ENGINEERING APPENDIX
CIVIL

NASSAU COUNTY BACK BAYS
COASTAL STORM RISK MANAGEMENT
FEASIBILITY STUDY

PHILADELPHIA, PENNSYLVANIA

APPENDIX C

AUGUST 2021

NCBB
Nassau County Back Bays Study

U.S. Army Corps of Engineers
Philadelphia District

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1 INTRODUCTION

The North Atlantic Coast Comprehensive Study (NACCS) was conducted to address the flood risks of vulnerable coastal populations in areas that were affected by Hurricane Sandy within the boundaries of the North Atlantic Division of the Corps. The Nassau County Back Bays (NCBB) area was identified as a "focus area" within the NACCS study. This Civil Engineering Appendix developed by USACE Philadelphia District (NAP) discusses the engineering and design work conducted to layout and evaluate potential structural, non-structural and other alternative design solutions for improved risk management against flooding in the NCBB Region of Long Island, New York. See Figure 1.1 below for reference of the project study area. The landward limit of the study area is bounded by the extent of the 500-year floodplain. The map below and all other figures related to structural plan formulation developed by NAP Civil Engineering have been included as Exhibit A "CENAP-EC-EC Map Deck" in this Appendix.

Figure 1.1 – NCBB Project Study Area (Courtesy of USACE-NAP)

The NACCS Tier 1 Screening provided pre-compiled reference data for initial screening of design alternatives. In addition to initial screening completed in the NACCS, information from the Coastal Storm Risk Management (CSRM) Feasibility Study Draft Report developed in December of 2018 by Moffat & Nichol, Inc. (M&N) for USACE New York District (NAN) was utilized as a reference in the development of the NAP structural plans. Two (2) separate documents developed by Moffat & Nichol are referenced herein: Structural Coastal Storm Risk Management Features and Conceptual Design for the East Rockaway Inlet, Jones Inlet and Fire Island Inlet Storm Surge Barriers (Moffat & Nichol, 2018). The plans and associated design features from the 2018 report helped inform the NAP planning process but did not govern any NAP formulation methodologies. See Section 2.1 for further discussion regarding initial screening.

After the study scope pivot occurred in early 2020 (See Main Report) the NAP team was tasked with refining the study to identify flood risk management solutions in Nassau County only. The spatial limitations of the project were further increased when guidance from the vertical team was given to consider the Coastal Barrier Resource Act (CBRA) System Unit as a restrictive measure in the plan formulation phase. See Figure 1.2 for the limits of the CBRA System Unit in relation to the Nassau County Back Bay Study Area. These scope reductions created significant alterations in the plan formulation phase and greatly influenced the plans and measures developed by NAP.

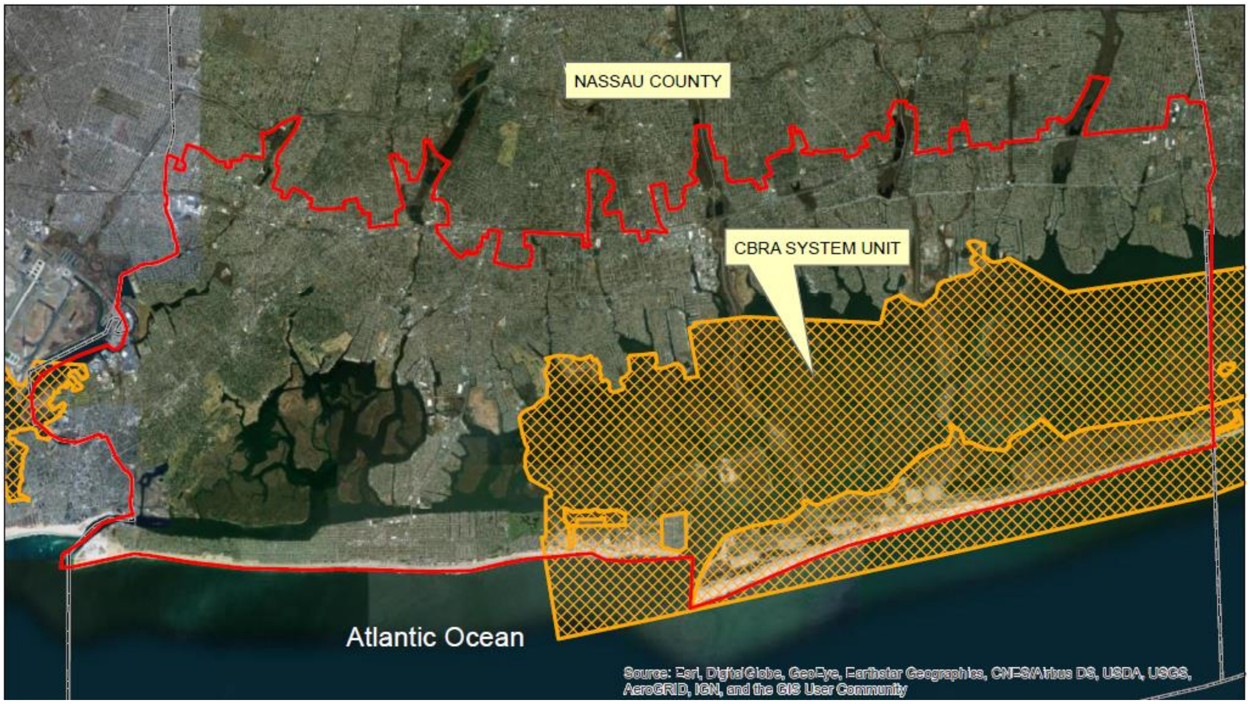


Figure 1.2 – CBRA Limits within NCBB Project Study Area (Courtesy of USACE-NAP)

The initial scope of the Civil Engineering portion of the NAP study was to develop two (2) structural flood control solution types to be evaluated as part of the plan formulation process: perimeter plans (consisting of floodwalls and levees) and storm surge barriers. Both solutions would be evaluated separately for screening analyses, similar to the approach taken by M&N in 2018. However, during initial screening, it was determined that Storm Surge Barriers would not be considered further in the plan formulation process due to multiple issues encountered in the initial screening. See Section 4.0 of this Appendix for more information regarding the removal of Storm Surge Barriers from the project scope.

Therefore, the perimeter plan would be the only structural solution used to determine a focused array of alternatives that was further evaluated to determine the Tentatively Selected Plan (TSP). Section 2.0 of this appendix covers the perimeter plan formulation and associated analyses. Designs from other USACE District studies were analyzed for suitability of incorporating these features as measures in this study. Parametric data from each were utilized for determination of with-project costs in the plan formulation study.

Civil Engineering was also tasked with supporting plan development for Non-Structural solutions and researching other potential design alternatives such as Natural and Nature Based Features (NNBF) and Adaptability Measures that could potentially be incorporated into the TSP. Discussion and Analyses regarding these alternative measures have also been included herein.

2 PERIMETER PLAN ANALYSES

2.1 Background

In 2018 M&N developed a CSRM Feasibility Study for the NCBB study area. The purpose of the report was to provide the necessary engineering basis to support the evaluation and development of conceptual designs for shoreline- based measures (SBMs) that were a part of the initial NCBB study alternatives. SBMs as defined in the 2018 report are composed of flood risk reduction measures such as levees and floodwalls located along or inward of the shoreline.

The report details several potential SBMs that could be implemented for CSRM benefits within the study area. The SBMs referenced include floodwalls, levees, bulkheads, seawalls, and revetments, road raising and navigational features. All these features combined to create a comprehensive perimeter plan that encompassed the majority of the communities within the scope of the NCBB study. See Figure 2.1.1 for reference of the M&N perimeter plan known as “Alternative 2”. The blue line represents the Alternative 2 plan. Note that this plan was formulated for the original study area which included Nassau County and portions of Suffolk County to the east and the New York City Borough of Queens to the west.

Figure 2.1.1 – M&N Perimeter Plan Alignment (Courtesy of Google Earth)

When the study was transitioned to NAP in 2019, the conceptual design work conducted by M&N in the aforementioned plan formulation would become the foundation for the NAP perimeter plan analyses. However, after a 2020 project scope pivot, this work would be refined by USACE-NAP project team to Nassau County only and would include a more rigorous screening process for determination of where SBMs would be applicable. Also, the USACE-NAP Project Development Team (PDT) drew from not only the M&N report, but several other USACE district studies, most notably the New Jersey Back Bay (NJBB) study, in the development of preliminary designs for SBMs.

2.2 Initial Development

In early 2020, the PDT was tasked with progressing the initial CSRM study efforts completed by M&N to a conceptual level of design suitable for evaluation and consideration in a TSP. The initial task in the perimeter plan development was for the PDT to determine where SBMs would be suitable based on the following elements: storm damage susceptibility, critical infrastructure density and socio-economic vulnerability. These factors were discerned in various PDT discussions and in conjunction with guidance from the vertical team as a part of the refinement of the study scope of work.

Once the plan formulation parameters were established, the team then defined those elements using mapping exercises. For storm damage susceptibility, all areas within the study region exhibit potential for flood damage at varying storm event levels. However, to achieve refinement of the scope, the metric chosen for storm damage susceptibility for this study would be the Average Annual Damages (AADs) suffered by infrastructure within a community. Infrastructure for the purposes of this evaluation refers to all existing vertical structures and their associated parcels, both public and private. See mapping below in Figure 2.2.1 of AADs for infrastructure within the 500-year floodplain of Nassau County. Refer to Economic Appendix F for additional information regarding data source of AADs in Nassau County. The presence of high AADs in a community in terms of both value and quantity helped determine which communities have the highest level of storm damage susceptibility.

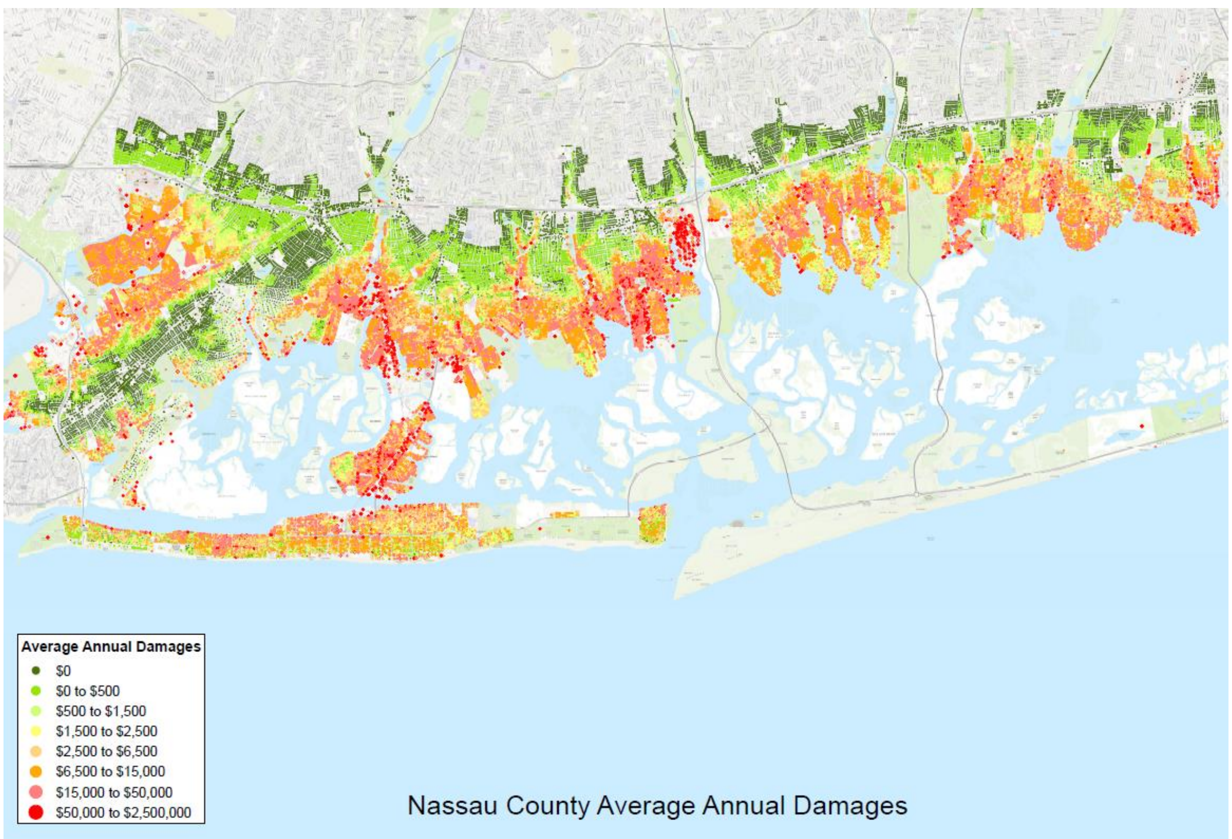


Figure 2.2.1 – NCBB Average Annual Damages (Courtesy of USACE-NAP)

The second element considered in the initial plan development is critical infrastructure density within the local communities. Critical infrastructure is defined in the NACCS and the Department of the Army Field

Manual (FM) 3-34.170 SWEAT-MSO (Sewage, Water, Electricity, Academics, Trash, Medical, Safety and Other Considerations) process as infrastructure that could be considered essential services, operations or necessary functions to ensure civil order. In discussion with the PDT, this definition can be further elaborated for the purposes of flood risk management as infrastructure that is essential to the community to resume functionality after a major coastal storm event. Those structures were also mapped by NAP to ascertain their location and locations where high concentrations of the types of facilities covered in Army Field Manual exist. See Figure 2.2.2 below for the NCBB Critical Infrastructure map.

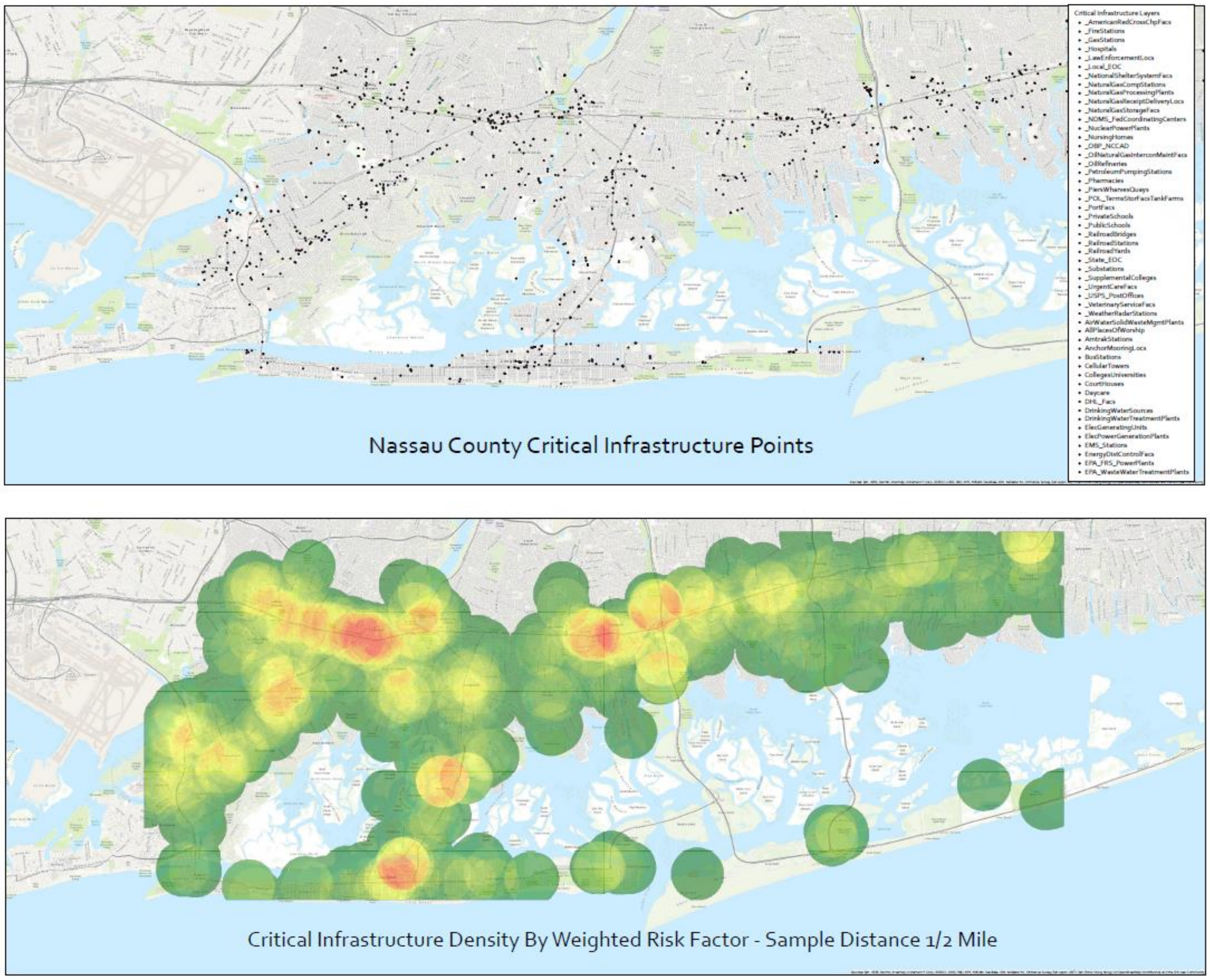


Figure 2.2.2 – NCBB Critical Infrastructure (Courtesy of USACE-NAP)

The last element of the scope refinement is the measure of socio-economic vulnerability in each community. Socio-economic vulnerability relates to the intrinsic demographics of the various communities that includes considerations such as age, income, race, etc. Communities with high socio-economic vulnerability are the third component in the discernment process of determining the Highly Vulnerable Areas (HVAs) within the NCBB study area. Figure 2.2.3 show the AADs heat map with socio-economic factors included. The rating index used to scale the AADs to socio-economic vulnerable areas in this figure was developed by the NAP Economics team. Refer to Appendix F for more information on the development of this scale and the associated results.

Nassau County Average Annual Damages (AAD)

Figure 2.2.3 – AAD Heat Map (Courtesy of USACE-NAP)

The information provided in the mapping exercises to discern SBM suitability provided the PDT with key information to determine HVAs within the study area. HVAs would be chosen for community specific perimeter plans, whereas the rest of the study area would be evaluated with Non-Structural CSRM solutions only. The HVAs identified by the PDT are shown in Figure 2.2.4 below. The four (4) areas chosen account for nearly 30% of the study area land mass.

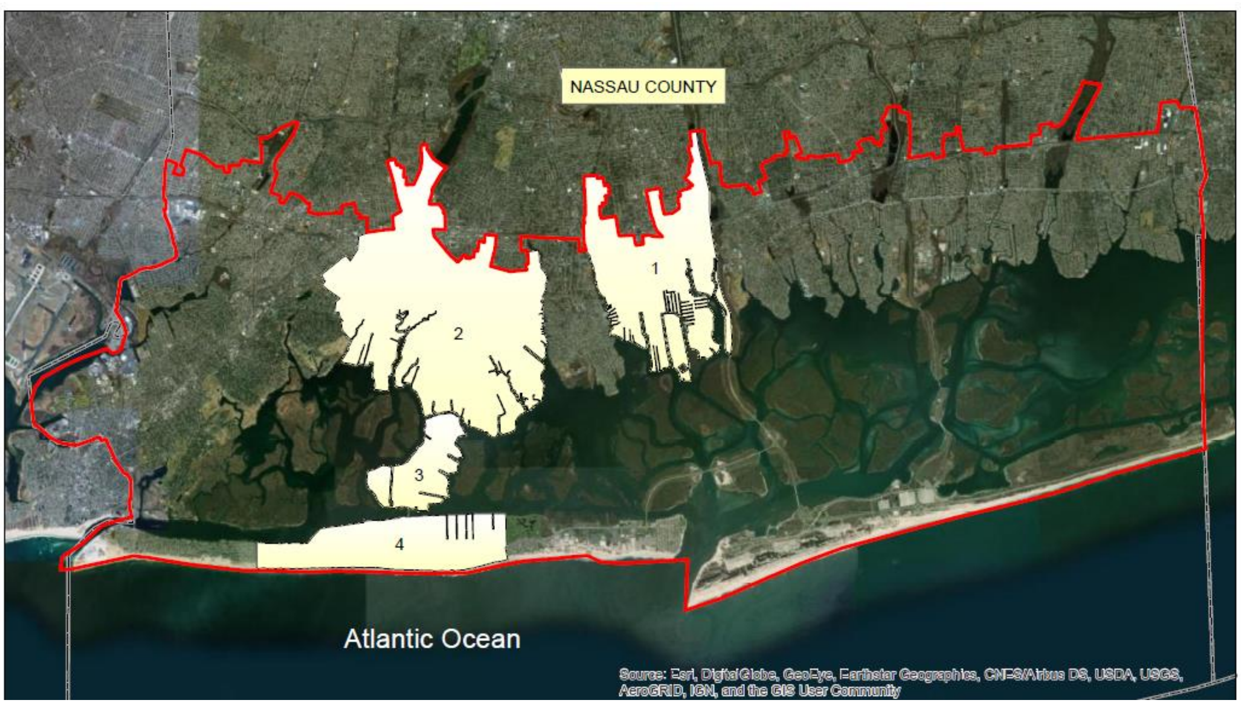


Figure 2.2.4 Highly Vulnerable Areas (Courtesy of USACE-NAP)

The areas shown in Figure 2.2.4 are defined as the following:

- Highly Vulnerable Area No.1 – Village of Freeport
- Highly Vulnerable Area No. 2 – Village of East Rockaway to the Hamlet of Oceanside (Includes Hamlets of Bay Park and Oceanside within the Town of Hempstead)
- Highly Vulnerable Area No. 3 – Village of Island Park & Vicinity (Includes Hamlet of Harbor Isle and Barnum Island within the Town of Hempstead)
- Highly Vulnerable Area No. 4 – City of Long Beach

With the HVAs identified, the entire study area could be divided into economic reaches by county and municipality. Reaches were then combined into groups based upon geographical conditions (hamlets, villages, towns, and cities), hydraulic connectivity and high vulnerability designation. Google Earth mapping was utilized to enclose each reach within a polygon for economic analysis. Water surface profiles were generated in HEC-FDA to determine the benefit pool for the reach and the Average Annual Net Benefits (AANB) were determined (See Appendix F for Economic Analysis). See Figure 2.2.5 for the resultant reaches defined by the PDT. Upon completion of the reach assignments, structural plan formulation within the highly vulnerable reaches could begin.

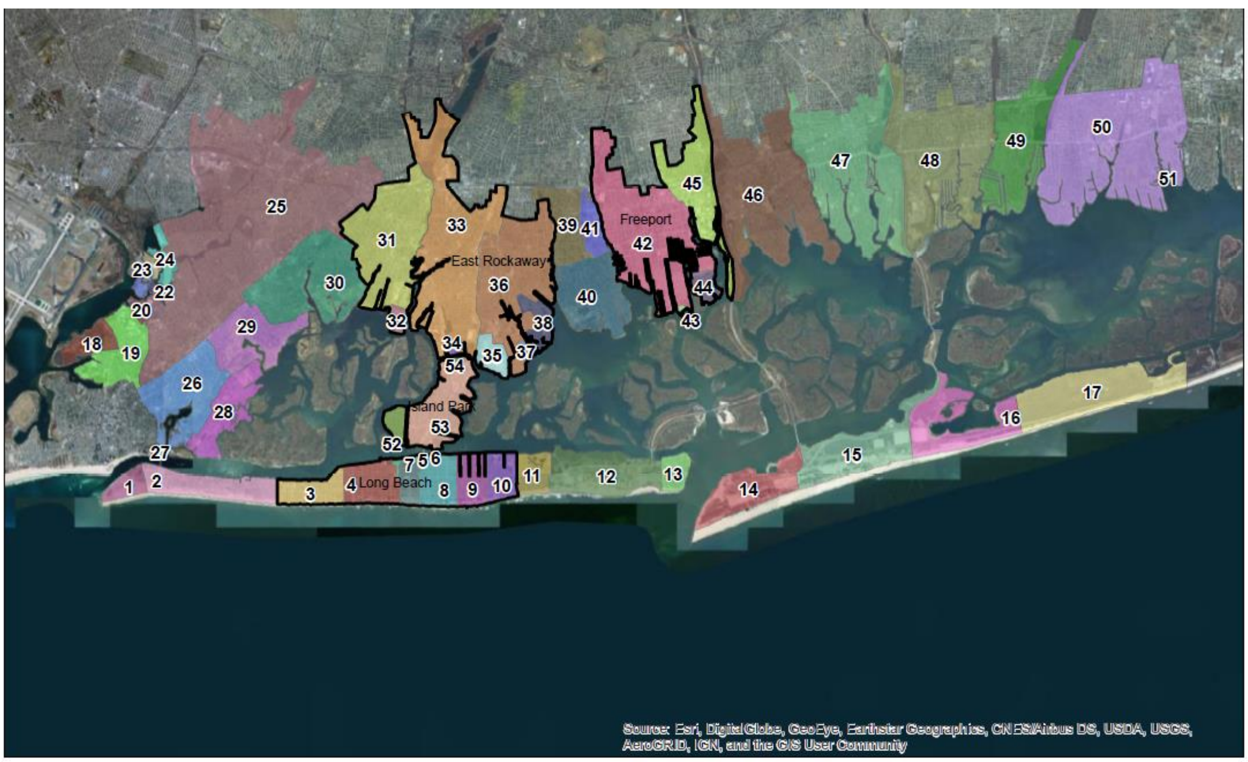


Figure 2.2.5 – NCBB Economic Reaches (Courtesy of USACE-NAP)

2.3 Perimeter Plan Screening

2.3.1 Structural Alignment Formulation

The initial step in the perimeter plan screening process was for the PDT to determine the alignments of structural risk management within the HVAs. In order to achieve this goal, the team had to determine the design intentions for alignment development. In accordance with the NJBB study, the team decided to look at the HVAs with consideration to the 20 year and 100-year floodplains or 5% and 1% Annual Exceedance Probability (AEP), respectively.

In addition to the 5% and 1% AEP floodplain, the team also decided to incorporate the 5-year floodplain or 20% AEP into the plan formulation screening. The decision to incorporate the 20% AEP event was influenced by PDT observations during the NJBB study. The 20% AEP captures high frequency events and nuisance flooding that municipalities often deal with annual maintenance costs. Many PDT members saw that New Jersey had significant damages associated with high frequency events which interested the team in incorporating a lower level of risk management into this study to analyze the potential AANB with respect to structural solutions. Also, NAN is currently working on a project in the nearby Jamaica Bay area where 20% AEP risk management is being evaluated within the Planning, Engineering and Design (PED) Phase. The water levels for this study were provided by the "Nassau County Back Bays Study Without Project Water Levels" report completed by USACE Engineer Research Development Center (ERDC) in 2015 for the NACCS. See Figure 2.3.1.1 for locations of the NCBB water monitoring stations. The Freeport, Island Park and Reynolds channel monitoring stations would provide the floodplain data for the HVAs. For more information regarding water levels and all other hydraulic design considerations, see Hydraulics and Hydrology (H&H) Appendix within this report.

Figure 2.3.1.1 NCBB Water Level Monitoring Stations (ERDC 2015)

The next major piece in the perimeter plan development was the mapping of the existing ground elevations of the study area. A Digital Elevation Model (DEM) for the entire study area had been provided by NAN. The DEM was mapped for each HVA and given an elevation scale for usability. Figure 2.3.1.2 contains an excerpt from the CENAP-EC-EC Map Deck for the DEM in the Village of Freeport. The floodplains inform the designer on where structural risk management is necessary to prevent the flooding during the associated design event. DEM mapping allows the designer to complete the termination points of those potential structural alignments based on existing ground elevation in relation to the associated level of design. The elevation ranges identified in the mapping were developed through a trial and error process to create the best visual aid for the designer.

