ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

SAN JACINTO RIVER AUTHORITY
HIGHLANDS SOUTH CANAL
SIPHON 29 IMPROVEMENTS - WO#2
HARRIS COUNTY, TEXAS

Reported to:
Texas Water Engineering, PLLC
Sugar Land, Texas

by

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REPORT NO. G155-18

March 2019

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March 11, 2019

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Reference: **Geotechnical Investigation**
San Jacinto River Authority - Highlands South Canal
Siphon 29 Improvements - Work Order No. 2
Harris County, Texas
AEC Report No. G155-18

Dear Ms. Foss,

Aviles Engineering Corporation (AEC) is pleased to present this report of the results of our geotechnical investigation for the above referenced project. Notice to Proceed for the project was provided by the San Jacinto River Authority (SJRA) on September 4, 2018. The project terms and conditions were in accordance with the Professional Services Agreement between Texas Water Engineering (TWE) and AEC, dated February 1, 2018 and Work Order 2, dated September 4, 2018. The project scope was based on AEC proposal G2018-07-08, dated July 19, 2018.

AEC appreciates the opportunity to be of service to you. Please call us if you have any questions or comments concerning this report or when we can be of further assistance.

Respectfully submitted,
Aviles Engineering Corporation
(TBPE Firm Registration No. F-42)

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EXECUTIVE SUMMARY

The report submitted herein presents the results of Aviles Engineering Corporation's (AEC) geotechnical investigation for the San Jacinto River Authority's (SJRA) proposed improvements of Highlands Division Siphon 29 on the Highlands South Canal, at Ellis School Road in Harris County, Texas (Houston Key Map 460 X). The project improvements include: (i) demolition and removal or abandonment in place of existing siphon pipes and headwall structures; (ii) installation and construction of new siphon pipes and headwall structures; and (iii) slope stability analysis for the existing canal bank.

1. Subsurface Soil Conditions: Based on AEC's borings, the subsurface soil conditions in the vicinity of Siphon 29 generally consist of approximately 18 feet of firm to hard fat clay (including fill) at the ground surface, underlain by approximately 17 to 20 feet of soft to very stiff lean clay which was encountered at depths of 18 to 38 feet. Approximately 4 to 7 feet of silty sand (SM) was encountered at depths of 24 to 28 feet and 31 to 38 feet in Boring B-1, respectively. Approximately 4 feet of sandy silt (ML) was encountered at depths of 36 to 40 feet in Boring B-2. Approximately 1 to 5 feet of silty sand (SM) was encountered at depths of 23 to 24 and 35 to 40 feet in G116-13 Boring B-46.

2. Subsurface Soil Properties: The subsurface cohesive soils encountered in the borings (including Boring B-46 of AEC report G116-13) have slight to very high plasticity, with liquid limits (LL) ranging from 28 to 81, and plasticity indices (PI) ranging from 10 to 59. The cohesive soils encountered are classified as "CL" and "CH" type soils and granular soils were classified as "SM" and "ML" type soils in accordance with ASTM D 2487.

3. Groundwater Conditions: Groundwater was initially encountered in Borings B-1, B-2, and G116-13 Boring B-46 at a depth of 18 to 23 feet below grade during drilling, and subsequently rose to a depth between 7.6 and 12.6 feet approximately 15 minutes after the initial encounter. Groundwater was measured at 6.2 to 6.5 feet below grade approximately 1 to 3 days after drilling was completed. Based on the groundwater level observed, the groundwater in the borings is likely to be pressurized. A summary of groundwater depths encountered in the borings is presented on Table 5 in Section 4.1 of this report.

4. Soil Dispersion Characteristics: AEC performed a total of 4 crumb tests from Borings B-1 and B-2, and also considered the crumb test results from G116-13 Boring B-46 to evaluate the dispersive characteristics of clay soils along the canal. The results indicate that the tested soil samples in the channel zone for Boring B-1 and B-2 are classified as non-dispersive, while the samples tested from G116-13 Boring B-46 are non-dispersive to dispersive.

5. Recommendations for the design and installation of siphon pipes by open cut or tunnel/trenchless methods are presented in Sections 5.2 and 5.3 of this report, respectively. Recommendations for design and installation of siphon inlet/outlet structures are presented in Section 5.4 of this report.

6. AEC performed slope stability analyses on a selected cross section of the canal to determine if the canal slopes will be stable. The slope stability analyses consider three different conditions: the short-term condition, long-term condition and rapid drawdown condition. AEC performed the stability analyses in general accordance with the December 2010 Harris County Flood Control District (HCFCD) Geotechnical Guidelines. Based on our analyses, the safety factor's (SF) for the

analyzed canal cross section meet HCFCD'S minimum requirements under short term, long term, and rapid-drawdown conditions.

This Executive Summary is intended as a summary of the investigation and should not be used without the full text of this report.

ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

SAN JACINTO RIVER AUTHORITY
HIGHLANDS SOUTH CANAL
SIPHON 29 IMPROVEMENTS - WO#2
HARRIS COUNTY, TEXAS

1.0 INTRODUCTION

1.1 General

The report submitted herein presents the results of Aviles Engineering Corporation's (AEC) geotechnical investigation for the San Jacinto River Authority's (SJRA) proposed improvements of Highlands Division Siphon 29 on the Highlands South Canal, at Ellis School Road in Harris County, Texas (Houston Key Map 460 X). A vicinity map is presented on Plate A-1, in Appendix A. The project improvements include: (i) demolition and removal or abandonment in place of existing siphon pipes and headwall structures; and (ii) installation and construction of new siphon pipes and headwall structures; and (iii) slope stability analysis for the existing canal bank.

