VIII. Coastal Storm Risk Management Measures

Coastal systems provide important social, economic, and ecological benefits to the Nation. However, our coasts are vulnerable to the influence of a combination of factors, including storms, changing climate, geological processes, and the pressures of ongoing development and urbanization. The overarching strategy to increase coastal resilience and reduce vulnerability can be achieved by 1) instituting land use changes over time to adapt to impacts that increase risks; 2) accommodating potential changes such as climate variability, sea level change, etc. to preserve the natural and built environment over time; and 3) employing risk reduction measures to reduce flood damages to property and infrastructure. In addition to policy and programmatic efforts to reduce risk, the NACCS Coastal Storm Risk Management Framework builds on three common adaptation categories used by the climate adaptation communities in the US and internationally: avoid (sometimes termed retreat), accommodate, and preserve (sometimes termed "protect") (Dronkers, J. et al. 1990; USACE 2014).

NNBF, non-structural, and structural are terms used to describe the full array of measures that can be employed to provide increased coastal resilience and risk reduction (USACE, 2013). An integrated, watershed-based approach that draws together a combination of measures as part of the above strategies will reduce risk and enhance coastal resilience over the long-term (USACE, 2013). A systems approach to evaluating comprehensive flood risk is necessary to evaluate the synergistic benefits of a combination of strategies, resilience and robustness of the coastal landscape, as well as to identify and communicate residual risk. Figure VIII-1 depicts the coastal landscape considering the three strategies and various management measures. The Framework describes the process local communities and other stakeholders could use to evaluate coastal flood risk, future vulnerability with respect to sea level change, and the strategies and measures to manage existing vulnerabilities and increasing risk over time.

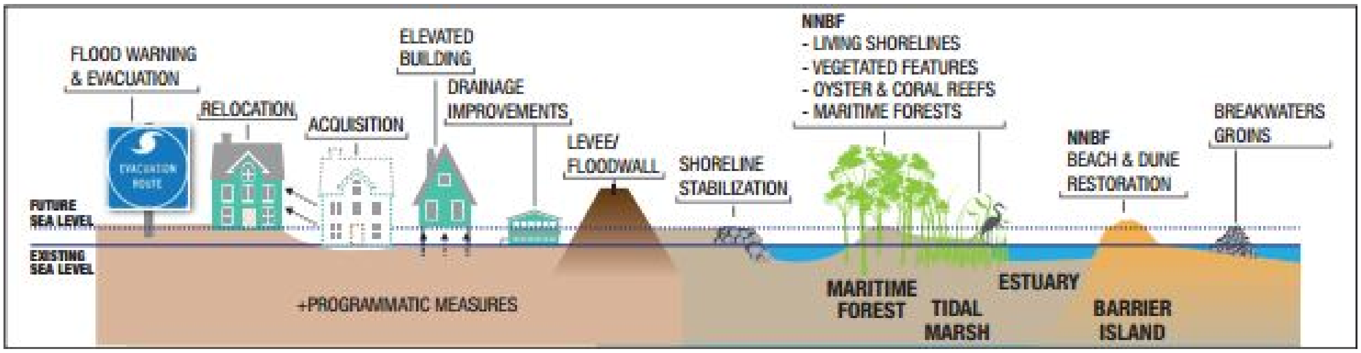

Figure VIII-1. Combinations of adaptable measures may be used to improve redundancy, robustness, and resilience associated with coastal flood risk management (not to scale)

Risk Management Measures Categorizations and Comparisons

A suite of coastal storm risk management measures was developed by taking an integrated approach that considers combinations of the full array of available measures (USACE, 2013). All of these measures were identified as potentially effective ways to reduce the vulnerability of coastal populations and increase resilience. The coastal storm risk management measures include structural, non-structural, NNBF, and programmatic measures. USACE convened a two day working meeting on June 26-27, 2013, at the Stevens Institute of Technology in Hoboken, NJ, with representatives from Federal, State, and local governments, as well as academia and private industry, to discuss the full array of potential measures. A master list of all the measures was compiled and filtered for duplication and consistency with study goals and objectives, then augmented based upon a literature review. The various measures were categorized as structural, non-structural, and NNBF in the final aggregated list. Some NNBF measures were identified for both the NNBF and structural categories because of their storm surge reduction potential. Additionally, programmatic measures were organized under the nonstructural category. Once the measures were aggregated into specific types, USACE staff evaluated the respective risk reduction capacity. Risk management measures were characterized by the degree to which they could 1) reduce coastal storm damages (through reductions in flooding, waves, or erosion), 2) produce multiple benefits, and 3) promote resilience and adaptive capacity (Table IV-4). This evaluation of the coastal storm risk management functions is based on professional experiences from previous coastal storm investigations. It was intended to present a qualitative assessment of the function, performance, utility, and resilience attributes of the various measures. Subsequent analyses could provide more refined and quantitative evaluations of the measures' risk reduction capacity. This process to compile and aggregate measures is illustrated in Table VIII-1.

Although many of the categories generally correspond to standard coastal risk management strategies, specific applications are not constrained to the usual solutions. Opportunities for innovative designs, technologies, materials, etc., should be considered when evaluating specific application of any of these measures. Furthermore, innovative combinations of standard measures are expected to be key to managing coastal risks and promote resilience. For example, shoreline stabilization measures, such as seawalls and revetments, can work effectively with beach restoration when designed to be exposed to waves only during extreme events to provide an additional line of defense without interrupting non-storm coastal processes (USACE, 2013).

Note that the actual design level associated with these measures could vary significantly depending on the specific application. At site-specific locations, design considerations of measures and corresponding assumptions will change. The values will change as assumptions change. For example, for the purposes of this study, beach restoration, alone or in combination with other structures such as groins or breakwaters, could be designed to reduce risks due to storm tides and waves to 1 percent flood level. Furthermore, USACE analyses of coastal flood risk management plans optimize net annual benefits compared to net annual costs of the plan as opposed to a specific design elevation. For general comparison and as part of the Framework evaluation of management measures, assumptions of a specific design elevation across the study area for the measure was required to compare to the corresponding the floodplain inundation scenario (10 percent flood and 1 percent flood plus 3 feet).

Table VIII-1. Storm Damage Reduction and Resilience Attributes Associated with the Full Array of Measures

Aggregated Measure Type[1]	Category[2]	Storm Damage Reduction Function			Multi-Benefits[3]	Resilience
		Flooding	Wave Attenuation	Erosion		Adaptive Capacity[4]
Acquisition (building removal) and relocation[5]	Non-STR	High	High	High	High	High
Building retrofit (e.g., floodproofing, elevating structures, relocating structures, ringwalls)	Non-STR	High	Low	Low	Low	Low
Enhanced flood warning and evacuation planning (early warning systems, emergency response systems, emergency access routes)	Non-STR	Low	None	None	Low	High
Land use management/conservation and preservation of undeveloped land, zoning and flood insurance	Non-STR	Medium	None	None	High	Medium
Deployable floodwalls	STR	Medium	None	None	None	Low
Floodwalls [6] and levees	STR	High	Low	None	Low	Low
Shoreline stabilization (seawalls, revetments, bulkheads)	STR	Low	High	High	Low	Low
Storm surge barriers	STR	High	Medium	None	Low	Low
Barrier Island preservation and beach restoration (beach fill, dune creation)	STR/NNBF	High	High	Medium	High	High
Beach restoration and breakwaters	STR/NNBF	High	High	High	High	Medium
Beach restoration and groins	STR/NNBF	High	High	High	High	Medium
Drainage improvements (e.g., channel restoration, water storage/retention features)	STR/NNBF	Medium	Low	Medium	Medium	Low
Living shorelines	STR/NNBF	Low	Medium	Medium	High	High

Aggregated Measure Type[1]	Category[2]	Storm Damage Reduction Function			Multi-Benefits[3]	Resilience
		Flooding	Wave Attenuation	Erosion		Adaptive Capacity[4]
Overwash fans (e.g., back bay tidal flats/fans)	NNBF	Low	Medium	High	Medium	High
Reefs	NNBF	Low	Medium	Medium	High	High
Submerged aquatic vegetation	NNBF	Low	Low	Low	High	Medium
Wetlands	NNBF	Low	Medium	Medium	High	High

[1] An extensive list of management measures was compiled as part of the NACCS Measures Working Meeting in June 2013. The measures presented here represent an aggregated list of the categories of measures and corresponding conceptual parametric unit cost estimates.
[2]STR = structural measure, Non-STR = nonstructural measure, and NNBF = Natural and Nature-Based Features measure. Multiple measures are listed if the aggregated measure type is made up of a combination of measures.
[3] Multi-benefits focus on socioeconomic contributions to human health and welfare above and beyond the risk reduction benefits already highlighted in this table (i.e., flooding, wave attenuation, etc.). These benefits could include increased recreational opportunities, development of fish and wildlife habitat, provisioning of clean water, production of harvestable fish or other materials, etc.
[4] Adaptive capacity is the assessment of a measure's ability to adjust through natural processes, operation and maintenance activities, or adaptive management, to preserve the measure's function.
[5] Acquisition, relocation, and buyouts do not actually prevent flooding and erosion but remove the population and associated development from its effects.
[6] The concept design identified for the floodwall category consists of a concrete structure. These structures might also require closure structures including stoplogs, miter gates, swing gates, or roller gates, which were not included in the development of the parametric unit cost estimate. A simple steel sheetpile I-wall may be more economical.

VIII.1. Applicability by Shoreline Type

In order to complete the NACCS Tier 1 assessment, the measures were further categorized based on shoreline type to generally identify a geographic location where they are best suited according to typical application opportunities, constraints, and best professional judgment. Shoreline types were derived from the NOAA Environmental Sensitivity Index Shoreline Classification dataset (http://stateofthecoast.noaa.gov/shoreline/esi_categories.html), (NOAA, n.d.). Nonstructural measures could be considered in all geographic contexts and were not specifically included in the Tier 1 assessment of management measures applicable to shoreline types for the various risk areas identified as part of the NACCS exposure and risk assessment. This categorization is summarized in Planning and State Appendices. Table VIII-2 presents the measures applicability by shoreline type.

Table VIII-2. Structural and NNBF Measure Applicability by NOAA-ESI Shoreline Type

Measures	Rocky shores (Exposed)	Rocky shores (Sheltered)	Beaches (Exposed)	Manmade structures (Exposed)	Manmade structures (Sheltered)	Scarps (Exposed)	Scarps (Sheltered)	Vegetated low banks (Sheltered)	Vegetated low banks (Sheltered)	Wetlands/Marshes/ Swamps (Sheltered)
Structural										
Storm Surge Barrier[1]										
Barrier Island Preservation and Beach Restoration (beach fill, dune creation)[2]			x							
Beach Restoration and Breakwaters[2]			x							
Beach Restoration and Groins[2]			x							
Shoreline Stabilization						x	x	x		
Deployable Floodwalls					x					
Floodwalls and Levees		x			x			x		
Drainage Improvements	x	x	x	x	x	x	x	x	x	x
Natural and Nature-Based Features										
Living Shoreline						x	x	x		x
Wetlands							x			x
Reefs	x	x				x				x
Submerged Aquatic Vegetation[3]										x
Overwash Fans[4]										
Drainage Improvements	x	x	x	x	x	x	x	x	x	x

[1] *The applicability of storm surge barriers cannot be determined based on shoreline type. It depends on other factors such as coastal geography.*
[2] *Beaches and dunes are also considered NNBF.*
[3] *Submerged aquatic vegetation is not associated with any particular shoreline type. It is initially assumed to apply to wetland shorelines.*
[4] *Overwash fans may apply to the back side of barrier islands, which are not explicitly identified in the NOAA Environmental Sensitivity Index Shoreline Classification dataset.*

Additionally, a conceptual analysis of geographic applicability of NNBF measures presented in Table VIII-3 was completed, including beach restoration, beach restoration with breakwaters/groins, living shorelines, reefs, submerged aquatic vegetation, and wetlands. The GIS operations that were used for the NNBF screening analysis are described in the ERDC NNBF Technical Report. In addition to the NOAA Environmental Sensitivity Index Shoreline Classification dataset (http://stateof thecoast.noaa.gov/shoreline/esi_categories.html) (NOAA, n.d.), other criteria that was considered was habitat type, impervious cover, water quality, and topography/bathymetry.

Table VIII-3. Structural and NNBF Measure Applicability by NOAA-ESI Shoreline Type

		Data Layers/Thresholds				
	Action/Operation	**Habitat Type**	**Urban Areas**	**Poor Water Quality**	**Shoreline Type**	**Topography/Bathymetry (ft +/- MSL)**
NNBF Measures		The Nature Conservancy Eco Regions; USFWS	Impervious Cover < 20% (Y or N)	EPA 303(d) Impaired Waterway	NOAA ESI Shoreline (NACCS aggregation)	10m DEM/NOAA bathymetry data (30m coastal relief data)
Barrier Island Preservation and Beach Restoration	NNBF Report Table 4-11 GIS Operation	Reference NNBF Report Table 4-11	N	Y	Beaches (exposed)	N/A
Breakwaters and Beach Restoration	NNBF Report Table 4-11 GIS Operation	Reference NNBF Report Table 4-11	N	Y	Beaches (exposed)	N/A
Groins and Beach Restoration	NNBF Report Table 4-11 GIS Operation	Reference NNBF Report Table 4-11	N	Y	Beaches (exposed)	N/A
Living Shoreline	NNBF Report Table 4-11 GIS Operation	Scrub-Shrub, Freshwater Emergent Wetland, Freshwater Forested/Shrub Wetland	N	Y	Scarps (exposed), Scarps (sheltered), Vegetated Low Banks (sheltered), Wetlands (Sheltered)	-1 to +2
Wetlands	NNBF Report Table 4-11 GIS Operation	Reference NNBF Report Table 4-11	Y	Y	Scarps (sheltered), Wetlands (sheltered)	0 to +2
Reefs	NNBF Report Table 4-11 GIS Operation	N/A	Y	N	Rocky Shores (exposed), Rocky Shores (sheltered), Scarps (exposed), Wetlands (sheltered)	-1 to -6
Submerged Aquatic Vegetation (SAV) Restoration	NNBF Report Table 4-11 GIS Operation	Reference NNBF Report Table 4-11	Y	N	Wetlands (sheltered)	-1 to -6

The NNBF measures presented in Table VIII-3 were evaluated using ESRI ArcGIS software to screen the relative geographic locations across the study area (ESRI, 2012). The primary features associated with the NNBF screening analysis were habitat type, shoreline type, and topography and bathymetry. The water quality components associated with the screening analysis represent areas of the study area that might impact the overall function of the respective features. The results of the NNBF screening analysis are presented in the State and District of Columbia Analyses Appendix.

VIII.2. Evaluation of Sea Level Affecting Marsh Model (SLAMM)

SLAMM "simulates the dominant processes involved in wetland conversions and shoreline modifications during long-term sea level rise" (Clough et al., 2010). Since its development in 1986, SLAMM has undergone multiple version releases (six in total) and has been broadly applied for assessing the long-term effects of sea level change on wetlands and shorelines. SLAMM is a spatially-explicit, raster-based model that applies a set of theoretical, empirical, and qualitative "rules" to capture the long-term effects of sea level change as they pertain to six key processes: inundation, salinity, saturation, accretion, erosion, and barrier island overwash (Clough and Larson 2010, Clough et al., 2010). Three of these processes (inundation, salinity, saturation) examine thresholds for switching to an alternative habitat type; the remaining three (accretion, erosion, overwash) address internal and external processes acting to maintain or degrade the current habitat type.

As part of the NACCS, USACE evaluated SLAMM to identify potential improvements for coastal marshes and wetlands affected by sea level change. The evaluation included consideration of assessing the effects of thin-layer placement of dredged materials as a potential mitigation option to reduce wetland losses due to sea level change, which could further exacerbate coastal flood risk. The purpose of this evaluation was to incorporate the opportunities for improvement of SLAMM that could be used by coastal managers. USACE staff conferred the developers of SLAMM to consider new process descriptions for the evaluation of primary productivity, the above and below ground production of organic materials, and the effectiveness of thin layer mineral placement typically associated with beneficial use of dredged materials. For those areas that could potentially utilize a combination of measures that incorporates NNBF and wetlands, particularly in back bays and estuarine conditions, the SLAMM could be utilized as part of subsequent Tier 2 and Tier 3 analyses.

VIII.3. Conceptual Designs for Risk Management Measures

Table VIII-4 summarizes the design criteria developed by the team for coastal storm risk management measures as part of the Framework. Generally, structural measures (e.g., beach restoration, levees, etc.) were assumed to be designed to the 1 percent flood elevation plus a 3-foot allowance to account for future sea level change. This 3-foot allowance is consistent with the USACE high scenario for projected sea level change by year 2068. Storm surge barriers were assumed to be designed to a higher storm tide level corresponding to a 0.2 percent flood elevation, also consistent with typical design standards, plus the same 3-foot sea level change allowance.