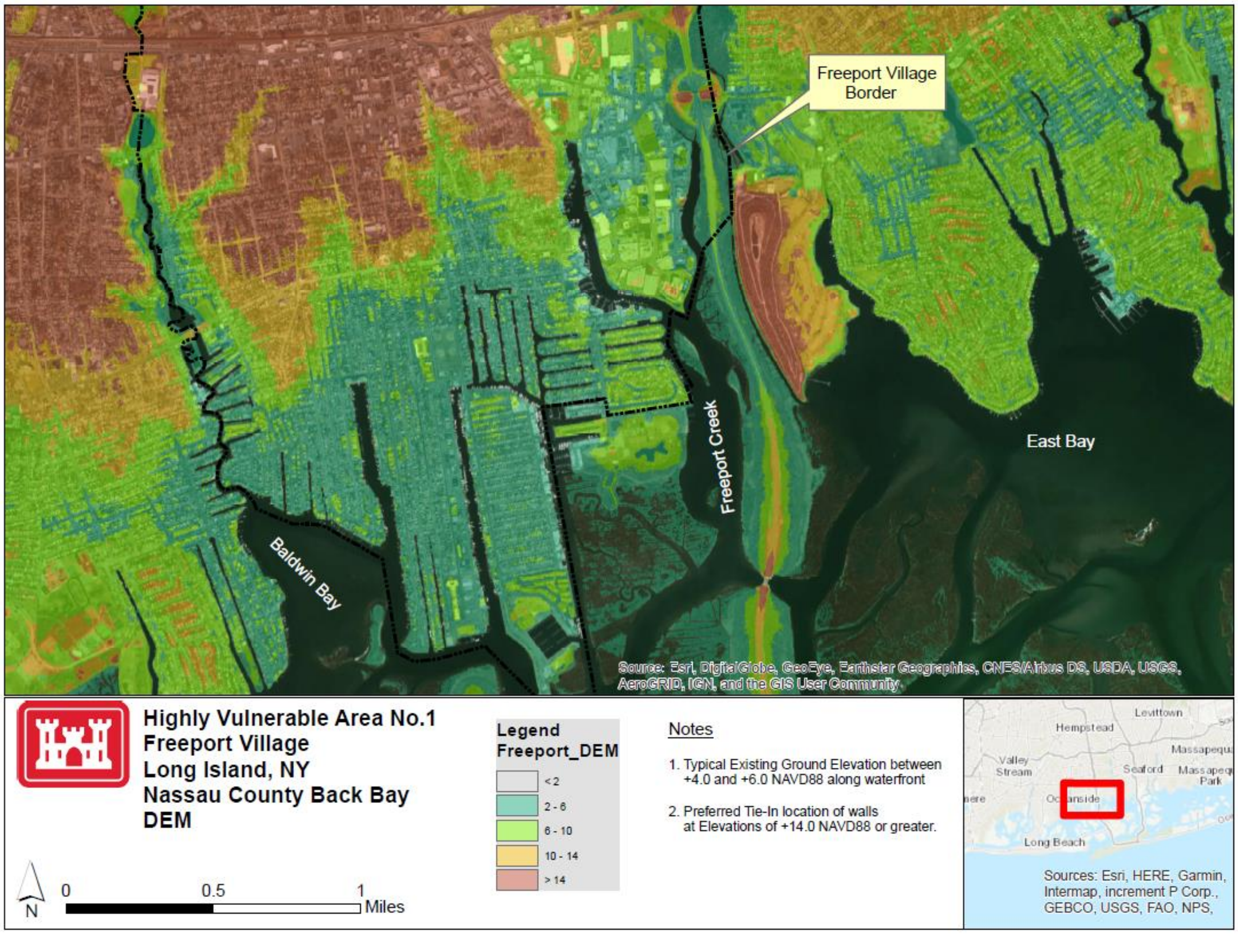


Figure 2.3.1.2 Village of Freeport DEM (Courtesy of USACE-NAP)

Using the NCBB information on floodplains and existing ground elevations, the PDT could create potential alignments with the HVAs. An alignment was laid out for each HVA (completed in Google Earth) along the bay frontage of each reach or at other suitable locations. Perimeter plan alignments were assumed to tie-in to dunes or seawalls of existing USACE projects on the ocean side of the barrier islands for the City of Long Beach. All other perimeter plans were to tie-in to high ground upland of the bay front based on the DEM. All upland tie-in locations were made at Elevation +14.0 NAVD88 or greater for the 5% and 1% AEP plans. The team also referenced the perimeter alignments developed in the 2018 M&N report to aid in the discernment of the risk management location along the shoreline. These layouts did not consider the best horizontal placement of the line but did approximate the existing shoreline or exposed perimeter. Figure 2.3.1.3 is an example of an alignment created using these design guides for the 100 year or 1% AEP level of risk management in the Village of Freeport. The SBMs referenced in the figure will be defined in Section 2.3.2 Perimeter Plan Measures of this appendix. It is also important to note that the team had to avoid laying out any alignments within the CBRA System Unit previously referenced in this appendix.

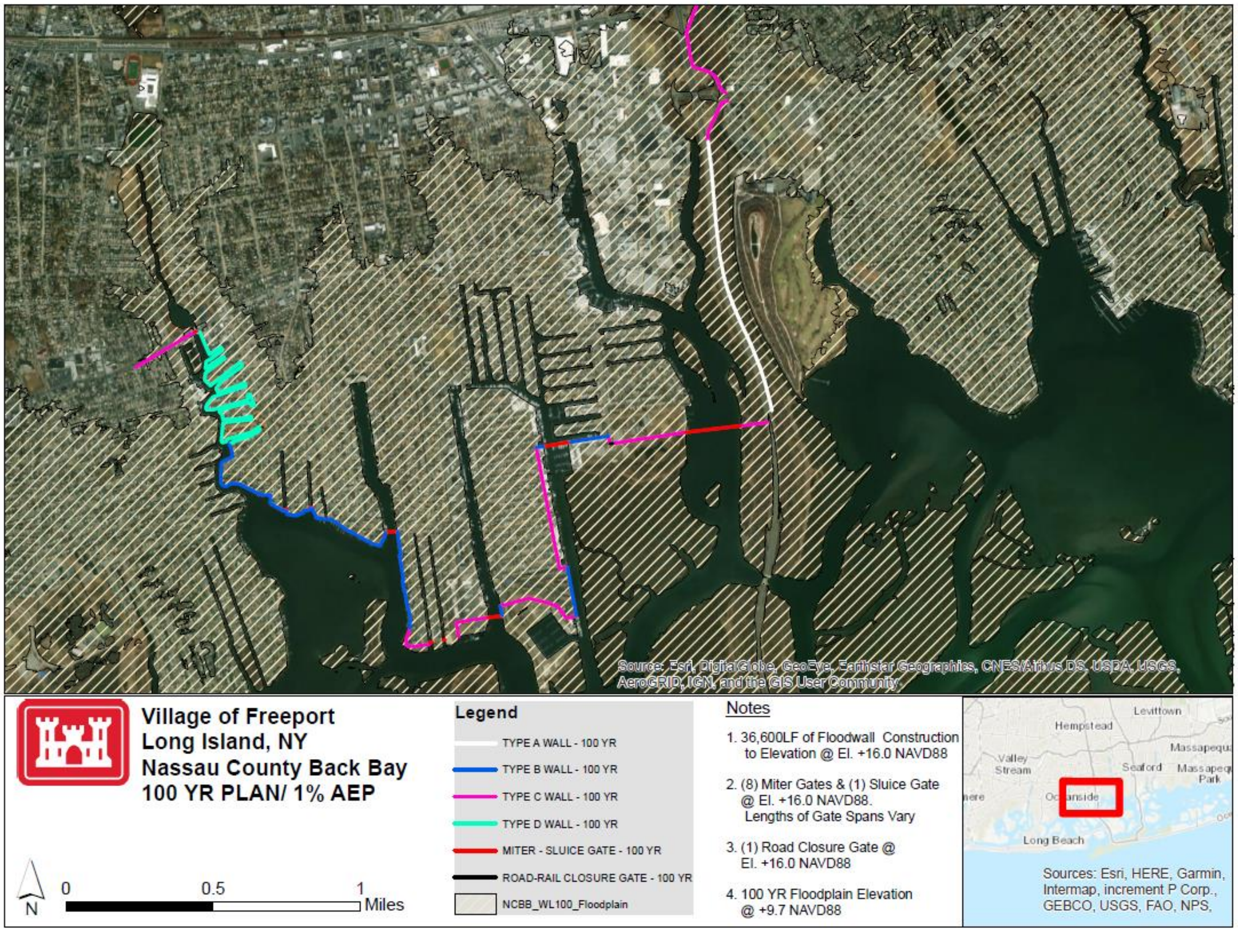


Figure 2.3.1.3 Village of Freeport 1% AEP Plan (Courtesy of USACE-NAP)

Another important element in alignment formulation was the determination of the potential SBMs crest elevation. SBM crest elevation was determined for each flood event scenario (20%, 5% and 1% AEP) using various hydraulic design factors such as still water level, design wave height, wave overtopping, RSLC and other considerations. For the evaluation and determination of the crest elevations, please see H&H Appendix. A summary of the design elevations can be found in table 2.3.1.1 below.

Table 2.3.1.1 – SBM Crest Elevation

AEP (%)	Return Period (YR)	Design Crest Elevation (FT)
20	5	+9.0
5	20	+13.0
1	100	+16.0

*All Elevations in NAVD88 Vertical Datum

The SBM crest elevations were very important in determining where to tie-in the perimeter plan alignments. For example, the 1% AEP plan would need to tie-in to a location at least at elevation +16.0 NAVD88 to ensure that a contiguous level of risk management is maintained throughout the alignment. It is also important to note that many of the tie-in locations are significantly inland due to the low-lying nature of the waterfront of the HVAs. This varies greatly from the M&N perimeter plan since Alternative 2 mostly is an all-encompassing perimeter plan. With terminations occurring so far inland in this study to reach the desired tie-in location, some H&H design constraints may be extraneous (such as wave overtopping) to include in the design crest elevation for areas not subject to direct impact from bay front hydrodynamic loading conditions. However, the process to determine multiple crest elevations for a single event based on geographic location would require a significant amount of modelling. Therefore, the design crest elevation is considered the same throughout the alignment for the purposes of this phase of the plan formulation. These design crest elevations will also be reflected in the SBMs the team has chosen to include within these alignments.

Alignments were developed in two screening cycles. An initial Cycle 1 Screening created perimeter plan alignments and critical infrastructure plan alignments in for the four (4) HVAs for the 20%, 5% and 1% AEP Plans. All floodwall and levee quantities associated with the Cycle 1 screening was utilized for the initial cost estimate and economic analyses to determine potential TSP options. After initial results were observed and discussed amongst the PDT and members of the Vertical Team, it was determined that a Cycle 2 Screening would be necessary. The goal of the Cycle 2 screening process was to refine Cycle 1 plans to maximize the efficiency of the developed plans in the Cycle 1 analysis based on field observations and any additional information collected during the TSP development. Refinement included but was not limited to: reduction of excess or unnecessary linear footage of SBMs, addition of linear footage of SBMs in areas not previously captured in Cycle 1, and refinement of location of SBMs based on additional design considerations. The Cycle 2 screening refined the various HVA plans developed in Cycle 1 and added three (3) additional critical infrastructure plans that were located outside of the HVAs based on PDT discussions and field observations during the TSP development. All quantities and alignments referenced herein refer to the finalized alignments created in Cycle 2.

2.3.2 Perimeter Plan Measures

For the NJBB study the NACCS Tier 1 floodwall was assumed for the entire line of protection to generate initial with-project quantities and costs. The NACCS floodwall is a pile supported, reinforced concrete T-Wall, with an unsupported stem height of 10 feet above ground and 2.5 feet thickness. Rows of piles spaced every 7 feet at lengths between 15 and 50 feet, depending on the soil conditions, form the foundation of the structure. See Figure 2.3.2.1 below. Piles are not shown for clarity.

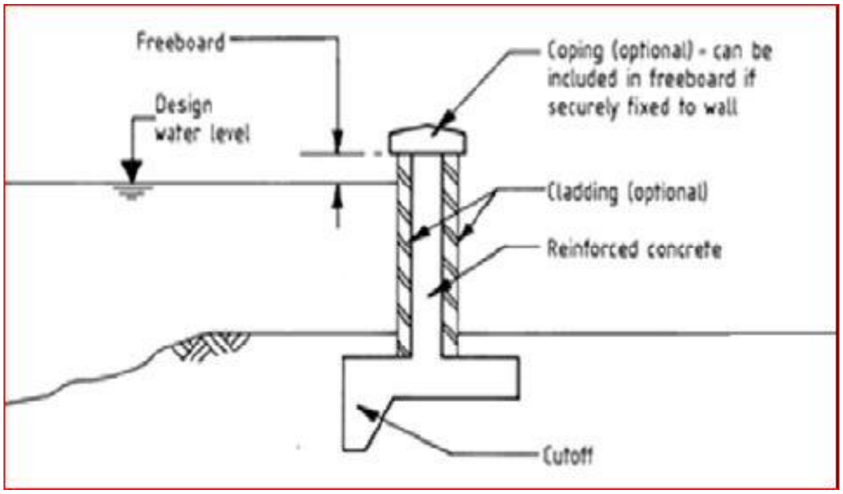


Figure 2.3.2.1 – NACCS Floodwall Cross Section (Concrete T-Wall)

The Tier 1 floodwall exercise was completed for NJBB to determine initial Benefit-Cost Ratio (BCR) results for the Cycle 1 Screening of potential Perimeter Plan alternative locations. The goal was to break down the NJBB economic reaches (35 total) based on their resultant initial with project BCR. "Favorable" (BCR above 2.0), 12 Groups considered "Possible" (BCR between 1.0 and 2.0), and 25 Groups considered "Screened Out" (BCR below 1.0). A further cycle of perimeter plan screening (Cycle 2) was applied to the 12 groups that received a “favorable” status.

For the purposes of NCBB, this initial step was not completed using this methodology because the alternative HVA identification was employed instead, which effectively screened out roughly 70% of the study area. An initial with project condition was not evaluated since the PDT was confident that structural plans formulated within the HVAs would present BCR’s greater than 1.0. Therefore, the PDT moved directly into the selection and design of the wall types for the NCBB study area. The Cycle 2 process for this study was utilized to refine the alignments developed in Cycle 1 to ensure the best possible BCR for each AEP Plan.

Early in the design process the PDT determined that floodwalls would be ideal for the NCBB environment due to the high density of structures along the oceanfront and varied real estate ownership along the waterfront. Floodwalls take up minimal permanent footprint as compared to larger structures such as seawalls and revetments. This would also be consistent with SBMs proposed in the 2018 M&N report and the NJBB study. Oppositely large rock structures like seawalls and revetments would not be ideal for the NCBB perimeter plan due to a various number of reasons. One, the size of the footprint these structures would demand could have high Real Estate impact costs (which are discussed further in Section 7.0 of this appendix) due to high property value in the region. Increased footprint also means an increased amount of permanent disturbance of potentially environmentally sensitive areas. Rock structures are also more likely to be implemented in high wave climate conditions synonymous with the oceanfront as opposed to the fetch limited conditions in the back bays. Lastly, seawall construction tends to be more specialized and tedious which increases construction costs and timeframes. Therefore, this measure was screened out for the purposes of this phase of plan formulation.

After identifying floodwalls as the preferred design feature, the PDT quickly realized that the design crest elevations for NCBB 5% and 1% AEP were similar to that of NJBB. See Section 8.0 of the Appendix B Hydraulics & Hydrology (H&H) for details regarding crest elevation evaluations. Due to the extensive amount of work completed in the screening process of the NJBB designs, the PDT decided to utilize the floodwalls discerned in the NJBB process for the development of the Cycle 1 Perimeter Plan for the NCBB study. The team gave great consideration to the use of the SBMs proposed by M&N in their 2018 report, but the amount of information available to the PDT from the NJBB project was the best choice for the progression of the plan formulation process in accordance with the desired TSP schedule. In addition to floodwalls, NJBB also proposed a levee design for certain areas of the bay front that were directly adjacent to the bay front. This SBM was also included in the NCBB conceptual design to keep consistent with NJBB and offer a design element that could provide potential environmental and aesthetic benefits at select locations. The opportunity to utilize such a structure within the HVAs was minimal, but it was included in the plan formulation.

The back bays shoreline ranges from coastal marshland to emergent beachhead to hard structure armoring (typically bulkhead) in areas of high-density development. Typical flood risk reduction floodwall and levee sections were generated for the Perimeter Plan Screening analysis based on these general conditions assumed along the proposed line of protection. Again, the design height of the risk management (elevation in feet NAVD88) was computed using still water elevation (SWEL) with required freeboard and anticipated relative sea level change (RSLC) to prevent wave overtopping during the design storm event. Crest elevations for floodwalls or earthen levees are similar if the levee includes a rubble slope on the flood side for wave attenuation. The three typical sections used in this analysis were a levee section (Type A), a floodwall section to be constructed in areas from the flood side of the structure (Type B), a floodwall section to be constructed from the protected side (Type C) and a steel king pile and sheet pile combined wall system - king pile/sheet pile combi-wall for short (Type D). Typical Sections of each wall type are shown in Figure 2.3.2.2 through

Figure 2.3.2.4 – Typical Section – Concrete Cantilever Wall on Piles – Type C
.3.2.5.

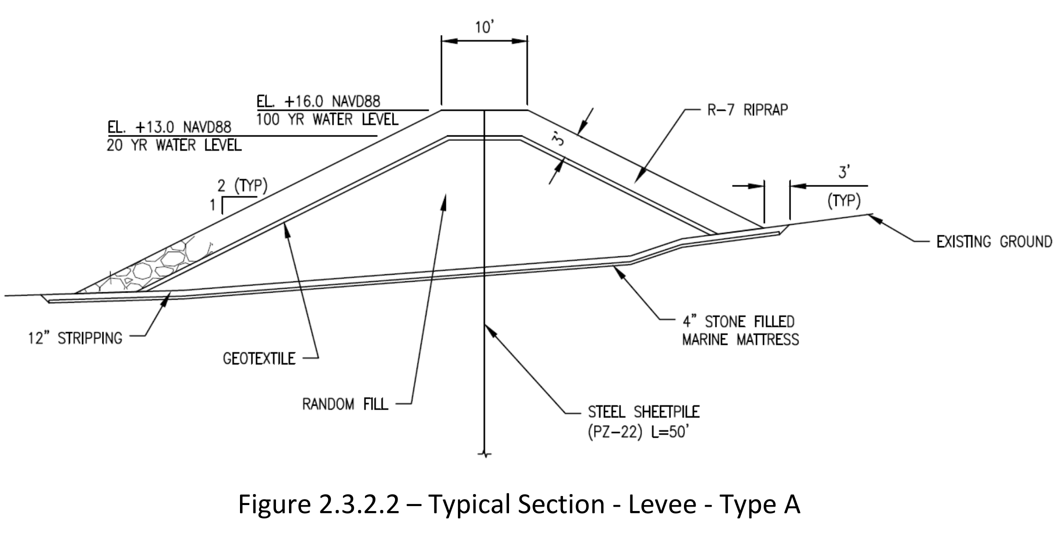


Figure 2.3.2.2 – Typical Section - Levee - Type A

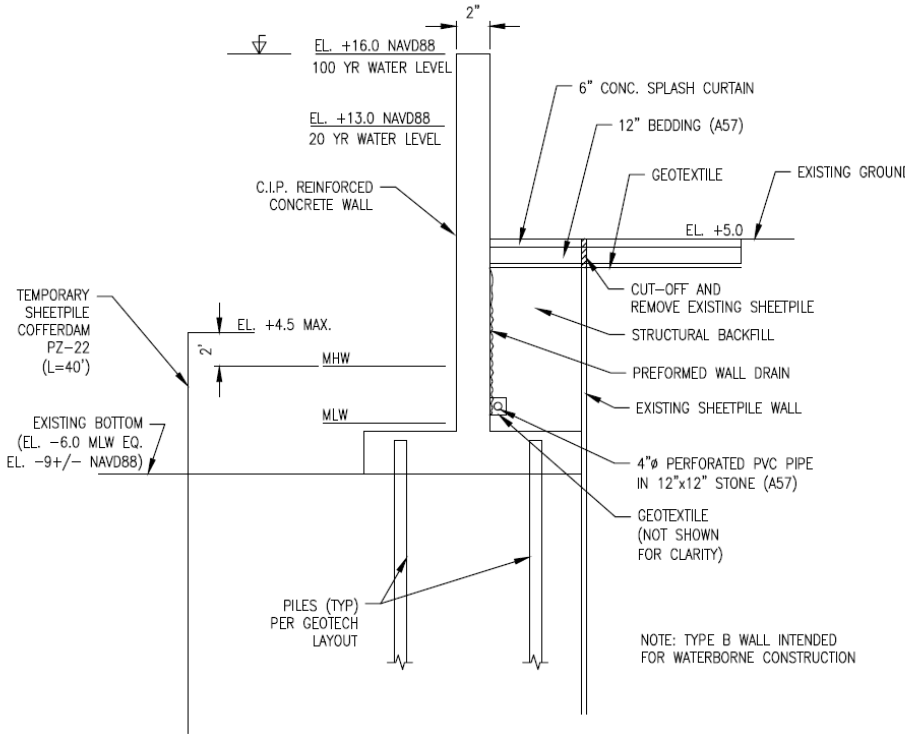


Figure 2.1.2.3 – Typical Section - Concrete Cantilever Wall on Piles - Type B

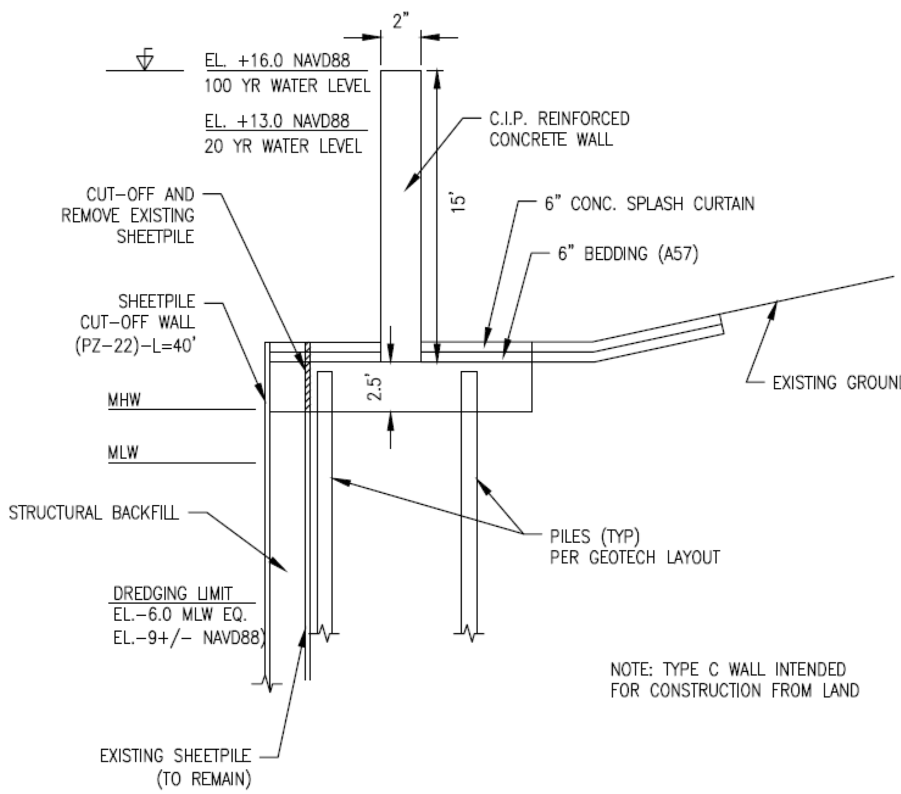


Figure 2.3.2.4 – Typical Section – Concrete Cantilever Wall on Piles – Type C

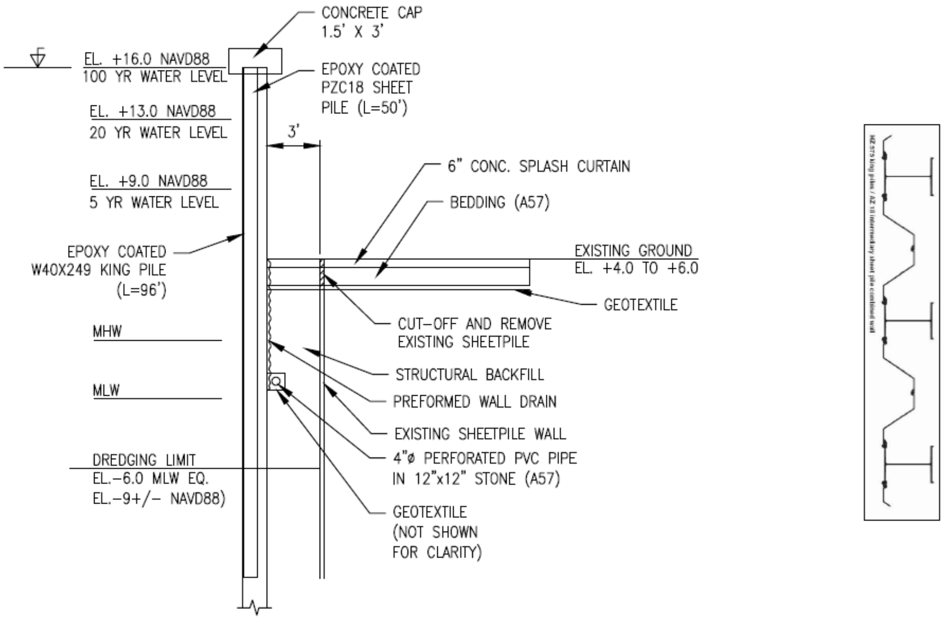


Figure 2.3.2.5 – Typical Section – King Pile/Sheet Pile Combi-Wall - Type D

All Floodwall and Levee Sections were chosen from the NJBB study for use in NCBB since these structures had been designed and vetted through NAP, rather than the PDT redesigning new features. The team would take these structures and adapt them to the NCBB study area. Another major factor in this decision was the lack of geotechnical information available to the team for the Nassau County area. Therefore, the PDT assumed that the geotechnical conditions in NCBB were at minimum similar to NJBB and would assume similar design conditions for the purposes of this stage in the plan formulation. See the Geotechnical Appendix E for commentary on the geotechnical conditions. It is understood that these floodwalls and levees will require optimization post-TSP and the area would need subsurface investigations if structural solutions are to be investigated further. This may result in refinement of the design of the typical sections or creation of subsets of each typical section that are further tailored to intrinsic design needs. A brief technical summary of the design of each wall type is provided herein.

Type A Levee sections were used in open space areas that transitioned from beach to water, or from undeveloped property to marshland, but generally avoided areas of coastal marsh or maritime forest for placement of the full levee section to minimize environmental impacts to these resources. If the alignment for the line of protection could not substantially avoid an environmentally sensitive area one of the floodwall types was utilized since its footprint is much less area than the levee. Very short sections of levee between floodwalls were also avoided for the sake of continuity at the screening level. Layout assumed a landward toe tie-in to existing ground higher than mean high water (MHW), with a sloped bottom extending to the flood side toe at an approximate depth of mean low water (MLW). Type A Levee is utilized in the 5% and 1% AEP alignments; the design crest height is El. +13.0 and El +16.0, respectively. The levee section, 10’ crest width with 2H:1V side slopes, includes a 3-foot-thick layer of riprap placed above a random fill interior. The riprap will protect the structure from, and reduce run-up by, wave action, and protect against erosion during overtopping. At the center of the levee section is a sheet pile wall to provide impermeability of the structure, and for cut-off protection against under seepage. Sections will be constructed on top of 4” thick, stone-filled marine mattresses with geotextile along the base to provide foundation support at the soil interface. Quantities include a 2 foot overbuild for expected settlement of the structure. Figure 2.3.2.6 is an example of a levee structure from the NACCS study. This structure would be similar to the Type A feature.

