# APPENDIX B
# PHYSICAL CONDITIONS

## Physical Conditions

A general description of the coastal processes that characterize the study area and provide the basis for design and evaluation of storm protection measures are provided in this section.

### Winds

Predominant wind directions in the Study Area are from the southwest, west and northwest. Although winds from the southeast are not predominate (less than 25% of all wind occurrences), due to the study area shoreline orientation (159 degrees from north) and the almost unlimited fetch distances to generate waves, they do have a marked influence on the study area coastal processes. Wind speeds are typically less than 27 knots, accounting for approximately 95 percent of all observations. The dominant wind speed range is from 7 to 16 knots, which occurs nearly 49 percent of the time. Wind speeds exceeding 27 knots (strong breeze) are less frequent with a total occurrence of approximately 5 percent (USACE-NAN, DRAFT, 2000).

### Astronomical Tides

Astronomical tides on the south shore of Long Island are semi-diurnal, rising and falling twice daily. The tidal range along the ocean shoreline increases from east to west. For example, the average range in the vicinity of Montauk Point is approximately 2 feet (most easterly); while near Fire Island Inlet the range is over 4 feet (most westerly).

### Storm Surge

Storm surge is water that is pushed toward the shore by the force of the winds and the decrease in astronomical area pressure during major storms. Water levels rise at the shoreline when the motion of wind driven waters is arrested by the coastal landmass.

Two types of storms are of primary significance along the south shore of Long Island: (1) tropical storms which typically impact the New York area from July to October, and (2) extratropical storms which are primarily winter storms occurring from October to March. These storms are often referred to as “nor’easters” due to the predominate direction from which the winds originate.

Hurricanes are the most powerful tropical storms to reach the study area with wind speeds in excess of 74 mph (by NOAA definition). Records indicate 26 hurricanes have impacted the study area in the past century. Nor’easters are less intense than hurricanes, with sustained wind speeds generally less than 57 mph. However, the durations of elevated water levels and waves during nor’easters are generally longer, enhancing the ability of nor’easters to cause coastal damage. Approximately 68 moderate to severe nor’easters have impacted the New York coastal region since 1865.

As a part of the Reformulation Study, the USACE has undertaken a comprehensive evaluation of storm-induced water levels for FIMP. Estimates of storm surge were made using a combination of models including the finite element hydrodynamic model ADCIRC (Advanced CIRCulation model) (Scheffner et al., 1994), the beach, berm, and dune erosion model SBEACH (Larson and Kraus ,1989), and the waves, currents, sediment transport, and morphological evolution modeling system Delft3D (WL | Delft Hydraulics, 2004). The models allow for the simulation of dune erosion, barrier island inundation, and breaching in order to identify the water surface elevations for different storm events and barrier island configurations. The output of these models led to the development of storm surge stage-frequency curves that identify the probability of these storm-induced water elevations occurring throughout the study area. The impact of open breaches on storm stages was quantified with the hydrodynamic model described above. Modeling results indicate that open breaches result in measurable changes in storm water levels and cause relative increase in the stage frequency curves of 6 to 18 inches.

Sea Level Rise

By definition, sea level rise (SLR) is an increase in the mean level of the ocean. Eustatic sea level rise is a change in global average sea level brought about by an alteration to the volume of the world's oceans. Relative sea level rise takes into consideration the eustatic increases in sea level as well as local land movements of subsidence or lifting. The historic sea level rise rate is approximately 0.0126 feet/year or about 1.3 feet/century. There are various projections of accelerated sea level rise, from 2.6 feet/century up to almost 5.4 feet/century. A significant increase in relative sea level could result in extensive shoreline erosion and inundation. Higher relative sea level elevates flood levels, and as a result, smaller, more frequent storms could result in flooding equivalent to larger less frequent storms. The more frequent flood events on top of higher sea level may affect more property, resulting in greater damages as sea level increases.

The current guidance (EC 1165-2-212) from the USACE states that proposed alternatives should be formulated and evaluated for a range of possible future eustatic rates of SLR. Three possible eustatic SLR rates, low, intermediate, and high, are provided in the guidance. These rates of rise correspond to 0.7 ft, 1.3 ft, and 2.7 ft over the 50 year period of analysis for the low, medium and high rates of relative sea level rise.

Currents

The rise and fall of tides is accompanied by the horizontal movement of the water called tidal current. When these waters are channeled through narrow passages such as inlets, the currents can become quite strong, first in one direction as the tide comes in (the flood) and then reversing as the tide falls (the ebb). For the inlets in the study area, the flood and ebb tidal currents generate ebb and flood shoals, which impact sediment bypassing across an inlet (see Inlets, below).

Waves

Waves are the dominant forcing mechanism for most coastal processes along the south shore of Long Island. During storm events, wave impact on beaches that cause erosion of the beach are combined with the increased water level from wave setup, which can lead to overwashing or breach formation. In the study area, significant wave heights, exceeding 3.3 feet occur approximately 25 to 30 percent of the time (USACE-NAN, DRAFT, 2000). Significant wave heights during extreme storm events may exceed 18 feet. The predominant wave period (time between successive wave crests) is between 5 and 9 seconds, which accounts for more than 60 percent of all waves.

Beach Characteristics

Along the study area the grain size distribution of the beach material varies. Typically, grain size increases from west to east, with mean grain size ranging from 0.39 mm to 0.52 mm.

## Offshore Sediment Characteristics

The inner continental shelf south and offshore of the study area is characterized by ridge and swale morphology. Surficial sediments are predominantly fine to medium grained sands. Fine-grained sediment outcrops exist in isolated areas of the inner shelf and shoreface. The geology of this area is complex and is characterized by Holocene sediments of variable thickness. These sediments generally consist of either organic-rich muds (backbarrier deposits typically found in the sheltered waters leeward of a barrier island) or modern marine and inlet-filling sands. The area west of Moriches Inlet is typified by a seaward-sloping wedge-shaped deposit of backbarrier sediments underlying marine sand. The maximum thickness of these Holocene sediments is 10 feet along the western portion of Fire Island thinning towards Moriches Inlet.

Since the 1960’s, efforts have been undertaken in the study area to identify locations offshore which contain sediment (sand) that would be a suitable source for beach nourishment. This includes considerations for compatibility to native beach grain size, the amount of volume available, environmental considerations, and distance to the project site. Twelve potential offshore sites and seven potential upland source sites were identified as possible sources for the beach nourishment measures. The specific results of the borrow area investigations for design purposes are included in the Borrow Area appendix.

## Shoreline Changes

Historic Shoreline Rate-of-Change (SRC) values in the FIMP study are documented in Gravens et al. (1999), which examined three non-overlapping time intervals using available shoreline data sets. The first period, representative of the epoch prior to significant human influence on the barriers, is 63 years long (1870 to 1933). The second period, representative of initial development on the barriers and the initiation of human intervention with natural processes including inlet stabilization and significant beach fill placements, is approximately 46 years long (1933 to 1979). The third period, representative of modern times and reflecting the most recent beach nourishment practices, is approximately 15 years long (1979 to 1995).

The Fire Island barrier has, in general, been eroding at a historically consistent rate of about 0.4 m/year (1.3 feet/year). Average shoreline recession has increased to 0.7 m/year (2.3 feet/year) over the most recent 15-year time interval on Fire Island. It is important to note that these SRC values are average values for the entire 30-mile barrier island and that the standard deviation in the SRC is between 3 and 4 times larger than the mean. The comparatively large SRC standard deviation indicates significant variation in the shoreline change signal along Fire Island.

At least part of the alongshore variability in the observed shoreline rate-of-change owes to undulating shoreline features that are locally referred to as longshore sand waves or erosion waves (Gravens et al., 1999). The impact of shoreline undulations on a typical beach fill design configuration was shown to be significant and could lead to greater than anticipated maintenance costs or a reduced level of protection at areas of erosional cusps. Explicit consideration of the presence of shoreline undulations in the development of alternative design configurations and the assessment of baseline and future without project conditions is essential for a successful project.

## Inlets

As presented previously, there are two inlets in the Project Area: Fire Island Inlet and Moriches Inlet, both of which are Federal navigation projects. Moriches and Fire Island inlets also increase the tidal prism and amplitude within the bays because the navigation channels are larger and more efficient than the unstructured tidal exchange. Both inlets allow the exchange of water, sediments, nutrients, planktonic organisms, and pollutants. They are exchanged between the open sea and the protected back-bays behind the barriers. The inlets play an important role in the regional sediment budget by either trapping sediment within its ebb and flood tidal shoals or bypassing sediment downdrift. Mature inlets with well-developed ebb and flood shoals are generally more efficient at bypassing sediment. The stabilization / jetties of the inlets act to confine flows within a relatively narrow area compared to natural inlets; they also act to deepen the inlet throat and shift the ebb tidal delta further offshore than a natural inlet. Accordingly, the inlets have acted to trap sand.

## Existing Shore Protection Activities

In response to the storm history described in Chapter 2, a number of construction measures have been implemented within the FIMI project area to mitigate storm impacts. These include measures which have been implemented either as other Federal initiatives, State actions, or undertaken by local municipalities, taxing districts, or by individual homeowner. Collectively, these actions have had a dramatic influence on the functioning of the existing coastal system.
The following section provides a description of the major coastal engineering actions which have been undertaken in the project area, which shape the current conditions. This section covers the major constructed elements along Fire Island, but does not try to capture all of the local projects that have been constructed, or all of the activities that have taken place along the back-bay areas. However, please note that stabilization of the barrier island also provides coastal risk management to the back-bay areas by minimizing overwash and breaches.

*Inundation Damages.* These occur when vulnerable structures are flooded by high tides and storm surges in the back-bay, where the water levels are sensitive to the conditions of the barrier islands. Inundation damages occur on the back-bay mainland and on the back-bay side of the barrier islands.
*Breach - Inundation.* Breach inundation damages occur when structures are flooded by increases in back-bay water elevations caused by breaches in the barrier islands remaining open for a period of time. These damages are limited to structures in back-bay mainland areas and on the back-bay side of the barrier islands.

## Ocean Beach Groins

Two shore perpendicular structures were constructed in the winter of 1970 within the Village of Ocean Beach, on Fire Island. Both groins are 200 feet long from landward crest to seaward crest, with the offshore portion about 85 feet of the total length. The groins were constructed in an area of higher erosion, to add stability to the ocean shoreline seaward of the Ocean Beach water tower and pumping stations (wells). Since this time, the water tower has been moved north in the Village, on Village owned land, however the three wells remain just landward of the eastern groin, within three village owned facilities. A separate Village maintenance facility is also located in the same Village property containing the wells.

