ENGINEERING APPENDIX
CIVIL

NEW JERSEY BACK BAYS
COASTAL STORM RISK MANAGEMENT
FEASIBILITY STUDY

PHILADELPHIA, PENNSYLVANIA

APPENDIX B.1

August 2021

NJBB
NEW JERSEY BACK BAYS STUDY

U.S. Army Corps of Engineers
Philadelphia District

THIS PAGE HAS BEEN INTENTIONALLY LEFT BLANK

Table of Contents

List of Figures

List of Tables

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 New Jersey Back Bays (NJBB) area was identified as a “focus area” within the NACCS study. This Civil Engineering Section discusses the engineering and design work conducted to layout and evaluate potential structural, non-structural and natural & nature-based (NNBF) solutions for protection against flooding in the New Jersey Back Bays Region. Two structural flood control solution types were evaluated: perimeter plans (line of protection placed at or near the shoreline or limit of development) and storm surge barriers (a system of barriers comprised of inlet and bay closures to prevent flood surge from entering the back bay(s)). Both solutions were evaluated separately for initial screening analyses, but components of each were combined to determine a focused array of alternatives that was further evaluated to determine the Tentatively Selected Plan (TSP).

The NACCS Tier 1 Screening provided pre-compiled reference data for initial screening of alternatives. Designs from other USACE District studies were also analyzed for suitability of incorporating these features as measures in this study. Parametric data from each were utilized for determination of with-project costs.

2 PERIMETER PLAN ANALYSES

2.1 Perimeter Plan Cycle 1 Screening

The entire back bays perimeter area was divided into economic reaches by county and municipality. Reaches were then combined into groups based upon geographical conditions (municipalities on a barrier island, etc.) or hydraulic connectivity (small island off the barrier) resulting in 50 groups. 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 C: Economics). A preliminary line of protection was laid out for each group (completed also in Google Earth) along the bay frontage of the polygon or at other suitable locations. The FEMA 500 year flood mapping was used to determine where to terminate the line of protection at existing high ground. Perimeter plan alignments were assumed to tie-in to dunes or seawalls of existing USACE projects on the ocean side of the barrier islands. This preliminary layout did not consider the best horizontal placement of the line but did approximate the existing shoreline or exposed perimeter. The linear foot length of the line of protection for each group is shown in Table 2-1 below. The Perimeter Plan Screening Analysis drawings, provided in the Drawings Annex, are labeled with the approximate location of each reach.

Table 2-1: Cycle 1 Reaches & Quantities of Floodwalls, Miter Gates and Road Closure Structure by Group

Group	County	Reaches	Floodwall (ft)	Miter Gates (ea)	Sluice Gates (ea)	Road Closures (ea)
1	Cape May	CM1	15,757	-	-	1
2	Cape May	LW1, WCR1, WCY1, NW1	54,070	1	-	9
3	Cape May	LW2	13,194	-	-	2
4	Cape May	WW1	11,727	-	-	1
5	Cape May	SH1, AV3	81,897	2	-	7
6	Cape May	MT1	7,948	-	-	2
7	Cape May	MT2	13,817	1	-	2
8	Cape May	AV2	5,465	-	-	1
9	Cape May	AV1	9,574	-	-	1
10	Cape May	SI1	34,954	2	-	4
11	Cape May	UP1	8,165	-	-	3
12	Cape May	OC1	78,573	3	-	4
13	Cape May	UP2	12,896	-	-	1
14	Atlantic	EG1	3,552	-	-	1
15	Atlantic	SP1	16,441	-	-	3
16	Atlantic	EG2	7,811	-	-	3
17	Atlantic	EG3	7,328	2	-	2
18	Atlantic	LP1, MG1, VN2, AC2	87,474	6	-	10
19	Atlantic	VN1	20,044	-	-	2
20	Atlantic	AC1	14,735	-	-	6
21	Atlantic	EG4	31,233	-	-	4
22	Atlantic	AB1	11,028	1	-	1
47	Atlantic	AB2	14,334	-	-	1
23	Atlantic	BC1	48,590	1	-	5
24	Ocean	LH1	68,775	5	-	-
25	Ocean	LH2, TK1	40,947	4	-	2
26	Ocean	LB5, BV1, LB4, SB1, SC1, LB3, HC1, LB1, BGL1	188,205	9	-	11
27	Ocean	SF1	49,526	5	3	3
28	Ocean	LB2	18,356	1	-	1
29	Ocean	BG1, OT1	26,287	3	-	-
30	Ocean	OT2	11,992	1	-	-
31	Ocean	OT3, OT4	16,238	5	-	-
32	Ocean	OT5	21,429	-	-	1
33	Ocean	LC1	28,330	3	2	1
34	Ocean	LC2	31,585	3	1	-
35	Ocean	LC3, BK1	74,450	8	-	2
36	Ocean	BK2	31,469	3	-	3
37	Ocean	BK3	22,715	2	1	4
38	Ocean	BK5, OG1, BK6, OG2	40,199	1	2	3
39	Ocean	IH1, TR2	59,492	9	-	3
40	Ocean	TR6	69,762	9	5	1
41	Ocean	BR2	91,679	9	4	-
42	Ocean	BK4, SSP1, SSH1, TR4, LL2, LL1, TR5, BR1, MK1, BH1, PPB1, PP2	178,744	16	-	6
48	Ocean	TR3, BK7	7,396	-	-	2
43	Ocean	BR3	37,716	1	1	1
51	Ocean	PP1, BR4	41,562	9	-	-
45	Monmouth	MQ1, BL1	22,642	3	-	2
46	Monmouth	BM1	14,028	1	1	-
50	Monmouth	ABS1	5,423	-	-	-

As an initial screening measure the NACCS Tier 1 floodwall was assumed for the line of protection to generate 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 foot 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, although these are not shown in the graphic (See Figure 2-1 below). The linear foot parametric cost of the wall includes drainage gates/outlet structures every 400 feet along the length of the floodwall. Additional structures (miter gates, sluice gates, and road closure structures) necessary to complete the continuous line of protection were also included to determine with-project quantities. Miter gates, 65 feet wide, were used to close off navigable canals or channels. Sluice gates, 60 feet wide, were used to maintain flow in areas where the floodwall will cut off flow to a small stream, tidal wetland or marsh, and where navigation is not required. Road closure structures (roller gate type) were used to close the line of protection during flooding events while allowing use of the roadway or municipal boat ramp during non-flood conditions. One road closure will accommodate two lanes of standard traffic; two road closures were used at locations with four lanes of traffic. The Norfolk District CSRM project provided preliminary designs for these structures and they are shown in the NJBB CSRM Perimeter Plan Screening Analysis drawings provided in the Drawings Annex.

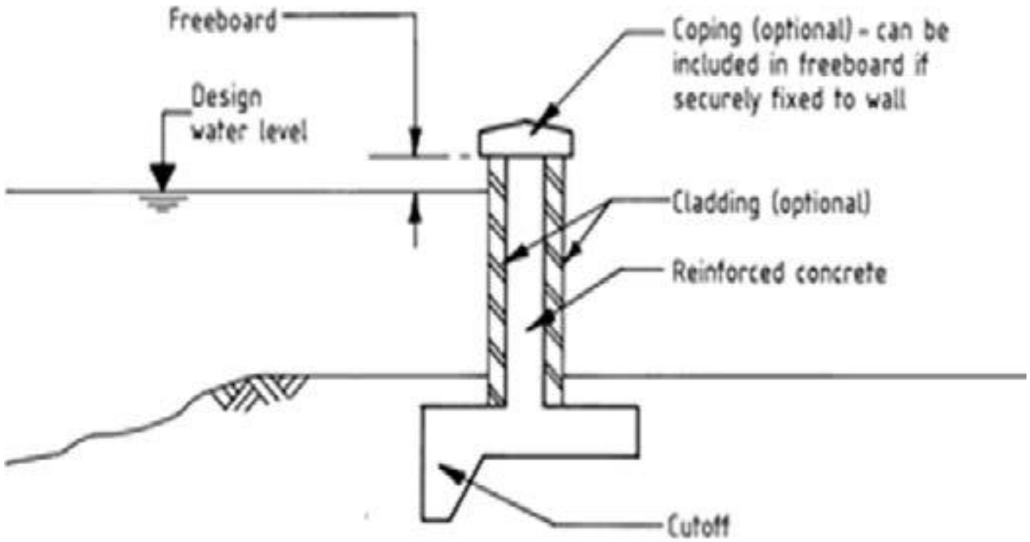


Figure 2-1: Representative NACCS Floodwall Cross Section (T-Wall)

Benefit-Cost Ratio results for the Cycle 1 Screening of potential Perimeter Plan alternative locations resulted in 12 Groups considered "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 screening (Cycle 2) was applied to the 12 groups that received a “favorable” status.

2.2 Perimeter Plan Cycle 2 Screening

A more detailed evaluation of the proposed preliminary line of protection was ultimately completed for a total of 13 groups for Perimeter Plan Cycle 2 Screening. The 13 groups included the 12 groups that advanced from the Perimeter Plan Cycle 1 Screening analysis (with some changes) and one additional group added to the analysis that had been overlooked in the original screening. Previous group compositions were revised to reorganize reaches for economic evaluation purposes, or to combine reaches differently due to hydraulic or structural reasons. The Perimeter Plan Cycle 2 Screening process applied to the 13 groups included refinement of the location of the line of protection, selection of a proposed structure type based upon preliminary consideration of existing conditions where it was to be placed, and computation of quantities based upon the updated layout and typical flood protection sections. Google Earth with elevation tools, the FEMA 500 Year Flood Plain Mapping, and NOAA Navigation Charts as an underlay were used to determine approximate nearshore conditions.

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 protection levee and floodwall sections were generated for the Perimeter Plan Cycle 2 Screening analysis based on these general conditions assumed along the proposed line of protection. The design height of the protection (elevation in feet NAVD88) was computed using still water elevation (SWEL) with required freeboard and anticipated relative sea level change (RSLC) in order 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. Approximate maximum required crest elevations are 13 feet NAVD88 everywhere except within Barnegat Bay, where the crest elevations are closer to 10 feet NAVD88. (See HH&C Appendix B.1 for design height calculations). For this level of screening the quantities assumed a maximum wall or levee top elevation of 13 feet NAVD88 for all locations. The three typical sections used in this analysis were a levee section (Type A), a floodwall section to be constructed in areas below water level (Type B), and a floodwall section to be constructed in areas above the mean tide zone (Type C). Typical Sections of each type are shown in Figure 2-2 through Figure 2-4.

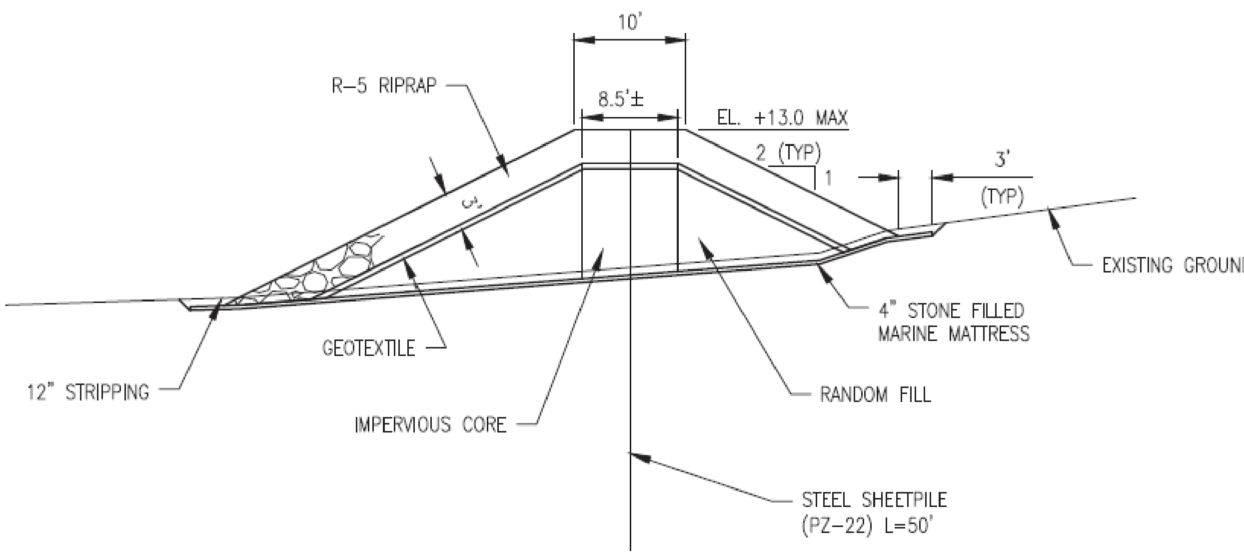


Figure 2-2: Cycle 2 Typical Section - Levee - Type A

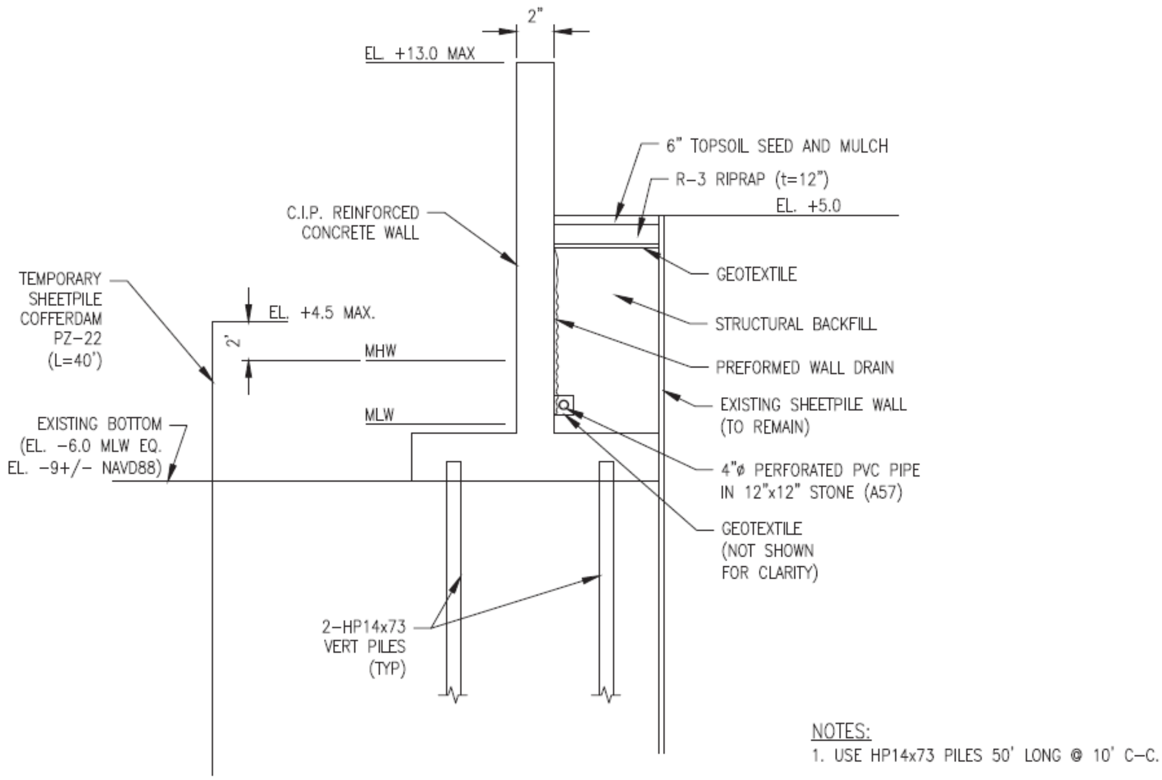


Figure 2-3: Cycle 2 Typical Section - Concrete Cantilever Wall on Piles - Type B

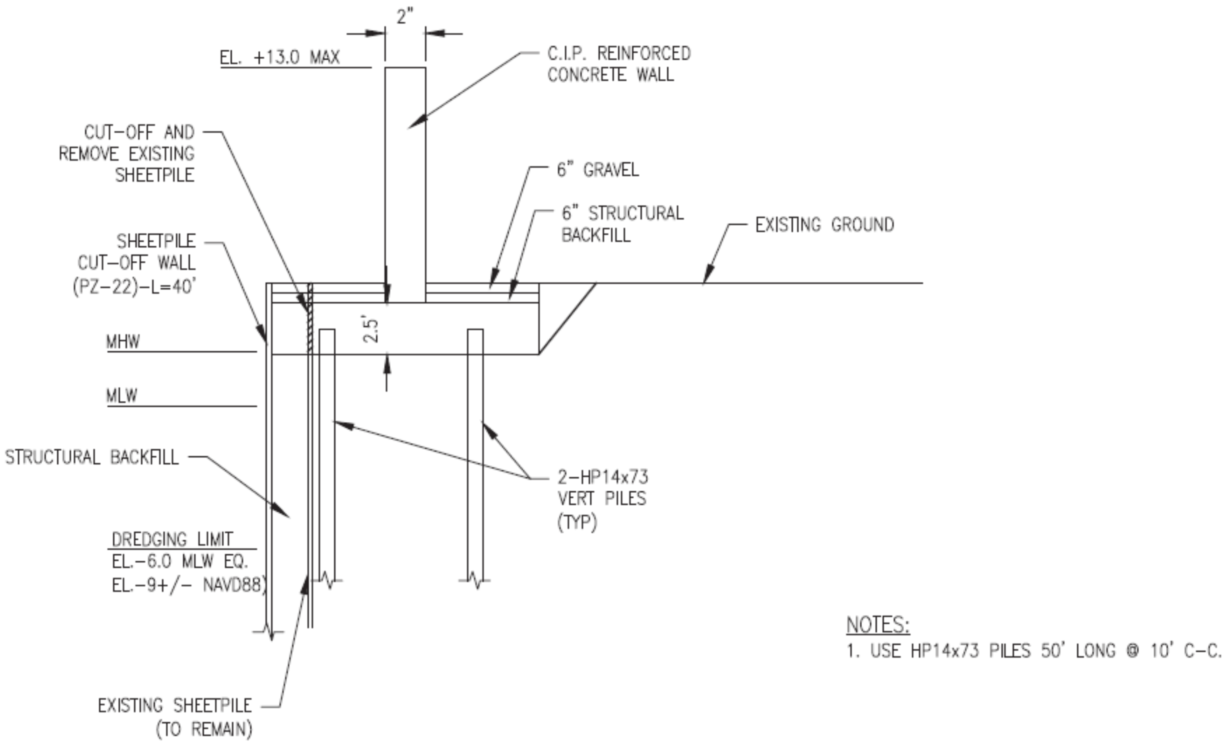


Figure 2-4: Cycle 2 Typical Section - Concrete Cantilever Wall - Type C

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 was 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). 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 sheetpile wall to provide impermeability of the structure, and for cut-off protection against underseepage. 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.

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. 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 will extend to a depth of approximately -9 feet NAVD88, which is the expected maximum dredging depth for the New Jersey Intracoastal Waterway (NJICWW). A temporary cofferdam is required for construction of the wall which will be completed using water-based methods. 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 sheetpile 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.

Floodwall placement in the vicinity of finger canals and other waterfront communities that included alternating lanes of bulkheaded waterway with developed or residential property was considered from an economic point of view. Perimeter floodwall placement would need to follow the existing bulkhead alignment, resulting in long linear foot lengths of structure and, thus, substantial with-project costs for these areas. A miter gate, therefore, was used across the opening of a waterway lane if it would eliminate 3000 feet or more of floodwall. This limit was determined by dividing the cost of a typical miter gate by the linear foot cost of floodwall. The linear foot lengths of the line of protection and number of gates and road closures needed for each group are shown in Table 2-2 below. The Norfolk District CSRM project again provided preliminary designs for these structures for the Cycle 2 level of screening, and they are shown in NJBB CSRM Perimeter Plan Screening Analysis drawings provided in the Drawings Annex.

Table 2-2: Cycle 2 Reaches & Quantities of Floodwalls, Miter Gates and Road Closure Structure by Group

Group	Group Name	County	Cycle1 Polyline Names	Floodwalls (ft)	Type A (feet)	Type B (feet)	Type C (feet)	Miter Gates (ea	Sluice Gates (ea	Road Closures (ea)
1	Cape May	Cape May	CM1	15,825	5,305	7,307	3,213	0	0	1
2	Wildwood Island	Cape May	LW1, NW1, WCR1, WCY1	54,171	24,296	26,618	3,257	1	0	9
4	West Wildwood	Cape May	WW1	11,726	3,728	7,998	-	0	0	1
5	Seven Mile Island	Cape May	AV1, AV2, AV3, SH1	97,225	8,446	85,428	3,350	2	0	9
10	Sea Isle City	Cape May	SI1	35,166	14,406	18,359	2,400	2	0	4
11	Strathmere	Cape May	UP1	8,187	1,048	3,304	3,835	0	0	3
12	Ocean City	Cape May	OC1	78,732	24,080	35,432	19,220	3	0	4
18	Absecon Island	Atlantic	AC1, AC2, LP1, MG1, VN1, VN2	111,112	11,398	70,041	29,672	8	0	11
23	Brigantine Island	Atlantic	BC1	48,699	593	36,743	11,363	1	0	5
26	Long Beach Island	Ocean	BGL1, BV1, HC1, LB1, LB2, LB3, LB4, LB5, SB1, SC1	209,124	18,201	164,947	25,975	10	0	10
42	Barnegat Bay Island	Ocean	BH1, BK4, BK7, BR1, LL1, LL2, MK1, PP2, PPB1, SSH1, SSP1, TR3, TR4, TR5	186,871	15,398	160,276	11,197	16	0	8
45	Manasquan	Monmouth	BL1, MQ1	22,820	10,741	9,328	2,751	3	0	2
52	West Cape May	Cape May	GP52	4,480	3,449	-	1,031	1	0	2
				884,138	**141,089**	**625,781**	**117,264**	**47**	**-**	**69**
				Wall Usage:	*16.0%*	*70.8%*	*13.3%*			

2.2.1 Focused Array of Alternatives

Cost estimates were generated for the following potential CSRM solutions: perimeter plan, storm surge barrier inlet or bay closures, and non-structural (NS) measures (see Nonstructural Analyses Appendix D). Each potential strategy was evaluated economically in isolation, then potential CSRM solutions were combined into multi-strategy alternatives. The following tables show 51 potential single and multi-strategy alternatives, though not all alternatives are considered complete or environmentally acceptable.

The 51 alternatives were separated into 5 regional groups that were each assigned a number to describe their location: (1) Entire Study Area, (2) Shark River, (3) Area between Manasquan Inlet and Little Egg Inlet; referred to as “North Region”, (4) Area south of Little Egg Inlet and north of Corson Inlet, referred to as “Central Region”, and (5) Areas south of Corson Inlet, referred to as “South Region”. Within each region, the alternatives were assigned a letter to describe the strategies implemented: (A) nonstructural strategy only, (B) perimeter strategy only (including locations that passed cycle 1 and cycle 2 analyses), (C) perimeter only in locations that passed cycle 2, (D) perimeter in locations that passed cycle 2 with nonstructural (plus permutations for perimeter locations that passed cycle 1), (E) storm surge barriers with nonstructural and/or perimeter, (F) storm surge barriers with nonstructural and/or perimeter and interior bay closures, and finally (G) storm surge barriers with nonstructural and/or perimeter and a different combination of interior bay closures. Individual maps for each of these alternative plans can be found in the Economics Appendix C.

Table 2-3: Comprehensive List of 51 Regional Alternatives

REGION	ALTERNATIVES	DESCRIPTION
STUDY WIDE	1A	Nonstructural ONLY
	1B	Perimeter (justified) ONLY
	1C	Storm Surge Barrier ALL INLETS
	1D	Storm Surge Barrier ALL INLETS minus Little Egg Harbor Inlet
SHARK RIVER	2A	Nonstructural ONLY
	2B	Perimeter ONLY
	2C	Storm Surge Barrier ONLY

NORTH REGION	3A	Nonstructural ONLY
	3B	Perimeter ONLY
	3C	Perimeter (Cycle 2) ONLY
	3D	Perimeter (Cycle 2) + Nonstructural
	3E(1)	Storm Surge Barrier ONLY
	3E(2)	Storm Surge Barrier + Nonstructural
	3E(3)	Storm Surge Barrier + Nonstructural + Perimeter
	3F(1)	Storm Surge Barrier + Bay Closure (Holgate)
	3F(2)	Storm Surge Barrier + Bay Closure (Holgate) + Nonstructural
	3G	Storm Surge Barrier + Bay Closure (Point Pleasant Canal)
CENTRAL REGION	4A	Nonstructural ONLY
	4B	Perimeter ONLY
	4C	Perimeter (Cycle 2) ONLY
	4D(1)	Perimeter (Cycle 2) + Nonstructural
	4D(2)	Perimeter (Cycle 1 and 2) + Nonstructural
	4E(1)	Storm Surge Barrier ONLY
	4E(2)	Storm Surge Barrier + Nonstructural
	4E(3)	Storm Surge Barrier + Nonstructural + South Ocean City Perimeter
	4E(4)	Storm Surge Barrier + Nonstructural + South Ocean City Bay Closure
	4F(1)	Storm Surge Barrier + Bay Closure (North Point)
	4F(2)	Storm Surge Barrier + Bay Closure (North Point) + Nonstructural
	4F(3)	Storm Surge Barrier + Bay Closure (North Point) + Nonstructural + South Ocean City Perimeter
	4F(4)	Storm Surge Barrier + Bay Closure (North Point) + Nonstructural + South Ocean City Bay Closure
	4G(1)	Storm Surge Barrier + Bay Closure (Absecon Blvd)
	4G(2)	Storm Surge Barrier + Bay Closure (Absecon Blvd) + Nonstructural
	4G(3)	Storm Surge Barrier + Bay Closure (Absecon Blvd) + Nonstructural + South Ocean City Perimeter
	4G(4)	Storm Surge Barrier + Bay Closure (Absecon Blvd) + Nonstructural + South Ocean City Bay Closure
	4G(5)	Storm Surge Barrier + Bay Closure (Absecon Blvd) + NS Brigantine + South Ocean City No-Action
	4G(6)	Storm Surge Barrier + Bay Closure (Absecon Blvd) + NS Brigantine + South Ocean City Nonstructural
	4G(7)	Storm Surge Barrier + Bay Closure (Absecon Blvd) + NS Brigantine + South Ocean City Perimeter
	4G(8)	Storm Surge Barrier + Bay Closure (Absecon Blvd) + NS Brigantine + South Ocean City Bay Closure
	4G(9)	Storm Surge Barrier + Bay Closure (Absecon Blvd) + PM Brigantine + South Ocean City No-Action
	4G(10)	Storm Surge Barrier + Bay Closure (Absecon Blvd) + PM Brigantine + South Ocean City Nonstructural
	4G(11)	Storm Surge Barrier + Bay Closure (Absecon Blvd) + PM Brigantine + South Ocean City Perimeter
	4G(12)	Storm Surge Barrier + Bay Closure (Absecon Blvd) + PM Brigantine + South Ocean City Bay Closure
SOUTH REGION	5A	Nonstructural ONLY
	5B	Perimeter ONLY
	5C	Perimeter (Cycle 2) ONLY
	5D(1)	Perimeter (Cycle 2) + Nonstructural

	5D(2)	Perimeter (Cycle 1 and 2) + Nonstructural
	5E(1)	Storm Surge Barrier ONLY
	5E(2)	Storm Surge Barrier + Nonstructural
	5F	Storm Surge Barrier + Nonstructural + Bay Closure (Sea Isle Blvd)
	5G	Storm Surge Barrier + Nonstructural + Bay Closure (Sea Isle Blvd, Wildwood Blvd, Stone Harbor Blvd)

*NS = Nonstructural, PM = Perimeter Measure

After developing the initial array of 51 alternatives across the 5 regional groups in the study area, the PDT narrowed the array down to the alternatives that had the highest benefits – a preliminary focused array of 20 alternatives as shown in the table below.

Table 2-4: Preliminary Focused Array of Alternative Plans

Region	Themes	Alternative	NONSTRUC	PERIMETER	INLET SSB	BAY CLOSURE
SHARK RIVER	2A	2A	X			
NORTH	3A	3A	X			
	3D	3D	X	X		
	3E	3E(2)	X		X	
		3E(3)	X	X	X	
CENTRAL	4A	4A	X			
	4D	4D(1)	X	X		
		4D(2)	X	X		
	4E	4E(2)	X		X	
		4E(3)	X	X	X	
		4E(4)	X		X	X
	4G	4G(6)	X		X	X
		4G(7)	X	X	X	X
		4G(8)	X		X	X
		4G(10)	X	X	X	X
		4G(11)	X	X	X	X
		4G(12)	X	X	X	X
SOUTH	5A	5A	X			
	5D	5D(1)	X	X		
		5D(2)	X	X		

2.3 Perimeter Plan Cycle 3 Screening

The Cycle 3 analysis for the Perimeter Plan consisted of the following tasks:

- New Wall Type D added (1 Levee, 3 Wall Types evaluated)
- All structures (levee and walls) evaluated for increased water levels from Cycle 2

- Preliminary Geotechnical and Structural analysis to verify design
- Revised Quantities
- Real Estate (Permanent and Temporary Easement) Acreage Estimates determined

Water levels for the 1% AEP were updated and wave overtopping was reanalyzed for this cycle, and it was determined that the approximate maximum required crest elevation for the flood protection structures is 16 feet NAVD88 (See HH&C Appendix B.1 for design height calculations). The previous Cycle 2 analysis assumed a maximum required crest/wall top elevation of 13 feet NAVD88.

A new Type D wall was introduced into the screening analysis. 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 greater water depths than previously assumed potentially exist. These locations are in narrow finger canals or adjacent to back bay channels that are close to the existing bulkhead line, respectively. Some of the back bay channels that hug the existing bulkhead line potentially exhibit water levels deeper than the NJIWW authorized depth of -6 feet MLW as determined by a review of NOAA Navigation Charts that were used to layout the proposed locations of the Type D wall.

The Cycle 3 preliminary design evaluation of the walls was completed using available geotechnical data for a stability analysis with proposed conditions to update the typical sections. Results of the analysis determined that the piles required to support Wall Type B increased to (2) 62-foot long HP14x73 piles spaced at 5 feet longitudinally, and piles required to support Wall Type C increased to (2) 57-foot long HP14x73 piles spaced at 8 feet longitudinally. No wall dimension changes were necessary for the Type B and Type C walls except for the increases to stem height. Wall Type D is a steel king pile and sheet pile combined wall system - king pile/sheet pile floodwall for short. The wall is comprised of W40X277 steel king piles at a length of 96 feet, interspaced by PZC18 sheet piling at a length of 50 feet. 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 overwash. (See Geotechnical and Structural Appendices for the Cycle 3 analyses).

Revised Cycle 3 Quantities were generated for all 4 structure types utilizing the updated typical sections (See Figure 2-8 and quantity estimations pages below). The NJBB CSRM Perimeter Plan Screening Analysis Cycle 3 drawings are provided in the Drawings Annex. The plan view layouts of the proposed structures are shown using a color key, and Sheet C-200 shows all 4 typical sections.

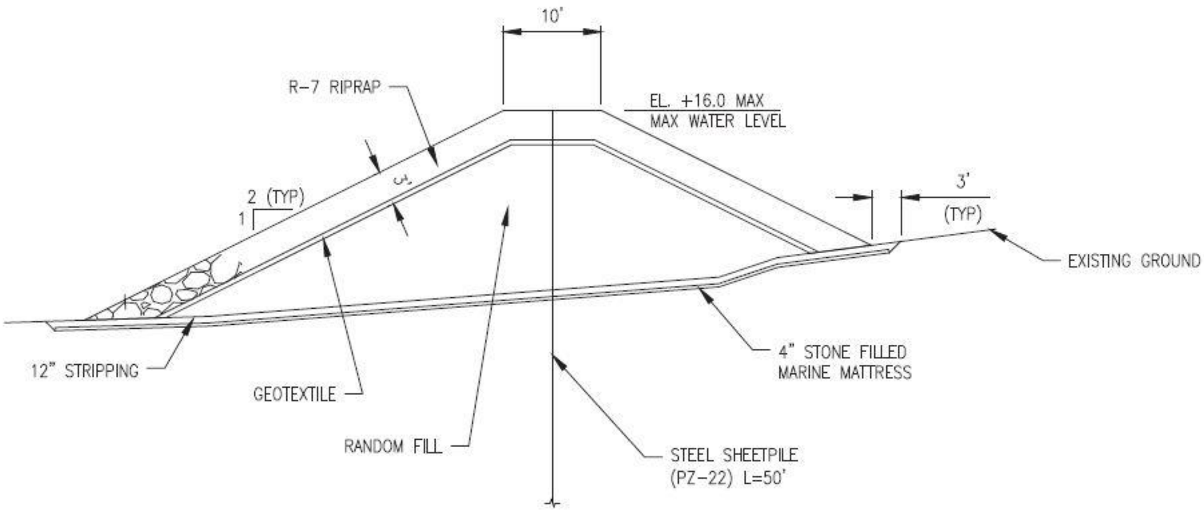


Figure 2-5: Cycle 3 Typical Section - Levee - Type A

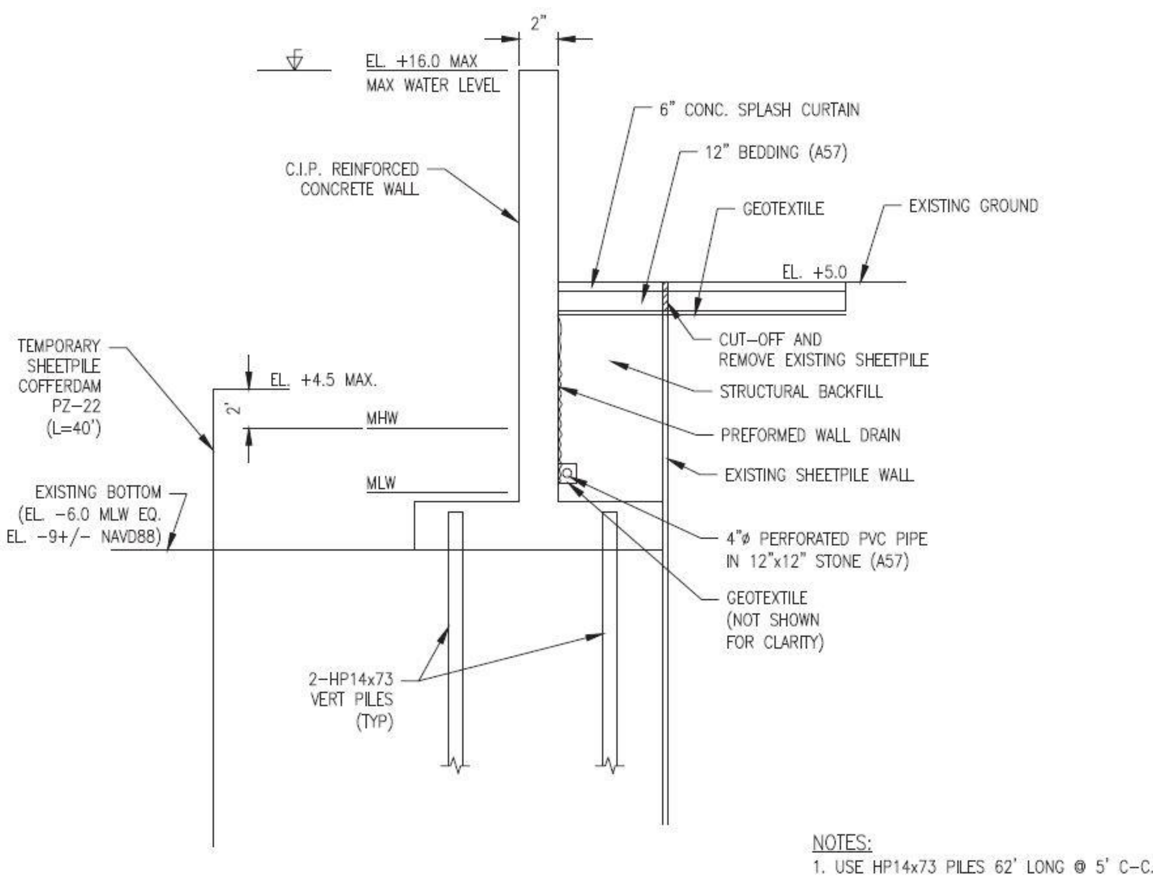


Figure 2-6: Cycle 3 Typical Section - Concrete Cantilever Wall on Piles - Type B

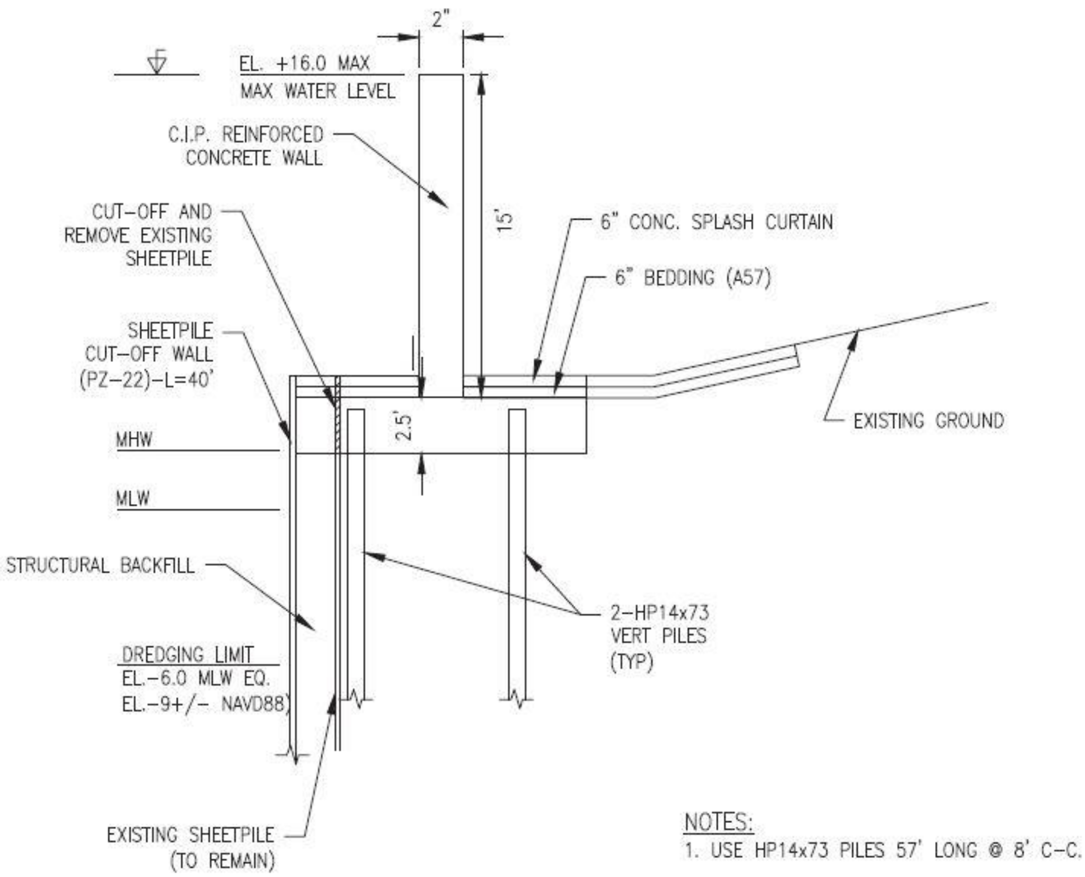


Figure 2-7: Cycle 3 Typical Section - Concrete Cantilever Wall - Type C

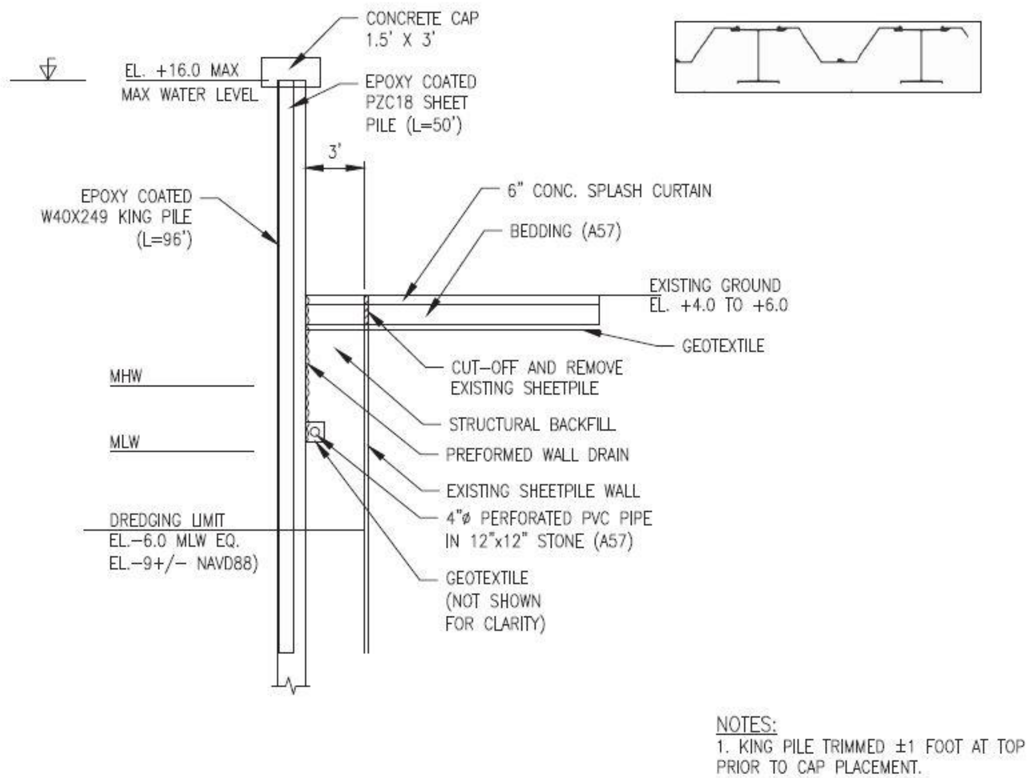


Figure 2-8: Cycle 3 Typical Section - Steel King Pile and Sheet Pile Combination Wall

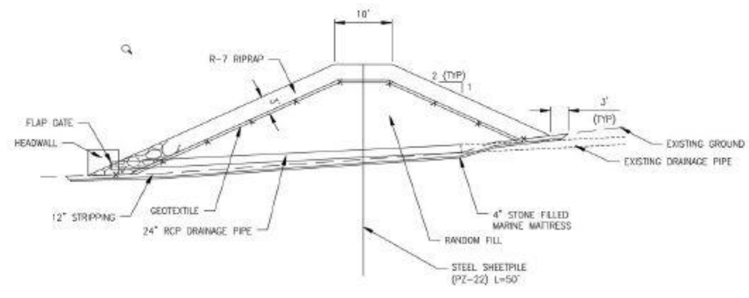
Type A 16' - Quantity per 100' Length of Structure								
Item	Length (ft)	Sheet-pile length (ft)	Width (ft)	Weight/ Lin. ft (lb/ft2)	Weight (lb)	Area (SF)	Volume (CY)	Quantity
Sheetpile	100	50		22	110,000			
Geotextile*	120		104			12,480		
Rip-Rap (R-7)	100					245	907	
4" Stone Filled Marine Matress	100		86			8,600		
Random Fill	100					600	2222	
12" Stripping	100		87			8,700		
24" RCP	0.25		60					15
Headwall (24")	0.25				7,250			1812.5
Flap Gate	0.25		1					0.25
Connections (assume 20% of total weight)					22,000			

Sheet Pile - Height of Sheetpile x Weight of sheetpile per SF x Length of Structure
Geotextile - Width from Cross Section x Length of Structure
Rip-Rip (R-5) - Area from Cross Section x Length of Structure x 1CY/27CF
4" Marine Matress - Width from Cross Section x Length of Structure. (Footprint of mattress)
Fill - Area from Cross Section x Length of Structure x 1CY/27CF
12" Stripping - Width from Cross Section x Length of Structure, DISPOSE OFFSITE
24" RCP- every 400'- Quantity= LF per 100' length of structure
Headwall- 1 every 400 '- Quanitiy= weight (lb)
Flap Gate- every 400'- Quantity= flap gate per 100' length of structure

Weight per LF of SP obtained from Pile Buck

*20% added to Geotextile to account for overlap

Quantities include 2' overbuild for settlement



TYPICAL SECTION – LEVEE – TYPE A
NTS
ELEV. 16'

				Total Quantities for Type A									Total Cost for Type A								
Group	Polyline Names	Length (ft)	# of 100' lengths	Sheetpile (lb)	Geotextile (SF)	Rip-Rap (R7) (CY)	Marine Mattress (SF)	Random Fill (CY)	12" Stripping (SF)	24" RCP (LF)	Headwall (lb)	24" Flap Gate (ea)	Sheetpile (lb)	Geotextile (SF)	Rip-Rap (R7) (CY)	Marine Mattress (SF)	Random Fill (CY)	12" Stripping (SF)	24" RCP (LF)	Headwall (lb)	24" Flap Gate (ea)
1	CM1	5,305	53	7,002,600	662,064	48,138	456,230	117,889	461,535	796	96153	13									
2	LW1, NW1, WCR1, WCY1	24,296	243	32,070,720	3,032,141	220,464	2,089,456	539,911	2,113,752	3644	440365	61									
4	WW1	3,728	37	4,920,960	465,254	33,828	320,608	82,844	324,336	559	67570	9									
5	AV1, AV2, AV3, SH1	8,446	84	11,148,720	1,054,061	76,640	726,356	187,689	734,802	1267	153084	21									
10	SI1	14,406	144	19,015,920	1,797,869	130,721	1,238,916	320,133	1,253,322	2161	261109	36									
11	UP1	1,048	10	1,383,360	130,790	9,510	90,128	23,289	91,176	157	18995	3									
12	OC1	24,080	241	31,785,600	3,005,184	218,504	2,070,880	535,111	2,094,960	3612	436450	60									
18	AC1, AC2, LP1, MG1, VN1, VN2	11,398	114	15,045,360	1,422,470	103,426	980,228	253,289	991,626	1710	206589	28									
23	BC1	593	6	782,760	74,006	5,381	50,998	13,178	51,591	89	10748	1									
26	BGL1, BV1, HC1, LB1, LB2, LB3, LB4, LB5, SB1, SC1	18,201	182	24,025,320	2,271,485	165,157	1,565,286	404,467	1,583,487	2730	329893	46									
42	BH1, BK4, BK7, BR1, LL1, LL2, MK1, PP2, PPB1, SSH1, SSP1, TR3, TR4, TR5	15,398	154	20,325,360	1,921,670	139,723	1,324,228	342,178	1,339,626	2310	279089	38									
45	BL1, MQ1	10,741	107	14,178,120	1,340,477	97,465	923,726	238,689	934,467	1611	194681	27									
52	GP52	3,449	34	4,552,680	430,435	31,296	296,614	76,644	300,063	517	62513	9									
	SUM	141,089	1,411	*186,237,480*	*17,607,907*	*1,280,252*	*12,133,654*	*3,135,311*	*12,274,743*	*21,163*	*2,557,238*	*353*	0	0	0	0	0	0	0	0	0

Partial Perimeter Plan Quantities

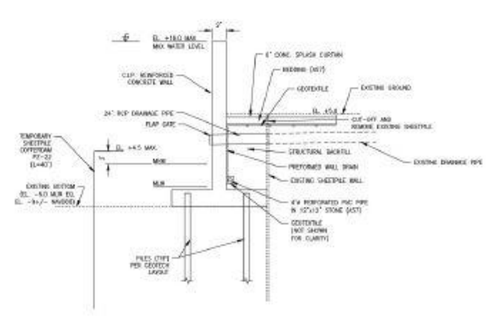
				Total Quantities for Type A									Total Cost for Type A								
Group	Polyline Names	Length (ft)	# of 100' lengths	Sheetpile (lb)	Geotextile (SF)	Rip-Rap (R7) (CY)	Marine Mattress (SF)	Random Fill (CY)	12" Stripping (SF)	24" RCP (LF)	Headwall (lb)	24" Flap Gate (ea)	Sheetpile (lb)	Geotextile (SF)	Rip-Rap (R7) (CY)	Marine Mattress (SF)	Random Fill (CY)	12" Stripping (SF)	24" RCP (LF)	Headwall (lb)	24" Flap Gate (ea)
12P	OC1 (Southern Ocean City Partial PP)	13,984	140	18,458,880	1,745,203	126,892	1,202,624	310,756	1,216,608	2098	253460	35									
26P	LB5, BV1, LB4, SB1 (Southern LBI Partial PP)	15,400	154	20,328,000	1,921,920	139,741	1,324,400	342,222	1,339,800	2310	279125	39									
	SUM	29,384	294	*38,786,880*	*3,667,123*	*266,633*	*2,527,024*	*652,978*	*2,556,408*	*4,408*	*532,585*	*73*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*

Type B 16' - Quantity per 100' Length of Structure										
Item	Length (ft)	Pile or Sheet Length (ft)	# piles	Width (ft)	Weight/ Lin. ft (lb/ft2)	Weight/ Lin. ft (lb/ft)	Weight (lb)	Area (SF)	Volume (CY)	Quantity
Cut and Remove 1.75' of Existing SheetPile	100	2			22		4,400			
Temp. Sheetpile Cofferdam	100	40			22		88,000			
Geotextile*	120			18				2,160		
Bedding AS7 Stone	100							15	56	
Structural Backfill	100							55	204	
HP14x73 piles (100')	100	62	40			73	181,040			
C.I.P. Reinforced Concrete Wall	100							78	289	
Preformed Wall Drain	100			8.75				875		
4" DIA perforated PVC Pipe in 12"x12" Stone (AS7)	100									
12"x12" Stone (AS7)	100							1	4	
24" RCP	0.25			8						2
Flap Gate	0.25			1						0.25
Splash Curtain (Concrete)	100							8	30	
Excavation	100							17	63	
Dispose of Excess Excavated Material	100							17	63	

Temp Sheetpile Cofferdam - Height of Sheetpile x Weight of per SF of Sheetpile x Length of Structure
Geotextile - (Width below riprap plus permiter around Perforated Pipe from Cross Section) x Length of Structure
Bedding (AS7) - Area from Cross Section x Length of Structure x 1CY/27CF
Structural Backfill - Area from Cross Section x Length of Structure x 1CY/27CF
HP14x73 Piles - Pile Length x Number of Piles per 100' x Weight per LF of Piles
C.I.P. Reinforced Concrete Wall - Area From Cross Section x Length of Structure x 1CY/27CF
Preformed Wall Drain - Height from Cross Section x Length of Structure
4" DIA Perforated PVC Pipe - Length of Structure. These empty into the interior drainage pipes every 400LF.
12"x12" Stone Encasement - Area from Cross Section x Length of Structure x 1CY/27CF
Cut and Removal of Sheetpile - Height of Sheetpile x Weight per SF of Sheetpile x Length of Structure
Excavation- Area from Cross Section x Length of Structure x 1CY/27CF
Excess Excavation - Area of Excavation - Area of Backfill x 1CY/27CF
24" RCP- every 400'- Quantity= LF per 100' length of structure
Flap Gate- every 400'- Quantity= flap gate per 100' length of structure
Splash Curtain- Area from cross section x 100 LFx 1CY/27CF

Weight per SF of SP obtained from Pile Buck
*20% added to Geotextile to account for overlap

H-Piles - 2 piles every 5 LF of Structure = (100'/5')*2= 40 piles
Weight per LF of Pile obtained fromSteel Construction Maunal (AISC)



TYPICAL SECTION – CONCRETE CANTILEVER WALL ON PILES – TYPE B
NTS

Group	Polyline Names	*Type B Length (ft)	*# of 100' Type B lengths	Total Quantities for Type B: Cut & Remove Sheetpile (lb)	Temp. Sheetpile (lb)	Geotextile (SF)	Stone (AS7) (CY)	Structural Backfill (CY)	HP14x73 (lb)	C.I.P. (CY)	Wall Drain (SF)	PVC Pipe (LF)	24" RCP (LF)	24" Flap Gate (ea)	Splash Curtain (CY)	Excavation (CY)	Dispose Excavation (CY)
1	CM1	1,845	18	81,180	1623600	39852	1093	3758	3340188	5330	16144	1845	37	5	547	1162	1162
2	LW1, NW1, WCR1, WCY1	11,361	114	499,884	9997680	245398	6732	23143	20567954	32821	99409	11361	227	28	3366	7153	7153
4	WW1	4,246	42	186,824	3736480	91714	2516	8649	7686958	12266	37153	4246	85	11	1258	2673	2673
5	AV1, AV2, AV3, SH1	63,970	640	2,814,680	56293600	1381752	37908	130309	115811288	184802	559738	63970	1279	160	18954	40277	40277
10	SI1	9,114	91	401,016	8020320	196862	5401	18566	16499986	26329	79748	9114	182	23	2700	5738	5738
11	UP1	3,304	33	145,376	2907520	71366	1958	6730	5981562	9545	28910	3304	66	8	979	2080	2080
12	OC1	25,715	257	1,131,460	22629200	555444	15239	52382	46554436	74288	225006	25715	514	64	7619	16191	16191
18	AC1, AC2, LP1, MG1, VN1, VN2	43,446	434	1,911,624	38232480	938434	25746	88501	78654628	125511	380153	43446	869	109	12873	27355	27355
23	BC1	36,743	367	1,616,692	32333840	793649	21774	74847	66519527	106146	321501	36743	735	92	10887	23134	23134
26	BGL1, BV1, HC1, LB1, LB2, LB3, LB4, LB5, SB1, SC1	135,136	1,351	5,945,984	118919680	2918938	80081	275277	244650214	390393	1182440	135136	2703	338	40040	85086	85086
42	BH1, BK4, BK7, BR1, LL1, LL2, MK1, PP2, PPB1, SSH1, SSP1, TR3, TR4, TR5	108,744	1,087	4,784,736	95694720	2348870	64441	221516	196870138	314149	951510	108744	2175	272	32220	68468	68468
45	BL1, MQ1	7,553	76	332,332	6646640	163145	4476	15386	13673951	21820	66089	7553	151	19	2238	4756	4756
52	GPS2	-	-	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	SUM	451,177	4,512	19,851,788	397,035,760	9,745,423	267,364	919,064	816,810,841	1,302,400	3,947,799	451,177	9,024	1,128	133,682	284,074	284,074

Group	Polyline Names	Total Cost for Type B: Cut & Remove Sheetpile (lb)	Temp. Sheetpile (lb)	Geotextile (SF)	Stone (AS7) (CY)	Structural Backfill (CY)	HP14x73 (lb)	C.I.P. (CY)	Wall Drain (SF)	PVC Pipe (LF)	24" RCP (LF)	24" Flap Gate (ea)	Splash Curtain (CY)	Excavation (CY)	Dispose Excavation (CY)
1	CM1														
2	LW1, NW1, WCR1, WCY1														
4	WW1														
5	AV1, AV2, AV3, SH1														
10	SI1														
11	UP1														
12	OC1														
18	AC1, AC2, LP1, MG1, VN1, VN2														
23	BC1														
26	BGL1, BV1, HC1, LB1, LB2, LB3, LB4, LB5, SB1, SC1														
42	BH1, BK4, BK7, BR1, LL1, LL2, MK1, PP2, PPB1, SSH1, SSP1, TR3, TR4, TR5														
45	BL1, MQ1														
52	GPS2														
	SUM	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Partial Perimeter Plan Quantities

Group	Polyline Names	*Type B Length (ft)	*# of 100' Type B lengths	Total Quantities for Type B: Cut & Remove Sheetpile (lb)	Temp. Sheetpile (lb)	Geotextile (SF)	Stone (AS7) (CY)	Structural Backfill (CY)	HP14x73 (lb)	C.I.P. (CY)	Wall Drain (SF)	PVC Pipe (LF)	24" RCP (LF)	24" Flap Gate (ea)	Splash Curtain (CY)	Excavation (CY)	Dispose Excavation (CY)
12P	OC1 (Southern Ocean City Partial PP)	3,156	32	138,864	2777280	68170	1870	6429	5713622	9117	27615	3156	63	8	935	1987	1987
26P	LB5, BV1, LB4, SB1 (Southern LBI Partial PP)	66,559	666	2,928,596	58571920	1437674	39442	135583	120498414	192282	582391	66559	1331	166	19721	41908	41908
	SUM	69,715	697	3,067,460	61,349,200	1,505,844	41,313	142,012	126,212,036	201,399	610,006	69,715	1,394	174	20,656	43,895	43,895

Group	Polyline Names	Total Cost for Type B: Cut & Remove Sheetpile (lb)	Temp. Sheetpile (lb)	Geotextile (SF)	Stone (AS7) (CY)	Structural Backfill (CY)	HP14x73 (lb)	C.I.P. (CY)	Wall Drain (SF)	PVC Pipe (LF)	24" RCP (LF)	24" Flap Gate (ea)	Splash Curtain (CY)	Excavation (CY)	Dispose Excavation (CY)
12P	OC1 (Southern Ocean City Partial PP)														
26P	LB5, BV1, LB4, SB1 (Southern LBI Partial PP)														
	SUM	0	0	0	0	0	0	0	0	0	0	0	0	0	0

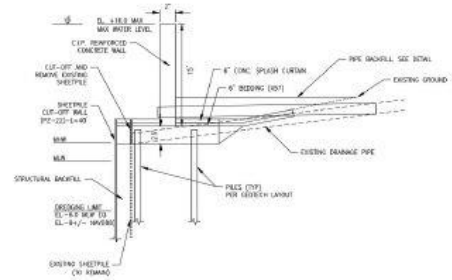
Type C 16' - Quantity per 100' Length of Structure										
Item	Length (ft)	Pile or Sheet Length (ft)	# piles	Width (ft)	Weight/ Lin. ft (lb/ft2)	Weight/ Lin. ft (lb/ft)	Weight (lb)	Area (SF)	Volume (CY)	Quantity
Cut and Remove 3.5' of Existing SheetPile	100	4			22		7,700			
6" Bedding (A57)	100							8	30	
6" Structural Backfill and between sheetpile walls	100							6	22	
HP14x73 piles	100	57	25			73	104,025			
C.I.P. Reinforced Concrete Wall	100							62	230	
Backfill	100							9	34	
Excavation	100							72	267	
Dispose of Excess Excavated Material	100							63	233	
Sheetpile Cut-off	100	40			22		88,000			
Connections (assume 20% of total weight of sheetpile)							17,600			
24" RCP	0.25			28						7
Flap Gate	0.25			1						0.25
Splash Curtain (Concrete)	100							8	30	
Pipe Backfill	0.25							40	0.4	
Demo & Dispose Existing Pipe	0.25			32						8

Cut and Removal of Sheetpile - Height of Sheetpile x Weight per SF of Sheetpile x Length of Structure
6" Bedding (A57)- Area from Cross Section x Length of Structure x 1CY/27CF
6" Structural fill - Area from Cross Section x Length of Structure x 1CY/27CF
HP14x73 Piles - Pile Length x Number of Piles per 100' x Weight per LF of Piles
C.I.P. Reinforced Concrete Wall - Area From Cross Section x Length of Structure x 1CY/27CF
Backfill - Area from Cross Section x Length of Structure x 1CY/27CF
Excavation - Area from Cross Section x Length of Structure x 1CY/27CF
Excess Excavation - Area of Excavation - Area of Backfill x 1CY/27CF
Sheetpile Cut-off - Sheetpile length x Weight per SF of Sheetpile x Length of Structure
24" RCP- every 400'- Quantity= LF per 100' length of structure
Flap Gate- every 400'- Quantity= flap gate per 100' length of structure
Splash Curtain- Area from cross section x 100 LF x 1CY/27CF
Demo & Dispose Existing Pipe- Length from Cross Section x 0.25 (because every 400'), quantity= LF

Weight per SF obtained from Pile Buck

H-Piles - 2 piles every 8 LF of Structure = (100'/8')*2 = 25 piles
Weight per LF of Pile obtained from Steel Construction Manual (AISC)

Weight per SF obtained from Pile Buck for PZ22



TYPICAL SECTION - CONCRETE CANTILEVER WALL ON PILES - TYPE C
NTS

Group	Polyline Names	Length (ft)	# of 100' lengths	Total Quantities for Type C: Remove 3.5' of Sheetpile (lb)	6" Bedding (CY)	6" Structural Backfill (CY)	HP14x73 (lb)	C.I.P. (CY)	Backfill (CY)	Excavation (CY)	Dispose Excess Excavation (CY)	Sheetpile Cut-off (lb)	24" RCP (LF)	24" Flap Gate (ea)	Splash Curtain (CY)	Pipe Backfill (CY)	Demo & Dispose Existing Pipe (LF)
1	CM1	3,213	32	247,401	952	714	3,342,323	7,378	1,095	8,568	7,473	3,392,928	225	8	952	12	257
2	LW1, NW1, WCR1, WCY1	3,257	33	250,789	965	724	3,388,094	7,479	1,110	8,685	7,576	3,439,392	228	8	965	12	261
4	WW1	-	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5	AV1, AV2, AV3, SH1	3,350	34	257,950	993	744	3,484,838	7,693	1,141	8,933	7,792	3,537,600	235	8	993	12	268
10	SI1	2,400	24	184,800	711	533	2,496,600	5,511	818	6,400	5,582	2,534,400	168	6	711	9	192
11	UP1	3,835	38	295,295	1,136	852	3,989,359	8,806	1,307	10,227	8,920	4,049,760	268	10	1136	14	307
12	OC1	19,220	192	1,479,940	5,695	4,271	19,993,605	44,135	6,549	51,253	44,704	20,296,320	1345	48	5695	71	1538
18	AC1, AC2, LP1, MG1, VN1, VN2	29,672	297	2,284,744	8,792	6,594	30,866,298	68,136	10,110	79,125	69,015	31,333,632	2077	74	8792	110	2374
23	BC1	11,363	114	874,951	3,367	2,525	11,820,361	26,093	3,872	30,301	26,429	11,999,328	795	28	3367	42	909
26	BGL1, BV1, HC1, LB1, LB2, LB3, LB4, LB5, SB1, SC1	25,975	260	2,000,075	7,696	5,772	27,020,494	59,646	8,851	69,267	60,416	27,429,600	1818	65	7696	96	2078
42	BH1, BK4, BK7, BR1, LL1, LL2, MK1, PP2, PPB1, SSH1, SSP1, TR3, TR4, TR5	11,197	112	862,169	3,318	2,488	11,647,679	25,712	3,815	29,859	26,043	11,824,032	784	28	3318	41	896
45	BL1, MG1	2,751	28	211,827	815	611	2,861,728	6,317	937	7,336	6,399	2,905,056	193	7	815	10	220
52	GP52	1,031	10	79,387	305	229	1,072,498	2,367	351	2,749	2,398	1,088,736	72	3	305	4	82
	SUM	117,264	1,173	*9,029,328*	*34,745*	*26,059*	*121,983,676*	*269,279*	*39,957*	*312,704*	*272,747*	*123,830,784*	*8,208*	*293*	*34,745*	*434*	*9,381*

Group	Total Cost for Type C: Remove 3.5' of Sheetpile (lb)	6" Bedding (CY)	6" Structural Backfill (CY)	HP14x73 (lb)	C.I.P. (CY)	Backfill (CY)	Excavation (CY)	Dispose Excess Excavation (CY)	Sheetpile Cut-off (lb)	24" RCP (LF)	24" Flap Gate (ea)	Splash Curtain (CY)	Pipe Backfill (CY)	Demo & Dispose Existing Pipe (LF)
1														
2														
4														
5														
10														
11														
12														
18														
23														
26														
42														
45														
52														
SUM	0	0	0	0	0	0	0	0	0	0	0	0		

Partial Perimeter Plan Quantities

Group	Polyline Names	Length (ft)	# of 100' lengths	Total Quantities for Type C: Remove 3.5' of Sheetpile (lb)	6" Bedding (CY)	6" Structural Backfill (CY)	HP14x73 (lb)	C.I.P. (CY)	Backfill (CY)	Excavation (CY)	Dispose Excess Excavation (CY)	Sheetpile Cut-off (lb)	24" RCP (LF)	24" Flap Gate (ea)	Splash Curtain (CY)	Pipe Backfill (CY)	Demo & Dispose Existing Pipe (LF)
12P	OC1 (Southern Ocean City Partial PP)	12,410	124	955,570	3,677	2,758	12,909,503	28,497	4,229	33,093	28,865	13,104,960	869	31	3677	46	993
26P	LB5, BV1, LB4, SB1 (Southern LBI Partial PP)	4,669	47	359,513	1,383	1,038	4,856,927	10,721	1,591	12,451	10,860	4,930,464	327	12	1383	17	374
	SUM	17,079	171	*1,315,083*	*5,060*	*3,795*	*17,766,430*	*39,218*	*5,820*	*45,544*	*39,724*	*18,035,424*	*1,196*	*43*	*5,060*	*63*	*1,366*

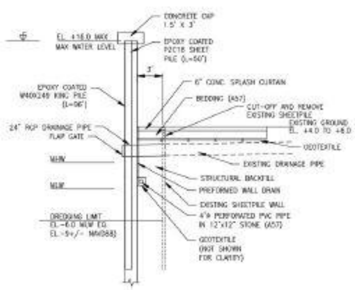
Group	Total Cost for Type C: Remove 3.5' of Sheetpile (lb)	6" Bedding (CY)	6" Structural Backfill (CY)	HP14x73 (lb)	C.I.P. (CY)	Backfill (CY)	Excavation (CY)	Dispose Excess Excavation (CY)	Sheetpile Cut-off (lb)	24" RCP (LF)	24" Flap Gate (ea)	Splash Curtain (CY)	Pipe Backfill (CY)	Demo & Dispose Existing Pipe (LF)
12P														
26P														
SUM	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*

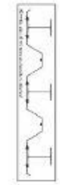
Type D 16' - Quantity per 100' Length of Structure										
Item	Length (ft)	Pile or Sheet Length (ft)	# piles	Width (ft)	Weight/ Lin. ft (lb/ft)	Weight/ Lin. ft (lb/ft2)	Weight (lb)	Area (SF)	Volume (CY)	Quantity
Cut and Remove 1.75' of Existing SheetPile	100	2			22		4,400			
Geotextile*	120			18				2,160		
Bedding A57 Stone	100							15	56	
Structural Backfill	100							31	115	
Epoxy Coated W40x249 King piles	100	96	18		249		430,272			
Epoxy Coated PZC18 Sheet Piles (LB Foster)**	100	50				24	120,000			
BBS-F Connectors (LB Foster)	100		18							18
BBS-M Connectors (LB Foster)	100		18							18
Preformed Wall Drain	100			8.75				875		
4" DIA perforated PVC Pipe in 12"x12" Stone (A57)	100									
12"x12" Stone (A57)	100							1	4	
24" RCP	0.25			5						1.25
Flap Gate	0.25			1						0.25
Splash Curtain (Concrete)	100							8	30	
Excavation	100							57	211	
Dispose of Excess Excavated Material	100							57	211	
Concrete Cap	100							4.5	17	

Geotextile - (Width below riprap plus permiter around Perforated Pipe from Cross Section) x Length of Structure
Bedding (A57) - Area from Cross Section x Length of Structure x 1CY/27CF
Structural Backfill - Area from Cross Section x Length of Structure x 1CY/27CF
W40x249 King Piles - Pile Length x Number of Piles per 100' x Weight per LF of Piles
BBS-F & BBS-M Connectors- 1 ea per king pile, Quantity = EA
Preformed Wall Drain - Height from Cross Section x Length of Structure
4" DIA Perforated PVC Pipe - Length of Structure. These empty into the interior drainage pipes every 400LF.
12"x12" Stone Encasement - Area from Cross Section x Length of Structure x 1CY/27CF
Cut and Removal of Sheetpile - Height of Sheetpile x Weight per SF of Sheetpile x Length of Structure
Excavation- Area from Cross Section x Length of Structure x 1CY/27CF
Dispose Excess Excavation - Area of Excavation - Area of Backfill x 1CY/27CF
24" RCP- every 400'- Quantity= LF per 100' length of structure
Flap Gate- every 400'- Quantity= flap gate per 100' length of structure
Splash Curtain- Area from cross section x 100 LFx 1CY/27CF
Cut and Removal of Sheetpile - Height of Sheetpile x Weight per SF of Sheetpile x Length of Structure
Concrete Cap - Area from cross section x 100 LF x 1CY/27CF

*20% added to Geotextile to account for overlap
**Length per discussion/guidance from Jeff (Geotech)
King Piles -spaced 67.81" O.C., 100/(67.81/12)= 17.70 ~18 per 100 LF

Weight per LF of sheetpiles obtained per spec attached.
Weight per LF of King Pile obtained from Steel Construction Manual (AISC)





TYPICAL SECTION – KING PILE COMBINED WITH STEEL SHEETPILE WALL – TYPE D
NTS

Group	Polyline Names	Length (ft)	# of 100' lengths	Remove 3.5' of Sheetpile (lb)	Geotextile (SF)	A57 Stone (CY)	Structural Backfill (CY)	W40x249 King Pile (lb)	PZC18 Sheet Pile (lb)	BBS-F Connectors (ea)	BBS-M Connectors (ea)	Wall Drain (SF)	PVC Pipe (LF)	24" RCP (LF)	24" Flap Gate (ea)	Splash Curtain (CY)	Excavation (CY)	Disposal Excavation (CY)	Concrete Cap (CY)
				Total Quantities for Type D															
1	CM1	5,402	55	240,128	117,979	3,237	6,271	23,501,457	6,554,400	983	983	47,793	5462	68	14	1618	11531	11531	910
2	LW1, NW1, WCR1, WCY1	15,257	153	671,308	329,551	9,041	17,517	65,646,599	18,308,400	2,746	2,746	133,499	15257	191	38	4521	32209	32209	2543
4	WW1	3,752	38	165,088	81,043	2,223	4,308	16,143,805	4,502,400	675	675	32,830	3752	47	9	1112	7921	7921	625
5	AV1, AV2, AV3, SH1	21,459	215	944,196	463,514	12,716	24,638	92,332,068	25,750,800	3,863	3,863	187,766	21459	268	54	6358	45302	45302	3577
10	SI1	9,246	92	406,824	199,714	5,479	10,616	39,782,949	11,095,200	1,664	1,664	80,903	9246	116	23	2740	19519	19519	1541
11	UP1	-	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12	OC1	9,717	97	427,548	209,887	5,758	11,157	41,809,530	11,660,400	1,749	1,749	85,024	9717	121	24	2879	20514	20514	1620
18	AC1, AC2, LP1, MG1, VN1, VN2	26,595	266	1,170,180	574,452	15,760	30,535	114,430,838	31,914,000	4,787	4,787	232,706	26595	332	66	7880	56145	56145	4433
23	BC1	-	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
26	BGL1, BV1, HC1, LB1, LB2, LB3, LB4, LB5, SB1, SC1	29,811	298	1,311,684	643,918	17,666	34,227	128,268,866	35,773,200	5,366	5,366	260,846	29811	373	75	8833	62934	62934	4969
42	BH1, BK8, BK7, BR1, LL1, LL2, MK1, PP2, PP81, SSH1, SSP1, TR3, TR4, TR5	51,533	515	2,267,452	1,113,113	30,538	59,168	221,732,070	61,839,600	9,276	9,276	450,914	51533	644	129	15269	108792	108792	8589
45	BL1, MQ1	1,775	18	78,100	38,340	1,052	2,038	7,637,328	2,130,000	320	320	15,531	1775	22	4	526	3747	3747	296
52	GPS1	-	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	SUM	174,607	1,746	7,682,708	3,771,511	103,471	200,475	751,285,031	209,528,400	31,429	31,429	1,527,811	174,607	2,183	437	51,735	368,615	368,615	29,101

Group	Remove 3.5' of Sheetpile (lb)	Geotextile (SF)	A57 Stone (CY)	Structural Backfill (CY)	W40x249 King Pile (lb)	PZC18 Sheet Pile (lb)	BBS-F Connectors (ea)	BBS-M Connectors (ea)	Wall Drain (SF)	PVC Pipe (LF)	24" RCP (LF)	24" Flap Gate (ea)	Splash Curtain (CY)	Excavation (CY)	Disposal Excavation (CY)	Concrete Cap (CY)
	Total Cost for Type D															
1																
2																
4																
5																
10																
11																
12																
18																
23																
26																
42																
45																
52																
SUM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Partial Perimeter Plan Quantities

Group	Polyline Names	Length (ft)	# of 100' lengths	Remove 3.5' of Sheetpile (lb)	Geotextile (SF)	A57 Stone (CY)	Structural Backfill (CY)	W40x249 King Pile (lb)	PZC18 Sheet Pile (lb)	BBS-F Connectors (ea)	BBS-M Connectors (ea)	Wall Drain (SF)	PVC Pipe (LF)	24" RCP (LF)	24" Flap Gate (ea)	Splash Curtain (CY)	Excavation (CY)	Disposal Excavation (CY)	Concrete Cap (CY)
				Total Quantities for Type D															
12P	OC1 (Southern Ocean City Partial PP)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
26P	LB5, BV1, LB4, SB1 (Southern LBI Partial PP)	10,804	108	475,376	233,366	6,402	12,405	46,486,587	12,964,800	1,945	1,945	94,535	10804	135	27	3201	22808	22808	1801
	SUM	10,804	108	475,376	233,366	6,402	12,405	46,486,587	12,964,800	1,945	1,945	94,535	10,804	135	27	3,201	22,808	22,808	1,801

Group	Remove 3.5' of Sheetpile (lb)	Geotextile (SF)	A57 Stone (CY)	Structural Backfill (CY)	W40x249 King Pile (lb)	PZC18 Sheet Pile (lb)	BBS-F Connectors (ea)	BBS-M Connectors (ea)	Wall Drain (SF)	PVC Pipe (LF)	24" RCP (LF)	24" Flap Gate (ea)	Splash Curtain (CY)	Excavation (CY)	Disposal Excavation (CY)	Concrete Cap (CY)
	Total Cost for Type D															
12P																
26P																
SUM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

2.3.1 Alternative Alignments

Preliminary Real Estate acreage requirements, I.E., permanent and temporary easement limits, were computed for all the structures, and provided to Baltimore District Real Estate. Refer to the figures in Section 6 Real Estate. Their initial analysis indicates that real estate values for the perimeter plan will be very high and may include compensation to front line property owners for loss of view. Currently investigations are ongoing to adjust the line of protection and complete a screening analysis for alternate alignments. Specifically, we are qualitatively looking at a case where the first row of properties along appropriate segments of the wall alternatives are bought out, and the less expensive wall (Type A) is constructed in that space. Another alignment change to be considered is to use the impermeable barrier section from the SSB screening and move the line of protection offshore. The new location would need to accommodate vessels to move along the bulkhead line with navigation and tidal gates to provide access and tidal exchange. It would need to reduce enough length of Type B or D wall utilized and negate high-cost real estate impacts to realize a savings in cost for that location.

3 STORM SURGE BARRIERS

3.1 SSB Cycle 2 Screening

A SSB Cycle 2 screening level analysis was completed in December 2018 to initially investigate storm surge barrier (SSB) options that would protect NJBB from coastal storm damages. USACE Engineering Research and Development Center (ERDC) performed three iterations of SSB modeling throughout the study area. The first iteration modeled a SSB at each individual inlet (one at a time). The second iteration modeled 15 alternatives, comprised of inlet and bay closures, to see how a system of barriers would reduce water levels. The third iteration modeled 8 alternatives with a larger storm set to establish hazard curves used for the HEC-FDA economic model. Based on the ERDC models, 11 inlets and 8 bay closures were identified for screening level analysis. Preliminary alignments of SSB components were estimated in AutoCAD Civil 3D for each location. Quantities were then estimated at each location and were provided to Cost Engineering which then estimated construction costs for each SSB. Construction costs were then used in the HEC-FDA economic model to determine the National Economic Development (NED) benefits for each barrier. Barriers with low NED benefits were screened out while barriers with high NED benefits were added to a focused array of alternatives. The focused array was then investigated in more detail during the Cycle 3 analysis in order to reach a tentatively selected plan (TSP). The following sections outline the process for determining SSB alignments and quantities for all 11 inlets and 8 bay closures.

3.1.1 Storm Surge Barrier Parametric Cost Model

The cost model used in this study was developed by USACE New York District and is based on statistical data and major design considerations. Design considerations include barrier crest elevations, lengths, depths and proportion of navigable and auxiliary flow features versus static elements. As seen in Table 3-5, cost engineers assembled a dataset of seventeen reference SSBs from around the world (Mooyart & Jonkman, 2017). As the study continues, this data set can be improved and expanded upon.

Table 3-5: Reference Set of Storm Surge Barriers

Reference Storm Surge Barrier	Country	Total Construction Duration [Years]	Initial Construction Cost [$, 2019Q1]	Average Height of Barrier (Sill to Crest) [FT]	Lengths: Dynamic Features, Nav + Aux [FT]	Lengths: Total (incl. dam) [FT]
Hollandsche Ijssel	Netherlands	4	$262,000,000	36	400	400
New Bedford	United States	4	$185,000,000	55	361	4495
Stamford	United States	4	$126,000,000	33	98	2854
Eider	Germany	6	$416,000,000	22	846	16076
Hull	United Kingdom	3	$29,000,000	35	134	134
Thames	United Kingdom	8	$2,521,000,000	42	1718	1718
Eastern Scheldt	Netherlands	17	$6,960,000,000	44	9206	25853
Maeslant	Netherlands	8	$1,010,000,000	82	2789	2789
Hartel	Netherlands	4	$219,000,000	31	763	820
Ramspol	Netherlands	5	$206,000,000	27	715	1348
Ems	Germany	3	$585,000,000	42	1516	2100
St. Petersburg	Russia	27	$9,948,000,000	24	7538	76280
IHNC	United States	3	$643,000,000	35	712	9449
Seabrook	United States	3	$192,000,000	34	325	469
Harvey Canal	United States	3	$368,000,000	24	282	394
GIWW	United States	4	$446,000,000	43	525	1706
MOSE	Italy	19	$7,540,000,000	46	5184	5184

The parametric cost model equation differentiates barrier components into three categories; navigable gate area (NA), auxiliary flow gate area (AA), and impermeable barrier/dam area (DA). Length or area of "dynamic" span of SSBs refers to those portions of a barrier system which can be opened either to allow flow for navigation or auxiliary flow. The values include both the width/area of the openings and the structures associated with operation and housing of such features. By contrast, length and area of "static" span refers to that of the closed off wall or dam portions of barrier systems. The model estimates construction costs at a specified % confidence interval based on available reference data for existing barriers all over the world. An example of the 50% confidence interval parametric cost equation is as follows:

$$Construction\ Cost_{50\%} = (\$19{,}200 * NA) + (\$13{,}900 * AA) + (\$3{,}000 * DA)$$

The construction cost is a function of the cross sectional area of each barrier component. Specific barrier widths for auxiliary flow were not analyzed as part of the Cycle 2 screening level analysis and were evaluated in more detail during Cycle 3. The SSB design heights were selected to be 20' NAVD88 at the inlets and 13' NAVD88 along the bay closures. Since bay closure locations are not as exposed to ocean waves and storm surge, the design heights requirements are not as high.

3.1.2 Navigable and Auxiliary Flow Gates

A navigable gate was analyzed at every inlet and bay closure to provide a navigable opening with unlimited vertical clearance. At this stage of the analysis, navigable gates were assumed to be sector gates due to their prevalence not only in the United States but all over the world. A sector gate contains two dynamic gates and two static gate housing structures. The dynamic gates remain in their housing structures, providing an open channel for navigation. The dynamic sector gates are horizontally closed during significant storm events. Due to the parametric cost model, the specific type of navigable gate does not affect the total construction cost. The parametric cost model references construction costs for a variety of navigable gate types. The specific type of navigable gate will need to be further evaluated and refined as the study continues.

Along bay closure alignments, sector gates were positioned across the NJIWW. At the inlets, sector gates were placed across federal navigation channels. To ensure channels were not restricted, the dynamic span of the sector gates were sized to provide a 10 foot buffer on either side of the NJIWW or federal navigation channel. The size of each dynamic gate and static housing structure was scaled off an existing SSB site in the United States, the Seabrook Flood Complex in New Orleans, LA (see **Error! Reference source not found.**). Not all inlets or bay closures have a federal navigation channel or NJIWW. In these instances, sector gates were positioned along the deepest portion of the waterway in order to promote tidal flow during open conditions. Some inlets, such as Townsends Inlet, have no Federal Navigation Channel but do have existing bridges with drawbridges. Sector gates were aligned directly in front of these drawbridges to support large vessel navigation.

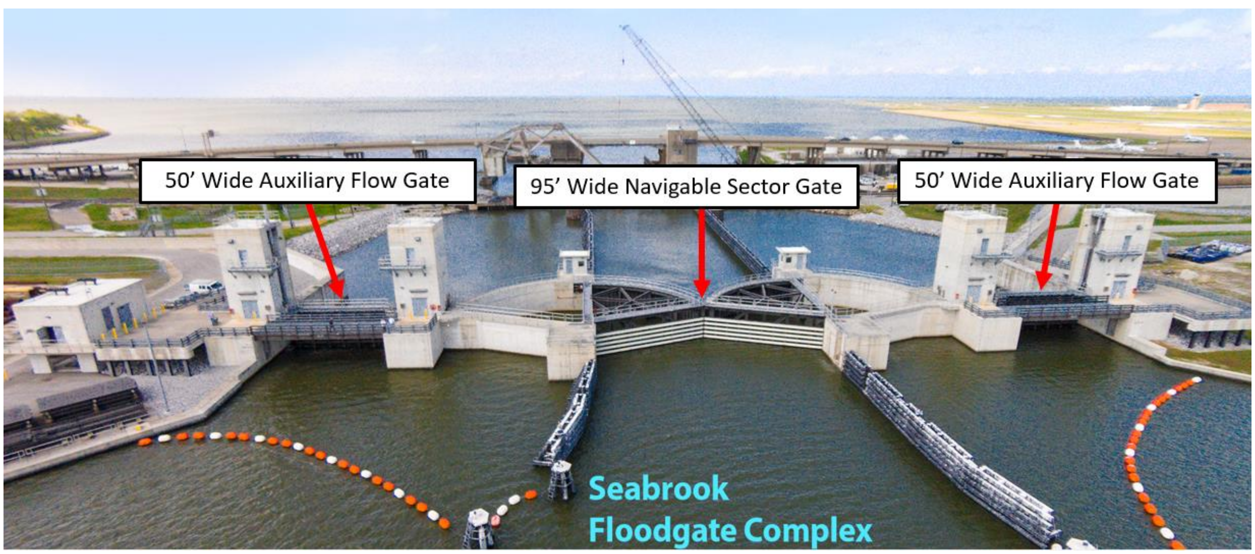


Figure 3-1: Seabrook Floodgate Complex in New Orleans, LA

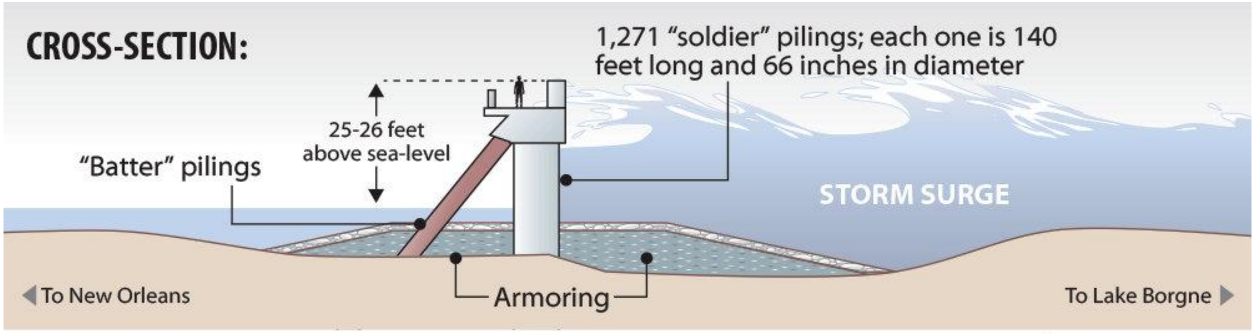
Auxiliary flow gates were positioned adjacent to navigable gates and throughout bay closures to maintain tidal flow. Auxiliary flow gates were placed throughout water depths that were deemed constructible and practical. For example, an area with water depths of only a foot may not generate enough flow in and out of a channel to justify the cost of an auxiliary flow gate. The minimum flow gate depth will need to be further investigated as the study continues. Auxiliary flow gates were assumed to be vertical lift gates because they are one of the more prevalent SSB gate types seen in the United States as well as overseas. Due to the parametric cost model, the specific type of auxiliary flow gate does not affect the total construction cost. The parametric cost model references construction costs for a variety of auxiliary flow gates including, but not limited to, vertical lift gates, segment gates, flap gates, and inflatable gates. The specific type of auxiliary flow gate will be further evaluated and refined as the study continues. The Seabrook Flood

Complex (see **Error! Reference source not found.**) was used as a template to initially size the vertical lift gates for this study. The dynamic portion of the gate is approximately 50 feet long, flanked by two housing structures that are each approximately 18 feet long. The length of movable gate was refined during Cycle 3 to minimize the flow restriction of the inlet. Vertical lift gates have limited vertical clearance but are capable of providing recreational navigation. For example, the Bayou Bienvenue vertical lift gate in New Orleans, LA (see **Error! Reference source not found.**) has enough vertical clearance to allow recreational boats to pass to and from Lake Borgne. The bottom of the gate rests at approximately 33' NAVD88 in the open condition.

