Flood Risk Management Feasibility Study

Peckman River Basin
New Jersey

Draft Integrated Feasibility Report and Environmental Assessment

Appendix C3: Geotechnical Engineering

US Army Corps of Engineers®
New York District

October 2019

Table of Contents

1.0 INTRODUCTION 1
2.0 LOCAL GEOLOGY 2
3.0 SUBSURFACE EXPLORATION 6
4.0 SUBSURFACE CONDITIONS 6
5.0 GEOTECHNICAL ANALYSES 8
5.1 LEVEES 10
5.1.1 SEEPAGE ANALYSIS 10
5.1.2 SLOPE STABILITY 12
5.1.3 SETTLEMENT 13
5.2 DIVERSION CULVERT 13
5.2.1 BEARING CAPACITY 14
5.2.3 SETTLEMENT 14
5.3 FLOODWALL AND RETAINING WALL STABILITY ANALYSIS 14
5.4 FLOODWALL SEEPAGE 14
6.0 CONCLUSIONS 15
7.0 ATTACHMENTS 15

List of Tables

Table 1: Summary of Stratigraphic Unit Properties.....10

Table 2: Permeability Constants.....11

Table 3: Slope Stability Soil Parameters.....12

List of Figures

Figure 1: Google Earth View of the Project Area.....1

Figure 2: Map of Soil Borings Performed.....3

Figure 3: NJGS Geological Paterson and Orange Quadrangle Maps and Cross-section for the Area of Interest South of Patterson.....4

Figure 4: NJGS Surficial Deposits with Bedrock Contours.....5

Figure 5: Google Earth Aerial of Project Site and Soil Boring Locations.....9

1.0 INTRODUCTION

The objective of the study is to determine the feasibility of constructing various features which include channel improvements, a diversion culvert, levees, and floodwalls proposed for the Peckman River Flood Risk Management Project. The overall project area evaluated in January 2012 is shown in Figure 1.

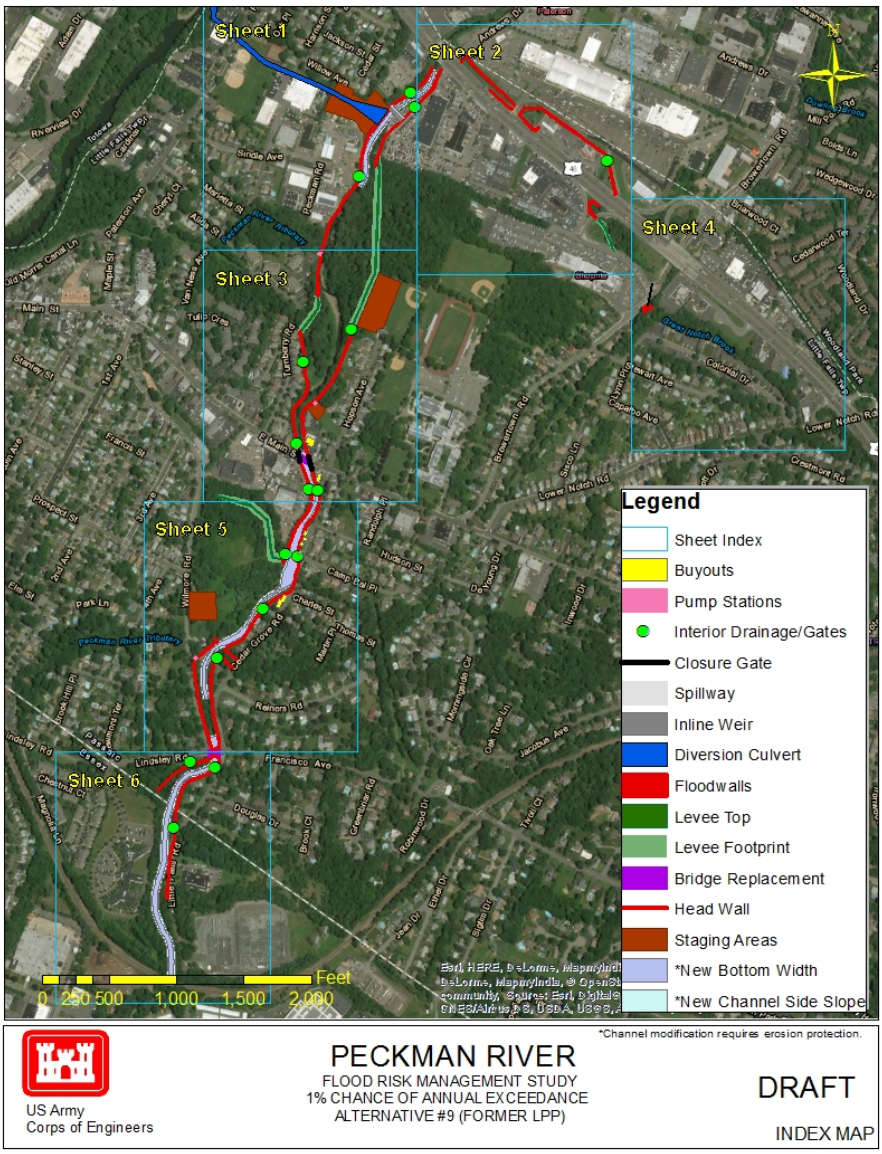


Figure 1: Google Earth View of the Project Area.

2.0 LOCAL GEOLOGY

The Peckman River Project area is located solely in the Paterson Quadrangle. NJGS mapping indicates Stream Terrace Deposits consisting of silt, clay, and fine sand underlying the project area. The uppermost surface (Stream Terrace Deposits) is described as moderately to well sorted, stratified; brown, yellowish-brown, reddish-brown; sand, pebble-to-cobble gravel, with minor silt. It is estimated to be as much as 20-foot thick and forms terraces with surfaces slightly above the modern floodplain along Peckman River and Preakness Brook. The lowest surface is defined as silt, clay, and/or fine sand up to 50-foot in thickness deposited on the lake bottom during the Great Notch Stage. Although the majority of the project area possesses the previous properties, the far eastern and western portions indicate two different soil types. The eastern edge is identified as Rahway Till, Yellow Phase, although it is discontinuous and generally less than 20-foot thick. This Till is described as silty sand, sandy silt, and silt with some to many subangular and subrounded pebbles and cobbles. Along the western project section the same soil properties are encountered with only the continuity changing (more continuous in western section). NJGS geologic mapping indicates Orange Mountain basalt within the western areas and Feltsville sandstone in all eastern areas. See attached Figures 3 and 4 with the project area of interest highlighted which combine the NJGS Bedrock Geologic and Surficial Geology Maps of the Paterson and Orange quadrangles, respectively. Figure 2, below, shows the 23 soil borings that were performed in the project area.

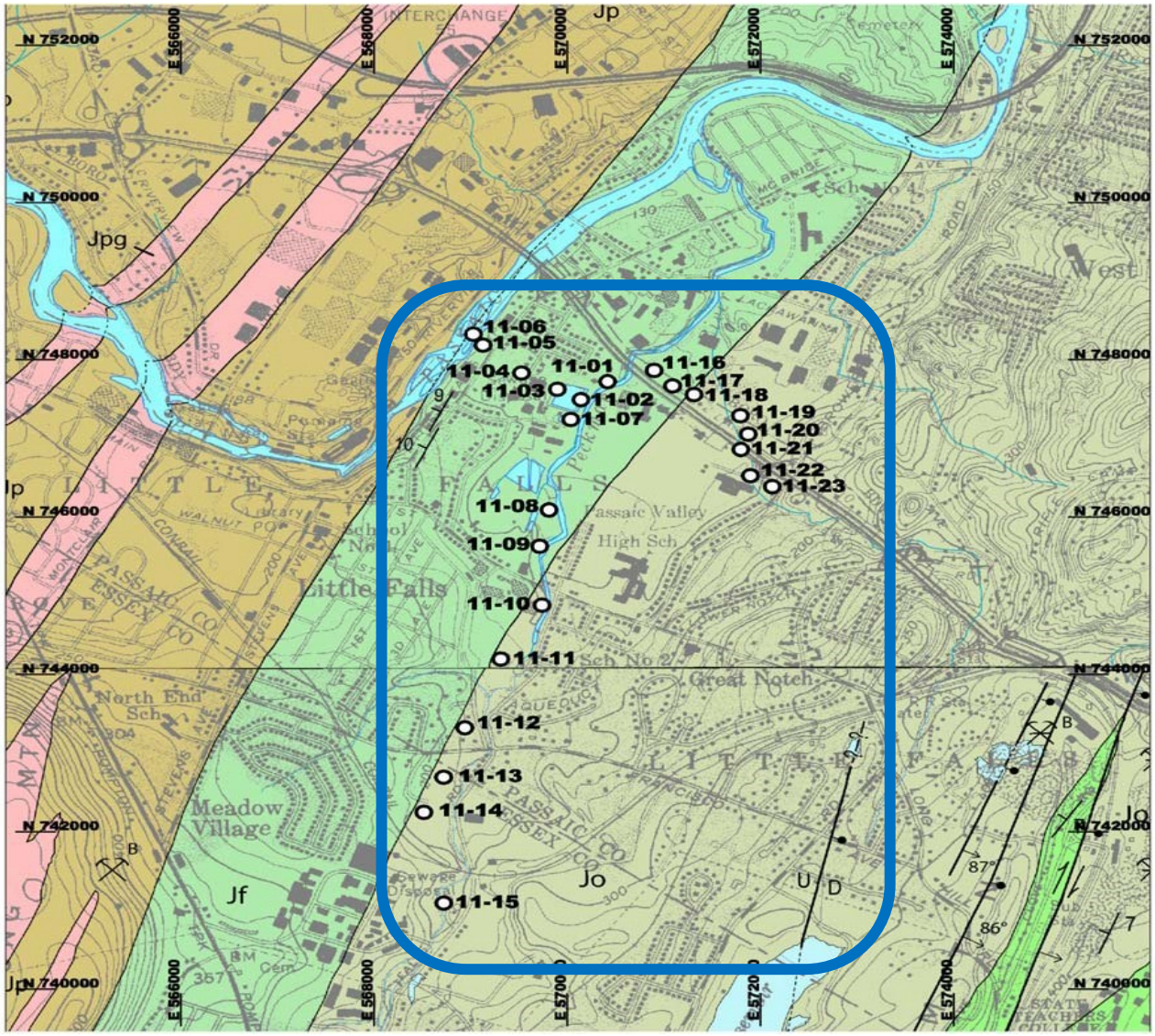


Figure 2: Map of Soil Borings Performed.

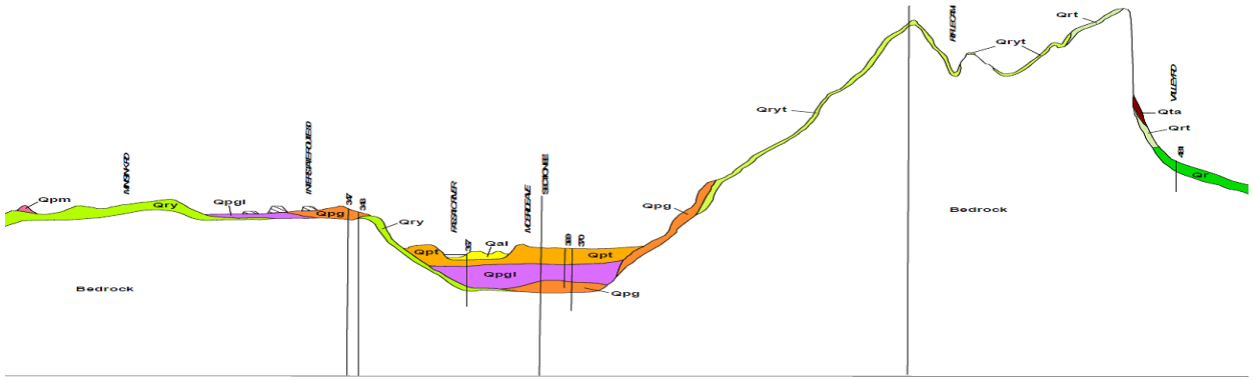


Figure 3: NJGS Geological Paterson and Orange Quadrangle Maps and Cross-section for the Area of Interest South of Patterson. Beige color (Jo) is Orange Mountain Basalt; Light green color (Jf) is Feltsville sandstone.

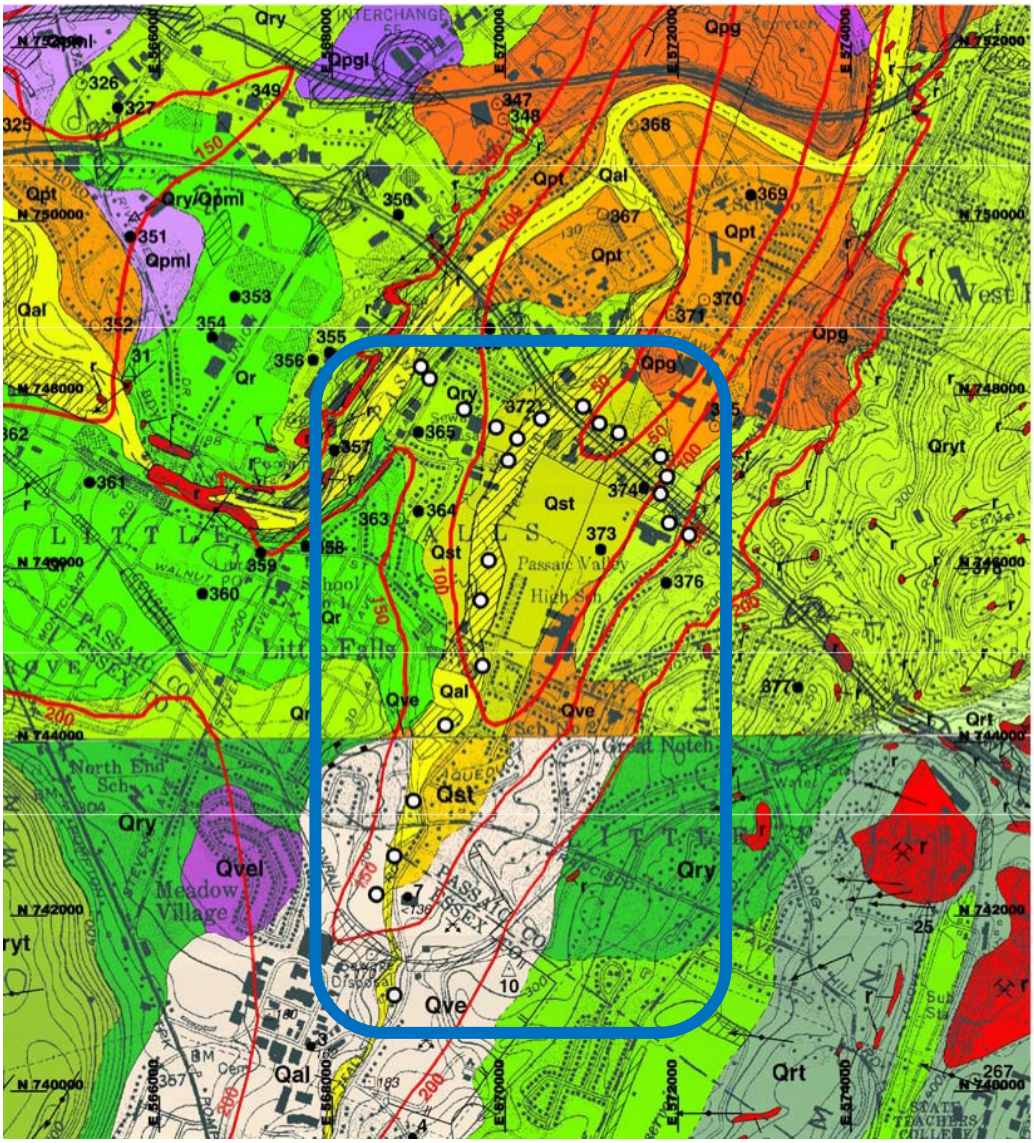


Figure 4: NJGS Surficial Deposits with Bedrock Contours.

3.0 SUBSURFACE EXPLORATION

A geotechnical investigation was performed for USACE by e4sciences | Earthworks, LLC in January 2012. The work was performed under IDC#204, Contract #W912DS-09-D-0001, Task Order #0026 and included a total of twenty-three (23) geotechnical borings drilled along the Peckman River Basin in New Jersey. The original scope of the project included levees, floodwalls channel improvements, and a diversion culvert from the Peckman River crossing under New Jersey Route 46 in in Passaic County to Little Falls Road near Old Bridge Road in Essex County.

The current project scope has been revised to be centrally localized and includes floodwalls and levees, with a diversion culvert and channel improvements proposed from the Peckman River south of New Jersey Route 46, north of Passaic Valley High School. The borings performed in the area of this updated project scope include borings PRB-11-01 through PRB-11-08. PRB-11-01, PRB-11-02, and PRB-11-07 were performed near the diversion culvert entrance retaining walls. PRB-11-08 was performed approximately 500 feet southwest of proposed levee section 2. Borings PRB-11-03 to PRB-11-06 were performed in the area of the proposed diversion culvert. However, explorations were not advanced in the immediate area of portions of the currently proposed levee and floodwall alignment. Consequently, additional explorations must be performed in the areas of the proposed levees and floodwalls pertaining to the current project scope so that subsurface conditions at these locations can be evaluated and confirmed in respect to this preliminary geotechnical assessment. Borings were drilled using the Standard Penetration Methods and Procedures. All SPT borings were drilled following ASTM standard D1586 with two modifications as follows:

-Blows per 6 inches were allowed to reach 100 blows.

-A 300 lb sampler was used upon refusal to advance the sample when it was determined that the refusal was due to cobbles.

As per Standard Penetration Methods and Procedures:

-A 140 lb hammer with a 30 inch drop was used to advance a 1 3/8 inch diameter split spoon sampler for drilling.

-When cohesive soils were present, a 3 inch diameter undisturbed piston tube was used.

-Any rocks or boulders that were encountered were cut with a NX size core.

-All soil sampling was continuous to a depth of 10 foot, then every five foot of depth thereafter.

4.0 SUBSURFACE CONDITIONS

The borings indicate that surficial soils, fill and recent river alluvium overlie glacial deposits that are underlain by bedrock. These deposits are broken down into seven stratigraphic units as follows:

a) Organic soil and silt: The soil and silt encountered is brown organic silt/soil, with

trace sand, grass and roots. Boring PRB-11-08 in the Peckman River channel did not encounter this organic layer. Boring PRB-11-10, drilled through East Main street, did not encounter this layer below the approximately one-foot layer of asphalt concrete (AC). N-values for the organic layer trended near 20. The thickness of these materials ranged from 0-foot to 4-foot.

b) Fill: Light brown, gray, or red in color, the fill encountered was a mix of silt, sand, and gravel with trace organic material. Fill material is native to the area and consists of reworked till, sands and gravels. Manmade materials such as glass, plastic, concrete, and asphalt are present in this unit. Fill was observed at all but two borings, PRB-11-08 and PRB-11-23. The uncorrected N-values of the fill ranged from 3 to 105, with an average of 31 depending on the clast size and concentration. The thickness of fill ranged from 0-foot to 18-foot.

c) Till: Red-brown silty sand and gravel, with varying amounts of clay was encountered in the subsurface investigation. Basalt and sandstone clasts supported by the silty sand matrix include pebbles, cobbles and boulders. Till was recorded at all but two locations, PRB-11-9B, and PRB-11-14. The uncorrected N-values of the till ranged from 16 to 200, with an average of 94 depending on the clast size, shape and concentration. The thickness of till ranged from 0-foot to >20-foot.

d) Sands and gravels: Sands and gravel units encountered were well sorted fluvial deposits. The uncorrected N-values of the materials ranged from 0 to 134, with an average of 29. This unit was encountered at borings PRB- 11-03, PRB-11-08, PRB-11-9B, PRB-11-11, PRB-11-13, and PRB-11-14. The thickness of these materials ranged from 0-foot to 10-foot.

e) Varved sand, silt, and clay: Red-brown silt, clay, or very fine sand. Clay composition and cohesive strength of the varved layers varied. Uncorrected N-values ranged between 10 and 132, with an average of 26 in these deposits. At boring PRB-11-08 sand, varved clay and silt alternates with sand layers below 13.0 feet. Varved silt and clay deposits were present in 14 of the 23 borings. In 12 of the borings, the varved deposit continued below the limit of the boring depth. The thickness of these materials ranged from 0-foot to 15.3 -foot.

f) Sandstone bedrock: The red/maroon micaceous Jurassic sandstone encountered is of the Feltville formation. Clasts of this material were encountered in the till and fill deposits. 5 foot rock core runs were advanced in Borings PRB-11-05 and PRB-11-06. The cores revealed intact Feltville sandstone. Coring rates in this formation averaged two minutes per foot. Rock Quality Designations (RQD) of the cores retrieved were 33 and 14, respectively, indicating a rock quality rating of poor to very poor in the upper five feet of bedrock.

g) Basalt bedrock: Orange Mountain Basalt is a dark gray to black Jurassic basalt with Calcite filled vugs. Clasts of this material were encountered in the till and fill deposits. Contact metamorphosed surfaces indicate pillow boundaries. The five foot rock core in PRB-11-23 indicated Orange Mountain Basalt. The coring rate averaged seven minutes per foot with an RQD of 18 indicating a rock quality rating of very poor for the top feet of rock. Basalt was encountered in only one of the borings advanced in the project vicinity.

Laboratory tests included the following: grain size analysis via the hydrometer analysis, moisture content, specific gravity, Atterberg limits, triaxial testing (when plastic soils were encountered), consolidated-undrained triaxial compression tests with pore water measurements, unconfined compression strength tests, and density of rock samples. Three of the borings recovered rock cores for testing.

The results of the laboratory tests generally classified the encountered materials into the following:

Fill Materials: The fill materials include sand, gravel and rock fragments with varying amounts of silt and clay. USCS classifications include GP, GW, GM and SP. Although no transmissivity testing was conducted on the samples retrieved, grain size analyses were conducted. Samples were noted to have 5 to 25 percent passing the #200 sieve; as a result, the soil deposit is considered to be porous.