## Smith Point County Park Bulkhead

Following the storms of the early and mid-1990’s Suffolk County constructed a steel sheetpile bulkhead fronting the existing pavilion at Smith Point County Park. In the mid-1990’s conditions were such that the pavilion and its infrastructure were at risk to future damage. The revetment was constructed in conjunction with a small beachfill project, to protect the bulkhead. Following construction of the revetment, a memorial for TWA Flight 800 (which crashed in the Atlantic Ocean off of Moriches Inlet in July 1996) was constructed. The memorial was located outside the alongshore extent of the revetment, and in a location vulnerable to erosion. In 2005, Suffolk County extended the revetment to provide protection inclusive of the memorial.

### Beachfill

Following the hurricane of 1938, there is a consistent record of beachfill activities undertaken in response to storm events. A large percentage of historical beachfill volumes have been placed adjacent to Fire Island and Moriches Inlet as a byproduct of inlet dredging. Following the 1962 nor'easter, the USACE contracted the placement of 9,529 linear feet of dune and 37,000 linear feet of berm along Fire Island as part of the Disaster Recovery Operation (USACE, 1963). Beachfill projects were also undertaken by local communities at Point of Woods, Cherry Grove and Ocean Beach following 1962. It is estimated that a total of 6.9 million cubic yards of beachfill was placed along Fire Island from 1933-1989 (Gravens et al, 1999).

Since 1990 beachfill has been performed by the USACE adjacent to the inlets as a byproduct of inlet maintenance dredging, and by the local communities in response to storm events. In response to the storms in the 1990's local communities placed approximately 1 million cubic yards of beachfill (CPE, 2013). In 1997 an additional 650,000 cubic yards of beachfill was placed by the communities in Fire Island Pines.

Two major beachfill projects were undertaken by local communities along Fire Island between 2000 and 2009. In 2003-2004 several communities in Fire Island placed approximately 1.28 million cubic yards of beachfill in Western Fire Island and Fire Island Pines, and in 2009 1.82 million cubic yards of sand was placed in eleven communities along Fire Island (CPE, 2013). In addition to these two major beachfill projects, 172,000 CY and 21,000 CY of sand were placed at Smith County Park and Davis Park respectively in 2007.

### Fire Island Inlet

Fire Island Inlet is located at the western end of Fire Island and connects the Atlantic Ocean with Great South Bay. Available records indicate that Fire Island Inlet has existed continuously since the early 1700's. The position of the inlet, however, has varied significantly over time and has migrated a total distance of about 5 miles from a point east of its present position between 1825 and 1940. Federal jetty construction at Democrat Point in 1941, as part of the Fire Island Inlet Navigation Project halted this westward migration. Due to chronic erosion on the western shore, modification of the Federal project was authorized in 1971 to provide for a sand bypassing system at Fire Island Inlet. Since this time, continued dredging of the inlet has been performed to both maintain a navigable channel, and to provide shore protection on the westerly, downdrift beaches and to protect the Ocean Parkway. Dredged material has also been back passed and placed in Robert Moses State Park to alleviate chronic erosion.

### Moriches Inlet

Moriches Inlet is located along the Atlantic Coast in the Town of Brookhaven and connects the Atlantic Ocean with Moriches Bay through the narrow barrier island. Available maps and records indicate that numerous inlets to Moriches Bay have existed during the last several centuries. The present Moriches Inlet was opened during a storm on 4 March 1931. The inlet migrated about 3,500 feet west from 1931 to 1947 at which time its migration was halted when non-federal interests constructed a long stone revetment on its western bank in an effort to stabilize the Inlet. During a storm on 15 May 1951 Moriches Inlet closed as a result of reduced hydraulic efficiency. Non-federal interests constructed jetties on both sides of the inlet from 1952 to 1953 and the inlet was reopened during construction by a storm on 18 September 1953.

In 1983, the USACE completed a General Design Memorandum for Moriches Inlet Navigation, which recommended Federal participation in inlet improvements including the following: (1) a 100-foot wide by 6-foot deep inner channel extending from the Intercoastal Waterway to Moriches Inlet, (2) an outer channel extending from the ocean to the inner channel with a width of 200 feet, a low water depth of 10 feet and an advanced maintenance deposition basin. Construction activities were completed by 1986, and since this time the inlet has been maintained as a Federal Navigation Channel.

### Sediment Budget

A sediment budget refers to the balance between sediment added to or removed from the coastal system, and is used to reflect the trends in alongshore sediment transport. Coastal erosion is a physical expression of a deficit in the sediment budget where nearshore processes remove more material from the shore than is added.

An existing (c. 2001) conditions sediment budget presenting estimates of volume changes and alongshore sediment transport rates within the FIMP study area were developed. The budget incorporates, to the extent possible, relevant long-term trends identified in previous studies as well as recent changes, including relatively new inlet and shoreline management practices at Shinnecock Inlet and the Westhampton Interim Project.

Overall, this budget shows that there is a gradient in the alongshore sediment transport rates from Montauk Point to Fire Island Inlet. Within the project area the sediment transport rate increases from approximately 275,000 cy/yr to 525,000 cy/yr from Smith Point County Park to Robert Moses State Park. Available surveys and assumptions regarding the effects of sea level rise on inlet morphology suggest that Moriches and Fire Island Inlet trap 33,000 and 141,000 cy/yr, respectively accounting for the volume of sand that is currently dredged and bypassed at each inlet for navigation. Therefore, the total loss to the system due to the inlets is 174,000 cy/yr. This value represents a significant percentage of the average alongshore sediment transport along the FIMP shoreline.

### Estuarial (Bayside) System Conditions

The project area estuarial system is comprised of Great South Bay and Moriches Bay and is connected to the Atlantic Ocean through Fire Island and Moriches Inlets respectively. The bays are also connected to each other through narrow tidal waterways of the Long Island Intracoastal Waterway (ICW). A summary of hydrodynamic and water quality conditions is presented in the following paragraphs.

### Hydrodynamics and Hydrology

Bay water levels are controlled by tidal elevations at Fire Island and Moriches Inlets. The uniformity of tide ranges throughout both Great South and Moriches Bays is a characteristic of the so-called “pumping mode” of inlet-bay hydraulics where water levels within an embayment remain nearly horizontal during ebb and flood tide phases. Bay tide amplitudes are generally less than ocean tides and lag the ocean tides. The difference between ocean and bay tides is particularly significant within eastern Great South Bay. The tidal range at the ocean end of Fire Island Inlet is approximately 4.3 feet, whereas the average tidal range in the bay is approximately 1 foot. The tidal range at the ocean side of Moriches Inlet is approximately 3.6 feet; the average tidal range within the bay is estimate to be 2 feet. Maximum current velocities occur near the inlet mouth, where values exceed 4 feet/second. Peak velocities in the bays away from the inlets are typically less than 1 feet/sec.

## Socio-Economic Conditions

The following details the development patterns and land use on Fire Island and the back-bay areas of Great South Bay and Moriches Bay.

Intensive human habitation was not documented on Fire Island until the second half of the 19th century. The establishment of permanent communities began shortly before the 20th century. The first of these, the Point O' Woods Association, began in 1898. Other communities quickly followed, although the youngest community, Dunewood, was formed in 1958. The number of buildings and the summer population began to grow. According to an analysis of aerial photographs, approximately 950 structures were found on Fire Island in 1928. This number grew slowly to 1,260 in 1955, and the number of structures had doubled to about 2,400 in 1962. The number of structures reached about 3,500 in the 1970's and now stands at approximately 4,150. All of the communities on Fire Island have greatly increased populations during the summer months from an influx of day visitors, short-term renters, and seasonal homeowners.

## Land Use and Management

Land use differs throughout the study area. The FIMI barrier island study area is generally more developed to the west in the communities of Saltaire, Ocean Beach, Cherry Grove and Fire Island Pines with no development in the middle, wilderness area. Smith Point County Park is located on the easternmost side of the FIMI project area, while Robert Moses State Park is located on the westernmost end of Fire Island. State coastal policies support protecting natural protective features, siting buildings and development in places that minimize risk, and avoiding actions that impair natural sediment processes. Additional Land Use and Management is included in Section 10.4 of this Report and Appendix H.

## Town of Babylon

The Town of Babylon includes communities on the mainland including the Villages of Amityville, Lindenhurst, and Babylon and the hamlets of Copiague and West Babylon. Land use in this area generally consists of medium-density detached homes, with high-density residential uses found close to the water's edge. There is very little agricultural use and more commercial/industrial use.

Commercial uses run along most of the length of Montauk Highway. The Babylon portion of the study area also includes several recreational and park uses which front the Great South Bay. In addition, the Town of Babylon includes part of Captree State Park on Captree Island and the easternmost tip of the Jones, Gilgo, and Oak beaches on the barrier island to the west of Fire Island Inlet.

Town of Islip

The study area within Islip is primarily residential, with some large open spaces (e.g., Great River and Connetquot River State Park on the bayshore) and commercial development concentrated along Montauk Highway. Communities in this area include West Bayshore, Bayshore, the Village of Brightwaters, Islip and East Islip, Great River, Oakdale, West Sayville, Sayville, and Bayport. Commercial development includes primarily small- to medium-sized shops and services, some of which are part of strip mall developments. Marine and marine-related commercial development is located near Great South Bay and its tributaries. There is no significant amount of industrial activity south of the Montauk Highway; industrial uses are located just outside of the study area, primarily along Union Boulevard.

Town of Brookhaven

With 260 square miles of land area, Brookhaven is the largest municipality on Long Island. Development in the municipality is generally less dense than Islip (with a notable exception being the area that includes Shirley and Mastic), with a number of undeveloped parcels. Communities in this area include Blue Point, the Village of Patchogue, Bellport, Brookhaven, Shirley, Mastic, Mastic Beach, Center Moriches, and East Moriches.

Within Brookhaven, retail commercial development is found along the Montauk Highway, especially in downtown Patchogue and in Shirley. Industrial uses, including maritime industry and boating, are found along the Patchogue River. There are also major open spaces and recreational amenities, including the Bellport Park Golf Course at South Country Road and South Howell’s Point Road, Smith Point County Marina near the Smith Point Bridge, and Wertheim National Wildlife Refuge, between Shirley and Brookhaven. There are also a number of smaller neighborhood parks and playgrounds.

Income

There is significant variation in the per capita and family income among study area towns as shown in Table 1 as follows. Per capita income in most of the study area is slightly above the state average. Median family incomes in the study area towns are all higher than the median family income for New York State.

**Table 1: Per Capita and Family Income**

| Location | Per Capita Income | Median Family Income |
|---|---|---|
| New York State | $31,796 | $69,202 |
| Suffolk County | $36,588 | $99,474 |
| Town of Babylon | $31,255 | $90,853 |
| Town of Islip | $31,493 | $92,482 |
| Town of Brookhaven | $34,201 | $97,520 |

*Source: American Community Survey 2007-2011 5-year Estimate*

## Economy

The largest segment of the study area population is employed in the education, health and social services sector. Retail trade, professional/management services and manufacturing also employ a large portion of the population. In the eastern end of the study area more people are employed in the agricultural field, while fewer are employed in the retail and manufacturing sectors.

