US Army Corps of Engineers®
New York District

Passaic River, New Jersey

Passaic River Tidal General Reevaluation Report

Lower Passaic River, New Jersey

Appendix J
Engineering and Design

February 2019

TABLE OF CONTENTS

List of Tables

List of Figures

List of Subappendices

1 INTRODUCTION

The Engineering and Design Appendix presents the supporting technical information used in updating the authorized design of features of the Passaic River, New Jersey, Tidal Flood Risk Management Project presented in the General Reevaluation Report (GRR) as well as the 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 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 detailed engineering data for the Recommended Plan. The plan will provide flood risk management for inland portions of the City of Newark. Drawings for the Recommended Plan are provided in Subappendix 1. Geotechnical and structural analyses for the National Economic Development (NED) Plan are provided in Subappendix 2.

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

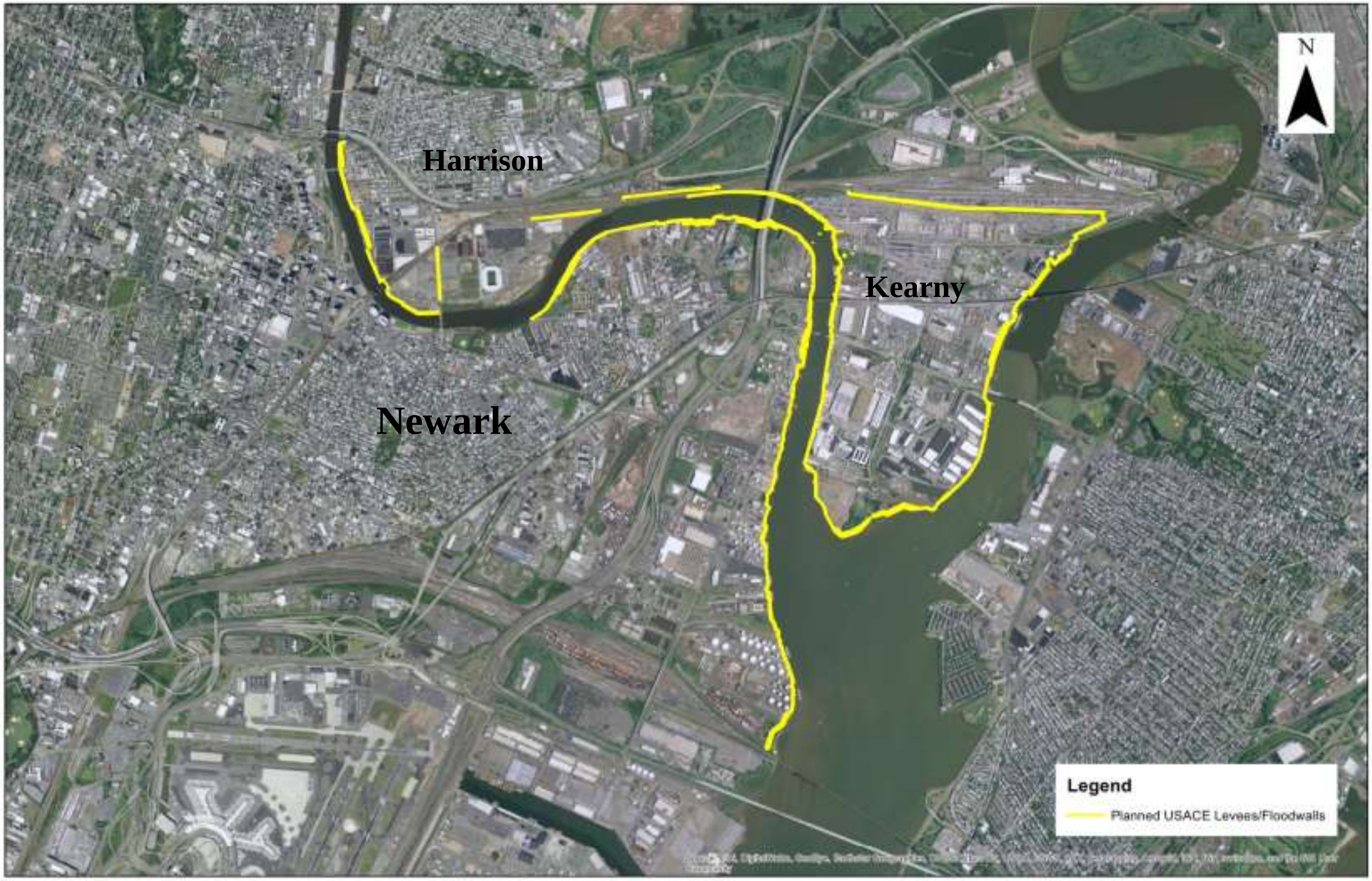

Figure 1: Passaic River Tidal Project Area – 1995 GDM Alignment

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 1 in 100 chance of occurring in any single year would be described as the 1 percent ACE event. This is the current accepted scientific terminology for expressing chance of exceedance. The annual recurrence interval, or return period, has historically been used by engineers to express probability of exceedance. For this document, due to the incorporation of historic information, both references may be used. Examples of equivalent expressions for exceedance probability for a range of ACEs are provided in **Table 1**.

Table 1: Annual Chance of Exceedance

ACE (as percent)	ACE (as probability)	Annual recurrence interval
50%	0.5	2-year
20%	0.2	5-year
10%	0.1	10-year
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 PROJECT PURPOSE

The purpose of the Passaic River, New Jersey, Integrated GRR and Environmental Assessment is to determine if the previously authorized or newly developed storm risk management projects in the study area are 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 NED PLAN DESCRIPTION

The Passaic Tidal study area was divided into six design areas based on geotechnical and engineering parameters, and for the economic analysis. The design areas are shown in **Figure 3**:

1) Harrison 1 – The area of Harrison included in the 1995 alignment.

2) Harrison 2 – An additional reach in Harrison which includes the Red Bull Arena and the PATH Service Station. This reach is eventually screened out as not economically viable and not included in the final NED plan. It is included in the cost engineering documentation for completeness.

3) Kearny – Also referred to as Kearny Point, this includes all of Kearny Point peninsula to the northern rail yard.

4) Newark – This area includes the areas of Newark subject to flooding from the east and was part of the 1995 alignment.

5) Minish – This area includes the alignment along Minish Park, providing flood risk management for ‘inland’ Newark.

6) Newark Flanking – This area includes floodwall and closure gates to limit flooding of the South Ironbound area of Newark from flood water flanking the alignment north of Newark Liberty International Airport.

Following plan formulation, the Harrison-2 component was screened out and the optimum NED design elevation determined to be a 16 feet NAVD88. The final NED Plan and associated floodplain is shown in **Figure 4**.

Figure 3: Passaic Tidal Project Reaches

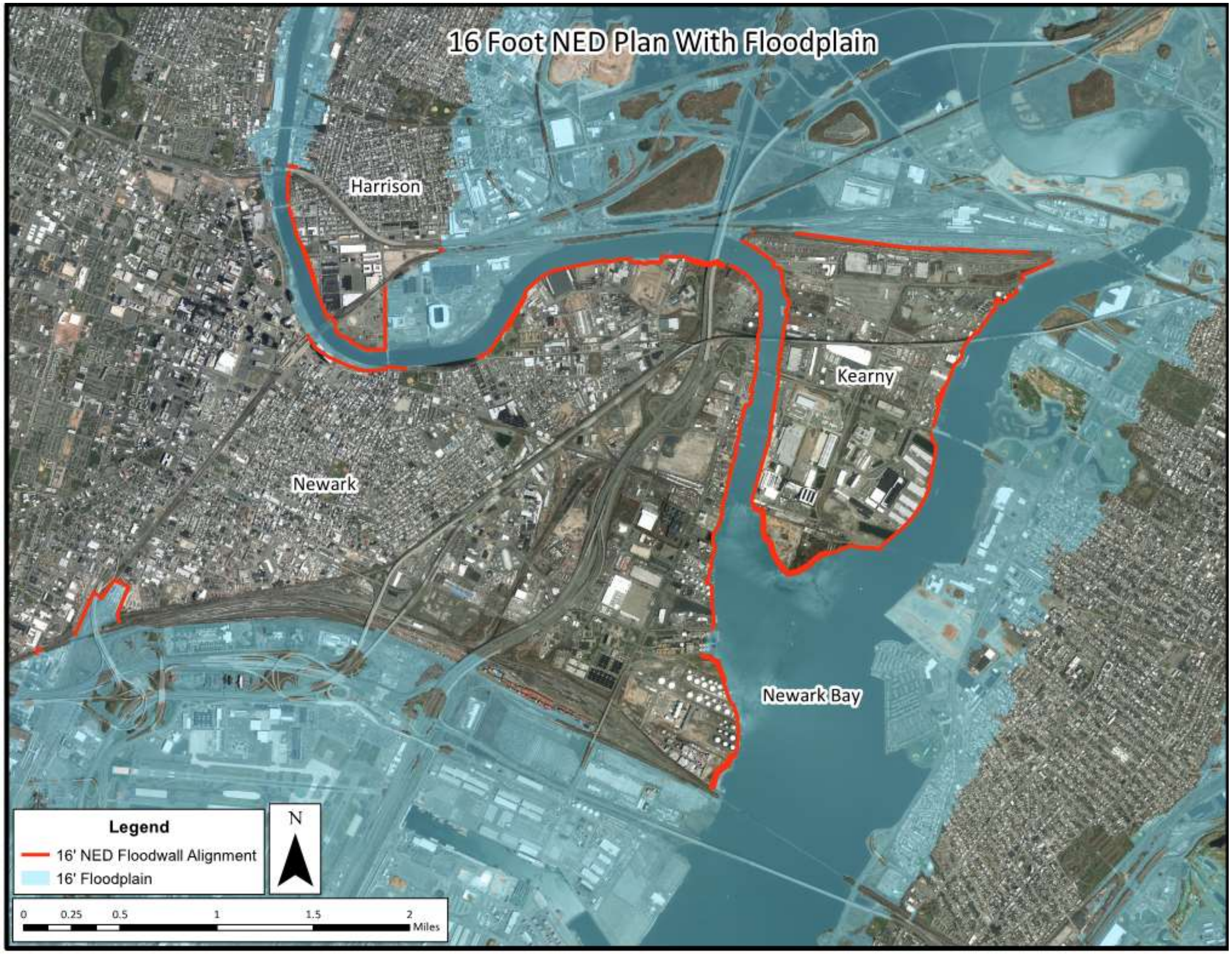

Figure 4: Passaic Tidal NED Plan – 16 feet NAVD88

5 RECOMMENDED PLAN

The Passaic Tidal Recommended Plan consists of seven segments of concrete floodwalls and gates along three reaches as described below. The design elevation is 14 feet NAVD88. The typical ground elevation at each segment is 6 to 10 feet NAVD88. For areas with a wall height of four feet or less, the wall is a concrete I-wall; for areas where the wall is greater than four feet, the wall is a pile-supported, concrete T-wall. The project reaches are shown in **Figure 5** and described below.

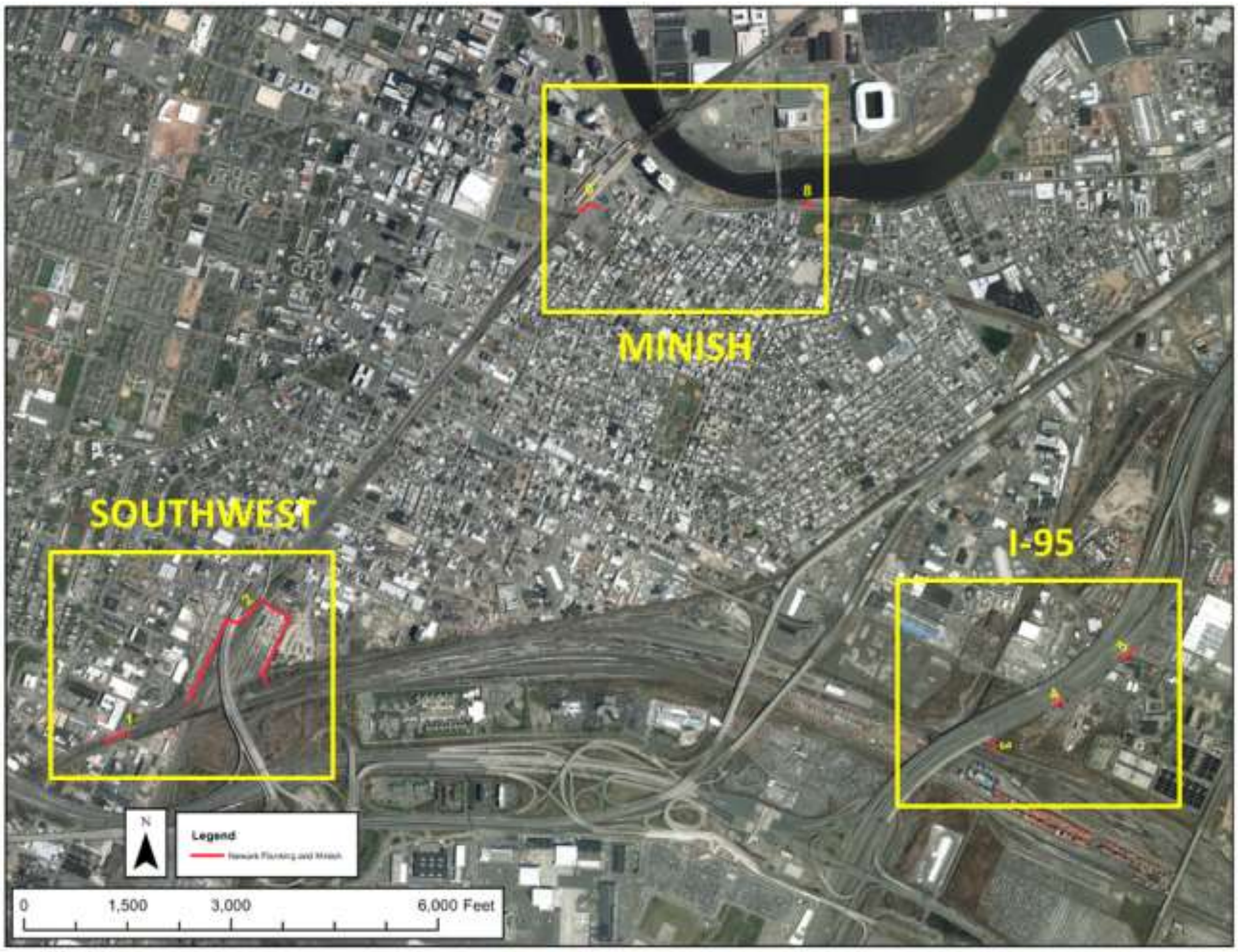

Figure 5: Passaic Tidal Project Reaches – Recommended Plan/Locally Preferred Plan

5.1 Southwest Reach

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

Segment 1: 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.

5.2 I-95 Reach

The I-95 Reach alignment includes two floodwall and one levee segment:

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.

5.3 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 6 through 15**. Interior drainage features are described in Section 6.