River & Till Deposits: River deposits have been identified throughout the project. These deposits are predominantly sand and silt, and have been classified as SP or SW. Most of the till deposits have fines less than 10 percent. These materials are also considered to be porous.

Varved Clay and Silt Deposits: The varved clay and silt deposits include fines (silts and clays) inter-bedded with fine sand. Due to the amount of fine-grained particles the varved clay and silt may be considered relatively impermeable and therefore be suitable for a cutoff barrier.

Bedrock: The bedrock underlying the site consists of sandstone and basalt. Bedrock was encountered at depths of 11.0, 17.5 to 18.1 feet in three borings. In the remaining 20 borings, no bedrock was encountered.

5.0 GEOTECHNICAL ANALYSES

For the geotechnical analyses a generalized profile was created using subsurface information obtained in the borings performed within the most recent project scope limits. These borings include PRB-11-01 to PRB-11-08 which are shown on Figure 5.

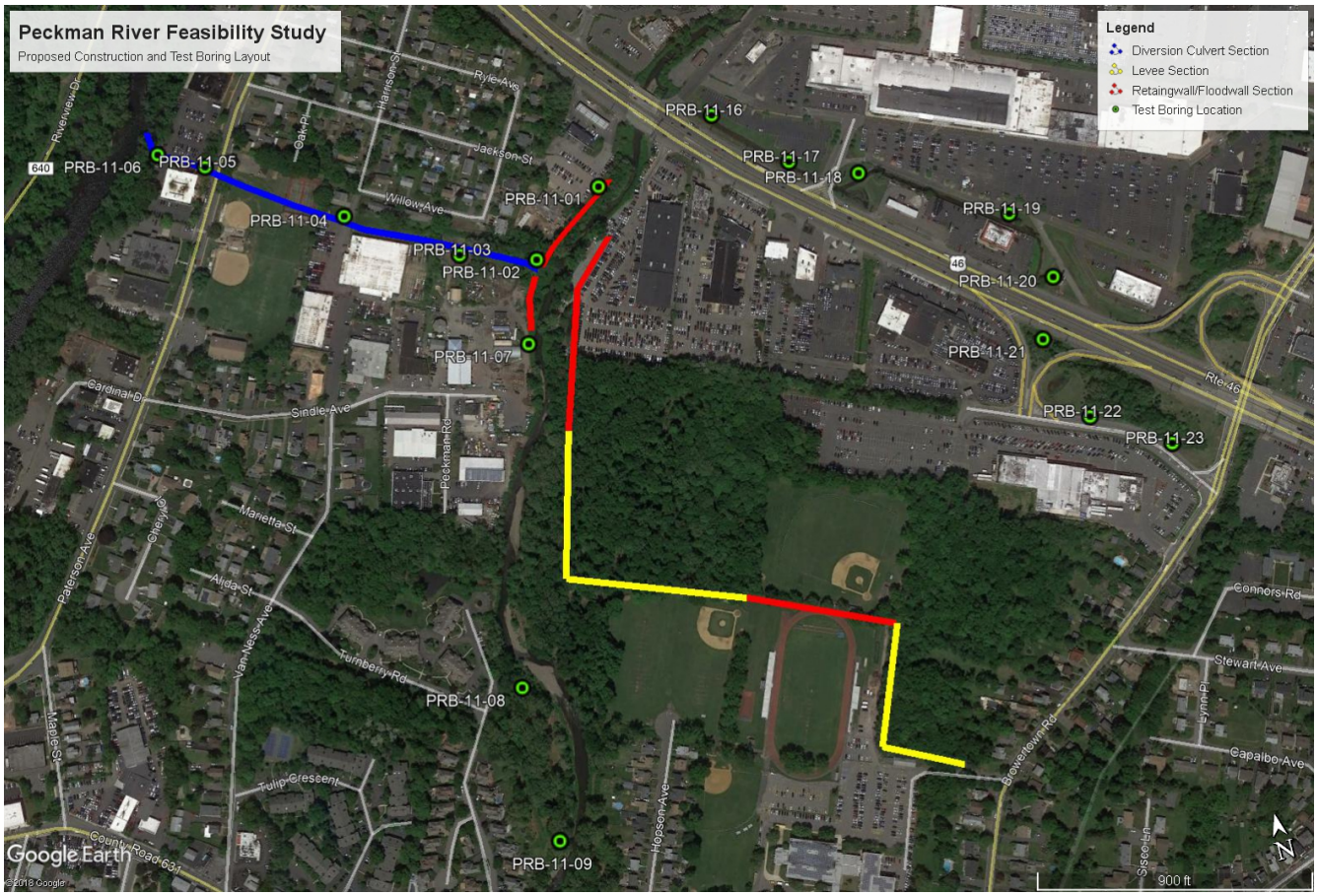


Figure 5: Google Earth Aerial of Project Site and Soil Boring Locations.

The subsurface conditions anticipated to be encountered within the limits of the revised project area can be generalized into the 5 stratum shown in the table below. Depths and average uncorrected N values shown were used to formulate geotechnical design parameters.

Table 1: Summary of Stratigraphic Unit Properties.

Strata	Depth (feet)	Average Uncorrected N-value/RQD
Organic Silt (OH)	0-3	20
FILL Sand and Gravel	3-8	31
Sand and Gravel Till (GM)	8-18	94
Varved Silt and Clay (ML/CL)	18-25	26
Sandstone Bedrock	5-7	14-33

5.1 LEVEES

5.1.1 SEEPAGE ANALYSIS

Geotechnical design parameters were based on the available existing field and laboratory test data obtained from the geotechnical investigation. Based on the current project limits, soil borings PRB-11-01 to PRB-11-08 were selectively chosen to develop the soil strength parameters. Embankment materials would likely be comprised of imported silt and/or clay soil (USCS Type ML/CL) for the impervious core and imported silty sands (USCS Type SM) or on-site material from the diversion culvert excavation (USCS Type GM/SM) for the remainder of the levee fill materials (embankment shell). The project specifications would be written to ensure that they conform to the assumed properties used during design. Fill would be specified to be placed in lifts and be placed in maximum 12-inch thick loose lifts compacted to 95 percent maximum density, in accordance with Modified Proctor test procedures ASTM D-1557. For embankment construction, it is recommended that the fill be placed approximately two percent above its optimum moisture content.

Seepage analyses were performed using the computer program SEEP/W, which is part of the GeoStudio suite of programs. The levee geometry is eight-foot high with a 12-foot wide crest and three horizontal to one vertical slopes. An impervious core and inspection trench was selected to minimize seepage through the levee. The proposed levee geometry satisfies the required dimensions required in EM 1110-2-1913 (see sections 6-1, 6-2, 7-2f). The following summarizes the estimated permeability constants used for seepage analysis:

Table 2: Permeability Constants.

Soil Area	Permeability		Anisotropy	GeoStudio Color
	feet/sec	cm/sec	kv/kh	
Levee Fill Material	3.28×10^{-6}	1×10^{-4}	1	
Impervious Core	3.28 x 10	1×10^{-7}	0.5	
Organic Silt	3.28×10^{-6}	1×10^{-4}	0.5	
FILL Sand and Gravel	3.28×10^{-5}	1×10^{-3}	1	
Sand and Gravel (Till)	3.28×10^{-5}	1×10^{-3}	1	
Varved Silt and Clay	3.28×10^{-9}	1×10^{-7}	0.5	

Permeability constants were estimated from intrinsic values referenced in “Applied Hydrology”, 4th ed., C.W. Fetter; Table 3.7 based on the grain size analyses obtained from the geotechnical investigation. The hydraulic conductivities used in the analysis are estimated from empirical data. Additional subsurface borings including falling head permeability tests along the levee and floodwall alignment should be performed in the next phase of this project in order to accurately determine the hydraulic conductivities of the existing in-situ soils.

During steady-state seepage analysis, it was determined that excessive uplift pressures are present on the impervious organic silt soil layer. It was determined that the factor of safety against uplift, or heave was approximately 0.7 at the toe of the levee. Excessive seepage (piping potential) though the shell of the embankment was also determined to be an issue. The determined horizontal factor of safety for piping at the toe of the levee was 2.5, which is less than the recommended value of 3 put forth in Jacksonville District’s “Herbert Hoover Dike Loadings and Factors of Safety for Use in Seepage and Stability Analysis.” To mitigate these excessive uplift pressures on the organic silt soil layer and piping through the levee shell, a one-foot diameter toe drain was placed five feet below the landside toe of the levee, in the sand and gravel fill layer. This design element reduced uplift pressures at the toe of the levee eliminating the potential for heave and resulted in factors of safety for piping above 3. It is noted that the toe drain should be designed with appropriate surrounding filter media to prevent clogging of the drain and will be done during the design phase of the project. The GeoStudio

results and calculations are attached.

5.1.2 SLOPE STABILITY

The following soil parameters were used for analyzing the slope stability:

Table 3: Slope Stability Soil Parameters.

Soil Area	Total Unit Weight	Cohesion	Friction Angle	GeoStudio Color
	pcf	psf	degrees	
Levee Shell	130	0	32	
Impervious Core	120	1500	0	
Organic Silt (OH)	110	200	0	
FILL Sand and Gravel	125	0	32	
Sand and Gravel Till (GM)	135	0	34	
Varved Silt and Clay (ML/CL)	120	2500	0	

The circular failure surface was analyzed using GeoStudio SLOPE/W for three different cases; end-of-Construction, rapid drawdown, and steady-state conditions. SLOPE/W performs a two-dimensional limit equilibrium stability analysis. The analyses were run using Spencer's Method. Pore pressure conditions for each SLOPE/W analysis were derived using the GeoStudio SEEP/W program except for the rapid drawdown case in which the "staged undrained strength" approach was used. This method follows that developed by Duncan, Wright and Wong (1990) as outlined in EM 1110-2-1902, Section G-3.

For the end-of-Construction case, the levee was assumed to not be hydraulically loaded and thus pore pressure only existed due to groundwater conditions. For the steady-state condition, it was assumed the levee would be loaded to its maximum height, 8 feet above the ground surface, for sufficient time such that fully saturated conditions are met. For the rapid drawdown case, calculations are done in two stages. The first stage involves the conditions before drawdown (maximum water height), and is considered an effective stress analysis before drawdown. The second stage uses R envelope parameters which consist of the effective strength parameters for the freely drained materials, and the undrained shear strengths determined from the result of

Stage 1 are used for materials that do not drain freely.

Factors of safety for the levee design with toe drain end of construction (Case I) and steady state (Case III) conditions was 1.6 for both cases. For rapid drawdown (Case II), the factor of safety was computed to be 1.3. These factors of safety meet required minimum factors of safety established in EM 110-2-1913, Table 6-1b.

5.1.3 SETTLEMENT

The borings indicate that the surficial soils are comprised of fill and recent river sediments over glacial deposits that overlie bedrock. The thickness of fill is approximately one to nine feet throughout the project area. The fill materials consist primarily of reworked sand and till deposits with fragments of manmade materials. The fill materials include sand, gravel and rock fragments with varying amounts of silt and clay. USCS classifications for the natural site soils include GP, GW, GM and SP. Uncorrected N-values for the fill layer range from 10 to 100 blows per foot, with an average uncorrected N-value of 31, and indicate that this soil deposit may be used to support the proposed embankments or flood walls providing the deposit is compacted or reworked to achieve the required design strength. Uncorrected N-values for the natural Till and sand and gravel ranged from 16 to 200 blows per foot, with an average value of 94 for the Till and 29 for the sand and gravel material. The Till and sand and gravel materials are suitable to support the proposed construction. In areas where loose or soft materials may be locally encountered beneath the levees or floodwalls, specialized ground improvement may be warranted, especially in areas where organic soils was encountered. As this condition is limited in area, the fill and organic soil deposits are not anticipated to have a significant design or construction impact to the project. Settlement calculations were not performed for the proposed levee as part of the feasibility study as significant settlement is not anticipated to occur based on the encountered subsurface conditions. As such, any minor settlement will not impact the feasibility and or cost of implementing the proposed project. However, settlement analyses will be performed during the design phase. It should be understood that localized conditions may require soil replacement, soil stabilization, or other foundation ground improvement methods.

5.2 DIVERSION CULVERT

An approximately 1,700 foot diversion culvert is proposed approximately 800 feet south of New Jersey Route 46 intersection with Andrews Drive. The culvert runs east to west, with the exit of the culvert discharging into the Passaic River. The reinforced concrete culvert is proposed to consist of a wide mouth entrance with a weir, transitioning to a double box design. The discharging end of the culvert will consist of a stilling basin before emptying to the Passaic River. The diversion culvert entrance and exits are proposed to include retaining walls on each side. Soil borings PRB-11-01, PRB-11-02, and PRB-11-07 were performed in the vicinity of the proposed culvert entrance retaining walls which are proposed to be approximately 100 feet in length. Additionally, in soil borings PRB-11-05 and PRB-11-06, bedrock was observed. This

bedrock included sandstone of the Feltville Formation. Due to the proposed bearing elevations of the culvert near the discharging end, bedrock excavation may be necessary and require special equipment or construction techniques.

5.2.1 BEARING CAPACITY

Analysis was conducted using the subsurface conditions observed in soil borings PRB-11-02 to PRB-11-06 to evaluate bearing conditions of the culvert on the soil. The stratigraphy along the culvert alignment revealed that a majority of the culvert excavation would consist of sand and gravel till material with a small amount of sand and gravel fill material near the surface of the excavation. The range of excavation for the culvert construction was between 16 and 24 feet below the existing ground surface. Analyzing stratigraphy in each of the five borings along the culvert alignment and the proposed loading conditions of the culvert and associated roadway above, it was determined that the culvert would have a fully compensated foundation. A fully compensated foundation case means that the bearing soil at the bottom of the culvert was subjected to higher stresses due to the excavated soil above, than from the full load of the culvert and backfill. The analysis is included in the Attachments section. The presumptive allowable bearing capacity for glacial till soils from the 2015 International Building Code, Table 1806.2 is 3,000 psf.

5.2.3 SETTLEMENT

As illustrated in section 5.2.1 and in the attached analysis, the culvert is determined to have a fully compensated foundation case where the bearing soil beneath the culvert was previously loaded to a higher magnitude than they load imposed after culvert construction. Due to this case, settlement of the culvert structure is would be negligible, if at all.

5.3 FLOODWALL AND RETAINING WALL STABILITY ANALYSIS

Where an earthen levee is cost prohibitive, floodwalls are proposed to provide flood risk management. Global stability analyses for both the proposed floodwalls and retaining walls are included in the Structural section of this appendix.

5.4 FLOODWALL SEEPAGE

Seepage below the proposed floodwalls was analyzed using SEEP/W software. The stratigraphic conditions used were the same as those used for the levee seepage analysis. Floodwall dimensions were taken from the proposed floodwall feasibility design drawings. Due to the impervious soil materials, it was determined that piping will not be considered as the impervious material is subject to uplift/heave. It was determined through the seepage analysis that excessive uplift pressures on the organic silt layer near the surface to approximately 40 feet beyond the end of the wall foundation exist, resulting in a factor of safety less than one against uplift. A one-foot diameter toe drain was implemented five feet below the surface in the sand and gravel fill layer, to relieve uplift pressures and lower exit gradients on the landside of the floodwall. When the toe

drain was implemented, the analysis showed uplift pressures with a factor of safety greater than one. The floodwall seepage analysis is included in the Attachments section.

6.0 CONCLUSIONS

The information provided in this report is for conceptual purposes only and details and assumptions provided are subject to change. Further evaluation/analysis and information (including, but not limited to, additional borings, field tests, laboratory tests, surveys) will be required in future phases in order to refine feature designs, layout and cost estimates.

7.0 ATTACHMENTS

Geotechnical Calculations

Attachments

Attachment 1 - Engineering Soil Parameters
Attachment 2 - Culvert Bearing and Settlement Calculations
Attachment 3 - GeoStudio Seepage – No Toe Drain
Attachment 4 - GeoStudio Seepage – Toe Drain
Attachment 5 - GeoStudio Slope Stability – No Toe Drain
Attachment 6 - GeoStudio Slope Stability – Toe Drain
Attachment 7 - Piping Factor of Safety Calculations
Attachment 8 - SAJ Herbert Hoover Dike Loadings and Factors of Safety for Use in Seepage and Stability Analysis

Geotechnical and HTRW Investigations for Peckman River Basin Project, New Jersey, dated January 27, 2012

Flood Risk Management Feasibility Study

Peckman River Basin
New Jersey

Draft Integrated Feasibility Report and Environmental Assessment

Appendix C3: Geotechnical Engineering

Attachment 1
Engineering Soil Parameters

US Army Corps of Engineers®
New York District

October 2019

Peckman River Feasibility Study 7/1/2019
EMO

Soil Parameters for Analysis

In general, 6 strata to consider

- Organic Silt/Soil (OM)
- FILL: Sand and Gravel (GM)
- Silty Sand and Gravel Till (GM)
- Varved Silt and Clay (ML/CL)
- Clay levee Core (CL)
- Levee Shell (SM/GM)

Using SPT blow counts, lab index testing, boring log soil descriptions, and NAVFAC DM7.1, assume the following:

Strata	ϕ'	γ (pcf)	C (psf)
Organic Silt/Soil	0	110	200
FILL: Sand + Gravel	32	125	0
Silty Sand + Gravel Till	34	135	0
Varved Silt + Clay	0	120	2500
Clay Core	0	120	1500
Levee Shell	32	130	0

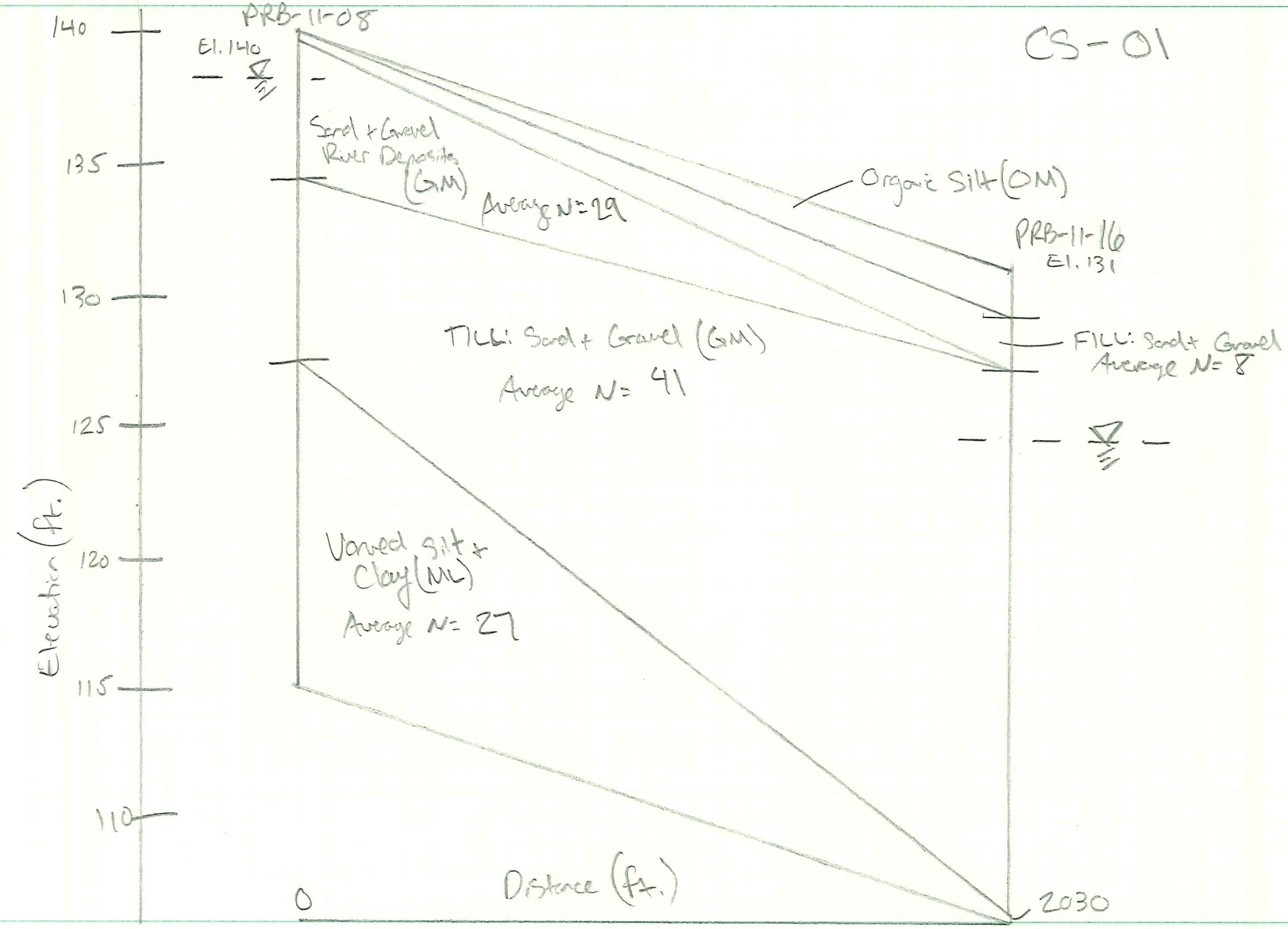
Peckman River Feasibility Study 7/1/2019

Hydraulic Conductivity Parameters for Analysis

Values are estimated from "Applied Hydrogeology," by C.W. Fetter, 4th Edition. and DM 7.1, Figure 1, page 7.1-139.