## Transportation

The Robert Moses Causeway provides access over Great South Bay to Captree State Park and then over the Fire Island Inlet to Robert Moses State Park. The William Floyd Parkway (County Route 46) provides access over Narrow Bay to Smith Point County Park and the FINS Smith Point Visitor Center.

Private transportation is the predominant method of access to Fire Island, with approximately 5.1 million visitors (70 percent of total visitors) accessing the island by automobile. 3.8 million visitors travel to Robert Moses State Park annually and over 1.5 million visitors travel to Smith Point County Park on an annual basis. Private access is also provided by boat, water taxi, bicycle and seaplane. Ferries account for approximately 1.2 million visitors travel to Fire Island annually.

## On Island Circulation

The only vehicular traffic currently on Fire Island is at the western and eastern ends of the island. Vehicular access to Fire Island is allowed at Robert Moses State Park and Smith Point County Park; other areas on the island are vehicle accessible only by a special permit issued by the town. Due to the lack of roadway infrastructure and prohibition of cars, travel around the island is an access issue. While on the island, day visitors can venture to neighboring communities by water taxi or on foot. Vehicles without a special permit are prohibited in the Fire Island National Seashore.

Water taxis provide convenient lateral transportation between the communities. The sandy “Burma Road” provides a route for construction, utility, and pedestrian traffic between the communities. Segments of Burma Road are difficult for pedestrian transit because of the large distance separating several communities. In addition, the sandy composition of Burma Road makes bicycle use difficult.

*Problem Identification*

This section describes the shorefront and back bay conditions in greater detail to more effectively characterize the relative risk to storm damages that have been accounted for in the project modeling.

1. Topography. Extensive information is available to characterize the existing topography along the Atlantic Ocean shoreline and along the back bay area. The topography of the shorefront can be characterized by the dune conditions (dune height, width, and volume), and the beach conditions (beach berm height, width, and slope), The back bay environment is more characterized by the overall elevation within the floodplain.

The shorefront conditions along the study area are quite variable. Changes in the beach and dune are reflected in seasonal changes, storm induced changes, human induced changes, and changes that can occur due to shoreline undulations linked to very site specific variations in the nearshore conditions.

To account for this variability over time, the study considers a range of conditions, from a baseline condition to a future vulnerable condition. A September 2000 topographic survey was used to establish a baseline condition. This topography served as the basis for the various modeling efforts undertaken to characterize the response to storm events. This September 2000 condition, however, only represents one condition that could exist within the project area. In fact, the September 2000 conditions represent a beach which is relatively wide and a dune condition which is relatively high and wide. In order to characterize the storm response under a range of future conditions, another topographic condition was established, which is called a “future vulnerable condition”. The future vulnerable condition is a condition derived from past survey information and a projection of future trends. It is intended to represent a more vulnerable condition. The future vulnerable condition was developed to be similar in nature to conditions that existed in the mid-90’s, except that ongoing fill actions which are identified as likely to occur in the future, such as the Westhampton Interim Project. The post-Sandy condition in many instances is now equal to this future vulnerable condition.

Along the backbay shoreline, the topography is more stable. Since the area is gently sloping, it is difficult to characterize the area relative to its topography. Instead, the relative heights of the backbay area are described in the next section which provides a description of the floodplain.

2. Existing Coastal Structures and Expected Future Response. As discussed in the existing conditions and w/o project conditions section, there are a number of beachfill projects and coastal structures in the study area. There is also a history of local beachfill efforts, which is expected to continue in the future. These existing projects and expected future activities significantly affect conditions that are likely to occur in the future. In areas where fill projects have occurred and are likely to occur in the future, it is expected that there is a limit to how degraded the shoreline conditions will become. It is also necessary to consider the long-term erosional trends that would likely occur with these projects in place and whether or not there is an existing coastal structure, or beachfill project that would likely occur in the future.

3. Long-Term Erosional Trends. Long-term erosional trends are those conditions which are due to differences in long-shore transport rates, physical conditions, or constructed features which impact long-shore transport. The long-term erosional trends are essential when assessing the

long-term changes that are likely to occur in a given area, and whether the area is erosional, stable, or accreting.

4. Shoreline Undulations. Shoreline undulations, in contrast to long-term erosion trends, are an erosional signature apparent to different degrees along the study area that are short-term in nature and somewhat ephemeral. Shoreline undulations are also referred to as "circulation cells", and "erosional hot spots". The exact cause of these shoreline undulations is unknown, but it is assumed that there is a correlation between the condition of the nearshore bar, and the localized erosion. Analysis has been undertaken to evaluate historic shorelines to identify locations where these undulations are likely to occur, and the likely magnitude of these shoreline undulations.

These analyses show that the undulations tend to form and migrate alongshore for a distance before disappearing. Although it appears that some areas are more prone to these undulations, analysis of the undulations indicates that they could occur anywhere along the shoreline. For purposes of analyses, it has been assumed that the undulations can occur anywhere, and are likely to range in size between 1 and 2 km in length (0.6 miles to 1.2 miles). The landward and seaward amplitude of the undulations were quantified as 16 meters (52 feet). It is important to note that in the analyses, it is assumed that locations which are experiencing accelerated erosion due to the presence of existing coastal structures are not subject to erosional undulations.

5. Sea Level Rise. Sea level rise is a critical factor when evaluating future impacts. For purposes of this analysis, an estimate for future sea level rise based upon the historical rate of change for the gage at Sandy Hook has been used. To reflect the fact that a significant degree of uncertainty surrounds the selection of a rate of sea level rise for use in this analysis, all modeling exercises allow variation of the rate of sea level rise from simulation to simulation, with the final results incorporating the average affects of sea level rise over many simulations. Based on the Sandy Hook gage, the most likely rate of sea level rise in the study area is estimated to be 0.127 feet per year, and that the sea level rise follows a log-normal probability distribution with a standard deviation of 0.0006 feet per year. It is acknowledged that the assumed most likely rate is a conservative estimate for purposes of alternative analysis, and may understate without-project damages and with-project benefits.

## Overview of the Modeling Approach

This section provides an overview of the specific hydrodynamics of the study area. To orient the reader, the following paragraphs summarize the modeling efforts undertaken for this study, including an overview of the hydrodynamic modeling and the estimation of frequency relationships.

- Storm-surge numerical modeling

Storm-surge numerical modeling was performed to produce peak storm water levels at 49 locations across the entire FIMP study area. These 49 locations were selected to capture the variability in storm water levels along the open coast and within the three bays. The storm-surge numerical modeling strategy for FIMP addressed a comprehensive list of physical processes (wind conditions, barometric pressure, astronomic tide, wave conditions, morphologic response, [namely barrier island overwash and breaching], and localized wind and wave setup) by merging hydrodynamic, wave, and sediment transport models. The integration of these modeling efforts is shown below. Each component shown is described below in Figure 1.

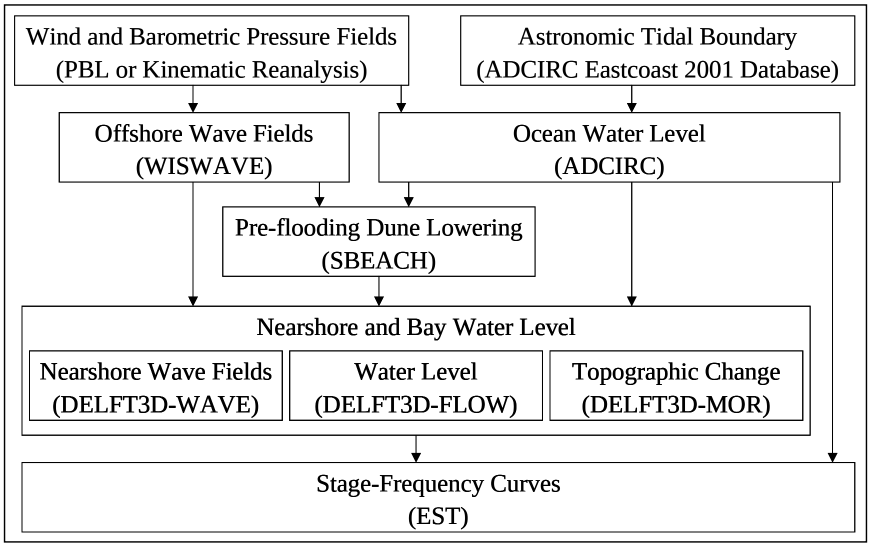

*Figure 1. FIMP storm water level modeling and stage-frequency methodology*

The six numerical models were applied to accomplish specific requirements for the study, as described below:

1. A Planetary Boundary Layer (PBL) model for wind field simulation was used to develop wind and pressure fields for tropical storms. (Thompson and Cardone, 1996).

2. An Interactive Kinematic Objective Analysis (IKOA) for wind field simulation was used to develop extratropical wind fields through data assimilation, based upon the National Center for Environmental Prediction (NCEP) database.

3. The offshore extreme storm wave conditions were generated using WISWAVE (also WAVAD) (Resio, 1981) a second generation, directional spectral wave model. WISWAVE output was used as input for the DELFT3D modeling and for SBEACH.

4. ADCIRC was used to simulate the ocean and nearshore (outside the surf zone) storm water levels (Luettich et al., 1992). ADCIRC is a long-wave hydrodynamic finite-element model that simulates water surface elevations and currents from astronomic tides, wind, and barometric pressure by solving the two-dimensional, depth-integrated momentum and continuity equations. The grid resolution varies from very coarse at the open ocean boundaries to 50-m in some nearshore locations. ADCIRC was forced with the winds and barometric pressure fields from 1 and 2 above, to capture meteorological effects on water levels, in conjunction with astronomic tidal constituents from the ADCIRC East Coast 2001 Tidal Constituent Database.

5. SBEACH was used for the hydrodynamic modeling, and separately to evaluate the shorefront response for the design and evaluation of beachfill alternatives. In the context of the hydrodynamic modeling, SBEACH was applied to estimate dune lowering that occurred prior to a dune being overtopped. SBEACH (Larson and Kraus 1989a; Larson, Kraus, and Byrnes 1990) is a numerical model for predicting beach, berm, and dune erosion due to storm waves and water levels. SBEACH is an empirically-based model of beach profile change developed to replicate dynamics of dune and berm erosion using standard data (topography, beach profiles, etc.) available in most engineering applications. In model simulations, the beach profile progresses to an equilibrium state as a function of the initial profile condition (including median grain size and shoreward boundary conditions) and storm conditions (wave height, period, and direction; wind

speed and direction; and water level). The model predicts profile response to storms including wave overtopping and dune lowering (Kraus and Wise 1993, Wise and Kraus 1993). For storm surge modeling, SBEACH storm simulations were performed for more than 200 beach profiles cut from the 2000 lidar topography. Dune crest elevation change just prior to inundation was extracted from the SBEACH simulation results and put into the DELFT3D topography grid to improve estimates of potential breaching and overwash processes.

