Passaic River Tidal – General Reevaluation Report

Geotechnical Report

April 6, 2016

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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.
2. "Design and Construction of Levees", EM1110-2-1913, United States Army Corps of Engineers, dated April 30, 2000.
3. Idriss, I. M., & Boulanger, R. W. (2008). Soil liquefaction during earthquakes. Earthquake engineering research institute.
4. "http://earthquake.usgs.gov/hazards/products/conterminous/2008/", Accessed December 14, 2015.
5. "AASHTO LRFD Bridge Design Specifications", 7th ed., American Association of State Highway and Transportation Officials, dated 2014.
6. Das, B. M. (2006). *Principles of geotechnical engineering*, Nelson, Ontario, Canada, 686 p.
7. "Retaining & Flood Walls", EM 1110-2-2502, United States Army Corps of Engineers, dated September 29, 1989.
8. "Design Guidance for Levee Underseepage", ETL-1110-2-569, United States Army Corps of Engineers, dated May a, 2005.

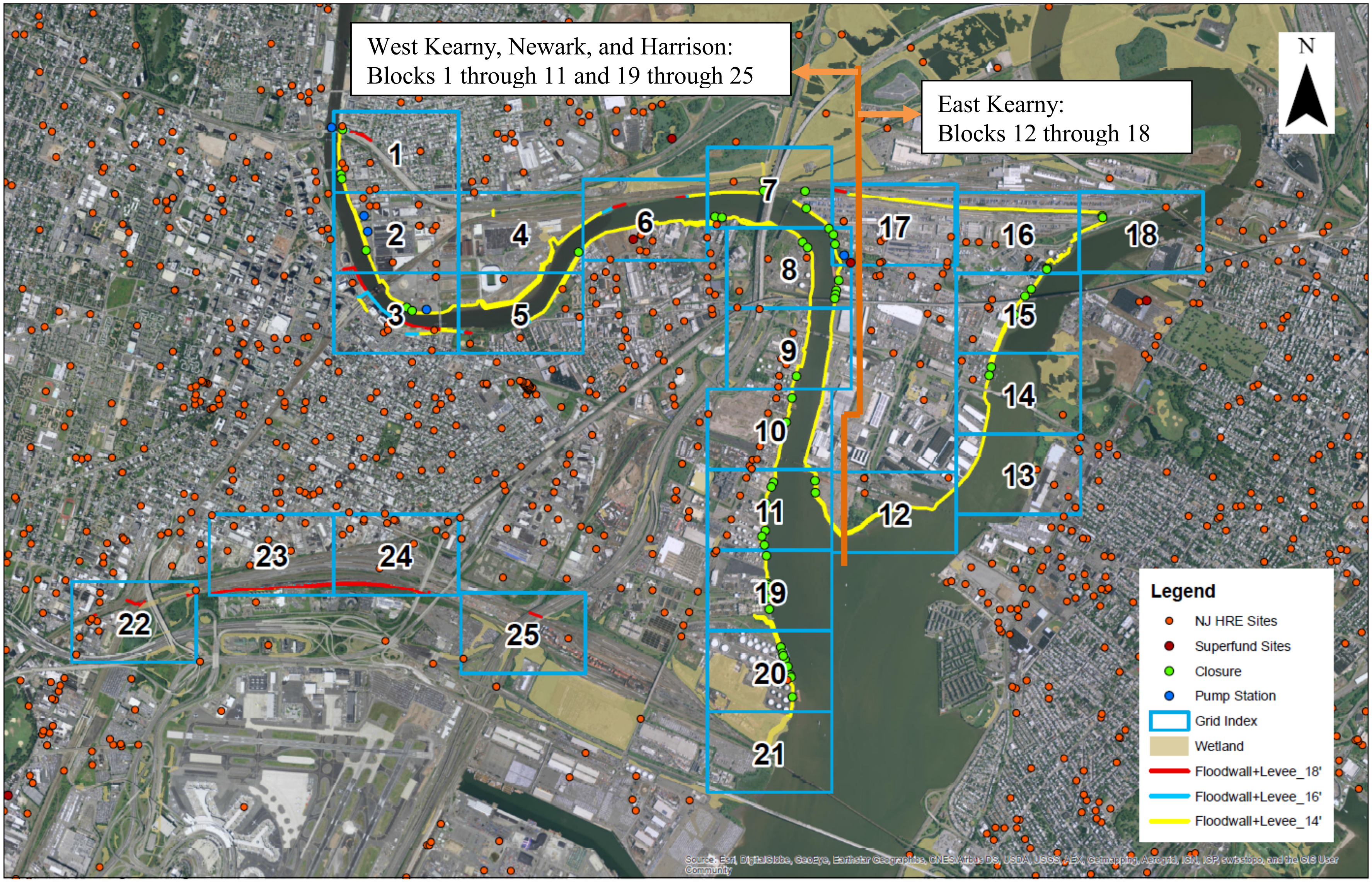

Figure 1. Site Location Plan and Segments

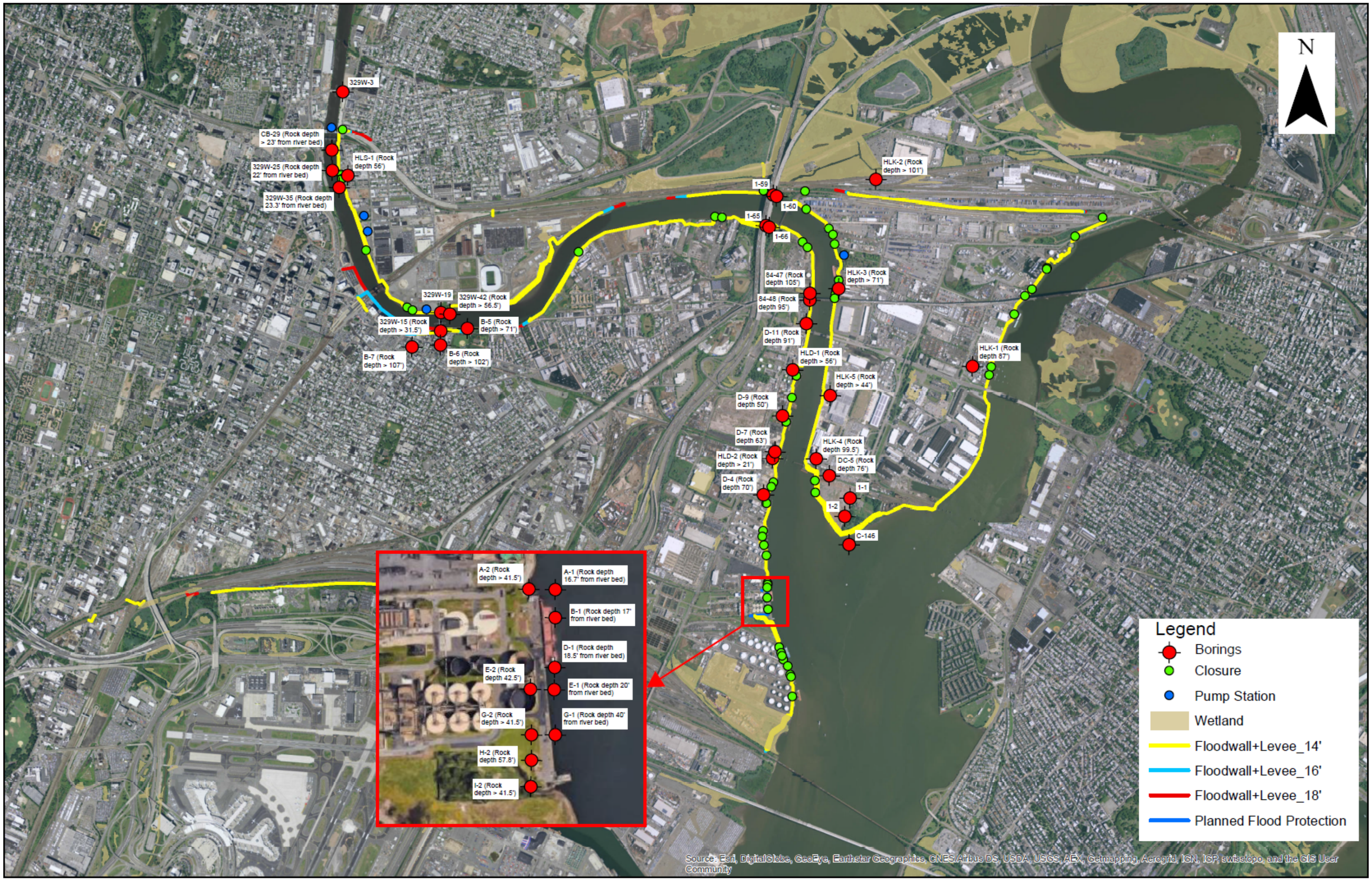

Figure 2. Boring Location Plan

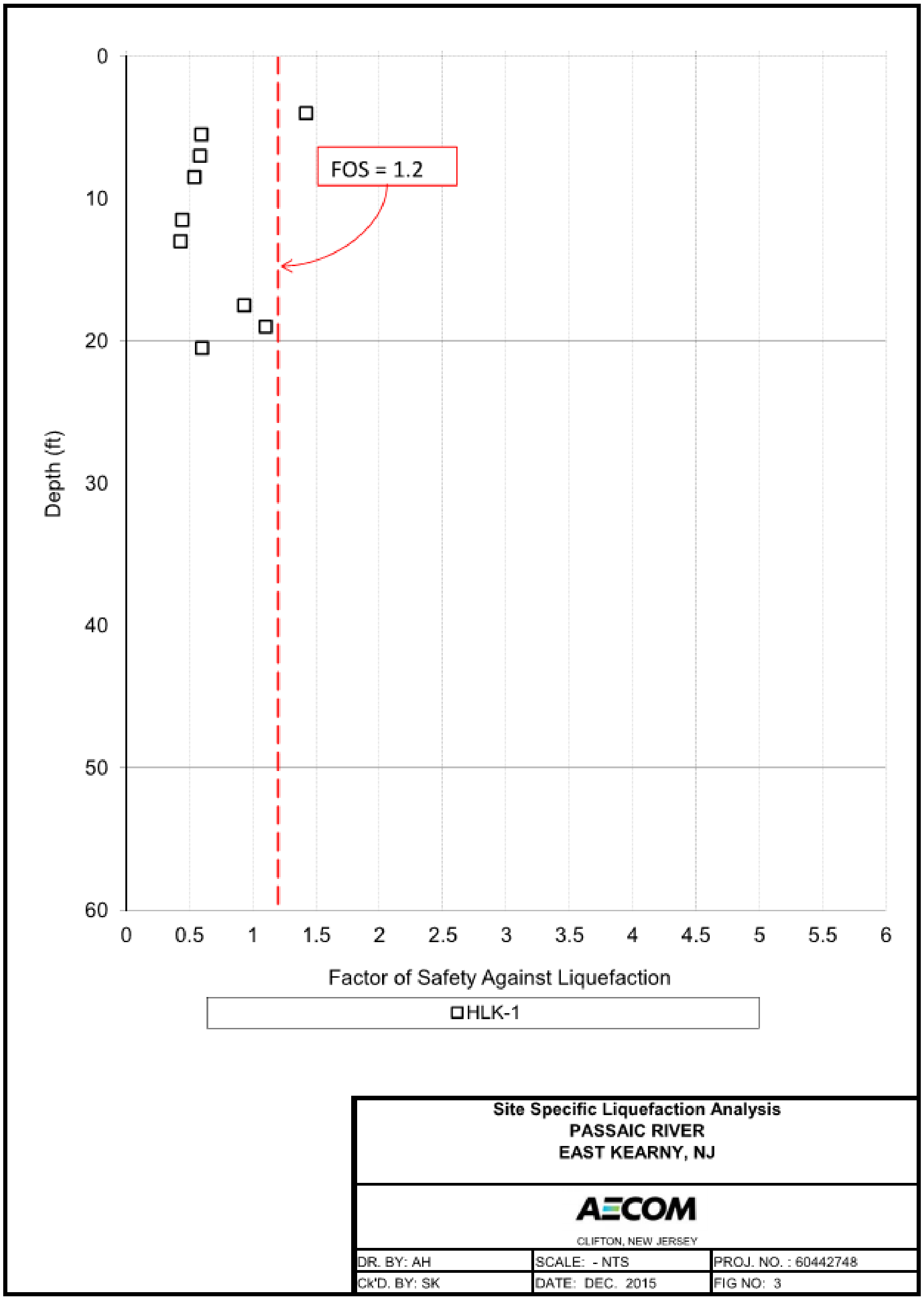

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Figure 3. FOS Against Liquefaction – East Kearny

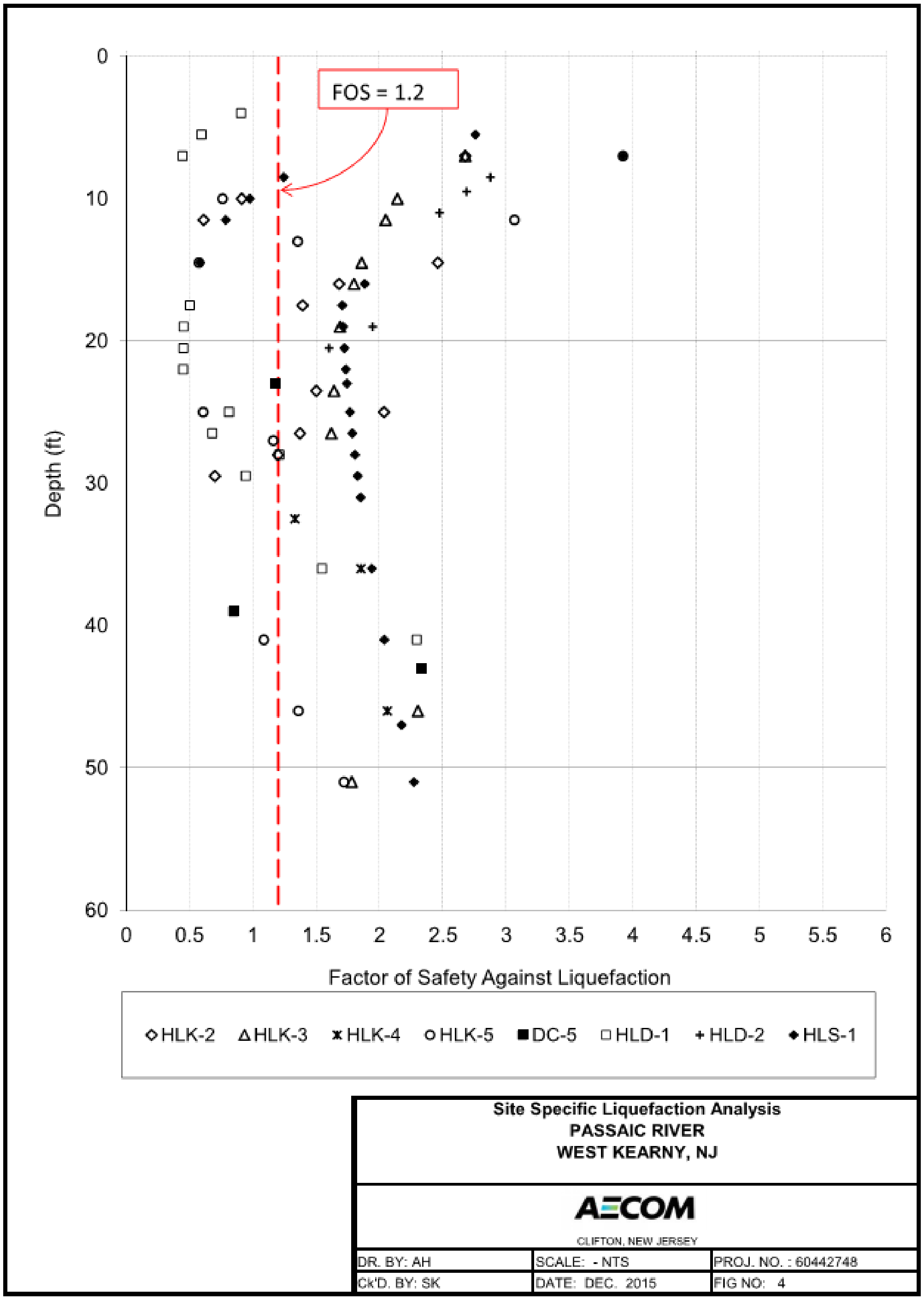

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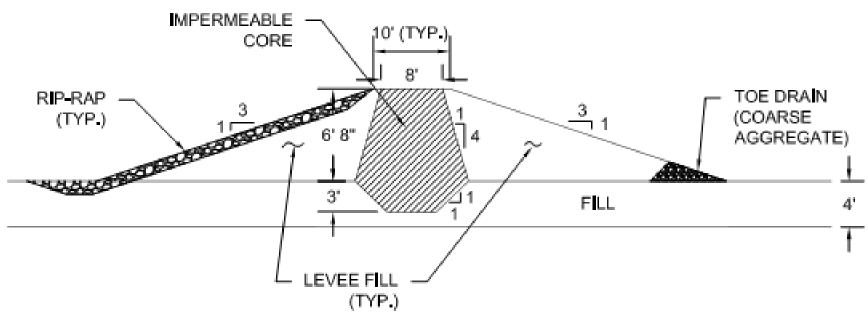
Figure 4. FOS Against Liquefaction – West Kearny, Newark, and Harrison