1.2 Purpose and Scope

The purpose of this geotechnical investigation is to evaluate the subsurface soil conditions at the site and develop geotechnical engineering recommendations for design and construction of the siphon pipes and siphon inlets/outlets, including demolition and removal or abandonment in place of existing siphon pipes and headwall structures. The scope of this geotechnical investigation is summarized below:

1. Drilling and sampling 2 geotechnical borings to 40 feet below existing grade;
2. Soil laboratory testing on selected soil samples;
3. Recommendations for demolition and removal or abandonment in place of existing siphon pipes and headwall structures;
4. Engineering analyses and recommendations for the installation of siphon pipes by open cut method, including loadings on pipes, bedding, lateral earth pressure parameters, trench stability, and backfill requirements;
5. Engineering analyses and recommendations for the installation of siphon pipes by trenchless method, including bore/auger launching and receiving shafts, reaction walls, and bore face stability;
6. Engineering analyses and recommendations for siphon inlets/outlets, including allowable bearing capacity and lateral earth pressure parameters for headwalls and wingwalls;
7. Engineering analyses and recommendations for the existing canal slope, including slope stability analysis on a selected cross section;

8. Construction recommendations for the siphon pipes and inlets/outlets.

2.0 SUBSURFACE EXPLORATION

As directed by SJRA, AEC drilled a total of two borings to 40 feet below existing grade at the site. The total drilling footage is 80 feet. G116-13 Boring B-46 was drilled to 40 feet below existing grade in June 2013 for AEC report G116-13, and is also included in this report for reference. The boring locations are shown on the Boring Location Plan on Plate A-2, in Appendix A. After completion of drilling, the boring locations were surveyed by GeoSolutions, LLC. Boring survey data in State Plane *Grid* Coordinates (Texas South Central Zone) is presented on Table 1 and on the boring logs.

Table 1. Summary of Borings Coordinates and Elevations

Boring No.	Boring Depth (ft)	Northing (Grid, ft)	Easting (Grid, ft)	Boring Surface Elevation (ft)
B-1	40	13,859,924.29	3,227,693.675	31.769
B-2	40	13,859,804.27	3,227,773.863	31.842

The field drilling was performed with a truck-mounted drilling rig. The borings were generally advanced initially using dry auger method, and then using wet rotary method once groundwater was encountered, or the borings began to cave in. Undisturbed samples of cohesive soils were obtained from the borings by pushing 3-inch diameter thin-wall, seamless steel Shelby tube samplers in general accordance with ASTM D 1587. Granular soils were sampled with a 2-inch split-barrel sampler in accordance with ASTM D 1586. Standard Penetration Test resistance (N) values were recorded for the granular soils as "Blows per Foot" and are shown on the boring logs. Strength of the cohesive soils was estimated in the field using a hand penetrometer. The undisturbed samples of cohesive soils were extruded mechanically from the core barrels in the field and wrapped in aluminum foil; all samples were sealed in plastic bags to reduce moisture loss and disturbance. The samples were then placed in core boxes and transported to the AEC laboratory for testing and further study. Groundwater readings were taken during drilling, after completion of drilling, and 3 days after completion of drilling. After the final groundwater readings were obtained, the borings were backfilled with bentonite chips.

3.0 LABORATORY TESTING PROGRAM

Soil laboratory testing was performed by AEC personnel. Samples from the borings were examined and classified in the laboratory by a technician under the supervision of a geotechnical engineer. Laboratory tests were performed on selected soil samples in order to evaluate the engineering properties of the foundation soils in accordance with applicable ASTM Standards. Atterberg limits, moisture contents, percent passing a No. 200 sieve, sieve analysis, and dry unit weight tests were performed on selected samples to establish the index properties and confirm field classification of the subsurface soils. For completeness, AEC also included the sieve and hydrometer analysis from AEC report G116-13 Boring B-46 in this report. Strength properties of cohesive soils were determined by means of torvane (TV), unconfined compression (UC), undrained-unconsolidated (UU), and consolidated-undrained (CU) triaxial tests performed on undisturbed samples. The test results are presented on the boring logs. Details of the soils encountered in the borings are presented on Plates A-3 through A-5, in Appendix A. A key to the boring logs, classification of soils for engineering purposes, terms used on boring logs, and reference ASTM Standards for laboratory testing are presented on Plates A-6 through A-9, in Appendix A. Sieve analysis results are presented on Plates A-10 and A-11, in Appendix A.

Crumb Tests: To evaluate the dispersive characteristics of clayey soils along the canal, four crumb tests were performed on selected soil samples in accordance with ASTM D 6572, Method A. The results of the crumb tests are summarized on Table 2 and are presented on Plate A-12, in Appendix A. AEC also considered the crumb test results performed on soil samples from G116-13 Boring B-46 and included them in Table 2 and as well as on Plate A-13, in Appendix A.

Table 2. Summary of Crumb Test Results

Sample ID and Description	Dispersive Grade	Dispersive Classification
B-1, 4’-6’, Fat Clay (CH)	1	Non-dispersive
B-1, 12’-14’, Fat Clay (CH)	1	Non-dispersive
B-2, 2’-4’, Fat Clay (CH)	1	Non-dispersive
B-2, 10’-12’, Fat Clay (CH)	1	Non-dispersive
G116-13 Boring B-46, 4’-6’, Fat Clay (CH)	1	Non-dispersive
G116-13 Boring B-46, 23’-25’, Lean Clay (CL)	3	Dispersive

<u>Consolidated-Undrained Triaxial Tests:</u> One CU triaxial shear test was performed in accordance with ASTM D 4767 to determine shear strength parameters of the soil. The CU test Mohr Coulomb Diagrams are included on Plate A-14, in Appendix A. The shear strength parameters obtained from the CU triaxial tests are summarized below in Table 3.

