US Army Corps
of Engineers®
New York District

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 A2

Jamaica Bay High Frequency Flood Risk Reduction Features
Engineering and Design Appendix

December 2018

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Jamaica Bay High Frequency Flood Risk Reduction Features (HFFRRF)

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
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
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

1 PROJECT DESCRIPTION

1.1 Overview of Engineering and 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 engineering-wise feasible, complete and economically justified.

The USACE transition to SMART Planning is another reason why sub-appendices are included within the study document. The initial study was initially limited to the Atlantic Ocean Shoreline 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 development of the High Frequency Flood Risk Reduction Features (HFFRRF) for Jamaica Bay. This appendix furthermore describes the development of HFFRRF Projects and a screening analysis of these projects to establish a viable plan to mitigate high frequency flood risk for Jamaica Bay.

1.2 Project Background

As a result of the Agency Decision Milestone (ADM), the storm surge barrier for the Jamaica Bay component of the previous Tentatively Selected Plan (see Draft General Reevaluation Report/Environmental Impact Statement (GRR/EIS) USACE-NAN (2016)), was moved into the New York - New Jersey Harbor and Tributaries Coastal Storm Risk Management Feasibility Study for further study and possible recommendation. Without the barrier, the communities surrounding Jamaica Bay still experience substantial risk of coastal flooding. Therefore, the study team sought to identify stand-alone features that could complement a potential future storm surge barrier, but also be economically justified on their own. Residents in many parts of the Jamaica Bay vicinity experience flooding due to high storm tides that occur relatively frequently. Since the proposed storm surge barrier would not be closed at every high tide, there is an opportunity to recommend features to mitigate flood risk in high frequency tidal flooding events in which the proposed storm surge barrier would remain open.

1.3 Report Organization

In accordance with USACE's SMART Planning principles, the HFFRRF projects were developed at a level of detail required to make the decision at hand. Due to the large geographical area, complexities associated with civil work designs in highly developed urban areas, and the potential interaction between the design of perimeter flood risk reduction alignments and interior drainage infrastructure, a two-phased approach was developed. A two-phased approach facilitates a systematic, yet efficient methodology to screen HFFRRF project alternatives. Phase 1 included project definitions, HFFRR-Feature designs and establishing project alignments; a detailed drainage analysis and an assessment of the real estate cost were deferred to a later phase. This approach allowed the Project Delivery Team (PDT) to establish project construction cost and project benefits for a large number of projects and complete a first round of screening based on Benefit Cost Ratios (BCRs). In Phase 2, the Phase 1 projects were further refined, and a detailed drainage analysis and an assessment of the real estate cost was included. As such, a two-tier screening approach was established where the time consuming and resource intensive interior drainage analysis was completed only for the viable projects that passed to Phase 2.

This appendix includes a description of the risk assessment (Chapter 2) and a basis of design (Chapter 3) for the development of the flood risk reduction features. All features used within this study are described in Chapter 4. The development of the projects and first phase of screening are summarized Chapter 5, while the second and final screening phases are summarized in Chapter 6. Complete overviews of all "Phase 1" and "Phase 2" HFFRRF projects are included in Sub-Appendix A and B respectively. Chapter 7 summarizes the HFFRRF Projects for Jamaica Bay that are part of the Recommended Plan.

2 RISK ASSESSMENT

2.1 Introduction

The starting point for the development of the *high-frequency flood risk reduction features* (HFFRRFs) for Jamaica Bay was to identify areas most at risk of coastal flooding due to storm events and projected sea level change. This involved the creation of maps that showed potential flooded areas corresponding to different storm surge magnitudes and future levels of risk in the Project Study Area, which includes the entire main land perimeter of Jamaica Bay, west of Alternative 1-E barrier alignment. The Study Area includes parts of Kings, Queens, and Nassau counties in Long Island, NY. The features developed for the project areas identified as high risk are designed to reduce flood risk for these low lying coastal areas during relatively high frequency flooding events when the proposed storm surge barrier at the Rockaway Inlet entrance to Jamaica Bay might not be utilized.

To this end, a methodology was developed to identify areas susceptible to coastal flooding at different future stage frequencies. Based on the level of flooding, and the length of shoreline that would need to be protected to mitigate the effects of flooding at a given stage frequency, a design flood frequency was selected for the HFFRRF for Jamaica Bay that would result in a manageable number of project features over the Study Area. Analysis of each HFFRRF must show that the design is cost effective in order to be recommended (i.e. the benefit to the nation must exceed the cost).

2.2 Mapping Areas at Risk

2.2.1 Stage Frequency Curves for Jamaica Bay

Flooding in the Jamaica Bay area is caused by a combination of astronomical tide and storm induced water-level rise. The storm induced water-level rise is caused by a combination of the wind induced shear stresses, decreasing atmospheric pressure, and storm waves raising water-levels along the shore. The water-level rise due to the former two effects is defined as storm surge, and together with astronomical tide level is called the total stage. The FEMA (2014) analysis was used as the basis for estimating expected stage elevations for this study, for sake of consistency with the Jamaica Bay Planning Reach Reformulation Study (USACE, 2016).

FEMA conducted coastal flood studies to analyze storm surge using ADCIRC (ADvanced CIRCulation) model in conjunction with SWAN (Simulating WAves Nearshore model). The combined modeling also accounted for the increase in stillwater level caused by the waves breaking in the nearshore, which is called 'wave setup'. This analysis produces the total stage elevations for the 1% annual chance flood event, or 1% annual exceedance probability (AEP) and are spatially variable. The FEMA (2014) modeling analysis led to the calculation of stage frequency statistics at several output points within the computational domain. The total stage elevations for the 20, 10, 5, 2, and 1% annual chance flooding- including the effects of wave setup- were gathered from the FEMA (2014) modeling analysis for model computational points within the Project Study area to establish a total stage frequency curve at each point. These stage elevations however do not take into account all effects from waves coming ashore during storm events such as wave runup and wave heights, which are accounted for separately.

The FEMA stage elevations are computed with respect to the Mean Sea Level (MSL) of the current NOAA tidal epoch[1], which is 1982-2001. These elevations were updated to the present sea levels by adding the observed change in sea level at Sandy Hook, NJ from the middle of the tidal epoch, 1992 to 2018.

Two NOAA gages are available near the Project site; the Battery and Sandy Hook. Both gages are similar distances from the project site, i.e. approximately 12-15 miles. However, the Sandy Hook gage and the project area are more similar geologically 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 ranges from 2.10 mm/yr at New London, CT to 3.97 mm/yr at Atlantic City, NJ, 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. This results in a correction of +0.34 ft to the stage elevations from FEMA. The final stage frequency curve at an example location within Jamaica Bay is presented in Table 2-1. The location of this point is shown in Figure 2-1.

Table 2-1: Total Stage Frequency Elevations (ft, NAVD[2]) based on FEMA* (2014)

Return Period[3] (years)	AEP (%)	Elevation based on year 1992 datum* (ft, MSL)	Elevation based on year 1992 datum* (ft, NAVD88)	Elevation based on year 2018 datum* (ft, NAVD88)
5	20	5.5	5.27	5.61
10	10	6.6	6.37	6.71
20	5	7.6	7.37	7.71
50	2	8.8	8.57	8.91
100	1	9.8	9.57	9.91

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

[1] The 19-year period over which sea level observations are taken and reduced to obtain mean values for datum definition.
[2] North American Vertical Datum
[3] Return Period (RP) is the average number of years expected between occurrences of storms of the same severity.

Figure 2-1: Location of output point CDM110 (also referred to as FIMP Station 61, FEMA at CDM Station 110 or NACCS Station 3992)

2.2.2 Projected Sea Level Change

The projected sea level change estimates the change in the mean level of the bay on the scale of decades. Several factors could contribute to long-term change in mean sea levels. 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, and other local effects due to regional ocean dynamics.

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 change (RSLC) 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. The relative sea level change was calculated for the difference between the current project start date (year 2018), and the end of the 50 year project planning horizon (year 2068) per USACE ER 1100-2-8162. These rates of rise correspond to 0.65 feet, 1.1 feet, and 2.54 feet over 50 years (year 2018-year 2068) for the low, intermediate and high rates of relative sea level rise. The intermediate projection scenario of 1.1 feet of relative sea level rise between year 2018 and year 2068 has been used to define water-levels for this study at the end of project service life. This is consistent with the selection of the intermediate case sea level rise projection used for planning purposes in the Atlantic Shorefront Planning Reach of the Reformulation Study, as well as the Jamaica Bay Planning Reach (USACE, 2016).

A uniform relative rate of rise of 1.1 feet was applied to the spatially varying total stage calculated based on FEMA modeling in the previous section, to calculate 2068 water-levels corresponding to various annual chances of occurrence. Table 2-2 shows the updated stage frequency elevations for 2068 for same the location shown in Figure 2-1. Although the modeled water levels vary spatially within Jamaica Bay, this table is illustrative of the trends in present and future stage frequencies within Jamaica Bay. The annual probability of occurrence of a given stillwater elevation is found to approximately double between 2018 and 2068 due to the effect of the projected relative sea level rise during this period.

Table 2-2: Total Stage Frequency Elevations (ft, NAVD88) based on FEMA* (year 2014) updated for 2068 MSL

Return Period (years)	AEP (%)	Elevation based on year 1992* datum (ft, NAVD88)	Elevation based on year 2018* datum (ft, NAVD88)	Elevation including RSLC to year 2068 (ft, NAVD88)
3	33	4.07	4.41	5.51
5	20	5.27	5.61	6.71
10	10	6.37	6.71	7.81
20	5	7.37	7.71	8.81
50	2	8.57	8.91	10.01
100	1	9.57	9.91	11.01

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

2.2.3 Existing Ground Elevations

Topographic data for New York City and Long Island based on LiDAR (Light Detection And Ranging) is available from USGS through NOAA's National Ocean Service. LiDAR surveys were most recently conducted in the Jamaica Bay area during March through April year 2014. The LiDAR measured surface elevations at an average post spacing of 0.7 meter or 2.3 feet. The elevation data was referenced to the NAVD88, GEOID12A vertical datum in metric units, and projected in horizontal coordinates of UTM, Zone 18, North American Datum of 1983 (year 2011), meters. The large extent of the survey data is broken up into smaller areas, with separate processed tiles of roughly 5,000 feet length available for download in LAZ format.

The LiDAR processing tiles in the study area were downloaded, and converted to raster images of 3-foot resolution to define existing ground elevations. Figure 2-2 shows the extracted elevations for the Hammels vicinity on the Rockaway peninsula.

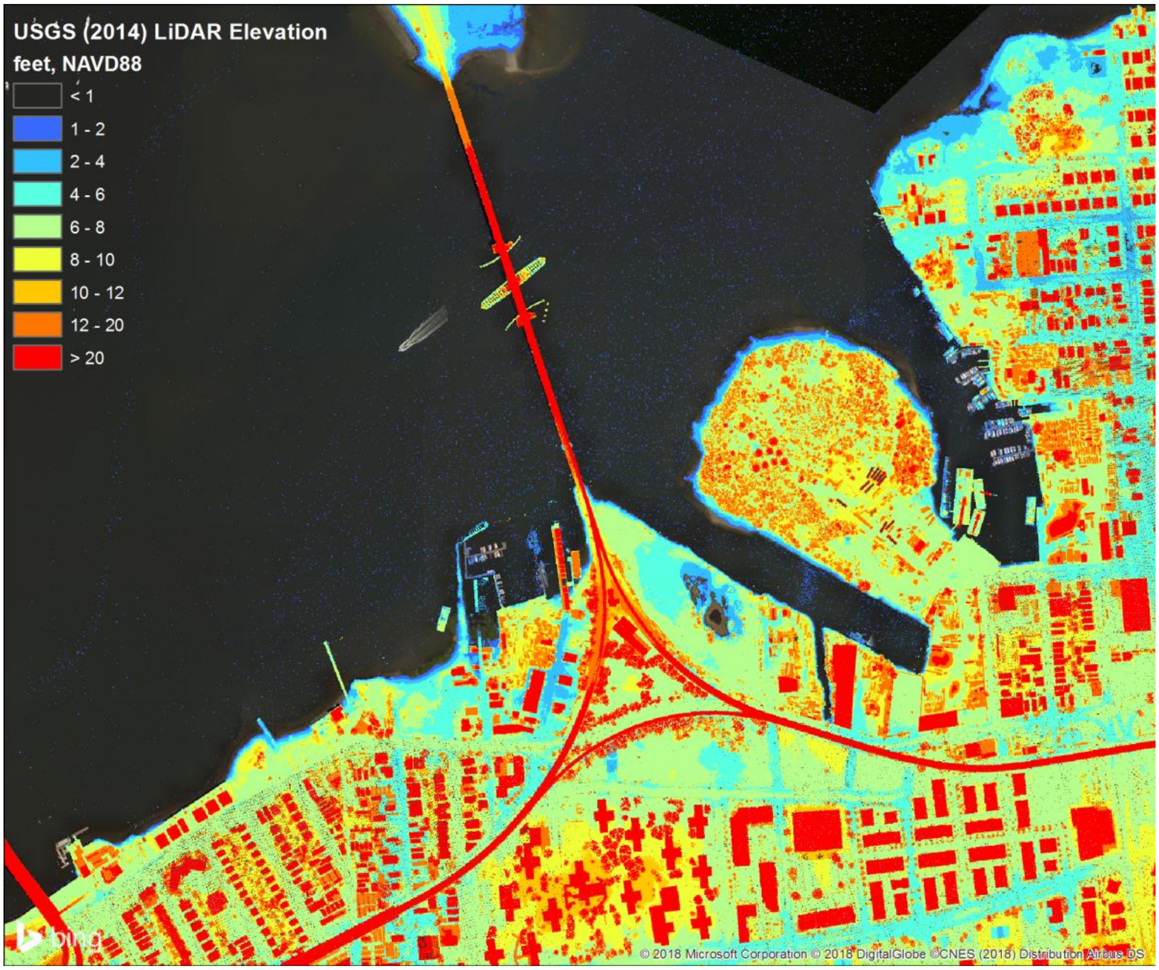


Figure 2-2: Map showing LiDAR Elevations extracted for Hammels vicinity, Rockaway

2.2.4 Methodology for Mapping Flood Extents

Areas at risk of future flooding due to tides, storm surge, wave setup, and projected sea level change, were mapped using a "bathtub" inundation analysis, which classifies coastal areas as flooded if their existing ground elevation is lower than the flood stage elevation, and are hydrologically connected to the bay. The year 2014 LiDAR ground elevations were subtracted from the spatially varying flood stage elevations computed as described in the previous sections to calculate an approximate inundation depth, and areas connected to the Jamaica Bay intertidal area with positive inundation depth were aggregated into the overall flood extent. Figure 2-3 shows a flowchart that summarizes the procedure followed for the mapping of approximate flood extents for the Jamaica Bay HFFRRF Study area.

However, it is important to note several simplifying assumptions made in the mapping of approximate flood extents. This analysis does not account for changing frequency of storms in the future due to climate change or other factors. It also does not account for any possible future changes in local bathymetry, shoreline, or topography. Moreover, the inundation analysis in relying mostly on the stage elevations along the coastline, and it does not take into account several factors that might affect overland flow such as the duration of the storm, local land-use category, sub-surface drainage infrastructure, or presence of vegetation.

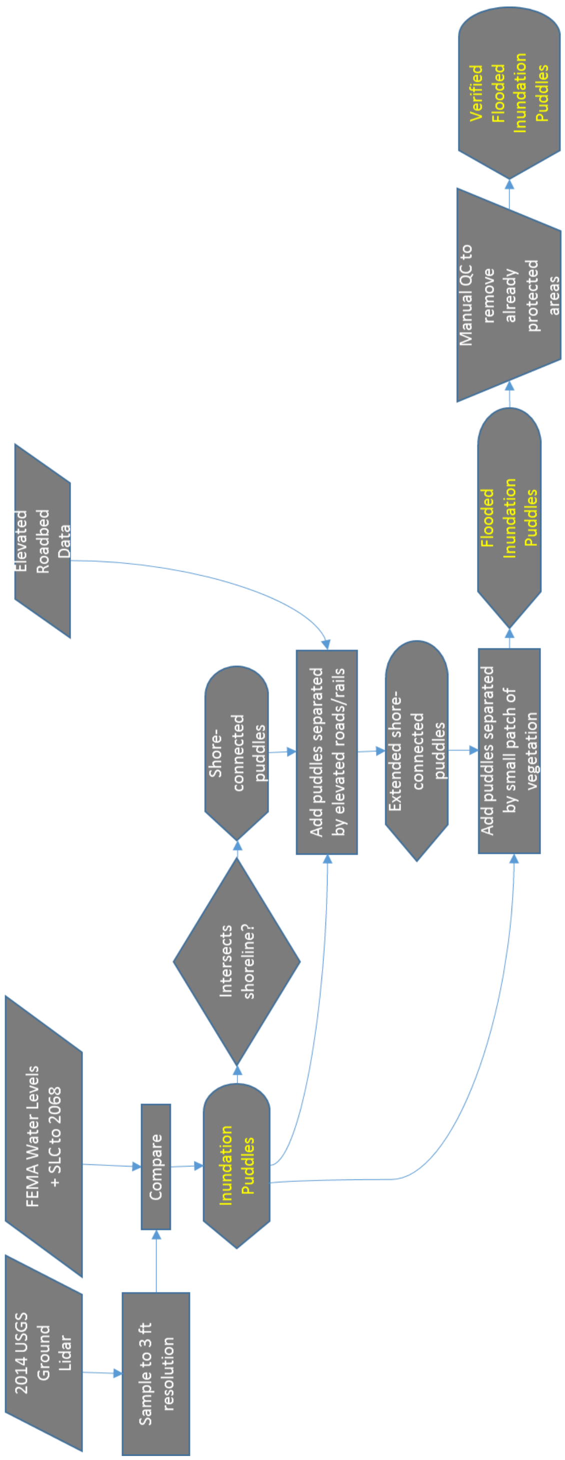


Figure 2-3: Flowchart illustrating methodology used for mapping flood extents for Jamaica Bay HFFRRF Study

2.3 Selection of Flood Frequency for HFFRRF Study

The flood extent mapping was performed starting with relatively lower stage elevations corresponding to a future 33% AEP (3-year RP) event in year 2068. The calculated flood extent map for such an event is shown in Figure 2-4. This shows relatively limited flooding, with mostly areas within the eastern Rockaway peninsula and Howard Beach and Canarsie neighborhood in Brooklyn being moderately affected. A similar flood extent map was calculated for approximately 1 foot higher stage elevations, corresponding to a future 20% AEP (5-year RP) event in year 2068. The corresponding flood extent map is also shown in Figure 2-4. In addition to the future 33% AEP (3-year RP) flood extent, this shows significantly higher flooding in Hammels, Arverne, and Edgemere neighborhoods of the Rockaway Peninsula, as well as increased flooding in the Five Towns area in Nassau County, and Broadchannel. The flood extents corresponding to even higher stage elevations corresponding to a future 10% AEP (10-year RP) event in year 2068 were also calculated as shown in Figure 2-4. This shows flooding occurring through most of the Jamaica Bay coastline of the Rockaway peninsula, as well as through most of Motts Creek into the Five Towns area of Nassau County. Providing effective risk reduction for such an event (10% AEP) would potentially require project features to be constructed along most of these coastlines, which were shown to be less efficient than operating the proposed tide gate at Rockaway Inlet.

In summary, flood extent maps were created for the Jamaica Bay side of the peninsula to assess the future year 2068 33%, 10% and 5% AEP floods - which are closely related to the current 20%, 10%, and 5% AEP floods (5, 10, and 20 year RP, respectively) floods. Inundation extents were analyzed to determine a tipping point of where significant flooding occurs. It is noted that the current 10% AEP showed significant flooding, as did the 5%, but that at the 5% the flooding was so extensive that it would require alignments more akin to the perimeter plan for Coastal Storm Risk Management (CSRM) (USACE-NAN, 2016). Since previously the barrier plan was already shown to be more cost effective than the perimeter plan and the HFFRRFs are meant to complement a potential future storm surge barrier and provide CSRM for the frequent flooding events for which the barrier would not be operated, thus the future 20% AEP flood frequency (5-year RP in year 2068) was selected as the flood frequency for the development of HFFRR features. The future 20% AEP (5 year RP in 2068), which amounts to a 10% AEP, or a 10 year RP in 2018 (see Table 2-2) incurs significant widespread flooding.

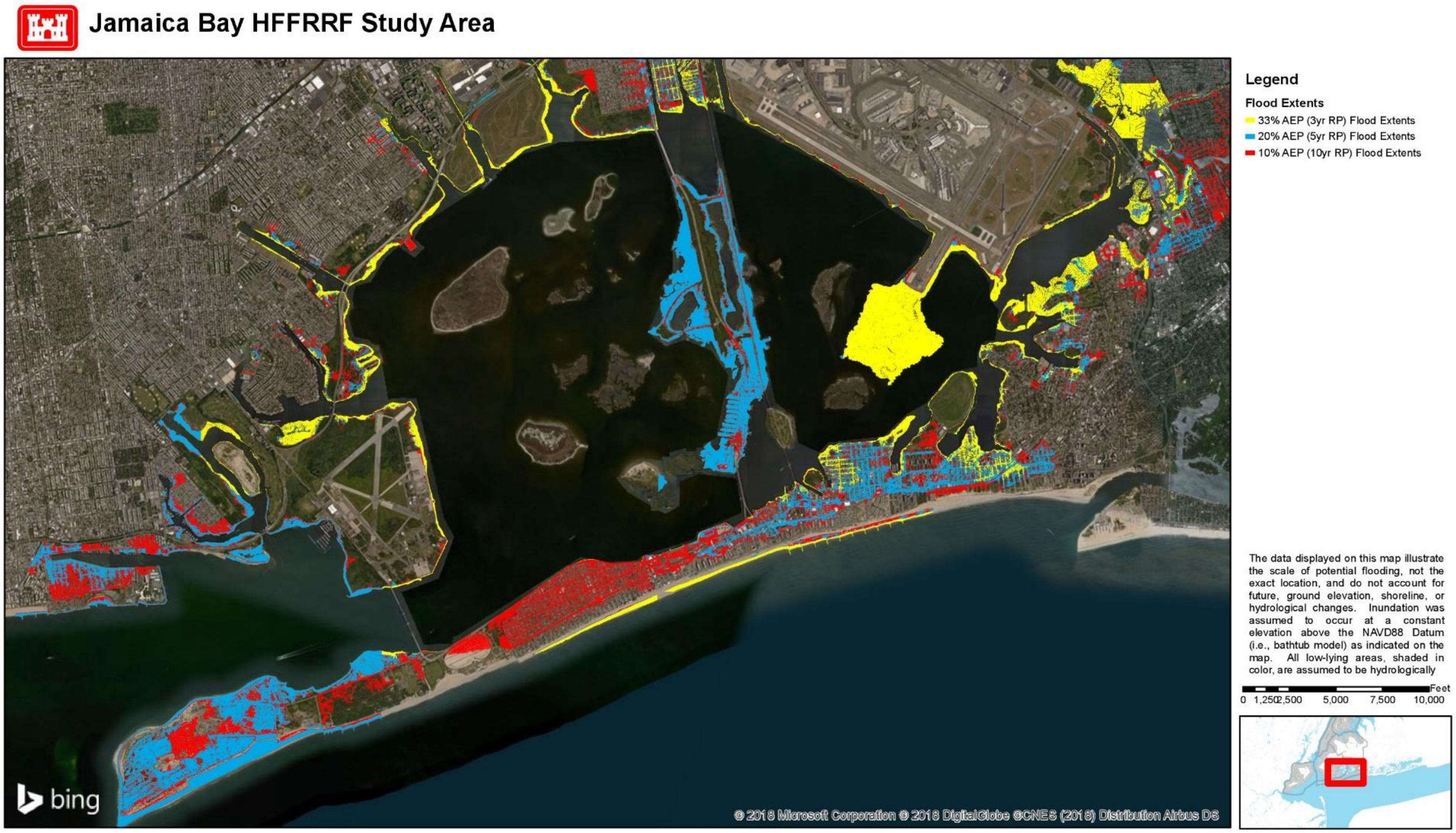


Figure 2-4: Future 33%, 20% and 10% AEP (year 2068) Stage Flood Extents for Jamaica Bay HFFRRF Study Area

2.4 Assets at Risk

The 20% AEP (5-year RP) flood frequency stage in year 2068 selected for the design of the HFFRR Features for Jamaica Bay affects over 75 miles of roads, and about 8,000 building footprints within the study area. For comparison, the 33% AEP (3-year RP) flood frequency affects about 16 miles of roads and 1,900 buildings, and the 10% AEP (10-year RP) flood frequency affects about 120 miles of roads and 14,000 buildings. These statistics are based on the road centerline and building footprints data from the NYC LION database and the Nassau County Five Towns study. Several of these assets serve as critical infrastructure systems for power, transportation, or emergency services.