6. The DELFT3D Modeling Suite (FLOW, WAVE, MOR) was used to compute the bay water levels under storm conditions, taking into account the contribution of storm surge, waves, winds and the contribution of overwash and/or breaching.

a. The DELFT3D-FLOW applied for this study simulates water level and currents from tidal, meteorological, and wave forcing by solving a two-dimensional depth-integrated flow and transport phenomena. The grid for this study extended from East Rockaway Inlet eastward to the east side of Shinnecock Bay. The model grid includes Great South, Moriches, and Shinnecock Bays, and their inlets, and extends up to 5 km from across the nearshore, with variable resolution. DELFT3D-FLOW was forced along its offshore boundary with water level time series from ADCIRC, throughout its domain with the storm wind and pressure fields, and with wave radiation stress fields.

b. The stationary wave model HISWA (DELFT3D-WAVE) was used to compute nearshore wave climate and resulting surf-zone radiation stresses (Holthuijsen et al., 1989). HISWA is a second generation wave model that computes wave propagation; wave generation by wind; non-linear wave-wave interactions and dissipation for a given bottom topography; and stationary wind, water level, and current field in waters of deep, intermediate and finite depth.

HISWA wave computations are carried out on a rectangular grid. A nested grid approach was also used for nearshore wave modeling and spans from East Rockaway Inlet to Montauk Point. The offshore grid was forced on its offshore boundary with significant wave height, peak period, and mean wave direction, input from the WISWAVE simulations, for each hourly input condition. The HISWA model has a dynamic interaction with DELFT3D-FLOW (i.e. two way wave-current interaction), which accounts for the effect of waves on currents and the effect of flow on waves, including wave setup, which allows for direct simulation of the impacts of wave setup on hydrodynamics, specifically water level at the coastline and in the estuarial bays.

c. The morphological changes, namely barrier island overwash and breaching, were simulated using DELFT3D-MOR. Three-dimensional transport of suspended sediment is calculated in DELFT3D by solving the three-dimensional advection-diffusion (mass-balance) equation for the suspended sediment. The local flow velocities and eddy diffusivities are based on the results of the hydrodynamic computations. Computationally, the three-dimensional transport of sediment is computed in exactly the same way as the transport of any other conservative constituent, such as salinity and heat. For the transport of non-cohesive sediment the Van Rijn (1993) formulation is used, which accounts for the effect of waves. Based on these sediment transport calculations, the elevation of the bed is dynamically updated at each computational time-step.

Collectively, these models simulate the impact that each modeled storm has on ocean and bay water elevations, lowering of the dune during the storm, and the morphological response due to a storm. The outputs from these models were input into a statistical modeling tool to estimate the likelihood of storm occurrence. The output from this analysis is presented in this report to represent the likelihood of various storm effects.

- Stage Frequency Methodology

The Empirical Simulation Techniques (EST) was applied to generate stage frequency curves. EST are a group of nonparametric methods for proceeding directly from hydrometeorological storm data to simulations of future storm activity and coastal impact, without introducing parametric assumptions concerning the probability law formulas and related parameters of the data (Scheffner *et al.*, 1999).

Two EST procedures, one univariate (1-D) and the other multivariate, were used in the FIMP studies. The 1-D EST methodology, using water level as the one dimension, was employed for stage-frequency development for the FIMP study. The multivariate EST was used in conjunction with SBEACH for modeling of beach profile response and estimation of storm-induced coastal changes, which is used to evaluate the beach and dune impacts for purpose of design and evaluation. (see Gravens *et al*., 1999).

For the FIMP study, the 1-D EST methodology was applied in a manner to account for the likelihood that historic storms could impact the areas at any tide condition. In order to apply this approach, 21 additional alternate tide events were run, to provide an improved estimate of the storm effects under different tide conditions. Along the open coast, the total surge generally can be added to the various tide conditions to develop the total surge effect, however, due to the complicated hydrodynamics of flows through the inlets and over the barrier island ,this approach does not work well within the bays,. With the inclusion of these alternate tide scenarios, final stage-frequency curves were generated to represent stage frequency relationships for the study area, at the 49 locations output from the model.

- Storms and Ocean Storm Induced Water Level

Storms are the major drivers for storm damage within the study area. The modeling efforts have been undertaken to characterize likely storm activity in the future, and the storm response that can be expected in the future, under different topographic conditions. The basis for our modeling effort in this study assumes that storms will occur in a manner similar to what has occurred in the past. Historic storms (as shown in Table 2) were used to develop statistics on storm recurrence, and the corresponding estimates of storm frequency, and the estimates of stage frequency. Figure 2 shows a stage-frequency curve along the Atlantic Ocean illustrative of the western portion of the FIMI project area. Two curves are shown on the figure: the storm water level alone, and the water elevation including the storm water level plus wave setup. The storm water level value is an elevation which is determined based upon procedures described above. This storm water level value is not representative of what an observer would see if standing on the beach.

In addition to the storm water level, there is an additional increase in water elevation due to wave setup, where the breaking of waves results in a localized increase in the water surface elevation along the coast. This value of storm water level, plus setup is what is shown as the higher elevation on this curve. Wave setup is a component of the water elevation which is difficult to quantify, and which can be variable depending upon the specific site conditions. As shown in the figure, wave setup adds an additional 2 feet of water to the storm water height under the conditions evaluated.

It is important to note that the combination of surge plus setup is intended to be representative of the still water elevation along the shoreline. To replicate conditions that would be representative of what an observer would see during a storm, one would also have to include the amount of wave runup which occurs. The amount of wave runup that occurs can be characterized as the average amount of runup, or the extreme amount of runup that occurs. These runup values are not directly used in the design and evaluation of alternatives, and are not presented here.

**Table 2: Historic Storms Modeled for FIMP**

| **Tropical Events (1930 – 2001)** | | | **Extratropical Events (1950 – 1998)** | |
|---|---|---|---|---|
| **Name** | **Start Date** | **Duration (hrs)** | **Start Date** | **Duration (hrs)** |
| not named | 10-Sep-1938 | 15 | 22-Nov-1950 | 34 |
| not named | 9-Sep-1944 | 10 | 04-Nov-1953 | 26 |
| Carol | 25-Aug-1954 | 5 | 11-Oct-1955 | 43 |
| Edna | 2-Sep-1954 | 7 | 25-Sep-1956 | 34 |
| Hazel | 5-Oct-1954 | 6 | 03-Mar-1962 | 56 |
| Connie | 3-Aug-1955 | 0 | 05-Nov-1977 | 28 |
| Donna | 29-Aug-1960 | 13 | 17-Jan-1978 | 16 |
| Esther | 10-Sep-1961 | 14 | 04-Feb-1978 | 27 |
| Doria | 20-Aug-1971 | 2 | 22-Jan-1979 | 19 |
| Agnes | 14-Jun-1972 | 18 | 22-Oct-1980 | 17 |
| Belle | 6-Aug-1976 | 7 | 26-Mar-1984 | 31 |
| Gloria | 16-Sep-1985 | 5 | 09-Feb-1985 | 17 |
| Bob | 16-Aug-1991 | 4 | 28-Oct-1991 | 50 |
| Floyd | 7-Sep-1999 | 3 | 01-Jan-1992 | 18 |
| | | | 08-Dec-1992 | 78 |
| | | | 02-Mar-1993 | 12 |
| | | | 10-Mar-1993 | 25 |
| | | | 28-Feb-1994 | 22 |
| | | | 21-Dec-1994 | 23 |
| | | | 05-Jan-1996 | 25 |
| | | | 6-Oct-1996 | 12 |
| | | | 02-Feb-1998 | 24 |

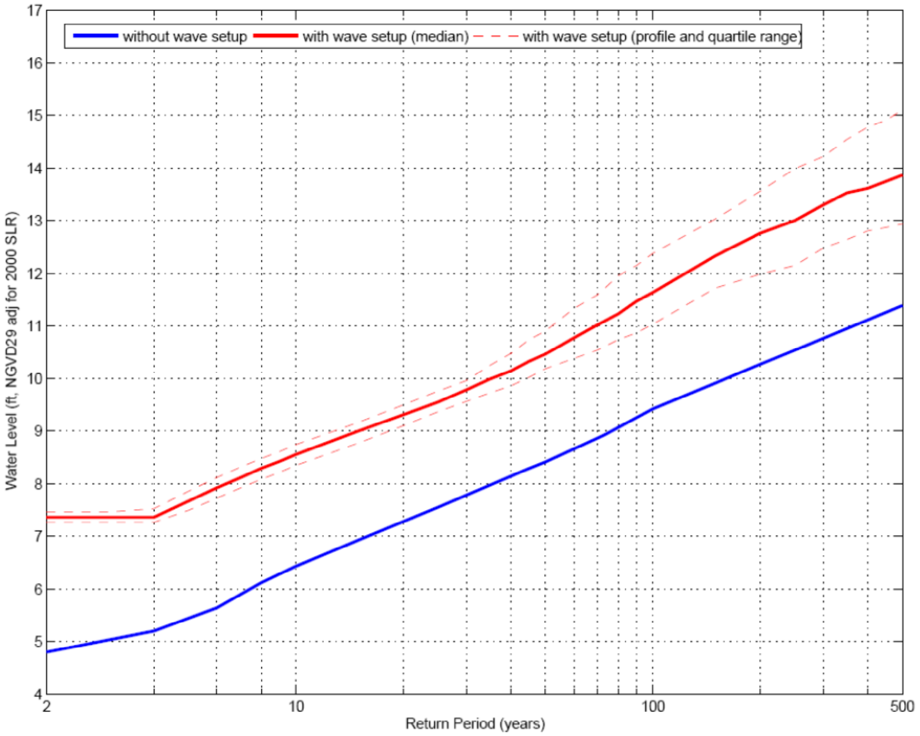

*Figure 2  Stage-frequency relationships with and without ocean wave setup for Station 23, Great South Beach.*

- Erosion Response.

The storm parameters were used as input in a variety of coastal engineering models to characterize the erosion response for various topographic conditions. The Corps model SBEACH was applied to characterize the erosional response along the barrier island. Characterization of the erosional response of a dune and beach and the impacts to existing development requires consideration of several important factors. These factors include: 1) the landward extent of erosion (erosion distance), 2) the amount of dune lowering that occurs during a storm, and the landward translation of the 0 ft contour which occurs during a storm. Figures 3 – 5 illustrate the erosional response for a typical location along the project area in Western Fire Island, for the various modeled profiles.