Table VIII-4. Criteria for Conceptual Design of NACCS Risk Reduction Measures

Measure Type	Criteria
Structural (not barriers) [1]	1 percent flood elevation + 3-foot sea level change allowance
Storm Surge Barriers	0.2 percent flood elevation + 3-foot sea level change allowance
Natural and Nature-Based Features	10 percent flood elevation
Non-structural (Floodproofing and Buyouts)	1 percent flood elevation + 3-foot sea level change allowance

[1] *Beaches and dunes are also considered Natural and Nature-Based Features.*

NNBF are not typically designed to provide significant risk reduction against storm tides. In fact, most of these measures allow for the storm tide and waves to propagate over or through the nature-based feature with minimum damage to it. This characteristic is what makes nature-based measures resilient but also inherently limits their ability to reduce coastal storm risks. For the purposes of this study, all nature-based features (e.g., living shorelines, wetlands, etc.) were assumed to be designed to provide risk reduction against the 10 percent flood. This design level may be high for some specific nature-based measures and low for others depending on specific site conditions and actual design details. For the NACCS evaluations, NNBF were assumed to provide risk reduction to the current 10 percent flood without an additional sea level change allowance. The assumption is that natural or managed adaptation processes would maintain the 10 percent flood design level as sea level changes over the life of the project. Site-specific conditions and combinations of site-specific NNBF, including break offshore waves, wave energy attenuation, slow inland water transfer, etc., would change the risk reduction performance (USACE, 2013).

Buildings are typically elevated (non-structural measure) to the FEMA-mandated 1 foot above the base flood elevation (BFE). However, many coastal communities have, or are enacting, more stringent elevation requirements of up to 3 feet above the BFE as a result of the magnitude and impact of Hurricane Sandy, and the uncertainty regarding the rate of sea level change. Therefore, for the purposes of this analysis, the more conservative requirement of 3 feet above the BFE was used as the non-structural design elevation.

The Hurricane Sandy Rebuilding Task Force announced on April 4, 2013 that all Sandy-related rebuilding projects funded by PL 113-2 must meet a single uniform flood risk reduction standard (FRRS) of one foot above the best available and most recent base flood elevation (BFE) information provided by FEMA, unless local standards are more restrictive. The NACCS incorporates this FRRS as part of the 1 percent flood plus three feet.

The design criteria identified in Table VIII-4 shows the coastal storm risk reduction levels that were assigned to measures. These design criteria are suggested design levels and actual risk reduction levels may vary depending upon site specific conditions. General benefits, impacts, and other considerations associated with the management measures were identified as well. Site specific evaluations as part of Tier 2 and Tier 3 analyses would refine impacts, particularly as they relate to social and environmental impacts and especially if a decision document that requires a NEPA environmental assessment or environmental impact statement.

VIII.3.1. Parametric Unit Cost Estimates

As part of the NACCS, conceptual design and parametric cost estimates were developed for the various coastal storm risk management measures for the NACCS. Initial, representative, concept designs have been developed for each measure together with quantities and parametric unit costs (typically per linear foot of shoreline) based on a combination of available cost information for existing projects and bottom-up estimates. The latter are based on quantity takeoffs for typical design sections and representative unit costs for all construction items (e.g., excavation, fill, rock, plantings) based on historical observations. Additionally, the parametric unit cost estimates, or total opportunity costs, are the total costs of the management measures per unit (linear foot or acre) derived from construction costs (which include assumptions for design) and operation and maintenance costs. Project timeframes represent a 50-year project life, unless otherwise noted. Assumptions associated with the parametric unit costs including the basis of parametric construction costs as well as operations and maintenance costs are included in the conceptual description of the management measures.

Initial conceptual designs used to estimate quantities and costs are representative of typical conditions in the study area and do not account for reach or site-specific variations in ground level, tidal range, or storm water levels. Furthermore, real estate costs were not included in the development of the parametric unit costs because no project recommendations identifying a specific location where various real properties would be affected were made. Real estate costs are so widely variable within the NACCS study area that they would cloud the information regarding the relative cost of the engineering measures available to reduce storm damages. As part of the NACCS framework Tier 1 assessment an initial screening of potentially applicable measures for each risk area is performed considering shoreline types and the estimated reduction in vulnerability for a given cost. In the Tier 2 of the NACCS framework, the designs and associated costs were adjusted for variability in relevant design parameters, including local design water levels (e.g., FEMA BFE). In addition, future parametric cost estimates adjustments will account for regional differences in the price of materials and transportation costs within the study area, as well as real estate lands, easements, rights-of-way (LER). A brief description of the measures considered by aggregated categories provided in the following paragraphs.

VIII.4. Non-Structural Measures

As listed in Table V111-1, Non-structural measures fall into four groups: (1) Acquisition/ Removal or relocation of structures from the risk; (2)retrofit measures, (3) warning systems and evacuation procedures to alert residents and implement plans to evacuate cultural resources to increased storm risks and facilitate easier evacuation from risk-prone areas, and (4) flood insurance and Land use Management/zoning. Non-structural measures falling in the first two categories typically reduce the potential for storm damage to a structure; however, risks to the surrounding property, vehicles, and emergency access are not reduced and property owners should evacuate vulnerable properties during storm events lest they become trapped.

Figure VIII-2. Typical Elevated Shorefront Structure (Courtesy: FEMA)

VIII.4.1 Acquisition/Building Removal or Relocation

Buildings may be removed from vulnerable areas by acquisition (buy-out), subsequent demolition, and relocation of the residents. Often considered a drastic approach to storm damage reduction, property acquisition and structure removal are usually associated with frequently damaged structures. Implementation of other measures may be effective but if a structure is subject to repeated storm damage, this measure may represent the best alternative to eliminating risks to the property and residents.

Costs for structure removal are estimated to be $70,000, in addition to the property purchase price. When acquiring properties, the government typically offers fair market value for a property.

This sub-category also includes moving a structure out of the vulnerable area, either within the same property boundaries or to another property. While often a costly endeavor, it may be applicable to structures subject to severe risk, but due to available space and structure value do not warrant demolition.

Costs for this category vary significantly from region-to-region, from coastal to inland communities, by the distance a structure may be moved, etc. Unlike relocation, removal of a structure requires acquisition of the entire property, demolition of the structure, removal of debris, excavation of underground utilities (if warranted), and restoration of the site to natural conditions. Acquired properties are usually deed restricted from further development.

VIII.4.2 Building Retrofit

Building retrofit measures include dry flood proofing or elevation of a structure. Dry floodproofing involves sealing flood prone structures from water with door and window barriers, small scale rapid deployable floodwalls, ring walls, or sealants. Elevation of structures is usually limited to residential structures or small commercial buildings. Whether a structure may be elevated depends on a number of factors including the foundation type, wall type, size of the structure, condition, etc.

Costs can vary significantly depending on those factors. However, fixed costs per structure include engineering and design, administrative fees, temporary housing for inhabitants, etc. As shown in Table VIII-5, elevation of a typical 1,400 square foot structure could cost up to $195,000.

Table VIII-5. Elevation (bldg. retrofit) - Construction Quantities & Costs

	Quantity		Parametric Estimate	
Item	Number	Unit	Unit Cost	Total Cost
Elevation 8 feet	1	ea	$122,600	$122,600
Temporary rehousing	1	ea	$10,000	$10,000
Subtotal				$132,600
Contingency	25%			$33,150
Total Construction				$165,750
E&D	$10,000			$10,000
S&A	10%			$16,575
Total Estimated First Construction Cost				$192,325
Annualized First Costs				$8,200
O&M	N/A			$0
Total Estimated Annual Average Cost				$8,200

Dry floodproofing of homes is technically feasible for flood depths of up to three feet. However, this significantly limits the level of effectiveness of floodproofing in reducing vulnerability. It is important to note that FEMA generally does not endorse floodproofing of residences and there are no reductions in flood insurance premiums for floodproofed homes.

Ring walls or ring levees are most often used for large commercial/industrial structures or multi-family/apartment buildings that cannot be elevated. Figure VIII-3 shows a small ring wall constructed around a garden apartment building. Ring walls require drainage outfalls or pumps to discharge runoff collected behind the wall, and gates for access and egress.

Figure VIII-3. Typical Apartment Ringwall

Sealing a structure could cost up to $100,000 for a 1,000 square foot structure; however, damage reduction is limited to a maximum of 3 feet due to potential hydrostatic pressure on the structures. A separate, 2,000 ringwall around a vulnerable structure would cost up to $4.8 million as shown in Table VIII-6.

Table VIII-6. Ringwall (Industrial Structure) - Construction Quantities & Costs

	Quantity		Parametric Estimate	
Item	Number	Unit	Unit Cost	Total Cost
Floodproof	1	ea	$2,861,332	$2,861,332
Roller gates	3	ea	$104,000	$312,000
Subtotal				$3,173,332
Contingency	25%			$793,333
Total Construction				$3,966,665
E&D	12%			$476,000
S&A	10%			$396,666
Total Estimated First Construction Cost				$4,839,331
Annualized First Costs				$206,319
O&M	N/A			$0
Total Estimated Annual Average Cost				$206,319

VIII.4.3 Flood Warning Systems and Evacuation

Flood warning systems and evacuation planning are applicable to vulnerable areas. Despite improved tracking and forecasting techniques, the uncertainty associated with the size of a storm, the path, or its duration necessitate that warnings be issued as early as possible. Evacuation planning is imperative for areas with limited access, such barrier islands, high density housing areas, elderly population centers, cultural resources, and areas with limited transportation options.

VIII.4.4 Flood Insurance

While not often thought of as a means of addressing vulnerable areas, adequate flood insurance is closely tied to effective flood warning systems and evacuation planning for a number of reasons:

(1) Residents that are uncertain about reducing risk to their belongings may be prone to attempt to remain in vulnerable areas during storm events, creating further risk. Knowing that personal property is insured, residents may be more comfortable with evacuating vulnerable areas at the approach of a storm.

(2) Flood insurance rates and regulations directly and indirectly impact property owners' decisions to reduce risk to their property though favorable construction practices. For instance, if a property owner in a vulnerable area makes an improvement to their structure, FEMA, the administrator of the NFIP, mandates that the improvement be constructed in accordance with FEMA regulations and if the improvement is warranted to be substantial (greater than 50% of the value of the structure), the unimproved portion of the structure must be improved to meet FEMA regulations (that is, less risk-prone).

(3) Community participation in the NFIP is conditional on meeting program guidelines. Participating communities must manage development within their floodplains in accordance with FEMA standards or risk removal from the program, which risks cancellation of all flood insurance policies within the community. Therefore, proper management of development and associated risk and vulnerability helps ensure the best possible flood insurance rates. Officials can help to further reduce flood insurance rates within their communities through the NFIP's Community Rating System. Reduced premium rates will make policies more attractive to uninsured residents, resulting in more complete coverage within a vulnerable community.

(4) Communities participating in the NFIP that are proactive in promoting floodplain management, flood risk awareness, etc. may help to further reduce the insurance costs to property owners through the NFIP's Community Rating System (CRS). Under the CRS, flood insurance premium rates are discounted to reward community actions that meet the three goals of the CRS, which are: (1) reduce flood damage to insurable property; (2) strengthen and support the insurance aspects of the NFIP; and (3) encourage a comprehensive approach to floodplain management.

The CRS uses a class rating system that is similar to fire insurance rating to determine flood insurance premium reductions for residents. CRS classes are rated from 10 to 1. As a community engages in additional mitigation activities, its residents become eligible for increased NFIP policy premium discounts. Each CRS Class improvement produces a 5 percent greater discount on flood insurance premiums for properties in the SFHA, with a Class 1 community receiving the maximum 45 percent premium reduction.

VIII.5. Structural Measures

As listed in Table VIII-1, the Structural Measures include Deployable floodwalls, Floodwalls, Dikes and levees, shoreline stabilization and Storm Surge Barriers.

Figure VIII-4. Rapid Deployment Floodwall (Courtesy: Plainschase.com)

VIII.5.1 Deployable Floodwalls

Description

Rapid Deployment Floodwalls (RDFWs) are structures that are temporarily erected along the banks of a river or estuary, or in the path of floodwaters to prevent water from reaching the area behind the structure. After the storm or flood, the structures are removed. This category also includes permanently installed, deployable flood barriers that rise into position during flooding due to the buoyancy of the barrier material and hydrostatic pressure. Some systems, such as stop logs, require a permanent base or footing, while others may be deployed without a base. Structural base components contribute to the overall effectiveness and level of risk management that an RDFW can provide. Figure VIII-4 shows an example of a stop log temporary floodwall.

Temporary measures like these are particularly useful for risk management in smaller areas, and are usually considered for areas where access to the waterfront is essential to the economy or character of a community. Often, traditional floodwalls, or levees are used to reduce risk to some portions of the waterfront, with intermittent closure structures like a RDFW. RDFWs provide the same benefits as similarly sized static floodwalls or levees, but height of the structure is somewhat limited.

The successful performance of RDFWs hinges on advance flood warning. Advance warning is needed prior to deployment to facilitate transportation and assembly. Therefore, use of RDFWs is not appropriate in areas subject to flooding shortly after a rain or storm event. Stop logs must be stored close nearby, typically in a separate, dedicated facility, and must be transported to the deployment site. Because of the relatively high cost to assemble, disassemble and store the RDFW, they are not desirable in areas of frequent flooding.

The wall width, distance between stationary anchors, and the use of bracing (shown in Figure VIII-4) limit the height that a wall may be constructed to. In some areas, RDFWs may be subject to minor wave action with proper construction.

Despite the limitations due to the effective level of risk management, storage and deployment requirements, and required personnel training, RDFWs are often a welcomed solution to providing flood risk management to areas with limited available real estate for permanent structural flood risk management measures and/or with valuable viewsheds, which would be impacted by permanent structural measures. RDFWs may be appropriate for implementation on rocky coasts, beaches, estuaries/lagoons, and urban shorelines.

Generic Design

A representative typical cross-section of a RDFW includes base or anchor plates, stanchions, gasketed stop logs, and bracing, if needed. The typical wall is 8 inches thick and 6 feet in height, which is the maximum height before bracing may be required. It is assumed that the typical application is not subject to wave action. Deployment of an RDFW requires training and practice, and maintenance of static foundations or bases and the deployable logs is required to ensure easy assembly when needed.

Parametric Costs

The cost estimate for the Rapid Deployment Floodwalls is shown in Table VIII-7, which provides first construction and annualized costs including operation and maintenance (O&M) costs. The costs were developed for a wall length of one mile and reduced to provide a cost per linear foot of RDFW. First construction costs are about $5,454 per linear foot of RDFW; annualized costs based on an interest rate of 3.5% and a 50-year project life are about $247 per linear foot. Maintenance of RDFW static foundations and the deployable stop logs is required to ensure easy assembly. Annual maintenance costs are assumed to be minimal and are not significant in the overall costs.

Table VIII-7. RDFW - Construction Quantities & Costs

	Quantity		Parametric Estimate	
Item	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$200,000	$200,000
Deployable Floodwall	1	1 Mile	$10,780,000	$10,780,000
Floodwall Construction	1	1 Mile	$6,471,035	$6,471,035
Stoplog Storage	1	ea	$445,000	$445,000
Drainage Outlets	13	ea		$988,000
Subtotal Construction				$18,884,035
Contingency	25%			$4,721,009
Total Construction				$23,605,043
E&D	12%			$2,832,605
S&A	10%			$2,360,504
Total Estimated First Construction Cost				$28,798,153
Total Estimated First Construction Cost per Foot				$5,454
Annualized First Costs				$233
O&M	$2/LF + $10,000 per drainage structure			$9
O&M	Install/Dismantle Deployable Wall			$5
Total Estimated Annual Average Cost				$247

Summary: Deployable Floodwalls Benefits, Impacts and other Considerations

While deployable floodwalls can generally be rapidly deployed prior to a predicted flooding condition, considerations needs to be given to the level of risk management required, ease of deployment and recovery, cost and ground disruption during construction, and where contained water will end up going.

Figure VIII-5. Typical Floodwall Construction

VIII.5.2 Floodwalls

Description

Floodwalls are structures used to reduce risk in relatively small areas or areas with limited space for flood risk management against lower levels of flooding. They can be similar to seawalls and are usually constructed from concrete. Unlike wider, more stable levees, narrow floodwalls require significant reinforcement and anchoring construction to prevent collapse from hydrostatic pressure. The significant amounts of steel sheeting and/or reinforced concrete used in constructing a typical wall make the feature extremely heavy. Because construction in a flood prone area, such as near a river or estuary, may occur on soft organic soil, pile reinforcement may be required under the base of the wall. The combination of steel sheeting, reinforcement, concrete, and pile support make a floodwall a much more costly structural risk management measure than a similar length and height levee. A typical floodwall is shown in Figure VIII-5. These structures might also require closure structures including stoplogs, miter gates, swing gates, or roller gates, which were not included in the development of the parametric unit cost estimate. A simple steel sheetpile I-wall may be more economical.

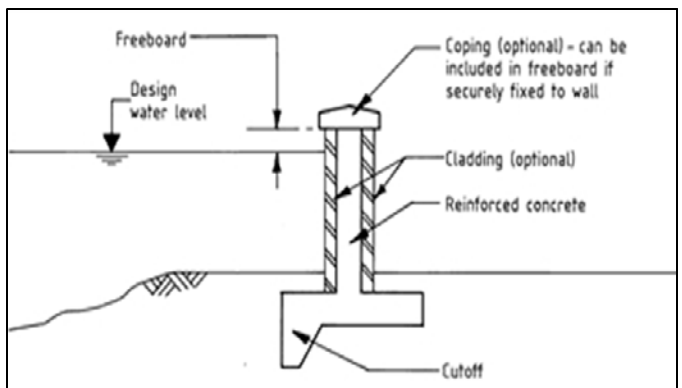

Figure VIII-6. Representative Floodwall Cross-section ("T"-wall)

Generic Design

A representative typical cross-section of a floodwall with a base ("T" wall, due to its shape) is shown in Figure VIII-6. Not shown in this figure are piles within the foundation. For areas where soils provide a poor foundation, the T-wall would be supported by up to 50-foot long piles every 7 feet along the wall. For areas with better foundations but still requiring piles, the wall would be supported by up to 15-foot long piles every 7 feet along the floodwall. The typical wall is 2.5 feet thick.