TYPICAL LEVEE SECTION

TYPICAL SECTION OF PROPOSED LEVEE

PASSAIC RIVER FLOOD RISK MANAGEMENT
Tidal Areas in Harrison, Kearny, and Newark, NJ

AECOM
CLIFTON, NJ

DGN. BY: AH	SCALE: NTS	PROJ. NO.: 60442748
CHK. BY: SK	DATE: 01/28/16	Figure. NO.: 5

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Figure 5. Typical Section of Proposed 6ft High Levee

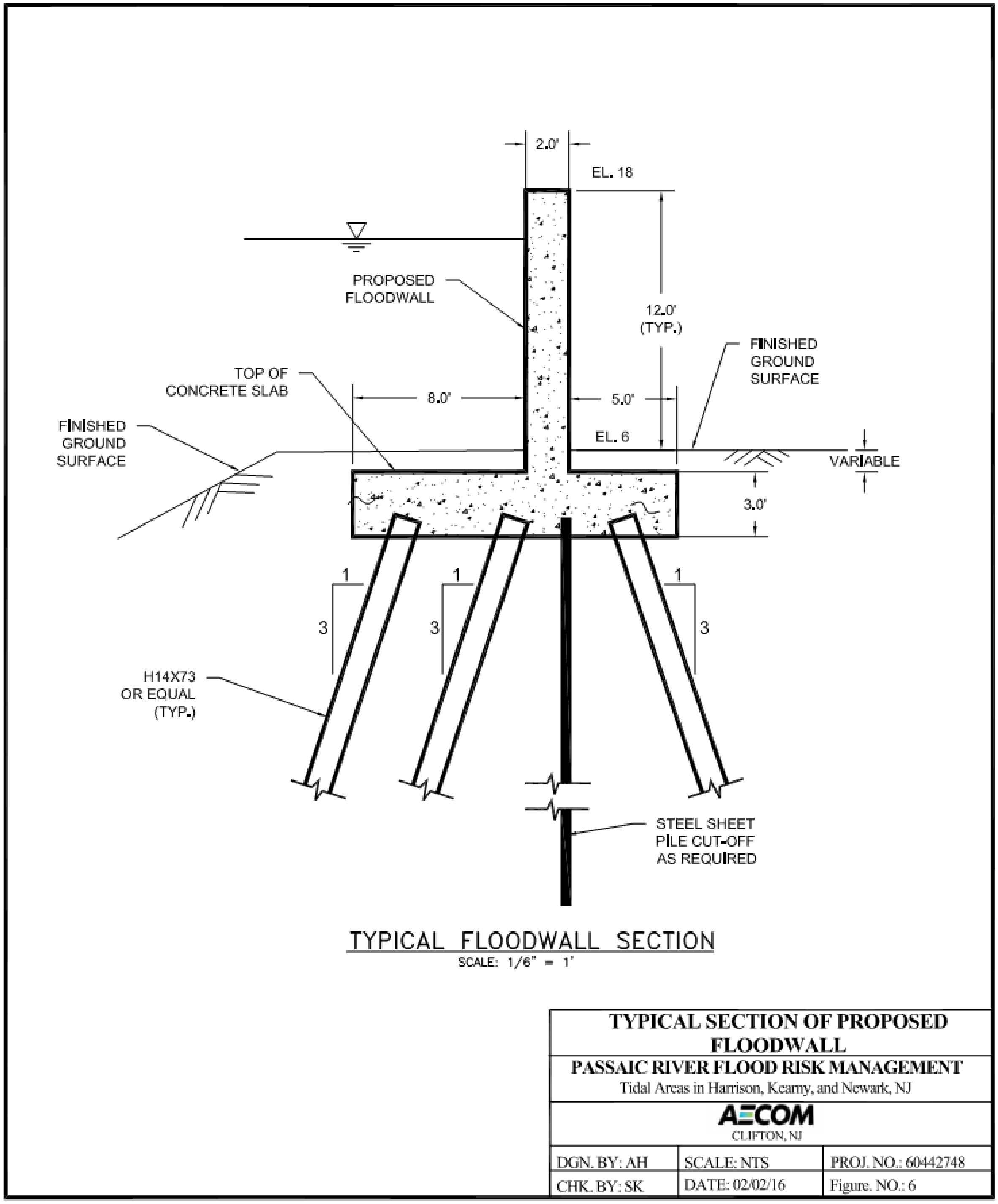


Figure 6. Typical Section of Floodwall

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		

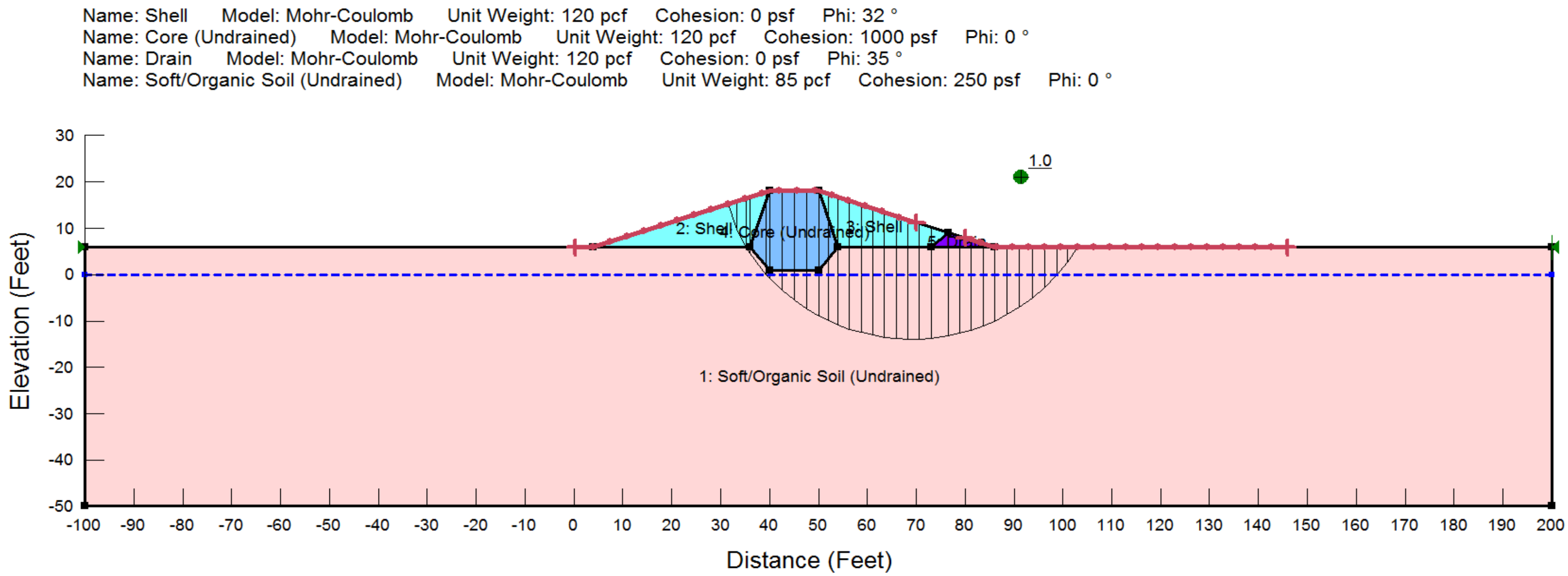
Table A.3: Summary of Slope Stability and Seepage Analyses Results for Case (b) Levee (6 ft high)

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.



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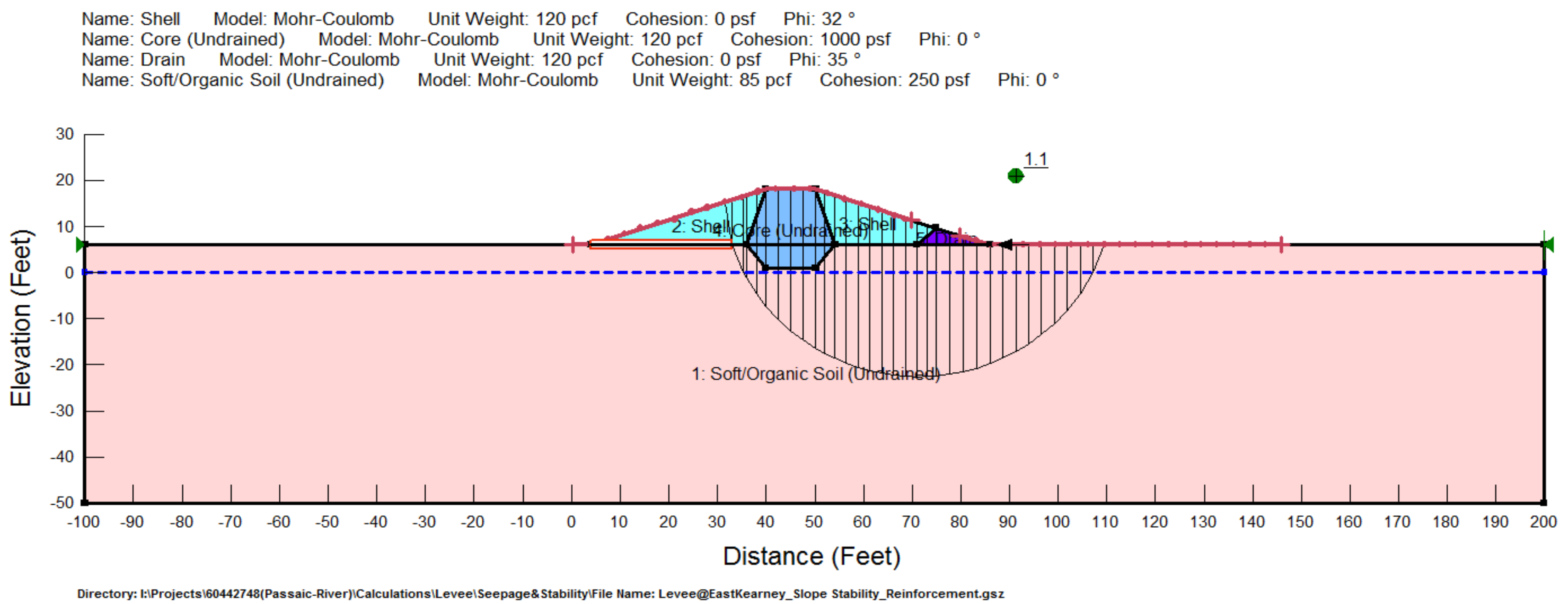
Critical Sliding Surface and Stability FOS for End of Construction (12' High Levee)

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

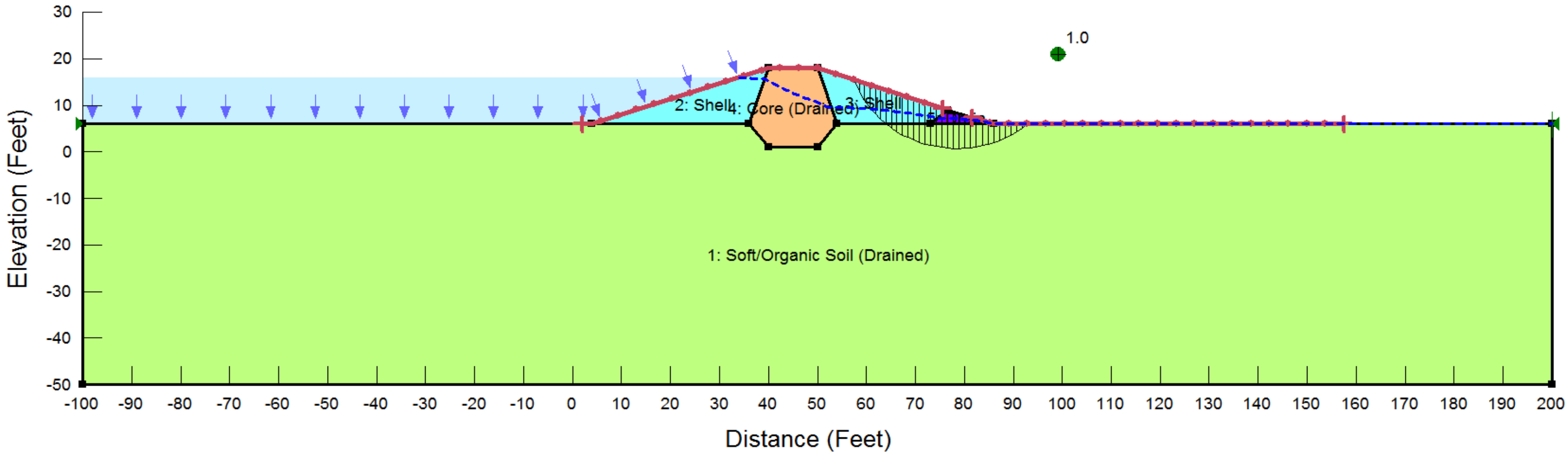


Critical Sliding Surface and Stability FOS for End of Construction using Fabric (12' High Levee)