Figure 3-2: Bayou Bienvenue Vertical Lift Gate in New Orleans, LA

3.1.3 Impermeable Barriers

Impermeable barriers flank the dynamic SSB components in order to tie the barrier into the upland. Impermeable barriers were also positioned along portions of low lying marsh land across bay closure alignments. The parametric cost equation does not estimate construction costs for a specific type of impermeable barrier, it applies a cost factor to a cross sectional area of static wall based on reference data for seventeen existing SSB sites (Table 3-5). A site specific impermeable barrier type has not been selected at this stage but will be further investigated as the study continues. **Error! Reference source not found.** shows one example of an existing impermeable barrier at Lake Borgne in New Orleans, LA.



Dan Swenson, NOLA.com | The Times-Picayune

Figure 3-3: Lake Borgne Impermeable Barrier in New Orleans, LA

3.1.4 Levees, Floodwalls and Seawalls

In areas that are not in open water or on open marsh land, levees, floodwalls and seawalls were used to tie barriers into high ground or existing adjacent oceanfront projects. Type A - levees were used in areas with little to no exposure to wave forcing. Type B and C - floodwalls were used in areas where the SSBs tie into the Perimeter Plan. In-water floodwalls were not used along low lying open marsh areas through bay closure alignments. The in-water floodwall design assumes there are adjacent existing sheet piles with backfill. To be conservative, impermeable barriers were selected for open marsh areas. A more detailed wall design will be investigated for low lying open marsh areas as the study continues. Seawalls were selected for low lying areas, such as beaches, that are still susceptible to waves and erosion but may not need a structure as robust as an impermeable barrier. As the study continues, beach and dune restoration measures will be investigated for these areas. Estimated seawall costs were scaled off construction costs for the Absecon Seawall in Atlantic City, NJ (see **Error! Reference source not found.**).

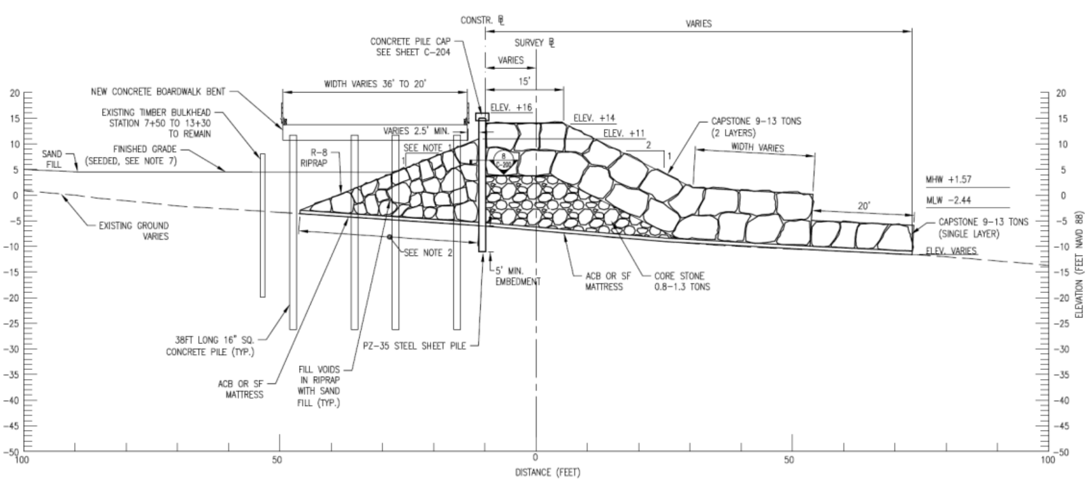


Figure 3-4: Typical Section – Absecon Seawall

3.1.5 Existing Data

Existing bathymetry and topography data was obtained from the U.S. Geological Survey's (USGS) Topobathymetric Model for New Jersey and Delaware. In response to storm damages induced from Hurricane Sandy, the USGS Coastal and Marine Geology Program in collaboration with the USGS National Geospatial Program (NGP) and National Oceanic and Atmospheric Administration (NOAA) developed three-dimensional 1-meter topobathymetric elevation models for the New Jersey/Delaware sub-region. The temporal range of input topography and bathymetry ranges from 1880 to 2014 and is referenced to NAVD88. USGS topobathymetric data was cross referenced against available USACE NAP bathymetric surveys which ranged from 2015-2018. The bathymetry data was used to estimate the total cross sectional area for each SSB component. The topographic data was used to tie SSBs into high ground. High ground was selected to be at approximately 13' NAVD88 or at an existing adjacent ocean front project. Not all ocean front projects were designed or maintained to a 13' NAVD88 elevation. Improving existing ocean front projects will need to be further evaluated as the study continues. Additional survey data will also be collected, as the study continues, to establish more accurate and representative site conditions.

3.1.6 Cycle 2 Results

Cycle 2 Quantities were measured from the Storm Surge Barrier Cycle 2 Screening drawings as shown in Appendix B.6 Drawings. Table 3-6 refers to the SSBs located at each inlet while Table 3-7 refers to the cross bay SSBs.

Table 3-6: Inlet Storm Surge Barrier Cycle 2 Quantities

Barrier Components	Inlet Storm Surge Barrier Locations										
	Cape May Canal	Cape May Inlet	Hereford Inlet	Townsends Inlet	Corsons Inlet	Great Egg Harbor Inlet	Absecon Inlet	Brigantine & Little Egg Inlet	Barnegat Inlet	Manasquan Inlet	Shark River Inlet
Navigable Gate Length (FT)	253	885	211	211	253	253	885	422	675	569	232
Navigable Gate Average Height (FT)[1]	27	48	58	51	43	63	67	55	38	32	48
Navigable Gate Area (SF)[2]	6838	42497	12227	10752	10880	15965	59261	23393	25408	18249	11131
Aux. Flow Gate Length (FT)	344	0	516	430	516	4214	774	4128	774	0	0
Aux. Flow Gate Average Height (FT)[1]	19	0	22	36	21	24	31	33	29	0	0
Aux. Flow Gate Area (SF)[2]	6613	0	11403	15577	10852	100798	23958	134891	22810	0	0
Impermeable Barrier Length (FT)	65	0	5112	1641	1124	1293	307	1927	174	0	165
Impermeable Barrier Average Height (FT)[1]	14	0	13	13	13	17	21	17	16	0	7
Impermeable Barrier Area (SF)[3]	903	0	64529	21973	14800	22322	6331	32945	2853	0	1185
Levee Length (FT)	2159	2435	0	0	0	0	0	0	1054	0	0
Seawall Length (FT)	0	302	1837	2516	2839	474	2567	48742	192	7833	0

Notes:

1. *Navigable gate average height, auxiliary gate average Height, and impermeable barrier average height is the average height from the existing bathymetry to a design height of 20' NAVD88 (see HH&C Appendix B.1 for design height calculations).*
2. *Gate area is the cross sectional surface area of the dynamic (moveable) span of barrier plus the cross sectional surface area of the housing structure associated with the gate.*
3. *The impermeable barrier area is the cross sectional surface area of the impermeable barrier.*

Table 3-7: Bay Closure Cycle 2 Quantities

Barrier Components	Bay Closure Locations							
	Wildwood Blvd Bay Closure	Stone Harbor Blvd Bay Closure	Sea Isle Blvd Bay Closure	South Ocean City Bay Closure	Absecon Blvd Bay Closure	North Point Bay Closure	Holgate Bay Closure	Point Pleasant Canal Bay Closure
Navigable Gate Length (FT)	253	253	253	253	253	253	253	253
Navigable Gate Average Height (FT)[1]	38	38	37	25	38	35	43	38
Navigable Gate Area (SF)[2]	9613	9611	9360	6324	9613	8910	10878	9613
Aux. Flow Gate Length (FT)	258	344	0	0	0	2666	3010	0
Aux. Flow Gate Average Height (FT)[1]	42	28	0	0	0	21	24	0
Aux. Flow Gate Area (SF)[2]	10793	9515	0	0	0	55961	73029	0
Impermeable Barrier Length (FT)	562	431	158	0	150	16331	10075	0
Impermeable Barrier Average Height (FT)[1]	13	14	16	0	17	13	12	0
Impermeable Barrier Area (SF)[3]	7073	5927	2488	0	2593	206342	118965	0
Levee Length (FT)	15585	20620	13096	9558	25733	15810	18074	0
Seawall Length (FT)	0	0	0	0	0	3953	9658	0
Floodwall - In the Wet (FT)	0	17546	1911	1205	9746	0	0	0
Floodwall - In the Dry (FT)	0	0	2400	2919	5503	0	0	0
Miter Gate (EA)	2	1	0	1	5	1	8	0
Sluice Gate (EA)	2	1	2	1	1	7	6	0
Road Closure (EA)	2	4	1	0	3	0	1	0

Notes:

1. *Navigable gate average height, auxiliary gate average height, and impermeable barrier average height is the average height from the existing bathymetry to a design height of 13' NAVD88 (see HH&C Appendix B.1 for design height calculations).*
2. *Gate area is the cross sectional surface area of the dynamic (moveable) span of barrier plus the cross sectional surface area of the housing structure associated with the gate.*
3. *The impermeable barrier area is the cross sectional surface area of the impermeable barrier.*

3.2 SSB Cycle 3 Screening

The SSB Cycle 3 screening analysis expanded upon the Cycle 2 screening to refine the focused array of alternatives into a TSP. Cycle 3 evaluated the following SSB locations; Southern Ocean City Bay Closure, Great Egg Harbor Inlet, Absecon Inlet, Absecon Bay Closure, Barnegat Inlet and Manasquan Inlet. The following sections outline the additional analysis performed and process for determining SSB Cycle 3 quantities.

3.2.1 Barrier Design and Assumptions

The preliminary design for Cycle 3 continues to utilize the parametric cost model from Cycle 2. This cost model was refined to increase the cost of the navigable area (NA) while decreasing the cost of the auxiliary flow (AA) area as well as the static dam area (DA):

$$Construction\ Cost_{50\%} = (\$20{,}200 * NA) + (\$11{,}800 * AA) + (\$2{,}200 * DA)$$

Similarly, to Cycle 2, the Cycle 3 preliminary SSB design assumes a combination of sector gates for navigation and vertical lift gates for auxiliary flow. The parametric cost model does not take into account the specific gate types. The equation only differentiates barriers by the three different sections; navigable, auxiliary and static. As the study progresses past the TSP to the ADM, actual quantities will need to be developed to refine the SSB cost estimate. A detailed multi-criteria gate type analysis will need to be performed post-TSP to evaluate all of the existing gate types and rank them accordingly for each proposed site location.

The size of the vertical lift gate was increased from 50 feet to 150 feet in order to promote additional conveyance. The Hartel barrier was used as an example to scale the 50 foot wide Cycle 2 vertical lift gate to 150 feet (see **Error! Reference source not found.**).

Figure 3-5: Hartel Barrier Vertical Lift Gates

The barrier is located in Spijkenisse, Netherlands and consists of two vertical lift gates approximately 162 feet and 322 feet in length. Various other design parameters were evaluated during Cycle 3 such as barrier alignment, sector gate size, sill elevation, and number of gates. ERDC-CHL modeled various Cycle 3 SSB designs in their open gate conditions to evaluate indirect impacts on tides, velocity, salinity and residence time through an Adaptive Hydraulic (AdH) Model (see HH&C Appendix B.1). Cycle 3 SSB drawings can be seen in Appendix B.6. Figure 3-6 is a rending for a potential SSB at Great Egg Harbor Inlet which includes a sector gate in the middle of the inlet that is flanked by a series of vertical lift gates on either side. All gates are being shown in their open condition.

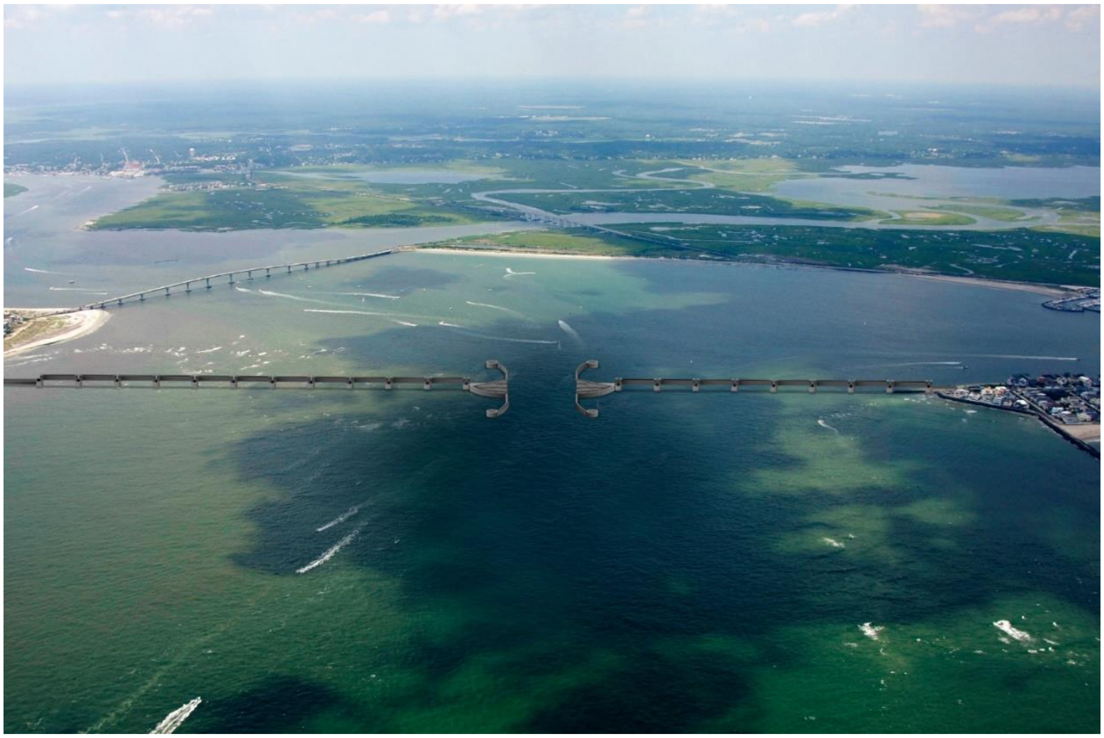


Figure 3-6: Great Egg Harbor Inlet – Storm Surge Barrier Rendering

3.2.2 Maritime Vessel Analysis

This maritime vessel analysis provides recommendations for minimum dimensions of navigable storm surge barrier gates under the NJBB CSRM Feasibility Study. Recommendations for navigation gate widths are based on vessel traffic data specific to each potential storm surge barrier location. Based on the available vessel traffic data, a specific design vessel was selected for each inlet to recommend a minimum dimension for a storm surge barrier navigation 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 bridge piers. Vessel traffic locations were also analyzed in order to recommend practical navigation gate locations at each inlet. 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 storm surge barriers are located across authorized federal navigation channels and must be sized to allow access through the entire authorized channel, outside of significant storm events. Future federal navigation channel relocating, widening, or deepening projects were not considered during this analysis but will be evaluated during the next phase of the study.
- 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.

- In the future, vessels such as tankers and cargo ships will increase in size in order to increase production. These types of large vessels are not generally reported in any of the NJBB AIS data sets and as a result, future vessel traffic was not considered when selecting design vessels. Future design vessels for the NJBB inlets will need to be further investigated in later phases of the study.
- The majority of the vessels reported through the NJBB inlets were smaller recreational vessels (pleasure crafts). Recommendations for sizing secondary navigation gates were based on this common recreational vessel.
- Preliminary navigable storm surge barrier gate widths are recommended for both one-way and two-way traffic. At this phase of the study, the assumption was made that two-way traffic should be maintained through navigable storm surge barriers. Future analysis should be performed to investigate the impacts of restricting channels to one-way traffic.
- Summer month AIS data (June through September) 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 storm surge barrier gates. Vessel dimensions reported in summer months were compared to winter months at Great Egg Harbor Inlet to verify this assumption. Vessel data was relatively similar between the two data sets but the largest vessel was reported during the summer.
- The USACE Engineering Manual (EM) 110-2-1613 Hydraulic Design of Deep-Draft Navigation Projects was initially used to calculate minimum gate widths. A separate gate width calculation was performed using the World Association for Waterborne Transport Infrastructure (PIANC) Report No. 121 Harbour Approach Channels Design Guidelines. Neither report specifically accounts for hard structures such as storm surge barriers.
- The USACE Engineering Research and Design Center (ERDC) is currently analyzing the effects of storm surge barrier structures on salinity, velocity and tidal prism (see H&H Engineering Appendix). The results of this analysis will need to be considered later in the study to properly size storm surge barrier 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.

3.2.2.1 AIS Data

AIS 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 an AIS Analysis Package (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.

3.2.2.2 Channel Design Guidance

Clear navigation gate width design guidance has not been established for storm surge barriers. For this reason, two different references were used to separately calculate and recommend safe navigation gate widths. Neither reference discusses how hard barrier structures affect the required channel width. In general, conservative assumptions and parameters were applied to the channel design criteria in an effort to account for hard structures.

Hydraulic Design of Deep-Draft Navigation Projects

The USACE Engineering Manual (EM) 1110-2-1613 Hydraulic Design of Deep-Draft Navigation Projects was used to size both one-way and two-way channel widths. Channel width design criteria provided in the EM are summarized in Table 3-8 and

Table 3-9 for one-way and two-way channels, respectively. Relatively high currents can be expected through all the NJBB inlets, especially through a constricted navigable storm surge barrier gate (see H&H Engineering Appendix). Due to maximum currents greater than 3 knots, the EM recommends ship simulations through navigable gates during more detailed phases of design. Based on the cross section definitions in the EM, most NJBB channel cross sections are not an exact match with any of cross section categories. The EM does not consider storm surge barriers when describing channel cross sections, making this a unique scenario. Additional safety factors need to be considered when navigating vessels through hard barrier structures. For this reason, the shallow channel cross section and the 1.5-3.0 knot maximum current was selected, yielding the highest and most conservative beam multiplier. The resulting beam multiplier is 5.5 for one-way traffic, assuming average aids to navigation, and 8.0 for two-way traffic.

Table 3-8: One-Way Ship Traffic Channel Width Design Criteria

One-Way Ship Traffic Channel Width Design Criteria

Channel Cross Section	Design Ship Beam Multipliers for Maximum Current, Knots		
	0.0 to 0.5	0.5 to 1.5	1.5 to 3.0
Constant Cross Section, Best Aids to Navigation			
Shallow	3.0	4.0	5.0
Canal	2.5	3.0	3.5
Trench	2.75	3.25	4.0
Variable Cross Section, Average Aids to Navigation			
Shallow	3.5	4.5	5.5
Canal	3.0	3.5	4.0
Trench	3.5	4.0	5.0

Table 3-9: Two-Way Ship Traffic Channel Width Design Criteria

Two-Way Ship Traffic Channel Width Design Criteria

Uniform Channel Cross Section	Design Ship Beam Multipliers for Maximum Current, Knots (ft/sec) 0.0 to 0.5 (0.0 to 0.8)	0.5 to 1.5 (0.8 to 2.5)	1.5 to 3.0 (2.5 to 5.0)
Best Aids to Navigation			
Shallow	5.0	6.0	8.0
Canal	4.0	4.5	5.5
Trench	4.5	5.5	6.5

Harbour Approach Channels Design Guidelines

The World Association for Waterborne Transport Infrastructure (PIANC) Report No. 121 Harbour Approach Channels Design Guidelines were used to size both one-way and two-way channel widths. Channel width design criteria provided in the PIANC report is summarized in Table 3-10 for all proposed NJBB storm surge barrier locations. According to PIANC, the two-way channel width can be estimated by doubling all of the one-way beam multiplier factors (except the bank clearance factors). Similarly to EM 1110-2-1613, the PIANC report does not consider storm surge barriers (hard structures) when developing beam multiplier factors. Conservative beam multiplier factors were selected, when applicable, for NJBB channel width parameters in an effort to apply additional factors of safety, yielding a more conservative total channel width beam multiplier. PIANC guidance also recommends ship maneuvering simulations (numerical models) be carried out in the detailed design phase to refine the preliminary design width and to quantify the safety and risk level of the final channel width. The total channel width beam multiplier is 5.3 for one-way traffic and 8.0 for two-way traffic. This compares well to the EM 1110-2-1613 beam multiplier results of 5.5 for one-way traffic and 8.0 for two-way traffic. The more conservative EM 1110-2-1613 beam multipliers (5.5 and 8.0) were chosen for the minimum practical channel calculations to help account for some of the unknown effects of the barrier structure.

Table 3-10: PIANC Channel Width Calculation for New Jersey Back Bay Sites

Parameter	Specification	Beam Multiplier Factor (One-Way)	Beam Multiplier Factor (Two-Way)
Basic Maneuvering Lane	Good	1.3	2.6
Vessel Speed	Fast (>=12kts)	0.1	0.2
Prevailing Cross Wind	Moderate (< 33 kts)	0.3	0.6
Prevailing Cross Current	Negligible (< 0.2 kts)	0.0	0.0
Prevailing Longitudinal Current	Strong (>= 3kts)	0.1	0.2
Wave Heights	(1m < Hs < 3m)	0.5	1.0
Aids to Navigation	Excellent - Good	0.0	0.0
Bottom Surface	Rough and Hard	0.2	0.4
Depth of Waterway	h < 1.25T	0.2	0.4
Bank Clearance, Red Side (left)	Steep and Hard Embankments, Structures	1.3	1.3
Bank Clearance, Green Side (Right)	Steep and Hard Embankments, Structures	1.3	1.3
Total Channel Width Beam Multiplier		**5.3**	**8.0**

3.2.2.3 Great Egg Harbor Inlet

Vessel Traffic Summary

AIS data was collected and evaluated at Great Egg Harbor Inlet from June 1, 2018 to September 30, 2018, representing a total of 121 days. Vessel traffic is assumed to be highest during the summer months and was therefore used to select a representative design vessel. Figure 3-7, Figure 3-8 and Figure 3-9 represent AIS vessel traffic summaries for Great Egg Harbor Inlet.

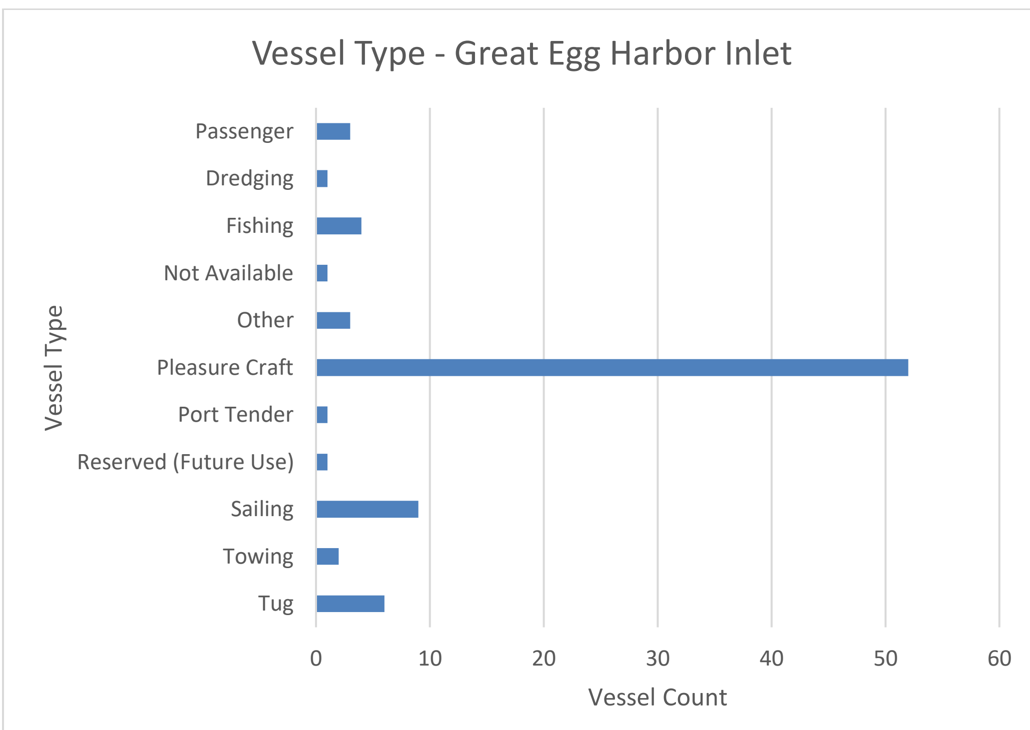


Figure 3-7: Vessel Type – Great Egg Harbor Inlet

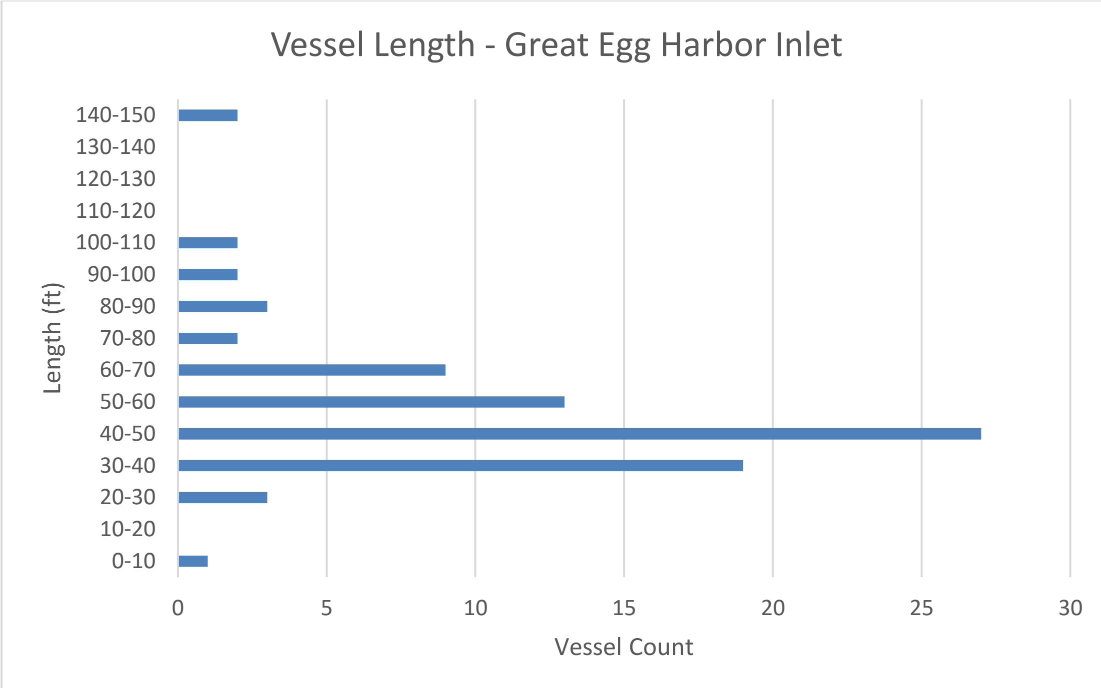


Figure 3-8: Vessel Length – Great Egg Harbor Inlet

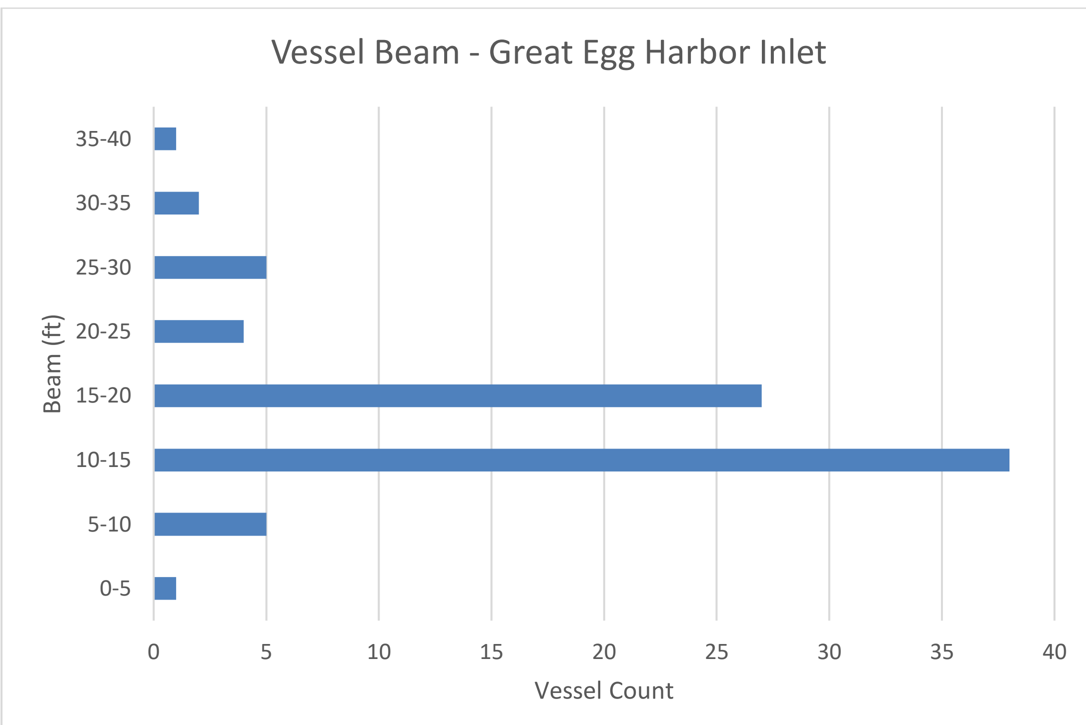


Figure 3-9: Vessel Beam – Great Egg Harbor Inlet

Minimum Practical Channel Width (Main Navigable Passage)

Main navigation openings are currently assumed to be sector gates. Secondary navigation gates may be added into the design to support the smaller recreational vessels and alleviate potential traffic through the main navigation gate. Further analysis is needed to determine the need, location and gate type for secondary navigation. Although there are vertical clearance restrictions to vertical lift gates, they can be designed to support recreational navigation. The current vertical lift gate design in the NJBB focused array of alternatives provides a 150-foot opening and may need to be increased in size to support secondary navigation. An additional sector gate could also be designed to support secondary navigation which results in unlimited vertical clearance. All reported vessels should have the ability to safely navigate through a storm surge barrier navigation gate. The vessel beam is the controlling factor to determine minimum navigation widths. For that reason, the largest reported vessel beam was selected as the design vessel as shown in Figure 3-10. The design vessel is approximately 144 feet long with a vessel beam of approximately 39 feet. Multiplying the 39 foot design vessel beam by the most conservative beam multipliers, see Section 3.2.2.2, results in a preliminary minimum channel width recommendation of approximately 215 feet for one-way traffic and 312 feet for two-way traffic.

Figure 3-10: Main Navigation Design Vessel – Great Egg Harbor Inlet

A 320 ft. wide navigable sector gate across Great Egg Harbor Inlet is currently being used in the focused array of alternatives. This 320 ft. dimension satisfies the minimum channel width recommendation of 312 ft. assuming a 39 ft. wide design vessel and two-way traffic. There is no authorized federal navigation channel at Great Egg Harbor Inlet. There are two bridges, the Ocean Drive Bridge and the JFK Memorial Bridge, adjacent to Great Egg Harbor Inlet that already constrain navigable widths to approximately 200 ft. and 100 ft. An alternative design could propose storm surge barriers directly seaward of these existing bridges and match the navigation gate widths to the existing bridge pier widths that already constrain navigation.

Minimum Practical Channel Width (Secondary Navigable Passage)

The AIS data, summarized in Figure 3-7 through Figure 3-9, shows a high volume of smaller recreational vessels (pleasure crafts) traversing through Great Egg Harbor Inlet. The highest number of recorded recreational vessels were in the 10-15 foot beam range. The selected design vessel has a length of approximately 40 feet and a vessel beam of approximately 13 feet (as shown in Figure 3-11). Using the secondary navigation design vessel and the most conservative beam multiplier from Section 3.2.2.2, a minimum channel width recommendation for a secondary navigation gate is approximately 69 feet for one-way traffic and approximately 104 feet for two-way traffic.

Figure 3-11: Secondary Navigation Design Vessel – Great Egg Harbor Inlet

Vessel Traffic Location

Within AISAP, vessel data can be displayed as vessel position heat maps. The thickness and color of the points on the map correlate to the number of vessels reported. Warmer colors (e.g. red, yellow, and white) represent areas with higher signal density and cooler colors (e.g. blue) represent areas with relatively less signal density. Figure 3-12 represents vessel data from June 1, 2018 to September 30, 2018 and suggests that vessels traverse all throughout Great Egg

Harbor Inlet but primarily enter the inlet through the middle and deepest portion. The highest intensity of vessels is located in the marina just inside the inlet.

Figure 3-12: Vessel Heat Map (6/1/2018 – 9/30/2018) – Great Egg Harbor Inlet

Summer month data was collected and analyzed in AISAP using a 5 minute sampling frequency. In order to get a more distinct representation of vessel tracks, higher sampling frequencies are needed. For this reason, 30 second sampling frequencies were used when collecting AIS data for the 4th of July weekend in 2019. The 4th of July weekend was assumed to be one of the busiest time periods of the year that would produce a manageable data set using a 30 second sampling frequency (e.g. higher sampling frequencies result in larger AISAP data outputs). Data sets would be too large to process if 30 second sampling frequencies were used to collect data for the entire summer. Figure 3-13 represents individual vessel reports recorded every 30 seconds while Figure 3-14 connects the individual reports to illustrate specific vessel traffic lines. The results from this higher frequency sampling rate confirm that the most traffic occurs in the middle of the channel. Based on these results, the recommended location of the main navigable storm surge barrier gate is in the middle and deepest section of the channel. The AIS data also shows that vessels are constrained through the Ocean Drive Bridge piers and JKF Memorial Bridge piers. Navigation gates could also be proposed adjacent to these bridges and sized to match the existing vessel constraints.

Figure 3-13: Vessel Heat Map (7/4/2019 – 7/8/2019) – Great Egg Harbor Inlet

Figure 3-14: Vessel Transit Map (7/4/2019 – 7/8/2019) – Great Egg Harbor Inlet

3.2.2.4 Absecon Inlet

Vessel Traffic Summary

AIS data was collected and evaluated at Absecon Inlet from June 1, 2018 to September 30, 2018, representing a total of 121 days. Vessel traffic is assumed to be highest during the summer months and was therefore used to select a representative design vessel. Figure 3-15, Figure 3-16, and Figure 3-17 represent AIS vessel traffic summaries for Absecon Inlet.

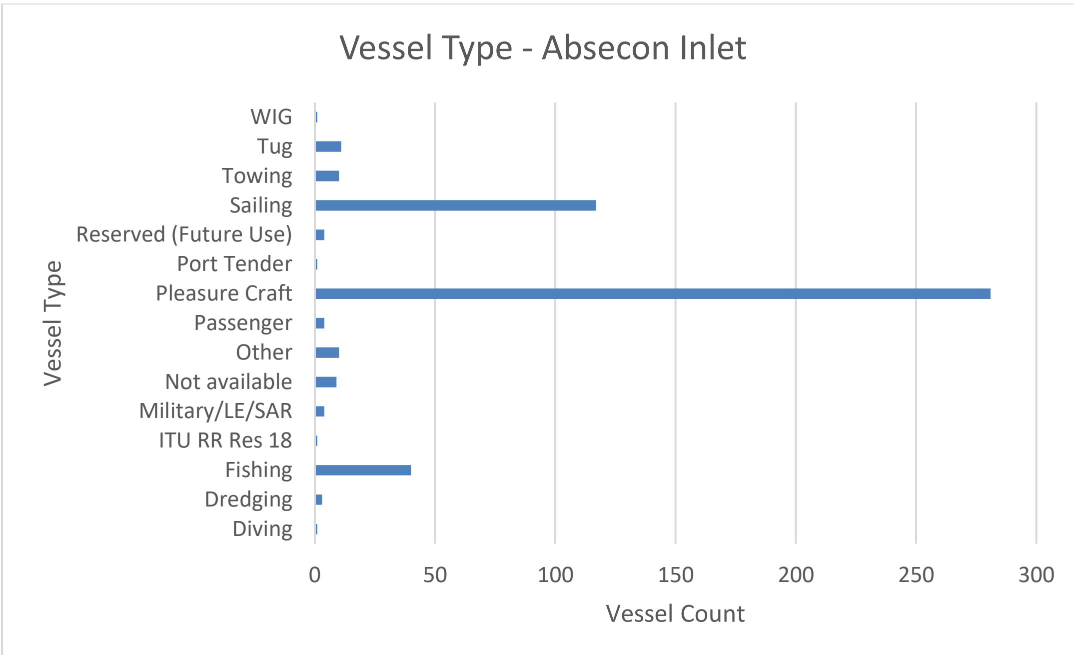


Figure 3-15: Vessel Type – Absecon Inlet

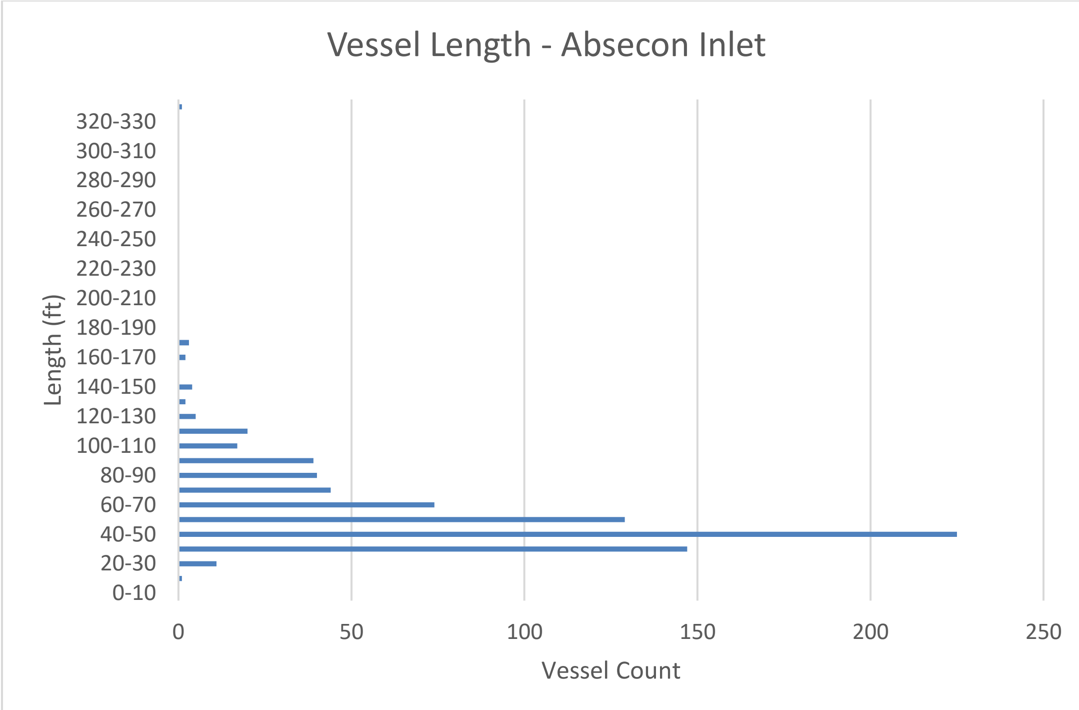


Figure 3-16: Vessel Length – Absecon Inlet

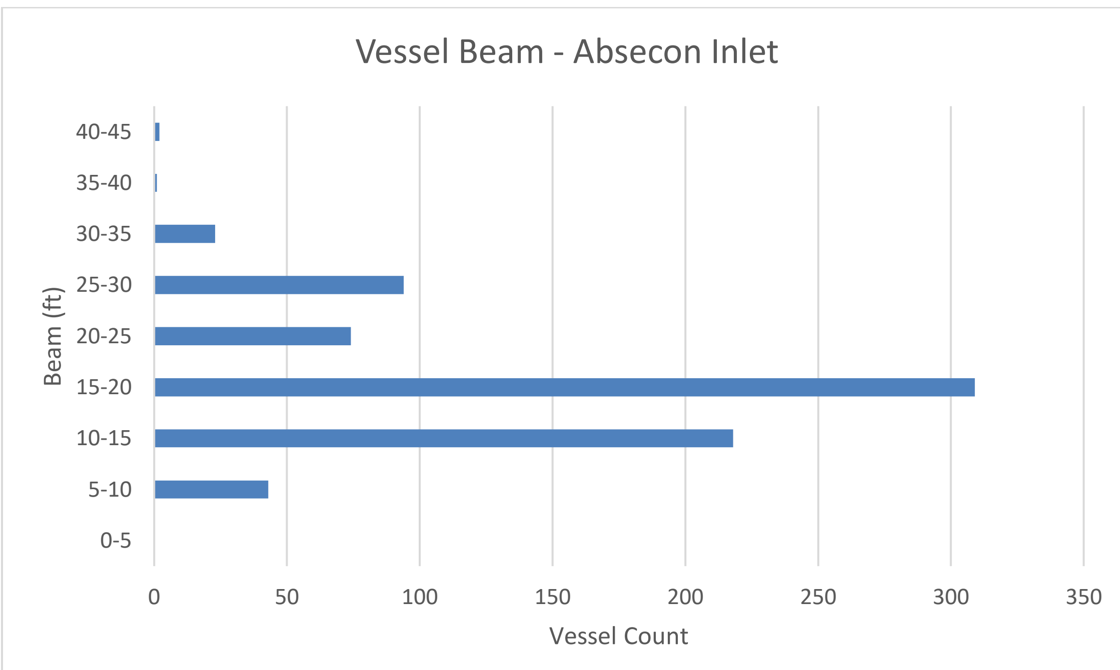


Figure 3-17: Vessel Beam – Absecon Inlet

Minimum Practical Channel Width (Main Navigation)

The largest reported vessel beam was selected as the design vessel for the main navigation gate (as shown in Figure 3-18). The design vessel is approximately 144 feet long with a vessel beam of approximately 43 feet. Multiplying the 43 foot design vessel beam by the most conservative beam multipliers, see Section 3.2.2.2, results in a preliminary minimum channel width recommendation of approximately 237 feet for one-way traffic and 344 feet for two-way traffic.

Figure 3-18: Main Navigation Design Vessel – Absecon Inlet

There is an existing authorized federal navigation channel at Absecon Inlet that is 400 ft. wide. At this phase of the study it is assumed that storm surge barriers must be sized to allow complete access through the entire authorized channel, outside of significant storm events. For this reason, a 420 ft. wide (10 ft. buffer on either side of federal channel) navigable sector gate across Absecon Inlet is currently being used in the focused array of alternatives. This 420 ft. dimension satisfies the minimum channel width recommendation of 344 ft. assuming a 43 ft. wide design vessel and two-way traffic. Brigantine Bridge is also located across Absecon Inlet and constrains navigation widths to approximately 115 ft. An alternative design could propose a storm surge barrier directly seaward of the existing bridge and match the navigation gate width to the existing width of the bridge piers that already constrain navigation.

Minimum Practical Channel Width (Secondary Navigation)

The AIS data, summarized in Figure 3-15 through Figure 3-17, shows a high volume of smaller recreational vessels (pleasure crafts) traversing through Absecon Inlet. The highest number of recorded recreational vessels were in the 15-20 foot beam range. The selected design vessel has a length of approximately 49 feet and a vessel beam of approximately 20 feet (as shown in Figure 3-19). Using the secondary navigation design vessel and the most conservative beam multiplier from Section 3.2.2.2, a minimum recommendation for a secondary channel is approximately 110 feet for one-way traffic and approximately 160 feet for two-way traffic.

Figure 3-19: Secondary Navigation Design Vessel – Absecon Inlet

Further analysis is needed to determine need, location and gate type for secondary navigation. Although there are vertical clearance restrictions to vertical lift gates, they can be designed to support recreational navigation. The current vertical lift gate design in the focused array of alternatives provides a 150 ft. opening and may need to be increased in size to support recreational navigation. An additional sector gate could also be designed to support secondary navigation which results in unlimited vertical clearance.

Vessel Traffic Location

Figure 3-20 displays the vessel position heat map from June 1, 2018 to September 30, 2018 and suggests vessels primarily enter Absecon Inlet through the authorized Federal Navigation channel (located closer to Atlantic City) which is also the deepest portion of the channel.

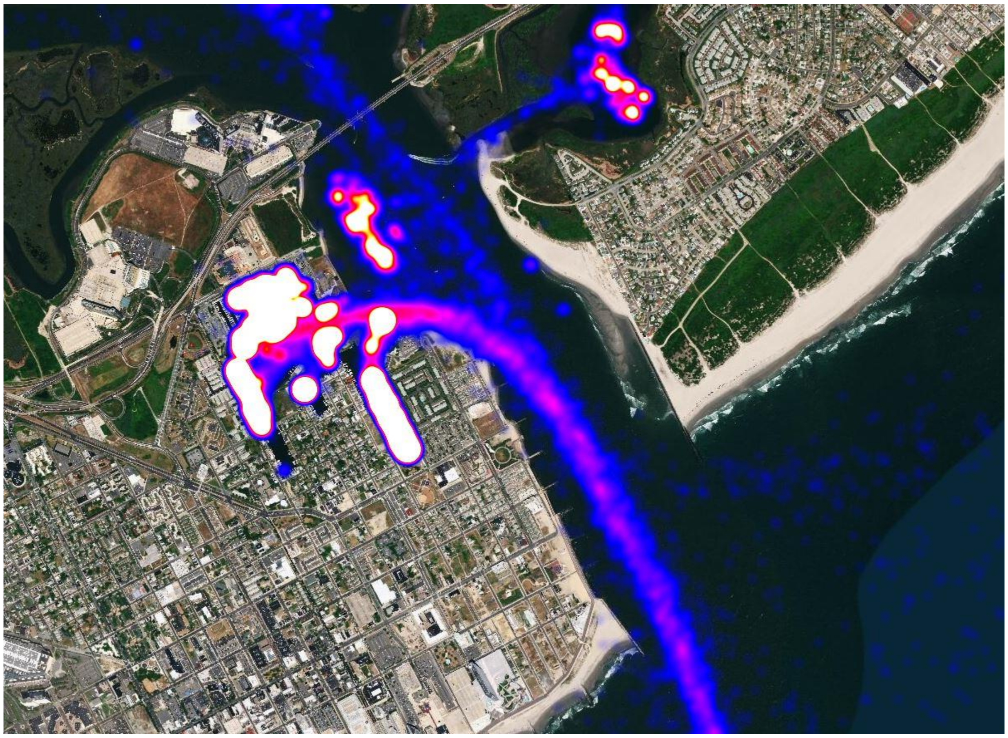


Figure 3-20: Vessel Heat Map (6/1/2018 – 9/30/2018) – Absecon Inlet

Summer month data was collected and analyzed in AISAP using a 5 minute sampling frequency. In order to get a more distinct representation of vessel tracks, higher sampling frequencies are needed. For this reason, 30 second sampling frequencies were used when collecting AIS data for the 4th of July weekend in 2019. The 4th of July weekend was assumed to be one of the busiest time periods of the year that would produce a manageable data set using a 30 second sampling frequency (e.g. higher sampling frequencies result in larger AISAP data outputs). Figure 3-21 represents individual vessel reports recorded every 30 seconds while Figure 3-22 connects the individual reports to illustrate specific vessel traffic lines. The results from this higher frequency sampling rate confirm that the most traffic occurs through the federal navigation channel closer to Atlantic City. Based on these results, the recommended location of the main navigable storm surge barrier gate is across the federal channel. The AIS data also shows that vessels are constrained to the north of the inlet through the Brigantine Bridge piers (approximately 115 ft. apart). Navigation gates could also be proposed adjacent to the bridge and sized to match the existing vessel constraints.

Figure 3-21: Vessel Heat Map (7/4/2019 – 7/8/2019) – Absecon Inlet

Figure 3-22: Vessel Transit Map (7/4/2019 – 7/8/2019) – Absecon Inlet

3.2.2.5 Barnegat Inlet

Vessel Traffic Summary

AIS data was collected and evaluated at Barnegat Inlet from June 1, 2018 to September 30, 2018, representing a total of 121 days. Vessel traffic is assumed to be highest during the summer months and was therefore used to select a representative design vessel. Figure 3-23, Figure 3-24, and Figure 3-25 represent AIS vessel traffic summaries for Barnegat Inlet.

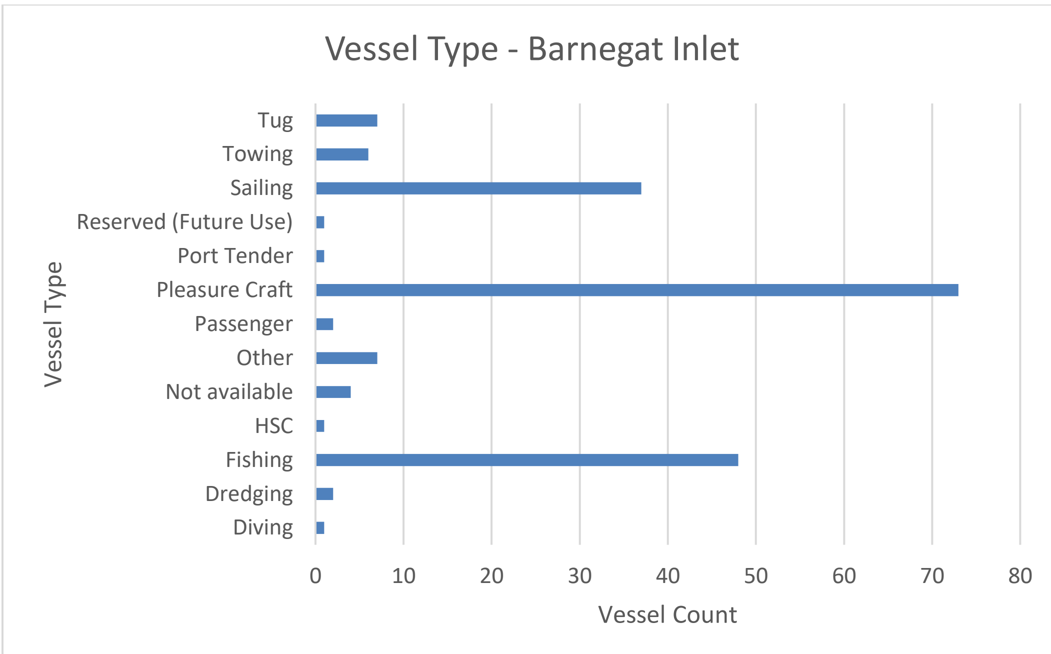


Figure 3-23: Vessel Type – Barnegat inlet

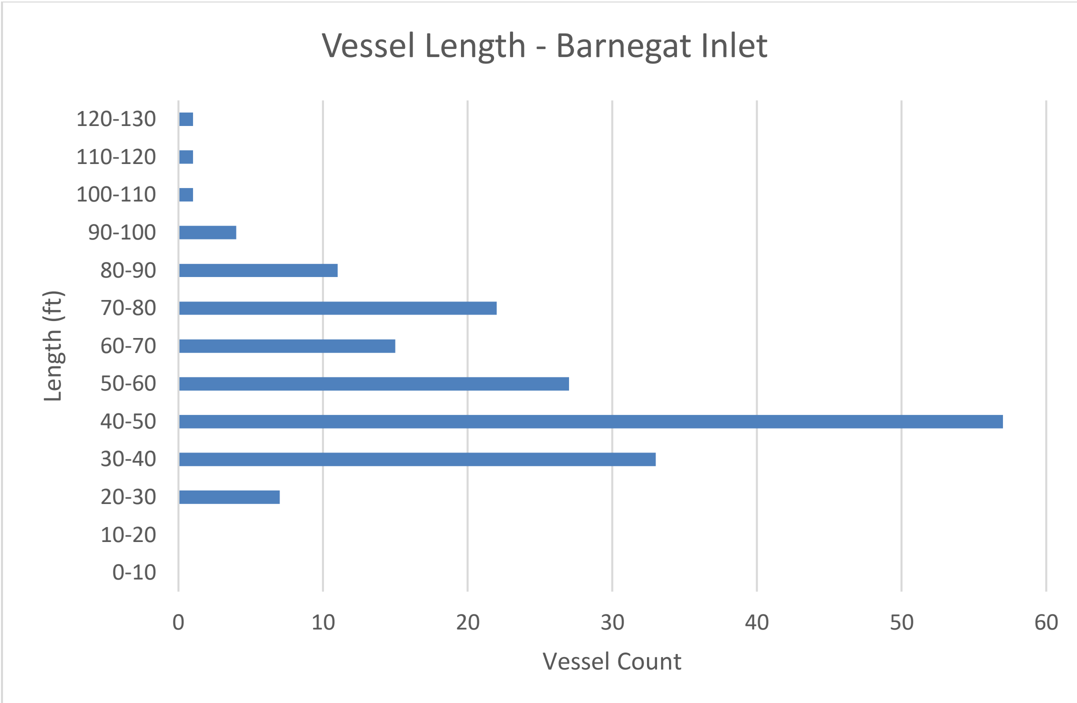


Figure 3-24: Vessel Length – Barnegat inlet

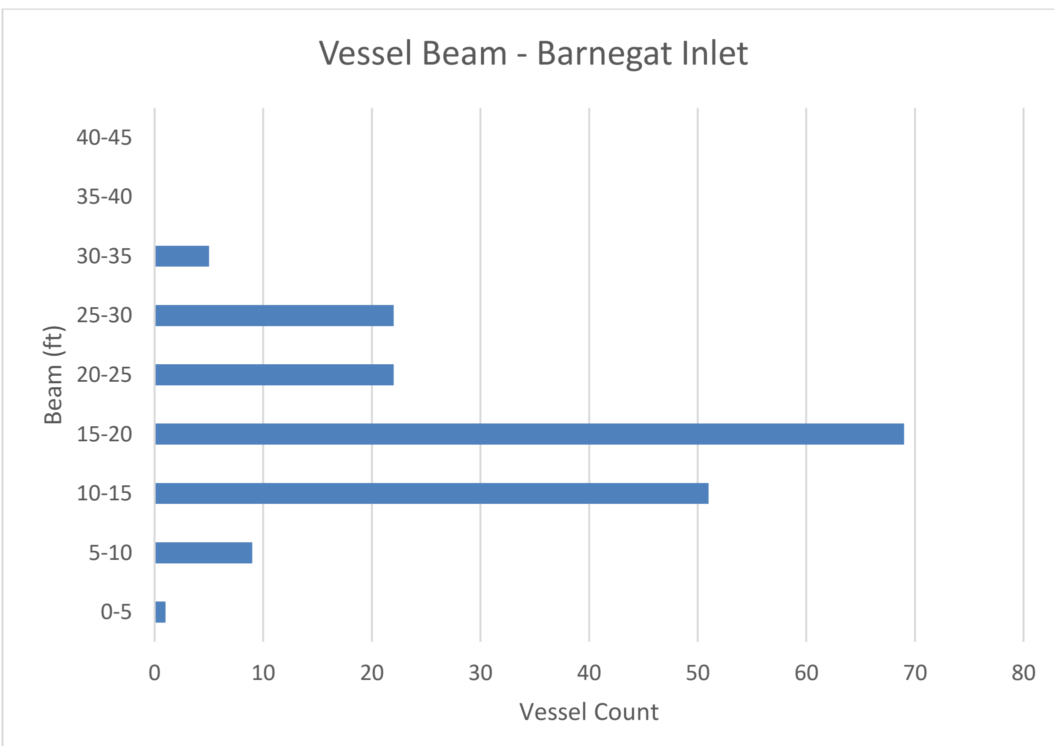


Figure 3-25: Vessel Beam – Barnegat inlet

Minimum Practical Channel Width (Main Navigable Passage)

The largest reported vessel beam was selected as the design vessel for the main navigation gate (as shown in Figure 3-26). The design vessel is approximately 89 feet long with a vessel beam of approximately 33 feet. Multiplying the 33 foot design vessel beam by the most conservative beam multipliers, see Section 3.2.2.2, results in a preliminary minimum channel width recommendation of approximately 182 feet for one-way traffic and 264 feet for two-way traffic.

Figure 3-26: Main Navigation Design Vessel – Barnegat inlet

There is an existing authorized federal navigation channel at Barnegat Inlet that is 300 ft. wide. At this phase of the study it is assumed that storm surge barriers must be sized to allow complete access through the entire authorized channel, outside of significant storm events. For this reason, a 320 ft. wide (10 ft. buffer on either side of channel) navigable sector gate across the channel is currently being used in the focused array of alternatives. This 320 ft. dimension satisfies the minimum channel width recommendation of 264 ft. assuming a 33 ft. wide design vessel and two-way traffic.

Minimum Practical Channel Width (Secondary Navigable Passage)

The AIS data, summarized in Figure 3-23 through Figure 3-25, shows a high volume of smaller recreational vessels (pleasure crafts) traversing through Barnegat Inlet. The highest number of recorded recreational vessels were in the 15-20 foot beam range. The selected design vessel has a length of approximately 53 feet and a vessel beam of approximately 20 feet (as shown in Figure 3-27). Using the secondary navigation design vessel and the most conservative beam multiplier from Section 3.2.2.2, a minimum recommendation for a secondary channel is approximately 110 feet for one-way traffic and approximately 160 feet for two-way traffic.

Figure 3-27: Secondary Navigation Design Vessel – Barnegat inlet

Further analysis is needed to determine need, location and gate type for secondary navigation. Although there are vertical clearance restrictions to vertical lift gates, they can be designed to support recreational navigation. The current vertical lift gate design in the focused array of alternatives provides a 150 ft. opening and may need to be increased in size to support recreational navigation. An additional sector gate could also be designed to support secondary navigation which results in unlimited vertical clearance.

Vessel Traffic Location

Figure 3-28 displays the vessel position heat map from June 1, 2018 to September 30, 2018 and suggests vessels primarily enter Barnegat Inlet closer to the north jetty. Once through the inlet, most vessels make a sharp turn towards Barnegat Lighthouse. This path generally follows the existing authorized federal navigation channel.

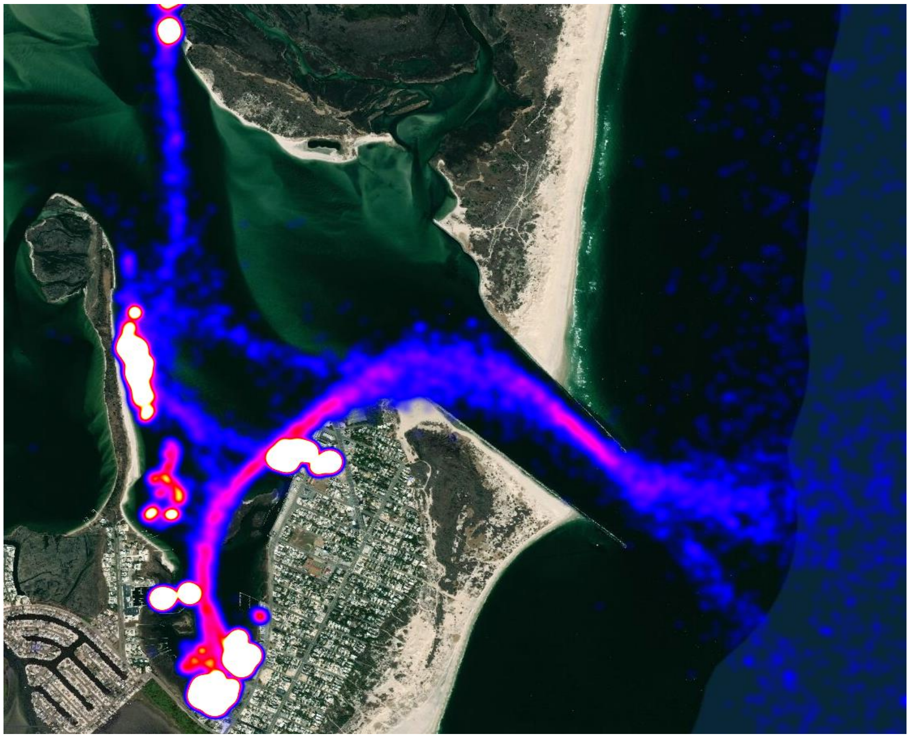


Figure 3-28: Vessel Heat Map (6/1/2018 – 9/30/2018) – Barnegat Inlet

Summer month data was collected and analyzed in AISAP using a 5 minute sampling frequency. In order to get a more distinct representation of vessel tracks, higher sampling frequencies are needed. For this reason, 30 second sampling frequencies were used when collecting AIS data for the 4th of July weekend in 2019. The 4th of July weekend was assumed to be one of the busiest time periods of the year that would produce a manageable data set using a 30 second sampling frequency (e.g. higher sampling frequencies result in larger AISAP data outputs). Figure 3-29 represents individual vessel reports recorded every 30 seconds while Figure 3-30 connects the individual reports to illustrate specific vessel traffic lines. The results from this higher frequency sampling rate confirm that most of the vessels traverse through the inlet closer to the north jetty and then make a sharp turn toward the Barnegat Lighthouse, generally following the navigation channel. The main navigable storm surge barrier should be located across the navigation channel in an area with the high heat intensity

Figure 3-29: Vessel Heat Map (7/4/2019 – 7/8/2019) – Barnegat Inlet

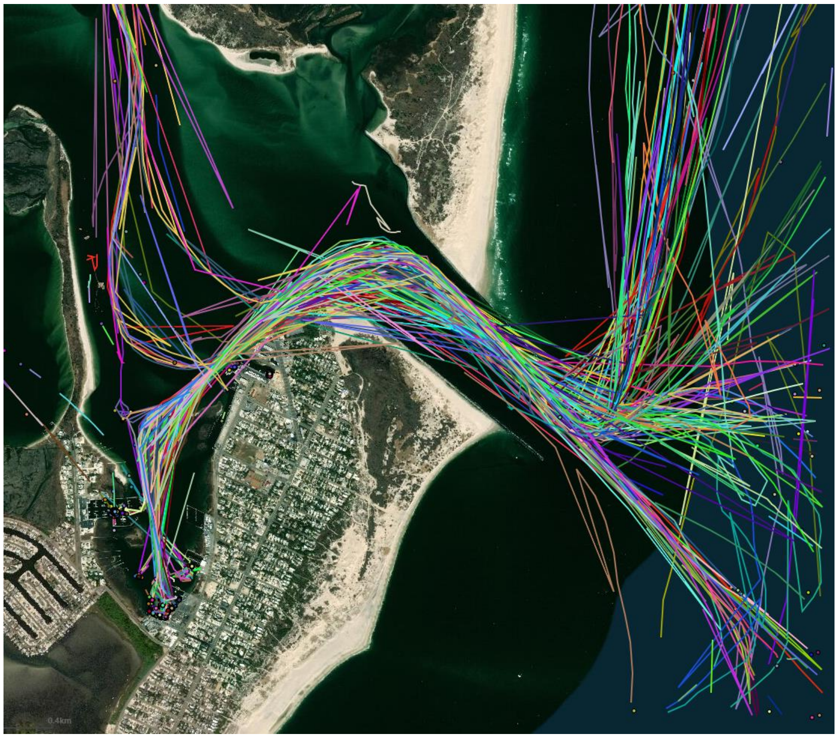


Figure 3-30: Vessel Transit Map (7/4/2019 – 7/8/2019) – Barnegat Inlet

3.2.2.6 Manasquan Inlet

Vessel Traffic Summary

AIS data was collected and evaluated at Manasquan Inlet from June 1, 2018 to September 30, 2018, representing a total of 121 days. Vessel traffic is assumed to be highest during the summer months and was therefore used to select a representative design vessel. Figure 3-31, Figure 3-32, and Figure 3-33 represent AIS vessel traffic summaries for Manasquan Inlet.

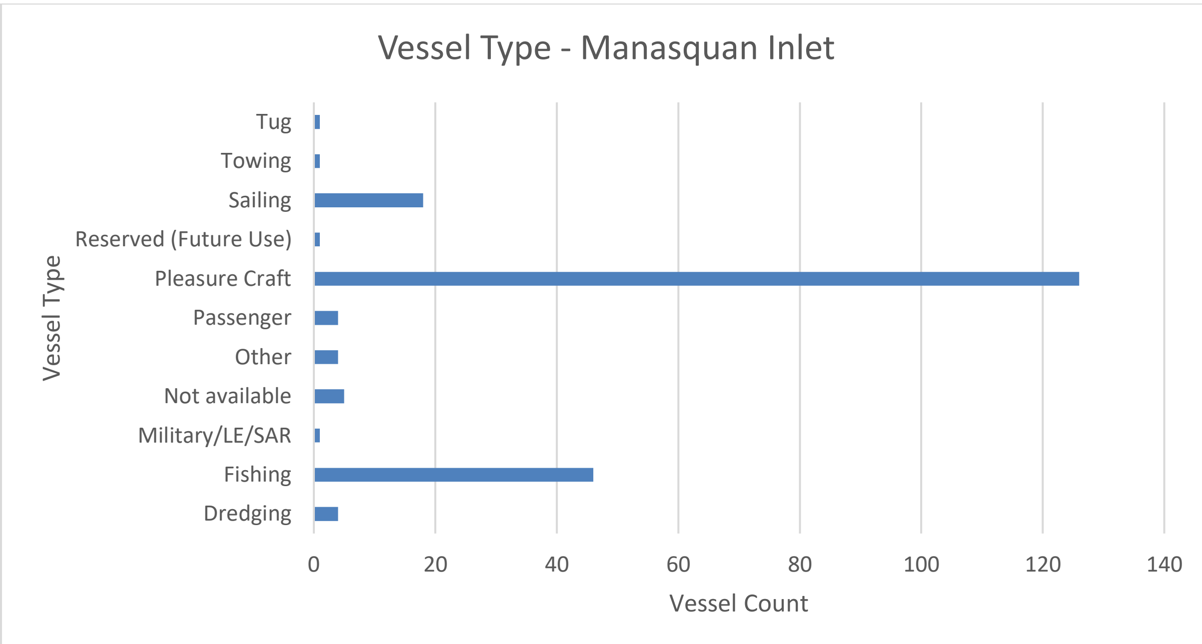


Figure 3-31: Vessel Type – Manasquan Inlet

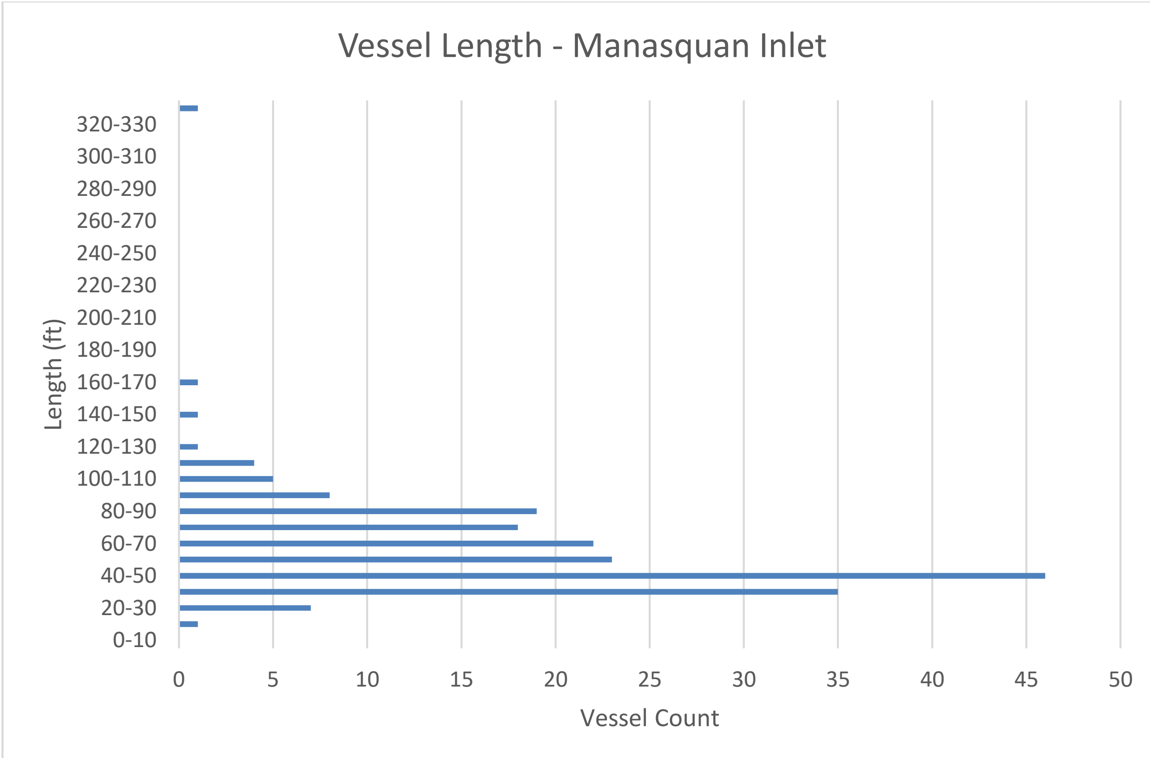


Figure 3-32: Vessel Length – Manasquan Inlet

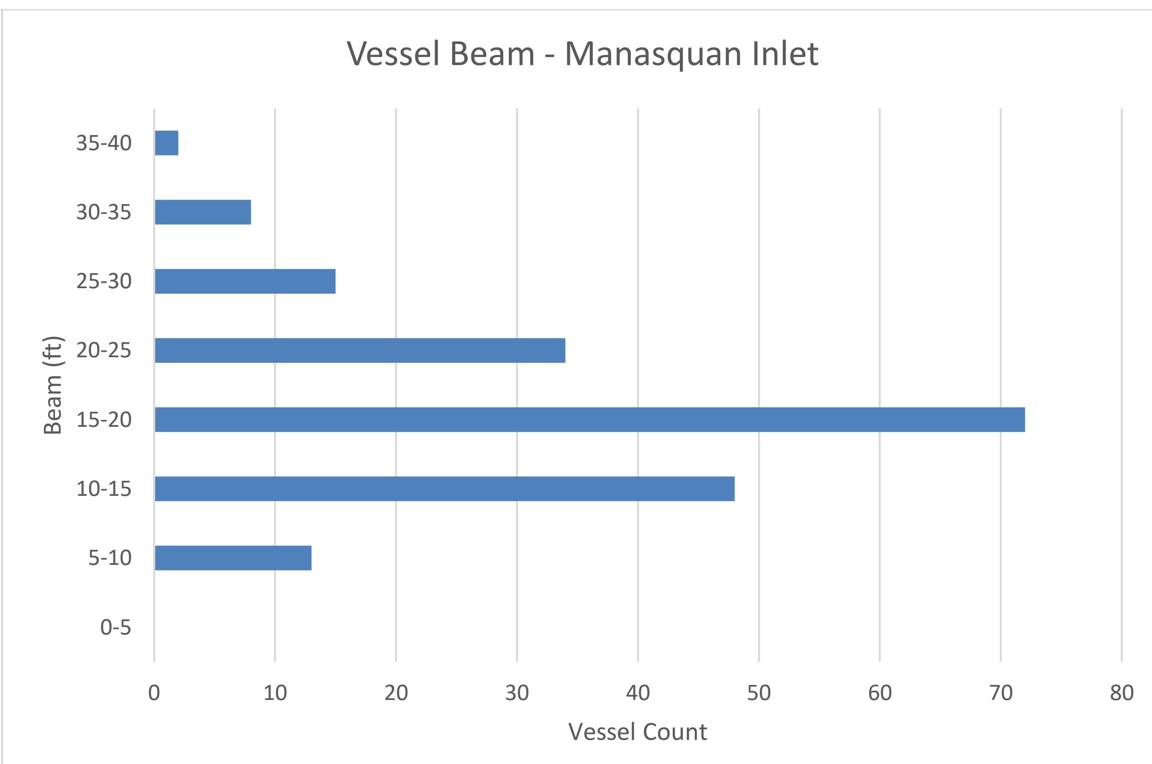


Figure 3-33: Vessel Beam – Manasquan Inlet

Minimum Practical Width (Main Navigable Passage)

The largest reported vessel beam was selected as the design vessel for the main navigation gate (as shown in Figure 3-34). The design vessel is approximately 335 feet long with a vessel beam of approximately 39 feet. Multiplying the 39 foot design vessel beam by the most conservative beam multipliers, see Section 3.2.2.2, results in a preliminary minimum channel width recommendation of approximately 215 feet for one-way traffic and 312 feet for two-way traffic.

Figure 3-34: Main Navigation Design Vessel – Manasquan Inlet

There is an existing authorized federal navigation channel at Manasquan Inlet that varies in size but is approximately 300 ft. wide. At this phase of the study it is assumed that storm surge barriers must be sized to allow complete access through the entire authorized channel, outside of significant storm events. For this reason, a 340 ft. wide navigable sector gate is currently being used in the focused array of alternatives. This 340 ft. dimension satisfies the minimum channel width recommendation of 312 ft. assuming a 39 ft. wide design vessel and two-way traffic. A 340 ft. wide navigation gate would essentially maintain access through the entire inlet, eliminating the need for a secondary navigation gate. The existing width of Manasquan Inlet varies but is approximately 400 ft.

Vessel Traffic Location

Figure 3-35 displays the vessel position heat map from June 1, 2018 to September 30, 2018 using a 5 minute sampling frequency. Figure 3-36 displays the vessel position heat map for the 2019 4th of July weekend at a 30 second sampling frequency. Both figures suggest that vessels traverse all throughout Manasquan Inlet. Vessel positions recorded on land show that there are some latitude and longitude errors in the results. The navigation gate location used in the focused array of alternatives maintains access through the authorized federal navigation channel and essentially maintains the existing navigable width of the inlet.

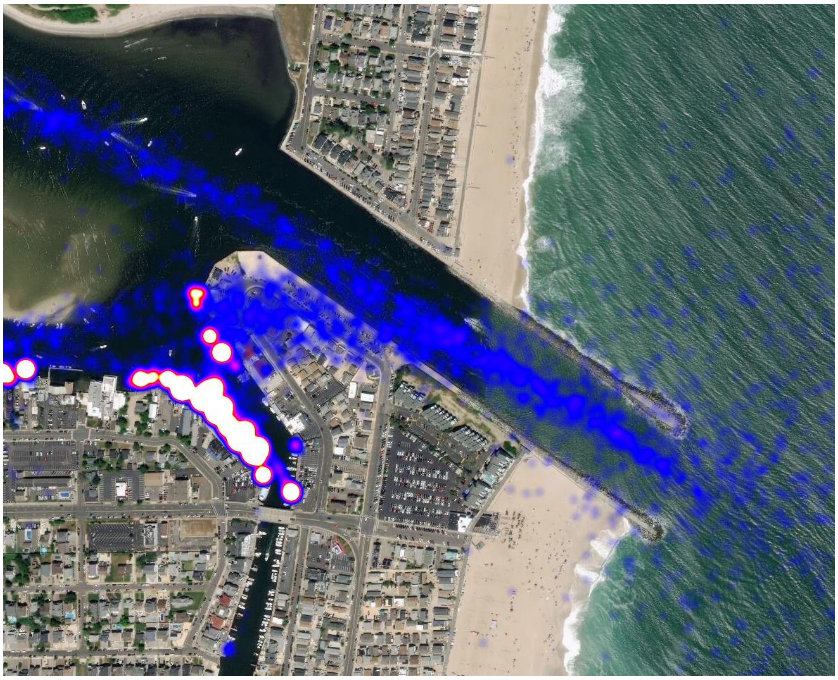


Figure 3-35: Vessel Heat Map (6/1/2018 – 9/30/2018) – Manasquan Inlet

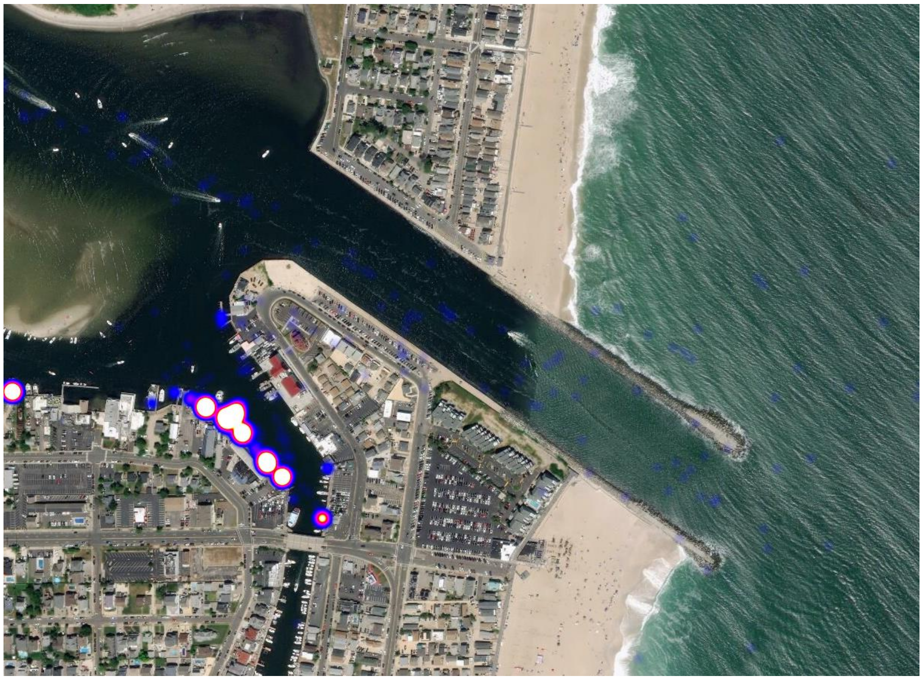


Figure 3-36: Vessel Heat Map (7/4/2019 – 7/8/2019) – Manasquan Inlet

3.2.2.7 Vessel Analysis Summary

This report documents a maritime vessel analysis for Great Egg Harbor Inlet, Absecon Inlet, Barnegat Inlet and Manasquan Inlet. There are proposed storm surge barriers at each one of these inlets within the focused array of alternatives. Design vessels are selected for each inlet based on AIS data. A minimum navigation channel is recommended based on the selected design vessel. Recommendations are preliminary and may be designed larger or smaller to meet specific criteria at each site. For example some navigation gates may need to be larger to maintain access to federal channels or to provide additional conveyance to reduce effects on tidal prism. Some navigation gates may be reduced in size to meet existing navigation constraints such as bridge piers. Table 3-11 provides a summary of the preliminary findings.

Table 3-11: Maritime Vessel Analysis Summary

Location	Main Navigation Design Vessel Beam (ft)	Main Navigation Minimum Opening (ft)	Seconday Navigation Design Vessel Beam (ft)	Seconday Navigation Minimum Opening (ft)
Great Egg Harbor Inlet	39	312	13	104
Absecon Inlet	43	344	20	160
Barnegat Inlet	33	264	20	160
Manasquan Inlet	39	312	N/A	N/A

Guidance from both EM 1110-2-1613 and PIANC Report No. 121 recommend ship maneuvering simulations (numerical models) be carried out in the detailed design phase to refine the preliminary design widths and to quantify the safety and risk level of the final channel width.

Additional factors that need to be considered in a vessel analysts include, but are not limited to; future design vessels, one-way vs. two-way traffic, wind and wave effects, visibility, navigation aids, currents, speed of design ship, project costs and vessel traffic intensity.

3.2.3 Cycle 3 Results

Cycle 3 Quantities, as shown in Table 3-12, were measured from the SSB Cycle 3 Screening drawings (see Appendix B.6). Information and results from Sections 3.2.1 and 3.2.2 were used to develop the Cycle 3 SSB designs. Various design parameters (gate alignment, sill elevation, number of gates, etc.) were investigated for each barrier location to evaluate indirect impacts on tides, velocities, salinity and residence time through the ERDC-CHL AdH Model. Due to scheduling constraints, the A1 alignments were selected for the Cycle 3 screening prior to receiving the AdH Model results. A1 alignments promoted more flow compared to other model runs and were assumed to have the smallest environmental impacts. The alignments, as well as other design parameters, may be refined post-TSP in order to optimize the design and minimize indirect impacts.