Assets classified as Critical Infrastructure in the NY/NJ Harbor and Tributaries Study conducted by USACE were included in the computation of statistics of the critical infrastructure facilities within the Jamaica Bay HFFRRF Study Area. The Critical Infrastructure facilities were divided into several categories – namely, electrical, oil and gas, emergency/health, transportation, and education facilities. Geographic data on Critical Infrastructures under each category was obtained from the Homeland Infrastructure Foundation Level Database (HIFLD) available under the public domain from the U.S. Department of Homeland Security (DHS). Datasets not under the public domain, and available only with the proprietary Homeland Security Infrastructure Program (HSIP)-Gold license were not included in this analysis. Most of the datasets were represented as point features, which were considered to fall within a floodplain if the corresponding structure was within a 5 foot distance of a calculated flooded area. The count of such Critical Infrastructure facilities within each calculated flood-extent computed in the same manner is shown in Table 2-3. Other datasets such as oil pipelines, railroads, and tunnels were represented as line features, and the miles of such features intersecting flooded areas are shown in Table 2-4.

Table 2-3: Count of Critical Infrastructure facilities within calculated flood extents for the Jamaica Bay HFFRRF Study Area

Category	Type	Name	Source	In 33% AEP (3-year RP) Floodplain (Year 2068)	In 20% AEP (5-year RP) Floodplain (Year 2068)	In 10% AEP (10-year RP) Floodplain (Year 2068)
Electrical	Point	Electricity Generating Units	HIFLD	Duplicated in Electricity Power Generation Dataset		
	Point	Electricity Power Generation	HIFLD	0	0	0
	Point	Energy Distribution Contents Facilities	HSIP Gold			
	Point	Substations	HIFLD	1	2	5

Category	Type	Name	Source	In 33% AEP (3-year RP) Floodplain (Year 2068)	In 20% AEP (5-year RP) Floodplain (Year 2068)	In 10% AEP (10-year RP) Floodplain (Year 2068)
	Point	Nuclear Power Plants	HSIP Gold			
Gas Oil Port Facilities	Point	Natural Gas Compressor Stations	HIFLD	0	0	0
	Point	Natural Gas and Delivery Plant	HIFLD	0	0	0
	Point	Natural Gas Storage Facilities	HIFLD	0	0	0
	Point	Oil and Natural Gas Interconnects	HIFLD	0	0	0
	Point	Oil Refineries	HIFLD	0	0	0
	Point	Petroleum Pumping Stations	HIFLD	0	0	0
	Point	PCL Terminal, storage Facilities, Tanks and Farms	HSIP Gold			
	Line	Oil and Natural Gas Pipelines	HSIP Gold		N.A.	
Emergency Services	Point	Communication Centers (Cellular Towers)	HIFLD	0	0	0
	Point	Emergency Medical Service	HIFLD	3	5	8
	Point	Fire Stations	HIFLD		Duplicated in Emergency Medical Service Dataset	
	Point	Hospitals	HIFLD	0	0	0
	Point	Law Enforcement Location	HIFLD	1	2	4
	Point	Local Emergency Operations Centers	HIFLD	0	0	0
	Point	Historical Shelter System	HSIP Gold			
	Point	Nursing Homes	HIFLD	0	5	5
	Point	Waste Water Treatment Plants	HIFLD	0	1	1
Transport-ation	Point	Amtrak Station	HIFLD	0	0	0
	Point	Railroad Stations	HIFLD		Duplicated in Intermodal Terminal Facilities Dataset	

Category	Type	Name	Source	In 33% AEP (3-year RP) Floodplain (Year 2068)	In 20% AEP (5-year RP) Floodplain (Year 2068)	In 10% AEP (10-year RP) Floodplain (Year 2068)
	Point	Intermodal Terminal Facilities	HIFLD	0	6	7
	Point	Railroad Yards	HSIP Gold			
	Point	Road and Railroad Bridges	HIFLD	2	2	9
	Line	Road and Railroad Tunnels	HIFLD		N.A.	
	Line	Railroad	HIFLD		N.A.	
	Polygon	Airport Boundaries	HSIP Gold			
Education	Point	Public Schools	HIFLD	0	10	18
	Point	Private Schools	HIFLD	0	2	8
	Point	Day Care Centers	HIFLD	3	16	23
TOTAL				**10**	**51**	**88**

Table 2-4: Miles of Critical Infrastructure facilities within calculated flood extents for the Jamaica Bay HFFRRF Study Area

Category	Type	Name	Source	In 33% AEP (3-year RP) Flood Plain (Year 2068)	In 20% AEP (5-year RP) Flood Plain (Year 2068)	In 10% AEP (10-year RP) Flood Plain (Year 2068)
Gas Oil Port Facilities	Line	Oil and Natural Gas Pipelines	HSIP Gold			
Transport-ation	Line	Road and Railroad Tunnels	HIFLD	0	0	0
	Line	Railroads	HIFLD	2.0	4.2	5.9
TOTAL				**2.0**	**4.2**	**5.9**

3 DESIGN BASIS

3.1 General

Due to the fact that the shoreline conditions, e.g., natural, manmade, gradient, etc., vary throughout Jamaica Bay, a number of generalizations and assumptions were made to develop generic flood risk reduction features that could be applied at various locations. The following sections describe the development of a design basis for the HFFRRFs in Jamaica Bay.

3.1.1 Vertical and Horizontal Datum

Vertical elevations (EL.) of project components and features are referenced to the North American Datum of year 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 year 1983 (NAD83) Long Island, New York State Plane with units in feet, unless specifically noted otherwise.

3.1.2 Bathymetry, Topography and Shoreline Elevation

Bathymetric information was based on NOAA Navigation Charts (NOAA chart 12350, 12401, 12337, 12402, 12339, and 12366) and NOAA 1/9 arc-second DEMs available for Jamaica Bay as part of the NCEI Hurricane Sandy Digital Elevation Models. These DEMs integrate both bathymetric and topographic data at the coast.

High resolution topographic data was gathered from the post-Hurricane Sandy USGS LiDAR (https://lta.cr.usgs.gov/) imagery collected for New York City and Long Island between March, 2014 and April, 2014.

3.2 Coastal Engineering

3.2.1 Water Levels

Following the analysis described in Chapter 2, the stillwater levels corresponding to a 20% AEP, 5-year return period stage frequency on top of 2068 sea levels was used as the requirement for functional design of project features for HFFRR in Jamaica Bay. These stillwater elevations include the effects of tide, storm surge, and wave setup. The design water levels for function of the individual project features were determined based on nearest available FEMA (2014) model output, and updated to include the effects of sea level change as per USACE guidance as described in Section 2.2.2.

3.2.2 Waves

Wave conditions typically vary throughout Jamaica Bay. Depending on the project location, the project may be subject to ocean swell (e.g. Manhattan Beach), wind waves, ship waves (e.g. Beach Channel) or little to no wave energy (e.g. sheltered basins and canals).

3.2.2.1 Wave-Heights for Functional Design

Wave-heights along the shoreline of Jamaica Bay corresponding to different Annual Exceedance Probabilities (AEPs) were obtained from the USACE (2015) North Atlantic Comprehensive Coastal Study (NACCS) database. The analysis of extreme waves was conducted as part of the NACCS based on an STWAVE model for the North Atlantic Region. The NACCS simulations include 1,050 synthetic storms, and 100 historical storms. Expected significant wave-heights for the 20% AEP (5-year RP) were extracted from the database at 137 Save Points along the perimeter of Jamaica Bay shown in Figure 3-1. The data from the save points was applied to all of Jamaica Bay using Natural Neighbor interpolation.

However, for project features that are relatively sheltered or set back over 150 feet (approximately at least 3 wave-lengths) from the shoreline, the NACCS model wave-heights calculated at the shoreline might overstate the wave-heights likely to be observed at the feature location. For such locations, during Phase 1 of the screening analysis, a maximum design wave-height of one foot was applied. As part of Phase 2, the expected wave-heights were updated to account for wave transformation, and is documented in Sub-Appendix D.

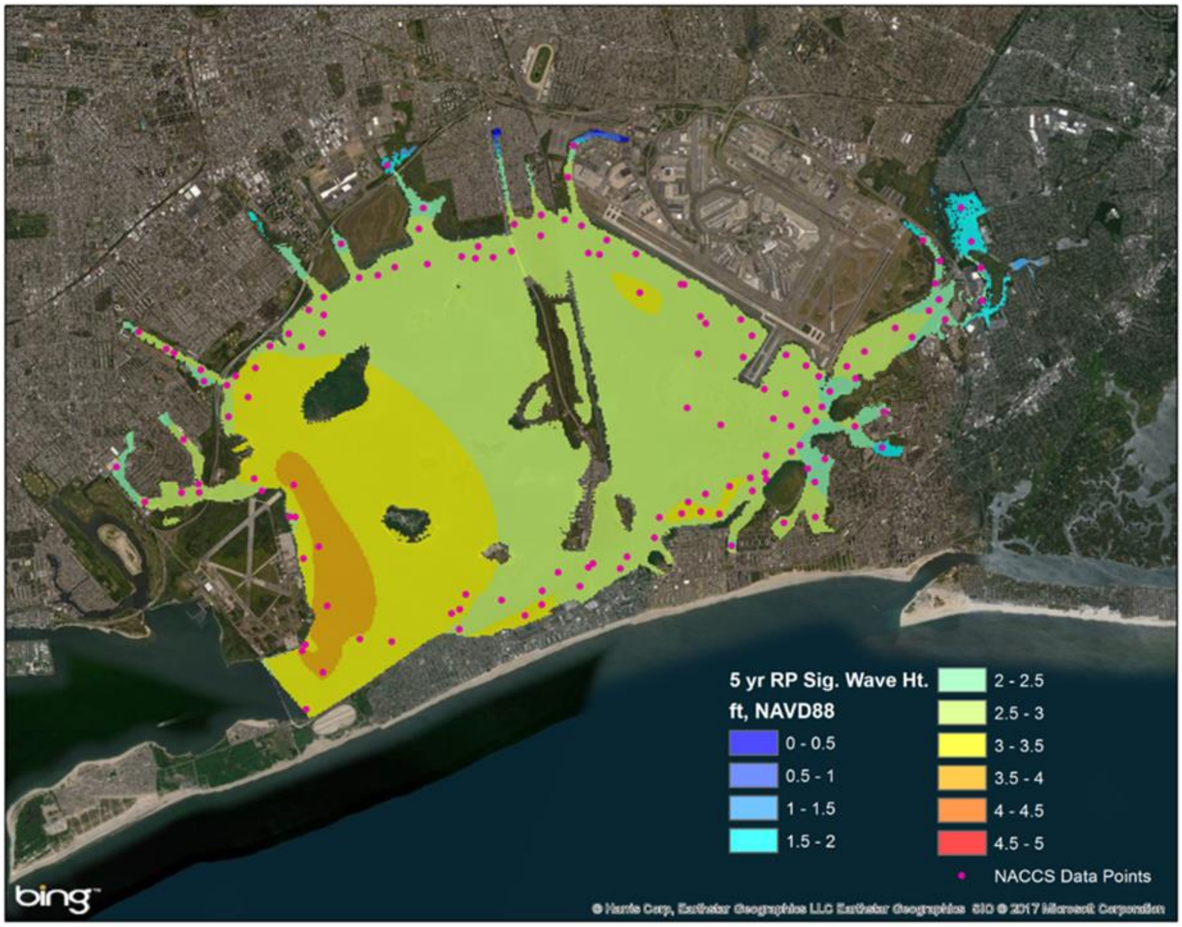


Figure 3-1: Distribution of 20% AEP (5-year RP) wave-height across Jamaica Bay.

Overtopping Criterion and Required Freeboard

Wave action superimposed with flood stage can generate significant overtopping of coastlines, potentially resulting in additional inundation of adjacent coastal areas. This would have implications for interior drainage infrastructure and the pump capacity requirements. A wave overtopping discharge of one liter per second per meter over about a thousand feet of shoreline would require a 50 cubic foot per second (cfs) discharge rate pump to provide drainage for additional inundation due to the overtopping. This requirement would increase should the wave overtopping discharge increases. Therefore, an overtopping threshold of one liter per second per meter was used to calculate freeboard requirements for features. The required wave freeboard was calculated for each project feature based on guidance in EurOtop II (2016) using the design wave-height at the project feature established as described previously.

3.2.2.2 Wave-Heights for Structural Design

For structural design or limit of feature failure, 100 year return period wave-heights were extracted for the Save Points within Jamaica Bay from the NACCS model database. The 100 year return period wave condition corresponds approximately to a wave-height of about 3.5 feet and wave period of three seconds over most of Jamaica Bay, and this was selected as the design wave condition for the project features at the screening level of analysis.

Wave Forces

The wave forces on the project features were computed following the methodology developed by Goda (1974) as recommended in the USACE (2002) Coastal Engineering Manual, based on the selected 100 year design wave-height and wave period. The wave forces were computed for each project feature for several combinations of water level and wave-height, and the combination leading to the highest wave force on the structure was chosen as the design condition for that feature. The highest wave force on the feature typically occurs when the maximum design wave-height meets the crest elevation of the structure.

3.3 Rainfall Run-off and Interior Drainage Considerations

The preliminary screening was conducted on a design governed by the 20% AEP still water levels. Additional effort to assess interior drainage impacts and related costs, as well as real estate costs was undertaken in Phase 2 subsequent to the preliminary screening in Phase 1. For the preliminary screening, the definition of project alignments focused on smaller and more isolated flood prone areas. The preliminary screening level analysis was completed with the understanding that a detailed drainage assessment would be completed after the initial screening of HFFRRF Projects. This approach allowed the PDT to screen out non-viable HFFRRF alternatives without completing a resource intensive interior drainage analysis of all areas where HFFRRF project alternatives are defined. In other words, if a project did not pass the initial screening without a diligent analysis of interior drainage impacts, then it was screened out without further analysis since the additional interior drainage requirements would only further decrease the benefit to cost ratio.

3.4 Structural Engineering

3.4.1 Design Basis References

The following codes, references, and standards were used as a basis for the design of the HFFRRF:

1. American Association of State Highway and Transportation Officials (AASHTO). *Standard Specifications for Highway Bridges, 17th Edition.*
2. AASHTO. *Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Customary U.S. Units, 6th Edition.*
3. American Concrete Institute (ACI). *ACI 350-06 Code Requirements for Environmental Engineering Concrete Structures.*
4. ACI. *ACI 318-11 Building Code Requirements for Reinforced Concrete.*
5. American Institute of Steel Construction (AISC). *Manual of Steel Construction, Load and Resistance Factor Design (LRFD), 14th Edition.*

6. American Society of Civil Engineers (ASCE). *ASCE 24-14 Flood Resistant Design and Construction.*
7. ASCE. ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures.*
8. American Welding Society (AWS). *ANSI/ AWS D1.1-2010, Structural Welding Code – Steel.*
9. NYC Building Code (NYCBC), 2014.
10. United States Steel (USS). *U.S. Steel Sheet Piling Design Manual, 1984.*
11. United States Army Corps of Engineers (USACE). *HSDRRSDG Hurricane and Storm Damage Risk Reduction Design Guidelines with June 2012 updates.*
12. USACE. *Engineer Manual (EM) 1110-2-584 Hydraulic Steel Structures.*
13. USACE. *Engineer Manual (EM) 1110-2-1614, Design of Coastal Revetments, Seawalls and Bulkheads.*
14. USACE. *Engineer Manual (EM) 1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures.*
15. USACE. *Engineer Manual (EM) 1110-2-2504 Design of Sheet Pile Walls.*
16. USACE. *Engineer Manual (EM) 1110-2-2906, Design of Pile Foundations.*
17. USACE. *Engineer Engineering Technical Letter (ETL) -1110-2-58, Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, And Appurtenant Structures.*

3.4.2 Design Elevations

The design elevation of each HFFRRF is dependent on the site conditions and the feature type. The elevation of the top of flood risk reduction features is set to 20% AEP water level at the expected end of the project service life (year 2068), considering the overtopping criterion (threshold of 1 liter/s/m, see Section 3.2.2.)

While actual existing ground elevations vary around Jamaica Bay, general assumptions had to be made for the upland (protected side) ground elevation of the HFFRRF such that the feature designs were generic enough and implementable at various locations within the study area. The ground elevation at the shoreline in general varies between +3 feet NAVD88 and +7 feet NAVD88; the generic features designed for this study accommodate this variation.

3.4.3 Geometric Considerations

The HFFRRF were developed based on a range of generalized geometric considerations. These geometric considerations are necessary to ensure the proper function of, and safe access to, the HFFRRF. The geometric considerations for the HFFRRF project are listed below:

Access Ramp Slope (maximum): 10%

Patrol Road/Access Ramp Width – Single Lane (minimum / desirable): 12 feet / 16 feet

Patrol Road/Access Ramp Width – Two-Lanes (minimum / desirable): 24 feet / 32 feet

Living Shoreline Riparian Zone Width (minimum / desirable): 30 feet / 100 feet

Living Shoreline Riparian Zone Slope (minimum / desirable): 20h:1v / 50h:1v

Berm Crest Width (minimum/ desirable): 5 feet / 10 feet

Berm Front Slope (minimum/ desirable): 2h:1v / 3h:1v

Berm Back Slope (minimum/ desirable): 2h:1v / 3h:1v

Revetment Front Slope (minimum/ desirable): 2h:1v / 3h:1v

3.4.4 Material Unit Weights

The primary material unit weights for the HFFRRF include steel, concrete, water, riprap and granular fill. Unit weights are used to convert volume to weight in order to calculate the self-weight of the various features. The self-weights are then used in structural and stability calculations. The unit weights are also used to determine the costs of the structures. Material unit weights used for the project are listed in Table 3-1.

Table 3-1: Material Unit Weights

Material	Unit Weight (lbs/ft^3)
Steel	490
Concrete (normal weight)	150
Water (salt)	64
Riprap (armor, toe and underlayer stone)[4]	132
Fully-Compacted Granular Fill, **Total**	120

3.4.5 Material Specifications

Material specifications are used to ensure that the materials used will be uniform across the HFFRRF and of sufficient quality to ensure its proper function for the duration of the project service life. The following material specifications were used as the minimum parameters for the HFFRRF features. All materials shall be new and of the best quality of their respective kinds as described or if not stated, to be at least in accordance with the relevant American Society of Testing Materials (ASTM) Standards.

The followings are noted with respect to material specifications:

- Structural steel shall conform to – American Society for Testing and Materials specification (ASTM) A992 for wide flanges, A572 Grade 50 for other structural members.
- Steel sheeting, combi-wall systems and HP sections (HP 14X73, HP 14X89 etc.) shall conform to ASTM A690 or ASTM A572 Grade 50, steel pipe piles shall conform to ASTM A252 Grade 3 (50 ksi) or ASTM A572 Grade 50.

[4] Weight of riprap may vary based on the filling of the riprap voids over time.

- Steel reinforcement in concrete shall conform to ASTM A615, Grade 60.
- Reinforced concrete shall have a minimum 28-day compressive strength (f'c) of 4,000 psi, maximum water/cement ratio 0.40.
- Lean concrete shall have a minimum 28-day compressive strength (f'c) of 3,000 psi.
- Minimum cover to reinforcement for concrete exposed to marine environment shall be 4 inches.
- Structural steel members exposed to marine environment shall be coated or galvanized. Steel foundations in the water, such as steel sheeting and steel piles shall be coated with coal tar epoxy and include 1/16 in. corrosion allowance in wall thickness (5/16 in. minimum).

3.4.6 Design Loads

Design loads refers to the various types of forces that can reasonably be expected to act on the HFFRRF and are used in the structural design and stability analysis. The following design loads were used as the minimum criteria for the HFFRRF. The design codes, references, and standards are listed in Section 3.4.1 Design Basis References. The followings are used for design load calculations:

- Hydrostatic Loads – hydrostatic loads for the 100 year return period Design Storm Condition is based on the design storm stillwater levels (SWL) listed in the Section 3.4.1.
- Hydraulic & Wave Loads – hydrostatic loads for the 100 year return period Storm Conditions in Combination with the 100 year return period wave conditions.
- Vessel Impact – Not considered for feasibility (will be considered during PED).
- Debris Impact – Not considered for feasibility (will be considered during PED).
- Seismic Load – as per American Society of Civil Engineers standards (ASCE) 7-10 and The New York City Seismic Code.
- Wind Load – for structure Category IV, ASCE 7-10.
- Temperature – uniform change, 40°F drop and 30°F rise for moderate climate for both steel and concrete.
- Basic Load Combinations – as per Table 5.1 of Hurricane and Storm Damage Risk Reduction System Design Guidelines (HSDRRSDG) – see below for additional details.

3.4.7 Stability Analysis for Gravity Structures

A stability analysis was performed to ensure the HFFRRF would not fail due to instability. The three modes of instability are:

1) Sliding: The structure moving horizontally
2) Bearing: The structure sinks into the ground, caused by a lack of soil bearing capacity and/or an insufficient foundation design.

3) Resultant Location: The entire base must be in compression for the usual load condition to maintain full contact between the structure and the foundation. For storm conditions, 60% of the base shall be in compression.

The stability analysis follows United States Army Corps of Engineers (USACE) publications, minimum factors of safety and resultant location limit are provided in Table 3-2 below.

Table 3-2: Minimum Factors of Safety or Resultant Location Limits (EM 1110-2-2502)

	Sliding, Factor of Safety	Bearing, Factor of Safety	Resultant Location Limits
Operational Condition	1.5	3.0	100% of base in compression
Design Storm Condition	1.25	1.5	60% of base in compression

3.4.8 Geotechnical Considerations

Geotechnical considerations refer to the subsurface soil composition on which the HFFRRF will be built, i.e. bearing capacity, soil type, etc. No site specific geotechnical analysis was completed. In general, the structural analysis relied on the geotechnical analysis and data completed previously and reported in USACE-NAN 2016.

3.4.9 Structural Conditions

A coastal structures condition survey was not conducted. It is conservatively assumed that the majority of coastal structures, bulkheads, retaining walls, revetments etc., within the project areas are in poor condition, deteriorated, and are no longer functioning effectively. The majority of these structures are built on private property, and there is little evidence that the coastal structures are under periodic maintenance. The majority of these structures are built on private property, and there is little evidence that the coastal structures are under periodic maintenance. For the HFFRRF projects it is assumed that all existing coastal structures will be replaced and materials will be hauled offsite to a disposal facility.

3.5 Easements

In order to construct and then maintain the HFFRRF, USACE will require two easements: 1) Perpetual Flood Protection Levee Easement, 2) Temporary Work Area Easement. The Perpetual Flood Protection Levee Easement is based on guidance from ETL-1110-2-583 which details the requirements for vegetation free and vegetation managed zones. The Perpetual Flood Protection Levee Easement is furthermore based on USACE NAN project experience from projects within the region, as is the Temporary Work Area Easement. The Temporary Work Area Easement has been established to allow for all construction, staging, grading, landscaping, and other construction-related activities. The Temporary Work Area Easement will remain active until final acceptance and contractor demobilization. The Perpetual Easement will be established to allow the HFFRRF structures to be inspected and maintained. It is permanent in nature and USACE will have the right to use the land within the easement lines. Given the close proximity to private property, additional refinement and site specific details will be required during the PED phase. Table 3-3 summarizes the Easement Limits.