Parametric Unit Costs

Costs, shown in Table VIII-8 were developed for T-walls of 6 to 16 feet high. For estimating purposes, the costs are based on the weighted average between the particular wall height on a poor foundation (50-foot piles) and a good foundation (15-foot piles). The cost of drainage gates/outlet structures every 400 feet along the length of the floodwall were considered in the cost of the structures.

For a 10 foot high floodwall construction, first construction costs are about $5,335 per linear foot; annualized costs are about $237 per linear foot. Operation and maintenance actions for floodwalls were assumed to be limited to periodic inspections and clearance of debris from outlet structures.

Table VIII-8. Floodwalls- Construction Quantities & Costs

	Quantity		Parametric Estimate	
Item	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$200,000	$200,000
Floodwall Construction	1	Mile	$17,284,524	$17,284,524
Drainage Outlets	13	ea		$988,000
Subtotal Construction				$18,472,524
Contingency	25%			$4,618,131
Total Construction				$23,090,655
E&D	12%			$2,770,879
S&A	10%			$2,309,065
Total Estimated First Construction Cost				$28,170,599
Total Estimated First Construction Cost per Foot				$5,335
Annualized First Costs				$227
O&M	$2/LF + $10,000 per drainage structure			$9
Total Estimated Annual Average Cost				$237

Summary: Floodwalls Benefits, Impacts and other Considerations

Permanent floodwalls reduce risk in a specific area from high water during storm events, but are costly, can require significant require land/real estate, may impact scenic views, and may impact habitat.

Floodwall considerations include level of risk management that is required, construction and real estate acquisition costs, how to deal with contained water, and ground disruption during construction.

VIII.5.3 Levees and Dikes

Description

Levees and dikes are embankments constructed along a waterfront to prevent flooding in relatively large areas. They are typically constructed by compacting soil into a large berm that is wide at the base and tapers toward the top, as shown in Table VIII-7. Grass or some other type of non-woody vegetation is usually planted on the levee/dike to add stability to the structure. If a levee or dike is located in an erosive shoreline environment, revetments may be needed on the waterfront side to reduce impacts from erosion, or in cases of extreme conditions, the dike face may be constructed entirely of rock.

Figure VIII-7. Typical Levee Construction

Levees may be constructed in urban areas or coastal areas; however, large tracts of real estate are usually required due to the levee width and required setbacks. The height and width usually limit access to the water for recreation and commercial activities, and like floodwalls, impact the view shed of coastal properties. In some cases levees have been incorporated into trail systems and frequently include amenities such as benches, street lighting and jogging paths. Structural measures, such as floodwalls, levees and dikes tend to trap rainfall runoff associated with storms on the landward side, creating a residual flooding risk. To reduce this residual risk, gravity outlets are installed along the length of the structure. In cases where significant runoff may be trapped behind the structure, ponding areas and pump stations are required. Depending on the density of development of a vulnerable area, levees and floodwalls are often constructed as a system whereby floodwalls are interspersed between levee segments as available property space dictates. Figure VIII-8 shows a levee/floodwall system before and during Hurricane Irene flooding in 2011. The floodwall section was constructed along the line of risk management behind a large commercial structure.

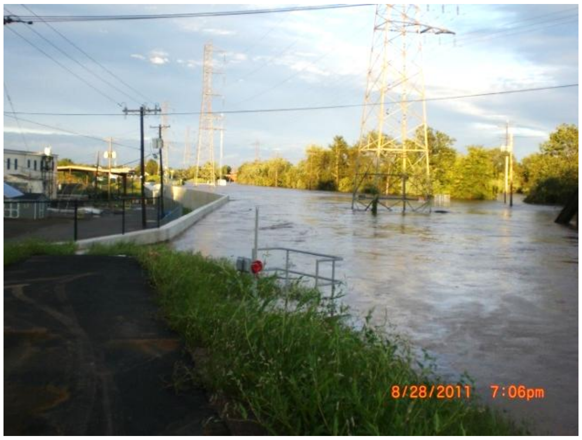

Figure VIII-8. Levee and Floodwall System, Bound Brook, NJ, before and after

If properly maintained, floodwalls, levees, and dikes are highly effective methods of flood risk management. However, if the design level of risk management is exceeded, water will overtop the structure, trapping floodwater behind it and risking erosion and failure of the feature.

Generic Design

Designs and costs were developed for levees of 6 to 16 feet high. Levees on poor foundations are subject to instability and settling, and therefore, require deeper excavation prior to construction. To account for this, the parametric cost was developed based on a weighted average of levees on poor and good foundations. The costs of drainage gates/outlet structures, which are assumed to be placed every 400 feet along the length of the structure, are considered within the cost of the structures. A typical levee section is shown in Figure VIII-9.

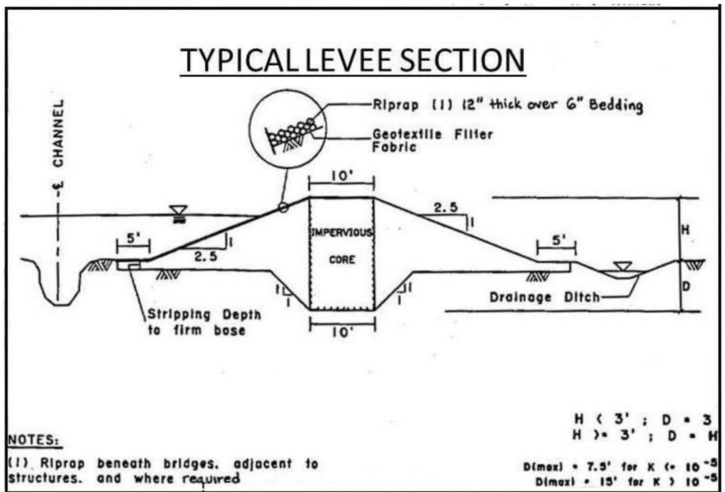

Figure VIII-9. Typical Levee Section

Parametric Unit Costs

For levee construction, first construction costs are about $1,578 per linear foot; annualized costs are about $77 per linear foot (Table VIII-9). Operation and maintenance actions for levees were also assumed to be limited to periodic inspections and clearance of debris from outlet structures. Costs for pump station maintenance would be significantly more but are site specific and were not considered in the parametric cost development.

Table VIII-9. Levee - Construction Quantities & Costs

	Quantity		Parametric Estimate	
Item	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$200,000	$200,000
Levee Construction	1	Mile	$4,744,478	$4,744,478
Drainage Outlets	13	ea	$40,000	$520,000
Subtotal				$5,464,478
Contingency	25%			$1,366,120
Total Construction				$6,830,598
E&D	12%			$819,672
S&A	10%			$683,060
Total Estimated First Construction Cost				$8,333,329
Total Estimated First Construction Cost per Foot				$1,578
Annualized First Costs				$67
O&M	$2/LF + $10,000 per drainage structure		$9	$9
Total Estimated Annual Average Cost			$80	$77

Summary: Levees and Dikes Benefits, Impacts and other Considerations

Similar to floodwalls, levees and dikes reduce risk to a specific area from high water during storm events, but are costly, can require significant require land/real estate, may impact scenic views, and may impact habitat.

VIII.5.4 Shoreline Stabilization

Description

Structures are often needed along shorelines to provide risk reduction from wave action or to stabilize and retain in situ soil or fill. Vertical structures are classified as either seawalls or bulkheads, according to their function, while protective materials laid on slopes are called revetments (USACE 1995). A bulkhead is primarily intended to retain or prevent sliding of the land, while reducing the impact of wave action is of secondary importance. Seawalls, on the other hand, are typically more massive structures whose primary purpose is interception of waves and reduction of wave-induced overtopping and flooding of the land structures behind. Note that under this definition seawalls do not include structures with the principal function of reducing risk to low-lying coastal areas. In those cases a high, impermeable, armored structure known as a sea dike is typically required to prevent coastal flooding (USACE 2002).

Revetments are onshore structures with the principal function of reducing the impacts to the shoreline from erosion and typically consist of a cladding of stone, concrete, or asphalt to armor sloping natural shoreline profiles (USACE 2002). They consist of an armor layer, filter layer(s), and toe protection. The armor layer may be a random mass of stone or concrete rubble or a well-ordered array of structural elements that interlock to form a geometric pattern. The filter assures drainage and retention of the underlying soil. Filter-type structures such as stone revetments are preferable where groundwater is part of the erosion process. Toe protection is needed to provide stability against undermining at the bottom of the structure (USACE 1995). Figure VIII-10 shows an example of a revetment at Poplar Island in Chesapeake Bay, MD (USACE 2002).

Figure VIII-10. Revetment at Poplar Island, MD

Bulkheads may be either cantilevered or anchored (like sheetpiling) or gravity structures (such as rock-filled timber cribs). Their use is limited to those areas where wave action can be resisted by such materials. In areas of intense wave action, massive concrete seawalls are generally required. These may have either vertical, concave, or stepped seaward faces (USACE 1995).

Revetments, bulkheads, and seawalls mainly reduce risk only the upland area behind them. All share the disadvantage of being potential wave reflectors that can erode a beach fronting the structure. This problem is most prevalent for vertical structures that are nearly perfect wave reflectors such as bulkheads and seawalls and is progressively less prevalent for curved, stepped, and rough inclined structures such as revetments that absorb or dissipate increasing amounts of wave energy (USACE 1995). Shoreline stabilization measures like those discussed in this section are appropriate for implementation on scarps and vegetated low banks along interior shorelines. It is assumed that existing man-made shorelines already include some form of shoreline stabilization/protection measure such as a riprap revetment, bulkhead or seawall.

Generic Design

Site-specific shoreline bank geometry, adjacent water depths, soil conditions, currents, waves, as well as other physical, environmental and economic factors will typically dictate the choice of shoreline stabilization/protection measure, i.e., vertical (bulkhead or seawall) vs. sloped (revetment). However, given the regional scale of the study it is impossible to account for these local, site-specific, conditions to determine which measure is most appropriate at each location. Therefore, for the purposes of regional framework development, a rock revetment is assumed as the standard shore stabilization/protection solution.

The principal components of a coastal revetment include:

- Protective rock armor & underlayer rock
- Toe elevation and protection
- Crest height
- Berm (if included)

The protective rock armor serves to hold the revetment in place and is often comprised of several layers of rock. Toe protection is normally an integral part of the revetment structure and is designed to prevent that structural component from undermining as a result of wave and/or current-induced scour. In some cases a revetment will be protected with concrete units rather than rock. A berm may or may not be included in the dike cross section. Where included, a berm can be used to limit wave runup and overtopping. The berm may also minimize the armoring requirements for the revetment and upper slope of the structure. Roadways or pathways are often included on or adjacent to revetments in order to provide access to hinterland areas and access for repairs to the revetments.

The generic revetment geometry used for the present work is comprised of toe protection, rock armor units (i.e. the seaward slope) and a short horizontal crest also comprised of rock. One of the more important variables of the dike 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 is assumed that the revetment is founded on reasonably competent soils which do not require foundation/ground improvements. Owing to the wide range of conditions within the study area, a number of other assumptions have been made to develop a generic revetment design as noted below:

- Revetments are only applicable to estuarial environments as distinct from open ocean environments
- Design waves conditions are characterized by a significant wave height, Hs, of 6 ft and a peak spectral period, Tp, of 6 seconds. These waves are considered representative of 1 percent annual chance of the design being exceeded design conditions in interior shorelines not exposed to ocean waves. In some locations the design wave will be controlled by exposure to ship wake and in others by locally generated wind-waves. Either way it is assumed that significant waves will not be larger than 6 ft.
- Local tidal conditions are used to size a revetment; calculations have been made for the tidal ranges (MLLW to MHHW) that typify NAD which range from 1-10 feet. The generic design, quantities and costs presented in this section are based on an average tidal range of 4 ft (Figure VIII-11).

- Crest elevation of the revetment is 6 feet above MHHW. This is considered a typical elevation for revetments and other structures used for shoreline stabilization in the study area. Flood risk reduction benefits associated with this elevation will depend on actual exposure to waves and related runup and overtopping, as well as storm surge elevations and could vary from 10 to 1 percent design level. A 10 percent annual chance of the design being exceeded is assumed given that revetments are typically constructed to armor and reduce impacts of erosion to a shoreline and not necessarily to reduce upland flooding.
- Bottom elevation of the revetment is 5 below MLLW. Actual elevations will vary widely across the study area, but this considered reasonable elevation for revetments along interior estuarine shorelines that are not directly adjacent to deep water areas such as navigation channels.
- Stone density is 165 lbs/ft^3
- Structure slope is 2 (Horizontal):1 (Vertical)
- Van der Meer's equations were used to size the revetment armor rock.

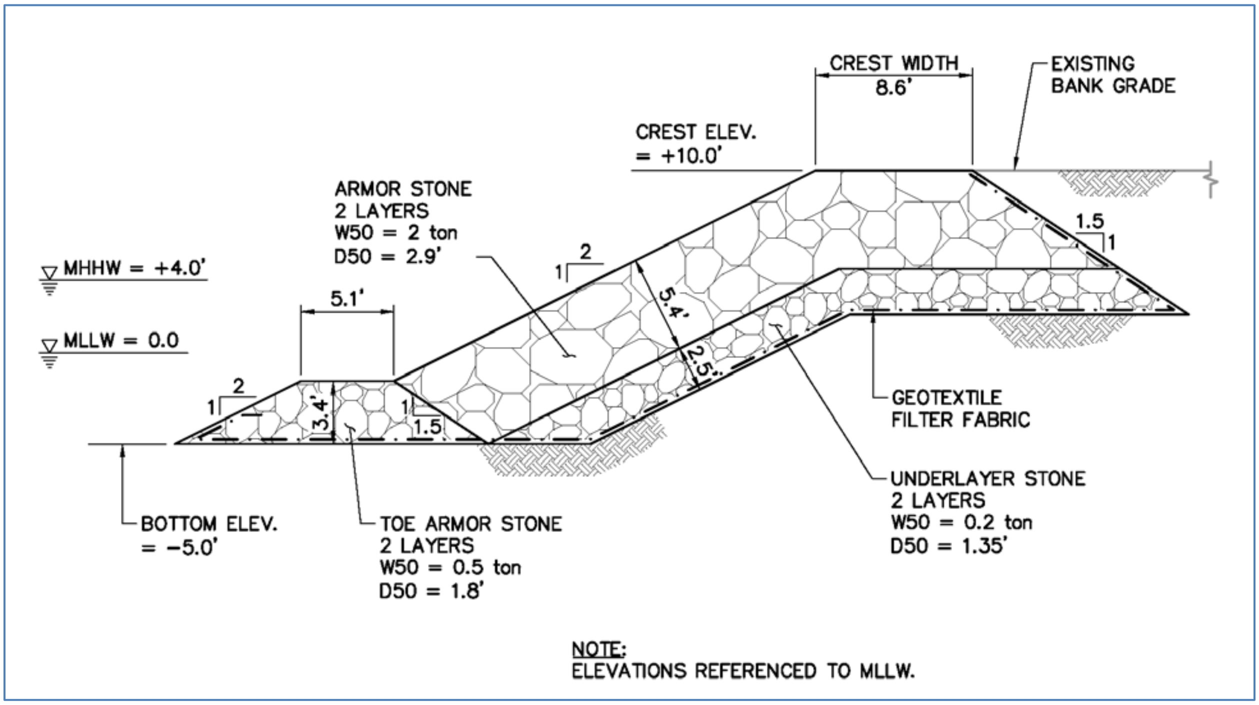

Figure VIII-11. Typical Section of a Rock Revetment

Parametric Unit Costs

A project length of 5,000 feet is used to determine total volumetric quantities required. Unit costs of $150 per ton of stone and $15 per sq.yd. of geotextile are applied in the parametric cost estimate. Cost estimates include 12% for engineering and design (E&D), 10% for construction management (S&A), and 1% for operation and maintenance (O&M). A contingency of 25% is applied to the cost estimate. Real estate costs associated with potential ocean front structure acquisitions/relocations, and easements are not included in the parametric cost estimate. Table VIII-10 provides a summary of first construction and annualized costs. Total annual costs are estimated using a 50-year project life and annual interest rate of 3.5%. The total estimated annual average cost is $263 per foot.

Table VIII-10. Revetment - Construction Quantities & Costs

Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$200,000	$200,000
Armor Stone	62,745	ton	$150	$9,411,750
Underlayer	26,335	ton	$150	$3,950,250
Toe Armor	11,085	ton	$150	$1,662,750
Geotextile	37,865	sq.yd.	$15	$567,975
Subtotal				$15,792,725
Contingency	25%			$3,948,181
Total Construction				$19,740,906
E&D	12%			$2,368,909
S&A	10%			$1,974,091
Total Estimated First Construction Cost				$24,083,906
Total Estimated First Construction Cost per Foot				$4,817
Annualized First Costs				$205
O&M	1%			$48
Total Estimated Annual Average Cost				$254

Summary: Shoreline Stabilization Benefits, Impacts and other Considerations

Erosion control methods such as stone revetments, gabions, bulkheads and rip-rap may be employed to reduce risk to beach areas, wetlands or other sensitive areas from wave energy and floodwaters.

