US Army Corps of Engineers®
New York District

Passaic River Tidal Protection Area, New Jersey
Coastal Storm Risk Management
Feasibility Study

Final Integrated Hurricane Sandy
General Reevaluation Report
& Environmental Assessment

Appendix J
Engineering and Design

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Subappendix 1 – Recommended Plan Geotechnical Report and Drawings
Subappendix 2 – NED Plan, Geotechnical and Structural Analyses

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**.

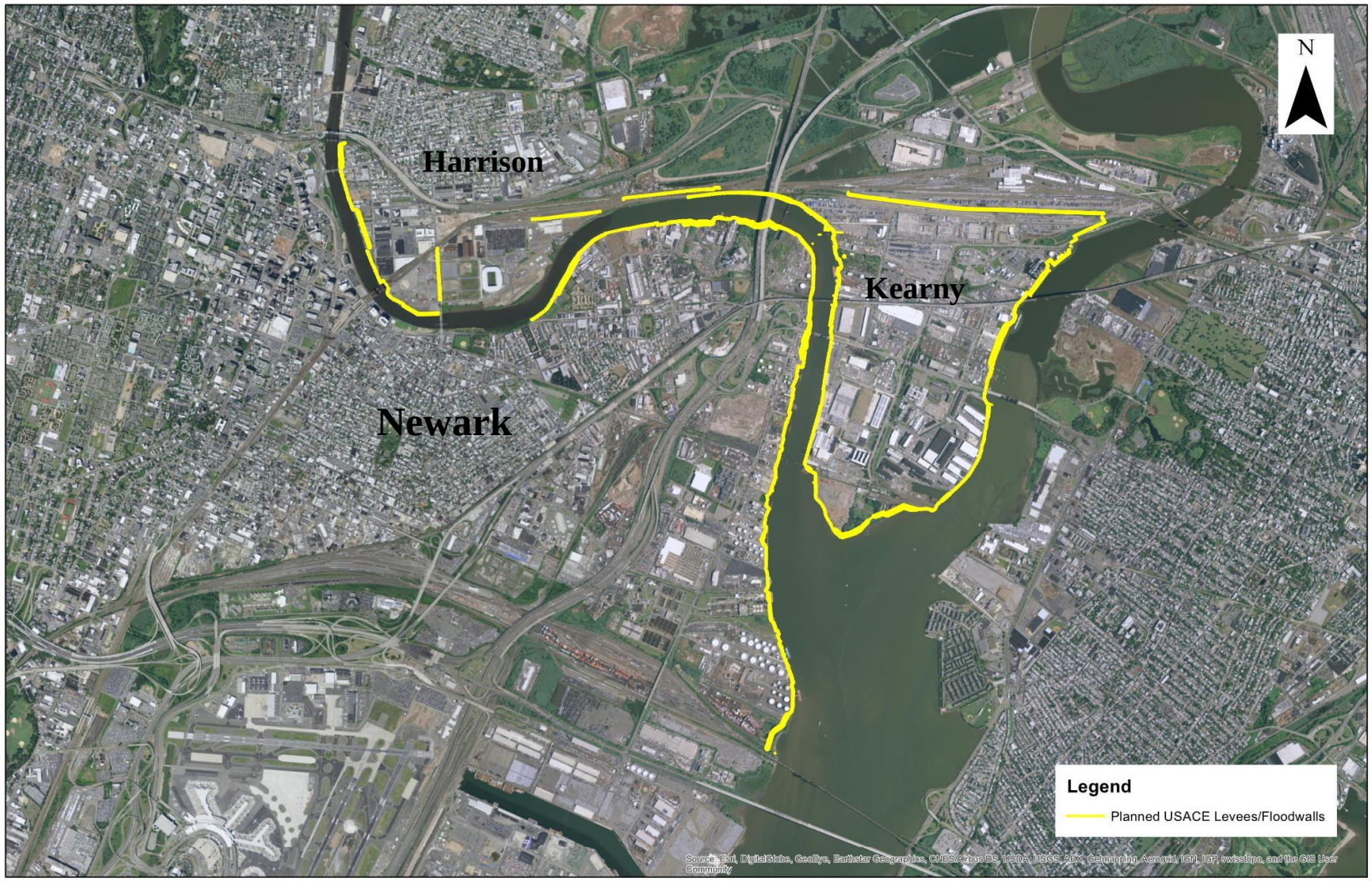

Figure 1: Passaic River Tidal Project Area – 1995 GDM Alignment

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**.

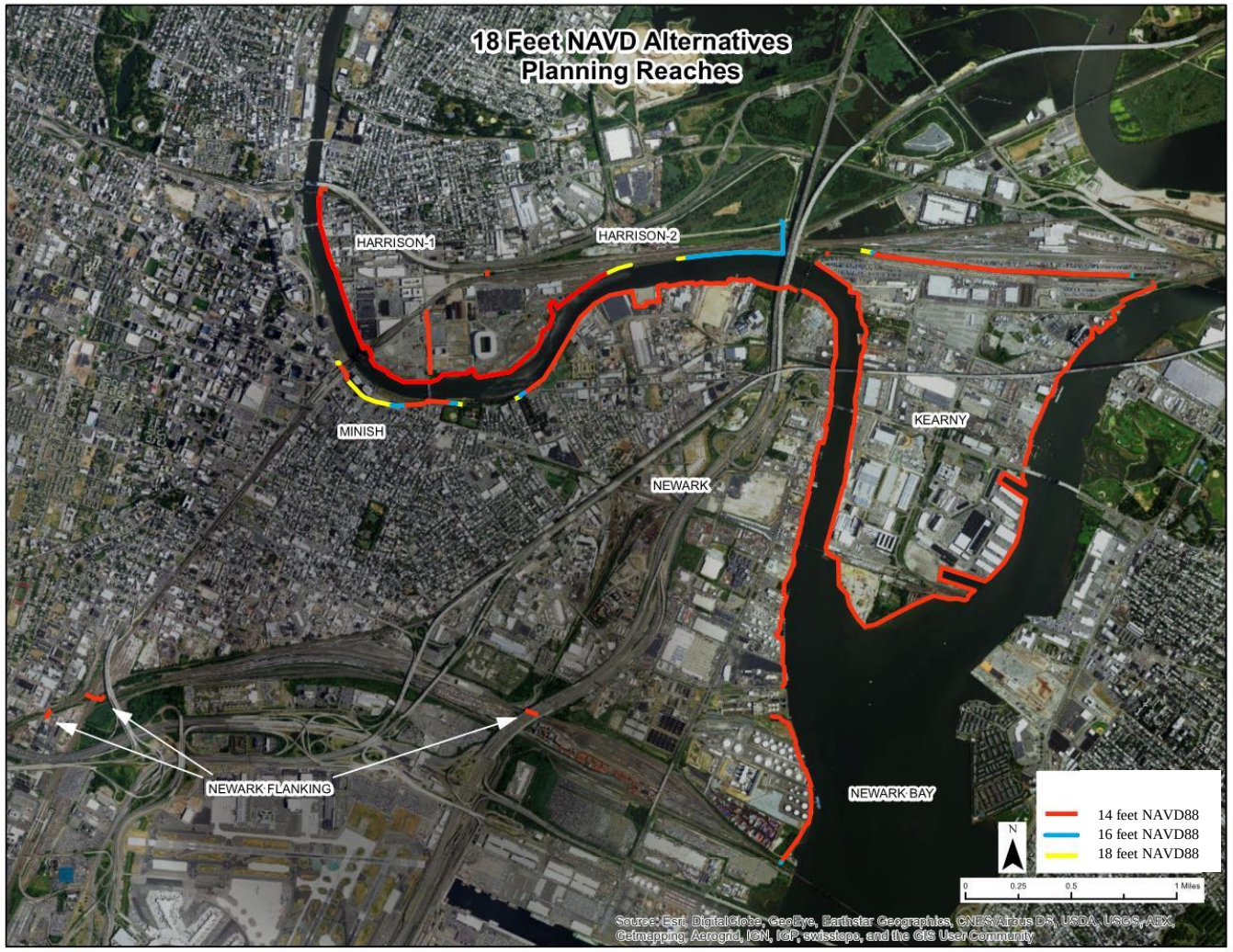

Figure 3: Passaic Tidal Project Reaches

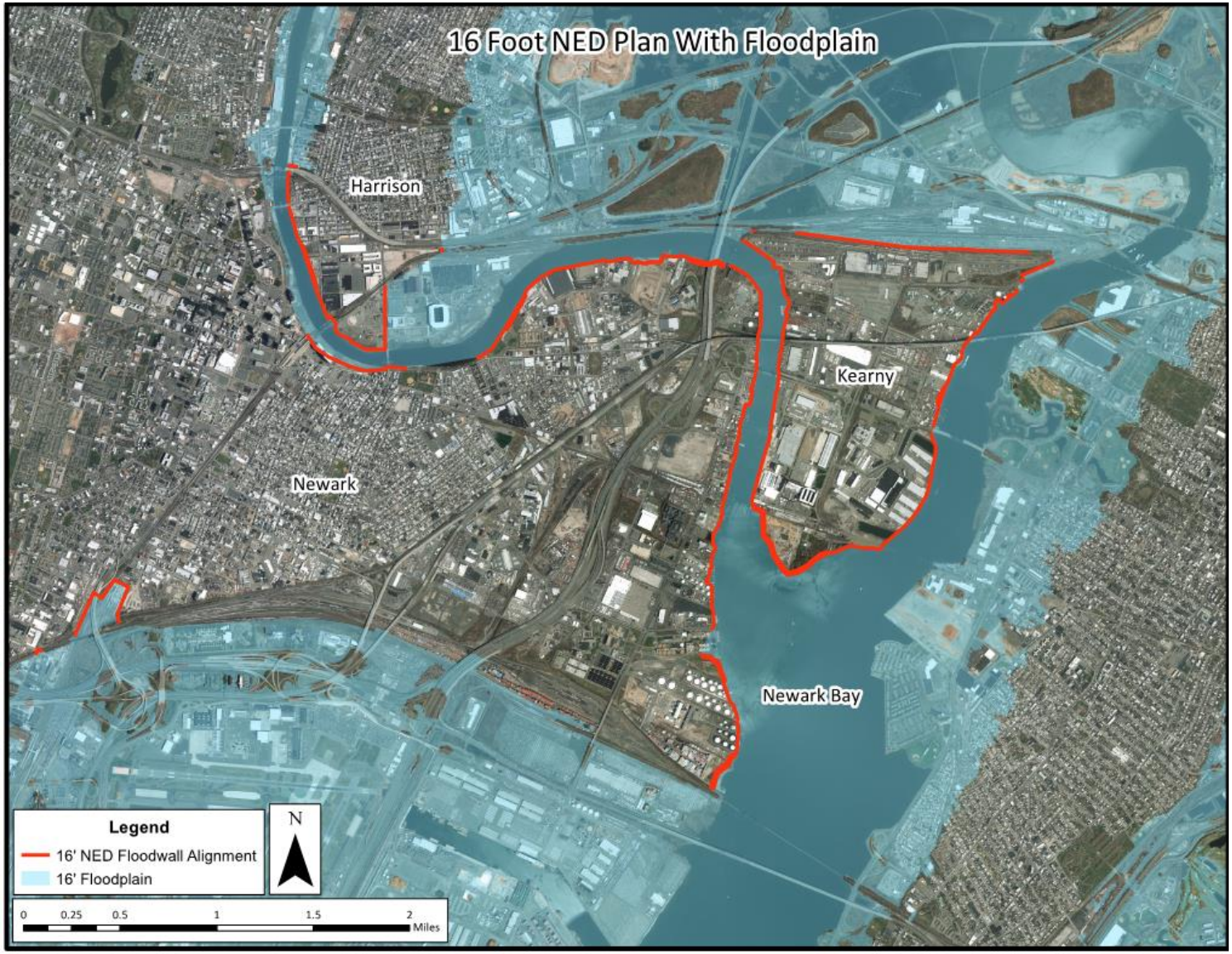

Figure 4: Passaic Tidal NED Plan – 16 feet NAVD88

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.

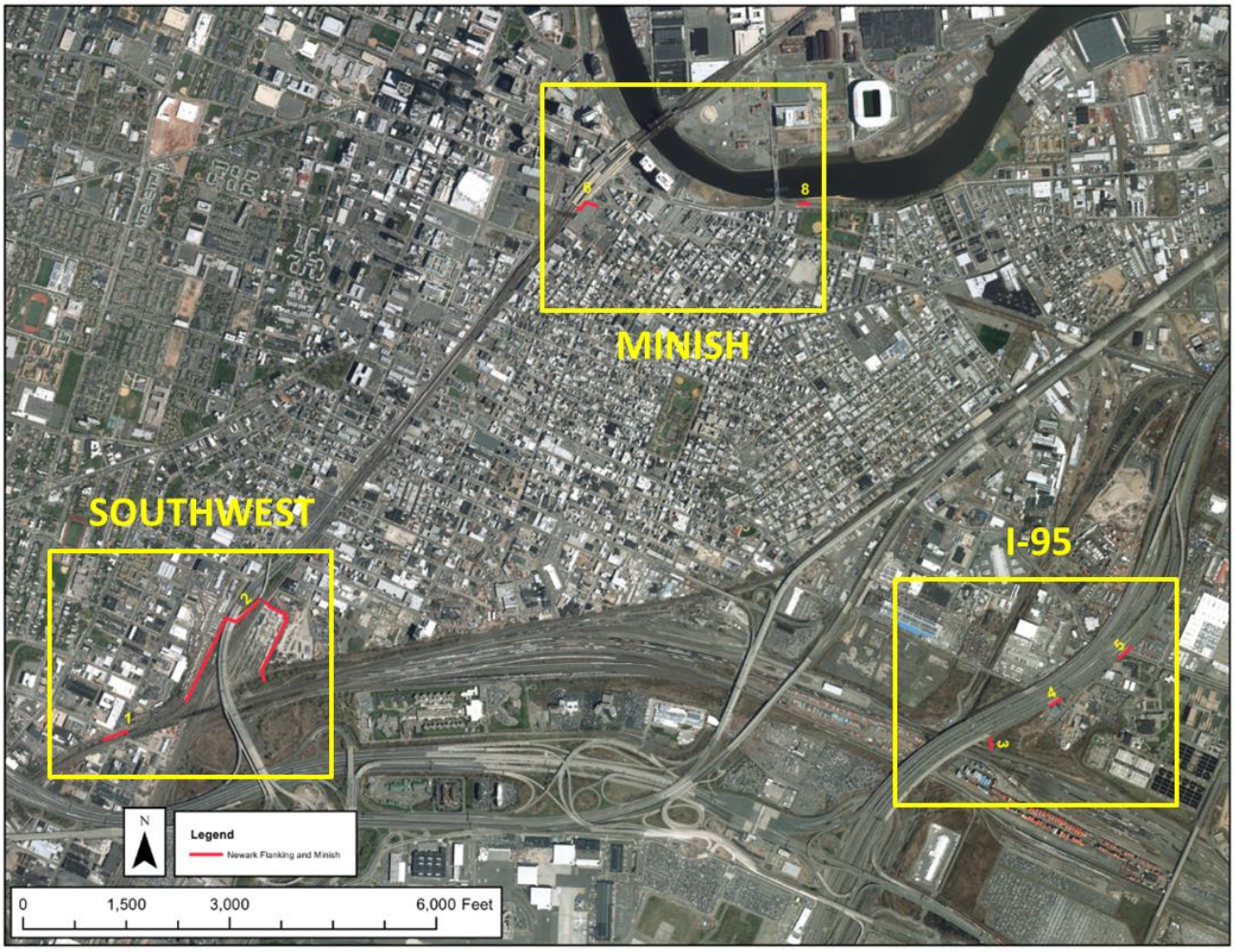

Figure 5: Passaic Tidal Project Reaches – Recommended Plan/Locally Preferred Plan

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.

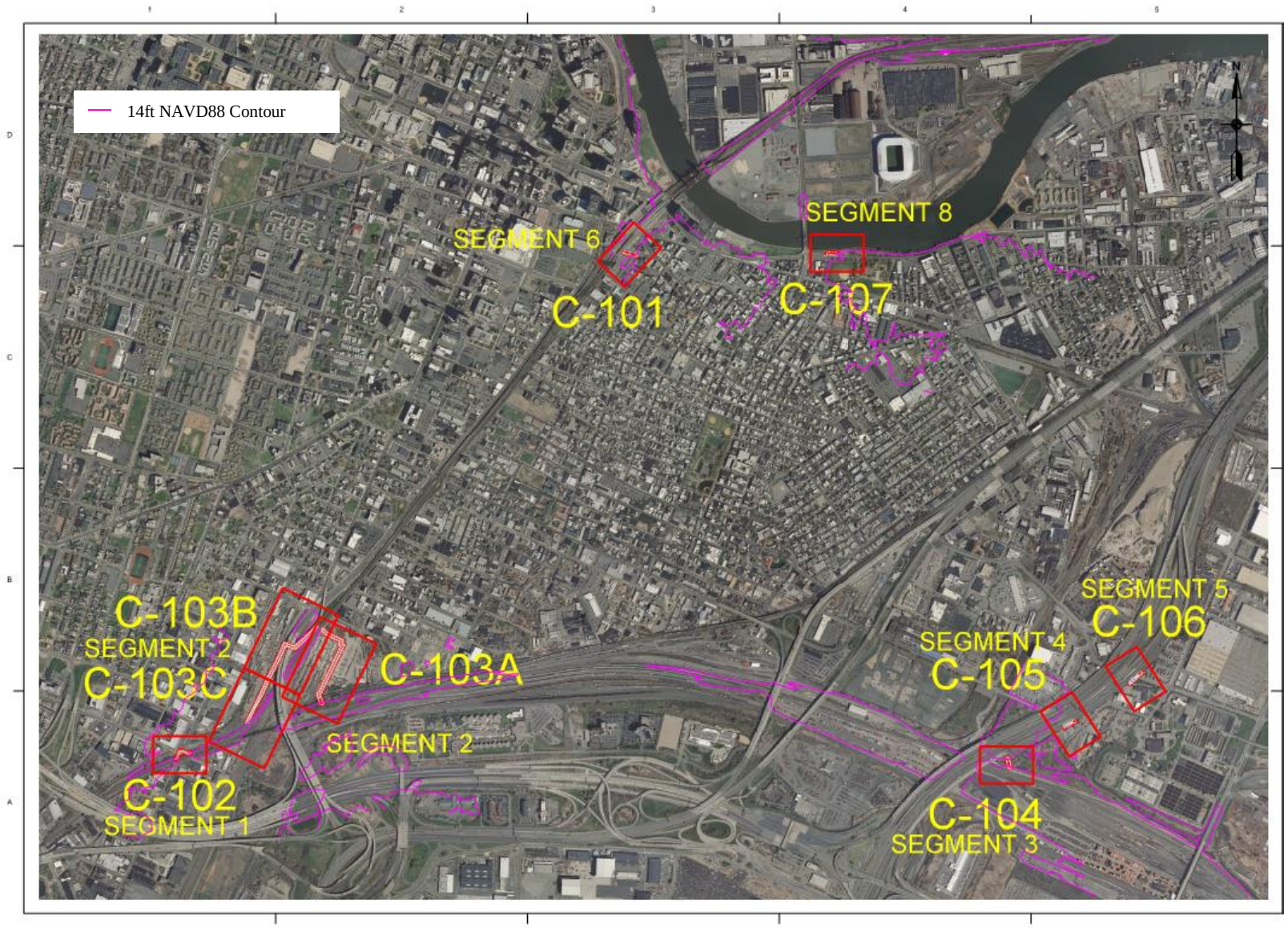

Figure 6: Recommended Plan Layout/Key Plan

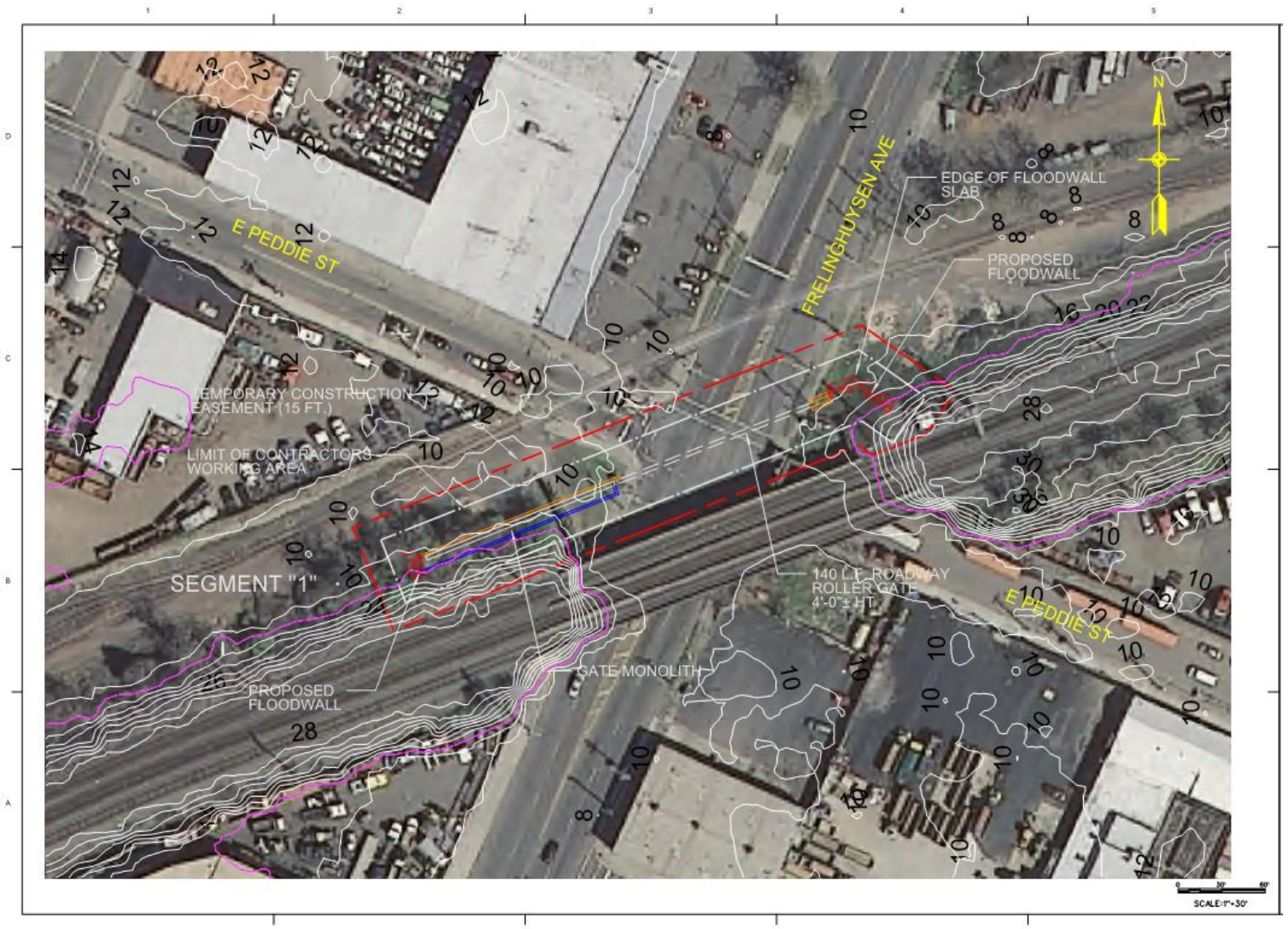

Figure 7: Southwest Reach - Segment 1

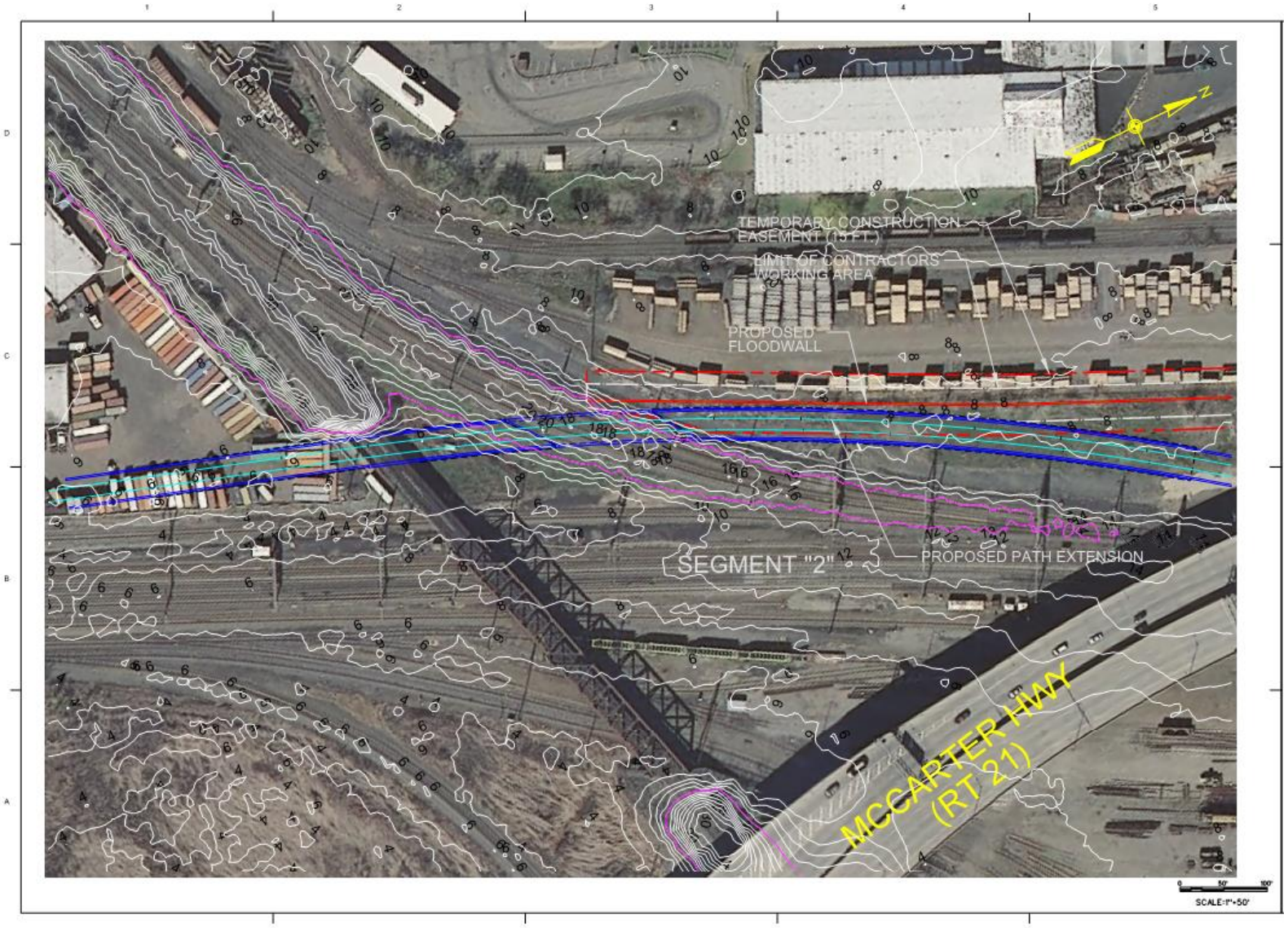

Figure 8: South West Reach - Segment 2A (South)

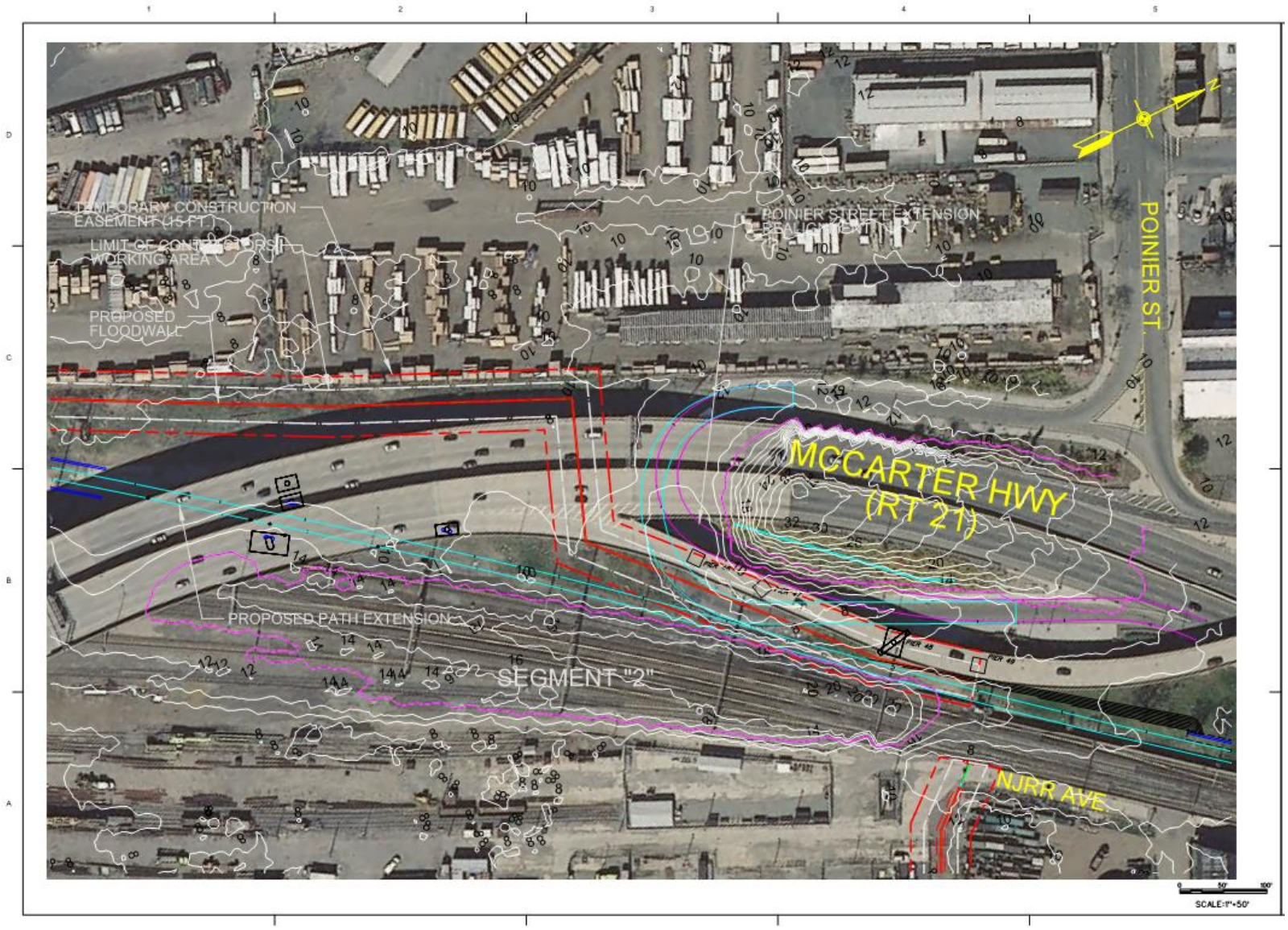

Figure 9: Southwest Reach - Segment 2A (North)

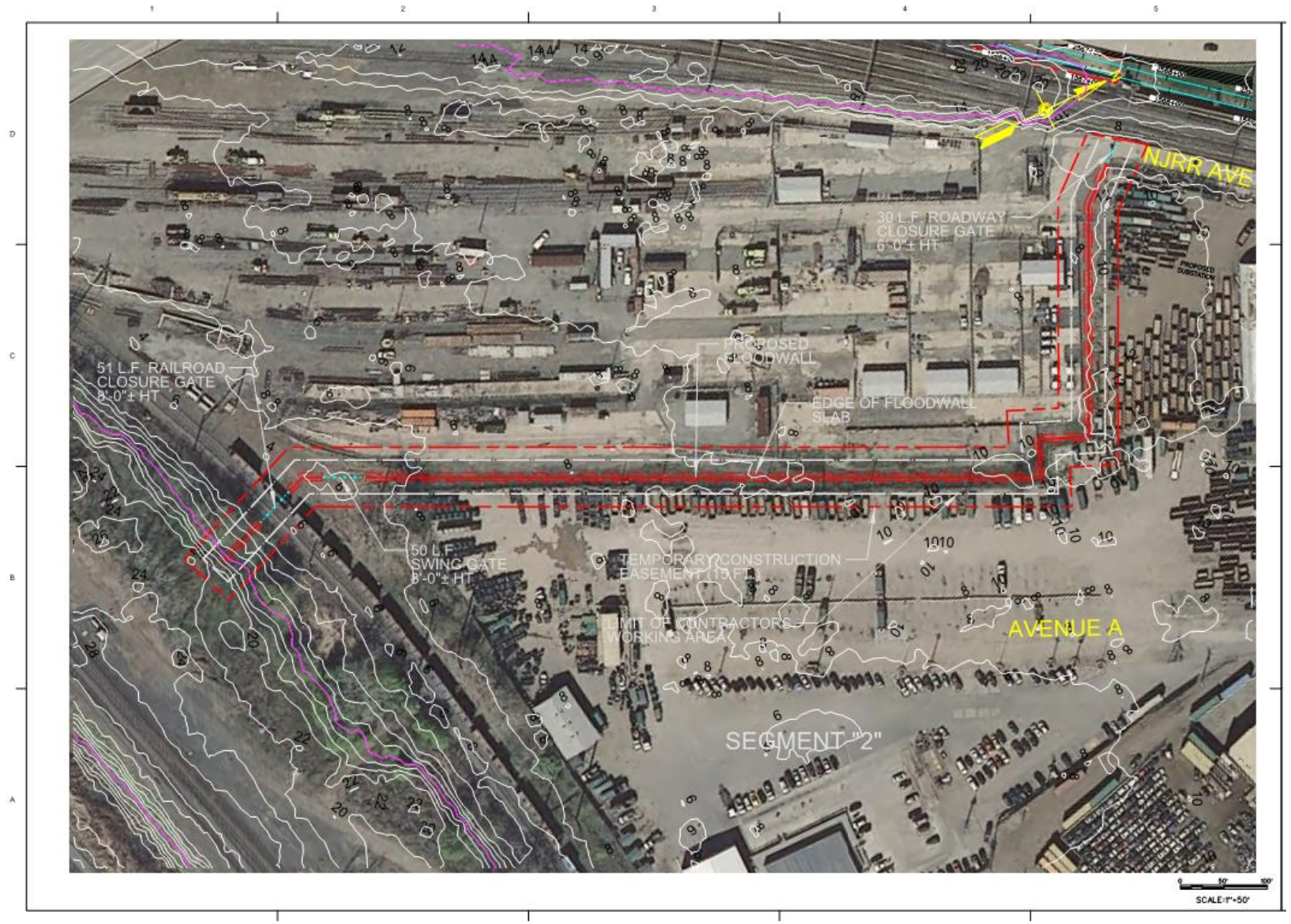

Figure 10: South West Reach - Segment 2B

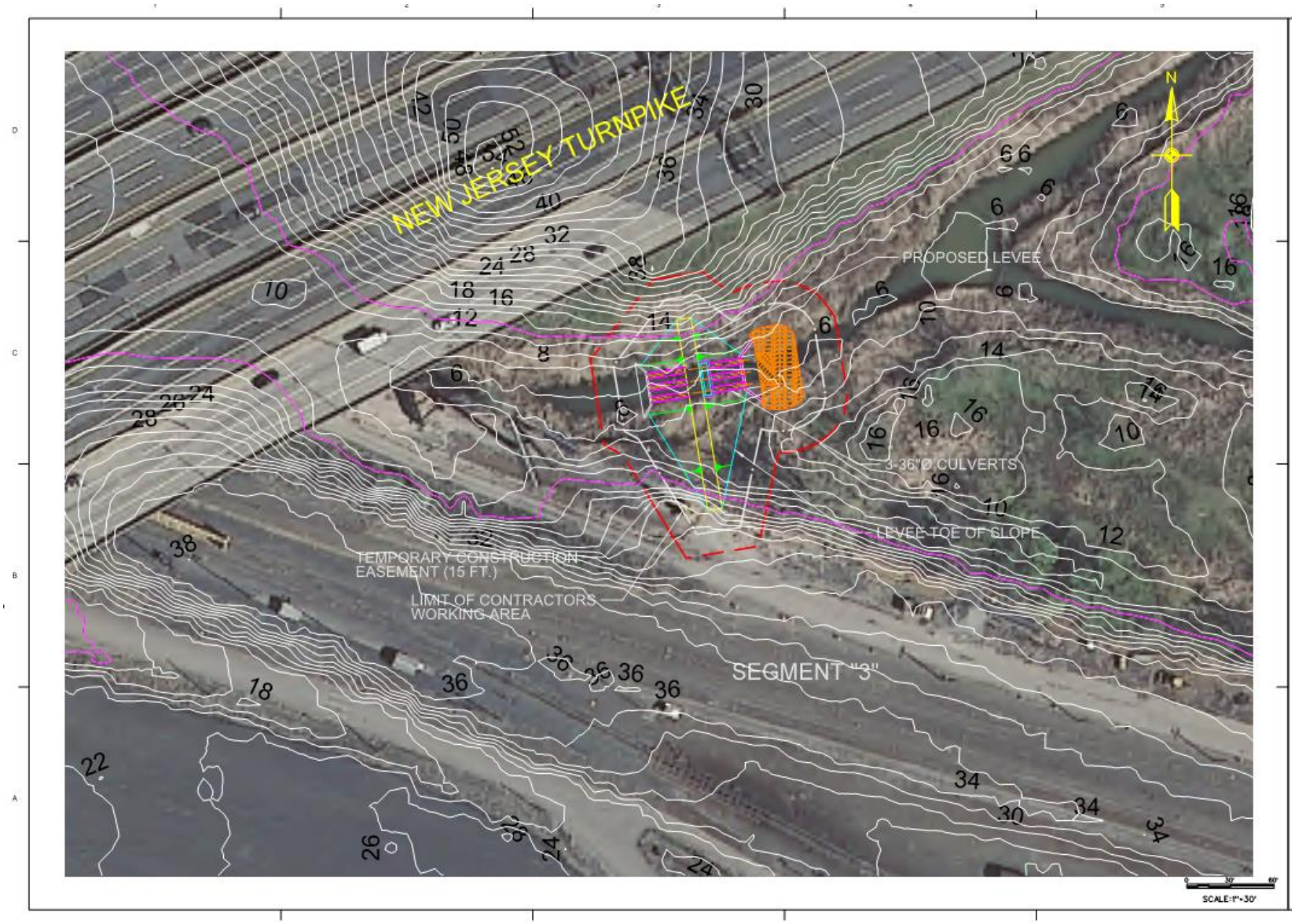

Figure 11: I-95 Reach - Segment 3

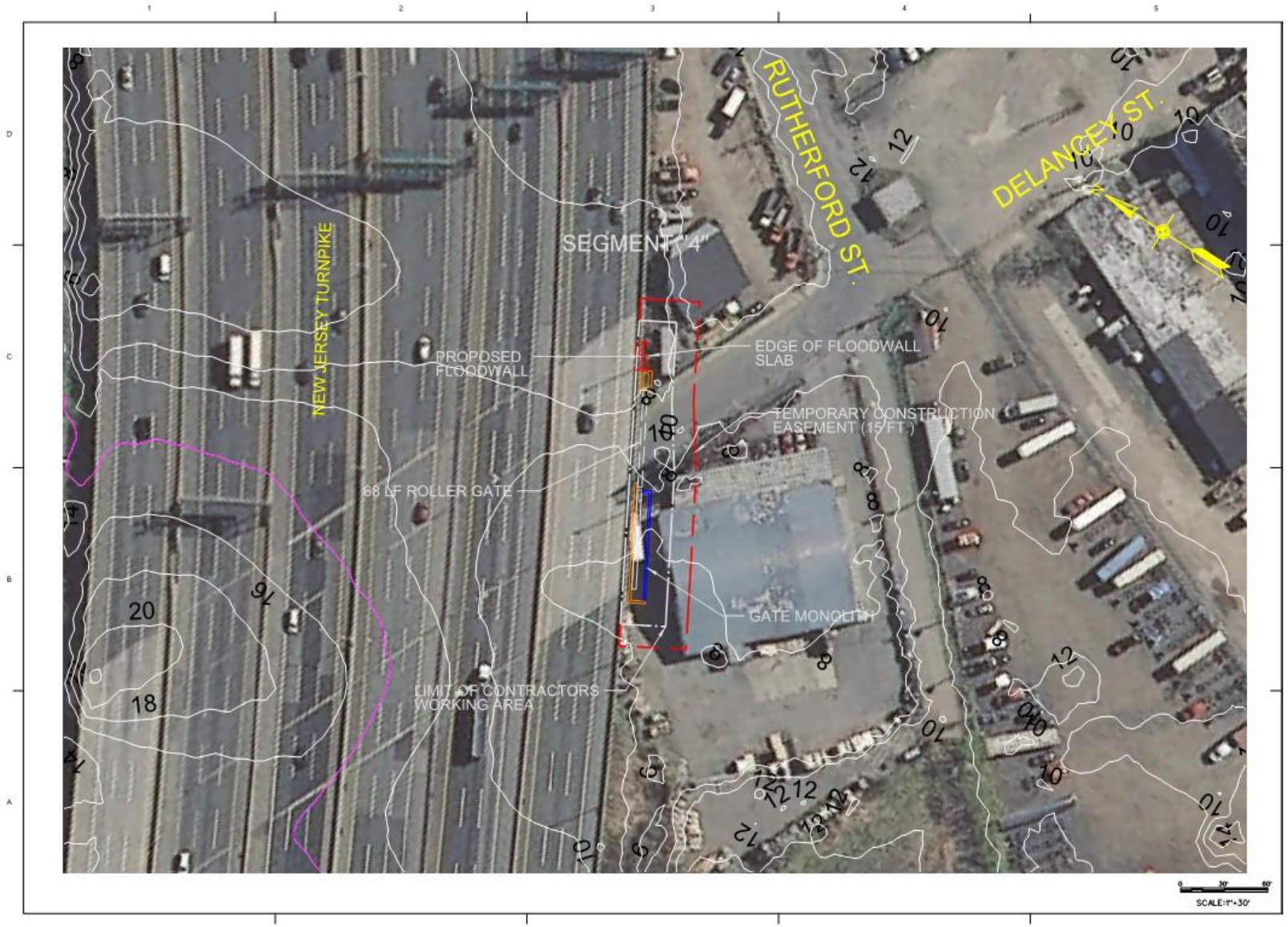

Figure 12: I-95 Reach - Segment 4

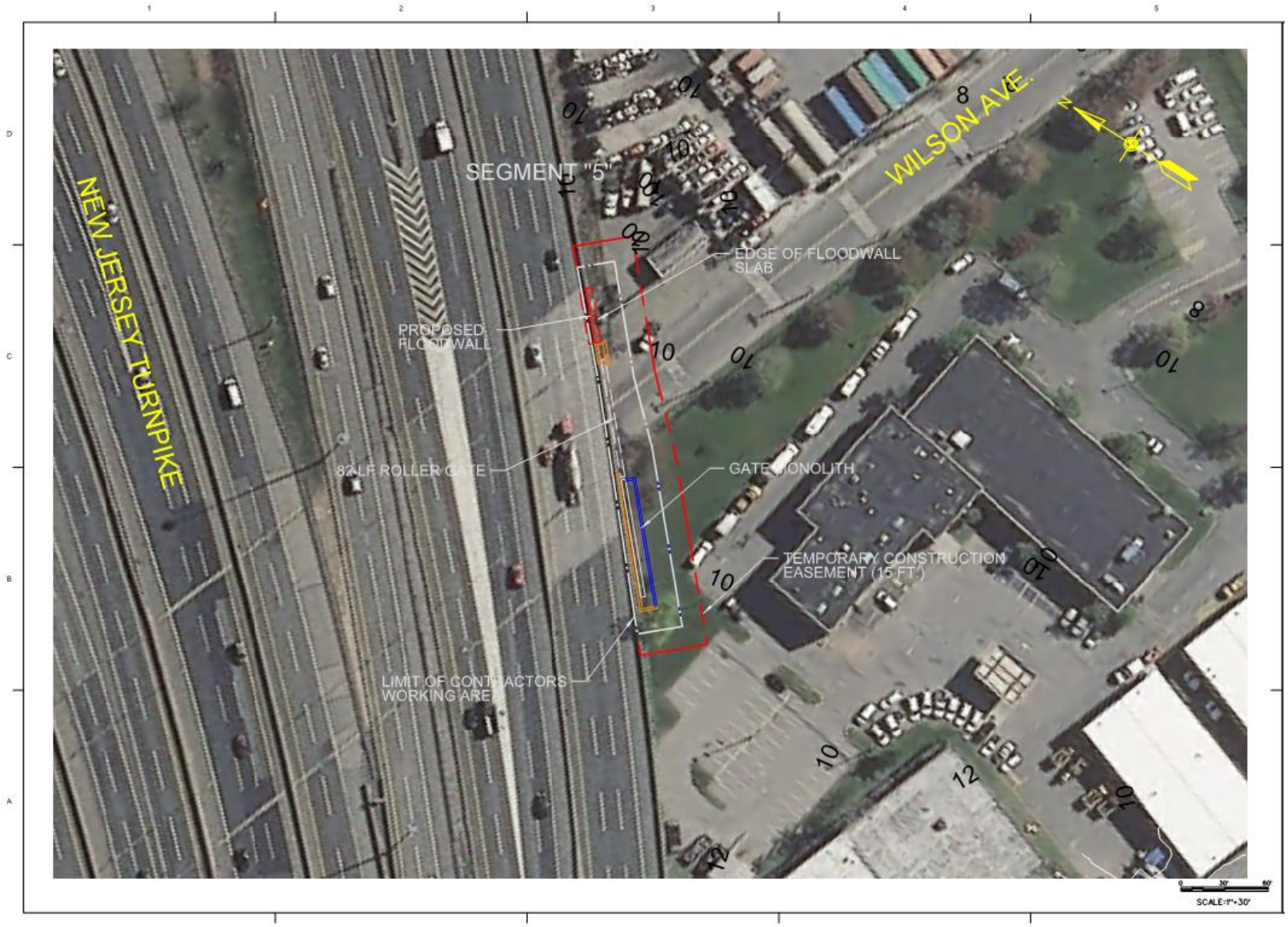

Figure 13: I-95 Reach - Segment 5

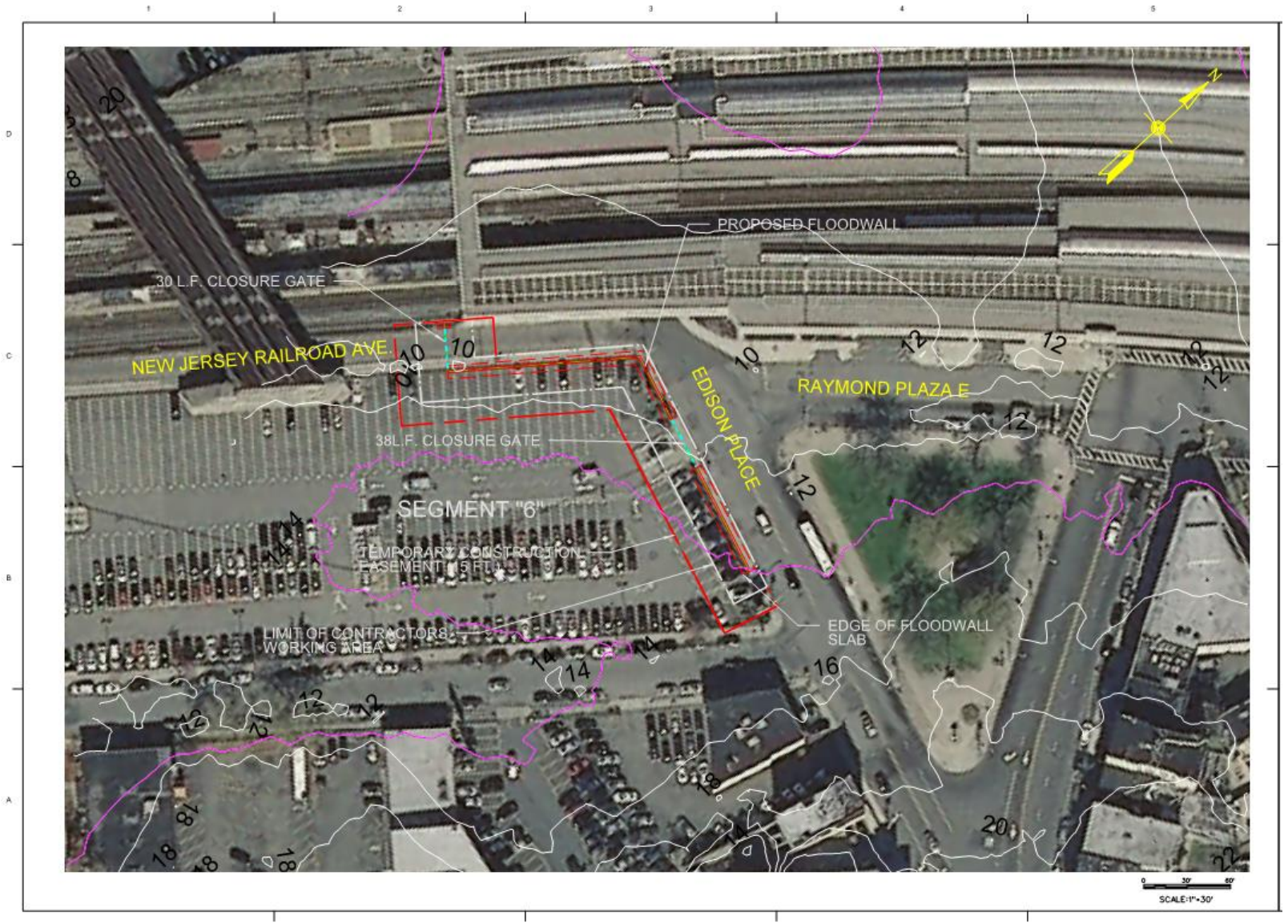

Figure 14: Minish Park Reach - Segment 6

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.

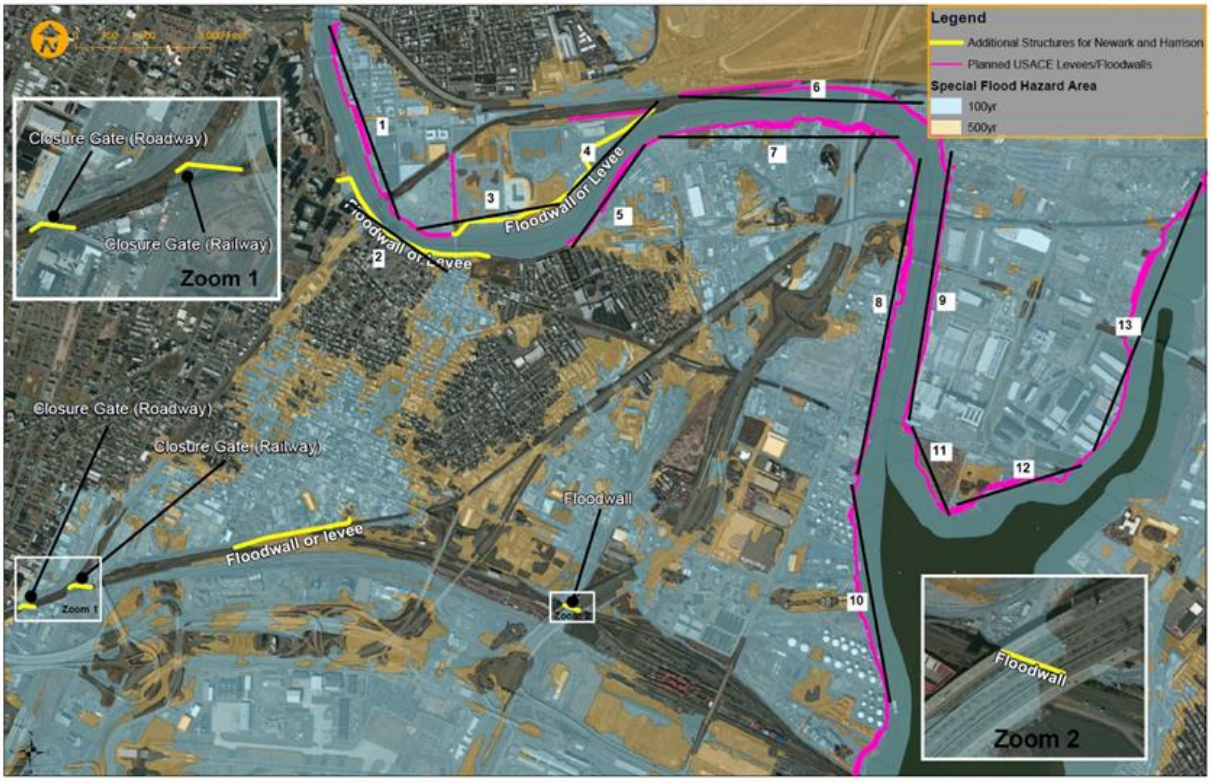

Figure 16: Segmentation of Levee / Floodwall System

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

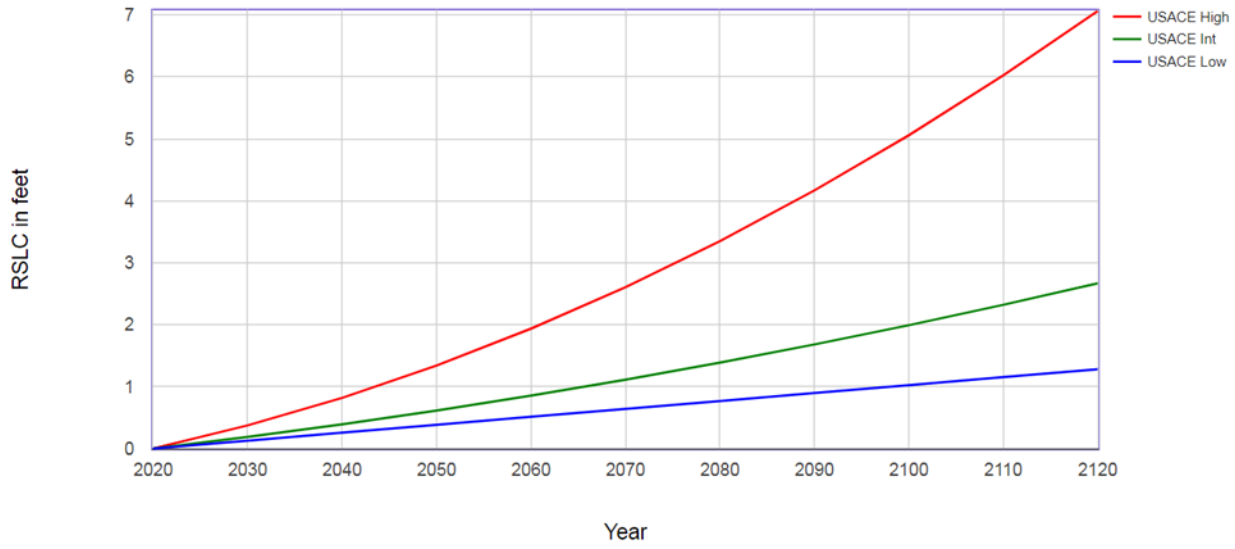

Figure 17: SLC Scenario Projections (Sandy Hook, NJ)

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

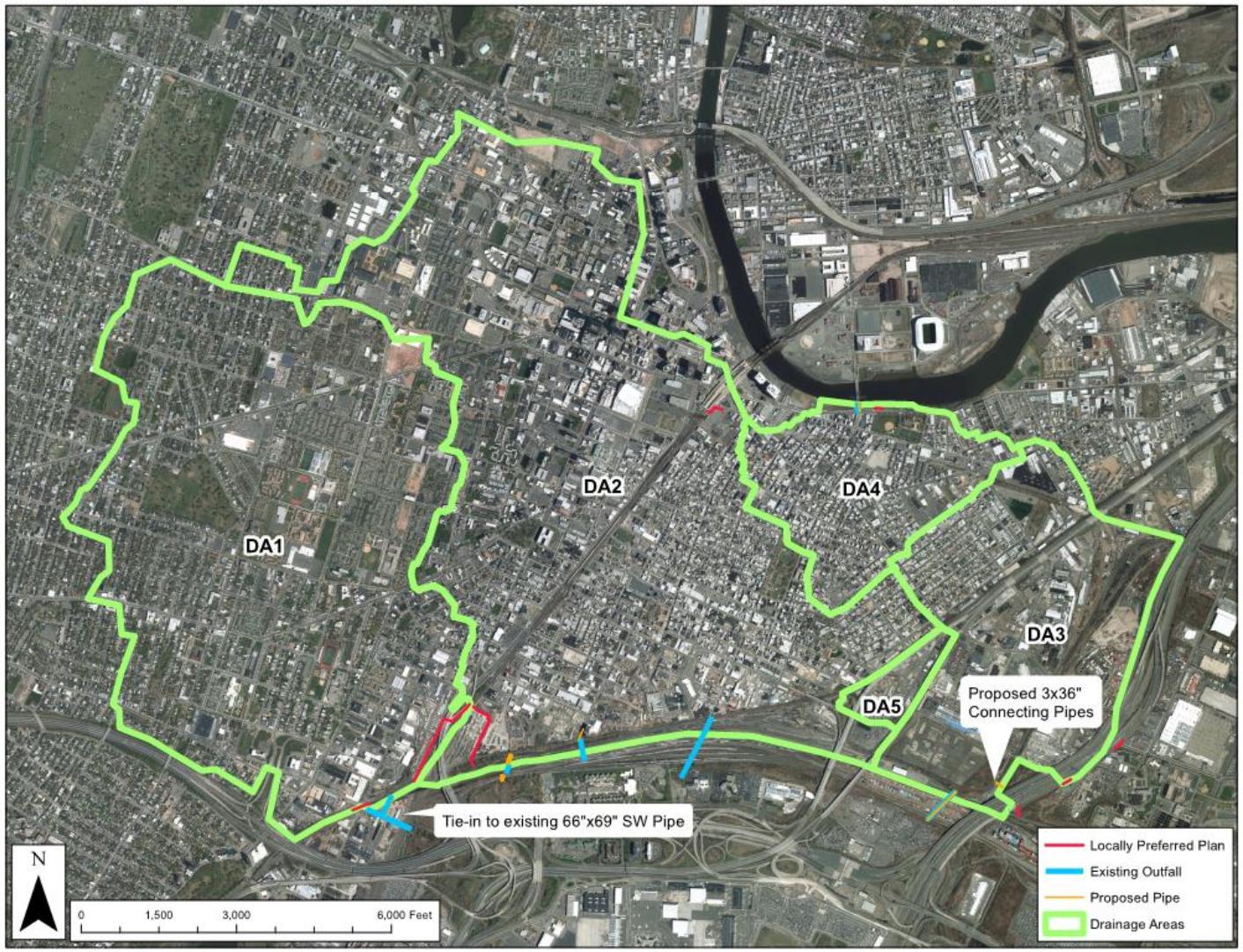

Figure 18: Interior Drainage Plan

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.

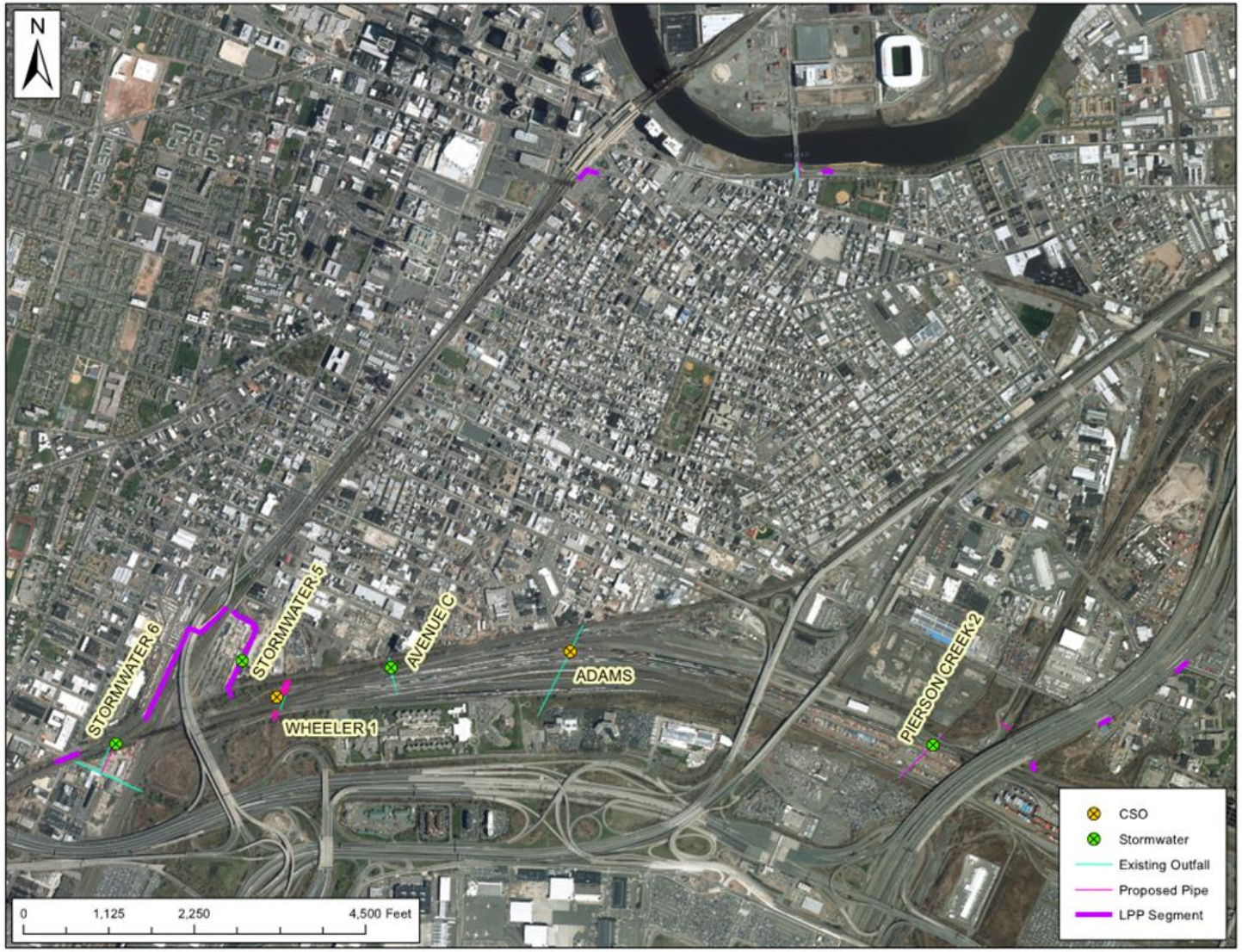

Figure 19: CSS/Stormwater Backflow Prevention

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**.

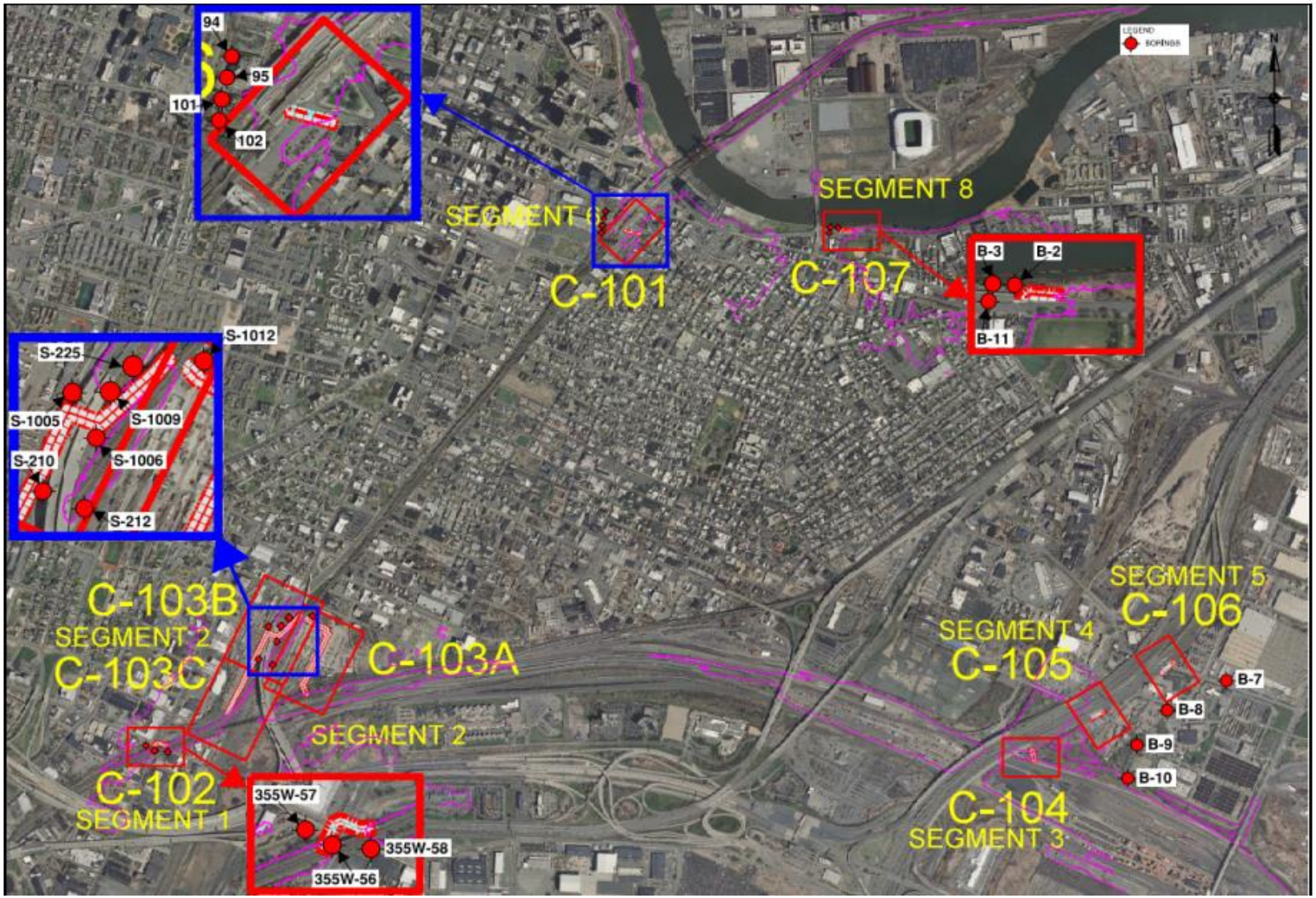

Figure 20: Recommended Plan Segments and Boring Locations

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 Drained: 10	250 50	3.28×10^{-6}
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 Drained: 10	250 50	3.28×10^{-6}
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 Drained: 22	2,500 200	3.28×10^{-8}

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. Construction. Dead load of the concrete wall components, no earthen backfill, no uplift. A 17 % overstress is permitted for this load case.

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

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

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

3a. Flood Stage at Stillwater, Debris Impact Load, Impervious Cutoff. 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 not 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.

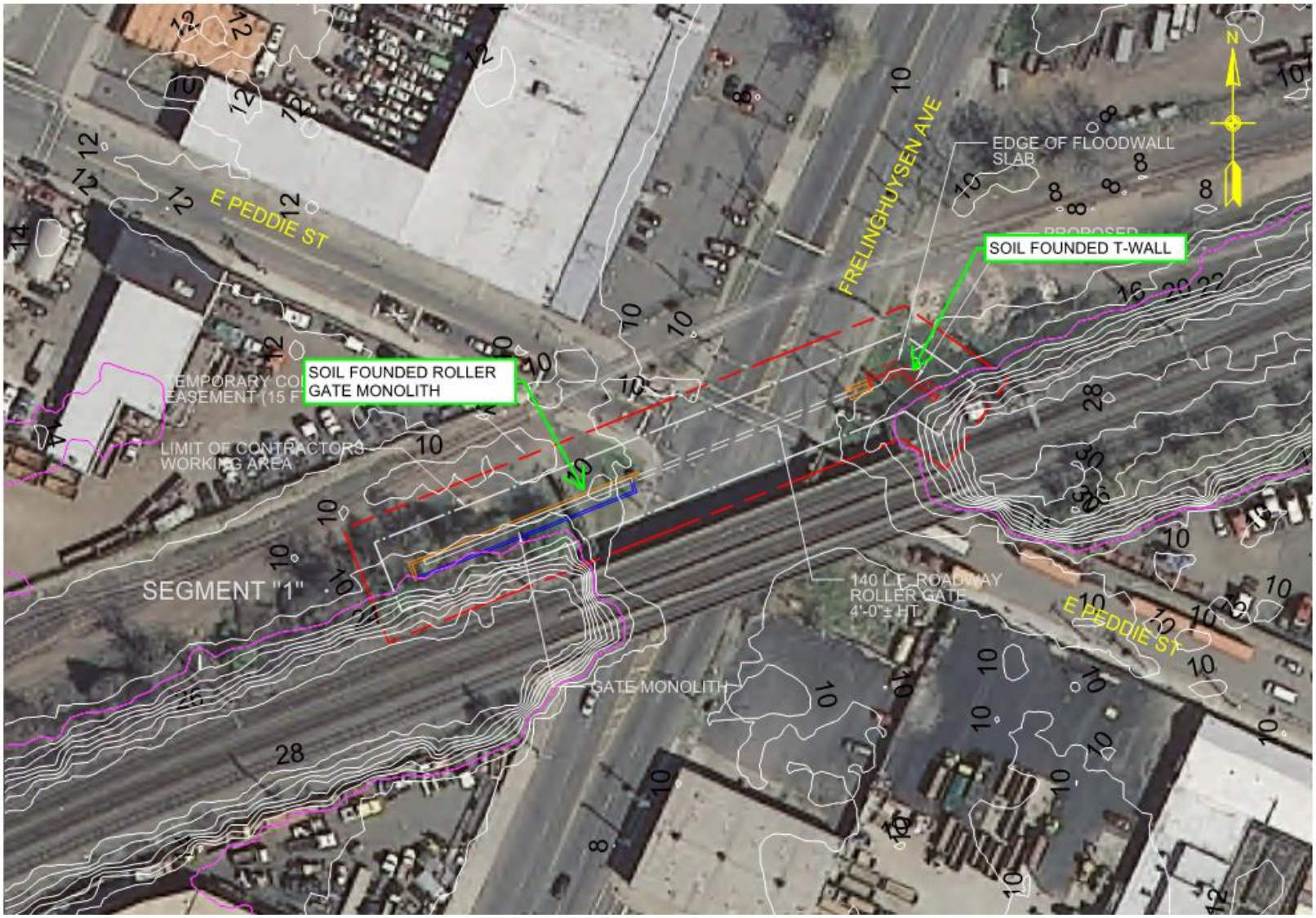

Figure 21: Segment 1 - Floodwall Type

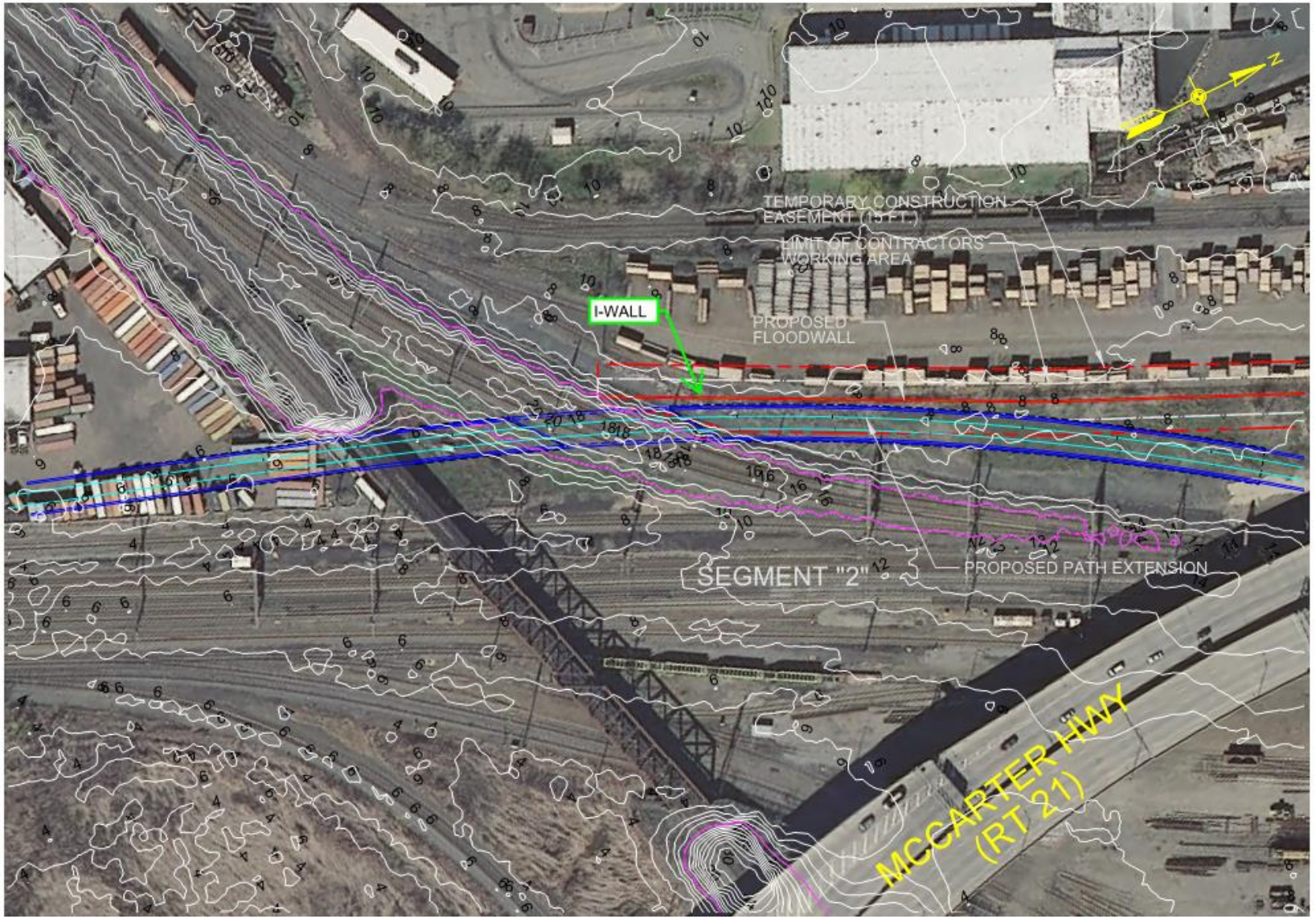

Figure 22: Segment 2A (South) - Floodwall Type

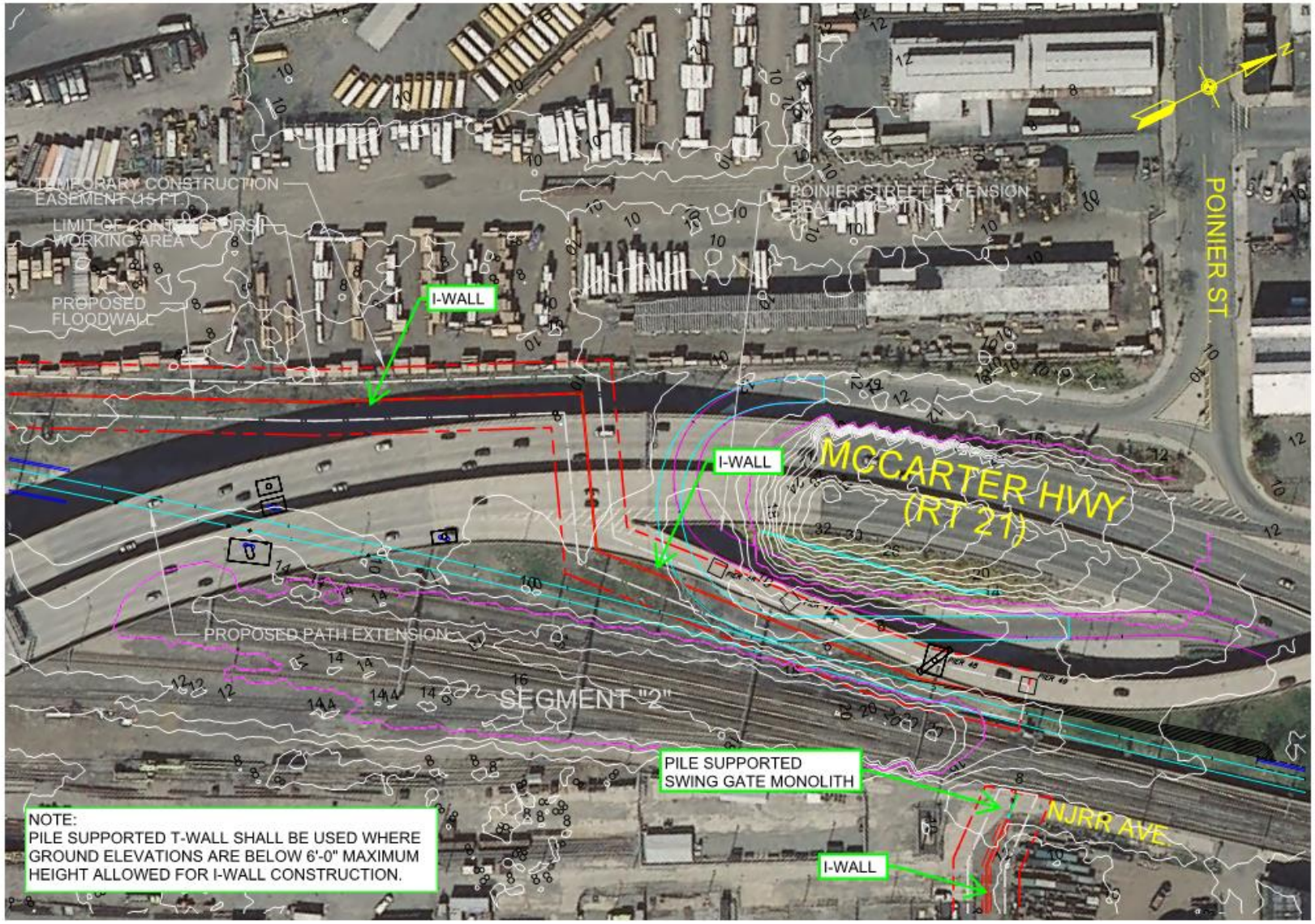

Figure 23: Segment 2A (North) - Floodwall Type

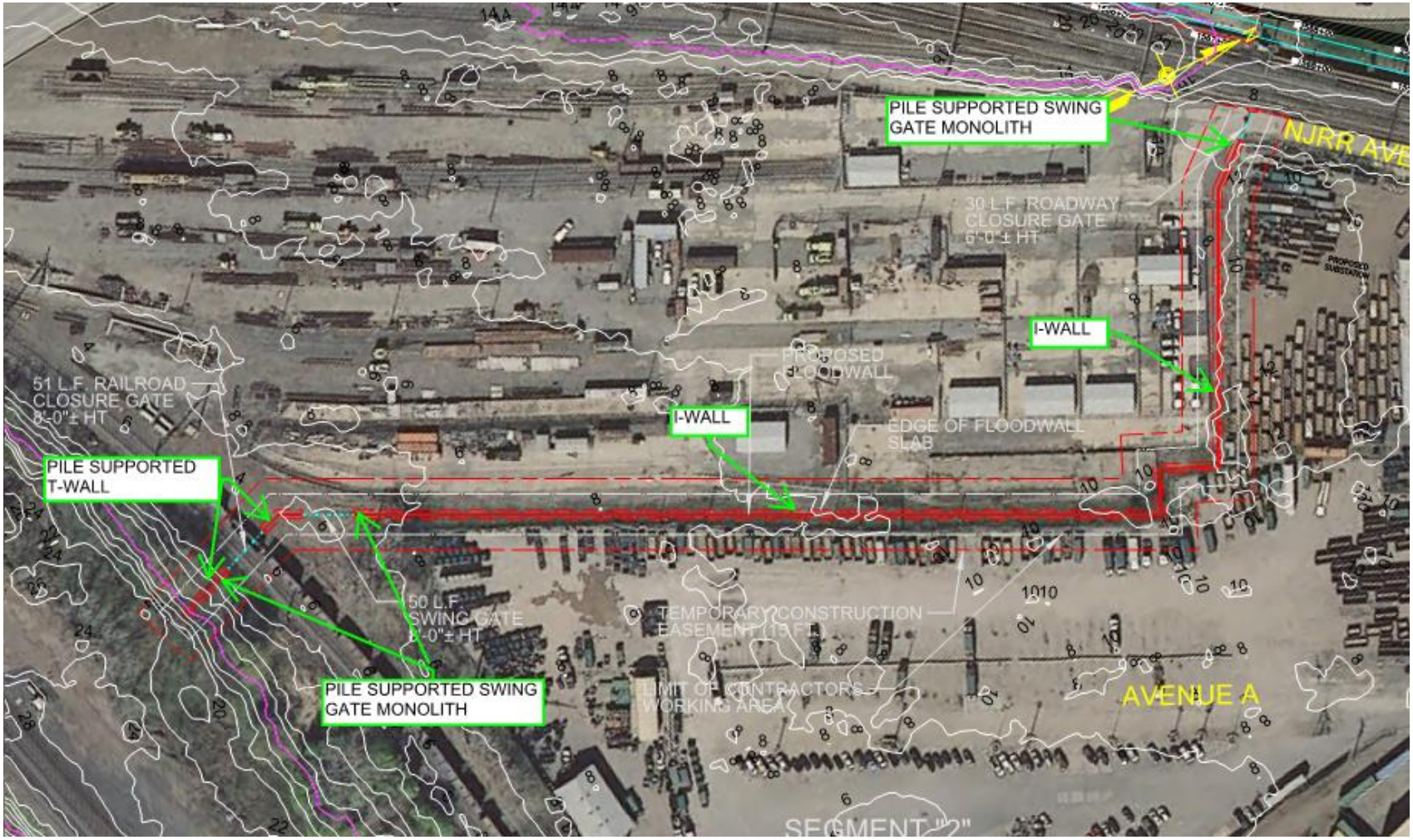

Figure 24: Segment 2B - Floodwall Type

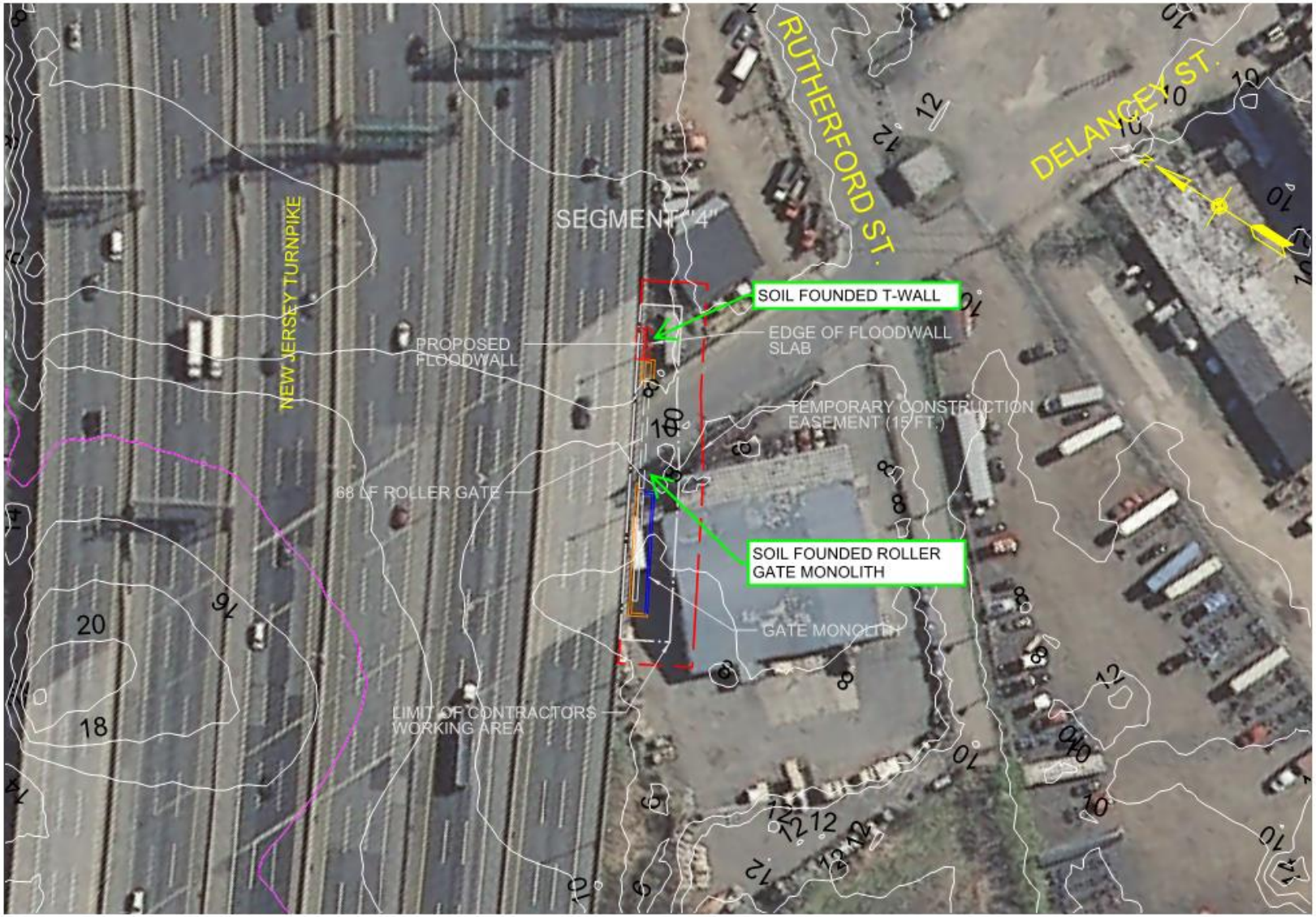

Figure 25: Segment 4 – Floodwall Type

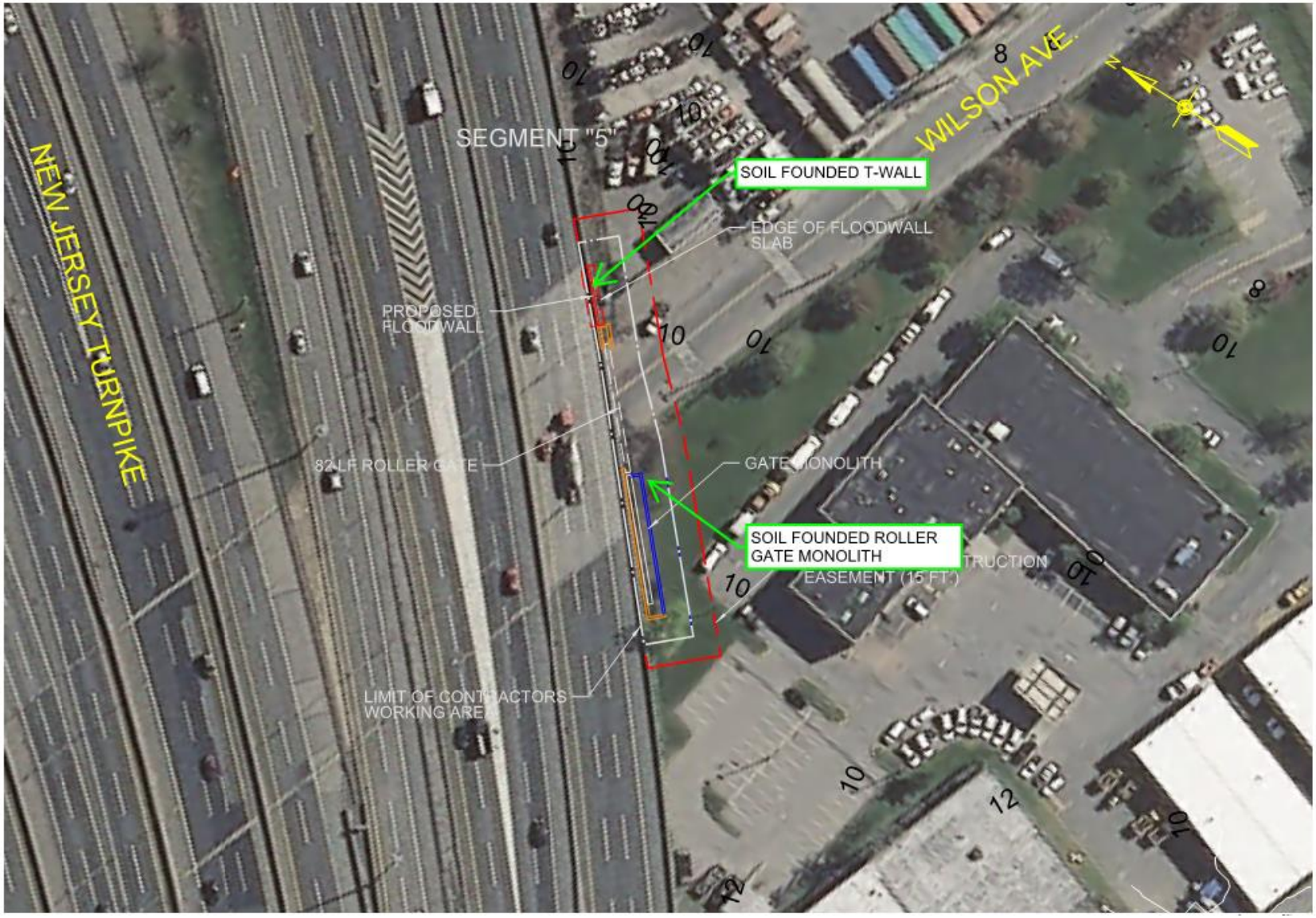

Figure 26: Segment 5 – Floodwall Type

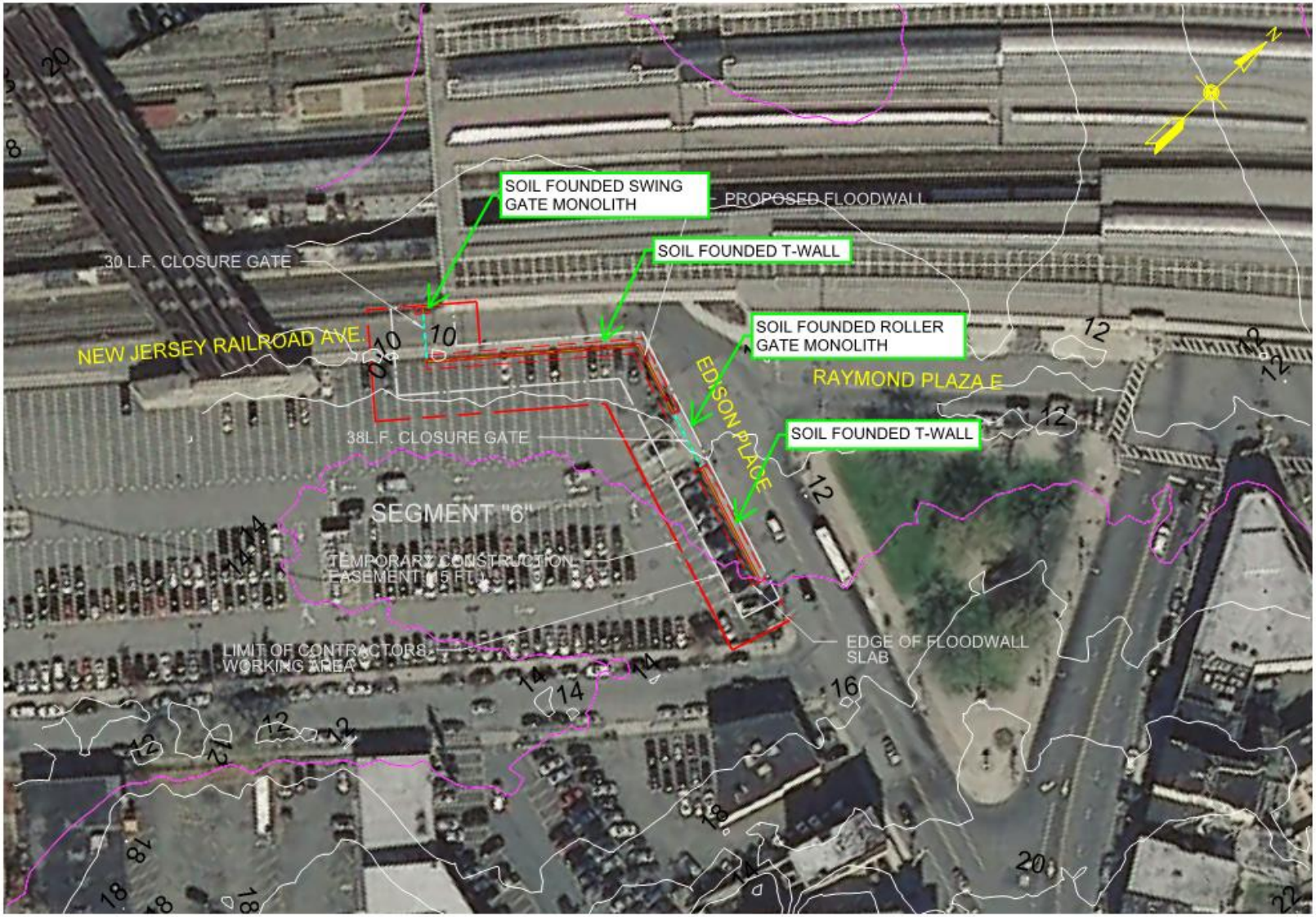

Figure 27: Segment 6 – Floodwall Type

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

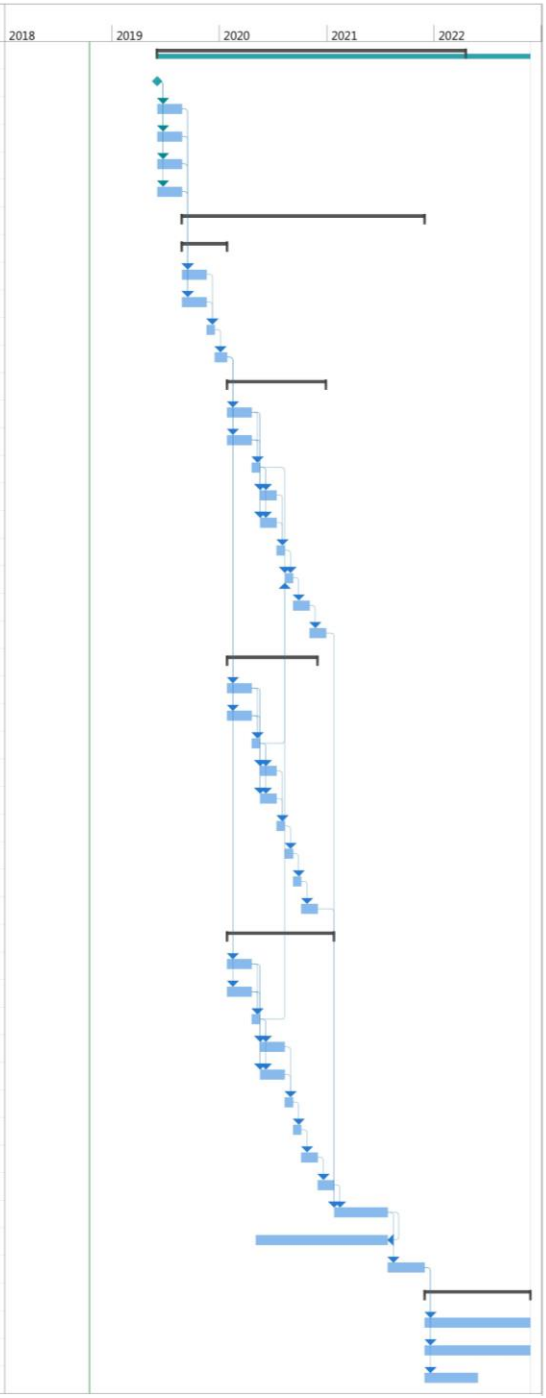

Figure 29: Recommended Plan Design and Construction Schedule

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.

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SUBAPPENDIX 1
Recommended Plan Geotechnical Report and Drawings

GEOTECHNICAL EVALUATION REPORT

PASSAIC RIVER TIDAL
NEWARK, NEW JERSEY

Prepared for

US Army Corps of Engineers
New York District

September 7, 2018

Prepared by:

AECOM

1255 Broad Street. Suite 201
Clifton, New Jersey 07013-3398

Project No: 60442748

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This report presents the results of the preliminary geotechnical study and the feasibility of concrete flood walls, flood gates and earth levee alternatives, and provides recommendations in support of the proposed flood protection system design and construction of the Tidal Portion of the Passaic River Flood Risk Management Plan.

1. INTRODUCTION

This report presents the findings of the feasibility assessment for the Passaic River Tidal Project in Newark, New Jersey. The following two alternatives were considered: 1) flood wall (T-wall and I-wall); and 2) earth levee. The project area is divided into seven (7) segments, designated to Segment # 1 to 6, and 8. The flood alternatives were analyzed for flood elevation of +14 ft (referenced to the North American Vertical Datum of 1988 (NAVD88)). The analyses include seepage, lateral load and pile axial capacity analysis for flood walls and flood gates, and seepage, slope stability and consolidation settlement analysis for earth levee. Liquefaction resistance was also evaluated for the flood walls, gates and levee.

2. GENERALIZED SUBSURFACE PROFILES

2.1 Previous Subsurface Investigation

Based on the available subsurface information in New Jersey Department of Transportation soil borings database[i] and a Memorandum prepared by AECOM for Passaic Valley Sewage Commission Wastewater Treatment Plant, Newark, New Jersey (2016)[ii], twenty two (22) borings near the proposed flood wall, flood gates, and levee alignment are considered in this study. The general locations of these borings are shown in Figure 1. 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 (Attachment A).

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 selected based on average SPT values from available boring logs in each area as shown in Figure 1.

Sufficient information on the SPT hammer was not available on many of the borings to make energy corrections for conversion to N_{60}, 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 as per the Reference iii (2013)[iii]. Corrections to N_{60} are considered for the liquefaction analyses in the next section of the report. Ground line elevations, where not given on some borings, are estimated from roadway surface elevations. Representative stratifications and selected soil properties for the seven segments are presented in Tables 1 through Table 5.

Table 1: Representative Stratification and Recommended Soil Properties for Segment 1

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	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 2: Representative Stratification and Recommended Soil Properties for Segment 2

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	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	$3.28 \times 10^{-}$

Table 3: Representative Stratification and Recommended Soil Properties for Segments 3, 4 and 5

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	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 4: Representative Stratification and Recommended 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 5: Representative Stratification and Recommended 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}

2.2 Recommendations

In order to obtain a better understanding of the subsurface condition with more accurate engineering and physical soil properties, additional field investigations and lab tests are recommended for the final design. The following recommendations are made for additional analyses to support final design:

1. Take additional soil borings where required to have a minimum of three (3) borings spaced at a maximum of 100 feet along each segment. Transverse soil profiles are developed typically with two (2) additional borings perpendicular to the levee flood wall alignment at each cross-section. Drill at least one test boring for each soil profile to a depth of bedrock or 100 ft deep for seismic site classification purposes.
2. Take additional disturbed and undisturbed samples for soil properties determination. Perform additional grain size analysis, unconsolidated-undrained (UU) tests and consolidation tests.
3. Make field permeability and/or field pumping tests, as necessary, for permeability estimation.

4. It is also recommended that seismic CPT soundings be performed for every 8 borings to obtain shear wave velocity of the subsurface soils. Seismic CPTs may assist to better define the site class, shear wave velocity, and liquefaction potential of the site.

3. LIQUEFACTION RESISTANCE

Factor of safety (FOS) against liquefaction for non-cohesive soils under the groundwater table at the seven segments were calculated. A design earthquake magnitude of M_w = 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)[iv] was used for liquefaction evaluation of non-cohesive soils (e.g., sand and gravel) in the top 50 ft. 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, $(N1)_{60}$ 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_σ. 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 the 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. The plot of factor of safety (FOS) against liquefaction for each segment is provided in Figures 2 to 6. Based on the liquefaction evaluation, occasional pockets of potentially liquefiable soils exist in the area of Segment 2 which require further investigation.

4. FLOOD WALLS AND GATES

The preliminary line of protection for each segment is provided in Figure 1. The flood wall alternative was considered for the all the Segments. As a representative section for areas of flood walls and gates, a T-Wall with height of 4 ft was considered for Segments 1, 4, 5, and 6. T-Walls supported on H-Piles with heights of 6 ft and 8 ft were considered for Segment 2. As an additional alternative, an I-Wall with height of 6 ft was considered for Segment 2. For Segment 6 and 8, T-Wall with height of 2ft 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. The summary of proposed flood protection systems is provided in Table 6. The design flood elevation was assumed to be el. +14 ft (NAVD88), and ground surface elevations were assumed to very between el. +6 and +12 ft.