Strata	K (cm/s)
Organic Silt/Soil	1×10^{-4}
FILL: Sand + Gravel	1×10^{-3}
Silty Sand + Gravel Till	1×10^{-3}
Varved Silt + Clay	1×10^{-6}
Clay Core	1×10^{-6}
Levee Shell	1×10^{-4}

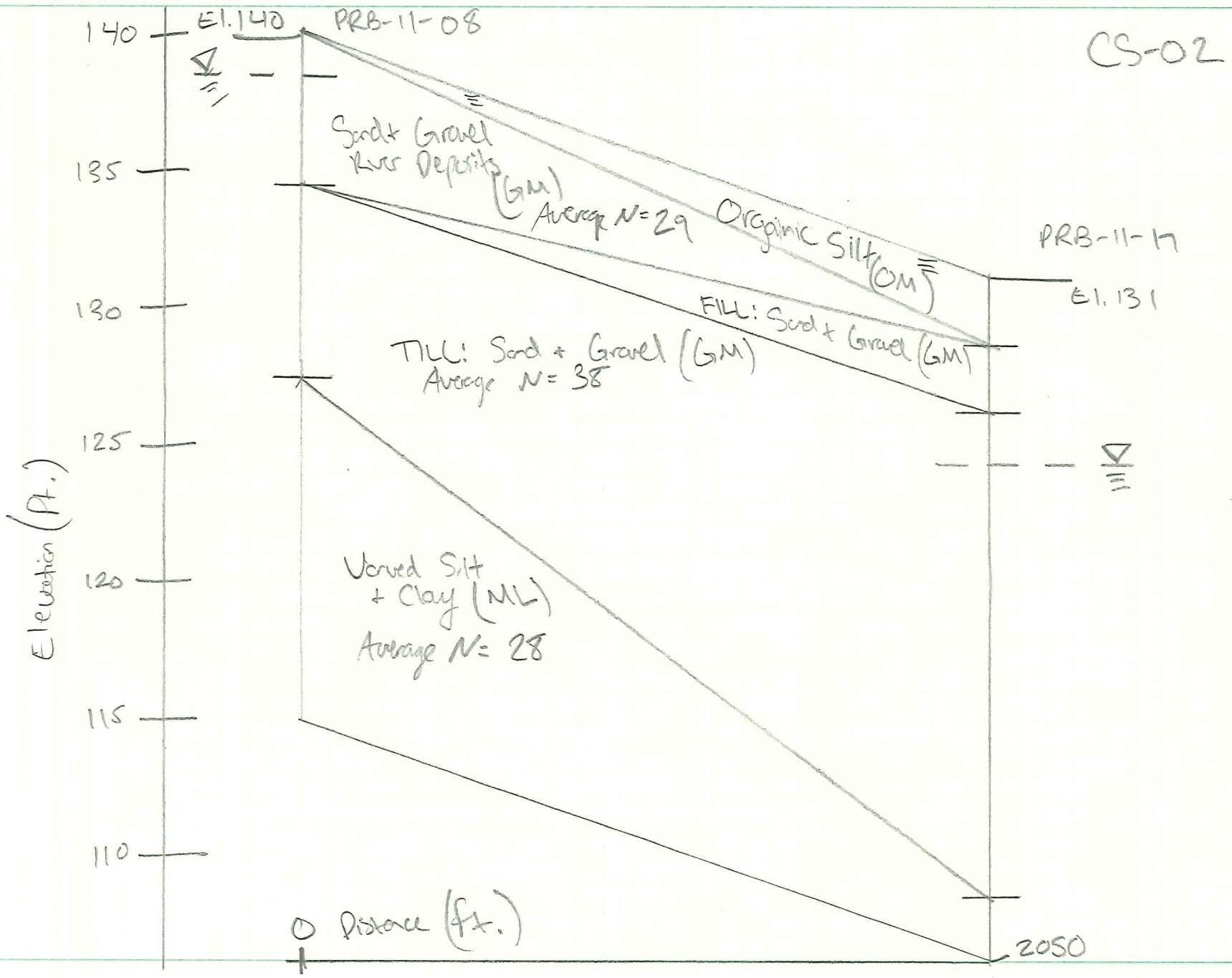
Due to the natural variability of glacial till, assume some anisotropy $\frac{K_y}{K_x} = 0.5$



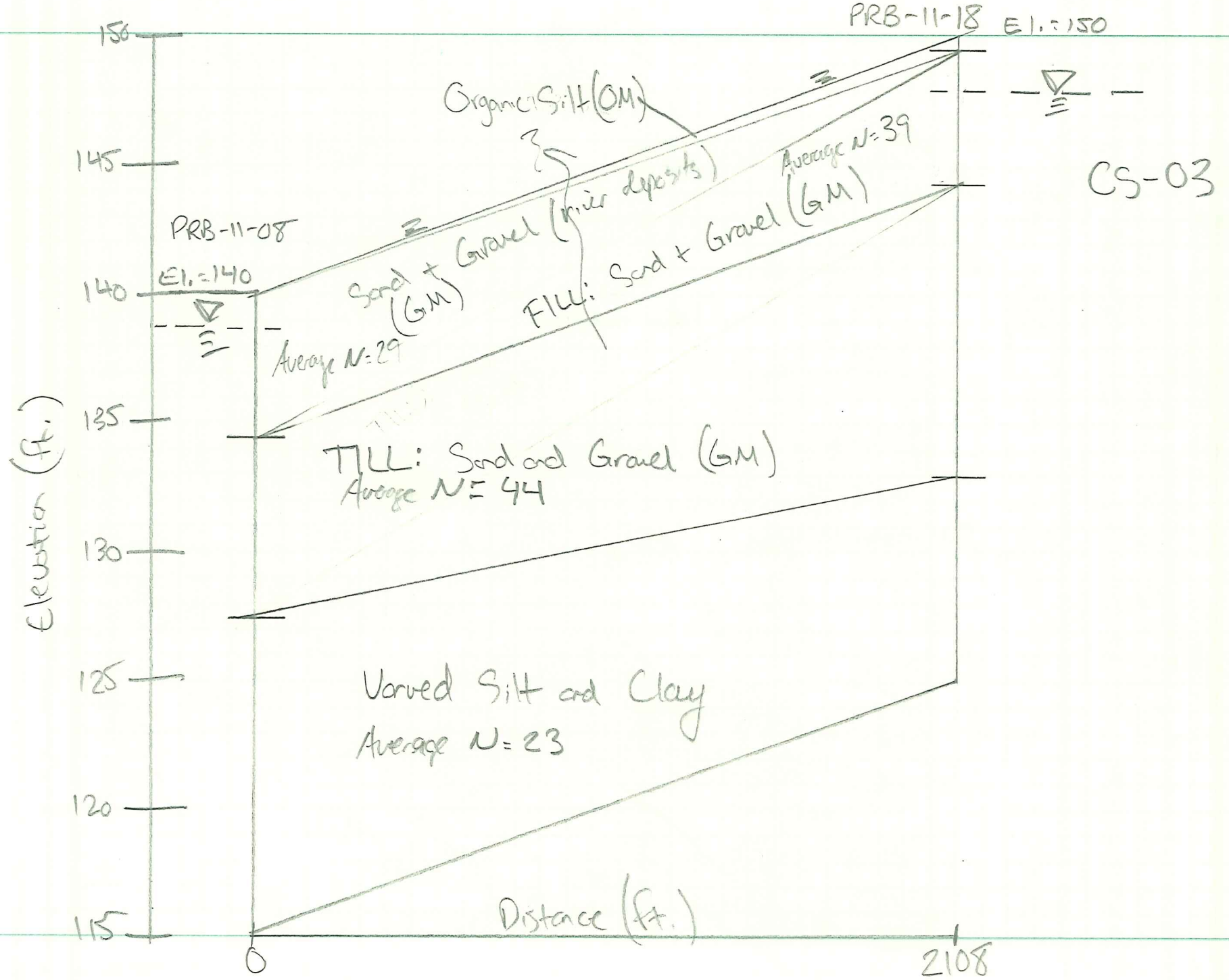
Peckman River Feasibility

Cross Sections

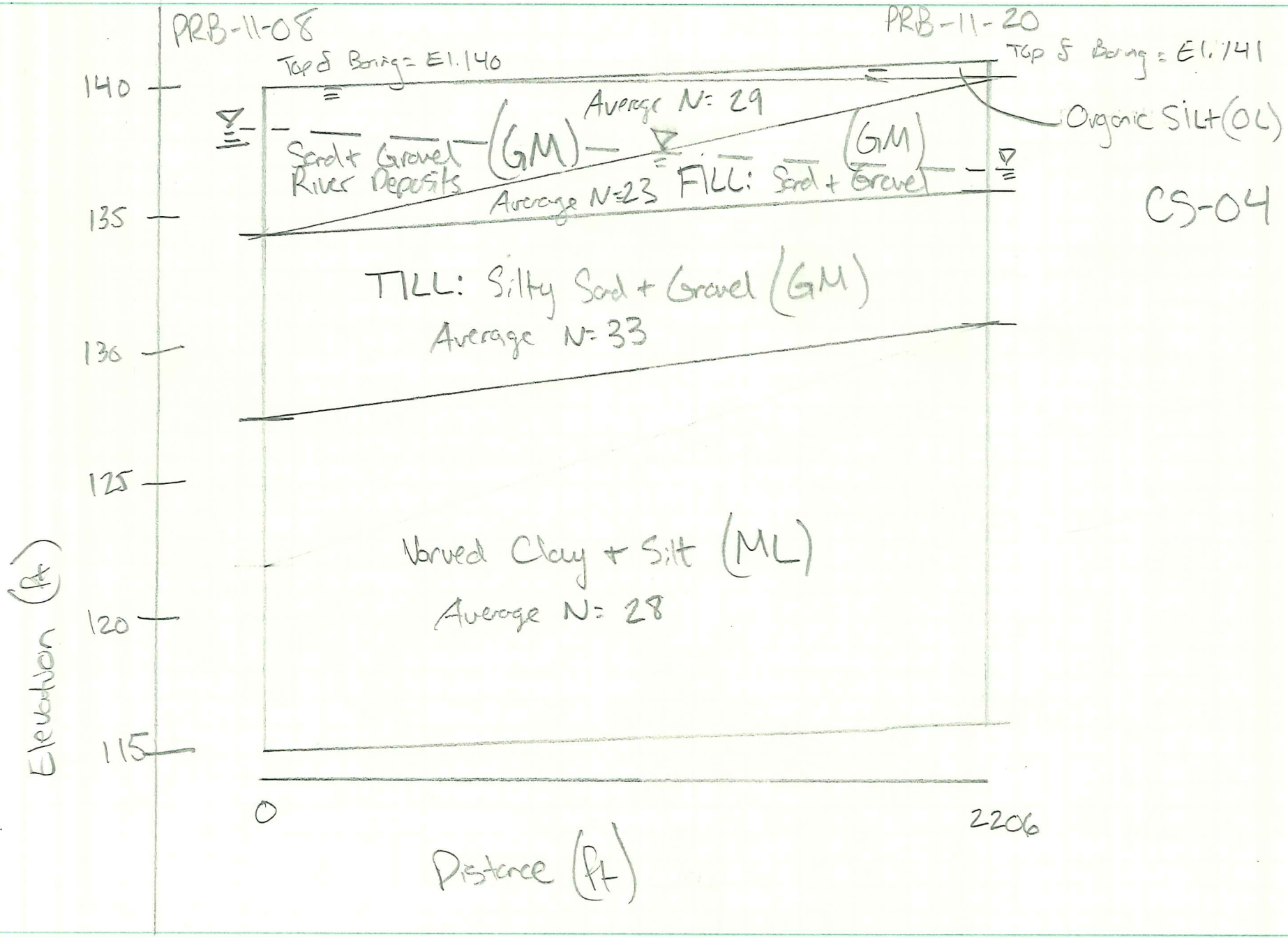
Emo 6/27/2019



Pickman River Feasibility | Cross Sections | ENO 6/27/2019



Peckman River Feasibility | Cross Sections | EVO 6/27/2019



Peckman River Feasibility Cross Sections | EMO 6/27/2019

US Army Corps of Engineers
New York District

MARK	DESCRIPTION	DATE	APPR.

MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT
NEW YORK
http://www.nan.usace.army.mil

26 FEDERAL PLAZA
ROOM 2109
NEW YORK, NY
10278-0090

DESIGNED BY: XXX	DATE: 15 APR 2015
DWN BY: XXX	CKD BY: XXX
SOLICITATION NO.: W-	
SUBMITTED BY: XXX	CONTRACT NO.:
PLOT SCALE: XXX	ISSUE DATE: XXXXXX
PLOT DATE: XXX	FILE NUMBER:
SIZE: XXX	FILE NAME: PECKMAN PLAN.DWG

GREEN BROOK FLOOD DAMAGE
REDUCTION PROJECT - SEGMENT C-1
SEBRINGS MILLS BRIDGE, PS, FLOODWALL,
& LEVEE (MIDDLESEX & GREEN BROOK, NJ)

SHEETTITLE1
SHEETTITLE2
SHEETTITLE3

Sheet Reference Number
X-XXX
Sheet XX of 175

Submitted by: USERNAME Plot Date: DD-MMM-YYYY HH:MM FILE NAME

e4sciences / Earthworks LLC
Peckman River Basin borings
LABORATORY TESTING DATA SUMMARY

BORING NO.	SAMPLE NO.	DEPTH (ft)	IDENTIFICATION TESTS WATER CONTENT (%)	LIQUID LIMIT (-)	PLASTIC LIMIT (-)	PLAS. INDEX (-)	USCS SYMB. (1)	SIEVE MINUS NO. 200 (%)	HYDRO. % MINUS 2 µm (%)	TOTAL UNIT WEIGHT (pcf)	DRY UNIT WEIGHT (pcf)	SPECIFIC GRAVITY (-)	STRENGTH Type Test @ STRESS (psi)	PEAK SHEAR STRESS (psi)	AXIAL STRAIN @ PEAK STRESS (%)	REMARKS
PRB-11-01	spoon 4	6-8	9.6				GW-GM	8.9	1							
PRB-11-02	spoon 7	19-21	12.2				GM	12.2	1			2.755				
PRB-11-03	spoon 3A	4-6	7.1				SP	4.7	0			2.696				
PRB-11-04	spoon 3	4-6	7.5				GC	24.7	4			2.702				
PRB-11-05	spoon 7	15-16.33	12.3				SM	20.7	1			2.725				
PRB-11-06	spoon 4	6-8	8.6				GP-GM	9.9	0			2.681				
PRB-11-07	spoon 8	18-20	29.2	28	20	8	CL	99.6	28			2.743				
PRB-11-08	spoon 6A	13-15	37.9	38	21	17	CL	99.2	48			2.755				
PRB-11-09	spoon 7	18-20	20.1	22	17	5	CL-ML	84.2	3			2.742				
PRB-11-11	spoon 9	23-25	24.7				SP-SM	6.1	0			2.688				
PRB-11-12	spoon 10	23-25	21.6	np	np	NP	ML	63.1	3			2.675				
PRB-11-15	spoon 6	14-16	7.7				GP-GM	9.7	1			2.705				
PRB-11-16	spoon 3	4-6	20.1				CL	56.0	7			2.682				
PRB-11-17	spoon 1	0-2	21.8				SM	33.4	3			2.670				
PRB-11-18	spoon 3A	4-6	19.5				SM	16.1	3			2.669				
PRB-11-18	spl 6	18-20								127.5						
PRB-11-18	spl 6	18.2	21.0													
PRB-11-18	spl 6 A	18.45	23.0				CL			127.9	104.0		CIU@5	12.5	19.4	T3171
PRB-11-18	spl 6 B	18.95	23.4	30	18	12	CL	99.4	29	127.5	103.3	2.703	CIU@10	17.3	15.3	T3172
PRB-11-18	spl 6 C	19.45	20.7				CL			130.6	108.2		CIU@20	29.6	20.9	T3173
PRB-11-19	spl 6	13-15								129.4						
PRB-11-19	spl 6	13.35	14.6													
PRB-11-19	spl 6 A	13.6	21.3				CL			129.7	106.9		CIU@5	19.6	19.7	T3168
PRB-11-19	spl 6	13.9	20.0													
PRB-11-19	spl 6 B	14.15	19.8	24	15	9	CL	86.6	21	130.9	109.2	2.695	CIU@10	62.4	19.3	T3169
PRB-11-19	spl 6	14.45	22.5													
PRB-11-19	spl 6 C	14.7	20.5				CL			130.3	108.2		CIU@20	31.0	10.2	T3170
PRB-11-20	spoon 7	18-20	26.8	27	19	8	CL	98.8	16			2.709				
PRB-11-22	spoon 6	13-15	12.7				SM	44.8	4			2.706				
PRB-11-23	spoon 3B	4-6	23.9				CL	86.7	22			2.661				

Note: (1) USCS symbol based on visual observation, Sieve results, and Atterberg Limits reported.

Prepared by: JR
Reviewed by: GET
Date: 11/17/2011

TerraSense, LLC
45H Commerce Way
Totowa, NJ 07512

e4sciences / Earthworks LLC
Peckman River Basin borings
LABORATORY TESTING DATA SUMMARY

BORING NO.	SAMPLE NO.	DEPTH (ft)	IDENTIFICATION TESTS										STRENGTH			REMARKS
			WATER CONTENT (%)	LIQUID LIMIT (-)	PLASTIC LIMIT (-)	PLAS. INDEX (-)	USCS SYMB. (1)	SIEVE MINUS NO. 200 (%)	HYDRO. % MINUS 2 μm (%)	TOTAL UNIT WEIGHT (pcf)	DRY UNIT WEIGHT (pcf)	SPECIFIC GRAVITY (-)	Type Test	PEAK SHEAR STRESS (tsf)	AXIAL STRAIN @ PEAK STRESS (%)	
PRB-11-01	Spoon 5	8-10	15.2				SW-SM	7.0	1							
PRB-11-05	Spoon 1	0-2	11.6				GM	14.8	3							
PRB-11-10	Spoon 7	18-20	10.9				SM	45.4	1			2.666				
PRB-11-11	Spoon 5B	8-10	10.0				GW-GM	6.5	1							
PRB-11-12	Spoon 6	10-11.5	1.5				GW	3.6	1							
PRB-11-13	Spoon 6	13-15	21.7				ML	73.1	2							
PRB-11-14	Spoon 3	4-6	18.7				ML	53.2	3			2.652				
PRB-11-14	Spoon 7	15-17	14.3				SP-SM	8.7	1							
PRB-11-14	Spoon 8	18-20	14.9				SW-SM	8.8	1							
PRB-11-17	Spoon 5	8-10	14.1				SM	14.2	2			2.765				
PRB-11-19	Spoon 7	15-17	23.8	25	16	9	CL	98.6	22			2.701				
PRB-11-21	Spoon 4	6-8	7.7				SM	24.0	5			2.699				

Note: (1) USCS symbol based on visual observation, Sieve results and Atterberg Limits reported.

Prepared by: JR
Reviewed by: GET
Date: 11/17/2011

TerraSense, LLC
45H Commerce Way
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DM7.1

TABLE 6
Typical Values of Soil Index Properties

	Particle Size and Gradation				Voids[1]					Unit Weight[2] (lb./cu.ft.)						
	Approximate Size Range (mm)		Approx. D_{10} (mm)	Approx. Range Uniform Coefficient C_u	Void Ratio			Porosity (%)		Dry Weight			Wet Weight		Submerged Weight	
	D_{max}	D_{min}			e_{max} loose	e_{cr}	e_{min} dense	n_{max} loose	n_{min} dense	Min loose	100% Mod. AASHO	Max dense	Min loose	Max dense	Min loose	Max dense
GRANULAR MATERIALS																
Uniform Materials																
a. Equal spheres (theoretical values)	-	-	-	1.0	0.92	-	0.35	47.6	26	-	-	-	-	-	-	-
b. Standard Ottawa SAND	0.84	0.59	0.67	1.1	0.80	0.75	0.50	44	33	92	-	110	93	131	57	69
c. Clean, uniform SAND (fine or medium)	-	-	-	1.2 to 2.0	1.0	0.80	0.40	50	29	83	115	118	84	136	52	73
d. Uniform, inorganic SILT	0.05	0.005	0.02	1.2 to 2.0	1.1	-	0.40	52	29	80	-	118	81	136	51	73
Well-graded Materials																
a. Silty SAND	2.0	0.005	0.02	5 to 10	0.90	-	0.30	47	23	87	122	127	88	142	54	79
b. Clean, fine to coarse SAND	2.0	0.05	0.09	4 to 6	0.95	0.70	0.20	49	17	85	132	138	86	148	53	86
c. Micaceous SAND	-	-	-	-	1.2	-	0.40	55	29	76	-	120	77	138	48	76
d. Silty SAND & GRAVEL	100	0.005	0.02	15 to 300	0.85	-	0.14	46	12	89	-	146[3]	90	155[3]	56	92
MIXED SOILS																
Sandy or Silty CLAY	2.0	0.001	0.003	10 to 30	1.8	-	0.25	64	20	60	130	135	100	147	38	85
Skip-graded Silty CLAY with stones or rk fgmts	250	0.001	-	-	1.0	-	0.20	50	17	84	-	140	115	151	53	89
Well-graded GRAVEL, SAND, SILT & CLAY mixture	250	0.001	0.002	25 to 1000	0.70	-	0.13	41	11	100	140	148[4]	125	156[4]	62	94
CLAY SOILS																
CLAY (30%-50% clay sizes)	0.05	0.5μ	0.001	-	2.4	-	0.50	71	33	50	105	112	94	133	31	71
Colloidal CLAY (-0.002 mm: 50%)	0.01	10Å	-	-	12	-	0.60	92	37	12	90	106	72	128	8	66
ORGANIC SOILS																
Organic SILT	-	-	-	-	3.0	-	0.55	75	35	40	-	110	87	131	25	69
Organic CLAY (30% - 50% clay sizes)	-	-	-	-	4.4	-	0.70	81	41	30		100	82	125	18	62