Gabion baskets corrode quickly in salt water applications. Structures require maintenance and may require reinforcement measures if erosion occurs in front of the structure. Level of risk management and integration with other similar nearby structures should be considered during the design phase.

Figure VIII-12. Fox Point Storm Surge Barrier, Providence RI (Source: Providence Journal)

VIII.5.5 Storm Surge Barriers

Description

Storm surge barriers reduce risk to estuaries against storm surge flooding and waves. In most cases the barrier consists of a series of movable gates that stay open under normal conditions to let the flow pass but are closed when storm surges are expected to exceed a certain level. The gates are sliding or rotating steel constructions supported in most cases by concrete structures on pile foundations (USACE 2002). Storm surge barriers are often chosen as a preferred alternative to close off estuaries and reduce the required length of flood risk management measures behind the barriers. Another important characteristic is that they are often (partly) opened during normal conditions to allow for navigation and saltwater exchange with the estuarine areas landward of the barrier. Nonetheless, storm surge barriers could have negative effects on the ecological system and on navigation. Famous examples are the storm surge barriers in The Netherlands in the southwest of the country (Jonkman et al 2013). In New Orleans, several storm surge barriers have been built after Hurricane Katrina (2005) to reduce risk to the city from surges and reduce the length of the directly exposed system. Figure VIII-12 shows an example of a storm surge barrier at Fox Point, Providence, RI.

Storm surge barriers range in scale from small/local gates reducing risk to a small coastal inlet to very large barrier "systems" reducing risk to a large estuary or bay and consist of a series of coastal dikes, gates, and in some cases navigation locks. Both are usually combined with other flood risk reduction measures such as levees and floodwalls. Designs that allow for navigation are important in port areas. The applicability of storm surge barriers cannot be determined based on shoreline type; it depends on other factors such as coastal geography, development density, physical and environmental conditions, etc.

Parametric Unit Costs

Potential sites for storm surge barriers include the following:

- Embayments characterized by relatively high development (such are needed to provide benefits to offset the relatively high costs of the barriers)
- Embayments with reasonably narrow entrances and therefore lower relative costs
- Some preference was also given to existing harbors featuring navigation channels

A list of candidate sites based on these considerations is provided in Table VIII-11. For the purposes of this discussion, engineering, economic, environmental, etc. constraints are not considered even though it is fully acknowledged that in most cases, some or all of these concerns would make actual implementation impossible. The goal is to provide enough information to be able to make a relative comparison to other coastal flood risk management strategies including local structural, natural and nature-based, and non-structural measures.

Storm surge barriers have not been built extensively throughout the world for a variety of reasons:

- Barriers are expensive and best applied to densely populated and low areas where damage costs from flooding are sufficient high to justify the barrier costs
- Barriers can have problematic impacts on the environment particularly when the barriers significantly change the tidal hydraulics of a natural estuarial basin.
- Barriers can complicate and/or compromise shipping

A construction cost estimate based on the actual design of a storm surge barrier for each location considered is well beyond the scope of this study. This would require knowing the general characteristics and dimensions of each component, including dikes, closure structures, gates, gate monoliths, etc. which would require a significant amount of additional study and design work.

Therefore, for this study an approach has been chosen which considers the actual construction costs of several storm surge barriers in various countries around the world. De Ridder (1996) developed a methodology for analyzing the capital costs of storm surge barriers. This approach involved three correlating construction costs to the combination of three variables: barrier width, barrier total height, and head (water differential) acting on the barrier. This methodology has also been used by other authors for similar conceptual level studies: Dircke et al (2012), Van Ledden et al (2012), and Jonkman et al (2013). Construction costs and relevant variables were collected for a number of storm surge barriers (Van Ledden et al 2012). These costs have been escalated to a price level of 2013 using the Civil Works Construction Cost System Index and are listed in Table VIII-11 plots the data in Figure VIII-13 and shows that there is very strong correlation between volume (height x head x width) and cost. For the purposes of this study, the average value of $32,200 per cubic meter or $912 per cubic foot (see Table VIII-11) was used to estimate cost of storm surge barriers within the study area.

Table VIII-11. Dimensions and costs for storm surge barriers around the world

Name, Country	Type	Year	Width (m)	Height (m)	Head (m)	Vol (m3)	FY13 costs (x $Million)	FY13 costs (x $1,000/ m3)
Ems Barrier, Germany	Sector gate	1980	360	8.5	3.8	11,628	566	49
Thames Barrier, UK	Sector	1980	530	17	7.2	64,872	2,229	31
Eastern Scheldt Barrier, NL	Lifting gates	1986	2400	14	5	168,000	6,185	33
Maeslant Barrier, NL	Floating gate	1991	360	22	5	39,600	1,009	23
Hartel Barrier, NL	Lifting gates	1991	170	9.3	5.5	8,696	220	23
Ramspol, NL	Bellow barrier	1996	240	8.2	4.4	8,659	203	21
Seabrook barrier, USA	Sector gates	2010	130	8	4	4,160	176	38
IHNC barrier, USA	Sector gates	2010	250	12	6	18,000	797	40
Average								32.2

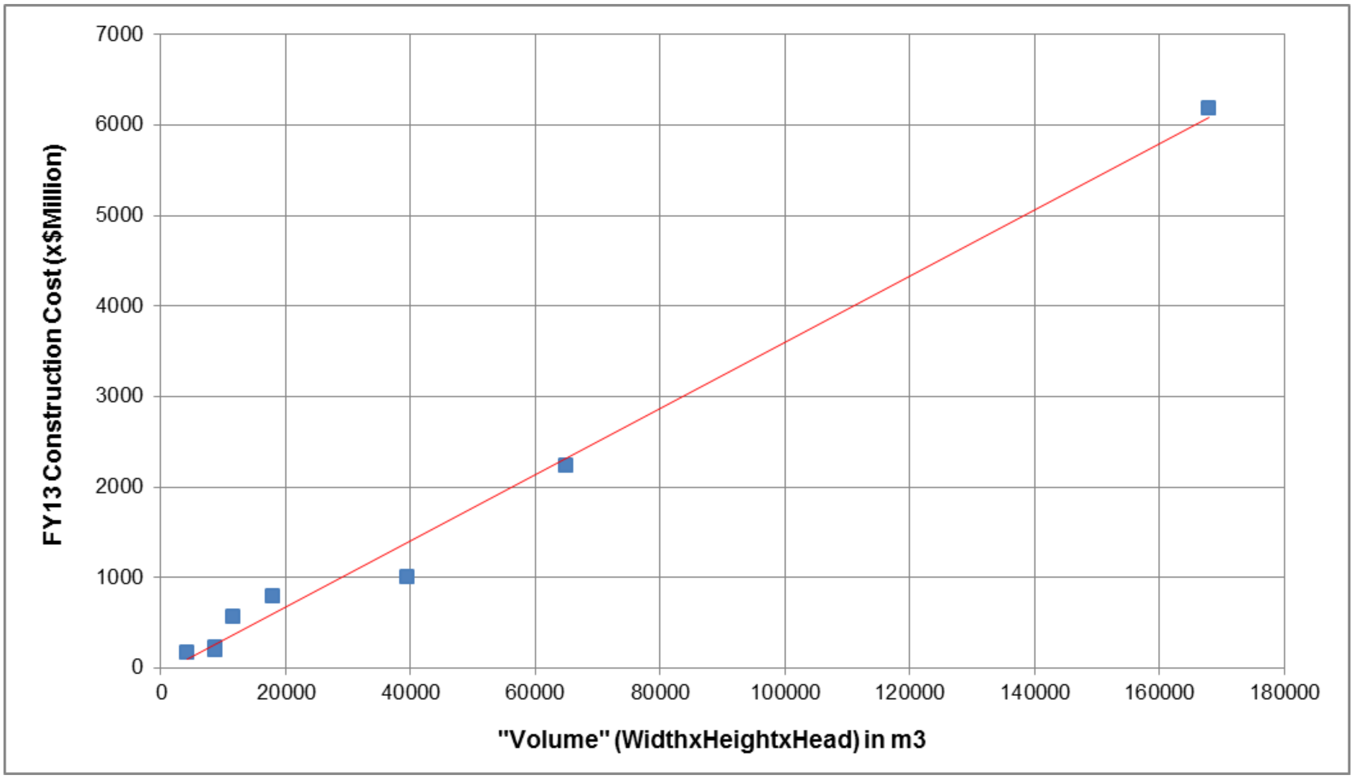

Figure VIII-13. Correlation between storm surge barrier "volume" and cost

In addition to the construction costs based on this empirical correlation, parametric cost estimates include 12% for engineering and design (E&D) and 10% for construction management (S&A). A contingency of 25% is applied to the cost estimate. Real estate costs associated with structure acquisitions/relocations, and easements vary considerable by project and are not included in the parametric cost estimate. Operation and maintenance costs of a large storm surge barrier will be substantial. From maintenance numbers of three large barriers in the world (Thames Barrier, Maeslant barrier, Eastern Scheldt barrier), it has been estimated that the annual maintenance costs are approximately 0.5% of the first construction costs (van Ledden et al, 2012). Table VIII-12 provides a summary of the first construction and annual costs on a unit basis (cubic foot). Total annual costs are estimated using a 50-year project life and annual interest rate of 3.5%.

Table VIII-13 presents storm surge barrier costs for each of the inlet and/or harbor opening considered in this study. The estimates were made on the basis of the cost per volume method described above. The table presents the design water level conditions, barrier dimensions, and corresponding cost estimates. A constant design water level of 11.5 ft above local MHHW, corresponding to approximately the 0.2 percent flood in the New York Bight, was used throughout the study area to determine the design hydraulic head. This value is based on the most recent FEMA modeling as part of their effort to update Flood Insurance rate Maps in for 14 coastal New Jersey counties and New York City. This value can be updated in the future as NACCS storm surge modeling results become available. In the meantime, the only local adjustment made is on the basis of the local tidal range. A 3 feet allowance to account for future sea level change was also included.

Table VIII-12. Storm Surge Barrier - Unit Construction Costs

Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Barrier Volume	1	cu.ft.	$912	$912
Subtotal				$912
Contingency	25%			$228
Total Construction				$1,140
E&D	12%			$137
S&A	10%			$114
Total Estimated First Construction Cost per cu.ft.				$1,391
Annualized First Costs				$59
O&M	0.5%			$7
Total Estimated Annual Average Cost per cu.ft.				$66

The resulting costs are a reasonable basis for planning and on the whole demonstrate that while storm surge barriers can be quite effective in managing coastal flood risk. Nevertheless, storm surge barriers are quite expensive especially for large structures (e.g. Sandy Hook-Breezy Point Barrier). Smaller structures such as Stamford CT, Fox Point, RI, and New Bedford, MA have performed well and have proven to be cost-effective. One of the additional challenges is that a storm surge barrier may be adequately designed but it will not perform satisfactorily unless it ties into surrounding areas of sufficient elevation to prevent flooding waters from simply flowing around the barriers.

Summary: Storm Surge Barrier System Benefits, Impacts and other Considerations

Large regional storm surge barrier systems provide for reliable, long-term engineered flood risk management for a large area. Barriers systems typically are deployed when unusually high tides are expected, but allow water traffic to pass through during normal conditions.

Potential impacts of large storm barrier systems include environmental disruptions and impacts to fish migration and also to shipping and water traffic which would need to be channeled through gates, sluices or passageways. Some installations have adversely affected historical properties.

Large regional storm surge barrier systems are very expensive and require long-term construction efforts coordinated in multiple locations. Systems may require strengthening or upgrade projects on existing dikes, floodwalls, etc. A key consideration in these projects is determining what level of risk management is desired.

Table VIII-13. Storm Surge Barriers – Parametric Cost Estimates										
Barrier Location		Local NAVD-MLLW (ft)	MHHW -MLLW (ft)	Length (ft)	Chart Depth (ft, MLLW)	Barrier Height (ft)	Hydraulic Head (ft)	Volume (x1000 cu.ft)	First Cost ($MILL)	Total Average Annual Cost ($MILL)
Boston Harbor	MA	5.4	10.1	2,000	40.0	64.6	24.6	3,183	2,903	211
Beverly	MA	5.1	9.7	900	15.0	39.2	24.2	854	779	57
Pt. Judith Harbor	RI	1.9	3.4	300	12.0	29.9	17.9	161	147	11
Bridgeport	CT	3.8	7.3	3,000	35.0	56.8	21.8	3,712	3,385	246
Milford	CT	3.6	6.9	180	7.0	28.4	21.4	109	100	7
Verrazano Narrows	NY	2.7	5.2	4,190	varies	varies	19.7	5,822	5,810	199
Arthur Kill	NY	2.9	5.6	2,700	35.0	55.1	20.1	2,992	2,729	26
Newtown Creek	NY	2.7	4.8	400	23.0	42.3	19.3	327	298	162
Rockaway Inlet	NY	2.8	5.3	2,800	23.2	43.0	19.8	4,769	4,349	
East Rockaway Inlet	NY	2.7	5.0	1,400	20.0	39.5	19.5	1,081	986	407
Jones Inlet	NY	2.6	4.8	2,250	23.0	42.3	19.3	1,842	1,680	198
Fire Island Inlet	NY	2.4	4.5	2,700	25.0	44.0	19.0	2,256	2,058	22
Moriches Inlet	NY	2.1	3.8	900	24.0	42.3	18.3	697	635	316
Shinnecock	NY	2.1	3.7	900	23.0	41.2	18.2	674	615	72
Cedar Beach	NY	3.6	7.0	600	25.0	46.5	21.5	599	546	122
Port Jefferson	NY	3.7	7.1	1,150	25.0	46.6	21.6	1,157	1,055	149
Huntington Bay	NY	4.0	7.7	2,700	25.0	47.2	22.2	2,823	2,575	46
Oyster Bay	NY	4.1	7.8	2,400	25.0	47.3	22.3	2,535	2,312	45
Sandy Hook-Breezy Point	NY/ NJ	2.8	5.2	28,500	varies	varies	19.7	39,124	35,681	2,592
Cheesequake	NJ	2.9	5.6	270	11.0	31.1	20.1	168	154	11
Shrewsbury River	NJ	2.8	5.2	1,650	16.0	35.7	19.7	1,164	1,062	77
Shark River	NJ	2.6	4.9	100	10.0	29.4	19.4	56	52	4
Manasquan Inlet	NJ	2.5	4.6	420	10.0	29.1	19.1	234	214	16
Indian River Inlet	DE	2.4	4.2	800	70.0	88.7	18.7	1,327	1,210	88
Christiana River	DE	2.8	6.0	1,250	38.0	58.5	20.5	1,501	1,369	99
Darby Creek	PA	2.9	6.1	420	2.0	22.6	20.6	196	179	13
Schuylkill	PA	2.9	6.3	720	28.0	48.8	20.8	732	668	49
Baltimore Patapsco	MD	0.8	1.6	2,250	50.0	66.1	16.1	1,776	1,620	151
Baltimore Bear Creek	MD	0.8	1.6	3,600	15.0	31.1	16.1	1,810	1,651	120

Solomons Island	MD	0.9	1.5	750	20.0	36.0	16.0	433	394	29
Ocean City	MD	2.3	3.8	2,000	28.0	46.3	18.3	1,694	1,545	112
Chincoteague Inlet	MD	2.2	4.1	6,500	15.0	33.6	18.6	4,051	3,695	268
Rudee Inlet	VA	2.5	3.7	100	15.0	33.2	18.2	60	55	4
Lynnhaven Inlet	VA	1.8	3.1	1,000	15.0	32.6	17.6	571	521	38
Little Creek	VA	1.7	2.8	950	22.0	39.3	17.3	647	591	43
Elizabeth River	VA	1.7	2.9	2,640	32.6	49.9	17.4	2,288	2,087	152

VIII.6. Structural/NNBF Measures

As listed in Table VIII-3, the Structural/NNBF Measures include Beach Restoration (beach fill, dune creation) – Barrier Island Preservation, Beach Restoration with Breakwaters, Beach Restoration with Groins, Drainage Improvements, and Living Shorelines.

VIII.6.1 Beach Restoration

Description

Beach restoration, also commonly referred to as beach nourishment or beachfill, typically includes the placement of sand fill to either replace eroded sand or increase the size (width and/or height) of an existing beach, including both the beach berm and dunes (Figure VIII-14) (USACE 2002). Material similar to the natural sand is artificially placed on the eroded part of the beach. Beach restoration might reduce risk not only the beach where it is placed and infrastructure landward of the beach, but also downdrift stretches by providing an updrift point source of sand (USACE 2002). Beach restoration can also be used to construct and/or restore barrier islands. Most coastal engineering practitioners consider beach restoration as a technically sound shore risk management engineering alterative when properly designed and placed in the appropriate location (NRC 1995).

The direction and rate of movement of the newly deposited sand along the shoreline should be considered to avoid shoaling and filling of any adjacent navigable waterways. As indicated by the numerous federal, state, and local beach restoration projects located throughout the study area, beach restoration is a very effective and thus commonly used method of storm damage reduction in the Northeast.

Figure VIII-14. Beach Restoration project under construction in June 2013 at Brant Beach, NJ

A well designed beach restoration project reduces risk to the structures and populations behind it by providing a buffer against the increased wave energy and storm surge generated during a coastal storm event. Beach restoration can also be used in combination with other structural shoreline risk management measures such as seawalls, breakwaters, and groins (see discussion below), but can also function well as a standalone measure. For this reason, beach restoration can be used in locations where the use of hard structures is not acceptable. Although very effective in reducing storm damage to the areas they are designed to reduce risk to, beach restoration projects are typically applicable only where there is an existing, gently sloping, sandy shoreline having a natural source of sand to help sustain the beach.