Table 6: Summary of Proposed Flood Protection Systems for each Segment

Segment #	Type of Flood Protection	Top of Wall Elevation [NAVD88] (ft)	Ground Elevation [NAVD88] (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

4.1 Bearing Capacity

Based on the average N-Values of the fill layer and conventional bearing capacity estimates were performed. More comprehensive bearing capacity calculation considering the lateral pressure will be done in the design phase of the study after performing the geotechnical investigation. The summary of allowable capacities is provided in below Table 7.

Table 7: 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

4.2 Seepage and Sliding Stability Analyses

Steady state seepage analyses at full flood stage were performed for the flood walls using the commercially available software GeoStudio 2007 SEEP/W© by Geoslope international, Ltd., and following the guidelines in EM1110-2-2502 (1989)[v]. The hydraulic conductivity values were assumed based on the soil type, and fines content. The assumed hydraulic conductivity values of each layer were provided in Tables 1 to 5. The maximum exit gradient and flow rate for the T-Walls and I-Wall at full flood stage are presented in Table 8. The estimated maximum gradients are lower than the allowable critical gradients, typically 0.5, according to EM 1110-2-2502 (1989)[ii]. Based on the estimated critical gradients for 4 ft 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 ft and 8 ft. Details of the seepage analyses for the T-walls are provided in Sheets C.1 to C.6 of Attachment C.

Table 8: Summary of Proposed Flood Protection Systems for each Segment

Segment #	Type of Flood Protection	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 (1989)[ii]. 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.

4.3 Global Stability Analysis

The global 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 (2016)[vi]. This segment 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 ft high but located in a kind of 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, soil stratification and parameters were 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 el. +1.5 ft to be same as the Passaic river level.

Case II was analyzed at flood level elevation of +14 ft 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. The values of FOS associated with the critical slip surfaces are greater than the required minimum USACE values as provided in Table 9.

Table 9: Slope Stability Analysis Results for 4ft High T-Wall in Segment 8

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

4.4 Lateral Load Analysis

An I-wall with 6ft free height alternative is considered for Segment 2 (see Figure 8 for schematic cross-section). The I-wall was analyzed using PYWALL, Ensoft, Inc. Long-term (drained) soil properties of the organic clay and clay layers were conservatively (resulting in higher active pressures on wall) used for the analysis. A summary of I-wall analysis results for Segment 2 is presented in Table 10. Considering a maximum allowable lateral deflection of 1 in at the top and approximately zero inches of deflection at the pile 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 ft is recommended. Plots of lateral defection, bending moment and shear force with depths of sheet piles are provided as Attachment D.

Table 10: Results of the Sheet Pile Analysis for I-walls in Segment 2

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

4.5 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 U.S. Army Corps of Engineers, Design of Pile Foundations, EM 1110-2-2906 (1991)[vii]. Skin friction from the Stratum 2 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 ft long pile are provided in below Table 11. The summaries of axial capacities for 30 ft to 60 ft pile lengths are presented in Attachment E.

Table 11: 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

5. EARTH LEVEE

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

5.1 Levee 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 (2016)[iii] and following the guidelines in U.S. Army Corps of engineers, Design and Construction of Levees, Engineering Manual, EM 1110-2-1913 (2000)[viii]. A 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. The details of the seepage analysis of Segment 3 and 4 cases of slope stability analyses for the earth levee are provided in Attachment F.

As shown in Sheet F.1, the estimated maximum exit gradients are lower than the allowable critical gradients, typically 0.5, according to ETL 1110-2-569 (2005)[ix].

The values of FOS associated with the critical slip surfaces are greater than the required minimum values, as shown in Sheets F.2 to F.6. Upstream and downstream seismic directions were considered in Case IV.

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 12 and 13.

Table 12: Seepage Analysis Results for 8 ft High Levee for Segment 3

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

Table 13: Slope Stability Analysis Results for 8 ft 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

5.2 Settlement Analysis

Based on the generalized soil profile for Segment 3 as provided in Table 3, the top 15 to 45 ft of the natural soil in the flood protection area consists of sandy/silty clay. The immediate or elastic settlement of soils will take 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 Geotechnical Reevaluation Report (2016)[x]. 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)[xi].

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 ft 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.

6. CONCLUSIONS AND RECOMMENDATIONS

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

- It is recommended to fill data gaps and validate the soil profiles by performing further geotechnical investigation at each segment. Guidelines for the investigation are provided in Section 2 of this report.
- T-walls supported on shallow foundation are feasible from seepage standpoint for the 2 ft flood height in Segment 8 and 4 ft flood height in Segment 1, 4, 5, 6 &8.
- T-walls with sheet piles and pile foundations are recommended for 6 and 8 ft flood heights for Segment 2.
- I-walls are feasible for 6 ft flood heights for Segment 2.
- Based on the results of seepage and global stability analyses, the levee alternative is feasible for flood height of 8 ft for Segment 3, where no organic soil was identified in the soil profiles.
- 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.

7. REFERENCES

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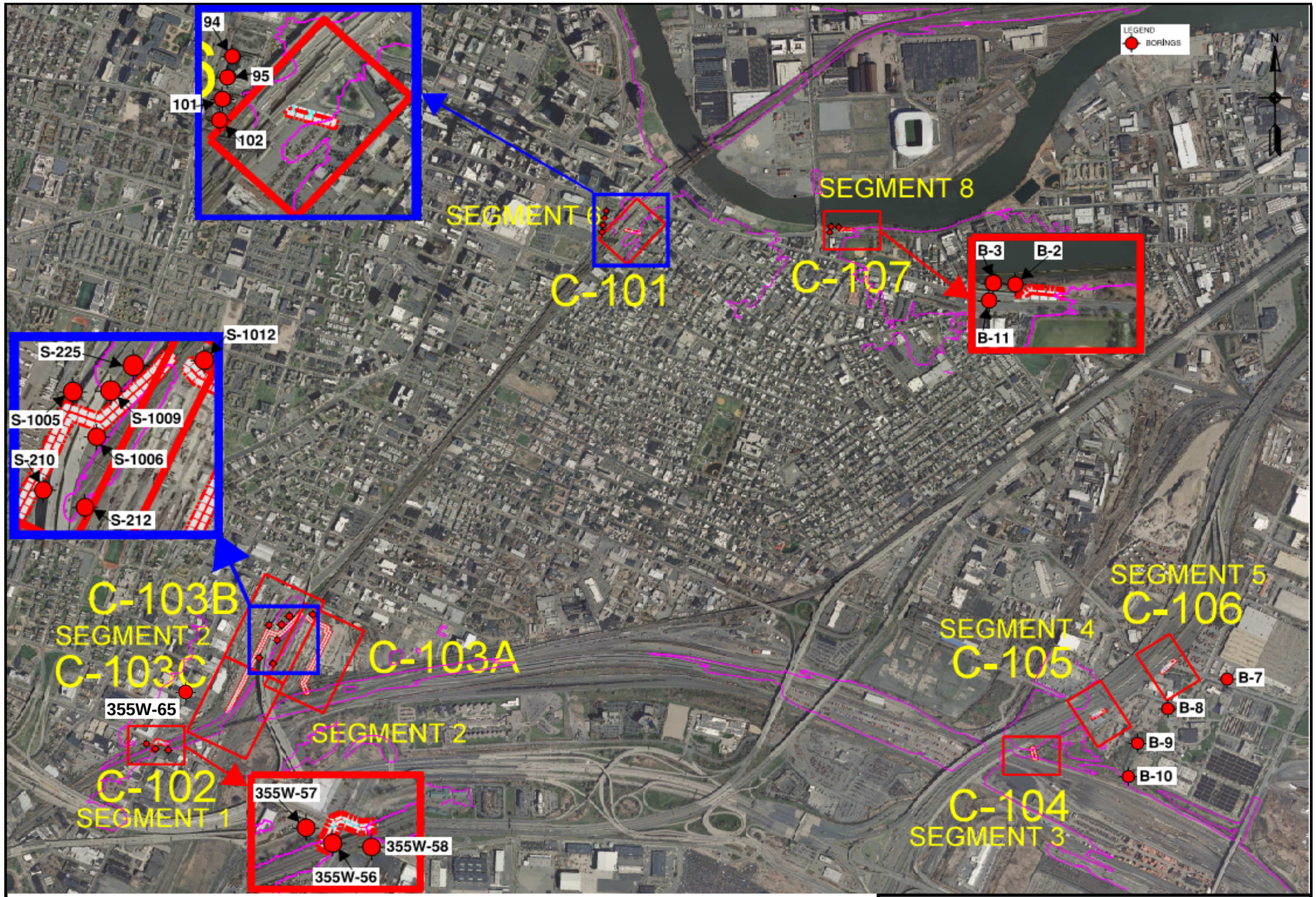
[vi] GEOSTUDIO 2016 software with Slope/W and Seep/W package

[vii] United States Army Corps of Engineers (1991), "*Engineering and Design: Design of Pile Foundations*", *EM 1110-2-2906*, USACE, Washington, DC

[viii] EM 1110-2-1913 (2000), "*Design and Construction of Levees*", US Army Corps of Engineers
[ix] ETL 1110-2-569 (2005), "*Design Guidance for Levee Underseepage*", US Army Corps of Engineers
[x] AECOM (2016), "*Passaic River Tidal – General Reevaluation Report*"
[xi] Das, B. M. (2006), "*Principles of Geotechnical Engineering*" 686. Ontario, Canada: Nelson

FIGURES

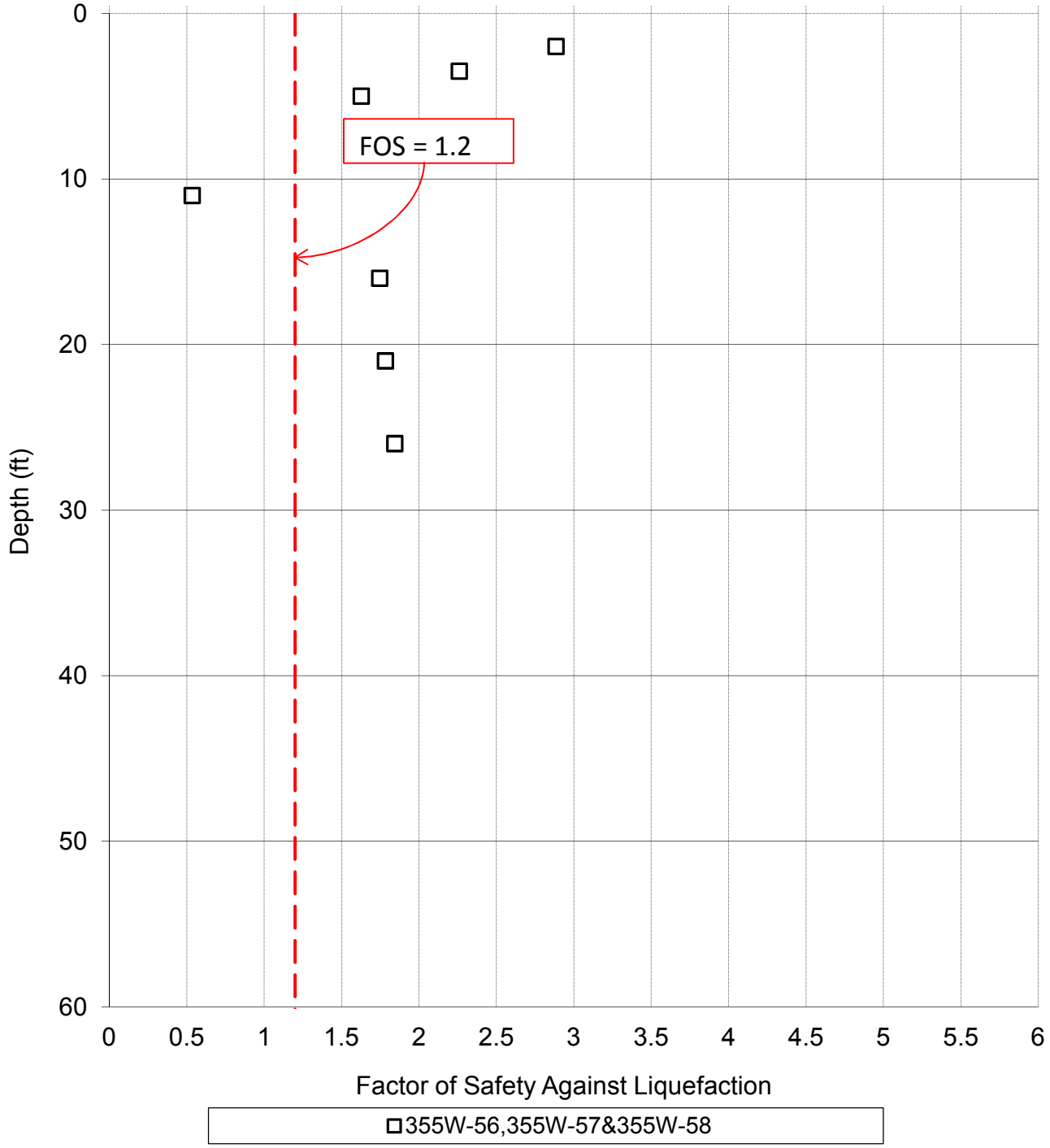


Segments and Boring Locations
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
CLIFTON, NEW JERSEY

PREP'D BY: VK	SCALE: AS SHOWN	PROJ NO: 60442748
CK'D BY: AH	DATE: 08/31/2018	FIG NO: 1

Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Report\Figures\Figure 1 - Segments and Boring Locations.xls



SITE SPECIFIC LIQUEFACTION ANALYSIS
SEGMENT 1
PASSAIC RIVER TIDAL GRR
LOWER PASSAIC RIVER, NEW JERSEY

AECOM

CLIFTON, NEW JERSEY

DR. BY: VK	SCALE: NTS	PROJ. NO. : 60442748
Ck'D. BY: AH	DATE: 08/22/2018	FIG NO: 2

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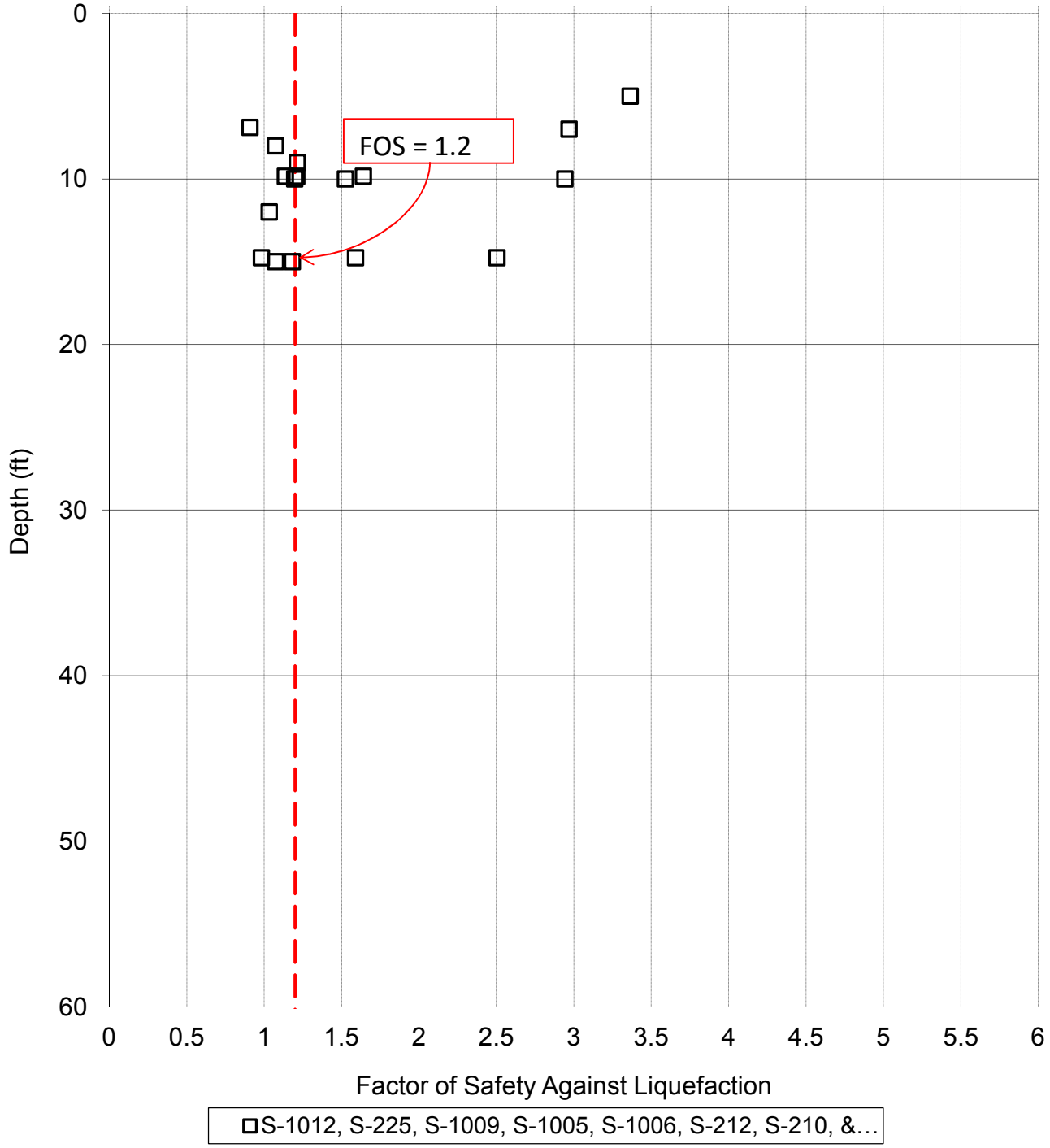
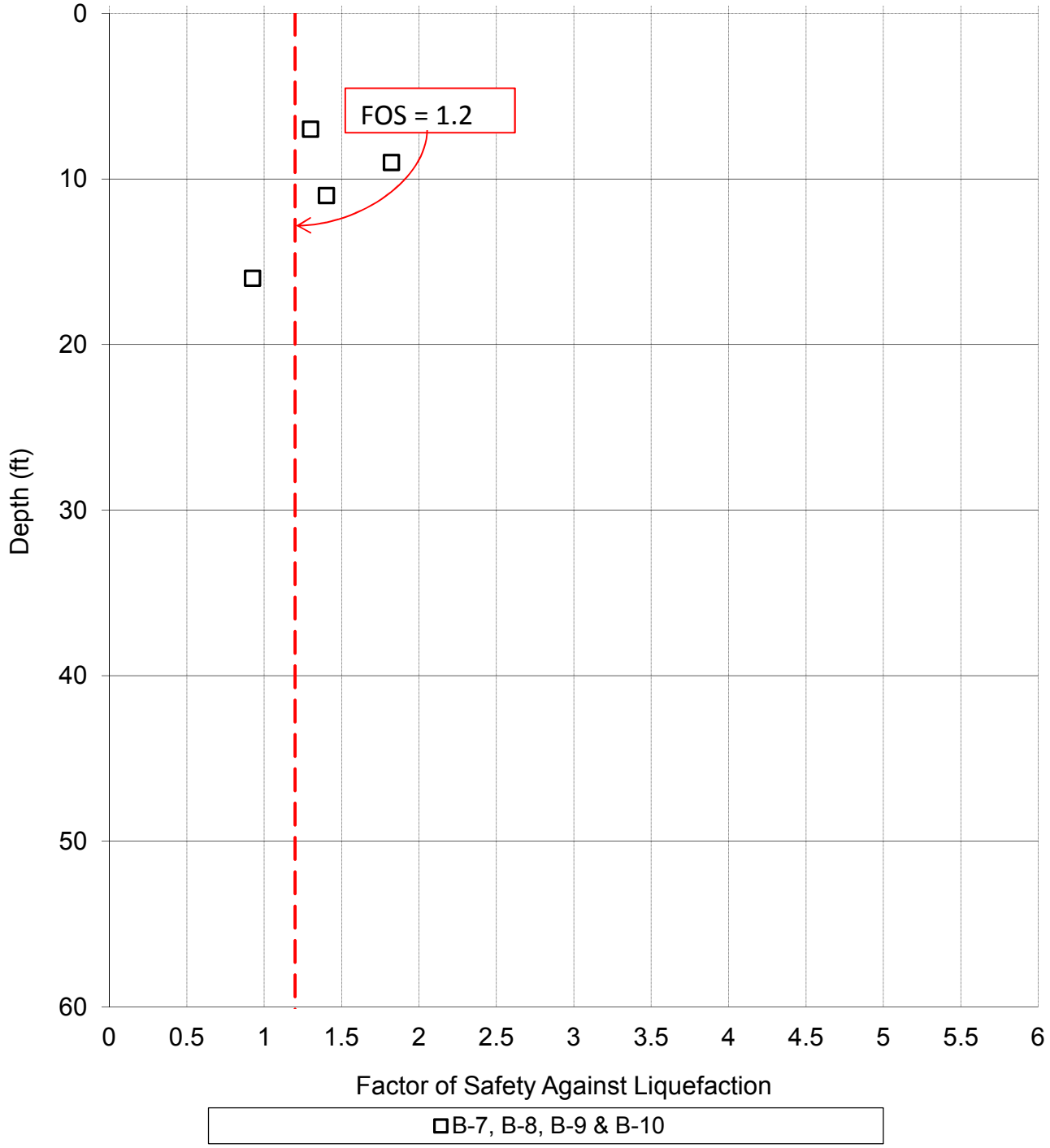


SITE SPECIFIC LIQUEFACTION ANALYSIS
SEGMENT 2
PASSAIC RIVER TIDAL GRR
LOWER PASSAIC RIVER, NEW JERSEY

AECOM
CLIFTON, NEW JERSEY

DR. BY: VK	SCALE: NTS	PROJ. NO. : 60442748
Ck'D. BY: AH	DATE: 08/22/2018	FIG NO: 3

Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Calculations\Liquefaction\Segment 2_Passaic River Tidal_Liquefaction Evaluation.xlsx

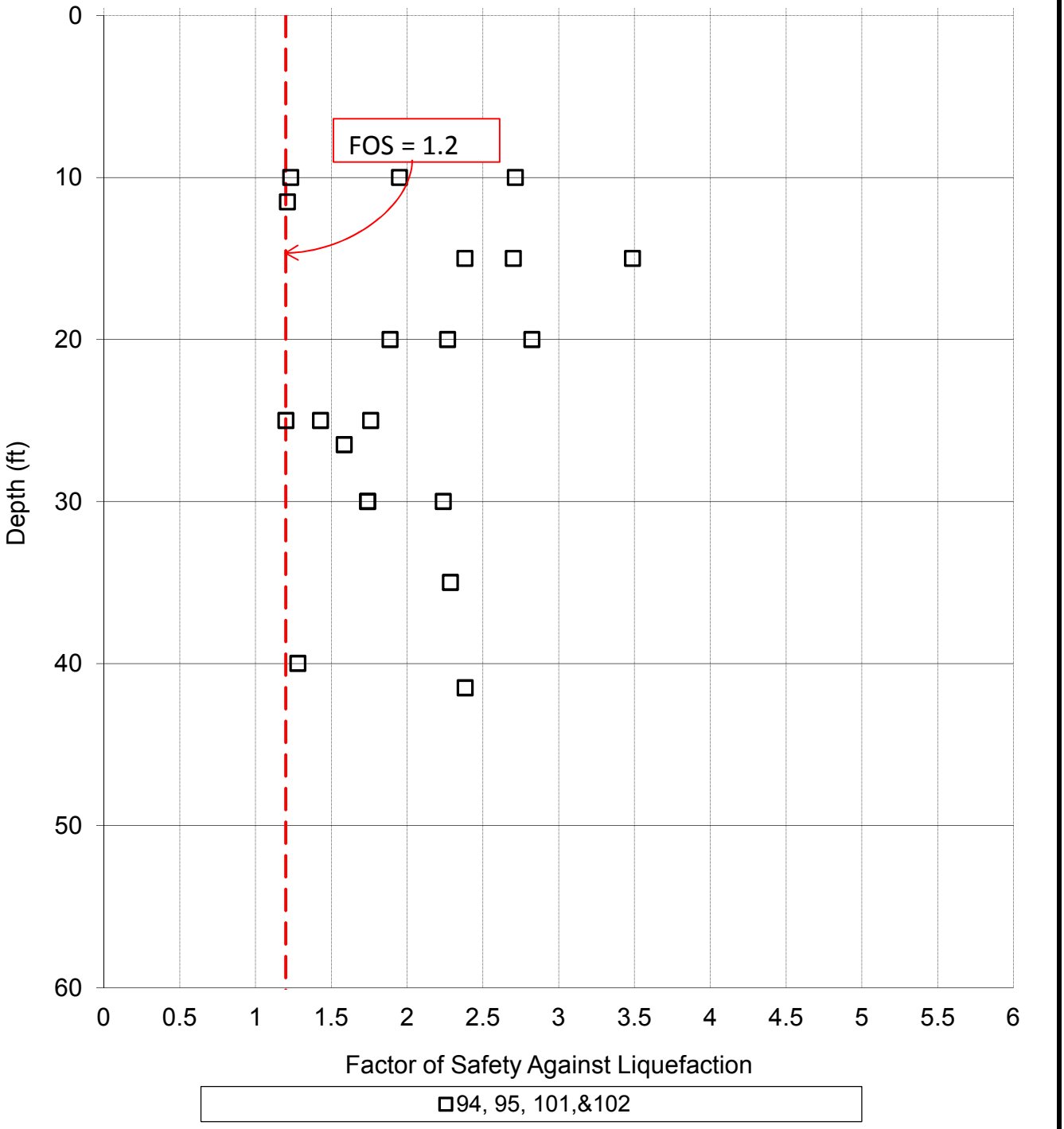


SITE SPECIFIC LIQUEFACTION ANALYSIS
SEGMENT 3, 4 & 5
PASSAIC RIVER TIDAL GRR
LOWER PASSAIC RIVER, NEW JERSEY

AECOM
CLIFTON, NEW JERSEY

DR. BY: VK	SCALE: NTS	PROJ. NO. : 60442748
Ck'D. BY: AH	DATE: 08/22/2018	FIG NO: 4

Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Calculations\Liquefaction\Segment 3,4&5_Passaic River Tidal_Liquefaction Evaluation.xlsx

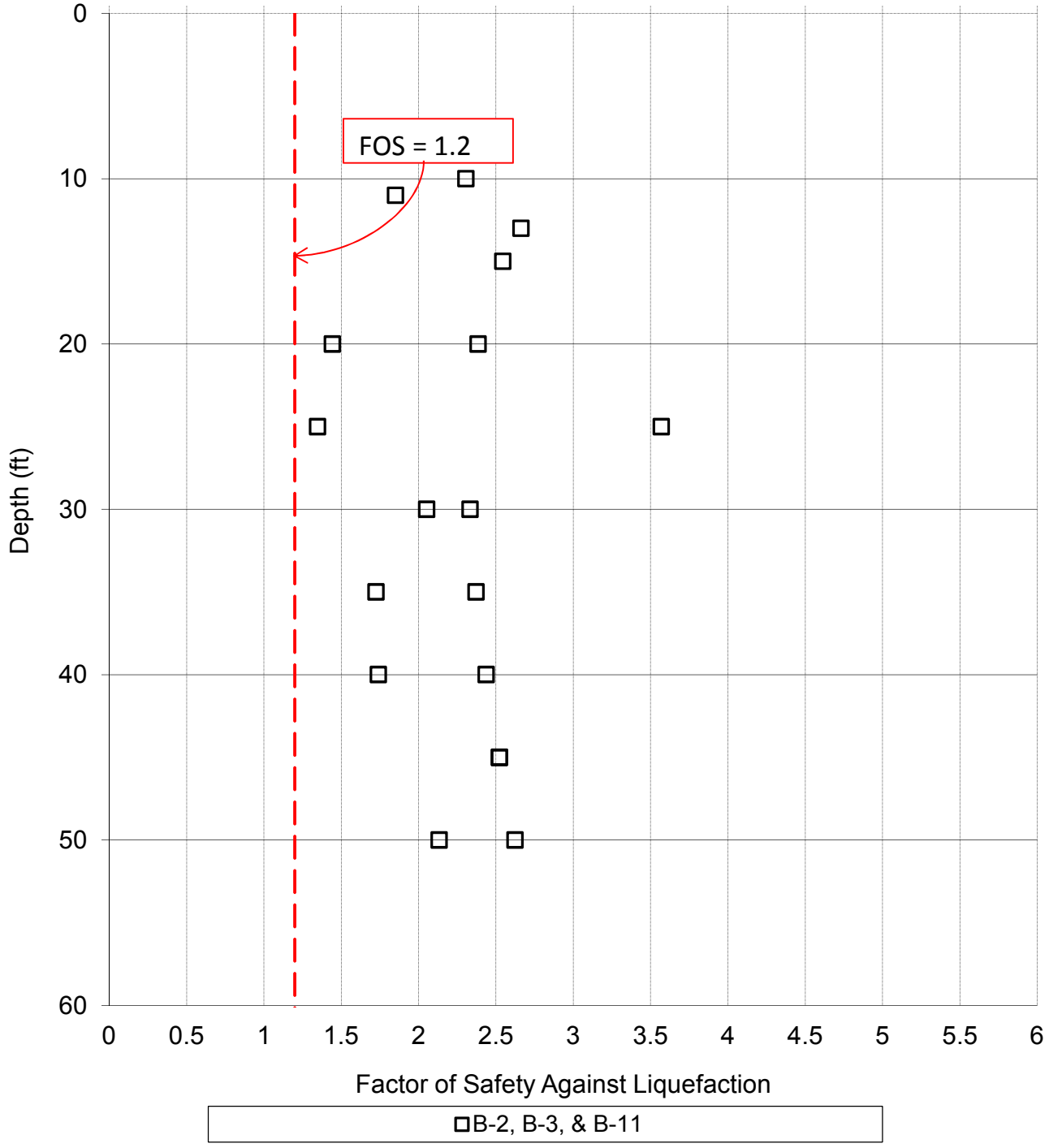


SITE SPECIFIC LIQUEFACTION ANALYSIS
SEGMENT 6
PASSAIC RIVER TIDAL GRR
LOWER PASSAIC RIVER, NEW JERSEY

AECOM
CLIFTON, NEW JERSEY

DR. BY: VK	SCALE: NTS	PROJ. NO. : 60442748
Ck'D. BY: AH	DATE: 08/22/2018	FIG NO: 5

Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Calculations\Liquefaction\Segment 6_Passaic River Tidal_Liquefaction Evaluation.xlsx



SITE SPECIFIC LIQUEFACTION ANALYSIS
SEGMENT 8
PASSAIC RIVER TIDAL GRR
LOWER PASSAIC RIVER, NEW JERSEY

AECOM
CLIFTON, NEW JERSEY

DR. BY: VK	SCALE: NTS	PROJ. NO. : 60442748
Ck'D. BY: AH	DATE: 08/22/2018	FIG NO: 6

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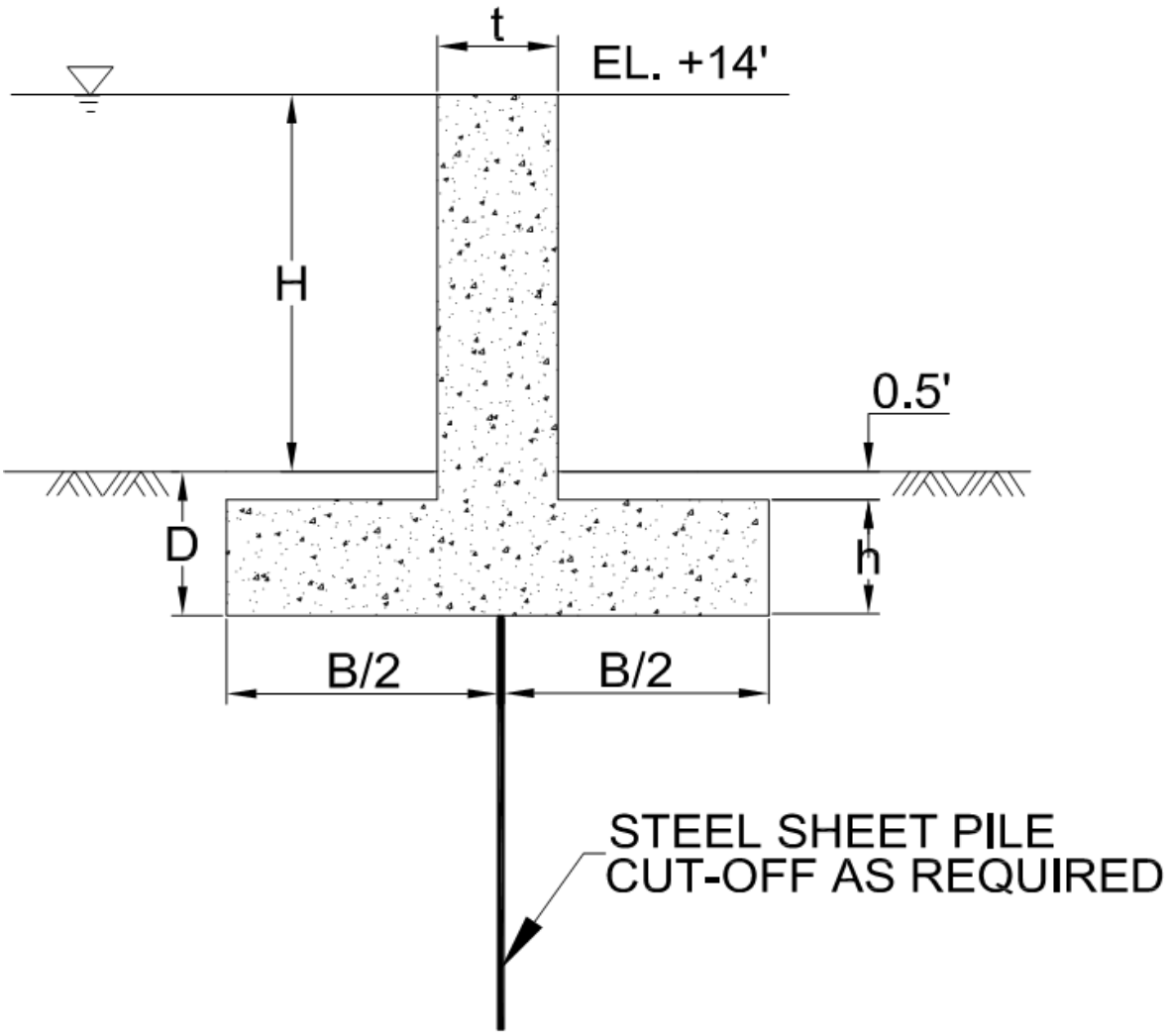
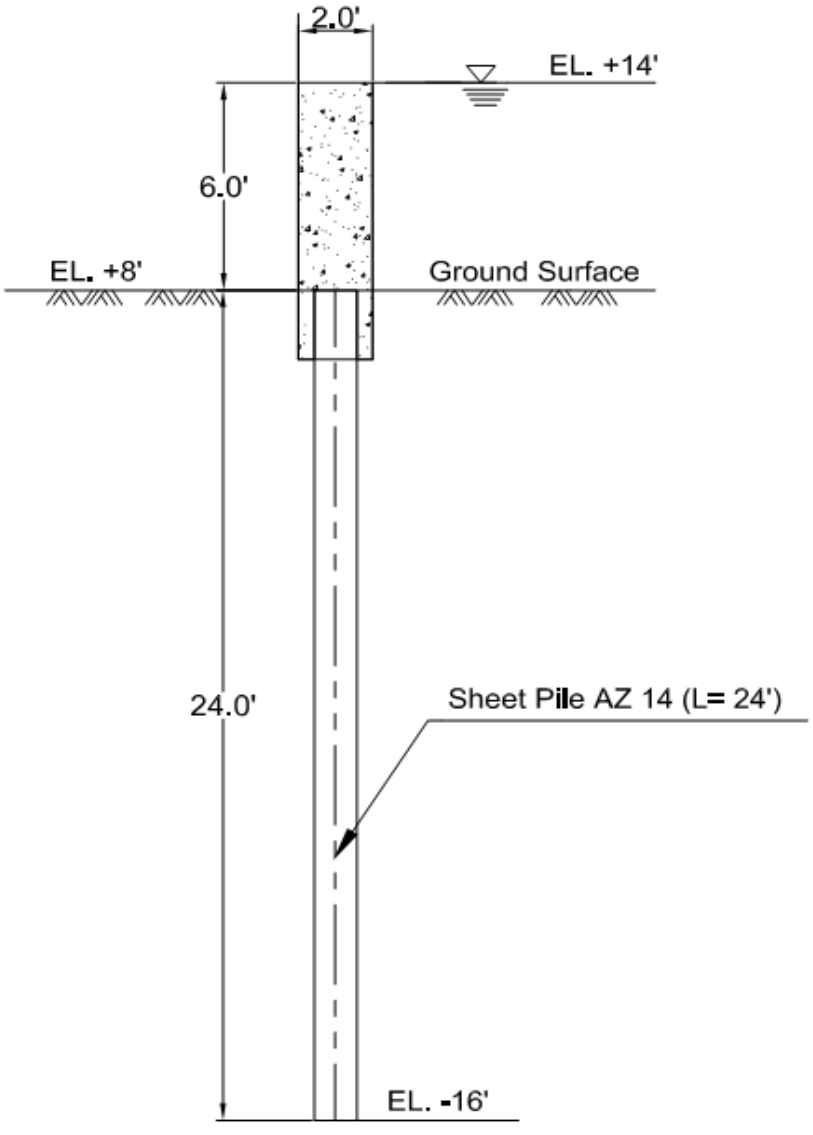


Segment #	H (ft)	B (ft)	D (ft)	h (ft)	t (ft)	Sheet Pile	Sheet Pile Length (ft)	H-Piles
1	4	12	3.0	2.5	1.5	-	-	-
2	6	10	3.0	2.5	1.5	Yes	10	Yes
	8	10	4.0	3.5	1.5	Yes	15	Yes
4	4	10	3.0	2.5	1.5	-	-	-
5	4	10	3.0	2.5	1.5	-	-	-
6	2	6	2.5	2.0	1.5	-	-	-
	4	10	3.0	2.5	1.5	-	-	-
8	2	6	2.5	2.0	1.5	-	-	-

SECTION OF PROPOSED T-WALL
PASSAIC RIVER TIDAL GRR
LOWER PASSAIC RIVER, NEW JERSEY

AECOM
CLIFTON, NEW JERSEY

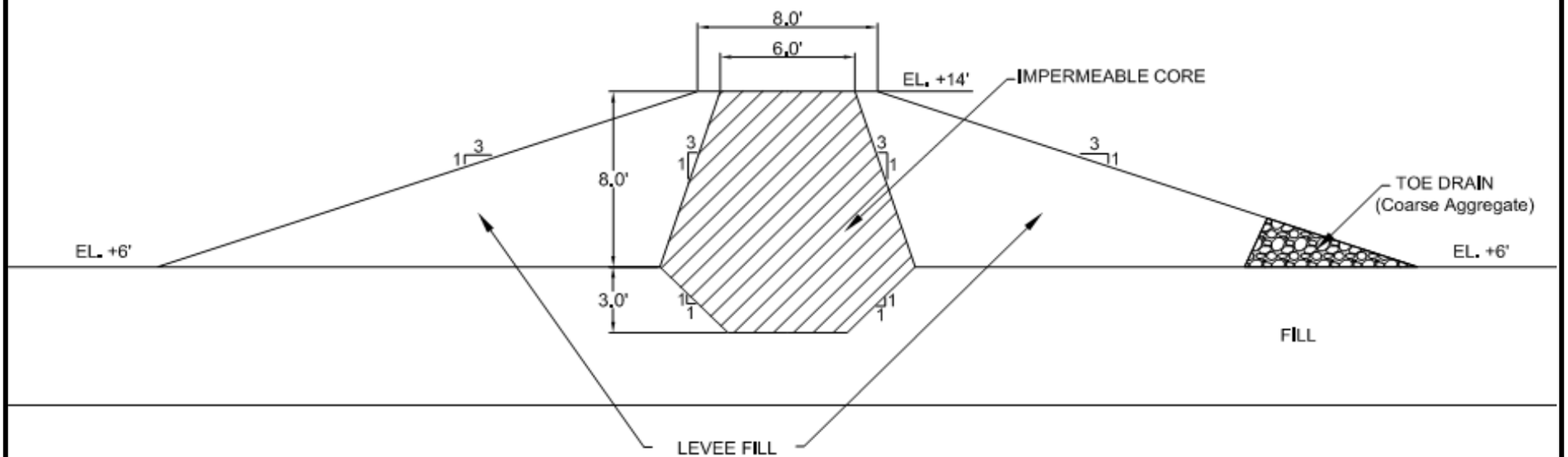
DR. BY: VK	SCALE: NTS	PROJ: 60442748
CH'D. BY: AH	DATE: 08/28/2018	FIG NO: 7



I-Wall Cross-Section for 6' Flood Height

SECTION OF PROPOSED I-WALL SEGMENT 2 PASSAIC RIVER TIDAL GRR LOWER PASSAIC RIVER, NEW JERSEY		
AECOM CLIFTON, NEW JERSEY		
DR. BY: VK	SCALE: NTS	PROJ: 60442748
Ck'D. BY: AH	DATE: 08/31/2018	FIG NO: 8

i:\projects\92c4235\task3\Figure 8 - Section of I-Wall.xls I-Wall



SECTION OF PROPOSED LEVEE
SEGMENT 3
PASSAIC RIVER TIDAL GRR
LOWER PASSAIC RIVER, NEW JERSEY

AECOM
CLIFTON, NEW JERSEY

DR. BY: VK	SCALE: NTS	PROJ: 60442748
Ck'D. BY: AH	DATE: 08/31/2018	FIG NO: 9

i:\projects\92c4235\task3\Figure 9 - Section of Proposed Levee.xls Segment 3

ATTACHMENT A – BORING LOGS

Form SO-2 7/74 NEW JERSEY DEPARTMENT OF TRANSPORTATION

ROUTE: LOCAL NAME: Roadway Boring TEST HOLE NO. 355W - 56

SECTION: Frelinghuysen Avenue, City of Newark

STATION: 106+07 OFFSET: 0 REFERENCE LINE: BL-Frelinghuysen Ave. GROUND LINE ELEVATION:

BORINGS MADE BY: Bowers DATE STARTED: 6/23/78

INSPECTOR: Lounsberry DATE COMPLETED: 6/23/78

Elevation G.W.T.
0 Hr. Filled in Dry 2' Down Date: 6/23/78
24 Hr. SAME Date: 6/26/78
_____ ft. P.P. Installed Date:

	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/6 6/12 12/18			REC.	Sample ID and Profile Change	
	---								3" Blacktop 9" Concrete	1.0'
	9	S-1	1.0'	2.5'	12	15	17	7"	Brown Grey & DULL RED CF GRAVEL and, CF Sand, trace Silt.	
	14									
	14	S-2	2.5'	4.0'	13	11	13	12"	DULL RED CF (+) SAND, some Silt, little (-) F Gravel.	
5	17									
	18	S-3	4.0'	5.5'	25	13	13	9"	DULL RED CF SAND, some Silt, little (-) MF Gravel.	
	24									
	20									
	9									
10	7									10.0'
	5	S-4	10.0'	11.5'	4	3	3	4"	Black & Red Fibrous Organic SILT some, CF Sand, trace (+) F Gravel.	
	6									
	7	S-5	11.5'	13.0'	3	3	3	17"	Light Grey Fibrous Clayey SILT little, F Sand.	
	7									
15	8	S-6	13.0'	14.5'	3	2	2	17"	Black Fibrous Organic SILT little (-), F Sand.	14.5'
	16									
	21	S-7	15.0'	16.5'	12	11	17	9"	DULL RED CF SAND, little Silt, little (+) MF Gravel.	
	28									
	53									
20	56									
	25	S-8	20.0'	21.5'	34	23	24	16"	DULL RED MF SAND, trace (+) Silt, trace F Gravel.	
	34									
	40									
	48									
25	27									
		S-9	25.0'	26.5'	24	21	23	17"	SAME	26.5'
									BOTTOM OF HOLE	
30										
35										
40										

Nominal I.D. of Drive Pipe	2½" XX
Nominal I.D. of Split Barrel Sampler	1½"
Weight of hammer on Drive Pipe	300 lbs.
Weight of hammer on Split Barrel Sampler	140 lbs.
Drop of hammer on Drive Pipe	24"
Drop of hammer on Split Barrel Sampler	30"

Core Dia. _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

The Contractor shall make his own subsurface investigations in order to satisfy himself of the actual subsurface conditions. The Information contained on this log is not warranted to show the actual subsurface conditions. The Contractor agrees that he will make no claims against the State if he finds that the actual conditions do not conform to those indicated by this log.

New Jersey Department of Transportation
Soils Bureau

Approximate Change in Strata _____

Inferred Change in Strata ----------

Form SO-2 7/74

NEW JERSEY DEPARTMENT OF TRANSPORTATION

ROUTE: LOCAL NAME: Roadway Boring TEST HOLE NO. 355W - 57

SECTION: Frelinghuysen Avenue, City of Newark

STATION: 106+47 OFFSET: 138' Lt. REFERENCE LINE: BL-Frelinghuysen Ave. GROUND LINE ELEVATION:

BORINGS MADE BY: Bowers DATE STARTED: 6/22/78

INSPECTOR: Lounsberry DATE COMPLETED: 6/22/78

Elevation G.W.T.

0 Hr. 7' Down Date: 6/22/78

24 Hr. Filled in Dry 8' Down Date: 6/23/78

ft. P.P. Installed Date:

Depth	Casing Blows	Sample No.	Depth From	Depth To	Blows on Spoon 0-6	6-12	12-18	Rec.	Sample ID and Profile Change	Strata
	---								4" Blacktop 6" Concrete	
	9	S-1	1.0'	2.5'	11	16	10	13"	DULL RED CF SAND, some Silt, trace (+) F Gravel	0.9'
	7									
	7	S-2	2.5'	4.0'	9	15	12	10"	DULL RED CF SAND, some (-) Silt, little F	
5	9								Gravel.	
	10	S-3	4.0'	5.5'	9	7	8	7"	DULL RED CF SAND, some (-) Silt, trace (+) F	
	8								Gravel.	
	6									
	4									
10	2									
		S-4	10.0'	11.5'	6	4	2	2"	DULL RED C (+) F GRAVEL.	
										11.5'
		S-5	11.5'	13.0'	2	4	12	8"	DULL RED SILT and CLAY some, CF (+) Sand.	13.0'
15									BOTTOM OF HOLE	
20										
25										
30										
35										
40										

Nominal I.D. of Drive Pipe	2½"	XX
Nominal I.D. of Split Barrel Sampler	1½"	
Weight of hammer on Drive Pipe	300 lbs.	
Weight of hammer on Split Barrel Sampler	140 lbs.	
Drop of hammer on Drive Pipe	24"	
Drop of hammer on Split Barrel Sampler	30"	

Core Dia.

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

The Contractor shall make his own subsurface investigations in order to satisfy himself of the actual subsurface conditions. The Information contained on this log is not warranted to show the actual subsurface conditions. The Contractor agrees that he will make no claims against the State if he finds that the actual conditions do not conform to those indicated by this log.

New Jersey Department of Transportation
Soils Bureau

Approximate Change in Strata

Inferred Change in Strata

Form SO-2 7/74 NEW JERSEY DEPARTMENT OF TRANSPORTATION

ROUTE: LOCAL NAME: Roadway Boring TEST HOLE NO. 355W - 58

SECTION: Frelinghuysen Avenue, City of Newark

STATION: 106+54 OFFSET: 124' Rt. REFERENCE LINE: BL-Frelinghuysen Ave. GROUND LINE ELEVATION:

BORINGS MADE BY: Johannes DATE STARTED: 6/22/78

INSPECTOR: Lounsberry DATE COMPLETED: 6/22/78

Elevation G.W.T.
0 Hr. 2' Down Filled in Dry Date: 6/22/78
24 Hr. Backfilled Date:
_____ ft. P.P. Installed Date:

	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/6	6/12	12/18	REC.	Sample ID and Profile Change	
	---								4" Blacktop 5" Concrete	
	15	S-1	1.0'	2.5'	12	13	15	4"	Blue-Grey & Red CF GRAVEL some, CF Sand, little Silt.	0.8'
	14									
	16	S-2	2.5'	4.0'	15	18	21	4"	Red CF SAND, little Silt, little (+) MF Gravel.	
5	60	S-3	4.0'	5.5'	18	21	27	3"	Red & Blue-Grey MF GRAVEL little, CF Sand.	
	179									
	21									
	12									
	23									
10	72									
		S-4	10.0'	11.5'	42	59	60	2"	Brown & Grey CF SAND, little (-) Silt, some F Gravel.	11.5'
									BOTTOM OF HOLE	
15										
20										
25										
30										
35										
40										

Nominal I.D. of Drive Pipe	2½" XX
Nominal I.D. of Split Barrel Sampler	1½"
Weight of hammer on Drive Pipe	300 lbs.
Weight of hammer on Split Barrel Sampler	140 lbs.
Drop of hammer on Drive Pipe	24"
Drop of hammer on Split Barrel Sampler	30"

Core Dia. _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

The Contractor shall make his own subsurface investigations in order to satisfy himself of the actual subsurface conditions. The Information contained on this log is not warranted to show the actual subsurface conditions. The Contractor agrees that he will make no claims against the State if he finds that the actual conditions do not conform to those indicated by this log.

New Jersey Department of Transportation
Soils Bureau

Approximate Change in Strata _____

Inferred Change in Strata ----------

ARORA and ASSOCIATES, P.C.

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1012

SECTION: 2N COORDINATES: N209265.1989 E650489.8494 FIELD BORING NO.

STATION: 5+214.510 OFFSET: 7.89 RT REFERENCE LINE: NBBROAD GROUND ELEVATION: 3.95

BORING BY: Warren George, Inc. DATE STARTED: 5-10-99

INSPECTOR: Y. S. Choksi DATE COMPLETED: 5-14-99

GROUND WATER ELEVATION

0 Hr. 1.66 Date: 5-14-99

24 Hr. 1.21 Date: 5-17-99

9.00 m P.P. Installed Date: 5-14-99

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-1	0.00	0.45	12	18	23	300	Brown and Gray cf SAND, some (-) cf Gravel, trace (+) Silt (FILL)	
	BIT									
	MUD									
1.5										1.20
		S-2	1.50	1.95	25	30	22	20	Gray c GRAVEL (FILL)	
		S-3	1.95	2.40	10	16	21	20	Gray cf GRAVEL (FILL)	2.40
		S-4	2.40	2.85	10	8	9	250	Reddish Brown mf SAND, little (-) Silt	
3										
		S-5	3.00	3.45	6	8	8	400	Reddish Brown cf SAND, trace Silt	
4.5										
		S-6	4.50	4.95	24	27	21	300	Reddish Brown cf SAND, little (-) Silt	
6										
		S-7	6.00	6.45	7	6	5	200	Reddish Brown cf SAND, trace Silt	
7.5										
		S-8	7.50	7.95	9	13	18	200	Reddish Brown f SAND, and (-) Silt	
9										
		S-9	9.00	9.45	12	17	19	250	SAME	
10.5										
		S-10	10.50	10.95	39	36	29	350	Reddish Brown f SAND, and Clayey Silt	
12										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

The subsurface information shown hereon was obtained for the Owner's design and estimate purposes. It is made available to authorized users only that they may have access to the same information available to the Owner. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Approximate Change in Strata ———————

Inferred Change in Strata — — — — — —

H:\1237\ENGINEER\Bonng\S-1012.wpd

NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1012

SECTION: 2N COORDINATES: N209265.1989 E650489.8494 FIELD BORING NO.

STATION: 5+214.510 OFFSET: 7.89 RT REFERENCE LINE: NBBROAD GROUND ELEVATION: 3.95

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-11	12.00	12.45	41	42	51	300	Reddish Brown f SAND, and Clayey Silt	
	BIT									
	MUD									
13.5										13.50
		S-12	13.50	13.95	43	41	38	300	Reddish Brown Clayey SILT, and f Sand	
15										
		S-13	15.00	15.45	14	12	14	350	Reddish Brown SILT & CLAY PP = 0.25 kg/cm²	
16.5										
		S-14	16.50	16.95	17	19	22	200	Reddish Brown CLAY & SILT, trace (+) lenses of f Sand	
									PP = 0.30 kg/cm²	
18										
		UD-1	18.00	18.60	-	-	-	600	Reddish Brown Silty CLAY Bottom PP = 0.75 kg/cm²	
		S-15	18.60	19.05	12	12	13	350	Reddish Brown Silty CLAY, trace (+) lenses of f Sand	
									PP = 0.50 kg/cm²	
19.5										
		S-16	19.50	19.95	14	16	23	250	Reddish Brown Silty CLAY, trace (+) lenses of f Sand,	
									trace f Gravel	
										20.70
21										
		S-17	21.00	21.40	55	86	100/100	300	Reddish Brown f SAND, some Clayey Silt	
22.5										
		S-18	22.50	22.95	32	38	50	250	Reddish Brown f SAND, some Silt, trace (-) mf Gravel	
24										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

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The subsurface information shown hereon was obtained for the Owner's design and estimate purposes. It is made available to authorized users only that they may have access to the same information available to the Owner. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1012

SECTION: 2N COORDINATES: N209265.1989 E650489.8494 FIELD BORING NO.

STATION: 5+214.510 OFFSET: 7.89 RT REFERENCE LINE: NBBROAD GROUND ELEVATION: 3.95

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-19	24.00	24.45	18	20	19	250	Reddish Brown cf SAND, trace (-) Silt, trace (-) f Gravel	
	BIT									
	MUD									
25.5										
		S-20	25.50	25.95	21	20	28	200	SAME	
27										
		S-21	27.00	27.45	25	58	35	150	Reddish Brown cf SAND, little cf Gravel (round edged rock fragments), trace Silt	
28.5										
		S-22	28.50	28.95	19	25	31	300	Reddish Brown cf SAND, trace (+) cf Gravel (rock fragments), trace Silt	
30										
		S-23	30.00	30.45	32	30	39	150	Reddish Brown cf SAND, some cf Gravel (round edged rock fragments), trace Silt	
										30.90
31.5										
		S-24	31.50	31.95	30	31	32	250	Reddish Brown cf GRAVEL (round edged rock fragments), little (+) cf Sand, trace Silt	
										32.70
33										
		S-25	33.00	33.45	45	61	71	300	Reddish Brown cf SAND, some (-) cf Gravel (round edged rock fragments), trace Silt	
										34.20
34.5										
		S-26	34.50	34.95	63	67	72	250	Reddish Brown cf GRAVEL (round edged rock fragments), some cf Sand, trace Silt	
36										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

The subsurface information shown hereon was obtained for the Owner's design and estimate purposes. It is made available to authorized users only that they may have access to the same information available to the Owner. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Approximate Change in Strata

Inferred Change in Strata

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NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1012

SECTION: 2N COORDINATES: N209265.1989 E650489.8494 FIELD BORING NO.

STATION: 5+214.510 OFFSET: 7.89 RT REFERENCE LINE: NBBROAD GROUND ELEVATION: 3.95

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-27	36.00	36.25	78	100/100	-	150	Reddish Brown cf GRAVEL (round edged rock fragments), trace (+) cf Sand, trace (-) Silt	
	BIT									
	MUD									
37.5										
		S-28	37.50	37.60	100/100	-	-	75	SAME	
39										
		S-29	39.00	39.45	52	58	62	300	Reddish Brown cf GRAVEL (round edged rock fragments), little cf Sand, trace (-) Silt	
40.5										
		S-30	40.50	41.95	36	32	36	250	SAME	
42										
		S-31	42.00	42.45	50	46	49	300	Reddish Brown cf GRAVEL (round edged rock fragments), some cf Sand, trace Silt	
43.5										
		S-32	43.50	43.95	38	49	55	200	SAME	
45										44.70
		S-33	45.00	45.45	25	27	30	300	Reddish Brown mf SAND, little Silt	
46.5										
		S-34	46.50	46.95	33	40	42	350	Reddish Brown f SAND, some Silt, trace f Gravel (rock fragments)	
48										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

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NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1012

SECTION: 2N COORDINATES: N209265.1989 E650489.8494 FIELD BORING NO.

STATION: 5+214.510 OFFSET: 7.89 RT REFERENCE LINE: NBBROAD GROUND ELEVATION: 3.95

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-35	48.00	48.45	38	47	53	300	Reddish Brown f SAND, some Silt, trace (-) f Gravel (rock fragments)	
	BIT									
	MUD									
49.5										
		S-36	49.50	49.95	31	37	46	250	SAME	
51										
		S-37	51.00	51.45	35	44	47	300	Reddish Brown cf SAND, little mf Gravel, (rock fragments), trace (+) Silt	
52.5										
		S-38	52.50	52.95	40	54	55	350	Reddish Brown mf SAND, trace (+) Silt, trace f Gravel (rock fragments)	
54										
		S-39	54.00	54.45	40	42	46	375	Reddish Brown cf SAND, trace f Gravel (rock fragments), trace Silt	
55.5										
		S-40	55.50	55.63	100/130	-	-	100	Reddish Brown cf SAND, little (+) cf Gravel (rock fragments), trace Silt	
57									Reddish Brown cf SAND, trace (+) mf Gravel (rock	
		S-41	57.00	57.08	100/80	-	-	75	fragments), trace Silt	
58.5									Bottom of Hole 57.08 m	
60										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

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NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-225
SECTION: 2N COORDINATES: N209248.1808 E650425.9558
STATION: 1+539.2881 OFFSET: 23.924 RT REF. LINE: RT21COMM G. L. EL: 3.34
BORINGS MADE BY: Site Engineers DATE STARTED: 10/20/94
INSPECTOR: A. Ari DATE COMPLETED: 10/20/94

El. G.W.T.
0 HR. 0.90 DATE: 10/21/94
24 HR. 0.90 DATE: 10/22/94
_____ m P.P. Inst. DATE:

Casing Blows	Sample No.	Depth		Blows on Spoon 0 / / 6	6 / /12	12/ /18	Rec.	Sample ID and Profiles Changes	
								8" CONCRETE PAVEMENT over 4" DGA	1.0
	S-1	1.0'	2.5'	7	6	8	1.5'	Black cf SAND, little cf Gravel, trace Silt (Fill)	
5									
A	S-2	5.0'	6.5'	7	7	12	1.5'	Reddish Brown cf SAND, trace f Gravel, trace Silt	
U									
G	S-3	6.5'	8.0'	9	8	7	1.5'	Same	
E	S-4	8.0'	9.5'	5	4	4	1.5'	Same	
10 R									
S	S-5	10.0'	11.5'	5	5	6	1.5'	Reddish Brown cf SAND, trace Silt	
15									
	S-6	15.0'	16.5'	6	5	7	1.0'	Same	
20									
	S-7	20.0'	21.5'	5	6	6	1.0'	Same	
25									
	S-8	25.0'	26.5'	5	7	13	1.5'	Reddish Brown mf SAND, some Silt	
30									
	S-9	30.0'	31.5'	10	11	10	1.5'	Reddish Brown f SAND, some Silt	
35									
	S-10	35.0'	36.5'	12	13	13	1.5'	Same	
									39.0
40 /									

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" 4"
Nominal I.D. of Split Barrel Sampler 1 1/2"
Weight of hammer on Drive Pipe 300 lbs.
Weight of hammer on Split Barrel Sampler 140 lbs.
Drop of hammer on Drive Pipe 24"
Drop of hammer on Split Barrel Sampler 30"

The subsurface information shown hereon was obtained for A & A design and estimated purposes. It is made available to authorized users only that they may have access to the same information available to A & A. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Core Dia. _____

Approximate Change in Strata _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata - - - - - - -

NOTE: Station, Offset, Ground Level and Groundwater Elevations are presented in metric.

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Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-225
SECTION: 2N COORDINATES: N209248.1808 E650425.9558
STATION: 1+539.2881 OFFSET: 23.924 RT REF. LINE: RT21COMM G. L. EL: 3.34
BORINGS MADE BY: Site Engineers DATE STARTED: 10/20/94
INSPECTOR: A. Ari DATE COMPLETED: 10/20/94

El. G.W.T.
0 HR. 0.90 DATE: 10/21/94
24 HR. 0.90 DATE: 10/22/94
_____ m P.P. Inst.
DATE:

Casing Blows	Sample No.	Depth		Blows on Spoon 0 / / 6	6 / /12	12/ /18	Rec.	Sample ID and Profiles Changes	
	S-11	40.0'	41.5'	4	5	7	1.3'	Reddish Brown Clayey SILT, little f Sand (V = 0.25 TSF H = 0.25 TSF)	
45 A U G E R	S-12	45.0'	46.5'	8	7	9	1.0'	Reddish Brown SILT & CLAY, trace f Sand (V = 0.10 TSF H = 0.30 TSF)	
50 S									50.0
	S-13	50.0'	51.5'	8	8	9	1.5'	Reddish Brown mf SAND, little (-) Clayey Silt	
55	S-14	55.0'	56.5'	7	7	10	1.5'	Reddish Brown cf SAND, trace Silt	
60	S-15	60.0'	61.5'	11	11	15	1.5'	Same	
65	S-16	65.0'	66.5'	13	18	16	1.5'	Same	
70	S-17	70.0'	71.5'	12	15	15	1.0'	Same	
75	S-18	75.0'	76.5'	14	15	15	1.5'	Same	
80									

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" 4"
Nominal I.D. of Split Barrel Sampler 1 1/2"
Weight of hammer on Drive Pipe 300 lbs.
Weight of hammer on Split Barrel Sampler 140 lbs.
Drop of hammer on Drive Pipe 24"
Drop of hammer on Split Barrel Sampler 30"

The subsurface information shown hereon was obtained for A & A design and estimated purposes. It is made available to authorized users only that they may have access to the same information available to A & A. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Core Dia. _____

Approximate Change in Strata _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata _ _ _ _ _ _ _

NOTE: Station, Offset, Ground Level and Groundwater Elevations are presented in metric.

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Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-225
SECTION: 2N COORDINATES: N209248.1808 E650425.9558
STATION: 1+539.2881 OFFSET: 23.924 RT REF. LINE: RT21COMM G. L. EL: 3.34
BORINGS MADE BY: Site Engineers DATE STARTED: 10/20/94
INSPECTOR: A. Ari DATE COMPLETED: 10/20/94

El. G.W.T.
0 HR. 0.90 DATE: 10/21/94
24 HR. 0.90 DATE: 10/22/94
_____ m P.P. Inst. DATE:

Casing Blows	Sample No. Depth			Blows on Spoon 0 / / 6	6 / /12	12/ /18	Rec.	Sample ID and Profiles Changes	
	S-19	80.0'	81.5'	14	16	15	1.0'	Same	
								Bottom of Hole	81.5
85									
90									
95									
100									
105									
110									
115									
120									

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" 4"
Nominal I.D. of Split Barrel Sampler 1 1/2"
Weight of hammer on Drive Pipe 300 lbs.
Weight of hammer on Split Barrel Sampler 140 lbs.
Drop of hammer on Drive Pipe 24"
Drop of hammer on Split Barrel Sampler 30"

The subsurface information shown hereon was obtained for A & A design and estimated purposes. It is made available to authorized users only that they may have access to the same information available to A & A. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Core Dia. _____

Approximate Change in Strata _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata - - - - - - -

NOTE: Station, Offset, Ground Level and Groundwater Elevations are presented in metric.

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ARORA and ASSOCIATES, P.C.

ROUTE: RT. 21 | LOCAL NAME: RT. 21 Viaduct | BORING NO. S-1009

SECTION: 2N | COORDINATES: N209202.5340 E650402.2717 | FIELD BORING NO.

STATION: 5+108.201 | OFFSET: 7.60 LT | REFERENCE LINE: NBBROAD | GROUND ELEVATION: 3.51

BORING BY: Warren George, Inc. | DATE STARTED: 5-26-99

INSPECTOR: Y. S. Choksi/ H. S. Bal | DATE COMPLETED: 6-2-99

GROUND WATER ELEVATION
0 Hr. 1.28 Date: 6-2-99
24 Hr. Filled for Safety Date: 6-2-99
P.P. Installed

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER								300 mm Concrete	0.30
	BIT	S-1	0.30	0.75	37	75	66	300	Gray cf SAND, little cf Gravel, trace (+) Clayey Silt (FILL)	
	MUD									
1.5										1.50
		S-2	1.50	1.95	2	3	2	200	Dk Brownish Gray organic Silty CLAY w/ vegetation roots	
									(FILL) PP = 0.50 kg/cm²	1.95
		S-3	1.95	2.40	2	2	2	150	Grayish Brown cf SAND, some Clayey Silt, trace mf Gravel (FILL)	2.40
		UD-1	2.40	3.00				450	Dk Gray organic Silty CLAY, little mf Gravel (FILL)	
3									(PP was not possible)	
		S-4	3.00	3.45	3	5	9	150	Greenish Gray and Brown CLAY & SILT, trace (+) f Sand (FILL)	3.30
4.5										
		S-5	4.50	4.95	7	6	12	200	Reddish Brown cf SAND, some Clayey Silt, trace f Gravel	
6										
		S-6	6.00	6.45	7	7	10	300	Reddish Brown cf SAND, little Silt, trace f Gravel	
7.5										
		S-7	7.50	7.95	10	11	13	300	SAME	
9										
		S-8	9.00	9.45	8	10	10	350	Reddish Brown mf SAND, little Silt, trace f Gravel	
10.5										
		S-9	10.50	10.95	9	12	13	450	Reddish Brown mf SAND, little (-) Silt	
12										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	NX

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

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NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1009

SECTION: 2N COORDINATES: N209202.5340 E650402.2717 FIELD BORING NO.

STATION: 5+108.201 OFFSET: 7.60 LT REFERENCE LINE: NBBROAD GROUND ELEVATION: 3.51

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-10	12.00	12.45	9	12	16	300	Reddish Brown f SAND, some Clayey Silt	
	BIT									
	MUD									
13.5										13.50
		S-11	13.50	13.95	13	14	17	450	Reddish Brown varved CLAY & SILT w/ lenses of f Sand PP = 0.50 kg/cm²	
		UD-2	14.40	15.00				500	SAME Bottom PP = 0.50 kg/cm²	
15										15.00
		S-12	15.00	15.45	23	28	43	450	Reddish Brown f SAND, some (+) Silt	
16.5										16.50
		S-13	16.50	16.95	44	33	27	400	Reddish Brown SILT & CLAY, little f Sand PP = 0.50 kg/cm²	
18										
		S-14	18.00	18.45	8	7	10	450	Reddish Brown CLAY & SILT PP = 0.50 kg/cm²	
		UD-3	18.90	19.50	-	-	-	500	Reddish Brown Silty CLAY Bottom PP = 0.50 kg/cm²	
19.5										
		S-15	19.50	19.95	7	9	9	450	SAME PP <0.25 kg/cm²	
21										
		S-16	21.00	21.45	13	13	15	350	Reddish Brown CLAY & SILT w/ lenses of f Sand PP = 0.50 kg/cm²	
22.5										
		S-17	22.50	22.95	30	37	48	325	Reddish Brown Clayey SILT, little (-) f Sand PP = 1.50 kg/cm²	
24										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	NX

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1009

SECTION: 2N COORDINATES: N209202.5340 E650402.2717 FIELD BORING NO.

STATION: 5+108.201 OFFSET: 7.60 LT REFERENCE LINE: NBBROAD GROUND ELEVATION: 3.51

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-18	24.00	24.45	30	33	37	300	Reddish Brown Clayey SILT, trace f Sand	
	BIT								PP = 1.50 kg/cm²	
	MUD									24.90
25.5										
		S-19	25.50	25.95	20	23	24	300	Reddish Brown f SAND, some Silt	
27										
		S-20	27.00	27.45	21	24	26	300	Reddish Brown mf SAND, trace Silt, trace f Gravel	
28.5										
		S-21	28.50	28.95	25	32	35	450	Reddish Brown f SAND, some (+) Clayey Silt, trace (-) f Gravel	
30										
		S-22	30.00	30.45	25	26	40	300	SAME	
31.5										
		S-23	31.50	31.95	26	32	38	450	SAME	
33										
		S-24	33.00	33.45	18	27	30	150	SAME	
34.5										
		S-25	34.50	34.95	16	24	29	300	SAME	
36										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	NX

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

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NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1009

SECTION: 2N COORDINATES: N209202.5340 E650402.2717 FIELD BORING NO.

STATION: 5+108.201 OFFSET: 7.60 LT REFERENCE LINE: NBBROAD GROUND ELEVATION: 3.51

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-26	36.00	36.45	25	28	30	325	Reddish Brown cf SAND, and Clayey Silt, little cf Gravel	
	BIT									
	MUD									
37.5										
		S-27	37.50	37.95	27	27	38	300	Reddish Brown f SAND, and Clayey Silt, trace (+) f Gravel	
39										
		S-28	39.00	39.45	25	27	28	450	Reddish Brown cf SAND, little Clayey Silt, trace f Gravel	
40.5										
		S-29	40.50	40.95	33	36	38	200	SAME w/ boulders	
42										
		S-30	42.00	42.44	40	68	100/140	300	SAME w/ boulders	
43.5										
		S-31	43.50	43.95	34	58	66	300	Reddish Brown cf GRAVEL (round and sharp edged), some cf Sand, trace Clayey Silt	
45										
		S-32	45.00	45.45	37	43	70	200	SAME w/ Decomposed SHALE	
46.5										
		S-33	46.50	46.77	54	100/120	-	150	SAME w/ boulders and Decomposed SHALE	
48										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	NX

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1009

SECTION: 2N COORDINATES: N209202.5340 E650402.2717 FIELD BORING NO.

STATION: 5+108.201 OFFSET: 7.60 LT REFERENCE LINE: NBBROAD GROUND ELEVATION: 3.51

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-34	48.00	48.45	23	35	37	300	Reddish Brown f SAND, and Clayey Silt	
	BIT									
	MUD									
49.5										
		S-35	49.50	49.95	26	25	25	200	Reddish Brown mf SAND, little Silt & Clay	
51										
		S-36	51.00	51.12	100/120	-	-	50	Red and Gray cf GRAVEL (round), little cf Sand, trace Silt w/ boulders	
52.5										52.50
		S-37	52.50	52.95	37	64	32	200	Reddish Brown highly Decomposed SHALE, friable to cf SAND, some mf Gravel, little Silt & Clay	
54										
		S-38	54.00	54.10	100/100	-	-	50	SAME	
									Top of Rock	55.20
55.5	CORING	C-1	55.20	56.70		REC	1125	75%	Reddish Brown slightly weathered, medium spaced fractured, soft SHALE	
						RQD	1050	70%		
57		C-2	56.70	58.20		REC	1350	90%	SAME w/ Gray veins	
						RQD	1125	75%		
58.5									Bottom of Hole 58.20 m	
60										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	NX

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

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NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ARORA and ASSOCIATES, P.C.

ROUTE: RT. 21 | LOCAL NAME: RT. 21 Viaduct | BORING NO. S-1005

SECTION: 2N | COORDINATES: N209198.4307 E650350.6688 | FIELD BORING NO.

STATION: 1+457.114 | OFFSET: 15.27 LT | REFERENCE LINE: RT21ML | GROUND ELEVATION: 3.10

BORING BY: Warren George, Inc. | DATE STARTED: 5-18-99

INSPECTOR: H. S. Bal | DATE COMPLETED: 5-24-99

GROUND WATER ELEVATION
0 Hr. 0.97 Date: 5-24-99
24 Hr. 0.81 Date: 5-25-99
m P.P. Installed

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	CASING	S-1	0.00	0.45	10	20	24	400	Brown and Dk Gray cf SAND, little Clayey Silt, little mf Gravel w/ marble pieces (FILL)	
1.5										
		S-2	1.50	1.95	5	3	5	350	Reddish Brown cf SAND, some cf Gravel, little (+) Silt & Clay (FILL)	
		S-3	1.95	2.40	4	4	2	100	Gray and Brown cf GRAVEL, trace cf Sand, trace Silt (FILL)	
		S-4	2.40	2.85	3	2	1	450	Brown SILT & CLAY, and cf Sand, little (+) cf Gravel, trace organics (FILL) PP <0.25 kg/cm²	
3										
		UD-1	2.85	3.45				600	SAME Top PP <0.25 kg/cm² Bottom PP = 0.50 kg/cm²	3.30
		S-5	3.45	3.90	4	6	5	50	Brown cf SAND, trace Silt Brown cf GRAVEL, trace cf Sand, trace Silt	
4.5										
		S-6	4.50	4.95	4	4	3	400	Reddish Brown cf SAND, little Silt, trace (-) f Gravel	
6										
		S-7	6.00	6.45	4	9	9	200	Reddish Brown f SAND, little Silt, trace mf Gravel	
7.5										7.50
		S-8	7.50	7.95	11	15	14	300	Reddish Brown Clayey SILT, little f Sand PP <0.25 kg/cm²	
9										
	ROLLER	S-9	9.00	9.45	9	15	15	350	SAME	
	BIT									
	MUD									
		UD-2	9.90	10.50				575	SAME Top PP <0.25 kg/cm² Bottom PP = 0.50 kg/cm²	
10.5										
		S-10	10.50	10.95	9	13	17	450	SAME PP = 0.35 kg/cm²	
12										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1005

SECTION: 2N COORDINATES: N209198.4307 E650350.6688 FIELD BORING NO.

STATION: 1+457.114 OFFSET: 15.27 LT REFERENCE LINE: RT21ML GROUND ELEVATION: 3.10

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-11	12.00	12.45	15	30	24	450	Reddish Brown Clayey SILT, some f Sand	
	BIT								PP = 0.50 kg/cm²	
	MUD									
13.5										
		S-12	13.50	13.95	15	17	17	150	Reddish Brown CLAY & SILT, some f Sand	
									PP = 0.50 kg/cm²	
									(Tube attempted twice)	
15										
		S-13	15.00	15.45	9	11	12	450	Reddish Brown SILT & CLAY, some f Sand, trace f Gravel	
									PP = 0.65 kg/cm²	
16.5										16.50
		S-14	16.50	16.95	10	10	9	400	Reddish Brown cf SAND, trace Silt, trace f Gravel	
18										
		S-15	18.00	18.45	12	14	17	450	SAME	
19.5										
		S-16	19.50	19.95	8	12	12	300	Reddish Brown cf SAND, little Silt, trace cf Gravel	
21										
		S-17	21.00	21.45	11	11	13	350	Reddish Brown cf SAND, little cf Gravel, trace Silt	
22.5										
		S-18	22.50	22.95	11	14	12	300	SAME	
24										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ROUTE: RT. 21	LOCAL NAME: RT. 21 Viaduct	BORING NO. S-1005
SECTION: 2N	COORDINATES: N209198.4307 E650350.6688	FIELD BORING NO.
STATION: 1+457.114	OFFSET: 15.27 LT REFERENCE LINE: RT21ML	GROUND ELEVATION: 3.10

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-19	24.00	24.45	14	23	22	400	Reddish Brown cf SAND, some cf Gravel, little (-) Silt	
	BIT									
	MUD									
25.5										
		S-20	25.50	25.95	13	21	25	350	SAME	
27										
		S-21	27.00	27.45	19	18	21	400	SAME	
28.5										
		S-22	28.50	28.95	18	34	28	420	Reddish Brown cf GRAVEL (rounded and sharp edged), some cf Sand, trace Silt w/ boulders, cobbles	
30										
		S-23	30.00	30.45	17	30	33	350	SAME w/ boulders	
31.5										
		S-24	31.50	31.95	18	19	47	200	Reddish Brown cf GRAVEL, little cf Sand, trace Silt & Clay w/ Decomposed SHALE	
33										
		S-25	33.00	33.45	28	36	34	275	SAME w/ boulders	
34.5										
		S-26	34.50	34.95	10	10	31	175	SAME	
36										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

ROUTE: RT. 21	LOCAL NAME: RT. 21 Viaduct	BORING NO. S-1005
SECTION: 2N	COORDINATES: N209198.4307 E650350.6688	FIELD BORING NO.
STATION: 1+457.114	OFFSET: 15.27 LT REFERENCE LINE: RT21ML	GROUND ELEVATION: 3.10

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-27	36.00	36.45	16	72	89	300	Reddish Brown cf GRAVEL (round edged), little cf Sand,	
	BIT								trace (-) Silt w/ cobbles and boulders	
	MUD									
37.5										
		S-28	37.50	37.55	100/50	-	-	50	Red and Gray cf GRAVEL (boulders and cobbles), little	
									cf Sand, trace Silt	
39										
		S-29	39.00	39.45	14	58	58	300	Reddish Brown cf SAND, and cf Gravel, trace Clayey Silt	
									w/ Decomposed SHALE	
40.5										
		S-30	40.50	40.77	60	100/120	-	120	Reddish Brown cf GRAVEL (highly Decomposed SHALE),	
									some cf Sand, little Clayey Silt w/ boulders	
42										
		S-31	42.00	42.28	35	100/130	-	280	Reddish Brown cf SAND, and cf Gravel (sharp edged),	
									little Silt& Clay w/ Decomposed SHALE, cobbles,	
43.5									boulders	
		S-32	43.50	43.95	20	52	77	350	Reddish Brown cf GRAVEL (round and sharp edged),	
									some cf Sand, trace Clayey Silt w/ Decomposed SHALE	
45										
		S-33	45.00	45.23	72	100/80	-	150	Red, Gray and White cf GRAVEL (round edged), trace cf	
									Sand w/ boulders and cobbles	45.60
									Red and Gray BOULDERS and COBBLES, and cf Gravel	
46.5		S-34	46.50	46.50	100/0	-	-	NR	Refusal	
									Bottom of Hole 46.50 m	
48										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ARORA and ASSOCIATES, P.C.

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1006

SECTION: 2N COORDINATES: N209181.0174 E650386.2934 FIELD BORING NO.

STATION: 1+457.131 OFFSET: 24.39 RT REFERENCE LINE: RT21ML GROUND ELEVATION: 3.28

BORING BY: Warren George, Inc. DATE STARTED: 5-21-99

INSPECTOR: Y. S. Choksi DATE COMPLETED: 5-26-99

GROUND WATER ELEVATION
0 Hr. 1.68 Date: 5-26-99
24 Hr. 0.99 Date: 5-27-99
P.P. Installed

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER								300 mm Concrete Pavement	0.30
	BIT	S-1	0.30	0.45	6	11	24	325	Gray cf SAND, little Silt, trace f Gravel (FILL)	
	MUD									
1.5										
		S-2	1.50	1.95	3	5	6	250	Yellowish Brown f SAND, some Silt, trace mf Gravel (FILL)	
										1.95
		S-3A	1.95	2.25	8	10	12	300	Gray cf SAND, and organic Clayey Silt, trace (+) cf Gravel,	
		S-3B	2.25	2.40					(Cinders) over Dk Gray organic Silty Clay (Peat)	2.25
3		UD-1	2.40	3.00	-	-	-	450	Brownish Gray organic Silty CLAY (PEAT)	
									PP = 0.50 kg/cm²	
		S-4	3.00	3.45	6	6	6	350	Dk Gray organic Silty CLAY, and cf Gravel, trace f Sand	
										3.90
4.5										
		S-5	4.50	4.95	10	15	14	300	Brown cf SAND, little Silt, trace mf Gravel	
6										
		S-6	6.00	6.45	10	11	11	450	Reddish Brown mf SAND, little Silt, trace f Gravel	
7.5										
		S-7	7.50	7.95	8	8	10	300	Reddish Brown mf SAND, trace (+) Silt, trace f Gravel	
9										
		S-8	9.00	9.45	8	10	13	400	SAME	
10.5										
		S-9	10.50	10.95	12	12	13	300	SAME	
12										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	NX

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1006

SECTION: 2N COORDINATES: N209181.0174 E650386.2934 FIELD BORING NO.

STATION: 1+457.131 OFFSET: 24.39 RT REFERENCE LINE: RT21ML GROUND ELEVATION: 3.28

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-10	12.00	12.45	12	13	16	350	Reddish Brown mf SAND, trace (+) Silt, trace f Gravel	
	BIT									
	MUD									
13.5										
		S-11	13.50	13.95	16	19	22	300	Reddish Brown mf SAND, little Silt, trace f Gravel	
15										15.00
		S-12	15.00	15.45	28	27	23	300	Reddish Brown Clayey SILT, and f Sand	
16.5										
		S-13	16.50	16.95	30	34	37	200	Reddish Brown Clayey SILT, some f Sand	
18										
		S-14	18.00	18.45	62	80	66	275	Reddish Brown SILT & CLAY PP = 0.50 kg/cm²	
19.5										
		S-15	19.50	19.95	13	18	18	350	Reddish Brown Silty CLAY interlayered w/ lenses of f SAND PP = 0.65 kg/cm²	
		UD-2	20.40	21.00	-	-	-	600	SAME	
21										
		S-16	21.00	21.45	10	10	15	400	Reddish Brown varved Silty CLAY PP = 0.80 kg/cm²	
22.5										22.20
		S-17	22.50	22.95	34	37	43	325	Reddish Brown f SAND, some Silt	
24										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	NX

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

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NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1006

SECTION: 2N COORDINATES: N209181.0174 E650386.2934 FIELD BORING NO.

STATION: 1+457.131 OFFSET: 24.39 RT REFERENCE LINE: RT21ML GROUND ELEVATION: 3.28

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-18	24.00	24.45	33	36	40	400	Reddish Brown f SAND, some Silt	
	BIT									
	MUD									
25.5										
		S-19	25.50	25.95	12	15	18	350	SAME	
27										
		S-20	27.00	27.45	17	18	21	300	SAME	
28.5										
		S-21	28.50	28.95	21	25	27	200	Reddish Brown cf SAND, trace mf Gravel, trace Silt	
30										
		S-22	30.00	30.45	19	25	31	250	Reddish Brown cf SAND, trace (+) cf Gravel, trace Silt	
31.5										
		S-23	31.50	31.95	23	23	25	300	Reddish Brown mf SAND, little Silt, trace f Gravel	
33										
		S-24	33.00	33.45	19	24	26	300	Reddish Brown cf SAND, little Clayey Silt	
34.5										34.50
		S-25	34.50	34.95	18	23	27	150	Reddish Brown cf GRAVEL (round edged rock fragments), some cf Sand, trace Silt	
36										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	NX

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Inferred Change in Strata — — — — — —

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NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1006

SECTION: 2N COORDINATES: N209181.0174 E650386.2934 FIELD BORING NO.

STATION: 1+457.131 OFFSET: 24.39 RT REFERENCE LINE: RT21ML GROUND ELEVATION: 3.28

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-26	36.00	36.45	16	17	24	100	Reddish Brown cf GRAVEL (round edged rock fragments), little (-) cf Sand, trace Silt	
	BIT									
	MUD									
37.5										
		S-27	37.50	37.95	17	21	26	150	SAME	
39										39.00
		S-28	39.00	39.45	15	19	25	75	Reddish Brown cf SAND, some cf Gravel (round edged rock fragments), little Silt	
40.5										
		S-29	40.50	40.95	14	34	29	150	Reddish Brown mf SAND, little (+) Silt, trace mf Gravel (round edged rock fragments)	
42										42.00
		S-30	42.00	42.45	9	10	11	200	Reddish Brown cf GRAVEL (round edged rock fragments), trace cf Sand, trace Silt	
43.5										
		S-31	43.50	43.95	41	45	47	225	Reddish Brown cf GRAVEL (round edged rock fragments), little cf Sand, trace Silt	
45										
		S-32	45.00	45.45	24	30	31	200	Reddish Brown cf GRAVEL (round edged rock fragments), trace (+) cf Sand, trace Silt	
46.5										
		S-33	46.50	46.95	17	45	100	150	SAME	
48										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	NX

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

ROUTE: RT. 21 LOCAL NAME: RT. 21 Viaduct BORING NO. S-1006

SECTION: 2N COORDINATES: N209181.0174 E650386.2934 FIELD BORING NO.

STATION: 1+457.131 OFFSET: 24.39 RT REFERENCE LINE: RT21ML GROUND ELEVATION: 3.28

DEPTH (m)	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/150	150/300	300/450	REC. (mm)	SOIL DESCRIPTION AND STRATIFICATION	(m)
	ROLLER	S-34	48.00	48.45	38	44	57	300	Reddish Brown cf GRAVEL (round edged rock fragments), little (-) cf Sand, trace Silt	
	BIT									
	MUD									
49.5										49.50
		S-35	49.50	49.90	53	58	100/100	300	Reddish Brown mf SAND, some Silt, trace f Gravel (round edged rock fragments)	
51										
		S-36	51.00	51.25	70	100/100	-	150	Reddish Brown cf SAND, little (+) Silt, trace (+) f Gravel (round edged rock fragments)	
52.5										
		S-37	52.50	52.62	100/120	-	-	100	Reddish Brown mf SAND, little cf Gravel (rock fragments), trace (+) Silt	
54										
		S-38	54.00	54.02	100/20	-	-	NR	Top of Rock	54.30
	CORING									
		C-1	54.30	55.80		REC	1200	80%	Reddish Brown slightly weathered, very closely to closely spaced fractured, medium hard SHALE	
55.5						RQD	900	60%		
		C-2	55.80	57.30		REC	1200	80%	Reddish Brown slightly to moderately weathered, very closely to medium spaced fractured, soft SHALE	
57						RQD	240	16%		
58.5									Bottom of Hole 57.30 m	
60										

Nominal I.D. of Drive Pipe / Hollow Stem Auger	100 mm
Nominal I.D. of Split Barrel Sampler	35 mm
Weight of Hammer on Drive Pipe	140 kg
Weight of Hammer on Split Barrel	63.5 kg
Drop of Hammer on Drive Pipe	600 mm
Drop of Hammer on Split Barrel	760 mm
Core Size	NX

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

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Approximate Change in Strata _____

Inferred Change in Strata — — — — — —

NEW JERSEY DEPARTMENT OF TRANSPORTATION
FORM SO-2M

Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-212
SECTION: 2N
STATION: 28+61 OFFSET: 65' RT REF. LINE: RT. 21 B.L. G. L. EL: 10.15
BORINGS MADE BY: Site Engineers DATE STARTED: 10/4/94 El. G.W.T.
INSPECTOR: E. Cifaldi DATE COMPLETED: 10/4/94 0 HR. 5.65 DATE: 10/4/94
24 HR. 4.55 DATE: 10/5/94
_____ ft. P.P. Inst. DATE:

Casing Blows		Sample No.	Depth		Blows on Spoon 0 / / 6	6 / /12	12/ /18	Rec.	Sample ID and Profiles Changes	
		S-1	0.0'	1.5'	34	24	31	0.5'	Dark Brown mf SAND, some mf Gravel, little Silt	
										3.0
5										
	A	S-2	5.0'	6.5'	2	2	2	1.0'	Dark Brown Clayey SILT, little mf Gravel, little Wood frag.	
	U									
	G	S-3	6.5'	8.0'	3	2	2	0.3'	Dark Brown SILT, little mf Sand, trace Wood frag.	8.0
	E									
10	R	S-4	8.0'	9.5'	3	3	4	0.8'	Reddish Brown mf SAND, little Silt	
	S	S-5	9.5'	11.0'	3	5	10	0.9'	Reddish Brown cf SAND, little Silt, little f Gravel	
15										
		S-6	15.0'	16.5'	4	5	5	1.0'	Reddish Brown mf SAND, little Silt	
20										
		S-7	20.0'	21.5'	5	6	6	1.1'	Reddish Brown f SAND, some Silt	
25										
		S-8	25.0'	26.5'	10	11	11	0.5'	Same	
30										
		S-9	30.0'	31.5'	4	5	5	1.1'	Reddish Brown cf SAND, some f Gravel, trace Silt	
35										
		S-10	35.0'	36.5'	5	6	6	0.5'	Reddish Brown cf SAND, little f Gravel, little Silt	
										37.5
40										

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" 4"
Nominal I.D. of Split Barrel Sampler 1 1/2"
Weight of hammer on Drive Pipe 300 lbs.
Weight of hammer on Split Barrel Sampler 140 lbs.
Drop of hammer on Drive Pipe 24"
Drop of hammer on Split Barrel Sampler 30"

The subsurface information shown hereon was obtained for A & A design and estimated purposes. It is made available to authorized users only that they may have access to the same information available to A & A. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Core Dia. _____

Approximate Change in Strata _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata _ _ _ _ _ _ _

Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-212
SECTION: 2N
STATION: 28+61 OFFSET: 65' RT REF. LINE: RT. 21 B.L. G. L. EL: 10.15
BORINGS MADE BY: Site Engineers DATE STARTED: 10/4/94
INSPECTOR: J. Walker DATE COMPLETED: 10/4/94

El. G.W.T.
0 HR. 5.65 DATE: 10/4/94
24 HR. 4.55 DATE: 10/5/94
_____ ft. P.P. Inst.
DATE:

Casing Blows		Sample No.	Depth		Blows on Spoon 0 / / 6	6 / /12	12/ /18	Rec.	Sample ID and Profiles Changes	
		S-11	40.0'	41.5'	2	3	3	1.1'	Reddish Brown SILT, some mf Gravel	
45	A									
	U	S-12	45.0'	46.5'	4	5	5	1.3'	Reddish Brown Clayey SILT	
	G									
	E									
	R									
50	S									
		S-13	50.0'	51.5'	11	9	10	1.2'	Same	
55										
		S-14	55.0'	56.5'	5	7	10	1.5'	Reddish Brown SILT, little f Sand	
60										
		S-15	60.0'	61.0'	25	41	50/ 0.0'	1.0'	Reddish Brown SILT, and f Sand	
65										65.0
		S-16	65.0'	66.5'	13	17	18	0.8'	Reddish Brown f SAND, some Silt	
70										70.0
		S-17	70.0'	71.5'	15	14	14	1.0'	Reddish Brown SILT, little f Sand	
75										75.0
		S-18	75.0'	75.0'	50/ 0.0'	-	-	0.0'	No Recovery Bottom of Hole	
80										

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" 4"
Nominal I.D. of Split Barrel Sampler 1 1/2"
Weight of hammer on Drive Pipe 300 lbs.
Weight of hammer on Split Barrel Sampler 140 lbs.
Drop of hammer on Drive Pipe 24"
Drop of hammer on Split Barrel Sampler 30"

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Core Dia. _____

Approximate Change in Strata _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata - - - - - - -

Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-210
SECTION: 2N
STATION: 28+61 OFFSET: 63' LT REF. LINE: RT. 21 B.L. G. L. EL: 9.88
BORINGS MADE BY: Site Engineers DATE STARTED: 10/5/94
INSPECTOR: J. Walker DATE COMPLETED: 10/5/94

El. G.W.T.
0 HR. 5.18 DATE: 10/5/94
24 HR. 5.58 DATE: 10/6/94
_____ft. P.P. Inst.
DATE:

Casing Blows		Sample No.	Depth		Blows on Spoon 0 / / 6	6 / /12	12/ /18	Rec.	Sample ID and Profiles Changes	
									8" CONCRETE PAVEMENT over 4" DGA	1.0
		S-1	1.0'	2.5'	13	21	18	1.1'	Brown mf SAND, little cf Gravel, little Brick frag. (Fill)	
5										
	A	S-2	5.0'	6.5'	11	11	10	1.0'	Brown and Black cf SAND, some (-) cf Gravel, little (+) Silt (Fill)	
	U									
	G	S-3	6.5'	8.0'	12	12	12	0.8'	Black and Dark Brown cf SAND, some cf Gravel, little (+) Silt	
	E									
10	R	S-4	8.0'	9.5'	3	7	3	0.9'	Gray cf SAND, and cf Gravel, trace Silt	
	S									
		S-5	10.0'	11.5'	3	3	3	1.1'	Dark Gray cf SAND, some (+) Silt, little Glass frag. (Fill)	
15										
		S-6	15.0'	16.5'	5	6	5	0.0'	No Recovery	
										18.0
20										
		S-7	20.0'	21.5'	1	1	1	1.5'	Dark Gray SILT & CLAY (V = 2.0 TSF H = 2.0 TSF)	
										23.0
25										
		S-8	25.0'	26.5'	5	10	16	1.5'	Reddish Brown cf SAND, little Silt, trace f Gravel	
30										
		S-9	30.0'	31.5'	9	8	9	1.1'	Reddish Brown mf SAND, little Silt	
35										
		S-10	35.0'	36.5'	9	11	11	1.2'	Same	
40										

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" 4"
Nominal I.D. of Split Barrel Sampler 1 1/2"
Weight of hammer on Drive Pipe 300 lbs.
Weight of hammer on Split Barrel Sampler 140 lbs.
Drop of hammer on Drive Pipe 24"
Drop of hammer on Split Barrel Sampler 30"

The subsurface information shown hereon was obtained for A & A design and estimated purposes. It is made available to authorized users only that they may have access to the same information available to A & A. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Core Dia. _____

Approximate Change in Strata _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata - - - - - - -

Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-210
SECTION: 2N
STATION: 28+61 OFFSET: 63' LT REF. LINE: RT. 21 B.L. G. L. EL: 9.88
BORINGS MADE BY: Site Engineers DATE STARTED: 10/5/94 El. G.W.T.
INSPECTOR: J. Walker DATE COMPLETED: 10/5/94 0 HR. 5.18 DATE: 10/5/94
24 HR. 5.58 DATE: 10/6/94
_____ ft. P.P. Inst. DATE:

Casing Blows		Sample No.	Depth		Blows on Spoon 0 / / 6	6 / /12	12/ /18	Rec.	Sample ID and Profiles Changes	
		S-11	40.0'	41.5'	7	7	9	1.1'	Reddish Brown f SAND, trace Silt	
45	A									
	U	S-12	45.0'	46.5'	7	11	16	1.5'	Reddish Brown f SAND, some (+) Silt	
	G									
	E									
	R									
50	S									
		S-13	50.0'	51.5'	7	6	8	1.2'	Reddish Brown f SAND, little Silt	
55										55.0
		S-14	55.0'	56.5'	9	11	13	1.0'	Reddish Brown SILT, and f Sand	
60										60.0
		S-15	60.0'	61.5'	9	10	10	1.0'	Reddish Brown f SAND, and Silt	
65										
		S-16	65.0'	66.5'	6	9	11	1.5'	Same	
70										
		S-17	70.0'	71.5'	7	7	8	1.5'	Same	
75										
		S-18	75.0'	76.5'	8	17	28	1.2'	Reddish Brown cf SAND, and cf Gravel, trace Silt	
80										

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" 4"
Nominal I.D. of Split Barrel Sampler 1 1/2"
Weight of hammer on Drive Pipe 300 lbs.
Weight of hammer on Split Barrel Sampler 140 lbs.
Drop of hammer on Drive Pipe 24"
Drop of hammer on Split Barrel Sampler 30"

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Core Dia. _____

Approximate Change in Strata _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata _ _ _ _ _ _ _

Arora and Associates, P.C.

ROUTE: RT. 21 LOCAL NAME: Newark Viaduct Replacement TEST HOLE NO. S-210
SECTION: 2N
STATION: 28+61 OFFSET: 63' LT REF. LINE: RT. 21 B.L. G. L. EL:
BORINGS MADE BY: Site Engineers DATE STARTED: 10/5/94 El. G.W.T. 9.88
INSPECTOR: J. Walker DATE COMPLETED: 10/5/94 0 HR. 5.18 DATE: 10/5/94
24 HR. 5.58 DATE: 10/6/94
_____ ft. P.P. Inst.
DATE:

Casing Blows		Sample No.	Depth		Blows on Spoon 0 / / 6	6 / /12	12/ /18	Rec.	Sample ID and Profiles Changes	
	A	S-19	80.0'	81.5'	21	33	42	1.0'	Same	
	U									
	G									
	E									
85	R									
	S	S-20	85.0'	86.5'	39	42	49	0.5'	Same	
									Bottom of Hole	86.5
90										
95										

Nominal I.D. of Drive Pipe 2 1/2" 3 1/2" 4"
Nominal I.D. of Split Barrel Sampler 1 1/2"
Weight of hammer on Drive Pipe 300 lbs.
Weight of hammer on Split Barrel Sampler 140 lbs.
Drop of hammer on Drive Pipe 24"
Drop of hammer on Split Barrel Sampler 30"

The subsurface information shown hereon was obtained for A & A design and estimated purposes. It is made available to authorized users only that they may have access to the same information available to A & A. It is presented in good faith, but is not intended as a substitute for investigations, interpretation or judgement of such authorized users.

Core Dia. _____

Approximate Change in Strata _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Inferred Change in Strata - - - - - - -

Form SO-2 7/74 NEW JERSEY DEPARTMENT OF TRANSPORTATION

ROUTE: LOCAL NAME: **Roadway Boring** TEST HOLE NO. **355W - 65**

SECTION: **Frelinghuysen Avenue, City of Newark**

STATION: **120+98** OFFSET: **2' Lt.** REFERENCE LINE: **BL-Frelinghuysen Ave.** GROUND LINE ELEVATION:

BORINGS MADE BY: **Bowers** DATE STARTED: **6/28/78**

INSPECTOR: **Lounsberry** DATE COMPLETED: **6/29/78**

Elevation G.W.T.
0 Hr. **10' Down** Date: **6/29/78**
24 Hr. **SAME** Date: **7/3/78**
_____ ft. P.P. Installed Date:

CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/6	6/12	12/18	REC.	Sample ID and Profile Change	
1								3" Blacktop 6" Concrete	
30	S-1	1.0'	2.5'	46	29	17	15"	Red & Gry.CF SAND,some(-)Silt,little MF Gravel.	0.8'
20	S-2	2.5'	4.0'	16	28	30	12"	SAME	
25	S-3	4.0'	5.5'	10	8	6	9"	Dull Red CF Sand, some (+) Silt, little MF Gravel.	
12									
11									
12									
23									
22									
23									
22	S-4	10.0'	11.5'	25	27	25	13"	Dull Red CF GRAVEL and, CF Sand, little (+) Silt.	
28									
33									
21									
17									
8	S-5	15.0'	16.5'	26	23	37	7"	Dull Red CF SAND, some Silt, little (-) MF Gravel.	
22									
32									
29									
21									
6	S-6	20.0'	21.5'	18	10	8	10"	Dull Red CF SAND, little (-) Silt, some (+) CF Gravel.	
14									
18									
25									
25								Dull Red CF GRAVEL some, CF Sand, trace (+) Silt.	
	S-7	25.0'	26.0'	40	120	--	7"		26.0'

BOTTOM OF HOLE

Nominal I.D. of Drive Pipe	2½" XX
Nominal I.D. of Split Barrel Sampler	1½"
Weight of hammer on Drive Pipe	300 lbs.
Weight of hammer on Split Barrel Sampler	140 lbs.
Drop of hammer on Drive Pipe	24"
Drop of hammer on Split Barrel Sampler	30"

Core Dia. _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

The Contractor shall make his own subsurface investigations in order to satisfy himself of the actual subsurface conditions. The Information contained on this log is not warranted to show the actual subsurface conditions. The Contractor agrees that he will make no claims against the State if he finds that the actual conditions do not conform to those indicated by this log.