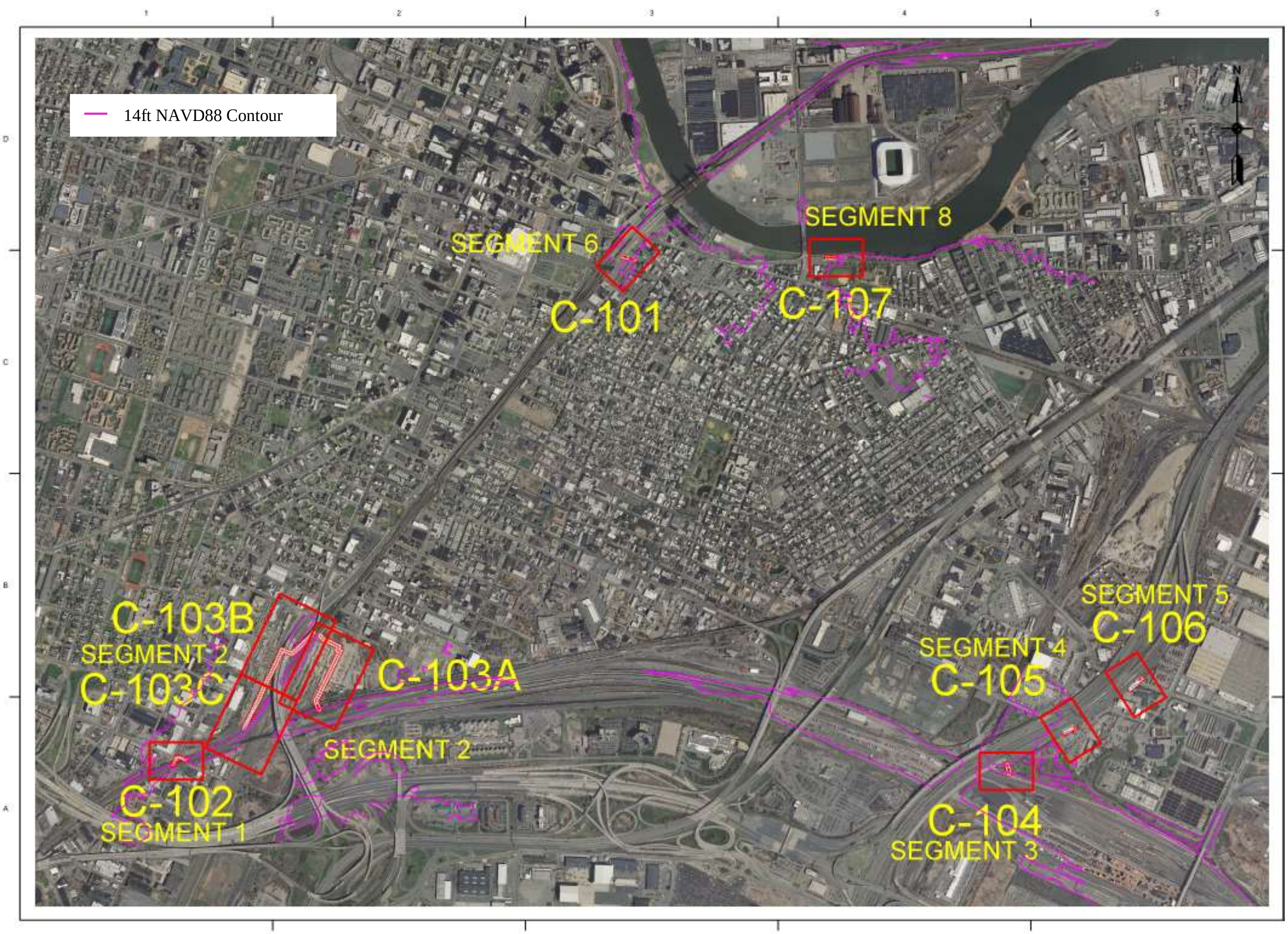

Figure 6: Recommended Plan Layout/Key Plan

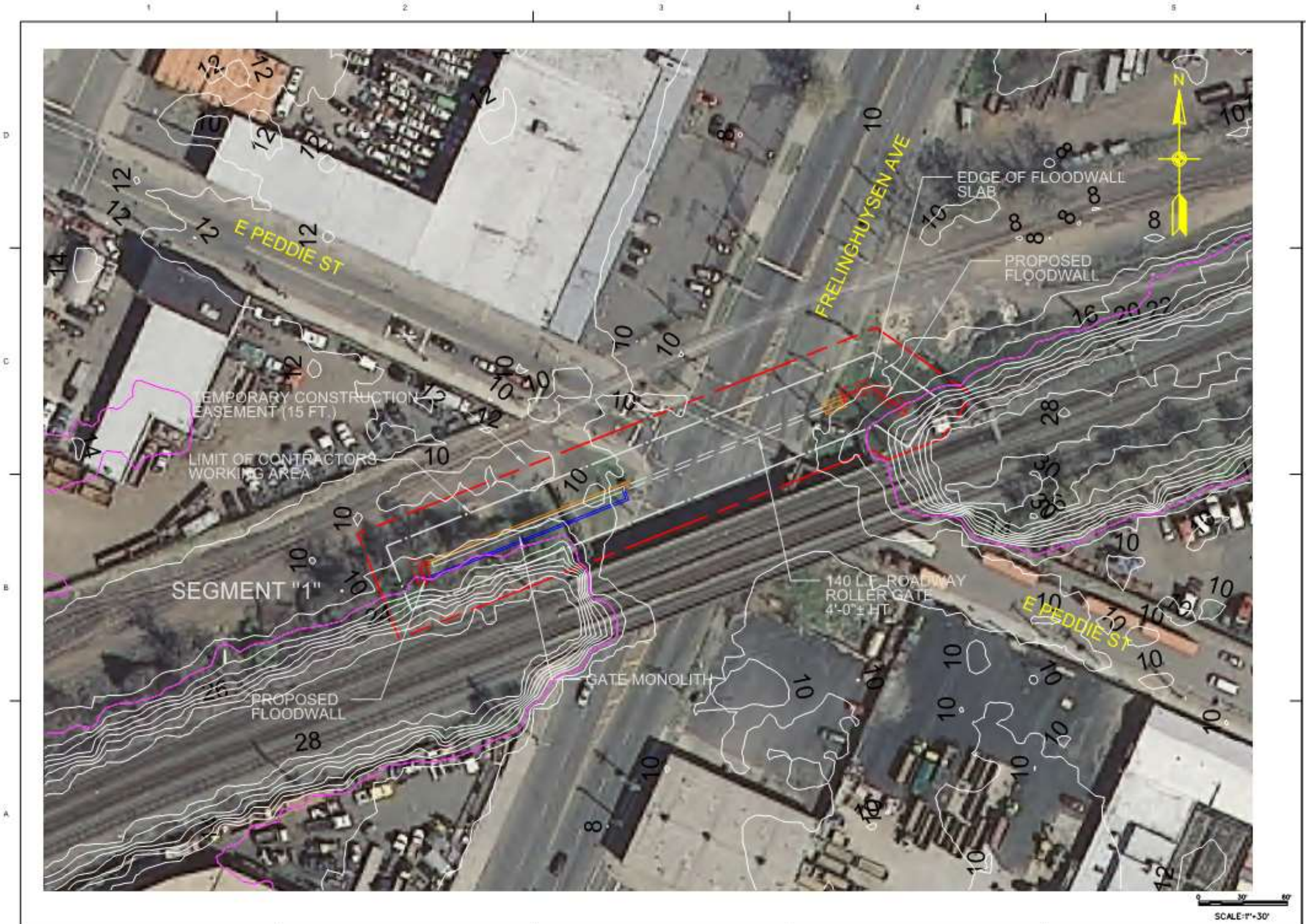

Figure 7: Southwest Reach - Segment 1

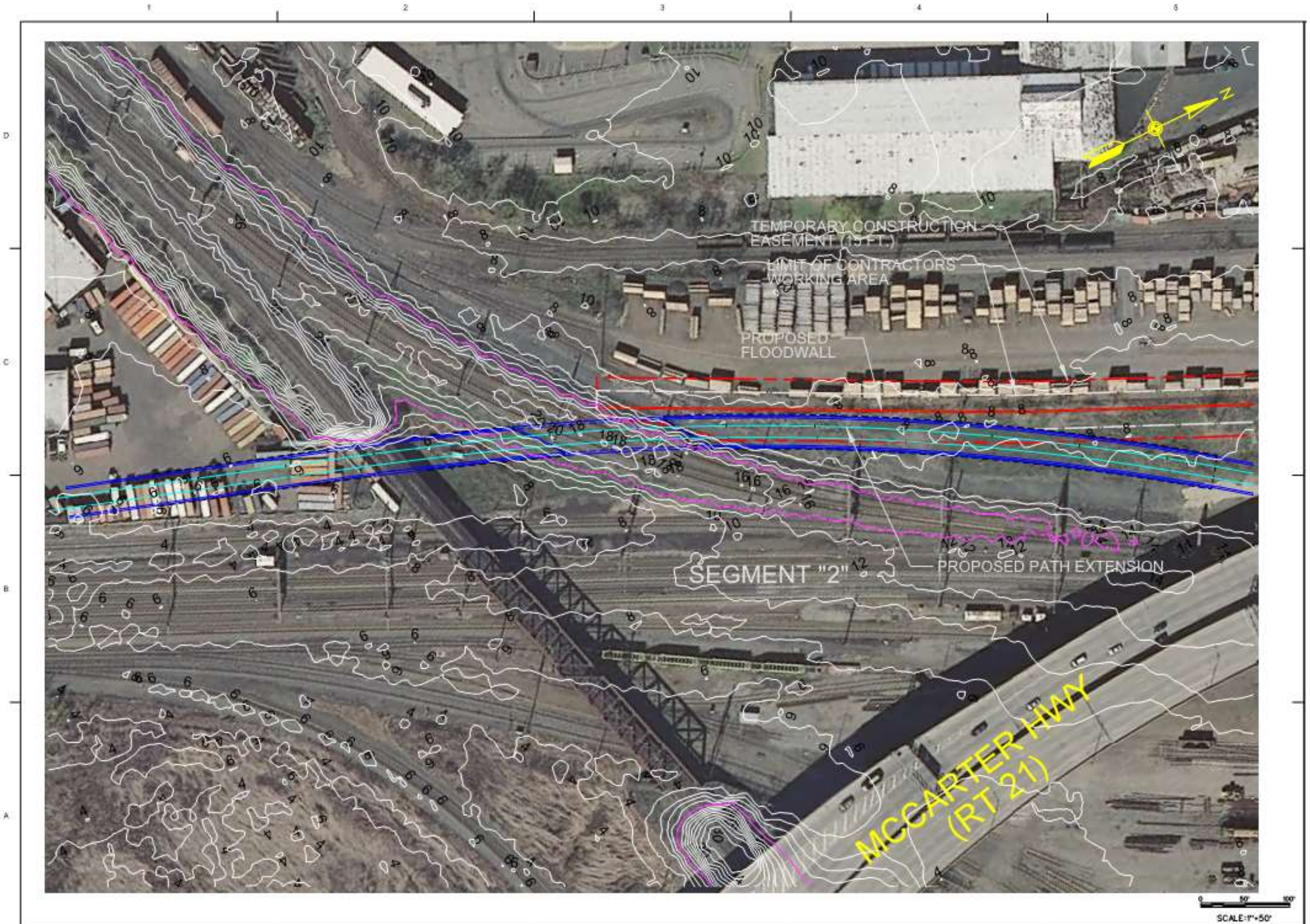

Figure 8: South West Reach - Segment 2A (South)

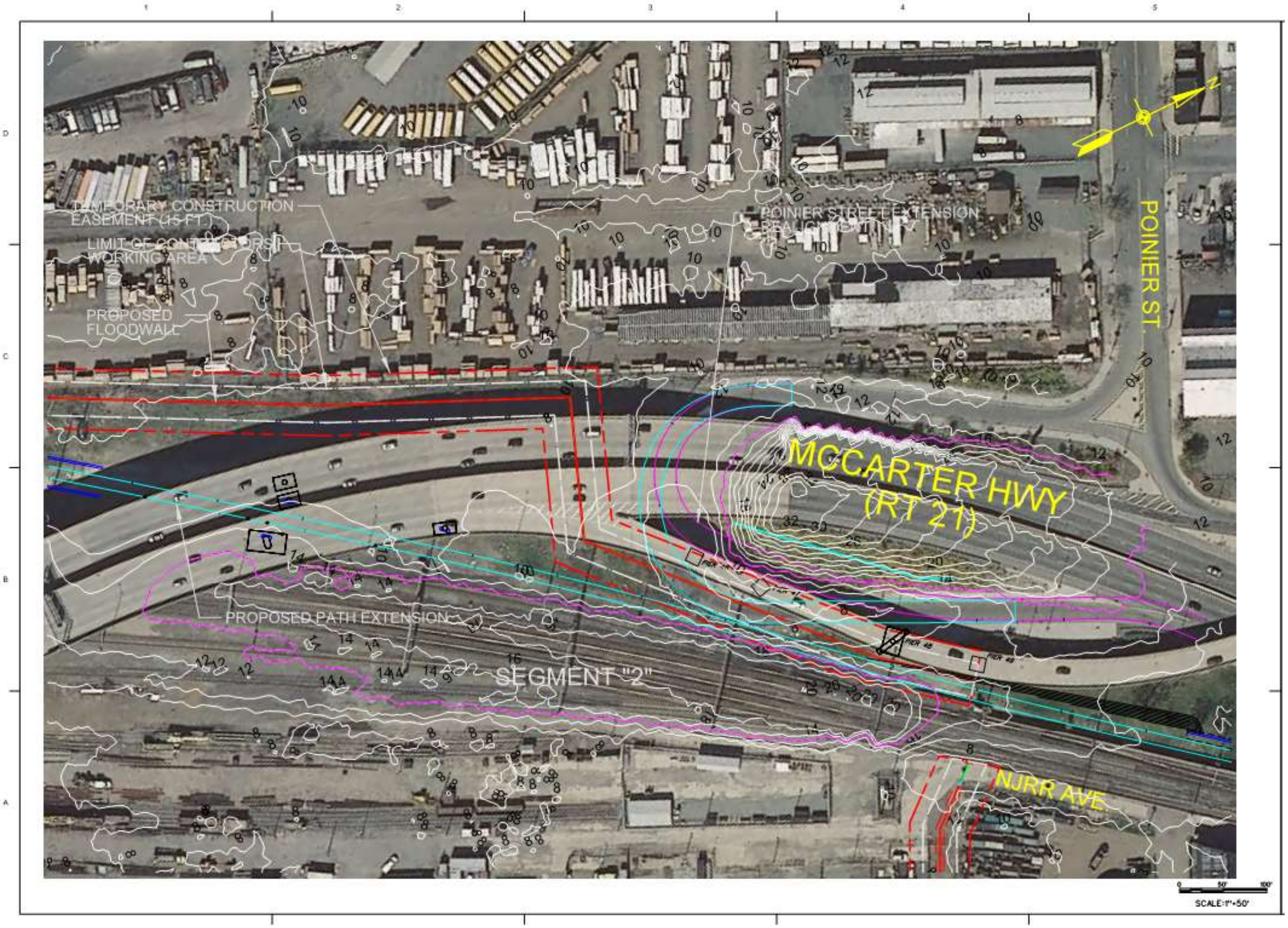

Figure 9: Southwest Reach - Segment 2A (North)