Table 3. Summary of Shear Strength Parameters from CU Triaxial Tests

Sample ID and Description	Effective Stress		Total Stress	
	c' (psf)	φ' (deg)	c_{cu} (psf)	φ_{cu} (deg)
B-2, 8'-10', Fat Clay (CH)	210	17.2	230	13.1

Notes: (1) c' = effective cohesion, φ' =effective friction angle, obtained from CU tests with pore pressure measurements;
(2) c_{cu} = cohesion in total stress, φ_{cu} = friction angle in total stress, obtained from CU tests.

<u>Consolidation Tests:</u> A one-dimensional consolidation test was performed on a selected soil sample in order to evaluate the general compressibility characteristics of clay soils at the site. The results of the consolidation test are presented on Plate A-15. The initial void ratio, compression index, recompression index, preconsolidation pressure, and estimated overconsolidation ratio (OCR) for the consolidation test are summarized in Table 4.

Table 4. Summary of Consolidation Test Results

Sample ID and Description	e_0	C_c	C_r	p_c (tsf)	OCR
B-1, 14'-16', Fat Clay (CH)	0.8371	0.2239	0.0301	4.4	7.0

Note: (1) e_0 = initial void ratio;
(2) C_c = compression ratio;
(3) C_r = recompression ratio, which is derived from the recompression curve within the stress range from 2 to 8 ksf;
(4) p_c = preconsolidation pressure; and
(5) OCR = overconsolidation ratio.

4.0 SITE CONDITIONS

The current siphon structure is located beneath Ellis School Road, which is a two lane (one lane in each direction) asphalt roadway with grass-lined roadside ditches. The existing siphon consists of dual 42-inch diameter concrete siphon pipes (based on as-built drawings dated 1978) and an additional 60-inch diameter concrete bypass siphon pipe (based on as-built drawings dated 1997). In 1998 the dual 42-inch pipes were slip-lined with 36-inch HOBAS Relining Pipes. Concrete headwalls for the existing siphon pipes are located on either side of Ellis School Road at the canal intersection. Based on AEC's site visits, horizontal movement of the headwalls and wing walls were observed at various locations. Loss of soils behind the wing walls, slope failures near the siphon structures, and cracks in the head walls and wing walls were also observed from various locations at Siphon 29. Selected photos in the vicinity of Siphon 29 are presented on Plates B-1 and B-2, in Appendix B.

4.1 Subsurface Conditions

Details of the soils encountered during drilling are presented in the boring logs. Soil strata encountered in our borings are summarized below.

Boring	Depth (ft)	Description of Stratum
B-1	0 - 4	Fill: very stiff, Fat Clay (CH), with shell
	4 - 18	Firm to very stiff, Fat Clay (CH), with slickensides
	18 - 24	Stiff, Sandy Lean Clay (CL), with silty clay pockets
	24 - 28	Silty Sand (SM), with fat clay seams, wet
	28 - 31	Stiff, Sandy Lean Clay (CL), with calcareous nodules
	31 - 38	Loose, Silty Sand (SM), with clayey sand pockets, wet
	38 - 40	Stiff, Fat Clay (CH), with ferrous stains
B-2	0 - 18	Firm to hard, Fat Clay (CH), with slickensides
	18 - 22	Very stiff, Lean Clay (CL), with silty clay and fat clay pockets
	22 - 36	Soft to stiff, Sandy Lean Clay (CL), with abundant silty clay pockets, wet
	36 - 40	Sandy Silt (ML), with silty clay seams, wet
G116-13	0- 4	Fill: very stiff, Fat Clay with Sand (CH), with calcareous nodules and roots
B-46	4 - 18	Firm to very stiff, Fat Clay (CH), with slickensides and ferrous stains
	18 - 23	Stiff, Lean Clay (CL), with silt partings
	23 - 24	Silt (ML), wet
	24 - 28	Stiff, Fat Clay (CH), with sand pockets
	28 - 35	Soft to stiff, Lean Clay with Sand (CL), with abundant silt partings
	35 - 40	Medium dense, Silty Sand (SM), with clayey sand pockets, wet

Subsurface Soil Properties: The subsurface cohesive soils encountered in the borings (including G116-13 Boring B-46) have slight to very high plasticity (see Plate A-7, in Appendix A), with liquid limits (LL) ranging from 28 to 81, and plasticity indices (PI) ranging from 10 to 59. The cohesive soils encountered are classified as "CL" and "CH" type soils and granular soils were classified as "SM" and "ML" type soils in accordance with ASTM D 2487. High plasticity clays can undergo significant volume changes due to seasonal changes in moisture contents. "CH" soils undergo significant volume changes due to seasonal changes in soil moisture contents. "CL" type soils with lower LL (less than 40) and PI (less than 20) generally do not undergo significant volume changes with changes in moisture content. However, "CL" soils with LL approaching 50 and PI greater than 20 essentially behave as "CH" soils and could undergo significant volume changes.

Groundwater: Groundwater levels encountered in the borings are presented in Table 5. AEC has also included the groundwater readings from G116-13 Boring B-46 in Table 5 for reference.