DM 7.1

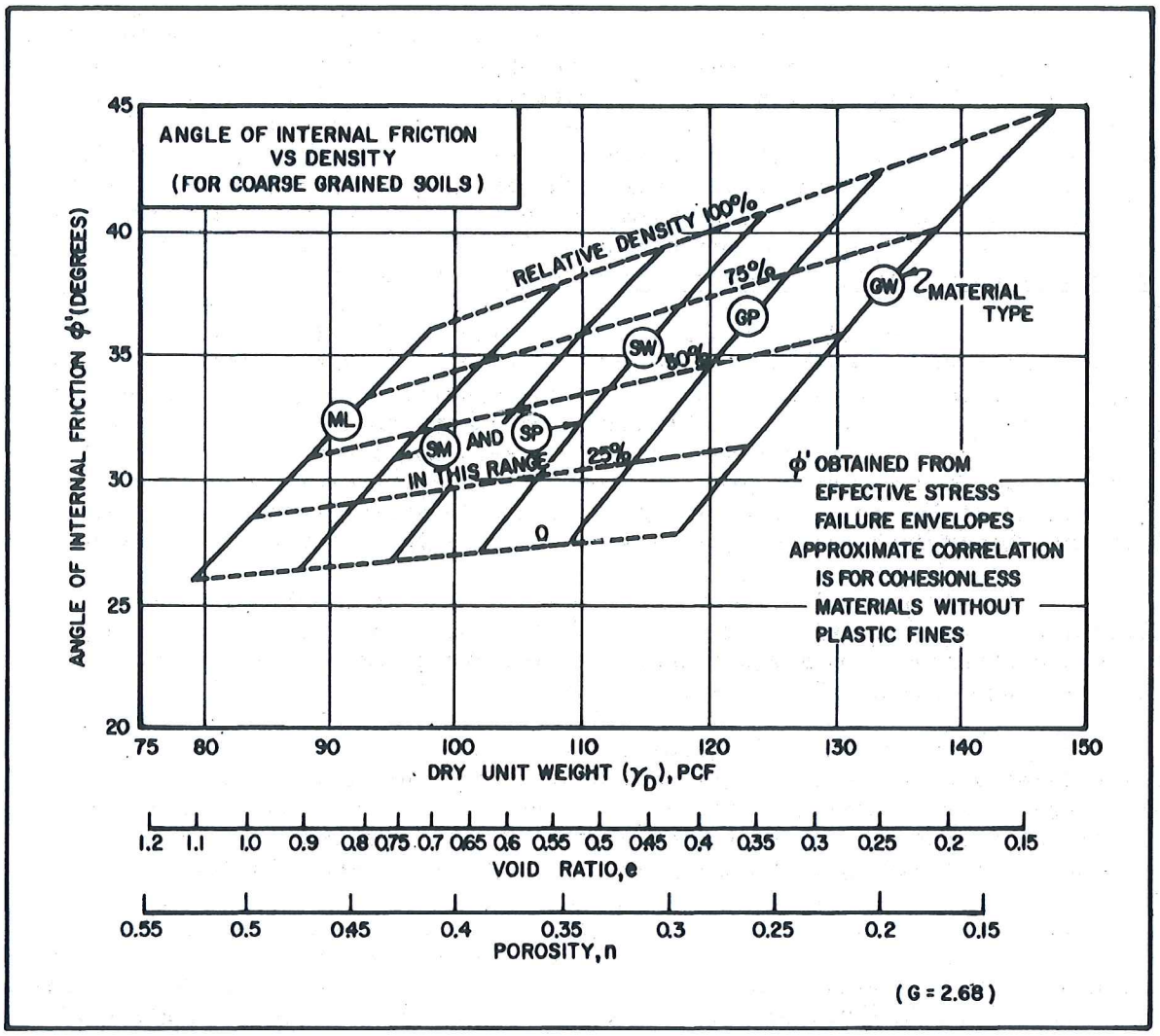

FIGURE 7
Correlations of Strength Characteristics for Granular Soils

DM 7.1

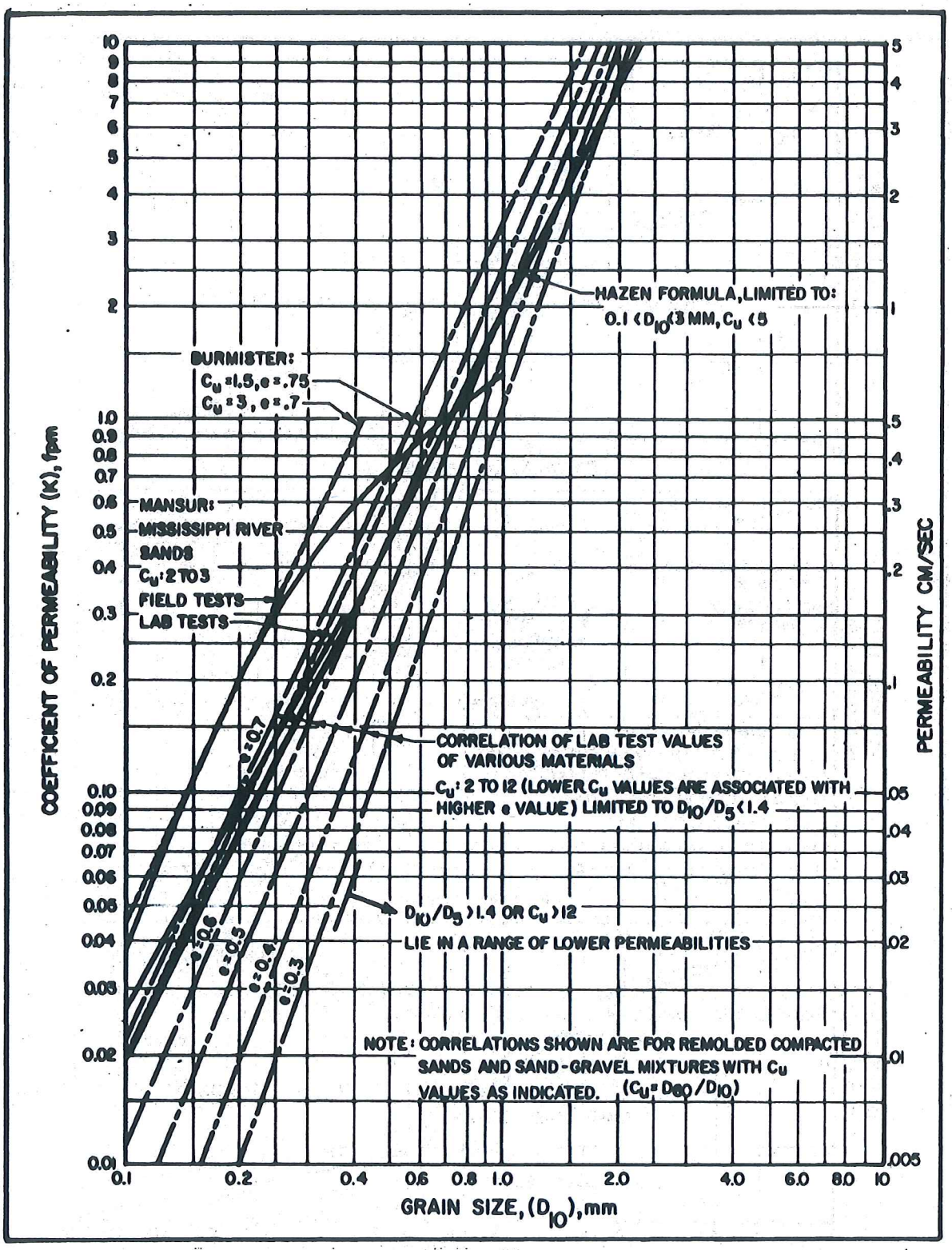

FIGURE 1
Permeability of Sands and Sand-Gravel Mixtures

Fetter 4th Edn.

Table 3.6 Conversion Values for Hydraulic Conductivity

1 gal/day/ft^2	=	0.0408 m/day
1 gal/day/ft^2	=	0.134 ft/day
1 gal/day/ft^2	=	4.72×10^{-5} cm/s
1 ft/day	=	0.305 m/day
1 ft/day	=	7.48 gal/day/ft^2
1 ft/day	=	3.53×10^{-4} cm/s
1 cm/s	=	864 m/day
1 cm/s	=	2835 ft/day
1 cm/s	=	21,200 gal/day/ft^2
1 m/day	=	24.5 gal/day/ft^2
1 m/day	=	3.28 ft/day
1 m/day	=	0.00116 cm/s

Table 3.7 Ranges of Intrinsic Permeabilities and Hydraulic Conductivities for Unconsolidated Sediments

Material	Intrinsic Permeability (darcys)	Hydraulic Conductivity (cm/s)
Clay	$10^{-6} - 10^{-3}$	$10^{-9} - 10^{-6}$
Silt, sandy silts, clayey sands, till	$10^{-3} - 10^{-1}$	$10^{-6} - 10^{-4}$
Silty sands, fine sands	$10^{-2} - 1$	$10^{-5} - 10^{-3}$
Well-sorted sands, glacial outwash	$1 - 10^{2}$	$10^{-3} - 10^{-1}$
Well-sorted gravel	$10 - 10^{3}$	$10^{-2} - 1$

obviously a wide-ranging continuum of permeability values for unconsolidated sediments (Table 3.7).

The intrinsic permeability is a function of the size of the pore opening. The smaller the size of the sediment grains, the larger the surface area the water contacts (Figure 3.14). This increases the frictional resistance to flow, which reduces the intrinsic permeability. For well-sorted sediments, the intrinsic permeability is proportional to the grain size of the sediment (Norris & Fidler 1965).

For sand-sized alluvial deposits, several factors relating intrinsic permeability to grain size have been noted (Masch & Denny 1966). These observations would hold true for all sedimentary deposits, regardless of origin of deposition.

1. As the median grain size increases, so does permeability, due to larger pore openings.
2. Permeability will decrease for a given median diameter as the standard deviation of particle size increases. The increase in standard deviation indicates a more poorly sorted sample, so that the finer material can fill the voids between larger fragments. (Figure 3.2B)

Flood Risk Management Feasibility Study

Peckman River Basin
New Jersey

Draft Integrated Feasibility Report and Environmental Assessment

Appendix C3: Geotechnical Engineering

Attachment 2
Culvert Bearing and Settlement Calculations

US Army Corps of Engineers®
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October 2019

Peckman River Feasibility Study 7/2/2019 EMO 1/4

Culvert Bearing Analysis

Purpose: Determine in-situ bearing pressures at base of proposed culvert foundation and compare to proposed culvert loading conditions. to show fully compensated foundation case.

Given:

- Boring logs for culvert borings
PRB-11-02, PRB-11-03, PRB-11-04, PRB-11-05
PRB-11-06
- Culvert dimensions and estimated bearing pressures from structural group (Tu Ha)
Entrance: B= 82.5 ft, 1000 psf bearing
DB Culvert: B= 45.5 ft, 1008 psf bearing
- Soil Parameters for analysis (see Soil parameters for analysis pack)

Strata	ϕ (A)	γ (pcf)
GM (Fill)	32	125
GM (Till)	34	135

Peckman River Feasibility Study 7/2/2019 EMO 2/4

Culvert Bearing Analysis

Using profile of culvert (see attached) with overlain boring logs (PRB-11-02-06), we will examine if there is a net pressure increase on the culvert foundation bearing stratum due to construction of the culvert.

Analysis

PRB-11-02 GS El. = 137 Bearing El. = 121

7 ft Fill GW table 9 ft b.g.s.

9 ft Till

7 ft (125 pcf) + 2 ft (135 pcf) + 7 ft (135 pcf - 62.4 pcf)

σ' = 1653.2 psf σ at culvert entrance = 1000 psf

1000 psf - 1653.2 psf = $\boxed{-653.2 \text{ psf} = \text{Net pressure decrease}}$

PRB-11-03 GS El. = 140 Bearing El. = 120

GW table 8 ft b.g.s

4 ft Fill

16 ft Till

4 ft (125 pcf) + 4 ft (135 pcf) + 12 ft (135 pcf - 62.4 pcf)

σ' = 1911.2 psf σ at double box culvert section = 1008 psf

1008 psf - 1911.2 psf = $\boxed{-903.2 \text{ psf} = \text{net pressure decrease}}$

Peckman River Feasibility

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EMO

3/4

Culvert Bearing Analysis

PRB-11-04 GS El. = 140 Bearing El. = 115
GW table 4 ft b.g.s.

4 ft Fill
21 ft Till

4 ft (125 pcf) + 21 ft (135 pcf - 62.4 pcf)
$\sigma' = 2024$ psf σ culvert = 1008 psf

1008 psf - 2024 psf = $\boxed{-1016 = \text{Net pressure decrease}}$

PRB-11-05 GS El. = 136 Bearing El. = 112
GW table 10 ft b.g.s.

2 ft Fill
18 ft Till
4 ft Bedrock (150 pcf)

2 ft (125 pcf) + 8 ft (135 pcf) + 10 ft (135 pcf - 62.4 pcf)
+ 4 ft (150 pcf - 62.4 pcf)

$\sigma' = 2406$ psf σ culvert = 1008 psf

1008 psf - 2406 psf = $\boxed{-1398 \text{ psf} = \text{net pressure decrease}}$

Peckman River Feasibility Study 7/2/2019 EMO 4/4

Culvert Bearing Analysis

PRB-11-06 GS. El. = 126 Bearing El. = 110

GW table 9 ft b.g.s.

6 ft Fill

4 ft Till

6 ft Bedrock

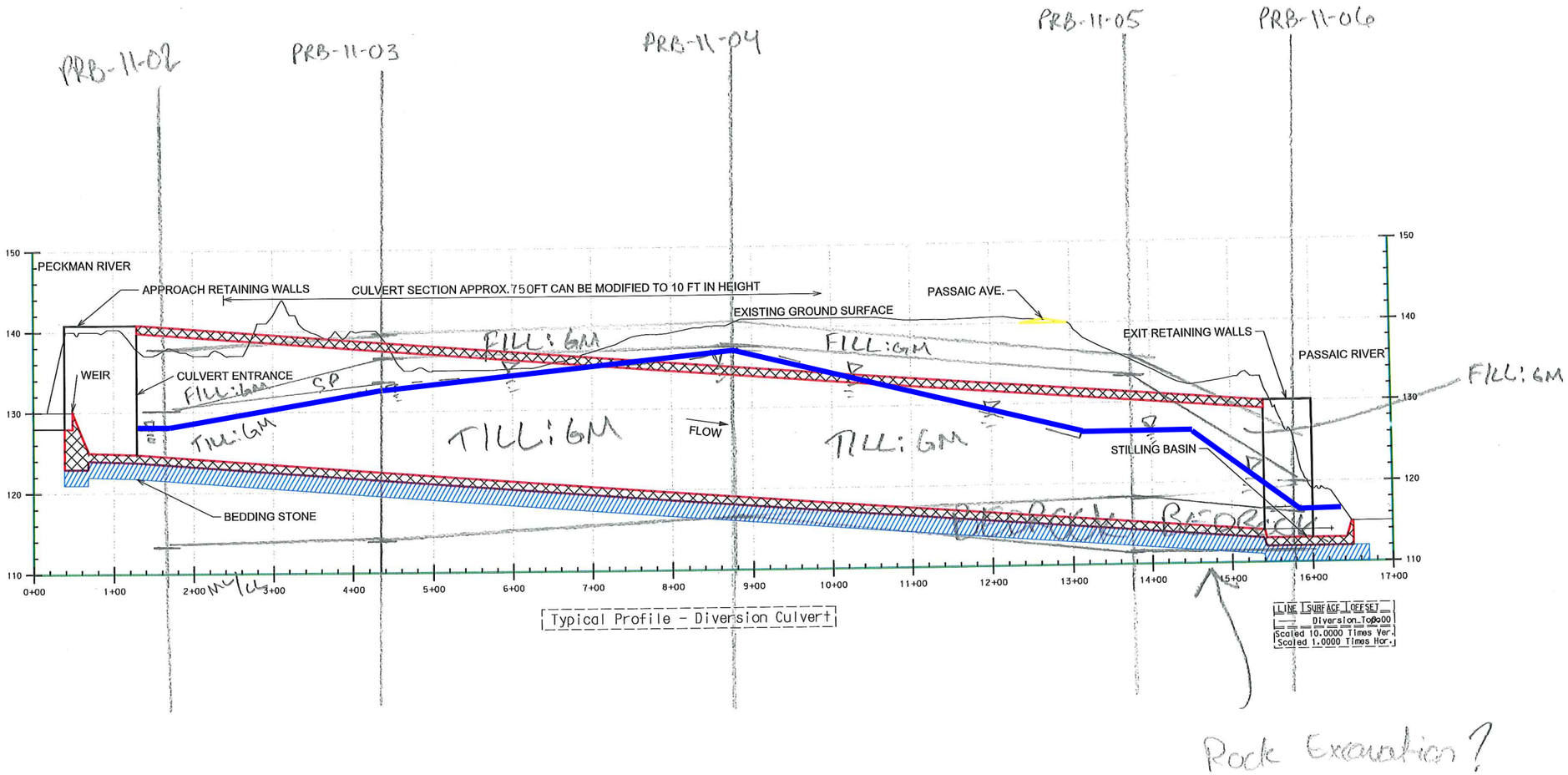
6 ft (125 pcf) + 3 ft (135 pcf) + 1 ft (135 pcf - 62.4 pcf)
+ 6 ft (150 pcf - 62.4 pcf)

$\sigma' = 1753$ psf σ' culvert = 1008 psf

1008 psf - 1753 psf $\boxed{-745 \text{ psf} = \text{net pressure decrease}}$

As seen from the analysis and general net pressure decrease on the soil as a result from the culvert construction, settlement will not be an issue.

A typical bearing capacity of this soil, taken from Table 1806.2 in the 2015 IBC, is 3,000 psf.



Typical Profile - Diversion Culvert

Approximate water level from test borings

EMO 7/11/2019 Peckman River Feasibility

SCALE 1"=75'

US Army Corps of Engineers

PECKMAN RIVER BASIN
ESSEX & PASSAIC COUNTIES
NEW JERSEY
FLOOD RISK MANAGEMENT STUDY

Sheet reference number

Submitted by: USERNAME Plot Date: DD-MMM-YYYY HHMM FILE NAME

Flood Risk Management Feasibility Study

Peckman River Basin
New Jersey

Draft Integrated Feasibility Report and Environmental Assessment

Appendix C3: Geotechnical Engineering

Attachment 3
GeoStudio Seepage – No Toe Drain

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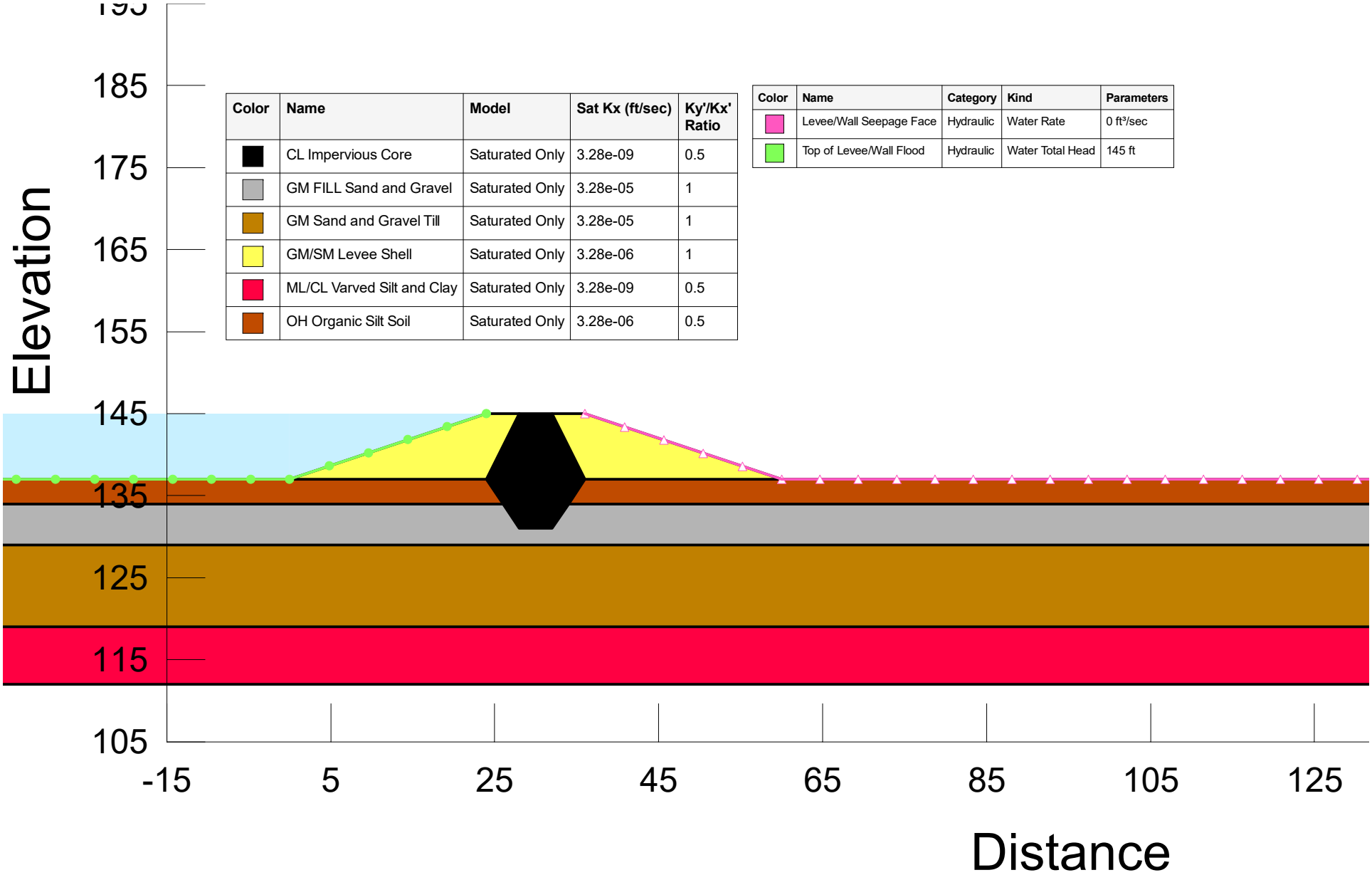
October 2019