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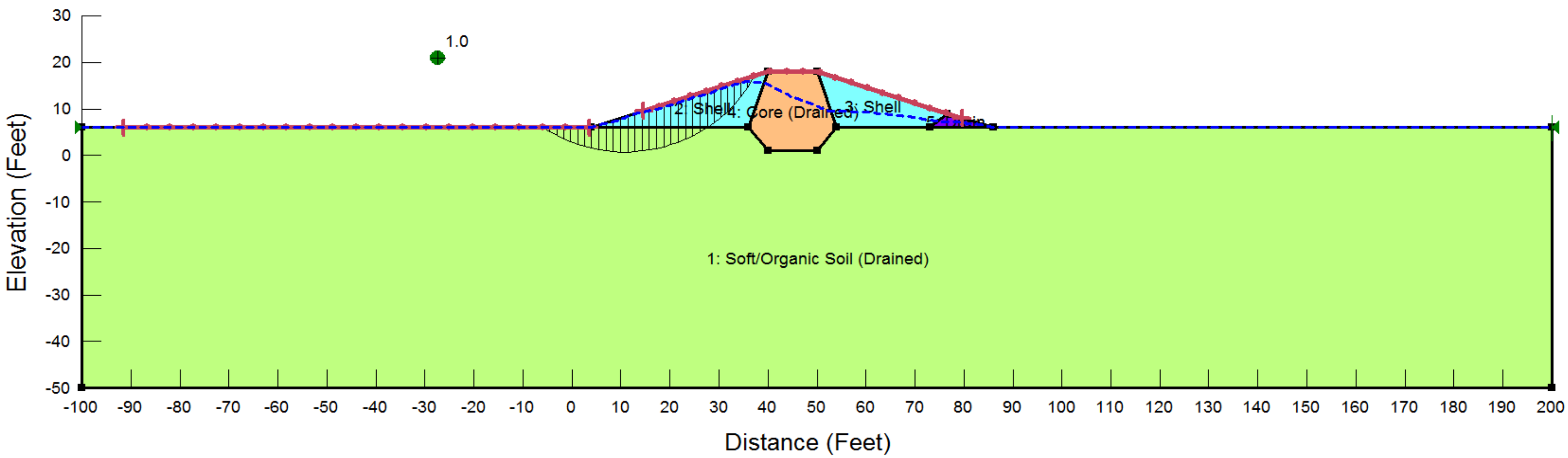
Critical Sliding Surface and Stability FOS for Steady Seepage (12' High Levee)

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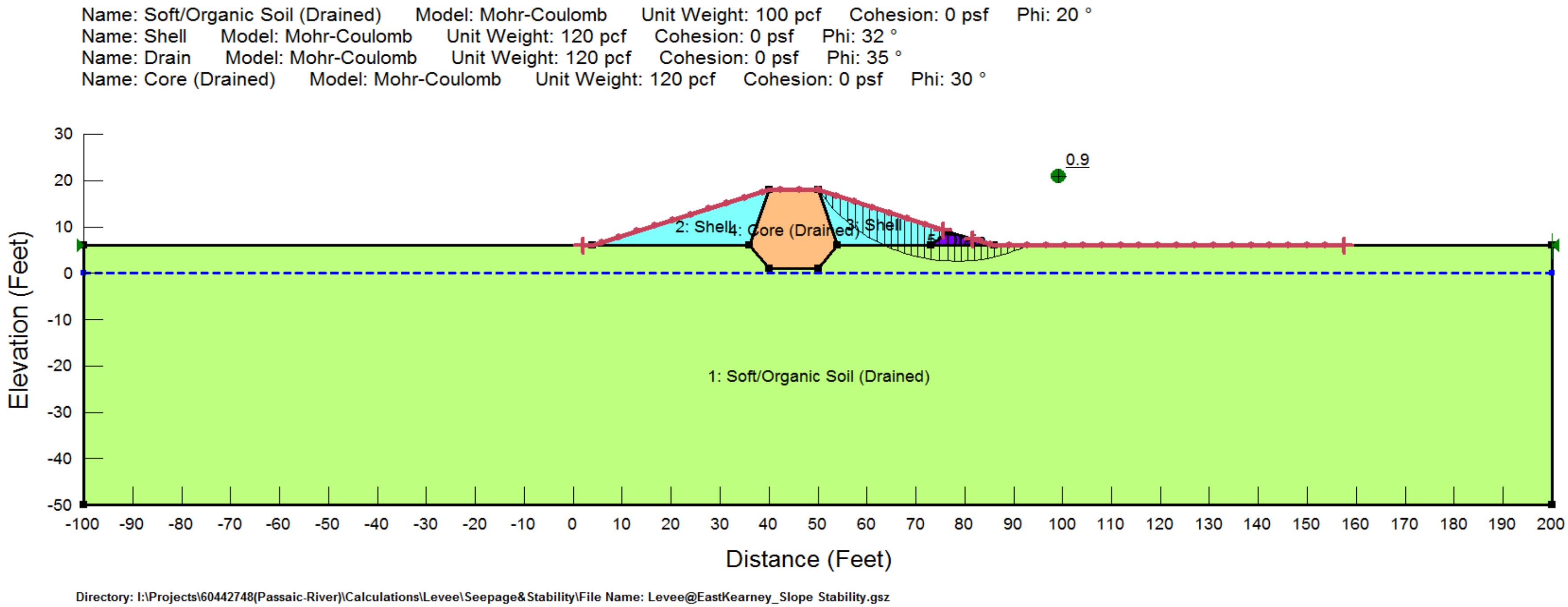
Critical Sliding Surface and Stability FOS for Rapid Drawdown (12' High Levee)

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

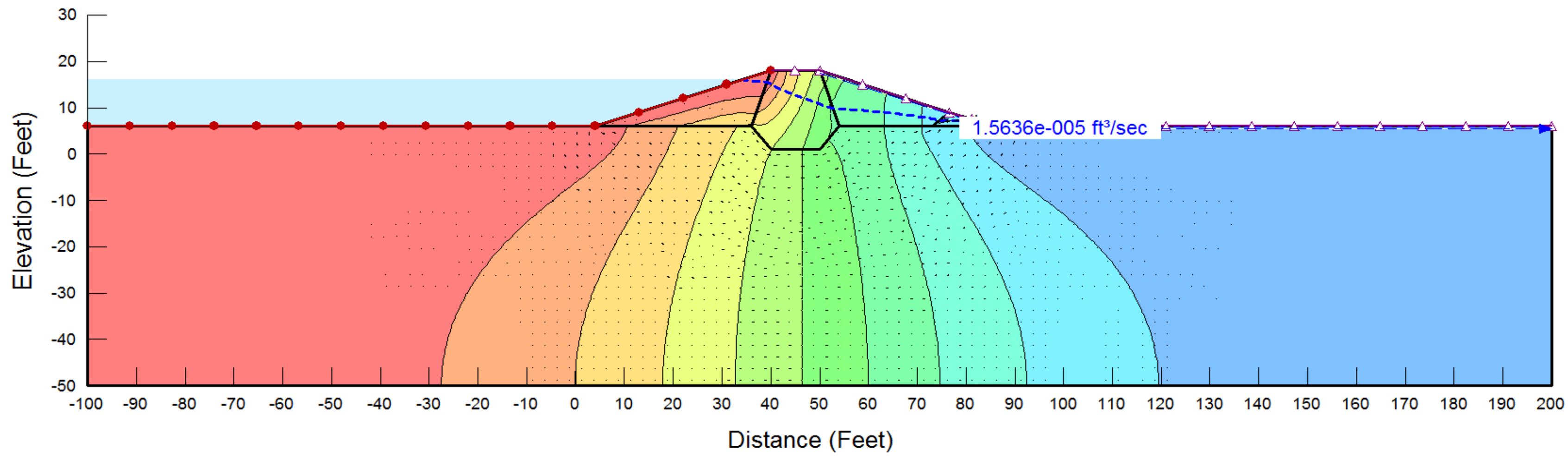


Critical Sliding Surface and Stability FOS for Seismic Condition (12' High Levee)

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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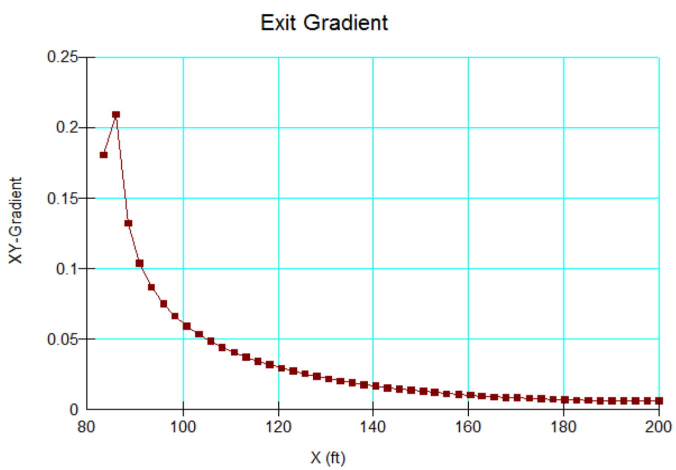
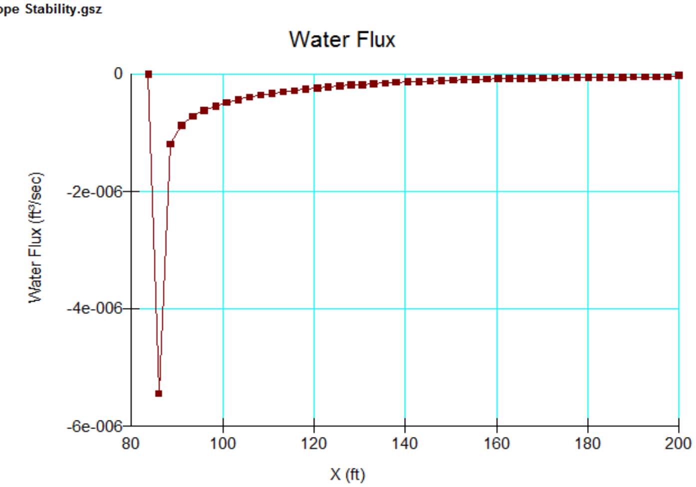


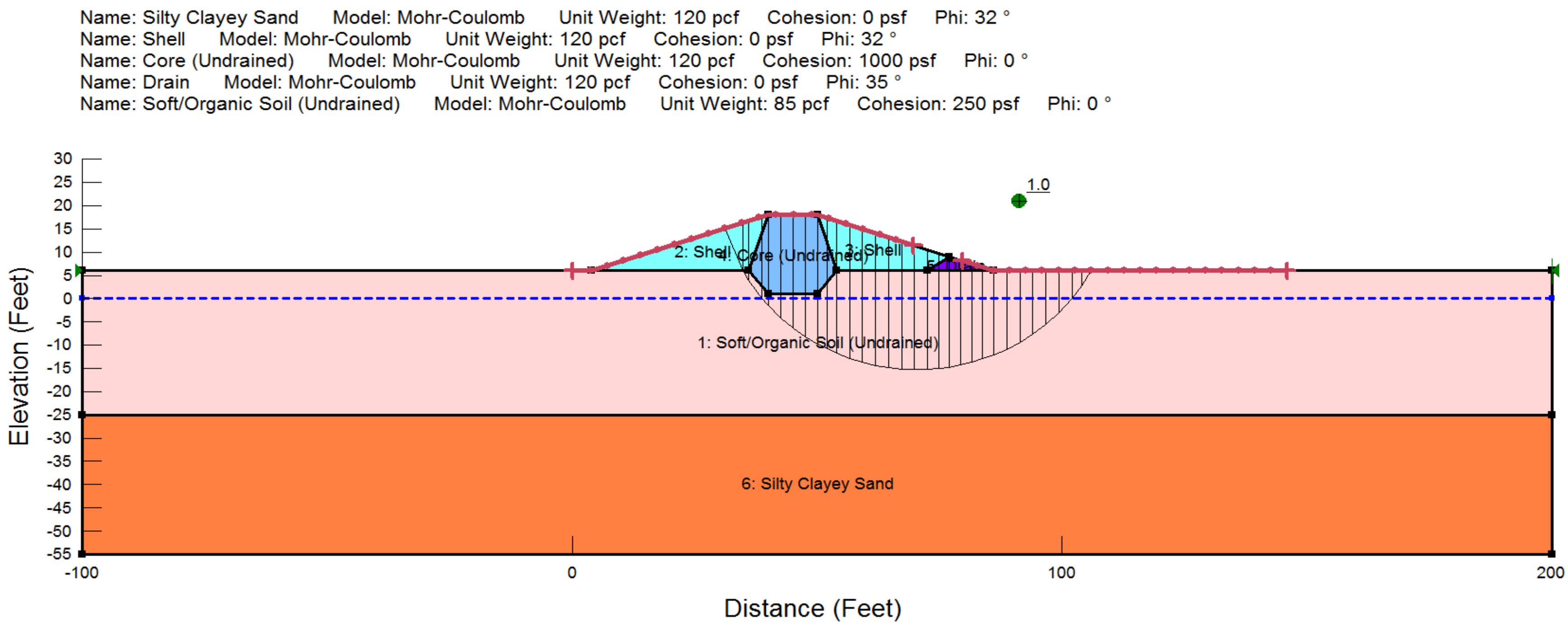


Steady-State Seepage with Flood Pool at El. 16 ft (12' High Levee)

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



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

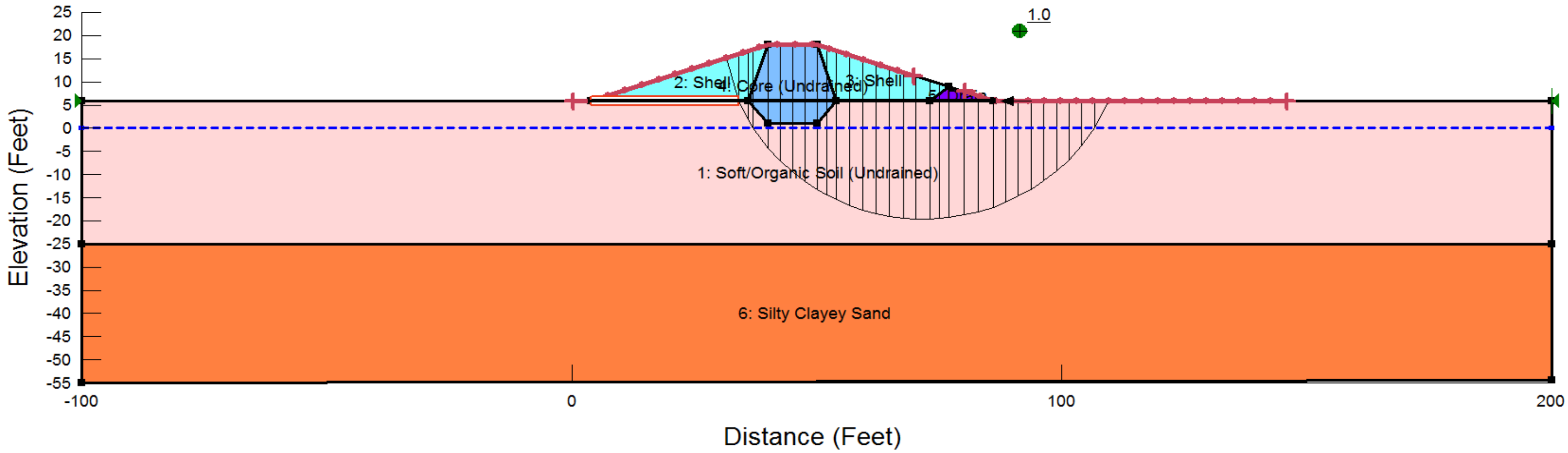
same FOS value was estimated for both Upstream and Downstream Slopes

