US Army Corps of Engineers®
New York District

Passaic River, New Jersey

Passaic River Tidal General Reevaluation Report

Lower Passaic River, New Jersey

Appendix F
Hydrology and Hydraulics

2019

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1 INTRODUCTION

The Hydrology and Hydraulics (H&H) 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 Recommended Plan, which is 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 risk management measures 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 multiple design elevations 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 levee elevations and alternative alignments 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 calculate estimated 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. The U.S. Army Corps of Engineers (USACE) selected the LPP as the Recommended Plan.

This appendix provides the detailed analysis of the project H&H for the NED plan and the Recommended Plan. Detailed discussions of the interior drainage analysis for each plan are included as subappendices.

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 Recommended Plan is shown in **Figure 2**.

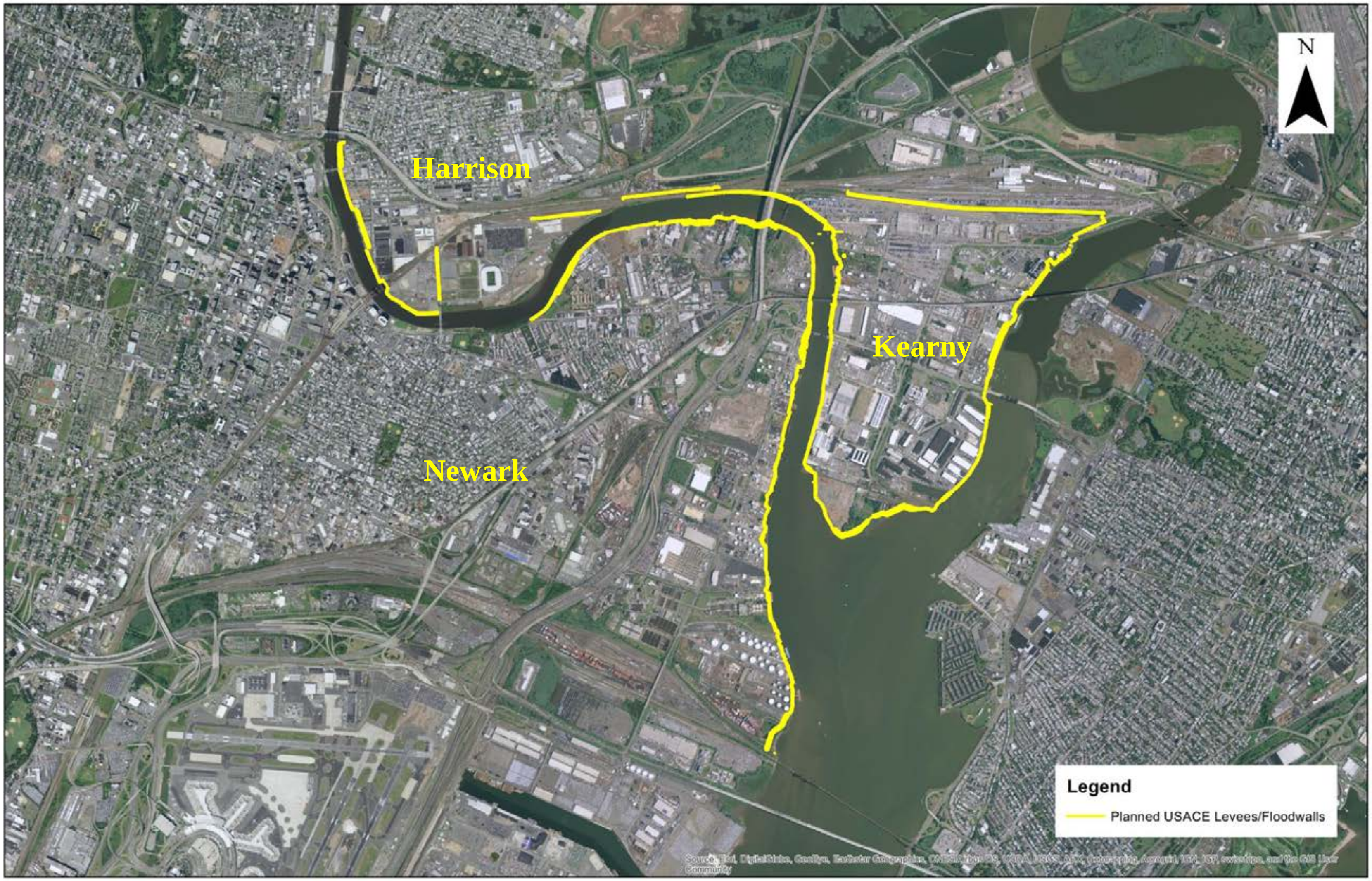

Figure 1: Passaic River Tidal Project Area, 1995 GDM Alignment

Figure 2: Passaic River Tidal Project, Recommended 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 one in 100 chance of occurring in any single year would be described as the one 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
4%	0.04	25-year
2%	0.02	50-year
1%	0.01	100-year
0.4%	0.004	250-year
0.2%	0.002	500-year

1.2 Survey and Datum

The latest topographic data used 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 (NGVD29). The conversion factor from NGVD29 to North American Vertical Datum of 1988 (NAVD88) is approximately -1.1 feet; therefore, the 1995 GDM design elevation of 14.9 feet NGVD29 is converted to 13.8 feet NAVD88. For ease in analysis, computation and discussions, the 1995 GDM design elevation is rounded to 14 feet NAVD88.

2 PROJECT PURPOSE

The purpose of the Passaic River, New Jersey, Tidal GRR is to determine if the previously authorized or newly developed storm risk management projects 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, 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 elements 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 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 (SLC), the project was evaluated at the design elevation and two additional design elevations +2 feet and +4 feet higher. Due to potential challenges presented by HTRW and Superfund sites' proximity to the authorized alignment, an additional alternative, the smaller Flanking Plan, was also considered.

4 RECOMMENDED PLAN

The Passaic Tidal Recommended Plan is the LPP and consists of concrete floodwalls and gates along three reaches as described below. The design elevation is 14 feet NAVD88 based on the limits of adjacent high ground which will limit flanking. The typical ground elevation at segment locations is 6 to 10 feet NAVD88. For areas with a wall height of six feet or less, the wall is primarily a concrete I-wall; for areas where the wall is greater than six feet, the wall is a pile-supported, concrete T-wall. The structural analysis is provided in Appendix J – Engineering & Design. The project reaches are shown in **Figure 3**.

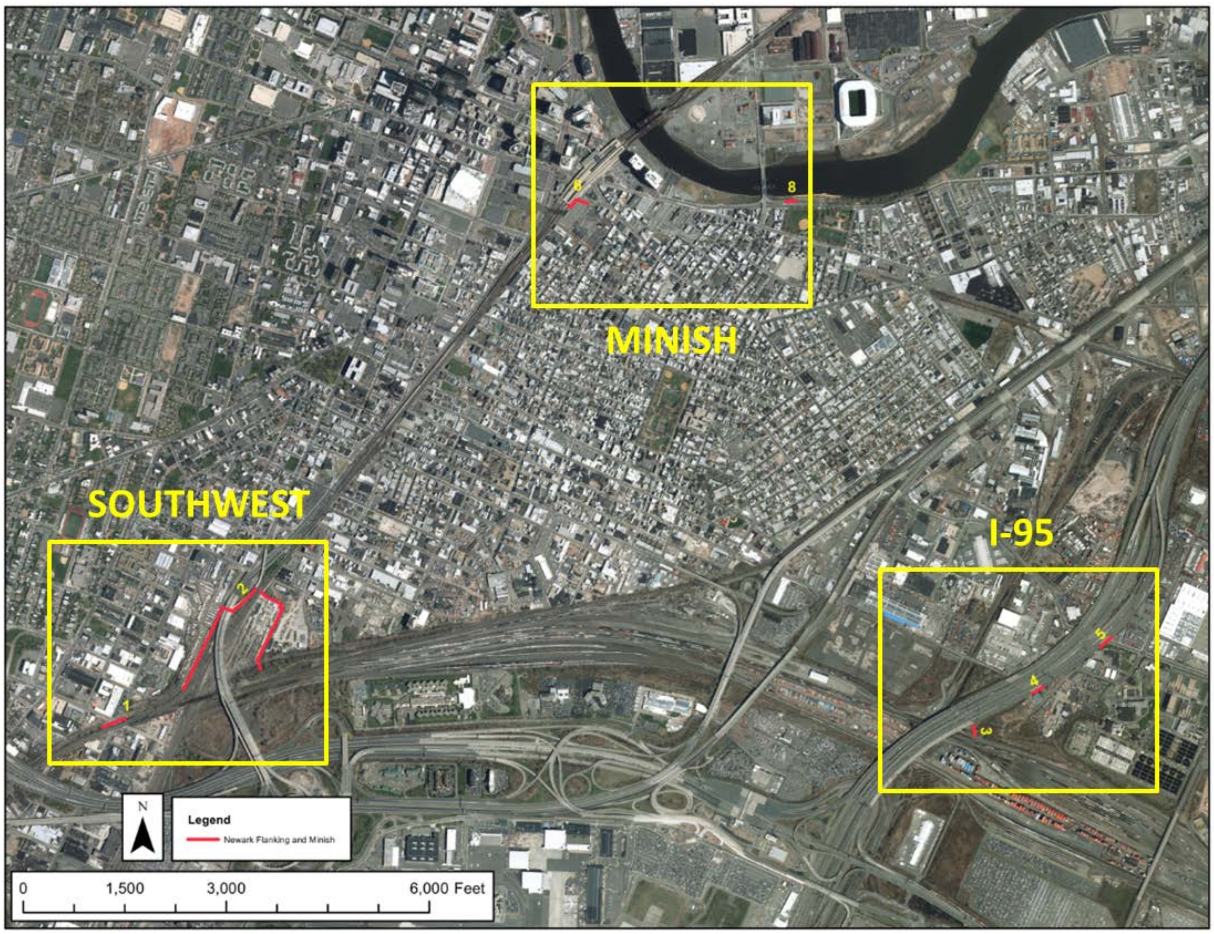

Figure 3: Passaic Tidal Project Reaches – Recommended Plan

4.1 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: 170 linear feet (LF) of floodwall with one closure gate: a 140 LF gate across the intersection of Frelinghuysen Avenue and East Peddie Street. The gate would be approximately

4.0 feet high above ground. The floodwall height above ground would range from approximately 2.6 to 4.0 feet and tie into the adjacent railroad embankment.

Segment 2A (western part of Segment 2): 1,990 LF of floodwall located between the main rail line to Newark Penn Station and the southern tie-off of the alignment. Segment 2A ties into the railroad embankments on each end of the wall. The Segment 2A alignment accommodates the proposed PATH railway extension from Newark Penn Station to the Newark Liberty Airport transit hub. Relocation of the Poinier Street ramp to McCarter Highway is planned to accommodate the PATH extension.

Segment 2B (eastern part of Segment 2): 1,450 LF of floodwall from the tie-in at the NJ Transit/Amtrak railroad to the southern alignment tie-in. This segment includes a gate at New Jersey Railroad (NJRR) Avenue and the southern rail line, and an additional gate north of the rail line for stormwater drainage during extreme rainfall events. Floodwall and gate height above ground along this segment would vary from 4.8 to 8.2 feet.

4.2 I-95 Reach

The I-95 Reach alignment includes three wall segments:

Segment 3: 135 LF of levee with three 36-inch culverts, headwalls, sluice gates, and backflow prevention devices. The levee crosses an unnamed tidal drainage ditch just east of the New Jersey Turnpike. The levee height above ground of this segment will be a maximum of approximately 9.4 feet.

Segment 4: 190 LF of floodwall across Delancy Street just east of the New Jersey Turnpike. The closure gate across Delancy Street would be approximately 70 LF and the floodwall height would range from approximately 4.1 to 4.8 feet.

Segment 5: 240 LF of floodwall across Wilson Avenue just east of the New Jersey Turnpike. The closure gate across Wilson Avenue would be approximately 85 LF and the floodwall height would range from approximately 3.1 to 3.2 feet above ground.

4.3 Minish Park Reach

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

Segment 6: 330 LF of floodwall along Edison Place and NJRR Avenue, and crossing NJRR Avenue to tie into the railroad embankment. The closure gate across NJRR Avenue would be approximately 30 LF. A closure gate was proposed along Edison Place at the Edison Park Fast. The height of the floodwall would range from approximately 0.9 to 3.1 feet above ground.

Segment 8: 150 LF of floodwall along the side of the off ramp from Raymond Boulevard to Jackson Street. This segment borders the sidewalk adjacent to Riverfront Park and would have a height ranging from approximately 1.3 to 3.4 feet above ground.

The total Recommended Plan alignment length is approximately 4,850 LF feet and includes seven closure gates and three 36-inch culverts. The Recommended Plan segments are shown in **Figures 4 through 13**.

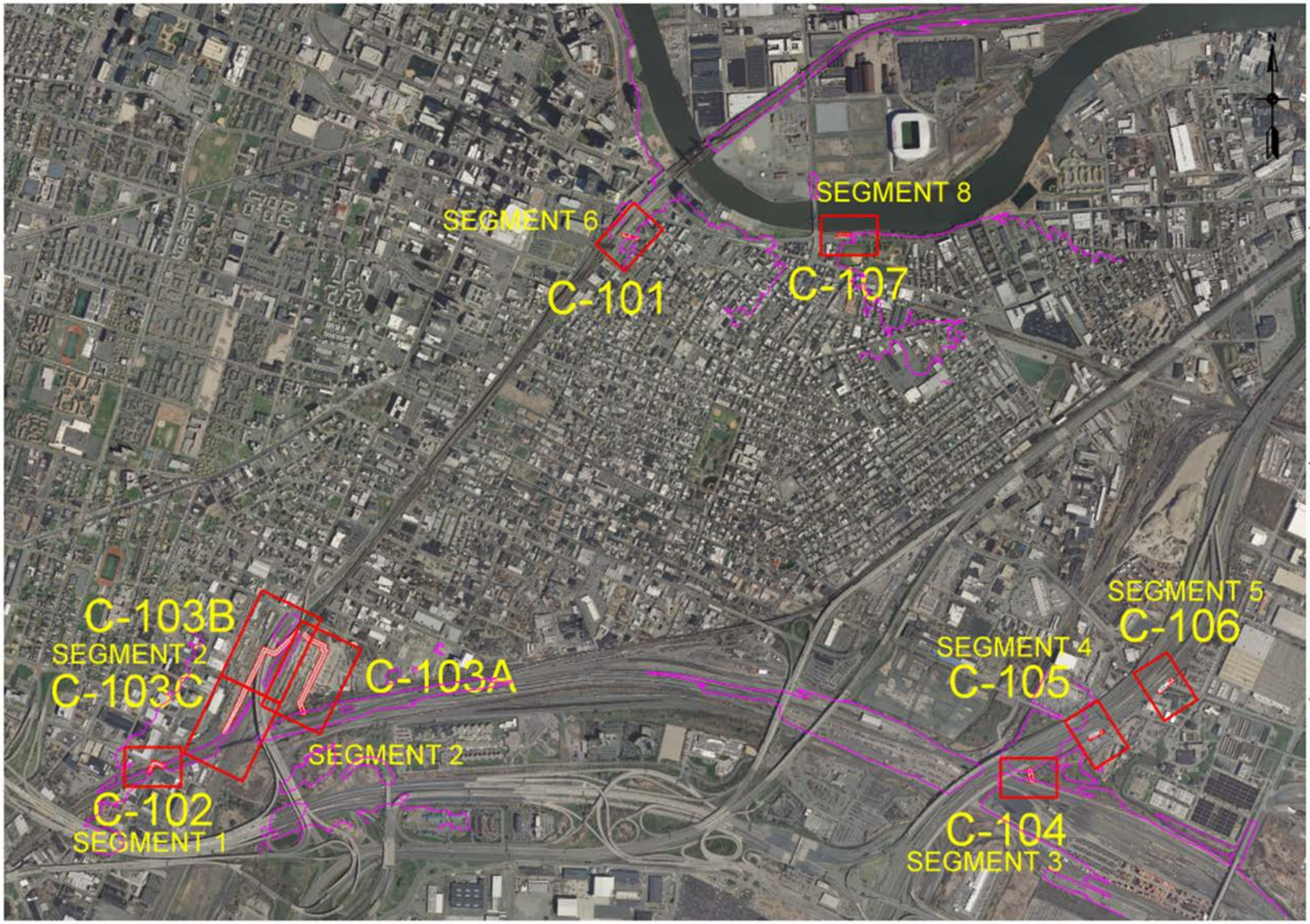

Figure 4: Recommended Plan Layout/Key Plan

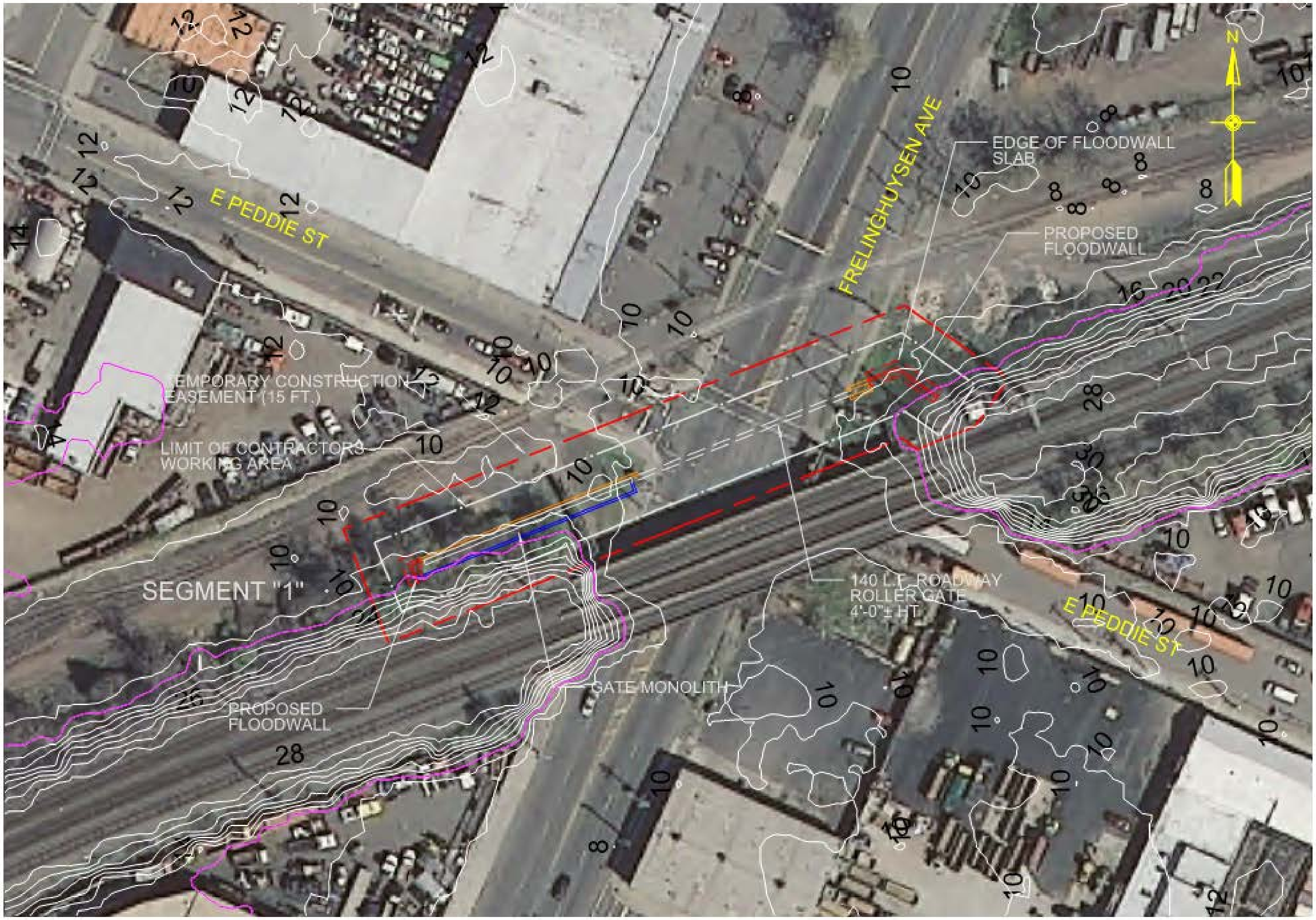

Figure 5: Southwest Reach – Segment 1

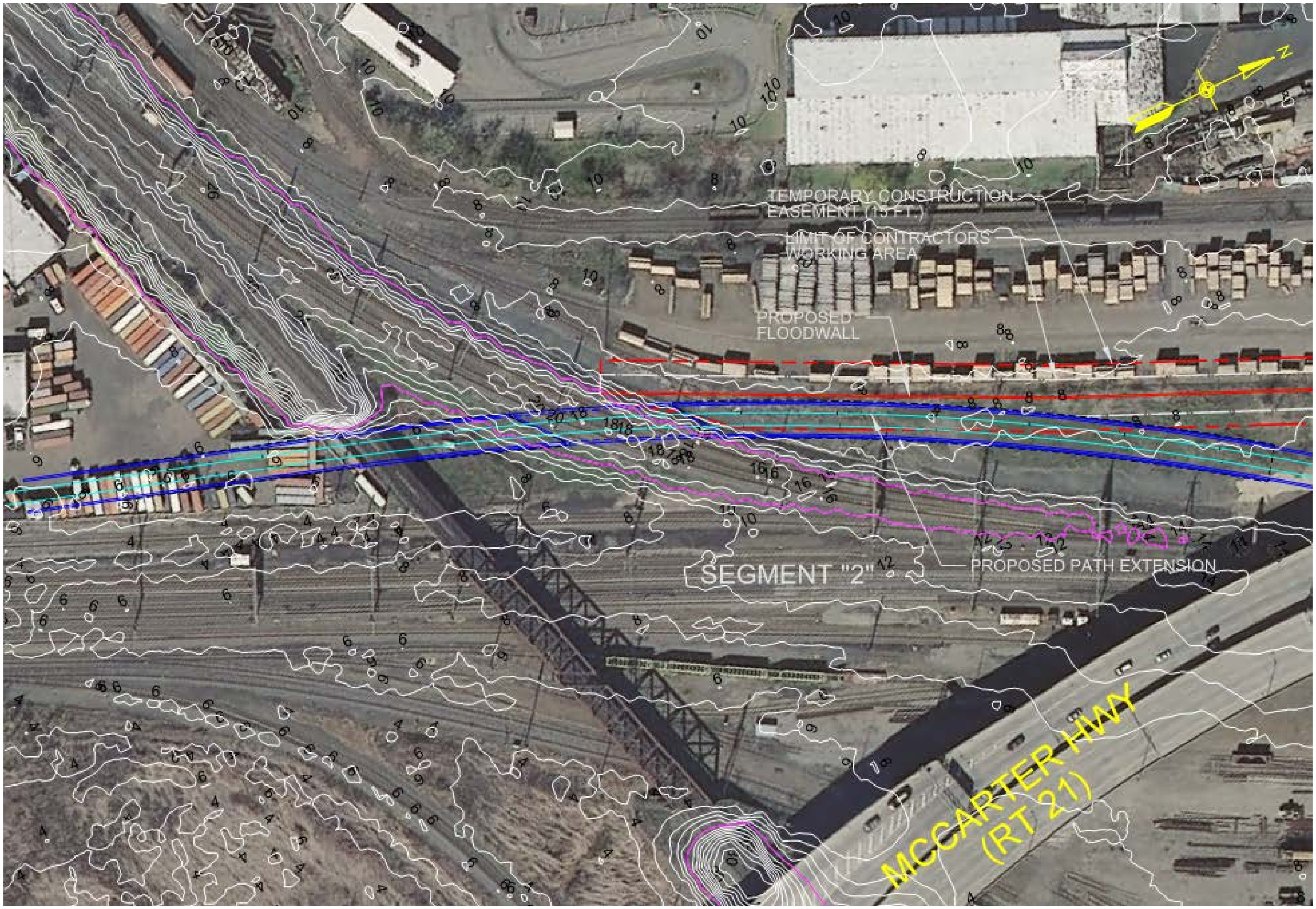

Figure 6: Southwest Reach - Segment 2A (South)

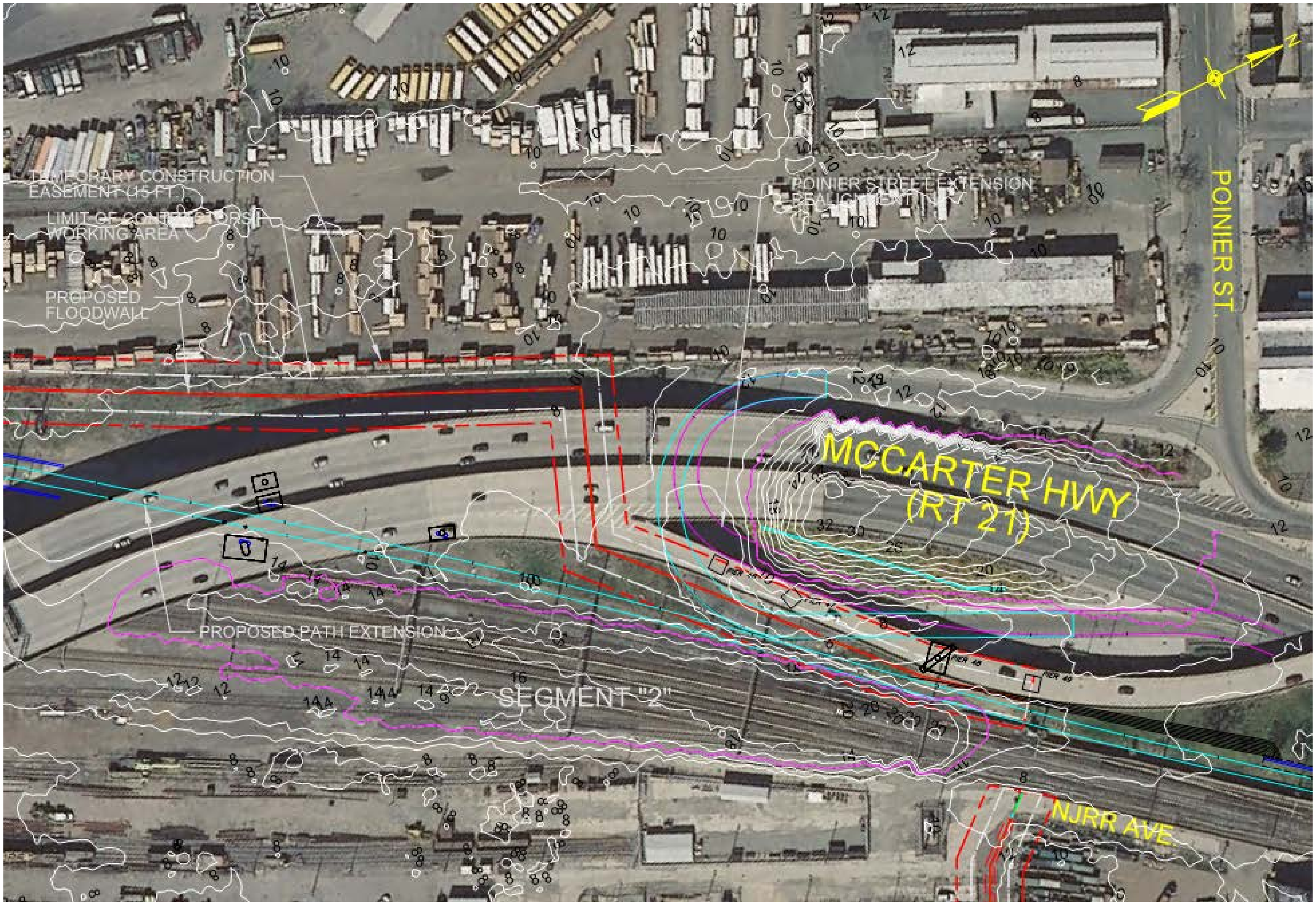

Figure 7: Southwest Reach - Segment 2A (North)

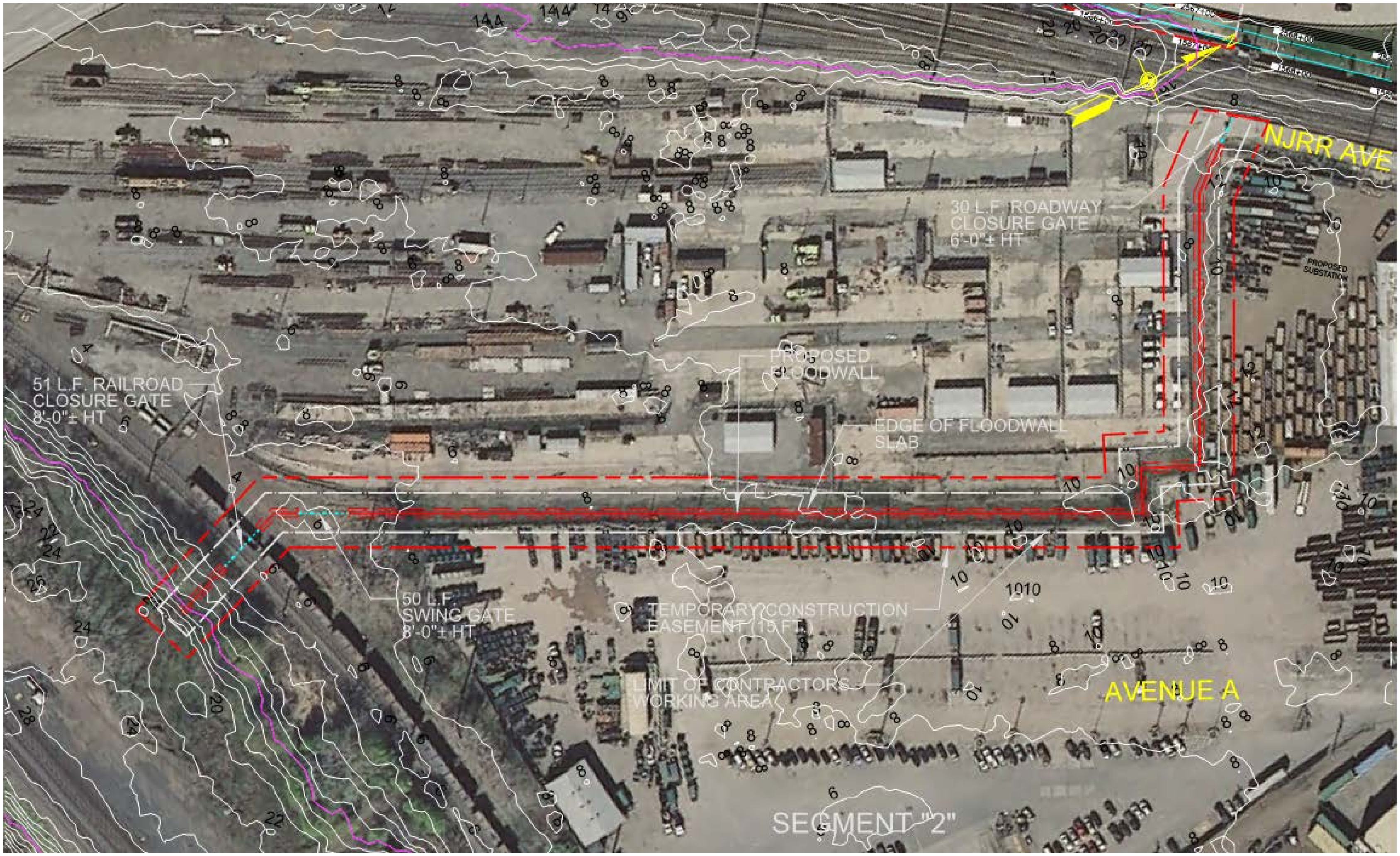

Figure 8: Southwest Reach - Segment 2B

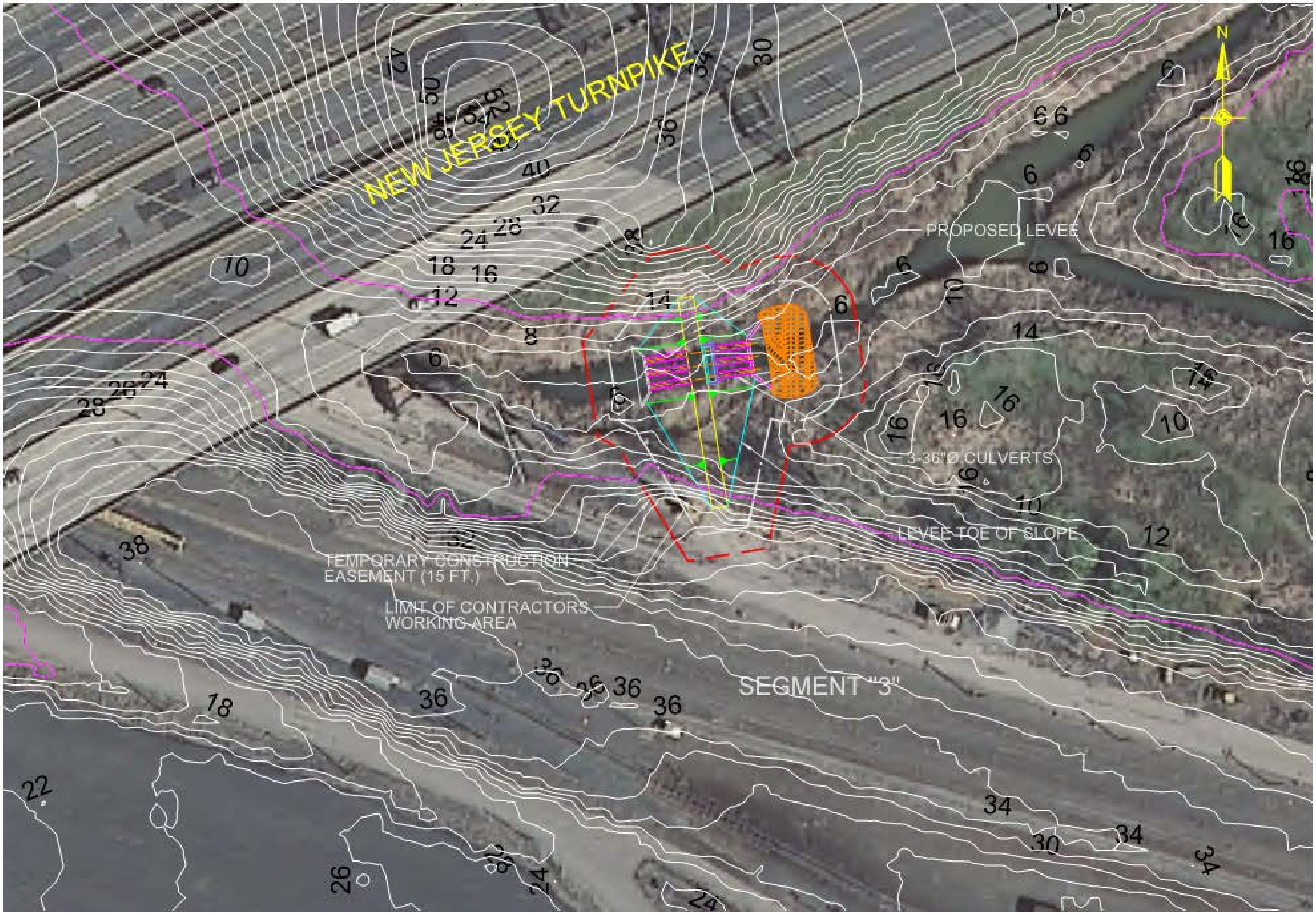

Figure 9: I-95 Reach - Segment 3

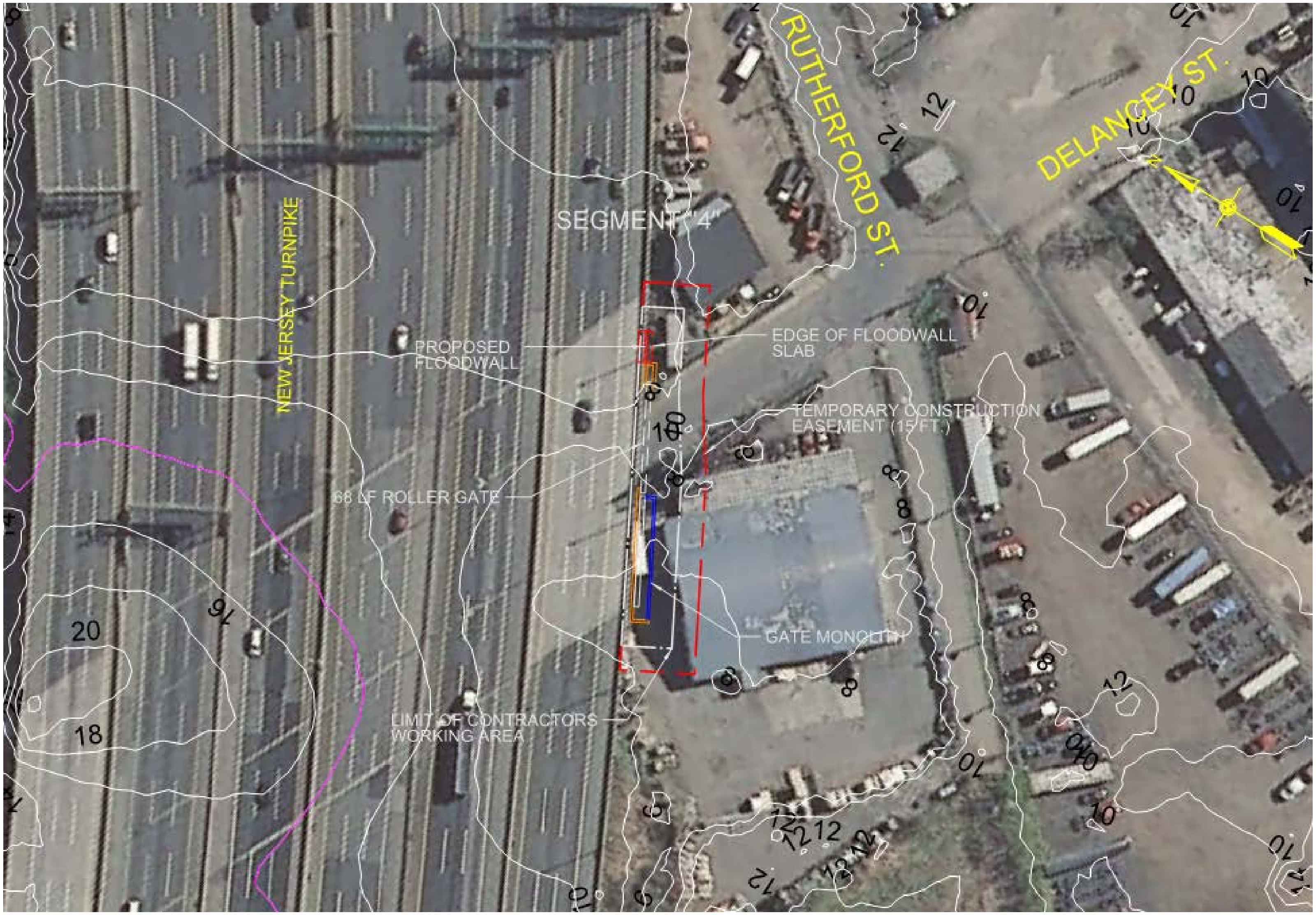

Figure 10: I-95 Reach - Segment 4

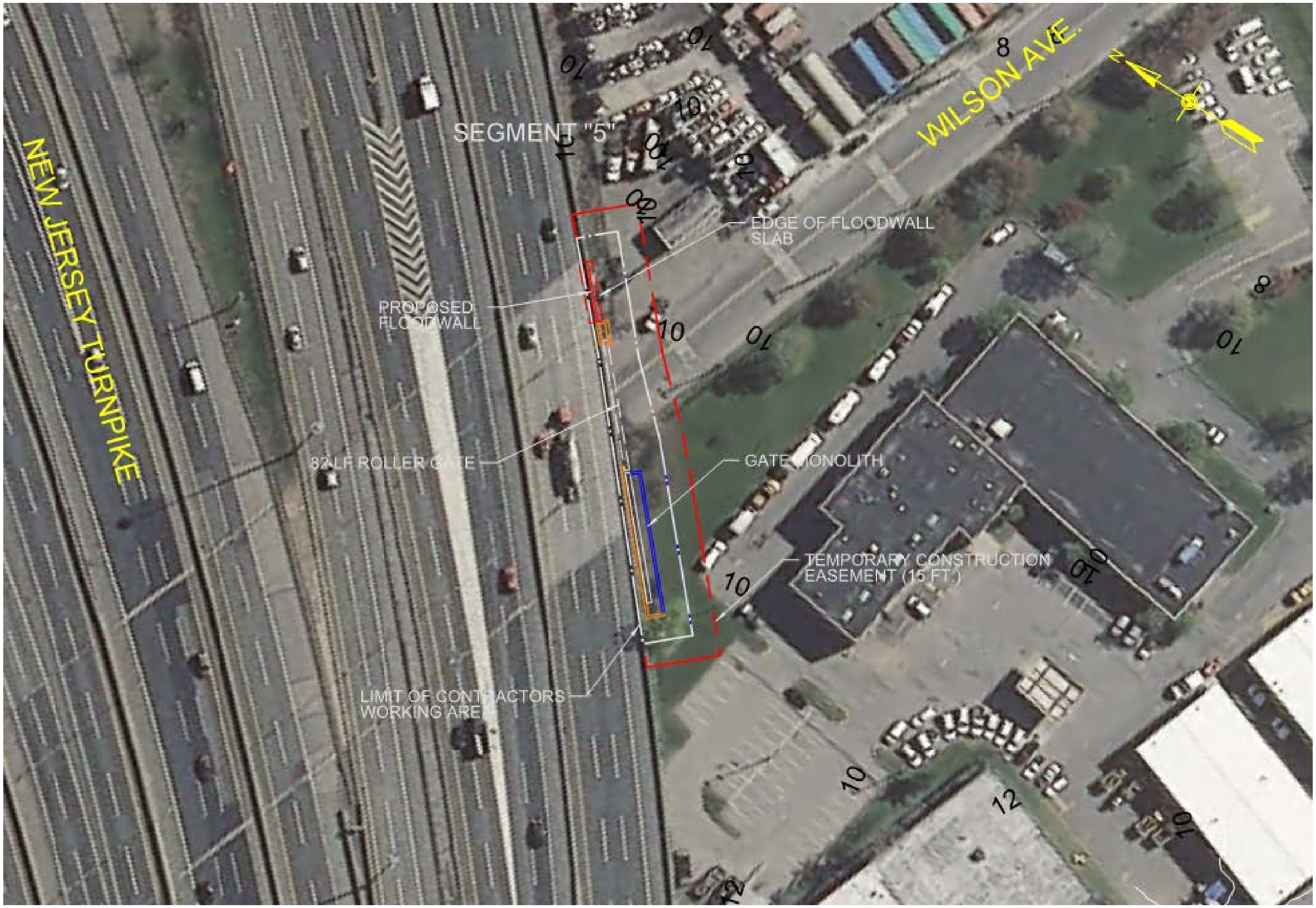

Figure 11: I-95 Reach - Segment 5

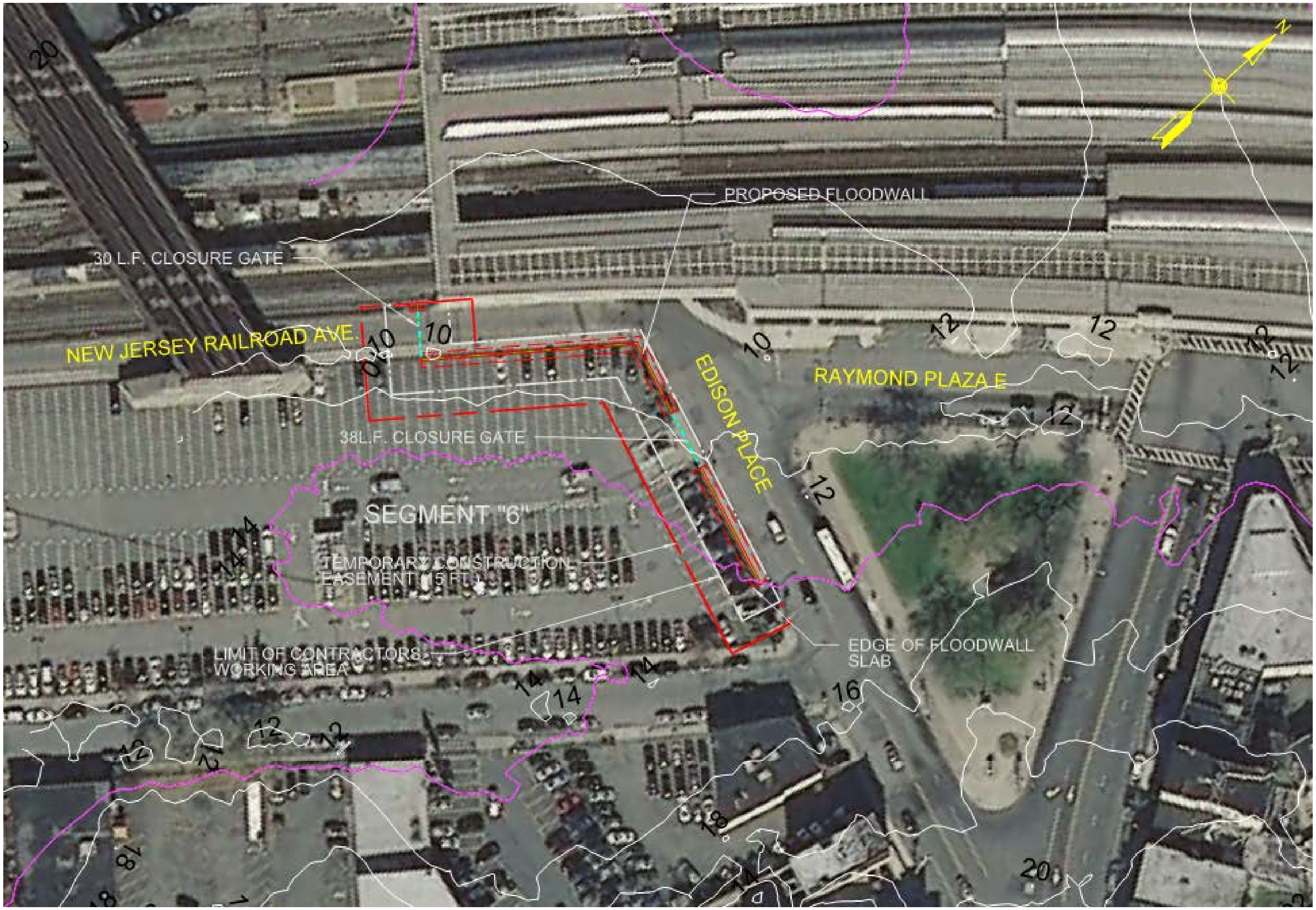

Figure 12: Minish Reach - Segment 6

Figure 13: Minish Reach - Segment 8

4.4 Existing Infrastructure Improvements

There are a number of stormwater and sanitary pipes that pass through or under the line of protection. These conduits require backflow prevention measures, with as backflow prevention devices and sluice gates or backflow chambers with devices and sluice gates, to limit tidal surge backflow into the flood risk management area. **Figure 14** shows the location of the major, existing conduits. Improvements to these features are discussed in Section 13. The presence, size, and condition of these pipes will need to be confirmed in the next phase of the project design.