New Jersey Department of Transportation
Soils Bureau

Approximate Change in Strata _____

Inferred Change in Strata ----------

Project: PVSC
Project Location: Newark, NJ
Project Number: 60344189

Log of Boring B-7

Date(s) Drilled	**5/6/16 - 5/9/16**	Logged By	**Roberto Lucidi**	Approximate Surface Elevation (feet)	**9.0**
Drilling Method	**Mud rotary**	Drilling Contractor	**Craig Boring Test**	Coordinates	**North:** **East:**
Casing Size/Type	**4" dia. flush joint steel**	Drill Rig Operator	**Eric Delmeier**	Total Depth Drilled (feet) **65.0**	Rock Depth (feet) **56.0**
Drill Rig Type	**CME-850 XR**	Drill Bit Size/Type	**3 7/8" tricone rollerbit**	Sampler Type(s)	**2" O.D. split spoon**
Groundwater Level and Date Measured	**N\A**	Hammer Wt/Drop **140lb/30" automatic**	Casing Hammer Wt/Drop **140lb/30" automatic**	Core Barrel Size/Type	**NX**
Boring Location and Comments	**See boring location plan**			No. of Samples **Dist.:12 Undist.:0 Core (ft):5**	

Depth, feet	Soil Samples: Type, Number	Recov. (ft)	Pen. Resist. (blows/6 in)	Rock Coring: Run Number	Recov. (%)	RQD (%)	Graphic Log	MATERIAL DESCRIPTION	Liquid Limit	Plastic Limit	Water Cont.(%)	% Fines	REMARKS/ OTHER TESTS
0								Topsoil					
								[FILL] Brown c-f SAND, some silt & clay, some c-f gravel, concrete and brick fragments					Hand-cleared from 0' to 6'
5	S-1	0.3	8 5 4 3					[FILL] Brick and concrete fragments, some c-f sand					
	S-2	0.9	4 5 10 11					[FILL] Brown c-f SAND, some silt, trace roots, brick fragments					
10	S-3	1.3	7 4 6 26					[FILL] Brown c-f SAND, some clay brick fragments					
													Casing driven to 15'
15	S-4	1.3	1 2 4 4					[FILL] Brown-black c-f SAND, some clay & silt, trace f gravel, some wood, brick fragments					
													Casing pushed to 20'
20	S-5	1.4	3 4 9 11					[CL] Red-brown silty CLAY, trace c-f sand, trace f gravel					
													Casing hard to drive from 23' to 25'
25													Casing driven to 25'

Project: PVSC
Project Location: Newark, NJ
Project Number: 60344189

Log of Boring B-7

Depth, feet	Soil Samples: Type, Number	Recov. (ft)	Pen. Resist. (blows/6 in)	Rock Coring: Run Number	Recov. (%)	RQD (%)	Graphic Log	MATERIAL DESCRIPTION	Liquid Limit	Plastic Limit	Water Cont.(%)	% Fines	REMARKS/ OTHER TESTS
25	S-6	1.8	9 11 12 13					[CL] Red-brown silty CLAY, trace f sand, trace f gravel			17	79	
30	S-7	0.3	7 13 12 13					[CL] Red-brown silty CLAY, trace f sand, trace f gravel					Rig chattering from 33' to 35'
35	S-8	1.9	7 8 11 13					[CL] Red-brown CLAY, some silt, trace c-f sand, trace f gravel					
40	S-9	1.8	6 6 9 12					[CL] Red-brown sandy lean CLAY, some silt, trace f gravel			14	51	
45	S-10	1.7	13 15 16 27					[CL] Red-brown sandy CLAY, some m-f gravel, trace silt					Rig chattering at 49'
50	S-11	0.7	19 80/3"					[GP] Red-brown m-f GRAVEL, some c-f sand, trace clay & silt					

Project: PVSC
Project Location: Newark, NJ
Project Number: 60344189

Log of Boring B-7

Depth, feet	Soil Samples: Type, Number	Soil Samples: Recov. (ft)	Soil Samples: Pen. Resist. (blows/6 in)	Rock Coring: Run Number	Rock Coring: Recov. (%)	Rock Coring: RQD (%)	Graphic Log	MATERIAL DESCRIPTION	Liquid Limit	Plastic Limit	Water Cont.(%)	% Fines	REMARKS/ OTHER TESTS
55	S-12	0.3	100/3"					[GP] Red-gray m-f GRAVEL					
60				R-1	98	65		Red, fine grained SILTSTONE, laminated, slightly weathered, low hardness, mod. fractured, occ. clay in fractures					3:00 min 5:00 min 5:00 min 4:00 min 4:00 min
65								End of boring at 65 ft B.G.S.					
70													
75													
80													

Project: PVSC
Project Location: Newark, NJ
Project Number: 60344189

Log of Boring B-8

Date(s) Drilled	**5/5/16 - 5/6/16**	Logged By	**Roberto Lucidi**	Approximate Surface Elevation (feet)	**10.50**
Drilling Method	**Mud rotary**	Drilling Contractor	**Craig Boring Test**	Coordinates	**North:** **East:**
Casing Size/Type	**4" dia. flush joint steel**	Drill Rig Operator	**Eric Delmeier**	Total Depth Drilled (feet) **35.0**	Rock Depth (feet) **25.0**
Drill Rig Type	**CME-850 XR**	Drill Bit Size/Type	**3 7/8" tricone rollerbit**	Sampler Type(s)	**2" O.D. split spoon**
Groundwater Level and Date Measured	**N\A**	Hammer Wt/Drop **140lb/30" automatic**	Casing Hammer Wt/Drop **140lb/30" automatic**	Core Barrel Size/Type	**NX**
Boring Location and Comments	**See boring location plan**			No. of Samples **Dist.:6** **Undist.:0** **Core (ft):10**	

Depth, feet	Soil Samples: Type, Number	Recov. (ft)	Pen. Resist. (blows/6 in)	Rock Coring: Run Number	Recov. (%)	RQD (%)	Graphic Log	MATERIAL DESCRIPTION	Liquid Limit	Plastic Limit	Water Cont.(%)	% Fines	REMARKS/ OTHER TESTS
0								Topsoil					
								[FILL] Brown c-f SAND, some clay, some m-f gravel, cobbles, concrete fragments					Hand-cleared from 0' to 6'
5	S-1	1.2	3 1 5 9					[FILL] Brown-black clayey m-f SAND, trace f gravel, trace organics					
	S-2	1.4	10 7 3 3					[FILL] Brown-black clayey m-f SAND, trace f gravel, trace organics					Casing driven to 10'
10	S-3	1.4	1 2 3 6					[CL] Red-brown sandy lean CLAY, trace f gravel			17	62	Perm Test performed at 10'
15	S-4	1.5	7 8 9 14					[CL] Red-brown sandy CLAY, trace f gravel					
20	S-5	0.8	8 12 12 10					[GC] Red-brown m-f GRAVEL, some clay, trace c-f sand					
25													Casing driven to 25' Perm Test performed at 25'

Project: PVSC
Project Location: Newark, NJ
Project Number: 60344189

Log of Boring B-8

Depth, feet	Soil Samples: Type, Number	Soil Samples: Recov. (ft)	Soil Samples: Pen. Resist. (blows/6 in)	Rock Coring: Run Number	Rock Coring: Recov. (%)	Rock Coring: RQD (%)	Graphic Log	MATERIAL DESCRIPTION	Liquid Limit	Plastic Limit	Water Cont.(%)	% Fines	REMARKS/ OTHER TESTS
25													
				R-1	88	23		Red, fine grained SILTSTONE, thinly laminated, mod. weathered, low hardness, closely fractured, occ. clay in fractures					4:00 min 5:00 min 4:30 min 4:30 min 4:30 min
30													
				R-2	100	27		Red, fine grained SILTSTONE, thinly laminated, mod. weathered, low hardness, v. close to closely fractured, occ. clay in fractures					3:30 min 5:00 min 6:00 min 5:00 min 4:30 min
35								End of boring at 35 ft B.G.S.					
40													
45													
50													

Project: PVSC
Project Location: Newark, NJ
Project Number: 60344189

Log of Boring B-9

Sheet 1 of 2

Date(s) Drilled	**5/5/16**	Logged By	**Roberto Lucidi**	Approximate Surface Elevation (feet)	**10.0**
Drilling Method	**Mud rotary**	Drilling Contractor	**Craig Boring Test**	Coordinates	**North: East:**
Casing Size/Type	**4" dia. flush joint steel**	Drill Rig Operator	**Eric Delmeier**	Total Depth Drilled (feet) **45.0**	Rock Depth (feet) **33.5**
Drill Rig Type	**CME-850 XR**	Drill Bit Size/Type	**3 7/8" tricone rollerbit**	Sampler Type(s)	**2" O.D. split spoon**
Groundwater Level and Date Measured	**N\A**	Hammer Wt/Drop **140lb/30" automatic**	Casing Hammer Wt/Drop **140lb/30" automatic**	Core Barrel Size/Type	**NX**
Boring Location and Comments	**See boring location plan**			No. of Samples **Dist.:8 Undist.:0 Core (ft):10**	

Depth, feet	Soil Samples: Type, Number	Recov. (ft)	Pen. Resist. (blows/6 in)	Rock Coring: Run Number	Recov. (%)	RQD (%)	Graphic Log	MATERIAL DESCRIPTION	Liquid Limit	Plastic Limit	Water Cont.(%)	% Fines	REMARKS/ OTHER TESTS
0								Topsoil					
								[FILL] Brown c-f SAND, some m-f gravel, some clay & silt, glass and plastic fragments					Hand-cleared from 0' to 6'
5	S-1	1.8	10 16 20 28					[FILL] Brown-red m-f SAND, some clay & silt, trace gravel					
	S-2	1.5	30 23 19 50/5"					[FILL] Dark brown m-f SAND, some clay, some gravel, wood and brick fragments			20	28	
10	S-3	0.8	83 38 16 23					[FILL] Brown-red clayey m-f SAND, concrete fragments, wood					Casing hard to drive from 10' to 13'
													Casing driven to 15'
15	S-4	0.3	3 3 5 9					[FILL] Gray clayey m-f SAND, wood					
													Casing driven to 20'
20	S-5	1.0	7 6 11 13					[CL] Red-brown sandy CLAY, some f gravel					PP = 1.5 tsf to 2.0 tsf
25													Casing driven to 25'

Project: PVSC
Project Location: Newark, NJ
Project Number: 60344189

Log of Boring B-9

Sheet 2 of 2

Depth, feet	Soil Samples: Type, Number	Recov. (ft)	Pen. Resist. (blows/6 in)	Rock Coring: Run Number	Recov. (%)	RQD (%)	Graphic Log	MATERIAL DESCRIPTION	Liquid Limit	Plastic Limit	Water Cont.(%)	% Fines	REMARKS/ OTHER TESTS
25	S-6	1.3	6 8 8 11					[CL] Red-brown sandy lean CLAY, trace m-f gravel			14	60	PP = 1.5 tsf Casing driven to 27'
30	S-7	0.9	6 17 100/5"					[CL] Red-brown silty CLAY, some f gravel, trace c-m sand					
35				R-1	87	23		Red, fine grained SILTSTONE, thinly laminated, slightly to mod. weathered, low hardness, v. close to closely fractured, occ. clay in fractures					4:00 min 5:30 min 7:30 min 5:00 min 4:30 min
40				R-2	100	38		Red, fine grained SILTSTONE, thinly laminated, slightly to mod. weathered, low hardness, v. close to mod. fractured, occ. clay in fractures					4:00 min 5:00 min 4:00 min 4:30 min 4:30 min
45								End of boring at 45 ft B.G.S.					
50													

Project: PVSC
Project Location: Newark, NJ
Project Number: 60344189

Log of Boring B-10

Date(s) Drilled	**5/4/16 - 5/5/16**	Logged By	**Roberto Lucidi**	Approximate Surface Elevation (feet)	**10.0**
Drilling Method	**Mud rotary**	Drilling Contractor	**Craig Boring Test**	Coordinates	**North:** **East:**
Casing Size/Type	**4" dia. flush joint steel**	Drill Rig Operator	**Eric Delmeier**	Total Depth Drilled (feet) **50.0**	Rock Depth (feet) **41.0**
Drill Rig Type	**CME-850 XR**	Drill Bit Size/Type	**3 7/8" tricone rollerbit**	Sampler Type(s)	**2" O.D. split spoon**
Groundwater Level and Date Measured	**N\A**	Hammer Wt/Drop **140lb/30" automatic**	Casing Hammer Wt/Drop **140lb/30" automatic**	Core Barrel Size/Type	**NX**
Boring Location and Comments	**See boring location plan**			No. of Samples **Dist.:9** **Undist.:0** **Core (ft):5**	

Depth, feet	Soil Samples: Type, Number	Recov. (ft)	Pen. Resist. (blows/6 in)	Rock Coring: Run Number	Recov. (%)	RQD (%)	Graphic Log	MATERIAL DESCRIPTION	Liquid Limit	Plastic Limit	Water Cont.(%)	% Fines	REMARKS/ OTHER TESTS
0								Topsoil					
								[FILL] Brown c-f SAND, some clay & silt, some c-f gravel, asphalt and brick fragments					Hand-cleared from 0' to 6'
5	S-1	1.7	11 10 9 11					[FILL] Brown-black c-f SAND, some clay, trace f gravel, asphalt, brick, wood					
	S-2	2.0	7 6 6 6					[FILL] Brown-black c-f SAND, some clay, trace f gravel, asphalt, brick, wood					
10	S-3	1.4	6 5 4 3					Top 5": [FILL] Brown-black c-f SAND, some clay, trace f gravel, asphalt, brick, wood Bottom 12": [FILL] Brown c-f SAND, trace silt, trace f gravel					Casing hard to drive from 10' to 13'
													Casing driven to 15'
15	S-4	1.8	13 65 93 100/5"					Top 5": [FILL] Brown c-f SAND, trace silt, trace f gravel Bottom 17": [FILL] Brown-black gravelly c-f SAND, trace silt, wood, asphalt, brick fragments					Rig chattering from 15' to 18'
													Casing driven to 20'
20	S-5	1.3	11 20 24 24					[ML] Red-brown SILT			22	95	
25													Casing driven to 25'

Project: PVSC
Project Location: Newark, NJ
Project Number: 60344189

Log of Boring B-10

Sheet 2 of 2

Depth, feet	Soil Samples: Type, Number	Recov. (ft)	Pen. Resist. (blows/6 in)	Rock Coring: Run Number	Recov. (%)	RQD (%)	Graphic Log	MATERIAL DESCRIPTION	Liquid Limit	Plastic Limit	Water Cont.(%)	% Fines	REMARKS/ OTHER TESTS
25	S-6	1.8	3 13 17 27					[SC] Red-brown c-f SAND, some clay, some m-f gravel					
30	S-7	0.8	6 10 12 17					[CL] Red-brown sandy CLAY, trace f gravel					Casing driven to 30'
35	S-8	1.1	16 24 13 4					[SC] Red-brown c-f SAND, some clay, some m-f gravel					Casing driven to 35'
40	S-9	0.1	50/1"					[GP] Red m GRAVEL					Casing pushed to 40'
45				R-1	97	55		Red, fine grained SILTSTONE, thinly laminated, slightly weathered, low hardness, closely to mod. fractured, occ. clay in fractures					4:00 min 5:40 min 5:00 min 4:30 min 5:00 min
50								End of boring at 50 ft B.G.S.					

EDWARDS AND KELCEY, INC.

SOIL ENGINEERING DIVISION

TEST BORING DATA — TEST HOLE NO. 94

Rt.: 21 Section: Local Name: Rt. 21 Freeway, Penn Plaza

Station: 18+50 Offset: 49'R Reference Line: Rt. 21 Ground Line Elevation:

Borings made by: Giles Date Started: 8/5/69 El. G.W.T. +4.2 @8:20 AM Date: 8/6/69

Inspector: Galdi Date Completed: 8/6/69 El. G.W.T. Date:

Depth in Feet	Casing Blows	Sample NO.	Depth		Blowson Spoon 0/6	6/12	12/18	Rec.	Sample Identification and Profile Change	
	-	S-1	0	1.0	19	47	-	12"	Fill: Blackish-Brown coarse to fine	
	174								SAND, t. Gravel, Cinders, pieces Brick	
	98									
	67									
	50									5.0
5	36	S-2	5.0	6.5	23	25	40	13"	Fill: Reddish-Brown SILT, l. medium	
	98								to fine Sand, pieces Sandstone	
	106									
	86									
	75									10.0
10	38	S-3	10.0	11.5	30	32	23	13"	Reddish-Brown coarse to fine SAND,	
	40								s. Silt, t. medium Gravel, pieces Sand-	
	42								stone	
	43									
	41									15.0
15	42	S-4	15.0	16.5	11	12	15	14"	Reddish-Brown fine SAND, l. (-)Silt.	
	40									
	36									
	33									
	30									19.5
20	30	S-5	20.0	21.5	8	11	15	12"	Reddish-Brown medium to fine SAND,	
	39								t. Silt	
	40									23.0
	46									
	42									
25	34	S-6	25.0	26.5	21	21	23	10"	Reddish-Brown fine SAND, l. Silt	
	48									
	48									
	49									
	60									
30	31	S-7	30.0	31.5	19	20	22	12"	Same as above	
	57									
	70									
	51									
	53									35.0
35	62	S-8	35.0	36.5	8	9	10	3"	Reddish-Brown SILT, l. fine Sand	
	69	S-9	36.5	38.0	11	12	13	12"		
	57									38.0
	66									
40	70									

I.D. Casing	2-1/2"	Wgt. Hammer on Casing	300#
I.D. Spoon	1-3/8"	Wgt. Hammer on Spoon	140#
Type Core Drill	Ax	Drop Hammer on Casing	24"
Core Dia.	1-1/8"	" " " Spoon	30"

Symbol	a.	s.	l.	t.
Proportions	and	some	little	trace
% By Wgt.	35 to 50	20 to 35	10 to 20	1 to 10

EDWARDS AND KELCEY, INC.

SOIL ENGINEERING DIVISION

TEST BORING DATA

TEST HOLE NO. 94

Rt.: 21 Section: Local Name: Rt. 21 Freeway, Penn Plaza

Station: 18+50 Offset: 49'R Reference Line: Rt. 21 Ground Line Elevation:

Borings made by: Giles Date Started: 8/5/69 El. G.W.T. +4.2 @8:20 AM Date: 8/6/69

Inspector: Galdi Date Completed: 8/6/69 El. G.W.T. Date:

Casing Blows	Sample NO.	Depth		Blowson Spoon 0/6	6/12	12/18	Rec.	Sample Identification and Profile Change	
71	S-10	41.5	43.0	11	12	14	12"	Reddish-Brown SILT, t. fine Sand	
35/36									43.0
								BOTTOM OF HOLE	

45

Depth in Feet

I.D.Casing 2-1/2"	Wgt.Hammer on Casing	300#	
I.D.Spoon 1-3/8"	Wgt.Hammer on Spoon	140#	
Type Core Drill Ax	Drop Hammer on Casing	24"	
Core Dia. 1-1/8"	" " " Spoon	30"	

Symbol	a.	s.	l.	t.
Proportions	and	some	little	trace
% By Wgt.	35 to 50	20 to 35	10 to 20	1 to 10

EDWARDS AND KELCEY, INC.

SOIL ENGINEERING DIVISION

TEST BORING DATA — TEST HOLE NO. 95

Rt.: 21 Section: Local Name: Rt. 21 Freeway, Penn Plaza

Station: 17+65 Offset: 47'R Reference Line: Rt. 21 Ground Line Elevation:

Borings made by: Giles Date Started: 8/5/69 El. G.W.T. -3.1 @2:15 PM Date: 8/5/69

Inspector: Galdi Date Completed: 8/5/69 El. G.W.T. Date:

Depth in Feet	Casing Blows	Sample NO.	Depth		Blowson Spoon 0/6	6/12	12/18	Rec.	Sample Identification and Profile Change	
	17									
	20	S-1	1.0	2.5	17	20	20	13"	Fill: Blackish-Brown coarse to fine	
	11								SAND, l. medium to fine Gravel, t. Silt,	
	14								pieces Brick	
	20									
5	31	S-2	5.0	6.5	16	17	40	13"	Same as above	
	70									
	190									
	200									
	190									10.0
10	91	S-3	10.0	11.5	17	20	14	12"	Reddish-Brown medium to fine SAND,	
	101								l. (-)Silt, l(-)Gravel, pieces Sandstone	
	114									
	117									
	105									
15	90	S-4A	15.0	15.5	17	20	24	14"	Reddish-Brown coarse to fine SAND,	15.5
	54	S-4B	15.5	16.5					t. fine Gravel t. (-)Silt	
	62									
	67									
	68									20.0
20	54	S-5	20.0	21.5	14	14	17	12"	Reddish-Brown coarse to fine SAND,	
	61								l. Silt, t. Gravel, pieces Sandstone	
	67									
	64									
	69									25.0
25	41	S-6	25.0	26.5	4	7	12	0"	Reddish-Brown medium to fine SAND,	
	40	S-7	27.0	28.5	10	11	13	12"	t. (+) Silt. Same as above	
	44									
	49									
	52									
30	91	S-8	30.0	31.5	17	19	40	12"	Same as above	
	94									
	150									
	126									
	140									35.0
35	91	S-9	35.0	36.5	17	22	22	14"	Brown medium to fine SAND, t. Silt,	
	87								t. (-)Gravel	
	88									
	94									38.5
40	100									

I.D.Casing 2-1/2"	Wgt.Hammer on Casing	300#	
I.D.Spoon 1-3/8"	Wgt.Hammer on Spoon	140#	
Type Core Drill Ax	Drop Hammer on Casing	24"	
Core Dia. 1-1/8"	" " " Spoon	30"	

Symbol	a.	s.	l.	t.
Proportions	and	some	little	trace
% By Wgt.	35 to 50	20 to 35	10 to 20	1 to 10

EDWARDS AND KELCEY, INC.

SOIL ENGINEERING DIVISION

TEST BORING DATA — TEST HOLE NO. 95

Rt.: 21 Section: Local Name: Rt. 21 Freeway, Penn Plaza

Station: 17+65 Offset: 47'R Reference Line: Rt. 21 Ground Line Elevation:

Borings made by: Giles Date Started: 8/5/69 El. G.W.T. -3.1 @2:15 PM Date: 8/5/69

Inspector: Galdi Date Completed: 8/5/69 El. G.W.T. Date:

Depth in Feet

Casing Blows	Sample NO.	Depth		Blowson Spoon 0/6	6/12	12/18	Rec.	Sample Identification and Profile Change	
	S-10	40.0	41.5	3	8	14	15"	Red fine SAND, s.(-) Silt	
	S-11	41.5	43.0	12	21	32	18"	Same as above	
									43.0
								BOTTOM OF HOLE	

45

I.D.Casing 2-1/2"	Wgt.Hammer on Casing	300#	
I.D.Spoon 1-3/8"	Wgt.Hammer on Spoon	140#	
Type Core Drill Ax	Drop Hammer on Casing	24"	
Core Dia. 1-1/8"	" " " Spoon	30"	

Symbol	a.	s.	l.	t.
Proportions	and	some	little	trace
% By Wgt.	35 to 50	20 to 35	10 to 20	1 to 10

EDWARDS AND KELCEY, INC.

SOIL ENGINEERING DIVISION

Sheet 1 of 1

TEST BORING DATA

TEST HOLE NO. 101

Rt.: 21 Section: Local Name: Rt. 21 Freeway, Penn Plaza

Station: 16+87 Offset: 77'R Reference Line: Rt. 21 Ground Line Elevation: 15.92

Borings made by: Giles Date Started: 8/4/69 El. G.W.T. +4.9 Date: 8/4/69

Inspector: French Date Completed: 8/4/69 El. G.W.T. Date:

Depth in Feet	Casing Blows	Sample NO.	Depth		Blowson Spoon 0/6	6/12	12/18	Rec.	Sample Identification and Profile Change	
	8									
	11	S-1	1.0	2.5	3	3	3	12"	Fill: Black coarse to fine SAND, l. (-) Silt, (Cinders)	
	17									
	12									
	14									5.0
5	11	S-2	5.0	6.5	5	4	4	17"	Tan Silt, l. (-) fine Sand, t. Clay	
	11									
	3									
	10									
	13									
10	14	S-3	10.0	11.5	3	9	11	14"	Reddish-Brown coarse to fine SAND, t. Silt	10.5
	13									
	21									
	20									
	29									
15	20	S-4	15.0	16.5	13	14	14	13"	Reddish-Brown SILT, l. fine Sand, pieces (Siltstone)	16.0
	27									
	29									
	35									19.0
	38									
20	31	S-5	20.0	21.5	12	11	13	14"	Reddish-Brown coarse to fine SAND, t. fine Gravel, t. Silt	
	32									
	35									
	29									
	33									
25	32	S-6	25.0	26.5	13	13	12	14"		
	30									
	36									
	39									
	37									
30		S-7	30.0	31.5	20	11	14	14"	Reddish-Brown fine SAND, t. Silt	30.5
										31.5
									BOTTOM OF HOLE	
35										
40										

I.D. Casing	2-1/2"	Wgt. Hammer on Casing	300#
I.D. Spoon	1-3/8"	Wgt. Hammer on Spoon	140#
Type Core Drill	Ax	Drop Hammer on Casing	24"
Core Dia.	1-1/8"	" " " Spoon	30"

Symbol	a.	s.	l.	t.
Proportions	and	some	little	trace
% By Wgt.	35 to 50	20 to 35	10 to 20	1 to 10

EDWARDS AND KELCEY, INC.

SOIL ENGINEERING DIVISION

Sheet 1 of 1

TEST BORING DATA

TEST HOLE NO. 102

Rt.: 21 Section: Local Name: Rt. 21 Freeway, Penn Plaza

Station: 16+20 Offset: 72'R Reference Line: Rt. 21 Ground Line Elevation: 17.09

Borings made by: Giles Date Started: 8/4/69 El. G.W.T. +3.4 Date: 8/5/69

Inspector: Galdi Date Completed: 8/5/69 El. G.W.T. Date:

Depth in Feet	Casing Blows	Sample NO.	Depth		Blowson Spoon 0/6	6/12	12/18	Rec.	Sample Identification and Profile Change	
	4	S-1	0	1.5	4	4	5	12"	Fill: Black coarse to fine SAND, l. (-) Silt, Cinders	
	5									
	6									
	5									
	6									
5	8	S-2	5.0	6.5	5	5	6	18"		6.0
	8								Reddish-Brown coarse to fine SAND, t. Silt, t. fine Gravel	
	11									
	12									
	13									10.0
10	7	S-3	10.0	11.5	6	7	4	18"	Reddish-Brown medium to fine SAND,	11.0
	6	S-4	11.5	13.0	2	4	7	17"	s. coarse Gravel, l. (-) Silt	12.0
	24								Black organic Silty Clay	
	27								Gray fine SAND a. SILT	14.0
	47									
15	21	S-5	15.0	16.5	8	11	14	18"	Reddish-Brown medium to fine SAND,	
	23								t. (-) Silt	
	33									18.0
	36									
	34									
20	29	S-6	20.0	21.5	14	13	15	9"	Red-Tan coarse to fine SAND, l. coarse to fine Gravel, t. Silt	
	31									
	38									23.0
	40									
	41									
25	27	S-7	25.0	26.5	9	10	12	13"	Reddish-Brown coarse to fine SAND, l. coarse to fine Gravel, l. Silt (pieces Sandstone)	
	34									
	33									
	35									
	36									
30		S-8	30.0	31.5	16	11	14	12"		
										31.5
									BOTTOM OF HOLE	
35										
40										

I.D. Casing	2-1/2"	Wgt. Hammer on Casing	300#
I.D. Spoon	1-3/8"	Wgt. Hammer on Spoon	140#
Type Core Drill	Ax	Drop Hammer on Casing	24"
Core Dia.	1-1/8"	" " " Spoon	30"

Symbol	a.	s.	l.	t.
Proportions	and	some	little	trace
% By Wgt.	35 to 50	20 to 35	10 to 20	1 to 10

GOODKIND & O'DEA, INC. FIELD NO. B-2

ROUTE: Raymond Blvd. LOCAL NAME: Jackson St. Bridge Rehabilitation TEST HOLE NO.

Station: Offset: Ref. Line: SHEET 1 of 2

STATION: 58+84 OFFSET: 83Lt REFERENCE LINE: Survey GROUND LINE ELEVATION: 10.8'

BORINGS MADE BY: J. McErlean DATE STARTED: 10-9-85

INSPECTOR: Art Leong DATE COMPLETED: 10-10-85

Elevation G.W.T. 0 Hr. Date 24 Hr. Date 10-11-85

0.6' ft. O.W.(Elevation) Date

Depth	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/6	6/12	12/18	REC.	Sample ID and Profile Change
	8	S-1	0'-	2'	7	16		1.0'	Brown f SAND, little(+) Silt, trace(+) m-f Gravel w/roots (fill)
	12				23	20			
	13	S-2	2'-	4'	16	15		1.4'	Brown f. SAND, little(+) Silt, trace f. Gravel, trace Brick & Shell fragments (fill).
	17				11	14			
5	33	S-3	4'-	4'10"	18	100/4"		0.8'	Red brown m-f SAND, little Silt, trace f. Gravel, trace Brick fragments (fill).
	53								
	26	S-4	6'-	8'	17	29		0.7'	Same (With boulder chips)
	16				22	14			
	19	S-5	8'-	10'	16	18		0.5'	Same
10	19				22	20			10.0
	16	S-6	10'	12'	16	8		1.2'	Red brown c-f SAND, little(+) f Gravel, trace(+) Silt
					14	22			
15		S-7	15'	17'	23	23		1.3'	Red brown c-f SAND, little(+) m-f Gravel, trace(+) Silt
					23	36			
20		S-8	20'	22'	25	19		1.5'	Same
					17	29			
25		S-9	25'	27'	47	31		1.1'	Same
					34	34			
30		S-10	30'	32'	50	38		1.0'	Red brown c-f SAND, little(-) f Gravel, trace Silt
					40	41			
35		S-11	35'	37'	26	28		2.0'	Red brown m-f SAND, trace Silt
					37	40			
40									

Revert Drilling Mud Used

Nominal I.D. of Drive Pipe	2½"	60'	4" 12'
Nominal I.D. of Split Barrel Sampler	1½"		
Weight of hammer on Drive Pipe	300 lbs.		
Weight of hammer on Split Barrel Sampler	140 lbs.		
Drop of hammer on Drive Pipe	24"		
Drop of hammer on Split Barrel Sampler	30"		

The Contractor shall make his own subsurface investigations in order to satisfy himself of the actual subsurface conditions. The information contained on this log is not warranted to show the actual subsurface conditions. The Contractor agrees that he will make no claims against the State if he finds that the actual conditions do not conform to those indicated by this log.

Core Dia. _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Approximate Change in Strata _____

Inferred Change in Strata _____

GOODKIND & O'DEA, INC. FIELD NO. B-2

ROUTE: Raymond Blvd. LOCAL NAME: Jackson St. Bridge Rehabilitation TEST HOLE NO.

Station: Offset: Ref. Line: SHEET 2 of 2

STATION: 58+84 OFFSET: 83Lt REFERENCE LINE: Survey GROUND LINE ELEVATION: 10.8'

BORINGS MADE BY: J. McEarlean DATE STARTED: 10-9-85

INSPECTOR: Art Leong DATE COMPLETED: 10-10-85

Elevation G.W.T.
0 Hr. Date
24 Hr. 0.6' ft. O.W. (Elevation) Date 10-11-85

	CASING BLOWS	SAMPLE NO.	DEPTH	Blows on Spoon 0/6	6/12	12/18	REC.	Sample ID and Profile Change	Depth
40'		S-12	40'- 42'	28	30		2.0'	Red brown f SAND, trace(+) Silt	
				31	38				43.0
45'		S-13A	45'- 46.5'	12	12	13	1.5'	Red brown SILT & CLAY	
		S-13B	46.5-47'	27			0.5'		46.5
								Red brown f SAND some(+) Silt	
50'		S-14	50'- 52'	29	39		2.0'	Red brown f SAND, little(+) Silt	
				29	42				53.0
55'		S-15	55'- 57'	30	30		2.0'	Red brown SILT, and f Sand	
				30	31				58.0
60'		S-16	60'- 62'	32	42		2.0'	Red brown f SAND, little Silt	
				49	44				
65'		S-17	65'- 67'	29	26		2.0'	Same	
				39	37				
70'		S-18	70'- 72'	32	35		2.0'	Same	
				52	57				72.0
75'								Bottom of Boring 72'	
80'									

Revert Drilling Mud Used

Nominal I.D. of Drive Pipe	2½"	4"
Nominal I.D. of Split Barrel Sampler	1½"	
Weight of hammer on Drive Pipe	300 lbs.	
Weight of hammer on Split Barrel Sampler	140 lbs.	
Drop of hammer on Drive Pipe	24"	
Drop of hammer on Split Barrel Sampler	30"	

The Contractor shall make his own subsurface investigations in order to satisfy himself of the actual subsurface conditions. The information contained on this log is not warranted to show the actual subsurface conditions. The Contractor agrees that he will make no claims against the State if he finds that the actual conditions do not conform to those indicated by this log.

Core Dia. _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Approximate Change in Strata _____

Inferred Change in Strata _____

GOODKIND & O'DEA, INC. FIELD NO. B-3

ROUTE: Raymond Blvd. LOCAL NAME: Jackson St. Bridge Rehabilitation TEST HOLE NO.

Station: Offset: Ref. Line: SHEET 1 of 2

STATION: 58+11.5 OFFSET: 95 Lt REFERENCE LINE: Survey GROUND LINE ELEVATION: 10.8'

BORINGS MADE BY: J. McEarlean DATE STARTED: 10/11/85

INSPECTOR: Art Leong DATE COMPLETED: 10/14/85

Elevation G.W.T.
0 Hr. +1.5 Date
24 Hr. Date 11/1/85
ft. O.W. Date

	CASING BLOWS	SAMPLE NO.	DEPTH	Blows on Spoon 0/6	6/12	12/18	REC.	Sample ID and Profile Change	Depth
	6	S-1	0'-2'	4	5		1.7'	Brown m-f SAND, little Silt, trace (+) f Gravel, trace brick & ashes (fill)	
	7			6	8				
	30								
	30								
5	28							Red brown c-f SAND, trace (+) m-f Gravel, trace (+) Silt, trace sandstone & ashes (fill)	
	12	S-2	5'-7'	29	20		1.0'		
	20			6	14				
	31	S-3	7'-9'	22	36		0.7'	Red brown c-f SAND, little m-f Gravel, trace (+) Silt (fill	
	37			32	25				
10	35	S-4	9'-11'	38	30		0.8'	Same	
	23			17	17			Red brown c-f SAND, some c-f Gravel, little Silt	12.0
		S-5	11'-13'	14	15		0.3'		
				5	5			Grey Clayey SILT, trace f Gravel, trace f Sand (w/fibers)	14.0
		S-6A	13'-14'	3	3		0.5'		
15		S-6B	14'-15'	15	27				
	Used	S-7	15'-17'	51	35		1.7'	Red brown c-f SAND, little m-f Gravel, little Silt	
				24	21			Same	
20	Mud								
		S-8	20'-22'	18	17		1.5'	Red brown m-f SAND, little (+) m-f Gravel, little Silt	
				20	18				
	Drilling								
25									
		S-9	25'-30'	19	16		1.5'	Red brown c-f SAND, some (-) m-f Gravel, little Silt	
	Gel"			16	21				
30									
		S-10	30'-32'	28	22		1.7'	Brown c-f SAND, little m-f Gravel, trace (+) Silt	
	Quick			17	18				
35									
		S-11	35'-37'	23	25		2.0'	Red brown m-f SAND, trace (+) Silt, trace (-) f Gravel	
				26	27				
40									

Nominal I.D. of Drive Pipe	2½"	55'	4" 17'
Nominal I.D. of Split Barrel Sampler	1½"		
Weight of hammer on Drive Pipe	300 lbs.		
Weight of hammer on Split Barrel Sampler	140 lbs.		
Drop of hammer on Drive Pipe	24"		
Drop of hammer on Split Barrel Sampler	30"		

The Contractor shall make his own subsurface investigations in order to satisfy himself of the actual subsurface conditions. The information contained on this log is not warranted to show the actual subsurface conditions. The Contractor agrees that he will make no claims against the State if he finds that the actual conditions do not conform to those indicated by this log.

Core Dia. _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Approximate Change in Strata _____

Inferred Change in Strata _____

GOODKIND & O'DEA, INC. FIELD NO. B-3

ROUTE: Raymond Blvd. LOCAL NAME: Jackson St. Bridge Rehabilitation TEST HOLE NO.

Station: Offset: Ref. Line: SHEET 2 of 2

STATION: 85+11.5 OFFSET: 95 Lt REFERENCE LINE: Survey GROUND LINE ELEVATION: 10.8'

BORINGS MADE BY: J. McEarlean DATE STARTED: 10/11/85

INSPECTOR: Art Leong DATE COMPLETED: 10/14/85

Elevation G.W.T.
0 Hr. Date
24 Hr. Date
ft. O.W. Date

	CASING BLOWS	SAMPLE NO.	DEPTH	Blows on Spoon 0/6	6/12	12/18	REC.	Sample ID and Profile Change	Depth
40		S-12	40' - 42'	35	49		2.0'	Red brown m-f SAND, trace Silt	
				50	47				
45		S-13	45' - 47'	30	31		1.8'	Red brown f SAND, little (+) Silt	
				39	43				
50		S-14	50' - 52'	29	40		2.0'	Same	
				48	43				54'
55	Used	S-15	55' - 57'	18	17		1.8'	Red brown SILT, little f Sand	
	Mud			25	38				
60		S-16	60' - 62'	30	14		2.0'	Red brown SILT, and f Sand	
	Drilling			31	53				62'
65		S-17	65' - 67'	30	41		1.5'	Red brown f SAND, little (+) Silt	
	"Quick Gel"			62	87				
70		S-18	70' - 72'	46	50		1.7'	Red brown f SAND, trace (+) Silt	
				46	96				72'
								Bottom of Boring 72'	
75									
80									

Nominal I.D. of Drive Pipe	2½"	4"
Nominal I.D. of Split Barrel Sampler	1½"	
Weight of hammer on Drive Pipe	300 lbs.	
Weight of hammer on Split Barrel Sampler	140 lbs.	
Drop of hammer on Drive Pipe	24"	
Drop of hammer on Split Barrel Sampler	30"	

The Contractor shall make his own subsurface investigations in order to satisfy himself of the actual subsurface conditions. The information contained on this log is not warranted to show the actual subsurface conditions. The Contractor agrees that he will make no claims against the State if he finds that the actual conditions do not conform to those indicated by this log.

Core Dia. _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Approximate Change in Strata _____

Inferred Change in Strata _____

GOODKIND & O'DEA, INC. FIELD NO. B-11

ROUTE: Raymond Blvd. LOCAL NAME: Jackson St. Bridge Rehabilitation TEST HOLE NO.

Station: Offset: Ref. Line: SHEET 1 of 2

STATION: 58+07 OFFSET: 64.5' Lt REFERENCE LINE: Survey GROUND LINE ELEVATION: 11.1'

BORINGS MADE BY: B. Nicolosi DATE STARTED: 10/11/85

INSPECTOR: Art Leong DATE COMPLETED: 10/14/85

Elevation G.W.T.
0 Hr. Date
24 Hr. +1.2 Date 11/1/85
ft. O.W. Date

	CASING BLOWS	SAMPLE NO.	DEPTH	Blows on Spoon 0/6	6/12	12/18	REC.	Sample ID and Profile Change	Depth
	10	S-1	0'-2'	8	11		1.7'	Brown m-f SAND, little Silt, trace m-f Gravel, trace ashes (fill)	
	15			6	9				
	14								
	35								
5		S-2	5'-7'	11	12		1.4'	Red brown c-f SAND, some (-) m-f Gravel, little Silt (fill)	
		S-3	7'-9'	12	11		1.2'	Red brown c-f SAND, little m-f Gravel, little Silt (fill)	
				15	19				
10		S-4A	9'-10'	12	11		0.5'	Same	10'
		S-4B	10'-11'	4	4		0.5'	Grey SILT & CLAY, w/organic fibers	
	Used	U-1	11'-13'	Press			0.9'		13'
		S-5	13'-15'	13	12		1.5'	Red brown c-f SAND, some (-) m-f Gravel, little Silt	
15				25	22				
	Mud								
20		S-6	20'-22'	8	10		1.8'	Red brown m-f SAND, trace (+) Silt	
				10	12				
	Drilling								
25		S-7	25'-27'	8	6		2.0'	Red brown m-f SAND, some (-) Silt, trace (-) f Gravel	
				10	9				
30									
	Revert	S-8	30'-32'	10	12		2.0'	Same	
				15	19				
35									
		S-9	35'-40'	11	13		1.5'	Red brown c-f SAND, little (+) m-f Gravel, little (-) Silt	
				13	21				
40									

Nominal I.D. of Drive Pipe	2½"	57'	4" 15'
Nominal I.D. of Split Barrel Sampler	1½"		
Weight of hammer on Drive Pipe	300 lbs.		
Weight of hammer on Split Barrel Sampler	140 lbs.		
Drop of hammer on Drive Pipe	24"		
Drop of hammer on Split Barrel Sampler	30"		

The Contractor shall make his own subsurface investigations in order to satisfy himself of the actual subsurface conditions. The information contained on this log is not warranted to show the actual subsurface conditions. The Contractor agrees that he will make no claims against the State if he finds that the actual conditions do not conform to those indicated by this log.

Core Dia. _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Approximate Change in Strata _____

Inferred Change in Strata _____

GOODKIND & O'DEA, INC. FIELD NO. B-11

ROUTE: Raymond Blvd. LOCAL NAME: Jackson St. Bridge Rehabilitation TEST HOLE NO.

Station: Offset: Ref. Line: SHEET 2 of 2

STATION: 58+07 OFFSET: 64.5' Lt REFERENCE LINE: Survey GROUND LINE ELEVATION: 11.1'

BORINGS MADE BY: B. Nicolosi DATE STARTED: 10/11/85

INSPECTOR: Art Leong DATE COMPLETED: 10/14/85

Elevation G.W.T.
0 Hr. Date
24 Hr. Date
____ hr. O.W. Date

	CASING BLOWS	SAMPLE NO.	DEPTH		Blows on Spoon 0/6 6/12 12/18	REC.	Sample ID and Profile Change	Depth
40		S-10	40'	42'	10 12 / 15 18	1.6'	Red brown m-f SAND, trace (+) Silt	
45		S-11	45'	47'	12 18 / 23 26	2.0'	Same	
50		S-12	50'	52'	13 16 / 15 20	1.5'	Red brown f SAND, little Silt	
55		S-13	55'	57'	14 15 / 17 16	1.0'	Red brown f SAND, some (-) Silt	58'
60	Revert Drilling Mud Used	S-14	60'	62'	7 12 / 12 13	1.2'	Red brown SILT & CLAY, trace f Sand	63'
65		S-15	65'	67'	13 15 / 20 20	1.5'	Red brown f SAND, trace Silt	
70		S-16	70'	72'	15 17 / 19 21	1.7'	Red brown f SAND, some Silt	72'
75							Bottom of Boring 72'	
80								

Nominal I.D. of Drive Pipe	2½"	4"
Nominal I.D. of Split Barrel Sampler	1½"	
Weight of hammer on Drive Pipe	300 lbs.	
Weight of hammer on Split Barrel Sampler	140 lbs.	
Drop of hammer on Drive Pipe	24"	
Drop of hammer on Split Barrel Sampler	30"	

The Contractor shall make his own subsurface investigations in order to satisfy himself of the actual subsurface conditions. The information contained on this log is not warranted to show the actual subsurface conditions. The Contractor agrees that he will make no claims against the State if he finds that the actual conditions do not conform to those indicated by this log.

Core Dia. _____

Soil descriptions represent a field identification after D.M. Burmister unless otherwise noted.

Approximate Change in Strata _____

Inferred Change in Strata _____

ATTACHMENT B – LIQUEFACTION EVALUATION

PASSAIC RIVER TIDAL GRR
LOWER PASSAIC RIVER, NEW JERSEY
LIQUEFACTION RESISTANCE EVALUATION
SEGMENT 1

DATA COLLECTION

Input by: VK Date: 8/16/18
Ck'd by: AH Date: 8/22/18
Source:

BORING INFORMATION

Boring No. 355W-56,355W-57&355W-58
Coordinates Station Offset
Surface Elev., ft 10.00 ∓ Total Depth, ft 26.0
Drilling Date

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method Mud Rotary

2500 yr.
Earthquake Magnitude 5.5
Magnitude Scaling Factor 1.69
G.S. Acc. (%g): 0.32

Groundwater: Depth, ft 2.0 Elev., ft 8.0 Remarks

Depth (ft)	Depth m	Elevation (ft)	**Layer**[1]	**N (bpf)**	Ave. Shear Wave Velocity (fps)	**Idealized Sat. Unit Weight (pcf)**	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	**Percent Fines (FC)**	FC Corrected $(N_1)_{60}$	CRR	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	K_{σ}	r_d	CRR	CSR*	FS, CRR/CSR*
2.00	0.60	8.0	**1**	**26**	794	**120**	240	0	240	1.70	1	2.60	0.75	20	33	**5**	33	0.60	2.0	240	240	0.24	1.00	1.00	0.60	0.21	2.89
3.50	1.05	6.5	**1**	**24**	790	**120**	420	94	326	1.70	1	3.05	0.80	19	33	**20**	37	0.60	3.5	420	326	0.23	1.00	0.99	0.60	0.27	2.26
5.00	1.50	5.0	**1**	**15**	668	**120**	600	187	413	1.70	1	3.50	0.80	12	20	**20**	25	0.29	5.0	600	413	0.14	1.00	0.98	0.48	0.30	1.63
11.00	3.30	-1.0	**1**	**6**	495	**120**	1320	562	758	1.70	1	5.30	0.85	5	9	**5**	9	0.11	11.0	1320	758	0.09	1.00	0.95	0.18	0.34	0.54
16.00	4.80	-6.0	**1**	**28**	888	**125**	1945	874	1071	1.30	1	6.80	0.95	27	35	**50**	40	0.60	16.0	1945	1071	0.26	1.00	0.91	0.60	0.34	1.75
21.00	6.31	-11.0	**1**	**47**	1071	**125**	2570	1186	1384	1.12	1	8.31	0.95	45	50	**50**	56	0.60	21.0	2570	1384	0.30	1.00	0.87	0.60	0.34	1.78
26.00	7.81	-16.0	**1**	**44**	1045	**125**	3195	1498	1697	1.07	1	9.81	0.95	42	45	**20**	49	0.60	26.0	3195	1697	0.30	1.00	0.83	0.60	0.33	1.85

[1]Layer Code — Soil Type
1 — Sand

* CSR = 0.65 $\alpha_{max}(\sigma_v/\sigma_v')\, r_d$

PASSAIC RIVER TIDAL GRR
LOWER PASSAIC RIVER, NEW JERSEY
LIQUEFACTION RESISTANCE EVALUATION
SEGMENT 2

DATA COLLECTION

Input by: VK Date: 8/16/18
Ck'd by: AH Date: 8/22/18
Source:

BORING INFORMATION

Boring No. S-1012, S-225, S-1009, S-1005, S-1006, S-212, S-210, & 355W-65
Coordinates Station Offset
Surface Elev., ft 13.00 ∓ Total Depth, ft 50.0
Drilling Date

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method Mud Rotary

2500 yr.
Earthquake Magnitude 5.5
Magnitude Scaling Factor 1.69
G.S. Acc. (%g): 0.32

Groundwater: Depth, ft 9.5 Elev., ft 3.5 Remarks

Depth (ft)	Depth m	Elevation (ft)	Layer[1]	N (bpf)	Ave. Shear Wave Velocity (fps)	Idealized Sat. Unit Weight (pcf)	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	Percent Fines (FC)	FC Corrected $(N_1)_{60}$	CRR	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	$K_{σ}$	r_d	CRR	CSR*	FS, CRR/CSR*
5.00	1.50	8.0	1	21	753	110	550	0	550	1.70	1	3.50	0.80	17	29	5	29	0.41	5.0	550	550	0.19	1.00	0.98	0.69	0.20	3.36
7.00	2.10	6.0	1	15	683	110	770	0	770	1.68	1	4.10	0.85	13	21	5	21	0.22	7.0	770	770	0.14	1.00	0.97	0.38	0.20	1.87
6.89	2.07	6.1	1	6	495	110	758	0	758	1.70	1	4.07	0.85	5	9	5	9	0.11	6.9	758	758	0.09	1.00	0.97	0.18	0.20	0.91
9.84	2.95	3.2	1	16	698	110	1082	21	1061	1.41	1	4.95	0.85	14	19	5	19	0.20	9.8	1082	1061	0.13	1.00	0.95	0.33	0.20	1.64
8.00	2.40	5.0	1	8	547	110	880	0	880	1.67	1	4.40	0.85	7	11	5	11	0.13	8.0	880	880	0.10	1.00	0.96	0.22	0.20	1.07
7.00	2.10	6.0	1	24	807	110	770	0	770	1.56	1	4.10	0.85	20	32	5	32	0.60	7.0	770	770	0.22	1.00	0.97	0.60	0.20	2.97
9.00	2.70	4.0	1	7	522	110	990	0	990	1.58	1	4.70	0.85	6	9	18	13	0.14	9.0	990	990	0.09	1.00	0.96	0.24	0.20	1.21
10.00	3.00	3.0	1	11	612	110	1100	31	1069	1.46	1	5.00	0.85	9	14	5	14	0.14	10.0	1100	1069	0.11	1.00	0.95	0.24	0.20	1.20
9.84	2.95	3.2	1	10	592	110	1082	21	1061	1.47	1	4.95	0.85	9	13	5	13	0.14	9.8	1082	1061	0.10	1.00	0.95	0.23	0.20	1.14
9.84	2.95	3.2	1	11	612	110	1082	21	1061	1.46	1	4.95	0.85	9	14	5	14	0.15	9.8	1082	1061	0.11	1.00	0.95	0.25	0.20	1.21
10.00	3.00	3.0	1	15	683	110	1100	31	1069	1.42	1	5.00	0.85	13	18	5	18	0.18	10.0	1100	1069	0.12	1.00	0.95	0.31	0.20	1.53
10.00	3.00	3.0	1	52	1067	110	1100	31	1069	1.21	1	5.00	0.85	44	53	5	53	0.60	10.0	1100	1069	0.30	1.00	0.95	0.60	0.20	2.94
14.76	4.43	-1.8	1	48	1079	110	1624	328	1295	1.14	1	6.43	0.95	46	52	5	52	0.60	14.8	1624	1295	0.30	1.00	0.92	0.60	0.24	2.51
12.00	3.60	1.0	1	6	495	110	1320	156	1164	1.44	1	5.60	0.85	5	7	25	12	0.14	12.0	1320	1164	0.08	1.00	0.94	0.23	0.22	1.03
14.76	4.43	-1.8	1	18	758	110	1624	328	1295	1.26	1	6.43	0.95	17	22	5	22	0.23	14.8	1624	1295	0.14	1.00	0.92	0.38	0.24	1.59
15.00	4.50	-2.0	1	12	656	110	1650	343	1307	1.29	1	6.50	0.95	11	15	5	15	0.15	15.0	1650	1307	0.11	1.00	0.92	0.26	0.24	1.08
14.76	4.43	-1.8	1	7	543	110	1624	328	1295	1.33	1	6.43	0.95	7	9	18	13	0.14	14.8	1624	1295	0.09	1.00	0.92	0.24	0.24	0.98
15.00	4.50	-2.0	1	10	615	110	1650	343	1307	1.30	1	6.50	0.95	10	12	18	16	0.17	15.0	1650	1307	0.10	1.00	0.92	0.28	0.24	1.18
15.00	4.50	-2.0	1	60	1171	110	1650	343	1307	1.10	1	6.50	0.95	57	63	5	63	0.60	15.0	1650	1307	0.30	1.00	0.92	0.60	0.24	2.49
15.00	4.50	-2.0	1	11	636	110	1650	343	1307	1.30	1	6.50	0.95	10	14	5	14	0.14	15.0	1650	1307	0.11	1.00	0.92	0.24	0.24	1.01
19.68	5.91	-6.7	1	11	636	110	2165	635	1530	1.19	1	7.91	0.95	10	12	5	12	0.14	19.7	2165	1530	0.10	1.00	0.88	0.23	0.26	0.88
19.68	5.91	-6.7	1	17	742	110	2165	635	1530	1.17	1	7.91	0.95	16	19	5	19	0.19	19.7	2165	1530	0.13	1.00	0.88	0.33	0.26	1.25
19.68	5.91	-6.7	1	22	814	110	2165	635	1530	1.15	1	7.91	0.95	21	24	5	24	0.27	19.7	2165	1530	0.16	1.00	0.88	0.45	0.26	1.75
20.00	6.01	-7.0	1	12	656	110	2200	655	1545	1.18	1	8.01	0.95	11	13	25	19	0.19	20.0	2200	1545	0.10	1.00	0.88	0.32	0.26	1.23
19.68	5.91	-6.7	1	18	758	110	2165	635	1530	1.16	1	7.91	0.95	17	20	5	20	0.20	19.7	2165	1530	0.13	1.00	0.88	0.35	0.26	1.33
20.00	6.01	-7.0	1	12	656	110	2200	655	1545	1.18	1	8.01	0.95	11	13	5	13	0.14	20.0	2200	1545	0.10	1.00	0.88	0.24	0.26	0.93
20.00	6.01	-7.0	1	18	758	110	2200	655	1545	1.16	1	8.01	0.95	17	20	5	20	0.20	20.0	2200	1545	0.13	1.00	0.88	0.34	0.26	1.32
24.60	7.39	-11.6	1	31	921	110	2706	942	1764	1.07	1	9.39	0.95	29	32	5	32	0.60	24.6	2706	1764	0.22	1.00	0.84	1.01	0.27	3.75
24.60	7.39	-11.6	1	24	840	110	2706	942	1764	1.08	1	9.39	0.95	23	25	5	25	0.28	24.6	2706	1764	0.16	1.00	0.84	0.47	0.27	1.77
24.60	7.39	-11.6	1	18	758	110	2706	942	1764	1.09	1	9.39	0.95	17	19	5	19	0.19	24.6	2706	1764	0.13	1.00	0.84	0.32	0.27	1.20
25.00	7.51	-12.0	1	20	787	110	2750	967	1783	1.08	1	9.51	0.95	19	21	5	21	0.21	25.0	2750	1783	0.14	1.00	0.84	0.36	0.27	1.33
25.00	7.51	-12.0	1	22	814	110	2750	967	1783	1.08	1	9.51	0.95	21	23	5	23	0.24	25.0	2750	1783	0.15	1.00	0.84	0.41	0.27	1.51
25.00	7.51	-12.0	1	100	1413	110	2750	967	1783	1.01	1	9.51	0.95	95	96	5	96	0.60	25.0	2750	1783	0.30	1.00	0.84	0.60	0.27	2.23
25.00	7.51	-12.0	1	26	864	110	2750	967	1783	1.07	1	9.51	0.95	25	26	5	26	0.33	25.0	2750	1783	0.17	1.00	0.84	0.56	0.27	2.07
29.52	8.86	-16.5	1	36	990	110	3247	1249	1998	1.02	1	10.86	1.00	36	37	5	37	0.60	29.5	3247	1998	0.29	1.00	0.80	0.60	0.27	2.22
29.52	8.86	-16.5	1	20	801	110	3247	1249	1998	1.03	1	10.86	1.00	20	21	5	21	0.21	29.5	3247	1998	0.14	1.00	0.80	0.36	0.27	1.32
29.52	8.86	-16.5	1	23	843	110	3247	1249	1998	1.02	1	10.86	1.00	23	24	5	24	0.26	29.5	3247	1998	0.15	1.00	0.80	0.44	0.27	1.62
30.00	9.01	-17.0	1	21	815	110	3300	1279	2021	1.02	1	11.01	1.00	21	21	5	21	0.22	30.0	3300	2021	0.14	1.00	0.80	0.38	0.27	1.40
30.00	9.01	-17.0	1	10	627	110	3300	1279	2021	1.03	1	11.01	1.00	10	10	5	10	0.12	30.0	3300	2021	0.09	1.00	0.80	0.20	0.27	0.75
30.00	9.01	-17.0	1	17	756	110	3300	1279	2021	1.02	1	11.01	1.00	17	17	15	21	0.21	30.0	3300	2021	0.12	1.00	0.80	0.36	0.27	1.33
34.44	10.34	-21.4	1	65	1228	110	3788	1556	2232	0.99	1	12.34	1.00	65	64	5	64	0.60	34.4	3788	2232	0.30	0.98	0.76	0.60	0.27	2.24
34.44	10.34	-21.4	1	25	868	110	3788	1556	2232	0.98	1	12.34	1.00	25	24	5	24	0.28	34.4	3788	2232	0.16	0.99	0.76	0.47	0.27	1.74
34.44	10.34	-21.4	1	25	868	110	3788	1556	2232	0.98	1	12.34	1.00	25	24	5	24	0.28	34.4	3788	2232	0.16	0.99	0.76	0.47	0.27	1.74
35.00	10.51	-22.0	1	26	880	110	3850	1591	2259	0.98	1	12.51	1.00	26	25	5	25	0.30	35.0	3850	2259	0.17	0.99	0.76	0.50	0.27	1.86
35.00	10.51	-22.0	1	12	668	110	3850	1591	2259	0.97	1	12.51	1.00	12	12	18	16	0.16	35.0	3850	2259	0.10	0.99	0.76	0.27	0.27	1.01
35.00	10.51	-22.0	1	22	829	110	3850	1591	2259	0.97	1	12.51	1.00	22	21	5	21	0.22	35.0	3850	2259	0.14	0.99	0.76	0.38	0.27	1.40
39.36	11.82	-26.4	1	93	1402	110	4330	1863	2466	0.99	1	13.82	1.00	93	92	5	92	0.60	39.4	4330	2466	0.30	0.95	0.72	0.60	0.26	2.28
39.36	11.82	-26.4	1	28	904	110	4330	1863	2466	0.94	1	13.82	1.00	28	26	5	26	0.33	39.4	4330	2466	0.17	0.97	0.72	0.54	0.26	2.06
39.36	11.82	-26.4	1	29	916	110	4330	1863	2466	0.95	1	13.82	1.00	29	27	5	27	0.36	39.4	4330	2466	0.18	0.97	0.72	0.59	0.26	2.26
40.00	12.01	-27.0	1	16	740	110	4400	1903	2497	0.92	1	14.01	1.00	16	15	5	15	0.15	40.0	4400	2497	0.11	0.98	0.71	0.26	0.26	0.98
44.28	13.30	-31.3	1	41	1038	110	4871	2170	2701	0.93	1	15.30	1.00	41	38	5	38	0.60	44.3	4871	2701	0.30	0.93	0.68	0.60	0.26	2.35
45.00	13.51	-32.0	1	27	893	110	4950	2215	2735	0.91	1	15.51	1.00	27	24	5	24	0.28	45.0	4950	2735	0.16	0.96	0.67	0.45	0.25	1.77
50.00	15.02	-37.0	1	14	706	110	5500	2527	2973	0.85	1	17.02	1.00	14	12	18	16	0.16	50.0	5500	2973	0.10	0.97	0.64	0.27	0.24	1.09

PASSAIC RIVER TIDAL GRR
LOWER PASSAIC RIVER, NEW JERSEY
LIQUEFACTION RESISTANCE EVALUATION
SEGMENT 3,4&5

DATA COLLECTION

Input by: VK Date: 8/16/18
Ck'd by: AH Date: 8/22/18
Source:

BORING INFORMATION

Boring No. B-7, B-8, B-9 & B-10
Coordinates Station Offset
Surface Elev., ft 10.50 ∓ Total Depth, ft 50.0
Drilling Date

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type Auto Liners No
Drilling Method Mud Rotary

2500 yr.
Earthquake Magnitude 5.5
Magnitude Scaling Factor 1.69
G.S. Acc. (%g): 0.32

Groundwater: Depth, ft 6.0 Elev., ft 4.5 Remarks

Depth (ft)	Depth m	Elevation (ft)	**Layer**[1]	**N (bpf)**	Ave. Shear Wave Velocity (fps)	**Idealized Sat. Unit Weight (pcf)**	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	**Percent Fines (FC)**	FC Corrected $(N_1)_{60}$	CRR	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	K_{σ}	r_d	CRR	CSR*	FS, CRR/CSR*
7.00	2.10	3.5	**1**	**6**	547	**90**	630	62	568	1.70	1.33	4.10	0.85	7	12	**30**	17	0.17	7.0	630	568	0.10	1.00	0.97	0.29	0.22	1.30
9.00	2.70	1.5	**1**	**10**	654	**90**	810	187	623	1.70	1.33	4.70	0.85	11	19	**30**	25	0.28	9.0	810	623	0.13	1.00	0.96	0.47	0.26	1.82
11.00	3.30	-0.5	**1**	**9**	630	**90**	990	312	678	1.70	1.33	5.30	0.85	10	17	**25**	22	0.24	11.0	990	678	0.12	1.00	0.95	0.40	0.29	1.40
16.00	4.80	-5.5	**1**	**6**	568	**90**	1440	624	816	1.70	1.33	6.80	0.95	8	13	**25**	18	0.18	16.0	1440	816	0.10	1.00	0.91	0.31	0.33	0.93

[1]Layer Code — Soil Type
1 — Sand

* CSR = 0.65 $\alpha_{max}(\sigma_v/\sigma_v')\, r_d$

PASSAIC RIVER TIDAL GRR
LOWER PASSAIC RIVER, NEW JERSEY
LIQUEFACTION RESISTANCE EVALUATION
SEGMENT 6

DATA COLLECTION

Input by: VK Date: 8/16/18
Ck'd by: AH Date: 8/22/18
Source:

BORING INFORMATION

Boring No. 94, 95, 101,&102
Coordinates Station Offset
Surface Elev., ft 17.00 ∓ Total Depth, ft 42.0
Drilling Date

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method Mud Rotary

2500 yr.
Earthquake Magnitude 5.5
Magnitude Scaling Factor 1.69
G.S. Acc. (%g): 0.32

Groundwater: Depth, ft 8.0 Elev., ft 9.0 Remarks

Depth (ft)	Depth m	Elevation (ft)	**Layer**[1]	**N (bpf)**	Ave. Shear Wave Velocity (fps)	**Idealized Sat. Unit Weight (pcf)**	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	**Percent Fines (FC)**	FC Corrected $(N_1)_{60}$	CRR	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	K_{σ}	r_d	CRR	CSR*	FS, CRR/CSR*
10.00	3.00	7.0	**1**	**11**	612	**120**	1200	125	1075	1.45	1	5.00	0.85	9	14	**12**	16	0.16	10.0	1200	1075	0.11	1.00	0.95	0.27	0.22	1.23
10.00	3.00	7.0	**1**	**34**	915	**120**	1200	125	1075	1.29	1	5.00	0.85	29	37	**5**	37	0.60	10.0	1200	1075	0.30	1.00	0.95	0.60	0.22	2.71
10.00	3.00	7.0	**1**	**20**	756	**120**	1200	125	1075	1.37	1	5.00	0.85	17	23	**5**	23	0.26	10.0	1200	1075	0.15	1.00	0.95	0.43	0.22	1.95
11.50	3.45	5.5	**1**	**11**	612	**120**	1380	218	1162	1.39	1	5.45	0.85	9	13	**15**	16	0.17	11.5	1380	1162	0.10	1.00	0.94	0.28	0.23	1.21
15.00	4.50	2.0	**1**	**25**	852	**120**	1800	437	1363	1.20	1	6.50	0.95	24	28	**5**	28	0.40	15.0	1800	1363	0.19	1.00	0.92	0.68	0.25	2.70
15.00	4.50	2.0	**1**	**27**	876	**120**	1800	437	1363	1.19	1	6.50	0.95	26	31	**5**	31	0.52	15.0	1800	1363	0.21	1.00	0.92	0.88	0.25	3.49
15.00	4.50	2.0	**1**	**28**	888	**120**	1800	437	1363	1.19	1	6.50	0.95	27	32	**5**	32	0.60	15.0	1800	1363	0.22	1.00	0.92	0.60	0.25	2.38
20.00	6.01	-3.0	**1**	**28**	888	**120**	2400	749	1651	1.10	1	8.01	0.95	27	29	**5**	29	0.44	20.0	2400	1651	0.20	1.00	0.88	0.75	0.27	2.83
20.00	6.01	-3.0	**1**	**26**	864	**120**	2400	749	1651	1.11	1	8.01	0.95	25	27	**5**	27	0.36	20.0	2400	1651	0.18	1.00	0.88	0.60	0.27	2.27
20.00	6.01	-3.0	**1**	**24**	840	**120**	2400	749	1651	1.11	1	8.01	0.95	23	25	**5**	25	0.30	20.0	2400	1651	0.16	1.00	0.88	0.50	0.27	1.89
25.00	7.51	-8.0	**1**	**22**	814	**120**	3000	1061	1939	1.04	1	9.51	0.95	21	22	**5**	22	0.23	25.0	3000	1939	0.14	1.00	0.84	0.39	0.27	1.43
25.00	7.51	-8.0	**1**	**19**	773	**120**	3000	1061	1939	1.04	1	9.51	0.95	18	19	**5**	19	0.19	25.0	3000	1939	0.13	1.00	0.84	0.32	0.27	1.20
25.00	7.51	-8.0	**1**	**25**	852	**120**	3000	1061	1939	1.04	1	9.51	0.95	24	25	**5**	25	0.28	25.0	3000	1939	0.16	1.00	0.84	0.48	0.27	1.76
26.50	7.96	-9.5	**1**	**24**	840	**120**	3180	1154	2026	1.02	1	9.96	0.95	23	23	**5**	23	0.25	26.5	3180	2026	0.15	1.00	0.83	0.43	0.27	1.59
30.00	9.01	-13.0	**1**	**25**	868	**120**	3600	1373	2227	0.98	1	11.01	1.00	25	25	**5**	25	0.28	30.0	3600	2227	0.16	0.99	0.80	0.47	0.27	1.74
30.00	9.01	-13.0	**1**	**42**	1047	**120**	3600	1373	2227	0.99	1	11.01	1.00	42	41	**5**	41	0.60	30.0	3600	2227	0.30	0.99	0.80	0.60	0.27	2.24
30.00	9.01	-13.0	**1**	**25**	868	**120**	3600	1373	2227	0.98	1	11.01	1.00	25	25	**5**	25	0.28	30.0	3600	2227	0.16	0.99	0.80	0.47	0.27	1.74
35.00	10.51	-18.0	**1**	**44**	1065	**120**	4200	1685	2515	0.95	1	12.51	1.00	44	42	**5**	42	0.60	35.0	4200	2515	0.30	0.95	0.76	0.60	0.26	2.29
40.00	12.01	-23.0	**1**	**22**	829	**120**	4800	1997	2803	0.89	1	14.01	1.00	22	20	**5**	20	0.20	40.0	4800	2803	0.13	0.96	0.71	0.33	0.25	1.28
41.50	12.46	-24.5	**1**	**53**	1140	**120**	4980	2090	2890	0.93	1	14.46	1.00	53	49	**5**	49	0.60	41.5	4980	2890	0.30	0.91	0.70	0.60	0.25	2.38

[1]Layer Code — Soil Type
1 — Sand

* CSR = 0.65 $\alpha_{max}(\sigma_v/\sigma_v')\, r_d$

PASSAIC RIVER TIDAL GRR
LOWER PASSAIC RIVER, NEW JERSEY
LIQUEFACTION RESISTANCE EVALUATION
SEGMENT 8

DATA COLLECTION

Input by: VK Date: 8/16/18
Ck'd by: AH Date: 8/22/18
Source:

BORING INFORMATION

Boring No. B-2, B-3, & B-11
Coordinates Station Offset
Surface Elev., ft 11.00 ∓ Total Depth, ft 50.0
Drilling Date

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method Mud Rotary

2500 yr.
Earthquake Magnitude 5.5
Magnitude Scaling Factor 1.69
G.S. Acc. (%g): 0.32

Groundwater: Depth, ft 9.5 Elev., ft 1.5 Remarks

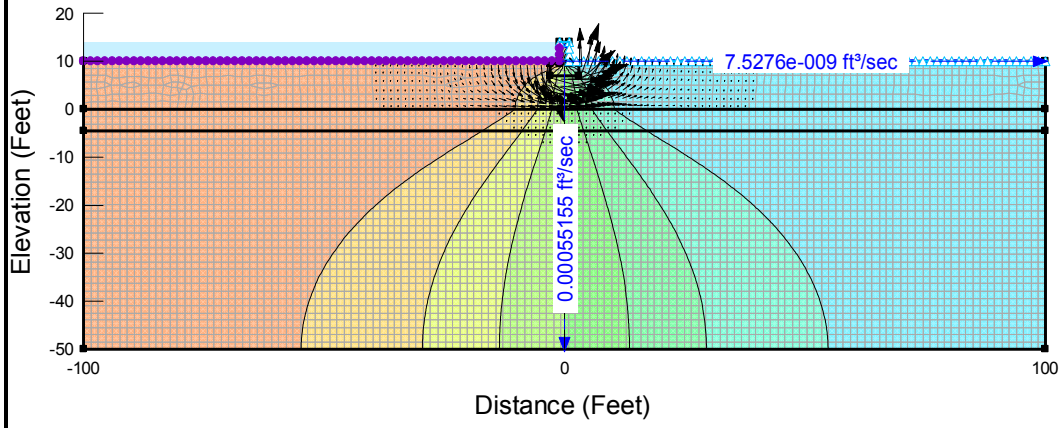
Depth (ft)	Depth m	Elevation (ft)	**Layer**[1]	**N (bpf)**	Ave. Shear Wave Velocity (fps)	**Idealized Sat. Unit Weight (pcf)**	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	**Percent Fines (FC)**	FC Corrected $(N_1)_{60}$	CRR	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	K_{σ}	r_d	CRR	CSR*	FS, CRR/CSR*
10.00	3.00	1.0	**1**	**22**	782	**120**	1200	31	1169	1.31	1	5.00	0.85	19	24	**5**	24	0.28	10.0	1200	1169	0.16	1.00	0.95	0.47	0.20	2.31
11.00	3.30	0.0	**1**	**20**	756	**120**	1320	94	1226	1.29	1	5.30	0.85	17	22	**5**	22	0.23	11.0	1320	1226	0.14	1.00	0.95	0.39	0.21	1.85
13.00	3.90	-2.0	**1**	**37**	943	**120**	1560	218	1342	1.18	1	5.90	0.85	31	37	**5**	37	0.60	13.0	1560	1342	0.29	1.00	0.93	0.60	0.23	2.66
15.00	4.50	-4.0	**1**	**46**	1062	**120**	1800	343	1457	1.11	1	6.50	0.95	44	48	**5**	48	0.60	15.0	1800	1457	0.30	1.00	0.92	0.60	0.24	2.55
20.00	6.01	-9.0	**1**	**20**	787	**120**	2400	655	1745	1.09	1	8.01	0.95	19	21	**5**	21	0.22	20.0	2400	1745	0.14	1.00	0.88	0.36	0.25	1.44
20.00	6.01	-9.0	**1**	**36**	972	**120**	2400	655	1745	1.07	1	8.01	0.95	34	37	**5**	37	0.60	20.0	2400	1745	0.29	1.00	0.88	0.60	0.25	2.39
25.00	7.51	-14.0	**1**	**16**	727	**120**	3000	967	2033	1.02	1	9.51	0.95	15	16	**20**	20	0.21	25.0	3000	2033	0.11	1.00	0.84	0.35	0.26	1.35
25.00	7.51	-14.0	**1**	**32**	932	**120**	3000	967	2033	1.02	1	9.51	0.95	30	31	**5**	31	0.54	25.0	3000	2033	0.21	1.00	0.84	0.92	0.26	3.57
30.00	9.01	-19.0	**1**	**27**	893	**120**	3600	1279	2321	0.97	1	11.01	1.00	27	26	**5**	26	0.32	30.0	3600	2321	0.17	0.98	0.80	0.53	0.26	2.05
30.00	9.01	-19.0	**1**	**39**	1019	**120**	3600	1279	2321	0.97	1	11.01	1.00	39	38	**5**	38	0.60	30.0	3600	2321	0.30	0.97	0.80	0.60	0.26	2.33
35.00	10.51	-24.0	**1**	**26**	880	**120**	4200	1591	2609	0.92	1	12.51	1.00	26	24	**5**	24	0.27	35.0	4200	2609	0.16	0.97	0.76	0.44	0.25	1.73
35.00	10.51	-24.0	**1**	**51**	1124	**120**	4200	1591	2609	0.95	1	12.51	1.00	51	49	**5**	49	0.60	35.0	4200	2609	0.30	0.94	0.76	0.60	0.25	2.37
40.00	12.01	-29.0	**1**	**27**	893	**120**	4800	1903	2897	0.89	1	14.01	1.00	27	24	**5**	24	0.27	40.0	4800	2897	0.16	0.95	0.71	0.43	0.25	1.74
40.00	12.01	-29.0	**1**	**61**	1200	**120**	4800	1903	2897	0.94	1	14.01	1.00	61	58	**5**	58	0.60	40.0	4800	2897	0.30	0.91	0.71	0.60	0.25	2.44
45.00	13.51	-34.0	**1**	**41**	1038	**120**	5400	2215	3185	0.89	1	15.51	1.00	41	36	**5**	36	0.60	45.0	5400	3185	0.28	0.88	0.67	0.60	0.24	2.52
45.00	13.51	-34.0	**1**	**70**	1262	**120**	5400	2215	3185	0.94	1	15.51	1.00	70	66	**5**	66	0.60	45.0	5400	3185	0.30	0.88	0.67	0.60	0.24	2.52
50.00	15.02	-39.0	**1**	**31**	938	**120**	6000	2527	3473	0.84	1	17.02	1.00	31	26	**5**	26	0.32	50.0	6000	3473	0.17	0.92	0.64	0.49	0.23	2.13
50.00	15.02	-39.0	**1**	**88**	1373	**120**	6000	2527	3473	0.97	1	17.02	1.00	88	85	**5**	85	0.60	50.0	6000	3473	0.30	0.85	0.64	0.60	0.23	2.62

[1]Layer Code — Soil Type
1 — Sand

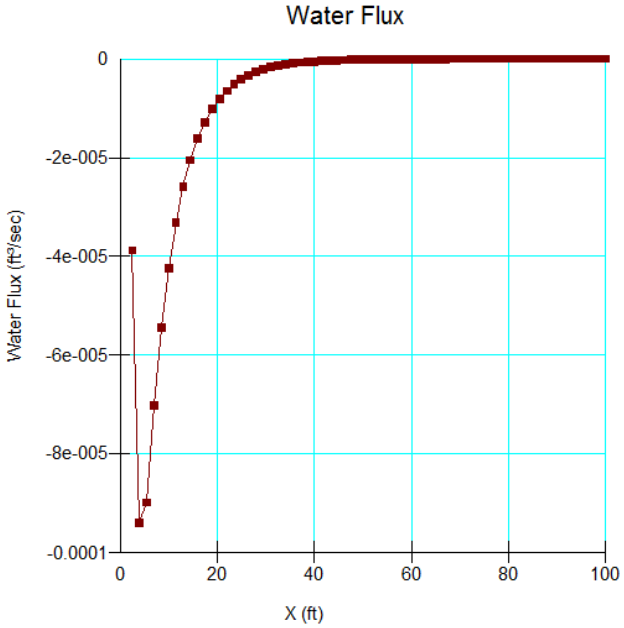
* CSR = 0.65 $\alpha_{max}(\sigma_v/\sigma_v')\, r_d$

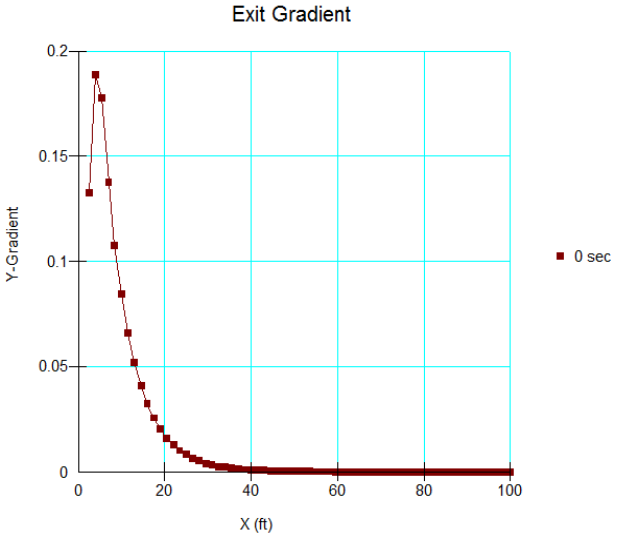
ATTACHMENT C – SEEPAGE AND STABILITY ANALYSIS FOR FLOOD WALL

Color	Name	Model	K-Function	Sat Kx (ft/sec)	Volumetric Water Content (ft^3/ft^3)	Vol. WC. Function
	Concrete	Saturated / Unsaturated	Concrete			
	Organic Clayey Silt	Saturated Only		3.28e-006	0	
	Sand with Silt and Gravel	Saturated Only		3.28e-006	0	
	Sand/Gravel (Fill)	Saturated / Unsaturated	Sand/Gravel (Fill)			VWC-Sand/Gravel (Fill)



Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Calculations\Floodwall\T-Wall\Seepage Analysis\

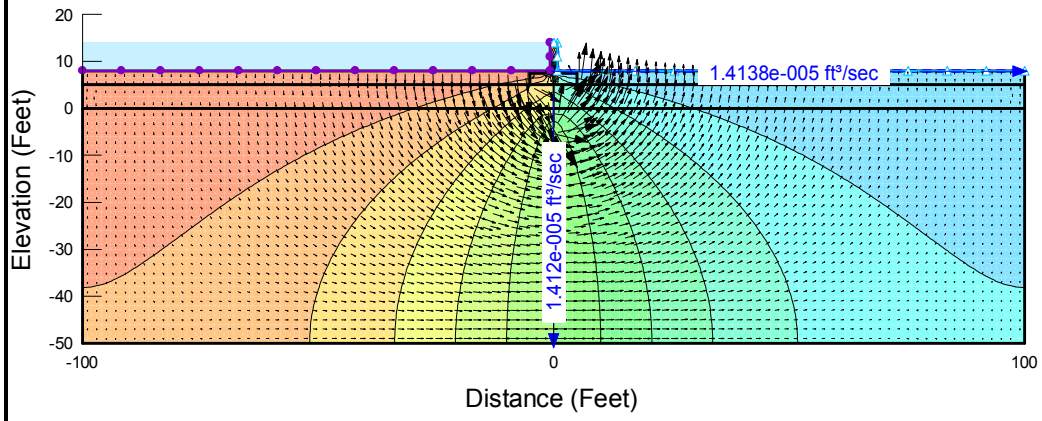




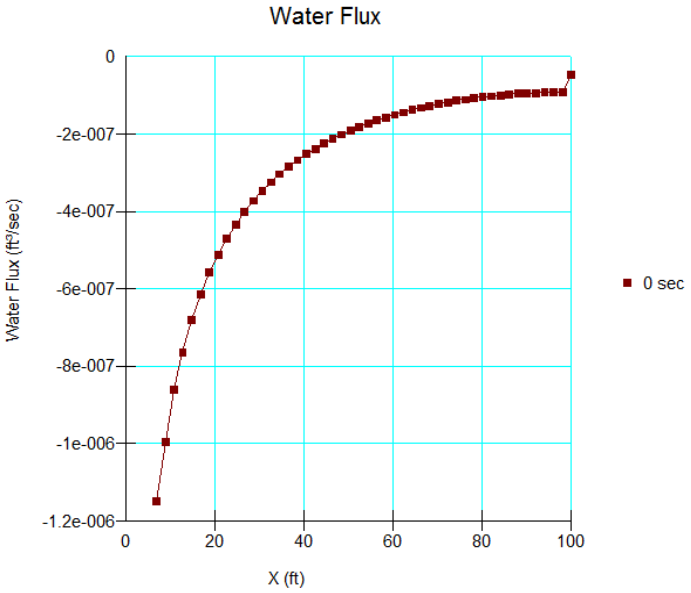
Steady-State Seepage for 4 ft High T-Wall (Full Flood at El. 14 ft)

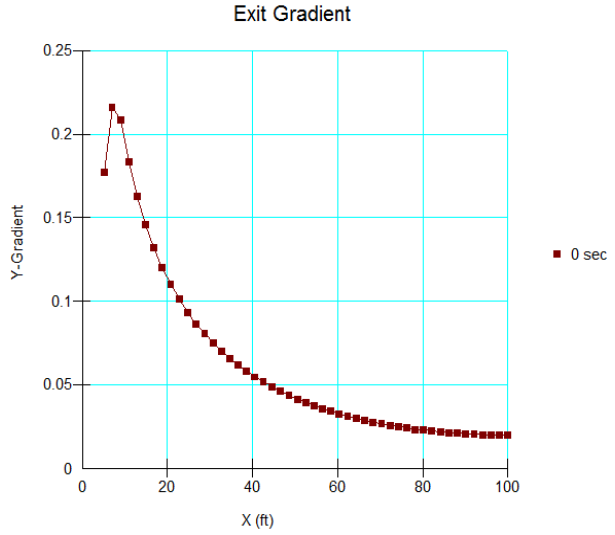
Seepage Analysis of T-Wall Segment 1 Passaic River Tidal GRR Lower Passaic River, New Jersey			
AECOM Clifton, NJ			
Prep'd by: VK	Date: 8/17/2018	Proj. No.:	60442748
Ck'd by: AH	Date: 8/20/2018	Sheet No.:	C.1

Color	Name	Model	Sat Kx (ft/sec)
	Concrete	Saturated / Unsaturated	
	Organic Clayey Silt	Saturated / Unsaturated	
	Sand (Fill)	Saturated / Unsaturated	
	Sheet Pile	Interface	
	Silty Sand	Saturated Only	3.28e-006



Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Calculations\Floodwall\T-Wall\Seepage Analysis\





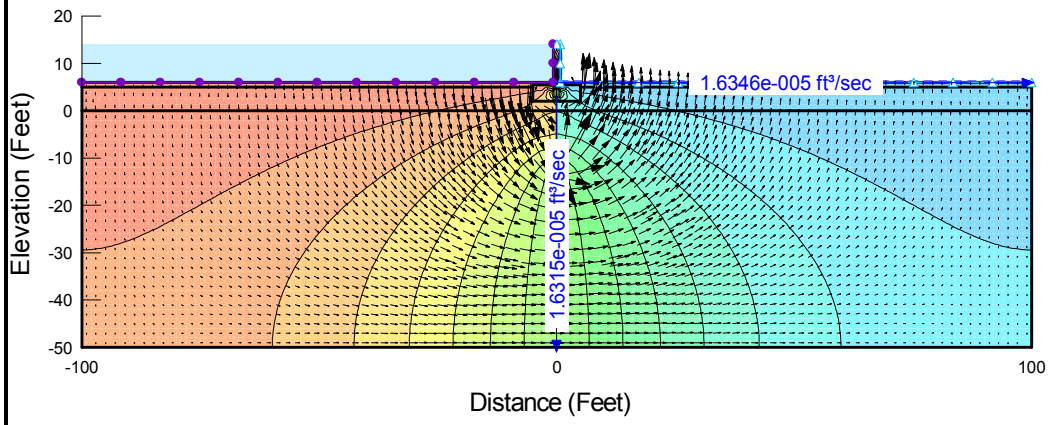
Steady-State Seepage for 6 ft High T-Wall (Full Flood at El. 14 ft)

Seepage Analysis of T-Wall
Segment 2
Passaic River Tidal GRR
Lower Passaic River, New Jersey

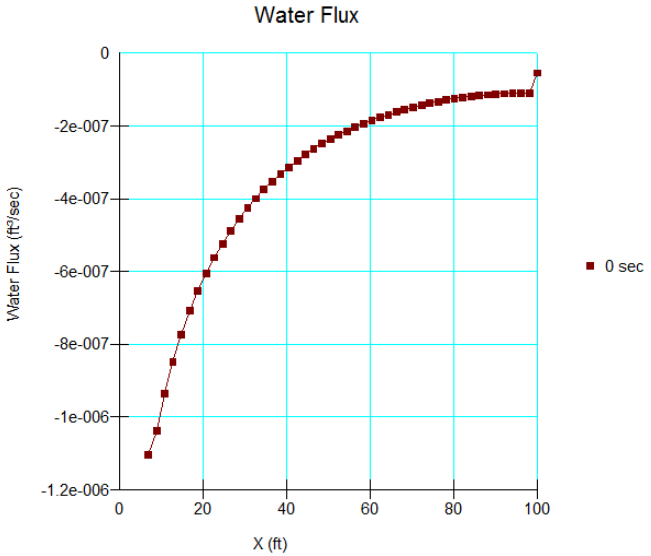
AECOM
Clifton, NJ

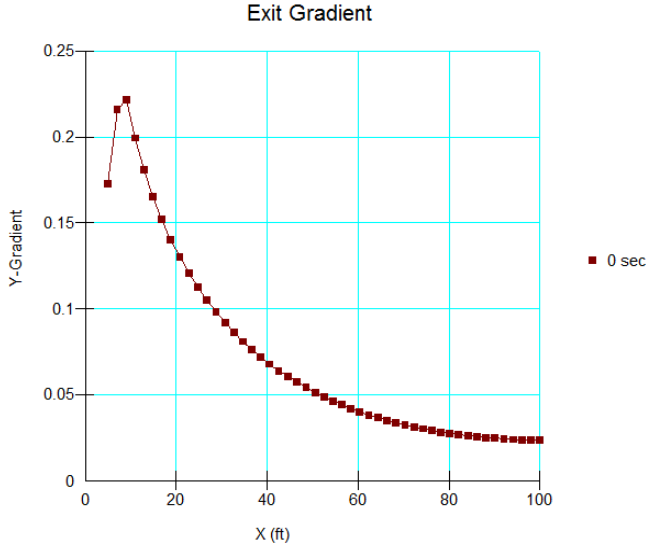
Prep'd by: VK	Date:	8/13/2018	Proj. No.:	60442748
Ck'd by: AH	Date:	8/16/2018	Sheet No.:	C.2

Color	Name	Model	Sat Kx (ft/sec)
	Concrete	Saturated / Unsaturated	
	Organic Clayey Silt	Saturated / Unsaturated	
	Sand (Fill)	Saturated / Unsaturated	
	Sheet Pile	Interface	
	Silty Sand	Saturated Only	3.28e-006



Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Calculations\Floodwall\T-Wall\Seepage Analysis\





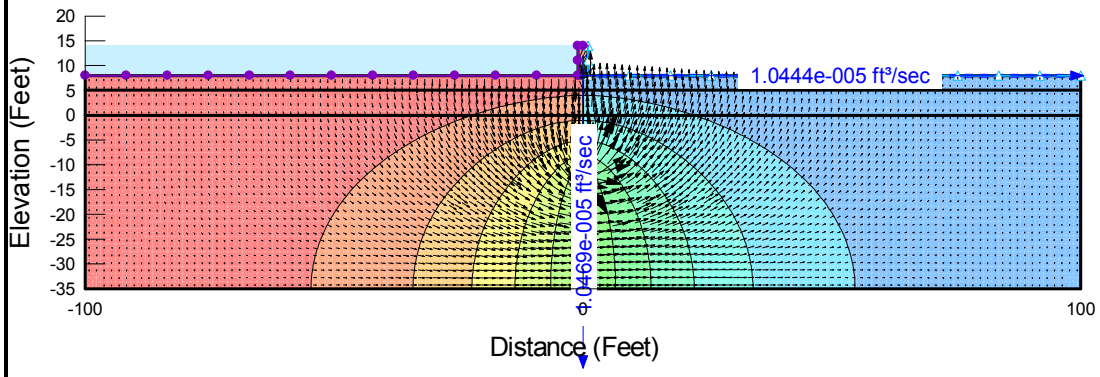
Steady-State Seepage for 8 ft High T-Wall (Full Flood at El. 14 ft)

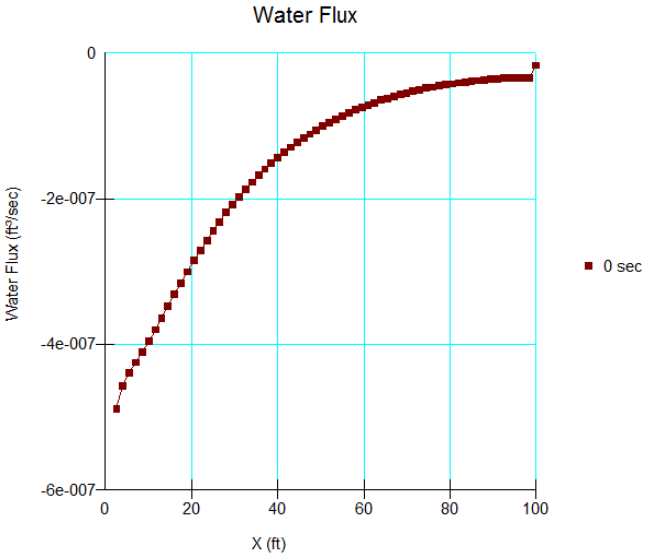
Seepage Analysis of T-Wall
Segment 2
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
Clifton, NJ

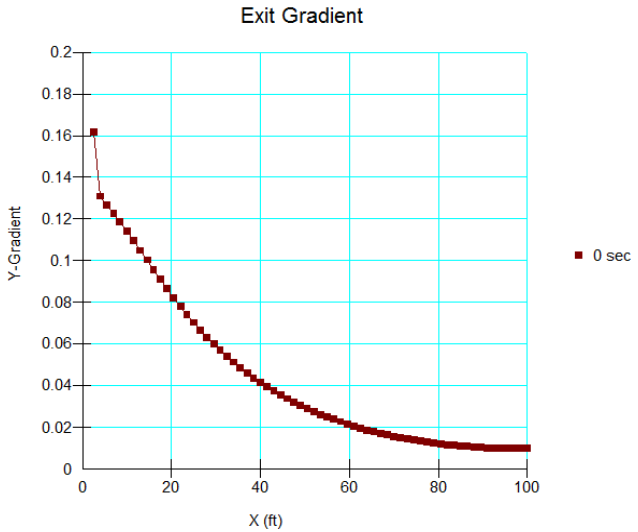
Prep'd by: VK	Date:	8/13/2018	Proj. No.:	60442748
Ck'd by: AH	Date:	8/16/2018	Sheet No.:	C.3

Color	Name	Model	K-Function	Vol. WC. Function	Sat Kx (ft/sec)	Volumetric Water Content (ft^3/ft^3)
	Concrete	Saturated / Unsaturated	Concrete			
	Organic Clayey Silt	Saturated / Unsaturated	Organic Clayey Silt	VWC-Organic Clayey Silt		
	Sand (Fill)	Saturated / Unsaturated	Fill	VWC-Fill		
	Sheet Pile	Interface				
	Silty Sand	Saturated Only			3.28e-006	0





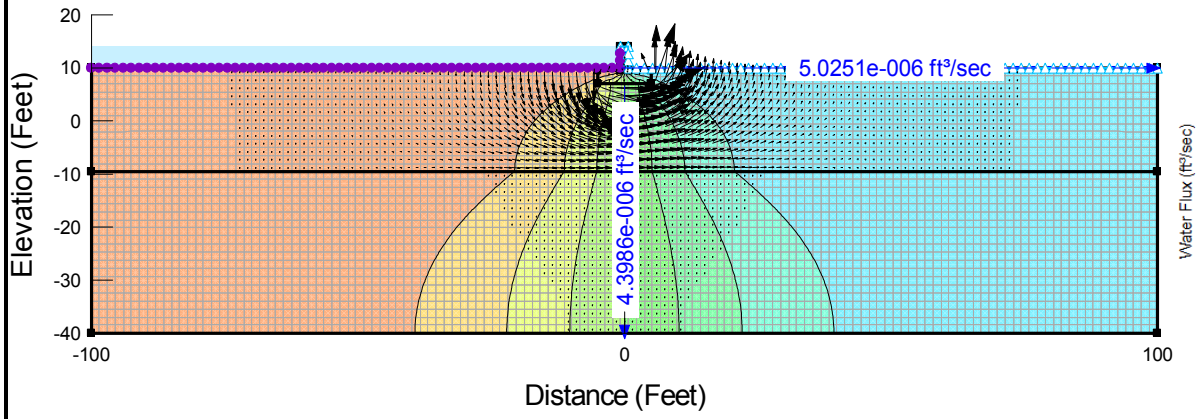
Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Calculations\Floodwall\I-Wall\Seepage Analysis\

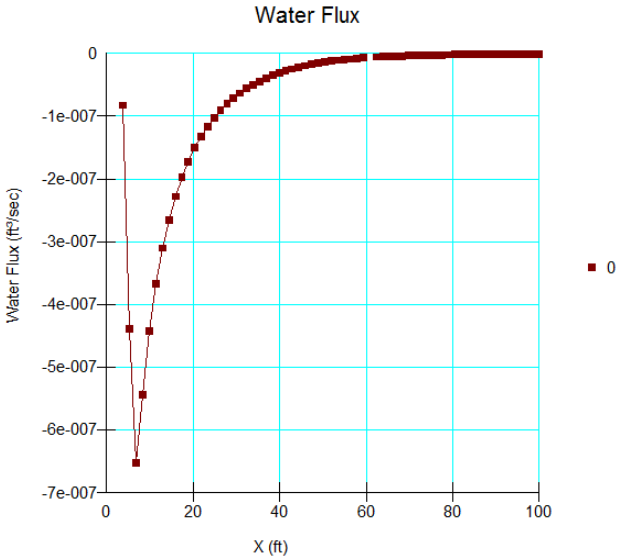


Steady-State Seepage for 6 ft High I-Wall with Sheetpile (Full Flood at El. 14 ft)

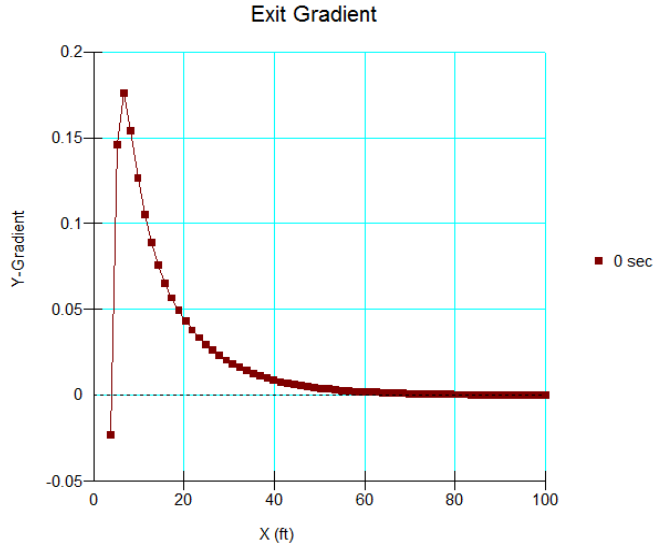
Seepage Analysis of I-Wall Segment 2 Passaic River Tidal GRR Lower Passaic River, New Jersey			
AECOM Clifton, NJ			
Prep'd by: VK	Date: 8/17/2018	Proj. No.:	60442748
Ck'd by: AH	Date: 8/22/2018	Sheet No.	C.4

Color	Name	Model	K-Function	Vol. WC. Function	Sat Kx (ft/sec)	Volumetric Water Content (ft^3/ft^3)
	Concrete	Saturated / Unsaturated	Concrete			
	Sand/Silt (Fill)	Saturated / Unsaturated	Sand/Silt (Fill)	VWC-Sand/Silt (Fill)		
	Sandy/Silty Clay	Saturated Only			3.28e-008	0





Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Calculations\Floodwall\T-Wall\Seepage Analysis\



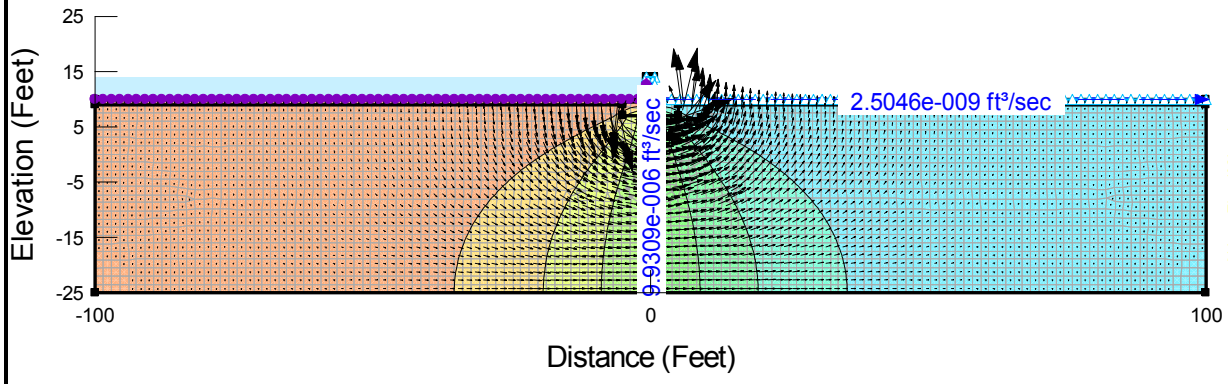
Steady-State Seepage for 4 ft High T-Wall (Full Flood at El. 14 ft)

Seepage Analysis of T-Wall
Segments 4&5
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
Clifton, NJ

Prep'd by: VK	Date:	8/13/2018	Proj. No.:	60442748
Ck'd by: AH	Date:	8/16/2018	Sheet No.:	C.5

Color	Name	Model	K-Function	Vol. WC. Function
	Concrete	Saturated / Unsaturated	Concrete	
	Sand with little Silt	Saturated / Unsaturated	Sand with little Silt	VWC-Sand with little Silt
	Sand/Silt (Fill)	Saturated / Unsaturated	Sand/Silt (Fill)	VWC-Sand/Silt (Fill)



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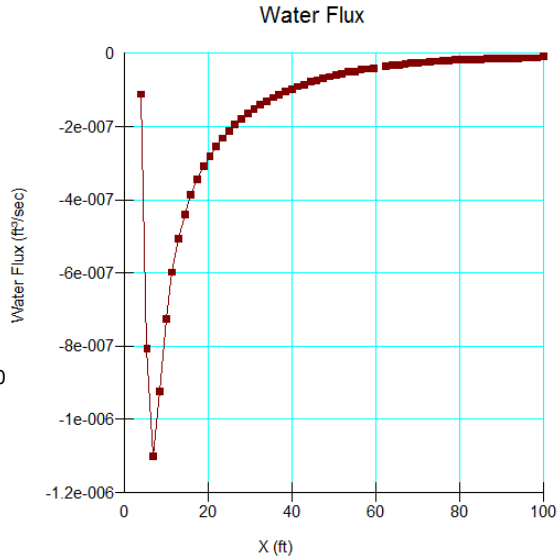
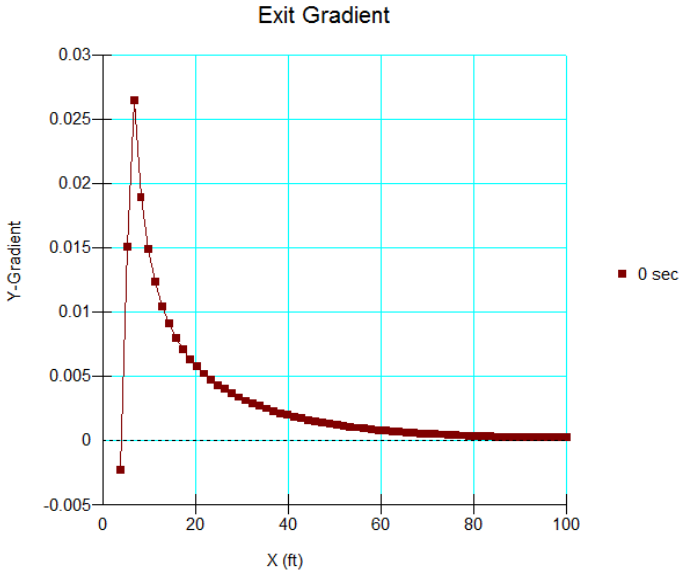
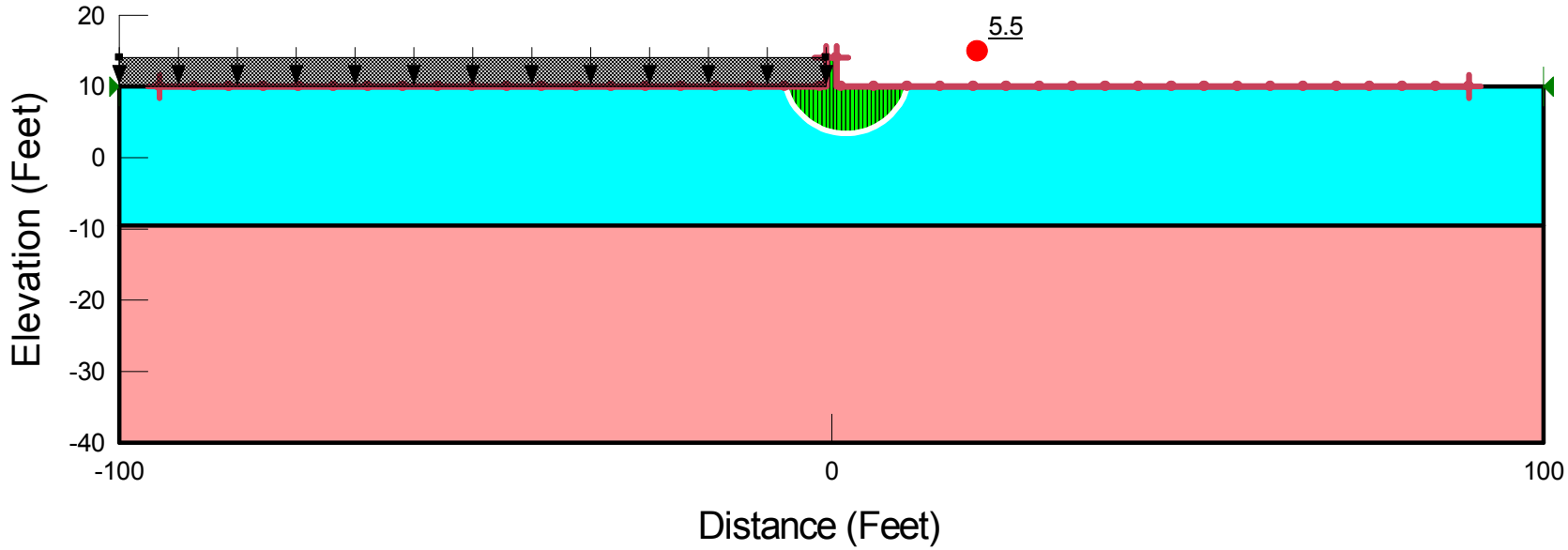