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)

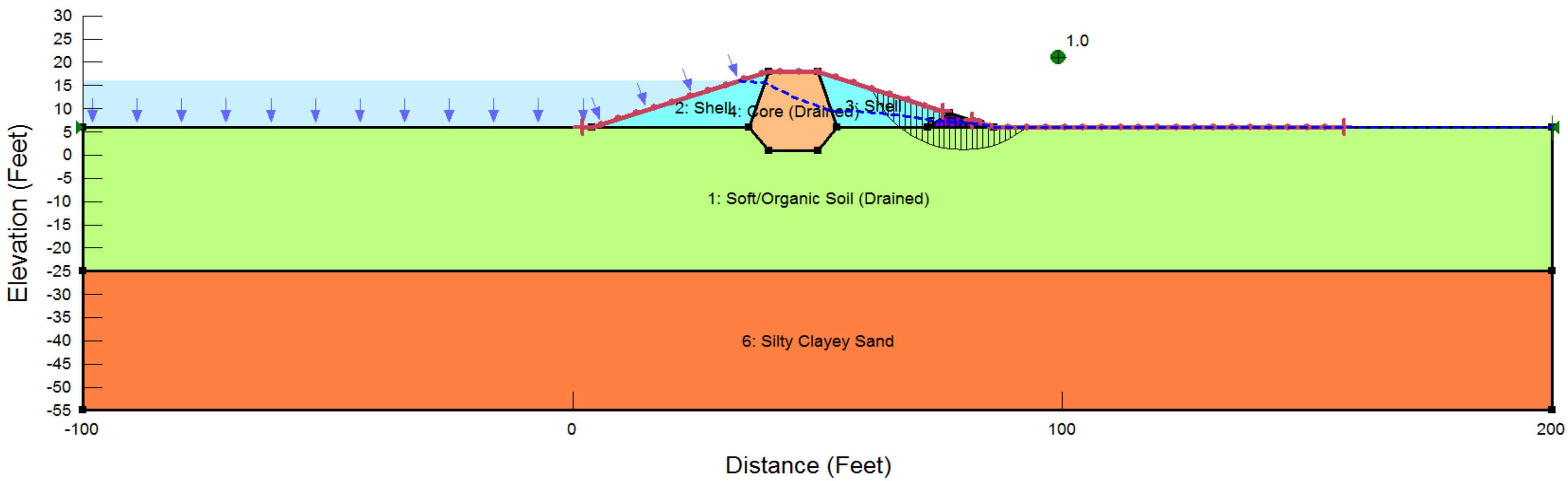
same FOS value was estimated for both Upstream and Downstream Slopes

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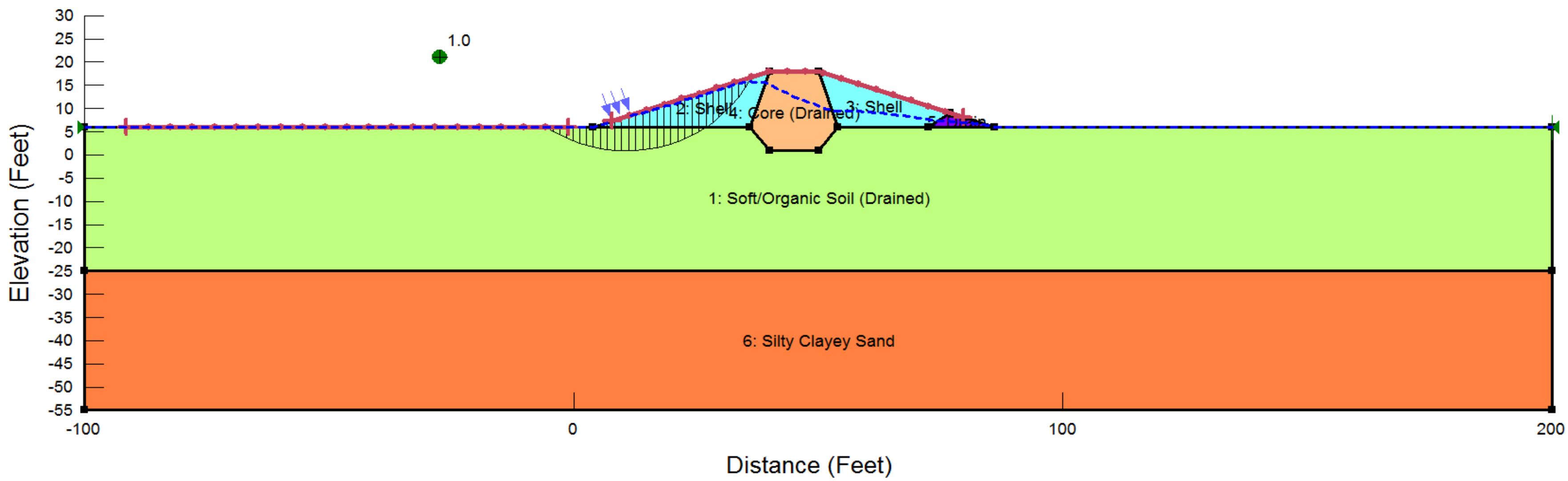
Critical Sliding Surface and Stability FOS for Steady Seepage (12' High Levee)

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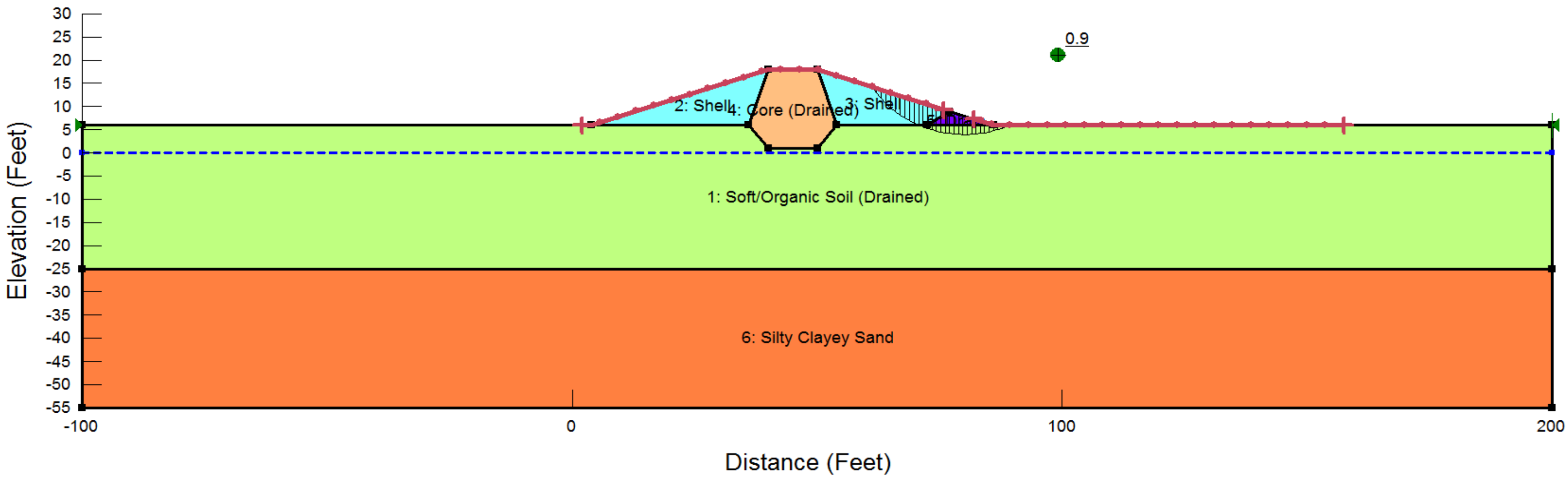
Critical Sliding Surface and Stability FOS for Rapid Drawdown (12' High Levee)

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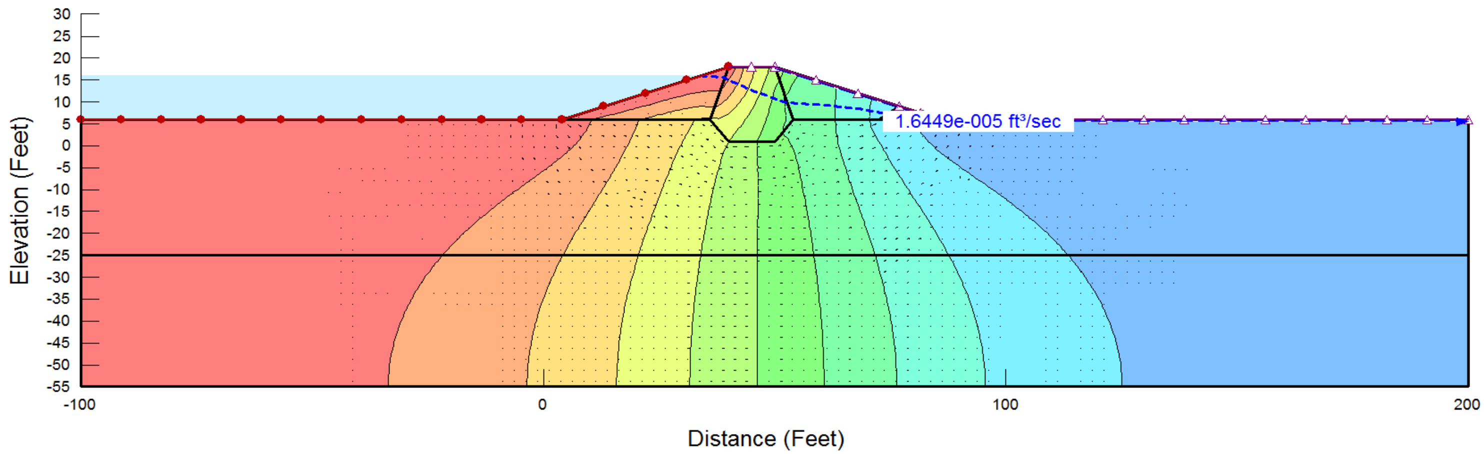
Critical Sliding Surface and Stability FOS for Seismic Condition (12' High Levee)

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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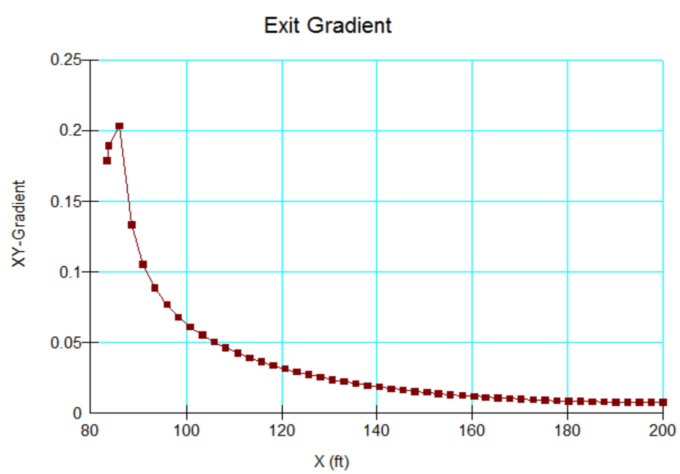
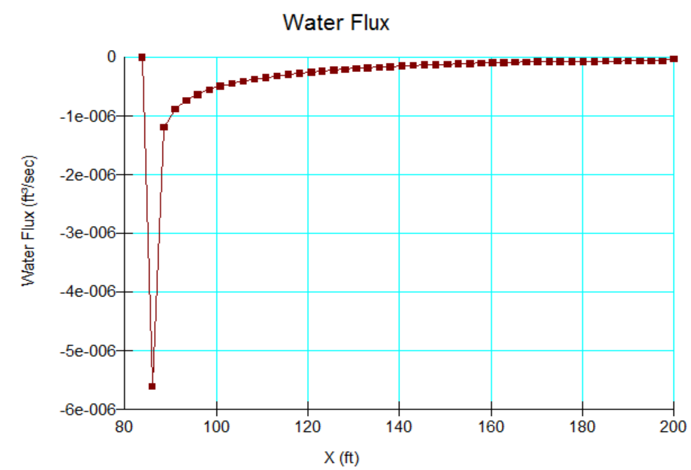




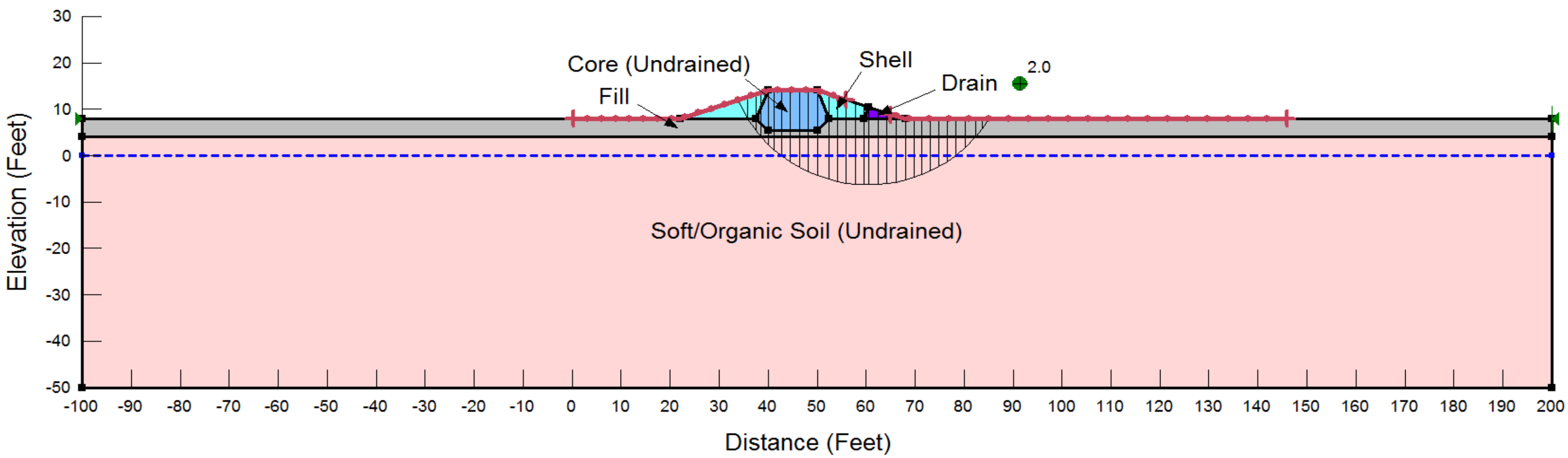
Steady-State Seepage with Flood Pool at El. 16 ft (12' High Levee)