4.5 Interior Drainage Features

The Recommended Plan's interior drainage plan is defined as the plan that maximizes the net excess benefits over cost. The interior drainage component for each sub-basin is presented in **Table 2** and shown in **Figure 14**. Selection of these features is discussed later in the appendix.

Table 2: **Recommended Plan Interior Drainage Plan Summary**

Basin	Description
Drainage Area 1	Tie low areas into existing 66" x 69" stormwater line
Drainage Area 2	50-foot gate adjacent to railroad
Drainage Area 3	3x36" Culverts in Segment 3 levee; 3x36" culverts under access road for drainage conduit
Drainage Area 4	No Additional Features
Drainage Area 5	No Additional Features

4.6 Flood Risk Management

The Recommended Plan provides flood risk management for the interior of the City of Newark up to a design elevation of 14 feet NAVD88. The 14 feet NAVD88 floodplain is shown in **Figure 15**. **Figure 16** depicts the with-project 14-foot NAVD88 floodplain.

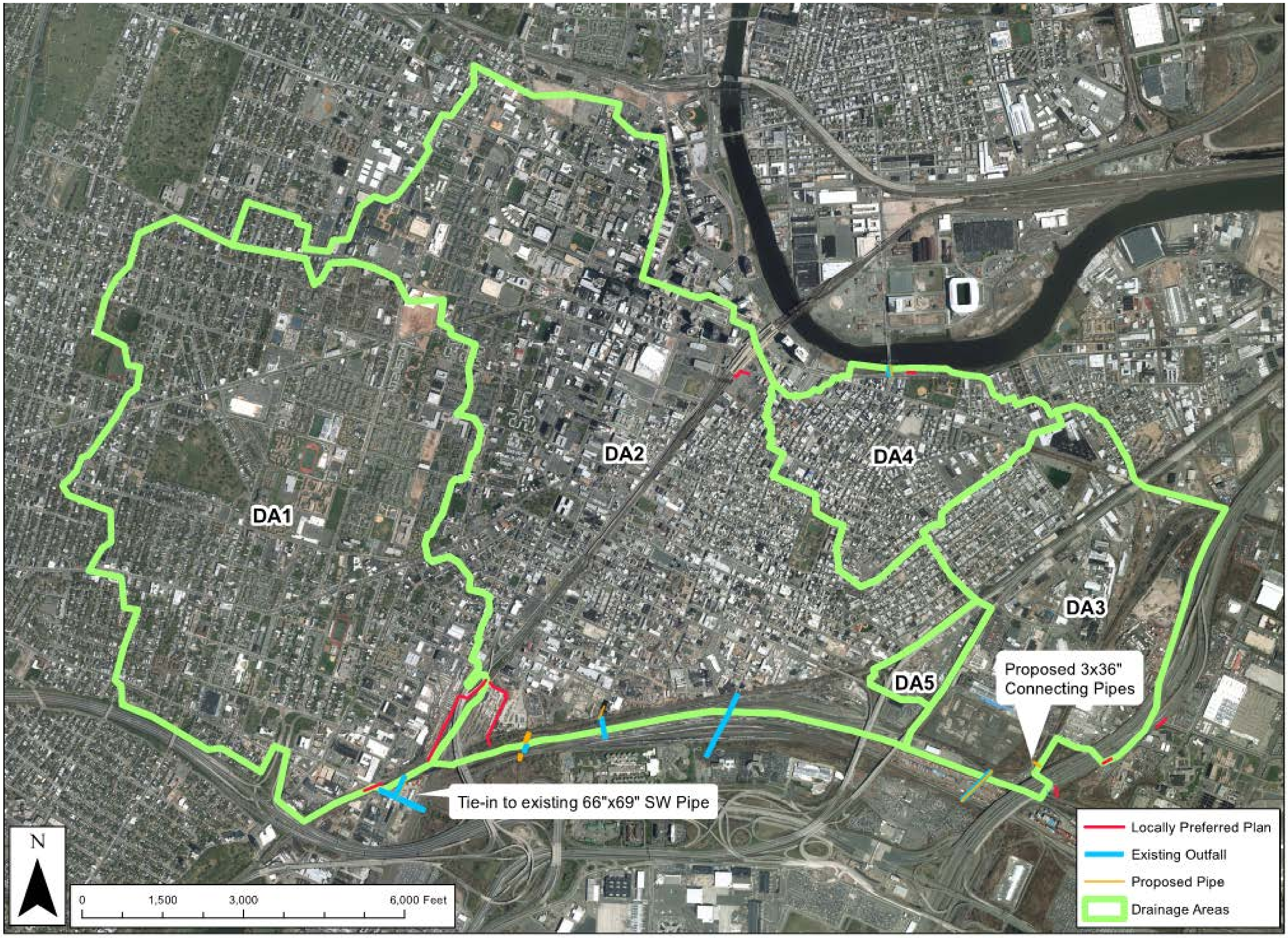

Figure 14: Passaic Tidal Project Existing and Interior Drainage Features

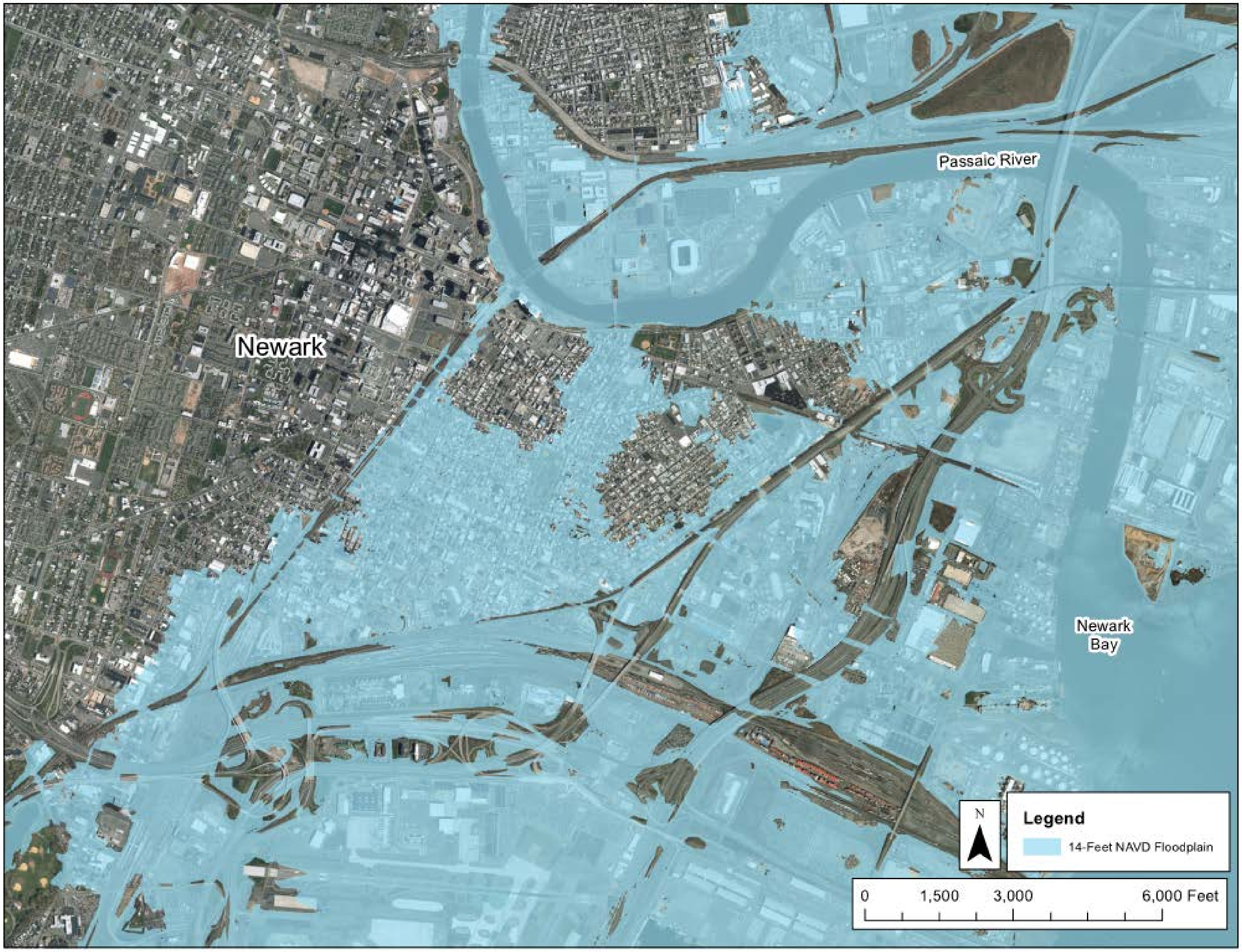

Figure 15: 14-feet NAVD88 Existing Floodplain

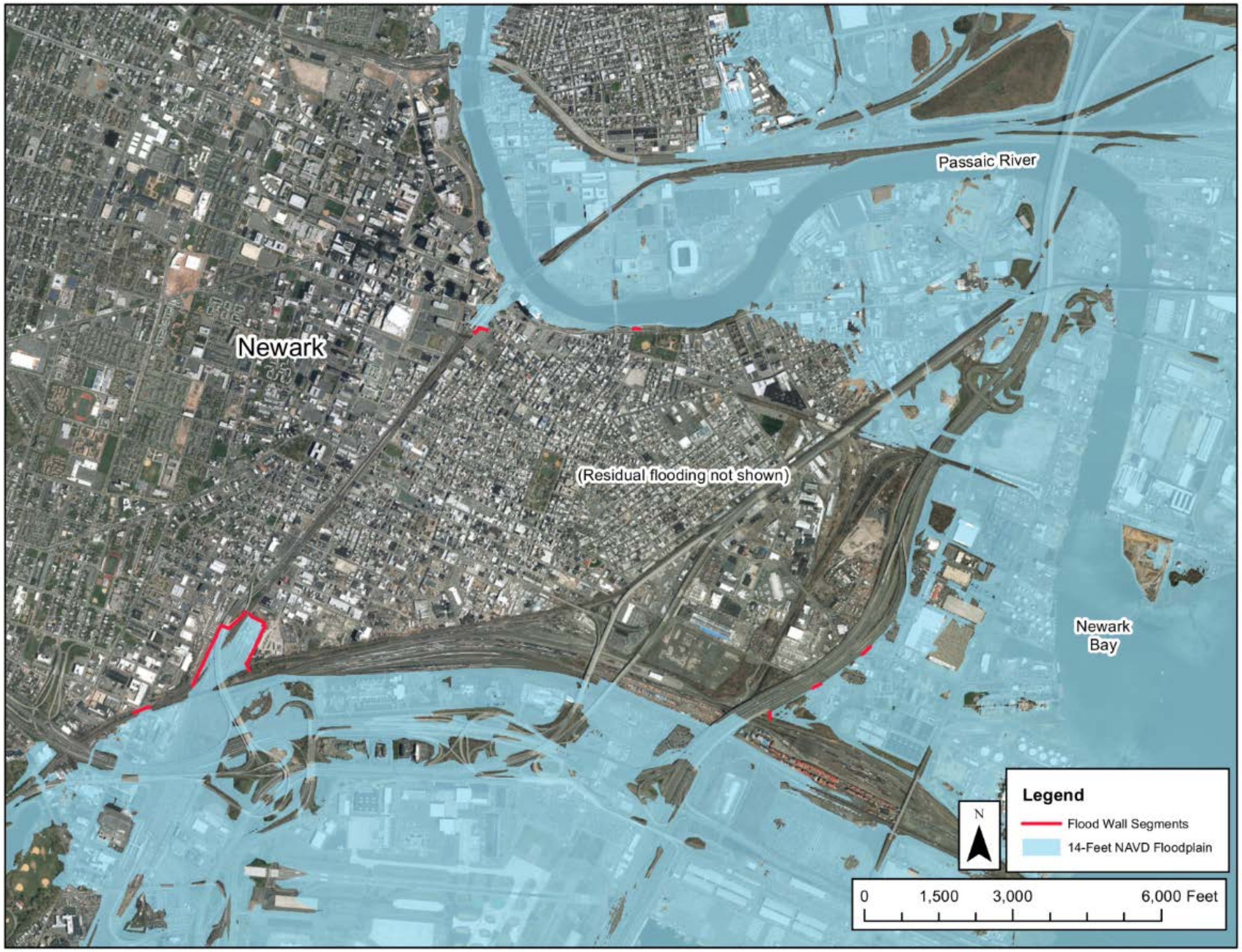

Figure 16: 14-feet NAVD88 With-Project Floodplain

5 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 (SWEL) 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 extratropical storms. 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 storms used in the model included over 1,000 synthetic tropical events and 100 extratropical events computed at over three million computational locations. The water levels were modeled to include the effects of storm surge, waves, and tides.

The 1992 tidal epoch was used in the initial NACCS coastal analysis; stillwater elevations in the project area were updated to 2020 levels using USACE Curve 1 projected sea level change data for the region (0.35 feet to 2020; 1.46 feet to 2070).

The NACCS stage versus frequency curve for the Passaic Tidal project area is shown in **Tables 3 and 4**.

Table 3: NACCS Stillwater Elevation - Stage versus Frequency (2020)

Annual Recurrence Interval (frequency)	ACE (probability)	SWEL (feet NAVD88)
1-year	0.99	5.37
2-year	0.5	6.23
5-year	0.2	7.41
10-year	0.1	8.34
25-year	0.04	9.57
50-year	0.02	10.80
100-year	0.01	12.09
250-year	0.004	13.67
500-year	0.002	14.99

Table 4: NACCS Stillwater Elevation - Stage versus Frequency (2070)

Annual Recurrence Interval (frequency)	ACE (probability)	SWEL (feet NAVD88)
1-year	0.99	6.48
2-year	0.5	7.34
5-year	0.2	8.52
10-year	0.1	9.44
25-year	0.04	10.67
50-year	0.02	11.90
100-year	0.01	13.19
250-year	0.004	14.78
500-year	0.002	16.10

6 INLAND STILLWATER LEVELS

Review of the 1995 GDM revealed that inland flooding as a result of potential flanking of the alignment in the vicinity of Newark Liberty International Airport (EWR) may not have been evaluated. However, closer review of the latest topographic mapping, Sandy Surge mapping, and Federal Emergency Management Agency (FEMA) preliminary flood mapping for Essex County, New Jersey (Reference 2) indicated that flanking was likely in the event of a large storm surge. As shown in **Figure 17**, the tidal surge from Hurricane Sandy inundated parts of the South Ironbound area of Newark via EWR. This is consistent with the FEMA preliminary flood mapping, shown in **Figure 18**.

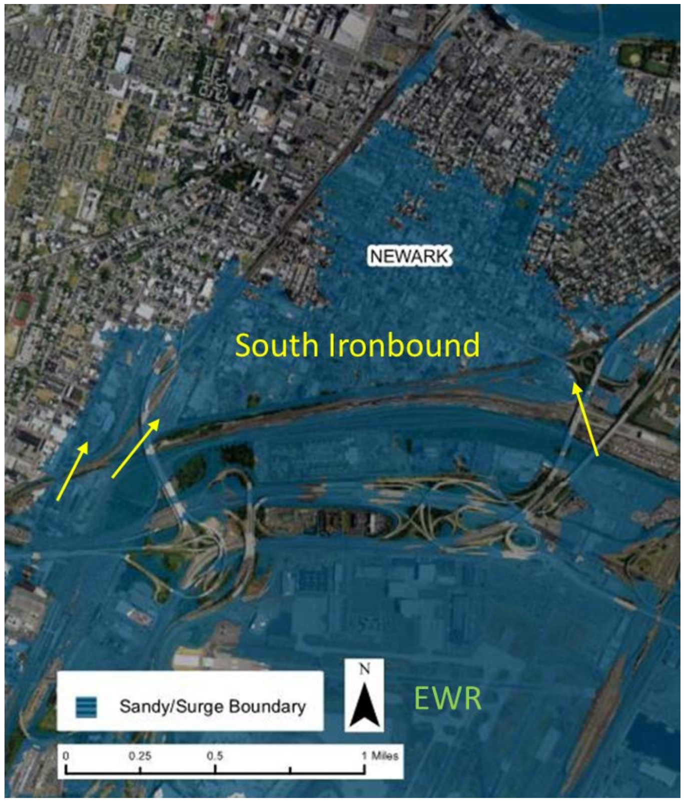

Figure 17: Sandy Surge – South Ironbound Area

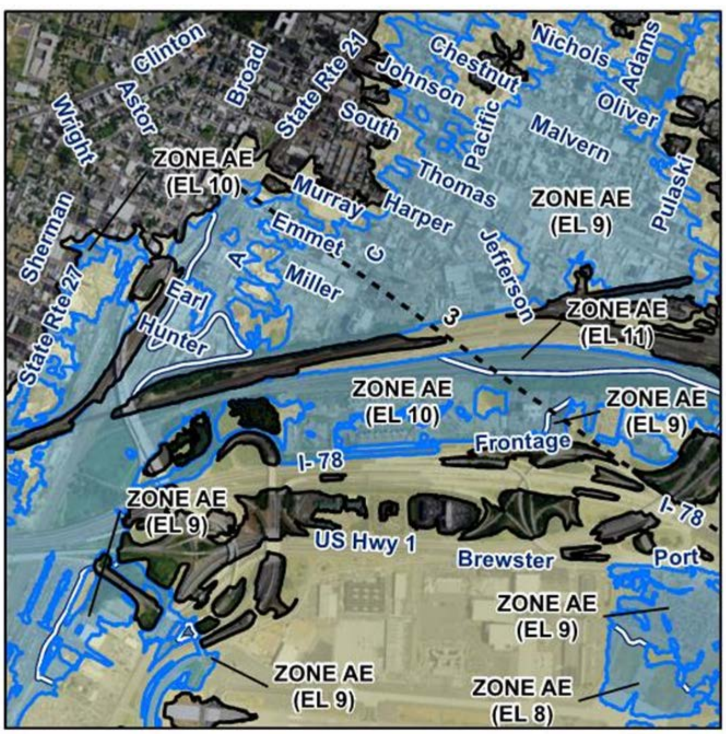

Figure 18: South Ironbound 1%/0.2% ACE Flooding (FEMA)

In order to assess the potential impacts of flanking at Segments 1, 2 and south of Segment 3, accurate inland flood elevations were necessary. The NACCS flood elevations are slightly higher than the FEMA flood elevations developed in 2013 for Newark Bay, as shown in **Table 5**; however, because the NACCS model did not include propagation of the surge inland, the FEMA model is a better representation of the inland surge elevations. The FEMA stillwater elevation in South Ironbound is lower but more accurately reflects potential flood risk for the area and allowed for a more accurate analysis of potential flood risk management measures.

Inland surge was further limited for high frequency storm events due to the tide gate and pump station on the Peripheral Ditch at Newark Liberty Airport (see **Figure 19**).

Table 5: NACCS/FEMA Stage versus Frequency Comparison

Annual Recurrence Interval (frequency)	ACE (probability)	NACCS SWEL (feet NAVD88)	FEMA (feet NAVD88)	South Ironbound (feet NAVD88)
2-year	0.5	6.2	3.8	2.8[3]
5-year	0.2	7.4	5.5	2.8[3]
10-year	0.1	8.3	6.9	2.8[3]
20-year	0.05	9.6	8.4 [1]	6.4
50-year	0.02	10.8	9.6	7.9
100-year	0.01	12.1	10.8	9.1
200-year	0.005	13.7	12.7 [2]	10.3
500-year	0.002	15.0	14.0	11.8

Notes:
[1] FEMA 25-year
[2] FEMA 250-year
[3] Controlled by Peripheral Ditch flood gate. Normal Tide elevation of 2.76 feet NAVD88.

Figure 19: Peripheral Ditch Tide Gate and Pump Station

7 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 on the GDM or coastal alignment. 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, and direction), and the nature of the beach, levee or wall being run up (i.e., slope, reflectivity, height, permeability, and 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 20**. 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. The runup and overtopping analysis includes levees as they were part of the 1995 alignment; however, levees were removed from the design following the geotechnical analysis. The discussion of levee runup and overtopping is included here for completeness.

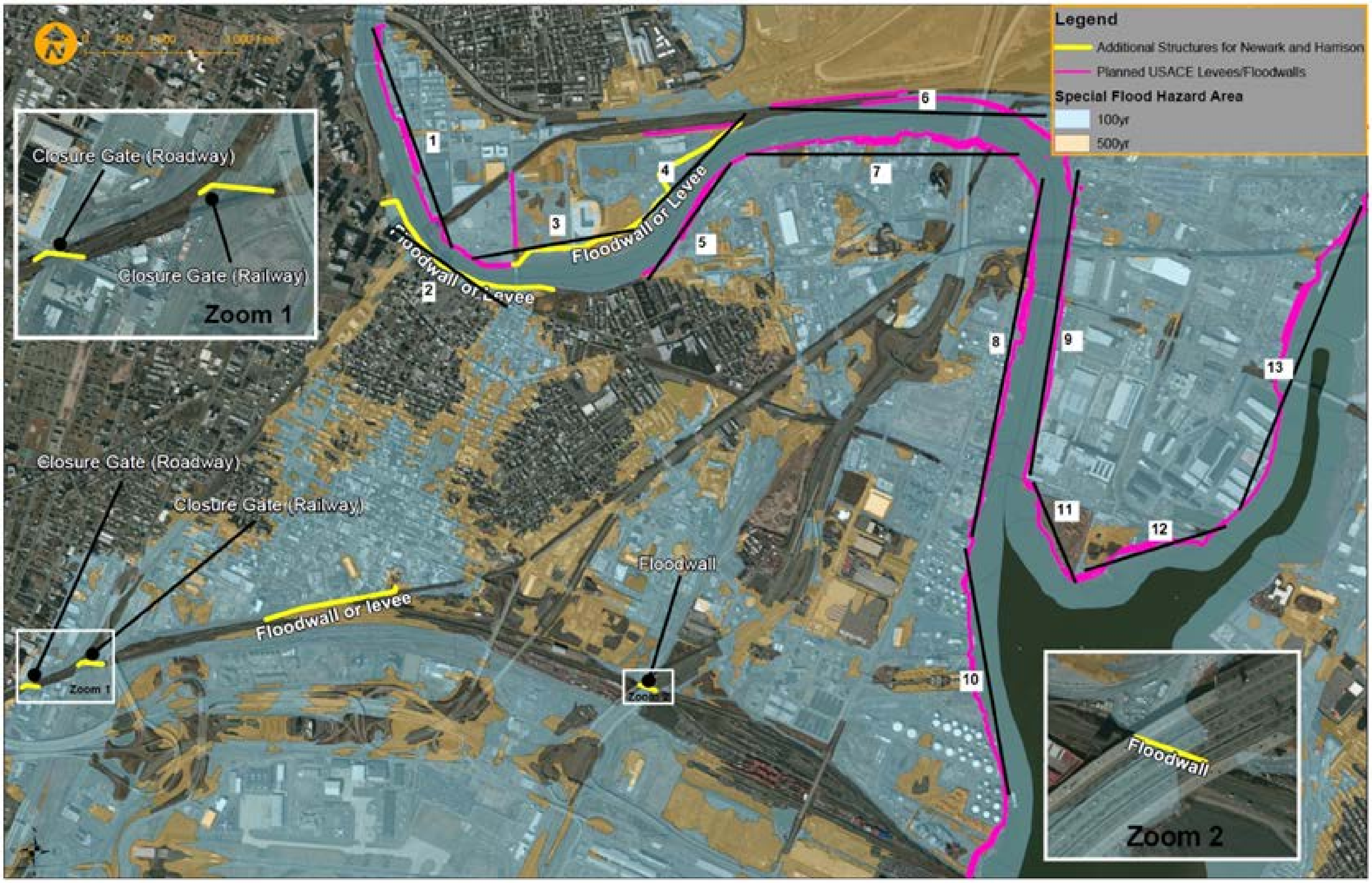

Figure 20: Segmentation of Levee / Floodwall System

7.1 Methods

Runup and overtopping were computed along Segments 10, 11, and 12 at the 100-, 200-, and 500-year recurrence intervals. The entire system is subject to storm surge and waves larger than 1.5 feet including both the Passaic River (Segments 1 through 9) and Hackensack River (Segment 13) during extreme events. However, only overtopping analyses were performed for representative upstream Segments 7 and 13. Storm surge elevations and wave parameters were calculated for the entire study area, and are reported below.

7.1.1 Storm Surge Water Surface Elevations

The USACE NACCS data provided the basis for the peak storm surge elevations at the 2-, 10-, 20-, 50-, 100-, 200-, and 500-year recurrence intervals. The NACCS data points in the study area were extracted from the model, converted to NAVD88 using USACE conversion factors, and extrapolated across the width of the local water body to create a continuous storm surge surface for each recurrence interval. **Figure 21** shows the NACCS data points along with the 100-year surge raster surface covering the study area.

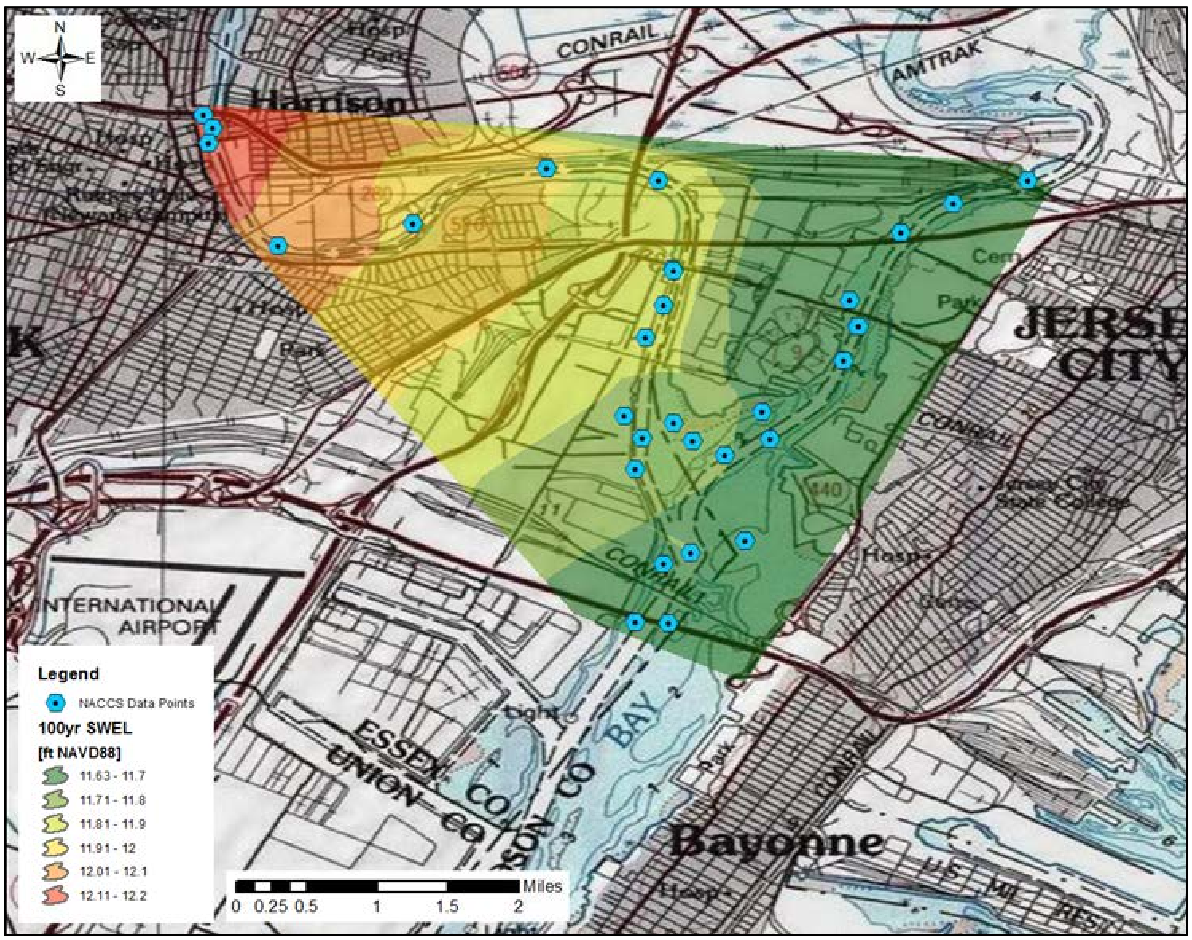

Figure 21: NACCS Data Points in the Study Area and Raster Surface (100-year SWEL)

7.1.2 Storm Surge Hydrograph

In order to facilitate future volumetric overtopping calculations, a time series of storm surge was created for the study area. Based on project experience in nearby areas, the created storm surge hydrograph assumed an extratropical storm. Tropical storms do impact the area, but are less common and typically less severe than extratropical. The peak of the hydrograph is normalized, so the main effect of this choice is to increase the event duration, as extratropical storms typically last longer. The extended duration of extratropical storms also makes them more conservative for most computations.

The method of creating a hydrograph is taken from Ayres et al. (Reference 3).

$$S_t(t) = S_P\left(1 - e^{-\left|\frac{D}{(t-t_0)}\right|}\right) \qquad (1)$$

Where:
S_t = storm surge elevation at time, t [feet]
S_p = peak storm surge elevation at time t_0 [feet]
$D = R/f$ = half the storm duration, [hours]

Where R = tropical storm radius of maximum wind, [nautical miles] and f = tropical storm forward speed [knots]. See the following paragraph for information on how this parameter was adapted for use in generating an extratropical surge hydrograph.

t = time [hours]
t_0 = time of landfall and peak surge [hours]

The approach described in the Ayres et al. document is designed for hurricane or tropical storm surges, given that it relies on a storm radius and forward speed, but it can be used to simulate extratropical surges as well. Because extratropical storms do not share the same organizational characteristics with tropical storms, the hydrograph cannot be created parametrically. Rather, we use the provided formula to define the shape of the hydrograph, and tune the width by altering the D parameter using historical storm data observed near the study site.

The nearest National Oceanic and Atmospheric Administration (NOAA) tide gauge to the study site is the Bergen Point West Reach New York gauge, number 8519483. The gauge site is at the southern end of Newark Bay, which is proximal to the confluence of the Hackensack and Passaic Rivers, and is proximal to the coast, which ensures the gage will record coastal storm surge, while the study area is at the northern end of the bay. Monthly water level maxima for the gauge historical record revealed a number of high surge events, and both predicted and observed data were downloaded targeting the highest events. Those events were cross checked against the National Hurricane Center's historical hurricane database to eliminate tropical storms and hurricanes from the sample. Five historical extratropical storms remained in the sample, and they were used to tune the D parameter in the synthetic hydrograph equation.

Five significant storms in the region including the December 12, 1992 and March 12-14, 2010 nor'easters, tropical storms Floyd (September 1999) and Irene (2011), and the April 2006 "Tax Day" storm were not included in the analysis. The nor'easters were not included in the development of the surge hydrograph because of data gaps at the Bergen Point gage. During December 1992, there was no reported hourly high/low water level or a highest water level published, so data from this month was not included in the analysis and selection of storms. The March 2010 nor'easter was also not included because the Bergen Point gage was inactive from December 2009 through March 2010. Tropical storms Floyd and Irene were also not included in the analysis because the study only investigated extratropical storms to develop the surge hydrograph. The analysis of extratropical storms in the surge hydrograph development is a conservative assumption because extratropical storms tend to produce wider (i.e. have longer durations) surge hydrographs than tropical storms; therefore, overtopping is calculated over greater periods of time. The extratropical and tropical storms do not share the same organizational characteristics; therefore, the study team determined that it was incorrect to use both types of storms to produce one surge hydrograph. The "Tax Day" nor'easter storm produced record breaking peak flows far upstream of the confluence of the rivers. The gage was more likely influenced by rainfall rather than coastal storm surge. Recorded high water levels at the Bergen Point gage during April 2006 are approximately 2/3 of the high surge values from storms used to create the surge hydrograph.

In order to compare disparate storms, the historical storm surge residuals were normalized (observed minus predicted water level), and time-adjusted to center the peak surge at time=0. A synthetic hydrograph was created for a 96-hour total duration, with 48 hours on each side of the peak surge also centered at time=0. These are shown in **Figure 22**. We then optimized the synthetic hydrograph rising and falling legs by adjusting the D parameter to minimize the root mean square error (RMSE) between the synthetic time series and all five historical storms. Because an asymmetric hydrograph was observed in some of the historic storms, the rising and falling legs were optimized separately; however, setting D = 6.3 hours provides the minimal RMSE for both the rising and falling legs. The unit synthetic storm surge hydrograph is provided in tabular format in Attachment 2, and can be used to obtain the surge hydrograph for the desired recurrence interval at the desired geographic location by multiplying the surge column by the appropriate peak surge value.

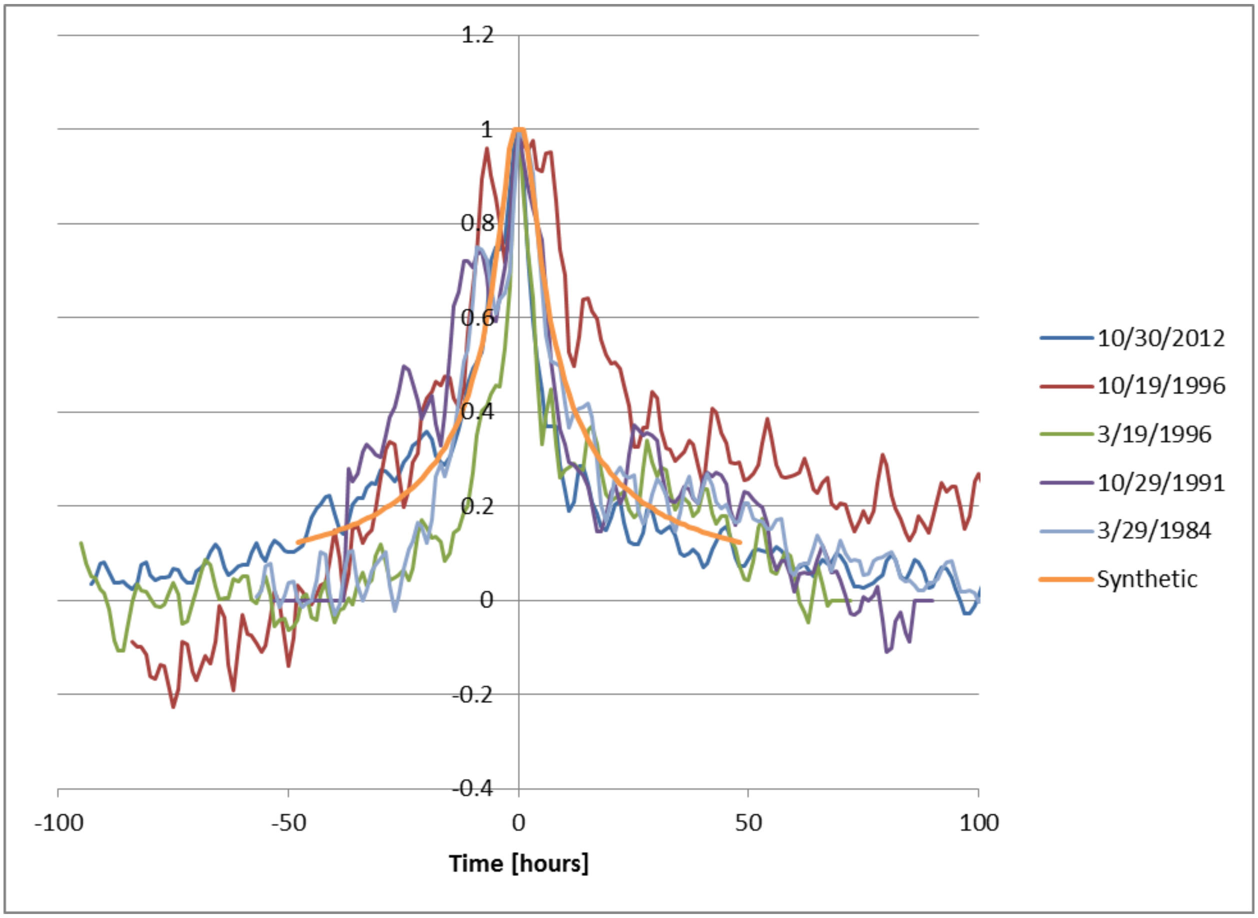

Figure 22: Normalized Historical Extratropical Storm Surge Hydrographs Compared with a Unit Synthetic Hydrograph (D=6.3 Hours)

7.1.3 Starting Wave Conditions

The study team determined the input wave conditions by processing the NACCS SWEL, wave height, and wave period datasets. In order to categorize storms with the appropriate recurrence interval, NACCS surge datasets were compared to the SWEL values described under the Storm Surge Water Elevations section of this report. NACCS storms within 0.3 meter of the 100-, 200-, or 500-year peak surge were compiled. The wave parameters from these groups of relevant storms were averaged for each recurrence interval to develop estimates of the wave height and SWEL for subsequent model inputs. Because the NACCS data are relatively dense, the wave heights and SWEL values were averaged to characterize the wave heights, periods, and SWEL along the upstream and downstream Passaic River, Newark Bay, and the Hackensack River.

To ensure the resulting NACCS wave heights and periods were reasonable, idealized fetch-limited test cases were simulated in the Automated Coastal Engineering System program with NACCS wind data. The resulting wave heights and periods were within the range of wave heights from the NACCS datasets, indicating compiled NACCS values were appropriate for analysis.

Tables 6, 7, and 8 show the input wave heights, periods, and SWELs applied to compute runup and overtopping for the 100-, 200-, and 500-year storms.

Table 6: **Input Wave Heights**

Wave Heights (feet)				
Annual Recurrence Interval (frequency)	Upstream Passaic (Reaches 1 to 7)	Downstream Passaic (Reaches 8 to 9)	Newark Bay (Reaches 10 to 12)	Hackensack River (Reach 13)
100-year	1.94	2.30	3.02	2.07
200-year	1.97	2.33	3.02	2.26
500-year	2.30	3.41	4.13	2.56

Table 7: **Input Wave Periods**

Wave Periods (seconds)				
Annual Recurrence Interval (frequency)	Upstream Passaic (Reaches 1 to 7)	Downstream Passaic (Reaches 8 to 9)	Newark Bay (Reaches 10 to 12)	Hackensack River (Reach 13)
100-year	2.7	3.0	3.1	2.8
200-year	2.7	3.3	3.2	2.9
500-year	2.7	3.5	3.5	2.9

Table 8: **Input SWEL Values**

SWEL (feet NAVD88)				
Annual Recurrence Interval (frequency)	Upstream Passaic (Reaches 1 to 5)	Mid/Lower Passaic and Newark Bay (Reaches 6 to 11)	Northeast Newark Bay (Reach 12)	Hackensack River (Reach 13)
100-year	12.10	11.80	11.70	11.50
200-year	13.09	13.03	13.04	12.59
500-year	14.52	14.71	14.64	14.07

7.1.4 Structure Dimensions

The USACE GDM (References 4 and 5) contains information on the design crest elevation for floodwalls and levees, along with design levee side slope. Throughout the system, the target design crest elevation is 14.9 feet NGVD29 (or 13.8 feet NAVD88). Levee side slopes are designed at 1V:3H. Floodwalls are designed to be vertical structures.

In order to consider alternatives to the GDM design elevations, runup and overtopping were evaluated for a 14 foot NAVD88 design elevation and for higher crests of +2 and +4 feet.

Segments 10, 11, and 12 were initially composed of both levees and floodwalls, and were subdivided further so that the appropriate runup and overtopping formulas could be used for each structure type. Those subdivisions are shown in **Table 9** and referenced to GDM stationing. There is a small section of high ground within Segment 12, from station 114+40 to 116+40, that does not need to be augmented with a structure in the original GDM design targeting a 14-feet NAVD88 structure elevation, but would fall below the GDM+2 feet and GDM+4 feet elevations and would need supplementation. For this reason, we treated the high ground as if it were part of the adjacent levee Segment 12 for the GDM and alternative design elevations.

Table 9: **Summary of Runup and Overtopping Analysis Subdivisions**

Segment	Structure Type	Approximate Stationing (Reference 5)	GDM Design Elevation (feet NAVD88)	Side Slope
10	Floodwall	160+00 – 223+00	13.8	Vertical
11	Floodwall	85+00 – 102+90	13.8	Vertical
11	Levee	102+90 – 105+00	13.8	1:3
12	Levee	105+00 – 129+45	13.8	1:3
12	Floodwall	129+45 – 135+00	13.8	Vertical

7.1.5 Runup

Wave runup height is defined as the vertical difference between the highest point of wave runup and the stillwater level. Runup is computed along the levees in the Passaic levee/floodwall system but is not calculated along the floodwalls. In this analysis, the wave runup height was computed for levee structures using the formulation provided in the EurOtop Manual (Reference 6), Equation 2, which determines the wave runup exceeded by two percent of incoming waves, for Segments 10, 11, and 12 in the Passaic levee/floodwall system using

$$R_{u2\%} = 1.65\gamma_b\gamma_f\gamma_\beta\varepsilon_{m-1,0}H_{m0} \qquad (2)$$

Where:

γ_b is the berm influence factor
γ_f is the slope roughness influence factor
γ_β is the oblique wave attack influence factor
$\varepsilon_{m-1,0}$ is the breaker parameter
H_{m0} is the wave height [feet]

In this analysis, it was assumed that there is no berm during the 100-, 200-, and 500-year events so γ_b equals one. It is also assumed that waves approach at a shore normal angle and that the levees are constructed of concrete so both γ_β and γ_f also equal one. We calculated runup assuming the face of the structure is higher than the actual runup. Furthermore, in Equation 2, $\varepsilon_{m-1,0}$, also known as the Iribarren number, is the breaker parameter defined by Equation 3.

$$\varepsilon_{m-1,0} = \frac{\tan\alpha}{\sqrt{\frac{H_{mo}}{L_o}}} \qquad (3)$$

In Equation 3, α is the structure's seaward slope steepness and L_0 is the deep water wave length defined as

$$L_0 = \frac{gT_p{}^2}{2\pi} \qquad (4)$$

Where g is gravity and T_p is the peak wave period.

Table 10 lists the wave runup heights along the levee portions of Segments 11 and 12 determined by the EurOtop Manual equations. As shown in **Table 9**, Segment 10 of the Passaic levee/floodwall system is constructed only with floodwalls; therefore, runup calculations are not performed along this segment. In general, increases in wave height – associated with stronger storm conditions – induce larger wave runup heights.

Table 10: **Wave Runup Heights along Segments 10, 11, and 12 of the Passaic Levee System**

Runup Elevations (feet NAVD88)						
Annual Recurrence Interval (frequency)	Segment 10		Segment 11		Segment 12	
	Levee	Floodwall	Levee	Floodwall	Levee	Floodwall
100-year	+	*	18.5	*	18.4	*
200-year	+	*	20.0	*	20.0	*
500-year	+	*	23.6	*	23.5	*
+ There are no levees planned for Segment 10 * Runup was not computed explicitly for floodwalls						

7.2 Overtopping

The overtopping methodologies for levees and floodwalls discussed in the EurOtop Manual were applied in these analyses. Overtopping equations are largely empirical, and multiple formulations exist to compute overtopping based upon the wave breaking and freeboard conditions. Freeboard is the height of a structure in excess of the local stillwater. A general discussion of overtopping along simple sloped structures and floodwalls is included in this section.

Equations for levee overtopping are largely dependent on whether the stillwater level is below, equal to, or greater than the crest elevation of the levee. As the difference between the water level and structure's crest elevation decreases, overtopping of the structure increases. Furthermore, overtopping is affected by the presence or absence of wave breaking, which is captured in the Iribarren number. For overtopping calculations, the same influence factors discussed for Equation 1 (i.e., γ_b, γ_β, and γ_f) are used. Additionally, an influence parameter for small, vertical walls commonly placed on top of levees to reduce overtopping is included in the equations. The value of this parameter is set equal to one for the computations in this analysis since the proposed levees will not be designed with a vertical wall at the crest. Depending on the amount of freeboard and the breaking parameter, overtopping is computed using multiple formulae provided in the EurOtop Manual.

Overtopping along floodwalls is a complex process that varies with wave impulsiveness, or breaking. Within the classes of non-impulsive and impulsive waves, multiple formulations to compute the overtopping can be used. Proper selection of the formula typically depends on some type of dimensionless freeboard criterion. In the analyses conducted, Equations 7.3, 7.5, 7.6, and 7.8 from the EurOtop manual are utilized to estimate floodwall overtopping; for simplicity those equations and their limitations are omitted from this document. These equations are intended for probabilistic design and comparison with data measurements for plain vertical walls.

7.2.1 Segments Subject to Waves

Tables 11, 12, and 13 show the overtopping flux per unit length in cubic feet per second per fott ($ft^3/s/ft$) along Segments 10, 11, and 12 during the 100-, 200-, and 500-year storms at the analysis heights of 14 feet, 16 feet, and 18 feet NAVD88, respectively.

Table VI-5-6 on page VI-5-24 of the Coastal Engineering Manual (Reference 7) 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 $ft^3/s/ft$. As shown in the following tables, the 16 feet NAVD88 alternative elevation limits overtopping to acceptable levels up to the 200-year recurrence interval; the 18 feet NAVD88 floodwall limits acceptable overtopping up to the 500-year recurrence interval.

Table 11: **Flux Per Unit Length for Coastal Segments, Alternative Elevation of 14 feet NAVD88**

Flux per Unit Length ($ft^3/s/ft$)						
Annual Recurrence Interval (frequency)	Segment 10		Segment 11		Segment 12	
	Levee	Floodwall	Levee	Floodwall	Levee	Floodwall
100-year	-	0.179	0.355	0.179	0.316	0.164
200-year	-	0.516	1.862	0.516	1.876	0.521
500-year	-	4.994	5.364	4.994	5.070	4.700

-There are no levees planned for Segment 10.

Table 12: Flux Per Unit Length for Coastal Segments, Alternative Elevation of 16 feet NAVD88

Flux per Unit Length ($ft^3/s/ft$)						
Annual Recurrence Interval (frequency)	Segment 10		Segment 11		Segment 12	
	Levee	Floodwall	Levee	Floodwall	Levee	Floodwall
100-year	-	0.032	0.034	0.032	0.031	0.029
200-year	-	0.092	0.166	0.092	0.168	0.093
500-year	-	0.848	2.708	0.848	2.587	0.811

-There are no levees planned for Segment 10.

Table 13: Flux Per Unit Length for Coastal Segments, Alternative Elevation of 18 feet NAVD88

Flux per Unit Length ($ft^3/s/ft$)						
Annual Recurrence Interval (frequency)	Segment 10		Segment 11		Segment 12	
	Levee	Floodwall	Levee	Floodwall	Levee	Floodwall
100-year	-	0.006	0.003	0.006	0.003	0.005
200-year	-	0.016	0.017	0.016	0.017	0.017
500-year	-	0.241	0.392	0.241	0.368	0.231

-There are no levees planned for Segment 10.

