US Army Corps of Engineers®
New York District

DRAFT FINAL

Integrated Hurricane Sandy General Reevaluation Report and Environmental Impact Statement

Atlantic Coast of New York

East Rockaway Inlet to Rockaway Inlet and Jamaica Bay

Appendix A1

Shorefront Engineering and Design Appendix

August 2018

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ACRONYMS

AASHTO	American Association of State Highway Transportation Officials
ACES	Automated Coastal Engineering System
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
BOD	Basis of Design
CPT	Cone Penetration Test
CY	Cubic Yards
DDR	Design Documentation Report
E&D	Engineering & Design
EL.	Elevation
EM	Engineering Manual
EMT	Electrical Metallic Tubing
ETL	Engineering Technical Letter
ER	Engineering Regulation
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
FS	Factor of Safety
FWOP	Future Without Project
FWP	Future With Project
FWS	Fish and Wildlife Service
GIWW	Gulf Intracoastal Waterway
HAT	Highest Astronomical Tide
HSDRRS	Hurricane Storm Damage Risk Reduction System
ITR	Independent Technical Review
ISO	International Standards Organization
LAT	Lowest Astronomical Tide
MHHW	Mean Higher-High Water
MHW	Mean High Water
MSL	Mean Sea Level
MLW	Mean Low Water
MLLW	Mean Lower-Low Water
NACCS	North Atlantic Comprehensive Study
NAN	USACE New York District
NAVD88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
O&M	Operation and Maintenance
PED	Pre-Construction Engineering and Design
QA	Quality Assurance

OSHA	US Department of Labor, Occupational Safety and Health Administration
RSLR	Relative Sea Level Rise
PVC	Polyvinyl chloride
Rev	Revision
RSLR	Relative Sea Level Rise
ROW	Right of Way
SLR	Sea Level Rise
SPT	Standard Penetration Test
TOW	Top of Wall
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey

East Rockaway Inlet to Rockaway Inlet and Jamaica Bay Reformulation Study

Draft Integrated Hurricane Sandy General Reevaluation Report and Environmental Impact Statement

Engineering Appendix A1

Shorefront Engineering and Design

1 PROJECT DESCRIPTION

1.1 Overview of Engineering & Design Appendix

This Atlantic Coast of New York, East Rockaway Inlet to Rockaway Inlet Hurricane Sandy General Revaluation Report Engineering Appendix summarizes the multiple models and analyses applied to evaluate and compare alternative features for each planning reach within the study area. Since each planning reach is exposed to different risk mechanisms while they must collectively function as a system, the engineering appendices document the evaluation process in separate sub appendices which detail the specific analyses applied to confirm the recommended plan is engineeringly feasible, complete and economically justified.

The USACE transition to SMART Planning is an additional reason which resulted in the inclusion of multiple sub-appendices. The initial study was initially limited to the Atlantic Ocean Shorefront Planning Reach and was conducted as a legacy study. The engineering analyses were conducted to satisfy a more rigorous design level and the Atlantic Ocean shorefront summary engineering documents were written to satisfy those study requirements. The Jamaica Bay Planning Reach analysis was added following Hurricane Sandy and was conducted to broaden the recommended plan to the entire authorized study area and was conducted under SMART planning guidelines.

SMART planning documents propose a 10% design, documentation of risks and efforts to mitigate risks, and decisions made to expedite the opportunity for public and agency comment on the recommendation. More detailed design decisions are generally deferred to the Planning, Engineering and Design phase.

This Engineering & Design Appendix provides an overview of the analyses supporting the Shorefront Erosion Control alternative analysis and Shorefront Coastal Storm Risk Management (CSRM) alternative analysis for the Atlantic Ocean Shorefront Planning Reach.

This appendix includes a description of the project area history (Chapter 2) and the performance of prior projects in Chapter 3. Existing conditions inclusive of the water level, storm surge and wave conditions for the Atlantic shorefront are described in Chapter 4. Borrow source areas and considerations are presented in Chapter 5. Chapter 6 presents the sediment budget for the planning reach prior to the description of the project alternatives in Chapter 6. Two categories of alternatives, 1) Erosion Control Alternatives and 2) Coastal Storm Risk Management (CSRM) alternatives are addressed within that chapter. The general approach to identifying a recommended plan is to evaluate erosion control alternatives in combination with a single beach restoration plan to select the most cost-effective approach to reducing project renourishment. This analysis is a lifecycle cost comparison to ensure that the most cost effective renourishment approach has been identified prior to the evaluation of alternatives that address coastal storm risk management. Secondly, the Coastal Storm Risk Management (CSRM) alternatives consist of various beachfill, dune and seawall measures to reduce future storm damages for Rockaway Beach are compared. Chapter 8 presents the recommended plan.

The Engineering Appendix for the Shorefront Planning Reach is organized into multiple sub-appendices, which are a compilation of engineering work products which summarize the design and performance specifications which informed the Recommended Plan.

Appendices

1.2 Reach Delineation

The Project Area has been divided into smaller segments known as reaches for the purpose of engineering and economic analysis. As shown in Figure 1-1, the Project Area is an 11 mile long narrow peninsula with Rockaway Inlet at the western project limit near Breezy Point and East Rockaway Inlet at the eastern limit near Beach 19 Street. The engineering analyses include historical erosion rates, sediment budget, historic shoreline changes and alternatives design considerations. The economic analyses include damages, cost, and benefit estimates. The reaches are developed based on physical, economic, and institutional differences including sediment transport rate boundaries, shoreline orientation, coastal structures, topographic elevations, and existing economic developments. The engineering reaches do not have to coincide with the economic reaches; however, they must be interrelated. The Project Area has been divided into six major reaches based on both engineering and economic considerations as follows:

- Reach 1: Rockaway Point to Beach 193rd Street
- Reach 2: Beach 193rd Street to Beach 149th Street
- Reach 3: Beach 149th Street to Beach 109th Street
- Reach 4: Beach 109th Street to Beach 86th Street
- Reach 5: Beach 86th Street to Beach 42nd
- Reach 6a: Beach 42nd Street to Beach 28th Street
- Reach 6b: Beach 28th Street to Beach 19th Street

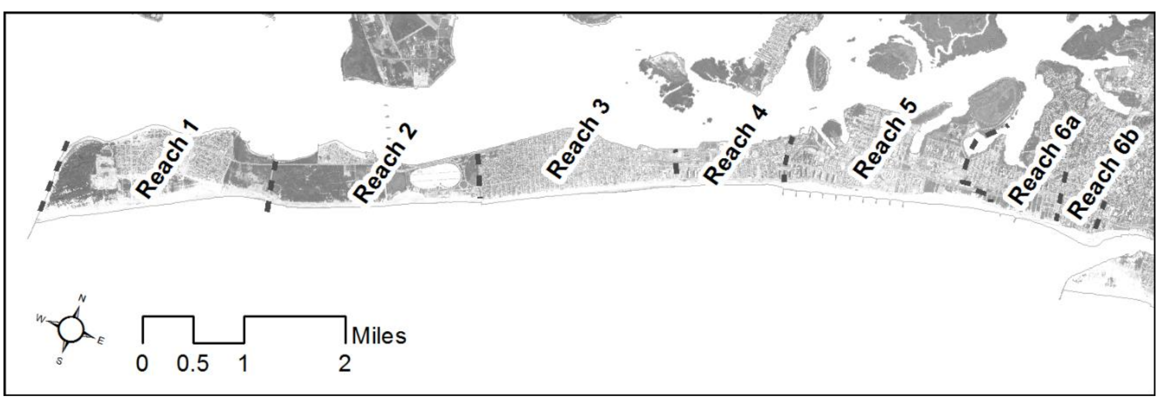


Figure 1-1: Rockaway Shorefront Engineering Reaches

1.3 Overview of Engineering Modeling

The numerical modeling strategy for The Atlantic Ocean Shorefront addresses a comprehensive list of physical processes by utilizing a range of hydrodynamic, wave, sediment transport, and shoreline change models. The following numerical models were applied in the study:

- ADCIRC – storm surge propagation;
- WISWAVE – regional wave transformation;
- STWAVE – nearshore wave transformation;
- SWAN – nearshore wave transformation;
- GENESIS – long-term shoreline evolution;
- SBEACH – storm induced profile change;
- XBEACH – cross-island flooding.

A detailed description of these modeling efforts is provided in the Engineering Modeling Appendix (Sub-Appendix A). The results of these modeling analyses are used throughout this Engineering and Design Appendix.

2 PROJECT AREA HISTORY

2.1 Storm History

The study area is subject to damages from hurricanes and extra-tropical storms known as "northeasters". Hurricanes strike the study area more frequently in summer months from June to October while northeaster strike in winter months from January through March. Hurricanes that most severely affect the study area usually approach from the south-southwest direction after re-curving around eastern Florida and skirting the Middle Atlantic States. The most severe hurricane on record for the study area is Hurricane Sandy, which occurred on 29 October 2012.

Northeasters sometimes develop into more complex storms. Relative location of high and low pressure centers may cause wind speeds in excess of what could be expected from a single storm cell. Winds reaching almost hurricane strength may occur over many thousands of square miles. Northeasters may form with little or no advance warning and may persist for as long as a week to ten days. The most severe northeaster on record that struck the study area occurred on 6-8 March 1962 which caused serious tidal flooding and wide spread damage all along the Middle Atlantic Coast. The following paragraphs describe a few of the more notable hurricanes and northeasters that impacted the study area. All elevations throughout this appendix are reference to the North American Vertical Datum of 1988 (NAVD88) unless specifically stated otherwise.

2.1.1 Hurricane of 21 September 1938

The center of this hurricane skirted the east coast of New Jersey and struck the south shore of Long Island near Moriches Inlet during a rising tide. The minimum barometric pressure at New York City was 28.72 inches. The U.S. Weather Bureau station near the Battery reported a gust velocity of 80 mph from the northwest. A maximum storm tide elevation of 4.3 ft NAVD88 was recorded at Sandy Hook, NJ. At Edgemere, about 500 summer cottages and 10 pleasure boats on Jamaica Bay were damaged. At Rockaway Beach, physical damage to waterfront private property and to pleasure boats was estimated at $25,000 and $56,000, respectively, as reported in 1938. A heavy sea swept away a large section of the breakwater near the Neponset Beach hospital east of Jacob Riis Park. It flooded the first floor, endangering approximately 130 children.

2.1.2 Hurricane (Donna) of 12 September 1960

Hurricane Donna made landfall approximately 100 miles east of Atlantic City, New Jersey at 11:00 am EST on September 12th. The hurricane eye passed near the study area and moved north during the high tide and became elongated and extended from New York City to Montauk Point. U. S. Air Force radar operators at Montauk Point indicated that the storm had separated into three eyes upon reaching this area. The weather bureau reported winds between 60 and 70 mph at 2:00 pm at New York City. The fastest speed recorded at LaGuardia Airport in Queens was 70 mph from northwest at 11:52 am with peak gust wind speed of 97 mph at 12:31 pm. The weather bureau at New York City reported a low barometric pressure of 28.65 inches at 1:40 pm. The near coincidence of the storm's passage with the time of high tide in the study area resulted in the highest recorded storm water elevation (at the time) in the study area. Storm tide elevations peaked at +7.3 feet NAVD88 at Sandy Hook, NJ. High water marks in the study area were documented in the post-storm survey report (USACE, 1961). Based on Plate 4 of this report, the maximum ocean high water marks were approximately +10.2 feet NAVD88 and maximum bay high water marks were approximately +8.3 feet NAVD88.

Total damages in Rockaway reached $8,774,000 as reported in 1960. Ocean flooding and wave attack damages were chiefly residential and, to a lesser extent, commercial. About 6,000 homes and hundreds of commercial establishments were damaged. There was extensive damage to passenger cars, as well as public and private schools. Many streets of the low-lying Rockaway Peninsula were under 3 to 4 feet of water, which remained several feet deep in many areas for four days because of inadequate drainage facilities. In Arverne and Far Rockaway (Beach 74th to 32nd streets), there was a major breach, as bay and ocean waters met, causing physical damages of more than $2,500,000 to homes. Commercial losses were exceptionally severe near Edgemere Avenue and Beach 34th street in Far Rockaway, where ten stores were completely ruined and subsequently went out of business.

2.1.3 Storm of 6-8 March 1962

This extratropical storm resulted from the merging of two storms. One storm moved easterly from the Midwest, while the other storm moved northerly up the coast. The storms merged off the mid-Atlantic coast and remained nearly stationary. Strong offshore winds over a long fetch of ocean affected the entire Atlantic coast for three days. This storm has been described as one of the most destructive extra tropical cyclones ever to hit the United States coastline, and is one of the most destructive to hit the study area. A continuous storm surge height of 3 to 5 feet coupled with spring tides resulted in prolonged storm tides with a maximum elevation of 6.6 ft NAVD88 at Sandy Hook, NJ. High water marks in the study area were documented in a post-storm survey report (USACE, 1963). Based on Plate 2 of this report, the maximum ocean high water marks were approximately +9.2 feet NAVD88 and maximum bay high water marks were approximately +7.7 feet NAVD88.

The Rockaway Peninsula suffered total estimated damage of $8,450,400 as reported in 1962. Damaging high waters occurred on five successive high tides over a period of 48 hours. The storm tides combined with the high waves to carry flood waters to buildings which would ordinarily be beyond the reach of ocean water. The storm's long duration caused unprecedented beach erosion, damage to groins and jetties, and destruction of houses on sites which had been considered safe for 60 to 80 years.

2.1.4 Storm of 28-30 March 1984

This extra-tropical storm threatened the study area, but available data indicated that it caused no damages except for beach erosion, which was caused by high waves driven by storm winds up to

84 mph. A maximum storm tide elevation of +5.9 ft NAVD88 was recorded at Sandy Hook, New Jersey.

2.1.5 Hurricane Gloria of 27 September 1985

This hurricane made landfall at Fire Island, 50 miles east of the study area, at 11:00 am EST on September 27th. The forward velocity was reduced by the rotational velocity at the study area, resulting in moderate damage to the study area. A storm surge height of 7.3 feet was recorded at Sandy Hook, NJ but coincided with low tide resulting in maximum storm tide elevation of only 5.7 ft NAVD88. If the storm had occurred closer to high tide the flooding impacts could have been severe.

2.1.6 Halloween Northeaster of 30 October, 1991

Overwash from this extratropical storm resulted in beach sand deposits on residential streets 100 to 200 feet landward of the boardwalk/beach. Beach erosion in the area between Beach 31st and Beach 34th streets resulted in the high water line (+1.3 ft NAVD88) reaching the boardwalk. A maximum storm tide elevation of +5.7 ft NAVD88 was recorded at Sandy Hook, NJ.

2.1.7 Northeaster of December 1992

One of the fiercest extratropical storms this century battered the project area for 48 hours with fierce rain, hurricane force winds, and near records storm tides. Ambrose Light station just southeast of New York City recorded sustained winds of 80 mph and gust to 93 mph. A record tying storm tide of +7.3 ft NAVD88 was recorded at Sandy Hook, NJ. The storm caused flooding and beach sand to wash up on residential streets. As a result of the flooding most of the roads on the peninsula were impassable and subway service was shutdown.

2.1.8 Hurricane Irene of 27 August 2011

In August 2011, Hurricane Irene was downgraded to a tropical storm right before it made landfall in New York City. In preparation the City issued the first-ever mandatory evacuation of coastal areas, including the entire project area. A storm tide of +7.0 ft NAVD88 was recorded at Sandy Hook, NJ. Damage to the boardwalk, beach erosion, and overwash and sand deposits were reported in the project area. The impacts of the storm were less than anticipated as the storm weakened just prior to making landfall.

2.1.9 Hurricane Sandy of 29 October 2012

On 29 October 2012, Hurricane Sandy made landfall approximately five miles south of Atlantic City, NJ, where it collided with a blast of arctic air from the north, creating conditions for an extraordinary and historic storm along the East Coast with the worst coastal impacts centered on the northern New Jersey, New York City, and the Long Island coastline. Hurricane Sandy's unusual track and extraordinary size generated record storm surges and offshore wave heights in the New York Bight. The maximum water level at The Battery, NY peaked at +11.3 feet NAVD88, exceeding the previous record by over 4 feet. The tide gauge at Sandy Hook, NJ reached +10.4 ft NAVD88 before failing. USGS deployed storm tide sensors and high water marks surveyed by the USGS after the storm indicate that the maximum water levels during Sandy varied between +12.9 ft NAVD88 and +10.3 ft NAVD88 within the Project Area (USGS, 2013).

Rockaway was one of the hardest hit areas by Hurricane Sandy. An overview of the extent of flooding in the project area is shown in Figure 2-1. As the storm surge rose the peninsula was flooded first with water from the ocean and then later with water from the bay. Strong ocean waves and currents carried water, sediment, and debris across the peninsula leaving behind a wake of destruction (Figure 2-2). Many homes and other buildings, including the boardwalk, were destroyed by waves or flooding and many more were severely damaged. At least four people are known to have died in this area. In addition to the direct effects of flooding, the storm caused the outbreak of multiple fires in Rockaway caused by the interaction electricity and sea water, including one in Breezy Point that destroyed over 100 homes. Critical services like electricity and water were knocked out leading to dangerous conditions, particularly in high-rise structures.

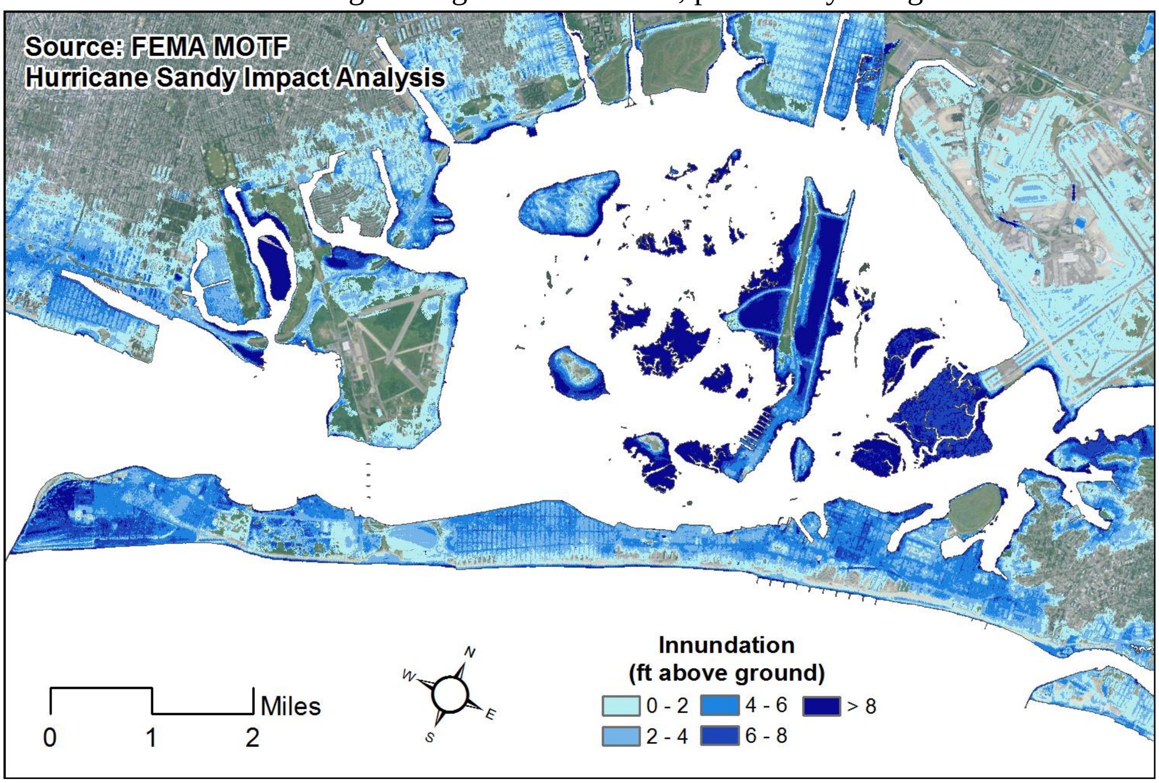


Figure 2-1: Hurricane Sandy Flood Inundation

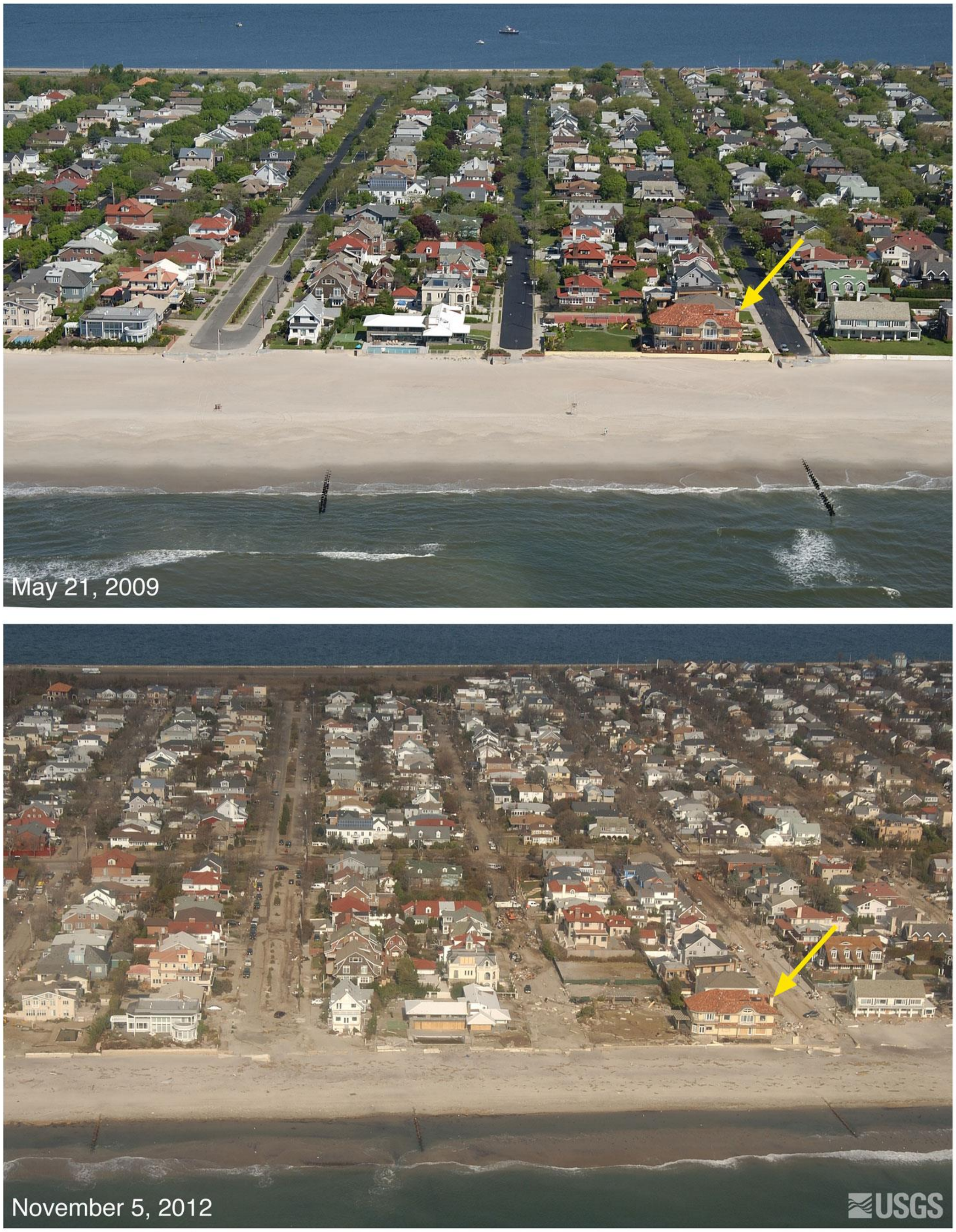


Figure 2-2: Pre- and Post-Storm Photo Comparison at Rockaway Beach

2.2 Shoreline History

2.2.1 Data Sources

High water shoreline surveys for the Rockaway Peninsula were obtained from U.S. Coast and Geodetic Survey records, and digitized aerial photography. Shoreline surveys show significant reconfiguration of the landforms over the period of record, which extends from year 1880 to present.

2.2.2 Analysis of Shorelines

The evolution of the shoreline along the Rockaway Peninsula may be characterized by several distinct time periods based on inlet stabilization, beachfill activities, and construction of erosion control structures (e.g. stone and timber groins).

1835-1934: Pre-Inlet Stabilization

Prior to the stabilization of Rockaway and East Rockaway Inlets the shoreline experienced large morphological changes including the growth of the Rockaway Peninsula to the southwest by more than 4 miles (Figure 2-3), and westward migration of East Rockaway Inlet (not shown). This time period is also characterized by construction of numerous bulkheads and groins (constructed between 1914 and 1930) and placement of fill for land development purposes.

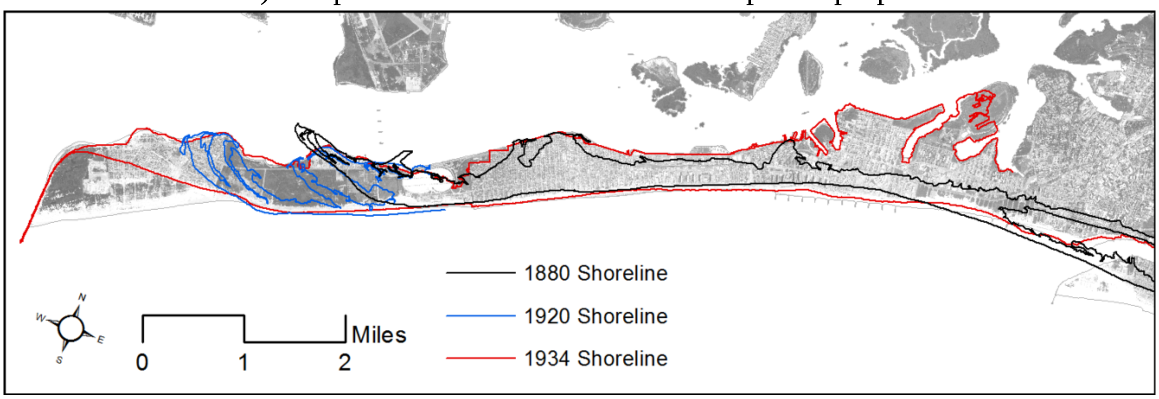


Figure 2-3: Shoreline Evolution: 1880-1934

1934-1961: Post-Inlet Stabilization

In 1933 an 8,400 ft long stone jetty was constructed on the updrift (eastern) side of Rockaway Inlet to stabilize the inlet. One year later, 1934, a 4,500 foot long stone jetty was constructed on the updrift (eastern) side of East Rockaway Inlet to stabilize the inlet. A second jetty was authorized for construction on the downdrift (eastern) side of East Rockaway Inlet but was never constructed. The construction of the jetties halted the westward migration of the inlets and stabilized their position. Shortly after the construction of the Rockaway Inlet jetty (1936) approximately 5 million cubic yards of fill was placed at the western end of Rockaway for land development. The fillet updrift of Rockaway Inlet continued to grow over this time period (Figure 2-4) creating the area now called Breezy Point. From 1930-1976 no maintenance dredging was required at Rockaway Inlet as nearly all of the sediment was impounded updrift of the jetty. This time period is also characterized by construction of additional stone and timber groins and sporadic beachfill projects. Construction of five stone groins in Reach 6a (Beach 36th St. to Beach 49th St) was completed in 1956. The seven stone groins in Reach 5 were constructed from 1961-1962.

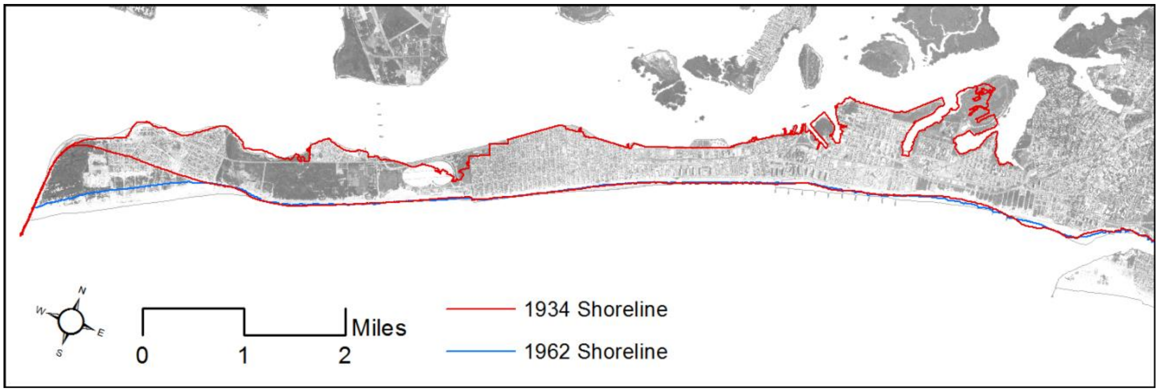


Figure 2-4: Shoreline Evolution: 1934-1962

1961-1973: No Engineering Activities

The Section 934 study (USACE 1993) examined shoreline changes between 1961 and 1973 based on profile surveys. This time period was chosen because no major beachfill operations occurred and all the shore stabilization groins were complete except the Beach 149th St. terminal groin, which was built in 1982. The only fill placement during this time period, 175,000 cubic yards, occurred in Reach 5 in 1962. Over this time period the shoreline was relatively stable, with shoreline change rates between +7.9 ft/yr and – 5.4 ft/yr. It is noted that in 1962 a very strong Nor'easter known as the "Ash Wednesday Storm of 1962" caused extensive coastal erosion as the storm persisted over 5 high tides. The 1962 shoreline is based on an aerial image taken on July of 1962, four months after the Ash Wednesday Storm.

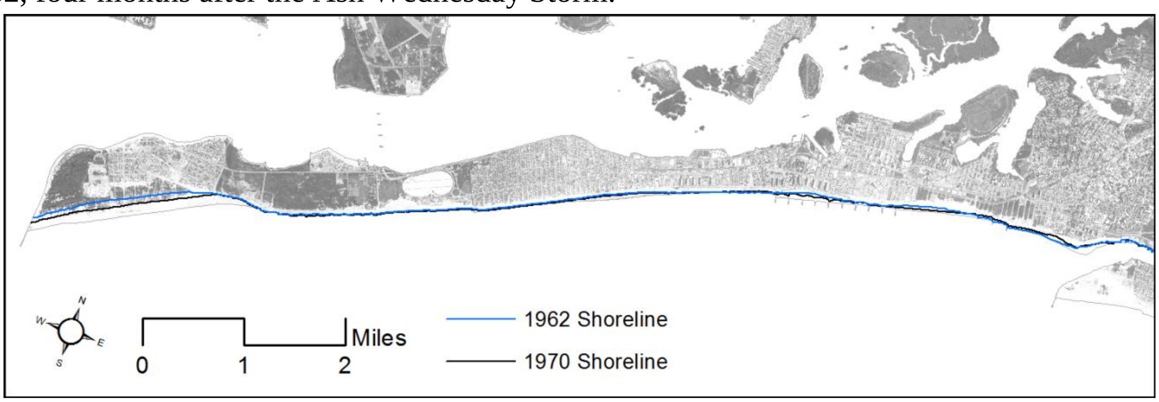


Figure 2-5: Shoreline Evolution: 1962-1970

1975-Present: WRDA 1974 & Section 934 WRDA 1986

This time period encompasses two major beach nourishment projects with intermittent renourishment and routine maintenance dredging of East Rockaway Inlet. An estimated 18.7 million cubic yards of beach fill was placed at Rockaway Beach during this time period. The shoreline positions fluctuated over this period in concert with the beach nourishment activities (Figure 2-6). The impact of 1975 beachfill project in Reach 4 and 5 is captured by the shoreward advance of the shoreline in these two reaches. Over this time period the net change in the shoreline position along the central portion of The Atlantic Ocean Shorefront (Reach 2 to Reach 5) is relatively small. The shoreline in Reach 1 and Reach 6b has advanced seaward by several hundred feet.

A detailed description of the performance of the two beach nourishment projects constructed during this period is provided in Section 3.

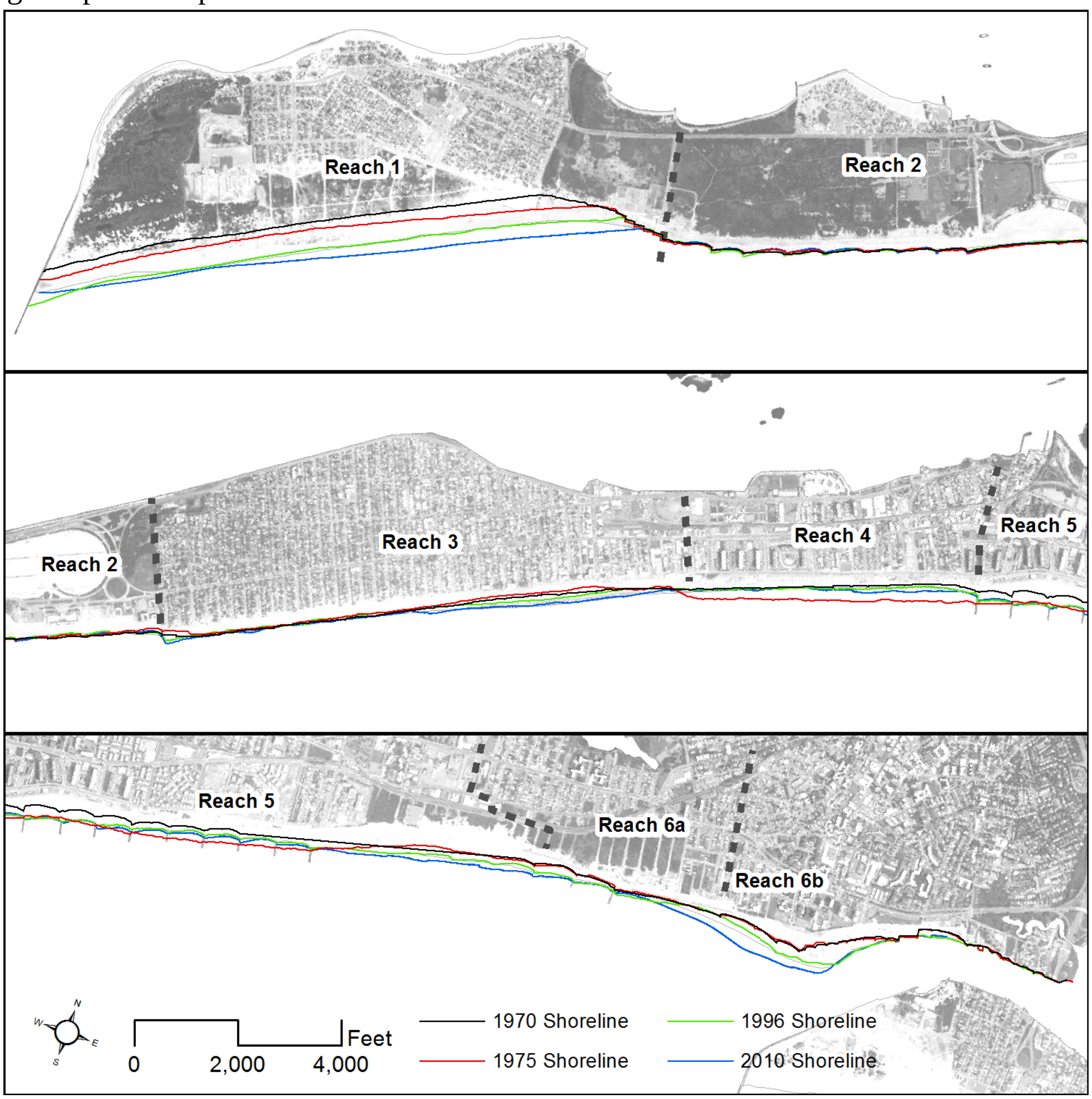


Figure 2-6: Shoreline Evolution: 1970-2010

2.3 Engineering Activities

2.3.1 Coastal Structures

Numerous shore protection structures were built along the Rockaway Peninsula during the 20th century. Table 2-1 summarizes the total number of groins and linear feet of bulkhead constructed during the 20th century. Table 2-2 lists coastal structures from west to east along the Peninsula as excerpted from the 1964 Survey Report, except structure #28, stone terminal groin at Beach 149th Street, built in 1982. As shown in the tables, the majority of the hard structures including over 200 groins and 25,300 linear feet of bulkhead were built prior to 1930. The next four decades saw the construction of additional 49 groins. The last ten stone groins were built in 1961-1962 until one final groin built in 1982 at the western end of the beach erosion control project (B149th St).

Based on a structure condition survey performed in spring of 2009, the majority of the timber groins are either buried or deteriorated and are no longer functioning effectively. The 24 stone groins constructed after 1943 are generally in fair to good condition.

Table 2-1: Groin and Bulkhead Construction by Decade

Time Period	# of Groins	Linear Feet of Bulkhead
Before 1930	207	25,300
1931-1940	14	0
1941-1950	20	0
1951-1960	5	0
1961-1970	10	0
1971-2015	1	0
TOTAL	257	25,300

Notes: Approximately 233 timber groins, all built on or before 1941, lengths range from 200 to 300 ft.

Table 2-2: List of Coastal Structures

Structure Number (West to East)	Type	Year Built
1	8,400 ft stone jetty at Rockaway Inlet	1933
2	7,600 ft bulkhead	1922
3	38 timber groins 200-300 ft long	1922
4	3 timber and stone groins 200 ft long	1922
5	8 stone groins 250-300'	1943
6	28 timber groins 200'	1922
7	25 timber groins 220'	1927
8	6 groins concrete& steel 300' inshore, timber offshore 300' long (500' total)	1940
9	6000 lf bulkhead	1930
10	20 timber groins 300'	1930
11	12 timber groins 200'	1941
28	1 terminal groin B149th St.	1982
12	39 timber groins 300'	1926
13	3 stone groins 500'	1962
14	1 timber groin 200'	1920
15	2 timber groins 160-220'	1919
16	4 stone groins 510'	1961
17	8 timber groins 400-600'	1938
18	7200 lf timber bulkhead	1928
19	21 timber groins 380'	1928
20	1 timber groin 300'	1917
21	5 stone groins 600'	1956
22	10 groins	1915
23	4,250 stone jetty E. Rock Inlet	1934
24	4,500 lf timber bulkhead	1930
25	17 timber groins	1930
26	3 stone groins 105'	1961
27	2 groins 60' timber inshore 150' stone offshore	1922

2.3.2 Beach Restoration

The Atlantic Ocean Shorefront has a long history of beachfill operations even before the start of the Federal Erosion Control Project in 1975. From 1926 to 2012 a total of approximately 27 million cubic yards of sand have been placed along Rockaway Beach. A tabulation of the historical beach fill volumes by reach and years is shown in Table 2-3. Historical beachfill operations have been either maintenance dredging of East Rockaway and Rockaway Inlets or larger beach restoration projects with sediment dredged from offshore borrow areas. A detailed account of inlet maintenance dredging is provided in the next section.

Since the Erosional Control Project began in 1975 approximately 19.5 million cubic yards of beach fill have been placed along Rockaway (Table 2-4). Nearly all of this beachfill over this time period has been placed east of Beach 149th Street in Reaches 3, 4, 5, and 6. However, Reach 4 and Reach 6 have received the greatest proportion of the beachfill volumes.

