US Army Corps of Engineers®
New York District

Passaic River, New Jersey

Passaic River Tidal General Reevaluation Report

Lower Passaic River, New Jersey

Appendix J
Engineering and Design

DRAFT

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1 INTRODUCTION

The Engineering and Design Appendix presents the supporting technical information used in updating the authorized design of features of the Passaic River, New Jersey, Tidal Flood Risk Management Project presented in the General Reevaluation Report (GRR) as well as the Tentatively Selected Plan (TSP), the Locally Preferred Plan (LPP). The New York District Corps of Engineers (NYD) produced a Draft General Design Memorandum (GDM) in 1995 and the first phase of a GRR for the entire Passaic River Watershed in 2013, both of which identified hurricane/storm surge/tidal protection to help manage flood risks in portions of Harrison, Kearny and Newark, New Jersey. The three "tidal" levees and floodwalls have since been separated out from the Main Passaic Watershed GRR and have been identified for separate funding and analysis as part of a series of Authorized but Unconstructed (ABU) Hurricane Sandy-related projects. The Harrison, Kearny and Newark tidal levees were analyzed at a GRR level of study making full use of the data acquired in 1995 and 2013, as well as the latest hydrologic, hydraulic, topographic and structural information.

The ABU Hurricane Sandy-related project was evaluated by comparing design heights to each other at a preliminary level of detail to compare costs and benefits to determine the optimum design height. The alternatives analyzed included the 1995 draft GDM elevation and lines of protection (LOP) with crest elevations 2 and 4 feet above the GDM elevation, as well as a smaller plan set back from the shoreline that provided flood risk management for the interior of the City of Newark. Preliminary typical levee and floodwall cross-sections were developed to estimate comparative quantities and costs.

After consideration of the potential Hazardous, Toxic, and Radioactive Waste (HTRW) impacts, potential environmental impacts, and the challenges associated with floodwall construction adjacent to several Superfund sites, the New Jersey Department of Environmental Protection (NJDEP), the non-Federal partner, selected a smaller alternative, known as the "Flanking Plan", as the LPP, which includes floodwall segments set back from the coastline.

This appendix provides the detailed cost estimate for the TSP, the LPP. The plan will provide flood risk management along portions of the Passaic River, and includes parts of Newark Bay in New Jersey.

A general project location map of the Passaic River Tidal Project Area (the ABU Project) is provided in Figure 1, which shows the 1995 line of protection alignment. The LPP is shown in Figure 2.

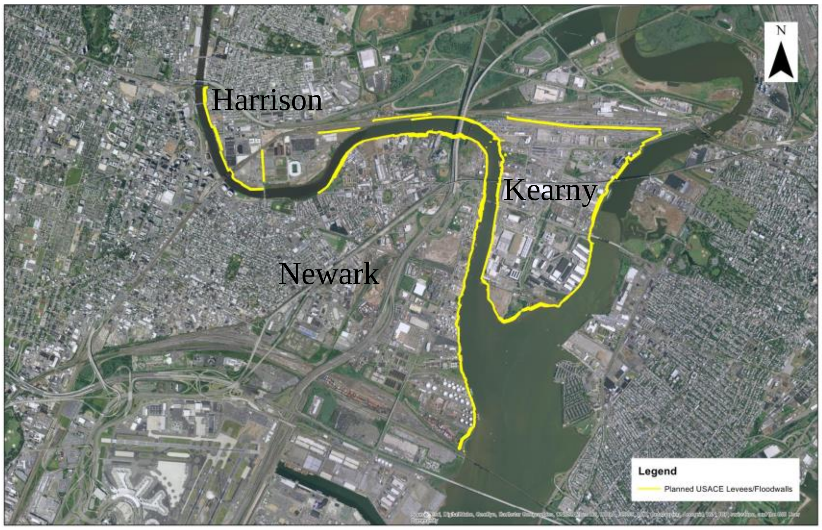

Figure 1: Passaic River Tidal Project Area – 1995 GDM Alignment

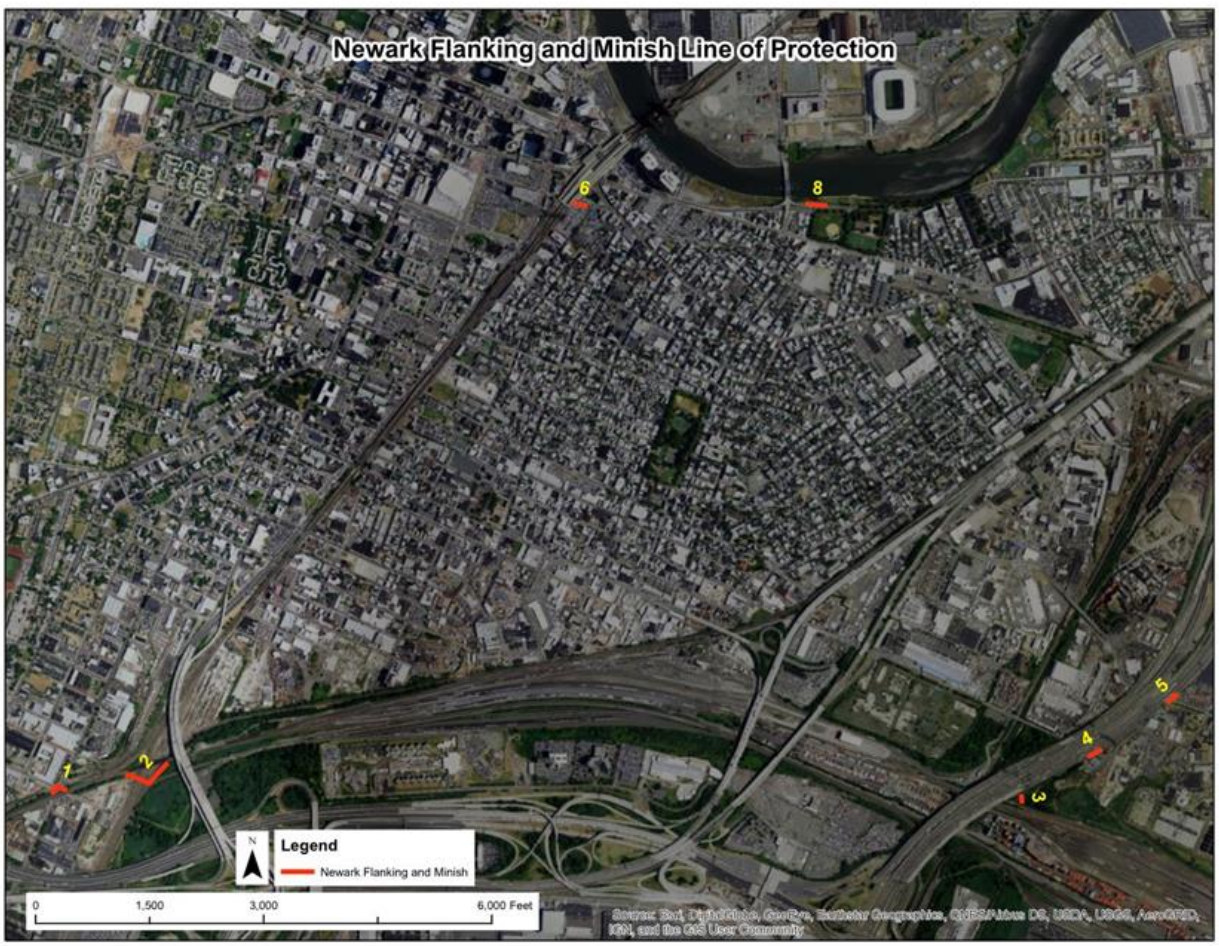

Figure 2: Passaic River Tidal Project, Locally Preferred Plan

1.1 Storm Frequency

The probability of exceedance describes the likelihood of a specified flood or storm event being exceeded in a given year. There are several ways to express the annual chance of exceedance (ACE) or annual exceedance probability. The ACE is expressed as a percentage. An event having a 1 in 100 chance of occurring in any single year would be described as the 1 percent ACE event. This is the current accepted scientific terminology for expressing chance of exceedance. The annual recurrence interval, or return period, has historically been used by engineers to express probability of exceedance. For this document, due to the incorporation of historic information, both references may be used. Examples of equivalent expressions for exceedance probability for a range of ACEs are provided in Table 1.

Table 1: Annual Chance of Exceedance

ACE (as percent)	ACE (as probability)	Annual recurrence interval
50%	0.5	2-year
20%	0.2	5-year
10%	0.1	10-year
5%	0.05	20-year
2%	0.02	50-year
1%	0.01	100-year
0.5%	0.005	200-year
0.2%	0.002	500-year

1.2 Survey and Datum

The latest topographic data was collected following the impact of Hurricane Sandy in 2012 and is based on Light Detection and Ranging (LiDAR) data. Previous analyses and designs are based on the National Geodetic Vertical Datum of 1929. The conversion factor from National Geodetic Vertical Datum (NGVD) to North American Vertical Datum (NAVD) is approximately -1.1 feet; therefore, the 1995 GDM design elevation of 14.9 feet NGVD is converted to 13.8 feet NAVD. For ease in analysis, computation and discussions, the 1995 GDM design elevation is rounded to 14 feet NAVD.

2 PROJECT PURPOSE

The purpose of the Passaic River, New Jersey, GRR is to document the development of the updated cost estimates, plan formulation and environmental impacts of the tidal portion for the Passaic River Flood Risk Management Project and determine if storm risk management in the study area is still in the federal interest.

3 PROJECT HISTORY

Flooding in the Passaic River Basin has been studied extensively over the past century at both the state and federal level. The State of New Jersey has produced numerous documents containing a variety of recommendation advancing flood storage as key to solving the problem in the Passaic River Basin. None of the local solutions were implemented upstream such that would reduce storm surge flooding in the tidal portion of the basin.

In 1936, the Corps of Engineers first became involved in the basin flood control planning effort as a direct result of the passage of the Flood Control Acts. Since that time, the Corps has issued reports containing recommendations eight times since 1939, the latest being 1995. Due to the lack of widespread public support, none of the basin-wide plans were implemented. Opposition was based on concerns of municipalities and various other interests throughout the basin.

The latest Feasibility Report was NYD's 'General Design Memorandum, Flood Protection Feasibility Main Stem Passaic River, December 1987," which was the basis for project authorization. This project at the time included a system of levees and floodwalls with associated closure structures, interior drainage and pump stations within the tidal portion of the Passaic River Basin.

Since authorization, the planning and design efforts were conducted and presented in NYD's "Draft General Design Memorandum (DGDM), Passaic River Flood Damage Reduction Project, Main Report and Supplement 1 to the Environmental Impact Statement, September 1995, and associated appendices." These efforts affirmed that the authorized project remained appropriate for the Passaic River Basin based on the problems, needs, and planning and design criteria at the time.