7.2.2 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. Along stretches of Segments 7 and 13, locations of high ground greater than 20 feet exist. The extents of the high ground are listed but are not used for calculating overtopping since they exceed all design elevations. **Table 14** indicates the stationing and structure type for the representative upstream segments.

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, 16, and 18 feet NAVD88 alternative elevations are listed in **Tables 15, 16, and 17**, respectively.

Table 14: Summary of Overtopping Subdivisions for Riverine Segments

Segment	Structure Type	Stationing (Reference 5)	GDM Design Elevation (feet NAVD88)	Side Slope
7	Floodwall	35+00 – 55+24 63+08 – 71+90 84+03 – 85+00	13.8	Vertical
7	Levee	55+24 – 63+08 71+90 – 82+90	13.8	1:3
7	High Ground	82+90 – 84+03	>20	Unknown
13	Floodwall	140+00 – 171+40 172+50 – 178+55 191+33 – 238+70	13.8	Vertical
13	High Ground	171+40 – 172+50	>20	Unknown
13	Levee	171+40 – 172+50	13.8	1:3

Table 15: **Flux Per Unit Length for Riverine Segments, Alternative Elevation of 14 feet NAVD88**

Flux per Unit Length ($ft^3/s/ft$)				
Annual Recurrence Interval (frequency)	Segment 7		Segment 13	
	Levee	Floodwall	Levee	Floodwall
100-year	0.065	0.032	0.058	0.029
200-year	0.632	0.174	0.427	0.153
500-year	3.463	3.261	1.770	1.503

Table 16: **Flux Per Unit Length for Riverine Segments, Alternative Elevation of 16 feet NAVD88**

Flux per Unit Length ($ft^3/s/ft$)				
Annual Recurrence Interval (frequency)	Segment 7		Segment 13	
	Levee	Floodwall	Levee	Floodwall
100-year	0.002	0.002	0.003	0.002
200-year	0.019	0.012	0.024	0.015
500-year	0.423	0.183	0.269	0.131

Table 17: **Flux Per Unit Length for Riverine Segments, Alternative Elevation of 18 feet NAVD88**

Flux per Unit Length ($ft^3/s/ft$)				
Annual Recurrence Interval (frequency)	Segment 7		Segment 13	
	Levee	Floodwall	Levee	Floodwall
100-year	0.000	0.000	0.000	0.000
200-year	0.001	0.001	0.001	0.002
500-year	0.020	0.019	0.018	0.017

7.3 Waves and the Locally Preferred Plan

Because the Recommended Plan alignment is set back from river and bay shorelines, it is not expected to experience any significant wave action during surge events. Any waves from Newark Bay or from the south will be dampened by existing buildings and wave-limiting flood depths. Therefore, wave impacts and overtopping were not considered in the structural and interior drainage analyses of the locally preferred plan.

8 SEA LEVEL CHANGE

Current 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 December 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. ER 1100-2-8162 spells out how SLC is to be computed and incorporated into levee/floodwall 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 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 equations at a low, intermediate, and high level. The output for the USACE equations can be seen in **Table 18**. The program also plots a chart of the sea level curves as seen in **Figure 23**.

The inclusion of SLC affects the design height performance and reliability, which can be evaluated using the probability of non-exceedance. The probability of non-exceedance is discussed in the Economics Appendix.

Table 18: Sea Level Change, Passaic Tidal Project Area

Year	USACE Low (feet)	USACE Int. (feet)	USACE High (feet)
2020	0.00	0.00	0.00
2030	0.13	0.19	0.37
2040	0.26	0.39	0.82
2050	0.38	0.61	1.34
2060	0.51	0.85	1.94
2070	0.64	1.11	2.61
2080	0.77	1.39	3.35
2090	0.90	1.68	4.17
2100	1.02	1.99	5.06
2110	1.15	2.32	6.02
2120	1.28	2.67	7.06

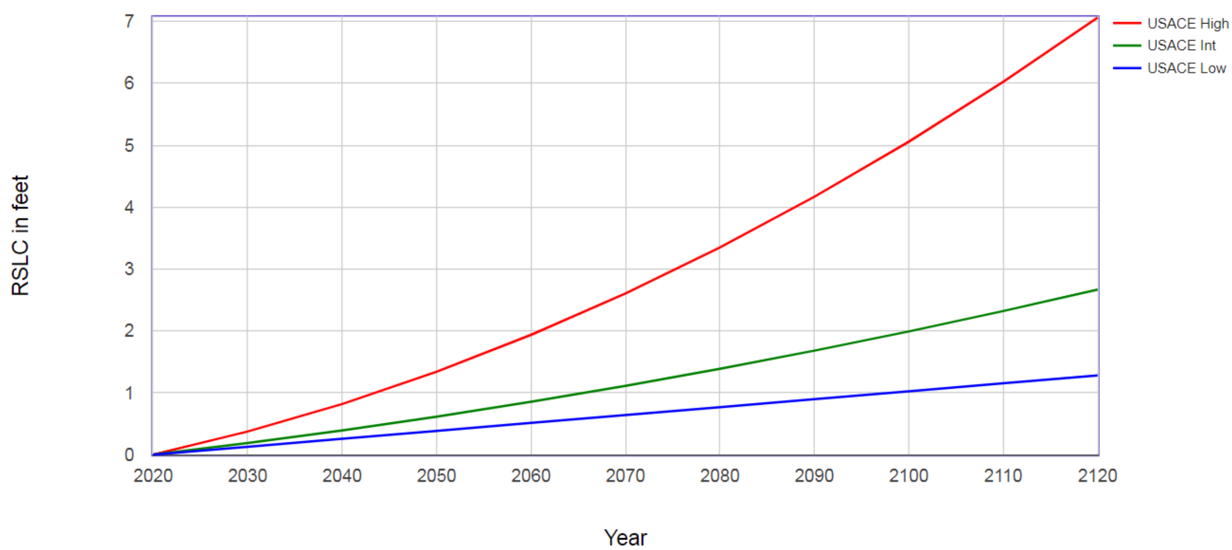

Figure 23: SLC Scenario Projections (Sandy Hook, NJ)

9 INTERIOR DRAINAGE ANALYSIS

9.1 Overview

Areas protected from exterior flood elevations are subject to interior residual flooding from stormwater runoff. Thus, interior drainage facilities may be required to safely store and discharge the runoff to limit interior residual flooding. The interior areas were studied to determine the specific nature of flooding and to formulate drainage alternatives to maximize NED benefits.

In accordance with USACE EM 1110-2-1413, *Hydrologic Analysis of Interior Areas*, the interior drainage facilities are evaluated separately from the alignment. First, a minimum facility plan is identified. The minimum facility plan is considered the smallest plan that can be implemented as part of the alignment that does not result in increased stormwater flooding as a result of project construction (residual damages). It is the starting point from which additional interior facilities planning commences.

Next, the benefits accrued from alternative interior drainage plans are attributed to the reduction in the residual flood damages which may have remained under the minimum facility condition. Finally, an optimum drainage alternative is selected based on meeting NED objectives.

The interior drainage facilities must be formulated to maximize NED benefits while meeting NED objectives to provide a complete, effective, efficient, and acceptable plan of flood risk management.

- **Completeness** is defined in Engineer Regulation (ER) 1105-2-100 as, *the extent to which the alternative plans provide and account for all necessary investments or other actions to ensure the realization of the planning objectives, including actions by other Federal and non-Federal entities*.

- **Effectiveness** is defined as, *the extent to which the alternative plans contribute to achieve the planning objectives.*

- **Efficiency** is defined as, *the extent to which an alternative plan is the most cost-effective means of achieving the objectives.*

- **Acceptability** is defined as, *the extent to which the alternative plans are acceptable in terms of applicable laws, regulations, and public policies.*

9.1.1 NED Plan Interior Drainage

As part of the GRR, the interior drainage plan from the 1995 GDM was remodeled and evaluated. The plan included 160 outfalls and six pump stations. The plan was not reformulated; therefore, new interior drainage alternatives for the GDM were not considered. The following is a description of the general components of the NED Plan 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 structure.

2) Pump Stations: There are six pump stations in the interior drainage plan. They range from 30 to 100 cfs.

The drainage areas analyzed for the NED Plan are similar to the areas in the 1995 GDM; however, the areas were verified/redelineated using updated topographic data from 2012. This resulted in some minor changes. Drainage area runoff parameters were unchanged from the 1995 GDM.

9.1.2 Recommended Plan Interior Drainage

The development of a Recommended Plan necessitated a new, separate interior drainage analysis of potential residual flooding with the Recommended Plan's alignment, which was not included as part of the NED Plan interior drainage analysis.

An overview of the interior drainage analysis of the Recommended Plan and results are discussed in the following sections. Detailed discussion of the interior drainage analyses for the Recommended Plan and NED Plan are included in **Subappendices 1 and 2**.

9.2 Recent Storm History

Essex County is subject to impacts from coastal storms, often characterized as nor'easters, which are most frequent between October and April. These storms track over the coastal plain or up to several hundred miles offshore, bringing strong winds and heavy rains. Rarely does a winter go by without at least one significant coastal storm and some years see upwards of five to ten. Tropical storms and hurricanes are also a special concern along the coast. In some years, they contribute a significant amount to the precipitation totals of the region. Damage during times of high tide can be severe when tropical storms or nor'easters affect the region.

Flooding in Essex County can occur during during any season of the year since New Jersey lies within the major storm tracks of North America. The worst storms have occurred in late summer or early fall when tropical disturbances (hurricanes) are most prevalent. Recent tropical events include Tropical Storm Floyd, Hurricane Irene, and Hurricane Sandy.

Hurricane Floyd originally made landfall in Cape Fear, North Carolina as a Category 2 hurricane. The storm crossed over North Carolina and southeastern Virginia before briefly entering the western Atlantic Ocean. The storm reached New Jersey on September 16, 1999, as a tropical storm. Record breaking flooding from rainfall exceeding 14 inches was recorded throughout the State of New Jersey. A Federal Emergency Declaration was issued on September 17, 1999 and a Major Disaster Declaration was issued on September 18, 1999.

Having earlier been downgraded to a tropical storm, Hurricane Irene came ashore in Little Egg Inlet in Southern New Jersey on August 28, 2011. In anticipation of the storm Governor Chris Christy declared a state of emergency on August 25, with President Obama reaffirming the declaration on August 27. Mandatory evacuations were ordered throughout the State of New Jersey. Wind speeds were recorded at 75 miles per hour (mph) and rainfall totals reached over 10 inches in many parts of the state. Extensive flooding throughout Essex County caused damage to homes, businesses, and public infrastructure. The flooding was exacerbated by high water levels in reservoirs and wetlands as a result of previous heavy rains. Over one million customers lost power during the storm. Overall damage estimates for the State of New Jersey came to over one billion dollars, with over 200,000 homes and buildings being damaged. A Major Disaster Declaration was issued on September 15, 2011.

Hurricane Sandy came ashore as an immense tropical storm in Brigantine, New Jersey, on October 29, 2012. Although rainfall was limited to less than 2 inches within Essex County, wind gusts were recorded up to 76 mph. A full moon made the high tides 20 percent higher than normal and amplified the storm surge. The New Jersey shore suffered the most damage. Seaside communities were damaged and destroyed up and down the coastline. Some 2.7 million households within New Jersey lost power. Initial reports suggested that 72,000 homes and businesses statewide were damaged or destroyed by the storm. Hurricane Sandy was estimated to cost the State of New Jersey over $36 billion. A Federal Emergency Declaration was issued on October 28, 2012 and a Major Disaster Declaration was issued on October 30, 2012.

9.3 Study Area

The study area encompasses 5.0 square miles in the City of Newark, 0.65 square miles in the Town of Harrison, and 2.73 square miles in the Town of Kearny. The Passaic and Hackensack Rivers intersect the study area as shown in **Figure 24**.

The study area is a mixed use area of industrial, commercial, and residential development. The waterfront is mostly developed for industrial uses including shipping (oil and gas, containers/consumer goods) and wastewater treatment. Related rail, barge, truck, and storage infrastructure line the waterfront. The NED Plan project segments are shown in **Figure 25**; the Recommended Plan segments are shown in **Figure 26**.

Figure 24: Interior Drainage Study Area

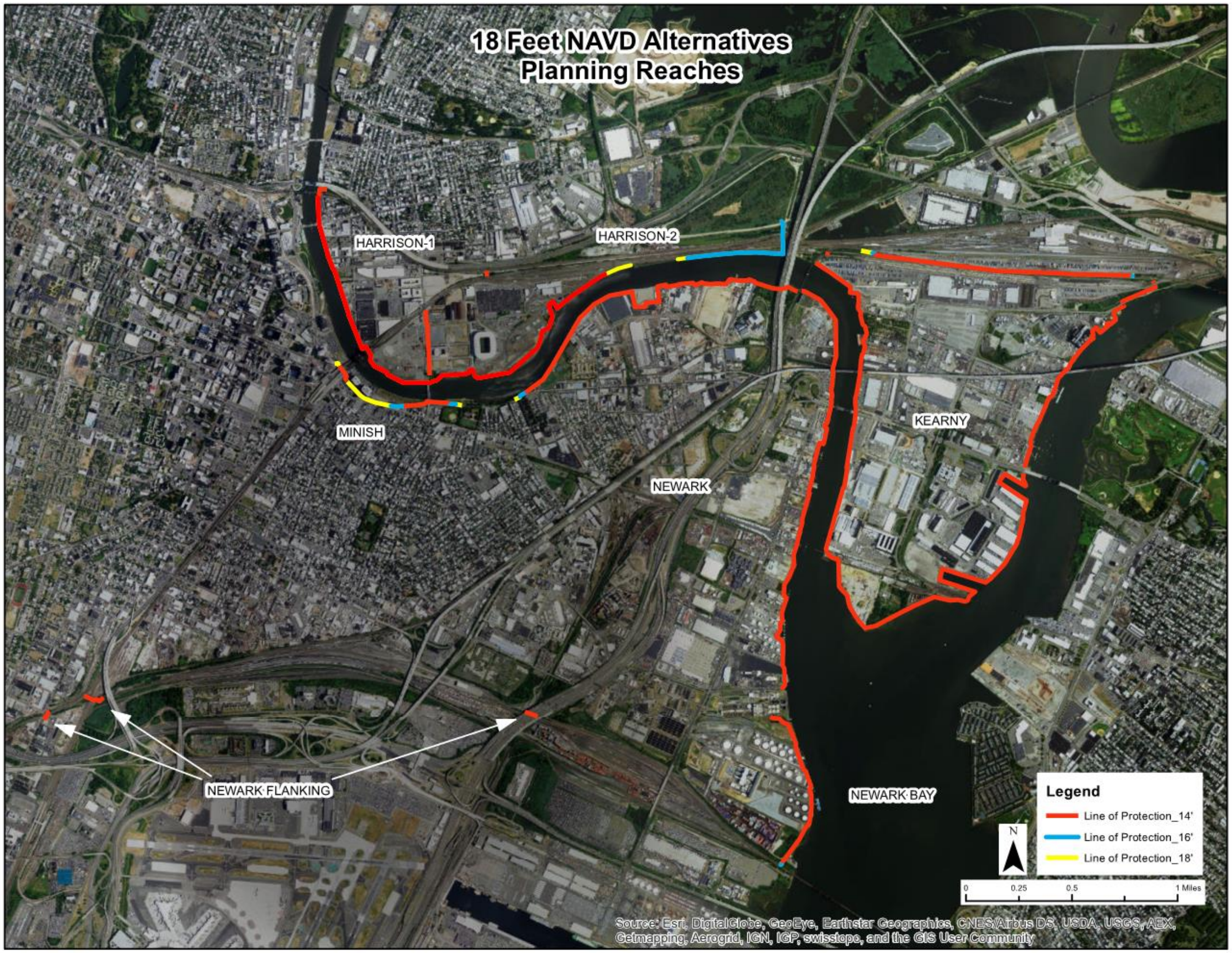

Figure 25: NED Plan Project Segments

Figure 26: Recommended Plan Project Segments

9.4 Interior Drainage Methodology

Areas protected from exterior flood elevations are subject to interior flooding from stormwater runoff. Thus, interior drainage facilities are required to safely store and discharge the runoff to limit interior residual flooding. Typically, the interior areas are studied to determine the specific nature of flooding and to formulate drainage alternatives to maximize National Economic Development (NED) benefits.

In accordance with EM 1110-2-1413, *Hydrologic Analysis of Interior Areas*, the interior drainage facilities are evaluated separately from the alignment. First, a minimum facility plan is identified. The minimum facility plan is considered the smallest plan that can be implemented as part of the alignment that does not result in increased stormwater flooding as a result of project construction. Starting from the minimum facilities analysis, alternatives to improve residual flooding conditions are evaluated to select an optimum plan. The interior drainage analysis for the GRR consisted of recreating the 1995 interior drainage model using the latest version of HEC-HMS in order to establish residual flooding impacts.

9.5 Rainfall and Storm Surge Correlation Analysis

For the with- and without-project conditions, the exterior stage (stillwater elevation within Newark Bay and the river mouth) is an important factor in the drainage of the interior precipitation runoff. The exterior stage is controlled by the tide cycle and storm surge elevations during storm events. Inland, the interior surface runoff is conveyed out into the rivers and bay via stormwater outfalls. In the without-project condition, these outfalls cease to operate when the exterior stage (tide/storm surge level) rises above the outfall opening because they rely on gravity to facilitate the transport of interior surface runoff. Similarly, if a new coastal storm risk management structure is introduced (with-project condition) to reduce the risk of storm surge entering the study area, the existing outfalls, under high exterior (tailwater) stage conditions would not operate. Therefore, it is important to develop an understanding of whether there is a relationship between interior surface runoff and exterior tidal events in both the with- and without-project conditions.

To understand the relationship between the interior and exterior stage conditions, if any, a correlation analysis needs to be performed. In accordance with EM 1110-2-1413, the correlation analysis should include a data analysis of the correlation, dependence, and coincidence of the interior and exterior stage relationship. In the vicinity of the Passaic Tidal study area, recent Corps correlation analyses have been conducted as part of the South River Hurricane and Storm Risk Management Project in 2002 and again for the South Shore of Staten Island Coastal Storm Risk Management Study in 2016 in order to quantify any correlation between the amount of precipitation and peak surge level during storms (locations shown in **Figure 27**).

From these three study areas, we can expect that the storm surge in the Newark Bay does not correlate to the precipitation events, is lightly dependent upon precipitation events, and that its peak stage is unpredictable but could coincide with peak interior discharges. Both previous Feasibility Studies are authorized projects and have a correlation analysis that was accepted through the USACE, Headquarters review process. A summary of the previous analyses and their applicability to the Passaic Tidal GRR is provided in this section and its subsections.

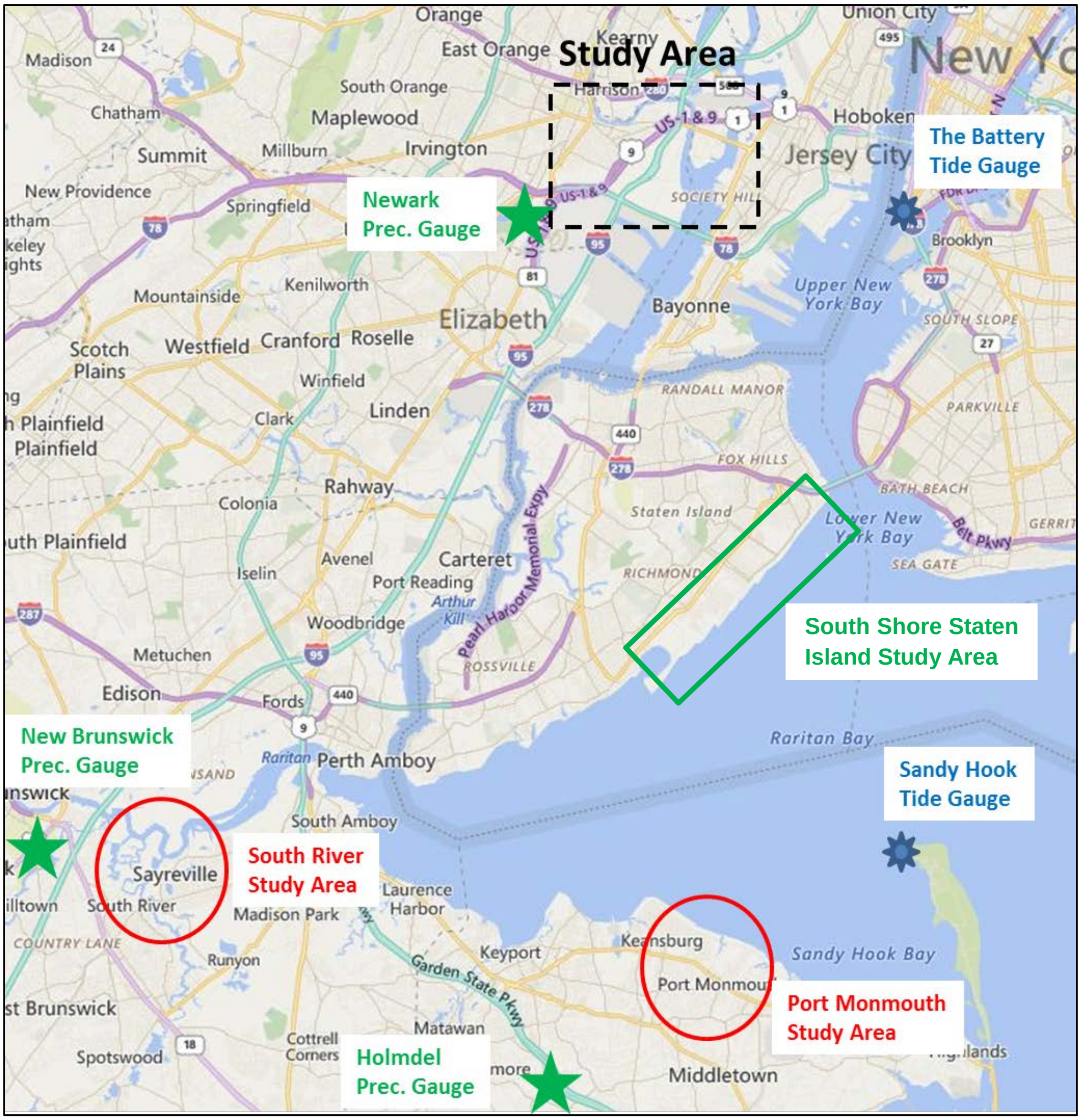

Figure 27: Gauge and Correlation Study Areas

The Passaic Tidal GRR, the South River, Port Monmouth, and South Shore Staten Island Feasibility Studies are within the New York/New Jersey Harbor area and have reasonably similar tidal conditions. Storm surge conditions during extreme events may vary slightly between the three study areas. A less than 0.5 feet peak stage difference was recorded between The Battery, NYC (see **Figure 27**) and Sandy Hook, NJ during Hurricane Sandy (National Hurricane Center – NOAA).

All four study areas are within approximately 20 miles from each other and have similar geomorphological conditions. They have experienced relatively similar rainfall conditions during past severe storm events. **Figure 27** shows the locations of three local rainfall gauges used to measure the variance in rainfall among the study areas. **Table 19** presents the total rainfalls during the last two severe weather events at these gauges. The observed variance in rainfall totals between study areas would not be significant enough to impact the correlation analysis results between sites.

Table 19: Rainfall Totals Near the Study Area During Irene and Sandy

Precipitation Gauge Location	Rainfall Total (inches)	
	Hurricane Irene	Hurricane Sandy
Holmdel	7.75	1.84
New Brunswick	8.08	1.77
Newark International Airport	8.92	1.06

In accordance with EM 1110-2-1413, the correlation analyses performed for the South River, Port Monmouth, and Staten Island studies considered the correlation, dependence, and coincidence of the exterior flood levels and interior flood levels.

9.5.1 Correlation

For the South River correlation analysis, hourly water surface elevations (WSEL) were obtained from the gauge at Sandy Hook for the time period from January 1933 to February 2000. They were then reduced to obtain daily high tide records for that time period (since these were hourly readings and not peak values, the actual peak values may have been slightly higher). Daily rainfall data for the same time period were also obtained from the New Brunswick precipitation gauge (location shown on Figure 26). After cleaning the datasets for unpaired data points and other suspect data, the aforementioned 67 years of systematic data (as adapted from the South River Study) along with the peak information from local storm events of record from the last 14 years (Hurricane Irene and Sandy) were combined and plotted on **Figure 28**.

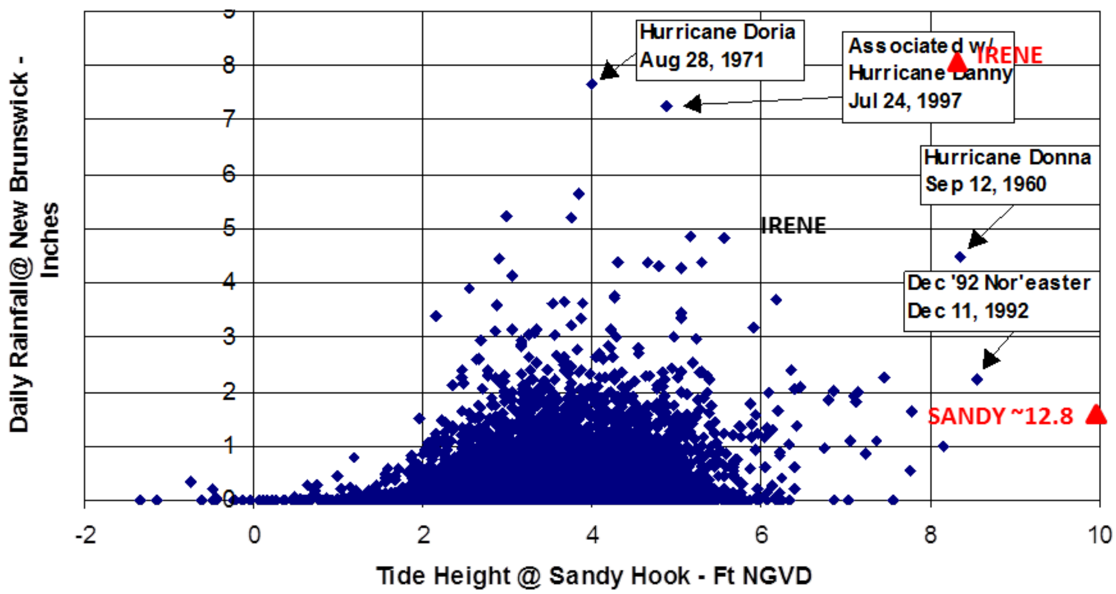

Figure 28: Tide-Rainfall Correlation Plot

As demonstrated in **Figure 28**, most of the higher tide events occurred with little rainfall, and most high rainfall events occurred with normal tides (normal tide range is shown on x-axis). This, along with the general wide scatter of precipitation amounts with a constant storm surge and vice versa indicates that there is no correlation between the surge events and precipitation. Therefore, it is not reasonable to say that we could predict one condition from the other based on these historic records.

9.5.2 Dependence

It is understood that the storms that typically produce tidal surges (i.e., hurricanes and nor'easters) can also produce somewhat significant rainfall. Likewise, many of the high rainfall events are accompanied by some degree of storm surge. If this were not true, the high surge events would not likely have any rainfall, and the paired data in Figure 27 would fall much closer to each axis. As expected, the figure reveals a minor dependence between the interior and exterior conditions. The fact that the main cluster of points that include some rainfall (one to two inches) also include a tide height greater than the mean tide level (0.9 feet NGVD29) is evidence of this.

9.5.3 Coincidence

The coincidence between the interior and exterior conditions involves the timing of the peak discharge from the interior drainage analysis and the timing of the peak exterior stage from the exterior storm surge analysis. In the exterior condition, the timing of the peak exterior stage is unpredictable because of the impacts of tidal fluctuation to the overall storm surge elevation. Therefore, predicting the coincidence of the peak exterior event and the peak interior flows is uncertain. Assuming that the interior and exterior events occur at the same time would be considered the worst case scenario and a conservative approach for modeling coincidence. Given that this coincidence was observed during Hurricane Donna in 1960, it has been incorporated into the model assumptions.

9.6 Analysis Approach

Due to the limited correlation between major rainfall/runoff events and tidal flooding events, it is considered most likely that only limited runoff will coincide with severe storm surge and significant storm surge will coincide with only moderately severe rainfall. Historical data indicate that the majority of interior runoff events will coincide with a storm surge level less than or equal to a 2-year storm. Similarly, the majority of significant storm surge events are likely to coincide with runoff equivalent to a 2-year event or less.

Therefore, the analysis was conducted for events with eight recurrence intervals: the 2-, 5-, 10-, 25-, 50-, 100-, 250- and 500- year frequency events (ACE probabilities of 50, 20, 10, 4, 2, 1, 0.4, and 0.2 percent, respectively). In order to develop a stage versus frequency relationship, the interior events were routed against exterior tidal marigrams. For the 'most likely' flooding scenarios, the eight interior storm events were routed against a 2-year exterior tide, and a 2-year interior storm event was routed against the nine exterior events. The highest SWEL of corresponding coincidental frequencies (i.e., 2-year interior and 10-year exterior, or 10-year interior and 2-year exterior) was identified as the most damaging flood level for the coincidental frequency, as shown in **Table 20**.

The upper and lower bound limits were used to represent the uncertainty in the analysis of residual damages and potential benefits from interior drainage features.

Table 20: Interior Drainage Analysis Approach

Combination of Interior and Exterior Conditions to be Analyzed											
Interior Flow	Exterior Stage	Time Condition	Peak Int. WSEL	Peak Ext. WSEL	Interior Flow	Exterior Stage	Time	Peak Int. WSEL	Peak Ext. WSEL	Max WS	Risk Condition
2-year	Normal	Current			N/a						Lower Bound
5-year	Normal	Current			N/a						Lower Bound
10-year	Normal	Current			N/a						Lower Bound
25-year	Normal	Current			N/a						Lower Bound
50-year	Normal	Current			N/a						Lower Bound
100-year	Normal	Current			N/a						Lower Bound
250-year	Normal	Current			N/a						Lower Bound
500-year	Normal	Current			N/a						Lower Bound
2-year	2-year	Current			2-year	2-year	Current				Most Likely (2-year)
5-year	2-year	Current			2-year	5-year	Current				Most Likely (5-year)
10-year	2-year	Current			2-year	10-year	Current				Most Likely(10-year)
25-year	2-year	Current	Model Output		2-year	25-year	Current	Model Output		Greatest WSEL for the Frequency Comb.	Most Likely(25-year)
50-year	2-year	Current			2-year	50-year	Current				Most Likely(50-year)
100-year	2-year	Current			2-year	100-year	Current				Most Likely(100-year)
250-year	2-year	Current			2-year	250-year	Current				Most Likely(250-year)
500-year	2-year	Current			2-year	500-year	Current				Most Likely(500-year)
2-year	10-year	Current			10-year	2-year	Current				Upper Bound
5-year	10-year	Current			10-year	5-year	Current				Upper Bound
10-year	10-year	Current			10-year	10-year	Current				Upper Bound
25-year	10-year	Current			10-year	25-year	Current				Upper Bound
50-year	10-year	Current			10-year	50-year	Current				Upper Bound
100-year	10-year	Current			10-year	100-year	Current				Upper Bound
250-year	10-year	Current			10-year	250-year	Current				Upper Bound
500-year	10-year	Current			10-year	500-year	Current				Upper Bound

9.7 Runoff and Surge Coincidence

There is little statistical information to determine where peak storm-related stormwater runoff should occur in relation to an approaching surge. Anecdotal meteorological evidence suggests that the maximum rainfall could be in any of the rain bands of a tropical storm, from out in the leading edge down to the eye wall, or behind the storm. Nor'easters are generally surge events but rainfall could occur and the impact is a function of the duration of the nor'easter. Therefore, in order to present a conservative modelling condition (maximum interior WSELs), the peak stormwater runoff was aligned to be coincidental with the maximum surge for a given annual chance event. This would result in the longest duration of gravity outlets being blocked and typically result in the highest interior water surface elevations for a particular storm/flood event. A graphic of typical modeled coincidence is shown in **Figure 29**.

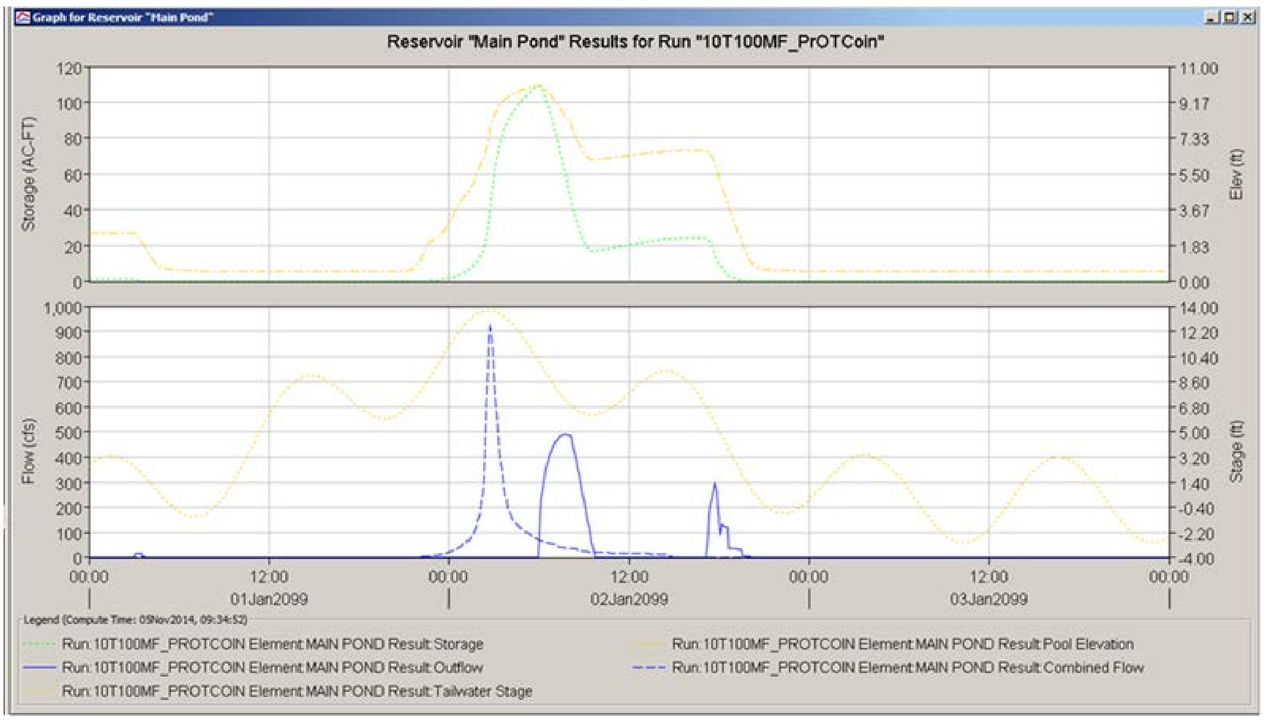

Figure 29: Typical Runoff/Surge Coincidence (HEC-HMS-Output)

9.8 Precipitation

Precipitation data were obtained from New Jersey 24-Hour Rain Fall Frequency Data for 2-, 5-, 10-, 25-, 50- and 100-year events and supplemented by NOAA Atlas 14, Volume 2, Version 3, for Newark, New Jersey, US Point Precipitation Frequency for various durations (5, 15, and 60 minutes; and 2, 3, 6, 12, 24 and 48 hours) and the estimated 500-year event. The 250-year event was interpolated from average recurrence interval/precipitation depth chart in NOAA Atlas 14. The rainfall data is shown in **Table 21**.

Table 21: **Rainfall Data**

Duration	Average Recurrence Interval (Years); Depth in Inches								
	1	2	5	10	25	50	100	250*	500
5-min	0.33	0.40	0.47	0.52	0.59	0.64	0.69	0.74	0.79
15-min	0.66	0.79	0.95	1.05	1.18	1.28	1.36	1.47	1.54
60-min	1.12	1.36	1.71	1.96	2.31	2.57	2.84	3.17	3.47
2-hour	1.37	1.67	2.12	2.46	2.94	3.33	3.74	4.26	4.74
3-hour	1.53	1.86	2.36	2.75	3.29	3.73	4.18	4.77	5.32
6-hour	1.96	2.39	3.02	3.53	4.24	4.84	5.47	6.31	7.11
12-hour	2.42	2.93	3.72	4.38	5.33	6.14	7.01	8.20	9.37
24-hour	2.71	3.29	4.20	4.99	6.16	7.18	8.30	9.85	11.40
48-hour	3.17	3.84	4.90	5.79	7.10	8.22	9.45	11.13	12.80

*Values interpolated based on 100-year and 500-year events.
Note: The data was unsmoothed in regard to depth versus duration for each frequency, and depth versus frequency, for each duration.

9.9 Boundary Conditions

9.9.1 Coastal Stillwater Elevations

Coastal stillwater elevation information from the NACCS is described in Section 5. The stillwater stage versus frequency data is shown in **Tables 3 and 4**.

9.9.2 Inland Stillwater Elevations

As described in Section 6, the FEMA inland stillwater elevations were more appropriate for inland analyses south of the project area.

10 Interior Drainage Results – Recommended Plan

The interior of the Recommended Plan was divided into five major drainage areas based on the contributions to the expected ponding or storage areas. As shown in **Figure 30**, Drainage Area 1 (DA1) contributes to ponding behind Segment 1. DA2, the largest drainage area, is the major contributor to ponding in the South Ironbound Area. DA3 contributes to ponding on the east side of the project area. DA5 is a small area of higher ground which ponds in the parking area but may overflow to DA2 and DA3. DA4 drains northward to the Passaic River. Should flood elevations reach 12 feet NAVD88 in DA4, flood waters will overflow into DA2.

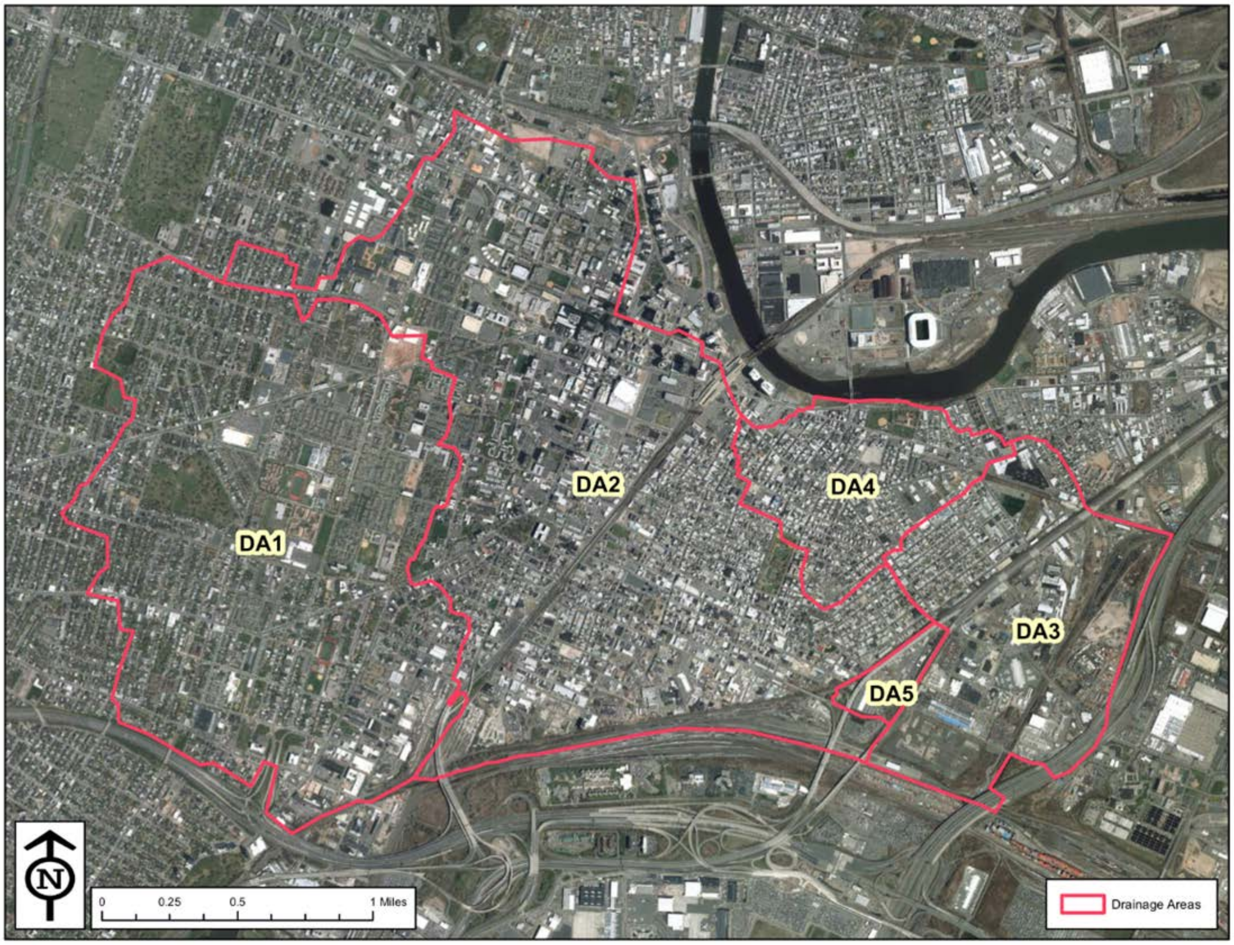

Figure 30: Recommended Plan Interior Drainage Areas

The details of the interior drainage analysis for the Recommended Plan are discussed in Subappendix 1. The following is a summary of the analysis results.

10.1 Recommended Plan – Minimum Facilities

The five drainage areas and associated ponding/stormwater storage areas were evaluated using HEC-HMS. Due to the unique location of the Recommended Plan flood risk management area – set back from the river and bay shore, and with several segments subject to limited tailwater effects during high frequency surge events – the combined sewer and existing stormwater drainage features limit any project-induced flooding in four of the five areas. In DA2, additional outfalls are required to limit induced flooding above the 100-year flood event. A summary of the minimum facilities for the Recommended Plan is shown in **Table 22**.

Table 22: Recommended Plan Minimum Facilities

Basin	Minimum Facilities Description
Drainage Area 1	Existing Conditions
Drainage Area 2	50-foot gate adjacent to railroad
Drainage Area 3	3x36” Culverts in Segment 3 levee; 3x36” culverts under access road for drainage conduit
Drainage Area 4	Existing Conditions
Drainage Area 5	Existing Conditions

10.2 Recommended Plan - Interior Drainage Alternatives

Interior drainage alternatives for the Recommended Plan included additional gravity outlets, additional storage, and pump stations. The alternatives considered for each drainage area are described below. Detailed results of the interior drainage analysis for the Recommended Plan are provided in Subappendix 1.

10.2.1 Drainage Area 1

Alternative 1: Additional gravity outlets near the existing drainage structures.

Alternative 2: 120 cubic feet per second (cfs) pump station.

Alternative 3: 60 cfs pump station.

Alternative 4: 1 x 66” additional steel drainage pipe.

10.2.2 Drainage Area 2

Alternative 1: Additional gravity outlets draining south from the drainage area.

Alternative 2: 1,200 cfs pump station.

Alternative 3: 500 cfs pump station.

10.2.3 Drainage Area 3

Alternative 1: Additional gravity outlets near the existing culvert.

Alternative 2: 240 cfs pump station.

Alternative 3: Excavated 3.8 acre pond storage with Alternative 1 outfalls.

Alternative 4: Excavated 3.8 acre pond with no additional outfalls.

10.2.4 Drainage Area 4

Alternative 1: Additional gravity outlets on Raymond Boulevard.

Alternative 2: 60 cfs pump station.

Alternative 3: 30 cfs pump station.

10.2.5 Drainage Area 5

Drainage Area 5 is small and primarily a group of parking lots; therefore; alternatives for improving interior drainage in DA5 were not considered.

10.3 Recommended Plan – Selected Interior Drainage Plan

The alternative interior drainage plans were formulated to provide safe and reliable protection from interior flooding. Due consideration was given to evaluating only feasible alternatives, i.e., alternatives that are implementable and provide equitable protection to properties within the alignment. Selection of an interior drainage plan thus focused on economics; i.e., providing the optimum reduction in damages for the cost of protection.

As outlined within the description of minimum facility, the planning and development of interior drainage facilities is performed independently from the alignment. Each interior drainage area is analyzed individually to determine the optimum alternative. Within each interior drainage area, the economics for a series of alternate facilities were evaluated and compared to determine which contributes the highest level of net excess benefits to the project. The optimum and selected interior drainage alternative for each sub-basin is presented in **Table 23**.

Table 23: **Selected Interior Drainage Plan Summary**

Basin	Plan	Description	Elevation that Flooding Starts (ft NAVD88)	Elevation of First Significant Damage (ft NAVD88)	10-year (10% ACE) Flood Elevation (ft NAVD88)
Drainage Area 1	Alternative DA1-4	Tie low areas into existing 66” x 69” stormwater line	8.0	8.5	No Flooding
Drainage Area 2	Minimum Facilities	50-foot gate adjacent to railroad	4.0	4.0	6.3
Drainage Area 3	Minimum Facilities	3x36” Culverts in Segment 3 levee; 3x36” culverts under access road for drainage conduit	5.0	6.0	5.1
Drainage Area 4	Minimum Facilities	No Additional Features	6.0	6.0	No Flooding
Drainage Area 5	Minimum facilities	No Additional Features	9.0	10.0	No Flooding

11 Interior Drainage Results – NED Plan

The GDM NED plan included 160 outfalls and six pump stations. The plan was not reformulated; therefore, interior drainage alternatives were not considered. The following is a summary of the interior drainage analysis of the NED Plan. A detailed discussion of the NED Plan interior drainage analysis is included in Subappendix 2.

11.1 Town of Harrison

There are three separate interior drainage areas that contribute to ponding behind the Harrison/South First Street Segment alignment, as shown in **Figure 31**:

1) S1: This 0.193-square mile north area drains by one 48-inch primary outlet, five 24-inch secondary outlets, and a 75-cfs pump station.
2) S2: This drainage area of 0.132-square mile drains to the west and is served by one 36 inch primary outlet, four 24-inch secondary outlets, and a 70-cfs pump station.
3) S3: This 0 .061-square mile drainage area discharges through one primary 36-inch outlet, three secondary 24-inch pipes, and a 30-cfs pump station.

Figure 31: Harrison/South First Street Drainage Areas

The Harrison interior drainage facilities data were verified and adopted from GDM, Appendix C-Hydrology and Hydraulics, and are shown in **Table 24**.

Table 24: Harrison Interior Drainage Features

Levee/Wall	Status	Type	Length* (feet)	Number	Gravity Size (inches)	Pump
S1	New	Primary	10	1	48	75 cfs
	New	Secondary	10	3	24	
	New	Secondary	10	2	24	
S2	New	Primary	10	1	36	70 cfs
	New	Secondary	10	1	24	
	New	Secondary	10	3	24	
S3	New	Primary	10	1	36	30 cfs
	New	Secondary	10	3	24	

*Through floodwall.