Table 3-12: Storm Surge Barrier Cycle 3 Quantities

Barrier Components	Storm Surge Barrier Locations					
	Manasquan Inlet - A1	Barnegat Inlet - A1	Absecon Inlet - A1	Great Egg Harbor Inlet - A1	Absecon Bay Closure - A1	Ocean City Bay Closure - A1
Navigable Gate Pier Area (SF)	14411	14893	27865	19149	4787	3457
Navigable Gate Moveable Area (SF)	13005	13440	25200	17280	4320	3120
Navigable Gate Total Area (SF)	27416	28333	53065	36429	9107	6577
Aux. Flow Gate Pier Area (SF)	0	12504	5310	19230	0	0
Aux. Flow Gate Moveable Area (SF)	0	54327	20250	84900	0	0
Aux. Flow Gate Total Area (SF)	0	66831	25560	104130	0	0
Impermeable Barrier Area (SF)	0	18365	7033	20716	14772	2906
Seawall Length (FT)	2366	795	2569	1275	0	0
Sluice Gate (SF)	0	3456	0	0	4272	768
Floodwall Type A Length (FT)	7280	0	0	974	27524	9467
Floodwall Type B Length (FT)	0	0	0	0	5193	1205
Floodwall Type C Length (FT)	0	897	0	0	5503	2919
Floodwall Type D Length (FT)	0	0	0	0	18194	0
Levee/Seawall Outfall	12	1	3	3	45	15
Floodwall Outfall	0	2	0	0	72	10
Road Closure (EA)	0	1	0	0	4	0

Notes:

1. *Navigable gate total area and auxiliary flow gate total area is the cross sectional surface area of the dynamic (moveable) span of barrier plus the cross sectional surface area of the housing structure associated with the gate.*
2. *The impermeable barrier area is the cross sectional surface area of the impermeable barrier.*

4 NON-STRUCTURAL

12 Groups considered "Possible" (BCR between 1.0 and 2.0), and 25 Groups considered "Screened Out" (BCR below 1.0) from the initial perimeter plan screening were not included for analysis in the perimeter plan cycle 2 screening. These areas, however, are appropriate for non-structural solutions. Raising structures (primarily residential) to elevate the first floor above the design flood level was the only non-structural solution considered for this phase of the screening process. Figure 4-1 below shows a graphic representation of this 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.

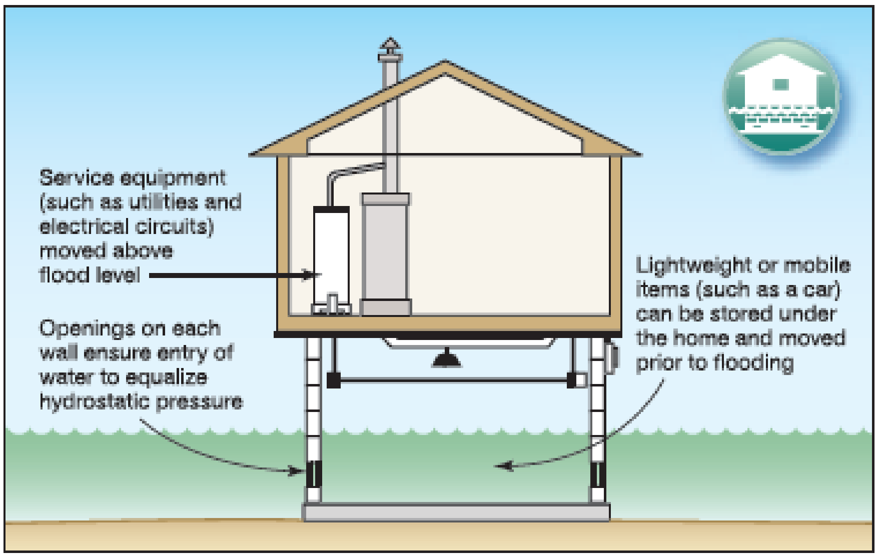


Figure 4-1: Non-Structural Flood Control Solution

Nonstructural measures fall into four broad groups resulting from the CSRM Inventory and Screening process including:

• Managed Coastal Retreat including Acquisition / Relocation,

• Building Retrofit (flood proofing, elevations, ring levees),

• Land Use Management (zoning changes, undeveloped land preservation), and

• Early Flood Warnings (evacuation planning, emergency response systems).

Detailed nonstructural analyses results can be found in Appendix D: Nonstructural.

5 NATURAL AND NATURE BASED FEATURES (NNBF) AND ENGINEERING WITH NATURE (EWN)

A qualitative screening effort was initially completed to identify perimeter plan and SSB areas for possible NNBF sites and measures. As a result, the array of measures was screened down to focus primarily on living shorelines and EWN (Engineering with Nature) modifications. Living shorelines may be created in areas where protection incorporates a dune and beach fill or along a levee frontage. EWN features, such as textured concrete, habitat benches, and ecologically enhanced revetments, can be incorporated into the design of floodwall and levee structures (See Figure 5-1 below). Preliminary costs of these items are considered to be within the contingency values for construction of the flood control feature. Subsequent to the initial screening effort USACE Philadelphia District partnered with our Engineering and Research Development Center (ERDC) to evaluate the effectiveness of NNBF and determine other opportunities for NNBF in the project. An initial suite of NNBF opportunities were identified by ERDC for each of the NJBB Regions (see Section 8.3.4 of the Main Report). A complete discussion of the entire range of NNBF strategies considered can be found in the Natural and Nature-Based Features Appendix inclusive of key design concepts which are documented in the latter sections of that Appendix.

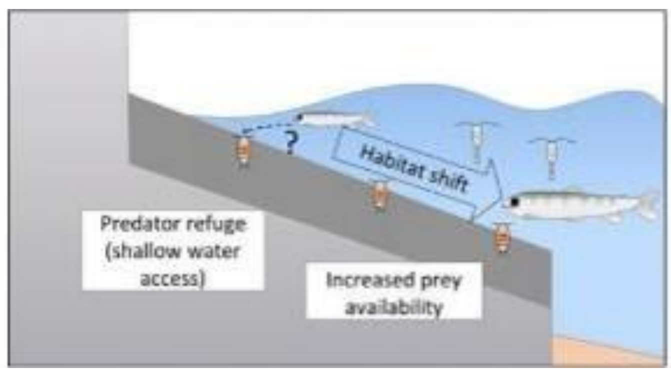


Figure 5-1: EWN Examples of Textured Concrete (left) and Habitat Bench (right)

6 REAL ESTATE

The Cycle 2 level Real Estate costs for the perimeter plan and SSB screening were estimated as a percentage of construction costs (refer to the Cost Estimating Technical Appendix). The Cycle 3 analysis includes quantification of permanent easement acreages based upon the proposed structure footprint and interior drainage modifications including required maintenance access, and temporary easement based upon required access during construction. Preliminary Real Estate acreage requirements, I.E. permanent and temporary easement limits, were computed for all the structures, and provided to Baltimore District Real Estate. (See Table 6-1 and Table 6-2 below).

Table 6-1: PP Real Estate Cycle 3 Quantities

NJBB PERIMETER PLAN EASEMENT AREA CALCULATION									
WALL TYPE	TEMPORARY EASEMENT AREA (SF)	TEMPORARY EASEMENT AREA (AC)	% OF TOTAL	PERMANENT EASEMENT AREA (SF)	PERMANENT EASEMENT AREA (AC)	% OF TOTAL	TOTAL AREA (SF)	TOTAL AREA (AC)	REFERENCE DETAIL
A	1428990	32.8	8%	15735610	361.2	92%	17164600	394.0	Sheet 1
B	4497297	103.2	19%	19406293	445.5	81%	23903590	548.7	Sheet 2
C	1154827	26.5	20%	4676888	107.4	80%	5831715	133.9	Sheet 3
D	1747559	40.1	20%	7089746	162.8	80%	8837305	202.9	Sheet 4
TOTALS	8828673	**203**		46908537	**1077**		55737210	**1280**	

Table 6-2: SSB Real Estate Cycle 3 Quantities

	WALL/BARRIER TYPE	TEMPORARY EASEMENT AREA (SF)	TEMPORARY EASEMENT AREA (AC)	% OF TOTAL	PERMANENT EASEMENT AREA (SF)	PERMANENT EASEMENT AREA (AC)	% OF TOTAL	TOTAL AREA (SF)	TOTAL AREA (AC)
GREAT EGG HARBOR INLET	SEAWALL	12774	0.3	36%	146898	3.2	37%	159672	3.5
	IMPERMEABLE BARRIER	11140	0.2	32%	128106	2.8	32%	139246	3.1
	TYPE A	11167	0.2	32%	121354	2.7	31%	132521	2.9
	TOTALS	**35081**	**0.8**		**396358**	**8.7**		**431439**	**9.5**
SOUTHERN OCEAN CITY BAY CLOSURE	TYPE A	96404	2.1	70%	1101480	24.3	87%	1197884	26.4
	TYPE B	12047	0.3	9%	51802	1.1	4%	63849	1.4
	TYPE C	29536	0.7	21%	117142	2.6	9%	146678	3.2
	TOTALS	**137987**	**3.0**		**1270424**	**24.3**		**1408411**	**31.0**
ABSECON INLET	SEAWALL	25690	0.57	92%	295438	6.5	80%	321128	7.1
	SECTOR GATE	2322	0.05	7%	72037	1.6	18%	74359	1.6
	TOTALS	**28012**	**0.6**		**367475**	**8.1**		**395487**	**8.7**
ABSECON BAY CLOSURE	TYPE A	262177.0	5.8	46%	3015021	66.5	70%	3277198	72.2
	TYPE B	68880	1.5	12%	296211	6.5	7%	365091	8.0
	TYPE C	54983	1.2	10%	242930	5.4	6%	297913	6.6
	TYPE D	181927	4.0	32%	727709	16.0	17%	909636	20.1
	TOTALS	**567967**	**12.5**		**4281871**	**94.4**		**4849838**	**106.9**
BARNEGAT INLET	SEAWALL	7798	0.2	47%	89676	2.0	71%	97474	2.1
	TYPE C	8967	0.2	53%	35869	0.8	29%	44836	1.0
	TOTALS	**16765**	**0.4**		**125545**	**2.8**		**142310**	**3.1**
MANASQUAN INLET	SEAWALL	23666	0.5	24%	272142	6.0	23%	295808	6.5
	SECTOR GATE	4558	0.1	5%	99527	2.2	8%	104085	2.3
	TYPE A	71900	1.6	72%	837200	18.5	69%	909100	20.0
	TOTALS	**100124**	**2.2**		**1208869**	**26.7**		**1308993**	**28.9**

For the perimeter plan, 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 approximate, and will be better developed in the post TSP design. Figures 6-1 and 6-2 show the minimum allowable dimensions of vegetation-free zone for a levee and floodwall. Figures 6-3 through 6-6 show the preliminary limits of permanent and temporary easement for each of the proposed perimeter flood protective structures.

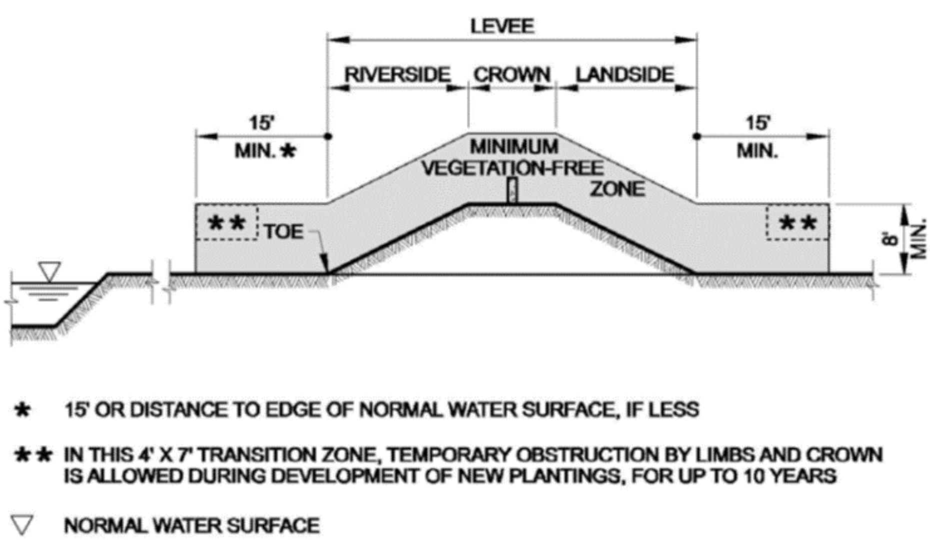
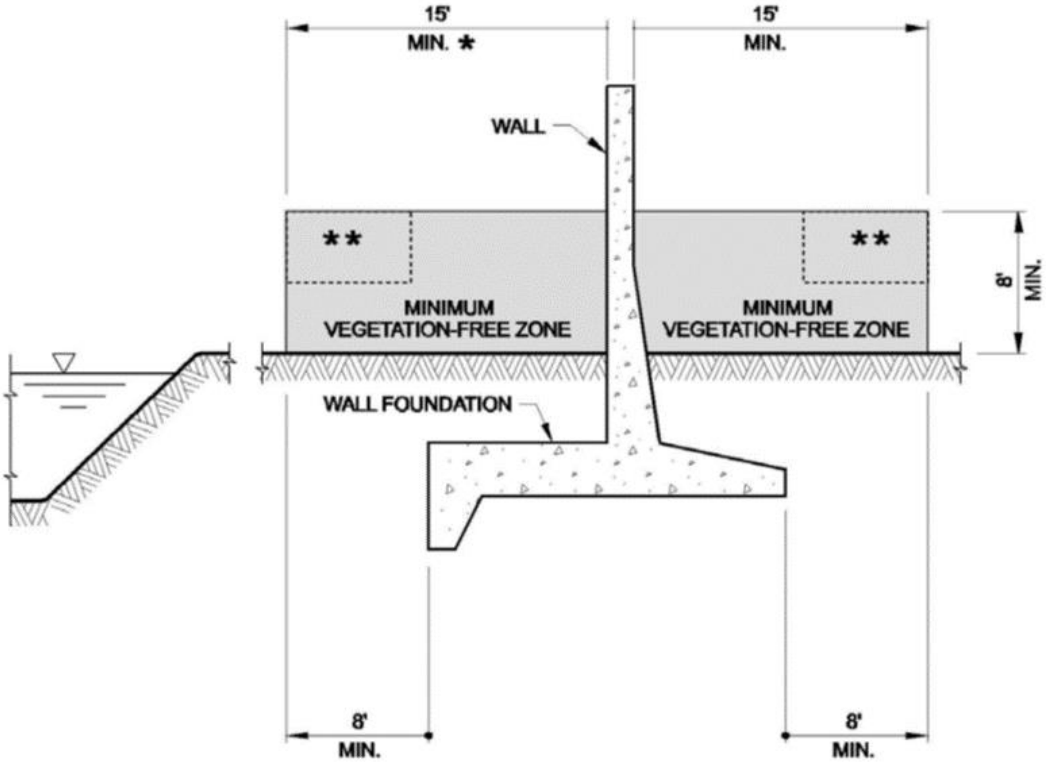


Figure 6-1: Vegetation-Free Zone at Levee



* 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 6-2: Vegetation-Free Zone at Floodwall

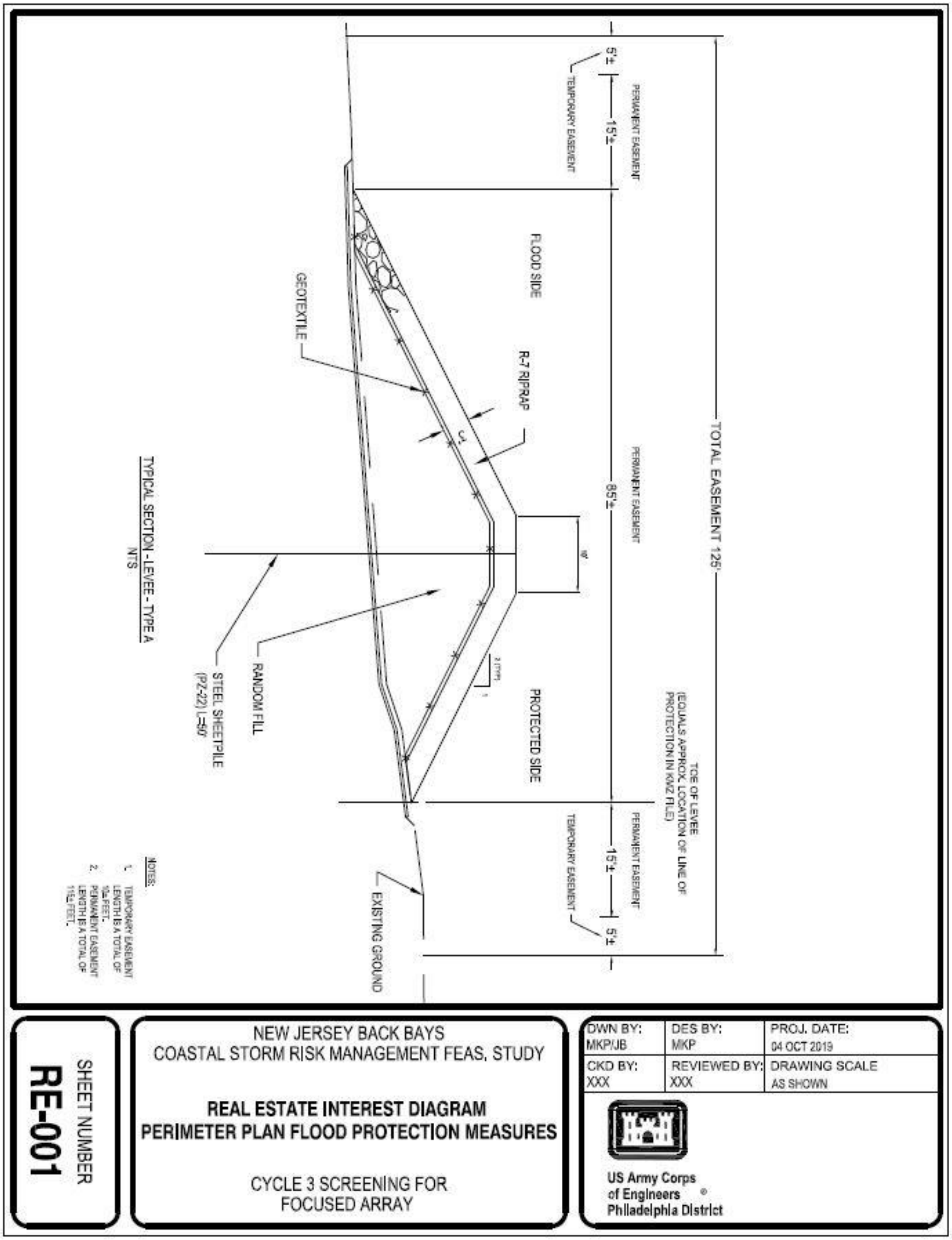


Figure 6-3: Typical Section with Real Estate - Type A Levee

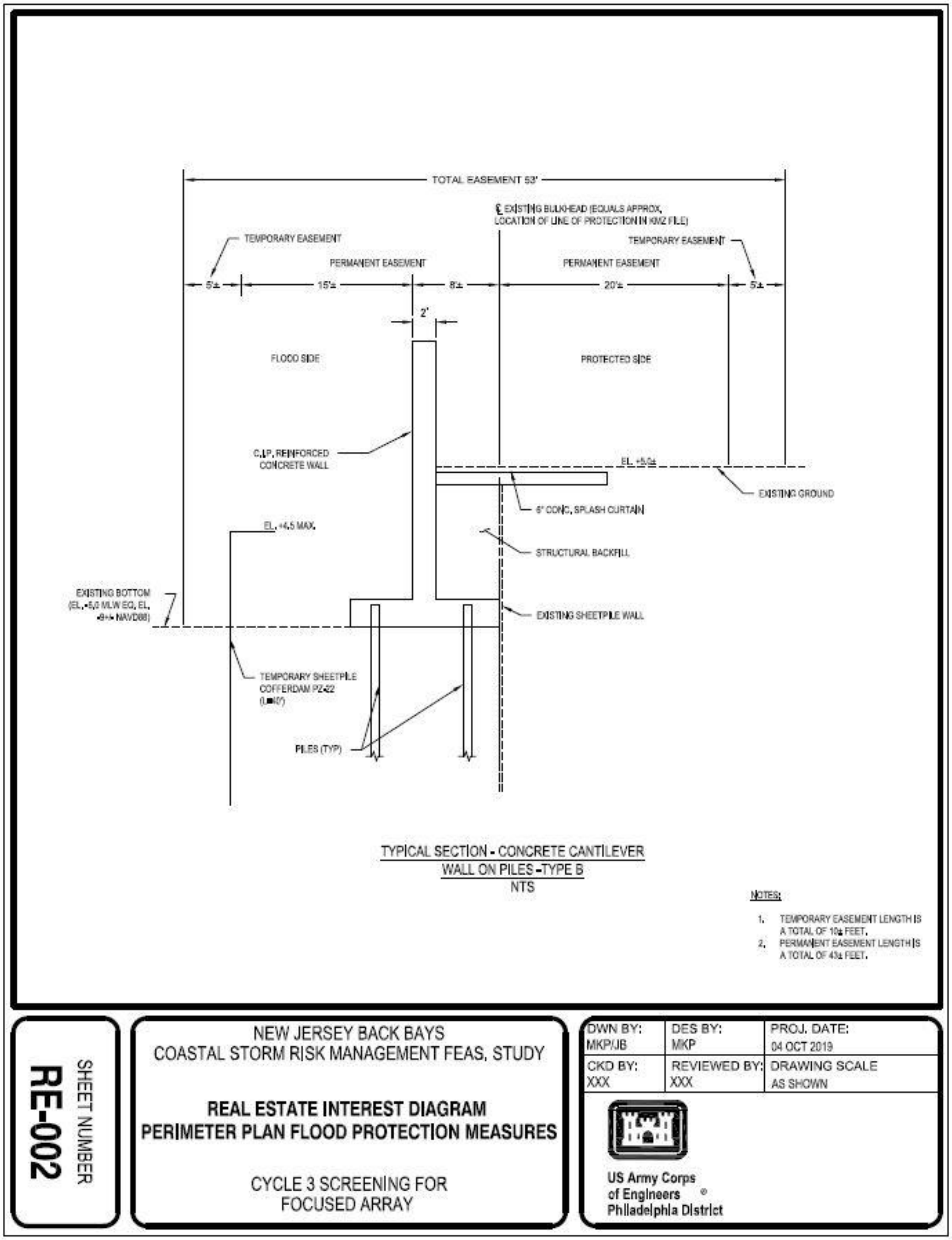


Figure 6-4: Typical Section with Real Estate - Type B Wall

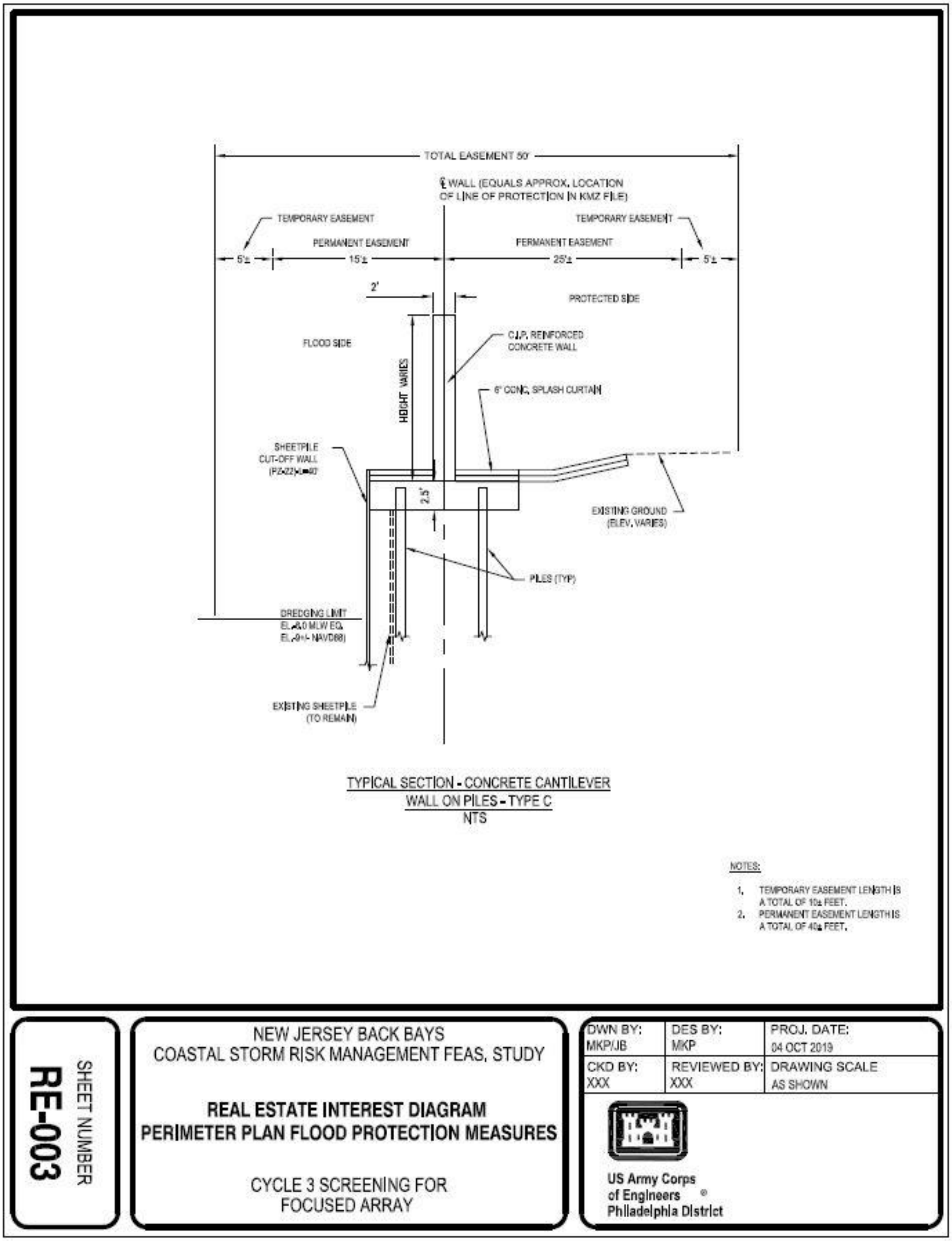


Figure 6-5: Typical Section with Real Estate - Type C Wall

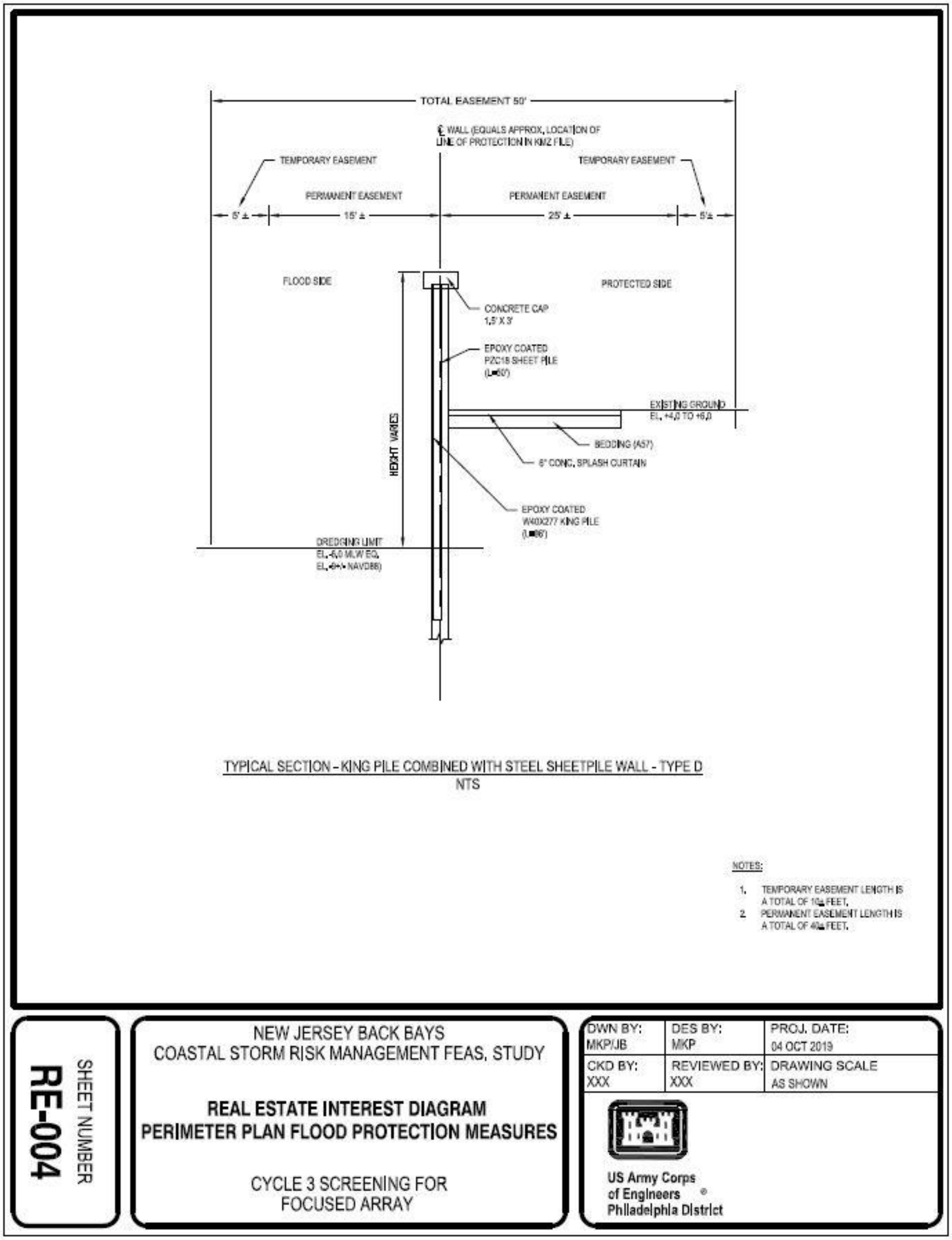


Figure 6-6: Typical Section with Real Estate - Type D Wall

ENGINEERING APPENDIX
GEOTECHNICAL

NEW JERSEY BACK BAYS
COASTAL STORM RISK MANAGEMENT
FEASIBILITY STUDY

PHILADELPHIA, PENNSYLVANIA

APPENDIX B.2

August 2021

NJBB
NEW JERSEY BACK BAYS STUDY

U.S. Army Corps of Engineers
Philadelphia District

THIS PAGE HAS BEEN INTENTIONALLY LEFT BLANK

Table of Contents

List of Figures

List of Tables

1 INTRODUCTION

This appendix presents the results of the Geotechnical engineering evaluation and analysis for the New Jersey Back Bays (NJBB) Coastal Storm Risk Management (CSRM) Study. This report will discuss in detail the existing subsurface information that was collected and reviewed and how that information was used in the formulation of soil properties and strength characteristics. Those characteristics were utilized to determine the preliminary foundation design for alternative structures to get to the TSP-IPR Milestone and Focused Array for the study. See Figure 3-1 for the projects limits.

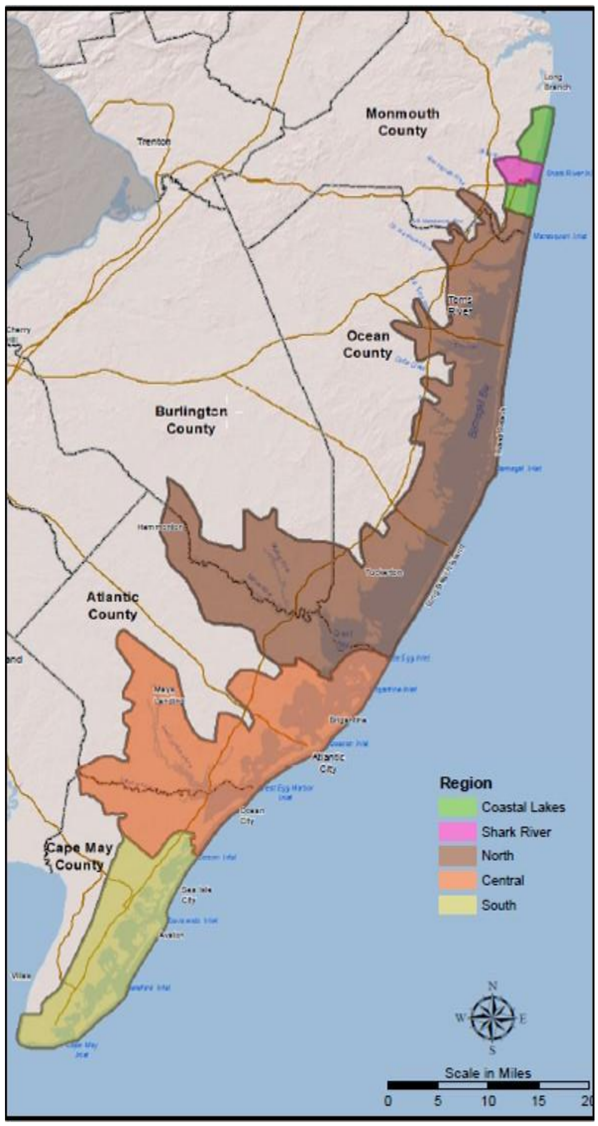


Figure 1-1: NJBB CSRM Feasibility Study Project Limits

2 EXISTING FIELD EXPLORATION DATA

In a preliminary overview of the NJBB Study Area, a search of existing subsurface data from previous geotechnical investigations was conducted. Existing subsurface investigation data consisting of field boring logs and laboratory testing was obtained from US Army Corps of Engineers (USACE) archive data, specifically from the New Jersey (N.J.) Inlets and Beaches project and Chelsea Heights Pump Station replacement project. Existing subsurface investigation data consisting of boring location plans and borings logs was also obtained from the New Jersey Department of Transportation (NJDOT) Geotechnical Data Management System (GDMS) data base. The following sections detail the relevance of the existing subsurface investigations used from each source.

The geotechnical investigations conducted as part of the N.J. Inlets and Beaches were performed in 1964 in the following areas: Corson's Inlet between Strathmere and Ocean City, NJ, Townsends Inlet between Avalon and Sea Isle City, NJ, and Hereford Inlet between Wildwood and Stone Harbor, NJ. The boring location plans with the exact locations of the existing borings are not available; however, the approximate investigation areas are known. The subsurface profile generally consisted of (in descending order): 1) granular soils with intermittent fine-grained soils and with organics extending to depths ranging from 13.0 feet to 44.0 feet, 2) organic fine-grained soils extending to depths ranging from 19.0 to 40.0 feet, and 3) granular soils extending to depths ranging from 30.0 to 50.0 feet (bottoms of the borings). The soils encountered are in general agreement with the published geologic data. The existing boring logs from these investigations can be found as Attachment 1 "U.S. Army Corps of Engineers Geotechnical Investigations N.J. Inlets and Beaches Boring Logs and Laboratory Testing" (Attachment 1) to this Appendix B.2.

Subsurface data was also collected from the geotechnical exploration and evaluation study as part of an overall project for Chelsea Heights Pump Station replacement and related sanitary sewer improvements in Atlantic City, NJ. The exploration was performed in March of 2002 on the north end of Annapolis Ave. in Atlantic City, NJ. The subsurface profile generally consisted of (in descending order): 1) granular fill extending to depths ranging from 6.0 feet to 8.0 feet, 2) organic fine-grained soils extending to depths ranging from 26.0 to 36.5 feet, and 3) granular soils extending to depths ranging from 29.0 to 42.0 feet (bottoms of the borings). The soils encountered are in general agreement with the published geologic data. The boring location plan, boring logs, and laboratory data can be found as Attachment 2 "Chelsea Heights Pump Station Boring Logs and Laboratory Testing" (Attachment 2) to this Appendix B.2.

The subsurface investigation data obtained from NJDOT GDMS data base contained boring location plans and boring logs from various NJDOT projects spanning Ocean City to Manasquan in relative close proximity to the major NJBB CSRM Feasibility Study alternative structures. The projects included bridge, approach, and state route structure subsurface investigations. Representative borings, based on their respective locations and depths, were included in the subsurface data gathering. The representative borings were drilled as recent as 2002 and as far back as 1973. The subsurface profile generally consisted of (in descending order): 1) granular soils with intermittent fine-grained soils and with organics extending to depths ranging from 0.0 feet to 69.0 feet, 2) organic fine-grained soils extending to depths ranging from 5.0 to 72.0 feet, and 3) Intermittent granular and fine-grained soil layers extending to depths ranging from 30.0 feet to 121.5 feet (bottoms of the borings). The soils encountered are in general agreement with the published geologic data. See Table 2-1 for a list of the representative Boring IDs selected for review and respective Project/Boring Location Plans. The existing Boring Location Plans and boring logs from these investigations can be found as Attachment 3 "New Jersey Department of Transportation Geotechnical Data Management System Boring Location Plans and Boring Logs" (Attachment 3) to this Appendix B.2.

Table 2-1: NJDOT Project with Associated Representative Boring Log ID

Boring Location Plan	Location	Boring IDs
Route 47 – Section 1F, George A. Redding Bridge, As-Drilled Location Plan	Lower Township, NJ Wildwood City, NJ	B-12, B-17, B-18, B-20, S-97, S-102, S-151, S-155
Reconstruction of Avalon Blvd. Plan of Test Borings	Avalon, NJ	B-17, B-29, B-34, B-35
Longport Bridge Replacement, Boring Location Plan	Ocean City, NJ	PB-5, PB-6, PB-16
Middle Thorofare Bridge & Approaches, General Plan and Elevation	Ocean City, NJ Upper Township, NJ	421W-16, 421W-17, 421W-18, 421W-19
Rainbow Thorofare Rehabilitation, General Plan & Elevation	Ocean City, NJ	420W-1
Bridges Over Inside, Beach, and Great Thorofares, As-Drilled Boring Plan	Atlantic City, NJ	S-7, S-8, S-17
Routes 30 & 87, Marina District Highway Improvements, Soil Borings	Atlantic City, NJ	O-1, O-15
Route 152 Somers Point to Longport, Boring Layout	Longport, NJ	D-36, D-37
Fox Island Creek, Route 72, Boring Location Plan	Long Beach Island, NJ	63-813-A, 63-813-B
Route 71 Section 3C, Boring Plan Location	Manasquan, NJ	234W-4, 234W-5

The existing borings collected spanned various projects from Cape May Inlet to Manasquan. The borings at each location were generally consistent in terms of the type of materials encountered, the variation in thickness of fine-grained soil layers, and the range of densities of their respective layers. For the purposes of this stage in the TSP, one representative boring was selected to be used in determining soil parameters for the foundation design of the alternative structures. The criteria in selecting a representative boring was to achieve the most conservative subsurface soil profile due to the variable nature of the New Jersey back bay soils and vast stretches of the potential structural solutions of this project. Of the borings considered, Boring O-15 drilled as part of the *Route 30 & 87* project in Atlantic City, NJ collected from the NJDOT GDMS data base was selected. Boring O-15 was drilled to a depth of 90.9 feet below existing ground surface on April 19, 1984 on Huron Ave. between Absecon Blvd. and Brigantine Blvd. in Atlantic City. Refer to Section 3.2 for a detailed description of the soils encountered and results of Standard penetration tests performed in Boring O-15.

3 SUBSURFACE CONDITIONS

3.1 Published Geologic Data

3.1.1 Geomorphology

The study area is situated along the New Jersey coast, which is located within the New Jersey section of the Coastal Plain Physiographic Province of Eastern North America. In New Jersey, the Coastal Plain Province extends from the southern terminus of the Piedmont Physiographic Province southeastward for approximately 155 miles to the edge of the Continental Shelf. The boundary between the rock units of the Piedmont and unconsolidated sediments of the Coastal Plain Physiographic Provinces is known as the Fall Line, which extends southwest across the state from Perth Amboy through Princeton Junction to Trenton. It is termed the Fall Line due to its linearity and the distinct elevation change that occurs across this border between the more rugged, generally higher rock terrain of the Piedmont and generally lower terrain of the soil materials comprising the Coastal Plain. The locations of the Physiographic Provinces in New Jersey and Fall Line are shown on Figure 3-2.

The Coastal Plain Province, lying southeast of the Fall Line, is part of the Atlantic Coastal Plain that extends along the entire eastern Atlantic Ocean coastline from Newfoundland to Florida. The Coastal Plain is the largest physiographic province in the state and covers approximately sixty percent of the surface area of New Jersey. This province encompasses an area of approximately 4,667 square miles, almost 3 million acres. More than half of the land area in the Coastal Plain is below an elevation of 50 feet above sea level (NGVD). The terrestrial portion of the Coastal Plain Province is bounded on the west and southwest by the Delaware River and Delaware Bay, on the north by the Fall Line and on the northeast by the Raritan Bay and Staten Island. The remaining portions of the Coastal Plain Province in New Jersey are bordered by the Atlantic Ocean. The Atlantic Coastal Plain has been further differentiated into the Inner and Outer Coastal Plain regions. The Inner Coastal Plain consists of lowlands and rolling hills underlain by Cretaceous deposits and is border to the north by the Piedmont Province. The Outer Coastal Plain is a region of low altitude where low-relief terraces are bounded by subtle erosional scarps, and consists of the unconsolidated Tertiary deposits of sand, silt and gravels. The eastern boundary of the Coastal Plain includes many barrier bars, bays, estuaries, marshes and meadowlands along the Atlantic coast extending from Sandy Hook in the north to Cape May Point at the southern tip of New Jersey.

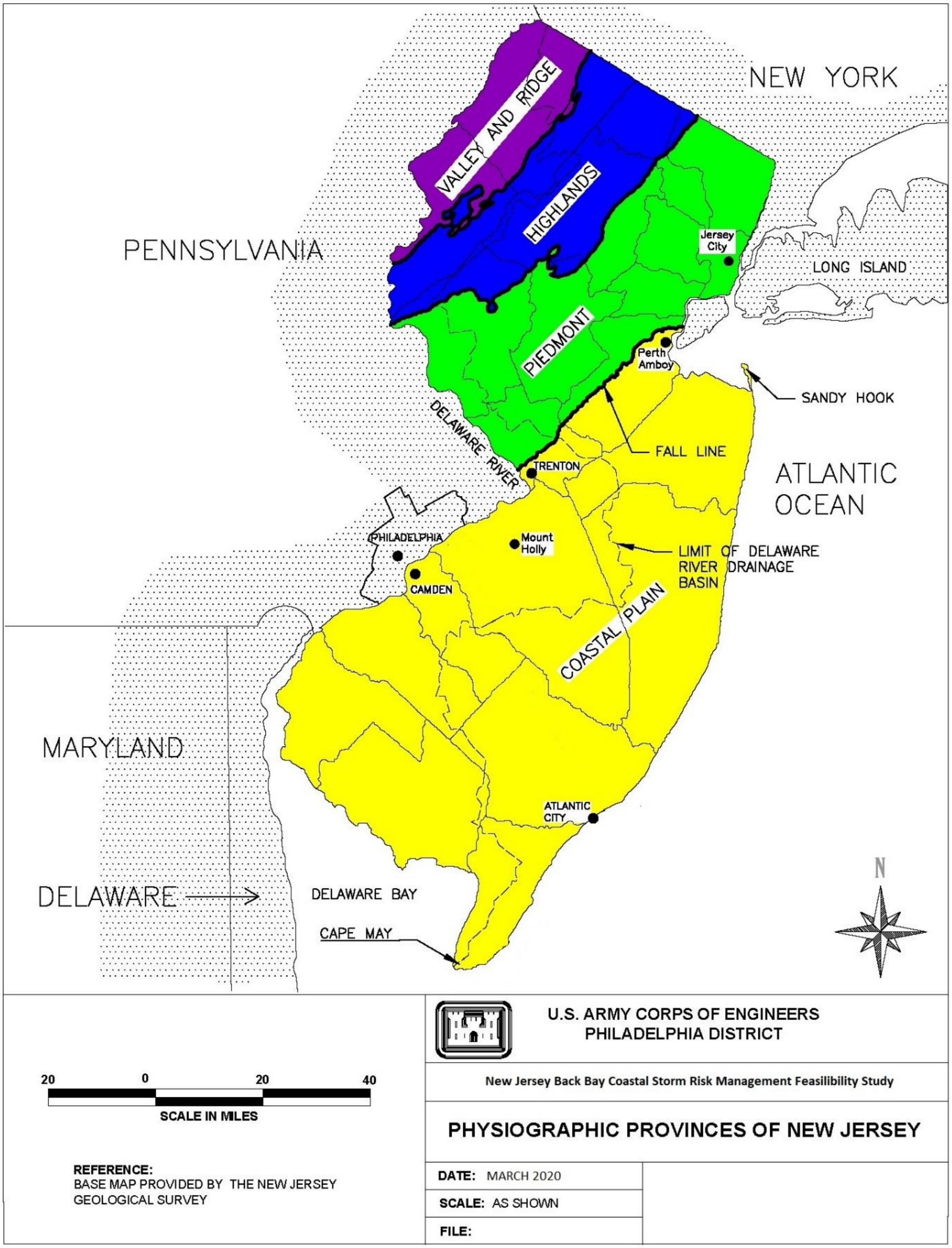


Figure 3-2: Physiographic Provinces of New Jersey

3.1.2 Physiography

The New Jersey shoreline, which is included in the Coastal Lowlands can be divided into those sections where the sea meets the mainland, at the northern and extreme southern ends of the State, and where the sea meets the barrier islands, in the central to southern portion of the State. The Coastal Lowlands include as many as three scarp-bounded terraces, which are underlain by marine and estuarine deposits. The outer margin of the terraces are surrounded by the tidal marshes, bays and the barrier islands. The barrier islands extend from Bay Head, down the coast for approximately 90 miles, to just north of Cape May Inlet and are generally continuous, except for the interruption by 10 inlets.

3.1.3 Barrier Islands

The New Jersey barrier islands, most of which are included in the study area, belong to a land form susceptible to comparatively rapid changes. The barrier islands range in width from around 1000 feet to 5,000 feet. Landward of the barrier beaches and inlets along the barrier islands are tidal bays, which range from 1 to 4 miles in width. These bays have been filled by natural processes until much of their area has been covered with tidal marshes. The remaining water area landward of the barrier islands consists of smaller bays connected by water courses called thorofares. Four geologic processes are considered to be responsible for the detritus (or loose material) in the bay area: (1) stream sedimentation, which contributes a small amount of upland material; (2) waves washing over the barrier islands during storms; (3) direct wind action blowing beach and dune sand into the lagoon; and (4) the work of tidal currents, which normally bring in more sediments in suspension from the ocean on flood tide than they remove on ebb tide. The vegetation of the lagoons, both in marshland and bays, serves to trap and retain the sediments.

3.1.4 Drainage of the Coastal Plain

The land surface in the Coastal Plain of New Jersey is divided into drainage basins, based on the area that contributes runoff to streams and their tributaries in a particular region. A drainage divide marks the topographic boundary between adjacent drainage basins. A major drainage divide in the Coastal Plain separates streams flowing to the Delaware River on the west and to the Atlantic Ocean on the east and southeast.

The surficial drainage system of the New Jersey Coastal Plain was developed at a time when sea level was lower than at present. The subsequent rise in sea level has drowned the mouth of coastal streams where tidal action takes place. This tidal effect extends up the Delaware River to Trenton, New Jersey, a distance of 139 miles. The formation of the barrier islands removed all direct stream connection with the ocean between Barnegat Bay and Cape May Inlet. These streams now flow into the lagoons formed in the back of these barrier beaches and their waters reach the Atlantic Ocean by way of the thorofares and inlets, discussed above. The significance of these features to the drainage system in the study area is that the Coastal Plain streams, whose upper courses carry little sediment, lose that little sediment in their estuaries, and in the lagoons, and supply virtually no beach nourishment to the ocean front areas.

The material present within the coastal lagoons and tidal marshes consists primarily of alluvium, and salt-marsh deposits. The alluvium, which was deposited was derived from weathered upland soils of the Bridgeton and Cohansey Formations, consists of gray and brown sand, silt, pebble gravel, cobbles, minor peat and shells. The salt-marsh deposits, which are comprised of organic muck and peat, silt clay and sand. Black, brown and gray organic muck includes remains of salt-tolerant grasses. Silt and sand occur as deposits along tidal creek margins. These salt-marsh deposits were deposited largely as suspended sediment in turbid bays or rivers during high tides.

3.1.5 Regional Geology

The New Jersey Coastal Plain Physiographic Province consists of sedimentary formations overlying crystalline bedrock known as the "basement complex." From well drilling logs, it is known that the basement surface slopes at about 155 feet per mile to a depth of more than 5,000 to 6,000 feet near the coast. Geophysical investigations have corroborated well-log findings and have permitted determination of the profile seaward to the edge of the continental shelf. A short distance offshore, the basement surface drops abruptly but rises again gradually near the edge of the continental shelf. Overlying the basement are semi-consolidated sedimentary formations of Lower to Middle Cretaceous sediments. The beds vary greatly in thickness, increasing seaward to a maximum thickness of 2.5 miles then decreasing to 1.5 miles near the edge of the continental shelf. On top of the semi-consolidated beds lie unconsolidated sediments of Upper Cretaceous and Tertiary formations. These sediments range from relatively thin beds along the northwestern margin at the Fall Line, to around 4,500 feet beneath Atlantic City to over 40,000 feet in the area of the Baltimore Canyon Trough located around 50 miles offshore of Atlantic City.

Based on information provided by the New Jersey Geological Survey (NJGS) and United States Geological Survey (USGS), the wedge shaped mass of unconsolidated sediments that comprise the New Jersey Coastal Plain discussed above are composed of sand, gravel, silt and clay. The wedge thins to a featheredge along the Fall Line and attains a thickness of over 6,500 feet in the southern part of Cape May County, New Jersey. The system is comprised of relatively highly permeable sand and gravel layers separated by semi-permeable to impermeable silt and clay interlayers that form confining layers and restrict the vertical flow of groundwater. These sediments range in age from Cretaceous to Upper Tertiary (i.e. Miocene - 144 to 5 million years ago), and can be classified as continental, coastal or marine deposits. The Cretaceous and Tertiary age sediments generally strike on a northeast-southwest direction and dip gently to the southeast from ten to sixty feet per mile. The Coastal Plain is mantled by discontinuous deposits of Late Tertiary to Quaternary (geologically recent) sediments, which, where present are basically flat lying. The unconsolidated Coastal Plain deposits, are unconformably underlain by a Pre-Cretaceous crystalline basement bedrock complex, which consists primarily of Precambrian and early Paleozoic age (>540 to 400 million years ago) rocks. Locally, along the Fall Line in Mercer and Middlesex Counties, Triassic age (circa 225 million years ago) rocks overlie the crystalline basement rocks and underlie the unconsolidated sediments.

3.1.6 Surficial Geology

As indicated above, the Coastal Plain of New Jersey consists of beds of gravel, sand, silt and clay, which dip gently towards the southeast. Fossil evidence indicates that these sediments range from the Cretaceous to Quaternary Period, with some more recent glacial period

Quaternary sediments mantling the surface. The older and lower layers outcrop at the surface along the northwest margin of the Coastal Plain and pass beneath successively younger strata in the direction of their dip. Since the formations dip toward the southeast, this results in a series of successive generally parallel outcrops with a northeast-southwest strike, with successively younger layers outcropping at the surface towards the southeast and progressing southward along the shore.

The sea successfully advanced and retreated across the 155 mile width of the Coastal Plain during the Cretaceous through Quaternary Periods (144 million years ago to present). Many sedimentary formations were deposited, exposed to erosion, submerged again and buried by younger sediments. The types of sorting, the stratification, and the fossil types in the deposits indicate that deposition took place offshore as well as in lagoons and estuaries, and on beaches and bars. Considerable changes in sea level continued to take place during Pleistocene time. Glacial periods brought a lowering in sea level as water was locked up in the large terrestrial ice masses. As the sea level fell to a beach line thousands of feet seaward of the present shoreline, Pleistocene sediments were deposited in valleys cut into older formations. The water released through glacial melt during interglacial periods brought a rising of sea level and beaches were formed far inland of the present shore.

Between Bay Head and Cape May City, the coastal lagoons, tidal marshes and barrier beaches that fringe the coast have contributed to the sands of the present beaches. During Quaternary time, changes in sea level caused the streams alternately to spread deposits of sand and gravel along drainage outlets and later to remove, rework, and redeposit the material over considerable areas, concealing earlier marine formations. One of these, the Cape May Formation consisting largely of sand and gravel, was deposited during the last interglacial stage, when the sea level stood 33 to 46 feet higher than at present. The material was deposited along valley bottoms, grading into the estuarine and marine deposits of the former shoreline. In most places along the New Jersey coast, there is a capping of a few feet of Cape May Formation. This capping is of irregular thickness and distribution, but generally forms a terrace about 25 to 35 feet above sea level. The barrier beaches, being of relatively recent origin, are generally composed of the same material as that found on the offshore bottom.

3.2 Soils Encountered

As mentioned, existing Boring O-15 was drilled as part of an overall project, Marina District Highway Improvements, commissioned by the State of New Jersey Department of Transportation involving US Route 30 (Absecon Blvd.) and NJ Route 87 (Brigantine Blvd.). Based on Boring O-15, the subsurface profile generally consisted of (in descending order): 1) fill, 2) soft fine-grained soils, and 3) dense granular soils. The soils encountered in Boring O-15 are in general agreement with the published geologic data. The details of the subsurface soil conditions are on the boring log presented within Attachment 3 of this Appendix B.2. The various soil layers encountered with their relevant properties are summarized in the ensuing paragraphs.

Boring O-15 was drilled to a depth of 90.9 feet below existing grade (or approximate elevation of El. -81.8) on the asphalt paved roadway. The boring was performed on eastbound Huron Ave. located between Absecon Blvd. and Brigantine Blvd. approximately 950 feet west of Brigantine Blvd. The surface material encountered in the boring comprised of only a 0.1 feet thick layer of bituminous pavement.

A 13.9-foot thick layer of fill, consisting of brown and black fine sand with topsoil, cinders, wood, glass, and concrete was encountered below the bituminous pavement. The fill is believed to be part of the roadway embankments. The relative density of the fill was typically very loose to medium dense with standard penetration N-values (N-values) ranging from 1 to 14 blows per foot (bf), averaging about 6 bpf.

A fine-grained soil layer was encountered underlying the fill that extended to a depth of 53.0 feet (or elevation of El. -43.9). The fine-grained soils generally consisted of dark gray silty clay with seams of fine sandy silty clay and silty clayey fine sand. The consistency of the fine-grained soils was very soft with N-values ranging from 0 to 2 bpf, averaging an N-value less than 1 bpf. An N-value of 0 typically means either the split spoon sampler was advanced simply by the weight of the hammer (WOH) or by the weight of the drill rods (WOR) with no hammer drop energy required. This signifies a very loose or very soft soil situation.

The fine-grained soils were underlain by granular soils that extended to the bottom of the boring (or elevation El. -81.8). The granular soil layer predominantly consisted of brown coarse to fine sand with trace gravel. The relative density of the granular soil layer was very dense with N-values ranging from 48 to values in excess of 100 bpf, averaging an N-value in excess of 100 bpf. An 8.0-foot thick layer of medium dense dark gray clayey medium to fine sand becoming very stiff dark gray silty clay was encountered at a depth of 60.0 feet (or elevation of El. -50.9).

3.3 Groundwater Conditions

Groundwater observations were not recorded for Boring O-15. Groundwater varies in the borings from being above, at, or below the ground surface due to the proximity to the shoreline. Given the various locations of all the borings collected, any soil parameters will be formulated with the assumption that groundwater is at the ground surface.

4 PRELIMINARY GEOTECHNICAL ANALYSIS

4.1 Soil Parameters

Overall, the soil profiles in the borings were somewhat similar. At Boring O-15, the fine-grained soil layer was encountered at a depth of 14.0 feet and extended to a depth of 53.0 feet for a thickness of 39.0 feet, which was observed to be the thickest fine-grained soil layer of the borings collected. This fine-grained soil layer was also observed to be very soft with an average N-value of less than 1 bpf. Therefore, Boring O-15 provides what would be considered as the most conservative soil profile. The soil profile below the ground surface in Boring O-15 is shown in Table 4-2.

Table 4-2: Soil Profile in Boring O-15

Soil Strata	Depth (ft)	Moist Unit Weight (pcf)	Submerged Unit Weight (pcf)	Angle of Internal Friction, φ (degrees)		Cohesion, c (psf)	
Fill	0 – 14	110	47.6	29 (drained)	29 (undrained)	0 (drained)	0 (undrained)
Silty Clay (CL)	14 – 53	100	37.6	17 (drained)	0 (undrained)	0 (drained)	250 (undrained)
Sand (SP)	53 – 90.9	120	57.6	34		0	

Soil properties used in the analyses are:

1. Submerged Unit weight of the soil - this is the total unit weight of the soil less the unit weight of water (62.4 pcf).
2. Phi (φ) - effective angle of internal friction.
3. Cohesion (c) - unconfined compressive strength.

The soil parameters shown in Table 4.2 will be used to perform pile foundation analysis described in Sections 4.2 and 4.3.

4.2 Levee (Type “A”) Analysis

As one of the structural alternatives in the overall NJBB CSRM Feasibility Study, the perimeter plan includes levee sections used in open space areas that transitioned from beach to water, or from undeveloped property to marshland. The levee section is planned to incorporate a 10’ crest width with 2H:1V side slopes, including 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 planned 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-inch thick, stone-filled marine mattresses with geotextile along the base to provide foundation support at the soil interface.

The level of detail on conceptual engineering analyses, calculations, and design is limited at this point in the study. No laboratory testing was available for the representative boring for preliminary analysis. Thus, the levee structure was not analyzed for stability. However, the levee geometry is typical of many levee structures used throughout USACE. Parametric estimates for some quantities have been used as described in the Civil Appendix. A higher level of design will be conducted during subsequent study and PED phases of the project.

4.3 Concrete Cantilever Flood Wall (Types “B” & “C”) Analyses

Another one of the structural alternatives in the overall NJBB CSRM Feasibility Study perimeter plan includes two types of concrete cantilever walls to retain flood water in the event of a storm surge. Both of these types of flood walls, Type “B” and Type “C”, will be founded on steel H-piles. Geotechnical analyses will include recommendations for pile capacity for the foundations. Based on preliminary pile analyses, steel H-piles with designation HP14x73 are considered suitable and have been analyzed. The relevant structural properties of the H-piles are provided in Table 4.3.

Table 4-3: Properties of H-pile HP 14x73

Section	Width (in)	Area (in^2)	Weight/Ft (lb)	Moment of Inertia (in^4)	Elastic Modulus (ksi)
HP14x73	14.6	21.1	73	716.5	29000

The resistances evaluated include axial (downward) resistance for single piles, uplift (upward) resistance for single piles, and lateral resistance for single piles. The computer softwares used to analyze the piles are:

1. A-PILE Offshore - By Ensoft, Inc., this performs axial capacity analysis on piles.
2. L-PILE – By Ensoft, Inc., this is based on FHWA COM624P computer software.

Table 4.4 below summarizes resistances evaluated, the appropriate factors of safety, and the basis for the factors according to EM 1110-2-2906, “Design of Pile Foundations”.

Table 4-4: Resistance, Software Used and Factors

Resistance	Software Used	Loading Condition	Factor of Safety	Comments
Axial (Downward)	A-PILE	Unusual	2.25	Theoretical or empirical prediction not verified by load test
Uplift (Upward)	A-PILE	Unusual	2.25	Theoretical or empirical prediction not verified by load test
Lateral – Single Pile	L-PILE	-	1.0	Ultimate lateral capacity is load required to induce maximum deflection determined by criteria set by structural engineer

In accordance with EM 1110-2-2906, when considering silt or clay soils the shear strength of these materials should be obtained for undrained (short term) and drained (long term) strength conditions and the pile capacities and lateral pile deflections obtained should be based on the more conservative of the two cases.

4.3.1 Flood Wall Type “B”

According to the typical section of Type “B” wall, the lowest bottom of footing elevation for Type “B” wall is 14 feet below the existing ground surface. Therefore, the upper most layer in the soil profile used to analyze the Type “B” wall is the top of the silty clay layer.

When comparing the axial capacities of the piles based on drained and undrained conditions, the drained shear strengths controlled the design. The structural loads (See Structural Appendix) dictate the need for two rows of vertical piles spaced at 11 feet that will act as a couple to resist those loads. An allowable capacity of 38 kips is required for both axial and uplift; therefore, uplift controls the design. An H-Pile with section HP14x73 driven to a depth of 60 feet below the bottom of footing elevation will perform satisfactorily under the given loads. The following Table 4-5 summarizes the results of the computed ultimate and allowable pile capacities using a factor of safety of 2.25 for the unusual case.

Table 4-5: Flood Wall Type “B” Vertical Capacities

Wall Type	Minimum Embedment (ft)	Ultimate Axial (Downward) Capacity (kips)	Allowable Axial Capacity (kips)	Ultimate Uplift (Upward) Capacity (kips)	Allowable Uplift Capacity (kips)
B	60.0	136.2	60.5	93.4	41.5

The flood wall structure will be subjected to hydrostatic forces and wave action, resulting in lateral loading. Lateral pile capacity is generally determined by the amount of allowable deflection that a structure or pile can be subject to, in addition to maximum internal moments within the member. Lateral pile analysis is performed by inputting lateral (horizontal) loads, external moments and vertical loads on the pile head. The results of the analysis provide maximum pile head deflection for the given loading criteria. Wave loads are cyclic which causes the deflections and moments of a single pile or a group of piles to increase rapidly with the number of cycles of load applied up to approximately 100 cycles, after which the deflection and moment are not significantly affected. As such, the lateral deflection analysis included cyclic loading of 100 cycles. Laterally loaded groups of piles deflect more than a single pile loaded with the same load per pile as the group. This increased deflection is due to overlapping zones of stress of the individual piles in the group. Consequently, a reduction factor or P-multiplier (Pm) must be applied depending on the center-to-center pile spacing in the direction of the load. The current pile layout has the piles within each row spaced 11 feet apart set vertically. For piles spaced at 6B or greater, the Pm is 1, i.e., not required. Guidance from the structural engineer allowed the analysis to be based on a fixed head condition. The applied external loads and the results of the lateral analyses are summarized in Table 4-6.

Table 4-6: Flood Wall Type “B” Lateral Analysis

Pile Length (ft)	Vertical Load (kips)	Lateral Load (kips)	Moment (kip-ft)	Horizontal Deflection (in)
60	38	27.75	0	1.34

The piles along the length of the wall are set at 5-feet center-to-center spacing. The spacing was chosen from an iterative process to optimize the pile lengths with the spacing. The larger the spacing, the more structural load is being carried by the individual piles, thus resulting in longer piles. As such, the larger the spacing, the more lateral load is carried by each pile, so the current 5-feet center-to-center spacing also allows for an acceptable deflection.

4.3.2 Flood Wall Type “C”

According to the typical section of Type “C” wall, the bottom of footing elevation for Type “C” wall is approximately 4 feet below the existing ground surface. Therefore, the upper most layer in the soil profile used to analyze the Type “B” wall begins 4 feet below the ground surface of the boring.

When comparing the axial capacities of the piles based on drained and undrained conditions, the drained shear strengths controlled the design. The structural loads (See Structural Appendix) dictate the need for two rows of vertical piles spaced at 11 feet that will act as a couple to resist those loads. An allowable capacity of 23.2 kips is required for both axial and uplift; therefore, uplift controls the design. An H-Pile with section HP14x73 driven to a depth of 55 feet below the bottom of footing elevation will perform satisfactorily under the given loads. The following Table 4-6 summarizes the results of the computed ultimate and allowable pile capacities using a factor of safety of 2.25 for the unusual case.

Table 4-7: Flood Wall Type “C” Vertical Capacities

Wall Type	Minimum Embedment (ft)	Ultimate Axial (Downward) Capacity (kips)	Allowable Axial Capacity (kips)	Ultimate Uplift (Upward) Cpacity (kips)	Allowable Uplift Capacity (kips)
C	55.0	120.4	53.5	77.5	34.4

The flood wall structure will be subjected to hydrostatic forces and wave action, resulting in lateral loading. The lateral deflection analysis included cyclic loading of 100 cycles. The reduction factor or P-multiplier (Pm) in this case, as was in Type “B”, is 1, i.e., not required. The current pile layout has the piles within each row spaced 11 feet apart set vertically. Guidance from the structural engineer allowed the analysis to be based on a fixed head condition. The applied external loads and the results of the lateral analyses are summarized in Table 4-6.

Table 4-8: Flood Wall Type "C" Lateral Analysis

Pile Length (ft)	Vertical Load (kips)	Lateral Load (kips)	Moment (kip-ft)	Horizontal Deflection (in)
55	23.2	48.4	0	1.47

The piles along the length of the wall are set at 8-feet center-to-center spacing. The spacing was chosen from an iterative process to optimize the pile lengths with the spacing. The larger the spacing, the more structural load is being carried by the individual piles, thus resulting in longer piles. As such, the larger the spacing, the more lateral load is carried by each pile, so the current 8-feet center-to-center spacing also allows for an acceptable deflection.

4.4 King Pile Flood Wall (Type "D") Analysis

Lastly of the structural alternatives in the overall NJBB CSRM Feasibility Study, the perimeter plan will also include the use of a cantilever king pile wall system to retain flood water in the event of a storm surge. This type of flood wall, designated Type "D", is proposed in areas along the bay shoreline where horizontal clearance or land space is inadequate to construct a Type "B" or "C" wall. The king pile wall system consists of W-shape piles (the king piles) with one pair of steel sheeting driven in between the king piles.

The king pile wall analysis was performed using the USACE Computed-Aided Structural Engineering (CASE) program CWALSHT. The program uses classical soil mechanics procedures for determining the required depth of penetration of a new wall or assesses the factors of safety for an existing wall. The minimum embedment for external stability and the maximum moment was calculated, and the structural engineer used this information to determine a W-Section suitable to the king piles.

The maximum wall height for Flood Wall Type "D" is 25 feet. The top of wall elevation is El. +16.0 and the lowest dredge line elevation is El. -9.0. The water level is assumed to be at the top of wall with a wave force of 3311 lb/ft at elevation EL. +15.15 acting as a line load. The results of the CWALSHT analysis are provided in Table 4-9.

Table 4-9: CWALSHT Analysis Results

Minimum Embedment (ft)	Total Pile Length (ft)	Maximum Moment (lb-ft)	Max. Moment Location (ft)
72.11	94.11	388520	El. -48.46

Based on CWALSHT analysis, the minimum embedment required for external stability, using active and passive pressure factors of safety of 1.0 and 1.5, respectively, is 72.11 feet for an overall length of pile of 94.11 feet. The maximum moment calculated, using a factor of safety of 1.0 for both active and passive pressure, is 388,520 lb-ft. Using the maximum moment, the structural engineer determined minimum king pile wall system based on its structural properties. It was determined that a king pile wall system, or combination wall system (also combi-wall system), with a beam section of W40x249 in combination with a pair of sheet sections PZC 18 would perform satisfactorily under the given loads. The relevant structural properties of the king pile (combi-wall) system are provided in Table 4.10.

Table 4-10: Properties of Combi-Wall Section W40x249 PZC18

Beam Section	Sheet Section	System Width (in)	Section Modulus (in^3)	Moment of Inertia (in^4)	Elastic Modulus (ksi)
W40x249	PZC18	21.1	187.9	3899	29000

The structural engineer is to determine the deflection criteria for the king pile wall. In the event of a 100-year storm, the loading on the wall will induce a deflection of 13 inches at the top of wall. This deflection may be acceptable since the loads required to induce this deflection are temporary loads.

5 BASIS FOR RECOMMENDATIONS

This engineering appendix has been prepared to serve as a preliminary foundation design for the proposed flood walls as part of the perimeter plan structural alternative in the overall NJBB CSRM Feasibility Study as well as a basis for preparation of the subsurface investigation program for a more refined foundation design for the proposed flood walls and preliminary design of the bay closure structures and storm surge barrier gate structures. The analyses and conclusions contained in this appendix are based upon the information available at the time of the actual calculations and on the site conditions, surface and subsurface, from previously published boring data from nearby projects. Further assumption has been made that the existing information from limited exploratory borings, in relation to lateral extent of the site, are loosely representative of conditions at the site as a whole. Inherent to these assumptions is a very conservative subsurface soil profile. The existing and proposed subsurface information together will still warrant a somewhat conservative design given that one test hole will be representative of miles of shoreline.

6 COMPLETED GEOTECHNICAL SUBSURFACE EXPLORATIONS

At the time of the enclosed conceptual engineering analyses, calculations, and design, the proposed preliminary geotechnical subsurface explorations had yet to be completed. The preliminary geotechnical subsurface investigation was performed in 2019 but was not incorporated into this Draft Integrated Report due to the level of design. The preliminary geotechnical subsurface investigation included six (6) Standard Penetration Test (SPT) borings with laboratory testing and thirty-three (33) Cone Penetrometer Test (CPT) Soundings. The purpose for the SPT boring explorations and laboratory testing was to obtain subsurface soil classification and strength data for design to be used in determining the feasibility of proposed flood walls, bay closure structures, and storm surge barrier gate structures. The boring data will help to fill in the gaps of the existing soil data gathered. The purpose for the CPT soundings was to develop a reliable profile of the subsurface material along the back bays for the various floodwall structures. The CPT method allows for a high quantity of sounding locations at significant depths along the 3,400 miles of coastline in the study area. The CPT data will be reviewed, and a determination can be made for the feasibility of the floodwall structures as well as specific target areas for future subsurface investigation and testing. The 2019 subsurface investigation results as well as future investigations will be integrated into the NJBB Study to inform a higher level of design during subsequent study and PED phases of the project.

REFERENCES

Das, Braja M. Principles of Foundation Engineering. 7th Ed. Cengage Learning. Stamford, CT. 2011.

EM 1110-1-1905. *Bearing Capacity of Soils*. October 30, 1992. Department of the Army, U.S. Army Corps of Engineers, Washington, DC 20314-1000.

EM 1110-2-2502. *Retaining Walls and Flood Walls*. September 29, 1989. Department of the Army, USACE, Washington DC 20314-1000.

ER 1110-2-2906. *Design of Pile Foundations*. January 15, 1991. Department of the Army, USACE, Washington DC 20314-1000.

Lambe, William T., Whitman, Robert V. *Soil Mechanics*. John Wiley & Sons, Inc. New York, NY. 1969.

United States Steel (USS). *Steel Sheet Piling Design Manual.* United States Department of Transportation. July, 2984

ENGINEERING APPENDIX
STURCTURAL

NEW JERSEY BACK BAYS
COASTAL STORM RISK MANAGEMENT
FEASIBILITY STUDY

PHILADELPHIA, PENNSYLVANIA

APPENDIX B.3

August 2021

NJBB
NEW JERSEY BACK BAYS STUDY

U.S. Army Corps of Engineers
Philadelphia District

THIS PAGE HAS BEEN INTENTIONALLY LEFT BLANK

Table of Contents

List of Figures

List of Tables

1 INTRODUCTION

This appendix presents the results of the Structural engineering preliminary analysis and design of the structural project features for the New Jersey Back Bays (NJBB) Coastal Storm Risk Management (CSRM) Study. This report will describe the methodology utilized for the preliminary design of the concrete T-wall and King Pile/Sheet Pile floodwalls. Additionally, this appendix will describe the general design criteria that shall be used for the design of the movable gates (hydraulic steel structures) to be used as storm surge barriers.

2 FLOODWALLS

There are three different types of floodwalls proposed for the Back Bays Perimeter Plan, which are identified as Type B, Type C, and Type D. Floodwalls Type B and C are composed of a cast-in-place reinforced concrete T-wall supported by two rows of steel H-piles. Type D is composed of a steel pile and sheet pile combination floodwall, also known as king pile/sheet pile wall. Type A is composed of a levee section and is covered under the Civil portion of this appendix. Type B and D walls will be constructed in areas below existing water level while Type C will be constructed in areas above the mean tide zone. Sketches of the floodwalls are shown in **Figures 1 to 3**.

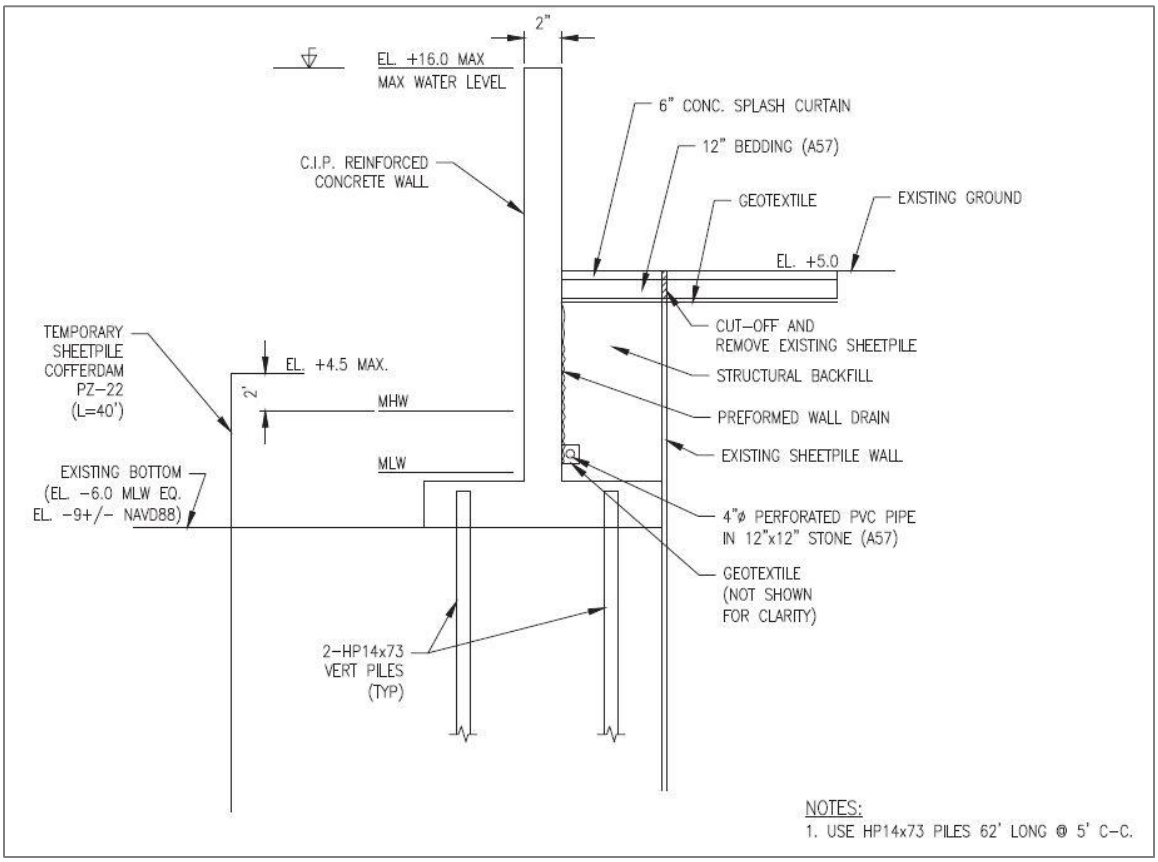


Figure 1: Floodwall Type B – Concrete T-Wall Supported by Steel Piles

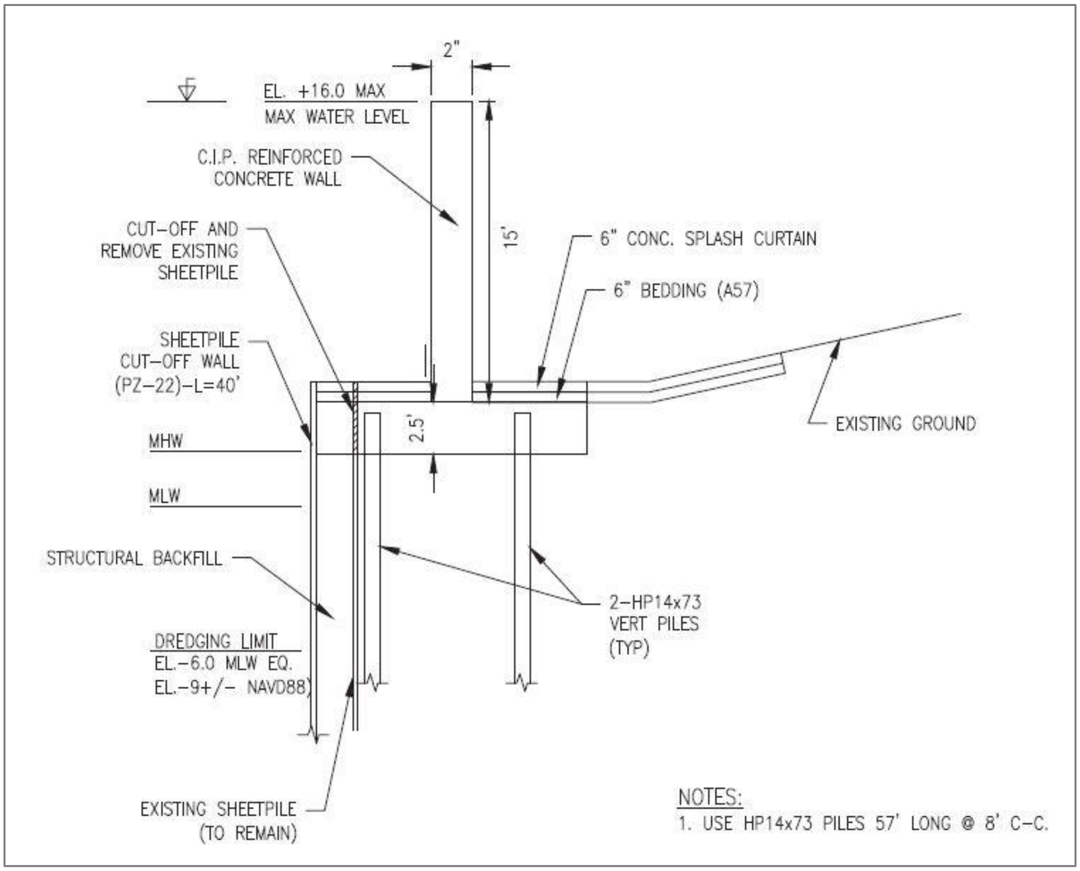


Figure 2: Floodwall Type C – Concrete T-Wall Supported by Steel Piles

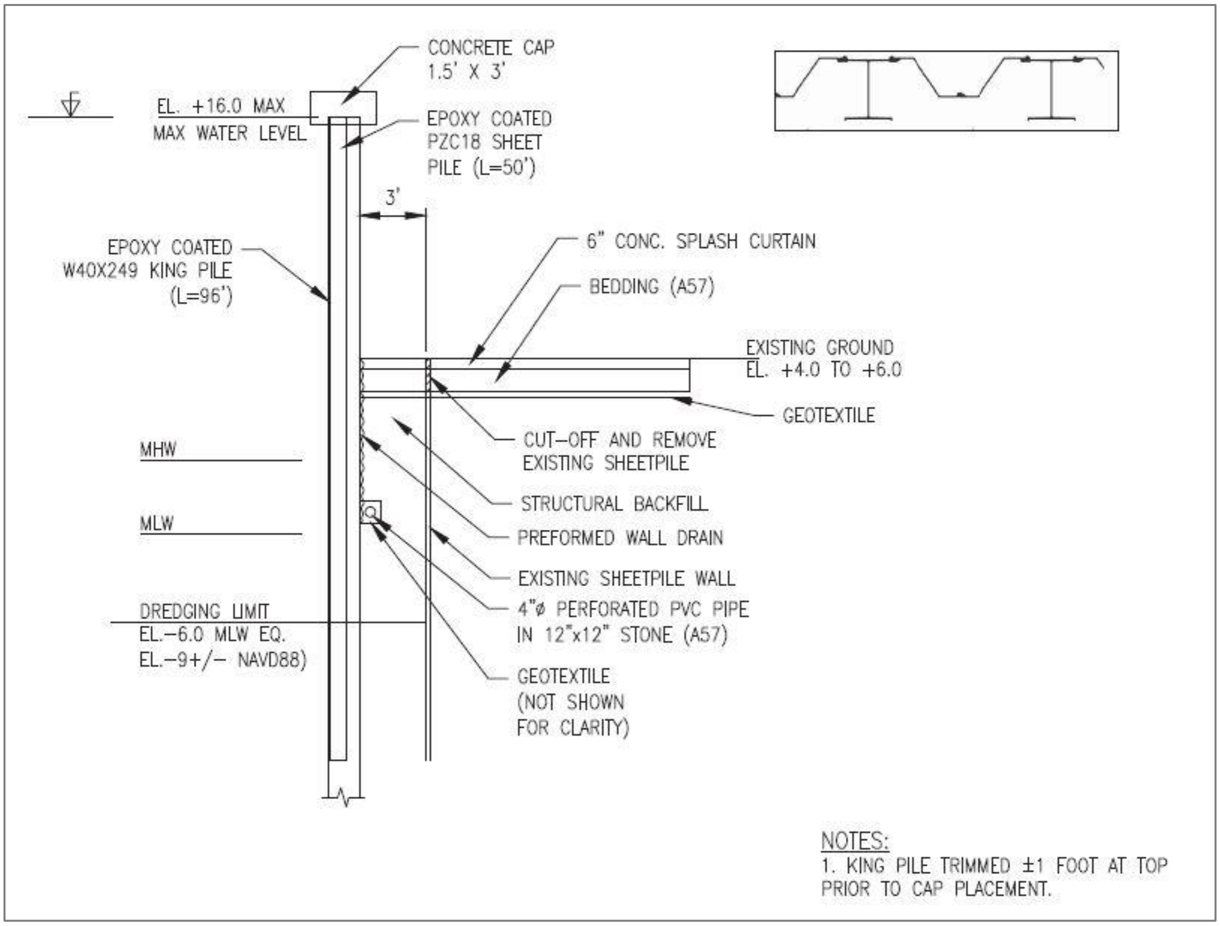


Figure 3: Floodwall Type D – Steel Pile and Sheet Pile Combination Wall

2.1 Analysis and Design of Floodwalls

The concrete T-walls were analyzed for global stability and structural strength based on the requirements established on EM 1110-2-2100 "Stability Analysis of Concrete Structures", EM 1110-2-2502 "Retaining and Floodwalls", Engineering and Construction Bulletin (ECB) No. 2017-2 "Revision and Clarification of EM 2100 and EM 2502", and EM 1110-2-2104 "Strength Design for Reinforced Concrete Hydraulic Structures".

Five different loading conditions were used during the analysis in accordance with Table B-5 of EM 1110-2-2100, see **Table 6-1**. An additional loading condition, Design Resiliency Check (DRC), was also used and includes water at the top of the wall with coincident wave. This case was taken from the New Orleans District Design Guidelines and applies to structures whose primary function is hurricane flood protection. The case was developed to verify the survivability of a structure during major storm events. As shown on **Figure 4** and considering the floodwalls as critical structures, Table 1 of ECB No. 2017-2 classifies these loading conditions into three (3) different categories: usual (<10 year recurrence interval), unusual (10-750 year recurrence interval), and extreme (>750 year recurrence interval).

The controlling case for the design of the floodwalls was the Design Resiliency Check (DRC) case, water at top of wall with coincident wave.