Steady-State Seepage for 4 ft High T-Wall (Full Flood at El. 14 ft)

Seepage Analysis of T-Wall Segment 6 Passaic River Tidal GRR Lower Passaic River, New Jersey			
AECOM Clifton, NJ			
Prep'd by: VK	Date: 8/13/2018	Proj. No.:	60442748
Ck'd by: AH	Date: 8/16/2018	Sheet No.	C.6

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	Concrete	Mohr-Coulomb	150	50,000	0	0
	Sand/Silt (Fill)	Mohr-Coulomb	100	0	29	0
	Sandy/Silty Clay (Undrained)	Mohr-Coulomb	125	2,500	0	0



Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Calculations\Floodwall\T-Wall\Slope Stability Analysis\

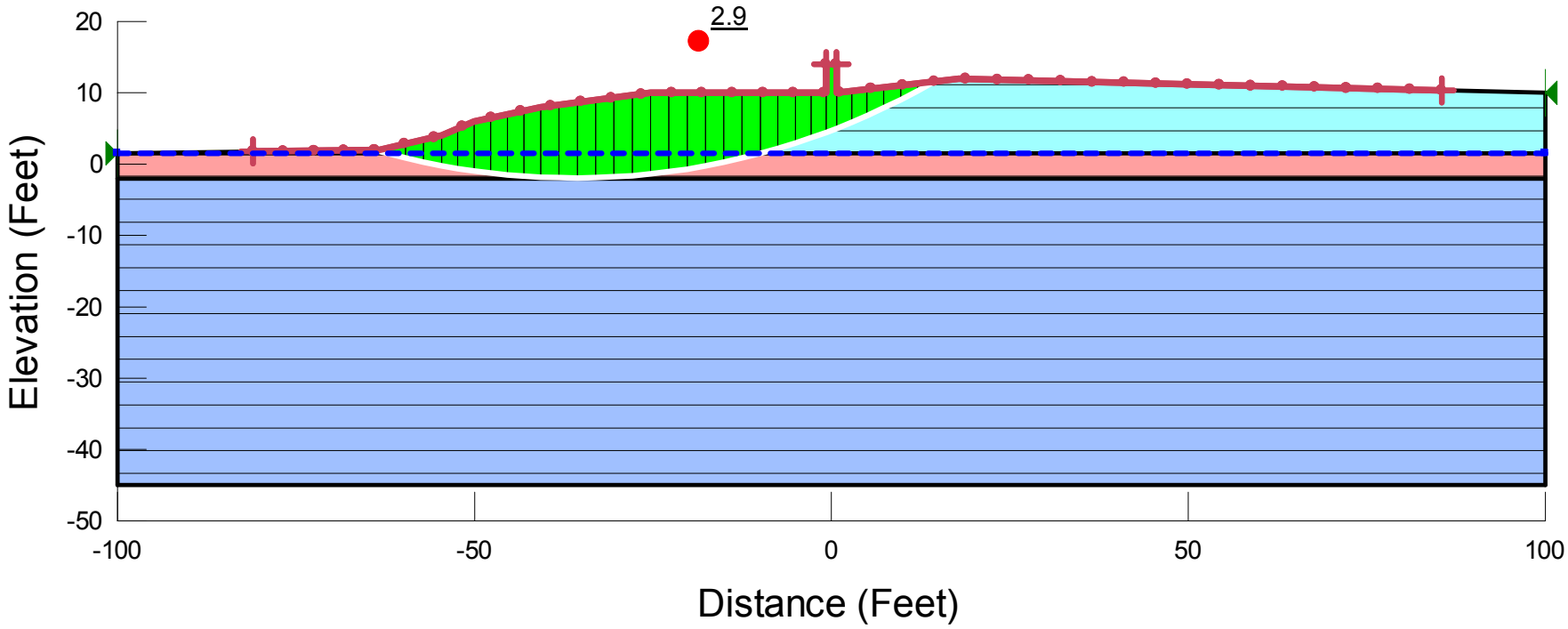
Full Flood as a Surcharge Load (4 ft High Floodwall)

Sliding Stability Analysis of T-Wall
Segments 4&5
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
Clifton, NJ

Prep'd by: VK	Date:	8/22/2018	Proj. No.:	60442748
Ck'd by: AH	Date:	8/27/2018	Sheet No.	C.7

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
	Concrete	Mohr-Coulomb	150	50,000	0	0	1
	Sand	Mohr-Coulomb	120	0	32	0	1
	Sand (Fill)	Mohr-Coulomb	120	0	29	0	1
	Silt and Clay (Undrained)	Mohr-Coulomb	90	250	0	0	1



Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Calculations\Floodwall\T-Wall\Slope Stability Analysis\

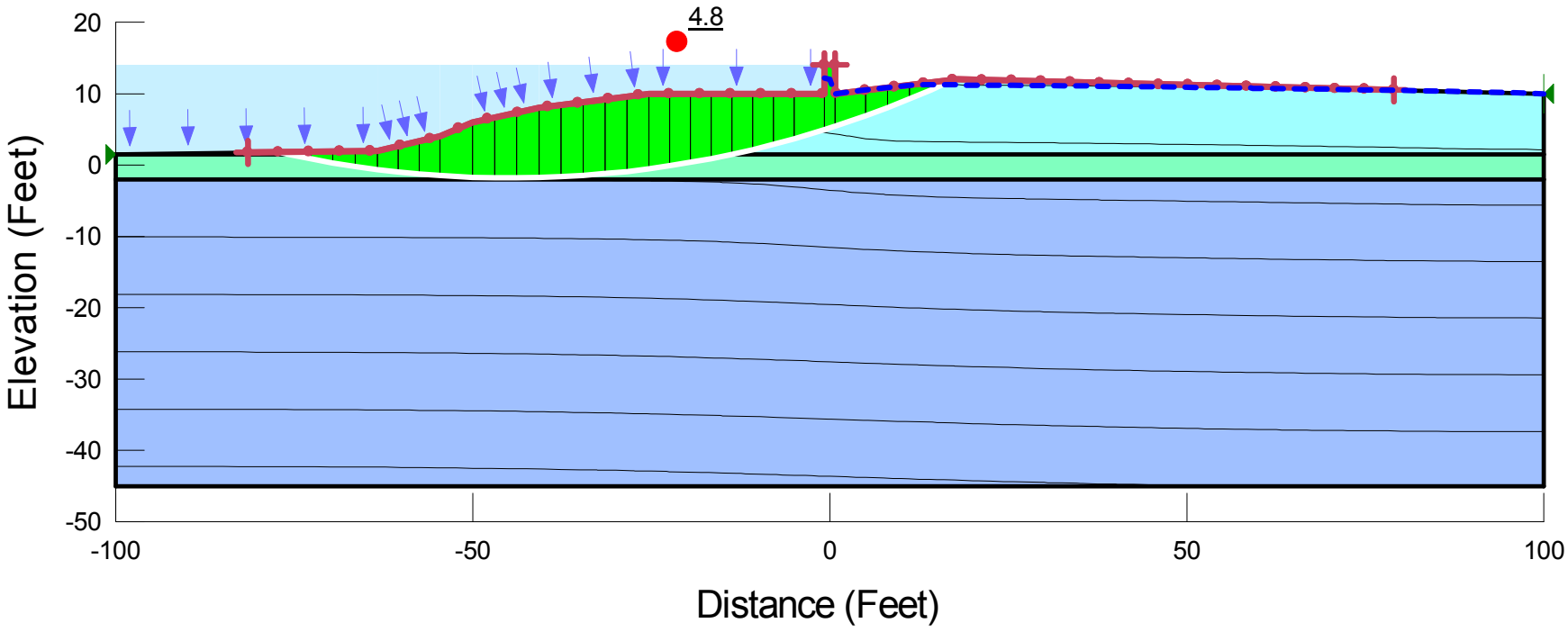
Critical Sliding Surface and Stability FOS for End of Construction (T-Wall)

Slope Stability of T-Wall
Segment 8
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
Clifton, NJ

Prep'd by: VK		Date:	8/24/2018	Proj. No.:	60442748
Ck'd by: AH		Date:	8/27/2018	Sheet No.	C.8

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)
	Concrete	Mohr-Coulomb	150	50,000	0	0
	Sand	Mohr-Coulomb	120	0	32	0
	Sand (Fill)	Mohr-Coulomb	120	0	29	0
	Silt and Clay (Drained)	Mohr-Coulomb	90	50	15	0



Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Calculations\Floodwall\T-Wall\Slope Stability Analysis\

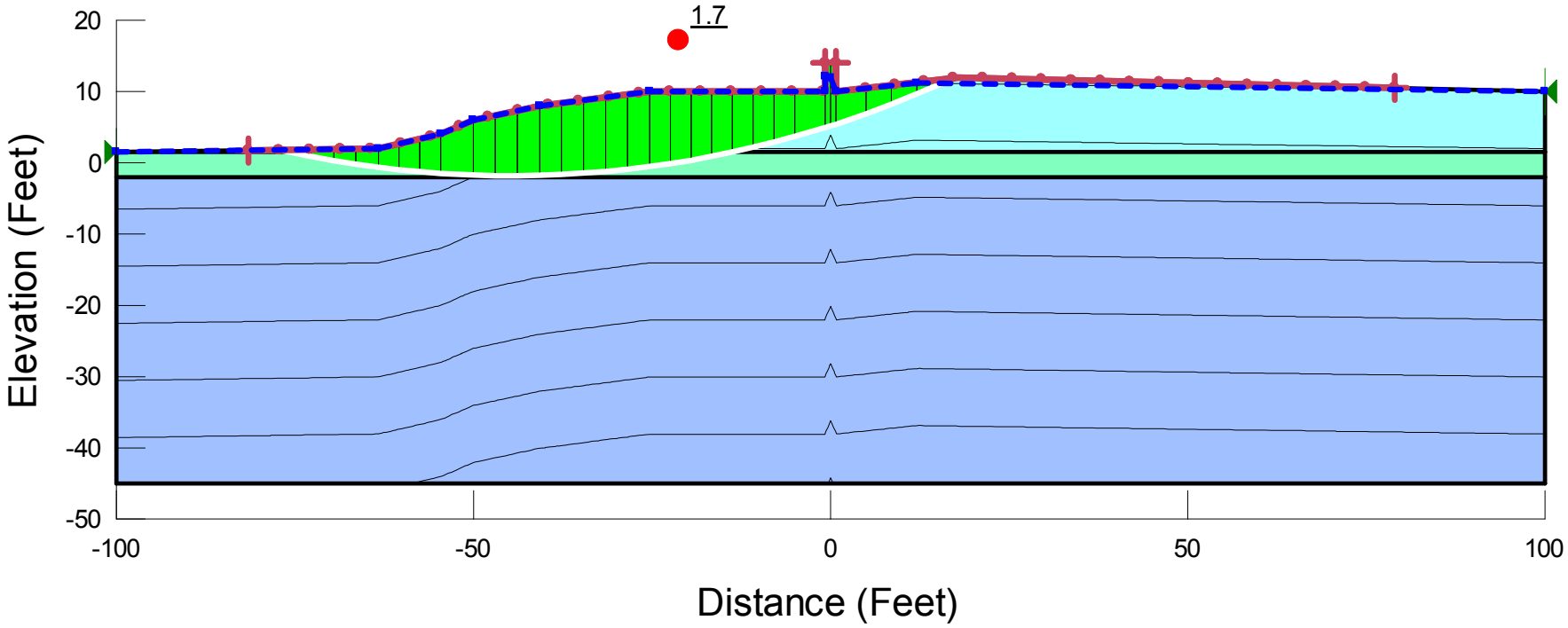
Critical Sliding Surface and Stability FOS for Steady State with Full Flood (T-Wall)

Slope Stability of T-Wall
Segment 8
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
Clifton, NJ

Prep'd by: VK		Date:	8/24/2018	Proj. No.:	60442748
Ck'd by: AH		Date:	8/27/2018	Sheet No.	C.9

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
	Concrete	Mohr-Coulomb	150	50,000	0	0	1
	Sand	Mohr-Coulomb	120	0	32	0	1
	Sand (Fill)	Mohr-Coulomb	120	0	29	0	1
	Silt and Clay (Drained)	Mohr-Coulomb	90	50	15	0	1



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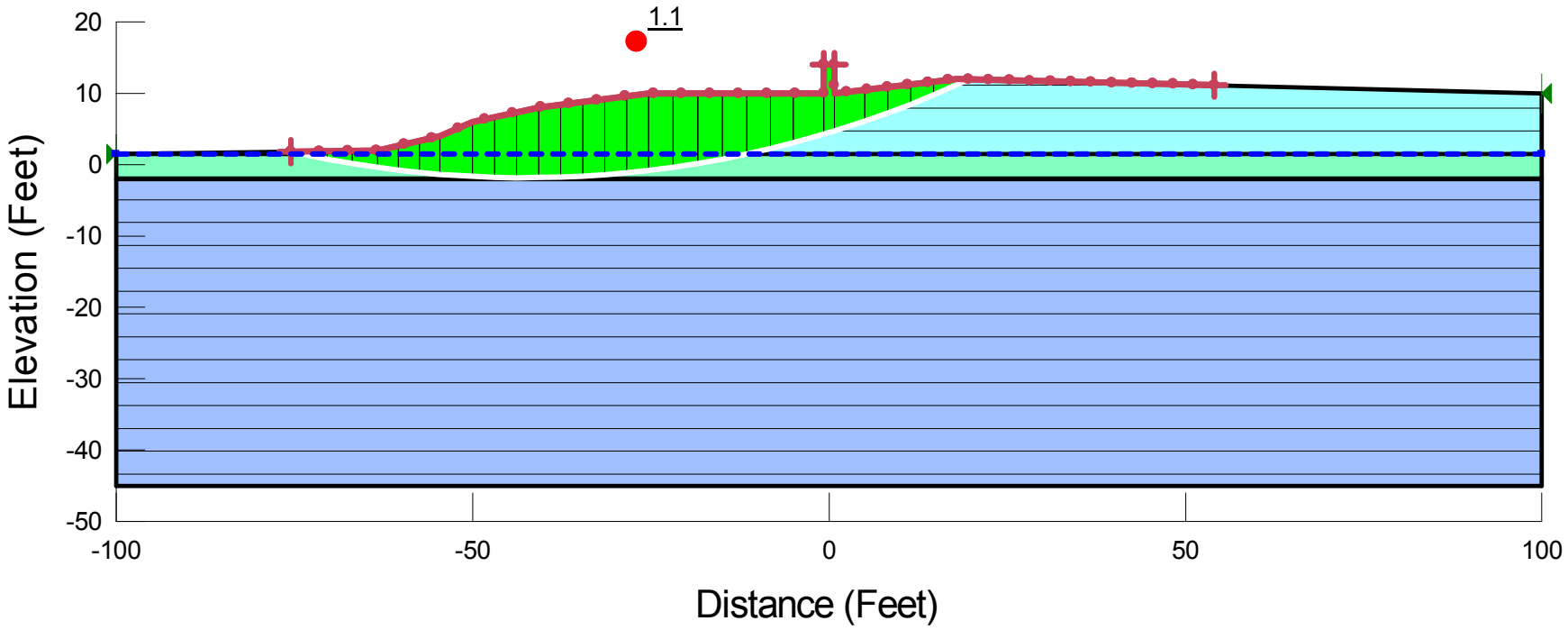
Critical Sliding Surface and Stability FOS for Rapid Drawdown from the Full Flood (T-Wall)

Slope Stability of T-Wall
Segment 8
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
Clifton, NJ

Prep'd by: VK		Date:	8/24/2018	Proj. No.:	60442748
Ck'd by: AH		Date:	8/27/2018	Sheet No.	C.10

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
	Concrete	Mohr-Coulomb	150	50,000	0	0	1
	Sand	Mohr-Coulomb	120	0	32	0	1
	Sand (Fill)	Mohr-Coulomb	120	0	29	0	1
	Silt and Clay (Drained)	Mohr-Coulomb	90	50	15	0	1



Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Calculations\Floodwall\T-Wall\Slope Stability Analysis\

Critical Sliding Surface and Stability FOS for Seismic Condition at Upstream (T-Wall)

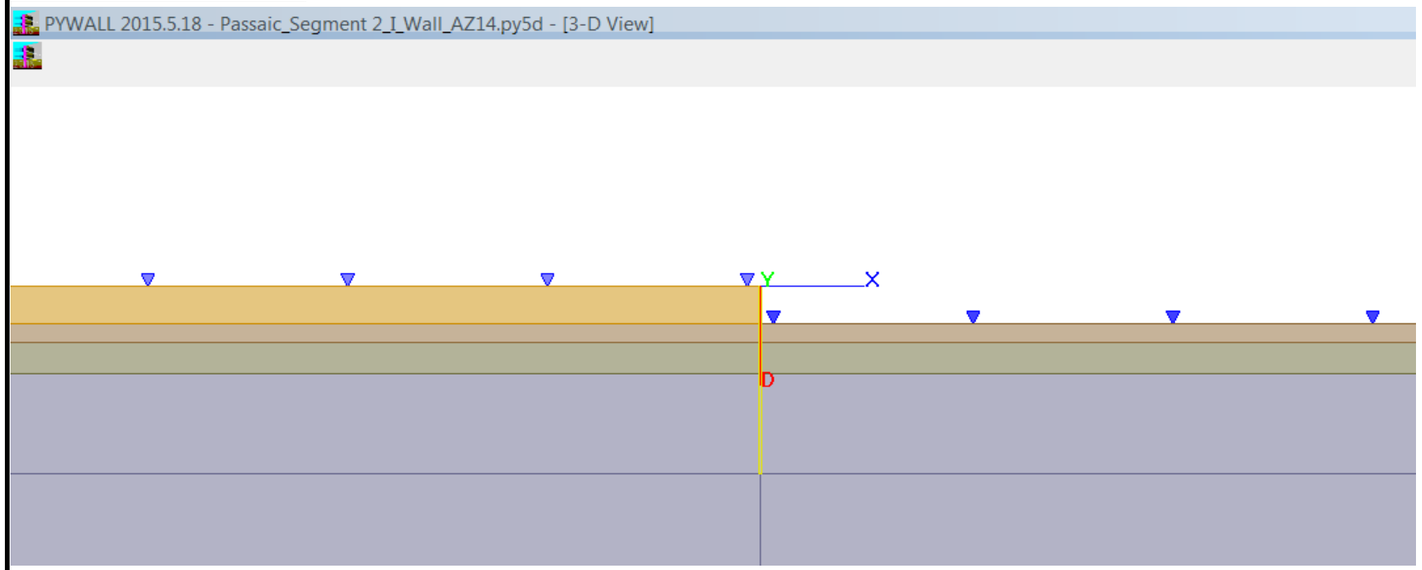
Slope Stability of T-Wall
Segment 8
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
Clifton, NJ

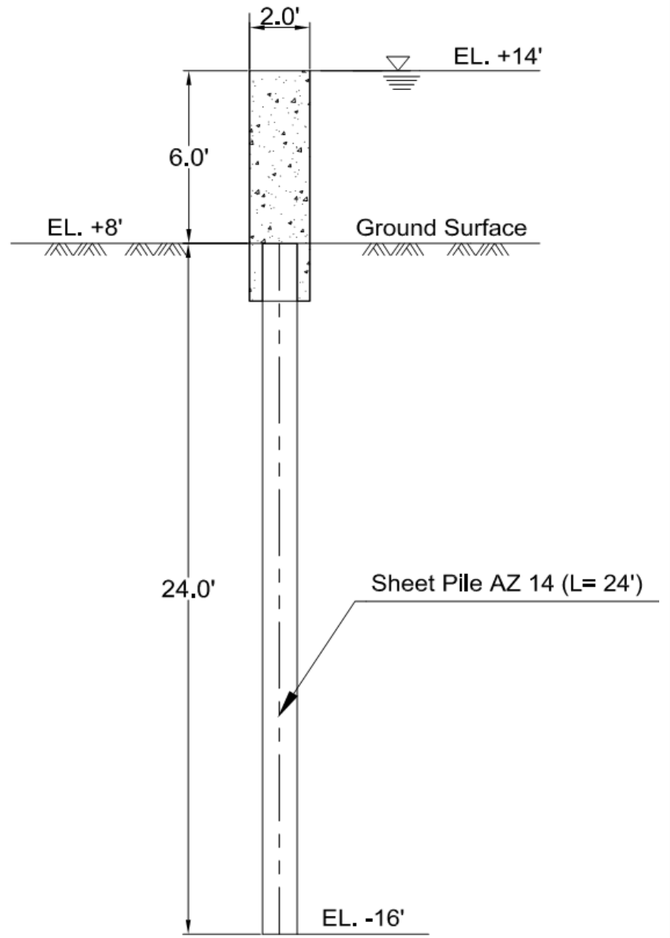
Prep'd by: VK		Date:	8/24/2018	Proj. No.:	60442748
Ck'd by: AH		Date:	8/27/2018	Sheet No.	C.11

ATTACHMENT D – LATERAL LOAD ANALYSIS FOR FLOOD WALL

PYWALL (Ensoft, Inc.) Model:

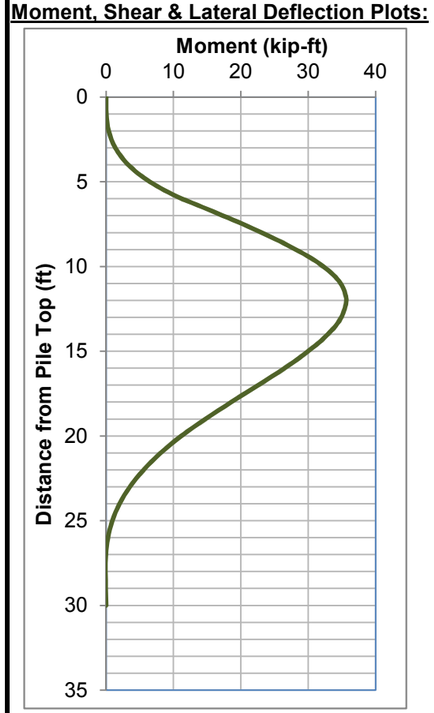


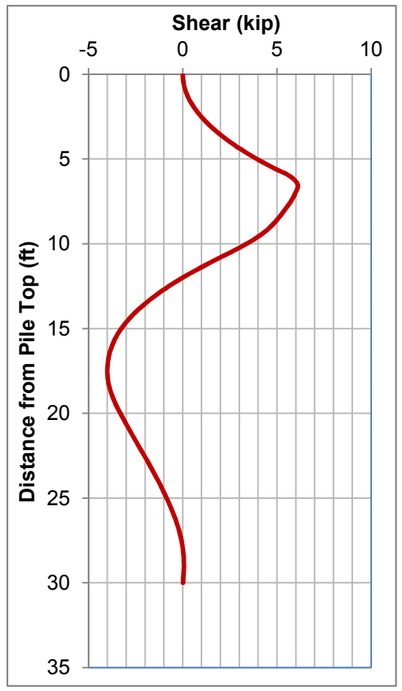
Schematic:

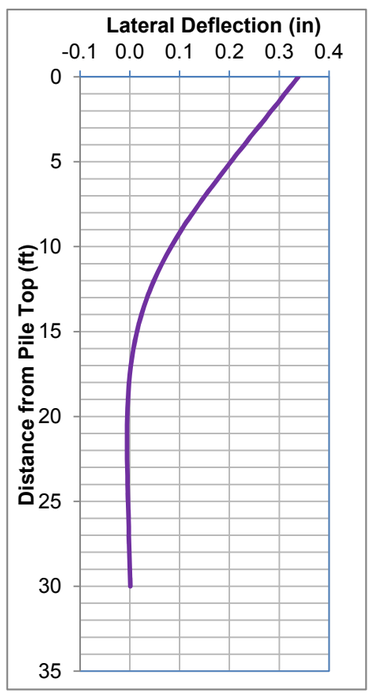


I-Wall Cross-Section for 6' Flood Height

Moment, Shear & Lateral Deflection Plots:







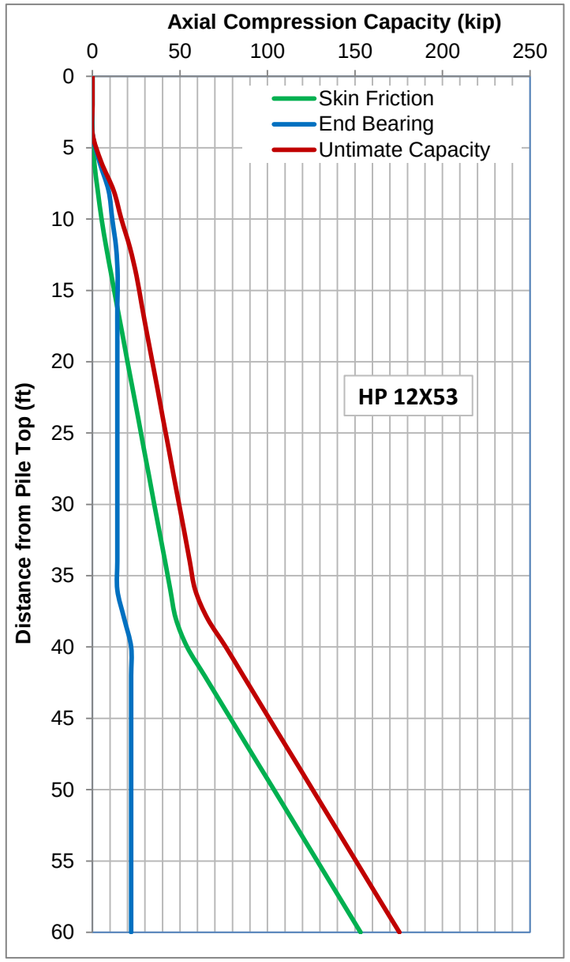
Analysis of I-Wall for 6 ft Flood Height
Segment 2
Passaic River Tidal GRR
Newark, New Jersey

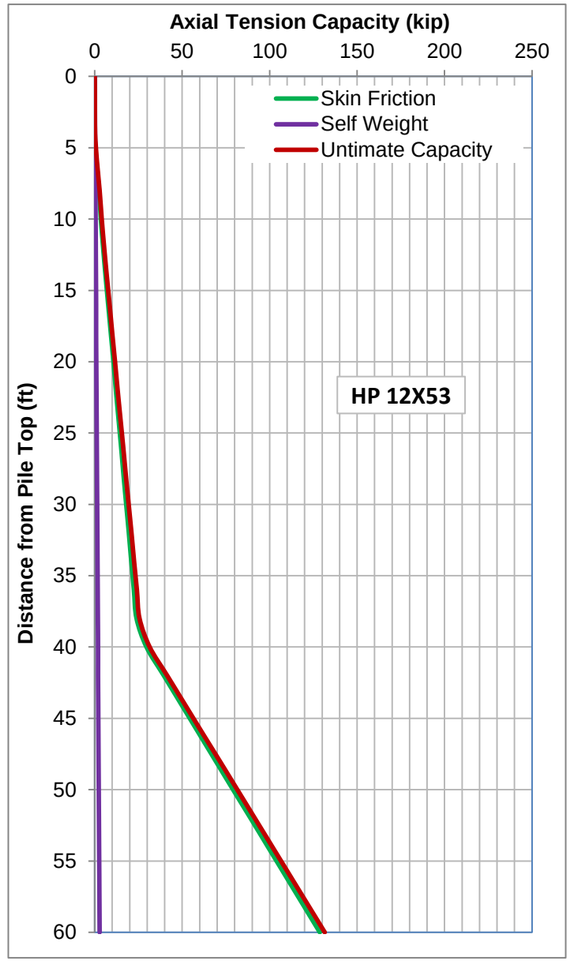
AECOM
Clifton, NJ

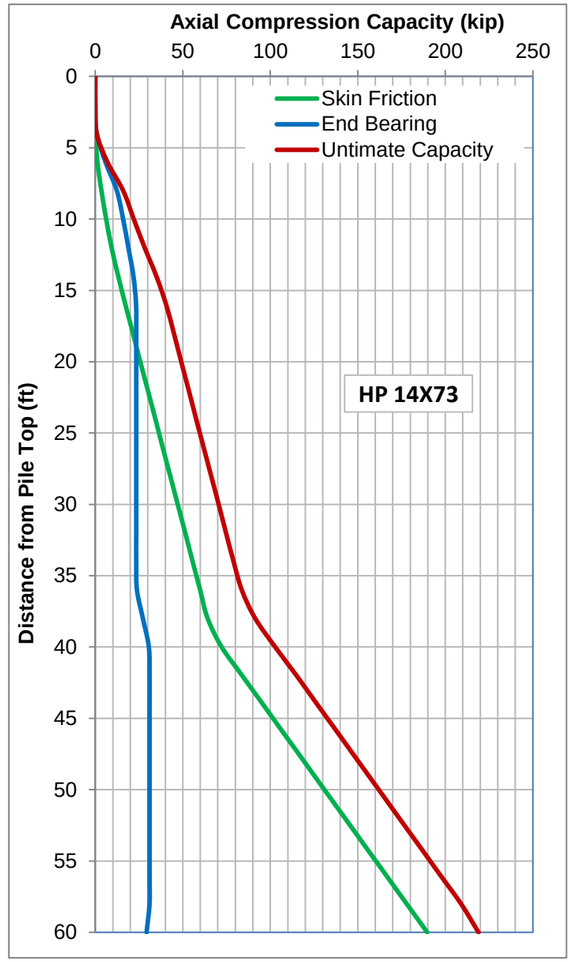
Prep'd by: VK	Date: 8/22/2018	Proj. No.: 60442748
Ck'd by: AH	Date: 8/22/2018	Sheet No D

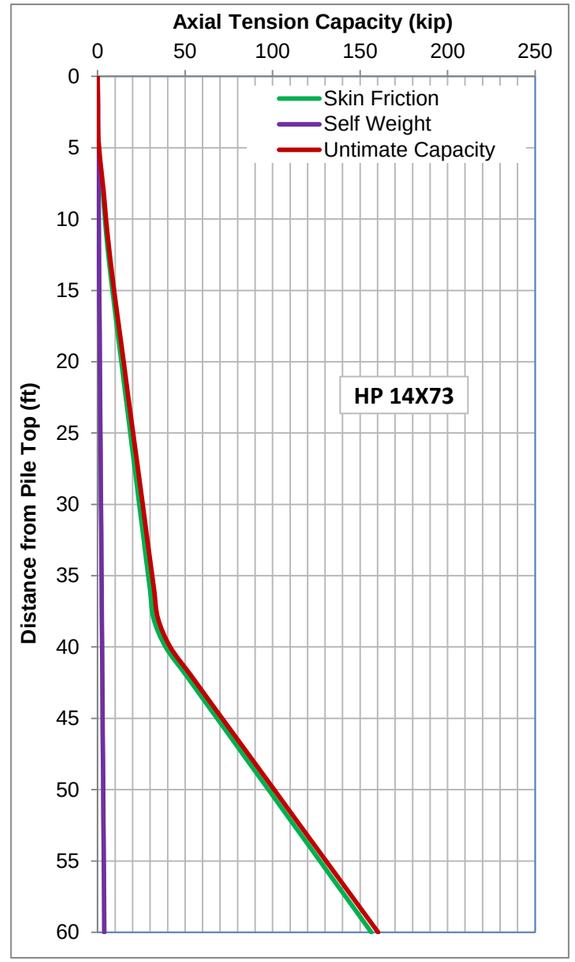
ATTACHMENT E – PILE CAPACITY ANALYSIS

Plots of Ultimate Axial Capacities from APILE (Ensoft, Inc.) Analyses:









Summary of Axial Capacities:

Pile Length	Ultimate Compression Capacity		Ultimate Tension Capacity		Minimum Factor of Safety	Allowable Compression Capacity		Allowable Tension Capacity	
(ft)	(kip)	(kip)	(kip)	(kip)		(kip)	(kip)	(kip)	(kip)
	HP 12X53	HP 14X73	HP 12X53	HP 14X73		HP 12X53	HP 14X73	HP 12X53	HP 14X73
30	50	71	19	26	2 (assuimg load test will be performed)	25	35	10	13
35	56	79	23	30		28	40	11	15
40	76	103	31	42		38	51	16	21
45	96	127	51	65		48	63	26	33
50	126	162	82	101		63	81	41	50
55	146	186	102	125		73	93	51	62
60	175	219	132	160		88	110	66	80

Reference:

United States Army Corps of Engineers (1991). “Engineering and Design: Design of Pile Foundations”, EM 1110-2-2906, USACE, Washington, DC.

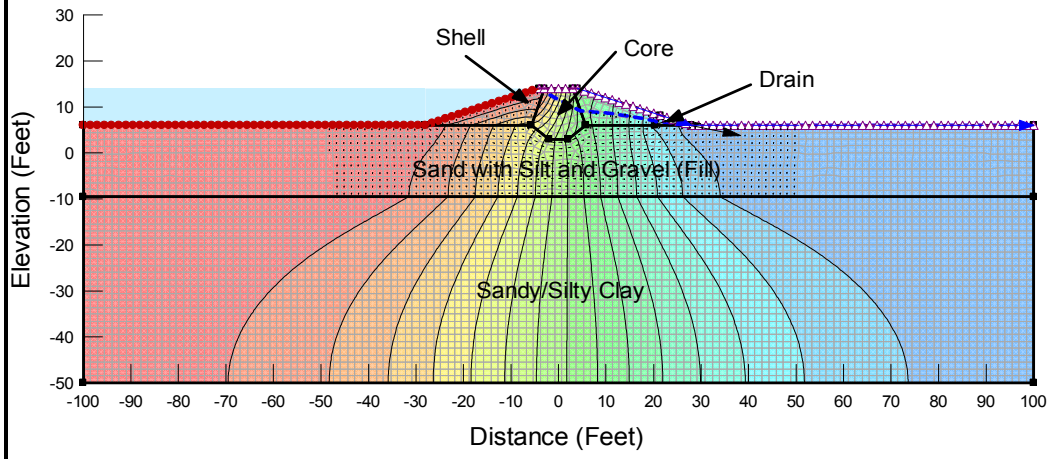
Axial Capacity Analyses of Driven Steel Piles for
Segment 2
Passaic River Tidal
Newark, New Jersey

AECOM
Clifton, NJ

Calc'd by: VK	Date: 8/20/2018	Proj. No.:	60442748
Ck'd by: AH	Date: 8/21/2018	Sheet No.:	E

ATTACHMENT F – LEVEE SEEPAGE AND STABILITY ANALYSIS

Color	Name	Model	K-Function	Vol. WC. Function	Sat Kx (ft/sec)	Volumetric Water Content (ft³/ft³)
	Core (Undrained)	Saturated / Unsaturated	Core	VWC-Core		
	Drain	Saturated / Unsaturated	Drain	VWC-Gravelly Sand		
	Sand with Silt and Gravel (Fill)	Saturated / Unsaturated	Sand with Silt and Gravel (Fill)	VWC-Sand with Silt and Gravel		
	Sandy/Silty Clay (Drained)	Saturated Only			3.28e-008	0
	Shell	Saturated / Unsaturated	shell	VWC-Shell		



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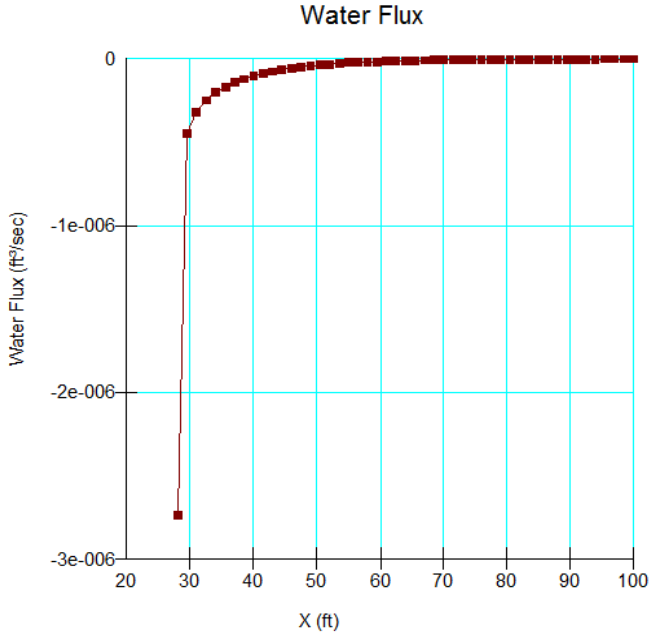
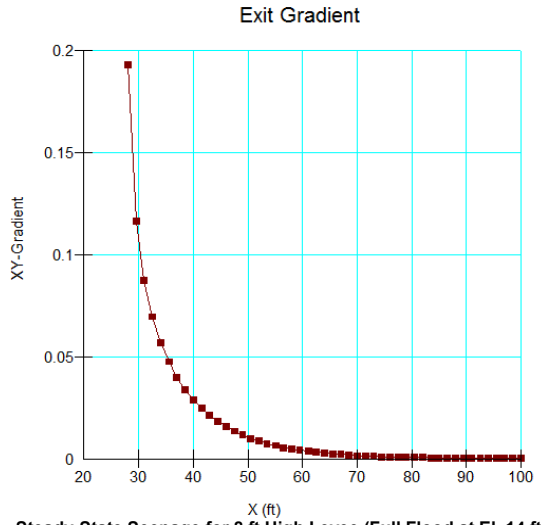




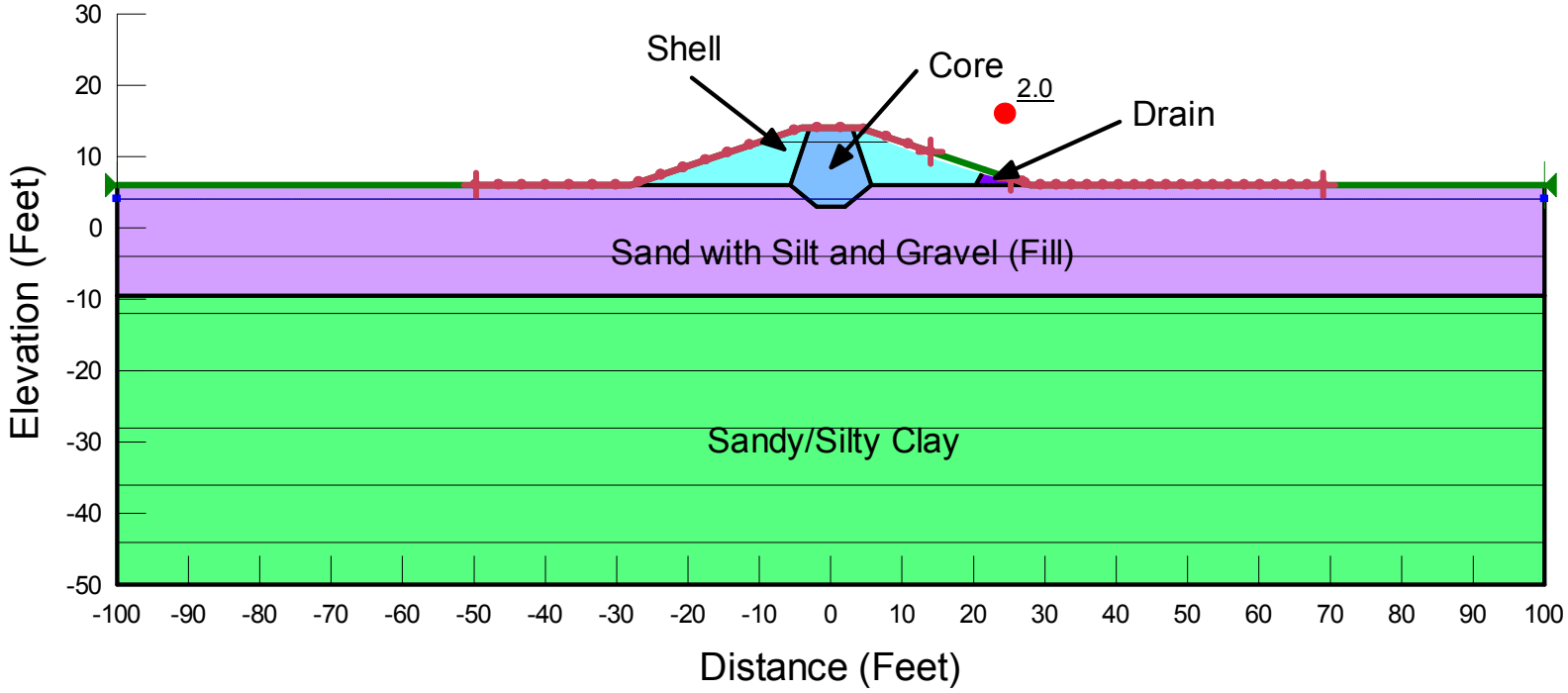
Steady-State Seepage for 8 ft High Levee (Full Flood at El. 14 ft)

Seepage Analysis of Levee
Segment 3
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
Clifton, NJ

Prep'd by: VK	Date:	8/17/2018	Proj. No.:	60442748
Ck'd by: AH	Date:	8/21/2018	Sheet No.:	F.1

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Core (Undrained)	Mohr-Coulomb	120	1,000	0
	Drain	Mohr-Coulomb	120	0	35
	Sand with Silt and Gravel (Fill)	Mohr-Coulomb	100	0	29
	Sandy/Silty Clay (Undrained)	Mohr-Coulomb	125	2,500	0
	Shell	Mohr-Coulomb	120	0	33



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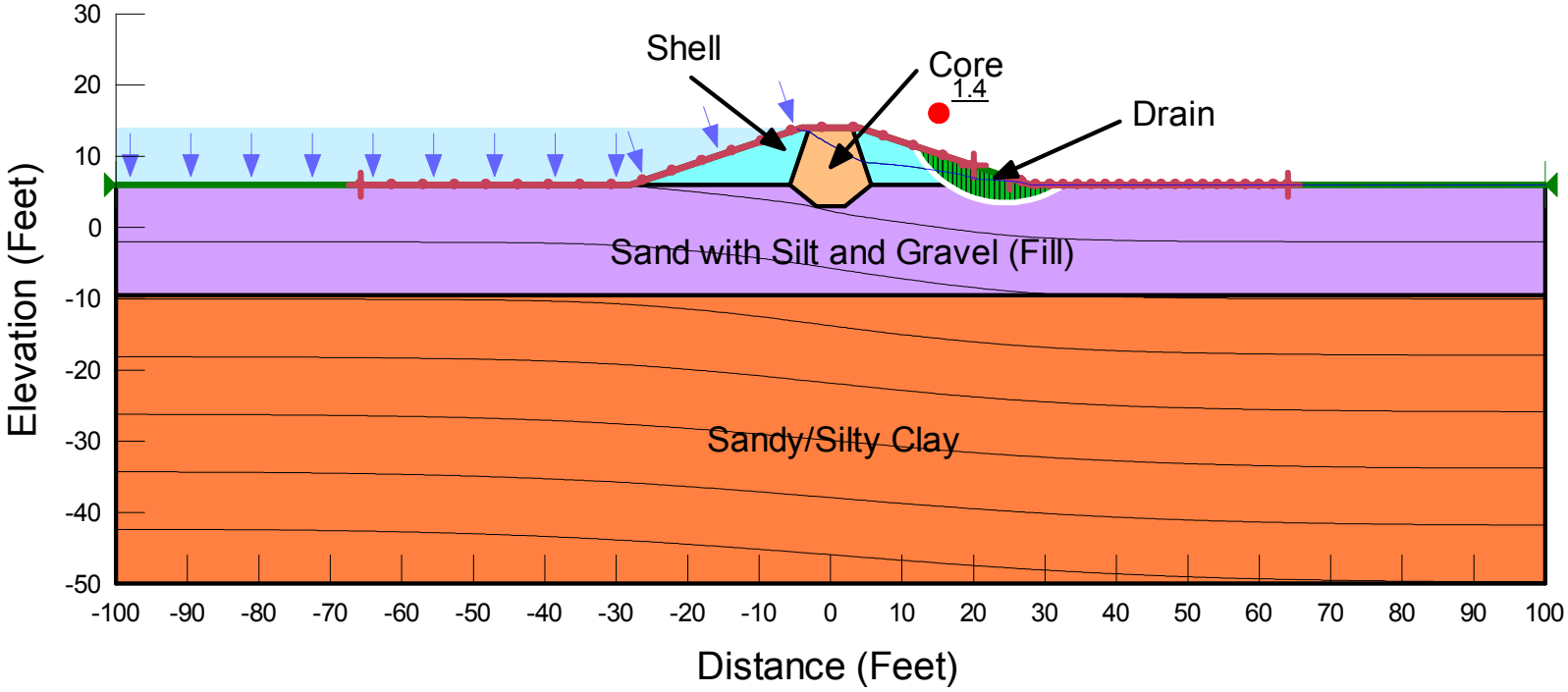
Critical Sliding Surface and Stability FOS for End of Construction (8 ft High Levee)

Slope Stability of Levee
Segment 3
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
Clifton, NJ

Prep'd by: VK		Date:	8/17/2018	Proj. No.:	60442748
Ck'd by: AH		Date:	8/21/2018	Sheet No.	F.2

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Core (Drained)	Mohr-Coulomb	120	0	30
	Drain	Mohr-Coulomb	120	0	35
	Sand with Silt and Gravel (Fill)	Mohr-Coulomb	100	0	29
	Sandy/Silty Clay (Drained)	Mohr-Coulomb	125	200	22
	Shell	Mohr-Coulomb	120	0	33



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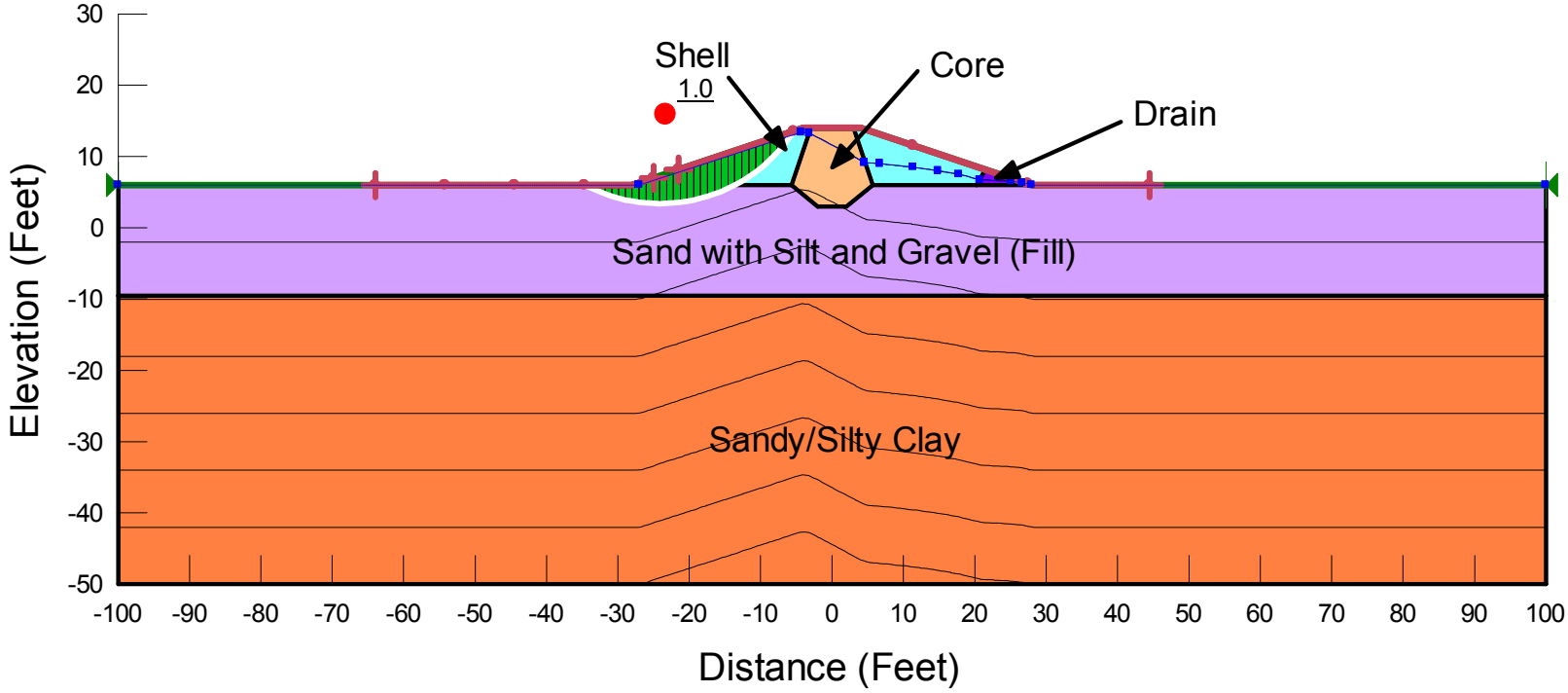
Critical Sliding Surface and Stability FOS for Steady State with Full Flood (8 ft High Levee)

Slope Stability of Levee
Segment 3
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
Clifton, NJ

Prep'd by: VK	Date:	8/17/2018	Proj. No.:	60442748	
Ck'd by: AH	Date:	8/21/2018	Sheet No.:	F.3	

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Core (Drained)	Mohr-Coulomb	120	0	30
	Drain	Mohr-Coulomb	120	0	35
	Sand with Silt and Gravel (Fill)	Mohr-Coulomb	100	0	29
	Sandy/Silty Clay (Drained)	Mohr-Coulomb	125	200	22
	Shell	Mohr-Coulomb	120	0	33



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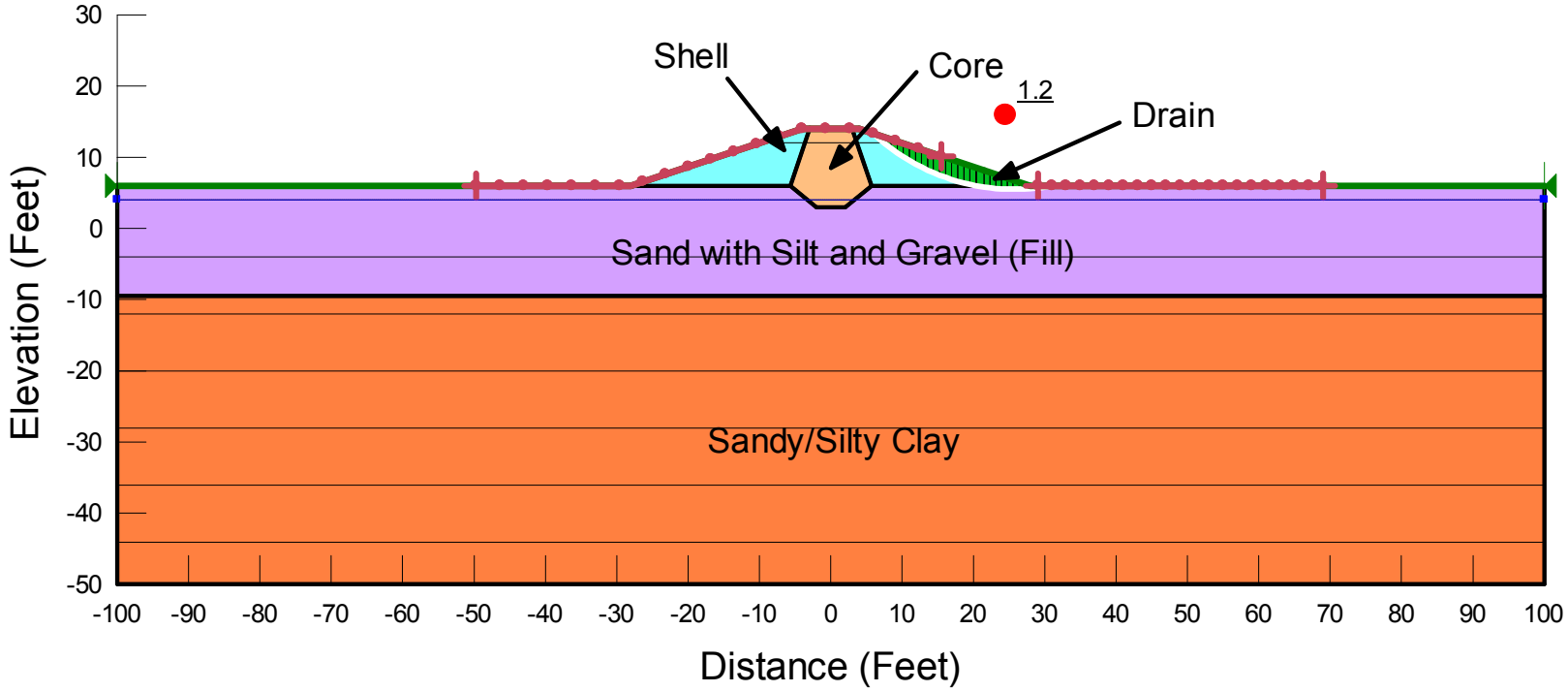
Critical Sliding Surface and Stability FOS for Rapid Drawdown from the Full Flood (8 ft High Levee)

Slope Stability of Levee
Segment 3
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
Clifton, NJ

Prep'd by: VK	Date:	8/17/2018	Proj. No.:	60442748	
Ck'd by: AH	Date:	8/21/2018	Sheet No.:	F.4	

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Core (Drained)	Mohr-Coulomb	120	0	30
	Drain	Mohr-Coulomb	120	0	35
	Sand with Silt and Gravel (Fill)	Mohr-Coulomb	100	0	29
	Sandy/Silty Clay (Drained)	Mohr-Coulomb	125	200	22
	Shell	Mohr-Coulomb	120	0	33



Directory: Q:\Geotechnical\60442748(Passaic-River)\Phase Two - 2018\Calculations\Levee\Slope Stability\File Name: Passaic_River_Tidal_GRR_Segment 3 Levee.gsz

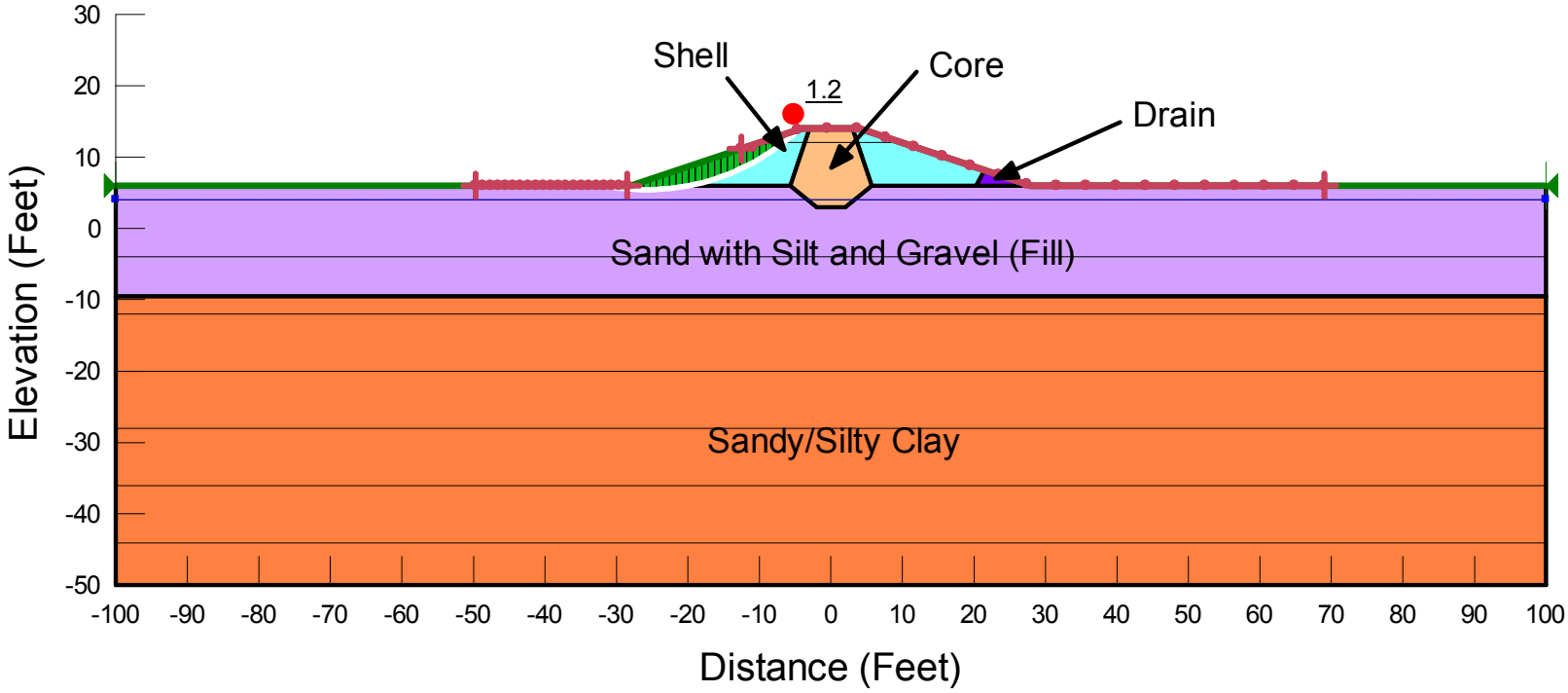
Critical Sliding Surface and Stability FOS for Seismic Condition at Downstream (8 ft High Levee)

Slope Stability of Levee
Segment 3
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
Clifton, NJ

Prep'd by: VK	Date:	8/17/2018	Proj. No.:	60442748	
Ck'd by: AH	Date:	8/21/2018	Sheet No.:	F.5	

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Core (Drained)	Mohr-Coulomb	120	0	30
	Drain	Mohr-Coulomb	120	0	35
	Sand with Silt and Gravel (Fill)	Mohr-Coulomb	100	0	29
	Sandy/Silty Clay (Drained)	Mohr-Coulomb	125	200	22
	Shell	Mohr-Coulomb	120	0	33



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Critical Sliding Surface and Stability FOS for Seismic Condition at Upstream (8 ft High Levee)

Slope Stability of Levee
Segment 3
Passaic River Tidal GRR
Lower Passaic River, New Jersey

AECOM
Clifton, NJ

Prep'd by: VK	Date:	8/17/2018	Proj. No.:	60442748
Ck'd by: AH	Date:	8/21/2018	Sheet No.:	F.6

ATTACHMENT G – LEVEE CONSOLIDATION SETTLEMENT ANALYSIS

SETTLEMENT ANALYSIS OF LEVEE
PASSAIC RIVER TIDAL GRR
LOWER PASSAIC RIVER, NEW JERSEY

CALCULATED BY: VK **DATE:** 8/16/2018 **CHECKED BY:** AH **DATE:** 8/21/2018

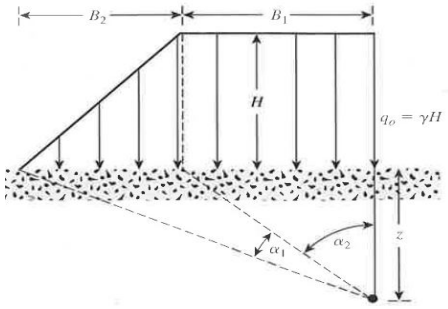
Soil Parameters:

Layer No.	Soil Description	Total Unit Weight (pcf)	Layer Thickness (ft)	Bottom Depth of Layer (ft)	Initial Void Ratio, e_0	Compression Index, C_c
1a	Sand/Silt (Fill)	100	2	2		
1b	Sand/Silt (Fill)	100	13.5	15.5		
2	Sandy/Silty Clay	125	30	45.5	0.94	0.18

Elevations:

Embankment top elavation: + 14 ft
Embankment bottom elavation: + 6 ft
Existing ground elavation: + 6 ft
Groundwater table elavation: + 4 ft

Increase in Vertical Stress in Soil due to Embankment Load:



$$\Delta\sigma_z = \frac{q_o}{\pi}\left[\left(\frac{B_1 + B_2}{B_2}\right)(\alpha_1 + \alpha_2) - \frac{B_1}{B_2}(\alpha_2)\right]$$

$q_o = \gamma H$
γ = unit weight of the embankment soil
H = height of the embankment

$$\alpha_1\,(\text{radians}) = \tan^{-1}\left(\frac{B_1 + B_2}{z}\right) - \tan^{-1}\left(\frac{B_1}{z}\right)$$

$$\alpha_2 = \tan^{-1}\left(\frac{B_1}{z}\right)$$

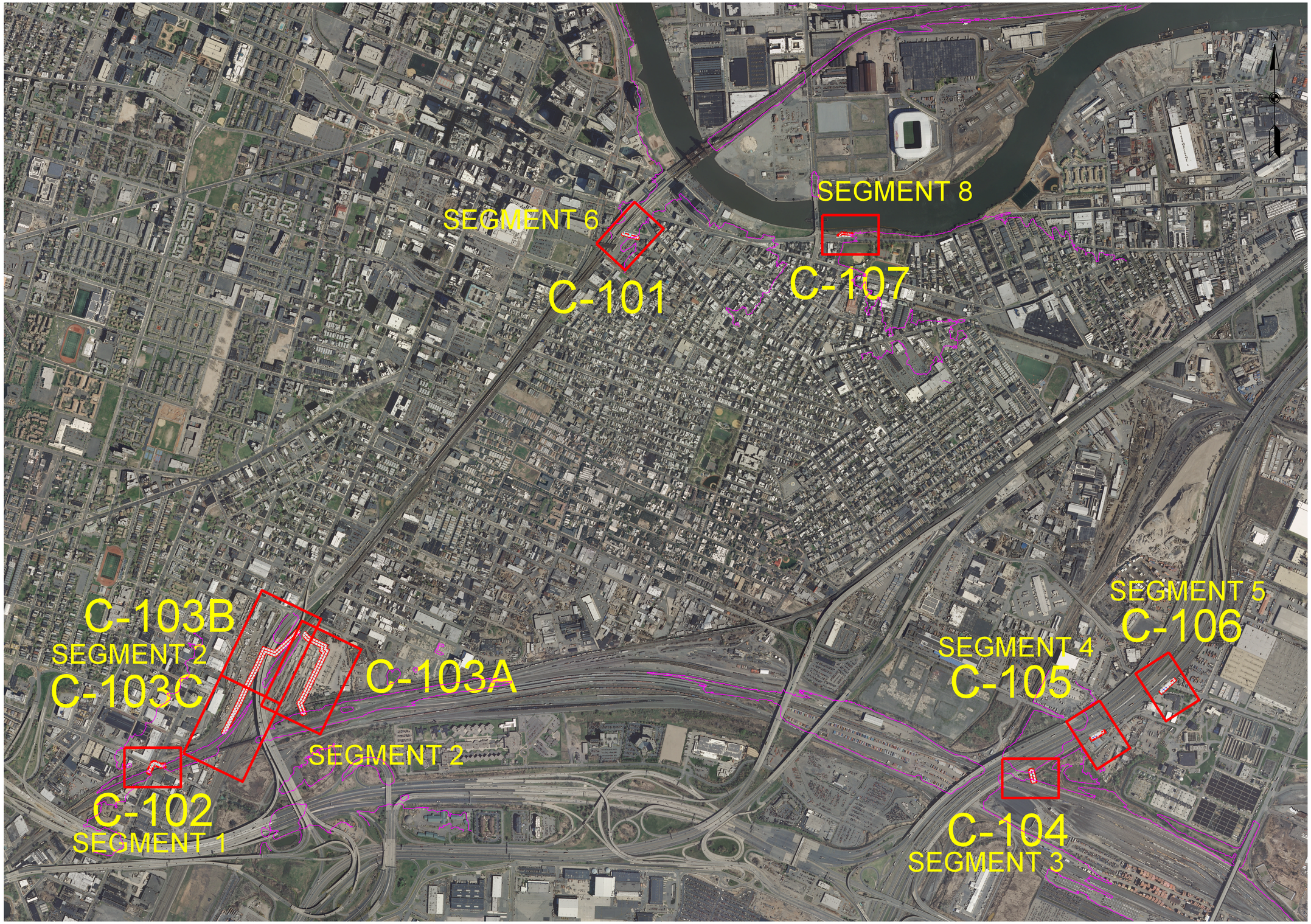
H =	8 ft	γ =	120 pcf
B_1 =	4 ft	q_0 =	960 psf
B_2 =	24 ft		

Settlement Calculation:

Sub-Layer No.	Thickness (ft)	Mid Depth of Sub-Layer (ft)	Initial Overburden Pressure, σ'_0 (psf)	α_1 (rad.)	α_2 (rad.)	Increase in Overburden Pressure, $\Delta\sigma'_z$ (psf)	$\sigma'_0 + \Delta\sigma'_z$ (psf)	C_c	Settlement (ft)
1	1	16.0	739	0.8	0.2	725	1464	0.18	0.028
2	1	17.0	802	0.8	0.2	707	1509	0.18	0.025
3	1	18.0	864	0.8	0.2	690	1554	0.18	0.024
4	1	19.0	927	0.8	0.2	674	1600	0.18	0.022
5	1	20.0	989	0.8	0.2	658	1647	0.18	0.021
6	1	21.0	1052	0.7	0.2	642	1694	0.18	0.019
7	1	22.0	1115	0.7	0.2	627	1741	0.18	0.018
8	1	23.0	1177	0.7	0.2	612	1789	0.18	0.017
9	1	24.0	1240	0.7	0.2	598	1838	0.18	0.016
10	1	25.0	1302	0.7	0.2	584	1886	0.18	0.015
11	1	26.0	1365	0.7	0.2	571	1936	0.18	0.014
12	1	27.0	1428	0.7	0.1	558	1985	0.18	0.013
13	1	28.0	1490	0.6	0.1	546	2036	0.18	0.013
14	1	29.0	1553	0.6	0.1	534	2086	0.18	0.012
15	1	30.0	1615	0.6	0.1	522	2137	0.18	0.011
16	1	31.0	1678	0.6	0.1	511	2189	0.18	0.011
17	1	32.0	1741	0.6	0.1	500	2240	0.18	0.010
18	1	33.0	1803	0.6	0.1	489	2292	0.18	0.010
19	1	34.0	1866	0.6	0.1	479	2345	0.18	0.009
20	1	35.0	1928	0.6	0.1	470	2398	0.18	0.009
21	1	36.0	1991	0.6	0.1	460	2451	0.18	0.008
22	1	37.0	2054	0.5	0.1	451	2504	0.18	0.008
23	1	38.0	2116	0.5	0.1	442	2558	0.18	0.008
24	1	39.0	2179	0.5	0.1	434	2612	0.18	0.007
25	1	40.0	2241	0.5	0.1	425	2667	0.18	0.007
26	1	41.0	2304	0.5	0.1	417	2721	0.18	0.007
27	1	42.0	2367	0.5	0.1	410	2776	0.18	0.006
28	1	43.0	2429	0.5	0.1	402	2831	0.18	0.006
29	1	44.0	2492	0.5	0.1	395	2887	0.18	0.006
30	1	45.0	2554	0.5	0.1	388	2942	0.18	0.006

TOTAL PRIMARY CONSOLIDATION SETTLEMENT: **4.6 in**

MAY 2009 NYD BORDER



US Army Corps
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New York District

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DWN BY: OGM	CKD BY: JDR	SOLICITATION NO.:	
SUBMITTED BY:		CONTRACT NO.:	
FILE NAME: 002-PR_KP001-2018.dgn			
SIZE: ANSI D	PLOTTED BY: Onsy.Mowad	PLOT DATE: 3/19/2018 10:17:52 AM	

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PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ

KEY PLAN

SHEET REFERENCE NUMBER
KP001
SHEET 2 OF 11

Submitted by: Onsy.Mowad Plot Date: 3/19/2018 10:17:51 AM
FILE NAME : K:\CADD\60442748-PR\PLANS\2018-PLANS\002-PR_KP001-2018.dgn

MAY 2009 NYD BORDER

PROPOSED FLOODWALL

30 L.F. CLOSURE GATE

NEW JERSEY RAILROAD AVE.

38L.F. CLOSURE GATE

EDISON PLACE

RAYMOND PLAZA E

SEGMENT "6"

TEMPORARY CONSTRUCTION EASEMENT (15 FT.)

LIMIT OF CONTRACTORS WORKING AREA

EDGE OF FLOODWALL SLAB

0 30' 60'

SCALE:1"=30'

US Army Corps of Engineers
New York District

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FILE NAME: 003-PRC-101-2016.dgn
SPEC. SECT. NO.:
PLOTTED BY: Cindy.Howard
PLOT DATE: 6/20/2018 AM
DATE: MAY. 2016
SOLICITATION NO.:
CONTRACT NO.:

PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ

SEGMENT "6" PLAN

- -

SHEET REFERENCE NUMBER
C-101
SHEET 3 OF 11

Submitted by: Ons,Mosaad Plot Date: 6/19/2018 8:34:28 AM
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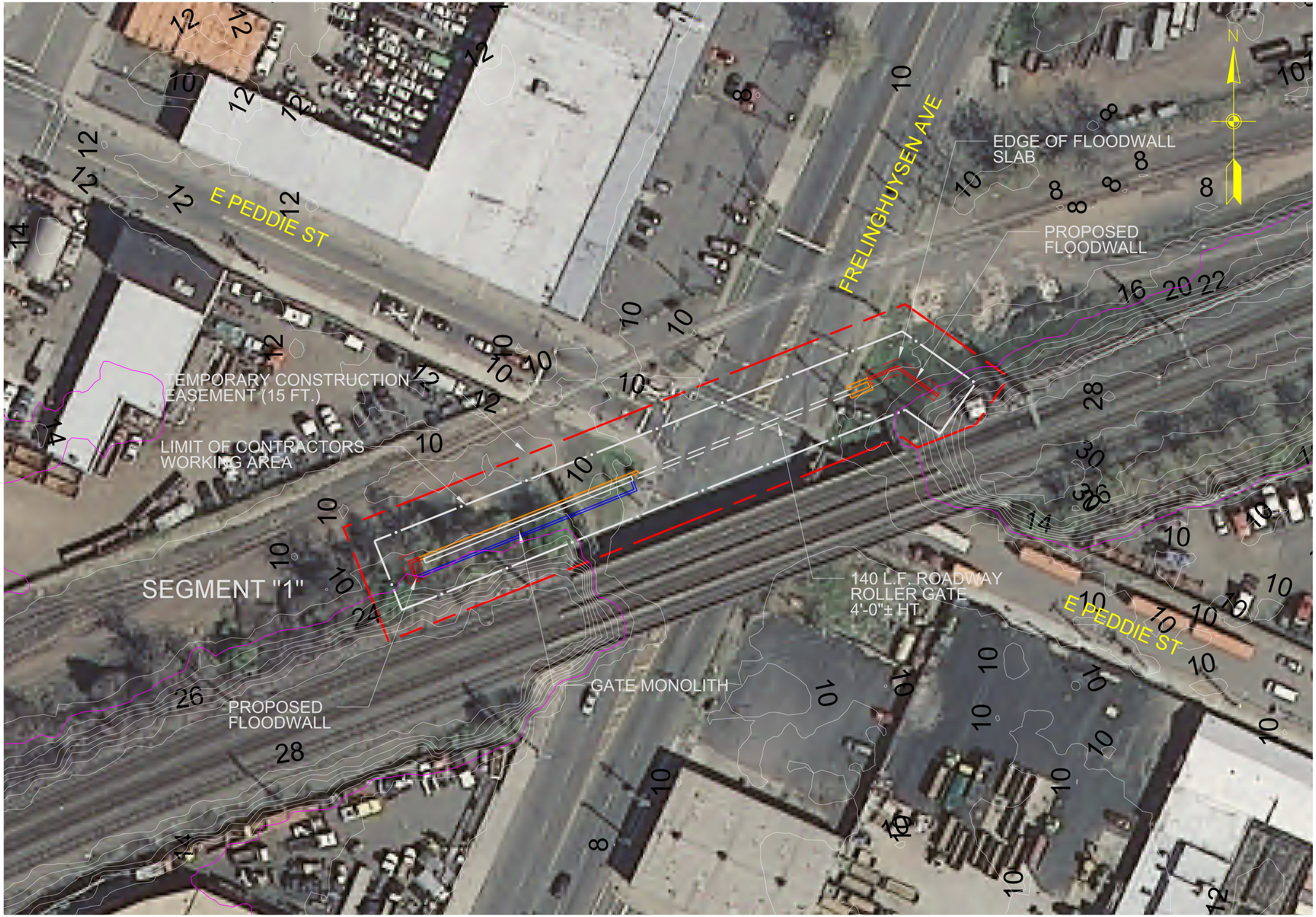
MAY 2009 NYD BORDER



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FILE NAME: 004-PRC-I102-2016.dgn			
ANS'D	PLOTTED BY: Cindy.Mcwood	PLOT DATE: 2/1/2018 PM	

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PASSAIC RIVER TIDAL
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LOWER PASSAIC RIVER, NJ

SEGMENT "1" PLAN

- -

SHEET REFERENCE NUMBER
C-102
SHEET 4 OF 11

Submitted by: Cindy.Mcwood Plot Date: 12/28/2018 3:17:08 PM
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MAY 2009 NYD BORDER

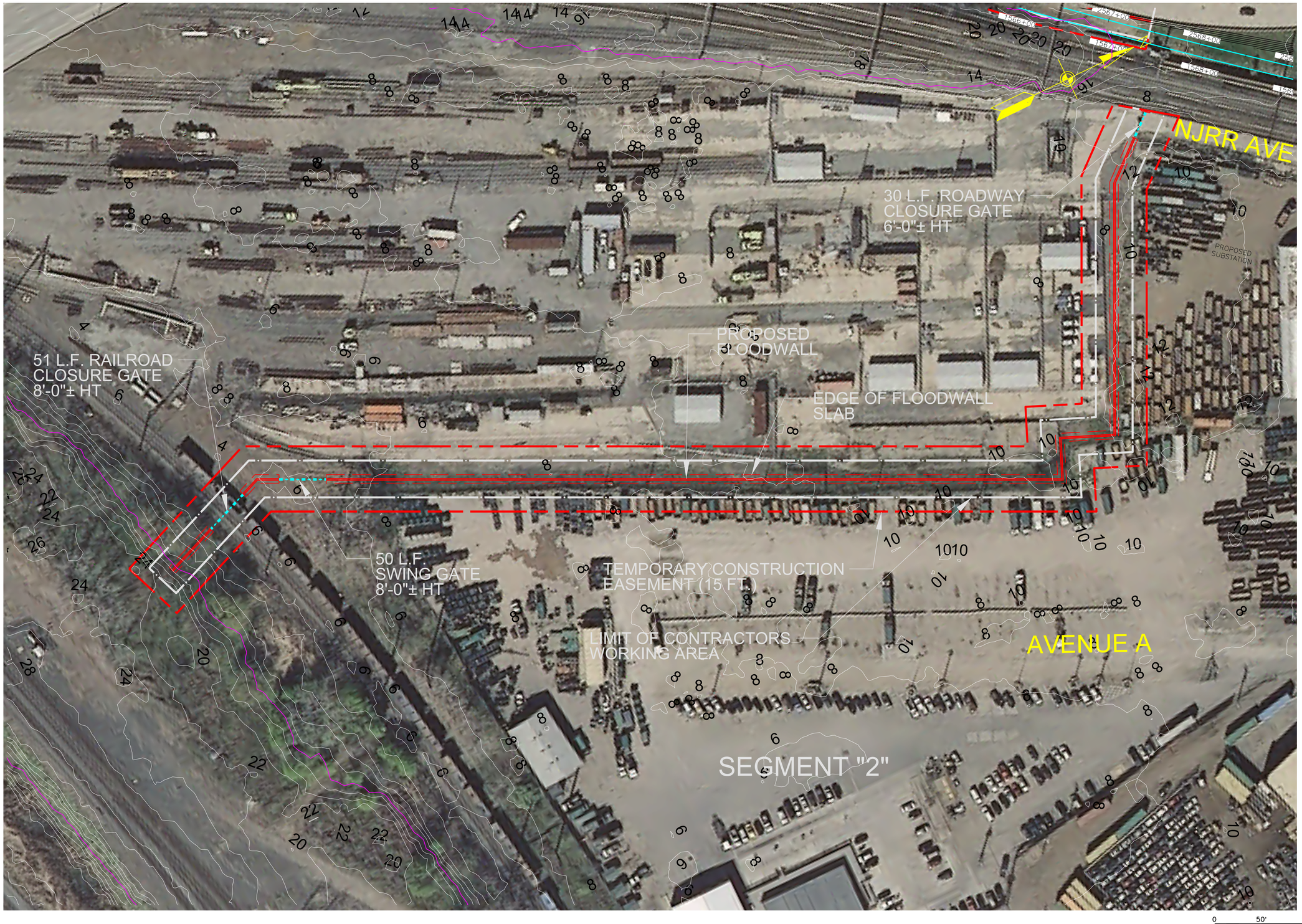
1 2 3 4 5

D

C

B

A



0 50' 100'

SCALE:1"=50'

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CONTRACT NO.:
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PASSAIC RIVER TIDAL
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LOWER PASSAIC RIVER, NJ

SEGMENT "2" PLAN

SHEET
REFERENCE
NUMBER
C-103A
SHEET 5A OF 11

Submitted by: Onsy.Mowad Plot Date: 10/2/2018 12:22:44 PM
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MAY 2009 NYD BORDER

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FILE NAME: 005B-PR-C-103B-2018.dgn
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PLOT DATE: 3:06:32 PM

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CONTRACT NO.:

PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ

SEGMENT "2" PLAN

SHEET REFERENCE NUMBER
C-103B
SHEET 58 OF 11

Submitted by: Ongy.Moawad Plot Date: 4/2/2018 3:06:32 PM
FILE NAME : K:\CADD\60442748-PR\PLANS\2018-PLANS\005B-PR-C-103B-2018.dgn

MAY 2009 NYD BORDER

TEMPORARY CONSTRUCTION EASEMENT (15 FT.)

LIMIT OF CONTRACTORS WORKING AREA

PROPOSED FLOODWALL

SEGMENT "2"

PROPOSED PATH EXTENSION

MCCARTER HWY (RT 21)

N

0 50' 100'

SCALE:1"=50'

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CONTRACT NO.:
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PLOTTED BY: Cindy.Howard
PLOT DATE: 3/30/18 PM

PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ

SEGMENT "2" PLAN

- -

SHEET REFERENCE NUMBER
C-103C
SHEET 5C OF 11

Submitted by: Onsy.Mooaad Plot Date: 4/2/2018 3:07:14 PM
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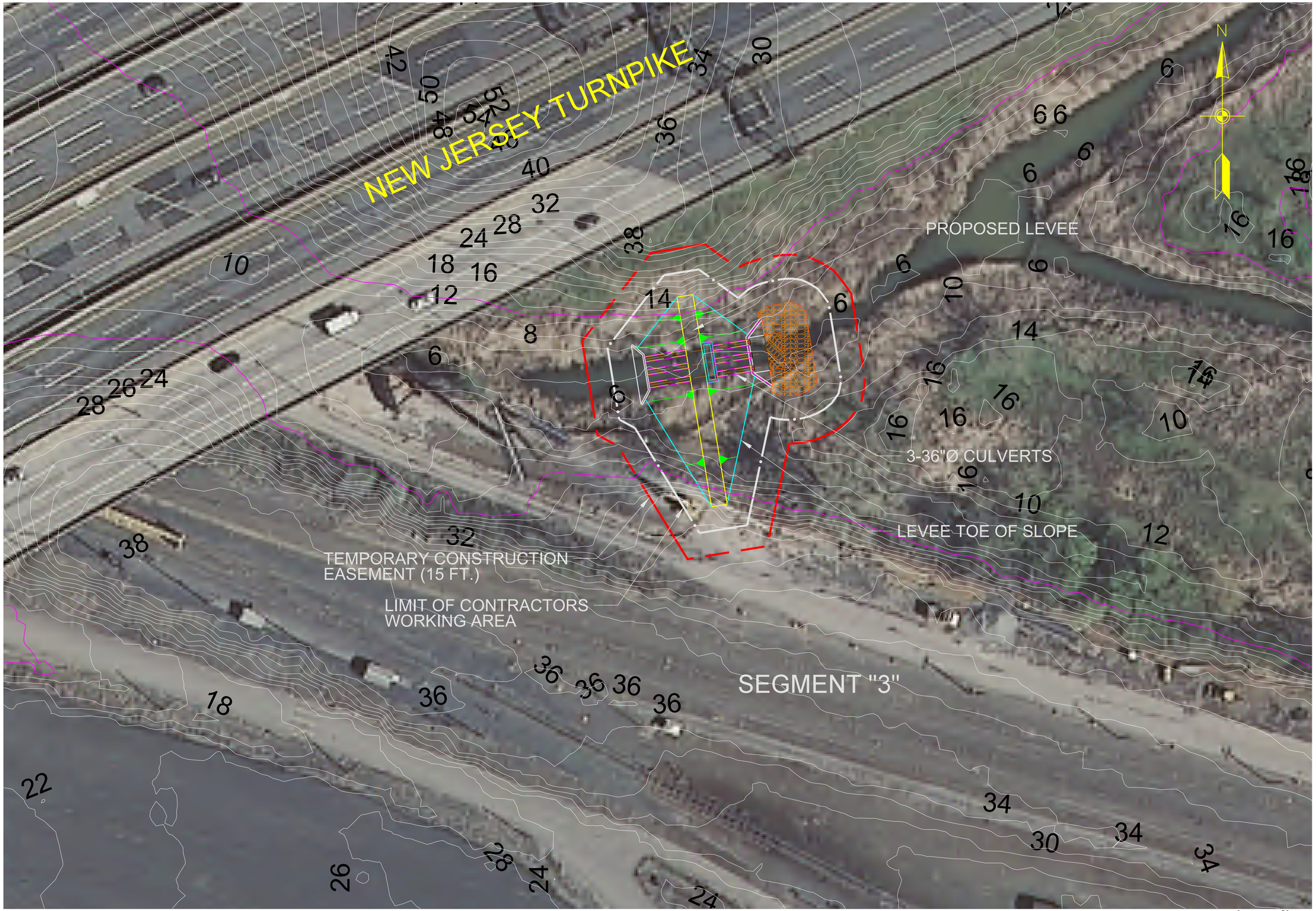
MAY 2009 NYD BORDER



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CONTRACT NO.:

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PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ

SEGMENT "3" PLAN

SHEET REFERENCE NUMBER
C-104
SHEET 6 OF 11

Submitted by: Onsy.Mowad Plot Date: 12/28/2018 3:27:13 PM
FILE NAME : K:\CADD\60442748-PR\PLANS\2018-PLANS\006-PR-C-104-2018.dgn

MAY 2009 NYD BORDER

SEGMENT "4"

NEW JERSEY TURNPIKE

RUTHERFORD ST.

DELANCEY ST.

PROPOSED FLOODWALL

EDGE OF FLOODWALL SLAB

TEMPORARY CONSTRUCTION EASEMENT (15 FT.)

68 LF ROLLER GATE

GATE MONOLITH

LIMIT OF CONTRACTORS WORKING AREA

0 30' 60'

SCALE:1"=30'

Submitted by: OnsyMoawad Plot Date: 3/19/2018 10:24:21 AM
FILE NAME : K:\CADD\60442748-PR\PLANS\2018-PLANS\007-PR-C-105-2018.dgn

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CKD BY: JDR
SUBMITTED BY:
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CONTRACT NO.:
FILE NAME: 007-PR-C-105-2018.dgn
SIZE: ANSI D
PLOTTED BY: OnsyMoawad
PLOT DATE: 7/5/22 AM

PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ

SEGMENT "4" PLAN

-

SHEET REFERENCE NUMBER
C-105
SHEET 7 OF 11

MAY 2009 NYD BORDER

SEGMENT "5"

NEW JERSEY TURNPIKE

WILSON AVE.

EDGE OF FLOODWALL SLAB

PROPOSED FLOODWALL

82 LF ROLLER GATE

GATE MONOLITH

TEMPORARY CONSTRUCTION EASEMENT (15 FT.)

LIMIT OF CONTRACTORS WORKING AREA

0 30' 60'

SCALE:1"=30'

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PLOTTED BY: Onsy.Mosaad
PLOT DATE: 3/19/2018 10:25:07 AM

DATE: MAY, 2016
SOLICITATION NO.:
CONTRACT NO.:

PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ

SEGMENT "5" PLAN

-

SHEET REFERENCE NUMBER
C-106
SHEET 8 OF 11

Submitted by: Onsy.Mosaad Plot Date: 3/19/2018 10:25:07 AM
FILE NAME : K:\CADD\60442748-PR\PLANS\2018-PLANS\008-PR-C-106-2018.dgn

MAY 2009 NYD BORDER

PASSAIC RIVER

SEGMENT "8"

PROPOSED FLOODWALL

15 FT. VEG-FREE ZONE

TEMPORARY CONSTRUCTION EASEMENT (15 FT.)

LIMIT OF CONTRACTORS WORKING AREA

EDGE OF FLOODWALL SLAB

RAYMOND BLVD

VAN BUREN ST

N

0 30' 60'

SCALE:1"=30'

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PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ

SEGMENT "8" PLAN

SHEET REFERENCE NUMBER
C-107
SHEET 9 OF 11

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SUBAPPENDIX 2
NED Plan Geotechnical and Structural Analysis, and Drawings

SUBAPPENDIX 2
2.1: NED Plan Geotechnical Analysis

Passaic River Tidal – General Reevaluation Report

Geotechnical Report

April 6, 2016

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Attachments

Attachment A: Levee Seepage and Stability Analysis
Attachment B: Liquefaction Evaluation
Attachment C: Levee Consolidation Settlement Analysis
Attachment D: Floodwall Seepage and Deep-Seated Sliding Analysis
Attachment E: Pile Capacity Analysis
Attachment F: Boring Log

This report presents the results of the preliminary geotechnical study and the feasibility of levee and floodwall alternatives, and provides recommendations in support of the proposed floodwall system design and construction of the Tidal Portion of the Passaic River Flood Risk Management Plan.

1. INTRODUCTION

The New York District Corps of Engineers (NYD) produced a Draft General Design Memorandum (GDM) in 1995 (Reference 1) and the first phase of a GRR for the entire Passaic River Watershed in 2013, both of which identified Hurricane/Storm Surge/Tidal levees to help manage flood risks in portions of Harrison, Kearny Point and Newark, NJ. The Tidal Protection of the Passaic River provides up to a 500 year level of protection and additional flood risk management to the area (see Figure 1). In this study, the 10.5 miles of protection areas are broken out into the following segments:

- Lister/Turnpike/Doremus Levee/Floodwall in Newark;
- South First Street Levee/Flood Wall in Harrison;
- Kearny Point Levee/Floodwall in Kearny.

Three different design levels of El. +14.0 ft, El. +16.0 ft, and El. +18.0 ft NAVD[1] were considered in the analysis. The ground level along the levee/floodwall alignment varies approximately from El. +6 ft to El. +8 ft. Thus, the design height of the levee/floodwall sections was considered from 6.0 ft to 12.0 ft.

2. SUBSURFACE CONDITIONS

2.1. PREVIOUS SUBSURFACE INVESTIGATION

Based on the available subsurface investigations included in the 1995 GDM (Reference 1) for the Passaic River Flood Damage Reduction Project, Passaic Valley Sewerage Commission Floodwall System Project, and New Jersey Department of Transportation soil borings database, a total of 42 borings along the proposed levee and floodwall alignment are currently available (see Attachment F). The general locations of these borings are shown in Figure 2. After reviewing the boring logs and in-situ and lab test results, the following Segments were assumed for the stability and seepage analyses of the levee and floodwall alternatives.

Soil Profile at East Kearny: Starts at the most eastern portion of the Kearny Segment and continues southcentral as shown in Figure 1.

Soil Profile at West Kearny, Newark and Harrison: Begins at the west end of East Kearny profile and continues west towards the Harrison Segment covering the Newark Segment as shown in Figure 1.

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

[1] All elevations are referenced to North American Vertical Datum of 1988 (NAVD).

purpose. The soil properties were selected based on SPT values and lab test results from available boring logs as shown in Figure 2, boring location plan.

1) East Kearny:

- Organics with Su = 250 psf, 55 feet thick, bottom elevation EL. -50.
- Silty Clay with Su = 500 psf, 30 feet thick, bottom elevation EL. -80.
- Rock (Weathered shale or siltstone), top of rock varies from EL. -80 to EL. -90.

2) West Kearny, Newark, and Harrison:

- Organics with Su = 250 psf, 30 feet thick, bottom elevation EL. -25.
- Silty Clayey Sand with ϕ = 32 psf, 10 ft to 30 feet thick, bottom elevation EL. -55.
- Rock (Weathered shale or siltstone), top of rock varies from EL. -30 to EL. -100

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

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

2.2. 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 at every 200 to 300 feet. Soil profiles typically with 3 borings in the traverse directions perpendicular to the levee-floodwall alignment in each cross-section need to be developed. At least one test boring for each soil profile should be drilled to a depth of bedrock or 100 ft 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) test and consolidation tests need to be performed.
4. It is also recommended that seismic CPT soundings be performed for every 8 borings to obtain shear wave velocity of the subsurface soils. Seismic CPTs may assist to better define the site class, shear wave velocity, and liquefaction potential of the site.
5. Field permeability and/or field pumping test shall be performed, as necessary, for permeability estimation.

3. GEOTECHNICAL ANALYSIS AND EVALUATION

3.1. SEISMIC CONSIDERATIONS

The recommended seismic site classification is Site Class E for all Segments. Depending on the severity of the expected earthquake and the importance of the levee, seismic analyses to determine liquefaction susceptibility may be required (Reference 2). A site-specific liquefaction assessment using the methods outlined in Reference 3 was performed for pockets of granular soils located below the groundwater level in the area of map blocks or sheets #1, 9, 10, 11, 14 and 17 as shown in Figure 1. These analyses require a peak ground surface acceleration (PGA) and an earthquake magnitude (Mw) to estimate the seismic shear stresses. Based on the 2008 USGS seismic hazard maps for return period of 2,475 years (Reference 4), a PGA of 0.32g (g is gravitational acceleration) and an Mw of 5.5 that is primarily based on historical earthquake information in the northeast is used in the analyses.

The factors of safety (FOS) against liquefaction using the site specific analysis for both Segments are shown in Figure 3 and Figure 4. According to Reference 5, the acceptable FOS against liquefaction triggering is 1.2. The results indicate that there is a potential for liquefaction within limited elevations in both Segments, which are 1) a 15 feet thick layer between El. +1 and El. -14 ft in the East Kearny Segment; and 2) a 25 feet thick layer between El. +3 and El. -22 ft in the West Kearny, Newark, and Harrison Segment. The details of the liquefaction analyses are provided in Attachment B.

Because of the liquefaction potential at specific soil layers contingency budgetary costs should be included for liquefaction mitigation measures. Additional subsurface investigations and additional soil boring and lab test data, as well as a more thorough detailed evaluation of the proximity of structures, utilities, etc. are necessary to evaluate the feasibility of the liquefaction mitigation methods such as Deep Dynamic Compaction (DDC)..

3.2. LEVEE

Three different design levels of El. +14.0 ft, El. +16.0 ft, and El. +18.0 ft NAVD were considered in the analysis as shown in Figure 1. The ground level at the line of protection is approximately at El. +6 ft to El. +8 ft. Thus, the design height of the levee varies from 6 ft to 12 ft. A typical levee cross-section with 12 ft height was selected for seepage and slope stability analysis. It is also assumed that riverside toe of levees are away from the top edge of the riverbank for proper stability. The new subsurface investigation and bathymetry survey of the river would be needed to evaluate the minimum distance from the river bank. It is certain that the minimum distance of the levee toe from the riverbank will vary along the line of protection. The maximum height of the levee that meets the minimum required safety factors was obtained by performing a similar slope stability analysis.

3.2.1. SEEPAGE AND SLOPE STABILITY ANALYSIS

3.2.1.1. METHODOLOGY

For preliminary analyses, one typical section for each Segment as described in Section 2.1 was selected for the analyses. The maximum height of the levee section is 12 ft with identical

upstream and downstream slopes of 3H:1V. In general, these cross-sections include an impervious clay core, a layer of high strength geotextile (Synteen® SC30K or approved equivalent) reinforcement at the bottom of proposed levees where necessary and a toe drain at the landside toe.

The seepage and slope stability analyses were performed using commercially available general purpose software SEEP/W© and SLOPE/W© (2007). According to the requirement of USACE EM 1110-2-1913 "Design and Construction of Levees", the following four different loading cases were considered for each Segment analysis:

1. Case I: End of Construction;
2. Case II: Steady Seepage from Full Flood Stage, fully developed phreatic surface;
3. Case III: Rapid Drawdown from Full Flood Stage; and,
4. Case IV: Seismic Loading, with groundwater conditions.

Selected soil shear strength parameters for free drain soils and low permeability soils are in accordance with the requirements of USACE EM 1110-2-1913. The permeability of each material was conservatively estimated based on soil types. Spencer's procedure for the method of slices was used to determine the minimum FOS values and the controlling/critical slip surface associated with the FOS values for all four loading cases.

For the Case I (end of construction) stability analyses, groundwater depth was modeled at El. +0 ft for all Segments. Considering that Case I is a short-term scenario, undrained strength parameters were used for soft organic and medium clay soils in the foundation layers.

Case II was analyzed at flood level elevation of El. +16.0 ft to estimate the conditions at a full flood stage. A 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 (rapid drawdown) 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/upstream 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.

3.2.1.2. RESULTS AND RECOMMENDATIONS

A summary of the calculated FOS and the corresponding required minimum factor of safety values are shown in Table 1, compared with the parameters for the 8-foot levee on 8 to 10 feet of fill, either inspected and approved for use in the foundation or excavated and replaced with controlled structural fill, calculated for the 1995 GDM. As seen from the table, the calculated

FOS values are lower than the minimum requirements of Reference 2 specifically for Case I and II. This is due to the presence of soft or organic soil stratum continuously along the region. Using geotextile slightly increased the stability safety factors but still the minimum required values weren't met. The details of all stability and seepage analysis results for both Segments are provided in Attachment A.

After performing similar slope stability and seepage analysis on levee with different heights it was obtained that 6 ft high levee would meet the minimum required stability safety factors if 4 ft from the subgrade level is replaced with controlled structural fill or the existing fill is at least 4 ft thick and is acceptable for use as foundation. An inspection trench along the centerline of the levee should be excavated to evaluate the existing fill. The slope stability safety factors and their comparison with the minimum required values are provided in

Table 2. The typical section of the proposed levee is shown in Figure 5.

Table 1. Slope Stability Analysis Results for 12 ft High Levee

	Required Minimum Factor of Safety (USACE)	Calculated Factor of Safety	1995 GDM Calculated Factor of Safety (8' levee on fill)
East Kearny Segment:			
Case I: End of Construction	1.3	1.0	1.7
Case II: Steady State - Full Flood Stage	1.4	1.0	2.4
Case III: Rapid Drawdown	1.0	1.0	1.2
Case IV: Seismic Load	1.0	0.9	n/a
West Kearny, Newark, and Harrison Segment:			
Case I: End of Construction	1.3	1.0	1.5
Case II: Steady State - Full Flood Stage	1.4	1.0	2.8
Case III: Rapid Drawdown	1.0	1.0	1.4
Case IV: Seismic Load	1.0	0.9	n/a

Table 2. Slope Stability Analysis Results for 6 ft High Levee on 4 ft Fill

	Required Minimum Factor of Safety (USACE)	Calculated Factor of Safety	1995 GDM Calculated Factor of Safety (8' levee on fill)
Both Segment:			
Case I: End of Construction	1.3	2.0	1.7
Case II: Steady State - Full Flood Stage	1.4	1.4	2.4
Case III: Rapid Drawdown	1.0	1.3	1.2
Case IV: Seismic Load	1.0	1.1	n/a

3.2.2. SETTLEMENT ANALYSIS

Based on the generalized soil profiles, the top 30 to 85 ft of the natural soil in the flood protection area consists of soft and organic soil and 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 settlement of the clayey soil layers.

3.2.2.1. METHODOLOGY

The generalized soil profile for East Kearny Segment was used to estimate the consolidation settlement of 6 ft high levee. The levee is underlain by a 4 ft thick existing fill or structural fill material.

One consolidation test data for silty clay soil is available at East Kearny Segment. The consolidation parameters as recommended in USACE 1995 memorandum was used for the top 12 ft of the organic soil.

In the settlement analysis, the compressible soil layers were divided into sub-layers of 2 feet thicknesses for obtaining better accuracy of calculations. Increase in vertical stresses at the mid depth of each sub-layer due to the embankment load was calculated using the elastic stress distribution methods as outlined in Reference 6.

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.

3.2.2.2. RESULTS AND RECOMMENDATIONS

It is estimated that a total primary consolidation settlement of 8-inch will occur in the compressible soils at the project site due to the construction of 6 ft high levee. In order to minimize the effect of permanent settlement on the levee, the estimated 8-inch consolidation settlement can be added to the construction height of the levee. The detail of the consolidation settlement calculation is provided in Attachment C.

3.3. FLOODWALL

Much of the proposed line of protection (LOP) does not have adequate space for levee construction; therefore, a floodwall alternative is considered in those reaches. Due to the soft foundation soils and unsatisfactory FOS obtained for levee over 6 ft high and also a need to remove unsuitable and uncontrolled existing fill material with varying thickness as discussed in Section 3.2, the floodwall alternative was considered for the entirety of each reach. A typical section of floodwall with sheetpile cutoff is shown in Figure 6.

3.3.1. SEEPAGE AND DEAP-SEATED SLIDING ANALYSIS

The seepage analyses of 12 ft high floodwall for all Segments were performed to estimate the exit gradient and flow rates with and without sheetpile cutoff. The exit gradient at the landside of floodwall with no sheetpile cutoff was 0.86 for both Segments. Per Reference 7, underseepage controls are needed where the calculated exit gradient exceeds an allowable gradient of typically

0.5. Using 20 ft deep sheetpile cutoff reduced the exit gradient to an acceptable value of 0.16. The flow rate for steady state seepage condition could be as high as 14 gallons/day per foot length of the wall. The details of floodwall seepage analyses are provided in Attachment D.

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

3.3.2. PILE BEARING CAPACITY

Pile capacity analyses were performed on three different pile options: H-Piles (HP14x73), 14" precast prestressed concrete piles[2], and Caissons or Micropiles with 8 and 12 inch diameter rock sockets. ENSOFT Software "APILE" was utilized for axial capacity analyses on driven H-piles and precast prestressed concrete piles (see Attachment E). To be conservative, skin resistance for the top 10 ft of the piles was eliminated. Downdrag effects were ignored due to limited information and shall be considered based on the results of additional borings and lab tests.

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

3.3.3. PILE FOUNDATION RECOMMENDATIONS

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

Similar pile capacity analysis performed on 14-inch prestressed precast concrete piles, showed that an 80 ft long concrete pile bearing on silty clay at the East Kearny Segment can provide 100 kips and 95 kips of ultimate compression and uplift capacities, respectively. In West Kearny, Newark, and Harrison Segment, a 60 ft long concrete pile bearing on silty clayey sand can provide approximately 205 kips of ultimate compression capacity and 160 kips of ultimate uplift capacity.

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

The final design shall include a study of pile group effect and pile deflections under lateral,

[2] Precast prestressed concrete (PPC) piles were analyzed as a potential alternative for construction in areas considered still impacted by HTRW. Use of PPC is not considered in the design at this stage of the analysis.

compression, and uplift loads, and potential downdrag effects.

4. PRELIMINARY INFORMATION AND ASSUMPTIONS

The preliminary information and assumptions made in this report that could have significant impacts on the project costs are summarized below:

1. The analyses and calculations performed in this report are preliminary in nature and all estimates were based on limited available data. The new subsurface investigation and laboratory testing program as recommended in Section 2.2 are necessary to meet USACE requirements.
2. A layer of highly variable fill materials up to approximately 20 feet in thickness exists in the area of protection. The top 4 ft of the fill needs to be removed and replaced with controlled structural fill if the existing fill is not acceptable for use in foundation.
3. Because of the liquefaction potential at specific soil layers contingency budgetary costs should be included for liquefaction mitigation measures. Where necessary, liquefaction mitigation methods such as dynamic compaction can be further studied at the project site.
4. The riverside toe of levees is assumed to be away from the top edge of the riverbank for proper stability. The new subsurface investigation and bathymetry of the river would be needed to evaluate the minimum distance from the river bank. It is certain that the minimum distance of the levee toe from the riverbank will vary along the line of protection.
5. For pile depth calculations, rock depths vary along the line of protection but pile lengths are assumed to be conservative (exceeding 100 feet in some locations).