Table 2-3: Historical Beach Fill Placement (Cubic Yards) by Reach

Year	Reach 1	Reach 2	Reach 3	Reach 4	Reach 5	Reach 6
1926				1,250,000	1,250,000	
1928						1,450,000
1930			1,250,000			
1936	5,000,000					
1940		400,000				
1958						1,250,000
1962					175,000	
1975				1,834,500[1]	1,834,500[1]	
1976						1,490,000
1977			1,205,000			
1978				453,000		231,000
1980						466,000
1982			163,300	414,000		479,000
1984				828,000		631,000
1986				654,000		691,000
1988				501,000		819,000
1990						206,000
1991						157,000
1995						606,400
1996			582,800	742,900[1]	742,900[1]	
1998						218,000
2000				504,644[1]	504,644[1]	241,000
2002						140,000
2004			94,968	317,279[1]	317,279[1]	271,953
2005						221,002
2007						260,000
2009						285,000
2010				137,000		
2012						272,000

Notes: [1]Distribution of beachfill between Reaches 4 and 5 unknown. Volume equally split between reaches.

Table 2-4: Historical Beach Fill Placement Summary (Cubic Yards)

Year	Reach 1	Reach 2	Reach 3	Reach 4	Reach 5	Reach 6
1926-1974	5,000,000	400,000	1,250,000	1,250,000	1,425,000	2,700,000
1975-2012			2,046,000	6,386,000	3,400,000	7,685,000

2.3.3 Inlet Maintenance Dredging

Federal navigation channels are located at both inlets bracketing the Rockaway Peninsula. The East Rockaway Inlet channel is shown in Figure 2-7, and the Rockaway Inlet channel, known as the Jamaica Bay channel, is shown in Figure 2-8.

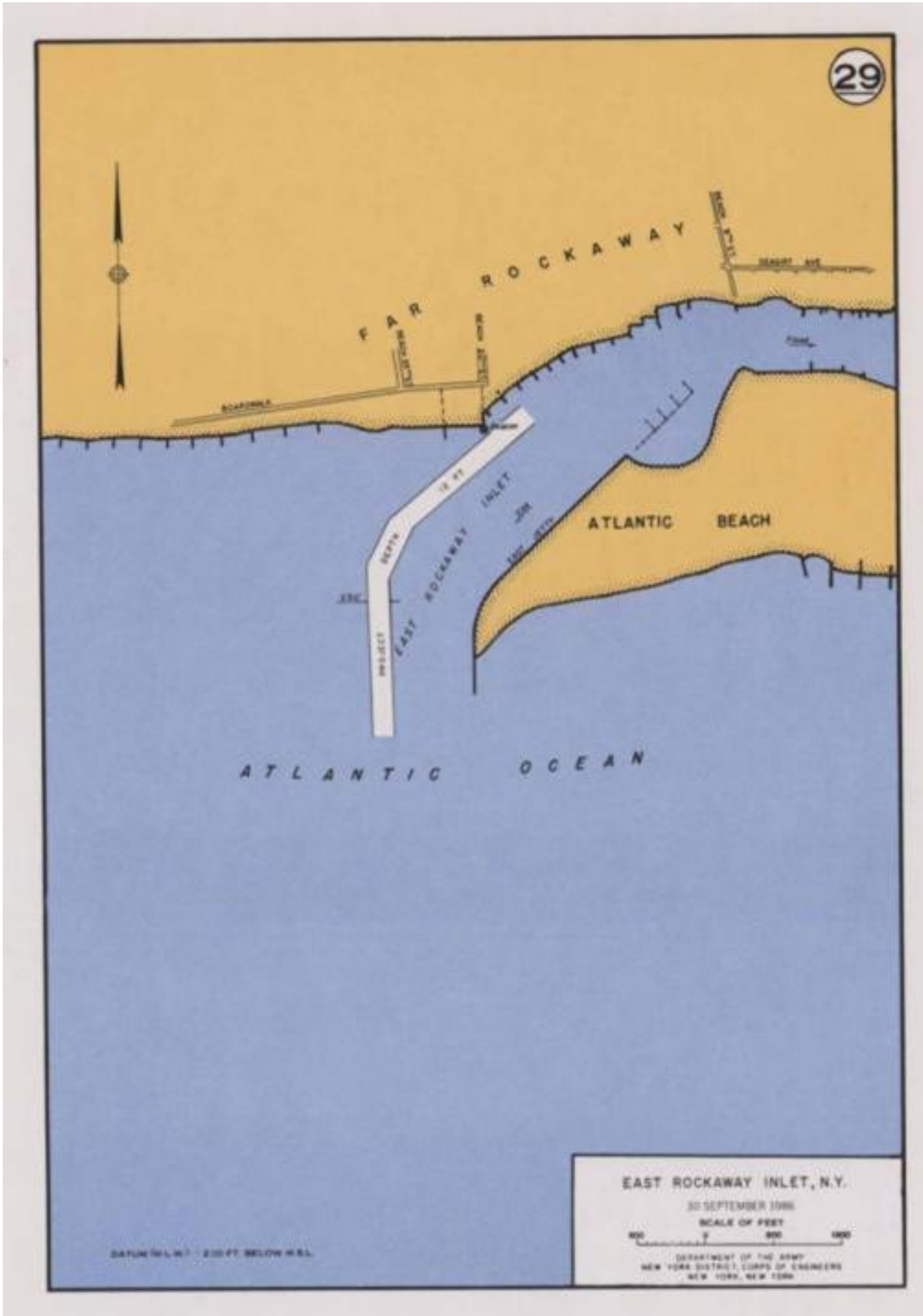


Figure 2-7: East Rockaway Inlet Channel

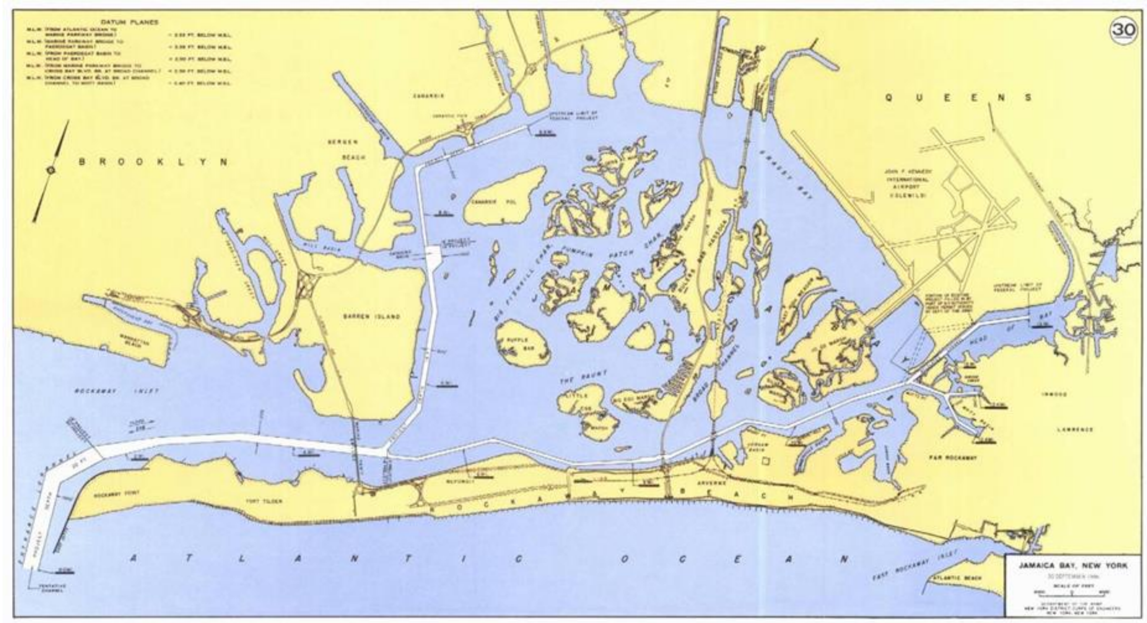


Figure 2-8: Rockaway Inlet Chanel

2.3.4 East Rockaway Inlet Channel

The East Rockaway Inlet Project was authorized by the Rivers and Harbors Act of 1930. The project allows navigation to proceed from the Atlantic Ocean into Reynolds Channel and the bays north of the Long Beach. The inlet provides for:

- A channel 12 ft. deep and approximately 250 ft. wide;
- One jetty constructed on the east side of the channel;
- One jetty (authorized but not constructed) on the west side of the channel.

Dredging records for East Rockaway channel show initial construction in 1935, maintenance dredging from 1938-1985, a channel realignment in 1988, and maintenance dredging from 1989 to present. Dredging intervals have historically varied from 1 year to 11 years, with recent dredging operations occurring 2 to 3 years apart. Table 2-5 gives the history of new work, defined as initial dredging to new authorized dimensions, and maintenance dredging. From 1938-1978 annual maintenance dredging volumes were approximately 32,500 cy/yr. A marked increase in the maintenance dredging volumes has occurred after 1979, with annual maintenance dredging volumes of 115,00 cy/yr. The increase in maintenance dredging volumes since 1979 is most likely due to the large amounts of beach fill placed on the Rockaway Peninsula since 1975, a portion of which migrates to the east into the channel.

Table 2-5: East Rockaway Inlet Channel Dredging History

FY	Depth (ft)	Reach	New Work (CY)	Maintenance (CY)
1935	12	Dredging Channel	171,230	
1938	12	Dredging Channel		75,963
1939	12	Dredging Channel		48,000
1941	12	Dredging Channel		42,967
1952	12	Dredging Channel		166,700
1954	12	Dredging Channel		56,600
1955	12	Dredging Realigned Channel		123,200
1956	12	Dredging Channel		48,700
1957	12	Dredging Channel		42,500
1958	12	Dredging Channel		20,300
1959	12	Dredging Channel		29,100
1960	12	Dredging Channel		4,868
1961	12	Dredging Channel		11,432
1962	12	Dredging Channel		7,900
1963	12	Dredging Channel		14,900
1964	14	Dredging Channel		26,298
1966	12	Channel		6,000
1967	12	Channel		8,650
1968	14	Channel		980
1968	12	Channel		24,880
1969	14	Channel		62,000
1970	12	Channel		53,485
1971	14	Channel		67,005
1972		Channel		52,024
1973	14	Channel		1,913
1973	14	Channel		22,952

FY	Depth (ft)	Reach	New Work (CY)	Maintenance (CY)
1974	12	Channel		46,714
1975	12	Channel		89,400
1976	12	Channel		71,930
1976	12	Channel		34,860
1977	14	Channel		23,482
1978	12	Channel Entrance		15,560
1979	12	Remove Shoals		241,100
1982	12	Offshore (12 +4) & Inshore Channels (18 +2)		197,743
1985	12	Channel		386,428
1988	12 (14+2)	Entire Channel, New Channel Alignment	1,006,000	
1989	12, 14	Inlet Channel		230,000
1990	12	East Rockaway Inlet		190,000
1991	14	Entrance		157,081
1993		Dredge		152,508
1996		Dredge		411,760
2002		Dredge		141,900
2004		Dredge		224,000
2007		Dredge		266,890
2009		Channel Maintenance		285,000
2010		Channel Maintenance		137,000
1938-1978 (CY/YR)				32,500
1979-2007 (CY/YR)				115,00
Total Maintenance 1938-2010 (CY)				4,323,000

2.3.5 Rockaway Inlet Channel

The Federal Navigation Maintenance Project of Jamaica Bay (Rockaway Inlet) was authorized by the Rivers and Harbor Acts of 1910 and subsequently modified by the Rivers and Harbors Acts of 1945 and 1950. Rockaway Inlet allows navigation to proceed from the Atlantic Ocean into Jamaica

Bay, splitting into access channels for both north and south sides of the Bay. The Project provides for:

- An entrance channel 20 ft. deep and approximately 1000 ft. wide extending from deep water in the Atlantic around Breezy Point;
- An 18-ft deep channel approximately 500 ft wide extending past the Marine Parkway Bridge;
- Northern and southern branch channels within Jamaica Bay;
- One rock jetty constructed on the east side of the entrance channel.

Dredging records for the Rockaway channel show initial construction in 1930. No maintenance dredging of the entrance channel occurred until 1976, after which records show regular maintenance of this portion of the channel. The lack of maintenance dredging until the 1976 is likely due to the impoundment capacity of the jetty at Rockaway Inlet. Once maintenance dredging began in 1976, dredging intervals varied from 1 year to 5 years. Table 2-6 provides a history of new work, defined as initial dredging to new authorized dimensions, and maintenance dredging for the channel. Maintenance dredge volumes have gradually increased over time and is likely due to the growth of the fillet and increasing bypassing around east jetty at Breezy Point. From 1976-2004 annual maintenance dredging volumes are 96,000 cy/yr.

Table 2-6: Rockaway Inlet Channel Dredging History

FY	Depth (ft)	Reach	New Work (CY)	Maintenance (CY)
1930	18	Inner Section of Entrance Channel	1,343,024	
1933		Filling in at Jetty	83,120	
1976		20 Foot Channel		89,696
1977		Entrance Channel		218,037
1979		20 Foot Channel		61,729
1981		Sea Leg of Channel - Vicinity Of Buoys		159,270
1985		Entrance Channel		181,685
1988		Entrance Channel		230,273
1991		Entrance		404,141
1992		Entrance		145,800
1994		Entrance		198,941
1996		Entrance		225,837
1998		Entrance		222,718
2002		Entrance		366,080
2004		Entrance		182,943
1976-2004 (CY/YR)				96,000
Total Maintenance 1930-2004 (CY)				2,687,000

3 PERFORMANCE OF PRIOR PROJECTS

3.1 WRDA 1974 Beach Erosion Control Project (1978-1988)

The multiple purpose beach erosion control and hurricane protection project was authorized by the Flood Control Act of 26 October 1965 in accordance with House Document No. 215, 89th Congress, First Session (USACE, 1993). It was then modified by Section 72 of the Water Resources Development Act of 6 March 1974, which authorized the separate construction of the beach erosion control portion. The project provided for the restoration of a protective beach along 6.2 miles of Rockaway Beach, between Beach 19th Street and Beach 149th Street (reaches 3, 4, 5 and 6). The project authorization also provided for Federal participation in the cost of periodic beach nourishment to stabilize the restored beach for a period not to exceed 10 years after the completion of the initial beach fill. A Post Authorization Change allowed the construction of 380-foot long quarry stone groin at the western limit of the project in the vicinity of Beach 149th Street in 1982.

The initial nourishment construction was completed in three contracts from 1975 to 1977. The authorized construction profile varied along Rockaway Beach and was comprised of a 100-foot berm between Beach 19th Street and Beach 126th Street, 150-foot berm between Beach 126th Street and Beach 110th Street, and 200-foot berm from Beach 110th Street to Beach 19th Street. The 150-foot and 200-foot berms were authorized based on separable recreation benefits. The storm damage reduction features of the authorized project consist only of a 100-foot berm width. The top of the berm elevation was constructed to +10 feet NGVD (+9 feet NAVD88). The constructed beach fill section was developed at a 1V:20H slope from +10 ft to -2 ft NGVD, and 1V:30H slope from -2 ft NGVD to closure. Figure 3-1 provides an overview of the authorized construction profile.

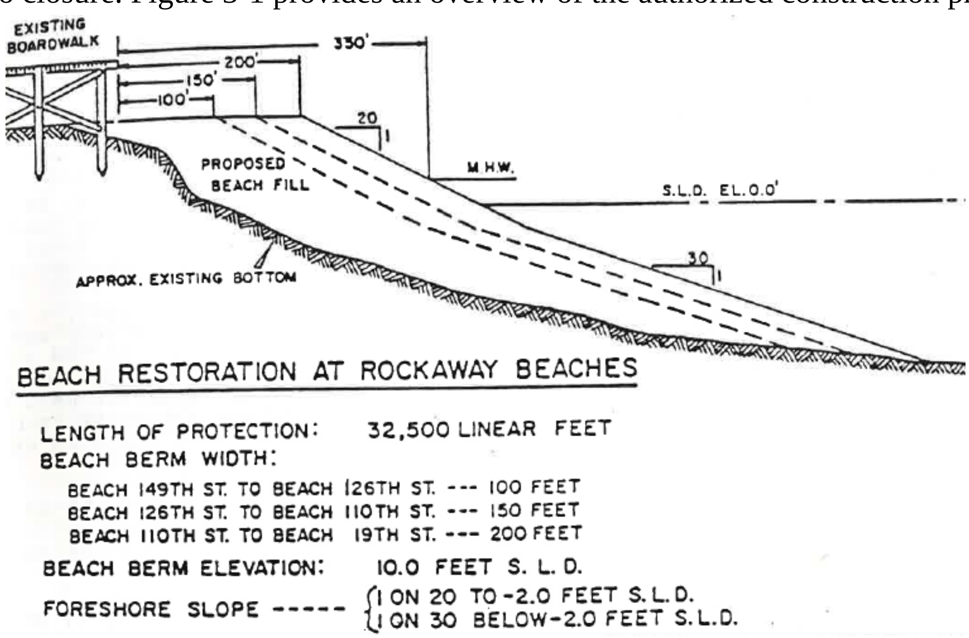


Figure 3-1: Authorized Construction Profile for Flood Control Act of 1965

Five renourishment operations and one emergency renourishment operation were performed over the 10 years following initial construction (Table 3-1). Renourishment operations occurred at two-year intervals and entailed constructing Feeder Beaches in the two most highly erosive areas in the project area:

- Western Feeder Beach (Reach 4) = ~5,700 feet
- Eastern Feeder Beach (Reach 6) = ~3,700 feet

The expectation was that the material would be eroded from those areas and would therefore supply, or feed, sand to the rest of the project area, thereby offsetting the long-term erosion.

During each renourishment operation, the beach along the designated Feeder Beaches was constructed to its construction profile dimension (200-foot berm). As shown in Table 3-1, this required placing on average 480,000 cy per operation in Reach 4 and 587,000 cy per operation in Reach 6. This is equivalent to placing 240,000 cy/yr and 293,500 cy/year in Reaches 4 and 6 respectively (533,500 cy/yr total). During each renourishment operation the shoreline at the feeder beach was extended seaward approximately 70 feet in Reach 4 and 130 feet in Reach 6. As anticipated, the material placed in the feeder beaches was quickly transported to other areas along The Atlantic Ocean Shorefront. The high erosion rates observed at the feeder beaches are attributed to the high background erosion rates in these areas as well as to beachfill diffusion (e.g. end losses). Beachfill diffusion is more pronounced when the alongshore length of the project is relatively short in comparison to the cross-shore width of beachfill, as was the case at the feeder beaches.

Monitoring of the shoreline positions between renourishment cycles showed that the authorized beach dimensions were not maintained along the project area (USACE, 1993). A total of 21 beach profile surveys were collected four years after the last renourishment event, 1988, and the existing beach dimensions were compared to the authorized beach dimensions. Only 3 of the 21 the profiles were close to meeting the design profile dimensions. The rest of the profiles had narrower beach dimensions than the authorized dimensions.

Table 3-1: WRDA 1974 Beachfill Events

Year	Description	Source of Fill	Reach	Total Fill Placed (cy)
1975	Initial Construction	Borrow area 1	Reach 4 & 5	3,669,000
1976	Initial Construction	Borrow area 3	Reach 6	1,490,000
1977	Initial Construction	Borrow area 2	Reach 3	1,205,000
1978	Emergency Renourishment	Borrow area 1	Reach 4	453,000
1978	Emergency Renourishment	Borrow area 3	Reach 6	231,000
1980	Renourishment 1	Borrow area 3	Reach 6	466,000
1982	Renourishment 2	Borrow area 2	Reach 4	414,000
1982	Renourishment 2	Borrow area 3	Reach 6	479,000
1982	Terminal Groin at B149th Street and fill	N/A	Reach 3	163,300
1984	Renourishment 3	Borrow area 2	Reach 4	828,000
1984	Renourishment 3	Borrow area 3	Reach 6	631,000
1986	Renourishment 4	Borrow area 2	Reach 4	654,000
1986	Renourishment 4	Borrow area 3	Reach 6	691,000
1988	Renourishment 5	Borrow area 2	Reach 4	501,000
1988	Renourishment 5	Borrow area 3	Reach 6	666,000

3.2 Section 934 Beach Erosion Control Project (1996-2004)

Additional erosion after the WRDA 1974 authorization expired led to a second major construction effort authorized through Section 934 of the Water Resources Development Act of 1986, which allowed continued Federal participation in periodic beach fill nourishment. Under this authority, a reevaluation report approved in May 1994 prescribed three additional nourishment cycles occurring three years apart although actual renourishment operations occurred four years apart. Initial construction was completed in 1996 and two renourishment operations occurred in 2000 and 2004 (Table 3-2). Due to the high cost of construction and continued nourishment, the New York District was directed in 2003 to reformulate the original plan, with the objective of finding a long term, cost-effective solution to the effects of continued erosion on the Rockaway peninsula. The eroded conditions of the beach in 1994, about two years before initial construction, are shown in Figure 3-2. The beach width is particular narrow in Reaches 4 and 6a. The shoreline had retreated all the way back to the boardwalk in Reach 6a.

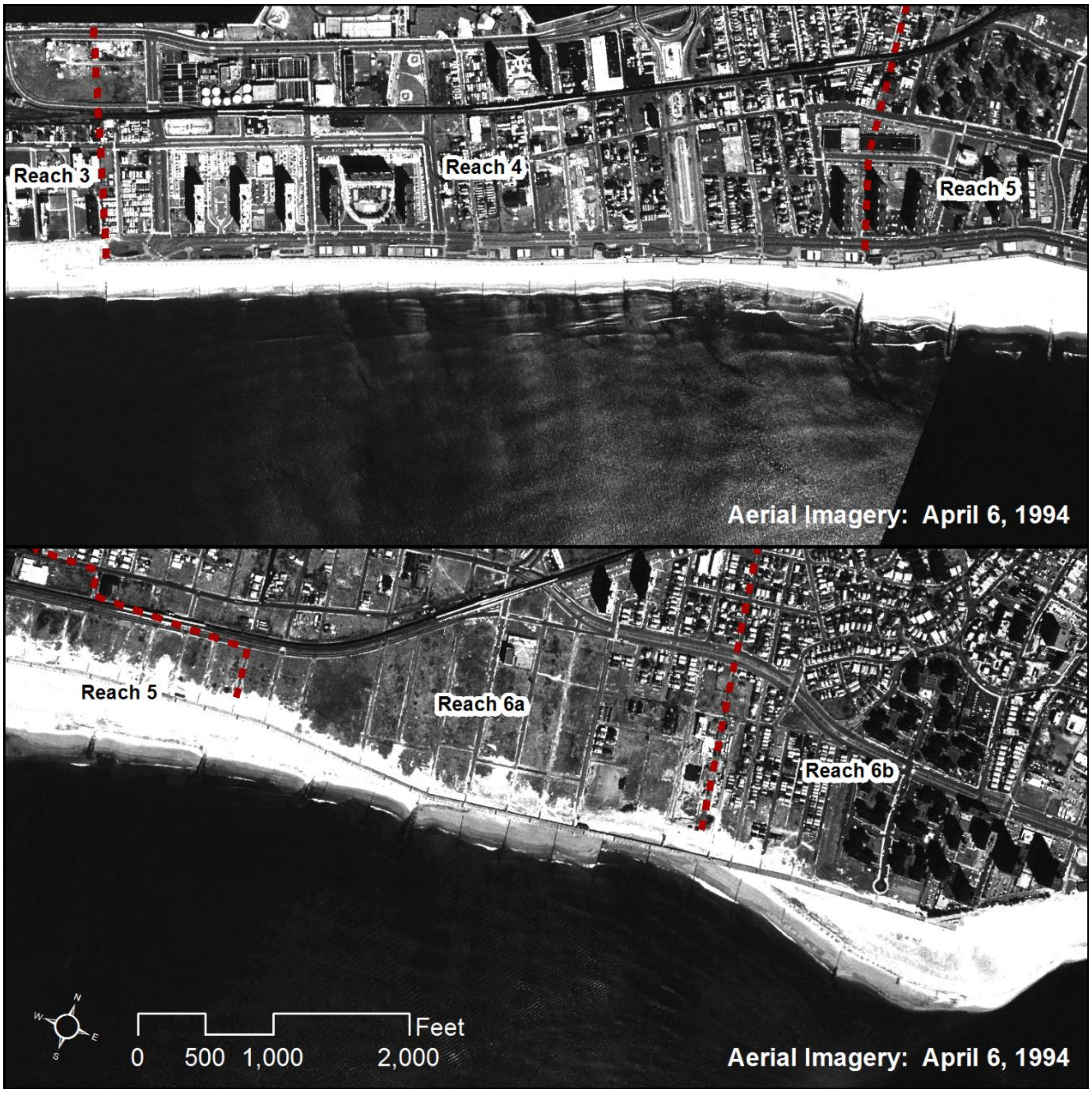


Figure 3-2: Erosional Hot Spots

The construction profile dimensions for the Section 934 Beach Erosion Control Project are the same as the WRDA 1974 Project except that all the berm widths are 100 feet. The 150 foot and 200 foot berm widths that were originally justified based on recreational benefits in the WRDA 1974 Project were not included in the Section 934 Project.

The Section 934 Project took a different approach to renourishment. Instead of relying on the feeder beaches to supply sediment to the entire project area and offset long-term erosion, the Section 934 Project placed renourishment along the entire project area during each renourishment operation. In addition, the Section 934 Project included advance fill in initial construction and renourishment operations. Advance fill is a sacrificial quantity of sand which acts as an erosional buffer against long-term and storm-induced erosion with the goal of preventing erosion of the design profile between renourishment cycles.

A summary of the Section 934 Project beachfill operations is provided in Table 3-2. Inlet maintenance dredging operations also occurred over the Section 934 Project period and a summary of these operations is included below in Table 3-2. During each renourishment the beach was restored to its authorized dimension plus advance fill. Including inlet maintenance dredging operations at total of 354,000 cy/year was placed in the project area in the eight years after initial construction between 1996 and 2004.

Table 3-2: Section 934 Beachfill Events

Year	Description	Source of Fill	Reach	Total Fill Placed (cy)
1996	Initial Construction	Borrow area 1B	Reach 4 & 5	1,485,800
1996	Initial Construction	Borrow area 1B	Reach 6	606,400
1996	Initial Construction	Borrow area 1B	Reach 3	592,800
1998	Inlet Maintenance Dredging	E. Rock Inlet	Reach 6	218,000
2000	Inlet Maintenance Dredging	E. Rock Inlet	Reach 6	241,000
2000	Renourishment 7	Borrow area 2	Reach 4 & 5	1,009,288
2002	Inlet Maintenance Dredging	E. Rock Inlet	Reach 6	140,000
2004-2005	Inlet Maintenance Dredging	E. Rock Inlet	Reach 6	221,002
2004	Renourishment 8	Borrow area 2	Reach 3	94,968
2004	Renourishment 8	Borrow area 2	Reach 4	634,557
2004	Renourishment 8	Borrow area 2	Reach 6	271,953

3.3 Section 934 Beach Profile Surveys

3.3.1 Data Availability

Atlantic Coast of New York Monitoring Program (ACNYMP) profiles from 1997 to 2001 are available at twelve locations along Rockaway Beach, NY. An overview of the profile locations is provided in Figure 3-3.

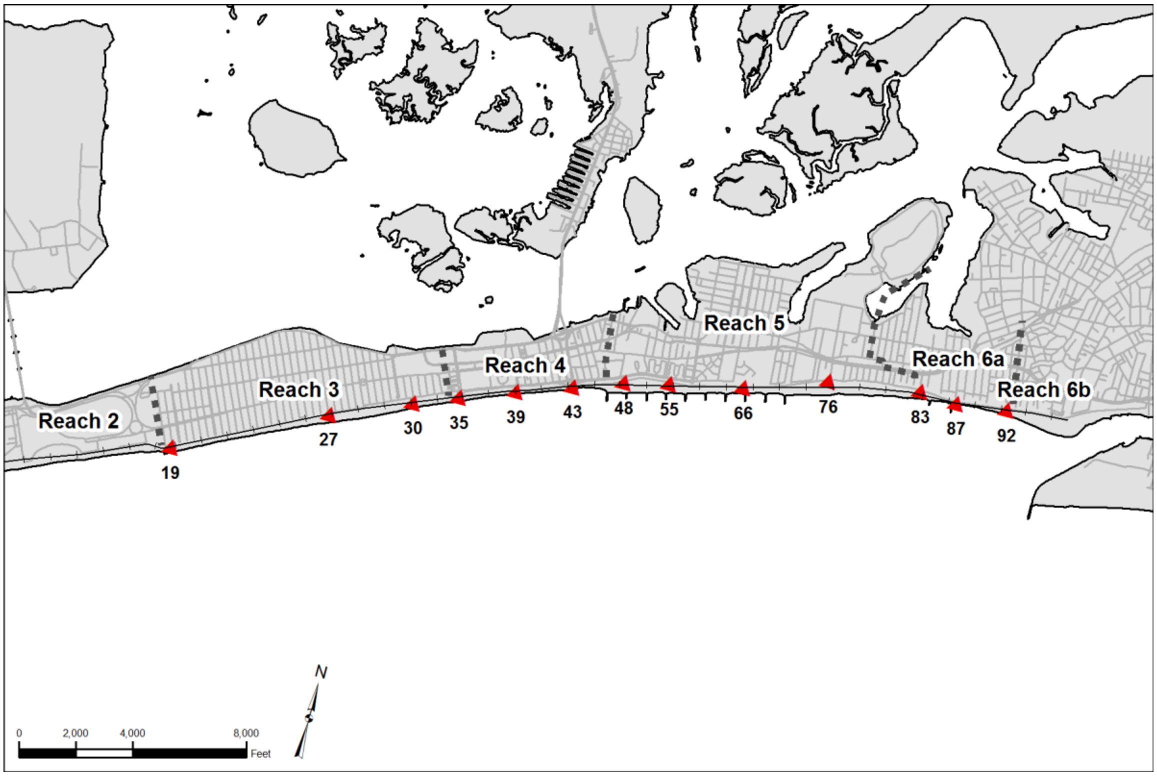


Figure 3-3: ACNYMP Locations

Profile availability over this time period is summarized in Table 3-3. The focus of this analysis is on spring profile surveys in 1997, 1998, and 2000. These profiles are used to quantify the performance of the Section 934 Project between initial construction and the first renourishment operation in 2000. The history of beachfill placement for the Section 934 Project is shown in Table 3-2. It is noted that the spring (S) 1997 profile was surveyed shortly after initial construction and the S2000 profile was surveyed prior to the first renourishment operation (Renourishment 7). The survey data from 1997 to 2000 captures a period without any beachfill operations. The only exception is Reach 6a, which received 218,000 cubic yards (cy) in the fall of 1998 as part of inlet maintenance dredging of East Rockaway Inlet.

Table 3-3: Profile Survey Availability

Reach	Profile	Station	S1997	F1997	S1998	F1998	S2000	S2001
Reach 3	19	319+00	x	x	x	x	x	x
Reach 3	27	262+00	x	x	x	x	x	x
Reach 3	30	232+00	x	x	x	x	x	x
Reach 4	35	216+00	x	x	x	x	x	x
Reach 4	39	195+00	x	x	x	x	x	x
Reach 4	43	175+00	x	x	x	x	x	x
Reach 5	48	157+00	x	x	x	x	x	x
Reach 5	55	141+00	x	x	x	x	x	x
Reach 5	66	115+00	x	x	x	x	x	x
Reach 5	76	85+00	x	x	x	x		x
Reach 6a	83	53+00	x	x	x		x	x
Reach 6a	87	40+00	x	x	x	x	x	x
Reach 6a	92	22+00	x	x	x	x		x

3.3.2 Volumetric Changes

Volumetric changes from S1997 to S1998 and S2000 are calculated at each profile location. Changes are computed over the full extent of the overlapping profiles (beyond depth of closure, -25 ft NAVD88, if possible). Table 3-4 presents the net volumetric changes (cy/ft) at each profile location. As mentioned earlier, the profiles in Reach 6a show an increase in volume between S1997 and S2000 because of beachfill placed as part of inlet maintenance dredging in the fall of 2008. Figure 3-4 and Figure 3-5 show ACNYMP profile surveys in the S1997, S1998, and S2000 at Profile 30 and 35.

Table 3-4: Volumetric Changes (cy/ft)

Reach	Profile	S1997-S1998	S1997-S2000
Reach 3	19	-7.6	8.1
Reach 3	27	-9.0	-3.8
Reach 3	30	-18.1	-43.6
Reach 4	35	-73.8	-125.9
Reach 4	39	-62.0	-76.8
Reach 4	43	-4.4	-63.5
Reach 5	48	-8.0	-55.8
Reach 5	55	-0.1	-36.5
Reach 5	66	6.2	-22.6
Reach 5	76	-42.9	
Reach 6a	83	-16.2	20.8
Reach 6a	87	5.2	2.9
Reach 6a	92	-6.6	

Note: Net Volumetric Changes (e.g. reflect volume added in Reach 6a as part of inlet maintenance dredging).

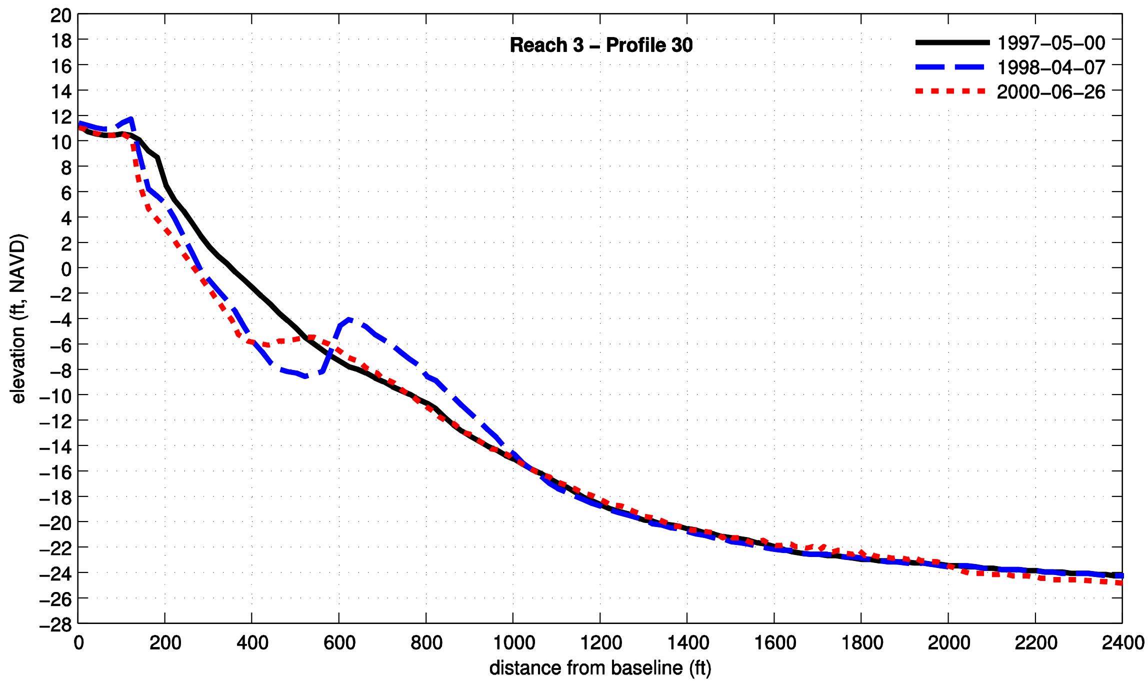


Figure 3-4: ACNYMP Data at Profile 30

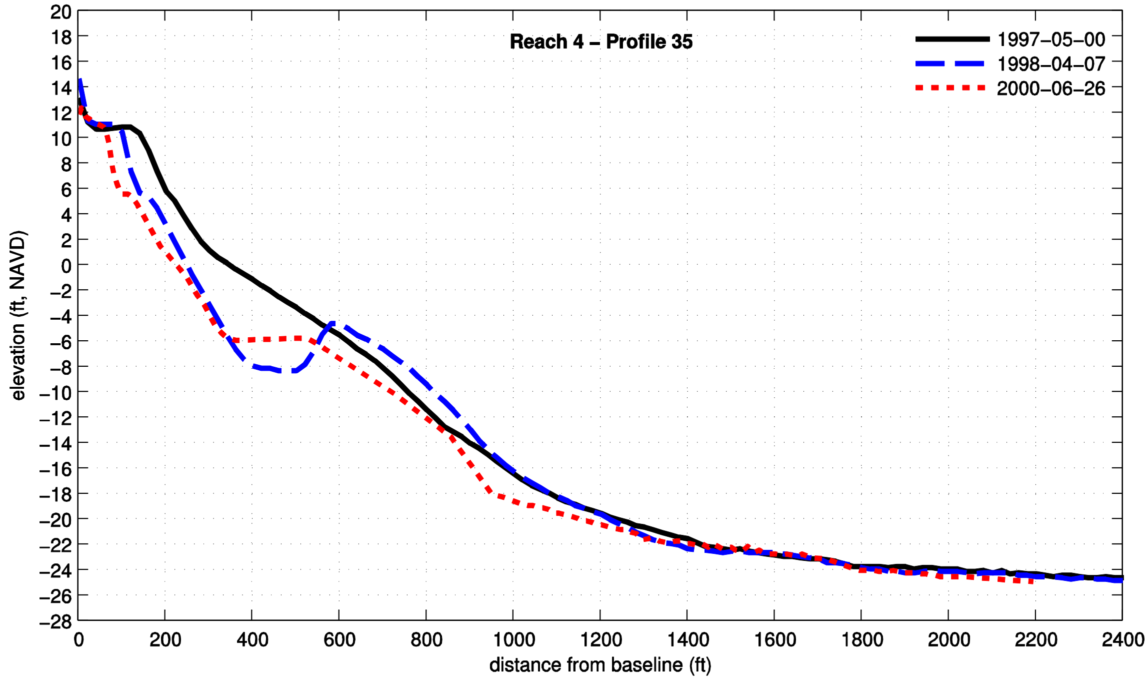


Figure 3-5: ACNYMP Data at Profile 35

Average end area calculations are performed based on the volumetric changes at each profile station to determine the reach-averaged changes. Volumetric changes are converted to equivalent shoreline changes based on the active height of the beach profile (33 feet). The equivalent shoreline changes presented in Table 3-5 represent the "gross" changes because the 1998 inlet maintenance dredging event (218,000 cy) has been subtracted from the reach-averaged volume changes in Reach 6a.

Table 3-5: Summary of Shoreline Changes (ft/yr)

Reach	Length (ft)	ACNYMP S1997-S1998	ACNYMP S1997-S2000
Reach 1	11,900		
Reach 2	11,100		
Reach 3	10,200	-12.7	-4.5
Reach 4	5,600	-35.3	-22.7
Reach 5	10,800	-11.2	-10.6
Reach 6a	3,600	-3.4	-14.5
Reach 6b	2,000		
Project Area	30,200	-15.3	-11.3

Note: Gross shoreline changes (e.g. inlet maintenance dredging event subtracted from Reach 6a)

3.4 Summary

A comparison of beachfill quantities for the WRDA 1974 and Section 934 Erosion Control Projects is provided in Table 3-6. Before comparing the two projects it is important to acknowledge the limitations of such a comparison:

- Neither project was successful at maintaining the design shoreline between renourishment operations. Also, it is not clear if one project provided a wider beach between renourishment operations than the other.
- The interval of renourishment operations for the two projects was different. It is likely that the shorter renourishment interval in the WRDA 1974 Project resulted in greater beachfill diffusion.
- The wave climate may be considerably different from year to year, resulting in very different erosion rates from year to year. It is possible that one project may have experienced a more energetic wave climate than the other.