Since 1996, the State has requested that the Corps proceed with three element of the Passaic River Basin project: the preservation of natural storage, the Joseph G. Minish Waterfront Park, and the Harrison portion of the tidal project area. In 2007, the NYD prepared a draft Limited Reevaluation Report (LRR) to reaffirm federal interest in construction of the tidal portion in Harrison.

Following the impact of Hurricane Sandy on the region in 2012, the NYD initiated a general reevaluation of the entire Passaic River Basin project to reaffirm project viability and move to construction. Due to the lapse of time since the last study and the current emphasis on design resiliency when considering sea level change, the project was evaluated at the design height and two additional design heights of +2 feet and +4 feet. Due to potential challenges presented by HTRW and Superfund site proximity to the authorized alignment, an additional alternative, the smaller Flanking Plan, was also considered.

4 NED PLAN DESCRIPTION

The Passaic Tidal study was divided into six components as noted below. The components are shown in Figure 3:

1) Harrison 1 – The area of Harrison included in the 1995 alignment.
2) Harrison 2 – Additional protection in Harrison which includes Red Bull Arena and the Path Service Station. NOTE: This reach is eventually screened out as not economically viable and not included final plan. It is included in the cost engineering documentation for completeness.
3) Kearny – Also referred to as Kearny Point, this includes all of Kearny Point peninsula to the northern rail yard.
4) Newark – This area includes the areas of Newark subject to flooding from east and was part of the 1995 alignment.
5) Minish – This component includes a line of protection along Minish Park, providing flood risk management for ‘inland’ Newark.
6) Newark Flanking – This component includes floodwall and closure gates to prevent flooding of the South Ironbound area of Newark from flood water flanking the line of protection north of Newark Liberty Airport.

Following plan formulation, the Harrison-2 component was screened out and the optimum (NED) design elevation determined to be a line of protection at 18 feet NAVD. The final NED Plan is shown in Figure 4.

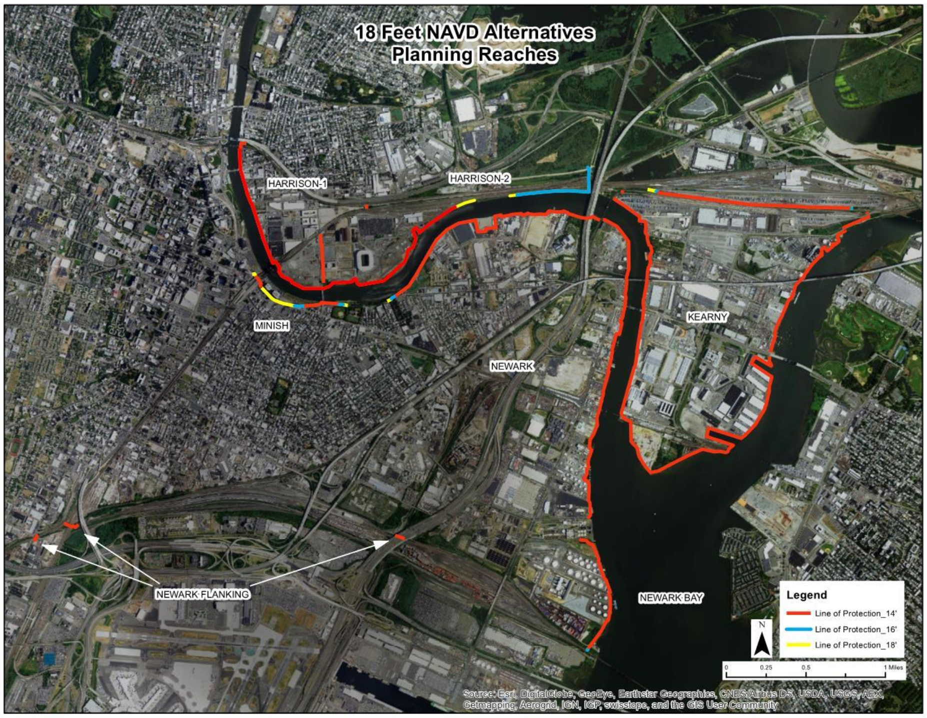

Figure 3: Passaic Tidal Project Reaches

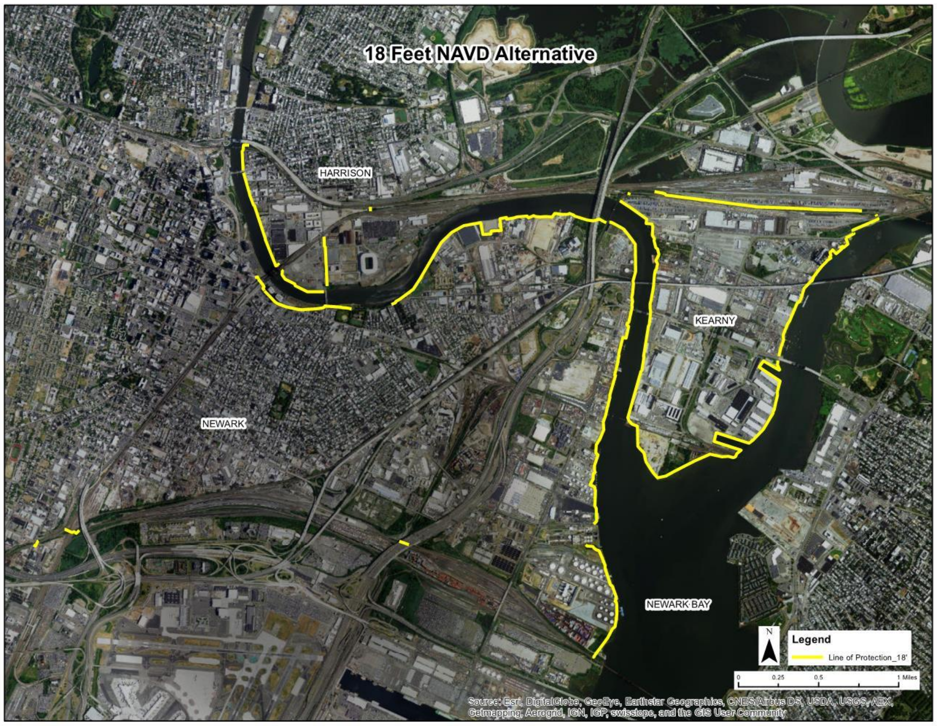

Figure 4: Passaic Tidal NED Plan

5 TENTATIVELY SELECTED PLAN

The Passaic Tidal TSP is the LPP and consists of concrete floodwalls and gates along three reaches as described below. The design elevation is 14 feet NAVD. The typical ground elevation is 6 to 10 feet NAVD. For areas with a wall height of four feet or less, the wall is a concrete I-wall; for areas where the wall is greater than four feet, the wall is a pile-supported, concrete T-wall. The project reaches are shown in Figure 5 and described below.

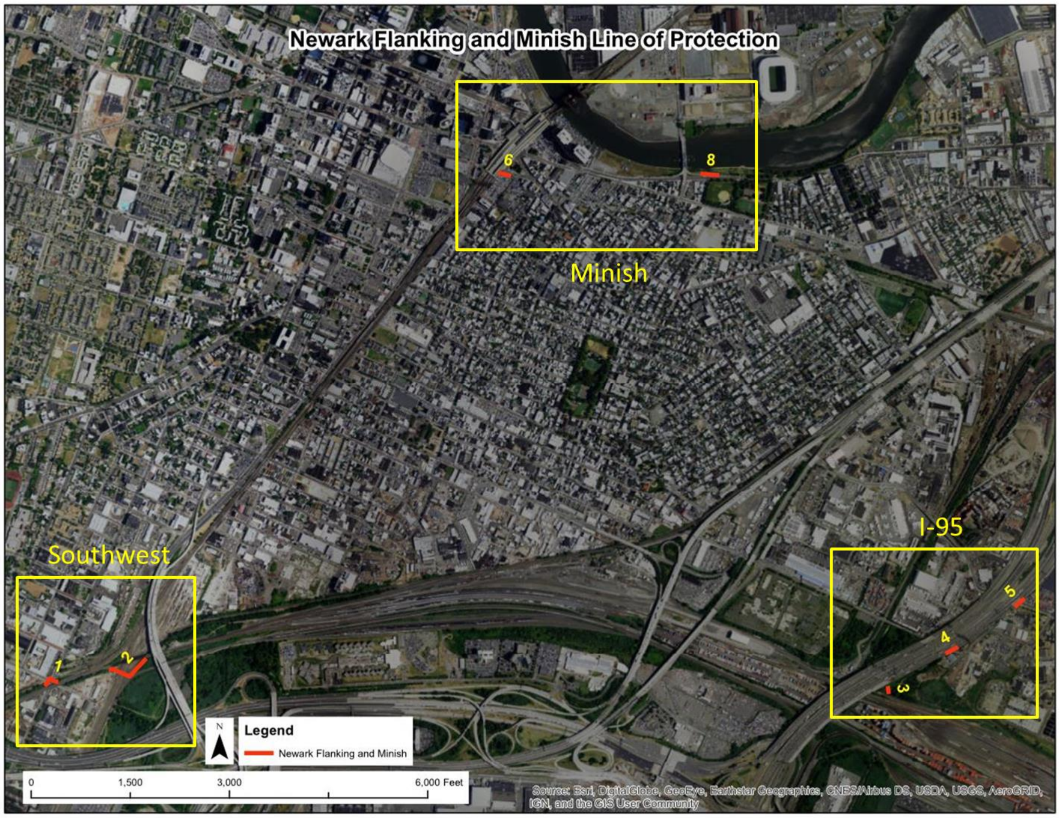

Figure 5: Passaic Tidal Project Reaches

5.1 Southwest Reach

The Southwest Reach alignment consists of two wall and gate segments that cut off flanking of the South Ironbound area of Newark by flood surge entering the Perimeter Ditch around Newark Liberty International Airport.

Segment 1: 290 linear feet (LF) of floodwall with two closure gates: a 65 LF gate across Frelinghuysen Avenue and a 45 LF gate across East Peddie Street. Both gates would be approximately 4.0 feet high. The floodwall height would range from approximately 2.6 to 4.0 feet.

Segment 2: 705 LF of floodwall located between McCarter Highway and Frelinghuysen Avenue, north of East Peddie Street. This segment includes five closure gates, totaling 190 LF to allow passage along the numerous railroad tracks at this location. Floodwall and gate height along this segment would vary from 4.8 to 8.2 feet.

5.2 I-95 Reach

The I-95 Reach includes three wall segments:

Segment 3: 139 LF of floodwall with a tide gate across an unnamed tidal creek just east of the New Jersey Turnpike. The floodwall height of this segment will be a maximum of 9.4 feet. The wall includes an outfall with a backflow prevention device.

Segment 4: 180 LF of floodwall across Delancy Street just east of the New Jersey Turnpike. The closure gate across Delancy Street would be approximately 60 LF and the floodwall height would range from approximately 4.1 to 4.8 feet.