11.2 City of Newark

There are two distinguished interior drainage areas that contribute to ponding behind the City of Newark Segment alignment, as shown in **Figure 32**. These contribute to ten ponding areas:

1) Northern Area at Lister Avenue and the New Jersey Turnpike includes: L1, L2, L3 and T drainage and ponding areas, and
2) Eastern Area at Doremus Avenue and Doremus Avenue Extension includes: D1, D2, D3A, D3B, D4 and D5 drainage and ponding areas.

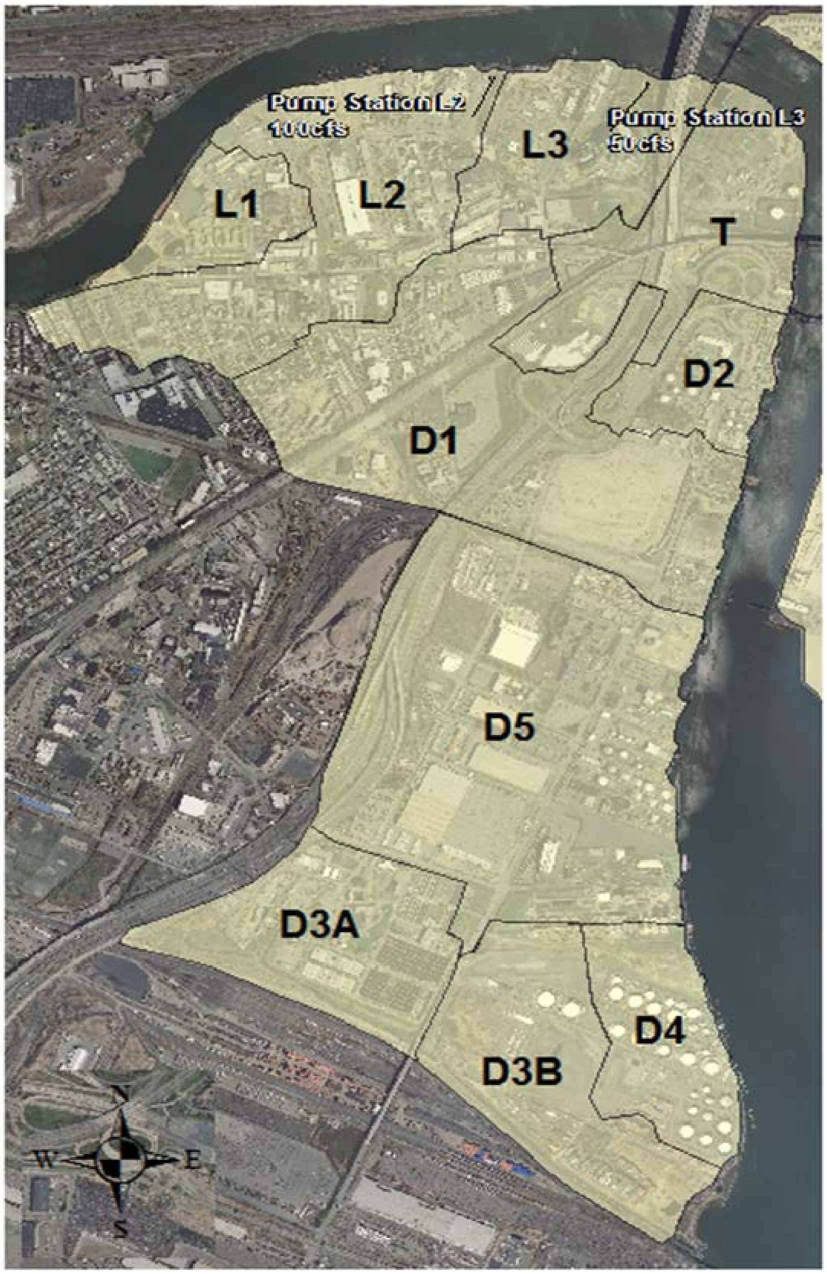

Figure 32: Newark Drainage Areas

The interior drainage features for Newark were determined in GDM for each of the ponding areas and are shown in **Table 25**. These features are independent of any culverts that may be required for wetlands flushing.

Table 25: Newark Interior Drainage Features

Levee/Wall	Status	Type	Length (feet)	Number	Size (inches)	Pump
Lister Avenue						
L1	New	Primary	10	1	36	
	New	Secondary	10	4	24	
	New	Secondary	10	1	24	
L2	Existing	Primary	10	2	72	100cfs
	New	Secondary	10	2	24	
	New	Secondary	10	5	24	
L3	New	Primary	10	1	48	50 cfs
	New	Secondary	10	5	24	
	New	Secondary	10	1	24	
	Existing	Secondary	10	1	24	
Turnpike						
T	New	Primary	10	1	48	
	New	Secondary	10	9	24	
Doremus Ave.						
D1	Existing	Primary	10	1	60	
	New	Secondary	10	5	24	
D2	New	Primary	10	1	48	
	New	Secondary	10	1	24	
	New	Secondary	10	3	24	
Doremus Ext.						
D3A	Existing	Primary	10	1	3x2 feet	
D3B	New	Primary	10	2	60	
	New	Secondary	10	1	36	
	New	Secondary	10	1	24	
D4	New	Primary	10	2	36	
	New	Secondary	10	7	24	
D5	New	Primary	10	1	36	
	New	Secondary	10	8	24	

11.3 Town of Kearny

Four interior drainage areas contribute to ponding behind the Kearny alignment, as shown in **Figure 33**.

1) K1: This 0.222-square mile area drains by one 36 inch primary outlet, eight 24-inch secondary outlets and a 75-cfs pump station.
2) K2: This drainage area of 0.036-square miles is served by one 48-inch primary outlet and three 24-inch secondary outlets.
3) K3: This 0.632-square mile drainage area discharges through one primary 66-inch outlet and six secondary 24-inch outlets into the Hackensack River.
4) K4: This 0.648 square mile drainage area discharges through three primary 36 inch outlet and 34 secondary 24 inch outlets into the Passaic and Hackensack rivers.

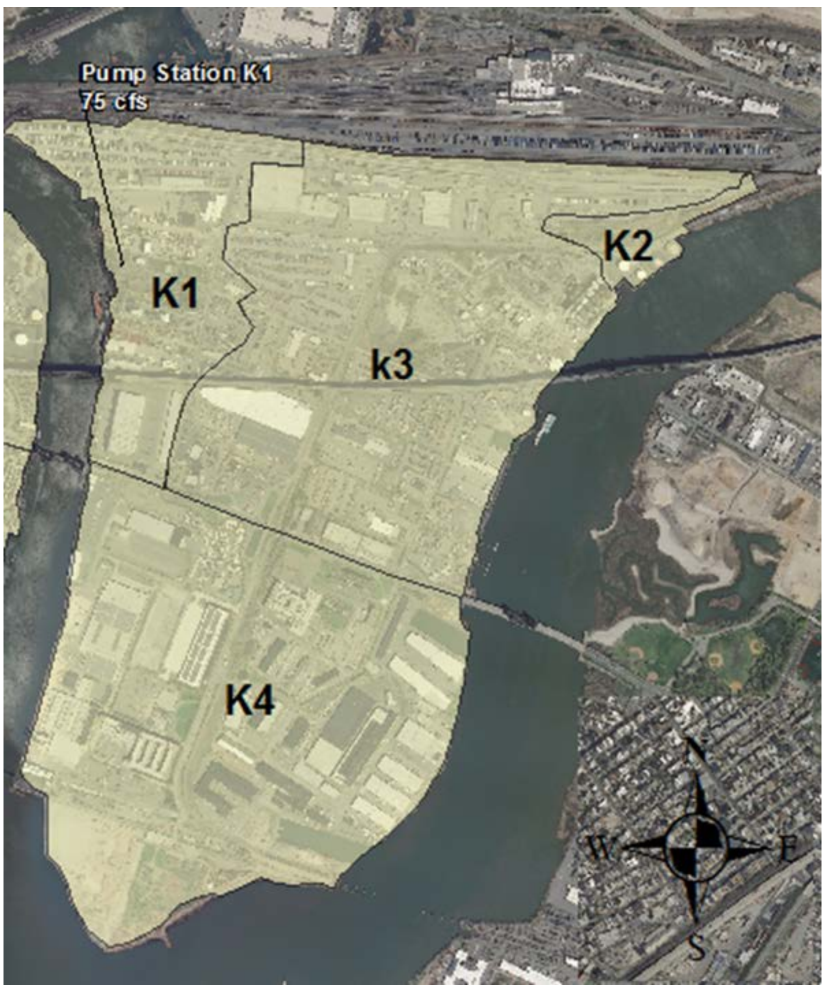

Figure 33: Kearny Drainage Areas

The Kearny interior drainage features are shown in **Table 26**.

Table 26: **Kearny Interior Drainage Features**

Levee/Wall	Status	Type	Length (feet)	Number	Size (inches)	Pump
K1	New	Primary	10	1	36	75 cfs
	New	Secondary	10	6	24	
	New	Secondary	10	2	24	
K2	New	Primary	10	1	48	
	New	Secondary	10	3	24	
K3	Existing	Primary	10	1	66	
	New	Secondary	10	2	24	
	New	Secondary	10	4	24	
K4	New	Primary	10	3	36	
	New	Secondary	10	17	24	
	New	Secondary	10	17	24	

12 Residual Damage

The interior drainage analysis for the Recommended Plan optimized interior drainage facilities to cost effectively reduce residual flooding. However, not all residual flooding can be eliminated. The residual flooding stage versus frequency curve for each drainage area is shown in **Table 27**. **Figures 34-38** show the approximate 50-year and 100-year residual floodplains in each drainage area for the selected interior drainage alternative. Elevations of first significant damage are shown in red.

Table 27: **Recommended Plan Residual Flooding Stage versus Frequency**

Frequency	Drainage Area Elevations (feet NAVD88)				
	DA1	DA2	DA3	DA4	DA5
2-year	2.76	4.65	3.06	4.71	6.25
5-year	2.81	5.5	4.01	5.11	7.02
10-year	3.04	6.25	5.1	5.81	7.92
25-year	5.00	7.04	5.98	6.15	8.72
50-year	5.89	7.43	6.66	6.28	9.17
100-year	8.55	7.86	7.26	6.61	9.50
250-year	10.48	8.38	8.06	7.32	10.01
500-year	11.34	8.76	8.47	8.00	10.17

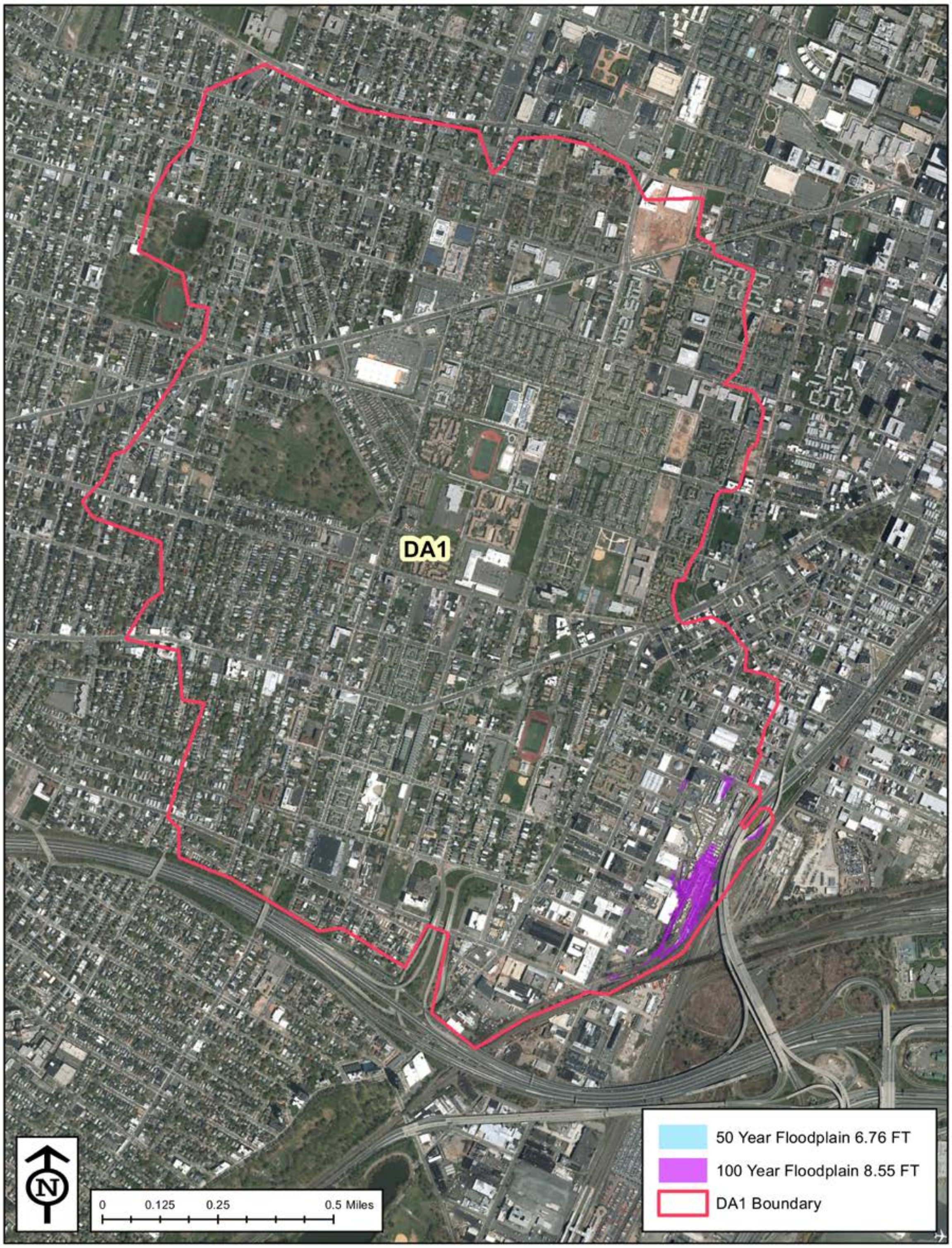

Figure 34: DA1 50-year and 100-year Residual Floodplains

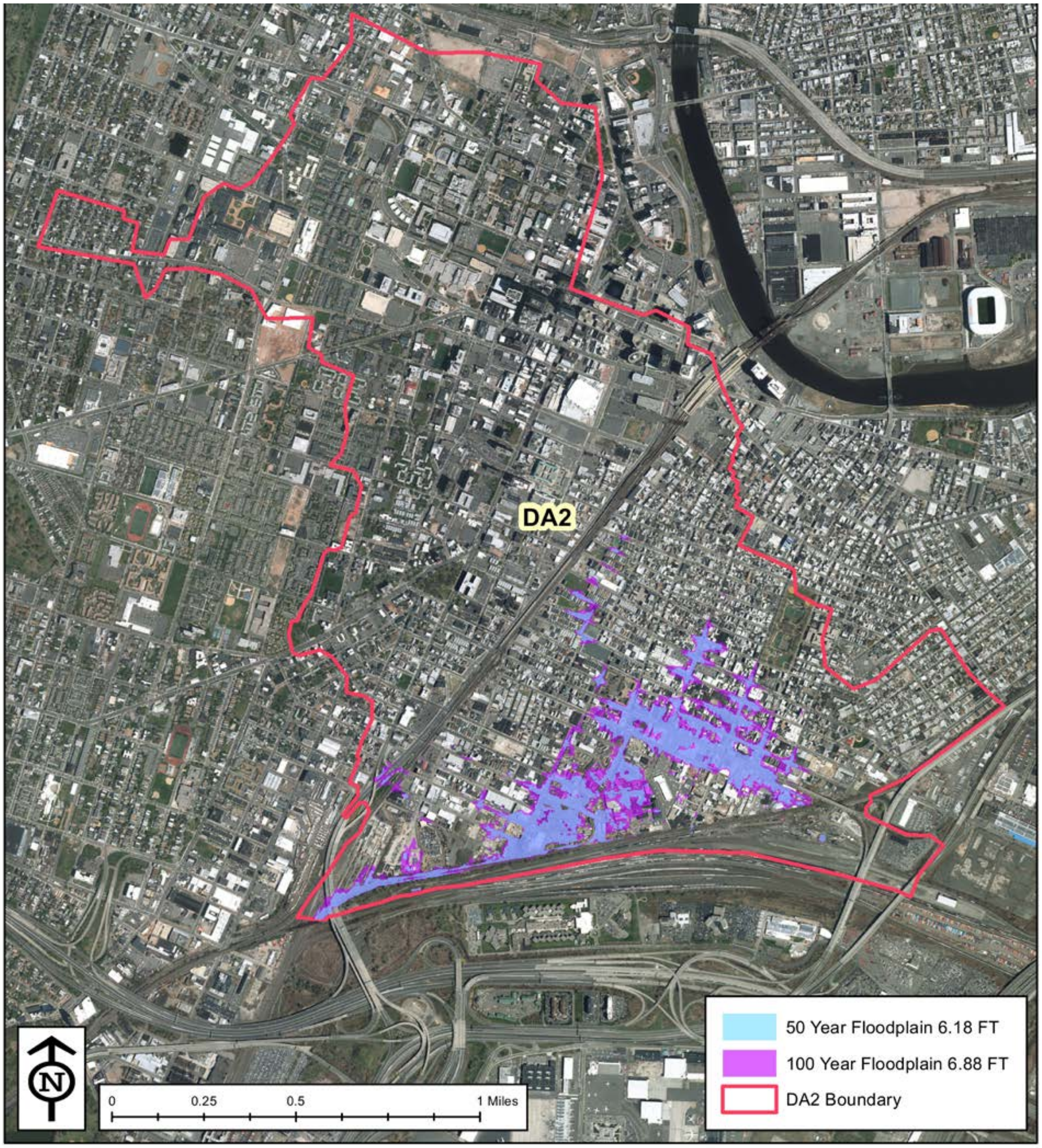

Figure 35: DA2 50-year and 100-year Residual Floodplains

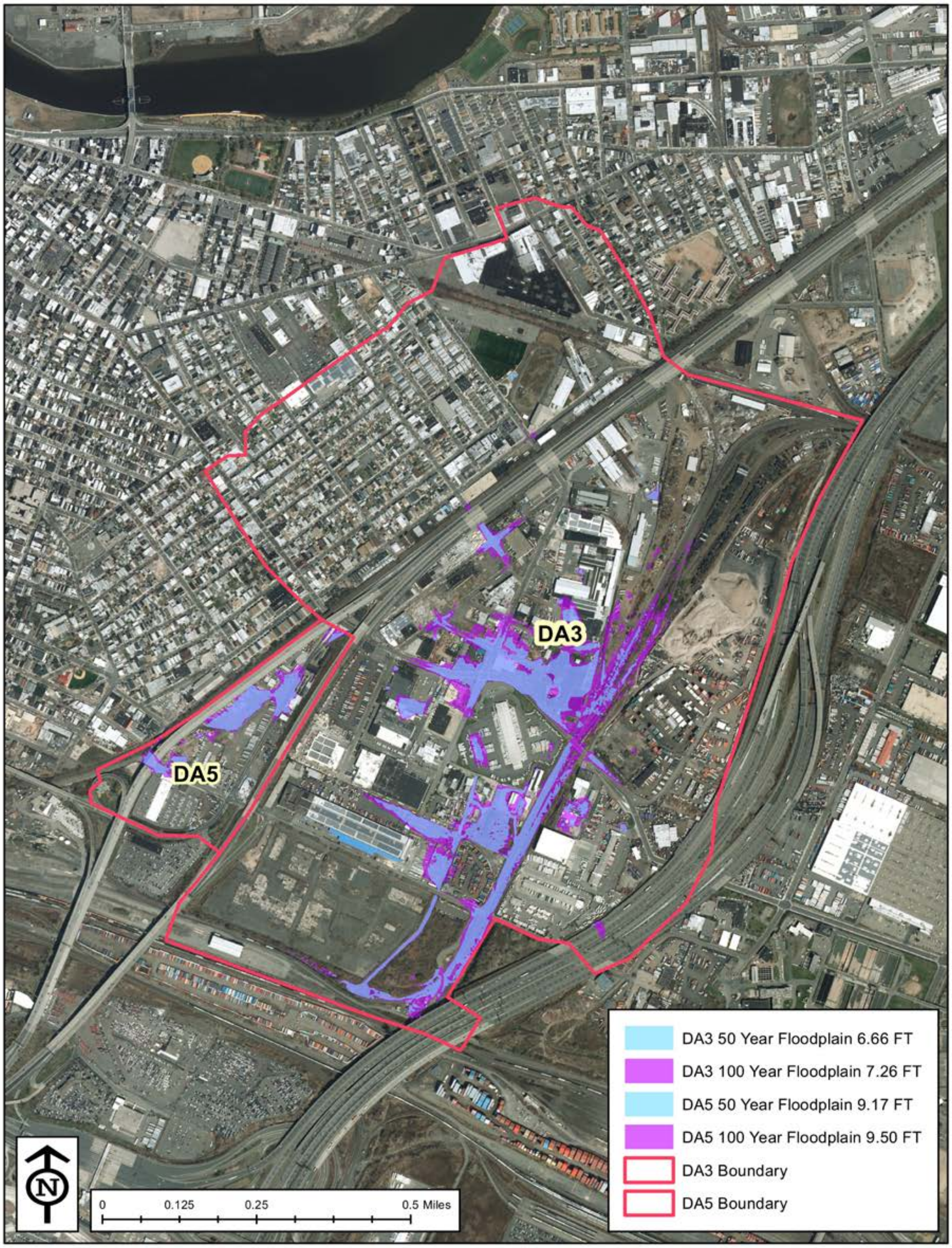

Figure 36: DA3 & DA5 50-year and 100-year Residual Floodplains

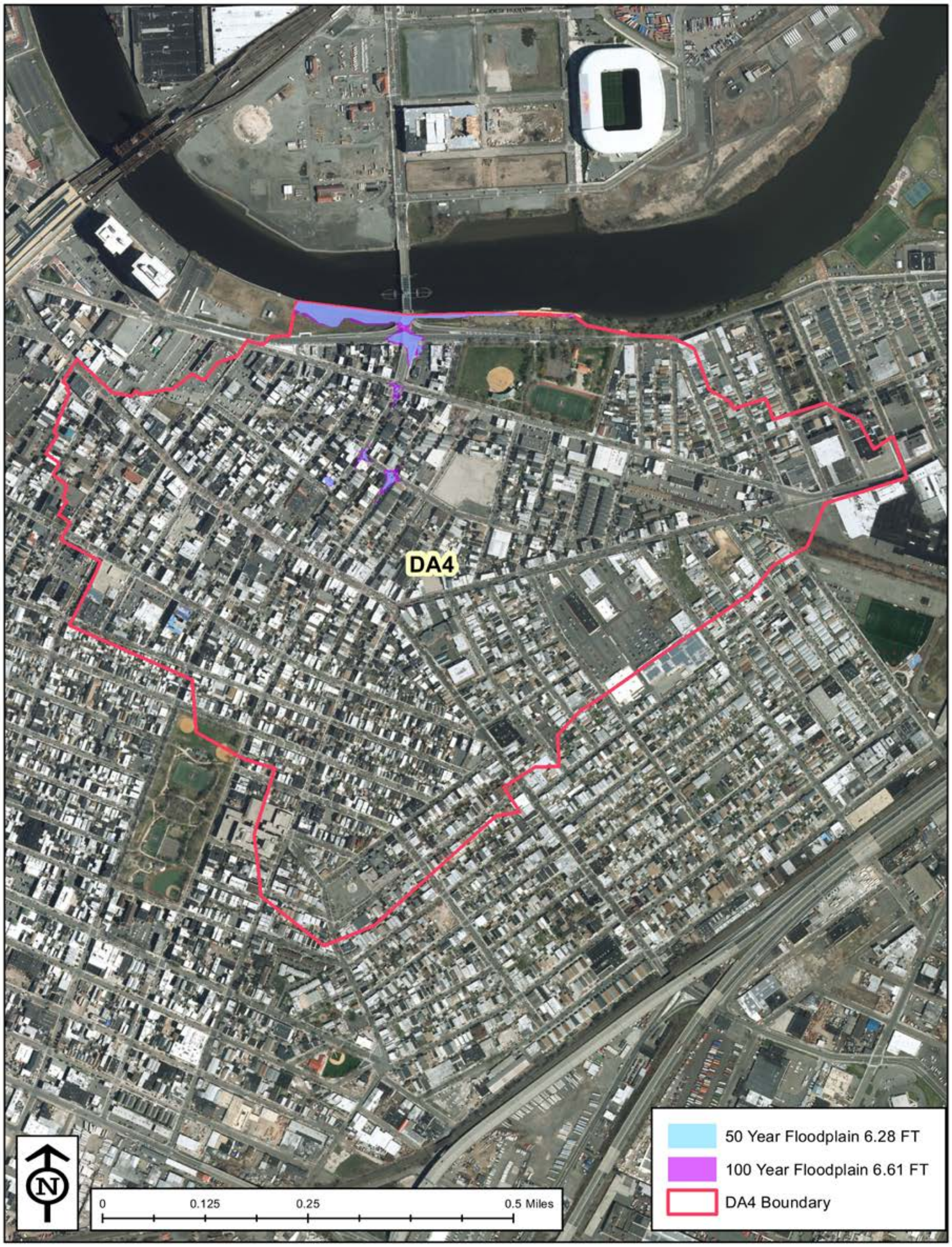

Figure 37: DA4 50-year and 100-year Residual Floodplains

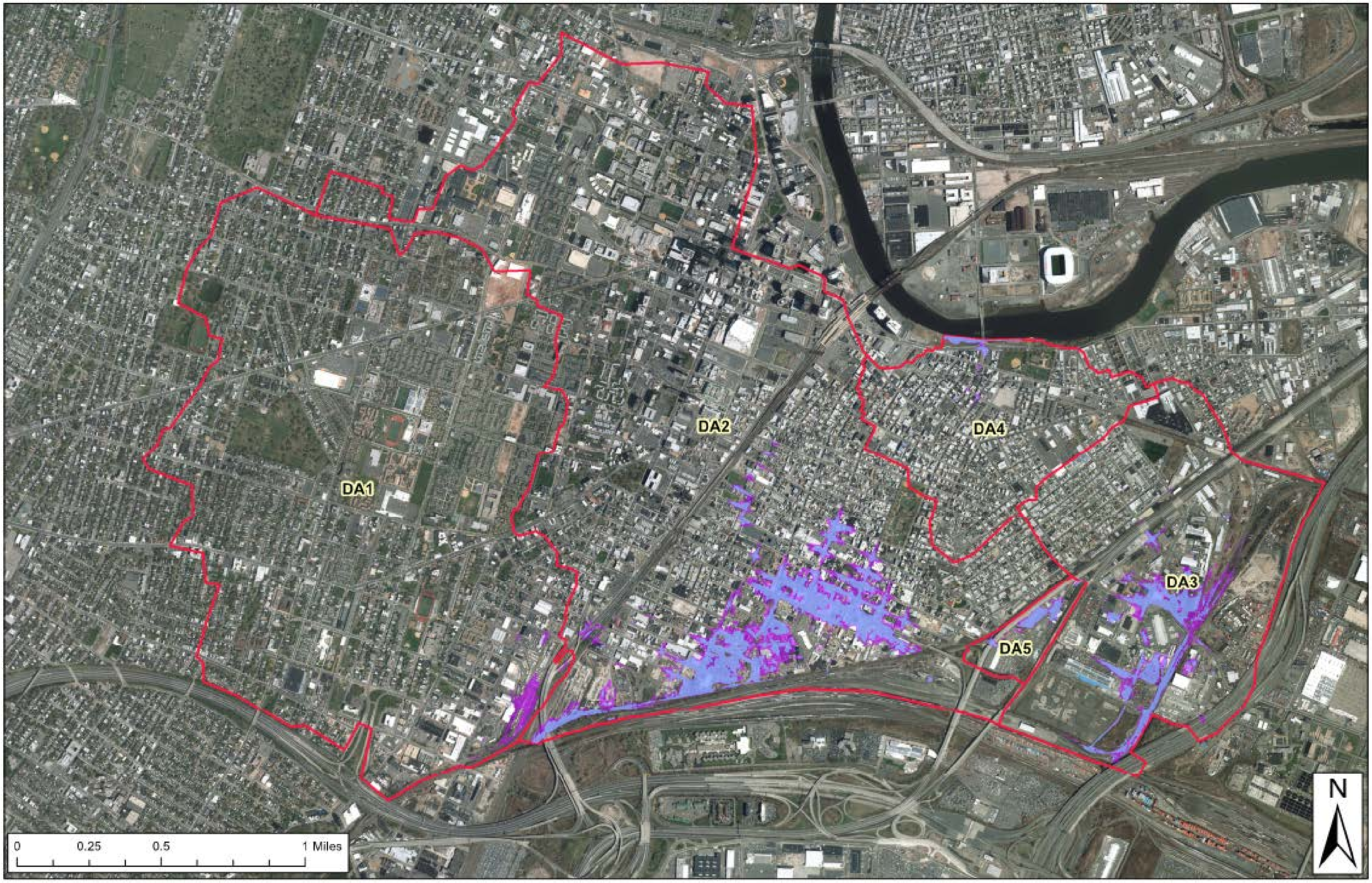

Figure 38: Recommended Plan 50-year and 100-year Residual Floodplains

13 BACKFLOW PREVENTION – EXISTING DRAINAGE STRUCTURES

13.1 Conduits

Stormwater drainage is managed within the City of Newark via the extensive combined sewer system (CSS) and some stormwater-only drainage features. During times of extensive rainfall, the CSS regulators allow by-pass of excess flow that exceeds the treatment plants capacity directly to the Passaic River and Newark Bay. If tide heights or storm surges block the CSS outfalls, combined drainage backs up into the city until processing can catch up. CSS outfalls typically have backflow prevent devices to limit backflow tidal surge into the city; however, these may not be located in line with the Recommended Plan alignment. Therefore, additional backflow devices may need to be installed. **Table 28** and **Figure 39** identify and show the locations of CSS conduits that are expected to require additional backflow prevention devices to limit tidal surcharging into the flood risk management area. Backflow prevention includes installation of a junction box, access, sluice gate, and backflow prevention device.

Likewise, few of the existing stormwater drainage or outfalls are believed to include measures to limit backflow into the drainage system. These conduits and outfalls will also need additional backflow prevention devices installed to further limit tidal and storm surges from entering the flood risk management area. The additional stormwater drainage backflow prevention device locations are also shown in **Table 28** and **Figure 39**.

Table 28: CSS and Stormwater Backflow Prevention Locations

Type	Name	Description	Location
Stormwater	Stormwater 5	15-inch Pipe	Hunter Street (Segment 2)
	Stormwater 6	66-inch Pipe	North of East Peddie Street
	Avenue C	36-inch RCP	End of Avenue C
	Pierson Creek 2	4' x 8' Box	Vicinity of Segment 3
CSS	Wheeler 1	5' x 8' Box	Vicinity of Avenue A (Segment 2)
	Adams 1	4' x 8' Box	End of Adams Street (Drainage Area 2)

13.2 Manholes

Due to the Recommended Plan alignment being set back from the waterfront, existing manholes that are part of the CSS, as well as manholes for other utility conduits will likely need to be sealed to prevent surcharging from tidal surge head above the manholes. This surcharge could backflow through smaller system pipes behind the alignment and cause backflow flooding. Therefore, it was assumed that 200 manholes will need to be sealed, pending a more detailed investigation during the design phase.

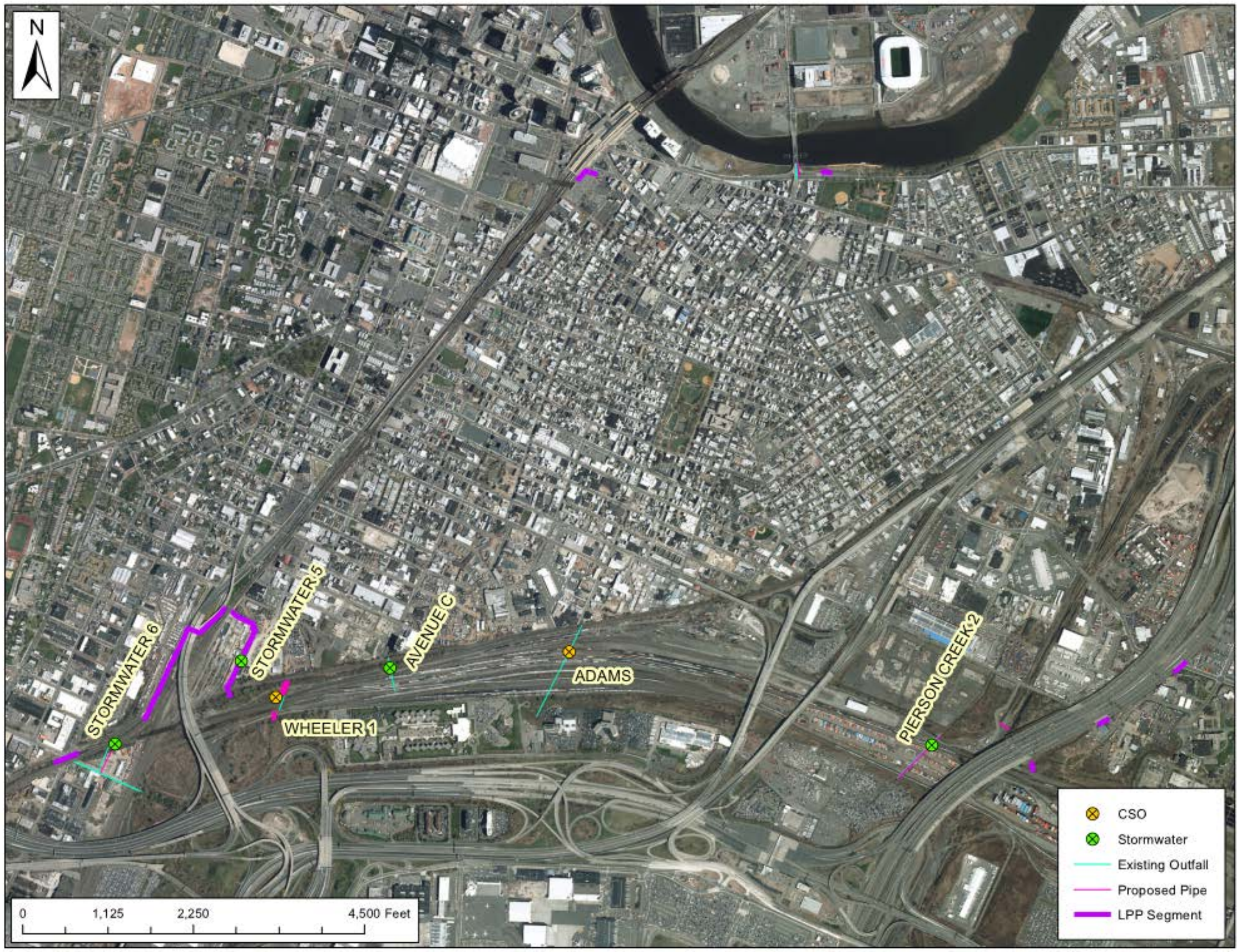

Figure 39: CSS/Stormwater Backflow Prevention

14 OPERATIONS AND MAINTENANCE

Development of a detailed Operations, Maintenance, Repair, Replacement and Rehabilitation Manual for the alignment and interior drainage features will be performed during the Construction Phase of the project.

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ATTACHMENT 1
EXECUTIVE SUMMARY: NACCS MODELING COMPONENT

Executive Summary: NACCS Modeling Component

The document summarizes the application of a suite of high-fidelity numerical models for the North Atlantic Coast Comprehensive Study (NACCS). The effort was conducted to provide information for computing the joint probability of coastal storm forcing parameters for the North Atlantic Coast of the United States because this information is critical for effective flood risk management project planning, design, and performance evaluation. The study was performed using the high-fidelity models within the Coastal Storm Modeling System (CSTORM-MS). The NACCS numerical modeling study produced nearshore wind, wave and water level estimates and the associated marginal and joint probabilities. Documentation of the statistical evaluation is provided in a separate Executive Summary.

The first major step in the numerical modeling effort was to select a suite of storms to simulate that are statistically significant to the region of interest. The NACCS coastal region is primarily affected by tropical, extratropical, and transitional storms. It is common to group the storms into statistical families of tropical and extratropical with transitional storms that were once tropical being mostly categorized as tropical. In this study, both tropical and extratropical storms were strategically selected to characterize the regional storm hazard. Extratropical storms were selected using the method of Nadal-Caraballo and Melby (2014) using an observation screening process. The tropical storm suite was developed using a modified version of the joint probability method (JPM) methodology (Ho and Myers 1975) with optimized sampling (JPM-OS) methods from Resio et al. (2007) and Toro et al. (2010). In this process, synthetic tropical storms are defined from a joint probability model of tropical cyclone parameters. The cyclone parameters describe the storm size, intensity, location, speed, and direction. This approach to statistical sampling is specifically designed to produce coastal hydrodynamic responses that efficiently span practical parameter and probability spaces to the study area.

With the storms selected, Oceanweather, Inc. (OWI) generated extratropical wind and pressure fields for the 100 historical extratropical events identified in the storm selection process for the NACCS effort for two working grids: the original Wave Information Study (WIS) Level II domain as well as a 0.125-deg domain covering 36-45N and 78-66W (NACCS domain covering Virginia to Maine). OWI performed a reanalysis of the storm core of winds generating the maximum ocean response and included the assessment/assimilation of coastal station data such as National Weather Service reporting stations and National Ocean Service stations not considered as part of the WIS effort. Background fields were sourced from the NCEP/NCAR reanalysis for the period from 1948 to 2012, preserving the enhancements applied in the WIS effort. Storms prior to 1948 were developed from the NCEP 20th Century Reanalysis project. Matching pressure fields on both grids were sourced from reanalysis products and interpolated onto the WIS/NACCS grids. Each extratropical storm event produced by OWI contains eight days of wind/pressure fields with the majority of the reanalysis effort concentrated on the coastal domain of the storm with high wind forcing.

In addition to the extratropical storm wind and pressure fields developed by OWI for the NACCS study, OWI provided developmental support and analysis associated with the generation of synthetic tropical storm wind and pressure fields. ERDC provided OWI with storm parameters associated with 1,050 tropical synthetic events and OWI was responsible (with input from ERDC) to expand these landfall parameters into a full storm track time history for each event. The development of a track path both pre- and post-landfall followed the same basic methodology as was applied in OWI's contribution to the FEMA Region IV Georgia/North Florida Surge study. Storm speed remained constant for the storm duration by applying the landfall speed specification supplied by ERDC. Post-landfall, the storm heading was preserved for a suitable amount of time (usually 24 hours) to allow sufficient spin-down time for the response (surge and wave) models. Prior to landfall, an analysis of mean track paths for three regional stratifications supplied by ERDC was evaluated to recommend a suitable turning rate (by stratification, if needed) of storm heading so that synthetic track paths were consistent with the historical record. Generation of synthetic tropical storm wind and pressure fields from three to five days prior to landfall/closest approach to one day post-landfall was accomplished with a tropical Planetary Boundary Layer (PBL) model. Wind (WIN) and pressure (PRE) output files of ten meter wind and sea level pressures were made on two target grids. The same WIS Level II and NACCS domains described in the extratropical wind and pressure field development were applied with the synthetic tropical storms.

With the storms selected and wind and pressure fields generated, the next major step was to apply CSTORM-MS to each event because this system provides a comprehensive methodology to simulate coastal storms and produce accurate surge and waves in the coastal zone. CSTORM-MS was applied with WAM for producing offshore deep water waves mainly intended for providing boundary conditions to the nearshore steady-state wave model STWAVE; ADCIRC to simulate the surge and circulation response to the storms; and STWAVE to provide the nearshore wave conditions including local wind generated waves. The CSTORM-MS coupling framework options used for the NACCS numerical modeling study tightly links the ADCIRC and STWAVE models in order to allow for dynamic interaction between surge and waves. Each model was validated separately prior to going into production mode.

An evaluation was conducted to assess the quality of the offshore wave model WAM estimates for several historical extratropical and tropical events. The testing also provided a means to evaluate the grid system, model resolutions, and forcing conditions. Validation was conducted by simulating five tropical and 17 extratropical storms based on high water level measurements and extreme wave dominated events and comparing to measured wave conditions for each event. The wave model results were evaluated at as many as 30 point-source measurements in the Atlantic Basin. The evaluation consisted of time, scatter, Quartile-Quartile graphics and a battery of statistical tests performed at each site for each grid level and for each of the 22 selected storm events. These results indicated that WAM provided high quality wave estimates compared to the measurement sites. From these tests, the need to initiate the Level1 WAM historical storm simulations at a minimum of ten days prior to the occurrence of the storm peak was also determined. This assured the nearshore wave climate contained sufficient far-field wave energy generated by synoptic-scale events in the entire Atlantic Ocean basin. The preproduction assessment also provided a means to develop and test the fully-automated system, generation of boundary condition information for STWAVE, and tools for quality checking the final model results used in the production portion of the work.

The ADCIRC mesh developed for the NACCS study encompasses the western North Atlantic, the Gulf of Mexico and the western extent of the Caribbean Sea with 3.1 million computational nodes and 6.2 million elements. Validation of this mesh was accomplished by comparisons of model simulated water levels to NOAA/NOS measured water-surface elevations. Model validation was conducted with the analysis of a long term tidal simulation as well as five tropical and two extratropical storms. From the harmonic analysis conducted for the long-term simulation it was determined that the model accurately predicts response to tidal forcing. Model accuracy was tested for the seven validation storms and showed that the model agrees with measured water surface elevations (WSELs) (time series and high water marks) at measurement locations throughout the study domain. Model accuracy is a function of the quality of the ADCIRC mesh, the accuracy of the bathymetry within the mesh, the representation of bottom friction characterized in the model, and the accuracy of the wind forcing. Small differences in modeled and measured WSELs for the validation storms are attributed to these factors.

Nearshore wave transformation for the NACCS was accomplished using the spectral wave model STWAVE applied to ten domains encompassing coastal Virginia to Maine. Prior to the production phase, STWAVE results were evaluated against measurements for the same five tropical and two extratropical storms used in the evaluation of ADCIRC. The evaluation consisted of time, scatter, Taylor diagrams, and a suite of statistics. Comparisons were most favorable for the most recent storms, likely due to development of more accurate wind and offshore forcing, more advanced buoy technology, and a larger measurement population size in recent time. STWAVE was also more accurate in estimating wave height than mean wave period. Although some sites did demonstrate persistent poor performance, STWAVE provided overall good wave estimates compared to measurement sites given the large extent and complexity of the model region.

Once the models were validated, NACCS production began on the suite of 1,150 storms for three conditions. With the 3,450 CSTORM-MS simulation requirement, a semi-automated process was needed to efficiently and accurately set up and execute this large simulation suite. Therefore, semi-automated production scripts for setting up CSTORM-MS simulations (CSTORM-PS) were created, tested, and verified for historical extratropical storms, historical tropical storms, and synthetic tropical storms; and scripts were executed for all production simulations. Because of the magnitude of this study, a visualization component (CSTORM-PVz) was created within the CSTORM-MS framework and automation scripts were generated to produce graphics, descriptive statistics, and digital reports for all NACCS results.

The products of this detailed, large-domain modeling study are intended to close gaps in data required for flood risk management analyses by providing statistical wave and water level information for the entire North Atlantic coast, while providing cost savings compared to developing coastal storm hazard data for individual local projects. The CSTORM-MS platform provides the raw model data (winds, waves, and water levels) as well as processed data (visualization products and statistics) and is available through the internet-based CHS. These data are available for engineering analyses and project design for coastal projects from Maine to Virginia.

ATTACHMENT 2
UNIT SYNTHETIC STORM HYDROGRAPH

Unit Synthetic Storm Surge Hydrograph

Multiply each of the surge values in the second column by the appropriate peak surge to obtain the hydrograph for the desired recurrence interval at the desired geographic location.

Time (hours)	Surge (feet)
-48	0.123002
-47	0.125447
-46	0.127992
-45	0.130642
-44	0.133404
-43	0.136284
-42	0.139292
-41	0.142435
-40	0.145723
-39	0.149166
-38	0.152775
-37	0.156563
-36	0.160543
-35	0.164730
-34	0.169140
-33	0.173792
-32	0.178707
-31	0.183906
-30	0.189416
-29	0.195264
-28	0.201484
-27	0.208110
-26	0.215185
-25	0.222755
-24	0.230874
-23	0.239602
-22	0.249011
-21	0.259182
-20	0.270211
-19	0.282211
-18	0.295312
-17	0.309672
-16	0.325477
-15	0.342953
-14	0.362372
-13	0.384066
-12	0.408445
-11	0.436015
-10	0.467408
-9	0.503415
-8	0.545019
-7	0.59343
-6	0.650062
-5	0.716346
-4	0.792992
-3	0.877544
-2	0.957148
-1	0.998164
0	1.000000
1	0.998164
2	0.957148
3	0.877544
4	0.792992
5	0.716346
6	0.650062
7	0.593430
8	0.545019
9	0.503415
10	0.467408
11	0.436015
12	0.408445
13	0.384066
14	0.362372
15	0.342953
16	0.325477
17	0.309672
18	0.295312
19	0.282211
20	0.270211
21	0.259182
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24	0.230874
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29	0.195264
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31	0.183906
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37	0.156563
38	0.152775
39	0.149166
40	0.145723
41	0.142435
42	0.139292
43	0.136284
44	0.133404
45	0.130642
46	0.127992
47	0.125447
48	0.123002

Subappendix 1
Recommended Plan Interior Drainage

DRAFT FOR DQC

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1 INTRODUCTION

The Hydrology and Hydraulics (H&H) 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 Recommended Plan, 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 risk management measures 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 multiple design elevations 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 alternative alignments 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 calculate estimated 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. The U.S. Army Corps of Engineers (USACE) selected the LPP as the Recommended Plan.

This subappendix documents the interior drainage analysis of the Recommended Plan.

A general project location map of the Recommended Plan is provided in **Figure 1**.

Figure 1: Passaic River Tidal Project Area – Recommended Plan Segments

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 one in 100 chance of occurring in any single year would be described as the one 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
4%	0.04	25-year
2%	0.02	50-year
1%	0.01	100-year
0.4%	0.004	250-year
0.2%	0.002	500-year

1.2 Survey and Datum

The latest topographic data used 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 (NGVD). The conversion factor from NGVD to North American Vertical Datum of 1988 (NAVD88) is approximately -1.1 feet; therefore, the 1995 GDM design elevation of 14.9 feet NGVD is converted to 13.8 feet NAVD88. For ease in analysis, computation and discussions, the 1995 GDM design elevation is rounded to 14 feet NAVD88.