5. CONCLUSION

The analyses and calculations performed in this report are preliminary in nature and all estimates were based on limited available data. The new subsurface investigation and laboratory testing program as recommended in Section 2.2 are necessary to meet USACE requirements.

5.1. LEVEE

Due to the presence of organic soils along the Segment, the proposed 6 ft high levee system requires a 4 ft of structural fill (or existing fill, if inspected and approved) beneath the levee to meet the minimum required stability. The fill material and soft soil along the Segments possess hydraulic exit gradient within an acceptable range. If it is intended to reduce the quantity of flow through the foundation below 7 gallons/day per foot, some seepage control methods such as sheetpile cutoff should be evaluated and utilized.

The recommended flood protection system for the areas with the top of wall elevation at El. +14 and ground surface at El. +8 ft in both Segments should be evaluated based on the construction cost of levee and floodwall. For the levee alternative inspecting the existing fill and possibly replacing it with a 4 ft thick structural fill should be considered in the cost estimate Depending on the severity of the expected earthquake and the importance of the levee, seismic analyses to determine liquefaction susceptibility may be required (Reference 2). Based on the evaluation

performed, there is liquefaction potential at specific locations as mentioned in Section 3.1 (see Figure 3 and Figure 4) and contingency budgetary costs should be included for liquefaction mitigation measures.

5.2. FLOODWALL

For the areas with lower ground elevation than El. +8 ft or higher top of wall elevation than El. +14 ft the levee system cannot be recommended due to the stability issues as discussed in Section 3.2.1. For these areas, it is recommended to use a floodwall system (T-Wall or I-Wall) with 20 ft deep sheetpile cutoff to control the seepage through the foundation. In areas with deeper rock elevation H-Pile or PPC piles may provide sufficient allowable compression and tension capacities. Micropiles or Caissons with rock socket can be utilized in areas with relatively shallow rock depth especially in West Kearny, Newark, and Harrison Segment.

6. REFERENCES

1. USACE (1995), General Design Memorandum (GDM), Passaic River Flood Damage Reduction Project, Appendix E- Geotechnical Design, Levees, Floodwalls and Miscellaneous, United States Army Corps of Engineers, dated September 1995.
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8. "Design Guidance for Levee Underseepage", ETL-1110-2-569, United States Army Corps of Engineers, dated May a, 2005.

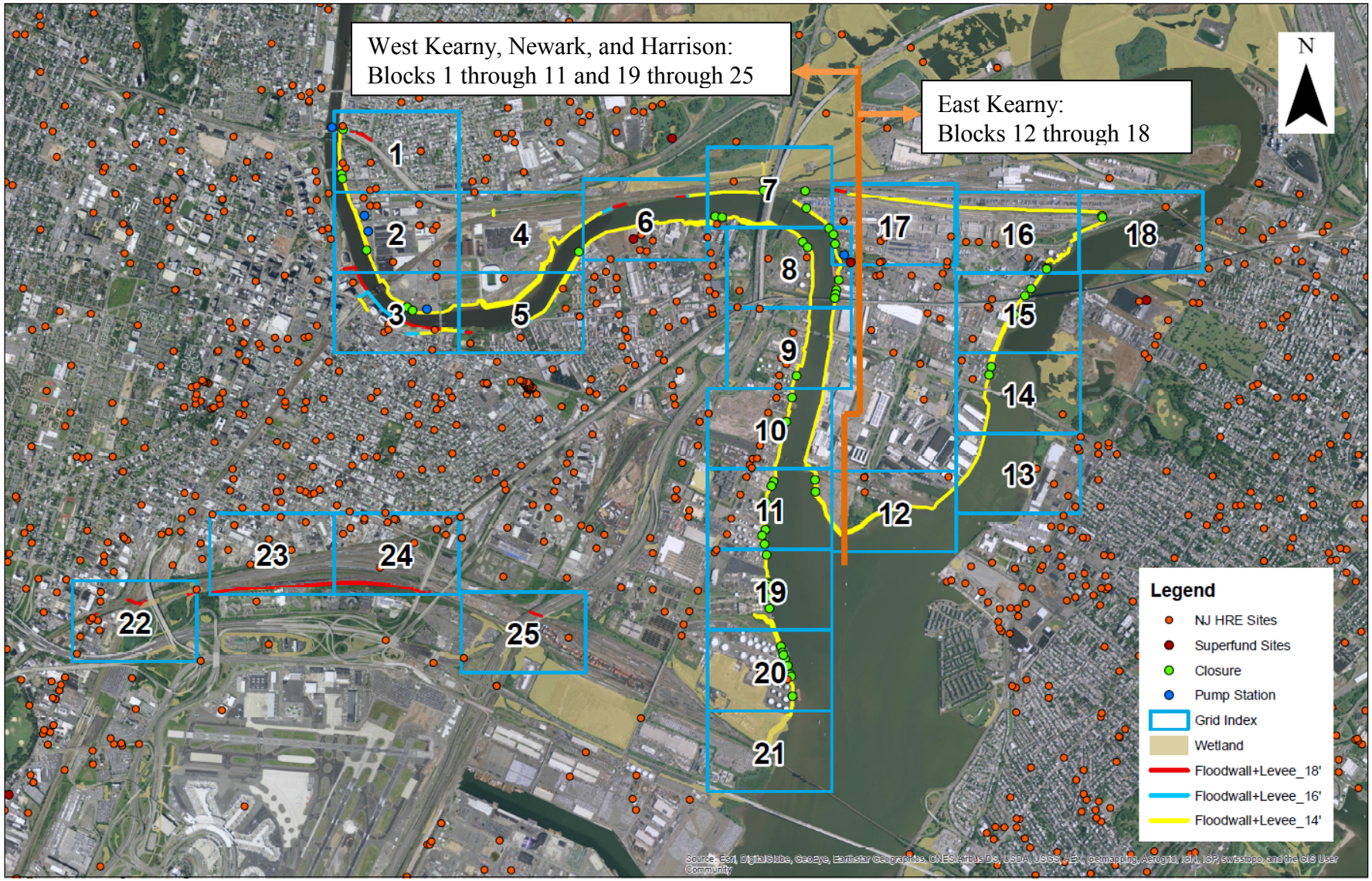

Figure 1. Site Location Plan and Segments

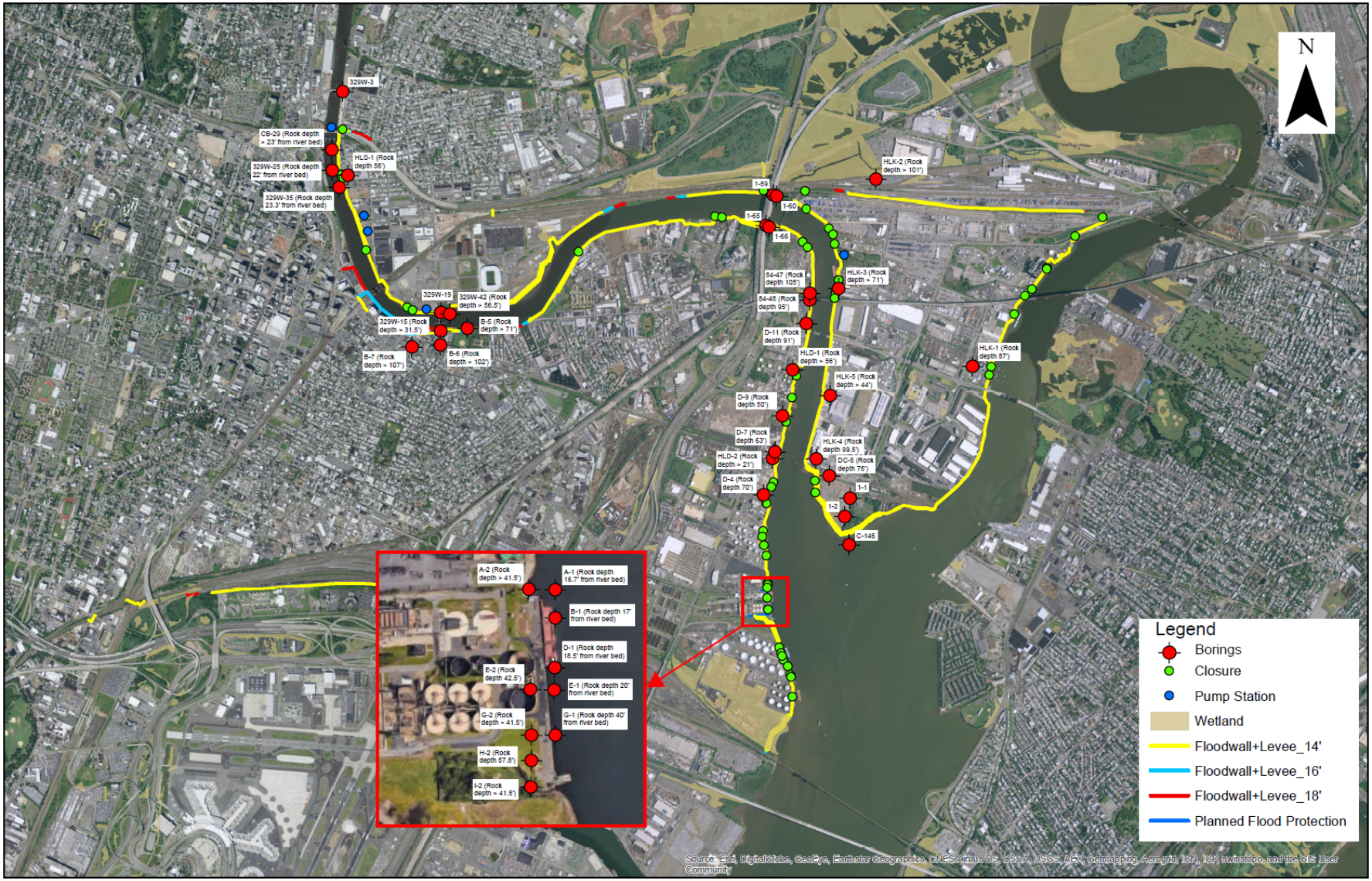

Figure 2. Boring Location Plan

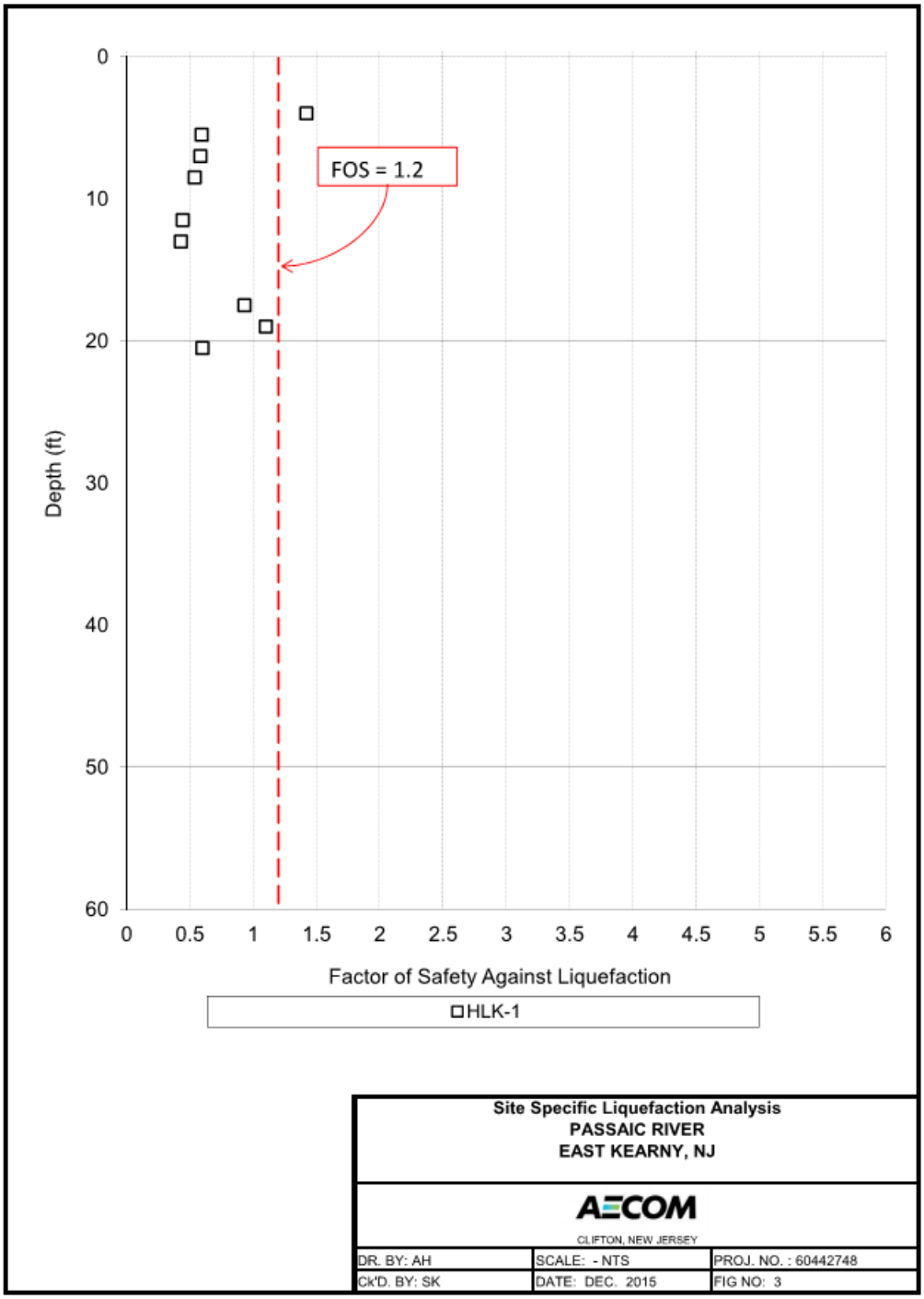

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Figure 3. FOS Against Liquefaction – East Kearny

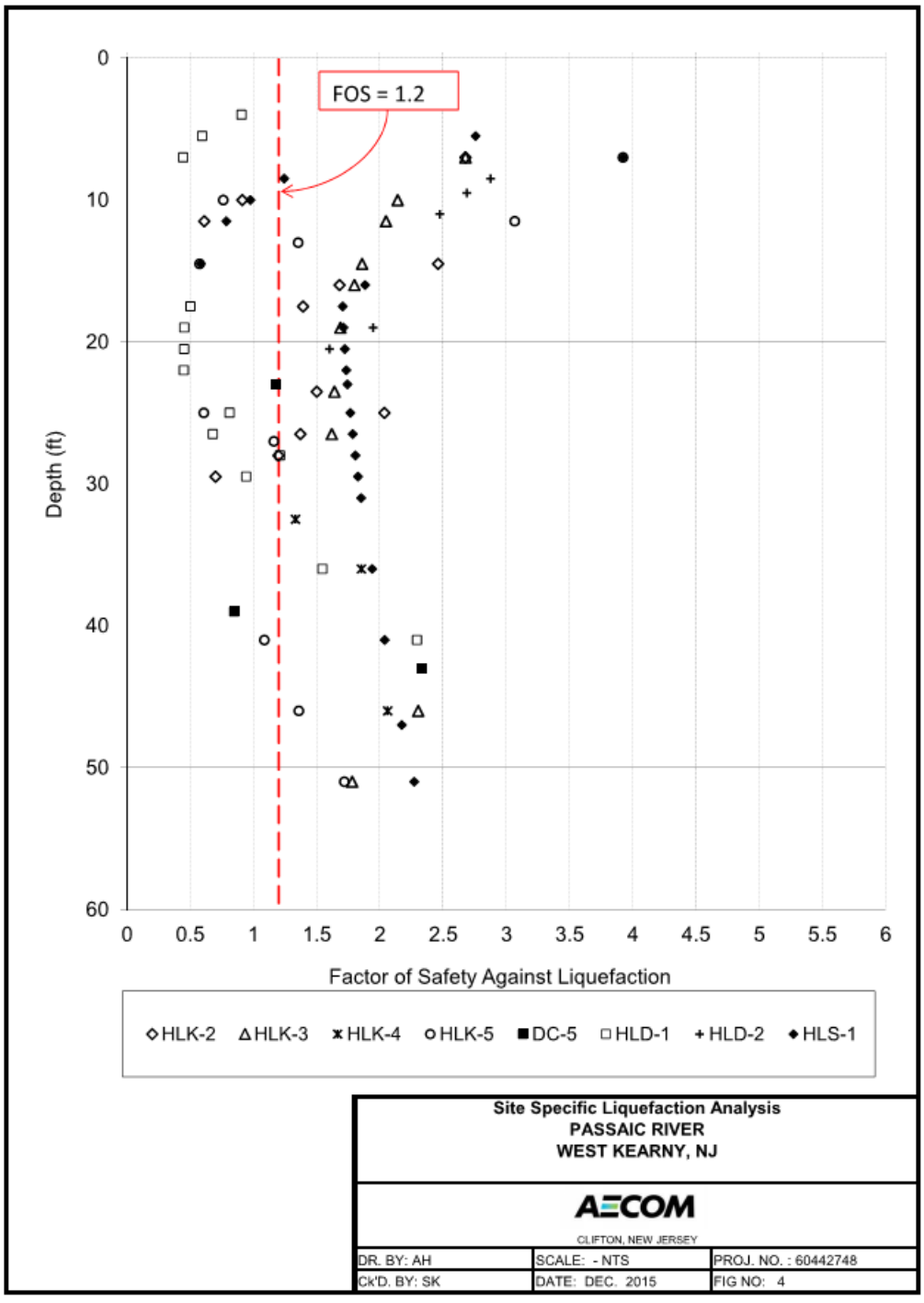

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Figure 4. FOS Against Liquefaction – West Kearny, Newark, and Harrison

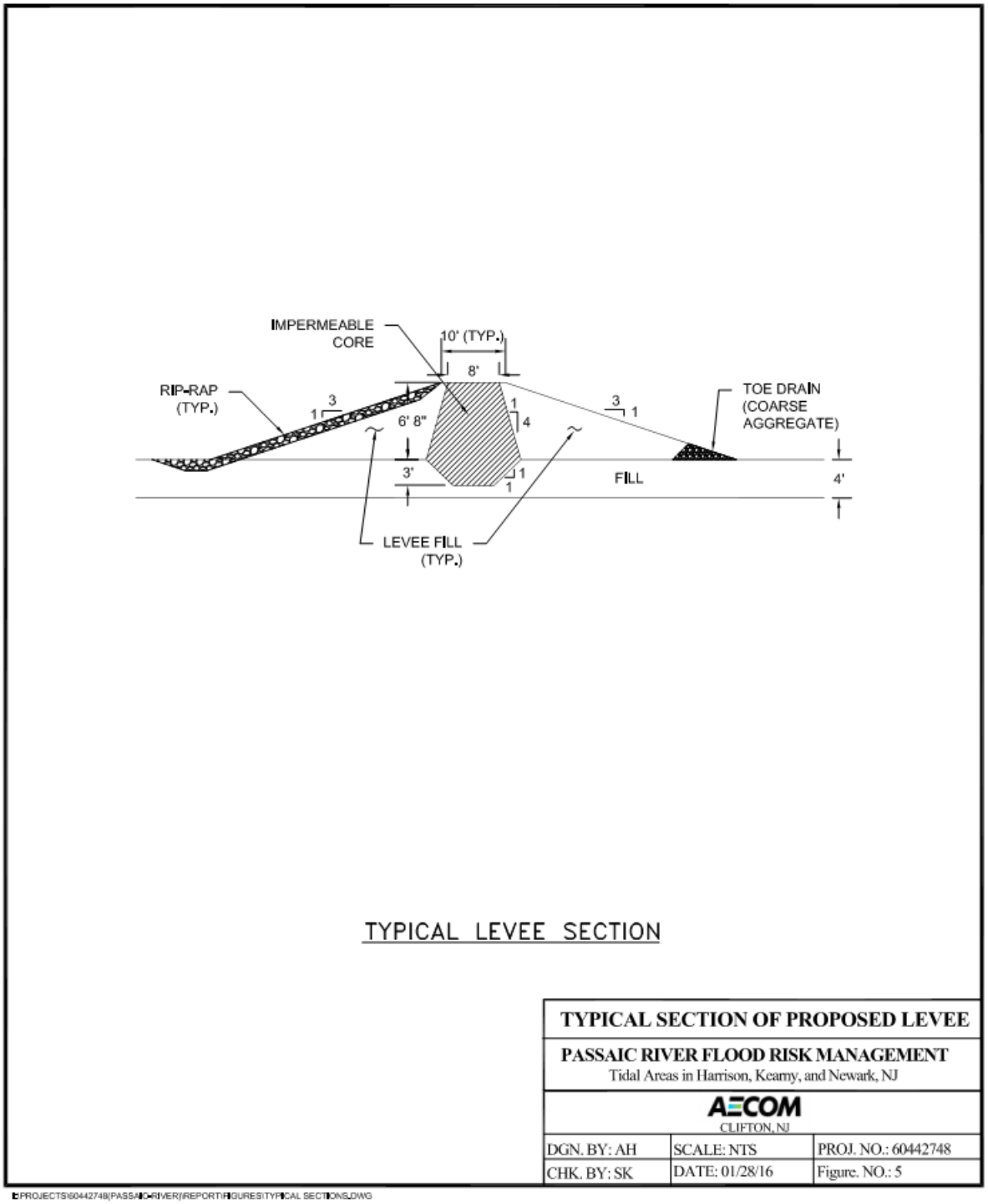


Figure 5. Typical Section of Proposed 6ft High Levee

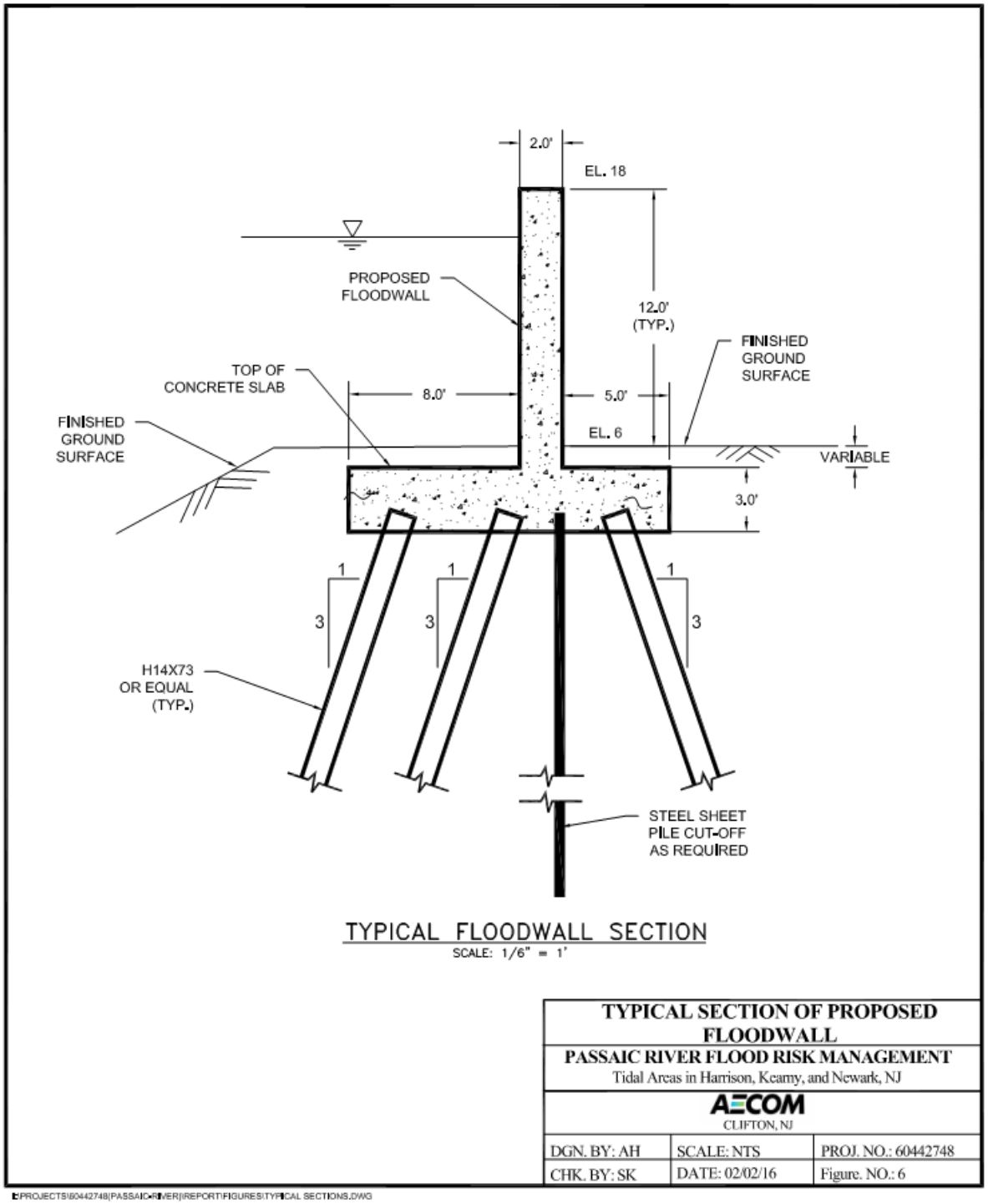


Figure 6. Typical Section of Floodwall

Attachment A

LEVEE SEEPAGE & SLOPE STABILITY ANALYSES

OBJECTIVES

1. To calculate exit hydraulic gradient and seepage flow through the levee
2. Obtain pore pressures for slope stability analyses for levee
3. Slope stability analyses for Upstream and Downstream slopes of proposed Earth Levee

ASSUMPTIONS

- Upstream Slope Angle: 1V:3H
- Downstream Slope Angle: 1V:3H
- Maximum Height of Levee: Case (a) 12 feet, Case (b) 6 feet
- Top of Levee: Case (a) El. +18 feet, Case (b) El. +14 feet (NAVD88)
- Flood Level: Case (a) El. +16 feet, Case (b) El. +13 feet (NAVD88)
- Top of ground surface: Case (a) El. +6 feet, Case (b) El. +8 feet (NAVD88)
- Static groundwater level: El. 0 feet
- Horizontal pseudo static seismic coefficient: 0.21
- Levee with separate shell and core
- High strength Geogrid (min. required Long Term Design Strength of 15000 lbs/ft) is used in the stability analysis for the case with Fabric.
- The riverside toe of levees is assumed to be away from the top edge of the riverbank for proper stability. The new subsurface investigation and bathymetry of the river would be needed to evaluate the minimum distance from the river bank. It is certain that the minimum distance of the levee toe from the riverbank will vary along the protection line.
- Embankment and subsurface soil properties as Table A.1 are considered for the analysis.

Table A.1: Properties for Embankment Material and Subsurface Soils

Zone	Segments		Materials	Unit Weight (pcf)	φ °	Cohesion (psf)	K (cm/sec)
Levee	All Segments		Shell	120	32	0	1.00E-05
		Short Term	Core	120	0	1000	1.00E-06
		Long Term	Core	120	30	0	1.00E-06
			Toe-Drain	120	35	0	1.00E-03
			Fill	115	30	0	1.00E-04
Foundation Soil	East Kearny	Short Term	Soft or Organic Soil	85	0	250	1.00E-04
		Long Term		100	20	0	1.00E-04
		Short Term	Silty Clay	120	0	500	1.00E-05
		Long Term		120	26	0	1.00E-05
	West Kearny	Short Term	Soft or Organic Soil	85	0	250	1.00E-04
		Long Term		100	20	0	1.00E-04
			Silty Clayey Sand	120	32	0	1.00E-04

METHODOLOGY

Seepage Analyses

A commercially available, general purpose seepage computer program, SEEP/W, was used to perform seepage analyses. Seepage flow and hydraulic exit gradient at toe were estimated for the steady state hydraulic conditions. The estimated exit gradient values were compared with allowable values recommended by the Army Corps of Engineers ETL 1110-2-569 (Reference 8) to assess the need for underseepage controls.

Slope Stability Analyses

A commercially available, general purpose slope stability computer program, SLOPE/W, was used to perform the slope stability analyses. SLOPE/W uses the limit equilibrium methods to compute the factor of safety (FOS) for a given slope geometry and loading conditions. Spencer's Procedure for the method of slices for circular failure was used to evaluate the slope stability as this procedure satisfies the complete static equilibrium for each slice. SLOPE/W automatically searches for the circular slip surface associated with the minimum FOS, which is considered the critical or controlling slip surface. The stability analyses were performed for the end of construction case and for piezometric conditions anticipated during flood events as listed below. In addition, stability under seismic loading and rapid drawdown conditions was also analyzed. All these analyses were performed with estimated effective stress strength parameters. However, for the end of construction case, total stress strength parameters were used for the clayey soils. In general accordance with EM 1110-2-1913, the following cases were analyzed:

Case I: End of Construction - Upstream/Downstream Slopes
Case II: Steady Seepage from Maximum Flood Level - Downstream Slope
Case III: Rapid Drawdown (from a fully developed steady state condition) - Upstream Slope
Case IV: Seismic Loading (Pseudo Static Coefficient of 0.21) - Downstream Slope

Pore pressures for use in the corresponding slope stability analyses were estimated from seepage analysis results for Cases II & III. The groundwater level was used for slope stability analyses of Cases I and IV.

Earthquake Conditions

It is a standard practice to consider the pseudo static coefficient as 2/3 of PGA/g in design where the PGA is Peak Ground Acceleration and the g is gravity acceleration. The seismic site class of this project site could be "E". Using the 2008 USGS seismic hazard maps, a PGA value of 0.32g was estimated for a 2,475 years seismic event. Accordingly, pseudo static coefficient of 0.21 (<= 2/3x0.32g/g) was estimated and used in the stability analyses.

RESULTS AND DISCUSSIONS

Seepage Analyses

- Steady-state seepage analysis results for Case (a) levee are provided in Figure A.6 and Figure A.12. As discussed below, Case (a) levee didn't meet the minimum required stability safety factors thus, seepage analysis results aren't discussed.

- Based on steady state seepage analyses, seepage flow under/through 6 ft high levee, Case (b), is estimated to be approximately 7 gpd per feet in both segments (see Figure A.17).
- Based on steady seepage analyses vertical hydraulic exit gradient for 6 ft high levee, Case (b), is approximately 0.14 in both segments (see Figure A.17). Note that this value is lower than the allowable gradient. Typically, the allowable hydraulic exit gradient is considered as 0.2, but it can be as much as 0.5 (Reference 8).

The fill material and soft soil along the Segments are estimated to possess hydraulic exit gradient within an acceptable range. This must be confirmed following the subsequent geotechnical investigation. If it is intended to reduce the quantity of flow through the foundation, some seepage control methods such as sheetpile cutoff should be evaluated and utilized.

Note that the estimated flow and exit hydraulic gradient values depend on the assumed permeability of embankment and subsurface soils. However, it is likely that nominal seepage control measures such as a toe drain may be sufficient to handle the flow through/under the proposed levee. Based on the estimated seepage flow, seepage flow will not likely exist through the embankment slope for steady seepage case. However, it is recommended that nominal slope protection measures such as vegetative cover (top soil/grass) be provided for both upstream and downstream slopes and the base as required.

Slope Stability Analyses

A summary of the calculated factors of safety and the corresponding required minimum factors of safety for 12 feet high (Case (a)) and 6 ft high (Case (b)) levees are given in Table A.2 and Table A.3, respectively. The output slope stability slip surfaces and seepage contours also shown in Figures A.1 through A.12 for Case (a) and Figures A.13 through A.17 for Case (b). As seen from the results, case (a) levee is not stable even with a layer of high strength geotextile reinforcement at the foundation interface. The calculated factors of safety satisfied the minimum required values for Case (b) levee which is 6 ft high levee underlain by 4 ft thick structural fill or inspected existing fill. Note that, for the End of Construction case, results are presented only for the downstream slope as the upstream slope is identical (both are 1V:3H) to the downstream slope.

Table A.2: Summary of Slope Stability and Seepage Analyses Results for Case (a) Levee (12 ft high)

Location	Design Condition	Case	Analyzed Slope	Factor of Safety (FOS)		Steady-State Seepage	
				Req. Minimum	Estimated	Flow Rate $ft^3/sec/ft$	Exit Gradient
East Kearny	End of Construction	Levee Without Fabric	Downstream	1.3	1.0	1.564E-05	0.21
	Seismic Loading	Levee Without Fabric	Downstream	1.0	0.9		
	Steady Seepage with Full Flood Stage	Levee Without Fabric	Downstream	1.4	1.0		
	Rapid Drawdown from the Full Flood	Levee Without Fabric	Upstream	1.0	1.0		
	End of Construction	Levee With Fabric	Downstream	1.3	1.1		
West Kearny, Newark, Harrison	End of Construction	Levee Without Fabric	Downstream	1.3	1.0	1.645E-05	0.20
	Seismic Loading	Levee Without Fabric	Downstream	1.0	0.9		
	Steady Seepage with Full Flood Stage	Levee Without Fabric	Downstream	1.4	1.0		
	Rapid Drawdown from the Full Flood	Levee Without Fabric	Upstream	1.0	1.0		
	End of Construction	Levee With Fabric	Downstream	1.3	1.0		

Table A.3: Summary of Slope Stability and Seepage Analyses Results for Case (b) Levee (6 ft high)

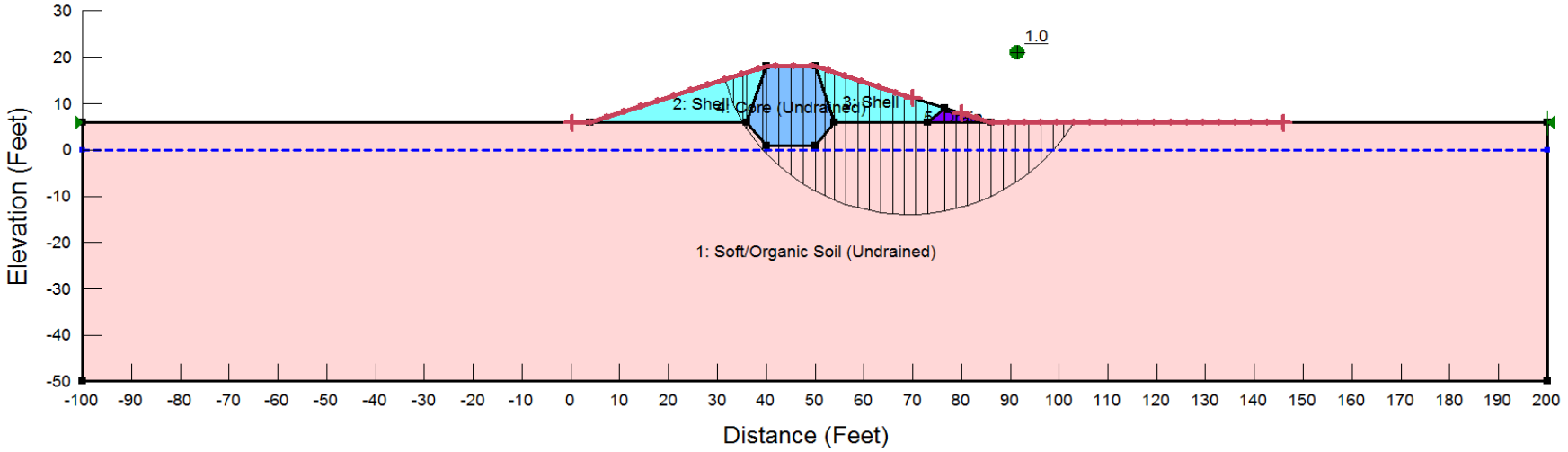
Location	Design Condition	Case*	Analyzed Slope	Factor of Safety (FOS)		Steady-State Seepage	
				Req. Minimum	Estimated Value	Flow Rate $ft^3/sec/ft$	Exit Gradient
All Segments	End of Construction	Levee With 4ft thick fill	Downstream	1.3	2.0	1.077E-05	0.14
	Seismic Loading	Levee With 4ft thick fill	Downstream	1.0	1.1		
	Steady Seepage with Full Flood	Levee With 4ft thick fill	Downstream	1.4	1.4		
	Rapid Drawdown from the Full	Levee With 4ft thick fill	Upstream	1.0	1.3		

* 4 ft thick existing fill material will be excavated and replaced with imported fill (The fill properties assumed in the analysis are provided in Table A.1).

REFERENCES

- EM 1110-2-1913, 2000. “Design and Construction of Levees”, US Army Corps of Engineers.
- ETL 1110-2-569, “Design Guidance for Levee Underseepage”, US Army Corps of Engineers.
- GEOSTUDIO 2007 with Slope/W and Seep/W package.

Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Core (Undrained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 1000 psf Phi: 0 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Soft/Organic Soil (Undrained) Model: Mohr-Coulomb Unit Weight: 85 pcf Cohesion: 250 psf Phi: 0 °



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Critical Sliding Surface and Stability FOS for End of Construction (12' High Levee)

same FOS value was estimated for both Upstream and Downstream Slopes

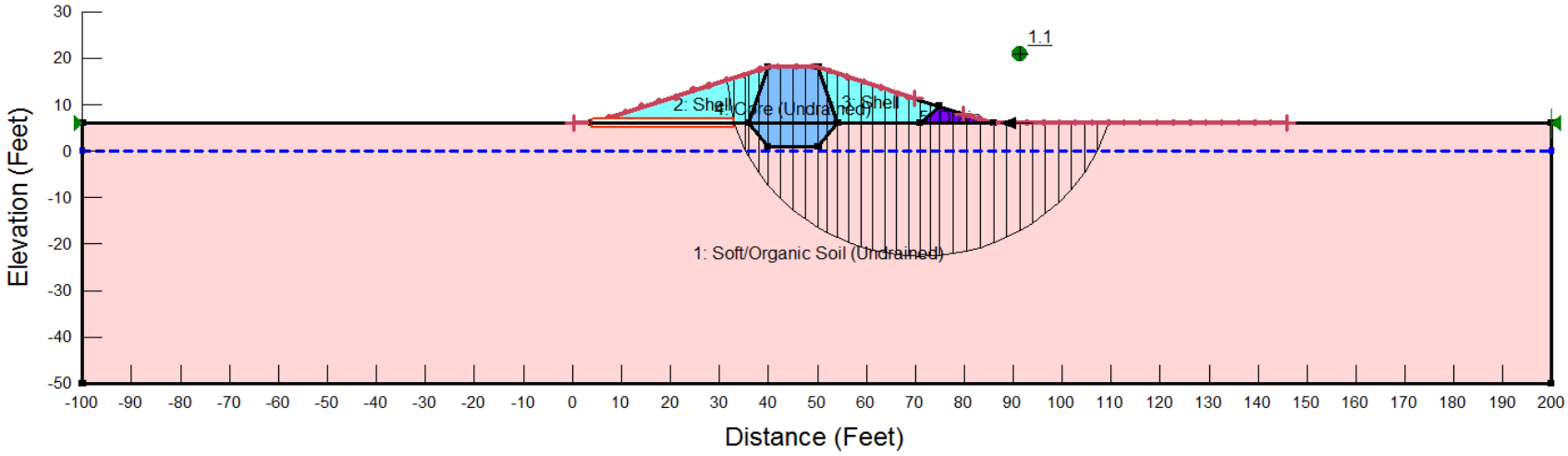
SLOPE STABILITY OF LEVEE ALTERNATIVE
East Kearny Segment

Critical Sliding Surface and Stability FOS

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.1

Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Core (Undrained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 1000 psf Phi: 0 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Soft/Organic Soil (Undrained) Model: Mohr-Coulomb Unit Weight: 85 pcf Cohesion: 250 psf Phi: 0 °



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Critical Sliding Surface and Stability FOS for End of Construction using Fabric (12' High Levee)

same FOS value was estimated for both Upstream and Downstream Slopes

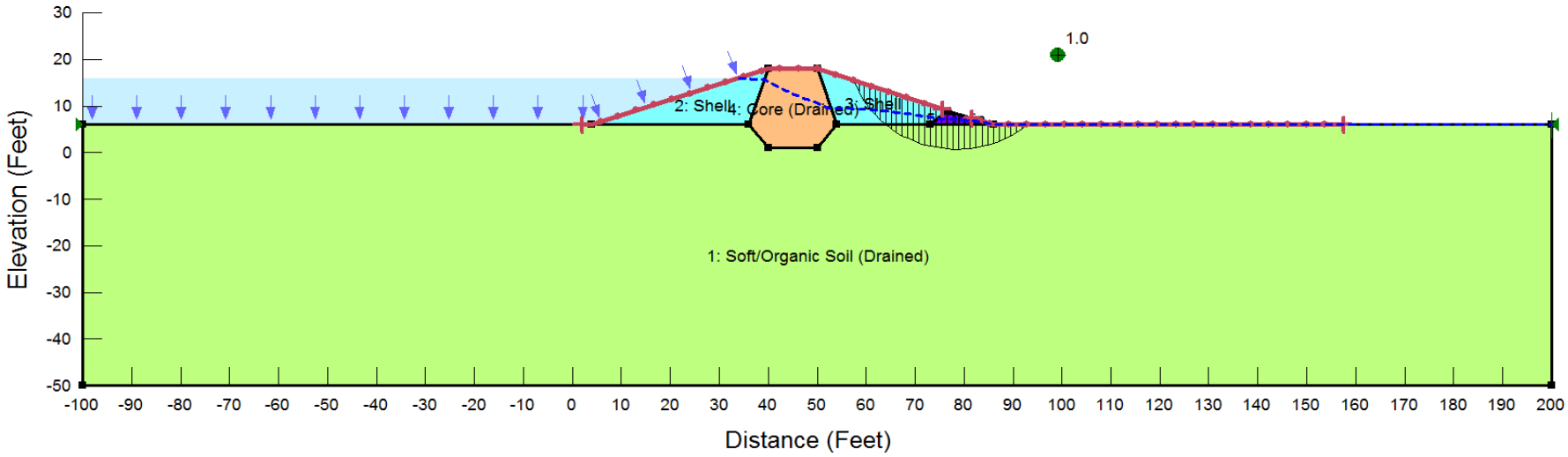
SLOPE STABILITY OF LEVEE ALTERNATIVE
East Kearny Segment

Critical Sliding Surface and Stability FOS

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.2

Name: Soft/Organic Soil (Drained) Model: Mohr-Coulomb Unit Weight: 100 pcf Cohesion: 0 psf Phi: 20 °
Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Core (Drained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 30 °



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Critical Sliding Surface and Stability FOS for Steady Seepage (12' High Levee)

Steady Seepage with Full Flood Pool (El. +16 Feet)

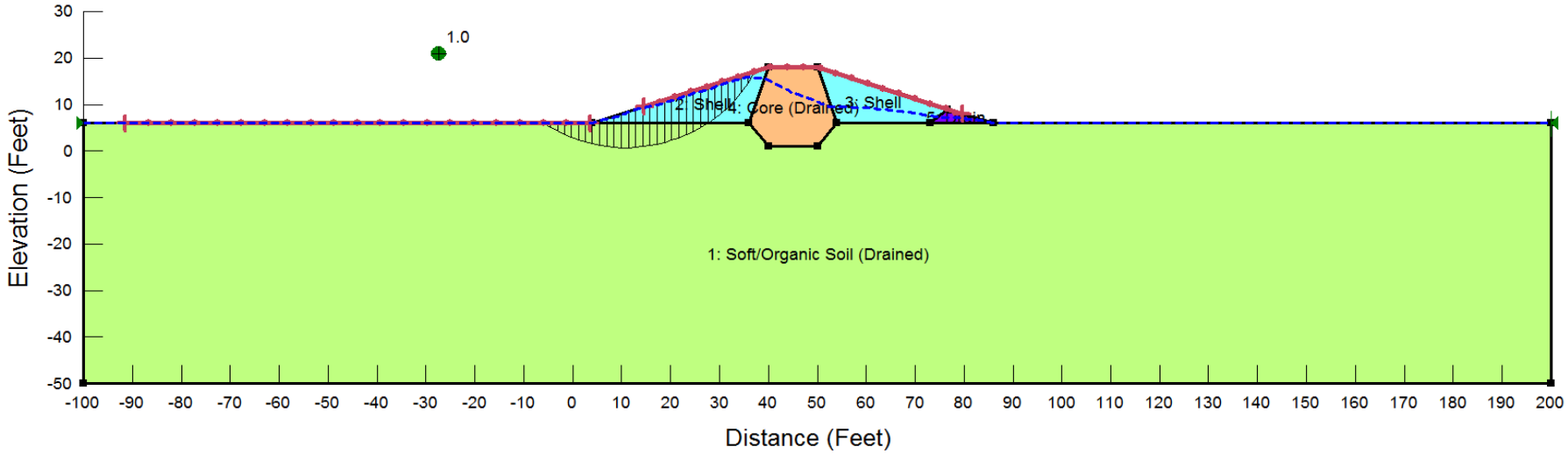
SLOPE STABILITY OF LEVEE ALTERNATIVE
East Kearny Segment

Critical Sliding Surface and Stability FOS

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.3

Name: Soft/Organic Soil (Drained) Model: Mohr-Coulomb Unit Weight: 100 pcf Cohesion: 0 psf Phi: 20 °
Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Core (Drained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 30 °



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Critical Sliding Surface and Stability FOS for Rapid Drawdown (12' High Levee)

Rapid Drawdown from the Full Flood Stage

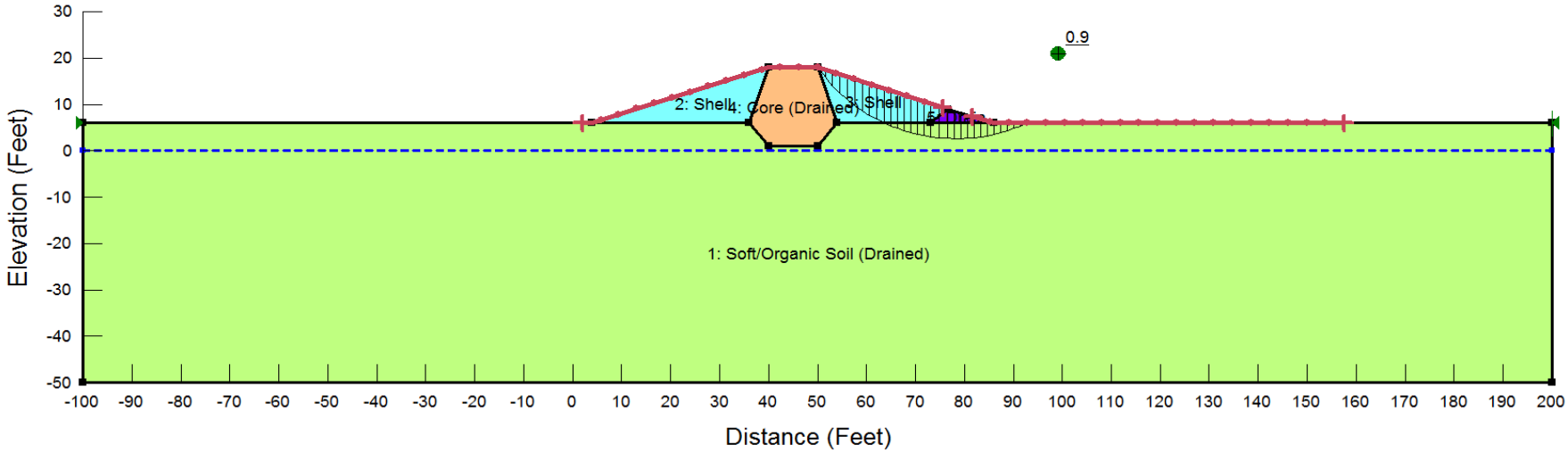
SLOPE STABILITY OF LEVEE ALTERNATIVE
East Kearny Segment

Critical Sliding Surface and Stability FOS

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.4

Name: Soft/Organic Soil (Drained) Model: Mohr-Coulomb Unit Weight: 100 pcf Cohesion: 0 psf Phi: 20 °
Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Core (Drained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 30 °



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Critical Sliding Surface and Stability FOS for Seismic Condition (12' High Levee)

Seismic Loading (Pseudo-Static Coefficient of 0.21)

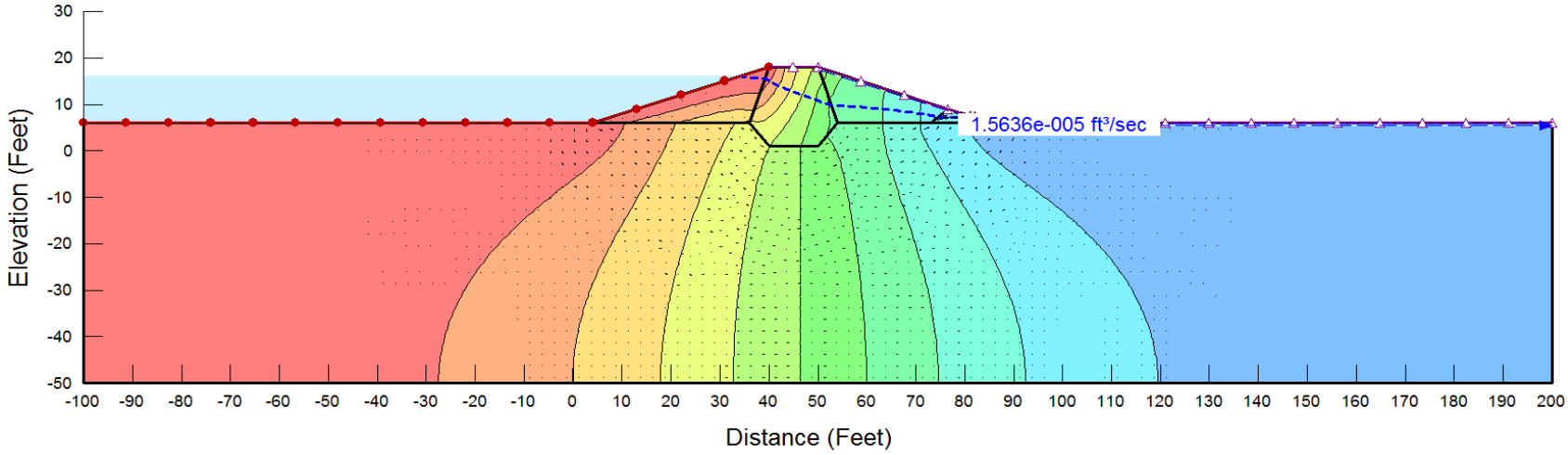
SLOPE STABILITY OF LEVEE ALTERNATIVE
East Kearny Segment

Critical Sliding Surface and Stability FOS

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.5

Name: Soft/Organic Soil (Drained) Model: Saturated Only K-Sat: 3.28e-006 ft/sec Volumetric Water Content: 0 ft³/ft³
Name: Shell Model: Saturated / Unsaturated K-Function: shell Vol. WC. Function: VWC-Shell
Name: Core (Undrained) Model: Saturated / Unsaturated K-Function: Core Vol. WC. Function: VWC-Core
Name: Drain Model: Saturated / Unsaturated K-Function: Drain Vol. WC. Function: VWC-Gravelly Sand



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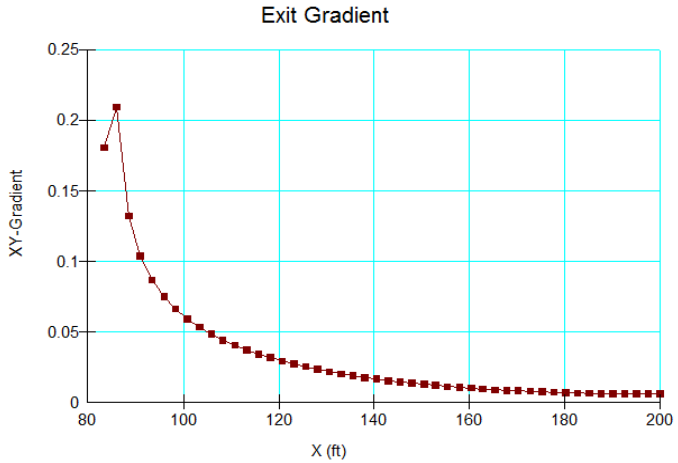
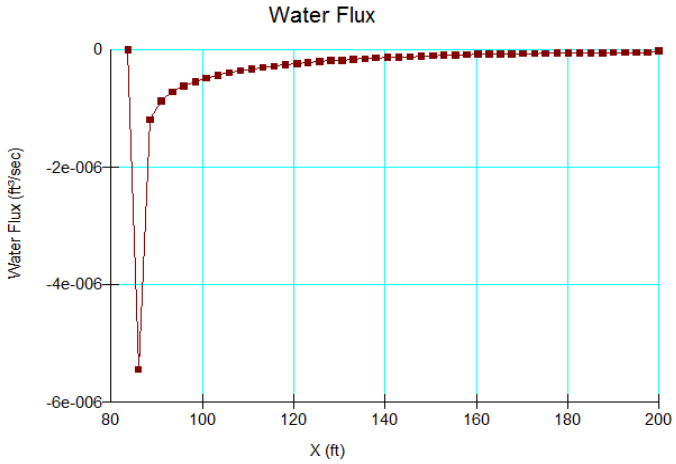




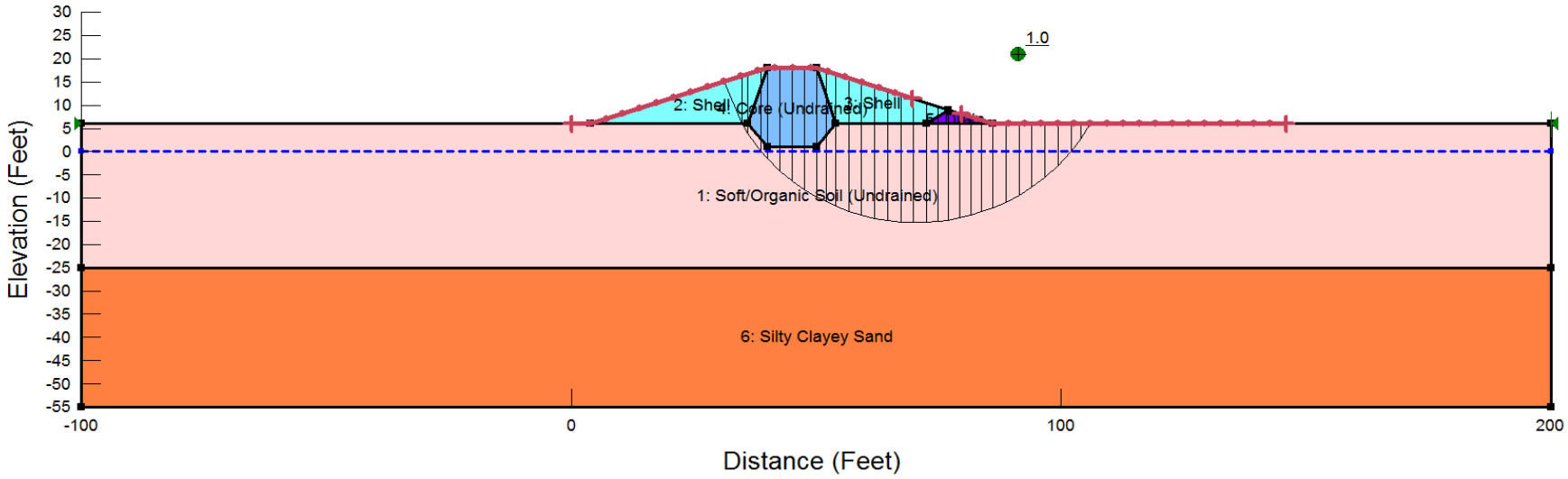
Steady-State Seepage with Flood Pool at El. 16 ft (12' High Levee)

SEEPAGE ANALYSIS OF LEVEE ALTERNATIVE
East Kearny Segment
Estimated Piezometric Conditions, Flow Rate, and Exit Gradient

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.6

Name: Silty Clayey Sand Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Core (Undrained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 1000 psf Phi: 0 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Soft/Organic Soil (Undrained) Model: Mohr-Coulomb Unit Weight: 85 pcf Cohesion: 250 psf Phi: 0 °



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Critical Sliding Surface and Stability FOS for End of Construction (12' High Levee)

same FOS value was estimated for both Upstream and Downstream Slopes

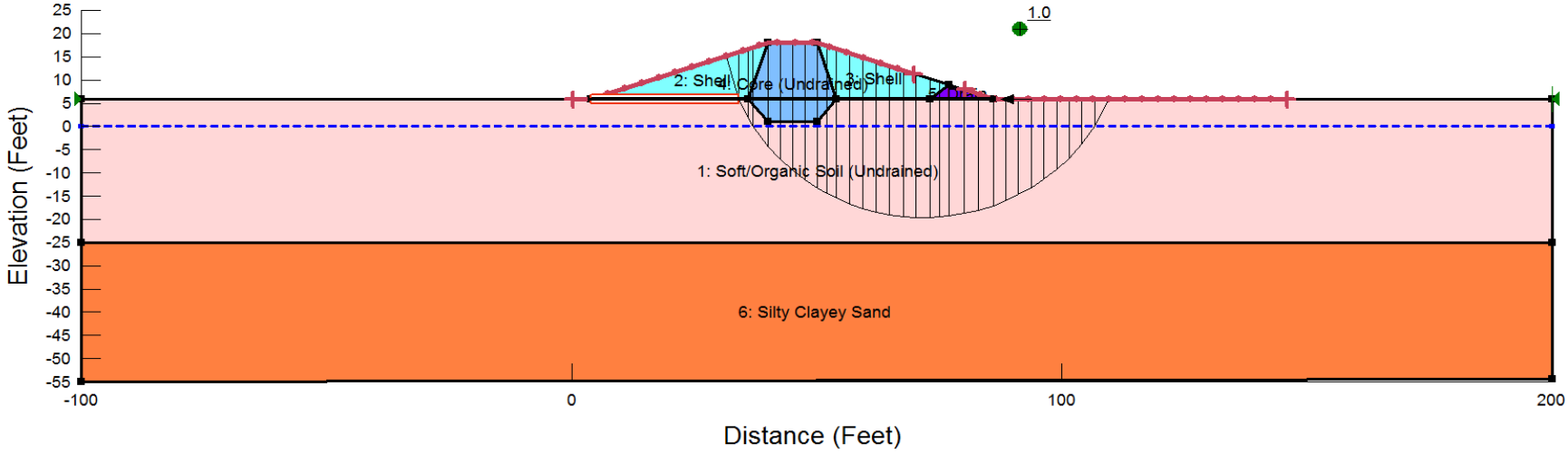
SLOPE STABILITY OF LEVEE ALTERNATIVE
West Kearny Segment

Critical Sliding Surface and Stability FOS

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.7

Name: Silty Clayey Sand Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Core (Undrained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 1000 psf Phi: 0 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Soft/Organic Soil (Undrained) Model: Mohr-Coulomb Unit Weight: 85 pcf Cohesion: 250 psf Phi: 0 °



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Critical Sliding Surface and Stability FOS for End of Construction using Fabric (12' High Levee)

same FOS value was estimated for both Upstream and Downstream Slopes

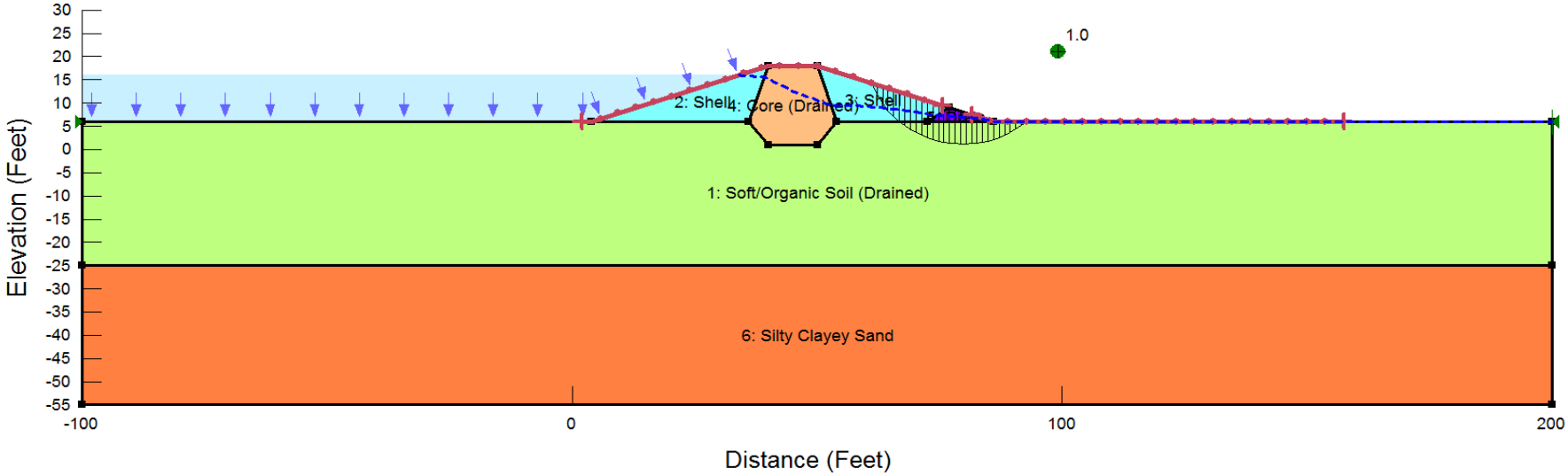
SLOPE STABILITY OF LEVEE ALTERNATIVE
West Kearny Segment

Critical Sliding Surface and Stability FOS

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.8

Name: Soft/Organic Soil (Drained) Model: Mohr-Coulomb Unit Weight: 100 pcf Cohesion: 0 psf Phi: 20 °
Name: Silty Clayey Sand Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Core (Drained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 30 °



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Critical Sliding Surface and Stability FOS for Steady Seepage (12' High Levee)

Steady Seepage with Full Flood Pool (El. +16 Feet)

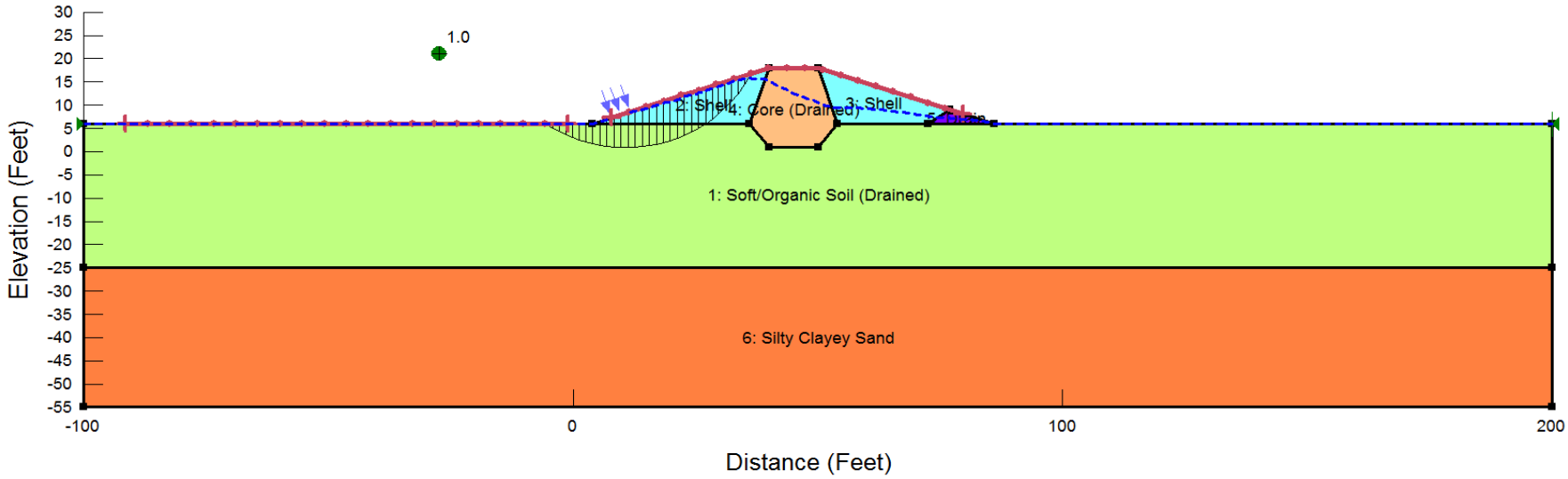
SLOPE STABILITY OF LEVEE ALTERNATIVE
West Kearny Segment

Critical Sliding Surface and Stability FOS

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.9

Name: Soft/Organic Soil (Drained) Model: Mohr-Coulomb Unit Weight: 100 pcf Cohesion: 0 psf Phi: 20 °
Name: Silty Clayey Sand Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Core (Drained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 30 °



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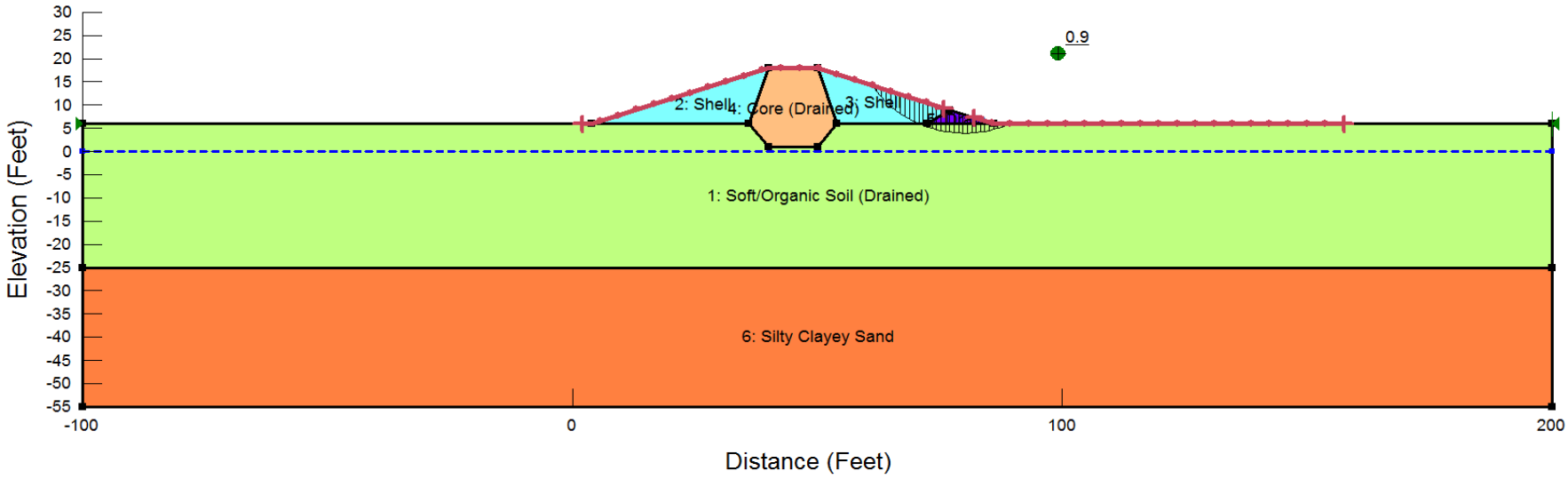
Critical Sliding Surface and Stability FOS for Rapid Drawdown (12' High Levee)

Rapid Drawdown from the Full Flood Stage

SLOPE STABILITY OF LEVEE ALTERNATIVE West Kearny Segment Critical Sliding Surface and Stability FOS		
AECOM		
DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.10

Name: Soft/Organic Soil (Drained) Model: Mohr-Coulomb Unit Weight: 100 pcf Cohesion: 0 psf Phi: 20 °
Name: Silty Clayey Sand Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Core (Drained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 30 °



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Critical Sliding Surface and Stability FOS for Seismic Condition (12' High Levee)

Seismic Loading (Pseudo-Static Coefficient of 0.21)

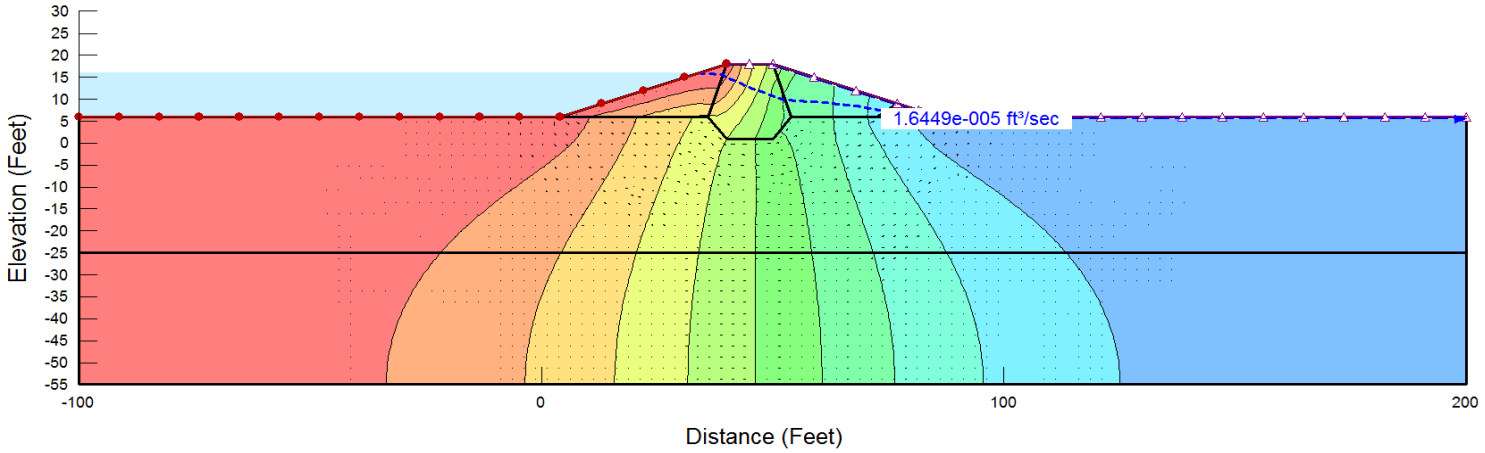
SLOPE STABILITY OF LEVEE ALTERNATIVE
West Kearny Segment

Critical Sliding Surface and Stability FOS

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.11

Name: Soft/Organic Soil (Drained) Model: Saturated Only K-Sat: 3.28e-006 ft/sec Volumetric Water Content: 0 ft³/ft³
Name: Silty Clayey Sand Model: Saturated Only K-Sat: 3.28e-006 ft/sec Volumetric Water Content: 0 ft³/ft³
Name: Shell Model: Saturated / Unsaturated K-Function: shell Vol. WC. Function: VWC-Shell
Name: Core (Undrained) Model: Saturated / Unsaturated K-Function: Core Vol. WC. Function: VWC-Core
Name: Drain Model: Saturated / Unsaturated K-Function: Drain Vol. WC. Function: VWC-Gravelly Sand



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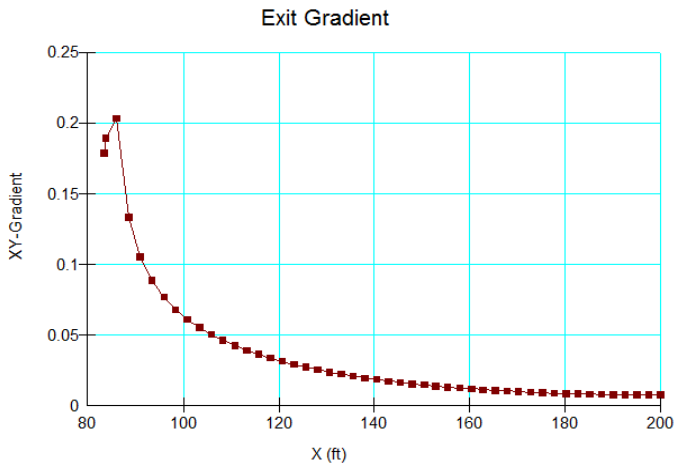
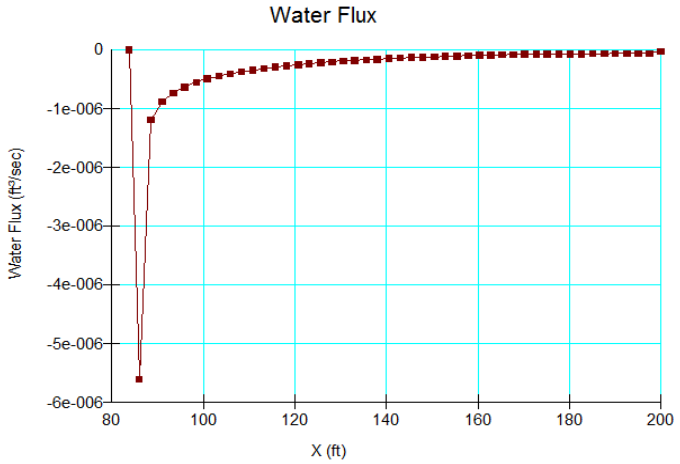




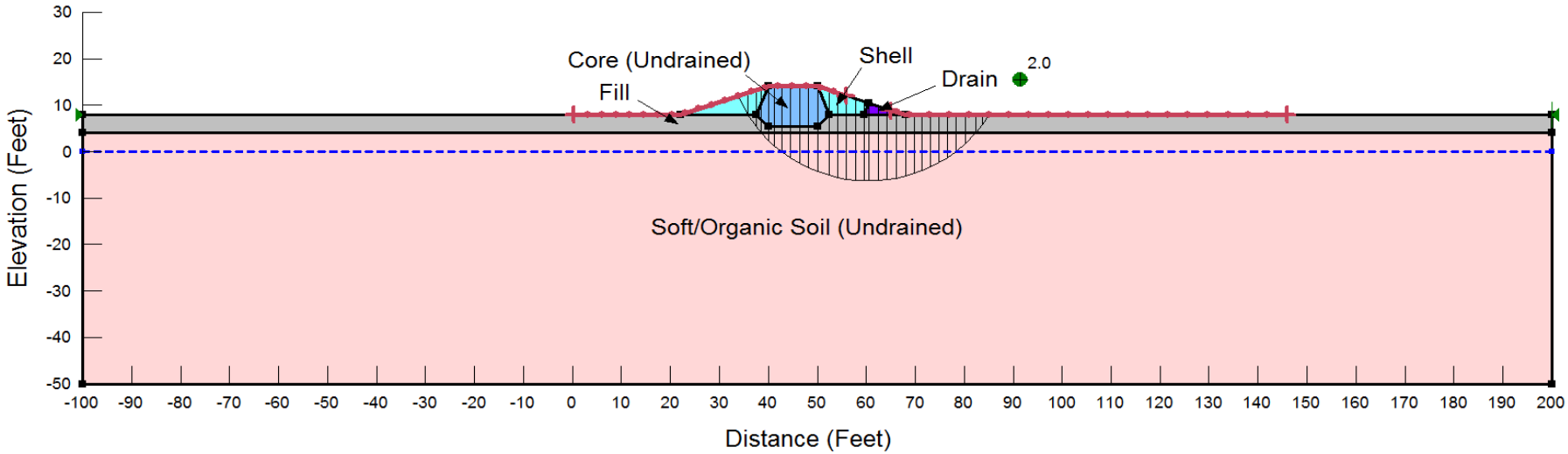
Steady-State Seepage with Flood Pool at El. 16 ft (12' High Levee)

SEEPAGE ANALYSIS OF LEVEE ALTERNATIVE
West Kearny Segment
Estimated Piezometric Conditions, Flow Rate, and Exit Gradient

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.12

Name: Fill Model: Mohr-Coulomb Unit Weight: 110 pcf Cohesion: 0 psf Phi: 30 °
Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Core (Undrained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 1000 psf Phi: 0 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Soft/Organic Soil (Undrained) Model: Mohr-Coulomb Unit Weight: 85 pcf Cohesion: 250 psf Phi: 0 °



Directory: I:\Projects\60442748(Passaic-River)\Calculations\Levee\Seepage&Stability\File Name: Levee@EastKearney_Slope Stability_6ft_tall.gsz

Critical Sliding Surface and Stability FOS for End of Construction (6' High Levee)

same FOS value was estimated for both Upstream and Downstream Slopes

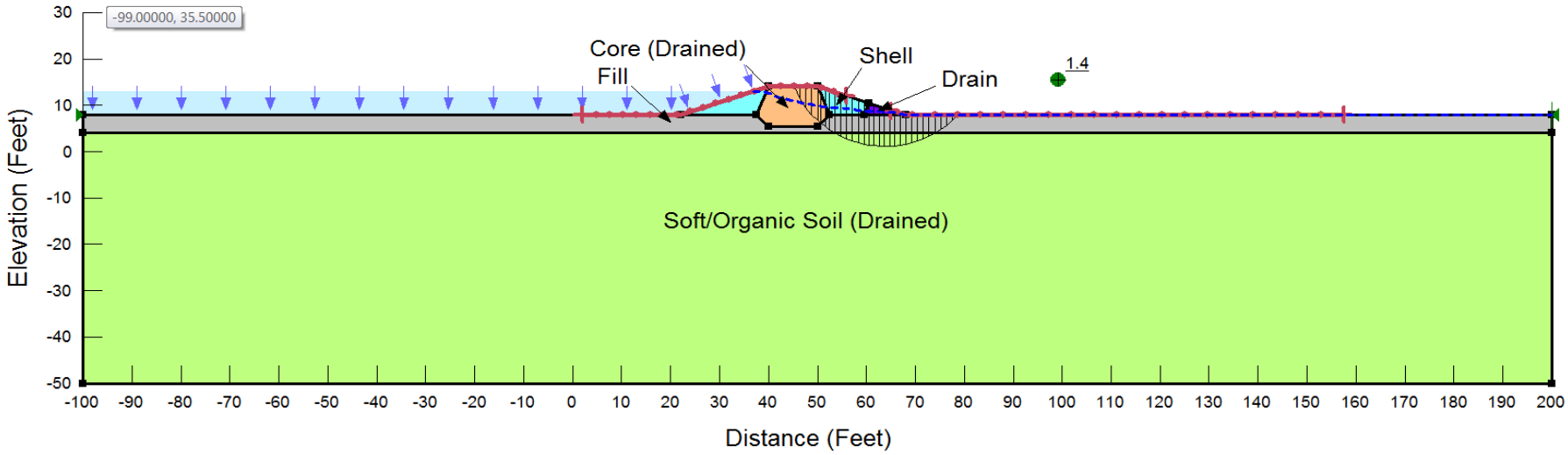
SLOPE STABILITY OF LEVEE ALTERNATIVE
East Kearny Segment

Critical Sliding Surface and Stability FOS

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.13

Name: Soft/Organic Soil (Drained) Model: Mohr-Coulomb Unit Weight: 100 pcf Cohesion: 0 psf Phi: 20 °
Name: Fill Model: Mohr-Coulomb Unit Weight: 110 pcf Cohesion: 0 psf Phi: 30 °
Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Core (Drained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 30 °



Directory: I:\Projects\60442748(Passaic-River)\Calculations\Levee\Seepage&Stability\File Name: Levee@EastKearney_Slope Stability_6ft_tall.gsz

Critical Sliding Surface and Stability FOS for Steady Seepage (6' High Levee)

Steady Seepage with Full Flood Pool (El. +16 Feet)

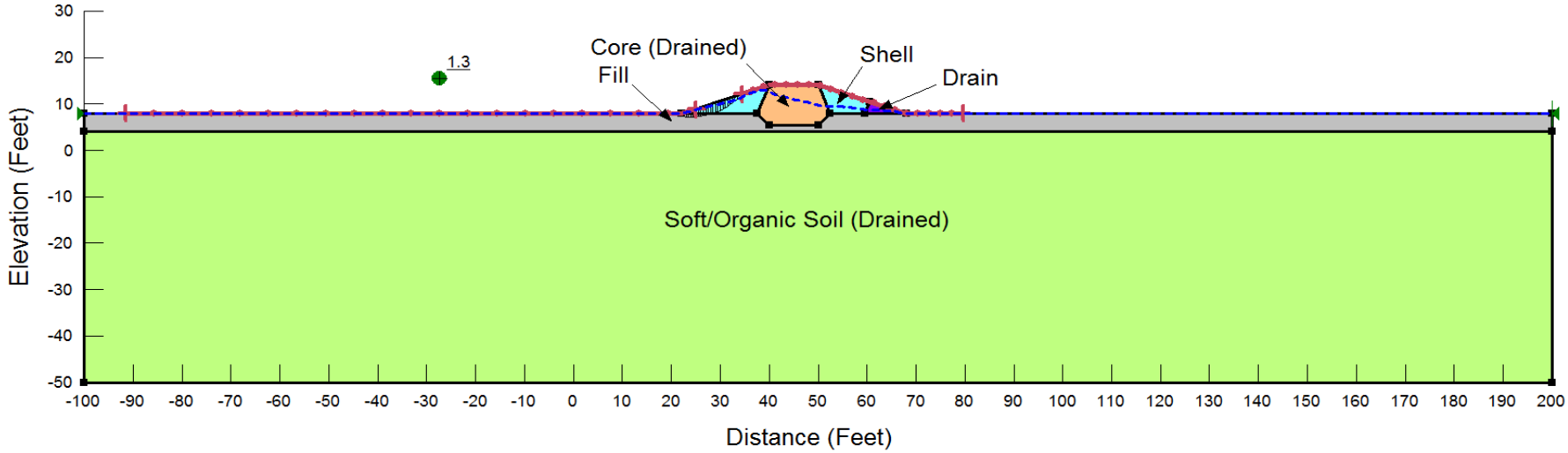
SLOPE STABILITY OF LEVEE ALTERNATIVE
East Kearny Segment

Critical Sliding Surface and Stability FOS

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.14

Name: Soft/Organic Soil (Drained) Model: Mohr-Coulomb Unit Weight: 100 pcf Cohesion: 0 psf Phi: 20 °
Name: Fill Model: Mohr-Coulomb Unit Weight: 110 pcf Cohesion: 0 psf Phi: 30 °
Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Core (Drained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 30 °



Directory: I:\Projects\60442748(Passaic-River)\Calculations\Levee\Seepage&Stability\File Name: Levee@EastKearney_Slope Stability_6ft_tall.gsz

Critical Sliding Surface and Stability FOS for Rapid Drawdown (6' High Levee)

Rapid Drawdown from the Full Flood Stage

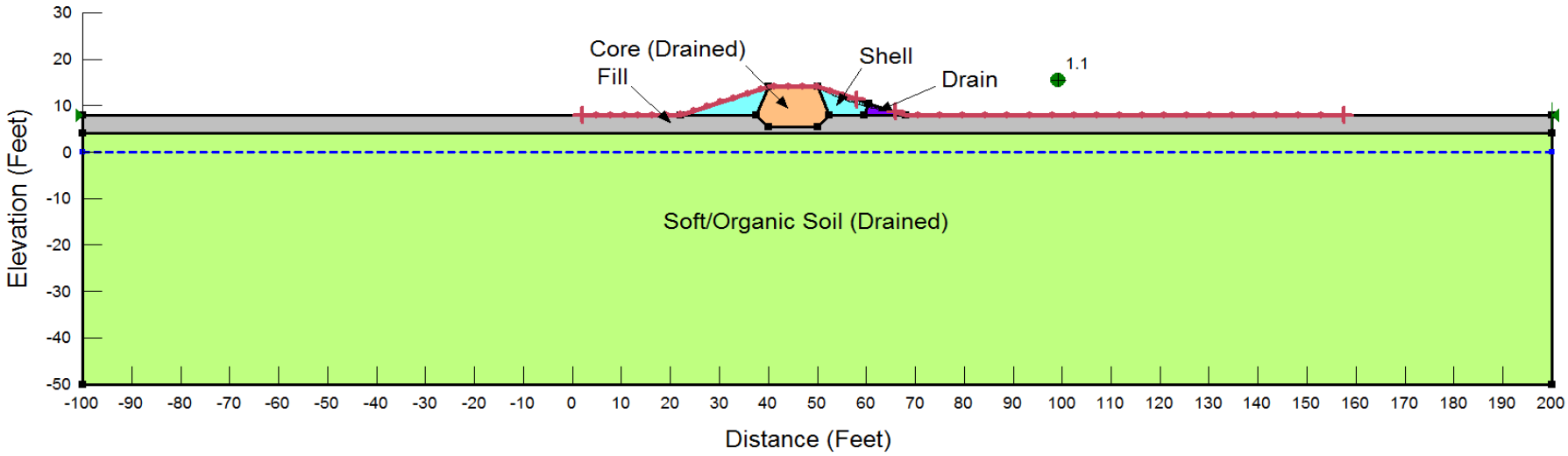
SLOPE STABILITY OF LEVEE ALTERNATIVE
East Kearny Segment

Critical Sliding Surface and Stability FOS

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.15

Name: Soft/Organic Soil (Drained) Model: Mohr-Coulomb Unit Weight: 100 pcf Cohesion: 0 psf Phi: 20 °
Name: Fill Model: Mohr-Coulomb Unit Weight: 110 pcf Cohesion: 0 psf Phi: 30 °
Name: Shell Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 32 °
Name: Drain Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 35 °
Name: Core (Drained) Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion: 0 psf Phi: 30 °



Directory: I:\Projects\60442748(Passaic-River)\Calculations\Levee\Seepage&Stability\File Name: Levee@EastKearney_Slope Stability_6ft_tall.gsz

Critical Sliding Surface and Stability FOS for Seismic Condition (6' High Levee)

Seismic Loading (Pseudo-Static Coefficient of 0.21)

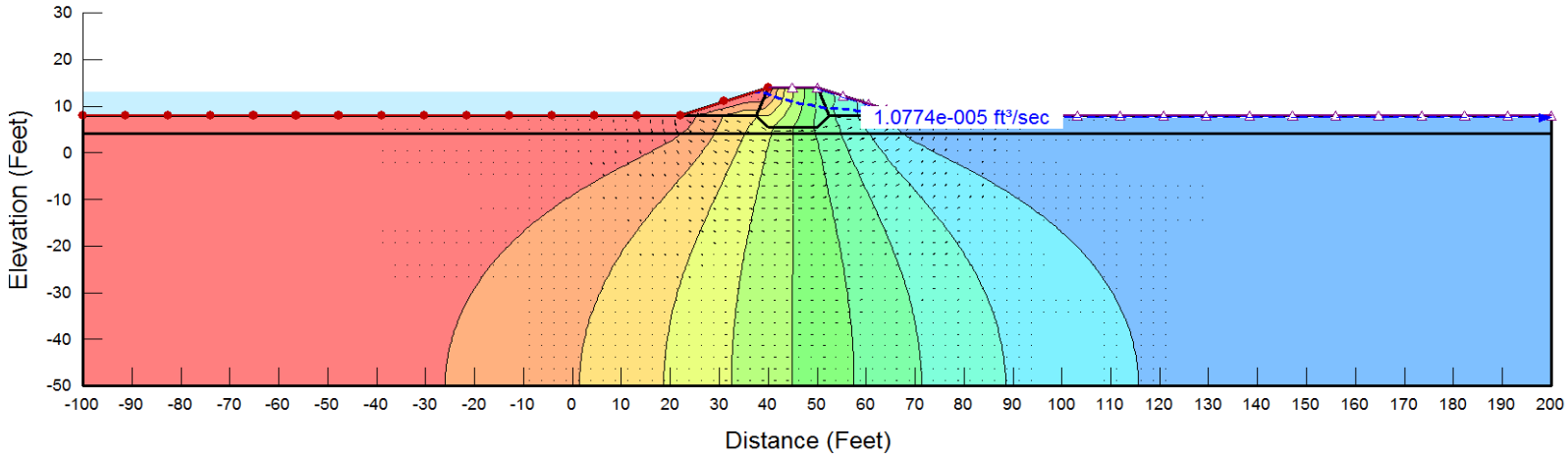
SLOPE STABILITY OF LEVEE ALTERNATIVE
East Kearny Segment

Critical Sliding Surface and Stability FOS

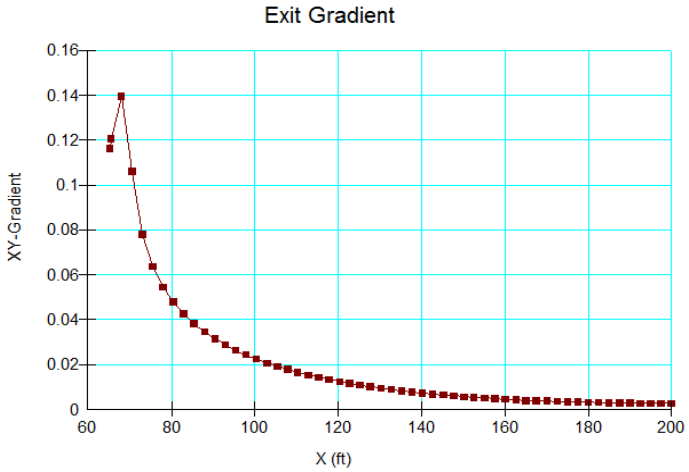
AECOM

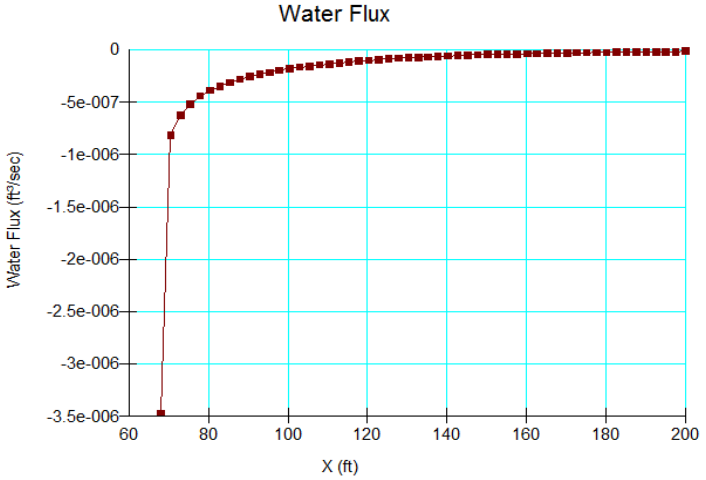
DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.16

Name: Soft/Organic Soil (Drained) Model: Saturated Only K-Sat: 3.28e-006 ft/sec Volumetric Water Content: 0 ft³/ft³
Name: Fill Model: Saturated / Unsaturated K-Function: Fill Vol. WC. Function: VWC-Fill
Name: Shell Model: Saturated / Unsaturated K-Function: shell Vol. WC. Function: VWC-Shell
Name: Core (Undrained) Model: Saturated / Unsaturated K-Function: Core Vol. WC. Function: VWC-Core
Name: Drain Model: Saturated / Unsaturated K-Function: Drain Vol. WC. Function: VWC-Gravelly Sand



Directory: I:\Projects\60442748(Passaic-River)\Calculations\Levee\Seepage&Stability\File Name: Levee@EastKearney_Slope Stability_6ft_tall.gsz





Steady-State Seepage with Flood Pool at El. 13 ft (6' High Levee)

SEEPAGE ANALYSIS OF LEVEE ALTERNATIVE
East Kearny Segment
Estimated Piezometric Conditions, Flow Rate, and Exit Gradient

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.17

Attachment B

LIQUEFACTION EVALUATION

OBJECTIVES

- To determine the factor of safety (FOS) against liquefaction for non-cohesive soils under the groundwater table at the referenced project site in Kearny in New Jersey.

GIVEN INFORMATION AND ASSUMPTIONS

- 9 boring logs reported in the memorandum by US Army Corps of Engineers (USACE 1995).

SEISMIC SITE CLASSIFICATION

The project site was divided into two areas, namely, East Kearny and West Kearny. The seismic site class determination was performed for both the project areas using weighted average standard penetration test (SPT) blow count (N-value) from the USACE 9 borings. Because there is a layer of peat and/or highly organic soil of thickness > 10 ft at most part of both project areas, the seismic site class is determined to be Class E - soft clay soil.

DESIGN EARTHQUAKE MAGNITUDE

A design earthquake magnitude of $M_w = 5.5$ 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.

PEAK GROUND ACCELERATION

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.

LIQUEFACTION EVALUATION METHODOLOGY

In the current 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 ft at the 9 borings. 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, $(N_1)_{60,cs}$ 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_σ. 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 the AASHTO (2014).

RESULTS

Based on the liquefaction evaluation, occasional pockets of potentially liquefiable soils exists in the area of Blocks 1, 9, 10, 11, 14 and 17 shown in Figure 1. The thickness of liquefiable soil pockets ranges from approximately 2 ft at Block 17 to 7 ft at Block 14.

REFERENCES

1. US Army Corps of Engineers (1995). "*General design memorandum: Passaic River flood damage reduction project*". New York.
2. Das, B. M. (2006). *Principles of geotechnical engineering*, Nelson, Ontario, Canada, 686 p.
3. Idriss, I. M., & Boulanger, R. W. (2008). *Soil liquefaction during earthquakes*. Earthquake engineering research institute.
4. "AASHTO LRFD Bridge Design Specifications", 7th ed., American Association of State Highway and Transportation Officials, dated 2014.