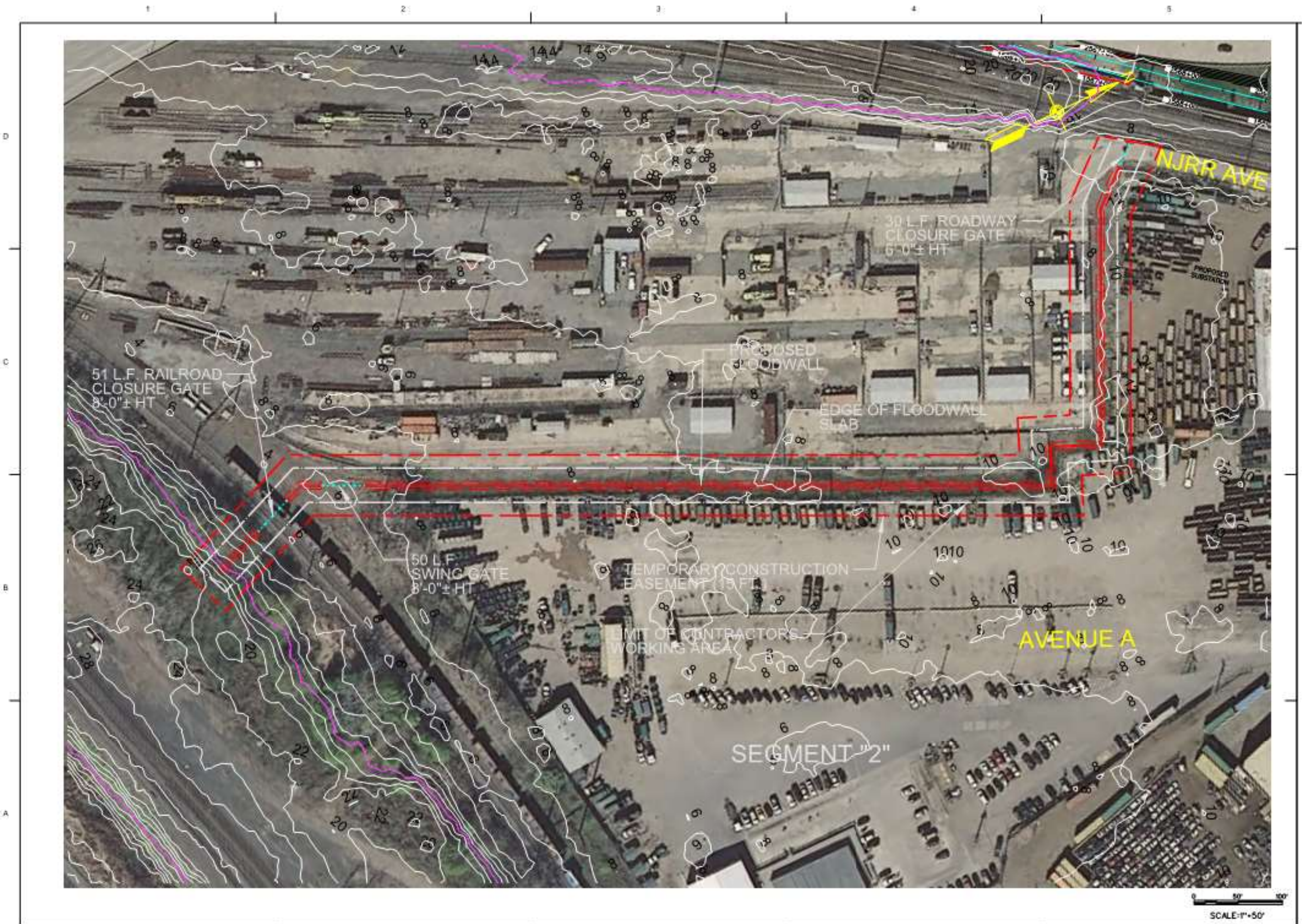

Figure 10: South West Reach - Segment 2B

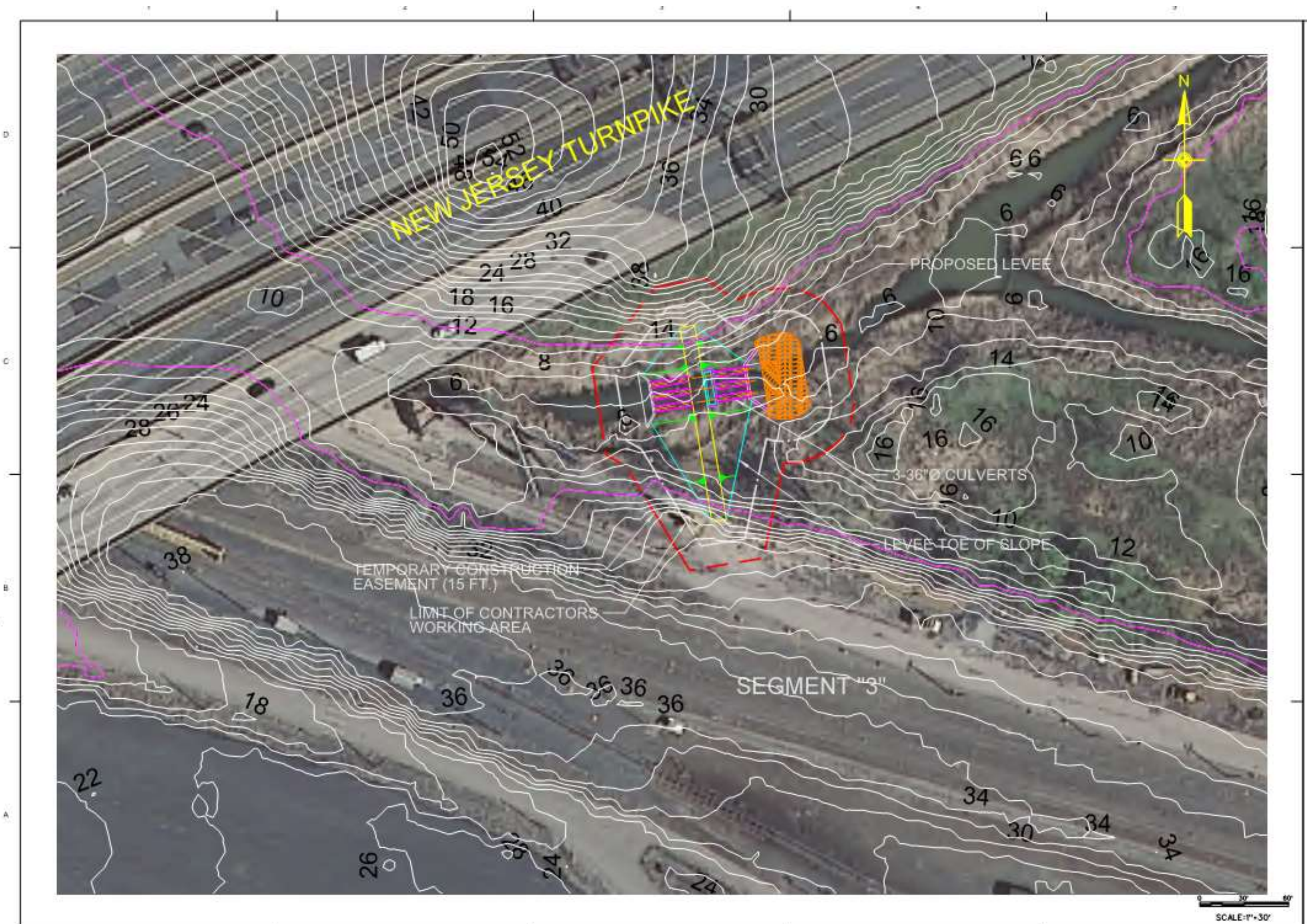

Figure 11: I-95 Reach - Segment 3

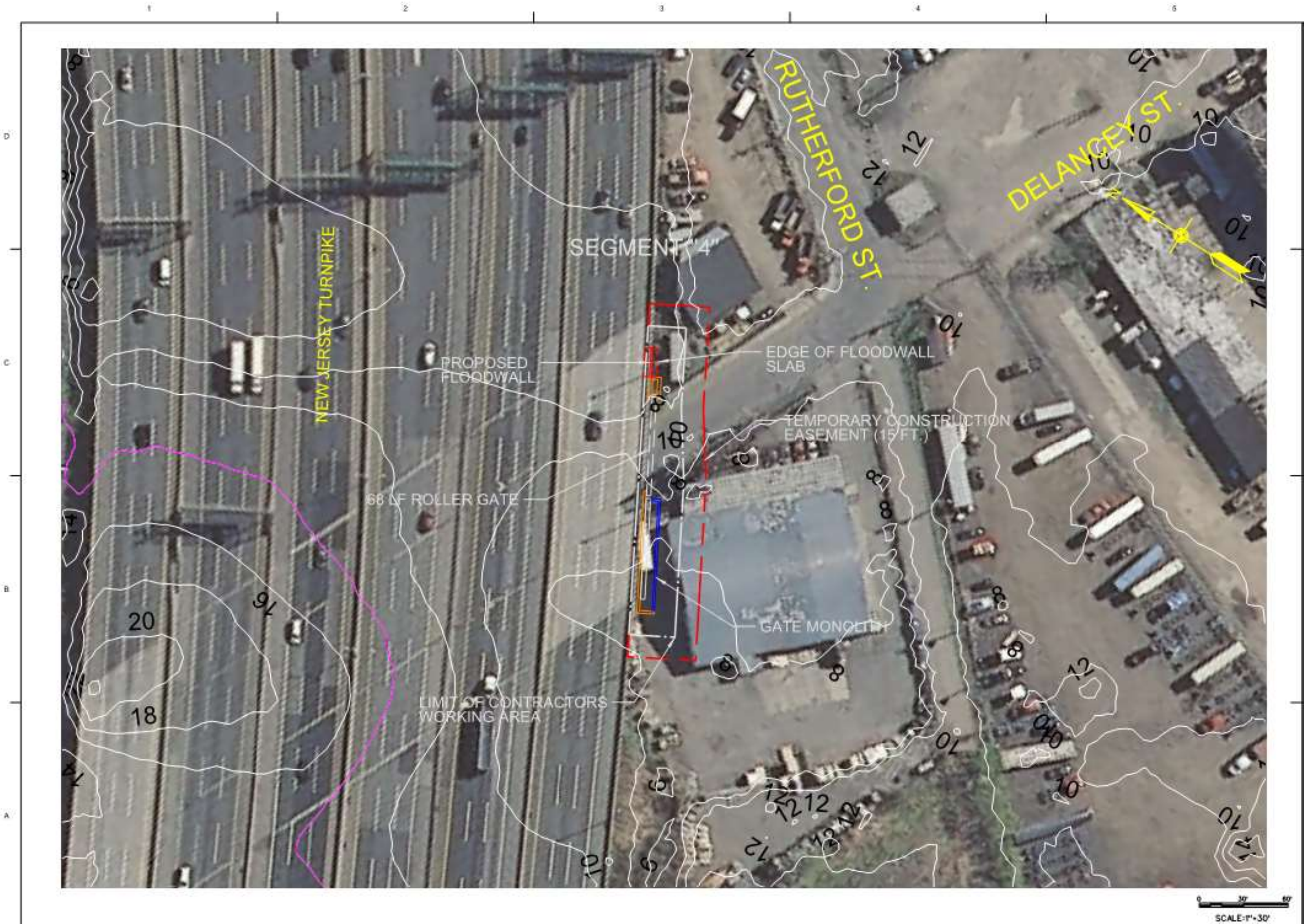

Figure 12: I-95 Reach - Segment 4

Figure 13: I-95 Reach - Segment 5

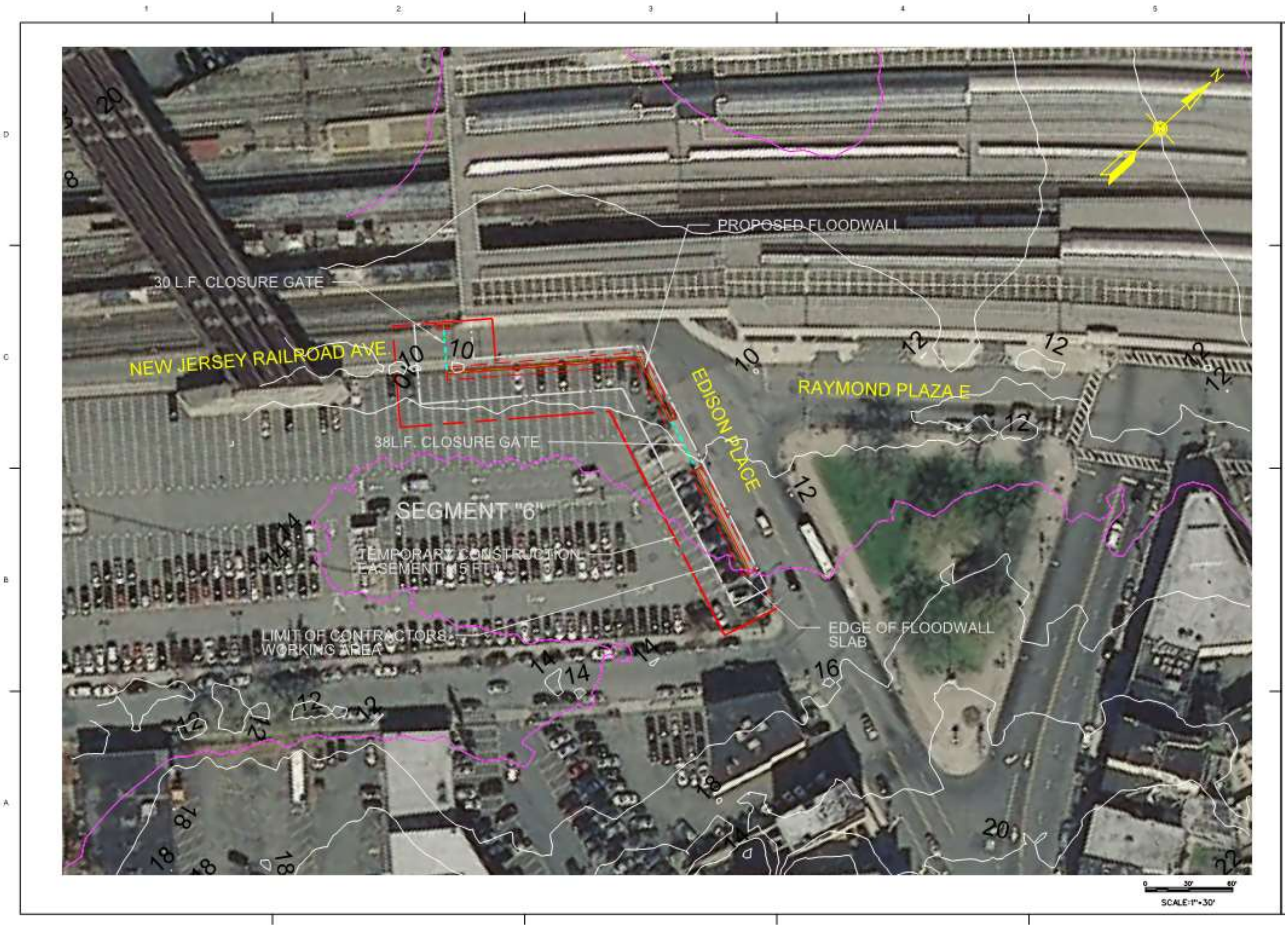

Figure 14: Minish Park Reach - Segment 6

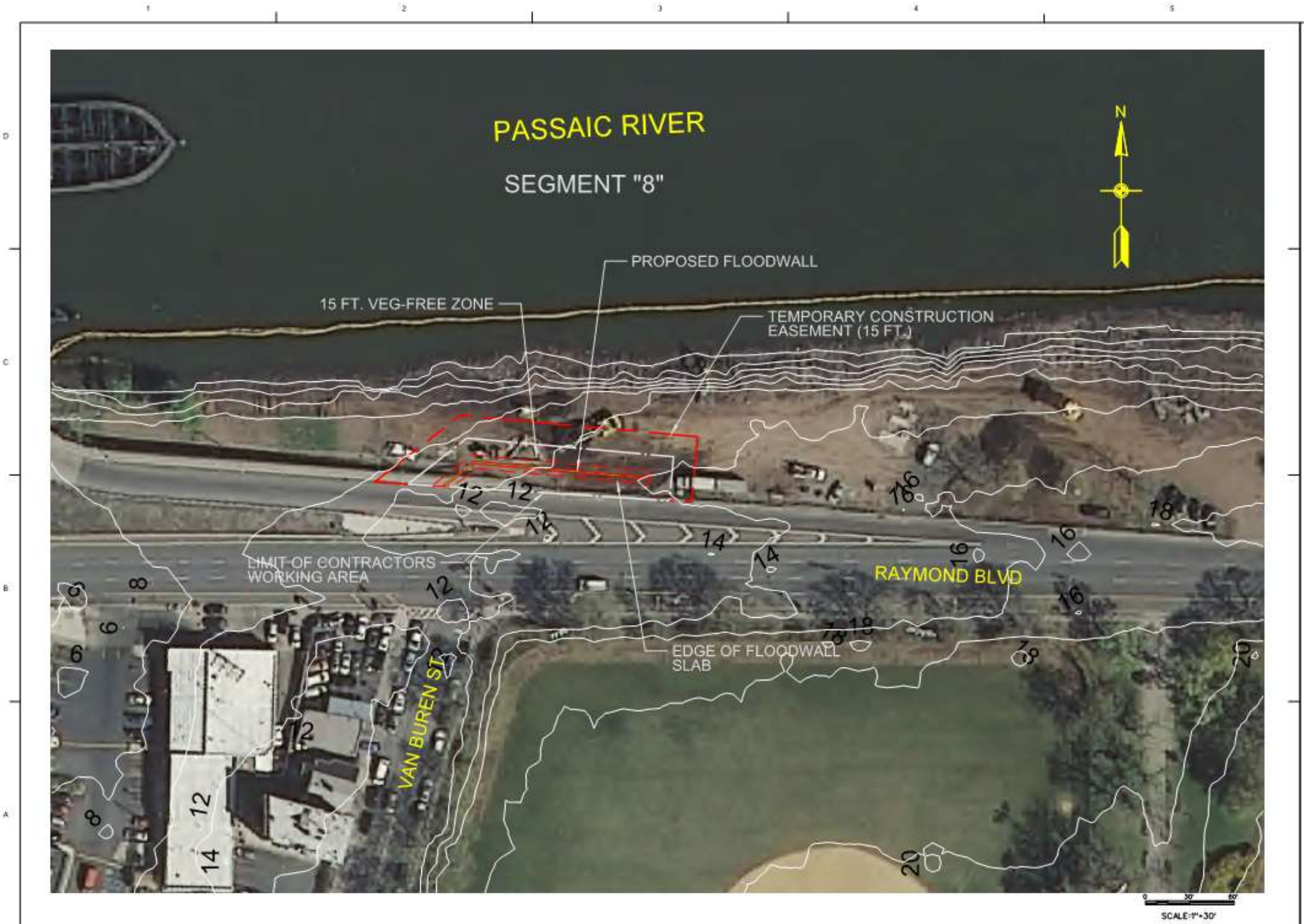

Figure 15: Minish Park Reach - Segment 8

6 HYDROLOGY AND HYDRAULICS

This section includes a summary of the hydrologic and hydraulic analyses completed as part of the general reevaluation. The analyses are presented in detail in Appendix F, Hydrology and Hydraulics (H&H).

6.1 Passaic River and Newark Bay Stillwater

The project is located near the mouth of the Passaic River and Hackensack River, and includes parts of Newark Bay in New Jersey. Stillwater Elevation (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 2 and 3**.