Despite these limitations some observations are made about the two projects. The renourishment quantities in the WRDA 1974 Project are 50 percent higher than the Section 934 Project. The high renourishment quantities during the WRDA 1974 Project are attributed to high beachfill diffusion associated with the feeder beaches and shorter renourishment interval. The renourishment approach in the Section 934 Project, nourishing the entire project area as needed, may have led to a decrease beachfill diffusion.

Table 3-6: Comparison of WRDA 1974 and Section 934 Renourishment Operations

Reach	Beachfill Quantity (cy/year)	
	WRDA 1974[1]	Section 934[2]
Reach 3		12,000
Reach 4	239,700	142,500
Reach 5		63,000
Reach 6	293,300	136,500
Total	533,000	354,000

Notes: [1] A total of 5 renourishment operations occurred in 2-year intervals from 1980-1988

[2] A total of 2 renourishment operations occurred in 4-year intervals from 1990-1994

4 EXISTING CONDITIONS

4.1 General

Existing conditions for the Rockway Shorefront were analyzed and documented prior to developing Erosion Control and CSRM alternatives.

Vertical elevations (EL.) of project components and features are referenced to the North American Datum of 1988 (NAVD88), Geoid12A vertical reference system. All elevations throughout the report are referenced to NAVD88 Geoid12A and presented in feet unless otherwise stated.

The horizontal datum shall be the North American Datum of 1983 (NAD83) Long Island, New York State Plane with units in feet, unless specifically noted otherwise.

4.2 Astronomical Tides

Daily tidal fluctuations at the project site are semi-diurnal, with two highs and two lows per 24-hour day. The tidal datum relationships on the ocean side and bay side are presented in Table 4-1.

Table 4-1: Tidal Datum Relationships

Datum	Ocean (ft, NAVD88)	Bay (ft, NAVD88)
MHHW	2.35	2.71
MHW	2.02	2.36
NAVD88	0.00	0.00
MSL	-0.22	-0.23
NGVD29	-1.11	-1.11
MLW	-2.54	-2.84
MLLW	-2.73	-3.04

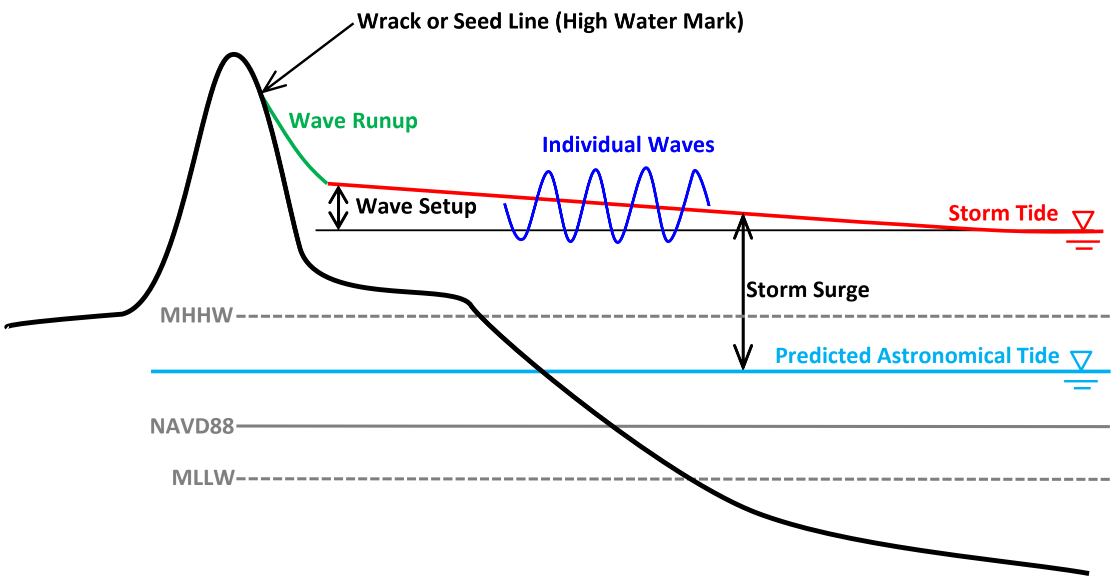
Notes: Tidal datums based on NOAA's VDATUM 1983-2001 Epoch

4.3 Storm Tides

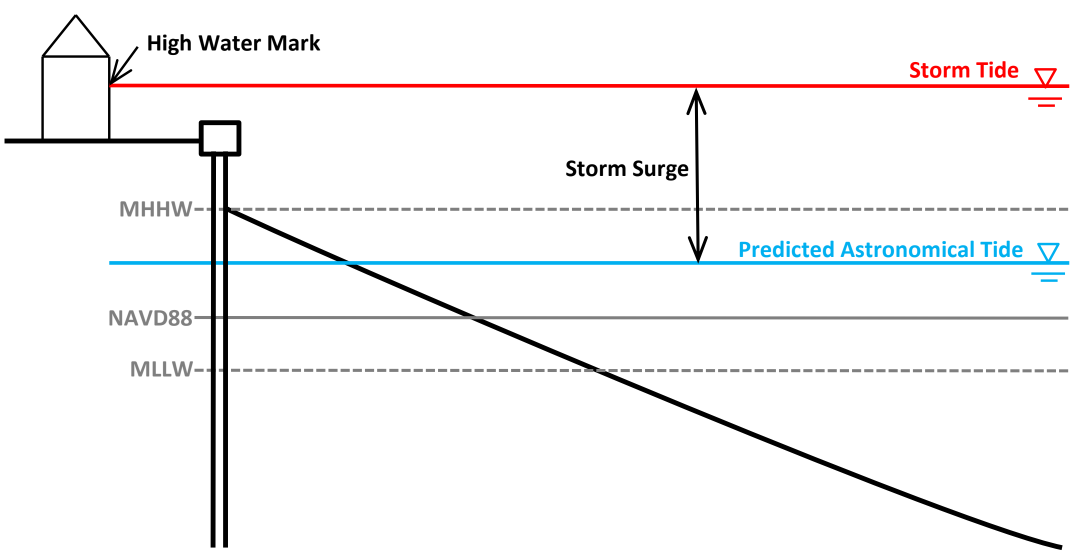
Storm tide is the total observed water level during a storm due to the combination of storm surge and astronomical tide. Storm surge is defined as a rise above normal water level on the open coast due to the action of wind stress on the water surface and the decrease in atmospheric pressure during major storms. Water levels rise at the shoreline when the motion of wind driven waters is arrested by the coastal landmass. Two types of storms, as mentioned in section 2.1) are of primary significance along in the Project Area: (1) tropical storms which typically impact occur from July to October, and (2) extratropical storms which are primarily winter storms occurring from October to March. These extratropical storms are often referred to as "nor'easters" due to the predominate direction from which the winds originate. Figure 4-1 shows the different contributions to the water surface elevation along an exposed coastline with waves (panel a) and along a sheltered coastline with little or no waves (panel b). The storm tide elevation does not include temporary fluctuations in the water surface elevation from individual waves or wave runup.

Several storm surge modeling studies have been completed for the region that may be used to define the storm tide frequency of occurrence relationships (i.e. stage frequency curves) in the Project Area. The storm surge studies reference here are:

- Camp Dresser McKee (CMD, 1981)
- Fire Island to Montauk Point (USACE-NAN, 2005)
- FEMA Flood Insurance Study (FEMA, 2013)
- North Atlantic Comprehensive Study (NACCS, 2015)



(a) Shoreline Exposed to Storm Tide and Waves



(b) Shoreline Exposed to Storm Tide (no waves)

Figure 4-1: Components of Water Surface Elevation

The stage frequency curves from FEMA (2013) were adopted and used to define the ocean and back bay stage frequency curves in the Project Area. Previous work on the Rockaway Reformulation Study used the FIMP modeling results to define the ocean-side stage frequency curves (USACE-NAN, 2005). The back bay stage frequency curves were previously based on modeling results from Camp Dresser McKee study (CDM, 1981). Preliminary FEMA stage frequency curves were released in the summer of 2013 and represent the best available information and science to date. The preliminary FEMA results indicate that the extreme water levels on both the ocean and bay-side of Rockaway are significantly higher than previously predicted by the FIMP and CDM studies (for stage frequency elevations up the 100 year return period).

Table 4-2: Ocean Stage Frequency Elevations (ft, NAVD88)

Return Period, yr	NACCS (2015)	FEMA (2013)	FIMP (2005)	CDM (1981)
5	6.0	5.3	4.9	
10	6.6	6.5	5.9	6.2
25		7.9	7.2	
50	8.3	9.1	8.1	7.9
100	9.6	10.5	9.2	8.6
200	11.3	12.2	10.1	9.6
500	13.5	14.7	11.2	11.1

FIMP Station 52, CDM Station 8, FEMA Station "FEMA - Offshore", NACCS Station 3917

Table 4-3: Bay Stage Frequency Elevations (ft, NAVD88)

Return Period, yr	NACCS (2015)	FEMA (2013)	FIMP (2005)	CDM (1981)
5	6.5	5.5	5.3	
10	7.2	6.6	6.2	4.7
25		7.9	7.4	
50	8.9	8.8	8.6	6.0
100	9.8	9.8	10.1	6.7
200	11.0	10.8	11.3	7.5
500	12.8	12.3	12.7	8.6

FIMP Station 61, CDM Station 110, FEMA at CDM Station 110, NACCS Station 3992

4.4 Sea Level Rise

By definition, sea level rise (SLR) is an increase in the mean level of the ocean. Eustatic sea level rise is an increase in global average sea level brought about by an increase to the volume of the world's oceans (thermal expansion). Relative sea level rise takes into consideration the eustatic increases in sea level as well as local land movements of subsidence or lifting. Two NOAA gages are available near the Project site; the Battery, and Sandy Hook. Both gages similar distances from the project site, i.e. approximately 12-15 miles. However, the Sandy Hook gage and the project area are more similar as they are located in the Coastal Plain geologic formation, whereas the

Battery gage is located on different geologic formations. Land subsidence is estimated at -2.17 mm/yr and -1.22 mm/yr at the Sandy Hook and Battery gages respectively. For comparison, Montauk Point, at the eastern end of Long Island, has an estimated vertical land movement of -1.23 mm/ yr (NOAA, 2013). Direct estimates of vertical land subsidence for the project area are unavailable. Regionally, sea level rise for New York, Connecticut, and New Jersey range from 2.10 mm/yr at New London, CT to 3.97 mm/yr at Atlantic City, NY, with Sandy Hook at 3.85 mm/yr (Gornitz at al. 2002). The Sandy Hook gage was chosen to represent sea level rise at the project site as the most appropriate available gage.

Historic information and local MSL trends used for the Study Area are provided by the NOAA/NOS Center for Operational Oceanographic Products and Services (CO-OPS) using the tidal gauge at Sandy Hook, New Jersey. The historic sea level rise rate (1935-2013) is approximately 0.0128 feet/year or about 1.3 feet/century.

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

The current guidance (ER 1100-2-8162) from the Corps states that proposed alternatives should be formulated and evaluated for a range of possible future local relative sea level rise rates. The relative sea level rates shall consider as a minimum a low rate based on an extrapolation of the historic rate, and intermediate and high rates which include future acceleration of the eustatic sea level rise rate. These rates of rise correspond to 0.7 ft, 1.1 ft, and 2.5 ft over 50 years (2018-2068) for the low, intermediate and high rates of relative sea level rise.

4.5 Waves

The offshore wave conditions for the Project Area are based on WISWAVE modeling results completed as part of the Fire Island to Montauk Point (FIMP) Reformulation Study (USACE-NAN, 2005). WISWAVE model simulations were performed for 36 historical storm events. A detailed discussion of the storm suite is provided in the Engineering Modeling Appendix (Sub-Appendix A). WISWAVE modeling results at station I07J04, located approximately 17 miles offshore of Rockaway at a water depth of 76 ft, were extracted and used to develop the offshore design wave conditions. Wave conditions during Hurricane Irene and Hurricane Sandy were added to the historical storm events based on observed wave heights at NOAA NDBC Buoy 44065.

Nearshore wave conditions are based on local wave modeling (STWAVE and SWAN) simulations for the historical storm events. Nearshore wave conditions are defined based on the modeling results at three locations along the Project Area at a water depth of 36 feet. Figure 4-2 shows the location of the nearshore and offshore wave stations.

Offshore and nearshore wave height frequency of occurrence relationships were developed using the Empirical Simulation Technique (EST) based on the peak wave heights for the historical storm events and probability of storm occurrence. A detailed description of the EST is provided in the Engineering Modeling Appendix (Sub-Appendix A. The results of the EST analysis, wave height frequency of occurrence relationships, are provided in Table 4-4.

Table 4-4: Offshore and Nearshore Wave Conditions

Return Period, yr	Offshore Significant Wave Height (ft)	Nearshore Significant Wave Height (ft)
2	14.8	7.7
10	22.1	15.3
25	24.8	18.6
50	27.3	21.4
100	30.7	23.9
200	35.4	26.3
500	41.6	29.3

Notes: Nearshore Wave Height at nominal depth of 36 ft

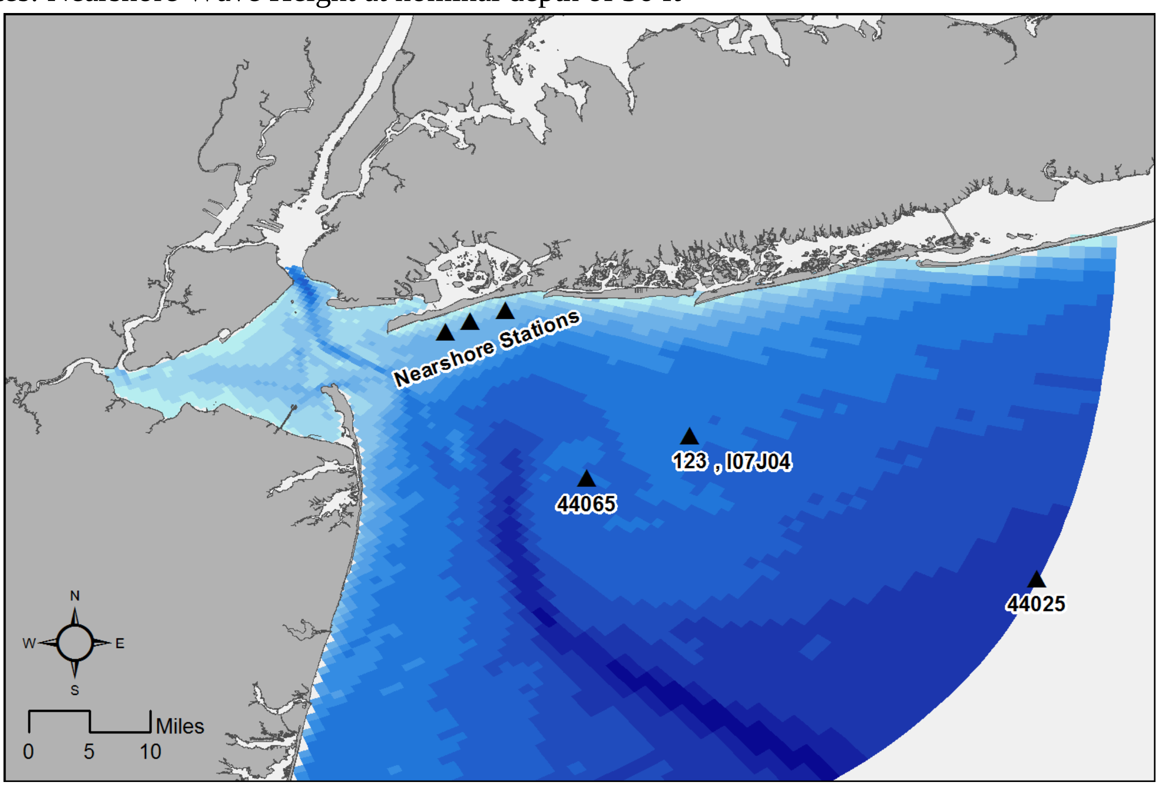


Figure 4-2: Location of Wave Stations

4.6 Wave Setup

Wave setup is characterized by a superelevation of the water surface in the surfzone as the result of wave breaking. As waves begin to break, the onshore-directed radiation stress, S_{xx}, decreases, resulting in onshore-directed forces. These forces are balanced by a shoreward slope in the water surface (wave setup). Wave setup for this study was calculated based on SBEACH model simulations of the Historical Storm Suite and Multivariate EST. The wave setup shown in Table 4-5 represents the wave setup contribution to the joint probability of wave setup and storm surge.

Table 4-5: Wave Setup

Return Period, yr	Wave Setup (ft)
2	1.4
5	2.0
10	2.3
25	2.6
50	3.1
100	4.0
200	4.4
500	4.4

In this study the wave setup value near the shoreline was extracted from the SBEACH model simulations. Figure 4-3 shows the cross-shore location at which the wave setup is extracted from SBEACH. The wave setup extracted from the cross-shore location of the intersection of the offshore water elevation and initial profile. This location varies with the offshore boundary conditions to SBEACH; however, this location is generally near the instantaneous shoreline. This is the same location that was used to determine the nearshore wave setup for FIMP (Gravens et al, 1999).

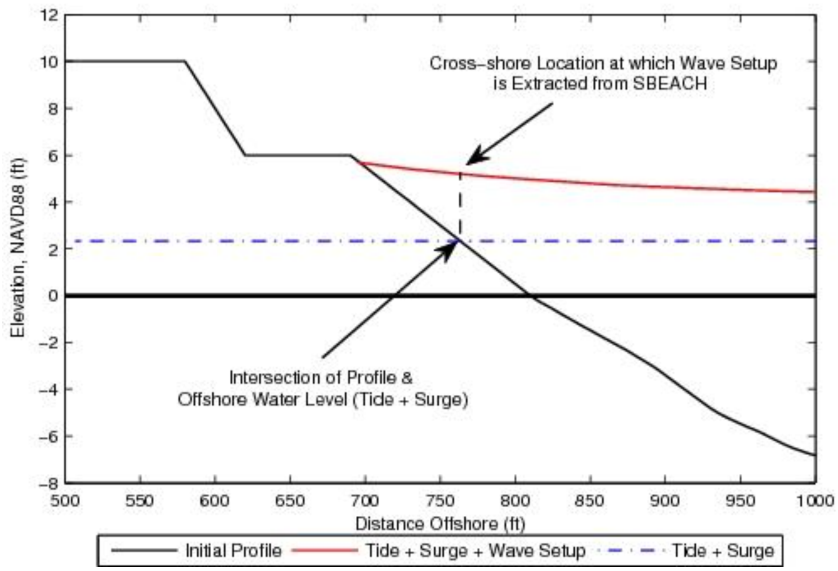


Figure 4-3: Cross-shore Location at which Wave Setup is extracted from SBEACH

4.7 Structural Condition

A coastal structures condition survey was conducted in spring of 2009, 26th to 27th of May. The condition survey was focused on determining the existing condition of shore protection structures, including groins, retaining walls, and remnants of timber groins (shown as pile groups). For rock groins, the crest elevation, width, side slope, armor size, core stone condition are inspected and documented. For wood groins (timber sheet piles), the elevation and existing conditions are inspected. Groin effectiveness in terms of updrift fillet formation and updrift/downdrift shoreline condition are inspected. The results of field survey/inspection are summarized in Table 2-15.

Based on the table, the majority of the timber groins are either buried or deteriorated and are no longer functioning effectively. All stone groins constructed after 1943 are generally in fair to good condition and are effective in trapping littoral material. The concrete retaining walls are generally in good condition. There is no evidence that the coastal structures are under periodic maintenance.

Table 4-6: Coastal Structures Condition Survey Summary

East Rockaway Reformulations Coastal Structures Condition Survey																	
Structures 1-43 surveyed 5/26/09, 44-103 surveyed on 5/27/09																	
									Stone Structures				Sheetpile		Updrift Fillet		Retaining Wall
Structure Number (field assigned)	Type	Street No	Construction Material	Side Slope (1v: h)	Crest Width (ft)	Width Crest + sides (ft)	Connection with wall, bulkhead, revetment?	Overall Structure Condition	Armor Diam. (ft)	Armor Unit Interlocking	Armor Stone Cracked?	Core Stone Visible btwn Armor units?	Sheetpile Type	Sheetpile Condition	Updrift Fillet Present?	Beach width offset (ft)	Wall Height above Grade (ft)
1	groin	B34	wood	na	na	na	none	nonfunctional					wood	nonfunctional	none	0	
2	groin	B34	wood	na	na	na	none	nonfunctional					wood	nonfunctional	none	0	
3	groin		stone	sides buried	10-15	20-25	none	good	4.0	good	intact	no			west side	25	
4	groin		stone	undiscernable	no flat crest	25	none	nonfunctional	4.0	nonfunctional	intact	no			none	0	
5	groin		stone	sides buried	15-20	25	none	good	4.5	good	intact	no			east side	20	
6	groin		stone	1.5	15-20	20-25	none	good	4.5	good	intact	no			none	0	
7	groin		stone	undiscernable	no flat crest	25	none	nonfunctional	4.5	nonfunctional	intact	no			none	0	
8	groin		stone	1.5	10-15	20-25	none	excellent	4.5	excellent	intact	no			east side	20	
9	single pile, pile group		wood	na	na	na	none	nonfunctional							na		
10	groin		stone	1.5	10-15	20-25	none	excellent	4.5	excellent	intact	no			west side	5	
11	single pile, pile group		wood	na	na	na	none	nonfunctional							na	na	
12	vehicle accessway	B64	cut under bdwk	na	na	na	boardwalk	good							na		
13	groin		stone	1.5	10-15	20-25	none	good	4.5	good	intact	no			east side	5	
14	single pile, pile group		wood	na	na	na	none	nonfunctional							na		
15	groin		stone	1.5	10-15	20-25	none	good	4.5	good	intact	no			east side	5	
16	vehicle accessway	B68	wood	na	na	na	boardwalk	good							na		
17	groin		stone	1	10-15	20	none	fair	4.5	fair	intact	no			west side	10	
18	single pile, pile group		wood	na	na	na	none	nonfunctional							na		
19	groin		stone	1.5	10-15	25	none	good	4.5	good	intact	no			west side	10	
20	single pile, pile group		wood	na	na	na	none	nonfunctional							na		
21	groin		stone	1.5	15-20	25	none	good	4.5	good	intact	yes			east side	10	
22	groin		stone	1.5	15-20	25	none	fair	4.5	fair	intact	no			none	0	
23	groin		stone	2	10-15	20-25	none	excellent	4.5	good	intact	no			west side	20	
24	groin		stone	1.5	10-15	25	none	excellent	4.5	good	intact	no			east side	75	
25	groin		wood	na	na	na	none	poor					wood	poor	none	0	
26	handicap access ramp	B91st	wood	na	na	na	boardwalk	good							na	na	
27	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
28	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
29	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
30	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
31	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
32	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
33	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
34	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
35	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
36	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
37	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
38	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
39	single pile, pile group		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
40	retaining wall	B130	cmu	na	na	na	na	good							na	na	3
41	retaining wall	B140	concrete	na	na	na	na	good							na	na	4
42	retaining wall	B149	concrete	na	na	na	na	good							na	na	4
43	groin	B149	stone	2	20-25	40	retaining wall	good	5.0	good	intact	no			east side	100	4
44	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
45	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
46	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
47	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
48	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
49	groin		wood	na	na	na	none	nonfunctional					wood	poor	none	0	
50	single pile, pile group		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
51	retaining wall		concrete	na	na	na	na	good							na	na	4

Structure Number (field assigned)	Type	Street No	Construction Material	Side Slope (1v: h)	Crest Width (ft)	Width Crest + sides (ft)	Connection with wall, bulkhead, revetment?	Overall Structure Condition	Stone Structures: Armor Diam. (ft)	Stone Structures: Armor Unit Interlocking	Stone Structures: Armor Stone Cracked?	Stone Structures: Core Stone Visible btwn Armor units?	Sheetpile: Sheetpile Type	Sheetpile: Sheetpile Condition	Updrift Fillet: Updrift Fillet Present?	Updrift Fillet: Beach width offset (ft)	Retaining Wall: Wall Height above Grade (ft)
51	retaining wall		concrete	na	na	na	na	good							na	na	4
52	retaining wall		concrete	na	na	na	na	good									5
53	vehicle accessway		concrete	na	na	na	na	good									
54	groin		combination	na	na	na	none	indeterminate					steel	indeterminate	none	0	
55	retaining wall		concrete	na	na	na	promenade	indeterminate									0.25
56	groin		combination	na	na	na	retaining wall	fair							east side	20	1
57	groin		wood	na	na	na	retaining wall	poor							none	0	
58	groin		combination	na	na	na	retaining wall	fair					concrete	indeterminate	east side	1-2' vert	
59	groin		wood	na	na	na	none	poor					wood	poor	none	0	
60	groin		wood	na	na	na	none	poor					wood	poor	none	0	
61	groin		wood	na	na	na	none	poor					wood	poor	none	0	
62	groin		wood	na	na	na	retaining wall	poor					wood	poor	none	0	
63	groin		wood	na	na	na	none	poor					wood	poor	none	0	
64	groin		wood	na	na	na	none	poor					wood	poor	none	0	
65	groin		wood	na	na	na	none	poor					wood	poor	none	0	
66	vehicle accessway	B169	concrete	na	na	na	retaining wall	good									
67	groin		stone	1.5	no flat crest	15	none	fair	4	fair	hairline cra	no			none	0	
68	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
69	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
70	groin		stone	1.5	no flat crest	10-20	none	fair	4	fair	hairline cra	no			none	0	
71	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
72	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
73	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
74	groin		stone	1.5	no flat crest	15	none	fair	4	fair	hairline cra	no			none	0	
75	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
76	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
77	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
78	groin		stone	1.5	no flat crest	10	none	fair	4	fair	hairline cra	no			none	0	
79	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
80	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
81	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
82	groin		stone	1.5	no flat crest	10	none	fair	4	fair	hairline cra	no			none	0	
83	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
84	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
85	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
86	groin		wood	na	na	na	none	nonfunctional					wood	indeterminate	none	0	
87	groin		stone	1.5	no flat crest	15	none	fair	4	fair	hairline cra	no			none	0	
88	groin		wood	na	na	na	defunct bulkhead	nonfunctional					wood	indeterminate	none	0	
89	groin		wood	na	na	na	defunct bulkhead	nonfunctional					wood	indeterminate	none	0	
90	groin		wood	na	na	na	defunct bulkhead	nonfunctional					wood	indeterminate	none	0	
91	groin		wood	na	na	na	defunct bulkhead	nonfunctional					wood	indeterminate	none	0	
92	groin		stone	1.5	no flat crest	15	none	good	4	good	hairline cra	no			none	0	
93	groin		wood	na	na	na	defunct bulkhead	nonfunctional					wood	indeterminate	none	0	
94	groin		wood	na	na	na	defunct bulkhead	nonfunctional					wood	indeterminate	none	0	
95	groin		wood	na	na	na	defunct bulkhead	nonfunctional					wood	indeterminate	none	0	
96	groin		stone	1.5	no flat crest	15	defunct bulkhead	good	4	fair	hairline cra	yes			east side	30	
97	revetment or toe stone		stone	2	no flat crest	10		poor	3	fair		no	wood	nonfunctional	na	na	
98	groin		wood	na	na	na	none	nonfunctional					wood	nonfunctional	none	0	
99	groin		wood	na	na	na	none	nonfunctional					wood	nonfunctional	none	0	
100	groin		combination	1.5	wood core	15	defunct bulkhead	fair	3.5	poor	intact	no	wood	poor	none	0	
101	groin		combination	1.5	wood core	15	defunct bulkhead	fair	3.5	poor	intact	no	wood	poor	none	0	
102	groin		combination	1.5	wood core	15	defunct bulkhead	fair	3.5	poor	intact	no	wood	poor	east side	75	
103	groin		wood	na	na	na		indeterminate					wood	indeterminate	none	0	
104	groin		wood	na	na	na		indeterminate					wood	indeterminate	none	1	

5 BORROW SOURCE

A borrow source investigation was carried out to identify and delineate sources of beachfill borrow material for use as initial design fill and future nourishment material for this project. Suitable grain size and distribution were investigated and present in available volume within a reasonable distance from the project shoreline. Grain size distributions and available volumes of the borrow sources were obtained from samples collected at various potential sources including upland, nearshore, and offshore locations. The grain sizes were compared with typical native beach model 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. The following summarize borrow source investigation results.

5.1 Historical Borrow Areas

Construction of the WRDA 1974 and Section 934 projects used several borrow areas offshore of Rockaway. Figure 5-1 shows the prior borrow areas and the delineation of the three borrow areas identified for the Reformulation Study.

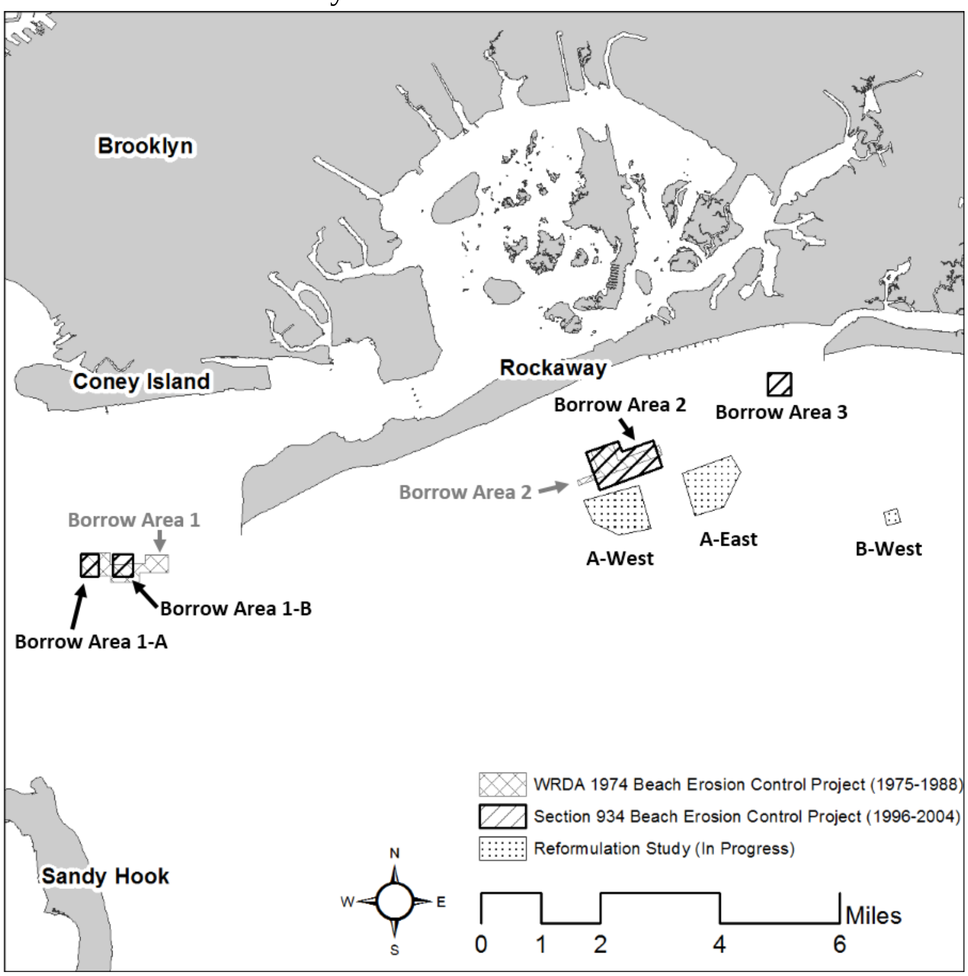


Figure 5-1: Historical Offshore Borrow Areas for Rockaway Beach

5.2 Native Beach Model

Eroded beaches that are in need of nourishment are considered to have remnant sediments of a grain size distribution that is more stable and in better equilibrium. 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 (i.e., pre-fill) 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 methodology from the USACE Shore Protection Manual, 1984 with the GSD curves of the borrow area sediments to determine the adjusted fill factor (Ra) and stability factor (Rj) of potential borrow sediments.

5.2.1 Native Beach Sediment Data

Native beach sediment samples were collected in 1961 and 1974 in pre-fill beach areas. The 1961 data consists of a summary of mean grain size, sorting coefficient, and a skewness coefficient, from which the 25th and 75th percentile grain sizes can be back calculated, and from that the 16th and 84 percentiles (required by current methodology) can be extrapolated. However, the 1974 data presents the raw grain size data, encompassing the 16th and 84th percentile. A comparison of the 1961 and 1974 mean grain size results shows, on average, the sediment neither becoming more coarse or more fine; therefore the more comprehensive 1974 data was used to estimate the native beach sand characteristics.

5.2.2 Native Beach Model Development

The 1986 monitoring report (unpublished) contains the following on-offshore spatial sediment composite information: Berm/Backshore, Mean High Water/Mean Tide Level/Mean Low Water, and -6/-12/-18/-24 ft. NGVD. Typically, beach fill equilibrates in shallower water; therefore, the -6/-12/-18/-24 ft. NGVD composite data was omitted from the model. The alongshore composite information was developed (in the monitoring report) for Beach Area A, which extends from B110th to B46th Streets; Area B, which extends from B46th to B19th Streets; and Area C, which extends from B149th to B100th Streets. As fill is proposed potentially in all three of these areas, all three areas were included in the model. The individual beach area sediment characteristics are shown in Table 5-1. The final beach model is determined by composition of all raw data (omitting the -6, -12, -18, -24, -30 ft. NGVD samples) for each beach area. The Rockaway Native Beach Model based on the mathematically mixed composition of all samples of the three beach areas (excluding deep samples) is shown on Table 5-2 below, and is 0.29 mm mean grain size, and standard deviation of 0.52 in phi units. Figure 5-2 shows the resulting native model grain size distribution curve.

Table 5-1: Average Values of the Rockaway Beach Sediment Samples by Beach Area

Beach Area	Sample Location	Phi 16 (ϕ_{16})	Phi 50 (ϕ_{50})	Phi 84 (ϕ_{84})	Mean Grain Size (ϕ)	Mean Grain Size (mm)	Standard Deviation (ϕ)
A (B110th to B46th)	Berm/Backshore	1.27	1.74	2.2	1.74	0.3	0.46
	MHW/MTL/MLW	1.09	1.74	2.27	1.7	0.31	0.59
	-6, -12, -18, -24, -30 ft.	1.55	2.5	3.46	2.5	0.18	0.96
B (B46th to B19th)	Berm/Backshore	1.31	1.79	2.29	1.79	0.29	0.49
	MHW/MTL/MLW	0.43	1.71	2.33	1.49	0.36	0.95
	-6, -12, -18, -24, -30 ft.	1.71	2.57	3.4	2.56	0.17	0.085
C (B149th to B110th)	Berm/Backshore	1.37	1.83	2.37	1.85	0.28	0.5
	MHW/MTL/MLW	1.31	1.83	2.54	1.9	0.27	0.62
	-6, -12, -18, -24, -30 ft.	1.55	2.87	3.57	2.67	0.16	1.01

Table 5-2: Native Beach Model Characteristics

Mean (ϕ)	**1.79**
Mean (mm)	0.29
Standard Deviation (ϕ)	0.52

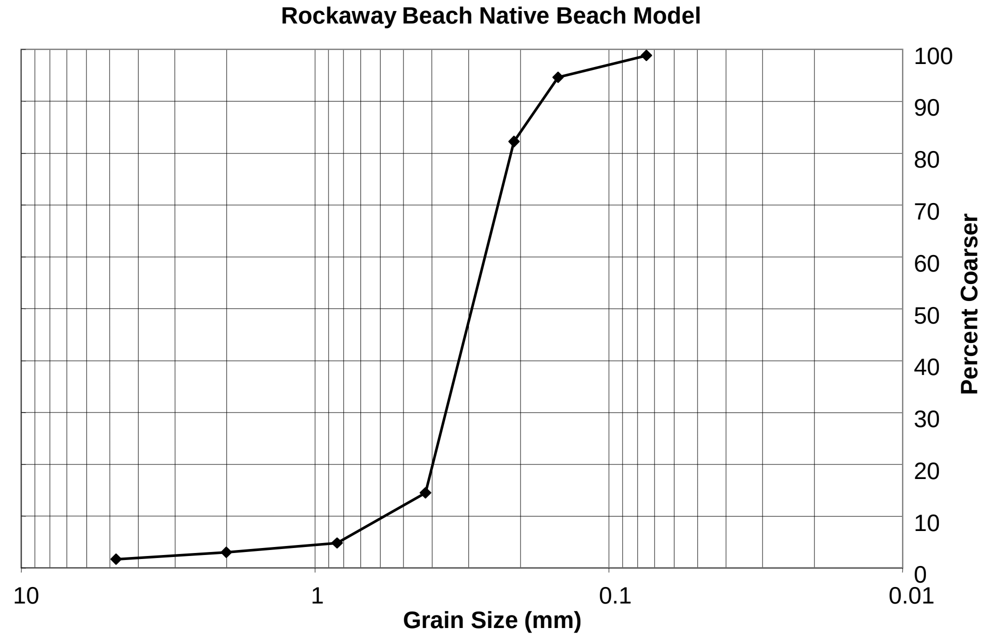


Figure 5-2: Native Beach Model

5.3 Potential Upland Source

An upland source using trucks to convey beach fill material to a project can be a cost effective alternative for small projects. However with large projects, the operational expense for the heavy equipment is often prohibitive and the environmental impact on the local communities may be prohibitive as well. However, if offshore sources are not available within reasonable traveling distance to the project site, it may be feasible to bring sand from upland suppliers by barge transfer. This could be the case if the sand suppliers have access to the waterways of Long Island Sound, the south shore of Long Island, Raritan Bay, New York Harbor or the Hudson River. Sand conveyed by barge in bulk can be fluidized and piped to the beach in the same manner that a cutter head dredge pumps sand ashore. Potential sites with suitable grain size, available volume, and within economic distance that warrant further investigation are described below and summarized in Table 5-3.

- Amboy Aggregates, South Amboy, NJ. This company is one of the largest suppliers of aggregate in the Unites States and the largest in the New York metropolitan area. One of its largest sources of sand and gravel is the channels leading into the New York Harbor (Ambrose, Chapel Hill, and Sandy Hook Channels, etc.). Amboy has a large processing plant in South Amboy, NJ that is capable of sorting dredged material into gradations needed by the construction industry. Recently, Amboy has begun importing coarse sediments from Canada, due to the scarcity of sand in the channels. Samples collected in 2000 varied in mean grain size from 0.26 to 0.56 mm, with a composite having a mean of 0.32 mm and sorting ratio of 1.15 in phi units, and were described as dark gray, fine to medium, poorly sorted, mainly quartz, but with small shell fragments (characteristic of marine sands).