Color	Name	Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	CL Impervious Core	Saturated Only	3.28e-09	0.5
	GM FILL Sand and Gravel	Saturated Only	3.28e-05	1
	GM Sand and Gravel Till	Saturated Only	3.28e-05	1
	GM/SM Levee Shell	Saturated Only	3.28e-06	1
	ML/CL Varved Silt and Clay	Saturated Only	3.28e-09	0.5
	OH Organic Silt Soil	Saturated Only	3.28e-06	0.5

Color	Name	Category	Kind	Parameters
	Levee/Wall Seepage Face	Hydraulic	Water Rate	0 ft³/sec
	Top of Levee/Wall Flood	Hydraulic	Water Total Head	145 ft

U.S. ARMY ENGINEER DISTRICT
NEW YORK

26 FEDERAL PLAZA
ROOM 2109
NEW YORK, NY
10278-0090

US Army Corps of Engineers®
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CONTRACT NO.:
DATE: 7/2/2019
DRAWN BY: EMO
CHECKED BY: EMF
FILE NAME: NAN Typical Levee Section.gsz

SEEPAGE INPUT
Levee, No Toe Drain

Flood Risk Management Feasibility Study
Peckman River Basin, New Jersey

PLATE
3-A1

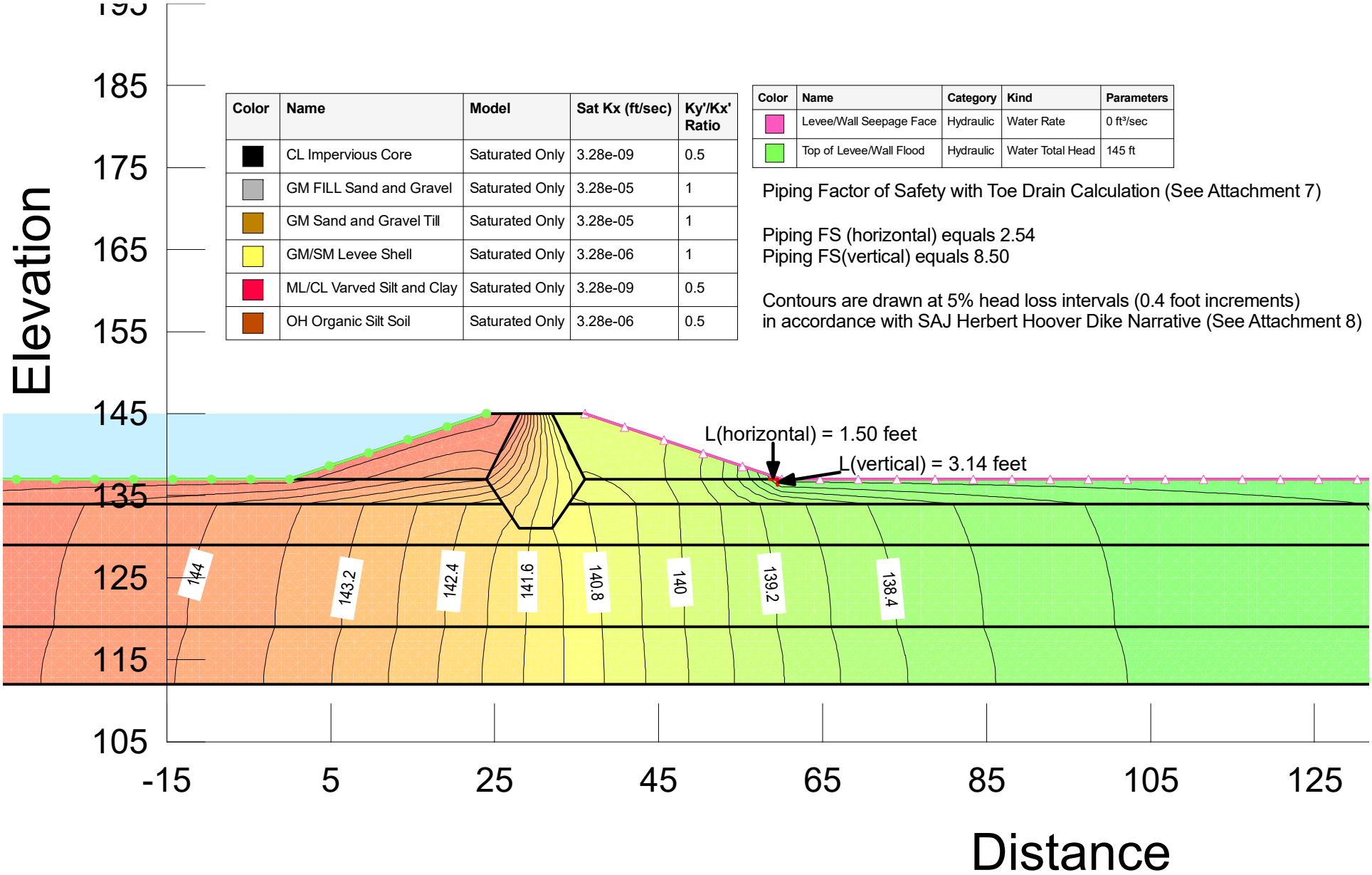
Color	Name	Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	CL Impervious Core	Saturated Only	3.28e-09	0.5
	GM FILL Sand and Gravel	Saturated Only	3.28e-05	1
	GM Sand and Gravel Till	Saturated Only	3.28e-05	1
	GM/SM Levee Shell	Saturated Only	3.28e-06	1
	ML/CL Varved Silt and Clay	Saturated Only	3.28e-09	0.5
	OH Organic Silt Soil	Saturated Only	3.28e-06	0.5

Color	Name	Category	Kind	Parameters
	Levee/Wall Seepage Face	Hydraulic	Water Rate	0 ft³/sec
	Top of Levee/Wall Flood	Hydraulic	Water Total Head	145 ft

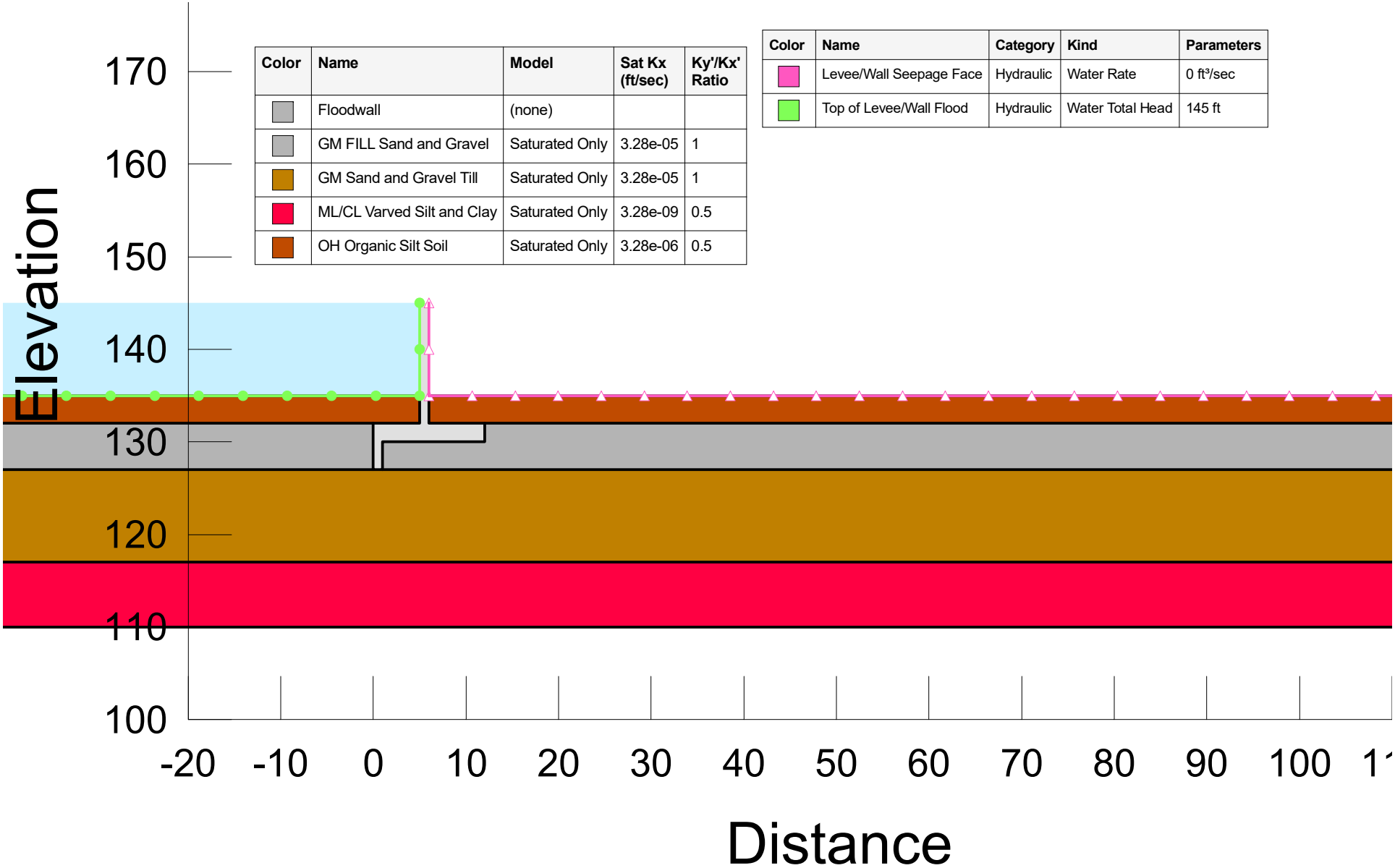
Piping Factor of Safety with Toe Drain Calculation (See Attachment 7)

Piping FS (horizontal) equals 2.54
Piping FS(vertical) equals 8.50

Contours are drawn at 5% head loss intervals (0.4 foot increments)
in accordance with SAJ Herbert Hoover Dike Narrative (See Attachment 8)



U.S. ARMY ENGINEER DISTRICT NEW YORK 26 FEDERAL PLAZA ROOM 2109 NEW YORK, NY 10278-0090	US Army Corps of Engineers® New York District	CONTRACT NO.: DATE: 7/2/2019 DRAWN BY: EMO CHECKED BY: EMF FILE NAME: NAN Typical Levee Section.gsz	SEEPAGE RESULTS Levee, No Toe Drain Flood Risk Management Feasibility Study Peckman River Basin, New Jersey	PLATE 3-A2



Color	Name	Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	Floodwall	(none)		
	GM FILL Sand and Gravel	Saturated Only	3.28e-05	1
	GM Sand and Gravel Till	Saturated Only	3.28e-05	1
	ML/CL Varved Silt and Clay	Saturated Only	3.28e-09	0.5
	OH Organic Silt Soil	Saturated Only	3.28e-06	0.5

Color	Name	Category	Kind	Parameters
	Levee/Wall Seepage Face	Hydraulic	Water Rate	0 ft^3/sec
	Top of Levee/Wall Flood	Hydraulic	Water Total Head	145 ft

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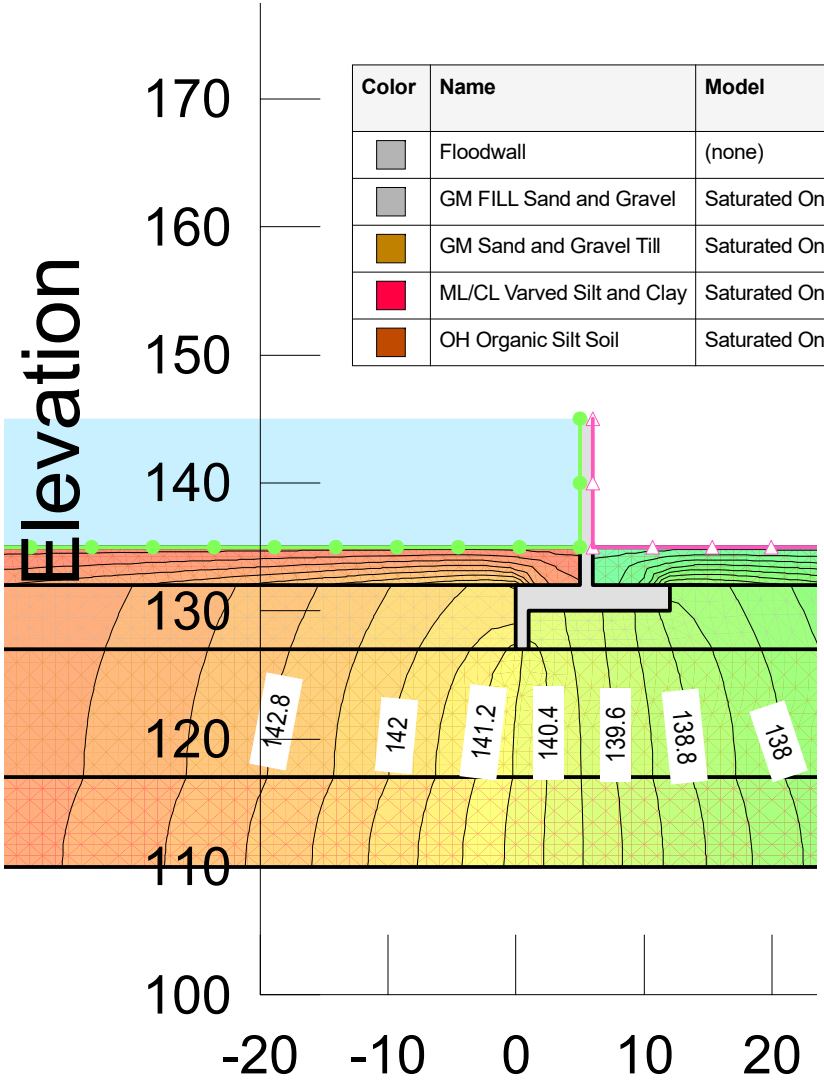
SEEPAGE INPUT
Floodwall, No Toe Drain

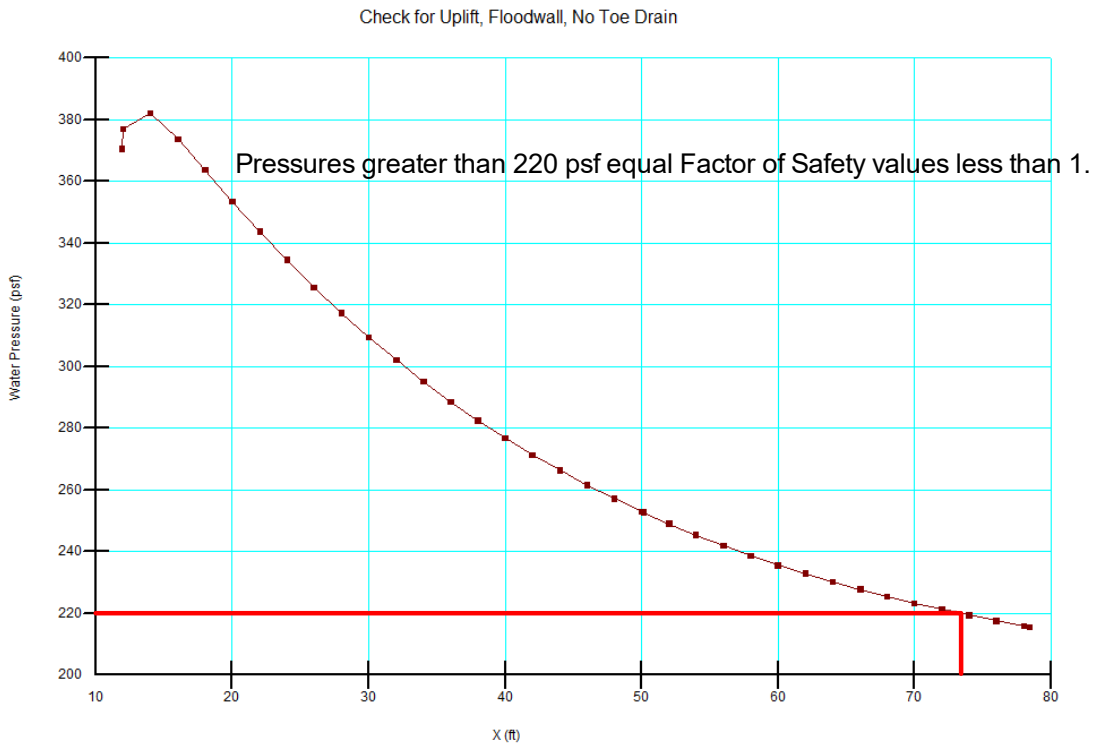
Flood Risk Management Feasibility Study
Peckman River Basin, New Jersey

PLATE
3-B1

Color	Name	Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	Floodwall	(none)		
	GM FILL Sand and Gravel	Saturated Only	3.28e-05	1
	GM Sand and Gravel Till	Saturated Only	3.28e-05	1
	ML/CL Varved Silt and Clay	Saturated Only	3.28e-09	0.5
	OH Organic Silt Soil	Saturated Only	3.28e-06	0.5

Color	Name	Category	Kind	Parameters
	Levee/Wall Seepage Face	Hydraulic	Water Rate	0 ft³/sec
	Top of Levee/Wall Flood	Hydraulic	Water Total Head	145 ft





Contours are drawn at 5% head loss intervals (0.4 foot increments)
in accordance with SAJ Herbert Hoover Dike Narrative (See Attachment 8)

Distance

U.S. ARMY ENGINEER DISTRICT NEW YORK 26 FEDERAL PLAZA ROOM 2109 NEW YORK, NY 10278-0090	US Army Corps of Engineers® New York District	CONTRACT NO.: DATE: 7/2/2019 DRAWN BY: EMO CHECKED BY: EMF FILE NAME: NAN Typical Levee Section.gsz	SEEPAGE RESULTS Floodwall, No Toe Drain Flood Risk Management Feasibility Study Peckman River Basin, New Jersey	PLATE 3-B2

Flood Risk Management Feasibility Study

Peckman River Basin
New Jersey

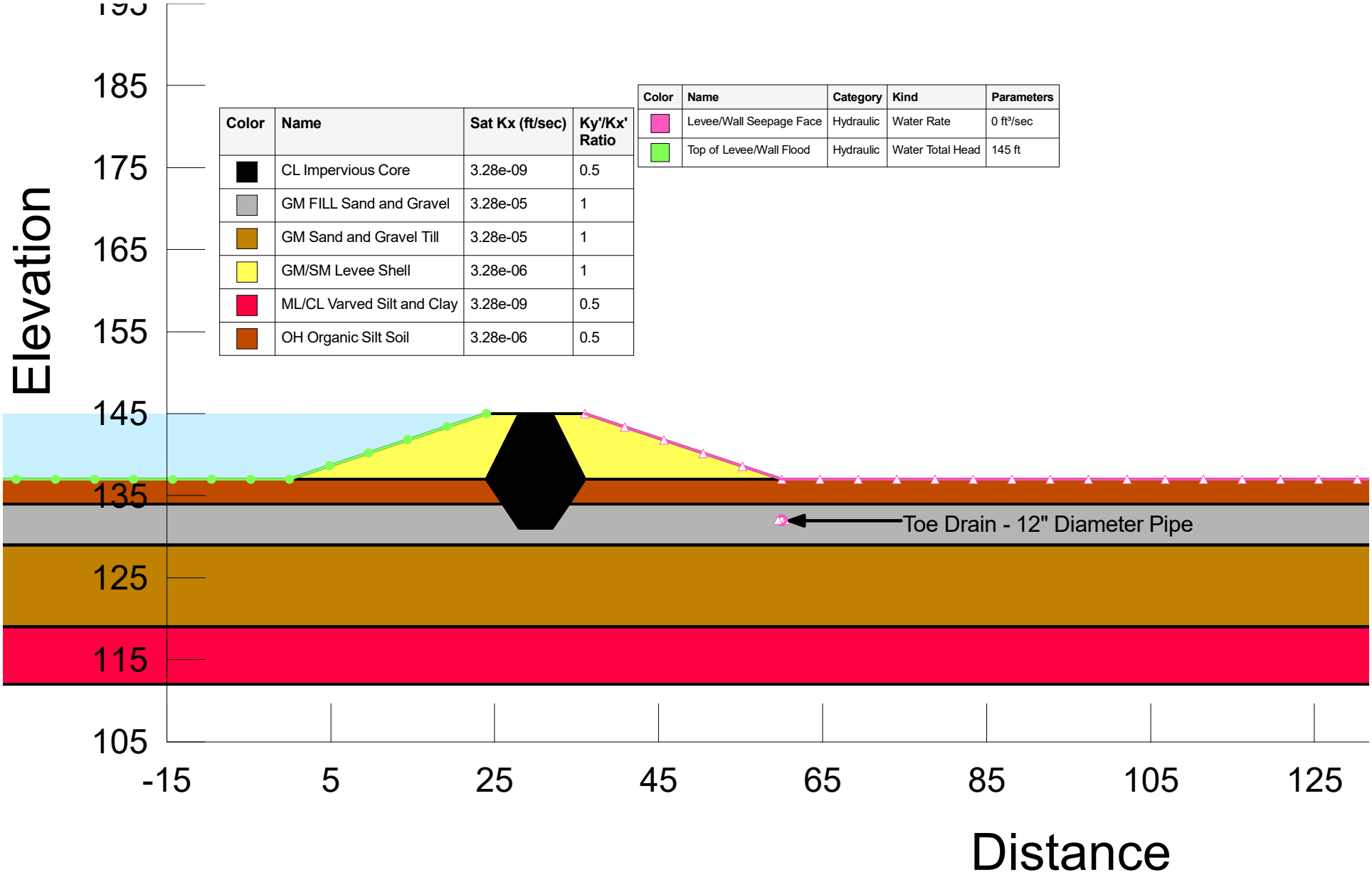
Draft Integrated Feasibility Report and Environmental Assessment