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

Annual Recurrence Interval (frequency)	ACE (probability)	SWEL (feet NAVD)
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 3: NACCS Stillwater Elevation - Stage versus Frequency (2070)

Annual Recurrence Interval (frequency)	ACE (probability)	SWEL (feet NAVD)
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

6.2 Waves and Overtopping

The study area is the shoreline along the Passaic River as it converges with the Hackensack River and flows into Newark Bay, in addition to a section of the shoreline of the Hackensack River at the same confluence. This area occupies parts of Hudson and Essex counties in New Jersey. The 1995 and 2013 studies did not consider wave runup or wave overtopping. Wave runup refers to the height above the water surface elevation reached by the swash. Runup is a complex phenomenon known to depend on the incident wave conditions (height, period, steepness, and direction), and the nature of the beach, levee or wall being run up (e.g. slope, reflectivity, height, permeability, and roughness). Wave overtopping refers to the volumetric rate at which runup flows over the top of the 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 16**. 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.

A detailed discussion of the wave model, wave heights, and overtopping are presented in Appendix F – Hydrology and Hydraulics.

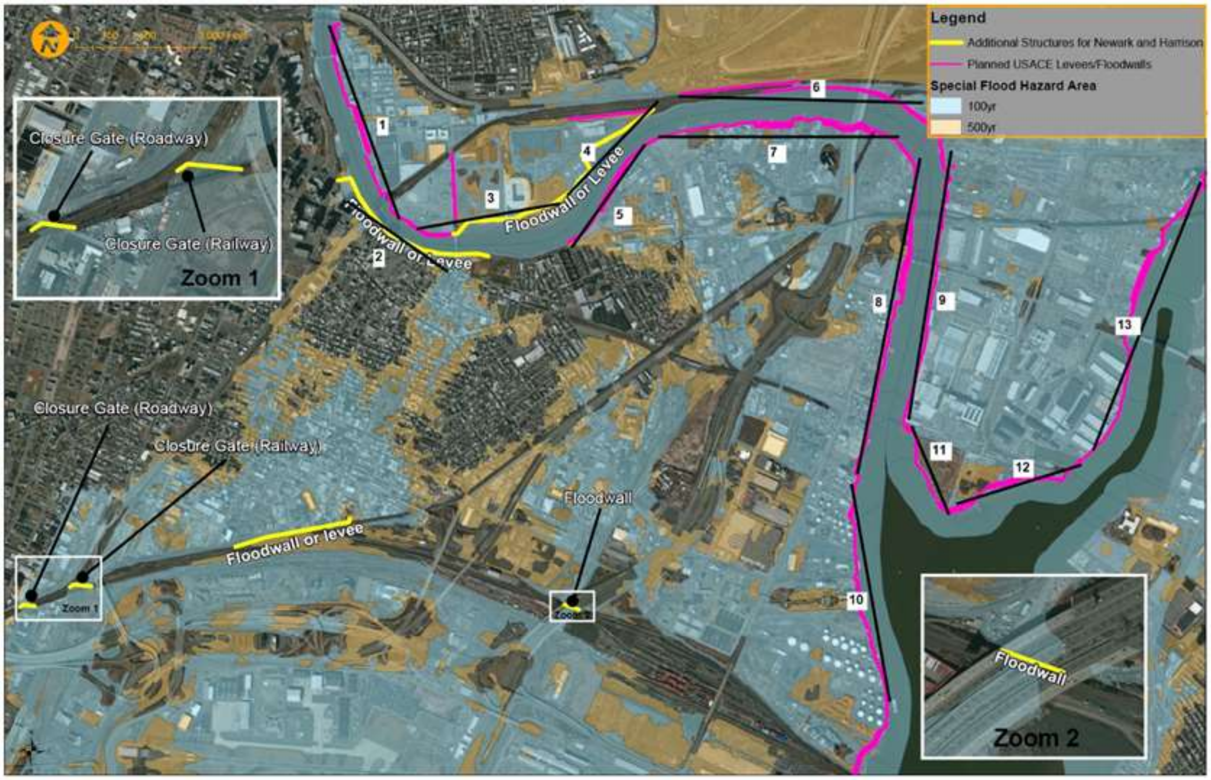

Figure 16: Segmentation of Levee / Floodwall System

6.3 Waves and the Recommended 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 infrastructure, and wave-limiting flood depths. Therefore, wave impacts and overtopping were not considered in the structural and interior drainage analyses of the Recommended Plan.

6.4 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 Dec 2013), which supersedes Engineer Circular (EC) 1165-2-212, *Sea Level Change Considerations for Civil Works Programs*. The ER refers to additional specific guidance in Engineer Technical Letter (ETL) 1100-2-1, *Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation*, which contains details previously contained in attachments to the old EC.

ER 1100-2-8162 states:

> *"Planning studies and engineering designs over the project life cycle, for both existing and proposed projects, will consider alternatives that are formulated and evaluated for the entire range of possible future rates of SLC, represented here by three scenarios of "low," "intermediate," and "high" SLC.*
>
> *…Once the three rates have been estimated, the next step is to determine how sensitive alternative plans and designs are to these rates of future local mean SLC, how this sensitivity affects calculated risk, and what design or operations and maintenance measures should be implemented to adapt to SLC to minimize adverse consequences while maximizing beneficial effects."*

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

The generated curves are based on USACE equations at a low, intermediate, and high level. The output for the USACE equations can be seen in **Table 4**. The program also plots a chart of the sea level curves as seen in **Figure 17**. SLC is discussed in more detail in the H&H Appendix.

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

Table 4: 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

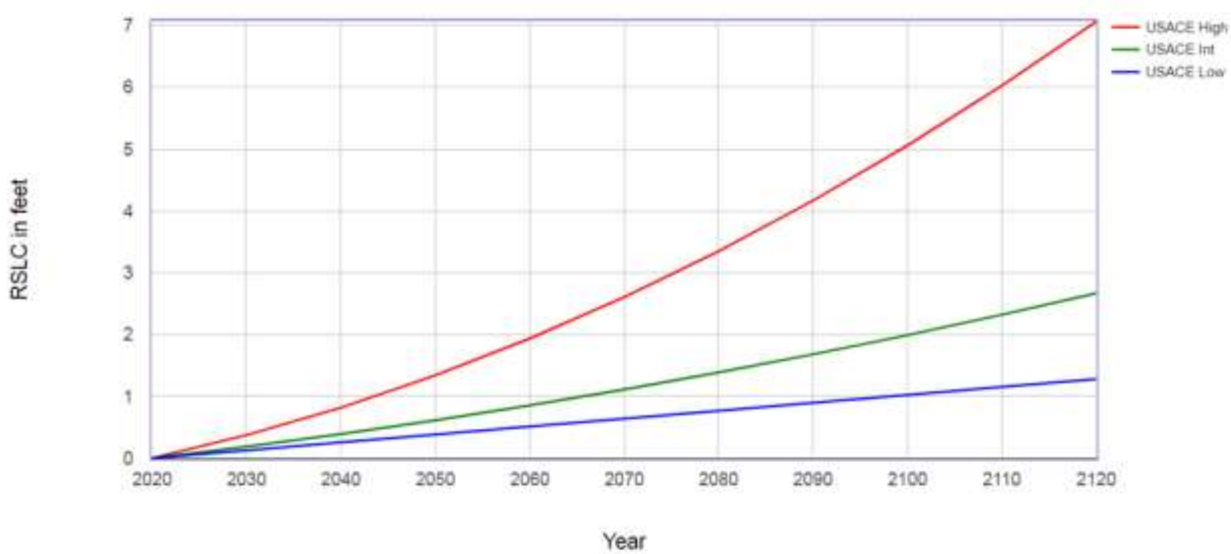

Figure 17: SLC Scenario Projections (Sandy Hook, NJ)

6.5 Interior Drainage Analysis

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.*

6.5.1 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.

6.5.2 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, respectively, of the H&H Appendix.

6.6 Recommended Plan - Interior Drainage Plan

The Recommended Plan's interior drainage plan is defined as the plan that maximizes the net excess benefits over cost. 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 alternatives were evaluated and compared to determine which contributes the highest level of net excess benefits to the project. The interior drainage component for each sub-basin is presented in **Table 5** and shown in **Figure 18**.

Table 5: **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

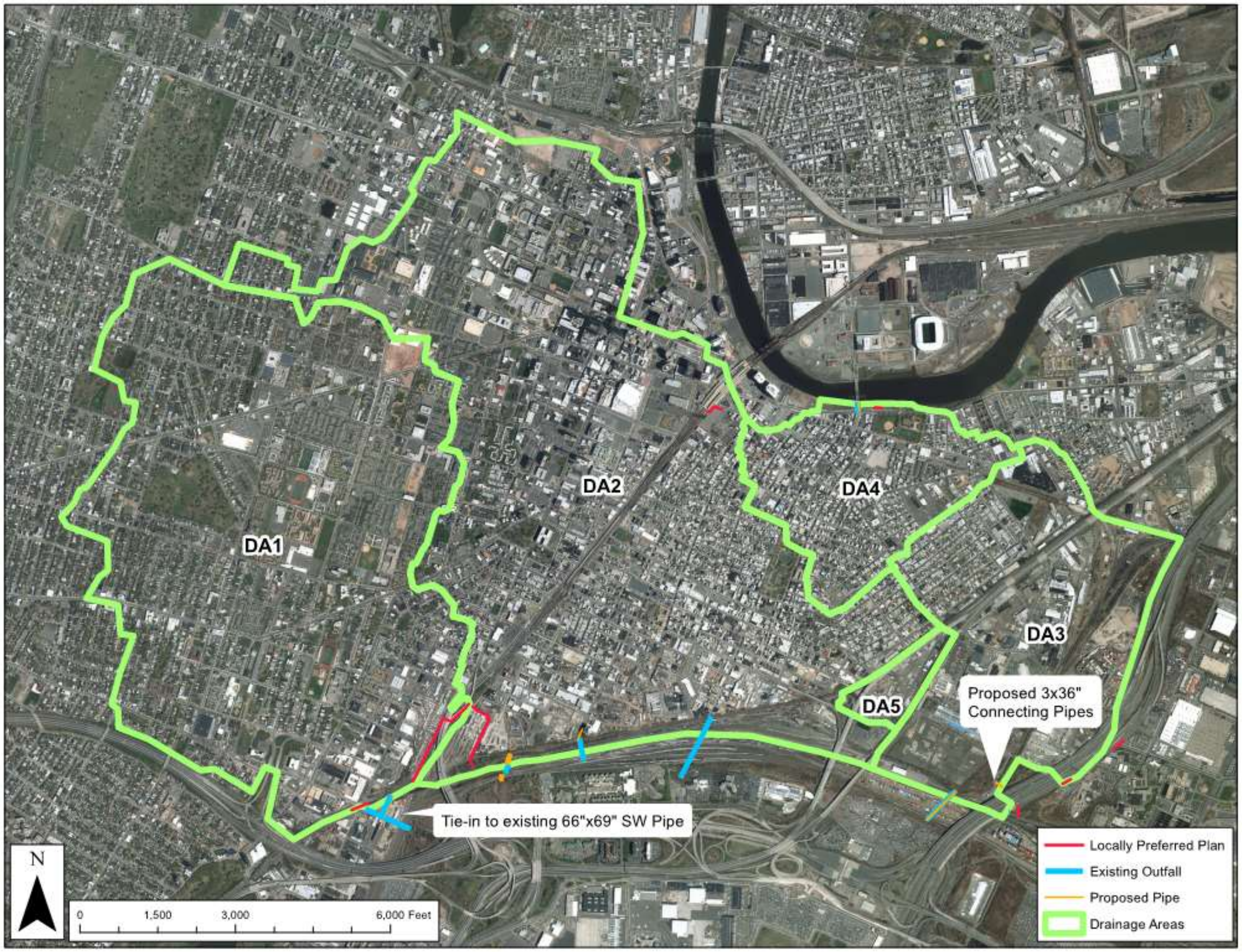

Figure 18: Interior Drainage Plan

7 BACKFLOW PREVENTION – EXISTING DRAINAGE STRUCTURES

7.1 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 6** and **Figure 19** 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 6** and **Figure 19**.

Table 6: CSS and Stormwater Backflow Prevention Locations

Type	Name	Description	Location
Stormwater	Stormwater 5	15-inch Pipe	Railyard at end of NJRR Avenue (Segment 2)
	Stormwater 6	66” x 69” Pipe	North of East Peddie Street
	Avenue C	36-inch Pipe	End of Avenue C
	Pierson Creek 2	4’ x 8’ Box	Vicinity of Segment 3
CSS	Wheeler 1	46” x 96” Ellipse	Vicinity of Avenue A (Segment 2)
	Adams 1	46” x 96” Ellipse	End of Adams Street (Drainage Area 2)

7.2 Sealing 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.

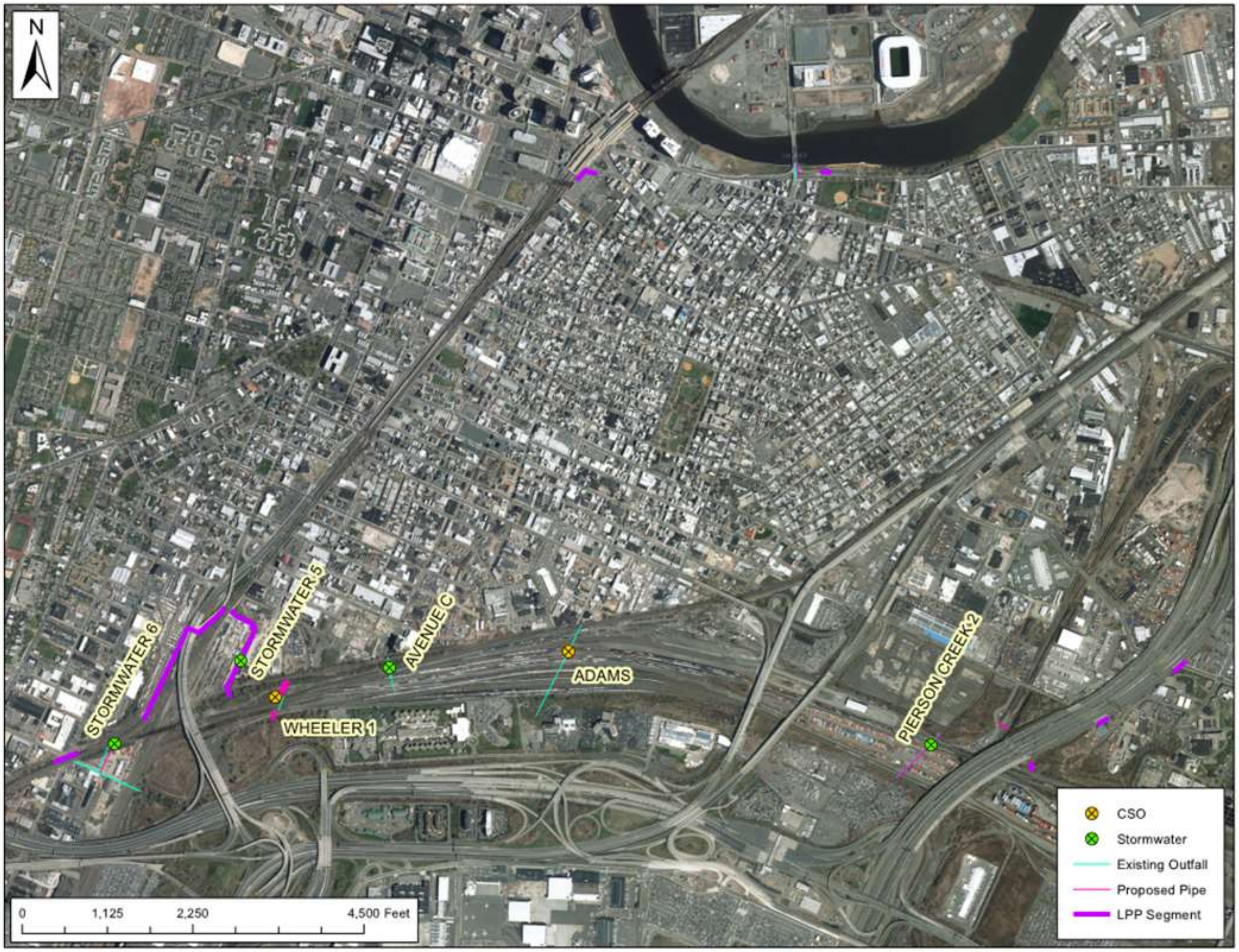

Figure 19: CSS/Stormwater Backflow Prevention

8 GEOTECHNICAL ANALYSIS

The following sections describe the geotechnical analysis associated with the Recommended Plan. The geotechnical analysis associated with the NED Plan is included in Subappendix 2.

The following two types of structures were considered for the Recommended Plan: 1) floodwall (T-and I-wall); and 2) earthen levee. The project area is divided into seven (7) segments, designated to Segment numbers 1 to 6, and 8. The flood alternatives were analyzed for flood elevation of +14 feet NAVD88. The analyses include seepage, lateral load and pile axial capacity analysis for floodwalls and flood gates, and seepage, slope stability and consolidation settlement analysis for the earthen levee. Liquefaction resistance was also evaluated for the floodwalls, gates and levee.