- R.W. Vogel, Barnegat, NJ. The samples collected were from the Jackson, NJ processing plant, and were described as light tan, moderately sorted, medium quartz sand, with a mean grain size varying from 0.59 to 0.71 mm, with a composite of 0.63 mm and a sorting ratio of 1.11 in phi units.
- Horan Sand and Gravel Corp., Syosset, NY. Mean grain size varied from 0.41 to 0.85 mm, with a composite of 0.66 mm and a standard deviation of 1.26 in phi units.
- Ranco Sand and Stone, Manorville, NY. Mean grain size varied from 0.48 to 1.31 mm, with a composite of 0.63 mm and a standard deviation of 0.84 in phi.
- East Coast Mines, Limited, East Quogue, NY. Material is described as coarse fine sand. The mean grain size was 0.61 mm, and the standard deviation was 1.11 in phi units.

Table 5-3: Characteristics of Upland Sand Sources for Rockaway

Name of Quarry	Location	Mean Size (mm)	Standard Deviation (ø)
Amboy Aggregates	South Amboy, NJ	0.32	1.15
R.W. Vogel	Barnegat, NJ	0.63	1.11
Horan Sand and Gravel	Bayshore, NY	0.66	1.26
Ranco Sand and Gravel	Manorville, NY	0.63	0.84
East Coast Mines	East Quogue, NY	0.61	1.11

5.4 Potential Nearshore Source

Sources investigated included the navigation channels and inlets including Rockaway Inlet, East Rockaway Inlet, Jones Inlet, and the Jamaica Bay Channels. The bay channels were ruled out due to environmental sensitivities. Furthermore, bay sediments tend to be unstable for ocean beach stability. East Rockaway Inlet sediments are currently placed on the Rockaway beach, however, are not an ideal initial beachfill material. The dredged material could be used for renourishment due to its close proximity to the project site.

5.5 Potential Offshore Borrow Areas

There are several potential offshore borrow areas with suitable grain size and available volume based on available core data. The following criteria were used to select offshore borrow areas for further investigation:

- suitable grain size compared with native grain size (mean size = 0.29mm);

- sufficient volume (greater than 75,000 contiguous cy);
- proximity (as close as possible to fill area for cost purposes;
- minimal adverse effect of the local wave conditions;
- a minimum of 30 feet water depth;
- minimal environmental constraints, fishing interests;
- clear of cables, pipelines, shipping lanes, etc..

Two potential sites are short-listed based on their available size, suitability, and environmental considerations. The sites are described in the following paragraphs. The location coordinates of the offshore sites are summarized in Table 5-4 and described in Figure 5-1.

5.6 Borrow Area A-West

The area for Borrow Area A-West is roughly rectangular in shape approximately 4,800 feet from east to west, and 4,000 feet from north to south. The average dredging depth is approximately 18 feet below grade. Due to numerous magnetic anomalies detected during the magnetometer investigation in this vicinity, a diver investigation is recommended prior to dredging to determine the nature of the anomalies. If the anomalies are small enough and without cultural impact, a hopper dredge with a screen could be utilized. In this case, it is estimated that the borrow area could supply approximately 9 million cubic yards (assuming 1V:3H side slopes and 25% of material to be unusable). If the anomalies are not small enough, or have cultural significance and the anomalies may not be disturbed, the borrow area could still supply approximately 4 million cubic yards (assuming a minimum 200 ft buffer surrounding each anomaly and 1V:3H side slopes and 35% of the material to be unusable). The average overfill factor for this area is approximately 1.08.

5.7 Borrow Area A-East

Borrow Area A-East is roughly rectangular (5,000 feet in the alongshore direction by 4,000 feet in the on-offshore direction. The average overfill factor for this delineation is approximately 1.15. The approximate depth of suitable materials is 17 feet. The volume contained in this area is approximately 8 million cubic yards (assuming 1V:3H side slopes and omitting approximately 25% for poor material interlayer found while dredging). Either a hopper dredge or a cutterhead dredge may be used for this area.

5.8 Borrow Area B-West

Borrow Area B-West is roughly a 1,200 by 1,200 feet box. The average overfill factor for this delineation is approximately 1.06. The approximate depth of suitable materials is 17.8 feet. The volume contained in this area is approximately 1 million cubic yards (assuming 1V:3H side slopes and omitting approximately 25% for poor material interlayers found while dredging). A cutterhead dredge would be the most efficient for this area. Environmental investigation must be performed on this area prior to use.

Table 5-4: Potential Borrow Area Coordinates

Borrow Area	Corner	Northing (feet)	Easting (feet)
A-West	1	137,150	1,031,900
A-West	2	139,100	1,031,050
A-West	3	140,500	1,035,900
A-West	4	136,650	1,037,000
A-West	5	136,100	1,034,150
A-East	1	137,750	1,040,850
A-East	2	141,550	1,039,750
A-East	3	143,100	1,044,100
A-East	4	141,700	1,044,900
A-East	5	138,550	1,043,450
B-West	1	136,950	1,057,900
B-West	2	138,100	1,057,600
B-West	3	138,400	1,058,750
B-West	4	137,250	1,059,100

Notes: NAD83 State Plane, Long Island Lambert System

5.9 Recommended Borrow Source for Further Investigation

Borrow Areas A-West and A-East are recommended for further investigation. The recommended borrow volume range from 13,000,000 to 18,000,000 cubic yards. Further investigation will be necessary prior to construction.

5.10 Impacts of Borrow Area Dredging

Nearshore wave impacts resulting from the excavation of proposed borrow areas offshore of Rockaway were evaluated using the spectral nearshore wave transformation model STWAVE. The simulated dredging conditions represent the removal of all suitable material (18 million cubic yards, USACE-NAN 2008) in borrow areas, A-West, A-East, B-West. Ten offshore wave conditions (six extreme events and six events representing normal conditions) were simulated to characterize the impact of excavating the proposed borrow areas on nearshore waves. The relative change of the nearshore wave conditions at the -39 ft contour was calculated based on the existing conditions and dredged conditions.

The impact of excavating the proposed borrow areas is dependent on the wave direction, and wave period. The excavated borrow areas can alter wave transformation over the pits, resulting in alternating zones of wave focusing and wave divergence with an increase in the wave height up 14.4% and a decrease up to 26%. Generally an increase in wave height occurs adjacent to the borrow pits whereas a decrease in the wave height occurs over and in the lee of the borrow pit. A change in the direction of the offshore waves can shift the zones of wave focusing and wave divergence along the shoreline. An example of the modeled change in wave heights is shown in Figure 5-3. Given the magnitude of the changes in the nearshore wave height resulting from dredging the borrow areas a 15% increase in the nearshore design wave heights is recommended.

However, the net impact of sediment transport can be neglected due to both decrease and increase of wave heights of the nearshore waves, which would result in minimal net impact. In addition, the borrow pits will not be excavated to the maximum capacity within one excavation, and the dredged pit will most likely re-filled before the next operation.

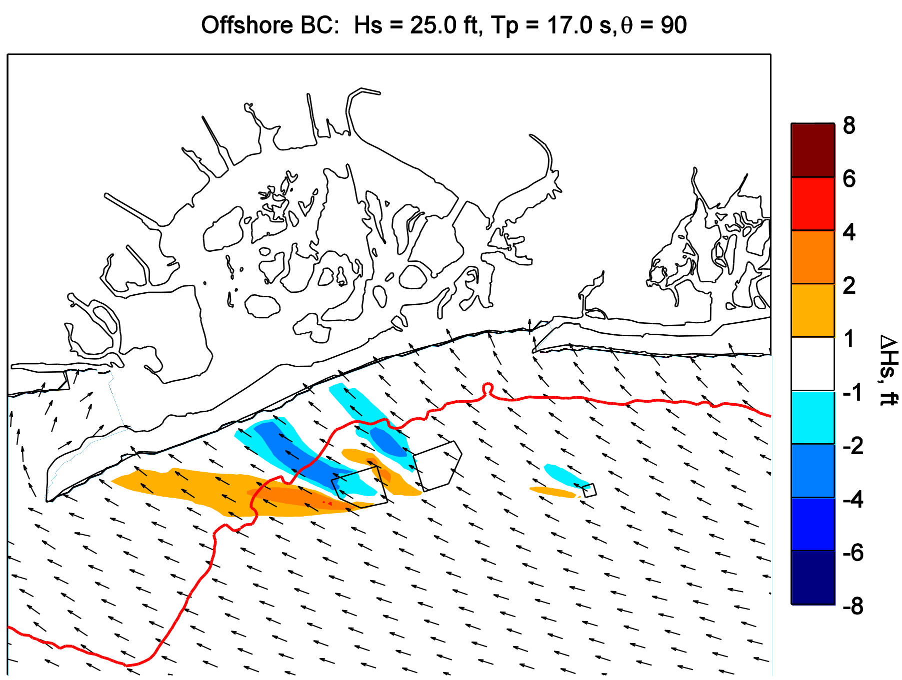


Figure 5-3: Example of Nearshore Wave Changes from Dredging

6 SEDIMENT BUDGET

6.1 Formulating a Sediment Budget

A sediment budget is an accounting of sediment gains and losses, or sources and sinks, within a specified control volume (cell), or a series of connecting cells, over a given period of time. Sediment budgets can provide a conceptual and quantitative model of sediment transport pathways in coastal systems, as well as a framework for understanding complex coastal systems and their responses to coastal engineering projects.

The sediment budgets developed for the Atlantic Ocean Shorefront include a number of cells and are based on the following balance:

$$\sum Qsource - \sum Qsink + P + BP - SLR = \Delta V + Residual$$

ΔV = net volume change within cell (eroding shoreline is a negative value)

P = volume of material placed within cell (positive contribution to cell)

BP = volume of material natural bypassed around inlet (positive contribution to cell)

SLR = volume of material lost to sea level rise (negative contribution to cell).

Qsource = Net longshore sediment transport (LST) into cell

Qsink = Net longshore sediment transport (LST) out of cell

6.2 Historical Sediment Budget

A historical sediment budget representing the time period between 1975-2010 was developed based on observed shoreline changes and engineering records of beachfill placement. This time period encompasses two major beach nourishment projects and intermittent renourishment and routine maintenance dredging of East Rockaway Inlet. The sediment budget was formulated based on the known quantities, setting the residual equal to zero, and solving for the net longshore sediment transport rates by balancing the budget.

Shoreline Changes (ΔV)

Net volume changes within the cells are calculated from observed shoreline changes between 1975 and 2010. The October 1975 shoreline represents the High Water Line (HWL) digitized from NOAA T-Sheets (TP00754 & TP00747). The August 2010 shoreline represents the Mean High Water Line (MWH) digitized from a Lidar survey performed by the USACE Joint Airborne Lidar Bathymetry Technical Center of Expertise (JALBTCX). Observed shoreline changes are converted to volumetric changes based on the height of the active profile (33 feet) and length of the shoreline within the sediment budget cell.

The observed shoreline changes over this time period are presented in Figure 6-1. The red areas in Figure 6-1 indicate accretion and the blue areas erosion. It is evident that the two ends of the

Rockaway Beach, Reach 1 and Reach 6, experienced the greatest rates of shoreline accretion with values up to +15 feet per year. In contrast the middle of the peninsula experienced smaller rates of shoreline change with Reaches 4 and 5 experiencing erosion rates up to -5 feet per year. It is noted that these shoreline change rates reflect the net shoreline change over this 35 year period and include the impact of beachfill operations. It is likely that the shoreline would have experienced much more erosion if it wasn't for the beachfill operations that helped stabilize the shoreline. The overall sediment budget will highlight the role beachfill operations had during this period in stabilizing the shoreline.

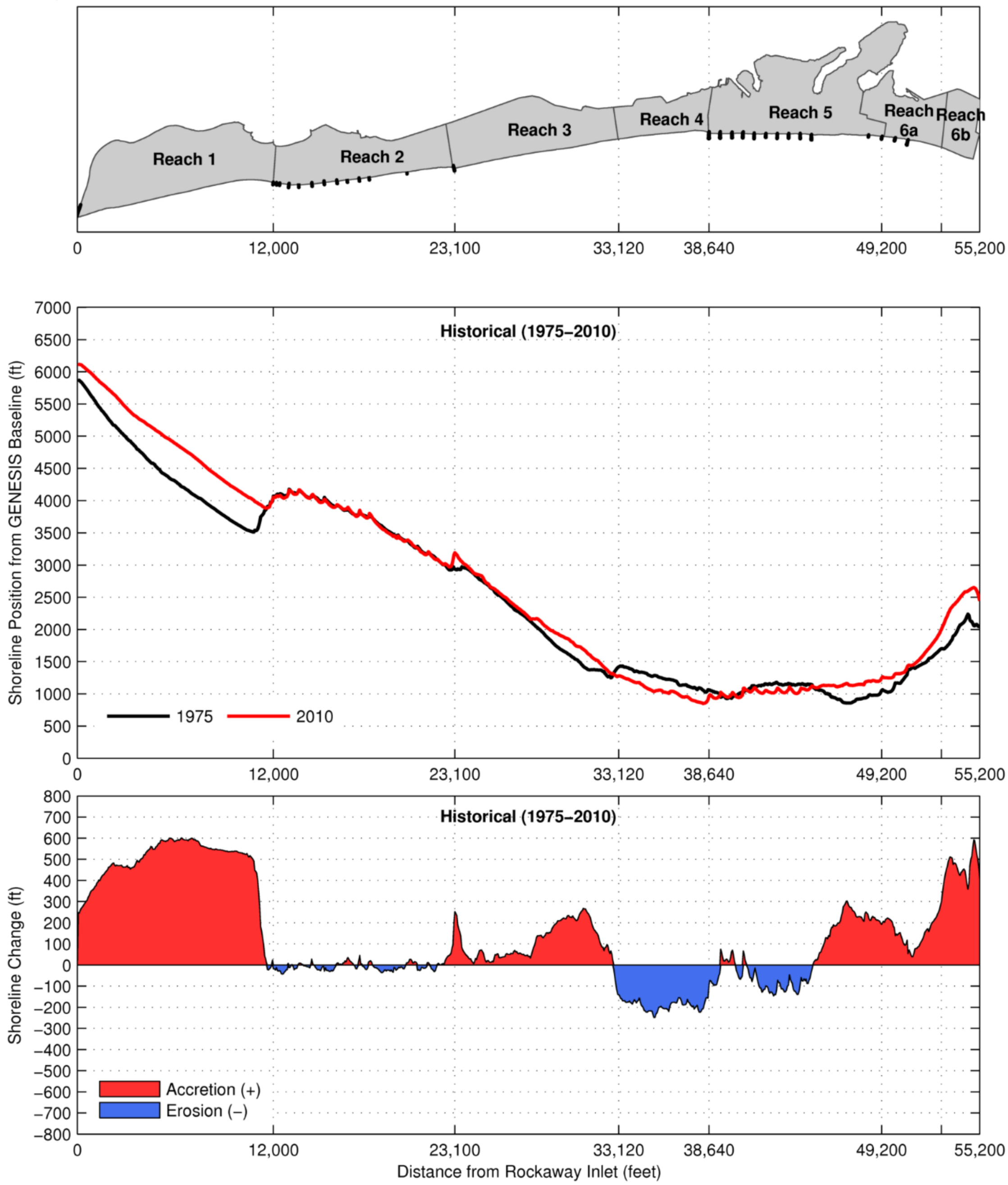


Figure 6-1: Historical Shoreline Changes (1975-2010)

Beachfill Placement (P)

A total 15.6 million cubic yards of beachfill were placed along Rockaway Beach from 1975-2010. This time period encompasses the WRDA 1974 Beach Erosion Control Project and the Section 934 Beach Erosion Control Project. A portion of the beachfill placed along Rockaway Beach is part of regular inlet maintenance dredging of East Rockaway Inlet. A majority (87%) of the beachfill over this time period was placed in Reaches 4, 5, and 6.

Natural Inlet Bypassing (BP)

East Rockaway Inlet is located at the eastern limit of Rockaway Beach. The East Rockaway Inlet is a Federal navigation channel 250-ft wide and maintained to -14 ft mean low water (MLW) plus 2 ft allowable overdepth. It is estimated that approximately 300,000 cy of updrift material needs to pass the Inlet in a westerly direction annually, either by natural bypassing or channel maintenance dredging (USACE NYD, 2012). Some material is lost permanently out of the littoral system. Historical dredging records indicate that the channel dredging rate increased from an average 30,000 cy/yr in the 1938-to-1978 time period to an average 115,000 cy/yr recently (USACE NYD, 2012). The increase in dredging volumes is associated with the growth of the updrift fillet at East Rockaway Inlet.

Previous studies have estimated that between 70,000 cy and 170,000 cy sediment naturally bypasses East Rockaway Inlet (OCTI, 2011 and USACE NYD, 2012). Recent shoreline modeling efforts as part of this study using GENESIS-T found that a bypassing rate of 100,000 cy/year provided the best model calibration for the 1996-2010 calibration time period.

Based on the available data and GENESIS-T modeling results the following Inlet Bypassing Rates are used in sediment Budgets:

- 1975-2010 Time Period: 50,000 cy/yr
- 1996-2010 Time Period: 100,000 cy/yr
- Future Conditions: 100,000 cy/yr

Sea Level Rise (SLR)

Cross-shore sediment losses due to sea level rise (SLR) are incorporated in the sediment budget after Bruun (1962).

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

S = change in sea level

θ = average profile slope over active beach profile

R = horizontal recession of beach

The historic rate of SLR in the study area is +0.0128 ft/yr (NOAA Sandy Hook Tide Gage). The average profile slope over the active beach profile, θ, was estimated to be 1V:60.5H based on long profile surveys in the project area. Over the 35 year time period the sea level (S) has risen approximately 0.45 ft corresponding to a horizontal recession of the beach (R) of 27 ft. This is equivalent to a volumetric loss of 53,000 cy/yr over the entire Rockaway Beach shoreline.

Two-Cell Sediment Budget

A two-cell sediment budget, Figure 6-2, was developed for Rockaway Beach based on the sediment sources and sinks described above. The two-cell sediment budget shows that net annual longshore sediment transport increases from -10,000 cy/yr (east) to +376,000 cy/yr (west) over the eastern cell. This sharp increase in sediment transport creates a 386,000 cy/yr deficit in the eastern sediment budget cell. Historically this sediment deficit has been offset by placing a 387,000 cy/yr of beachfill in this cell. The sediment budget shows that without this beachfill this cell would have experienced considerably more erosion.

The western sediment budget cell has a surplus of sediment since much more sediment is entering the cell at Beach 110th Street than is leaving the cell at Rockaway Inlet. The surplus of sediment resulted in shoreline accretion, most notably in Reach 1 (Figure 6-1).

The two-cell historical sediment budget clearly shows that net longshore sediment transport rates increasing from east to west along Rockaway Beach are the primary cause of shoreline erosion in eastern Rockaway Beach and shoreline accretion in western Rockaway Beach. Shoreline erosion in eastern Rockaway has been largely avoided by placing a considerable amount of beachfill in this area.

Seven-Cell Sediment Budget

A more detailed seven-cell sediment budget, Figure 6-3, was also developed for Rockaway Beach based on the same set of data to provide additional detail about areas that have historically been erosional “hotspots”. The WRDA 1976 Erosion Control Project only performed renourishment operations in the two Feeder Beaches (Reach 4 and Reach 6a). These feeder beaches were also identified prior to the WRDA 1976 Project as erosional hot spots. It is clear from the sediment budget that Reaches 4 and 6a are indeed erosional hotspots. A nodal point in the net annual longshore sediment transport occurs within Reach 6a and there sediment transport rates cause a sediment deficit of 223,000 cy/yr. This deficit has been historically offset by beachfill placement (212,000 cy/yr).

The other erosional hotspot, Reach 4, appears to be caused by the groin field in Reach 5 which suppresses the sediment transport, starving Reach 4 of sand. Once again the sediment deficit has been managed by beachfill placement (130,000 cy/yr).

The sediment budget shows that Reaches 2, 3, and 5 have been relatively stable and have about the same net longshore sediment transport entering and leaving the cells. These cells have likely been beneficiaries of updrift beachfill operations.

Reaches 1 and 6b have a sediment surplus because more net annual longshore sediment transport is entering than leaving these cells.

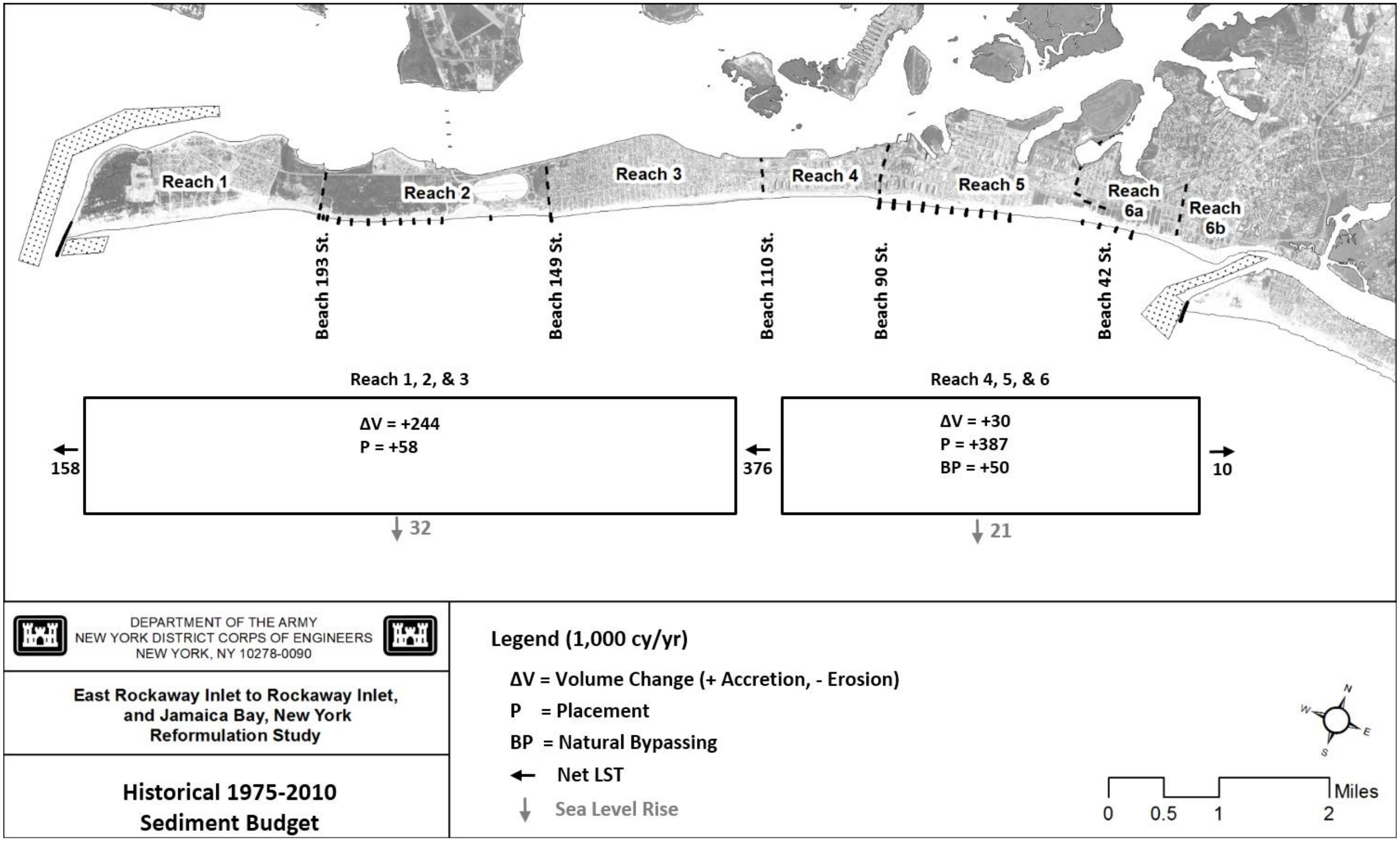


Figure 6-2: 2-Cell Historical Sediment Budget (1975-2010)

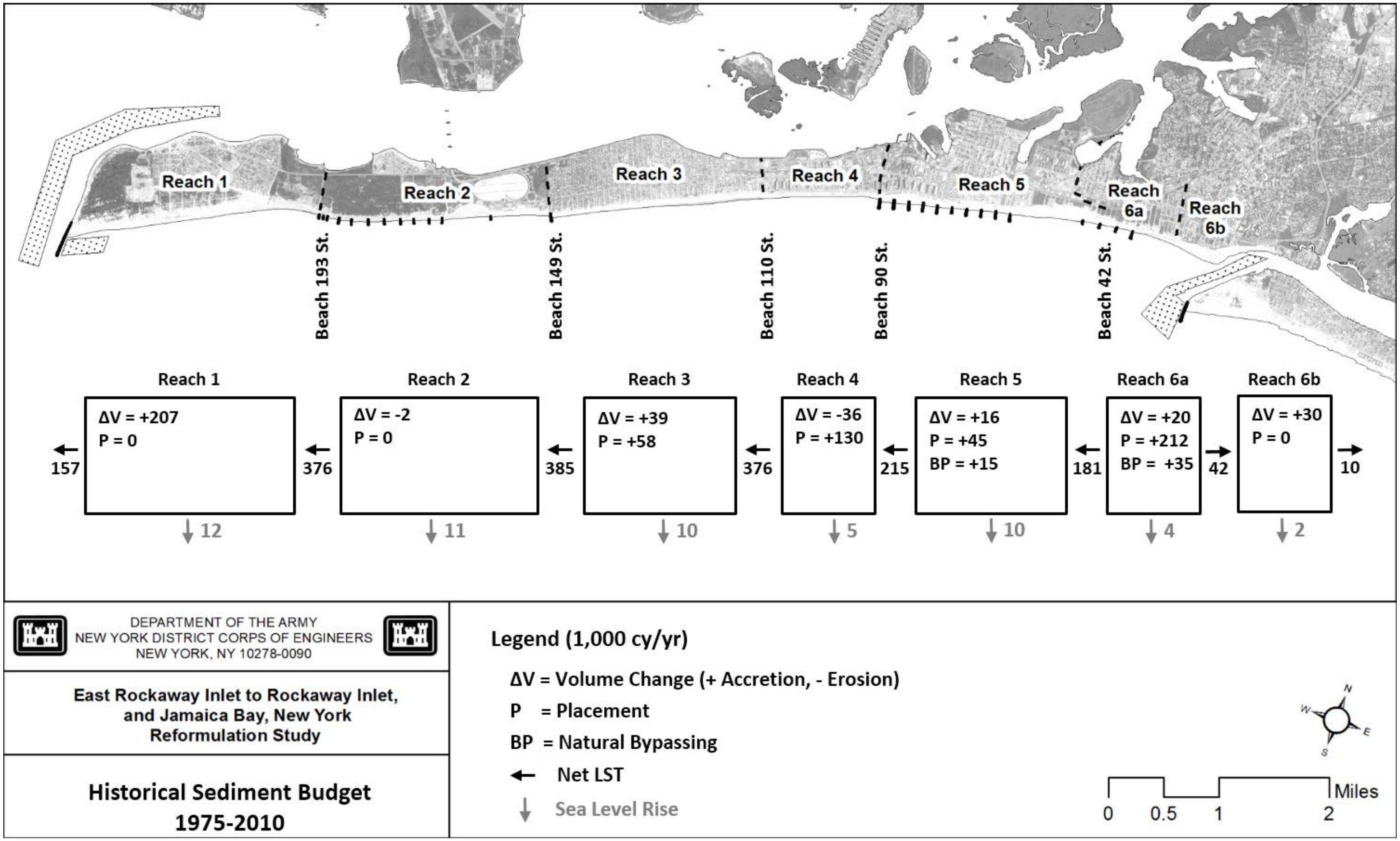


Figure 6-3: 7-Cell Historical Sediment Budget (1975-2010)

6.3 Without Project Sediment Budget

The Without Project Future Condition (WOPFC) is by definition the projection of the most-likely future conditions in the project area if no actions are taken as a result of this study. The WOPFC must be representative of what is most likely to occur in the absence of a Federal project, and as such must be based upon historic practice and trends. The WOPFC serves as the base conditions for all the alternative analyses.

Identifying the WOPFC at The Atlantic Ocean Shorefront is particularly challenging because the historical conditions include a Federal project. Therefore, historical data alone may not be used to describe the shoreline and beach conditions if no actions are taken in the project area. Instead, a shoreline change model (GENESIS-T) is used to simulate longshore sediment transport and shoreline changes that are likely to occur in the WOPFC.

In defining the WOPFC, the following assumptions are made to establish the framework of what is likely to occur.

Beachfill Placement (P)

As defined by existing Federal/State navigation authorities, the existing inlets (Rockaway Inlet and East Rockaway Inlet) and their corresponding approach and back-bay navigation channels will be maintained near the present widths depths, and locations. Approximately 230,000 cubic yards of material will be dredged from East Rockaway Inlet every 2 years and placed in Reach 6a.

Natural Inlet Bypassing (BP)

A natural inlet bypassing rate of 100,000 cy/year at East Rockaway Inlet is used to characterize the WOPFC. This bypassing rate provided the best calibration in GENESIS-T and is within the range of previous estimates (OCTI, 2011 and USACE NYD, 2012).

6.3.1 GENESIS-T Modeling

GENESIS-T is designed to simulate long-term shoreline change based on spatial and temporal differences in longshore sediment transport induced primarily by wave action while accounting for coastal structures and beach fills. The GENESIS-T model was calibrated to historical conditions from 1996-2010. A detailed description of the GENESIS-T model development is provided in the Numerical Modeling Appendix (Sub-Appendix A).

A 16-year GENESIS-T simulation was performed to characterize the WOPFC. The wave conditions for the 16-year period are based on the wave conditions from 1996 to 2012. The predicted net annual longshore sediment transport from GENESIS-T is used in the WOPFC sediment budget. Figure 6-4 shows the predicted net annual longshore sediment transport rates and corresponding shoreline changes. The WOPFC simulations include both natural inlet bypassing and inlet maintenance dredging, both of which reduce the shoreline erosion in Reach 6a. The GENESIS-T simulations do not include the impact of sea level change or any other cross-shore coastal processes.

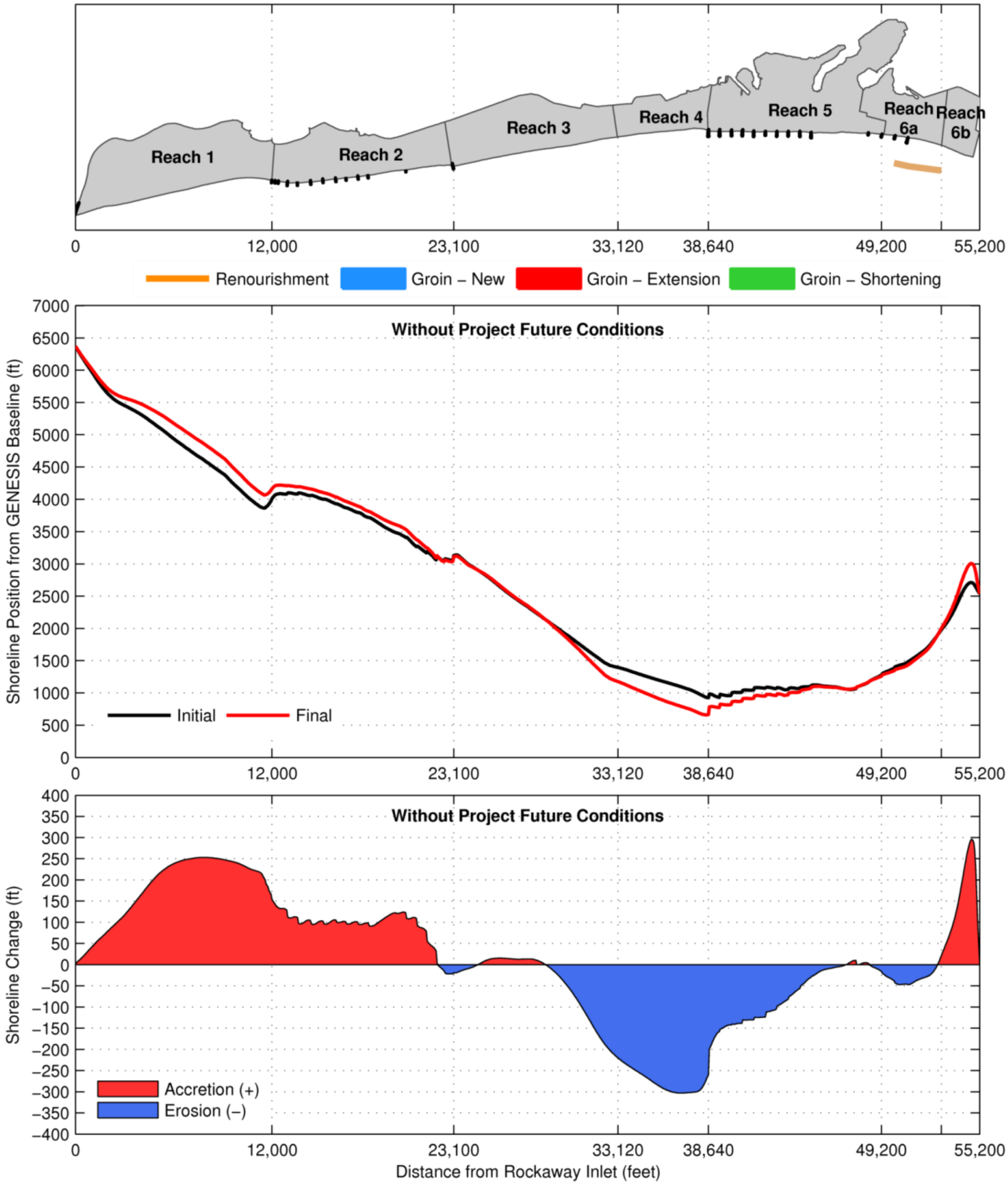


Figure 6-4: WOPFC Shoreline Changes

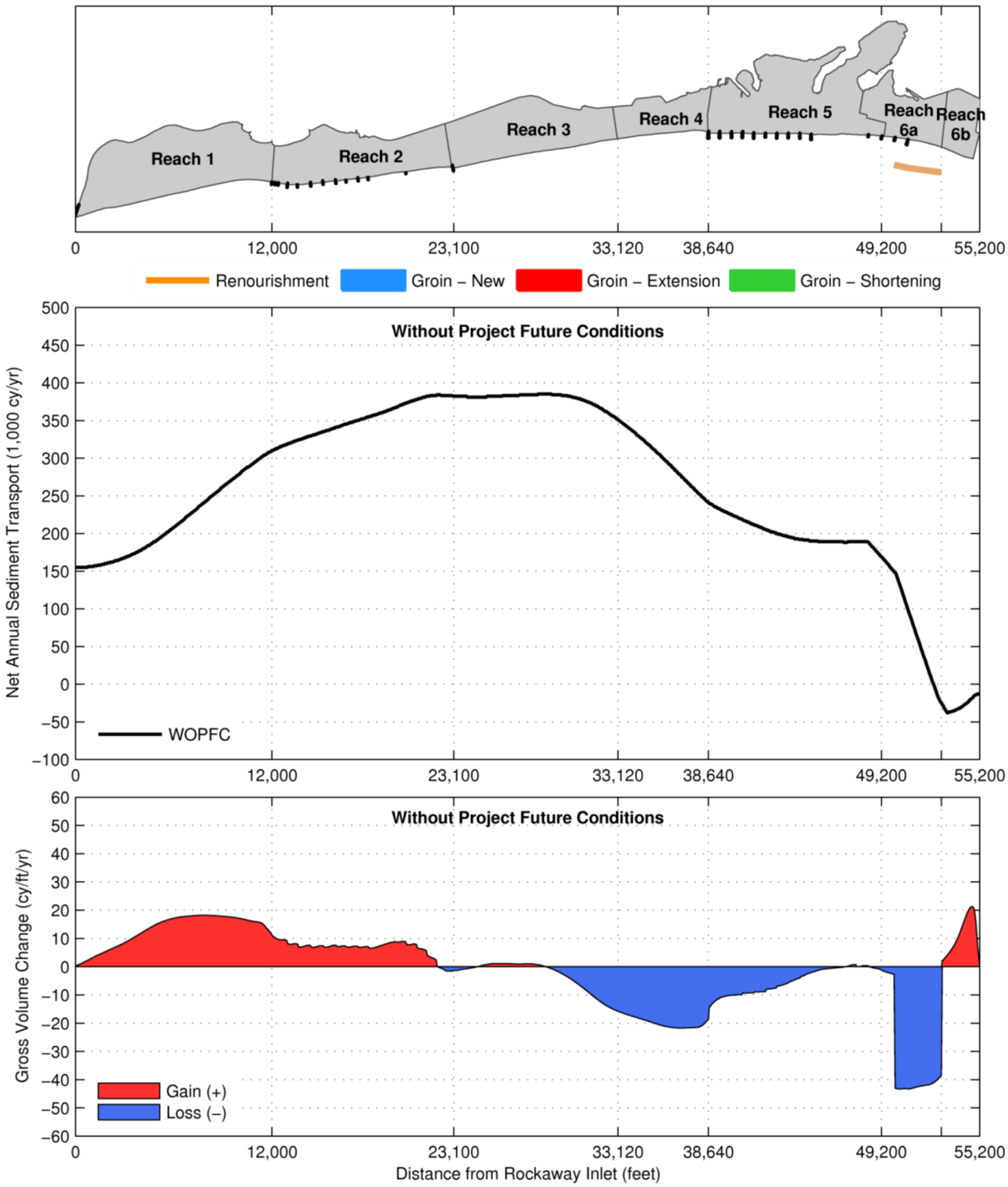


Figure 6-5: WOPFC Sediment Transport & Gross Volume Changes.

6.3.2 Without Project Future Conditions Sediment Budget

A WOPFC sediment budget was developed based on modeled shoreline changes, modeled net annual longshore sediment transport rates, sea level rise, and inlet bypassing and inlet maintenance dredging assumptions.

6.3.3 Sea Level Rise (SLR)

Cross-shore sediment losses due to sea level rise (SLR) are incorporated in the sediment budget after Bruun (1962). The WOPFC sediment budget uses the historic rate of SLR at the NOAA Tide Gage at Sandy Hook, NJ. The sensitivity to of the WOPFC to higher rates of sea level rise is shown based on current USACE guidance (ER 1100-2-8162). Future SLR rates were evaluated for a 50-year period from 2018-2068. Table 6-1 provides an overview of the impact sea level rise.

Table 6-1: Sea Level Rise Impacts on Shoreline Changes and Sediment Budget

Sea Level Rise Scenario	SLR over 50-years (ft)	Shoreline Change (ft/yr)	Volumetric Loss[1] (cy/yr)
USACE-Low (Historical)	0.64	-0.78	53,000
USACE-Intermediate	1.09	-1.32	90,000
USACE-High	2.80	-3.07	209,000

[1]Volumetric Loss over the entire Rockaway Beach Shoreline (55,650 feet)

6.3.4 Two-Cell Sediment Budget

As expected, the WOPFC sediment budget is similar to the Historical Sediment Budget, except that the only beachfill placement in the WOPFC is associated with East Rockaway Inlet maintenance dredging. The two-cell WOPFC sediment budget, Figure 6-7, shows that eastern half of Rockaway Beach has a sediment deficit of approximately 145,000 cy/yr while the western half has a sediment surplus of approximately 200,000 cy/yr. As a result it is expected that the eastern half of Rockaway will continue to experience shoreline erosion and the western half shoreline accretion in the absence the WOPFC. The corresponding shoreline change rates for the WOPFC are presented in Table 6-2. The impact SLR acceleration has on the shoreline change rates is also shown in Table 6-2.