Appendix C3: Geotechnical Engineering

Attachment 4
GeoStudio Seepage – Toe Drain

US Army Corps of Engineers®
New York District

October 2019



Color	Name	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	CL Impervious Core	3.28e-09	0.5
	GM FILL Sand and Gravel	3.28e-05	1
	GM Sand and Gravel Till	3.28e-05	1
	GM/SM Levee Shell	3.28e-06	1
	ML/CL Varved Silt and Clay	3.28e-09	0.5
	OH Organic Silt Soil	3.28e-06	0.5

Color	Name	Category	Kind	Parameters
	Levee/Wall Seepage Face	Hydraulic	Water Rate	0 ft³/sec
	Top of Levee/Wall Flood	Hydraulic	Water Total Head	145 ft

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SEEPAGE INPUT
Levee, Toe Drain

Flood Risk Management Feasibility Study
Peckman River Basin, New Jersey

PLATE
4-A1

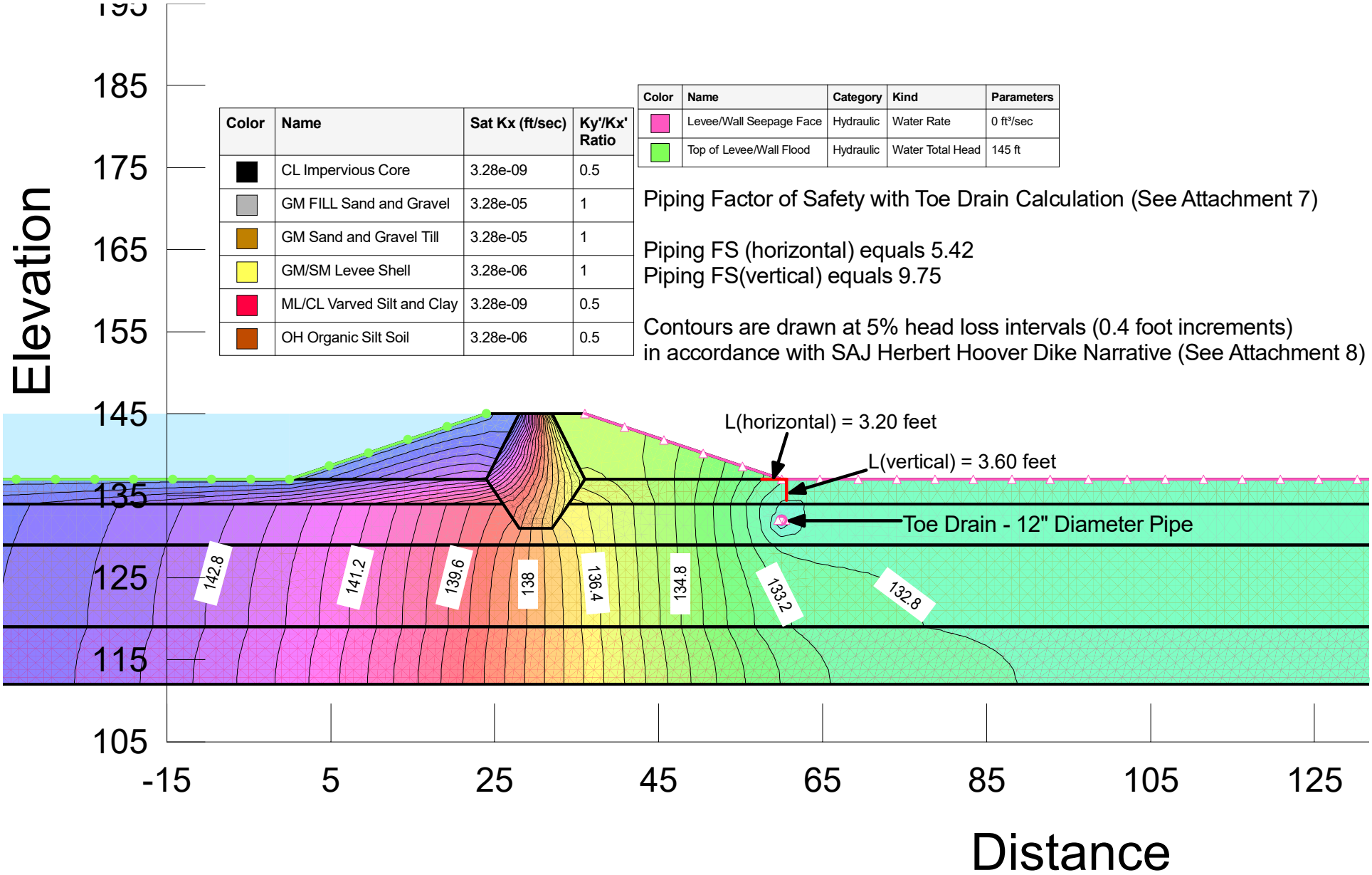
Color	Name	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	CL Impervious Core	3.28e-09	0.5
	GM FILL Sand and Gravel	3.28e-05	1
	GM Sand and Gravel Till	3.28e-05	1
	GM/SM Levee Shell	3.28e-06	1
	ML/CL Varved Silt and Clay	3.28e-09	0.5
	OH Organic Silt Soil	3.28e-06	0.5

Color	Name	Category	Kind	Parameters
	Levee/Wall Seepage Face	Hydraulic	Water Rate	0 ft³/sec
	Top of Levee/Wall Flood	Hydraulic	Water Total Head	145 ft

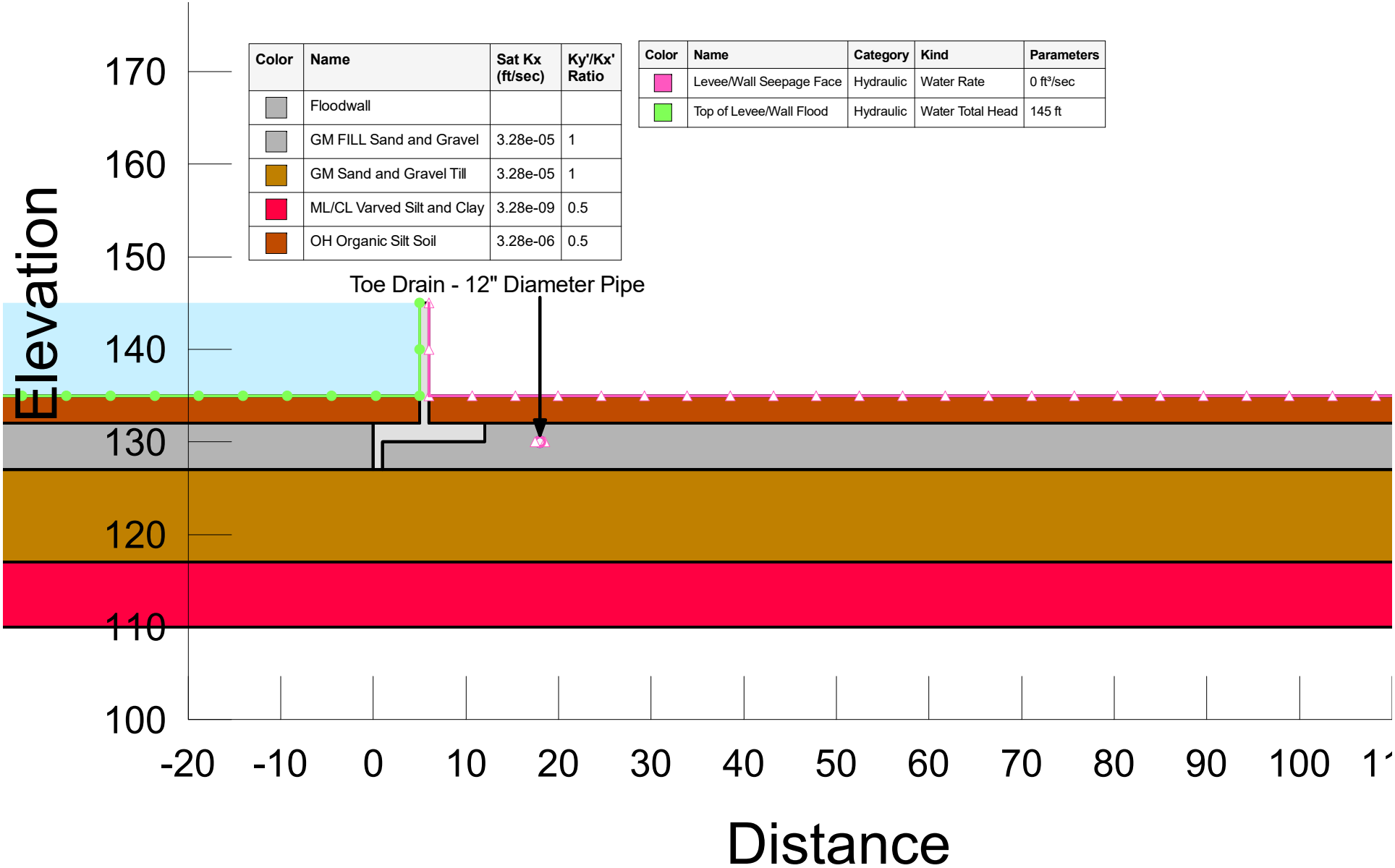
Piping Factor of Safety with Toe Drain Calculation (See Attachment 7)

Piping FS (horizontal) equals 5.42
Piping FS(vertical) equals 9.75

Contours are drawn at 5% head loss intervals (0.4 foot increments)
in accordance with SAJ Herbert Hoover Dike Narrative (See Attachment 8)



U.S. ARMY ENGINEER DISTRICT NEW YORK 26 FEDERAL PLAZA ROOM 2109 NEW YORK, NY 10278-0090	US Army Corps of Engineers® New York District	CONTRACT NO.: DATE: 7/2/2019 DRAWN BY: EMO CHECKED BY: EMF FILE NAME: NAN Typical Levee with Toe Drain.gsz	SEEPAGE RESULTS Levee, Toe Drain Flood Risk Management Feasibility Study Peckman River Basin, New Jersey	PLATE 4-A2



Color	Name	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	Floodwall		
	GM FILL Sand and Gravel	3.28e-05	1
	GM Sand and Gravel Till	3.28e-05	1
	ML/CL Varved Silt and Clay	3.28e-09	0.5
	OH Organic Silt Soil	3.28e-06	0.5

Color	Name	Category	Kind	Parameters
	Levee/Wall Seepage Face	Hydraulic	Water Rate	0 ft³/sec
	Top of Levee/Wall Flood	Hydraulic	Water Total Head	145 ft

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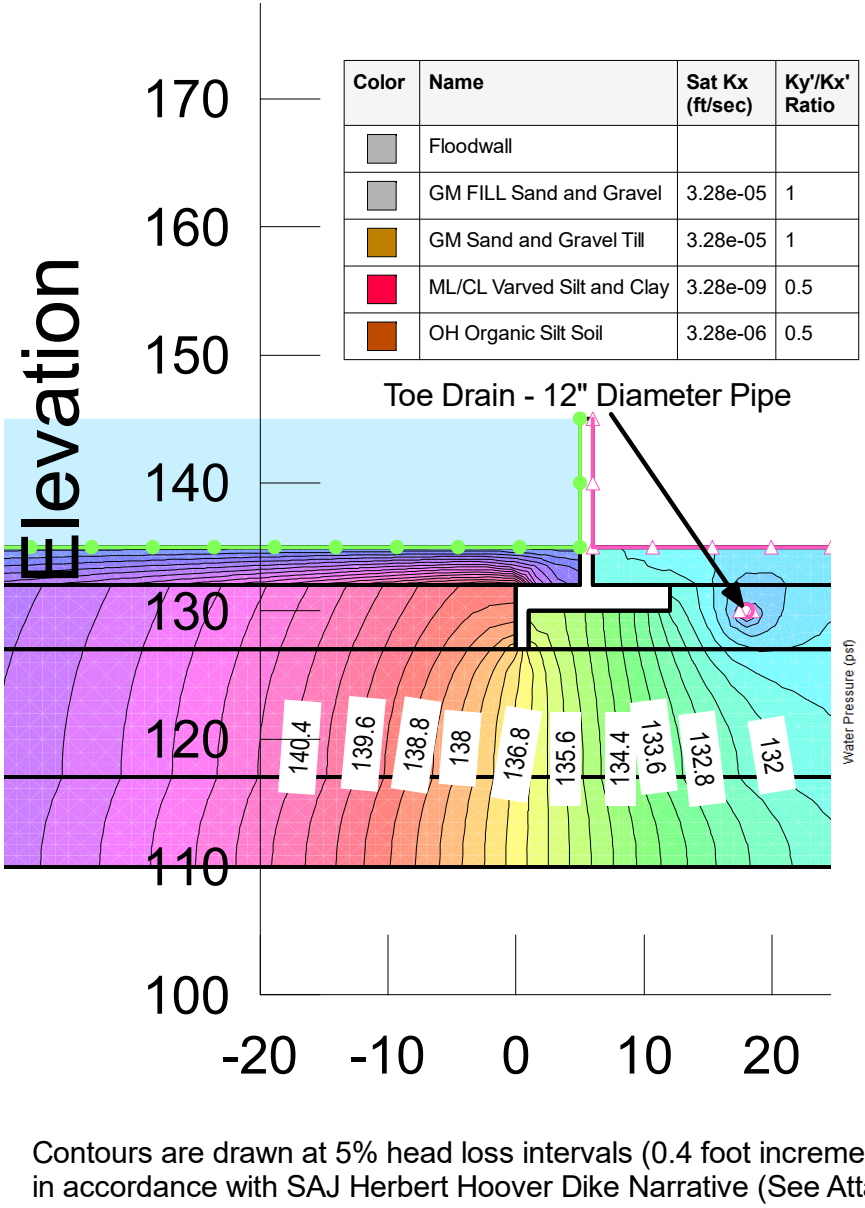
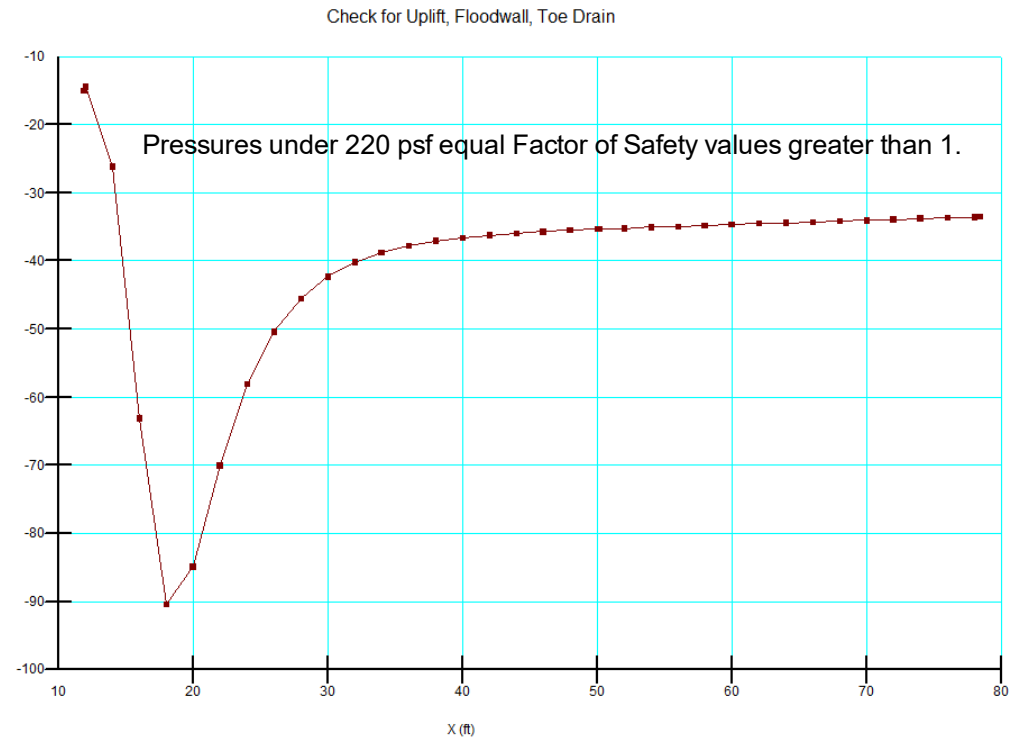
Color	Name	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	Floodwall		
	GM FILL Sand and Gravel	3.28e-05	1
	GM Sand and Gravel Till	3.28e-05	1
	ML/CL Varved Silt and Clay	3.28e-09	0.5
	OH Organic Silt Soil	3.28e-06	0.5

Color	Name	Category	Kind	Parameters
	Levee/Wall Seepage Face	Hydraulic	Water Rate	0 ft³/sec
	Top of Levee/Wall Flood	Hydraulic	Water Total Head	145 ft





Contours are drawn at 5% head loss intervals (0.4 foot increments)
in accordance with SAJ Herbert Hoover Dike Narrative (See Attachment 8)

U.S. ARMY ENGINEER DISTRICT NEW YORK 26 FEDERAL PLAZA ROOM 2109 NEW YORK, NY 10278-0090	US Army Corps of Engineers® New York District	CONTRACT NO.: DATE: 7/2/2019 DRAWN BY: EMO CHECKED BY: EMF FILE NAME: NAN Typical Levee with Toe Drain.gsz	SEEPAGE RESULTS Floodwall, Toe Drain Flood Risk Management Feasibility Study Peckman River Basin, New Jersey	PLATE 4-B2

Flood Risk Management Feasibility Study

Peckman River Basin
New Jersey

Draft Integrated Feasibility Report and Environmental Assessment

Appendix C3: Geotechnical Engineering

Attachment 5
GeoStudio Slope Stability – No Toe Drain

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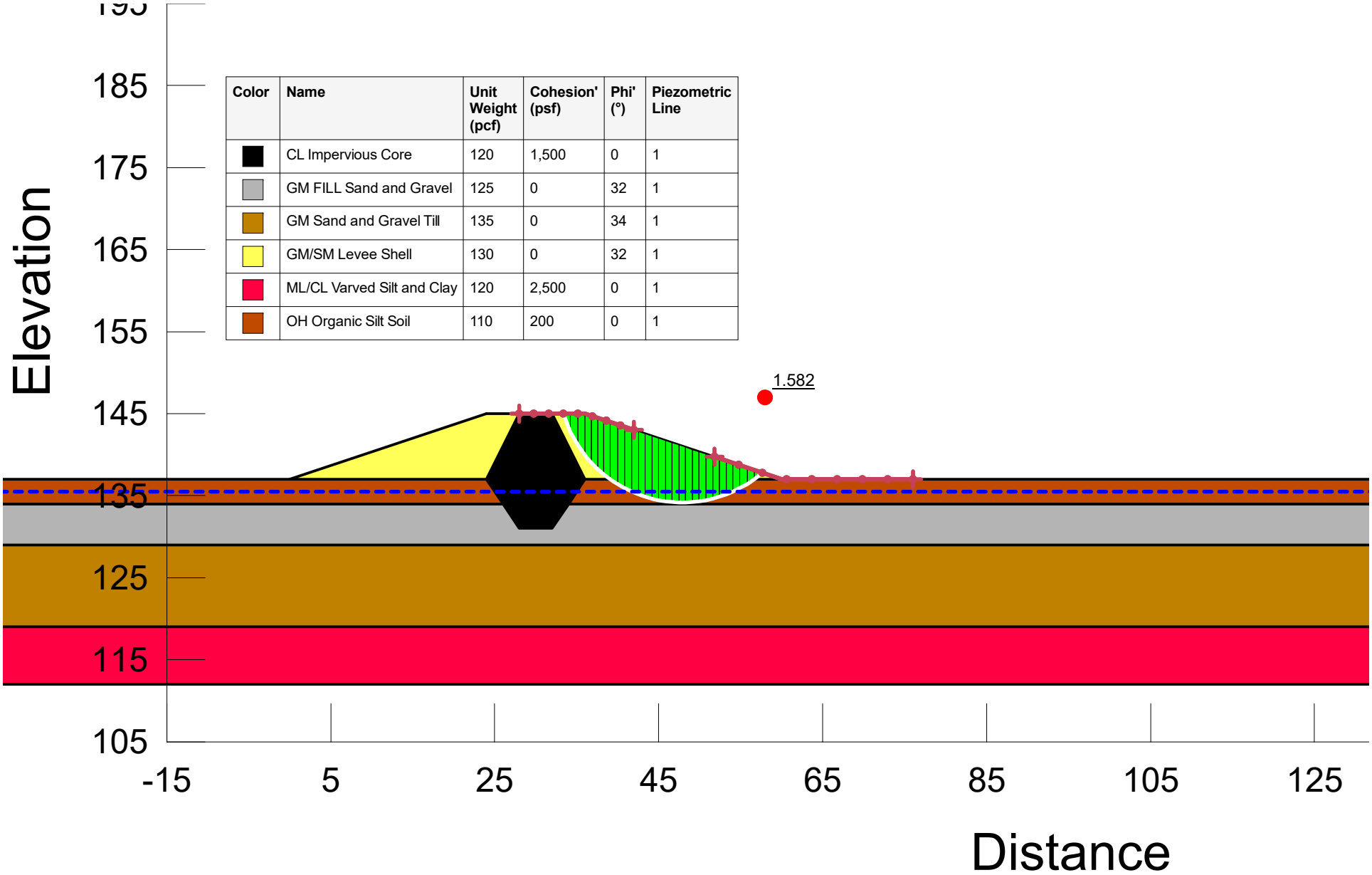

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Piezometric Line
	CL Impervious Core	120	1,500	0	1
	GM FILL Sand and Gravel	125	0	32	1
	GM Sand and Gravel Till	135	0	34	1
	GM/SM Levee Shell	130	0	32	1
	ML/CL Varved Silt and Clay	120	2,500	0	1
	OH Organic Silt Soil	110	200	0	1