PASSAIC RIVER
EAST KEARNY, NJ
SUMMARY OF SOIL BORING RESULTS AND LIQUEFACTION RESISTANCE EVALUATION

DATA COLLECTION

Input by: AH Date: 12/14/15
Ck'd by: Date:
Source:

BORING INFORMATION

Boring No. HLK-1
Coordinates Station Offset
Surface Elev., ft 7.00 ∓ Total Depth, ft 90.4
Drilling Date 9/20/1994 - 9/23/1994

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method MUD ROTARY
Groundwater: Depth, ft 3.0 Elev., ft 4.0 Remarks

2500 yr.
Earthquake Magnitude 5.5
Magnitude Scaling Factor 1.69
G.S. Acc. (%g): 0.32

Depth (ft)	Depth m	Elevation (ft)	Soil Symbol	**Layer[1]**	**N (bpf)**	Ave. Shear Wave Velocity (fps)	**Idealized Sat. Unit Weight (pcf)**	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	**Percent Fines (FC)**	FC Corrected $(N_1)_{60}$	CRR 7.5	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	K_σ	r_d 5.5	CRR 5.5	CSR* 5.50	FS_l CRR/CSR* 5.5
4.00	1.20	3.0	GM	**3**	**11**	599	**90**	360	62	298	1.70	1	3.20	0.8	9	15	**30**	20	0.21	4.0	360	298	0.11	1.00	0.99	0.35	0.25	1.42
5.50	1.65	1.5	GM	**3**	**2**	332	**90**	495	156	339	1.70	1	3.65	0.8	2	3	**30**	8	0.11	5.5	495	339	0.07	1.00	0.98	0.18	0.30	0.60
7.00	2.10	0.0	GM	**3**	**3**	390	**90**	630	250	380	1.70	1	4.10	0.85	3	4	**30**	10	0.12	7.0	630	380	0.07	1.00	0.97	0.20	0.33	0.58
8.50	2.55	-1.5	GM	**3**	**3**	390	**90**	765	343	422	1.70	1	4.55	0.85	3	4	**30**	10	0.12	8.5	765	422	0.07	1.00	0.96	0.20	0.36	0.54
11.50	3.45	-4.5	SM	**3**	**2**	339	**90**	1035	530	505	1.70	1	5.45	0.85	2	3	**30**	8	0.11	11.5	1035	505	0.07	1.00	0.94	0.18	0.40	0.45
13.00	3.90	-6.0	GM	**3**	**2**	339	**90**	1170	624	546	1.70	1	5.90	0.85	2	3	**30**	8	0.11	13.0	1170	546	0.07	1.00	0.93	0.18	0.42	0.43
17.50	5.26	-10.5	SP	**1**	**14**	693	**90**	1575	905	670	1.70	1	7.26	0.95	13	23	**5**	23	0.24	17.5	1575	670	0.15	1.00	0.90	0.41	0.44	0.93
19.00	5.71	-12.0	ML	**4**	**12**	656	**90**	1710	998	712	1.70	1	7.71	0.95	11	19	**50**	25	0.29	19.0	1710	712	0.13	1.00	0.89	0.49	0.44	1.10
20.50	6.16	-13.5	ML	**4**	**6**	514	**90**	1845	1092	753	1.70	1	8.16	0.95	6	10	**50**	15	0.16	20.5	1845	753	0.09	1.00	0.87	0.27	0.45	0.60

[1]Layer Code | Soil Type | Assumed Fines Content (%)

1	GW, GP, SW, SP	5
2	Duel Symbols	10
3	GM, GC, SM, SC	30
4	ML	50

* CSR = $0.65\ \alpha_{max}(\sigma_v/\sigma_v')r_d$

PASSAIC RIVER
WEST KEARNY, NJ
SUMMARY OF SOIL BORING RESULTS AND LIQUEFACTION RESISTANCE EVALUATION

DATA COLLECTION

Input by: AH Date: 12/14/15
Ck'd by: Date:
Source:

BORING INFORMATION

Boring No. HLK-2
Coordinates Station Offset
Surface Elev., ft 8.00 ∓ Total Depth, ft 101.5
Drilling Date 12/5/1994 - 12/7/1994

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method MUD ROTARY

2500 yr.
Earthquake Magnitude 5.5
Magnitude Scaling Factor 1.69
G.S. Acc. (%g): 0.32

Groundwater: Depth, ft 6.0 Elev., ft 2.0 Remarks

Depth (ft)	Depth m	Elevation (ft)	Soil Symbol	**Layer[1]**	**N (bpf)**	Ave. Shear Wave Velocity (fps)	**Idealized Sat. Unit Weight (pcf)**	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	**Percent Fines (FC)**	FC Corrected $(N_1)_{60}$	CRR 7.5	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	K_σ	r_d 5.5	CRR 5.5	CSR^* 5.50	FS_l CRR/CSR^* 5.5
7.00	2.10	1.0	ML	**4**	**30**	874	**90**	630	62	568	1.69	1	4.10	0.85	26	43	**50**	49	0.60	7.0	630	568	0.30	1.00	0.97	0.60	0.22	2.68
10.00	3.00	-2.0	SM	**3**	**6**	495	**90**	900	250	650	1.70	1	5.00	0.85	5	9	**30**	14	0.15	10.0	900	650	0.09	1.00	0.95	0.25	0.27	0.91
11.50	3.45	-3.5	GM	**3**	**2**	339	**90**	1035	343	692	1.70	1	5.45	0.85	2	3	**30**	8	0.11	11.5	1035	692	0.07	1.00	0.94	0.18	0.29	0.61
14.50	4.35	-6.5	SM-SP	**2**	**19**	773	**90**	1305	530	775	1.59	1	6.35	0.95	18	29	**10**	30	0.47	14.5	1305	775	0.19	1.00	0.92	0.79	0.32	2.46
16.00	4.80	-8.0	SM-SP	**2**	**17**	742	**90**	1440	624	816	1.57	1	6.80	0.95	16	25	**10**	27	0.33	16.0	1440	816	0.17	1.00	0.91	0.56	0.33	1.68
17.50	5.26	-9.5	SM-SP	**2**	**16**	727	**90**	1575	718	857	1.55	1	7.26	0.95	15	24	**10**	25	0.28	17.5	1575	857	0.15	1.00	0.90	0.48	0.34	1.39
23.50	7.06	-15.5	SM-SP	**2**	**19**	773	**90**	2115	1092	1023	1.40	1	9.06	0.95	18	25	**10**	26	0.33	23.5	2115	1023	0.16	1.00	0.85	0.55	0.37	1.50
25.00	7.51	-17.0	SM-SP	**2**	**22**	814	**90**	2250	1186	1064	1.35	1	9.51	0.95	21	28	**10**	29	0.44	25.0	2250	1064	0.19	1.00	0.84	0.75	0.37	2.04
26.50	7.96	-18.5	SM-SP	**2**	**19**	773	**90**	2385	1279	1106	1.35	1	9.96	0.95	18	24	**10**	25	0.30	26.5	2385	1106	0.16	1.00	0.83	0.51	0.37	1.37
28.00	8.41	-20.0	SM-SP	**2**	**17**	756	**90**	2520	1373	1147	1.33	1	10.41	1	17	23	**10**	24	0.26	28.0	2520	1147	0.15	1.00	0.81	0.45	0.37	1.20
29.50	8.86	-21.5	SM-SP	**2**	**10**	627	**90**	2655	1466	1189	1.37	1	10.86	1	10	14	**10**	15	0.15	29.5	2655	1189	0.11	1.00	0.80	0.26	0.37	0.70

[1]Layer Code | Soil Type | Assumed Fines Content (%)

Layer Code	Soil Type	Assumed Fines Content (%)
1	GW, GP, SW, SP	5
2	Duel Symbols	10
3	GM, GC, SM, SC	30
4	ML	50

* $CSR = 0.65\ \alpha_{max}(\sigma_v/\sigma_v')\ r_d$

PASSAIC RIVER
WEST KEARNY, NJ
SUMMARY OF SOIL BORING RESULTS AND LIQUEFACTION RESISTANCE EVALUATION

DATA COLLECTION

Input by: AH Date: 12/14/15
Ck'd by: Date:
Source:

BORING INFORMATION

Boring No. HLK-3
Coordinates Station Offset
Surface Elev., ft 7.00 ∓ Total Depth, ft 70.2
Drilling Date 8/10/1994 - 8/16/1994

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method MUD ROTARY

2500 yr.
Earthquake Magnitude 5.5
Magnitude Scaling Factor 1.69
G.S. Acc. (%g): 0.32

Groundwater: Depth, ft 6.0 Elev., ft 1.0 Remarks

Depth (ft)	Depth (m)	Elevation (ft)	Soil Symbol	**Layer**[1]	**N (bpf)**	Ave. Shear Wave Velocity (fps)	**Idealized Sat. Unit Weight (pcf)**	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	**Percent Fines (FC)**	FC Corrected $(N_1)_{60}$	CRR 7.5	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	K_σ	r_d 5.5	CRR 5.5	CSR* 5.50	FS_L CRR/CSR* 5.5
7.00	2.10	0.0	GM	**3**	**38**	952	**90**	630	62	568	1.58	1	4.10	0.85	32	51	**30**	56	0.60	7.0	630	568	0.30	1.00	0.97	0.60	0.22	2.68
10.00	3.00	-3.0	GM	**3**	**15**	683	**90**	900	250	650	1.70	1	5.00	0.85	13	22	**30**	27	0.35	10.0	900	650	0.14	1.00	0.95	0.59	0.27	2.14
11.50	3.45	-4.5	GM	**3**	**24**	807	**90**	1035	343	692	1.63	1	5.45	0.85	20	33	**30**	39	0.60	11.5	1035	692	0.24	1.00	0.94	0.60	0.29	2.05
14.50	4.35	-7.5	SM	**3**	**18**	758	**90**	1305	530	775	1.60	1	6.35	0.95	17	27	**30**	33	0.60	14.5	1305	775	0.18	1.00	0.92	0.60	0.32	1.86
16.00	4.80	-9.0	SM	**3**	**19**	773	**90**	1440	624	816	1.55	1	6.80	0.95	18	28	**30**	33	0.60	16.0	1440	816	0.18	1.00	0.91	0.60	0.33	1.80
19.00	5.71	-12.0	GM	**3**	**15**	710	**90**	1710	811	899	1.53	1	7.71	0.95	14	22	**30**	27	0.35	19.0	1710	899	0.14	1.00	0.89	0.59	0.35	1.69
23.50	7.06	-16.5	GM	**3**	**25**	852	**90**	2115	1092	1023	1.35	1	9.06	0.95	24	32	**30**	37	0.60	23.5	2115	1023	0.22	1.00	0.85	0.60	0.37	1.64
26.50	7.96	-19.5	GM	**3**	**21**	801	**90**	2385	1279	1106	1.33	1	9.96	0.95	20	27	**30**	32	0.60	26.5	2385	1106	0.17	1.00	0.83	0.60	0.37	1.62
46.00	13.81	-39.0	SM	**3**	**22**	829	**90**	4140	2496	1644	1.11	1	15.81	1	22	25	**30**	30	0.48	46.0	4140	1644	0.16	1.00	0.67	0.80	0.35	2.30
51.00	15.32	-44.0	SM	**3**	**75**	1295	**90**	4590	2808	1782	1.02	1	17.32	1	75	77	**30**	82	0.60	51.0	4590	1782	0.30	1.00	0.63	0.60	0.34	1.78

[1]Layer Code | Soil Type | Assumed Fines Content (%)
1 | GW, GP, SW, SP | 5
2 | Duel Symbols | 10
3 | GM, GC, SM, SC | 30
4 | ML | 50

* CSR = 0.65 $\alpha_{max}(\sigma_v/\sigma_v')\, r_d$

PASSAIC RIVER
WEST KEARNY, NJ
SUMMARY OF SOIL BORING RESULTS AND LIQUEFACTION RESISTANCE EVALUATION

DATA COLLECTION

Input by: AH Date: 12/14/15
Ck'd by: Date:
Source:

BORING INFORMATION

Boring No. HLK-4
Coordinates Station Offset
Surface Elev., ft 9.00 ∓ Total Depth, ft 504.7
Drilling Date 7/21/1994 - 8/30/1994

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method MUD ROTARY

Earthquake Magnitude 2500 yr.
Magnitude Scaling Factor 5.5
G.S. Acc. (%g): 1.69
0.32

Groundwater: Depth, ft 9.0 Elev., ft 0.0 Remarks

Depth (ft)	Depth m	Elevation (ft)	Soil Symbol	**Layer**[1]	**N** (bpf)	Ave. Shear Wave Velocity (fps)	**Idealized Sat. Unit Weight** (pcf)	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	**Percent Fines (FC)**	FC Corrected $(N_1)_{60}$	CRR 7.5	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	K_{σ}	r_d 5.5	CRR 5.5	CSR* 5.50	FS_L CRR/CSR* 5.5
32.50	9.76	-23.5	SM	**3**	**15**	723	**90**	2925	1466	1459	1.20	1	11.76	1	15	18	**30**	23	0.26	32.5	2925	1459	0.12	1.00	0.78	0.43	0.32	1.33
36.00	10.81	-27.0	SM	**3**	**35**	980	**90**	3240	1685	1555	1.11	1	12.81	1	35	39	**30**	44	0.60	36.0	3240	1555	0.30	1.00	0.75	0.60	0.32	1.85
46.00	13.81	-37.0	ML	**4**	**21**	815	**90**	4140	2309	1831	1.07	1	15.81	1	21	22	**50**	28	0.38	46.0	4140	1831	0.15	1.00	0.67	0.65	0.31	2.06

[1]Layer Code | Soil Type | Assumed Fines Content (%)
1 | GW, GP, SW, SP | 5
2 | Duel Symbols | 10
3 | GM, GC, SM, SC | 30
4 | ML | 50

* CSR = 0.65 $\alpha_{max}(\sigma_v/\sigma_v')\, r_d$

PASSAIC RIVER
WEST KEARNY, NJ
SUMMARY OF SOIL BORING RESULTS AND LIQUEFACTION RESISTANCE EVALUATION

DATA COLLECTION

Input by: AH Date: 12/14/15
Ck'd by: Date:
Source:

BORING INFORMATION

Boring No. HLK-5
Coordinates Station Offset
Surface Elev., ft 7.50 ∓ Total Depth, ft 51.5
Drilling Date 9/16/1994 - 9/19/1994

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method MUD ROTARY

2500 yr.
Earthquake Magnitude 5.5
Magnitude Scaling Factor 1.69
G.S. Acc. (%g): 0.32

Groundwater: Depth, ft 5.0 Elev., ft 1.5 Remarks

Depth (ft)	Depth m	Elevation (ft)	Soil Symbol	**Layer**[1]	**N (bpf)**	Ave. Shear Wave Velocity (fps)	**Idealized Sat. Unit Weight (pcf)**	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	**Percent Fines (FC)**	FC Corrected $(N_1)_{60}$	CRR 7.5	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	K_σ	r_d 5.5	CRR 5.5	CSR* 5.50	FS_l CRR/CSR* 5.5
7.00	2.10	0.5	GM	**3**	**18**	728	**90**	630	125	505	1.70	1	4.10	0.85	15	26	**30**	31	0.59	7.0	630	505	0.17	1.00	0.97	0.99	0.25	3.92
10.00	3.00	-2.5	SM	**3**	**5**	465	**90**	900	312	588	1.70	1	5.00	0.85	4	7	**30**	13	0.14	10.0	900	588	0.08	1.00	0.95	0.23	0.30	0.76
11.50	3.45	-4.0	GM	**3**	**18**	728	**90**	1035	406	629	1.70	1	5.45	0.85	15	26	**30**	31	0.59	11.5	1035	629	0.17	1.00	0.94	0.99	0.32	3.07
13.00	3.90	-5.5	GM	**3**	**13**	649	**90**	1170	499	671	1.70	1	5.90	0.85	11	19	**30**	24	0.27	13.0	1170	671	0.13	1.00	0.93	0.46	0.34	1.35
14.50	4.35	-7.0	SM	**3**	**3**	405	**90**	1305	593	712	1.70	1	6.35	0.95	3	5	**30**	10	0.12	14.5	1305	712	0.08	1.00	0.92	0.20	0.35	0.57
25.00	7.51	-17.5	SM	**4**	**5**	483	**90**	2250	1248	1002	1.59	1	9.51	0.95	5	8	**50**	13	0.14	25.0	2250	1002	0.08	1.00	0.84	0.24	0.39	0.61
27.00	8.11	-19.5	SM	**4**	**13**	687	**90**	2430	1373	1057	1.42	1	10.11	1	13	19	**50**	24	0.27	27.0	2430	1057	0.13	1.00	0.82	0.46	0.39	1.16
41.00	12.31	-33.5	SW	**4**	**14**	706	**90**	3690	2246	1444	1.21	1	14.31	1	14	17	**50**	23	0.24	41.0	3690	1444	0.12	1.00	0.71	0.41	0.38	1.09
46.00	13.81	-38.5	ML	**4**	**17**	756	**90**	4140	2558	1582	1.15	1	15.81	1	17	19	**50**	25	0.29	46.0	4140	1582	0.13	1.00	0.67	0.49	0.36	1.36
51.00	15.32	-43.5	SM	**3**	**46**	1083	**90**	4590	2870	1720	1.06	1	17.32	1	46	49	**30**	54	0.60	51.0	4590	1720	0.30	1.00	0.63	0.60	0.35	1.72

[1]Layer Code	Soil Type	Assumed Fines Content (%)
1	GW, GP, SW, SP	5
2	Duel Symbols	10
3	GM, GC, SM, SC	30
4	ML	50

* CSR = 0.65 $\alpha_{max}(\sigma_v/\sigma_v')r_d$

PASSAIC RIVER
WEST KEARNY, NJ
SUMMARY OF SOIL BORING RESULTS AND LIQUEFACTION RESISTANCE EVALUATION

DATA COLLECTION

Input by: AH Date: 12/14/15
Ck'd by: Date:
Source:

BORING INFORMATION

Boring No. DC-5
Coordinates Station Offset
Surface Elev., ft 8.90 ∓ Total Depth, ft 510.2
Drilling Date

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method MUD ROTARY

2500 yr.
Earthquake Magnitude 5.5
Magnitude Scaling Factor 1.69
G.S. Acc. (%g): 0.32

Groundwater: Depth, ft 8.9 Elev., ft 0.0 Remarks

Depth (ft)	Depth m	Elevation (ft)	Soil Symbol	**Layer[1]**	**N (bpf)**	Ave. Shear Wave Velocity (fps)	**Idealized Sat. Unit Weight (pcf)**	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	**Percent Fines (FC)**	FC Corrected $(N_1)_{60}$	CRR 7.5	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	K_σ	r_d 5.5	CRR 5.5	CSR* 5.50	FS_l CRR/CSR* 5.5
23.00	6.91	-14.1	SM	**3**	**12**	656	**90**	2070	880	1190	1.35	1	8.91	0.95	11	15	**30**	21	0.22	23.0	2070	1190	0.11	1.00	0.85	0.36	0.31	1.18
39.00	11.71	-30.1	SM	**3**	**9**	604	**90**	3510	1878	1632	1.16	1	13.71	1	9	10	**30**	16	0.16	39.0	3510	1632	0.09	1.00	0.72	0.27	0.32	0.85
43.00	12.91	-34.1	SP-SM	**2**	**26**	880	**90**	3870	2128	1742	1.08	1	14.91	1	26	28	**10**	29	0.44	43.0	3870	1742	0.19	1.00	0.69	0.74	0.32	2.33

[1]Layer Code | Soil Type | Assumed Fines Content (%)
1 | GW, GP, SW, SP | 5
2 | Duel Symbols | 10
3 | GM, GC, SM, SC | 30
4 | ML | 50

* CSR = 0.65 $\alpha_{max}(\sigma_v/\sigma_v')r_d$

PASSAIC RIVER
WEST KEARNY, NJ
SUMMARY OF SOIL BORING RESULTS AND LIQUEFACTION RESISTANCE EVALUATION

DATA COLLECTION

Input by: AH Date: 12/14/15
Ck'd by: Date:
Source:

BORING INFORMATION

Boring No. HLD-1
Coordinates Station Offset
Surface Elev., ft 7.00 ∓ Total Depth, ft 90.4
Drilling Date 9/20/1994 - 9/23/1994

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method MUD ROTARY

Groundwater: Depth, ft 3.0 Elev., ft 4.0 Remarks

2500 yr.
Earthquake Magnitude 5.5
Magnitude Scaling Factor 1.69
G.S. Acc. (%g): 0.32

Depth (ft)	Depth m	Elevation (ft)	Soil Symbol	**Layer[1]**	**N (bpf)**	Ave. Shear Wave Velocity (fps)	**Idealized Sat. Unit Weight (pcf)**	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	**Percent Fines (FC)**	FC Corrected $(N_1)_{60}$	CRR 7.5	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	K_σ	r_d 5.5	CRR 5.5	CSR* 5.50	FS_L CRR/CSR* 5.5
4.00	1.20	3.0	SM	**3**	**5**	455	**90**	360	62	298	1.70	1	3.20	0.8	4	7	**30**	12	0.13	4.0	360	298	0.08	1.00	0.99	0.23	0.25	0.91
5.50	1.65	1.5	SM	**3**	**2**	332	**90**	495	156	339	1.70	1	3.65	0.8	2	3	**30**	8	0.11	5.5	495	339	0.07	1.00	0.98	0.18	0.30	0.60
7.00	2.10	0.0	SM	**3**	**0**	0	**90**	630	250	380	1.70	1	4.10	0.85	0	0	**30**	5	0.09	7.0	630	380	0.05	1.00	0.97	0.15	0.33	0.44
17.50	5.26	-10.5	SM	**3**	**4**	447	**90**	1575	905	670	1.70	1	7.26	0.95	4	6	**30**	12	0.13	17.5	1575	670	0.08	1.00	0.90	0.22	0.44	0.50
19.00	5.71	-12.0	SM	**3**	**3**	405	**90**	1710	998	712	1.70	1	7.71	0.95	3	5	**30**	10	0.12	19.0	1710	712	0.08	1.00	0.89	0.20	0.44	0.45
20.50	6.16	-13.5	SM	**3**	**3**	405	**90**	1845	1092	753	1.70	1	8.16	0.95	3	5	**30**	10	0.12	20.5	1845	753	0.08	1.00	0.87	0.20	0.45	0.45
22.00	6.61	-15.0	SM	**3**	**3**	405	**90**	1980	1186	794	1.70	1	8.61	0.95	3	5	**30**	10	0.12	22.0	1980	794	0.08	1.00	0.86	0.20	0.45	0.45
25.00	7.51	-18.0	SM	**3**	**10**	615	**90**	2250	1373	877	1.62	1	9.51	0.95	10	15	**30**	21	0.22	25.0	2250	877	0.11	1.00	0.84	0.36	0.45	0.81
26.50	7.96	-19.5	SM	**3**	**8**	569	**90**	2385	1466	919	1.61	1	9.96	0.95	8	12	**30**	18	0.18	26.5	2385	919	0.10	1.00	0.83	0.30	0.45	0.68
28.00	8.41	-21.0	SM	**3**	**14**	706	**90**	2520	1560	960	1.48	1	10.41	1	14	21	**30**	26	0.32	28.0	2520	960	0.14	1.00	0.81	0.54	0.44	1.21
29.50	8.86	-22.5	SW	**1**	**16**	740	**90**	2655	1654	1001	1.43	1	10.86	1	16	23	**5**	23	0.25	29.5	2655	1001	0.15	1.00	0.80	0.42	0.44	0.94
36.00	10.81	-29.0	SW	**1**	**27**	893	**120**	3435	2059	1376	1.18	1	12.81	1	27	32	**5**	32	0.60	36.0	3435	1376	0.22	1.00	0.75	0.60	0.39	1.55
41.00	12.31	-34.0	ML	**4**	**22**	829	**120**	4035	2371	1664	1.11	1	14.31	1	22	24	**50**	30	0.48	41.0	4035	1664	0.16	1.00	0.71	0.82	0.36	2.29

[1]Layer Code | Soil Type | Assumed Fines Content (%)

1	GW, GP, SW, SP	5
2	Duel Symbols	10
3	GM, GC, SM, SC	30
4	ML	50

* CSR = 0.65 $\alpha_{max}(\sigma_v/\sigma_v')r_d$

PASSAIC RIVER
WEST KEARNY, NJ
SUMMARY OF SOIL BORING RESULTS AND LIQUEFACTION RESISTANCE EVALUATION

DATA COLLECTION

Input by: AH Date: 12/14/15
Ck'd by: Date:
Source:

BORING INFORMATION

Boring No. HLD-2
Coordinates Station Offset
Surface Elev., ft 12.00 ∓ Total Depth, ft 22.5
Drilling Date 8/3/1994 - 8/4/1994

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method MUD ROTARY

2500 yr.
Earthquake Magnitude 5.5
Magnitude Scaling Factor 1.69
G.S. Acc. (%g): 0.32

Groundwater: Depth, ft 8.0 Elev., ft 4.0 Remarks

Depth (ft)	Depth m	Elevation (ft)	Soil Symbol	**Layer[1]**	**N (bpf)**	Ave. Shear Wave Velocity (fps)	**Idealized Sat. Unit Weight (pcf)**	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	**Percent Fines (FC)**	FC Corrected $(N_1)_{60}$	CRR 7.5	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	K_σ	r_d 5.5	CRR 5.5	CSR* 5.50	FS_l CRR/CSR* 5.5
8.50	2.55	3.5	GM	**3**	**50**	1052	**90**	765	31	734	1.35	1	4.55	0.85	43	57	**30**	63	0.60	8.5	765	734	0.30	1.00	0.96	0.60	0.21	2.88
9.50	2.85	2.5	GM	**3**	**50**	1052	**90**	855	94	761	1.34	1	4.85	0.85	43	57	**30**	62	0.60	9.5	855	761	0.30	1.00	0.96	0.60	0.22	2.69
11.00	3.30	1.0	GM	**3**	**61**	1131	**90**	990	187	803	1.25	1	5.30	0.85	52	65	**30**	70	0.60	11.0	990	803	0.30	1.00	0.95	0.60	0.24	2.47
19.00	5.71	-7.0	GM	**3**	**24**	840	**90**	1710	686	1024	1.36	1	7.71	0.95	23	31	**30**	36	0.60	19.0	1710	1024	0.21	1.00	0.89	0.60	0.31	1.95
20.50	6.16	-8.5	GM	**3**	**15**	710	**90**	1845	780	1065	1.41	1	8.16	0.95	14	20	**30**	25	0.30	20.5	1845	1065	0.13	1.00	0.87	0.51	0.32	1.60

[1]Layer Code | Soil Type | Assumed Fines Content (%)
1 | GW, GP, SW, SP | 5
2 | Duel Symbols | 10
3 | GM, GC, SM, SC | 30
4 | ML | 50

* CSR = 0.65 $\alpha_{max}(\sigma_v/\sigma_v')r_d$

PASSAIC RIVER
WEST KEARNY, NJ
SUMMARY OF SOIL BORING RESULTS AND LIQUEFACTION RESISTANCE EVALUATION

DATA COLLECTION

Input by: AH Date: 12/14/15
Ck'd by: Date:
Source:

BORING INFORMATION

Boring No. HLS-1
Coordinates Station Offset
Surface Elev., ft 6.50 ∓ Total Depth, ft 61.2
Drilling Date 11/14/1994 - 11/18/1994

Existing Height of Embankment (ft) 0
Proposed Height of Embankment (ft) 0

SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method MUD ROTARY

Groundwater: Depth, ft 5.0 Elev., ft 1.5 Remarks

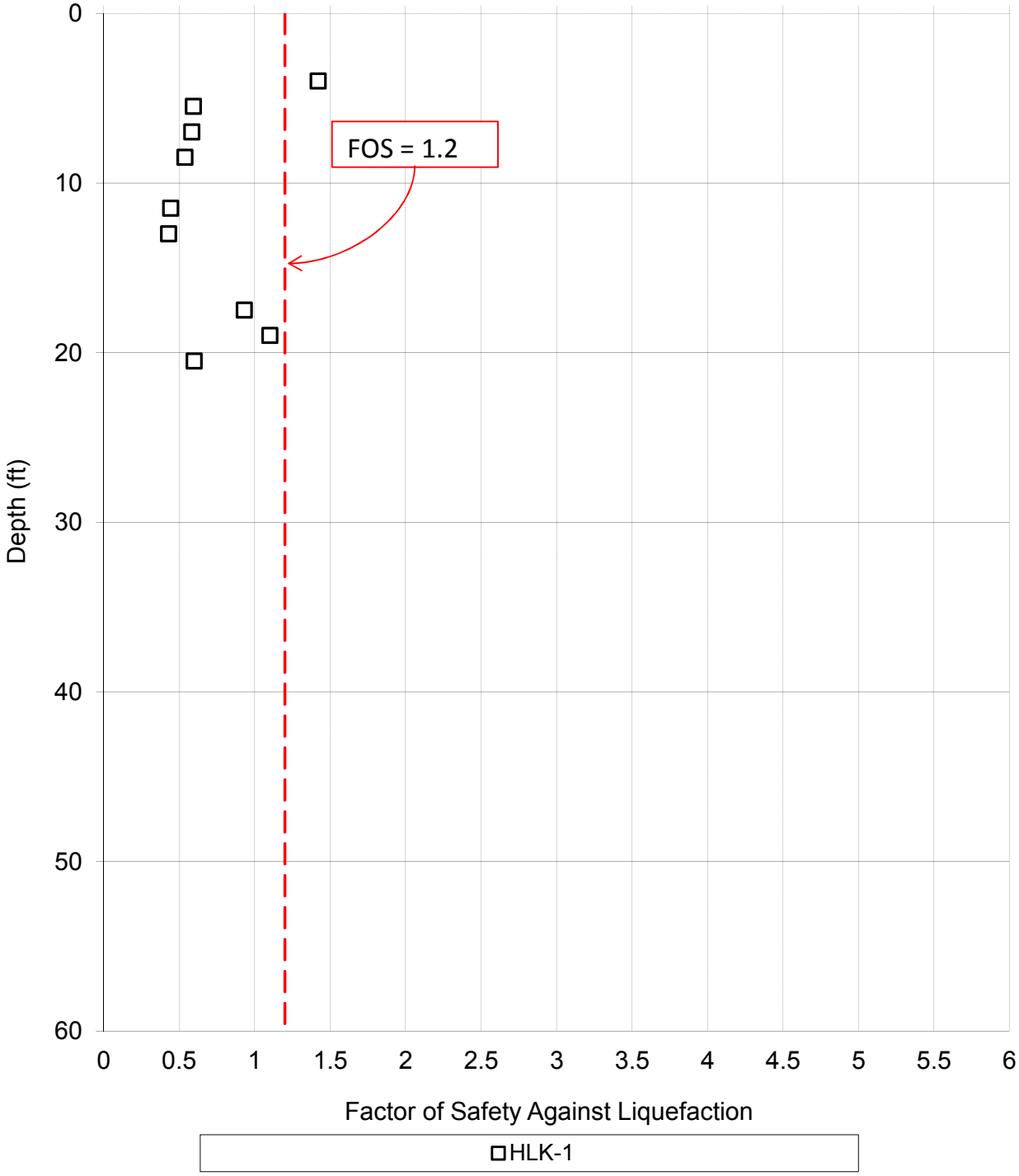
2500 yr.
Earthquake Magnitude 5.5
Magnitude Scaling Factor 1.69
G.S. Acc. (%g): 0.32

Depth (ft)	Depth m	Elevation (ft)	Soil Symbol	**Layer Code[1]**	**N (bpf)**	Ave. Shear Wave Velocity (fps)	**Idealized Sat. Unit Weight (pcf)**	Total Overburden Stress (psf)	Pore Pressure (psf)	Effective Overburden Stress (psf)	C_N	Hammer Energy Correction	Rod Length (m)	Corrction for Rod Length	N_{60} (bpf)	$(N_1)_{60}$ (bpf)	**Percent Fines (FC)**	FC Corrected $(N_1)_{60}$	CRR 7.5	Depth Below top of Embank. (ft)	Proposed Total Overburden Stress (psf)	Proposed Effective Overburden Stress (psf)	Cσ	K_σ	r_d 5.5	CRR 5.5	CSR* 5.50	FS_l CRR/CSR* 5.5
5.50	1.65	1.0	ML	**4**	**24**	790	**90**	495	31	464	1.70	1	3.65	0.8	19	33	**50**	38	0.60	5.5	495	464	0.23	1.00	0.98	0.60	0.22	2.76
7.00	2.10	-0.5	SM	**3**	**18**	728	**90**	630	125	505	1.70	1	4.10	0.85	15	26	**30**	31	0.59	7.0	630	505	0.17	1.00	0.97	0.99	0.25	3.92
8.50	2.55	-2.0	ML	**4**	**10**	592	**90**	765	218	547	1.70	1	4.55	0.85	9	14	**50**	20	0.21	8.5	765	547	0.11	1.00	0.96	0.35	0.28	1.24
10.00	3.00	-3.5	ML	**4**	**8**	547	**90**	900	312	588	1.70	1	5.00	0.85	7	12	**50**	17	0.18	10.0	900	588	0.10	1.00	0.95	0.30	0.30	0.98
11.50	3.45	-5.0	ML	**4**	**6**	495	**90**	1035	406	629	1.70	1	5.45	0.85	5	9	**50**	14	0.15	11.5	1035	629	0.09	1.00	0.94	0.25	0.32	0.79
14.50	4.35	-8.0	ML	**4**	**3**	405	**90**	1305	593	712	1.70	1	6.35	0.95	3	5	**50**	10	0.12	14.5	1305	712	0.08	1.00	0.92	0.20	0.35	0.58
16.00	4.80	-9.5	SP	**1**	**19**	773	**120**	1485	686	799	1.56	1	6.80	0.95	18	28	**5**	28	0.39	16.0	1485	799	0.19	1.00	0.91	0.66	0.35	1.88
17.50	5.26	-11.0	SM	**3**	**36**	972	**120**	1665	780	885	1.34	1	7.26	0.95	34	46	**30**	51	0.60	17.5	1665	885	0.30	1.00	0.90	0.60	0.35	1.71
19.00	5.71	-12.5	SM	**3**	**48**	1079	**120**	1845	874	971	1.23	1	7.71	0.95	46	56	**30**	61	0.60	19.0	1845	971	0.30	1.00	0.89	0.60	0.35	1.71
20.50	6.16	-14.0	SM	**3**	**26**	864	**120**	2025	967	1058	1.32	1	8.16	0.95	25	33	**30**	38	0.60	20.5	2025	1058	0.23	1.00	0.87	0.60	0.35	1.72
22.00	6.61	-15.5	SM-SP	**2**	**36**	972	**120**	2205	1061	1144	1.23	1	8.61	0.95	34	42	**10**	43	0.60	22.0	2205	1144	0.30	1.00	0.86	0.60	0.35	1.74
23.00	6.91	-16.5	SM	**3**	**46**	1062	**120**	2325	1123	1202	1.17	1	8.91	0.95	44	51	**30**	56	0.60	23.0	2325	1202	0.30	1.00	0.85	0.60	0.34	1.74
25.00	7.51	-18.5	SM	**3**	**35**	962	**120**	2565	1248	1317	1.18	1	9.51	0.95	33	39	**30**	44	0.60	25.0	2565	1317	0.30	1.00	0.84	0.60	0.34	1.77
26.50	7.96	-20.0	SM	**3**	**51**	1103	**120**	2745	1342	1403	1.11	1	9.96	0.95	48	54	**30**	59	0.60	26.5	2745	1403	0.30	1.00	0.83	0.60	0.34	1.79
28.00	8.41	-21.5	GM	**3**	**41**	1038	**120**	2925	1435	1490	1.11	1	10.41	1	41	45	**30**	51	0.60	28.0	2925	1490	0.30	1.00	0.81	0.60	0.33	1.81
29.50	8.86	-23.0	GM	**3**	**29**	916	**120**	3105	1529	1576	1.12	1	10.86	1	29	32	**30**	38	0.60	29.5	3105	1576	0.23	1.00	0.80	0.60	0.33	1.83
31.00	9.31	-24.5	GM	**3**	**34**	970	**120**	3285	1622	1663	1.09	1	11.31	1	34	37	**30**	42	0.60	31.0	3285	1663	0.29	1.00	0.79	0.60	0.32	1.85
36.00	10.81	-29.5	GM	**3**	**46**	1083	**120**	3885	1934	1951	1.02	1	12.81	1	46	47	**30**	52	0.60	36.0	3885	1951	0.30	1.00	0.75	0.60	0.31	1.94
41.00	12.31	-34.5	GM	**3**	**100**	1440	**120**	4485	2246	2239	1.00	1	14.31	1	100	100	**30**	105	0.60	41.0	4485	2239	0.30	0.98	0.71	0.60	0.29	2.04
47.00	14.11	-40.5	ML-GM	**2**	**100**	1440	**120**	5205	2621	2584	1.00	1	16.11	1	100	100	**10**	101	0.60	47.0	5205	2584	0.30	0.94	0.66	0.60	0.28	2.17
51.00	15.32	-44.5	SM	**3**	**70**	1262	**120**	5685	2870	2815	0.96	1	17.32	1	70	67	**30**	73	0.60	51.0	5685	2815	0.30	0.92	0.63	0.60	0.26	2.27

[1]Layer Code

Layer Code	Soil Type	Assumed Fines Content (%)
1	GW, GP, SW, SP	5
2	Duel Symbols	10
3	GM, GC, SM, SC	30
4	ML	50

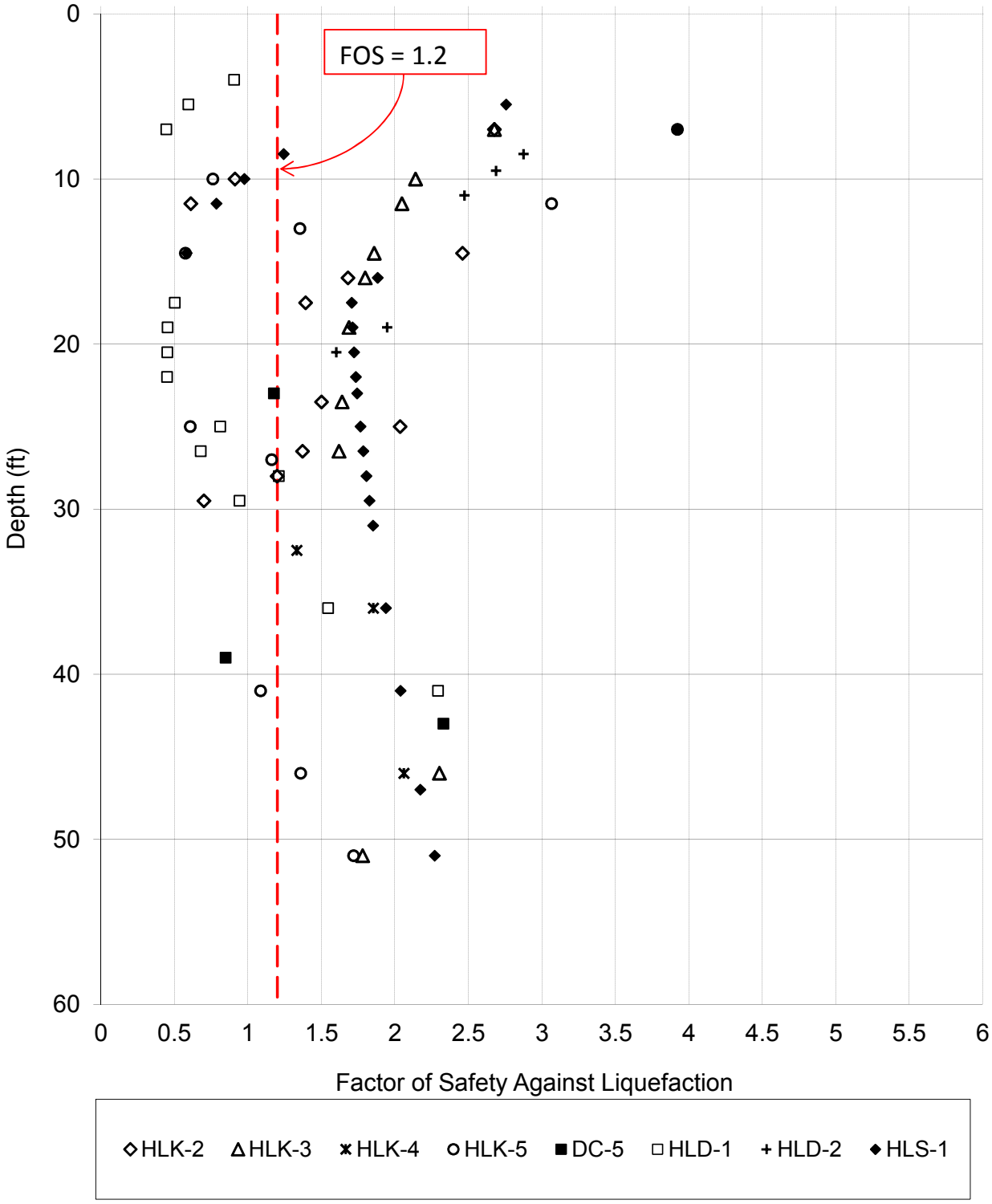
* CSR = 0.65 $\alpha_{max}(\sigma_v/\sigma_v')r_d$



Site Specific Liquefaction Analysis
PASSAIC RIVER
EAST KEARNY, NJ

AECOM
CLIFTON, NEW JERSEY

DR. BY: AH	SCALE: - NTS	PROJ. NO. : 60442748
Ck'D. BY: SK	DATE: DEC. 2015	FIG NO: B.1



Site Specific Liquefaction Analysis
PASSAIC RIVER
WEST KEARNY, NJ

AECOM
CLIFTON, NEW JERSEY

DR. BY: AH	SCALE: - NTS	PROJ. NO. : 60442748
Ck'D. BY: SK	DATE: DEC. 2015	FIG NO: B.2

Attachment C

LEVEE CONSOLIDATION SETTLEMENT ANALYSIS

OBJECTIVES

- To estimate the primary consolidation settlement for proposed levee.

GIVEN INFORMATION AND ASSUMPTIONS

- The height of proposed embankment is 6 ft.
- The soil profile consists of 4 ft of suitable fill, 51 ft of soft/organic soil, and 30 ft of silty clay.
- All soft/organic soil and silty clay are normally consolidated.
- Conservatively, East Kearny soil profile was considered for primary conslidation settlement calculation since the thickness of clayey layers at West Kearny Segment is lower than the East Kearny Segment.

METHODOLOGY

The consolidation parameters used for the silty clay stratum were obtained from the results of a consolidation test performed on undisturbed samples reported in the memorandum by US Army Corps of Engineers (USACE 1995). Consolidation parameters for the soft/organic soil stratum as recommended in USACE 1995 memorandum was used in this calculation.

The generalized soil profile used in this report for the settlement analysis of the levee system considered that the soft/organic soil stratum extends to a depth of 55 ft from the existing ground surface. However, spatial variability at the site exists as evident from the available boring logs. For example, the undisturbed sample representative of silty clay stratum was taken from the depths of 28 to 31 ft at Boring HLK-1. To avoid an overly conservative settlement estimate, the soft/organic stratum was divided into two layers;1) the 12 ft top layer was assigned the same soil unit weight and consolidation parameters as the soft/organic stratum; and 2) the 39 ft bottom layer was assigned the same consolidation parameters as the silty clay stratum, while the soil unit weight remaining the same as the soft/organic stratum.

This report recommends excavating a 4 ft deep inspection trench along the centerline of the levee prior to construction to evaluate the existing fill for use as a foundation. If the existing fill is found to be intermixed with unsuitable devaying organic material, debris, woods, metal and general building demolition rubble, then it is proposed that the top 4 ft of the existing fill to be removed and replaced by a new compacted structural fill.

In this calculation, the compressible soil layers were divided into sub-layers of 2 feet thicknesses for obtaining better accuracy of incremental settlement. Increase in vertical stresses at the mid depth of each sub-layer due to the embankment load was calculated using the procedure outlined in Das (2006).

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

RESULTS

It is estimated that a total primary consolidation settlement of 8 inch will occur in the compressible soils due to the construction of 6 ft high levee.

REFERENCES

1. US Army Corps of Engineers (1995). "*General design memorandum: Passaic River flood damage reduction project*". New York.
2. Das, B. M. (2006). *Principles of geotechnical engineering*, Nelson, Ontario, Canada, 686 p.

SETTLEMENT CALCULATION BASED ON LABORATORY CONSOLIDATION TEST RESUTLS
PASSAIC RIVER
EAST KEARNY, NJ

CALCULATED BY: AH **DATE:** 1/22/2016 **CHECKED BY:** SK

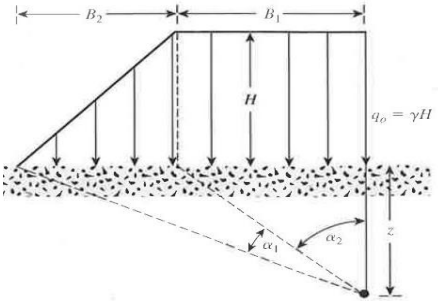
Soil Parameters:

Layer No.	Soil Description	Total Unit Weight (pcf)	Layer Thickness (ft)	Bottom Depth of Layer (ft)	Initial Void Ratio, e_0	Compression Index, C_c
1	Fill	115	4	4		
2	Soft/organic soil	85	12	16	1.46	0.45
3	Soft/organic soil	85	39	55	0.94	0.18
4	Silty clay	120	30	85	0.94	0.18

Elevations:

Embankment top elavation: + 14 ft
Embankment bottom elavation: + 8 ft
Existing ground elavation: + 8 ft
Groundwater table elavation: + 4 ft

Increase in Vertical Stress in Soil due to Embankment Load:



$$\Delta\sigma_z = \frac{q_o}{\pi}\left[\left(\frac{B_1 + B_2}{B_2}\right)(\alpha_1 + \alpha_2) - \frac{B_1}{B_2}(\alpha_2)\right]$$

$q_o = \gamma H$
γ = unit weight of the embankment soil
H = height of the embankment

$$\alpha_1\,(\text{radians}) = \tan^{-1}\left(\frac{B_1 + B_2}{z}\right) - \tan^{-1}\left(\frac{B_1}{z}\right)$$

$$\alpha_2 = \tan^{-1}\left(\frac{B_1}{z}\right)$$

H = 6 ft
B_1 = 5 ft
B_2 = 18 ft

γ = 120 pcf
q_0 = 720 psf

Settlement Calculation:

Sub-Layer No.	Thickness (ft)	Mid Depth of Sub-Layer (ft)	Initial Overburden Pressure, σ'_0 (psf)	α_1 (rad.)	α_2 (rad.)	Increase in Overburden Pressure, $\Delta\sigma'_z$ (psf)	$\sigma'_0 + \Delta\sigma'_z$ (psf)	C_c	Settlement (ft)
1	2	5	483	0.6	0.8	347	830	0.45	0.086
2	2	7	528	0.7	0.6	334	862	0.45	0.078
3	2	9	573	0.7	0.5	318	891	0.45	0.070
4	2	11	618	0.7	0.4	302	920	0.45	0.063
5	2	13	663	0.7	0.4	286	949	0.45	0.057
6	2	15	709	0.7	0.3	270	979	0.45	0.051
7	2	17	754	0.6	0.3	255	1009	0.18	0.024
8	2	19	799	0.6	0.3	241	1040	0.18	0.021
9	2	21	844	0.6	0.2	228	1073	0.18	0.019
10	2	23	889	0.6	0.2	216	1106	0.18	0.018
11	2	25	935	0.5	0.2	205	1140	0.18	0.016
12	2	27	980	0.5	0.2	195	1175	0.18	0.015
13	2	29	1025	0.5	0.2	185	1210	0.18	0.013
14	2	31	1070	0.5	0.2	177	1247	0.18	0.012
15	2	33	1115	0.5	0.2	169	1284	0.18	0.011
16	2	35	1161	0.4	0.1	161	1322	0.18	0.010
17	2	37	1206	0.4	0.1	154	1360	0.18	0.010
18	2	39	1251	0.4	0.1	148	1399	0.18	0.009
19	2	41	1296	0.4	0.1	142	1438	0.18	0.008
20	2	43	1341	0.4	0.1	136	1478	0.18	0.008
21	2	45	1387	0.4	0.1	131	1518	0.18	0.007
22	2	47	1432	0.3	0.1	127	1558	0.18	0.007
23	2	49	1477	0.3	0.1	122	1599	0.18	0.006
24	2	51	1522	0.3	0.1	118	1640	0.18	0.006
25	2	53	1567	0.3	0.1	114	1681	0.18	0.006
26	2	55	1613	0.3	0.1	110	1723	0.18	0.005
27	2	57	1728	0.3	0.1	107	1835	0.18	0.005
28	2	59	1843	0.3	0.1	103	1946	0.18	0.004
29	2	61	1958	0.3	0.1	100	2059	0.18	0.004
30	2	63	2073	0.3	0.1	97	2171	0.18	0.004
31	2	65	2189	0.3	0.1	95	2283	0.18	0.003
32	2	67	2304	0.3	0.1	92	2396	0.18	0.003
33	2	69	2419	0.2	0.1	90	2509	0.18	0.003
34	2	71	2534	0.2	0.1	87	2621	0.18	0.003
35	2	73	2649	0.2	0.1	85	2734	0.18	0.003
36	2	75	2765	0.2	0.1	83	2848	0.18	0.002
37	2	77	2880	0.2	0.1	81	2961	0.18	0.002
38	2	79	2995	0.2	0.1	79	3074	0.18	0.002
39	2	81	3110	0.2	0.1	77	3187	0.18	0.002
40	2	83	3225	0.2	0.1	75	3301	0.18	0.002
41	2	85	3341	0.2	0.1	74	3414	0.18	0.002

TOTAL PRIMARY CONSOLIDATION SETTLEMENT: **8 in**

Attachment D

FLOODWALL SEEPAGE & DEEP-SEATED SLIDING ANALYSIS

OBJECTIVES

1. To calculate exit hydraulic gradient and seepage flow through the floodwall with and without sheetpile cutoff.
2. To perform deep-seated sliding analysis to check for sliding within weak layers beneath the floodwall. This is also called a global stability analysis check.

ASSUMPTIONS

- Maximum Height of Floodwall: 12 feet
- Top of Floodwall: El. +18 feet (NAVD88)
- Flood Level: El. +18 feet (NAVD88)
- Top of ground surface: El. +6 feet (NAVD88)
- Bottom Width of Floodwall: 15 feet
- Soil Properties: Given in Table D.1

Table D.1: Properties for Subsurface Soils

Segments		Materials	Unit Weight (pcf)	φ^o	Cohesion (psf)	K (cm/sec)
East Kearny	Short Term	Soft or Organic Soil	85	0	250	1.00E-04
	Long Term		100	20	0	1.00E-04
	Short Term	Silty Clay	120	0	500	1.00E-05
	Long Term		120	26	0	1.00E-05
West Kearny, Newark, and Harrison	Short Term	Soft or Organic Soil	85	0	250	1.00E-04
	Long Term		100	20	0	1.00E-04
		Silty Clayey Sand	120	32	0	1.00E-04

METHODOLOGY

Seepage Analyses

A commercially available, general purpose seepage computer program, SEEP/W, was used to perform seepage analyses for the floodwall alternative with and without sheetpile cutoff. Seepage flow and hydraulic exit gradient at downstream side were estimated for the steady state hydraulic conditions. The estimated exit gradient values were compared with allowable recommended values to assess the need for underseepage controls.

Deep-Seated Stability Analyses

Deep-seated sliding analysis should be performed to check for sliding within weak layers beneath structures. A commercially available, general purpose seepage computer program, SLOPE/W, was used for this purpose. In this analysis it is assumed that floodwall is a T-Wall with 15 ft wide base rested on batter piles. The vertical water pressure due to the flood is conservatively assumed to be a surcharge load on the ground surface.

RESULTS AND DISCUSSIONS

Seepage Analyses

- Based on the steady state seepage condition for floodwall without sheetpile, underseepage flow for 12 ft high floodwall is estimated to be approximately 25 gpd per foot in both Segments (see Figures D.1 and D.3). The same analysis showed the exit hydraulic gradient of approximately 0.86 in both segments (see Figures D.1 and D.3). Note that this value is much higher than the allowable gradient. Typically, the allowable hydraulic exit gradient is considered as 0.2, but it can be as much as 0.5.
- The steady state seepage analysis for the same floodwall with 20 ft deep sheetpile cutoff resulted in seepage flow of approximately 14 gpd per foot and vertical hydraulic exit gradient of 0.16 in both segments (see Figures D.2 and D.4). The vertical hydraulic exit gradient is within the acceptable range if 20 ft long sheet pile is used. It is also important to note that sheetpiles are not fully impervious and water may flow through the connections but the hydraulic exit gradient is expected to be close to the estimated value. If the estimated flow quantity is a concern or unacceptable, the depth of the sheetpile cutoff needs to be increased.

The summary of seepage analysis results are provided in Table D.2.

Table D.2: Summary of Seepage Analyses Results for 12 ft High Floodwall

Location	Design Condition	Case	Steady-State Seepage	
			Flow Rate $ft^3/sec/ft$	Exit Gradient
East Kearny	Steady Seepage with Full Flood	Floodwall without Sheetpile	3.911E-05	0.86
	Steady Seepage with Full Flood	Floodwall with 20' deep Sheetpile	2.120E-05	0.16
West Kearny, Newark, Harrison	Steady Seepage with Full Flood	Floodwall without Sheetpile	3.953E-05	0.86
	Steady Seepage with Full Flood	Floodwall with 20' deep Sheetpile	2.170E-05	0.16

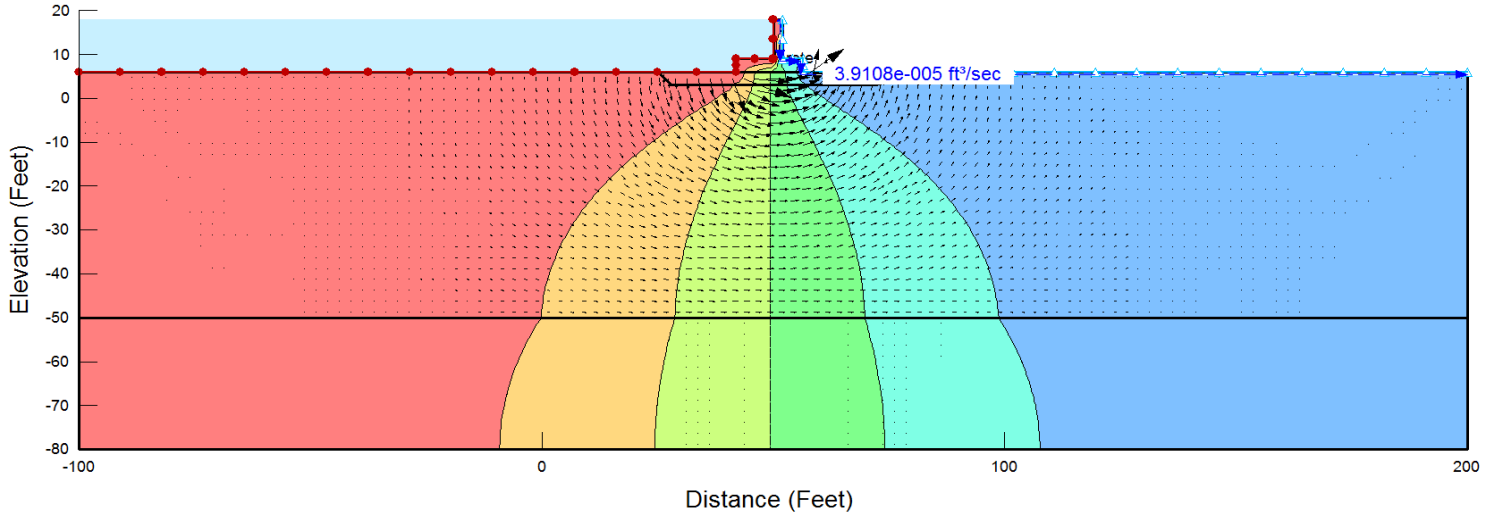
Deep-Seated Stability Analyses

As shown in Figure D.5, the minimum global stability safety factor obtained for the critical slipping surface is 1.50 which meets the minimum required value per EM 1110-2-2502. In this analysis the lateral resistances of the foundation piles and sheetpiles are conservatively neglected.

REFERENCES

- GEOSTUDIO 2007 with Slope/W and Seep/W package.
- "Retaining & Flood Walls", EM 1110-2-2502, United States Army Corps of Engineers, dated September 29, 1989.

Name: Silty Clay Model: Saturated Only K-Sat: 3.28e-007 ft/sec Volumetric Water Content: 0 ft³/ft³
Name: Fill Model: Saturated Only K-Sat: 3.28e-006 ft/sec Volumetric Water Content: 0 ft³/ft³
Name: Soft/Organic Soil (Undrained) Model: Saturated Only K-Sat: 3.28e-006 ft/sec Volumetric Water Content: 0 ft³/ft³
Name: Concrete Model: Saturated / Unsaturated K-Function: Core



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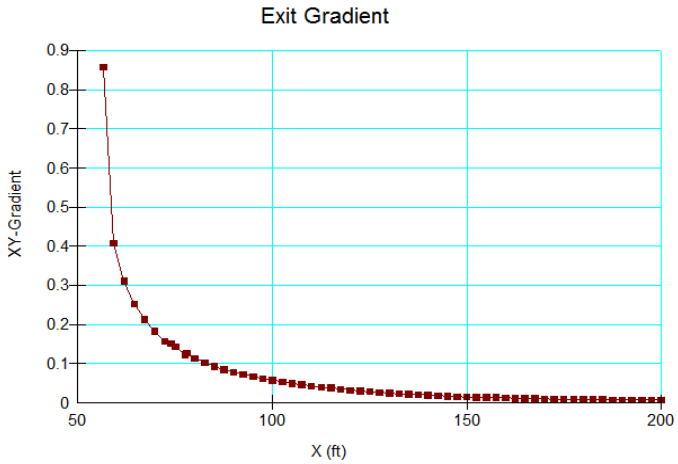

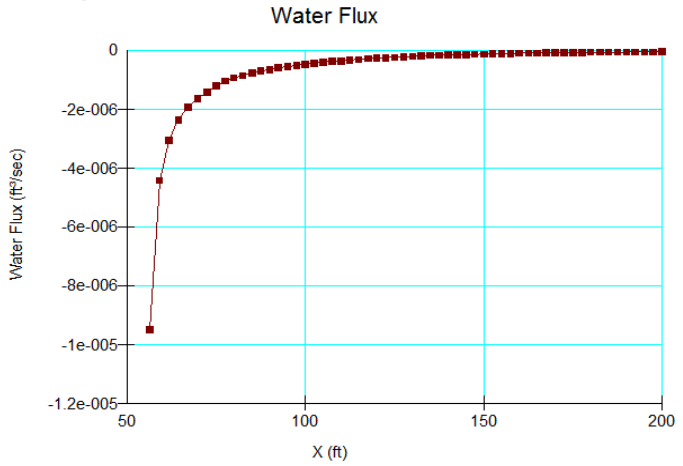

Steady-State Seepage for 12 ft High Floodwall Without Sheetpile (Flood Pool at El. 18 ft)

SEEPAGE ANALYSIS OF FLOODWALL
East Kearny Segment
Estimated Piezometric Conditions, Flow Rate, and Exit Gradient

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO:
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: 1

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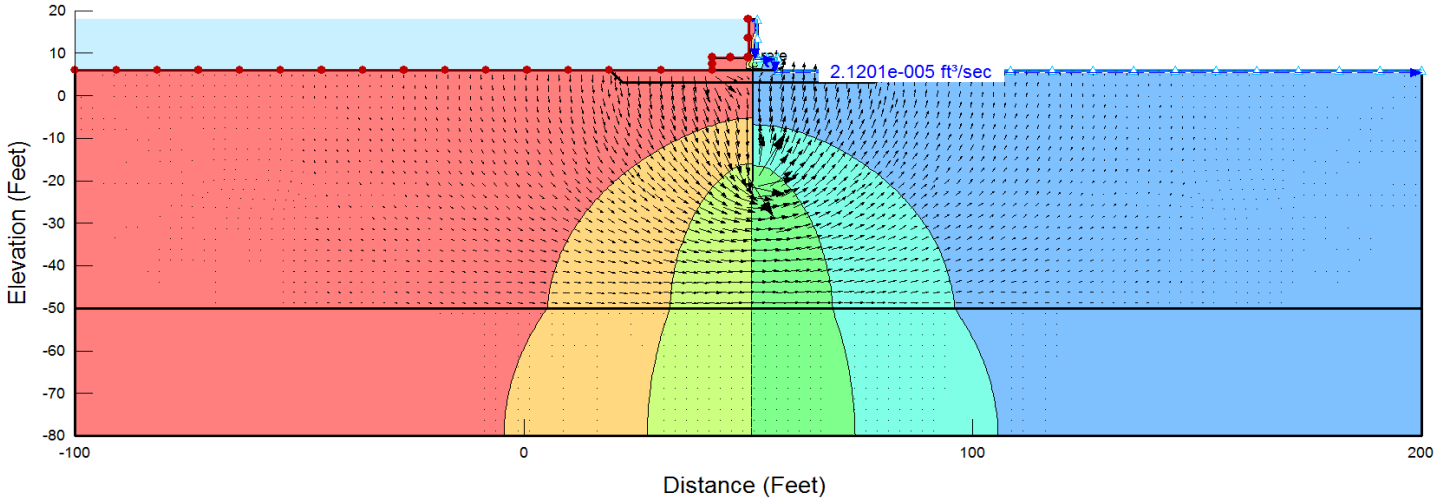
Name: Fill Model: Saturated Only K-Sat: 3.28e-006 ft/sec Volumetric Water Content: 0 ft³/ft³
Name: Soft/Organic Soil (Undrained) Model: Saturated Only K-Sat: 3.28e-006 ft/sec Volumetric Water Content: 0 ft³/ft³
Name: Concrete Model: Saturated / Unsaturated K-Function: Core
Name: Sheet Pile Model: Interface K-Transpose: 0 ft/sec K-Normal: 0 ft/sec Dry Air Conductivity: 0 ft/sec



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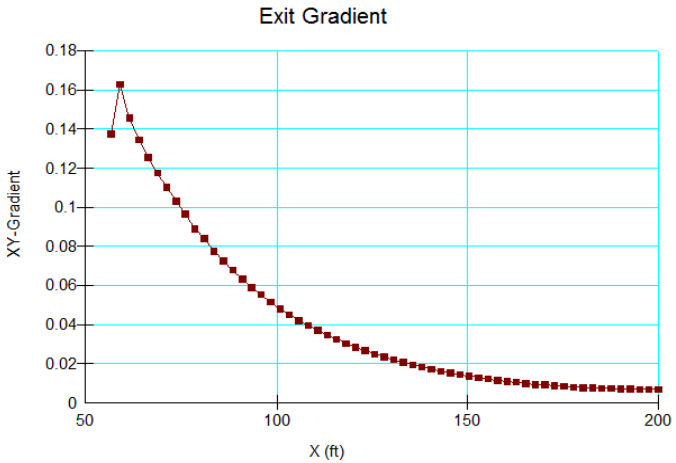

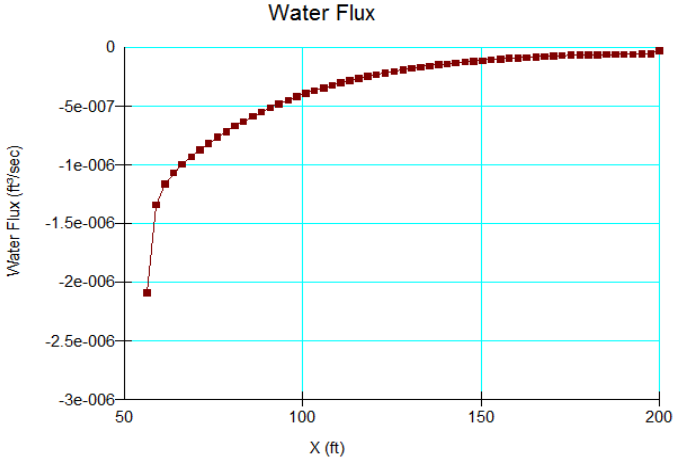

Steady-State Seepage for 12 ft High Floodwall With 20ft Long Sheetpile (Flood Pool at El. 18 ft)

SEEPAGE ANALYSIS OF FLOODWALL WITH SHEETPILE
East Kearny Segment
Estimated Piezometric Conditions, Flow Rate, and Exit Gradient

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO:
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: 2

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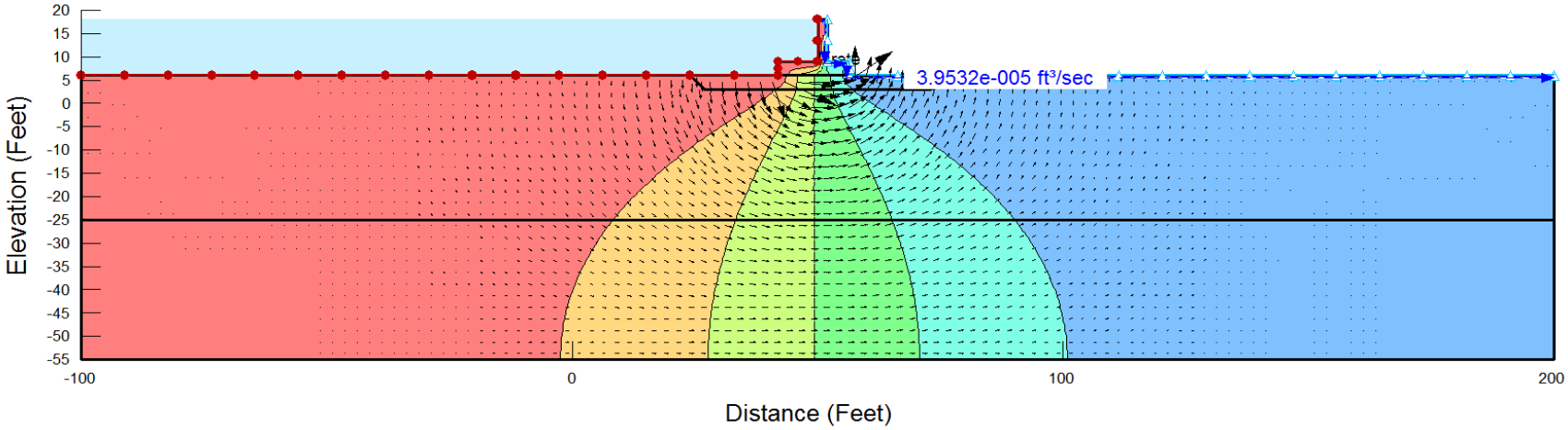
Name: Soft/Organic Soil (Undrained) Model: Saturated Only K-Sat: 3.28e-006 ft/sec Volumetric Water Content: 0 ft³/ft³
Name: Concrete Model: Saturated / Unsaturated K-Function: Core



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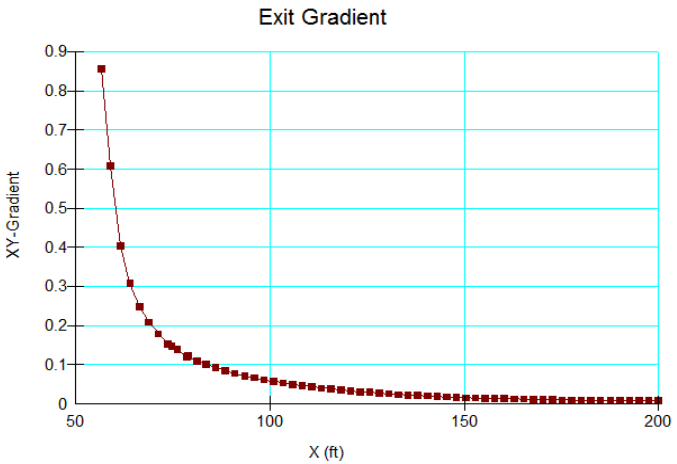

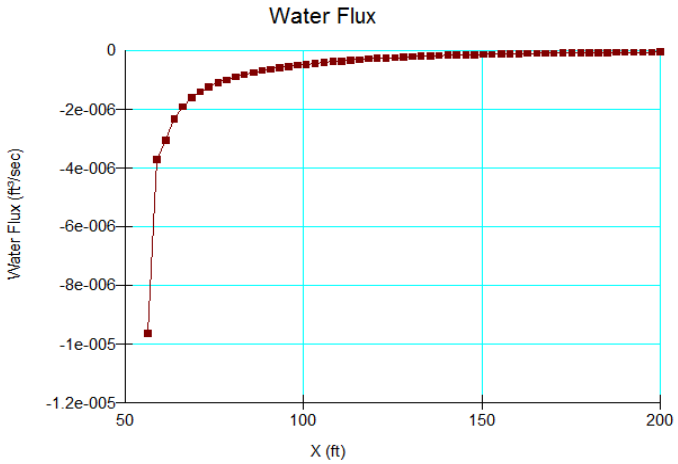

Steady-State Seepage for 12 ft High Floodwall Without Sheetpile (Flood Pool at El. 18 ft)

SEEPAGE ANALYSIS OF FLOODWALL
West Kearny Segment
Estimated Piezometric Conditions, Flow Rate, and Exit Gradient

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO:
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: 3

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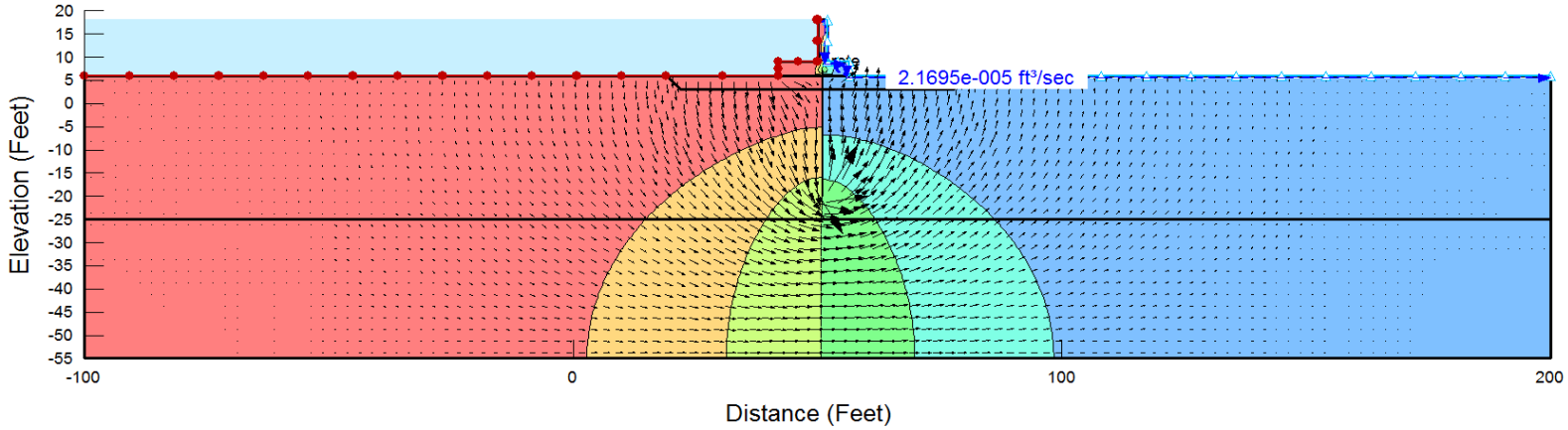
Name: Fill Model: Saturated Only K-Sat: 3.28e-006 ft/sec Volumetric Water Content: 0 ft³/ft³
Name: Soft/Organic Soil (Undrained) Model: Saturated Only K-Sat: 3.28e-006 ft/sec Volumetric Water Content: 0 ft³/ft³
Name: Concrete Model: Saturated / Unsaturated K-Function: Core
Name: Sheet Pile Model: Interface K-Transpose: 0 ft/sec K-Normal: 0 ft/sec Dry Air Conductivity: 0 ft/sec



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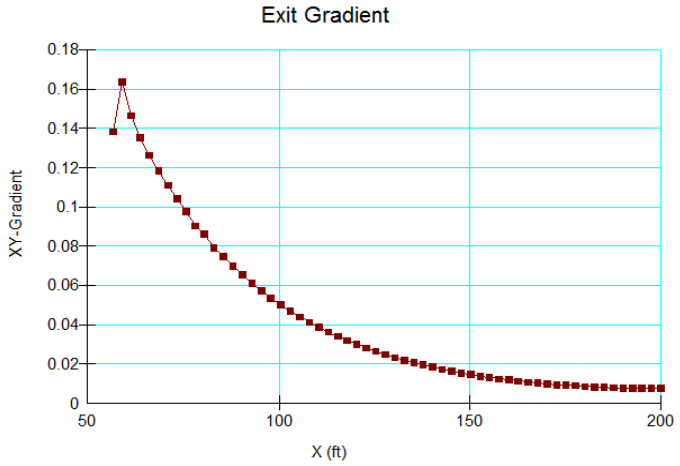
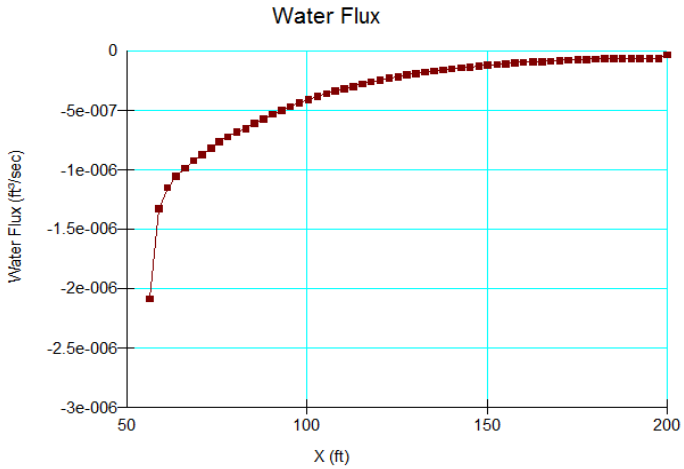


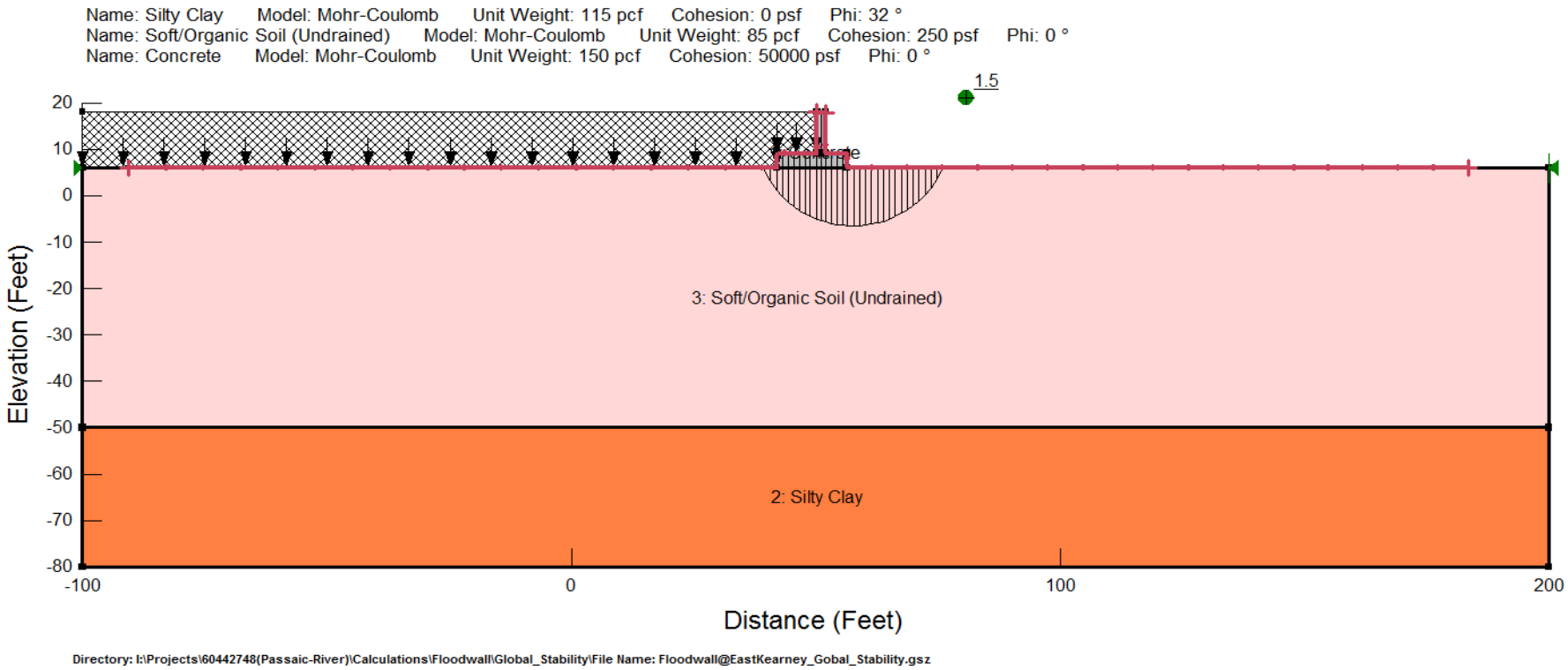


Steady-State Seepage for 12 ft High Floodwall With 20ft Long Sheetpile (Flood Pool at El. 18 ft)

SEEPAGE ANALYSIS OF FLOODWALL WITH SHEETPILE
West Kearny Segment
Estimated Piezometric Conditions, Flow Rate, and Exit Gradient

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO:
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: 4



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Full Flood as a Surcharge Load (12 ft High Floodwall)

DEEP-SEATED STABILITY ANALYSIS OF FLOODWALL
East Kearny Segment

Critical Global Sliding Surface and Stability FOS

AECOM

DR BY: MS	SCALE: N.T.S	PROJECT NO:
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: 5

Attachment E

PILE CAPACITY ANALYSIS

OBJECTIVES

1. To calculate compression and tension capacities of H-Piles (HP14x73) bearing on soil and rock.
2. To calculate compression and tension capacities of 14-inch prestressed precast concrete (PPC) piles bearing on soil.
3. To calculate compression and tension capacities of 8-inch and 12-inch (O.D.) rock sockets for Caissons/Micropiles.

ASSUMPTIONS

- Depth to Rock at East Kearny Segment: 80 ft or less
- Depth to Rock at West Kearny, Newark, and Harrison Segment: 60ft or less
- Downdrag effect on piles: Negligible
- Allowable bonding resistance of rock-socket interface (compression): 50 psi
- Allowable bonding resistance of rock-socket interface (tension): 30 psi

METHODOLOGY

HP and PPC Piles

A commercially available, general purpose pile capacity calculation computer program, APILE v.5.0, was used to perform driven pile capacity calculation analyses for the HP and PPC piles. The method of FHWA was used in the computation. The engineering properties of the soil as provided in Table-1 of Attachment A were used. The compression capacities of the piles were estimated with the assumption of piles bearing on soil.

Micropiles/Caissons

The compression and tension capacities of the rock sockets were calculated using the FHWA Micropile 2005 guidelines. In this project Micropiles with rock-sockets may be used in the areas with shallower rock depth. In estimating the total capacities of the Micropiles the skin resistance from the soil was neglected. The geotechnical compression and tension capacities of the rock sockets were compared with the structural capacities and the minimum values were recommended for the preliminary design.

RESULTS AND DISCUSSIONS

HP and PPC Piles

Based on the soil stratification and results of the pile bearing capacity analysis using APILE, H-Pile (HP14x73) embedded at least 80 ft into soft/organic clay and silty clay can approximately provide an ultimate compression and uplift capacity of 95 kips at East Kearny Segment (see Figure E.1). In West Kearny, Newark, and Harrison Segment H-Pile embedded at least 60 ft into soft/organic clay and silty clayey sand can provide approximately 110 kips of ultimate compression capacity and 100 kips of ultimate uplift capacity (see Figure E.2). For H-Piles bearing on a competent rock the ultimate compression capacity will be determined by structural capacity with the limit of 200 kips.

Similar pile capacity analysis performed on 14-inch prestressed precast concrete piles

showed that concrete piles embedded at least 80 ft into soft/organic clay and silty clay at East Kearny Segment can provide 100 kips and 95 kips of ultimate compression and uplift capacities, respectively (see Figure E.3). In West Kearny, Newark, and Harrison Segment concrete piles embedded at least 60 ft into soft/organic clay and silty clayey sand can provide approximately 205 kips of ultimate compression capacity and 160 kips of ultimate uplift capacity (see Figure E.4).

Micropiles/Caissons

The allowable compression and tension capacities of 8-inch and 12-inch rock sockets for different lengths are calculated based on the Micropile design guidelines and details as provided in Figure E.5 and Figure E.6. The summary of the estimated capacities are also given in Table E.1. As seen from the table, the maximum allowable compression and tension capacities of 9-5/8-inch Micropile with 20-feet long (8-inch O.D.) rock socket is 150 and 100 tons, respectively. The maximum allowable capacities increase to 240 and 150 tons, respectively, if the rock socket diameter is increased to 12-inch.

Table E.1: Summary of estimated Micropile capacity

Steel Casing Outside Diameter (in.)	Steel Casing Thickness (Minimum) (in.)	Rock Socket Diameter (in.)	Rebar Size	Rock Socket (Minimum) (ft)	Maximum Allowable Capacity (tons)	
					Compression	Tension
9-5/8	0.545	8	#24 (1)	10	80	50
				15	120	75
				20	150	100
				25	180	125
				30	180	150
13-3/8	0.480	12	#24 (1)	10	120	75
				15	180	100
				20	240	150
				25	260	155
				30	260	155

REFERENCES

- APILE v.5.0, A Program for the Study of Driven Piles under Axial Loads, ENSOFT, INC.
- FHWA Publication No. NHI-05-039, Micropile Design and Construction Reference Manual, 2005.

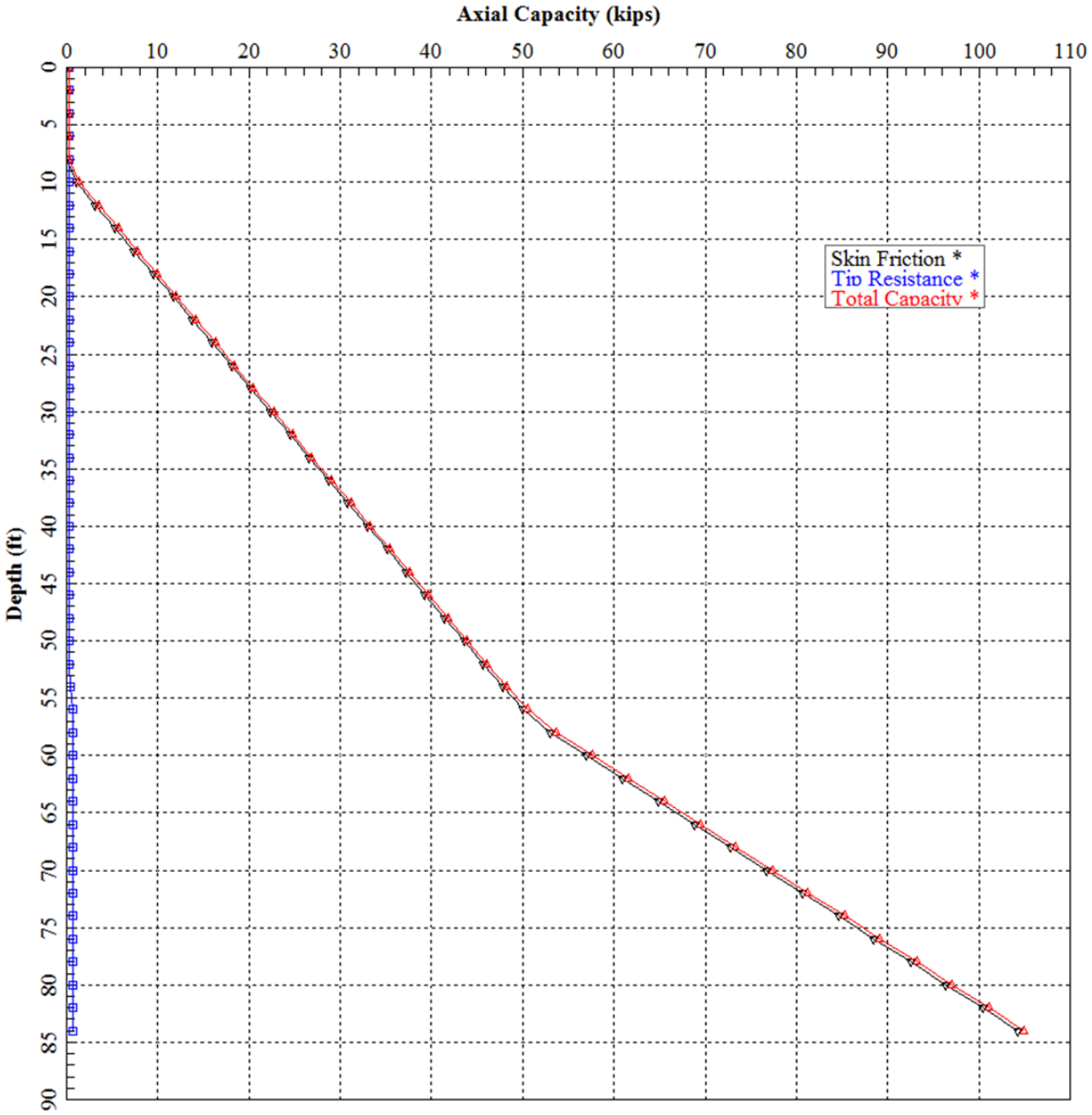

Figure E.1: Skin Friction and Total Capacity Distribution of HP14x73 Pile with Depth (East Kearny Segment)

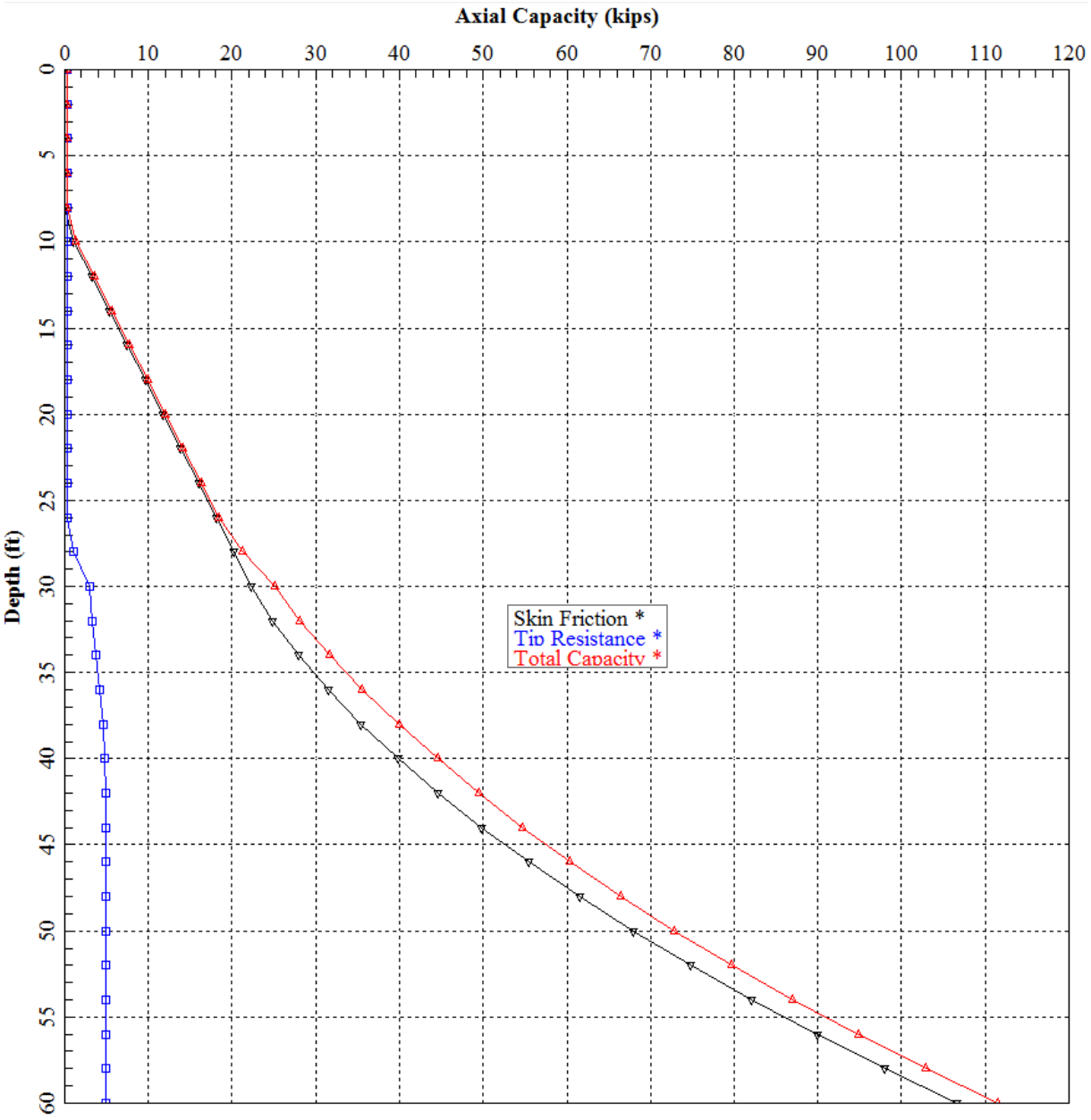

Figure E.2: Skin Friction and Total Capacity Distribution of HP14x73 Pile with Depth (West Kearny, Newark, and Harrison Segment)

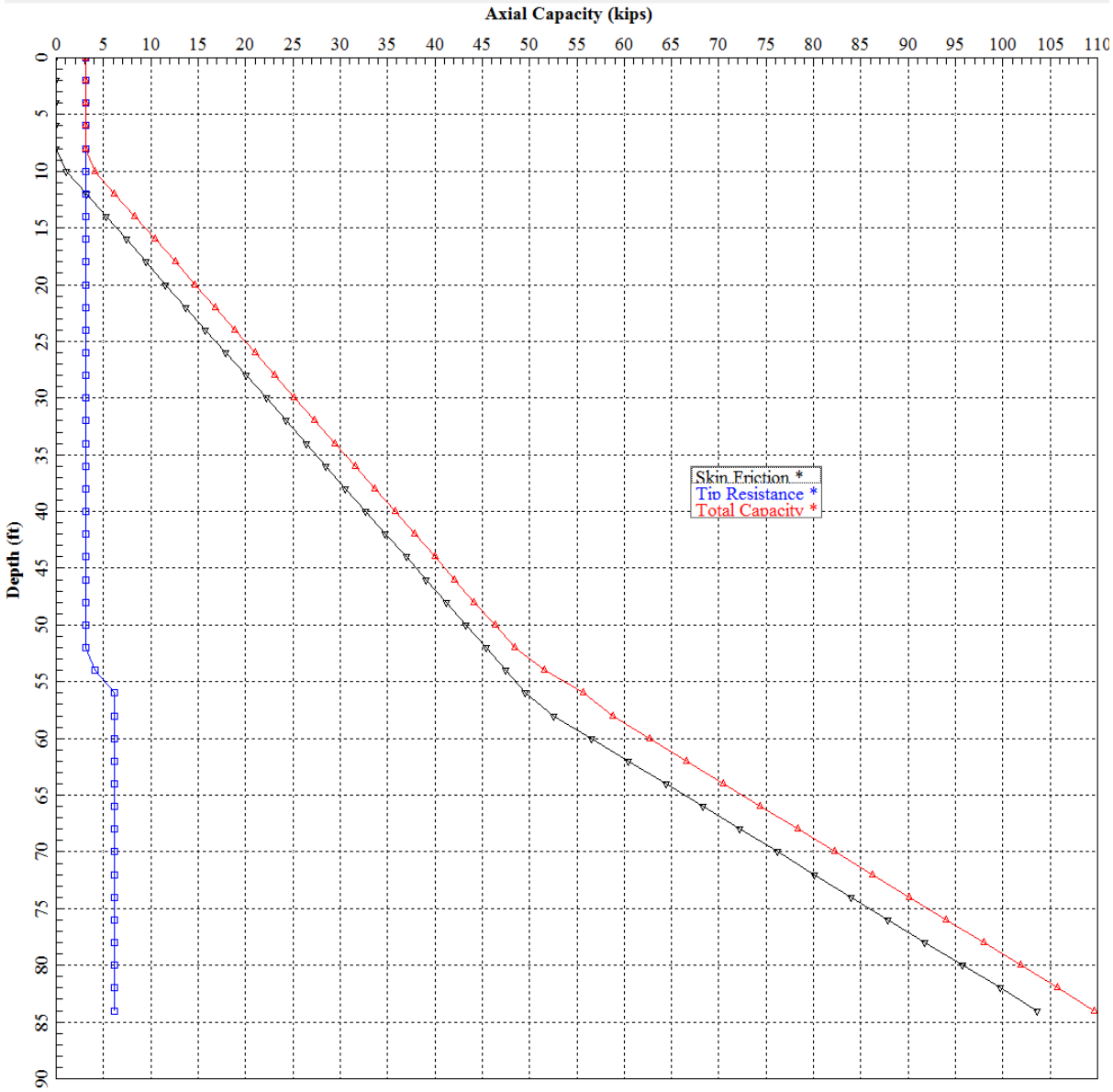

Figure E.3: Skin Friction and Total Capacity Distribution of 14-inch Prestressed Precast Concrete Pile with Depth (East Kearny Segment)

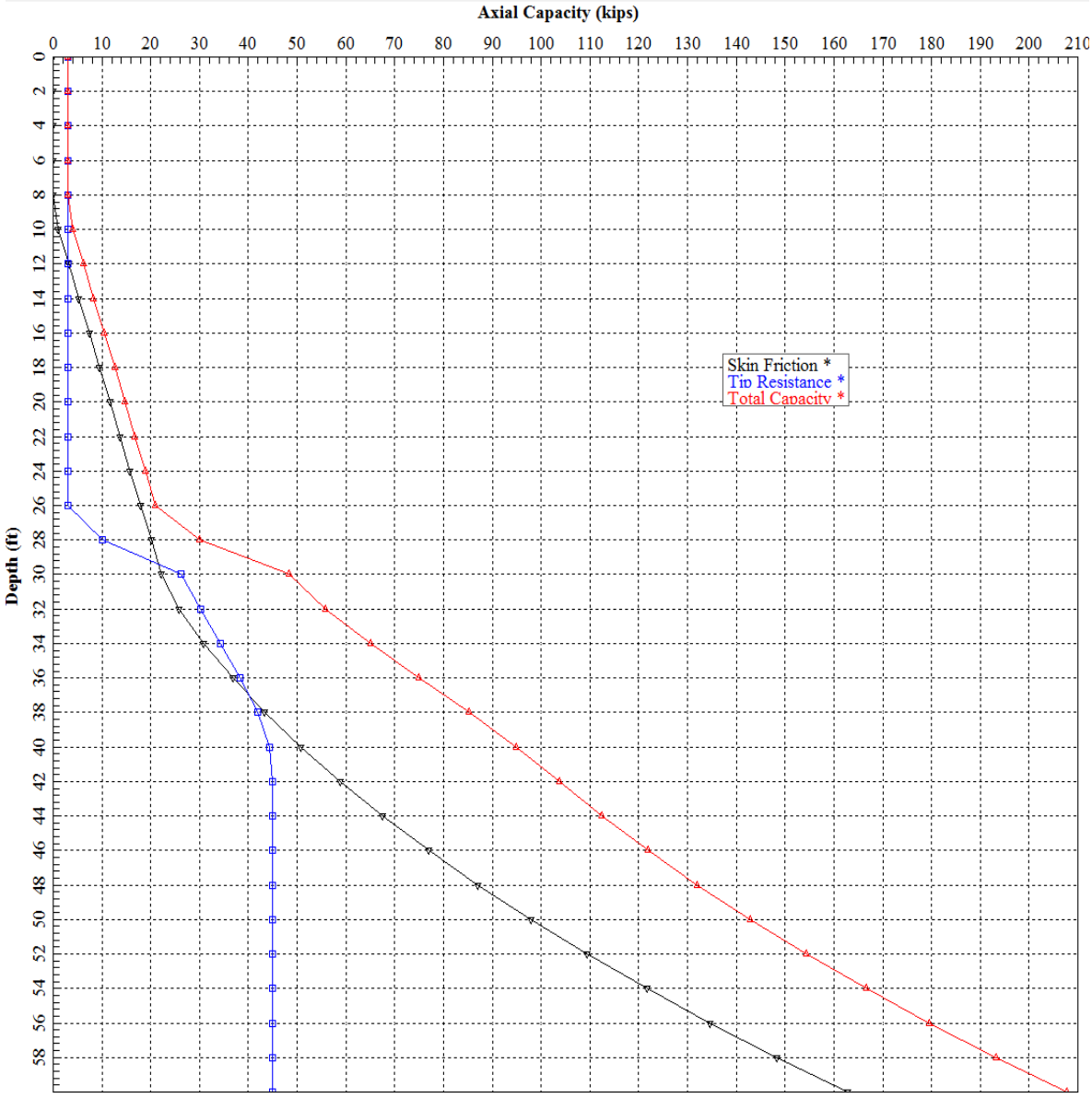

Figure E.4: Skin Friction and Total Capacity Distribution of 14-inch Prestressed Precast Concrete Pile with Depth (West Kearny, Newark, and Harrison Segment)

PILE BEARING CAPACITY STUDY
PASSAIC RIVER FLOOD RISK MANAGEMENT PROJECT
TIDAL AREAS IN HARRISON, KEARNEY, AND NEWARK, NJ

ESTIMATE OF THE CAPACITY OF DRILLED SHAFTS IN ROCK - FHWA MICROPILE JUNE 2005 GUIDELINES

Project Name: Passic-River **Calculated by : M.S.**

Project Number : **Checked by : S.K.**

Outside Diameter of Casing:	***9.625*** in.
Thickness of Casing:	***0.545*** in.
Inside Diameter of Casing	8.535 in.
Diameter of Rock Socket :	***8*** in.
Perimeter of Rock Socket :	25.1 in.
Plunge Length :	**12.0** in.
Area of Rock Socket at Bottom:	50.3 sq.in.
Center to Center Spacing :	**10.0** ft
Depth to Rock :	**75** ft
Soil Unit Weight :	**38** pcf
Soil Weight above Socket :	620 tons
Rock unit weight	**145** pcf
Allowable Bond Stress (Compression):	**50** psi
Allowable Bond Stress (Tension):	**33** psi

Based on a cone with 30 degree angle and CC spacing

Est. Rock Cap. at Bottom :	**20** tsf
Est. Resistance at Bottom :	7 tons

Geotechnical Capacity - In Accordance with FHWA

A. Compression		
Length of Socket	Side Resistance (tons)	Allowable Resistance (tons)
5	38	45
10	75	82
15	113	120
20	151	158
25	188	184
30	226	184

B. Tension			
Length of Socket	Competent Rock - Failure at Grout/Rock Interface (tons)	Competent Rock - Spacing Consideration (tons)	Fractured Rock - Single Drilled Shaft (tons)
5	25	120	153
10	50	153	153
15	75	153	153
20	101	153	153
25	126	153	153
30	151	153	153

Structural Capacity - In Accordance with FHWA

Rebar Diameter:	3 in	Rebar Number:	24
Number of Rebars:	**1**		
Total Rebar Area:	7.07 sq.in.		
Rebar Steel Yield Stress:	**75** ksi		
Casing Steel Yield Stress :	**50** ksi		
Conc. Compr. Stress:	**6** ksi		
Casing Steel Area:	15.5 sq.in.		
Grout Area in Casing :	50.1 sq.in.		
Grout Area in Socket :	43.2 sq.in.		

Cased Length Capacity

Steel Strength (Comp.):	266 tons	Steel Strength (Tension):	311 tons
Grout Strength :	60 tons		
Total :	**326** tons		

Rock Socket (Uncased Length) Capacity

Allowable Transfer Load :	8 tons		
Steel Strength (Comp.):	125 tons	Steel Strength (Tension):	153 tons
Grout Strength :	52 tons		
Total :	**184** tons		

Total Structural Capacity (FHWA) : 184 tons

Figure E.5: Estimate of Capacity of 8-inch Rock-Sockets

PILE BEARING CAPACITY STUDY
PASSAIC RIVER FLOOD RISK MANAGEMENT PROJECT
TIDAL AREAS IN HARRISON, KEARNEY, AND NEWARK, NJ

ESTIMATE OF THE CAPACITY OF DRILLED SHAFTS IN ROCK - FHWA MICROPILE JUNE 2005 GUIDELINES

Project Name: Passic-River **Calculated by : M.S.**

Project Number : **Checked by : S.K.**

Outside Diameter of Casing:	***13.375*** in.
Thickness of Casing:	***0.48*** in.
Inside Diameter of Casing	12.415 in.
Diameter of Rock Socket :	***12*** in.
Perimeter of Rock Socket :	37.7 in.
Plunge Length :	**12.0** in.
Area of Rock Socket at Bottom:	113.1 sq.in.
Center to Center Spacing :	**10.0** ft
Depth to Rock :	**75** ft
Soil Unit Weight :	**38** pcf
Soil Weight above Socket :	624 tons
Rock unit weight	**145** pcf
Allowable Bond Stress (Compression):	**50** psi
Allowable Bond Stress (Tension):	**33** psi

Based on a cone with 30 degree angle and CC spacing

Est. Rock Cap. at Bottom :	**20** tsf
Est. Resistance at Bottom :	16 tons

Geotechnical Capacity - In Accordance with FHWA

A. Compression

Length of Socket	Side Resistance (tons)	Total Resistance (tons)
5	57	72
10	113	129
15	170	185
20	226	242
25	283	263
30	339	263

B. Tension

Length of Socket	Competent Rock - Failure at Grout/Rock Interface (tons)	Competent Rock - Spacing Consideration (tons)	Fractured Rock - Single Drilled Shaft (tons)
5	38	120	157
10	75	157	157
15	113	157	157
20	151	157	157
25	157	157	157
30	157	157	157

Structural Capacity - In Accordance with FHWA

Rebar Diameter:	3 in	Rebar Number:	24
Number of Rebars:	**1**		
Total Rebar Area:	7.07 sq.in.		
Rebar Steel Yield Stress:	**75** ksi		
Casing Steel Yield Stress :	**50** ksi		
Conc. Compr. Stress:	**6** ksi		
Casing Steel Area:	19.4 sq.in.		
Grout Area in Casing :	114.0 sq.in.		
Grout Area in Socket :	106.0 sq.in.		

Cased Length Capacity

Steel Strength (Comp.):	312 tons	Steel Strength (Tension):	365 tons
Grout Strength :	137 tons		
Total :	**448** tons		

Rock Socket (Uncased Length) Capacity

Allowable Transfer Load :	11 tons		
Steel Strength (Comp.):	125 tons	Steel Strength (Tension):	157 tons
Grout Strength :	127 tons		
Total :	**263** tons		

Total Structural Capacity (FHWA) : 263 tons

Figure E.6: Estimate of Capacity of 12-inch Rock-Sockets

SUBAPPENDIX 2
2.2: T-Wall Structural Analysis

Passaic River Tidal – General Reevaluation Report

Floodwall Design Criteria

Revised
April 15, 2016

FLOODWALL DESIGN CRITERIA

1. General

This design criteria addresses the design of floodwalls in typical reaches along the Passaic River extending from Kearny to Newark, NJ. The design elements defined herein represent a preliminary design (i.e., 30-percent level) using the best available information. The analysis was limited to Stability. Pile foundations provide stability against overturning, sliding and flotation resistance. Soil conditions along this reach of the Passaic River were divided into two reaches; East Kearny and West Kearny. The elevation of the bedrock was assumed based on current limited information (see the Geotechnical Report); pile lengths must be refined as more soil data becomes available.

Floodwall designs were also provided which may be used to address Hazardous, Toxic, and Radioactive Waste (HTRW) reaches in which ground disturbance may not permitted[1] (i.e., excavation, augering and drilling of piles is not permitted). In this situation, our recommendation was to construct the T-Wall on top of the existing ground surface. Pile types requiring drilling or augering were not allowed. H-Piles, Pipe Piles and concrete piles were considered; prestressed concrete piles were selected for use in these HTRW reaches. The concrete piles are more resistant to corrosion that is typically found in HTRW soils. Vinyl sheet piling may be a consideration for use as cutoff piling. Although not unconditionally accepted by the USACE, there have been several projects constructed by the Corps that have included vinyl sheeting. Interim guidance is provided in USACE document; ***"General Design Guide: PVC Sheet Pile", dated May 2005***. Given the concern for long term durability, coated steel sheet piling has been included in our proposed designs. An L-Wall design was also developed for the HTRW reaches. In building the Floodwall on top of the ground surface, the overall height of the T-Wall was reduced to a level where L-Walls are a consideration. The L-Wall would only be applicable in the HTRW reaches. The sheet pile cutoff wall acts as both seepage cutoff and axial capacity. Where axial capacity is required, steel pilings would be required, vinyl should not be considered for this structural application. The L-Wall would <u>not</u> be recommended where corrosion rates are proven to be severe as the steel sheet pile would need to include significant, long-term corrosion protection and monitoring Soil testing for corrosive properties and stray currents should be performed in advance of final design. The level of corrosion protection, to include coatings and sacrificial thickness, can then be more accurately determined. In summary, Micro piles and H-Piles were considered in Typical T-wall reaches. Prestressed concrete piles were only considered for use in the HTRW reaches. L-Wall designs should be considered but only in wall heights less than 8ft where corrosion is determined to be moderate. Design calculations for this phase can be found in Appendix x.

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[1] This is a potential construction condition, considered in the analysis for completeness.

For cost comparison purposes, three wall heights were considered; Top of Wall (TOW) at El 18.0, El 16.0 and El 14.0 NAVD[2]. The Still Water Elevation (SWL) was assumed to be 2 feet below the TOW elevation. The typical ground elevation was assumed to be El 7 NAVD throughout the project.

2. Codes and Standards

The following is an abbreviated list of general U.S. Army Corps of Engineers (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 7th Edition, 2014..
- 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, 14th 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.
- USACE EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures.
- USACE EM 1110-2-2502, Retaining and Flood Walls.
- 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.

3. General Design Load Parameters

3.1.A. Load Combinations

The preliminary design includes four Basic Load Cases; these are the loadings that typically control floodwall designs. Other loadings must also be analyzed in the final design, including Seismic Load Cases for both Operating and Maximum Earthquake conditions. Additionally, sufficient hydraulic modeling should be performed as part of

[2] All elevations are referenced to North American Vertical Datum of 1988 (NAVD).

the future design to establish wave properties and forces. Typically, on inland waterways, when the wall is overbuilt to include uncertainty and sea-level rise the static head to top of wall is similar in force to that imparted by a wave; sufficiently close for this conceptual design. The load cases included in the design are:

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

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

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

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

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

3b. Flood Stage with Water to Top of Wall, Pervious Cutoff. Dead load of concrete wall, At-Rest lateral earth pressures, and hydrostatic loading for water to the TOW; Uplift forces assume the sheet pile to be pervious varying linearly from flood side TOW elevation to the ground water elevation on the Protected Side. Wave force is not included. A 33% overstress is permitted.

4a. Flood Stage at Still Water, Debris Impact Load, Impervious Cutoff. Loadings include: Dead load of concrete wall, At-Rest lateral earth pressures, and hydrostatic loading for water to the SWL. Uplift forces assume the sheet pile to be impervious. A debris load of 500lbs/LF is applied at the SWL. 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 HSDRRS design guidance that was developed for the New Orleans District post-Katrina and considered applicable for this flood protection project

3.1.A. Hydraulic Stages

Table 1 – Hydraulic Stages and Design Water Surface Elevations

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

SWL – Still Water Level
TOW – Top of Wall

3.2. Load Cases

3.2.1. Dead Loads (D)

Dead loads shall be determined in accordance with applicable engineering manuals and ASCE 7-02, 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.