Table 5. Groundwater Depths below Existing Ground Surface

Boring No.	Date Drilled	Boring Depth (ft)	Groundwater Depth (ft)
B-1	9/7/18	40	18 (Drilling) 7.6 (15 min.) 6.3 (9/10/18)
B-2	9/7/18	40	20 (Drilling) 12.6 (15 min.) 6.5 (9/10/18)
G116-13 B-46	6/4/13	40	23.4 (Drilling) 7.8 (15 min.) 6.2 (6/5/13)

The information in this report summarizes conditions found on the dates the borings were drilled. It should be noted that our groundwater observations are short-term; groundwater depths and subsurface soil moisture contents will vary with environmental variations such as frequency and magnitude of rainfall and the time of year when construction is in progress.

4.2 Hazardous Materials

No signs of visual staining or odors were encountered during field drilling or during processing of the soil samples in the laboratory.

4.3 Subsurface Variations

It should be emphasized that: (i) at any given time, groundwater depths can vary from location to location, and (ii) at any given location, groundwater depths can change with time. Groundwater depths will vary with seasonal rainfall and other climatic/environmental events. Subsurface conditions may vary away from and in between the boring locations.

Clay soils in the Greater Houston area typically have secondary features such as slickensides, calcareous and ferrous nodules, and contain sand/silt seams/lenses/layers/pockets. It should be noted that the information in the boring logs is based on 3-inch diameter soil samples. Soil samples were obtained from the borings continuously at intervals of 2 feet from the ground surface to a depth of 20 feet, then at intervals of 5 feet thereafter to the boring termination depths. A detailed description of the soil secondary features may not have been obtained due to the small sample size and sampling interval between the samples. Therefore, while a boring log shows some soil secondary features, it should not be assumed that the features are absent where not indicated on the boring logs.

5.0 GEOTECHNICAL ENGINEERING RECOMMENDATIONS

The project improvements include: (i) demolition and removal or abandonment in place of existing siphon pipes and headwall structures; and (ii) installation and construction of new siphon pipes and headwall structures.

Based on information provided by Texas Water Engineering (TWE), AEC understands that: (i) two 72-inch siphon pipes are proposed with an approximate flowline elevation of 19 feet above Mean Sea Level (MSL) and (ii) the existing siphon pipes will either be removed or abandoned in place, depending on which method will be used to construct the new siphon pipes. Based on the drawing provided by TWE, the foundation slab of the siphon inlet/outlet headwalls will bear at an elevation of 16.5 feet above MSL.

5.1 Demolition and Removal of Siphon Pipes and Headwalls

Based on the information provided by TWE, AEC understands that the existing 42 inch siphon pipes will be either removed or grouted in place depending on how the new siphon pipes are installed. If the new pipes are installed by open-cut method, then the existing siphon pipes will be removed. If the new pipes will be installed by tunnel or trenchless method, then the existing siphon pipes will be abandoned in place. AEC assumes that whichever method is used to install the new siphon pipes, all three (both 42 inch diameter and 60 inch diameter) of the existing siphon pipes will all be either removed or abandoned together.

Demolition and Backfill: Assuming the new siphon pipes will be installed by open cut, then demolition of the existing siphon pipes, headwalls, wingwalls, and their foundations should be performed in accordance with Section 02 41 13.13 of the latest edition of the SJRA Construction Specifications. The contractor should take care to ensure that the surrounding soils are not excessively disturbed while removing the existing siphon pipes, headwalls, wingwalls, and footings. Backfilling of the removed siphon pipes, headwalls, wingwalls, and footings should be performed in accordance with Section 31 21 33 of the latest edition of the SJRA Construction Specifications.

Abandon in Place: If the new siphon pipes will be installed by tunnel or trenchless methods, then the existing structures and siphon pipes will be abandoned in place. AEC recommends that the existing siphon pipes and any other to be abandoned structure cavities be properly backfilled with flowable fill. Flowable fill should be in accordance with Item 401 of the Texas Department of Transportation (TxDOT) Standard Construction Recommendations for the Construction and Maintenance of Highways, Streets, and Bridges, or equivalent SJRA Construction Specification.

5.2 Installation of Siphon Pipes by Open-Cut Method

According to TWE, AEC understands that the new siphon pipes will likely be installed by open cut method. Siphon pipes installed by open-cut methods should be designed and installed in accordance with Section 31 21 33 of the latest edition of the SJRA Construction Specifications.

5.2.1 Geotechnical Parameters for Siphon Pipes

Recommended geotechnical parameters for the subsurface soils at the site to be used for design of siphon pipes are presented on Plate C-1, in Appendix C. The design values are based on the results of field and laboratory test data on individual boring logs as well as our experience. It should be noted that because of the variable nature of soil stratigraphy, soil types and properties along the alignment or at locations away from a particular boring may vary substantially.

5.2.2 Loadings on Pipes

Underground utilities support the weight of the soil and water above the crown, as well as roadway traffic and any structures that exist above the utilities.