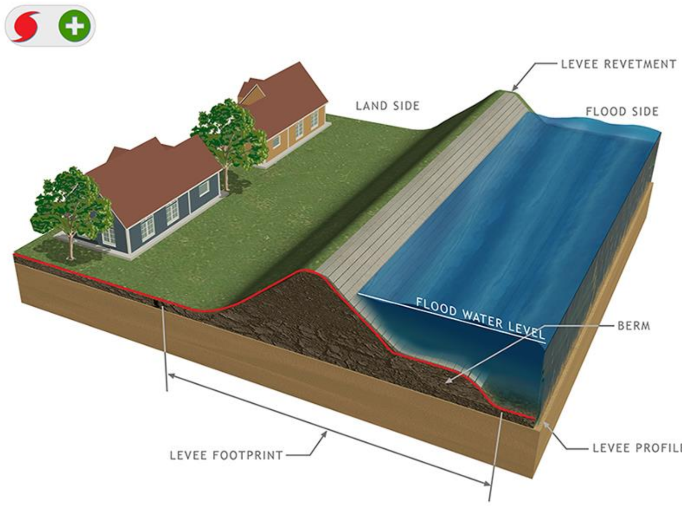


Figure 2.3.2.6 – Example of a Type A Levee (NACCS)

Both floodwalls Type B and Type C are assumed to be similar in composition but different in size, location of placement, and means and methods needed for construction. Both floodwalls are reinforced concrete T-Walls, with a stem thickness of 2 feet, base thickness of 2.5 feet, supported by (2) 50-foot-long HP14x73 piles spaced at 10 feet longitudinally. The crest elevations for both floodwall structures are El. +13.0 and El. +16.0 for the 5% and 1% AEP alignments, respectively. Construction of the Type B wall assumes placement just bayward of an existing bulkhead structure that will remain in place and provide support of excavation. The base of the Type B wall extends to a depth of approximately -9.0 feet NAVD88. This assumption is borrowed from the NJBB study which is the expected maximum dredging depth for the New Jersey Intracoastal Waterways. The expected depth is similar to average depths observed in NCBB waterways. For the purposes of this phase in the plan formulation, the assumed dredging depth was kept similar to NJBB to keep all quantity calculations the same. The actual dredging depths can be optimized post-TSP selection. Lastly, a temporary cofferdam is required for construction of the wall which will be completed using water-based methods. An example of a concrete floodwall from the NACCS is shown below in Figure 2.3.2.7 below. This structure would be similar to a Type B or C floodwall.

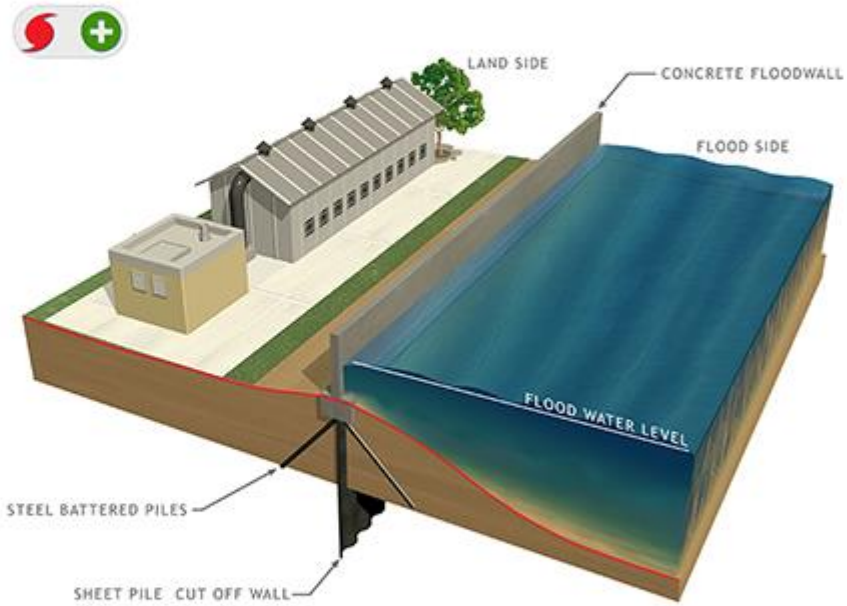


Figure 2.3.2.7 – Example of a Type B/C Floodwall (NACCS)

The Type C wall will be constructed from land at a base depth above or close to the tidal zone. The wall dimensions are based upon constructing the concrete base above the lowest MHW level in the bay (0 feet +/- NAVD88) which results in a stem height of 10.5 feet. The unsupported stem height is estimated to be as high as 9.5 feet. The Type C wall assumes construction behind an existing bulkhead (condition unknown) or at the land edge. In either case, the installation of a sheet pile cut-off wall in front of the structure is assumed to be required for protection of soil below and beyond the base from scour. The depth, number, or size and spacing of piles for either of the floodwalls was not analyzed at this screening level, however, selection of these elements and their parameters was based upon other walls of similar type proposed in other studies. Figure 2.3.2.8 is a rendering of a section of the 1% AEP alignment in the City of Long Beach that contains both Type B (Blue) and Type C (Magenta) walls from Google Earth in the study area.

Figure 2.3.2.8 – Floodwall Alignment Rendering (Courtesy of Google Earth)

The PDT also used the Type C wall in areas upland of the waterfront where no in water work would be required. The team decided not to utilize a different wall type as the changes that would be made at this level of design would be very minimal due to the number of unknowns with existing utilities, geotechnical conditions, excavation requirements, and so forth. Therefore, the Type C wall would be shown in areas upland of the existing waterfront alignment. The Type B floodwall however would strictly be shown along the waterfront as it is intended for waterborne construction.

Wall Type D is a steel king pile and sheet pile combined wall system - king pile/sheet pile floodwall for short. Type D walls have design crest elevations of +9.0, +13.0 and +16.0 for the 20% AEP, 5% AEP and 1% AEP, respectively The wall is comprised of W40X249 steel king piles at a length of 96 feet, interspaced by PZC18 sheet piling at a length of 50 feet for the 1% AEP. A 33X118 steel king pile at a length of 96 feet is assumed for the 5% AEP. A 33X118 steel king pile would also be used for the 20% AEP but the steel king pile length would be reduced due to the 4 foot reduction in the crest height from the 5% AEP. A rule of thumb of 2:1 ratio for embedded vs exposed cantilevered wall was then applied to the length, resulting in a total of a 12 foot reduction to a total length of 84 feet. The PZC18 sheet piling was also reduced 4 feet due to the reduction of the crest height. The wall will be capped with concrete and have a 20 foot wide by 6 inch thick splash curtain on the landward side for protection against over wash. The Type D wall has the narrowest construction footprint of all the types proposed. It will be utilized in areas where there are expected horizontal constraints, or in areas where permanent project footprint is a real estate concern. These locations are in narrow finger canals or adjacent to back bay channels that are close to the existing bulkhead line, respectively. See Figure 2.3.2.9 for an example of a sheet pile flood risk management structure from the NACCS. Note that the structure shown in the image has a helical pile anchor support system whereas the Type D walls proposed in this study are cantilevered structures.

Calculations and assumptions for each wall type at each designated crest elevation can be found in Exhibit B “NCBB Structural Plan Quantities Summary” attached to this appendix. Table 2.3.2.1 summarizes the four (4) selected wall types and their respective design crest elevations. Notes on design assumptions and calculated quantities are included within the document. The quantities developed in this spreadsheet were utilized by the Cost Engineer of the PDT to develop costs associated with the structural alignments. Each wall type includes drainage gates/outlet structures every 400 feet along the length of the floodwall, as this was a similar assumption in the Norfolk CSRM study.

Figure 2.3.2.9 – Example of a Type D Floodwall (NACCS)

Table 2.3.2.1 – Summary of Selected Floodwall & Levee Design Crest Elevations

AEP (%)	Type A Levee	Type B Concrete Floodwall	Type C Concrete Floodwall	Type D King Pile/Sheet Pile Combination Wall
20	N/A	N/A	N/A	9.0
5	13.0	13.0	13.0	13.0
1	16.0	16.0	16.0	16.0

Type D walls are exclusively used in the 20% AEP alignments due to the narrow construction footprint and permanent structure footprint for real estate acquisition purposes. The 20% AEP Plans are meant to provide the locals with risk management from high frequency events, so while the costs incurred by the municipalities may be significant, the AANB may not be as high as the 5% and 1% AEP Plans. Therefore, the team wanted to use a wall type with the least real estate impact, to keep the costs of the plans lower. Also, the team noted that most of the locations in need of risk management for the 20% AEP event would be finger canals where vinyl, timber or steel bulkheads already exist, so the Type D wall would be a similar feature aesthetically. An example of a 20% AEP Plan is shown below in Figure 2.3.2.10 from East Rockaway to Oceanside. The example included below is the Cycle 1 version of the 20% AEP Plan.

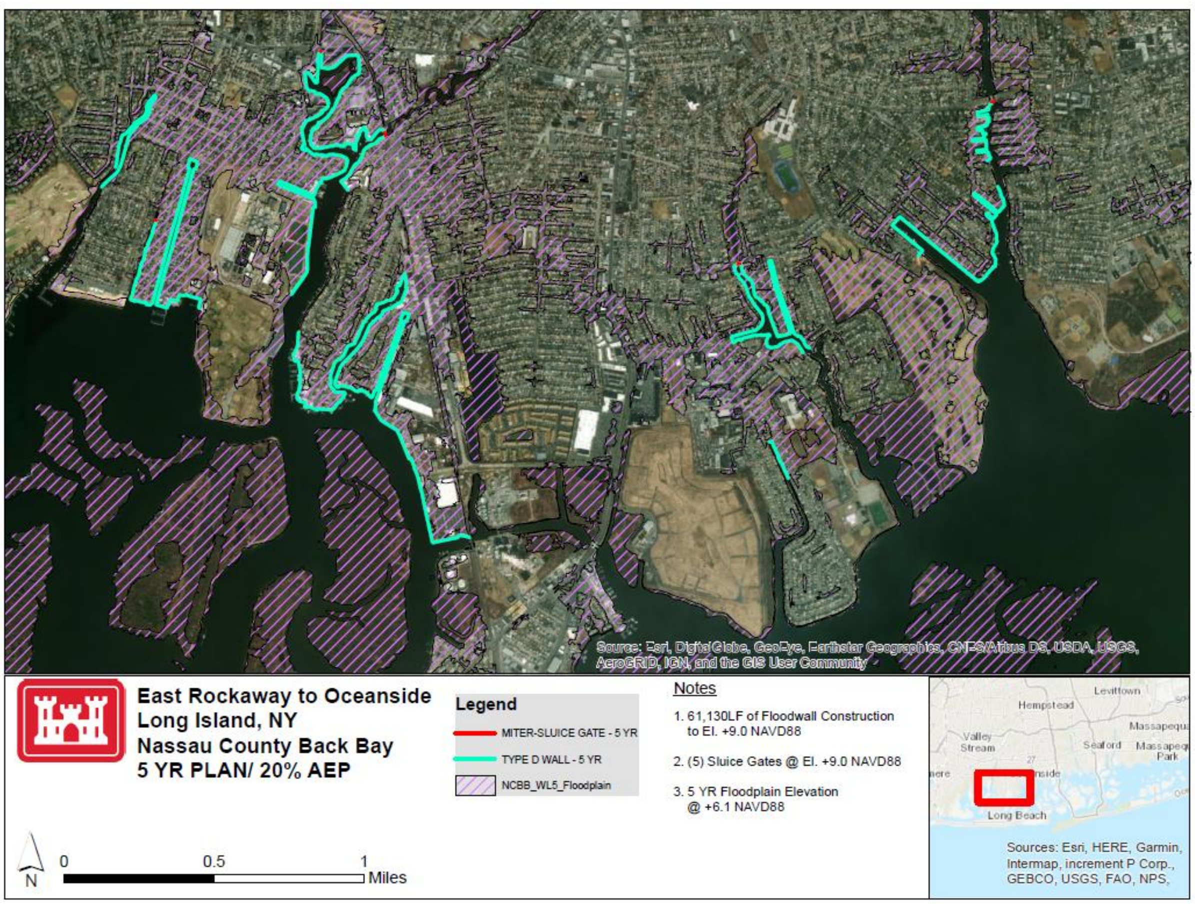


Figure 2.3.2.10: East Rockaway to Oceanside 20% AEP Plan – C1 (Courtesy of USACE-NAP)

As exhibited by the map, the 20% AEP mostly perpetrates the communities through finger canals and upper creek reaches. The use of Type D wall for these types of constricted space locations was necessary. Therefore, due to the high volume of use for the purposes of the 20% AEP risk management it was determined to keep the risk management contiguous and utilize the Type D throughout. Further discussion regarding the selected structural plan alignments can be found in Section 2.3.5 Perimeter Plan Alignments.

For Cycle 1, the alignment formulation strategy for the 20% AEP Plans included little to no closure structures utilized in the layout. Closure structures refer to Road or Rail closures in the upland area and Miter or Sluice Gate structures for in-water closures. The logic behind minimizing the use of gate structures in the 20% AEP Plan was to reduce the operation and maintenance needed to be performed by the municipalities. The 20% AEP Plan is meant to address high frequency events, which means gates would need to be opened and closed too frequently for the proper performance of the design. This puts an unecessary strain on local community resources. However, after much discussion with the team it was determined that this approach would not be effective as hoped as the long linear footages required to protect the finger canals and lagoons increased plan costs significantly.

It was also noted that gates at less frequent AEP events would still need to be closed at lower frequency events to ensure that the protection remain contiguous to maintain effectiveness against events lesser than the design storm. For example, if a system is desgned for a 20% AEP event and a predicted event is projected at a 50% AEP level, the municipality would still need to close the gate to ensure that the area directly behind the gate is not subject to flooding. A 50% AEP event is a very frequent event that can occur often, in some areas much more than the 2-year period associated with it. This means that operation and maitenance costs for gates will be similar at all plan iteration levels. Therefore, no matter the system design event, gates will need to be closed often to protect the community against high frequency events. Even with projected significant operation and maintenance costs associated with gate closures, the reduction of Type D wall to include gates was the best path forward identified by the PDT for the 20% AEP Plans. The 20% AEP Plans were revised in the Cycle 2 screening process to include Miter Gates and reduce the linear footage of Type D wall. Figure 2.3.2.11 shows the refined East Rockaway to Oceanside Plan developed in Cycle 2.

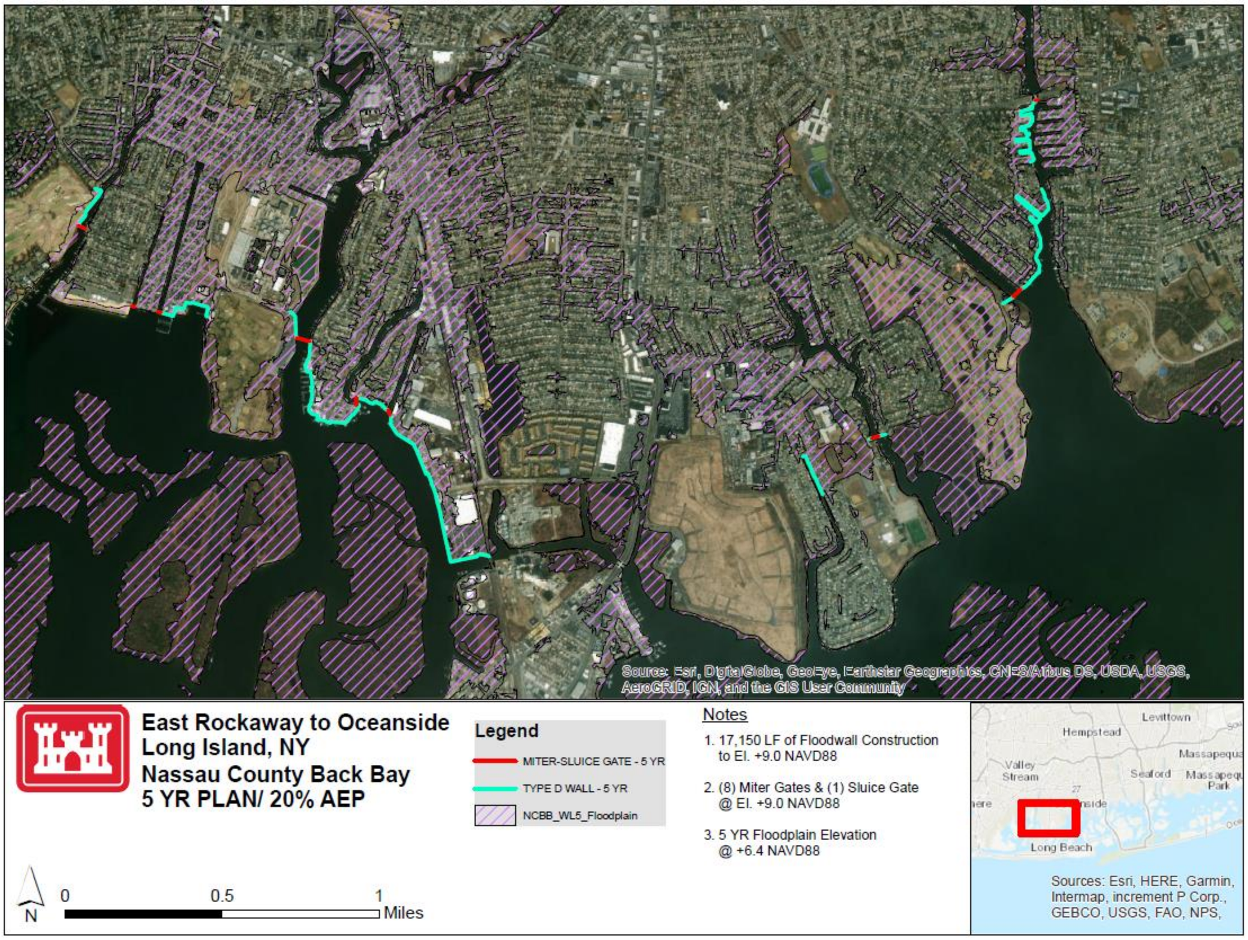


Figure 2.3.2.10: East Rockaway to Oceanside 20% AEP Plan – C2 (Courtesy of USACE-NAP)

For the 20%, 5% and 1% AEP Plans, if the perimeter floodwall placement would need to follow the existing bulkhead alignment, it would result in long linear foot lengths of structure and, thus, substantial with-project costs for those plans. A miter gate, therefore, was used to close off navigable canals or channels if it would eliminate a significant portion of floodwall (~3000 feet minimum per the NJBB study). This limit was determined by dividing the cost of a typical miter gate developed in the Norfolk CSRM study by the linear foot cost of floodwall. Sluice gates were used to maintain flow in areas where the floodwall will cut off flow to a small stream, creek, tidal wetland or marsh, and where navigation is not required. The PDT tried to use closure gate features as minimally as possible due to the increased costs from operations and maintenance associated with these types of structures. More information regarding the closure gates chosen for this phase of the study can be found in the next section.

2.3.3 Closure Gates

In addition to floodwalls and levees, additional structures such as miter gates, sluice gates and road (or rail) closure gates would be necessary to complete the continuous line of protection in the various perimeter plan alignments. Road closure structures (roller gate or swing gate) could be used to close the line of protection during flooding events while allowing use of the roadway during non-flood conditions. For the NCBB study a roller gate structure was assumed at each gate closure location for the purposes of the initial plan evaluation. One road closure will accommodate two lanes of standard traffic. The USACE

Norfolk District (NAO) CSRM project provided parametric costs for road closure gates that were utilized for perimeter plan cost estimating. The team also referenced the road closure gate design from the USACE NAN Port Monmouth Flood Protection Project in Port Monmouth, New Jersey for the design of these structures to develop potential real estate impacts. An excerpt from the referenced design is the attached "NCBB CSRM Structural Plan" drawing set in Exhibit C of this appendix. An example of a roller gate road closure feature is shown in Figure 2.3.3.1 below and a typical design detail in Figure 2.3.3.2.

Figure 2.3.3.1 – Roller Gate Road Closure (Moffat & Nichol, 2018)

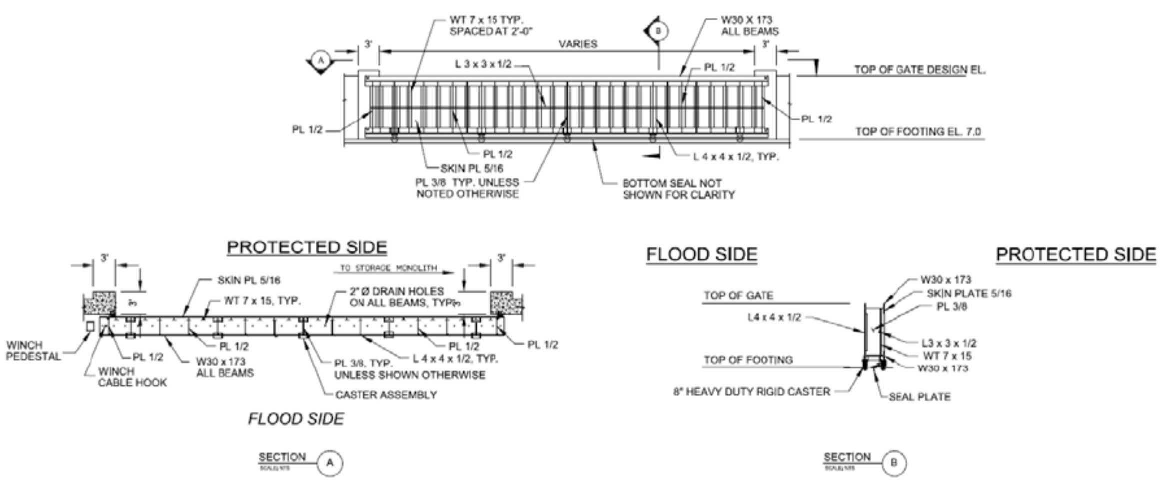


Figure 2.3.3.2 – Roller Gate Typical Section (Moffat & Nichol, 2018)

Road closure gates are used in several locations in the perimeter plan alignments where road or rail closure is necessary. These features could be used in even more locations after a post-TSP optimization process. Additional locations would be created not only based off design refinement, but also during coordination efforts with the municipalities since road closure gates can greatly affect traffic patterns and ingress/egress to private property. Further study of existing traffic patterns and impacts of road closure gates on those patterns would be required if significant closures are selected as a part of the TSP. In terms of positioning of the road closure gates, the PDT also considers accessibility for operation and maintenance, maximum width and potential alternative closure options. For example, a boat ramp or a small local road could have a roller gate road closure type feature but could also employ a stop-log or deployable flood barrier feature instead. Examples of simpler access options can be found in Section 2.3.4 where the need for pedestrian access measures precipitated by the floodwalls are discussed. Figure 2.3.3.3 is an excerpt from the perimeter plan alignment in Oceanside, NY in Google Earth. The approximated road closure gate location is denoted in black.

Figure 2.3.3.3: Road Closure Gate Alignment Example (Courtesy of Google Earth)

In some situation roller gate closures will be need for rail lines as well. While the design would be slightly different that a roadway roller gate, for this phase in the study the Norfolk CSRM design was also assumed for rail closures. Parametric costs from the Norfolk CSRM study were applied to the road/rail closure gates as well as the miter and sluice gates.

Miter and Sluice Gates are utilized in the study area to complete the continuous line of protection in the various perimeter plan alignments. Miter Gates would be used in navigable waterways while Sluice Gates would be used to prevent flooding in upper creek reaches where navigation is not a concern or in areas where the floodwall will cut off flow to a small stream, creek, tidal wetland or marsh. For the purposes of this phase of the study, the PDT chose to utilize the design and parametric costs for Miter and Sluice Gates presented in the Norfolk CSRM study.

The PDT decided to employ Miter Gates at the 20%, 5% and 1% AEP (see Section 2.3.5 for more information on design assumptions), as longer lengths of high floodwalls would be required to achieve risk reduction from the respective design events. Utilizing the rule of thumb developed in the NJBB study (1 Miter Gate = 3,000 LF of floodwall) the team carefully selected Miter Gate locations in the study area. Selection of the locations where a Miter Gate would be required was simple in many locations due to the use of the NJBB design assumption and referencing suggested Miter Gate placement locations in the M&N report. Figure 2.3.3.4 is an example of a canal span that would be protected by a Miter Gate with Type B floodwalls tying into the structure on either side in the City of Long Beach, NY. The proposed gate location is denoted in red. Gate locations are similar for the 20% and 100% AEP Plans as the floodplain extents are very similar and do not vary greatly. However, the 5% AEP Miter Gate locations differ due to higher variations in the floodplain extents and best engineering judgement.

Figure 2.3.3.4: Miter Gate Alignment Example (Courtesy of Google Earth)

Due to the high density of developed property in the area, Miter Gates would most likely need to conform to the width of the existing canal or channel otherwise a buyout of the adjacent property would be required. Therefore, the PDT needed to complete preliminary analyses to determine if the identified spans within the study area had the ability to accommodate construction of a Miter Gate and still maintain its navigable purposes. To achieve this goal, the team looked to USACE Automatic Identification System Analysis Package (AISAP) to determine average vessel characteristics in the study area, particularly within the back bays adjacent to the HVAs.

AISAP is a real-time shipboard broadcast system sending signals to other ships and shore-based receivers. The system was designed as a collision avoidance system. Broadcasted data includes information such as time stamps, latitude and longitude, vessel ID, vessel type, and vessel dimensions. AIS is mandatory for almost all commercial vessels and is also used by some recreational vessels. The Nationwide Automatic Identification System (NAIS) is run by the U.S. Coast Guard and is a network of land-based receivers and transmitters that listen for AIS broadcasts. NAIS collects and archives AIS signal data. USACE developed AISAP, enabling users to pull data from the NAIS archive into the USACE database. AISAP is a web-based tool for acquiring, analyzing, and visualizing near-real-time and archival data from the U.S. Coast Guard. Users can search for all vessels in an area during a specific time or limit their search to specific vessels during a given time range. Figure 2.3.3.5 is a screenshot of the AISAP program used to collect data for this study.

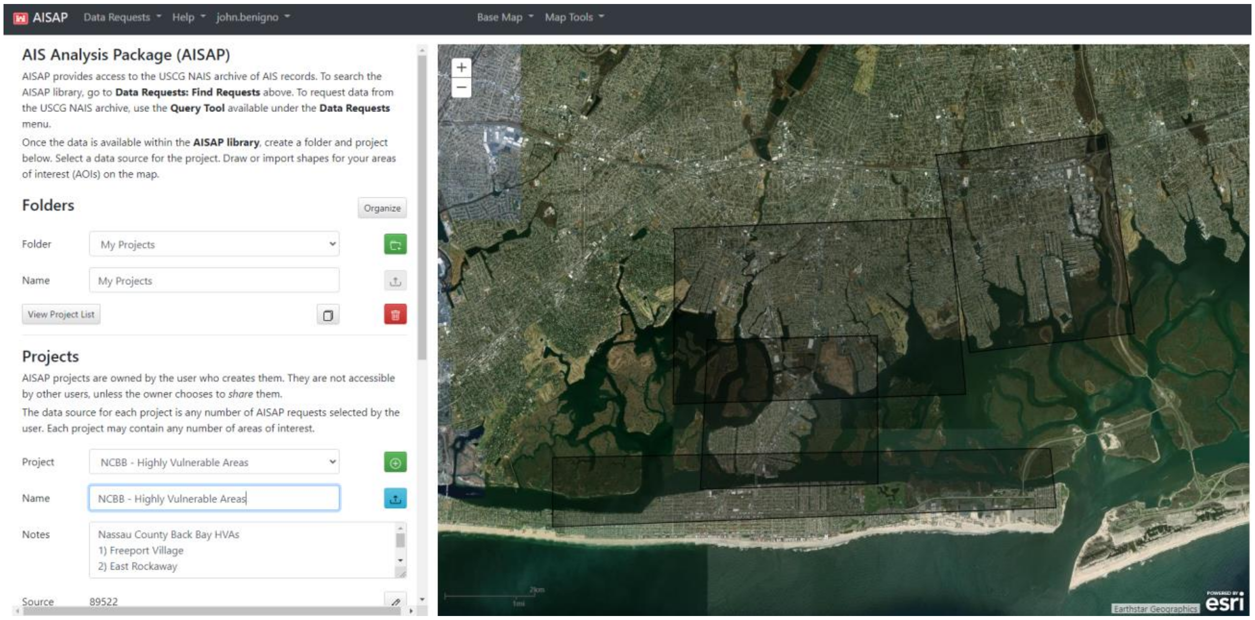


Figure 2.3.3.5 – AISAP Program Interface – NCBB Vessel Analysis Project (USACE 2020)

This maritime vessel analysis provides recommendations for minimum dimensions of navigable Miter gates under the NCBB CSRM Feasibility Study. Recommendations for navigation gate widths are based on vessel traffic data specific to each potential HVA gate location. Based on the available vessel traffic data, a design vessel was selected to recommend a minimum dimension for a miter gate. The purpose of this analysis is only to provide general gate width recommendations. The selected navigation gate dimensions could be larger or smaller depending on existing conditions at each site. Gates may be larger if additional conveyance is needed for environmental or ecological considerations or to maintain access to existing federal navigation channels. Gates may be smaller if navigable widths are already constrained by existing structures such as piers, piles or other existing obstructions. Recommendations for gate widths and locations are preliminary and will be further evaluated in additional phases of the study. The following assumptions were made during this analysis:

- Navigable Miter Gates are located across finger canals and creeks and must be sized to allow property owners ingress/egress of their vessels to their homes or marinas outside of significant storm events. Future channel relocating, widening, or deepening projects were not considered during this analysis but will be evaluated during the next phase of the study if Miter Gates were to be included in the TSP.
- Data is limited by the number of vessels using the Automated Identification System (AIS), the sampling rate used to collect AIS data in a particular area, and the accuracy of the vessel information inputted into the system. The goal of this analysis is not to report every single vessel traversing through an inlet and its exact location, but rather to generate a general representation of vessels.
- This analysis does not factor in the potential growth of the size of the average ship. It is assumed that most of the vessels that will be passing through the Miter Gates will be recreational and will be constrained by their owner's current dock or slip size.
- Most of the vessels reported through the NCBB area were smaller recreational vessels (pleasure crafts).