The summary of subsurface conditions or stratigraphy of both segments and soil properties used in this study are given in more detail in the Geotechnical Report (Subappendix 1).

8.1 Previous Subsurface Investigation

Based on the available subsurface information in New Jersey Department of Transportation soil borings database and a memorandum prepared by AECOM for the Passaic Valley Sewage Commission Wastewater Treatment Plant, Newark, New Jersey (2016), twenty two (22) borings near the proposed floodwall, flood gates, and levee alignment are considered in this analysis. The general locations of these borings are shown in **Figure 20**. In order to characterize the subsurface conditions of each segment, a representative stratification and set of soil properties were assigned to each segment after carefully examining the existing boring logs.

The depth, thickness, type and continuity of soil layers vary between the seven segment areas; therefore, site-specific stratification and soil properties were estimated for each area. The soil properties were estimated based on average standard penetration test (SPT) values from available boring logs in each area.

Sufficient information on the SPT hammer was not available on many of the borings to make energy corrections for conversion to N60, so blow counts of the second plus third 6-inch penetration intervals determined an uncorrected N-value for estimating soils property parameters. The drained parameters for organic soils were assumed. Corrections to N60 were considered for the liquefaction analyses in the next section. Ground line elevations where not given on some borings and were estimated from roadway surface elevations. The representative stratifications and soil properties for the seven segments are presented in **Tables 7 to 11**.

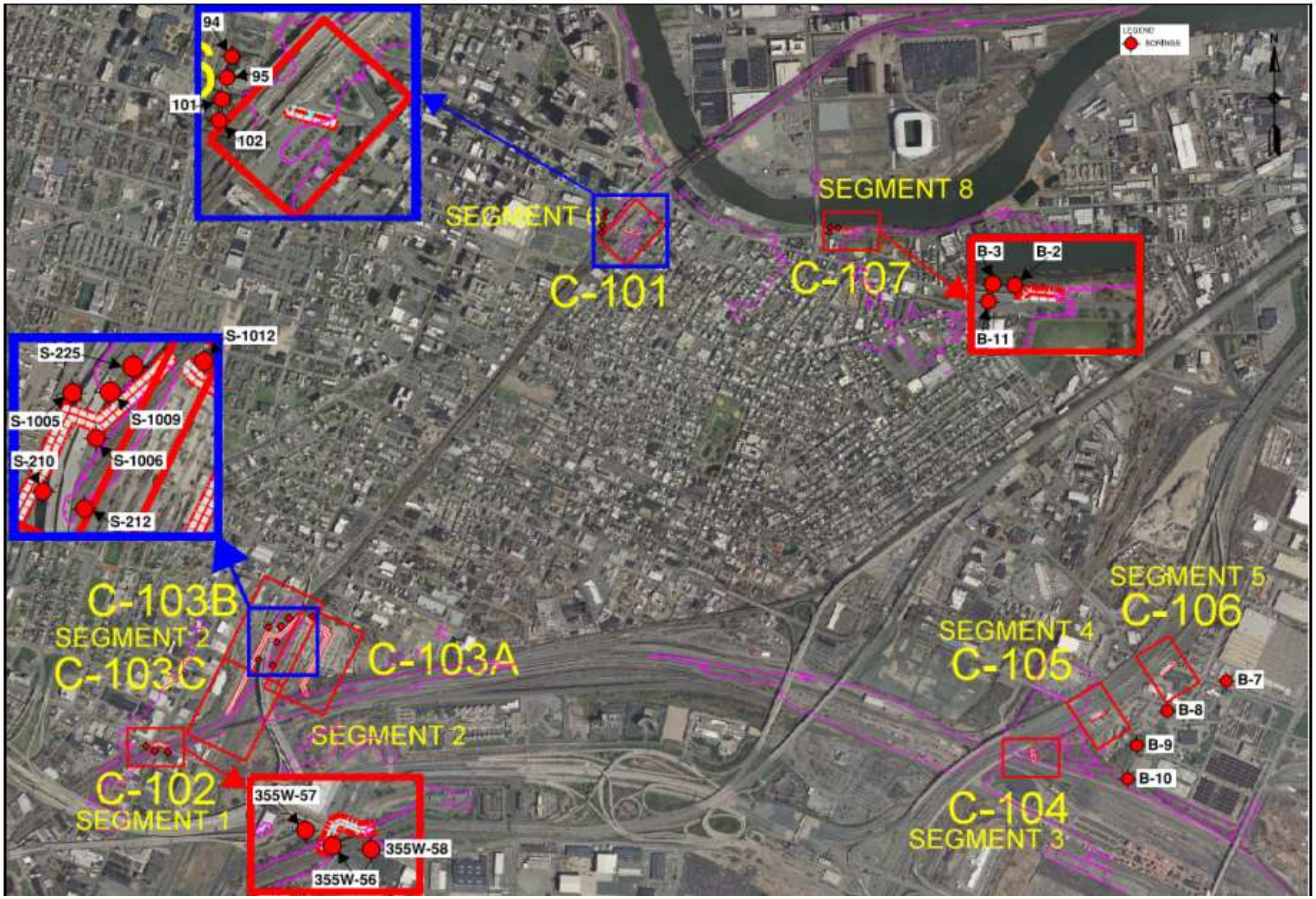

Figure 20: Recommended Plan Segments and Boring Locations

Table 7: Representative Stratification and Estimated Soil Properties for Segment 1

Stratum No.	Top Elevation (ft)	Bottom Elevation (ft)	Material	Unit Weight, γ (lb/ft³)	Friction Angle, φ (degree)	Cohesion, c (lb/ft²)	Hydraulic Conductivity, k (ft/sec)
1	10	0	Medium Sand/Gravel, Little/Some Silt (Fill)	120	29	0	3.28×10^{-4}
2	0	-4.5	Soft to Medium Organic Silt/Clayey Silt	90	Undrained: 0	250	3.28×10^{-6}
					Drained: 10	50	
3	-4.5	-	Dense Sand, Little/Trace Silt, Trace Gravel	125	35	0	3.28×10^{-6}

Table 8: Representative Stratification and Estimated Soil Properties for Segment 2

Stratum No.	Top Elevation (ft)	Bottom Elevation (ft)	Material	Unit Weight, γ (lb/ft³)	Friction Angle, φ (degree)	Cohesion, c (lb/ft²)	Hydraulic Conductivity, k (ft/sec)
1	13	5	Loose Sand, Little/Some Silt, Trace Gravel (Fill)	100	29	0	2.30×10^{-6}
2	5	0	Soft Organic Clayey Silt/Silty Clay (Peat)	90	Undrained: 0	250	3.28×10^{-6}
					Drained: 10	50	
3	0	-31	Loose to Medium Sand, Little/Some Silt, Trace Gravel	110	30	0	3.28×10^{-6}

Table 9: Representative Stratification and Estimated Soil Properties for Segments 3, 4, & 5

Stratum No.	Top Elevation (ft)	Bottom Elevation (ft)	Material	Unit Weight, γ (lb/ft³)	Friction Angle, φ (degree)	Cohesion, c (lb/ft²)	Hydraulic Conductivity, k (ft/sec)
1	14	-9.5	Loose Sand, Little/Some Silt, Trace Gravel, Debris (Fill)	100	29	0	2.30×10^{-6}
2	-9.5	-39.5	Very Stiff Sandy/Silty Clay	125	Undrained: 0	2,500	3.28×10^{-8}
					Drained: 22	200	

Table 10: Representative Stratification and Estimated Soil Properties for Segment 6

Stratum No.	Top Elevation (ft)	Bottom Elevation (ft)	Material	Unit Weight, γ (lb/ft^3)	Friction Angle, φ (degree)	Cohesion, c (lb/ft^2)	Hydraulic Conductivity, k (ft/sec)
1	17	9	Loose to Medium Sand/Silt, Trace Gravel, Debris (Fill)	110	29	0	3.28×10^{-5}
3	9	-25	Medium Sand, Trace/Little/Some Silt, Trace Gravel	120	32	0	3.28×10^{-6}

Table 11: Representative Stratification and Estimated Soil Properties for Segment 8

Stratum No.	Top Elevation (ft)	Bottom Elevation (ft)	Material	Unit Weight, γ (lb/ft^3)	Friction Angle, φ (degree)	Cohesion, c (lb/ft^2)	Hydraulic Conductivity, k (ft/sec)
1	11	1.5	Medium Sand, Little Silt, Trace Gravel (Fill)	120	29	0	3.28×10^{-5}
2	1.5	-2	Very Stiff Silt and Clay with Organics	90	Undrained: 0 Drained: 15	250 50	3.28×10^{-6}
3	-2	-44	Medium Sand, Little Gravel, Trace Silt	120	32	0	3.28×10^{-6}

8.2 Preliminary Information and Assumptions

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

1) The analyses and calculations performed as part of this study are preliminary in nature and all estimates were based on limited available data. The new subsurface investigation and laboratory testing program as recommended later in this section are necessary to meet USACE requirements for final design.
2) For pile depth calculations, rock depths vary along the alignment but pile lengths are assumed to be conservative.

8.3 Recommendations

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

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

8.4 Liquifaction Resistance

Factors of safety (FOS) against liquefaction for non-cohesive soils under the groundwater table at the seven segments were calculated. A design earthquake magnitude of Mw = 5.5 corresponding to 2% probability of exceedance in 50 years (return period ~ 2,475 years) was used in this evaluation based on the historic earthquake information in the northeast. Using the 2008 USGS seismic hazard maps, a peak ground acceleration (PGA) value of 0.32g was estimated for a 2,475 years seismic event.

In the analysis, the SPT-based simplified procedure outlined by Idriss and Boulanger (2008) was used for liquefaction evaluation of non-cohesive soils (e.g., sand and gravel) in the top 50 feet. The simplified procedure involves estimation of the seismic demand, expressed in terms of the cyclic stress ratio (CSR); and the capacity of the soil to resist liquefaction, expressed in terms of the cyclic resistance ratio (CRR). CSR at a particular depth is a function of the PGA, the total and effective vertical stresses at the depth of interest, and a shear stress-reduction coefficient. CRR is estimated based on clean sand corrected normalized SPT blow-counts, (N1)60, cs values.

A Magnitude Scaling Factor (MSF) was used to normalize the CRR values to the design earthquake magnitude. The CRR was also adjusted for overburden effects using the correction factor, $K\sigma$. Values of FOS against liquefaction were calculated dividing CRR by CSR. FOS of 1.2 was considered as the threshold value for the triggering of liquefaction according to AASHTO (2014). The fines content was estimated from the soil quantity descriptions based on the Burmeister classifications. However, the additional subsurface investigation will provide more accurate information on the site-specific fines content and may change the liquefaction analysis results. Details of the liquefaction evaluation are provided in Attachment B to the Geotechnical Report. The plot of FOS against liquefaction for each segment is also provided.

Based on the liquefaction evaluation, occasional pockets of potentially liquefiable soils exist in the area of Segment 2. The liquefaction is not a concern in other segments.

8.5 Floodwalls and Gates

The preliminary alignment for each segment is provided in **Figure 20**. The floodwall alternative was considered for all the segments. As a representative section for areas of floodwalls and gates, a T-Wall with height of 4 feet was considered for Segments 1, 4, 5, and 6. T-Walls supported on H-Piles with heights of 6 feet and 8 feet were considered for Segment 2. As an additional alternative, an I-Wall with height of 6 feet was considered for Segment 2. For Segment 6 and 8, T-Wall with height of 2 feet was also considered. If the existing soil is not suitable for construction, it must be replaced by proper structural fill. Bearing capacity and seepage analyses were performed for T-Walls. The sections of the T-wall and I-wall are provided in Figures 7 and 8 of the Geotechnical Report. The summary of proposed flood risk reductions systems is provided in **Table 12**. The design flood elevation was assumed to be elevation +14 feet NAVD88, and ground surface elevations were assumed to very between elevation +6 and +12 feet NAVD88.

Table 12: Summary of Proposed Flood Risk Reduction Systems for Each Segment

Segment #	Type of Structure	Top of Wall Elevation [NAVD] (ft)	Ground Elevation [NAVD] (ft)	Base Width (ft)	Wall Height (ft)
1	T-Wall or Gate Structure	14	10	12	4
2	T-Wall or Gate Structure or I-Wall	14	6 and 8	10 (T-Wall)	6 and 8
4	T-Wall or Gate Structure	14	10	10	4
5	T-Wall or Gate Structure	14	10	10	4
6	T-Wall or Gate Structure	14	10 and 12	6 and 10	2 and 4
8	T-Wall	14	12	6	2

8.6 Bearing Capacity

Based on the average N-values of the fill layer conventional bearing capacity estimates were performed. A more comprehensive bearing capacity calculation considering the lateral pressure will be done in the design phase of the project after performing the geotechnical investigation. The summary of allowable capacities is provided in **Table 13**.

Table 13: Summary of Bearing Capacities for Each Segment

Segment #*	Allowable Bearing Capacity (ksf)
1	1.0
3, 4&5	3.0
6	3.0
8	1.0

*Analysis of Segment 2 not needed.

8.7 Seepage and Sliding Stability Analyses

Steady state seepage analyses at full flood stage were performed for the floodwalls using the commercially available software GeoStudio 2007 SEEP/W by Geoslope International, Ltd., and following the guidelines in EM 1110-2-2502. The hydraulic conductivity values were assumed based on soil type and fines content. The assumed hydraulic conductivity values of each layer were provided in **Tables 7 to 11**. The maximum exit gradient and flow rate for the T-wall and I-wall at full flood stage are presented in **Table 14**. The estimated maximum gradients are lower than the allowable critical gradients, typically 0.5, according to EM 1110-2-2502. Based on the estimated critical gradients for 4 foot flood height, sheet pile cutoff is not required for T-walls or gate structures in Segments 1, 4, 5, 6, and 8. However, sheet pile cutoff is required to reduce the critical gradient in Segment 2 for flood heights 6 feet and 8 feet. Details of the seepage analyses for the T-walls are provided in Sheets C.1 to C.6 of Attachment C to the Geotechnical Report.

Table 14: Summary of Proposed Alignment for Each Floodwall Segment

Segment #	Type of Structure	Wall Height (ft)	Maximum Exit Gradient	Sheet Pile Cutoff	Sheet Pile Cutoff Length (ft)
1	T-Wall or Gate Structure	4	0.19	No	-
2	T-Wall or Gate Structure	6	0.22	Yes	10
		8	0.22	Yes	15
	I-Wall	6	0.16	Yes	-
4	T-Wall or Gate Structure	4	0.18	No	-
5	T-Wall or Gate Structure	4	0.18	No	-
6	T-Wall or Gate Structure	2	-	No	-
		4	0.03	No	-
8	T-Wall	2	-	No	-

Sliding stability analysis was performed to check the sliding within weak layers below the base of the T-wall. The vertical water pressure due to the flood was conservatively assumed to be a surcharge load on the ground surface. The minimum global stability safety factor obtained for the critical slipping surface is 5.50 which meets the minimum required value per EM 1110-2-2502. In this analysis, the lateral resistance of the foundation piles was conservatively neglected. Details of the sliding stability analyses for the T-walls are provided in Sheet C.7 of Attachment C to the Geotechnical Report.

8.8 Global Stability Analysis

The slope stability analyses for the T-wall in Segment 8 was performed using the commercially available software GeoStudio SEEP/W and SLOPE/W by Geoslope International, Ltd. This segment was selected because of the topography which is sloped from the wall towards the river and will be critical in terms of stability FOS. The other segments that have floodwall without pile foundation are 4 feet high but located on relatively flat ground and may not govern. The following four cases were considered in the analyses:

Case I: End of construction;

Case II: Steady seepage from full flood stage; fully developed phreatic surface;

Case II: Rapid drawdown from full flood stage; and,

Case IV: Seismic loading, no flood condition.

Spencer's procedure for the method of slices was used to determine the minimum FOS values and the critical slip surface associated with the FOS values for all four loading cases.

For Case I stability analysis, groundwater was modeled as provided in **Table 5**. Considering that Case I is a short-term scenario, undrained strength parameters were used for cohesive soil layers. The groundwater was at elevation +1.5 feet NAVD88 to be same as the Passaic River level.

Case II was analyzed at flood level elevation of +14 feet NAVD88 to estimate the conditions at a full flood stage. Seepage analysis was performed for this case to estimate flow and exit gradient characteristics and to develop the phreatic surface for use in the stability analyses.

Case III was performed to estimate the conditions when the water level adjacent to the riverside slope lowers rapidly. This case generally has a greater influence on soils with lower permeability since the dissipation of pore pressure is slower in these materials. For this case, the phreatic surface was conservatively modeled as in Case II while keeping the flood level lowered along the riverside slope to the toe.