Table 1: Coastal Floodwall Loading Condition Classification

Load Case	Loading Description	Classification
C1	Surge Stillwater + Coincident Wave	UN/E[1]
C2a	Coincident Pool + OBE	UN
C2b	Coincident Pool + MDE	E
C3	Construction	UN
C4	Normal Operating	UN
Additional Case (DRC)[2]	Water at Top of Wall + Coincident Wave	UN/E

[1] UN = Unusual, E = Extreme
[2] DRC = Design Resiliency Check

(EM 1110-2-2100, Table 3-1) Load Condition Probabilities

Load Condition Categories	Annual Probability (p)	Return Period (t_r)
Usual	Greater than or equal to 0.10	Less than or equal to 10 years
Unusual (normal structures)	Less than 0.10 but greater than or equal to 0.0033	Greater than 10 years but less than or equal to 300 years
Unusual (critical structures)	Less than 0.10 but greater than or equal to 0.00133	Greater than 10 years but less than or equal to 750 years
Extreme (normal structures)	Less than 0.0033	Greater than 300 years
Extreme (critical structures)	Less than 0.00133	Greater than 750 years

Figure 4: ECB 2017-2, Loading Condition Categories

A description of the different water surface elevations acting on the floodwalls as shown in Figures 6-1 to 6-3 can be found in the Civil portion of this Appendix. Using a conservative approach during the analysis, all floodwalls were evaluated for the maximum possible surge stillwater and coincident pool water levels. Additionally, wave forces were calculated and provided by the Hydraulics, Hydrology, and Coastal Section for both the Surge Stillwater and Coincident Wave (C1) case and the Water at Top of Wall and Coincident Wave (DRC) cases. The wave forces were calculated and applied as point loads acting on the floodwalls at certain heights.

A set of spreadsheets was developed in Mathcad to analyze the walls considering all applicable loading conditions. For Type B and C walls, concrete member sizes were designed based on all vertical, gravity, and horizontal forces acting on the structures. **Figure 5** below provides a schematic of the different forces taken into consideration during the analysis.

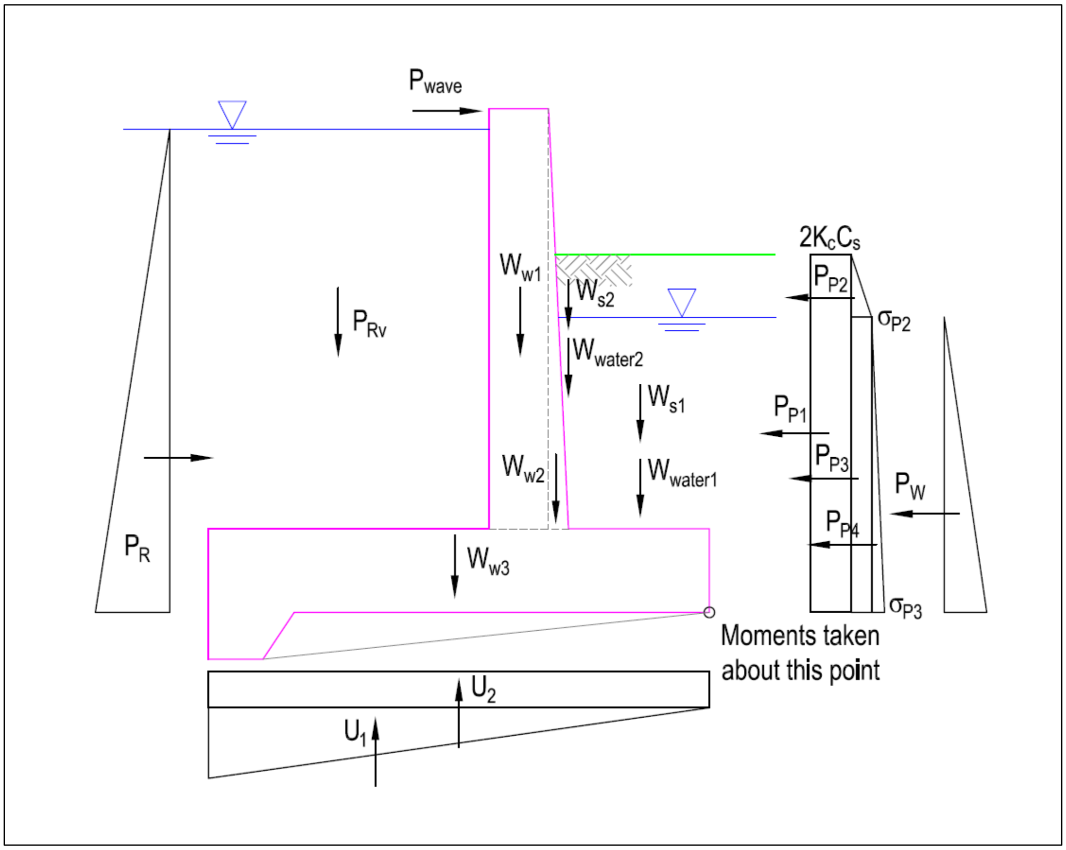


Figure 5: Forces Acting on Floodwalls

Resultant forces from the global stability analysis were provided to the Geotechnical Section for the design of the steel H-piles that will support the walls. The vertical force was calculated as part of a system of forces (couple) formed when dividing the moment acting on the wall by the distance between the two rows of piles supporting the structure. This distance between rows of piles is equal to 11ft for both Type B and C walls. The resultant horizontal force was calculated based on all horizontal loads acting on the walls. **Table 2** provides a summary of the resultant forces acting on the walls for the controlling case (worst-case scenario).

Table 2: Resultant Forces Acting on Floodwalls - Controlling Case

Wall Type	Vertical Force (kips/ft)	Horizontal Force (kips/ft)
B	11.6	11.1
C	5.6	12.1

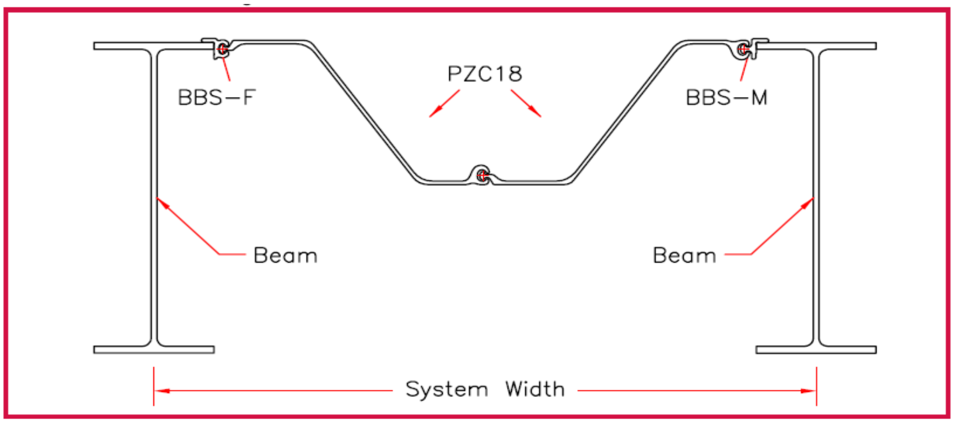
The preliminary design results for T-wall types B and C are provided in **Table 3** below.

Table 3: T-wall Design Results

Wall Type	Footing		Stem		Piles (2 Rows)		
	Width (ft)	Thickness (ft)	Height (ft)	Thickness (ft)	Size	Length[1] (ft)	Spacing (ft)
B	14	2.5	22.5	2	HP14x73	62	5
C	14	2.5	15	2	HP14x73	57	8

[1] Note: This length includes 2ft that will be embedded in the wall base. Subtract 2ft to obtain the length of the pile below the bottom of the wall footing

For the Type D walls, the Geotechnical Section performed a pile analysis and provided the forces acting on the wall. A required section modulus was then calculated based on these forces and the required king pile/sheet pile combination was selected from a dimensions and properties table provided by one of the manufacturers of this type of wall in the United States. The selected wall is composed of a combination of 96’ long W40x249 king piles spaced at 67.81" on center with 50’ long PZC18 sheet piles in between. **Figure 6** below shows the dimensions and properties table for the king pile/sheet pile wall.



Dimensions and Properties

Beam	System Width	Section Modulus	Moment of Inertia	Weight in Pounds		
				lb / ft²		
	in.	in³ / ft	in⁴ / ft	100%	80%	60%
W33 x 118	63.54	82.5	1502	44.1	39.7	35.4
W33 x 130	63.57	91.2	1658	46.3	42.0	37.6
W36 x 135	64.01	96.6	1882	47.0	42.6	38.3
W40 x 149	63.87	110.5	2295	49.7	45.3	41.0
W40 x 167	63.87	127.0	2642	53.1	48.7	44.4
W40 x 183	63.87	142.0	2966	56.1	51.7	47.4
W40 x 199	67.81	149.1	3076	55.6	51.6	47.5
W40 x 215	67.81	164.2	3398	58.5	54.4	50.3
W40 x 249	67.81	187.9	3899	64.5	60.4	56.3

Figure 6: Dimensions and Properties Table for King Pile/Sheet Pile Wall

Additional information regarding floodwalls can be found in Appendix B.1, Civil.

3 STORM SURGE BARRIERS

Storm surge barriers consist of a series of movable gates (hydraulic steel structures, HSS) that stay open under normal conditions to allow navigation and tidal flow to pass but are closed during storm surge events.

3.1 Analysis and Design of Movable Gates (Hydraulic Steel Structures, HSS)

The design of the hydraulic steel structures will be in accordance with ETL 1110-2-584 "Design of Hydraulic Steel Structures", Engineering and Construction Bulletin (ECB) 2019-10 "Guidance for Design of Hydraulic Steel Structures", and ECB 2021-6 "Guidance for Design of Hydraulic Steel Structures and Design and Evaluation of I-Walls Including Sheet Pile Walls". The established design philosophy is intended to provide ductile structures and to prevent brittle behavior. The design must consider all failure modes including general yielding or excessive plastic deformation, buckling or general instability, fatigue damage, fracture, excessive elastic deformation, and damage from excessive vibration. A risk analysis should be performed on HSS where life safety or significant economic loss would occur in the event of a failure. The design must also satisfy all applicable limit states. A limit state is a controlling condition in which a structural system or component becomes unfit for its intended purpose. Limit states include strength, serviceability, fatigue, and fracture. All HSS members and connections must satisfy the following general equation for each limit state:

$$\Sigma \gamma_i Q_{ni} \leq \alpha\phi R_n$$

where:

γ_i = load factors that account for variability in loads to which they are assigned

Q_{ni} = nominal (code-specified) load effects

α = performance factor

ϕ = resistance factor that reflects the uncertainty in the resistance for the particular limit state

R_n = nominal resistance as specified in the American Institute of Steel Construction (AISC) Manual

Load factors for each load and loading condition are defined for each HSS. The general equation for combining loads for the strength limit state is as follows:

$$U = \Sigma\gamma_p L_p + \gamma_{pr} L_{pr} + \Sigma\gamma_c L_{tc} + \gamma_c L_{dc}$$

where:

U = factored applied load

γ_p = load factor applied to permanent loads

L_p = permanent loads

γ_{pr} = load factor applied to principal loads

L_{pr} = principal loads

L_t = temporary loads

L_d = dynamic loads

c = designates companion loads

γ_c = load factor applied to companion loads

Applicable loads with their respective load factors are shown in **Figure 6-7**.

Limit State		Serviceability and Fatigue[5]	Strength				
Load Category		Usual and Unusual	Permanent, Companion	Principle Load Factors, γ_{pr} Usual	Unusual	Extreme[7]	
Return Period - Critical		< 750	< 10	< 10	10-750	750-10,000	> 10,000
Return Period - Normal		< 300	< 10	< 10	10-300	300-3,000	> 3,000
Permanent Loads, L_P			γ_p				
Dead	D	1.0	1.2[1], 0.9[2]	1.4	NA	NA	NA
Gravity (Mud/Ice)	G	1.0	1.6[1], 0[2]	NA	NA	NA	NA
Temporary Loads, L_T			γ_c				
Hydrostatic	Hs	1.0	1.0	1.5[3]	1.4[3]	1.3	1.2[7]
Ice, Thermal Expansion	IX	1.0	1.0	NA	NA	1.3	1.2[7]
Operating Equipment	Q	1.0	1.0	1.5[3]	1.4[3]	1.3	1.2[7]
Live Load	L	1.0	1.0[4]	NA	1.6[4]	NA	NA
Self Straining	T	1.0	0.75[4]	1.0[4]			
Gate Operation Friction	F	1.0	1.4	NA	NA	NA	NA
Dynamic Loads, L_D			γ_c				
Hydrodynamic- Temporal Head, Prop wash, downdrag, intertial resistance, overtopping impingement	Hd	1.0	1.0	NA	NA	NA	1.2[7]
Wave	Hw	1.0	1.0	NA	NA	NA	1.2[7]
Debris/Floating Ice	IM	1.0	1.0	NA	NA	NA	1.2[7]
Vessel Impact	BI	1.0	1.0	NA	NA	NA	1.2[7]
Wind	W	1.0	0.5[4]	NA	NA	1.0[4]	
Earthquake	EQ	NA	NA	NA	1.5	1.0 or 1.25[6]	1.0 or 1.25[6]

Notes:

1. Applied when loads add to the predominant load effect.
2. Applied when loads subtract from the predominant load effect.
3. Usual or Unusual loads used as principal loads for strength design when they are the maximum possible loads.
4. From ASCE 7. Where other standards are referenced, load cases and load factors from those standards will be used for design when those loads are primary loads. See load descriptions for details.
5. Load factors for finite fatigue life are shown. Load factors for infinite fatigue life are 2.0 for all loads. See paragraph 5.1.3.
6. For site specific earthquake the load factor is 1.0. Otherwise the higher load factor is used. See Paragraph 4.4.
7. When the return period is not known, a load factor of 1.3 will be used. Otherwise the load factor is 1.2.

Figure 7: Loads and Load Factors for HSS

The loads applied to the hydraulic steel structures can be separated into categories based on their probability of occurrence. Loads with less probability of occurrence can have lower safety factors and different performance requirements to achieve the same reliability. Loads associated with different average annual return periods (or annual exceedance probability (AEP)) are categorized as usual, unusual, and extreme. The probability of loading associated with the Usual, Unusual, and Extreme load categories is illustrated in **Figure 8**.

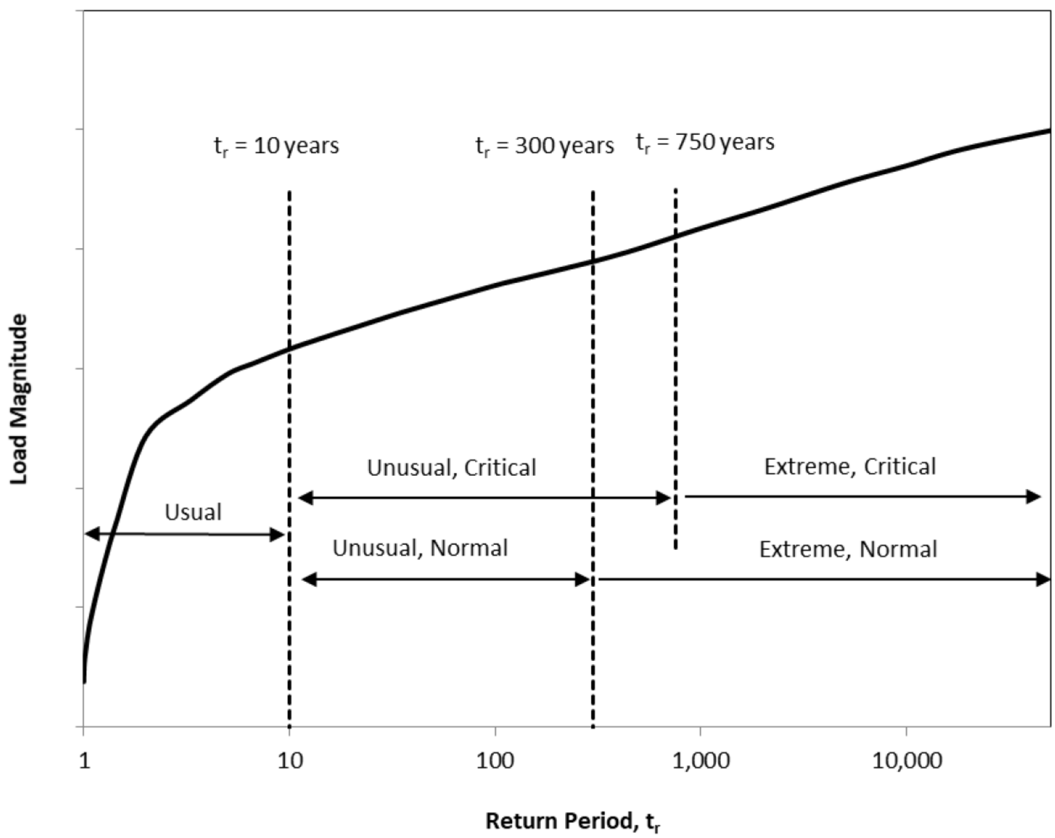


Figure 8: Load Category versus Return Period

All cyclically-loaded HSS must be designed for the fatigue limit state. The stress life procedures as defined in the AISC or American Association of State Highway and Transportation Officials (AASHTO) manuals must be used for fatigue design. Other considerations on the design of hydraulic steel structures include constructability, safety, and serviceability with consideration for inspectability and economy.

3.2 Sector Gates – Navigable Gates

Sector Gates will be installed at inlets and bay closures to provide a navigable opening with unlimited vertical clearance. These gates consist of two leaves that join at the center of the opening and rotate about a vertical axis into recessed areas in permanent housing structures when in open position, providing an open channel for navigation. Each leaf is shaped as a sector of a cylinder. The gates are horizontally closed during significant storm events. The advantage of sector gates is that they can be opened and closed under small differential heads. Loads that are applicable to sector gate design include dead load, gravity loads, hydrostatic and hydrodynamic loads, operating loads, barge and other impact loads, ice loads, wave loads, and earthquake loads.

Figure 9 shows the typical components of a sector gate.

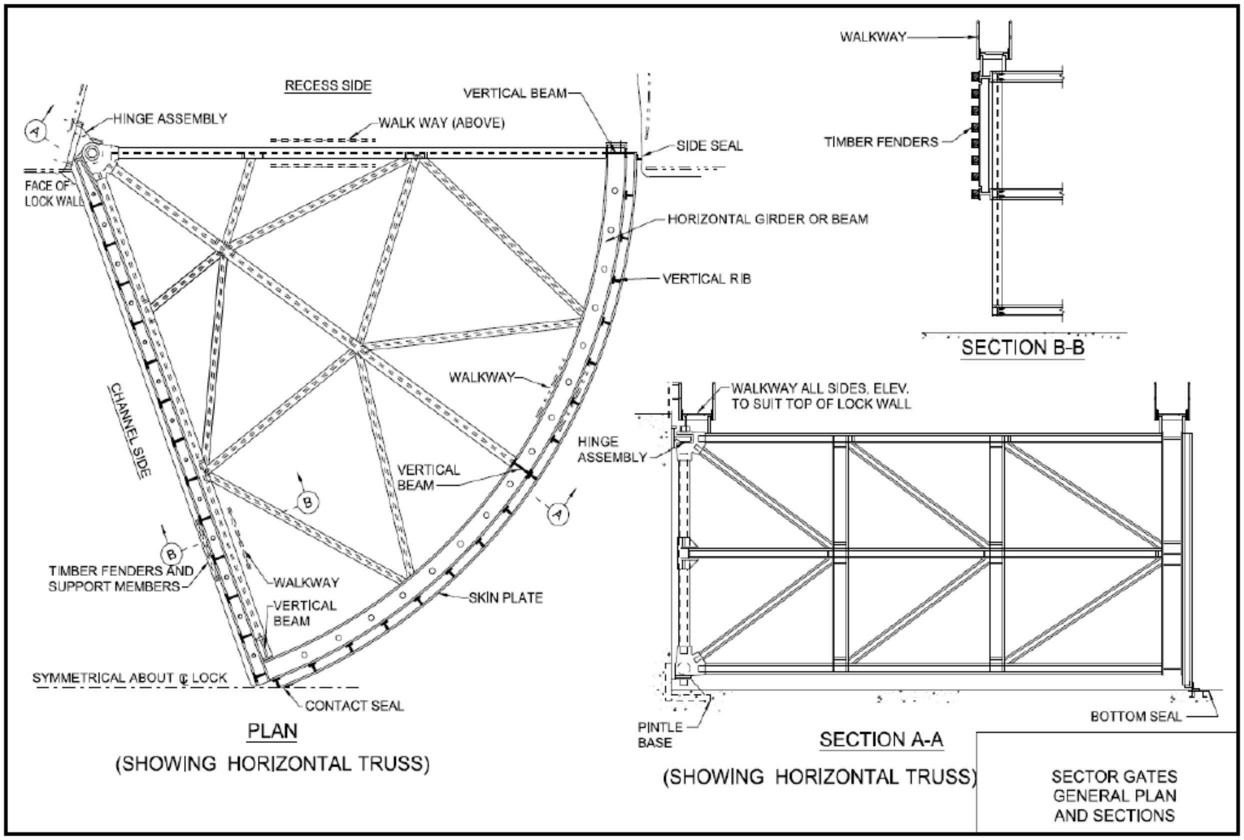


Figure 9: Typical Components of Sector Gate

3.3 Vertical Lift Gates – Auxiliary Flow Gates

Vertical Lift Gates will be positioned adjacent to sector (navigable) gates and throughout bay closures to maintain tidal flow. Vertical lift gates have limited vertical clearance but are capable of providing recreational navigation. These gates will be placed throughout water depths that are deemed constructible and practical. Loads that are applicable to vertical lift gate design include dead load, gravity loads, hydrostatic and hydrodynamic loads, operating loads, environmental loads, impact loads, and earthquake loads.

Figure 10 shows typical views of a vertical lift gate.

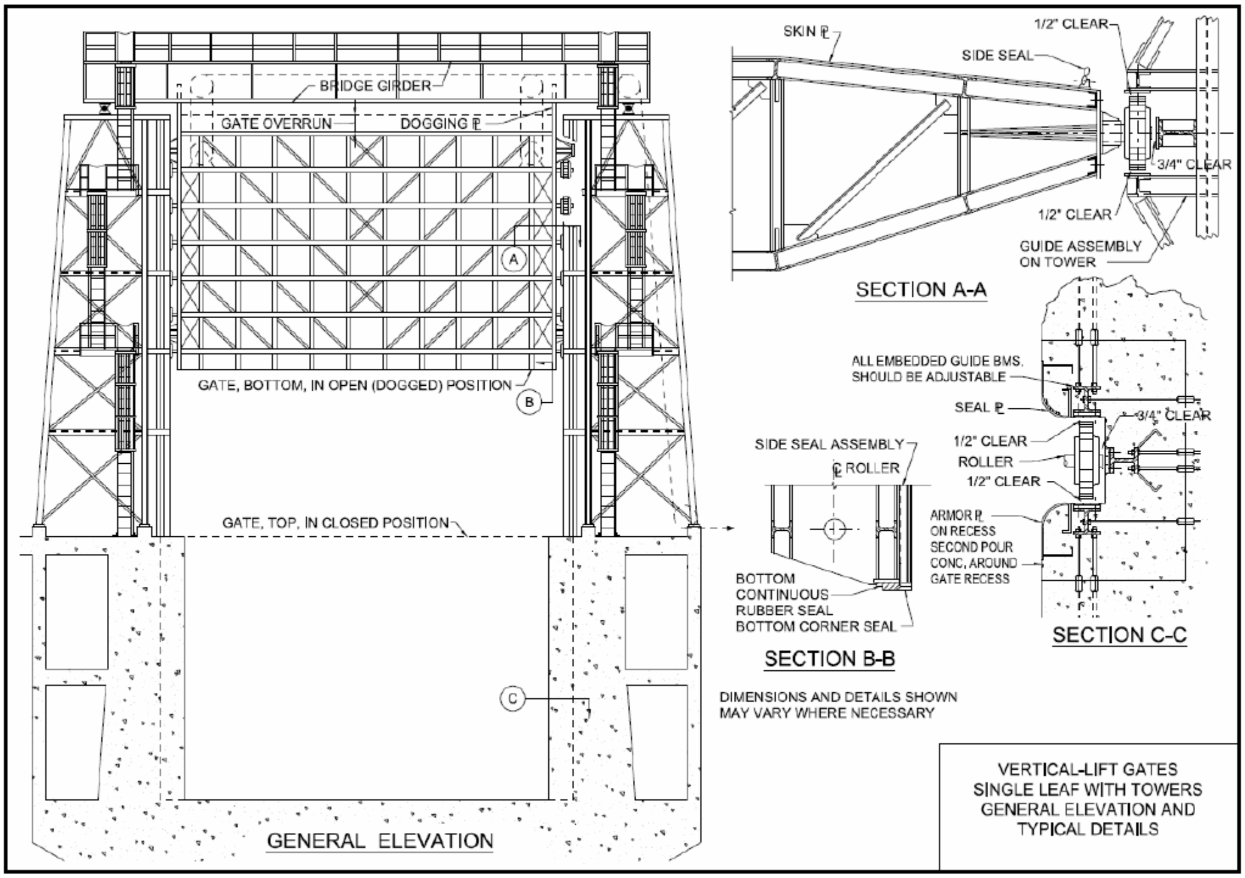


Figure 10: Typical Views of Vertical Lift Gate

Refer to Appendix B.1, Civil for additional information regarding storm surge barriers.

REFERENCES

EM 1110-2-2100. *Stability Analysis of Concrete Structures*. Department of the Army, U.S. Army Corps of Engineers, Washington, DC 20314-1000.

EM 1110-2-2104. *Strength Design for Reinforced Concrete Hydraulic Structures*. Department of the Army, U.S. Army Corps of Engineers, Washington, DC 20314-1000.

EM 1110-2-2502. *Retaining and Floodwalls*. Department of the Army, U.S. Army Corps of Engineers, Washington, DC 20314-1000.

Engineering and Construction Bulletin 2017-2. *Revision and Clarification of EM 1110-2-2100 and EM 1110-2-250*. Department of the Army, U.S. Army Corps of Engineers, Washington, DC 20314-1000.

ETL 1110-2-584. *Design of Hydraulic Steel Structures*. Department of the Army, U.S. Army Corps of Engineers, Washington, DC 20314-1000.

Engineering and Construction Bulletin 2019-10. *Guidance for Design of Hydraulic Steel Structures*. Department of the Army, U.S. Army Corps of Engineers, Washington, DC 20314-1000.

Engineering and Construction Bulletin 2021-6. *Guidance for Design of Hydraulic Steel Structures and Design and Evaluation of I-Walls Including Sheet Pile Walls*. Department of the Army, U.S. Army Corps of Engineers, Washington, DC 20314-1000.

ER 1110-2-8157. *Responsibility for Hydraulic Steel Structures*. Department of the Army, U.S. Army Corps of Engineers, Washington, DC 20314-1000.

AISC. *Steel Construction Manual, 15th Edition*. American Institute of Steel Construction, Chicago, IL 60601.

AASHTO. *The Manual for Bridge Evaluation, 3rd Edition*. American Association of State highway and Transportation Officials, Washington, DC 20004.

ENGINEERING APPENDIX
HYDROLOGY, HYDRAULICS AND COASTAL

NEW JERSEY BACK BAYS
COASTAL STORM RISK MANAGEMENT
FEASIBILITY STUDY

PHILADELPHIA, PENNSYLVANIA

APPENDIX B.4

August 2021

NJBB
NEW JERSEY BACK BAYS STUDY

U.S. Army Corps of Engineers
Philadelphia District

THIS PAGE HAS BEEN INTENTIONALLY LEFT BLANK

Table of Contents

List of Figures

List of Tables

B-4) HYDROLOGY, HYDRAULICS AND COASTAL

Introduction

This appendix presents the results of the Hydraulic, Hydrology and Coastal (HH&C) engineering evaluation and analysis for the New Jersey Back Bays (NJBB) Coastal Storm Risk Management (CSRM) Study. The NJBB study area is shown in **Figure 13**. This report will discuss in detail all the existing information that was reviewed and how that information was used in the HH&C engineering evaluation and analysis to come up with the contribution of the elements to get to the TSP Milestone and Draft Feasibility Report for the study.

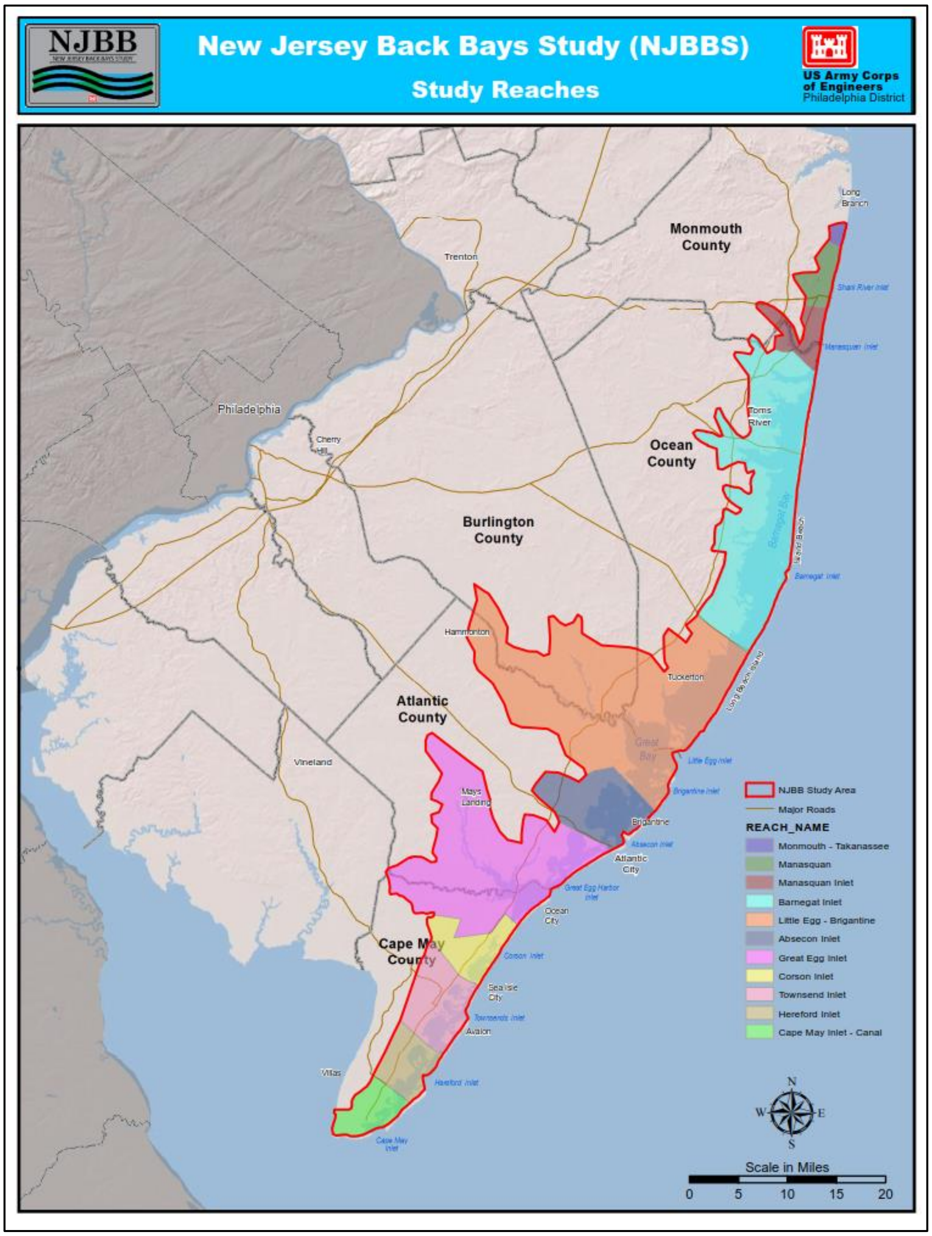


Figure 1: Study Area and Regions

Vertical Datum

In accordance with ER 1110-2-8160 the NJBB Feasibility Study is designed to North American Vertical Datum of 1988 (NAVD88), the current orthometric vertical reference datum within the National Spatial Reference System (NSRS) in CONUS. The study area is subject to tidal influence and is directly referenced to National Water Level Observation Network (NWLON) tidal gages and coastal hydrodynamic tidal models established and maintained by the U.S. Department of Commerce (NOAA). The current NWLON National Tidal Datum Epoch (NTDE) is 1983-2001.

More than one NWLON tidal gage is required to reference tidal water levels to NAVD88 due to the vast size of the study area. Four NWLON tidal stations within the study area are as presented in **Table 16**. The location of NOAA tidal stations is shown in **Error! Reference source not found.14**. The local NAVD88-MSL relationship at locations between gages is estimated using NOAA VDatum models of the project region (EM 1110-2-6056).

Table 1: NOAA Tidal Gage Datum Relationships

Datum[1]	Cape May	Atlantic City	Barnegat Inlet	Sandy Hook
	(Feet)	(Feet)	(Feet)	(Feet)
MHHW	2.42	1.99	1.33	2.41
MHW	1.99	1.58	1.10	2.09
NAVD88	0.00	0.00	0.00	0.00
MSL	-0.45	-0.40	-0.02	-0.23
MLW	-2.86	-2.44	-1.06	-2.62
MLLW	-3.02	-2.61	-1.18	-2.81
MN[2]	4.85	4.02	2.16	4.71

Notes: [1]Tidal datums based on 1983-2001 Tidal Epoch

[2]Mean Tidal Range (MHW-MLW)

Hydrodynamic modeling completed for this study was performed in meters, MSL in the current NTDE. Water elevations are converted to feet, NAVD88 using NOAA VDatum. VDatum is a vertical datum transformation software tool, that provides conversions between various tidal datums fields and mean sea level as well as between mean sea level and North American Vertical Datum of 1988 (NAVD88). The tidal datums fields (MHHW, MHW, MSL, MLW, MLLW) are derived from hydrodynamic simulations using the hydrodynamic model, ADCIRC (Yang et al. 2008). NOAA ADCIRC model results were validated by comparing with observations water level stations maintained by the NOAA's Center for Operational Oceanographic Products and Services (CO-OPS). **Figure 14** presents the mean tidal range (MHW - MLW) for the study area. **Table 17** presents the NOAA VDatum results for MHHW and mean tidal range (MN) at the four NOAA tidal stations. Comparison of the values in in **Table 16** and **Table 17** show that the VDatum results are in agreement with the NOAA tidal stations.

Table 2: NOAA VDatum Tidal Datum Relationships

Datum[1]	Cape May	Atlantic City	Barnegat Inlet	Sandy Hook
MHHW	2.42	1.99	1.34	2.40
MN[2]	4.85	4.02	2.14	4.66

Notes: [1]Tidal datums based on 1983-2001 Tidal Epoch

[2]Mean Tidal Range (MHW-MLW)

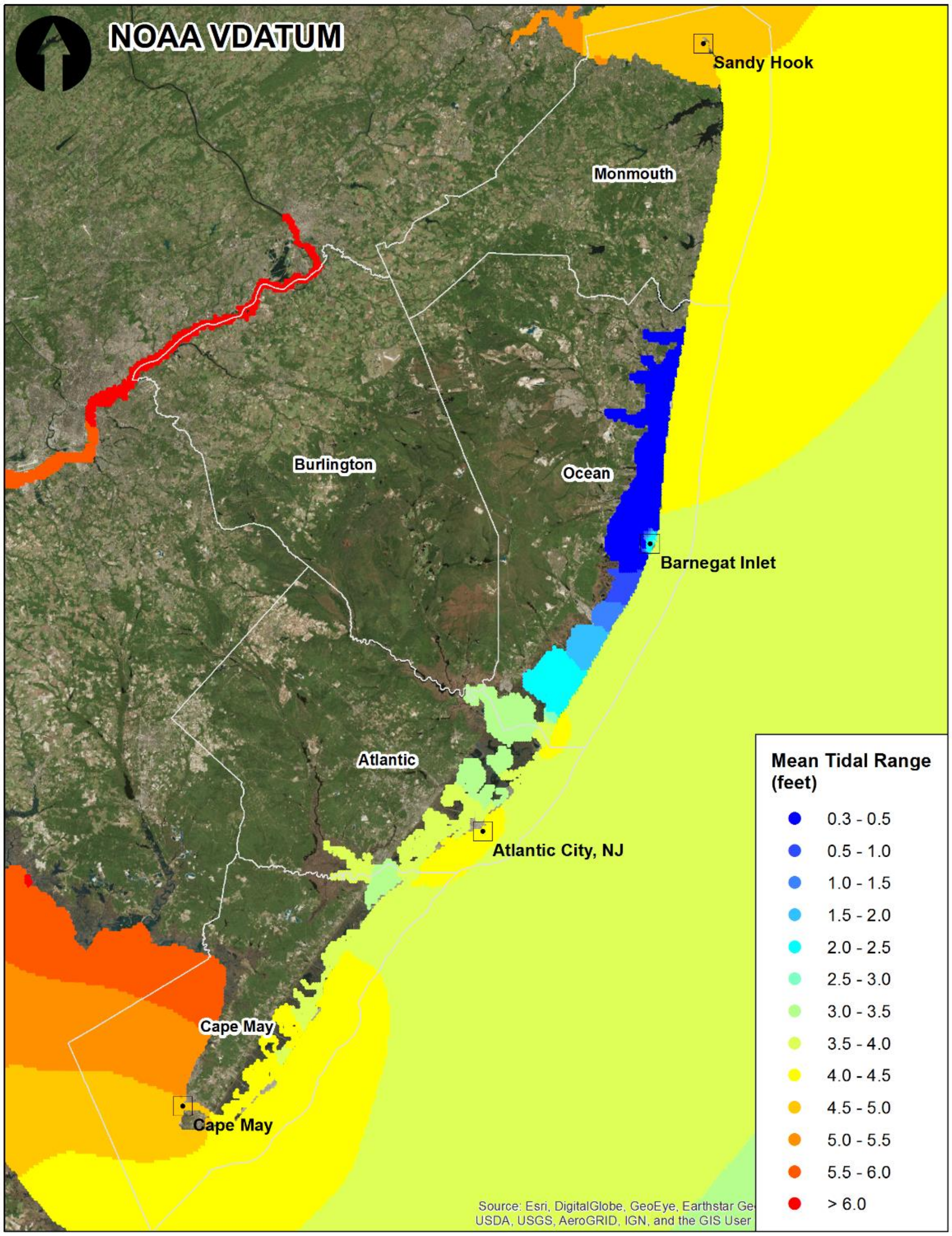


Figure 2: Mean Tidal Range in Study Area

Sea Level Change

Background on SLC

Global sea level change (SLC) is often caused by the global change in the volume of water in the world's oceans in response to three climatological processes: 1) ocean mass change associated with long-term forcing of the ice ages ultimately caused by small variations in the orbit of the earth around the sun; 2) density changes from total salinity; and most recently, 3) changes in the heat content of the world's ocean, which recent literature suggests may be accelerating due to global warming. Global SLC can also be caused by basin changes through such processes as seafloor spreading. Thus, global sea level, also sometimes referred to as global mean sea level, is the average height of all the world's oceans.

Relative (local) SLC is the local change in sea level relative to the elevation of the land at a specific point on the coast. Relative SLC is a combination of both global and local SLC caused by changes in estuarine and shelf hydrodynamics, regional oceanographic circulation patterns (often caused by changes in regional atmospheric patterns), hydrologic cycles (river flow), and local and/or regional vertical land motion (subsidence or uplift).

USACE Guidance

In accordance with ER 1100-2-8162, potential effects of relative sea level change (RSLC) were analyzed over a 50-yr economic analysis period and a 100-yr planning horizon. Research by climate science experts predict continued or accelerated climate change for the 21st century and possibly beyond, which would cause a continued or accelerated rise in global mean sea level. ER 1100-2-8162 states that planning studies will formulate alternatives over a range of possible future rates of SLC and consider how sensitive and adaptable the alternatives are to SLC.

ER 1100-2-8162 requires planning studies and engineering designs consider three future sea level change scenarios: low, intermediate, and high. The historic rate of SLC represents the "low" rate. The "intermediate" rate of SLC is estimated using the modified National Research Council (NRC) Curve I. The "high" rate of SLC is estimated using the modified NRC Curve III. The "high" rate exceeds the upper bounds of IPCC estimates from both 2001 and 2007 to accommodate the potential rapid loss of ice from Antarctica and Greenland, but it is within the range of values published in peer-reviewed articles since that time.

Historical SLC

Historical RSLC for this study (4.09 mm/yr) is based on NOAA tidal records at Atlantic City, NJ. Additional historic RSLC rates within the study area are available at Cape May, NJ (4.63 mm/yr) and Sandy Hook, NJ (4.09 mm/yr). **Error! Reference source not found.15** and **Error! Reference source not found.16** show historical RSLC at Atlantic City. Several metrics for sea level are presented, the monthly mean sea level (light blue), 5-year moving average (orange), and 19-year moving average (dark blue). It is apparent that over long-time scales (19 years) mean sea level is steadily increasing. However, over shorter time scales mean sea level may increase or decrease. The monthly mean sea level, light blue line in **Error! Reference source not found.16**, goes up and down every year capturing the seasonal cycle in mean sea level and is slightly different in the two figures based moving average for the time period shown. The 5-year moving average, orange line in **Error! Reference source not found.15** captures the interannual variation (2 or more years).

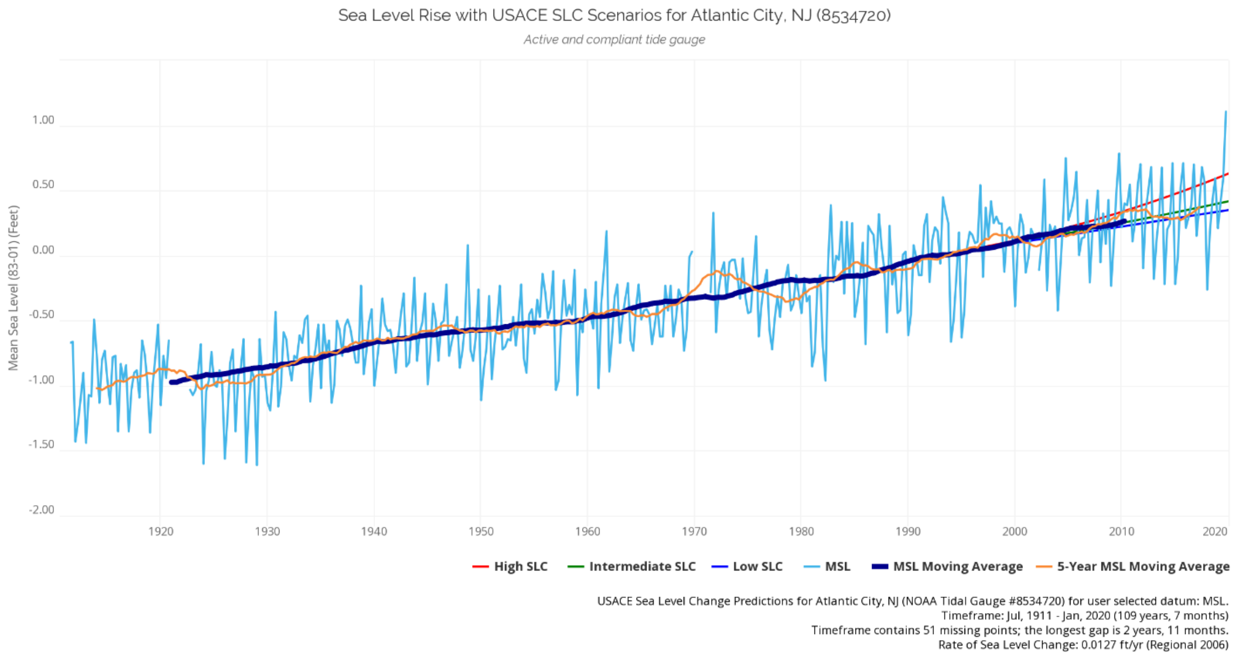


Figure 3: Historical (1911-2020) Relative Sea Level Change at Atlantic City, NJ

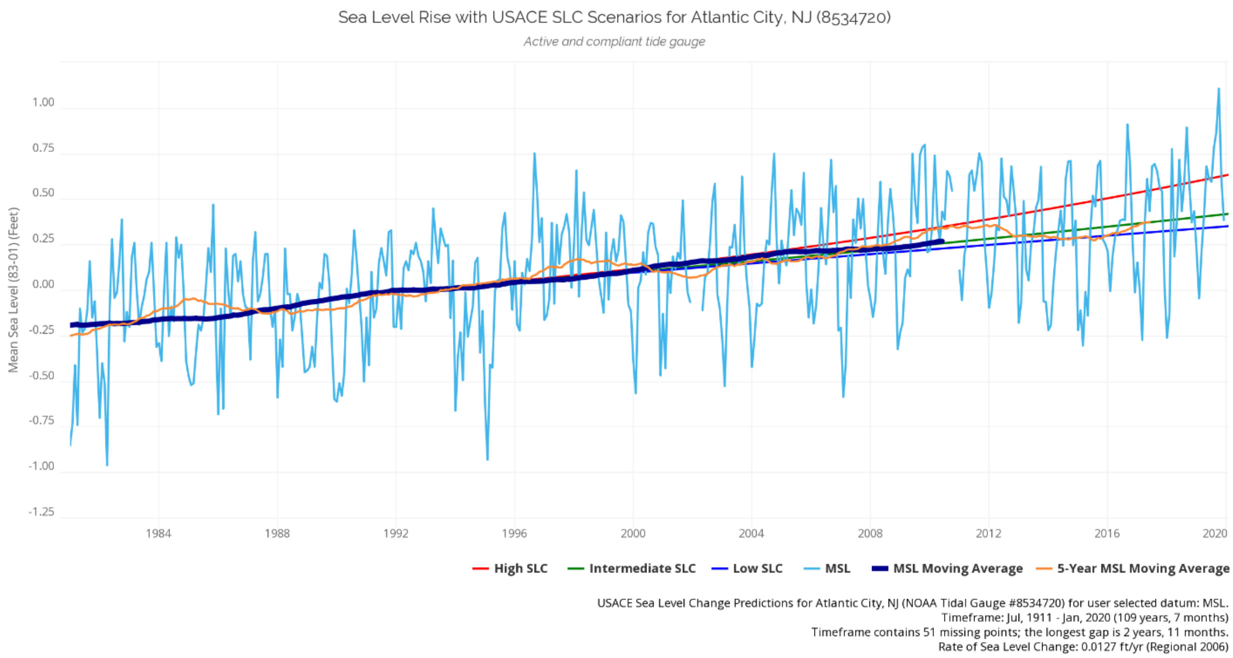


Figure 4: Historical (1983-2020) Relative Sea Level Change at Atlantic City, NJ

USACE SLC Scenarios

USACE low, intermediate, and high SLC scenarios over the 100-yr planning horizon at Atlantic City, NJ are presented in **Table 18** and **Error! Reference source not found.17**. Water level elevations at year 2030 are expected to be between 0.5 and 1.0 feet higher than the current NTDE. Water elevations at year 2080 are expected to be between 1.15 and 4.02 feet higher than the current NTDE.

Hydrodynamic modeling performed for this study was completed in the current NTDE. Therefore, the modeled water levels represent MSL in 1992. Future water levels are determined by adding the SLC values in **Table 18**. For example, a water level elevation of 10 feet NAVD88 based on the current National Tidal Datum Epoch (1983-2001), will have an elevation in the year 2080 of 11.15, 11.84, and 14.02 feet NAVD88 under the USACE low, intermediate, and high SLC scenario respectively.

Table 3: USACE Sea Level Change Scenarios (Derived from Atlantic City, NJ)

Year	USACE - Low (ft, MSL[1])	USACE - Int (ft, MSL[1])	USACE - High (ft, MSL[1])
1992	0.00	0.00	0.00
2000	0.11	0.11	0.13
2019	0.35	0.42	0.62
2030	0.50	0.63	1.03
2050	0.76	1.06	2.01
2080	1.15	1.84	4.02
2100	1.41	2.54	5.74
2130	1.81	3.50	8.87

[1]Mean Sea Level based on National Tidal Datum Epoch (NTDE) of 1983-2001

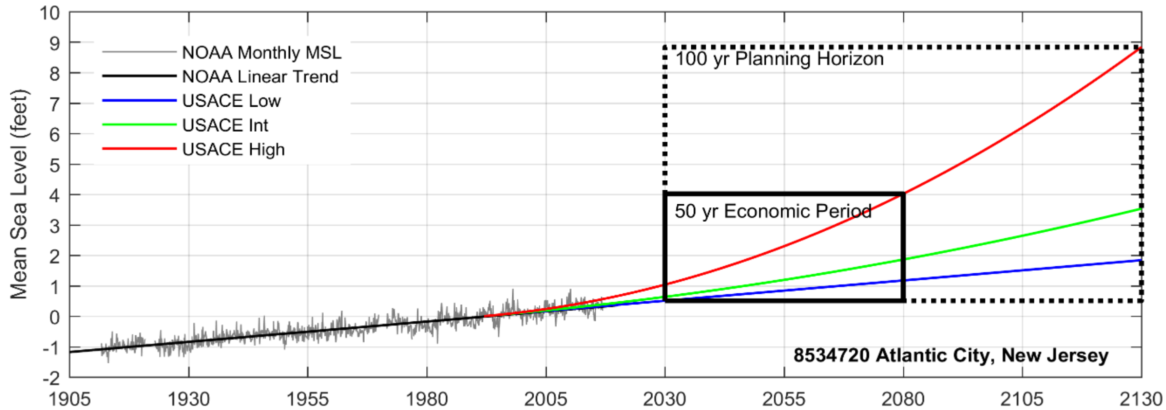


Figure 5: Relative Sea Level Change Projections at Atlantic City, NJ

NJ Science and Technical Advisory Panel (STAP)

NJ Climate Adaptation Alliance convened a 2nd Science and Technical Advisory Panel (STAP) in 2019 to identify and evaluate the most current science on sea level rise projections and changing coastal storms, consider the implications for the practices and policies of local and regional stakeholders, and provide practical options for stakeholders to incorporate science into risk-based decision processes. The 2019 report titled "New Jersey's Rising Seas and Changing Coastal Storms: Report of the 2019 Science and Technical Advisory Panel" (Kopp et al. 2019) contains a detailed description of the basis for the STAP's projected SLR estimates. The following is an excerpt from the Executive Summary on SLR conclusions:

1. *From 1911 (the start of the Atlantic City tide-gauge record) to 2019, sea-level rose 17.6 inches (1.5 feet) along the New Jersey coast, compared to a 7.6-inch (0.6 feet) total change in the global mean sea-level.*
2. *Over the last forty years, from 1979-2019, sea-level rose 8.2 inches (0.7 feet) along the New Jersey coast, compared to a 4.3-inch (0.4 feet) change in global mean sea-level.*
3. *New Jersey coastal areas are likely (at least a 66% chance) to experience SLR of 0.5 to 1.1 ft between 2000 and 2030, and 0.9 to 2.1 ft between 2000 and 2050. It is extremely unlikely (less than 5% chance) that SLR will exceed 1.3 ft by 2030 and 2.6 ft by 2050.*
4. *While near-term SLR projections through 2050 exhibit only minor sensitivity to different emissions scenarios (<0.1 feet), SLR projections after 2050 increasingly depend upon the pathway of future global greenhouse gas emissions.*
    a. *Under a high-emissions scenario, consistent with the strong, continued growth of fossil fuel consumption, coastal areas of New Jersey are likely (at least a 66% chance) to see SLR of 1.5 to 3.5 ft between 2000 and 2070, and 2.3 to 6.3 ft between 2000 and 2100. It is extremely unlikely (less than a 5% chance) that SLR will exceed 4.4 ft by 2070 and 8.8 ft by 2100.*
    b. *Under a moderate-emissions scenario, roughly consistent with current global policies, coastal areas of New Jersey are likely (at least a 66% chance) to see SLR of 1.4 to 3.1 ft between 2000 and 2070, and 2.0 to 5.2 ft between 2000 and 2100. It is extremely unlikely (less than a 5% chance) that SLR will exceed 3.8 ft by 2070 and 6.9 ft by 2100.*

c. *Under a low-emissions scenario, consistent with the global goal of limiting warming to 2oC above early industrial (1850-1900) levels, coastal areas of New Jersey are likely (at least a 66% chance) to see SLR of 1.3 to 2.7 ft between 2000 and 2070, and 1.7 to 4.0 ft between 2000 and 2100. It is extremely unlikely (less than a 5% chance) that SLR will exceed 3.2 ft by 2070 and 5.0 ft by 2100.*

STAP Projected SLC Estimates, **Table 19**, are with respect to mean sea level from 1991-2009, with a midpoint of 2000. USACE SLC estimates are based on mean sea level over the current National Tidal Datum Epoch (1983-2001) with a midpoint of 1992. The STAP moderate emissions scenario falls between the USACE intermediate and high scenarios as shown in **Error! Reference source not found. 18**.

Table 4: STAP Projected SLC Estimates for New Jersey (Kopp et al. 2019)

		2030	2050	2070			2100			2150		
				Emissions								
	Chance SLR Exceeds			Low	Mod.	High	Low	Mod.	High	Low	Mod.	High
Low End	> 95% chance	0.3	0.7	0.9	1	1.1	1.0	1.3	1.5	1.3	2.1	2.9
Likely Range	> 83% chance	0.5	0.9	1.3	1.4	1.5	1.7	2.0	2.3	2.4	3.1	3.8
	~50 % chance	0.8	1.4	1.9	2.2	2.4	2.8	3.3	3.9	4.2	5.2	6.2
	<17% chance	1.1	2.1	2.7	3.1	3.5	3.9	5.1	6.3	6.3	8.3	10.3
High End	< 5% chance	1.3	2.6	3.2	3.8	4.4	5.0	6.9	8.8	8.0	13.8	19.6

**2010 (2001-2019 average) Observed = 0.2 ft*

Notes: All values are 19-year means of sea-level measured with respect to a 1991-2009 baseline centered on the year indicated in the top row of the table. Projections are based on Kopp et al. (2014), Rasmussen et al. (2018), and Bamber et al. (2019). Near-term projections (through 2050) exhibit only minor sensitivity to different emissions scenarios (<0.1 feet). Low and high emissions scenarios correspond to global-mean warming by 2100 of 2°C and 5°C above early Industrial (1850-1900) levels, respectively, or equivalently, about 1°C and 4°C above the current global-mean temperature. Moderate (Mod.) emissions are interpolated as the midpoint between the high- and low-emissions scenarios and approximately correspond to the warming expected under current global policies. Rows correspond to different projection probabilities. There is at least a 95% chance of SLR exceeding the values in the 'Low End' row, while there is less than a 5% chance of exceeding the values in the 'High End' row. There is at least a 66% chance that SLR will fall within the values in the 'Likely Range'. Note that alternative methods may yield higher or lower estimates of the chance of low-end and high-end outcomes.

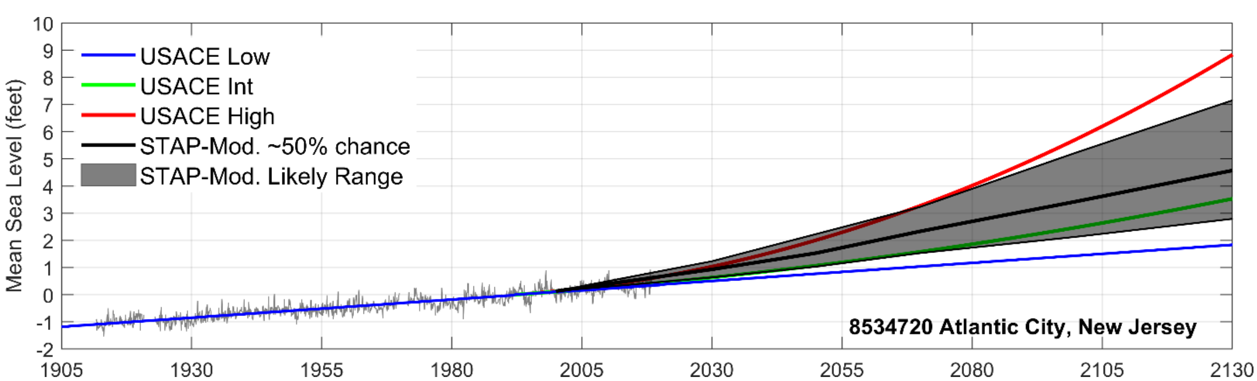


Figure 6: USACE and STAP RSLC Projections at Atlantic City, NJ

Existing Conditions

Astronomical Tide

Daily tidal fluctuations at the project site are semi-diurnal, with a full tidal period that averages 12 hours and 25 minutes; hence there are nearly two full tidal cycles per day. The mean tidal range in the ocean is 4.0 feet at Atlantic City. The rise and fall of the tide in the ocean lead to tidal flow through the inlets that causes a corresponding rise and fall of water levels in the back bays **Figure 14** shows the mean tidal range for the study area.

The southern half of the study area, from Little Egg Harbor Inlet south to Cape May Inlet, experiences a mean tide range that is only slightly reduced relative to the mean range in the open ocean at Atlantic City, typically in the 3.5 to 4.0 foot mean range. This is due to the relatively shorter distance along the coast between inlets, and the relatively short distances from the open ocean, through the inlets, to the inland extent of the bays.

North of Little Egg Harbor Inlet the mean tide range in the back bays gradually decreases such that at Mantoloking, near the head of Barnegat bay, the mean range is about 0.9 feet. The reduction in mean tide range is due to the long, narrow, and shallow geometry of Barnegat Bay and the relatively greater distances between inlets; it is about 24 miles from Manasquan Inlet south to Barnegat Inlet, and then an additional 21 miles south to Little Egg Harbor Inlet.

Seasonal and Interannual Fluctuations in Sea Level

The average seasonal cycle of mean sea level, shown in **Figure 19Error! Reference source not found.**, is caused by <u>regular fluctuations</u> in coastal temperatures, salinities, winds, atmospheric pressures, and ocean currents and on average causes a 0.5 foot (0.17 m) difference in sea level from September (highest) to January (lowest).

Interannual (2 or more years) variations in sea level, shown in **Figure 20**, are caused by <u>irregular fluctuations</u> in coastal ocean temperatures, salinities, winds, atmospheric pressures, and ocean currents (El Niño).

Seasonal and interannual fluctuations in sea level are significant in the study area and will be incorporated in design water elevations in subsequent phases of the feasibility study.

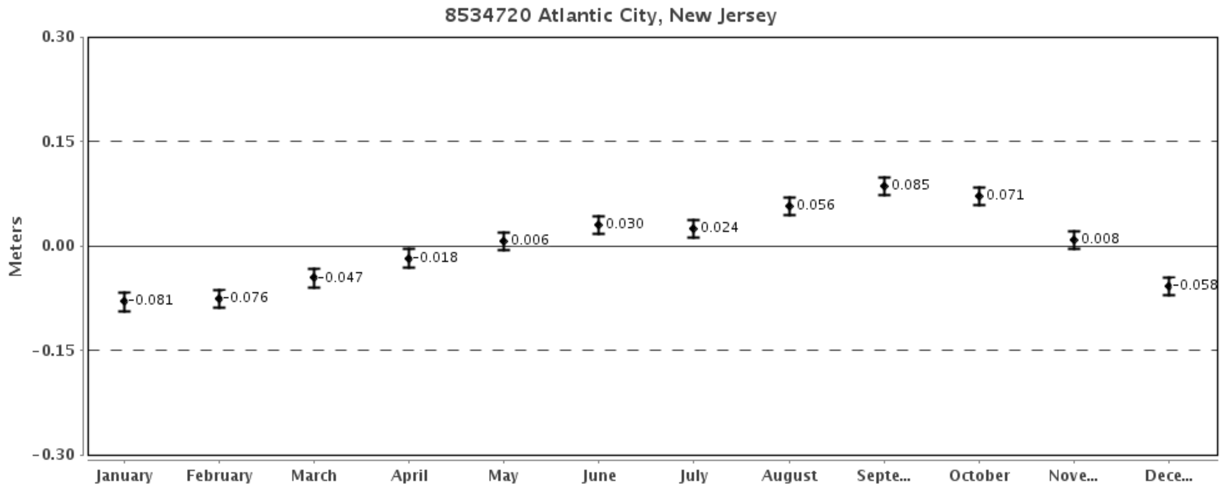


Figure 7: Average Seasonal Cycle in Sea Level at Atlantic City, NJ

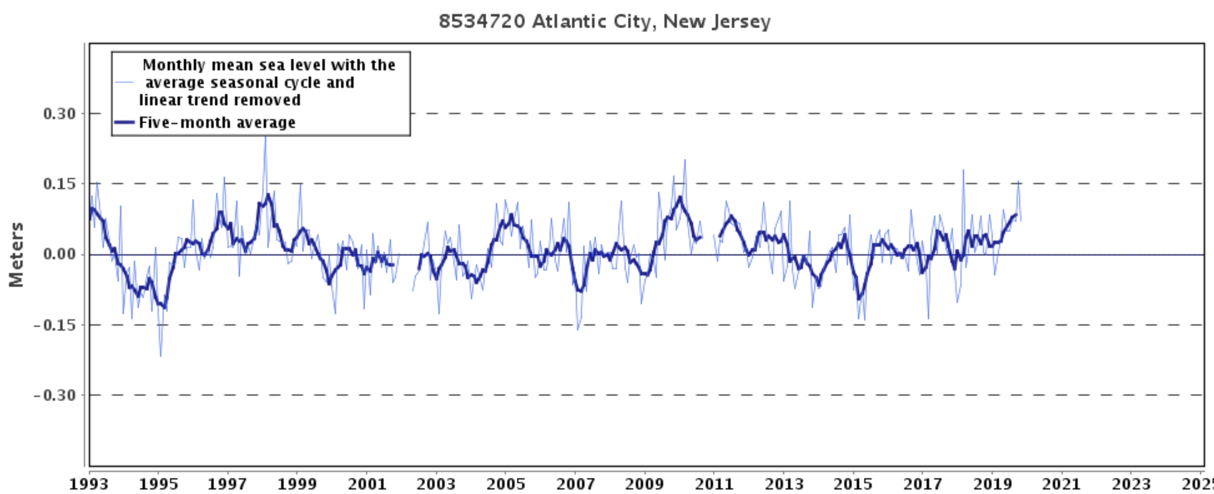


Figure 8: Interannual Variation in Sea Level at Atlantic City, NJ

Storm Surge

Storm surge is the increased water level above the predicted astronomical tide due to storm winds over the ocean and the resultant wind stress on the ocean surface. The principal factor that creates flood risk for the study area is storm surge that propagates into the back bays through the twelve inlets distributed along the New Jersey coast. The magnitude of the storm surge is calculated as the difference between the predicted astronomic tidal elevation and the actual water surface elevation at any time. Wind blowing over the ocean surface is capable of generating storm surge. However, the largest and most damaging storm surges develop as a result of either tropical cyclones (hurricanes and tropical storms) or extra-tropical cyclones (“nor’easters”). Although the meteorological origins of the two types of storms differ, both can generate large, low-pressure atmospheric systems with intense wind fields that rotate counterclockwise (in the northern hemisphere). The relatively broad and shallow continental shelf along the east coast allows the generation of larger storm surge values than are typically experience on the US Pacific coast.

Storm surge propagation into the back bays broadly mirror the tidal propagation, with storm surge in the southern portions of the study area in similar magnitude to the ocean coastline and attenuated storm surge in Barnegat Bay. However, storm surge in Barnegat Bay is highly dependent on wind speed and direction. Strong winds are capable of “pushing” accumulated storm surge from either the southern end or northern end of the Little Egg Harbor-Barnegat Bay system in the direction that the wind is blowing. The effect of the wind is that storm surges at the southern or northern ends of the Little Egg Harbor-Barnegat Bay system may be similar in elevation to storm surge elevations on the ocean even though tidal amplitudes in the bay are muted relative to the ocean. Storm surge elevations along the middle of the bay are lowest, and generally less than the ocean, because the wind effects are less signficant.

Waves

Wave conditions in the NJBB study area are fetch-limited and generated by local wind conditions. In fetch-limited conditions, wave heights are limited by the distance of open water in which the waves are able to grow. Wave conditions throughout the bay are also affected by the shallow water depths, marshes, and orientation relative to the wind directions. The 100-year wave conditions in the back bays are generally between 3 and 4 feet with a peak wave period of 3 to 4 seconds. At some back bay locations wave conditions may be dominated vessel wakes.

The ocean coastline and inlets are exposed to significantly greater wave energy associated with the ocean. Wave conditions offshore may exceed 30 feet during 100-year wave conditions with peak wave periods between 9 and 16 seconds. Wave conditions inside the inlets are affected by complex wave transformation process (wave refraction, shoaling, breaking, diffraction, reflection, and wave-current interactions) associated with the dynamic bathymetry and ebb shoals and rubble mound structures (jetties).

Historical Storms

The study area has experienced flooding from both tropical cyclones and extratropical cyclones. **Table 20** displays the top ten historical storms at Cape May, Atlantic City, and Sandy Hook NOAA tidal stations. Note that the historical water levels have not been adjusted for sea level rise.

Table 5: Historical Peak Water Levels at NOAA Stations

Cape May, NJ (since 1965)			Atlantic City, NJ (since 1911)			Sandy Hook, NJ (since 1932)		
Date	**Type**	**Feet NAVD88**	**Date**	**Type**	**Feet NAVD88**	**Date**	**Type**	**Feet NAVD88**
23-Jan-2016	E	5.96	11-Dec-1992	E	6.37	29-Oct-2012	T	11.30
29-Oct-2012	T	5.87	14-Sep-1944	T	6.23	12-Sep-1960	T	7.27
27-Sep-1985	T	5.79	29-Oct-2012	T	6.15	11-Dec-1992	E	7.26

29-Oct-2011	E	5.67	27-Sep-1985	T	5.96	28-Aug-2011	T	6.95
25-Oct-1980	E	5.64	31-Oct-1991	E	5.85	7-Nov-1953	E	6.87
11-Dec-1992	E	5.53	6-Mar-1962	E	5.83	6-Mar-1962	E	6.57
4-Jan-1992	E	5.52	9-Aug-1976	T	5.83	14-Sep-1944	T	6.57
3-Mar-1994	E	5.50	25-Nov-1950	E	5.63	13-Mar-2010	E	6.21
28-Aug-2011	T	5.37	29-Mar-1984	E	5.38	25-Nov-1950	E	6.17
14-Oct-1977	T	5.25	23-Jan-2016	E	5.23	12-Nov-1968	E	5.99

High-Frequency Flooding

High-frequency flooding, also known as nuisance flooding, recurrent flooding, or sunny-day flooding, are flood events caused by tides and/or minor storm surge that occur more than once per year. High-frequency flooding mostly affects low-lying and exposed assets or infrastructure, such as roads, public storm-, waste- and fresh-water systems (Sweet et. al 2018) and is likely more disruptive (a nuisance) than damaging. However, the cumulative effects of high-frequency flooding may be a serious problem to residents who live and work in these low-lying areas. The number of high-frequency flood days is accelerating in the study area in response to RSLC.

Flooding from rainfall and inadequate stormwater systems are closely related to high-frequency flooding but are treated separated in this study. It is common for municipalities in the study area to have gravity-based stormwater systems that are unable to drain water when tidal level exceeds the elevation of the storm drain. When this happens, water starts ponding around the drain and may flood many of the same low-lying areas as high-frequency flooding. The frequency and impact of rainfall flooding will increase as the probability of the tide level exceeding storm drains will increases in response to RSLC. Some municipalities are addressing this problem by installing pump stations that are capable of draining water during elevated water levels.

National Weather Service Flood Stages

The National Weather Service (NWS) with the help of NOAA and USGS provide real time flood status of stream gages and tidal stations (**Figure 21**). The National Weather Service (NWS) has established three coastal flood severity thresholds: minor, moderate, and major flood stages. The NWS minor and moderate flood stages are the most representative of high-frequency flooding events right now. However, all three flood stages will be evaluated here since NWS major flood stage could eventually occur at frequency consistent with high-frequency flooding in the future in response to RSLC.

The definition of minor, moderate, and major flooding is provided herein by NWS. The definitions are taken from the NWS website for Atlantic City, NJ so that impacts are specific to Ocean and Atlantic County. However, impacts experienced described at this station are generally representative of the entire study area.

- **Minor Flooding** - Minimal or no property damage, but possibly some public threat;
- **Moderate Flooding** - widespread flooding of roadways begins due to high water and/or wave action with many roads becoming impassable in the coastal communities of Ocean

County and Atlantic County. Lives may be at risk when people put themselves in harm's way. Some damage to vulnerable structures may begin to occur;

- **Major Flooding** - flooding starts to become severe enough to begin causing structural damage along with widespread flooding of roadways in the coastal communities of Ocean County and Atlantic County. Vulnerable homes and businesses may be severely damaged or destroyed as water levels rise further above this threshold. Numerous roads become impassable and some neighborhoods may be isolated. The flood waters become a danger to anyone who attempts to cross on foot or in a vehicle.

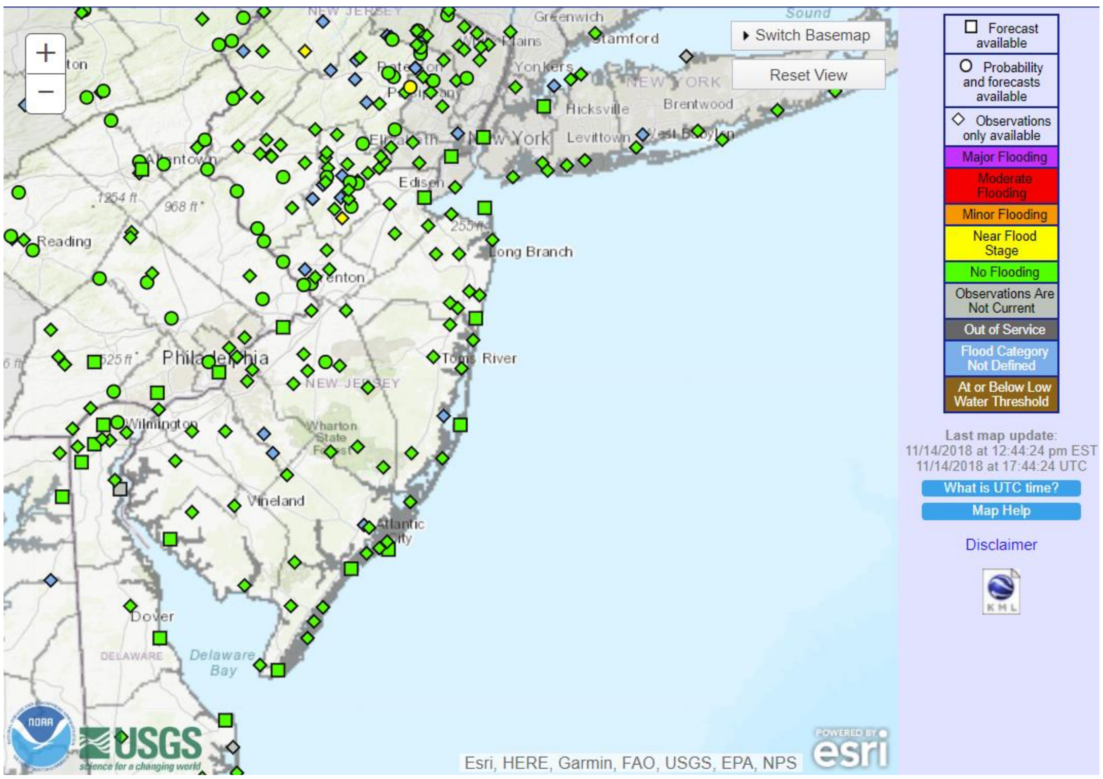


Figure 9: NWS Real-Time Flood Monitoring Network

An example of the flood inundation area associated with the three NWS Flood stages is shown in **Figure 22**, **Figure 23**, and **Figure 24** at Atlantic City, Wildwood, and Cape May. The impact of minor flooding can be seen to be very limited to a few particularly low-lying areas. The impact of moderate flooding is more widespread impacting some streets and properties and major flooding is widespread impacting several streets and blocks near the bay shoreline.

There are 17 NWS stations in the study area with documented flood stages. The flood stages are reported on the NWS website in feet MLWW:

https://water.weather.gov/ahps/region.php?state=nj

The NWS flood stages are converted to feet NAVD88 in **Table 21** for floodplain mapping. NWS minor flood stages are typically 1 to 1.5 feet above MHHW. Moderate and major flood stages are an additional 1 and 2 feet, respectively, above the minor flood stage. The NWS minor flood stage elevations are pretty consistent across the study area, 3.2 to 3.7 feet NAVD88, with the exception of Barnegat Bay where the tidal range is smaller.

Table 6: NWS Flood Stages

Location	Gage	Minor	Moderate	Major
		NAVD88		
Belmar	BLMN4	3.7	4.7	5.7
Manasquan	MSNN4	3.2	4.2	5.2
Mantaloking	MTLN4	1.4	2.4	3.4
Bayshore	BASN4	1.4	2.4	3.4
Barnegat Light	BGLN4	2.3	3.3	4.3
Ship Bottom	SBTN4	2.1	3.1	4.1
Tuckerton	TKTN4	2.6	3.6	4.6
Atlantic City Marina	ATLN4	3.3	4.3	5.3
Atlantic City	ALCN4	3.5	4.5	5.5
Atlantic City (ocean front)	ACYN4	3.4	4.4	5.4
Margate	MGTN4	3.3	4.3	5.3
Ocean City	ONCN4	3.2	4.2	5.2
Sea Isle City	SICN4	3.3	4.3	5.3
Avalon	AVLN4	3.5	4.5	5.5
Stone Harbor	SHBN4	3.4	4.4	5.4
Cape May	CMAN4	3.7	4.7	5.7
Cape May Harbor	CAPN4	3.4	4.4	5.4

Note: Locations are sorted from North to South. Grey-shaded locations are in Barnegat Bay.

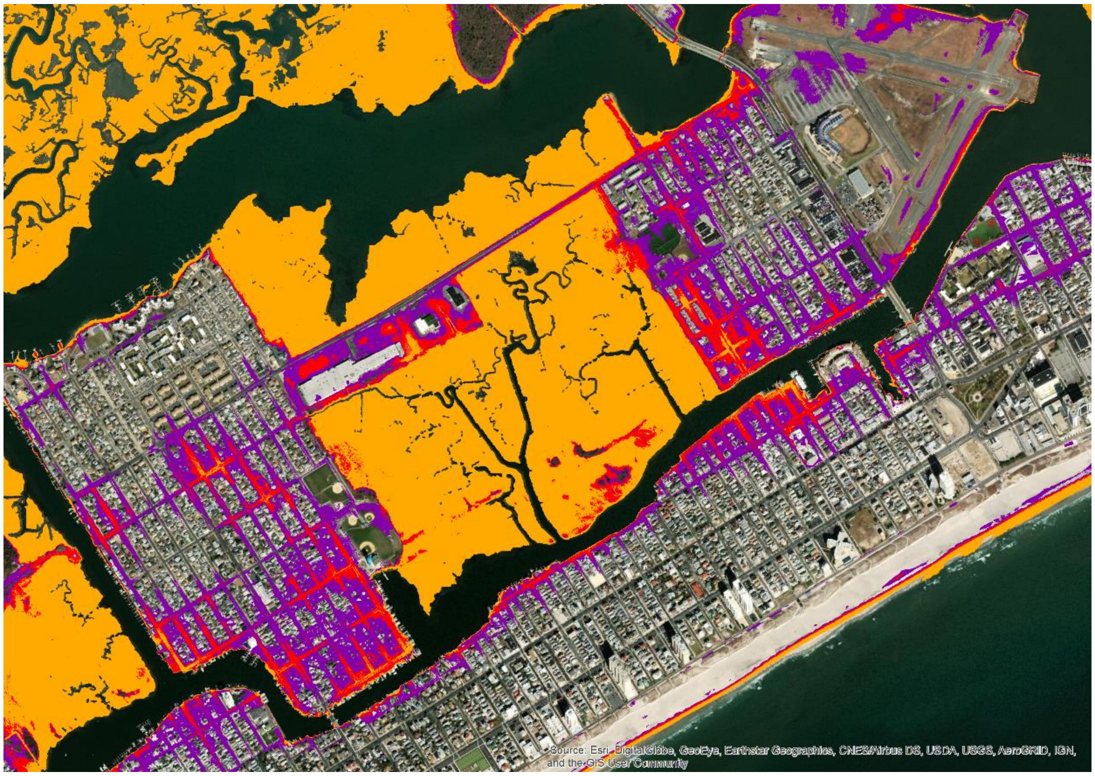


Figure 10: Floodplain associated with NWS Stages at Atlantic City, NJ

Figure 11: Floodplain associated with NWS Stages at Wildwood, NJ

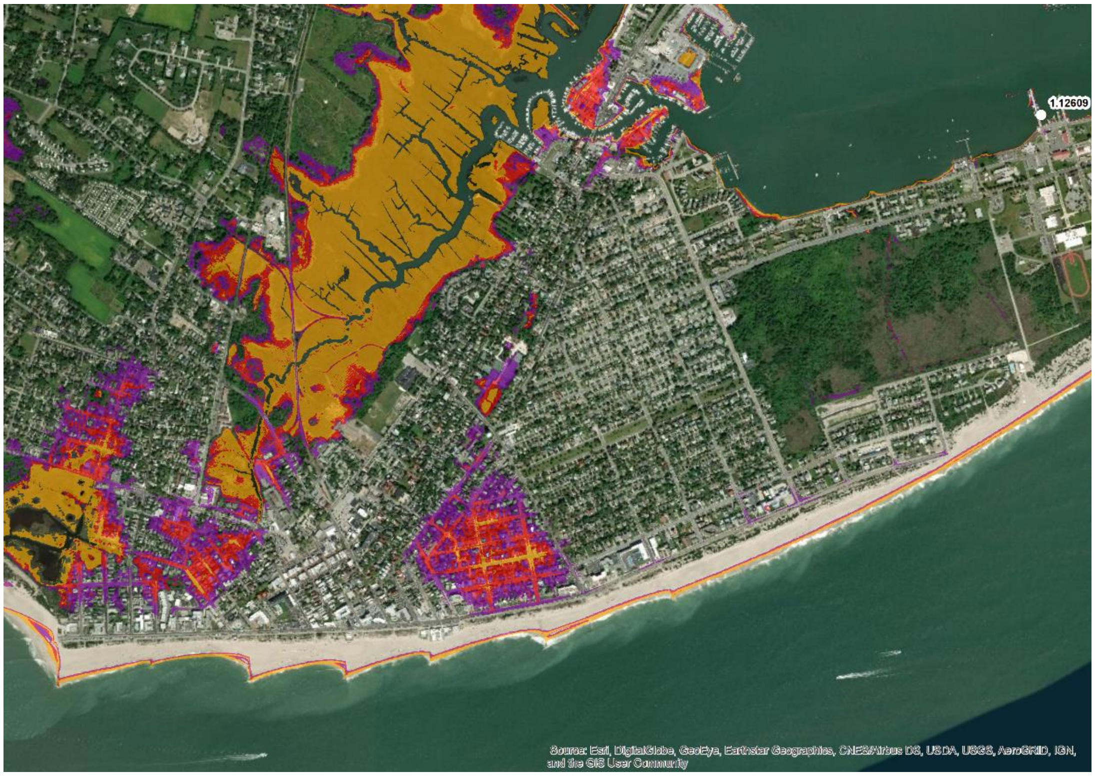


Figure 12: Floodplain associated with NWS Stages at Cape May, NJ

Historical High-Frequency Flooding at Atlantic City, NJ

Atlantic City, NJ has the longest tidal record (1911-Present) out of any of NOAA or USGS stations and is therefore best suited for investigating how often high-frequency flooding has occurred in the past and how rate of flooding has been affected by historic RSLC. Hourly verified data from NOAA CO-OPS station at Atlantic City, NJ was downloaded from 1911-2018. The number of days in which the daily maximum water level equaled or exceeded the NWS flood stages was calculated. The top panel of **Figure 25** shows historic record of water levels and a dot for any day in which the NWS flood stages were exceeded. The bottom panel of **Figure 25** shows a histogram of the total number of days in a given year that the NWS flood stages were exceeded. It is readily observed from **Figure 25** that annual rate of NWS minor flooding has increased over time, with a dramatic increase in the 1990's. The annual rate of NWS moderate flooding has a seen a small but visible increase and with little or no increase in NWS major flooding.

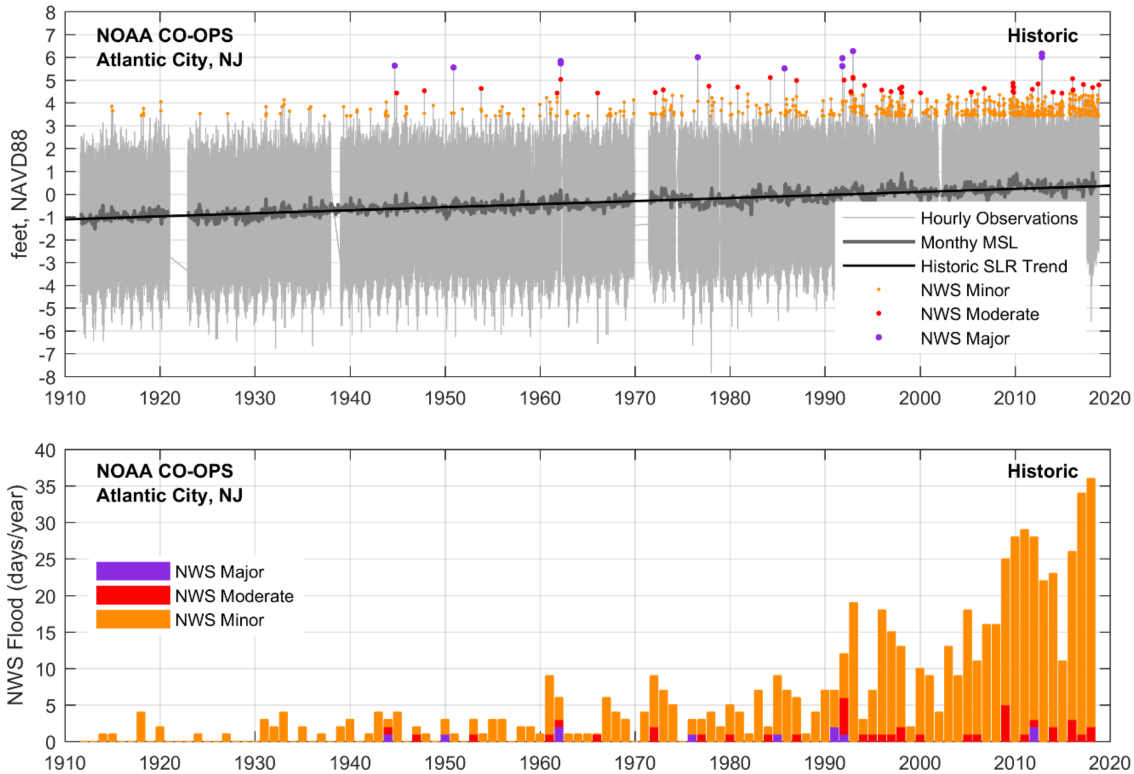


Figure 13: Historic High-Frequency Flooding at Atlantic City, NJ

To isolate the impact of historic RSLC on the frequency of flooding, the analysis was repeated with the historic SLR trend removed so that the mean sea level remained the same as in 1910 over the period of record. **Figure 26** shows that if no RSLC had occurred since 1910, the frequency of NWS minor flooding would be still be a couple times per year, significantly lower than today, and that primary driver of the increase in high-frequency flooding over the last 100 years has been RSLC not changes in the tidal range or meteorological conditions.