- It is assumed that these gates will be able to accommodate one lane of traffic, as most of the existing canals and creeks currently have narrow widths to operate within. Future analysis would need to be performed post-TSP to investigate the impacts of restricting channels to one-way traffic.
- Summer month AIS data, May through September, capturing peak recreational use (Memorial Day through Labor Day) was used to select design vessels and track vessel locations. Summer months were assumed to have the most traffic and the most representative design vessel to size navigable Miter Gates.
- This analysis does not focus on other critical design parameters including, but not limited to, environmental, ecological, and cost considerations. Additional parameters will need to be evaluated in more detail as the study continues. Parametric costs from the Norfolk CSRM study were used for the Miter Gates.

AISAP Results

AIS data was collected and evaluated in the Back Bay area adjacent to HVAs from May 21, 2018 to September 9, 2018, representing a total of 111 days. Vessel traffic is assumed to be highest during the summer months and was therefore used to select a representative design vessel. Figure 2.3.3.6 contains the heat map of the vessel transit data captured during this timeframe.

Figure 2.3.3.6 – AISAP Vessel Heat Map – NCBB HVAs (USACE 2020)

Raw results of the vessel data statistics for each HVA have been included in Exhibit D of this Appendix titled “AISAP Vessel Data Results”. A total of 21,000+ vessel reports with 7,000+ transits were recorded during this timeframe for the four HVAs. A summary of the recorded vessel data for each HVA has been included in Table 2.3.3.1 below. The far-right column is the averaged value of each notable vessel characteristic.

Table 2.3.3.1 – AISAP Vessel Traffic Summary Statistics

VESSEL TRAFFIC SAMPLE STATISTICS						
Report Date Range: 5/21/2018 12:05:00 AM to 9/9/2018 12:50:00 PM						
Description	**Unit**	**East Rockaway**	**Long Beach**	**Island Park**	**Freeport**	**Average**
Number of Reports	EA	8328	3884	4423	5009	5411
Number of Unique Vessels	EA	15	29	20	17	20
Number of Transits	EA	3286	1362	877	2139	1916
Vessel Draft (Mean)	FT	1.75	2.06	2	0.39	1.6
Vessel Draft (Std. Deviation)	FT	3.68	4.21	4.27	1.6	3.4
Vessel Length (Mean)	FT	43.74	46.83	47.73	46.32	46.2
Vessel Length (Std. Deviation)	FT	28.35	29.05	27.08	33.47	29.5
Vessel Width (Mean)	FT	13.77	15.16	14.11	13.69	14.2
Vessel Width (Std. Deviation)	FT	8.43	11.59	7.83	7.98	9.0
Vessel Speed (Mean)	KN	0.14	1.84	1.35	0.58	1.0
Vessel Speed(Std. Deviation)	KN	0.04	0.7	0.58	0.2	0.4

Civil design then needed to use this data to determine a minimum design width for the Miter Gate. Table 2.3.3.2 contains the averaged HVA mean value for each notable vessel characteristic and the averaged standard deviation from each vessel characteristic. Miter Gates cannot be designed for the mean vessel, since that would potentially exclude vessels that currently access existing finger canals and creeks. Two potential design values were considered, Averaged Mean + Average Standard Deviation and Highest Averaged Mean + Highest Averaged Standard Deviation. The latter was sure to capture the largest possible vessel that transits the Back Bay area near the HVAs.

Table 2.3.3.2 – Design Vessel Formulation

DETERMINATION OF DESIGN VALUES					
Items	**Unit**	**Averaged Mean**	**Averaged Std. Dev.**	**Avg Mean + Avg Std. Deviation**	**Max Value (Highest Avg Mean + Highest Std. Dev.)**
Vessel Draft	FT	1.6	3.4	5.0	6.3
Vessel Length	FT	46.2	29.5	75.6	81.2
Vessel Width	FT	14.2	9.0	23.1	26.8
Vessel Speed	KN	1.0	0.4	1.4	2.4

With the values in column 3 and 4 being very similar, it was determined that the best course of action was to use the highest design condition as the gate width opening. The team would then compare that value to all the opening widths available at the locations designated for a Miter Gate and determine if the vessel design is a one size fits all for all finger canals and creeks or if vessel design is needed to be refined for each specific gate location. Table 2.3.3.3 contains the selected design values based off the Max Value reported in column 4 of the table above.

Table 2.3.3.3 – Vessel Design Values

VESSEL DESIGN VALUES		
Items	**Unit**	**Value**
Vessel Draft	FT	7.0
Vessel Length	FT	82.0
Vessel Width	FT	27.0
Vessel Speed	KN	3.0

The calculated values were rounded up to the nearest whole number for design purposes. The design values are higher than the calculated value to ensure accommodation. The vessel width is the most important value as it will be the minimum span opening for the Miter Gate. A vessel with these characteristics would be representative of a small yacht like the one observed during an October 2020 site visit by Civil Engineering staff (See Exhibit E for "Civil Site Visit Summary Report"). See Figure 2.3.3.7 below. However, in most scenarios these vessels will not be accessing finger canals and creeks and will be restricted to the waterways the run parallel to the existing waterfront. Therefore, the assumption is that the design values include a high factor of safety since the max vessel will most likely not access the smaller navigable waterways. If Miter Gates were selected as a part of the TSP, further vessel analysis would need to be completed to ground truth these observations such as a vessel traffic count or vessel census in the area.

Figure 2.3.3.7 – NCBB Navigable Gate Design Vessel (Courtesy of USACE Staff)

With the minimum opening width determined of 27 feet. The next step in the design process was to identify the minimum gate width. The Norfolk CSRM study identified a minimum 65 foot wide gate structure. Using the detail for their gate dimensions shown in Figure 2.3.3.8, the 27 foot opening was added to the two 19.0 ft wide concrete buttresses, which creates a minimum gate structure length of 65 feet for the NCBB study. The 65 feet needed for the gate width could then be compared to the potential canal, channel or creek spans that need to be crossed by a navigable gate structure or a combination of navigable gate structures. For the purposes of this phase in the study, only one Miter Gate was assumed at each opening location. The number of gates at each span could be refined post-TSP if Miter Gates were selected as part of the TSP.

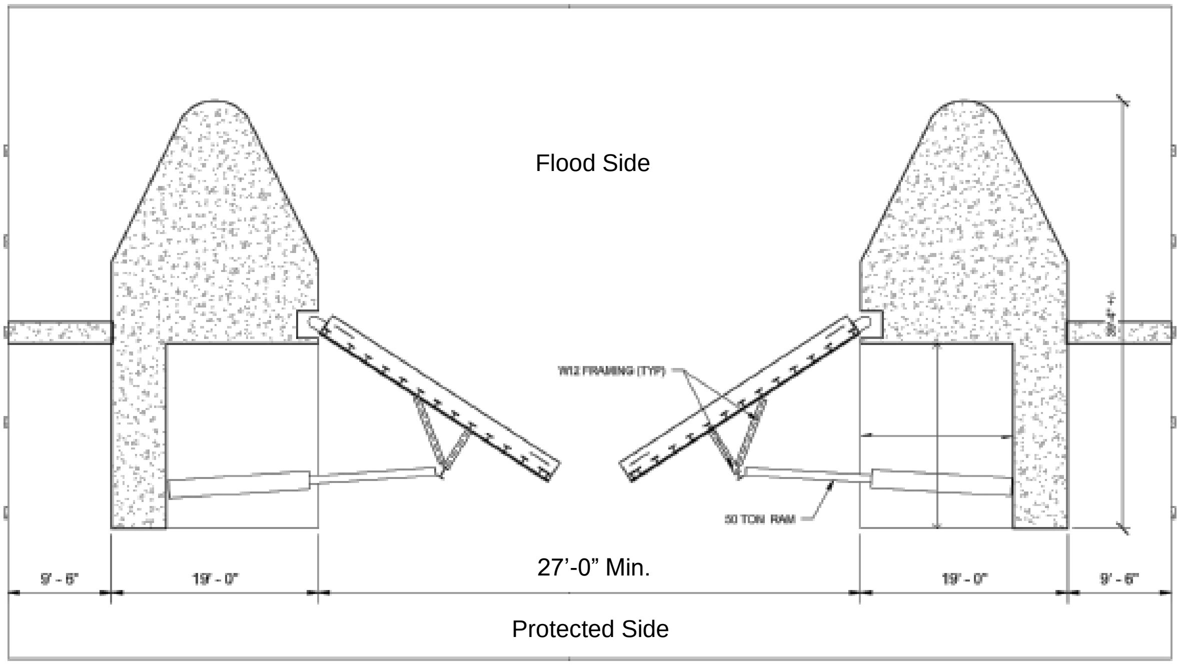


Figure 2.3.3.8 NCBB Miter Gate Design (Courtesy of USACE-NAP)

The next step in the design phase was to compare the minimum 65' span required for each potential opening to the actual existing opening size to see if this proposed gate design would be able to be accommodated. Table 2.3.3.4 contains a summary of the Miter Gate locations and the opening width. Of the 24 Miter Gate locations proposed for the 5% and 1% AEP Plans in the four (4) HVAs, 18 of them passed the minimum width requirement. The locations highlighted in yellow did not meet the requirement. Table 2.3.3.5 contains a summary of the Miter Gate locations and the opening widths for the 20% AEP Plans in the four HVAs. 18 of the 23 proposed gate locations met the minimum requirement for these plans. Note that the 20% AEP gate locations may differ from the 5% and 1% AEP Plans in some locations. Refer to Exhibit A for reference of gate locations.

Table 2.3.3.4 – Navigational Gate Location Opening Widths (5% & 1% AEP)

NAVIGATIONAL GATE LOCATIONS OPENING WIDTHS				
Miter Gate #	**Freeport**	**East Rockaway**	**Island Park**	**Long Beach**
1	60.4	52.2	131.0	89.4
2	47.5	56.7	187.0	83
3	209.6	71.0	151.0	79
4	60.2	230.7	N/A	70.9
5	40.0	80.8	N/A	167.3
6	250.0	106.5	N/A	N/A
7	432.5	170.6	N/A	N/A
8	1035.4	195.5	N/A	N/A
**Gate locations are reported from Left to Right from each HVA Perimeter Plan*				

Table 2.3.3.5 – Navigational Gate Location Opening Widths (20% AEP)

NAVIGATIONAL GATE LOCATIONS OPENING WIDTHS				
Miter Gate #	**Freeport**	**East Rockaway**	**Island Park**	**Long Beach**
1	60.4	139.0	131.0	89.4
2	47.5	56.0	187.0	83.0
3	209.6	69.3	151.0	79.0
4	60.2	248.0	N/A	70.9
5	40.0	112.0	N/A	N/A
6	250.0	104.0	N/A	N/A
7	432.5	147.0	N/A	N/A
8	188.0	205.0	N/A	N/A
**Gate locations are reported from Left to Right from each HVA Perimeter Plan*				

Some locations are short by a very minimal amount, so it is possible the following could be completed post-TSP for the highlighted locations; further vessel analysis to potentially reduce the minimum gate width in these select locations, potentially look at reducing the Gate buttress size, since the gate buttress will have to be further evaluated against local Geotechnical conditions post-TSP. Possibly replace the Miter Gate with a bulkhead or floodwall option if the economics are close enough. Lastly, widening the span of the proposed location through property acquisition and dredging or excavation could be explored, but would likely be a very costly alternative. It is also worth noting that Miter Gate locations 7 & 8 in Freeport shown in Table 2.3.3.4 exceed the 400' max width design criteria used in the NJBB study. Therefore, these locations would need to be evaluated for additional Miter Gates or other structures that permit the flow of water for environmental purposes. For the purposes of this phase study all locations are assumed to accommodate the 65' design so that a parametric cost could be assigned to the various gate locations. Further evaluation and design would need to occur post-TSP to refine the gate cost estimates at each proposed location. Renderings of what the 65' Miter Gate Design in a finger canal opening would look like for the 20%, 5% and 1% AEP Plans are shown in Figures 2.3.3.9 to 2.3.3.11. The sample location is the finger canal that is adjacent to Pine Avenue in Long Beach, NY.

Figure 2.3.3.9 – NCBB Miter Gate – 20% AEP Plan (Courtesy of USACE-NAP)

Figure 2.3.3.10 – NCBB Miter Gate – 5% AEP Plan (Courtesy of USACE-NAP)

Figure 2.3.3.11 – NCBB Miter Gate – 1% AEP Plan (Courtesy of USACE-NAP)

In addition to Miter Gates, Sluice Gates were also utilized to inhibit flow in the upper reaches of some existing creeks and finger canals. The team utilized Sluice Gates mainly in locations where overpasses existed and assumed that the existing structure would be retrofitted with a sluice gate structure to block the 5% and 1% AEP floodplain and in some cases the 20% AEP floodplain. For example, see Figure 2.3.3.11 for an example of a Sluice Gate alignment in Freeport, NY that ties into the bridge connecting Baldwin Harbor and the Village of Freeport. The Sluice Gate alignment is shown in Red. In this plan the gate is required to keep the floodplain from extending into the upper reach. The bridge also provides high ground to terminate the Type D (cyan) wall. The gate would be placed on the south side face of the existing bridge structure rather than at the north side face of the structure as to not impound water from high flow events beneath the bridge deck potentially causing scour to the existing substructure.

Figure 2.3.3.11 – NCBB Sluice Gate Alignment Example (Courtesy of USACE-NAP)

The Sluice Gate that was used for parametric design and costs is the gate that was proposed in the Norfolk CSRM Study. The gate in that study was a 60 foot wide gate. Sluice Gates were selected in opening locations for the 20%, 5% and 1% AEP Plans. The opening widths vary, but for the purposes of this phase of the study, it is assumed that the parametric cost can be applied to all. See Table 2.3.3.6 for summary of the opening widths for the potential Sluice Gate locations and their respective plans.

Table 2.3.3.6 – Sluice Gate Locations Opening Widths

SLUICE GATE LOCATIONS OPENING WIDTHS					
Sluice Gate #	**Freeport 20% AEP**	**Freeport 5% & 1% AEP**	**East Rockaway 20% AEP**	**East Rockaway 5% & 1% AEP**	**Island Park CI 1% AEP**
1	42.2	42.2	50.0	50.0	106.2
2	N/A	N/A	N/A	N/A	177.5
**Gate locations are reported from Left to Right from each HVA Perimeter Plan*					

Per the table, the openings designated for a sluice gate are close or less than the 60 foot wide assumption from the Norfolk Study. The Island Park locations that exceed this width may require multiple sluice gates at a single location. For this phase of the study only one gate was assumed at the span locations and the parametric cost was applied to all plans. Post-TSP optimization would require the design of a gate that would match the desired spans or a scaling of the parametric costs to reflect the decreased or increased size or number of gates needed. An example of a Sluice Gate cross section similar to what would be implemented in this study is shown in Figure 2.3.3.12 for reference. Real Estate acquisition limits for both Sluice Gates and Miter Gates can be found in Section 7.0 Real Estate of this appendix.

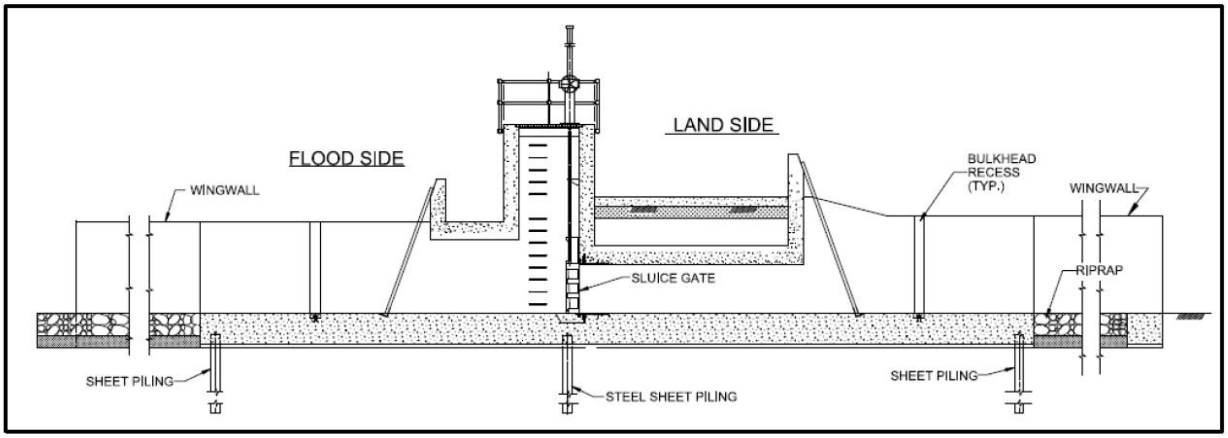


Figure 2.3.3.12 – Sluice Gate Typical Section (Courtesy of USACE-NAO)

Road & Rail Closure Gates and Miter & Sluice Gates round out the SBMs chosen to be identified within the Perimeter Plan Alignments. The summary of the completion of the Perimeter Plan Alignments is given in Section 2.3.5. In addition to the SBMs discussed, there are several features that could potentially be included in the structural plan that are currently captured in the contingency of the cost estimate. Those features are means of access over and through the various floodwalls and levees.

2.3.4 Access Measures

The implementation of SBMs along the perimeter of the HVAs will impact existing crossovers, ramps, walkways and other access features that currently convey pedestrians/patrons to the waterfront. Existing locations where access exists cannot be restricted and the project must maintain access at all private and public locations as a portion of this design. For the purposes of this phase of the study, all access features that would be required have been captured in the contingency of the alignment cost estimates. Dependent upon the selection of the TSP, any structural plans chosen to be incorporated in the Feasibility level of design will include accommodations for pedestrian and in some cases handicap (ADA) access.

The team began to look at potential access options over or through the various types of flood risk management features on a conceptual level, to develop an idea of what type of access features could be designed during the Feasibility Phase. Several conceptual renderings were developed to illustrate what access features may look like in relation to the proposed floodwalls. Access features include steps, ramps, swing gates, roller gates and stop logs. The Freeport Waterfront Park in the Village of Freeport was chosen as the sample location to illustrate potential access measures through/over the various floodwall types, since the protection alignments for the 20%, 5% and 1% AEP Plans have been developed for this area. Figure 2.3.4.1 contains an aerial view of the location of the sample renderings with the proposed floodwall alignment for the 5% and 1% AEP Plans. Figure 2.3.4.2 shows the existing conditions of the waterfront park from the view of the street end. A scaled figure at approximately six (6) feet tall has been included in the image for visual reference.

Figure 2.3.4.1 Freeport Waterfront Park – Aerial View (Courtesy of USACE-NAP)

Figure 2.3.4.2 Freeport Waterfront Park – Street View (Courtesy of USACE-NAP)

The floodwalls shown in Figure 2.3.4.1 are located landward of the existing waterfront bulkhead for the purposes of maintaining the open space in its current condition. As can be seen in the aerial view, all equipment storage structures, and playground equipment are protected by the floodwall. A part of the design intention for alignments near open space are to ensure that view sheds of open public space are not encumbered by floodwalls if risk management is not necessary. This topic is to be further discussed in Section 2.3.5. With that intention in mind, the 20% AEP in this location would follow the existing waterfront bulkhead alignment shown. However, for the purposes of the rendering all three plan elevation heights were shown in the same location to compare the impact of the wall heights on pedestrian access and view shed. Figures 2.3.4.3 thru 2.3.4.5 contain the renderings for the 20%, 5% and 1% AEP plans respectively.

Figure 2.3.4.3 – El. +9.0 NAVD88 Floodwall – 20% AEP Plan (Courtesy of USACE-NAP)

Figure 2.3.4.4 – El. +13.0 NAVD88 Floodwall – 5% AEP Plan (Courtesy of USACE-NAP)

Figure 2.3.4.5 – El. +16.0 NAVD88 Floodwall – 1% AEP Plan (Courtesy of USACE-NAP)

A concrete access ramp is shown in each figure as the standard means for providing conveyance over the floodwall, these ramps would be ADA compliant. However other access measures would also be explored in the Feasibility Design phase as previously discussed. Figure 2.3.4.6 shows examples of various additional access measures that could be utilized.

Figure 2.3.4.6 – Additional Access Measures (Courtesy of USACE-NAP)

The image in the top left of the figure would be a simple staircase that could be used to traverse the lower walls in locations were ADA access is not required. These could be favorable options at locations where floodwall would be placed on private property. Figure 2.3.4.7 shows an example of a simple timber staircase and ramp structure.

Figure 2.3.4.7 – Simple Staircase & Ramp Example (Courtesy of USACE-NAP)

The image in the top right of Figure 2.3.4.6 is a rapidly deployable stop log structure. This structure would provide standard access to pedestrians, ADA access and access for maintenance equipment. Steel, Aluminum or Timber stop logs would be deployed in the event of a storm to create a uniform level of risk management. An example of a stop log structure is shown in Figure 2.3.4.8. A stop log structure can also be used in Non-Structural solution applications which is further discussed in Section 5.0 Non-Structural Solutions of this appendix.

Figure 2.3.4.8 – Stop Log Example (Courtesy of Google Images)

The two features on the bottom left and right of Figure 2.3.4.6 show swing and roller gate access options. These features may require higher design and construction costs but provide the community with a product that is highly desirable in terms of operation and maintenance. Deployable features like stop logs require storage of the material in a nearby facility. Depending on how many stop log systems are constructed, the logistics of storing, handling and deploying could prove cumbersome. This is an issue when considering that the stop logs must be put in place prior to a storm event, which would increase the required response time needed. A swing gate or roller gate is permanently affixed to the floodwall and can easily be closed prior to a storm event. These types of access features would be ideal in the most vulnerable locations (i.e. access locations along the waterfront) where closures can be achieved quickly. Figures 2.3.4.9 and 2.3.4.10 show examples of roller gates and swing gates that have been designed by USACE for other flood control projects.

Figure 2.3.4.9 – Roller Gate Example – Port Monmouth, NJ (Courtesy of USACE-NAN)

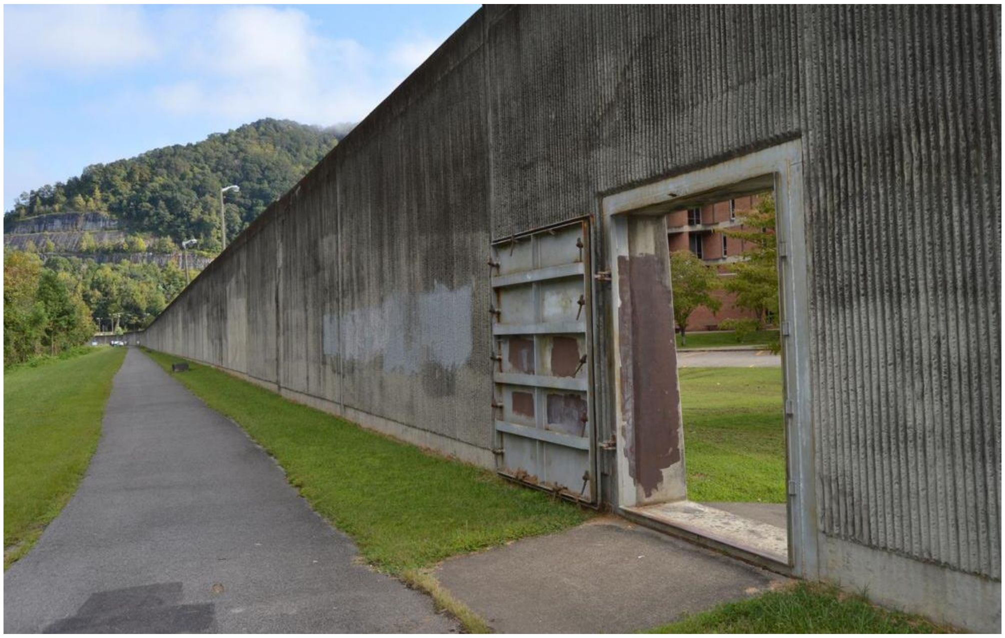


Figure 2.3.4.10 Swing Gate Example – Matewan, WV (Courtesy of USACE-LRH)

One wall type that was not shown in the renderings is the Type A Levee. This is since the levee is only used in open space areas that access to the waterfront is not available or feasible. Levees are not utilized within the proposed alignments where they would obstruct existing access on private or public property. See Section 2.3.5 for more information regarding levee alignment considerations. The crest of the levee could be used for a scenic walking path or possibly the leeward side as an area for leisure activities. Figure 2.3.4.11 is a conceptual figure of potential post construction uses of a levee.

Figure 2.3.4.11 – Levee Post-Construction Use Example (Courtesy of Google Images)

2.3.5 Perimeter Plan Alignments

The work completed by the team referenced in the previous sections culminated in the development of the perimeter plan alignments for the HVAs within the Nassau County Study Area. Several design assumptions and concepts, discussed previously, were utilized to develop the various alignments. A summary of these major design assumptions is provided below.

- Determination of the HVAs
  - Narrowed the location to where perimeter plan alignments could be developed.
- Design Storm Events & Elevations
  - Selected the level of recurrence to formulate to and the respective elevations SBMs should be designed to.
- Floodwalls & Levees
  - Determined the types of SBMs that could be implemented in any proposed perimeter plan.
- Gates
  - Preliminary gate design geometry informed decisions on where to locate gates in the perimeter alignment formulation.

- Access
    - The need to maintain ingress and egress to existing properties and the known potential access solutions are influential in locating floodwalls along the waterfront.
- Use of Existing Work
    - Utilized the work completed by M&N in 2018 to help inform where to locate alignments or discern if alignments should be modified to achieve study goal.

These highlighted design assumptions both informed the design team and provided the tools necessary to develop and lay out the final Perimeter Plans for the Highly Vulnerable Areas. A summary of the perimeter plan alignments and their respective quantities can be found in Table 2.3.5.1 below. Included herein are some additional assumptions specific to the layout of the perimeter plan alignments that would be necessary to complete formulation. These design assumptions have been summarized in the topics below.

Table 2.3.5.1 – Perimeter Plan Alignment Summary Table

PERIMETER PLAN			QUANTITY								
HVA	Cycle 2 Polyline Names	AEP %	Floodwall / Levee (ft)	Type A	Type B	Type C	Type D	Miter Gates (ea)	Sluice Gates (ea)	Road Closures (ea)	Rail Closures (ea)
Freeport	FPV5	20	42,264	-	-	-	42,264	8	1	0	0
Freeport	FPV20	5	36,602	5,211	8,254	12,904	10,233	8	1	1	0
Freeport	FPV100	1	36,602	5,211	8,254	12,904	10,233	8	1	1	0
Long Beach	LBC5	20	13,865	-	-	-	13,865	4	0	0	0
Long Beach	LBC20	5	25,629	-	14,953	10,677	-	5	0	4	1
Long Beach	LBC 100	1	46,440	-	14,953	31,488	-	5	0	4	1
Island Park	IPV5	20	30,026	-	-	-	30,026	3	0	0	0
Island Park	IPV20	5	36,317	1,442	10,254	4,719	19,902	3	0	1	2
Island Park	IPV 100	1	36,317	1,442	10,254	4,719	19,902	3	0	1	2
East Rockaway	EROC5	20	17,137	-	-	-	17,137	8	1	0	0
East Rockaway	EROC20	5	51,786	2,655	6,417	25,270	17,444	8	1	3	1
East Rockaway	EROC100	1	51,786	2,655	6,417	25,270	17,444	8	1	3	1

1. Existing SBMs

Existing measures within the HVAs had a major influence in the determination of where a perimeter alignment was placed. For example, if an existing bulkhead structure lines the waterfront of a community, it is most likely that this existing alignment would be matched with the improved risk management proposed by one of the floodwalls included within this phase of the study. This approach was taken in most scenarios, since locating the wall where structure exists maximize the amount of land to be protected and decreases the likelihood of property buyouts. Locating a floodwall landward of existing SBMs can impact existing private property, utilities and roadways while locating a floodwall waterward of the existing SBMs can increase impacts to environmentally sensitive areas or impede navigation in existing waterways. It is important to note that the proposed alignments may follow the alignment of the existing SBMs, but the centerline of the floodwall or levee structures may vary dependent upon where the existing structure would fall in relation to the design assumptions. Figure 2.3.5.1 shows an example of a series of waterfront properties in the Hamlet of Oceanside, NY that each have varying types of bulkhead protections. The perimeter plan development in this area is shown directly below in blue. Per the figure, the alignment of the floodwall follows the alignment of the existing bulkhead protection very closely.