Table 6-2: Two-Cell WOPFC Shoreline Changes

Sea Level Rise Scenario	Shoreline Change (ft/yr)	
	Reaches 1, 2, 3	Reaches 4, 5, 6
USACE-Low (Historical)	+4.1	-6.2
USACE-Intermediate	+3.5	-6.8
USACE-High	+2.2	-8.5

6.3.5 Seven-Cell Sediment Budget

The seven-cell WOPFC sediment budget, Figure 6-8, provides a more detailed look at the sediment budget and identifies erosional hotspots along Rockaway Beach. The net annual longshore sediment transport rates are similar to the Historical Conditions, and increase from east to west along Rockaway Beach peaking in Reach 3. The steady increase in net annual longshore transport rate creates a sediment deficit in Reaches 3, 4, 5, and 6a. The overall trend in longshore sediment transport is driven by the alongshore variability in the wave conditions. Figure 6-6 shows the alongshore variability in the net annual longshore sediment transport problems.

The primary difference between the WOPFC and Historical Conditions sediment budgets is that there is no beachfill in the WOPFC to offset the sediment deficit created by the overarching trend longshore sediment transport. Table 6-3 shows the corresponding shoreline change rates based on the seven-cell WOPFC sediment budget. The most striking cell is Reach 4, which is predicted to erode by 17.5 ft/yr. This erosion hotspot is caused by 1) overarching trend in longshore sediment transport along eastern Rockaway Beach, 2) sediment impoundment of updrift groin field in Reach 5.

Table 6-3: Seven-Cell WOPFC Shoreline Changes

Reach	Shoreline Change (ft/yr)
1	+9
2	+4.4
3	-3.2
4	-17.5
5	-3.8
6a	-5.3[1]
6b	+9.4

[1]Shoreline change rate in Reach 6a would be much greater if not for beach fill from Inlet Maintenance Dredging

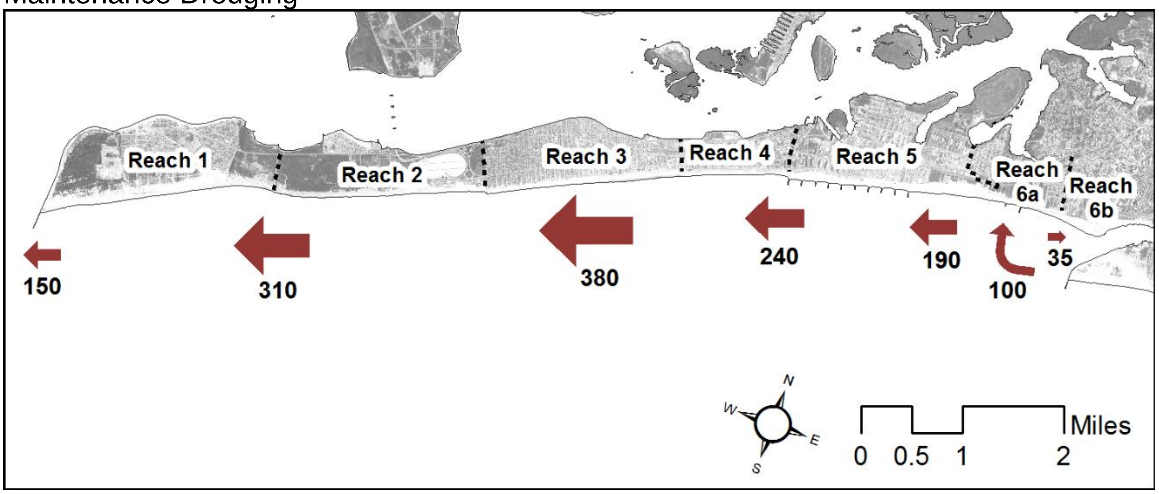


Figure 6-6: WOPFC Sediment Transport Pathways at Rockaway Beach

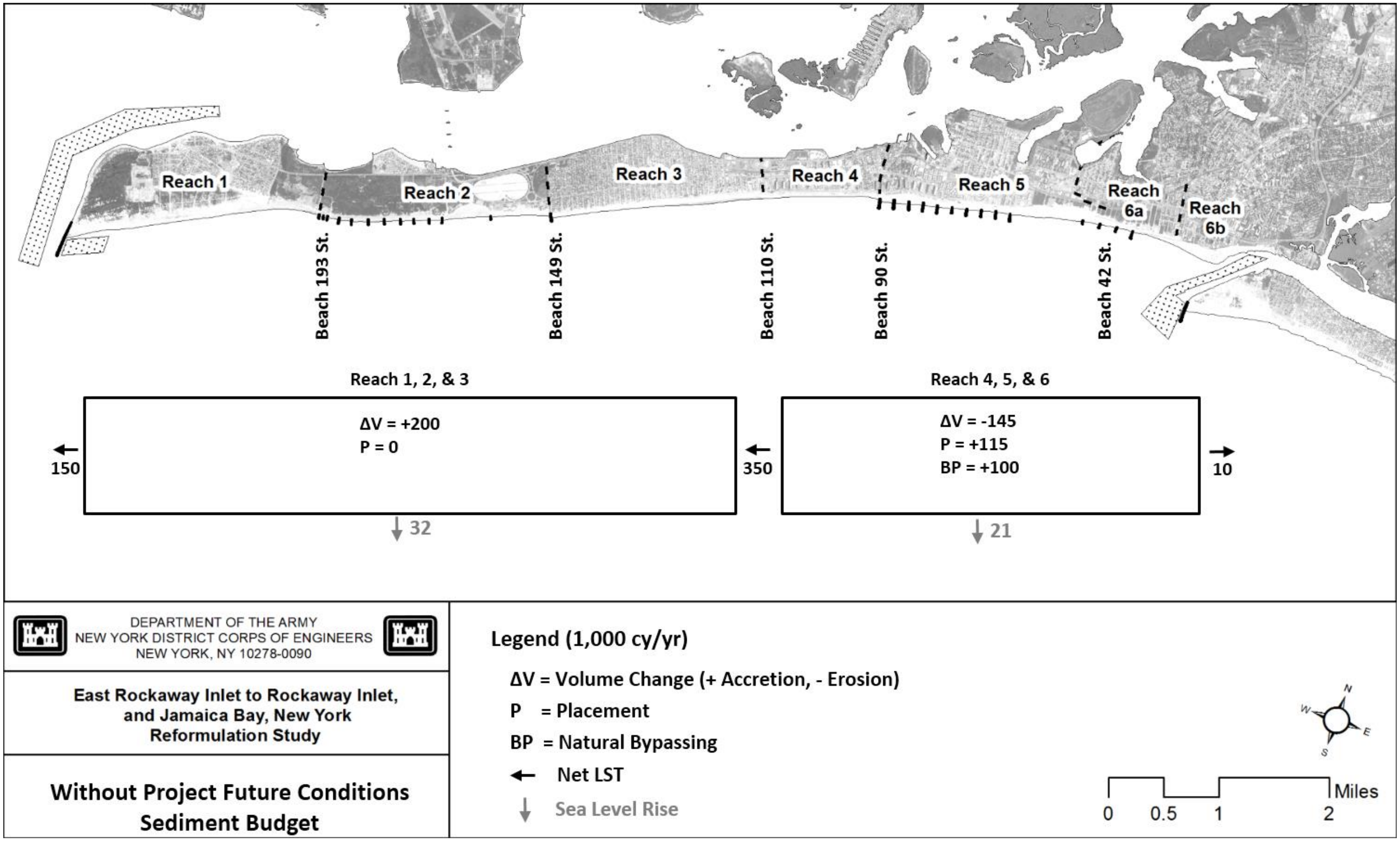


Figure 6-7: 2-Cell WOPFC Sediment Budget

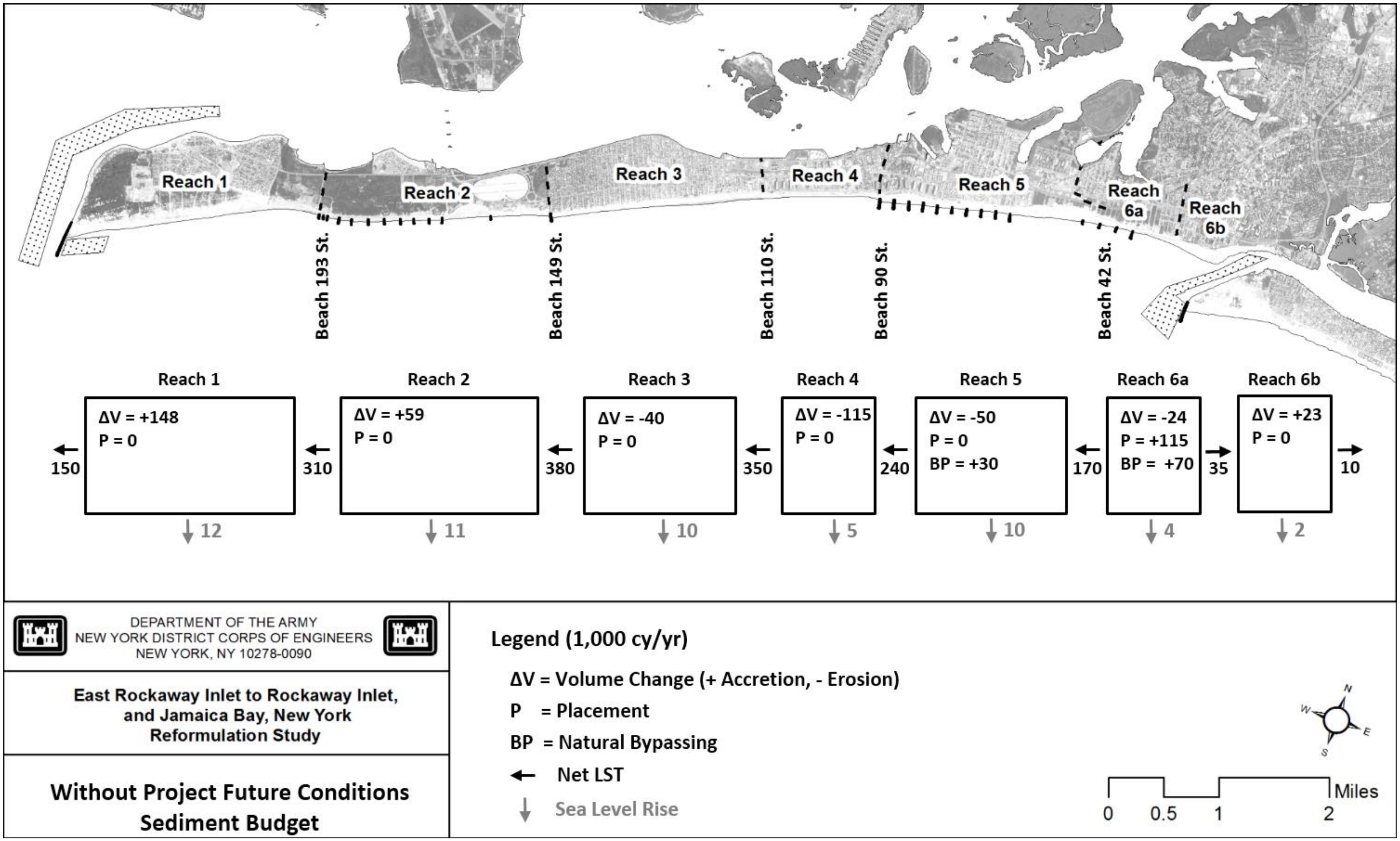


Figure 6-8: 7-Cell WOPFC Sediment Budget

6.4 With Project Sediment Budget

The GENESIS-T modeling results for each alternative were simplified by reaches and used to create With Project Conditions (WPFC) sediment budgets. The net annual longshore sediment transport rates and renourishment quantities from the GENESIS-T simulations are incorporated into the sediment budgets (Figure 6-9 to Figure 6-11). A detailed description of the GENESIS-T modeling results is provided in the Engineering Modeling Appendix (Sub-Appendix A).

The WPFC sediment budgets account for the low (historic) sea level change prediction, resulting in a 29,000 cy/yr increase in renourishment quantities from the GENESIS-T modeling results which do not account for sea level rise. Renourishment quantities would increase by an additional 20,000 and 84,000 cy/yr respectively if the intermediate and high sea level change predictions are applied.

This section documents the evaluation of several beach restoration and erosion control alternatives for the Atlantic Ocean Shorefront. The alternative evaluation focuses on ways to reduce future renourishment requirements and life-cycle costs. The alternative analysis does not consider storm damage reduction benefits; all of the alternatives provide roughly the same level of risk reduction since the alternatives have the same design profile (e.g. berm and dune dimensions). Different measures, such as new groins, shortening / lengthening of existing groins, and boardwalk relocation, are evaluated based on their ability to reduce future renourishment requirements and life-cycle costs. The three alternatives evaluated are (Figure 7-1 in Section 7.1):

- Alternative 1: Beach Restoration
- Alternative 2: Beach Restoration + Reduced Erosion Control
- Alternative 3: Beach Restoration + Increased Erosion Control

The objective of all three alternatives is the same: maintain the design beach profile over the life of the project at the lowest possible cost. The focus of Alternatives 2 and 3 are managing the two historical erosional hot spots in Reach 4 and 6 with either reduced erosion control (groin shortening and boardwalk relocation) or increased erosion control (new groins and extension of existing groins).

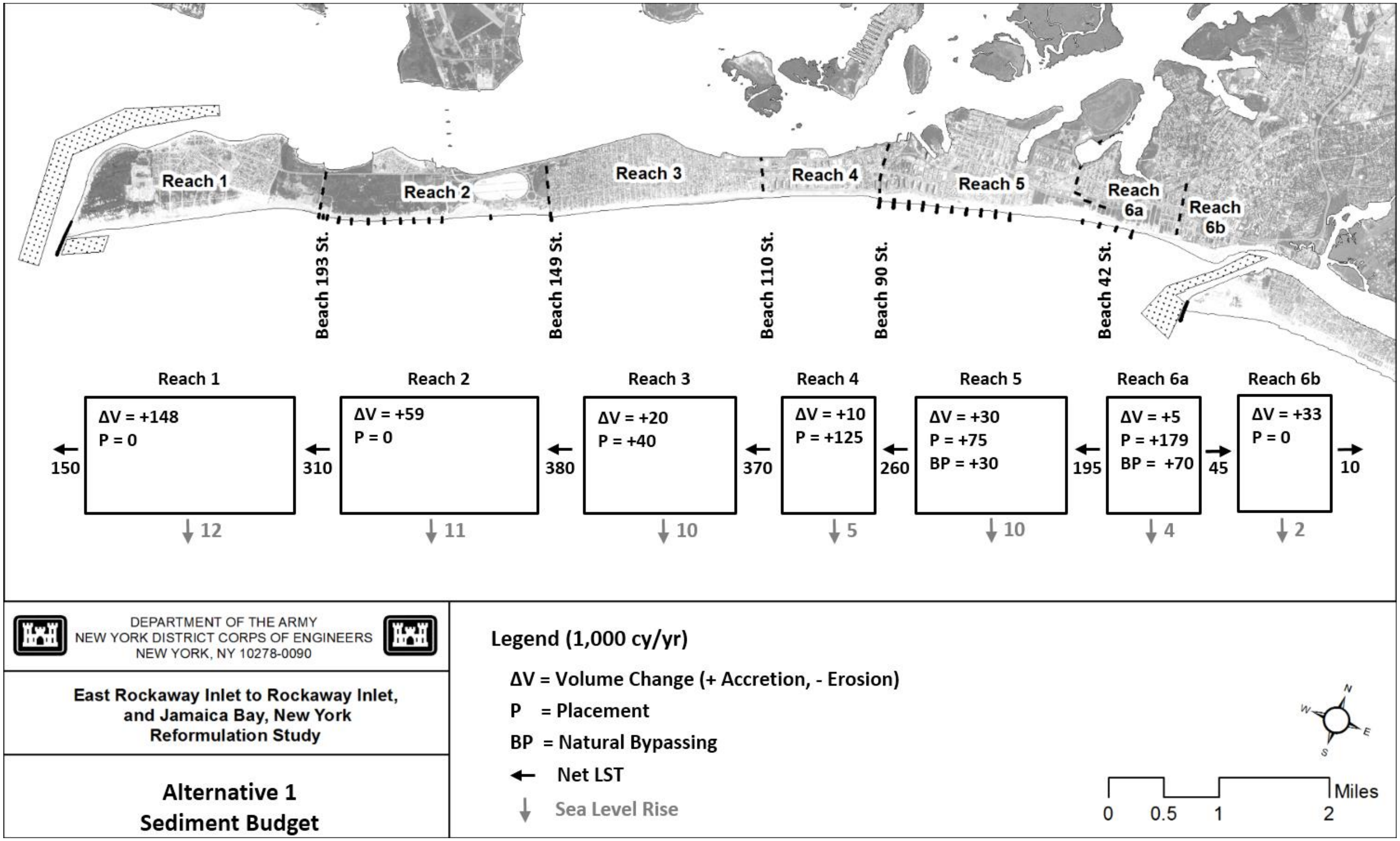


Figure 6-9: 7-Cell Alternative 1 Sediment Budget

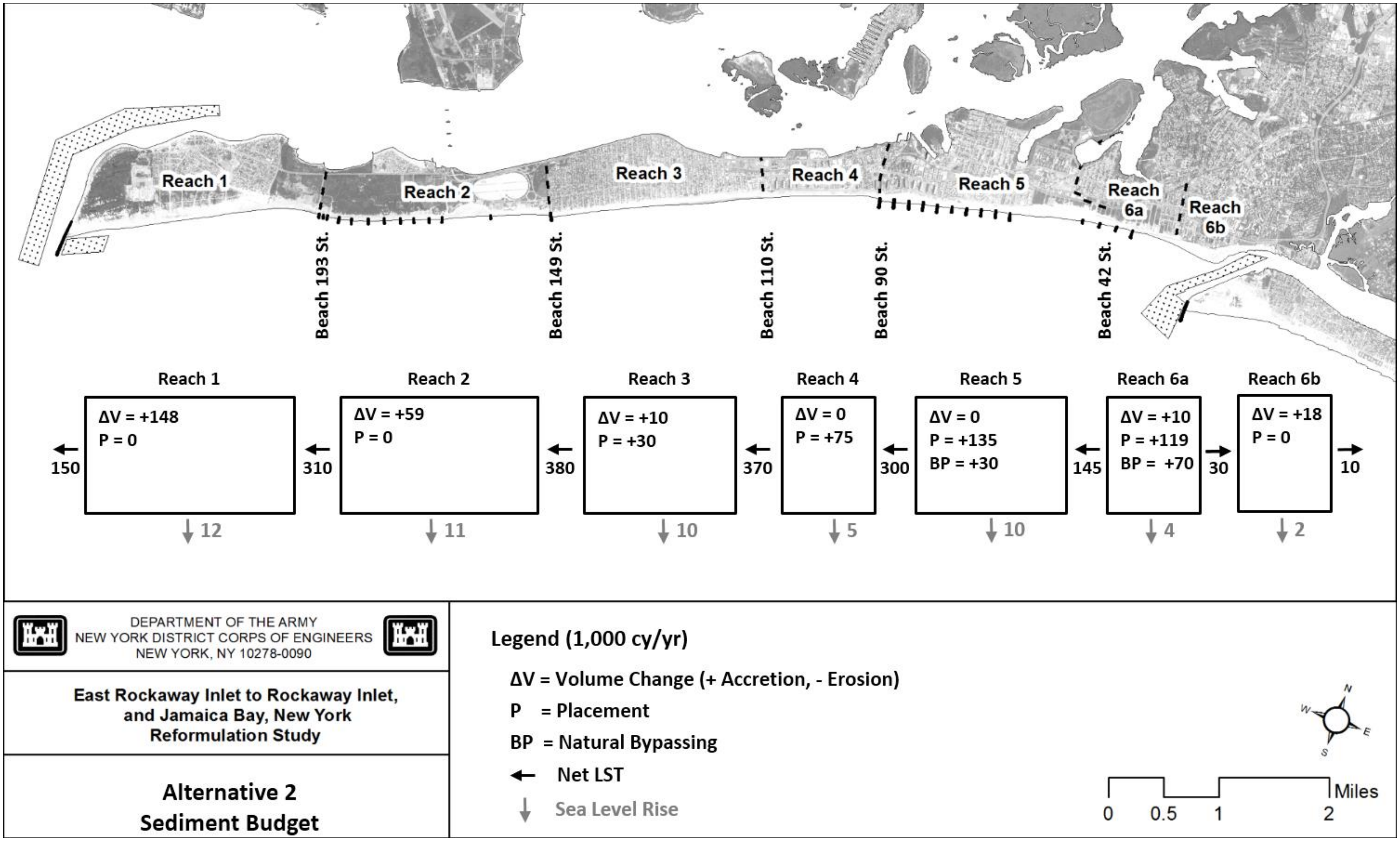


Figure 6-10: 7-Cell Alternative 2 Sediment Budget

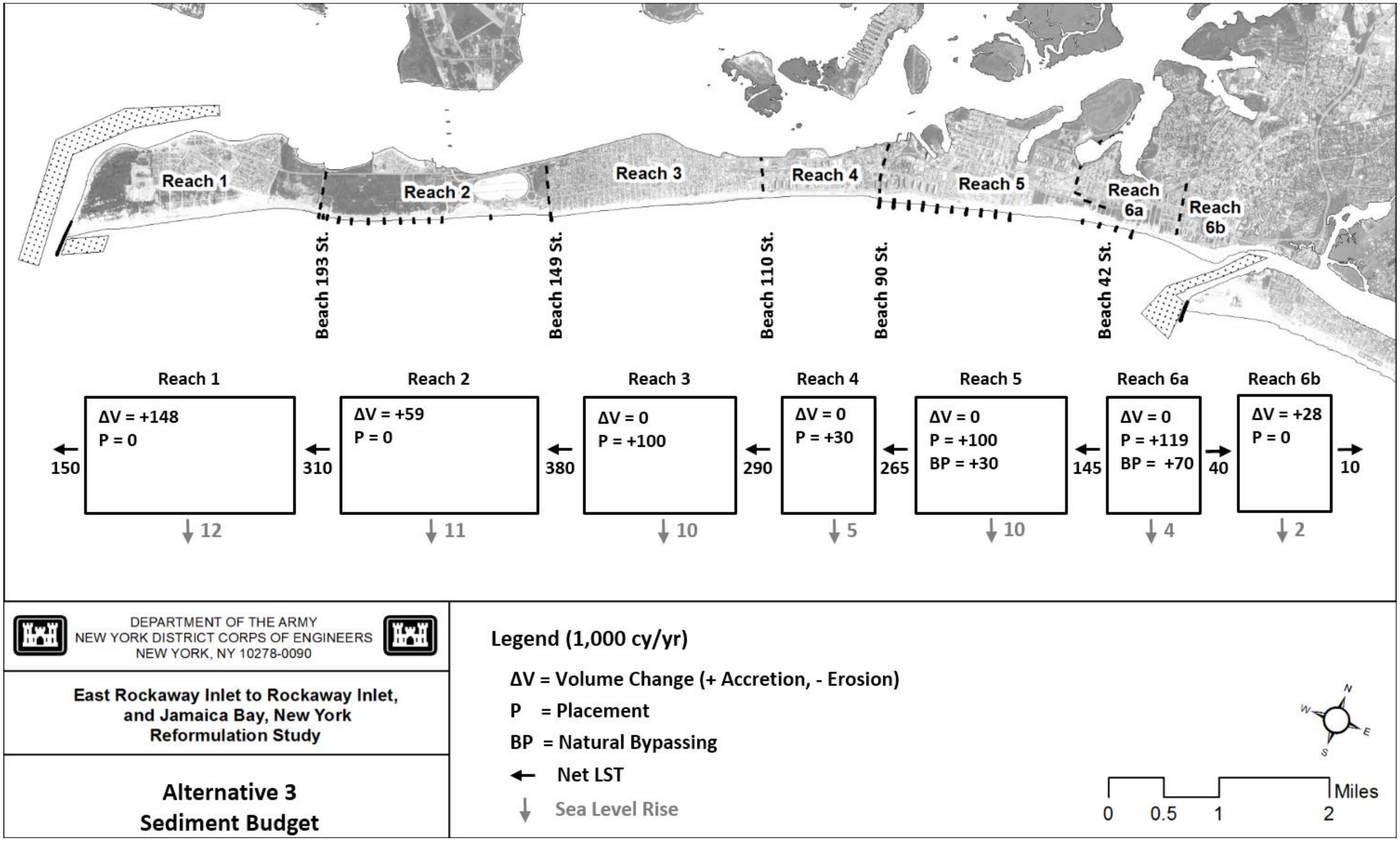


Figure 6-11: 7-Cell Alternative 3 Sediment Budget

7 ALTERNATIVE DEVELOPMENT

7.1 Overview

An initial screening effort was undertaken to identify potentially feasible erosion control and storm protection measures. These measures have been incorporated in a series of plans to address the identified problems.

The general approach to identifying a tentatively selected plan is to evaluate erosion control alternatives in combination with a single beach restoration plan to select the most cost effective approach to reducing project renourishment. This analysis is a lifecycle cost comparison to ensure that the most cost effective renourishment approach has been identified prior to the evaluation of alternatives for the coastal storm risk management.

The Coastal Storm Risk Management (CSRM) plans consist of various beachfill, dune and seawall measures to reduce future storm damages for the Atlantic Ocean Shorefront (Reach 3 through Reach 6b). The plans were evaluated based on a comparison of their quantified storm risk management benefits in comparison to their costs. The plan that provides the greatest net CSRM benefits in excess of costs is identified as the National Economic Development Plan.

Recreation benefits are also being evaluated and will be incorporated into the final Benefit to Cost Ratio (BCR). For any plan to be implemented the BCR must be greater than 1.0.

The alternatives comparison is initially performed based on the low/ historic sea level rise scenario, which assumes a continuation of historic sea level changes. The scenario analysis considers two additional accelerated sea level change scenarios.

7.2 Erosion Control Alternatives

Plan formulation of the erosion control alternatives focused on identifying the least-costly solution to maintaining a wide beach and dune over the 50-year planning horizon. The erosion control alternative analysis did not consider storm damage reduction benefits; each of the erosion control alternatives were evaluated based on the same generic design berm and dune. Four erosion control alternatives (Figure 7-1) were short-listed by the PDT and selected to be evaluated in detail:

- Alternative 0: No Action
- Alternative 1: Beach Restoration
- Alternative 2: Beach Restoration + Reduced Erosion Control
- Alternative 3: Beach Restoration + Increased Erosion Control

The short-listed alternatives include various measures such as new groins, shortening/lengthening of existing groins, and boardwalk relocation that have the potential to reduce future renourishment requirements and life-cycle costs

Detailed one-dimensional shoreline change modeling (GENESIS) was conducted to identify future renourishment requirements for each alternative. The screening level design consisting of plan layouts, cross-sections, quantities, and costs, was performed for each alternative to estimate the life-cycle costs.

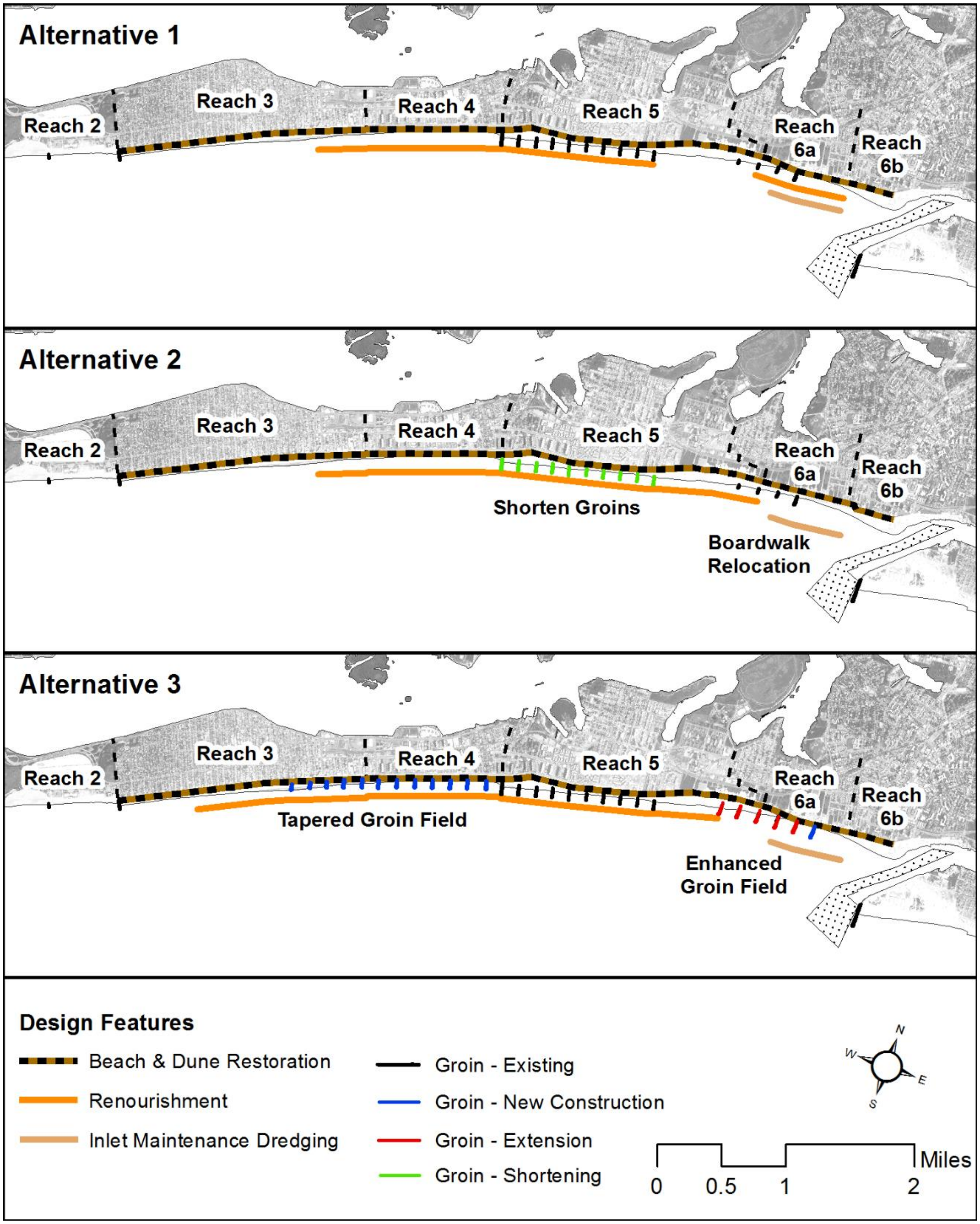


Figure 7-1: Short-Listed Alternatives

7.2.1 Screening Level Design

This section presents the screening level design for the alternatives. The screening level design consisted of developing layouts, cross-sections, quantities, and costs. The objective of the screening level design is to develop enough detail regarding the designs to be able to reliably estimate the life-cycle costs.

7.2.2 Preliminary Design Profile

At the start of Erosion Control Alternative evaluation the optimal design profile was still being evaluated as part of the Beach-fx study, which led to the selection of an optimal design berm width and dune height. Some features of the design profile (i.e. berm elevation, dune slope, foreshore slope) were already known. In order to allow the Erosion Control Alternative Analysis to proceed an assumption regarding the design profile was necessary. It is not expected that changes in the design profile would have a significant impact on the relative cost of the alternatives. All elevations throughout this appendix are reference to the North American Vertical Datum of 1988 (NAVD88) unless specifically stated otherwise.

Figure 7-2 shows the design profile used in the erosion control analysis, which has a dune elevation of +16 ft NAVD88 and a berm width of 60 ft. The advance berm width is a sacrificial quantity of sand which acts as an erosional buffer against long-term and storm-induced erosion with the goal of preventing erosion of the design profile between renourishment cycles. Specifically, the advance fill is to offset the expected losses between initial construction and the first renourishment operation. Advance berm widths vary by alternative and reach since it the advance berm width is proportional to the expected shoreline erosion between renourishment operations.

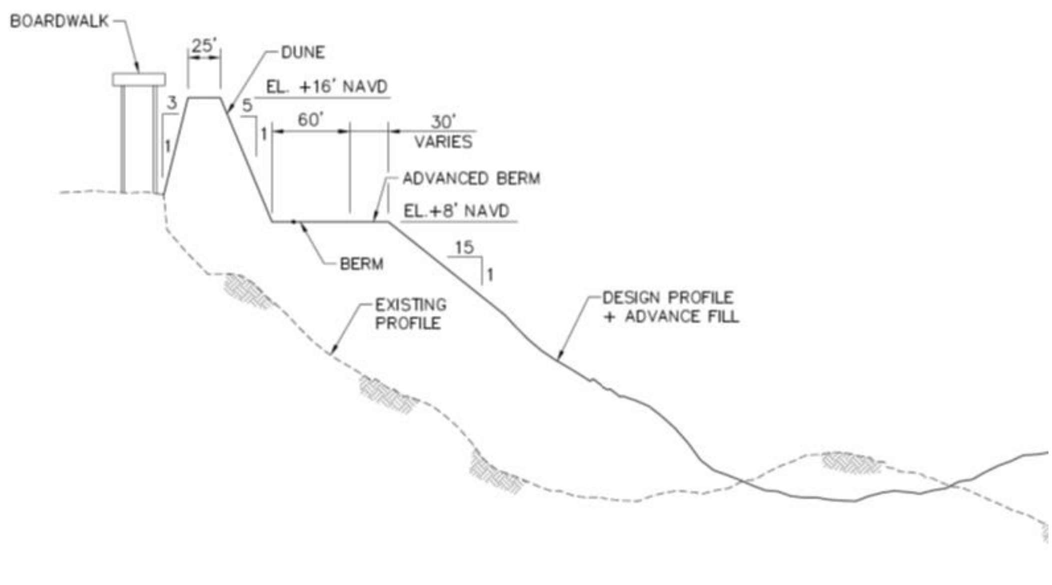


Figure 7-2: Preliminary Design Profile (Applied in Erosion Control Evaluation)

The New York District has over twenty years of successful project performance with the Westhampton Project (Long Beach, NY) which is comparable and has an equal berm width. The 25 ft dune crest width was selected based on Section 934 beach renourishment design profile which was constructed through the Flood Control and Coastal Emergencies Act, PL 84-99, (post Sandy). This 25 ft dune crest is considered to be a pre-existing condition and will be preserved.

7.2.3 Project Baseline & Alignment

The project baseline controls the alignment of the project and sets the location of the design shoreline. The project baseline for this project represents that landward toe of the dune (+10’ NAVD88). The seaward edge of the design berm (excluding advance fill) is always located 143 ft seaward of the baseline. In most instances the project baseline follows the seaward edge of the boardwalk. However, in some locations the project baseline deviates from the boardwalk in order to ensure a relatively straight design shoreline that follows the natural shoreline curvature. However, the dune always follows the boardwalk and is not constrained by the project baseline. In some locations, such as Reach 5, the berm width measured from the boardwalk is much greater than 143 feet. The project baseline is the same in all three alternatives, with the exception of Reach 6a where the baseline and boardwalk are relocated further inland in Alternative 2.

7.2.4 Initial Construction Beachfill Quantities

It is impossible to predict the exact shoreline position for the point in time that construction is expected to start since the wave conditions vary from year to year and affect shoreline change rates. Beachfill quantities required for initial construction of each alternative are estimated based on the expected shoreline position in December 2019. A hybrid approach based on numerical model results and historical shoreline change rates have been used to estimate the shoreline position in December 2019. This includes a 3.5 year GENESIS-T numerical model simulation representative of typical wave conditions (detailed in Sub-Appendix A) and the reach specific erosion rates discussed in section 6.3.

Beachfill quantities are based on the difference in the design shoreline position (including advance fill) and the estimated December 2019 shoreline. For every foot that the shoreline at the start of construction needs to be translated seaward it is estimated that it requires 1.22 cy/ft of fill, based on berm elevation of +8 ft NAVD88 and a depth of closure of -25 ft NAVD88. Average end area calculations were performed to convert the difference in shoreline positions to volumes over the entire project area.

The design shoreline is a constant offset from the project baseline since it is uniform throughout the project area. Advance fill is included such that the constructed advance shoreline varies along the project area in conjunction with the expected shoreline change rates (further detailed in Sub-Appendix A). All beachfill quantities, Table 7-1, include an overfill factor of 11% based on the compatibility analysis for the borrow areas. In addition the initial construction quantities include an additional 15% for construction tolerance. It is noted that the advance fill and renourishment quantities do not include tolerance since the purpose of the advance fill and renourishment is to place a specific volume of sediment to offset anticipated losses between renourishment operations, rather than build a specific template.

Table 7-1: Initial Construction Beachfill Quantities

Reach	Length (ft)	Alternative 1	Alternative 2	Alternative 3
Reach 1	12,480			
Reach 2	11,090			
Reach 3	10,320	279,000	290,000	331,000
Reach 4	5,380	383,000	305,000	221,000
Reach 5	10,650	221,000	310,000	290,000
Reach 6a	3,730	375,000	70,000	234,000
Reach 6b	2,000	20,000	20,000	20,000
Total	**55,650**	**1,278,000**	**995,000**	**1,096,000**

7.2.5 Renourishment Operations

GENESIS-T model simulations were used to determine the renourishment quantities (cy/yr), and advance fill requirements for each alternative. Renourishment intervals for the three alternatives is based on a combination of GENESIS-T model simulations and past project performance.

Renourishment Interval

The two prior beach restoration projects, WRDA 1974 and Section 934, had renourishment operations in two-year and four-year cycles respectively. The two-year renourishment cycles during the WRDA 1974 project may have been partially attributed to the nature of the project, feeder beaches, which led to high end losses. Nonetheless, it is apparent from the renourishment quantities that nearly all of the fill placed in the feeder beaches had transported elsewhere two years later.

In contrast the Section 934 project had renourishment operations every four years. However, the beach conditions at the end of the four year cycle were eroded well beyond the design template and a shorter renourishment interval or wider advance berm would likely have been needed to maintain the design template between renourishment operations.

GENESIS-T model simulations for all three alternatives were conducted based on four-year renourishment intervals. The models results for Alternative 1 show that an advance berm width of nearly 80 ft is required in Reach 4 to maintain the design shoreline between renourishment operations. Similarly, in Reach 6a an advance berm width of 55 ft plus the continued inlet maintenance dredging and placement (~115,000 cy/yr) is required. These relatively high advance berm widths are indicative of the erosive nature of these reaches.

One of the advantages of Alternative 2 and Alternative 3 is that With Project shoreline rates are more uniform over the project area. As a result the advance berm widths are more even throughout the project area and less than 60 ft. As a result, it is expected that Alternative 2 and Alternative 3 will on average allow for longer renourishment intervals than Alternative 1.

Based on past project performance a renourishment interval of 3 years is applied in the cost estimates for Alternative 1. A renourishment interval of 4 years is applied to Alternatives 2 and 3.

Shoreline change modeling results indicate that the design beach profile can be maintained over the life of project in all three alternatives if sufficient advance fill is placed and regular renourishment operations are performed. Alternative 1 experienced high sediment losses in the two historical erosional hot spots (EHS) requiring large renourishment quantities and a relatively short 3-year renourishment cycle.