Table 3-3: Easement Limits

	Perpetual Easement		Temporary Work Area Easement	
Feature Type	Flood Side [ft.]	Protected Side [ft.]	Flood Side [ft.]	Protected Side [ft.]
Low Floodwall	**18**	**15**	**25**	**25**
Medium Floodwall	**15**	**17**	**25**	**25**
High Floodwall	**15**	**19**	**25**	**25**
Low Berm	**28.5**	**39.5**	**30**	**40**
Medium Berm	**33.5**	**42.5**	**35**	**45**
High Berm	**43.5**	**54.5**	**45**	**55**
Hybrid Berm	**15**	**43**	**25**	**45**
Shallow Bulkhead	**17**	**15**	**25**	**15**
Deep Bulkhead	**18**	**15**	**25**	**15**
Revetment with Floodwall	**52**	**20**	**55**	**25**
In Water Gate	**18**	**15**	**25**	**25**

4 HIGH FREQUENCY FLOOD RISK REDUCTION FEATURES

4.1 Development of Project Features

4.1.1 Development of Generic HFFRR-Features

To generate project alternatives that would reduce the risk of flooding from high frequency storm events (i.e. the 20% AEP storm event in the year 2068), features that provide a flood risk reduction function were developed and designed. In accordance with USACE's SMART Planning principles, the features were prepared and developed at a level of detail required to make the decision at hand. Detailed design and analysis has progressed through the Feasibility Study and will be furthered in the Pre-Construction Engineering and Design (PED) Phase. As such, the two-tier screening approach detailed within this appendix resulted in a refinement of Phase 1 project features in Phase 2 and the development of additional HFFRRF to better match the site conditions of the Phase 2 projects. Generic features which are shared across both phases of the study include, among others, floodwalls, berms, bulkheads, and revetments. In conjunction with the flood risk reduction features, where erosion is a concern and the physical constraints of their locations are conducive, a series of Natural and Nature Based Features (NNBFs) were developed to control erosion and improve overall function and resilience of the HFFRRF. Especially for berms where NNBFs serve in the design as an integral engineering feature to the CSRM design in order to control erosion and help manage coastal flood risk.

In addition to the coastal flood risk reduction features, a series of drainage features were developed to be used in conjunction with the line of risk reduction. Drainage features are necessary to remove rain water runoff and overtopping stormwater from the protected side of the HFFRRF.

Apart from the generic HFFRRF described in this chapter, a few more specific feature designs were developed where required. Such features include a breakwater, in water gate structures, hybrid berm, road ramp, and road raising, amongst others. As certain Phase 1 project areas were screened out, some of the Phase 1 feature types were also screened out. A table detailing which feature types were included in the two phases of this study is shown below.

Table 4-1: Feature Type Inclusion per Phase

HFFRRF Type	Phase 1	Phase 2
Low Floodwall	√	√
Medium Floodwall	√	√
High Floodwall	√	√
Low Berm	√	√
Medium Berm		√
High Berm	√	√
Shallow Bulkhead	√	√
Shallow Bulkhead – Urban Application	√	√
Deep Bulkhead	√	√
Deep Bulkhead – Urban Application	√	
Street End Bulkhead	√	
Revetment with Floodwall	√	√
In Water Gate	√	
Hybrid Berm		√
Vehicular Gate	√	√
Road Ramp		√
Road Raising	√	
Breakwater	√	
NNBF	√	√
Drainage Features	√	√

4.1.2 Existing Shoreline Considerations

Existing shoreline features for all the project sites were assessed using publicly available satellite images. In general, the existing shoreline features were classified into the categories shown in Table 4-2. The potential for selection of the HFFRRF is partially informed by the existing conditions. This table presents the applicability of the generic measures by shoreline type. The design and variations of these features are informed on the existing shoreline type, shoreline condition, and land use. Typical application opportunities, constraints, and existing conditions were considered as well. A project alignment that reduces the risk of flooding consists of a series of adjoining HFFRRF. All features used within this study are described in the following sections.

Table 4-2: Existing shoreline features and general applicability of HFFRRF

	HFFRRF TYPE															
Existing Shoreline Features	Low Floodwall	Medium Floodwall	High Floodwall	Low Berm	Medium Berm	High Berm	Shallow Bulkhead	Shallow Bulkhead - Urban	Deep Bulkhead	Deep Bulkhead - Urban	Revetment with Floodwall	Natural and Nature Based Feature	Hybrid Berm	Vehicular Gate	Road Ramp	Road Raising
Natural Shoreline	√	√	√	√	√	√						√	√			
Revetment											√					
Bulkhead							√	√	√	√						
Parks or Wetlands	√			√	√	√						√	√			
Street End	√	√	√				√	√	√	√				√	√	√
Urban Waterfront Development	√	√	√				√	√	√	√				√	√	√
Industrial Waterfront Development	√	√	√				√	√	√	√				√	√	√

4.2 Low Floodwall, Medium Floodwall and High Floodwall

Floodwall systems are independent, single purpose structures that aim to provide flood protection. A floodwall is a reinforced concrete structure supported on steel H-piles. A steel sheet pile cut-off wall was provided for seepage control. Within the realm of providing risk reduction features to prevent inundation as a result of high frequency flooding, three types of prototypical floodwalls were designed and labeled “low,” “medium,” and “high.”

For the low floodwall design, the approximate existing ground elevation was assumed to be at El. 5 feet which is deemed appropriate for the typical site conditions. Note that there is no specific intent to significantly alter the existing upland ground elevations, however in order to provide a generic conceptual floodwall design generalizations were needed such that one prototypical design could be applicable at multiple locations. The L-shape reinforcement concrete structure terminates at El. 8 feet and is supported on the battered H-piles with the vertical steel sheet piles used as seepage control measure. A typical cross-section for the low floodwall is shown in Figure 4-1.

For the medium and high floodwall, the approximate existing ground elevations were assumed to be El. 5 feet and El. 3 feet and top of wall elevations were set at El. 10 feet and El. 11 feet, respectively. The reinforced concrete structure is shaped like an inverted “T” and is supported on pairs of vertical H-piles. Typical cross-sections for medium and high floodwall are shown in Figure 4-2 and Figure 4-3, respectively. Pile design depends on design loads and soil parameters. For this study, soil characteristics as described in Chapter 3 were used.

Due to the relatively small footprint, a floodwall is deemed suitable for flood-prone urban waterfront areas, both directly at the shoreline and farther inland, where there are no existing structures. It should be noted that flood-prone waterfront areas are likely to have poor soil conditions and require excavation and backfilling prior to construction.

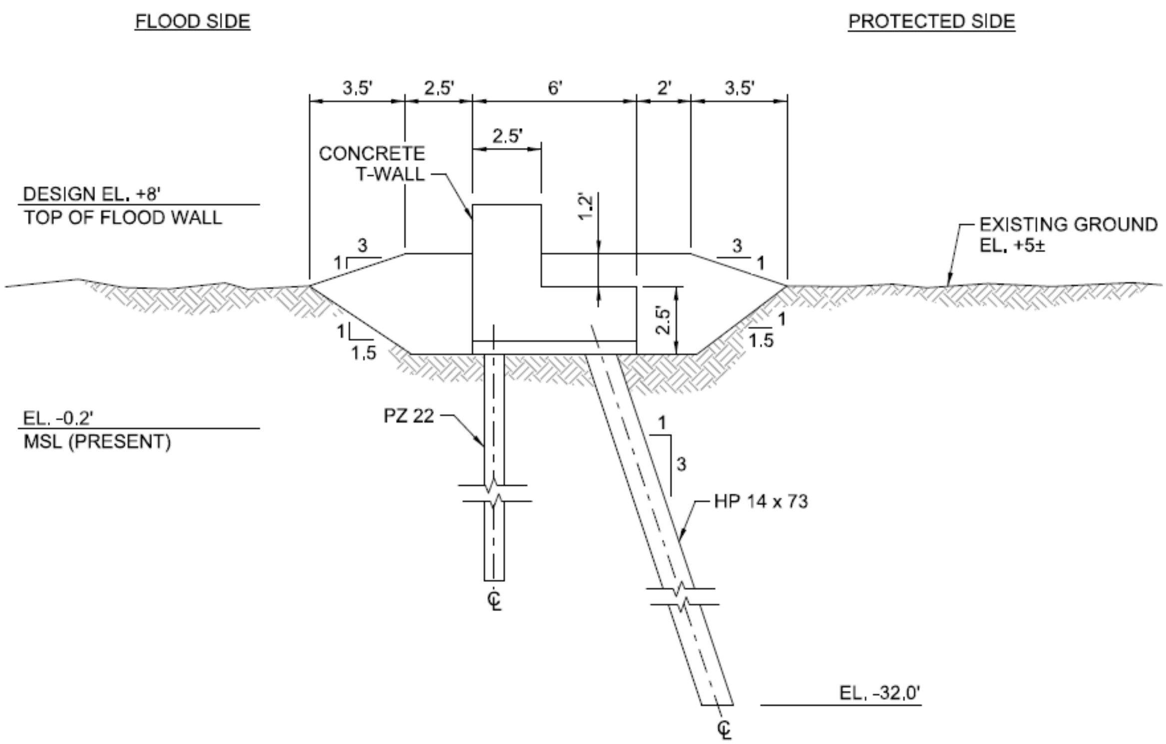


Figure 4-1: Low Floodwall Cross-Section

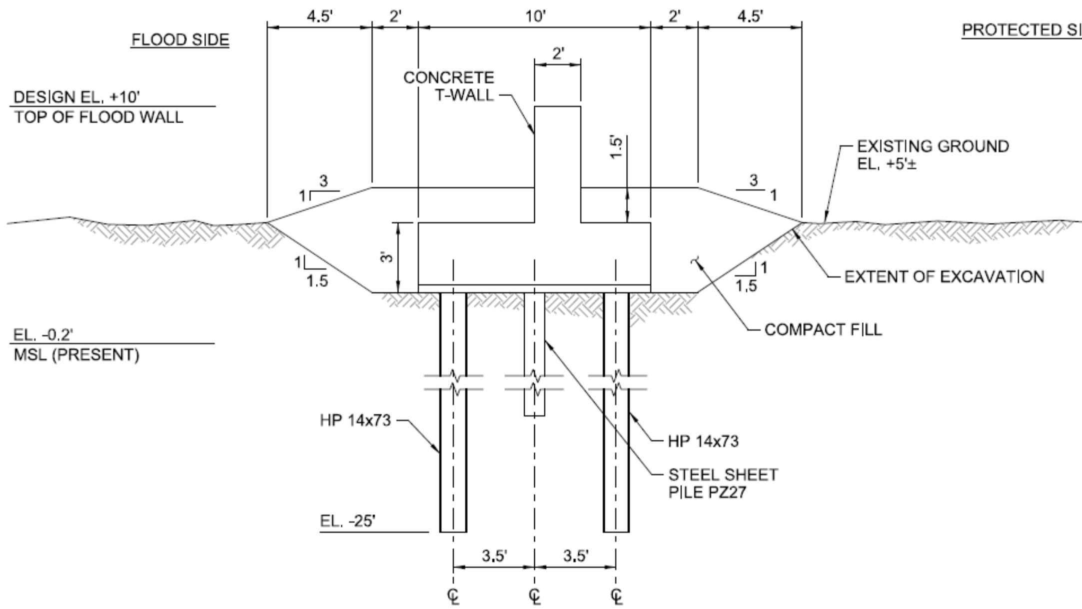


Figure 4-2: Medium Floodwall Cross-Section

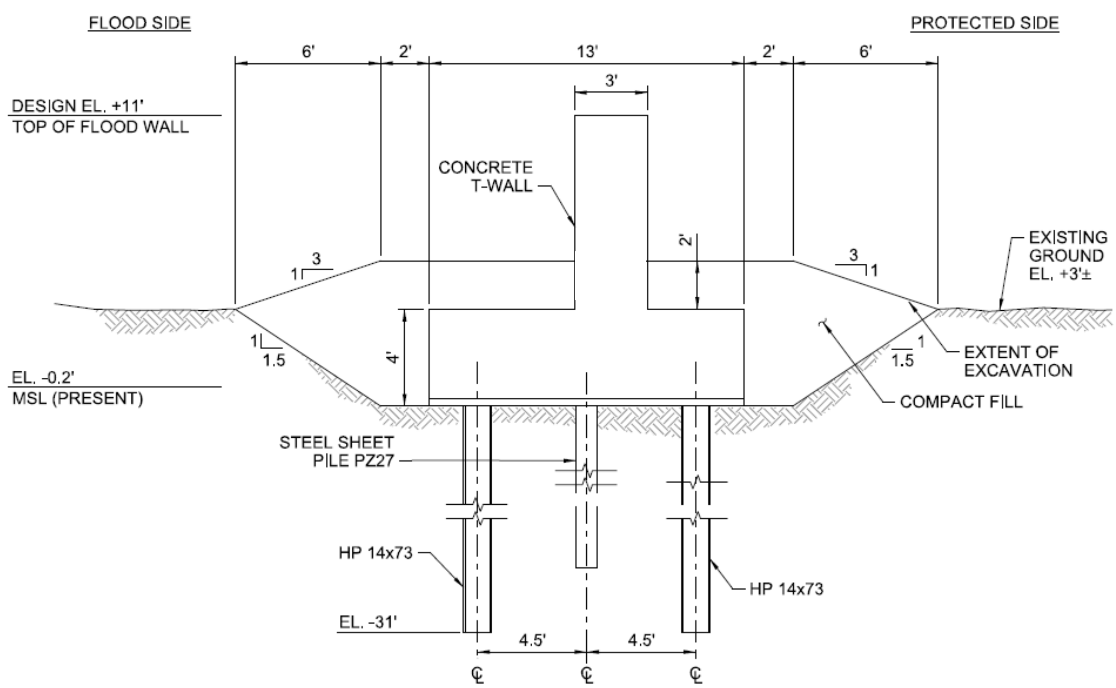


Figure 4-3: High Floodwall Cross-Section

4.3 Low Berm, Medium Berm and High Berm

Figure 4-4: Typical Berm

Whereas floodwalls are made of materials such as concrete and steel, berms are made of compacted soil and vegetation and are considered to be a nature-based feature. Figure 4-4 shows the natural appearance of a typical berm. Berms are commonly used along rivers and bodies of water to prevent flooding of the adjacent inland grade elevation.

Berms are typically constructed by piling earth on a cleared and leveled surface; soil is compacted into a large earthen structure that is wide at the base and tapers toward the top. The interior of the berm is a core composed of impervious material, usually clay, to form a watertight barrier to prevent or minimize seepage. Grass or some other type of non-woody vegetation are commonly planted on the berm to add stability and protection from erosion. The vegetation on the berm increases its aesthetic appeal. Berms can be further enhanced through combination with tidal marsh and rock sill NNBFs.

Berms on poor soil are subject to instability and settling, and therefore, require deeper excavation prior to construction. For this study, it was assumed the berm is founded on soil of medium quality. As discussed in Section 3.4.8, no site specific geotechnical analysis was completed. For the proto-typical design for the low berm, three feet of soft-consistency material would be excavated from the ground elevation of El. +5.5 feet. Similarly, for the medium and high berm, material would be excavated from the existing ground elevation of El. +4 feet and El. +3 feet, respectively.

To minimize seepage concerns, facilitate maintenance, and allow for ease of construction, a crest width of 5 feet and 7 feet was used for the low and medium berm design, respectively. Since the high berm also has to meet roadway requirements and future emergency needs, a 10-foot crest width was used in the design of the high berm. A side slope of 1 vertical on 2.5 horizontal was used for both the low, medium and high berm to minimize erosion and scour potential, and provide sufficient stability.

Due to the berm width and required setbacks, relatively large tracts of real estate are usually required. For this reason, berms are best suited along natural shoreline or parallel to the course of streams and basins within Jamaica Bay and set away some distance from the developed areas.

Figure 4-5, Figure 4-6 and Figure 4-7 show the typical low berm, medium berm, and high berm HFFRRF, respectively.

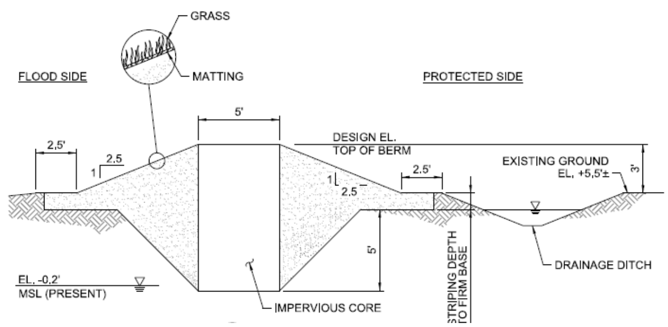


Figure 4-5: Low Berm Cross-Section

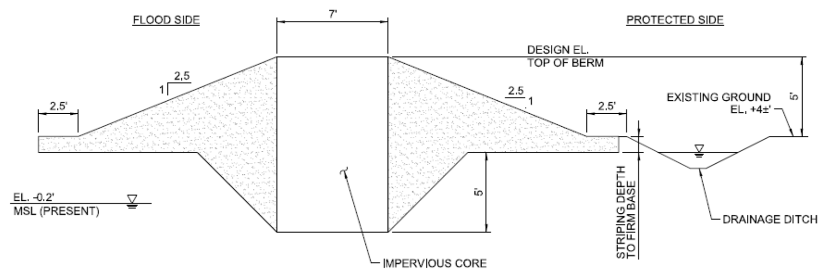


Figure 4-6: Medium Berm Cross-Section

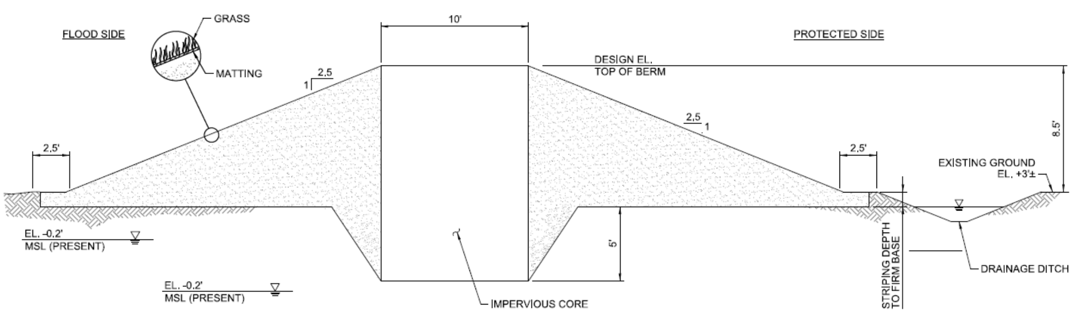


Figure 4-7: High Berm Cross-Section

4.4 Shallow Bulkhead and Deep Bulkhead

4.4.1 Bulkhead Design

A bulkhead wall is typically comprised of a steel sheet pile wall with or without a pile cap. The sheet pile wall is a row of vertical interlocking piles driven to form an integrated straight wall. Figure 4-8 shows an example of a bulkhead with concrete cap in Brooklyn, New York.

For this study, the bulkhead wall consists of a steel sheet pile wall with a reinforced concrete pile cap. On the protected side, the concrete cap extends down from El. 11 feet to the existing ground elevation at El. 7 feet. A concrete splash pad 5 feet wide at design existing ground elevation of El.7 feet was provided for scour protection. To fill in the gap between the new sheet piling and the existing bulkhead/shoreline, backfill was provided.

While their main function is usually to retain and prevent sliding of land, bulkheads, if vertically extended beyond existing grade and constructed watertight, can also reduce the risk of upland flooding. Bulkheads on poor soil require longer sheet pilings. Because flood-prone waterfront areas in Jamaica Bay are likely to have poor soil material, it was assumed that the soil in front of the sheet piling is characterized by poor sand. Soil behind the sheet piling was assumed to be backfill of medium sand up to existing ground elevation. Two different existing mudline elevations, El. -3 feet and El. -8 feet, were used to establish the design of the prototypical shallow and deep water bulkhead as HFFRR-Features, respectively. This was done to capture the varying conditions throughout Jamaica Bay in which bulkheads would be applied as generic measure. The deeper the water (lower mudline elevation), the heavier and longer the sheet piling required. Sheet size and length for the shallow and deep water bulkheads are shown in Figure 4-9 and Figure 4-10.

The relatively small footprint of a bulkhead renders it a preferred solution to urban or developed waterfront areas that are subjected to flooding. At some locations the Jamaica Bay waterfront can be characterized by a series of discontinuous and heterogeneous existing bulkheads that are privately owned with limited real estate for new structures. The Meadowmere (Queens), Arverne (Queens), Howard Beach (Queens) neighborhoods are just a few examples where such conditions exist. In order to develop a prototypical feature, as in the case of bulkhead construction, the existing bulkhead structure is assumed to be non-functional, because privately-owned bulkheads typically have no comprehensive maintenance program in place and hence likely experience some deterioration.