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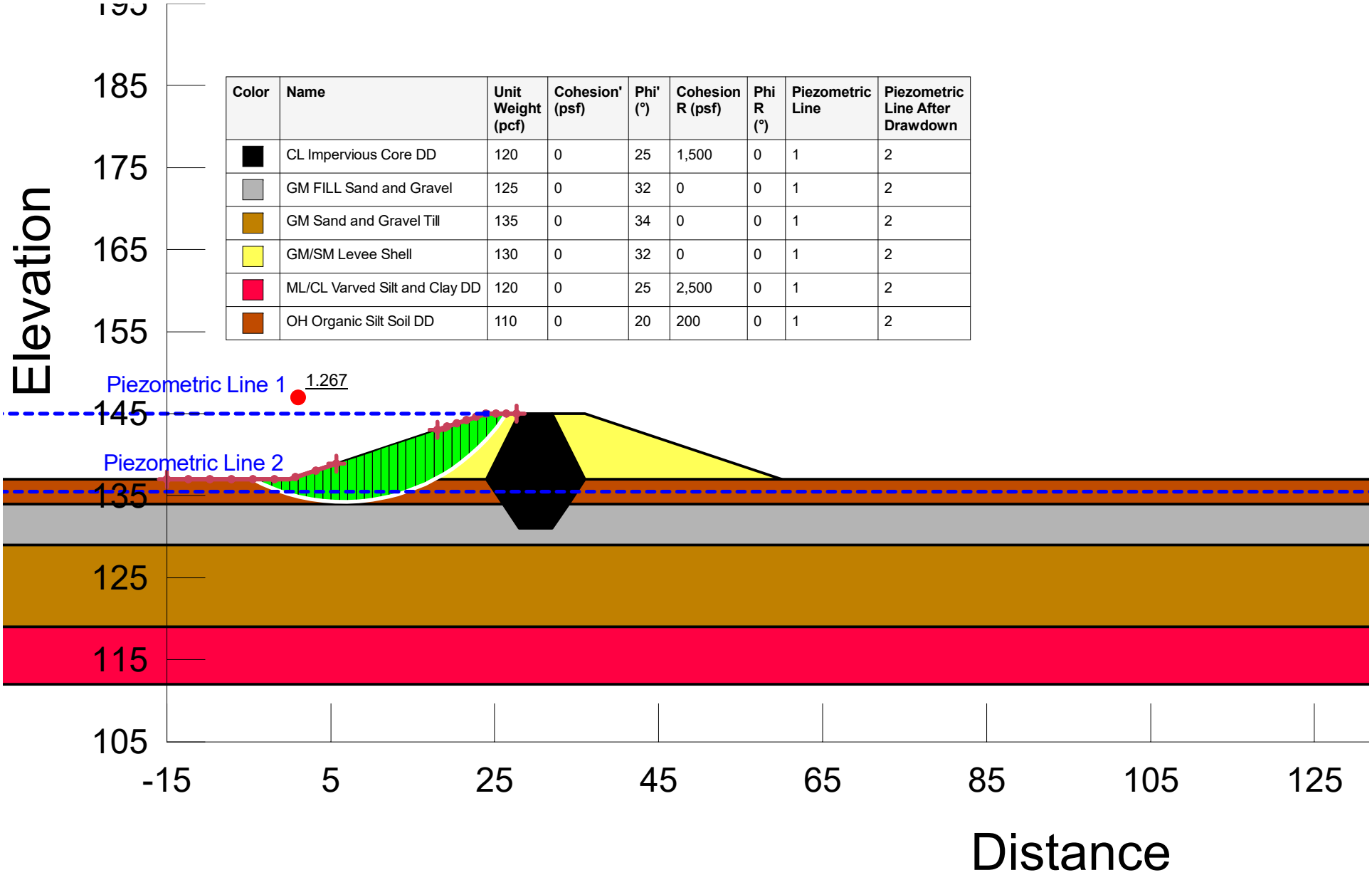
FILE NAME: NAN Typical Levee Section.gsz

SLOPE STABILITY RESULTS
Case I - End of Construction
Method: Spencer

Flood Risk Management Feasibility Study
Peckman River Basin, New Jersey

PLATE
5A



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion R (psf)	Phi R (°)	Piezometric Line	Piezometric Line After Drawdown
	CL Impervious Core DD	120	0	25	1,500	0	1	2
	GM FILL Sand and Gravel	125	0	32	0	0	1	2
	GM Sand and Gravel Till	135	0	34	0	0	1	2
	GM/SM Levee Shell	130	0	32	0	0	1	2
	ML/CL Varved Silt and Clay DD	120	0	25	2,500	0	1	2
	OH Organic Silt Soil DD	110	0	20	200	0	1	2

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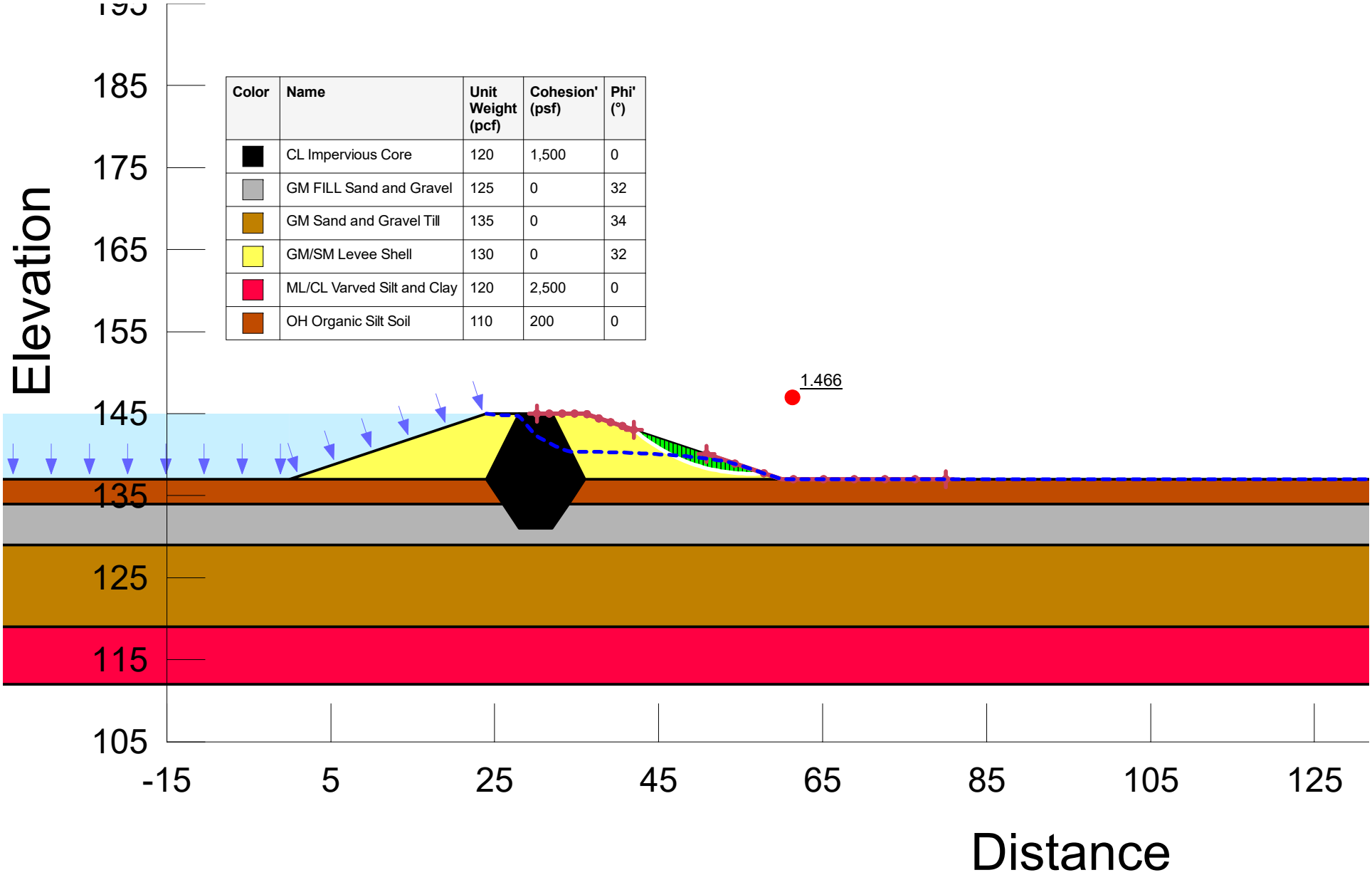
FILE NAME: NAN Typical Levee Section.gsz

SLOPE STABILITY RESULTS
Case II - Rapid Drawdown
Method: Spencer

Flood Risk Management Feasibility Study
Peckman River Basin, New Jersey

PLATE
5B



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	CL Impervious Core	120	1,500	0
	GM FILL Sand and Gravel	125	0	32
	GM Sand and Gravel Till	135	0	34
	GM/SM Levee Shell	130	0	32
	ML/CL Varved Silt and Clay	120	2,500	0
	OH Organic Silt Soil	110	200	0

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FILE NAME: NAN Typical Levee Section.gsz

SLOPE STABILITY RESULTS
Case III - Steady State
Method: Spencer

Flood Risk Management Feasibility Study
Peckman River Basin, New Jersey

PLATE
5C

Flood Risk Management Feasibility Study

Peckman River Basin
New Jersey

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Appendix C3: Geotechnical Engineering

Attachment 6
GeoStudio Slope Stability – Toe Drain

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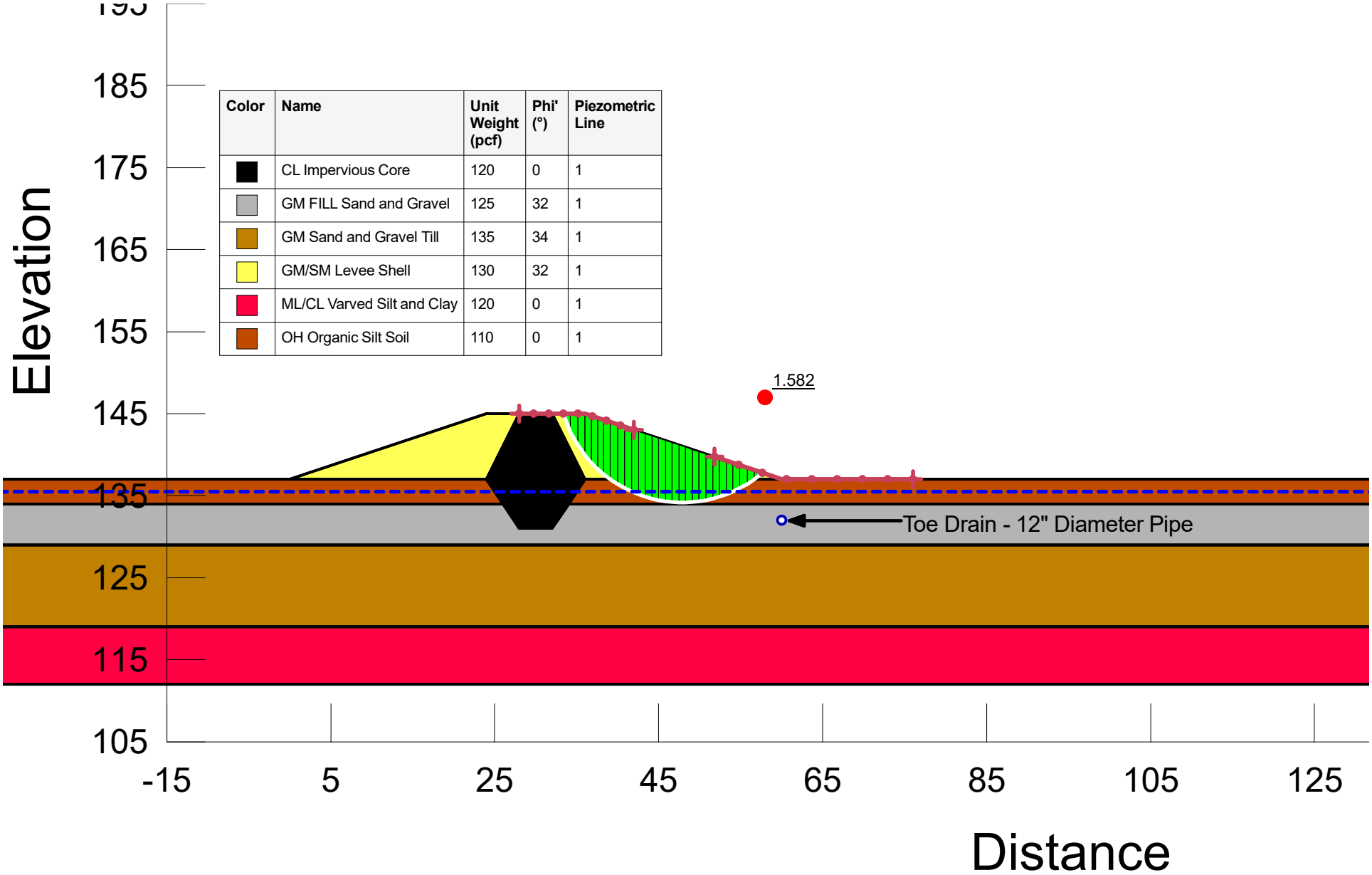

Color	Name	Unit Weight (pcf)	Phi' (°)	Piezometric Line
	CL Impervious Core	120	0	1
	GM FILL Sand and Gravel	125	32	1
	GM Sand and Gravel Till	135	34	1
	GM/SM Levee Shell	130	32	1
	ML/CL Varved Silt and Clay	120	0	1
	OH Organic Silt Soil	110	0	1

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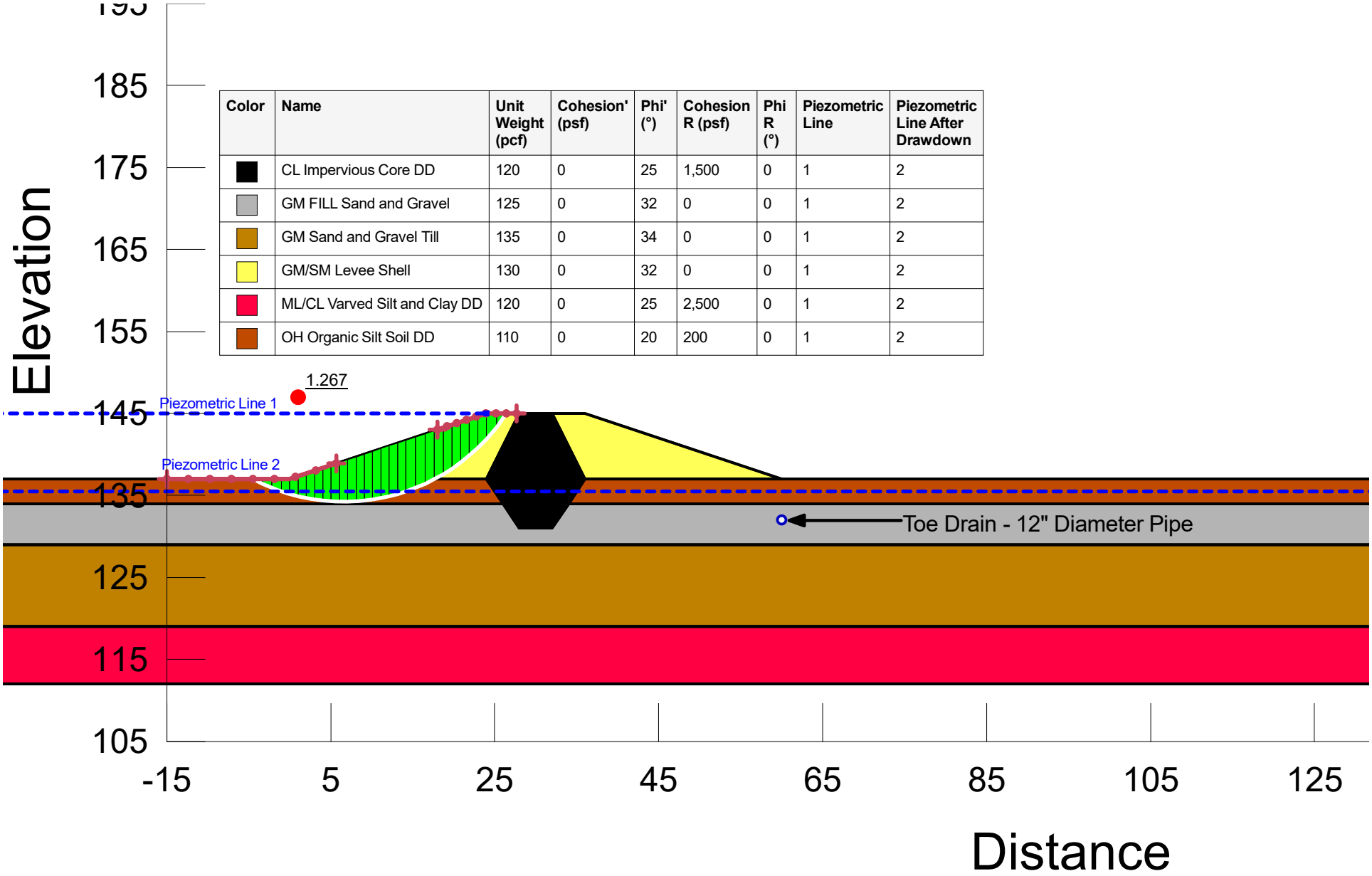
FILE NAME: NAN Typical Levee with Toe Drain.gsz

SLOPE STABILITY RESULTS
Case I - End of Construction, Toe Drain
Method: Spencer

Flood Risk Management Feasibility Study
Peckman River Basin, New Jersey

PLATE
6A



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion R (psf)	Phi R (°)	Piezometric Line	Piezometric Line After Drawdown
	CL Impervious Core DD	120	0	25	1,500	0	1	2
	GM FILL Sand and Gravel	125	0	32	0	0	1	2
	GM Sand and Gravel Till	135	0	34	0	0	1	2
	GM/SM Levee Shell	130	0	32	0	0	1	2
	ML/CL Varved Silt and Clay DD	120	0	25	2,500	0	1	2
	OH Organic Silt Soil DD	110	0	20	200	0	1	2

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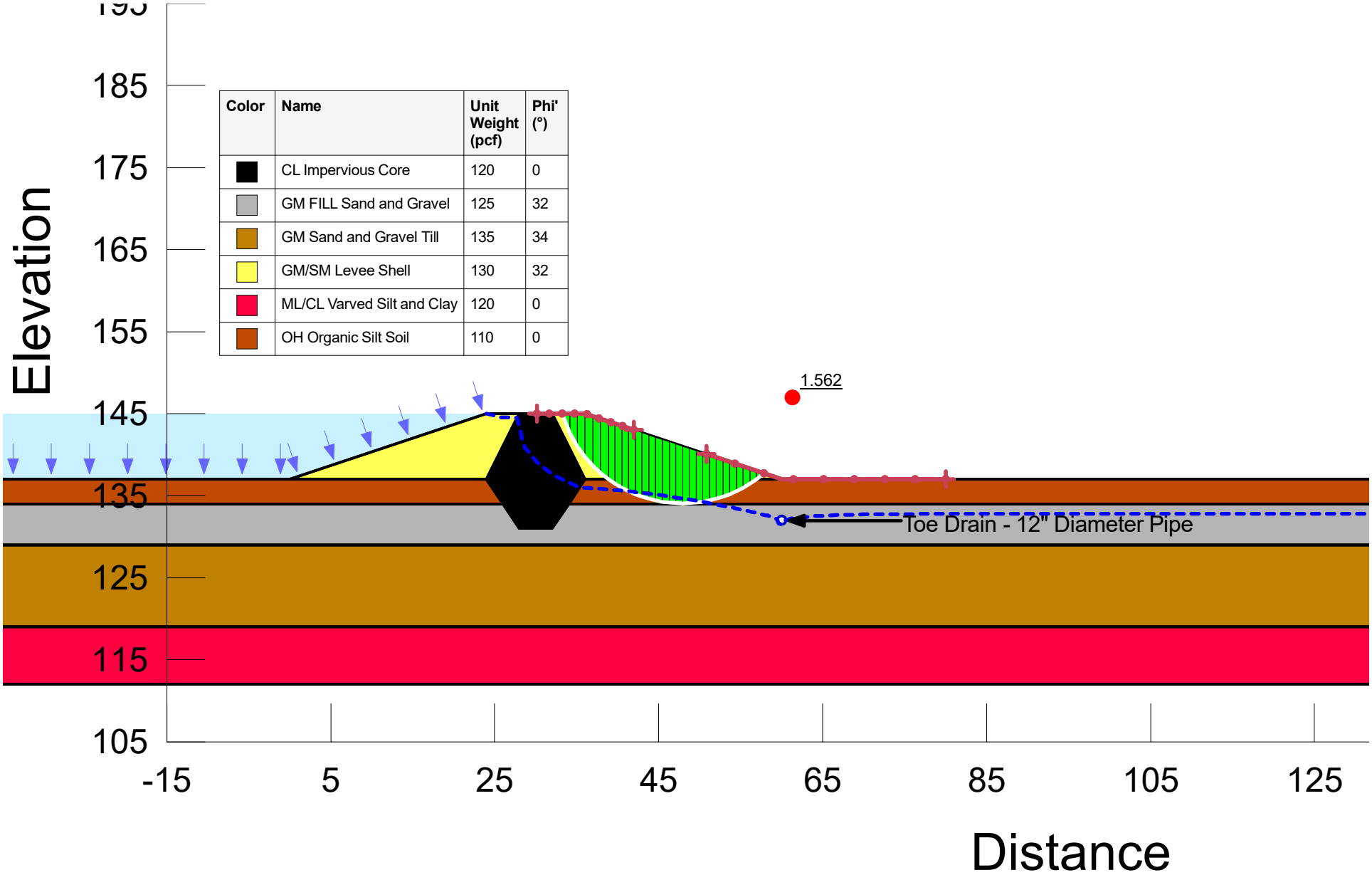
FILE NAME: NAN Typical Levee with Toe Drain.gsz

SLOPE STABILITY RESULTS
Case II - Rapid Drawdown, Toe Drain
Method: Spencer

Flood Risk Management Feasibility Study
Peckman River Basin, New Jersey

PLATE
6B



Color	Name	Unit Weight (pcf)	Phi' (°)
	CL Impervious Core	120	0
	GM FILL Sand and Gravel	125	32
	GM Sand and Gravel Till	135	34
	GM/SM Levee Shell	130	32
	ML/CL Varved Silt and Clay	120	0
	OH Organic Silt Soil	110	0

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FILE NAME: NAN Typical Levee with Toe Drain.gsz

SLOPE STABILITY RESULTS
Case III - Steady State, Toe Drain
Method: Spencer

Flood Risk Management Feasibility Study
Peckman River Basin, New Jersey

PLATE
6C

Flood Risk Management Feasibility Study

Peckman River Basin
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Appendix C3: Geotechnical Engineering

Attachment 7
Piping Factor of Safety Calculations

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October 2019

FS Piping by 5% Method

Peckman River Feasibilty - Typical Levee Section - Seepage and Slope Stability Analysis Results (Max Head)

	Geometry					Pool Elevation								Seepage Analyses			Slope Stability
Analysis Condition	Upstream Slope	Downstream Slope	Crest Width	Feature Being Analyzed	Remediation Feature ??	Upstream	Downstream	Exit Face Material #	$L_{horizontal}$ (ft)	$L_{vertical}$ (ft)	Δh	$i_{actual\ Hor}$ ($\Delta h/L$)	$i_{actual\ Vert}$ ($\Delta h/L$)	$FS_{Piping\ Horizontal}$ ($i_{cr\ hor}/i_{actual}$)	$FS_{Piping\ Vertical}$ ($i_{cr\ vert}/i_{actual}$)	Seepage Quantities (ft3/d/ft)	$FS_{Slope\ Stability}$
No Toe Drain	3 to 1	3 to 1	12	Toe of Embank.	No	145	137	1	1.50	3.14	0.40	0.27	0.13	2.54	8.50		1.46
Toe Drain	3 to 1	3 to 1	12	Toe of Embank.	No	145	137	1	3.20	3.60	0.40	0.13	0.11	5.42	9.75		1.56

Material 1	Embankment	Material 2		Material 3		Material 4	
γ_{SAT} =	130	γ_{SAT} =		γ_{SAT} =		γ_{SAT} =	
Φ' =	32	Φ' =		Φ' =		Φ' =	
$i_{cr\ vert}$ =	1.08	$i_{cr\ vert}$ =		$i_{cr\ vert}$ =		$i_{cr\ vert}$ =	
$i_{cr\ hor}$ =	0.68	$i_{cr\ hor}$ =		$i_{cr\ hor}$ =		$i_{cr\ hor}$ =	

Blue, Green, and Gray = Input Value
White and Yellow = Calculated Value

Notes:
1. Infinite elements in the neg and pos x direction were used.
2. Gradients were calculated in the Geostudio SEEP/W analysis by first graphing total head contours equal to 5% of head across embankment, then measuring the distance between the exit face node and the nearest total head contour.
3. Models were based on borings; PRB-11-01, PRB-11-02, PRB-11-03, PRB-11-04, PRB-11-05, PRB-11-06, PRB-11-07, and PRB-11-08.
4. Upstream pool elevation was not modeled with drainage gravel because FS requirements were already met.