Alternative 2 and 3, reduced the sediment losses in the two historical EHS by either increasing sediment flow into the hot spots (Alternative 2) or reducing sediment flow out of the hot spots (Alternative 3). As a result, Alternative 2 and 3 had lower renourishment quantities and a longer, 4-year, renourishment cycle than Alternative 1.

Renourishment Quantities

A summary of the renourishment quantities, per operation, including overfill and losses from sea level rise is provided in Table 7-2. It is noted that the renourishment quantities for Alternative 1 are based on a 3-year renourishment cycle, whereas the quantities for Alternative 2 and 3 are based on a 4-year cycle.

Table 7-2: Renourishment Quantities per Operation

Reach	Renourishment (cy)		
	Alternative 1[3]	Alternative 2[4]	Alternative 3[4]
Reach 3	133,000	133,000	444,000
Reach 4	416,000	333,000	133,000
Reach 5	250,000	599,000	444,000
Reach 6a	200,000[2]	0[2]	0[2]
Total	999,000	1,065,000	1,021,000

Notes [1] All renourishment quantities include SLR and overfill (11%)

[2] Excludes Inlet Maintenance Dredging (115,000 cy/yr)

[3] 3-year renourishment cycle

[4] 4-year renourishment cycle

7.2.6 Boardwalk Relocation

The Boardwalk Relocation measure entails the demolition and reconstruction of 2,900 feet of boardwalk and is only applicable to Alternative 2. The reconstructed boardwalk would be relocated about 200 to 300 feet landward so that the design shoreline follows the natural curvature of the shoreline reducing renourishment costs.

The old boardwalk was destroyed during Hurricane Sandy, and a new boardwalk has been designed and constructed by NYC Parks. The new boardwalk includes 24" steel pipe piles with a pre-cast concrete pile cap and precast concrete deck plank. The boardwalk also includes sand retaining wall made out of HP 14x89 steel piles and precast concrete panels. The cost estimate for Alternative 2 assumes that some of the materials could be salvaged and reused, such as the 24" steel pipe piles, HP 14x89 piles, and precast concrete panels. However, the majority of the boardwalk would have to be demolished, disposed of, and rebuilt with new materials.

In addition, 6.61 acres of permanent easements are required. The majority of the easements are on parcels owned by the New York City Housing Preservation (HUD). The costs for the permanent easements could be very high making this measure cost prohibitive.

7.2.7 Groin Design

Three types of groin measures are considered in the alternative analysis: new groin construction, groin extension, and groin shortening. The exact dimensions and stone sizes of the existing groins at Rockaway Beach are not available. Therefore, it is assumed that the existing groins in Reaches 5 and 6 are similar to the proposed new groin designs.

Generally a groin is comprised of three sections: 1) horizontal shore section (HSS) extending along the design berm; (2) an intermediate sloping section (ISS) extending from the berm to the design shoreline, and (3) an outer section (OS) that extends from the shoreline to offshore. The head section (HD) is part of the OS and is typically constructed at a flatter slope than the trunk of the groin and may require larger stone due to the exposure to breaking waves.

The spacing between groins in this study is based on the existing spacing in Reach 5 (720 ft) and Reach 6a (780 ft). The required lengths of the new groins is based on the GENESIS-T model simulations.

After identifying the groin lengths and spacing the following steps are applied in the groin design:

1) Layout groin section on typical profile surveys to determine appropriate bed elevations along the groin.
2) Calculate required armor stone size for stability under 100-year wave conditions (CIRIA, 2007).
3) Determine remaining groin dimensions and quantities for a typical groin. The crest width of the groin varies by section and is controlled by the armor stone size ($3D_{50}$).

The typical groin profiles and sections for Reaches 3& 4, Reach 6a, and Reach 5 are shown in Figure 7-3 to Figure 7-7. The groin sections in Reach 5 are the same as in Reaches 3 and 4. A complete list of the groin measures for each alternative, including the length, location, and feature is provided in Table 7-3. It is noted that groin extensions and shortening require removal of the existing groin head (50 ft), stockpiling nearby, and then reconstructing the head with the stockpiled stone. All of the re handling of the stone is reflected in the cost estimate for groin shortening. The cost of removing the head and stockpiling the stone for groin extensions in Reach 6a is approximately equal to the cost of obtaining and transporting the stone to the project site. Therefore, the cost estimate for the groin extensions only reflects the cost to construct 200 to 300 feet of new groin. Typical quantities required to build the groins are presented in Table 7-4 to Table 7-6.

Table 7-3: Summary of Groin Lengths

Alternative	Reach	Number	Groin ID	Street	Horizontal Shore Section (ft)	Intermediate Sloping Section (ft)	Outer Section (ft)	Total (ft)	Notes
Alt 2	5	1		60th St	90	108	278	476	shorten 100'
Alt 2	5	2		62nd St	90	108	178	376	shorten 100'
Alt 2	5	3		65th St	90	108	178	376	shorten 100'
Alt 2	5	4		68th St	90	108	178	376	shorten 100'
Alt 2	5	5		71st St	90	108	153	351	shorten 100'
Alt 2	5	6		74th St	90	108	203	401	shorten 100'
Alt 2	5	7		77th St	90	108	178	376	shorten 100'
Alt 2	5	8		80th St	90	108	203	401	shorten 100'
Alt 2	5	9		83rd St	90	108	228	426	shorten 100'
Alt 2	5	10		86th St	90	108	278	476	shorten 100'
Alt 3	6a	1	63	34th St	62	108	328	498	new 498'
Alt 3	6a	2	62	37th St	55	108	328	491	extension 209'
Alt 3	6a	3	61	40th St	90	108	328	526	extension 307'
Alt 3	5	4	53	43rd St	90	108	228	426	extension 114'
Alt 3	5	5	52	46th St	90	108	228	426	extension 155'
Alt 3	5	6	51	49th St	90	108	228	426	extension 180'
Alt 3	4	1	47	92nd St	66	108	128	302	new 302'
Alt 3	4	2	46	95th St	62	108	128	298	new 298'
Alt 3	4	3	45	98th St	63	108	128	299	new 299'
Alt 3	4	4	44	101st St	62	108	128	298	new 298'
Alt 3	4	5	43	104th St	66	108	128	302	new 302'
Alt 3	4	6	42	106th St	67	108	128	303	new 303'
Alt 3	4	7	41	108th St	66	108	128	302	new 302'
Alt 3	3	8	35	110th St	90	108	153	351	new 351'
Alt 3	3	9	34	113th St	90	108	178	376	new 376'
Alt 3	3	10	33	115th St	90	108	178	376	new 376'
Alt 3	3	11	32	118th St	90	108	178	376	new 376'
Alt 3	3	12	31	121st St	63	108	128	299	new 299'

Table 7-4: Typical Groin Quantities for New Construction Reaches 3 & 4 (Alt 3)

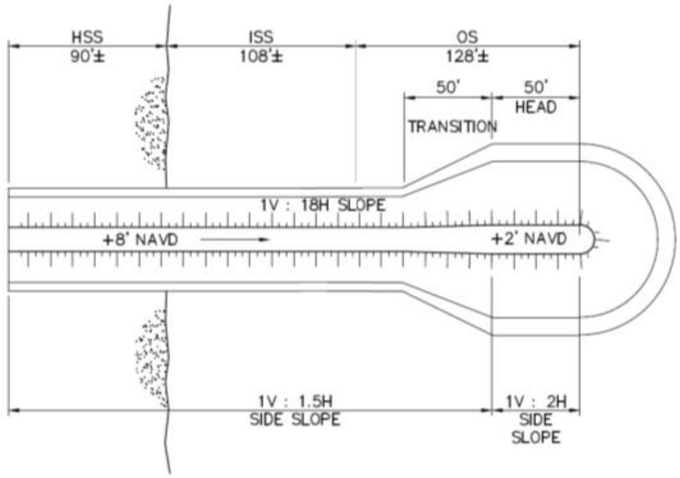
Feature	QTY/ft	UNIT	Length (ft)	Quantity
Armor Stone	13.5	TON	4,087	55,175
Underlayer Stone	4.4	TON	4,087	17,983
Core Stone	2.7	TON	4,087	11,035
Blanket Stone	6.9	TON	4,087	28,200
Geotextile Filter	7.7	SY	4,087	31,470
Excavation - Sand	23.1	CY	4,087	94,410

Table 7-5: Typical Groin Quantities for New Construction Reach 6a (Alt 3)

Feature	QTY/ft	UNIT	Length (ft)	Quantity
Armor Stone	13.0	TON	526	6,838
Underlayer Stone	4.6	TON	526	2,420
Core Stone	2.8	TON	526	1,473
Blanket Stone	6.3	TON	526	3,314
Geotextile Filter	7.5	SY	526	3,945
Excavation - Sand	18.6	CY	526	9,784

Table 7-6: Typical Groin Quantities for Extensions Reach 6a (Alt 3)

Feature	QTY/ft	UNIT	Length (ft)	Quantity
Armor Stone	24.7	TON	800	19,760
Underlayer Stone	5.2	TON	800	4,160
Core Stone	3.3	TON	800	2,640
Blanket Stone	10.4	TON	800	8,320
Geotextile Filter	9.4	SY	800	7,520
Excavation - Sand	15.3	CY	800	12,240



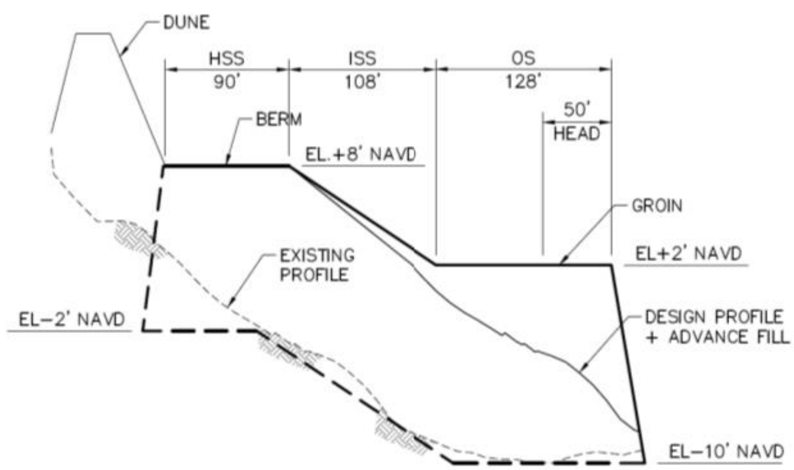


Figure 7-3: Typical Groin in Reaches 3 & 4 – Profile View

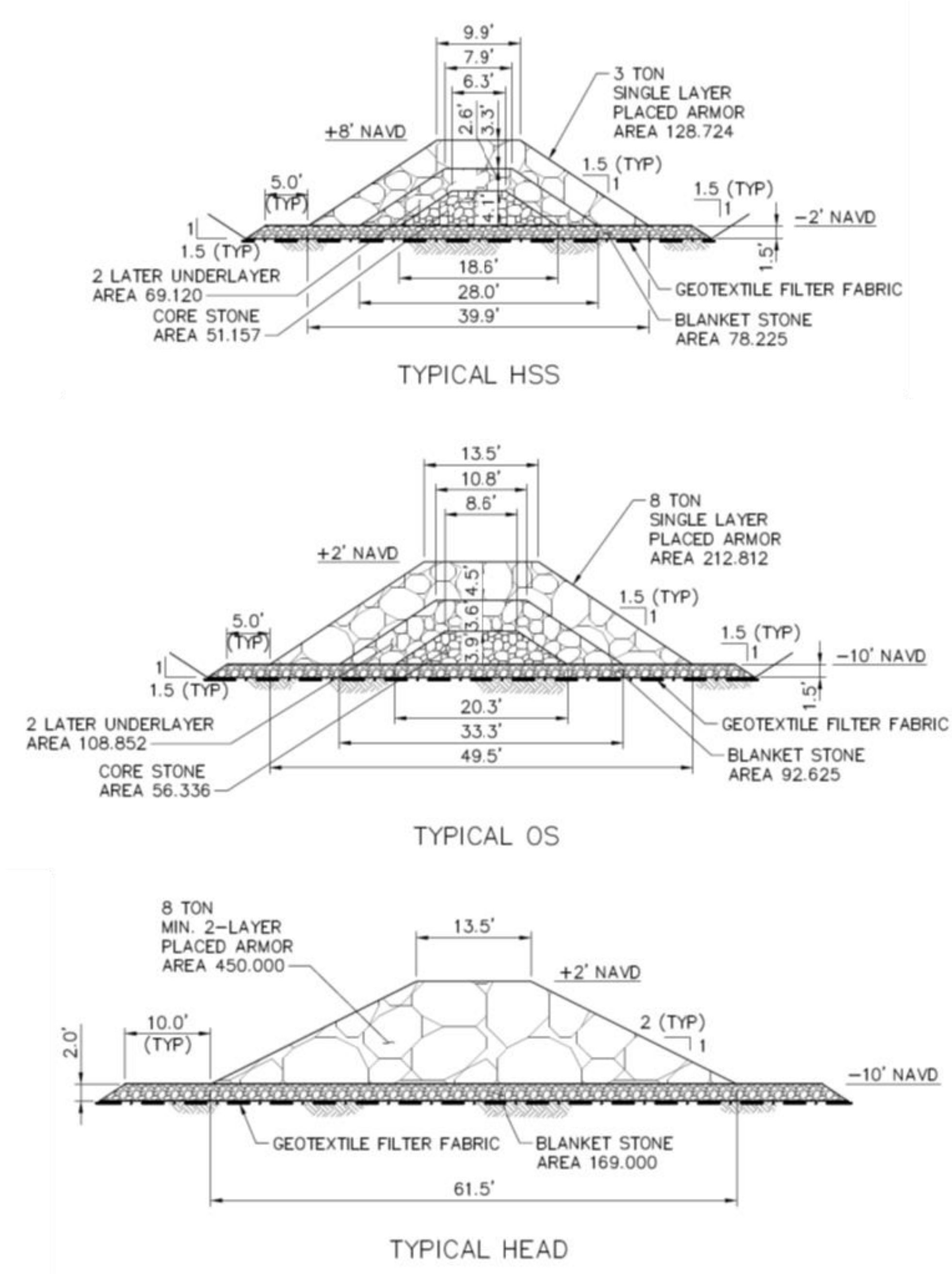


Figure 7-4: Typical Groin in Reaches 3 & 4 – Section View

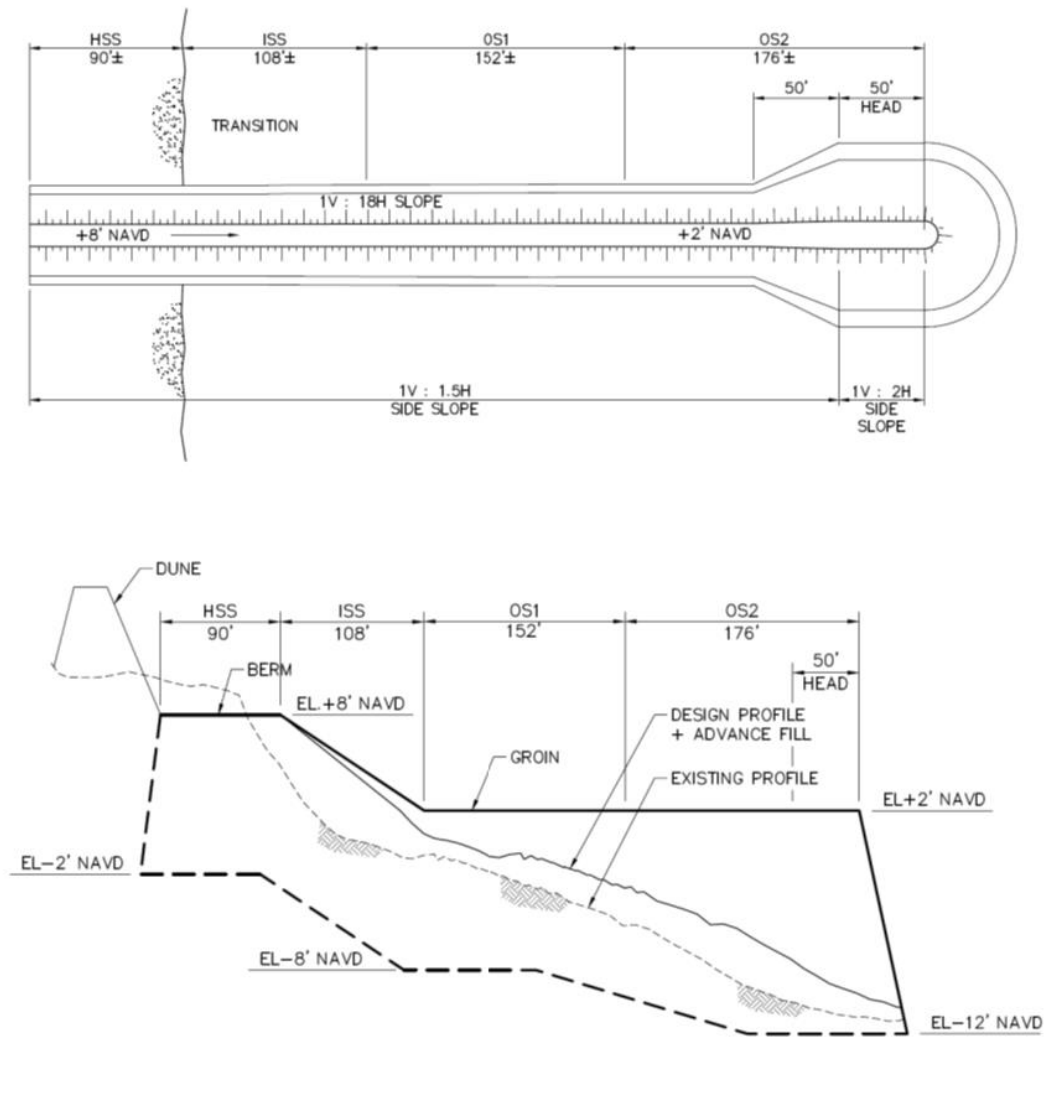


Figure 7-5: Typical Groin in Reach 6a – Profile View

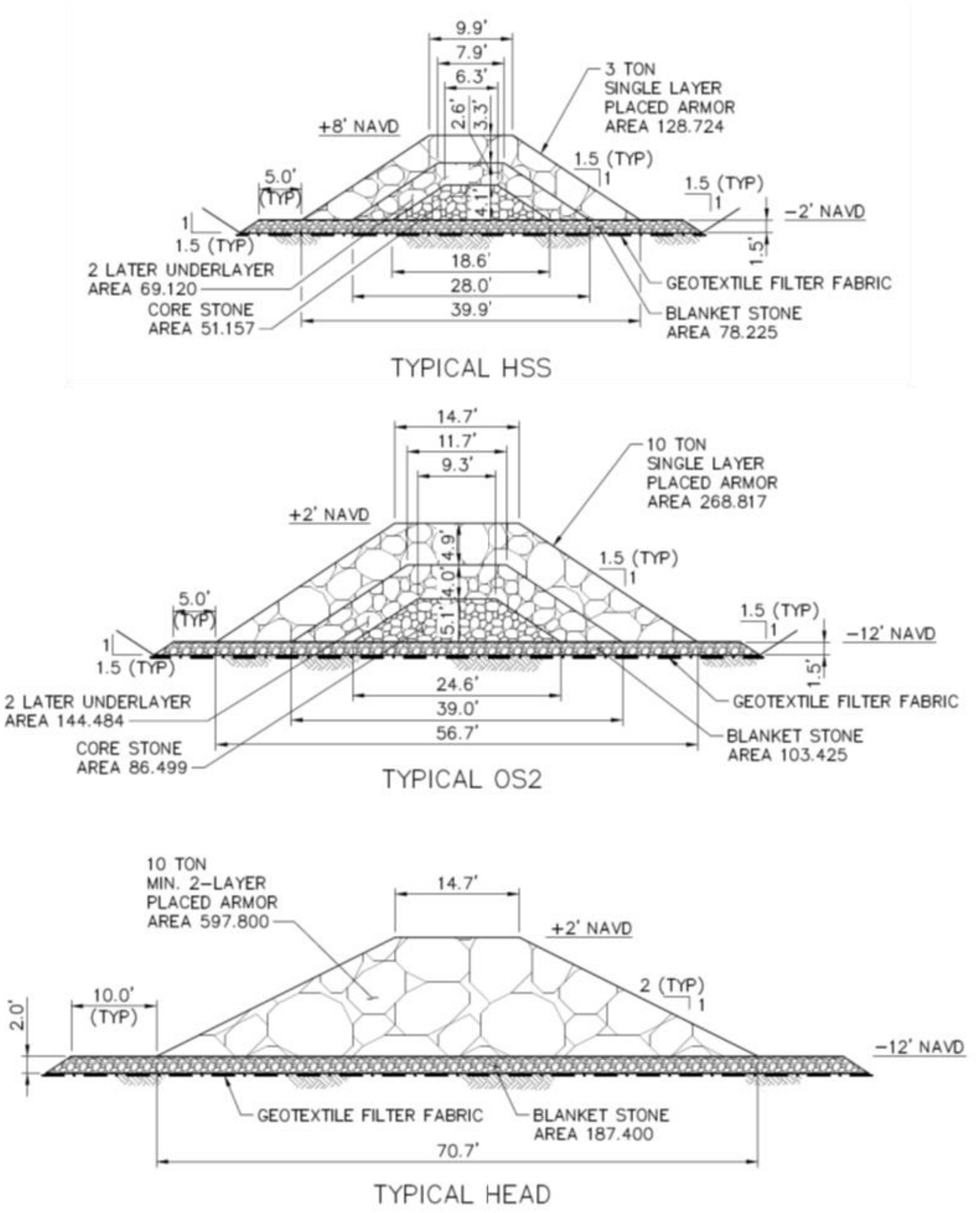


Figure 7-6: Typical Groin in Reach 6a – Section View

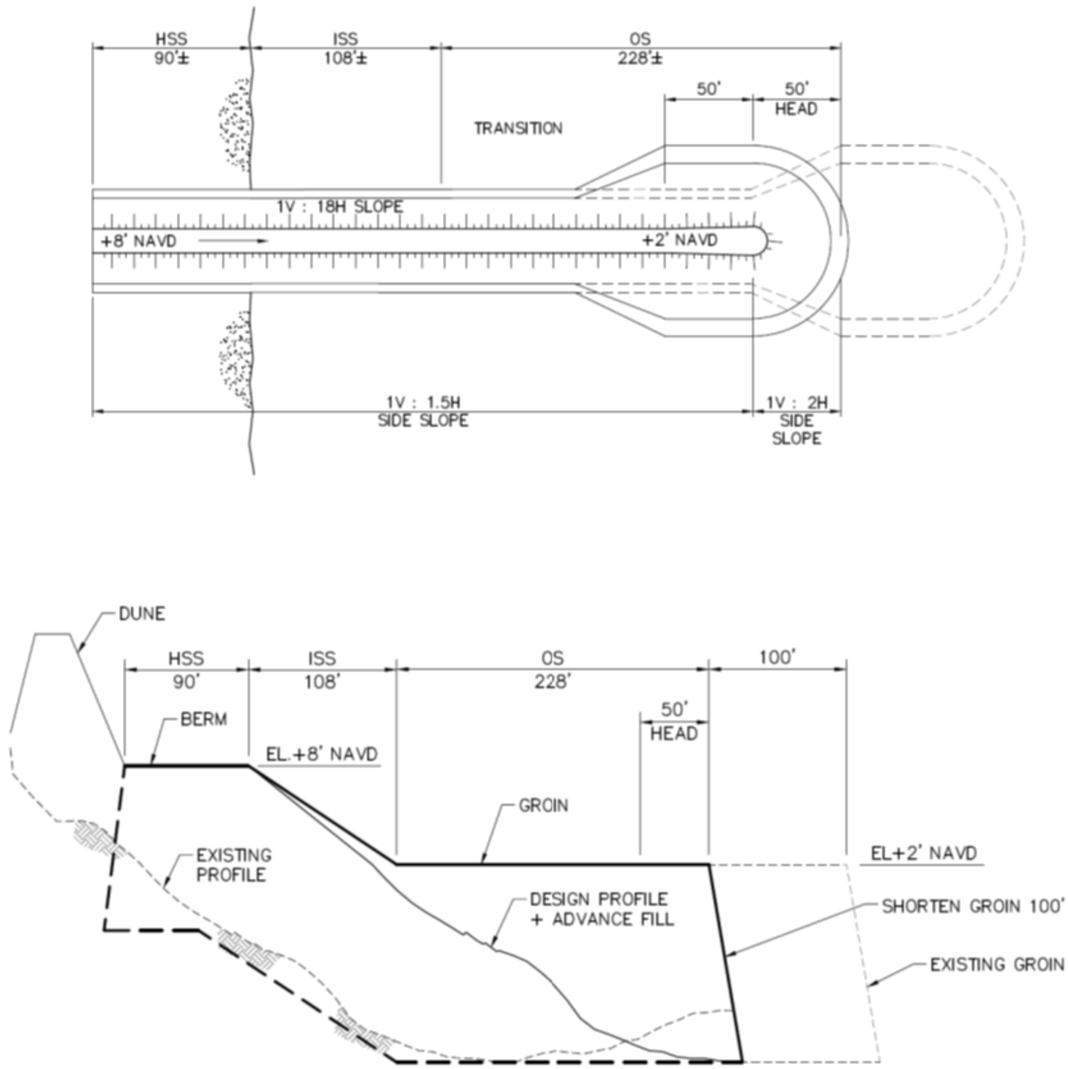


Figure 7-7: Typical Groin Shortening in Reach 5 – Profile View

7.2.8 Cost Estimates

Life-cycle cost estimates for the three alternatives are presented here. An exhaustive set of cost estimates is provided in Appendix C "Cost Estimates". For brevity only a summary of the initial construction, renourishment, and annualized costs are presented here. The effective price level for the cost estimate is the second quarter of 2018, and a 2.750% interest rate, and 50-year project life is used to determine the annualized cost. Interest during construction (IDC) is also included for each of the alternatives based on their respective construction durations.

Dredging costs for the project are estimated based on the USACE Dredging Software CEDEP and a MII cost estimate for the shore crew. A beachfill unit price of $12.88 per cubic yard and mob/demob of $3.3 million is used throughout the cost estimates.

The unit prices for groin construction are based on an MII cost estimates using local labor rates. An overview of unit prices for all construction items is provided in Table 7-7.

Table 7-7: Unit Prices

Item	Unit Price	Source
Construction		
Armor Stone Placement	$193.30 per TON	MII Cost Estimate
Underlayer Stone Placement	$104.68 per TON	MII Cost Estimate
Core/Bedding Stone Placement	$92.87 per TON	MII Cost Estimate
Geotextile Placement	$23.68 per SY	MII Cost Estimate
Beachfill Placement	$12.88 per CY	CEDEP & MII Cost Estimate
Dune Grass Placement	$20,000 per Acre	SSSI Cost Estimate
Sand Excavation	$7.24 per CY	MII Cost Estimate
Armor Stone Rest	$66.38 per TON	MII Cost Estimate
Bedding Stone Rest	$27.54 per TON	MII Cost Estimate
Removal		
Armor Stone Removal	$59.74 per TON	MII Cost Estimate
Underlayer Stone Removal	$43.38 per TON	MII Cost Estimate
Core/Bedding Stone Removal	$34.70 per TON	MII Cost Estimate
Geotextile Removal	$9.18 per SY	MII Cost Estimate
Disposal		
Armor Stone	$118 per TON	~2/3 of Construction Costs
Underlayer Stone	$66 per TON	~2/3 of Construction Costs
Core/Bedding Stone	$52 per TON	~2/3 of Construction Costs
Geotextile	$9 per SY	~1/3 of Construction Costs

Notes: effective price level January 2015

The total initial cost for the demolition and reconstruction of the 2,900 ft long section of the boardwalk is $27,129,000 (including a 33.46% contingency). The real estate costs associated with obtaining the permanent easements could be as high as $40 million to $60 million due to (1) loss of first and possibly second floor views; and (2) the loss of air rights for a high rise project. In addition the loss of property reduces the FAR (Floor Area Ratio) leading to a potential loss in profit from less developable space on the remaining parcel. The high costs associated with the real estate preclude Alternative 2 from being viable. The life-cycle cost estimates use $59,588,000 (including a 48.97% contingency) as the estimate for the real estate costs associated with boardwalk relocation.

The cost estimates also include Planning, Engineering and Design (PED) and Construction Management. PED is estimated to be equal to 10% of the construction costs. Construction management is estimated as a function of the construction costs and is approximately 8% the construction costs. An Abbreviated Risk Analysis (ARA) was completed to estimate risk based contingencies associated with the various construction features for each alternative.

Initial construction and renourishment operation cost estimates for the three alternatives are presented in Table 7-8 and Table 7-9. Alternative 1 has the lowest initial construction costs, but the highest renourishment costs. The cost of each renourishment operation in Alternative 1 is about the same as in Alternative 2 and 3 but because renourishment operations are required every 3 years for Alternative 1, instead of every 4 years the total renourishment costs for Alternative 1 are much higher.

Renourishment costs for Alternative 2 and 3 are similar, however the initial construction costs for Alternative 3 are much lower. The initial construction costs for Alternative 2 are relatively high due to the real estate costs associated with boardwalk relocation.

A summary of the overall life-cycle cost estimate for each alternative is presented in Table 7-10. The recommended alternative is Alternative 3 Beach Restoration and increased erosion control. This alternative had the lowest annualized costs over the 50-year project life and the lowest renourishment costs over the project life. However, the difference in the annualized cost estimates between Alternative 1 and Alternative 3 is relatively small, 2%, and well within the margin of uncertainty in the cost estimates.

Table 7-8: Erosion Control - Initial Construction Cost Estimates

Item	Alternative 1	Alternative 2	Alternative 3
Beachfill	$22,848,000	$19,190,000	$20,504,000
Groins	$0	$11,498,000	$27,844,000
Boardwalk Relocation	$0	$59,677,000	$0
PED	$2,285,000	$5,037,000	$4,835,000
Construction Management	$1,794,000	$3,591,000	$3,462,000
Contingency	$5,923,000	$42,276,000	$14,372,000
Total	**$32,850,000**	**$141,269,000**	**$71,017,000**

Notes: effective price level Q2 2018

Table 7-9: Erosion Control - Renourishment Cost Estimates

Item	Alternative 1[1]	Alternative 2[2]	Alternative 3[2]
Beachfill	$16,167,000	$17,017,000	$16,450,000
PED	$1,617,000	$1,702,000	$1,645,000
Construction Management	$1,319,000	$1,382,000	$1,341,000
Contingency	$6,183,000	$6,507,000	$6,291,000
Total Per Operation	**$25,286,000**	**$26,608,000**	**$25,727,000**
Total Over Project Life[3]	**$186,579,322**	**$143,626,085**	**$140,943,415**

Notes: [1] 3-year renourishment cycle

[2] 4-year renourishment cycle

[3] Present Worth using 2.750% interest rate

Table 7-10: Erosion Control Life-Cycle Cost Estimates – Atlantic Ocean Shorefront Formulation Summary

		Low SLR		
		Alternative 1	**Alternative 2**	**Alternative 3**
Initial Cost	Initial Construction	$32,850,000	$141,269,000	$71,017,000
	IDC	$215,000	$2,207,000	$1,307,000
	Investment Cost	**$33,065,000**	**$143,476,000**	**$72,324,000**
Annualized Cost	Investment Cost	$1,225,000	$5,314,000	$2,679,000
	Renourishment (Planned/Emergency)	$7,975,000	$6,153,000	$5,950,000
	O&M	$403,000	$465,000	$579,000
	Major Rehab	$332,000	$332,000	$332,000
	SLR Adaption	$0	$0	$0
	Total Annual Cost	**$9,935,000**	**$12,264,000**	**$9,540,000**

Note: Effective price level Q2 2018

7.3 Coastal Storm Risk Management Alternatives

Five coastal storm risk management alternatives are under consideration:

1. Beach Restoration, +16 foot Dune, 60 foot Berm
2. Beach Restoration, +18 foot Dune, 80 foot Berm
3. Beach Restoration, +20 foot Dune, 100 foot Berm
4. Beach Restoration, +18 foot Reinforced Dune – Buried Seawall
5. Beach Restoration, +18 foot Reinforced Dune – Composite Seawall

All of the alternatives include beach restoration and a dune. The most cost effective erosional control features described above will be included as part of all five of the coastal storm risk management alternatives.

A screening analysis was performed prior to the detailed economic modeling to narrow down the number of possible alternatives to five. The screening analysis evaluated the performance under a range of dune and berm dimensions as well as reinforced dunes to aid in the selection of appropriate dimensions. Other factors such as prior projects at Rockaway Beach, project constraints, stakeholder concerns, and engineering judgment were also applied in the selection of the final set of alternatives.

7.3.1 Prior Projects

There have been three major beach restoration project at Rockaway Beach since the 1970s:

- WRDA 1974 Beach Erosion Control Project (1975-1988)
- Section 934 Beach Erosion Control Project (1996-2004)
- FCCE Project (2013-2014)

A direct comparison of the design profiles for the WRDA 1976, Section 934, and FCCE projects is difficult since these projects were defined by construction templates. Construction templates are the profile constructed by the dredging contractor in field and typically have a wider berm and steeper slope than the natural profile. Immediately after construction the construction template begins to adjust to the local wave conditions often resulting in a cross-shore transfer of sediment from the upper half to the lower half of the profile.

Design profiles are an idealized profile that has adjusted to the local site conditions and matches the natural shape of the beach profile. If the pre-construction profile is known the design profile dimensions may be estimated from the construction template. However, the adjusted design profile dimensions are very sensitive to the pre-construction profile. At Rockaway Beach for example, the adjusted design dimensions will be very close to the constructed dimensions if pre-construction profile is relatively wide (e.g. Reaches 3 and 5). However, if the pre-construction profile is much narrower than the construction template (i.e. Reach 4) then adjusted design berm width may be approximately half the width of the construction berm width. The pre-construction conditions at Beach 108th Street in Reach 4 are used here to characterize the three projects. Figure 7-8 to Figure 7-10 show the construction template and corresponding design profile for all three projects in Reach 4. Table 7-11 provides a summary of the construction beach berm width and design beach berm. The beach berm width is measured from the seaward edge of the boardwalk to the seaward crest of the berm.

Table 7-11: Adjusted Beach Berm Width for Prior Projects (Including Advance Fill)

Project	Construction Berm Width (ft)[1]	Design Beach Berm Width (ft)[1]
WRDA 1974	200	120 (134[2])
Section 934	160	106 (120[2])
FCCE	280	180

Notes: [1] Design Beach Berm Width measured from profile origin/boardwalk.

[2] Design Berm Width at +8' NAVD88

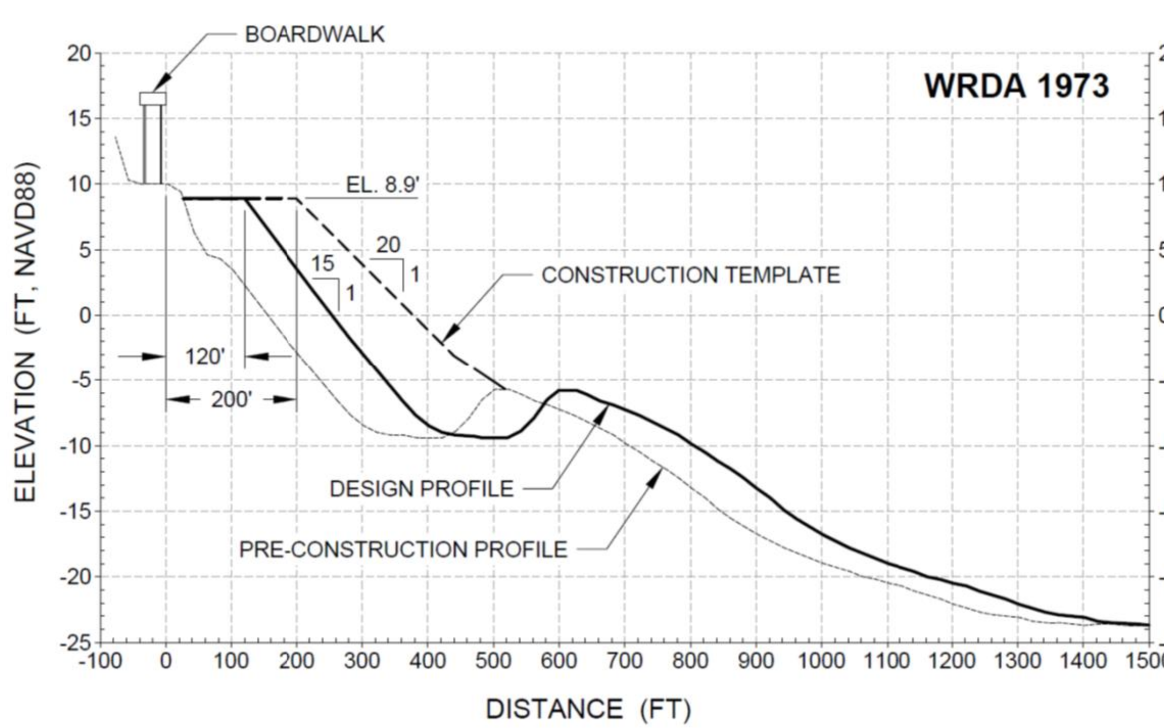


Figure 7-8: WRDA 1974 Construction Template and Design Profile (Reach 4)

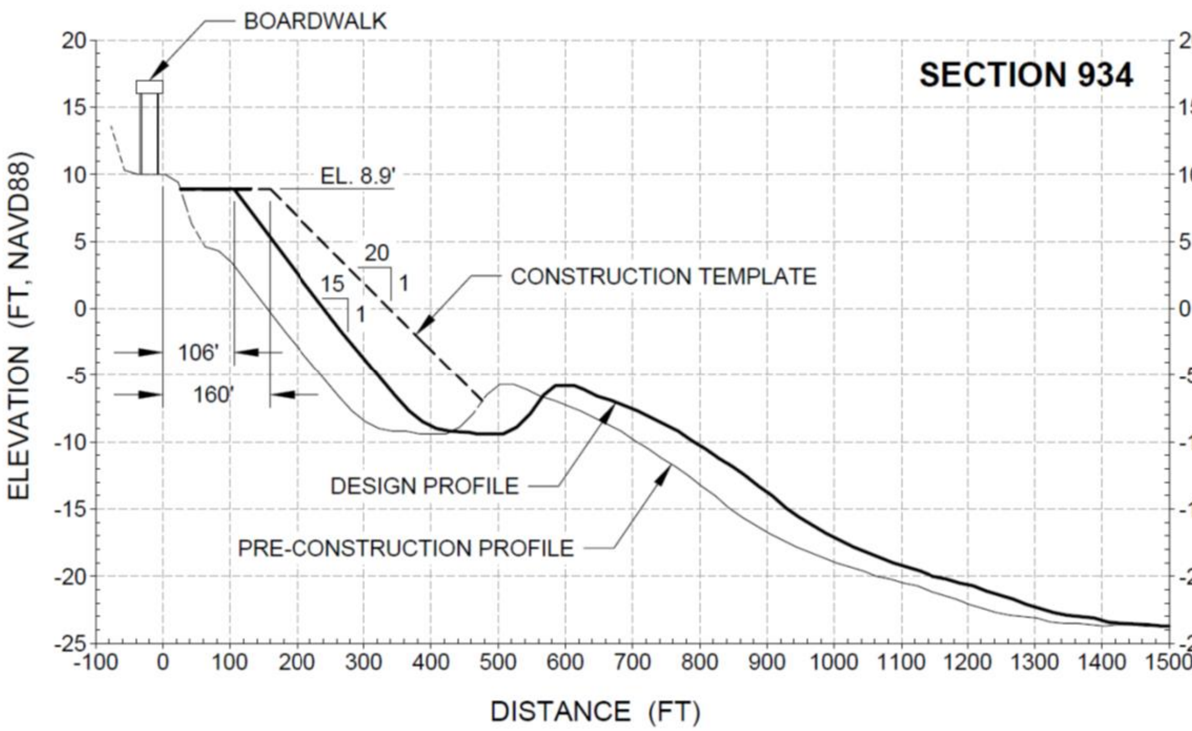


Figure 7-9: Section 934 Construction Template and Design Profile (Reach 4)

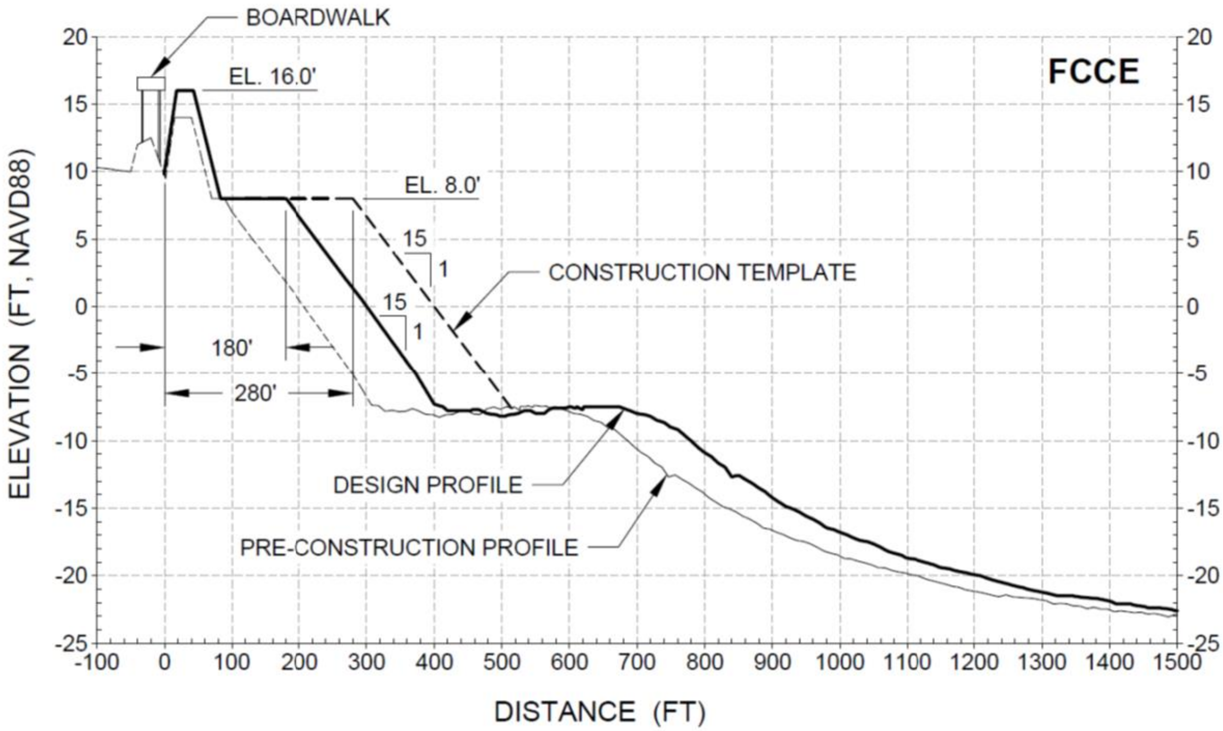


Figure 7-10: FCCE Construction Template and Design Profile (Reach 4)

7.3.2 Performance

A screening analysis was performed for a range of design beachfill profile and reinforced dune sections. The screening analysis considered a range of dune heights and berm widths as well as several different reinforced dune design concepts. The purpose of the screening analysis is to narrow down the number of possible profile combinations for further evaluation in Beach-fx. The screening analysis led to the selection of three design beachfill profiles and two reinforced dune designs to be further evaluated in Beach-fx.