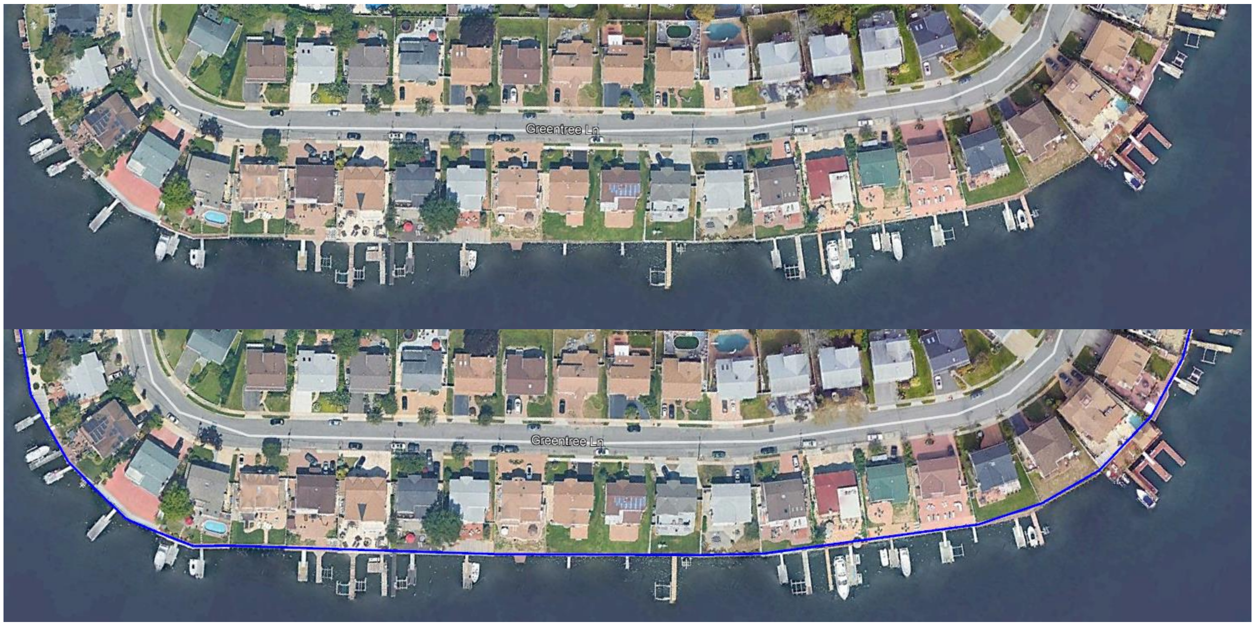


Figure 2.3.5.1 – Perimeter Plan Development Before & After (Courtesy of Google Earth)

2. Floodplain Extents

The elevation of the floodplain at the various AEP levels impact the elevation of the structures required, which has been discussed at length in this appendix. However, the spatial extents of the floodplains have significant bearing on the placement of the perimeter plans as well. In order to achieve the desired level of risk reduction in a community, the floodwall must successfully block the floodplain from extending into the community. Figure 2.3.5.2 is an example of the 5 year or 20% AEP floodplain in the Village of Island Park, NY. In this example structural risk management is implemented where the floodplain extends into the reach beyond the waterfront limits. In some cases, risk reduction is not provided along the waterfront where the floodplain only extends into open space or marsh area. In those locations, risk reduction is not needed since the limits of the floodplain do not pose a threat to any existing property or roadways. This approach to perimeter plan development was applied to the alignment formulation for all AEP events. Due to the low elevations the existing ground of communities within the study area, much of the HVAs require complete perimeter alignments for the 5% and 1% AEP events.

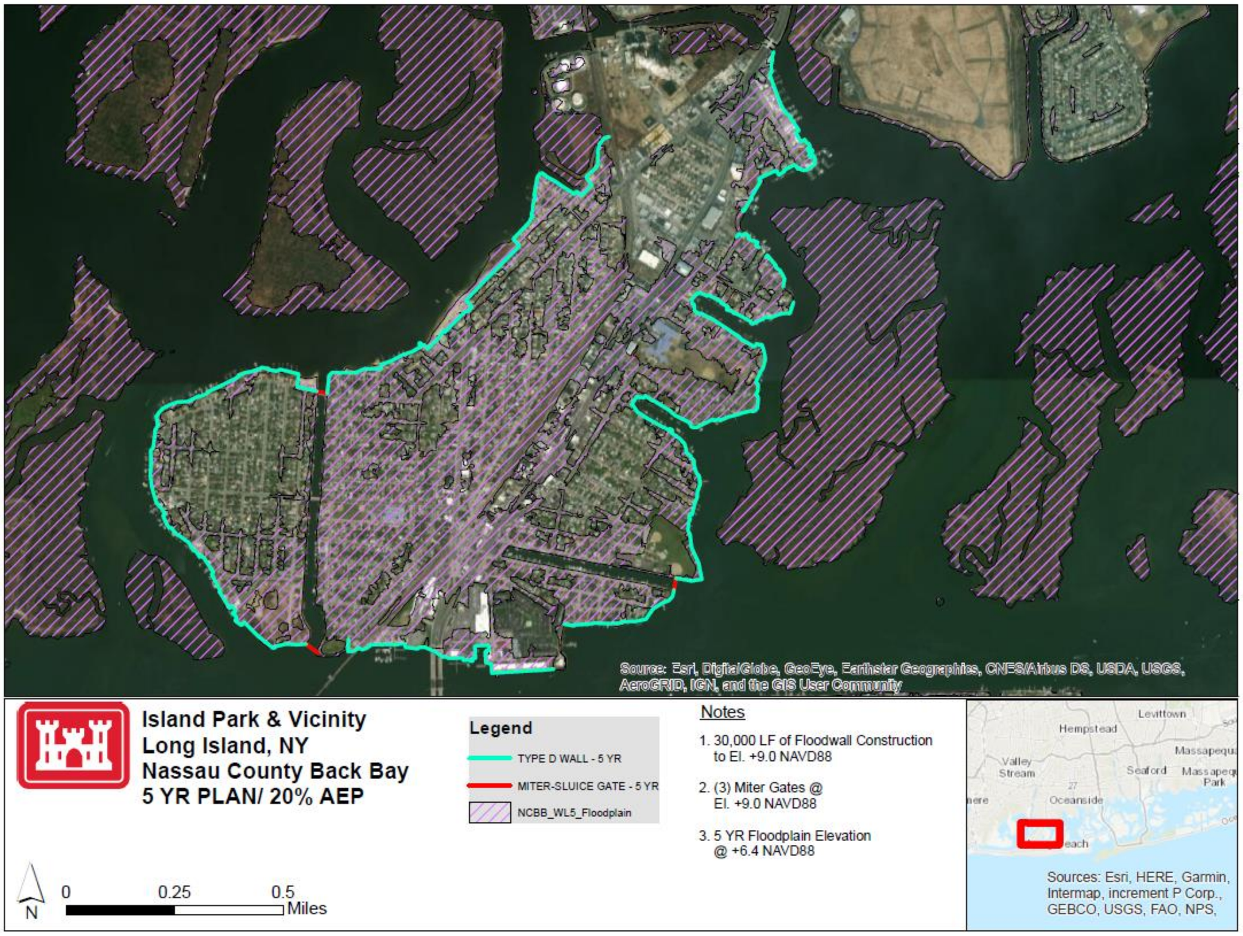


Figure 2.3.5.2 – Floodplain Considerations for Alignment (Courtesy of USACE-NAP)

3. Open Space Considerations

   Several parks, beaches and other open spaces exist along bay frontage in Nassau County. Open spaces are meant for communities to use for leisure activities such as fishing, walking, biking, sports and other activities. Implementation of floodwalls in or around open space can potentially reduce the design intention of the open space and public interest must be treated with consideration when formulating. In many scenarios structural protection was placed landward of the open space unless a building or facility was within the limits. This provides the public with an alignment that secures the community from the floodplain extents while maintaining the integrity of their open space. Namely providing them with an unencumbered viewshed and direct access to the waterfront for leisure activities. An example of this plan formulation technique at Hewlett Point Park in Figure 2.3.5.3. This figure illustrates the implementation of floodwalls upland of the open space for the 5% and 1% AEP Plans.

Figure 2.3.5.3 – Open Space Perimeter Alignment (Courtesy of Google Earth)

4. Tide Gate Placement Considerations

Placement of closure gates within tidally flowed areas (Miter and Sluice Gates) is a difficult formulation decision because of the effects that are associated with placing operational structures in a marine environment. Inhibition of channels, creeks or streams can create significant impacts to water quality, sediment transport and channel dynamics. Tidal gates also create significant operation and maintenance concerns for the local sponsor. Closure of tidal gates will need to be managed by the local communities as well as maintaining functionality of the gates through habitual testing and upkeep. The implementation of tidal gates exposes the local community to increased liability if they fail to properly operate the structures during a design storm event that could cause damages to upland property that otherwise would've been protected. Also, the use of Miter Gates in navigable waterways create operational issues with ingress and egress of local vessel traffic and create the need for the development of a marine traffic notification system for closure alerts. Due to these considerations, the team placed the minimal number of gates as possible in the perimeter plan alignments developed.

The team used the rule of thumb developed in the NJBB study that was previously referenced in Section 2.3.3 that a gate would be placed only if it would mean the reduction of a minimum of 3,000 LF of floodwall. The determination of that linear footage was based on an economic "break-even" point where the cost of the gates would be equal to the cost of the floodwall. See Figure 2.3.5.4 for an example of a location that meets gate placement criteria in the City of Long Beach. The finger canals along Pine Avenue well exceed 3,000 LF and become excellent candidates for Miter Gates due to the potential cost savings.

Figure 2.3.5.4 – Miter Gate Placement Example (Courtesy of Google Earth)

The figure above shows the alignment in the City of Long Beach for the 5% and 1% AEP Plans. In this area the gate locations are also the same for the 20% AEP Plan but the alignment of the floodwall will differ based off the extent of the floodplain for that design event. Miter Gates were not originally formulated in any of the 20% AEP Plans but were added in the Cycle 2 analysis. The 20% AEP Plan is designed to handle a lower level storm event. To implement a Miter Gate at this lower level design interval, it would require gate closures for storm events not only at the 5 year level but at higher frequency events as well to minimize damage. The difficulty with implementing a structure like a Miter Gate for the purposes of low-level events is that the gates would require closure several times a year and identifying high frequency events and their potential severity is much more difficult than identifying events such as the 20 and 100 year events. This could be costly for operations and maintenance and a great hindrance to the residents for ingress and egress of their vessels. High frequency storm events occur during the timeframe of peak vessel use (May to September), and attempting to coordinate frequent closures during peak vessel use could become cumbersome for the sponsor to manage and taxing on the residents to endure. However, after several discussions with the PDT, it was also considered that the gates designed for the 20 and 100 year events would also need to be closed for the lower level events as to not incur damages related to those events. Therefore, high frequency of closure and significant operation and maintenance is applicable to gates in all plans.

Existing flood risk management alignment was followed in the finger canals and lagoons in the 20% AEP Plan where the 3,000 LF replacement criteria was not met. Many local communities are currently constructing risk management measures at elevations like the El. +9.0 NAVD88 elevation recommended for this design event, which the local communities have accepted. For example, at the location in the figure below, the City of Long Beach is currently constructing bulkheads to this study's 20% AEP Plan elevation. Figure 2.3.5.5 shows the construction of the new bulkhead along the finger canal on the left and an image of the new structure compared to the elevation of existing structure on the right.

Figure 2.3.5.5 – Bulkhead Construction in Long Beach, NY (Courtesy of USACE-NAP)

Initially the 20% AEP Plan for the location shown in Figure 2.3.5.5 was to continue the alignment throughout the finger canal and match the construction elevation. However, the approach was changed to place a Miter Gate at the canal opening for the previous reasons stated. Therefore, the 20% AEP Plan provides less risk reduction and a similar level of operation and maintenance to the 5% and 1% AEP Plans. The only major advantage is the potential for a reduction in impact of real estate acquisition costs. Lower elevation floodwall and gates mean less of a viewshed encumbered by the structure. The real estate acquisition costs for the reduced viewshed impacts were not considered in the cost screening of the alignments, however if the 20% AEP Plan was selected for the TSP or even as a Locally Preferred Plan (LPP), that analysis would need to be further vetted by USACE Real Estate. Figure 2.3.5.6 shows street view of the 20%, 5% and 1% AEP Plan floodwall and gate structure at Elevations +9.0, +13.0 and +16.0 NAVD88. Compare these images to the bulkhead constructed at this location shown in the previous figure.

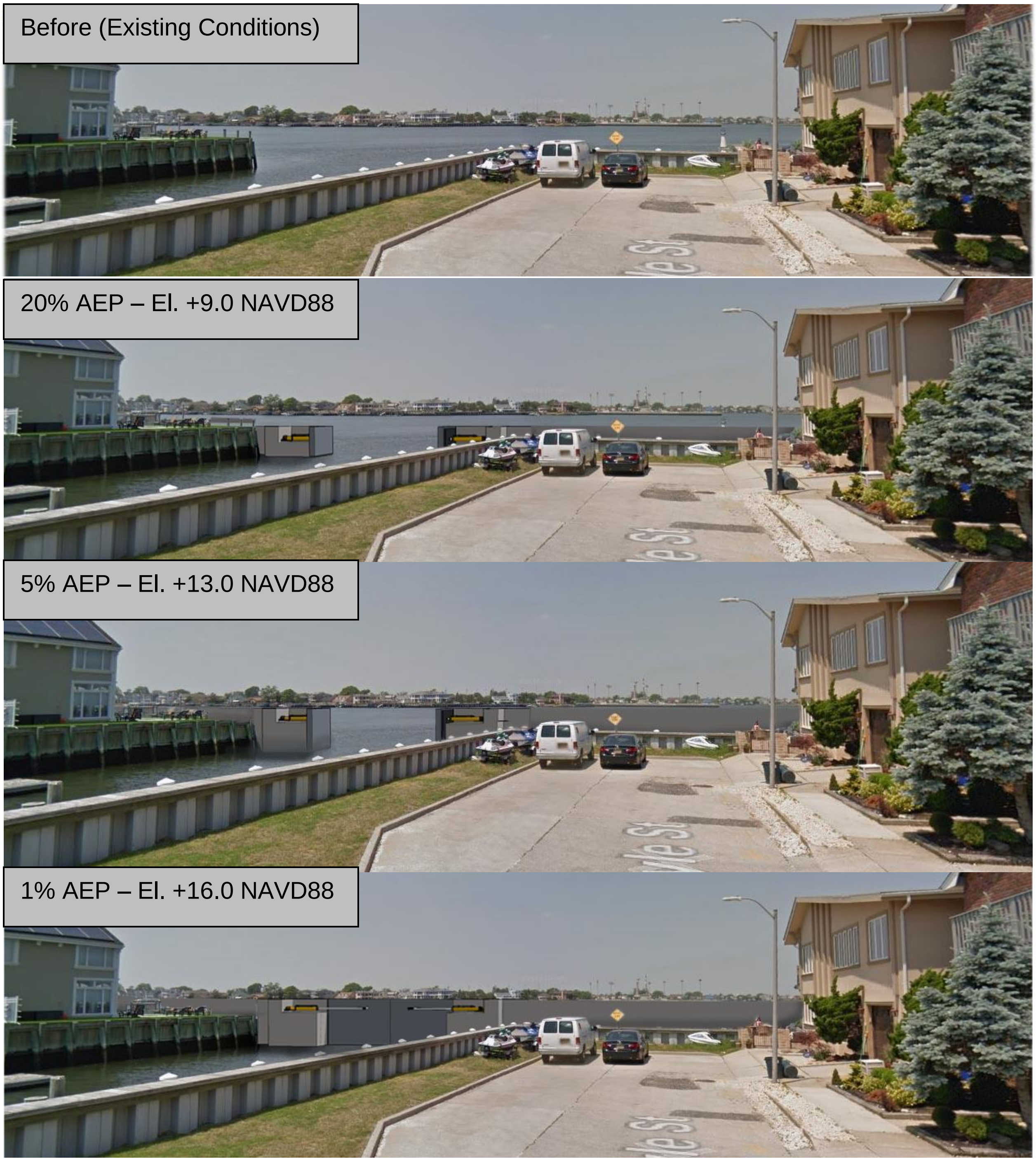


Figure 2.3.5.6 – Street View of Floodwall Alignment (Courtesy of USACE-NAP)

5. Termination Point Assumptions

All perimeter plans formulated within the HVAs generally follow the waterfront of the community. However, since the idea of a comprehensive perimeter plan was screened out during the initial plan formulation phase, each HVA plan is essentially independent of one another. The lack of continuity means that the plans must terminate into an upland location that is at a ground elevation equal to or greater the design intention of the associated plan. This termination must be completed to ensure the designed level of risk management is contiguous throughout the plan alignment. Since risk management was identified for HVAs only in the plan formulation, the termination points for many of the plans would extend inland a significant distance to reach a location that is of an acceptable elevation. To determine the appropriate tie-in location, the team used Digital Elevation Models (DEMs) of each HVA to discern where to terminate the structural alignment (See Exhibit A for DEM mapping). An excerpt of the Village of Freeport 1% AEP Plan is shown in Figure 2.3.5.7 below.

The Freeport plan terminates at a location where the elevation is a minimum of El. +14.0 NAVD88. The Type C wall shown in pink extends along the roadway to the point where the floodplain extents have been blocked and the tie-in elevation has been met. El. +14.0 satisfies the tie-in requirements in Freeport and East Rockaway since the tie-in locations are so far upland. The team did not need to account for design considerations like wave overtopping in these locations since these far inland areas are not opened to direct wave attack. The El. +14.0 tie-in also satisfies the tie-in requirement for the 5% AEP Plan since the structural crest elevation for that plan is El. +13.0. Post-TSP optimization of any selected plans could potentially increase or decrease the terminus elevation through further hydraulic modelling which could shorten or extend the alignment lengths. Note the alignment shown would require access measures to accommodate residents and changes in the traffic pattern to avoid the implementation of road closure gates (i.e. creation of one-way streets). Also, the alignment would require a sluice gate located at the face of the bridge to block tidal flow from flanking the protection termination.

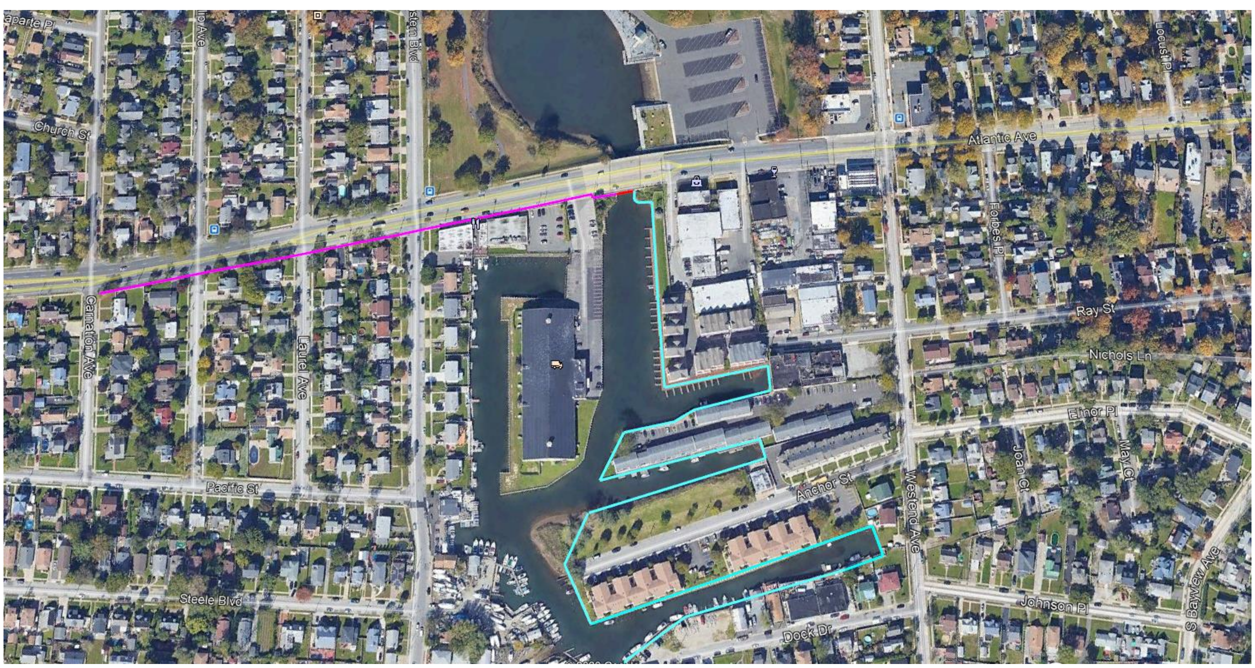


Figure 2.3.5.7 – Upland Tie-In of Freeport Alignment (Courtesy of USACE-NAP)

If the perimeter plan could not be tied into an acceptable elevation, the HVA then needed to be completely encompassed by structural risk management. Complete encircling of a community is worrisome, since it creates higher risk for life safety. An encircled community may use structural risk management as a false sense of security and not upgrade existing infrastructure within the perimeter plan alignment. This potential neglect could lead to catastrophic losses to life and property within the encompassed community. However, for communities closer to the oceanfront such as Island Park and Long Beach this encompassing plan is unavoidable. If a community with an all-encompassing plan is selected for TSP, a life and safety risk analysis would need to be completed during the plan optimization phase. Figure 2.3.5.8 shows the 1% AEP Plan for the City of Long Beach. The 1% AEP Plan completely encompasses the community, with a floodwall shown within the dune alignment of the existing Federal Dune & Beach Project. This floodwall is necessary since the existing Federal Project dune crest elevation is El. +14.0 and is in a location open to direct wave attack. Therefore, the plan must provide adequate risk management in this location to ensure a contiguous line of protection. The 5% AEP Plan here does not require floodwall along the oceanfront since the dune elevation exceeds the El. +13.0 design crest. Further explanation regarding tie-in to the Federal Project can be found in Section 11.0 of the H&H Appendix.

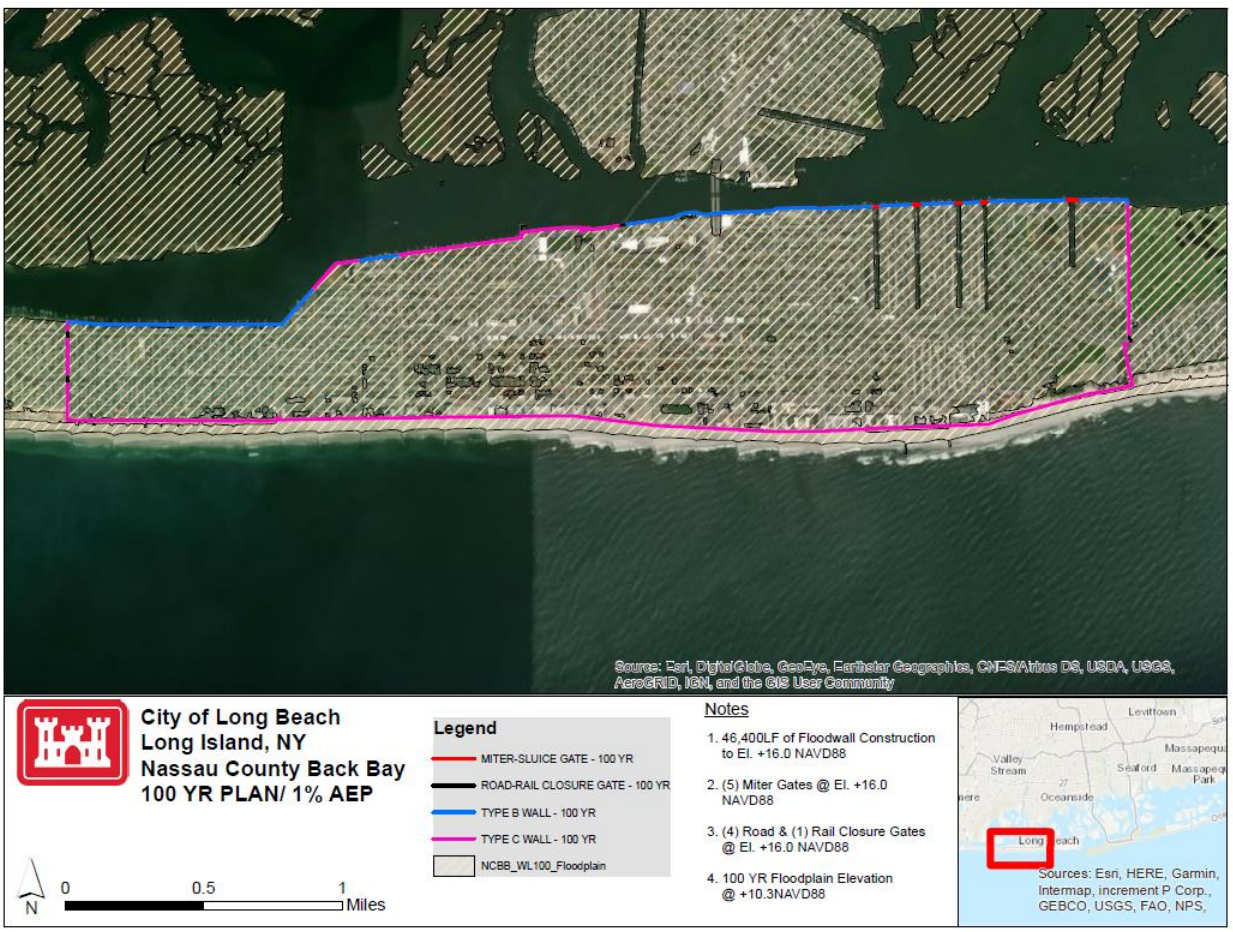


Figure 2.3.5.8 – City of Long Beach 1% AEP Plan (Courtesy of USACE-NAP)

6. Pump Stations

All plans referenced in Table 2.3.5.1 would require the design and installation of a pump station. Implementation of floodwalls in the communities will affect the existing drainage conditions. The floodwalls will be effective in keeping flooding out but will also keep stormwater from intense rain events in the community's streets. For this level of design only a cost for the potential pump stations was applied to the individual perimeter plans based off parametric values developed by the USACE-NAP H&H design team. See Appendix B, Section 10.0 for more information regarding assumptions for Pump Station design assumptions and considerations incorporated into this plan. If a structural alignment were chosen as part of the TSP, the Pump Station design would be further evaluated with extensive stormwater modelling and hydraulic design. See Figure 2.3.5.9 for an example of a typical Pump Station that would be required.