Segment 5: 226 LF of floodwall across Wilson Avenue just east of the New Jersey Turnpike. The closure gate across Wilson Ave would be approximately 60 LF and the floodwall height would range from approximately 3.1 to 3.2 feet.

5.3 Minish Park Reach

The Minish Park Reach alignment includes one segment at Minish Park and one at Newark Penn Station:

Segment 6: 204 LF of floodwall along Edison Place and across New Jersey Railroad Avenue at Edison Place. The closure gate across NJRR Avenue would be approximately 24 LF and the height of the floodwall would range from approximately 0.9 to 3.1 feet.

Segment 8: 297 LF of floodwall along the side of the off ramp from Raymond Blvd to Jackson Street. This segment boarders the sidewalk adjacent to Riverfront Park and would have a height ranging from approximately 1.3 to 3.4 feet.

The total LPP alignment length is approximately 2,040 LF feet and includes 8 closure gates and a tidal culvert. Interior drainage features have not yet been identified.

6 HYDROLOGY AND HYDRAULICS

A summary of the hydrologic and hydraulic analyses completed as part of the general reevaluation is included in Section 6. The analyses are presented in detail in Appendix B, Hydrology and Hydraulics.

6.1 Passaic River and Newark Bay Stillwater

The Project is located near the mouth of the Passaic River and Hackensack River, and includes parts of Newark Bay in New Jersey. Stillwater Elevation (SWL) data were obtained from the recent North Atlantic Comprehensive Coastal Study (NACCS) coastal surge model.

The NACCS model, finalized in 2015, computed the coastal storm hazard for the east coast region from Maine to Virginia as a primary requirement for the NACCS project performance evaluation. The primary focus was on storm winds, waves and water levels along the coast for both tropical and extra-tropical storm events. The method for computing winds, waves and water levels was to apply a suite of high-fidelity numerical models within the Coastal Storm Modeling System. The storm events included in the model included over 1,000 synthetic tropical events and 100 extra-tropical events computed at over 3 million computational locations. The water levels were modeled to include the effects of storm surge, waves, and tides. The NACCS model is discussed in more detail in the Hydrology and Hydraulics (H&H) Appendix.

The stage frequency curve for the Passaic Tidal project area is shown in Table 2.

Table 2: NACCS SWL Stage versus Frequency

Year	Prob.	SWL (feet NAVD)
2-yr	0.5	5.8
5-yr	0.2	7.0
10-yr	0.1	7.9
20-yr	0.05	8.9
50-yr	0.02	10.4
100-yr	0.01	11.8
200-yr	0.005	13.2
500-yr	0.002	14.8

6.2 Waves and Overtopping

The study area is the shoreline along the Passaic River as it converges with the Hackensack River and flows into Newark Bay, in addition to a section of the shoreline of the Hackensack River at the same confluence. This area occupies parts of Hudson and Essex counties in New Jersey. The 1995 and 2013 studies did not consider wave runup or wave overtopping. Wave runup refers to the height above the water surface elevation reached by the swash. Runup is a complex phenomenon known to depend on the incident wave conditions (height, period, steepness, direction), and the nature of the beach, levee or wall being run up (e.g. slope, reflectivity, height, permeability, roughness). Wave overtopping refers to the volumetric rate at which runup flows over the top or crest of a slope the beach, levee, or vertical wall.

If not accounted for in the design, wave runup and overtopping may result in levee slope erosion and possible levee/wall failure. Levees are often designed to limit wave overtopping below a certain wave overtopping threshold.

The project coastline was segmented into 13 parts according to alignment and fetch exposure and the segments are labeled in Figure 6. Levee/floodwall segments 10, 11, and 12 have exposures to the long fetches across Newark Bay, and are assumed to be most susceptible to runup and overtopping due to waves. The most rigorous analyses, which include runup and overtopping, were performed on Segments 10, 11, and 12; representative upstream segments underwent a cursory analysis that only considered overtopping.

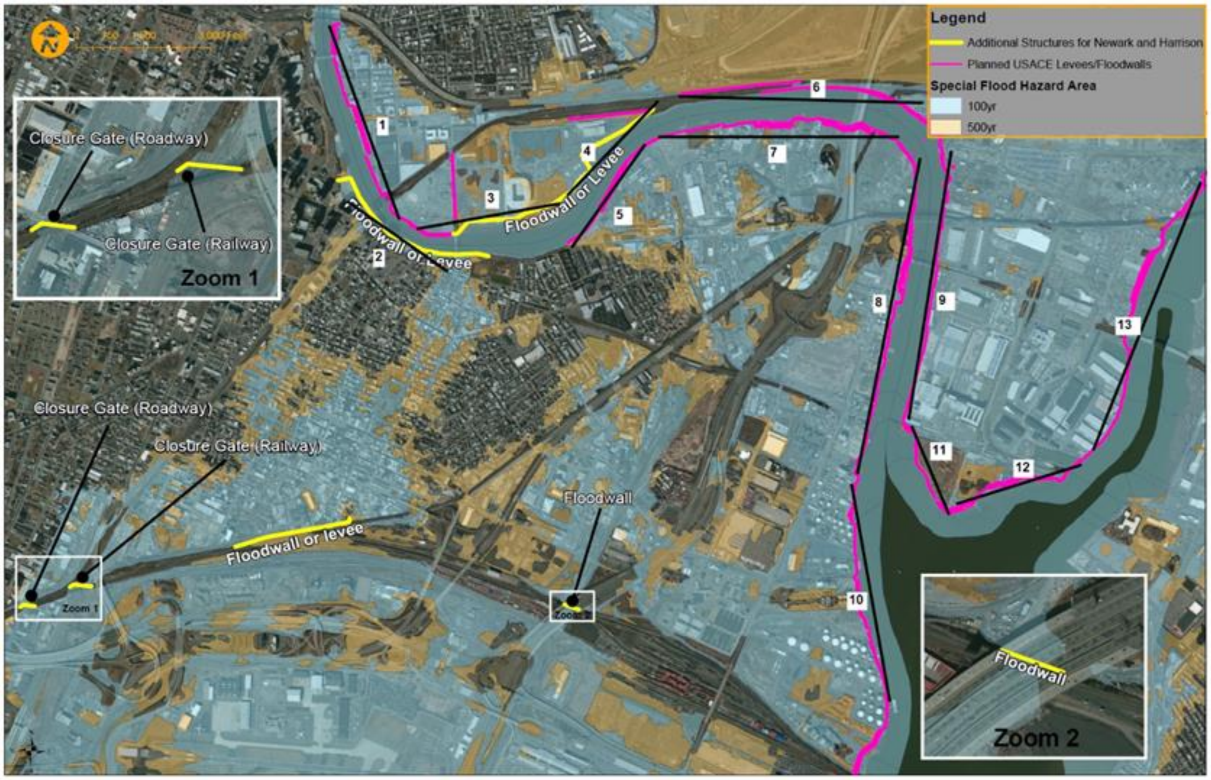

Figure 6: Segmentation of Levee / Floodwall System

6.2.1 Segments Subject to Waves

Details of the wave and overtopping analysis are shown in the H&H Appendix. Tables 3, 4, and 5 show the overtopping flux per unit length along Segments 10, 11, and 12 during the 100-, 200-, and 500-year storms at the analysis heights of 14 feet NAVD, 16 feet NAVD, and 18 feet NAVD, respectively.

Table VI-5-6 on page VI-5-24 of the Coastal Engineering Manual suggests that damage to embankment seawalls with unprotected crests will begin at a flux of approximately 0.022 ft^3/s/ft. Damage to fully protected embankment seawalls will begin at a flux of approximately 0.54 cubic feet per second per foot (ft^3/s/ft). As shown in the following tables, the 16 feet NAVD alternative elevations limits overtopping to acceptable levels up to the 200 year recurrence interval; the 18 feet NAVD floodwall limits acceptable overtopping up to the 500 year recurrence interval.

Table 3: Flux Per Unit Length for Alternative Elevation of 14 feet NAVD

Flux per Unit Length ($ft^3/s/ft$) for Design Elevation 14 ft NAVD88						
Storm Recurrence Interval (yr)	Segment 10		Segment 11		Segment 12	
	Levee	Floodwall	Levee	Floodwall	Levee	Floodwall
100	-	0.179	0.355	0.179	0.316	0.164
200	-	0.516	1.862	0.516	1.876	0.521
500	-	4.994	5.364	4.994	5.070	4.700

-There are no levees planned for Segment 10.

Table 4: Flux Per Unit Length for Alternative Elevation of 16 feet NAVD

Flux per Unit Length ($ft^3/s/ft$) for Design Elevation 16 ft NAVD88						
Storm Recurrence Interval (yr)	Segment 10		Segment 11		Segment 12	
	Levee	Floodwall	Levee	Floodwall	Levee	Floodwall
100	-	0.032	0.034	0.032	0.031	0.029
200	-	0.092	0.166	0.092	0.168	0.093
500	-	0.848	2.708	0.848	2.587	0.811

-There are no levees planned for Segment 10.

Table 5: Flux Per Unit Length for Alternative Elevation of 18 feet NAVD

Flux per Unit Length ($ft^3/s/ft$) for Design Elevation 18 ft NAVD88						
Storm Recurrence Interval (yr)	Segment 10		Segment 11		Segment 12	
	Levee	Floodwall	Levee	Floodwall	Levee	Floodwall
100	-	0.006	0.003	0.006	0.003	0.005
200	-	0.016	0.017	0.016	0.017	0.017
500	-	0.241	0.392	0.241	0.368	0.231

-There are no levees planned for Segment 10.

6.2.2 Upstream Segments

Segments 7 and 13, which are representative upstream segments for the Passaic and Hackensack Rivers, respectively, were also analyzed. The upstream segments are composed of levees and floodwalls with the same geometries and elevations as those described for Segments 10, 11, and 12. These segments are not subject to any significant waves due to both orientation to and the fetch length of significant winds.

Overtopping in upstream segments was calculated utilizing the same equations for the levees and floodwalls documented for the areas subject to more significant waves. The resultant fluxes per unit length for the 14 feet, 16 feet, and 18 feet NAVD alternative elevations are listed in Tables 6, 7, and 8, respectively.