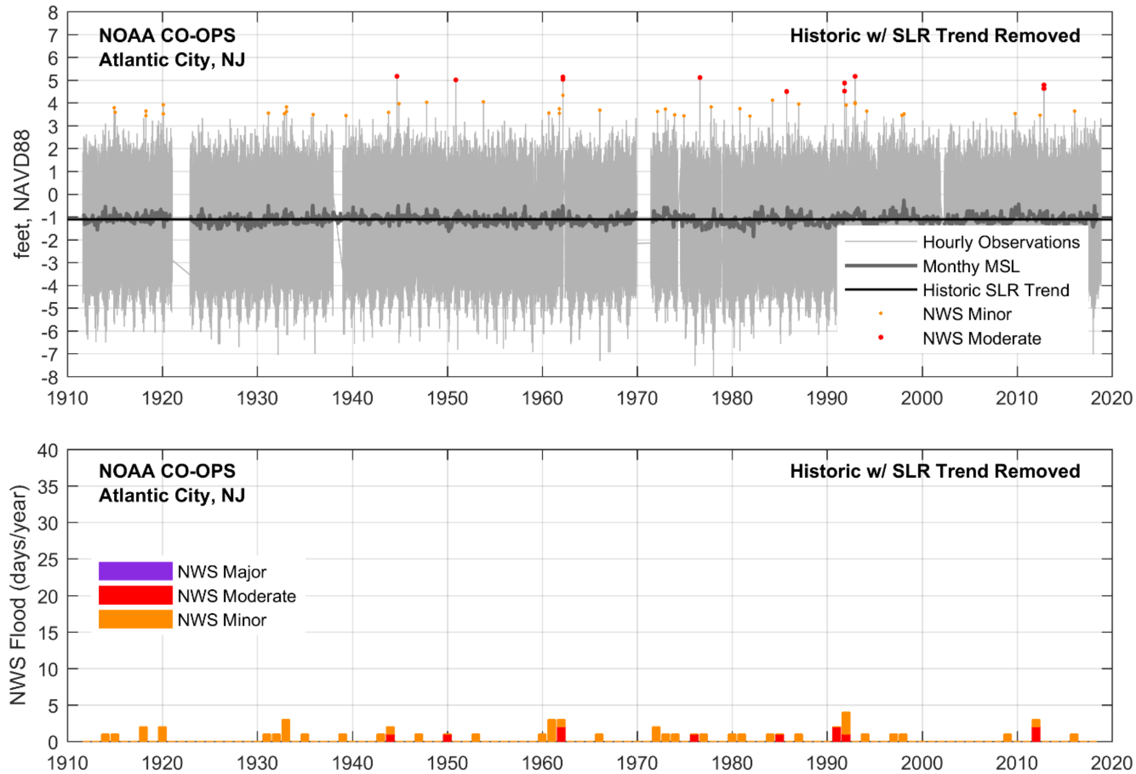


Figure 14: Impact of SLC on Historic High-Frequency Flooding

Future High-Frequency Flooding at Atlantic City, NJ

The previous section showed the dramatic impact RSLC has had on frequency of flooding over the last 100 years. This section shows how the rate of high-frequency flooding will be affected by future RSLC. To complete this analysis the last 25 years of the NOAA tidal record (1992-2017, skipping 2002 which had data gaps) was assumed to repeat over and over again until 2130. However, the three USACE SLC projections were added to the observed water levels. The top panel of **Figure 27** shows the hourly water level observations and future projections with the USACE-Low SLC scenario applied and a dot for any day in which the NWS flood stages were exceeded. The middle and bottom panel of **Figure 27** shows a histogram of the total number of days in a given year that the NWS flood stages were exceeded. The bottom panel shows the same information as the middle panel, but zooms in on NWS flood days (per year) between 0 and 40. The results in **Figure 27** show that Atlantic City is experiencing an acceleration in NWS minor flood days that will only get worse in the future. It also indicates that the increase already underway in NWS minor flooding will begin to occur in the future for the NWS moderate and major flooding. A significant increase in NWS moderate and major flooding appears to occur after 2030 and 2080 respectively.

The same analysis was repeated for the USACE-Intermediate and USACE-High RSLC scenarios in **Figure 28** and **Figure 29**. Annual NWS flood days from the analyses are tabulated in **Table 22** It is difficult to say or know what the tipping point (days per year) for NWS minor, moderate, and major flooding before the impacts to roads and infrastructure are unacceptable. However, the analysis shows that major investments in bulkheads and storm water systems (i.e. pump stations) are likely to be required in the future for the portions of the study area to be inhabitable.

Table 7: High-Frequency Flood Occurrences (Per Year)

Year	NWS Minor Flood			NWS Moderate Flood			NWS Major Flood		
	Low	Int	High	Low	Int	High	Low	Int	High
1930	1.1			0.0			0.0		
1955	1.7			0.2			0.1		
1980	3.6			0.5			0.2		
2005	14.5			0.7			0.0		
2015	26.5			2.2			0.5		
2030	54.7	73.2	139.8	4.7	5.9	21.1	0.1	0.3	1.0
2055	98.0	164.5	325.8	9.5	25.5	191.6	0.5	2.1	37.7
2080	153.8	282.6	356.2	23.1	100.9	349.9	1.5	11.1	298.3
2105	218.6	342.0	356.3	50.1	243.2	356.3	4.4	69.6	356.3
2130	258.5	350.6	352.3	78.1	327.3	352.3	5.8	182.3	352.3

Note: 10-year running mean filter applied to determine annual flood occurrences

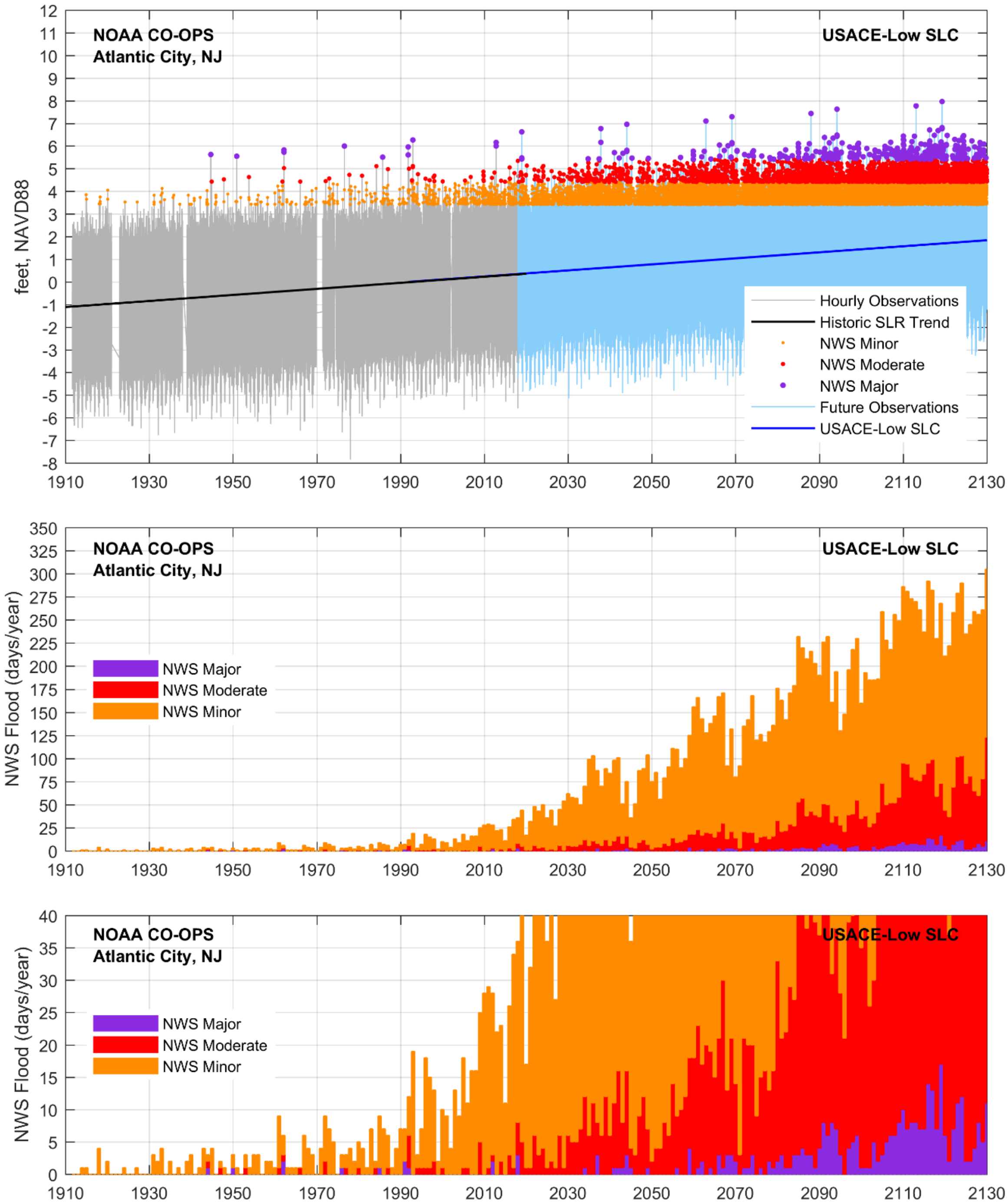


Figure 15: Future High-Frequency Flooding – USACE-Low SLC

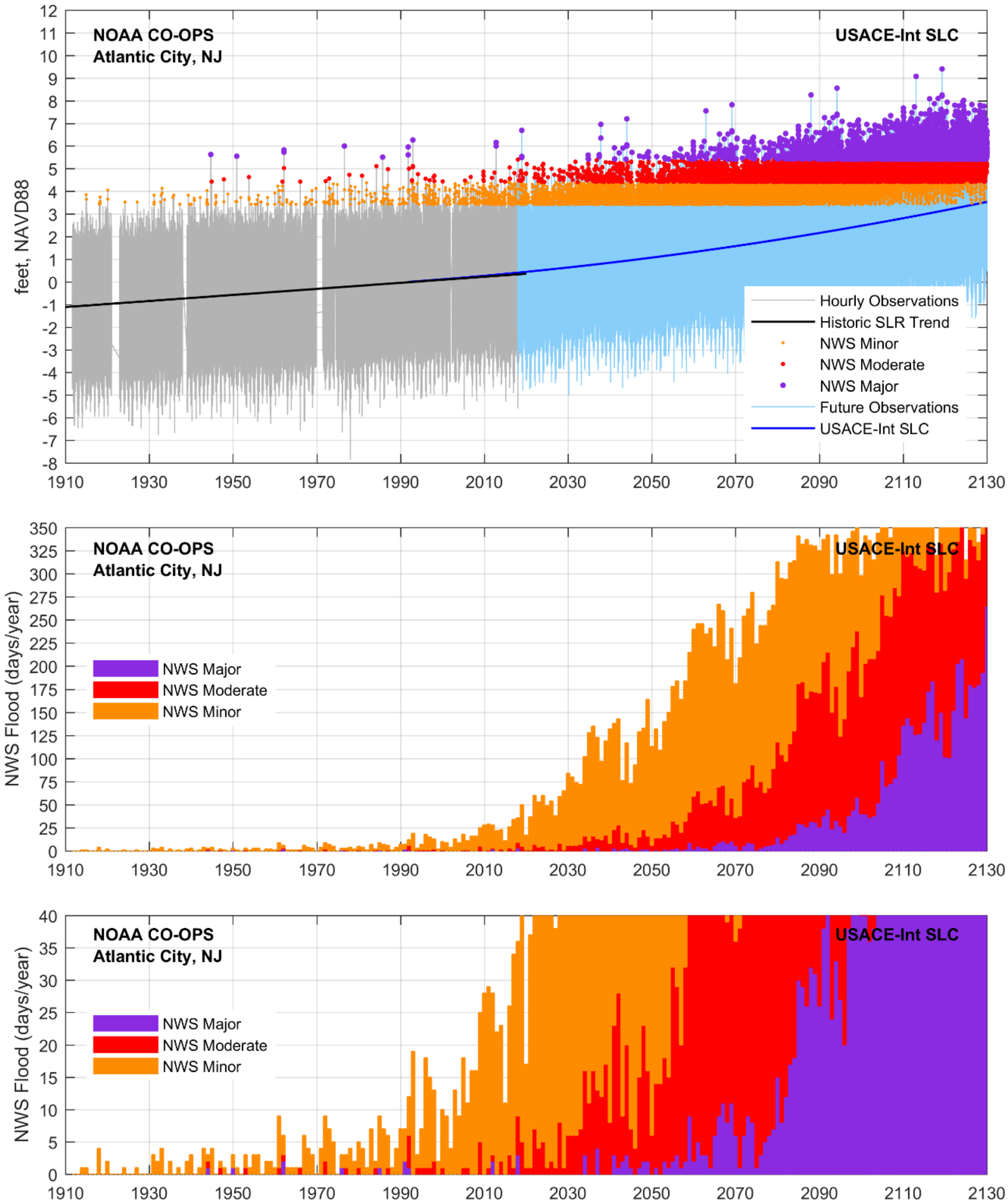


Figure 16: Future High-Frequency Flooding – USACE-Intermediate SLC

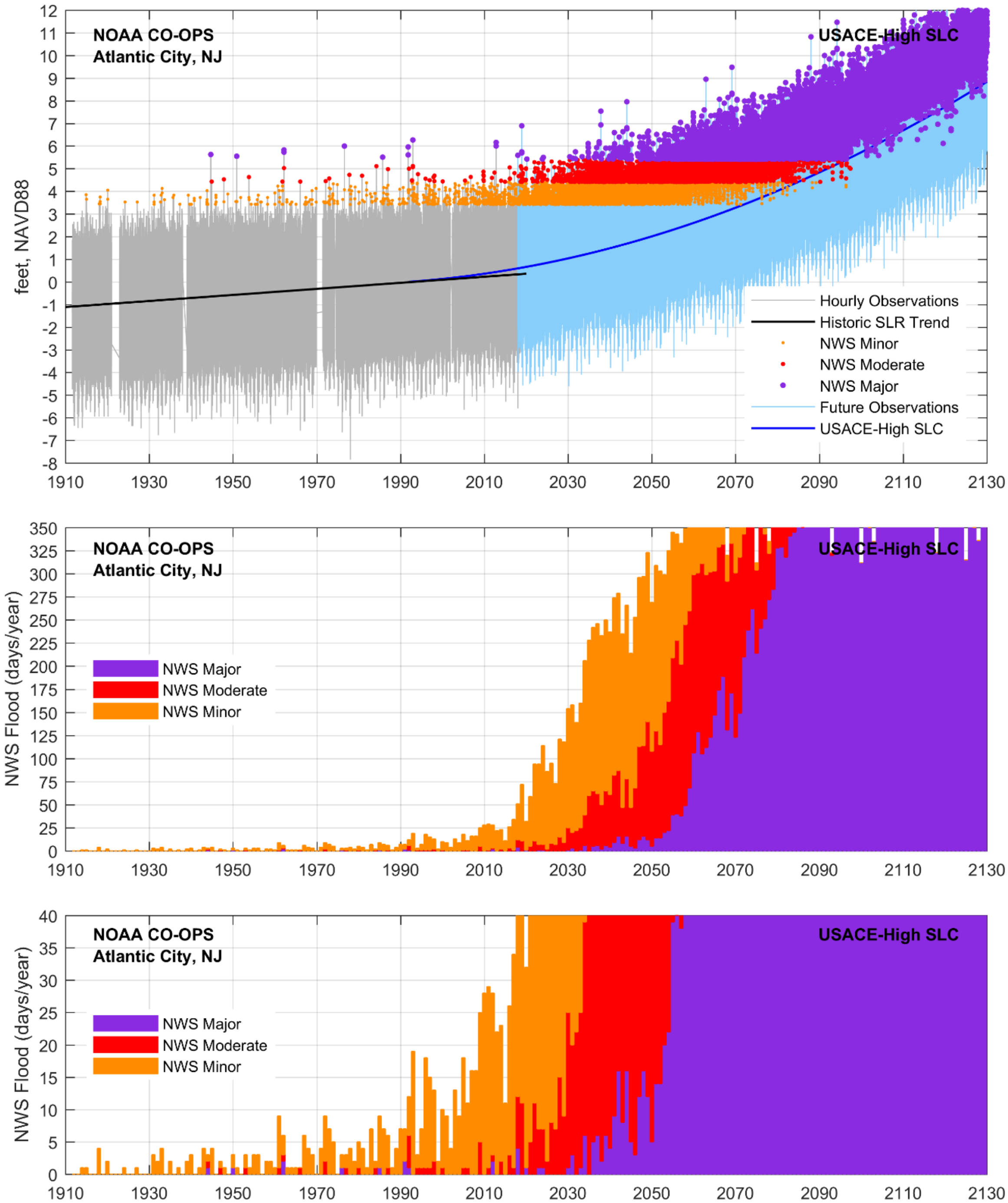


Figure 17: Future High-Frequency Flooding – USACE-High SLC

Storm Surge Modeling

NACCS

The North Atlantic Coast Comprehensive Study (NACCS) was authorized under the Disaster Relief Appropriations Act, PL 113-2, in response to Superstorm Sandy. The Act provided the USACE up to $20 Million to conduct a study with the goal to (1) reduce flood risk to vulnerable coastal populations, and (2) promote resilient coastal communities to ensure a sustainable and robust coastal landscape system, considering future sea level change and climate change scenarios.

As part of the NACCS, the US Army Engineer Research and Development Center (ERDC) completed a coastal storm wave and water level modeling effort for the U.S. North Atlantic Coast. This modeling study provides nearshore wind, wave, and water level estimates and the associated marginal and joint probabilities critical for effective coastal storm risk management. This modeling effort involved the application of a suite of high-fidelity numerical models within the Coastal Storm Modeling System (CSTORM-MS) to 1050 synthetic tropical storms and 100 historical extra-tropical storms. Documentation of the numerical modeling effort is provided in Cialone et al. 2015 and documentation of the statistical evaluation is proved in Nadal-Caraballo et al. 2015. Products of the study are available for viewing and download on the Coastal Hazards System (CHS) website: https://chs.erdc.dren.mil/.

Modifications for NJBB

The USACE Engineer Research and Development Center (ERDC), Coastal and Hydraulics Lab (CHL) conducted a numerical modeling study to evaluate the effectiveness of storm surge barriers in reducing water levels in the study area. As part of this numerical modeling study the existing condition water levels in the study area were updated to ensure that the existing and with-project water levels were consistent and derived from a common model, set of storms, and statistical evaluation. A detailed discussion of the ERDC numerical modeling report is provided in the Draft Technical Report XX attachment.

The ERDC numerical modeling study reused the CSTORM-MS developed for NACCS. While the original mesh boundary was maintained, Chesapeake Bay and coastal Long Island in the NACCS grid were subject to a “de-refining” procedure, which locally reduces a mesh resolution in areas that are distant from the area of interest. During the 1st phase of the CSTORM modeling (Iterations 1, 2, and 3) the model bathymetry was only updated to raise the barrier islands elevations from Manasquan to Lower Cape May Meadows to represent 2018 existing conditions with the recent construction of several USACE beach restoration projects that were not captured in the original NACCS model. During the 2nd phase of the CSTORM modeling the model bathymetry at Little Egg Inlet, Barnegat Inlet, and Barnegat Bay was updated with newer bathymetric data.

A total of 1050 synthetic tropical cyclones were designed and simulated in the NACCS. However, not all of these storms affect the NJBB region. Using Gaussian process metamodeling (GPM) and a design of experiments (DoE) approach, CHL selected subset of the NACCS synthetic tropical cyclones to maximize coverage of the storm parameter and probability spaces and produce storm surges across the NJBB region while reducing the hydrodynamic modeling requirements. A set of approximately 60 tropical cyclones was selected for modeling in order to complete the frequency distributions of response for both the with- and without-project conditions.

Modeling results are applied throughout the NJBB study to define wave and water level Annual Exceedance Probabilities (AEP). The water level AEP are based on the "Base + Linear superposition of 96 random tides" simulations and the mean confidence interval. The wave height AEP are based on the "Base Conditions + 1 random tide" simulations and the mean confidence interval. The water levels represent the peak water level observed during a storm due to the combination of storm surge, astronomical tide, wave-setup, currents, and winds. The water levels are computed stillwater levels, which do not include individual wave crests that could increase the instantaneous water surface.

Model Validation

ADCIRC Model Validation

The NACCS model validation procedure, documented in Cialone et al. (2015), included a harmonic analysis to ensure that the model is responding correctly to astronomical forcing 143 NOAA gage locations, 3 of which are in the study area: Sandy Hook, NJ; Atlantic City, NJ, and Cape May, NJ. In addition, a comparison of model to measurements for seven storm conditions to ensure that the model is responding to meteorological forcing. The seven storms are Hurricanes Sandy, Irene, Isabel, Josephine, and Gloria and extratropical storms ET070 (North American Blizzard of 1996) and ET073. Cialone et al. (2015) concluded that "consistency in the model's ability to predict water levels for the seven validation storm events provided a level of confidence in what can be expected from the model", and "from the harmonic analysis conducted for the long-term simulation, it was determined that the model accurately predicts response to tidal forcing".

Since model validation conducted for the NACCS study focused on the available NOAA gage locations, which are located in the Atlantic Ocean, the Philadelphia District asked ERDC-CHL to perform an additional analysis for USGS gages located in the back bays (**Figure 31**). The additional model validation analyses compared observed water levels to modeled (ADCRIC) water levels for all seven of the validation storm events and at any USGS gage that were active during the storm events. **Figure 30** compares the observed and modeled peak water levels. For water levels above 6 feet NAVD88 the ADCIRC model may be biased and over-predict water levels in the study area. It was concluded from the model validation that the model was acceptable for a planning study, but that the mean water level values, rather than a higher confidence interval, should be used for design.

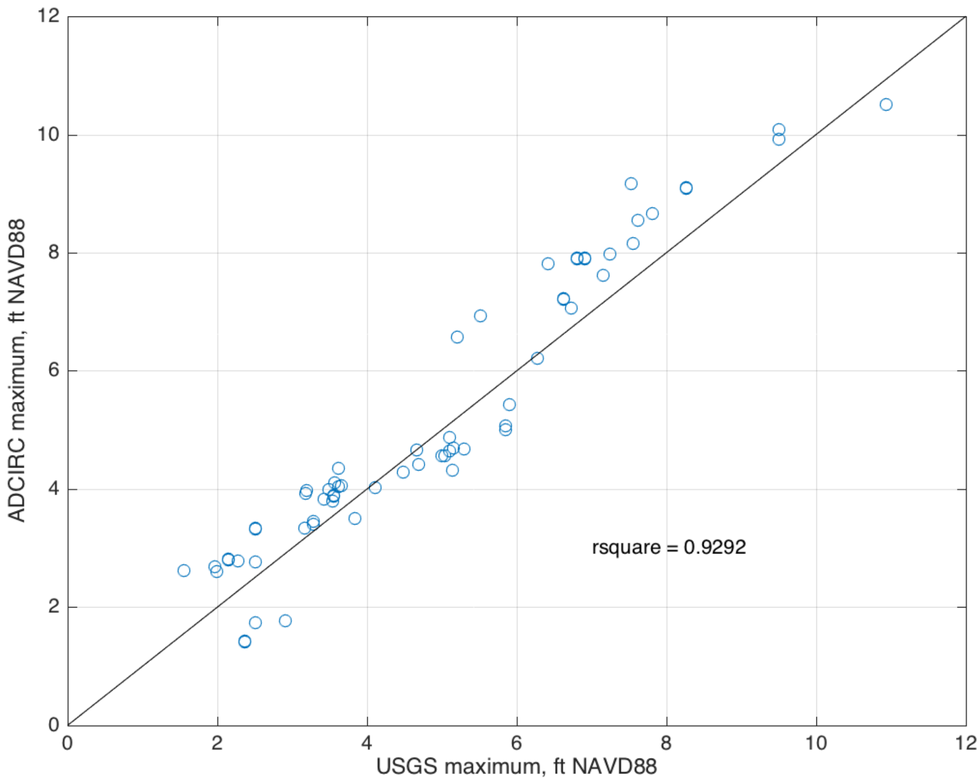


Figure 18: NACCS Model Validation at USGS Gages

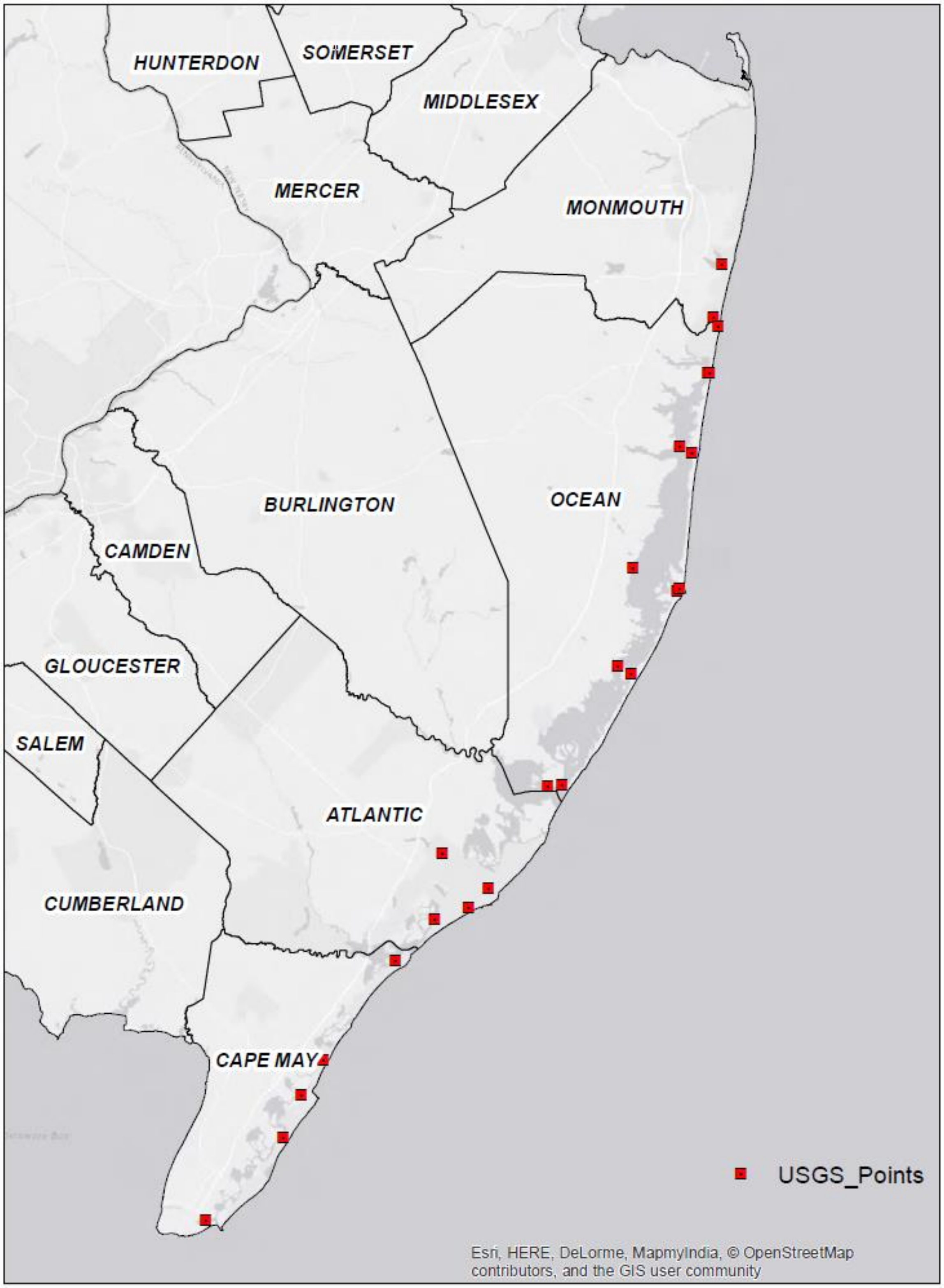


Figure 19: USGS Model Validation Gages

Baseline Water Levels

Save Points

A reduced set of 96, out of a possible 772, NACCS save points was selected to represent the AEP water levels in the economic model HEC-FDA. **Figure 33** shows the subset of 96 NACCS save points. The reduced set of points was selected by first removing points that appeared to be outliers relative to nearby points and then selecting a save point about every half-mile along the coastline, prioritizing open water save points and save points that seemed to best represent the nearby points. A smaller subset of save points would likely have been possible to characterize the FWOP conditions due to the homogeneity in water levels, but it is anticipated that there will be more variability in the water levels for the storm surge barrier alternatives. Sharp gradients in the water levels may occur between adjacent inlets when one inlet is closed and the other is open. Each save point is assigned to a specific reach and damage elements (i.e. structures) in HEC-FDA based on its location. The same set of save points and reaches is used in the FWOP and With Project HEC-FDA model simulations.

NJBB Hazard Curves

The NACCS and NJBB water level hazard curves, or AEP water levels, are the final product of high-fidelity numerical climate and hydrodynamic modeling, and rigorous joint probability methods. The methods include joint probability methods characterizing the storm climate, efficient sampling of probability space to develop efficient storm samples, high-fidelity numerical modeling of climate and hydrodynamics, and computation of response joint probabilities and epistemic uncertainties (Nadal-Caraballo et al. 2015).

The NJBB water level hazard curves are equivalent to the NACCS "Base + Linear superposition of 96 random tides" hazard curves and include the total contribution of storm surge, astronomical tide, wave-setup, currents, and winds. The water levels represent the still water level and do not include the height of individual waves, wave runup, and seasonal and interannual water level variations in mean sea level.

An example of the hazard curve information produced at the 96 save points applied in the study is shown in **Figure 32**, including the best estimate (mean) and 2%, 16%, 84%, and 98% confidence limits.

Representative Stations

In early stages of the NJBB Feasibility Study it was convenient to break the study area into separate hydraulic reaches (shown in **Figure 33**). These hydraulic reaches are no longer used in the study but are still useful in characterizing the AEP water levels. **Table 23** presents the AEP water levels for a representative station in each of the hydraulic reaches. The variability in water levels within hydraulic reaches is captured by **Figure 33**, which shows a map of the 1% AEP water levels, and **Figure 34** and **Figure 35** which show the AEP curves at all of the 96 HEC-FDA save points within each hydraulic reach, as well as the locations listed in **Table 23**. It is apparent from these tables and figures that the back bay AEP water levels are relatively homogenous, except for Barnegat Bay where there is more variability in the AEP water levels are 1 to 3 feet lower than the rest of the study area.

Table 8: NJBB Baseline Water Level AEP at Representative Stations

Location	Save Point	Hydraulic Reach	Return Period (years)							
			1	2	5	10	20	50	100	500
			Annual Exceedance Probability							
			100%	50%	20%	10%	5%	2%	1%	0.2%
Cape May	15566	Cape May Inlet	3.9	4.7	5.9	7.1	7.9	9.0	10.3	12.9
Wildwood	11282	Hereford Inlet	4.0	5.1	6.4	7.4	8.0	9.2	10.5	13.5
Avalon	13470	Townsend Inlet	3.9	5.0	6.2	7.0	7.7	9.3	10.7	14.4
Strathmere	7531	Corson Inlet	4.1	5.2	6.3	7.0	7.7	9.1	10.3	13.9
Ocean City	11309	Great Egg Inlet	4.2	5.2	6.2	6.9	7.7	9.1	10.2	13.0
Atlantic City	11356	Absecon Inlet	3.9	4.9	6.0	6.7	7.5	8.9	10.2	12.8
Mystic Island	11273	Little Egg Inlet	4.1	4.9	5.9	6.8	7.7	9.1	10.4	13.1
Lavallette	13694	Barnegat Inlet	2.9	3.5	4.5	5.3	6.2	7.6	8.6	10.9
Point Pleasant	13716	Manasquan Inlet	4.0	5.0	5.9	6.5	7.3	8.8	10.0	12.1
Belmar	13721	Shark River Inlet	4.3	5.3	6.4	7.2	8.1	9.3	10.3	12.3
Asbury Park	3742	Coastal Lakes	4.4	5.3	6.3	7.0	7.7	8.7	9.7	12.8

Note: All elevations are in feet NAVD88, relative to NTDE (1983-2001)

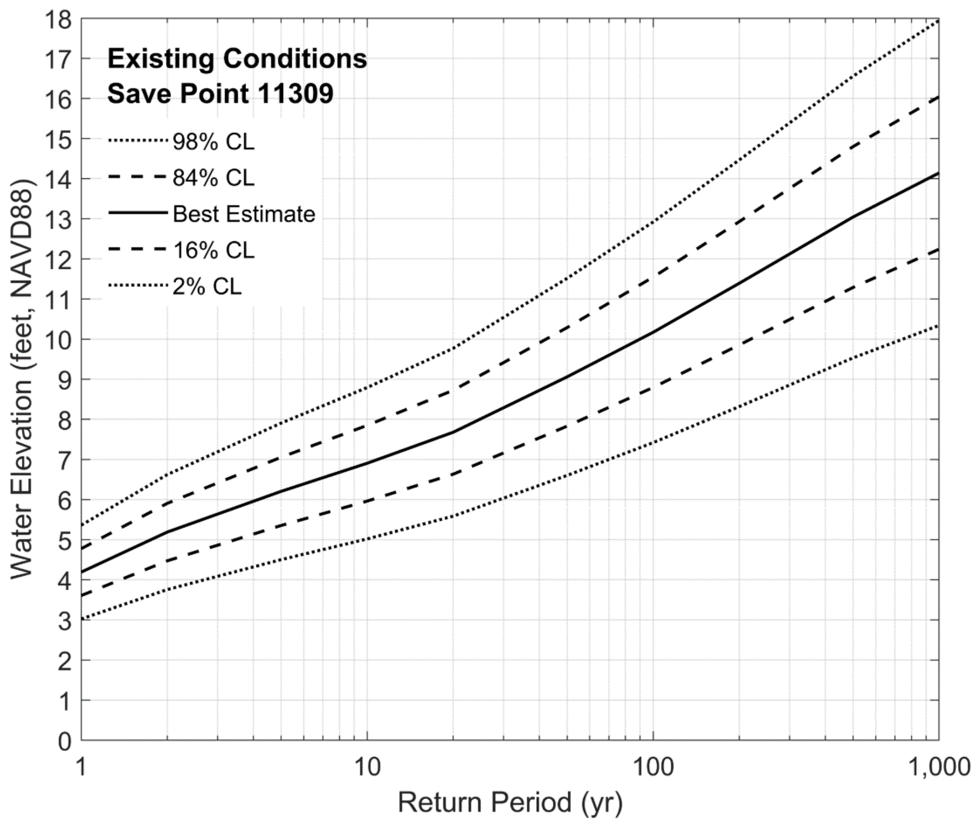


Figure 20: NJBB Baseline Hazard Curve at Ocean City, NJ

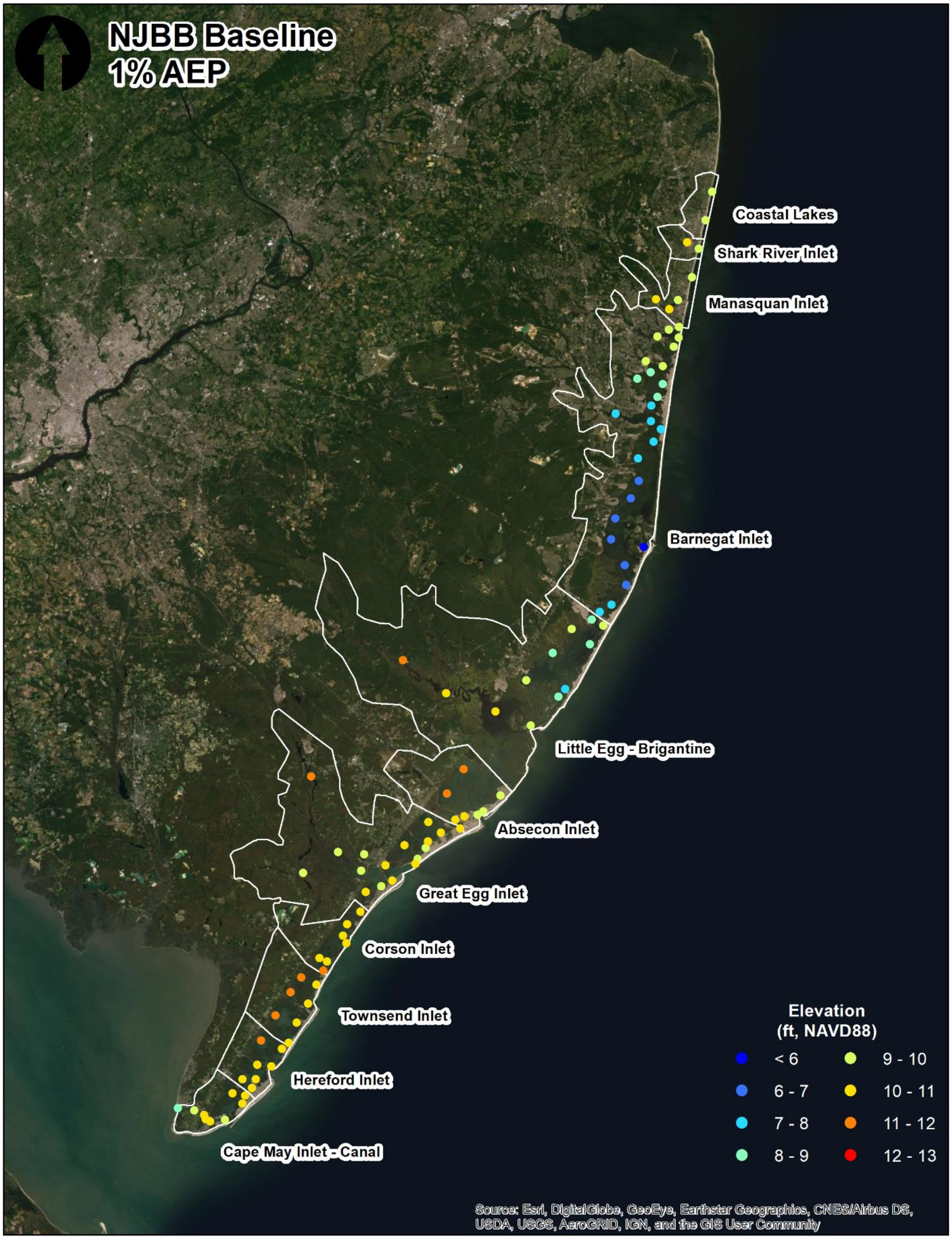


Figure 21: NJBB Baseline 1% AEP Water Level at HEC-FDA Stations

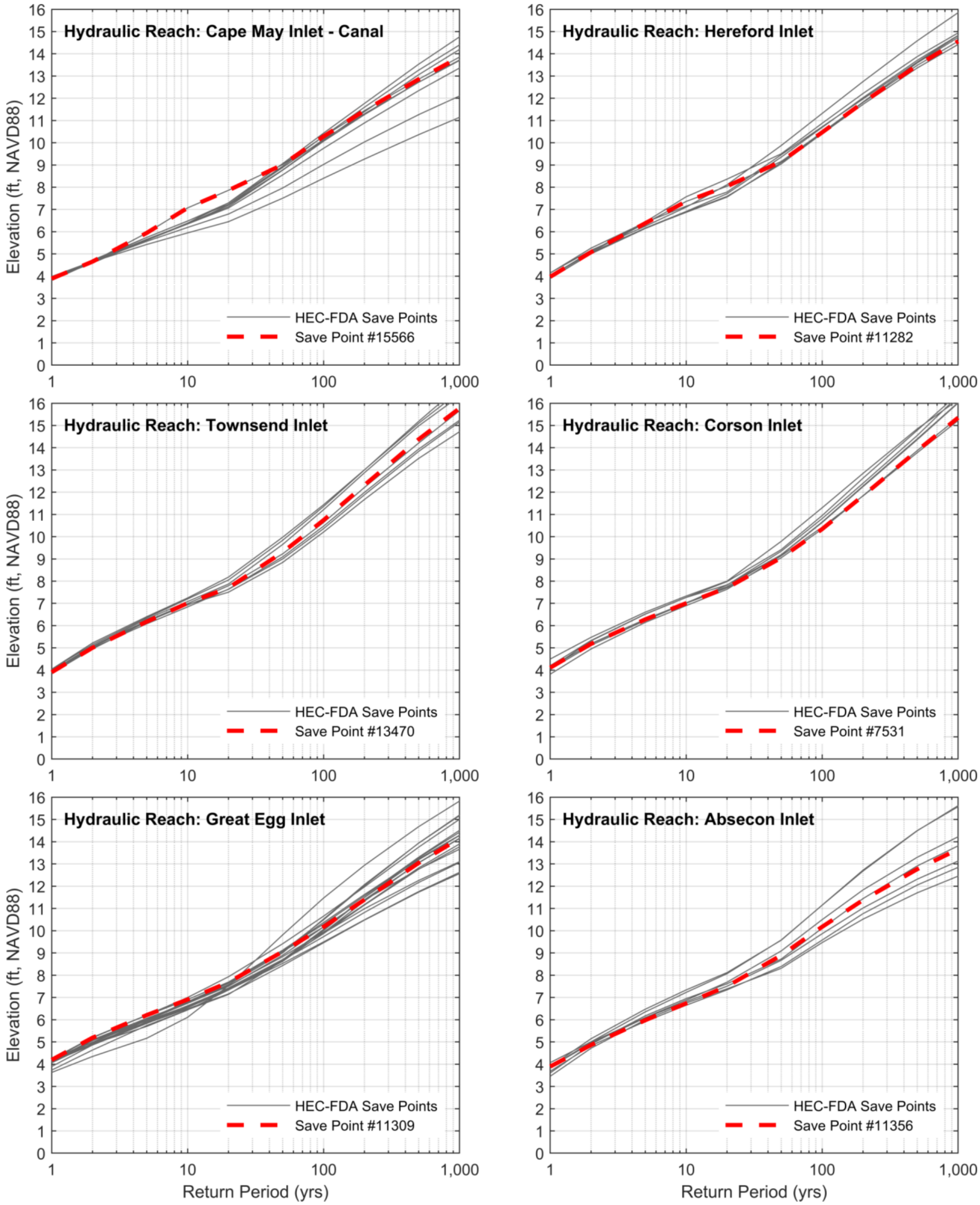


Figure 22: NACCS: Cape May Inlet to Absecon Inlet

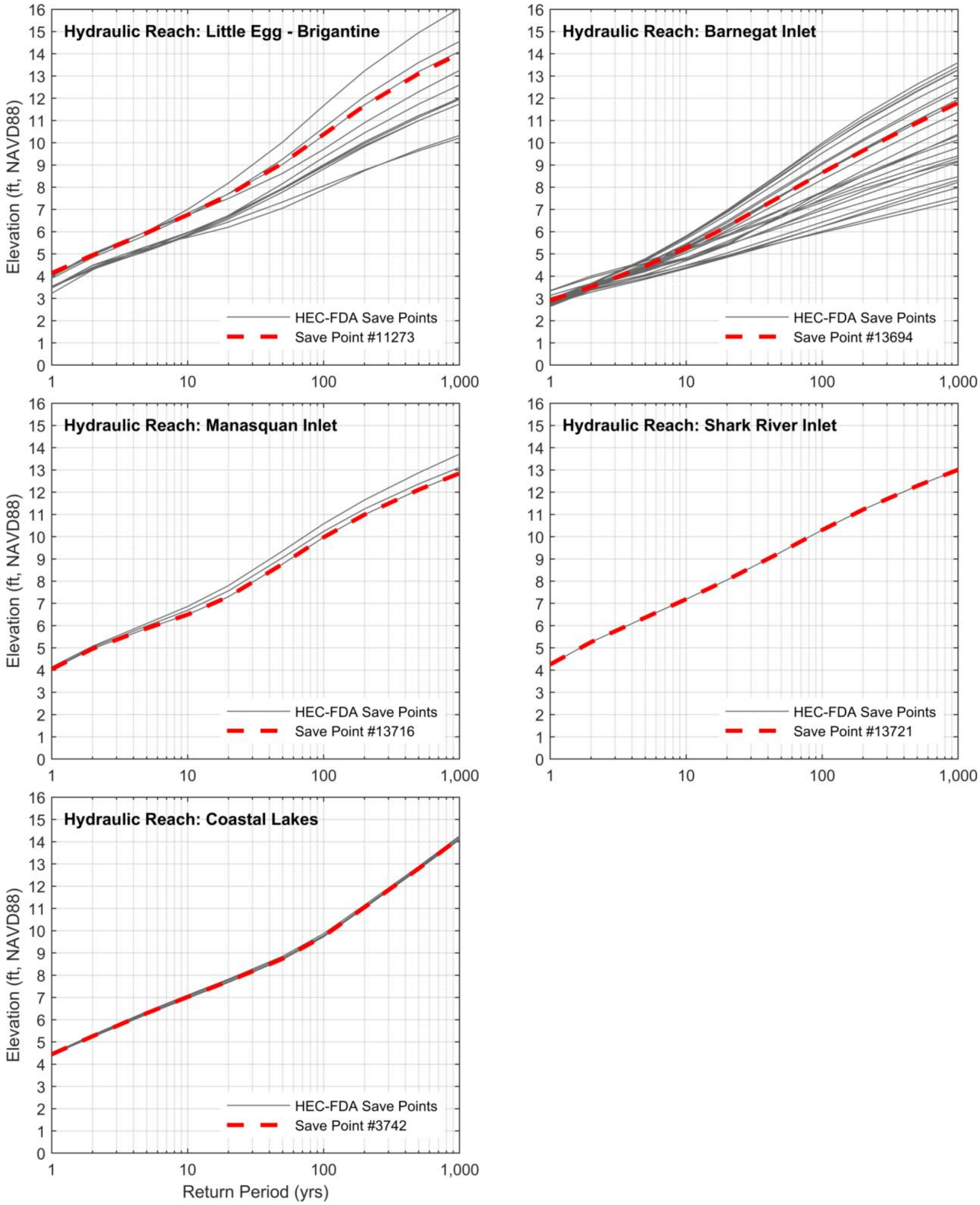


Figure 23: NACCS: Little Egg to Coastal Lakes

Hazard Curve Comparison to Historical Tide Gauge Analysis

A statistical analysis of the historical tide gauge record may suggest that what has occurred in the past will occur in the future, thus may underestimate the risk, especially at lower frequencies. Modeling, such as performed for the NACCS, provides an opportunity to evaluate impacts from stronger hypothetical storms that may not have occurred on record, but could occur. The historical record at the NOAA stations primarily reflects maximum water levels from nor'easters, tropical storms, or Category 1 type storms. The historical maximum water levels are approximately equal to a 10% to 1% AEP event.

NACCS Hazard Curves in NJBB study area are higher than hazard curves derived from historical tide gage analysis of NOAA and USGS data but are in agreement at the 99% AEP. Differences between NACCS and NOAA hazard curves at ocean stations are consistent with differences between NACCS and USGS hazard curves at back bay stations. The overall accuracy of the NACCS results at the NOAA ocean stations was reviewed and accepted during the publication of the NACCS.

Adjustments to the hazard curves ensure that comparisons between the different data sources are made to a common vertical datum (feet, NAVD88) and sea level rise trends are removed to be consistent with the current National Tidal Datum Epoch (NTDE) of 1983-2001 with a midpoint of 1992.5. The local NAVD88-MSL relationship is based on published values at NOAA and USGS gages and at locations between gages estimated using NOAA VDatum. In addition, all water level statistics represent the mean or 50% probability curve unless otherwise noted.

NOAA (Ocean) Stations

The NOAA Center for Operational Oceanographic Products and Services (CO-OPS) operates several tidal gages in the region with a reliable history of water level observations dating as far back as 1911 at Atlantic City, NJ. A list of the four stations in the region that have periods of record long enough for NOAA to reliably estimate sea level trends and perform extreme water level analyses is shown in **Table 24**.

Table 9: NOAA Long-term Tidal Stations

Station Name	Station Number	Records Since
Sandy Hook, NJ	8531680	1932
Atlantic City, NJ	8534720	1911
Cape May, NJ	8536110	1965
Lewes, DE	8557380	1919

Water level statistics are presented here from two different sources, both derived from the NOAA water level measurements, NOAA's published extreme water levels and Nadal-Caraballo and Melby (2014, TR-14-7).

The statistical analysis methodology employed by NOAA is based on the use of monthly maximum data fitted by the generalized extreme value (GEV) distribution. GEV is a family of continuous probability distributions developed within extreme value theory to combine the Gumbel, Fréchet and Weibull families. Since the NOAA GEV water level statistics are based on monthly maximums, smaller storm events that occur in the same month as a larger storm event are omitted from the analysis and could cause an underestimation of the 99% annual exceedance probability (AEP).

Nadal-Caraballo and Melby (2014, TR-14-7) calculated water level statistics at 23 NOAA tidal gages based on verified water level measurements using a peak over threshold (POT) and Generalized Pareto Distribution (GPD) and Monte Carlo Life-Cycle. The Monte Carlo Life-Cycle allows tidal variations (i.e. spring/neap and high/low tides) to be incorporated. The water level statistics from Nadal-Caraballo and Melby (2014) were shown to agree well with those computed by NOAA GEV at the 10% AEP to 1% AEP. At the 99% AEP the TR-14-7 water levels are greater than the NOAA GEV as expected since the NOAA GEV misses some of the storm events that occur within the same month.

An overview of the NACCS 1% AEP water levels at the four NOAA tidal stations in the study area is presented in **Table 25**.

Table 10: Hazard Curve Comparison at NOAA Stations

Station / Source	**Annual Exceedance Probability**						
	99%	**50.0%**	**20.0%**	**10.0%**	**5.0%**	**2.0%**	**1.0%**
	Return Period (years)						
	1	**2**	**5**	**10**	**20**	**50**	**100**
Water Level Elevations in Feet, NAVD88 (Current NTDE, 1983-2001)							
Sandy Hook, NJ							
NACCS	4.7	5.5	6.6	7.5	8.3	9.6	10.9
NOAA TR-14-7	4.8			6.3		7.8	8.5
NOAA GEV	4.0	5.0		6.4			9.2
Atlantic City, NJ							
NACCS	3.9	4.8	5.7	6.4	7.1	8.3	9.7
NOAA TR-14-7	4.2			5.2		6.1	6.5
NOAA GEV	3.6	4.4		5.3			6.8
Cape May, NJ							
NACCS	3.4	4.7	5.9	6.4	6.7	7.0	7.4
NOAA TR-14-7	4.6			5.3		5.7	5.9
NOAA GEV	3.8	4.7		5.4			6.0
Lewes, DE							
NACCS	4.1	4.8	5.6	6.1	6.6	7.5	8.6
NOAA TR-14-7	4.1			5.2		5.9	6.3
NOAA GEV	3.6	4.4		5.4			6.0

USGS (Bay) Stations

The USGS operates several tidal gages in the NJBB study area with water level observations dating as far back as 1993. Four USGS tidal gages are selected for analysis shown in **Table 26**. The period of record for USGS tidal gages is not as long as the NOAA stations, but still provides meaningful information about the high-frequency water level statistics. Water level statistics at these 4 USGS tidal gages is calculated using peak over threshold analysis with the data fitted by the GEV distribution (Gumbel, Fréchet and Weibull). The results of the water level statistics are shown in **Table 27**.

Table 11: USGS Tidal Stations

Station Name	Station Number	Records Since
Shark River at Belmar, NJ	1407770	Apr 2000
Barnegat Bay at Waretown, NJ	1409110	July 1993
Inside Thorofare at Atlantic City, NJ	1410560	May 2000
Cape May Harbor at Cape May, NJ	1411390	May 2000

Table 12: Hazard Curve Comparison at USGS Stations

Location and Source	Annual Exceedance Probability						
	99%	**50.0%**	**20.0%**	**10.0%**	**5.0%**	**2.0%**	**1.0%**
	Return Period (years)						
	1	**2**	**5**	**10**	**20**	**50**	**100**
Shark River at Belmar, NJ							
NACCS #13721	4.3	5.3	6.4	7.2	8.1	9.3	10.3
USGS	4.9	5.6	6.8	8.1	9.8	12.8	16.0
Barnegat Bay at Waretown, NJ							
NACCS #11424	2.7	3.4	4.0	4.5	5.0	5.8	6.4
USGS	2.0	2.2	2.7	3.3	3.9	5.2	6.4
Inside Thorofare at Atlantic City, NJ							
NACCS #11356	3.9	4.9	6.0	6.7	7.5	8.9	10.2
USGS	4.1	4.4	4.8	5.3	5.9	7.1	8.2
Cape May Harbor at Cape May, NJ							
NACCS #7546	3.9	4.6	5.7	6.5	7.2	8.9	10.1
USGS	4.1	4.3	4.6	5.0	5.4	6.3	7.1

Hazard Curve Comparison to FEMA

NACCS and the FEMA Region II study (FEMA 2014) are based on the Joint Probability Method

(JPM). The JPM was adopted by federal agencies for the critical post-Katrina determinations of hurricane surge frequencies. In standard JPM implementations, it is necessary to consider a very large number of combinations of storm parameters, and each such combination (or synthetic storm) requires the simulation of wind, waves, and surge. The JPM is a very robust methodology, and it is also very complex. The complexity arises from the fact that it has multiple components and probabilistic models that could be executed in different ways, or different developers could choose to use different models. FEMA water level statistics represent the 84-percent confidence limit, not the mean.

The results of the NJBB Baseline (CSTORM) and FEMA water level frequencies for the 1% AEP are shown side by side in **Figure 36** to give a visual understanding of the differences. **Figure 37** shows a scatter plot comparison of the NACCS save points and FEMA save points. With the exception of a few save points, the NACCS and FEMA 1% AEP water levels are within 2 feet of each other. The NACCS values tend to be a higher, especially south of Little Egg Inlet. The purpose of comparing FEMA and NACCS is to provide some context of how the NACCS data compares to the FEMA BFE which may be more familiar to stakeholders and the public.

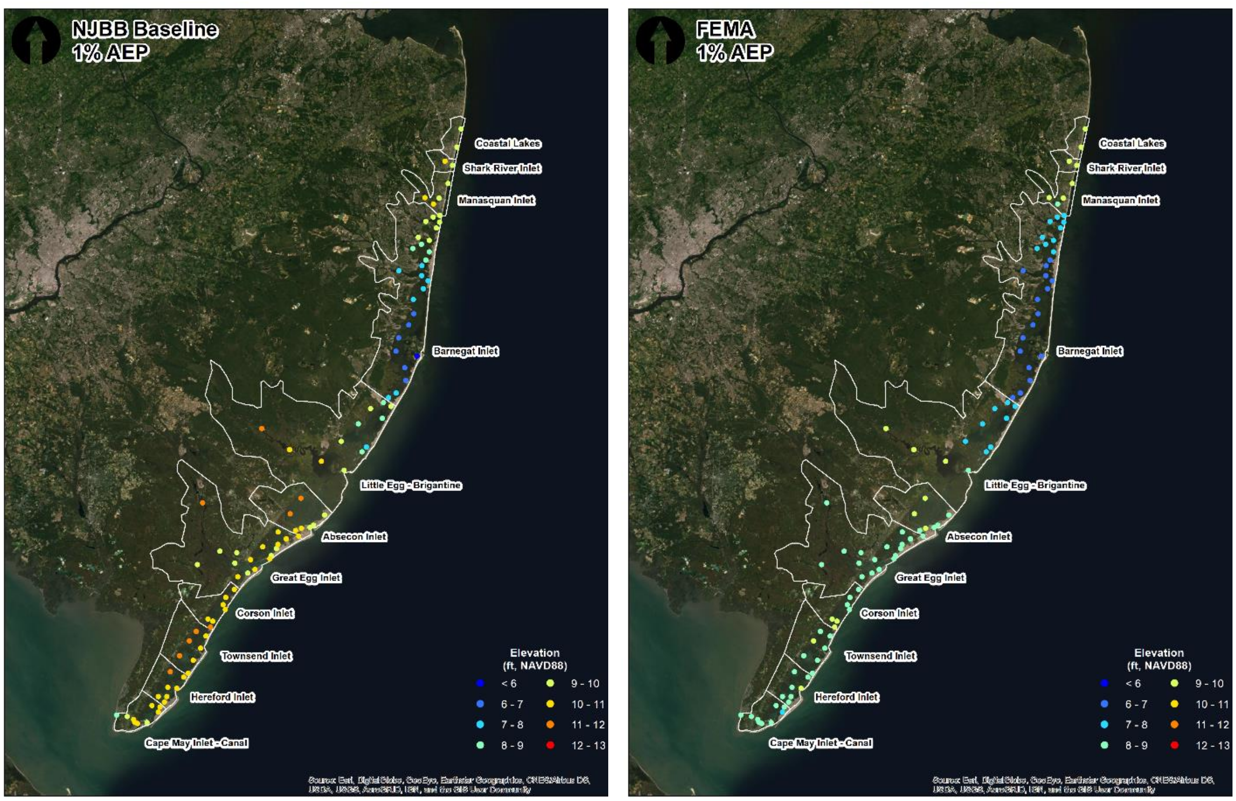


Figure 24: NJBB Baseline and FEMA 1% AEP Water Level Map

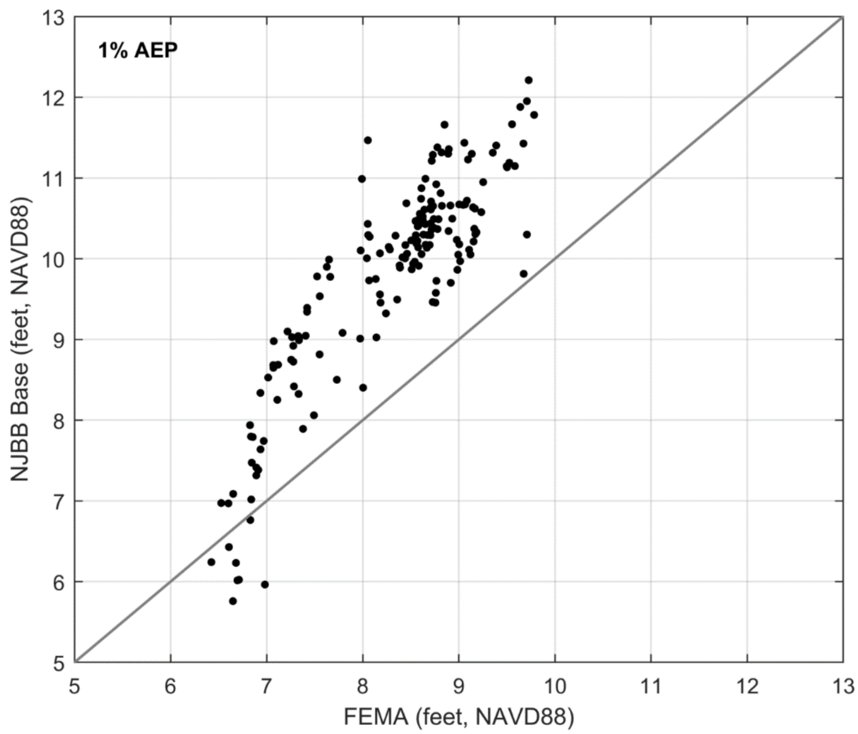


Figure 25: NJBB Baseline and FEMA 1% AEP Water Level Scatter Figure

Storm Surge Barriers

Approach to Modeling Storm Surge with Storm Surge Barriers

Due to the complex network of inlets and bays that control the flow of water between the ocean and back bays, NAP requested assistance from ERDC-CHL in evaluating the effectiveness of inlet closures in reducing water levels in the NJBB study area. More specifically, NAP wanted help determining how much inlet closures reduce back-bay flooding? How effective inlet closures are at reducing water levels if other inlets are open and if multiple inlet closures could work as system? To answer these questions ERDC-CHL leveraged the existing NACCS CSTORM-MS. Draft Technical Report by Slusarczyk et al. (2020) provides a detailed description of the storm surge modeling effort and discussion of the modeling results. The storm surge modeling work was completed in two phases: Phase 1 between the AMM and TSP-IPR milestones, and Phase 2 between the TSP-IPR and TSP milestones.

Phase 1

In Phase 1 an iterative modeling approach was devised that would allow a large number of inlet closures and potential inlet closure combinations to be considered before converging on a smaller final set of inlet closure alternatives. The iterative modeling approach begins with model simulations of one inlet closure at a time to improve understanding of the hydraulic influence of each inlet. The second iteration evaluated a large number of possible inlet closure combinations, before moving on to the final iteration of a smaller final set of alternatives. Model simulations for the final set of alternatives is used to develop frequency distributions of peak water levels that may be applied in economic analyses of flood damages. The iterative modeling approach is made feasible by utilizing a very small subset of 10 extreme cyclones for Iterations 1 and 2. A more robust set of 60 tropical cyclones was selected for Iteration 3 in order to develop the frequency distributions.

- Iteration 1: Model the hydraulic influence of each barrier island inlet by modeling one inlet at a time.
- Iteration 2: Model the effectiveness of large set of possible inlet closure combinations.
- Iteration 3: Model the effectives of final set of inlet closure alternatives and develop frequency distributions of peak water levels.

Workshops with the CHL, the NJBB Project Delivery Team (PDT), and non-Federal sponsor (NJDEP) were held on January 31, 2018 and April 13, to review the model results from Iteration 1 and Iteration 2 and selected the closure configurations to be brought forward in the study. Many of the closure configurations for Iteration 2 are designed around leaving the most environmentally sensitive inlets open: Little Egg/Brigantine, Corson, and Hereford. Closures across the interior bays "bay closures" are added to several configurations to reduce water levels where environmental sensitive inlets open. The study area was also broken up into 3 regions (north, central, and south) based on the relative hydraulic independence of the configurations identified for these regions. Since many of the configurations are designed around leaving Little Egg and Corson inlets open, these two inlets were natural boundaries for the three regions.

Phase 2

In Phase 2 the CSTORM model bathymetry was updated in Barnegat Bay and at several of the inlets with more recent survey data. After updating the model bathymetry, the same set of 60 tropical cyclones from Phase 1 – Iteration 3 was simulated in CSTORM and the hazard curves were updated. CSTORM simulations were also performed for the three primary storm surge barrier alternatives in the Focused Array of Alternatives:

- North: Closures at Manasquan and Barnegat Inlets.
- Central 1: Closures at Absecon and Great Egg Inlets.
- Central 2: Closure at Great Egg Inlet and bay closures at Absecon Blvd and Southern Ocean City.

Summary of Storm Surge Barrier Model Results (Phase I)

A detailed discussion of the storm surge barrier modeling results is provided in Draft Technical Report by Slusarczyk et al. (2020). Only a summary of modeling results is provided here.

Iteration 1 focused on the ability of individual surge barriers to alter maximum water levels compared to a base condition with no closures in place. It was found that individual closures can reduce back bay flooding, mainly in the bays closest to the closure location, but adjacent inlets may allow flow into the bay then water level reductions can be less significant. Individual storm surge barriers at Great Egg Inlet, Barnegat Inlet, and Shark River Inlet were most effective. Individual storm surge barriers from Cape May to Corson Inlet were not as effective and would perform better as part of system of storm surge barriers. A storm surge barrier at Manasquan Inlet was effective for storms where the predominate wind direction was south, however, storms with north winds could push storm surge north into Barnegat Bay and Manasquan limiting the barriers effectiveness.

Iteration 2 focused on evaluating systems (multiple) of storm surge barriers including cross-bay storm surge barriers ("bay closures"). Many of the storm barrier alternatives were designed around leaving the most environmentally sensitive inlets open: Little Egg/Brigantine, Corson, and Hereford. The numerical modeling results show that many of the Iteration 2 alternatives are effective at reducing back bay water levels. However, some of the alternatives such as All Closures Less 2 showed considerable sensitivity to the storm and wind directions and it was unclear what the impact would be on the hazard curve. Iteration 2 also showed that many of the bay closures have the potential to increase surge on the unprotected side of the closure as wind-blown water piles up against the closure. Increases in surge were not limited to the immediate vicinity of the closure and significant impacts may be felt 5 to 10 miles away.

Figure 38, **Figure 39**, and **Figure 40** show the modeling results for three storm surge barrier alternatives, All Closures and All Closures Less 2, and C3, respectively. The All Closures Less 2 alternative has storm surge barriers at all the inlets except Little Egg and Corson inlets. C3 has storm surge barriers at Great Egg Inlet, Absecon Inlet, and a bay closure north of Brigantine.

Iteration 3 focused on the 8 alternatives identified during the April 13, 2018 workshop with the CHL, the NJBB Project Delivery Team (PDT), and non-Federal sponsor (NJDEP). These 8 alternatives were selected based on their ability to generate the greatest NED benefits (flood damages reduce minus project costs) and be environmentally acceptable. Several alternatives were included that are not likely to be environmentally acceptable to ensure that alternatives were not eliminated too early before a more thorough plan formulation evaluation is applied.

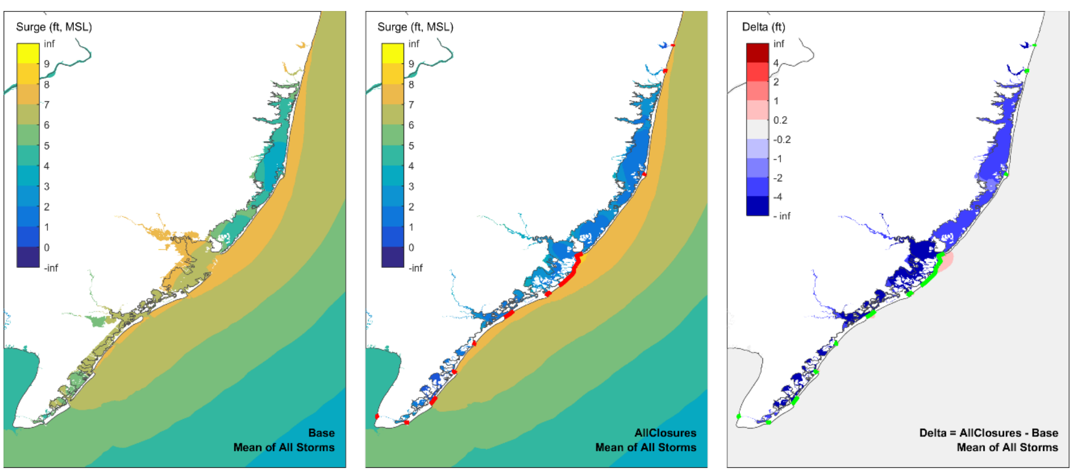


Figure 26: CSTORM Model Results – All Closures

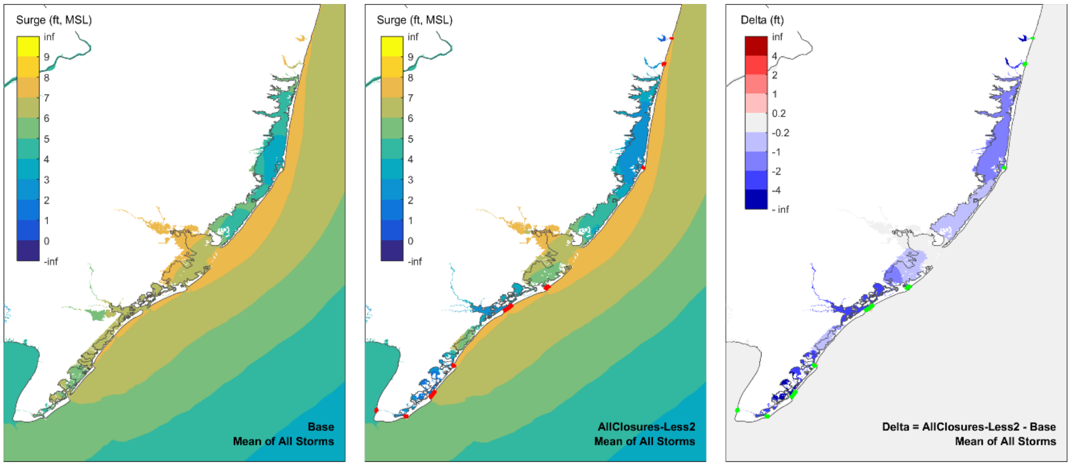


Figure 27: CSTORM Model Results – All Closures Less 2

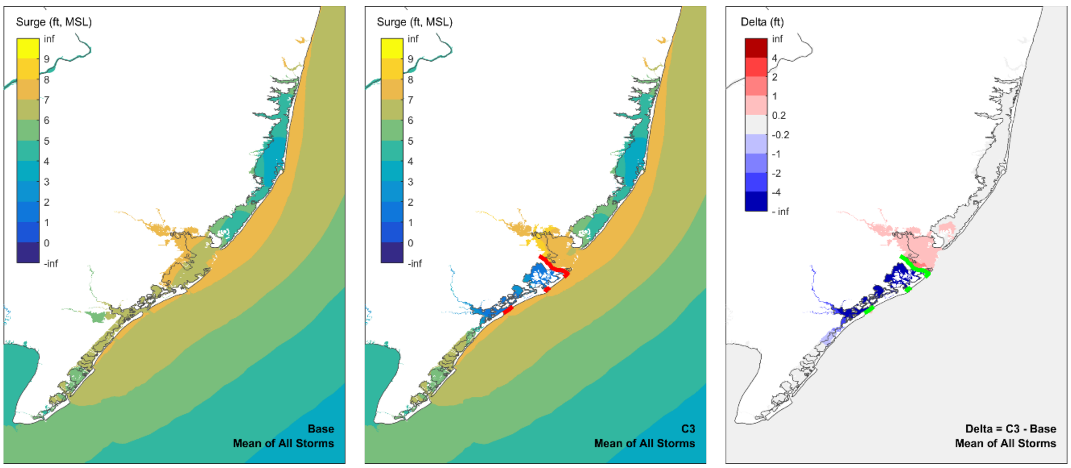


Figure 28: CSTORM Model Results – C3

Hazard curves were generated for the Iteration 3 alternatives based on simulations for storm suite of 60 tropical cyclones. An example of the hazard curves at six locations (**Figure 41**) for Baseline, All Closures, and All Closures Less 2 alternatives is provided in **Figure 42**. The Baseline and All Closures hazard curves may be thought of as bracketing the possible performance of other storm surge barrier alternatives. Less effective storms surge barrier alternatives have hazard curves close to the Baseline curve and more effective storm surge barrier alternatives have hazard curves close to the All Closures curve. **Figure 42** shows that the performance of All Closures Less 2 varies within the study area. At some locations like Ocean City the performance of All Closures Less 2 is similar to All Closed, and other areas like Lavallette, closer to open inlets, the performance is more similar to the Baseline conditions.

A 1- or 2-foot reduction in storm surge may not seem significant, but a 2-foot reduction in storm surge at Lavallette may be the difference in a 6 foot (NAVD88) storm surge event being a 100-year event versus a 20-year event. It is unclear until the economic model is completed if a 1- or 2-foot reduction in water level in places like Barnegat Bay will translate into a significant reduction in damages. The purpose of Iteration 3 was to generate the water level hazard curves that may be applied in HEC-FDA to calculate benefits.

Figure 29: Example Hazard Curve Locations in Central and North Regions

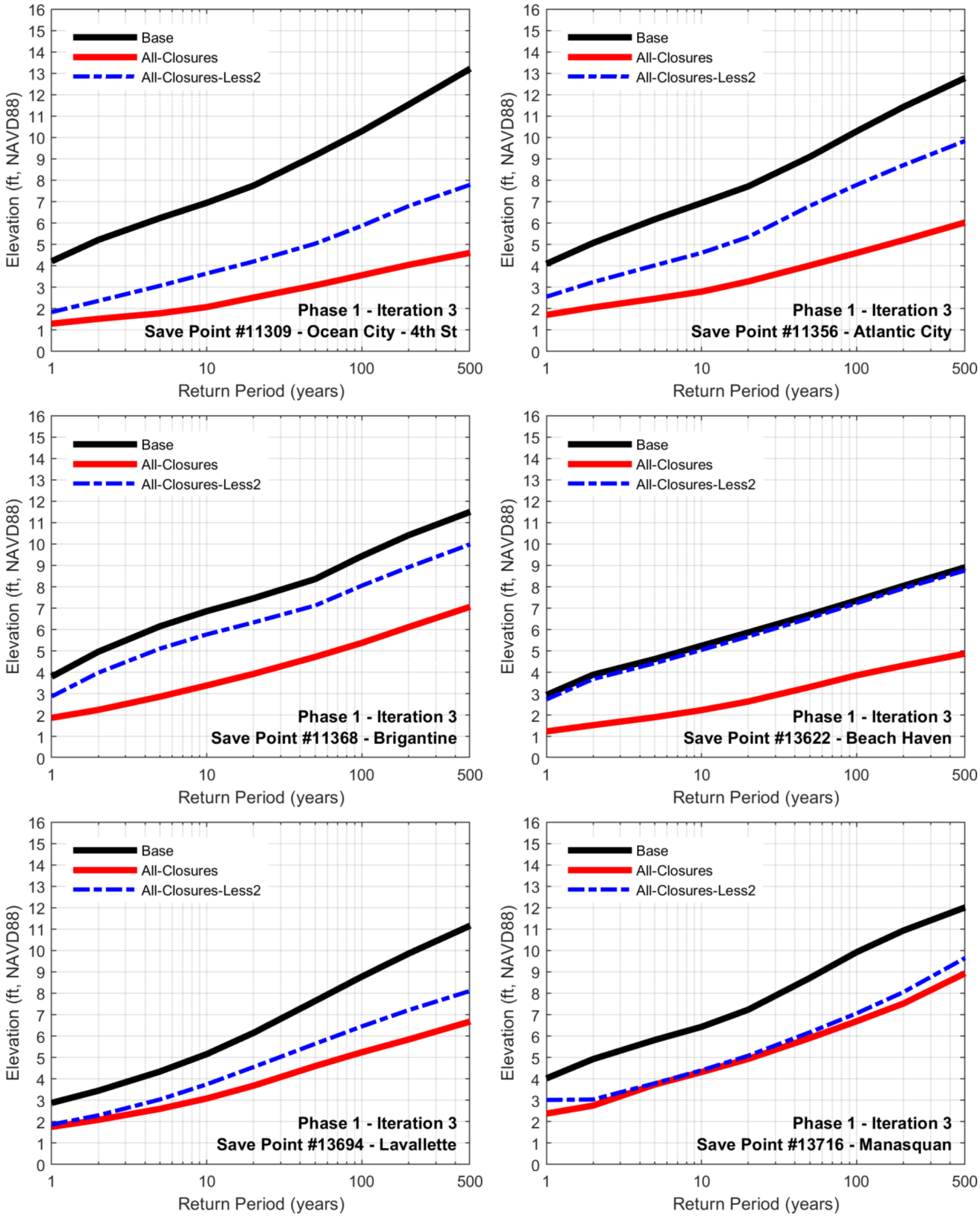


Figure 30: Storm Surge Barrier Hazard Curves (Phase 1 – Iteration 3)

Summary of Storm Surge Barrier Model Results (Phase 2)

Phase 2 of the storm surge modeling work focused on updating the model bathymetry and revising the baseline and storm surge barrier hazard curves. In general, the model results and hazard curves in Phase 2 are consistent with the findings from Phase 1 with small differences. The most significant changes in the model bathymetry occurred along the marshes in Barnegat Bay and Great Bay Boulevard (between Little Egg Inlet and Barnegat Bay) so the largest differences in the model results occurred within lower Barnegat Bay, between Surf City and Beach Haven. Differences between Phase 1 and Phase 2 model results north of Surf City were insignificant.

Model results for North (closures at Manasquan and Barnegat Inlets) and Central 1 (closures at Absecon and Great Egg Inlets) alternatives s closely mirror the results for All Closed Less 2 since it is the same set of closures in the North and Central Regions. All Closed Less 2 included additional closures in the South Region, but these closures have little impact on the results in the Central and North Regions. An example of the hazard curves at six locations (**Figure 41**) for Baseline, North, Central 1, and Central 2 alternatives is provided in **Figure 43**.

North alternative, peak SWLs in upper Barnegat Bay and Manasquan River are 1.5 to 3 ft lower than the base conditions at the 100-year return period. In the Peak SWLs in lower Barnegat Bay are only 0 to 1 ft lower than the base conditions at the 100-year return period confirming earlier observations the peak SWLs in lower Barnegat Bay are dominated by flow from Little Egg Inlet.

Central 1 alternative, peak SWLs in the area dominated by Great Egg Inlet (most of Ocean City and Atlantic City) are 3 to 5 ft lower than the base conditions at the 100-year return period. Peak SWLs in the vicinity of Absecon Inlet are approximately 2 ft lower than the base conditions at the 100-year return period.

Central 2 alternative, with closures at Great Egg Inlet and bay closures at Absecon Blvd. and Southern Ocean City, produces similar results to Central 1 in the area dominated by Great Egg Inlet (most of Ocean City and Atlantic City) indicating that the benefits of the bay closures are more localized. Peak SWLs in the vicinity of Absecon Inlet are nearly the same as the base conditions.

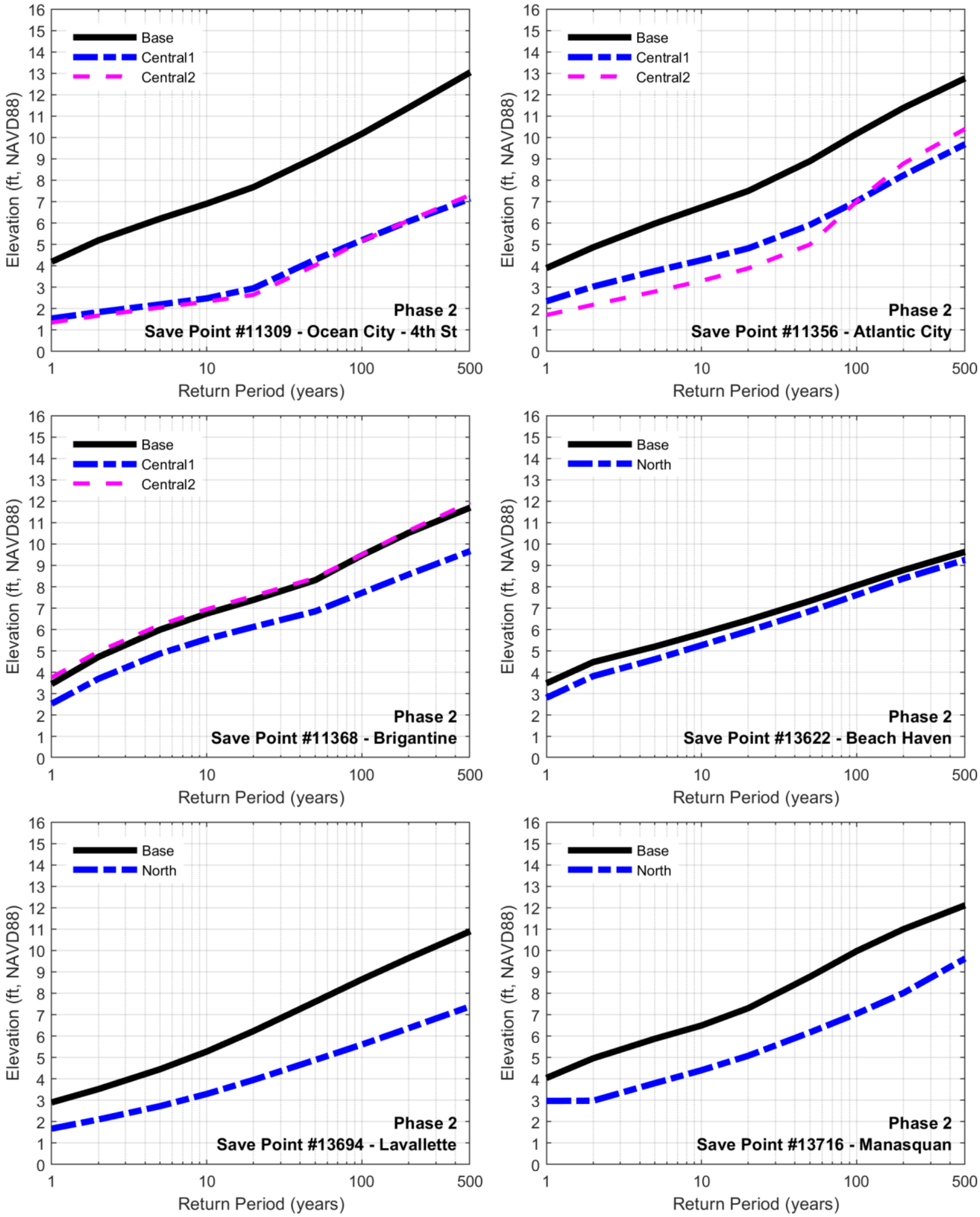


Figure 31: Storm Surge Barrier Hazard Curves (Phase 2)

Impact of Storm Surge Barriers on Ocean-Facing Beaches

Modeling results show that the storm surge barriers may cause an increase in water levels in the immediate vicinity of the storm surge barrier. Beyond a distance of 1 mile of the storm surge barrier no discernable (less than 1 inch) increase in water levels was observed. **Figure 44** shows a comparison of the peak surge in the baseline conditions, All Closures Less 2 alternative, and the difference between All Closures Less 2 and the baseline conditions. An increase in ocean water levels of 6 to 12 inches is observed at the storm surge barrier and increase of 2 to 6 inches within ½ mile of the barrier, and 1 to 2 inches within 1 mile of the barrier. It is noted that the values reported here and shown in **Figure 44** are based on mean of all 10 tropical storms in NJBB Iteration 1 and 2 storm suites, and increase, proportionally, with stronger storms.

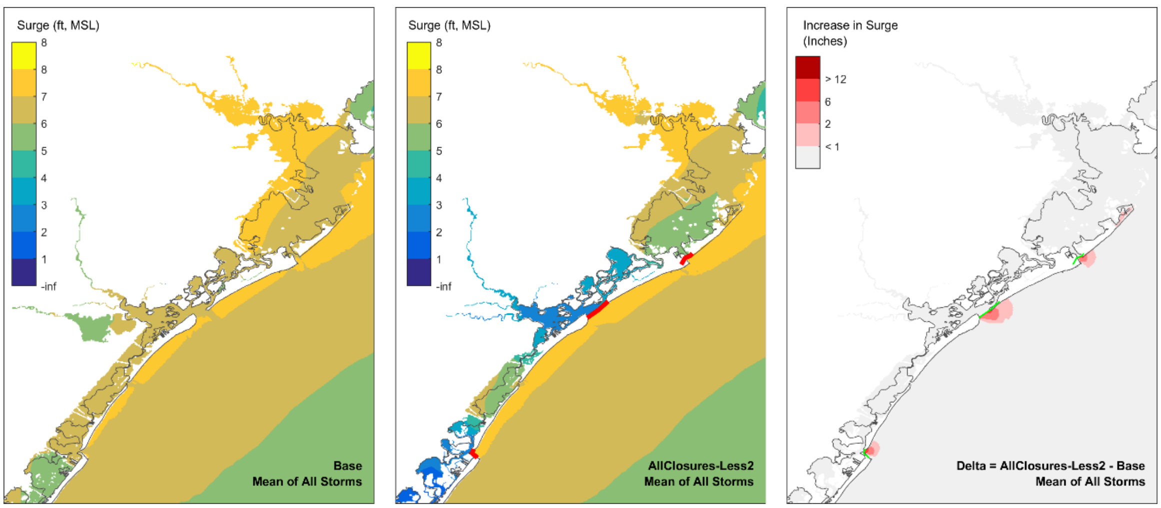


Figure 32: Impact of Storm Surge Barrier on Ocean-Facing Beaches

Wave Overtopping (In Progress)

Overview

Wave overtopping is of principal concern for structures constructed for flood risk management. The design crest elevation of flood risk management structure is often determined by the design still water level and required freeboard, height above still water level, to prevent wave overtopping from damaging the structure during the design storm event.