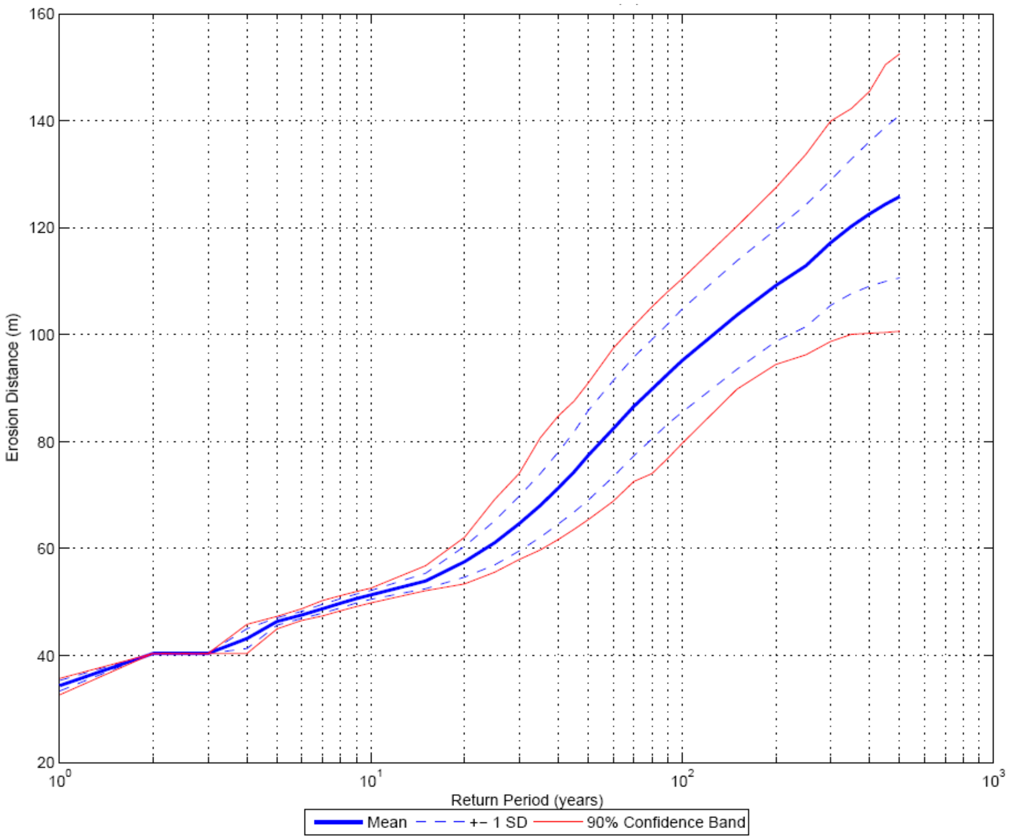

*Figure 3  Erosion distance vs. frequency for Fire Island subreach F-R2 (1 m = 3.28 ft).*

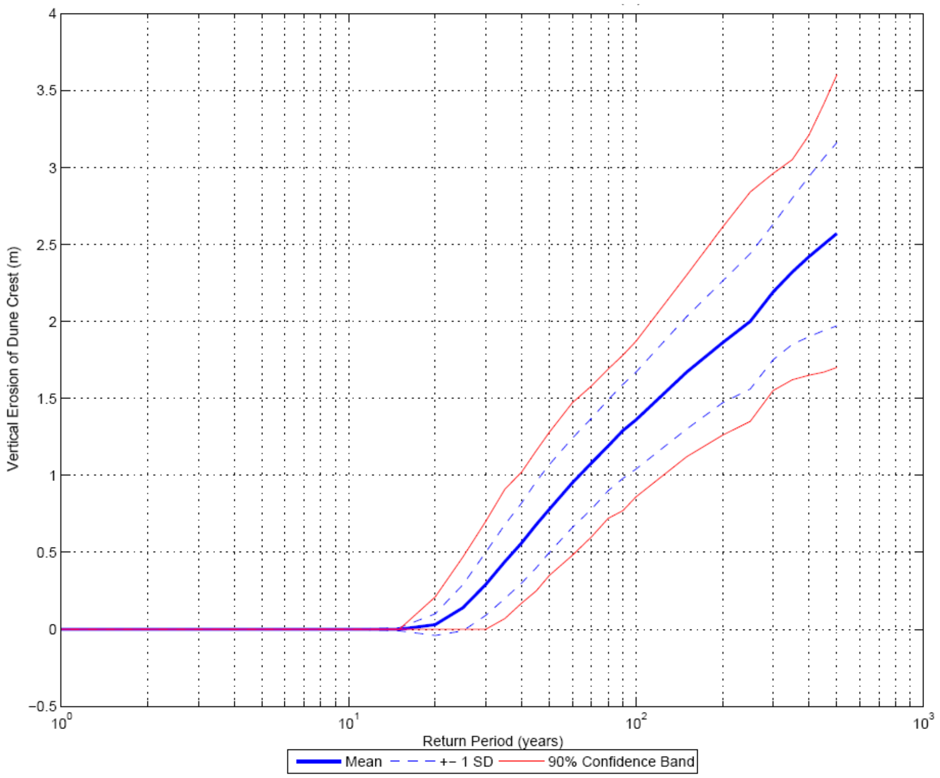

*Figure 4 Vertical erosion of dune crest vs. frequency for Fire Island subreach F-R2 (1 m = 3.28 ft).*

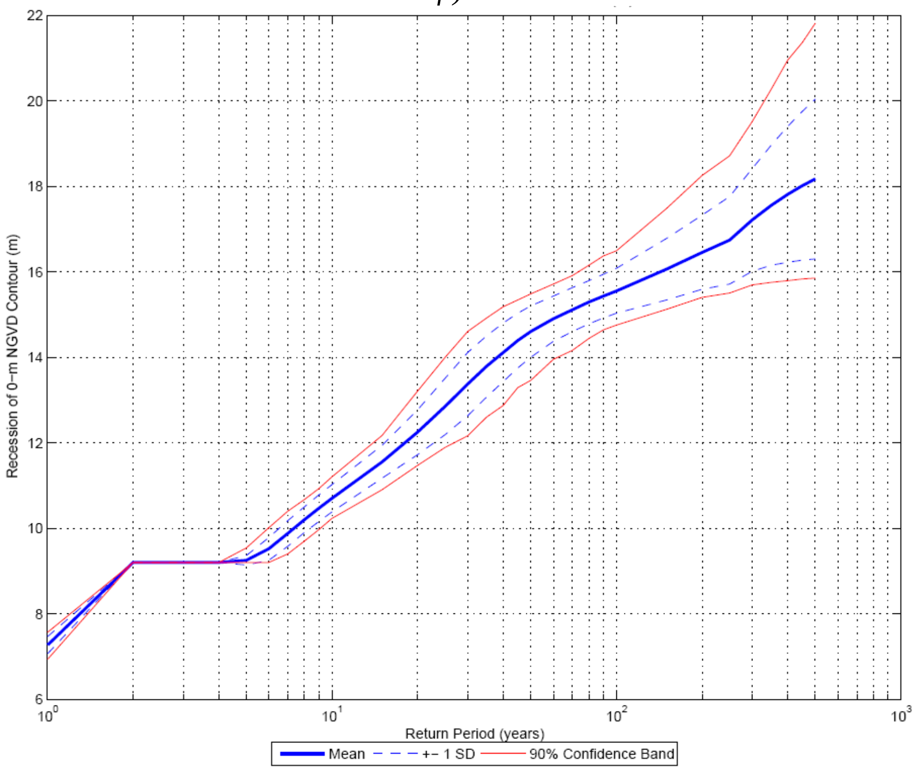

*Figure 5. Recession of 0-ft NGVD29 elevation vs. frequency for Fire Island subreach F-R2*

- Post-Storm Recovery.

It is important to note that the SBEACH modeling, described above captures the erosion which occurs during a storm event. What has been observed to occur is, immediately after a storm event, beaches tend to begin to recover when long-period waves move the sand from the nearshore back onto the beach. When determining how the study area evolves over time, it is important to estimate the amount of recovery expected in an area. The amount of recovery, expressed as a percentage of the volume lost, depends upon a number of factors, including the sediment budget.

The estimated amount of beach recovery has been established for various shoreline locations. These recovery amounts have been developed in order to match the long-term erosional trends for each location, and establish whether the area is erosional, stable or accreting in the long-term.

### Overwashing and Breaching Models

A major problem is erosion response that compromises the integrity of the barrier island and results in an overwash or breach. Characterizing the complicated breaching process required the application of a number of models to evaluate the likelihood of overwashing or breaching, and the concomitant impact on bay water elevations. Additional engineering analyses were also necessary. The following elements were reflected in the modeling, and are described further below.

1. The likelihood of overwash and breaching for various topographic conditions.

2. The likely evolution of these breaches if they are allowed to remain open.

3. The impact these breaches have on the water elevations in the backbay environment.

4. The expected sediment exchange associated with these processes.

#### Definition of Morphological Response Terminology

In order to have a common understanding of overwashing and breaching, a common set of terminology was established for characterizing the response. This terminology is summarized here. The terms overwash, washover, runup, overtopping, breaching, partial breaching and full breaching are specifically described below for clarity of the analyses of these coastal processes.

Overwash is "(a) a mass of water representing the part of the uprush that runs over the berm crest (or other structure) without flowing directly back to the sea or lake and (b) the flow of water in restricted areas over low parts of barriers or spits, especially during high tides or storms," (Glossary of Geology, American Geological Institute, 1987). Overwash tends to erode or flatten dunes during a storm with an attendant deposition of eroded sediment on the landward side of the barrier island (washover). This terminology is commonly used in most of the relevant research in the area of barrier island morphodynamics and in reports of large storm damage available in the literature and has been adopted for FIMP.

Note, however, that engineers and researchers sometimes use the term overwash to refer specifically to the intermittent volume of water that overtops the dune due solely to wave runup, defined as the peak elevation of wave uprush above still-water level. Wave uprush consists of two components: super elevation of the mean water level due to wave action (wave setup) and fluctuations about that mean (swash). This intermittent flow occurs only when the total water level (tide + storm surge + wave setup) remains below the dune crest elevation. Others use the term overtopping instead to refer to this intermittent water flow and the term overwash to refer to the sediment transport associated with it. For the purposes of this study the intermittent flow due to runup will be referred to as overtopping, whereas the continuous flow that occurs after the dune is inundated by setup will be denoted as overflow. Overwash will be used according to the more general definition provided in the previous paragraph, which could include both overtopping and overflow.