Table 6: **Flux Per Unit Length for Riverine Segments, 14 feet NAVD**

Flux per Unit Length ($ft^3/s/ft$) for Alternative Elevation 14 ft NAVD				
Storm Recurrence Interval (yr)	**Segment 7**		**Segment 13**	
	Levee	**Floodwall**	**Levee**	**Floodwall**
100	0.065	0.032	0.058	0.029
200	0.632	0.174	0.427	0.153
500	3.463	3.261	1.770	1.503

Table 7: **Flux Per Unit Length for Riverine Segments, 16 feet NAVD**

Flux per Unit Length ($ft^3/s/ft$) for Alternative Elevation 16 ft NAVD				
Storm Recurrence Interval (yr)	**Segment 7**		**Segment 13**	
	Levee	**Floodwall**	**Levee**	**Floodwall**
100	0.002	0.002	0.003	0.002
200	0.019	0.012	0.024	0.015
500	0.423	0.183	0.269	0.131

Table 8: **Flux Per Unit Length for Riverine Segments, 18 feet NAVD**

Flux per Unit Length ($ft^3/s/ft$) for Design Elevation 18 ft NAVD				
Storm Recurrence Interval (yr)	**Segment 7**		**Segment 13**	
	Levee	**Floodwall**	**Levee**	**Floodwall**
100	0.000	0.000	0.000	0.000
200	0.001	0.001	0.001	0.002
500	0.020	0.019	0.018	0.017

6.3 Sea Level Change

Current U.S. Army Corps of Engineers (USACE) guidance requires incorporation of SLC into Civil Works projects. This is outlined in Engineer Regulation (ER) 1100-2-8162, *Incorporating Sea Level Change in Civil Works Programs* (31 Dec 2013), which supersedes Engineer Circular (EC) 1165-2-212, *Sea Level Change Considerations for Civil Works Programs*. The ER refers to additional specific guidance in Engineer Technical Letter (ETL) 1100-2-1, *Procedures to Evaluate Sea Level Change: Impacts Responses and Adaptation*, which contains details previously contained in attachments to the old EC.

ER 1100-2-8162 states:

> *"Planning studies and engineering designs over the project life cycle, for both existing and proposed projects, will consider alternatives that are formulated and evaluated for the entire range of possible future rates of SLC, represented here by three scenarios of "low," "intermediate," and "high" SLC.*

> *…Once the three rates have been estimated, the next step is to determine how sensitive alternative plans and designs are to these rates of future local mean SLC, how this sensitivity affects calculated risk, and what design or operations and maintenance measures should be implemented to adapt to SLC to minimize adverse consequences while maximizing beneficial effects."*

Based on an expected project life of 50 years, SLC must be calculated for 2070 conditions from a base year of 2020. USACE issued ER 1100-2-8162, Incorporating Sea Level Change in Civil Works Programs. This ER spells out how SLC is to be computed and incorporated into levee height calculations. To assist in the calculation of SLC mandated by ER 1100-2-8162, USACE has created a tool to assist with the calculations. The tool is located at the website http://www.corpsclimate.us/ccaceslcurves.cfm. This website uses information from ER 1100-2-8162 and National Oceanic and Atmospheric Administration (NOAA) Technical Report OAR CPO-1, Global Sea Level Rise Scenarios for the United States National Climate Assessment published in December 2012. For the Newark Bay area, the Sandy Hook, New Jersey gauge was used.

The generated curves are based on USACE and NOAA equations at a low, intermediate, and high level. The output for the USACE and NOAA equations can be seen in Table 9. The program also plots a chart of the sea level curves as seen in Figure 7. SLC is discussed in more detail in the H&H Appendix.

The inclusion of SLC affects the design height performance and reliability, which can be evaluated using the probability of non-exceedance (PNE). The PNE is discussed later in the appendix.

Table 9: Sea Level Change, Passaic Tidal Project Area

Passaic Tidal 8531680, Sandy Hook, NJ NOAA's Published Rate: 0.01280 feet/yr					
Year	USACE Low NOAA Low	USACE Int NOAA Int Low	NOAA Int High	USACE High	NOAA High
2016	0.07	0.12	0.23	0.28	0.36
2020	0.12	0.19	0.34	0.41	0.52
2025	0.18	0.28	0.49	0.59	0.74
2030	0.25	0.38	0.66	0.78	0.98
2035	0.31	0.48	0.84	1.00	1.25
2040	0.37	0.58	1.03	1.23	1.55
2045	0.44	0.69	1.24	1.48	1.87
2050	0.50	0.80	1.46	1.75	2.22
2055	0.57	0.92	1.70	2.04	2.59
2060	0.63	1.04	1.95	2.34	2.99
2065	0.69	1.17	2.22	2.67	3.42
2066	0.71	1.19	2.27	2.74	3.50

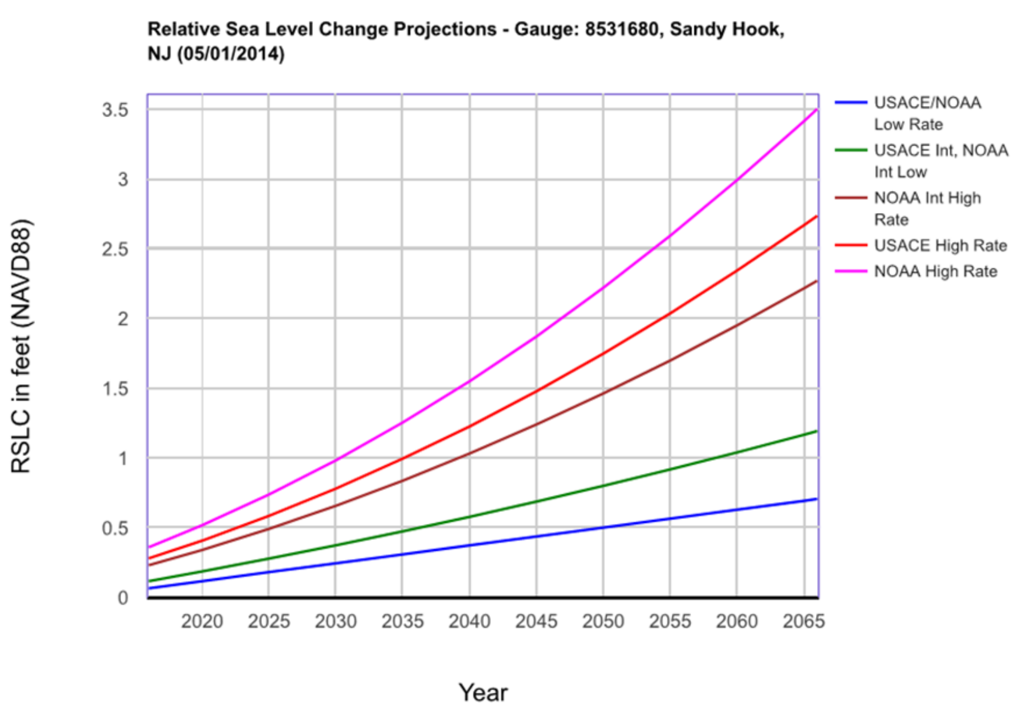

Figure 7: SLC Scenario Projections

6.4 Interior Drainage Analysis

As part of the GRR, the interior drainage plan for the GDM Plan (NED Plan) was remodeled and evaluated. The plan included 160 outfalls and six pump stations. The plan was not reformulated; therefore, interior drainage alternatives were not considered. The following is a description of the general components of the interior drainage features.

1) Outfalls: There are 160 outfalls ranging in size from 24 to 60 inches. Each outfall, whether new or an extension of an existing outfall, includes a sluice gate, backflow prevention, and a catch basin structures.
2) Pump Stations: There are six pump stations in the interior drainage plan. They range from 30 to 100 cubic feet per second (cfs).

A detailed description of the interior drainage model and results for the NED Plan is provided in the H&H Appendix. An interior drainage analysis for the LPP, the Flanking Plan, has not yet been conducted. While estimated costs for interior drainage features have been included in the LPP First Costs, interior drainage features have not yet been identified. The interior drainage analysis for the LPP will be completed prior to the final GRR.

The interior drainage modeling completed for the GRR and described in this appendix was accomplished using a much more robust software platform (HEC-HMS, v4.1) than the 1995 study. However, it was a remodeling effort only and not a re-optimization of interior drainage features. Therefore, with the new model and more detailed topographic mapping, a more refined interior drainage analysis can be achieved, possibly resulting in the removal (or addition) of one or more of the proposed pump stations. The receipt of detailed utilities and stormwater network information will also help refine the future interior drainage analysis.

7 GEOTECHNICAL ANALYSIS

For the geotechnical analysis, the 10.5 miles of line of protection were divided into the following segments:

1) Lister/Turnpike/Doremus Levee/Floodwall in Newark
2) South First Street Levee/Flood Wall in Harrison
3) Kearny Point Levee/Floodwall in Kearny

The three design elevations of 14.0 feet NAVD, 16.0 feet NAVD, and 18.0 feet NAVD were analyzed. The ground level along the levee/floodwall alignment varied from approximately 6 feet NAVD to 8 feet NAVD. Thus, the design height of the levee/floodwall sections was considered from 6.0 feet to 12.0 feet. For the 2013 report for Harrison, the geotechnical analysis of the prior, existing data was updated based on the then-revised line of protection (LOP) height of 16.0 feet NAVD. The results of the latest geotechnical analyses are summarized below and are discussed in more detail in Attachment 1 – Geotechnical Report.

Detailed analyses were not conducted for the Flanking Plan alignments, which are located inland from the shoreline. However, to be conservative, the preliminary results of the geotechnical analysis were considered applicable to these locations. More site specific analyses for the Flanking Plan elements will be conducted prior to the final report.

7.1 Previous Subsurface Investigation

Based on the available subsurface investigations included in the 1995 GDM for the Passaic River Flood Damage Reduction Project, the recent Passaic Valley Sewerage Commission Floodwall System Project, and New Jersey Department of Transportation soil borings database, a total of 42 borings along the proposed levee and floodwall alignment are currently available (see Figure 6). After reviewing the boring logs and in-situ and lab test results, the following segments were assumed for the stability and seepage analyses of the levee and floodwall alternatives.

<u>Soil Profile at East Kearny:</u> Starts at the most eastern portion of the Kearny Segment and continues southcentral as shown in Figure 8.

<u>Soil Profile at West Kearny, Newark and Harrison:</u> Begins at the west end of East Kearny profile and continues west towards the Harrison Segment covering the Newark Segment as shown in Figure 9.

The depth, thickness, type, and continuity of soil layers vary between the two segments, however, the following soil profiles were selected as typical of each for slope stability analysis purpose:

1) East Kearny:
   Organics with Su = 250 pounds per square foot (psf), 55 feet thick, bottom elevation -50 feet NAVD.
   Silty Clay with Su = 500 psf, 30 feet thick, bottom elevation -80 feet NAVD.
   Rock (Weathered shale or siltstone), top of rock varies from -80 to -90 feet NAVD.

2) West Kearny, Newark, and Harrison:
   Organics with Su = 250 psf, 30 feet thick, bottom elevation -25 feet NAVD.
   Silty Clayey Sand with φ = 32 degrees, 10 ft to 30 feet thick, bottom elevation -55 feet NAVD.
   Rock (weathered shale or siltstone), top of rock varies from -30 to -100 feet NAVD.

The natural soils throughout the alignment of the floodwall/levee system are overlain by a layer of highly variable fill materials up to approximately 20 feet in thickness. These materials are predominantly granular soils intermixed with silt, clay, and decaying organic soil that are placed uncontrolled and include wood, metal, and general building demolition rubble and debris.