Case IV (seismic loading) utilizes the pseudo-static slope stability analysis. The piezometric line was modeled the same as in Case I. It is standard practice to consider the pseudo-static coefficient as 2/3 of PGA/g. Accordingly, a pseudo-static coefficient of 0.21 (2/3x0.32g/g) estimated from 2008 USGS seismic hazard maps for return period of 2,475 years was estimated and used in the stability analyses. Further, it was assumed that liquefaction mitigation measures will be implemented if liquefaction is a concern. Details of the slope stability analyses for the T-wall in Segment 8 are provided in Sheets C.8 to C.11 of Attachment C to the Geotechnical

Report. The values of FOS associated with the critical slip surfaces are greater than the required minimum values as provided in **Table 15**.

Table 15: Slope Stability Analysis Results for 4-foot High T-Wall in Segment 2

Analysis Case	Required Minimum Factor of Safety (USACE)	Calculated Factor of Safety
Case I: End of Construction	1.3	2.9
Case II: Steady State – Full Flood Stage	1.4	4.5
Case III: Rapid Drawdown	1.0	1.7
Case IV: Seismic Load	1.0	1.1

8.9 Lateral Load Analysis

I-wall with 6 feet free height alternative was considered for Segment 2. I-wall was analyzed using PYWal by Ensoft, Inc. Long-term (drained) soil properties of the organic clay and clay layers were conservatively (higher active pressure on wall) used for the analysis. A summary of I-wall analysis results for Segment 2 is presented in **Table 16**. Considering a maximum allowable lateral deflection of 1 inch at the top and approximately zero inches of deflection at the tip of the wall, AZ14 sections are recommended for the sheet piles. A minimum sheet pile length of the free height of the wall plus 24 feet is recommended. Plots of lateral defection, bending moment and shear force with depths of sheet piles are provided in Attachment D of the Geotechnical Report.

Table 16: Results of the Sheet Pile Analysis for I-walls in Segment2

Segment #	Sheet Pile Section	Allowable Moment Capacity (kip-in)	Sheet Pile Length (ft)	Maximum Deflection (in)	Maximum Moment (kip-in)
2	AZ14	1910	24 (Below G.S)	0.35	35

8.10 Pile Axial Capacity Analysis

The geotechnical compression and tension capacities of the driven HP 12X53 and HP 14X73 piles were estimated for T-wall or gate structure in Segment 2 using the commercially available software APILE v2015 by Ensoft, Inc. and following the procedures outlined in the USACE, *Design of Pile Foundations*, EM 1110-2-2906. Skin friction from organic layer was ignored. A minimum factor of safety of 2.0 for compression was used assuming that the compression capacity will be verified by pile load test. The allowable compression and tension capacities of 50 foot long pile are provided in **Table 17**. The summaries of axial capacities are presented in Attachment E of the Geotechnical Report.

Table 17: Summary of Allowable Capacities of a 50-foot Long H-Pile

Pile Type	Pile Size	Pile Length (feet)	Est. Allowable Pile Compression Capacity (kips)	Est. Allowable Pile Tension Capacity (kips)
H-Pile	HP 12X53	50	63	41
	HP 12X73	50	81	50

8.11 Earthen Levee

An earthen levee was considered for Segment 3. The ground level at the alignment is approximately at elevation +6.0 feet NAVD88. Thus, the design height of the levee is 8 feet. Prior to the construction of the earth levee, the soil must be inspected down to 6 feet depth by excavating trenches. A typical levee cross-section with 8 feet height was selected for seepage and slope stability analyses.

8.11.1 Seepage and Slope Stability Analyses

Similar to the T-wall in Segment 8, the seepage and slope stability analyses for the earth levees performed using the commercially available software GeoStudio SEEP/W and SLOPE/W by Geoslope International, Ltd. and following the guidelines in USACE, *Design and Construction of Levees*, EM 1110-2-1913. The levee constructed with cohesionless structural fill with a clay core wall in the middle was considered in our analyses. The cross section of the levee used for the analysis is provided in Figure 9 of the Geotechnical Report. The details of the seepage and slope stability analyses for the earth levee are provided in Attachment F of the Geotechnical Report. As shown in Sheet E.1, the estimated maximum exit gradients are lower than the allowable critical gradients, typically 0.5, according to ETL 1110-2-569. The values of FOS associated with the critical slip surfaces are greater than the required minimum values, as shown in Sheets E.2 to E.6 in the Subappendix 1. The summary of the exit gradient from the seepage analysis and the factor of safety values obtained for the four cases are provided in **Tables 18 and 19**.

Table 18: Seepage Analysis Results for 8 foot High Levee for Segment 3

Segment #	Type of Structure	Wall Height (ft)	Maximum Exit Gradient
3	Levee	8	0.19

Table 19: Slope Stability Analysis Results for 8 foot High Levee for Segment 3

Analysis Case	Required Minimum Factor of Safety (USACE)	Calculated Factor of Safety
Case I: End of Construction	1.3	2.0
Case II: Steady State – Full Flood Stage	1.4	1.4
Case III: Rapid Drawdown	1.0	1.0
Case IV: Seismic Load	1.0	1.2

8.11.2 Settlement Analysis

Based on the generalized soil profile for Segment 3 as provided in **Table 9**, the top 15 to 45 feet of the natural soil in the flood protection area consists of sandy/silty clay. The immediate or elastic settlement of soils will take place during the construction. Therefore, settlement analysis was only performed to estimate the primary consolidation of the clayey soil layers.

The consolidation test data ($e_o = 0.94$ and $C_c = 0.18$) for sandy/silty clay for the present study was obtained from previous Geotechnical Report (Subappendix 2). In the settlement analysis, the compressible layers were divided into sub-layers of 1 feet thickness for obtaining better accuracy of calculations. Increase in vertical stresses at the mid depth of each layer due to the embankment load was calculated using the elastic stress distribution methods as outlined in Das. B. M. (2006).

The time rate of primary consolidation and secondary consolidation was not estimated in this analysis due to lack of sufficient deformation-time data. Additional consolidation testing on undisturbed sample(s) will be required for obtaining information regarding the rate of consolidation.

Based on the analysis, it is estimated that a total primary consolidation settlement of 5-inch will occur in the compressible soils at the project site due to the construction of 8 foot high levee. In order to minimize the effect of permanent settlement on the levee, the estimated 5-inch consolidation settlement can be added to the construction height of the levee. The detail of the consolidation settlement calculation is provided in Attachment G of the Geotechnical Report.

8.12 Conclusions and Recommendations

Following are the conclusions and recommendations based on the findings of this feasibility study level geotechnical analysis:

1) It is recommended to validate the soil profiles by performing a geotechnical investigation at each segment.
2) T-walls supported on shallow foundation are feasible from seepage standpoint for the 2 foot flood height in Segment 8 and 4 foot flood height in Segment 1, 4, 5, 6 & 8.

3) T-walls with sheet piles and pile foundations are recommended for the 6 and 8 foot flood heights for Segment 2.
4) I-walls are feasible for the 6 foot flood height for Segment 2.
5) Based on the results of seepage and global stability analyses, the levee alternative is feasible for flood height of 8 foot for Segment 3, where no organic soil was identified in the soil profiles.
6) In order to minimize the effect of permanent settlement on the levee, the estimated 5-inch consolidation settlement can be added to the construction height of the levee.

9 SURVEYING, MAPPING AND OTHER GEOSPATIAL DATA

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

10 FLOODWALL DESIGN

10.1 General

This design criteria addresses the design of tidal floodwalls in typical reaches along the Passaic River extending in Newark, NJ. The design elements defined herein represent a feasibility design using the best available information. The analysis is limited to foundation stability. Soil founded T-walls and gate monoliths are proposed to minimize impact on subsurface utilities where soil capacity is equal or in excess of 1,000 psf. Pile foundations are proposed to provide stability against overturning, sliding and flotation resistance where soil bearing capacity is insufficient for soil founded foundations. Sheet pile I-wall is proposed in these areas with pile supported T-wall being proposed where wall height exceeds 6 feet. Soil conditions in the area are limited and are based on current information (see the Geotechnical Report); pile lengths must be refined as more soil data becomes available. The SWEL is assumed to be at the TOW elevation 14.0 feet NAVD88. The typical ground elevation is assumed to range from 6.0 NAVD88 to 12.0 feet NAVD88 throughout the project.

10.2 Codes and Standards

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

AASHTO, American Association of State Highway and Transportation Officials, LRFD Bridge Design 8th Edition, 2017.

ACI 318-14 American Concrete Institute, Building Code Requirements for Structural Concrete.

ACI 350-06 American Concrete Institute, Environmental Engineering Concrete Structures.

AISC, American Institute of Steel Construction, Inc., Manual of Steel Construction, 15th Edition.

ASCE 7-10 American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures.

ASTM, American Society for Testing and Materials.

AWS D1.1-15 American Welding Society, Structural Welding Code, latest edition.

Hurricane and Storm Damage Risk Reduction Systems Design Guidelines (HSDRRSDG), June 2012

USACE EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures.

USACE EM 1110-2-2502, Retaining and Floodwalls.

USACE EM 1110-2-2906, Design of Pile Foundations.

USACE ETL 1110-2-584, Design of Hydraulic Steel Structures.

USACE ETL 1110-2-575, Evaluation of I-Walls.

10.3 General Design Load Parameters

10.3.1 Load Combinations

The feasibility design includes two basic load cases, the construction load case and the water to TOW case; these are the loadings that typically control floodwall designs. Other loadings must also be analyzed in the final design, including Seismic Load Cases for both operating and maximum earthquake conditions. Typically, on inland waterways, when the wall is overbuilt to include uncertainty and sea-level rise the static head to top of wall is similar in force to that imparted by a wave and are sufficiently close for feasibility-level designs. Some of the load cases that will be included in the final design are:

1a. <u>Construction</u>. Dead load of the concrete wall components, no earthen backfill, no uplift. A 17 % overstress is permitted for this load case.

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

2a. <u>Flood Stage with Water to Top of Wall, Impervious Cutoff.</u> Dead load of concrete wall, At-Rest lateral earth pressures, and hydrostatic loading for water to the TOW; Uplift forces assume the sheet pile to be impervious. Wave force is not included. A 33% overstress is permitted.

2b. <u>Flood Stage with Water to Top of Wall, Pervious Cutoff.</u> Dead load of concrete wall, At-Rest lateral earth pressures, and hydrostatic loading for water to the TOW; Uplift forces assume the sheet pile to be pervious varying linearly from flood side TOW elevation to the ground water elevation on the protected side. Wave force is not included. A 33% overstress is permitted.

3a. <u>Flood Stage at Stillwater, Debris Impact Load, Impervious Cutoff.</u> Loadings include: Dead load of concrete wall, At-Rest lateral earth pressures, and hydrostatic loading for water to the design elevation. Uplift forces assume the sheet pile to be impervious. A debris load of 500lbs/LF is applied at the design elevation. Wave force is not included. A 33% overstress is permitted.

The overstress factors listed in each load case above reflect the stress levels permitted in the Hurricane and Storm Damage Risk Reduction Systems Design Guidelines (HSDRRSDG) that were developed for the New Orleans District post-Katrina and considered applicable for this flood risk management project

10.3.2 Hydraulic Stages

Design elevations are shown in **Table 20**.

Table 20: **Hydraulic Stages and Design Water Surface Elevations**

Stage (NAVD88)	Flood Side (NAVD88)	Protected Side (NAVD88)
TOW El 14.0		
TOW Water	EL. 14.0	EL. 6.0

TOW – Top of Wall

10.4 Load Cases

10.4.1 Dead Loads (D)

Dead loads shall be determined in accordance with applicable engineering manuals and ASCE 7-10, and shall include the self-weight of all permanent construction components including foundations, slabs, walls, roofs, actual weights of equipment, overburden pressures, and all permanent non-removable stationary construction. Applicable unit weights are shown in **Table 21**.

Table 21: Unit Weights

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

10.4.2 Live Loads (L)

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

10.4.3 Live Load Surcharge (LS)

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

10.4.4 Soil Pressures (S)

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

Lateral Soils at-rest Pressure Coefficients:

K_O = 0.53 for Granular Material.

10.4.5 Hydrostatic Loads (H)

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

10.4.6 Uplift Loads (U)

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

10.4.7 Wind Loads (W)

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

10.5 Concrete Design Criteria

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

Structural Concrete: 4,000 psi @ 28 days with a maximum water/cement ratio = 0.40

Steel reinforcement: 60,000 psi (ASTM A615)

10.6 Steel Design Criteria

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

(a) Structural steel rolled shapes	ASTM 572, Grade 50
	ASTM A992, Grade 50
(b) Plates	ASTM A36, Grade 36
(c) Bolts and nuts	ASTM A325, min. ¾ inch
	ASTM A490
(d) Anchor Bolts	ASTM A449, (¾ inch diameter and/or greater)
(e) Corrosion stainless steel	ASTM A304 (freshwater)
	ASTM A316 (saltwater)
(f) Sheet Piles	ASTM A328, Grade 50
	ASTM A572, Grade 50
(g) Stainless Steel Embedded Anchors	ASTM A276 or UNS S21800

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

10.7 Pile Foundation Design Criteria

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

Piles may be micro-piles with continuous casings to bedrock, steel pipe piles, steel H piles or pre-stressed concrete. Pipe piles satisfy ASTM A252 with minimum yield strength of 45 ksi. H-piles satisfy Grade 50 Steel. Steel piles are designed structurally per AISC ASD, 14th Edition, as modified by EM 1110-2-2906. Concrete square piles have a design strength equal to 6,000 psi at 28 days, pre-stressing strands are Low-Lax, Grade 270. Pres-stressed concrete piles are designed to satisfy both strength and serviceability requirements. Strength design follows the basic criteria set forth by ACI, except the strength reduction factor is 0.7 for all failure modes and the load factor is 1.9 for both dead and live loads. The pre-stressed concrete pile is designed for an axial strength limited to 80 percent of pure axial strength and a minimum eccentricity equal to 10 percent of the pile width. Control of cracking is achieved by limiting the concrete compressive stress to 0.4f’c and the tensile stress to zero. Combined axial and bending are considered when analyzing the stresses in the piles.

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

10.8 Floodwall Type by Segment

Figures 21 through 28 detail the proposed floodwall type at each project segment.