Figure 4-8: Bulkhead at South 5th Street Brooklyn, NY

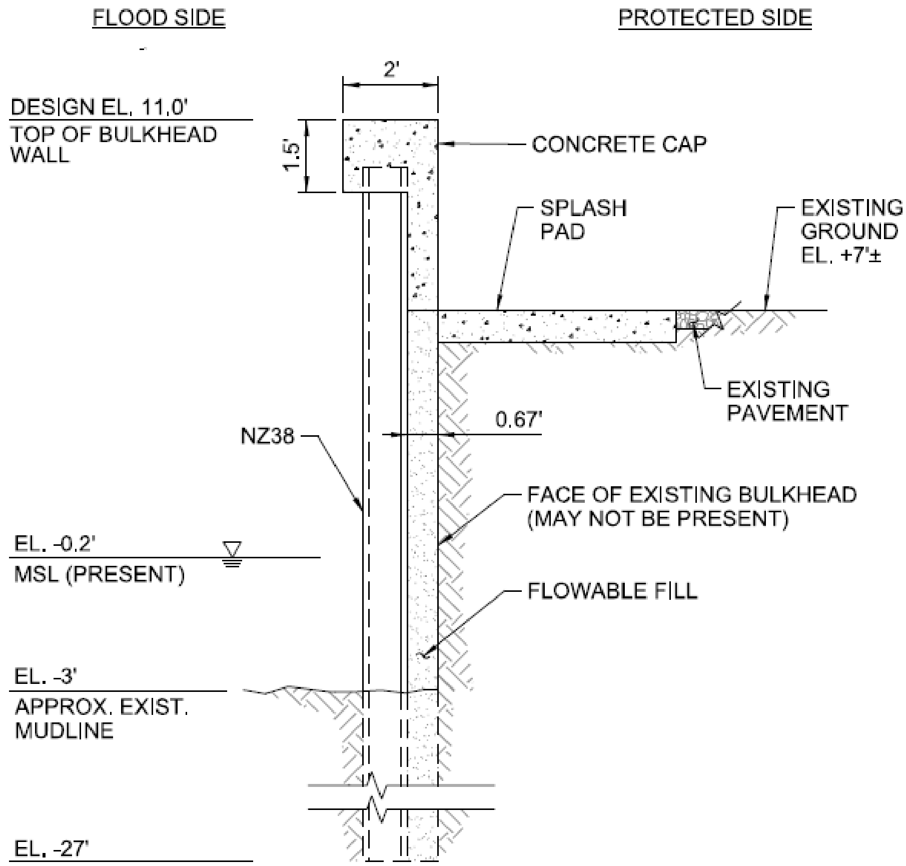


Figure 4-9: Shallow Bulkhead Cross-Section

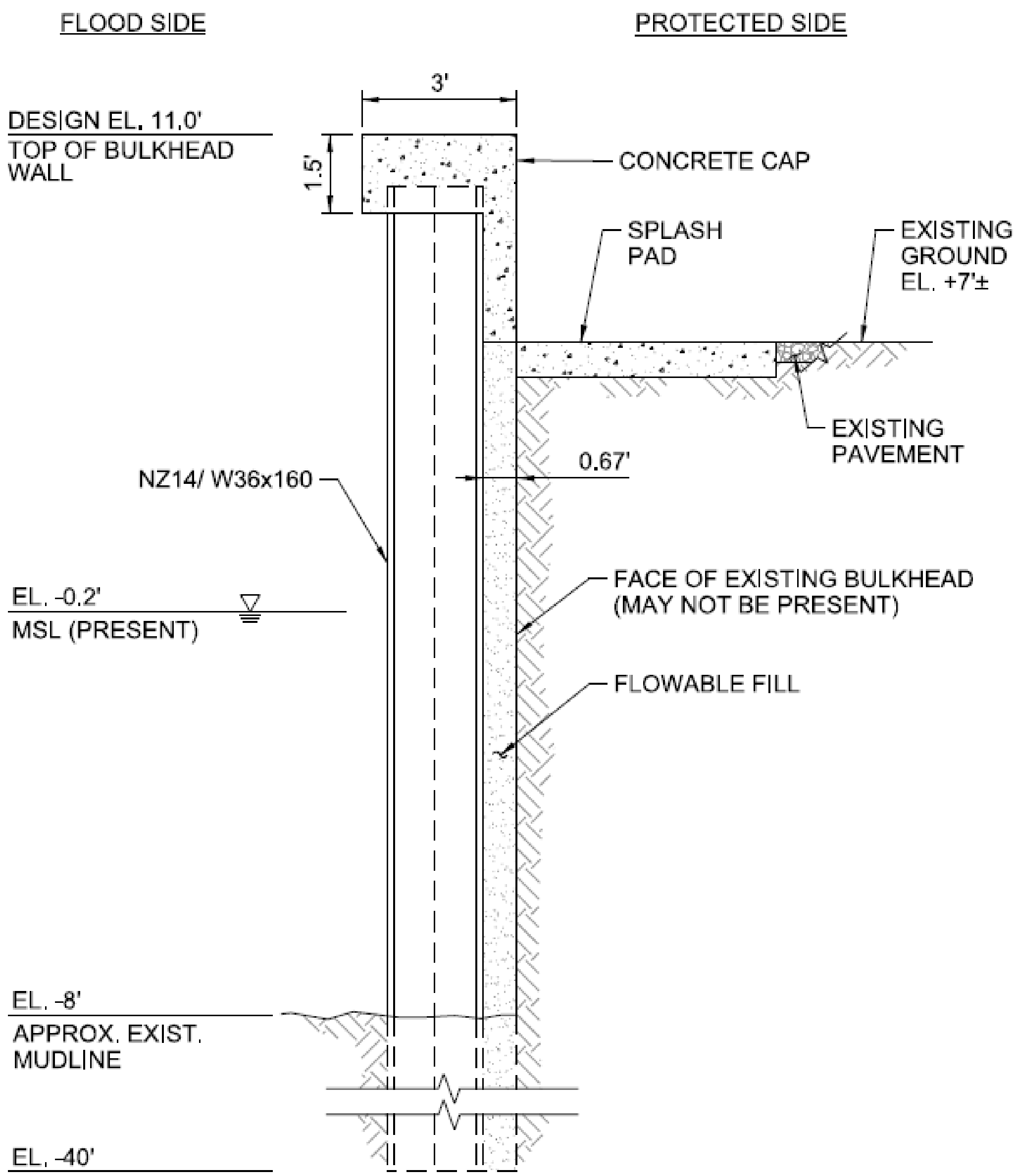


Figure 4-10: Deep Bulkhead Cross-Section

4.4.2 Bulkhead for Urban Application

In some cases, the base cost of bulkheads had to be adjusted to account for complex site conditions at locations where the upland is urban and is heavily developed, and associated complications that would arise during construction. These include a large number of overhanging porches, piles for floating boat docks, stairs, ramps, and various other structures obstructing the bulkhead. The cost of removing these structures and then replacing them with additional structures to allow access during construction, as well as the delay in construction were taken into consideration for bulkheads followed by the suffix "urban application."

4.4.3 Street End Bulkhead

This feature was a special feature type designed to tie into the new Broad Channel bulkheads which have been constructed as part of an unrelated street raising project in Broad Channel (DDC Project ID-HWQ1182A). The bulkheads are newly constructed, and removing them would unnecessarily increase cost; however they are not tall enough to meet the required design elevation. New piles will be driven to the ends of the Broad Channel bulkheads and concrete header beams will be added to raise the top of the structure to meet the design elevation required for this study. Figure 4-11 shows an excerpt from the Broad Channel street raising project plans. Figure 4-12 shows the bulkhead pile-cap extension for the street end tie-ins.

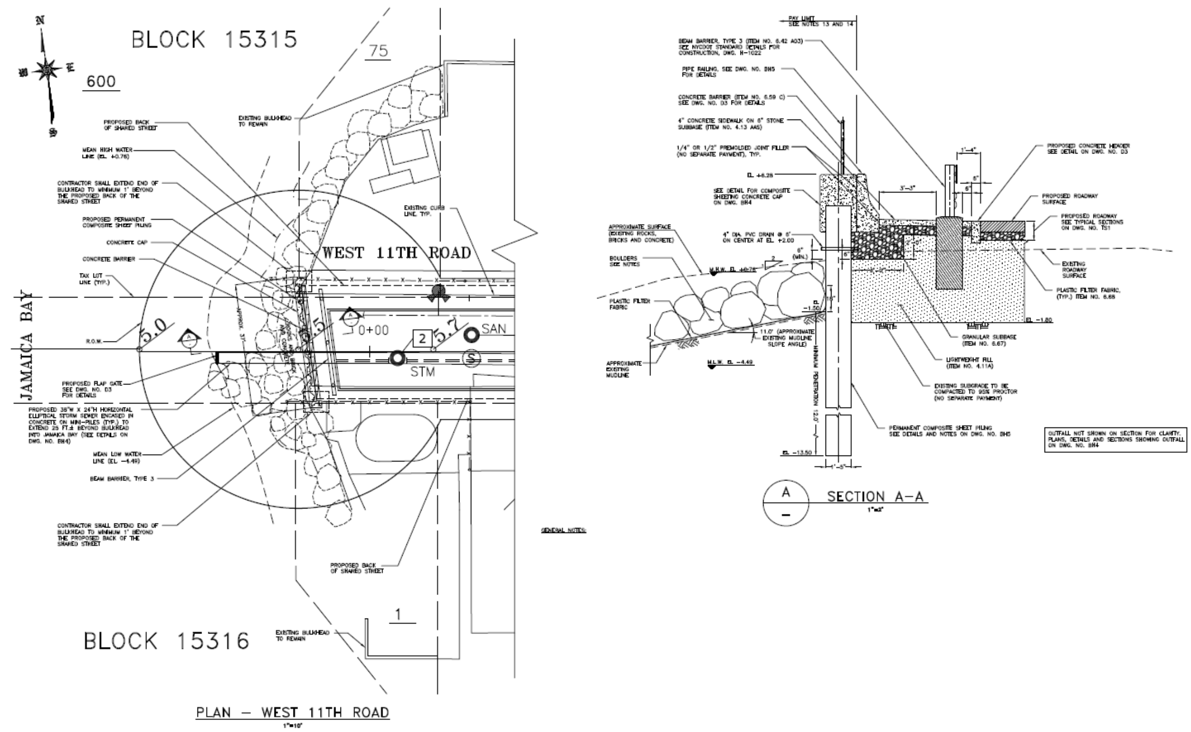


Figure 4-11: Broad Channel Street End Bulkhead Typical Plan and Section (taken from DDC Project ID-HWQ1182A for illustrative purposes only)

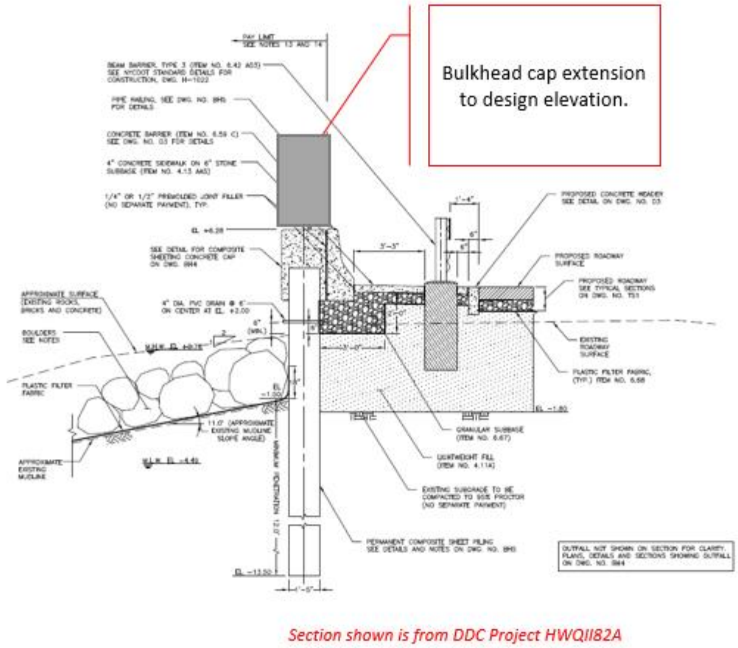


Figure 4-12: Street End Bulkhead Cross-Section (typical section taken from DDC Project ID-HWQ1182A for illustrative purposes only)

4.5 Revetment with Floodwall

Revetments are onshore structures made of erosion resistant material such as stone or concrete. They are typically built to protect the shoreline from erosion. Revetments are comprised of an armor layer, filter layer(s), and toe protection. The armor layer is designed to maintain the revetment's cross-section during wave action. The filter layer supports the armor, and it allows passage of water while retaining the underlying soil. The toe is to provide stability against undermining at the bottom of the structure. Figure 4-13 shows an example of a revetment in Hunter's Point New York City.

Figure 4-13: Revetment Hunters Point, NY (photo credit: Nicole Avella)

The generic revetment geometry is comprised of toe protection, underlayer and rock armor units (i.e. the seaward slope) and a short horizontal crest also comprised of rock. A concrete floodwall supported on steel sheet pile is provided behind the revetment. The cross-section of revetment with floodwall is shown in Figure 4-14.

It was assumed that for a prototypical design, with applications throughout Jamaica Bay, a revetment with 2-foot diameter armor stone, 5-inch diameter underlayer stone, 1.3-foot diameter toe armor stone and a slope of 2 (Horizontal):1(Vertical) would provide sufficient stability. The protective rock armor serves to hold the revetment in place and consist of two layers of rock. The underlayer acts as a drain parallel to the slope to prevent a build-up of water pressure under the armor layer and a filter to prevent the underlying soil from washing out. The two-layer underlayer would be on top of a geotextile. Toe protection is normally an integral part of the revetment structure and was designed to prevent the structural component from undermining as a result of wave and/or current-induced scour. The toe was comprised of two layers of toe armor stone with a width of 3.5 feet. The crest would be 6 feet wide and constructed in front of a floodwall. The floodwall consists of a concrete cap and steel sheet pilings. The top of the floodwall is at El. 9.5 feet and the design existing ground elevation is at El. 6 feet.

One of the more important variables of the revetment design is the seaward side slope which, together with the crest height, is generally dictated by soil conditions and revetment construction methods. For the purposes of this study, it was assumed that the revetment was founded on reasonably good quality soils which would not require foundation/ground improvements. Bottom elevation of the revetment was assumed to be at El. -8 feet. Actual elevations will vary across the study area, but for feasibility level analysis, it was considered a reasonable elevation for the revetment toe along Jamaica Bay shorelines.

The revetment, whereas effective at dissipating wave energy, cannot prevent coastal flooding since it is porous. The impervious concrete floodwall would be installed to prevent flooding. Revetments, especially the ones with stone armor, integrate well with the natural shoreline, their natural look in particular has a high aesthetic appeal.

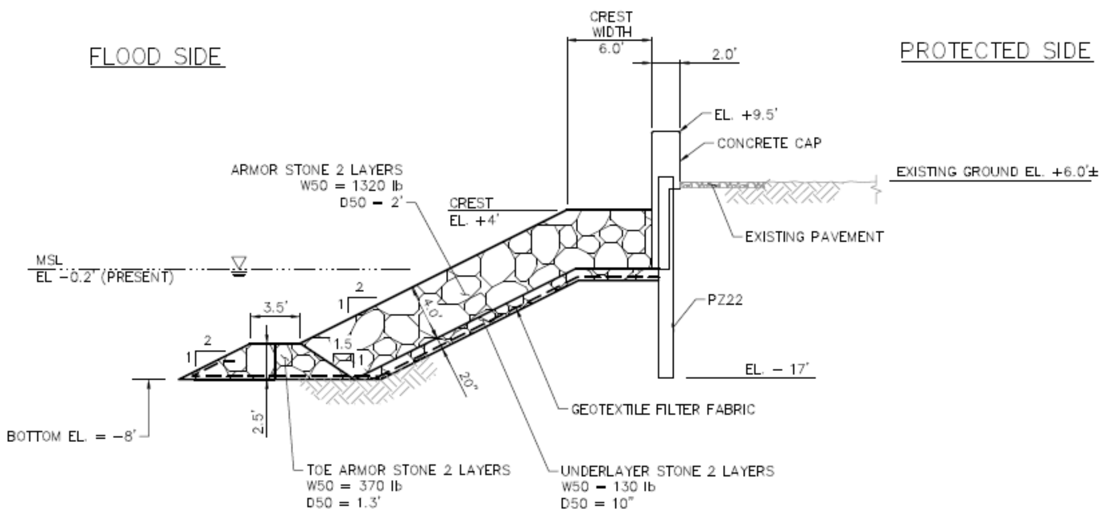


Figure 4-14: Revetment with Floodwall Cross-Section

4.6 Vehicular Gate

Vehicular gates are features added to a line of flood risk reduction, across a road or driveway, which allows for unimpeded access across the line of flood risk reduction during normal day-to-day conditions. Vehicular gates can be either manually or automatically operated. The HFFRRF prototypical vehicular gate is designed to be manually operated. Manually operated gates require operations personnel to physically go to the location of the gate and close it during storm conditions. The gate would then be locked into place to prevent tampering, and access to the flood side of the line of flood risk reduction would be impeded.

In general, swing gates and roller gates were considered initially, both gates have the advantage of simple and quick operation where no special skill or equipment are required. The roller gate was not selected because a level storage area immediately adjacent to the closure opening is necessary for roller gate operation and a (level) track will need to be maintained and inspected. A swing gate was used given the site constraints and upland use of the waterfront area.

4.7 Road Raising

Road raising consists of raising an existing road's surface elevation in order to use the road itself as a berm-like feature, thus reducing the risk of flooding on one side of the road. In order to raise the road surface, any connecting driveway or side street needs to be raised and ramped to meet the raised road. In addition, buried retaining walls are used to support the increased height of the roadway. The various construction activities required to complete the road raising often necessitate relocating and/or raising buried utilities and adding drainage inlets and pipes at the bottom of driveways to convey stormwater away from homes and businesses. Figure 4-15 shows a prototypical section for the road raising feature.

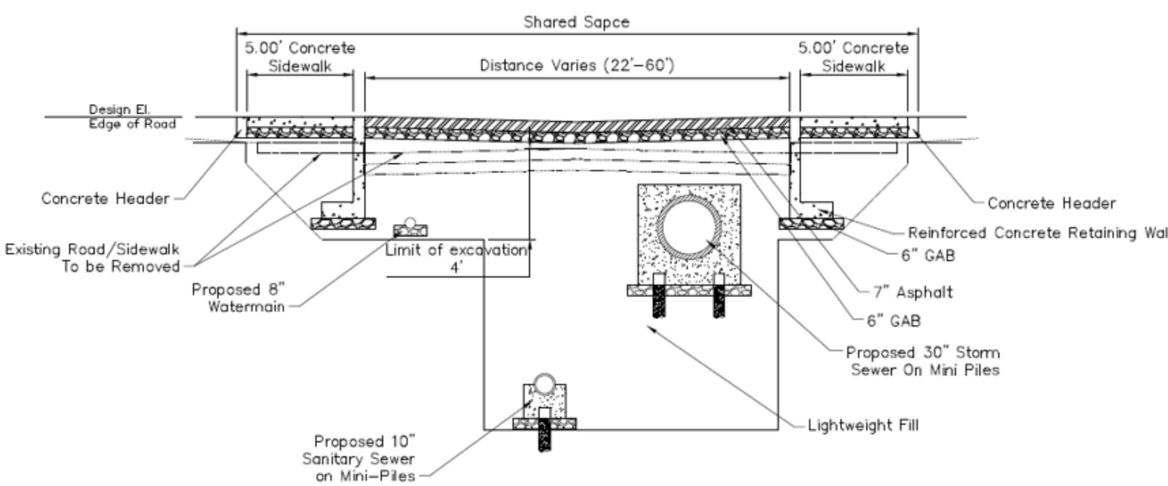


Figure 4-15: Road Raising Cross-Section

4.8 Breakwater

Breakwaters are structures constructed in the water, either offshore or nearshore, that can absorb wave energy and reduce the wave exposure of flood risk reduction features landward of it. It should be noted that the feature was used in this study only for the Broad Channel project. For that location, a rubble mound breakwater with a side slope of 1 vertical on 1.5 horizontal was designed;

the breakwater has two layers of armor stones on top of two layers of underlayer stone. The underlayer stone is in turn placed over a core of smaller bedding stone.

The breakwater will provide habitat to mussels, shellfish and other marine life. Since shellfish thrive in intertidal zones, to maximize the habitat for reef shellfish, the crest was determined to be 20 feet, with a toe width of approximately 127 feet on each side of the breakwater. Figure 4-16 shows a typical cross-section of a breakwater.

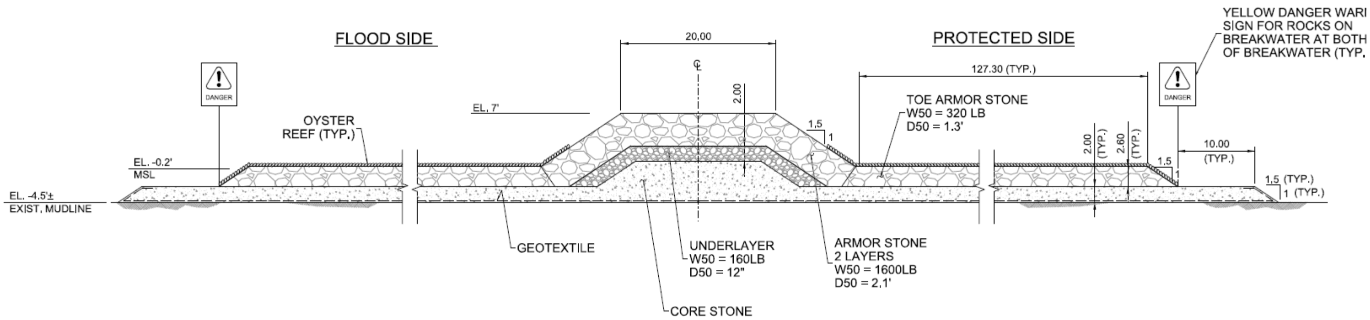


Figure 4-16: Breakwater Cross-Section

4.9 In Water Gates and Related Structures

The designs for in water floodgates and tie-in structures for Head of Bay and Old Howard Beach were developed based on, and assumed to be equal to the preliminary gate design outlined in USACE-NAN (2016). In general, the in water gate structure is comprised of a steel gate, reinforced concrete monolith and pile foundation. The gate monolith would be tied into high ground on each side using tie-in T-walls.

At Head of Bay, a combination of a Sector Gate and Lift Gate was used. The use of a Sector Gate will allow for unrestricted navigation. It is anticipated that both the sector gate and lift gate will remain open for all normal channel operating conditions and closed prior to the arrival of a storm. The top of the gates as well as the associated tie-in structures were determined to be El. 10.3 feet. The sill elevation was assumed to be at El. -20 feet.

At Howard Beach, Sector Gates were used at Shellbank Creek and Hawtree Basin. The operation and design of the two sets of Sector Gate are similar to the one at Head of Bay. However, the top of the gates was set at El. 8 feet while the sill elevation was assumed to be at El. -20 feet.

4.10 Hybrid Berm

As explained in Section 4.3, in general, berms integrate well into the natural landscape but have a relatively large footprint.

In areas where a regular berm as a HFFRRF would be appropriate but a lack of available real estate renders the option impractical, a hybrid berm was used. The hybrid berm has the aesthetic advantage of a regular berm on the protected side, as well as the benefit of a reduced footprint. The hybrid berm is comprised of a berm on the protected side, riprap on the flood side and a vertical steel sheet pile wall in the middle. The steel sheet pile wall is equipped with a reinforced concrete pile cap that runs flush with the top of the berm. Since the sheet pile wall will act against any seepage concerns the impervious core has been replaced with regular earth. The riprap with a slope of 1 vertical on 2 horizontal was used to provide scour protection. Similar to the low, medium and high berms, a protected side berm slope of 1 vertical on 2.5 horizontal was used. The hybrid berm has a crest width of 5 feet, a design height of 3.5 feet and a design existing ground elevation of El. +6'. Figure 4-17 shows the typical cross-section of a hybrid berm.

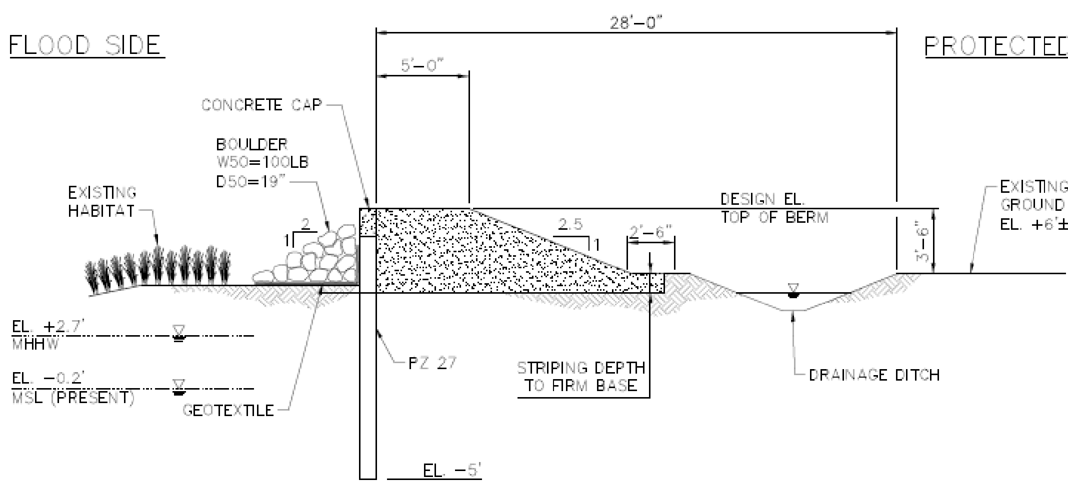


Figure 4-17: Hybrid Berm Cross-Section

4.11 Road Ramp

Road ramps are a means of allowing both vehicular and pedestrian access to the flood side of a line of risk reduction. They consist of two drive lanes and can be designed with sidewalks to allow for safe pedestrian access. The road ramps for this project were designed to be used in conjunction with low floodwalls. In order to allow a vehicle towing a boat to use the ramps without bottoming out, a design length of 85 feet from the bottom of either end of the ramp was used. A culvert would be put under the road ramp to convey drainage to nearby outfalls/pump stations, as needed.