The summary of subsurface conditions or stratigraphy of both segments and soil properties used in this study are given in the Geotechnical Report. In all segments, the soft organic silt or clay layer were continuously encountered along the region.

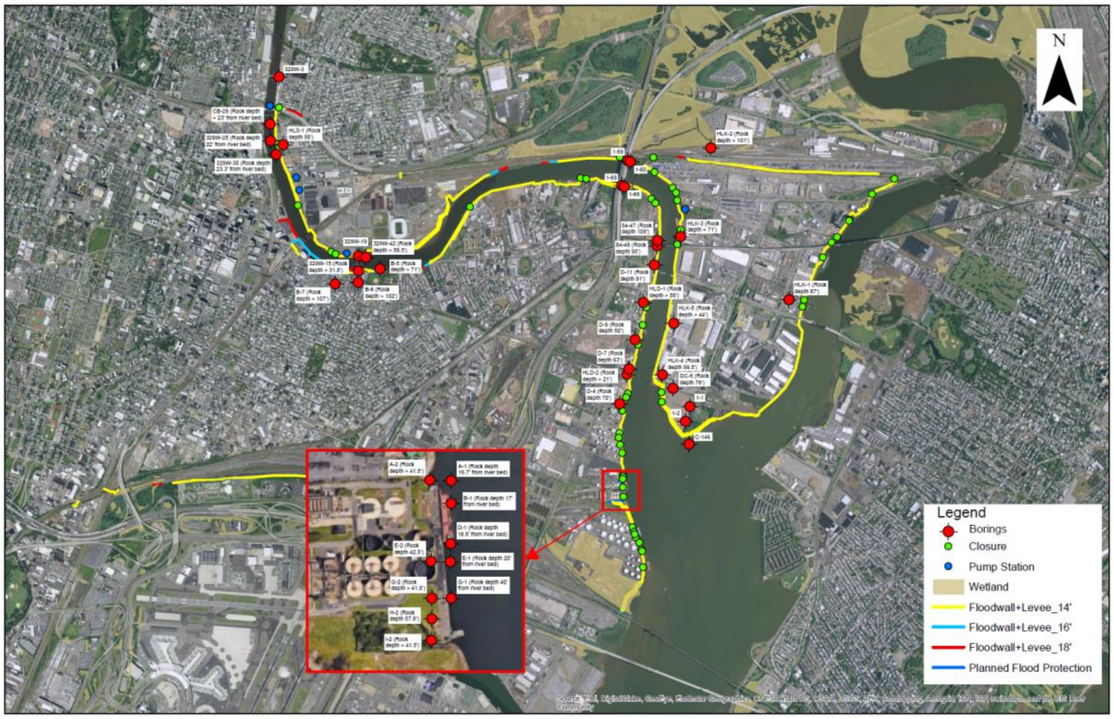

Figure 8: Existing Boring Data

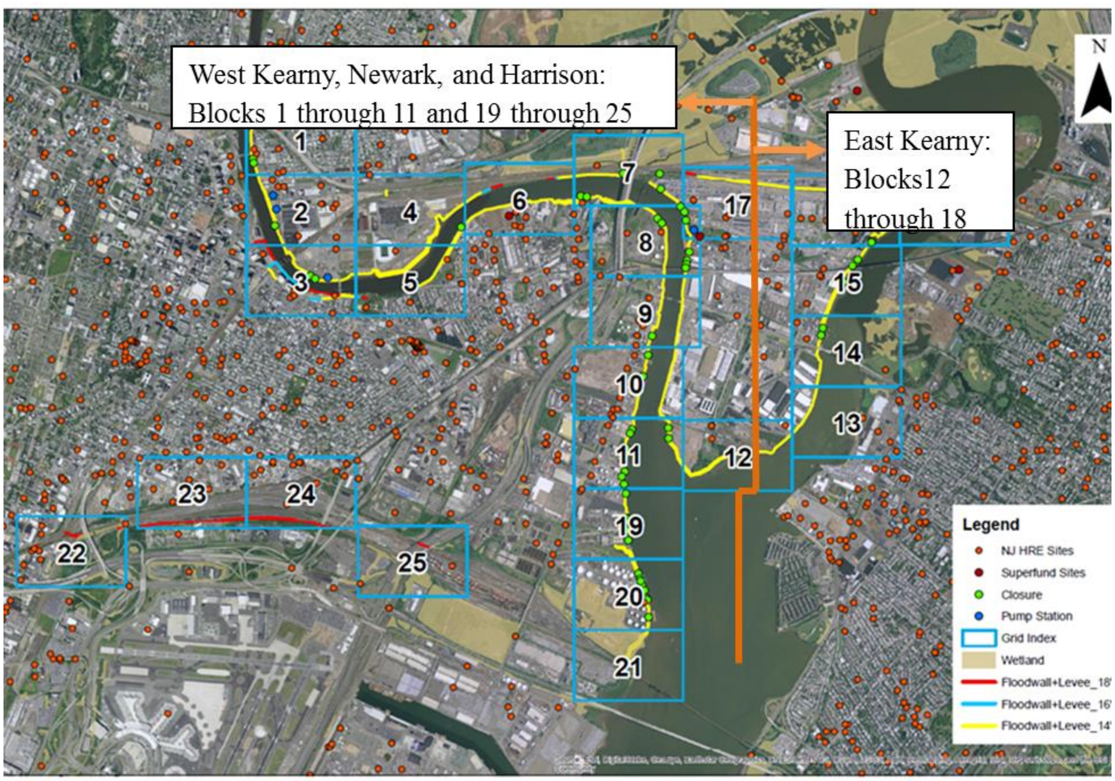

Figure 9: Geotechnical Segments

7.2 Preliminary Information and Assumptions

The preliminary information and assumptions made in the geotechnical analysis are summarized below:

1) The analyses and calculations performed as part of this study are preliminary in nature and all estimates were based on limited available data. The new subsurface investigation and laboratory testing program as recommended later in this section are necessary to meet USACE requirements for final design.
2) A layer of highly variable fill materials up to approximately 20 feet in thickness exists in the area of protection. The fill material is intermixed with silt, clay, and decaying organic soil that are placed uncontrolled and include wood, metal, and general building demolition rubble and debris.
3) For pile depth calculations, rock depths vary along the line of protection but pile lengths are assumed to be conservative (exceeding 100 feet in some locations).

7.3 Levee Construction Determination

Due to the presence of organic soils along the previously designed levee segments, excavation and backfill of foundation material would be required in order to provide adequate stability for the levee segments. Furthermore, liquefaction mitigation may be required pending further geotechnical analysis. Therefore, based on the extreme uncertainty in levee foundation design parameters and anticipated access limitations, all levee segments in the previous 1995 GDM design were replaced with floodwall segments. The details of the levee analysis are included in Attachment 1.

7.4 Floodwall Geotechnical Analysis

Because much of the proposed line of protection does not have adequate space or adequate foundation for levee construction, a floodwall alternative is considered in those reaches. Due to the unsatisfactory Factor of Safety obtained for a levee over 6 feet high and also the need to remove unsuitable and uncontrolled existing fill material with varying thickness as discussed above, the floodwall alternative was considered for the entirety of each reach. A typical section of floodwall with sheetpile cutoff is shown in Figure 10.

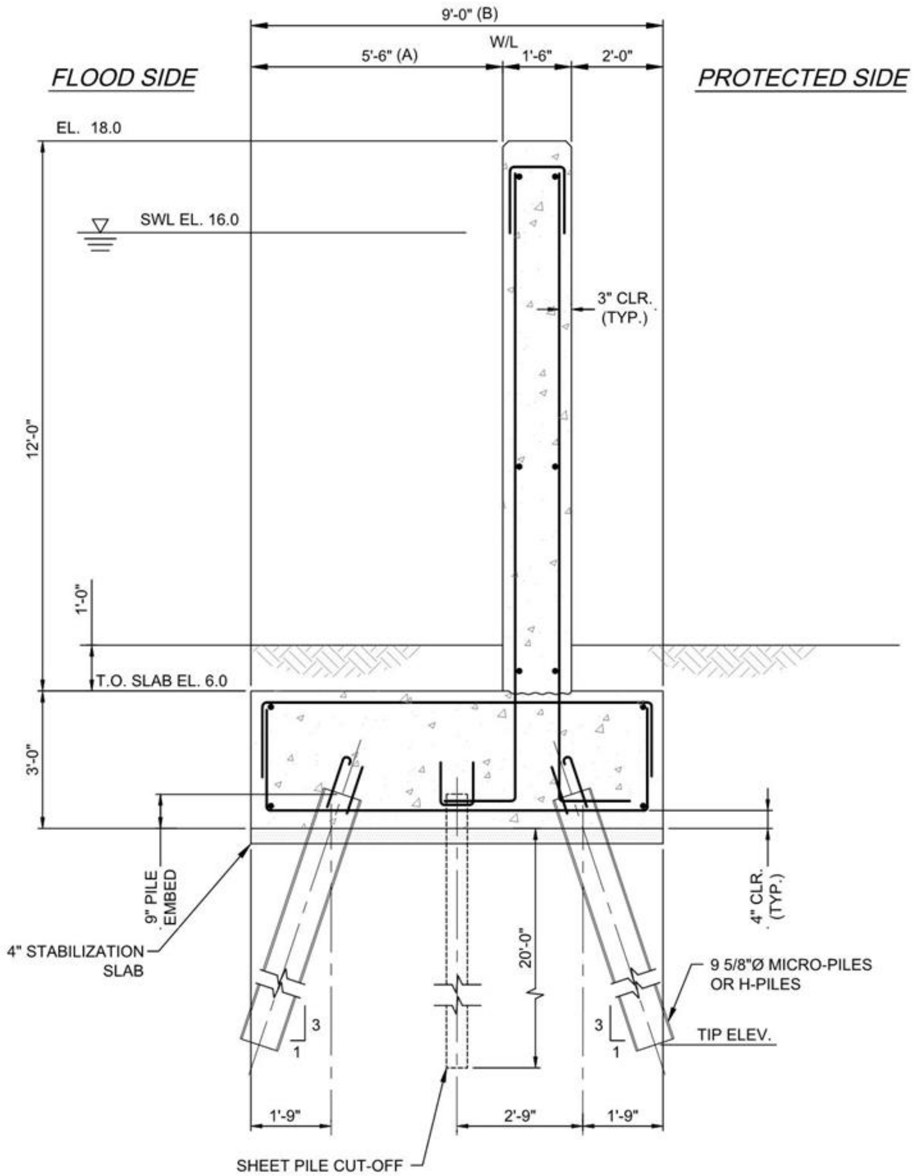

Figure 10: Typical Floodwall Section

7.5 Seepage and Deep-Seated Sliding Analysis

The seepage analyses of a 6 and 12 foot high floodwall for all Segments were performed to estimate the exit gradient and flow rates with and without sheetpile cutoff. The exit gradient at the landside of the 12-foot high floodwall with no sheetpile cutoff was 0.86 for both Segments. Per Reference 7, underseepage controls are needed where the calculated exit gradient exceeds an allowable gradient of typically 0.5. Using 20 foot deep sheetpile cutoff reduced the exit gradient to an acceptable value of 0.16. The flow rate for steady state seepage condition could be as high as 14 gallons/day per foot length of the wall. The details of floodwall seepage analyses are provided in Attachment 1.