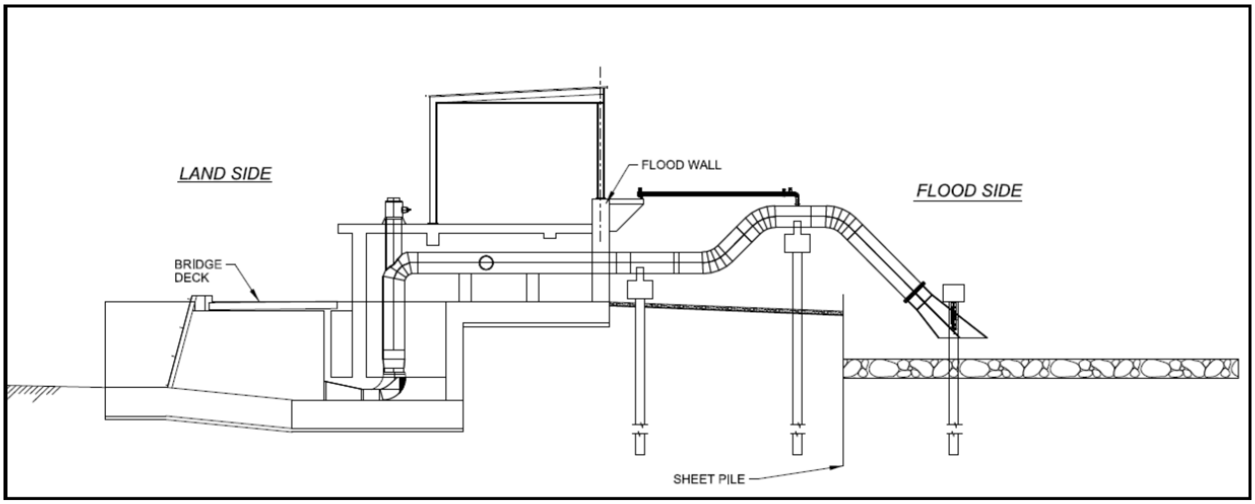


Figure 2.3.5.9 – Pump Station Typical Section (Courtesy of USACE-NAO)

7. Real Estate Considerations

Preliminary Real Estate acreage requirements (permanent and temporary easement limits) were computed for all the structures shown in the various plan alignments and provided to Baltimore District Real Estate. Refer to the figures provided in Section 7 Real Estate. The goal of the alignments was to impact existing properties as little as possible and limit the number of buyouts required to construct the plan. The initial analysis indicates that real estate values for the perimeter plan will be very high due to the amount of floodwall required and the amount of easement area required for the footprint of the proposed structures on private property. This may include compensation to front line property owners for loss of view as previously mentioned. An alignment change that was considered for future work in the NJBB formulation was to move the line of protection offshore, which was suggested by the Non-Federal Sponsor. This idea was not explored for NCBB due to the presence of highly sensitive environmental areas (CBRA System Unit), potential impacts to existing navigable waterways, potential increase in construction costs, and various other reasons. Another alignment alternative would be to buyout all owners adjacent to the waterfront and build structural protection in those locations landward of the existing protection. That option was not explored as a portion of this study due to the high density of structures, both residential and commercial, in the area that are situated along the waterfront and are essential to the stability of the municipalities. These additional alignment strategies could be explored post-TSP at request of the sponsor, but for the purposes of this phase of the study the plan formulation follows the current alignments for all the design assumptions and considerations previously referenced within this appendix. The team did explore the option of creating smaller perimeter plans focused on managing risk to critical infrastructure essential to each community that are further discussed in Section 3.0 of this appendix.

3 CRITICAL INFRASTRUCTURE PLAN ANALYSES

3.1 Critical Facility Identification

Critical Infrastructure Plans were developed as an alternative to community perimeter structural risk management for this study. Critical Infrastructure is identified in the Army Field Manual (FM) 3-34.170 as SWEAT-MSO. A list of facility categories that fall under SWEAT-MSO have been identified in Table 3.1.1 below. Due to the highly developed nature of the NCBB study area several facilities could be classified within the SWEAT-MSO identification guidance. It is important to note that some facilities may not fall within the table below but are still considered critical to the community. As discussed with the vertical team, Critical Infrastructure is also represented by any facility that is essential to community recovery after a major storm event. Therefore, essential economic drivers such as industrial facilities, commercial areas or tourist attractions can also be deemed as critical for the purposes of plan formulation. These facilities would fall into the "Other" category listed below.

Table 3.1.1 – Army Field Manual SWEAT-MSO

Conditions		Indicators	Metrics
S	Sewage	Collection	Kind of Toilet; Sewage Service Availability Perception (simulated)
		Treatment	Distance to Wastewater Treatment
W	Water	Production	Distance to Water Tower; Functionality of Water Facilities
		Distribution	Cooking/Drinking Water Source; Laundry/Bathing Water Source; Water Availability Perception (simulated); Water Security Disruption
E	Electricity	Generation	Distance to Electrical Transformer
		Distribution	Fuel for Lighting; Fuel for Cooking; Has Washing Machine; Has Refrigerator; Has Television Set; Electricity Availability Perception (simulated)
A	Academics	Facilities	Distance to School
		Services	Literacy; School Attendance; Highest Grad Completed; School Availability Perception (simulated)
T	Trash	Collection	Manner of Garbage Disposal; Trash Collected Perception (simulated)
		Disposal	Distance to City Dump
M	Medical	Facilities	Distance to Medical Facility
		Services	Has Disability; Medical Availability Perception (simulated)
S	Safety	Facilities	Distance to Police/Fire Station; Distance to Government Administration Building
		Services	Has Television Set; Has Radio; Has Telephone; Police Perception (simulated); Army Perception (simulated)
O	Other	Transportation	Distance to Major Road; Traffic Perception (simulated)

The PDT used mapping techniques to identify all critical infrastructure that met the above field manual categories within Nassau County. All critical infrastructure identified within the study area that is susceptible to AADs will be protected at minimum by a Non-Structural measure (Elevation, Dry-Floodproofing, Wet-Floodproofing, etc.). However, the challenge for the team was to determine what facilities would be good candidates for a Structural measure in the form of a localized perimeter plan. To accomplish this a Critical Infrastructure screening process occurred.

3.2 Critical Infrastructure Plan Screening

The team developed the following criterion for selecting critical facility locations eligible for structural measures:

- Must meet Army SWEAT-MSO guidelines for Critical Infrastructure.
- Must fall within the 1% AEP floodplain limits.
- Protection must maintain the functionality of the facility.
- No adverse impacts to surrounding properties/facilities.
- Cannot be within the CBRA System Unit.

These criteria effectively screened out several locations since many critical facilities are in highly developed areas that floodwall risk management would not only impact the functionality of the critical facility, but also impact other properties in terms of stormwater conveyance, property encroachment and viewshed impacts. An example of a critical facility that was chosen for Non-Structural evaluation only is shown in Figure 3.2.1 below. The Island Park Fire Department is located on a busy thoroughfare in a densely populated community. At this location it is not feasible to construct a perimeter plan around the property as floodwall footprints would both encroach upon and increase stormwater runoff onto adjacent properties. Also, the function of a Fire Department is to quickly mobilize equipment and personnel to and from the firehouse. A wall around the perimeter would inhibit this mobility unless several closure gates are installed. This could be achieved but could block driver viewshed when exiting the building which could impact traffic patterns. Therefore, the best course of action is a Non-Structural measure that maintains the current facility footprint. See Section 5.3 for more information.

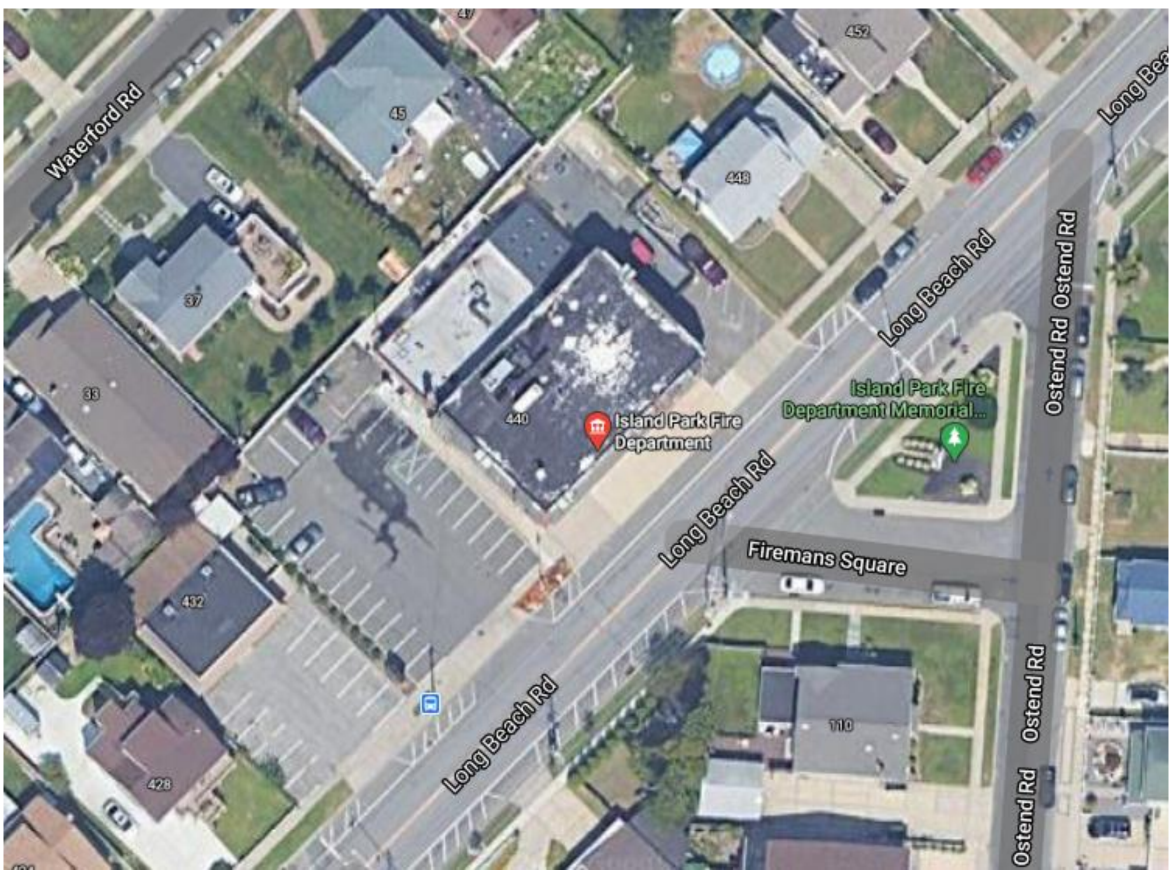


Figure 3.2.1 – Island Park Fire Department Aerial View (Courtesy of Google Images)

An example of a location that would meet the criteria developed for the screening is the Bay Park Wastewater Treatment Plan (WWTP). In this location a perimeter concrete floodwall and levee was constructed by the County after serious damage was sustained during Superstorm Sandy. The site plan of the proposed construction is shown in Figure 3.2.2 below.

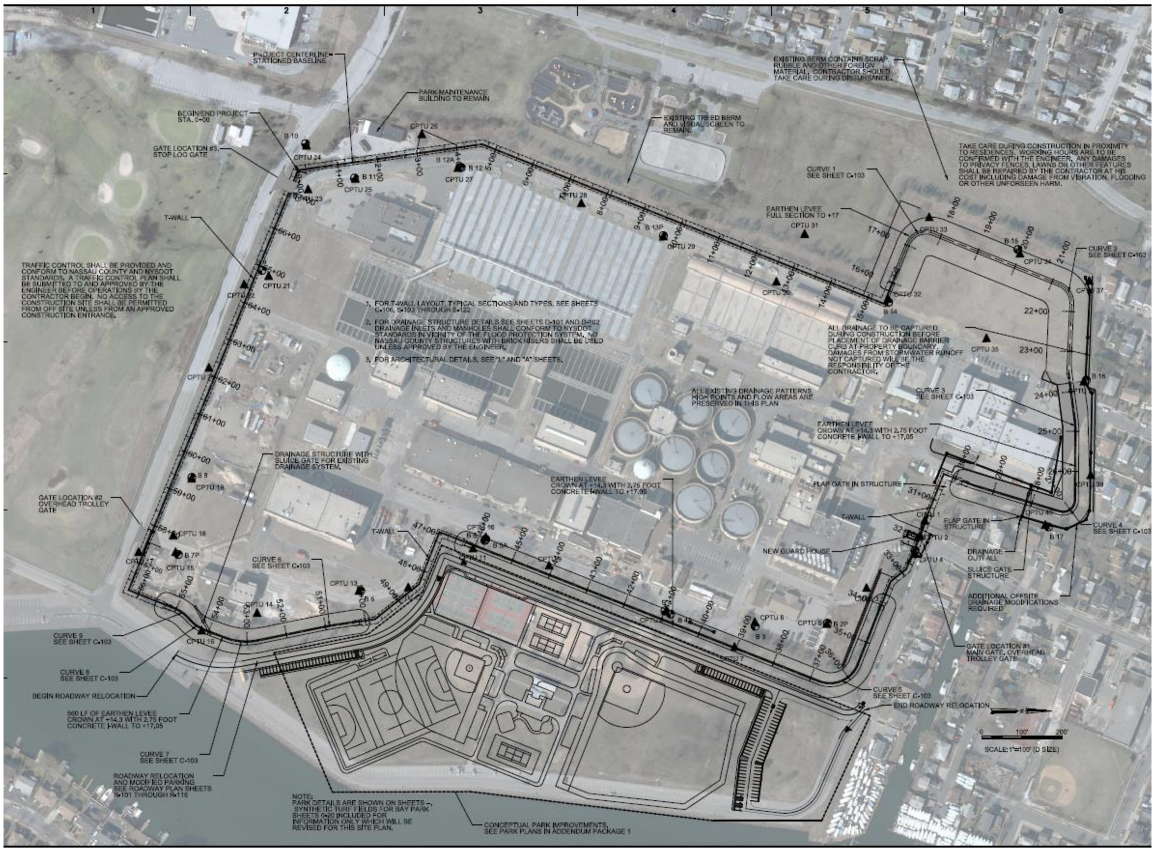


Figure 3.2.2 – Bay Park WWTP Protection Plan (Courtesy of Google Images)

The location of this facility is ideal for a structural solution due to its proximity to a large amount of open space, the number of structures within the facility (i.e. perimeter plan is more ideal for several component facilities rather than an individual facility) and proximity to the waterfront. The floodwall height at this location was designed for the 0.2% AEP storm event or 500 year. For this phase of the NCBB study, Critical Infrastructure Plans were formulated to the 1% AEP design storm event (El. +16.0 NAVD88), but formulation to the 0.2% AEP storm event could be optimized post-TSP if critical infrastructure plans are selected. For this phase of the study, all critical infrastructure is assumed to remain in its current location and not to be relocated. Per E0-11988 the team must then ensure that if a facility must remain within the 1% AEP floodplain and is to be protected by a floodwall or levee then it must be designed to protect from the 0.2% AEP floodplain elevation. The 0.2% AEP storm event differs from the floodplain elevation, but this optimization could increase or decrease the Critical Infrastructure Plan heights. Figure 3.2.3 shows the height of the floodwall constructed around the Bay Park WWTP. The placard shown in the image is the flood elevation recorded at this location during Superstorm Sandy.

Figure 3.2.3 – Bay Park WWTP Floodwall (Courtesy of USACE-NAP)

Using the stated assumptions and design criteria initially three (3) intrinsic structural plans were developed that protected critical infrastructure. An example of one of these plans is depicted in Figure 3.2.4. This plan is risk management for the E.F. Barret Power Generation Station in Island Park, NY. This station is crucial to supplying power to Nassau County and damages sustained to the plant in a storm event could result in major economic and social hardships to the community. All Critical Infrastructure Plans can be found in the maps and drawings attached to this appendix.

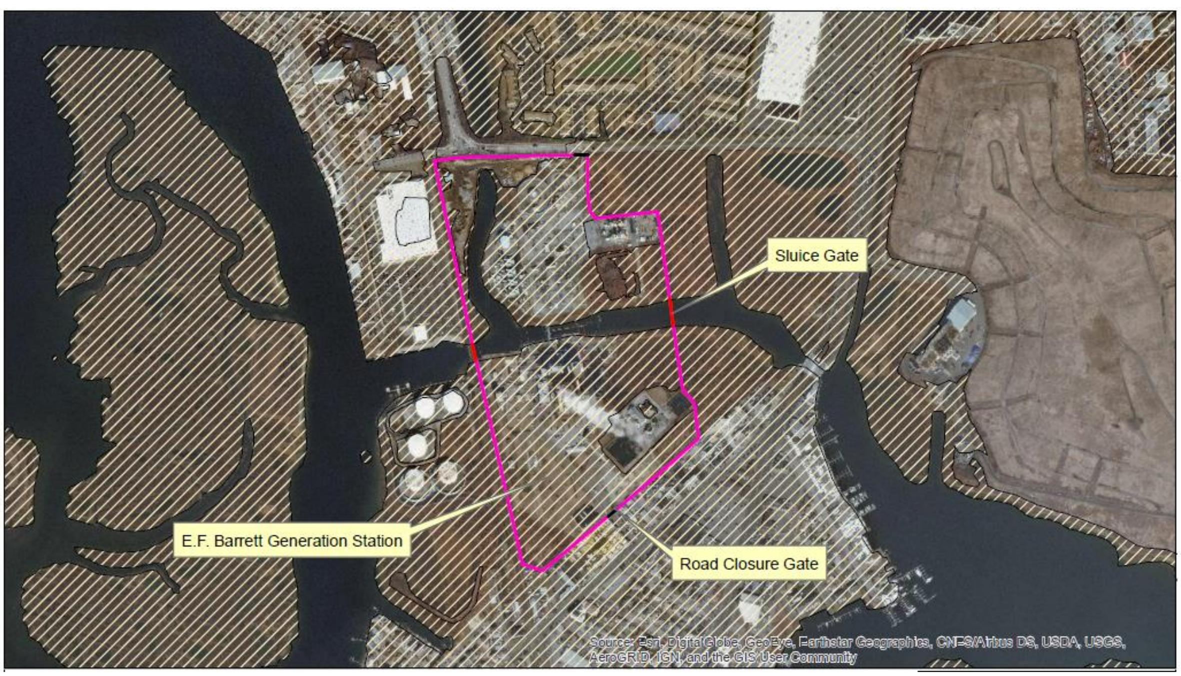


Figure 3.2.4 – Island Park Critical Infrastructure Plan (Courtesy of USACE-NAP)

The three plans developed were in the Village of Freeport, Village of Island Park and the City of Long Beach, which are all HVAs. The team did not want to limit structural plans for critical facilities just to HVAs but could not determine any candidates outside of the HVAs. The team reached out to the Non-Federal Sponsor and coordinated a Site Visit to identify any additional areas that would meet the established criterion. See Exhibit E for more information on the site visit referenced. From that visit, the Cedar Creek Wastewater Treatment Plant (WWTP) in Wantagh, NY was identified as another location.

Per a progress review meeting with the vertical team, the PDT was tasked to evaluate if risk management for the evacuation routes within the study area was possible. For the purposes of structural critical infrastructure plan development, evacuation routes can be considered as a critical facility within the "Other" category of the SWEAT-MSO guidance. Figure 3.2.5 shows the four (4) major evacuation routes within Nassau County. Portions of Evacuation Routes No.1 and No. 4 that where within the 1% AEP floodplain were considered for a structural risk management plan. Evacuation Route No. 2 was not considered since it is encompassed by contiguous risk management from the HVA plans. If the HVA plans were not considered for the TSP, then Non-Structural measures would be required to protect the evacuation route which would be evaluated post-TSP. No plan was proposed for Evacuation Route No. 3 since the portion within the 1% AEP floodplain is also within the CBRA System Unit. With the addition of plans at Evacuation Routes No. 1 & No. 4, a total of seven (7) intrinsic Critical Infrastructure Plans were developed for this phase of the study. Table 3.2.1 contains a summary of the Critical Infrastructure Plans.

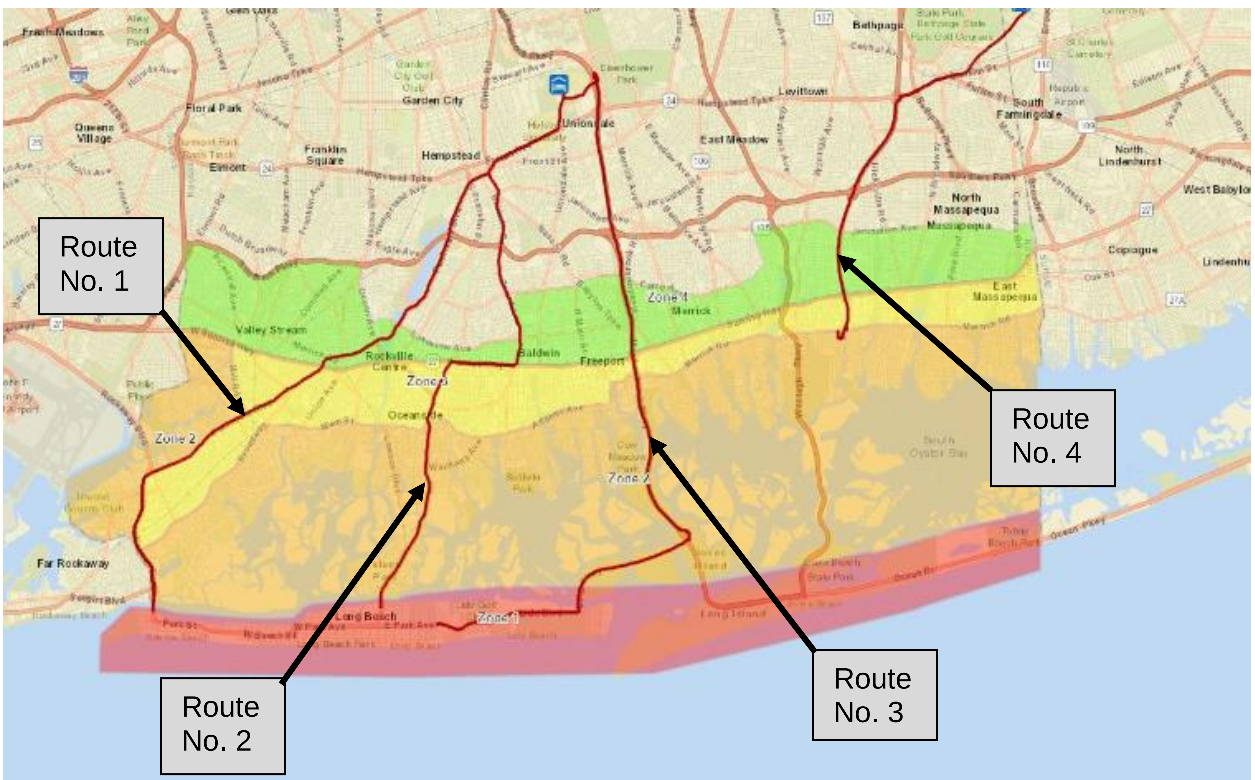


Figure 3.2.5 – Nassau County Evacuation Routes (Nassau County 2020)

Table 3.2.1 – Critical Infrastructure Alignment Summary Table

PERIMETER PLAN			QUANTITY								
Location	Cycle 2 Polyline Names	AEP%	Floodwall / Levee (ft)	Type A	Type B	Type C	Type D	Miter Gates (ea)	Sluice Gates (ea)	Road Closures (ea)	Rail Closures (ea)
Freeport	FPV-CI100	1	12,245	-	8,221	4,024	-	0	0	3	0
Long Beach	LBC5-CI	20*	1,505	-	-	-	1,505	0	0	0	0
Long Beach	LBC100-CI	1	10,283	-	-	10,283	-	0	0	3	1
Island Park	IPV100-CI	1	6,951	-	-	6,951	-	0	2	2	0
Far Rockaway	FROC100-CI-EVAC	1	7,059	-	-	7,059	-	0	1	4	0
Wantagh	WTG100-CI	1	6,080	-	-	6,080	-	0	0	1	0
Wantagh	WTG100-CI-EVAC	1	792	-	-	792	-	0	0	0	0
*20% AEP Critical Infrastructure Plan completed at Long Beach only											

A 20% AEP Critical Infrastructure Plan was also developed for the City of Long Beach only. This plan does not meet the EO 11988 guidance for critical infrastructure risk management plans (i.e. designed for a minimum of 1% AEP). However, it was developed to build off an existing plan that is currently being proposed by the City and FEMA. The Long Beach plan is to construct an El. +9.0 floodwall to protect critical infrastructure adjacent to the back bay waterfront. See Figure 3.2.6 for the alignment of this plan. The USACE plan would extend this alignment to ensure the 20% AEP floodplain is fully repelled in the area of the critical facilities. This plan is something the local sponsor can consider as part of an LPP or to utilize to increase the scope of their existing design. See Exhibit E for the USACE plan.

Figure 3.2.6 – Proposed Long Beach Critical Infrastructure Plan (Long Beach 2020)

3.3 Critical Infrastructure Plan Future Work

If any of the Critical Infrastructure Plans are selected for TSP, optimization will be conducted. There will be a difficulty in selecting these plans quantitatively due to the difficulty for the Economics team to identify benefits outside of the structural risk management benefits of the individual plans. For example, it is currently quantifiable for the team to determine damage to the facilities within a power plant during a storm event and then correspond those benefits to the cost of the implementation of a SBM around that facility. However, it is not quantifiable for the team to ascertain economic damages to residential and commercial facilities when a power plant goes offline during an event. The costs associated with the indirect impacts to the community when a critical infrastructure element is damaged or temporarily out of service can be significant and may affect the BCR of that plan greatly. Therefore, once this abstract analysis is completed post-TSP, a more informed decision regarding the use of critical infrastructure plans.

4 STORM SURGE BARRIERS

4.1 SSB Background

The second structural alternative that this study focused on was the implementation of Storm Surge Barriers (SSBs). The M&N Report from 2018 also explored the possibility of utilizing Storm Surge Barriers (SSBs) as coastal storm risk management solutions. SSB alignments were proposed in Alternative 1A and 1B. Figure 4.1.1 and Figure 4.1.2 below show the respective plans. The SSBs for Alternative 1A would span East Rockaway, Jones and Fire Island Inlets within Nassau County. Alternative 1B would have SSBs span East Rockaway and Jones Island, while providing a cross-bay barrier along Wantagh Parkway.

Figure 4.1.1 – Storm Surge Barrier Alternative 1A (Moffat & Nichol 2018)

Figure 4.1.2 – Storm Surge Barrier Alternative 1B (Moffat & Nichol 2018)

When the project was transferred to USACE-NAP the team further expanded on the SSB plan alternatives, creating four different alignments. See H&H Appendix for more information regarding the SSB plan formulation and hydraulic modelling results. Figure 4.1.3 contains the Alternatives 1A-1D that were hydraulically modeled with the help of ERDC.

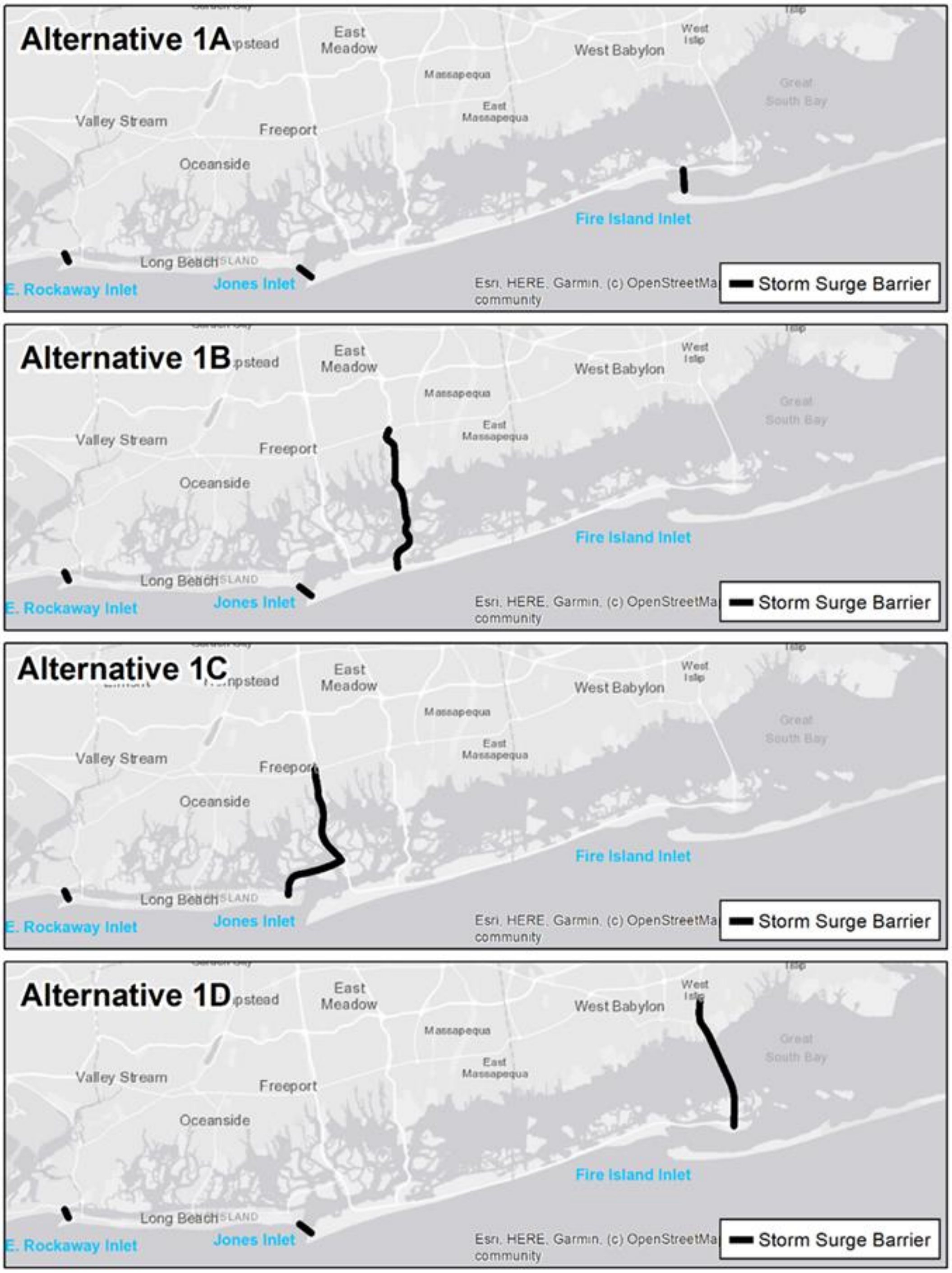


Figure 4.1.3 – USACE-NAP SSB Alternatives (Courtesy of USACE-NAP)

4.2 SSB Screening

Underwhelming hydraulic modelling results coupled with constraints from formulation within the CBRA System Unit effectively screened out all proposed SSB solutions for coastal storm risk management. Therefore, all associated structures such as seawalls, revetments, sector gates, vertical lift gates and other potential structures that would have been included in these alternatives were all screened out with the reduction of SSBs from consideration for TSP. A succinct explanation for the hydraulic screening of the SSBs has been included in the H&H Appendix. Figure 4.1.4 shows the SSB plans in relation to the CBRA System Unit.