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 °



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

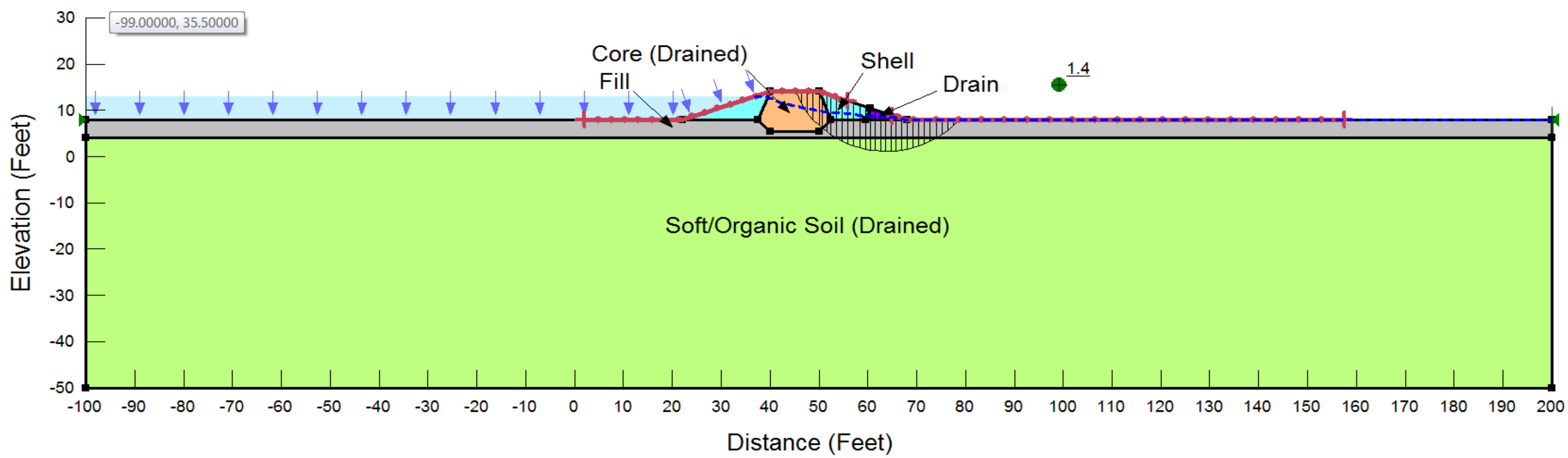
same FOS value was estimated for both Upstream and Downstream Slopes

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 °



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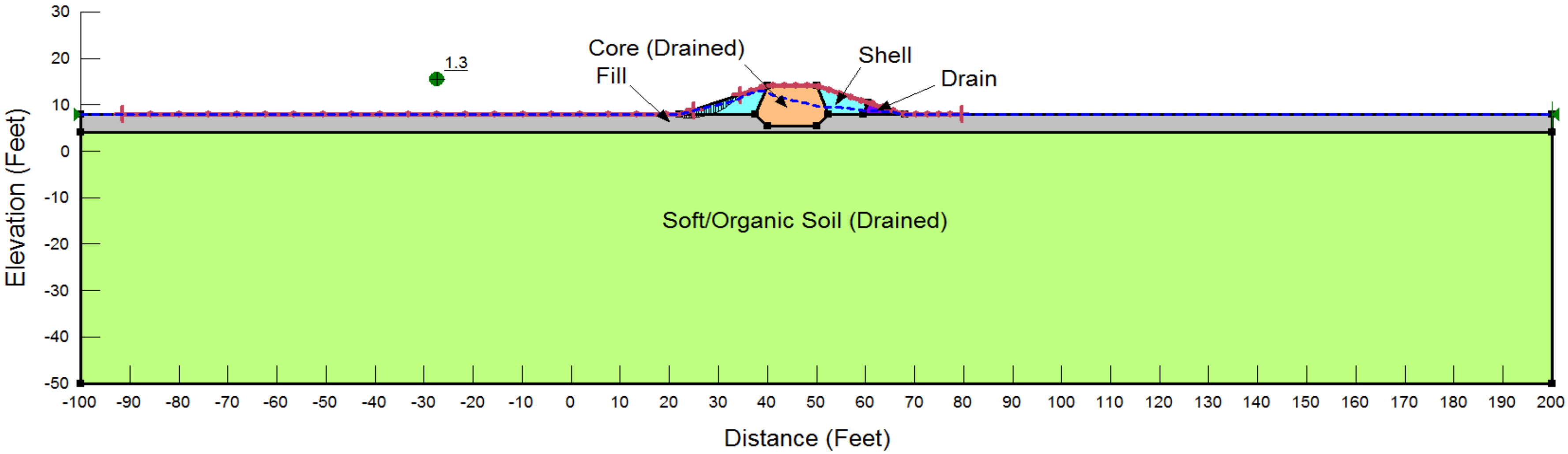
Critical Sliding Surface and Stability FOS for Steady Seepage (6' High Levee)

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 °



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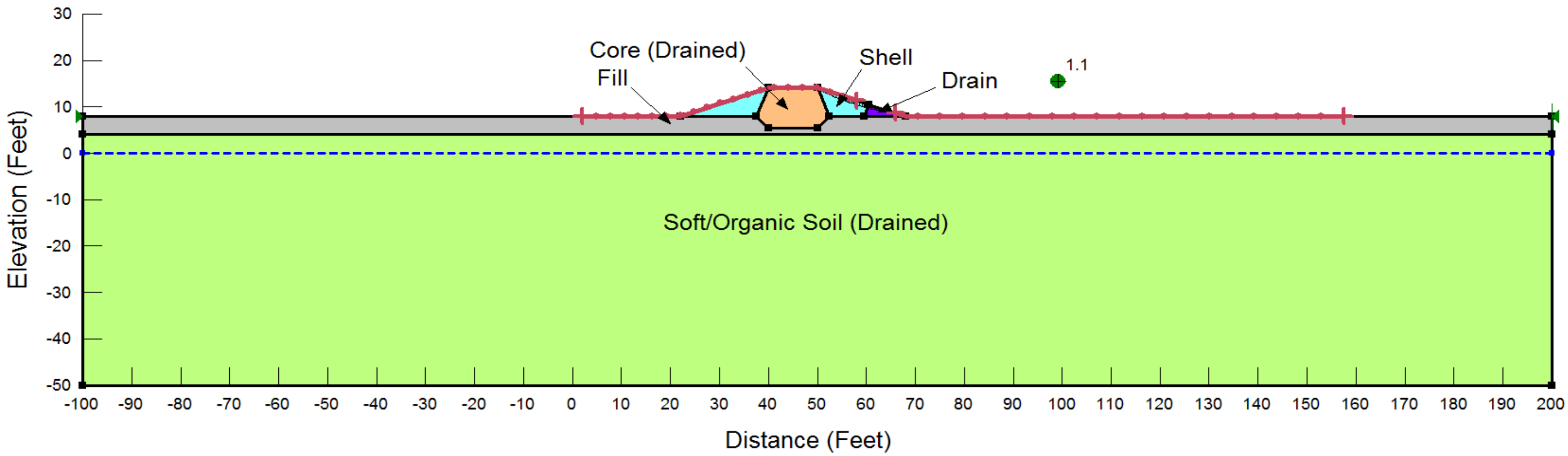
Critical Sliding Surface and Stability FOS for Rapid Drawdown (6' High Levee)

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 °



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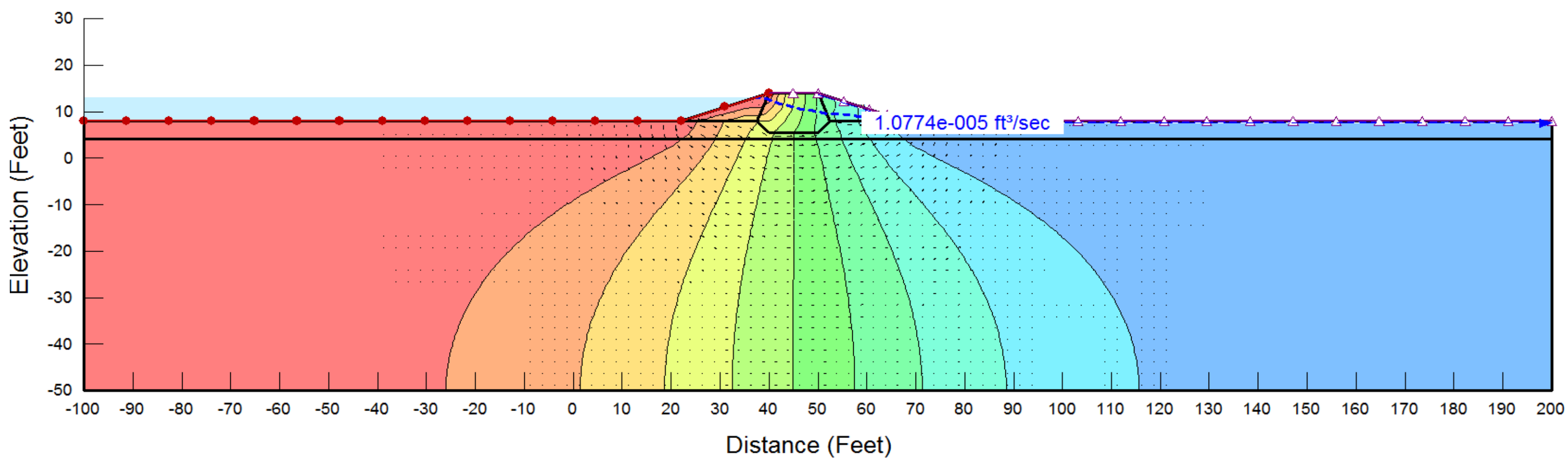
Critical Sliding Surface and Stability FOS for Seismic Condition (6' High Levee)

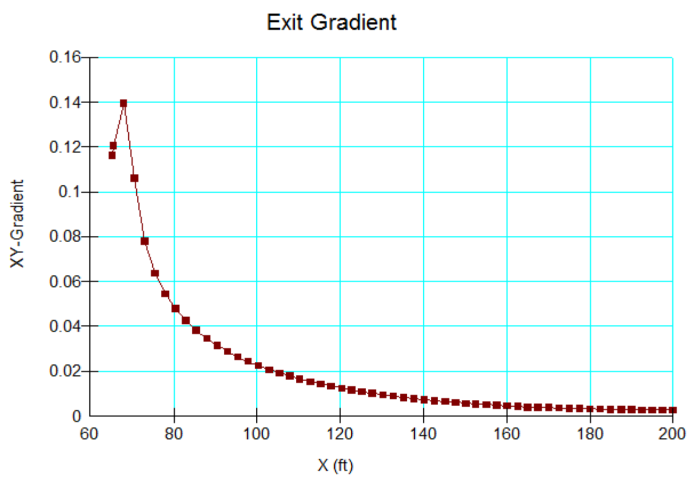
Seismic Loading (Pseudo-Static Coefficient of 0.21)

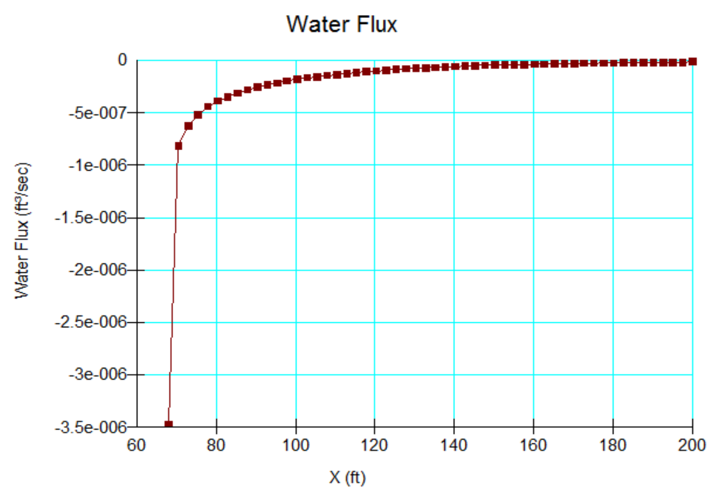
SLOPE STABILITY OF LEVEE ALTERNATIVE **East Kearny Segment** **Critical Sliding Surface and Stability FOS**		
AECOM		
DR BY: MS	SCALE: N.T.S	PROJECT NO: 60442748
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: A.16

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

Groundwater: Depth, ft 6.0 Elev., ft 2.0 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
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 (%)
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
Groundwater: Depth, ft 6.0 Elev., ft 1.0 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)	$C\sigma$	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
Groundwater: Depth, ft 9.0 Elev., ft 0.0 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
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
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[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)	$C\sigma$	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
Groundwater: Depth, ft 8.9 Elev., ft 0.0 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)	$C\sigma$	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
Groundwater: Depth, ft 8.0 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
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

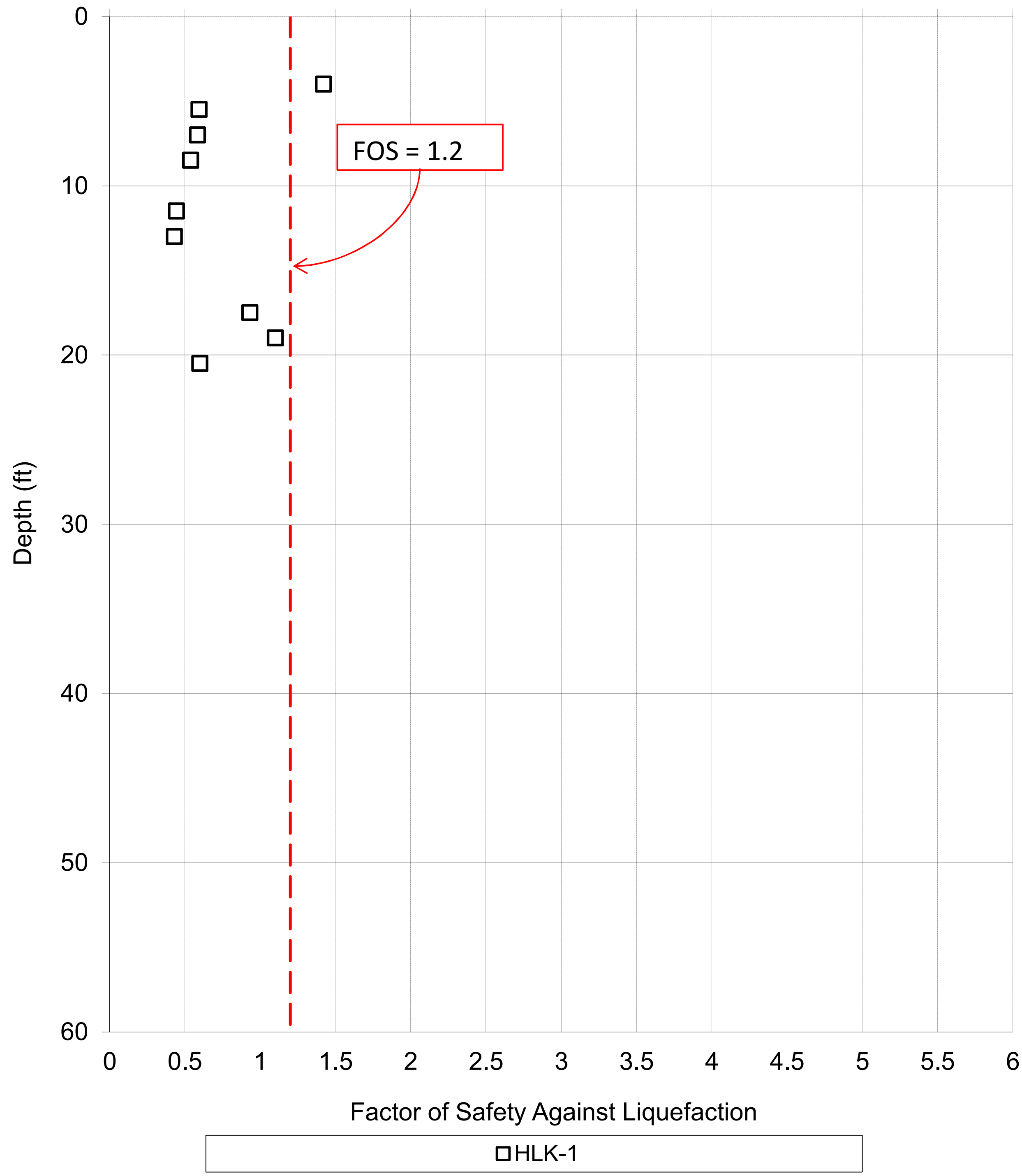
SPT Hammer Weight, lbs 140 Drop, in 30 Type SH Liners No
Drilling Method MUD ROTARY
Groundwater: Depth, ft 5.0 Elev., ft 1.5 Remarks