4.12 Natural and Nature Based Features (NNBF)

As described in Section 4.1, a series of Natural and Nature Based Features (NNBFs) were developed as an integral engineering feature to the CSRM design in order to control erosion and reduce wave energy exposure to HFFRRF. Especially for berms, which are expected to be overtopped frequently throughout their project life, NNBFs can improve the overall function and resilience of the HFFRRFs. In addition, they provide a variety of ecosystems services increasing the overall ecological resiliency of the bay. They provide adaptive features, such as improved wetlands habitats and a more natural shoreline that can migrate with rising sea levels in the future. It should be noted that the NNBF design has progressed through the two phases of screening. During Phase 1 generic prototypical NNBF designs were established which would be applicable along Jamaica Bay shorelines, while in Phase 2 a more site specific preliminary NNBF design was established that was informed by the updated site conditions of the screened project and project alignments.

Figure 4-18: Rock Sill and Wetland as NNBF in Brooklyn Bridge Park, Brooklyn, NY

4.12.1 Natural and Nature Based Features (NNBF) – Phase 1

The NNBF design includes placement of a stone toe protection and rock sill structure at or just off the existing shoreline to attenuate wave action and allow tidal marsh to establish between the rock sill and the berm. Sometimes termed "living shoreline," the sill structures provide protection for the subtidal and intertidal habitats as well as providing hard bottom habitat for increased ecological production. The shore slope behind the structure will be regraded to reduce risk of erosion further and create elevation gradients and substrates for establishment of tidal marsh. In addition, the graded habitat behind the structure will be designed to allow the shoreward migration of various habitats with rising sea levels, thereby extending the life of these important ecological systems. The wetland and benthic habitat created by the NNBFs also helps to offset the impacts to habitat from the construction of the HFFRRFs and make the project self-mitigating as a whole. Beyond that, the NNBFs provide numerous additional long-term ecological benefits including provision of various ecosystem services, including carbon and nutrient sequestration, increase productivity by restoration of aquatic and terrestrial wildlife habitats, including primary nursery areas for fisheries, and cultural benefits such as aesthetic benefits to the community. The dynamic character of NNBF's and full ecological benefits are fully described in the ERDC report SR-15-1 (Bridges et al., 2014). The report identifies numerous ecological services, in addition to the structural and erosion protection services provided by breakwater and sill structures.

This type of NNBF also provides opportunity for shellfish and ribbed mussel NNBF habitat, as the rock sills can provide excellent settling and growth spaces for shellfish. Many living shoreline applications are developed with shellfish attractant materials and substrates incorporated for just that reason. The protected subtidal habitats created behind the rock sills also promote the establishment of more productive habitats such as development of seagrasses and other diverse habitats, thereby improving ecological function.

The areas where NNBFs are proposed have erosion problems which have resulted in degraded habitats and degraded coastal storm risk management capacity. The proposed NNBFs are a mechanism for controlling erosion in the system and have the added benefit of providing additional habitat, which allow the project to self-mitigate for other impacts to wetlands caused by the HFFRRF berms and floodwalls. The NNBFs were planned in co-location with the flood risk reduction features in order to take advantage of their capacity to improve the function and resilience of those "grey" features. Located on the unprotected side of the flood risk reduction features, the toe features and bands of tidal wetland vegetation included in the NNBFs will act to dampen incoming waves during storm conditions and potentially reduce wave overtopping. Tidal wetlands also continually adapt to the changing water elevations as sea levels rises. As the sea slowly rises, the lower and intermediate marsh will gradually migrate landward, replacing the scrub shrub habitats, and will continue to provide a protective buffer for the flood risk reduction features over time and increasing the inherent adaptability of the HFFRRF to sea level rise.

The primary factor governing the form of the NNBFs is the amount of horizontal space available between the flood risk reduction feature and the existing shoreline and/or the near-shore shallow littoral shelf. The design will, where possible, re-establish a graded habitat environment which is less susceptible to erosion, more inherently adaptable to sea level rise, and has the added benefit of increased habitat diversity, including valuable maritime forest and intertidal habitats (sub-tidal, smooth cordgrass and salt hay habitats) along shorelines in front of flood risk reduction features where that horizontal space was 200-250 feet or less, the NNBF only includes two stages of Intertidal Marsh vegetation: the low marsh stage, typically dominated by Smooth Cordgrass

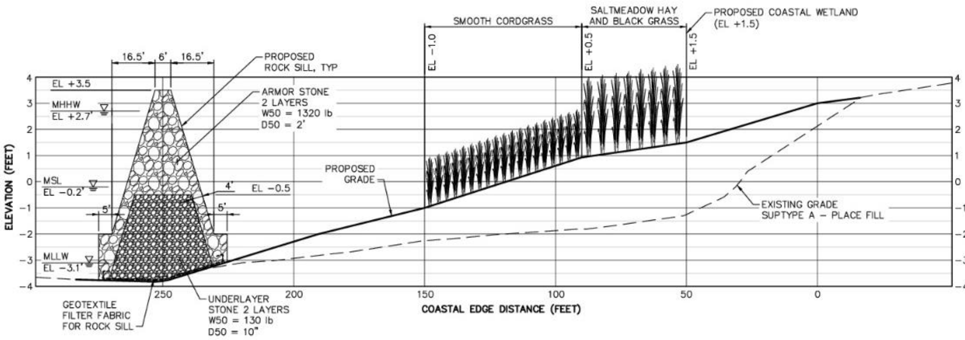
(*Spartina alternaflora*); and the intermediate marsh stage, typically dominated by Salt Meadow Hay (*Spartina patens*) and Black Grass (*Juncus gerardi*). These are referred to as Type 1 NNBFs. Where the horizontal space was more than 250 feet wide, a third stage of marsh was added: High Marsh, which includes shrub species such as Marsh Elder (*Iva frutescens*) and Groundsel Bush (*Baccharis halimifolia*). These are referred to as Type 2 NNBFs.

In some locations too narrow to accommodate the NNBF, the constraint could be overcome by setting the rock toe protection elements of the NNBF slightly off the existing shoreline and placing sufficient fill behind them to achieve the elevations and slope gradients necessary to establish tidal marsh. This strategy could only be utilized if the bathymetry of the existing shoreline exhibited gradual slopes to shallow bottoms just offshore, and it was given preference where aerial photography or field reconnaissance indicated active erosion and/or shoreline retreat from past extents. These NNBFs are referred to as Subtype A.

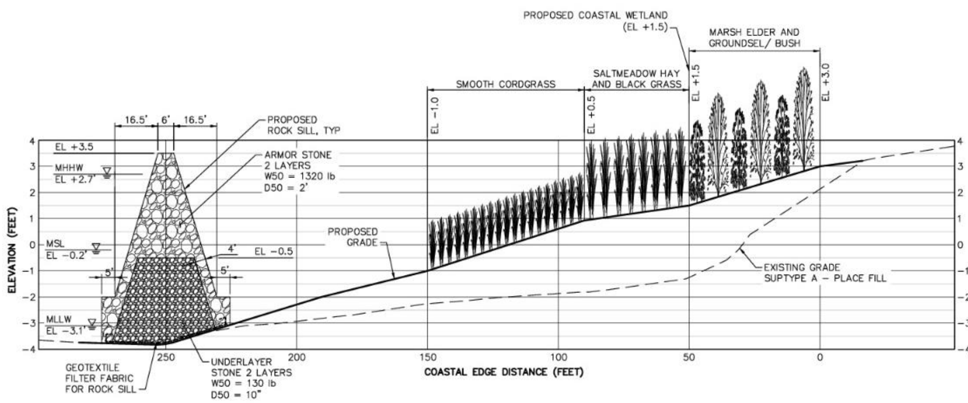
Some existing marsh areas and fringes between the flood control features and the water's edge have been invaded by the non-native invasive, Common Reed (*Phragmites australis*). Along these shorelines it was deemed desirable to excavate landward to remove the invasive and establish appropriate slopes and elevations for restoration of native species marsh. In these instances, preliminary plans call for removal of the top 2.0-2.5 feet of existing soil to eliminate the *Phragmites* rhizomes. This approach was also more conducive where rapidly descending bathymetry made stepping off shore with the slope toe structure impractical. Such NNBFs are referred to as Subtype B.

Combination of these NNBF approaches resulted in four NNBF prototypes used for the Phase 1 analysis as follows (See Figure 4-19):

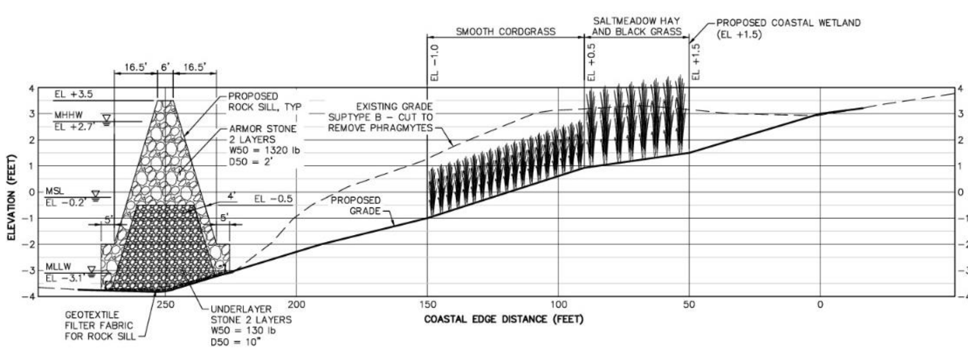
1. NNBF Type 1A – Shoreline Extension with Intertidal Marsh
2. NNBF Type 2A – Shoreline Extension with Intertidal and High Marsh
3. NNBF Type 1B – Shoreline Excavation for *Phragmites* Removal with Intertidal Marsh
4. NNBF Type 2B – Shoreline Excavation for *Phragmites* Removal with Intertidal and High Marsh



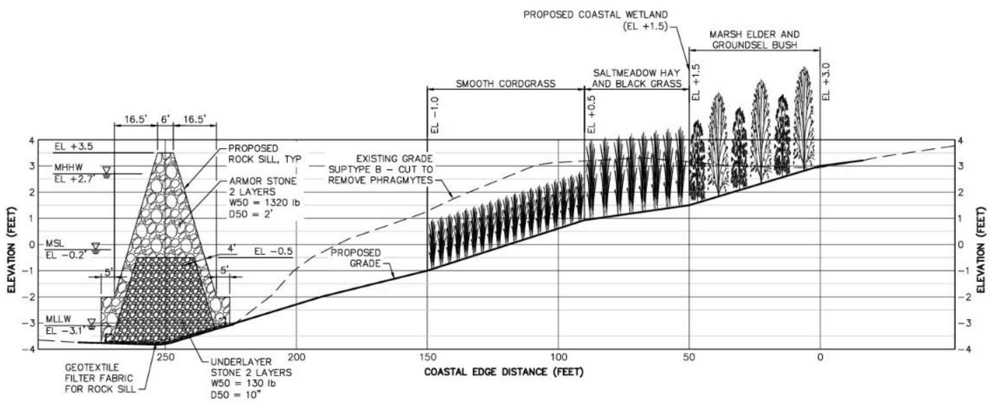
NATURAL AND NATURE BASED FEATURES (NNBF)– TYPE 1A
N.T.S.



NATURAL AND NATURE BASED FEATURES (NNBF)– TYPE 2A
N.T.S.



NATURAL AND NATURE BASED FEATURES (NNBF)– TYPE 1B
N.T.S.



NATURAL AND NATURE BASED FEATURES (NNBF)– TYPE 2B
N.T.S.

Figure 4-19: NNBF Feature Type 1A, 1B, 2A and 2B Cross-Sections

4.12.2 Natural and Nature Based Features (NNBF) – Phase 2

In Phase 2 of the screening, the NNBFs were further refined and co-located with the flood risk reduction features in order to take advantage of their capacity to improve the function and resilience of the structural features.

The refinement of the NNBFs include additional provisions for restoring impacted or eroded shorelines which includes both re-grading of the shoreline and removal of invasive species. Inclusion of features such as the hybrid berm also allowed for additional habitat areas. Regrading for specific sub-habitat types will ecologically engineer the shoreline to provide the maximum habitat diversity in the available space. The plans include the following potential elements (moving shoreward from sill to uplands):

- Placement of a hard substrate (Rock Sill) which will allow encrusting organisms to develop including mussels and other shellfish
- establishment of a low energy subtidal habitat, providing essential nursery habitats conducive to the establishment of submerged aquatic vegetation (SAV)
- Re-establishment of low intertidal marsh habitat, (Common Smoothgrass) especially in shallow eroded intertidal shoreline areas
- Placement and enhancement of intermediate intertidal habitats (Salt Meadow Hay; Black Grass)
- Removal of invasive species (*Phragmites*)
- Development of high marsh (infrequently flooded) dominated by scrub/shrub habitat (Marsh Elder; Groundsel)

The planting plan allows for a re-establishment of a more natural shoreline gradient, using excavation/fill in the appropriate segments, and the establishment of natural marsh elements appropriate for the elevation and tidal regime. The design includes the identification of potential locations where the project team believes these elements will likely fit, but final design and siting will depend on a final feature alignments and detailed delineation of existing grades and elevations.

The final rock sill geometries were refined adaptively, taking into account a more refined bathymetry, adjacent shoreline features, detailed wave environment and where to establish passages for both improved tidal exchange and organism movement. In some cases, the elevation may be adjusted if local wave conditions warrant and/or if aesthetic reasons changes are necessary.

4.13 Drainage Features

As introduced in Section 4.1, a series of drainage features were developed to be used in conjunction with the HFFRRF.

4.13.1 Phase 1 Drainage Features

4.13.1.1 Drainage Considerations

Structures such as the floodwall, bulkhead, revetment and berm have the potential to trap rainfall runoff associated with storms on the landward side, creating an additional flooding hazard. To mitigate this hazard, drainage infrastructure was considered for such projects. Because of the large number and variations of projects in Phase 1 of this feasibility study, a uniform cost per linear foot was developed for two scenarios, based on 1,000-foot hypothetical design lengths. The first scenario was for systems installed where open space is available and flood risk reduction berms could be constructed in conjunction with drainage ditches leading to outlet vaults. The second scenario was for sites with limited space, where hard structures like floodwalls and bulkheads would be constructed along with storm drainage piping leading to outlet vaults. In both cases, it was assumed that the drainage infrastructure would only collect runoff currently flowing to the surface water over land, and that there would be no interaction between the proposed infrastructure and existing drainage infrastructure. The design assumptions are detailed in the following sections.

4.13.1.2 Watershed Delineations

In order to lay out a conceptual storm drainage design that could collect runoff from areas currently draining over land directly to the surface water, the size of the direct drainage areas had to be considered. Approximate watershed boundaries were determined at some sites using LiDAR topographic data in conjunction with Google Earth to determine the locations of any existing storm drainage inlets. It was found that most of the direct drainage watersheds were fairly narrow and that inlet locations would not typically be based upon flow capacity requirements.

4.13.1.3 Inlet Spacing (limited space scenario only)

Since the direct drainage watershed areas were typically long and narrow, inlets were spaced based upon practical considerations, such as maximum practical pipe lengths between access points for maintenance, or for making turns in the pipe system, rather than being controlled by inlet flow collection capacity. For estimating drainage infrastructure costs, the assumption was made that there would be four inlets per thousand feet. At sites where space was available for constructing open channels, collection piping and inlet costs were not included in the cost estimate.

4.13.1.4 Outlet Vault Spacing

Regular discharge points from the drainage system must be incorporated in order to avoid very long pipe or open channel runs and thus excessively deep pipes and structures. The assumption was made that the upstream ends of pipe runs would have a minimum of 1.5 feet of cover. This would result in a 15-inch pipe having an upstream pipe invert depth of approximately 3 feet. Assuming a pipe slope of 0.5 percent, the downstream pipe invert would be 5 feet deep on a 400-foot run and 6 feet deep on a 600-foot run (assuming flat topography). Since a high water table is likely be present in most of the system locations, the assumption was made that drainage piping and open channel runs would be limited to 250 feet in length before reaching an outlet, which would result in a downstream invert elevation a little over 4 feet deep.

4.13.1.5 Pipe Sizing (limited space scenario only)

Collection piping was not sized in detail at this conceptual stage of the project. General assumptions were made about widths of direct drainage watershed areas and watershed impervious percentages based on surveys of the project locations. Using these assumptions, it was estimated that a 15-inch pipe on a 0.5- percent slope would reach flow capacity during the 10% AEP (10-year RP), 5-minute storm peak intensity (crown of the pipe) after collecting runoff from approximately 500 feet of project length. Since the maximum pipe runs until reaching an outlet vault are 250 feet, 15-inch or 18-inch diameter pipes along the runs are expected to suffice. An 18-inch pipe diameter for the full run length was assumed during cost estimating to be conservative and to potentially size the system to route larger storm events. At sites where space was available for constructing open channels, collection piping costs were not included in the cost estimate. Discharge pipes leaving the vaults were assumed to be 24-inch diameter.

4.13.1.6 Vault Design

Vaults were sized based upon assumed invert depths of incoming pipes and the length and width necessary to house a sluice gate that fits a 24-inch by 24-inch opening. The vault was assumed to include two chambers. The first chamber functions as a junction for the incoming pipes, with a single outlet opening to the second chamber (over which the sluice gate is installed). A 24-inch Prestressed Concrete Cylinder Pipe (PCCP) would route stormwater from the second chamber to the downstream surface water. An elastomeric check valve would be installed at the end of each outlet pipe, along with a concrete headwall and riprap energy dissipater. The use of PCCP was assumed in the cost estimates because they come in longer sections, thus fewer joints, which can be beneficial in levee applications where seepage can be a concern. The joints can also be secured to reduce the likelihood of separation from settling or other forces.

4.13.2 Phase 2 Drainage Features

A key characteristic of Phase 2 was the completion of an interior drainage analysis for each of the Phase 2 project locations. The interior drainage analysis and characteristics regarding the drainage features for Phase 2 is described in Sub-Appendix D. The Phase 2 interior drainage analysis resulted in updates to and site specific designs of stormwater management for each of the Phase 2 projects – which then replaced all of the work done in Phase 1, thus replacing and updating the cost estimates for drainage features in Phase 2.

5 PHASE 1 PROJECT SCREENING

5.1 Phase 1 Projects

5.1.1 Introduction

Following the risk assessment presented in Chapter 2, low lying coastal neighborhood areas within Jamaica Bay were identified as areas where HFFRRF could be implemented. The Phase 1 preliminary screening and subsequent feasibility design and analysis was performed only on the areas identified as potentially having viable economically justified stand-alone projects.

Other parts of the Jamaica Bay shoreline that are subject to flooding were not included; such areas are characterized by natural or undeveloped areas or isolated structures. Inclusion of HFFRRF projects for such locations would garner minimal reduction to the overall flood risk within Jamaica Bay and as such only marginally contribute to the overall objective of the project. Examples of such exclusions, amongst others, are geographically much smaller areas with very few assets at risk, undeveloped urban lots adjacent to Jamaica Bay, isolated developed but privately-owned lots with one single owner, and natural shorelines and parklands.

In Phase 1 of the Feasibility Study for HFFRRF for Jamaica Bay, areas for study were identified, analyzed, and screened for feasibility. A general grouping of viable low lying coastal neighborhoods was completed, and the following areas were identified where HFFRRF could be implemented:

1. Mid-Rockaway Jamaica Bay side, which includes Hammels, Arverne, and Edgemere
2. Motts Basin, Norton Basin and the Inwood Marina Area,
3. Head of Bay and the adjoining Nassau County watershed, including Cedarhurst-Lawrence, and Meadowmere,
4. Old Howard Beach,
5. Canarsie, and
6. Broad Channel.

These general areas are graphically presented in Figure 5-1.

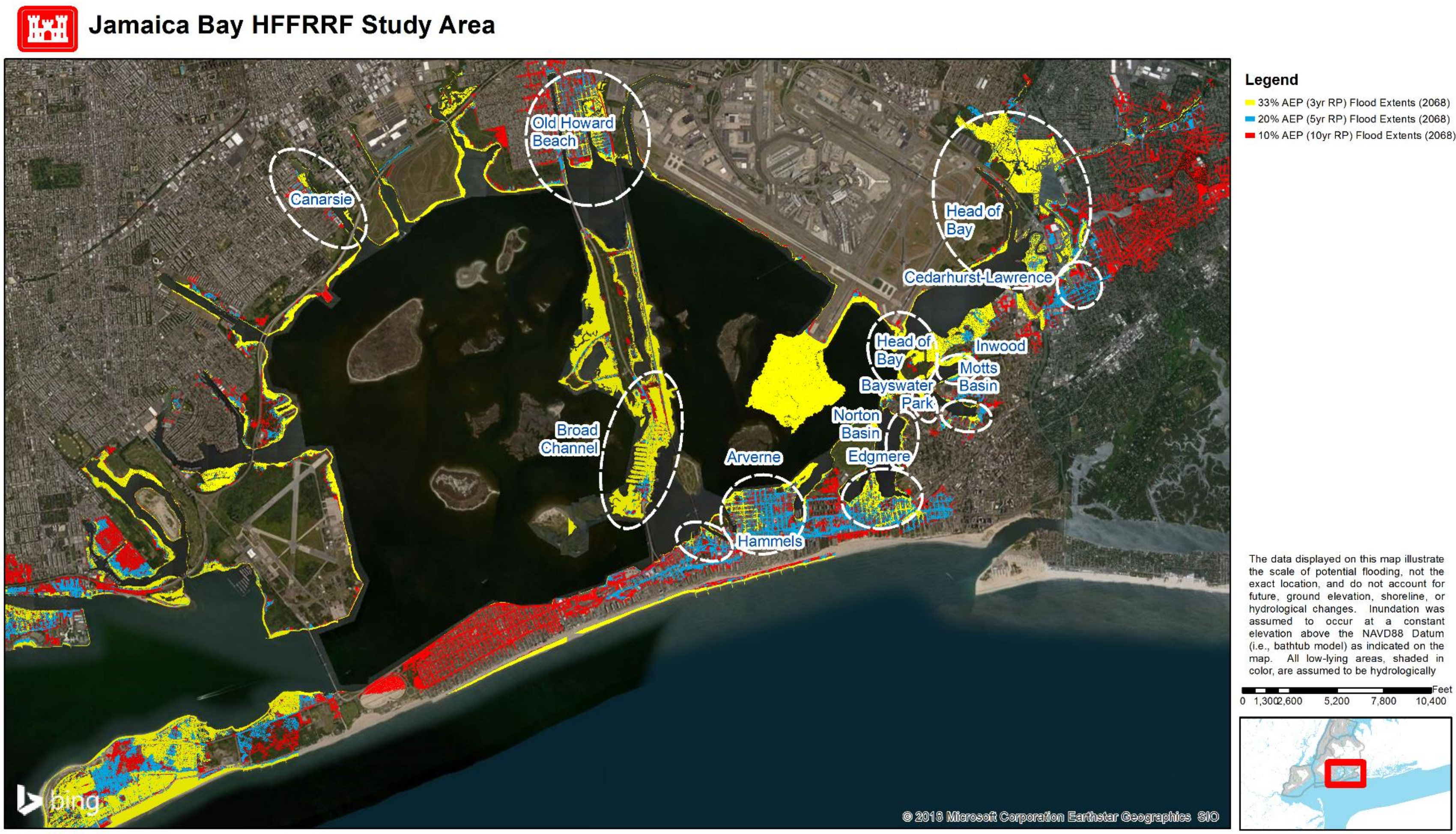


Figure 5-1: Jamaica Bay Inundation Extents for the 20% AEP chance flooding (5-year RP) for the year 2068 and the areas of interest where HFFRRF Projects are developed

For these areas project alignments were defined following the coastal edge while keeping as many assets on the land side of the alignment as practically deemed feasible. Projects were developed by considering realistic project extents, established based on shoreline type, length, topography, neighborhood, land use, planning considerations, project scope, inundation extents, flooding pathway, and existing topography. The goal was to ensure adequate tie in of the HFFRRF projects to higher ground elevations. Project segments were aggregated by the project area they fell within, and project IDs were assigned to each combination of segments. A detailed description of the projects is provided in the next section.