2 ANALYSIS APPROACH

2.1 General

Areas protected from exterior flood elevations are subject to interior flooding from stormwater runoff. Thus, interior drainage facilities are required to safely store and discharge the runoff to limit interior residual flooding. The interior areas were studied to determine the specific nature of flooding and to formulate drainage alternatives to maximize National Economic Development (NED) benefits.

In accordance with USACE Engineer Manual (EM) 1110-2-1413, *Hydrologic Analysis of Interior Areas,* the interior drainage facilities are evaluated separately from the alignment. First, a minimum facility plan is identified. The minimum facility plan is considered the smallest plan that can be implemented as part of the alignment that does not result in increased stormwater flooding as a result of project construction (residual damages). It is the starting point from which additional interior facilities planning commences.

Next, the benefits accrued from alternative interior drainage plans are attributed to the reduction in the residual flood damages which may have remained under the minimum facility condition. Finally, an optimum drainage alternative is selected based on meeting NED objectives.

The interior drainage facilities must be formulated to maximize NED benefits while meeting NED objectives to provide a complete, effective, efficient, and acceptable plan of protection.

- **Completeness** is defined in Engineer Regulation (ER) 1105-2-100 as, *the extent to which the alternative plans provide and account for all necessary investments or other actions to ensure the realization of the planning objectives, including actions by other Federal and non-Federal entities*.

- **Effectiveness** is defined as, *the extent to which the alternative plans contribute to achieve the planning objectives.*

- **Efficiency** is defined as, *the extent to which an alternative plan is the most cost-effective means of achieving the objectives.*

- **Acceptability** is defined as, *the extent to which the alternative plans are acceptable in terms of applicable laws, regulations, and public policies.*

2.2 Minimum Facilities

The minimum facilities are the starting point from which additional interior drainage facilities will be compared. The minimum facilities should provide interior flood relief such that, during low exterior stages, the local storm drainage system functions essentially as it did without flood protection in place, up to that of the local storm sewer design. For this project, minimum facilities represent the minimum drainage required such that no induced flooding occurs during low exterior stages.

The determination of interior drainage facilities was conducted using guidance from EM 1110-2-1413. The strategy outlined under this guidance follows the premise that interior drainage facilities will be planned and evaluated separately from the project alignment, and should provide adequate drainage at least equal to that of the existing infrastructure. This initial plan represents the minimum interior facilities required to implement the Recommended Plan.

2.3 Interior Drainage Methodology

Due to the limited correlation between major rainfall/runoff events and tidal flooding events, it is considered most likely that only limited runoff will coincide with severe storm surge and significant storm surge will coincide with moderate rainfall. Historical data indicate that the majority of interior runoff events will coincide with a storm surge level less than or equal to a 2-year storm. Similarly, the majority of significant storm surge events are likely to coincide with runoff equivalent to a 2-year event or less.

This analysis was conducted for events with eight recurrence intervals: the 2-, 4-, 10-, 25-, 50-, 100-, 250- and 500- year frequency events (ACE probabilities of 50, 25, 10, 4, 2, 1, 0.4, and 0.2 percent, respectively). In order to develop a stage versus frequency relationship, the interior events were routed against exterior tidal marigrams. For the 'most likely' flooding scenarios, the eight interior storm events were routed against a 2-year exterior tide, and a 2-year interior storm event was routed against the eight exterior events. The highest water surface elevation of corresponding coincidental frequencies (i.e., 2-year interior and 10-year exterior, or 10-year interior and 2-year exterior) was identified as the most damaging flood level for the coincidental frequency, as shown in **Table 2**. Normal tide and 10-year tailwaters were used as the lower and upper bound limits, respectively, for the economic analysis.

For Segments 1 and 2, and the southern outfall of Segment 3, the normal tailwater was also used for the Most Likely and Upper Bound analysis because the Peripheral Ditch flood gates and pump station limited tidal surge inundation into the ditch up to the 10-year event.

2.4 Risk and Uncertainty

Risk and Uncertainty (R&U) associated with the hydrology and hydraulics was incorporated into the economic analysis using the lower and upper bound approach described in the Interior Drainage Methodology section above. HEC-FDA was used for the economic analysis of the proposed interior drainage alternatives. R&U associated with the HEC-FDA model is discussed in the Economics Appendix.

Table 2: Interior Drainage Analysis Approach

Combination of Interior and Exterior Conditions to be Analyzed											
Interior Flow	Exterior Stage	Time Condition	Peak Int. WSEL	Peak Ext. WSEL	Interior Flow	Exterior Stage	Time	Peak Int. WSEL	Peak Ext. WSEL	Max WS	Risk Condition
2-year	Normal	Current			N/a						Lower Bound
5-year	Normal	Current			N/a						Lower Bound
10-year	Normal	Current			N/a						Lower Bound
25-year	Normal	Current			N/a						Lower Bound
50-year	Normal	Current			N/a						Lower Bound
100-year	Normal	Current			N/a						Lower Bound
250-year	Normal	Current			N/a						Lower Bound
500-year	Normal	Current			N/a						Lower Bound
2-year	2-year	Current			2-year	2-year	Current				Most Likely (2-year)
5-year	2-year	Current			2-year	5-year	Current				Most Likely (5-year)
10-year	2-year	Current			2-year	10-year	Current				Most Likely(10-year)
25-year	2-year	Current			2-year	25-year	Current				Most Likely(25-year)
50-year	2-year	Current			2-year	50-year	Current				Most Likely(50-year)
100-year	2-year	Current			2-year	100-year	Current				Most Likely(100-year)
250-year	2-year	Current			2-year	250-year	Current				Most Likely(250-year)
500-year	2-year	Current			2-year	500-year	Current				Most Likely(500-year)
2-year	10-year	Current			10-year	2-year	Current				Upper Bound
5-year	10-year	Current			10-year	5-year	Current				Upper Bound
10-year	10-year	Current			10-year	10-year	Current				Upper Bound
25-year	10-year	Current			10-year	25-year	Current				Upper Bound
50-year	10-year	Current			10-year	50-year	Current				Upper Bound
100-year	10-year	Current			10-year	100-year	Current				Upper Bound
250-year	10-year	Current			10-year	250-year	Current				Upper Bound
500-year	10-year	Current			10-year	500-year	Current				Upper Bound

3 PROJECT AREA CONDITIONS

3.1 Correlation Analysis

The authorized storm damage reduction features will trap local drainage behind the alignment. In order to release the interior runoff to the Passaic River, Newark Bay, or other drainage conduits, outlet pipes with flap valves and sluice gates to control backflow may be provided along the alignment to supplement the existing combined sewer system. Since the gravity structures cannot discharge runoff against high tailwater stages, it was important to develop an understanding of the relationship between the precipitation events creating significant interior runoff and storm events creating high exterior stages that block the gravity outlets.

A review of historical precipitation and tide data was performed for the South River Hurricane and Storm Risk Management Project in 2002 and again for the South Shore of Staten Island Coastal Storm Risk Management Study in 2016 in order to quantify any correlation between the amount of precipitation and peak surge level during storms. The results of both analyses showed that there was no link between severe precipitation and severe storm surge events and that the simultaneous occurrence of either was random.

3.2 Precipitation

Precipitation data was obtained from New Jersey 24-Hour Rain Fall Frequency Data for 2, 5, 10, 25, 50 and 100-year events and supplemented by NOAA Atlas 14, Volume 2, Version 3, for the estimated 24 hour, 500-year event. The 250-year event was interpolated from average recurrence interval/precipitation depth chart in NOAA Atlas 14.

3.3 Initial Abstraction

Stormwater drainage in the City of Newark is primarily conveyed through a combined sewer system (CSS) to the Passaic Valley Sewage Commission (PVSC) facility at Newark Bay (see **Figure 3**). Some separate stormwater drainage exists (see existing drainage discussion for each project drainage area); however, the primary means is via the combined system. Modeling the stormwater runoff in the CSS in conjunction with existing, separate stormwater drainage features, as well as the large amount of overland flow captured during large events would be exceedingly cumbersome. Without a robust, detailed CSS model, the accuracy of the results would be uncertain. Therefore, using anecdotal information about the CSS performance and capacity of the PVSC plant before combined sewer outfall regulators are triggered, it was determined that the system has a typical maximum capacity to handle approximately 3 inches of rainfall in a 24-hour period, the period used for the analysis. All excess rainfall would be conveyed as overland flow and temporarily ponded, where applicable, until it could be successfully drained by the CSS.

Therefore, an initial abstraction of 3 inches was accounted for in the analysis; the 24-hour rainfall for each frequency was adjusted by reducing the total rainfall amount by 3 inches.

Additional initial abstraction for ground cover (as expressed by CN value) was calculated in the HEC-HMS model. The additional initial abstraction amounts were minor due to the urban nature of the drainage areas.

Figure 2: Portion of the Newark CSS – GIS Data

3.4 Project Stillwater Levels

3.4.1 Coastal Stillwater Elevations

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 (SWEL) data were obtained from the recent North Atlantic Comprehensive Coastal Study (NACCS) coastal surge model and updated to the project years 2020 and 2070.

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 extratropical storms. 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 storms used in the model included over 1,000 synthetic tropical events and 100 extratropical events computed at over three million computational locations. The water levels were modeled to include the effects of storm surge, waves, and tides.

The 1992 tidal epoch was used in the initial NACCS coastal analysis; stillwater elevations in the project area were updated to 2020 levels using USACE Curve 1 projected sea level change data for the region (0.35 feet to 2020; 1.46 feet to 2070). The stillwater stage versus frequency data is shown in **Tables 3 and 4**. The NACCS model effort Executive Summary is provided in Attachment 1 of the Hydrology and Hydraulics (H&H) Appendix. The model information is described in more detail in Reference 1.

Table 3: NACCS Stillwater Elevation - Stage versus Frequency (2020)

Annual Recurrence Interval (frequency)	ACE (probability)	SWEL (feet NAVD88)
2-year	0.5	6.23
5-year	0.2	7.41
10-year	0.1	8.34
25-year	0.04	9.57
50-year	0.02	10.80
100-year	0.01	12.09
250-year	0.004	13.67
500-year	0.002	14.99

Table 4: NACCS Stillwater Elevation - Stage versus Frequency (2070)

Annual Recurrence Interval (frequency)	ACE (probability)	SWEL (feet NAVD88)
2-year	0.5	7.34
5-year	0.2	8.52
10-year	0.1	9.44
25-year	0.05	10.67
50-year	0.02	11.90
100-year	0.01	13.19
250-year	0.005	14.78
500-year	0.002	16.10

3.4.2 Inland Stillwater Elevations

In order to assess the potential impacts of flanking at Segments 1 and 2, accurate inland flood elevations were necessary. The NACCS flood elevations are slightly higher than the FEMA flood elevations developed in 2013 for Newark Bay, as shown in **Table 5**. However, because the NACCS model did not include propagation of the surge inland from the shore, the FEMA model is a better representation of the inland surge elevations. The FEMA stillwater elevation in South Ironbound is lower than the NACCS values but more accurately reflects potential flood risk for the area and allowed for a more accurate analysis of potential flood risk management measures.

Table 5: NACCS/FEMA Stage versus Frequency Comparison

Annual Recurrence Interval (frequency)	ACE (probability)	NACCS SWEL (feet NAVD88)	FEMA (feet NAVD88)	South Ironbound (feet NAVD88)
2-year	0.5	6.2	3.8	2.8 (3)
5-year	0.2	7.4	5.5	2.8 (3)
10-year	0.1	8.3	6.9	2.8 (3)
20-year	0.05	9.6	8.4 (1)	6.4
50-year	0.02	10.8	9.6	7.9
100-year	0.01	12.1	10.8	9.1
200-year	0.005	13.7	12.7 (2)	10.3
500-year	0.002	15.0	14.0	11.8

Notes:
(1) FEMA 25-year
(2) FEMA 250-year
(3) Controlled by Peripheral Ditch Flood Gate. Normal Tide elevation of 2.76 feet NAVD88.

3.4.3 Inland Backwater Conditions

As noted in **Table 5**, Segments 1and 2 are subject to backwater controlled by Liberty International Airport's Peripheral Ditch and its tide gates and pump station. The gate height is at approximate elevation 8 feet NAVD88. The elevation of the pump station, parking lot and nearby controlling elevations is uncertain but appears to range from approximately 6.5 feet to 8 feet NAVD88. For the interior drainage analysis of Segments 1 and 2, it was assumed that the tide gate limited upstream surge up to the 10% ACE (10-year) surge (8.3 feet NAVD). This is based on the minimal, if any, head over the dam and adjacent controlling elevations and expected time for that overflow to reach 4 miles upstream towards Segments 1 and 2, as shown in **Figure 3**.

3.5 Waves and Overtopping

Because the Recommended Plan alignment is set back from river and bay shorelines or in areas not subject to significant waves due to limited fetch, it is not expected to experience any significant wave action during surge events. Any waves from Newark Bay or from the south will be dampened by existing buildings and infrastructure. Therefore, wave impacts and overtopping were not considered in the interior drainage analysis.

3.6 Drainage Area Connectivity

The interior of the project area was divided into five major drainage areas based on the contributions to the expected ponding or storage areas. As shown in **Figure 4**, Drainage Area 1 (DA1) contributes to ponding behind Segment 1. DA2, the largest drainage area, is the major contributor to ponding in the South Ironbound Area. DA3 contributes to ponding on the east side of the project area. DA5 is a small area of higher ground which ponds in the parking area but may overflow to DA2 and DA3. DA4 drains northward to the Passaic River. Should flood elevations reach 12 feet NAVD in DA4, flood waters will overflow into DA2.

3.7 Drainage Area Descriptions

3.7.1 Drainage Area 1

DA1, a 2.2-square mile area, includes the area adjacent to Segment 1 and the uphill contributing drainage area. The drainage area is shown in **Figure 5** with existing conditions ponding.

3.7.2 Drainage Area 2

DA2, a 2.7-square mile area, includes the South Ironbound area and the uphill contributing drainage area. The drainage area is shown in **Figure 6** with existing conditions ponding.

3.7.3 Drainage Areas 3 and 5

DA3 and DA5, 0.88 and 0.06 square miles in area, respectively, are in the eastern side of the project area along I-95. DA5 is a small, higher elevation, somewhat isolated area consisting primarily of parking lots. Excessive ponding in DA5 due to extreme rainfall events exceeding the

500-year event may overflow into DA2 and DA3. The drainage areas are shown in **Figure 7** with existing conditions ponding.

3.7.4 Drainage Area 4

DA4, on the north side of the project area, 0.45 square miles in area, drains north to the Passaic River near Minish Park. The drainage area is shown in **Figure 8** with existing conditions ponding.

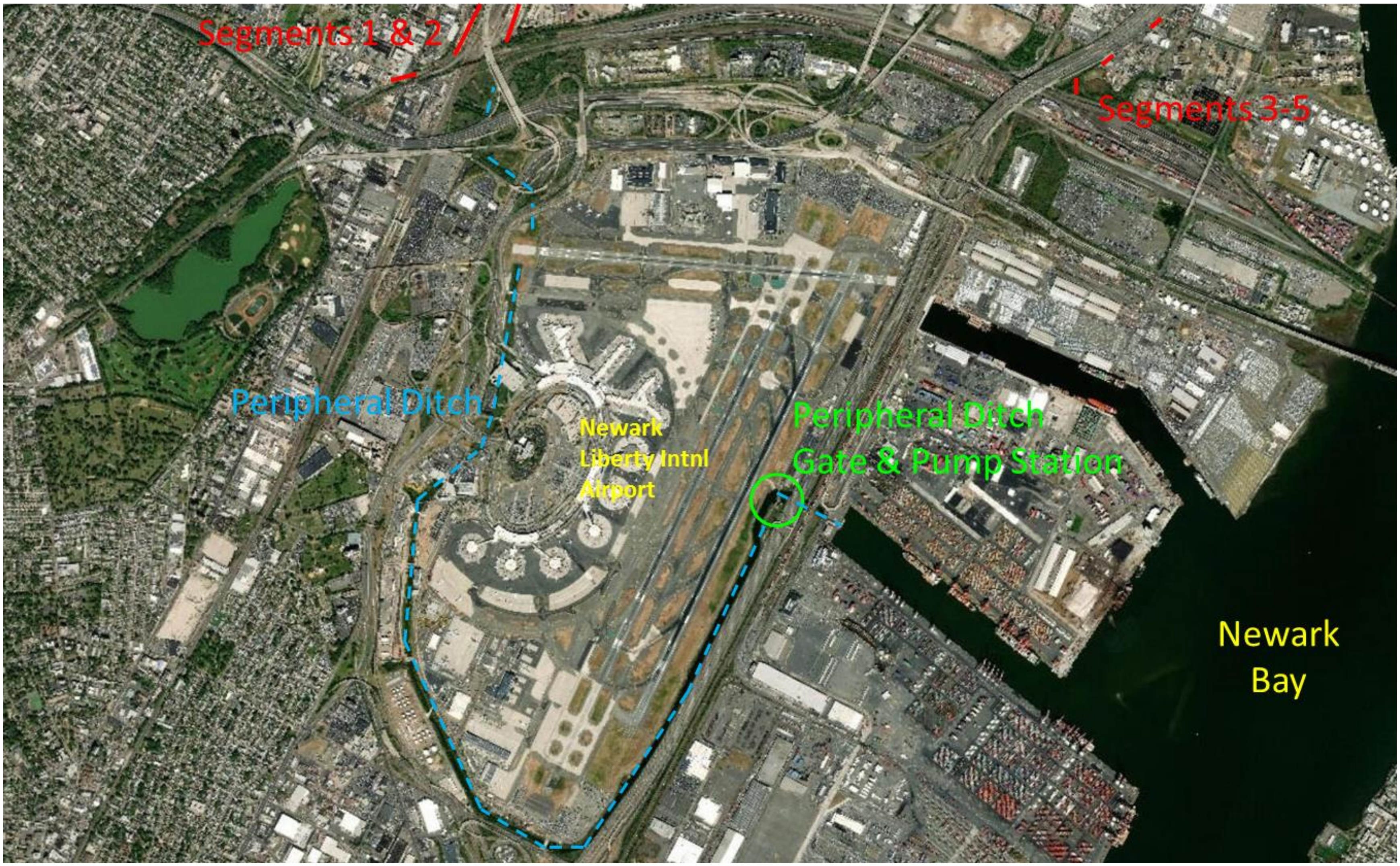

Figure 3: Peripheral Ditch

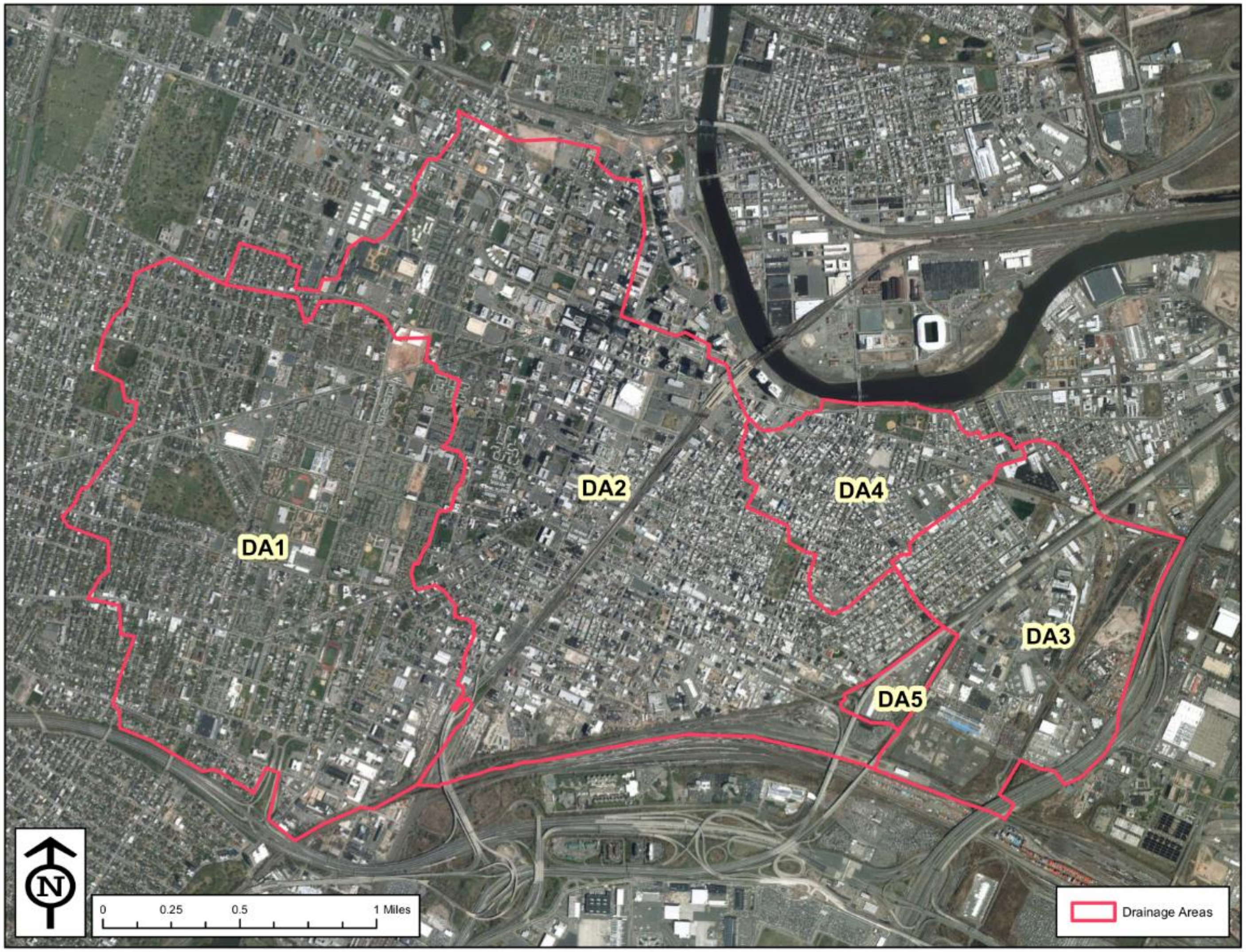

Figure 4: Interior Drainage Areas

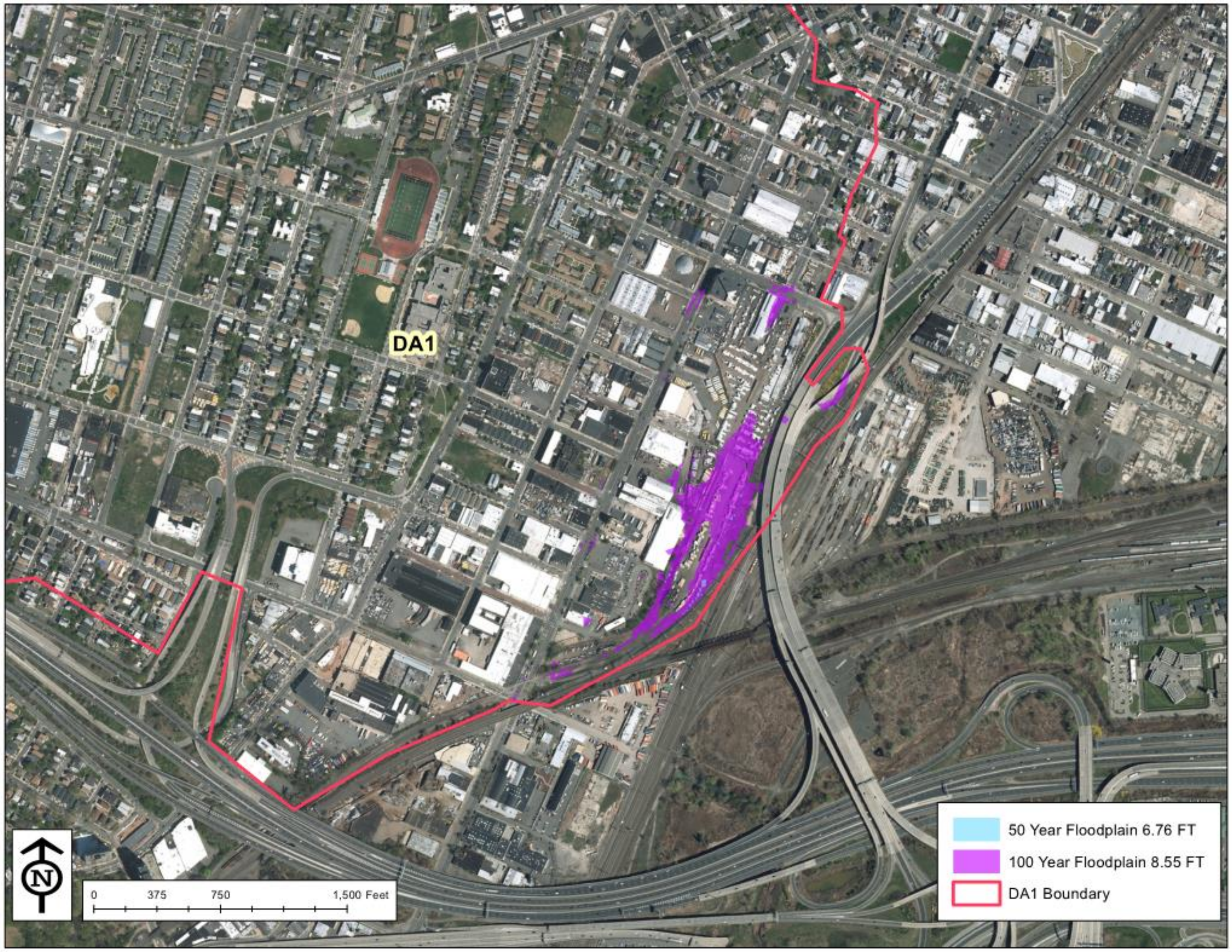

Figure 5: Drainage Area 1

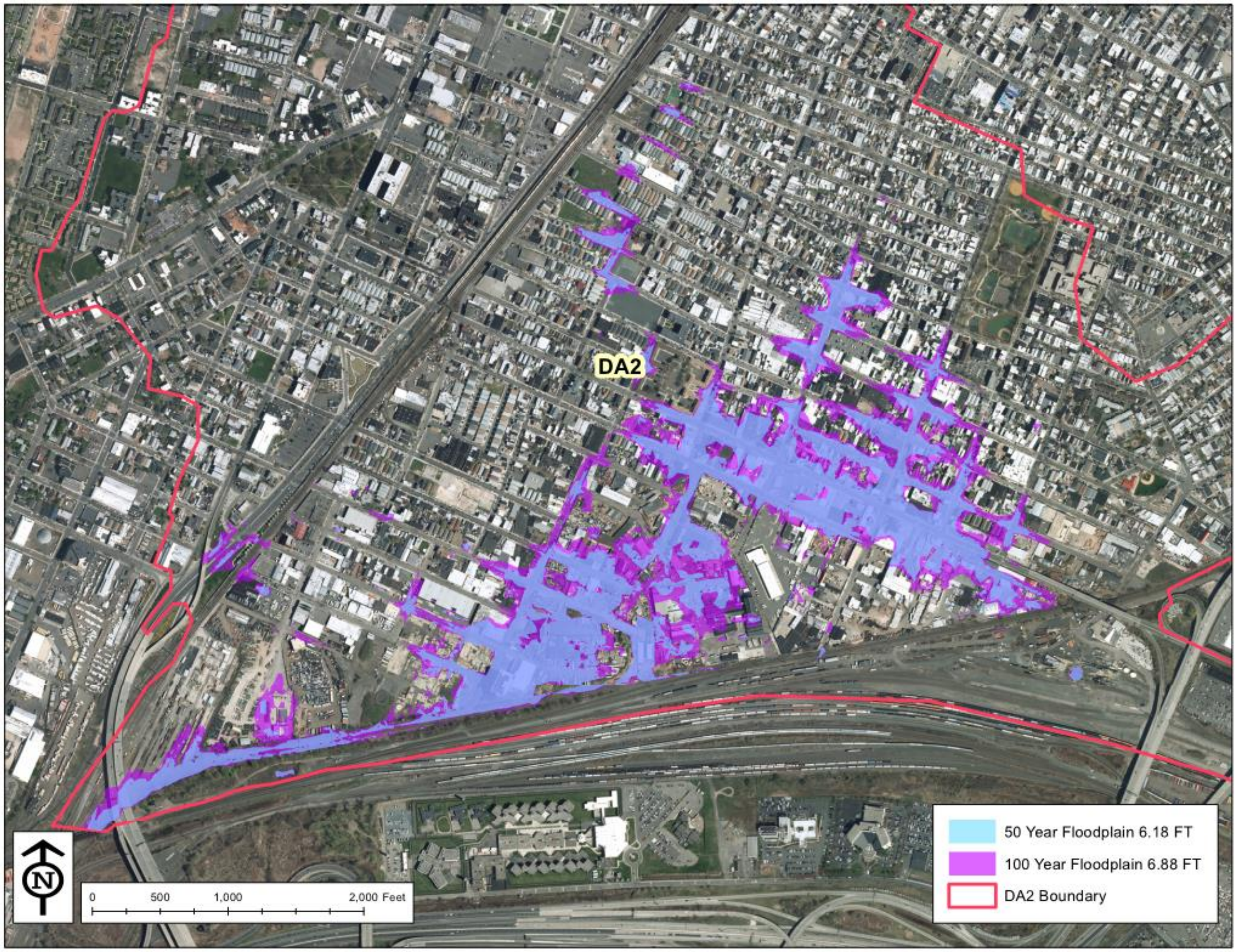

Figure 6: Drainage Area 2

Figure 7: Drainage Areas 3 and 5

Figure 8: Drainage Area 4

3.8 Existing Drainage Features

Review of existing stormwater drainage information available in the City of Newark CSS model, older PDF stormwater drainage plans, and inspection via aerial imagery and site visits identified a number of stormwater drainage features within the drainage areas, in addition to the CSS features. These include drainage ditches, culverts, and overland flow paths, which were incorporated into the HEC-HMS model as “weir” or “spillway” structures. The existing features are described in the following sections.

3.8.1 Drainage Area 1

DA1 existing stormwater drainage includes two existing stormwater outfalls. The intersection’s underpass is also a flow path for excessive flood flow. A portion of the drainage area is shown in **Figure 9** with existing conditions ponding elevations and drainage features. Items noted as “spillways” are controlling high ground which may function as ‘spillways’ when ponding reaches a particular depth.

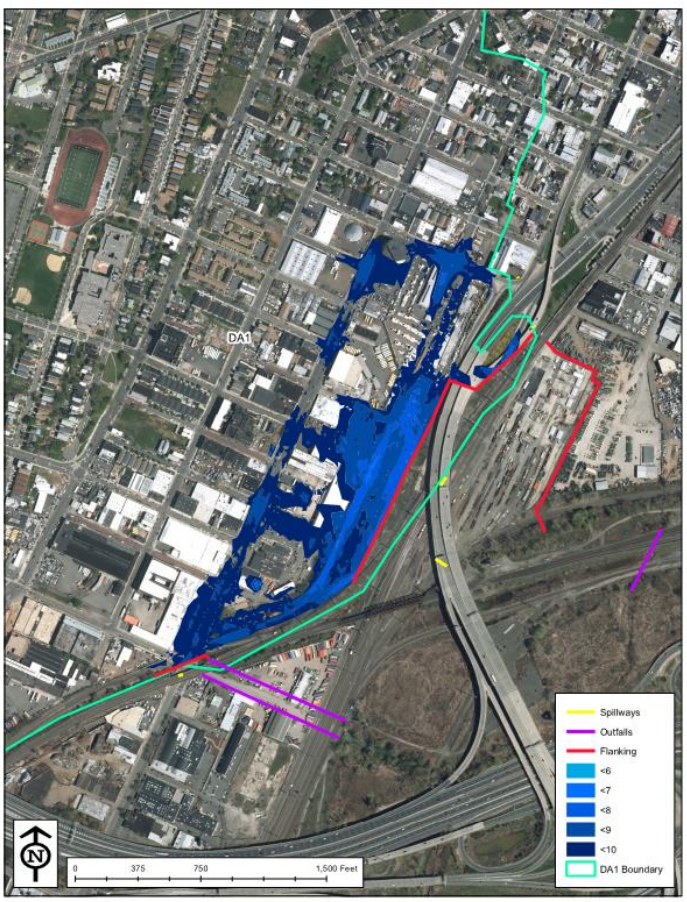

Figure 9: Drainage Area 1 Existing Drainage and Ponding Elevations

3.8.2 Drainage Area 2

DA2 existing stormwater drainage also includes two existing stormwater outfalls. High ground to the west along the line of protection at the NJ Transit rail lines and rail yard also act as controlling elevations for overflow of excessive flooding. At approximate elevation 13 feet NAVD88, the main railroad tracks forming the southern alignment form a spillway. A portion of the drainage area is shown in **Figure 10** with ponding elevations and drainage features.

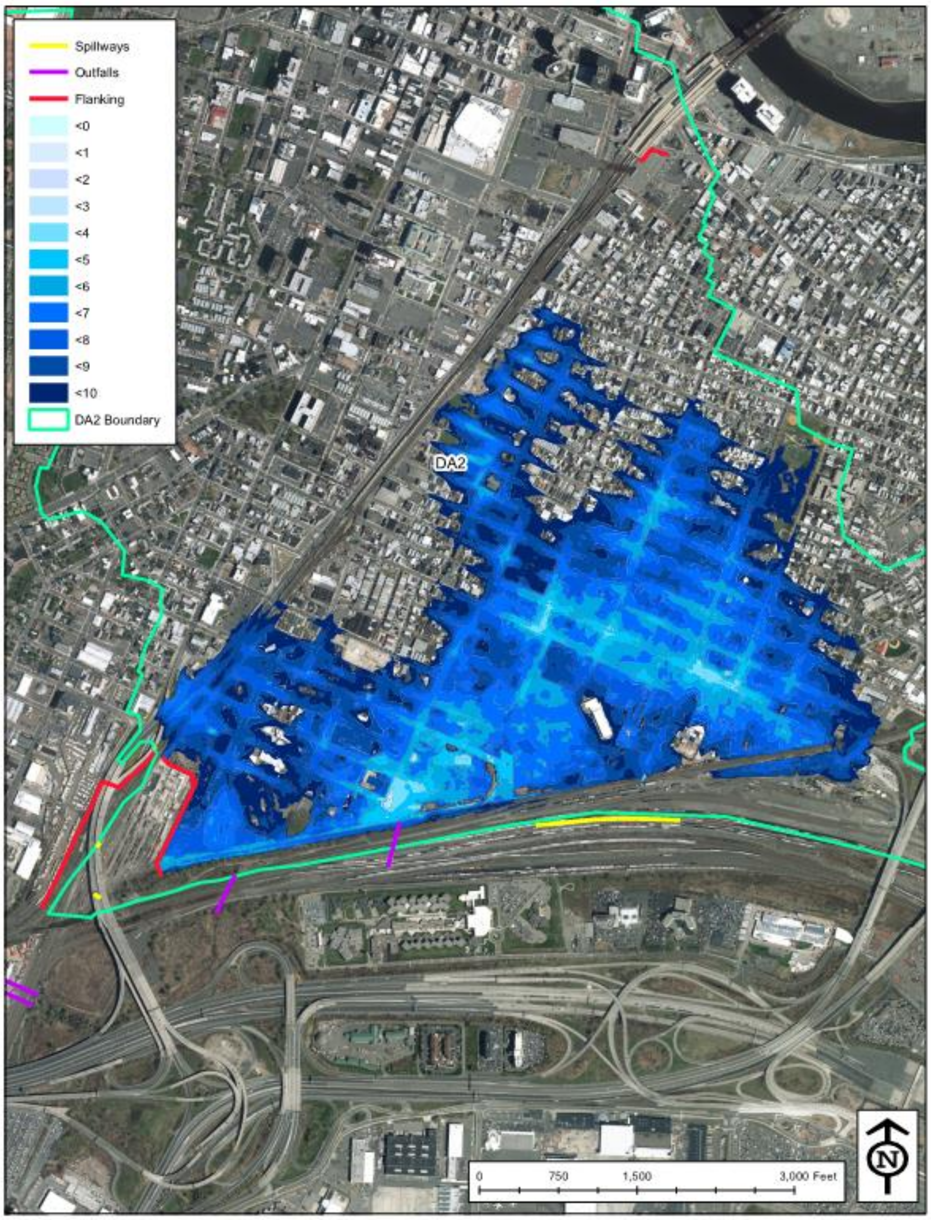

Figure 10: Drainage Area 2 Existing Drainage and Ponding Elevations

3.8.3 Drainage Areas 3 and 5

DA3 and DA5 existing stormwater drainage also includes two existing stormwater outfalls. The underpasses at Delancy Street and Wilson Avenue function as spillways at elevation 10 feet NAVD; however, they are not active even during extreme rainfall events up to the 0.2% ACE. A portion of the drainage areas are shown in **Figure 11** with existing conditions ponding elevations and drainage features.

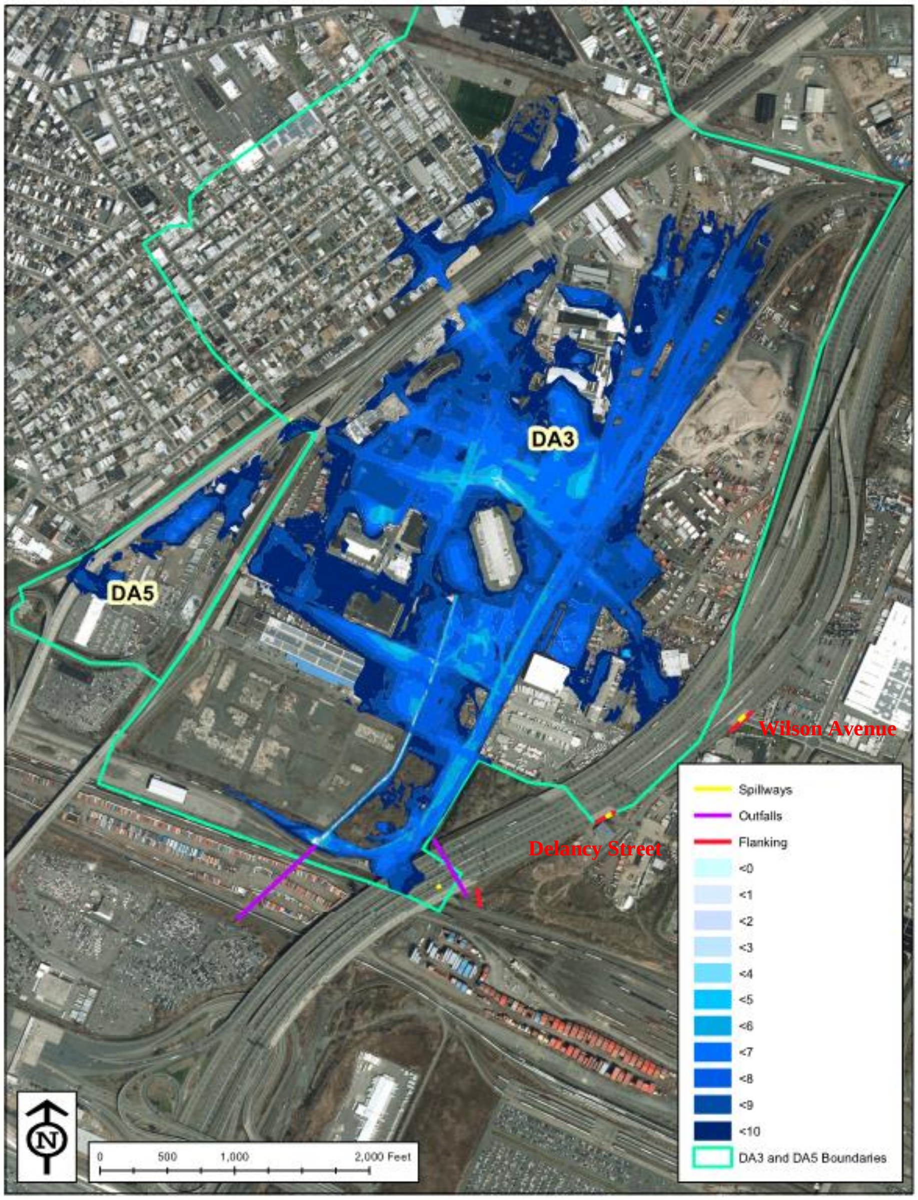

Figure 11: Drainage Areas 3 and 5 Existing Drainage and Ponding Elevations

3.8.4 Drainage Area 4

DA4 has a low, ponding spot on Raymond Boulevard under the Jackson Street Bridge. Several catch basins are located here, which connect to the combined sewer system in the vicinity of the combined sewer outfall regulator. If the regulator is blocked due to excessive tidal surge, ponding will occur. Ponding exceeding 12 feet NAVD88 will flow towards DA2 and the South Ironbound area before flowing up against the proposed Segment 8 wall. The base of the Segment 8 floodwall section at Riverside Park is at 12.5 foot NAVD88; therefore, it does not affect ponded runoff in this drainage area. Thus, no additional drainage features are proposed for DA4. The drainage area is shown in **Figure 12** with existing conditions ponding elevations.

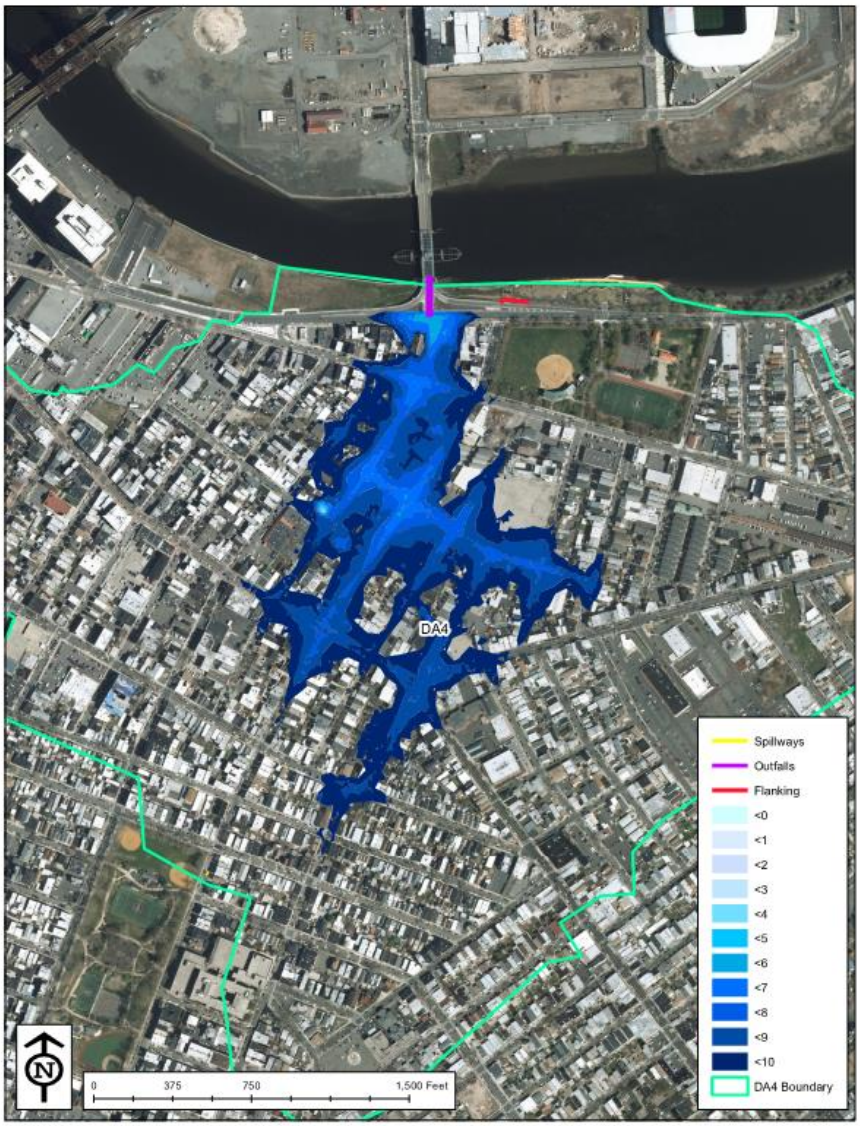

Figure 12: Drainage Area 4 Existing Drainage and Ponding Elevations

4 INTERIOR DRAINAGE MODEL

The interior drainage analysis for the Recommended Plan was conducted using HEC-HMS, a rainfall runoff and routing software. Recent, post-Sandy collected Light Detection and Ranging (LiDAR) data was used as base terrain information.

4.1 HEC-HMS Model

The HEC-HMS runoff model schematic is shown in **Figure 13**. The ponding areas are shown linked by auxiliary 'weirs' or high ground over which stored water flows when storage elevations are exceeded. Local stormwater drainage and topographic features direct all of the runoff toward the combined sewer system or the ponding/storage areas. The following features are incorporated into the interior drainage HEC-HMS model:

1) Interior ponds/storage areas which consist of the natural storage available in existing ditches and low-lying areas.
2) Existing stormwater drainage features including culverts, pipes, and controlling elevation weirs.
3) Proposed gravity outlets through the alignment and supplemental pump stations, as evaluated.

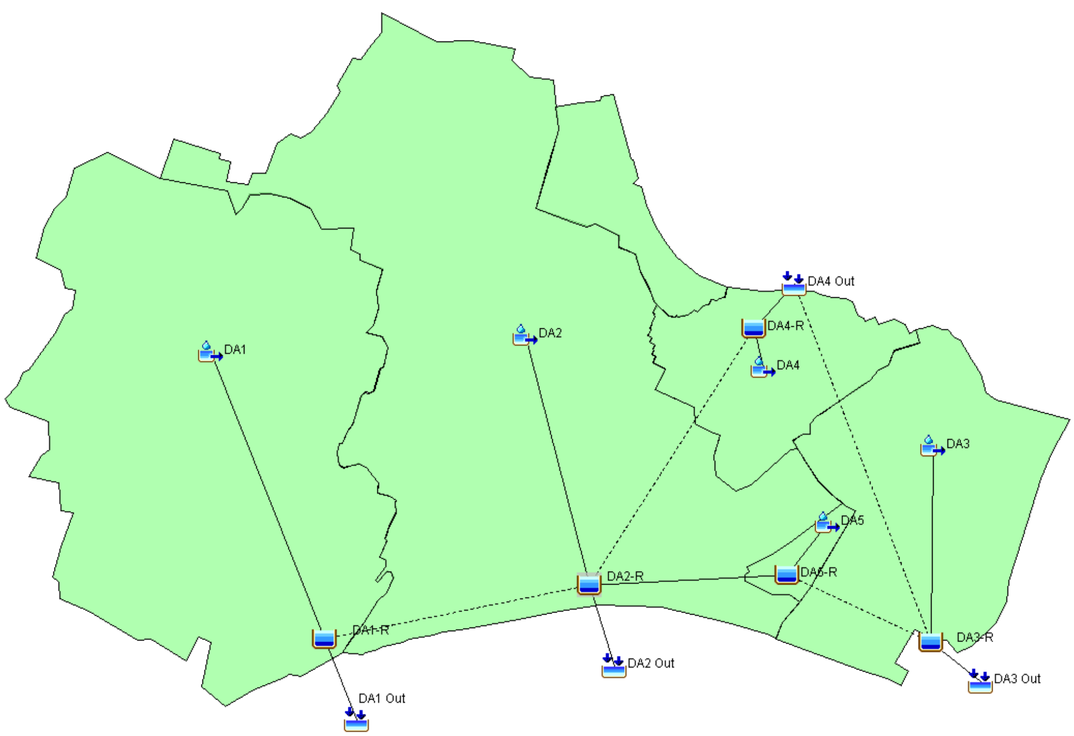

Figure 13: Interior Drainage Model Schematic

4.2 Model Parameters

The drainage area and runoff parameters are shown in **Tables 6 through 8**. Runoff from these areas is reflected in the HEC-HMS model schematic shown in **Figure 12.** The ponding areas are shown linked by weirs, which represent high ground between the storage areas. Should the water depths in the ponding areas exceed the adjacent weir heights, flow would be diverted to the adjacent pond with a lower elevation.