Beach restoration alone is a viable solution for the reduction of storm damages at locations where shore erosion is not severe. Beach restoration could be limited in its effectiveness in areas where renourishment/rehabilitation is required frequently (e.g. adjacent to inlets or erosional hot spots). At these highly erosive locations, it is often advisable to combine beachfill with other methods for reducing erosion (e.g., groins, breakwaters or seawalls). The longevity of a beach restoration project is also related to the length of the filled shoreline. Consequently, beach restoration projects are ideally applied to long segments and are less suitable for local, isolated storm risk management.

Generic Design

Typically beachfill design templates or cross-sections, dune height/width and berm width, are designed to provide a certain level of risk management. Beachfill designs must also consider the quantity of sand and frequency of renourishments that are required to maintain the design berm and dune over the life of the project. There are many other site specific design criteria that are not discussed in detail here but must be considered for during detailed beach restoration design: identification of onshore or offshore sources of compatible sediment, beachfill tapers, dune crest alignment, etc.

Beachfill design templates are defined by the berm elevation, berm width, foreshore slope, dune elevation, dune width, dune slope. Berm elevations are typically designed to correspond with existing beach conditions. USACE Engineering Manual 1110-2-3301 suggests, *"if possible, constructed berm elevations should be designed to be the same or slightly less than the natural berm crest elevations".* Natural berm elevations are controlled by normal tide and wave conditions are typically about 6 feet above Mean Higher High Water (MHHW). A berm elevation of +8 feet NAVD was selected for the generic design based typical MHHW elevations along ocean shorelines within the study area. If fill materials are compatible with the native sediment than the seaward beach slopes will mirror native beach conditions offshore to the closure depth. A representative closure depth at the -25 feet NAVD contour has been identified based on typical ocean wave heights in the study area.

The berm width and dune elevation/width are designed to provide risk management during a 1 percent storm. The berm width and volume must be sufficient to reduce wave energy during storm events and the dune must be high and wide enough to prevent significant wave overtopping and erosion during storm events. Previous planning and design studies for ocean shorelines in the study area evaluated the level of risk management provided by different dune and berm combinations. The results indicate that a dune crest elevation approximately 8 feet above the 1 percent flood, a dune crest width of 25 feet, and dune slope of 1V:5H provides approximately a 1 percent flood level of risk management when combined with a berm width of 120 feet. Therefore, the proposed dune crest elevation for the generic design profile is +18 feet NAVD (8 feet above a representative 1 percent flood +10 feet NAVD).

In addition, it is assumed that the existing beach berm width is about 50% of the required design beach berm (i.e., 60 ft of additional berm required) and that the existing dune is small or non-existing (i.e., 100% of the 18 ft dune will be required). This leads to approximately 100 cubic yards of beachfill per foot shoreline required, a number typical of many beach restoration projects in the study area.

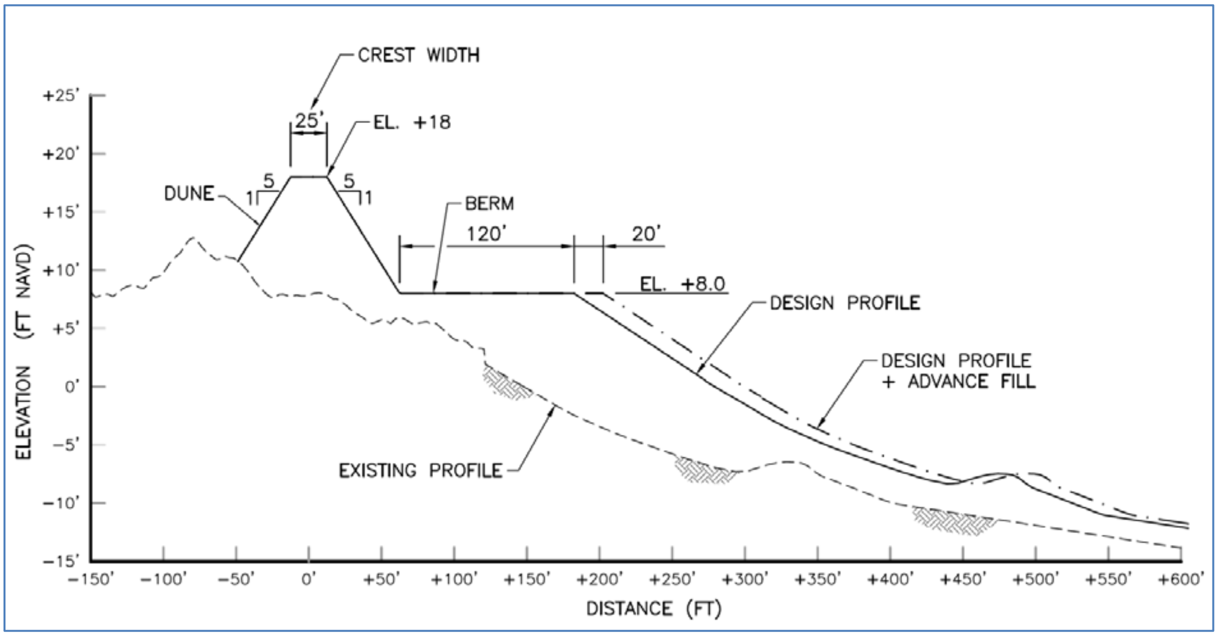

Figure VIII-15. Typical Section of Beach Restoration

Beachfill alone does not alter pre-existing shoreline erosion rates. Generally it is assumed that the background shoreline erosion will continue at the same rate as before the project. Typically, background erosion is caused by a deficit in sediment budget. Beachfill projects typically experience additional erosion from “spreading out” or diffusion of sand resulting from the shoreline anomaly or “bump” created by the beachfill. Diffusion losses are function of the longshore length of the beachfill, cross-shore width of the beachfill, and wave climate (diffusivity). The rate of diffusion is particularly sensitive to the longshore length of the beachfill project. Shorter projects will generally experience a much higher rate of diffusion than longer projects. A typical shoreline erosion rate of 5 feet/year, encompassing background erosion and beachfill diffusion, was applied in the generic beachfill design estimates. In addition, a RSLC of 3 feet over a hundred years, equating to approximately 1.5 ft/yr of shoreline erosion, was added for a total erosion rate of 6.5 feet/year.

Parametric Unit Costs

Beach restoration is normally constructed using either hopper or pipeline hydraulic dredges. Fill material is typically obtained from offshore borrow areas located in the vicinity of the project area. Initial beachfill quantities are usually determined by comparing survey profiles to the design template. Initial beachfill quantities are site specific and will vary considerably depending on the existing beach width and dune heights. In order to develop parametric costs it is estimated that initial construction of each beach fill will require placement of 50% of the design berm width, 100% of the dune fill, and 100% of the required advance fill.

Advance beachfill is required to maintain the design section before the first scheduled renourishment. Advance fill requirements are based on the expected shoreline erosion between the initial fill and first renourishment and are equivalent to renourishment volumes. The interval between renourishment events is dependent on the expected shoreline erosion rate; a shorter renourishment interval is generally required for higher erosion rates. A renourishment interval of four years is applied in this study and is typical of existing projects in the area. All fill quantity estimates include dredging tolerance (15%) and overfill (10%) allowances. Table VIII-14 shows the estimated first fill and renourishment fill quantities.

Unit beachfill costs may vary considerably based on the type of dredge used and distance to sediment source (e.g. borrow area). A value of $12.0 per cubic yard is applied in the parametric costs based recent bids and detailed cost estimates for beachfill projects performed with hopper dredges and a sediment source within approximately 10 miles of the placement site. In addition, recent bids indicate that each mobilization/demobilization costs approximately $3 million. A small project length (3,000 feet) will require 1 mob/demob whereas a larger project length (15,000 feet) may still only require 1 mob/demob. Therefore, the relative cost of the mobilization will be much higher for a small beachfill project resulting in a greater parametric cost. A typical project length of 10,000 feet (~ 2 miles) is used to determine the parametric beachfill costs.

An additional cost associate with beachfill projects are berm fill maintenance costs. Berm maintenance ($15 per foot) is typically required to address shoreline undulations and erosional hotspots. Regular fill maintenance, such as tiling, is included under the regular operation & maintenance.

Cost estimates include 12% for engineering and design (E&D), 10% for construction management (S&A), and 1% for operation and maintenance (O&M). A contingency of 25% is applied to the cost estimate. Real estate costs associated with structure acquisitions/relocations, and easements vary considerable by project and are not included in the parametric cost estimate. Table VIII-14 and Table VIII-15 provide a summary of the first construction and renourishment quantities and costs.

Total annual costs are estimated using a 50-year project life and annual interest rate of 3.5%. Table VIII-16 presents the annualized costs for first costs, renourishment costs, fill maintenance, and O&M. The total annual cost is approximately $488 per foot.

Table VIII-14. Beach Restoration - First Construction Quantities & Costs

Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$3,000,000	$3,000,000
Design Beach Fill Volume	1,279,056	cu.yd.	$12	$15,348,672
Advance Fill Volume	401,989	cu.yd.	$12	$4,823,868
Subtotal				$23,172,540
Contingency	25%			$5,793,135
Total Construction				$28,965,675
E&D	12%			$3,475,881
S&A	10%			$2,896,568
Total Estimated First Construction Cost				$35,338,124
Total Estimated First Construction Cost per Foot				$3,534

Table VIII-15. Beach Restoration - Renourishment Quantities & Costs				
Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$3,000,000	$3,000,000
Renourishment Fill Volume	401,989	cu.yd.	$12	$4,823,868
Subtotal				$7,823,868
Contingency	25%			$1,955,967
Total Construction				$9,779,835
E&D	12%			$1,173,580
S&A	10%			$977,984
Total Estimated Renourishment Cost				$11,931,399
Total Estimated Renourishment Cost per Foot				$1,193

Table VIII-16. Beach Restoration - Annualized Costs per Foot		
Annualized First Costs		$151
Annualized Renourishment Costs		$279
Fill Maintenance		$23
O&M	1%	$35
Total Estimated Annual Average Cost		$488

Summary: Beach Restoration Benefits, Impacts and other Considerations

Beach fill or beach replenishment increases beach width which provides a buffer zone against storm erosion to reduce risk to property and vulnerable population. Increased beach area also provides more recreational space or "towel area" as an added benefit. Beach fill avoids the construction of expensive, hard, permanent structures such as seawalls, revetments and groins and can also provide for the replacement of lost habitat. Beach fill reduces storm damage and may often help to increase tourism.

Beach fill impacts include damage to habitat in borrow areas and also to the habitat areas that is being filled. Beach fill can cause short term water quality impacts due to turbidity and may disrupt the natural beach system due to variations in the introduced sand grain size mix. Beach fill may also create steeper beaches with ledges and scarp.

Beach fill considerations include the both initial cost, and the long term need for continued renourishment and maintenance.

VIII.6.2 Beach Restoration with Groins

Description


Figure VIII-16. Groin Field at Westhampton, NY

Most coastlines experience waves and currents that transport sand parallel to shore; this is generally referred to as longshore sediment transport. On some coastlines there is more sand leaving the area via longshore sediment transport than there is sand arriving thus causing a net deficit of sand and attendant erosion. Groins are structures that extend perpendicularly from the shoreline. They are usually built to stabilize a stretch of natural or artificially nourished beach against erosion that is due primarily to a net longshore loss of beach material. The effect of a single groin is accretion of beach material on the updrift side and erosion on the downdrift side; both effects extend some distance from the structure. Consequently, a groin system (series of groins) results in a saw-tooth-shaped shoreline within the groin field and a differential in beach level on either side of the groins (USACE 2002). In most cases, groins are sheet-pile or rubble-mound constructions. An example of a groin field at Westhampton, on the Atlantic coast of Long Island, NY, is shown in Figure VIII-16 (USACE 2002).

Groins are occasionally constructed non-perpendicular to the shoreline, can be curved, have fishtails, or have a shore-parallel T-head at their seaward end. Also, shore-parallel spurs are provided to shelter a stretch of beach or to reduce the possibility of offshore sand transport by rip currents (USACE 2002). Groins can be long or short and high or low. Long and/or high groins will trap more sediment than comparatively shorter and/or lower ones. Some cross-groin transport is beneficial for obtaining a well-distributed retaining effect along the coast. For the same reason permeable groins, which allow sediment to be transported through the structure and may reduce rip currents, may be advantageous. Proper spacing of groins allows for sand to accumulate along the entire length of the area between the groins. The relatively high initial construction costs with groins may be offset by a reduction in the quantity and frequency of future renourishments over the project life.

Generic Design

The beach restoration and groin design assumes that the beachfill cross-section is unchanged and that the groin compartments would be filled initially to promote sand bypassing. The optimal groin field layout (groin geometry, length and spacing) is typically determined by balancing the initial cost of the groins with the cost reductions in renourishments (i.e. groin retention efficiency). Groin retention efficiency is the reduction of beachfill losses with groins and typically increases with groin length (G) and shorter groin spacing (L). The Shore Protection Manual (USACE 1984) recommends groin spacing to length ratios (L/G) between 2 and 3, where the groin length is measured from the seaward berm crest. Based on previous alternative screening studies performed for ocean shorelines within the study area, a groin spacing of 1,150 feet and groin length of 412 feet was selected for the generic design (L/G = 2.8) providing a retention efficiency of 55%. Figure VIII-17 shows the groin field layout.

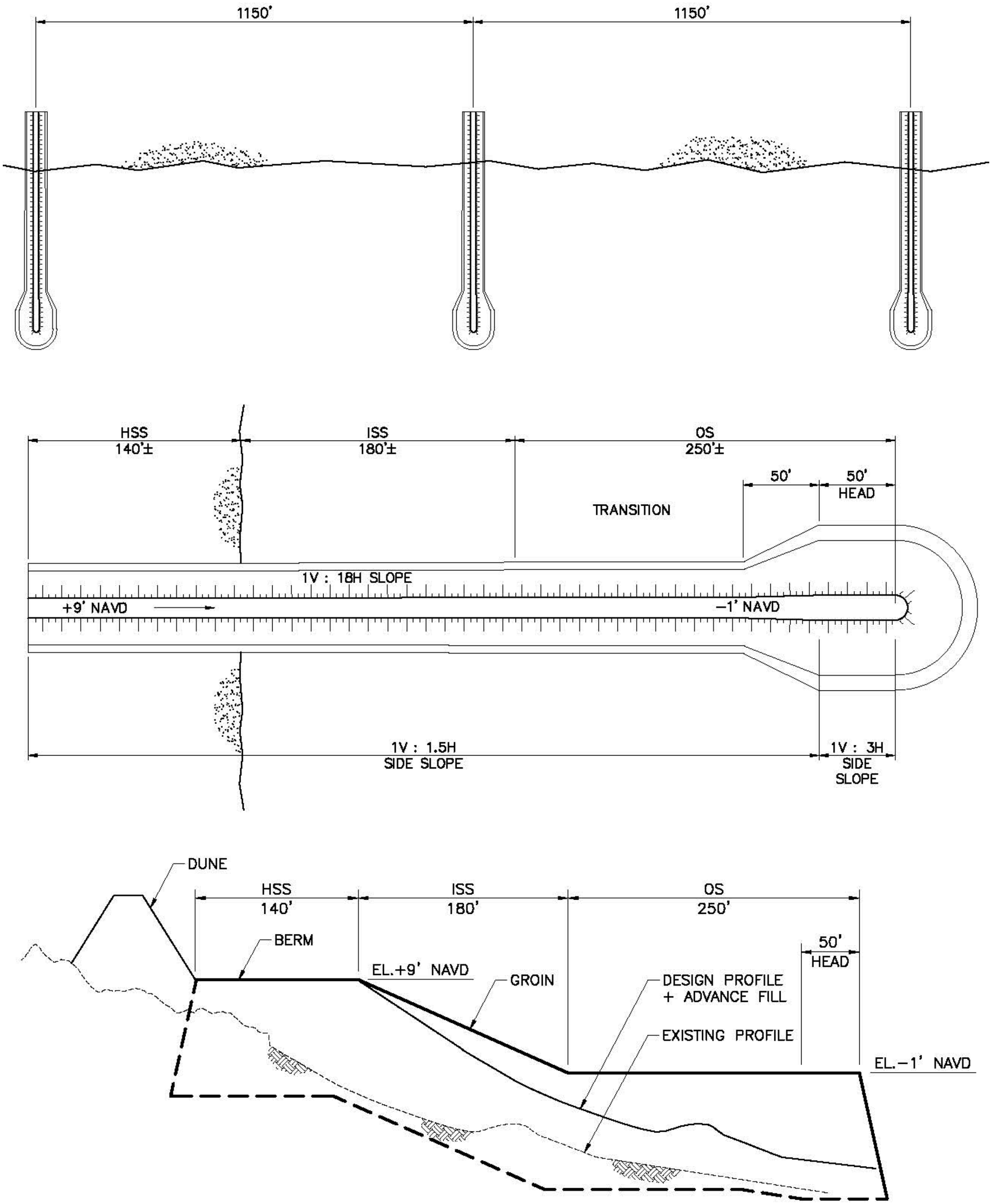

Figure VIII-17. Typical Groin Layout

Groin design is summarized as: (1) a horizontal shore section (HSS) extending from a crest elevation of +8 feet NAVD to a bottom elevation of -2 feet NAVD; (2) an intermediate sloping section (ISS) extending from a crest elevation of +8 to -1 feet NAVD at a slope of 1V:18H; and (3) an outer sloping section (OS) extending from a crest elevation of -1 feet NAVD to a bottom elevation of -13 feet NAVD. Figure VIII-17 depicts the three groin sections and the length of each section. The SPM groin length is defined as the ISS and OS sections (412 feet).