In accordance with USACE's SMART Planning principles, and similar to the development of the project features (HFFRRF), the projects were developed at a level of detail required to make the decision at hand. For Phase 1 this meant that a detailed drainage analysis or an evaluation of the real estate cost was temporarily omitted. This approach allowed the PDT to establish project construction cost and benefits efficiently for a large number of projects and complete a first round of screening based on Phase 1 BCRs. As such, a two-tier screening approach was established where the time consuming and resource intensive interior drainage analysis was only completed for the viable projects that passed on to Phase 2. Through this two-tier screening approach, the time, effort, and expense of doing a more detailed analysis is limited to a smaller number of projects most likely to be included in the Recommended Plan.

5.1.2 HFFRRF Projects for Jamaica Bay

The USACE planning approach supports an integrated approach to reducing coastal risks and increasing human and ecosystem community resilience through a combination of NNBF, non-structural measures and structural measures.

A total of twenty-three (23) HFFRRF projects were delineated and designed. Each project consists of a single or multiple alignment(s), which in turn consist of one single or multiple HFFRRF (s). For ease of reference numerical project IDs were assigned to each project's combination of HFFRRFs. A total of seventeen (17) HFFRRF projects were defined where the design would only include prototypical structural measures (i.e. HFFRRF as described in section 4.2 through 4.11). These are projects 1 through 17, and an overview of these projects is provided in the Table 5-1 below. Maps that display the alignments are included in Sub-Appendix A. An additional six (6) projects were designed for those areas where the structural measures could be integrated with NNBFs. These projects were given the numeric project IDs of 102 through 107.

Not all projects included co-located NNBFs because many of the project locations were not suitable for development of such applications. In order to plan the NNBFs, the site of each flood risk reduction feature was evaluated for the presence and quality of existing sensitive habitats (e.g. tidal marshes, maritime forests, submerged aquatic vegetation). Those sites, or portions of sites, where healthy wetlands and maritime forests already exist did not require constructed NNBF to perform their CSRM functions, and there was thus no need to cause impacts to the existing high quality habitat. Shorelines where the NNBF conflicted with existing anthropogenic infrastructure, such as docks and marina facilities, were also avoided. NNBFs were also deemed infeasible on shorelines where the localized bathymetry descended rapidly to deep water, or the amount of horizontal space between the flood control feature and the descent to deep water was too narrow to create sufficient marsh to make the NNBF effective and sustainable.

Conversely, shorelines with suitable bathymetry, available horizontal space, and limited existing infrastructure, were deemed suitable for the NNBFs to be included in the HFFRRF design. With regard to habitats, the NNBFs were deliberately targeted to locations where erosion is a concern as evidenced by the fact that wetland habitats have been lost or degraded. In actively eroding shorelines and tidal marshes where the native species have been displaced by invasive *Phragmites*, sometimes known as Common Reed, vegetation management will be necessary since the root structures of *Phragmites* are thick and extensive and could potentially compromise an adjacent berm. Therefore, for those NNBF-suitable locations where *Phragmites* has invaded, the NNBFs were planned to include excavation of the top layers of soil where the *Phragmites* rhizomes exist, and replacement with clean soil and re-establishment of native wetland vegetation species.

It should also be noted that the living shoreline applications represented by these NNBFs represent excellent opportunities to integrate hard-bottom or reef restoration efforts, as oyster reefs, along with other bivalves, to help to manage coastal storm risk. Oyster restoration has been an ongoing effort in Jamaica Bay, which had been one of the most abundant and valuable oyster fisheries in the region up until the 1920s. Recent oyster restoration efforts have been exerted in Jamaica Bay under the Billion Oyster Project (2018), but that project has experienced mixed success. Zarnoch and Schreibman (2012) pointed to challenges including very limited availability of suitable substrate for oyster attachment and growth, and lack of sufficient densities of adult oysters to produce larval stock. However, their study also indicated that water quality was not preventing oyster growth and maturation in the Bay. Oysters (and coral reefs) provide CSRM benefits such as breaking of waves, attenuation of wave energy and slowing inland water transfer (Bridges et al. 2014). In that light, the NNBFs set forth herein, with their stone toe protection/breakwater elements, could represent excellent opportunities to introduce important suitable shellfish reef substrate into the shorelines of Jamaica Bay. The stone rock sill elements could be constructed of material conducive to shellfish attachment and growth and could be covered with a veneer of bagged shellfish shells, or other proprietary shellfish attractant surfaces to help establish a productive reef. In any case, the addition of suitable hard substrate in the littoral areas of Jamaica Bay will provide an excellent resource as a source of perennial shellfish larvae for the bay as a whole.

Additionally, there is an opportunity to place ribbed mussels in these areas, similar to the projects initiated by the New York DEP in 2011, which provided hard structures for ribbed mussel attachment (DEP, 2016). Along with oysters and other shellfish, ribbed mussels provide valuable filtration services, improving water quality in the bay. They also form the basis of a complex estuarine food web and they increase overall secondary productivity, support a broader and healthier estuarine community thereby supporting several fisheries. By including both oysters and ribbed mussels as target restoration species, there is a higher likelihood that at least one will successfully recruit and reproduce on their own.

Details on the HFFRRF Projects that are inclusive of NNBFs are provided in Table 5-1, and maps displaying the alignments of these projects and the locations for NNBFs are included in Sub-Appendix A. It should be noted that all structural HFFRRF for these projects are identical to the equally named, but differently numbered, projects that include NNBFs; the only difference is the addition of the NNBFs.

Table 5-1: HFFRRF Phase 1 Projects

ID	Project Name (Borough or County)	Notes	NNBF Type*	Length [ft]
1	Hammels (Queens)	This project consists of a series of floodwalls and is set back from the coastline to minimize conflicts with existing waterfront facilities. A total of six (6) vehicular gates are included to maintain access to the waterfront.[5]		3,100
2	Arverne (Queens)	Project follows the coastline of the Arverne peninsula and includes a total of 11 HFFRRF segments to suit the changing conditions in landscape and land use along the proposed project. Four (4) vehicular gates are included to maintain access to the waterfront industrial sites and marina.[6]		12,300
102	Arverne with NNBF	The Arverne project is enhanced with NNBFs at three locations.		12,300
		1) The north-west corner of the peninsula (Brant Point). Existing habitat include mud flats, high marsh and invasive marsh (*Phragmites*). The proposed NNBF would employ the installation of rock sills off the existing, eroding shoreline to protect the toe of the slope and dampen incoming waves so the existing shoreline could be regraded and potentially extended seaward. The proposed NNBF also includes the removal of the	2A	

[5] This was a conservative estimate and in Phase 2 all but one vehicular gate were switched to road ramps.

[6] This was a conservative estimate and in Phase 2 all but one vehicular gate were switched to road ramps.

ID	Project Name (Borough or County)	Notes	NNBF Type*	Length [ft]
		Phragmites and expansion/restoration of the intertidal wetland habitat and high marsh. The existing upland maritime forest between the berm feature and the wetlands are to remain undisturbed.		
		2) At the north-east corner of the peninsula where there is currently a narrow beach (DuBois Point), between Beach 69th and just east of Beach 65th Street a NNBF is proposed that includes the construction of rock sills to create an intertidal flat. Further upslope and to the east intertidal marsh can be restored. The existing upland maritime forest is to remain undisturbed.	1A	
		3) To the east of Marina 59, much of the existing marsh along the shoreline is dominated by *Phragmites*. The proposed NNBF includes restoration of an intertidal flat, supported by rock sills, and excavation of the *Phragmites* such that intertidal and high marsh can be restored.	2B	
3	Edgemere (Queens)	Project consists out of two approx. 3,000 foot segments, i.e. a medium floodwall for the west side and a high berm on the east side of the peninsula. One vehicular gate is included to maintain access to the waterfront.		6,300
103	Edgemere with NNBF	On the east side of the Edgemere neighborhood the proposed NNBF would restore and further enhance existing wetland habitat. A large area of wetland habitat is proposed to be restored and created between the HFFRRF high berm and the newly constructed rock sill, just off of the existing coastline. The proposed NNBF includes the removal of the *Phragmites* and restoration of the intertidal habitat and high marsh such that both type 2A and type 2B are implemented.	2A/2B	6,300
4	Norton Basin (Queens)	Project follows the coastal edge of Norton Drive and consist of approximately 2,400 foot segment of floodwall.		2,400
104	Norton Basin with NNBF	At Norton Basin the proposed NNBF includes creation/restoration of the intertidal habitat and high marsh adjacent to Norton Drive. The wetland habitat and appropriate grades along the extended shoreline would be supported by the construction of a rock sill in the water side	1A/2A	2,400

ID	Project Name (Borough or County)	Notes	NNBF Type*	Length [ft]
		and construction of a medium floodwall on the landward side.		
5	Bayswater Park (Queens)	This project consists of an approximately 1,400 foot long berm that follows the coastal edge.		1,500
105	Bayswater Park With NNBF	Although currently healthy upland maritime forest exists at this location, there is an opportunity to enhance the HFFRRF with the creation of additional wetland habitat. The proposed NNBF includes the removal of *Phragmites* and restoration of the intertidal marsh. A rock sill would be constructed to support an extension of the existing shoreline. The existing upland maritime forest is to remain undisturbed.	1A/1B	1,500
6	Motts Basin S (Queens)	Project follows the southern perimeter of Motts Basin residential neighborhood as well as the low lying coastline of the Long Island Power Authority substation and the Inwood material terminal.		3,800
106	Motts Basin S With NNBF	For Motts Basin South an opportunity exists to enhance wetland habitat between Dickens Street and Pinson Street on the water side of the proposed HFFRRF. In the horizontal direction there are few constraints at this location, and the proposed NNBF includes extension of the shoreline and restoration of the intertidal and high marsh.	2A	3,800
7	Motts Basin N (Nassau County)	Project follows a short section of roadway (Waterfront Blvd.) in Nassau County set back from the northern perimeter of Motts Basin. Construction of a low floodwall would reduce the risk of coastal flooding of residential and commercial parcels on the north side of Motts Basin		700
107	Motts Basin N With NNBF	Similarly as with Motts Basin South, good conditions exist to enhance the HFFRR-Feature with NNBFs and habitat restoration. The proposed NNBF includes extension of the shoreline and restoration of the intertidal and high marsh.	1A /2A	700
8	Inwood Marina (Nassau County)	This project provides flood risk reduction to the residential neighborhood to the east of the		2,700

ID	Project Name (Borough or County)	Notes	NNBF Type*	Length [ft]
		Inwood Marina. Two (2) vehicular gates are included to maintain waterfront access.		
9	Head of Bay Gate (Queens / Nassau County)	Equal to the alternative as proposed in Appendix A2 of the GRR, a storm surge barrier at this location would provide flood risk reduction for low lying coastal areas at the far eastern extent of Jamaica Bay and along adjoining waterbodies in Nassau County. A barrier at this single location could reduce risk for the extensive area for eastern end of Jamaica Bay. (If this barrier is deemed not viable, additional smaller projects provide options for isolated areas, as included in project 12 through 16 below.)		3,000
10	Old Howard Beach (Queens)	Similar to the alternative as proposed in Appendix A2 of the GRR, storm surge barriers at both Shell Bank Creek and Hawtree Basin and connecting HFFRRF to tie the alignment in to higher ground would provide flood risk reduction for the Howard Beach area.		3,700
11	Canarsie (Brooklyn)	This project includes flood risk reduction features along Fresh Creek such that the lowest portions of the shoreline would be elevated. Revetments would be placed where revetments currently exist. A floodwall is proposed to be constructed along a portion of E 108th Street.		2,700
12	Cedarhurst-Lawrence (Nassau County)	This project follows a section of the Nassau Expressway with a low floodwall for approximately 1,100 feet.[7] The project also includes two sections of bulkhead on either side of the canal next to the Lawrence High School. A short section of floodwall connects the bulkhead on the west side to high ground.		1,800
13	Meadowmere (Queens)	The Meadowmere alignment consists of a 3,700 foot length of bulkhead around the northern end of the Meadowmere Park Island. A low berm (650 ft) on the west side and a floodwall (1300) on the east side connect the		6,700

[7] This section was later removed as an unrelated road raising project will provide a flood barrier in this area.

ID	Project Name (Borough or County)	Notes	NNBF Type*	Length [ft]
		bulkhead to a 1,000 foot long revetment on the southern end of the island.		
14	Meadowmere N (Queens)	This project provides flood risk reduction to the residential neighborhoods along Bayview Avenue and Broad Street in the Meadowmere area. The alignment consists a 1,000 foot berm (700 foot high and 300 foot low berm) and approximately a 3,700 foot bulkhead.		4,800
15	Meadowmere E (Queens)	This project consists of a 1,600 foot bulkhead around the peninsula parallel with 1st and 3rd Streets off of Rockaway Boulevard.		1,600
16	Rosedale (Queens)	The Rosedale project is a 1,900 floodwall that connects Brookville Boulevard in the south and high ground further north.		1,900
17	Broad Channel (Queens)	This project consists of urban bulkheads and road raisings on the west side of the island and berms and road raisings on the east side. There are also berms around two parks, at the northern and southern ends of the island. The project also has a 1,600 foot breakwater off of the west side of the island.		28,700

***NNBF TYPES ARE AS FOLLOWS PER DESCRIPTION IN SECTION 4.12.1**

NNBF TYPE 1A – SHORELINE EXTENSION WITH INTERTIDAL MARSH
NNBF TYPE 1B – SHORELINE EXTENSION WITH INTERTIDAL AND HIGH MARSH
NNBF TYPE 2A – SHORELINE EXCAVATION FOR *PHRAGMITES* REMOVAL WITH INTERTIDAL MARSH
NNBF TYPE 2B – SHORELINE EXCAVATION FOR *PHRAGMITES* REMOVAL WITH INTERTIDAL AND HIGH MARSH

5.2 Phase 1 Project Costs

Project costs were estimated following the completion of the conceptual design for each project inclusive of structural features and NNBFs. Project cost include the construction cost, Pre-construction Engineering and Design (PED), construction administration, Operations and Maintenance (O&M) cost and contingencies. The details with respect to establishing the total project cost estimates for Phase 1 are documented within Sub-Appendix C. Table 5-2 below provides an overview of the project costs for the Phase 1 projects. As noted previously, the project cost developed for Phase 1 did not include real estate cost or mitigation cost.

Table 5-2: Total Preliminary Project Cost for All Projects without and with inclusion if Natural and Nature Based Features—used for Preliminary Screening

Project ID	Project Name	Perimeter Length (ft)	Total Project Cost (Q4 2017 price level)*
1	Hammels	3,100	$17.2 M
2	Arverne	12,300	$58.1 M
102	Arverne with NNBF	12,300	$69.6 M
3	Edgemere	6,300	$25.8 M
103	Edgemere with NNBF	6,300	$34.2 M
4	Norton Basin	2,400	$13 M
104	Norton Basin with NNBF	2,400	$20.7 M
5	Bayswater Park	1,500	$1.3 M
105	Bayswater Park with NNBF	1,500	$5.2 M
6	Motts Basin South (S)	3,800	$21.9 M
106	Motts Basin South with NNBF	3,800	$25.8 M
7	Motts Basin North (N)	700	$1.7 M
107	Motts Basin North with NNBF	700	$5.9 M
8	Inwood Marina	2,700	$13.1 M
9	Head of Bay Gate	3,000	$787.9 M
10	Old Howard Beach	3,700	$259.4 M
11	Canarsie	2,700	$8.4 M
12	Cedarhurst-Lawrence	1,800	$8.4 M
13	Meadowmere	6,700	$44.3 M
14	Meadowmere North (N)	4,800	$34.8 M
15	Meadowmere East (E)	1,600	$14.1 M
16	Rosedale	1,900	$10.3 M
17	Broad Channel	28,700	$287.8 M

*Does not include real estate nor mitigation costs

5.3 Preliminary Screening of the HFFRRF Project Alternatives

In support of the first round of screening of the HFFRRF projects the economic benefits were analyzed. Benefits modeling is detailed in the Economics Appendix (Appendix B), the Benefit Cost Ratio (BCR) of each of the alternatives was calculated and the project characteristics were tabulated to facilitate screening. Screening results are presented in Table 5-3.

All projects where NNBFs had been identified were included in the preliminary screening with NNBFs, except for Motts Basin North. For Motts Basin North further analysis of the existing habitat showed high quality mudflats, with mussel reefs. This is an existing NNBF and conversion to intertidal marsh would negatively impact the existing habitat with a habitat transfer. As such, the NNBF part of the Motts Basin North design was screened out.

For more than half of the Phase 1 projects the BCR was well below unity, i.e. Bayswater, Norton Basin, Motts Basin South, Inwood Marina, Head of Bay Gate, Meadowmere, Meadowmere North, Meadowmere East, Rosedale and Broad Channel. Without consideration of real estate cost the annualized costs exceed the benefits, and as a result these projects were screened out and not carried forward for further analysis. Conversely, Canarsie, Cedarhurst-Lawrence, Hammels, Arverne and Edgemere have BCRs above unity, resulting in a positive screening outcome.

Finally, the Old Howard Beach project has a positive BCR (1.0) but its total project costs are estimated to exceed the $259 Million shown in Table 5-2. First, the calculated BCR is currently at unity while a key assumption of the Phase 1 screening is that real estate cost and a detailed interior drainage analysis have not yet been included. Hence, if this project would advance to the second phase of analysis, the project costs are likely to increase, and the BCR would decrease below unity. Secondly, given that the storm surge barrier for Jamaica Bay is still recommended for further study – and the HFFRRF Feasibility Study's goal is to reduce flood risk in anticipation of the construction of a storm surge barrier and thereby reduce the need for frequent operation (and as such reduce storm surge barrier O&M cost) – it does not seem to be supportable to invest more than $260 M in this second large civil works project that would incur its own expensive O&M.

Table 5-3: Phase 1 Screening Results. Benefits and Costs in 1,000 of Dollars

Project	Without Project EAD[8]	With Project EAD	Annual Benefits	Total Project Cost	Annual Cost	Net Benefits	BCR	Passed (Y/N)	Reason for Screening Out	# of Struc-tures
Canarsie	$5,245	$4,001	$1,244	$8,403	$367	$877	3.4	YES		222
Hammels	$6,921	$5,358	$1,563	$17,215	$733	$830	2.1	YES		88
Arverne with NNBF	$23,613	$17,525	$6,088	$69,616	$2,899	$3,189	2.1	YES		715
Motts Basin North	$709	$572	$137	$1,707	$77	$60	1.8	YES		18
Edgemere with NNBF	$13,733	$12,298	$1,435	$34,204	$1,408	$27	1.0	YES	Best buy is with NNBF	702
Old Howard Beach	$32,578	$21,686	$10,892	$259,395	$10,719	$173	1.0	NO	Total cost negates objective	986
Bayswater with NNBF	$312	$296	$16	$5,239	$225	-$209	0.1	NO	BCR <1	9
Norton Basin With NNBF	$458	$429	$29	$20,703	$828	-$799	0.0	NO	BCR <1	19
Motts Basin South with NNBF	$2,510	$2,229	$281	$25,826	$1,055	-$774	0.3	NO	BCR <1	118
Inwood Marina	$1,689	$1,346	$343	$13,059	$553	-$210	0.6	NO	BCR <1	60
Head of Bay Gate	$115,378	$100,956	$14,422	$787,940	$32,423	-$18,001	0.4	NO	BCR <1	1,368
Cedarhurst-Lawrence	$12,649	$9,713	$2,936	$8,401	$352	$2,584	8.3	YES		128
Meadowmere	$2,726	$2,203	$523	$44,330	$1,814	-$1,291	0.3	NO	BCR <1	99
Meadowmere N	$6,917	$6,338	$579	$34,841	$1,399	-$820	0.4	NO	BCR <1	38

[8] Equivalent Annual Damage: This is the annualized damage accounting for changes in expected damage over time – in this case due to sea level change between the base year and the final year of the analysis period.

Project	Without Project EAD[8]	With Project EAD	Annual Benefits	Total Project Cost	Annual Cost	Net Benefits	BCR	Passed (Y/N)	Reason for Screening Out	# of Struc-tures
Meadowmere E	$682	$358	$324	$14,135	$565	-$241	0.6	NO	BCR <1	25
Rosedale	$978	$630	$348	$10,316	$423	-$75	0.8	NO	BCR <1	104
Broad Channel	$11,204	$7,967	$3,237	$287,842	$10,622	-$7,385	0.3	NO	BCR <1	764

5.4 Phase 1 Project Screening Synopsis

During the first phase of screening process in support of the feasibility study for HFFRRF Projects for Jamaica Bay, an initial screening effort was undertaken to identify potentially feasible projects. From the twenty-three (23) projects analyzed a select set of six (6) HFFRRF projects were carried forward into the Phase 2 analysis. The Hammels, Arverne and Edgemere projects were combined going into Phase 2 because all three projects passed the initial screening and their geographic proximity and adjoining storm water sewer sheds justifies aggregation of these project areas into one larger project area. These three projects aggregated and the project was renamed to "Mid-Rockaway Jamaica Bay with NNBFs."

6 PHASE 2 PROJECT SCREENING

6.1 Introduction

Upon completion of the Phase 1 screening as presented in Chapter 5, the second Phase of the two-tier screening approach was initiated. Phase 2 included a more time consuming and resource intensive interior drainage analysis for the smaller number of viable projects passing out of Phase 1. As a result of Phase 1 the following low lying coastal neighborhood areas were identified as areas where High Frequency Flood Risk Reduction Features (HFFRRF) could be implemented and further refinement was warranted to complete the feasibility level study:

1. Mid-Rockaway Backbay with NNBFs, which includes Hammels, Arverne, and Edgemere
2. Motts Basin North
3. Canarsie
4. Cedarhurst-Lawrence

These four project locations are graphically presented in Figure 6-1.

Additional analyses were completed in this second phase to progressively converge to higher level of detail after completion of the preliminary screening documented within the previous chapter. The second phase of the screening included:

- An analysis of existing drainage infrastructure and an analysis of impacts to the existing drainage system as a result of the construction of HFFRRF Projects

- A cost estimate to account for modifications to the existing drainage infrastructure and/or construction of new drainage infrastructure as part of the HFFRRF projects

- Analysis of wave-height for the project areas and establishing the required freeboard for the features

- A more detailed analysis of the potential impacts to wetland habitat and a more detailed analysis of the NNBF designs that are part of the screened HFFRRF Projects

Jamaica Bay HFFRRF Screening Results

Figure 6-1: Jamaica Bay Inundation Extents for the 20% AEP chance flooding (5-year RP) for the year 2068 and the four (4) project areas for Phase 2 analysis.

6.2 HFFRRF Phase 2 Projects

6.2.1 Phase 2 Project Refinement and Interior Drainage

A total of four (4) HFFRRF projects were delineated and refined during the second phase of the HFFRRF feasibility study. Each project consists of a single or multiple alignment(s), which in turn consist of one single or multiple HFFRRF. During Phase 2 the alignments and feature types of the projects were updated and modified where needed as a result of ongoing refinement of the designs. The most notable difference compared to the Phase 1 screening was the completion of the interior drainage analysis (documented in Sub-Appendix D) for the Phase 2 projects.

As stated in U.S. Army Corps of Engineers EM 1110-2-1413, "Hydrologic Analysis of Interior Areas", the design Minimum Facility should provide interior flood relief such that during low exterior stages (at gravity conditions for normal astronomic tide) the local storm drainage system (typical 10-year design storm) functions essentially as it would without the Coastal Storm Risk Management System in place.

The Minimum Facility is intended to ensure that the existing drainage system performs the same with and without the project put in place as to avoid induced flood damages. This is the starting point from which all additional interior drainage alternatives can be evaluated. Additional interior drainage measures may be designed to further reduce interior water levels beyond the Minimum Facility. These additional interior facilities must be incrementally justified. For each project area and within each interior drainage subbasin, the economics for a series of alternative interior drainage measures were evaluated to determine the alternative providing the highest level of net benefits to the individual project areas. Sub-Appendix E includes the Interior Drainage analysis.