The screening analysis evaluated the probability of exceedance for a range of dune and berm dimensions as well as reinforced dunes is performed to aid in the selection of appropriate dimensions. Other factors such as prior projects at Rockaway Beach, project constraints, stakeholder concerns, and engineering judgment were also applied in the selection of the final set of design profiles.

7.3.3 Beach Restoration and Dune Alternatives

Range of Proposed Beachfill Configurations

To screen potential design beach profiles the following combinations of berm width and dune height are evaluated:

- Berm Widths: 0, 50, 100, 120, 150, 180, and 210 feet;

- Dune Height: 14, 16, 18, 20, and 22 feet NAVD88.

The berm elevation and dune width for all the beachfill design sections are +8 feet NAVD88 and 25 feet, respectively. The berm width is measured from the seaward toe of the dune. Whereas, the "beach berm width" is measured from the project baseline / boardwalk.

Screening Criteria

The level of risk reduction for each of the design beachfill profile configurations was estimated based on three separate screening criteria: dune lowering, upland flooding, and wave overtopping. The level of risk reduction is defined as the return period of an event that produces storm conditions that exceed the design thresholds. The return period is the inverse of the probability that the event will be exceeded in any one year. For example, a 10 year flood has a 1/10 or 10% chance of being exceeded in any one year and a 500 year flood has a 1/500 or 0.2% chance of being exceeded in any one year.

Three beach profile response parameters are used to evaluate the level of risk reduction associated with each of the design beach profiles. These response parameters and corresponding failure threshold values are:

- Dune Lowering: 20% reduction in dune height (measured from the berm);
- Upland Flood Depth: 1.0 feet;
- Wave Overtopping: 1.0 cfs per foot (93 liters/s/m).

Two of the three criteria, dune lowering and wave overtopping, were also considered in the selection of the design beach profiles for Fire Island to Montauk Point (FIMP) (Alfageme, 2001). A third criterion, upland flood depth, is considered here to provide a link between the damages calculated by Beach-fx and the level of risk reduction. The upland flood depth is defined in this study as the difference in elevation between the post-storm profile and the maximum water surface elevation including wave setup but excluding wave crests and wave runup. There are three damage driving parameters in Beach-fx: water level, wave height, and vertical erosion. Since the upland wave heights are depth limited, the upland flood depth and upland wave height are very closely correlated and it would be redundant to also consider upland wave heights. Vertical erosion is not expected to be a major driver of damages at the Atlantic Ocean Shorefront under the with-project conditions and is therefore not included in the screening criteria.

Note that although these values were selected somewhat arbitrarily all three criteria capture the onset of significant erosion from dune lowering, upland flooding, or wave overtopping. In addition all three criteria yield similar results providing an additional level of comfort and suggest that the selected criteria are consistent. Ultimately, the wave overtopping criterion, 1.0 cfs per foot, was used to determine the level of risk reduction of the profiles in FIMP (Alfageme, 2001).

A comparison of the level of risk reduction evaluated from each of the three criteria is provided below. Similarly to FIMP, wave overtopping generally limits and defines the level of risk reduction.

Screening Methodology

SBEACH model simulations, previously used to develop the Beach-fx Storm Response Database (SRD), are reused here to develop response versus frequency curves for dune lowering, upland flood depth, and wave overtopping. SBEACH model simulations were performed for each profile and all 456 historical storm combinations. Profile responses (e.g. dune lowering, and upland flood depth) were extracted from the SBEACH model output.

Wave overtopping on the beachfill profiles dune was calculated based on the Van der Meer (1995, 1998) methodology intended for estimating wave overtopping on coastal structures, the Kobayashi et al. (1996) extension of the Van der Meer formula to sandy dunes based on an equivalent uniform beach slope parameter, and the Alfageme (2001) empirical coefficient adjustment based on large scale tests performed by Delft Hydraulics Laboratory (1983). Attachment A "FIMP - Selection of Alternative Design Beach Profiles" provides additional details about the wave overtopping calculations. Note that a different wave overtopping analysis is applied for the seawall structures and is discussed in more detail later in the memorandum.

Response versus frequency curves were generated for the selected screening criteria by applying a multivariate Empirical Simulation Technique (EST) analysis. Multivariate EST is a statistical procedure used to develop design parameter frequency relationships as a function of the input parameters (e.g. offshore wave height, storm surge, tide) that are descriptive of the storm event but have unknown joint probabilities. The multivariate EST approach applied in this study is the same as the one used for FIMP (Gravens et al., 1999). Details of the approach, as applied to wave setup along Rockaway Beach, are provided in Gravens et al. (1999) "Fire Island Inlet to Montauk Point, Reformulation Study (FIMP): Historical and Existing Condition Coastal Processes Assessment." The nine input parameters capture the variability in the maximum and average value of the storm surge, tide, and wave height as well as the duration of the storm event.

The multivariate EST analysis was modified to include three additional responses: upland flood depth, upland wave height, and wave overtopping.

Results

The smallest design beachfill profile recommended for further consideration is slightly narrower than the FCCE project but wider than the prior WRDA 1974 and Section 934 projects, with a dune height of +16 ft NAVD88 and a berm width of 60 feet. The other two design profiles expand on this profile with wider berms, higher dunes, and greater levels of risk reduction. The dimensions of the three design beach profiles and associated level of risk reduction is provided in Table 7-12. The total beach berm width including advance fill, measured from the boardwalk, is also shown in Figure 7-11. Although Beach-fx was applied to optimize the (minimum required) beachfill design configuration, it is common practice to add advance fill until the next scheduled nourishment to guarantee the design level of protection and reduce the risk of damages due to unexpected severe storm occurrence prior to the first renrounishment. The advance fill within the project area varies from reach to reach based on the expected erosion rates with a typical value along the shoreline of approximately 30 feet.

Table 7-12: Design Beach Profiles

Design	Dune Height (feet, NAVD88)	Design Berm Width (feet)	Beach Berm Width (feet)	Level of Risk Reduction (average return period in years)
Medium	+16	60	173[1]	44
Large	+18	80	209[1]	70
XL	+20	100	245[1]	100

Notes: [1]Includes an advance berm width of 30 feet.

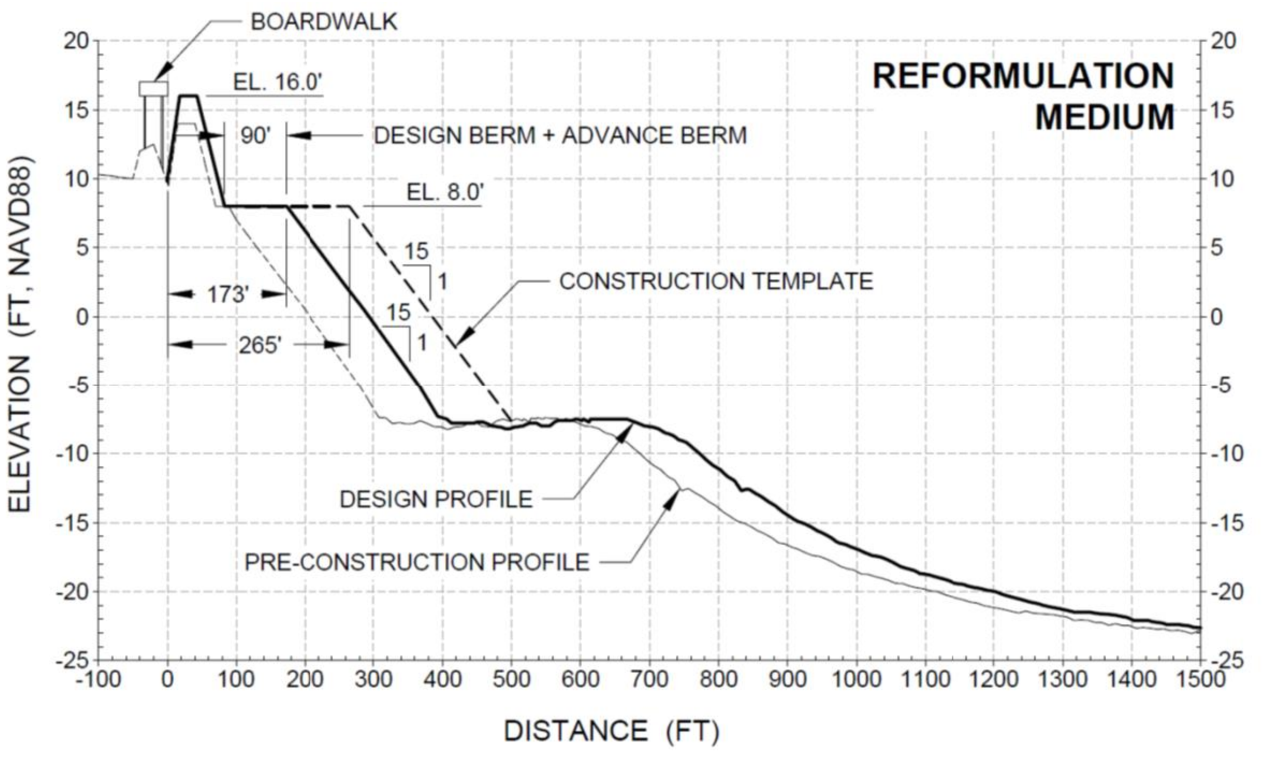


Figure 7-11: Medium Design Profile

Reinforced Dune Alternatives

Two reinforced dune concepts have been proposed for the Atlantic Ocean Shorefront. The first type, buried seawall, is designed to protect inland areas from erosion and wave damages during severe storm events such as Hurricane Sandy. The second type, composite seawall, is designed to also limit storm surge inundation and cross-island flooding during severe storm events. The composite seawall is compatible with a comprehensive storm surge barrier for Jamaica Bay.

Buried seawalls are essentially dunes with a reinforced rubble mound core. An example of constructed buried seawall is shown in Figure 7-12 (Dam Neck, Virginia). Buried seawalls were developed as an alternative to larger standalone seawalls and are designed to function in conjunction with beach restoration projects and dunes. The primary advantage of buried seawalls over traditional dunes are the additional protection against erosion and wave attack provided by the stone core. As an example, during Hurricane Sandy a long forgotten stone seawall in Bay Head, NJ originally built in 1882 that had formed the core of a dune, protected a community from severe damage from wave attack (Irish et al. 2013). The adjacent community (Mantoloking, NJ) only had a sandy dune which was eroded during Hurricane Sandy exposing the island to damaging waves and leading to the formation of a breach. The relic buried seawall in Bay Head, NJ was shown by Irish et al. to reduce potential wave loads by a factor of two and was the difference between widespread destruction (Mantoloking, NJ) and minor structural impacts (Bay Head, NJ).

The proposed buried seawall for the Atlantic Ocean Shorefront is very similar to the examples in Bay Head, NJ and Dam Neck, VA. Since the purpose of the structure is wave protection, it may be constructed intermittingly along the shoreline in the most vulnerable areas.

Figure 7-12: Buried Seawall Example at Dam Neck, Virginia (USACE, 1999)

The second reinforced dune concept is a composite seawall with an impermeable core (i.e. steel sheet pile). The purpose of the composite seawall is to not only protect against erosion and wave attack but also to limit storm surge inundation and cross-island flooding. The composite seawall provides a high level of risk reduction that may not be practical to achieve with a dune because of the necessary height and footprint of such a dune. In addition, the composite seawall is compatible with a comprehensive storm surge barrier for Jamaica Bay.

Several design concepts were initially considered for the seawall: rubble mound, vertical steel sheet pile wall, vertically composite, and horizontally composite structure. The vertical sheet pile wall and vertically composite wall were eliminated from further consideration because of the large lateral forces acting on the steel sheet pile and the required length and size of steel piles to withstand these forces. The armor stone in horizontally composite structures has been demonstrated to significantly reduce the wave breaking pressure (Takahashi, 2002). This allows smaller steel sheet pile walls to be used in the design if the face of the wall is completely protected by armor stone as shown in Figure 7-13 (d).

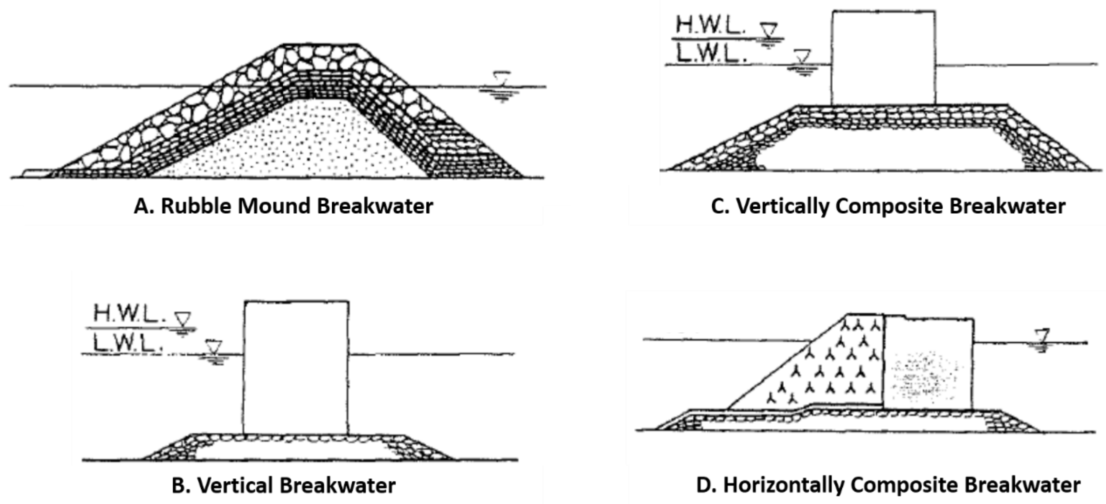


Figure 7-13: Seawall/Breakwater Structure Types (from Takahashi, 2002)

The rubble mound seawall and horizontally composite seawall have been selected for further evaluation and screening level cost estimates. The rubble mound seawall is necessary in locations where there is not an existing boardwalk (i.e. Reach 3). Both seawalls behave similarly from a wave overtopping perspective since both structures have a rubble slope on the seaward side of the structure.

The remaining sections will discuss the screening level analyses applied in the selection of an appropriate crest elevation for the reinforced dune structures.

7.4 Buried Seawall

The required crest elevation of the buried seawall was set in order to provide wave protection against sandy scale storm events. An analysis of wave transmission coefficients for low-crested breakwaters over a range of crest elevations was performed. Wave transmission through a low-crested stone breakwater is dependent on the structure geometry, principally the crest freeboard, crest width, water depth, and permeability, as well as the wave period and surf similarity parameter. The simplified prediction method from CIRIA (2001) which relates the relative crest height of the structure to the wave transmission coefficient is used here to estimate the wave transmission (Figure 7-14). The CIRIA wave transmission relationship indicates that roughly a 50% decrease in the transmitted wave height occurs if the structure freeboard (Rc) is zero. Therefore, the crest elevation of the structure must be roughly equal to the local water level at the toe of the structure (including wave setup) to achieve a 50% reduction in the wave transmitted wave height. Table 7-13 provides a summary of the wave transmission coefficient analysis.

Table 7-13: Wave Transmission Coefficients

Return Period (years)	Local Water Level[1] (ft, NAVD88)	Crest Elevation (ft, NAVD88)			
		12'	14'	16'	18'
50	12.2	0.51	0.10	< 0.1	< 0.1
100	14.5	0.80	0.56	0.17	< 0.1
150	15.7	> 0.8	0.75	0.41	0.10
200	16.6	> 0.8	0.80	0.55	0.24
250	17.2	> 0.8	0.80	0.63	0.34
500	19.1	> 0.8	> 0.8	0.80	0.59

Notes: [1]Includes wave setup.

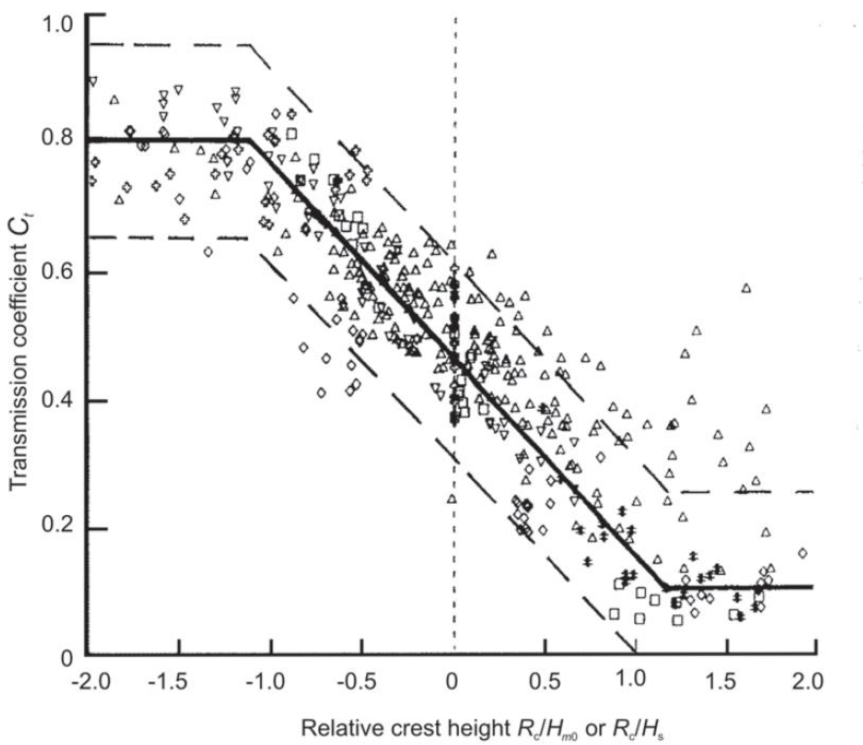


Figure 7-14: CIRIA Relationship for Wave Transmission at Low-Crested Structures

7.5 Composite Seawall

Rubble mound or horizontally composite seawalls are considered here as an alternative approach to large dunes to provide high levels of risk reduction. All the seawall configurations are based on the same general seawall design: 1-layer rubble structure with an impermeable core, and a 1V:2H slope.

7.5.1 Range of Proposed Seawall Configurations

To determine the optimum seawall sections the following seawall sections are evaluated:

- Seawall Crest Elevation: 14, 16, 18, and 20 feet NAVD88;
- Berm Widths: 0, 50, 100, 120, 150, 180, and 210 feet.

The berm elevation for all the design sections is +8 feet NAVD88.

7.5.2 Screening Criteria

Two response parameters are used to evaluate the level of risk reduction of the seawall sections:

- Upland Flood Depth: 1.0 feet;
- Wave Overtopping: 1.0 cfs per foot (93 liters/s/m).

It is important to note that seawall will provide additional benefits even after the level of risk reduction is exceeded as the seawall will still likely prevent direct wave attack on the leeside of the structure.

7.5.3 Screening Methodology

Similarly to the beachfill analysis, SBEACH modeling results and a multivariate EST are used to develop response versus frequency curves for upland flood depth and wave overtopping.

The methods in the EurOtop Overtopping Manual (2007) were adopted to perform the wave overtopping analysis on rubble seawalls. The wave overtopping methodology applied in the previous section on dunes is not applicable to rubble seawalls. The input parameters, significant wave height at the toe of the structure are extracted from SBEACH simulations and the peak wave period and still water level are extracted from wave and water level boundary condition files. The upland flood depth was then calculated based on the wave overtopping rate and uniform open channel flow theory (Manning's formula). Details of the upland flooding approach may be found in Sub-Appendix A.

7.5.4 Results

A crest elevation of +16 ft NAVD88 is recommended for the buried seawall because it reduces the transmitted wave heights by 50% during a 150 year wave event. A smaller buried seawall (e.g. +14 ft NAVD88) would not provide as significant reduction in the transmitted wave heights during large storm events. A larger buried seawall may become costly and would face the same problems as the XL dune (e.g. view shed and footprint). The crest elevation of the proposed structure is the same as the buried seawall in Bay Head, NJ which also had a crest elevation of +16 ft NAVD88.

A crest elevation of +17 feet NAVD88 is recommended for the composite seawall, which is the same elevation as the boardwalk. The composite seawall provides an alternative to the XL dune which may not be supported by stakeholders due to its impact on view shed and its relatively large footprint on the beach. The composite seawall is also compatible with a comprehensive storm surge barrier for Jamaica Bay. A larger horizontally composite seawall is not recommended for further consideration because of the jump in costs and stakeholder concerns if the structure were to exceed the crest elevation of the boardwalk.

Table 7-14: Recommended of Reinforced Dunes

Structure Type	Structure Crest Elevation (feet, NAVD88)	Dune Elevation	Design Berm Width (feet)	LORR[1] (years)
Buried Seawall	+16	+18	60	100[2]
Composite Seawall	+17	+18	60	150[2]

Notes: [1] Level of Risk Reduction (LORR)

[2] Provides additional risk reduction for wave attack

7.5.5 Screening Level Design

7.5.5.1. Beach Restoration and Dune Alternatives

The smallest design beach fill profiles alternatives under consideration is slightly narrower than the Flood Control and Coastal Emergencies (FCCE) project but wider than the prior WRDA 1974 and Section 934 projects, with a dune height of +16 ft NAVD88 and a berm width of 60 feet. The two additional design beach fill profiles under consideration have wider berms and higher dunes (Figure 7-15). The dimensions of the three design beach profiles and associated level of risk reduction is provided in Table 7-15. Using Figure 7-15 and the figures presented in section 7.3.1 allows for the comparison of the alternative dimensions to prior projects.

Table 7-15: Recommended Design Beachfill Profiles

Design	Dune Height (feet, NAVD88)	Design Berm Width (feet)	LORR[1] (years)
Medium	+16	60	44
Large	+18	80	70
XL	+20	100	100

Notes: [1] Level of Risk Reduction (LORR)

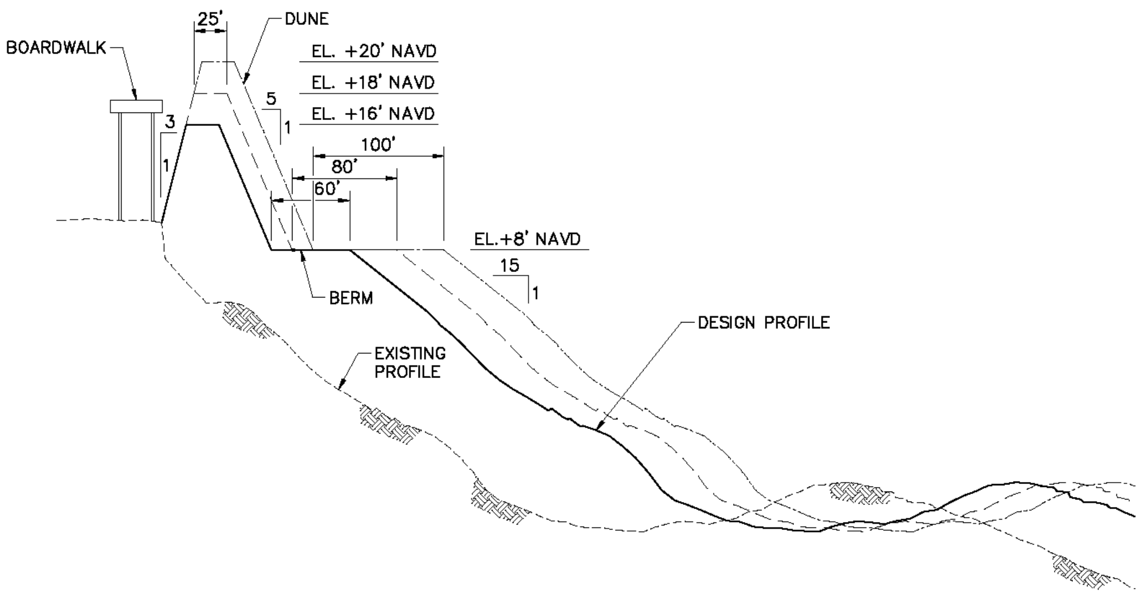


Figure 7-15: Beach Restoration and Dune Alternatives

7.5.5.2. Beach Restoration and Reinforced Dune Alternatives

Two reinforced dune concepts have been proposed for the Atlantic Ocean Shorefront. The first type, buried seawall, is designed to protect inland areas from erosion and wave damages during severe storm events such as Hurricane Sandy. The second type, composite seawall, is designed to also limit storm surge inundation and cross-island flooding during severe storm events. The composite seawall is compatible with a comprehensive storm surge barrier for Jamaica Bay. A typical section of the buried seawall and composite seawall is shown in Figure 7-16.

The first concept is a buried seawall. Buried seawalls are essentially dunes with a reinforced rubble mound core and were developed as an alternative to larger standalone seawalls. Buried seawalls are designed to function in conjunction with beach restoration projects and dunes. The primary advantage of buried seawalls over traditional dunes is the additional risk reduction provided by the stone core for damage due to erosion and wave attack. Since the purpose of the buried seawall is the reduction of wave energy for the area landward of it, it may be constructed intermittingly along the shoreline in the most vulnerable areas.

The second reinforced dune concept is a composite seawall with an impermeable core (i.e. steel sheet pile). The purpose of the composite seawall is to not only protect against erosion and wave attack but also to limit storm surge inundation and cross-island flooding. The composite seawall provides a high level of risk reduction that may not be practical to achieve with a dune because of the necessary height and footprint of such a dune. In addition, the composite seawall is compatible with a comprehensive storm surge barrier for Jamaica Bay.

Several design concepts were initially considered before selecting a horizontally composite seawall: rubble mound, vertical steel sheet pile wall, vertically composite, and horizontally composite structure. The vertical sheet pile wall and vertically composite wall were eliminated from further consideration because of the large lateral forces acting on the steel sheet pile and the required length and size of steel piles to withstand these forces. The armor stone in horizontally composite structures has been demonstrated to significantly reduce the wave breaking pressure (Takahashi, 2002). This allows smaller steel sheet pile walls to be used in the design if the face of the wall is completely protected by armor stone.

Table 7-16: Recommended Level of Risk Reduction of Reinforced Dunes

Structure Type	Structure Crest Elevation (feet, NAVD88)	Dune Elevation	Design Berm Width (feet)	LORR[1] (years)
Buried Seawall	+16	+18	60	70[2]
Composite Seawall	+17	+18	60	150[2]

Notes: [1] Level of Risk Reduction (LORR)

[2] Provides additional risk reduction against wave attack

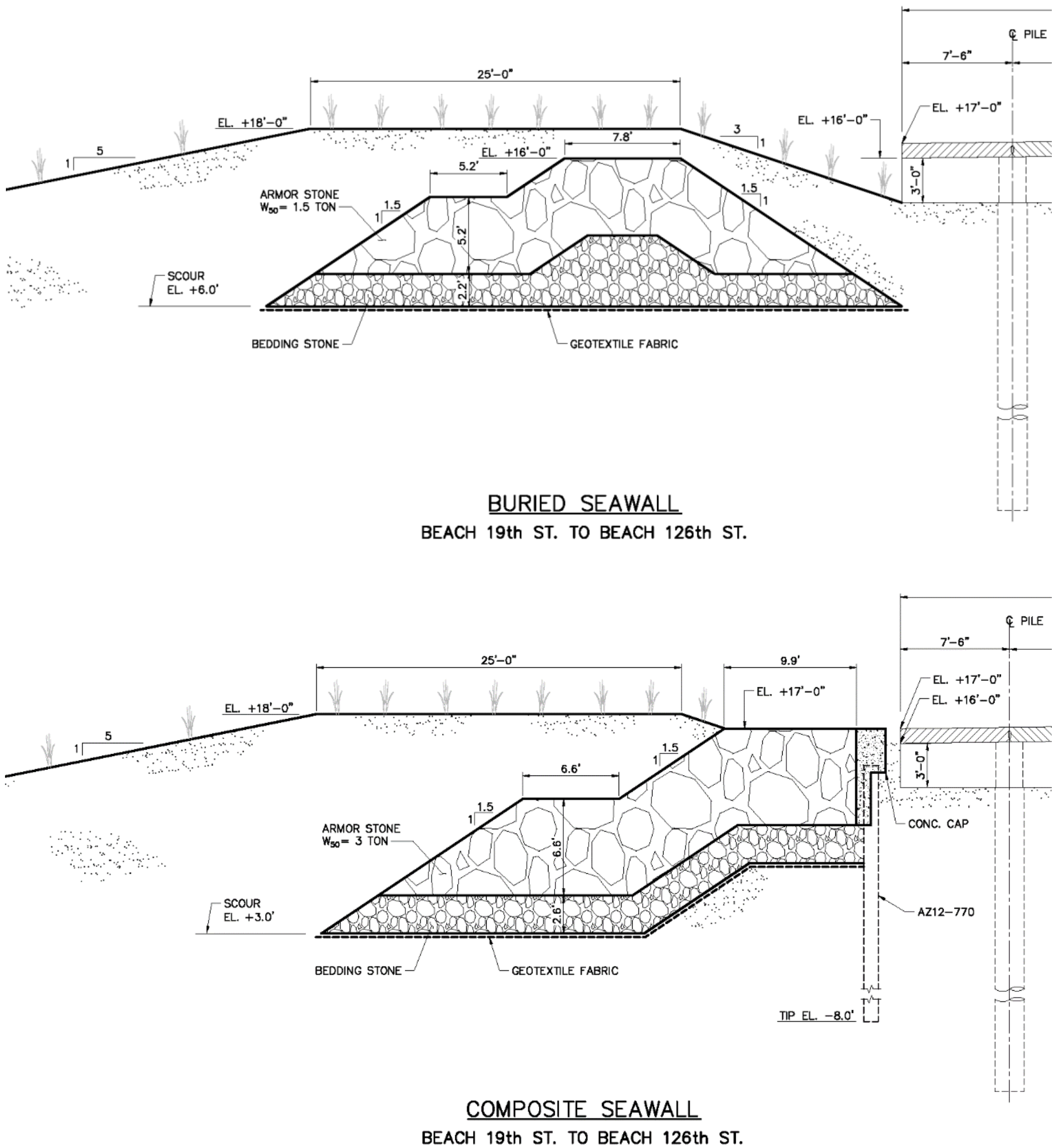


Figure 7-16: Reinforced Dune Alternatives (Buried Seawall in op panel, composite seawall in lower panel)

7.6 Plan Adaptations to SLC

The data compilation and analysis presented above reflect future conditions based on the Low Sea Level Rise scenario, which assumes a continuation of historic sea level changes. The scenario analysis considers two additional accelerated sea level change scenarios as required under USACE guidance (ER 1100-2-8162 and ETL 1100-2-1).

The assessment of sea level rise impacts include a technical analysis of the adaptability of each alternative to accelerate sea level rise under intermediate (Curve 1) and high (Curve 3) scenarios. Annual costs and benefits under these scenarios were recalculated to allow an assessment of whether the plans identified under the low sea level rise scenario remain appropriate and cost effective under the accelerated sea level rise scenario.

7.6.1 Dune and Beach Restoration

It is relatively easy to adapt the dune and beach restoration alternatives to sea level change. Additional sediment can be included in each renourishment operation to offset losses from sea level rise. The natural berm elevation will rise in concert with the rising sea surface, so the design berm should be adjusted accordingly. The dune crest elevation will also need to be raised in response to sea level rise to maintain the design performance. It is recommended that the design berm elevation and dune crest elevation be increased in 1-foot increments in the future to accommodate sea level rise.

The Bruun rule is used to evaluate the relationship between sea level rise and beach erosion. Using the Bruun rule, estimates can be made of the losses due to sea level rise and the volume of sediment required to raise the beach profile to offset losses from sea level rise. The additional renourishment quantities on the Rockaways, based on Bruun rule, are manageable, with increases of approximately 22,000 cy/yr and 93,000 cy/yr for the USACE intermediate and USACE high sea level change scenarios, respectively.

Approximately 550,000 CY would be required each time the dune crest elevation needs to be raised 1 foot. This quantity includes both the volume of sediment within the footprint of the dune as well as the quantity of sediment required to shift the berm and entire active profile 8 ft seaward. An additional cost associated with raising the dune is the impact on existing beach access. It is assumed that the 8 concrete access ramps would be elevated the first time the 18 ft dune and 20 ft dune is raised. It is assumed that up to 2 ft increases in the 16 ft dune height could be accommodated within the existing ramps.

Revised life-cycle cost estimates were prepared for all the alternatives under the USACE intermediate and USACE high sea level rise scenarios. The following adaptability costs were included in the life-cycle cost estimates:

1) Additional beachfill in each renourishment operation (i.e. Bruun Rule).
2) Raising the dune crest elevation in 1 foot increments.
3) Demolishing and rebuilding concrete boardwalk access ramps after 1st dune height change (excluding the +16 foot dune).

The trigger for adaptation measure 2 is when sea level rise exceeds the design, 0.7 ft, which was based on the USACE low scenario. In the USACE intermediate scenario adaptation measures #2 and #3 occur in year 32 (2050). In the USACE high scenario adaptation measures #2 and #3 occur in year 12 (2030) and adaptation measure #2 occurs again in year 32 (2050).

7.6.2 Seawalls

The buried seawall and composite seawall both may be adapted in the future to rising sea levels by adding 1-layer of armor stones as shown in Figure 7-17. The composite seawall would also require extending the concrete cap up to the elevation of the armor stone. Since the size of the median diameter of armor stone is fixed the adapted height of the seawalls may actually increase more than the sea level. It's not feasible to increase the height of the seawalls by exactly 1 or 2 feet by adding smaller armor stone because the smaller stone would not be stable under design storm conditions. Raising only the concrete cap in the composite seawall is also not feasible because the wave forces on the cap and steel pile would increase without the the armor stone in front. Consequently, there is considerably less flexibility in the adaptation of the seawalls in comparison to the dunes.

In addition to adapting the seawalls, the dune height would increase to keep the seawalls buried and renourishment fill quantities would need to increase to offset losses due to sea level rise.

Revised life-cycle cost estimates were prepared for the two seawall alternatives under the USACE intermediate and USACE high sea level rise scenarios. The following adaptability costs were included in the life-cycle cost estimates:

1) Additional beachfill in each renourishment operation (i.e. Bruun Rule).
2) Adding 1-layer of armor stone to the seawalls & increasing dune height.
3) Demolishing and rebuilding concrete boardwalk access ramps after increasing seawall height.

The trigger for adaptation measure 2 is when sea level rise exceeds the design, 0.7 ft, which was based on the USACE low scenario. In the USACE intermediate scenario adaptation measures #2 and #3 occur in year 32 (2050). In the USACE high scenario adaptation measures #2 and #3 occur in year 12 (2030).