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	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 $\alpha_{max}(\sigma_v/\sigma_v')\,r_d$

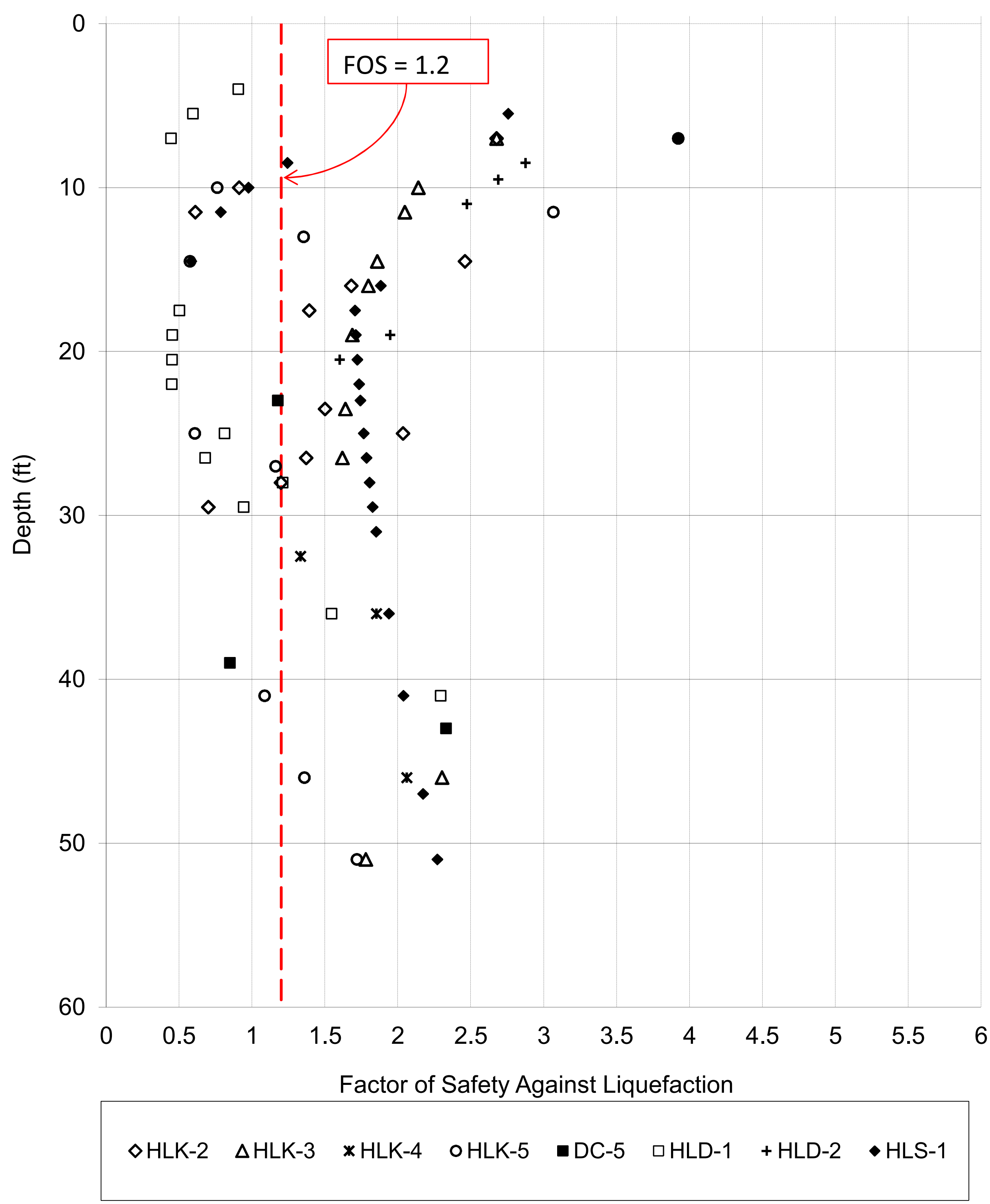


Site Specific Liquefaction Analysis
PASSAIC RIVER
EAST KEARNY, NJ

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

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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

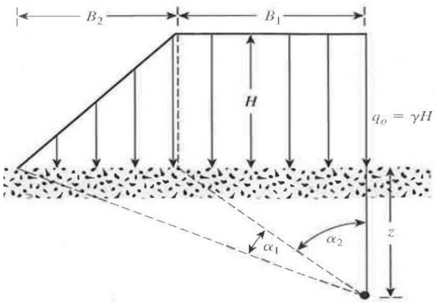
Soil Parameters:

Layer No.	Soil Description	Total Unit Weight	Layer Thickness	Bottom Depth of Layer	Initial Void Ratio, e_0	Compression Index, C_c
		(pcf)	(ft)	(ft)		
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	Mid Depth of Sub-Layer	Initial Overburden Pressure, σ'_0	α_1	α_2	Increase in Overburden Pressure, $\Delta\sigma'_z$	$\sigma'_0 + \Delta\sigma'_z$	C_c	Settlement
	(ft)	(ft)	(psf)	(rad.)	(rad.)	(psf)	(psf)		(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

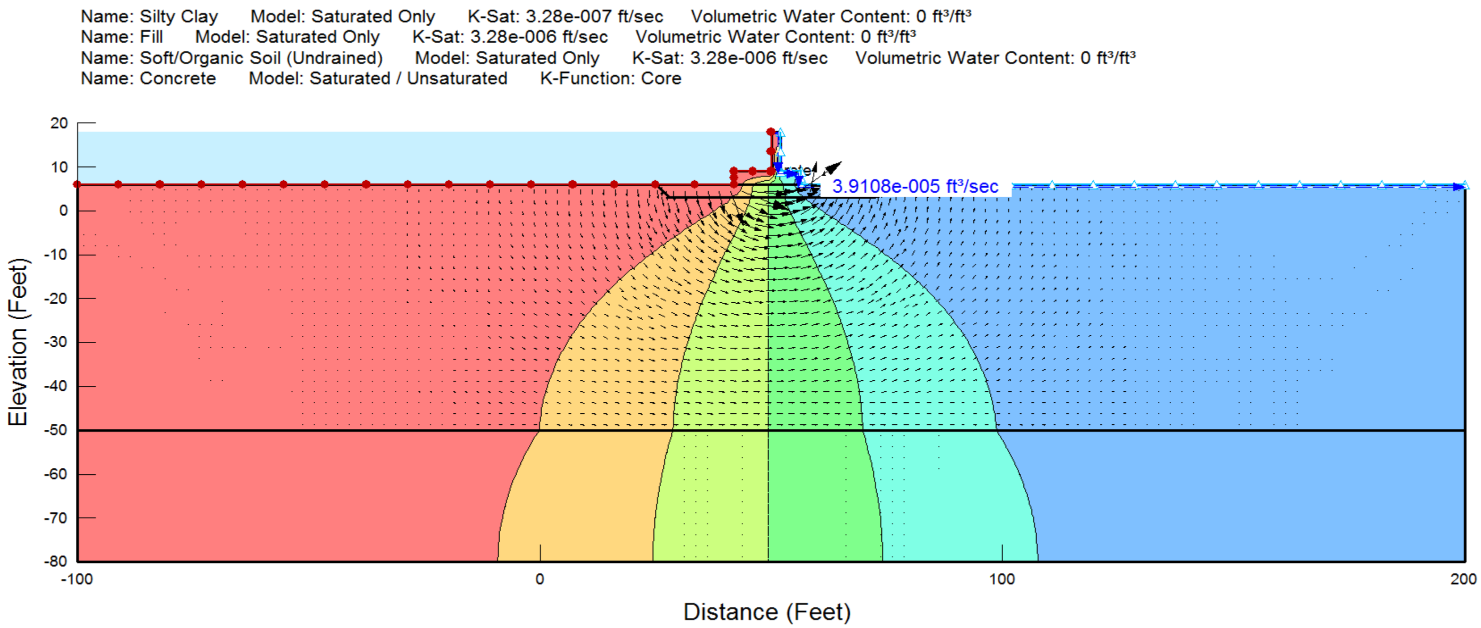
Location	Design Condition	Case	Steady-State Seepage	
			Flow Rate $ft^3/sec/ft$	Exit Gradient
East Kearny	Steady Seepage with Full Flood Steady Seepage with Full Flood	Floodwall without Sheetpile Floodwall with 20' deep Sheetpile	3.911E-05 2.120E-05	0.86 0.16
West Kearny, Newark, Harrison	Steady Seepage with Full Flood Steady Seepage with Full Flood	Floodwall without Sheetpile Floodwall with 20' deep Sheetpile	3.953E-05 2.170E-05	0.86 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.



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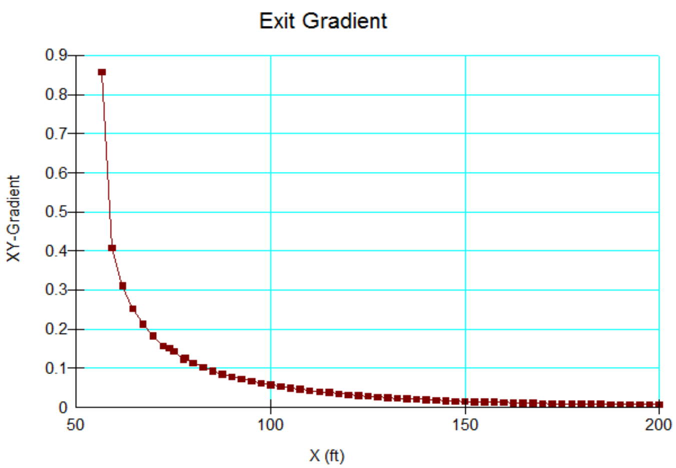
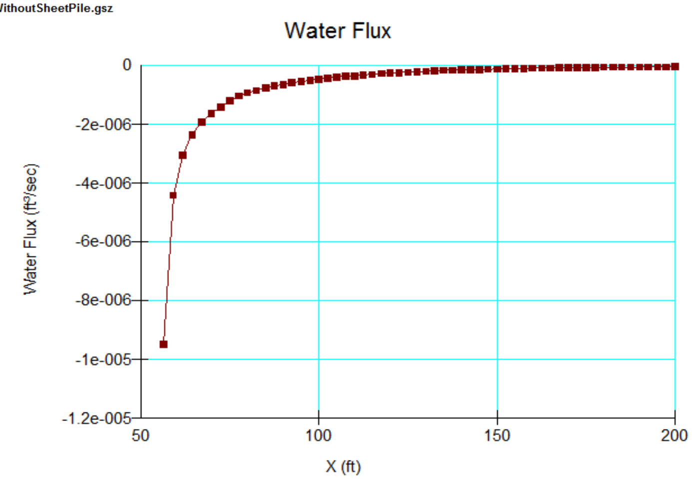




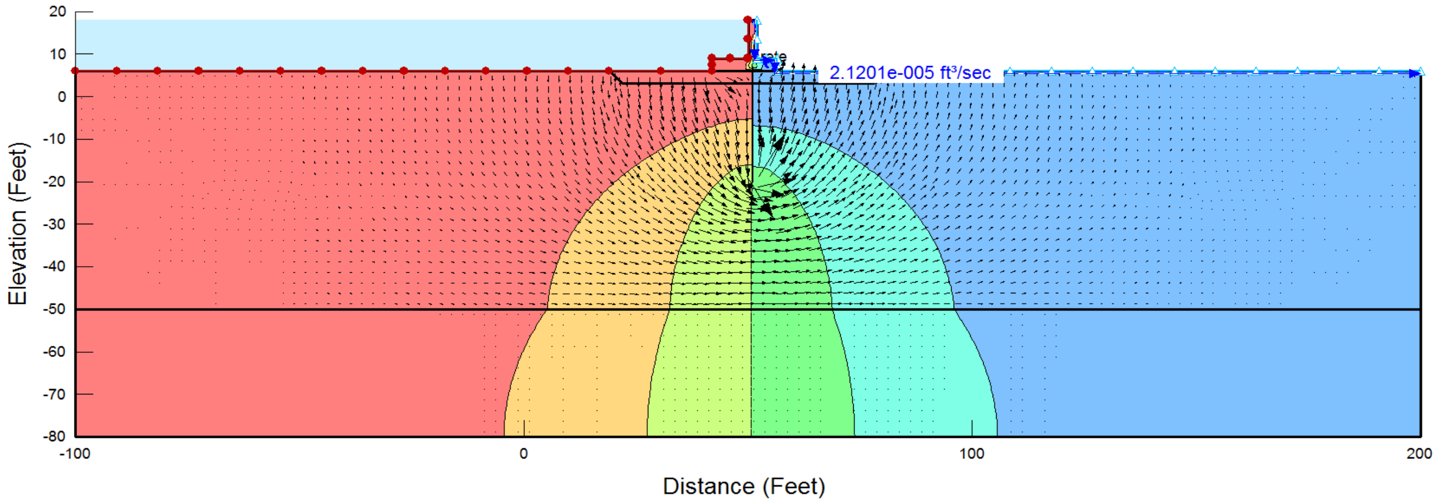
Steady-State Seepage for 12 ft High Floodwall Without Sheetpile (Flood Pool at El. 18 ft)