Deep-seated sliding analysis was performed to check the sliding within weak layers beneath the sheetpile. The vertical water pressure due to the flood was conservatively assumed to be a surcharge load on the ground surface. The minimum global stability safety factor obtained for the critical slipping surface is 1.50 which meets the minimum required value per EM 1110-2-2502 (Reference 7). In this analysis the lateral resistances of the foundation piles and sheetpile were conservatively neglected.

7.6 Pile Bearing Capacity

Pile capacity analyses were performed on two different pile options: H-Piles (HP14x73) and Caissons or Micropiles with 8 and 12 inch diameter rock sockets. ENSOFT Software "APILE" was utilized for axial capacity analyses on driven H-piles. To be conservative, skin resistance for the top 10 feet of the piles was eliminated. Downdrag effects were ignored due to limited information and will be considered based on the results of additional borings and lab tests.

The compression and tension capacities of rock sockets for caissons were calculated using the spreadsheets with details as provided in Attachment 1.

7.7 Pile Foundation Recommendations

Due to the existing soft or organic soil, proposed piles will be advanced to a stiffer or denser soil stratum to achieve required compression and tension capacities. Based on the soil stratification and results of the pile capacity analysis, an 80 foot long H-Pile (HP14x73) bearing on silty clay can provide an ultimate compression and uplift capacity of approximately 95 kips at the East Kearny Segment. In West Kearny, Newark, and Harrison Segment, a 60 foot long H-Pile bearing on silty clayey sand can provide approximately 110 kips of ultimate compression capacity and 100 kips of ultimate uplift capacity. For H-Piles bearing on a competent rock the ultimate compression capacity will be determined by structural capacity with the limit of 200 kips.

The allowable compression and tension capacities of 20 foot long (12-inch O.D.) rock socket for Caissons/Micropiles were estimated 240 and 150 tons, respectively.

The final design will include a study of pile group effect and pile deflections under lateral, compression, and uplift loads, and potential downdrag effects.

7.8 Recommendations

In order to obtain a better understanding of the subsurface condition and more accurate engineering and physical soil properties, additional field investigation and lab testing need to be performed for the final design. The following are recommendations for additional analyses to support final design:

1) Additional soil borings will be performed, typically at every 200 to 300 feet. Soil profiles typically with three borings in the traverse directions perpendicular to the levee-floodwall alignment in each cross-section need to be developed. At least one test boring for each soil profile should be drilled to a depth of bedrock or 100 feet for seismic site classification purpose.
2) Additional disturbed and undisturbed samples are needed for soil properties interpretation purposes.
3) Additional grain size analysis, unconsolidated-undrained test and consolidation tests need to be performed.
4) It is also recommended that seismic cone penetration test soundings be performed for every eight borings to obtain shear wave velocity of the subsurface soils. Seismic CPTs may assist to better define the site class, shear wave velocity, and liquefaction potential of the site.
5) Field permeability and/or field pumping test will be performed, as necessary, for permeability estimation.

8 SURVEYING, MAPPING AND OTHER GEOSPATIAL DATA

Terrain data used to update the alignment was developed from 2012 LiDAR collected for the USACE NACCS. The vertical datum for this study is the North American Vertical Datum of 1988 (NAVD). Horizontal datum is North American Datum of 1983 (NAD83).

9 FLOODWALL DESIGN

9.1 General

The design elements defined herein represent a preliminary design (i.e., 30-percent level) using the best available information. The structural analysis was limited to stability. Pile foundations provide stability against overturning, sliding and flotation resistance. Soil conditions along this reach of the Passaic River were divided into two reaches, East Kearny and West Kearny (which includes Harrison and Newark). The elevation of the bedrock was assumed based on current limited information (see Section 7 – Geotechnical Analysis); pile lengths must be refined as more soil data becomes available.

Micro piles and H-Piles were considered in Typical T-wall reaches. Design calculations for the structural analysis can be found in Attachment 2. For cost comparison purposes, three wall heights were considered: Top of Wall (TOW) elevations at 18.0 feet NAVD, 16.0 feet NAVD and 14.0 feet NAVD. For the structural calculations, the SWL was assumed to be 2 feet below the TOW elevation. The typical ground elevation was assumed to be 6 feet NAVD throughout the project.

9.2 Codes and Standards

The following is an abbreviated list of general USACE references, and industry codes and standards which are applicable to structural and foundation design for this preliminary design effort. Additional codes must be referenced for the final construction Plans & Specifications. Considered in this design are:

1) AASHTO, American Association of State Highway and Transportation Officials, LRFD Bridge Design 7th Edition, 2014.
2) ACI 318-14 American Concrete Institute, Building Code Requirements for Structural Concrete.
3) ACI 350-06 American Concrete Institute, Environmental Engineering Concrete Structures.
4) AISC, American Institute of Steel Construction, Inc., Manual of Steel Construction, 14th Edition.
5) ASCE 7-10 American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures.
6) ASTM, American Society for Testing and Materials.
7) AWS D1.1-15 American Welding Society, Structural Welding Code, latest edition.
8) UFC 3-201-01, Civil Engineering.
9) USACE EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures.
10) USACE EM 1110-2-2502, Retaining and Flood Walls.
11) USACE EM 1110-2-2906, Design of Pile Foundations.
12) USACE EM 1110-2-2909, Conduits, Culverts and Pipes.
13) USACE ETL 1110-2-584, Design of Hydraulic Steel Structures.
14) USACE ETL 1110-2-575, Evaluation of I-Walls.

9.3 General Design Load Parameters

9.3.1 Load Combinations

The preliminary design includes four Basic Load Cases; these are the loadings that typically control floodwall designs. Other loadings must also be analyzed in the final design, including Seismic Load Cases for both Operating and Maximum Earthquake conditions. Additionally, sufficient hydraulic modeling should be performed as part of the future design to establish wave properties and forces. Typically, on inland waterways, when the wall is overbuilt to include uncertainty and sea-level rise the static head to top of wall is similar in force to that imparted by a wave; sufficiently close for this conceptual design. The load cases included in the design are shown below. Detailed load calculations are shown in Attachment 2.

1a. Construction. Dead load of the concrete wall components, no earthen backfill, no uplift. (A 17 percent overstress is permitted for this load case)

1b. Construction with Wind. Dead load of the concrete wall components, no earthen backfill, no uplift; a conservative wind load of 50 psf is applied to the wall stem. (A 33 percent overstress is permitted for this load case.)

2a. Flood Stage at Still Water, Impervious Cutoff. Dead load of concrete wall, At-Rest lateral earth pressures, and hydrostatic loading for water to the SWL; Uplift forces assume the sheet pile to be impervious. Wave force is not included.

2b. Flood Stage at Still Water, Pervious Cutoff. Dead load of concrete wall, At-Rest lateral earth pressures, and hydrostatic loading for water to the SWL; Uplift forces assume the sheet pile to be pervious varying linearly from flood side SWL to the ground water elevation on the Protected Side. Wave force is not included.

3a. Flood Stage with Water to Top of Wall, Impervious Cutoff. Dead load of concrete wall, At-Rest lateral earth pressures, and hydrostatic loading for water to the TOW; Uplift forces assume the sheet pile to be impervious. Wave force is not included. (A 33 percent overstress is permitted.)

3b. Flood Stage with Water to Top of Wall, Pervious Cutoff. Dead load of concrete wall, At-Rest lateral earth pressures, and hydrostatic loading for water to the TOW; Uplift forces assume the sheet pile to be pervious varying linearly from flood side TOW elevation to the ground water elevation on the Protected Side. Wave force is not included. ( )4a. Flood Stage at Still Water, Debris Impact Load, Impervious Cutoff. Loadilude: Dead load of concrete wall, At-Rest lateral earth pressures, and hydrostatic loading for water to the SWL. Uplift forces assume the sheet pile to be impervious. A debris load of 500 pounds per linear foot is applied at the SWL. Wave force is not included. (A 33 percent overstress is permitted.)

The overstress factors listed in each load case above reflect the stress levels permitted in the HSDRRS design guidance that was developed for the New Orleans District post-Katrina and considered applicable for this flood protection project. The overstress was used to normalize loads in the calculations found in Appendices 2 and 3. In the Detailed Design the load factors found in EM 1110-2-2104 for concrete design and ETL 1110-2-584 for steel design shall be used in lieu of the overstress.

9.3.2 Hydraulic Stages

Water and ground surface elevations for the structural analysis are shown in Table 10.

Table 10: Hydraulic Stage and Design Water Surface Elevations

Stage (feet NAVD)	Flood Side (feet NAVD)	Protected Side (feet NAVD)
TOW El 14.0		
SWL Water	EL. 12.0	EL. 6.0
TOW Water	EL. 14.0	EL. 6.0
TOW El 16.0		
SWL Water	EL. 14.0	EL. 6.0
TOW Water	EL. 16.0	EL. 6.0
TOW El 18.0		
SWL Water	EL. 16.0	EL. 6.0
TOW Water	EL. 18.0	EL. 6.0

SWL – Still Water Level
TOW – Top of Wall

9.4 Load Cases

9.4.1 Dead Loads (D)

Dead loads will be determined in accordance with applicable engineering manuals and ASCE 7-02, and will include the self-weight of all permanent construction components including foundations, slabs, walls, roofs, actual weights of equipment, overburden pressures, and all permanent non-removable stationary construction. Typical unit weights are shown in Table 11.

Table 11: Unit Weights

Item	Weight [Pcf]
Water (Fresh)	62.4
Semi-compacted Fill	110
Fully Compacted Granular Fill, wet	120
Fully Compacted Granular Fill, Effective	58
Fully Compacted Clay Fill, wet	110
Fully Compacted Clay Fill, Effective	48
Riprap	130
Silt	94
Reinforced Concrete (Normal weight)	150
Steel	490

9.4.2 Live Loads (L)

Live loads for building structures will be determined in accordance with applicable engineering manuals and ASCE 7-02.

9.4.3 Live Load Surcharge (LS)

A minimum live load surcharge of 200 psf will be applied during construction.

9.4.4 Soil Pressures (S)

Structures are designed for lateral and vertical soil pressures. Lateral pressures are determined using the at-rest coefficients, KO obtained from the Geotechnical Report:

Lateral Soils at-rest Pressure Coefficients:

Ko = 0.8 for Clay.

Ko = 0.48 for Granular Material.