Table 2 – 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

3.2.2. Live Loads (L)

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

3.2.2.1 Live Load Surcharge (LS)

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

3.2.3. 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:
  Ko = 0.8 for Clay.
  Ko = 0.48 for Granular Material.

3.2.4. Hydrostatic Loads (H)

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

3.2.5. Uplift Loads (U)

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

3.2.6. 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.

4. 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)

5. 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 A992, Grade 36
(c)	Bolts and nuts	ASTM A325, min. ¾“ ASTM A490
(d)	Anchor Bolts	ASTM A449, (¾“ dia. & 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 mils min.) epoxy. Vertical lift gates and steel sheet pile structures shall be painted with an epoxy painting system.

6. 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 10ft deep rock socket. H-Pile and Concrete 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 prestressed 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, prestressing strands are Low-Lax, Grade 270. Prestressed 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 prestressed 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.

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

Although not commonly used in the Northeast, Precast Prestressed Concrete (PPC) piles were included for use in reaches that are considered HTRW and have an increased rate of corrosion, in the event that construction on HTRW sites is pursued. The concrete pile is far more resistant to corrosion than steel. Stress levels shall be controlled to prevent cracking of the concrete when experiencing both service loads and driving stresses.

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

GEOTECHNICAL

MICROPILE DESIGN - BASED ON FHWA MICROPILE GUIDELINES (2005)

Project Name: Passic-River - Preliminary Floodwall Design — Calculated by: M.S.

Project Number : — Checked by:

SUMMARY OF ESTIMATED MICROPILE CAPACITY

Steel Casing Outside Diameter (in.)	Steel Casing Thickness (Minimum) (in.)	Rebar Size	Rock Socket (Minimum) (ft)	Maximum Allowable Capacity (tons)	
				Compression	Tension
9-5/8	0.545	#24 (1)	10	80	50
			15	120	75
			20	150	100
			25	180	125
			30	180	150
13-3/8	0.480	#24 (1)	10	120	75
			15	180	100
			20	240	150
			25	260	155
			30	260	155

H-Pile Capacity - BASED ON APILE ANALYSIS USING FHWA METHOD

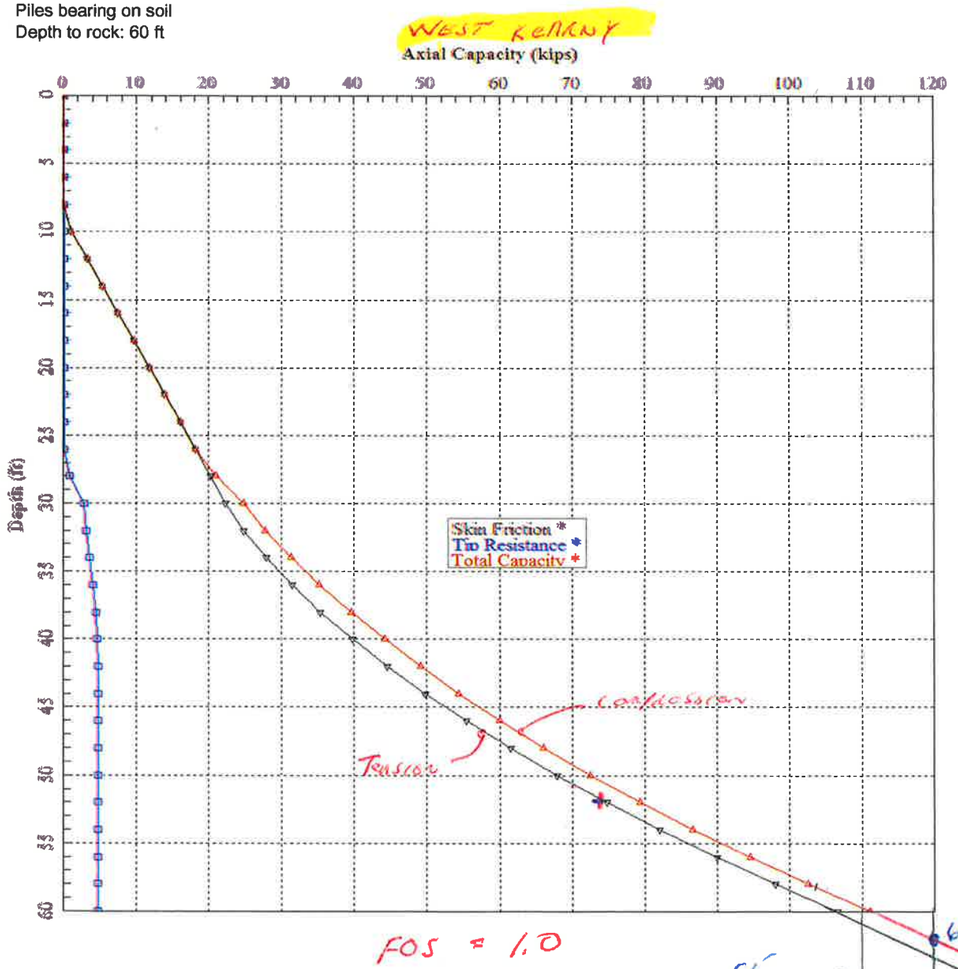
Project Name: Passic-River - Preliminary Floodwall Design — Calculated by: M.S.

Project Number : — Checked by:

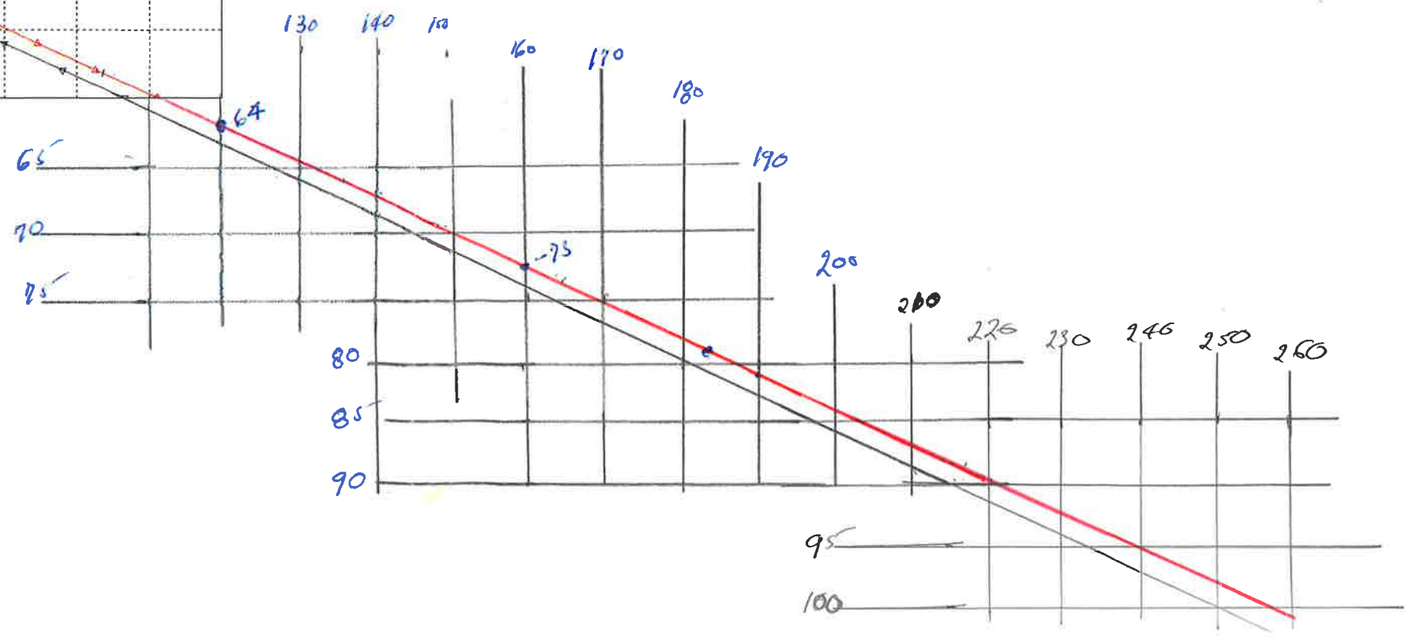
Piles bearing on soil

Depth to rock: 60 ft

WEST KENNY



FOS = 1.0



COM

JOB TITLE _____

JOB NO. _____ CALCULATION NO. _____

ORIGINATOR _____ DATE _____

REVIEWER _____ DATE _____

SCALE _____ SHEET NO. _____ OF _____

H-Pile Capacity - BASED ON APILE ANALYSIS USING FHWA METHOD

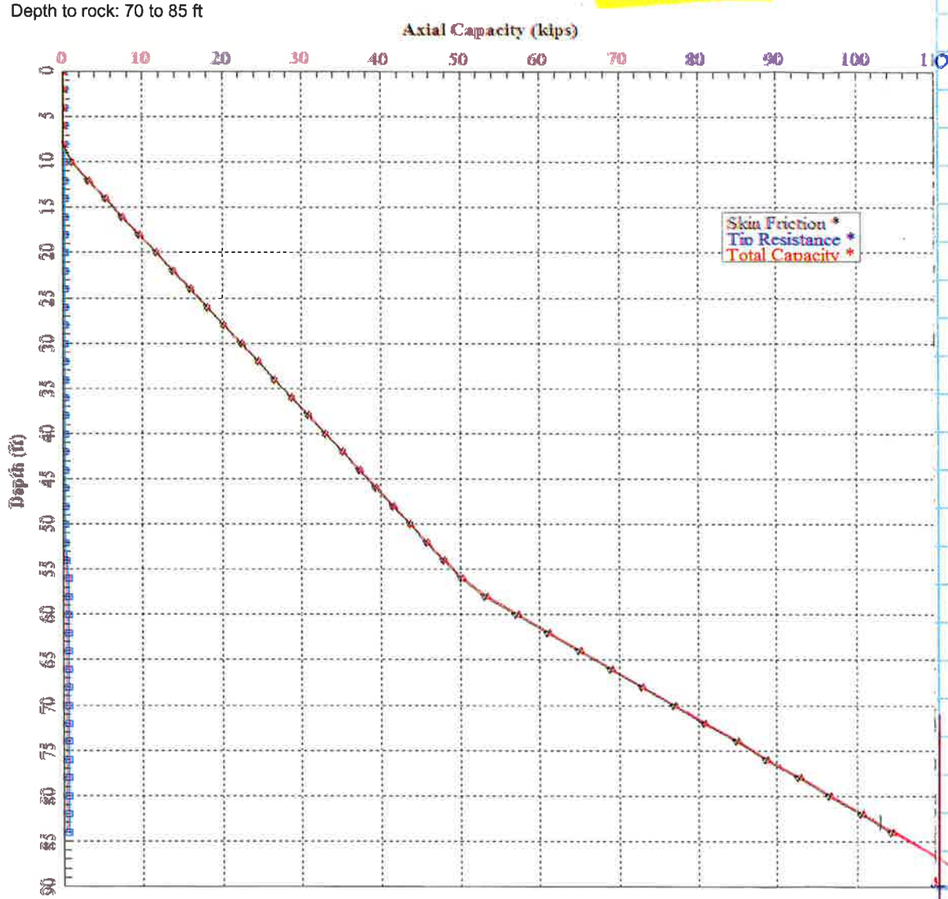
Project Name: Passic-River - Preliminary Floodwall Design — Calculated by: M.S.

Project Number : — Checked by:

Piles bearing on soil

Depth to rock: 70 to 85 ft

EAST KEARNY



FOS = 1.0

120 130 140 150 160 170 180 190 200 210 220

95 100 105 110 115 120 125 130 135 140

Grid: 4x4 = 1"

14" Concrete Prestressed Pile Capacity - BASED ON APILE ANALYSIS USING FHWA METHOD

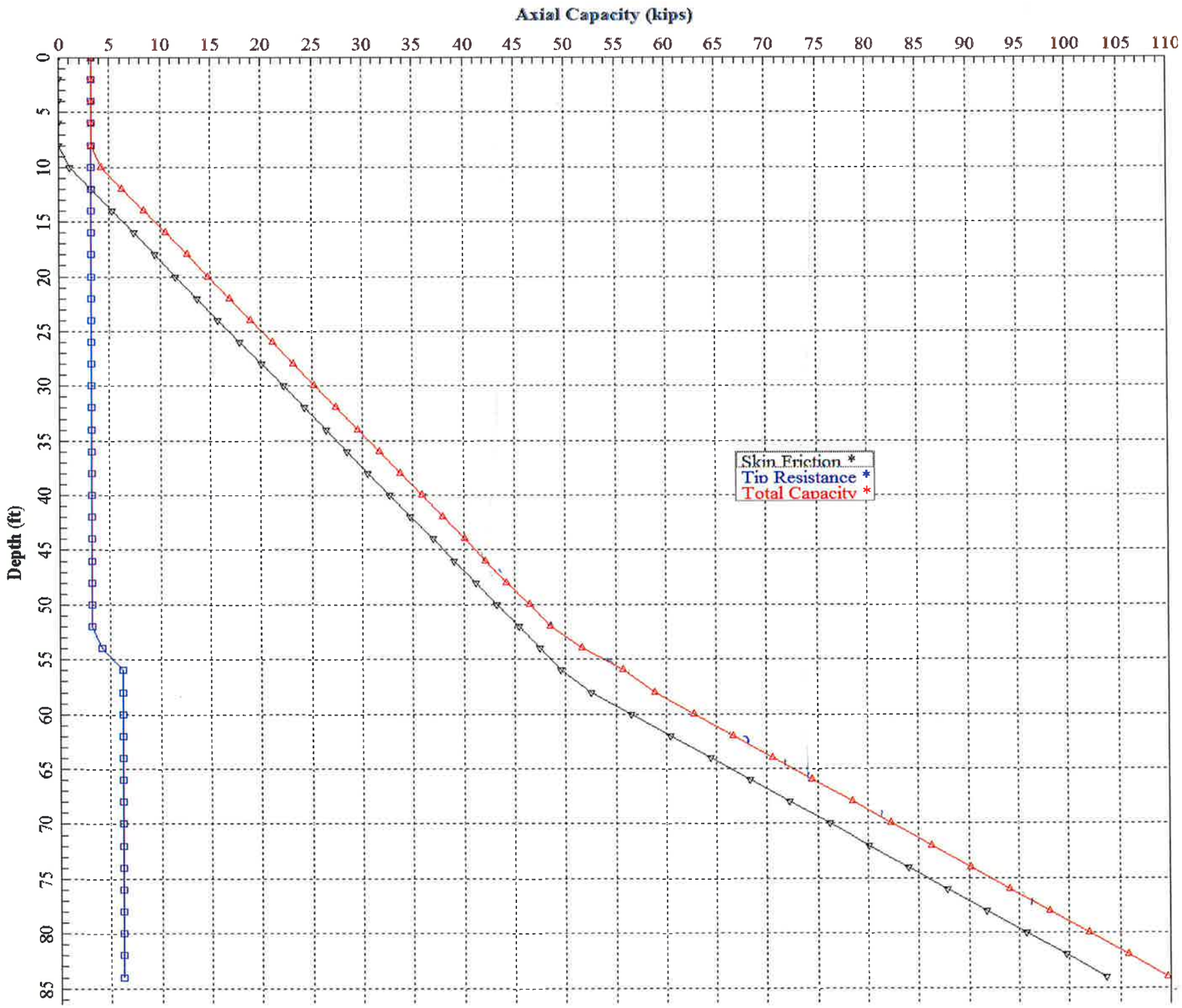
Project Name: Passic-River - Preliminary Floodwall Design — Calculated by: M.S.

Project Number : — Checked by:

Piles bearing on soil

Depth to rock: 70 to 85 ft

EAST OF KEARNY



14" CONC PRESTRESSED

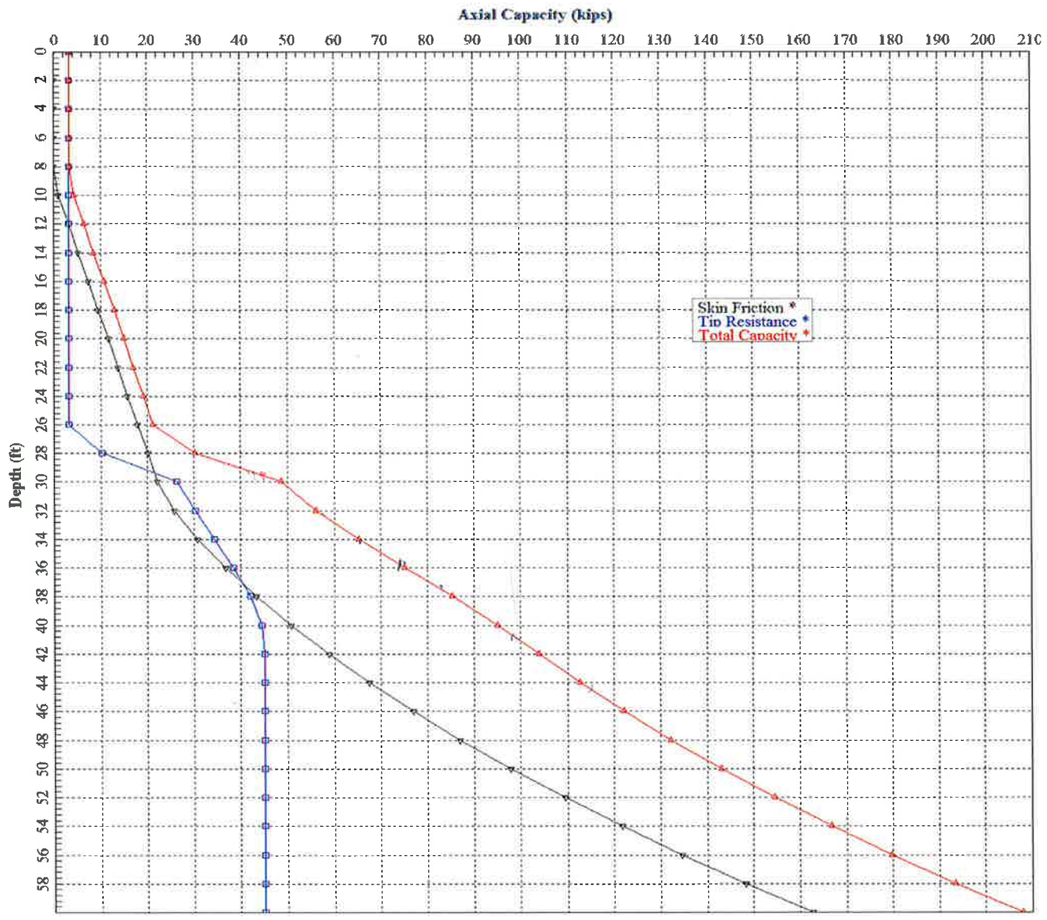
~~H-Pile~~ **Capacity - BASED ON APILE ANALYSIS USING FHWA METHOD**

Project Name: Passic-River - Preliminary Floodwall Design Calculated by: M.S.

Project Number : Checked by:

Piles bearing on soil WEST OF KEARNY

~~Depth to rock: 60 ft~~



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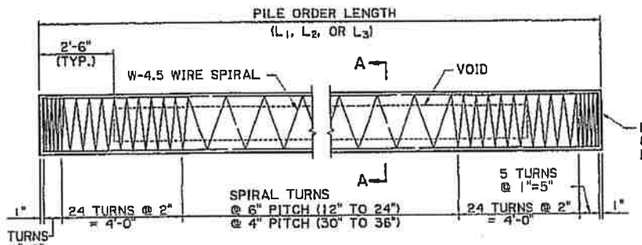
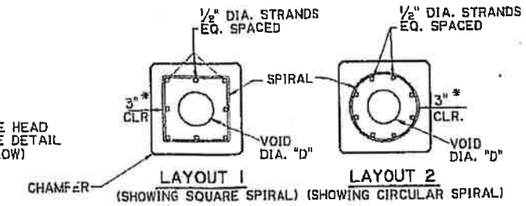
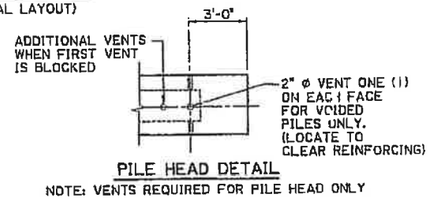


12"-24" ø SOLID
24", 30", 36" ø VOIDED
PRECAST-PRESTRESSED CONCRETE PILES
(FOR CIRCULAR AND SQUARE SPIRAL LAYOUT)



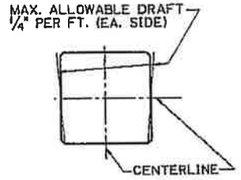
SECTION A-A

* 2" FOR 12" & 14" PILE SIZE

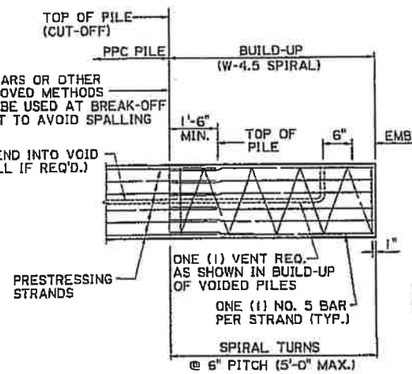


PILE HEAD DETAIL

NOTE: VENTS REQUIRED FOR PILE HEAD ONLY

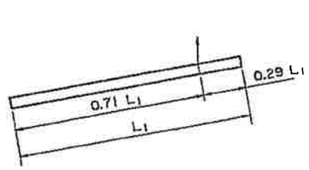


DRAFT DETAIL

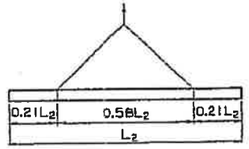


TYP. BUILD-UP WHERE
REDRIVING IS NOT REQUIRED

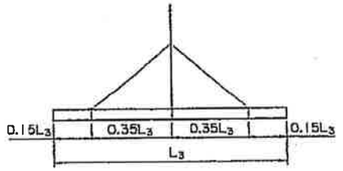
(FOR BUILD-UP IN EXCESS OF 5'-0"
SUBMIT DESIGN AND DETAILS FOR APPROVAL)



1-POINT PICK-UP



2-POINT PICK-UP



3-POINT PICK-UP

PICK-UP DETAILS

GENERAL NOTES

CONCRETE: THE CONTRACTOR SHALL DESIGN AND SUBMIT FOR APPROVAL A CONCRETE MIX WITH MINIMUM COMPRESSIVE CYLINDER STRENGTH OF 6000 psi AT 28 DAYS. CONCRETE STRENGTH AT THE TIME OF TRANSFER OF PRESTRESSED FORCE SHALL BE 4000 psi OR GREATER. BUILD-UP CONCRETE SHALL BE THE SAME DESIGN AS THE PRESTRESS CONCRETE.

PRESTRESSING STEEL: PRETENSIONED REINFORCEMENT SHALL BE ½" DIA. SEVEN-WIRE, UNCOATED LOW-RELAXATION, GRADE 270 AND SHALL CONFORM TO THE REQUIREMENTS OF AASHTO M203. AN INITIAL TENSION OF 30,980 LBS. SHALL BE APPLIED TO EACH STRAND.

DEFORMED REINFORCING STEEL: REINFORCING STEEL SHALL BE DEFORMED BILLET STEEL BARS, GRADE 60 AND SHALL MEET THE REQUIREMENTS OF AASHTO M31.

SPIRAL REINFORCING STEEL: SPIRAL REINFORCEMENT SHALL BE SIZE W-4.5 COLD-DRAWN STEEL WIRE AND SHALL CONFORM TO AASHTO M 32M.

FABRICATION TOLERANCES: MANUFACTURE OF THE PILING AND FABRICATION TOLERANCES SHALL BE IN ACCORDANCE WITH THE "MANUAL FOR QUALITY CONTROL FOR PLANTS AND PRODUCTION OF STRUCTURAL PRECAST CONCRETE PRODUCTS (MNL-116, LATEST EDITION)" PUBLISHED BY PCI.

CHAMFERS AND CORNERS: ON PILES 18" ø OR SMALLER, ALL EXPOSED CONCRETE CORNERS ARE TO HAVE ¾" CHAMFERS. ON PILES 20" ø OR LARGER, ALL EXPOSED CONCRETE CORNERS ARE TO HAVE 1½" CHAMFERS. A 1" RADIUS CURVE WILL BE PERMITTED IN LIEU OF CHAMFERS SHOWN ABOVE. HOWEVER, ALL PILES FURNISHED SHALL BE OF THE SAME CONFIGURATION.

PICK-UP AND HANDLING: LOADING CRITERIA ARE BASED ON CAREFUL HANDLING OF THE PILE. ROTATION OF THE PILE IN THE SLING IS TO BE PREVENTED UNTIL THE PILE IS IN THE VERTICAL POSITION. PICK-UP POINTS FOR ALL PILES ARE TO BE CLEARLY MARKED ON PILES. SUPPORTS FOR STORAGE SHALL BE AT PICK-UP POINTS (FOR 1-POINT PICK-UP USE SUPPORT 0.29L_1 FROM EACH END). PILES WILL BE MADE AT A CENTRAL PLANT AND BE TRANSPORTED TO THE BRIDGE SITE. ALL PRESTRESSED PILING SHALL BE HELD AT THE PLANT FOR 14 DAYS AFTER CASTING, PROVIDING THE COMPRESSIVE STRENGTH OF 6000 psi HAS BEEN ATTAINED. PICK-UP POINTS SHOWN MAY BE MODIFIED FOR TRANSPORTATION PURPOSES, PROVIDED THE PILE STRESSES ARE IN ACCORDANCE WITH DESIGN CRITERIA. THE MODIFIED PICK-UP POINTS SHALL BE SENT TO THE BRIDGE DESIGN ENGINEER FOR REVIEW.

DRIVING: PILES SHALL BE DRIVEN TO AT LEAST THE MINIMUM TIP ELEVATION AS SHOWN ON CONTRACT PLANS UNLESS OTHERWISE DIRECTED BY THE ENGINEER. PILES SHALL BE DRIVEN TO TOLERANCES SPECIFIED IN THE STANDARD SPECIFICATIONS.

PRESTRESS LOSSES: BASED ON "RECOMMENDATION FOR ESTIMATING PRESTRESSED LOSSES" PCI JOURNAL VOL. 20 JULY/AUGUST, 1975. PERCENT OF ULTIMATE SHRINKAGE EQUAL TO 31% AND 62% FOR 14 DAYS AND 90 DAYS RESPECTIVELY. PERCENT ULTIMATE CREEP EQUAL TO 26% AND 51% FOR 14 DAYS AND 90 DAYS RESPECTIVELY.

ALLOWABLE STRESSES: THE MAXIMUM LENGTHS FOR PICK-UP HAVE BEEN DETERMINED USING THE FOLLOWING ALLOWABLE STRESS (1998 AASHTO LRFD BRIDGE SPECS. 5.9.4.2.1, 5.9.4.1.2 & 5.13.4.4.3) AT BOTH 14 DAYS AND 90 DAYS.

ALLOWABLE TENSILE STRESS (psi): $5\sqrt{f'_c}$
ALLOWABLE COMPRESSIVE STRESS (psi): $0.45f'_c$
IMPACT FACTOR: 1.5
MIN. FINAL COMPRESSIVE STRESS: 725 psi

6000 psi

DETAILS THIS SHEET NOT TO SCALE

PILE INFORMATION

PILE SIZE (in.)	SECTION PROPERTIES					SQUARE SPIRAL LAYOUTS						CIRCULAR SPIRAL LAYOUTS					
	VOID "d" (in.)	AREA (in.²)	SECTION MODULUS OF (in.³x10³)	WEIGHT PER FOOT (lb/ft)	CHAMFER (in.)	NO. OF STRANDS	PRESTRESS IN CONCRETE (psi)		MAX. CASTING LENGTH (ft)			NO. OF STRANDS	PRESTRESS IN CONCRETE (psi)		MAX. CASTING LENGTH (ft)		
							AT RELEASE	AT 90 DAYS	L_1	L_2	L_3		AT RELEASE	AT 90 DAYS	L_1	L_2	L_3
12 SOLID	0	144	288	150	¾"	4	830	774	53.7	76.1	108.6	6	1227	1133	61.5	87.0	124.2
14 SOLID	0	196	457	204	¾"	8	1203	1116	66.0	93.4	133.4	8	1203	1116	66.0	93.4	133.4
16 SOLID	0	256	683	267	¾"	12	1373	1273	67.6	95.7	136.7	11	1264	1175	70.1	99.2	141.7
18 SOLID	0	324	972	338	¾"	12	1096	1026	72.6	102.7	146.7	13	1183	1106	74.6	105.6	150.8
20 SOLID	0	400	1333	417	1½"	16	1180	1106	78.7	111.3	159.0	16	1180	1106	78.7	111.3	159.0
24 SOLID	0	576	2304	600	1½"	24	1227	1154	86.7	122.7	175.2	24	1227	1154	86.7	122.7	175.2
24 VOIDED	10.5	489	2254	510	1½"	20	1204	1119	92.9	131.4	187.7	20	1204	1119	92.9	131.4	187.7
30 VOIDED	16.5	686	4257	715	1½"	28	1203	1120	107.8	152.6	217.9	28	1203	1120	107.8	152.6	217.9
36 VOIDED	22.5	898	7077	936	1½"	36	1182	1102	120.8	170.9	244.1	37	1213	1131	121.9	172.5	246.4

JENAH HAKHLEH
REG. No. 19592
REGISTERED
PROFESSIONAL ENGINEER
IN
CIVIL ENGINEERING
7/30/2003

SHEET NUMBER 130a
JEFFERSON | 61301(008) | 742-26-0030
DESIGNED A. ALLEN | CHECKED P. FOSSIER | DETAILED R. SCALLAN | CHECKED J. NAKHLEH | DATE 7-30-2003 | SHEET 19 OF 9
PRECAST-PRESTRESSED CONCRETE PILES
12, 14, 16, 18, 20 AND 24 INCH SOLID
24, 30 AND 36 INCH VOIDED
CLASS PMJ CONCRETE
BRIDGE AND STRUCTURAL DESIGN

$$ ---- SUBMITTAL STAGE ---- $$

FOUNDATION ANALYSIS

FOUNDATION ANALYSIS SUMMARY

The analysis was performed as needed to recommend a stable and economical pile-founded floodwall. Geotechnical data was limited to previous data. As such, for the East and West of Kearny typical floodwall reaches, foundations were developed for both a shallow bedrock and a deeper bedrock. Micro-Piles were selected were rock was shallow. A 10-foot anchor socket was used which developed sufficient tension capacity. Where bedrock was deep, deeper than 50 feet, an H-Pile was used. The deeper piles gained most of their capacity through friction, very little was attributed to end bearing. Multiple pile spacing's were considered. With limited geotechnical data, an acceptable design was considered achieved when at least 75% of the available soil capacity or 75% of the structural combined bending/axial capacity was reached. To assure redundancy, no less than two rows of four piles per row were considered for each 50-foot monolith. Pile capacities included a factor of safety equal to 2.0 for normal operational load cases. Pile foundations were checked considering the pile head to base connection as both fixed and pinned. Pile foundation analysis did not include down-drag, or instability forces. Down-drag would occur if the foundation design included a fill surcharge load. Instability would occur where the piles would experience lateral forces from a wedge failure (similar to soil nailing).

Reaches, short in length and at undetermined locations, may require special HTRW consideration. It was assumed that no excavation and drilling/coring of piles was permitted in these limited reaches. Floodwalls may be constructed on top of the existing ground surface. Driven piles provide bearing, sheet piling provides cutoff. Given the potential for increased corrosion, as is found in contaminated soils, the precast prestressed concrete (PPC) pile was recommended for bearing and vinyl sheet piling for cutoff. Where the soil properties are low to moderate in corrosion severity, H-piles and steel sheet piling are acceptable. L-Walls should also be considered if one of the lower top of wall elevations (El 12 or 14 feet NAVD) is selected in the final design. In that the L-Pile cutoff piling also acts as a bearing pile, the sheet piling must be steel. Vinyl is acceptable for piling acting purely as cutoff, but not when it is also providing support and subject to both axial and flexural stresses. In the final design, it is recommended that the rate of corrosion be established testing both the soil and extent of stray currents.

Limited space for floodwall construction along riverfront reaches required special consideration. There exists a footprint of approximately 15 feet in width between the river top of bank and industrial buildings. A narrow corridor for floodwall construction. Driving battered piles, standard practice for structures resisting lateral loads in soft soils, would be problematic. Piles battered towards the protected side could conflict with the building foundations. Battered Piles driven towards the river would need to be hung over the buildings during driving and would have reduced capacity given the close proximity to the slope. The solution provided is an all vertical pile foundation. The number of piles was increased to maintain the established criteria. Additionally, pile embedment was increased into the base to assure a fixed connection was established.

Passaic River
Tidal Flood Protection System

T-Wall Foundation Analysis

Note: All axial forces below are local forces in acting in the axis of the pile.

East of Kearny - Typical Monolith (Bedrock < 50')

Monolith	50 ft.	Pile	9.5" Micropile
Top of Wall	EL 118	Pile Qty	10
Top of Slab	EL 106	F/S Batter	3 : 1
SWL	EL 116	P/S Batter	3 : 1

LC	Load Case Description	Axial Load, Tension (kip)	Axial Load, Compression (kip)
1	Construction	0.0	46.7
2	Construction + Wind on F.S.	0.0	51.9
3	Water to SWL, Impervious	32.0	105.9
4	Water to SWL, Pervious	29.3	103.1
5	Water to SWL + Debris Load, Impervious	42.6	116.4
6	Water to T.O. Wall, Impervious	57.8	132.9
7	Water to T.O. Wall, Pervious	54.6	129.8

East of Kearny - Typical Monolith (Bedrock > 50')

Monolith	50 ft.	Pile	HP14x73
Top of Wall	EL 118	Pile Qty	10
Top of Slab	EL 106	F/S Batter	3 : 1
SWL	EL 116	P/S Batter	3 : 1

LC	Load Case Description	Axial Load, Tension (kip)	Axial Load, Compression (kip)
1	Construction	0.0	46.5
2	Construction + Wind on F.S.	0.0	50.9
3	Water to SWL, Impervious	32.6	106.5
4	Water to SWL, Pervious	30.4	104.2
5	Water to SWL + Debris Load, Impervious	42.6	116.4
6	Water to T.O. Wall, Impervious	58.3	133.4
7	Water to T.O. Wall, Pervious	55.7	130.9

West of Kearny - Typical Monolith (Bedrock < 50')

Monolith	50 ft.	Pile	9.5" Micropile
Top of Wall	EL 118	Pile Qty	10
Top of Slab	EL 106	F/S Batter	3 : 1
SWL	EL 116	P/S Batter	3 : 1

LC	Load Case Description	Axial Load, Tension (kip)	Axial Load, Compression (kip)
1	Construction	0.0	46.7
2	Construction + Wind on F.S.	0.0	51.9
3	Water to SWL, Impervious	32.0	105.9
4	Water to SWL, Pervious	29.3	103.1
5	Water to SWL + Debris Load, Impervious	42.6	116.4
6	Water to T.O. Wall, Impervious	57.8	132.9
7	Water to T.O. Wall, Pervious	54.6	129.8

West of Kearny - Typical Monolith (Bedrock > 50')

Monolith	50 ft.	Pile	HP14x73
Top of Wall	EL 118	Pile Qty	10
Top of Slab	EL 106	F/S Batter	3 : 1
SWL	EL 116	P/S Batter	3 : 1

LC	Load Case Description	Axial Load, Tension (kip)	Axial Load, Compression (kip)
1	Construction	0.0	46.5
2	Construction + Wind on F.S.	0.0	51.0
3	Water to SWL, Impervious	33.1	107.0
4	Water to SWL, Pervious	30.9	104.7
5	Water to SWL + Debris Load, Impervious	43.1	117.0
6	Water to T.O. Wall, Impervious	59.0	134.1
7	Water to T.O. Wall, Pervious	56.4	131.5

HTRW Sites - Typical Monolith (Bedrock > 50')

Monolith	50 ft.	Pile	12" PPC
Top of Wall	EL 118	Pile Qty	16
Top of Slab	EL 110.5	F/S Batter	3 : 1
SWL	EL 116	P/S Batter	3 : 1

LC	Load Case Description	Axial Load, Tension (kip)	Axial Load, Compression (kip)
1	Construction	0.0	24.4
2	Construction + Wind on F.S.	0.0	25.0
3	Water to SWL, Impervious	1.4	35.9
4	Water to SWL, Pervious	0.3	34.8
5	Water to SWL + Debris Load, Impervious	6.5	41.0
6	Water to T.O. Wall, Impervious	10.6	45.9
7	Water to T.O. Wall, Pervious	5.1	40.4

Water Front (Bedrock > 50')

Monolith	50 ft.	Pile	HP14x89
Top of Wall	EL 118	Pile Qty	20
Top of Slab	EL 106	F/S Batter	Vertical
SWL	EL 116	P/S Batter	Vertical

LC	Load Case Description	Axial Load, Tension (kip)	Axial Load, Compression (kip)
1	Construction	0.0	22.6
2	500yr SWL, Impervious	0.0	28.8
3	500yr SWL, Pervious	31.5	66.5
4	500yr SWL + 500 yr Wave, Impervious	29.4	64.5
5	500yr SWL + 100 yr Wave + Debris, Impervious	39.1	74.2
6	Water to T.O. Wall, Impervious	50.0	85.7
7	Water to T.O. Wall, Pervious	47.6	83.3

T-WALL

EAST OF KEARNY

USACE Passaic River Flood Protection

Feasibility Level Flood Protection

TYPICAL EAST OF KEARNY MONOLITH
TOW EL. 18; TOS EL 6.0

URS Project : 60442748

Monolith Foundations

1515 Poydras Street

Suite 2700

New Orleans, LA 70112

(504) 586-8111

Computed by: RBJ Checked by: _____

Date: Jan-16 Date: _____

AECOM

Job USACE Passaic River Flood Protection
Feasibility Level Flood Protection

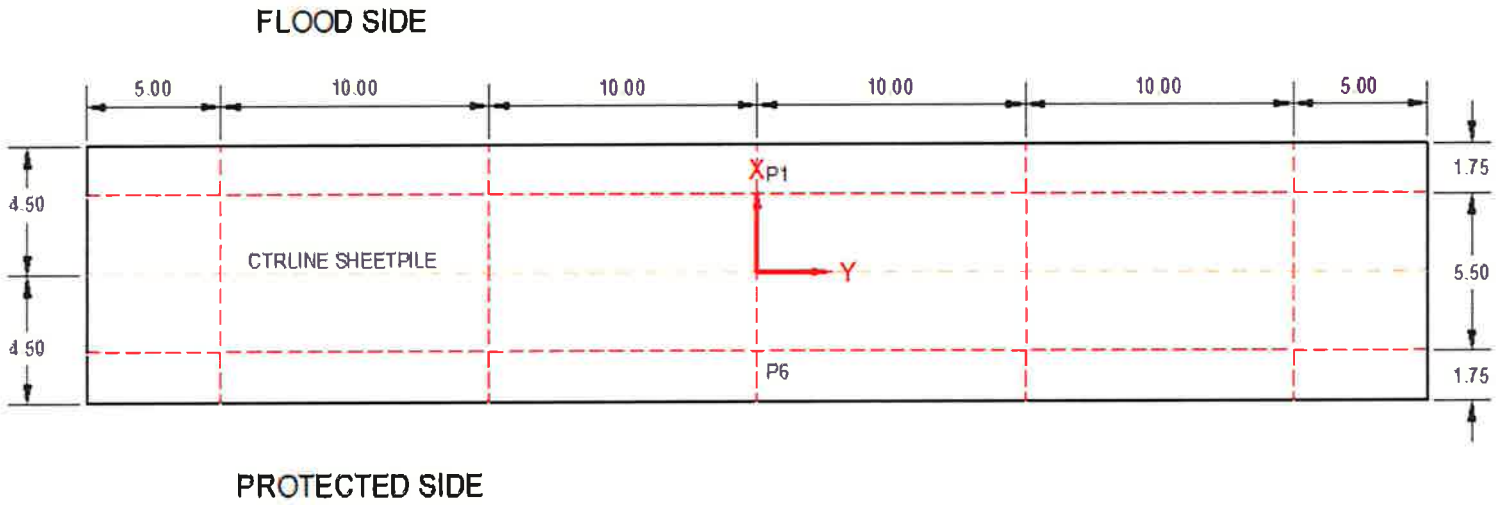
Description TYPICAL WEST OF KEARNY MONOLITH
TOW EL. 18; TOS EL 6.0
Monolith Foundation Layout & Axis

Project No. 60442748

Computed by RBJ Date Jan-16

Checked by 0 Date Jan-00

TOW EL. 18; TOS EL 6.0



AECOM

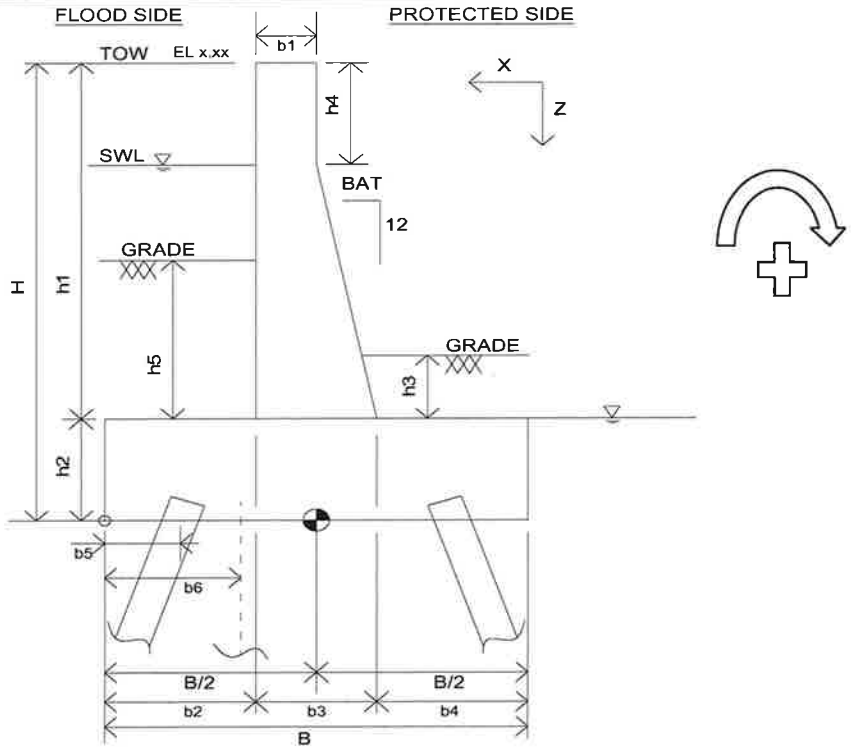
Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL EAST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00

References

LOAD CALCULATIONS

TOW EL. 18; TOS EL 6.0

Top of Wall EL.	18.0	NAVD88
Normal Water El.	6.0	NAVD88
Still Water El.	16.0	NAVD88
Top of Slab EL.	6.0	NAVD88
H=	15.0	ft.
h1=	12.0	ft.
h2=	36.0	in. (Base Slab Height)
h3=	12.0	in. (P.S. Soil Height)
h4=	0.0	ft.
h5=	1.00	ft. (F.S. Soil Height, **T.O. Slab**)
B=	9.0	ft. (Base Slab Width)
b1=	18.0	in. (Wall Stem Width, top)
b2=	66.0	in. (F.S. Slab Width)
b3=	18.0	in. (Wall Stem Width, bottom)
b4=	24.0	in. (P.S. Slab Width)
b5=	21.0	in. (F.S. Pile Row Edge Space)
b6=	54.0	in. (Sheet Pile Edge Space)
BAT=	0.0	(Wall Batter, N/A)
PS Grade =	7.00	NAVD88 (Average of PS soil for all)
Monolith Length	50.0	ft
Bottom Of Slab =	3.0	NAVD88



T-WALL CROSS-SECTION

Notes: 1) positive 'Y' axis is into page
2) pile batters vary from those show in diagram

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL EAST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

ASSUMED UNIT WEIGHTS:

Unit Weight of Storm Water:
62.4 pcf

Wet Unit Weight of Soil: 110 pcf

Unit Weight of Riprap: 0 pcf

Sat Unit Weight of Soil:
47.6 pcf

Unit Weight of Concrete
150 pcf

Unbalanced Load for Stability Analysis:

F_{cap} (k/ft) = 0.00 (Operation Case; Force acts at bottom of slab)

F_{cap} (k/ft) = 0.00 (Water to TOW Case; Force acts at bottom of slab)

K_0, Clay fill = 0.80 (for lateral soil forces)

EM 1110-2-2502: $P_{hz} = K_0 * (\gamma * z_w + \gamma' * (z - z_w))$

- PS design water elevation <BOS elevation, no PS uplift or PS hydrostatic loads

FS Wind force above SWL= 50.0 psf

Construction Surcharge Pressure = 200 psf

Wave Point Load = 0 kip/ft

Dist from B.O. Slab to Wave Load = 0 ft

	LOAD CASES ANALYZED:	FS Water El.	PS Water El.	Overstress for Design	Notes
1	Construction	6.0	7.0	1.17	Temporary Construction Case
2	Construction + Wind on F.S.	6.0	7.0	1.33	Unusual Construction Case
3	Water to SWL, Impervious	16.0	7.0	1.00	Still Water Level Flood, Impervious uplift
4	Water to SWL, Pervious	16.0	7.0	1.00	Still Water Level Flood, Pervious uplift
5	Water to SWL + Debris Load, Impervious	16.0	7.0	1.00	Still Water Level Flood, Impervious uplift
6	Water to T.O. Wall, Impervious	18.0	7.0	1.33	Flood, Impervious uplift
7	Water to T.O. Wall, Pervious	18.0	7.0	1.33	Flood, Pervious uplift

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL EAST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

All forces on this page are per foot of monolith length

Soil Forces:

Water EL.	FS Soil EL.	PS Soil EL.	Wt. of FS Soil (k/ft)	Wt. of PS Soil (k/ft)	FS Soil Lateral Force (k/ft)	PS Soil Lateral Force (k/ft)
7.0	7.00	7.00	**0.2618**	**0.220**	**-0.3046**	**0.7040**
16.0	7.00	7.00	**0.2618**	**0.220**	**-0.3046**	**0.7040**
18.0	7.00	7.00	**0.2618**	**0.220**	**-0.3046**	**0.7040**

Water Forces:

Water EL.	Wt. of FS Water (k/ft)	FS Water Lateral Force (k/ft)
7.0	**0.3432**	**-0.499**
16.0	**3.432**	**-5.273**
18.0	**4.1184**	**-7.020**

Wind Force: 50 psf * monolith height / 1000 = **-0.75 k/ft**

MDE Earthquake Force:

	Vertical (k-ft)	Lateral (k-ft)		
Soil		**0.127**	Z-coord = -2.67	
Concrete	**0.270**	**0.844**	Z-coord = -4.50	X-coord = -0.700

Surcharge Forces:

F.S. = 200 psf * F.S. width / 1000 = **1.1 k/ft**

P.S. = 200 psf * P.S. width / 1000 = **0.4 k/ft**

Unbalanced Load:

Operating: 0.000 k/ft in (+) X Direction, acting at bottom of slab (Z-coordinate = 0)

TOW: 0.000 k/ft in (+) X Direction, acting at bottom of slab (Z-coordinate = 0)

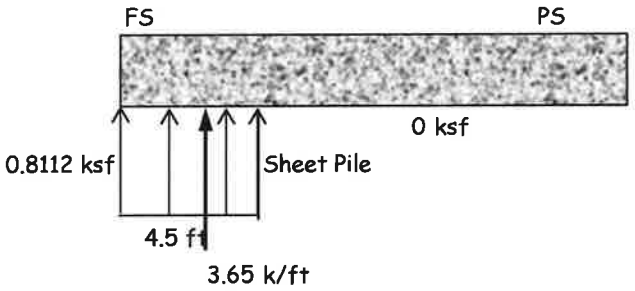
AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL EAST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
					References

Uplift Loads:

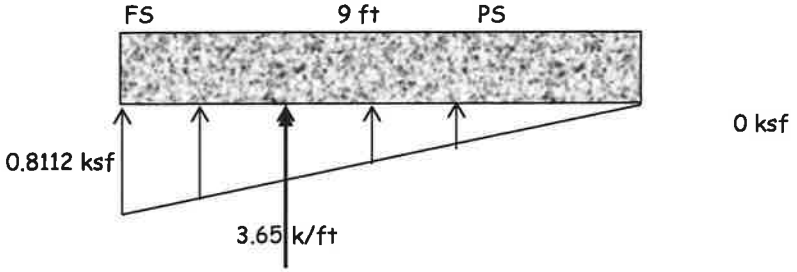
Water to SWL:

Impervious:



X-coordinate = 2.25

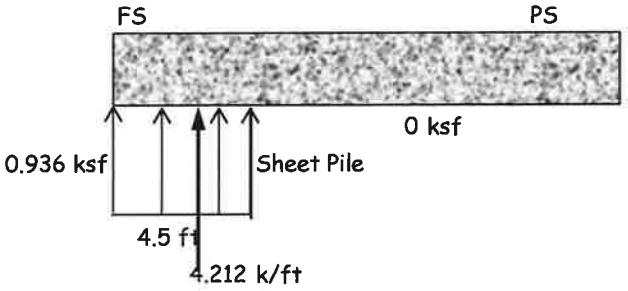
Pervious:



X-coordinate = 1.500

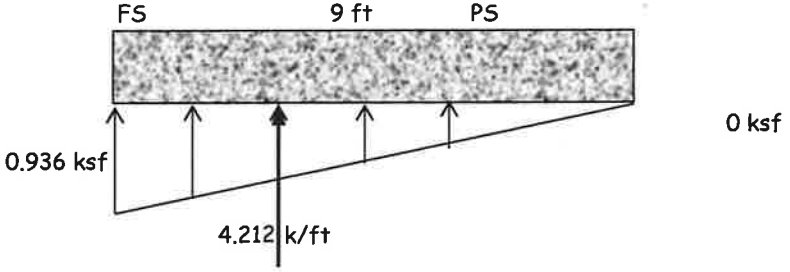
Water to TOW:

Impervious:



X-coordinate = 2.25

Pervious:



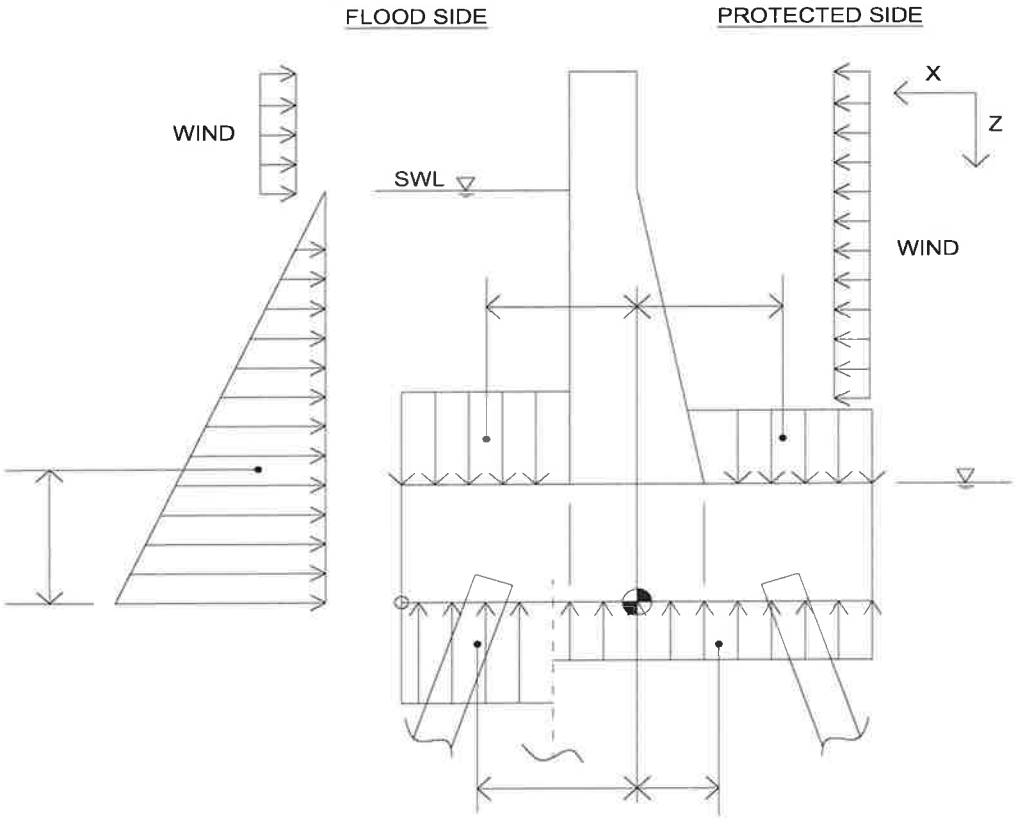
X-coordinate = 1.500

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL EAST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00

References

LOAD DIAGRAMS FOR IMPERVIOUS SHEET PILE CONDITIONS



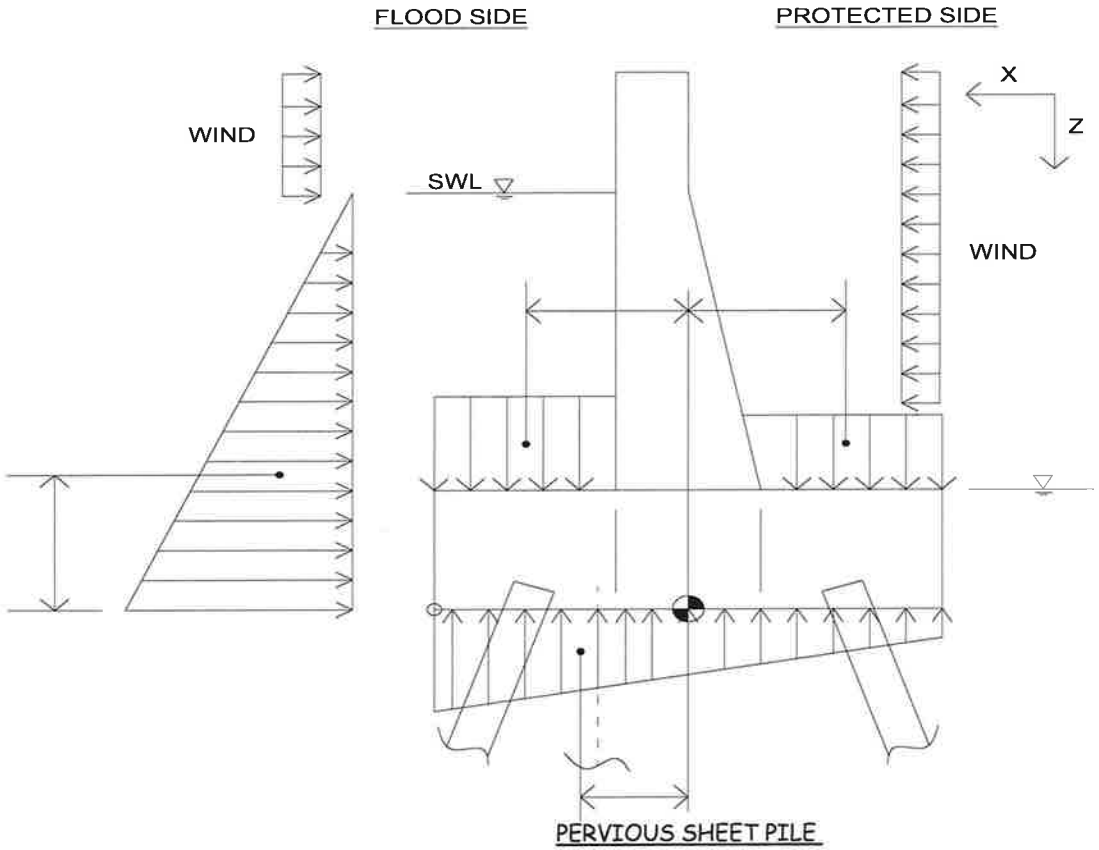
IMPERVIOUS SHEET PILE

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL EAST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00

References

LOAD DIAGRAMS FOR PERVIOUS SHEET PILE CONDITIONS



AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748
	Feasibility Level Flood Protection		
Description	TYPICAL EAST OF KEARNY MONOLITH	Computed by	RBJ
	TOW EL. 18; TOS EL 6.0	Date	Jan-16
	Foundation Load Cases & Combinations	Checked by	0
		Date	Jan-00
			References

LC 1: Construction, 1.17 allowable overstress								
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FOOT OF WALL LENGTH								
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
		2.70	-1.75				4.73		wall stem weight
		4.05	0.00				0.00		base slab weight
		0.22	-3.50				0.77		P.S. soil weight
	0.70				-1.33		-0.94		P.S. lateral soil force
		0.26	1.75				-0.46		F.S. soil weight
	-0.30				-1.33		0.41		F.S. lateral soil force
	0.00				0.00		0.00		Unbalanced load, no water
Subtotal A	*0.40*	*7.23*				*0.00*	*4.50*	*0.00*	*Dry Soil Weight Subtotal*
		1.10	1.75				-1.93		F.S. Equipment Surcharge
		0.40	-3.50				1.40		P.S. Equipment Surcharge
SUM:	**0.40**	**8.73**				**0.00**	**3.98**	**0.00**	***LC 1 CPGA Forces***

LC 2: Construction + Wind on F.S., 1.33 allowable overstress								
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FOOT OF WALL LENGTH								
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
		2.70	-1.75	0			4.73	0	wall stem weight
		4.050	0				0.00		base slab weight
	-0.75				-7.50	0	5.63	0	FS Wind Load
SUM:	**-0.75**	**6.75**				**0.00**	**10.35**	**0.00**	***LC 2 CPGA Forces***

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL EAST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00

References

LC 3: Water to SWL, Impervious, 1.0 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FOOT OF WALL LENGTH									
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
		2.70	-1.75				4.73		wall stem weight
		4.05	0.00				0.00		base slab weight
		0.22	-3.50				0.77		P.S. soil weight
	0.70				-1.33		-0.94		P.S. lateral soil force
		0.26	1.75				-0.46		F.S. soil weight, saturated
	-0.30				-1.33		0.41		F.S. lateral soil force, saturated
	-5.27				-4.33		22.85		Lateral water force
		3.43	1.75				-6.01		Vertical water force
	0.00				0		0.00		Unbalanced load, water TOW
Subtotal B	*-4.87*	*10.66*				*0.00*	*21.35*	*0.00*	*SWL Subtotal*
		-3.65	2.25				8.21		Impervious FS hydrostatic uplift
		0.00					0.00		Impervious PS hydrostatic uplift
SUM:	**-4.87**	**7.01**				**0.00**	**29.56**	**0.00**	**LC 3 CPGA Forces**

LC 4: Water to SWL, Pervious, 1 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FOOT OF WALL LENGTH									
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
Subtotal B	*-4.87*	*10.66*				*0.00*	*21.35*	*0.00*	*SWL Subtotal*
		-3.65	1.500				5.48		Pervious hydrostatic uplift
SUM:	**-4.87**	**7.01**				**0.00**	**26.82**	**0.00**	**LC 4 CPGA Forces**

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL EAST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00

References

LC 5: Water to SWL + Debris Load, Impervious, 1.0 allowable overstress								
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FT. OF WALL								
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
Subtotal B	*-4.87*	*10.66*				*0.00*	*21.35*	*0.00*	*SWL Subtotal*
		-3.65	2.25				8.21		Impervious FS hydrostatic uplift
		0.00					0.00		Impervious PS hydrostatic uplift
	-0.50				-13.00		6.50		Debris Load
SUM:	**-5.37**	**7.01**				**0.00**	**36.06**	**0.00**	**LC 5 CPGA Forces**

LC 6: Water to T.O. Wall, Impervious, 1.33 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FT. OF WALL									
S NO.	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
		2.70	-1.75				4.73		wall stem weight
		4.05	0.00				0.00		base slab weight
		0.22	-3.50				0.77		P.S. soil weight
	0.70				-1.33		-0.94		P.S. lateral soil force
		0.26	1.75				-0.46		F.S. soil weight, saturated
	-0.30				-1.33		0.41		F.S. lateral soil force, saturated
	-7.02				-5.00		35.10		Lateral water force
		4.12	1.75				-7.21		Vertical water force
	0.00				0		0.00		Unbalanced load, water TOW
Subtotal B	*-6.62*	*11.35*				*0.00*	*32.40*	*0.00*	*TOW Subtotal*
		-4.21	2.25				9.48		Impervious FS hydrostatic uplift
		0.00					0.00		Impervious PS hydrostatic uplift
SUM:	**-6.62**	**7.14**				**0.00**	**41.87**	**0.00**	**LC 6 CPGA Forces**

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL EAST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

LC 7: Water to T.O. Wall, Pervious, 1.33 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FT. OF WALL									
S NO.	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
Subtotal B	-6.62	11.35				0.00	32.40	0.00	*TOW Subtotal*
		-4.21	1.500				6.32		Pervious hydrostatic uplift
SUM:	**-6.62**	**7.14**				**0.00**	**38.72**	**0.00**	**LC 7 CPGA Forces**

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL EAST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

Summary of Forces and Moments (For Full Length of Monolith):

LOAD CASE NUMBER	Fx	Fy	Fz	Mx	My	Mz
	kips	kips	kips	(kip-ft)	(kip-ft)	(kip-ft)
1	20.0	0.0	436.6	0.0	199.0	0.0
2	-37.5	0.0	337.5	0.0	517.5	0.0
3	-243.7	0.0	350.7	0.0	1478.0	0.0
4	-243.7	0.0	350.7	0.0	1341.1	0.0
5	-268.7	0.0	350.7	0.0	1803.0	0.0
6	-331.0	0.0	356.9	0.0	2093.7	0.0
7	-331.0	0.0	356.9	0.0	1935.8	0.0

AECOM

Job USACE Passaic River Flood Protection
Feasibility Level Flood Protection

Project No. 60442748

Description TYPICAL EAST OF KEARNY MONOLITH
TOW EL. 18; TOS EL 6.0
Soil & Pile Information Required for CPGA

Computed by RBJ Date Jan-16

Checked by 0 Date Jan-00

References

Pile Layout, SW-11:

Row 1			Row 2		
pile no.	x	y	pile no.	x	y
1	2.25	-20.00	6	-2.25	-20.00
2	2.25	-10.00	7	-2.25	-10.00
3	2.25	0.00	8	-2.25	0.00
4	2.25	10.00	9	-2.25	10.00
5	2.25	20.00	10	-2.25	20.00

Tip Elevation:

(For CPGA, need Tip Elevation as a function of CPGA Axis at B.O. Slab, +Z points downward)

B.O.S. Elevation = 4 NAVD88 * Socketed 12' min into bedrock
Bedrock EL = -45 NAVD89
"TIP" in CPGA = 61 ft

Pile Properties & Attributes: (See hand calculations for micropiles)

E_s = 3,605,000 psi
A = 181 in^2
$I_x = I_y$ = 1534 in^4
C_{33} = 1.2 (factor for method of axial load transfer from pile to soil; = 1 for full tip bearing, = 2 for full skin friction)

Allowable Compression (AC) = 160 kips
Allowable Tension (AT) = 100 kips
PO = 1597 kips
PT = 1010 kips
PB = 550 kips
MB = 2825 kip-in
MO = 2200 kip-in

Es Value for CPGA Run:

Monolith width = 50 ft
N_h = 30 pci = 0.03

GROUP FACTORS	
Pile Spacing in Direction of Loading	From EM1110-2-2906
	D
3B	0.33
4B	0.38
5B	0.45
6B	0.56
7B	0.71
8B	1

Group reduction is based on distance between piles in direction of loading. This includes distance due to battering and is taken over the distance 10*d_{pile} (point of fixety).

Assume a batter of 3
B = d_{pile} = 9.63 in = 0.803 ft

Distance between piles at B.O. Slab = 4.50 ft
Average distance between piles over 10*dpile = 9.85 ft

Average distance between piles in terms of pile width B = 12.274 B

Group Reduction "D" value for this distance = 1

Therefore, Es including group reduction = 0.0300 kci

AECOM

Job	USACE Passaic River Flood Protection		Project No.	60442748	
	Feasibility Level Flood Protection				
Description	TYPICAL EAST OF KEARNY MONOLITH		Computed by	RBJ	Date Jan-16
	TOW EL. 18; TOS EL 6.0				
	CPGA Input & Output Files		Checked by	0	Date Jan-00

TOM EL18, H-pile 10' O.C.

Input file:

```
100 EAST OF KEARNY, TOW EL18, FIX, 1:3 BATTER, HP14x73, 10'OC
200 PROP 29000 261 729 26.1 1.7 0 ALL
300 SOIL ES 0.03 TIP 60 0 ALL
400 FIX ALL
500 ALLOW H 200 53 535 498 895 2675 ALL
600 FOVSTR 1.17 1.17 1
700 FOVSTR 1.33 1.33 2 6 7
1000 BATTER 3 ALL
1100 ANGLE 180 6 TO 10
1200 PILE 1 2.5 -20 0
1300 PILE 6 -2.5 -20 0
1400 ROW Y 5 1 4 AT 10
1500 ROW Y 5 6 4 AT 10
4500 LOAD 1 20 0 436.6 0 199 0
4600 LOAD 2 -37.5 0 337.5 0 517.5 0
4700 LOAD 3 -243.7 0 350.7 0 1478 0
4800 LOAD 4 -243.7 0 350.7 0 1341 0
4900 LOAD 5 -268.7 0 350.7 0 1803 0
5000 LOAD 6 -331 0 356.9 0 2094 0
5100 LOAD 7 -331 0 356.9 0 1936 0
6000 FOUT 1 2 3 4 5 6 7 EL18HP10.doc
6100 PFO ALL
6200 PLB ALL
```

T-WALL

WEST OF KEARNY

USACE Passaic River Flood Protection

Feasibility Level Flood Protection

TYPICAL WEST OF KEARNY MONOLITH
TOW EL. 18; TOS EL 6.0

URS Project : 60442748

Monolith Foundations

1515 Poydras Street

Suite 2700

New Orleans, LA 70112

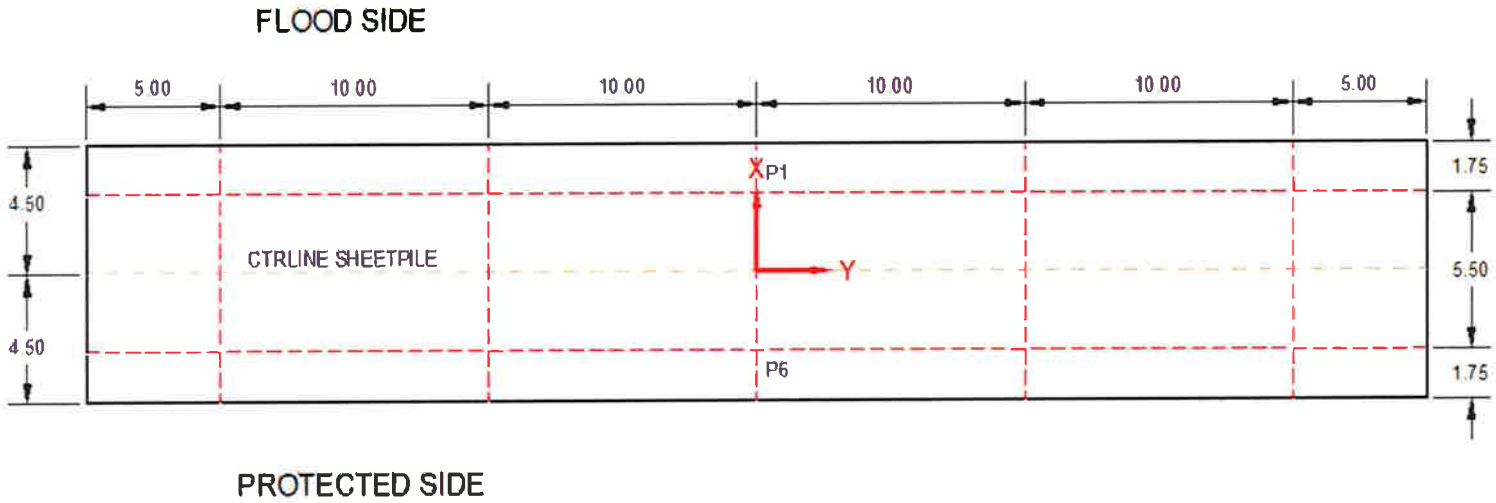
(504) 586-8111

Computed by:	RBJ	Checked by:	
Date:	Jan-16	Date:	

AECOM

Job	USACE Passaic River Flood Protection Feasibility Level Flood Protection	Project No.	60442748
Description	TYPICAL WEST OF KEARNY MONOLITH TOW EL. 18; TOS EL 6.0 Monolith Foundation Layout & Axis	Computed by	RBJ
		Date	Jan-16
		Checked by	0
		Date	Jan-00

TOW EL. 18; TOS EL 6.0



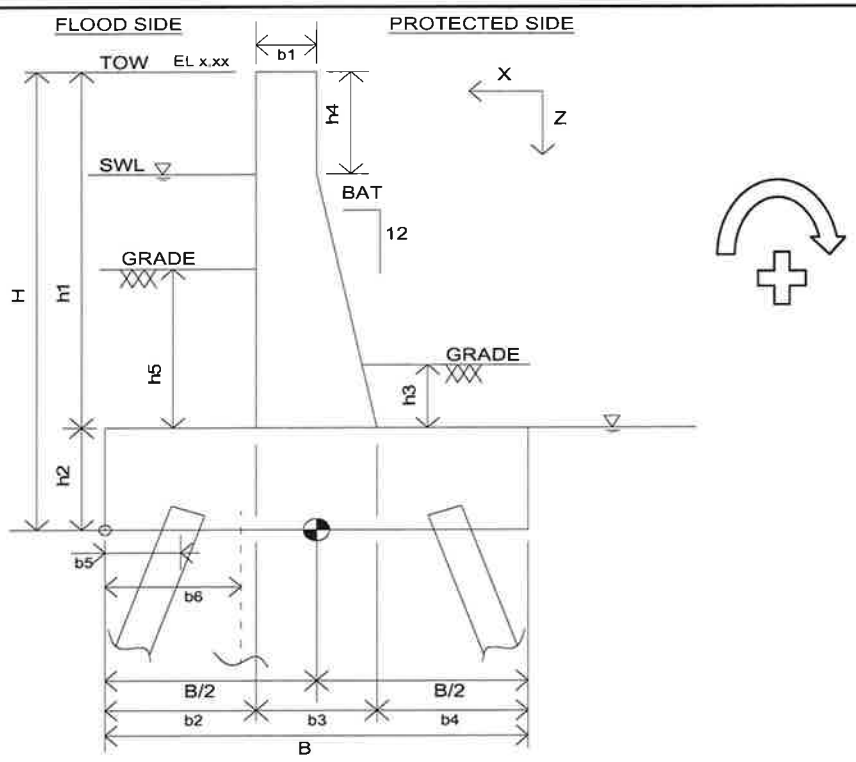
AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL WEST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

LOAD CALCULATIONS

TOW EL. 18; TOS EL 6.0

Top of Wall EL.	18.0	NAVD88
Normal Water El.	6.0	NAVD88
Still Water El.	16.0	NAVD88
Top of Slab EL.	6.0	NAVD88
H=	15.0	ft.
h1=	12.0	ft.
h2=	36.0	in. (Base Slab Height)
h3=	12.0	in. (P.S. Soil Height)
h4=	0.0	ft.
h5=	1.00	ft. (F.S. Soil Height, **T.O. Slab**)
B=	9.0	ft. (Base Slab Width)
b1=	18.0	in. (Wall Stem Width, top)
b2=	66.0	in. (F.S. Slab Width)
b3=	18.0	in. (Wall Stem Width, bottom)
b4=	24.0	in. (P.S. Slab Width)
b5=	21.0	in. (F.S. Pile Row Edge Space)
b6=	54.0	in. (Sheet Pile Edge Space)
BAT=	0.0	(Wall Batter, N/A)
PS Grade =	7.00	NAVD88 (Average of PS soil for all)
Monolith Length	50.0	ft
Bottom Of Slab =	3.0	NAVD88



T-WALL CROSS-SECTION

Notes: 1) positive 'Y' axis is into page
2) pile batters vary from those show in diagram

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL WEST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

ASSUMED UNIT WEIGHTS:

Unit Weight of Storm Water:
62.4 pcf

Wet Unit Weight of Soil: 100 pcf

Unit Weight of Riprap: 0 pcf

Sat Unit Weight of Soil:
37.6 pcf

Unit Weight of Concrete
150 pcf

Unbalanced Load for Stability Analysis:

F_{cap} (k/ft) = 0.00 (Operation Case; Force acts at bottom of slab)

F_{cap} (k/ft) = 0.00 (Water to TOW Case; Force acts at bottom of slab)

K_0, Clay fill = 0.80 (for lateral soil forces)

EM 1110-2-2502: $P_{hz} = K_0 * (\gamma * z_w + \gamma' * (z - z_w))$

- PS design water elevation <BOS elevation, no PS uplift or PS hydrostatic loads

FS Wind force above SWL= 50.0 psf

Construction Surcharge Pressure = 250 psf

Wave Point Load = 0 kip/ft

Dist from B.O. Slab to Wave Load = 0 ft

	LOAD CASES ANALYZED:	FS Water El.	PS Water El.	Overstress for Design	Notes
1	Construction	3.0	3.0	1.17	Temporary Construction Case
2	Construction + Wind on F.S.	3.0	3.0	1.33	Unusual Construction Case
3	Water to SWL, Impervious	16.0	6.0	1.00	Still Water Level Flood, Impervious uplift
4	Water to SWL, Pervious	16.0	6.0	1.00	Still Water Level Flood, Pervious uplift
5	Water to SWL + Debris Load, Impervious	16.0	6.0	1.00	Still Water Level Flood, Impervious uplift
6	Water to T.O. Wall, Impervious	18.0	6.0	1.33	TOW Flood, Impervious uplift
7	Water to T.O. Wall, Pervious	18.0	6.0	1.33	TOW Flood, Pervious uplift

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL WEST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
					References

All forces on this page are per foot of monolith length

Soil Forces:

Water EL.	FS Soil EL.	PS Soil EL.	Wt. of FS Soil (k/ft)	Wt. of PS Soil (k/ft)	FS Soil Lateral Force (k/ft)	PS Soil Lateral Force (k/ft)
7.0	7.00	7.00	**0.2068**	**0.200**	**-0.2406**	**0.6400**
16.0	7.00	7.00	**0.2068**	**0.200**	**-0.2406**	**0.6400**
18.0	7.00	7.00	**0.2068**	**0.200**	**-0.2406**	**0.6400**

Water Forces:

Water EL.	Wt. of FS Water (k/ft)	FS Water Lateral Force (k/ft)
7.0	**0.3432**	**-0.499**
16.0	**3.432**	**-5.273**
18.0	**4.1184**	**-7.020**

Wind Force: 50 psf * monolith height / 1000 = **-0.75 k/ft**

MDE Earthquake Force:

	Vertical (k-ft)	Lateral (k-ft)		
Soil		**0.116**	Z-coord = -2.67	
Concrete	**0.270**	**0.844**	Z-coord = -4.50	X-coord = -0.700

Surcharge Forces:

F.S. = 250 psf * F.S. width / 1000 = **1.375 k/ft**

P.S. = 250 psf * P.S. width / 1000 = **0.5 k/ft**

Unbalanced Load:

Operating: 0.000 k/ft in (+) X Direction, acting at bottom of slab (Z-coordinate = 0)

TOW: 0.000 k/ft in (+) X Direction, acting at bottom of slab (Z-coordinate = 0)

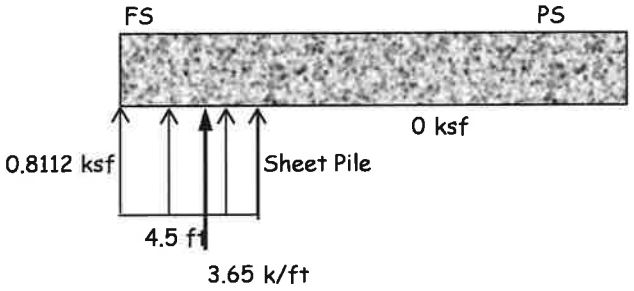
AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL WEST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

Uplift Loads:

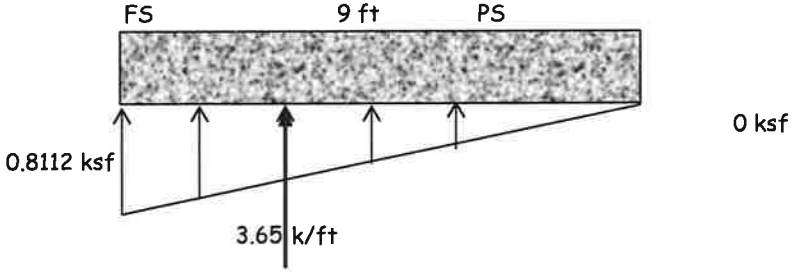
Water to SWL:

Impervious:



X-coordinate = 2.25

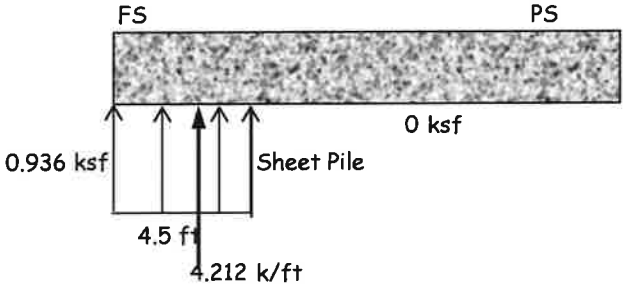
Pervious:



X-coordinate = 1.500

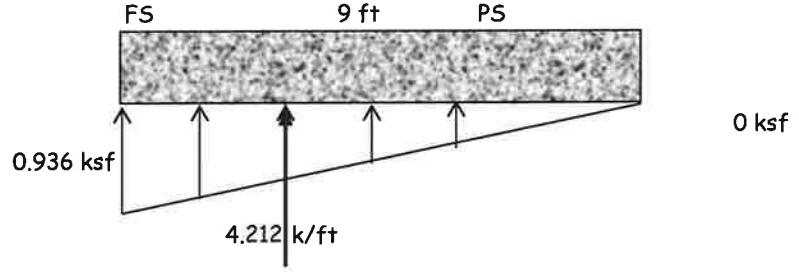
Water to TOW:

Impervious:



X-coordinate = 2.25

Pervious:



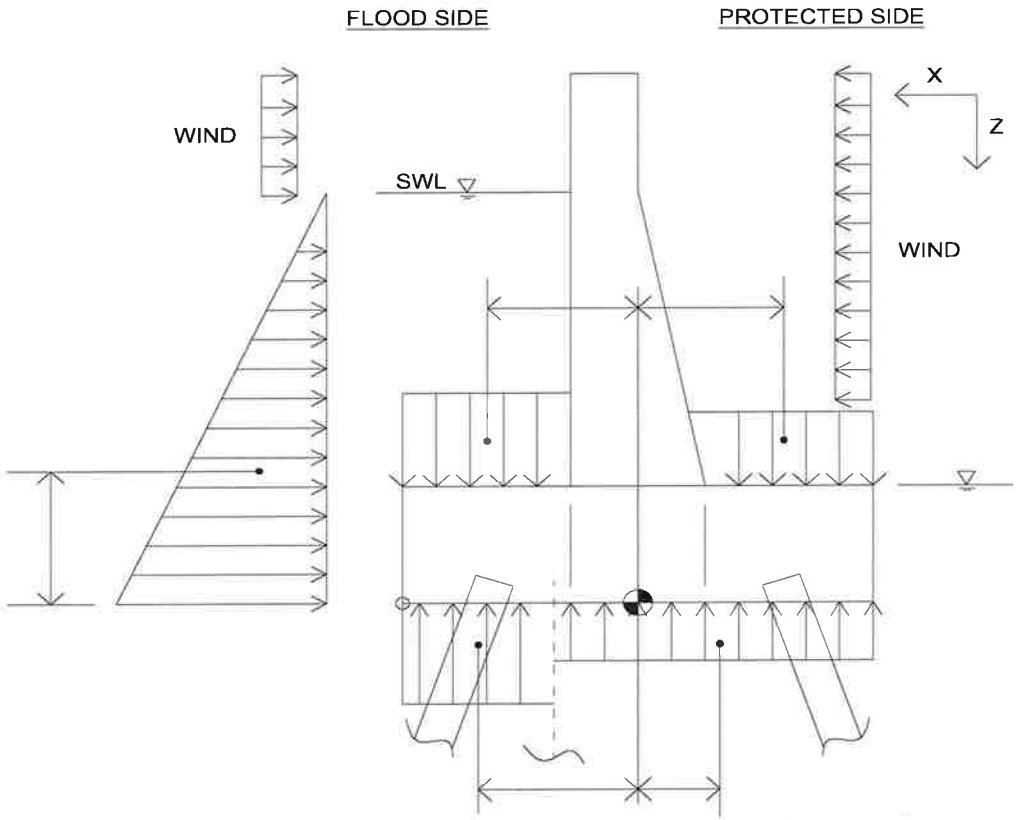
X-coordinate = 1.500

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL WEST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00

References

LOAD DIAGRAMS FOR IMPERVIOUS SHEET PILE CONDITIONS

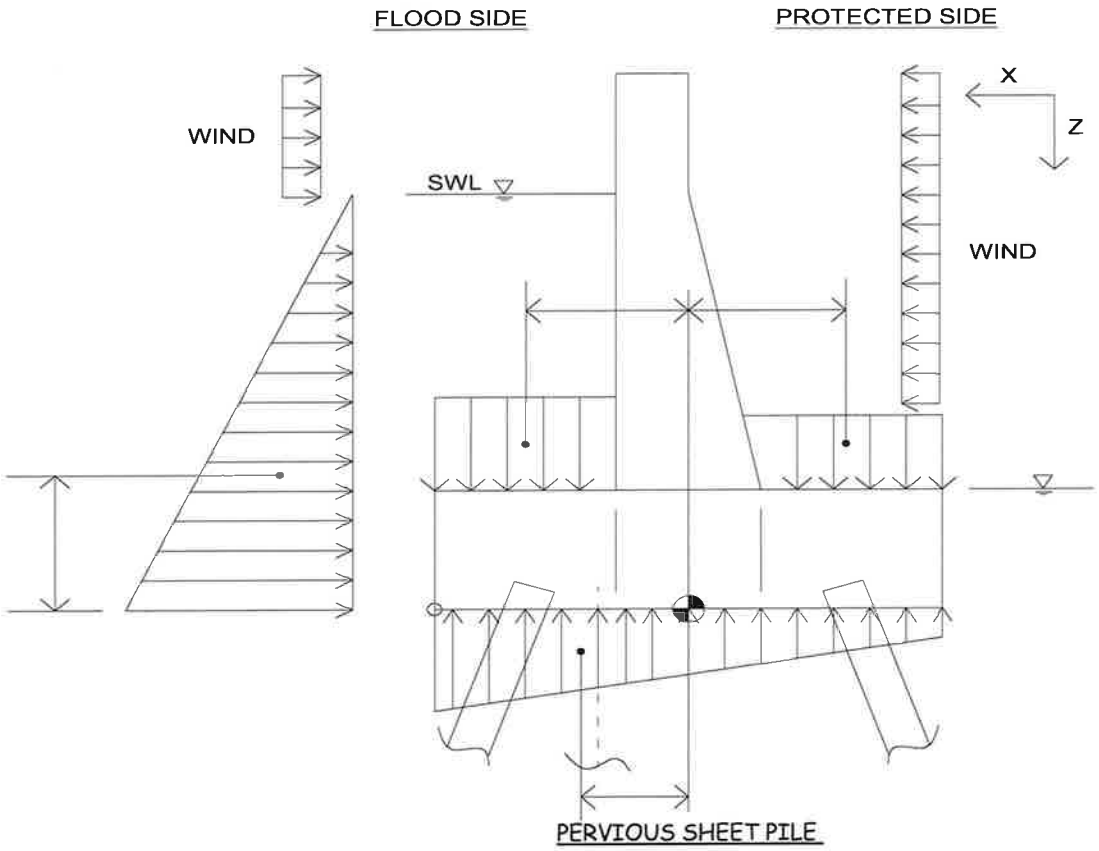


IMPERVIOUS SHEET PILE

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL WEST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
					References

LOAD DIAGRAMS FOR PERVIOUS SHEET PILE CONDITIONS



AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL WEST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00

References

LC 1: Construction, 1.17 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FOOT OF WALL LENGTH									
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
		2.70	-1.75				4.73		wall stem weight
		4.05	0.00				0.00		base slab weight
		0.20	-3.50				0.70		P.S. soil weight
	0.64				-1.33		-0.85		P.S. lateral soil force
		0.21	1.75				-0.36		F.S. soil weight
	-0.24				-1.33		0.32		F.S. lateral soil force
	0.00				0.00		0.00		Unbalanced load, no water
Subtotal A	*0.40*	*7.16*				*0.00*	*4.53*	*0.00*	*Dry Soil Weight Subtotal*
		1.38	1.75				-2.41		F.S. Equipment Surcharge
		0.50	-3.50				1.75		P.S. Equipment Surcharge
SUM:	**0.40**	**9.03**				**0.00**	**3.87**	**0.00**	**LC 1 CPGA Forces**

LC 2: Construction + Wind on F.S., 1.33 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FOOT OF WALL LENGTH									
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
		2.70	-1.75	0			4.73	0	wall stem weight
		4.050	0				0.00		base slab weight
	-0.75				-7.50	0	5.63	0	FS Wind Load
SUM:	**-0.75**	**6.75**				**0.00**	**10.35**	**0.00**	**LC 2 CPGA Forces**

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL WEST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00

References

LC 3: Water to SWL, Impervious, 1.0 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FOOT OF WALL LENGTH									
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
		2.70	-1.75				4.73		wall stem weight
		4.05	0.00				0.00		base slab weight
		0.20	-3.50				0.70		P.S. soil weight
	0.64				-1.33		-0.85		P.S. lateral soil force
		0.21	1.75				-0.36		F.S. soil weight, saturated
	-0.24				-1.33		0.32		F.S. lateral soil force, saturated
	-5.27				-4.33		22.85		Lateral water force
		3.43	1.75				-6.01		Vertical water force
	0.00				0		0.00		Unbalanced load, water TOW
Subtotal B	*-4.87*	*10.59*				*0.00*	*21.37*	*0.00*	*SWL Subtotal*
		-3.65	2.25				8.21		Impervious FS hydrostatic uplift
		0.00					0.00		Impervious PS hydrostatic uplift
SUM:	**-4.87**	**6.94**				**0.00**	**29.59**	**0.00**	**LC 3 CPGA Forces**

LC 4: Water to SWL, Pervious, 1 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FOOT OF WALL LENGTH									
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
Subtotal B	*-4.87*	*10.59*				*0.00*	*21.37*	*0.00*	*SWL Subtotal*
		-3.65	1.500				5.48		Pervious hydrostatic uplift
SUM:	**-4.87**	**6.94**				**0.00**	**26.85**	**0.00**	**LC 4 CPGA Forces**

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL WEST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00

References

LC 5: Water to SWL + Debris Load, Impervious, 1.0 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FT. OF WALL									
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
Subtotal B	*-4.87*	*10.59*				*0.00*	*21.37*	*0.00*	*SWL Subtotal*
		-3.65	2.25				8.21		Impervious FS hydrostatic uplift
		0.00					0.00		Impervious PS hydrostatic uplift
	-0.50				-13.00		6.50		Debris Load
SUM:	**-5.37**	**6.94**				**0.00**	**36.09**	**0.00**	**LC 5 CPGA Forces**

LC 6: Water to T.O. Wall, Impervious, 1.33 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FT. OF WALL									
S NO.	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
		2.70	-1.75				4.73		wall stem weight
		4.05	0.00				0.00		base slab weight
		0.20	-3.50				0.70		P.S. soil weight
	0.64				-1.33		-0.85		P.S. lateral soil force
		0.21	1.75				-0.36		F.S. soil weight, saturated
	-0.24				-1.33		0.32		F.S. lateral soil force, saturated
	-7.02				-5.00		35.10		Lateral water force
		4.12	1.75				-7.21		Vertical water force
	0.00				0		0.00		Unbalanced load, water TOW
Subtotal B	*-6.62*	*11.28*				*0.00*	*32.42*	*0.00*	*TOW Subtotal*
		-4.21	2.25				9.48		Impervious FS hydrostatic uplift
		0.00					0.00		Impervious PS hydrostatic uplift
SUM:	**-6.62**	**7.06**				**0.00**	**41.90**	**0.00**	**LC 6 CPGA Forces**

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL WEST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

LC 7: Water to T.O. Wall, Pervious, 1.33 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FT. OF WALL									
S NO.	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
Subtotal B	-6.62	11.28				0.00	32.42	0.00	*TOW Subtotal*
		-4.21	1.500				6.32		Pervious hydrostatic uplift
SUM:	**-6.62**	**7.06**				**0.00**	**38.74**	**0.00**	**LC 7 CPGA Forces**

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL WEST OF KEARNY MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
					References

Summary of Forces and Moments (For Full Length of Monolith):

LOAD CASE NUMBER	Fx	Fy	Fz	Mx	My	Mz	
	kips	kips	kips	(kip-ft)	(kip-ft)	(kip-ft)	
1	20.0	0.0	451.6	0.0	193.7	0.0	Temporary Construction Case
2	-37.5	0.0	337.5	0.0	517.5	0.0	Unusual Construction Case
3	-243.7	0.0	346.9	0.0	1479.3	0.0	Still Water Level Flood, Impervious uplift
4	-243.7	0.0	346.9	0.0	1342.4	0.0	Still Water Level Flood, Pervious uplift
5	-268.7	0.0	346.9	0.0	1804.3	0.0	Still Water Level Flood, Impervious uplift
6	-331.0	0.0	353.2	0.0	2095.0	0.0	TOW Flood, Impervious uplift
7	-331.0	0.0	353.2	0.0	1937.1	0.0	TOW Flood, Pervious uplift

AECOM

Job USACE Passaic River Flood Protection
Feasibility Level Flood Protection

Project No. 60442748

Description TYPICAL WEST OF KEARNY MONOLITH
TOW EL. 18; TOS EL 6.0

Computed by RBJ

Checked by 0

Soil & Pile Information Required for CPGA

Date Jan-16

Date Jan-00

References

Pile Layout, SW-11:

Row 1			Row 2		
pile no.	x	y	pile no.	x	y
1	2.25	-20.00	6	-2.25	-20.00
2	2.25	-10.00	7	-2.25	-10.00
3	2.25	0.00	8	-2.25	0.00
4	2.25	10.00	9	-2.25	10.00
5	2.25	20.00	10	-2.25	20.00

Tip Elevation: (For CPGA, need Tip Elevation as a function of CPGA Axis at B.O. Slab, +Z points downward)

B.O.S. Elevation = 4 NAVD88 * Socketed 12' min into bedrock

Bedrock EL = -45 NAVD89

"TIP" in CPGA = 61 ft

Pile Properties & Attributes: (See hand calculations for micropiles)

E_s = 3,605,000 psi

A = 181 in^2

$I_x = I_y$ = 1534 in^4

C_{33} = 1.2 (factor for method of axial load transfer from pile to soil; = 1 for full tip bearing, = 2 for full skin friction)

Allowable Compression (AC) = 160 kips

Allowable Tension (AT) = 100 kips

PO = 1597 kips

PT = 1010 kips

PB = 550 kips

MB = 2825 kip-in

MO = 2200 kip-in

Es Value for CPGA Run:

Monolith width = 50 ft

N_h = 30 pci = 0.03

GROUP FACTORS	
Pile Spacing in Direction of Loading	From EM1110-2-2906
	D
3B	0.33
4B	0.38
5B	0.45
6B	0.56
7B	0.71
8B	1

Group reduction is based on distance between piles in direction of loading. This includes distance due to battering and is taken over the distance $10*d_{pile}$ (point of fixety).

Assume a batter of 3

$B = d_{pile}$ = 9.63 in = 0.803 ft

Distance between piles at B.O. Slab = 4.50 ft

Average distance between piles over 10*dpile = 9.85 ft

Average distance between piles in terms of pile width B = 12.274 B

Group Reduction "D" value for this distance = 1

Therefore, Es including group reduction = 0.0300 kci

AECOM

Job	USACE Passaic River Flood Protection		Project No.	60442748	
	Feasibility Level Flood Protection				
Description	TYPICAL WEST OF KEARNY MONOLITH		Computed by	RBJ	Date Jan-16
	TOW EL. 18; TOS EL 6.0				
	CPGA Input & Output Files		Checked by	0	Date Jan-00

TOM EL18, Micropile 5' O.C.