6.2.2 Phase 2 Project Descriptions

Additional refinements to the project designs were a result of many factors, including but not limited to Non-Federal sponsor feedback, adjustments to accommodate interior drainage and storage capacity considerations, and minimization of impacts to wetland habitat. As detailed in Chapter 4, specific features such as the Hybrid Berm and the Road Ramp were developed to allow for project design refinement and improve overall completeness and acceptability of each project. In addition, the NNBF designs moved away from the more generic prototypical application from Phase 1 and towards site specific NNBF designs in Phase 2 (see also section 4.12). Furthermore, wave-heights were assessed in more detail on a site-by-site basis, which allowed for the refinement of project elevations as well as an update to the rock sill design which is part of the NNBF. The Phase 2 wave modeling is documented in Sub-Appendix D.

Brief descriptive overviews of the Phase 2 refined projects are provided in Table 6-1. Maps displaying the project alignments and extents are included in Sub-Appendix B. A detailed overview of the pump station capacities, new outfalls and outfall modifications is included within Sub-Appendix E and a summary is provided in Table 6-2.

Table 6-1: HFFRRF Phase 2 Projects

ID	Project Name (Borough or County)	Notes	Length [ft]
1	Mid-Rockaway Backbay with NNBFs (Queens)	This project area includes the Edgemere, Arverne, and Hammels project alignments from Phase 1. Of the previously used eleven (11) vehicular gates all but one has been replaced with road ramps.	22,700
	Hammels	The Mid-Rockaway Backbay HFFRRF project alignment that consists out of approximately 2,550 ft of Low Floodwall and a total of six road ramps that provide risk reduction to the Hammels area. Three (3) new outfalls (5 ft x 3 ft) are included within the project. The three (3) existing outlets will be modified to add a valve chamber that will include a sluice gate and flap valve to prevent high tides or storm surge to result in flow reversal and cause flooding through the drainage system. In addition, two (2) new pump stations are included within the design.	
	Arverne	The Mid-Rockaway Backbay HFFRRF project alignment the follows the coastal edge of Arverne and constitutes of multiple HFFRR-Features to best match the existing shoreline conditions as well as minimizing impacts. The alignment consists of the construction of approximately 3,170 ft of Low Floodwall, 480 ft of Medium Floodwall, 440 ft of High Floodwall, 2,490 ft of Low Berm, 580 ft of hybrid Berm, 3,950 ft of Bulkhead and 990 ft of Revetment as well as three areas where NNBFs are integrated into the design (discussed below separately). Three (3) road ramps and one (1) vehicular gate are included to maintain access to the waterfront. Eight (8) existing outlets will be modified to add a valve chambers that will include a sluice gate and flap valve to prevent high tides or storm surge to result in flow reversal and cause flooding through the drainage system. Eight (8) new outfalls (5 ft x 3 ft) are included within the project. In addition, three (3) new pump stations are included within the design.	
	Arverne NNBF description	Arverne 1) The north-west corner of the Arverne peninsula (Brant Point). Existing habitat include mud flats, intermediate and high marsh with some fringes of invasive marsh (*Phragmites*). The proposed NNBF would employ the installation of rock sills off the existing, eroding shoreline to protect the toe of the	

	slope and dampen incoming waves so the existing shoreline could be regraded and potentially extended seaward. The proposed NNBF also includes the removal of the *Phragmites* and creation/restoration of the intertidal wetland habitat and high marsh. Some existing uplands features are to be regraded to high marsh. A portion of the existing upland maritime forest between the berm feature and the wetlands are to remain undisturbed and expanded where practical.
Arverne NNBF description	Arverne 2) At the north-east corner (CS-103) of the peninsula where there is currently a narrow beach (DuBois Point), in between Beach 69th and just east of Beach 65th Street a NNBF is proposed that includes the construction of rock sills to create an intertidal flat and replanting with smooth cordgrass (low marsh). The rock sills provide an excellent habitat for attached fauna such as ribbed mussels and shellfish, which help to attenuate wave action. Based on coordination with local and state agencies conducting shellfish (i.e. ribbed mussel) restoration in Jamaica Bay, we will consider in the final design, additional options of materials or techniques (such as pre-seeding mats with bivalves) to ensure success. Further upslope and to the east intertidal marsh can be regraded to provide high marsh habitat adjacent to the existing upland habitats providing a buffer in anticipation of rising sea-level.
Arverne NNBF description	Arverne 3) To the east of Marina 59, much of the existing intertidal marsh along the shoreline is healthy low and intermediate marsh with some fringes infested by *Phragmites*. The proposed NNBF includes restoration of an intertidal flat, protected by rock sills, and regrading of the higher elevations areas to accommodate the establishment of intertidal marsh similar to the adjacent natural marsh areas. The rock sills provide an excellent habitat for attached fauna such as ribbed mussels and shellfish, which help to attenuate wave action.
Edgemere	The Mid-Rockaway Backbay HFFRRF project alignment then follows the coastal edge of Edgemere where a series of HFFRR-Features are interlinked to form the perimeter line of risk reduction to best match the existing shoreline conditions and avoid and minimize impacts. This area also includes two areas where NNBFs are implemented, one on the east and one on the west side of the peninsula (descriptions provided below. The alignment consists out of approximately 480ft of Medium Floodwall, 660 ft of high floodwall, 1,510 ft of Low Berm, 2,060 ft of Medium Berm, 80 ft of High Berm, 2,260 ft of Hybrid Berm and 440 ft of Bulkhead. One (1) road ramp is included to

		maintain access to the waterfront. Three (3) existing outlets will be modified to prevent high tides or storm surge to result in flow reversal and cause flooding through the drainage system. Twelve (12) new outfalls (5 f tx 3 ft) are included within the project and three (3) new pump stations are included within the design.	
	Edgemere NNBF description	Edgemere 1) On the west side of the Edgemere neighborhood, the proposed NNBF design with the establishment of the rock sill, will protect some of the existing eroding wetlands habitats, both subtidal and intertidal, and provide for some areas where high Marsh – Scrub/Shrub habitat can be established. The new habitats will also help provide protection for the berm. The rock sills provide an excellent habitat for attached fauna such as ribbed mussels and shellfish, which help to attenuate wave action as well.	
	Edgemere NNBF description	Edgemere 2) On the east side of the Edgemere neighborhood the proposed NNBF would restore and further enhance existing wetland habitat. A large area of wetland habitat is proposed to be restored and created between the HFFRRF berm and hybrid berm and the newly constructed rock sill, just off of the existing coastline. A hybrid berm was selected in many locations as feature here are placed as far upland to minimize the impacts on the existing wetlands. It allows for minimal habitat impacts and provides space for additional natural habitat development which protects the berm from erosion. The proposed NNBF includes the removal of the *Phragmites* where appropriate, and restoration of the intertidal habitats including planting of smooth cordgrass and high marsh at appropriate elevations, as well as ribbed mussel and reef restoration, which will aid in attenuating wave action.	
2	Motts Basin N (Nassau County)	Project follows a short section of roadway (Waterfront Blvd.) in Nassau County set back from the northern perimeter of Motts Basin. Construction of a low floodwall would reduce the risk of coastal flooding of residential and commercial parcels on the north side of Motts Basin	700
3	Canarsie (Brooklyn)	This project includes flood risk reduction features along Fresh creek such that the lowest portions of the shoreline would be elevated. Revetments would be placed where revetments currently exist. A floodwall is proposed to be constructed along a portion of E 108th Street.	2,800

4	Cedarhurst-Lawrence (Nassau County)	This project includes a section of bulkhead around the end the basin/canal that is situated to the north of Johnny Jack Park and west of the Lawrence High School. As well as small section of floodwall to connect the bulkhead on the west side to high ground. When the existing drainage outlets are blocked by high tail waters a storm drain system will direct runoff towards a new pump station. The preliminary pump station capacity is estimated to be approximately 40cfs.	1,000

Table 6-2: Interior Drainage Infrastructure for Phase 2 HFFRRF Projects

Project Name (Drainage Basin)	Drainage Sub-Basin	Outfall Size	Outfall Location
Mid-Rockaway Backbay with NNBFs (Hammels)	H1	TBD	Existing Outfall ROC-656
	H1	5’x3’	Outfall H1-1, Approximately 70 feet east of Beach 85th Street
	H1	TBD	Existing Outfall ROC-657
	H2	5’x3’	Outfall H2-1, Approximately 350 feet west of Beach 80th Street
	H2	5’x3’	Outfall H2-2, Approximately 100 feet east of Beach 79th Street
	H2	TBD	Existing Outfall ROC-653
Mid-Rockaway Backbay with NNBFs (Arverne)	A1	TBD	Existing Outfall ROC-633
	A1	TBD	Existing Outfall ROC-634
	A1	TBD	Existing Outfall ROC-40062
	A1	5’x3’	Outfall A1-1 located at the end of Hillmeyer Avenue.
	A1	5’x3’	Outfall A1-2 located adjacent to Hillmeyer Avenue and Barbadoes Avenue.
	A1	TBD	Existing Outfall ROC-658
	A1	5’x3’	Outfall A1-3
	A1	TBD	Existing Outfall ROC-659
	A2	5’x3’	Outfall A2-1 located on Bayfield Avenue 150 feet west of Beach 65th Street.
	A2	5’x3’	Outfall A2-2 located at the east end of DE Costa Avenue.
	A2	5’x3’	Outfall A2-3 located at the east end of Burchell Road.
	A3	TBD	Existing Outfall ROC Located at the east end of Thursby Avenue.
	A3	TBD’	Existing Outfall ROC-636

	A3	5’x3’	Outfall A3-1 located 250 north of Beach Channel Drive on 58 Street.
	A3	TBD	Existing Outfall ROC-635
	A3	5’x3’	Outfall A3-2 located 50 north of Beach Channel Drive on 58 Street.
Mid-Rockaway Backbay with NNBFs (Edgemere)	E1	TBD	Existing Outfall ROC-648
	E1	5’x3’	Outfall E1-1 located on Norton Avenue between Beach 47th and 48th Streets.
	E1	5’x3’	Outfall E1-2 located on Norton Avenue between Beach 46th and 45th Streets.
	E1	5’x3’	Outfall E1-3 located on Beach 45th Street north of Hough Place.
	E1	5’x3’	Outfall E1-4 located on the north end of Beach 45th Street.
	E1	5’x3’	Outfall E1-5 located 550 feet north of Hough Place.
	E1	5’x3’	Outfall E1-6 located 500 feet north of Hough Place.
	E1	TBD	Existing Outfall ROC-637
	E1	5’x3’	Outfall E1-7 located north of Beach 40th Street.
	E2	TBD	Existing Outfall ROC-638
	E2	5’x3’	Outfall E2-1 located 50 feet east of Beach 37th Street.
	E2	5’x3’	Outfall E2-2 located 50 feet east of Beach 37th Street.
	E2	5’x3’	Outfall E2-3 located 50 feet east of Beach 36th Street.
	E2	5’x3’	Outfall E2-4 located 50 feet east of Beach 36th Street.
	E2	5’x3’	Outfall E2-5 located between Beach 36th Street and Beach 35th Street.
Motts Basin	L1	TBD	Existing Outfall
Cedarhurst-Lawrence	L1	TBD	Existing Outfall
	L1	TBD	Existing/New Culvert (500 feet from Peninsula Boulevard).
	L1	TBD	Existing/New Culvert (500 feet from Peninsula Boulevard).
	L1	5’x3’	Outfall L-1, Approximately 250 feet from Peninsula Boulevard
Canarsie			Based on the evaluation of the interior water surface elevation and net benefits, no interior drainage plan that would result in a HFFRRF with a BCR above 1.0 was identified. Accordingly there is not a Preferred Drainage Plan identified for the Canarsie drainage basin. Even with the pumps and improved gravity outlet drainage system, flood elevations for a 50% AEP rainfall occurring with the design storm tide are not reduced significantly (see Sub-Appendix E for details)_

TBD: Outfall size To Be Determined, pending surveys during PED

6.2.3 Habitat Restoration and Enhancement to Offset Permanent Impacts

The opportunity to create and/or restore subtidal and intertidal wetlands habitats is one of the key features of the NNBF approach as detailed within this appendix. A series of NNBFs were developed as part of the proposed HFFRRFs to not only control erosion and help manage coastal flood risk, but to provide opportunities for habitat restoration and enhancement to offset unavoidable permanent impacts to federal and state regulated areas. These NNBFs provide the ecological benefits and were incorporated in final design to also recognize future federal, state, and city permitting requirements:

- Restoration and/or creation of both low and high marsh habitats. Specifically, these efforts target the following:
  - Restoration of low marsh habitat in existing mudflat areas proximate to highly erosional shorelines.
  - Restoration and/or creation, as well as enhancement, of high marsh habitat in adjacent uplands that are dominated by common reed (*Phragmites australis*) and other invasive species.
- Creation of rock sill features that provide protection for the subtidal and intertidal habitats, as well as provide a hard bottom habitat for increased ecological production. These features provide additional opportunities for shellfish and ribbed mussel habitat creation.
- Restoration of maritime forest (upland) within upland ruderal and urban habitats that have been significantly impacted by historic and current anthropogenic disturbance. While in upland habitats, these efforts account for anticipated state and city level permitting requirements.

A full analysis of the mitigation is provided in the body of the main report text. Table 6-3 provides an overview of the habitat created. Based on the current HFFRRF alignments and existing habitats, it is estimated that approximately 9.4 acres of new habitat will result from the current configuration, while the permanent impact is estimated at 3.7acres (see main body text, section 6.5 of this GRR/EIS).

Table 6-3: Habitat Created for Mid-Rockaway Backbay with NNBFs in acres

Habitat Type	Restoration/Creation		Enhancement		Total
	Mid-Rockaway Backbay with NNBFs		Mid-Rockaway Backbay with NNBFs		
	Edgemere	Arverne	Edgemere	Arverne	
Intertidal Wetland	3.042	4.606	0.468	-	8.116
Maritime Forest	-	1.348	-	-	1.348
Total	3.042	5.954	0.468		9.464

The designs of the HFFRRF and the habitats created will enhance the ecological resilience of the area by providing a diverse set of habitats that will dynamically change with changing conditions such as sea level rise.

6.3 Phase 2 Project Costs

Project construction costs were estimated once the Phase 2 conceptual design for each project was completed. For Phase 2 the project construction is inclusive of all structural features such as the interior drainage features, pump stations and the HFFRRFs (inclusive of NNBFs). Total project cost include the construction cost, Pre-construction Engineering and Design (PED), construction administration, Real Estate and Operations and Maintenance (O&M) cost and contingencies. The details with respect to establishing the total project cost estimates are documented within the Sub-Appendix C. Table 6-4 below provides an overview of the project costs for the Phase 2 projects.

Table 6-4: Total Project Cost for Phase 2 Projects with inclusion of Natural and Nature Based Features—used for Final Screening

Project ID	Project Name	Perimeter Length (ft)	Project Cost
1	Mid-Rockaway Backbay with NNBFs	22,700	$194 M
2	Motts Basin N	700	$2.6 M
3	Canarsie	2,800	$27.7 M
4	Cedarhurst-Lawrence	1,000	$13.6 M

The inclusion of the costs for drainage infrastructure (new outfalls, modification to existing outfalls and inclusion of pump stations) and real estate increased the project cost compared to the Phase 1 project costs. The Canarsie project cost increased most substantially from $8.4 Million in Phase 1 to $27.7 Million in Phase 2.

6.4 Final Screening of HFFRRF Projects

Along with the refinement of the project designs, the benefits modeling was updated and refined to accurately capture the changes in the project design. The inclusion of interior drainage features and pump stations resulted in changes in residual damages and thus changes in project benefits. After completion of the benefits modeling and interior drainage optimization (see Sub-Appendix E) the Benefit Cost Ratio (BCR) was calculated. Screening results for the Phase 2 projects are presented in Table 6-5.

The results of the Phase 2 screening results, based on BCR, presented in Table 6-5 indicate that three (3) out of the four (4) projects are cost effective. Benefit estimates include the reduced damages as result of coastal flooding as well as a reduction in damages as a result of all interior flooding. The Canarsie project has a BCR below unity and is not selected to move forward. The other three project alternatives will be included within the TSP.

Table 6-5: Phase 2 Screening Results

		Mid-Rockaway Backbay with NNBF's	Canarsie	Cedarhurst - Lawrence	Motts Basin North (no Pumps)
Without Project Damages	Annual Damages	$44,304,000	$4,424,000	$12,655,000	$710,000
With Project Damages	Line of Risk Reduction Damages	$30,585,000	$3,557,000	$6,858,000	$484,000
	Interior Drainage Damages	$1,845,000	$692,000	$643,000	$86,000
	Annual Damages	$32,430,000	$4,249,000	$7,501,000	$570,000
Benefits	Annual Benefits	$11,874,000	$175,000	$5,154,000	$140,000
Costs	Total Project Cost	$194,009,000	$27,675,000	$13,573,000	$ 2,596,000
	Annual Cost	$8,507,000	$1,262,000	$607,000	$111,000
Net Benefits	Net Annual Benefits	$3,367,000	($1,087,000)	$4,547,000	$29,000
	BCR	1.4	0.1	8.5	1.3

6.5 Phase 2 Synopsis

6.5.1 Phase 2 Screening

The second phase of the screening detailed within this chapter included an analysis of existing drainage infrastructure and an analysis of the Minimum Facility, which is intended to ensure that the existing drainage system performs the same with and without the project put in place as to avoid induced flood damages. Project Cost was calculated and included estimates to account for modifications to the existing drainage infrastructure and construction of new drainage infrastructure as part of the HFFRRF projects. In addition a more detailed analysis of the potential impacts to wetland habitat was completed and the project NNBF designs were further refined and planned in co-location with the flood risk reduction features in order to take advantage of their capacity to improve the function and resilience of the structural features.

6.5.2 Phase 2 Projects Selected

In Phase 2 additional analyses were completed to progressively converge to a higher level of detail for the HFFRRF Projects. Four (4) projects were screened and three (3) projects were identified based on a BCR ratio greater than 1.0 (See Table 6-5). However due to changes to the cost estimates resulting from the Cost Certification Review, the estimated cost to construct Motts Basin North increased by approximately 20% as result of an increase in both estimated construction and real estate cost. The increase in cost was due to an increase in the sheet pile quantities for the low-floodwall and the project's sensitivity to an increase in construction cost for this particular HFFRR-Feature. This caused the benefit to cost ratio to decrease below 1.0 and be eliminated from the Recommended Plan as it has negative net benefits and is therefore not economically justified.

In summary, the following two HFFRRF projects were brought forward to the recommended plan:

1. Mid-Rockaway Backbay with NNBFs
2. Cedarhust-Lawrence

7 THE RECOMMENDED PLAN FOR JAMAICA BAY

7.1 The Recommended Plan

The communities surrounding Jamaica Bay experience substantial risk for coastal flooding. Therefore, the study team sought to identify stand-alone project features that could complement a potential future storm surge barrier, but also be economically justified on their own. Residents in many parts of the Jamaica Bay vicinity experience flooding due to high tides that occur frequently. Since a storm surge barrier would not be closed at every high tide, there is an opportunity to recommend features to mitigate flood risk in high frequency tidal flooding events in which the proposed storm surge barrier would remain open.

Low lying coastal neighborhood areas within Jamaica Bay were identified as areas where High Frequency Flood Risk Reduction Features (HFFRRF) could be implemented. Features that provide a flood risk reduction function were developed and designed to generate project alternatives that would reduce the risk of flooding from high frequency storm events. The future (year 2068) 20% AEP stage elevation is selected for this study[9]. The 20% AEP stage elevation, albeit spatially variable, is approximately equal to an elevation of +7 feet NAVD88 for the study area. HFFRRF include, amongst others, floodwalls, berms, bulkheads and revetments. With ground elevations varying, prototypical HFFRRF heights (measured from ground elevation) range between 3 feet and 8 feet.

Project alignments were defined through an approach that generally selected and placed HFFRRF along the coastal edge while protecting as many existing assets as was practically feasible. Projects were developed by considering realistic project extents, where the determination of a realistic project extents was established based on shoreline type, length, topography, neighborhood, land use, planning considerations, project scope, inundation extents, flooding pathway, and existing topography.

Two phases of feasibility design, analysis and project screening on cost and benefit was performed to evaluate viable economically justified stand-alone projects. Two (2) projects were selected to be included within the Recommended Plan:

1. Mid-Rockaway Backbay with NNBFs
2. Cedarhurst-Lawrence

Detailed plans for the projects and sections for the HFFRRF that are included within the Recommended Plan are provided in Sub-Appendix F.

[9] The future 20% AEP (5 year Return Period in 2068) amounts to a 10% AEP in 2018, or a 10 year Return Period stage elevation.

7.1.1 Mid-Rockaway Backbay with NNBFs

The Mid-Rockaway Backbay with NNBFs project consists of a risk reduction alignment that encompasses three (3) neighborhoods on the coastal edge of Jamaica Bay between Beach 35th Street and Beach 88th Street in Queens, NY. For ease of reference the three areas are described separately.

Hammels: A HFFRRF project alignment that consists of approximately 2550 feet of Low Floodwall and a total of six road ramps provides risk reduction to the Hammels area (Figure 7-1). Three (3) new outfalls (5ft x 3ft) at Beach 85th, Beach 80th and Beach 79th Street are included within the project. The three (3) existing outlets will be modified to add a valve chamber that will include a sluice gate and flap valve to prevent high tides or storm surge to result in flow reversal and cause flooding through the drainage system. In addition, two (2) new pump stations are included within the design: one 100 cfs pump station at the end of Beach 87th Street, and one 180 cfs pump station near the intersection of Beach Channel Drive and Beach 78th Street.

Arverne: The project alignment follows the coastal edge of Arverne and consists of multiple HFFRRF to best match the existing shoreline conditions and minimize impacts. The alignment consists of the construction of approximately 3170 feet of Low Floodwall, 480 feet of Medium Floodwall, 440 feet of High Floodwall, 2490 feet of Low Berm, 1140 feet of Medium Berm, 580 feet of Hybrid Berm, 3950 feet of Bulkhead, and 990 feet of Revetment as well as three areas where NNBFs are integrated into the design (Figure 7-2). Wetland habitat is created in co-location with the berms. Since the NNBFs are located on the unprotected side of the HFFRRF, the toe protection features and bands of tidal wetland vegetation included in the NNBFs will act to dampen incoming waves during storm conditions and reduce the risk of overtopping. The NNBFs also present an opportunity to re-establish sub-tidal, intertidal and supratidal habitats to create a nature based shoreline that is resilient to both moderate storm events and encroaching sea level and will allow for the migration of these natural features shoreward, increasing the lifetime of the habitats. As part of the project eight (8) existing outlets will be modified to add a valve chamber that will include a sluice gate and flap valve to prevent high tides or storm surge to result in flow reversal and cause flooding through the drainage system. Eight (8) new outfalls (5ft x 3ft) are included within the project. In addition, three (3) new pump stations are included within the design. The pump stations are preliminarily located within the vicinity of the intersection of Beach 72nd street and De Costa Avenue, north of De Costa Avenue just east of the intersection of Beach 63rd Street and De Costa Avenue, and at the eastern end of Thursby Avenue. Furthermore, three (3) road ramps and one (1) vehicular gate are included to maintain access to the waterfront.

Edgemere: The project alignment follows the coastal edge of Edgemere and includes two areas, one on the east and one on the west side of the peninsula, where NNBFs are implemented (Figure 7-3). A series of HFFRRF are interlinked to form the perimeter line of risk reduction to best match the existing shoreline conditions and avoid and minimize impacts. The alignment consists of approximately 480 feet of Medium Floodwall, 660 feet of high floodwall, 1510 feet of Low Berm, 2060 feet of Medium Berm, 80 feet of High Berm, 2260 feet of Hybrid Berm, and 440 feet of

Bulkhead. One (1) road ramp is included to maintain access to the waterfront. Three (3) existing outlets will be modified to add a valve chamber that will include a sluice gate and flap valve to prevent high tides or storm surge to result in flow reversal and cause flooding through the drainage system. Twelve (12) new outfalls (5ft x 3ft) are included within the project. In addition, three (3) new pump stations are included within the design. Due to the size of the area and difficulties in draining all of the drainage area to a single site, one of the drainage subbasins (E1) is proposed to have two pump stations. One pump station would be located near Norton Avenue and Beach 49th Street, and the other would be near Beach 43rd Street and Hough Place, with a combined capacity of approximately 210 cfs. A third pump station drains the remainder of the Edgemere area and is proposed to be located near Beach 38th Street with an estimated capacity of 120 cfs.