The term overwash (or overwash area) is also used in this report to denote the resulting storm-induced barrier island response (topographic change) to water moving over the barrier island by overwash and overflow processes. In this report, the term overwash when referring to storm-induced morphological change will indicate lowering of the barrier island, between its pre-storm elevation and the Mean High Water (MHW) datum.

Breaching refers to the condition where overflow cuts a channel across the island that permits the exchange of ocean and bay waters under normal tidal conditions. For this report, two degrees of morphological response to breaching will be used. A partial breach is a storm-induced barrier island cut that has a scoured depth between MHW and Mean Low Water (MLW) while a full breach is a storm-induced barrier island cut that has a scoured depth at or below Mean Low Water (MLW). A partial breach will allow for water to exchange between the ocean and bay during a portion of the normal tidal cycle while a full breach will allow water exchange during the complete tidal cycle. A partial or full breach may potentially develop into a permanent breach during normal tide conditions following a storm.

Overwashing and breaching are interrelated. For example, severe overwashing can lead to breaching. A storm that results in breaching is also likely to result in overwash. The breach or overwash area may be temporary or permanent (i.e., a new inlet) depending on the size of the breach, adjacent bay water depths, potential tidal prism, littoral drift, etc.

- Potential Breach Locations

In the application of the model, a number of locations were identified that met the conditions necessary to be prone to breaching, considering dune and beach conditions, and barrier island width. Through a series of model tests, these potential breach areas were further refined. The result of these tests identified the following areas identified as “potential breach locations”, which are shown in Figure 6. It is important to note that these are the areas identified as likely breach locations based upon current conditions, and reasonably foreseeable future conditions, but should not be taken to imply that other locations in the Study Area would not breach in the future, but for purposes of analysis, the impact of breaching focuses on these areas. It is also important to note that in order to consider the hydrodynamic impacts of these breaches on the back-bay, assumptions have been made that breaches in proximity to each other would have a similar effect on the back-bay. For purposes of hydrodynamic impacts, we have considered the impact relative to breaching reach, as identified in Table 3.

Table 3 Breach locations and Breach reach

| Breach Area | Potential Breach Locations | Breach Reach* |
|---|---|---|
| 1 | Fire Island Lighthouse Tract | Western Great South Bay |
| 2 | Kismet to Corneille States | Western Great South Bay |
| 3 | Talisman to Blue Pt. Beach | Central Great South Bay |
| 4 | Davis Park | Central Great South Bay |
| 5 | Old Inlet West | Eastern Great South Bay |
| 6 | Old Inlet East | Eastern Great South Bay |
| 7 | Smith Point County Park | Moriches Bay |
| * For evaluating hydrodynamic response in the bay | | |

- Overwash and Breaching Frequency

As described above, the engineering modeling effort was undertaken for a range of topographic conditions. The Empirical Simulation Technique analysis established frequency relationships and estimated the likelihood of overwash, partial breaching, and breaching by location and for the baseline and future vulnerable condition. As these results are intended to be applied in a lifecycle model, it is important that the complete range of future conditions is modeled. We have determined that if a breach occurs in the future, it is likely that it would be closed in a manner similar to the existing Breach Contingency Plan. Since this is a possible condition in the future, without project condition this barrier island condition was also evaluated for frequency of overwash and breaching. A summary of the results is shown in Table 4, which shows the range of return periods for the different morphological responses expected.

The table illustrates that in the baseline condition, the probability of breaching is relatively low, and the probability of breaching increases significantly in the future vulnerable condition as shown in Table 5. This difference in response can be attributed to both dune height and beach width, although in areas where a dune is vulnerable, it appears the primary driver is beach width. In conditions where the beach is wide, there are limited forces acting on the dune. If there is no dune lowering, due to wave action, it is rare for the dune to be overtopped

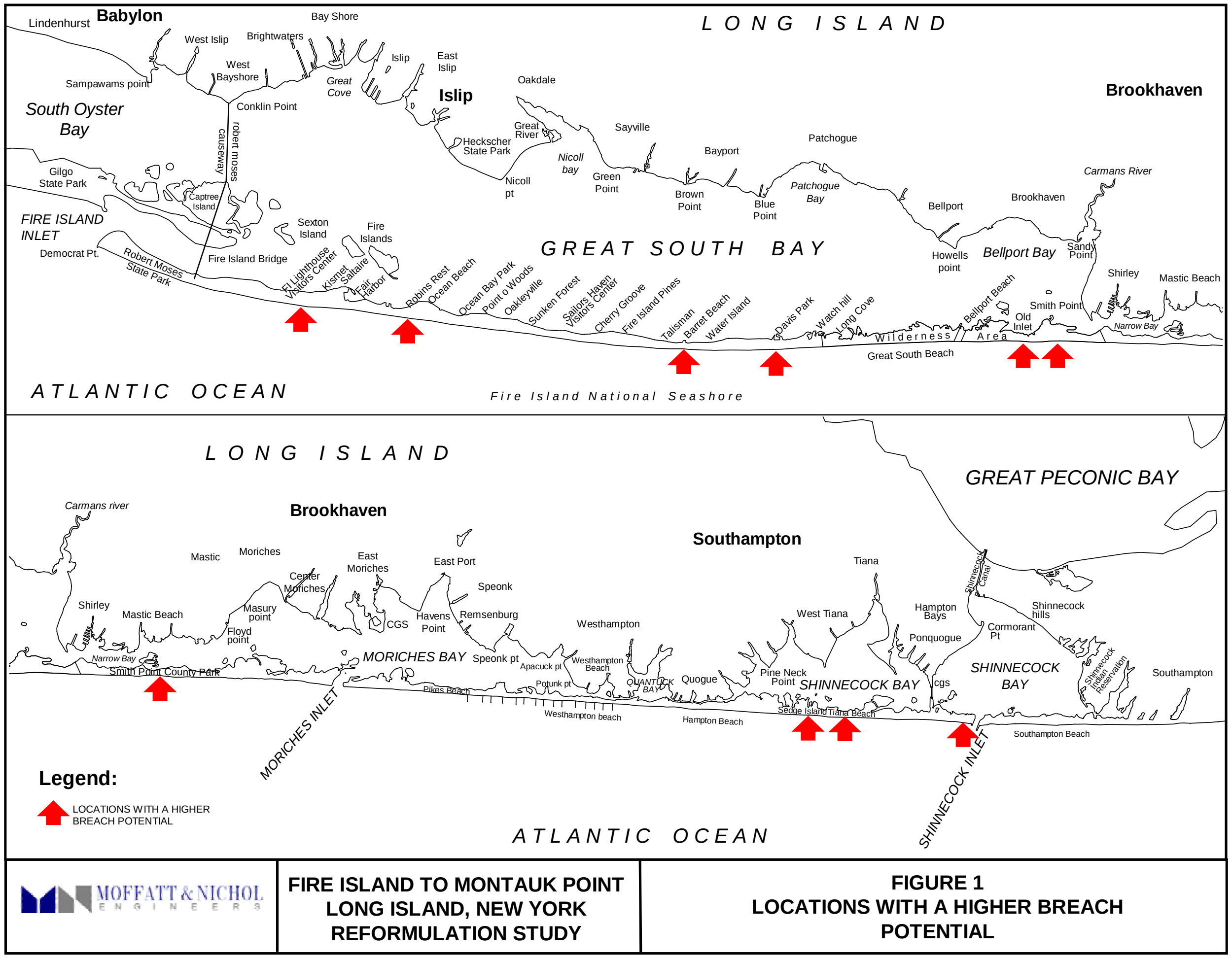

*Figure 6 Vulnerable Breach Locations*

Table 4  Return Periods for Overwash/Breaching

| | Fire Island Lighthouse Tract | Kismet to Corneille Estates | Talisman to Blue Point Beach | Davis Park | Old Inlet, West | Old Inlet, East | Smith Point County Park |
|---|---|---|---|---|---|---|---|
| Baseline Conditions (return period in years) | | | | | | | |
| Overwash | 14 - 184 | 9 – 141 | 20 – 213 | 22 – 145 | 10 – 45 | 5 – 24 | 8 – 26 |
| Partial Breaching | 184 – 500 | 141 – 500 | 213 – 500 | 145 – 500 | 45 – 82 | 24 – 118 | 26 – 145 |
| Full Breaching | > 500 | > 500 | > 500 | > 500 | > 82 | > 118 | > 145 |

| FVC (return period in years) | | | | | | | |
|---|---|---|---|---|---|---|---|
| Overwash | 3 – 34 | 5 – 15 | 5 – 12 | 15 – 73 | 4 -7 | 5 – 19 | 4 – 9 |
| Partial Breaching | 34 – 106 | 15 – 34 | 12 – 31 | 73 – 288 | 7 – 22 | 19 – 84 | 9 – 141 |
| Full Breaching | > 106 | > 34 | > 31 | > 288 | > 22 | > 84 | > 141 |

| Breach Closed (return period in years) | | | | | | | |
|---|---|---|---|---|---|---|---|
| Overwash | 5 – 21 | 5 – 17 | 5 – 39 | 12 – 26 | 4 – 12 | 5 – 34 | 5 – 20 |
| Partial Breaching | 21 – 43 | 17 – 37 | 39 -80 | 26 – 108 | 12 – 67 | 34 – 191 | 20 – 139 |
| Full Breaching | > 43 | > 37 | > 80 | > 108 | > 67 | > 191 | > 139 |

To apply these results in the lifecycle modeling, it is important to acknowledge that the breach response can also be considered as the total ocean still water elevation necessary to result in a morphological change. The morphological response information is used in the lifecycle model in this manner, which allows the life-cycle model to account for increases in sea level, which contribute to the total ocean still water elevation. Adding sea level, in effect, requires a lower storm surge height to trigger the morphological response, and in turn shows that overwash and breaching becomes more frequent with sea level rise.

- Breach Evolution

The consideration of breach frequency is an important component of the analysis. Equally important is the consideration of how these morphological responses evolve over time. In this regard, it is important to characterize 1) the likelihood of a breach remaining open and growing, and 2) the rate at which a breach would grow. Both of these considerations are important for the "partial breach" and the "full breach" conditions.

Evaluating the likelihood of a breach growing is a difficult, complex process drawing upon past experience with breaching, numerical modeling and engineering judgment to project breach evolution. These analyses established that the likelihood for a breach to grow is dependent upon the initial condition (whether it is a full breach or partial breach), and upon the time of year a breach occurred (during the winter, nor'easter season, or tropical, summer season). It is recognized that if any area has a full breach, and water is exchanging throughout the full tidal cycle, the breach will grow. However, it is also assumed that there is a limit to how many breaches can be sustained in a bay at any given time, which could limit how many breaches grow, in the instance of multiple breaches.