Table 6: Interior Drainage Area Parameters

Subarea	Drainage Areas (square miles)	SCS Curve Number	SCS Unitgraph	
			Tc (min)	Lag (min)
DA1	2.20	91	120.30	72.18
DA2	2.70	92	172.56	103.54
DA3	0.88	91	177.96	106.78
DA4	0.45	93	112.20	67.32
DA5	0.06	94	37.62	22.57

Table 7: Precipitation Rata

Recurrence Interval (yrs)	Precipitation Depths (in)
2	3.29
5	4.22
10	5.01
25	6.19
50	7.22
100	8.35
250	9.62
500	11.50
* Source: NoAA Atlas Volume 2, Ver 3; Location: Lat 40.7259, Long -74.1565	

Table 8: Interior Drainage Area – Peak Runoff

Subarea	Runoff (cfs)							
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	250-yr	500-yr
DA1	17	136	336	842	1,363	1,968	2,944	3,647
DA2	22	165	382	865	1,351	1,910	2,854	3,593
DA3	7	54	126	286	448	631	930	1,147
DA4	4	32	79	195	312	445	657	805
DA5	1	6	19	57	93	134	191	228

4.3 Gate Operational Parameters

As discussed in Section 3.5, Segments 1 – 3 are subject to backwater controlled by Liberty International Airport's Peripheral Ditch and its tide gates and pump station. This is reflected in the FEMA inland flood elevations shown in Table 5; it was assumed that the tide gate limited upstream surge up to the 10% ACE (10-year) surge (8.3 feet NAVD). Therefore, the gates in Segment 1 and 2 were assumed to remain open and provide a conduit for interior flood waters up to and including the 10-year surge.

The fact that during higher frequency surge events (up to the 10-year) stormwater runoff from the interior of Newark is still able to outflow to the Peripheral Ditch needs to be incorporated into the Operations Manual for the gates. That is, the gates should remain open during moderate surge to allow for the conveyance of stormwater during coincidental extreme rainfall events.

5 ECONOMIC CRITERIA

5.1 Conditions

The analysis of benefits and costs for formulation of the interior drainage plans was conducted using a discount rate of 2.875% over a 50-year project life.

5.2 Costs

5.2.1 General

Interior drainage facility costs are based on incremental improvements and are additive to features integral to the alignment (i.e., the minimum facilities). These costs consist of first construction costs, real estate costs, and annual operation and maintenance (O&M) expenses. Each of these is described below.

5.2.2 First Construction Costs

First construction costs assigned to interior drainage facilities include gravity outlets, intake structures, and gates associated with the outlets and pump stations. Interior drainage costs do not include major alignment costs, but rather are limited to project features that may be altered by the interior drainage design. First costs for items were estimated based on prevailing unit costs. First costs include Engineering and Design (E&D, 15%), Supervision and Administration (S&A, 8%), and Contingency (36%). First land costs include Survey, Appraisal and Administration (9%), and Contingency (10%).

5.2.3 Real Estate Costs

Real estate acquisitions associated with interior drainage facilities are based on the purchase of a permanent drainage easement where interior features are required beyond the alignment and buffer footprint (e.g., diversion pipe easement, ponds, etc.).

Real estate acquisition costs include an adjustment to the base value of the land for surveying, appraisal, and administration (9%), and contingencies (19%).

5.2.4 Operation and Maintenance

Annual charges attributed to the operation and maintenance (O&M) of interior drainage facilities consist of labor charges for the care and cleaning of pond areas, outlets and pump stations, as well as anticipated energy charges and annualized replacement costs, if applicable.

5.2.5 Replacement Costs

Replacement costs for flap gates, trash racks, and sluice gates were assumed to occur every 25 years. The replacement costs were estimated by multiplying the cost of the components by the present worth. The following present worth factor was used: $1 / (1 + \text{Discount Rate})^{25}$. The resultant cost was annualized and included in the O&M costs.

5.2.6 Cost Estimate Assumptions

The following assumptions were made when developing the interior drainage facilities' estimated costs:

- Mobilization/Demobilization, Dewatering: Construction of the interior drainage facilities is assumed to occur simultaneously with the alignment construction; therefore, mobilization, demobilization, and dewatering costs may not required as part of the interior drainage costs unless not part of the alignment.
- Toe ditch / drainage construction: Toe ditch construction costs are assumed to be part of the alignment costs.

5.3 Benefits

5.3.1 General

Benefits due to the reduction of interior flooding are summarized for each interior drainage facility listed in subsequent sections. The benefits for interior drainage facilities are calculated as the difference between minimum facility residual damages and residual damages associated with the interior drainage plan alternative being evaluated.

5.3.2 Residual Flood Damages

As described in the Economics Appendix, the expected damage to structures and vehicles, as well as expected emergency costs, were calculated for various depths of interior or residual flooding; that is, flooding which occurs as a result of the alignment preventing runoff. The residual damages with the minimum facility plan represent the starting point from which additional interior facilities planning commences. The benefits accrued from alternative interior drainage plans are attributable to the reduction in the residual flood damages which may have remained under the minimum facility condition.

5.3.3 Residual Annual Damages

Residual damages were calculated using risk based simulation techniques. The damage analysis assumed that there will be no significant coincidence between the residual interior flooding from rainfall and residual flooding from storms exceeding the alignment. In accordance with EM 1110-2-1413, interior damage was calculated for a full range of interior flood events up to and including the 500-year storm.

6 MINIMUM FACILITIES ANALYSIS

The five drainage areas and associated stormwater storage areas were evaluated using HEC-HMS. Due to the unique location of the Recommended Plan flood risk management area – set back from the river and bay shore, and with several segments subject to limited tailwater effects during high frequency surge events – the combined sewer and existing stormwater drainage features limit any project-induced flooding in four of the five areas. In DA2, minimum facilities can be met with by adding an additional gate to relieve potential ponding at extreme rainfall events (100-, 250-, and 500-year). A summary of the minimum facilities is shown in **Table 9.** Tables showing the hydraulic analysis results are presented in Section 9.

Table 9: Minimum Facilities Features

Basin	Minimum Facilities Description
Drainage Area 1	Existing Conditions
Drainage Area 2	50-foot gate adjacent to railroad
Drainage Area 3	3x36” Culverts in Segment 3 levee; 3x36” culverts under access road for drainage conduit
Drainage Area 4	Existing Conditions
Drainage Area 5	Existing Conditions

7 ALTERNATIVES ANALYSIS

7.1 General

The benefits accrued from alternative plans are attributable to the reduction in the residual flooding and damages which may have remained under the minimum facility condition. For an alternative plan to be justified, it must be implementable and reasonably maximize benefits versus the additional cost required for its construction, operation and maintenance. Plan alternatives examined include the use of pump stations and the diversion of upland runoff. The following is a general description of various plan alternatives that were considered during the development of interior drainage facilities.

7.2 Gravity Outlets

One of the five drainage areas required additional stormwater outlets to meet minimum facilities requirements. Therefore, additional outfalls were initially considered to see if the residual damages could be measurably reduced.

7.3 Excavated Ponds

In general, the urban nature of Newark limits land available for ponding. Therefore, only DA3 was evaluated for a potential ponding alternative due to undeveloped land adjacent to the existing outfalls.

7.4 Pumps

Pumping plans incorporate the use of pump stations in conjunction with the minimum facility features developed for each interior area. Pump stations were considered as a means of reducing residual flood heights within interior storage areas through the mechanical displacement of accumulated surface runoff from the interior watershed. To determine how efficiently a pump station reduces flooding, the resultant reduction in residual flood damages is compared to the initial and annual costs of developing and operating the pump station.

The costs of pumping alternatives are additive to the minimum facility cost. The construction of a pump station creates additional capital or first project costs and also increases annual operation and maintenance costs. Capital expenditures affected by the addition of pump stations include mechanical equipment and associated housing.

7.5 Interior Drainage Alternatives

Interior drainage alternatives for the Recommended Plan included additional gravity outlets, additional storage, and pump stations. The alternatives considered for each drainage area are described below.

7.5.1 Drainage Area 1

Alternative 1: Additional gravity outlets near the existing drainage structures.

Alternative 2: 120 cubic feet per second (cfs) pump station.

Alternative 3: 60 cfs pump station.

Alternative 4: 1 x 66” additional steel drainage pipe.

7.5.2 Drainage Area 2

Alternative 1: Additional gravity outlets draining south from the drainage area.

Alternative 2: 1,200 cfs pump station.

Alternative 3: 500 cfs pump station.

7.5.3 Drainage Area 3

Alternative 1: Additional gravity outlets near the existing culvert.

Alternative 2: 240 cfs pump station.

Alternative 3: Excavated 3.8 acre pond storage with Alternative 1 outfalls.

Alternative 4: Excavated 3.8 acre pond with no additional outfalls.

7.5.4 Drainage Area 4

Alternative 1: Additional gravity outlets on Raymond Boulevard.

Alternative 2: 60 cfs pump station.

Alternative 3: 30 cfs pump station.

7.5.5 Drainage Area 5

Drainage Area 5 is small and primarily a group of parking lots; therefore; alternatives for improving interior drainage in DA5 were not considered.

7.6 Summary of the Alternatives Analysis

7.6.1 Selection Criteria

The selection of viable interior drainage alternatives was guided by specific criteria in each drainage area:

DA1: This drainage area had minimal ponding; therefore, robust (i.e., pump) alternatives were not expected to be viable given the cost versus expected benefits. Locations of potential additional gravity outlets were identified based on topographic characteristics and limitations of the nearby urban development.

DA2: The main stormwater storage area was located well inland behind a wide railroad embankment. Pump size was expected to be limited due to the lack of adequately sized receiving waters nearby for discharge or large volumes of pumped stormwater. The “wet” side of the railroad embankment is a developed area (prison) with mostly underground stormwater drainage.

DA3: While DA3 had significant ponding, the area includes several natural drainage ditches and a significant amount of open space. This provided the opportunity to include existing storage before significant damage occurred from residual flooding as well as space for ponding alternatives that were not viable in other drainage areas due to urban development.

DA4: DA4 ponding is a direct result of the existing stormwater/CSS system and is not impacted by the proposed Recommended Plan. Furthermore, the ponding begins at the existing Raymond Boulevard underpass, thus limiting potential alternatives: creating of ponding areas is not feasible and any additional gravity outlets or pump stations would need to be installed near or under the Jackson Street ramps, through the existing retaining walls, emptying onto or just below the recently developed Minish Park.

DA5: This drainage area is primarily a parking area, which drains to DA3 and DA2 depending on the intensity of the rainfall. Potential benefits are minimal.

7.6.2 Analysis Results

The following is a summary of the alternatives analysis for each drainage area. Detailed cost and benefits are presented in the associated tables.

DA1: An additional gravity outlet in Alternative DA1-1 provides a significant reduction in residual flooding; however, the need to jack the pipe under the railroad and tie-into the downstream outfall makes this alternative more costly. Alternative DA1-4 requires tying into the existing stormwater drainage pipe (approximately 66”x69”) and is much less costly (see **Figure 14**). Pump station alternatives do not provide positive net benefits. The results of the analysis are shown in **Table 10**.

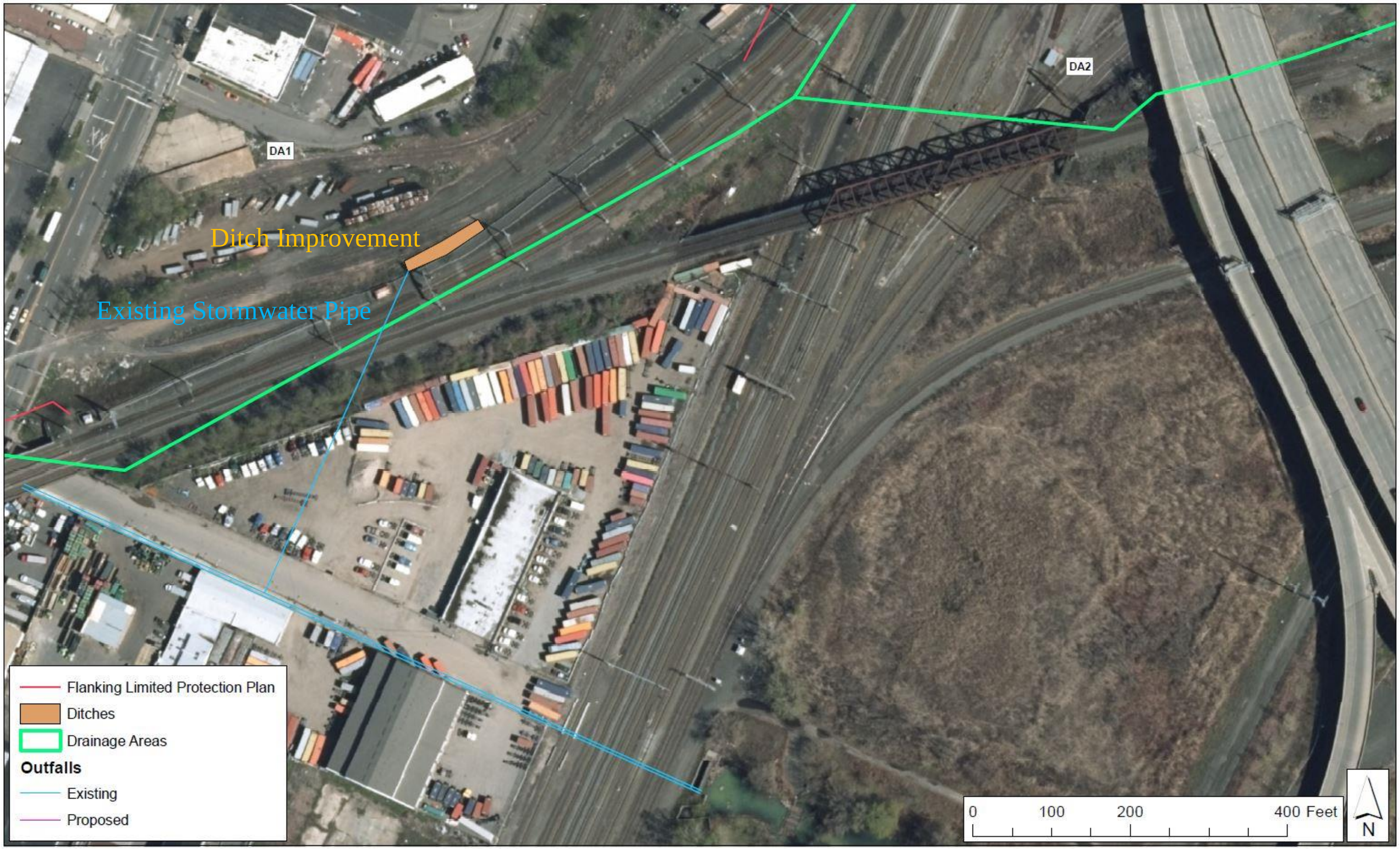

Figure 14: Drainage Area 1 - Interior Drainage Recommended Plan (Alternative DA1-4)

Table 10: DA1 Interior Drainage Analysis Results

	Minimum Facilities	Alt DA1-1 (Gravity)	Alt DA1-2 (120cfs)	Alt DA1-3 (60cfs)	Alt DA1-4 (Tie-in)
Average Annual Damage (AAD)	$192,420	$104,480	$159,750	$174,400	$143,730
AAD Reduced	$0	$87,940	$32,670	$18,020	$48,690
First Cost of Construction	$0	$1,893,797	$7,654,387	$4,872,016	$461,130
Interest During Construction	$0	$24,827	$100,346	$63,870	$6,045
Total Investment Cost	$0	$1,918,624	$7,754,733	$4,935,886	$467,175
Annual Cost	$0	$70,773	$286,053	$182,073	$17,233
Annual O&M	$0	$3,000	$78,340	$67,004	$3,000
Total Annual Costs	$0	$73,773	$364,393	$249,077	$20,233
Annual Benefits		$87,940	$32,670	$18,020	$48,690
BCR	**N/A**	1.19	**0.09**	**0.07**	**2.41**
NET BENEFITS		$14,167	-$331,723	-$231,057	**$28,457**
Construction Duration: 12 months Annual Interest Rate: 2.875%					

DA2: In DA2, the primary stormwater ponding or storage area is set back from the alignment, as shown in **Figure 10**, and is not directly influenced by the project's alignment. Both of the pump stations considered as Alternatives DA2-1 (1,200 cfs) and DA2-2 (500 cfs) appear to provide significant reductions in residual damages; however, the location of the pump stations (inland) and the infrastructure needed to convey the large discharges to suitable receiving waters are prohibitively costly. Furthermore, the potential environmental and induced flooding impacts of channel improvements and the increased discharge on the downstream receiving waters - tributaries to the Peripheral Ditch and the Peripheral Ditch – cannot be easily quantified but are expected to be significant and require significant mitigation. Therefore, the pump stations were screened out as not feasible as interior drainage components of the project. There are no other viable interior drainage alternatives for DA1, as shown in **Table 11**.

Table 11: DA2 Interior Drainage Analysis Results

	Minimum Facility	Alt DA2-1 (Gravity)	Alt DA2-2 (1200 cfs)	Alt DA2-3 (500cfs)
Average Annual Damage (AAD)	$6,416,030	$6,226,490	$733,150	$1,822,960
AAD Reduced	$0	$189,540	$5,682,880	$4,593,070
First Cost of Construction	$0	$7,468,865	$134,962,988	$88,288,031
Interest During Construction	$0	$97,914	$1,769,317	$1,157,425
Total Investment Cost	$0	$7,566,779	$136,732,305	$89,445,456
Annual Cost	$0	$279,120	$5,043,722	$3,299,425
Annual O&M	$0	$9,851	$1,283,400	$1,141,700
Total Annual Costs	$0	$288,971	$6,327,122	$4,441,125
Annual Benefits		$189,540	$5,682,880	$4,593,070
BCR	**N/A**	**0.66**	**0.90**	**1.03**
NET BENEFITS		-$99,431	-$644,242	$151,945
Construction Duration: 12 months Annual Interest Rate: 2.875%				

DA3: High costs associated with lengthy jacked pipe sections, a pump station, or an excavated pond limit the effectiveness of the alternatives for DA3, as shown in **Table 12**. There are no cost effective interior drainage alternatives for DA3. The Interior Drainage Recommended Plan remains the Minimum Facilities plan as shown in **Figure 15**.

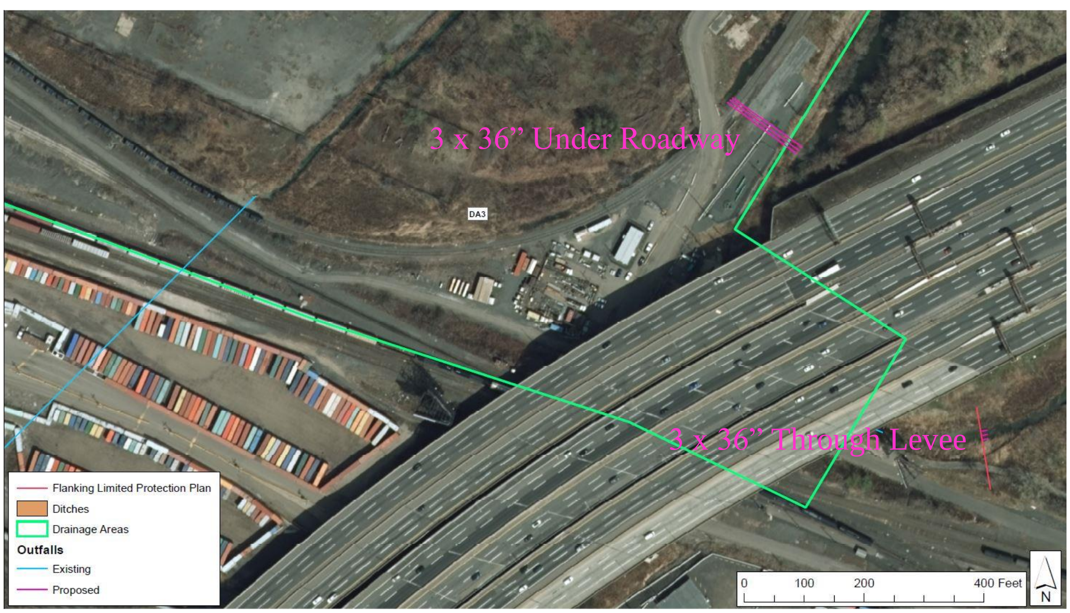

Figure 15: Drainage Area 3 Interior Drainage Recommended Plan

Table 12: DA3 Interior Drainage Analysis Results

	Minimum	Alt DA3-1	Alt DA3-2	Alt DA3-3	Alt DA3-4
	Facilities	(Gravity)	(240 cfs)	(Pond + Gravity	(Pond Only)
Average Annual Damage (AAD)	$196,520	$117,850	$68,920	$96,010	$154,230
AAD Reduced	$0	$78,670	$127,600	$100,510	$42,290
First Cost of Construction	$1,267,251	$7,198,568	$11,081,559	$14,293,945	$7,554,420
Interest During Construction	$16,613	$94,371	$145,275	$187,389	$99,036
Total Investment Cost	$1,283,864	$7,292,939	$11,226,834	$14,481,333	$7,653,455
Annual Cost	$47,359	$269,019	$414,131	$534,181	$282,317
Annual O&M	$0	$8,509	$131,680	$8,509	$3,000
Total Annual Costs		$277,528	$545,811	$542,691	$285,317
Annual Benefits		$78,670	$127,600	$100,510	$42,290
BCR	**N/A**	**0.28**	**0.23**	**0.19**	**0.15**
NET BENEFITS		-$198,858	-$418,211	-$442,181	-$243,027
Construction Duration: 12 months Annual Interest Rate: 2.875%					

DA4: There is no viable interior drainage alternative that does not have significant cost or impact associated with it: additional gravity drains would require extensive jacking through the Jackson Street ramps and pump stations are too costly, as shown in **Table 13**.

Table 13: DA4 Interior Drainage Analysis Results

	Minimum	Alt DA2-1	Alt DA2-2	Alt DA2-3
	Facility	(Gravity)	(1200 cfs)	(500cfs)
Average Annual Damage (AAD)	$102,570	$94,930	$60,390	$77,820
AAD Reduced	$0	$7,640	$42,180	$24,750
First Cost of Construction	$0	$2,365,230	$3,371,485	$2,356,550
Interest During Construction	$0	$31,007	$44,199	$30,894
Total Investment Cost	$0	$2,396,238	$3,415,684	$2,387,444
Annual Cost	$0	$88,391	$125,996	$88,067
Annual O&M	$0	$5,764	$61,336	$57,085
Total Annual Costs	$0	$94,156	$187,332	$145,152
Annual Benefits	$0	$7,640	$42,180	$24,750
BCR	**N/A**	**0.08**	**0.23**	**0.17**
NET BENEFITS	$0	-$86,516	-$145,152	-$120,402
Construction Duration: 12 months Annual Interest Rate: 2.875%				

DA5: As a parking lot, there are not viable alternatives for interior drainage improvement for DA5.

8 SELECTED INTERIOR DRAINAGE PLAN

The alternative interior drainage plans were formulated to provide safe and reliable protection from interior flooding. Due consideration was given to evaluating only feasible alternatives, i.e., alternatives that are implementable and provide equitable protection to properties within the alignment. Selection of an interior drainage plan thus focused on economics; i.e., providing the optimum reduction in damages for the cost of protection.

As outlined within the description of minimum facility, the planning and development of interior drainage facilities is performed independently from the alignment. Each interior drainage area is analyzed individually to determine the optimum alternative. Within each interior drainage area, the economics for a series of alternate facilities were evaluated and compared to determine which contributes the highest level of net excess benefits to the project. The optimum and selected interior drainage alternative for each sub-basin is presented in **Table 14**.

Table 14: **Selected Interior Drainage Plan Summary**

Basin	Plan	Description	Elevation that Flooding Starts (ft NAVD88)	Elevation of First Significant Damage (ft NAVD88)	10-year (10% ACE) Flood Elevation (ft NAVD88)
Drainage Area 1	Alternative DA1-4	Tie low areas into existing 66” x 69” stormwater line	8.0	8.5	No Flooding
Drainage Area 2	Minimum Facilities	50-foot gate adjacent to railroad	4.0	4.0	6.3
Drainage Area 3	Minimum Facilities	3x36” Culverts in Segment 3 levee; 3x36” culverts under access road for drainage conduit	5.0	6.0	5.1
Drainage Area 4	Minimum Facilities	No Additional Features	6.0	6.0	No Flooding
Drainage Area 5	Minimum facilities	No Additional Features	9.0	10.0	No Flooding

9 H&H ANALYSIS RESULTS

Tabular results of the interior drainage analysis are presented in **Tables 15 through 20**. Only the minimum facilities and selected plans for the base year (2020) are shown. Other alternatives and future analyses results are not shown for brevity.

Table 15: DA1 Minimum Facilities Analysis Results (2020; feet NAVD88)

Passaic Tidal - Minimum Facility, No Added Outfalls, Present Tides														
Subbasin DA1 - Stages (feet, NAVD88) and Inflows (cfs)														
Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Interior Inflow	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Interior Inflow	Peak Exterior Stage	Max Interior WSEL	Max Peak Interior Inflow	Risk
2 yr	Normal	Present	2.76	17	2.76							2.76	17	Low
5 yr	Normal	Present	2.81	136	2.76							2.81	136	Low
10 yr	Normal	Present	3.06	336	2.76							3.06	336	Low
25 yr	Normal	Present	4.43	842	2.76							4.43	842	Low
50 yr	Normal	Present	6.24	1363	2.76							6.24	1363	Low
100 yr	Normal	Present	8.89	1968	2.76							8.89	1968	Low
250 yr	Normal	Present	10.84	2944	2.76							10.84	2944	Low
500 yr	Normal	Present	11.62	3647	2.76							11.62	3647	Low
2 yr	2 yr	Present	2.76	17	2.76	2 yr	2 yr	Present	2.76	17	2.76	2.76	17	Likely
5 yr	2 yr	Present	2.81	136	2.76	2 yr	5 yr	Present	2.76	17	2.76	2.81	136	Likely
10 yr	2 yr	Present	3.06	336	2.76	2 yr	10 yr	Present	2.76	17	2.76	3.06	336	Likely
25 yr	2 yr	Present	4.43	842	2.76	2 yr	25 yr	Present	5.00	17	6.40	5.00	842	Likely
50 yr	2 yr	Present	6.24	1363	2.76	2 yr	50yr	Present	5.73	17	7.90	6.24	1363	Likely
100 yr	2 yr	Present	8.89	1968	2.76	2 yr	100yr	Present	6.45	17	9.10	8.89	1968	Likely
250 yr	2 yr	Present	10.84	2944	2.76	2 yr	250 yr	Present	7.32	17	10.30	10.84	2944	Likely
500 yr	2 yr	Present	11.62	3647	2.76	2 yr	500 yr	Present	8.08	17	11.80	11.62	3647	Likely
2 yr	10 yr	Present	2.76	17	2.76	10 yr	2 yr	Present	3.06	336	2.76	3.06	336	High
5 yr	10 yr	Present	2.81	136	2.76	10 yr	5 yr	Present	3.06	336	2.76	3.06	336	High
10 yr	10 yr	Present	3.06	336	2.76	10 yr	10 yr	Present	3.06	336	2.76	3.06	336	High
25 yr	10 yr	Present	4.43	842	2.76	10 yr	25 yr	Present	6.53	336	6.40	6.53	842	High
50 yr	10 yr	Present	6.24	1363	2.76	10 yr	50yr	Present	7.64	336	7.90	7.64	1363	High
100 yr	10 yr	Present	8.89	1968	2.76	10 yr	100yr	Present	8.56	336	9.10	8.89	1968	High
250 yr	10 yr	Present	10.84	2944	2.76	10 yr	250 yr	Present	9.44	336	10.30	10.84	2944	High
500 yr	10 yr	Present	11.62	3647	2.76	10 yr	500 yr	Present	10.02	336	11.80	11.62	3647	High

Table 16: DA1 Alternative 4 Analysis Results (2020; feet NAVD88)

Passaic Tidal - Alternative 4, 1x66" RCP, Present Tides														
Subbasin DA1 - Stages (feet, NAVD88) and Inflows (cfs)														
Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Interior Inflow	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Interior Inflow	Peak Exterior Stage	Max Interior WSEL	Max Peak Interior Inflow	Risk
2 yr	Normal	Present	2.76	17	2.76							2.76	17	Low
5 yr	Normal	Present	2.83	136	2.76							2.83	136	Low
10 yr	Normal	Present	3.18	336	2.76							3.18	336	Low
25 yr	Normal	Present	4.98	842	2.76							4.98	842	Low
50 yr	Normal	Present	6.76	1363	2.76							6.76	1363	Low
100 yr	Normal	Present	8.54	1968	2.76							8.54	1968	Low
250 yr	Normal	Present	10.41	2944	2.76							10.41	2944	Low
500 yr	Normal	Present	11.27	3647	2.76							11.27	3647	Low
2 yr	2 yr	Present	2.76	17	2.76	2 yr	2 yr	Present	2.76	17	2.76	2.76	17	Likely
5 yr	2 yr	Present	2.83	136	2.76	2 yr	5 yr	Present	2.76	17	2.76	2.83	136	Likely
10 yr	2 yr	Present	3.18	336	2.76	2 yr	10 yr	Present	2.76	17	2.76	3.18	336	Likely
25 yr	2 yr	Present	4.98	842	2.76	2 yr	25 yr	Present	5.00	17	6.40	5.00	842	Likely
50 yr	2 yr	Present	6.76	1363	2.76	2 yr	50yr	Present	5.73	17	7.90	6.76	1363	Likely
100 yr	2 yr	Present	8.54	1968	2.76	2 yr	100yr	Present	8.55	17	9.10	8.55	1968	Likely
250 yr	2 yr	Present	10.41	2944	2.76	2 yr	250yr	Present	9.43	17	10.30	10.41	2944	Likely
500 yr	2 yr	Present	11.27	3647	2.76	2 yr	500yr	Present	10.02	17	11.80	11.27	3647	Likely
2 yr	10 yr	Present	2.76	17	2.76	10 yr	2 yr	Present	3.18	336	2.76	3.18	336	High
5 yr	10 yr	Present	2.83	136	2.76	10 yr	5 yr	Present	3.18	336	2.76	3.18	336	High
10 yr	10 yr	Present	3.18	336	2.76	10 yr	10 yr	Present	3.18	336	2.76	3.18	336	High
25 yr	10 yr	Present	4.98	842	2.76	10 yr	25 yr	Present	6.53	336	6.40	6.53	842	High
50 yr	10 yr	Present	6.76	1363	2.76	10 yr	50yr	Present	7.63	336	7.90	7.63	1363	High
100 yr	10 yr	Present	8.54	1968	2.76	10 yr	100yr	Present	8.55	336	9.10	8.55	1968	High
250 yr	10 yr	Present	10.41	2944	2.76	10 yr	250yr	Present	9.43	336	10.30	10.41	2944	High
500 yr	10 yr	Present	11.27	3647	2.76	10 yr	500yr	Present	10.02	336	11.80	11.27	3647	High

Table 17: DA2 Minimum Facilities Analysis Results (2020; feet NAVD88)

Passaic Tidal - Minimum Facility, 100' Gate, Present Tides														
Subbasin DA2 - Stages (feet, NAVD88) and Inflows (cfs)														
Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Interior Inflow	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Interior Inflow	Peak Exterior Stage	Max Interior WSEL	Max Peak Interior Inflow	Risk
2 yr	Normal	Present	4.65	22	5.00							4.65	22	Low
5 yr	Normal	Present	5.50	165	5.00							5.50	165	Low
10 yr	Normal	Present	6.25	382	5.00							6.25	382	Low
25 yr	Normal	Present	7.04	865	5.00							7.04	865	Low
50 yr	Normal	Present	7.43	1351	5.00							7.43	1351	Low
100 yr	Normal	Present	7.86	1910	5.00							7.86	1910	Low
250 yr	Normal	Present	8.38	2844	5.00							8.38	2844	Low
500 yr	Normal	Present	8.76	3572	5.00							8.76	3572	Low
2 yr	2 yr	Present	4.65	22	5.00	2 yr	2 yr	Present	4.65	22	5.00	4.65	22	Likely
5 yr	2 yr	Present	5.50	165	5.00	2 yr	5 yr	Present	4.65	22	5.00	5.50	165	Likely
10 yr	2 yr	Present	6.25	382	5.00	2 yr	10 yr	Present	4.65	22	5.00	6.25	382	Likely
25 yr	2 yr	Present	7.04	865	5.00	2 yr	25 yr	Present	4.42	22	6.40	7.04	865	Likely
50 yr	2 yr	Present	7.43	1351	5.00	2 yr	50yr	Present	4.55	22	7.90	7.43	1351	Likely
100 yr	2 yr	Present	7.86	1910	5.00	2 yr	100yr	Present	4.59	22	9.10	7.86	1910	Likely
250 yr	2 yr	Present	8.38	2844	5.00	2 yr	250 yr	Present	4.61	22	10.30	8.38	2844	Likely
500 yr	2 yr	Present	8.76	3572	5.00	2 yr	500 yr	Present	4.62	22	11.80	8.76	3572	Likely
2 yr	10 yr	Present	4.65	22	5.00	10 yr	2 yr	Present	6.25	382	5.00	6.25	382	High
5 yr	10 yr	Present	5.50	165	5.00	10 yr	5 yr	Present	6.25	382	5.00	6.25	382	High
10 yr	10 yr	Present	6.25	382	5.00	10 yr	10 yr	Present	6.25	382	5.00	6.25	382	High
25 yr	10 yr	Present	7.04	865	5.00	10 yr	25 yr	Present	6.24	382	6.40	7.04	865	High
50 yr	10 yr	Present	7.43	1351	5.00	10 yr	50yr	Present	6.28	382	7.90	7.43	1351	High
100 yr	10 yr	Present	7.86	1910	5.00	10 yr	100yr	Present	6.35	382	9.10	7.86	1910	High
250 yr	10 yr	Present	8.38	2844	5.00	10 yr	250 yr	Present	6.65	382	10.30	8.38	2844	High
500 yr	10 yr	Present	8.76	3572	5.00	10 yr	500 yr	Present	6.95	382	11.80	8.76	3572	High

Table 18: DA3 Minimum Facilities Analysis Results (2020; feet NAVD88)

Passaic Tidal - Minimum Facility, No Added Outfalls, Present Tides														
Subbasin DA3 - Stages (feet, NAVD88) and Inflows (cfs)														
Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Interior Inflow	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Interior Inflow	Peak Exterior Stage	Max Interior WSEL	Max Peak Interior Inflow	Risk
2 yr	Normal	Present	3.06	7	2.76							3.06	7	Low
5 yr	Normal	Present	3.77	54	2.76							3.77	54	Low
10 yr	Normal	Present	4.60	126	2.76							4.60	126	Low
25 yr	Normal	Present	5.87	286	2.76							5.87	286	Low
50 yr	Normal	Present	6.55	448	2.76							6.55	448	Low
100 yr	Normal	Present	7.19	631	2.76							7.19	631	Low
250 yr	Normal	Present	8.01	930	2.76							8.01	930	Low
500 yr	Normal	Present	8.41	1147	2.76							8.41	1147	Low
2 yr	2 yr	Present	3.06	7	2.76	2 yr	2 yr	Present	3.06	7	2.76	3.06	7	Likely
5 yr	2 yr	Present	4.01	54	2.76	2 yr	5 yr	Present	3.06	7	2.76	4.01	54	Likely
10 yr	2 yr	Present	5.10	126	2.76	2 yr	10 yr	Present	3.06	7	2.76	5.10	126	Likely
25 yr	2 yr	Present	5.98	286	2.76	2 yr	25 yr	Present	4.88	7	6.40	5.98	286	Likely
50 yr	2 yr	Present	6.66	448	2.76	2 yr	50yr	Present	5.05	7	7.90	6.66	448	Likely
100 yr	2 yr	Present	7.26	631	2.76	2 yr	100yr	Present	5.08	7	9.10	7.26	631	Likely
250 yr	2 yr	Present	8.06	930	2.76	2 yr	250 yr	Present	5.09	7	10.30	8.06	930	Likely
500 yr	2 yr	Present	8.47	1147	2.76	2 yr	500 yr	Present	5.10	7	11.80	8.47	1147	Likely
2 yr	10 yr	Present	3.06	7	2.76	10 yr	2 yr	Present	5.10	126	2.76	5.10	126	High
5 yr	10 yr	Present	4.47	54	2.76	10 yr	5 yr	Present	5.45	126	2.76	5.45	126	High
10 yr	10 yr	Present	5.65	126	2.76	10 yr	10 yr	Present	5.65	126	2.76	5.65	126	High
25 yr	10 yr	Present	6.61	286	2.76	10 yr	25 yr	Present	5.97	126	6.40	6.61	286	High
50 yr	10 yr	Present	7.16	448	2.76	10 yr	50yr	Present	6.21	126	7.90	7.16	448	High
100 yr	10 yr	Present	7.62	631	2.76	10 yr	100yr	Present	6.51	126	9.10	7.62	631	High
250 yr	10 yr	Present	8.27	930	2.76	10 yr	250 yr	Present	6.85	126	10.30	8.27	930	High
500 yr	10 yr	Present	8.67	1147	2.76	10 yr	500 yr	Present	7.06	126	11.80	8.67	1147	High

Table 19: DA4 Minimum Facilities Analysis Results (2020; feet NAVD88)

Passaic Tidal - Minimum Facility, No Added Outfalls, Present Tides														
Subbasin DA4 - Stages (feet, NAVD88) and Inflows (cfs)														
Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Interior Inflow	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Interior Inflow	Peak Exterior Stage	Max Interior WSEL	Max Peak Interior Inflow	Risk
2 yr	Normal	Present	2.76	4	2.76							2.76	4	Low
5 yr	Normal	Present	2.76	32	2.76							2.76	32	Low
10 yr	Normal	Present	2.96	79	2.76							2.96	79	Low
25 yr	Normal	Present	4.15	195	2.76							4.15	195	Low
50 yr	Normal	Present	5.13	312	2.76							5.13	312	Low
100 yr	Normal	Present	6.07	445	2.76							6.07	445	Low
250 yr	Normal	Present	7.27	657	2.76							7.27	657	Low
500 yr	Normal	Present	8.00	805	2.76							8.00	805	Low
2 yr	2 yr	Present	4.71	4	6.23	2 yr	2 yr	Present	4.71	4	6.23	4.71	4	Likely
5 yr	2 yr	Present	4.75	32	6.23	2 yr	5 yr	Present	5.11	4	7.41	5.11	32	Likely
10 yr	2 yr	Present	5.81	79	6.23	2 yr	10 yr	Present	5.17	4	8.34	5.81	79	Likely
25 yr	2 yr	Present	6.15	195	6.23	2 yr	25 yr	Present	5.20	4	9.57	6.15	195	Likely
50 yr	2 yr	Present	6.28	312	6.23	2 yr	50yr	Present	5.21	4	10.80	6.28	312	Likely
100 yr	2 yr	Present	6.61	445	6.23	2 yr	100yr	Present	5.21	4	12.09	6.61	445	Likely
250 yr	2 yr	Present	7.32	657	6.23	2 yr	250 yr	Present	5.22	4	13.67	7.32	657	Likely
500 yr	2 yr	Present	8.00	805	6.23	2 yr	250 yr	Present	5.22	4	13.67	8.00	805	Likely
2 yr	10 yr	Present	5.17	4	8.34	10 yr	2 yr	Present	5.81	79	6.23	5.81	79	High
5 yr	10 yr	Present	6.11	32	8.34	10 yr	5 yr	Present	6.80	79	7.41	6.80	79	High
10 yr	10 yr	Present	7.38	79	8.34	10 yr	10 yr	Present	7.38	79	8.34	7.38	79	High
25 yr	10 yr	Present	8.02	195	8.34	10 yr	25 yr	Present	8.01	79	9.57	8.02	195	High
50 yr	10 yr	Present	8.26	312	8.34	10 yr	50yr	Present	8.34	79	10.80	8.34	312	High
100 yr	10 yr	Present	8.53	445	8.34	10 yr	100yr	Present	8.63	79	12.09	8.63	445	High
250 yr	10 yr	Present	9.04	657	8.34	10 yr	250 yr	Present	8.94	79	13.67	9.04	657	High
500 yr	10 yr	Present	9.33	805	8.34	10 yr	250 yr	Present	8.94	79	13.67	9.33	805	High

Table 20: DA5 Minimum Facilities Analysis Results (2020; feet NAVD88)

Passaic Tidal - Minimum Facility, No Added Outfalls, Present Tides														
Subbasin DA5 - Stages (feet, NAVD88) and Inflows (cfs)														
Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Interior Inflow	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Interior Inflow	Peak Exterior Stage	Max Interior WSEL	Max Peak Interior Inflow	Risk
2 yr	Normal	Present	6.25	1	-							6.25	1	Low
5 yr	Normal	Present	7.02	6	-							7.02	6	Low
10 yr	Normal	Present	7.92	19	-							7.92	19	Low
25 yr	Normal	Present	8.72	57	-							8.72	57	Low
50 yr	Normal	Present	9.17	93	-							9.17	93	Low
100 yr	Normal	Present	9.50	134	-							9.50	134	Low
250 yr	Normal	Present	10.01	191	-							10.01	191	Low
500 yr	Normal	Present	10.17	228	-							10.17	228	Low
2 yr	2 yr	Present	6.25	1	-	2 yr	2 yr	Present	6.25	1	-	6.25	1	Likely
5 yr	2 yr	Present	7.02	6	-	2 yr	5 yr	Present	6.25	1	-	7.02	6	Likely
10 yr	2 yr	Present	7.92	19	-	2 yr	10 yr	Present	6.25	1	-	7.92	19	Likely
25 yr	2 yr	Present	8.72	57	-	2 yr	25 yr	Present	6.25	1	-	8.72	57	Likely
50 yr	2 yr	Present	9.17	93	-	2 yr	50yr	Present	6.25	1	-	9.17	93	Likely
100 yr	2 yr	Present	9.50	134	-	2 yr	100yr	Present	6.25	1	-	9.50	134	Likely
250 yr	2 yr	Present	10.01	191	-	2 yr	250 yr	Present	6.25	1	-	10.01	191	Likely
500 yr	2 yr	Present	10.17	228	-	2 yr	500 yr	Present	6.25	1	-	10.17	228	Likely
2 yr	10 yr	Present	6.25	1	-	10 yr	2 yr	Present	7.92	19	-	7.92	19	High
5 yr	10 yr	Present	7.02	6	-	10 yr	5 yr	Present	7.92	19	-	7.92	19	High
10 yr	10 yr	Present	7.92	19	-	10 yr	10 yr	Present	7.92	19	-	7.92	19	High
25 yr	10 yr	Present	8.72	57	-	10 yr	25 yr	Present	7.92	19	-	8.72	57	High
50 yr	10 yr	Present	9.17	93	-	10 yr	50yr	Present	7.92	19	-	9.17	93	High
100 yr	10 yr	Present	9.50	134	-	10 yr	100yr	Present	7.92	19	-	9.50	134	High
250 yr	10 yr	Present	10.01	191	-	10 yr	250 yr	Present	7.92	19	-	10.01	191	High
500 yr	10 yr	Present	10.17	228	-	10 yr	500 yr	Present	7.92	19	-	10.17	228	High

Subappendix 2
NED Plan Interior Drainage

DRAFT

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1 INTRODUCTION

The Hydrology and Hydraulics (H&H) 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 Recommended Plan, 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 risk management measures 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 multiple design elevations 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 alternative alignments 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 calculate estimated 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. The U.S. Army Corps of Engineers (USACE) selected the LPP as the Recommended Plan.

This appendix provides the Interior Drainage Analysis for the National Economic Development (NED) Plan.

A general project location map of the Passaic River Tidal Project Area (the ABU Project) is provided in **Figure 1**, which shows the 1995 plan alignment.

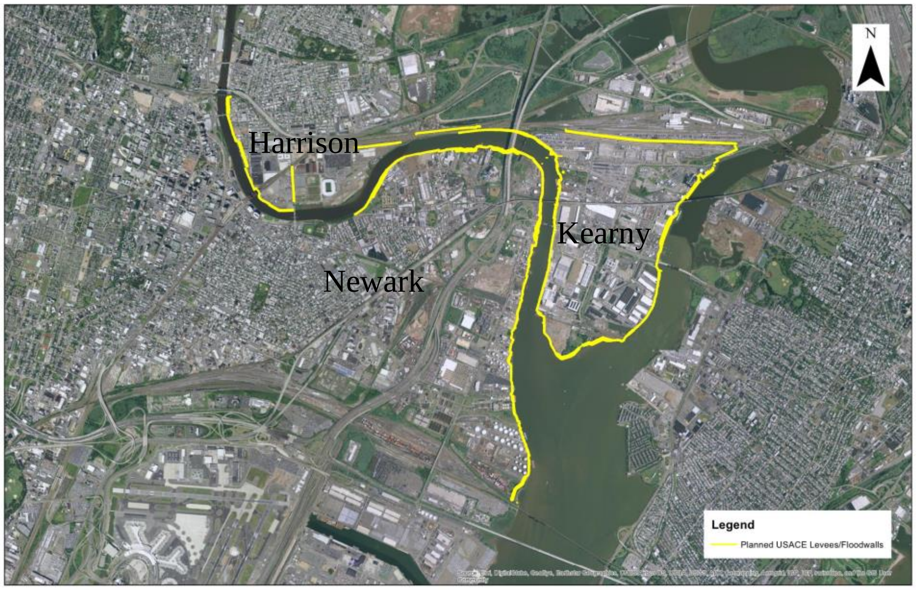

Figure 1: Passaic River Tidal Project Area – 1995 GDM Alignment

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 one in 100 chance of occurring in any single year would be described as the one 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
4%	0.04	25-year
2%	0.02	50-year
1%	0.01	100-year
0.4%	0.004	250-year
0.2%	0.002	500-year

1.2 Survey and Datum

The latest topographic data used 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 (NGVD29). The conversion factor from NGVD29 to North American Vertical Datum of 1988 (NAVD88) is approximately -1.1 feet; therefore, the 1995 GDM design elevation of 14.9 feet NGVD29 is converted to 13.8 feet NAVD88. For ease in analysis, computation and discussions, the 1995 GDM design elevation is rounded to 14 feet NAVD88.