Armor stone sizes increase along the groin with water depth and were determined based on assumed 1 percent storm wave conditions which will be limited by depth at the toe of the structure and therefore a function of the storm tide. The groin trunk consists of side slopes of 1V:1.5H, one layer of armor stone with sizes from 8 to 10 ton, underlayer with 2 layers of stone, core and blanket layer comprised of 9 to 180 pound stone, and geotextile filter. At the groin head a minimum of two armor stone layers (16.4 ton) are placed. Typical sections at the HSS, OS, and Head section are shown in Figure VIII-18.

Parametric Unit Costs

The design beach fill volumes and costs for first construction are the same as the beach restoration only alternative. However, due to the increased sediment retention (55%) a longer renourishment interval, 8 years, is applied. Volumetric losses from RSLC (1.5 feet/year) remain the same as the beach restoration only alternative, only the volumetric losses associated with background erosion and diffusion (5 feet/year) are reduced. A project length of 10,000 feet (~2 miles) is used to determine the number groins and total volumetric quantities required. A more expensive mobilization/demobilization is required for the additional equipment required for the groin construction. A 1 foot tolerance is applied to the armor stone quantity estimates. Unit costs of $150 per ton of stone, $15 per sq.yd. of geotextile, and $13 per cu.yd of excavation are applied in the parametric cost estimate. Berm fill maintenance, typically required in beach restoration only alternatives to address shoreline undulations and erosional hotspots, is not included since the groin field is expected to stabilize the shoreline.

Cost estimates include 12% for engineering and design (E&D), 10% for construction management (S&A), and 1% for operation and maintenance (O&M). A contingency of 25% is applied to the cost estimate. Real estate costs associated with potential ocean front structure acquisitions/relocations, and easements are not included in the parametric cost estimate Table VIII-17 and Table VIII-18 provide a summary of the first construction and renourishment quantities and costs.

Total annual costs are estimated using a 50-year project life and annual interest rate of 3.5%. Table VIII-19 presents the annualized costs for first costs, renourishment costs, fill maintenance, and O&M. The total annual cost is $532 per foot.

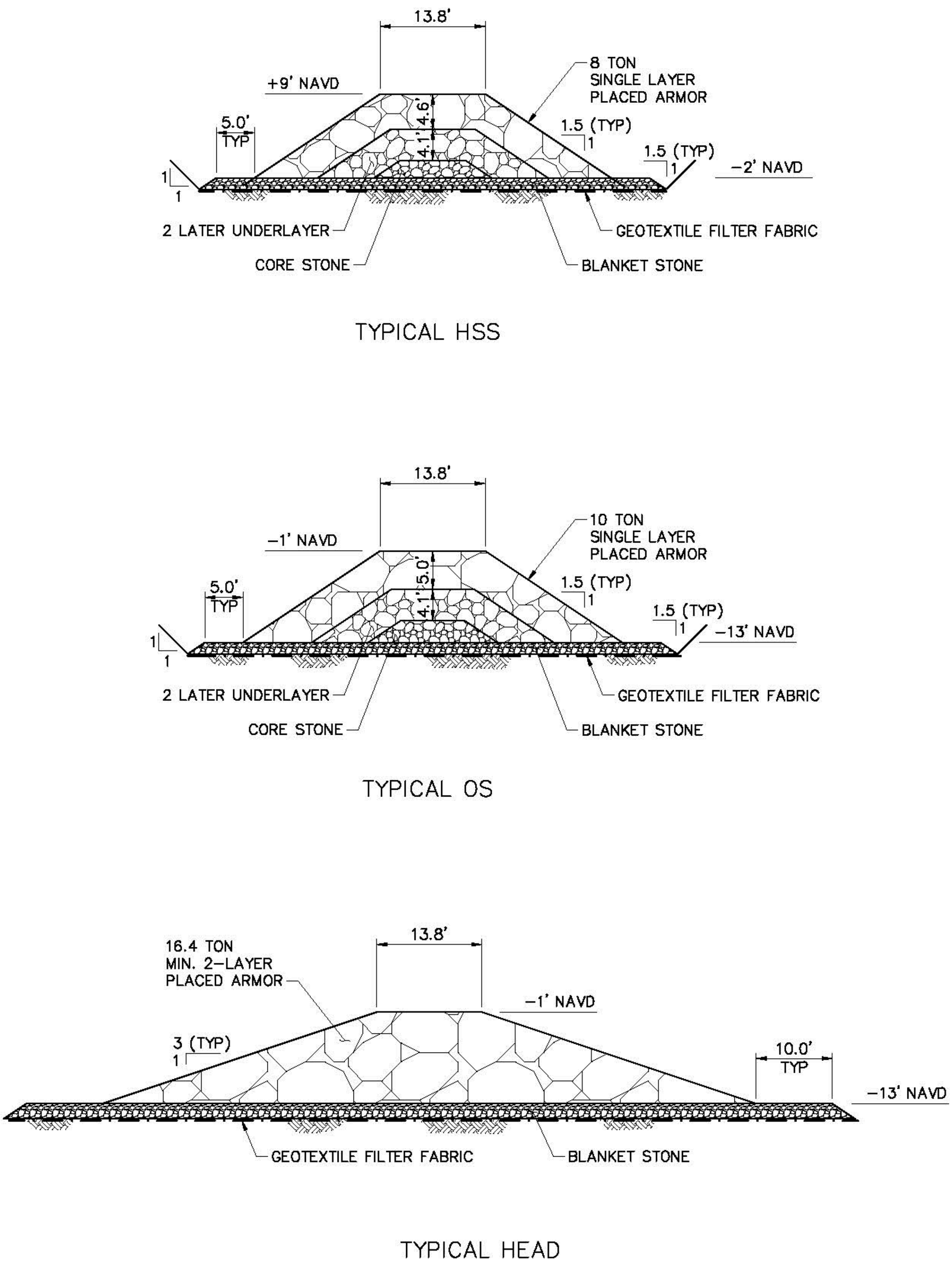

Figure VIII-18. Typical Groin Section

Table VIII-17. Beach Restoration with Groins - First Construction Quantities & Costs				
Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$4,000,000	$4,000,000
Design Beach Fill Volume	1,279,056	cu.yd.	$12	$15,348,672
Advance Fill Volume	463,833	cu.yd.	$12	$5,565,996
Armor Stone	79,676	ton	$150	$11,951,400
Underlayer / Core Stone	31,092	ton	$150	$4,663,800
Blanket Stone	36,875	ton	$150	$5,531,250
Geotextile	38,219	sq.yd.	$15	$573,285
Excavation	75,621	cu.yd.	$13	$983,073
Subtotal				$48,617,476
Contingency	25%			$12,154,369.00
Total Construction				$60,771,845
E&D	12%			$7,292,621
S&A	10%			$6,077,185
Total Estimated First Construction Cost				$74,141,651
Total Estimated First Construction Cost per Foot				$7,414

Table VIII-18. Beach Restoration with Groins - Renourishment Quantities & Costs				
Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$3,000,000	$3,000,000
Renourishment Fill Volume	463,833	cu.yd.	$12	$5,565,996
Subtotal				$8,565,996
Contingency	25%			$2,141,499
Total Construction				$10,707,495
E&D	12%			$1,284,899
S&A	10%			$1,070,750
Total Estimated Renourishment Cost				$13,063,144
Total Estimated Renourishment Cost per Foot				$1,306

Table VIII-19. Beach Restoration with Groins - Annualized Costs per Foot		
Annualized First Costs		$316
Annualized Renourishment Costs		$142
Fill Maintenance		$0
O&M	1%	$74
Total Estimated Annual Average Cost		$532

Summary: Beach Restoration with Groins Benefits, Impacts and other Considerations

By trapping a portion of the littoral drift sand groins help to sustain a beach by preventing further erosion. The beach in turn helps to reduce risk to the shoreward coastal property and population.

Groins typically create deposition and erosion problems by upsetting the natural equilibrium between the sources of beach sediment and the littoral drift pattern. Groin fields tend to shift the zone of erosion out of the immediate area to the down drift neighbor.

Figure VIII-19. Breakwater Field at Ocean View

VIII.6.3 Beach Restoration with Breakwaters

Description

In general, breakwaters are structures designed to reduce risk to shorelines, beaches, or harbor areas from the impacts of wave action thereby reducing shoreline erosion and storm damage. When used as harbor risk management structures they are typically attached to the shore and enclose the harbor basin to reduce the impacts from waves. Shoreline risk reduction breakwaters are usually built some distance from the shore (detached breakwaters), in relatively shallow water, and roughly parallel to it so as to maximize amount of risk reduction they provide and to optimize their efficiency at reducing erosion. Figure VIII-19 shows an example of a field of detached breakwaters (USACE 2002). Beach restoration may be combined with offshore breakwaters along severely eroding shorelines to increase the longevity of a project by increasing the sediment retention. The relatively high initial construction costs with breakwaters may be offset by a reduction in the quantity and frequency of future renourishments over the project life.

Breakwaters are usually built as rubble-mound structures (USACE 2002) though they can be constructed from a variety of materials such as geotextile and concrete. The dissipation of wave energy allows sand to be deposited behind the breakwater. This accretion further reduces risk the shoreline and may also widen the beach. In some cases the beach “salient” formed by the accretion effect connects to the breakwater thus forming a “tombolo”; whether or not the detached breakwaters become attached to shore is a function of placement distance offshore and length of the structure. The gaps between the breakwaters are in most cases on the same order of magnitude as the length of one individual structure. Breakwaters, usually in combination with beach restoration, are appropriate for implementation on beaches as a stabilization measure.

Generic Design

In contrast to the beach restoration and groin design, the design beachfill cross-section changes with the inclusion of offshore breakwaters. A 33% reduction in the design berm width is justified by an equivalent reduction in the incident wave energy along the shoreline. The dune dimensions are not altered since the breakwaters would have little impact on the storm tide.

The objective of the breakwater layout is to stabilize the shoreline with the formation of salients and avoid excessive erosion in the gaps between breakwaters. If the spacing between breakwaters is too small or if the breakwaters are too close to the shoreline, tombolos may form behind the breakwaters. Tombolos block the longshore sediment transport and essentially function as groins. Criteria established for breakwater design was applied to determine the appropriate breakwater length, spacing, distance from shoreline, and depth (Chasten et al, 1993, and Rosati 1990) for a typical ocean shoreline. The generic breakwater layout consists of breakwater segments of 300 feet, 400 foot gaps between segments, and breakwaters located 500 feet seaward of the design shoreline. Figure VIII-20 shows the breakwater layout. For the purpose of plan comparison, an increased sediment retention efficiency of 65% (relative to beachfill alone) is estimated.

The breakwater cross-section is similar to the design of the groin trunk and consists of 2 layers of 18 ton armor stone, an underlayer with 2 layers of 1.8 ton stone, and a core and blanket layer comprised of 9 to 180 pound stone. A typical section for the breakwater is shown in Figure VIII-20. The armor stone sizes were determined based on typical 1 percent storm wave conditions in the study area.

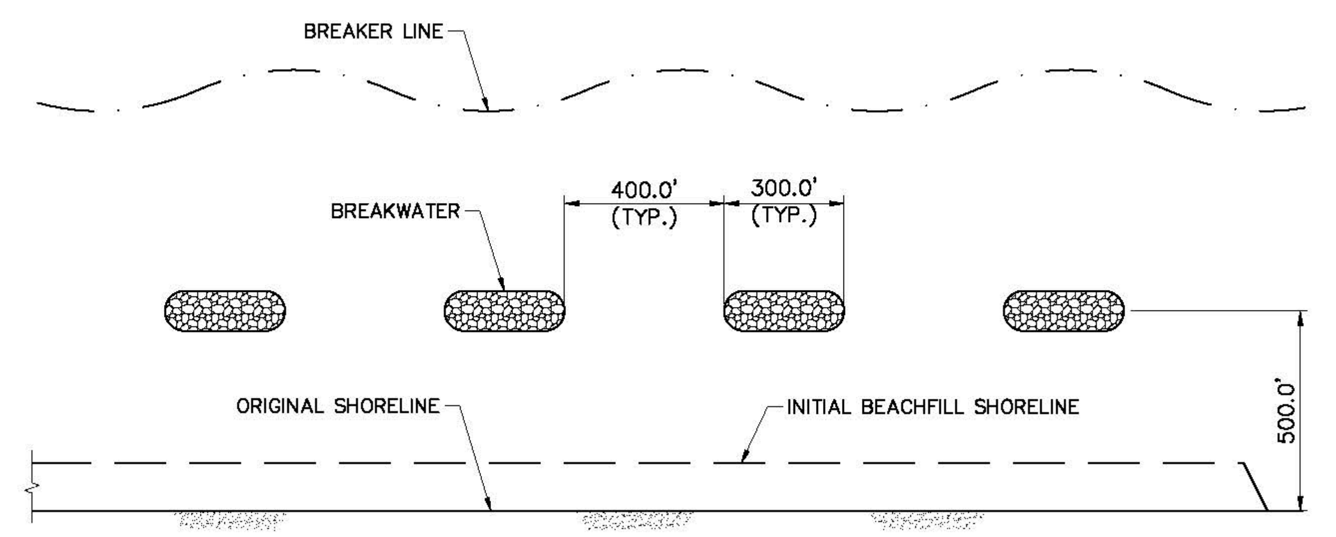

Figure VIII-20. Typical Offshore Breakwater Layout

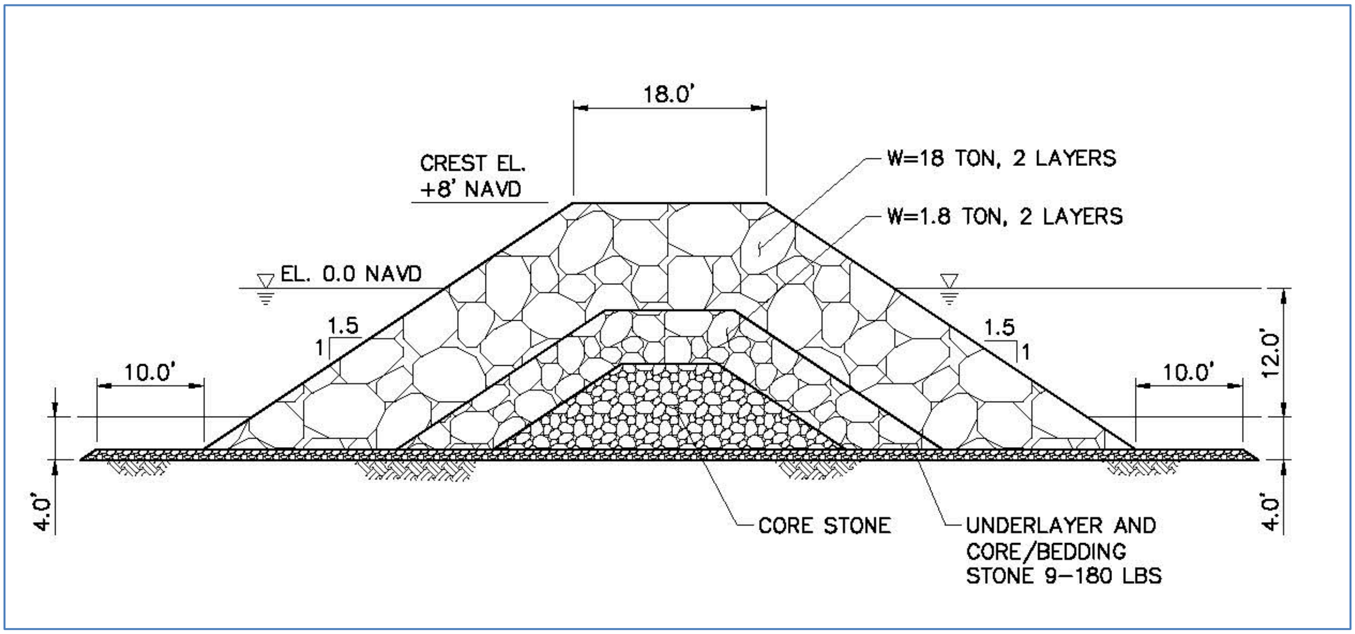

Figure VIII-21. Typical Breakwater Section

Parametric Unit Costs

The design beach fill volumes for first construction decrease significantly since the design berm is reduced to 80 feet. In addition, advance fill volumes and renourishment quantities are lower due to the increased sediment retention (65%). An 8 year renourishment interval is applied (same as beach restoration with groins). Volumetric losses from sea level change remain the same as the beach restoration only alternative, only the volumetric losses associated with background erosion and diffusion (5 feet/year) are reduced. A project length of 10,000 feet (~2 miles) is used to determine the number breakwaters and total volumetric quantities required. A more expensive mobilization/demobilization is required for the additional equipment required for the breakwater construction. A 1 foot tolerance is applied to the armor stone quantity estimates. Unit costs of $150 per ton of stone are applied in the parametric cost estimate. Berm maintenance, typically required in beach restoration only alternatives to address shoreline undulations and erosional hotspots, is not included since the offshore breakwaters are expected to stabilize the shoreline.