For a partial breach, it is recognized that there is a probability that the breach does not grow, but closes naturally. The probability that a partial breach will grow is affected by the time of year that the breach occurred. Wave conditions during the winter, extratropical season are more extreme than the typical summer month conditions. Therefore, conditions are such that a partial breach would be more likely to remain open in the winter months than in the summer months. The analysis projects a 50% likelihood of a partial breach closing naturally during the winter months, and a 75% likelihood of a partial breach closing naturally in the summer months.

- Breach Growth

In scenarios where it is established that a breach or partial breach is likely to grow, observed breaches were interpreted to estimate the rate of breach growth. Observations of past breach growth shows that the growth rate can be fit to an exponential curve, to project the width and cross-sectional area of the breach. The rate of growth is dependent upon the tidal prism of the back bay. Based upon this analysis, projected growth rates were developed for each of the potential breach areas, based upon the back bay conditions. The results of this original analysis are shown in Tables 5 and 6, which show the expected width and cross-sectional area for each of the potential breach locations. As discussed above, it is acknowledged that the breach growth rate for a particular breach is dependent upon whether or not there is another breach into the bay. For a single breach, it is assumed that it would grow to this size, for more than one breach it is assumed that the breaches collectively would grow to this size.

*Table 5  Estimated long-term potential breach widths (used in modeling).*

| Project Reach | Range Value | Breach Widths (feet) | | | | |
|---|---|---|---|---|---|---|
| | | **1 Month** | **3 Months** | **6 Months** | **9 Months** | **12 Months** |
| GSB | Minimum | 720 | 1,870 | 3,070 | 3,830 | 4,320 |
| | Maximum | 1,340 | 3,070 | 4,320 | 4,820 | 5,030 |
| MB | Minimum | 320 | 830 | 1,360 | 1,690 | 1,910 |
| | Maximum | 750 | 1,600 | 2,080 | 2,220 | 2,270 |

*Table 6  Estimated long-term potential breach cross-sectional areas (used in modeling).*

| Project Reach | Range Value | Breach Areas (sq. feet) | | | | |
|---|---|---|---|---|---|---|
| | | **1 Month** | **3 Months** | **6 Months** | **9 Months** | **12 Months** |
| GSB | Minimum | 5,040 | 13,120 | 21,480 | 26,820 | 30,220 |
| | Maximum | 9,380 | 21,480 | 30,220 | 33,770 | 35,210 |
| MB | Minimum | 2,230 | 5,800 | 9,490 | 11,850 | 13,360 |
| | Maximum | 5,270 | 11,180 | 14,550 | 15,560 | 15,870 |

The breach dimensions shown in Tables 5 and 6 are the breach cross-sectional areas that were used as the basis for modeling, and describing the changes in stage frequency curves under various scenarios.  For purposes of lifecycle modeling, these breach growth rates have been updated to account for the observed data that is available following hurricane Sandy, particularly the breach at Old Inlet.  These updated breach growth curves are shown in Table 7 and 8.

*Table 7  Updated, post-Sandy estimated long-term potential breach widths (used for lifecycle modeling).*

| Project Reach | Range Value | Breach Widths (feet) | | | | |
|---|---|---|---|---|---|---|
| | | **1 Month** | **3 Months** | **6 Months** | **9 Months** | **12 Months** |
| GSB | Minimum | 120 | 310 | 510 | 640 | 720 |
| | Maximum | 1,240 | 2,840 | 3,990 | 4,460 | 4,650 |
| MB | Minimum | 320 | 830 | 1,360 | 1,690 | 1,910 |
| | Maximum | 750 | 1,600 | 2,080 | 2,220 | 2,270 |

*Table 8 Updated, post-Sandy estimated long-term potential breach cross-sectional areas (used for lifecycle modeling).*

| **Project Reach** | **Range Value** | **Breach Areas (sq. feet)** | | | | |
|---|---|---|---|---|---|---|
| | | **1 Month** | **3 Months** | **6 Months** | **9 Months** | **12 Months** |
| GSB | Minimum | 840 | 2,170 | 3,560 | 4,440 | 5,010 |
| | Maximum | 8,680 | 19,880 | 27,960 | 31,250 | 32,580 |
| MB | Minimum | 2,230 | 5,800 | 9,490 | 11,850 | 13,360 |
| | Maximum | 5,270 | 11,180 | 14,550 | 15,560 | 15,870 |

- Back-Bay Water Elevations

As described previously, the water elevations in the bays due to storms are sensitive to the barrier island conditions. The modeling and analyses described above were used to generate water elevations in the back bay which are representative of the different barrier island conditions. To represent the range of possible future conditions, the following scenarios were evaluated:

1 – Baseline condition, representative of the September 2000 Conditions,
2 – The future vulnerable condition, representative of a more degraded condition,
3 – Breach open conditions for breaches at varying locations (5 breach reaches), and of varying sizes (a breach open for 3 months, and a breach open for 12 months, corresponding to the dimensions in Tables 4 and 5), and
4 – Breach closed conditions, representative of the BCP template.

It is a challenge to show the differences in the water elevations, due to the number of possible scenarios. Figures 7 to 10 show the differences in the stage frequency curves for a representative location in Great South Bay, and Moriches Bay. Two sets of curves are provided for each station. The first set compares baseline conditions, breach closed conditions, and future vulnerable conditions. The second set compares the baseline condition and various breach open conditions.

These figures are complicated, but illustrate the effect that changes in topography have on the flooding conditions within the bays, and illustrate the following:

For Great South Bay:

- The baseline condition stage frequency curves are very flat (very small difference between a 2-yr event and a 500-yr event), with an elevation of +2.8 ft NGVD corresponding to a 2-yr event, and an elevation of +5.2 ft NGVD corresponding to a 500-yr event.
- Under a Future Vulnerable Condition, flooding is greater than the baseline condition beginning at a 10 year event, and has the effect of increasing the height of flooding by 1 ft to 1.5 ft.
- Under a breach closed scenario, flooding is greater than the baseline condition beginning at a 5 yr event, and can have the effect of increasing flooding up to 2 feet, as compared to the baseline condition.
- Under Breach Open conditions, a single breach of the barrier island, when open for 3 months can increase flooding 1 to 1.5 feet above normal. Multiple breaches can increase this flooding up to 2 feet above baseline conditions

- Under Breach Open conditions, a single breach of the barrier island, when open for 12 months can increase flooding 1.5 to 2 feet above normal. Multiple breaches can increase this flooding up to 3 feet above baseline conditions
- Since these curves are so flat, these scenarios have a tremendous impact on the flooding regime. Under FVC topography, and breach-closed topography, the elevation of a 500-yr event (+5.2 ft NGVD) would be experienced with a 25 yr event or 20-yr event respectively.
- Under breach-open scenarios, the equivalent of a 500-yr event would be experienced by a storms ranging from a 2-yr event to 10-yr event.
- Under breach-open conditions, the expected flooding would be significantly higher than is currently accounted for in the floodplain management regulations.

For Moriches Bay

- The baseline condition stage frequency curves have a greater range as compared to the curves in Great South Bay., with an elevation of +4 ft NGVD corresponding to a 2-yr event, and +7.7 ft NGVD corresponding to a 500-yr event.
- Under a Future vulnerable condition, flooding is greater than the baseline condition beginning at a 10 year event, and has the effect of increasing the height of flooding by 1 to 2 ft
- Under a breach-closed scenario, flooding is very similar to the baseline condition, and can have the effect of increasing flooding up to 2 feet.
- Under Breach Open conditions, the effect is dependent upon the location of the breach; for a single breach of the barrier island, when open for 3 months can increase flooding 1 to 2 feet above normal. Multiple breaches can also result in flooding up to 2 feet above baseline conditions
- Under Breach Open conditions, a single breach of the barrier island, when open for 12 months can increase flooding 1.5 to 2 feet above normal. Multiple breaches can increase this flooding up to 3 feet above baseline conditions
- Although the baseline curves are not as flat as in Great South Bay, these scenarios have a tremendous impact on the flooding regime. Under FVC topography, and breach closed topography, the elevation of a 500-yr event (+7.7 ft NGVD) would be experienced with a 60 yr event, and a 100-yr event experienced with a 40-yr event.
- Under breach-open scenarios, the equivalent of a 500-yr event would be experienced by a storms ranging from a 30-yr event to 50-yr event, and the equivalent of a 100-yr event could be experienced by a 10-yr event.

Overall, these figures illustrate that there are measurable changes in the flooding that can occur along the mainland of Long Island, when there is the potential for increased water to enter into the bay during a storm that results in a breach, or when a breach is open. This illustrates that flooding typically encountered under extreme storm events, such as a 100-yr event or 500-yr event could be experienced due to more frequent storm events.

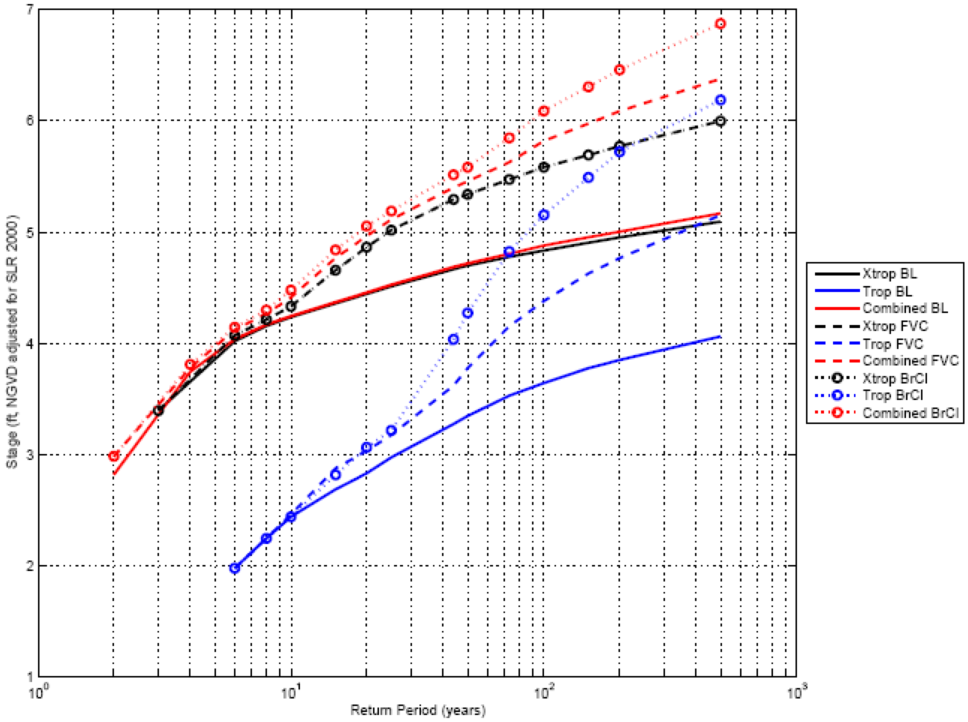

*Figure 7. Comparison between Breach Closed (BCC, BrCl), Baseline BLC, (BL), and Future Vulnerable (FVC) stage-frequency curves at Station 5, Connetquot River, in Great South Bay.*