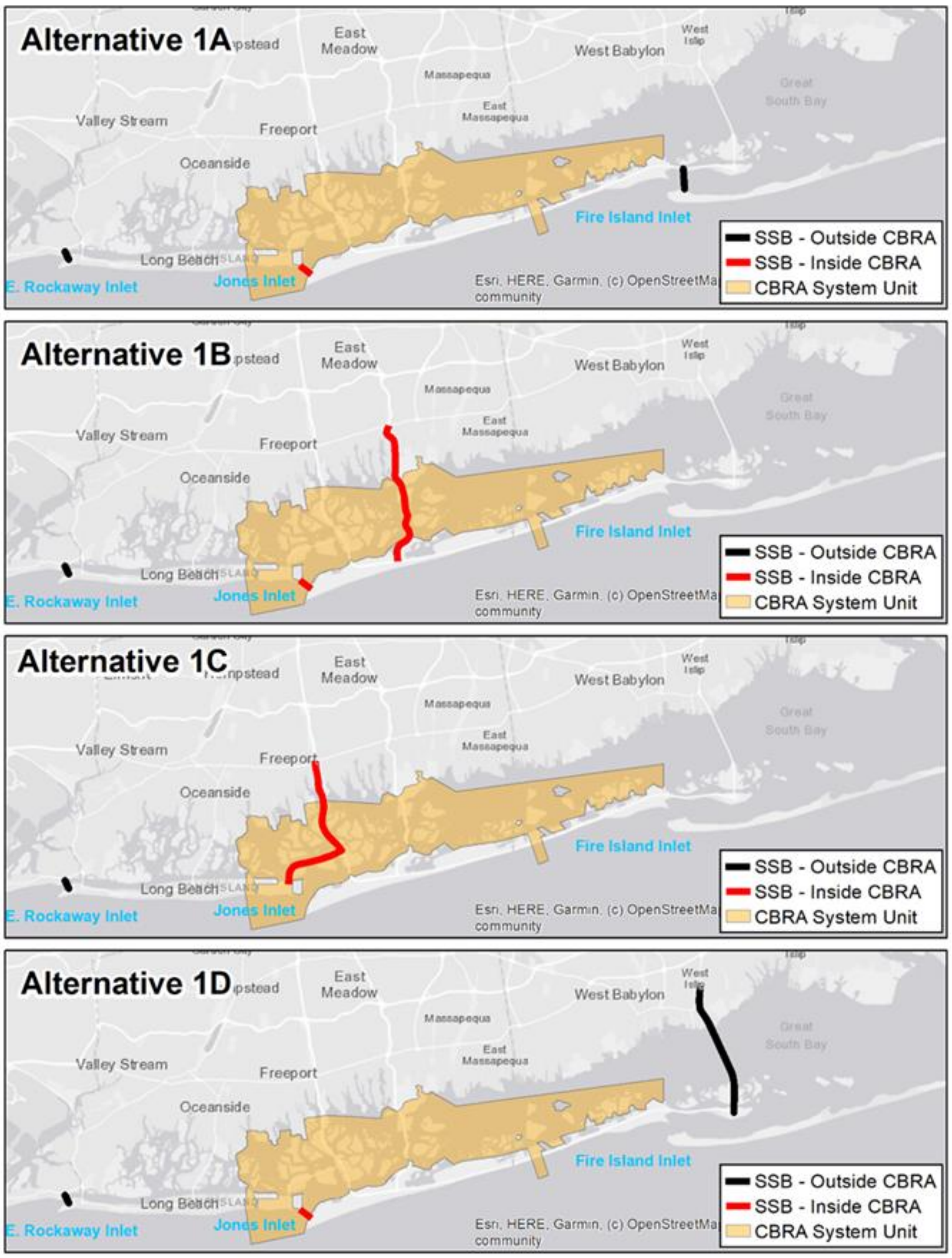


Figure 4.1.4 – USACE-NAP SSB Alternatives w/ CBRA System Unit (Courtesy of USACE-NAP)

5 NON-STRUCTURAL

5.1 Background

All areas that were not identified as HVAs in the initial plan development and screening process would still be included for risk management in this study by non-structural solutions. Raising structures (primarily residential) to elevate the first floor above the design flood level and Dry-Floodproofing large public and commercial facilities have been considered for this phase of the screening process. Due to the large inventory of structures, for this phase of the study parametric costs from other USACE projects have been utilized for elevations and dry-floodproofing. Figure 5.1.1 below shows a graphic representation of an elevation alternative. Refer to the Economic Technical Appendix for information on the analysis. Future alternative analyses will consider other non-structural measures such as flood proofing, deployable flood walls, ring levees/floodwalls, etc. All Non-Structural Analyses can be found in Appendix A of this report.

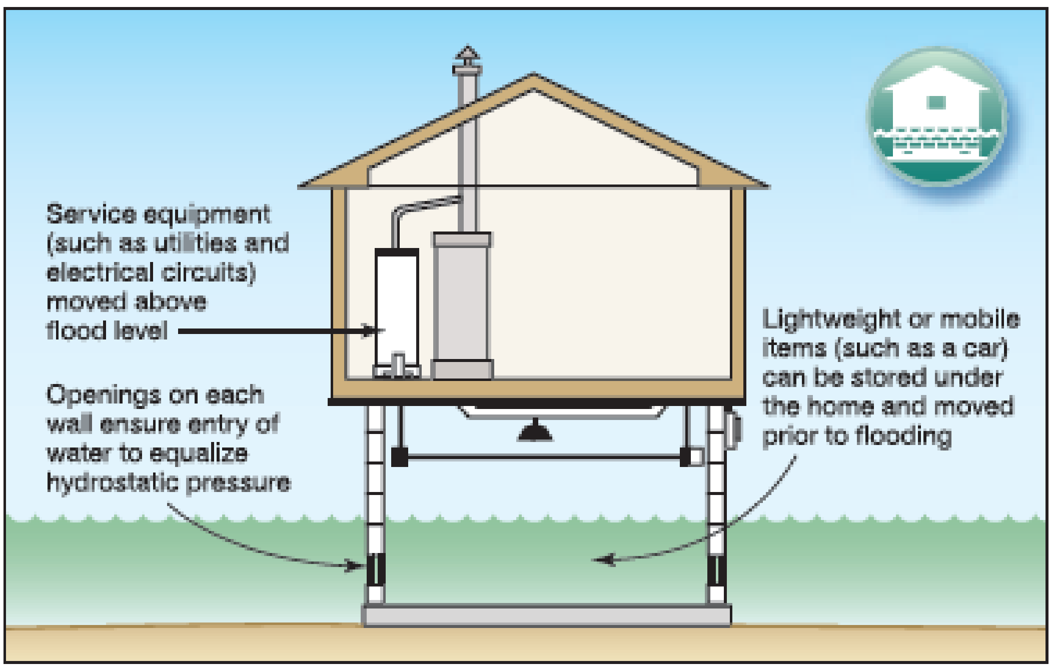


Figure 5.1.1 – Home Elevation Concept Diagram (Courtesy of Google Images)

Non-structural CSRM measures are divided into two primary categories, physical and non-physical. Physical non-structural measures include: buyout/acquisition, dry flood proofing, wet flood proofing, elevation and relocation. Non-physical non-structural measures include: evacuation plans, flood emergency preparedness plans, floodplain mapping, land use regulation, risk communication, zoning, flood Insurance and flood warning systems.

Despite the use of parametric design costs for this portion of the study, Civil Design was tasked with developing conceptual design sand figures for both Elevation and Dry-Floodproofing solutions that could be carried forward post-TSP and used for developing designs and cost estimates intrinsic to this study. The work completed by the team for both items has been included herein.

5.2 Elevations

The National Non-Structural Committee (NNC) has provided guidelines for Non-Structural mitigation measures. Per the NNC Non-Structural Measures Matrix that the team referenced during plan formulation there are six (6) different design options available for elevation of private residencies. These include: Extended Foundation, Piers, Posts, Columns, Piles and Fill. Per a site visit conducted by the team, it was noticed that the most common form of elevation of existing properties was an extended foundation. A comprehensive inventory of structures designated for elevation will be gathered post-TSP and statistics will be compiled on the existing structure foundation and the appropriate elevation methodology. A series of conceptual figures developed by the team and sample photographs taken by the team have been attached in Exhibit F "Elevation Concept Drawings" for reference. The extended foundation sketch has been included in Figure 5.2.1 below. Figure 5.2.2 contains an example of existing residences elevated using the extended foundation elevation method in Long Beach, Nassau County.

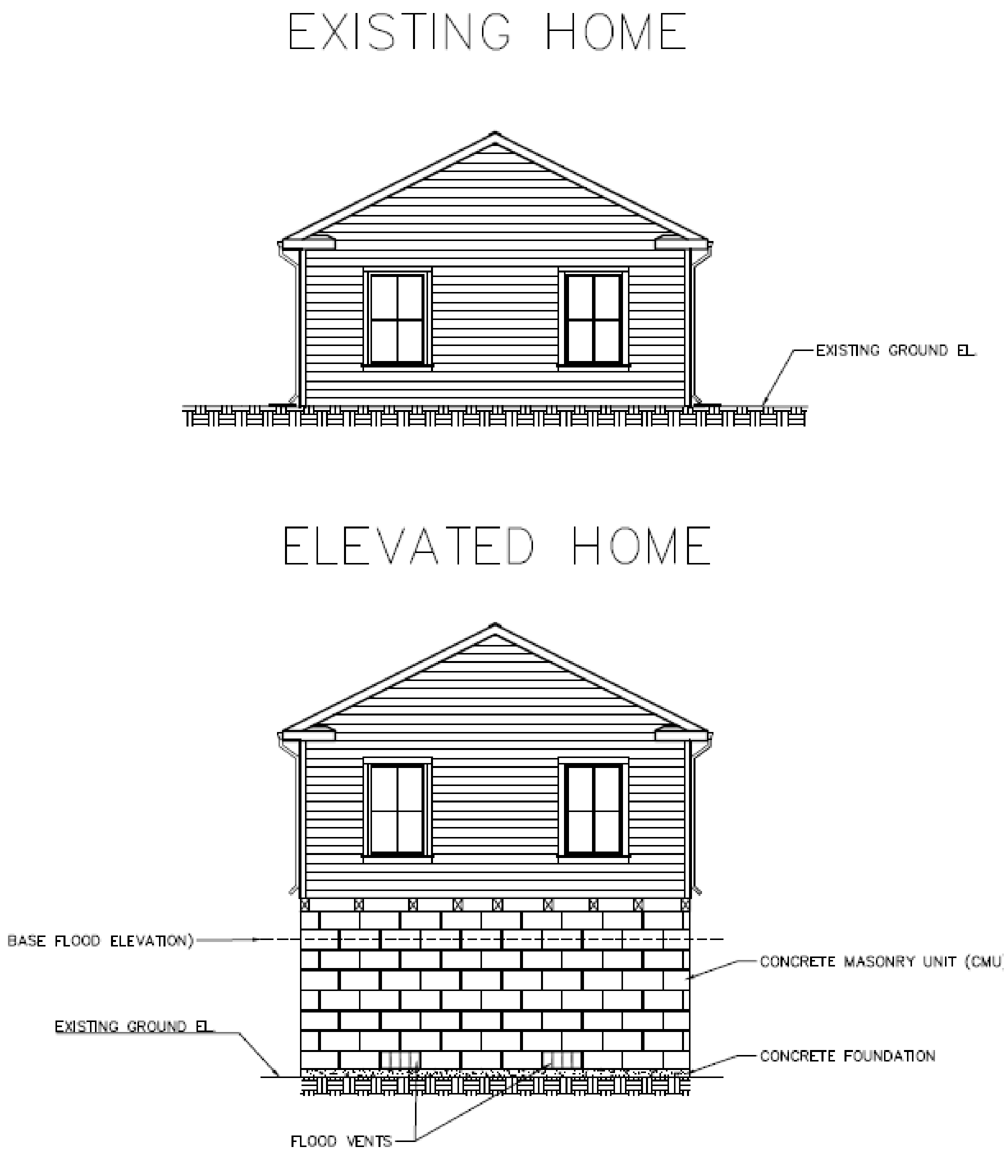


Figure 5.2.1 – Extended Foundation Concept Drawing (Courtesy of USACE-NAP)

Figure 5.2.2 – Existing Extended Foundations in Long Beach, NY (Courtesy of Google Images)

Directly adjacent to this location provided an excellent sample area for renderings of other potential Non-Structural elevation solutions implemented over a single block. A before and after of three (3) different Non-Structural Measures is shown in Figure 5.2.3. These measures include the following (as shown from left to right in the figure):

- Basement Fill (Compacted Fill & Sealing) with Utility Relocation. A third floor has been added to recover lost space from basement closure.
- Extended Foundation Elevation. Shown here for conceptual purposes, other options are possible.
- Wet Floodproofing with the installation of flood vents at first floor level. Utility elevation with raising the HVAC unit on a wooden pedestal.

It is important to note that all structural elevations cannot be elevated over 12 feet above ground level. If 12 feet or more elevation was required or if the property was in poor structural condition for elevation, a buyout would be recommended. All structural elevations are based off floodplain elevations developed through hydraulic modelling and must comply with all local, state and federal elevation requirements. Appendix A further describes these requirements. The alternative to these Non-Structural Measures at this location is the implementation of a floodwall from the perimeter plan alignments. Figure 5.2.4 shows the what the street would look like with a concrete floodwall at the 1% AEP Plan.

All elevation techniques will need to be further evaluated post-TSP if Non-Structural Measures were selected. The additional evaluation may be in the form of performing inspections of specific sample properties and creating associated designs that can be utilized to develop intrinsic quantities and refine cost estimates for the various alternatives. These costs can then be extrapolated over the entire inventory of NCBB structures selected for elevation.

Figure 5.2.3 – Non-Structural Measure Rendering (Courtesy of USACE-NAP)

Figure 5.2.4 – Structural Alternative at Rendering Location (Courtesy of USACE-NAP)

5.3 Dry-Floodproofing

Compared to Elevation methods which look to extend the bottom elevation of the first floor above the flood elevation, the Dry-Floodproofing methodology maintains a structure at its current elevation but ensures that the building is impermeable to floodwaters. As previously stated, most residential structures will receive some form of elevation. In contrast, large public, industrial or commercial facilities are too complex to elevate and will require Dry-Floodproofing. This technique is proposed to protect Critical Infrastructure that has been identified for Non-Structural solutions.

Various critical facilities exist within Nassau County as discussed in Section 3.0 of this appendix. With several hundred facilities in need of risk management at various sizes, functionalities and first floor elevations, the team identified a series of sample facilities that represented the array of structures within the study area. The intention of the sample facility selection is to develop a conceptual Dry-Floodproofing risk management plan for each sample that could be quantified and costed. From which these costs could be extrapolated over the structure inventory for the facilities with a similar classification creating a more refined estimate, as compared to using parametric costs from other USACE projects.

Table 5.3.1 shows the range of facilities identified for this phase of the study that captures the critical facilities located within Nassau County. This list will change during the optimization phase, likely expanding to include a wider variety of structures. Conceptual Dry-Floodproofing plans for each of these locations are shown in Exhibit G “CENAP-EC-EC Non-Structural Map Deck”.

Table 5.3.1 – Sample Facilities selected for Dry-Floodproofing

Facility Type	Sample Facility	Location
Small-Medium Public	Island Park Fire Department	Island Park
Large Public	Long Beach City Hall	Long Beach
Small-Medium Medical	South Nassau Emergency Facility (Urgent Care)	Long Beach
Large Medical	Mt. Sinai South Nassau Hospital	Oceanside
Small-Medium School	Francis X Hagerty Elementary School	Island Park
Large School	Long Beach High School	Long Beach
Industrial	Farber Plastics	Freeport

Note: A conceptual plan for Mt. Sinai South Nassau Hospital is not included in Exhibit G.

The Island Park Fire Department was previously discussed as a location not suitable for intrinsic structural risk management but will be protected through Non-Structural Measures. For this phase of the study the facility was selected as a candidate for Dry-Floodproofing. Dry-Floodproofing will include the following actions:

- Application of a permeable membrane (up to 3 feet above first floor elevation per NNC guidance) in the form of an epoxy paint/sealer.
- Installation of flood shields and stop logs installed in front of all openings that require ingress and egress. This includes access panels, doorways, garage openings, etc.
- Sealing of all pipe penetrations from the building exterior to ensure impermeability.
- Elevation of all external utilities susceptible to flood damage above design flood elevation.

See the Appendix A Non-Structural for the full evaluation of this and other sample facilities.

Figure 5.3.1 shows the proposed conceptual risk management plan of the Island Park Fire Department with Dry-Floodproofing Methods. Figure 5.3.2 contains a rendering of what the Dry-Floodproofing risk management would look like at this location.

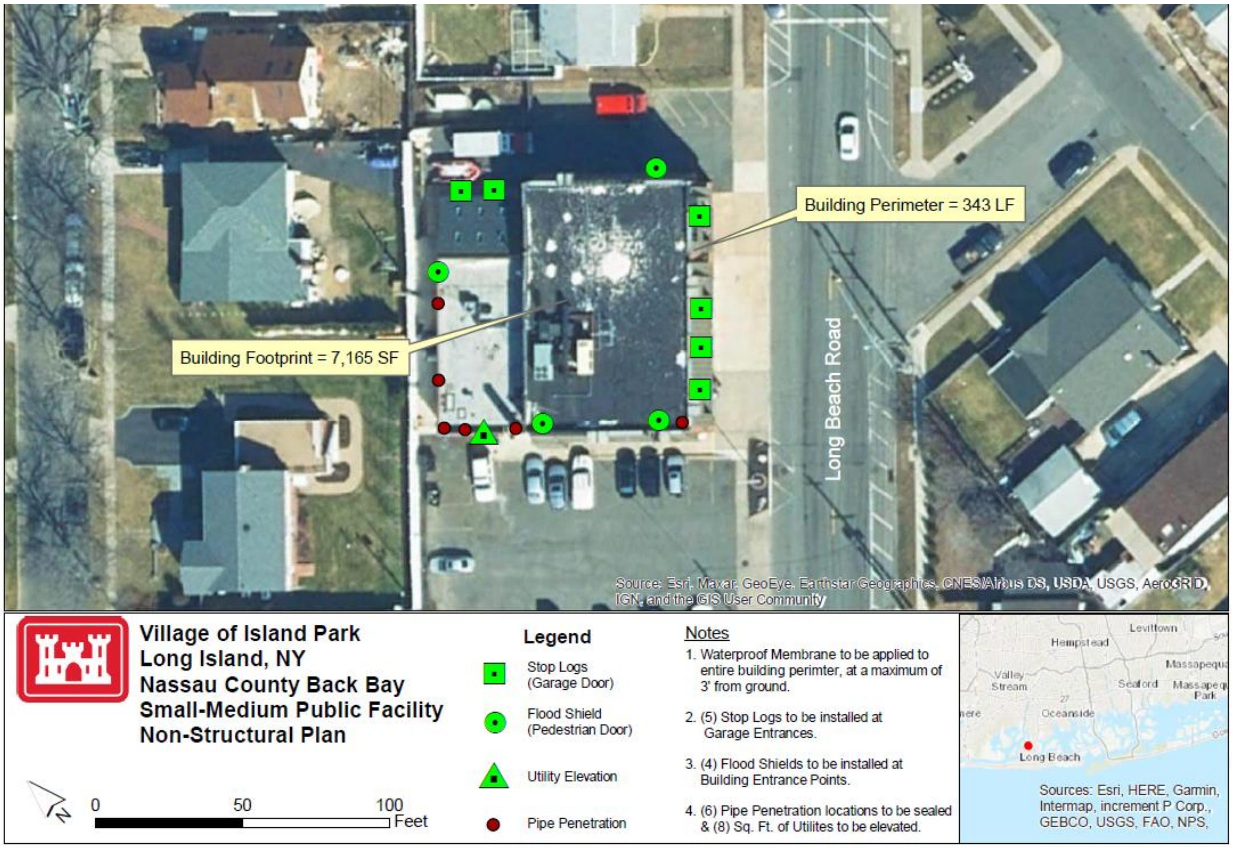


Figure 5.3.1 – Island Park FD Dry-Floodproofing Concept Plan (Courtesy of USACE-NAP)

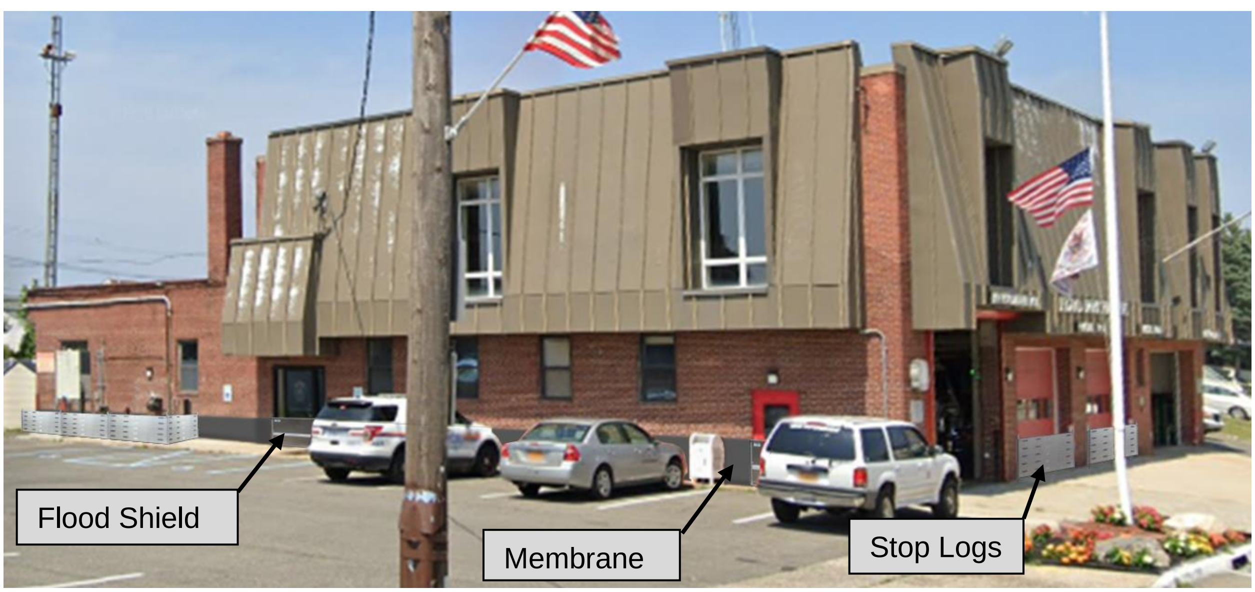


Figure 5.3.2 – Island Park FD Dry-Floodproofing Rendering (Courtesy of USACE-NAP)

6 ALTERNATIVE DESIGN MEASURES

6.1 Natural and Nature Based Features (NNBF)

Post-TSP an additional screening effort will be completed to identify portions of the study area for possible NNBF sites and measures. For the NJBB study, an initial level of screening for NNBF was completed an array of measures was screened down to focus primarily on living shorelines and EWN (Engineering with Nature) modifications. Refer to the Environmental Technical Appendix G for information on the analysis completed by USACE-NAP. Living shorelines may be created in areas where risk management incorporates levee frontage. EWN features, such as textured concrete, habitat benches, ecologically enhanced revetments, horizontal levees and other options can be incorporated into the design of floodwall and levee structures. Preliminary costs of potential NNBF solutions have been captured within the contingency values applied to the cost estimate of the flood control features. Figure 6.1.1 contains a rendering from NACCS that shows the before and after construction.

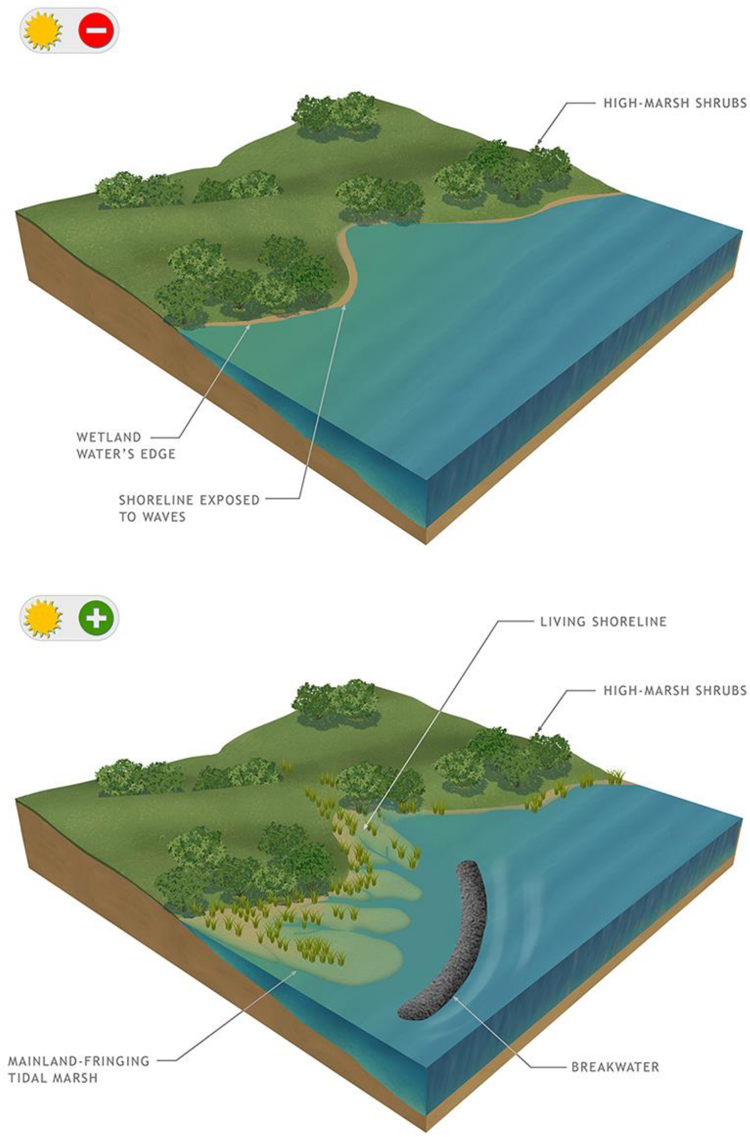


Figure 6.1.1 – Living Shorelines Example (Courtesy of NACCS)

6.2 Adaptability Measures

Post-TSP an additional screening effort will be completed to identify if any adaptability measures can be added to the selected plan that will extend the intended design use. Per USACE guidance EC 1165-2-11 the planning phase must give a consideration for adaptive management of risks related to RSLC. Current design elevations discussed in this study account for the 2080 intermediate RSLC projection. However, it is understood that the possibility exists that the design RSLC may differ from actual RSLC rate. Therefore, the team must investigate potential adaptable measures in the feasibility design to be able to increase the level of risk management of any coastal storm risk management feature to meet the desired design lifespan. Herein are potential adaptable measures for both Structural and Non-Structural solutions.