9.4.5 Hydrostatic Loads (H)

Hydrostatic loads for which structures will be designed refer to the vertical and horizontal loads induced by a static water head and buoyant pressures, excluding uplift pressures. Dynamic Wave Forces have not been included.

9.4.6 Uplift Loads (U)

Uplift loads for which structures will be designed to two uplift conditions: Uplift Condition A, assumes the sheet pile cutoff wall is fully effective (Impervious), and Uplift Condition B, assumes the sheet pile cutoff wall is ineffective (Pervious) (pressure assumed to be vary linearly across the base).

9.4.7 Wind Loads (W)

Structures are designed for wind loads established by ASCE No. 7, “Minimum Design Loads for Buildings and Other Structures,” but in no case less than 50 psf. The basic sustained wind speed is 110 miles per hour, and the exposure category is “C”. Architectural roofs will be designed for a 135 mile-per-hour sustained wind. An importance factor of 1.15 is included in wind calculations.

9.5 Concrete Design Criteria

Concrete design will utilize EM 1110-2-2104 and the ACI 350R Concrete Sanitary Engineering Structures and will comply with the ACI 318 latest edition strength design method, unless otherwise required:

- Structural Concrete: 4,000 psi @ 28 days with a maximum water/cement ratio = 0.40
- Steel reinforcement 60,000 psi (ASTM A615)

9.6 Steel Design Criteria

Steel design will utilize the ETL 1110-2-584 and the AISC Steel Construction Manual, 14th edition. Load combinations will be in accordance with ASCE 7-02. Typical design values are as follows unless otherwise noted:

(a) Structural steel rolled shapes	ASTM 572, Grade 50
	ASTM A992, Grade 50
(b) Plates	ASTM A992, Grade 36
(c) Bolts and nuts	ASTM A325, minimum ¾ inch
	ASTM A490
(d) Anchor Bolts	ASTM A449, (¾ inch diameter and greater)
(e) Corrosion stainless steel	ASTM A304 (freshwater)
	ASTM A316 (saltwater)
(f) Sheet Piles	ASTM A328, Grade 50
	ASTM A572, Grade 50
(g) Stainless Steel Embedded Anchors	ASTM A276 or UNS S21800

Normally, components that will be exposed to the elements are either hot-dipped galvanized or primed, painted and sealed with coats of (10 mils min.) epoxy. Vertical lift gates and steel sheet pile structures will be painted with an epoxy painting system.

9.7 Pile Foundation Design Criteria

All forces applied to T-Wall structures are resisted by the pile foundation. T-wall monoliths are assumed to act independent of adjacent monoliths, no load transfer is considered between monoliths. Pile designs are based on a soil structure interactive analysis with the pile supports input in accordance with EM 1110-2-2906. Lateral resistance of the soil is based on the soil horizontal subgrade modulus. In future designs, pile capacities will be determined utilizing springs based on P-Y and T-Z curves generated by geotechnical analysis. Factors for group effects have been included in this analysis. Pile capacities have been determined using all-friction and a combination of friction and end bearing. Micropiles will be considered where bedrock is reasonably shallow (e.g., <50 feet). Micro-pile capacities include a 10 foot deep rock socket. H-Pile capacities mainly consider friction; very little end bearing was included. Piles embedded the standard 6”-9” were analyzed as both fixed and pinned pile heads. Recent research conducted by the New Orleans and St. Paul districts has indicated that piles with minimal embedment act as partially fixed, more fixed than pinned. As such, recent practice is to bracket the connection design with a pinned and fixed analysis. Monoliths with all vertical piles were rigidly connected to the base and only analyzed as fixed. In order to assure a very rigid connection, these piles were embedded two pile diameters into the base.

Piles may be Micropiles with continuous casings to bedrock, steel pipe piles, or steel H piles. Pipe piles satisfy ASTM A252 with minimum yield strength of 45 ksi. H-piles satisfy Grade 50 Steel. Steel piles are designed structurally per AISC ASD, 14th Edition, as modified by EM 1110-2-2906. Combined axial and bending are considered when analyzing the stresses in the piles.

Vertical piles were used only where space restraints prevented the installation of the more efficient battered pile. This condition mainly occurred were the floodwall alignment was sandwiched between the Passaic River/Hackensack River/Newark Bay and buildings located near the top of bank. Cross sections of the bank and infrastructure were not available; therefore, it was assumed that a 15 feet top of bank crown at 8 feet NAVD exists with a flood side bank slope down to the thalwag of the river. The vertical pile design used only a fixed pile head. To assure this fixity occurred, the piles were embedded a minimum of two pile diameters into the base. The pile foundation can be used for bearing and also to stabilize the bank slope, similar to soil nailing, if stability factors of safety are low.

CPGA pile design software was used for this preliminary design. Settlement and ground instability were not considered to be a factor. Forces from downdrag and unbalanced loads were not included in the pile design. It was assumed that pile load tests will be conducted in advance of construction, a Factor of Safety = 2.0 was included for normal load cases and 1.5 for unusual load cases.

9.8 Construction and Vegetation-Free Zones

Project easements were reviewed for compliance with respect to the USACE’s vegetation management policy, ETL 1110-2-583, 30 April 2014, *Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams and Appurtenant Structures*. The current vegetation management guidelines were not in place when the 1995 GDM was completed. The new guidance requires 15 feet from levee toes, drains or structural features and 15 feet from the faces of floodwalls and a minimum of 8 feet beyond the footing.

This allows for operation and maintenance, surveillance, and access during high-water events. Vegetation has potential to impact the operations and degrade the performance of the system. The root-free zone provides a margin of safety between the greatest expected extent of plant roots critical to the performance and reliability of the flood damage risk reduction system. The typical configuration for a floodwall, as set forth under USACE’s vegetation management policy, is shown in Figure 11.

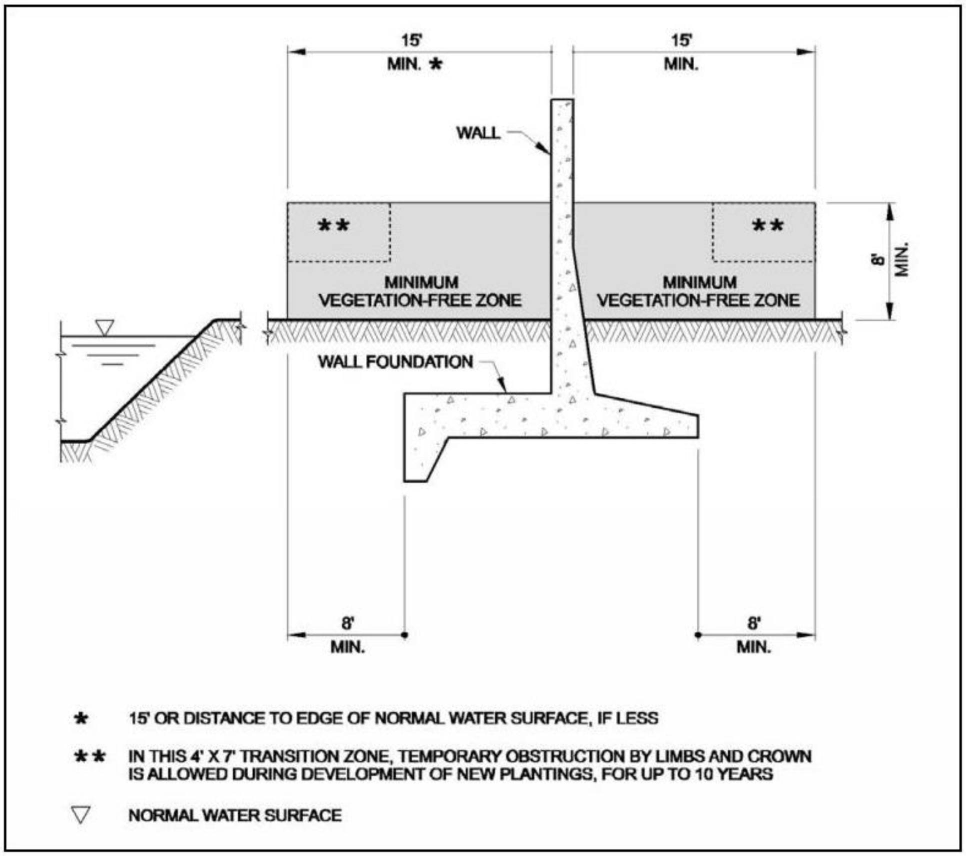

Figure 11: Typical Vegetation-Free Zone Configuration at Floodwall

9.9 Armoring

The potential for sizeable waves along the three Newark Bay reaches of the NED Plan line of protection may necessitate armoring the exterior of the floodwall to help prevent wave-induced erosion. Articulated block armoring (such as ArmorFlex®) can provide needed protection while also allowing for grass cover within the block openings for better aesthetic appearance and a more natural, environmentally friendly surface. Typical articulated block armoring installation is shown in Figure 12.

Image courtesy CONTECH Engineered Solutions

Figure 12: Articulated Block Armoring

10 CLOSURE GATE DESIGN

10.1 General

There are 64 closure gates in the current NED Plan line of protection and eight in the LPP. The gates are mostly exterior gates associated with access through the line of protection to the waterfront. The inventory of gates is shown in Attachment 3 and the project drawings. The closure gates were grouped into several different categories based on gate openings, heights and types. The gate types used were predominantly swing gates with the exception of roller gates for openings of 50 feet or larger. The gates are assumed a mix of closures to span railroads, highways and pedestrian crossings.

The current design level includes four basic load cases which are loadings that typically control floodwall/closure gate structures designs. A full array of load cases for each gate will need to be investigated in the final design phase. The load cases included in the current design are:

1) Construction + Wind: Dead load of the concrete monolith and steel gate, a conservative wind load of 50 psf, no earthen backfill, no uplift, no construction surcharge. A 33 percent overstress is permitted for this load case.

2) Flood stage at still water (SWL) at 2 feet below top of gate structure with debris impact loading of 500 lbs/ft applied at the SWL. A 33 percent overstress is permitted for this load case.

3) Flood stage at water to top of gate (TOG).Wave force is not included. A 33 percent overstress is permitted for this load case.

4) Flood stage at SWL at two feet below top of gate structure. A 0 percent overstress is permitted for this load case.

The gate members (girders, intercostals, and skin plates), concrete monolith (abutments/footings), and foundations were sized to carry these anticipated loads as mentioned above for all different gate categories which have been selected. Secondary gate features such as any hinge assemblies, connections, casters, trolleys, or hanger systems were conceptually shown based on previous similar projects and engineering judgment. Calculations were not performed to size these types of features. Wave loadings are expected to be minimal due to topographic conditions and lack of proximity/exposure to full coastal storm surge associated with hurricanes. It is also assumed, per technical discussions, that there will be no unbalanced loading or downdrag forces seen by the gates at this level of design. This will require more in-depth analysis and can be fully vetted during later design stages. Complex pile group analysis; therefore, was not required. Seismic forces were not considered to govern and were not applied at this level of design.