Cost estimates include 12% for engineering and design (E&D), 10% for construction management (S&A), and 1% for operation and maintenance (O&M). A contingency of 25% is applied to cost estimate. Real estate costs associated with potential ocean front structure acquisitions/relocations, and easements are not included in the parametric cost estimate. Table VIII-20 and Table VIII-21 provide a summary of the first construction and renourishment quantities and costs.

Total annual costs are estimated using a 50-year project life and annual interest rate of 3.5%. Table VIII-22 presents the annualized costs for first costs, renourishment costs, fill maintenance, and O&M. The total annual cost is $613 per foot.

Table VIII-20. Beach Restoration with Breakwaters - First Construction Quantities & Costs				
Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$4,000,000	$4,000,000
Design Beach Fill Volume	660,611	cu.yd.	$12	$7,927,332
Advance Fill Volume	401,989	cu.yd.	$12	$4,823,868
Armor Stone	223,328	ton	$150	$33,499,200
Underlayer	58,165	ton	$150	$8,724,750
Core/Bedding Stone	8,025	ton	$150	$1,203,750
Subtotal				$60,178,900
Contingency	25%			$15,044,725
Total Construction				$75,223,625
E&D	12%			$9,026,835
S&A	10%			$7,522,363
Total Estimated First Construction Cost				$91,772,823
Total Estimated First Construction Cost per Foot				$9,177

Table VIII-21. Beach Restoration with Breakwaters - Renourishment Quantities & Costs				
Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$3,000,000	$3,000,000
Renourishment Fill Volume	401,989	cu.yd.	$12	$4,823,868
Subtotal				$7,823,868
Contingency	25%			$1,955,967
Total Construction				$9,779,835
E&D	12%			$1,173,580
S&A	10%			$977,984
Total Estimated Renourishment Cost				$11,931,399
Total Estimated Renourishment Cost per Foot				$1,193

Table VIII-22. Beach Restoration with Breakwaters - Annualized Costs per Foot		
Annualized First Costs		$391
Annualized Renourishment Costs		$130
Fill Maintenance		$0
O&M	1%	$92
Total Estimated Annual Average Cost		$613

Summary: Beach Restoration with Breakwaters Benefits, Impacts and other Considerations

Breakwaters reduce risk to a portion of the shoreline area from wave erosion, which in turn reduces risk to property and vulnerable populations.

Breakwaters typically create deposition and erosion problems by upsetting the natural equilibrium between the sources of beach sediment and the littoral drift pattern. Shorelines near breakwaters must change their configuration in an attempt to reach a new equilibrium. Breakwaters do not provide direct tide surge risk management.

VIII.6.4 Drainage Improvements

Measures in this category include pump stations, culverts/drains/inlets, and water storage/retention features. A drainage system can perform two functions: it carries water away via conveyance systems and, during times of high water, may store water until it can be carried away in storage facilities. Conveyance systems utilize measures such as pump stations, culverts, drains, and inlets to remove water from a site quickly and send it to larger streams. Storage facilities or features are used to store excess water until the storm or flood event has ended. Drainage improvement measures are appropriate for implementation on all shoreline types. The most significant application of drainage improvements in coastal flood storm management is as part of any plan that uses structures, such as seawalls, gates or levees, to create a line of risk management against tidal inundation. Drainage outlets, flood storage, or pumps are needed to control flooding from rainfall runoff from behind the line of risk reduction or from waves overtopping the structures.

Summary: Drainage Improvements Benefits, Impacts and other Considerations

Drainage improvements enable more rapid and efficient evacuation of rain and floodwaters from a specific area to a receiving body of water, reducing the risk of flood water buildup.

Considerations include cost and maintenance requirements and also potential impacts to utilities during construction.

VIII.6.5 Living Shoreline

Description

Living shorelines represent a shoreline management option that combines various erosion control methods and/or structures while restoring or preserving natural shoreline vegetation communities and enhancing resiliency. Typically, creation of a living shoreline involves the placement of sand, planting marsh flora; and, if necessary, construction of a rock structure on the shoreline or in the near shore (VIMS 2013b). An example of a living shoreline application is shown in Figure VIII-22. However, living shorelines can use a variety of stabilization and habitat restoration techniques that span several habitat zones and use a variety of materials. Specifically, living shorelines can be used on upland buffer/backface zones, coastal wetlands and beach strand zones, and the subtidal water zone. Living shoreline materials may include sand fill, clean dredge material, tree and grass roots, marsh grasses, mangroves, natural fiber logs, rock, concrete, filter fabric, seagrasses, etc. (Maryland DNR, 2007).

The benefits of living shorelines include stabilization of the shoreline, reduction of impacts to surrounding riparian and intertidal environment, reduction of impacts to cultural resources particualrly prehistoric resources along the coast improvement of water quality via filtration of upland run-off, and creation of habitat for aquatic and terrestrial species (Chesapeake Bay Foundation, 2007). Living shorelines are generally applicable to relatively low current and wave energy environments in estuaries, rivers, and creeks. Areas exposed to larger waves do not benefit significantly from a living shoreline application since the marsh vegetation and underlying soils would likely be eroded. Some instances of living shoreline applications in the Delaware Bay and Chesapeake Bay have indicated success in coastal storm risk management.


Figure VIII-22. Living Shoreline

Living shorelines are essentially tidal wetlands constructed along a shoreline to reduce coastal erosion, maintain dynamic shoreline processes, and provide habitat for organisms such as fish, crabs and turtles. They are natural landscape features that function primarily under normal tidal range conditions and provide a varied mix of habitat such as: shallow water, intertidal, beach, marsh and dune. They provide some benefits as a wave reducing component by functioning as shallow water under high water and storm conditions. A typical living shoreline is relatively narrow, and they have been promoted in embayments and other lower energy areas to replace revetments, bulkhead and other hard structures to serve as shoreline risk management. An essential component of a living shoreline is constructing a rock structure (breakwater/sill) offshore and parallel to the shoreline to serve as risk management from wave energy that would impact the wetland area and cause erosion of the substrate and damage or removal of the tidal plants. Also, the rock structure serves to hold the sand that is located shoreward in place, maintaining the substrate for the plants.

Two other items of importance to incorporate into a living shoreline are: 1) ensure there is adequate sunlight for the plants, and 2) take measures to prevent waterfowl (primarily Canada geese) from eating the plants. Since living shorelines are located close to the land, and possibly in areas with high banks, trees may overhang the area and significantly reduce exposure of the plants to sunlight. Tidal wetland plants generally thrive in areas where there are no trees, and the presence of them could affect the growth of the tidal plants. Non-migratory Canada geese are common along the east coast, and a flock of them can very quickly destroy newly planted vegetation, often pulling a new plant out by the roots. Goose-exclusion fencing is mandatory to prevent this predation and allow the marsh to grow and develop into a mature system. The fencing should be installed to prevent geese from flying or walking into the marsh. Once the grasses have had time to develop a strong root system, the fencing is no longer required and the waterfowl can eat the grasses without destroying the marsh.

Design

Typically, the living shoreline rock structure is designed for average, regular wave conditions, i.e., the crest elevation is at or slightly above mean high water (MHW) or mean higher high water (MHHW). The rock size is also designed to withstand average wave condition, which generally allows for the use of common riprap sizes and gradations. It is assumed that under storm flood conditions, the living shoreline would be under water and the larger waves would pass over the top and impact on the shore at a higher elevation. Thus, using a large rock size (with concomitant higher costs) is not required. Side slopes of the outer side of the breakwater range from 1.5H:1V to 3H:1V. Due to the small height of the breakwater, the difference in rock quantity for the slopes is not significant.

A living shoreline is constructed in fairly shallow water, usually less than 5 ft below mean lower low water (MLLW). The actual water depth is site-specific, but the shallower the water the lower the material quantities and subsequent construction costs for a given length of shoreline.

Another important feature of the living shoreline is openings in the rock structure to allow fish, crabs, turtles and other organisms to move from the deeper, open water into the wetland area for feeding and shelter. These openings can be either low-crested regions (crest elevation at about MLLW) about 5 to 10 ft wide, or the breakwater can be segmented. If segmented, smaller breakwaters can be constructed either inside or outside the alignment at the openings to minimize wave energy through them.

The sand that is placed behind the breakwater should be relatively coarse to minimize loss of material from the waves and currents that can enter through the breakwater. It is common to specify sand material with a maximum fines content of 10 percent. The slope of the sand should be fairly flat, with a maximum slope of 10H:1V.

Living shorelines should be designed to have both low and high marsh vegetation, and a 50/50 design ratio is preferred and typical. Site specific conditions as well as local preference could change this ratio, as well as environmental conditions following construction. It is practical and acceptable to allow the ratio to vary over time and not be strict about maintaining a certain ratio. Low marsh vegetation is typically Spartina alterniflora and high marsh vegetation is typically Spartina patens.

Figure VIII-23 shows a schematic of a representative typical cross-section of a living shoreline that includes the rock breakwater/sill, sand fill behind the breakwater and vegetative marsh grass plantings. For the purposes of the generic design and parametric costs estimates it is assummed that the living shoreline is located in -2 ft MLLW and has a fill width of about 50 ft. The crest elevation of the sill would be set at +4 ft (approximately MHHW). The outside side slope of the breakwater is 1.5H:1V and it is constructed of riprap with a median weight of 200 pounds. The assumed fetch distance is on the order of one to two miles and the average design waves are about one to two feet. This assumed generic or typical design and dimensions could easily be adapted to other specific site conditions such water depth and and tidal range once the potential application areas are identified. Water depth as well as tidal range (the difference between MHHW and MLLW) could be determined from avaiable coastal charts. On the other hand, design waves are more difficult to determine as they would require at least a desktop study of local wind statistics, fetch lenghts, and analytical wave calcualtions. For more detailed design a numerical wave model may be required.

Note that the design of living shoreline is not very sensitive to the extreme flood elevations (e.g., FEMA BFE) because, as explained above, it is assumed that under storm flood conditions, the living shoreline would be under water and the larger waves would pass over the top and impact on the shore at a higher elevation. Therefore, extreme water levels were not considered as an input design parameter. However, sea level change would have an impact over time on the performance of the living shoreline as the new mash is exposed to higher and higher elevations. Depending on the SLR scenario it is possible that the living shoreline would lose its marsh. Alternatively, the marsh could be "renourished" with additional fill material and even new plantings. The rock sill could also be raised with new riprap as required.

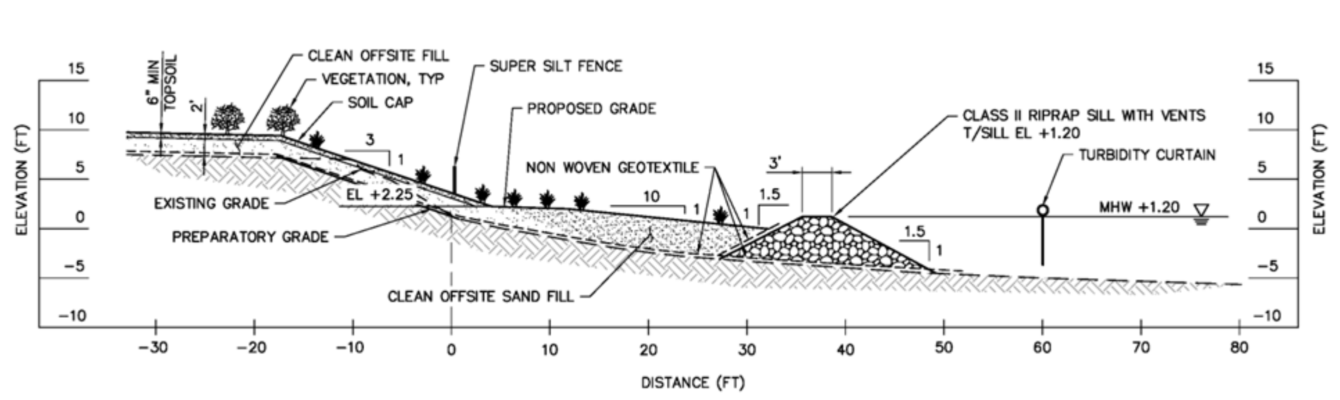

Figure VIII-23. Typical Section of Living Shoreline

Parametric Unit Cost Estimate

A parametric cost estimate based on the generic living shoreline design presented above is summarized in Table VIII-23. The costs are developed for a representative shoreline length of 5,000 feet and reduced to provide a cost per linear foot of living shoreline. The costs are based on representative unit costs for similar projects in study area. However, it is acknowledged that there will be significant variability in these unit costs depending location, material availability, local transportation costs, etc. After specific locations are selected for the application of a living shoreline, the unit costs, as well as the design, will be adjusted accordingly. First construction costs are about $1,415 per linear foot of living shoreline; annualized costs are about $67 per linear foot.

Table VIII-23. Living Shoreline - Construction Quantities & Costs

Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$500,000	$500,000
Armor Stone	20,000	ton	$150	$3,000,000
Geotextile	16,667	sq.yd.	$15	$250,000
Sand Fill	27,778	cu.yd.	$20	$555,556
Grass Plantings	166,667	each	$2	$333,333
Subtotal				$4,638,889
Contingency	25%			$1,159,722
Total Construction				$5,798,611
E&D	12%			$695,833
S&A	10%			$579,861

Table VIII-23. Living Shoreline - Construction Quantities & Costs				
Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Total Estimated First Construction Cost				$7,074,306
Total Estimated First Construction Cost per Foot				$1,415
Annualized First Costs				$60
O&M	0.5%			$7
Total Estimated Annual Average Cost				$67

Summary: Living Shoreline Benefits, Impacts and other Considerations

A Living Shoreline is generally considered to be a shoreline with bank stabilization using plants, sand and limited rock or other materials. The term is often expanded to include living breakwaters such as oyster reefs and systems of manmade wave attenuation devices (WADs) which are designed to promote habitat growth within and on the devices.

Living shoreline measures are aesthetically pleasing, preserve/create habitat, may retain runoff and pollutants, and can be less expensive than hard structure shoreline erosion risk management. Vegetation must be segregated to reduce impacts from human traffic by providing designated walkways and access paths.

The living shoreline approach is generally works best in low-erosional settings. More research is needed with regard to the effectiveness of living breakwaters in preventing beach erosion.

VIII.7. Natural and Nature-Based Features

As discussed in the previous section NNBF can be used in combination with structural and non-structural interventions to provide an integrated approach to reducing coastal risks while increasing human and ecosystem resilience across the North Atlantic Coast. Natural features are created and evolve over time through the action of physical, biological, geologic, and chemical processes operating in nature. Nature-based features are those that may mimic characteristics of natural features, but are created by human design, engineering, and construction to provide specific services such as coastal risk reduction. Nature-based features are acted upon by the same physical, biological, geologic, and chemical processes operating in nature, and as a result, generally must be maintained to reliably provide the expected level of service. Natural and nature-based features can enhance the resilience of coastal areas challenged by RSLC (Borsje et al. 2011) and coastal storms (e.g., Gedan et al. 2011, Lopez 2009).

As listed in Table VIII-3, the NNBF measures include overwash fans, reefs, submerged aquatic vegetation (SAV and wetlands).

VIII.7.1 Overwash Fans

Description

Overwash is the landward transport of beach sediments across a dune area. Large coastal storms and their associated high winds, waves, and tides can result in overwash of the beach and dune system. During storm conditions, elevated storm tides and high waves may erode beaches and dunes, and the eroded sand can be carried landward by surging water. The sand and water may wash over or break through the dunes, and spill out onto the landward side of the barrier island. This deposit is usually fan-shaped and therefore is known as an overwash fan (or washover) fan (Delaware Sea Grant, 2009). An example of an overwash fan is shown in Figure VIII-24.

Figure VIII-24. Overwash at the Pea Island National Wildlife Refuge, Kinnakeet, NC (Credit: USGS Coastal & Marine Geology)

Consequences of natural overwash processes may include loss of, or damage to, property; or loss of access to property, roads and infrastructure as a result of flooding and sediment intrusion. In addition, if existing dunes are lowered by overwash barrier island may be more susceptible to breaching and therefore lose some of their flood risk management capacity (Donnelly et al. 2004). On the other hand, overwash fans are component of the sediment budget of barrier islands (Pierce 1969) and are also believed to be a relevant process in the rollover or retreat mechanism of some coastal barriers in response to RSLC (Dillon 1970, Kraft et al. 1973) by increasing the island width and providing a new foundation for back bay wetland growth. However, new inlet and flood tidal delta formation are believed to be a larger contributor to barrier island migration (Leatherman 1976) along the Atlantic coast.

Prevention of overwash and breaching may eliminate sand transport to the lagoon system and possibly preclude the ability of barrier islands to adapt to rising sea levels (Smith et al. 2008). Overtime, the lack of cross-barrier sediment transport may lead to a relatively narrow barrier island fronting relatively deep back bay water depths and therefore, more susceptible to catastrophic breaching and back bay flooding.

Allowing for natural overwash processes in developed barriers or barrier and back bay systems that are already very susceptible to breaching and flooding is risky and rarely feasible. A potential, albeit not yet commonly implemented, alternative is to construct overwash fans that mimic the beneficial effects of natural overwash without the damages typically associated with overwash. Engineered overwash fans would increase overall barrier island stability and back bay flood risk management capacity by increasing its width/volume and providing a substrate suitable for wetland growth. Sandy sediment could be mined from borrow sources “outside” the barrier island sediment budget system such as offshore borrow sites similar to those use for beach restoration projects. Other sources may include beneficial reuse of dredged sediments from adjacent back bay and inlet channels.