Structural Measures

Floodwalls and Levees are permanent hard structures that once designed can prove difficult to adapt. Levee adaptation can be costly, but from a design perspective can be simple. A Levee sub-structure could be designed in such a way to ensure additional future fill to raise the design crest elevation would not destabilize the structure. For Floodwalls, adaptability measures can be more difficult to design and implement. An example of a potential adaptability measure for a Floodwall is a Wave Bumper Fenceblade. This product is currently being utilized by USACE-NAP on the Manasquan to Barnegat Coastal Storm Damage Reduction Project. At Grant Avenue in Seaside Heights, NJ a steel bulkhead with concrete cap has been constructed along the beachfront. The concrete cap located at the location where vehicle access is planned to be maintained will be built to match the elevation of the existing adjacent boardwalk El. +15.5 NAVD88 with a deployable feature that can extend the risk reduction to El. +18.0 NAVD88 to be contiguous with the surrounding level of risk management. Figure 6.2.1 shows the detail for the floodwall design and Figure 6.2.2 shows the completed product.

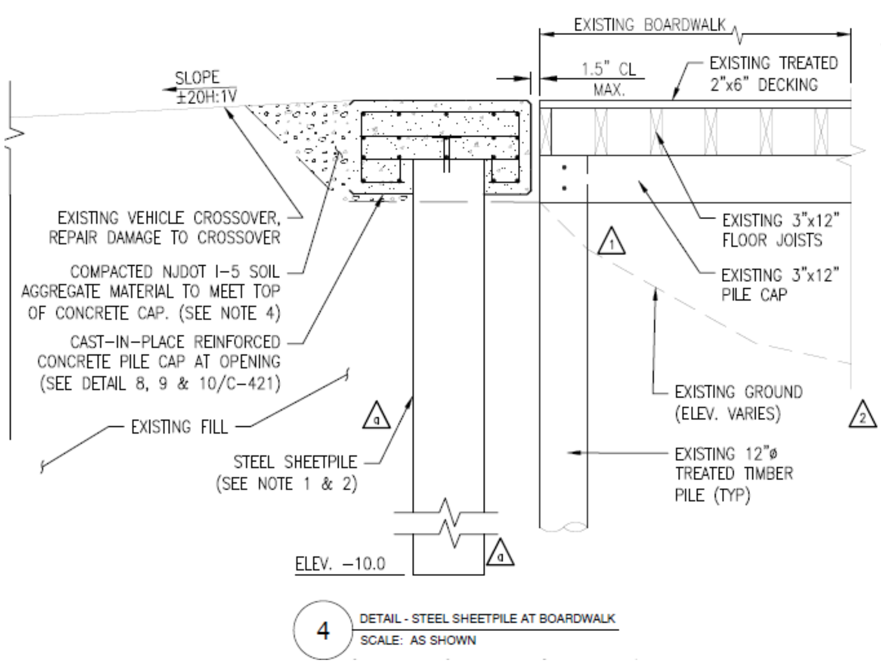


Figure 6.2.1 – Grant Avenue Adaptability Design (Courtesy of USACE-NAP)

Figure 6.2.2 – Grant Avenue Fenceblade Barrier System (Courtesy of USACE-NAP)

The Fenceblade is a rapidly deployable flood barrier that also provides additional wave deflection benefits due to its concave design. Incorporating a feature like the Fenceblade to a structural solution can greatly increase the floodwall's adaptability. For example, a structure that is designed for a 20% AEP event at El. +13.0 NAVD88, could have a Fenceblade component that allows the structure to be elevated to a 1% AEP event level of risk management when such a storm is predicted. The ability to rapidly implement a feature like this over a large amount of floodwall length such as those proposed in the Perimeter Plan Alignments may not be feasible due to the lead time needed and the manpower required but could be better suited for the smaller scale Critical Infrastructure Plans. This type of feature, among others, would be further vetted post-TSP if structural solutions are selected for Feasibility Phase analyses. Figure 6.2.3 shows a rendering of the Fenceblade product.

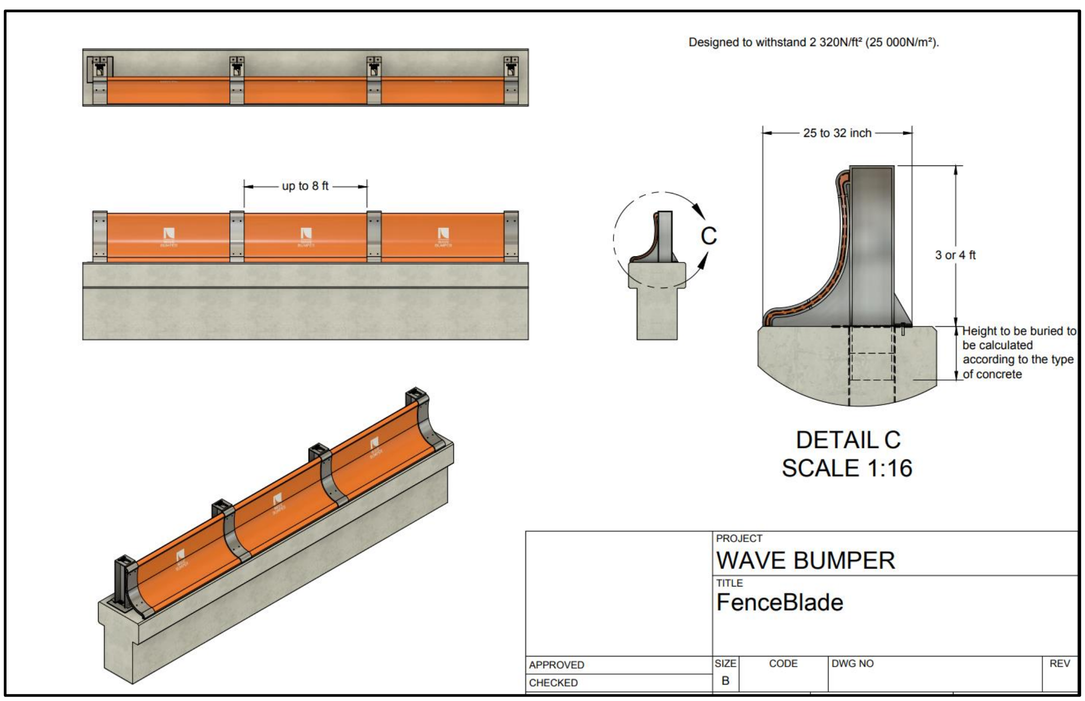


Figure 6.2.3 – Wave Bumper FenceBlade Rendering (Courtesy of USACE-NAP)

Non-Structural Measures

For Non-Structural solutions such as elevation of structures, it may be difficult to adapt a design elevation once a structure has been elevated. However, for Critical Infrastructure there may be several opportunities for adaptable features. For Dry-Floodproofing for example, if projected flood elevations increase due to a more rapid increase in RSLC, flood shields and panels can be adapted to increase in height with space for additional panels above the intended design flood event elevation. Impermeable membrane can also be applied further up the exterior of a structure to increase surface area risk management. Another adaptable measure is the use of a rapid deployable barrier that can be put in place prior to a major storm event and sized according to the design event over the lifespan of the project. One structure that could be used in the AquaFence. AquaFence is an approved product by the NNC and is a rapidly deployable flood barrier that can close openings in a facility or completely encompass a facility perimeter for risk management during a major flood event. A product like AquaFence could be supplied to the local communities to store in a nearby location and quickly deploy prior to an event. These would most likely be utilized at Critical Infrastructure locations where elevation solutions are not possible. Figure 6.2.4 shows an example of an AquaFence in use. Adaptability measures like this product and many others will be further explored in the Feasibility Design of Non-Structural solutions.

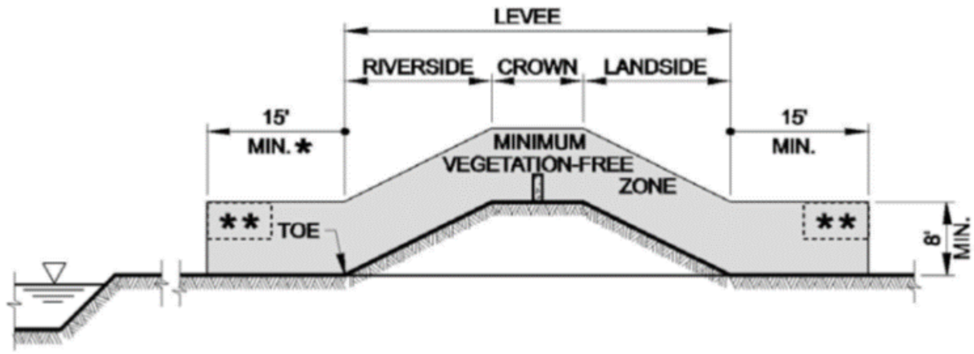
Figure 6.2.4 – AquaFence Non-Structural Measure in Use (Courtesy of Google Images)

7 REAL ESTATE

The Real Estate impact costs for the perimeter plans were estimated as a percentage of construction costs (refer to the Cost Estimating Technical Appendix D). The percentages used for the NCBB study were like the assumptions made for the NJBB study. Perimeter Plan analyses includes quantification of permanent easement acreages based upon the proposed structure footprint and interior drainage modifications including required maintenance access. It was also assumed that a temporary easement area would be required for access during construction. Real Estate limits were laid out in relation to the feature alignment location in AutoCAD Civil 3D. From this lay out, preliminary Real Estate acreage requirements, permanent and temporary easement limits, were computed for all the structures and at all AEP Plans and provided to Baltimore District Real Estate. Figure 7.1.1 shows an excerpt from AutoCAD of the Real Estate footprint layout in Freeport, NY. All calculations from this exercise have been attached in Exhibit H “Real Estate Footprint Quantities”.

Figure 7.1.1 – Real Estate Impact Layout in AutoCAD Civil 3D (Courtesy of USACE-NAP)

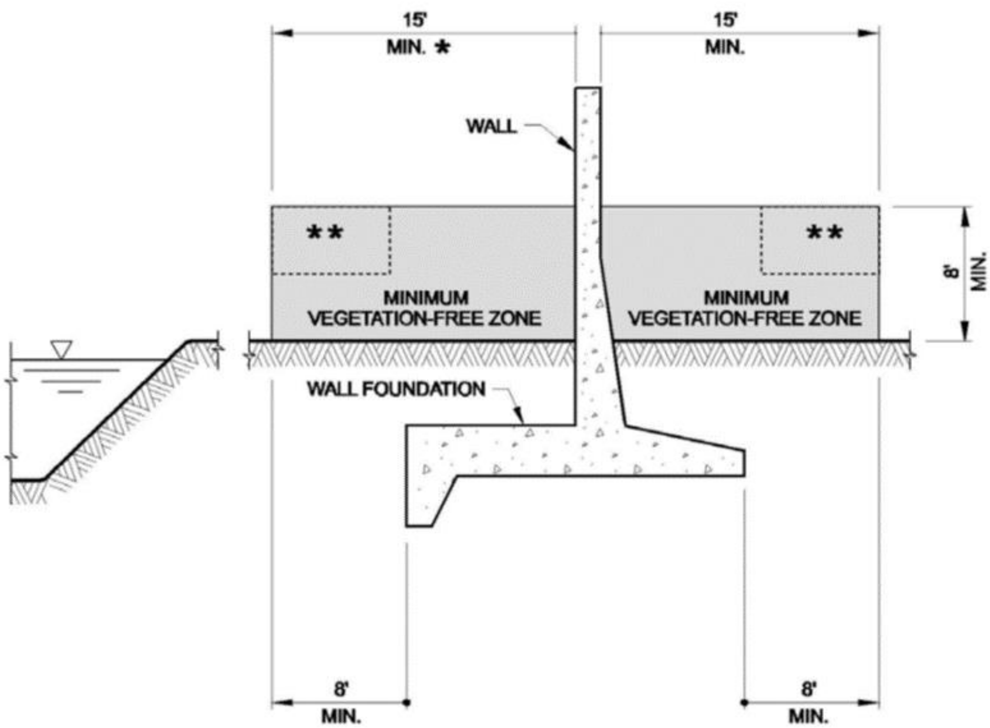
For the perimeter plan alignments, ETL 1110-2-571 Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures provides the minimum acceptable buffer between vegetation and flood damage reduction structures. The vegetation-free zone is a three-dimensional corridor surrounding any levee and floodwall and applies to all vegetation, except grass, which is permitted for erosion control purposes. The primary purpose of the vegetation-free zone is to provide access free of obstructions by personnel and equipment for surveillance, inspection, maintenance, monitoring, and flood-fighting. These limits provide the basis for the determination of permanent easement. The addition of temporary easement is essentially a place holder value and will be better developed in the post TSP design. Temporary easement area will vary greatly at different locations and dependent upon property ownership nuances such as right-of-way’s, deeds, riparian grants, etc. that can make property access difficult to obtain. Figure 7.1.2 shows the minimum allowable dimensions of vegetation-free zone for a levee and floodwall. Exhibit I “Real Estate Footprint Drawings” included in this appendix shows the preliminary limits of the permanent and temporary easements for each of the proposed perimeter flood risk management measures. Also included in the drawing set are preliminary real estate impact limits for the Miter Gate, Road/Rail Closure Gate and Sluice Gate structures that have been included within the study. The Norfolk CSRM Study was referenced when determining Real Estate impact limits for these structures.



* 15' OR DISTANCE TO EDGE OF NORMAL WATER SURFACE, IF LESS

** IN THIS 4' X 7' TRANSITION ZONE, TEMPORARY OBSTRUCTION BY LIMBS AND CROWN IS ALLOWED DURING DEVELOPMENT OF NEW PLANTINGS, FOR UP TO 10 YEARS

▽ NORMAL WATER SURFACE



* 15' OR DISTANCE TO EDGE OF NORMAL WATER SURFACE, IF LESS

** IN THIS 4' X 7' TRANSITION ZONE, TEMPORARY OBSTRUCTION BY LIMBS AND CROWN IS ALLOWED DURING DEVELOPMENT OF NEW PLANTINGS, FOR UP TO 10 YEARS

▽ NORMAL WATER SURFACE

Figure 7.1.2 – Vegetation Free Zone at Levee & Floodwall (ETL 1110-2-571)

Tables 7.1.1 and 7.1.2 contain the amount of easement area, both temporary and permanent required for all SBMs included within the structural alignments of this study.

Table 7.1.1 – Easement Footprints for Floodwalls & Levees

Floodwall/Levees		Temporary Easement		Permanent Easement	
Wall Type	Description	Land Side	Flood Side	Land Side	Flood Side
A	Levee	5	5	15	100
B	Floodwall	5	5	20	23
C	Floodwall	5	5	25	15
D	Floodwall	5	5	25	15
*All Distances reported in Feet (FT)					

Table 7.1.2 – Easement Footprints for Other SBMs

Other SBMs	Temporary Easement		Permanent Easement	
Description	Land Side	Flood Side	Land Side	Flood Side
Sluice Gate	0	0	15	15
Miter Gate	0	0	35	25
Road/Rail Closure Gate	5	5	15	15
*All Distances reported in Feet (FT)				

8 TENTATIVELY SELECTED PLAN (TSP)

The work completed by the NAP Civil Design Section during the screening analyses of the Structural and Non-Structural Alternatives aided Cost Engineering in the development of Alternative Plan Costs. See Appendix D for Cost Engineering results. Those costs were then utilized in a subsequent economic analysis conducted by the NAP Economics Section. The economic analysis completed evaluated the perimeter plan alternatives developed for the HVAs within Nassau County against the Non-Structural alternative in those same areas as Non-Structural was essentially selected for the remainder of the study area. The Non-Structural solution was also coupled with the intrinsic Critical Infrastructure Plans to create a third alternative for analysis. Therefore, the following action alternatives were evaluated for TSP by the PDT.

- Structural Plan in HVAs, Non-Structural throughout rest of Nassau County
- Non-Structural Plan throughout Nassau County
- Non-Structural Plan throughout Nassau County with localized Critical Infrastructure Plans

An overview of the Economic Results for the three plan approaches can be found in Figure 8.1.1 below. All benefits, costs and risks were evaluated considering an Intermediate RSLC. See Appendix F for further explanation on the analyses conducted for alternative screening.

PROJECT	Nassau County Back Bay CSRM Feasibility Study
LOCATION	**FREEPORT + LONG BEACH ISLAND + ISLAND PARK + EAST ROCKAWAY**
PREPARED	14-May-21
Estimated Price Level	1-Oct-20
SLC Rate	Intermediate

WBS STRUCTURE		**ESTIMATED COST in Thousands (1 OCT 2020)**			
WBS NUMBER	Civil Works Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	**TOTAL ($K)**
A	B	C	D	E	F
100YR	VILLAGE OF FREEPORT 100YR	$1,373,107	$647,735		**$2,020,842**
100YR	CITY OF LONG BEACH ISLAND 100YR	$1,066,515	$487,337		**$1,553,852**
100YR	ISLAND PARK 100YR	$1,324,617	$575,408		**$1,900,025**
100YR	EAST ROCKAWAY 100YR	$1,878,075	$874,292		**$2,752,367**
CI 100YR	VILLAGE OF FREEPORT CI 100YR/20YR	$294,218	$131,533		**$425,752**
CI 100YR	CITY OF LONG BEACH ISLAND CI 100YR/20YR	$208,795	$95,991		**$304,786**
CI 100YR	ISLAND PARK CI 100YR/20YR	$172,396	$81,515		**$253,911**
CI 100YR	CEDAR CREEK WASTEWATER				
CI 100YR	BAY PARK RECLAMATION FACILITY				
2.50%	Federal Discount Rate				
50	Period of Analysis				
0.03526	Capital Recovery Factor				

Inventory - 2030 FWOP - All structures and vehicles with FFE below 50% AEP elevated above 1% AEP floodplain
Nonstructural costs not yet updated
Critical infrastructure depreciated replacement values not yet updated

Critical infrastructure secondary benefits not quantified

Nonstructural - 2080 Intermediate - Residential structures with FFE below 5% AEP elevated above 1% AEP floodplain
Nonstructural - 2080 Intermediate - Non-residential structures with FFE below 5% AEP floodproofed
Nonstructural - 2080 Intermediate - Critical Infrastructure with FFE below 1% AEP floodproofed

Structural HEC-FDA in Thousands								
Group	**EAD Reduced**	**Init. Const.**	**IDC**	**Annual OMRR&R**	**AAC**	**BCR**	**AANB**	**Residual Risk**
1	$145,596	$2,020,842	$128,681	$10,104	$85,892	1.7	$59,704	7.6%
2	$104,605	$1,553,852	$98,944	$7,769	$66,044	1.6	$38,561	14.1%
3	$82,226	$1,900,025	$120,987	$9,500	$80,757	1.0	$1,469	8.0%
4	$175,426	$2,752,367	$175,262	$13,762	$116,984	1.5	$58,442	10.5%
1	$37,507	$425,752	$27,110	$2,129	$18,096	2.1	$19,411	7.2%
2	$1,465	$304,786	$19,408	$1,524	$12,954	0.1	-$11,490	7.8%
3	$5,918	$253,911	$16,168	$1,270	$10,792	0.5	-$4,874	12.4%

Nonstructural HEC-FDA in Thousands								
Group	**EAD Reduced**	**Init. Const.**	**IDC**	**Annual OMRR&R**	**AAC**	**BCR**	**AANB**	**Residual Risk**
1	$126,436	$572,512	$1,770	$0	$20,248	6.2	$106,188	19.8%
2	$65,631	$605,962	$1,873	$0	$21,431	3.1	$44,200	46.1%
3	$64,176	$387,209	$1,197	$0	$13,694	4.7	$50,481	28.2%
4	$121,840	$709,754	$2,194	$0	$25,102	4.9	$96,738	37.9%
ROC	$232,488	$1,562,392	$4,830	$0	$55,257	4.2	$177,231	48.0%

Nonstructural with CRIT Structural HEC-FDA in Thousands								
Group	**EAD Reduced**	**Init. Const.**	**IDC**	**Annual OMRR&R**	**AAC**	**BCR**	**AANB**	**Residual Risk**
1	$132,191	$967,350	$28,785	$2,129	$37,251	3.5	$94,941	16.1%
2	$66,278	$909,292	$21,277	$1,524	$34,334	1.9	$31,944	45.6%
3	$70,091	$640,586	$17,364	$1,270	$24,468	2.9	$45,624	21.6%
4	$121,843	$709,754	$2,194	$0	$25,102	4.9	$96,741	37.9%
ROC	$232,488	$1,562,392	$4,830	$0	$55,257	4.2	$177,231	48.0%

Figure 8.1.1 – Economic Results for All Potential TSP Plans

From this evaluation two (2) plans showed the most promise. These plans included:

- Structural Plan in Long Beach Only, Non-Structural throughout rest of Nassau County
- Non-Structural throughout rest of Nassau County

Both potential plans were articulated to the Non-Federal Sponsor over the course of several public meetings and presentations prior to final analyses and selection. Table 8.1.1 contains the identified National Economic Development (NED) Plan of the Non-Structural Alternative highlighted in green.

Table 8.1.1 – Economic Selection of TSP

	Jan 5th Memo Results - HEC-FDA in Thousands - Intermediate SLC								
Jan 5th Memo	**Plan**	**EAD Reduced**	**Init. Const.**	**IDC**	**Annual OMRR&R**	**AAC**	**BCR**	**AANB**	**Residual Risk**
1	No Action	$0	$0	$0	$0	$0	1.0	$0	100.0%
2	Long Beach Struc Plan	$649,545	$4,785,719	$108,935	$7,769	$180,345	3.6	$469,200	35.8%
3	Total Benefits	$622,893	$4,789,373	$74,449	$4,922	$176,411	3.5	$446,481	38.4%
4	Nonstructural (NED)	$610,571	$3,837,829	$11,864	$0	$135,733	4.5	$474,839	39.7%
5	LPP	-	-	-	-	-	-	-	-

Results

The selection of the Non-Structural Plan throughout the entirety of the Nassau County Back Bay Study Area effectively screens out a Structural Plan in the form of a contiguous floodwall or levee for protection from inundation. This also screens out localized Critical Infrastructure Plans. Critical Infrastructure Plans were most likely screened out due to the abstract nature of trying to capture benefits associated with ability to maintain the integrity of a critical facility for the design inundation event. NAP Civil Design will move forward with the observations and recommendations made in the Non-Structural analysis of this report to further expound on the design and quantities for Structure Elevations, Dry-Floodproofing and Adaptable Flood Risk Measures that can provide risk management to the vulnerable communities on the back bays of Nassau County. The PDT has not completely ruled out the use of Critical Infrastructure Plans post-TSP as their economic benefits were difficult to discern during the TSP selection process.

Therefore, the team will most likely explore Critical Infrastructure Plans further in the Feasibility Design once more robust economic information is available. Refined Critical Infrastructure Plans could increase Net Benefits to include them into the Non-Structural Plan as additional scope or as a replacement to areas otherwise designated for a Non-Structural solution. NNBF solutions will also be continually evaluated post-TSP to further enhance the project risk management and potentially satisfy any mitigation requirements deemed necessary during environmental review. An example of a plan that includes Non-Structural, Critical Infrastructure and NNBF risk management is shown in Figure 8.2.1 at the Cedar Creek Sewer Treatment Facility in Wantagh, NY.

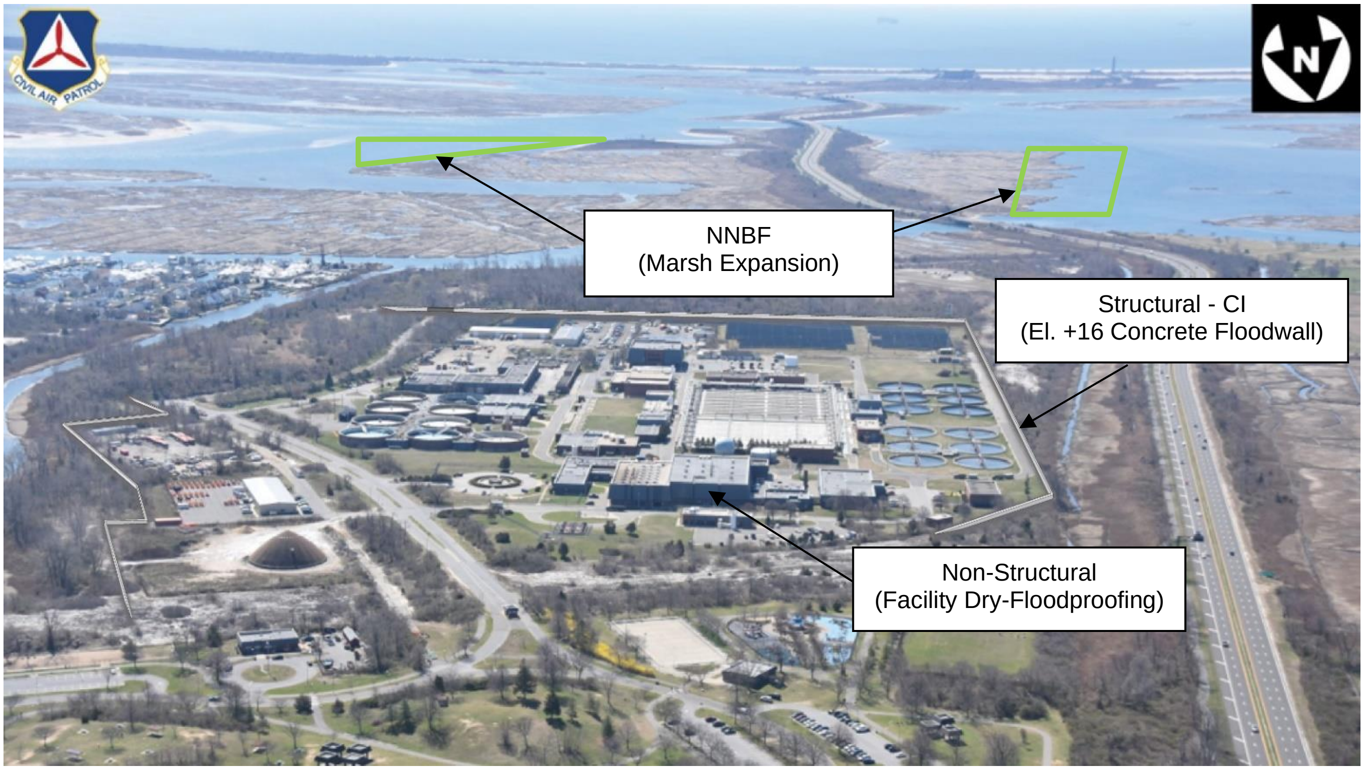


Figure 8.1.2 – Wantagh Treatment Plant Conceptual Critical Infrastructure Plan
(Courtesy of Civil Air Patrol/USACE-NAP)

9 REFERENCES

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10 DESIGN GUIDANCE

Below is a list of United States Army Corps of Engineering Design Guidance referenced in the development of the Nassau County Back Bay Structural & Non-Structural Plan Formulation:

1. United States Army Corps of Engineers (USACE). HSDRRSDG Hurricane and Storm Damage Risk Reduction Design Guidelines with June 2012 updates.

2. USACE. Engineer Manual (EM) 1110-2-1614 Design of Coastal Revetments, Seawalls and Bulkheads.

3. USACE. Engineer Manual (EM) 1110-2-2100 Stability Analysis of Concrete Structures

4. USACE. Engineer Manual (EM) 1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures.

5. USACE. Engineer Manual (EM) 1110-2-2504 Design of Sheet Pile Walls.

6. USACE. Engineer Manual (EM) 1110-2-2602 Planning and Design of Navigation Locks

7. USACE. Engineer Manual (EM) 1110-2-2906 Design of Pile Foundations.

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

9. USACE. Engineer Engineering Technical Letter (ETL) 1110-2-58 Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, And Appurtenant Structures.

10. USACE. Engineer Engineering Technical Letter (ETL) 1110-2-2105 Design of Hydraulic Steel Structures.

11. USACE. Executive Order (EO) 11988 Flood Risk Management.

11 EXHIBITS

A. CENAP-EC-EC Structural Map Deck – 29 Pages

B. Floodwall/Levee Quantities – 10 Pages

C. NCBB CSRM CAD Drawing Set – 48 Pages

D. AISAP Data & Results – 10 Pages

E. Civil Site Visit Summary Report – 17 Pages

F. Elevation Concept Drawings – 6 Pages

G. CENAP-EC-EC Non-Structural Map Deck – 17 Pages

H. Real Estate Quantities – 7 Pages

I. Real Estate Footprint Drawings – 7 Pages