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

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
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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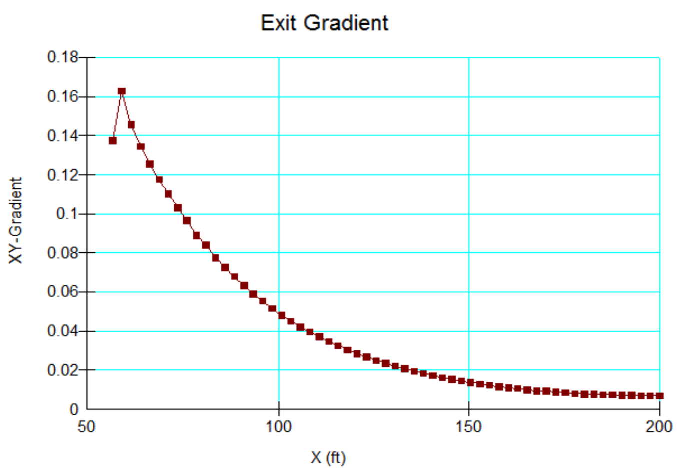
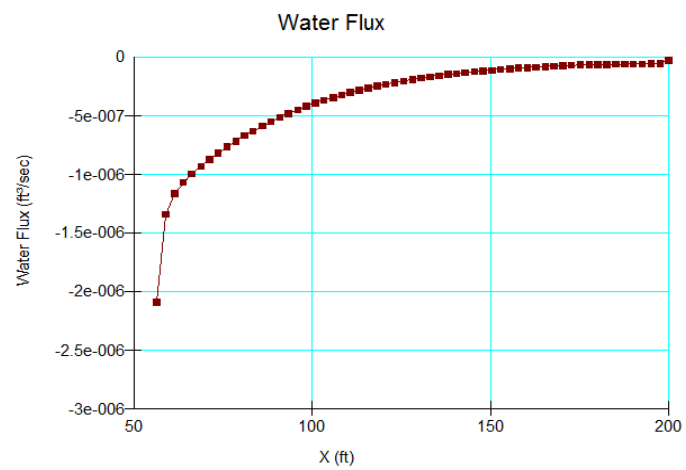
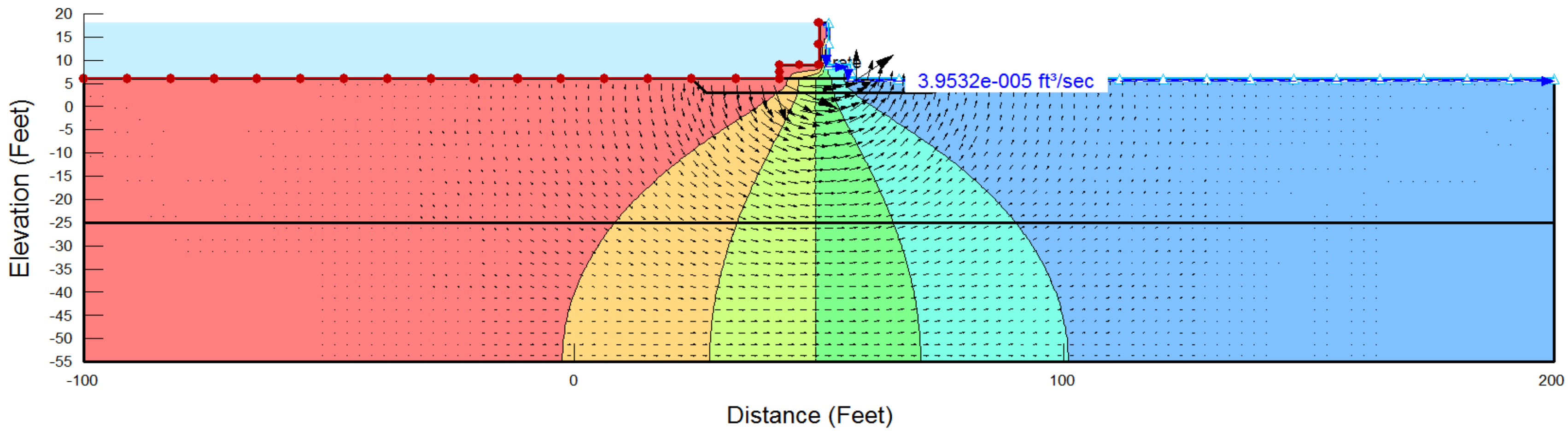




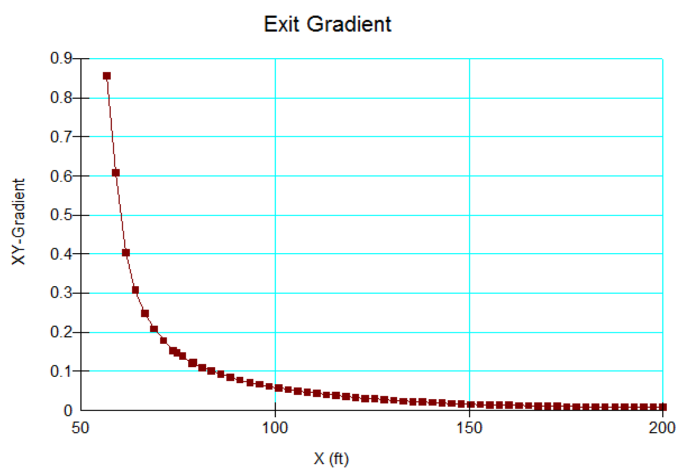
Steady-State Seepage for 12 ft High Floodwall With 20ft Long Sheetpile (Flood Pool at El. 18 ft)

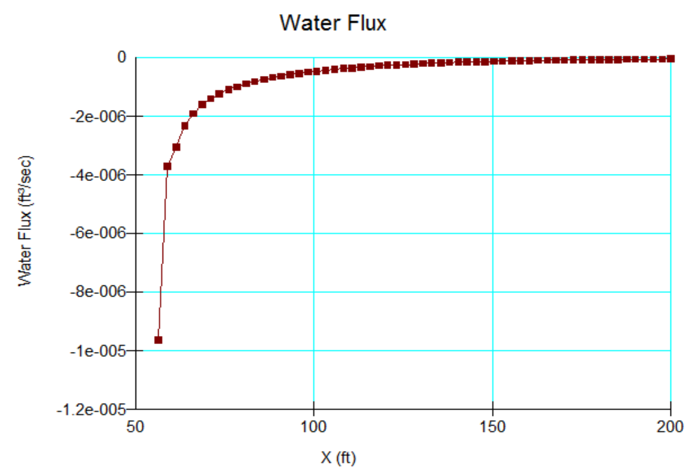
SEEPAGE ANALYSIS OF FLOODWALL WITH SHEETPILE **East Kearny Segment** **Estimated Piezometric Conditions, Flow Rate, and Exit Gradient**		
AECOM		
DR BY: MS	SCALE: N.T.S	PROJECT NO:
CHKD BY: SK	DATE: 12/18/15	FIGURE NO: 2

Name: Silty Clayey Sand Model: Saturated Only K-Sat: 3.28e-006 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



Directory: I:\Projects\60442748(Passaic-River)\Calculations\Floodwall\Seepage\File Name: Floodwall@WestKEarny&Newark&Harrison_Seepage_WithoutSheetPile.gsz

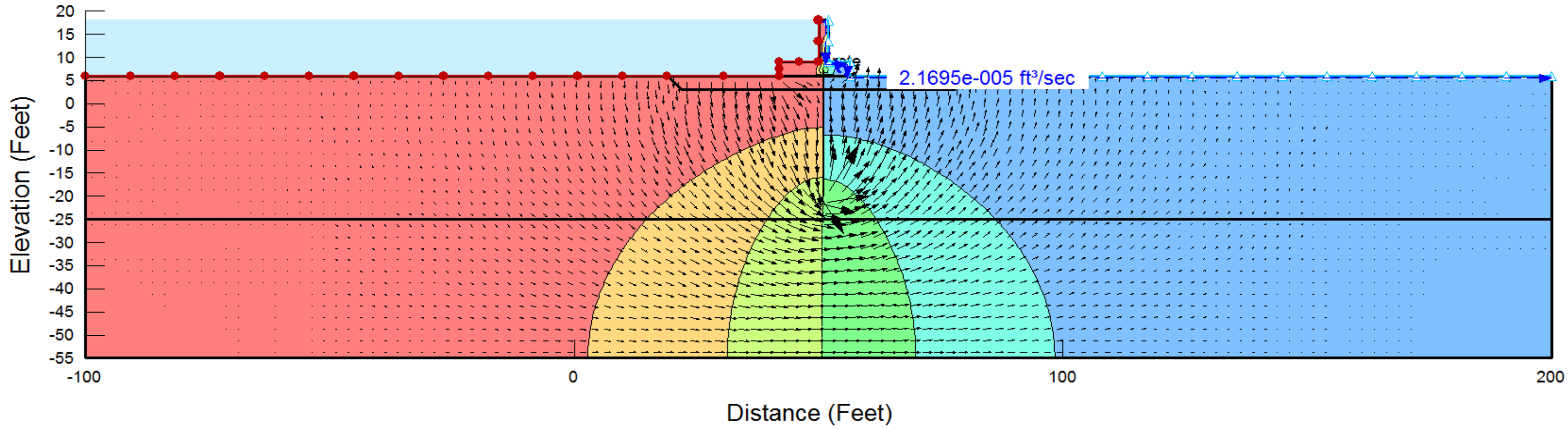




Steady-State Seepage for 12 ft High Floodwall Without Sheetpile (Flood Pool at El. 18 ft)

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

Name: Silty Clayey Sand Model: Saturated Only K-Sat: 3.28e-006 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
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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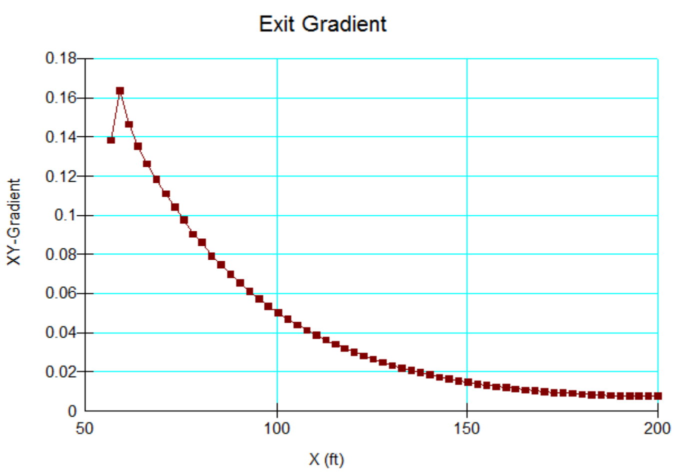
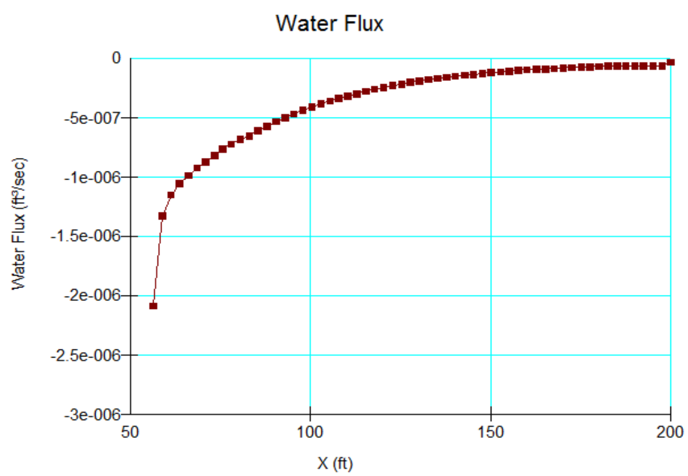


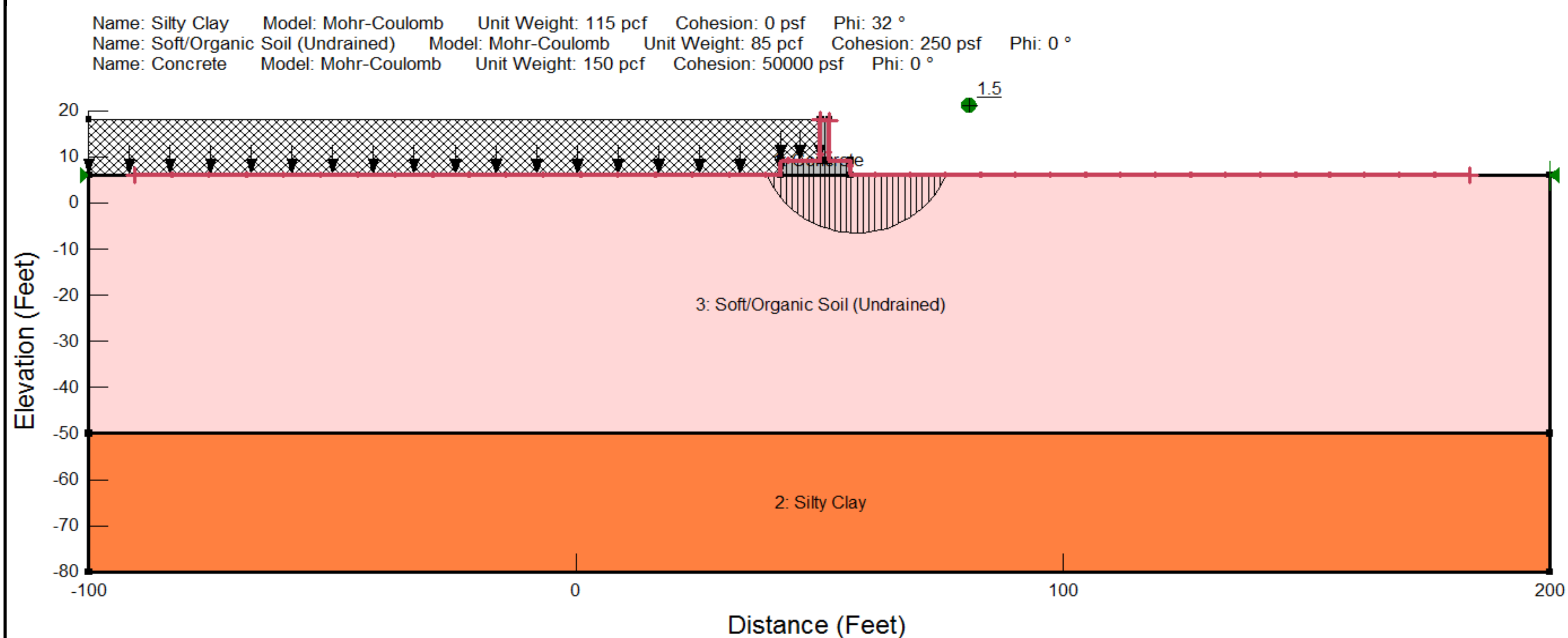


Steady-State Seepage for 12 ft High Floodwall With 20ft Long Sheetpile (Flood Pool at El. 18 ft)

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.