Earth Loads: For underground utilities to be installed using open cut methods, the vertical soil load W_e can be calculated as the larger of the two values from Equations (1) and (3):

$$W_e = C_d \gamma B_d^2 \quad \text{...........Equation (1)}$$

$$C_d = [1 - e^{-2K\mu'(H/B_d)}]/(2K\mu') \quad \text{...........Equation (2)}$$

$$W_e = \gamma B_c H \quad \text{...........Equation (3)}$$

where: W_e = trench fill load, in pounds per linear foot (lb/ft);
C_d = trench load coefficient, see Plate C-4, in Appendix C;
γ = effective unit weight of soil over the conduit, in pounds per cubic foot (pcf);
B_d = trench width at top of the conduit < 1.5 B_c (ft);
B_c = outside diameter of the conduit (ft);
H = variable height of fill (ft);
when the height of fill above the top of the conduit $H_c > 2 B_d$, $H = H_h$ (height of fill above the middle of the conduit). When $H_c < 2 B_d$, H varies over the height of the conduit; and
$K\mu'$ = 0.1650 maximum for sand and gravel,
0.1500 maximum for saturated top soil,
0.1300 maximum for ordinary clay,
0.1100 maximum for saturated clay.

When underground conduits are located below groundwater, the total vertical dead loads should include the weight of the projected volume of water above the conduits.

Traffic Loads: The vertical stress on top of an underground conduit, p_L (psf), resulting from traffic loads (from a HS-20 truck) can be obtained from Plate C-5, in Appendix C. The live load on top of the underground conduit can be calculated from Equation (4):

$$W_L = p_L B_c \quad \text{...........Equation (4)}$$

where: W_L = live load on the top of the conduit (lb/ft);
p_L = vertical stress (on the top of the conduit) resulting from traffic loads (psf);
B_c = outside diameter of the conduit, (ft);

Lateral Loads: The lateral soil pressure p_l can be calculated from Equation (5); hydrostatic pressure should be added, if applicable.

$$p_l = 0.5 (\gamma H_h + p_s) \quad \text{...........Equation (5)}$$

where: H_h = height of fill above the center of the conduit (ft);
γ = effective unit weight of soil over the conduit (pcf);
p_s = vertical pressure on conduit resulting from traffic and/or construction equipment (psf).

5.2.3 Trench Stability

Cohesive soils in the Houston area contain many secondary features which affect trench stability, including sand seams and slickensides. Slickensides are shiny weak failure planes which are commonly present in fat clays; such clays often fail along these weak planes when they are not laterally supported, such as in an open excavation. The Contractor should not assume that slickensides and sand seams/layers/pockets are absent where not indicated on the logs.

The Contractor should be responsible for designing, constructing and maintaining safe excavations. The excavations should not cause any distress to existing structures.

Trenches 20 feet and Deeper: The Occupational Safety and Health Administration (OSHA) requires that shoring or bracing for trenches 20 feet and deeper be specifically designed by a licensed professional engineer.

Trenches Less than 20 Feet Deep: Trench excavations that are less than 20 feet deep may be shored, sheeted and braced, or laid back to a stable slope for the safety of workers, the general public, and adjacent structures, except for excavations which are less than 5 feet deep and verified by a competent person to have no cave-in potential. The excavation and trenching should be in accordance with OSHA Safety and Health Regulations, 29 CFR, Part 1926. Recommended OSHA soil types for trench design for existing soils can be found on Plate C-1, in Appendix C. Fill soils are considered OSHA Class ‘C’; submerged cohesive soils should also be considered OSHA Class ‘C’, unless they are dewatered first.

Critical Height is defined as the height a slope will stand unsupported for a short time; in cohesive soils, it is used to estimate the maximum depth of open-cuts at given side slopes. Critical Height may be calculated based on the soil cohesion. Values for various slopes and cohesion are shown on Plate D-1, in Appendix D. Cautions listed below should be exercised in use of Critical Height applications:

1. No more than 50 percent of the Critical Height computed should be used for vertical slopes. Unsupported vertical slopes are not recommended where granular soils or soils that will slough when not laterally supported are encountered within the excavation depth.

2. If the soil at the surface is dry to the point where tension cracks occur, any water in the crack will increase the lateral pressure considerably. In addition, if tension cracks occur, no cohesion should be assumed for the soils within the depth of the crack. The depth of the first waler should not

exceed the depth of the potential tension crack. Struts should be installed before lateral displacement occurs.

3. Shoring should be provided for excavations where limited space precludes adequate side slopes, e.g., where granular soils will not stand on stable slopes and/or for deep open cuts.

4. All excavation, trenching and shoring should be designed and constructed by qualified professionals in accordance with OSHA requirements.

The maximum (steepest) allowable slopes for OSHA Soil Types for excavations less than 20 feet are presented on Plate D-2, in Appendix D.

If limited space is available for the required open trench side slopes, the space required for the slope can be reduced by using a combination of bracing and open cut as illustrated on Plate D-3, in Appendix D. Guidelines for bracing and calculating bracing stress are presented below.

Computation of Bracing Pressures: The following method can be used for calculating earth pressure against bracing for open cuts. Lateral pressure resulting from construction equipment, traffic loads, or other surcharge should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure. Hydrostatic pressure, if any, should also be considered. The active earth pressure at depth z can be determined by Equation (6). The design soil parameters for trench bracing design are presented on Plate C-1 in Appendix C.

$$p_a = (q_s + \gamma h_1 + \gamma' h_2)K_a - 2c\sqrt{K_a} + \gamma_w h_2 \qquad \text{...........Equation (6)}$$

where:
- p_a = active earth pressure (psf);
- q_s = uniform surcharge pressure (psf);
- γ, γ' = wet unit weight and buoyant unit weight of soil (pcf);
- h_1 = depth from ground surface to groundwater table (ft);
- h_2 = $z-h_1$, depth from groundwater table to the point under consideration (ft);
- z = depth below ground surface for the point under consideration (ft);
- K_a = coefficient of active earth pressure;
- c = cohesion of clayey soils (psf); c can be omitted conservatively;
- γ_w = unit weight of water, 62.4 pcf.