The level of risk reduction associated with engineered overwash features could vary significantly depending on the size of the overwash and specific site conditions. For example, a large overwash fan behind an existing low, narrow, barrier island could significantly reduce the likelihood of a breach and therefore the risk of back bay flooding during extreme events (up to a 1 percent flood). However, generally back bay flooding is mostly a function of the storm tide penetrating through existing inlets, particularly for the more frequent, smaller, coastal flood events. Combined with reasonable limitations in the size and elevation, this means that in most cases overwash fans will have relatively low risk reduction capacity (around a 10 percent flood). Nonetheless, over the long term engineered overwash fans may be essential to the overall resiliency of barrier islands, particularly those with high levels of development and limited opportunity for natural barrier island rollover and migration processes.

Generic Design and Parametric Cost Estimate

For the purposes of this study it was assumed that the engineered overwash fan would be approximately 2,000 feet long and 200 feet wide. It was further assumed that the average thickness of the overwash fan is 9 feet (from an existing bottom depth of 5 ft below MLLW to 4 ft above MLLW). Parametric costs assuming are summarized in Table VIII-24. Given the relatively small volume it was assumed that the fan would be built with a small to medium size hydraulic dredge and using a back bay source of sand. Alternatively overwash fan(s) could be constructed as part of larger beach restoration projects with offshore sand sources. This would approach would help offset the very costly mob/demob associated with oceangoing dredges.

Table VIII-24. Overwash Fan - Construction Quantities & Costs

Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$500,000	$500,000
Overwash Fill Volume	133,333	cu.yd.	$20	$2,666,667
Subtotal				$3,166,667
Contingency	25%			$791,667
Total Construction				$3,958,333
E&D	12%			$475,000
S&A	10%			$395,833
Total Estimated First Construction Cost				$4,829,167
Total Estimated First Construction Cost per Foot				$2,415
Annualized First Costs				$103
O&M	0%			$0
Total Estimated Annual Average Cost				$103

Summary: Overwash Fans Restoration Benefits, Impacts and other Considerations

Overwash fans occur when storm tides surge over or through low points in a dune system. The overwash water introduces sand on the landward side of the dune which is often configured in a fan shape. Natural processes usually introduce vegetation on the overwash fan creating new dune growth. This process of landward movement of beach sand is considered vital to the barrier beach system (island transgression). But Overwash fan deposits can often occur on private real estate and manmade features.

Overwash fan formation is part of natural barrier island survival. Overwash fan formation should not be considered a means of prevention or mitigation of storm surge damage. Overwash fan formation is often considered an unwanted consequence of dune washovers by storm surge waters.

VIII.7.2 Reefs

Description

Artificial reefs are established for various reasons; they may be used to restore degraded or damaged natural reefs, to provide three dimensional habitat structure above the bottom, to provide fishing and scuba diving opportunities, to deter illegal netting, and other purposes. Artificial reefs also enhance the resilience of coastal areas by reducing the degradation and shoreline erosion that would occur during a storm event.

Oyster reef restoration in particular provides spatially-complex substrate and benthic structure that is important for many estuarine organisms. A well-developed reef will typically consist of intricately layered formations of live oysters on the exterior and layers of old oyster shell forming the base and reef interior. Deep crevices created by the oyster shell provide refuge for numerous species of small aquatic organisms (USACE 2009).

Overall, embayments in the North Atlantic have been subject to erosion and subsequent deposition from the heavy sediment loads. Principal sediment sources are upland runoff that enters the bays from the watershed's river systems and shoreline erosion. As a result, productive natural reefs, especially oyster reefs, have been degraded or covered with silt, and have reduced productivity or function. Former natural reefs that are covered with sediment provide bottom habitat for certain benthic marine organisms, but do not support thriving reef communities or distinguishable aggregation sites, such as for schooling prior to annual migrations, and typically would not provide conditions associated with finfish foraging. The development of artificial reefs in the bays would provide a means to reestablish and enhance reef communities, while at the same time providing shoreline erosion risk management. This erosion risk management thus serves two beneficial purposes: reducing risk to fastland as well as structures, and preventing sediment from covering the reef. The structural material provides suitable surfaces for attachment of small filter feeders such as barnacles and marine vegetation, whereas voids and passages in the reef structures provide cover from predators for crabs and juvenile and small fish. Sedimentation effects are reduced, as the vertical height of artificial reef structures provides longevity relative to existing reefs that are relatively level, near the bottom and more susceptible to the effects of sedimentation.

Reef sites may be developed using natural materials such as oyster shells, clam shells, or rock. Additionally, reef material may be obtained from discarded construction debris such as clean, rebar-free concrete, slag, metals (steel, aluminum), rubber or plastic. Also, man-made structures specifically designed for reef creation can be used, such as Reef Balls™ which are made of concrete, or other similar designs. The use of the latest generation of designed reef structures with specific biologically oriented features provides a significant improvement over debris materials and earlier designed structures. One benefit to Reef Balls™ is that their design and performance are supported by readily available engineering, scientific and monitoring data and there is a proven track record in providing value habitat and fishing opportunities in the mid-Atlantic region (e.g., New York, Virginia, New Jersey, and Massachusetts reefs) for benthic organisms, crustaceans, and multiple fish species. They are not specifically designed, however, for shore risk management and may need to be incorporated with other reef structures and materials to achieve this goal.

Generic Design

Reef design and restoration technology has advanced to a state of practice in which reef products that are specifically designed and proven to achieve biological objectives have demonstrated a significant potential to provide three dimensional structures for colonization by benthic marine organisms, cover for crabs and juvenile and small fish, and foraging sites for larger fish. Modification of the reef design to also consider shoreline erosion risk management can readily be accomplished using the technology components available for reef construction.

The water depth in which the reef would be located is an important cost factor for achieving the goal of shoreline risk management. Deeper water would require more material to create a reef with a top elevation high enough to break large waves that would occur during the storm events with high water elevations. For this application, it is proposed that the top elevation of the reef be established at -1 ft MLLW that will maintain the structure underwater for most of the time while placing it as high as possible to reduce wave energy during storms. Note that generally wave reduction is not the controlling design factor in oyster reef projects. Instead these are typically driven and ecological restoration goals. Therefore, in most cases restored reefs are relatively low relief (1 to 2 feet above the existing bottom elevation). A higher relief reef will be more effective at reducing waves but it will also be significantly more costly for the same restoration area.

For generic design and costs estimating purposes it is assumed that the reef is located in -5 ft MLLW and has a width relative to the shoreline of about 100 ft. The reef is constructed using riprap as a base material up to elevation -2 ft MLLW, then placing a one-foot layer of oyster shell on top to bring the final elevation up to -1 ft MLLW (i.e., 4 feet above the bottom). The riprap would have a median weight of 50 pounds and would be obtained from a local quarry. Oyster shell would likely have to be hauled by rail from a quarry in Florida (near Tallahassee) that currently is the only location to obtain the material. Both the riprap and the oyster material would be transferred to a shallow-draft barge for placement in the water.

Parametric Unit Cost Estimate

Table VIII-25 presents construction cost estimates for the schematic reef design. The costs are developed for a shoreline length of 5,000 feet and reduced to provide a cost per linear foot linear foot of reef. First construction costs are about $4,752 per linear foot of reef; annualized costs are about $203 per linear foot.

Table VIII-25. Oyster Reef - Construction Quantities & Costs				
Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$250,000	$250,000
Base Stone	83,333	ton	$150	$12,500,000
Oyster Reef Material	18,519	cu.yd	$200	$2,314,815
Seeding of Top Layer	11.5	acre	$45000	$516,529
Subtotal				$15,581,344
Contingency	25%			$3,895,336
Total Construction				$19,476,680
E&D	12%			$2,337,202
S&A	10%			$1,947,668
Total Estimated First Construction Cost				$23,761,549
Total Estimated First Construction Cost per Foot				$4,752
Annualized First Costs				$203
O&M	0.0%			$0
Total Estimated Annual Average Cost				$203

Summary: Reef Benefits, Impacts and other Considerations

Reef breakwaters can provide shoreline risk management by reducing wave energy and creating sand deposition areas which grow the nearby shoreline. Reef breakwaters can be installed with minimal environmental impact and can provide area for habitat growth. A variety of manufactured structures such as reef balls and wave attenuation devices (WADs) can be used. These structures are designed to encourage marine habitat growth.

Calm waters in lee of the reefs encourages accumulation of sediment in the vicinity of the reef as an intended consequence, however, this condition often creates areas of erosion down shore. Reef breakwaters can become obstacles to boat traffic in lower tide conditions, depending on specific construction applications.

Living reef breakwaters are a relatively new technology and new specific applications would require some site-specific research into their effectiveness in preventing beach erosion.

VIII.7.3 SAV Restoration

Description

Submerged aquatic vegetation (SAV) are grasses that grow to the surface of shallow water, but do not emerge from the water surface. SAV performs many important ecosystem functions, including wave attenuation and sediment stabilization, water quality improvement, primary production, food web support for secondary consumers, and provision of critical nursery and refuge habitat for fisheries species, as well as for the attachment of epiphytic organisms (USACE 2008).

SAV grows only in shallow water to allow sufficient light to reach them. SAV generally grows taller than five or six feet but have to be submerged almost all the time to avoid drying out. Special air cells allow SAV to float upright in the water column (NOAA Chesapeake Bay Office n.d.)

Generic Design

For this study, it is assumed that the top elevation of the SAV substrate will be established at -1 ft MLLW that will maintain the vegetation underwater for most of the time while placing it as high as possible to reduce wave energy during storms.

To construct the SAV bed, sand would be placed in a layer on the bottom with a small hydraulic dredge to build it up, the individual plants would be installed using snorkel. The depth of the final elevation would be shallow enough to permit snorkel versus SCUBA. This would require scheduling placement around low tide. Alternatively, SCUBA could be used if it is desired to plant SAV at any phase of the tide.

It is assumed that the SAV bed is constructed over and existing bottom at -5 ft MLLW and has a fill width of about 300 ft with a generally flat slope.

Parametric Cost Estimate

Table VIII-26 presents construction cost estimates for the schematic SAV bed design. The costs are developed for a shoreline length of 5,000 feet and reduced to provide a cost per linear foot linear foot of reef. First construction costs are about $2,423 per linear foot of SAV bed; annualized costs are about $103 per linear foot.

Table VIII-26. SAV Restoration - Construction Quantities & Costs

Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$500,000	$500,000
Sand Fill	222,222	cu.yd.	$20	$4,444,444
SAV Plantings	750,000	each	$4	$3,000,000
Subtotal Construction				$7,944,444
Contingency	25%			$1,986,111
Total Construction				$9,930,556
E&D	12%			$1,191,667
S&A	10%			$993,056
Total Estimated First Construction Cost				$12,115,278
Total Estimated First Construction Cost per Foot				$2,423
Annualized First Costs				$103
O&M	0.0%			$0
Total Estimated Annual Average Cost				$103

Summary: SAV Restoration Benefits, Impacts and other Considerations

Submerged aquatic vegetation (SAV) helps to buffer shorelines by stabilizing sediments with plant roots. SAV also provides habitat, food and shelter for an array of marine life, improves water quality and clarity and traps suspended particles.

Since SAV's are fragile, SAV restoration zones must be safeguarded from significant human activities.

Local water quality is a critical factor in SAV restoration success. Suitability for SAV restoration must be assessed at particular target sites.

VIII.7.4 Wetlands

Description

Coastal wetlands may contribute to coastal risk flood risk management wave attenuation and sediment stabilization. The dense vegetation and shallow waters within wetlands can slow the advance of storm surge somewhat and slightly reduce the surge landward of the wetland or slow its arrival time (Wamsley et al. 2010). Wetlands can also dissipate wave energy; potentially reducing the amount of destructive wave energy, though evidence suggests that slow-moving storms and those with long periods of high winds that produce marsh flooding can reduce this benefit (Resio and Westerlink 2008). The magnitude of these effects depends on the specific characteristics of the wetlands, including the type of vegetation, its rigidity and structure, as well as the extent of the wetlands and their position relative to the storm track.

Figure VIII-25. Elders East Wetland Restoration, Jamaica Bay, NY, Under Construction (Galvin Brothers, Inc.)

Functionally restored wetlands act in the same manner as natural wetlands, though design features may be included to enhance risk reduction or account for adaptive capacity considering future conditions (e.g., by allowing for migration due to changing sea levels). An example of an engineered wetland under construction at the Gateway National Recreational Area in Jamaica Bay, NY is shown in Figure VIII-25.

Generic Design

For this study, the tidal wetlands that would be constructed along a shoreline do not have a protective rock breakwater/sill along the outer edge. As the goal is to reduce coastal erosion from flooding while maintaining dynamic shoreline processes and providing habitat for organisms such as fish, crabs and turtles, it would be necessary for wetland designs to be wider than that for a living shoreline. The top elevation of the wetland will be placed at MHHW (assumed about + 4 ft above MLLW) to protect the plants from being washed away during a tidal cycle and from regular, frequently occurring waves.

Further, the material to be used in the beach region will be relatively coarse sand material with a minimal amount of fines (less than 10 percent passing a #100 sieve). The sand material in the wetland area behind the beach could contain a higher quantity of fines; however sand material is preferred in the wetland to allow plant roots to develop more effectively.

Tidal wetlands are natural landscape features that function primarily under normal tidal range conditions and provide a varied mix of habitat such as: shallow water, intertidal, beach, marsh and dune. They provide some benefits as a wave reducing component by functioning as shallow water under high water and storm conditions. A typical wetland for this study would be fairly wide to incorporate the beach and provide an effective region for wave breaking and wave energy reduction even if significant erosion of portions of the wetland would occur during a storm event.

The wetland would be constructed in fairly shallow water, usually less than 5 ft below mean lower low water (MLLW). The actual water depth would site-specific, and for shallower water material quantities and subsequent construction costs for a given length of shoreline would be lower. For purposes of this generic design, it is assumed that the water depth that the wetland would be constructed is -5 ft below MLLW.

The sand that is placed to construct the wetland should be relatively coarse to minimize loss of material from the waves and currents that can enter through the breakwater. It is common to specify sand material with a maximum fines content of 10 percent. The slope of the wetland surface should be fairly flat. It also would be necessary to install tidal channels into the wetland to allow more effective water exchange and allow for fish and other aquatic organisms to enter and utilize the wetland plants and refuge.

Wetlands should be designed to have both low and high marsh vegetation, and a 50/50 design ratio is preferred and typical. Site specific conditions as well as local preference could change this ratio, as well as environmental conditions following construction. It is practical and acceptable to allow the ratio to vary over time and not be strict about maintaining a certain ratio. Low marsh vegetation is typically *Spartina alterniflora* and high marsh vegetation is typically *Spartina* patens.

Another item of importance to incorporate into a wetland is to take measures to prevent waterfowl (primarily Canada geese) from eating the plants. Non-migratory Canada geese are common along the east coast, and a flock of them can very quickly destroy newly planted vegetation, often pulling a new plant out by the roots. Goose-exclusion fencing is mandatory to prevent this predation and allow the marsh to grow and develop into a mature system. The fencing should be installed to prevent geese from flying or walking into the marsh. Once the grasses have had time to develop a strong root system, the fencing is no longer required and the waterfowl can eat the grasses without destroying the marsh.

For quantity a cost estimating purposes it was assumed that a typical wetland restoration would consist of a 300 feet wide platform constructed to +4 ft MLLW (approximately MHHW). The outside side slope of the wetland is assumed to be 15H:1V. The existing bottom slope is also assumed to be approximately 15H:1V. The design also includes vegetative marsh grass plantings 1.5 ft on center. It is assumed that the wind fetch distance is relatively short (on the order of one to two miles) and the average waves are about one to two feet so that additional wave risk management measures along the exposed wetland perimeter are not required.

Parametric Unit Cost Estimate

Table VIII-27 presents construction cost estimates for the generic wetland design. The costs are developed for a shoreline length of 5,000 feet and reduced to provide a cost per linear foot linear foot of wetland. A small hydraulic dredge would be used to pump the sand into the wetland area. Post-placement shaping of the wetland to create tidal channels would be performed using low-ground pressure construction equipment. First construction costs are about $2,593 per linear foot of wetland; annualized costs are about $123 per linear foot.

Table VIII-27. Wetlands - Construction Quantities & Costs

Item	Quantity		Parametric Estimate	
	Number	Unit	Unit Cost	Total Cost
Mob/demob	1	LS	$500,000	$500,000
Sand Fill	333,333	cu.yd.	$20	$6,666,667
Grass Plantings	666,667	each	$2	$1,333,333
Subtotal				$8,500,000
Contingency	25%			$2,125,000
Total Construction				$10,625,000
E&D	12%			$1,275,000
S&A	10%			$1,062,500
Total Estimated First Construction Cost				$12,962,500
Total Estimated First Construction Cost per Foot				$2,593
Annualized First Costs				$111
O&M	0.5%			$13
Total Estimated Annual Average Cost				$123

Summary: Wetlands Benefits, Impacts and other Considerations

Wetlands trap and hold floodwaters and absorb wave energy which would otherwise degrade a shoreline. Wetlands recharge groundwater, remove pollution and provide diverse habitat as well as recreational activities.

Due to land acquisition costs, Restoration/preservation of existing wetlands is likely to be more successful than creation of new wetlands. Wetland restoration design considerations include site selection criteria, hydrology, water source and quality, substrate and plant material selection and handling, buffer zone placement and long term management.