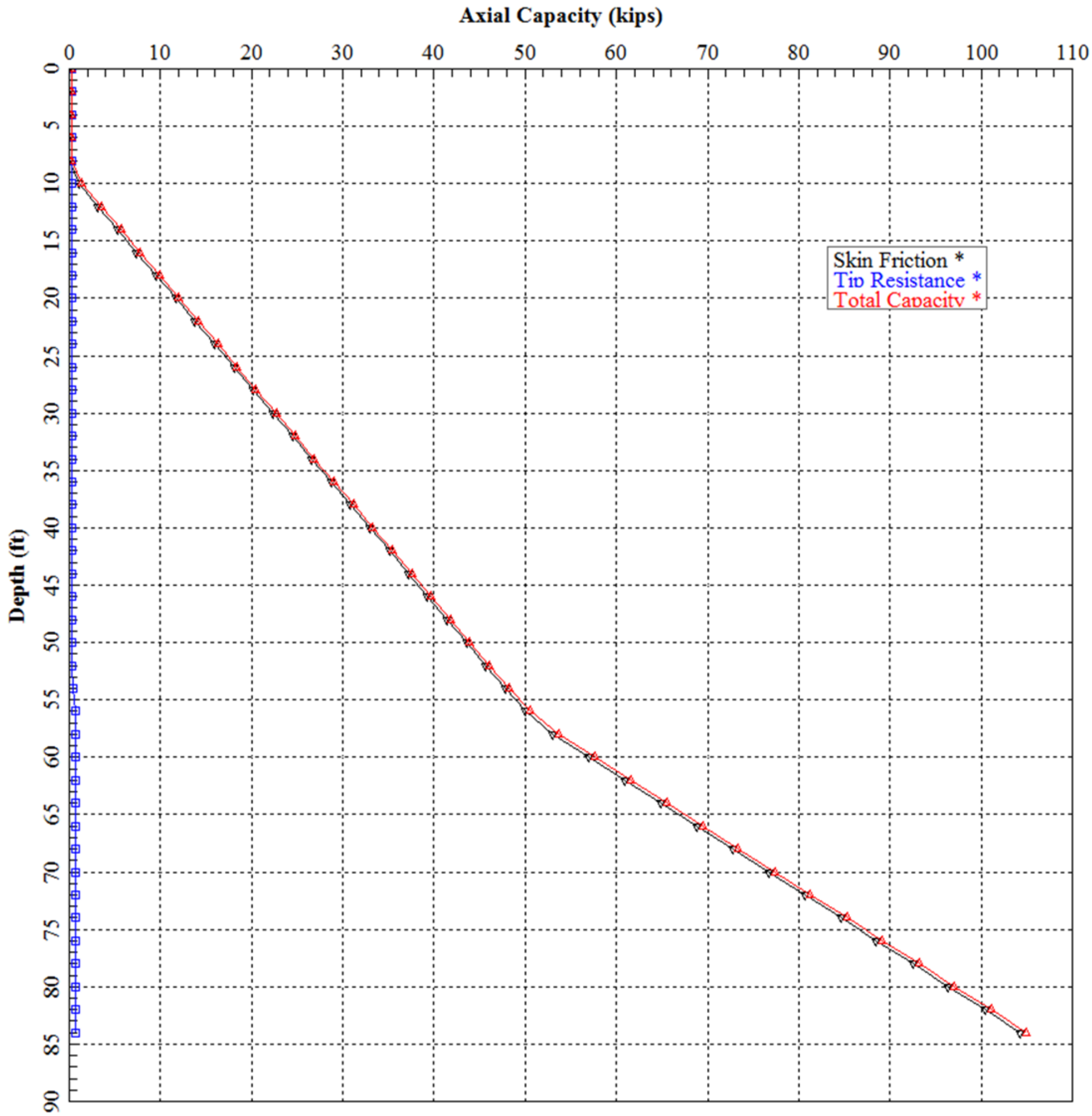

Figure E.1: Skin Friction and Total Capacity Distribution of HP14x73 Pile with Depth (East Kearny Segment)

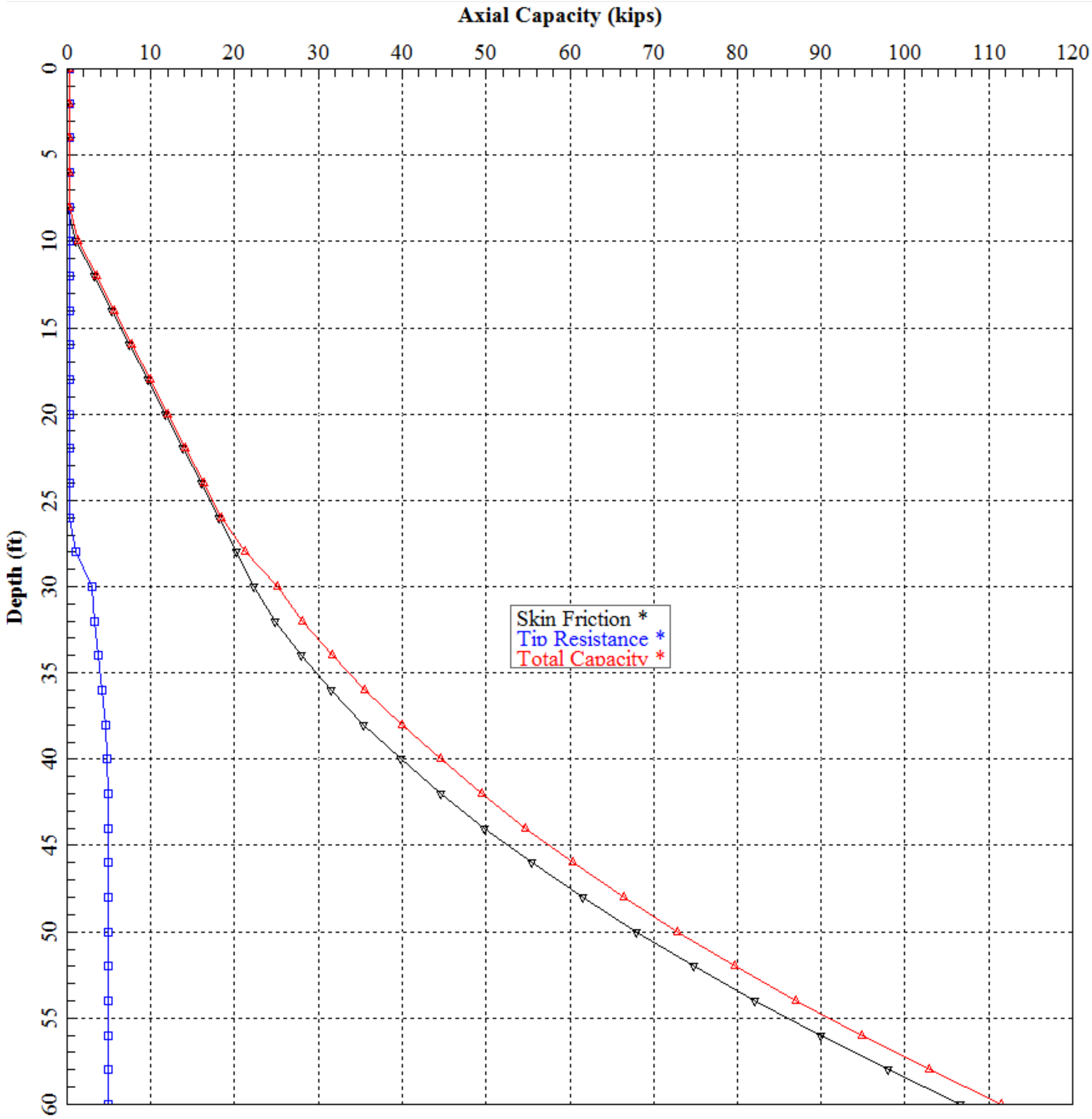

Figure E.2: Skin Friction and Total Capacity Distribution of HP14x73 Pile with Depth (West Kearny, Newark, and Harrison Segment)

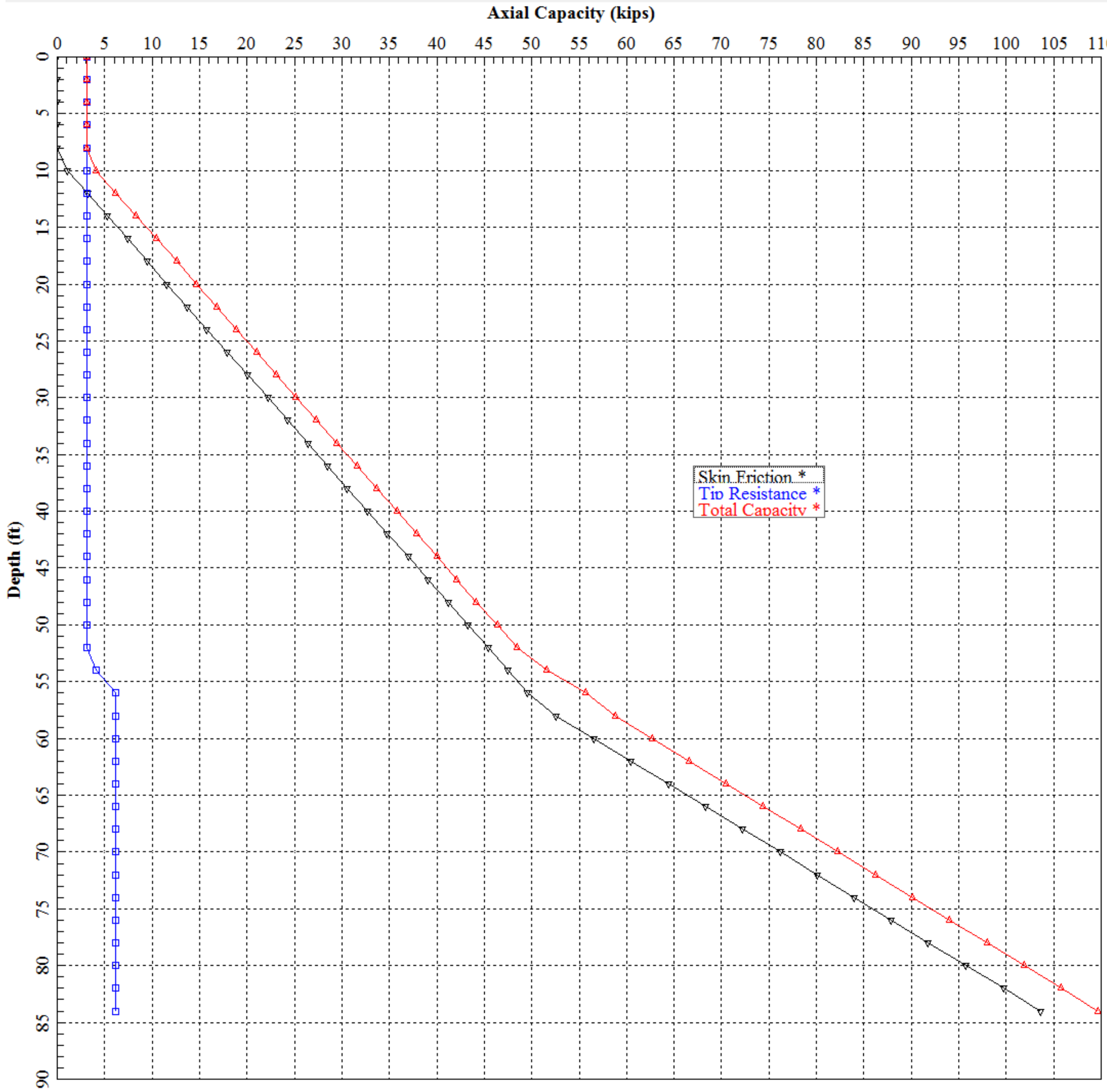

Figure E.3: Skin Friction and Total Capacity Distribution of 14-inch Prestressed Precast Concrete Pile with Depth (East Kearny Segment)

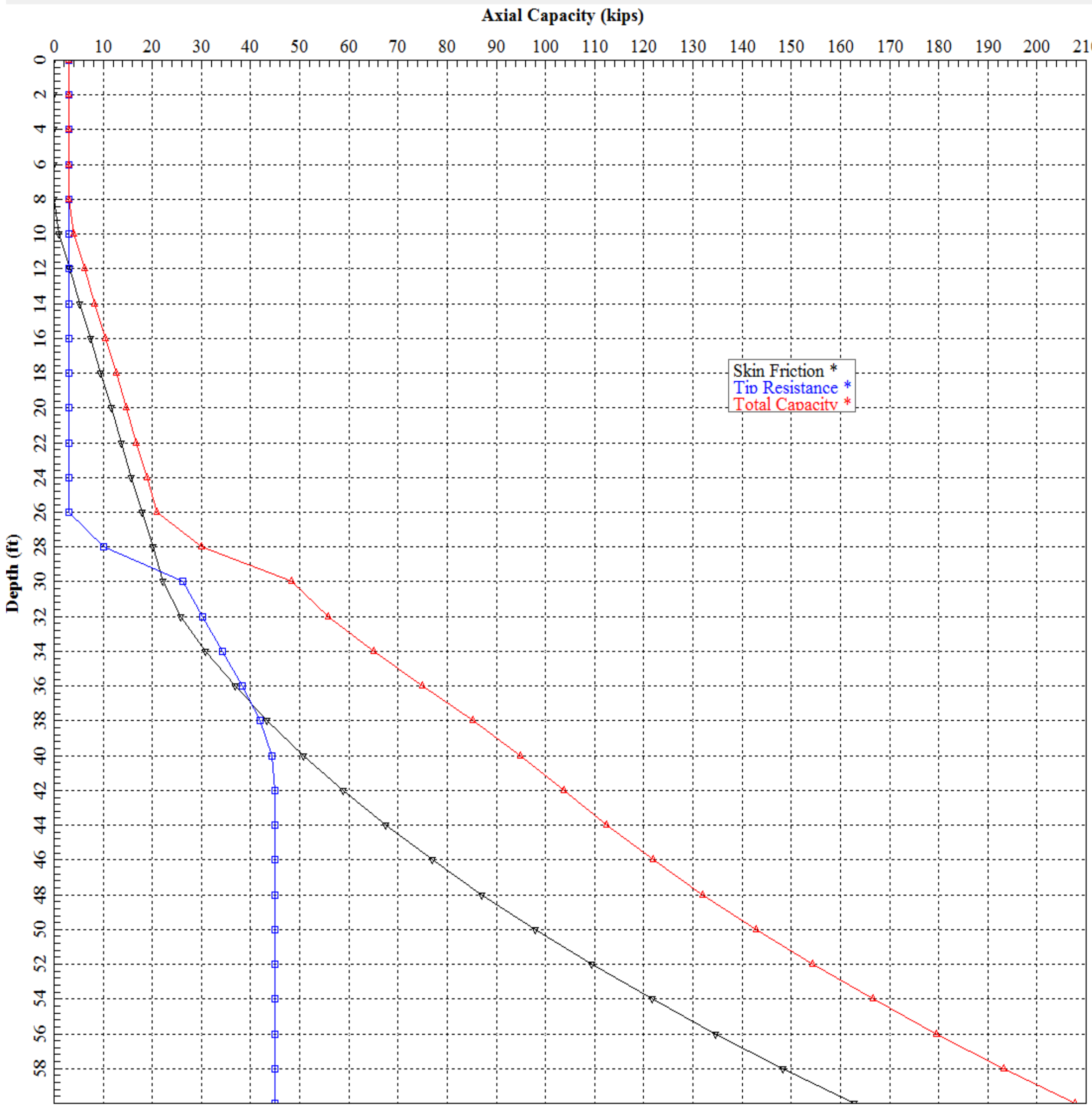

Figure E.4: Skin Friction and Total Capacity Distribution of 14-inch Prestressed Precast Concrete Pile with Depth (West Kearny, Newark, and Harrison Segment)

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

Soil Weight above Socket: 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
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.

Rebar Number: 24

Cased Length Capacity

Steel Strength (Comp.):	312	tons
Grout Strength :	137	tons
Total :	**448**	tons

Steel Strength (Tension): 365 tons

Rock Socket (Uncased Length) Capacity

Allowable Transfer Load :	11	tons
Steel Strength (Comp.):	125	tons
Grout Strength :	127	tons
Total :	**263**	tons

Steel Strength (Tension): 157 tons

Total Structural Capacity (FHWA) : 263 tons

Figure E.6: Estimate of Capacity of 12-inch Rock-Sockets