Pressure distribution for the practical design of struts in open cuts for clays and sands are illustrated on Plates D-4 through D-6, in Appendix D.

Bottom Stability: In open-cuts, it is necessary to consider the possibility of the bottom failing by heaving, due to the removal of the weight of excavated soil. Heaving typically occurs in soft plastic clays when the excavation depth is sufficiently deep enough to cause the surrounding soil to displace vertically due to bearing capacity failure of the soil beneath the excavation bottom, with a corresponding upward movement of the soils in the bottom of the excavation. In fat and lean clays, heave normally does not occur unless the ratio of Critical Height to Depth of Cut approaches one. In very sandy and silty lean clays and granular soils, heave can occur if an artificially large head of water is created due to installation of impervious sheeting while bracing the cut. This can be mitigated if groundwater is lowered below the excavation by dewatering the area. Guidelines for evaluating bottom stability in clay soils are presented on Plate D-7, in Appendix D.

According to the drawing provide by TWE, the flow line of the new siphon pipes will be at a depth of approximately 12.8 to 13.1 feet below grade (i.e. at an approximate elevation of 19 feet above MSL). Based on the borings and the proposed siphon invert depths, AEC anticipates that the open cut trench excavations will primarily encounter firm to hard fat clay (CH) throughout the trench and pipe bedding zone. Based on Table 5 in Section 4.1 of this report, open cut excavations are likely to encounter groundwater within the trench or pipe bedding zone (i.e. at a depth below approximately 6 feet). Groundwater control during trench excavation operations may be required; groundwater control recommendations are presented in Section 6.2 of this report.

If the excavation extends below groundwater and the soils at or near the bottom of the excavation are mainly sands or silts, the bottom can fail by blow-out (boiling) when a sufficient hydraulic head exists. The potential for boiling or in-flow of granular soils increases where the groundwater is pressurized. To reduce the potential for boiling of excavations terminating in granular soils below pressurized groundwater, the groundwater table should be lowered at least 3 feet below the bottom of the excavation to maintain a stable surface. Dewatering should be performed in accordance with Section 01 57 23.02 of the latest edition of the SJRA Construction Specifications.

Calcareous nodules, silt/sand seams, and fat clays with slickensides were encountered in our borings. These secondary structures may become sources of localized instability when they are exposed during excavation, especially when they become saturated. Such soils have a tendency to slough or cave in when not laterally confined, such as in trench excavations. The Contractor should be aware of the potential for cave-in of the soils. Low plasticity soils (silts and clayey silts) will lose strength and may behave like granular soils when saturated.

Stockpile and Equipment Surcharge: To avoid surcharging the excavation walls, stockpile of excavated materials immediately adjacent to the excavation face should be prohibited. We recommend stockpiled materials be placed at least 6 feet away from the edge of an excavation face, and no higher than 3 feet. Construction equipment working near the trench may also induce excessive surcharge loads; AEC recommends appropriate shoring or shield system be provided considering these impacts in addition to the lateral earth and hydrostatic pressures.

5.2.4 Bedding and Backfill

Trench excavation, pipe embedment material, and backfill for the proposed siphon pipes should be in general accordance with Section 31 21 33 of the latest edition of the SJRA Construction Specifications.

5.3 Installation of Siphon Pipes by Tunnel or Trenchless Methods

According to TWE, AEC understands that the intent is to install the siphon pipes by open cut method. However, considering that open cut installation would likely require Ellis School Road to be closed during construction, open cut installation may not be possible if the roadway cannot be closed. If open cut installation cannot be performed then the siphon pipes would need to be installed by either tunnel method or trenchless method.

The Contractor is responsible for designing, constructing, implementing, and monitoring safe tunneling excavation and protecting existing structures in the vicinity from adverse effects resulting from construction, and retaining professionals who are qualified and experienced to perform the tasks and who are capable of modifying the system, as required. The following discussion provides general guidelines to the Contractor for reference purposes.

Tunnel Method: With tunneling methods, the excavation face is advanced using either a boring shield (with hand mining, or mechanized means such as backacter or cutting boom), microtunnel boring machine (MTBM), or mechanized tunnel boring machine (TBM). In cases with soft or unstable ground, then a slurry TBM or earth pressure balance machine (EPBM) will be used instead. The boring shield or machine is typically placed in front of the carrier pipe (in a one pass method) or in front of a primary liner (in a two pass method). Tunneling methods should be performed in accordance with Section 33 05 23.19 of the latest edition of the SJRA Standard Specifications.

Trenchless Method: With trenchless method, the excavation face is advanced using either dry auger/bore method (either above groundwater, or with lowering the groundwater table), or slurry auger/bore method (below groundwater or within saturated sands/silts). The tip of the auger is in line with the front of the carrier pipe (in a one pass method) or primary liner (in a two pass method). Trenchless methods should be performed in accordance with Section 33 05 23.23 of the latest edition of the SJRA Standard Specifications.

Excavation Walls: For this report, the term “Excavation walls” can refer to the soils surrounding the pipes whether trenchless or tunnel methods are used. Excavation walls can be constructed using either: (i) a one pass method, where the siphon carrier pipe is pushed through the ground by jacking it into the soil and the carrier pipe directly supports the excavation walls; or (ii) a two pass method, which includes first installing a primary liner (which supports the excavation walls and has a larger diameter than the carrier pipe), and then installing the siphon carrier pipe within the primary liner. For two pass methods, primary liner options include: (i) steel casing; (ii) rib and lagging; (iii) steel liner plate; or (iv) segmented concrete. Two pass method construction should be in accordance with Section 31 71 00.01 of the latest edition of the SJRA Construction Specifications.

