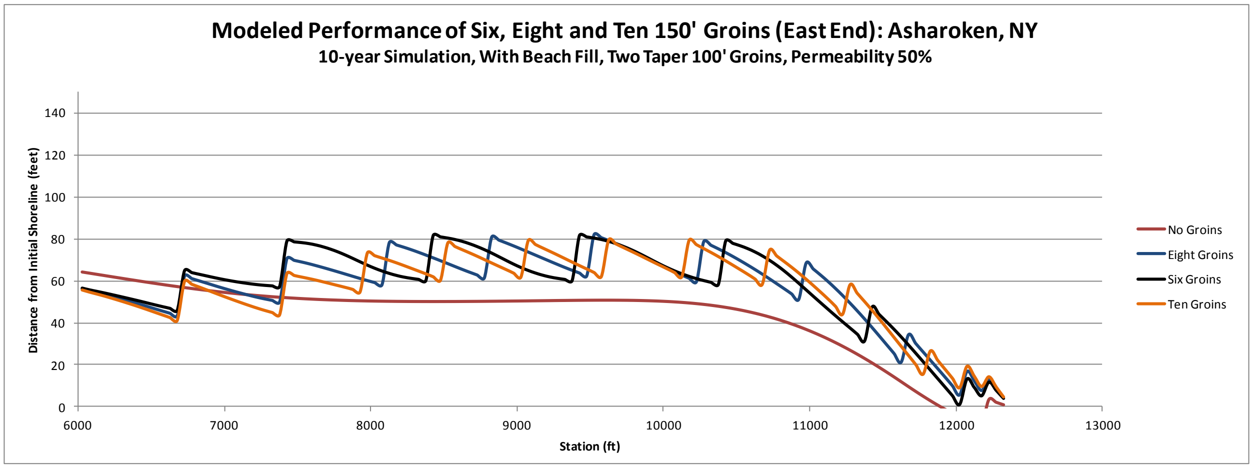
b. Eight groins plus taper

Two 100 ft long groins are placed at station 60+00 and 67+00. Eight 150 ft long groins are spaced at 714 ft intervals, placed at station 74+00, 81+14, and 124+00.

c. Ten groins plus taper

Two 100 ft long groins are placed at station 60+00 and 67+00. Ten 150 ft long groins are spaced at 555 ft intervals, placed at station 74+00, 79+55, and 124+00.

The modeled performance of the three alternatives is **depicted in the figure below.**



8 Volume Changes

Volume changes due to each alternative are calculated by taking the difference between the final shoreline position of the 10-year simulation and the initial shoreline position with the design beach fill. The shoreline change in feet is assumed to equate to one cubic yard of volume change. The critical erosion shoreline located at the western and eastern end of the project was analyzed separately in order to more easily assess the impact of each alternative as follows:

- West Critical Shoreline: Station 0+00 to 10+00, approximately 1,000 ft shore distance fronting the steel sheetpile bulkhead seawall;
- East Critical Shoreline : Station 87+50 to 112+50, approximately 2,500 ft shore distance fronting the timber bulkhead retaining wall;

Note that the reference stations marking the critical shoreline boundaries are based on the preliminary layout plan.

For west shoreline model simulation, the results of shoreline position and equivalent volume changes after 10-year of initial fill for the three alternatives are summarized in Table 1. As shown in the table, removal of the existing groin would provide limited improvement of updrift sediment supply, however, would de-stabilize the established dune and beach system. The model indicates that the groin field would retain almost 96% of design width and reduce the annual re-nourishment rate by approximately 4,000 **cy**/year. Approximately 500 ft of downdrift shoreline will be affected by the groin field. The affected littoral deficit can be mitigated with tapered beachfill and stockpile of feeder beach downdrift of the groin field.

For east shoreline model simulation, the results of shoreline position and equivalent volume changes after 10-year of initial fill for the alternatives are summarized in Table 2. Note that the initial shoreline widths include a 50 ft advance fill. As shown in the table, all structural alternatives provide minor renourishment reductions except Alternative 4. The modeling results indicate six groins plus two tapers provide the best cost-effective groin field layout.

9.Modeling of the Effect of Offshore Hole

Additional model simulation of the effect of offshore hole located north of the eastern critical shoreline (figure 5) was performed to determine the response of shoreline changes assuming the hole was filled with sand. Both the without and with beachfill shoreline conditions were simulated. The model results indicated a very limited changes of shoreline responses with or without the offshore hole. The reason for the limited changes is that the entire offshore bathymetry has been adjusted since the creation of the offshore hole in the 1960’s. Therefore, it would take a similar long period of time (on the order of 50 years) to re-adjust the offshore bathymentry back to normal condition.

Table 1 GENESIS Simulation Results for West Critical Area

West Shoreline Alternatives, Station 0+00 to 10+00, 1,000 ft Shoreline Length				
	Beachfill Only Baseline Condition	Alternative 1 Remove 50% Existing Groin	Alternative 2 Remove Existing Groin	Alternative 3 Construct Tapered 3-Groin Field
Design Beach Width (ft)	100	100	100	100
Beach Width after 10 years	56	56	71	96
10-year Nourishment Volume (CY)	44,000	44,000	29,000	4,000
10-year Nourishment Savings (CY)	-	0	15,000	40,000
Advantages	-	none	Continuous Littoral Transport	Retain 95% Shoreline Width
Disadvantages	-	Will De-stabilize Updrift Shoreline	Will De-stabilize Updrift Shoreline	Need Mitigate Downdrift Erosion

Table 2 GENESIS Simulation Results for East Critical Area

East Shoreline Alternatives, Station 87+50 to 112+50, 2,500 ft Shoreline Length					
	Beachfill Only Baseline Condition	Alternative 1 Two Terminal Groins or Breakwaters	Alternative 2 Two Terminal Groins and One Breakwater	Alternative 3 Three Low Crest, Short l Groins	Alternative 4 Eight Low Crest, Short Groins
Design Beach Width (ft)	100	100	100	100	100
Beach Width after 10 years	47	49	49.5	49	70
10-year Nourishment Volume (CY)	132,750	127,250	126,250	127,750	75,000
10-year Nourishment Savings (CY)	0	5,500	6,500	5,000	60,000
Advantages	-	Retain Design Beach Updrift of Structure	Retain Design Beach Updrift of Structure	Retains Design Berm Width	Retains Design Berm Width
Disadvantages	-	May Cause Downdrift Erosion	May Cause Downdrift Erosion	-	High Initial Cost

Notes:

5. **Beach width changes are based on model results, East critical shoreline includes advance fill;**
6. **Volume estimate is based on the assumption 1 ft shore change = 1 cubic yard volume change;**
7. **Required nourishment = volume necessary to restore to design berm width.**

9 References

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