2 PASSAIC RIVER AND NEWARK BAY STILLWATER

2.1 Coastal Stillwater Elevations

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 (SWEL) data were obtained from the recent North Atlantic Comprehensive Coastal Study (NACCS) coastal surge model and updated to the project years 2020 and 2070.

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 extratropical storms. 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 storms used in the model included over 1,000 synthetic tropical events and 100 extratropical events computed at over three million computational locations. The water levels were modeled to include the effects of storm surge, waves, and tides.

The 1992 tidal epoch was used in the initial NACCS coastal analysis; stillwater elevations in the project area were updated to 2020 levels using USACE Curve 1 projected sea level change data for the region (0.35 feet to 2020; 1.46 feet to 2070). The stillwater stage versus frequency data is shown in **Tables 2 and 3**. The NACCS model effort Executive Summary is provided in Attachment 1 of the Hydrology and Hydraulics (H&H) Appendix. The model information is described in more detail in Reference 1.

Table 2: NACCS Stillwater Elevation - Stage versus Frequency (2020)

Annual Recurrence Interval (frequency)	ACE (probability)	SWEL (feet NAVD88)
2-year	0.5	6.23
5-year	0.2	7.41
10-year	0.1	8.34
25-year	0.04	9.57
50-year	0.02	10.80
100-year	0.01	12.09
250-year	0.004	13.67
500-year	0.002	14.99

Table 3: NACCS Stillwater Elevation - Stage versus Frequency (2070)

Annual Recurrence Interval (frequency)	ACE (probability)	SWEL (feet NAVD88)
2-year	0.5	7.34
5-year	0.2	8.52
10-year	0.1	9.44
25-year	0.05	10.67
50-year	0.02	11.90
100-year	0.01	13.19
250-year	0.005	14.78
500-year	0.002	16.10

2.2 Inland Stillwater Elevations

In order to assess the potential impacts of flanking at Segments 1, 2 and south of Segment 3, accurate inland flood elevations were necessary. The NACCS flood elevations are slightly higher than the FEMA flood elevations developed in 2013 for Newark Bay, as shown in **Table 4**. However, because the NACCS model did not include propagation of the surge inland from the shore, the FEMA model is a better representation of the inland surge elevations. The FEMA stillwater elevation in South Ironbound is lower than the NACCS values but more accurately reflects potential flood risk for the area and allowed for a more accurate analysis of potential flood risk management measures.

Table 4: NACCS/FEMA Stage versus Frequency Comparison

Annual Recurrence Interval (frequency)	ACE (probability)	NACCS SWEL (feet NAVD88)	FEMA (feet NAVD88)	South Ironbound (feet NAVD88)
2-year	0.5	6.2	3.8	2.8 (3)
5-year	0.2	7.4	5.5	2.8 (3)
10-year	0.1	8.3	6.9	2.8 (3)
20-year	0.05	9.6	8.4 (1)	6.4
50-year	0.02	10.8	9.6	7.9
100-year	0.01	12.1	10.8	9.1
200-year	0.005	13.7	12.7 (2)	10.3
500-year	0.002	15.0	14.0	11.8

Notes:
(1) FEMA 25-year
(2) FEMA 250-year
(3) Controlled by Peripheral Ditch Flood Gate. Normal Tide elevation of 2.76 feet NAVD88.

3 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 stillwater 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 (i.e., 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.

A detailed discussion of waves in the project area and alignment overtopping is provided in the H&H Appendix main text.

4 INTERIOR DRAINAGE ANALYSIS

As part of the GRR, the interior drainage plan from the 1995 GDM was remodeled and evaluated. The plan included 160 outfalls and six pump stations. The plan was not reformulated;

therefore, new interior drainage alternatives were not considered. The following is a description of the general components of the NED Plan 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 structure.
2) Pump Stations: There are six pump stations in the interior drainage plan. They range from 30 to 100 cfs.

The drainage areas analyzed for the ABU plan are similar to the areas in the 1995 GDM; however, the areas were verified/redelineated using updated topographic data from 2012. This resulted in some minor changes. Drainage area runoff parameters were unchanged from the 1995 GDM. A detailed description of the interior drainage model and results is discussed in the following sections.

4.1 Recent Storm History

Essex County is subject to impacts from coastal storms, often characterized as nor'easters, which are most frequent between October and April. These storms track over the coastal plain or up to several hundred miles offshore, bringing strong winds and heavy rains. Rarely does a winter go by without at least one significant coastal storm and some years see upwards of five to ten. Tropical storms and hurricanes are also a special concern along the coast. In some years, they contribute a significant amount to the precipitation totals of the region. Damage during times of high tide can be severe when tropical storms or nor'easters affect the region.

Flooding in Essex County can occur during any season of the year since New Jersey lies within the major storm tracks of North America. The worst storms have occurred in late summer or early fall when tropical disturbances (hurricanes) are most prevalent. Recent tropical events include Tropical Storm Floyd, Hurricane Irene, and Hurricane Sandy.

Hurricane Floyd originally made landfall in Cape Fear, North Carolina as a Category 2 hurricane. The storm crossed over North Carolina and southeastern Virginia before briefly entering the western Atlantic Ocean. The storm reached New Jersey on September 16, 1999, as a tropical storm. Record breaking flooding from rainfall exceeding 14 inches was recorded throughout the State of New Jersey. A Federal Emergency Declaration was issued on September 17, 1999 and a Major Disaster Declaration was issued on September 18, 1999.

Having earlier been downgraded to a tropical storm, Hurricane Irene came ashore in Little Egg Inlet in Southern New Jersey on August 28, 2011. In anticipation of the storm Governor Chris Christy declared a state of emergency on August 25, with President Obama reaffirming the declaration on August 27. Mandatory evacuations were ordered throughout the State of New Jersey. Wind speeds were recorded at 75 miles per hour (mph) and rainfall totals reached over 10 inches in many parts of the state. Extensive flooding throughout Essex County caused damage to homes, businesses, and public infrastructure. The flooding was exacerbated by high water levels in reservoirs and wetlands as a result of previous heavy rains. Over one million customers lost power during the storm. Overall damage estimates for the State of New Jersey came to over one billion dollars, with over 200,000 homes and buildings being damaged. A Major Disaster Declaration was issued on September 15, 2011.

Hurricane Sandy came ashore as an immense tropical storm in Brigantine, New Jersey, on October 29, 2012. Although rainfall was limited to less than 2 inches within Essex County, wind gusts were recorded up to 76 mph. A full moon made the high tides 20 percent higher than normal and amplified the storm surge. The New Jersey shore suffered the most damage. Seaside communities were damaged and destroyed up and down the coastline. Some 2.7 million households within New Jersey lost power. Initial reports suggested that 72,000 homes and businesses statewide were damaged or destroyed by the storm. Hurricane Sandy was estimated to cost the State of New Jersey over $36 billion. A Federal Emergency Declaration was issued on October 28, 2012 and a Major Disaster Declaration was issued on October 30, 2012.

4.2 Study Area

The study area encompasses 5.0 square miles in the City of Newark, 0.65 square miles in the Town of Harrison, and 2.73 square miles in the Town of Kearny. The Passaic and Hackensack Rivers intersect the study area as shown in **Figure 2**.

The study area is a mixed use area of industrial, commercial, and residential development. The waterfront is mostly developed for industrial uses including shipping (oil and gas, containers/consumer goods) and wastewater treatment. Related rail, barge, truck, and storage infrastructure line the waterfront. The NED Plan project segments are shown in **Figure 3**.

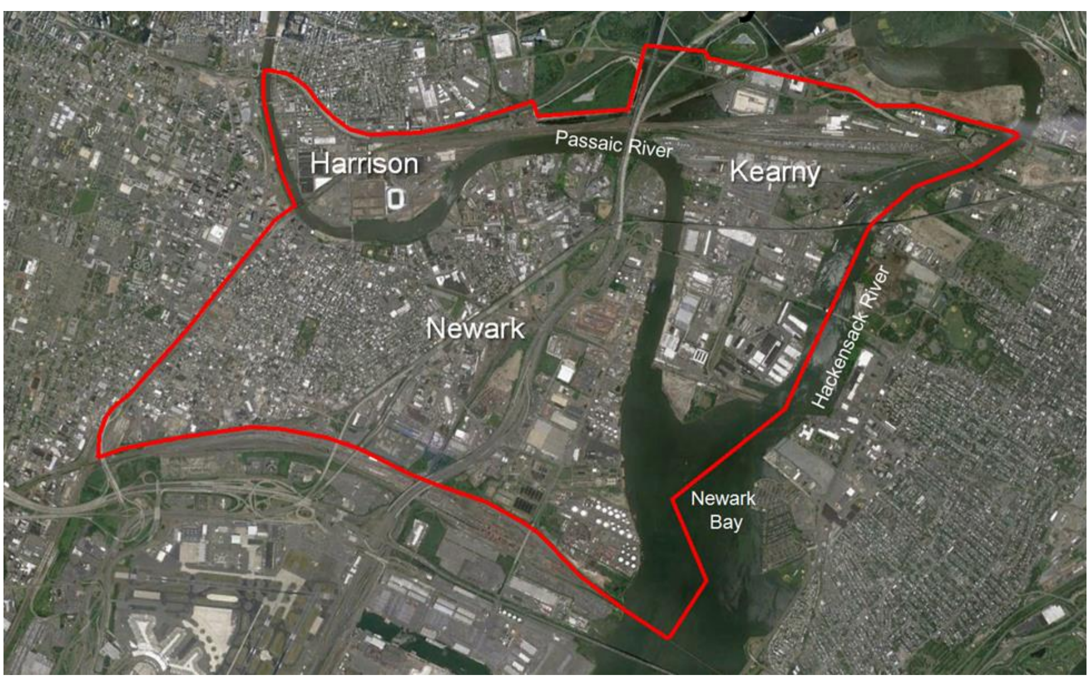

Figure 2: Interior Drainage Study Area

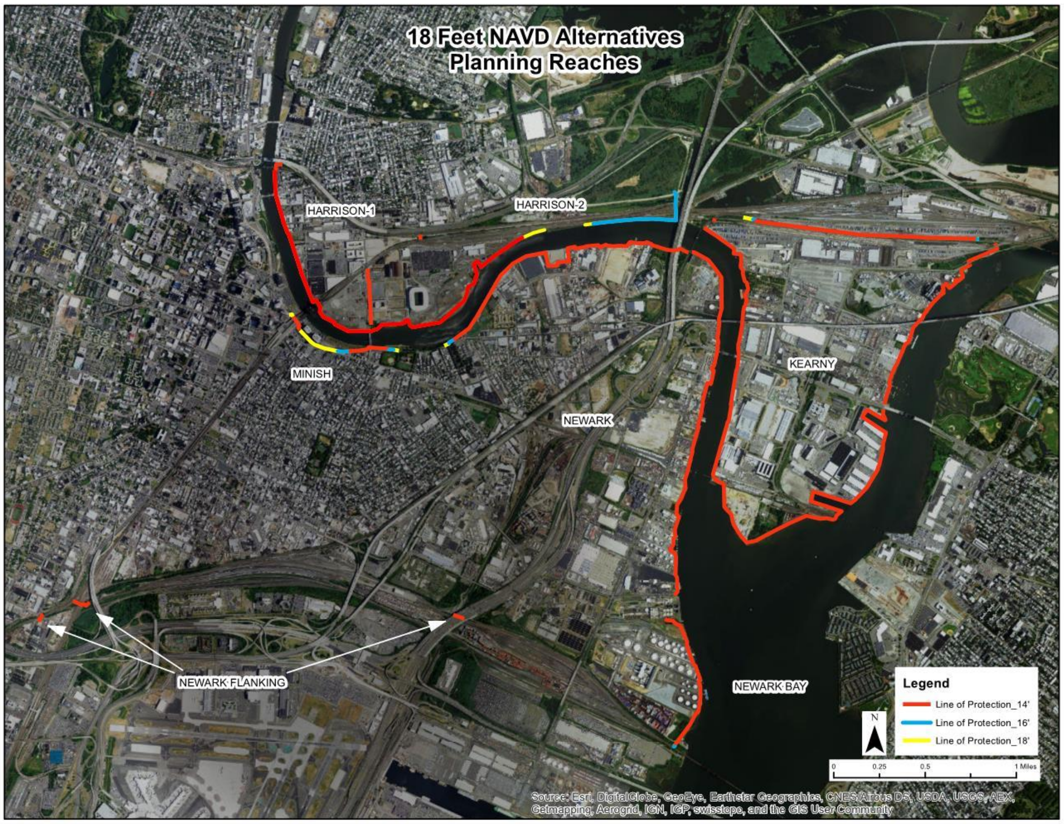

Figure 3: Project Segments

4.3 Interior Drainage Methodology

Areas protected from exterior flood elevations are subject to interior flooding from stormwater runoff. Thus, interior drainage facilities are required to safely store and discharge the runoff to limit interior residual flooding. Typically, the interior areas are studied to determine the specific nature of flooding and to formulate drainage alternatives to maximize National Economic Development (NED) benefits.

In accordance with Engineer Manual (EM) 1110-2-1413, *Hydrologic Analysis of Interior Areas*, the interior drainage facilities are evaluated separately from the alignment. First, a minimum facility plan is identified. The minimum facility plan is considered the smallest plan that can be implemented as part of the alignment that does not result in increased stormwater flooding as a result of project construction. Starting from the minimum facilities analysis, alternatives to improve residual flooding conditions are evaluated to select an optimum plan. The interior drainage analysis for the GRR consisted of recreating the 1995 interior drainage model using the latest version of HEC-HMS in order to establish residual flooding impacts.

4.4 Analysis Approach

Due to the limited correlation between major rainfall/runoff events and tidal flooding events, it is considered most likely that only limited runoff will coincide with severe storm surge and significant storm surge will coincide with only moderately severe rainfall. Historical data indicate that the majority of interior runoff events will coincide with a storm surge level less than or equal to a 2-year storm. Similarly, the majority of significant storm surge events are likely to coincide with runoff equivalent to a 2-year event or less.

Therefore, the analysis was conducted for events with nine recurrence intervals: the 2-, 5-, 10-, 25-, 50-, 100-, 250- and 500- year frequency events (ACE probabilities of 50, 20, 10, 4, 2, 1, 0.4, and 0.2 percent, respectively). In order to develop a stage versus frequency relationship, the interior events were routed against exterior tidal marigrams. For the 'most likely' flooding scenarios, the nine interior storm events were routed against a 2-year exterior tide, and a 2-year interior storm event was routed against the nine exterior events. The highest WSEL of corresponding coincidental frequencies (i.e., 2-year interior and 10-year exterior, or 10-year interior and 2-year exterior) was identified as the most damaging flood level for the coincidental frequency, as shown in **Table 5**.

Table 5: Interior Drainage Analysis Approach

Combination of Interior and Exterior Conditions to be Analyzed											
Interior Flow	Exterior Stage	Time Condition	Peak Int. WSEL	Peak Ext. WSEL	Interior Flow	Exterior Stage	Time	Peak Int. WSEL	Peak Ext. WSEL	Max WS	Risk Condition
2-year	Normal	Current	Model Output		N/a			Model Output		Greatest WSEL for the Frequency Comb.	Lower Bound
5-year	Normal	Current			N/a						Lower Bound
10-year	Normal	Current			N/a						Lower Bound
25-year	Normal	Current			N/a						Lower Bound
50-year	Normal	Current			N/a						Lower Bound
100-year	Normal	Current			N/a						Lower Bound
250-year	Normal	Current			N/a						Lower Bound
500-year	Normal	Current			N/a						Lower Bound
2-year	2-year	Current			2-year	2-year	Current				Most Likely (2-year)
5-year	2-year	Current			2-year	5-year	Current				Most Likely (5-year)
10-year	2-year	Current			2-year	10-year	Current				Most Likely(10-year)
25-year	2-year	Current			2-year	25-year	Current				Most Likely(25-year)
50-year	2-year	Current			2-year	50-year	Current				Most Likely(50-year)
100-year	2-year	Current			2-year	100-year	Current				Most Likely(100-year)
250-year	2-year	Current			2-year	250-year	Current				Most Likely(250-year)
500-year	2-year	Current			2-year	500-year	Current				Most Likely(500-year)
2-year	10-year	Current			10-year	2-year	Current				Upper Bound
5-year	10-year	Current			10-year	5-year	Current				Upper Bound
10-year	10-year	Current			10-year	10-year	Current				Upper Bound
25-year	10-year	Current			10-year	25-year	Current				Upper Bound
50-year	10-year	Current			10-year	50-year	Current				Upper Bound
100-year	10-year	Current			10-year	100-year	Current				Upper Bound
250-year	10-year	Current			10-year	250-year	Current				Upper Bound
500-year	10-year	Current			10-year	500-year	Current				Upper Bound

4.5 Runoff and Surge Coincidence

There is little statistical information to determine where peak storm-related stormwater runoff should occur in relation to an approaching surge. Anecdotal meteorological evidence suggests that the maximum rainfall could be in any of the rain bands of a tropical storm, from out in the leading edge down to the eye wall, or behind the storm. Nor'easters are generally surge events but rainfall could occur and the impact is a function of the duration of the nor'easter. Therefore, in order to present a conservative modelling condition (maximum interior WSELs), the peak stormwater runoff was aligned to be coincidental with the maximum surge for a given annual chance event. This would result in the longest duration of gravity outlets being blocked and typically result in the highest interior WSELs for a particular storm/flood event. A graphic of typical modeled coincidence is shown in **Figure 4**.

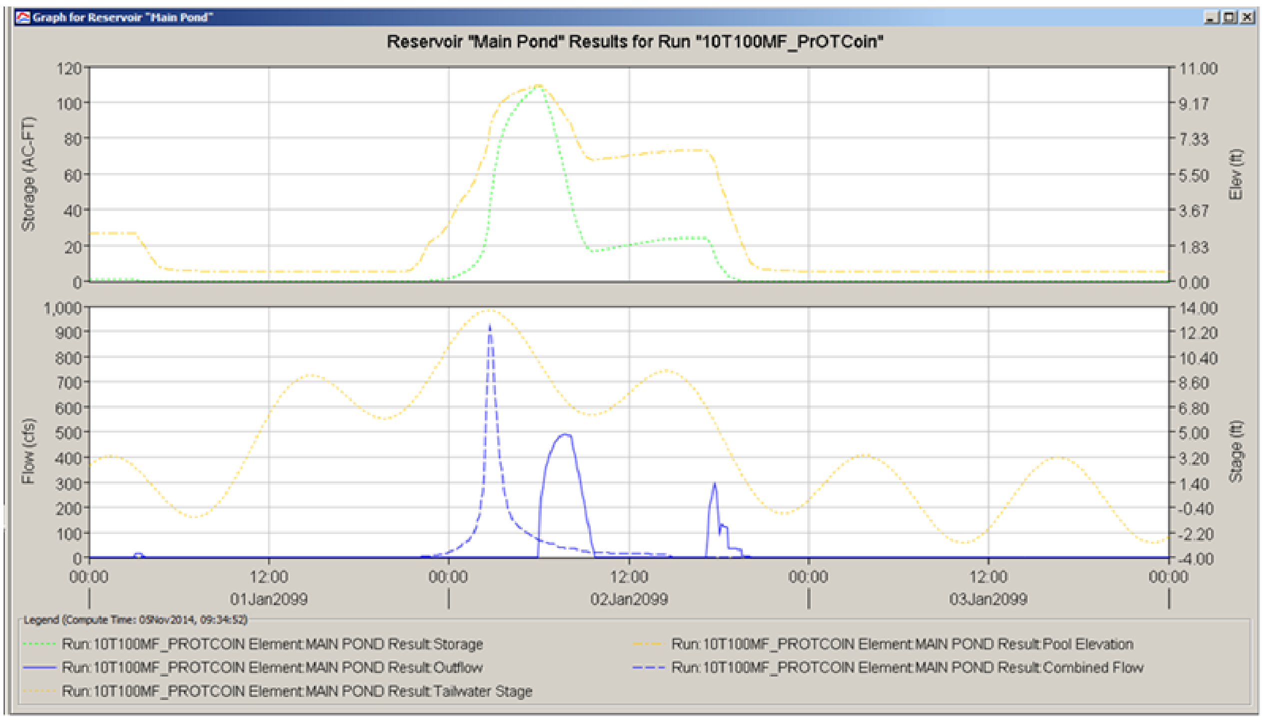

Figure 4: Typical Runoff/Surge Coincidence (HEC-HMS-Output)

4.6 Precipitation

Precipitation data were obtained from New Jersey 24-Hour Rainfall Frequency Data for 2-, 5-, 10-, 25-, 50- and 100-year events and supplemented by NOAA Atlas 14, Volume 2, Version 3, for Newark, New Jersey, US Point Precipitation Frequency for various durations (5, 15, and 60 minutes; and 2, 3, 6, 12, 24, and 48 hours) and the estimated 500-year event. The 250-year event was interpolated from average recurrence interval/precipitation depth chart in NOAA Atlas 14. The rainfall data is shown in **Table 6**.

Table 6: **Rainfall Data**

Duration	Average Recurrence Interval (Years); Depth in Inches								
	1	2	5	10	25	50	100	250*	500
5-min	0.33	0.40	0.47	0.52	0.59	0.64	0.69	0.74	0.79
15-min	0.66	0.79	0.95	1.05	1.18	1.28	1.36	1.47	1.54
60-min	1.12	1.36	1.71	1.96	2.31	2.57	2.84	3.17	3.47
2-hour	1.37	1.67	2.12	2.46	2.94	3.33	3.74	4.26	4.74
3-hour	1.53	1.86	2.36	2.75	3.29	3.73	4.18	4.77	5.32
6-hour	1.96	2.39	3.02	3.53	4.24	4.84	5.47	6.31	7.11
12-hour	2.42	2.93	3.72	4.38	5.33	6.14	7.01	8.20	9.37
24-hour	2.71	3.29	4.20	4.99	6.16	7.18	8.30	9.85	11.40
48-hour	3.17	3.84	4.90	5.79	7.10	8.22	9.45	11.13	12.80

*Values interpolated based on 200-year and 500-year events.

Note: The data was unsmoothed in regard to depth versus duration for each frequency, and depth versus frequency, for each duration.

4.7 Town of Harrison

4.7.1 Interior Drainage Areas

There are three separate interior drainage areas that contribute to ponding behind the Harrison/South First Street Segment alignment, as shown in **Figure 5**:

1) S1: This 0.193-square mile north area drains by one 48-inch primary outlet, five 24-inch secondary outlets, and a 75-cfs pump station.
2) S2: This drainage area of 0.132-square mile drains to the west and is served by one 36 inch primary outlet, four 24-inch secondary outlets, and a 70-cfs pump station.
3) S3: This 0 .061-square mile drainage area discharges through one primary 36-inch outlet, three secondary 24-inch outlets, and a 30-cfs pump station.

The drainage area parameters are shown in **Table 7**. Runoff from these areas is reflected in the HEC-HMS model schematic shown in **Figure 6**. The three ponding areas are shown linked by weirs, which represent high ground between the ponds. Should the water depths in the ponding areas exceed the adjacent weir heights, flow would be diverted to the adjacent pond with a lower elevation.

Table 7: Drainage Area Parameters – Harrison/South First Street

Subarea	Drainage Areas (square miles)	SCS Curve Number	SCS Unitgraph	
			Tc (hour)	Lag (min)
S1	0.1930	83	0.74	26.6
S2	0.1318	83	0.72	25.9
S3	0.0611	83	0.66	23.8
Source: Passaic River Draft GDM, Appendix C H&H, Table C-76				

Figure 5: Harrison/South First Street Drainage Areas

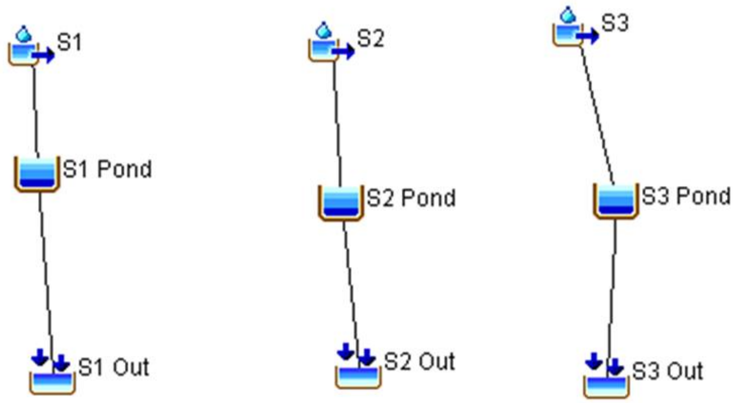

Figure 6: Harrison/South First Street HEC-HMS Model Schematic

Local stormwater drainage and topographic features direct all of the runoff toward the alignment. The following features are incorporated into the interior drainage HEC-HMS model:

1) Interior ponds which consist of the natural storage available in existing ditches and low-lying areas.
2) Gravity outlets through the levee/walls and supplemental pump stations.

4.7.2 Harrison Interior Drainage Plan

The Harrison interior drainage facilities data were verified and adopted from GDM, Appendix C-Hydrology and Hydraulics, and are shown in **Table 8**. The detailed results of the analysis are shown in **Tables 9 through 11**.

Table 8: **Harrison Interior Drainage Features**

Levee/Wall	Status	Type	Length* (feet)	Number	Gravity Size (inches)	Pump
S1	New	Primary	10	1	48	75 cfs
	New	Secondary	10	3	24	
	New	Secondary	10	2	24	
S2	New	Primary	10	1	36	70 cfs
	New	Secondary	10	1	24	
	New	Secondary	10	3	24	
S3	New	Primary	10	1	36	30 cfs
	New	Secondary	10	3	24	

*Through floodwall.

Gravity outlets consisted of extending of the existing storm sewers through levee/walls. In areas drained by established drainage ditches, 48-inch to 72-inch outlets were provided. In those areas where no utility information was available, the outfall structure was assumed to be 36-inch primary outlet and 24-inch secondary outlets spaced 400 feet apart, hydraulically connected – either by pipe or by ditch.

Table 9: Harrison S1 Analysis Results (feet NAVD88)

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	5.43	3.20						**5.43**	**Lower Bound**
5 yr	Normal	Present	5.92	3.20						**5.92**	**Lower Bound**
10 yr	Normal	Present	6.01	3.20						**6.01**	**Lower Bound**
25 yr	Normal	Present	6.12	3.20						**6.12**	**Lower Bound**
50 yr	Normal	Present	6.25	3.20						**6.25**	**Lower Bound**
100 yr	Normal	Present	6.40	3.20						**6.40**	**Lower Bound**
250 yr	Normal	Present	6.60	3.20						**6.60**	**Lower Bound**
500 yr	Normal	Present	6.80	3.20						**6.80**	**Lower Bound**
2 yr	2 yr	Present	5.96	5.71	2 yr	2 yr	Present	5.96	5.71	**5.96**	**Most Likely**
5 yr	2 yr	Present	6.05	5.71	2 yr	5 yr	Present	6.03	7.30	**6.05**	**Most Likely**
10 yr	2 yr	Present	6.15	5.71	2 yr	10 yr	Present	6.11	8.90	**6.15**	**Most Likely**
25 yr	2 yr	Present	6.30	5.71	2 yr	25 yr	Present	6.13	10.80	**6.30**	**Most Likely**
50 yr	2 yr	Present	6.44	5.71	2 yr	50 yr	Present	6.13	12.30	**6.44**	**Most Likely**
100 yr	2 yr	Present	6.60	5.71	2 yr	100 yr	Present	6.13	13.70	**6.60**	**Most Likely**
250 yr	2 yr	Present	6.80	5.71	2 yr	250 yr	Present	6.13	16.40	**6.80**	**Most Likely**
500 yr	2 yr	Present	7.00	5.71	2 yr	500 yr	Present	6.13	17.30	**7.00**	**Most Likely**
2 yr	10 yr	Present	6.11	8.90	10 yr	2 yr	Present	6.15	5.71	**6.15**	**Upper Bound**
5 yr	10 yr	Present	6.36	8.90	10 yr	5 yr	Present	6.32	7.30	**6.36**	**Upper Bound**
10 yr	10 yr	Present	6.58	8.90	10 yr	10 yr	Present	6.58	8.90	**6.58**	**Upper Bound**
25 yr	10 yr	Present	6.91	8.90	10 yr	25 yr	Present	6.59	10.80	**6.91**	**Upper Bound**
50 yr	10 yr	Present	7.20	8.90	10 yr	50 yr	Present	6.59	12.30	**7.20**	**Upper Bound**
100 yr	10 yr	Present	7.50	8.90	10 yr	100 yr	Present	6.59	13.70	**7.50**	**Upper Bound**
250 yr	10 yr	Present	7.84	8.90	10 yr	250 yr	Present	6.59	16.40	**7.84**	**Upper Bound**
500 yr	10 yr	Present	8.17	8.90	10 yr	500 yr	Present	6.59	17.30	**8.17**	**Upper Bound**

Table 10: Harrison S2 Analysis Results (feet NAVD88)

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	5.39	3.20						5.39	Lower Bound
5 yr	Normal	Present	5.39	3.20						5.39	Lower Bound
10 yr	Normal	Present	5.51	3.20						5.51	Lower Bound
25 yr	Normal	Present	5.99	3.20						5.99	Lower Bound
50 yr	Normal	Present	6.05	3.20						6.05	Lower Bound
100 yr	Normal	Present	6.13	3.20						6.13	Lower Bound
250 yr	Normal	Present	6.24	3.20						6.24	Lower Bound
500 yr	Normal	Present	6.36	3.20						6.36	Lower Bound
2 yr	2 yr	Present	5.76	5.71	2 yr	2 yr	Present	5.76	5.71	5.76	Most Likely
5 yr	2 yr	Present	6.00	5.71	2 yr	5 yr	Present	5.96	7.30	6.00	Most Likely
10 yr	2 yr	Present	6.05	5.71	2 yr	10 yr	Present	6.02	8.90	6.05	Most Likely
25 yr	2 yr	Present	6.15	5.71	2 yr	25 yr	Present	6.06	10.80	6.15	Most Likely
50 yr	2 yr	Present	6.24	5.71	2 yr	50 yr	Present	6.06	12.30	6.24	Most Likely
100 yr	2 yr	Present	6.34	5.71	2 yr	100 yr	Present	6.06	13.70	6.34	Most Likely
250 yr	2 yr	Present	6.48	5.71	2 yr	250 yr	Present	6.06	16.40	6.48	Most Likely
500 yr	2 yr	Present	6.62	5.71	2 yr	500 yr	Present	6.06	17.30	6.62	Most Likely
2 yr	10 yr	Present	6.02	8.90	10 yr	2 yr	Present	6.05	5.71	6.05	Upper Bound
5 yr	10 yr	Present	6.19	8.90	10 yr	5 yr	Present	6.10	7.30	6.19	Upper Bound
10 yr	10 yr	Present	6.31	8.90	10 yr	10 yr	Present	6.31	8.90	6.31	Upper Bound
25 yr	10 yr	Present	6.50	8.90	10 yr	25 yr	Present	6.31	10.80	6.50	Upper Bound
50 yr	10 yr	Present	6.70	8.90	10 yr	50 yr	Present	6.31	12.30	6.70	Upper Bound
100 yr	10 yr	Present	6.92	8.90	10 yr	100 yr	Present	6.31	13.70	6.92	Upper Bound
250 yr	10 yr	Present	7.15	8.90	10 yr	250 yr	Present	6.31	16.40	7.15	Upper Bound
500 yr	10 yr	Present	7.48	8.90	10 yr	500 yr	Present	6.31	17.30	7.48	Upper Bound

Table 11: **Harrison S3 Analysis Results (feet NAVD88)**

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	4.19	3.20						4.19	Lower Bound
5 yr	Normal	Present	4.48	3.20						4.48	Lower Bound
10 yr	Normal	Present	4.70	3.20						4.70	Lower Bound
25 yr	Normal	Present	5.01	3.20						5.01	Lower Bound
50 yr	Normal	Present	5.02	3.20						5.02	Lower Bound
100 yr	Normal	Present	5.18	3.20						5.18	Lower Bound
250 yr	Normal	Present	5.44	3.20						5.44	Lower Bound
500 yr	Normal	Present	5.68	3.20						5.68	Lower Bound
2 yr	2 yr	Present	5.31	5.71	2 yr	2 yr	Present	5.31	5.71	5.31	Most Likely
5 yr	2 yr	Present	5.77	5.71	2 yr	5 yr	Present	5.26	7.30	5.77	Most Likely
10 yr	2 yr	Present	5.90	5.71	2 yr	10 yr	Present	5.31	8.90	5.90	Most Likely
25 yr	2 yr	Present	6.02	5.71	2 yr	25 yr	Present	5.36	10.80	6.02	Most Likely
50 yr	2 yr	Present	6.06	5.71	2 yr	50 yr	Present	5.36	12.30	6.06	Most Likely
100 yr	2 yr	Present	6.11	5.71	2 yr	100 yr	Present	5.81	13.70	6.11	Most Likely
250 yr	2 yr	Present	6.18	5.71	2 yr	250 yr	Present	5.82	16.40	6.18	Most Likely
500 yr	2 yr	Present	6.25	5.71	2 yr	500 yr	Present	5.82	17.30	6.25	Most Likely
2 yr	10 yr	Present	5.31	8.90	10 yr	2 yr	Present	5.90	5.71	5.90	Upper Bound
5 yr	10 yr	Present	6.00	8.90	10 yr	5 yr	Present	6.03	7.30	6.03	Upper Bound
10 yr	10 yr	Present	6.08	8.90	10 yr	10 yr	Present	6.08	8.90	6.08	Upper Bound
25 yr	10 yr	Present	6.20	8.90	10 yr	25 yr	Present	6.08	10.80	6.20	Upper Bound
50 yr	10 yr	Present	6.30	8.90	10 yr	50 yr	Present	6.08	12.30	6.30	Upper Bound
100 yr	10 yr	Present	6.41	8.90	10 yr	100 yr	Present	6.08	13.70	6.41	Upper Bound
250 yr	10 yr	Present	6.55	8.90	10 yr	250 yr	Present	6.08	16.40	6.55	Upper Bound
500 yr	10 yr	Present	6.72	8.90	10 yr	500 yr	Present	6.08	17.30	6.72	Upper Bound

4.8 City of Newark

4.8.1 Interior Drainage Areas

There are two distinguished interior drainage areas that contribute to ponding behind the City of Newark Segment alignment, as shown in **Figure 7**. These contribute to ten ponding areas:

1) Northern Area at Lister Avenue and the New Jersey Turnpike includes: L1, L2, L3 and T drainage and ponding areas,
2) Eastern Area at Doremus Avenue and Doremus Avenue Extension includes: D1, D2, D3A, D3B, D4 and D5 drainage and ponding areas.

The drainage area parameters are shown in **Table 12**. Some of the ponding areas are linked by an area of high ground (D1-D2, D3B-D4 and L3-T), which is modeled as a diversion weir in the HEC-HMS model. Runoff from these areas and the interior ponding areas are reflected in the HEC-HMS model schematics as shown in **Figures 8 and 9**.

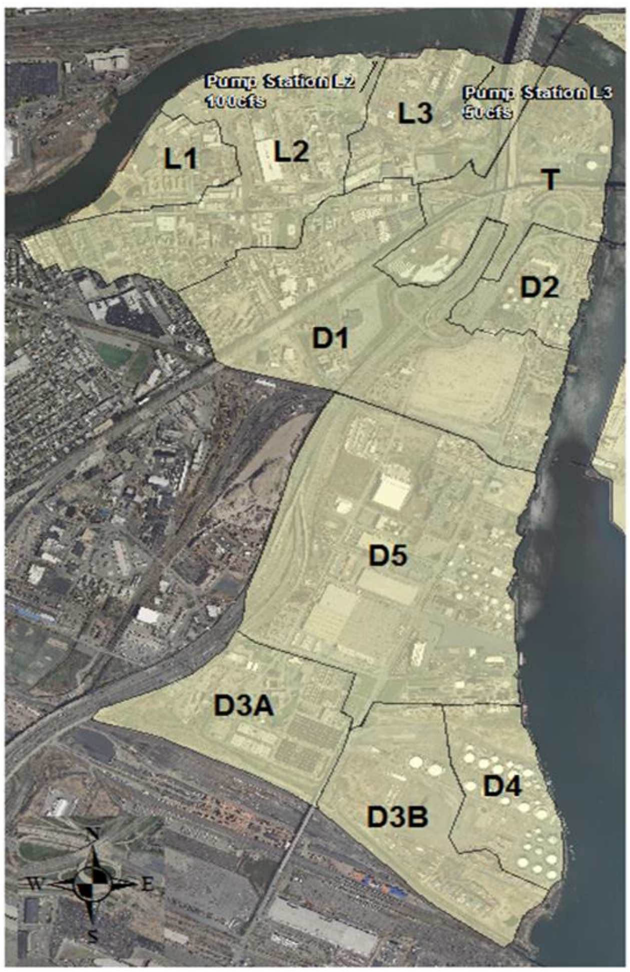

Figure 7: Newark Drainage Areas

Table 12: Drainage Area Parameters – Newark

Subarea	Drainage Areas (square miles)	SCS Curve Number	SCS Unitgraph	
			Tc (hour)	Lag (min)
Lister Avenue				
L1	0.0757	83	0.64	23.0
L2	0.3127	83	0.76	27.4
L3	0.1589	85	0.72	25.9
Turnpike				
T	0.2025	78	0.71	25.6
Doremus Avenue				
D1	0.4572	83	0.75	27.0
D2	0.0879	80	0.69	24.8
Doremus Extension				
D3A	0.1907	87	0.79	28.4
D3B	0.2092	78	0.75	27.0
D4	0.1272	80	0.70	25.2
D5	0.5545	81	0.81	29.2
Source: Passaic River Draft GDM, Appendix C H&H, Table C-76				

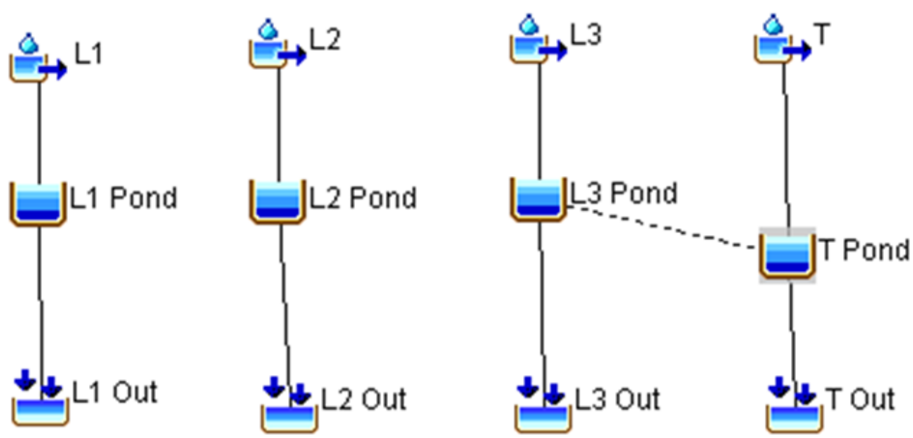

Figure 8: Lister Avenue and Turnpike HEC-HMS Model Schematic

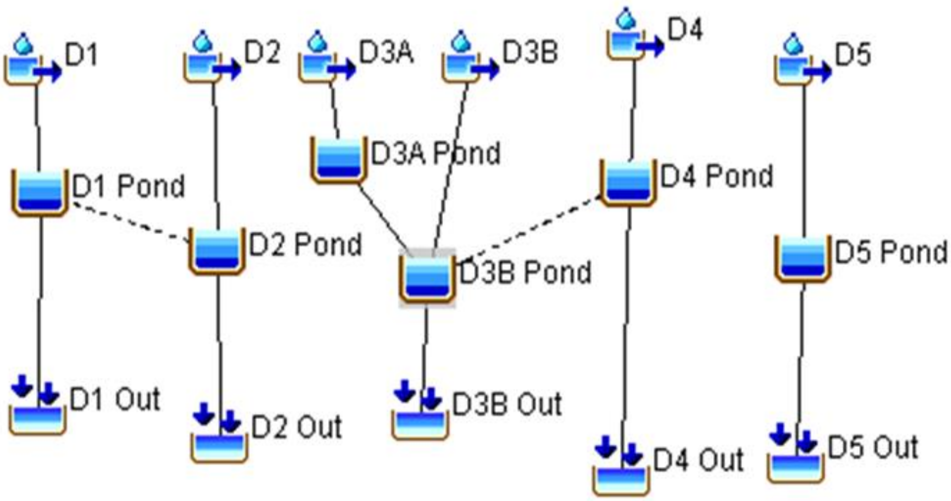

Figure 9: Doremus Avenue and Doremus Extension HEC-HMS Model Schematic

4.8.2 Newark Interior Drainage Plan

The interior drainage features for Newark were determined in GDM for each of the ponding areas and are shown in **Table 13**. The results of the analysis are shown in **Tables 14 through 23**. These features are independent of any culverts that may be required for wetlands flushing.