For the design effort, the following codes and standards were used, as well as the applicable portions of the HSDRRSDG (Hurricane Storm Damage Risk Reduction System Design Guidelines) and the existing project GDM:

- EM 1110-2-2705 – Structural Design of Closure Structures for Local Flood Protection Projects
- EM 1110-2-2104 – Strength Design for Concrete Hydraulic Structures
- EM 1110-2-2105 – Strength Design for Hydraulic Steel Structures.

Once the preliminary gate designs were compiled for all different gate selections, costs were developed based on the major contributing "bid" items that would typically be present in final documents such as: concrete monolith structure (abutments and footings), structural steel gate (gate overall weight plus detail factor), concrete reinforcing for monolith structure, and pile foundation (total pile length for the gates). Items such as steel embeds, seals, turnbuckles, casters, hinge assemblies, access ladders, etc. were included in the structural steel gate item. Unit prices were based on recent, similar construction projects and adjusted for any regional effects and applied to the various bid item quantities.

10.2 Grouping

The final closure gate inventory has 64 closure gate structures that fluctuate in gate opening width and gate height. The gate heights for all 64 closure gates were determined based on the design water elevation of 14 feet NAVD and their respective existing grade elevations. In addition, evaluations were completed for gate heights 2 and 4 feet above the 14-foot elevation.

All gates were grouped into several scenarios based on gate openings and heights as shown in Table 12. The Kearny, Newark and Flanking areas consist of H-pile foundation whereas the Harrison area consists of concrete micropile foundation. Any opening width equal to 10-feet or smaller was grouped with the 10-foot gate opening. The 20-foot gate opening was grouped with a series of opening widths ranging from 15 to 20 feet. The majority of opening widths in the inventory was for the 30-foot width. The 30-foot gate opening was grouped from 25 to 30 feet. The 35-foot, 40-foot, 45-foot and 50-foot gate openings were grouped individually, since their gate opening width is considered to be on the larger end of the swinging gate spectrum.

Table 12: Gate Grouping Scenarios

GATE OPENING (Feet)	**SWING GATE(H-Pile Foundation)**								
	GATE HEIGHTS(Feet)								
10	6	8	10	12	14	16	-	-	-
20	5	7	9	11	13	-	-	-	-
30	2	4	6	8	10	12	14	16	18
35	9	11	13	15	17	-	-	-	-
40	10	12	14	-	-	-	-	-	-
45	5	7	9	-	-	-	-	-	-
50	6	8	10	-	-	-	-	-	-
GATE OPENING (Feet)	**SWING GATE(Micro Pile Foundation, Harrison Area)**								
	GATE HEIGHTS(Feet)								
30	11	13	15	17	-	-	-	-	-
40	2	4	6	8	10	12	-	-	-
GATE OPENING (Feet)	**ROLLER GATE**								
	GATE HEIGHTS(Feet)								
50	10	12	14	16	-	-	-	-	-

Once the gates were group as described above, the smallest gate height and the tallest gate height for each respective group was determined and a 2-feet incremental height increase was implemented starting from the minimum to the maximum gate heights. Typically gates for openings larger than 38 feet would be considered at the threshold for the swing gates. Roller gates predominantly are seen for openings larger than 38 feet. The gate opening width identified in the flanking area of the final closure structure inventory ranged from 40 to 150 feet. After further assessment of the gate openings in the flanking area, the roller gate option will not be feasible due to the limited space in this area which does not facilitate the construction of the larger concrete monolith structure. Therefore, the 150 feet opening was divided into three swing gates with an opening of 50 feet. The inventory list also includes four gate widths opening of 50 feet which have been grouped together as roller gates since the vicinity permitted a larger concrete monolith structure. The same grouping procedure described above was followed with respect to gate heights.

10.3 Gate Design

The structural design of the swing/roller gate includes the layout and design of the major structural elements of the concrete monolith structure and floodgate. This includes the gate steel members, the concrete gate bay walls and support columns, base slab and the pile foundations. The structural steel gate members include top and bottom girders spanning horizontally between concrete bay columns, vertical intercostal framing spaced at approximately 2 feet on center and spanning between top and bottom girders, steel skin plate spanning between the vertical intercostal, and steel cross bracing and horizontal bracing. The concrete monoliths are comprised of two concrete gate bay walls/columns on either side which are formed into the base slab and pile foundation. The concrete monoliths are supported by the pile foundations. Steel H-piles and concrete micropiles were applied during design for consistency with the floodwall team. It is assumed that each gate monolith structure will be flanked by the floodwall structures in the adjacent reaches. The floodgate drawings are included in the drawing set. The sections and views on the drawings are grouped as described in Table 12. Based on the gate width and heights, the design elements will vary in size, location and spacing accordingly.

The analysis of the steel gate and concrete monolith was performed based on the load cases noted in the introduction. The governing load case was typically the flood stage with water at the top of the gate. Loads were applied as hydrostatic pressures corresponding to the water surface elevations on the flood-side. A debris impact uniform loading (500 lbs/ft) was applied at the appropriate water surface elevations. The skin plate was designed as a fixed end beam spanning between the vertical intercostals and the deflection was limited to 0.4 of the thickness to ensure that the flat plate theory is applicable. The horizontal girders were designed as larger wide flange simply supported beams spanning between the bearing points on the concrete columns making them true beam elements allowing for flexural stresses. The vertical intercostals were designed as simple beams spanning between horizontal girders. The vertical intercostals consist of a WT section welded to the skin plate and were designed as a combined section utilizing the steel skin plate as the tension flange of the total combined section. The analysis of the reinforced concrete monolith walls and columns was performed considering fixed support at the interface of the bottom of the wall and top of slab. The wall analysis considered a 1 foot unit width of the wall acting as a cantilever and connected only to the base slab. The column analysis considered half of the gate width and width of the column loading on the column acting as a cantilever and connected only to the base slab. The entire analysis for the floodgate and concrete monolith was carried out by hand calculations for one gate width and height which than an excel spreadsheet program was developed to generated the analysis design for all chosen gate scenarios listed in Table 12. Gate calculations are provided in Attachment 3.

Gate cost estimates were developed based on the results of the analysis above. The cost estimate was broken down into four items corresponding to each individual gate width and height. The four cost items are the structural steel gate, concrete monolith structure, concrete reinforcing and pile foundation with a final total project cost.

11 PUMP STATIONS

The 1995 GDM includes six pump stations for interior drainage, ranging from 30 to 100 cfs. The GRR did not include conceptual design of the pump stations; rather, the pump station costs were updated based on a cost curve developed from a range of pump station sizes.

12 UTILTIES RELOCATION/PROTECTION

Similar to the pump stations, there is insufficient detail to estimate the scope and cost for utilities relocations and/or protections for features passing through the line of protection. However, a cost allotment for utilities relocation was included in the cost estimate. Features such a pipe sleeves through or under the floodwall were considered in the Abbreviated Risk Analysis.

13 DESIGN AND CONSTRUCTION SCHEDULE

The preliminary design and construction schedule is shown in Attachment 4.

14 DRAWINGS

The TSP (LPP) drawings are provided as Attachment 5.

15 ADDITIONAL ANALYSES AND DATA COLLECTION

Additional analyses and data collection are required to finalize the project design. These work efforts will be conducted as part of the next phase of the project or during the development of Plans and Specifications (P&S) and include:

15.1 Future Geotechnical Needs

In order to obtain a better understanding of the subsurface condition and more accurate soil physical properties, additional field investigation and lab testing need to be performed for the final design. The recommendations listed below may be considered to augment the previous investigations:

1) Additional soil borings will be performed typically every 200 feet, and in the traverse direction perpendicular to the floodwall alignment.

15.2 Future Survey Needs

The following survey efforts are required in order to produce final P&S:

1) Additional detailed topographic surveys along the authorized LOP and in the locations of project features will be required to support 30-scale design drawings.
2) Detailed utilities surveys along the LOP and near project features will also be required.

15.3 Future Interior Drainage Analysis

Due to the availability of more robust modeling software and more detailed topographic mapping, a more refined interior drainage analysis can be achieved, possibly resulting in the removal of one or more of the proposed pump stations. The receipt of detailed utilities and stormwater network information will also help refine the analysis.

An interior drainage analysis for the LPP, the Flanking Plan, has not yet been conducted. While estimated costs for interior drainage features have been included in the LPP First Costs, interior drainage features have not yet been identified. The interior drainage analysis for the LPP will be completed prior to the final GRR.

16 PERMITS AND APPLICATIONS

Permits and applications will be identified and developed as part of the development of P&S. The following is a list of permits required for construction; however, this list is not exclusive:

1) New Jersey Flood Hazard Area
2) Individual Freshwater Wetlands
3) General Permit 12 (GP-12), Survey and Investigating
4) Soil Erosion and Sediment Control
5) New Jersey Pollutant Discharge Elimination System
6) New Jersey Dept. of Transportation permits
7) Treatment Works Approval (TWA) for any modifications to existing sanitary sewers

17 EMERGENCY ACTION PLAN

An Emergency Action Plan will be developed during the Plans and Specifications Phase of the project. The coordination of this effort will include the non-Federal partner, county and municipalities.

18 OPERATION AND MAINTENANCE

Development of an Operation, Maintenance, Repair, Replacement and Rehabilitation Manual will be performed during the Construction Phase of the project.

19 REFERENCES

1. USACE (1995), General Design Memorandum (GDM), Passaic River Flood Damage Reduction Project, Appendix E- Geotechnical Design, Levees, Floodwalls and Miscellaneous, United States Army Corps of Engineers, dated September 1995.

2. “Design and Construction of Levees”, EM1110-2-1913, United States Army Corps of Engineers, dated April 30, 2000.

3. Idriss, I. M., & Boulanger, R. W. (2008). Soil liquefaction during earthquakes. Earthquake engineering research institute.

4. “http://earthquake.usgs.gov/hazards/products/conterminous/2008/”, Accessed December 14, 2015.

5. “AASHTO LRFD Bridge Design Specifications”, 7th ed., American Association of State Highway and Transportation Officials, dated 2014.

6. Das, B. M. (2006). Principles of geotechnical engineering, Nelson, Ontario, Canada, 686 p.

7. “Retaining & Flood Walls”, EM 1110-2-2502, United States Army Corps of Engineers, dated September 29, 1989.

8. “Design Guidance for Levee Underseepage”, ETL-1110-2-569, United States Army Corps of Engineers, dated May a, 2005.

ATTACHMENT 1
GEOTECHNICAL ANALYSIS

ATTACHMENT 2
T-WALL STRUCTURAL ANALYSIS

ATTACHMENT 3
CLOSURE GATE INVENTORY AND CALCULATIONS

ATTACHMENT 4
DESIGN AND CONSTRUCTION SCHEDULE

ATTACHMENT 5
DRAWINGS