EurOtop (2016) describes wave overtopping as:

> *Overtopping discharge occurs because of waves running up the face of a seawall or dike. If wave run-up levels are high enough water will reach and pass over the crest of the structure. This defines the 'green water' overtopping case where a continuous sheet of water passes over the crest. In cases where the structure is vertical, the wave may impact against the wall and send a vertical plume of water of the crest. A second form of overtopping occurs when waves break on the seaward face of the structure and produce significant volumes of splash 'whitewater'. These droplets may then be carried over the wall either under their own momentum or as a consequence of an onshore wind.*

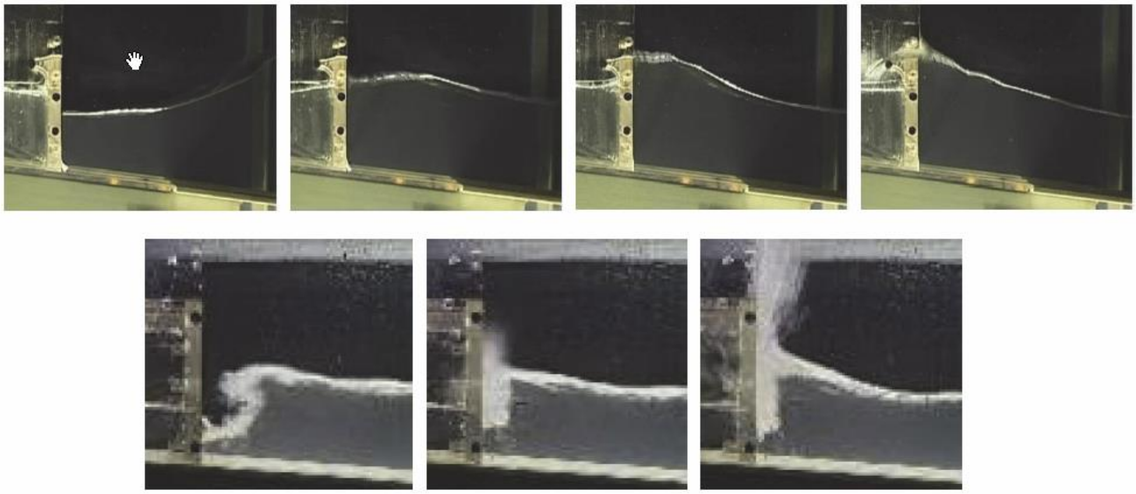


Figure 33: Wave Overtopping at Vertical Wall (EurOtop, 2016)

The top panel and bottom panel of **Figure 45** show an example 'green water' and 'white water' overtopping at a vertical structure respectively.

The wave overtopping rate, q, reported in this study is the mean overtopping discharge (liters/s/m). In actuality wave overtopping occurs in sporadic short pulses and is not constant over time. It is coastal engineering practice to use mean wave overtopping rates in engineering applications since available design formulas are based on the mean overtopping rate due to its ability to be easily measured in laboratory studies.

Wave Conditions

Wave conditions in the NJBB study area are fetch-limited waves generated by local wind conditions. In fetch-limited conditions, wave heights are limited by the distance of open water in which the waves are able to grow. Wave conditions throughout the bay are also affected by the shallow water depths, marshes, and orientation relative to the wind directions. A sampling of the 100-year wave conditions at 11 representative locations throughout the study area is provided in **Table 28**.

In the design or assessment of coastal structures with respect to wave overtopping, the two primary hydraulic parameters (water level and wave height and wave period) may be derived from a joint probability analysis (EurOtop, 2016). If both water level and wave height are determined for a certain return period, then the wave overtopping discharge for the combination of these extreme conditions will be larger than the actual wave overtopping occurring with the return period (EurOtop, 2016). This is caused by the fact that the combination of these two extreme values will have a lower probability of occurrence if the two are not fully correlated (EurOtop, 2016).

The "H_{m0} – Joint" and "Tp – Joint" columns in **Table 28** represent the joint probability or most likely wave height and wave period associated with the 1% AEP water level event. The joint probability of the wave height and water levels was determined from time series of NACCS model results at each of the representative stations. The maximum wave height within 1 hour of the maximum water level was identified from the time series. Scatter plots showing the relationship between the peak water level and wave height is presented in **Figure 47** to **Figure 49**. These figures also show the relationship between the wave height and wave period associated with the peak water levels. A 2nd order polynomial curve was fit to the scatter data to obtain the joint probability relationship.

Table 13: Representative Wave Conditions, Joint Probability for 1% AEP

Station	ID	SWEL (ft, NAVD88)	H_{m0} (ft)	H_{m0}-Joint (ft)	Tp-Joint (s)
Cape May	15566	10.3	3.8	3.4	3.1
Wildwood	11282	10.5	3.6	3.1	3.8
Ocean City	11309	10.2	3.6	2.4	3.3
Somers	11230	10.7	3.7	3.6	3.5
Atlantic City	13554	10.0	3.6	2.3	3.3
Beach Haven	11399	7.9	4.2	2.8	3.5
Tuckerton	11444	8.5	4.7	4.1	4.3
Lavallette	11511	8.3	3.3	2.3	3.4
Island Heights	13684	7.9	3.5	2.0	3.4
Mantoloking	13706	9.5	3.4	1.9	2.9
Manasquan	13711	10.2	3.4	2.1	2.8

Note: Still Water Elevation (SWEL), Joint probabilities values shown based on curve fit.

A joint probability analysis was not conducted at the storm surge barrier locations at inlets, as it is assumed at this stage of the study that the 1% ACE water level and wave event occur simultaneously. A representative design wave height of 12 feet and wave period of 12 seconds is used in the analysis based on available NACCS wave data near the location of the storm surge barriers at inlets. A single representative wave condition is applied to all the inlet closures at this phase of the study, however detailed modeling will be performed in subsequent phases of the modeling to determine the design wave conditions at each storm surge barrier location.

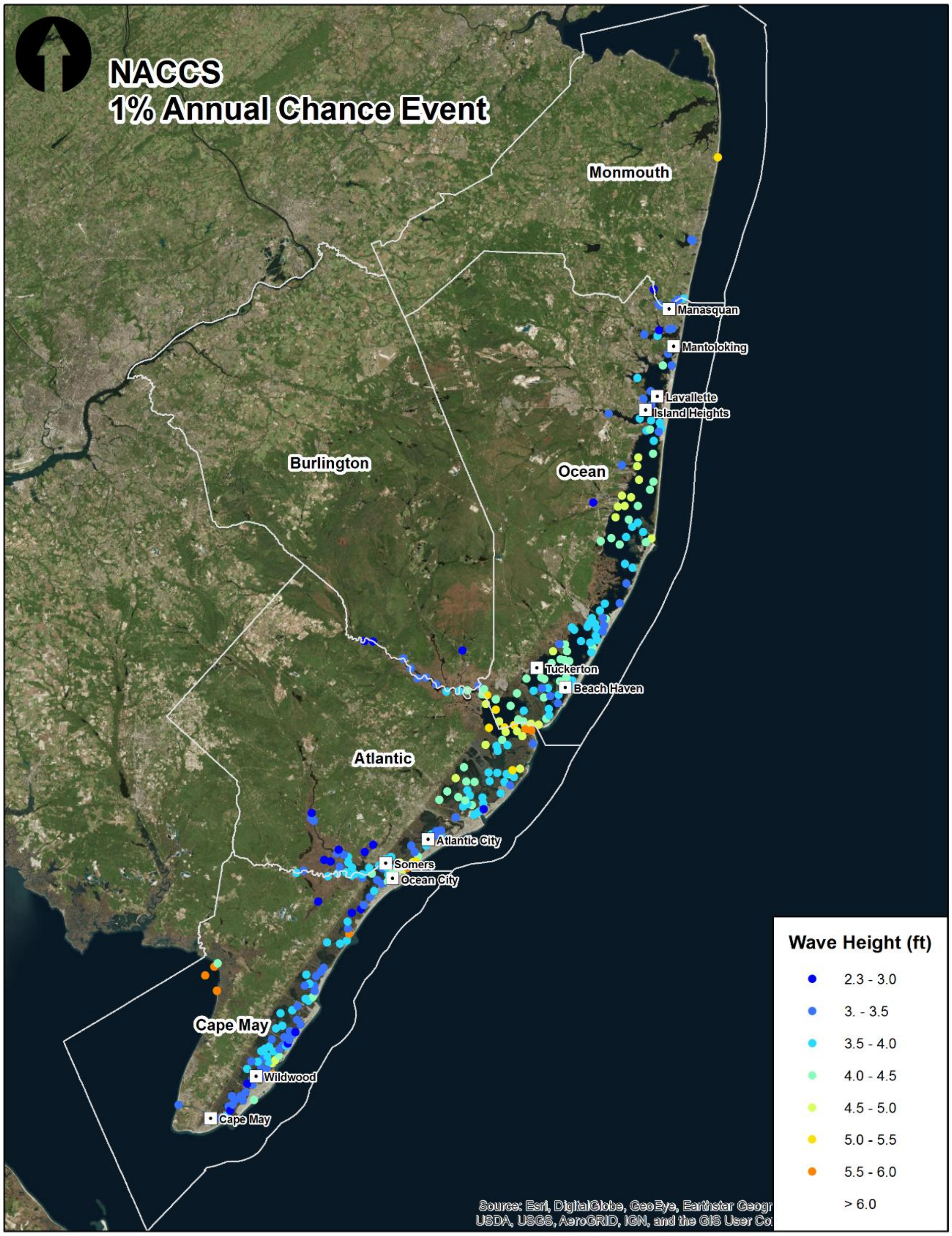


Figure 34: NACCS 1% AEP Peak Wave Height and Representative Stations

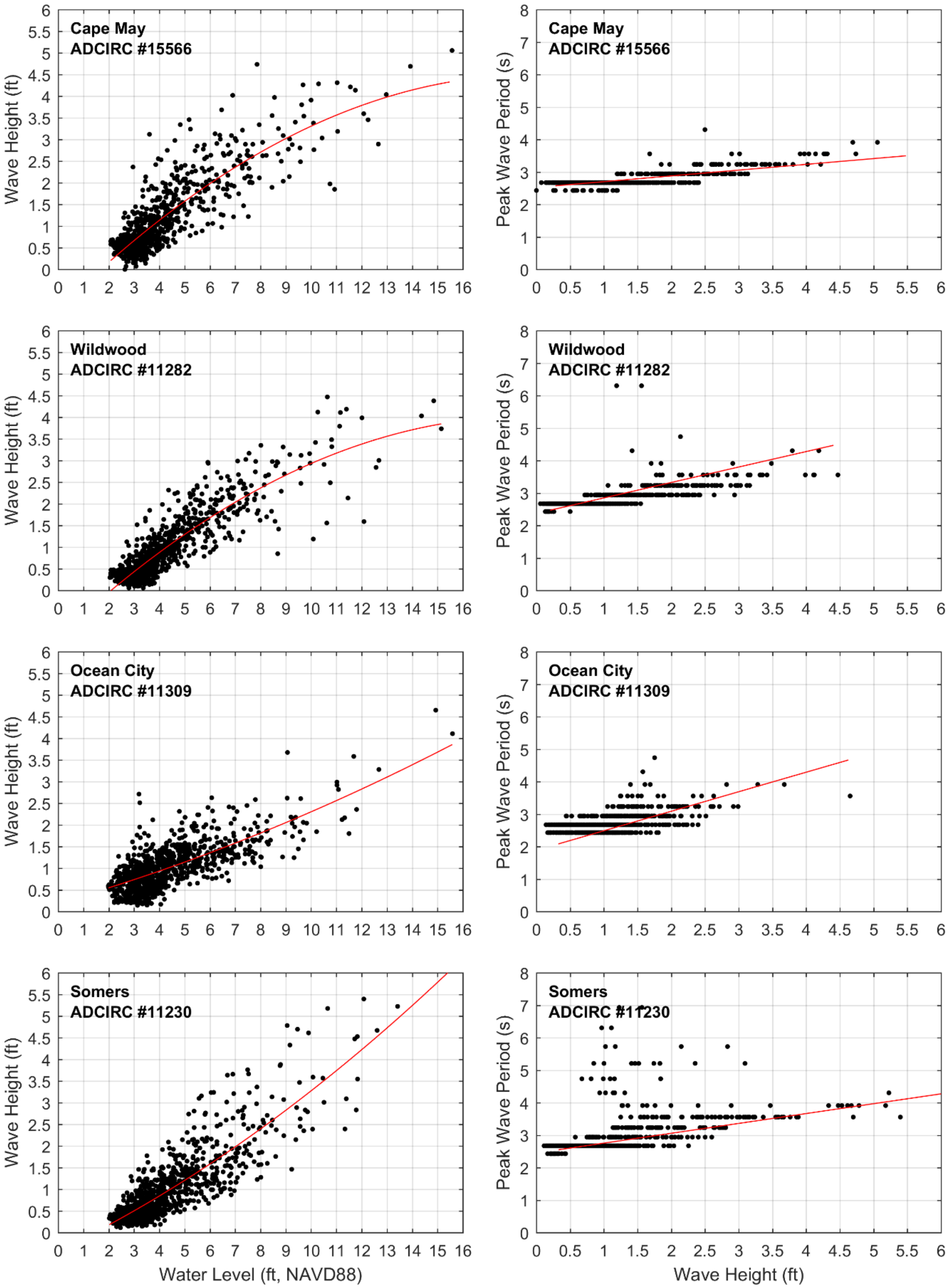


Figure 35: Joint Wave Probability, Cape May to Somers

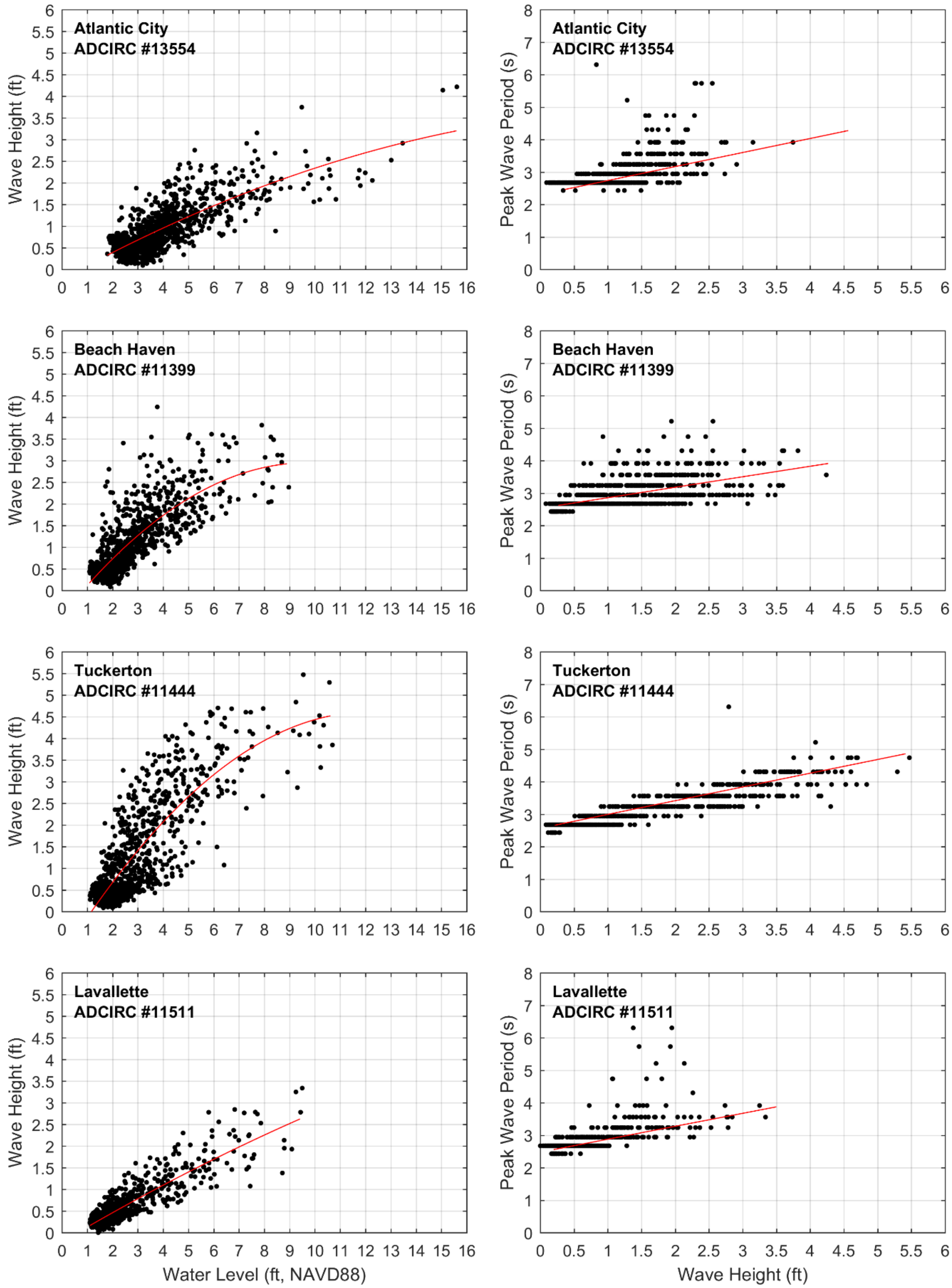


Figure 36: Joint Wave Probability, Atlantic City to Lavallette

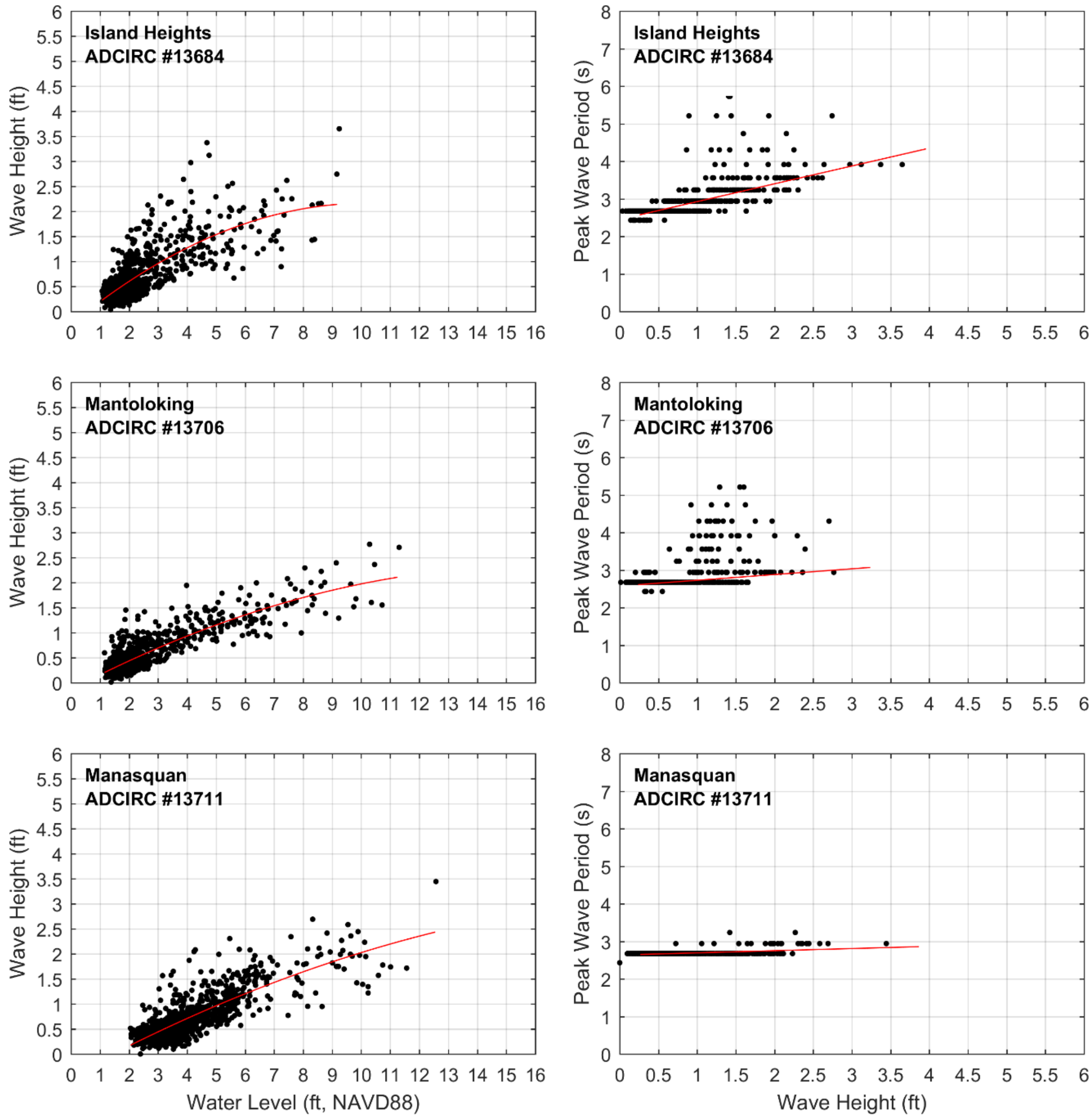
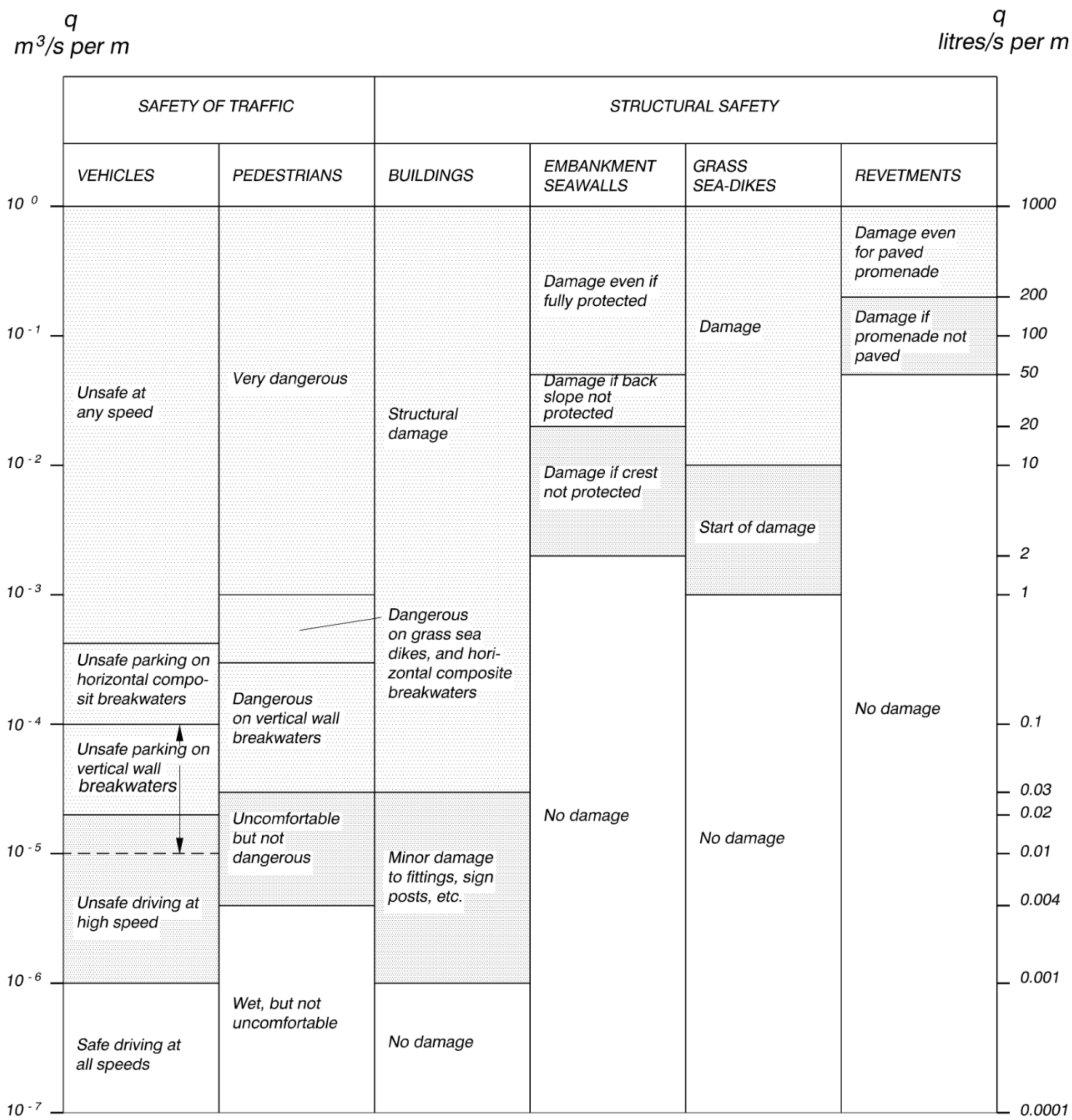


Figure 37: Joint Wave Probability, Island Heights to Manasquan

Tolerable Wave Overtopping Rates

Floodwalls that are exposed to heavy wave overtopping for many hours are susceptible to structural failure (Goda, 2000). Therefore, floodwalls are often designed to limit wave overtopping below a tolerable overtopping rate based on the structure type, property and operation, and people and vehicles. EM 1110-2-1100 provides guidelines for critical mean wave overtopping rates of several structure types before the structure begins to exhibit damage which may eventually lead to structural failure. Based on available literature including European and United States reference documents including **Table 29**, a tolerable mean wave overtopping rate of 10 liters/s/m is selected for floodwalls, rubble slopes (armored levees), and bay closures in the NJBB study. A tolerable mean wave overtopping rate of 200 liters/s/m is selected for storm surge barriers located at inlets. Floodwalls with an adequate splash apron could handle higher rates of wave overtopping before suffering structural damage and failure. However, houses and infrastructure are located in close proximity to the floodwalls and higher rates of overtopping could cause damage to homes and infrastructure, localized flooding/ponding, and threat to life safety.

Table 14: Tolerable Values of Mean Wave Overtopping (EM 1110-2-1100)



EurOtop (2016) and EM 1110-2-1100 highlight the importance of peak wave overtopping from a single wave on tolerable wave overtopping values. Overtopping discharge from a single wave can be more than 100 times the mean overtopping discharge during the storm peak (EM 1110-2-1100) and is often responsible for structural damages. Peak wave overtopping volumes have been shown to be strongly dependent on the wave height (EurOtop, 2016). For a given mean overtopping discharge, small waves only give small overtopping volumes, whereas large waves may give a much larger overtopping volumes for a single wave (EurOtop, 2016). In that sense mean tolerable overtopping rates should also be coupled to the wave height (EurOtop, 2016). Since the design wave conditions in the NJBB study area are relatively small the tolerable mean wave overtopping rate selected for this study should be considered conservative relative to higher wave energy environments.

Overtopping Formulas

Vertical Wall

Mean wave overtopping rates are calculated for vertical walls using empirical formulas provided by EurOtop (2016). Results from EurOtop are compared to Franco and Franco (1999) as described in EM 1110-2-1100 and Ward and Ahrens (1992). The primary parameters in all of these wave overtopping formulas are the crest freeboard (R_c) and wave height (H_{m0}) as shown in **Figure 50**. The water depth (h), slope of foreshore (1:m), and wave period are important parameters in shallow water.

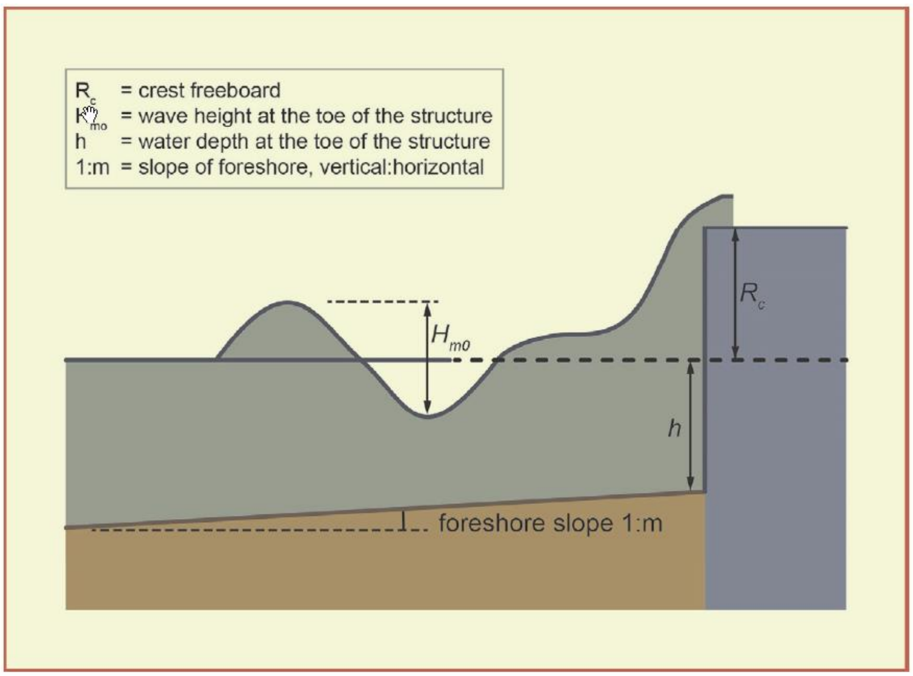


Figure 38: Wave Overtopping Parameters (EurOtop, 2016)

The five wave overtopping formulas for vertical walls evaluated here are:

- EurOtop equations 7.1 and 7.2 for non-impulsive wave conditions;
- EurOtop equations 7.5 and 7.6 for non-impulsive wave conditions with an influencing foreshore;
- EurOtop equations 7.6, 7.8, 7.9, and 7.10 for impulsive wave conditions;
- Franco & Franco (1999), Table VI-5-13 in EM 1110-2-1100;
- Ward & Ahrens (1992), Group 1 Seawalls.

The general equation for the empirical formulas is:

$$Q = a\ exp[-(bR)^c]$$

where Q and R are the non-dimensional representation of the mean wave overtopping rate, *q*, and freeboard, R_c,

$$Q = \frac{q}{\sqrt{gH_{m0}^3}}\ , \qquad R = \frac{R_c}{H_{m0}}$$

and *a*, *b*, and *c* are constants. This general equation is used by Franco & Franco (1999) and the EurOtop formulas for non-impulsive (i.e. non-breaking) wave conditions. The empirical formulas for Ward and Ahrens (1992) and EurOtop formula for impulsive wave conditions follow this general form but also include parameters based on the water depth, slope of foreshore, and wave period. A comparison of three EurOtop formulas are shown in **Figure 51**, where the strong dependence of wave overtopping on the relative freeboard is shown. It is apparent from Error! Reference source not found. that under small relative freeboard conditions, $R_c/H_{m0} < 1$, the three wave overtopping formulas produce similar results. As the relative freeboard increases the impulsive wave (breaking wave) conditions produce higher rates of wave overtopping and the impact of the foreshore becomes more significant.

The EurOtop Manual provides two sets of formulas, the “Mean value approach” and “Design or assessment approach”. The mean value approach should be used to predict or compare with test data and the design or assessment approach includes a partial safety factor with one standard deviation above the mean value approach. The difference between the approaches is shown in **Figure 52** for non-impulsive wave conditions.

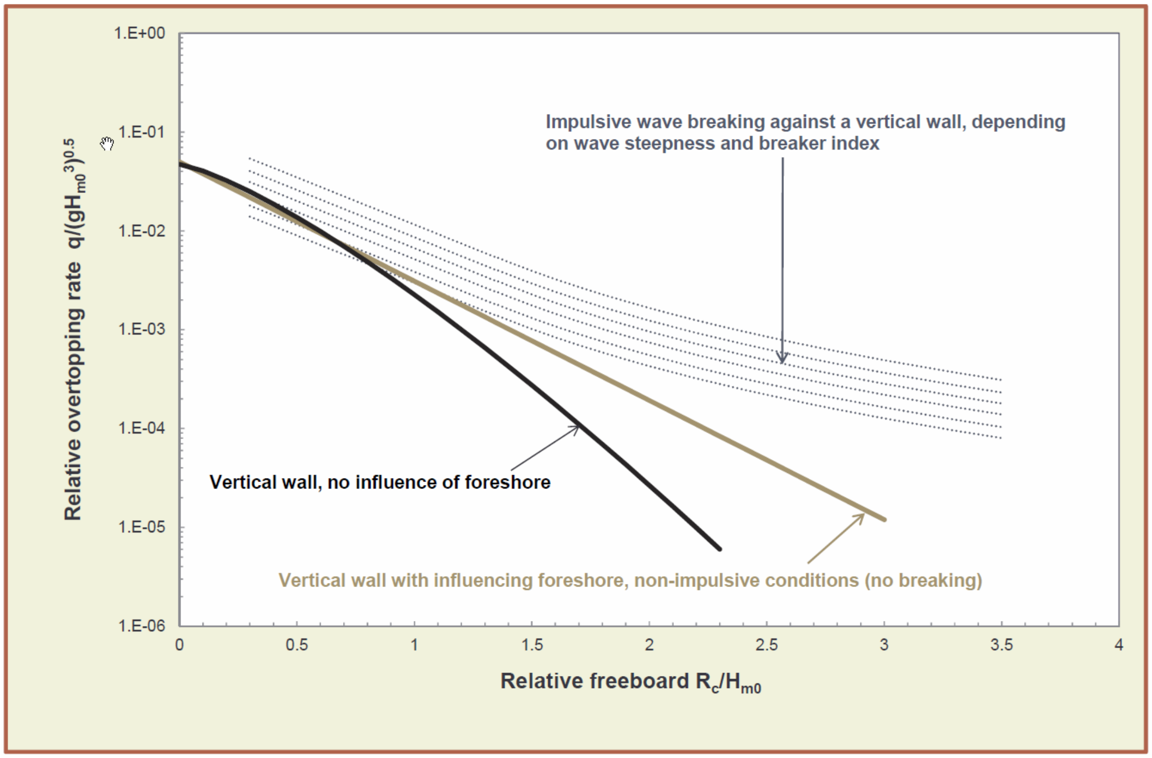


Figure 39: Non-dimensional Overtopping and Freeboard (EurOtop, 2016)

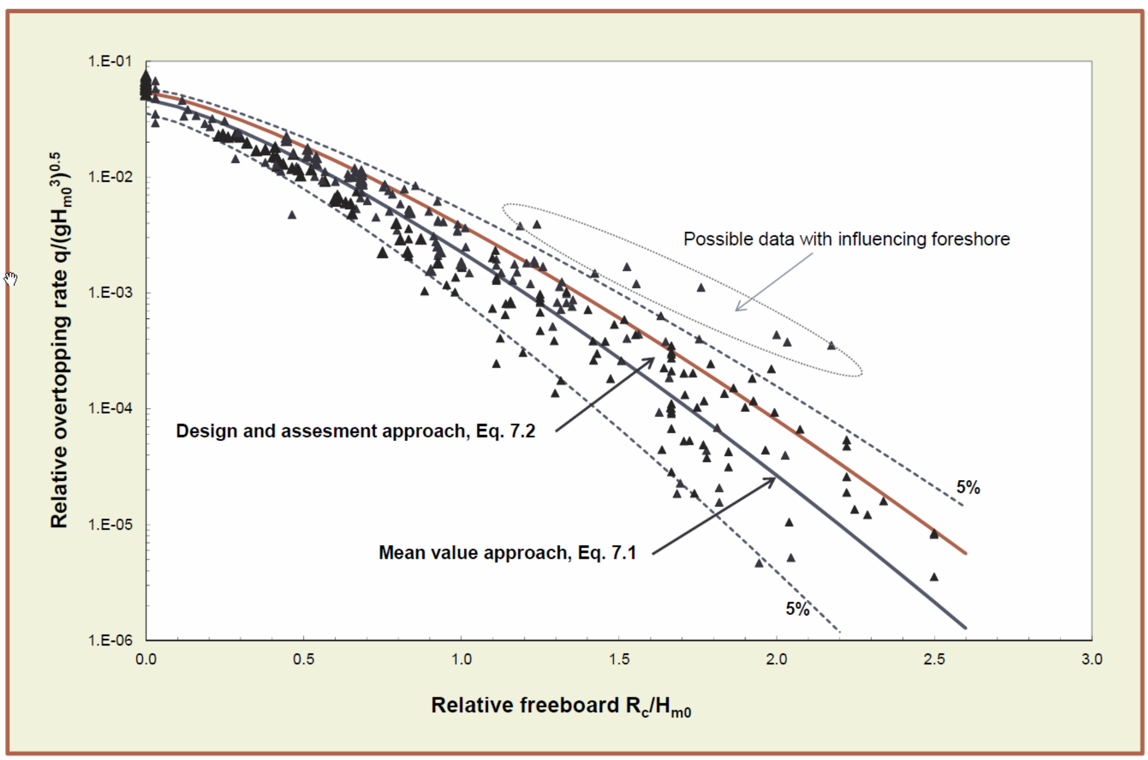


Figure 40: Mean Value and Design Approaches (EurOtop, 2016)

Rubble Slope

The primary focus of the wave overtopping analysis is on vertical walls (i.e. floodwalls) since they are the primary measure under consideration in the Perimeter Plan. However, there are some locations where a rubble slope (i.e. armored levee) is more appropriate and economical. Mean wave overtopping rates are calculated for rubble slopes using empirical formulas provided by EurOtop (2016). The general formula for the rubble slope is the same as the vertical wall with other influence factors that account for roughness associated with the armor stone, oblique wave attack, crest berm, composite slopes, and wave wall at crest. EurOtop (2016) provides a formula for the "Mean value approach" and "Design or assessment approach".

Comparison of Formulas

Due to the size of the study area, there will be considerable variability in the local site conditions, such as the wave conditions, water depth, and foreshore slope. Rather than perform a detailed analysis at every site, several representative sites are selected throughout the study area and the sensitivity to the wave overtopping formulas is evaluated. This approach provides confidence in the results and a deeper understanding of the most important parameters governing wave overtopping in the study area.

Three sets of wave conditions are evaluated:

- Wave Height = 1 m, Wave Period = 4 s, Water Depth = 3m;
- Wave Height = 2 m, Wave Period = 8 s, Water Depth = 3m;
- Wave Height = 4 m, Wave Period = 12 s, Water Depth = 10m;

The first set of wave conditions are fairly representative of the design wave conditions found in the NJBB study area. The second set of wave conditions are included to illustrate how the results are affected by the wave conditions. The third set of wave conditions is representative of the conditions at the storm surge barriers located inside the tidal inlets. **Figure 53** and presents the wave overtopping results on a vertical wall for the first two wave conditions over a range of freeboard heights in terms of the relative wave overtopping and relative freeboard. **Figure 54** presents the wave overtopping results for the third wave condition, representative of the wave conditions at the storm surge barriers.

In order to provide context to the non-dimensional figures, the tolerable wave overtopping rate of 50, 10, and 2 liters/s/m, is plotted in **Figure 53**. The intersection of the wave overtopping formulas and tolerable rate of wave overtopping represents the relative freeboard, R_c/H_{m0}, required to limit wave overtopping below this tolerable rate. For the 1-meter wave height conditions, a relative freeboard of about 0.8 is required to limit wave overtopping below 10 liters/s/m for all the formulas except Ward & Ahrens, which requires a higher freeboard. Said differently, the freeboard must be equal to or greater than 80 percent of the wave height. For the 2-meter wave height conditions a relative freeboard of 1.2 is required to limit wave overtopping below the tolerable rate.

It is apparent from this analysis that the required relative freeboard for a vertical wall is not very sensitive to the wave overtopping formula, especially in the 1-meter waves, with the exception of Ward & Ahrens. Ward & Ahrens based their formula on physical lab experiments with impulsive wave conditions with wave heights generally greater than 2m and wave periods between 8 and 12 seconds. Therefore, the Ward & Ahrens formula is better suited for larger wave conditions not found within the NJBB study area. It can be seen from **Figure 53** that Ward & Ahrens produce similar results to the impulsive EurOtop formulas for the 2 meter wave conditions within the 50/liter/s/m to 2/liters/s/m overtopping range.

Wave overtopping for the rubble slope (solid blue line) is very similar to vertical walls and it is expected that the required relative freeboard will be similar between the vertical wall and rubble slope.

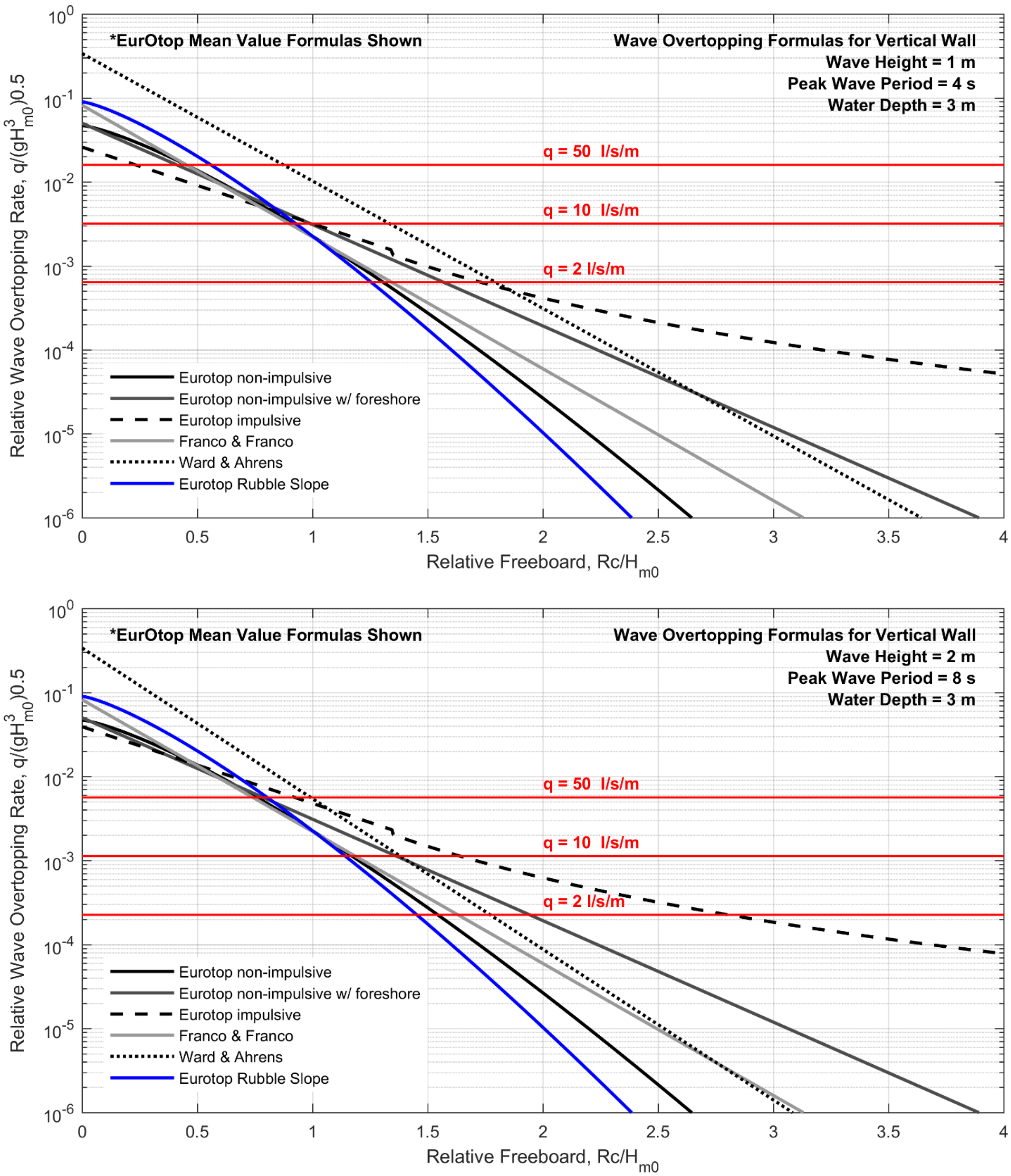


Figure 41: Wave Overtopping Formulas for Vertical Wall

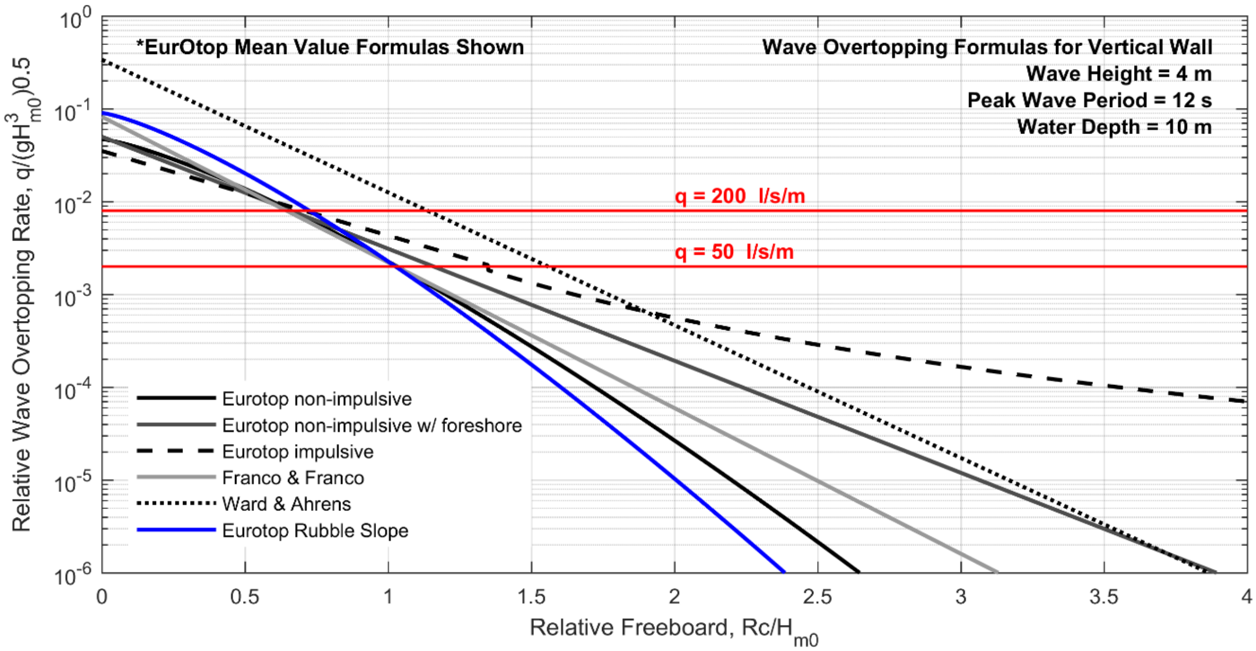


Figure 42: Wave Overtopping Formulas Applied to Storm Surge Barriers

Overtopping Results

Vertical Wall

The results from the wave overtopping analysis at the 11 representative locations are presented in **Table 30**. The required relative freeboard, R_c/H_{m0}, and freeboard height, R_c, to keep wave overtopping below the tolerable threshold, 10 liters/s/m, are given. The results in **Table 30** are based on the EurOtop equation for non-impulsive conditions with an influencing foreshore. The more conservative "design approach" formula was applied. The required relative freeboard increases with wave height and varies between 0.5 in northern Barnegat Bay where the wave conditions are the smallest, to 1.0 at Tuckerton where the wave conditions are the largest. The actual freeboard height varies between 1.6 and 5.2 feet, with all but Tuckerton and Somers below 3.9 feet.

The sensitivity of the relative freeboard height to EurOtop "mean value" and "design approach", as well as the Franco & Franco equation, are presented in **Table 31**. Differences between the three equations are relatively small and the EurOtop "design approach" generally requires the greatest relative freeboard. Results for Ward & Ahrens are not presented here because the wave conditions in the NJBB are smaller than the range of values used in their laboratory experiment. It is more likely that the wave conditions will be non-impulsive during the design conditions considering the small wave periods, small wave heights, and water depths during the 1% AEP.

Table 15: Wave Overtopping Results at Vertical Wall, Relative Freeboard

Station	ID	SWEL (ft, NAVD88)	H_{m0}-Joint (ft)	Tp-Joint (s)	R_C/H_{m0} (-)	Rc (ft)
Cape May	15566	10.3	3.4	3.1	0.8	3.9
Wildwood	11282	10.5	3.1	3.8	0.8	3.3
Ocean City	11309	10.2	2.4	3.3	0.6	2.2
Somers	11230	10.7	3.6	3.5	0.8	4.3
Atlantic City	13554	10.0	2.3	3.3	0.6	2.2
Beach Haven	11399	7.9	2.8	3.5	0.8	3.0
Tuckerton	11444	8.5	4.1	4.3	1.0	5.2
Lavallette	11511	8.3	2.3	3.4	0.6	2.2
Island Heights	13684	7.9	2.0	3.4	0.6	1.8
Mantoloking	13706	9.5	1.9	2.9	0.5	1.6
Manasquan	13711	10.2	2.1	2.8	0.6	1.8

Table 16: Relative Freeboard Sensitivity, Vertical Wall

Station	ID	EurOtop w/ Foreshore Mean Value Approach	EurOtop w/ Foreshore Design Approach	Franco & Franco
Cape May	15566	1.01	1.16	0.91
Wildwood	11282	0.96	1.10	0.87
Ocean City	11309	0.81	0.95	0.77
Somers	11230	1.04	1.19	0.93
Atlantic City	13554	0.81	0.95	0.77
Beach Haven	11399	0.92	1.05	0.85
Tuckerton	11444	1.11	1.27	0.99
Lavallette	11511	0.81	0.95	0.77
Island Heights	13684	0.74	0.87	0.71
Mantoloking	13706	0.70	0.84	0.68
Manasquan	13711	0.74	0.89	0.71

Rubble Slope

The results from the wave overtopping analysis at the 11 representative locations are presented in **Table 32**. The required relative freeboard, R_c/H_{m0}, and freeboard height, R_c, to keep wave overtopping below the tolerable threshold, 10 liters/s/m, are given. The results in **Table 32** are based on the EurOtop equation for rubble slopes using the more conservative "design approach" formula. The required relative freeboard increases with wave height and varies between 0.7 in northern Barnegat Bay where the wave conditions are the smallest, to 0.9 at Tuckerton where the wave conditions are the largest. The actual freeboard height varies between 1.7 and 4.7 feet, with all but Tuckerton and Somers below 3.6 feet.

Table 17: Wave Overtopping Results at Rubble Slope, Relative Freeboard

Station	ID	SWEL (ft, NAVD88)	H_{m0}-Joint (ft)	Tp-Joint (s)	R_C/H_{m0} (-)	Rc (ft)
Cape May	15566	10.3	3.4	3.1	0.9	3.6
Wildwood	11282	10.5	3.1	3.8	0.8	3.2
Ocean City	11309	10.2	2.4	3.3	0.7	2.2
Somers	11230	10.7	3.6	3.5	0.9	3.9
Atlantic City	13554	10.0	2.3	3.3	0.7	2.2
Beach Haven	11399	7.9	2.8	3.5	0.8	2.9
Tuckerton	11444	8.5	4.1	4.3	0.9	4.7
Lavallette	11511	8.3	2.3	3.4	0.7	2.2
Island Heights	13684	7.9	2.0	3.4	0.7	1.8
Mantoloking	13706	9.5	1.9	2.9	0.7	1.7
Manasquan	13711	10.2	2.1	2.8	0.7	1.9

Storm Surge Barriers

The results from the wave overtopping analysis at the storm surge barrier locations are presented in **Table 33**. The required relative freeboard, R_c/H_{m0}, and freeboard height, R_c, to keep wave overtopping below the tolerable threshold, 200 liters/s/m, are given. The results in **Table 33** are based on the EurOtop equation for non-impulsive conditions with an influencing foreshore. The more conservative "design approach" formula was applied. Wave conditions at the mouth of the inlet are transformed through the inlet to the location of the storm surge barrier using Dalrymple's 1992 paper on "Water Wave Propagation in Jettied Channels". In subsequent phases of the study more detailed wave modeling will be performed to determine the wave conditions at the storm surge barriers.

Table 18: Wave Overtopping at Storm Surge Barriers, Relative Freeboard

Station	SWEL (ft, NAVD88)	H_{m0} (ft)	Tp (s)	R_C/H_{m0} (-)	Rc (ft)
Great Egg Inlet	10.6	8.8	14.0	0.58	5.2
Absecon Inlet	10.0	9.7	14.0	0.63	6.1
Barnegat Inlet	8.8	8.6	14.0	0.57	4.9
Manasquan Inlet	9.3	11.1	14.0	0.70	7.8

The required freeboard at storm surge barriers at the cross-bay closures, is the equal to the results provided for the vertical floodwalls and rubble slopes inside the bays. The wave conditions and tolerable wave overtopping rate of 10 liters/s/m for the bay closures are within the range of values evaluated for the vertical walls and rubble slopes located inside the back bays.

Total Water Level and Crest Elevations

Total Water Level Components

The total water level component analysis identifies all the contributions to the water surface elevation applied in the design structural crest elevations. The significant water level components for the NJBB study area are shown below:

- **Mean Sea Level**
  - Mean Sea Level (MSL) is a tidal datum, is mean or average sea level computed over a 19-year period, known as the National Tidal Datum Epoch (NTDE). The present 19-year reference period used by NOAA is the 1983-2001 NTDE.
  - Relative Sea Level Change (RSLC) is a combination of both global and local SLC including local vertical land motion (subsidence or uplift).
- **Astronomical Tide** is the semi-diurnal (twice daily) periodic rise and fall of a body of water resulting from gravitational interactions between Sun, Moon, and Earth.
- **Non-Tidal Residuals**
  - Seasonal variations in sea level from regular fluctuations in coastal temperatures, salinities, winds, atmospheric pressures, and ocean currents.
  - Interannual variations in sea level from irregular fluctuations in coastal temperatures, salinities, winds, atmospheric pressures, and ocean currents (El Niño).
  - Storm Surge is the increased water level due to storm winds over the ocean and the resultant wind stress on the ocean surface.
- **Wave-induced Components**
  - Wave Setup is the increase in water level from wave breaking in the nearshore.

– Freeboard is additional height of a structure (i.e. levee, floodwall) above the still water level required to limit wave overtopping below a tolerable discharge. On sloped structures such as levees the freeboard height is related to wave runup.

Design Crest Elevations (1% AEP)

Preliminary crest elevations for structural measures (Floodwalls, Levees, Storm Surge Barriers) are based on the 1% AEP with 50% assurance provided in the NJBB Baseline and NACCS hazard curves. It is emphasized that there is no policy requirement that USACE projects be designed to the 1% AEP water level or any minimum performance standard. In subsequent phases of the NJBB Feasibility Study the performance of the measures will be optimized to maximize NED benefits, which could result in higher or lower performance. The decision to design structures to the 1% AEP water level at this stage of the study is consistent with the parametric designs in NACCS and ECB 2013-33 that required all Sandy rebuilding projects receiving funds for construction under the Sandy supplemental (Public Law 113-2) be meet a flood risk reduction standard of one foot above the best available and most recent base flood elevation.

The relative contribution of the each respective total water level component towards the perimeter plan design crest elevation at three representative locations is provided in **Table 34**. The NJBB CSTORM and NACCS water level hazard curves include several of the total water level components: MSL, astronomical tide, storm surge, and wave setup. The water level hazard curves represent the join probability of all the components combined and the exact relative contribution of each component is not well defined. However, the relative contribution of each component is estimated here based on the well-known tidal amplitudes (MHW) and approximate estimates of wave setup based on the wave heights.

RSLC is included by adding 2 feet, based on the USACE Intermediate SLC scenario. The required freeboard for each structure was determined based on wave overtopping calculations and tolerable overtopping rate. Seasonal variations in sea level are included based on average seasonal fluctuation during peak hurricane season (August, September, October) observed NOAA tidal gage at Atlantic City. Inter-annual variations in sea level are not included in the TWL estimate or design crest elevations at this time and rarely exceed 0.5 feet.

Design and cost estimates of the perimeter plan floodwalls and levees are based on a crest elevation of 16 feet NAVD88.Due to the spatial variability in water levels, wave conditions, and wave overtopping there are some locations where the required crest elevation of the perimeter plan features could be lower than 16 feet NAVD88 and a few locations where the perimeter plan may need to be slightly higher.

Table 19: Perimeter Plan Crest Elevations and Total Water Level Components

Component	Wildwood (feet)		Ocean City (feet)		Beach Haven (feet)	
MSL (feet, NAVD88)	-0.40	10.5[2]	-0.40	10.2[2]	-.40	7.9[2]
Astronomical Tide	1.8[1]		1.6[1]		1.2[1]	

Storm Surge	8.9		8.8		8.4	
Wave Setup	0.2		0.2		0.2	
RSLC	2.0		2.0		2.0	
Seasonal Variations	0.3		0.3		0.3	
Freeboard	3.3[3]		2.2[3]		3.0[3]	
Total Water Level (feet, NAVD88)	**16.1**		**14.7**		**13.2**	

Notes: [1]MHW shown; [2]Value from NACCS hazard curve in feet, NAVD88; [3]Freeboard based on wave overtopping of vertical wall.

Conceptual design and cost estimates of the storm surge barriers are based on a crest elevation of 17 to 20 feet NAVD88 as shown in **Table 35**. Design crest elevations for the bay closures are set to the same elevation as the perimeter plan, 16 feet NAVD88. Additional refinement and granularity will be included in design crest elevations in subsequent phases of the Feasibility Study.

Table 20: Storm Surge Barrier Crest Elevations and Total Water Level Components

Component	**Great Egg Inlet (feet)**		**Absecon Inlet (feet)**		**Barnegat Inlet (feet)**		**Manasquan Inlet (feet)**	
MSL (feet, NAVD88)	-0.40	10.6[2]	-0.40	10.0[2]	-0.40	8.8[2]	-0.40	9.3[2]
Astronomical Tide	1.6[1]		1.6[1]		1.6[1]		1.6[1]	
Storm Surge	9.4		8.8		7.6		8.1	
Wave Setup	0.0		0.0		0.0		0.0	
SSB Induced	1.0		1.0		1.0		1.0	
RSLC	2.0		2.0		2.0		2.0	
Seasonal Variations	0.3		0.3		0.3		0.3	
Freeboard	5.1[3]		6.1[3]		4.9[3]		7.8[3]	
Total Water Level (feet, NAVD88)	**19.0**		**19.4**		**17.0**		**20.4**	

Notes: [1]MHW shown; [2]Value from NACCS hazard curve in feet, NAVD88; [3]Freeboard based on wave overtopping of vertical wall.

Performance

ER 1105-2-101 requires risk assessment for CSRM studies. At this stage of the NJBB CSRM Study the risk assessment provides additional information about the relative project performance, structural performance and reliability, and life safety that is not provided by the NED economic results. In addition, the impact of sea level change on the system performance is helpful to consider the strengths, weaknesses, and adaptability of different alternatives. The focus here is on nonstructural (elevating structures) and perimeter plans (floodwalls), storm surge barriers are not included. Definitions for a few commonly used terms in this section are provided below:

Annual Exceedance Probability (AEP) - The probability that a certain threshold may be exceeded at a location in any given year, considering the full range of possible values, and if appropriate, incorporation of project performance. The AEP is expressed as a percentage. An event having a one in 100 chance of occurring in any single year would be described as the one percent AEP event.

Assurance - The probability that a target stage will not be exceeded during the occurrence of a flood of specified exceedance probability considering the full range of uncertainties. Term selected to replace "conditional non-exceedance probability" (CNP).

Long-Term Exceedance Probability (LTEP) - The probability of capacity exceedance during a specified period. For example, 30-year exceedance probability refers to the probability of one or more exceedances of the capacity of a measure during a 30-year period, formerly long-term risk. This accounts for the repeated annual exposure to flood risk over time.

Structural Performance and Reliability

There are significant differences in the reliability and consequences of failure between nonstructural, perimeter plans, and storm surge barriers. Nonstructural plans generally provide exceptional reliability, require little active intervention, and independent failure points. Failure of a single structure will not lead to failure of the entire system. In addition, people located inside elevated structures will be able to evacuate vertically inside the structure or to the roof to greater elevations, potentially reducing life loss.

Perimeter plans in the NJBB study area are far from trivial in extent and complexity. Every road closure, rail closure, miter gate, and structure transition are failure points that require active intervention in advance of storm events. In addition, perimeter plans are exposed to waves, wave overtopping, and possible failure over several miles along the system. Storage capacity of perimeter plans is limited, and flood damages occur rapidly after wave overtopping begins. Failure of a floodwall, transition, or closure/gate will quickly overwhelm any storage capacity resulting in high velocities, rapid increases in flood elevations, conditions that may increase the risk of life loss.

Storm surge barriers are similar to perimeter plans in that both are extensive and complex structural solutions. A few of the advantages of the storm surge barriers is that the number of failure points is reduced and concentrated on larger gate structures (i.e. sector gates and vertical lift gates) with the linear extent of the system of gates, floodwalls, and levees significantly reduced from the perimeter plan. A significant advantage of the storm surge barriers is additional storage capacity of the back bays to accommodate wave overtopping, breaches, and failures resulting in slower and less severe increases in water levels. Another advantage of the storm surge barrier alternatives is the adaptive capacity of the system or the ability to add complimentary measures such as non-structural or smaller perimeter plans over time in response to RSLR to maintain a high level of performance.

Project "System" Performance

Project performance is evaluated for four plans:

- Nonstructural plans with structures elevated to the 1% AEP
- Perimeter Plan (1% AEP)
- Storm Surge Barriers

At this stage of the study both the alternatives have been designed for 2 feet of RSLR based on the USACE-Intermediate SLC scenario. Project Performance is evaluated by determining the AEP, LTEP, and assurance associated with the water level exceeding the design first floor elevation (Nonstructural) or floodwall crest elevation. It is assumed that when these water elevations are reached the elevated structures will begin to experience significant damages and in the case of the perimeter plan wave overtopping or wall failure will lead to significant damages.

Unlike nonstructural and perimeter plans, the storm surge barriers transform the back bay water levels by reducing storm surge propagation into the back bays. The water levels for the 2% AEP event are not zero and some of the more vulnerable structures with lower first floor elevations (FFE) may still experience damages. Therefore, the performance of the storm surge barrier alternatives is evaluated by determining when structures over a range of FFEs would experience damage from the With-Project water level exceeding that FFE.

Project performance (AEP, LTEP, and assurance) in the year 2080 assuming RSLR has followed the USACE-Intermediate SLC scenario (2 feet) is presented in **Table 36**. Since both the nonstructural plan and 1% AEP perimeter plan are designed to the 1% AEP in 2080, the AEP is equal to 1%, and the LTEP and assurances are the same. For the storm surge barrier alternatives, the performance is very high for structures with relatively high FFE, but the more vulnerable structures with a lower FFE experience relatively lower performance and may be good candidates for complimentary NS plans.

Table 21: Project Performance: AEP, LTEP, Assurance at Year 2080 (USACE Int. SLC)

Plan	AEP		LTEP			Assurance by Event				
	Expected	90% Assurance	10-yr Period	30-yr Period	50-yr Period	10%	2%	1%	0.4%	0.2%
Nonstructural	0.91%	2.37%	8.8%	24.1%	36.8%	99.9%	85.0%	54.6%	17.9%	6.1%
Perimeter Plan	0.91%	2.37%	8.8%	24.1%	36.8%	99.9%	85.0%	54.6%	17.9%	6.1%
Storm Surge Barrier, FFE 14'	0.01%	0.01%	0.1%	0.3%	0.5%	99.9%	99.9%	99.9%	99.9%	99.9%
Storm Surge Barrier, FFE 12'	0.01%	0.06%	0.1%	0.3%	0.5%	99.9%	99.9%	99.9%	99.9%	99.8%
Storm Surge Barrier, FFE 10'	0.09%	0.27%	0.9%	2.8%	4.6%	99.9%	99.9%	99.9%	97.7%	80.2%
Storm Surge Barrier, FFE 8'	0.48%	0.95%	4.7%	13.5%	21.5%	99.9%	99.9%	91.8%	38.9%	12.4%
Storm Surge Barrier, FFE 6'	1.84%	2.79%	17.0%	42.8%	60.5%	99.9%	59.5%	7.9%	0.5%	0.1%

Sea Level Change and Adaptability

ER 1110-2-8162 requires the performance of alternatives to be evaluated under all three USACE SLC scenarios to determine the alternatives overall potential performance. Not only is it possible that RSLC could be lower or greater than the USACE Intermediate SLC scenario, it is also possible that the plans will have a service life well beyond 50 years. Therefore, it is important to consider the sensitivity of the project performance to RSLC and the adaptive capacity of the alternatives.

The project performance of the four plans over a 100 year planning period is provided in **Table 37**. The table captures the impact RSLC has on project performance. The left side of **Table 37** shows the AEP in 2030, 2055, 2080, 2105, and 2130. The right side of the table shows the LTEP over four 25-year periods.

In the USACE-Intermediate SLC scenario the AEP for the 1% AEP Perimeter Plan increases from 0.43% at the start of the project (2030) to 0.91% in 2080 and then 2.59% in 2130. Another way to look at the project performance, is that the LTEP (probability of a single event exceeding the design water level) increases from 12.5% in the first 25-years of the project (2030-2055) to 17.2% in the second 25-years of the project (2055-2080). In the last 50-years of the project the LTEP increases from 25.1% (2080-2105) and 38.6% (2105-2030).

Table 22: Project Performance: AEP, LTEP sensitivity to SLC

Plan	Annual Exceedance Probability, Expected					Long-Term Exceedance Probability			
	2030	2055	2080	2105	2130	2030-2055	2055-2080	2080-2105	2105-2130
USACE Low SLC Scenario									
Nonstructural	0.43%	0.51%	0.62%	0.74%	0.90%	11.1%	13.1%	15.6%	18.5%
Perimeter Plan	0.43%	0.51%	0.62%	0.74%	0.90%	11.1%	13.1%	15.6%	18.5%
Storm Surge Barrier, FFE 14'	0.01%	0.01%	0.01%	0.01%	0.01%	0.2%	0.2%	0.2%	0.2%
Storm Surge Barrier, FFE 12'	0.01%	0.01%	0.01%	0.01%	0.01%	0.2%	0.2%	0.2%	0.2%
Storm Surge Barrier, FFE 10'	0.02%	0.03%	0.04%	0.06%	0.09%	0.6%	0.9%	1.3%	1.9%
Storm Surge Barrier, FFE 8'	0.17%	0.22%	0.29%	0.37%	0.47%	4.8%	6.1%	7.8%	9.9%
Storm Surge Barrier, FFE 6'	0.78%	1.00%	1.21%	1.48%	1.81%	19.8%	24.1%	28.6%	33.7%
USACE Int SLC Scenario									
Nonstructural	0.46%	0.63%	0.91%	1.47%	2.59%	12.5%	17.2%	25.1%	38.6%
Perimeter Plan	0.46%	0.63%	0.91%	1.47%	2.59%	12.5%	17.2%	25.1%	38.6%
Storm Surge Barrier, FFE 14'	0.01%	0.01%	0.01%	0.01%	0.01%	0.2%	0.2%	0.2%	0.2%
Storm Surge Barrier, FFE 12'	0.01%	0.01%	0.01%	0.02%	0.07%	0.2%	0.2%	0.3%	1.0%
Storm Surge Barrier, FFE 10'	0.02%	0.05%	0.09%	0.19%	0.37%	0.8%	1.6%	3.3%	6.4%
Storm Surge Barrier, FFE 8'	0.19%	0.29%	0.48%	0.85%	1.50%	5.7%	8.9%	14.8%	24.9%
Storm Surge Barrier, FFE 6'	0.86%	1.23%	1.84%	3.07%	6.86%	22.8%	31.4%	44.9%	65.5%
USACE High SLC Scenario									
Nonstructural	0.58%	1.21%	3.66%	24.92%	100.00%	18.5%	41.1%	92.9%	100.0%
Perimeter Plan	0.58%	1.21%	3.66%	24.92%	100.00%	18.5%	41.1%	92.9%	100.0%
Storm Surge Barrier, FFE 14'	0.01%	0.01%	0.01%	0.13%	1.02%	0.2%	0.2%	1.2%	10.0%
Storm Surge Barrier, FFE 12'	0.01%	0.01%	0.11%	0.63%	3.62%	0.3%	1.2%	7.0%	33.7%
Storm Surge Barrier, FFE 10'	0.04%	0.14%	0.55%	2.34%	100.00%	1.9%	7.0%	26.1%	100.0%
Storm Surge Barrier, FFE 8'	0.26%	0.68%	2.06%	37.92%	100.00%	9.9%	26.3%	85.9%	100.0%
Storm Surge Barrier, FFE 6'	1.13%	2.48%	23.96%	100.00%	100.00%	33.8%	79.7%	100.0%	100.0%

It is apparent from in **Table 37** that in the USACE High RSLR scenario the NS and perimeter plan alternatives begin to experience relatively low performance between 2055 and 2080, and would be completely overwhelmed after the initial 50 years of the project. The adaptability of the NS and perimeter plans is limited. In the "footprint" of the plan may be expanded over time to add additional structures that become more vulnerable due to RSLC but going back and re-elevating a structure is not simple or inexpensive. The perimeter plan may be adapted in the future by raising the crest elevation of the structures only if the original foundation is designed to handle the increased loads. Since the foundation is a substantial part of the structure, the initial cost required to over-design the foundation to accommodate a future adaption is significant.

The adaptive capacity of the storm surge barrier structure is low, as it is not feasible to increase the height of vertical lift gate or sector gate, however additional nonstructural or perimeter measures can be implemented over time in adjustment to the SLC rate being experienced without adding expensive adaptability costs to initial construction. Even under the High SLC curve, the initial storm surge barrier design proposed for the TSP can be adapted to maintain project performance over a 100-year planning horizon.

An example of how a relatively high performance in the storm surge barrier alternative may be maintained even under the USACE High SLC curve over a 100-year period is shown in Error! Reference source not found. by adding complimentary NS plans in years 2080 and 2105.

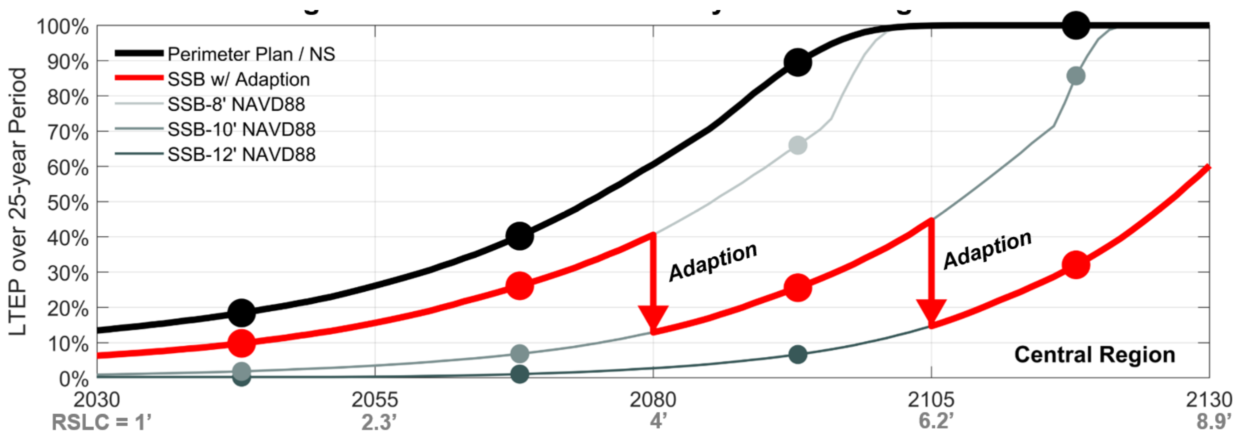


Figure 43: LTEP in USACE High SLC Scenario

AdH Modeling And PTM

Overview

Storm surge barriers are a combination of static impermeable barriers and dynamic gates that may be closed during storm events to reduce storm surges in the back bays. During normal conditions the gates will remain open allowing for tidal exchange between the ocean and bays. However, even under normal conditions when the gates are open, the gate housings, piers, and impermeable barriers will reduce the cross-sectional area across the inlet. The reduction in cross-sectional area causes an increase in velocities through the open gates and has the potential to reduce tidal exchange between the ocean and bays. A reduction in tidal exchange could lead to other physical impacts including changes in back bay tidal ranges, salinity, sediment transport, and other physical factors. These physical impacts may in turn affect water quality, wetlands, ecological processes, and living resources (Orten et al. 2019).

The U.S. Army Engineer District, Philadelphia, requested the U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, to perform hydrodynamic and salinity modeling with the Adaptive Hydraulics (AdH) model and particle tracking model (PTM) of proposed storm surge protection measures at several inlets from the Atlantic Ocean. The twodimensional (2D) AdH model has been developed based on the available data and known primary influences on the physics within the system. The model includes freshwater inflows, tides, salinity, and wind in an effort to reproduce the field for water surface elevation, velocity magnitude and direction, and salinity over a wide range of conditions. The AdH model was validated to available field data for all including a data collection effort performed in February 2019 to collect salinity and discharge/velocity data at three major inlets over a 13-hour tidal cycle – Barnegat, Little Egg, and Great Egg. A detailed description of the model setup and validation are presented as well as the results of several proposed alternatives is provided in Draft Technical Reports by McAlpin & Ross (2020) and Lackey et al. (2020).

AdH modeling was conducted for the TSP and five other alternatives/variations to understand the potential physical impacts of the storm surge barriers and well as the sensitivity of the physical impacts to current design choices: bottom sill elevation, number of gates, location/alignment. In general, the TSP design has the smallest reduction in the existing cross-sectional area and therefore has the smallest physical impacts. The AdH modeling effort did not attempt to model every single alternative in the focused array, but it did capture the two primary SSB alternatives in the Central Region and the only SSB alternative in the North Region in the Focused Array. An overview of the 6 alternative configurations simulated in ADH is provided in **Table 38** and **Figure 56**.

Table 23: AdH With Project Storm Surge Barrier Alternative Configurations

Storm Surge Barrier	WP1*	WP2	WP3	WP4	WP5	WP6
Manasquan Inlet	A1	A1	A1	A1	B1	A1
Barnegat Inlet	A1	A2	A3	A4	B1	C1
Absecon Inlet					A1	A2
Great Egg Inlet	A1	A2	A3	B1	A1	A1
Absecon Bay Blvd	A1	A1	A1	A1		
South Ocean City	A1	A1	A1	A1		

*WP1 is the TSP

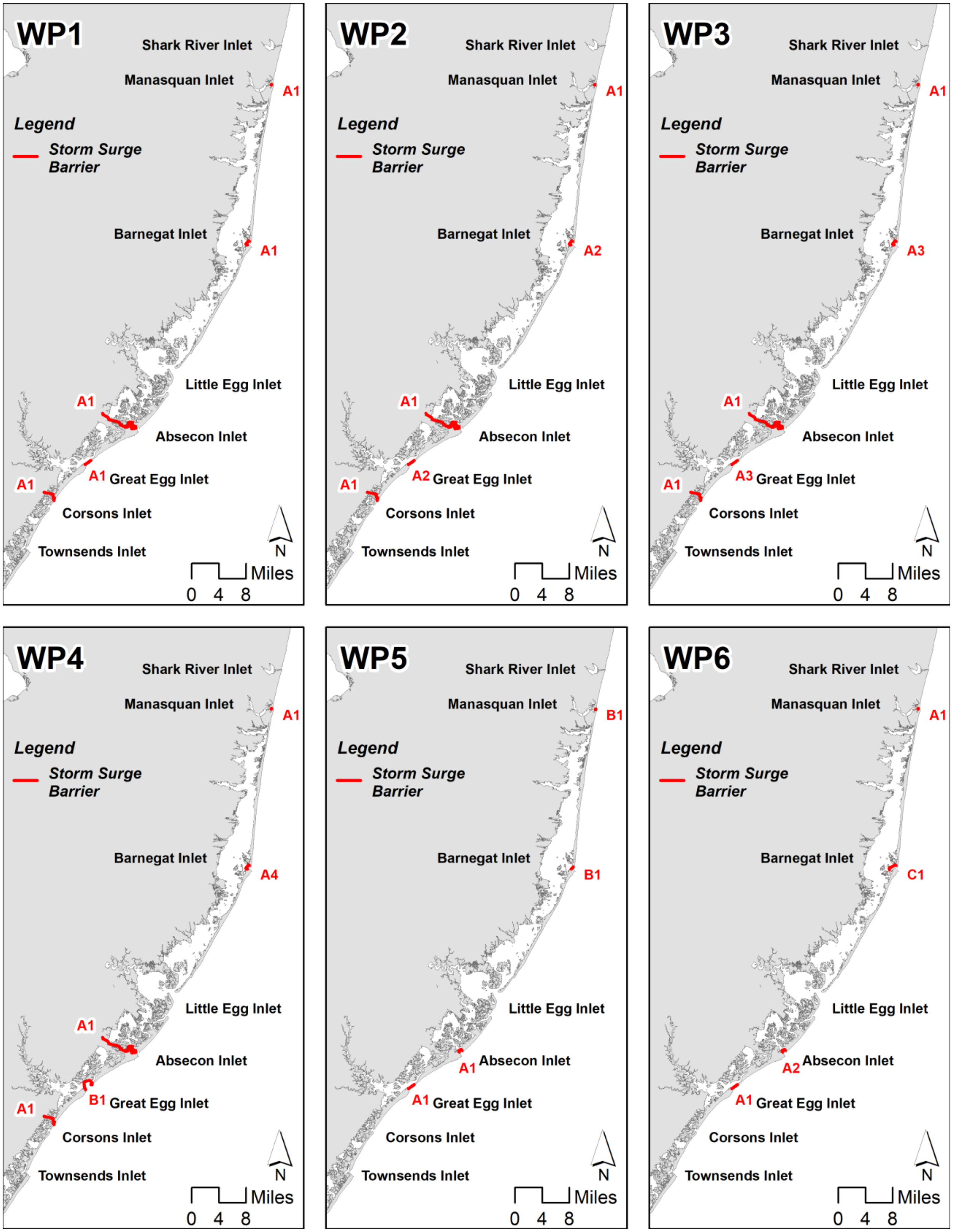


Figure 44: AdH With Project Storm Surge Barrier Alternative Configurations

One of the strengths of the AdH model is its ability to resolve the geometry of the storm surge barriers with really small grid elements. **Figure 57** shows an example of the storm surge barrier design (A1) at Barnegat Inlet and the model resolution. A summary of the different individual storm surge barrier designs evaluated in AdH is provided below, for detailed drawings of each of the storm surge barrier designs see Engineering Appendix B.

- **Manasquan Inlet A1 - TSP Design**
- Manasquan Inlet B1 - Alternative alignment farther seaward
- **Barnegat Inlet A1 - TSP Design**
- Barnegat Inlet A2 - Shallow sill at sector gate
- Barnegat Inlet A3 - Reduced number of vertical lift gates
- Barnegat Inlet A4 - Larger sector gate
- Barnegat Inlet B1 - Alternative alignment across narrow part of inlet
- Barnegat Inlet C1 - Alternative alignment farther into the bay
- **Absecon Inlet A1 - Recommended Design (Not part of TSP)**
- Absecon Inlet A2 - Shallow sill at sector gate
- **Great Egg Inlet A1 - TSP Design**
- Great Egg Inlet A2 - Shallow sill at sector gate
- Great Egg Inlet A3 - Reduced number of vertical lift gates
- Great Egg Inlet B1 - Alternative alignment farther into the bay
- **Absecon Bay Blvd A1- TSP Design**
- **South Ocean City A1 - TSP Design**

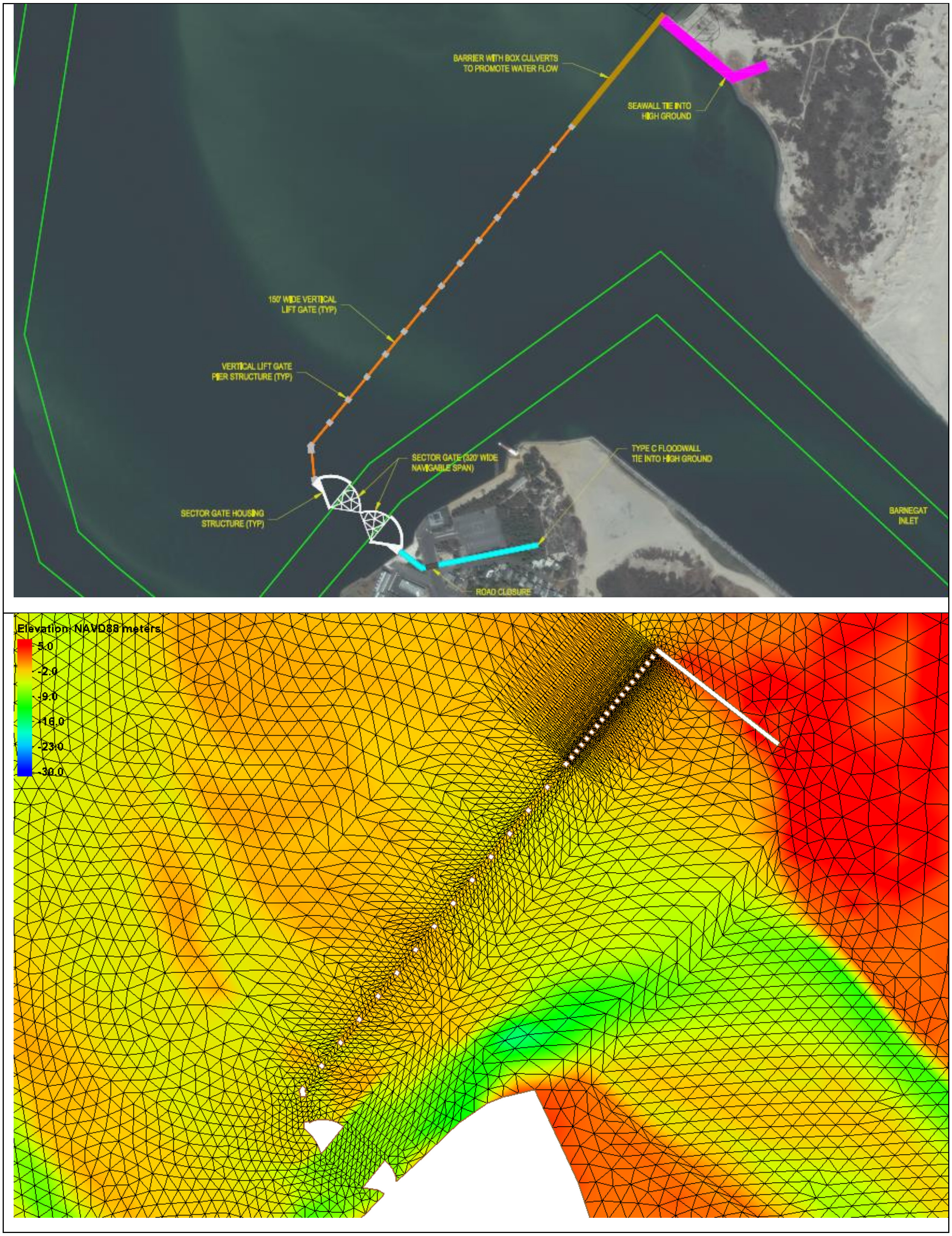


Figure 45: AdH Model Representation of Storm Surge Barrier at Barnegat Inlet (A1)

AdH and PTM Results

Modeling of the storm surge barrier alternatives focused on the physical impacts and the relative changes in tidal prism, tidal amplitudes, near-field and far-field velocities, salinity, and residence time (PTM). AdH is well suited to this application and has been employed in several other US Army Corps Studies investigating potential impacts of storm surge barriers on the environment (Coastal Texas, NYNJ HATS). Although the AdH model is calibrated and validated to field conditions over a range of conditions, the model is best used for determining trends and impacts in a percentage change and range of results type of analyses.