1C

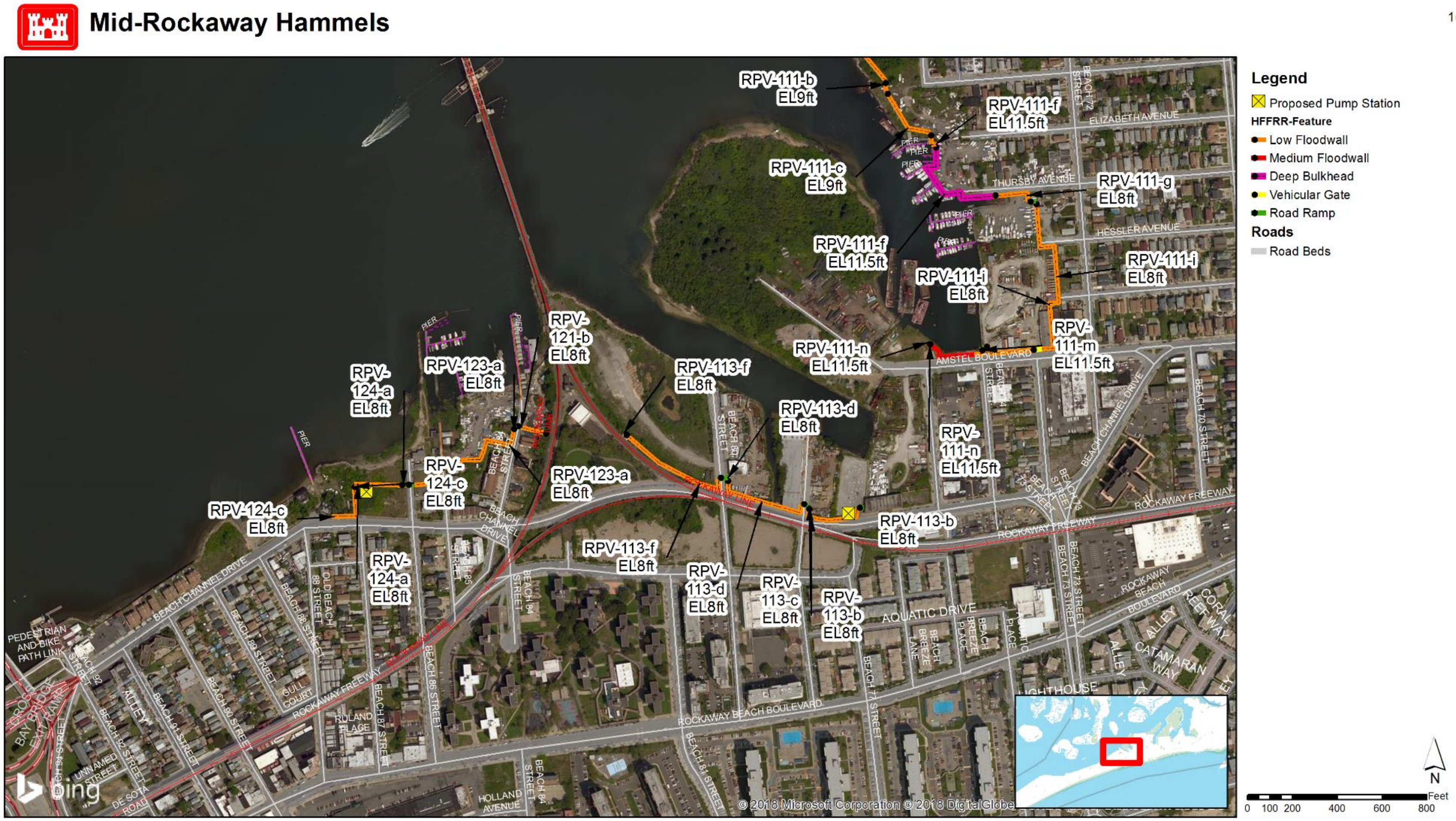


Figure 7-1: Mid-Rockaway Backbay with NNBFs – Hammels Vicinity

Figure 7-2: Mid-Rockaway Backbay with NNBFs – Arverne Vicinity

Figure 7-3: Mid-Rockaway Backbay with NNBFs – Edgemere Vicinity

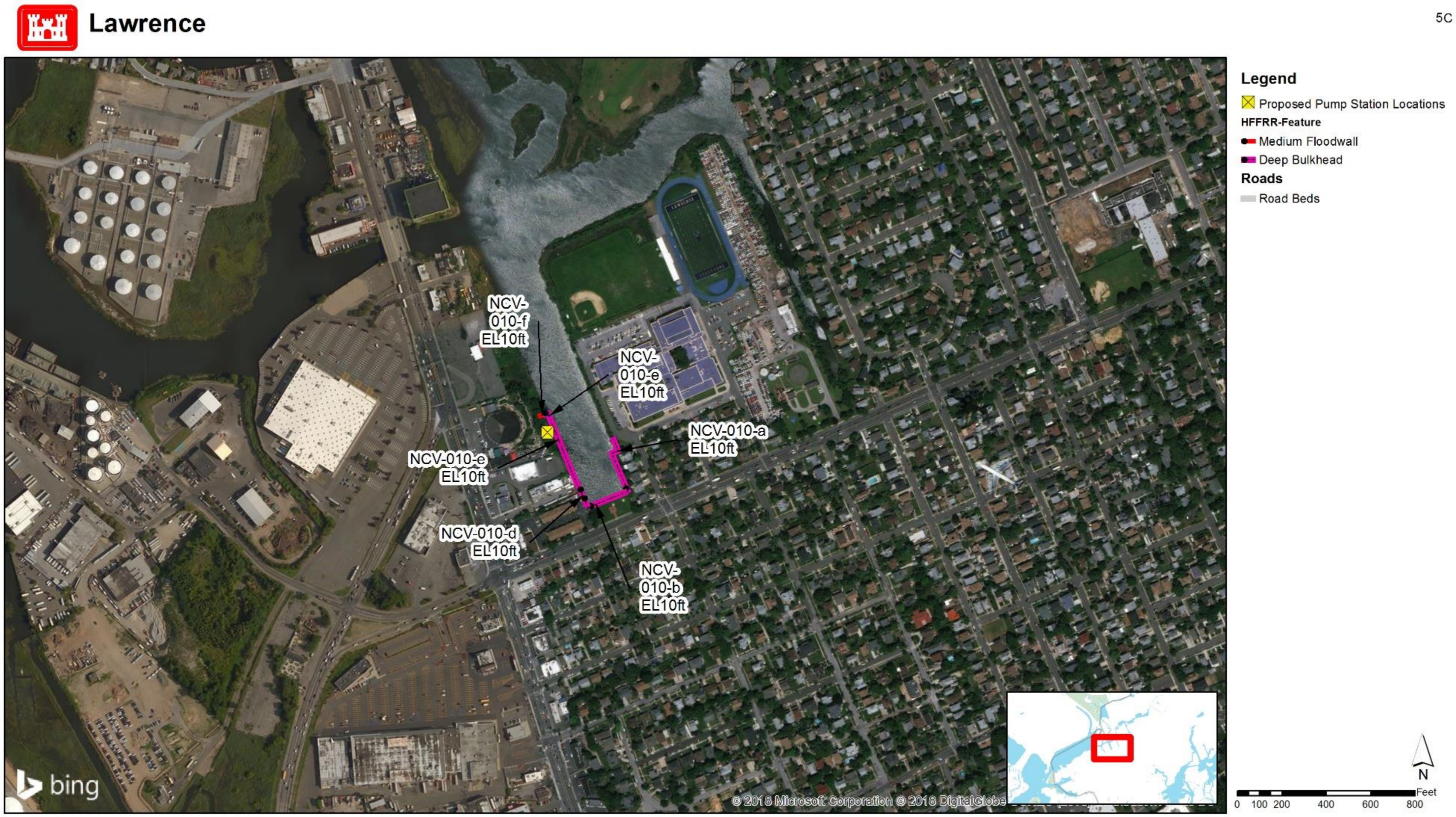


Figure 7-4: Cedarhurst-Lawrence

7.1.2 Cedarhurst-Lawrence

The Cedarhurst-Lawrence project is located in Nassau County and crosses the border between the Village of Cedarhurst and the town of Hempstead. The project alignment follows the edge of the basin/canal that is situated to the north of Johnny Jack Park and to the west of the Lawrence High School. The project alignment follows the water's edge and includes approximately 960 feet of Bulkhead and a short section of approximately 25 feet of Medium Floodwall. There are three (3) existing outfalls in the area where the bulkhead will be raised. Each of the existing outlets will be modified to add a valve chamber that will include a sluice gate and flap valve to prevent high tides or storm surge from flooding through the drainage system. When the drainage outlets are blocked by high tides, a storm drain system will direct runoff towards a new pump station. The preliminary pump station capacity is estimated to be approximately 40 cfs, and will be situated at the north-eastern end of the project area as shown in Figure 7-4.

7.2 Quantities and Cost

Cost estimates for the Recommended Plan were developed at an April 1st 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 F, and full details on the development of the Recommended Plan cost estimate are provided within 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. To estimate the cost of the pump stations, cost curves were used. The cost risk analysis determined the contingency to be 28.36%, making the total project cost (fully funded) $261.6 million for budgeting purposes. The Civil Work Breakdown Structure (CWBS) feature codes as shown in Table 7-1 are utilized to establish the project cost. The project cost presented in Table 7-1 are a summary of the detail in Appendix C.

Table 7-1: MII Estimate Recommended Plan – HFFRRF for Jamaica Bay

CWBS account code #	HFFRRF Project Account Code Description	Total Project Cost (Fully Funded)
	Mid-Rockaway Backbay with NNBFs	**$199,798,000**
01	Lands and Damages	$17,687,000
02	Utility Relocations	$5,636,000
11	Levees and Floodwalls	$127,473,000
13	Pump Stations	$47,256,000
18	Cultural Resource Preservation	$1,746,000
	Cedarhurst-Lawrence	**$15,208,000**
01	Lands and Damages	$915,000
02	Utility Relocations	$238,000
11	Levees and Floodwalls	$9,214,000
13	Pump Stations	$3,809,000
18	Cultural Resource Preservation	$1,032,000
30	Pre-Construction Engineering and Design	$23,545,000
31	Construction Management	$13,054,000
	Total	**$251,605,000**

7.3 Construction Schedule

The construction schedule for the Jamaica Bay Reach is included in Appendix C. The total duration is approximately 46 months. Figure 7-5 shows the current construction schedule for the 2 projects that are part of the recommended plan.

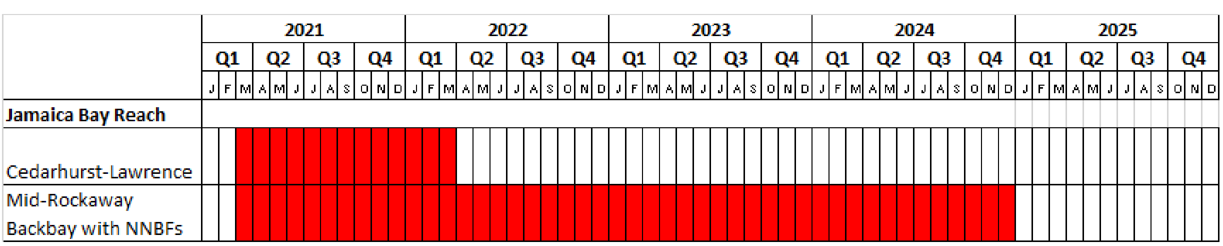


Figure 7-5: Schematic overview of the construction schedule for the Jamaica Bay Reach of the Recommended Plan

7.4 Recommendations for PED

This Engineering & Design Appendix provides an overview of the analyses supporting the development of the HFFRRF for Jamaica Bay. This appendix furthermore describes the development of HFFRRF Projects and a screening analysis of these projects to establish a feasible plan to mitigate for high frequency flood risk in Jamaica Bay. Based on the data gathered during the feasibility study and engineering analyses, a preliminary design for the HFFRRF projects has been completed. It should be noted that HFFRRF designs are prototypical in nature and are not site-specific designs. The dimensions and sizing of the individual features in this study are preliminary, based on the study area conditions and sufficient for feasibility level study. It is expected that HFFRRF designs would be further refined in PED. The preliminary designs shall not be construed as requirements for actual dimensions for implementation. Significant additional engineering analysis is required to substantiate the designs of the flood risk reduction features, the drainage infrastructure and the pump stations including, but not limited to, a full evaluation of topographical and bathymetric elevations, subsurface soil conditions, inventory and investigation of existing structures and utilities. More details regarding recommendations are provided below.

7.4.1 Component Revisions and Analyses

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.

Of special note is also the analysis of the adaptability of the HFFRRF projects in the face of changing sea levels. Feature heights will be finalized during the PED phase and additional analysis is recommended to document the adaptability of the projects under the consideration of sea level rise scenarios. The HFFRRF have been designed for future 20% AEP water levels with consideration of the intermediate SLC scenario. If the realized SLC exceeds the design SLC and closely resembles the USACE high SLC scenario, then adaptation is expected to be required in the year 2044 for those feature that provide the flood risk reduction function. In general there are two adaptability options: 1) With a storm surge barrier for Jamaica Bay in place; operate the storm surge barrier more frequently and 2) Without a storm surge barrier in place; address larger wave overtopping volumes with collection systems and/or pumps and retrofit HFFRRF features to allow for an increase in elevation.

In addition, rock sills and the NNBFs are considered to be adaptable. The placement of additional stone over time allows for an increase in crest elevation of rock sills with sea level rise in the event that realized RSLC exceeds the projection. The designs also include an expectation that the protected habitats behind the sill features would migrate shoreward and the fill/cut elevations chosen in most cases are designed to allow for that migration. Thus, the NNBFs proposed herein are intrinsically adaptive features and consist of improved wetland habitats and a more natural shoreline that can migrate with rising sea levels in the future.

Finally, albeit that the HFFRRF elevations exceed the year 2018 10% AEP water levels, over time, and with rising sea levels is not expected that all HFFRRF elevations will be above furtur 10% AEP water levels. Finally, in all instances the 1% AEP water level is expected to exceed the lowest HFFRR-Feature elevation and therefore the project elevation. The reader is referred to Figure 7-6 for a graphic representation of the adaptability of HFFRRF Projects. A revised and detailed analysis of adaptability is recommended for the PED phase.

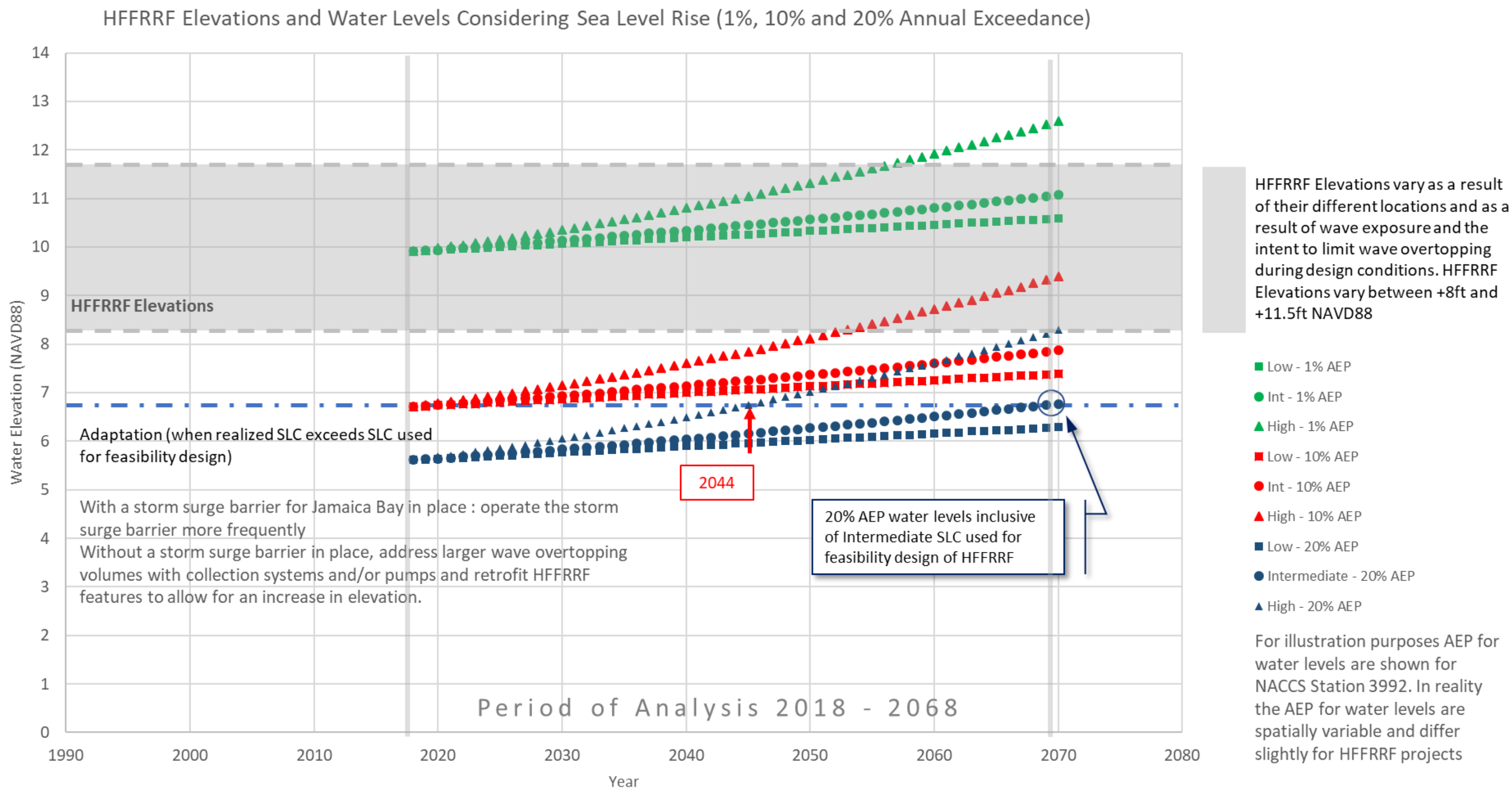


Figure 7-6: Graphic Presentation of Adaptability of HFFRF Projects

7.4.2 Recommended PED Analyses

A preliminary, non-exhaustive listing of potential future engineering analyses and design refinements for PED include the following:

- Geotechnical data collection (site specific borings and geotechnical data collection),
- Bathymetric and topographic data collection,
- Utility survey and an inventory and condition assessment of existing structures and utilities for the project area,
- Site specific design for all HFFRRF including detailed structural, geotechnical and civil engineering analyses and design,
- Design and engineering analysis regarding HFFRRF site integration, notably roadway design for road ramps and all features in close proximity to DOT right-of-way,
- Given the close proximity to private property, additional refinement and site-specific details will need to be worked out to establish the permanent and temporary easements,
- Refinement of project elevations and design of the HFFRRF transitions between different feature types,
- For the AEP stage elevations relevant to the HFFRRF design it can be noted that the underlying tides play a more dominant role and it is recommended to further communicate this concept with the public to increase awareness of risk. For example, the 20% AEP water level can be a result of high tides with a relative small storm surge or a low tide with a relatively large storm surge. For extreme events, i.e. lower AEP events, the storm surge component is the dominant contributor to the total stage elevation regardless of whether the event coincides with high or low tides. This is important from a risk communication perspective since smaller meteorological events may not deserve the same attention but can result in flooding if they coincide with high tides.
- Bayswater Park and Beach 35th St are currently being redesigned by DOT and Parks. The design of the proposed berm will be further coordinated with both agencies.
- Refined engineering analyses and design for the pump stations, pump capacities and new drainage infrastructure,
- Refined design and engineering analyses of modifications and connections to existing drainage infrastructure,
- NNBF designs include the preliminary identification of locations where these elements will likely fit, but final design and NNBF siting will depend on final feature alignments and detailed delineation of existing grades and elevations,
- Detailed 2D wave modeling and analysis of wakes from Ferry/Commercial/Recreation vessels at different tidal elevations to optimize rock sill designs and freeboard requirements of HFFRRF, and
- Analysis of temporary construction features.

It is further recommended that all refinements and analyses are coordinated with the appropriate agencies with respect to future and ongoing infrastructure upgrades, park and recreational developments, environmental remediation and housing developments.

REFERENCES

Billion Oyster Project. 2018. Information on the effort to restore a sustainable oyster population and to foster awareness, affinity, and understanding of the New York Harbor by engaging New Yorkers directly in the work of restoring one billion oysters. New York (NY). https://billionoysterproject.org/ (accessed July 3rd 2018).

Bridges, T. S., Wagner, P. W., Burks-Copes, K. A., Bates, M. E., Collier, Z., Fischenich, C. J., Gailani, J. Z., Leuck, L. D., Piercy, C. D., Rosati, J. D., Russo, E. J., Shafer, D. J., Suedel, B. C., Vuxton, E. A., and Wamsley, T. V. (2014). *Use of natural and nature-based features (NNBF) for coastal resilience*. ERDC SR-15-1. Vicksburg, MS: U.S. Army Engineer Research and Development Center.

Cialone, M. A., Massey, T. C., Anderson, M. E., Grzegorzewski, A. S., Jensen, R. E., Cialone, A., Mark, D. J., Pevey, K. C., Gunkel, B. L., McAlpin, T. O., Nadal-Caraballo, N. C., Melby, J. A., Ratcliff, J. J. (2015) North Atlantic Coast Comprehensive Study (NACCS) coastal storm model simulations: waves and water levels. Technical report, Coastal and Hydraulics Laboratory, US Army Corps of Engineers (USACE), Engineer Research and Development Center, Vicksburg, MS.

EurOtop (2016). Manual on wave overtopping of sea defences and related structures. An overtopping manual largely based on European research, but for worldwide application. Second Edition. Authors: Van der Meer, J. W., N. W. H., Allsop, T. Bruce, J. DeRouck, A. Kortenhaus, T. Pullen, H. Schüttrumpf, P. Troch, and B. Zanuttigh

Gornitz, et, al "Impacts of Sea Level Rise in the New York City Metropolitan Area", Global and Planetary Changes 32 (2002) 61- 88, 2002 Elsevier Science B.V.

Jamaica Bay Watershed Protection Plan, 2016 Update. 2016. New York City Department of Environmental Protection - Bureau of Environmental Planning and Analysis. October 1, 2016 DEP

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

RAMPP (2014). Region II Storm Surge Project – Recurrence Interval Analysis of Coastal Storm Surge Levels and Wave Characteristics, FEMA to HSFE02-09-J-001.

USACE (2002). Coastal Engineering Manual (EM 1110-2-1100). Washington, DC: Department of the Army.

USACE (2013). Incorporating Sea Level Change in Civil Works Programs (ER-1100-2-8162).

USACE (2015). North Atlantic Coast Comprehensive Study (NACCS): Coastal Storm Hazards from Virginia to Maine. ERDC/CHL TR-15-5 Technical Report, Nov 2015. Authors: Nadal-Caraballo N.C., J.A. Melby, V.M. Gonzalez, and A.T. Cox

USACE-NAN (2016). Atlantic Coast of New York, East Rockaway Inlet to Rockaway Inlet and Jamaica Bay - Draft Integrated Hurricane Sandy General Reevaluation Report and Environmental Impact Statement.

USGS Lidar https://lta.cr.usgs.gov/

C. B. Zarnoch and M. P. Schreibman 2012. Growth and Reproduction of Eastern Oysters, Crassostrea Virginica, in a New York City Estuary: Implications for Restoration. Urban Habitats. Volume 7 March 2012.