Loadings on Pipes: Recommendations for computation of loadings on pipes from HS-20 trucks are presented in Section 5.2.1 of this report.

5.3.1 Tunnel Access Shafts

For the purposes of this investigation, AEC considers bore launching and receiving pits (if trenchless methods are used) to be essentially the same as a tunnel access shaft (if tunneling method is used). Based on the information provided by SJRA, the flow line elevation of the proposed siphon pipes will be at an elevation of 19 feet above MSL. Based on our experience with other projects involving tunneling, AEC assumes that the bottom of the tunnel access shafts will be approximately 3 feet lower than the siphon pipe invert depth, i.e. at an elevation of 16 feet above MSL. AEC should be notified if the access shaft bottom will be more or less than 3 feet below the siphon pipe invert depth so that our recommendations can be updated if necessary. Tunnel access shafts should be constructed in accordance with Section 31 75 00 of the latest edition of the SJRA Construction Specifications. The approximate tunnel invert depths and possible subsurface conditions at the tunnel access shafts are summarized in Table 6 below.

Table 6. Subsurface Conditions in Borings near Tunnel Access Shafts

Soil Boring	Tunnel Invert Depth (ft)	Soil Types Encountered within Tunnel Access Shaft(1)	Ground Water Depth below Existing Ground Surface (ft)
B-1	12.8	0'-4': fill: very stiff CH 4'-16': firm to very stiff CH	18 (Drilling) 7.6 (15 min.) 6.3 (9/10/18)
B-2	12.8	0'-16': firm to hard CH	20 (Drilling) 12.6 (15 min.) 6.5 (9/10/18)
G116-13 B-46	13.1	0'-4': fill: very stiff CH 4'-16': firm to very stiff CH	23.4 (Drilling) 7.8 (15 min.) 6.2 (6/5/13)

Note: (1) Taken from ground surface to 3 feet below tunnel invert depth.
(2) CH = Fat Clay.

Based on Table 6, excavation for the tunnel access shafts are likely to encounter groundwater. AEC recommends that the groundwater table be lowered to at least 3 feet below the bottom of the access shaft excavation to be able to work on a firm surface. Groundwater control should be in accordance with Section 01 57 23.02 of the latest edition of the SJRA Construction Specifications. Possible groundwater dewatering measures include: (i) sump and pump (in clay soils); (ii) deep wells or multi-staged wellpoints (in sandy soils); or (iii) eductors (in silts and silty sands).

If cohesive soils contain significant secondary features, seepage rates will be higher. This may require larger sumps, or if significant granular layers are interbedded within the cohesive soils, methods used for granular soils may be required. Where it is present, pressurized groundwater will also yield higher seepage rates.

AEC notes that extended and/or excessive dewatering can result in settlement/differential settlement of existing structures, pavement, or utilities in the vicinity of the dewatering operations. General groundwater control recommendations are presented in Section 6.2 of this report. The options for groundwater control presented here are for reference purposes only; it is the Contractor's responsibility to take the necessary precautions to minimize the effect on existing structures in the vicinity of the groundwater control operations.

If typical dewatering operations outside of the access shaft locations are not feasible because lowering the groundwater table will have a negative impact on adjacent structures, utilities, or pavements, then using either: (i) internally braced water-tight sheet pile cut-off walls, steel shaft liner, or drilled shaft wall; or (ii) jet grouting of sandy soils in the immediate surrounding area can be considered. It is AEC's opinion that using either water-tight: internally-braced sheet piles, steel shaft liner, or drilled shaft wall (with grout between the shafts) would be an effective option for tunnel access shaft shoring; in addition, these methods also reduce and/or eliminate the need for groundwater control outside of the access shaft excavations.

Sheet Piling: Design soil parameters for sheet pile design are presented on Plate C-1, in Appendix C. AEC recommends that the sheet pile design consider both short-term and long-term parameters; whichever is critical should be used for design. The determination of the pressures exerted on the sheet piles by the retained soils shall consider active earth pressure, hydrostatic pressure, and uniform surcharge (including construction equipment, soil stockpiles, and traffic load, whichever surcharge is more critical).

Sheet pile design should be based on the following considerations:

(1) Consider the ground water elevation to be at the top of the ground surface on the retained side (i.e. outside of the access shaft).
(2) Consider the ground water elevation to be at least 3 feet below the bottom of the access shaft excavation, whether or not the groundwater level is actually lowered to 3 feet below the bottom of the excavation during construction. This should result in a more conservative design for sheet piling length, due to an increase of the hydrostatic pressure head differential.
(3) Neglect cohesion for active pressure determination, based on Equation (6) in Section 5.3.2 of this report;
(4) The design retained height should extend from the ground surface to the bottom of the access shaft excavation.
(5) A 300 psf uniform surcharge pressure from construction equipment or soil stockpiles should be considered at the top of the sheet piles; loose soil stockpiles during access shaft construction should be limited to 3 foot high or less.
(6) Use a Factor of Safety of 2.0 for passive earth pressure in front of (i.e. the shaft side) the sheet piles.