The modeling results show the storm surge barriers cause an increase velocity in vicinity of the structures, the greater the reduction in cross-sectional area the greater the increase in velocities. The alignment of the storm surge barrier was also found to be important and shifting the alignment away from the strongest currents at an inlet can reduce the overall impacts. Many of the alternative design configurations with shallower sills (A2) or with reduced number of vertical lift gates (A3) caused a greater reduction in cross-section area and subsequently the greatest increases in velocities. An example of the impact of the storm surge barrier designs on the near-field velocities is shown in **Figure 58**, **Figure 59**, and **Figure 60** and for Barnegat Inlet, Great Egg Inlet, and Manasquan Inlet respectively. The velocity patterns and magnitudes at the proposed structure locations are greatly changed, as expected, but the impact to velocity magnitudes away from the structures is very little. The velocity at the inlets and structures should be reviewed for impacts to navigation as well as potential sedimentation impacts. However, the changes produced by modifying the flow at the inlets is fairly localized.

One of the primary questions is what impact the storm surge barriers have on the exchange of water between the ocean and bay. The volume of water that enters and leaves an inlet during an average tidal cycle is called the tidal prism. The tidal prism may also be thought of as the surface area of a bay multiplied by the average tidal range. AdH modeling results were used to compute the tidal prism under baseline conditions and each with project configuration for 9 areas that roughly correspond to specific bays or primary influence of each inlet as shown in **Figure 61**. The tidal prism results, **Table 39**, show that nearly all the storm surge barriers result in a reduction of the tidal prism with the greatest impact associated with the more constrictive design configurations. The TSP (WP1) is estimated to have relatively no impact on the tidal prism at Manasquan River, and reduce the tidal prism in Barnegat Bay and Great Egg Harbor by 2.5% and 4.8% respectively. The impacts of the TSP extend beyond the immediate bays at which the closures are located with reductions in tidal prism less than 1.6% elsewhere.

The modeling results proved to be sensitive to the design configurations with tidal prism reductions up to 6.5% in Barnegat Bay for the B1 design and 9.3% at Great Egg Harbor for the A3 design. The Absecon Inlet storm surge barriers, which are not part of the TSP, had the greatest impact on the Absecon Inlet and Little Egg Inlet areas with tidal prism reductions of up to 8.7% and 2% respectively. The impacts to tidal amplitudes are not evenly distributed throughout the bays with individual reductions in tidal amplitude ranging from 1.3% to 8.3% through Barnegat Bay and 0.1% to 4.5% in Great Egg Harbor for the TSP (WP1).

Overall, the impact of the storm surge barriers on salinities is small, and the mean salinity does not vary by more than 2 ppt for any given location and alternative. The variation at specific times may be larger but overall, the impact is small. Given the well mixed nature of the inlets, ocean salinity is pushed into the back bay areas and allowed to move easily throughout the area. The restrictions created by the alternative structures and the reduction in tidal prism are not large enough to significantly impact the salinity at the analysis locations.

Table 24: AdH Tidal Prism Results

Area	Base	WP1*	WP2	WP3	WP4	WP5	WP6
Manasquan Inlet	2,392,363	0.13	-0.62	0.33	0.09	2.19	0.56
Barnegat Inlet	79,322,854	-2.43	-3.80	-4.89	-2.16	-6.53	-1.73
Little Egg Inlet	59,313,965	-0.44	-0.61	-0.60	-0.63	-0.71	-1.98
Absecon Inlet	27,489,518	-0.97	-1.12	-1.15	-1.08	-3.75	-8.70
Great Egg Inlet	51,041,642	-4.76	-6.75	-9.25	-3.20	-4.46	-5.11
Corson Inlet	8,307,946	-1.62	-1.95	-2.06	-1.42	-0.45	-0.82
Townsend Inlet	16,539,248	-1.02	-1.10	-2.37	-2.30	-0.03	-1.49
Hereford Inlet	13,290,585	-0.79	-0.95	-1.69	-1.65	-0.01	-1.11
Cape May Inlet	10,479,052	-0.33	-0.59	-0.73	-0.74	0.01	-0.63

*WP1 is the TSP

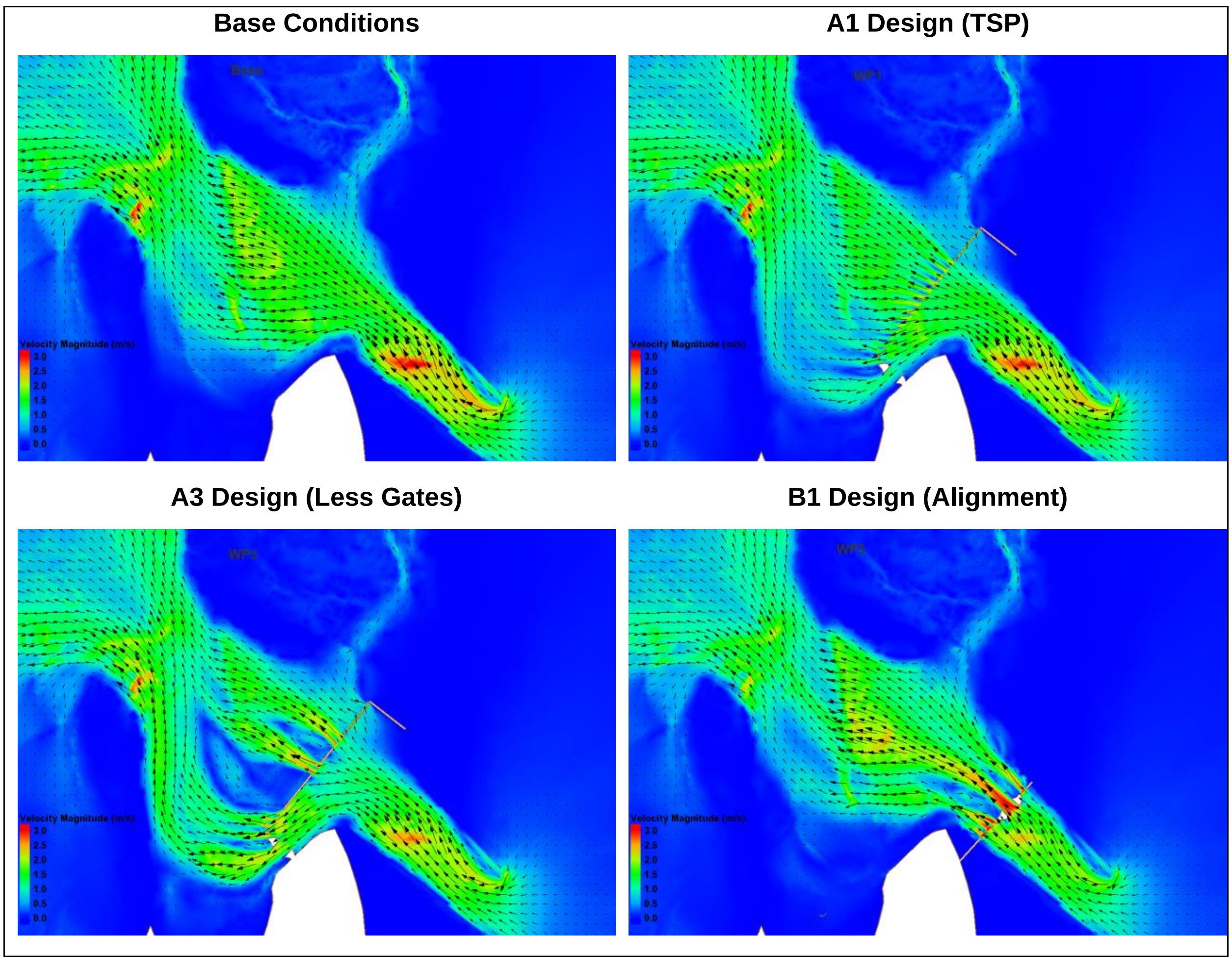


Figure 46: Flood Velocities for Barnegat Inlet Storm Surge Barrier Concepts

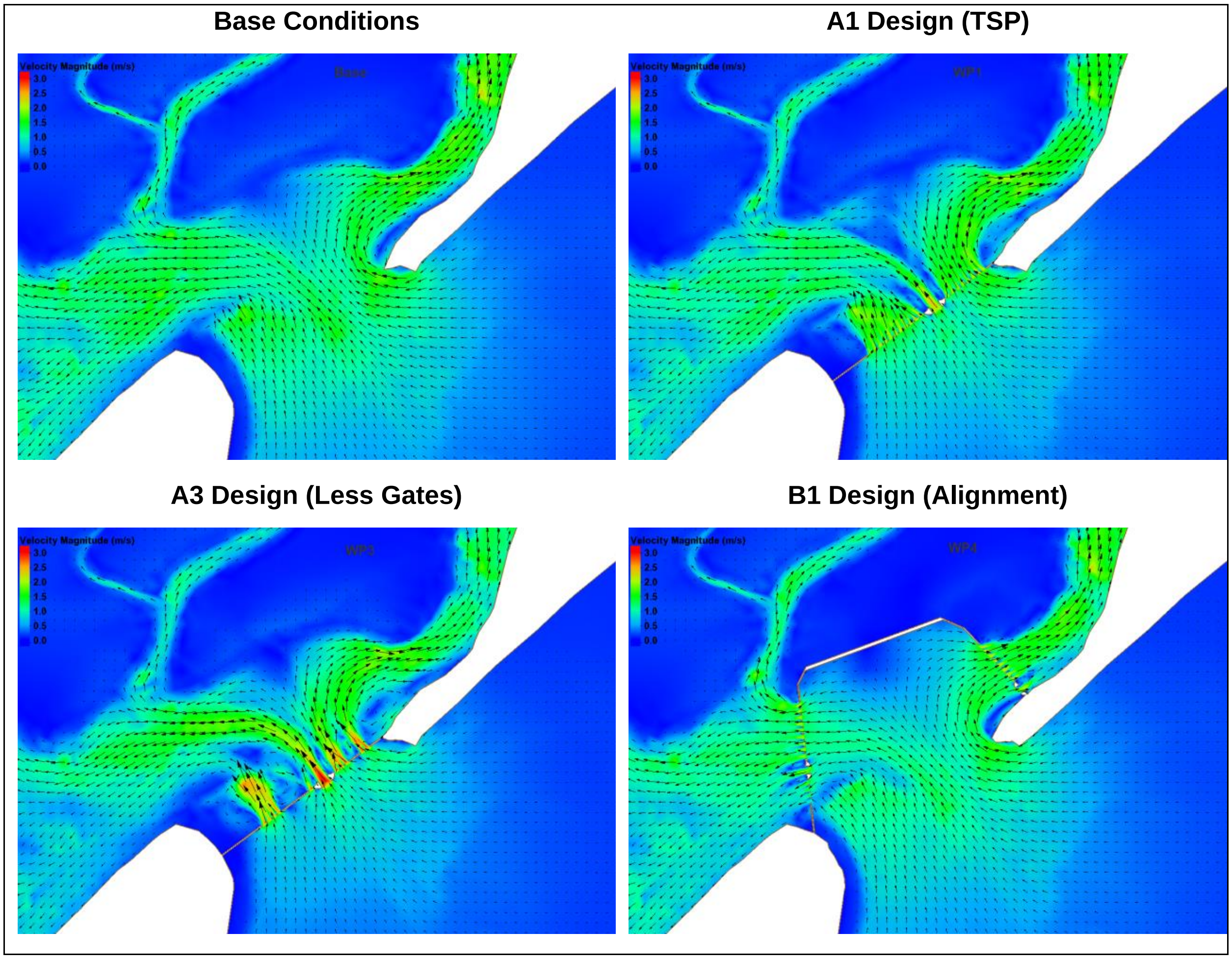


Figure 47: Flood Velocities for Great Egg Inlet Storm Surge Barrier Concepts

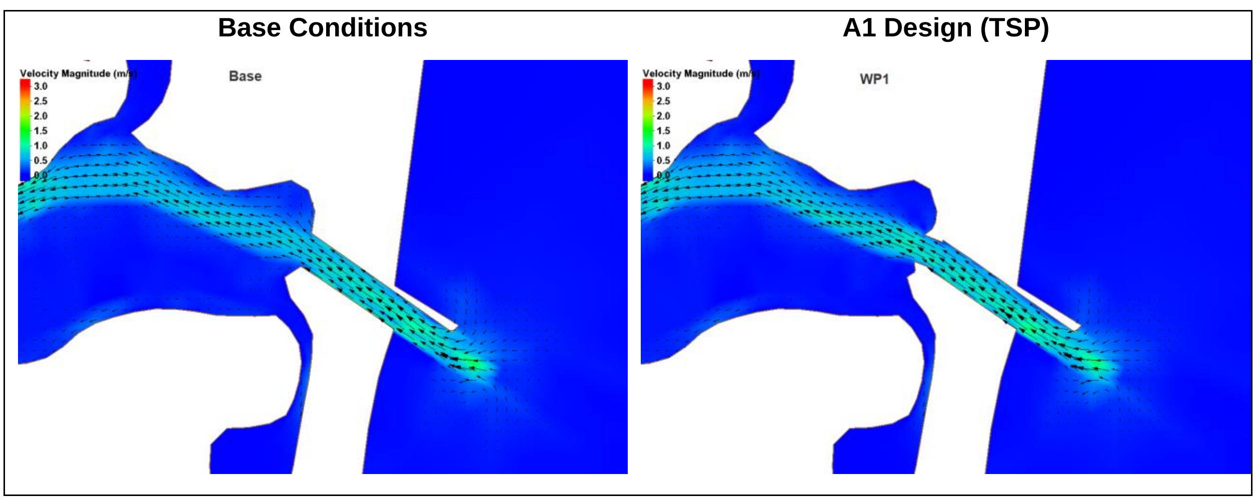


Figure 48: Flood Velocities for Manasquan Inlet Storm Surge Barrier Concept

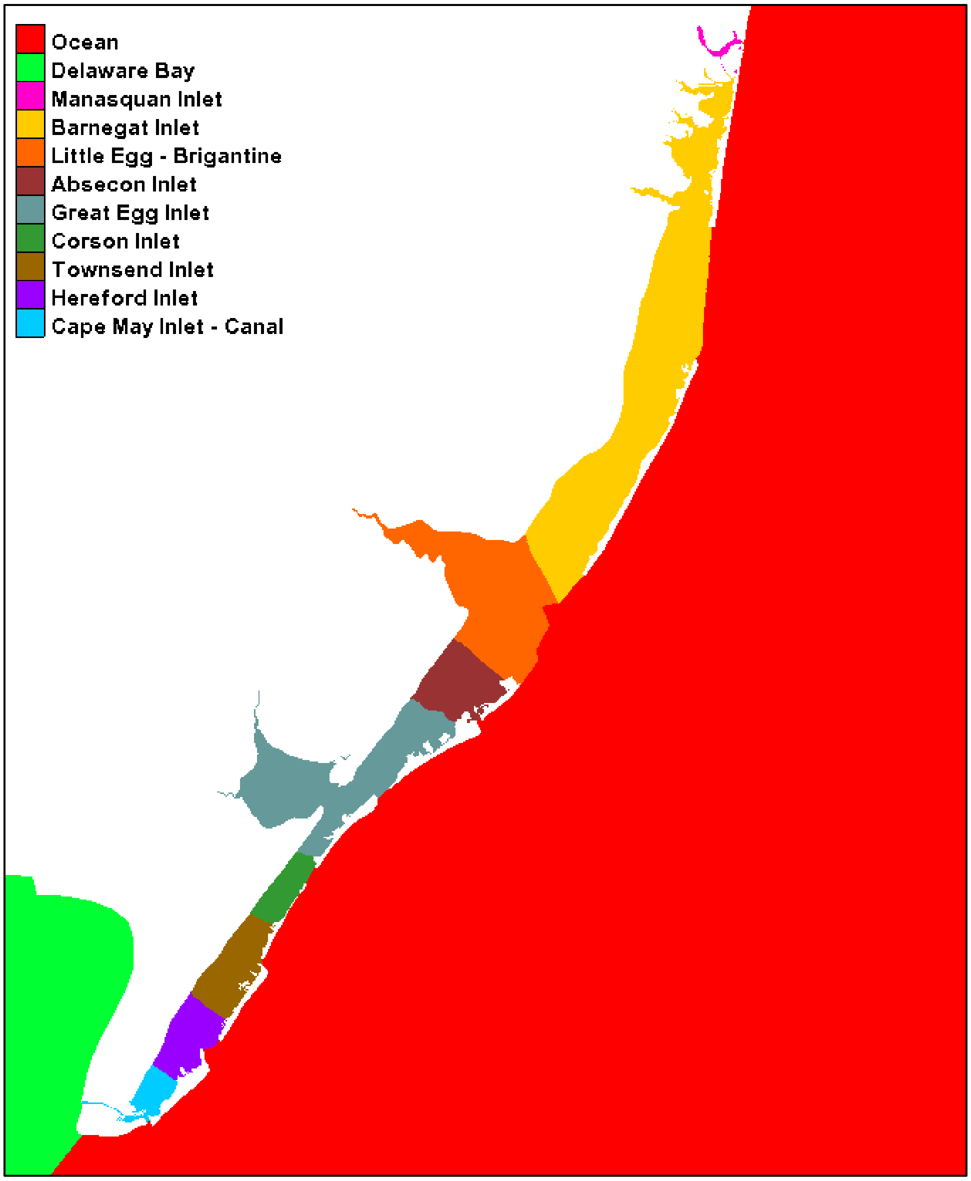


Figure 49: Tidal Prism Areas

One of the other major concerns is the potential impact of storm surge barriers on flushing, residence time, eutrophication, and water quality. Some areas in the study area, such as Barnegat Bay, already suffer from eutrophication, and poor water quality (USGS). Detailed water quality models () and investigations of residence time for Barnegat Bay have already been completed. Defne and Ganju (2014) use a combination of hydrodynamic and particle tracking modeling to identify the mechanisms controlling flushing and residence time in Barnegat Bay. Defne and Ganju (2014) also explain the link between residence time and eutrophication:

> *Estuarine eutrophication is a fundamental consequence of anthropogenic nutrient loading to the coast (Bricker et al. 1999). Typical symptoms include phytoplankton blooms (Paerl 1988), macroalgae proliferation (Valiela et al. 1997), seagrass dieback (Duarte 2002), and hypoxia (Rabalais and Turner 2001). Ultimately, eutrophication impairs the ecological*

function of estuaries in terms of biodiversity, habitat quality, and trophic structure.

One primary physical control on eutrophication is estuarine flushing and ultimately residence time (González et al. 2008), which is defined as the time elapsed until a water parcel leaves a water body through one of its outlets. Estuaries with poor flushing and long residence times tend to retain nutrients within the system leading to high primary productivity rates (Lancelot and Billen 1984). Conversely, well-flushed estuaries are more resilient to nutrient loading due to reduced residence time and greater exchange with less impacted coastal waters.

The AdH hydrodynamic model results were applied to a PTM to evaluate the impact of the storm surge barriers have on residence time in the NJBB study area. Overall, the PTM results showed that the structures had little discernable changes to residence time with modeled differences general within the uncertainty range from innate model randomness caused by diffusion (**Figure 62Error! Reference source not found.**). Model results, **Table 40** and **Table 41**, show that the TSP in general increases in residence time in South and Central regions by 2 to 5 days and actually reduces residence item in North region by 1 to 2 days. Up to now the focus of the AdH and PTM has been on the physical impacts of storm surge barriers during normal conditions when the gates are open. Additional work may be required in the future to assess the impact of the storm surge barrier during storm events.

Table 25: Average Residence Time (Days)

Location	Base	Wp1	Wp2	Wp3	Wp4	Wp5	Wp6
Cape May	10.9	9.9	9.6	11.7	11.7	11.0	12.7
Hereford	25.0	27.0	26.2	28.6	28.6	26.8	25.7
Townsends	36.0	39.9	38.9	39.3	39.3	40.0	25.2
Corson	19.1	24.0	24.2	22.0	22.0	22.6	17.9
Great Egg Harbor	19.6	22.1	22.0	23.0	23.0	22.2	22.9
Absecon Bay	26.2	27.9	25.6	29.1	29.1	29.3	25.5
Little Egg Inlet	20.0	19.1	19.4	19.2	19.2	19.9	23.2
Barnegat Bay	30.5	29.6	29.0	29.6	29.6	29.9	24.8
Manasquan River	29.7	27.4	17.1	28.2	28.2	27.9	21.3

Table 26: Change in Average Residence Time (Days)

Location	Base	Wp1	Wp2	Wp3	Wp4	Wp5	Wp6
Cape May	1.8	-1.0	-1.3	0.8	0.8	0.1	-1.8
Hereford	3.0	2.0	1.3	3.7	3.7	1.9	-0.7
Townsends	4.4	3.9	2.9	3.4	3.4	4.1	10.8
Corson	3.7	4.8	5.1	2.8	2.8	3.4	1.2

Great Egg Harbor	0.7	2.5	2.4	3.4	3.4	2.6	-3.3
Absecon Bay	2.4	1.7	-0.7	2.9	2.9	3.1	0.7
Little Egg Inlet	1.4	-0.9	-0.6	-0.9	-0.9	-0.2	-3.2
Barnegat Bay	0.4	-0.9	-1.5	-0.9	-0.9	-0.6	5.7
Manasquan River	6.8	-2.3	-12.6	-1.5	-1.5	-1.7	8.4

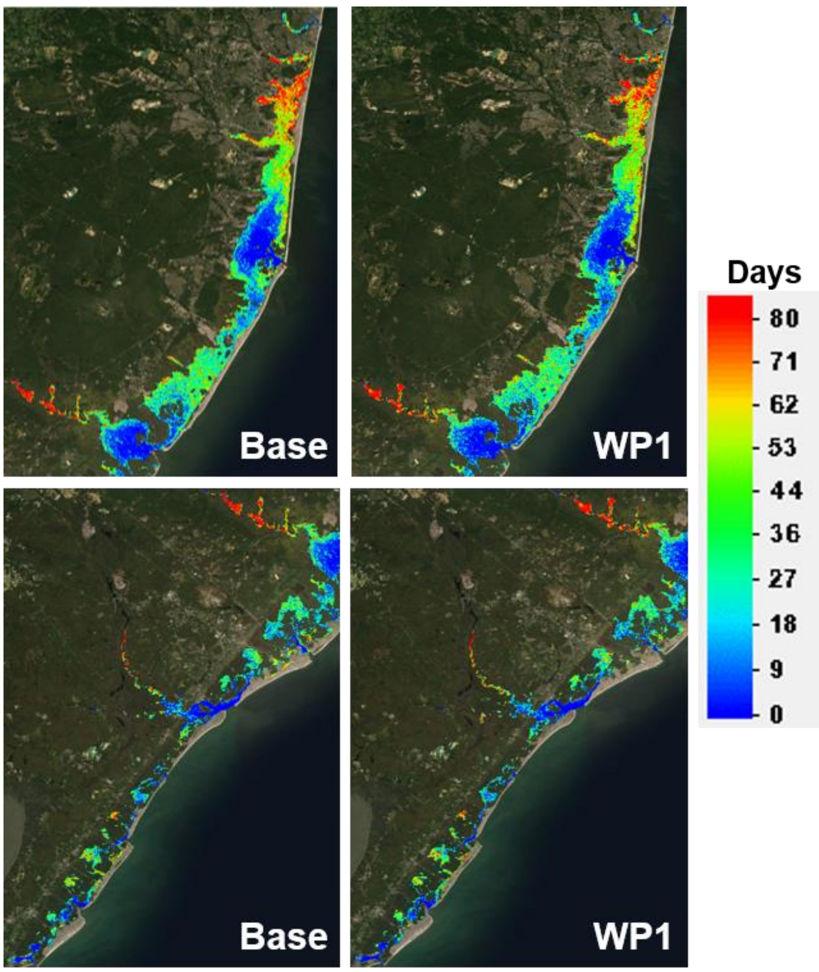


Figure 50: Modeled Residence Time (PTM)

Interior Drainage

Any perimeter plan with-project (WP) conditions implemented in the study area would require upgrades to existing stormwater infrastructure. Given the large study area, and initial phase of screening, detailed assessment for each reach (e.g. determination of runoff, storage, pipe sizing, minimum facilities, pump sizing, etc.) was infeasible. As such, a conservative assumption was made that all necessary stormwater management upgrades would be in the form of pump stations. Following Cycle 1 screening of the perimeter plan, a desktop assessment was performed to estimate the number of pump stations required in each reach of the proposed perimeter plan. This desktop effort focused on reaches determined most feasible in Cycle 1. **Figure 63** depicts a flow chart describing the desktop process developed for this assessment. In general, a distinction was made between areas with existing bulkhead, which currently prohibits stormwater runoff from flow toward the bay (where perimeter plan was assumed to have no impact), and areas without existing bulkhead (where perimeter plan was assumed to have an impact that would be address through installation of pump station(s)). The subsequent process is described in step-wise, bullet-point fashion, below:

- Estimated percentage of existing shoreline that had bulkhead greater than 2ft height above grade, using available aerial photography and existing DEM data
  - Less than 2 ft height categorized as unprotected
- Assumed if existing bulkhead greater than 2ft height => no WP impact anticipated; if existing bulkhead less than 2ft (or unprotected) => WP will have impact, pump stations required
- Determined percentage of assumed impacted shoreline (e.g. length of unprotected shoreline (or less than 2 ft bulkhead) / total shoreline length)
- Obtained drainage area to each reach of perimeter plan
  - Used NJDEP HUC14 watershed boundaries, follow identifiable breakpoint in DEM between drainage to oceanside/bayside
- Applied percentage of assumed impacted shoreline to drainage area
- Assumed pump station required for every 60 acres of adjusted drainage area
  - Based on previous USACE and NJDOT studies (Chelsea Heights FRM Feasibility, NJDOT Seaside Park Route 35 Stormwater Improvements)
- Applied area reduction factor of 50% to any contiguous areas dissimilar to majority of study area (i.e. any areas that were noticeably NOT long and narrow typical of a barrier island), assuming less pump stations would be necessary to treat same land area shaped differently
- Applied reduction factor of 25% globally to account for likelihood that a portion of the identified pump station locations have existing available storage/may not be economically justified
- Calculated additional metrics for back check
  - Above method averages approximately 3 pump stations per municipality
    - ♦ NJDOT Seaside Park Improvements included 3 pump stations for one municipality
  - Above method averages approximately 1,200 ft shoreline spacing between pump stations/outfalls
    - ♦ Oceanside outfall spacing is approximately 1450 ft on average (outfalls on bayside difficult to visually identify)
  - Back checks appear reasonable

Results of the assessment and calculations are shown in **Table 42**. Given the coarse desktop nature of this assessment, it is expected that with additional analysis, including available storage (on streets, open areas, pipe systems), actual increase in flooding/damages, assessment of minimum facilities, etc.; some of the identified pump stations may not be economically justified. As such, this is likely a conservative estimate, appropriate for, and consistent with, this phase of screening.

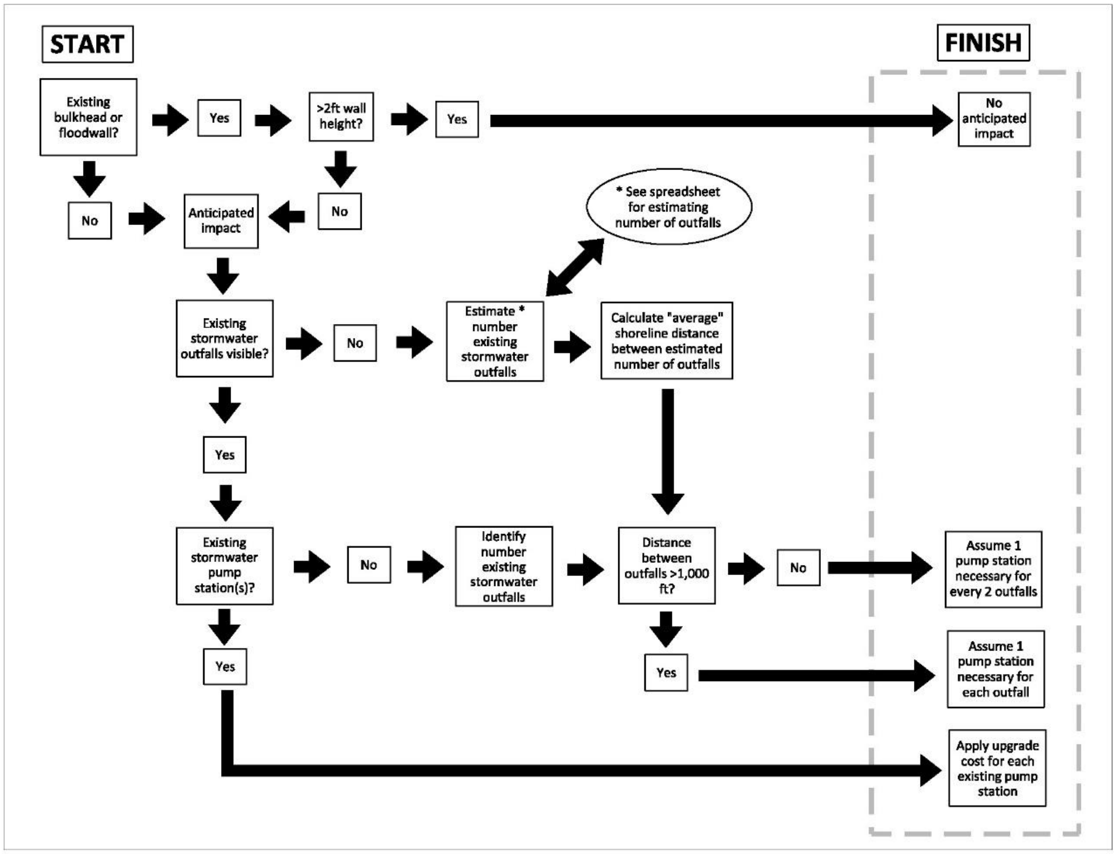


Figure 51: Flow chart for pump station assessment

Table 27: Summary of Estimated Number of Pump Stations by Reach

Group Number	Reach	Total Floodwall Length (ft)	Approx. shoreline Length with Existing Bulkhead > 2ft Height (ft)	Total Baseline Length (ft)	Ratio of Baseline to Total Length	% of Shoreline with Existing Bulkhead > 2 ft	Approx. Drainage Area to BB (ac)	Approx. Drainage Area to BB Factored (ac)	Number of Proposed Pump Stations
1	CM1	15757	0	10000	0.63	0	490	367.5	6
2	LW1	9312	0	30750	0.57	63	1340	1005	6
2	NW1	21841	11150						
2	WCR1	7255	7255						
2	WCY1	15662	15662						
4	WW1	11727	6950	5000	0.43	59	113	84.75	1
5	AV3	50997	36397	33500	0.41	73	892	669	3
5	SH1	30900	23400						
9	AV1	9574	0	3700	0.39	0	56	42	1
10	SI1	34954	9200	20000	0.57	26	473	354.75	4
12	OC1	78573	52000	37100	0.47	66	1807	1355.25	9
18	AC2	43263	30263	44000	0.50	75	3083	2312.25	15
18	LP1	10016	10016						
18	MG1	19953	18953						
18	VN2	14242	6242						
23	BC1	48590	39390	19500	0.40	81	1244	933	3
26	BGL1	12565	0	97000	0.52	45	2301	1725.75	17
26	BV1	21691	13441						
26	HC1	28070	26570						
26	LB1	23056	0						
26	LB3	10349	0						
26	LB4	44084	29074						
26	LB5	17438	3144						
26	SB1	17445	0						
26	SC1	13507	11807						
42	BH1	12786	2878	75000	0.42	9	3581	2067	32
42	BK4	6990	0						
42	BR1	22767	0						
42	LL1	10047	0						
42	LL2	11698	0						
42	MK1	18712	7015						
42	PP2	4471	0						
42	PPB1	10976	0						
42	SSH1	7259	0						
42	SSP1	19253	5988						
42	TR4	15486	0						
42	TR5	38299	0						
45	BL1	7638	0	10500	0.46	0	949.875	949.875	16
45	MQ1	15004	0						
52	GP52	-----	-----	-----	-----	-----	-----	-----	1

Existing Beach/Dune Conditions

A map of existing USACE CSRM projects in New Jersey, **Figure 64Error! Reference source not found.**, shows that nearly the entire Atlantic Ocean facing shoreline, from Cape May to Sandy Hook, is part of an existing USACE CSRM project. The only exception is Island Beach State Park and few sand spits or shorelines adjacent to inlets where there is little infrastructure at risk. Several of the USACE CSRM projects were authorized but unconstructed until Hurricane Sandy in October of 2012. Following Hurricane Sandy, nearly all of the projects have been constructed or are currently under construction.

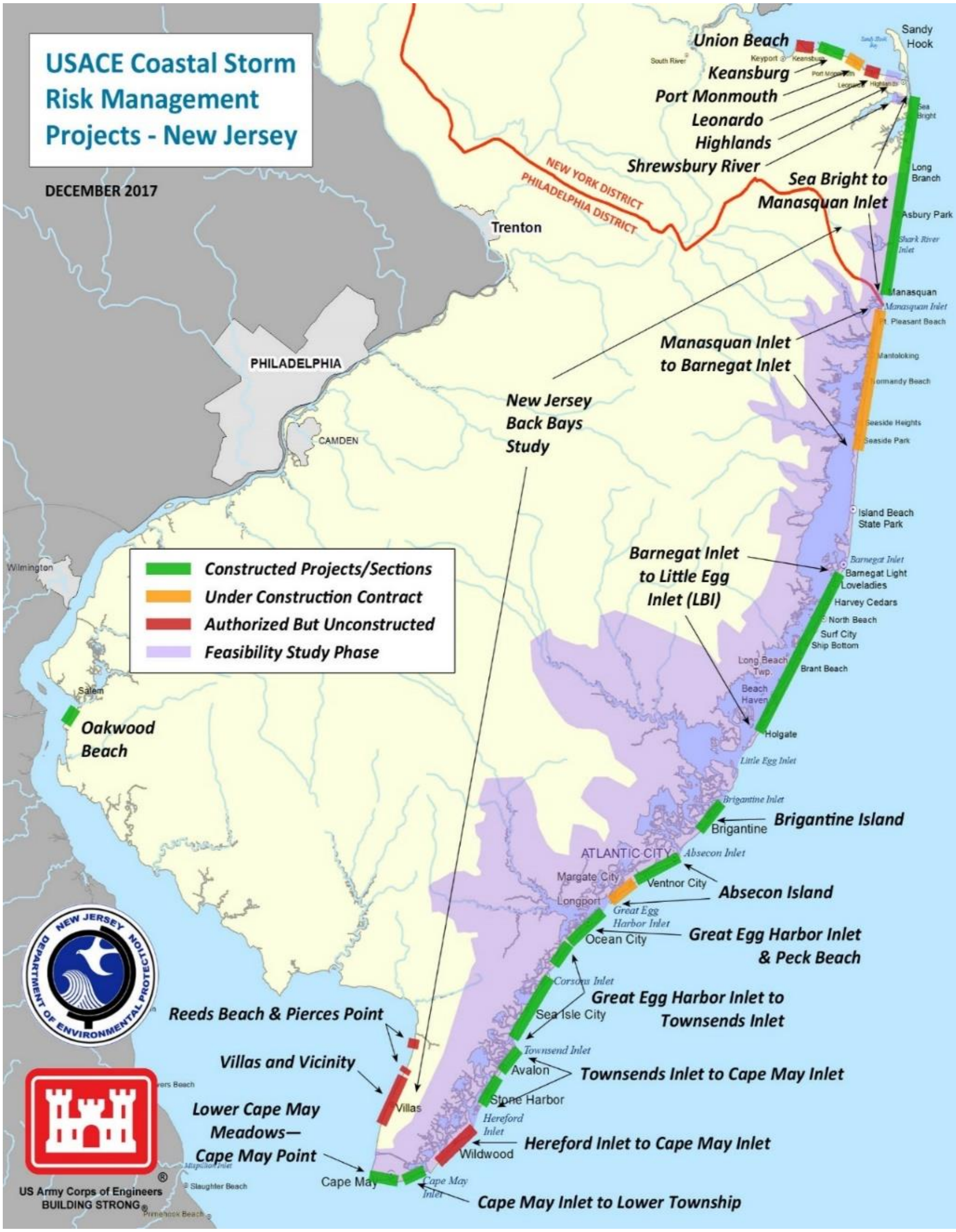


Figure 52: USACE CSRM Projects along Ocean Shorelines

Feasibility studies for each of the USACE CSRM projects were completed independently of each other and determined design dune and berm conditions by optimizing NED benefits within each respective study area. Due to unique nature of each study area the optimization resulted in variability in the design dune dimensions up and down the coast. There is even variability in the design dune heights in some of the projects and two projects don't have an authorized dune as part of the project. A summary of the existing USACE-CSRM projects authorized design dune/seawall heights is provided in **Table 43**. These studies optimized the dune and berm dimensions with the understanding that back-bay flooding could still occur during storm events, thus limiting the potential flood inundation benefits provided by dunes along the ocean. Therefore, it is possible that the risk of back-bay flooding constrained the optimized dune heights in some studies.

Table 28: Existing USACE CSRM Projects in Study Area

Project	Location	Authorized Crest Elevation (ft, NAVD88)
Manasquan Inlet to Barnegat Inlet	Northern Point Pleasant Beach and Seaside Heights	18
Manasquan Inlet to Barnegat Inlet	Rest of Project Area	22
Barnegat Inlet to Little Egg Inlet	Long Beach Island	22
Brigantine Island	Brigantine Island	10
Absecon Island	Absecon Seawall	16
Absecon Island	Atlantic City	14.75
Absecon Island	Ventnor, Margate, Longport	12.75
Great Egg Harbor Inlet & Peck Beach	Ocean City - North	n/a
Great Egg Harbor Inlet to Townsends Inlet	Ocean City - South	12.8
Great Egg Harbor Inlet to Townsends Inlet	Strathmere and Sea Isle City	14.8
Townsends Inlet to Cape May Inlet	Townsends Seawall	11.7
Townsends Inlet to Cape May Inlet	Avalon	14.75
Townsends Inlet to Cape May Inlet	Stone Harbor	14.75
Townsends Inlet to Cape May Inlet	Hereford Seawall	11.7
Hereford Inlet to Cape May Inlet	Wildwood	16
Cape May Inlet to Lower Township	Cape May	n/a
Lower Cape May Meadows	Cape May Meadows	16.75

Note: Grey-shaded rows are Seawalls, not dunes

Philadelphia District coastal engineers and coastal planners, familiar with the existing USACE CSRM projects, got together to discuss how these existing projects would mesh with the NJBB CSRM alternatives. Since the beginning of the NJBB study there have been questions about whether the existing USACE CSRM projects dunes are robust and reliable enough to be part of NJBB storm surge barrier alternative or bay shoreline floodwall alternative (i.e. perimeter plan). The purpose of the meeting was to discuss the complexities of answering this question and identifying a path forward for evaluating the interaction between the ocean dunes and NJBB alternatives.

During the meeting it was pointed out that it is unlikely that a storm surge barrier alternative would need to maintain an uninterrupted line of impregnable dunes along the shoreline. Dune erosion and overtopping would allow more water into the bay and increase bay water levels; however, it is not an “all or nothing situation” where any dune failure would completely negate the benefits of the storm surge barriers. It was also noted during the meeting that ocean shoreline is exposed to significantly larger waves than the bay and therefore design crest elevations for CSRM measures along the bay are likely to be lower than ocean for the same design level.

Another important discussion during the meeting was that the existing CSRM projects along the ocean may provide a practical upper limit to the design level on NJBB bay alternatives. If a NJBB alternative did require modifications to the existing CSRM projects, such as higher dunes, the cost associated with these modifications would extend well beyond the additional sand required to construct the dune. Increasing the dune height would increase the footprint of the dune and push the design profile further seaward, increasing fill quantities and periodic nourishment quantities/frequency. In some erosion hot spots, it may be difficult to maintain the expanded design profile between periodic nourishment operations. Modifying the dune height may also require obtaining new easements, since the existing easements are based on specific dune crest elevation. Despite these complexities, it was noted during the meeting that an evaluation would need to be completed to determine if costly dune modifications would be offset by a reduction in damages and still be part of an optimized NED plan.

The path forward identified during the meeting was to first get a better understanding of the sensitivity of back-bay water levels to the dune conditions and the performance of the NJBB alternatives without any modifications to the existing USACE CSRM projects. To complete this analyses ADCIRC simulation will be completed for three dune conditions: (1) Existing/authorized dune heights, (2) Partially eroded, 50% of dune height removed, and (3) No dune. The ADCIRC simulations will be performed for a small subset of representative storms.

The second step is to improve our understanding of how likely the existing USACE CSRM projects are to become eroded during storm events. This will be accomplished by running SBEACH simulations for the existing/authorized dune heights for a small subset of representative storms.

The third step, if necessary, is to develop designs and cost estimates for modifications to the existing USACE CSRM projects.

An evaluation of the ADCIRC and SBEACH modeling and cost evaluation of potential modifications to existing USACE CSRM projects will completed prior to the release of the Final Report.

References

Cialone, M. A., T. C. Massey, M. E. Anderson, A. S. Grzegorzewski, R. E. Jensen, A. Cialone, D. J. Mark, K. C. Pevey, B. L. Gunkel, T. O. McAlpin, N. C. Nadal-Caraballo, J. A. Melby, J. J. Ratcliff, 2015. North Atlantic Coast Comprehensive Study (NACCS) Coastal Storm Model Simulations: Waves and Water Levels. ERDC/CHL TR-15-14. Vicksburg, MS: U.S. Army Engineer Research and Development Center.ECB 2013-33

Dalrymple, Robert A. (1992). *Water Wave Propagation in Jettied Channels*. Coastal Engineering, Chapter 233.

Defne, Zafer and Ganju, Neil K. (2014) Quantifying the residence time and flushing characteristics of a shallow, back-barrier estuary: Application of hydrodynamic and particle tracking models. Estuaries and Coasts. Volume 38, Issue 5. Pages 1719-1734.

EM 1110-2-1100. *Coastal Engineering Manual*. Department of the Army, U.S. Army Corps of Engineers, Washington, DC 20314-1000.

EM 1110-2-6056. *Standards and Procedures for Referencing Project Elevation Grades to Nationwide Vertical Datums*. December 31, 2010. Department of the Army, USACE, Washington DC.

ER 1110-2-8160. *Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums*. March 1, 2009. Department of the Army, USACE, Washington DC.

ER 1100-2-8162. *Incorporating Sea Level Change in Civil Works Programs*. December 31, 2013. Department of the Army, USACE, Washington DC.

EurOtop, 2016. *Manual on wave overtopping of sea defenses and related structures. An overtopping manual largely based on European research, but for worldwide application*. Van der Meer, J.W., Allsop, N.W.H., Bruce, T., De Rouck, J., Kortenhaus, A., Pullen, T., Schüttrumpf, H., Troch, P. and Zanuttigh, B., www.overtopping-manual.com.

Franco, C. & Franco, L. 1999 *Overtopping formulae for caisson breakwaters with non-breaking 3-d waves*. Journal of Waterway, Port, Coastal & Ocean Engineering, Vol 125, No 2, pp 98-107, ASCE, New York.

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

Kopp, R.E., A. Broccoli, B. Horton, D. Kreeger, R. Leichenko, J.A. Miller, J.K. Miller, P. Orton, A. Parris, D. Robinson, C.P.Weaver, M. Campo, M. Kaplan, M. Buchanan, J. Herb, L. Auermuller and C. Andrews. 2016. *Assessing New Jersey's Exposure to Sea-Level Rise and Coastal Storms: Report of the New Jersey Climate Adaptation Alliance Science and Technical Advisory Panel*. Prepared for the New Jersey Climate Adaptation Alliance. New Brunswick, New Jersey.

Kopp, R.E., C. Andrews, A. Broccoli, A. Garner, D. Kreeger, R. Leichenko, N. Lin, C. Little, J.A. Miller, J.K. Miller, K.G. Miller, R. Moss, P. Orton, A. Parris, D. Robinson, W. Sweet, J. Walker, C.P. Weaver, K. White, M. Campo, M. Kaplan, J. Herb, and L. Auermuller. *New Jersey's Rising Seas and Changing Coastal Storms: Report of the 2019 Science and Technical Advisory Panel*. Rutgers, The State University of New Jersey. Prepared for the New Jersey Department of Environmental Protection. Trenton, New Jersey.

Lackey T., Mays N., McAlpin J., and Kim S, 2020. *DRAFT: Residence Time Analysis to Predict*

Impact of Proposed Storm Protection Structures in New Jersey Back Bays (NJBB). ERDC/CHL TR-20-X. Vicksburg, MS: U.S. Army Engineer Research and Development Center.

McAlpin J., and Ross C., 2020. *DRAFT: Analysis of Proposed Storm Protection Structures on the Hydrodynamics and Salinity in New Jersey Back Bays (NJBB)*. ERDC/CHL TR-20-X. Vicksburg, MS: U.S. Army Engineer Research and Development Center.

Nadal-Caraballo, Norberto C. and Melby Jeffrey A. (2014). *North Atlantic Coast Comprehensive Study Phase I: Statistical Analysis of Historical Extreme Water Levels with Sea Level Change.* ERDC/CEHL TR-14-7, Coastal and Hydraulics Laboratory, U.S. Army Engineer and Development Center, 3090 Halls Ferry Road, Vicksburg, MS.

Nadal-Caraballo, N. C., J. A. Melby, V. M. Gonzalez, and A. T. Cox, 2015. *North Atlantic Coast Comprehensive Study (NACCS) Coastal Storm Hazards from Virginia to Maine*. ERDC/CHL TR-15-5. Vicksburg, MS: U.S. Army Engineer Research and Development Center.

New Jersey Climate Adaption Alliance, http://njadapt.rutgers.edu/

NOAA VDATUM, https://vdatum.noaa.gov/about.html

NWS Flood Stages, https://water.weather.gov/ahps/region.php?state=nj

Orton P., Fernald S., Marcell K., Brooks B., van Prooijen B., and Chen Z. 2019. Surge Barrier Environmental Effects and Empirical Experience Workshop Report.

Slusarcyzk G., Cialone M. A., Nadal-Caraballo N. C., and Hampson R. W., 2020. *DRAFT: Storm Surge Comparison for Proposed New Jersey Back Bays Inlet Closures*. ERDC/CHL TR-20-X. Vicksburg, MS: U.S. Army Engineer Research and Development Center.

Sweet, W. V., Dusek, G., Obeysekera, J., Marra, J. J., 2018. Patterns and Projections of High Tide Flooding Along the U.S. Coastline Using a Common Impact Threshold. NOAA Technical Report NOS CO-OPS 086. Silver Spring, Maryland.

USACE-NAO, 2017. *City of Norfolk Coastal Storm Risk Management Study, Hydraulics, Hydrology, & Coastal Sub-Appendix*. Norfolk Virginia, October 2017.

Ward., D. L., Ahrens, J. P., 1992. Overtopping Rates for Seawalls. Miscellaneous Paper CERC-92-3. Department of the Army. Waterways Experiment Stations, Corps of Engineers, 3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199.

Yang Z., Myers E. P., Wong A. M., and White S. A., 2008. *VDATUM for Chesapeake Bay, Delaware Bay and Adjacent Coastal Water*

ENGINEERING APPENDIX
COST

NEW JERSEY BACK BAYS
COASTAL STORM RISK MANAGEMENT
FEASIBILITY STUDY

PHILADELPHIA, PENNSYLVANIA

APPENDIX B.5

August 2021

U.S. Army Corps of Engineers
Philadelphia District

THIS PAGE HAS BEEN INTENTIONALLY LEFT BLANK

Table of Contents

List of Tables

NEW JERSEY BACK BAY COASTAL STORM RISK MANAGEMENT COST NARRATIVE

Scope of Work:

The 70-year storm risk management plan, the Tentatively Selected Plan (TSP) for the NJBB Coastal Storm Risk Management Project includes the following civil works feature accounts:

- Account 01 - Lands and Damages: For both structural and nonstructural features of work, real estate costs due to construction impacts are assessed and provided by NAB Real Estate Division and are shown in Table 2 for the TPCS.
- Account 02 - Relocations: It is anticipated that existing boat docks would need to be replaced during construction of Type B T-walls and Type D King Pile Combined w/ Steel Sheetpile (Combi) walls, which are built from the water side. Fifty percent of the Type B and Type D wall lengths were used as a placeholder based on recent site visits and engineering judgment. Utility relocation costs were not included in the cost estimate since existing utility data was not obtained from the local utility companies. Utility relocation costs will be included in the next phase of this study.
- Account 06 – Fish and Wildlife Facilities: The proposed project plan shows Elements of Measures that include floodwalls and levees for multiple areas, three storm surge barriers (SSB) at Manasquan Inlet, Barnegat Inlet and Great Egg Harbor Inlet and two bay closures at South Ocean City and Absecon Boulevard. Environmental mitigation costs were provided for each of these elements by NAP District environmental PDT Member. Environmental mitigation includes work such as creation of tidal open water sub-tidal (shellfish), SAV beds, tidal open water hardened shoreline, intertidal rocky shore line, intertidal mudflat, intertidal beach, saline low and high marshes, scrub shrub deciduous and coniferous, wetlands and forested areas.
- Account 10 – Breakwaters and Seawalls: The proposed project alignment shows Elements of Measures that include three SSBs at Manasquan Inlet, Barnegat Inlet and Great Egg Harbor Inlet and two bay closures at South Ocean City and Absecon Boulevard. Lengths and heights of SSBs were provided for each of these elements by NAP District civil engineer PDT Member. Preliminary quantity take-offs for the SSBs and bay closures were conservatively estimated based on the Google Earth kmz aerial mapping files and the proposed lengths for SSBs and bay closures, assuming averaged elevation of the project alignment will be the same as the constant desired height for the proposed SSB or bay closure. All costs in connection with construction work for SSBs and bay closures were estimated using cost model parametric equations, documented in Kluijver et al. (2019), including construction durations calculated using schedule model parametric equations for construction scheduling of SSBs and bay closures. In addition, the concerns for Environmental Mitigation and Natural and Nature Based Features (NNBF) are included in the costs shown in Account 06.
- Account 11 – Levees and Floodwalls: The proposed project alignment shows Elements of Measures that include walls and levee construction for multiple areas. Floodwalls consisting of concrete T-wall and Combi walls were used. Combi wall design includes W40 x 249 king piles combined with PZC18 steel sheet piles with reinforced concrete pile cap. Earthen levees consisting of fill material placed on both sides of steel sheet piling, covered with riprap were also used. Length of wall and levees and draft detail drawings for the

walls and levees were provided by Philadelphia District structural engineer PDT member. Preliminary quantity take-offs for the wall and levee were conservatively estimated based on the detail drawings and the proposed lengths for wall and levee, assuming averaged elevation of the project alignment will be the same as the constant desired height for the proposed wall and levee. In cases of a wall or levee near a body of water, water diversion such as drainage pipe with flap gates were added to the MII estimate. Seismic monitoring of structures will also be required for driving of new steel sheeting and removal of existing or temporary sheeting and is estimated with work being done by a subcontractor based on similar previous projects at NAP. Street intersections in the vicinity of the work will need traffic control consisting of new traffic signals, vehicle barriers, traffic signs and flag person. All costs in connection with construction work for floodwalls and levees were estimated in MII using MII software, Cost Book Library 2016 edition, R.S. Means 2019 edition, and historic cost data. In addition, the concerns for Environmental Mitigation and Natural and Nature Based Features (NNBF) are included in the costs shown in Account 06.

- Account 13 – Pumping Plant: The NAP preliminary estimate for a 100 cfs pump station at East Creek, Raritan & Sandy Hook FRM Project Monmouth County, NJ with three 250 H.P. pumps (14,900 gpm) was used to estimate pump station costs for the areas within the project alignment . The size of concrete sump chamber, sluice gates, pipes, electrical and other appropriate items are used to accommodate the number of pumps. All costs in connection with construction work for pump stations were estimated in MII using MII software, Cost Book Library 2016 edition, R.S. Means 2019 edition, and historic cost data.
- Account 15 – Floodway Control and Diversion Structures: In several areas where the project alignment goes thru a body of water, a tidal barrier which includes box culvert, miter gates and sluice gates were parametrically estimated based on the size adjustments and historical costs from City of Norfolk Coastal Storm Risk Management Study, Norfolk, VA. Similarly, on the land side, there are street roller gates in some areas where the alignment intersects streets that were also parametrically estimated based on the previously mentioned study. The costs for all these gates are escalated by using escalation factors from CWCCIS dated 30 Sep 2019 to bring historical costs to the current price level.
- Account 18 – Cultural Resource Preservation: The proposed project alignment has potential impacts on cultural resources that may require extensive archaeological mitigations. Since no surveys were done, areas that are currently considered as significant sites may potentially have extensive impacts or none at all. A conservative approach was taken to count as if most sites are high probability sites and will have substantial archaeological mitigations. The cost for archaeological mitigation was conservatively estimated and provided by NAP District cultural resources PDT Member.
- Account 19 – Buildings, Grounds and Utilities: The proposed project alignment shows Elements of Measures that include Non Structural flood mitigation consisting of raising and wet flood proofing of existing structures. The cost for non-structural flood mitigation was conservatively estimated and provided by NAP District flood plain mapping PDT Member. All costs in connection with construction work for non-structural flood mitigation were estimated in MII using MII software, Cost Book Library 2016 edition, R.S. Means 2019 edition, and historic cost data.

- Account 30 – Preconstruction Engineering & Design (PE&D): PE&D costs include local cooperative agreements, environmental and regulatory activities, general design memorandum, preparation of plans and specifications, engineering during construction, A/E liability actions, cost engineering, construction and supply contract award activities, project management, and the development of the PCA. PE&D costs were estimated as unit costs for Measures 2A, 3E(2), 4G(8) and 5A based on similar projects constructed by NAP. A contingency factor of 33% was included in the PE&D costs.
- Account 31 – Construction Management (S&A): Construction Management costs include contract administration, review of shop drawings, inspection and quality assurance, project office operation, contractor initiated claims and litigations, and government initiated claims and litigations. S&A related costs were estimated as lump sum amount for construction staffing requirements using on a 240 month construction duration based on similar projects constructed by NAP. A contingency factor of 33% was included in the S&A costs.

Construction Cost Estimate:

The following methodology is used in the preparation of the cost estimate for New Jersey Back Bays Coastal Storm Risk Management Study:

a. The estimate is in accordance with the guidance contained in ER 1110-2-1302.

b. The estimate is presented in Civil Works Work Breakdown Structure.

c. The price level for the TSP estimate is in 4th Quarter of FY2019.

d. Construction costs developed by Cost Engineering Section, Engineering Division, Philadelphia District are based on a concept design developed by NAP Engineering team. Unit costs are developed using the MCACES Second Generation (MII) software containing the 2019 English Cost Book Library which was used as a starting point. Historical cost data from similar projects are used for parametric estimate, and vendor quotes were used for non-Cost Book data. The estimate is documented with notes to explain the assumed construction methods, crews, productivity, and other specific information. The intent is to provide or convey a “fair and reasonable” estimate that which depicts the local market conditions.

e. Labor costs are based on Davis Bacon wage rates for Ocean County, NJ.

f. Bid competition: No contracting plan is done at this point. Bidding competition is assumed to be unrestricted since the overall work is typical to the area and the massive size of the project will likely draw multiple national level large size contractors to bid on the project. This assessment is reflected in the Cost and Schedule Risk Analysis.

g. Contract Acquisition Strategy: Acquisition strategy is not yet determined at this point. However to reflect the historical market condition for this type of work, Prime Contractor is assumed to perform earth work and concrete placement and will sub-contract out most remaining work.

h. Labor Shortages: It is assumed that there will be a normal labor market.

i. Materials: Most material costs are from the Cost Book Library. Vendor quotes were used for non-Cost Book items such as silt curtain, Aqua Barrier and Portadam rent costs. Assumptions include:

1. Rent materials will be part of the construction contract. No government furnished materials are assumed. Quoted delivery charge is used for hauling cost.
2. Materials will be rented from local nearest available sources.
3. Hauling: most hauling will be done by trucks. For trucking, it is assumed that the average speed is 30 mph factoring traffic hours in often congested major routes.

j. Equipment: Rates used are based from the latest USACE EP-1110-1-8, Region I. Adjustments are made for fuel and facility capital cost of money (FCCM). Judicious use of owned verses rental rates was considered based on typical contractor usage and local equipment availability. Full FCCM/Cost of Money rate is latest available; MII program takes EP recommended discount, no other adjustments have been made to the FCCM.

k. Fuels (gasoline, on and off-road diesel) were based on local market averages for on-road and off-road fuels in Atlantic County, NJ. Since fuels fluctuate irrationally, an average was used.

l. Major crew and productivity rates were developed and studied by senior USACE estimators familiar with the type of work. All of the work is typical to the Philadelphia District. The crews and productivities were checked by local NAP estimators and comparisons with historical cost data. Major crews include steel erection, hauling, concrete, stonework, and planting.

m. All crew work hours are assumed to be 8 hrs. 5 days/week which is typical to the area. It is anticipated that no overtime is required for reasons such as noise and night lighting bans due to close proximity to existing residences and inability to work from the water side during hours of darkness.

n. Mobilization and demobilization: Contractor mobilization and demobilization are based on the assumption that most of the contractors will take at least one month to mobilize and one month to demobilize. Contractors located within 500 miles from the project site using readily available, off-the-shelf construction equipment would do the work. Construction access would be by local streets and from the water side using a nearby dock as a land to water transfer point for material. Mob and demob cost is estimated at 3% of total construction costs based on the North Atlantic Comprehensive Coastal Study (NACCS).

o. Field Office Overhead: Typically civil works project has field office overhead ranging from 9% to 11%. 10% was used for Prime Contractor Job Office Overhead. Overhead assumptions may include: Superintendent, office manager, pickups, periodic travel, costs, communications, temporary offices (contractor and government), office furniture, office supplies, computers and software, as-built drawings and minor designs, tool trailers, staging setup, camp and kitchen maintenance and utilities, utility service, toilets, safety equipment, security and fencing, small hand and power tools, project signs, traffic control, surveys, temp fuel tank station, generators, compressors, lighting, and minor miscellaneous.

p. Home Office Overhead: 10% was used for HOOH based upon estimating and negotiating experience, and consultation with local construction representatives.

q. Profit: Since the Construction Cost Estimate is currently in a budgetary phase, profit is typically included at 10% for Prime Contractor. However, due to large size of project and general expectation that there will be some competition, 6% profit was used for Prime and Prime's Profit on Sub's work. Sub-contractors' profit is mostly 8.5%.

r. Sales Tax: Only State sales tax was applied. No local sales tax was included in the estimate.

s. Bond: Bond is calculated at 1.0% based upon estimating and negotiating experience, and consultation with local construction representatives.

t. Contingency: The estimated cost for each major subdivision or feature of the tentatively recommended project includes an item for "contingencies". The contingency allowances used in the development of the cost estimate for the tentatively selected project were estimated as an appropriate percentage using Crystal Ball software for preparing risk analysis. Thirty three percent was applied to the work to account for concerns about the level of design, weather delays, available funding available from the Sponsor, and environmental mitigation requirements.

u. Escalation: No escalation to midpoint of construction according to tentative construction start dates is included in the MII estimate and non-MII estimates provided by NAP. Escalation will only be included in the Total Project Cost Summary (TPCS) to avoid duplication.

v. HTRW: Contaminated material for Hazardous, Toxic, and Radioactive Waste (HTRW) was not included in the estimate since HTRW contamination is expected to be localized to older structures containing lead paint, asbestos or storage tanks for heating oil.

w. Monitoring Costs: Monitoring costs include coastal, bay side and environmental monitoring during initial construction and post construction. Monitoring costs are included in the PE&D amount.

x. Adaptive Management Costs: Adaptive management costs include coastal, bay side and environmental adaptive management during initial construction post construction. Adaptive management costs are included in the PE&D amount.

y. Operation, Maintenance Repair Replacement and Rehabilitation (OMRR&R) Costs: Total annualized OMRR&R costs are $x,xxxx,xxx,xxx.

Plan Formulation for the TSP:

A cycled iterative approach was used during plan formulation to determine the TSP:

Cycle 0 was a qualitative exercise where the PDT "screened out" areas for perimeter measures because they had near zero damageable structures. No cost, no benefits.

Cycle 1 was the quantitative analysis of all the perimeter measures for mainland and barrier island (1% design level).

Cycle 2 (a and b) (Dec 2018) was the further quantitative analysis of the potentially economically viable sites (all barrier islands).

- Alternatives: 50 to 20
- Design: Iterations of earthen berm, concrete T-wall constructed from land side and concrete T-wall constructed from water side; level of design = 5%.
- Cost update: parametric cost based on R.S. Means, historic data and MII cost book.
- SSB: 7 barriers screened out.
- PP: Long Beach Island, Island Beach and Strathmere screened out.

Cycle 3 (a, b and c) (Oct 2019/ Dec 2019/ Jan 2020) was the further quantitative analysis of the incrementally justified sites.

- Alternatives: 20 to 8 to TSP
- Design: Risk-based analysis including 16 ft and 13 ft berm and wall heights. Added Combi wall design and improved design for berm and concrete walls; level of design = 15%.
- Cost update: berm and wall costs in MII. Improved parametric cost formula for the SSBs.
- SSB: Absecon Inlet screened out.
- PP: Still warrants further investigation.

For additional information regarding plan formulation, see Appendix A Plan Formulation describing the plan formulation for the TSP.

Total First Cost for the TSP:

Initial construction costs are based on a Dec 2019 price level and a 240 month construction duration. For more information, refer to the Main Report describing the TSP. Initial construction costs are shown in Table 1.

Table 1: Total First Cost TSP

CWWBS		FEATURE OF WORK	Price Level: December 2019 Construction duration: 240 months			Price Level: October 2020		
			CONTRACT COST	CONTINGENCY (*)	TOTAL	CONTRACT COST	CONTINGENCY	TOTAL
01.		**LANDS AND DAMAGES**	$656,553,350	$216,662,606	$873,215,956	$680,939,554	$224,710,053	$905,649,607
02.		**RELOCATIONS**						
	01.	Mob, Demob & Prep Work	$108,379	$35,765	$144,144	$115,131	$37,993	$153,125
	02.	Relocations	$3,612,622	$1,192,165	$4,804,787	$3,837,708	$1,266,444	$5,104,151
06.		**FISH AND WILDLIFE FACILITIES**	$282,096,199	$93,091,746	$375,187,945	$295,630,972	$97,558,221	$393,189,193
10.		**BREAKWATERS AND SEAWALLS**						
	01.	Mob, Demob & Prep Work - Included in LS amounts.						
	02.	Barnegat Inlet SSB	$1,300,769,625	$429,253,976	$1,730,023,601	$1,324,039,747	$436,933,116	$1,760,972,863
	03.	Manasquan Inlet SSB	$517,079,406	$170,636,204	$687,715,610	$526,329,699	$173,688,801	$700,018,500
	04.	Great Egg Harbor Inlet SSB	$1,875,759,528	$619,000,644	$2,494,760,172	$1,909,315,933	$630,074,258	$2,539,390,191
	05.	South Ocean City Bay Closure SSB	$115,345,619	$38,064,054	$153,409,673	$117,409,095	$38,745,001	$156,154,096
	06.	Absecon Blvd Bay Closure	$190,011,601	$62,703,828	$252,715,429	$193,410,814	$63,825,569	$257,236,383
11.		**LEVEES AND FLOODWALLS**						
	01.	Mob, Demob & Prep Work	$21,882,988	$7,221,386	$29,104,374	$22,456,198	$7,410,545	$29,866,743
	02.	Levees	$288,130,457	$95,083,051	$383,213,508	$295,677,832	$97,573,684	$393,251,516
	03.	Floodwalls: T-wall and Combi Wall	$436,922,119	$144,184,299	$581,106,418	$448,366,986	$147,961,105	$596,328,091
	04.	Associated General Items	$2,059,528	$679,644	$2,739,172	$2,113,476	$697,447	$2,810,923
13.		**PUMPIMG PLANT**						
	01.	Mob, Demob & Prep Work	$434,916	$143,522	$578,438	$456,139	$150,526	$606,665
	02.	Pump Station	$14,497,200	$4,784,076	$19,281,276	$15,204,648	$5,017,534	$20,222,182
15.		**FLOODWAY CONTROL AND DIVERSION STRUCTURES**						
	01.	Mob, Demob & Prep Work	$5,267,037	$1,738,122	$7,005,159	$5,519,745	$1,821,516	$7,341,261
	02.	Gates, Stop Logs and Associated Structur	$175,567,884	$57,937,402	$233,505,286	$183,991,505	$60,717,197	$244,708,702
18.		**CULTURAL RESOURCE PRESERVATION**	$70,013,523	$23,104,463	$93,117,986	$73,430,110	$24,231,936	$97,662,046
19.		**BUILDINGS, GROUNDS, AND UTILITIES**						
	01.	Mob, Demob & Prep Work (costs included in unit prices)						
	02.	Non Structural, Buildings	$3,592,523,010	$1,185,532,593	$4,778,055,603	$3,767,545,374	$1,243,289,973	$5,010,835,348
		CONSTRUCTION ESTIMATE TOTALS	**$8,892,081,640**	**$2,934,386,941**	**$11,826,468,581**	**$9,184,851,113**	**$3,031,000,867**	**$12,215,851,980**
30.		**PLANNINNG, ENGINEERING AND DESIGN (P,E & D)**	$566,950,000	$187,093,500	$754,043,500	$575,454,250	$189,899,903	$765,354,153
31.		**CONSTRUCTION MANAGEMENT (S & A)**	$181,415,900	$59,867,247	$241,283,147	$184,137,136	$60,765,255	$244,902,392
		TOTAL TSP PROJECT FIRST COST	**$10,297,000,890**	**$3,398,010,294**	**$13,695,011,184**	**$10,625,382,053**	**$3,506,376,078**	**$14,131,758,131**
			$10,297,001,000	$3,398,010,000	$13,695,011,000	$10,625,382,000	$3,506,376,000	$14,131,758,000

* Contingency amount is 33% and is based on Crystal Ball Analysis

Table 2: Total Project Cost Summary

**** TOTAL PROJECT COST SUMMARY ****

PROJECT: New Jersey Back Bays CSRM TSP Feasibility Study
PROJECT N(P2 402964
LOCATION: Atlantic County, NJ

DISTRICT: Philadelphia District **PREPARED:** 7/6/2021
POC: CHIEF, COST ENGINEERING, Joseph J. Hannings

This Estimate reflects the scope and schedule in report; Draft Feasibility Report (TSP) March 2020

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)						TOTAL PROJECT COST (FULLY FUNDED)			
						Program Year (Budget EC): 2022 Effective Price Level Date: 1 OCT 21									
WBS NUMBER A	Civil Works Feature & Sub-Feature Description B	COST ($K) C	CNTG ($K) D	CNTG (%) E	TOTAL ($K) F	ESC (%) G	COST ($K) H	CNTG ($K) I	TOTAL ($K) J	Spent Thru: 1-Oct-19 ($K)	TOTAL FIRST COST ($K) K	NFLATEC (%) L	COST ($K) M	CNTG ($K) N	FULL ($K) O
02	RELOCATIONS	$3,953	$1,304	33.0%	$5,257	8.5%	$4,289	$1,415	$5,704	$0	$5,704	81.7%	$7,795	$2,572	$10,367
06	FISH & WILDLIFE FACILITIES	$295,631	$97,558	33.0%	$393,189	7.0%	$316,428	$104,421	$420,849	$0	$420,849	67.0%	$528,329	$174,349	$702,678
10	BREAKWATER & SEAWALLS	$4,070,505	$1,343,267	33.0%	$5,413,772	4.0%	$4,231,759	$1,396,480	$5,628,239	$0	$5,628,239	63.4%	$6,913,071	$2,281,313	$9,194,384
11	LEVEES & FLOODWALLS	$768,614	$253,643	33.0%	$1,022,257	4.8%	$805,583	$265,842	$1,071,425	$0	$1,071,425	78.8%	$1,440,317	$475,305	$1,915,622
13	PUMPING PLANT	$15,661	$5,168	33.0%	$20,829	7.1%	$16,776	$5,536	$22,312	$0	$22,312	81.7%	$30,489	$10,061	$40,550
15	FLOODWAY CONTROL & DIVERSION STR	$189,511	$62,539	33.0%	$252,050	7.0%	$202,843	$66,938	$269,781	$0	$269,781	72.1%	$349,084	$115,198	$464,282
18	CULTURAL RESOURCE PRESERVATION	$73,430	$24,232	33.0%	$97,662	7.1%	$78,657	$25,957	$104,614	$0	$104,614	61.3%	$126,877	$41,869	$168,746
19	BUILDINGS, GROUNDS & UTILITIES	$3,767,545	$1,243,290	33.0%	$5,010,835	7.1%	$4,035,735	$1,331,793	$5,367,528	$0	$5,367,528	59.8%	$6,450,162	$2,128,553	$8,578,715
	CONSTRUCTION ESTIMATE TOTALS:	$9,184,851	$3,031,001		$12,215,852	5.5%	$9,692,069	$3,198,383	$12,890,451	$0	$12,890,451	63.5%	$15,846,124	$5,229,221	$21,075,344
01	LANDS AND DAMAGES	$680,940	$224,710	33.0%	$905,650	5.9%	$721,307	$238,031	$959,338	$5,300	$964,638	51.6%	$1,093,742	$360,935	$1,459,977
30	PLANNING, ENGINEERING & DESIGN	$575,454	$189,900	33.0%	$765,354	3.9%	$597,969	$197,330	$795,299	$0	$795,299	32.8%	$793,839	$261,967	$1,055,806
31	CONSTRUCTION MANAGEMENT	$184,138	$60,765	33.0%	$244,903	3.9%	$191,342	$63,143	$254,485	$0	$254,485	63.7%	$313,170	$103,346	$416,516
	PROJECT COST TOTALS:	$10,625,383	$3,506,376	33.0%	$14,131,759		$11,202,687	$3,696,887	**$14,899,573**	$5,300	**$14,904,873**	61.1%	$18,046,876	$5,955,469	$24,007,644

ESTIMATED TOTAL PROJECT COST: $24,007,644

_______________ **CHIEF, COST ENGINEERING, Joseph J. Hannings**

_______________ **PROJECT MANAGER, J. B. Smith**

_______________ **CHIEF, REAL ESTATE, Craig R. Homesley**

_______________ **CHIEF, PLANNING, Peter R. Blum, P.E.**

_______________ **CHIEF, ENGINEERING, Andrew J. Schwaiger, P.E.**

_______________ **CHIEF, OPERATIONS, Michael A. Landis, P.E.**

_______________ **CHIEF, CONSTRUCTION, John A. Delferro, P.E.**

_______________ **CHIEF, CONTRACTING, Nora L. Cherry**

_______________ **CHIEF, PM-PB, Nathan C. Barcomb**

_______________ **CHIEF, DPM, Curtis A. Heckelman**

****** TOTAL PROJECT COST SUMMARY ******

****** CONTRACT COST SUMMARY ******

PROJECT: New Jersey Back Bays CSRM TSP Feasibility Study
LOCATION: Atlantic County, NJ
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (TSP) March 2020

DISTRICT: Philadelphia District
POC: CHIEF, COST ENGINEERING, Joseph J. Hannings
PREPARED: 7/6/2021

Civil Works Work Breakdown Structure		**ESTIMATED COST**				**PROJECT FIRST COST (Constant Dollar Basis)**				**TOTAL PROJECT COST (FULLY FUNDED)**				
		Estimate Prepared:			**26-Dec-19**	Program Year (Budget EC):			2022					
		Effective Price Level:			1-Oct-19	Effective Price Level Date:			1 OCT 21					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	RISK BASED CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	INFLATED (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	NON STRUCTURAL MEASURE 2A													
02	RELOCATIONS	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
10	BREAKWATER & SEAWALLS	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
11	LEVEES & FLOODWALLS	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
13	PUMPING PLANT	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
15	FLOODWAY CONTROL & DIVERSION STRU	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
18	CULTURAL RESOURCE PRESERVATION	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
19	BUILDINGS, GROUNDS & UTILITIES	$25,715	$8,486	33.0%	$34,201	7.1%	$27,546	$9,090	$36,636	2040Q1	59.8%	$44,026	$14,528	$58,554
	CONSTRUCTION ESTIMATE TOTALS:	$25,715	$8,486	33.0%	**$34,201**		$0	$0	$0			$44,026	$14,528	$58,554
01	LANDS AND DAMAGES	$3,137	$1,035	33.0%	$4,173	5.9%	$3,323	$1,097	$4,420	2040Q1	59.8%	$5,311	$1,753	$7,064
30	PLANNING, ENGINEERING & DESIGN													
0.9%	Project Management	$222	$73	33.0%	$295	3.9%	$230	$76	$306	2025Q1	7.7%	$248	$82	$330
0.6%	Planning & Environmental Compliance	$161	$53	33.0%	$214	3.9%	$167	$55	$222	2025Q1	7.7%	$180	$59	$239
1.7%	Engineering & Design	$441	$146	33.0%	$587	3.9%	$458	$151	$610	2025Q1	7.7%	$494	$163	$656
0.0%	Reviews, ATRs, IEPRs, VE	$13	$4	33.0%	$17	3.9%	$13	$4	$18	2025Q1	7.7%	$14	$5	$19
0.1%	Life Cycle Updates (cost, schedule, risks)	$34	$11	33.0%	$45	3.9%	$35	$12	$47	2025Q1	7.7%	$38	$13	$51
0.0%	Contracting & Reprographics	$13	$4	33.0%	$17	3.9%	$13	$4	$18	2025Q1	7.7%	$14	$5	$19
1.8%	Engineering During Construction	$455	$150	33.0%	$605	3.9%	$473	$156	$629	2040Q1	59.7%	$755	$249	$1,005
0.2%	Planning During Construction	$55	$18	33.0%	$74	3.9%	$58	$19	$77	2040Q1	59.7%	$92	$30	$122
0.5%	Adaptive Management & Monitoring	$122	$40	33.0%	$163	3.9%	$127	$42	$169	2040Q1	59.7%	$203	$67	$270
0.4%	Project Operations	$95	$31	33.0%	$127	3.9%	$99	$33	$132	2025Q1	7.7%	$107	$35	$142
31	CONSTRUCTION MANAGEMENT													
1.7%	Construction Management	$427	$141	33.0%	$568	3.9%	$444	$146	$590	2040Q1	59.7%	$709	$234	$943
0.0%	Project Operation:	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
0.4%	Project Management	$107	$35	33.0%	$142	3.9%	$111	$37	$148	2040Q1	59.7%	$177	$58	$236
	CONTRACT COST TOTALS:	$30,998	$10,229		$41,227		$5,552	$1,832	**$7,385**			$52,368	$17,281	**$69,649**

**** TOTAL PROJECT COST SUMMARY ****

**** CONTRACT COST SUMMARY ****

PROJECT: **New Jersey Back Bays CSRM TSP Feasibility Study**
LOCATION: Atlantic County, NJ
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (TSP) March 2020

DISTRICT: Philadelphia District
POC: CHIEF, COST ENGINEERING, Joseph J. Hannings
PREPARED: 7/6/2021

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared:			**26-Dec-19**	Program Year (Budget EC):			2022					
		Effective Price Level:			1-Oct-19	Effective Price Level Date:			1 OCT 21					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	INFLATED (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	MEASURE 3E(2)													
02	RELOCATIONS	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$108,035	$35,652	33.0%	$143,687	7.0%	$115,635	$38,160	$153,795	2035Q3	41.3%	$163,396	$53,921	$217,317
10	BREAKWATER & SEAWALLS	$1,850,369	$610,622	33.0%	$2,460,991	4.0%	$1,923,672	$634,812	$2,558,484	2035Q3	41.3%	$2,718,213	$897,010	$3,615,224
11	LEVEES & FLOODWALLS	$56,142	$18,527	33.0%	$74,669	4.8%	$58,843	$19,418	$78,261	2035Q3	41.3%	$83,147	$27,438	$110,585
13	PUMPING PLANT	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
15	FLOODWAY CONTROL & DIVERSION STRU	$45,220	$14,922	33.0%	$60,142	7.0%	$48,401	$15,972	$64,373	2035Q3	41.3%	$68,392	$22,569	$90,961
18	CULTURAL RESOURCE PRESERVATION	$37,117	$12,249	33.0%	$49,366	7.1%	$39,759	$13,121	$52,880	2035Q3	41.3%	$56,181	$18,540	$74,721
19	BUILDINGS, GROUNDS & UTILITIES	$1,557,293	$513,907	33.0%	$2,071,199	7.1%	$1,668,147	$550,489	$2,218,636	2040Q1	59.8%	$2,666,136	$879,825	$3,545,961
	CONSTRUCTION ESTIMATE TOTALS:	$3,654,176	$1,205,878	33.0%	**$4,860,054**		$3,854,457	$1,271,971	$5,126,428			$5,755,465	$1,899,304	$7,654,769
01	LANDS AND DAMAGES	$83,430	$27,532	33.0%	$110,962	5.9%	$88,376	$29,164	$117,540	2029Q3	20.6%	$106,625	$35,186	$141,811
30	PLANNING, ENGINEERING & DESIGN													
0.9%	Project Management	$31,498	$10,394	33.0%	$41,892	3.9%	$32,730	$10,801	$43,531	2025Q1	7.7%	$35,247	$11,631	$46,878
0.6%	Planning & Environmental Compliance	$22,836	$7,536	33.0%	$30,372	3.9%	$23,729	$7,831	$31,560	2025Q1	7.7%	$25,554	$8,433	$33,987
1.7%	Engineering & Design	$62,672	$20,682	33.0%	$83,354	3.9%	$65,124	$21,491	$86,615	2025Q1	7.7%	$70,132	$23,143	$93,275
0.0%	Reviews, ATRs, IEPRs, VE	$1,817	$600	33.0%	$2,417	3.9%	$1,888	$623	$2,511	2025Q1	7.7%	$2,033	$671	$2,705
0.1%	Life Cycle Updates (cost, schedule, risks)	$4,846	$1,599	33.0%	$6,445	3.9%	$5,035	$1,662	$6,697	2025Q1	7.7%	$5,423	$1,789	$7,212
0.0%	Contracting & Reprographics	$1,817	$600	33.0%	$2,417	3.9%	$1,888	$623	$2,511	2025Q1	7.7%	$2,033	$671	$2,705
1.8%	Engineering During Construction	$64,691	$21,348	33.0%	$86,039	3.9%	$67,222	$22,183	$89,406	2035Q3	41.1%	$94,877	$31,309	$126,186
0.2%	Planning During Construction	$7,874	$2,599	33.0%	$10,473	3.9%	$8,182	$2,700	$10,883	2035Q3	41.1%	$11,549	$3,811	$15,360
0.5%	Adaptive Management & Monitoring	$17,364	$5,730	33.0%	$23,094	3.9%	$18,043	$5,954	$23,998	2047Q3	97.8%	$35,697	$11,780	$47,477
0.4%	Project Operations	$13,528	$4,464	33.0%	$17,992	3.9%	$14,057	$4,639	$18,696	2025Q1	7.7%	$15,138	$4,996	$20,133
31	CONSTRUCTION MANAGEMENT													
1.5%	Construction Management	$55,388	$18,278	33.0%	$73,667	3.9%	$57,556	$18,993	$76,549	2035Q3	41.1%	$81,233	$26,807	$108,040
0.0%	Project Operation:	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
0.4%	Project Management	$13,848	$4,570	33.0%	$18,417	3.9%	$14,389	$4,749	$19,138	2035Q3	41.1%	$20,309	$6,702	$27,011
	CONTRACT COST TOTALS:	$4,035,786	$1,331,809		$5,367,596		$4,252,679	$1,403,384	**$5,656,063**			$6,261,314	$2,066,233	**$8,327,547**

**** TOTAL PROJECT COST SUMMARY ****

Printed:7/23/2021
Page 4 of 5

**** CONTRACT COST SUMMARY ****

PROJECT: **New Jersey Back Bays CSRM TSP Feasibility Study**
LOCATION: Atlantic County, NJ
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (TSP) March 2020