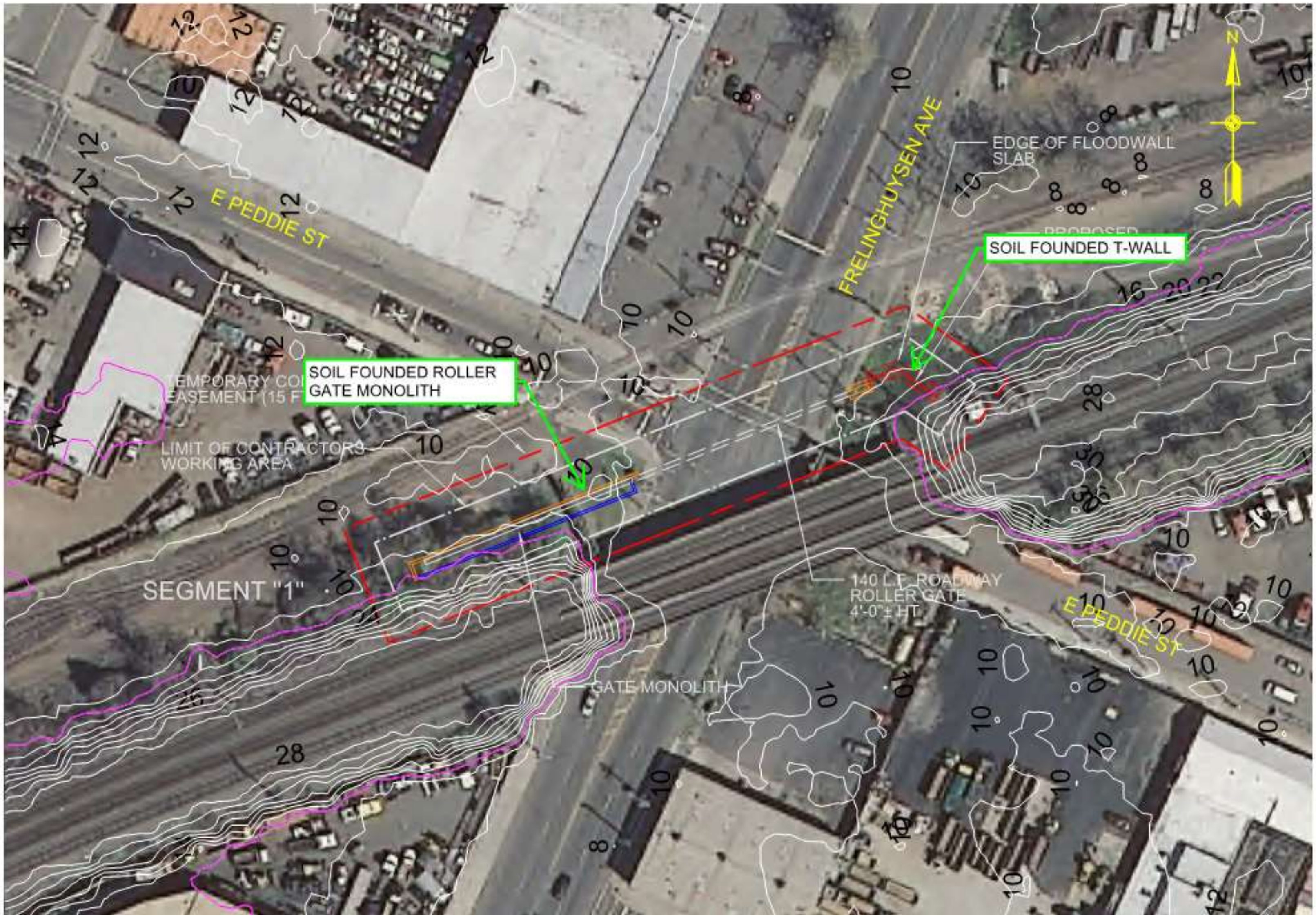

Figure 21: Segment 1 - Floodwall Type

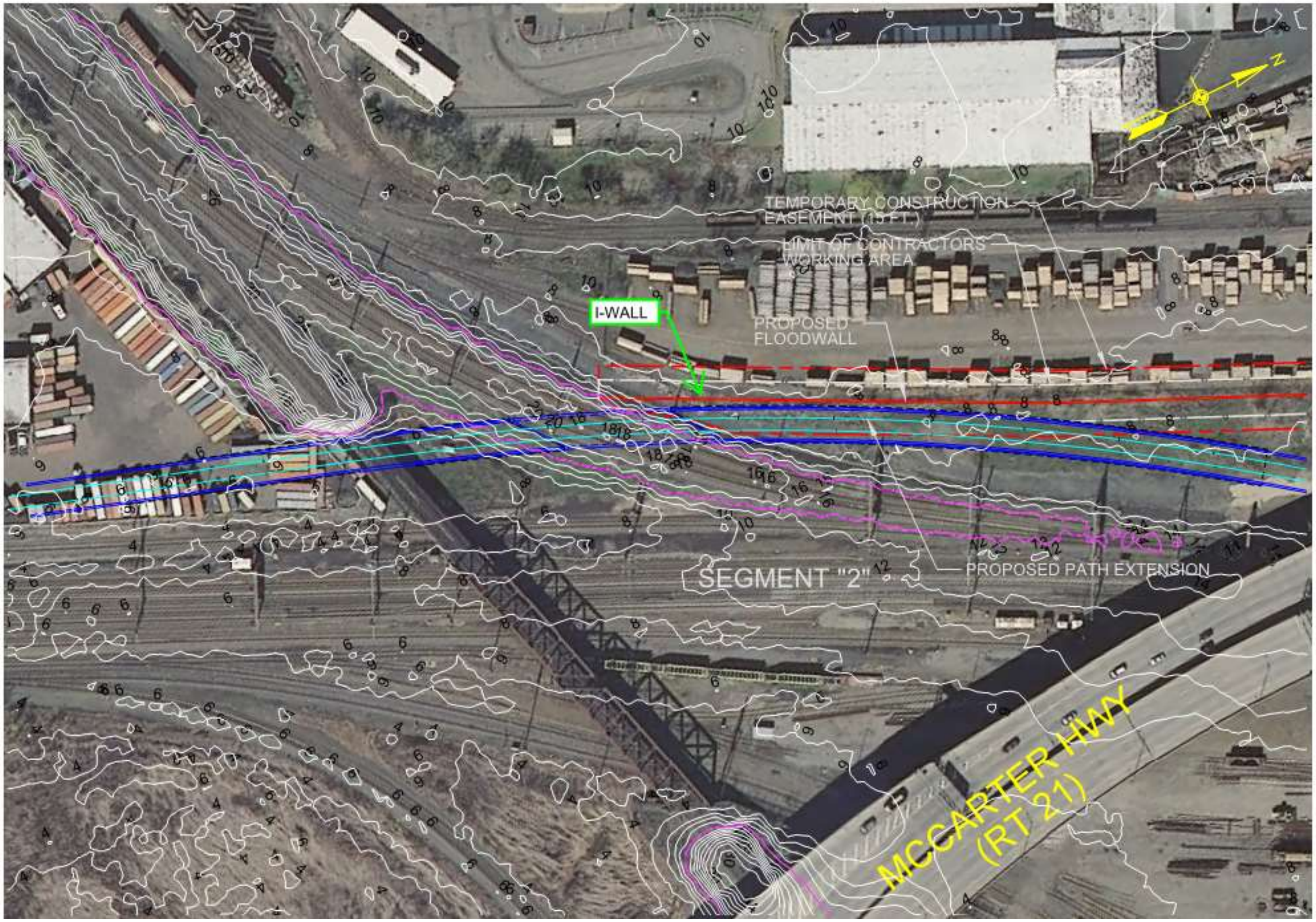

Figure 22: Segment 2A (South) - Floodwall Type

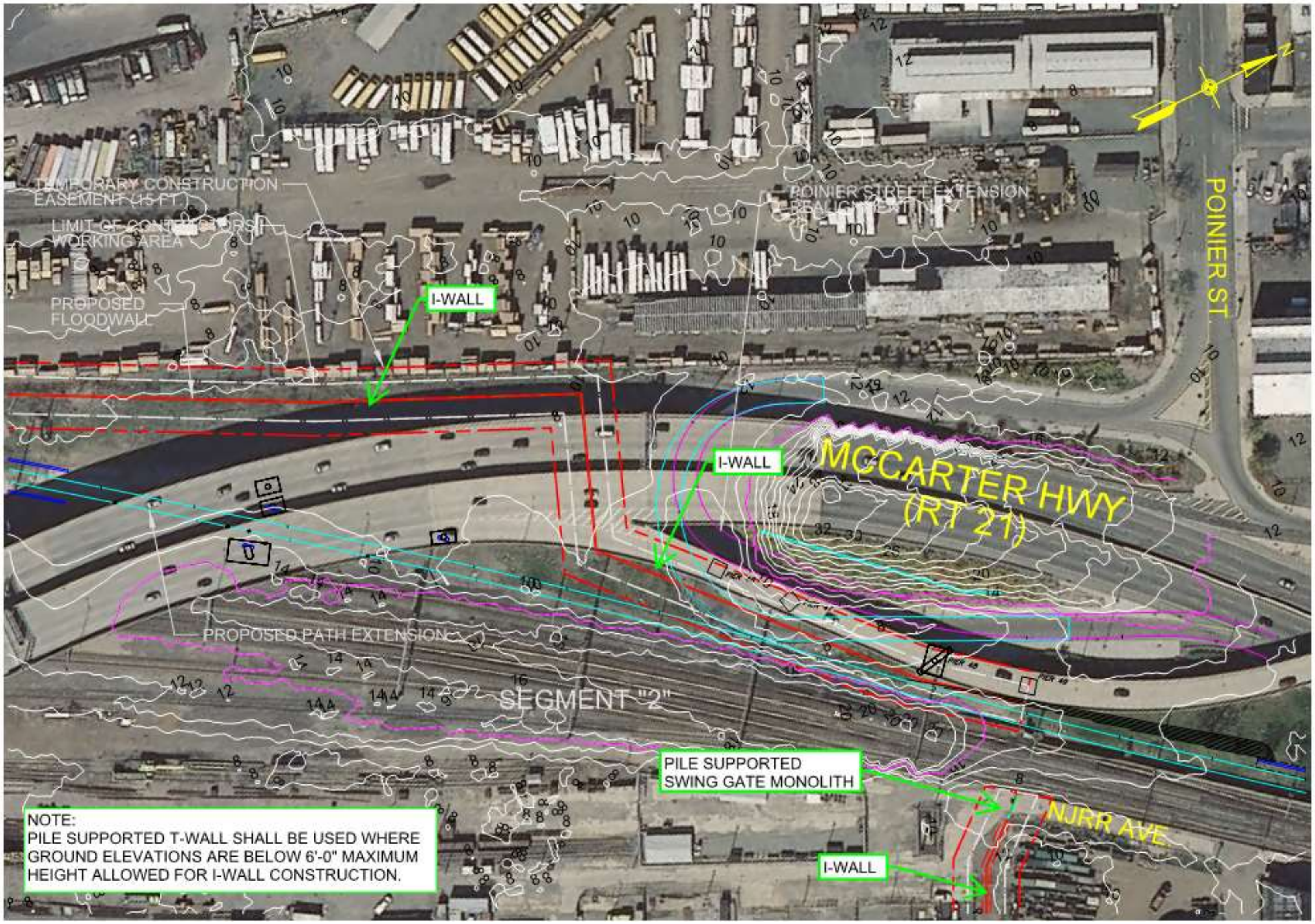

Figure 23: Segment 2A (North) - Floodwall Type

Figure 24: Segment 2B - Floodwall Type

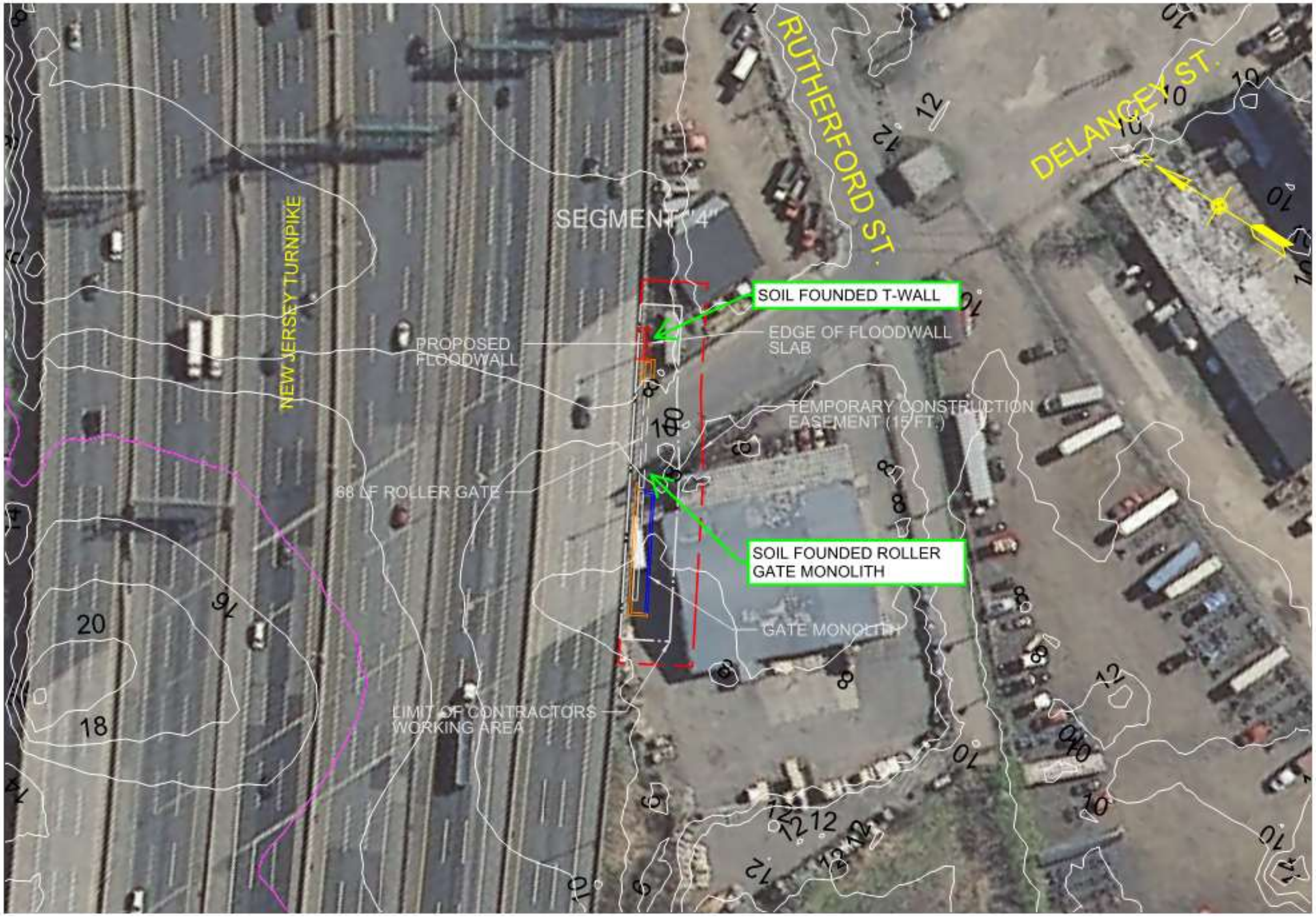

Figure 25: Segment 4 – Floodwall Type

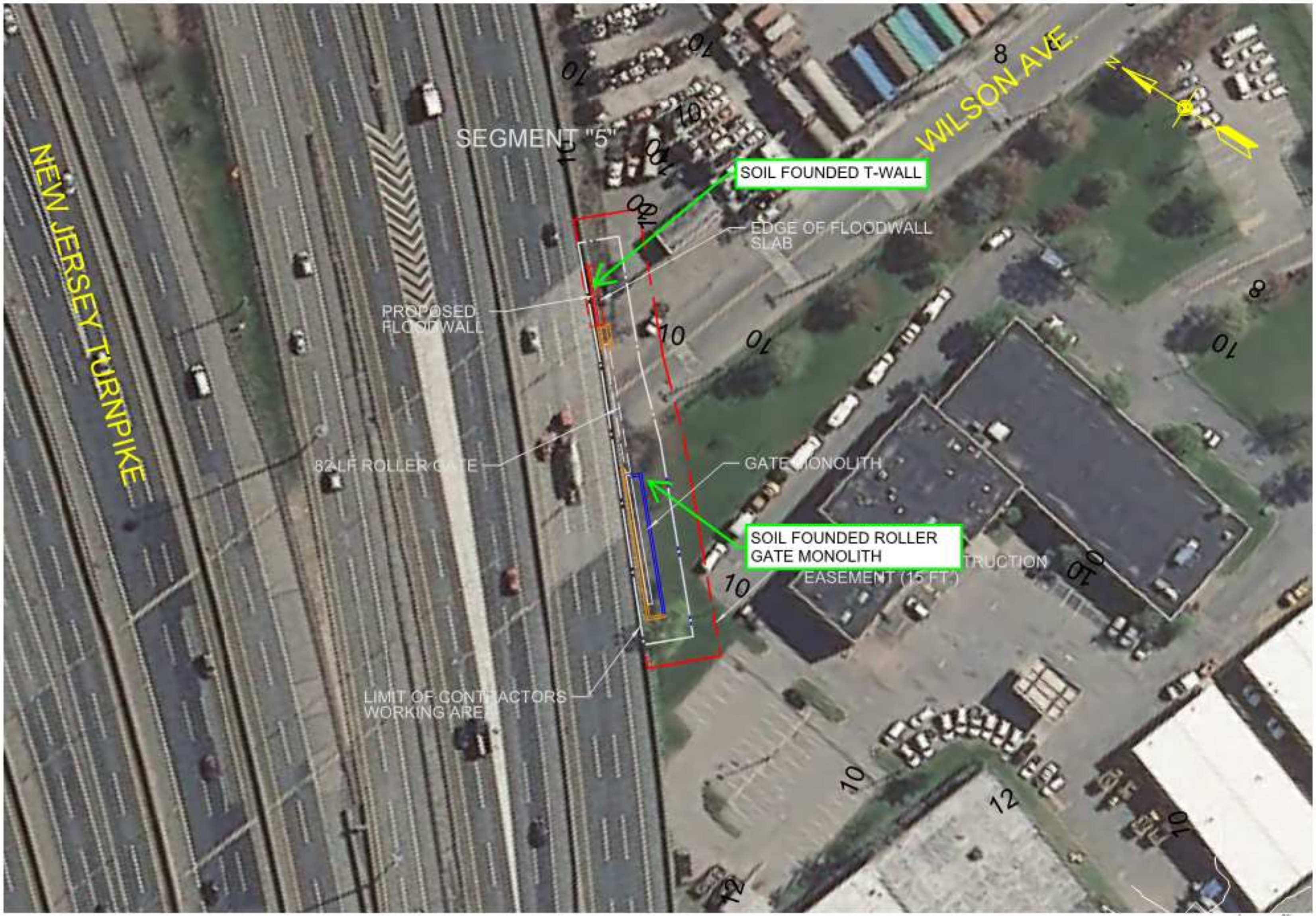

Figure 26: Segment 5 – Floodwall Type

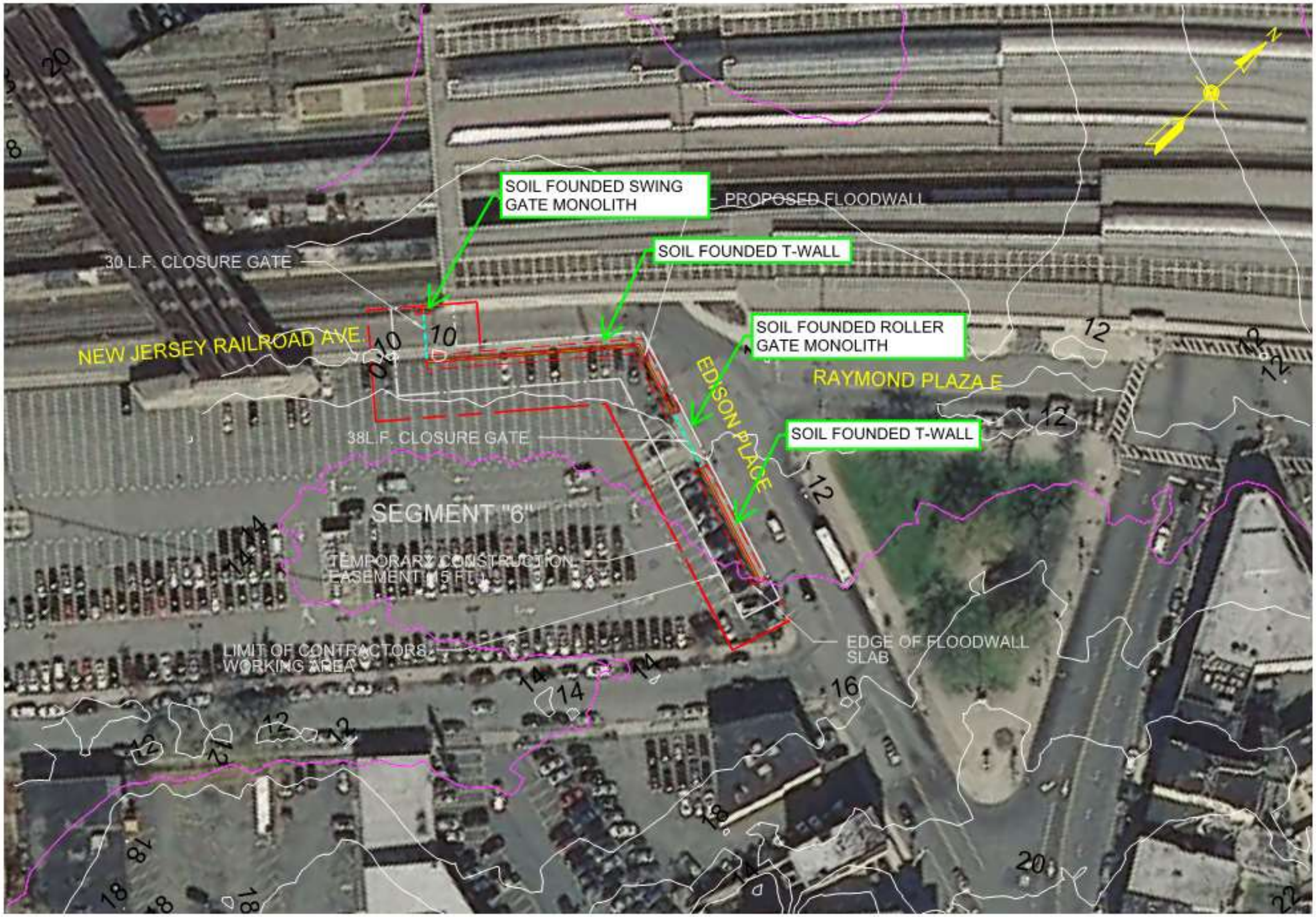

Figure 27: Segment 6 – Floodwall Type

Figure 28: Segment 8 – Floodwall Type

11 CLOSURE GATE DESIGN

11.1 General

There were 64 closure gates in the NED Plan alignment and eight in the Recommended Plan. The gates in the NED plan were mostly exterior gates associated with access through the alignment to the waterfront. The gates in the Recommended Plan are primarily roadway gates. The inventory of the gates in the Recommended Plan is shown in **Table 22** and the project drawings. The gate types used were both swing and roller gates.

Table 22: Recommended Plan Gates

Segment	Gate Type / Size (Length x Height)	Location
Segment 1	Roller / 140ft x 4ft	Intersection of Frelinghuysen Avenue and East Peddie Street
Segment 2	Swing / 30ft x 4ft	NJRR Avenue
Segment 2	Swing / 51ft x 8ft	Railroad
Segment 2	Swing / 50ft x 8ft	North of Railroad - Drainage
Segment 4	Roller / 68ft x 6ft	Delancy Street
Segment 5	Roller / 82ft x 4ft	Wilson Street
Segment 6	Swing / 30ft x 4ft	NJRR Avenue
Segment 6	Roller / 30ft x 2ft	Parking Lot

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

1) Construction + Wind: Dead load of the concrete monolith and steel gate, a conservative wind load of 50 psf, no earthen backfill, no uplift, no construction surcharge. A 33% overstress is permitted for this load case.
2) Flood stage two feet below top of gate structure with debris impact loading of 500 lbs/ft applied at the SWEL. A 33% overstress is permitted for this load case.
3) Flood stage at water to the top of gate (TOG).Wave force is not included. A 33% overstress is permitted for this load case.
4) Flood stage two feet below top of gate structure. A zero percent overstress is permitted for this load case.

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

For the design effort, the following codes and standards were used, as well as the applicable portions of the HSDRRSDG and the existing project GDM:

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

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

11.2 Gate Design

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

The analysis of the steel gate and concrete monolith was performed based on the load cases noted in the introduction. The governing load case was typically the flood stage with water at the top of the gate. Loads were applied as hydrostatic pressures corresponding to the water surface elevations on the flood side. The skin plate was designed as a fixed end beam spanning between the vertical intercostals and the deflection was limited to 0.4 of the thickness to ensure that the flat plate theory is applicable. The horizontal girders were designed as larger wide flange simply supported beams spanning between the bearing points on the concrete columns making them true beam elements allowing for flexural stresses. The vertical intercostals were designed as simple beams spanning between horizontal girders. The vertical intercostals consist of a WT section welded to the skin plate and were designed as a combined section utilizing the steel skin plate as the tension flange of the total combined section. The analysis of the reinforced concrete monolith walls and columns was performed considering fixed support at the interface of the bottom of the wall and top of slab. The wall analysis considered a 1 foot unit width of the wall acting as a cantilever and connected only to the base slab. The column analysis considered half of the gate width and width of the column loading on the column acting as a cantilever and connected` only to the base slab.

12 PUMP STATIONS

12.1 NED Plan – Interior Drainage

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

12.2 Recommended Plan - Interior Drainage

The Recommended Plan interior drainage plan does not include pump stations.

13 UTILTIES RELOCATION/PROTECTION

There is currently insufficient detail to accurately estimate the scope and cost for utilities relocations and/or protections for features passing through the proposed alignment. Therefore, a reasonable cost allotment for typical utility relocations was included in the cost estimate. Uncertainty in the quantity of features such a pipe sleeves through or under the floodwall were considered in the Cost and Schedule Risk Analysis.

14 DESIGN AND CONSTRUCTION SCHEDULE

The preliminary design and construction schedule is shown in **Figure 29**.

ID	Task Name	Duration	Start	Finish
1	**Passaic River Tidal GRR (LPP)**	**751 days**	**Mon 6/3/19**	**Mon 4/18/22**
2	**NOTICE TO PROCEED**	**1 day**	**Mon 6/3/19**	**Mon 6/3/19**
3	Field Survey	60 days	Tue 6/4/19	Mon 8/26/19
4	Geotechnical Survey	60 days	Tue 6/4/19	Mon 8/26/19
5	HTRW Assessment	60 days	Tue 6/4/19	Mon 8/26/19
6	Cultural/F&W Assessment	60 days	Tue 6/4/19	Mon 8/26/19
7	**PRECONSTRUCTION ENGINEERING & DESIGN**	**590 days**	**Tue 8/27/19**	**Mon 11/29/21**
8	**30% P&S All Segments**	**110 days**	**Tue 8/27/19**	**Mon 1/27/20**