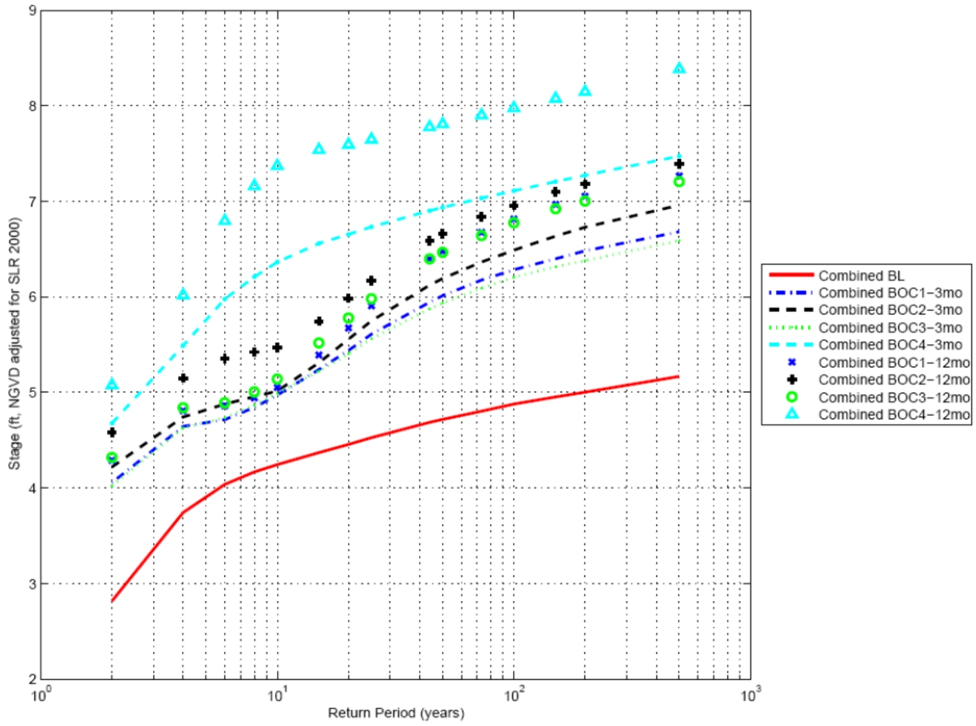

*Figure 8. Comparison between Baseline (BLC, BL) and Breach Open (BOC) stage-frequency curves at Station 5, Connetquot River, in Great South Bay.*

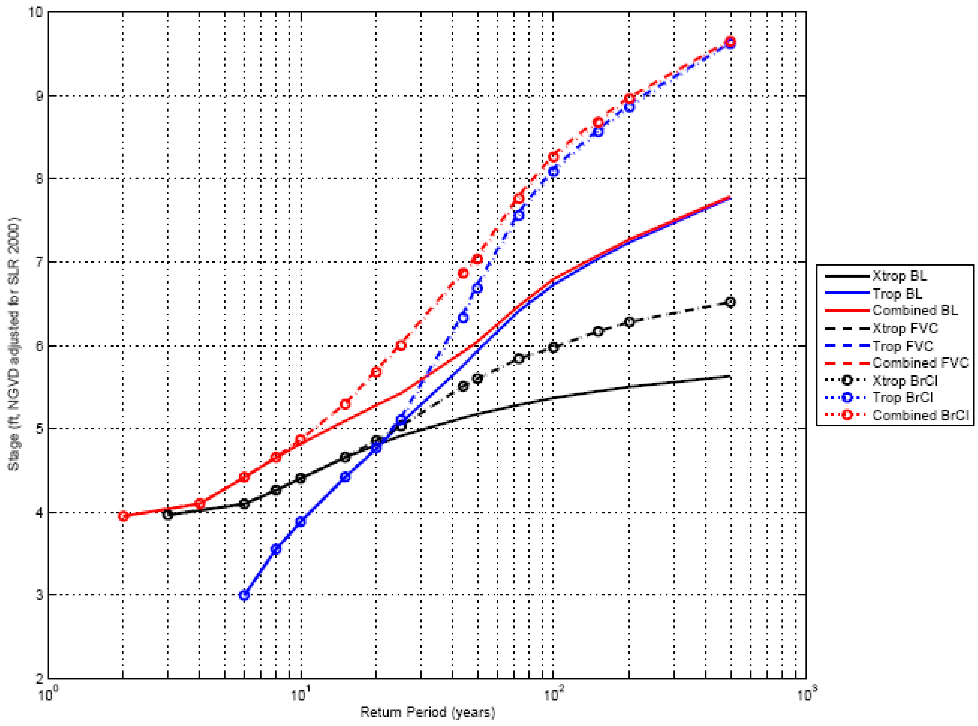

*Figure 9. Comparison between Breach Closed (BCC, BrCl), Baseline (BLC, BL), and Future Vulnerable (FVC) stage-frequency curves at Station 13, Apacuck Point, in Moriches Bay.*

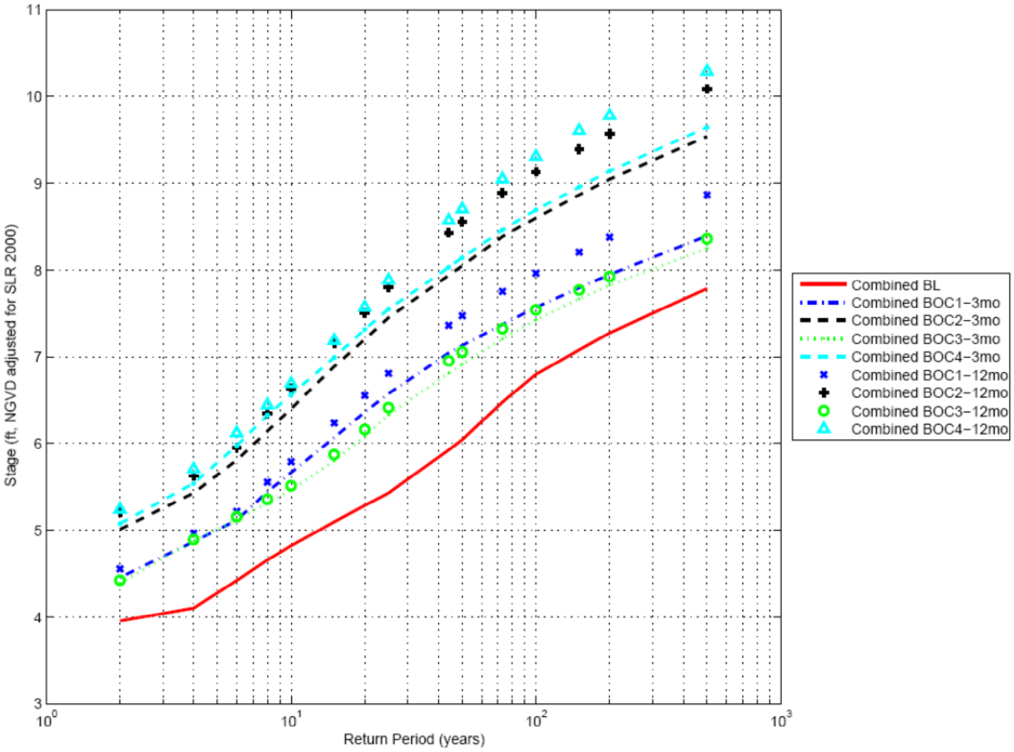

*Figure 10. Comparison between Baseline (BLC, BL) and Breach Open (BOC) stage-frequency curves at Station 13, Apacuck Point, in Moriches Bay*

## D. Lifecycle Considerations

The information summarized above has been used as input into several lifecycle models, which are used to project the storm damages that are likely to occur in the future. Separate models have been developed to consider backbay damages, which consider barrier island changes, and the concomitant change in back bay water elevations, and shorefront models, which evaluate damages along the shorefront. The changes in the barrier island condition, and dune conditions are governed by storm response, post-storm recovery, long-term erosional trends, and shoreline undulations. The results of these models indicate that the risk of breaching will increase in the future due to the combined impacts of sea level rise, storms and barrier island erosion. Table 7 provides a summary of the important physical factors used to project changes in barrier island conditions at the most vulnerable breach locations.

**Table 7 Vulnerable Breach Location Data**

| **Bay Location** | GSB | GSB | GSB | GSB | GSB | GSB | MOR |
|---|---|---|---|---|---|---|---|
| | FI Lighthouse | Kismet/Corneille | Talisman/Blue Pt. | Davis Park | Old Inlet W | Old Inlet E | SPCP |
| **Effective Beach Widths for Input Conditions (ft)** | | | | | | | |
| Baseline | 200 | 150 | 150 | 250 | 200 | 200 | 200 |
| Future Vulnerable | 50 | 50 | 50 | 50 | 50 | 50 | 50 |
| Breach Closed | 35 | 53 | 50 | -13 | 101 | 97 | 109 |
| **Background (non-storm) Shoreline Change (ft/yr)** | | | | | | | |
| Mean/Year | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| Std Dev/Year | 4 | 4 | 4 | 4 | 4 | 4 | 4 |
| **Shoreline Undulation** (Standard Deviation in ft) | 52 | 0 | 52 | 52 | 52 | 52 | 52 |
| **Profile Recovery from Storm Induced Erosion** | 76% | 91% | 100% | 100% | 98% | 98% | 98% |
| Baseline | 6.97 | 6.38 | 7.28 | 7.40 | 6.34 | 5.31 | 6.00 |
| Future Vulnerable | 5.12 | 5.52 | 5.44 | 6.93 | 5.05 | 5.31 | 5.05 |
| Breach Closed | 5.52 | 5.52 | 5.44 | 6.60 | 5.05 | 5.31 | 5.24 |
| **Partial Breach** | | | | | | | |
| Baseline | 10.18 | 9.88 | 10.35 | 9.87 | 8.21 | 7.44 | 7.54 |
| Future Vulnerable | 8.09 | 7.06 | 6.65 | 8.92 | 5.78 | 7.15 | 6.18 |
| Breach Closed | 7.47 | 7.22 | 6.65 | 7.61 | 6.60 | 7.87 | 7.20 |
| **Full Breach** | | | | | | | |
| Baseline | 11.57 | 11.57 | 11.49 | 11.49 | 8.99 | 9.48 | 9.71 |
| Future Vulnerable | 9.58 | 8.09 | 7.82 | 10.72 | 7.33 | 9.03 | 9.66 |
| Breach Closed | 8.38 | 8.19 | 7.92 | 9.51 | 8.71 | 10.06 | 9.68 |