DISTRICT: Philadelphia District
POC: CHIEF, COST ENGINEERING, Joseph J. Hannings

PREPARED: 7/6/2021

Civil Works Work Breakdown Structure		**ESTIMATED COST**				**PROJECT FIRST COST (Constant Dollar Basis)**				**TOTAL PROJECT COST (FULLY FUNDED)**				
		Estimate Prepared:			**26-Dec-19**	Program Year (Budget EC):			2022					
		Effective Price Level:			1-Oct-19	Effective Price Level Date:			1 OCT 21					
WBS NUMBER *A*	Civil Works Feature & Sub-Feature Description *B*	COST ($K) *C*	CNTG ($K) *D*	CNTG (%) *E*	TOTAL ($K) *F*	ESC (%) *G*	COST ($K) *H*	CNTG ($K) *I*	TOTAL ($K) *J*	Mid-Point Date *P*	INFLATED (%) *L*	COST ($K) *M*	CNTG ($K) *N*	FULL ($K) *O*
	MEASURE 4G(8)													
02	RELOCATIONS	$3,953	$1,304	33.0%	$5,257	8.5%	$4,289	$1,415	$5,704	2044Q3	81.7%	$7,795	$2,572	$10,367
06	FISH & WILDLIFE FACILITIES	$187,596	$61,907	33.0%	$249,502	7.0%	$200,793	$66,262	$267,054	2044Q3	81.7%	$364,933	$120,428	$485,361
10	BREAKWATER & SEAWALLS	$2,220,136	$732,645	33.0%	$2,952,781	4.0%	$2,308,087	$761,669	$3,069,755	2044Q3	81.7%	$4,194,858	$1,384,303	$5,579,161
11	LEVEES & FLOODWALLS	$712,472	$235,116	33.0%	$947,588	4.8%	$746,740	$246,424	$993,164	2044Q3	81.7%	$1,357,171	$447,866	$1,805,037
13	PUMPING PLANT	$15,661	$5,168	33.0%	$20,829	7.1%	$16,776	$5,536	$22,312	2044Q3	81.7%	$30,489	$10,061	$40,550
15	FLOODWAY CONTROL & DIVERSION STRU	$144,292	$47,616	33.0%	$191,908	7.0%	$154,442	$50,966	$205,408	2044Q3	81.7%	$280,693	$92,629	$373,321
18	CULTURAL RESOURCE PRESERVATION	$36,313	$11,983	33.0%	$48,296	7.1%	$38,898	$12,836	$51,734	2044Q3	81.7%	$70,695	$23,329	$94,025
19	BUILDINGS, GROUNDS & UTILITIES	$227,547	$75,091	33.0%	$302,638	7.1%	$243,745	$80,436	$324,181	2040Q1	59.8%	$389,568	$128,558	$518,126
	CONSTRUCTION ESTIMATE TOTALS:	$3,547,969	$1,170,830	33.0%	**$4,718,799**		$3,713,769	$1,225,544	$4,939,312			$6,696,201	$2,209,746	$8,905,947
01	LANDS AND DAMAGES	$358,273	$118,230	33.0%	$476,502	5.9%	$379,512	$125,239	$504,750	2038Q3	53.4%	$582,088	$192,089	$774,177
30	PLANNING, ENGINEERING & DESIGN													
0.9%	Project Management	$30,582	$10,092	33.0%	$40,674	3.9%	$31,779	$10,487	$42,266	2025Q1	7.7%	$34,222	$11,293	$45,516
0.6%	Planning & Environmental Compliance	$22,172	$7,317	33.0%	$29,489	3.9%	$23,040	$7,603	$30,643	2025Q1	7.7%	$24,811	$8,188	$32,999
1.7%	Engineering & Design	$60,851	$20,081	33.0%	$80,931	3.9%	$63,231	$20,866	$84,098	2025Q1	7.7%	$68,093	$22,471	$90,564
0.0%	Reviews, ATRs, IEPRs, VE	$1,764	$582	33.0%	$2,347	3.9%	$1,833	$605	$2,438	2025Q1	7.7%	$1,974	$652	$2,626
0.1%	Life Cycle Updates (cost, schedule, risks)	$4,705	$1,553	33.0%	$6,258	3.9%	$4,889	$1,613	$6,502	2025Q1	7.7%	$5,265	$1,737	$7,002
0.0%	Contracting & Reprographics	$1,764	$582	33.0%	$2,347	3.9%	$1,833	$605	$2,438	2025Q1	7.7%	$1,974	$652	$2,626
1.8%	Engineering During Construction	$62,811	$20,728	33.0%	$83,539	3.9%	$65,269	$21,539	$86,807	2045Q3	86.8%	$121,951	$40,244	$162,194
0.2%	Planning During Construction	$7,646	$2,523	33.0%	$10,169	3.9%	$7,945	$2,622	$10,566	2045Q3	86.8%	$14,844	$4,899	$19,743
0.5%	Adaptive Management & Monitoring	$16,859	$5,564	33.0%	$22,423	3.9%	$17,519	$5,781	$23,300	2055Q1	145.1%	$42,947	$14,173	$57,120
0.4%	Project Operations	$13,135	$4,334	33.0%	$17,469	3.9%	$13,649	$4,504	$18,153	2025Q1	7.7%	$14,698	$4,850	$19,548
31	CONSTRUCTION MANAGEMENT													
1.7%	Construction Management	$59,425	$19,610	33.0%	$79,035	3.9%	$61,750	$20,377	$82,127	2045Q3	86.8%	$115,376	$38,074	$153,450
0.0%	Project Operation:	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
0.4%	Project Management	$14,856	$4,903	33.0%	$19,759	3.9%	$15,437	$5,094	$20,532	2045Q3	86.8%	$28,844	$9,518	$38,362
	CONTRACT COST TOTALS:	$4,202,812	$1,386,928		**$5,589,740**		$4,401,454	$1,452,480	**$5,853,934**			$7,753,289	$2,558,585	**$10,311,875**

****** TOTAL PROJECT COST SUMMARY ******

****** CONTRACT COST SUMMARY ******

PROJECT: New Jersey Back Bays CSRM TSP Feasibility Study
LOCATION: Atlantic County, NJ
This Estimate reflects the scope and schedule in report; Draft Feasibility Report (TSP) March 2020

DISTRICT: Philadelphia District
POC: CHIEF, COST ENGINEERING, Joseph J. Hannings

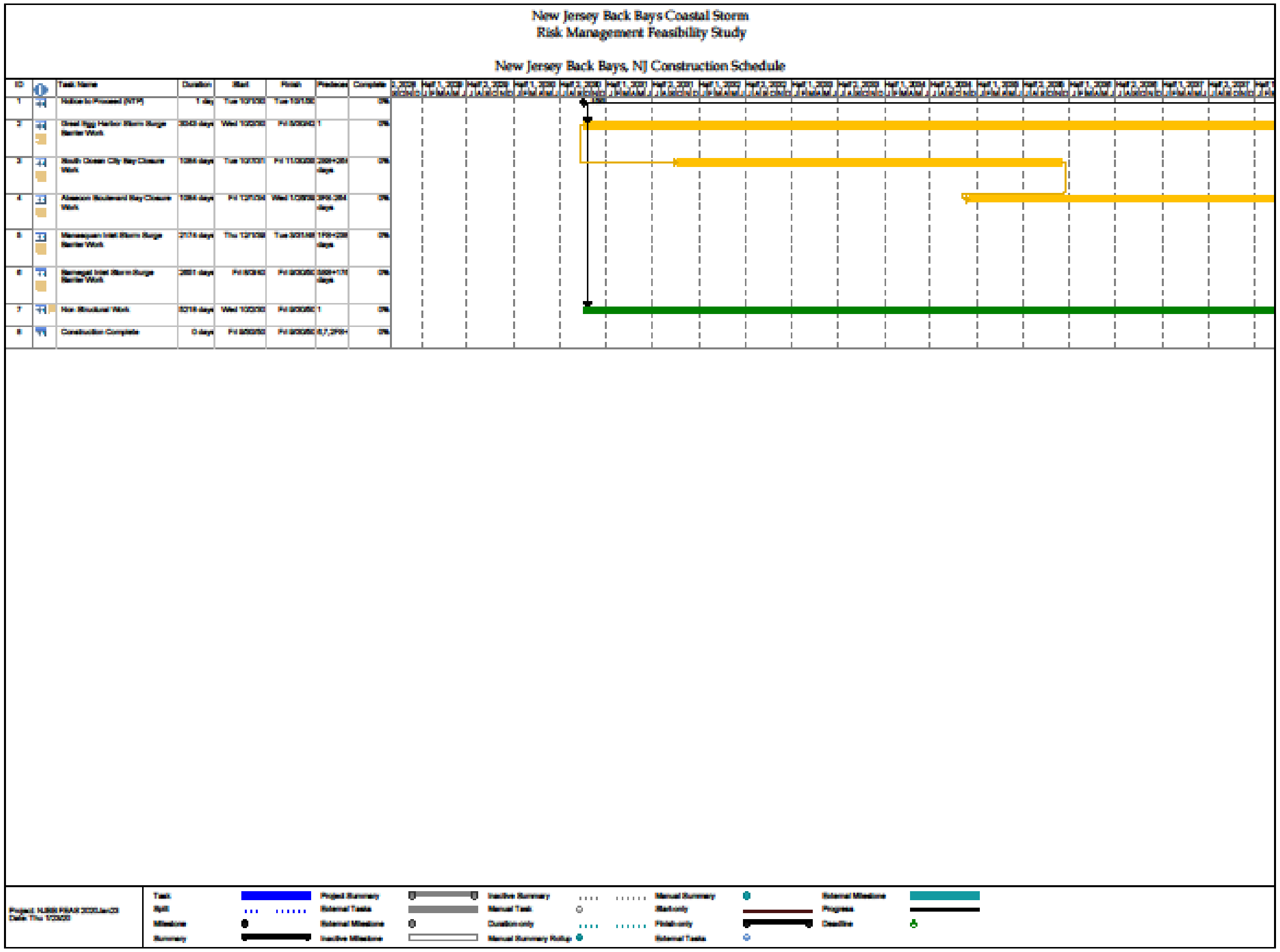
PREPARED: 7/6/2021

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared:			**26-Dec-19**	Program Year (Budget EC):			2022	FULLY FUNDED PROJECT ESTIMATE				
		Effective Price Level:			1-Oct-19	Effective Price Level Date:			1 OCT 21					
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	INFLATED (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	***B***	***C***	***D***	***E***	***F***	***G***	***H***	***I***	***J***	***P***	***L***	***M***	***N***	***O***
	NON STRUCTURAL MEASURE 5A													
02	RELOCATIONS	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
10	BREAKWATER & SEAWALLS	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
11	LEVEES & FLOODWALLS	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
13	PUMPING PLANT	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
15	FLOODWAY CONTROL & DIVERSION STRU	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
18	CULTURAL RESOURCE PRESERVATION	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
19	BUILDINGS, GROUNDS & UTILITIES	$1,956,990	$645,807	33.0%	$2,602,797	7.1%	$2,096,297	$691,778	$2,788,075	2040Q1	59.8%	$3,350,432	$1,105,642	$4,456,074
	CONSTRUCTION ESTIMATE TOTALS:	$1,956,990	$645,807	33.0%	**$2,602,797**		$2,096,297	$691,778	$2,788,075			$3,350,432	$1,105,642	$4,456,074
01	LANDS AND DAMAGES	$236,099	$77,913	33.0%	$314,012	5.9%	$250,096	$82,532	$332,627	2040Q1	59.8%	$399,719	$131,907	$531,626
30	PLANNING, ENGINEERING & DESIGN													
0.9%	Project Management	$16,869	$5,567	33.0%	$22,435	3.9%	$17,529	$5,784	$23,313	2025Q1	7.7%	$18,876	$6,229	$25,105
0.6%	Planning & Environmental Compliance	$12,230	$4,036	33.0%	$16,265	3.9%	$12,708	$4,194	$16,902	2025Q1	7.7%	$13,685	$4,516	$18,201
1.7%	Engineering & Design	$33,564	$11,076	33.0%	$44,640	3.9%	$34,877	$11,509	$46,387	2025Q1	7.7%	$37,559	$12,394	$49,953
0.0%	Reviews, ATRs, IEPRs, VE	$973	$321	33.0%	$1,294	3.9%	$1,011	$334	$1,345	2025Q1	7.7%	$1,089	$359	$1,448
0.1%	Life Cycle Updates (cost, schedule, risks)	$2,595	$856	33.0%	$3,452	3.9%	$2,697	$890	$3,587	2025Q1	7.7%	$2,904	$958	$3,862
0.0%	Contracting & Reprographics	$973	$321	33.0%	$1,294	3.9%	$1,011	$334	$1,345	2025Q1	7.7%	$1,089	$359	$1,448
1.8%	Engineering During Construction	$34,645	$11,433	33.0%	$46,078	3.9%	$36,001	$11,880	$47,881	2040Q1	59.7%	$57,493	$18,973	$76,465
0.2%	Planning During Construction	$4,217	$1,392	33.0%	$5,609	3.9%	$4,382	$1,446	$5,828	2040Q1	59.7%	$6,998	$2,309	$9,308
0.5%	Adaptive Management & Monitoring	$9,299	$3,069	33.0%	$12,368	3.9%	$9,663	$3,189	$12,852	2040Q1	59.7%	$15,432	$5,093	$20,524
0.4%	Project Operations	$7,245	$2,391	33.0%	$9,636	3.9%	$7,528	$2,484	$10,013	2025Q1	7.7%	$8,107	$2,675	$10,782
31	CONSTRUCTION MANAGEMENT													
1.6%	Construction Management	$32,069	$10,583	33.0%	$42,652	3.9%	$33,324	$10,997	$44,321	2040Q1	59.7%	$53,218	$17,562	$70,780
0.0%	Project Operation:	$0	$0	33.0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
0.4%	Project Management	$8,017	$2,646	33.0%	$10,663	3.9%	$8,331	$2,749	$11,080	2040Q1	59.7%	$13,304	$4,390	$17,695
	CONTRACT COST TOTALS:	$2,355,787	$777,410		**$3,133,196**		$2,515,455	$830,100	**$3,345,556**			$3,979,905	$1,313,369	**$5,293,273**

Construction and Funding Schedule for the TSP:

The construction and project schedules of the TSP are given in Tables 3 and 4 respectively of this Engineering Technical Appendix. The schedules are based on the timeliness of the report's approval and allocation of funds by OMB, the foregoing construction procedures, and the ability of local interests to implement the necessary items of local cooperation.

Table 3: Construction Schedule



New Jersey Back Bays Coastal Storm
Risk Management Feasibility Study

New Jersey Back Bays, NJ Construction Schedule

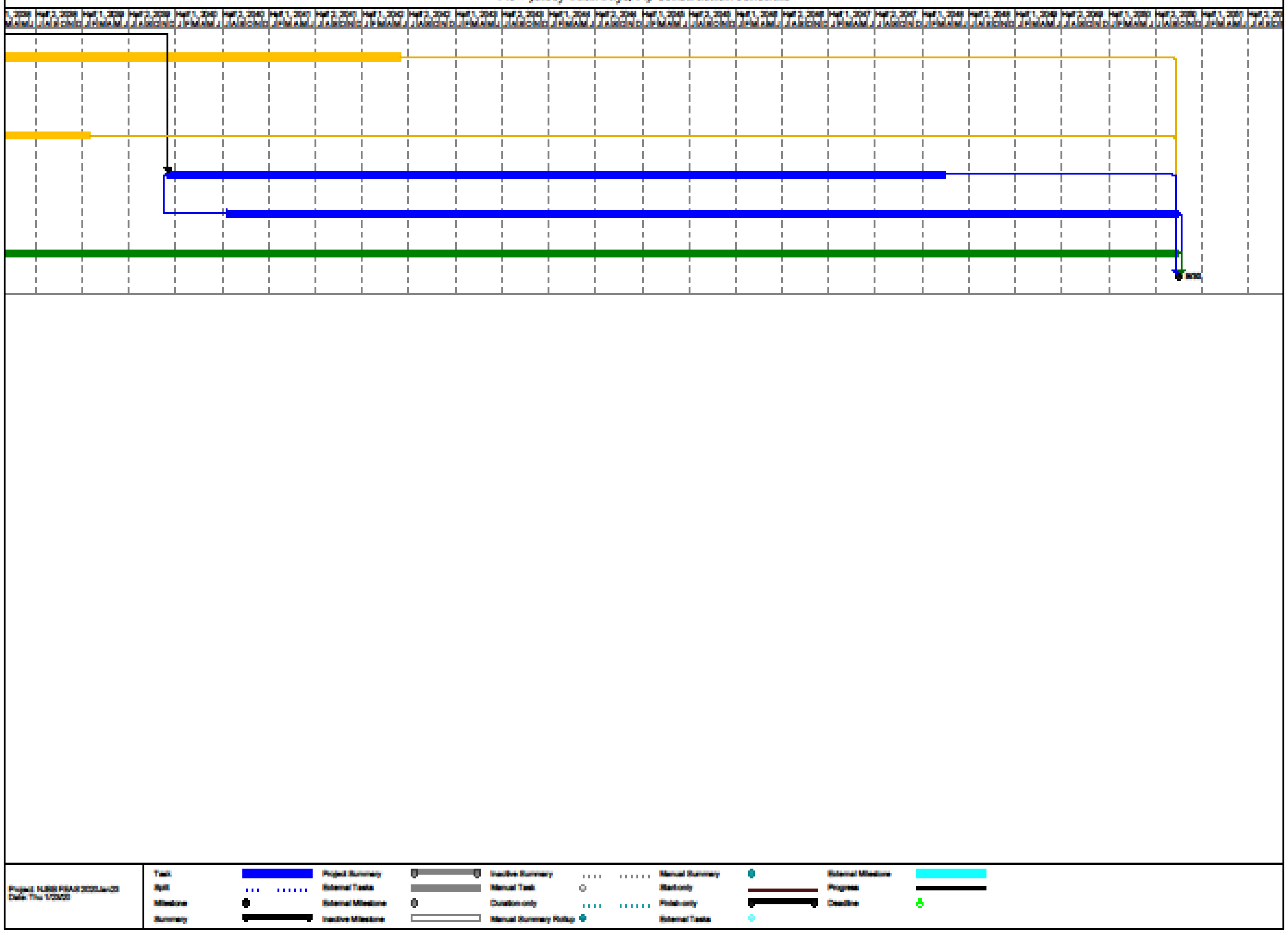
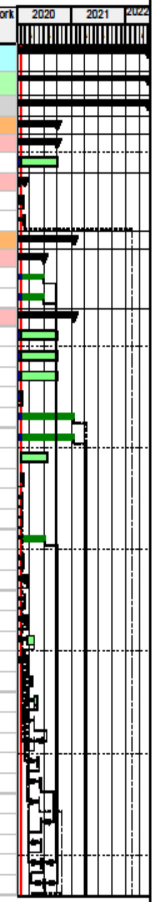


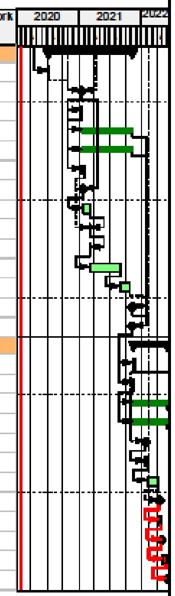
Table 4: Project Schedule

NEW JERSEY BACKBAYS COASTAL RESILIENCE STUDY, NJ	1/2			Data Date: 01-Feb-20				Print Date : 24-Jan-20		
Activity ID	**Activity Name**	**Original Duration**	**Remaining Duration**	**Total Float**	**Start**	**Finish**	**Remaining Total Cost**	**Actual Total Cost**	**At Completion Total Cost**	**CEFMS Work Item Code**
402964	**NEW JERSEY BACKBAYS COASTAL RESILIENCE STUDY, NJ**	1417	583	0	03-Oct-16 A	27-May-22	$14,790,804.54	$3,732,195.46	$18,523,000.00	
402964.CW	**Standard Civil Works Project**	1417	583	0	03-Oct-16 A	27-May-22	$14,790,804.54	$3,732,195.46	$18,523,000.00	
402964.CW.22000	**Feasibility Studies**	1417	583	0	03-Oct-16 A	27-May-22	$14,790,804.54	$3,732,195.46	$18,523,000.00	
402964.CW.22000.02	**Scoping**	850	169	414	03-Oct-16 A	30-Sep-20	$0.00	$201,269.68	$201,269.68	
402964.CW.22000.02.1	**Scoping Phase PDT Wide Tasks**	250	169	414	03-Oct-16 A	30-Sep-20	$0.00	$201,269.68	$201,269.68	
SCP1360	Budget FY17	250	169	414	03-Oct-16 A	30-Sep-20	$0.00	$201,269.68	$201,269.68	5D07J7
402964.CW.22000.02.2	**Study Initiation**	25	16	547	21-Jan-20 A	25-Feb-20	$0.00	$0.00	$0.00	
SCP1220	Prepare Model Review Plan	20	11	547	21-Jan-20 A	18-Feb-20	$0.00	$0.00	$0.00	
SCP1245	Model Certification (If needed)	5	5	547	19-Feb-20	25-Feb-20	$0.00	$0.00	$0.00	
402964.CW.22000.03	**Alternative Evaluation & Analysis**	1015	242	341	03-Jan-17 A	19-Jan-21	$9,644,804.54	$3,530,925.78	$13,175,730.32	
402964.CW.22000.03.1	**Alternatives Phase PDT Wide Tasks**	332	104	182	03-Jan-17 A	29-Jun-20	$933,965.90	$1,173,914.25	$2,107,880.15	
ALT1420	TSP-IPR Evaluation In-House Support - Fed (FY18)	332	104	182	03-Jan-17 A	29-Jun-20	$755,835.22	$861,546.29	$1,617,381.51	5J94L0
ALT1430	TSP-IPR Evaluation In-House Support - Non-Fed (FY18)	332	104	182	03-Jan-17 A	29-Jun-20	$178,130.68	$312,367.96	$490,498.64	5241G7
402964.CW.22000.03.3	**Tentative Selected Plan**	534	242	341	03-Dec-18 A	19-Jan-21	$8,710,838.64	$2,357,011.53	$11,067,850.17	
ALT1510	TSP-IPR to TSP Evaluation Non-Fed	278	169	414	03-Dec-18 A	30-Sep-20	$1,945,713.21	$1,520,349.55	$3,466,062.76	5K9F1B
ALT1550	TSP-IPR to TSP Evaluation Fed #2	175	169	414	01-May-19 A	30-Sep-20	$2,525,504.07	$812,863.34	$3,338,367.41	5L8B09
ALT1560	NNBF Workshop	252	169	414	01-Aug-19 A	30-Sep-20	$1,621.36	$23,798.64	$25,420.00	55KHKK
ALT1040	Negotiate IEPR Contract	45	3	253	02-Dec-19 A	05-Feb-20	$0.00	$0.00	$0.00	
ALT1520	TSP to ADM Evaluation Fed	250	242	51	21-Jan-20 A	19-Jan-21	$2,000,000.00	$0.00	$2,000,000.00	557KC1
ALT1530	TSP to ADM Evaluation Non-Fed	250	242	51	21-Jan-20 A	19-Jan-21	$2,118,000.00	$0.00	$2,118,000.00	54304L
ALT1060	USFWS Final 2B Report	124	124	459	03-Feb-20	28-Jul-20	$0.00	$0.00	$0.00	
ALT1460	Refine TSP	12	12	182	03-Feb-20	19-Feb-20	$0.00	$0.00	$0.00	
ALT1020	Initiate IEPR Contract	1	1	253	06-Feb-20	06-Feb-20	$0.00	$0.00	$0.00	
ALT1090	IEPR Contract Awarded	1	1	578	07-Feb-20	07-Feb-20	$0.00	$0.00	$0.00	
ALT1095	IEPR Contract - Budget	103	103	182	10-Feb-20	06-Jul-20	$120,000.00	$0.00	$120,000.00	503F82
ALT1470	TSP MFR	2	2	182	20-Feb-20	21-Feb-20	$0.00	$0.00	$0.00	
ALT1480	DQC of Draft Report	17	17	182	24-Feb-20	17-Mar-20	$0.00	$0.00	$0.00	
ALT1490	Update Report Summary, Risk Register, DMP and Report Consistent w/ TSP	1	1	234	24-Feb-20	24-Feb-20	$0.00	$0.00	$0.00	
ALT1175	Prepare Enviromental Notice of Availability for EPA (NOA)	17	17	234	25-Feb-20	18-Mar-20	$0.00	$0.00	$0.00	
ALT1160	Concurrent Review of Draft Feasibility/ EIS Report - ATR, IEPR, Policy, HQ, Cost and Public	30	30	182	18-Mar-20	28-Apr-20	$0.00	$0.00	$0.00	
ALT1180	NOA Filed in Federal Register	1	1	234	19-Mar-20	19-Mar-20	$0.00	$0.00	$0.00	
ALT1440	Final Comprehensive RE Recon Cost Estimate for TSP	11	11	528	06-Apr-20*	20-Apr-20	$0.00	$0.00	$0.00	
ALT1210	Public Draft Report and NEPA Comment Period of EIS	15	15	182	29-Apr-20	19-May-20	$0.00	$0.00	$0.00	
ALT1230	IEPR Review / Final IEPR Report	2	2	195	29-Apr-20	30-Apr-20	$0.00	$0.00	$0.00	
ALT1240	Receive IEPR Comments	1	1	195	30-Apr-20	30-Apr-20	$0.00	$0.00	$0.00	
ALT1235	Develop Public Response Matrix	9	9	182	20-May-20	02-Jun-20	$0.00	$0.00	$0.00	
ALT1250	Respond to IEPR Comments	8	8	182	03-Jun-20	12-Jun-20	$0.00	$0.00	$0.00	
ALT1410	Risk Informed Decision Workshop #2	1	1	207	03-Jun-20	03-Jun-20	$0.00	$0.00	$0.00	
ALT1260	Receive Final IEPR Report	1	1	182	11-Jun-20	12-Jun-20	$0.00	$0.00	$0.00	
ALT1390	Final Draft of Feasibility Study Report and EIS	1	1	182	12-Jun-20	15-Jun-20	$0.00	$0.00	$0.00	
ALT1270	Prepare Read Ahead for Agency Decision Milestone Meeting	10	10	182	15-Jun-20	29-Jun-20	$0.00	$0.00	$0.00	
ALT1380	Distribute Agency Dession Milestone Meeting Read Aheads	4	4	182	30-Jun-20	06-Jul-20	$0.00	$0.00	$0.00	



proj 402964- Ongoing Schedule-01-Feb-20

NEW JERSEY BACKBAYS COASTAL RESILIENCE STUDY, NJ	2/2	Data Date: 01-Feb-20									Print Date : 24-Jan-20
Activity ID	**Activity Name**	**Original Duration**	**Remaining Duration**	**Total Float**	**Start**	**Finish**	**Remaining Total Cost**	**Actual Total Cost**	**At Completion Total Cost**	**CEFMS Work Item Code**	
402964.CW.22000.04 Feasibility Level Analysis		**339**	**339**	**42**	**07-Jul-20**	**10-Nov-21**	**$4,148,000.00**	**$0.00**	**$4,148,000.00**		
FEA1005	ASA Policy Exception Letter Signed (If necessary)	3	3	182	07-Jul-20	09-Jul-20	$0.00	$0.00	$0.00		
FEA1020	Agency Decision Milestone	0	0	52		20-Jan-21*	$0.00	$0.00	$0.00		
FEA1080	Submit Exemption Package for Post Sept 2020	1	1	69	21-Jan-21	21-Jan-21	$0.00	$0.00	$0.00		
FEA1560	ADM to Final Report Evaluation Fed	205	205	42	21-Jan-21	10-Nov-21	$2,074,000.00	$0.00	$2,074,000.00	59GBK7	
FEA1570	ADM to Final Report Evaluation Non-Fed	205	205	42	21-Jan-21	10-Nov-21	$2,074,000.00	$0.00	$2,074,000.00	5J4308	
FEA1145	Comprehensive Cost Schedule Risk Analysis (Optimization)	10	10	69	22-Jan-21	04-Feb-21	$0.00	$0.00	$0.00		
FEA1030	Agency Decision MFR	0	0	51		27-Jan-21	$0.00	$0.00	$0.00		
FEA1040	HQ Finalize Comments and Project Guidance Memo	25	25	51	28-Jan-21	04-Mar-21	$0.00	$0.00	$0.00		
FEA1050	Cost Certification from Cost DX	1	1	51	04-Mar-21	04-Mar-21	$0.00	$0.00	$0.00		
FEA1065	District Submits Report to NAD	1	1	51	04-Mar-21	04-Mar-21	$0.00	$0.00	$0.00		
FEA1070	Complete Draft of Final FR/EIS/EA	126	126	51	05-Mar-21	31-Aug-21	$0.00	$0.00	$0.00		
FEA1090	DQC of Final Report	40	40	51	01-Sep-21	28-Oct-21	$0.00	$0.00	$0.00		
FEA1100	Final Report Complete	0	0	42		10-Nov-21*	$0.00	$0.00	$0.00		
FEA1110	Submit Final Report (Division Engineer's Notice)	0	0	42		10-Nov-21*	$0.00	$0.00	$0.00		
402964.CW.22000.05 Chief's Report Milestone		**136**	**136**	**0**	**12-Nov-21**	**27-May-22**	**$998,000.00**	**$0.00**	**$998,000.00**		
CHR1010	Prepare Package for State and Agency Review	10	10	42	12-Nov-21	26-Nov-21	$0.00	$0.00	$0.00		
CHR1110	Senior Leader Panel Workshop	1	1	51	12-Nov-21	12-Nov-21	$0.00	$0.00	$0.00		
CHR1580	Final Report to Chief's Report Evaluation Fed	135	135	0	15-Nov-21	27-May-22	$499,000.00	$0.00	$499,000.00	53B77G	
CHR1590	Final Report to Chief's Report Evaluation Non-Fed	135	135	0	15-Nov-21	27-May-22	$499,000.00	$0.00	$499,000.00	58L789	
CHR1020	State and Agency Review (Final FR/EA/EIS and Draft Chief's Report)	0	0	6		20-Jan-22*	$0.00	$0.00	$0.00		
CHR1030	Response Letters to S&A comments (If required)	10	10	6	21-Jan-22	03-Feb-22	$0.00	$0.00	$0.00		
CHR1040	OWPR & RIT Coordination of Final Report Packet & Chief's Report	44	44	6	04-Feb-22	07-Apr-22	$0.00	$0.00	$0.00		
CHR1050	Chief Signs Report of the Chief of Engineers	0	0	0		15-Apr-22*	$0.00	$0.00	$0.00		
CHR1060	Chief's Report Forwarded to ASA(CW) (RIT TASK)	10	10	0	18-Apr-22	29-Apr-22	$0.00	$0.00	$0.00		
CHR1070	ASA(CW) Signs Record of Decision If Not Authorized	0	0	0		29-Apr-22	$0.00	$0.00	$0.00		
CHR1080	Feasibility Report Transmittal to Congress	20	20	0	02-May-22	27-May-22	$0.00	$0.00	$0.00		
CHR1090	Feasibility Report to Congress	0	0	0		27-May-22	$0.00	$0.00	$0.00		



REFERENCES

ER 1110-2-1302, Civil Works Cost Engineering

Kluijver, M., C. Dols, S.N. Jonkman and L.F. Mooyaart. 2019. Advances in the Planning and Conceptual Design of Storm Surge Barriers – Application to the New York Metropolitan Area. New York-New Jersey Harbor and Tributaries Study (NYNJ HATS). U.S. Army Corps of Engineers, New York District.

MII Cost Estimate Summary

NJBB TSP

New Jersey Back Bays
Coastal Storm Risk Management
Feasibility Study

Tentatively Selected Plan (TSP)
March 2020

Estimated by	NAP Cost Section
Designed by	NAP Civil Section
Prepared by	William Welk
Preparation Date	2/21/2020
Effective Date of Pricing	11/29/2019
Estimated Construction Time	7,300 Days

Designed by	
NAP Civil Section	
Estimated by	
NAP Cost Section	
Prepared by	
William Welk	

Design Document	TSP
Document Date	2/3/2020
District	Philadelphia District
Contact	William Welk
Budget Year	2020
UOM System	Original

Direct Costs

LaborCost
EQCost
MatlCost
SubBidCost

Timeline/Currency

Preparation Date	2/21/2020
Escalation Date	11/29/2019
Eff. Pricing Date	11/29/2019
Estimated Duration	7300 Day(s)
Currency	US dollars
Exchange Rate	1.000000

Costbook CB16EN: 2016 MII English Cost Book

Labor Region 1: Labor Region 1 -2019

Labor Rates

LaborCost1
LaborCost2
LaborCost3
LaborCost4

Equipment EP16R01: MII Equipment 2016 Region 01

01 NORTHEAST

Sales Tax	7.00
Working Hours per Year	1,360
Labor Adjustment Factor	1.16
Cost of Money	1.88
Cost of Money Discount	25.00
Tire Recap Cost Factor	1.50
Tire Recap Wear Factor	1.80
Tire Repair Factor	0.15
Equipment Cost Factor	1.00
Standby Depreciation Factor	0.50

Fuel

Electricity	0.143
Gas	2.900
Diesel Off-Road	2.880
Diesel On-Road	3.390

Shipping Rates

Over 0 CWT	17.43
Over 240 CWT	12.24
Over 300 CWT	9.98
Over 400 CWT	8.61
Over 500 CWT	7.45
Over 700 CWT	7.45
Over 800 CWT	10.71

Labor ID: Region 1 EQ ID: EP16R01 Currency in US dollars TRACES MII Version 4.4

Date	Author	Note
6/6/2016	ww	1. Prepared by the U.S. Army Corps of Engineers, Philadelphia District, Wanamaker Building, 100 Penn Square East, Philadelphia, PA 19107-3391.
6/6/2016	ww	2. SUMMARY OF WORK: Work includes, but is not limited to:
2/2/2018	ww	Shark River Region - Measure 2A (non-structural only).
7/31/2019	ww	Northern Region - Measure 3(E)2: Construct Barnegat Inlet Storm Surge Barrier and Manasquan Inlet Storm Surge Barrier. In addition, construct 7,280 LF of Type A levee and 897 LF of Type C concrete floodwall. Environmetal mitigation and non-structural work is also included for the above.
7/31/2019	ww	Central Region - Measure 4(G)8: Construct Great Egg Harbor Inlet Storm Surge Barrier, Absecon Blvd Bay Closure and South Ocean City Bay Closure. In addition, construct 37,965 LF of Type A levee, 6,398 LF of Type B, 8,422 LF of Type C concrete floodwalls and 18,194 LF of Type D king pile combined w/ steel sheetpile floodwall. Environmetal mitigation and non-structural work is also included for the above.
7/31/2019	ww	Southern Region - Measure 5A (non-structural only).
7/31/2019	ww	3. Construction schedule:
7/31/2019	ww	- Report completion (Chief of Engineers Report) - April 2022
2/3/2020	ww	- Estimated start of construction - October 2030
2/3/2020	ww	- Mid-point of construction - October 2050 based on 20-year construction duration.
2/3/2020	ww	4. Used Ocean County, NJ labor rates, General Decision Number NJ140050, Mod. No. 4 dated 10/04/19.
2/3/2020	ww	5. Real estate costs (project feature 01) provided through PL-PC and furnished by CENAB-RE.
2/3/2020	ww	6. P,E&D costs (project feature 30) are based on similar COE projects constructed by NAP. S&A costs (project feature 31) calculated based on construction staffing plan for similar COE projects constructed by NAP.
2/3/2020	ww	7. Price level: October 2019.
2/3/2020	ww	8. Contingencies are based on Crystal Ball software for preparing risk analysis and is 33% for all project costs.
2/3/2020	ww	9. Critical assumptions:
2/3/2020	ww	- There will be no environmental construction windows.
2/3/2020	ww	- There will be no severe weather events during construction.
2/3/2020	ww	- Construction work will take place 5 days a week, 8 hours per day.
2/3/2020	ww	- Job will be open bid.
2/3/2020	ww	10. Used R.S. Means, MII Cost Book, price quotes and historic data for material costs as needed.
2/3/2020	ww	

Labor ID: Region 1   EQ ID: EP16R01   Currency in US dollars   TRACES MII Version 4.4

Direct Cost Markups	Category	Method
Productivity	Productivity	Productivity
Overtime	Overtime	Overtime

	Days/Week	Hours/Shift	Shifts/Day	1st Shift	2nd Shift	3rd Shift
Standard	5.00	8.00	1.00	8.00	0.00	0.00
Actual	5.00	8.00	1.00	8.00	0.00	0.00

Day	OT Factor	Working	OT Percent	FCCM Percent
Monday	1.50	Yes	0.00	0.00
Tuesday	1.50	Yes		
Wednesday	1.50	Yes		
Thursday	1.50	Yes		
Friday	1.50	Yes		
Saturday	1.50	No		
Sunday	2.00	No		

Sales Tax	TaxAdj	Running % on Selected Costs
MatlCost		

Contractor Markups	Category	Method
JOOH	JOOH	Running %
JOOHCALC (Small Tools)	JOOH	% of Labor
JOOHCALC	JOOH	JOOH (Calculated)
HOOH	HOOH	Running %
Profit	Profit	Running %
Bond	Bond	Running %
Excise Tax	Excise	Running %

Owner Markups	Category	Method
Escalation	Escalation	Escalation

StartDate	StartIndex	EndDate	EndIndex	Escalation
12/29/2017	844.49	12/28/2018	882.99	4.56

Contingency	Contingency	Running %
SIOH	SIOH	Running %

Labor ID: Region 1 EQ ID: EP16R01 Currency in US dollars TRACES MII Version 4.4

Description	Quantity	UOM	ContractCost	Escalation	Contingency	SIOH	ProjectCost
Project Cost Summary			9,132,207,290	416,427,700	0	0	9,548,634,990
			9,132,207,289.54				*9,548,634,989.95*
Tentatively Selected Plan (TSP) -	1.0	EA	9,132,207,290	416,427,700	0	0	9,548,634,990
Shark River Region - Measure 2A	1.0	LS	26,606,091	1,213,238	0	0	27,819,329
01 Lands and Damages	1.0	LS	2,892,997	131,921	0	0	3,024,918
01 Lands and Damages	1.0	LS	2,892,997	131,921	0	0	3,024,918
19 Buildings Grounds and Utitities	1.0	LS	23,713,094	1,081,317	0	0	24,794,411
19A Buildings Grounds and Utitities - Shark River Region	1.0	LS	23,713,094	1,081,317	0	0	24,794,411
Northern Region - Measure 3E(2)	1.0	LS	3,461,617,525	157,849,759	0	0	3,619,467,284
01 Lands and Damages	1.0	LS	76,934,329	3,508,205	0	0	80,442,535
01 Lands and Damages	1.0	LS	76,934,329	3,508,205	0	0	80,442,535
06 Fish and Wildlife Facilities	1.0	LS	98,593,201	4,495,850	0	0	103,089,051
06A Fish and Wildlife Facilities - Barnegat Inlet Storm Surge Barrier (SSB)	1.0	LS	71,615,176	3,265,652	0	0	74,880,828
06B Fish and Wildlife Facilities - Manasquan Inlet Storm Surge Barrier (SSB)	1.0	LS	26,978,024	1,230,198	0	0	28,208,222
10 Breakwaters and Seawalls	1.0	LS	1,738,574,129	79,278,980	0	0	1,817,853,109
10A Breakwaters and Seawalls - Barnegat Inlet SSB	1.0	LS	1,244,045,368	56,728,469	0	0	1,300,773,837
10B Breakwaters and Seawalls - Manasquan Inlet SSB	1.0	LS	494,528,760	22,550,511	0	0	517,079,272
11 Levees and Floodwalls	1.0	LS	52,323,214	2,385,939	0	0	54,709,153
11A Levees and Floodwalls - Barnegat Inlet SSB	1.0	LS	6,403,546	292,002	0	0	6,695,548
11B Levees and Floodwalls - Manasquan Inlet SSB	1.0	LS	45,919,668	2,093,937	0	0	48,013,605
15 Floodway Control and Diversion Structures	1.0	LS	41,267,187	1,881,784	0	0	43,148,971
15A Floodway Control and Diversion Structures - Barnegat Inlet SSB	1.0	LS	41,267,187	1,881,784	0	0	43,148,971
18 Cultural Resource Preservation	1.0	LS	33,846,796	1,543,414	0	0	35,390,210

Currency in US dollars

Description	Quantity	UOM	ContractCost	Escalation	Contingency	SIOH	ProjectCost
18A.86 Identification, Data Analysis and Reports	1.0	LS	33,846,796	1,543,414	0	0	35,390,210
19 Buildings Grounds and Utitities	1.0	LS	1,420,078,669	64,755,587	0	0	1,484,834,256
19A Buildings Grounds and Utitities - Barnegat Inlet SSB	1.0	LS	1,420,078,669	64,755,587	0	0	1,484,834,256
Central Region - Measure 4G(8)	1.0	LS	3,641,687,910	166,060,969	0	0	3,807,748,879
01 Lands and Damages	1.0	LS	330,376,697	15,065,177	0	0	345,441,875
01 Lands and Damages	1.0	LS	330,376,697	15,065,177	0	0	345,441,875
02 Relocations	1.0	LS	3,559,351	162,306	0	0	3,721,657
02A Relocations - Absecon Blvd Bay Closure	1.0	LS	3,345,159	152,539	0	0	3,497,698
02B Relocations - South Ocean City Bay Closure	1.0	LS	214,192	9,767	0	0	223,959
06 Fish and Wildlife Facilities	1.0	LS	171,200,409	7,806,739	0	0	179,007,148
06A Fish and Wildlife Facilities - Great Egg Harbor Inlet Storm Surge Barrier (SSB)	1.0	LS	98,293,744	4,482,195	0	0	102,775,939
06B. Fish and Wildlife Facilities - Absecon Blvd Bay Closure	1.0	LS	55,933,207	2,550,554	0	0	58,483,761
06C. Fish and Wildlife Facilities - South Ocean City 52nd Street Bay Closure	1.0	LS	16,973,458	773,990	0	0	17,747,448
10 Breakwaters and Seawalls	1.0	LS	2,085,988,241	95,121,064	0	0	2,181,109,305
10A Breakwaters and Seawalls - Great Egg Harbor Inlet SSB	1.0	LS	1,793,948,309	81,804,043	0	0	1,875,752,352
10B Breakwaters and Seawalls - Absecon Blvd Bay Closre	1.0	LS	181,724,816	8,286,652	0	0	190,011,467
10C. Breakwaters and Seawalls - South Ocean City Bay Closre	1.0	LS	110,315,116	5,030,369	0	0	115,345,485
11 Levees and Floodwalls	1.0	LS	663,988,694	30,277,884	0	0	694,266,579
11A Levees and Floodwalls - Great Egg Harbor SSB	1.0	LS	6,141,614	280,058	0	0	6,421,672
11B Levees and Floodwalls - Absecon Blvd Bay Closure	1.0	LS	563,477,155	25,694,558	0	0	589,171,713
11C Levees and Floodwalls - South Ocean City Bay Closure	1.0	LS	94,369,925	4,303,269	0	0	98,673,194
13 Pumping Plant	1.0	LS	14,280,912	651,210	0	0	14,932,121
13A. Pumping Plant - Absecon Blvd Bay Closure	1.0	LS	10,710,684	488,407	0	0	11,199,091

Labor ID: Region 1 EQ ID: EP16R01 Currency in US dollars TRACES MII Version 4.4

Description	Quantity	UOM	ContractCost	Escalation	Contingency	SIOH	ProjectCost
13B. Pumping Plant - South Ocean City Bay Closure	1.0	LS	3,570,228	162,802	0	0	3,733,030
15 Floodway Control and Diversion Structures	1.0	LS	131,682,717	6,004,732	0	0	137,687,449
15A Floodway Control and Diversion Structures - Absecon Blvd Bay Closure	1.0	LS	103,770,723	4,731,945	0	0	108,502,668
15B Floodway Control and Diversion Structures - South Ocean City Bay Closure	1.0	LS	27,911,994	1,272,787	0	0	29,184,781
18 Cultural Resource Preservation	1.0	LS	33,113,047	1,509,955	0	0	34,623,002
18A.86 Identification, Data Analysis and Reports	1.0	LS	33,113,047	1,509,955	0	0	34,623,002
19 Buildings Grounds and Utitities	1.0	LS	207,497,841	9,461,902	0	0	216,959,743
19A Buildings Grounds and Utitities - Central Region	1.0	LS	207,497,841	9,461,902	0	0	216,959,743
Southern Region - Measure 5A	1.0	LS	2,002,274,886	91,303,735	0	0	2,093,578,621
01 Lands and Damages	1.0	LS	217,716,164	9,927,857	0	0	227,644,021
01 Lands and Damages	1.0	LS	217,716,164	9,927,857	0	0	227,644,021
19 Buildings Grounds and Utitities	1.0	LS	1,784,558,722	81,375,878	0	0	1,865,934,600
19A Buildings Grounds and Utitities - South Region	1.0	LS	1,784,558,722	81,375,878	0	0	1,865,934,600

Labor ID: Region 1 EQ ID: EP16R01 Currency in US dollars TRACES MII Version 4.4

Description	DirectLabor	DirectEQ	DirectMatl	DirectSubBid	DirectCost	CostToPrime	ContractorOwnCost
Contractor Indirect Summary							
Prime	136,236,028	25,573,315	267,387,072	2,837,075,866	3,266,272,280	3,266,272,280	4,247,456,392
Service Subcontractor	1,199,303	173,584	79,105	0	1,451,992	1,829,092	1,829,092
Construction Subcontractor	7,314,538	2,283,052	1,593,792	93,120	11,284,502	14,413,265	14,413,265
Prime - No Markups	0	0	0	4,879,952,483	4,879,952,483	4,879,952,483	4,879,952,483

Currency in US dollars

ENGINEERING APPENDIX
LIFE SAFETY RISK ASSESSMENT

NEW JERSEY BACK BAYS
COASTAL STORM RISK MANAGEMENT
FEASIBILITY STUDY

PHILADELPHIA, PENNSYLVANIA

APPENDIX B.6

August 2021

NJBB
NEW JERSEY BACK BAYS STUDY

U.S. Army Corps of Engineers
Philadelphia District

THIS PAGE HAS BEEN INTENTIONALLY LEFT BLANK

Table of Contents

List of Figures

List of Tables

B-6) LIFE SAFETY RISK ASSESSMENT

USACE recognizes that risks to human life are a fundamental component of all flood risk management studies and must receive explicit consideration in the planning process. Current USACE guidance (PCB 2019-4, ECB 2019-03, ECB 2019-15, and the January 2021 Policy Directive – Comprehensive Documentation of Benefits in Decision Documents) on risk assessments in planning studies specifies how studies should be performed on new or existing dams and levees. This risk assessment's purpose is to make sure that the feasibility level designs follow the four Tolerable Risk Guidelines:

a. TRG 1 – Understanding the Risk
b. TRG 2 – Building Risk Awareness
c. TRG 3 – Fulfilling Daily Responsibilities
d. TRG 4 – Actions to Reduce Risk

While all of these guidelines are important, TRGs 1 and 4 are critical to Planning studies. The risk assessment below is the first step to Understanding the Risk (TRG 1) of the proposed features and makes recommendations on changes that could Reduce the Risk (TRG 4).

An additional benefit of the risk assessment is the identification of areas of concern in the proposed design that may require extra attention during design or changes to design to ensure minimal risk to the public.

For this study, the life safety risk consideration was accomplished by performing an abbreviated Life Safety Consequence Assessment and a feasibility screening level Potential Failure Mode Assessment.

As part of the life safety analysis the three base alternatives in the final array were evaluated: Future Without Project (FWOP); a Perimeter Plan, including levees, floodwalls, etc., (PP); and a Storm Surge Barrier (SSB), which also include levees and floodwalls. The Central Reach was the focus of this assessment with 48,655 properties in this reach. Only residential structures (45,291) were used in this screening, for the simplified evaluation and as it is likely businesses would be closed during the storm.

Project Summary

The objective of the New Jersey Back Bays (NJBB) Coastal Storm Risk Management (CSRM) Feasibility Study is to investigate CSRM problems and solutions to reduce damages from coastal flooding that affects population, critical infrastructure, critical facilities, property, and ecosystems.

The Atlantic Coast of New Jersey is fronted by an effective Federal CSRM program (USACE, 2013). However, the NJBB region currently lacks a comprehensive CSRM program. As a result, the NJBB region experienced major impacts and devastation during Hurricane Sandy and subsequent coastal events, thus damaging property and disrupting millions of lives owing to the low elevation areas and highly developed residential and commercial infrastructure along the coastline.

The Central Region extends from Little Egg Inlet south to Corson Inlet, with an area of 312 square miles and all or portions of 21 municipalities in Atlantic and Cape May Counties (**Figure 65**).

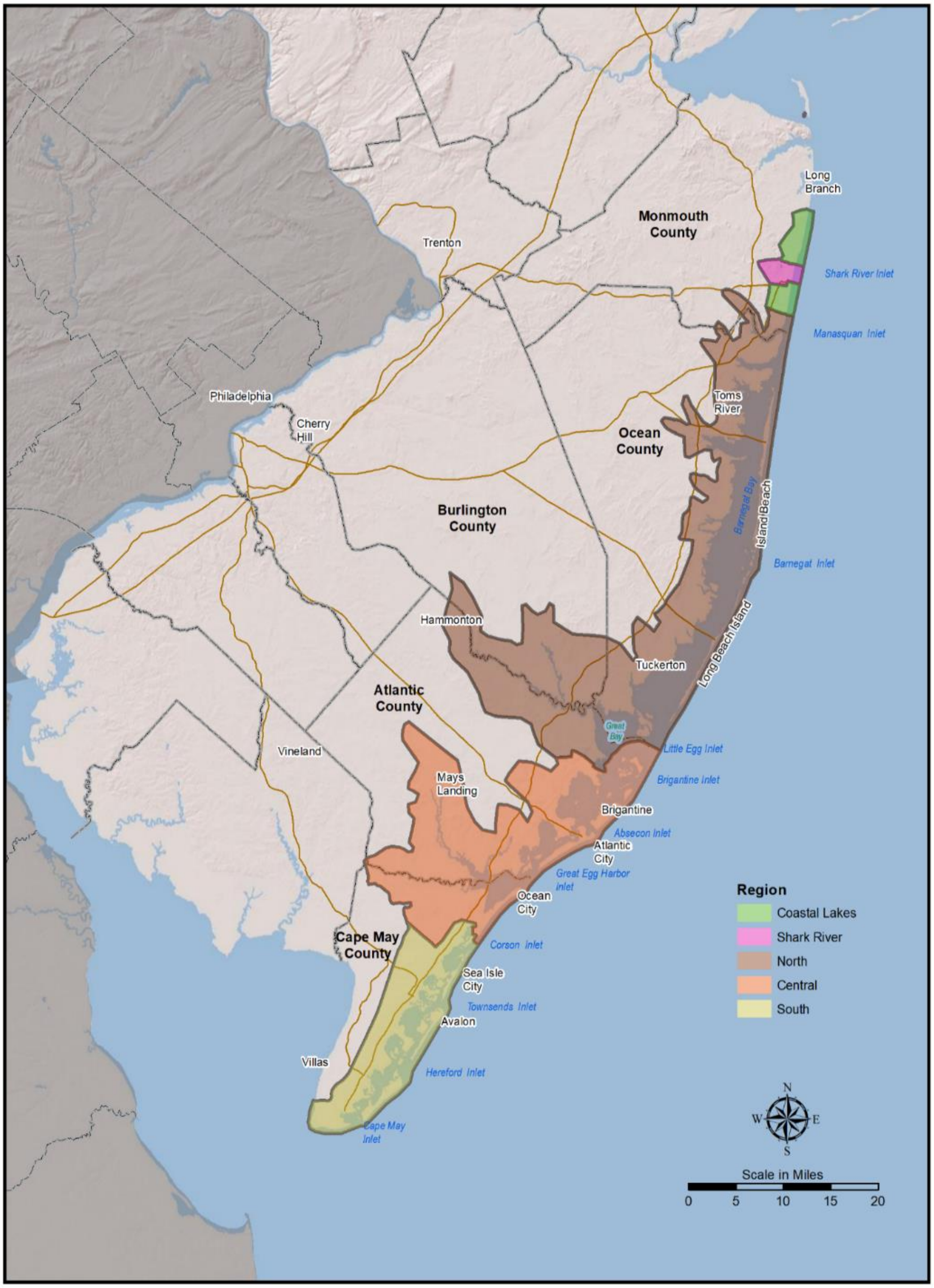


Figure 1: Multiple Reaches of Proposed Project Plan

The ocean shoreline length of this region is about 27 miles and includes five tidal inlets: Little Egg, Brigantine, Absecon, Great Egg, and Corson. The relatively short distance between inlets compared to those of the North Region makes the back bays of this reach susceptible to relatively higher 1% ACE storm surge elevations.

The back bay shorelines of the barrier islands are essentially fully developed with medium density residential and business infrastructure. However, the western (mainland) shorelines of the Central Region are significantly less densely developed than some of the other reaches in the study.

Alternatives: Full descriptions of the plans are in the main report, but the summaries are presented here:

The Future without Project alternative is the no action plan. The only change from the current conditions would be sea level rise between now and 2080. Existing shoreline protection projects would remain in place and protect from ocean side flooding.

The Perimeter Plan (PP) utilizes levees and floodwalls on the bay side of the barrier islands, which would tie into the existing shoreline protection projects (**Figure 66**).

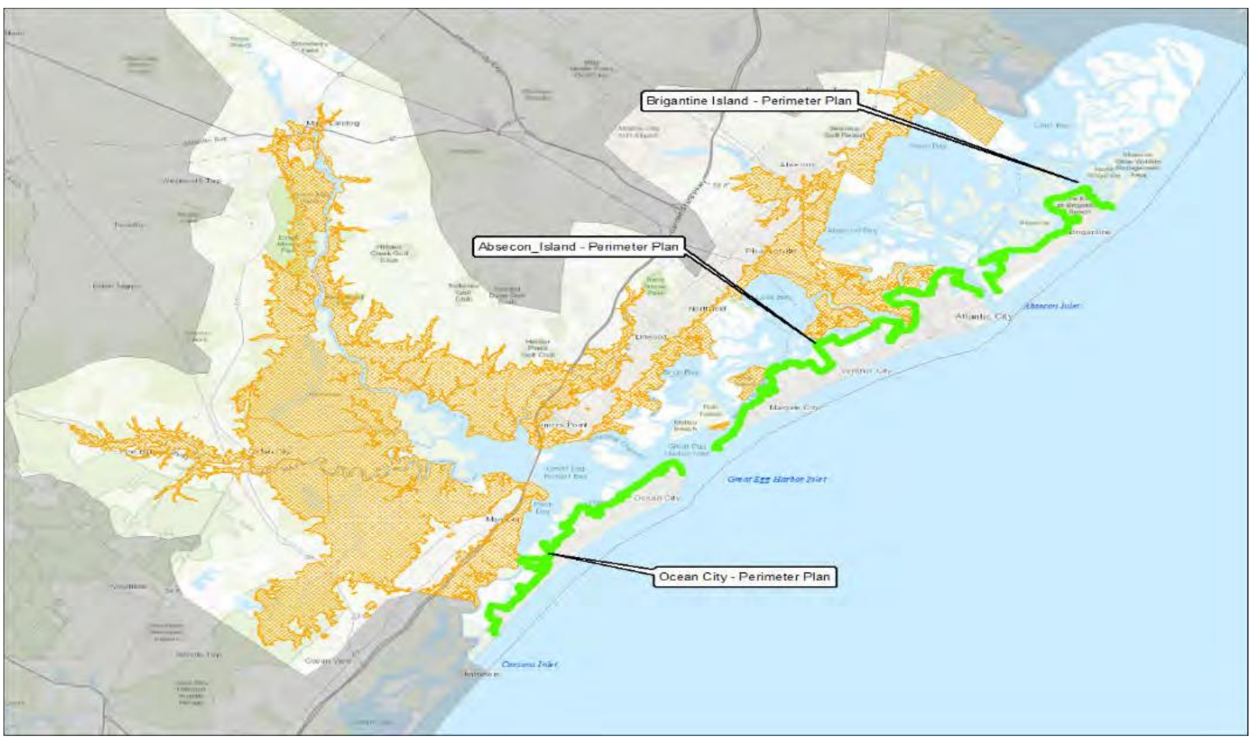


Figure 2: Possible Perimeter Plan Layout

The Storm Surge Barrier Plan (SSB) utilizes gated barriers to prevent storm surge from entering the inlets and bays, likely using levees or floodwalls to tie into the existing shoreline protection projects (**Figure 67**).

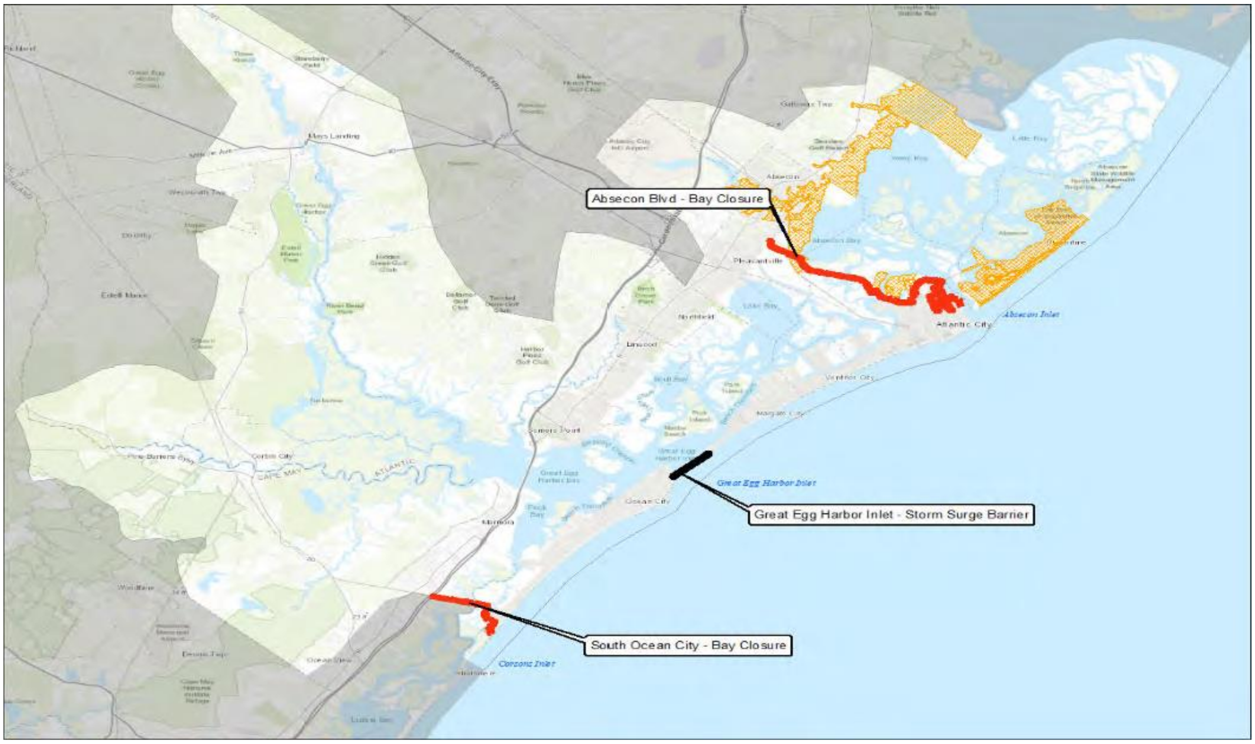


Figure 3: Possible Storm Surge Barrier Layout

Consequences

While no formal life loss modeling (LifeSim or FIA) was performed, a screening level life loss assessment was evaluated based on structures protected, building first floor elevations, number of stories, historic evacuation rates, and modeled flood depths. Hurricane Sandy evacuation rates were used for this analysis, where 42.5% of the population in the hazard area evacuated before the storm. Due to the simplistic evaluation, evacuation time was not considered. It was assumed that anyone who evacuated before the storm made it to safety, and no one evacuated during the storm or during subsequent failure warnings.

Multiple elevations for the lines of protection were considered for the study and the TSP will be optimized to maximize project benefits. For the purposes of this life loss study, an Annual Exceedance Probability (AEP) of 0.01 was used for the storm event and the level of protection for the alternatives. By matching the AEP of the storm event and the level of protection, the difference in water levels between the ocean level and the protected side of project will be the largest possible. This largest water level difference will provide the largest difference in life loss between the failure and non-failure conditions of the project (**Table 49**). That largest difference is considered to have the greatest risk for life loss.

Table 1: Total Number of Residential Structures Impacted Based on Exceedance Probability (Depths at First Floor Levels)

AEP	No Water	0-2ft Depth	2-6ft Depth	>6ft Depth	Total
0.1	30,812	12,598	1,881	-	45,291
0.5	18,109	12,152	14,964	-	45,291
0.01	12,991	9,221	22,446	633	45,291
0.005	10,483	5,401	24,902	4,505	45,291
0.002	8,308	2,990	18,323	15,670	45,291

Population at Risk (PAR)

The population at risk is the number of people who would likely be in this reach during a storm event. This area of New Jersey has a high number of long term and weekly rentals that may or may not be occupied at the time of a storm. For this preliminary assessment, the PAR was calculated by taking the residences in this area and multiplying them by the average number of inhabitants per household by locality based on the 2010 US Census (**Table 50**). The PAR for this reach was 101,548 in residential buildings.

Table 2: 2010 US Census Persons per Household by City

Municipality	Persons per Household	Municipality	Persons per Household
Absecon City	2.59	Longport Boro	2.1
Atlantic City	2.41	Margate City	2.04
Brigantine City	2.18	Northfield City	2.79
Corbin City	3	Ocean City	2.08
Egg Harbor Twp.	3	Pleasantville City	2.95
Estell Manor City	2.9	Somers Point City	2.36
Galloway Twp.	2.58	Upper Twp.	2.45
Hamilton Twp.	2.59	Ventnor City	2.31
Linwood City	2.63	Weymouth Twp.	2.3

Warnings and Warning Times

As this study is assessing protection from coastal storms, which can be predicted with some accuracy up to a few days out, warning time for evacuations before the storm are all considered to be over two hours. Due to the simplistic method being used in this assessment, timing of evacuations attempted cannot be evaluated, and everyone who attempts to evacuate before the storm is assumed to make it to a safe location.

The primary warning for a coastal storm in this evaluation is considered when the storm is forecast to make landfall. This primary warning would mobilize the group of people who are most likely to evacuate during the storm. Double warnings are considered when the line of protection (flood wall, levee, storm surge barrier) is likely to be overtopped or if the Storm Surge Barrier fails to close. This second warning would likely mobilize more of the remaining population as it indicates that additional flooding will occur. Since the water level caused by the rainfall event considered for this assessment is matched by the height of the line of protection, overtopping is not considered as the leading risk driver. No additional warning was provided for overtopping in this assessment. A mechanical failure of the storm surge barrier, where the barrier could not close prior to the storm would generate additional warnings, and the same evacuation rate of 42.5% was applied again to all flood depths evenly. This additional warning time evacuation rate would need to be revised for future assessments.

Warning times during a failure are divided into two categories for this assessment: slow failure and rapid failure. Slow failures are a result of a storm surge barrier being unable to close and flood waters slowly rising with the ocean levels. This is aided by the available storage capacity of the bay, allowing additional time to evacuate resulting in a double warning. Rapid failures are due to a breach (internal erosion of a levee, monolith failure of a flood wall, etc.) and would rapidly flood the interior protected area to match the already high ocean level with little to no warning.

Generally, if a rapid failure of a SSB plan would breach into the bay, it would also have a longer warning time due to the storage capacity of the bay. It would take some time for the bay levels to rise and then impact the now unprotected areas, instead of the rapid impact of a failure of a floodwall or levee (in the perimeter plan) immediately next to the structures inside the floodwalls and levees.

Evacuations

Determining how many people would evacuate is challenging at any level of a study, but without the Milletti and Sorenson survey, and in coastal storm conditions, it can be more challenging. During a dam or levee failure a large majority of the population is likely to evacuate, or attempt to evacuate, if there is a failure. Coastal storms are more challenging to assess because every person has their own experiences, which weigh into their decision making. Based on the results of the 2014 New Jersey Behavioral Risk Factor Survey, it was reported that 42.5% of the population in impacted areas with mandatory evacuation orders evacuated prior to Hurricane Sandy (22 October 2012). Although evacuation trends change based on experience, and people may change their evacuation plans based on Hurricane Sandy, this evacuation rate was used for this analysis and even distributed to all flood depths.

Since LifeSim was not utilized for this preliminary assessment, it is assumed that all people who attempted to evacuate were successful and that no one attempted to evacuate after the storm started unless a double warning is provided. Based on the 2014 assessment, this assumption is incorrect, as 25.3% of evacuees left during the storm. This will be assessed later in the study process with LifeSim. Vertical evacuations were utilized in this assessment when a second story was available and flooding depth would have been over two feet deep.

The population remaining after the evacuations are a subset of the PAR and are considered the Threatened Population (**Table 51**).

Table 3: Threatened Population During 0.01 AEP Event

1st Floor Water Depth	FWOP	Perimeter Protection	Storm Surge Barrier (Rapid)	Storm Surge Barrier (2x Warning)
0 ft	16,766	37,106	40,715	23,411
<2ft	11,900	12,388	11,228	6,456
2-6ft	12,517	4,547	3,514	2,021
6-13ft	548	289	31	18
>13ft	-	-	-	-
Vertical Evacuation	16,719	4,121	2,963	1,704

Life Loss

Since the basic life loss calculations utilized in this assessment do not assess people caught evacuating, the calculated life loss is likely underestimated. Additionally, the only lives lost accounted for in this assessment are directly related to exposure to flood waters. Deaths caused by associated conditions (heart attack, structure collapse, etc.) are not included. As a result of these short comings, the life loss numbers should be used as relative numbers for comparison to the other alternatives and not total life loss. This is not as accurate as a LifeSim model, which is recommended for use later in the study.

There are many factors that are used to determine fatality rates, including proximity to assistance, response capabilities, age of population, air temperature, and many others. For the purposes of the screening, fatality rates are based solely on depth of water and only include loss of life due to exposure to the water. Life loss calculations are based on the traditional fatality rate table (**Table 52**), which is based on depth of water. The fatality rates used were taken from the 2016 Jadwin Dam Issue Evaluation Study.

Table 4: Fatality Rates

1st Floor Water Depth	Probability
0-2ft	0%
2-13ft	0.02%
13-15ft	12%
>15ft	91%

Life Loss from Non-Breach

Life loss from non-breach is important because it assumes that all features of the project work according to plan. Life loss from non-breach is generally limited to locations outside of the protected area (**Table 53**). While the Non-Breach double warning is not likely to occur, it is possible and utilized to calculate incremental life loss.

Table 5: Non-Breach Life Loss by Alternative for 0.01 AEP Event

1st Floor Water Depth	FWOP	Perimeter Plan	SSB (Rapid Failure)	SSB (2x Warning)
0 ft	0.00	0.00	0.00	0.00
<2ft	0.00	0.00	0.00	0.00
2-6ft	2.50	0.91	0.70	0.40
6-13ft	0.11	0.06	0.01	0.00
>13ft	0.00	0.00	0.00	0.00
Total	2.61	0.97	0.71	0.41

Life Loss from Breach

Life loss from breach is the most commonly considered life loss by the public and is a result of loss of life due to project failure. Breach life loss is calculated in the same way as non-breach, utilizing water depths at first floor elevations. Due to the limitations of this assessment and not modeling evacuations during the storm, the loss of life of the Perimeter Plan and SSB (Rapid Failure) will match the FWOP, due to having the same final water elevations. The Storm Surge Barrier with double warning has a smaller loss of life due to the evacuations from the second warning. See **Table 54** for the Breach life loss.

Table 6: Breach Life Loss by Alternative for 0.01 AEP Event

1st Floor Water Depth	FWOP	Perimeter Plan	SSB (Rapid Failure)	SSB (2x Warning)
0 ft	0.00	0.00	0.00	0.00
<2ft	0.00	0.00	0.00	0.00
2-6ft	2.50	2.50	2.50	1.43
6-13ft	0.11	0.11	0.11	0.06
>13ft	0.00	0.00	0.00	0.00
Total	2.61	2.61	2.61	1.49

For comparison, the CDC reports Hurricane Sandy caused 40 deaths, but only 4 deaths in New Jersey were directly attributed to flooding (AEP of 0.05).

Incremental Life Loss

Incremental Life Loss is the life loss plotted on the f-N chart for a risk assessment. Incremental life loss is the breach life loss with the non-breach life loss subtracted from it. This shows the true loss of life due to a failure at the project, by not including any life loss that may occur during non-breach conditions. See **Table 55** for the Incremental Life Loss based on a 0.01 AEP event.

Table 7: Incremental Life Loss by Alternative and Depth for 0.01 AEP Event

1st Floor Water Depth	FWOP	Perimeter Plan	SSB (Rapid Failure)	SSB (2x Warning)
0 ft	0.00	0.00	0.00	0.00
<2ft	0.00	0.00	0.00	0.00
2-6ft	2.50	1.59	1.80	1.03
6-13ft	0.11	0.05	0.10	0.06
>13ft	0.00	0.00	0.00	0.00
Total	2.61	1.64	1.90	1.09

Key Limitations / Lessons Learned

- The methodology for the simplified consequence analysis seems appropriate for the level of risk assessment conducted and phase of the project. Modeled Life loss consequences were not available but will be available for the updated risk analysis during the design phase.
- Not being able to calculate successful and unsuccessful evacuations once the storm starts has an unknown impact on the results.
- The limited length of wall and levee in the SSB alternative would make almost all failures of the SSB plan fall in the double warning category.
- The SSB plan is modeled as if one gate failure floods the entire protected area. There is one storm surge barrier and two inlet gates and the impact the failure at each gate should be modeled separately.
- Breaches in the existing dune system are not included. Their inclusion in future modeling should be considered.

Conclusions

Based on the preliminary screening, the storm surge barrier has fewer non-breach lives lost than the perimeter plan, which makes sense because more residences are within the line of protection.

The perimeter plan has fewer incremental lives lost when both alternatives have one warning, but when the storm surge barrier has a double warning, which is likely the typical condition, the incremental life loss is less than the perimeter plan (**Table 56**).

Table 8: Summary of Life Loss at 0.01 AEP

.01 AEP	FWOP	Perimeter	SSB (Rapid Fail)	SSB (2x Warning)
Non-Breach	2.61	0.97	0.71	0.41
Incremental	2.61	1.65	1.90	1.09

Potential Failure Mode Assessment

While the PP and SSB have some similarities in features (flood walls, levees), it is the locations of these features and the Storm surge barrier itself that set them apart. The locations will impact the consequences more than the PFMs. The similarities in features allow for one discussion of the failure modes with separate conclusions at the end. The PP wraps around the protected side of the island and is located between the bay and the protected area. The SSB is located at the ocean side of the barrier island and the storm surge barrier closes to keep storm surge out of the bay, keeping the bay water elevation much lower than the ocean level. The existing dune systems are a key piece of the line of protection but are not included in this risk assessment. If any or multiple dunes failed, flooding of the protected area would occur. The existing dunes are the largest unknown of the system because many were not built by USACE. While the dunes need to be considered as a non-project segment to the protection system, at this point of the evaluation, the design and condition of all of the segments of dunes is unknown and cannot be assessed. Since both alternatives use the existing dunes, this unknown risk is carried in both alternatives and will not impact this assessment.

Brainstorming PFMs

The Perimeter Plan consists of floodwalls and levees that would line the oceanward side of the bays to prevent water from flowing through the bay and flooding the barrier islands from the bay side. For this evaluation the terms “levee” and “embankment” are interchangeable. The brainstorming session identified 25 PFMs (**Table 57**) spanning the following categories of performance: embankment and foundation internal erosion, embankment stability, embankment erosion, closure systems, interior drainage, and floodwall stability. For the brainstorming effort, consideration was given to the current design described in the Feasibility Report, limited knowledge of the subsurface, likely levee materials, locations of potential construction difficulties, and likely operations and maintenance issues that could occur over time. For the purposes of this evaluation, other than mechanical failure of the pumps, any failure mode that could be attributed to a pump station (sliding, global instability, leakage around a conduit, etc) would be equal to or less likely than the flood wall PFMs of the same nature, due to the robustness of the pump station. Therefore, the items listed below as floodwall have also been considered for the pump station in this evaluation. The chart below lists these brainstormed PFMs sorted by the affected feature:

Table 9: Brainstormed Potential Failure Modes

#	Plan	Potential Failure Mode	Feature
1	Both	Overturning of the floodwall	Wall
2	Both	Sliding of the floodwall	Wall
3	Both	Seepage and piping through the foundation (wall)	Wall
4	Both	Seepage and piping through the foundation (levee)	Levee
5	Both	Concentrated Leak erosion at the foundation/ embankment interface	Levee
6	Both	Concentrated Leak erosion at the abutment (dune)/ embankment/wall interface	Levee
7	Both	Overtopping of wall, scour at the toe – wall failure	Wall
8	Both	Overtopping of the levee, crest erosion – levee failure	Levee
9	Both	Global stability of the floodwall	Wall
10	Both	Backward erosion piping of the embankment	Levee
11	Both	Scour at embankment/floodwall interface	Wall/Levee
12	Both	Concentrated leak erosion along conduit (through embankment)	Levee
13	Both	Concentrated leak erosion along conduit (through wall) - Sealant	Wall
14	Both	Differential settlement floodwall monoliths	Wall
15	Both	Differential settlement at the embankment /floodwall/ dune interfaces	Wall/Levee
16	Both	Obstructed conduit restricts exit flow	Conduits
17	Both	Obstructed conduit allows backflow into the protected area	Conduits
18	Both	Failure of the existing pipes beneath the levee or floodwall	Levee/Floodwall
19	Both	Overtopping flows scour embankment at the wall/ embankment/ dune interface	Wall/Levee
20	PP	Failure of road closure (operational/not closed)	Closure
21	PP	Failure of road closure (closure fails)	Closure
22	SSB	Failure of SSB to close	SSB
23	SSB	Premature opening of SSB	SSB
24	SSB	Overtopping of SSB	SSB
25	Both	Pump Station Failure (Operational – Not Pumping)	Pump Failure

Evaluating PFMs

Many of the brainstormed PFMs are easily avoidable with typical design features, construction Quality Assurance (QA), or a standard Emergency Action Plan (EAP). A follow up and more formal risk assessment will occur during the design process, when more design decisions have been made and the level of protection has been optimized. A formal life loss assessment (using FIA or LifeSim) will also be performed at this time.

For this screening level assessment, qualitative methods were used to determine life loss likelihoods if that failure method occurred (**Table 58**). This did not take into account the probability of failure from this level of design. A "Low" likelihood represents a slow rise of water AND providing large amounts of warning time; a "Moderate" likelihood represents EITHER slow failure rate OR large amounts of warning time; "High" likelihood represents rapid failure rate AND little to no warning time. Uncertainty between levels were given hyphenated ratings. The ease of prevention and the life loss likelihood were evaluated, and a decision was made if further evaluation was required at this point. Even if the potential for failure was high, if the evaluation states that it is a typical design consideration, no additional evaluation is required at this stage.

Table 10: Evaluation of Potential Failure Modes

#	PFM Description	Evaluation	Likelihood of Consequences in case of Failure	Additional Risk Evaluation Required at this Stage?
1	Overturning of the floodwall	Analyzed in design phase, and typically designed against. Not enough information at feasibility phase (typical design considerations)	High	NO
2	Sliding of the floodwall	Analyzed in design phase, and typically designed against. Not enough information at feasibility phase (typical design considerations)	High	NO
3	Seepage and piping through the foundation (wall)	While likely on a sand foundation, common designs include sheeting to depth or bedrock to minimize seepage.	Low-Mod	NO
4	Seepage and piping through the foundation (levee)	Subsurface data will be analyzed in design phase, not enough information at feasibility phase (typical design considerations)	Low-Mod	NO

#	PFM Description	Evaluation	Likelihood of Consequences in case of Failure	Additional Risk Evaluation Required at this Stage?
5	Concentrated Leak erosion at the foundation/ embankment interface	Seepage key and sheeting will be designed/ analyzed in design phase, not enough information at feasibility phase (typical design considerations)	Low-Mod	NO
6	Concentrated Leak erosion at the abutment (dune)/ embankment/wall interface	Seepage key and connections will be designed/ analyzed in design phase, not enough information at feasibility phase (typical design considerations)	Low-Mod	NO
7	Overtopping of wall, scour at the toe, wall failure	Scour Pad will be designed/ analyzed in design phase, not enough information at feasibility phase (typical design considerations)	Low	NO
8	Overtopping of the levee, crest erosion, levee failure	The levee height has not yet been optimized. Appropriate armoring and vegetation will be incorporated in design phase to minimize scour (typical design considerations)	Moderate	NO
9	Global stability of the floodwall	Wall stability will be designed/ analyzed in design phase, not enough information at feasibility phase (typical design considerations)	Mod-High	NO
10	Backward erosion piping of the embankment	A seepage analysis will be performed in design phase, with the parameters of the soil being used. Appropriate measures will be incorporated in the design of the levee to minimize backward erosion piping. There is not enough information at feasibility phase (typical design considerations)	Low-Mod	NO
11	Scour at embankment/floodwall interface	The wall will be keyed into the embankment. The length of embedment will be designed/ analyzed in design phase, not enough information at feasibility phase (typical design considerations)	Low	NO

#	PFM Description	Evaluation	Likelihood of Consequences in case of Failure	Additional Risk Evaluation Required at this Stage?
12	Concentrated leak erosion along conduit (through embankment)	Seepage and piping along the outside of a conduit through or under the embankment is a common failure mode. Proper zoning of materials and material placement will minimize the risk of this failure mode, which will be designed/ analyzed in design phase, not enough information at feasibility phase (typical design considerations)	Moderate	NO
13	Concentrated leak erosion along conduit (through wall) - Sealant	A leak in the conduit/flood wall interface may leak water, but it will not lead to a failure of the flood protection system	Low	NO
14	Differential settlement between floodwall monoliths	Settlement will be analyzed in design phase, not enough subsurface information at feasibility phase (typical design considerations)	Low-Mod	NO
15	Differential settlement at the embankment /floodwall /dune interfaces	Settlement of the wall will be evaluated during design, but extra attention is required at the interface between the wall and levee	Low-Mod	NO
16	Obstructed conduit restricts exit flow	Obstructed flap gates or conduits are an Operations and Maintenance concern which can lead to flooding of the protected area, but the buildup of water would be slow allowing adequate evacuation time. Potential Failure Mode 17 is of greater concern for a more rapid failure.	Low	NO
17	Obstructed conduit allows backflow into the protected area	Backflow through an open conduit would force interior drainage to back up and water levels on the inside of the protected area to approach levels on the outside. However, flow into the protected area would be throttled by the size of the conduit.	Low	NO

#	PFM Description	Evaluation	Likelihood of Consequences in case of Failure	Additional Risk Evaluation Required at this Stage?
18	Failure of the existing pipes beneath the levee or floodwall	At this point the location of floodwalls and levees are uncertain so the locations of any pipes beneath the line of protection is also uncertain. Any conduits passing beneath the line of protection will be camera inspected prior to construction, and on a 5-year cycle throughout the life of the project. While this is an important failure mode, regular Operations and Maintenance should minimize the impact of this failure mode	Low-Mod	NO
19	Overtopping flows scour embankment at the wall /embankment /dune interface	This PFM would be a subset of PFM 8, overtopping of the embankment	Low	NO
20	Failure of road closure gate (operational/not closed)	At this point it is uncertain where road closures will be located on the final design, however the inability to close a road closure gate would lead to a slow rise of water with adequate warning.	Low	NO
21	Failure of road closure gate (closure fails)	At this point it is uncertain where road closures will be located on the final design, however the inability to close a road closure gate would lead to a slow rise of water with adequate warning.	Moderate	NO
22	Failure of SSB to close	Failure of a storm surge barrier to close would allow water to enter and start to fill the bay at the same rate as sea level rise during the storm. This failure would be known early in the storm and additional evacuations could be performed.	Low	NO

#	PFM Description	Evaluation	Likelihood of Consequences in case of Failure	Additional Risk Evaluation Required at this Stage?
23	Premature opening of SSB	While premature opening of the SSB would be the most damaging failure mode of the SSB, the inflow of water would have to fill the bay prior to impacting the protected area. This would allow some additional time for evacuations. Many gates systems have a default position as closed, so if the power went out, the gates would not open.	Low	NO
24	Overtopping of SSB	Overtopping of the SSB would have water flowing into the bay, which would provide additional storage and time before flood waters reached the protected area. Additionally, many SSB are designed for overtopping to occur with no damage to the barrier.	Low	NO
25	Failure of Pump Station (Operational – Not Pumping	Failure of a pump station to pump would result in flooding from the low-lying areas and slowly spreading uphill. While economic damages are likely, life loss is unlikely to occur due to the slow and predictable water movement and levels. Typical pump station design include redundancy in the system, with more pumping capacity available than needed. Additionally, pumps could be rented and used to pump the area if needed.	Low	NO

While none of the failure modes considered stood out as risk drivers at this phase of the feasibility study, these failure modes should be considered during design of the project and will be re-evaluated once the design is more substantial.

Since the storm surge barrier and inlet barriers are mechanical closure gates, a fault tree assessment will be performed at the next risk assessment on each gate. The assessment is typical for mechanical gates and includes frequency of operation and likelihood that each individual component will work as designed. While a probability of failure is unable to be reasonably determined for the SSB at this time, it will be higher than a levee or floodwall based on the multiple moving parts.

Similar to the storm surge barrier, a fault tree assessment should be performed for the typical pump station design at the next risk assessment. This would allow for a better understanding of the probability of failure for the pumps,and would help with Operations and Maintenance recommendations for replacing and restocking parts for replacing parts before they failed.

Typical Risks

Since the designs and locations of the alternatives are still in the feasibility phase, a full risk assessment cannot be performed. The two structural alternatives are similar in features as the storm surge barrier plan would likely include flood walls or levees to tie into the existing dune. However, the increase in risk caused by the length of the line of protection on the perimeter plan would not match the increase in risk based on the storm surge barriers.

For comparative purposes, an appropriately designed levee and floodwall have relatively low risks based on USACE's Levee Screening Tool and are in the 10^{-5} range based on limited information. Having subsurface data and full design details would reduce the risk further during a quantitative risk assessment. The storm surge barrier would likely be one to two orders of magnitude higher than the perimeter plan.

Additional Recommendations

- While no risks were outstanding during the risk assessment, designing the features with the idea that inspection, operations, and maintenance are going to require access to both sides of the line of protection is a way to reduce risk.
- Public awareness is an effective way to reduce risk (TRG 4). Having the public aware of the project and having local EMAs familiar with the EAP and emergency exercises would help increase evacuation effectiveness and reduce life loss.
- Consider phasing construction based on concerns other than environmental window availability. Consider earlier construction for reaches with anticipated higher settlement, and/or ways to minimize embankment flaws created from work stoppages. While considering these alternative sequences, it is understood that the environmental windows are the top priority in construction sequencing.

It is also recommended that when more rigorous and quantitative risk assessments are conducted in the D&I phase, the full list of 25 PFMs be consulted as part of the process, not just the Storm Surge Barrier. One reason this is important is that future design will likely affect the risk from one or more PFMs identified here (such as design of underseepage control measures). Another reason is the qualitative nature of this assessment limits the impact of multiple PFMs added together on the full project risk. Further brainstorming of PFMs should also occur in future risk assessments since new design or geological issues may develop in the meantime.

Key Limitations / Lessons Learned

- The methodology for the simplified potential failure mode assessment seems appropriate for the feasibility level study. It identifies the potential for risks but cannot fully quantify the risks until more information is available on the design and existing conditions.
- The existing dunes that will be incorporated into the oceanward line of protection should be investigated to determine the existing conditions, design level of protection, and subsurface conditions. The dunes will likely be the weakest link in the system due to the frequent wave impact in normal conditions.
- Following brainstorming, the Levee Safety Tool was used to ensure all common levee PFMs were considered by the team. Although no new PFMs were added, it provided some assurance that none had been missed.

Conclusions

At the feasibility phase of the project, the screening level risk assessment did not identify any potential failure modes that would favor one alternative significantly over the other or that would lead to elimination of an alternative. Due to the multiple components of the Storm Surge Barrier, the probability of failure would likely be one to two orders of magnitude higher than the static features of the perimeter plan. Additional information, including modeled life loss evaluations, subsurface investigations, and advancing design will allow for a more thorough and quantitative evaluation.