9	DDR - 30%	60 days	Tue 8/27/19	Mon 11/18/19
10	P&S - 30%	60 days	Tue 8/27/19	Mon 11/18/19
11	30% DQC review	20 days	Tue 11/19/19	Mon 12/16/19
12	VE Study	30 days	Tue 12/17/19	Mon 1/27/20
13	**I-95 Section: Plans and Specs**	**240 days**	**Tue 1/28/20**	**Mon 12/28/20**
14	DDR - 60%	60 days	Tue 1/28/20	Mon 4/20/20
15	1 - P&S - 60%	60 days	Tue 1/28/20	Mon 4/20/20
16	60% DQC Review	20 days	Tue 4/21/20	Mon 5/18/20
17	DDR - 90%	40 days	Tue 5/19/20	Mon 7/13/20
18	1 - P&S - 90%	40 days	Tue 5/19/20	Mon 7/13/20
19	90% DQC, BCOES Reviews	20 days	Tue 7/14/20	Mon 8/10/20
20	Revised 90% for review	20 days	Tue 8/11/20	Mon 9/7/20
21	90% ATR & IEPR Reviews	40 days	Tue 9/8/20	Mon 11/2/20
22	1 - 100% DDR and P&S	40 days	Tue 11/3/20	Mon 12/28/20
23	**Minish Section: Plans and Specs**	**220 days**	**Tue 1/28/20**	**Mon 11/30/20**
24	DDR - 60%	60 days	Tue 1/28/20	Mon 4/20/20
25	2 - P&S - 60%	60 days	Tue 1/28/20	Mon 4/20/20
26	60% DQC Review	20 days	Tue 4/21/20	Mon 5/18/20
27	DDR - 90%	40 days	Tue 5/19/20	Mon 7/13/20
28	2 - P&S - 90%	40 days	Tue 5/19/20	Mon 7/13/20
29	90% DQC, BCOES Reviews	20 days	Tue 7/14/20	Mon 8/10/20
30	2 - Revised 90% for review	20 days	Tue 8/11/20	Mon 9/7/20
31	90% ATR & IEPR Reviews	20 days	Tue 9/8/20	Mon 10/5/20
32	2 - 100% DDR and P&S	40 days	Tue 10/6/20	Mon 11/30/20
33	**Southwest Section: Plans and Specs**	**260 days**	**Tue 1/28/20**	**Mon 1/25/21**
34	DDR - 60%	60 days	Tue 1/28/20	Mon 4/20/20
35	3 - P&S - 60%	60 days	Tue 1/28/20	Mon 4/20/20
36	60% DQC Review	20 days	Tue 4/21/20	Mon 5/18/20
37	DDR - 90%	60 days	Tue 5/19/20	Mon 8/10/20
38	3 - P&S - 90%	60 days	Tue 5/19/20	Mon 8/10/20
39	90% DQC, BCOES Reviews	20 days	Tue 8/11/20	Mon 9/7/20
40	3 - Revised 90% for review	20 days	Tue 9/8/20	Mon 10/5/20
41	90% ATR & IEPR Reviews	40 days	Tue 10/6/20	Mon 11/30/20
42	3 - 100% DDR and P&S	40 days	Tue 12/1/20	Mon 1/25/21
43	Real Estate Acquisition	130 days	Tue 1/26/21	Mon 7/26/21
44	Railroad Coordination	320 days	Tue 5/5/20	Mon 7/26/21
45	Bid and Award	90 days	Tue 7/27/21	Mon 11/29/21
46	**CONTRACT 1 - CONSTRUCTION ACTIVITIES**	**258 days**	**Mon 11/29/21**	**Thu 11/24/22**
47	Southwest Construction	12 emons	Mon 11/29/21	Thu 11/24/22
48	I-95 Construction	12 emons	Mon 11/29/21	Thu 11/24/22
49	Minish Construction	6 emons	Mon 11/29/21	Sat 5/28/22

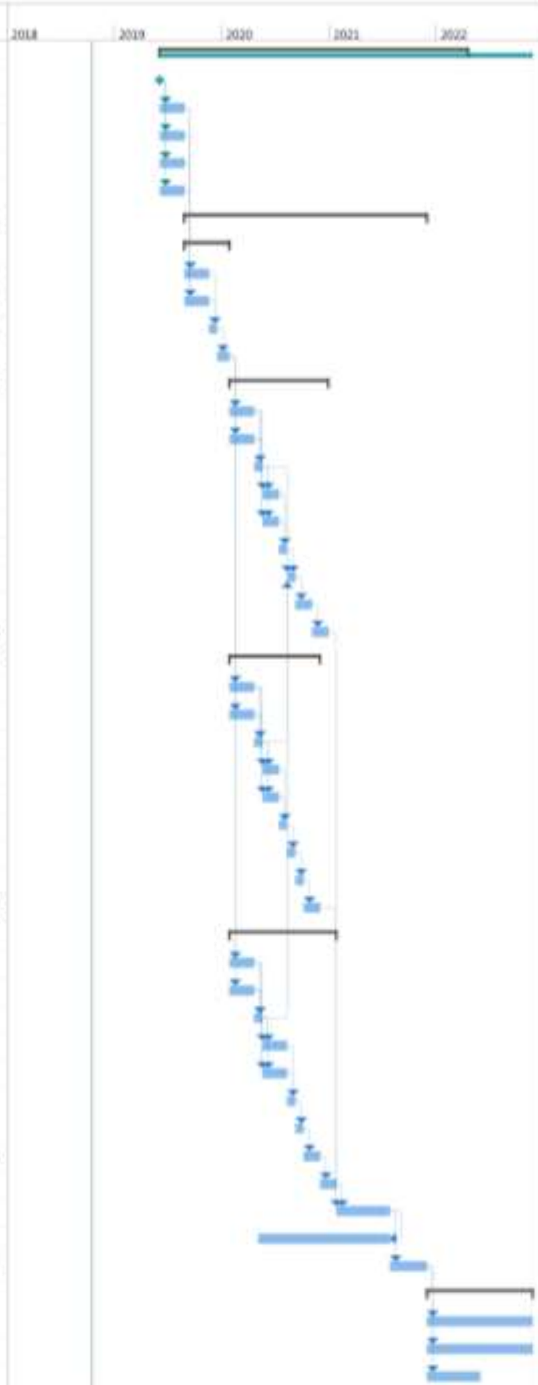

Figure 29: Recommended Plan Design and Construction Schedule

15 ADDITIONAL ANALYSES AND DATA COLLECTION

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

15.1 Geotechnical Needs

In order to obtain a better understanding of the subsurface condition and more accurate soil physical properties at each segment location, additional field investigation and lab testing need to be performed for the final design.

15.2 Field Survey Needs

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

1) Detailed topographic surveys along the Recommended Plan alignment and in the locations of project features will be required to support 30-scale design drawings.

2) Detailed utilities surveys along the project segments and proposed drainage features will be required.

3) Survey of manholes and other structures that may contribute to tidal surcharge conveyance behind the alignment and will need to be sealed.

15.3 Interior Drainage Refinement

The interior drainage analysis should be revisited with more detailed information regarding the capacity of the City’s existing combined sewer system (CSS). The current analysis included an estimate of the CSS initial capacity or abstraction. The remaining runoff contributed to residual ponding with in the project area. Refinement of the initial abstraction will help to better define the proposed interior drainage features.

16 PERMITS AND APPLICATIONS

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

1) New Jersey Flood Hazard Area,
2) Individual Freshwater Wetlands,
3) General Permit 12 (GP-12) Survey and Investigating,
4) Soil Erosion and Sediment Control,
5) New Jersey Pollutant Discharge Elimination System,

6) New Jersey Department of Transportation permits,
7) Treatment Works Approval (TWA) for any modifications to existing sanitary sewers.

17 EMERGENCY ACTION PLAN

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

18 OPERATION AND MAINTENANCE

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

19 REFERENCES

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

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

3. Idriss, I. M., & Boulanger, R. W. (2008). Soil liquefaction during earthquakes. Earthquake Engineering Research Institute.

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

5. "AASHTO LRFD Bridge Design Specifications", 7th Edition, American Association of State Highway and Transportation Officials, 2014.

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

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

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

9. United States Army Corps of Engineers (1989), "*Engineering and Design: Retaining and Flood Walls*", *EM 1110-2-2502*, USACE, Washington, DC.

SUBAPPENDIX 1
Recommended Plan Geotechnical Report and Drawings

SUBAPPENDIX 2
NED Plan Geotechnical and Structural Analysis, and Drawings

SUBAPPENDIX 2
2.1: NED Plan Geotechnical Analysis

SUBAPPENDIX 2
2.2: T-Wall Structural Analysis

SUBAPPENDIX 2
2.3: Closure Gates

SUBAPPENDIX 2
2.4: NED Plan Drawings