Table 13: Newark Interior Drainage Features

Levee/Wall	Status	Type	Length (feet)	Number	Size (inches)	Pump
Lister Avenue						
L1	New	Primary	10	1	36	
	New	Secondary	10	4	24	
	New	Secondary	10	1	24	
L2	Existing	Primary	10	2	72	100cfs
	New	Secondary	10	2	24	
	New	Secondary	10	5	24	
L3	New	Primary	10	1	48	50 cfs
	New	Secondary	10	5	24	
	New	Secondary	10	1	24	
	Existing	Secondary	10	1	24	

Table 13 (cont.): Newark Interior Drainage Features

Levee/Wall	Status	Type	Length (feet)	Number	Size (inches)	Pump
Turnpike						
T	New	Primary	10	1	48	
	New	Secondary	10	9	24	
Levee/Wall	**Status**	**Type**	**Length (feet)**	**Number**	**Size (inches)**	**Pump**
Doremus Avenue						
D1	Existing	Primary	10	1	60	
	New	Secondary	10	5	24	
D2	New	Primary	10	1	48	
	New	Secondary	10	1	24	
	New	Secondary	10	3	24	
Doremus Extension						
D3A	Existing	Primary	10	1	3x2 feet	
D3B	New	Primary	10	2	60	
	New	Secondary	10	1	36	
	New	Secondary	10	1	24	
D4	New	Primary	10	2	36	
	New	Secondary	10	7	24	
D5	New	Primary	10	1	36	
	New	Secondary	10	8	24	

Table 14: **Newark L1 Results (feet NAVD88)**

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	3.77	3.20						3.77	Lower Bound
5 yr	Normal	Present	4.08	3.20						4.08	Lower Bound
10 yr	Normal	Present	4.32	3.20						4.32	Lower Bound
25 yr	Normal	Present	4.66	3.20						4.66	Lower Bound
50 yr	Normal	Present	4.93	3.20						4.93	Lower Bound
100 yr	Normal	Present	5.25	3.20						5.25	Lower Bound
250 yr	Normal	Present	5.68	3.20						5.68	Lower Bound
500 yr	Normal	Present	6.01	3.20						6.01	Lower Bound
2 yr	2 yr	Present	5.98	5.71	2 yr	2 yr	Present	5.98	5.71	5.98	Most Likely
5 yr	2 yr	Present	6.07	5.71	2 yr	5 yr	Present	6.52	7.30	6.52	Most Likely
10 yr	2 yr	Present	6.15	5.71	2 yr	10 yr	Present	7.01	8.90	7.01	Most Likely
25 yr	2 yr	Present	6.28	5.71	2 yr	25 yr	Present	7.12	10.80	7.12	Most Likely
50 yr	2 yr	Present	6.39	5.71	2 yr	50 yr	Present	7.17	12.30	7.17	Most Likely
100 yr	2 yr	Present	6.50	5.71	2 yr	100 yr	Present	7.19	13.70	7.19	Most Likely
250 yr	2 yr	Present	6.66	5.71	2 yr	250 yr	Present	7.36	16.40	7.36	Most Likely
500 yr	2 yr	Present	6.80	5.71	2 yr	500 yr	Present	7.36	17.30	7.36	Most Likely
2 yr	10 yr	Present	7.01	8.90	10 yr	2 yr	Present	6.15	5.71	7.01	Upper Bound
5 yr	10 yr	Present	7.48	8.90	10 yr	5 yr	Present	6.88	7.30	7.48	Upper Bound
10 yr	10 yr	Present	7.85	8.90	10 yr	10 yr	Present	7.85	8.90	7.85	Upper Bound
25 yr	10 yr	Present	8.08	8.90	10 yr	25 yr	Present	8.02	10.80	8.08	Upper Bound
50 yr	10 yr	Present	8.18	8.90	10 yr	50 yr	Present	8.07	12.30	8.18	Upper Bound
100 yr	10 yr	Present	8.26	8.90	10 yr	100 yr	Present	8.08	13.70	8.26	Upper Bound
250 yr	10 yr	Present	8.33	8.90	10 yr	250 yr	Present	8.16	16.40	8.33	Upper Bound
500 yr	10 yr	Present	8.43	8.90	10 yr	500 yr	Present	8.16	17.30	8.43	Upper Bound

Table 15: **Newark L2 Results (feet NAVD88)**

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	4.03	3.20						4.03	Lower Bound
5 yr	Normal	Present	4.24	3.20						4.24	Lower Bound
10 yr	Normal	Present	4.43	3.20						4.43	Lower Bound
25 yr	Normal	Present	4.73	3.20						4.73	Lower Bound
50 yr	Normal	Present	4.97	3.20						4.97	Lower Bound
100 yr	Normal	Present	5.10	3.20						5.10	Lower Bound
250 yr	Normal	Present	5.34	3.20						5.34	Lower Bound
500 yr	Normal	Present	5.61	3.20						5.61	Lower Bound
2 yr	2 yr	Present	5.11	5.71	2 yr	2 yr	Present	5.11	5.71	5.11	Most Likely
5 yr	2 yr	Present	5.55	5.71	2 yr	5 yr	Present	5.11	7.30	5.55	Most Likely
10 yr	2 yr	Present	5.75	5.71	2 yr	10 yr	Present	5.11	8.90	5.75	Most Likely
25 yr	2 yr	Present	5.95	5.71	2 yr	25 yr	Present	5.14	10.80	5.95	Most Likely
50 yr	2 yr	Present	6.04	5.71	2 yr	50 yr	Present	5.15	12.30	6.04	Most Likely
100 yr	2 yr	Present	6.10	5.71	2 yr	100 yr	Present	5.15	13.70	6.10	Most Likely
250 yr	2 yr	Present	6.19	5.71	2 yr	250 yr	Present	5.15	16.40	6.19	Most Likely
500 yr	2 yr	Present	6.28	5.71	2 yr	500 yr	Present	5.15	17.30	6.28	Most Likely
2 yr	10 yr	Present	5.11	8.90	10 yr	2 yr	Present	5.75	5.71	5.75	Upper Bound
5 yr	10 yr	Present	5.55	8.90	10 yr	5 yr	Present	5.86	7.30	5.86	Upper Bound
10 yr	10 yr	Present	6.00	8.90	10 yr	10 yr	Present	6.00	8.90	6.00	Upper Bound
25 yr	10 yr	Present	6.18	8.90	10 yr	25 yr	Present	6.04	10.80	6.18	Upper Bound
50 yr	10 yr	Present	6.33	8.90	10 yr	50 yr	Present	6.06	12.30	6.33	Upper Bound
100 yr	10 yr	Present	6.50	8.90	10 yr	100 yr	Present	6.06	13.70	6.50	Upper Bound
250 yr	10 yr	Present	6.70	8.90	10 yr	250 yr	Present	6.06	16.40	6.70	Upper Bound
500 yr	10 yr	Present	7.02	8.90	10 yr	500 yr	Present	6.06	17.30	7.02	Upper Bound

Table 16: **Newark L3 Results (feet NAVD88)**

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	3.76	3.20						3.76	Lower Bound
5 yr	Normal	Present	4.03	3.20						4.03	Lower Bound
10 yr	Normal	Present	4.12	3.20						4.12	Lower Bound
25 yr	Normal	Present	4.26	3.20						4.26	Lower Bound
50 yr	Normal	Present	4.39	3.20						4.39	Lower Bound
100 yr	Normal	Present	4.53	3.20						4.53	Lower Bound
250 yr	Normal	Present	4.73	3.20						4.73	Lower Bound
500 yr	Normal	Present	4.94	3.20						4.94	Lower Bound
2 yr	2 yr	Present	4.99	5.71	2 yr	2 yr	Present	4.99	5.71	4.99	Most Likely
5 yr	2 yr	Present	5.10	5.71	2 yr	5 yr	Present	5.01	7.30	5.10	Most Likely
10 yr	2 yr	Present	5.34	5.71	2 yr	10 yr	Present	5.01	8.90	5.34	Most Likely
25 yr	2 yr	Present	5.65	5.71	2 yr	25 yr	Present	5.01	10.80	5.65	Most Likely
50 yr	2 yr	Present	5.82	5.71	2 yr	50 yr	Present	5.01	12.30	5.82	Most Likely
100 yr	2 yr	Present	5.99	5.71	2 yr	100 yr	Present	5.01	13.70	5.99	Most Likely
250 yr	2 yr	Present	6.07	5.71	2 yr	250 yr	Present	5.01	16.40	6.07	Most Likely
500 yr	2 yr	Present	6.18	5.71	2 yr	500 yr	Present	5.01	17.30	6.18	Most Likely
2 yr	10 yr	Present	5.00	8.90	10 yr	2 yr	Present	5.34	5.71	5.34	Upper Bound
5 yr	10 yr	Present	5.10	8.90	10 yr	5 yr	Present	5.34	7.30	5.34	Upper Bound
10 yr	10 yr	Present	5.35	8.90	10 yr	10 yr	Present	5.35	8.90	5.35	Upper Bound
25 yr	10 yr	Present	5.82	8.90	10 yr	25 yr	Present	5.44	10.80	5.82	Upper Bound
50 yr	10 yr	Present	6.10	8.90	10 yr	50 yr	Present	5.55	12.30	6.10	Upper Bound
100 yr	10 yr	Present	6.32	8.90	10 yr	100 yr	Present	5.55	13.70	6.32	Upper Bound
250 yr	10 yr	Present	6.58	8.90	10 yr	250 yr	Present	5.55	16.40	6.58	Upper Bound
500 yr	10 yr	Present	6.92	8.90	10 yr	500 yr	Present	5.55	17.30	6.92	Upper Bound

Table 17: Newark T Results (feet NAVD88)

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	4.27	3.20						4.27	**Lower Bound**
5 yr	Normal	Present	4.51	3.20						4.51	**Lower Bound**
10 yr	Normal	Present	4.71	3.20						4.71	**Lower Bound**
25 yr	Normal	Present	4.99	3.20						4.99	**Lower Bound**
50 yr	Normal	Present	5.22	3.20						5.22	**Lower Bound**
100 yr	Normal	Present	5.46	3.20						5.46	**Lower Bound**
250 yr	Normal	Present	5.75	3.20						5.75	**Lower Bound**
500 yr	Normal	Present	6.02	3.20						6.02	**Lower Bound**
2 yr	2 yr	Present	5.08	5.71	2 yr	2 yr	Present	5.08	5.71	5.08	**Most Likely**
5 yr	2 yr	Present	5.49	5.71	2 yr	5 yr	Present	5.35	7.30	5.49	**Most Likely**
10 yr	2 yr	Present	5.71	5.71	2 yr	10 yr	Present	5.42	8.90	5.71	**Most Likely**
25 yr	2 yr	Present	5.96	5.71	2 yr	25 yr	Present	5.46	10.80	5.96	**Most Likely**
50 yr	2 yr	Present	6.05	5.71	2 yr	50 yr	Present	5.46	12.30	6.05	**Most Likely**
100 yr	2 yr	Present	6.14	5.71	2 yr	100 yr	Present	5.46	13.70	6.14	**Most Likely**
250 yr	2 yr	Present	6.24	5.71	2 yr	250 yr	Present	5.46	16.40	6.24	**Most Likely**
500 yr	2 yr	Present	6.36	5.71	2 yr	500 yr	Present	5.46	17.30	6.36	**Most Likely**
2 yr	10 yr	Present	5.42	8.90	10 yr	2 yr	Present	5.71	5.71	5.71	**Upper Bound**
5 yr	10 yr	Present	5.86	8.90	10 yr	5 yr	Present	6.04	7.30	6.04	**Upper Bound**
10 yr	10 yr	Present	6.06	8.90	10 yr	10 yr	Present	6.06	8.90	6.06	**Upper Bound**
25 yr	10 yr	Present	6.22	8.90	10 yr	25 yr	Present	6.08	10.80	6.22	**Upper Bound**
50 yr	10 yr	Present	6.36	8.90	10 yr	50 yr	Present	6.09	12.30	6.36	**Upper Bound**
100 yr	10 yr	Present	6.50	8.90	10 yr	100 yr	Present	6.09	13.70	6.50	**Upper Bound**
250 yr	10 yr	Present	6.67	8.90	10 yr	250 yr	Present	6.09	16.40	6.67	**Upper Bound**
500 yr	10 yr	Present	6.89	8.90	10 yr	500 yr	Present	6.09	17.30	6.89	**Upper Bound**

Table 18: Newark D1 Results (feet NAVD88)

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	5.45	3.20						5.45	Lower Bound
5 yr	Normal	Present	6.03	3.20						6.03	Lower Bound
10 yr	Normal	Present	6.17	3.20						6.17	Lower Bound
25 yr	Normal	Present	6.37	3.20						6.37	Lower Bound
50 yr	Normal	Present	6.55	3.20						6.55	Lower Bound
100 yr	Normal	Present	6.74	3.20						6.74	Lower Bound
250 yr	Normal	Present	6.97	3.20						6.97	Lower Bound
500 yr	Normal	Present	7.26	3.20						7.26	Lower Bound
2 yr	2 yr	Present	6.04	5.71	2 yr	2 yr	Present	6.04	5.71	6.04	Most Likely
5 yr	2 yr	Present	6.20	5.71	2 yr	5 yr	Present	6.32	7.30	6.32	Most Likely
10 yr	2 yr	Present	6.34	5.71	2 yr	10 yr	Present	6.38	8.90	6.38	Most Likely
25 yr	2 yr	Present	6.55	5.71	2 yr	25 yr	Present	6.44	10.80	6.55	Most Likely
50 yr	2 yr	Present	6.72	5.71	2 yr	50 yr	Present	6.47	12.30	6.72	Most Likely
100 yr	2 yr	Present	6.89	5.71	2 yr	100 yr	Present	6.49	13.70	6.89	Most Likely
250 yr	2 yr	Present	7.08	5.71	2 yr	250 yr	Present	6.58	16.40	7.08	Most Likely
500 yr	2 yr	Present	7.35	5.71	2 yr	500 yr	Present	6.58	17.30	7.35	Most Likely
2 yr	10 yr	Present	6.38	8.90	10 yr	2 yr	Present	6.34	5.71	6.38	Upper Bound
5 yr	10 yr	Present	6.68	8.90	10 yr	5 yr	Present	6.71	7.30	6.71	Upper Bound
10 yr	10 yr	Present	6.92	8.90	10 yr	10 yr	Present	6.92	8.90	6.92	Upper Bound
25 yr	10 yr	Present	7.27	8.90	10 yr	25 yr	Present	7.04	10.80	7.27	Upper Bound
50 yr	10 yr	Present	7.56	8.90	10 yr	50 yr	Present	7.14	12.30	7.56	Upper Bound
100 yr	10 yr	Present	7.75	8.90	10 yr	100 yr	Present	7.16	13.70	7.75	Upper Bound
250 yr	10 yr	Present	7.89	8.90	10 yr	250 yr	Present	7.34	16.40	7.89	Upper Bound
500 yr	10 yr	Present	8.01	8.90	10 yr	500 yr	Present	7.34	17.30	8.01	Upper Bound

Table 19: Newark D2 Results (feet NAVD88)

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	4.05	3.20						4.05	**Lower Bound**
5 yr	Normal	Present	4.16	3.20						4.16	**Lower Bound**
10 yr	Normal	Present	4.26	3.20						4.26	**Lower Bound**
25 yr	Normal	Present	4.41	3.20						4.41	**Lower Bound**
50 yr	Normal	Present	4.53	3.20						4.53	**Lower Bound**
100 yr	Normal	Present	4.66	3.20						4.66	**Lower Bound**
250 yr	Normal	Present	4.83	3.20						4.83	**Lower Bound**
500 yr	Normal	Present	5.00	3.20						5.00	**Lower Bound**
2 yr	2 yr	Present	4.62	5.71	2 yr	2 yr	Present	4.62	5.71	4.62	**Most Likely**
5 yr	2 yr	Present	4.90	5.71	2 yr	5 yr	Present	4.75	7.30	4.90	**Most Likely**
10 yr	2 yr	Present	5.11	5.71	2 yr	10 yr	Present	4.82	8.90	5.11	**Most Likely**
25 yr	2 yr	Present	5.38	5.71	2 yr	25 yr	Present	4.86	10.80	5.38	**Most Likely**
50 yr	2 yr	Present	5.56	5.71	2 yr	50 yr	Present	4.98	12.30	5.56	**Most Likely**
100 yr	2 yr	Present	5.71	5.71	2 yr	100 yr	Present	4.99	13.70	5.71	**Most Likely**
250 yr	2 yr	Present	5.86	5.71	2 yr	250 yr	Present	4.99	16.40	5.86	**Most Likely**
500 yr	2 yr	Present	6.01	5.71	2 yr	500 yr	Present	4.99	17.30	6.01	**Most Likely**
2 yr	10 yr	Present	4.82	8.90	10 yr	2 yr	Present	5.11	5.71	5.11	**Upper Bound**
5 yr	10 yr	Present	5.18	8.90	10 yr	5 yr	Present	5.35	7.30	5.35	**Upper Bound**
10 yr	10 yr	Present	5.49	8.90	10 yr	10 yr	Present	5.49	8.90	5.49	**Upper Bound**
25 yr	10 yr	Present	5.91	8.90	10 yr	25 yr	Present	5.60	10.80	5.91	**Upper Bound**
50 yr	10 yr	Present	6.10	8.90	10 yr	50 yr	Present	5.68	12.30	6.10	**Upper Bound**
100 yr	10 yr	Present	6.56	8.90	10 yr	100 yr	Present	5.88	13.70	6.56	**Upper Bound**
250 yr	10 yr	Present	7.23	8.90	10 yr	250 yr	Present	5.89	16.40	7.23	**Upper Bound**
500 yr	10 yr	Present	8.02	8.90	10 yr	500 yr	Present	5.89	17.30	8.02	**Upper Bound**

Table 20: **Newark D3A Results (feet NAVD88)**

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	4.32	3.20						4.32	Lower Bound
5 yr	Normal	Present	4.51	3.20						4.51	Lower Bound
10 yr	Normal	Present	4.67	3.20						4.67	Lower Bound
25 yr	Normal	Present	4.91	3.20						4.91	Lower Bound
50 yr	Normal	Present	5.12	3.20						5.12	Lower Bound
100 yr	Normal	Present	5.35	3.20						5.35	Lower Bound
250 yr	Normal	Present	5.68	3.20						5.68	Lower Bound
500 yr	Normal	Present	6.04	3.20						6.04	Lower Bound
2 yr	2 yr	Present	4.32	5.71	2 yr	2 yr	Present	4.32	5.71	4.32	Most Likely
5 yr	2 yr	Present	4.51	5.71	2 yr	5 yr	Present	4.32	7.30	4.51	Most Likely
10 yr	2 yr	Present	4.67	5.71	2 yr	10 yr	Present	4.32	8.90	4.67	Most Likely
25 yr	2 yr	Present	4.91	5.71	2 yr	25 yr	Present	4.32	10.80	4.91	Most Likely
50 yr	2 yr	Present	5.12	5.71	2 yr	50 yr	Present	4.32	12.30	5.12	Most Likely
100 yr	2 yr	Present	5.35	5.71	2 yr	100 yr	Present	4.32	13.70	5.35	Most Likely
250 yr	2 yr	Present	5.68	5.71	2 yr	250 yr	Present	4.32	16.40	5.68	Most Likely
500 yr	2 yr	Present	6.04	5.71	2 yr	500 yr	Present	4.32	17.30	6.04	Most Likely
2 yr	10 yr	Present	4.32	8.90	10 yr	2 yr	Present	4.67	5.71	4.67	Upper Bound
5 yr	10 yr	Present	4.51	8.90	10 yr	5 yr	Present	4.67	7.30	4.67	Upper Bound
10 yr	10 yr	Present	4.67	8.90	10 yr	10 yr	Present	4.67	8.90	4.67	Upper Bound
25 yr	10 yr	Present	4.91	8.90	10 yr	25 yr	Present	4.67	10.80	4.91	Upper Bound
50 yr	10 yr	Present	5.12	8.90	10 yr	50 yr	Present	4.67	12.30	5.12	Upper Bound
100 yr	10 yr	Present	5.35	8.90	10 yr	100 yr	Present	4.67	13.70	5.35	Upper Bound
250 yr	10 yr	Present	5.68	8.90	10 yr	250 yr	Present	4.67	16.40	5.68	Upper Bound
500 yr	10 yr	Present	6.04	8.90	10 yr	500 yr	Present	4.67	17.30	6.04	Upper Bound

Table 21: **Newark D3B Results (feet NAVD88)**

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	4.28	3.20						4.28	Lower Bound
5 yr	Normal	Present	4.62	3.20						4.62	Lower Bound
10 yr	Normal	Present	4.90	3.20						4.90	Lower Bound
25 yr	Normal	Present	5.30	3.20						5.30	Lower Bound
50 yr	Normal	Present	5.63	3.20						5.63	Lower Bound
100 yr	Normal	Present	5.98	3.20						5.98	Lower Bound
250 yr	Normal	Present	6.12	3.20						6.12	Lower Bound
500 yr	Normal	Present	6.28	3.20						6.28	Lower Bound
2 yr	2 yr	Present	5.70	5.71	2 yr	2 yr	Present	5.70	5.71	5.70	Most Likely
5 yr	2 yr	Present	5.91	5.71	2 yr	5 yr	Present	6.26	7.30	6.26	Most Likely
10 yr	2 yr	Present	6.03	5.71	2 yr	10 yr	Present	6.41	8.90	6.41	Most Likely
25 yr	2 yr	Present	6.14	5.71	2 yr	25 yr	Present	6.53	10.80	6.53	Most Likely
50 yr	2 yr	Present	6.24	5.71	2 yr	50 yr	Present	6.62	12.30	6.62	Most Likely
100 yr	2 yr	Present	6.34	5.71	2 yr	100 yr	Present	6.69	13.70	6.69	Most Likely
250 yr	2 yr	Present	6.47	5.71	2 yr	250 yr	Present	7.11	16.40	7.11	Most Likely
500 yr	2 yr	Present	6.61	5.71	2 yr	500 yr	Present	7.11	17.30	7.11	Most Likely
2 yr	10 yr	Present	6.41	8.90	10 yr	2 yr	Present	6.03	5.71	6.41	Upper Bound
5 yr	10 yr	Present	6.70	8.90	10 yr	5 yr	Present	6.70	7.30	6.70	Upper Bound
10 yr	10 yr	Present	6.94	8.90	10 yr	10 yr	Present	6.94	8.90	6.94	Upper Bound
25 yr	10 yr	Present	7.29	8.90	10 yr	25 yr	Present	7.16	10.80	7.29	Upper Bound
50 yr	10 yr	Present	7.57	8.90	10 yr	50 yr	Present	7.36	12.30	7.57	Upper Bound
100 yr	10 yr	Present	7.83	8.90	10 yr	100 yr	Present	7.44	13.70	7.83	Upper Bound
250 yr	10 yr	Present	8.04	8.90	10 yr	250 yr	Present	7.87	16.40	8.04	Upper Bound
500 yr	10 yr	Present	8.17	8.90	10 yr	500 yr	Present	7.87	17.30	8.17	Upper Bound

Table 22: Newark D4 Results (feet NAVD88)

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	3.69	3.20						3.69	Lower Bound
5 yr	Normal	Present	4.01	3.20						4.01	Lower Bound
10 yr	Normal	Present	4.16	3.20						4.16	Lower Bound
25 yr	Normal	Present	4.42	3.20						4.42	Lower Bound
50 yr	Normal	Present	4.63	3.20						4.63	Lower Bound
100 yr	Normal	Present	4.85	3.20						4.85	Lower Bound
250 yr	Normal	Present	5.16	3.20						5.16	Lower Bound
500 yr	Normal	Present	5.45	3.20						5.45	Lower Bound
2 yr	2 yr	Present	5.76	5.71	2 yr	2 yr	Present	5.76	5.71	5.76	Most Likely
5 yr	2 yr	Present	5.97	5.71	2 yr	5 yr	Present	6.19	7.30	6.19	Most Likely
10 yr	2 yr	Present	6.04	5.71	2 yr	10 yr	Present	6.25	8.90	6.25	Most Likely
25 yr	2 yr	Present	6.13	5.71	2 yr	25 yr	Present	6.29	10.80	6.29	Most Likely
50 yr	2 yr	Present	6.21	5.71	2 yr	50 yr	Present	6.30	12.30	6.30	Most Likely
100 yr	2 yr	Present	6.29	5.71	2 yr	100 yr	Present	6.32	13.70	6.32	Most Likely
250 yr	2 yr	Present	6.39	5.71	2 yr	250 yr	Present	6.40	16.40	6.40	Most Likely
500 yr	2 yr	Present	6.49	5.71	2 yr	500 yr	Present	7.08	17.30	7.08	Most Likely
2 yr	10 yr	Present	6.25	8.90	10 yr	2 yr	Present	6.04	5.71	6.25	Upper Bound
5 yr	10 yr	Present	6.50	8.90	10 yr	5 yr	Present	6.50	7.30	6.50	Upper Bound
10 yr	10 yr	Present	6.72	8.90	10 yr	10 yr	Present	6.72	8.90	6.72	Upper Bound
25 yr	10 yr	Present	7.02	8.90	10 yr	25 yr	Present	6.82	10.80	7.02	Upper Bound
50 yr	10 yr	Present	7.28	8.90	10 yr	50 yr	Present	6.88	12.30	7.28	Upper Bound
100 yr	10 yr	Present	7.60	8.90	10 yr	100 yr	Present	6.90	13.70	7.60	Upper Bound
250 yr	10 yr	Present	8.01	8.90	10 yr	250 yr	Present	7.54	16.40	8.01	Upper Bound
500 yr	10 yr	Present	8.23	8.90	10 yr	500 yr	Present	8.09	17.30	8.23	Upper Bound

Table 23: Newark D5 Results (feet NAVD88)

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	4.46	3.20						4.46	**Lower Bound**
5 yr	Normal	Present	4.84	3.20						4.84	**Lower Bound**
10 yr	Normal	Present	5.17	3.20						5.17	**Lower Bound**
25 yr	Normal	Present	5.67	3.20						5.67	**Lower Bound**
50 yr	Normal	Present	6.03	3.20						6.03	**Lower Bound**
100 yr	Normal	Present	6.18	3.20						6.18	**Lower Bound**
250 yr	Normal	Present	6.37	3.20						6.37	**Lower Bound**
500 yr	Normal	Present	6.63	3.20						6.63	**Lower Bound**
2 yr	2 yr	Present	5.39	5.71	2 yr	2 yr	Present	5.39	5.71	5.39	**Most Likely**
5 yr	2 yr	Present	5.86	5.71	2 yr	5 yr	Present	5.81	7.30	5.86	**Most Likely**
10 yr	2 yr	Present	6.08	5.71	2 yr	10 yr	Present	5.97	8.90	6.08	**Most Likely**
25 yr	2 yr	Present	6.27	5.71	2 yr	25 yr	Present	6.03	10.80	6.27	**Most Likely**
50 yr	2 yr	Present	6.43	5.71	2 yr	50 yr	Present	6.05	12.30	6.43	**Most Likely**
100 yr	2 yr	Present	6.61	5.71	2 yr	100 yr	Present	6.13	13.70	6.61	**Most Likely**
250 yr	2 yr	Present	6.81	5.71	2 yr	250 yr	Present	6.13	16.40	6.81	**Most Likely**
500 yr	2 yr	Present	7.12	5.71	2 yr	500 yr	Present	6.13	17.30	7.12	**Most Likely**
2 yr	10 yr	Present	5.97	8.90	10 yr	2 yr	Present	6.08	5.71	6.08	**Upper Bound**
5 yr	10 yr	Present	6.24	8.90	10 yr	5 yr	Present	6.34	7.30	6.34	**Upper Bound**
10 yr	10 yr	Present	6.44	8.90	10 yr	10 yr	Present	6.44	8.90	6.44	**Upper Bound**
25 yr	10 yr	Present	6.73	8.90	10 yr	25 yr	Present	6.53	10.80	6.73	**Upper Bound**
50 yr	10 yr	Present	6.97	8.90	10 yr	50 yr	Present	6.59	12.30	6.97	**Upper Bound**
100 yr	10 yr	Present	7.23	8.90	10 yr	100 yr	Present	6.62	13.70	7.23	**Upper Bound**
250 yr	10 yr	Present	7.55	8.90	10 yr	250 yr	Present	6.75	16.40	7.55	**Upper Bound**
500 yr	10 yr	Present	7.95	8.90	10 yr	500 yr	Present	6.75	17.30	7.95	**Upper Bound**

4.9 Town of Kearny

4.9.1 Interior Drainage Areas

Four interior drainage areas contribute to ponding behind the Kearny alignment, as shown in **Figure 10**. The drainage area parameters are shown in **Table 24**.

1) K1: This 0.222-square mile area drains by one 36 inch primary outlet, eight 24-inch secondary outlets and a 75-cfs pump station.
2) K2: This drainage area of 0.036-square miles is served by one 48-inch primary outlet and three 24-inch secondary outlets.
3) K3: This 0.632-square mile drainage area discharges through one primary 66-inch outlet and six secondary 24-inch pipes into the Hackensack River.
4) K4: This 0.648 square mile drainage area discharges through one primary 36 inch outlet and 34 secondary 24 inch pipes into the Passaic and Hackensack rivers.

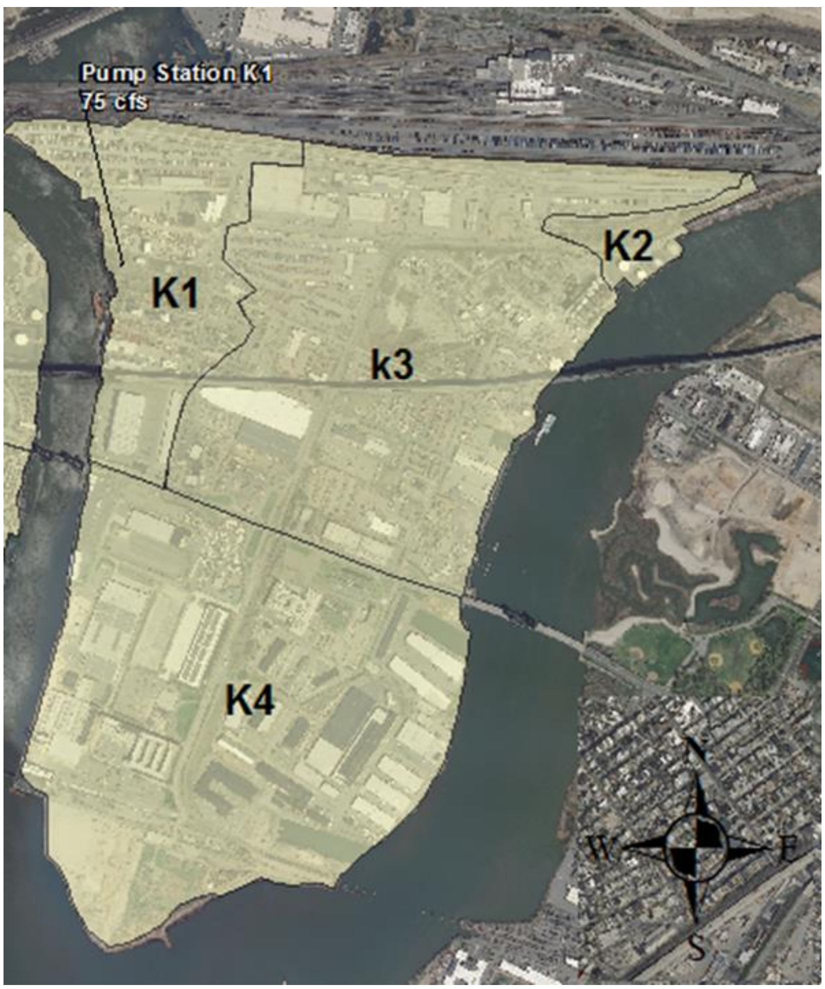

Figure 10: Kearny Drainage Areas

Table 24: Drainage Area Parameters – Kearny

Subarea	Drainage Areas (square miles)	SCS Curve Number	SCS Unitgraph	
			Tc (hour)	Lag (min)
K1	0.2215	81	0.76	27.4
K2	0.0364	80	0.72	25.9
K3	0.6319	85	0.8	28.8
K4	0.6476	80	0.82	29.5
Source: Passaic River Draft GDM, Appendix C H&H, Table C-76				

Some of the ponding areas are linked by an area of high ground (K2-K3), which is modeled as a diversion weir in the HEC-HMS model. Runoff from these areas and the interior ponding areas are reflected in the HEC-HMS model schematics shown in **Figure 11**.

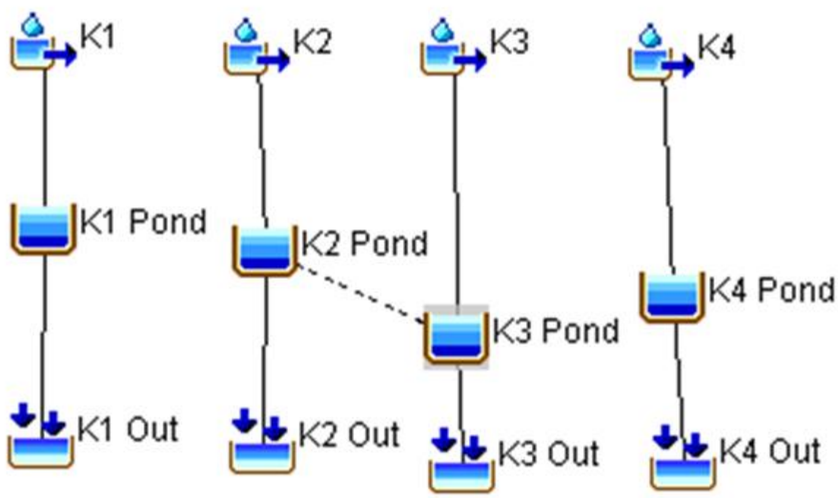

Figure 11: Kearny HEC-HMS Model Schematic

4.9.2 Kearny Interior Drainage Plan

The Kearny interior drainage features are shown in **Table 25**. The results of the analysis are shown in Tables **26 through 29**.

Table 25: Kearny Interior Drainage Features

Levee/Wall	Status	Type	Length (feet)	Number	Size (inches)	Pump
K1	New	Primary	10	1	36	75 cfs
	New	Secondary	10	6	24	
	New	Secondary	10	2	24	
K2	New	Primary	10	1	48	
	New	Secondary	10	3	24	
K3	Existing	Primary	10	1	66	
	New	Secondary	10	2	24	
	New	Secondary	10	4	24	
K4	New	Primary	10	3	36	
	New	Secondary	10	17	24	
	New	Secondary	10	17	24	

Table 26: Kearny K1 Results (feet NAVD88)

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	4.60	3.20						4.60	Lower Bound
5 yr	Normal	Present	4.98	3.20						4.98	Lower Bound
10 yr	Normal	Present	5.07	3.20						5.07	Lower Bound
25 yr	Normal	Present	5.56	3.20						5.56	Lower Bound
50 yr	Normal	Present	6.00	3.20						6.00	Lower Bound
100 yr	Normal	Present	6.06	3.20						6.06	Lower Bound
250 yr	Normal	Present	6.16	3.20						6.16	Lower Bound
500 yr	Normal	Present	6.26	3.20						6.26	Lower Bound
2 yr	2 yr	Present	5.80	5.71	2 yr	2 yr	Present	5.80	5.71	5.80	Most Likely
5 yr	2 yr	Present	6.01	5.71	2 yr	5 yr	Present	6.01	7.30	6.01	Most Likely
10 yr	2 yr	Present	6.06	5.71	2 yr	10 yr	Present	6.02	8.90	6.06	Most Likely
25 yr	2 yr	Present	6.16	5.71	2 yr	25 yr	Present	6.02	10.80	6.16	Most Likely
50 yr	2 yr	Present	6.26	5.71	2 yr	50 yr	Present	6.02	12.30	6.26	Most Likely
100 yr	2 yr	Present	6.37	5.71	2 yr	100 yr	Present	6.02	13.70	6.37	Most Likely
250 yr	2 yr	Present	6.50	5.71	2 yr	250 yr	Present	6.02	16.40	6.50	Most Likely
500 yr	2 yr	Present	6.64	5.71	2 yr	500 yr	Present	6.02	17.30	6.64	Most Likely
2 yr	10 yr	Present	6.02	8.90	10 yr	2 yr	Present	6.06	5.71	6.06	Upper Bound
5 yr	10 yr	Present	6.15	8.90	10 yr	5 yr	Present	6.16	7.30	6.16	Upper Bound
10 yr	10 yr	Present	6.26	8.90	10 yr	10 yr	Present	6.26	8.90	6.26	Upper Bound
25 yr	10 yr	Present	6.45	8.90	10 yr	25 yr	Present	6.28	10.80	6.45	Upper Bound
50 yr	10 yr	Present	6.62	8.90	10 yr	50 yr	Present	6.28	12.30	6.62	Upper Bound
100 yr	10 yr	Present	6.82	8.90	10 yr	100 yr	Present	6.28	13.70	6.82	Upper Bound
250 yr	10 yr	Present	7.06	8.90	10 yr	250 yr	Present	6.28	16.40	7.06	Upper Bound
500 yr	10 yr	Present	7.47	8.90	10 yr	500 yr	Present	6.28	17.30	7.47	Upper Bound

Table 27: Kearny K2 Results (feet NAVD88)

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	3.74	3.20						3.74	Lower Bound
5 yr	Normal	Present	3.94	3.20						3.94	Lower Bound
10 yr	Normal	Present	4.01	3.20						4.01	Lower Bound
25 yr	Normal	Present	4.08	3.20						4.08	Lower Bound
50 yr	Normal	Present	4.14	3.20						4.14	Lower Bound
100 yr	Normal	Present	4.21	3.20						4.21	Lower Bound
250 yr	Normal	Present	4.30	3.20						4.30	Lower Bound
500 yr	Normal	Present	4.38	3.20						4.38	Lower Bound
2 yr	2 yr	Present	4.39	5.71	2 yr	2 yr	Present	4.39	5.71	4.39	Most Likely
5 yr	2 yr	Present	4.58	5.71	2 yr	5 yr	Present	4.48	7.30	4.58	Most Likely
10 yr	2 yr	Present	4.74	5.71	2 yr	10 yr	Present	4.52	8.90	4.74	Most Likely
25 yr	2 yr	Present	4.95	5.71	2 yr	25 yr	Present	4.54	10.80	4.95	Most Likely
50 yr	2 yr	Present	5.11	5.71	2 yr	50 yr	Present	4.62	12.30	5.11	Most Likely
100 yr	2 yr	Present	5.27	5.71	2 yr	100 yr	Present	4.62	13.70	5.27	Most Likely
250 yr	2 yr	Present	5.43	5.71	2 yr	250 yr	Present	4.62	16.40	5.43	Most Likely
500 yr	2 yr	Present	5.60	5.71	2 yr	500 yr	Present	4.63	17.30	5.60	Most Likely
2 yr	10 yr	Present	4.52	8.90	10 yr	2 yr	Present	4.74	5.71	4.74	Upper Bound
5 yr	10 yr	Present	4.76	8.90	10 yr	5 yr	Present	4.88	7.30	4.88	Upper Bound
10 yr	10 yr	Present	4.96	8.90	10 yr	10 yr	Present	4.96	8.90	4.96	Upper Bound
25 yr	10 yr	Present	5.26	8.90	10 yr	25 yr	Present	5.04	10.80	5.26	Upper Bound
50 yr	10 yr	Present	5.50	8.90	10 yr	50 yr	Present	5.12	12.30	5.50	Upper Bound
100 yr	10 yr	Present	5.76	8.90	10 yr	100 yr	Present	5.21	13.70	5.76	Upper Bound
250 yr	10 yr	Present	6.04	8.90	10 yr	250 yr	Present	5.21	16.40	6.04	Upper Bound
500 yr	10 yr	Present	6.34	8.90	10 yr	500 yr	Present	5.22	17.30	6.34	Upper Bound

Table 28: **Kearny K3 Results (feet NAVD88)**

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	4.58	3.20						4.58	Lower Bound
5 yr	Normal	Present	4.87	3.20						4.87	Lower Bound
10 yr	Normal	Present	5.10	3.20						5.10	Lower Bound
25 yr	Normal	Present	5.45	3.20						5.45	Lower Bound
50 yr	Normal	Present	5.74	3.20						5.74	Lower Bound
100 yr	Normal	Present	6.02	3.20						6.02	Lower Bound
250 yr	Normal	Present	6.16	3.20						6.16	Lower Bound
500 yr	Normal	Present	6.35	3.20						6.35	Lower Bound
2 yr	2 yr	Present	4.76	5.71	2 yr	2 yr	Present	4.76	5.71	4.76	Most Likely
5 yr	2 yr	Present	5.07	5.71	2 yr	5 yr	Present	4.92	7.30	5.07	Most Likely
10 yr	2 yr	Present	5.32	5.71	2 yr	10 yr	Present	4.98	8.90	5.32	Most Likely
25 yr	2 yr	Present	5.67	5.71	2 yr	25 yr	Present	5.05	10.80	5.67	Most Likely
50 yr	2 yr	Present	5.96	5.71	2 yr	50 yr	Present	5.16	12.30	5.96	Most Likely
100 yr	2 yr	Present	6.09	5.71	2 yr	100 yr	Present	5.21	13.70	6.09	Most Likely
250 yr	2 yr	Present	6.24	5.71	2 yr	250 yr	Present	5.21	16.40	6.24	Most Likely
500 yr	2 yr	Present	6.43	5.71	2 yr	500 yr	Present	5.22	17.30	6.43	Most Likely
2 yr	10 yr	Present	4.98	8.90	10 yr	2 yr	Present	5.32	5.71	5.32	Upper Bound
5 yr	10 yr	Present	5.38	8.90	10 yr	5 yr	Present	5.62	7.30	5.62	Upper Bound
10 yr	10 yr	Present	5.72	8.90	10 yr	10 yr	Present	5.72	8.90	5.72	Upper Bound
25 yr	10 yr	Present	6.07	8.90	10 yr	25 yr	Present	5.84	10.80	6.07	Upper Bound
50 yr	10 yr	Present	6.22	8.90	10 yr	50 yr	Present	5.94	12.30	6.22	Upper Bound
100 yr	10 yr	Present	6.37	8.90	10 yr	100 yr	Present	6.05	13.70	6.37	Upper Bound
250 yr	10 yr	Present	6.57	8.90	10 yr	250 yr	Present	6.05	16.40	6.57	Upper Bound
500 yr	10 yr	Present	6.78	8.90	10 yr	500 yr	Present	6.05	17.30	6.78	Upper Bound

Table 29: **Kearny K4 Results (feet NAVD88)**

Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Interior Flow	Exterior Stage	Time Condition	Peak Interior WSEL	Peak Exterior Stage	Max Interior WSEL	Risk
2 yr	Normal	Present	3.83	3.20						3.83	**Lower Bound**
5 yr	Normal	Present	4.02	3.20						4.02	**Lower Bound**
10 yr	Normal	Present	4.07	3.20						4.07	**Lower Bound**
25 yr	Normal	Present	4.17	3.20						4.17	**Lower Bound**
50 yr	Normal	Present	4.25	3.20						4.25	**Lower Bound**
100 yr	Normal	Present	4.34	3.20						4.34	**Lower Bound**
250 yr	Normal	Present	4.46	3.20						4.46	**Lower Bound**
500 yr	Normal	Present	4.58	3.20						4.58	**Lower Bound**
2 yr	2 yr	Present	4.41	5.71	2 yr	2 yr	Present	4.41	5.71	4.41	**Most Likely**
5 yr	2 yr	Present	4.62	5.71	2 yr	5 yr	Present	4.51	7.30	4.62	**Most Likely**
10 yr	2 yr	Present	4.77	5.71	2 yr	10 yr	Present	4.54	8.90	4.77	**Most Likely**
25 yr	2 yr	Present	4.99	5.71	2 yr	25 yr	Present	4.56	10.80	4.99	**Most Likely**
50 yr	2 yr	Present	5.16	5.71	2 yr	50 yr	Present	4.65	12.30	5.16	**Most Likely**
100 yr	2 yr	Present	5.32	5.71	2 yr	100 yr	Present	4.65	13.70	5.32	**Most Likely**
250 yr	2 yr	Present	5.48	5.71	2 yr	250 yr	Present	4.65	16.40	5.48	**Most Likely**
500 yr	2 yr	Present	5.68	5.71	2 yr	500 yr	Present	4.65	17.30	5.68	**Most Likely**
2 yr	10 yr	Present	4.54	8.90	10 yr	2 yr	Present	4.41	5.71	4.54	**Upper Bound**
5 yr	10 yr	Present	4.80	8.90	10 yr	5 yr	Present	4.51	7.30	4.80	**Upper Bound**
10 yr	10 yr	Present	5.01	8.90	10 yr	10 yr	Present	4.54	8.90	5.01	**Upper Bound**
25 yr	10 yr	Present	5.31	8.90	10 yr	25 yr	Present	4.56	10.80	5.31	**Upper Bound**
50 yr	10 yr	Present	5.57	8.90	10 yr	50 yr	Present	4.65	12.30	5.57	**Upper Bound**
100 yr	10 yr	Present	5.83	8.90	10 yr	100 yr	Present	4.65	13.70	5.83	**Upper Bound**
250 yr	10 yr	Present	6.06	8.90	10 yr	250 yr	Present	4.65	16.40	6.06	**Upper Bound**
500 yr	10 yr	Present	6.19	8.90	10 yr	500 yr	Present	4.65	17.30	6.19	**Upper Bound**

4.10 Minish Park

The Minish Park portion of the alignment, shown in **Figure 12**, was added to the project following closer review of the design elevations and potential for tidal surge inundation. This portion of the project was not in the 1995 GDM plan. As a new alignment, there were no existing interior drainage features. For this level of analysis and based on the short segment of the wall, it is assumed that minor interior drainage features would be needed; however, the interior drainage will be revised to include Minish Park during the next phase of the study.

Figure 12: Minish Park Alignment

4.11 Newark Flanking Area

Similar to Minish Park, the small floodwall and gate sections which comprise the Newark Flanking components of the alignment are new to the project. Based on topography, these small areas could be subject to run off from a very large drainage area; however, that drainage area is within the heart of Newark. As such, it has an extensive, existing stormwater drainage system which may or may not drain runoff to the locations of the flanking alignment. Therefore, the interior drainage at these locations cannot be reasonably modeled without more extensive stormwater drainage data collection, which will be accomplished in the next phase of the study.

4.12 Residual Damage

As noted previously, the interior drainage analysis competed as part of this GRR did not involve optimization. Instead, the previous facilities were incorporated into an up-to-date HEC-HMS model and the results reported. The residual flooding stage versus frequency curve for each drainage area is shown in **Tables 30 through 33**. **Figure 13** shows the approximate 100-year residual floodplain and exterior Sandy surge.

Table 30: **Harrison Residual Flooding Stage versus Frequency (S Areas)**

Frequency	S1	S2	S3
2-year	5.96	5.76	5.31
5-year	6.05	6.00	5.77
10-year	6.15	6.05	5.90
25-year	6.30	6.15	6.02
50-year	6.44	6.24	6.06
100-year	6.60	6.34	6.11
250-year	6.80	6.48	6.18
500-year	7.00	6.62	6.25
Elevations in feet NAVD88.			

Table 31: **Newark Residual Flooding Stage versus Frequency (D Areas)**

Frequency	D1	D2	D3A	D3B	D4	D5
2-year	6.04	4.62	4.32	5.70	5.76	5.39
5-year	6.32	4.90	4.51	6.26	6.19	5.86
10-year	6.38	5.11	4.67	6.41	6.25	6.08
25-year	6.55	5.38	4.91	6.53	6.29	6.27
50-year	6.72	5.56	5.12	6.62	6.30	6.43
100-year	6.89	5.71	5.35	6.69	6.32	6.61
250-year	7.08	5.86	5.68	7.11	6.40	6.81
500-year	7.35	6.01	6.04	7.11	7.08	7.12
Elevations in feet NAVD88.						

Table 32: **Newark Residual Flooding Stage versus Frequency (L and T Areas)**

Frequency	L1	L2	L3	T
2-year	5.98	5.11	4.99	5.08
5-year	6.52	5.55	5.10	5.49
10-year	7.01	5.75	5.34	5.71
25-year	7.12	5.95	5.65	5.96
50-year	7.17	6.04	5.82	6.05
100-year	7.19	6.10	5.99	6.14
250-year	7.36	6.19	6.07	6.24
500-year	7.36	6.28	6.18	6.36
Elevations in feet NAVD88.				

Table 33: **Kearny Residual Flooding Stage versus Frequency (K Areas)**

Frequency	K1	K2	K3	K4
2-year	5.80	4.39	4.76	4.41
5-year	6.01	4.58	5.07	4.62
10-year	6.06	4.74	5.32	4.77
25-year	6.16	4.95	5.67	4.99
50-year	6.26	5.11	5.96	5.16
100-year	6.37	5.27	6.09	5.32
250-year	6.50	5.43	6.24	5.48
500-year	6.64	5.60	6.43	5.68
Elevations in feet NAVD88.				

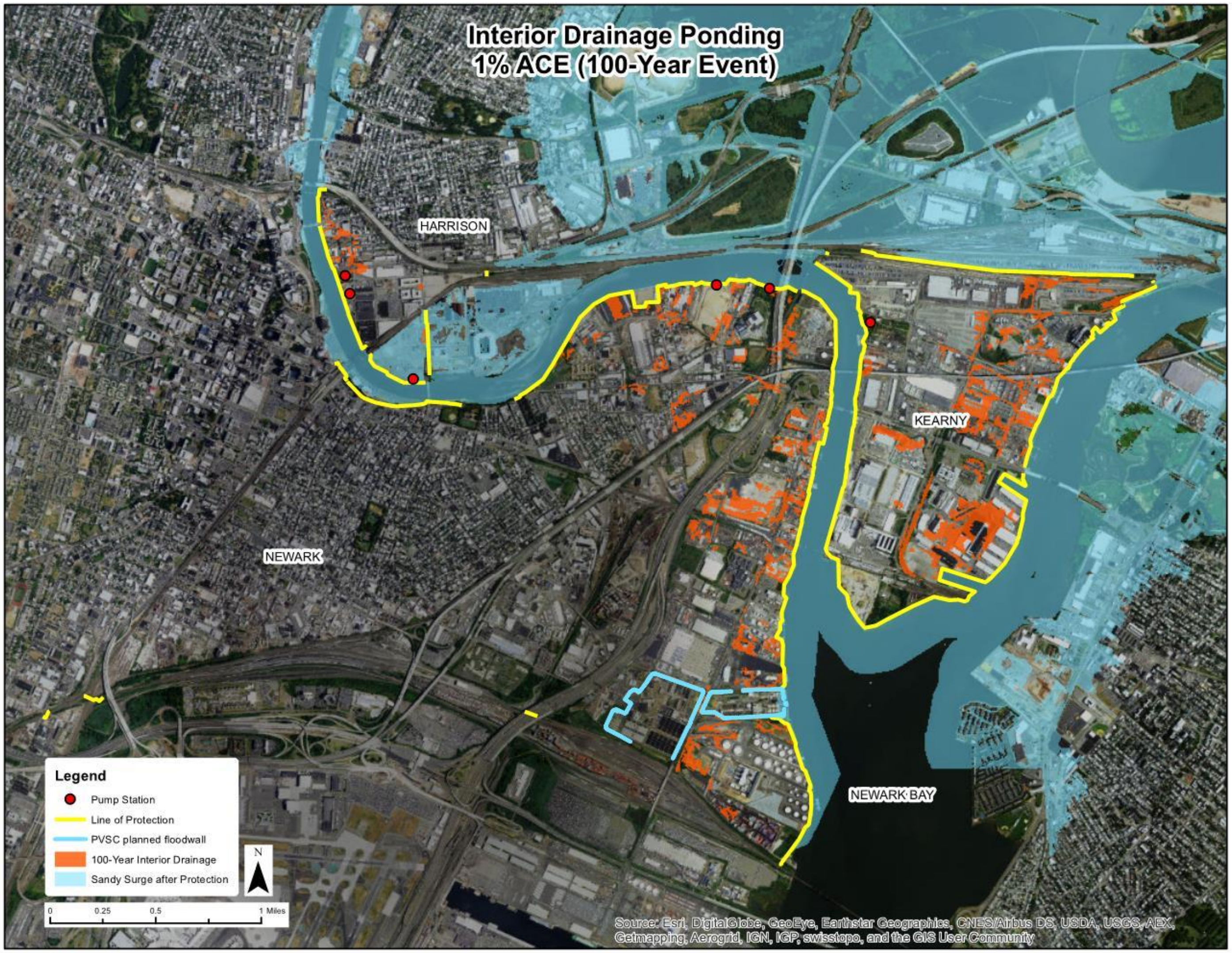

Figure 13: Residual Floodplain, 100-year (1% ACE)

5 PUMP STATIONS

The 1995 GDM plan includes six pump stations for interior drainage, ranging from 30 to 100 cfs as noted for Harrison, Newark, and Kearny. The GRR does 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.

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