Input file:

```
100 WEST OF KEARNY, TOW EL18, FIX, 1:3 BATTER, MICROPILE, 5'OC
200 PROP 3605 1534 1534 181 1.2 0 ALL
300 SOIL ES 0.03 TIP 60 0 ALL
400 FIX ALL
500 TENSION 0.8 ALL
600 DLS R 160 100 1597 1010 550 2825 2200 H 15.18 ALL
700 FOVSTR 1.17 1.17 1
800 FOVSTR 1.33 1.33 2 6 7
1000 BATTER 3 ALL
1100 ANGLE 180 11 TO 20
1200 PILE 1 2.5 -22.5 0
1300 PILE 11 -2.5 -22.5 0
1400 ROW Y 10 1 9 AT 5
1500 ROW Y 10 11 9 AT 5
4500 LOAD 1 20 0 436.6 0 199 0
4600 LOAD 2 -37.5 0 337.5 0 517.5 0
4700 LOAD 3 -243.7 0 350.7 0 1478 0
4800 LOAD 4 -243.7 0 350.7 0 1341 0
4900 LOAD 5 -268.7 0 350.7 0 1803 0
5000 LOAD 6 -331 0 356.9 0 2094 0
5100 LOAD 7 -331 0 356.9 0 1936 0
6000 FOUT 1 2 3 4 5 6 7 EL18MP05.doc
6100 PFO ALL
6200 PLB ALL
```

RIVER FRONTING WALLS ALL-VERTICAL PILES

USACE Passaic River Flood Protection

Feasibility Level Flood Protection

TYPICAL FRONTING MONOLITH
TOW EL. 18; TOS EL 6.0

URS Project : 60442748

Monolith Foundations

1515 Poydras Street

Suite 2700

New Orleans, LA 70112

(504) 586-8111

Computed by:	RBJ	Checked by:	
Date:	Jan-16	Date:	

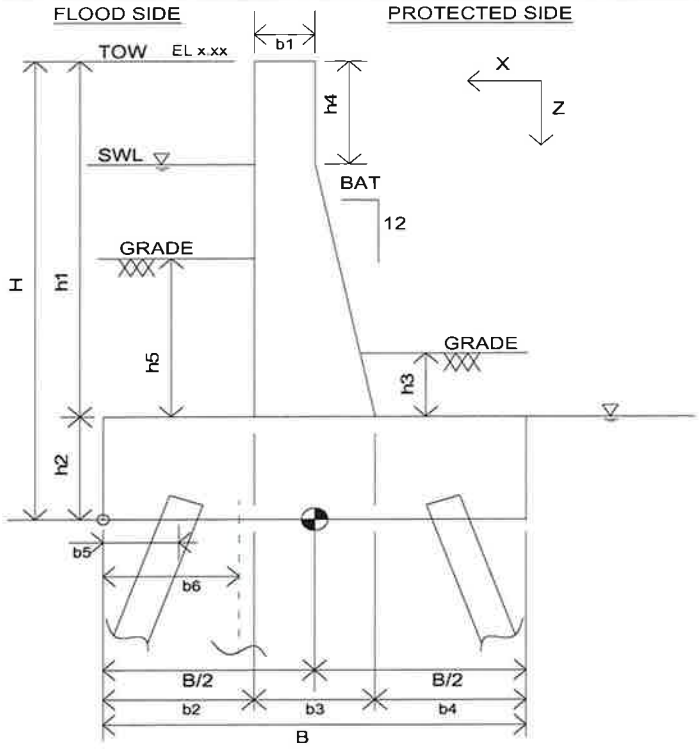
AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL FRONTING MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

LOAD CALCULATIONS

TOW EL. 18; TOS EL 6.0

Top of Wall El.	18.0	NAVD88
Normal Water El.	6.0	NAVD88
Still Water El.	16.0	NAVD88
Top of Slab EL.	6.0	NAVD88
H=	15.0	ft.
h1=	12.0	ft.
h2=	36.0	in. (Base Slab Height)
h3=	12.0	in. (P.S. Soil Height)
h4=	0.0	ft.
h5=	1.00	ft. (F.S. Soil Height, **T.O. Slab**)
B=	9.0	ft. (Base Slab Width)
b1=	18.0	in. (Wall Stem Width, top)
b2=	66.0	in. (F.S. Slab Width)
b3=	18.0	in. (Wall Stem Width, bottom)
b4=	24.0	in. (P.S. Slab Width)
b5=	21.0	in. (F.S. Pile Row Edge Space)
b6=	54.0	in. (Sheet Pile Edge Space)
BAT=	0.0	(Wall Batter, N/A)
PS Grade =	7.00	NAVD88 (Average of PS soil for all)
Monolith Length	50.0	ft
Bottom Of Slab =	3.0	NAVD88



T-WALL CROSS-SECTION

Notes: 1) positive 'Y' axis is into page
2) pile batters vary from those show in diagram

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL FRONTING MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

ASSUMED UNIT WEIGHTS:

Unit Weight of Storm Water:
62.4 pcf

Wet Unit Weight of Soil: 100 pcf

Unit Weight of Riprap: 0 pcf

Sat Unit Weight of Soil:
37.6 pcf

Unit Weight of Concrete
150 pcf

Unbalanced Load for Stability Analysis:

F_{cap} (k/ft) = 0.00 (Operation Case; Force acts at bottom of slab)

F_{cap} (k/ft) = 0.00 (Water to TOW Case; Force acts at bottom of slab)

K_0, Clay fill = 0.80 (for lateral soil forces)

EM 1110-2-2502: $P_{hz} = K_0 * (\gamma * z_w + \gamma' * (z - z_w))$

- PS design water elevation <BOS elevation, no PS uplift or PS hydrostatic loads

FS Wind force above SWL= 50.0 psf

Construction Surcharge Pressure = 250 psf

Wave Point Load = 0 kip/ft

Dist from B.O. Slab to Wave Load = 0 ft

	LOAD CASES ANALYZED:	FS Water El.	PS Water El.	Overstress for Design	Notes
1	Construction	3.0	3.0	1.17	Temporary Construction Case
2	Construction + Wind on F.S.	3.0	3.0	1.33	Unusual Construction Case
3	Water to SWL, Impervious	16.0	6.0	1.00	Still Water Level Flood, Impervious uplift
4	Water to SWL, Pervious	16.0	6.0	1.00	Still Water Level Flood, Pervious uplift
5	Water to SWL + Debris Load, Impervious	16.0	6.0	1.00	Still Water Level Flood, Impervious uplift
6	Water to T.O. Wall, Impervious	18.0	6.0	1.33	TOW Flood, Impervious uplift
7	Water to T.O. Wall, Pervious	18.0	6.0	1.33	TOW Flood, Pervious uplift

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL FRONTING MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
					References

All forces on this page are per foot of monolith length

Soil Forces:

Water EL.	FS Soil EL.	PS Soil EL.	Wt. of FS Soil (k/ft)	Wt. of PS Soil (k/ft)	FS Soil Lateral Force (k/ft)	PS Soil Lateral Force (k/ft)
7.0	7.00	7.00	**0.2068**	**0.200**	**-0.2406**	**0.6400**
16.0	7.00	7.00	**0.2068**	**0.200**	**-0.2406**	**0.6400**
18.0	7.00	7.00	**0.2068**	**0.200**	**-0.2406**	**0.6400**

Water Forces:

Water EL.	Wt. of FS Water (k/ft)	FS Water Lateral Force (k/ft)
7.0	**0.3432**	**-0.499**
16.0	**3.432**	**-5.273**
18.0	**4.1184**	**-7.020**

Wind Force: 50 psf * monolith height / 1000 = **-0.75 k/ft**

MDE Earthquake Force:

	Vertical (k-ft)	Lateral (k-ft)		
Soil		**0.116**	Z-coord = -2.67	
Concrete	**0.270**	**0.844**	Z-coord = -4.50	X-coord = -0.700

Surcharge Forces:

F.S. = 250 psf * F.S. width / 1000 = **1.375 k/ft**

P.S. = 250 psf * P.S. width / 1000 = **0.5 k/ft**

Unbalanced Load:

Operating: 0.000 k/ft in (+) X Direction, acting at bottom of slab (Z-coordinate = 0)

TOW: 0.000 k/ft in (+) X Direction, acting at bottom of slab (Z-coordinate = 0)

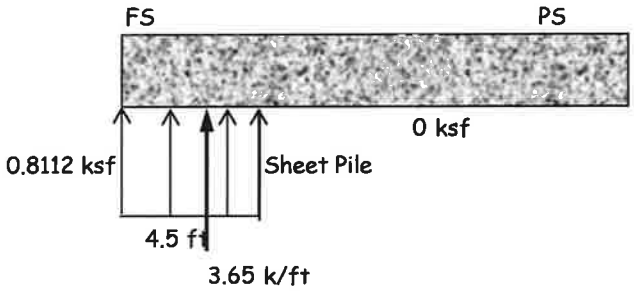
AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL FRONTING MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

Uplift Loads:

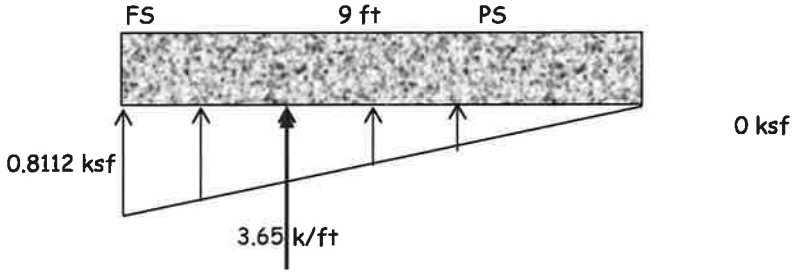
Water to SWL:

Impervious:



X-coordinate = 2.25

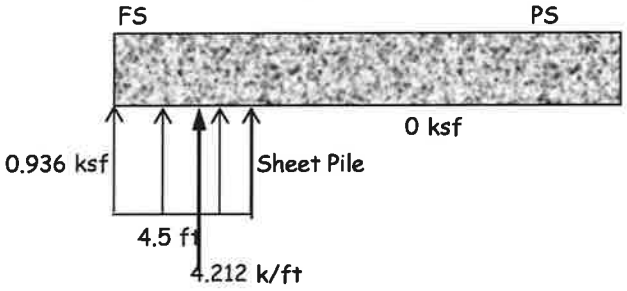
Pervious:



X-coordinate = 1.500

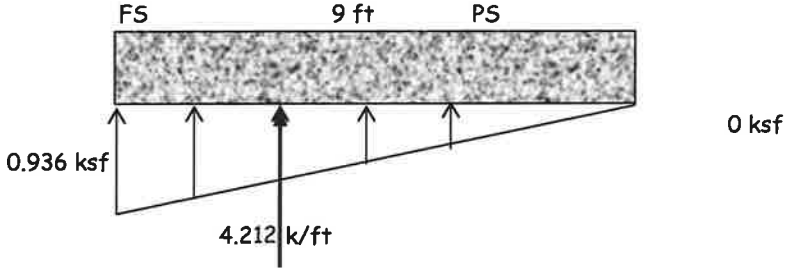
Water to TOW:

Impervious:



X-coordinate = 2.25

Pervious:



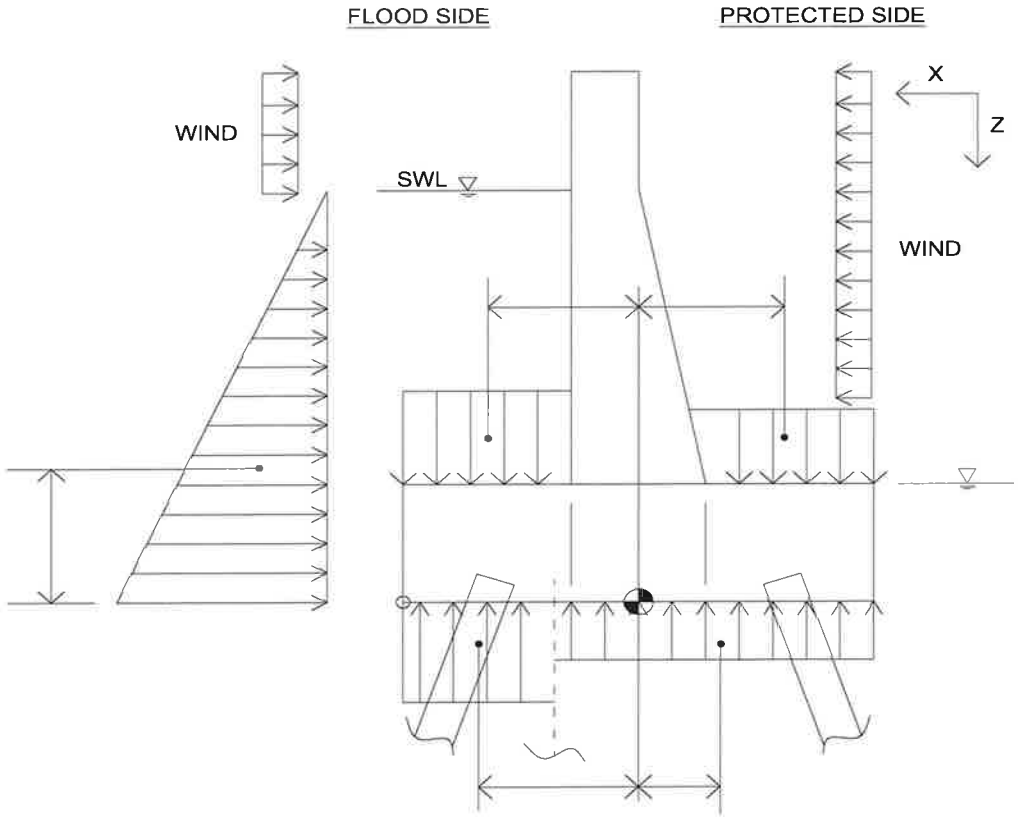
X-coordinate = 1.500

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL FRONTING MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00

References

LOAD DIAGRAMS FOR IMPERVIOUS SHEET PILE CONDITIONS



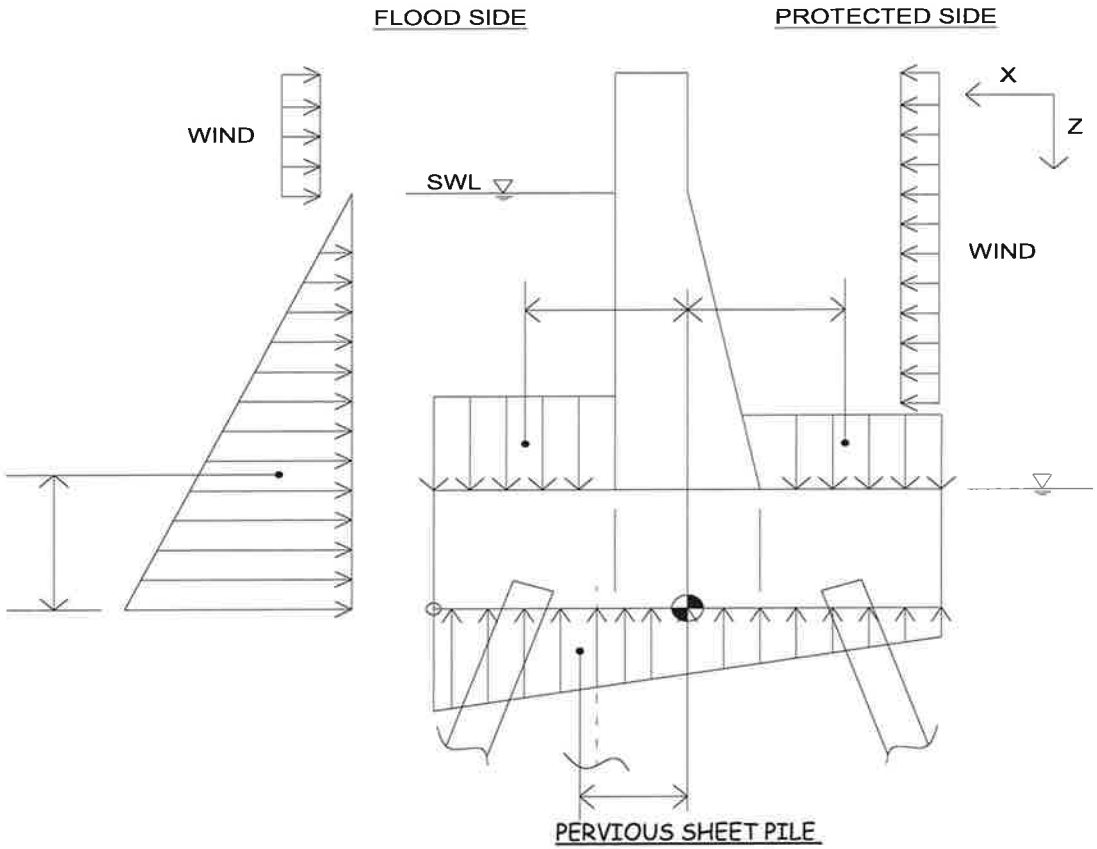
IMPERVIOUS SHEET PILE

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL FRONTING MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00

References

LOAD DIAGRAMS FOR PERVIOUS SHEET PILE CONDITIONS



AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL FRONTING MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

LC 1: Construction, 1.17 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FOOT OF WALL LENGTH									
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
		2.70	-1.75				4.73		wall stem weight
		4.05	0.00				0.00		base slab weight
		0.20	-3.50				0.70		P.S. soil weight
	0.64				-1.33		-0.85		P.S. lateral soil force
		0.21	1.75				-0.36		F.S. soil weight
	-0.24				-1.33		0.32		F.S. lateral soil force
	0.00				0.00		0.00		Unbalanced load, no water
Subtotal A	*0.40*	*7.16*				*0.00*	*4.53*	*0.00*	*Dry Soil Weight Subtotal*
		1.38	1.75				-2.41		F.S. Equipment Surcharge
		0.50	-3.50				1.75		P.S. Equipment Surcharge
SUM:	**0.40**	**9.03**				**0.00**	**3.87**	**0.00**	**LC 1 CPGA Forces**

LC 2: Construction + Wind on F.S., 1.33 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FOOT OF WALL LENGTH									
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
		2.70	-1.75	0			4.73	0	wall stem weight
		4.050	0				0.00		base slab weight
	-0.75				-7.50	0	5.63	0	FS Wind Load
SUM:	**-0.75**	**6.75**				**0.00**	**10.35**	**0.00**	**LC 2 CPGA Forces**

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL FRONTING MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

LC 3: Water to SWL, Impervious, 1.0 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FOOT OF WALL LENGTH									
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
		2.70	-1.75				4.73		wall stem weight
		4.05	0.00				0.00		base slab weight
		0.20	-3.50				0.70		P.S. soil weight
	0.64				-1.33		-0.85		P.S. lateral soil force
		0.21	1.75				-0.36		F.S. soil weight, saturated
	-0.24				-1.33		0.32		F.S. lateral soil force, saturated
	-5.27				-4.33		22.85		Lateral water force
		3.43	1.75				-6.01		Vertical water force
	0.00				0		0.00		Unbalanced load, water TOW
Subtotal B	*-4.87*	*10.59*				*0.00*	*21.37*	*0.00*	*SWL Subtotal*
		-3.65	2.25				8.21		Impervious FS hydrostatic uplift
		0.00					0.00		Impervious PS hydrostatic uplift
SUM:	**-4.87**	**6.94**				**0.00**	**29.59**	**0.00**	**LC 3 CPGA Forces**

LC 4: Water to SWL, Pervious, 1 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FOOT OF WALL LENGTH									
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
Subtotal B	*-4.87*	*10.59*				*0.00*	*21.37*	*0.00*	*SWL Subtotal*
		-3.65	1.500				5.48		Pervious hydrostatic uplift
SUM:	**-4.87**	**6.94**				**0.00**	**26.85**	**0.00**	**LC 4 CPGA Forces**

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL FRONTING MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

LC 5: Water to SWL + Debris Load, Impervious, 1.0 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FT. OF WALL									
	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
Subtotal B	*-4.87*	*10.59*				*0.00*	*21.37*	*0.00*	*SWL Subtotal*
		-3.65	2.25				8.21		Impervious FS hydrostatic uplift
		0.00					0.00		Impervious PS hydrostatic uplift
	-0.50				-13.00		6.50		Debris Load
SUM:	**-5.37**	**6.94**				**0.00**	**36.09**	**0.00**	**LC 5 CPGA Forces**

LC 6: Water to T.O. Wall, Impervious, 1.33 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FT. OF WALL									
S NO.	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
		2.70	-1.75				4.73		wall stem weight
		4.05	0.00				0.00		base slab weight
		0.20	-3.50				0.70		P.S. soil weight
	0.64				-1.33		-0.85		P.S. lateral soil force
		0.21	1.75				-0.36		F.S. soil weight, saturated
	-0.24				-1.33		0.32		F.S. lateral soil force, saturated
	-7.02				-5.00		35.10		Lateral water force
		4.12	1.75				-7.21		Vertical water force
	0.00				0		0.00		Unbalanced load, water TOW
Subtotal B	*-6.62*	*11.28*				*0.00*	*32.42*	*0.00*	*TOW Subtotal*
		-4.21	2.25				9.48		Impervious FS hydrostatic uplift
		0.00					0.00		Impervious PS hydrostatic uplift
SUM:	**-6.62**	**7.06**				**0.00**	**41.90**	**0.00**	**LC 6 CPGA Forces**

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL FRONTING MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
				References	

LC 7: Water to T.O. Wall, Pervious, 1.33 allowable overstress									
EXTERNAL FORCES & MOMENTS ABOUT ORIGIN PER FT. OF WALL									
S NO.	Fx	Fz	'X' Centroid	'Y' Centroid	'Z' Centroid	Mx	My	Mz	NOTES:
	(kip/ft)	(kip/ft)	(ft)	(ft)	(ft)	(kip-ft/ft)	(kip-ft/ft)	(kip-ft/ft)	
Subtotal B	-6.62	11.28				0.00	32.42	0.00	*TOW Subtotal*
		-4.21	1.500				6.32		Pervious hydrostatic uplift
SUM:	**-6.62**	**7.06**				**0.00**	**38.74**	**0.00**	**LC 7 CPGA Forces**

AECOM

Job	USACE Passaic River Flood Protection	Project No.	60442748		
	Feasibility Level Flood Protection				
Description	TYPICAL FRONTING MONOLITH	Computed by	RBJ	Date	Jan-16
	TOW EL. 18; TOS EL 6.0				
	Foundation Load Cases & Combinations	Checked by	0	Date	Jan-00
					References

Summary of Forces and Moments (For Full Length of Monolith):

LOAD CASE NUMBER	Fx	Fy	Fz	Mx	My	Mz	
	kips	kips	kips	(kip-ft)	(kip-ft)	(kip-ft)	
1	20.0	0.0	451.6	0.0	193.7	0.0	Temporary Construction Case
2	-37.5	0.0	337.5	0.0	517.5	0.0	Unusual Construction Case
3	-243.7	0.0	346.9	0.0	1479.3	0.0	Still Water Level Flood, Impervious uplift
4	-243.7	0.0	346.9	0.0	1342.4	0.0	Still Water Level Flood, Pervious uplift
5	-268.7	0.0	346.9	0.0	1804.3	0.0	Still Water Level Flood, Impervious uplift
6	-331.0	0.0	353.2	0.0	2095.0	0.0	TOW Flood, Impervious uplift
7	-331.0	0.0	353.2	0.0	1937.1	0.0	TOW Flood, Pervious uplift

AECOM

Job	USACE Passaic River Flood Protection		Project No.	60442748	
	Feasibility Level Flood Protection				
Description	TYPICAL WEST OF KEARNY MONOLITH		Computed by	RBJ	Date Jan-16
	TOW EL. 18; TOS EL 6.0				
	CPGA Input & Output Files		Checked by	0	Date Jan-00

TOM EL18, HP14x89 5' O.C. CERTICAL PILES

Input file:

```
100 FROPNTING WALL, 50' MONOLITH TOW EL18-
110 H-PILE, 5'OC, VERTICAL FS & PS
200 PROP 29000 326 905 26.1 1 0 ALL
300 SOIL ES 0.11 TIP 60 0 ALL
400 FIX ALL
500 TENSION 0.8 ALL
600 ALLOW H 200 60 653 607 1108 3275 ALL
700 FOVSTR 1.17 1.17 1
800 FOVSTR 1.33 1.33 2 6 7
1200 PILE 1 3 -22.5 0
1300 PILE 11 -3 -22.5 0
1400 ROW Y 10 1 9 AT 5
1500 ROW Y 10 11 9 AT 5
4500 LOAD 1 20 0 436.6 0 199 0
4600 LOAD 2 -37.5 0 337.5 0 517.5 0
4700 LOAD 3 -243.7 0 350.7 0 1478
4800 LOAD 4 -243.7 0 350.7 0 1341 0
4900 LOAD 5 -268.7 0 350.7 0 1803 0
5000 LOAD 6 -331 0 356.9 0 2094 0
5100 LOAD 7 -331 0 356.9 0 1936 0
6000 FOUT 1 2 3 4 5 6 7 FWHP.doc
6100 PFO ALL
6200 PLB ALL
6200 PLB ALL
```

SUBAPPENDIX 2
2.3: Closure Gates

Memo

Date:	Tuesday, April 12, 2016
Project:	Passaic Tidal Closure Structure
To:	John Dromsky-Reed
From:	Wes Jacobs Bogdan Bogdanovic
Subject:	**Preliminary Design and Cost Study of the Passaic River Closure Gate Structures in Support of the Passaic River Ecosystem Restoration Feasibility Studies Conducted under Indefinite Delivery Contract IDC-PL-3005: W912DS-11-D-0008**

This memorandum presents the results of our preliminary (planning level) designs and cost study of the Passaic River closure gate structures in the tidal areas of Kearney, Newark, and Harrison, NJ.

1. INTRODUCTION

This study addresses the design of closure gates in typical reaches along the Passaic River extending from Kearny to Harrison, NJ. The design and cost element defined herein represents a 30-percent level conceptual design using the latest revision inventory document of the closure gates titled "**Passaic River-Final Closure Gate Inventory**" dated March 10, 2016 (see Appendix A). The closure gates were grouped into several different categories based on gate openings, heights and types. The gate types used were predominantly swing gates with the exception of roller gates for openings of 50 feet or larger. The gates are assumed a mix of closures to span railroads, highways and pedestrian crossings.

Through coordination with technical staff from AECOM, as well as standard engineering practice, the 30% design includes four basic load cases which are loadings that typically control floodwall/closure gate structures designs. A full array of load cases will need to be investigated in the final design phase. The load cases included in the 30% design are:

- 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.
- Flood stage at still water (SWL) at 2 feet below top of gate structure with debris impact loading of 500lbs/ft applied at the SWL. A 33% overstress is permitted for this load case.
- Flood stage at water to top of gate (TOG).Wave force is not included. A 33% overstress is permitted for this load case.
- Flood stage at SWL at 2 feet below top of gate structure. A 0% overstress is permitted for this load case.

The gate members (girders, intercostals, and skin plates), concrete monolith (abutments/footings), and foundations were sized to carry these anticipated loads as mentioned above for all different gate categories which have been selected. Secondary gate features such as any hinge assemblies, connections, casters, trolleys, or hanger systems were conceptually shown based on previous similar projects and engineering judgment. Calculations were not performed to size these types of features. Wave loadings are expected to be minimal due to topographic conditions and lack of proximity/exposure

to full coastal storm surge associated with hurricanes. It is also assumed, per technical discussions, that there will be no unbalanced loading or downdrag forces seen by the gates at this level of design. This will require more in-depth analysis and can be fully vetted during later design stages. Complex pile group analysis, therefore, was not be required. Seismic forces were not considered to govern and were not applied at this level of design.

For the 30 % design effort the following codes and standards will be used, as well as the applicable portions of the HSDRRSDG (Hurricane Storm Damage Risk Reduction System Design Guidelines) and the existing project GDM:

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

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

2. DISCUSSION AND RESULTS

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

All gates were grouped into several scenarios based on gate openings and heights as shown in table 1. The Kearny, Newark and Flanking areas consist of H-pile foundation whereas the Harrison area consists of concrete micro pile foundation. Any opening width equal to 10-feet or smaller was grouped with the 10-foot gate opening. The 20-foot gate opening was grouped with a series of opening widths ranging from 15 to 20 feet. The majority of opening widths in the inventory was for the 30-foot width. The 30-foot gate opening was grouped from 25 to 30 feet. The 35-foot, 40-foot, 45-foot and 50-foot gate openings were grouped individually, since their gate opening width is considered to be on the larger end of the swinging gate spectrum.

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

larger concrete monolith structure. The same grouping procedure described above was followed with respect to gate heights.

Table 1. Gate Grouping Scenarios

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

The structural design of the swing/roller gate includes the layout and design of the major structural elements of the concrete monolith structure and floodgate. This includes the gate steel members, the concrete gate bay walls and support columns, base slab and the pile foundations. The structural steel gate members include top and bottom girders spanning horizontally between concrete bay columns, vertical intercostal framing spaced at approximately 2 feet on center and spanning between top and bottom girders, steel skin plate spanning between the vertical intercostal, and steel cross bracing and horizontal bracing. The concrete monoliths are comprised of two concrete gate bay walls/columns on either side which are formed into the base slab and pile foundation. The concrete monoliths are supported by the pile foundations. Steel H-piles and concrete micro piles were applied during design for consistency with the floodwall team. It is assumed that each gate monolith structure will be flanked by the floodwall structures in the adjacent reaches. The floodgate drawings in Appendix B are preliminary in nature and not to be used for construction. The sections and views on the drawings are grouped as described above in table 1. Based on the gate width and heights, the design elements will vary in size, location and spacing accordingly.

The analysis of the steel gate and concrete monolith was performed based on the load cases noted in the introduction. The governing load case was typically the flood stage with water at the top of the gate. Loads were applied as hydrostatic pressures corresponding to the water surface elevations on the flood-side. A debris impact uniform loading (500lbs/ft.) was applied at the appropriate water surface elevations. The skin plate was designed as a fixed end beam spanning between the vertical intercostals and the deflection was limited to 0.4 of the thickness to ensure that the flat plate theory is applicable. The horizontal girders were designed as larger wide flange simply supported beams spanning between the bearing points on the concrete columns making them true beam elements allowing for flexural stresses.

The vertical intercostals were designed as simple beams spanning between horizontal girders. The vertical intercostals consist of a WT section welded to the skin plate and were designed as a combined section utilizing the steel skin plate as the tension flange of the total combined section. The analysis of the reinforced concrete monolith walls and columns was performed considering fixed support at the interface of the bottom of the wall and top of slab. The wall analysis considered a 1 foot unit width of the wall acting as a cantilever and connected only to the base slab. The column analysis considered half of the gate width and width of the column loading on the column acting as a cantilever and connected only to the base slab. The entire analysis for the floodgate and concrete monolith was carried out by hand calculations for one gate width and height which than an excel spreadsheet program was developed to generated the analysis design for all chosen gate scenarios listed in table 1. The calculations are provided in Appendix C.

Opinions of probable cost (using unit prices from similar, recent projects) were developed based on the results of the analysis above. The cost estimate was broken down into four items corresponding to each individual gate width and height. The four cost items are the structural steel gate, concrete monolith structure, concrete reinforcing and pile foundation with a final total project cost. The cost breakdown for all listed scenarios is provided in table format in Appendix D. In addition, compiled cost curve graphs for each gate opening width based on total project cost versus gate height to gate opening width were developed and are shown below for each gate type.

Figure 1. Cost curve graph for swing gates

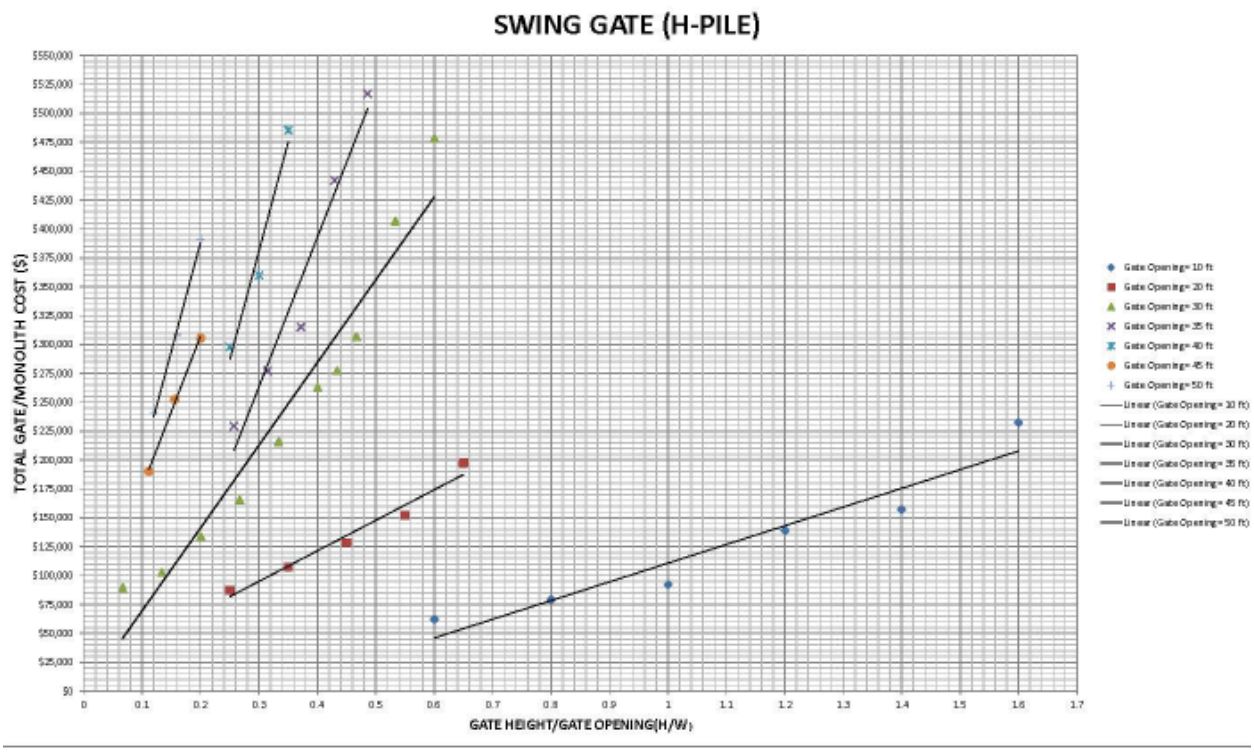


Figure 2. Cost curve graph for swing gates (micro pile, Harrison Area)

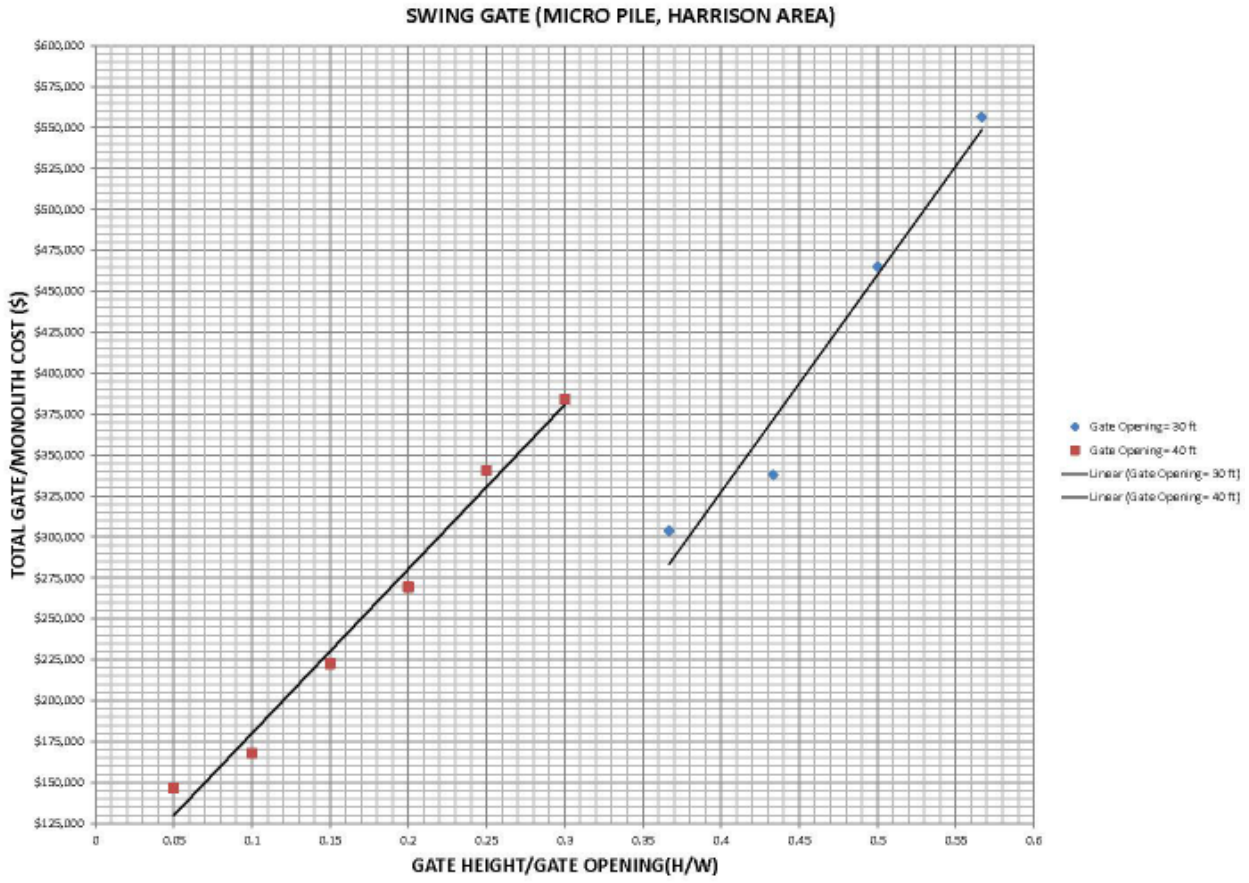
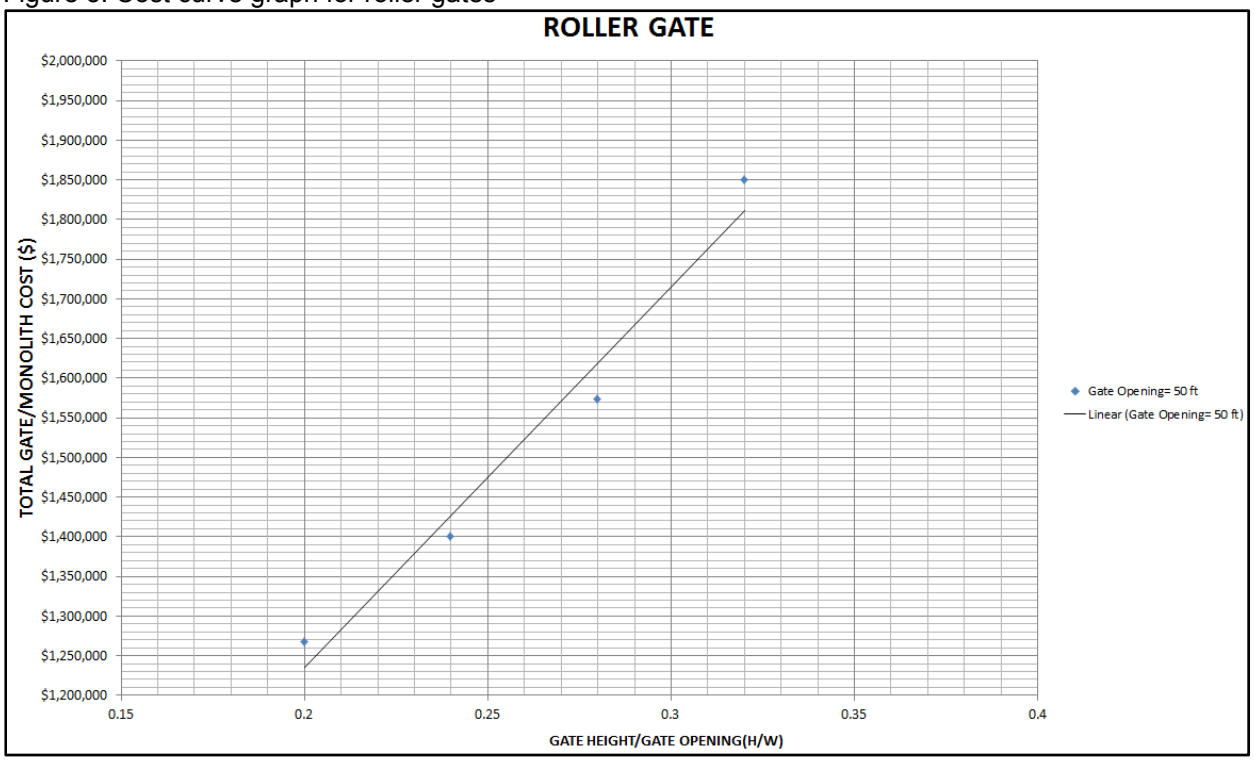


Figure 3. Cost curve graph for roller gates



The intent to these curves is to be able to achieve an approximate construction cost estimate based on the gate width opening for varying gate height to width ratio. Opinions of probable construction cost for each of the 64 gates in the inventory were developed from these cost curves and are reported in Appendix A. The closure gate costs, by reach, are reported in table 2.

Table 2. Summary of Closure Gate Costs By Project Reach

	Project Reach				
Design Water Surface Elevation[1]	Kearney	Newark	Harrison 1	Newark Flanking	TOTAL
14 ft (GDM)	$8,247,020	$4,023,917	$2,403,056	$1,558,707	$16,232,701
16 ft (GDM + 2 ft)	$9,896,957	$5,402,250	$3,242,335	$2,108,385	$20,649,928
18 ft (GDM + 4 ft)	$11,556,451	$6,780,583	$3,956,389	$2,658,063	$24,951,486

[1] All elevations reference the NAVD 88 vertical datum.

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Appendix A:
Passaic River-Final Closure Gate Inventory and Cost

PASSAIC RIVER - FINAL CLOSURE GATE INVENTORY

Passaic River Ecosystem Restoration Feasibility Studies Conducted under Indefinite Delivery Contract IDC-PL-3005: W912DS-11-D-0008

USACE New York District

FINAL Closure Structure Inventory, 10 March 2016

Total Number of Closure Structures: 64

Source Documents: GDM September 1995 - Vol II of II; AECOM e-mail transmittal, 11/11/15

Originator: Bogdan Bogdanovich

QC: Michael Vecchio, 12/8/15; Revision 1 - 1/19/16, Revision 2 - 3/10/16 for Closure Gate Inventory, Post-QC Revisions: 4/14/2016

Incorporation of Cost Equations: K Hayden - 3/29/2016; K Hayden Addition of micro-pile 40-ft wide gate cost equations -0 4/21/16

QC of Cost Equations: Jaak Van den Sype, 3/31/16; Kim Hayden - 4/14/2016; Michae Vecchio - 4/21/16

QC of Transmitted Table: Michael Murphy, 4/7/2016

Gate No.	Revised Reach	GIS file description from AECOM	Gate Type	Original GDM Height (ft)[4]	Opening Width (O) (ft)[4]	Existing Grade (ft)[1]	GDM DWSE (ft)[2]	GDM + 2 ft[3]	GDM + 4 ft[3]	Computed GDM Height (H) (ft)	H/O for GDM	GDM Construction Cost[5]	Computed GDM +2 Height (ft)	H/O for GDM +2 ft	GDM + 2 ft Construction Cost[5]	Computed GDM +4 Height (ft)	H/O for GDM+4 ft	GDM + 4 ft Construction Cost[5]
5	Kearny	RAILROAD CLOSURE	swing	8.0	35	8.0	14.0	16.0	18.0	6.0	0.2	$ 97,258	8.0	0.2	$ 171,216	10.0	0.286	$ 245,173
6	Kearny	±15 L.F. CLOSURE ±7.0' HT. T.O.L. 14.9'	swing	7.0	15	7.2	14.0	16.0	18.0	6.8	0.5	$ 59,431	8.8	0.6	$ 75,456	10.8	0.723	$ 91,481
7	Kearny	±15 L.F. CLOSURE ±3.5' HT. T.O.L. 14.9'	swing	3.5	15	7.5	14.0	16.0	18.0	6.5	0.4	$ 56,337	8.5	0.6	$ 72,362	10.5	0.698	$ 88,387
8	Kearny	±15 L.F. CLOSURE ±3.5' HT. T.O.L. 14.9'	swing	8.5	15	8.6	14.0	16.0	18.0	5.4	0.4	$ 47,608	7.4	0.5	$ 63,633	9.4	0.625	$ 79,658
9	Kearny	±35 L.F. CLOSURE ±3.5' HT. T.O.L. 14.9'	swing	8.5	35	5.1	14.0	16.0	18.0	8.9	0.3	$ 204,849	10.9	0.3	$ 278,806	12.9	0.369	$ 352,764
10	Kearny	±25 L.F. CLOSURE ±7.0' HT. T.O.L. 14.9'	swing	7.0	25	6.8	14.0	16.0	18.0	7.2	0.3	$ 61,879	9.2	0.4	$ 74,816	11.2	0.447	$ 87,752
11	Kearny	±50 L.F. CLOSURES ±8.5' HT.	roller	8.5	50	1.6	14.0	16.0	18.0	12.4	0.2	$ 1,465,076	14.4	0.3	$ 1,656,990	16.4	0.328	$ 1,848,905
12	Kearny	±50 L.F. CLOSURES ±8.5' HT.	roller	8.5	50	3.8	14.0	16.0	18.0	10.2	0.2	$ 1,255,561	12.2	0.2	$ 1,447,476	14.2	0.284	$ 1,639,390
13	Kearny	±50 L.F. CLOSURES ±8.5' HT.	roller	8.5	50	2.6	14.0	16.0	18.0	11.4	0.2	$ 1,365,937	13.4	0.3	$ 1,557,852	15.4	0.307	$ 1,749,766
14	Kearny	30' CLOSURE ±6.9' HT.	swing	6.9	30	7.4	14.0	16.0	18.0	6.6	0.2	$ 124,496	8.6	0.3	$ 181,378	10.6	0.354	$ 238,259
15	Kearny	30' CLOSURE ±7.2' HT.	swing	7.2	30	6.0	14.0	16.0	18.0	8.0	0.3	$ 163,383	10.0	0.3	$ 220,265	12.0	0.400	$ 277,147
16	Kearny	±30 L.F. CLOSURE ±8.4' HT. T.O.L. 14.9'	swing	8.4	30	3.9	14.0	16.0	18.0	10.1	0.3	$ 222,050	12.1	0.4	$ 278,932	14.1	0.469	$ 335,814
17	Kearny	±30 L.F. CLOSURE ±8.4' HT. T.O.L. 14.9'	swing	8.4	30	4.0	14.0	16.0	18.0	10.0	0.3	$ 220,416	12.0	0.4	$ 277,298	14.0	0.467	$ 334,180
18	Kearny	±30 L.F. CLOSURE ±8.9' LOCATION MAY	swing	8.9	30	6.0	14.0	16.0	18.0	8.0	0.3	$ 163,630	10.0	0.3	$ 220,512	12.0	0.400	$ 277,394
19	Kearny	±35 L.F. CLOSURE ±8.9' HT. T.O.L. 14.9'	swing	8.9	35	0.7	14.0	16.0	18.0	13.3	0.4	$ 366,789	15.3	0.4	$ 440,747	17.3	0.494	$ 514,704
20	Kearny	±50 L.F. CLOSURE ACROSS EXISTING DISC	roller	6.0	50	7.3	14.0	16.0	18.0	6.7	0.1	$ 914,901	8.7	0.2	$ 1,106,815	10.7	0.213	$ 1,298,729
21	Kearny	±20 L.F. CLOSURE 5.0' HT. T.O.L. 14.9'	swing	6.0	20	5.8	14.0	16.0	18.0	8.2	0.4	$ 124,304	10.2	0.5	$ 150,731	12.2	0.611	$ 177,158
21A	Kearny	±20 L.F. CLOSURE 5.0' HT. T.O.L. 14.9'	swing	6.0	20	5.2	14.0	16.0	18.0	8.8	0.4	$ 131,638	10.8	0.5	$ 158,066	12.8	0.639	$ 184,493
22	Kearny		roller	?	50	6.3	14.0	16.0	18.0	7.7	0.2	$ 1,012,969	9.7	0.2	$ 1,204,883	11.7	0.234	$ 1,396,798
23	Kearny		swing	5.0	30	12.9	14.0	16.0	18.0	1.1	0.0	$ 83,158	3.1	0.1	$ 96,494	5.1	0.169	$ 119,386
24	Kearny		swing	5.0	30	8.0	14.0	16.0	18.0	6.0	0.2	$ 105,349	8.0	0.3	$ 162,230	10.0	0.332	$ 219,112
	KEARNY SUBTOTALS											**$ 8,247,020**			**$ 9,896,957**			**$ 11,556,451**
26	Newark	±30 L.F. CLOSURE ±7.5' HT. T.O.L. 14.9'	swing	7.5	30	4.3	14.0	16.0	18.0	9.7	0.3	$ 210,647	11.7	0.4	$ 267,529	13.7	0.455	$ 324,410
27	Newark	±30 L.F. CLOSURE ±6.0' HT. T.O.L. 14.9'	swing	6.0	30	6.6	14.0	16.0	18.0	7.4	0.2	$ 146,688	9.4	0.3	$ 203,570	11.4	0.380	$ 260,452
28	Newark	±30 L.F. CLOSURE ±6.0' HT. T.O.L. 14.9'	swing	6.0	30	6.9	14.0	16.0	18.0	7.1	0.2	$ 137,273	9.1	0.3	$ 194,155	11.1	0.369	$ 251,037
29	Newark	±30 L.F. CLOSURE ±6.0' HT. T.O.L. 14.9'	swing	6.0	30	5.8	14.0	16.0	18.0	8.2	0.3	$ 168,956	10.2	0.3	$ 225,838	12.2	0.406	$ 282,720
30	Newark	±30 L.F. CLOSURE ±6.0' HT. T.O.L. 14.9'	swing	6.0	30	6.9	14.0	16.0	18.0	7.1	0.2	$ 137,158	9.1	0.3	$ 194,040	11.1	0.369	$ 250,922
31	Newark	±30 L.F. CLOSURE ±9.0' HT. T.O.L. 14.9'	swing	9.0	30	4.9	14.0	16.0	18.0	9.1	0.3	$ 194,953	11.1	0.4	$ 251,835	13.1	0.437	$ 308,716
32	Newark	±30 L.F. CLOSURE ±7.5' HT. T.O.L. 14.9'	swing	7.5	30	6.4	14.0	16.0	18.0	7.6	0.3	$ 152,538	9.6	0.3	$ 209,419	11.6	0.387	$ 266,301
33	Newark	±30 L.F. CLOSURE ±8.0' HT. T.O.L. 14.9'	swing	6.0	30	3.6	14.0	16.0	18.0	10.4	0.3	$ 231,966	12.4	0.4	$ 288,847	14.4	0.480	$ 345,729
34	Newark	±30 L.F. CLOSURE ±5.5' HT. T.O.L. 14.9'	swing	5.5	30	5.8	14.0	16.0	18.0	8.2	0.3	$ 170,512	10.2	0.3	$ 227,394	12.2	0.408	$ 284,276
35	Newark	±30 L.F. CLOSURE ±5.5' HT. T.O.L. 14.9'	swing	5.5	30	8.8	14.0	16.0	18.0	5.2	0.2	$ 83,994	7.2	0.2	$ 140,876	9.2	0.307	$ 197,757
36	Newark	±10 L.F. CLOSURE ±10.0' HT. T.O.L. 14.9'	swing	10.0	10	3.6	14.0	16.0	18.0	10.4	1.0	$ 116,652	12.4	1.2	$ 148,977	14.4	1.436	$ 181,301
38	Newark	±30 L.F. CLOSURE ±9.0' HT. T.O.L. 14.9'	swing	9.0	30	5.8	14.0	16.0	18.0	8.2	0.3	$ 168,277	10.2	0.3	$ 225,159	12.2	0.405	$ 282,040
39	Newark	±30 L.F. CLOSURE ±9.0' HT. T.O.L. 14.9'	swing	9.0	30	4.7	14.0	16.0	18.0	9.3	0.3	$ 201,141	11.3	0.4	$ 258,023	13.3	0.444	$ 314,905
40	Newark	±30 L.F. CLOSURE ±9.0' HT. T.O.L. 14.9'	swing	9.0	30	6.2	14.0	16.0	18.0	7.8	0.3	$ 159,251	9.8	0.3	$ 216,133	11.8	0.395	$ 273,015
41	Newark	±30 L.F. CLOSURE ±9.0' HT. T.O.L. 14.9'	swing	9.0	30	5.4	14.0	16.0	18.0	8.6	0.3	$ 181,027	10.6	0.4	$ 237,909	12.6	0.420	$ 294,791
42	Newark	PROVIDE 1-10' CLOSURE 2-4' CLOSURE	swing		10	5.4	14.0	16.0	18.0	8.6	0.9	$ 88,451	10.6	1.1	$ 120,775	12.6	1.262	$ 153,100
43	Newark	PROVIDE 1-10' CLOSURE 2-4' CLOSURE	swing		4	8.5	14.0	16.0	18.0	5.5	1.4	$ 61,365	7.5	1.9	$ 86,333	9.5	2.381	$ 111,301
44	Newark	PROVIDE 1-10' CLOSURE 2-4' CLOSURE	swing		4	9.6	14.0	16.0	18.0	4.4	1.1	$ 47,397	6.4	1.6	$ 72,365	8.4	2.101	$ 97,333
47	Newark	±10 L.F. CLOSURE ±6.0' HT. T.O.L. 14.9'	swing	6.0	10	7.7	14.0	16.0	18.0	6.3	0.6	$ 50,601	8.3	0.8	$ 82,926	10.3	1.028	$ 115,250
48	Newark	±10 L.F. CLOSURE ±6.0' HT. T.O.L. 14.9'	swing	6.0	10	6.6	14.0	16.0	18.0	7.4	0.7	$ 68,304	9.4	0.9	$ 100,628	11.4	1.137	$ 132,953
49	Newark	20 L.F. CLOSURE ±6.0' HT.	swing	6.0	20	7.2	14.0	16.0	18.0	6.8	0.3	$ 105,350	8.8	0.4	$ 131,778	10.8	0.539	$ 158,205
49A	Newark	±10 L.F. CLOSURE ±6.0' HT. T.O.L. 14.9'	swing	6.0	10	8.0	14.0	16.0	18.0	6.0	0.6	$ 46,120	8.0	0.8	$ 78,444	10.0	1.000	$ 110,769
50	Newark	±10 L.F. CLOSURE ±6.0' HT. T.O.L. 14.9'	swing	6.0	10	6.8	14.0	16.0	18.0	7.2	0.7	$ 64,901	9.2	0.9	$ 97,225	11.2	1.116	$ 129,550
51	Newark	30' L.F. CLOSURE ±4.5' HT.	swing	4.5	30	6.7	14.0	16.0	18.0	7.3	0.2	$ 144,461	9.3	0.3	$ 201,343	11.3	0.378	$ 258,225
52	Newark	30' L.F. CLOSURE ±4.5' HT.	swing	4.5	30	6.5	14.0	16.0	18.0	7.5	0.3	$ 150,648	9.5	0.3	$ 207,530	11.5	0.385	$ 264,412
53	Newark	20' CLOSURE ±4' HT.	swing	4.0	20	9.6	14.0	16.0	18.0	4.4	0.2	$ 74,104	6.4	0.3	$ 100,531	8.4	0.421	$ 126,958
	NEWARK SUBTOTALS											**$ 4,023,917**			**$ 5,402,250**			**$ 6,780,583**
53A	Harrison 1	30' L.F. CLOSURES ±6.4' HT. & ±7.4' HT.	swing		40	7.5	14.0	16.0	18.0	6.5	0.2	$ 243,026	8.5	0.2	$ 293,132	10.5	0.263	$ 343,237
54	Harrison 1	30' L.F. CLOSURES ±6.4' HT. & ±7.4' HT.	swing	6.4	40	7.9	14.0	16.0	18.0	6.1	0.2	$ 233,598	8.1	0.2	$ 283,704	10.1	0.253	$ 333,810
55	Harrison 1	30' CLOSURES ±8.4' HT. NEED TO CHECK	swing	7.4	30	5.1	14.0	16.0	18.0	8.9	0.3	$ 189,288	10.9	0.4	$ 277,729	12.9	0.429	$ 366,170
56	Harrison 1	30' L.F. CLOSURES ±6' HT.	swing	7.4	30	4.3	14.0	16.0	18.0	9.7	0.3	$ 227,249	11.7	0.4	$ 315,690	13.7	0.458	$ 404,131
57	Harrison 1	30' L.F. CLOSURES ±6' HT.	swing	8.4	30	3.9	14.0	16.0	18.0	10.1	0.3	$ 244,463	12.1	0.4	$ 332,904	14.1	0.471	$ 421,345
58	Harrison 1	30' L.F. CLOSURES ±6' HT.	swing	6.0	30	1.4	14.0	16.0	18.0	12.6	0.4	$ 354,728	14.6	0.5	$ 443,169	16.6	0.554	$ 531,610
59	Harrison 1	30' L.F. CLOSURES ±6' HT.	swing	6.0	30	4.1	14.0	16.0	18.0	9.9	0.3	$ 234,237	11.9	0.4	$ 322,678	13.9	0.463	$ 411,119
59A	Harrison 1		swing		40	9.3	14.0	16.0	18.0	4.7	0.1	$ 196,894	6.7	0.2	$ 247,000	8.7	0.217	$ 297,105
59B	Harrison 1		swing		40	8.8	14.0	16.0	18.0	5.2	0.1	$ 211,145	7.2	0.2	$ 261,251	9.2	0.231	$ 311,357
59C	Harrison 1		swing		40	6.5	14.0	16.0	18.0	7.5	0.2	$ 268,428	9.5	0.2	$ 318,534	11.5	0.288	$ 368,639
59D	Harrison 1		swing		40	14.0	14.0	16.0	18.0	0.0	0.0		2.0	0.1	$ 146,546	4.0	0.100	$ 167,867
	HARRISON 1 SUBTOTALS											**$ 2,403,056**			**$ 3,242,335**			**$ 3,956,389**
60	Newark Flanking	- Western most section (1)	swing	4	45	12.0	14.0	16.0	18.0	2.0	0.0	$ 105,033	4.0	0.1	$ 162,756	6.0	0.133	$ 220,478
61	Newark Flanking	1 of 2	swing	4	45	9.4	14.0	16.0	18.0	4.6	0.1	$ 179,386	6.6	0.1	$ 237,109	8.6	0.191	$ 294,832
62	Newark Flanking	2 of 2	swing	4	45	9.6	14.0	16.0	18.0	4.4	0.1	$ 173,077	6.4	0.1	$ 230,799	8.4	0.186	$ 288,522
63	Newark Flanking	RR 1	swing	8	30	6.0	14.0	16.0	18.0	8.0	0.3	$ 163,365	10.0	0.3	$ 220,246	12.0	0.400	$ 277,128
64	Newark Flanking	(3) 50-foot swing gates in lieu of 150-foo	swing	8	50	8.3	14.0	16.0	18.0	5.7	0.1	$ 682,255	7.7	0.2	$ 908,021	9.7	0.194	$ 1,133,787
65	Newark Flanking	RR 3	swing	10	40	4.7	14.0	16.0	18.0	9.3	0.2	$ 255,592	11.3	0.3	$ 349,454	13.3	0.333	$ 443,316
	NEWARK FLANKING SUBTOTALS											**$1,558,707**			**$2,108,385**			**$2,658,063**
	TOTALS											**$16,232,701**			**$20,649,928**			**$24,951,486**

1 Data provided by AECOM

2 Provided by AECOM

3 Per scope of work.

4 Taken from 1/27/90 GDM drawings.

5 Construction includes the following materials, exclusively: structural steel gate; concrete monolith structure; concrete reinforcing; piles foundatio

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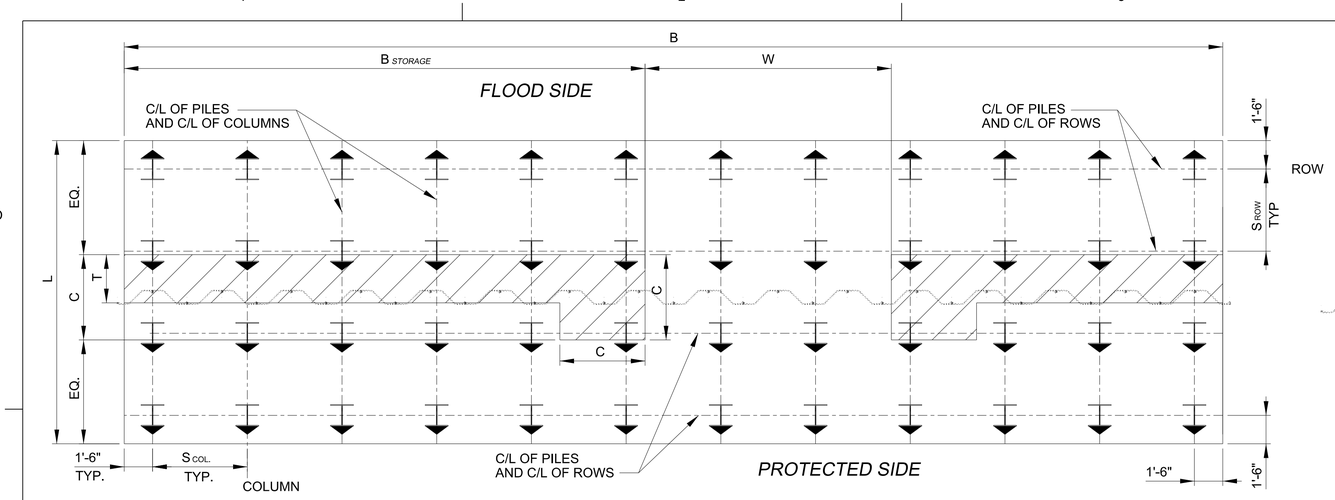
Appendix B:
Typical Flood-Gate and Monolith Concrete Structure Drawings

MAY 2009 NYD BORDER

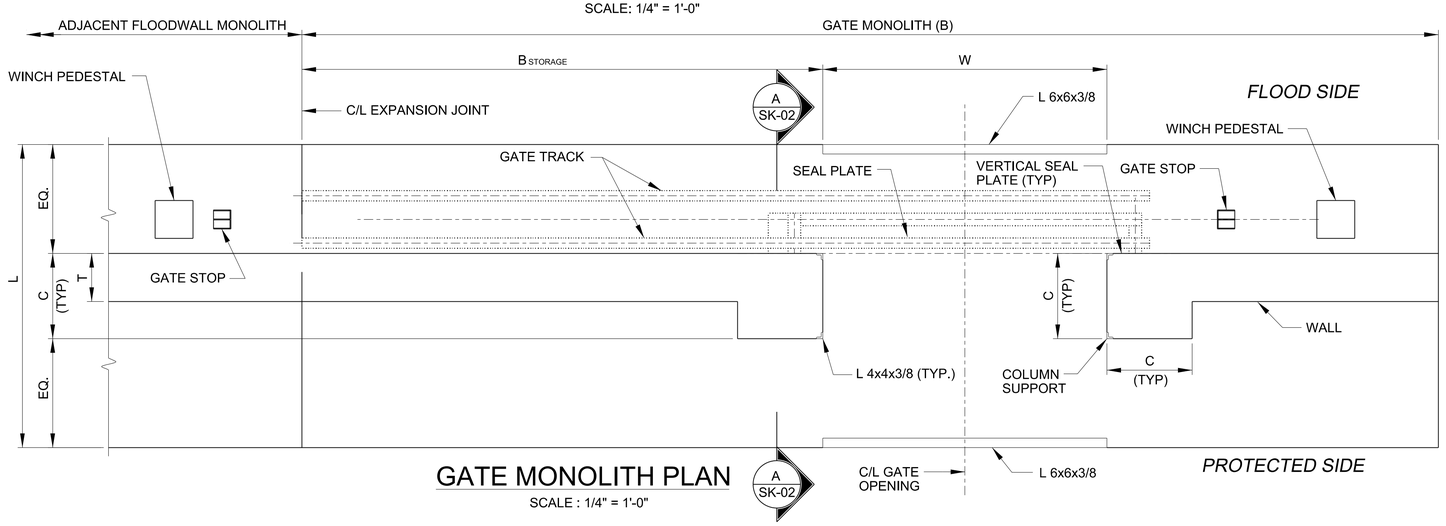


LEGEND

- BATTERED STEEL PILE (3V ON 1H)
- SHEET PILE

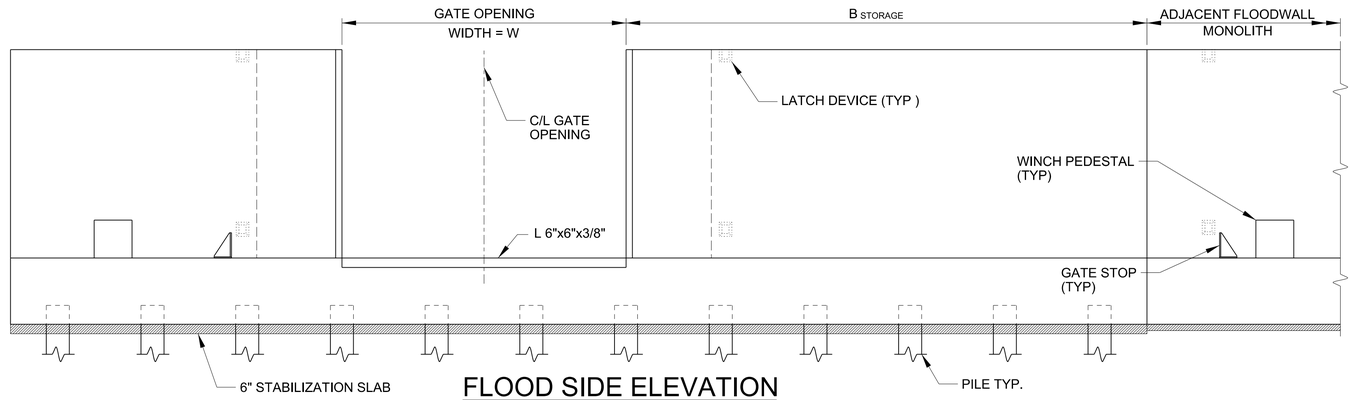
PILE AND FOUNDATION PLAN - GATE MONOLITH

SCALE: 1/4" = 1'-0"



GATE MONOLITH PLAN

SCALE : 1/4" = 1'-0"



FLOOD SIDE ELEVATION

SCALE : 1/4" = 1'-0"

1) WEST KEARNY AREA (H-PILE)

ROLLER GATE STEEL MEMBER SIZES

GATE DESIGNATIONS	GATE HEIGHTS "H" (ft)			
	10	12	14	16
Top Girder	W24x62	W30x90	W33x118	W33x118
Bottom Girder	W24x146	W30x148	W33x201	W33x241
Intercostal	WT7x11	WT7x11	WT7x11	WT7x11
X-Bracing	C10x20	C10x20	C10x20	C10x20
Skin Plate	3/8	3/8	3/8	3/8
Horiz. Brace	L4x4x0.25	L4x4x0.5	L4x4x0.5	L4x4x0.5
GATE OPENING WIDTH "W"=	**50 ft**			

CONCRETE MONOLITH DIMENSION

DIMENSION DESIGNATIONS (ft)	GATE HEIGHTS "H" (ft)			
	10	12	14	16
L	15	15	15	17
B	120	120	120	120
$B_{storage.}$	60	60	60	60
A	4	4	4	4
C	3.5	3.5	4.5	4.5
T	2	2	2.5	2.5
GATE OPENING WIDTH "W"=	**50 ft**			

PILE DIMENSIONS AND INFORMATION

PILE TYPE 14 H-PILE	GATE HEIGHTS "H" (ft)			
	10	12	14	16
# Piles	96	96	96	120
Tip Elevation (ft)	-50	-55	-60	-60
Pile Length(ft)	53	58	64	64
# Rows	4	4	4	5
# Columns	24	24	24	24
$S_{ROW.}$ (ft)	4	4	4	3.5
$S_{COL.}$ (ft)	5'-1"	5'-1"	5'-1"	5'-1"
GATE OPENING WIDTH "W"=	**50 ft**			

US Army Corps
of Engineers
New York District

MARK	DESCRIPTION	DATE	APPR.
MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT
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DWN BY:
CKD BY:
SUBMITTED BY:
DATE: MAY, 2016
SOLICITATION NO.:
CONTRACT NO.:
FILE NAME: $FILE$
SIZE: ANSI D
PLOTTED BY: $USERNAME$
PLOT DATE: $SYTIME$

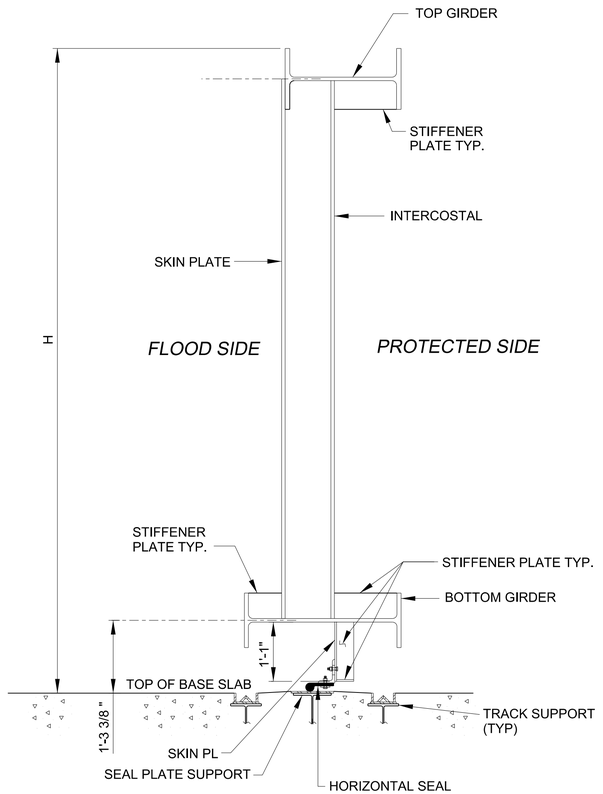
PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ

MONOLITH PLAN AND ELEVATIONS
ROLLER GATES

SHEET REFERENCE NUMBER
SK-01
SHEET 1 OF 6

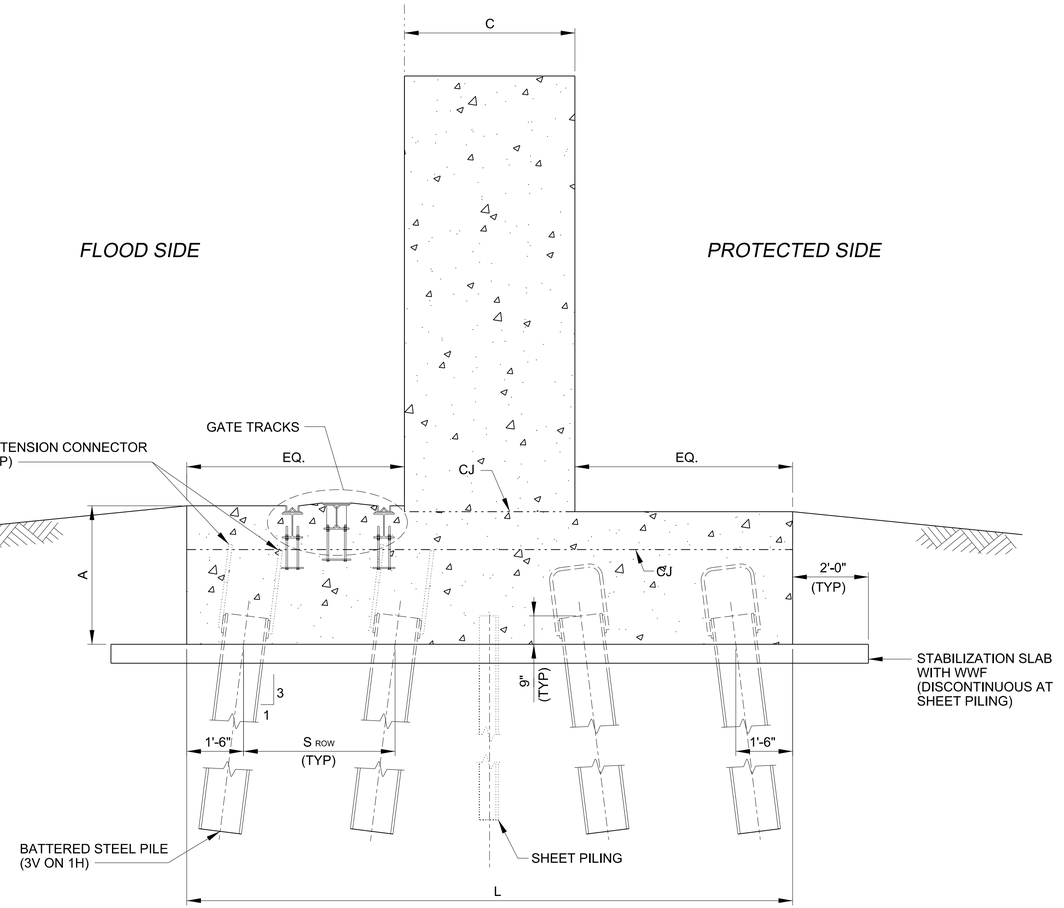
Submitted by: $USERNAME$
FILE NAME : $FILE$
Plot Date: $DATE$ $SYTIME$

MAY 2009 NYD BORDER



ROLLER GATE SECTION (B / SK-03)

SCALE: 3/4" = 1'-0"



MONOLITH SECTION (A / SK-01)

SCALE: 1/2" = 1'-0"

US Army Corps of Engineers
New York District

MARK	DESCRIPTION	DATE	APPR.
MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY:
DWN BY:
CKD BY:
SUBMITTED BY:
DATE: MAY 2016
SOLICITATION NO.:
CONTRACT NO.:
FILE NAME: $FILE$
SIZE: ANSI D
PLOTTED BY: $USERNAME$
PLOT DATE: $SYTIME$

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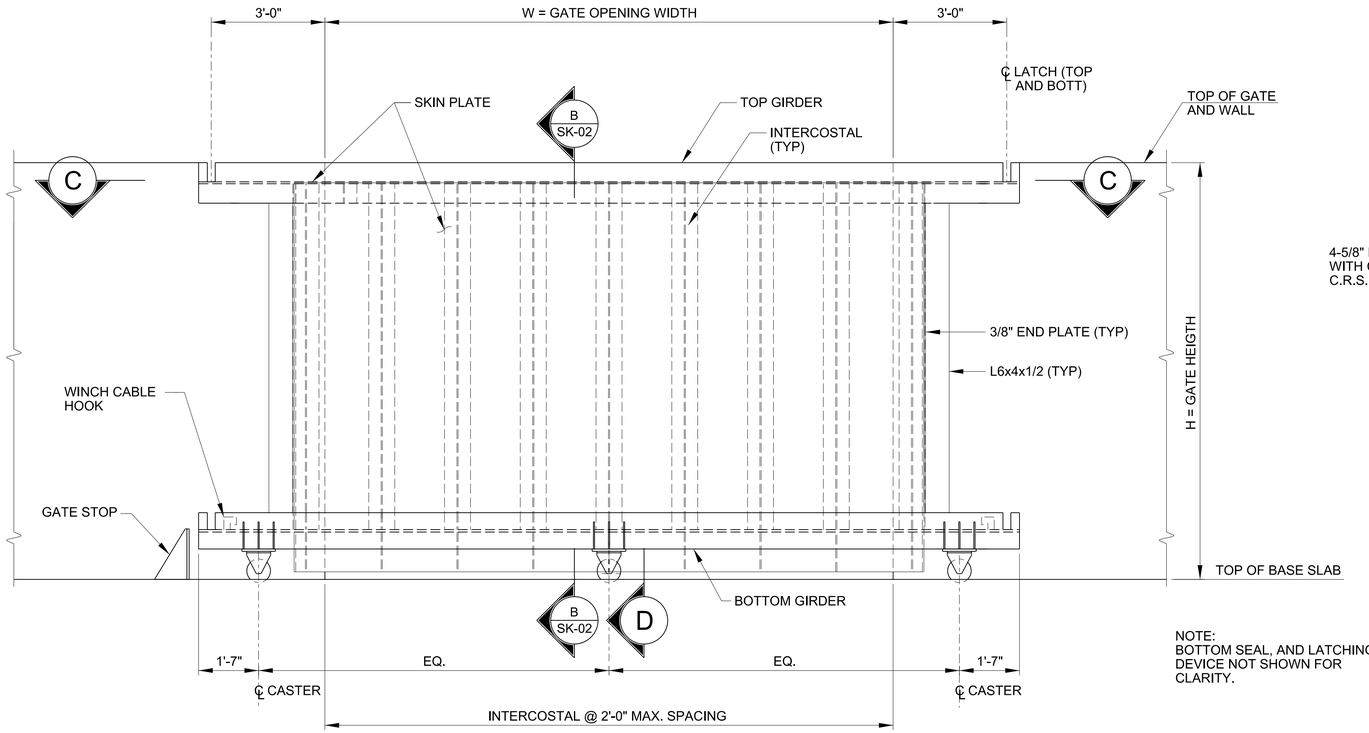
PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ

TYPICAL MONOLITH AND ROLLER GATE SECTIONS

SHEET REFERENCE NUMBER
SK-02
SHEET 2 OF 6

Submitted by: $USERNAME$
FILE NAME : $FILE$
Plot Date: $DATE$ $SYTIME$

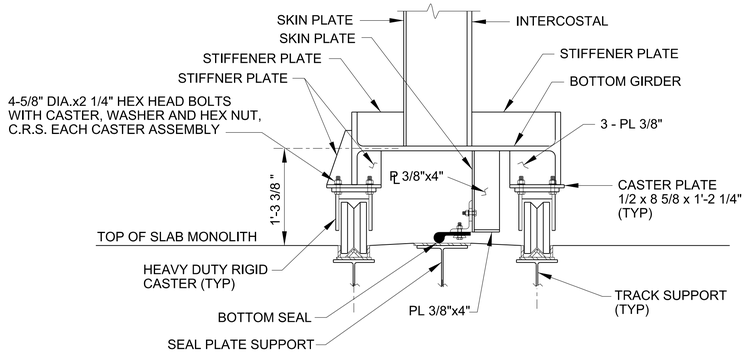
MAY 2009 NYD BORDER



NOTE:
BOTTOM SEAL, AND LATCHING DEVICE NOT SHOWN FOR CLARITY.

ROLLER GATE FLOOD SIDE ELEVATION

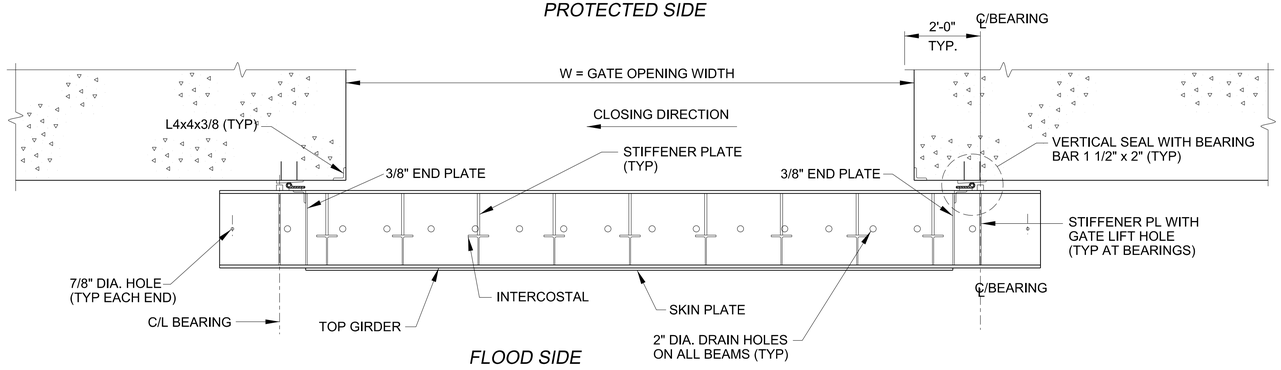
CLOSED POSITION



GATE DETAIL

VIEW D

SCALE: 1" = 1'-0"



NOTE: LATCHING DEVICES NOT SHOWN FOR CLARITY.

SECTION C

SCALE: 1/2" = 1'-0"

US Army Corps of Engineers
New York District

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY:
DWN BY:
CKD BY:
SUBMITTED BY:
DATE: MAY 2016
SOLICITATION NO.:
CONTRACT NO.:
FILE NAME: $FILE$
SIZE: ANSI D
PLOTTED BY: $USERNAME$
PLOT DATE: $SYTIME$

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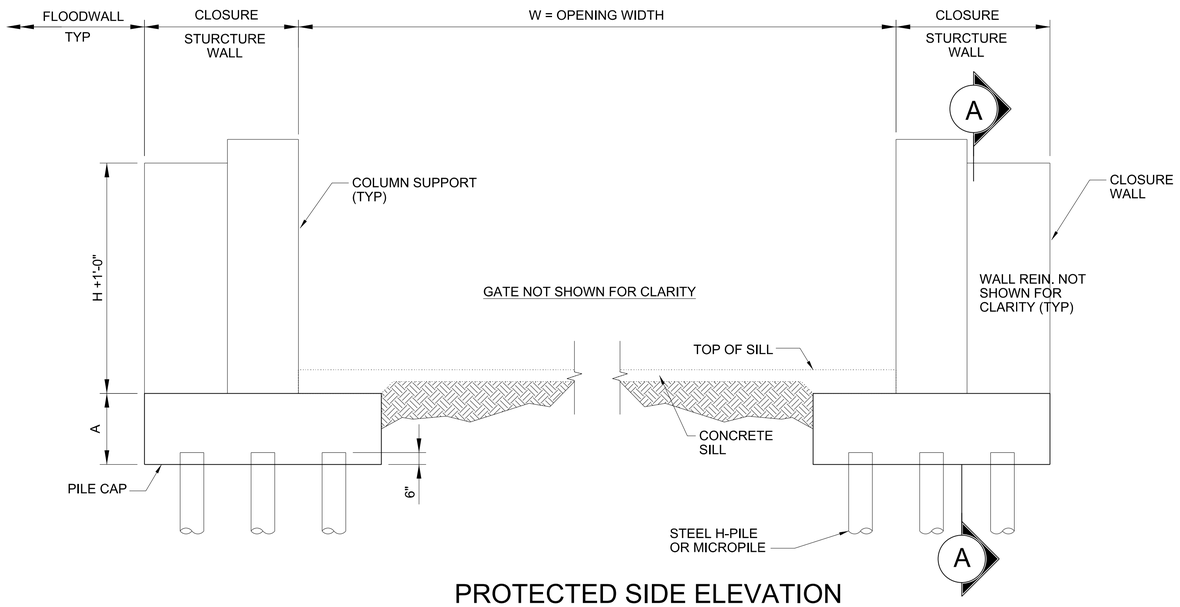
PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ

TYPICAL
SECTIONS & ELEVATIONS
ROLLER GATES

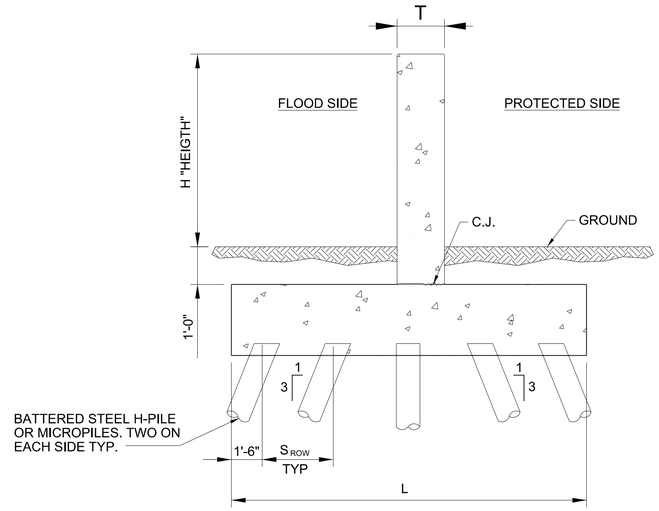
SHEET REFERENCE NUMBER
SK-03
SHEET 3 OF 6

Submitted by: $USERNAME$
FILE NAME : $FILE$
Plot Date: $DATE$ $SYTIME$

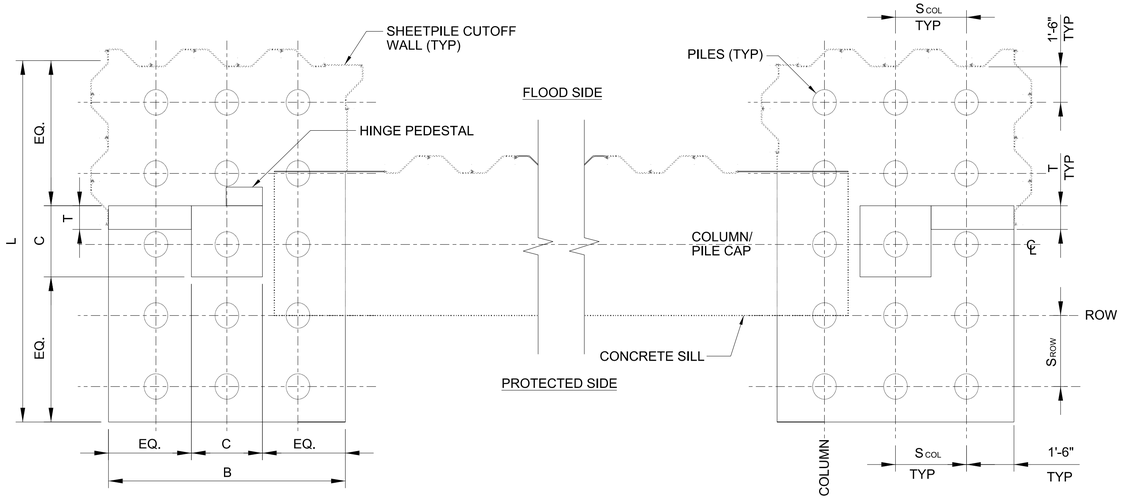
MAY 2009 NYD BORDER



PROTECTED SIDE ELEVATION



SECTION A-A



GATE MONOLITH AND PILE PLAN

SCALE: 3/4" = 1'-0"

US Army Corps of Engineers
New York District

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY:
DWN BY:
CKD BY:
SUBMITTED BY:
DATE: MAY, 2016
SOLICITATION NO.:
CONTRACT NO.:
FILE NAME: $FILE$
SIZE: ANSI D
PLOT DATE: $SYTIME$
PLOTTED BY: $USERNAME$

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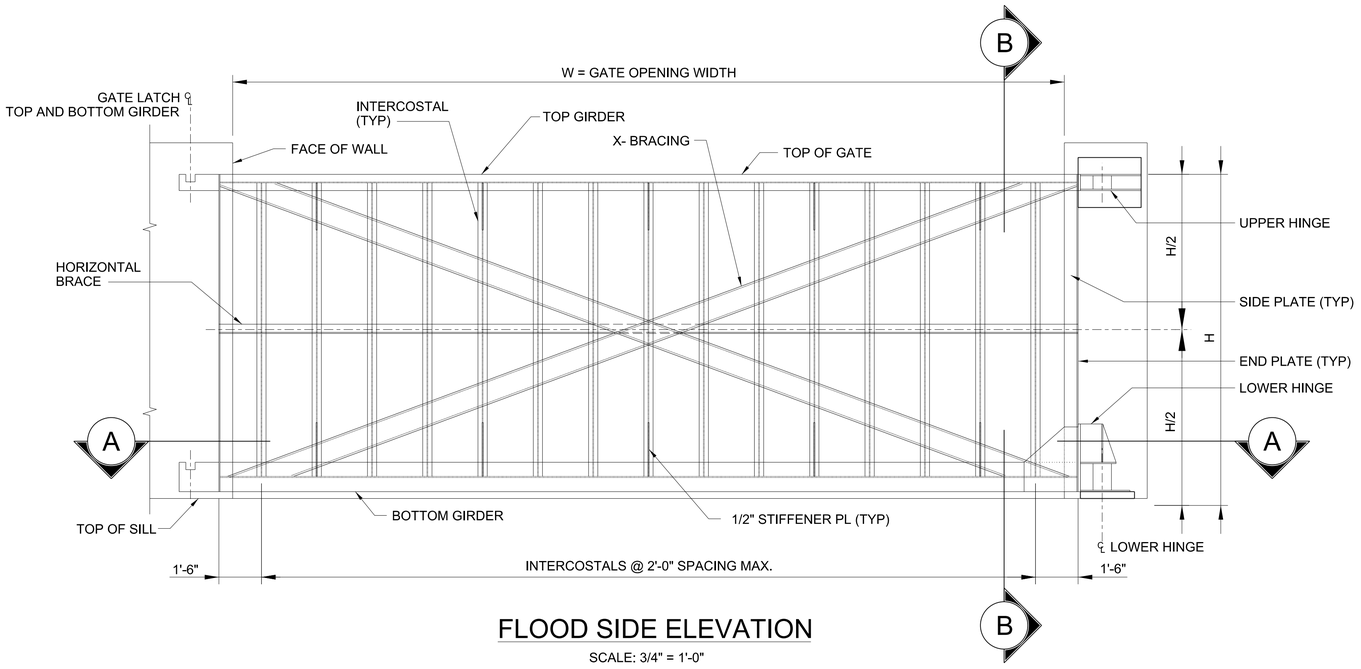
PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ

TYPICAL PROPOSED MONOLITH
SUPPORT STRUCTURE
SWING GATE

SHEET REFERENCE NUMBER
SK-04
SHEET 4 OF 6

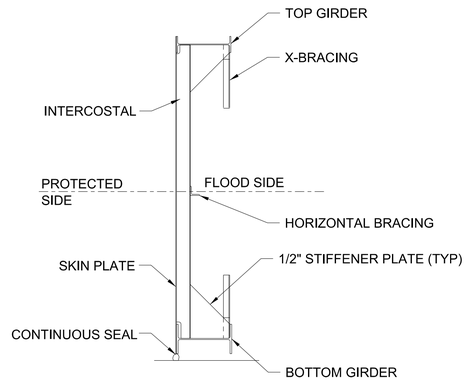
Submitted by: $USERNAME$ Plot Date: $DATE$ $SYTIME$
FILE NAME : $FILE$

MAY 2009 NYD BORDER



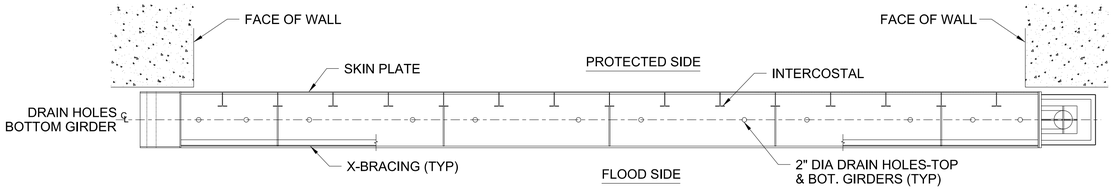
FLOOD SIDE ELEVATION

SCALE: 3/4" = 1'-0"



SECTION B-B

SCALE: 1/4" = 1'-0"



SECTION A-A

SCALE: 3/4" = 1'-0"

US Army Corps
of Engineers
New York District

MARK	DESCRIPTION	DATE	APPR.	MARK	DESCRIPTION	DATE	APPR.

DATE: MAY, 2016
SOLICITATION NO.:
CONTRACT NO.:
PLOT DATE: $SYTIME$

DESIGNED BY:
DWN BY:
CKD BY:
SUBMITTED BY:
FILE NAME: $FILE$
SIZE: ANSI D
PLOTTED BY: $USERNAME$

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PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ

TYPICAL PROPOSED GATE
PLAN AND ELEVATION
SWING GATE

SHEET
REFERENCE
NUMBER
SK-05
SHEET 5 OF 6

Plot Date: $DATE$ $SYTIME$
Submitted by: $USERNAME$
FILE NAME : $FILE$

EAST/WEST OF KEARNY, NEWARK AND HARRISON AREA (H-PILE)

1)

SWING GATE STEEL MEMBER SIZES						
GATE DESIGNATIONS	GATE HEIGHTS "H" (ft)					
	6	8	10	12	14	16
Top Girder	Wx8x13	W8x13	W10x12	W12x16	W14x22	W14x22
Bottom Girder	W8x18	W8x18	W10x22	W12x26	W14x30	W14x34
Intercostal	WT3x6	WT3x6	WT4x6.5	WT5x7.5	WT6x8	WT7x11
X-Bracing	C10x20	C10x20	C10x20	C10x20	C10x20	C10x20
Skin Plate	5/16	5/16	5/16	5/16	6/16	6/16
Horiz. Brace	L4x4x0.25	L4x4x0.5	L4x4x0.5	L4x4x0.5	L4x4x0.5	L4x4x0.5
GATE OPENING WIDTH "W"=	**10 ft**					

CONCRETE MONOLITH DIMENSION						
DIMENSION DESIGNATIONS	GATE HEIGHTS "H" (ft)					
	6	8	10	12	14	16
L	7	9	9	12	12	15
B	6	6	6	9	9	10
A	2.5	2.5	2.5	3	3	4
C	2	2	2.5	3	3	3.5
T	1	1	1	1.5	1.5	1.75
GATE OPENING WIDTH "W"=	**10 ft**					

PILE DIMENSIONS AND INFORMATION						
PILE TYPE 14 H-PILE	GATE HEIGHTS "H" (ft)					
	6	8	10	12	14	16
# Piles	4	6	6	9	9	15
Tip Elevation (ft)	-35	-40	-45	-50	-55	-55
Pile Length(ft)	37	43	48	53	58	58
# Rows	2	3	3	3	3	5
# Columns	2	2	2	3	3	3
$S_{ROW.}$ (ft)	4	3	3	4.5	4.5	3
$S_{COL.}$ (ft)	3	3	3	3	3	3.5
GATE OPENING WIDTH "W"=	**10 ft**					

2)

CONCRETE MONOLITH DIMENSION					
DIMENSION DESIGNATIONS (ft)	GATE HEIGHTS "H" (ft)				
	5	7	9	11	13
L	7	9	9	9	10
B	6	6	6	6	9
A	2.5	2.5	2.5	2.5	3
C	2	2	2	2.5	2.5
T	1	1	1	1.5	1.5
GATE OPENING WIDTH "W"=	**20 ft**				

PILE DIMENSIONS AND INFORMATION					
PILE TYPE 14 H-PILE	GATE HEIGHTS "H" (ft)				
	5	7	9	11	13
# Piles	4	6	6	6	9
Tip Elevation (ft)	-35	-40	-50	-55	-55
Pile Length(ft)	37	43	53	58	58
# Rows	2	3	3	3	3
# Columns	2	2	2	2	3
$S_{ROW.}$ (ft)	4	3	3	3	3.5
$S_{COL.}$ (ft)	3	3	3	3	3
GATE OPENING WIDTH "W"=	**20 ft**				

3)

SWING GATE STEEL MEMBER SIZES										
GATE DESIGNATIONS	GATE HEIGHTS "H" (ft)									
	2*	4	6	8	10	12	13	14	16	18
Top Girder	W12x16	W12x16	W12x22	W16x26	W18x35	W24x55	W24x55	W24x55	W24x62	W24x68
Bottom Girder	W12x26	W12x26	W12x35	W16x50	W18x65	W24x84	W24x84	W24x94	W24x117	W24x146
Intercostal	WT3x6	WT3x6	WT3x6	WT3x6	WT4x6.5	WT5x7.5	WT5x7.5	WT6x8	WT7x11	WT7x11
X-Bracing	C10x20	C10x20	C10x20	C10x20	C10x20	C10x20	C10x20	C10x20	C10x20	C10x20
Skin Plate	5/16	5/16	5/16	5/16	5/16	5/16	5/16	3/8	3/8	3/8
Horiz. Brace	L4x4x0.25	L4x4x0.25	L4x4x0.25	L4x4x0.5	L4x4x0.5	L4x4x0.5	L4x4x0.5	L4x4x0.5	L4x4x0.5	L4x4x0.5
GATE OPENING WIDTH "W"=	**30 ft**									
*** Gate Height was rounded up to 2ft**										

CONCRETE MONOLITH DIMENSION										
DIMENSION DESIGNATIONS (ft)	GATE HEIGHTS "H" (ft)									
	2*	4	6	8	10	12	13	14	16	18
L	6	6	9	9	10	12	12	12	15	17
B	6	6	6	6	9	9	9	9	10	10
A	2.5	2.5	2.5	2.5	3	3	3	3	4	4
C	1.5	1.5	2	2	2.5	3	3	3	3.5	4
T	1	1	1	1	1	1.5	1.5	1.5	1.75	2
GATE OPENING WIDTH "W"=	**30 ft**									
*** Gate Height was rounded up to 2ft**										

PILE DIMENSIONS AND INFORMATION										
PILE TYPE 14 H-PILE	GATE HEIGHTS "H" (ft)									
	2*	4	6	8	10	12	13	14	16	18
# Piles	4	4	6	6	9	9	9	9	15	15
Tip Elevation (ft)	-40	-40	-40	-50	-50	-55	-60	-60	-60	-60
Pile Length(ft)	43	43	43	53	53	58	64	64	64	64
# Rows	2	2	3	3	3	3	3	3	5	5
# Columns	2	2	2	2	3	3	3	3	3	3
$S_{ROW.}$ (ft)	3	3	3	3	3.5	4.5	4.5	4.5	3	3.5
$S_{COL.}$ (ft)	3	3	3	3	3	3	3	3	3.5	3.5
GATE OPENING WIDTH "W"=	**30 ft**									
*** Gate Height was rounded up to 2ft**										

4)

SWING GATE STEEL MEMBER SIZES					
GATE DESIGNATIONS	GATE HEIGHTS "H" (ft)				
	9	11	13	15	17
Top Girder	W18x35	W24x55	W24x55	W24x68	W27x84
Bottom Girder	W18x65	W24x76	W24x103	W24x131	W27x146
Intercostal	WT4x6.5	WT5x7.5	WT6x8.0	WT7x11	WT7x17
X-Bracing	C10x20	C10x20	C10x20	C10x20	C10x20
Skin Plate	5/16	5/16	5/16	3/8	3/8
Horiz. Brace	L4x4x0.25	L4x4x0.5	L4x4x0.5	L4x4x0.5	L4x4x0.5
GATE OPENING WIDTH "W"=	**35 ft**				

CONCRETE MONOLITH DIMENSION					
DIMENSION DESIGNATIONS (ft)	GATE HEIGHTS "H" (ft)				
	9	11	13	15	17
L	10	12	12	15	17
B	9	9	9	10	10
A	3	3	3	4	4
C	2.5	3	3	3.5	4
T	1	1.5	1.5	1.75	2
GATE OPENING WIDTH "W"=	**35 ft**				

PILE DIMENSIONS AND INFORMATION					
PILE TYPE 14 H-PILE	GATE HEIGHTS "H" (ft)				
	9	11	13	15	17
# Piles	9	9	9	15	15
Tip Elevation (ft)	-50	-55	-60	-60	-60
Pile Length(ft)	53	58	64	64	64
# Rows	3	3	3	5	5
# Columns	3	3	3	3	3
$S_{ROW.}$ (ft)	3.5	4.5	4.5	3	3.5
$S_{COL.}$ (ft)	3	3	3	3.5	3.5
GATE OPENING WIDTH "W"=	**35 ft**				

HARRISON AREA (MICROPILES)

5)

SWING GATE STEEL MEMBER SIZES			
GATE DESIGNATIONS	GATE HEIGHTS "H" (ft)		
	10	12	14
Top Girder	W24x55	W24x62	W24x76
Bottom Girder	W24x84	W24x117	W24x162
X-Bracing	C10x20	C10x20	C10x20
Skin Plate	5/16	5/16	3/8
Horiz. Brace	L4x4x0.25	L4x4x0.5	L4x4x0.5
GATE OPENING WIDTH "W"=	**40 ft**		

CONCRETE MONOLITH DIMENSION			
DIMENSION DESIGNATIONS (ft)	GATE HEIGHTS "H" (ft)		
	10	12	14
L	12	12	15
B	9	9	10
A	3	3	4
C	3	3	3.5
T	1.5	1.5	1.75
GATE OPENING WIDTH "W"=	**40 ft**		

PILE DIMENSIONS AND INFORMATION			
PILE TYPE 14 H-PILE	GATE HEIGHTS "H" (ft)		
	10	12	14
# Piles	9	12	15
Tip Elevation (ft)	-55	-60	-60
Pile Length(ft)	58	64	64
# Rows	3	4	5
# Columns	3	3	3
$S_{ROW.}$ (ft)	4.5	3	3
$S_{COL.}$ (ft)	3	3	3.5
GATE OPENING WIDTH "W"=	**40 ft**		

6)

SWING GATE STEEL MEMBER SIZES			
GATE DESIGNATIONS	GATE HEIGHTS "H" (ft)		
	5	7	9
Top Girder	W16x26	W21x44	W21x48
Bottom Girder	W16x45	W21x62	W21x93
Intercostal	WT3x6	WT3x6	WT3x6
X-Bracing	C10x20	C10x20	C10x20
Skin Plate	5/16	5/16	5/16
Horiz. Brace	L4x4x0.25	L4x4x0.5	L4x4x0.5
GATE OPENING WIDTH "W"=	**45 ft**		

CONCRETE MONOLITH DIMENSION			
DIMENSION DESIGNATIONS (ft)	GATE HEIGHTS "H" (ft)		
	5	7	9
L	9	10	12
B	6	9	9
A	3	3	3
C	2.5	3	3
T	1.5	1.5	1.5
GATE OPENING WIDTH "W"=	**45 ft**		

PILE DIMENSIONS AND INFORMATION			
PILE TYPE 14 H-PILE	GATE HEIGHTS "H" (ft)		
	5	7	9
# Piles	6	9	9
Tip Elevation (ft)	-45	-45	-50
Pile Length(ft)	48	48	53
# Rows	3	3	3
# Columns	2	3	3
$S_{ROW.}$ (ft)	3	3.5	4.5
$S_{COL.}$ (ft)	3	3	3
GATE OPENING WIDTH "W"=	**45 ft**		

7)

SWING GATE STEEL MEMBER SIZES			
GATE DESIGNATIONS	GATE HEIGHTS "H" (ft)		
	6	8	10
Top Girder	W18x40	W24x55	W24x62
Bottom Girder	W18x65	W24x84	W24x131
Intercostal	WT3x6	WT3x6	WT3x6
X-Bracing	C10x20	C10x20	C10x20
Skin Plate	5/16	5/16	5/16
Horiz. Brace	L4x4x0.25	L4x4x0.5	L4x4x0.5
GATE OPENING WIDTH "W"=	**50 ft**		

CONCRETE MONOLITH DIMENSION			
DIMENSION DESIGNATIONS (ft)	GATE HEIGHTS "H" (ft)		
	6	8	10
L	9	10	12
B	6	9	9
A	3	3	3
C	2.5	3	3
T	1.5	1.5	1.5
GATE OPENING WIDTH "W"=	**50 ft**		

PILE DIMENSIONS AND INFORMATION			
PILE TYPE 14 H-PILE	GATE HEIGHTS "H" (ft)		
	6	8	10
# Piles	6	9	12
Tip Elevation (ft)	-50	-50	-55
Pile Length(ft)	53	53	58
# Rows	3	3	4
# Columns	2	3	3
$S_{ROW.}$ (ft)	3	3.5	3
$S_{COL.}$ (ft)	3	3	3
GATE OPENING WIDTH "W"=	**50 ft**		

1)

SWING GATE STEEL MEMBER SIZES				
GATE DESIGNATIONS	GATE HEIGHTS "H" (ft)			
	11	13	15	17
Top Girder	W21x44	W21x44	W24x55	W24x68
Bottom Girder	W21x68	W21x93	W24x104	W24x146
Intercostal	WT5x7.5	WT6x8.0	WT7x17	WT7x17
X-Bracing	C10x20	C10x20	C10x20	C10x20
Skin Plate	5/16	5/16	3/8	3/8
Horiz. Brace	L4x4x0.25	L4x4x0.5	L4x4x0.5	L4x4x0.5
GATE OPENING WIDTH "W"=	**30 ft**			

CONCRETE MONOLITH DIMENSION				
DIMENSION DESIGNATIONS (ft)	GATE HEIGHTS "H" (ft)			
	11	13	15	17
L	12	12	15	17
B	9	9	10	10
A	3	3	4	4
C	3	3	3.5	4
T	1.5	1.5	1.75	2
GATE OPENING WIDTH "W"=	**30 ft**			

PILE DIMENSIONS AND INFORMATION				
PILE TYPE 9" MICROPILES	GATE HEIGHTS "H" (ft)			
	11	13	15	17
# Piles	9	9	12	15
Tip Elevation (ft)®	-60	-60	-60	-60
Pile Length(ft)	64	64	64	64
# Rows	3	3	4	5
# Columns	3	3	3	3
$S_{ROW.}$ (ft)	4.5	4.5	4	3.5
$S_{COL.}$ (ft)	3	3	3.5	3.5
GATE OPENING WIDTH "W"=	**30 ft**			

®MICRO PILE TIP INCLUDES 10' SOCKET, ASSUMED BED ROCK @ EL. -50.0 SOCKET TIP @ EL. -60.0

2)

SWING GATE STEEL MEMBER SIZES						
GATE DESIGNATIONS	GATE HEIGHTS "H" (ft)					
	2	4	6	8	10	12
Top Girder	W12x19	W12x26	W12x30	W18x40	W24x62	W24x76
Bottom Girder	W12x35	W12x35	W12x53	W18x76	W24x94	W24x117
Intercostal	WT3x6	WT3x6	WT3x6	WT5x7.5	WT5x7.5	WT6x8.0
X-Bracing	C10x20	C10x20	C10x20	C10x20	C10x20	C10x20
Skin Plate	5/16	5/16	5/16	5/16	5/16	5/16
Horiz. Brace	L4x4x0.25	L4x4x0.25	L4x4x0.25	L4x4x0.25	L4x4x0.25	L4x4x0.25
GATE OPENING WIDTH "W"=	**40 ft**					

CONCRETE MONOLITH DIMENSION						
DIMENSION DESIGNATIONS (ft)	GATE HEIGHTS "H" (ft)					
	2	4	6	8	10	12
L	6	6	9	9	12	12
B	6	6	6	6	6	6
A	2.5	2.5	2.5	3	3	3
C	1.5	2	2	2.5	3	3
T	1	1.5	1.5	1.5	1.5	1.5
GATE OPENING WIDTH "W"=	**40 ft**					

PILE DIMENSIONS AND INFORMATION						
PILE TYPE 9" MICROPILES	GATE HEIGHTS "H" (ft)					
	2	4	6	8	10	12
# Piles	4	4	6	6	8	8
Tip Elevation (ft)®	-60	-60	-60	-60	-60	-60
Pile Length(ft)	64	64	64	64	64	64
# Rows	2	2	3	3	4	4
# Columns	2	2	2	2	2	2
$S_{ROW.}$ (ft)	3	3	3	3	3	3
$S_{COL.}$ (ft)	3	3	3	3	3	3
GATE OPENING WIDTH "W"=	**40 ft**					

®MICRO PILE TIP INCLUDES 10' SOCKET, ASSUMED BED ROCK @ EL. -50.0 SOCKET TIP @ EL. -60.0

US Army Corps of Engineers
New York District

MARK	DESCRIPTION	DATE	APPR.
MARK	DESCRIPTION	DATE	APPR.

DESIGNED BY:
DWN BY:
CKD BY:
SUBMITTED BY:
DATE: MAY 2016
SOLICITATION NO.:
CONTRACT NO.:
FILE NAME: $FILE$$
SIZE: ANSI D
PLOTTED BY: $USERNAME$
PLOT DATE: $SYTIME$

U. S. ARMY ENGINEER DISTRICT
NEW YORK
http://www.nan.usace.army.mil

PASSAIC RIVER TIDAL
GENERAL REEVALUATION REPORT
LOWER PASSAIC RIVER, NJ
GATE / MONOLITH
SIZE, DIMENSIONS AND INFORMATION
TABLES
SWING GATE

SHEET REFERENCE NUMBER
SK-06
SHEET 6 OF 6

MAY 2009, NYD BORDER
Submitted by: $USERNAME$
FILE NAME : $FILE$
Plot Date: $DATE$ $SYTIME$