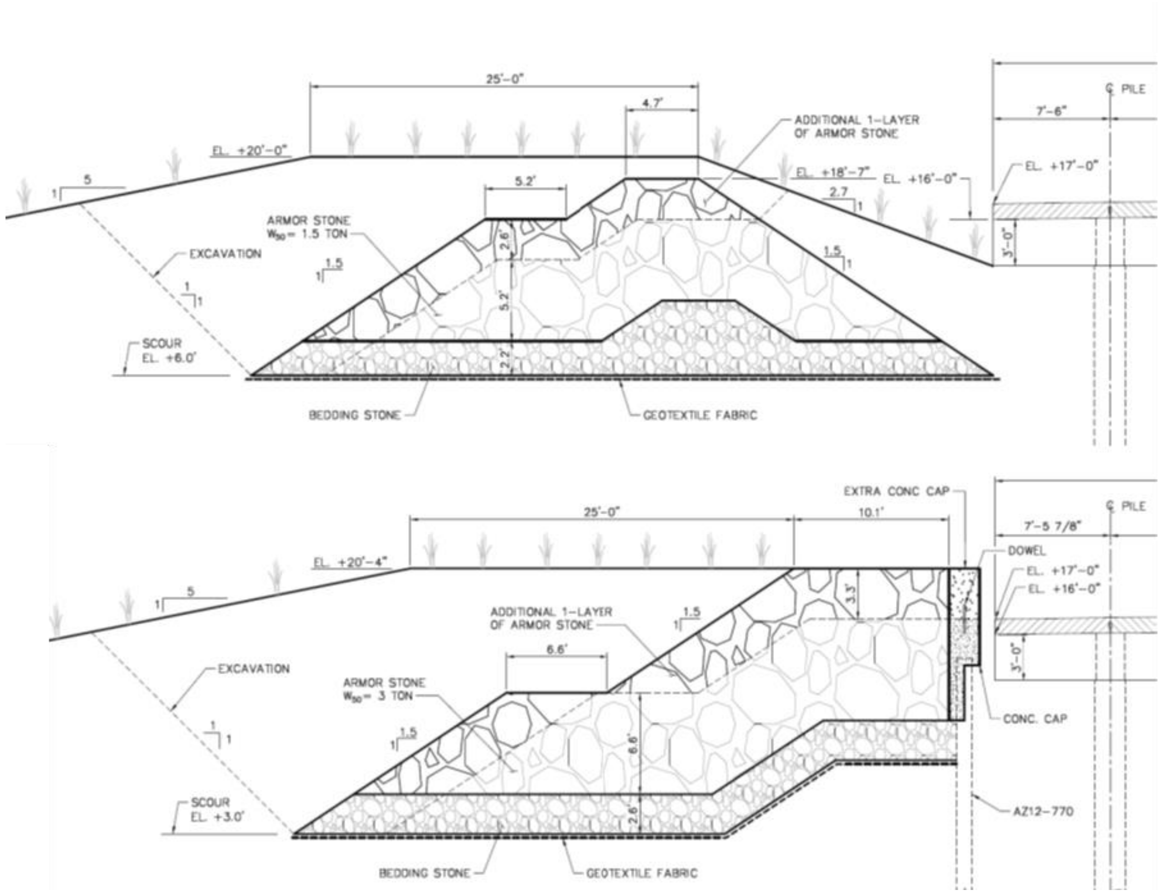


Figure 7-17: Seawall Adaptability Measures

7.6.3 Groins

Due to the uncertainty in sea level change as well as the design/performance of the groin system it is recommended that the groins be adapted in the future by adjusting renourishment quantities and placement locations. Even without considering sea level changes there will be some differences in the actual performance of the groins and the expected or modeled performance that will need to be adapted to by adjusting fill placement.

The relative difference in sea level rise between the USACE low and USACE high sea level rise scenarios is about 2 feet. Typically groins are designed with a crest elevation between MHHW and MLLW. Considering that the existing tidal range in the project area is 5 feet, the impact of sea level rise on sediment bypassing over the groins is anticipated to be relatively minor since even the high sea level rise changes fall well within the tidal range.

Groin performance could also be impacted as the design beach profile is adjusted in the future in response to sea level change. As discussed above, the dune height and berm elevation should be raised in response to sea level rise. A 2 ft increase in the dune and berm elevation results in a 46 ft seaward projection in the design profile, which would decrease the effective length of the groins by 46 ft. As the effective groin length decreases, bypassing around the tips of the groins will increase.

Both of the groin performance impacts above can be managed by adjusting renourishment fill quantities and placement locations. As bypassing over and around the groins increases, more fill will be required within the groin field and less fill will be required downdrift of the groins. Adapting the groins structurally by increasing the groin elevation or length could also be considered, but is likely to be expensive and not justified based on the small improvements in groin performance. Structural adaptation of the groins could entail either adding an additional layer of armor stone or extending the groin seaward.

7.6.4 CSRM Alternatives Comparison

The cost and benefits for each of the alternatives were evaluated. The results of the comparison, presented in Table 7-17, indicate that all of the alternative plans are cost effective and that the highest net benefits are provided by the composite seawall. Among the beach restoration and dune alternatives, the highest net benefits are provided by the largest alternative considered.

The benefit estimates include reduced damages for the shorefront area, reduced damages from cross island flooding, and reduced future maintenance costs. For the shorefront areas the Beach-fx models were revised to incorporate each design profile and to adjust future profiles to reflect the planned renourishment to maintain the design profile into the future. The reduced damage due to cross shore flooding was estimated by using the HEC-FDA levee function to truncate/eliminate damages for storm events that would not generate significant overtopping volumes (1.0 cfs). Because the project will maintain the design profile there will be no need for non-federal actions to repair the design profile after major storm events. These future costs avoided are estimated to add $577,000 in average annual benefits to each plan.

The initial construction costs for each of the CSRM alternatives includes all of the required project features including erosion control measures, beach, dune and seawall features for each plan, any modifications to existing structures, such as boardwalk access ramps, and associated costs for engineering design and construction management. For economic comparison purposes, interest during construction has been added to the initial construction costs to reflect different investment opportunity costs between alternatives.

Average annual costs include the amortized value of the initial construction (50 years, 2.750% interest rate), annualized value of periodic renourishment, Operations and Maintenance (O&M), average annual costs for structure repair after major storm events, and any costs for adapting the structure for accelerated sea level rise (assumed zero under the low sea level rise or base condition). Screening level cost estimate and supporting data are included within Sub-Appendix B.

The analysis of sea level rise included the average annual costs of future plan adaptations and the change in with and without project damage and benefits associated with higher water levels and higher rates of shoreline change. Shorefront benefits under these scenarios were recalculated in Beach-fx. Back Bay inundation damages were estimated to increase in response to higher flood levels in Jamaica Bay. Because of the higher flood levels in Jamaica bay, the area subject to cross shore flooding becomes smaller in the accelerated sea level rise scenarios. As a result the damages and benefits associated with cross shore flooding become smaller as sea level rise increases

As seen in Table 7-18 and Table 7-19, all of the plans considered remain cost effective. Because of the high cost of modifying the structural alternatives and the reduction in cross shore flood benefits the seawall plans become relatively less cost effective. Under the intermediate sea level rise scenario the composite seawall plan continues to provide the overall highest net benefits while the highest net benefits of the dune and beach restoration plans is provided by the 20 foot dune alternative. Under the high sea level rise scenario the composite seawall plan continues to provide the overall highest net benefits while the 20 foot dune alternative provides slightly higher net benefits than the composite seawall alternative.

Table 7-17: Low SLR CSRM Benefits

Atlantic Ocean Shorefront Formulation Summary		Low SLR					
		Without Project	16 Foot Dune	18 Foot Dune	20 Foot Dune	Buried Seawall	Composite Seawall
Initial Cost	Initial Construction	$0	$71,017,000	$95,497,000	$147,199,000	$155,483,000	$220,988,000
	IDC	$0	$1,307,000	$2,129,000	$3,462,000	$3,752,000	$6,760,000
	Investment Cost	$0	$72,324,000	$97,626,000	$150,661,000	$159,235,000	$227,748,000
Annualized Cost	Investment Cost	$0	$2,679,000	$3,616,000	$5,581,000	$5,898,000	$8,436,000
	Renourishment (Planned/Emergency)	$867,000	$5,950,000	$6,392,000	$6,829,000	$5,950,000	$5,950,000
	O&M	$0	$579,000	$598,000	$621,000	$727,000	$836,000
	Major Rehab	$0	$332,000	$332,000	$332,000	$332,000	$332,000
	SLR Adaption	$0	$0	$0	$0	$0	$0
	Total Annual Cost	$867,000	$9,540,000	$10,938,000	$13,363,000	$12,907,000	$15,554,000
Damages	Damages - Shore Front	$17,502,000	$8,389,000	$5,180,000	$2,752,000	$5,097,000	$1,986,000
	Damages - Cross Shore Flood Damages	$28,757,000	$26,393,000	$19,350,000	$15,413,000	$19,350,000	$11,360,000
	Back Bay Damages	$65,548,000	$65,548,000	$65,548,000	$65,548,000	$65,548,000	$65,548,000
	Total Damages	$111,807,000	$100,330,000	$90,078,000	$83,713,000	$89,995,000	$78,894,000
Benefits	Total Benefits (Reduced Damages)	-	$9,113,000	$12,322,000	$14,750,000	$12,405,000	$15,516,000
	Cost Avoided (Emergency Nourishment)	-	$867,000	$867,000	$867,000	$867,000	$867,000
	Shorefront Benefit (Reduced Damage Plus Cost Avoided)	-	$9,980,000	$13,189,000	$15,617,000	$13,272,000	$16,383,000
	Cross Shore Flood Damage Reduced	-	$2,364,000	$9,407,000	$13,344,000	$9,407,000	$17,397,000
	Total Storm Damage Reduction Benefits	-	$12,344,000	$22,596,000	$28,961,000	$22,679,000	$33,780,000
	Recreation Benefits	-	$29,430,000	$29,430,000	$29,430,000	$29,430,000	$29,430,000
	Total Benefits	-	$41,774,000	$52,026,000	$58,391,000	$52,109,000	$63,210,000
	Net Benefits (Damage Reduction Only)	-	$2,804,000	$11,658,000	$15,598,000	$9,772,000	$18,226,000
BCR		**-**	**4.4**	**4.8**	**4.4**	**4.0**	**4.1**

Table 7-18: Intermediate SLR CSRM Benefits

Atlantic Ocean Shorefront Formulation Summary		Intermediate SLR					
		Without Project	16 Foot Dune	18 Foot Dune	20 Foot Dune	Buried Seawall	Composite Seawall
Initial Cost	Initial Construction	$0	$ 71,017,000	$ 95,497,000	$147,199,000	$155,483,000	$220,988,000
	IDC	$0	$ 1,307,000	$ 2,129,000	$ 3,462,000	$ 3,752,000	$ 6,760,000
	Investment Cost	$0	$ 72,324,000	$ 97,626,000	$150,661,000	$159,235,000	$227,748,000
Annualized Cost	Investment Cost	$0	$2,679,000	$3,616,000	$5,581,000	$5,898,000	$8,436,000
	Renourishment (Planned/Emergency)	$943,000	$6,364,000	$6,801,000	$7,243,000	$6,364,000	$6,364,000
	O&M	$0	$579,000	$598,000	$621,000	$728,000	$836,000
	Major Rehab	$0	$332,000	$332,000	$332,000	$332,000	$332,000
	SLR Adaption	$0	$210,000	$373,000	$377,000	$1,020,000	$1,453,000
	Total Annual Cost	$943,000	$10,164,000	$11,720,000	$14,154,000	$14,342,000	$17,421,000
Damages	Damages - Shore Front	$18,512,000	$7,750,000	$4,882,000	$2,641,000	$4,783,000	$2,245,000
	Damages - Cross Shore Flood Damages	$27,384,000	$25,191,000	$18,515,000	$14,794,000	$18,515,000	$10,947,000
	Back Bay Damages	$70,505,000	$70,505,000	$70,505,000	$70,505,000	$70,505,000	$70,505,000
	Total Damages	$116,401,000	$103,446,000	$93,902,000	$87,940,000	$93,803,000	$83,697,000
Benefits	Total Benefits (Reduced Damages)	-	$8,172,000	$11,040,000	$13,281,000	$11,139,000	$13,677,000
	Cost Avoided (Emergency Nourishment)	-	$626,368	$626,368	$626,368	$626,368	$626,368
	Shorefront Benefit (Reduced Damage Plus Cost Avoided)	-	$8,798,368	$11,666,368	$13,907,368	$11,765,368	$14,303,368
	Cross Shore Flood Damage Reduced	-	$2,193,000	$8,869,000	$12,590,000	$8,869,000	$16,437,000
	Total Storm Damage Reduction Benefits	-	$10,991,368	$20,535,368	$26,497,368	$20,634,368	$30,740,368
	Recreation Benefits	-	$35,292,000	$35,292,000	$35,292,000	$35,292,000	$35,292,000
	Total Benefits	-	$46,283,368	$55,827,368	$61,789,368	$55,926,368	$66,032,368
	Net Benefits (Damage Reduction Only)	$0	$827,368	$8,815,368	$12,343,368	$6,292,368	$13,319,368
BCR		**-**	**4.2**	**4.5**	**4.1**	**3.7**	**3.6**

Table 7-19: High SLR CSRM Benefits

Atlantic Ocean Shorefront Formulation Summary		High SLR					
		Without Project	16 Foot Dune	18 Foot Dune	20 Foot Dune	Buried Seawall	Composite Seawall
Initial Cost	Initial Construction	$0	$71,017,000	$95,497,000	$147,199,000	$155,483,000	$220,988,000
	IDC	$0	$1,307,000	$2,129,000	$3,462,000	$3,752,000	$6,760,000
	Investment Cost	$0	$72,324,000	$97,626,000	$150,661,000	$159,235,000	$227,748,000
Annualized Cost	Investment Cost	$0	$2,679,000	$3,616,000	$5,581,000	$5,898,000	$8,436,000
	Renourishment (Planned/Emergency)	$1,299,000	$7,666,000	$8,108,000	$8,544,000	$7,666,000	$7,666,000
	O&M	$0	$579,000	$598,000	$621,000	$554,000	$417,000
	Major Rehab	$0	$332,000	$332,000	$332,000	$332,000	$332,000
	SLR Adaption	$0	$564,000	$849,000	$859,000	$2,197,000	$2,288,000
	Total Annual Cost	$1,266,000	$11,820,000	$13,503,000	$15,937,000	$16,647,000	$19,139,000
Damages	Damages - Shore Front	$18,302,000	$9,559,000	$6,321,000	$3,728,000	$6,114,000	$3,330,000
	Damages - Cross Shore Flood Damages	$22,511,000	$21,191,000	$15,865,000	$12,924,000	$15,865,000	$9,663,000
	Back Bay Damages	$90,505,000	$90,505,000	$90,505,000	$90,505,000	$90,505,000	$90,505,000
	Total Damages	$131,318,000	$121,255,000	$112,691,000	$107,157,000	$112,484,000	$103,498,000
Benefits	Total Benefits (Reduced Damages)	-	$8,743,000	$11,981,000	$14,574,000	$12,188,000	$14,972,000
	Cost Avoided (Emergency Nourishment)	-	$1,266,000	$1,266,000	$1,266,000	$1,266,000	$1,266,000
	Shorefront Benefit (Reduced Damage Plus Cost Avoided)	-	$10,009,000	$13,247,000	$15,840,000	$13,454,000	$16,238,000
	Cross Shore Flood Damage Reduced	-	$1,320,000	$6,646,000	$9,587,000	$6,646,000	$12,848,000
	Total Storm Damage Reduction Benefits	-	$11,329,000	$19,893,000	$25,427,000	$20,100,000	$29,086,000
	Recreation Benefits	-	$29,430,000	$29,430,000	$29,430,000	$29,430,000	$29,430,000
	Total Benefits	-	$40,759,000	$49,323,000	$54,857,000	$49,530,000	$58,516,000
	Net Benefits (Damage Reduction Only)	-	-$491,000	$6,390,000	$9,490,000	$3,453,000	$9,947,000
BCR		**-**	**3.4**	**3.7**	**3.4**	**3.0**	**3.1**

8 THE RECOMMENDED PLAN

8.1 The Recommended Plan for the Atlantic Ocean Shorefront

The Atlantic Ocean shorefront reach is subject to wave attack, wave run up, and over topping along the Rockaway peninsula. The general approach to developing CSRM along the Atlantic Ocean Shorefront (Reach 3 through Reach 6b of the Atlantic Ocean Shorefront) was to evaluate erosion control alternatives in combination with a single beach restoration plan to select the most cost effective renourishment approach prior to the evaluation of alternatives for coastal storm risk management. The most cost effective erosion control alternative is beach restoration with increased erosion control. This erosion control alternative had the lowest annualized costs over the 50-year project life and the lowest renourishment costs over the project life.

A screening analysis was performed to evaluate the level of risk reduction provided by a range of dune and berm dimensions and by reinforced dunes, which would be combined with beach restoration with increased erosion control to optimize CSRM at the Atlantic Ocean Shorefront. Other factors such as prior projects at Rockaway Beach, project constraints, stakeholder concerns, and engineering judgment were also applied in the evaluation and selection. A composite seawall was selected as the best coastal storm risk management alternative. The composite seawall protects against erosion and wave attack and also limits storm surge inundation and cross-peninsula flooding. The Recommended Plan spans from Reach 3 through Reach 6b and combines Beach restoration and Erosion Control and has the following features.

The Recommended Plan for the Atlantic Ocean Shorefront reach (Figure 8-1) consists of:

- A composite seawall with a structure crest elevation of +17 feet (NAVD88), the dune elevation is +18 feet (NAVD88), and the design berm width is 60 feet;
- A beach berm elevation of +8 ft NAVD88 and a depth of closure of -25 ft NAVD88;
- A total beach fill quantity of 1,596,000 cy for the initial placement, including tolerance, overfill and advanced nourishment with a 4-year renourishment cycle of 1,021,00 cy, resulting in a minimum berm width of 60 feet;
- Extension of 5 existing groins; and
- Construction of 13 new groins.
- Construction of two beachfill taper sections at both the project east and west ends, with 3 new groins as part of the west taper.

The following sections provide additional details on the reinforced dune with composite seawall, the shorefront beach restoration and the groins, respectively. A set of preliminary plans and sections for the Recommended Plan for the Atlantic Ocean Shorefront reach is provided in Sub-Appendix C.

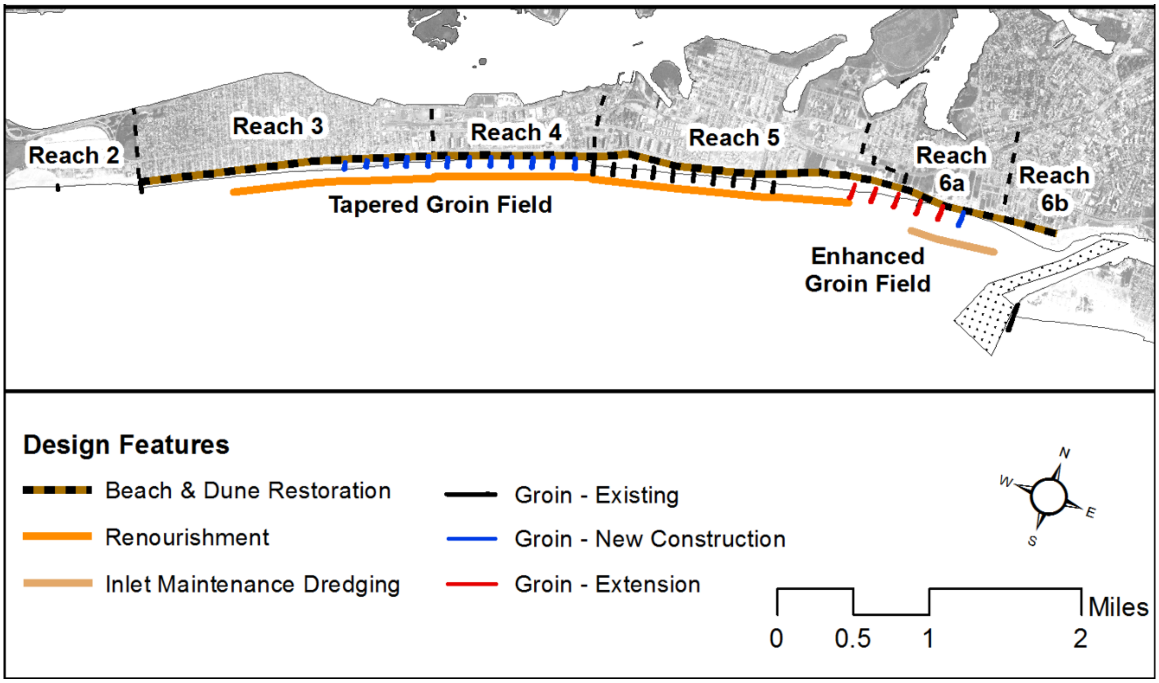


Figure 8-1: Schematic overview of the Beach Restoration and Groin features of the Recommended Plan for the Atlantic Ocean Shorefront Reach

8.1.1 Reinforced Dune – Composite Seawall

A composite Seawall is proposed for the Atlantic Ocean Shorefront from Beach 149th Street up to Beach 20th Street. The composite seawall alignment follows the existing boardwalk alignment. The Composite seawall concept consists of an impermeable core (i.e. sheet pile wall with concrete cap) and rubble mound structure on the seaward side of the wall. The composite seawall is covered with sand and only the top and concrete cap are exposed on the land side of the dune (see Figure 8-2). The structure crest elevation is +17 feet (NAVD88), the dune elevation is +18 feet (NAVD88), and the design berm width is 60 feet. The armor stone in the horizontally composite structure significantly reduces wave breaking pressure, which allows smaller steel sheet pile walls to be used in the design if the face of the wall is completely protected by armor stone. The composite seawall may be adapted in the future to rising sea levels by adding 1-layer of armor stone and extending the concrete cap up to the elevation of the armor stone. Due to spatial constraints within Reach 3 between Beach 149th Street and Beach 126th Street, a modified version of the composite seawall that includes a splash apron on the leeward side of the sheet pile wall is proposed for this section (Figure 8-3). Detailed plans and sections are provided in Sub-Appendix C.

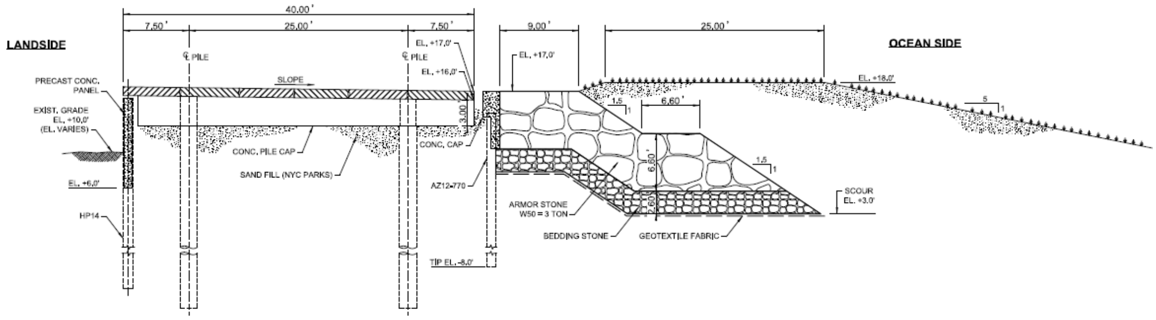


Figure 8-2: Composite Seawall

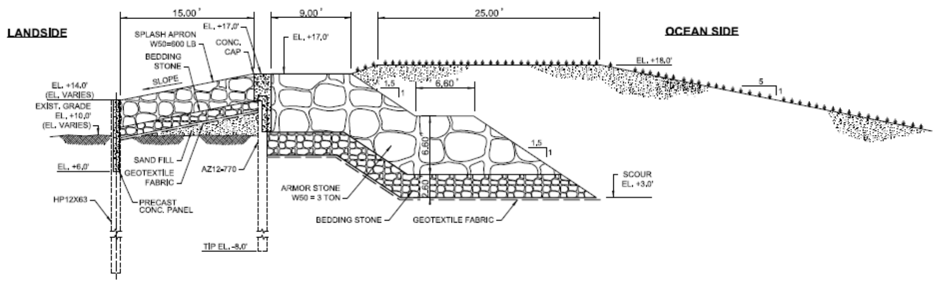


Figure 8-3: Composite Seawall (Beach 126th Street to Beach 149th Street)

8.1.2 Beach Restoration

The Recommended Plan includes all the features of Alternative 3 as earlier presented in Chapter 7 and consists of beach restoration for Reach 3 through 6. A design profile is proposed that includes a dune with a 25 ft wide with a crest at elevation +18 ft NAVD88 and a back slope of 1V:3H and a front slope of 1V:5H. The design includes a berm with a minimum width of 60 ft at an elevation of +8 ft NAVD88. The width of the design berm is controlled by the alignment of the baseline. The baseline is aligned with the natural shoreline and the distance from the baseline to the design shoreline is always 243 ft. The alignment of the dune follows the unnatural alignment of the boardwalk and as a result the distance between the toe of the dune and the seaward crest of the berm varies. Detailed plans and sections are provided in Sub-Appendix C.

For the transitions at the project ends two smaller beachfill taper sections are included within the Recommended Plan. The east beachfill taper is approximately 3,000 ft in shorefront length from Beach 19th Street east to Beach 9th Street. The taper comprises of approximately 1,000 ft of dune and beach taper including reinforced dune feature and approximately 2,000 ft of dune and beach fill without reinforced dune feature. In addition to the tapering of berm width, the dune elevation also tapers from an elevation of +18 ft NAVD88 at 19th Street down to approximately +12 ft NAVD88 at Beach 9th Street which will be tied into the existing grade. The west beachfill taper is approximately 5,000 ft in shorefront length from Beach 149th Street west to Beach 169th street fronting Riis Park. The beachfill taper will be beach fill only with a berm width tapered from the design width at 149th Street to the existing width and height at 169th Street.

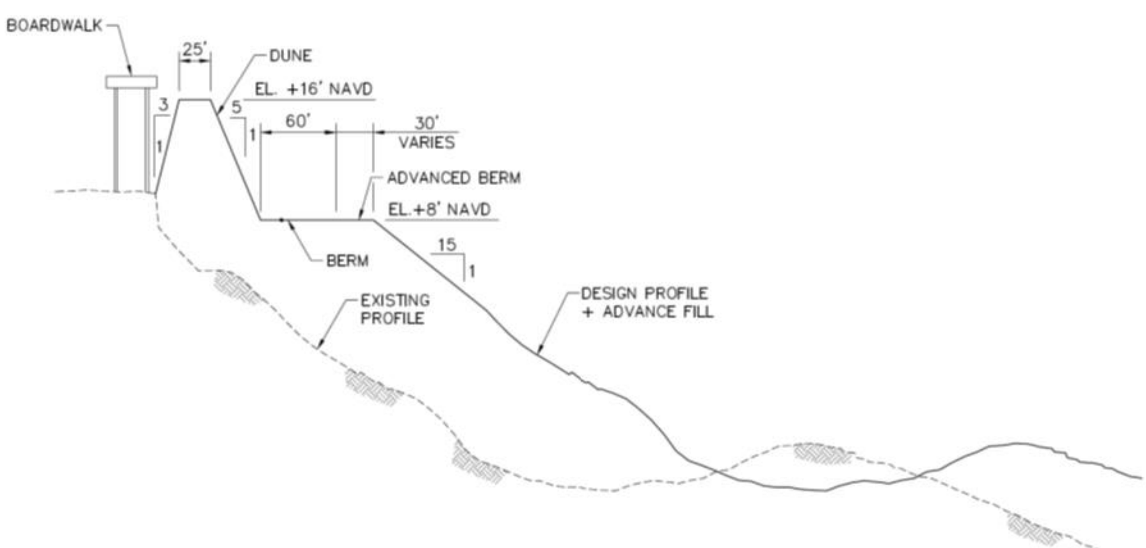


Figure 8-4: Design Beach Profile (note: Baseline location varies)

The initial beach fill construction quantities per reach are provided in Table 8-1.

Table 8-1: Initial Construction Beachfill Quantities (CY)

Reach	Length (ft)	Recommended Plan
Reach 2 (West Taper)		306,000
Reach 3	10,320	356,000
Reach 4	5,380	294,000
Reach 5	10,650	321,000
Reach 6a	3,730	250,000
Reach 6b	2,000	20,000
East Taper		49,000
Total	55,650	1,596,000

A summary of the renourishment quantities, per operation, including overfill and losses is provided in Table 8-2. The renourishment quantities for the Recommended Plan are based on a 4-year cycle. Inlet Maintenance Dredging is assumed to continue and dredged materials to be paced in Reach 6a.

Table 8-2: Renourishment Quantities per Operation[1]

Reach	Renourishment (cy) based on a 4-year renourishment cycle
	Tentatively Selected Plan
Reach 3	444,000
Reach 4	133,000
Reach 5	444,000
Reach 6a	0[2]
Total	1,021,000

Notes [1] All renourishment quantities account for SLR and overfill (11%)

[2] Excludes Inlet Maintenance Dredging (115,000 cy/yr)

8.1.3 Groins

New Groins are proposed for the Atlantic Ocean Shorefront Recommended Plan and include two types of groin measures: new groin construction and groin extension. Existing groins are extended in Reach 5 and 6 and 1 new groin is constructed in Reach 6. In reach 4 seven (7) new groins are constructed and in Reach 3 five (5) new groins are constructed (Figure 8-1). In addition to the beachfill taper in Reach 2, a tapered groin system comprised of three (3) rock groins is also included for this section. Table 8-3 provides and overview of the groin length, type and location. Detailed plans and sections are provided in Sub-Appendix C.

Table 8-3: Summary of Groin Lengths for the Recommended Plan

Reach	Number	Groin ID	Street	Horizontal Shore Section (ft)	Intermediate Sloping Section (ft)	Outer Section (ft)	Total Length (ft)	Notes
6a	1	63	34th St	62	108	328	498	new 498 ft
6a	2	62	37th St	55	108	328	491	extension 209'
6a	3	61	40th St	90	108	328	526	extension 307 ft
5	4	53	43rd St	90	108	228	426	extension 114 ft
5	5	52	46th St	90	108	228	426	extension 155 ft
5	6	51	49th St	90	108	228	426	extension 180 ft
4	1	47	92nd St	66	108	128	302	new 302 ft
4	2	46	95th St	62	108	128	298	new 298 ft
4	3	45	98th St	63	108	128	299	new 299 ft

Reach	Number	Groin ID	Street	Horizontal Shore Section (ft)	Intermediate Sloping Section (ft)	Outer Section (ft)	Total Length (ft)	Notes
4	4	44	101st St	62	108	128	298	new 298 ft
4	5	43	104th St	66	108	128	302	new 302 ft
4	6	42	106th St	67	108	128	303	new 303 ft
4	7	41	108th St	66	108	128	302	new 302 ft
3	8	35	110th St	90	108	153	351	new 351 ft
3	9	34	113th St	90	108	178	376	new 376 ft
3	10	33	115th St	90	108	178	376	new 376 ft
3	11	32	118th St	90	108	178	376	new 376 ft
3	12	31	121st St	63	108	128	299	new 299 ft
2	1	23	Reach 2	161	108	100	369	new 369 ft
2	2	22	Reach 2	205	108	100	413	new 413 ft
2	3	21	Reach 2	223	108	100	431	new 431 ft

8.2 Quantities & Costs

Cost estimates for the Recommended Plan were developed at a second quarter 2018 price level for labor, material, and equipment. The material quantities for the Recommended Plan have been developed from the plans shown in Sub-Appendix C (sub-Appendix to this Appendix A1) and full details on the development of the cost estimate are provided in Appendix C. The MII Estimate is included in Appendix C and includes the details of the estimate including the different tasks required to complete the construction. Details provided for these tasks include the production rate of the crews and the crew composition, including the equipment used and the number and description of labor categories required. It should be noted that the cost estimate was updated from the values used for the alternative analysis. All labor is assumed to be from prevailing wage rates for New York City and equipment rates were estimated for USACE Region 1, with supplemental information from published Blue Book Rates for equipment. The total project cost, fully funded, for the Atlantic Ocean Shorefront is $232.0 million, without contingency. The cost risk analysis determined the contingency to be 24.91%, making the total project cost $289,8 million for budgeting purposes (2018 price levels). Obtaining the armor stone is difficult for quarries to supply and the project requires over 350,000 tons of armor stone. The armor stone represented over 89% of the project's expected variability, with no other item representing over 10% of the project's total cost variation. The Civil Work Breakdown Structure (CWBS) feature codes as shown in Table

8-4 are utilized to establish the project cost. The project cost presented in Table 8-4 are a summary of the detail in Appendix C.

Table 8-4: MII Estimate Recommended Plan – Atlantic Ocean Shorefront Reach

CWBS account code #	Account Code Description	Total Project Cost (Fully Funded)
01	Lands and Damages	$65,000
10	Breakwaters & Seawalls	$218,930,000
17	Beach Replenishment	$35,232,000
30	Planning, Engineering and Design	$19,660,000
31	Construction Management	$15,906,000
	Total	**$289,793,000**

8.3 Construction Schedule

The construction schedule for the Atlantic Ocean Shorefront Reach is included in Appendix C. The total duration is approximately 43 months. The first construction activity will be the beachfill starting in December 2019 with a duration of approximately 6.5 months. Starting Spring of 2020 the groins and the composite seawall and dune will be constructed in parallel with project completion mid-2023.

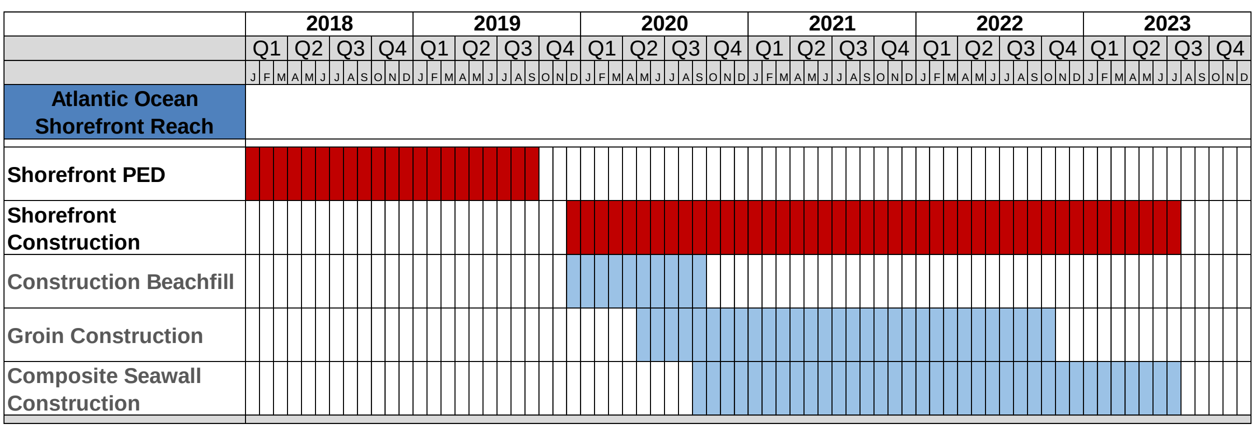


Figure 8-5: Schematic overview of the construction schedule for the Atlantic Ocean Shorefront Reach of the Recommended Plan

8.4 Recommendations for PED

This Engineering & Design Appendix provides an overview of the analyses supporting the development of the recommended plan for the Atlantic Ocean Shorefront Reach. This appendix furthermore describes the development of erosion control and CSRM alternatives and a screening analysis of these alternatives to establish the Recommended Plan. Based on the data gathered during the feasibility study and engineering analyses, a preliminary design for the project has been completed. The dimensions and sizing of the specific project features such as the composite seawall, the groins and splash apron are preliminary but of sufficient detail for feasibility study.

It is expected that the project feature's designs and the composite seawall design would be further developed in PED. It can be noted that based upon the review of the proposed project during the feasibility phase, DEC will require further justification or component revisions to ensure the protection of water quality, habitat quality, and public access during the PED Phase. In addition, initial beach fill quantities will need to be updated based on current survey data. The preliminary designs shall not be construed as requirements for actual dimensions for implementation and significant additional engineering and design is required to substantiate the designs including, but not limited to, a full evaluation of topographical and bathymetric elevations, subsurface soil conditions and an inventory and condition assessment of existing structures and utilities for the project area. A preliminary list of potential future engineering analyses and design refinements include the following:

- Bathymetric and topographic survey to establish beachfill quantities and inform detailed designs for the groins and buried seawall,
- Geotechnical data collection (site specific borings, samples and geotechnical data collection),
- Utility survey and an inventory and condition assessment of existing structures and utilities for the project area,
- Morphological modeling analysis to assess the performance of the tapered groin field in Reach 2 and beyond the terminal groin at 149th street and incorporate recent survey data on the performance of the FCCE project post Hurricane Sandy,
- Borrow area analysis, including, but not limited to surveys, grain size compatibility and site capacity analyses,
- Refined engineering analyses for the armor rock sizing (both for the coastal groins and the composite seawall) and geometry of the coastal groin structures,
- Refined engineering analyses for the beach tapers, dune transitions and tie-ins to higher ground at the project ends,
- Engineering analyses and detailed design to integrate the existing baffle wall into the design of the composite seawall between Beach 126th and Beach 49th Street,
- Detailed design to account for the existing boardwalk, beach access (stairs and ramps) and modifications needed during construction of the composite seawall,
- Design of transition of the composite seawall at Beach 126th Street as well as at the project ends, and

- Analyses of temporary construction features and construction sequencing.

REFERENCES

Bedient, P.B., and Huber, W.C. (1992). "Hydrology and Floodplain Analysis (2nd Edition)." Addison-Wesley Publishing Company: Reading, Massachusetts.

Chow, V.T. (1959). "Open-Channel Hydraulics." McGraw-Hill: New York.

EurOtop. (2007). "Wave Overtopping of Sea Defences and Related Structures: Assessment Manual." Eds. T. Pullen., N.W.H Allsop, T. Bruce., A. Kortenhaus, H. Schuttrumpf, and J.W. van der Meer.

Federal Emergency Management Agency (FEMA). (2007). "Guidelines and Specifications for Flood Hazard Mapping Partners." Section D.2.7 Overland Wave Propagation.

Goda, Y. (2000). "Random Seas and Design of Maritime Structures." World Scientific: New Jersey.

M&N. (2001). "Selection of Alternative Design Beach Profiles." Draft Memorandum. Fire Island Inlet to Montauk Point (FIMP) Study.

NOAA Technical Report NOS CO-OPS 065 'Estimating Vertical Land Motion form Long Term Tide Guage Rec-ords', May 2013.

Smith, J. M., Sherlock, A. R., Resio, D. T., 2001. "STWAVE: Steady-State Spectral Wave Model User's Manual for STWAVE, Version 3.0". ERDC/CHL SR-01-1. Vicksburg, MS: U.S. Army Engineer Research and Development Center.

Smith, J. M. 2007. "Full-plane STWAVE: II. Model overview". ERDC TN-SWWRP-07-05. Vicksburg, MS: U.S. Army Engineer Research and Development Center.

Wamsley, V., and Kraus, N.C. (2004). "SBEACH dune lowering sensitivity analysis for the Fire Island to Montauk Point, New York, study." USACE ERDC/CHL TR-05.

U.S. Army Corps of Engineers (USACE).(2004). "Coastal overwash Part1: Overview of processes." USACE ERDC/RSM-TN-14

U.S. Army Corps of Engineers, New York District (USACE-NAN), (2005). "Baseline Conditions Storm Surge Modeling and Stage Frequency Generation: Fire Island to Montauk Point Reformulation Study", U.S. Army Corps of Engineers, New York District.

U.S. Geological Survey (USGS) by Arcement, G.J. and Schenider, V.R. (1984). "Guide for Selecting Manning's Roughness Coefficients for Natural Channels ad Flood Plains." U.S. Geological Survey Water-supply Paper 2339.