Blue, Yellow, and Gray are input boxes
White boxes are calculated

Flood Risk Management Feasibility Study

Peckman River Basin
New Jersey

Draft Integrated Feasibility Report and Environmental Assessment

Appendix C3: Geotechnical Engineering

Attachment 8
SAJ Herbert Hoover Dike Loadings and Factors of Safety for Use in Seepage and Stability Analysis

US Army Corps of Engineers®
New York District

October 2019

CESAJ-EN-G

MEMORANDUM FOR RECORD

SUBJECT: Herbert Hoover Dike Loadings and Factors of Safety for Use in Seepage and Stability Analyses

1.0 Background. The need for this guidance document arises from the challenges associated with interpreting and applying existing USACE guidance and criteria to Herbert Hoover Dike (HHD). It will also serve to offer a consistent and unifying set of criteria that current and future HHD design teams, both within USACE and by its A/E designers, shall use during designs and analyses for both seepage and slope stability.

2.0 USACE Guidance. Existing USACE guidance regarding seepage criteria is somewhat vague and is currently being revised through HQUSACE. Lake Okeechobee is operated similar to a reservoir, meaning that water is usually impounded on HHD for prolonged periods. Since HHD was originally authorized as a levee, the designer is first referred to EM 1110-2-1913 *Design and Construction of Levees*, paragraph 1-5 a.1. which states "embankments that are subject to water loading for prolonged periods (longer than normal flood protection requirements) or permanently should be designed in accordance with earth dam criteria rather than the levee criteria given herein." This statement causes the designer to refer to EM 1110-2-1901 *Seepage Analysis and Control for Dams*, which states that design Factors of Safety (FS) typically range between 1.5 and 15. ETL 1110-2-569 *Design Guidance for Levee Underseepage* offers interim design guidance for levee underseepage. Paragraph 6 Recommended Design Guidance states that deviation from the [ETL 569] guidance is acceptable when based on sound engineering judgment and experience.

3.0 Seepage. The need for seepage control is determined from a study of the thickness and characteristics of the natural blanket material (peat, silt, and clay), the thickness and permeability of the foundation (sands and limestone), the effective seepage entrance and exit, and the maximum differential head on the Dike created by the design Lake elevation. Seepage enters the foundation beneath and through the Dike, through the bed of the Lake, and through the rim canal or borrow pit. This results in a hydraulic gradient and an excess pressure head at the base of landside blanket as the water flows from the Lake to the landward side of the Dike. The pressures that develop landward of the Dike during sustained flood stages may, unless controlled, cause uplift, or rupture, of the blanket areas resulting in a concentration of seepage flow and the formation of sand boils which could potentially lead to catastrophic failure. The semi-confining blanket is considered infinite for analysis and design purposes in extent along many project reaches landward of the Dike toe. Measures that provide pressure relief, drainage, stability, filter functionality, or combinations of each should be considered to control the potentially adverse effects of excess seepage pressures in the permeable foundation and aquifer below the semi-confining blanket where present, and as well to control potentially adverse situations where a blanket is absent.

4.0. SAJ Guidance on Seepage Load Cases and Factors of Safety. The authorized level of protection for HHD is the Standard Project Flood (SPF), which, for Lake Okeechobee, corresponds to an elevation slightly above 26 feet, referencing the National Geodetic Vertical Datum of 1929 (NGVD29). This elevation was derived through hydrologic analyses performed for preparation of the Major Rehabilitation Report, approved in November 2000. The conversion to North American Vertical Datum of 1988 (NAVD88) is somewhat variable around Lake Okeechobee due to the large surface area and ranges from about 1.1 to 1.4 feet less than NGVD29. For the purposes of geotechnical analyses, the conversion of the SPF event to an NAVD88 elevation of 25.0 is deemed to be sufficiently accurate. This corresponds to the static pool and applies to all reaches.

4.1. Factors of safety for uplift and piping will be evaluated at sufficient locations to determine minimum FS for each analysis case. Locations to be evaluated will include the toe area of Dike, the modified toe of slope if slope modifications are made, and any low areas or areas of thinner blanket that typically occur at existing or proposed drainage features or excavations. A typical cross-section for analyses is shown in Figure 1.

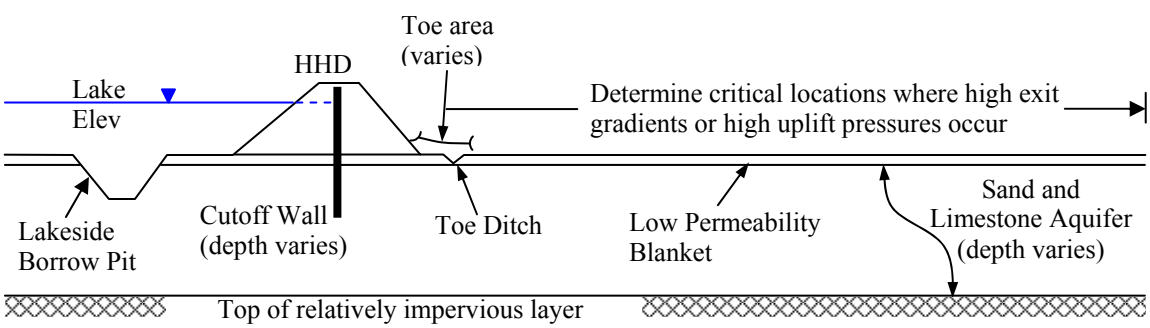


Figure 1: Typical cross-section of HHD for use in evaluation of seepage analyses

4.2. It has been estimated that the SPF elevation has a return period of approximately 935 years. The most extreme hydrologic event which could occur for the Lake corresponds to the Probable Maximum Flood (PMF) which will be developed as part of an on-going Hydrologic Frequency Analysis not expected to be completed until late 2009. For the purposes of on-going analyses, the maximum possible static pool to be analyzed for risk management purposes should correspond to the lowest perimeter crest elevation which is about 30.0 ft-NAVD88, above which overtopping would begin to occur on the western side of Lake Okeechobee.

4.2.1. Piping. The potential failure mode of most concern for HHD is piping. The FS against piping of foundation soils due to under seepage, FS_{piping}, shall be calculated (see ex. in Figure 2) in accordance with Equation 1 as adapted from Chapter 4 of EM 1901. It is noted that this is an effective stress calculation. Design FS values in Table 2 were selected to account for uncertainty with material properties in combination with consequences and HHD's classification as a high hazard dam. It is noted that the FS are

well within the range quoted by EM 1901 guidance as well as consistent with recommendations contained in Harry Cedergren's book titled *Seepage, Drainage, and Flow Nets.*

$$FS_{piping} = \frac{i_{cv}}{i_v} = \frac{(\gamma_s - \gamma_w) * z}{\gamma_w * h} \qquad (1)$$

where: i_{cv} = critical vertical gradient for flotation of foundation soil particles
i_v = average vertical hydraulic gradient (h/z)
γ_w = unit weight of water
γ_s = saturated unit weight of foundation soil susceptible to piping
z = distance over which head loss is 5 % of the total head loss across the dike; e.g. if the lake elevation is 20 ft, and the ground surface elevation is 0 ft at a location of emerging seepage, then plot head contour lines with a 1 ft interval and measure z as the orthogonal distance between the 1 ft contour line and the ground surface
h = excess head above ground surface (or tail water elevation if it is higher than ground surface) to be dissipated across z

For all soils, with the notable exceptions of peat and intact limestone, the critical vertical gradient, i_{cv}, shall be calculated in accordance with Equation 2, also as adapted from Chapter 4 of EM 1901. Limestone, being rock, is not pipe-able, and the potential movement of peat is addressed below in uplift computations.

$$i_{cv} = \frac{\gamma_s - \gamma_w}{\gamma_w} \qquad (2)$$

The FS against piping of embankment and foundation soils due to through seepage, also to be considered as an FS_{piping} value, shall be calculated in accordance with Equation 3 as adapted from ERDC Technical Report GL-97-2. This approach shall be applied for granular (piping-prone) soils with horizontal emerging seepage located at the dike toe and/or at a toe ditch/swale location.

$$FS_{piping} = \frac{i_{ch}}{i_h} = \frac{(\gamma_s - \gamma_w)\tan\phi * z}{\gamma_w * h} \qquad (3)$$

where: i_{ch} $= i_{cv} \tan\phi$ = critical hydraulic gradient for horizontal flow
i_h = average horizontal hydraulic gradient (h/z)
ϕ = angle of internal friction for soil
h = head to be dissipated across z

4.2.2. Uplift. Where the top soil strata exhibit vertical permeabilities that are significantly less than the horizontal permeability of the underlying soils, this condition will be referred to as a blanket-aquifer case. To determine the FS against uplift, or heave, of blanket materials, Equation 4 shall be utilized to calculate FS_{uplift} (see ex. in Figure 2). Note that the example calculations in Figure 2 show cases where the head at the ground surface is equal to the ground surface, however, the piping and uplift equations are also applicable for cases where there is a submerged seepage exit surface (i.e. the equations are also applicable when tail water is above the ground surface – refer to the definition of variable *h* given above). Per EM 1901, paragraph 4-9c, heave is defined as a phenomenon that "occurs when the upward seepage force due to differential head equals the overlying buoyant unit weight of soil." It is noted that use of the buoyant unit weight of soil implies an effective stress calculation. The distinction between Equations 4 and 1 is subtle but exists as the difference between materials used when calculating the numerator ($\gamma_s - \gamma_w$) of each equation.

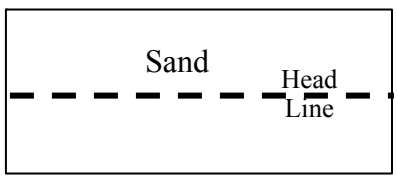
$$FS_{uplift} = \frac{\sum_{j=1}^{n} (\gamma_s - \gamma_w)_j * z_j}{\gamma_w * h} \qquad (4)$$

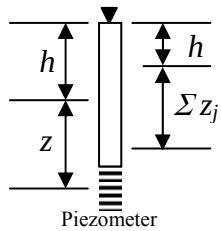
where: j = soil layer
γ_s = saturated unit weight of blanket materials
z_j = thickness of each soil layer. Σz_j will equal total blanket thickness.

$$FS_{piping} = \frac{i_{cv}}{i_v} = \frac{(\gamma_s - \gamma_w) * z}{\gamma_w * h}$$

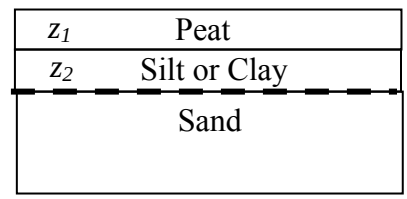
$$FS_{uplift} = \frac{\sum_{j=1}^{n} (\gamma_s - \gamma_w)_j * z_j}{\gamma_w * h}$$

Non-Blanket





Blanket-Aquifer



where: γ_s = 115 pcf
γ_w = 62.4 pcf
z = 10 ft
h = 2.0 ft

where: γ_{s1} = 66 pcf
γ_{s2} = 70 pcf
γ_w = 62.4 pcf
z_1 = 6 ft
z_2 = 3 ft
h = 0.5 ft

$$FS_{piping} = \frac{(115 - 62.4) * 10}{62.4 * 2}$$

$$FS_{uplift} = \frac{[(66 - 62.4) * 6 + (70 - 62.4) * 3]}{62.4 * 0.5}$$

$$FS_{piping} = 4.2$$

$$FS_{uplift} = 1.4$$

Figure 2: Example under seepage FS calculations, for illustrative purposes only

4.2.3. Pore pressures. Steady-state analyses shall be conducted with the Lake at SPF (25.0) and extreme pool (30.0) elevations. This is judged appropriate considering that these events will build slowly and will likely last for several months. A transient analysis shall be conducted under the temporary surcharge condition that could occur from wind setup atop the SPF. Pore pressures from finite element modeling for each analysis shall be used in the slope stability analyses for the corresponding load cases.

4.3. Analysis software. At a minimum, Geostudio 2004, or later versions, developed by GEO-SLOPE, International Ltd. of Alberta, Canada, should be used for both seepage and stability analyses and is preferred by SAJ.

5.0. Use of Minimum Seepage Factors of Safety. Traditional USACE guidance describes a FS based on the use of a maximum allowable exit gradient of 0.5 for saturated unit weights of blanket material greater than or equal to 110 pounds per cubic foot (pcf). However, blanket materials that surround HHD are much lighter than 110 pcf. As indicated in Table 1, use of a maximum allowable exit gradient on HHD for blanket unit weights less than 110 pcf would lead to unconservative designs with theoretical FS ranging from 0.24 to 1.53. Therefore, the Jacksonville District will present design criteria in terms of Factors of Safety instead of allowable seepage gradients. This is also consistent with ETL 569 which indicates that saturated unit weights of the blanket soils must be at or above 110 pcf to design to an allowable exit gradient of 0.5.

Table 1: Apparent Seepage FS Calculated for Different Soil Unit Weights and an Exit Gradient of 0.5

Saturated Unit Weight for Blanket (pcf)	Seepage FS for Exit Gradient = 0.5
70	0.24
80	0.56
90	0.88
100	1.21
110	1.53
120	1.85

6.0. SAJ Guidance on Slope Stability Load Cases and Factors of Safety. EM 1110-2-1902 *Slope Stability* contains USACE guidance. For all analyses, slope stability will be ensured by satisfying design FS using Spencer's Method of analysis. Table 3 contains load cases and minimum required FS, based on Table 3-1 as contained within EM 1902, for use in analyzing slope stability during design. In all cases, the landside slope stability is of interest. Static pool elevations shall apply to all reaches and stationing. The surcharge is dependent on stationing ranges and wind setup calculations.

6.1. Extreme Pool. For the Lake, this event is considered to correspond to an elevation of about 30.0 feet NAVD88. As stated previously, FS values will be

calculated and reported for this extreme load case, and shall then be evaluated for risk management purposes.

6.2. End-of-Construction. As HHD has been in its current configuration for nearly 40 years, this case may not be fully applicable. The requirement to perform End-of-Construction will apply when a significant and permanent alteration to the existing ground surface line will occur due to any constructed feature needed for added stability or seepage rehabilitation.

6.3. Maximum Storage Pool. For the Lake, this event is considered to be the SPF, corresponding to an elevation of 25.0 feet NAVD88.

6.4. Surcharge. This condition usually occurs as a temporary rise in pool or river level above the maximum storage pool until flood discharges are released through a control structure or downstream flood conditions recede and allow the pool or river to return to pre-flood conditions. Neither situation is applicable to HHD and the Lake. However, wind setup is a reality due to tropical systems or other wind events. Wind setup is a temporary stacking of water above the static pool due to wind shear across the water surface. Previous storm events and analyses have shown 12 hours as a reasonable estimate for wind setup to build. In actuality, since this is a temporary event, a transient seepage analysis should be performed to determine the pore pressure distribution which occurs at the end of 12 hours and then slope stability analyses would be performed with the distributed loading from the wind setup acting on the lakeside slope of HHD. The initial condition will be defined as the phreatic surface developed under the SPF elevation 25. A two-point Head vs. Time boundary function should be defined within SEEP/W. Set the Lake elevation to 25 at time 0 and direct the program to use the pore pressures developed from the steady-state analysis at this elevation. Time 1 will occur 12 hours later and the Lake elevation will correspond to the station being analyzed as shown in Design Documentation Report, Vol. 1, Section 7, Project Hydrology.

6.5 Rapid Drawdown. The need to perform this analysis is usually predicated on operating conditions which exist in other reservoirs where the pool can be lowered faster than the pore pressures within the embankment can dissipate. A rigorous analysis is not applicable to HHD for the following reasons: 1) no outlet works or pumps exist with sufficient capacity to lower Lake Okeechobee fast enough to qualify as a rapid drawdown and 2) due to the vast surface area and storage volume within the Lake, a catastrophic breach of HHD would still not lower the Lake fast enough to qualify as a rapid drawdown. Previous analyses have demonstrated sufficiently adequate FS and those results will be presented to satisfy the EM 1902 requirement, thus no further analysis will be pursued.

Table 2: Minimum Factors of Safety for Seepage

	Load Case	Toe of Dike		Landward**	
		Uplift	Piping	Uplift	Piping
Design	SPF	≥ 3.0	≥ 3.0	≥ 1.6	≥ 2.0
Design	Surcharge*	≥ 1.5	≥ 1.5	≥ 1.5	≥ 1.5
Report	Extreme Pool	Report FS (for risk management evaluation)			

* See definition of Surcharge given above in paragraph 6.4
**Lower required Factors of Safety are applicable for areas landward of the toe area of the originally constructed Dike and before any previous or proposed slope modification. If a landward constraint exists less than 150 ft from toe of Dike, then use linear interpolation to select the appropriate FS from Figure 3.

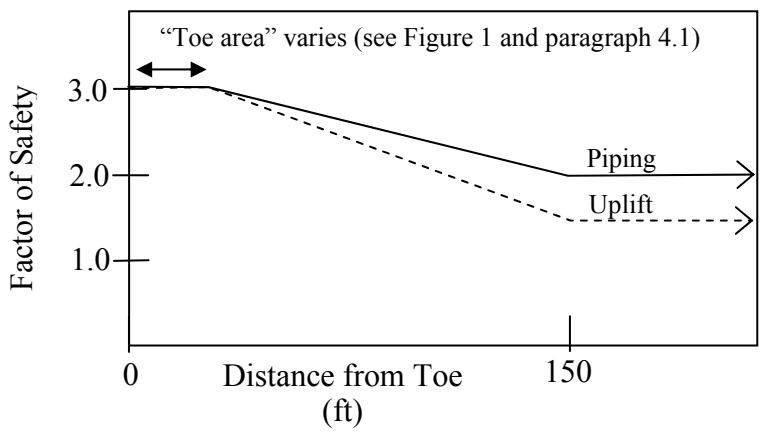


Figure 3: Graph Showing FS Decrease With Distance From Toe Of Dike

Table 3: Minimum Factors of Safety for Slope Stability

	Load Case	Factor of Safety
Design	SPF	≥ 1.5
Design	End-of-Construction	≥ 1.3
Design	Surcharge*	≥ 1.4
Design	Rapid Drawdown	No additional analyses needed.
Report	Extreme Pool	Report FS

*See definition of Surcharge given above in paragraph 6.4

7.0 Review and Concurrence. This MFR has undergone an Independent Technical Review within USACE and revisions have been made after considering reviewers' comments. In addition, via teleconference on Feb 20, 2008 with Messrs. Dave Pezza (HQ), Greg Baer (SAD), Tracy Hendren (SAD), Bob Fulton (SAD), Alan Bugg (SAJ), Stephen Duba (SAJ), David Dollar (SAJ), Scott Leech (SAJ), Jacob Davis, (SAJ), Mike Rogalski (SAJ), Michael Ramsbotham (SPK), Jeffrey Schaefer (LRL), Erich Guy (LRH), and Noah Vroman (ERDC), consensus was reached on selected Factors of Safety for use in designs and analyses for the HHD project.

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