Design, construction, and monitoring of sheet piles should be performed by qualified personnel who are experienced in this operation. Sheet piles should be driven in pairs, and proper construction controls provided to maintain alignment along the wall and prevent outward leaning of the sheet piles. Construction of sheet piles should be in accordance with Section 31 62 17 of the latest edition of the SJRA Construction Specifications.

Stockpile and Equipment Surcharge: To avoid surcharging the access shaft excavation walls, stockpile of excavated materials immediately adjacent to the excavation face should be prohibited. We recommend stockpiled materials be placed at least 6 feet away from the edge of an excavation face, and no higher than 3 feet. Construction equipment working near the access shaft excavations may also induce excessive surcharge loads; AEC recommends appropriate shoring or shield system be provided considering these impacts in addition to the lateral earth and hydrostatic pressures.

Bottom Stability: Recommendations for evaluating tunnel access shaft bottom stability are presented in Section 5.3.2 of this report.

Reaction Walls: Reaction walls (especially if a one pass method is used) will be part of the tunnel shaft walls; they will be rigid structures and support tunneling/trenchless operations by mobilizing passive pressures of the soils behind the walls. The passive earth pressure can be calculated using Equation (7); we recommend that a factor of safety of 2.0 be used for passive earth pressure. The design soil parameters that can be used for reaction wall design are presented on Plate C-1 in Appendix C.

$$p_p = \gamma\, zK_p + 2c(K_p)^{1/2} \qquad \text{...........Equation (7)}$$

where,
- p_p = passive earth pressure (psf);
- γ = wet unit weight of soil (pcf);
- z = depth below ground surface for the point under consideration (ft);
- K_p = coefficient of passive earth pressure;
- c = cohesion of clayey soils (psf).

Due to subsurface variations, soils with different strengths and characteristics will likely be encountered at a given location. The soil resulting in the lowest passive pressure should be used for design of the walls. The soil conditions should be checked by geotechnical personnel to confirm the recommended soil parameters.

5.3.2 Tunnel/Trenchless Face Stability during Construction

A general description of tunnel and trenchless methods are presented in Section 5.3 of this report. In general, tunneling methods include using either a boring shield or TBM to advance the excavation face, while trenchless methods include using an auger (either dry or with slurry) to advance the excavation face. For tunneling method, AEC notes that the MTBM, SFM, and EPBM are all able to counteract hydrostatic pressures from groundwater. In a similar vein, for trenchless methods, a slurry auger/bore will also be able to counteract hydrostatic pressures from groundwater. All of these methods reduce or eliminate the need for groundwater control during tunneling/trenchless methods. Conversely, for tunneling method, an open face or partial boring shield or a mechanized close face TBM cannot counteract hydrostatic pressures.

Similarly for trenchless methods, a dry auger/bore will also be unable to counteract hydrostatic pressures. If these methods are used, the groundwater table either needs to be lowered during construction, or saturated sands/silts need to be grouted prior to tunneling/trenchless operations.

Trenchless Method: Based on AEC's discussions with TWE (and considering the siphon carrier pipe will be 72 inches in diameter), AEC understands that the most likely method to construct the siphon pipe will be a bore and jack method. A one pass method will be used to jack the siphon carrier pipe directly into the ground (supporting the excavation wall) while a bore/auger (i.e. trenchless method) will be used to advance the excavation face. For this method, AEC prefers that the trenchless method use a slurry auger/bore to advance the excavation, since groundwater is likely to be encountered within the auger/bore zone during construction. However, selection of auger/bore method (whether dry auger or slurry auger) will be up to the trenchless Contractor. AEC notes that bore and jack is also frequently performed as a two pass method, where a steel casing is first jacked into the ground, the soil within the casing is bored out, and then the carrier pipe is installed. However, according to TWE, AEC understands that the two pass method is less likely to be used.

Tunneling Method: If a tunneling method will be used, AEC also assumes that a one pass method is likely (because of the relatively short siphon pipe length, and based on our discussions with TWE), where the tunnel boring shield/TBM and carrier pipe are jacked directly into the ground. However, AEC notes that two pass systems (with primary liner) are also commonly used in these applications. For tunneling method (based on the AEC's borings and considering the siphon carrier pipe will be 72 inches in diameter), feasible tunneling methods include tunnel boring shield, close faced mechanized TBM, or MTBM. However, due to the likely presence of groundwater within the tunneling zone, AEC prefers using a MTBM (with a slurry face) because dewatering operations are not typically required during construction. However, if a boring shield or close-faced mechanized TBM are used, dewatering operations along the tunnel alignment are likely to be required. Selection of tunneling method (whether boring shield, mechanized TBM, or MTBM) will ultimately be up to the tunneling Contractor.

Tunneling using a Boring Shield: AEC has the following precautions if tunneling using a boring shield is used, especially when granular soils (such as sands or silts, saturated or not) are present either within or immediately above or below the tunnel zone. These precautions include: (i) slower process compared to excavation with a mechanized TBM, which will in turn lengthen the period of dewatering, causing consolidation of the soils above the tunnel, will cause additional disturbance and increased settlement of the ground surface/roadway above; (ii) less control over the volume of soil removed compared to a mechanized TBM or MTBM, which can increase the volume of excavated soil approximately 1 to 2 percent, resulting in more ground surface settlement. Using mechanized processes (such as a backacter or cutting boom) may be faster than digging by hand, which may help to mitigate the amount of surface settlement. Workman safety against possible tunnel face collapse (especially if flowing soils are encountered), including protection against potential buildup of toxic/noxious gases (if any) will be the sole responsibility of the Contractor.

5.3.2.1 General

The stability of a tunnel or bore face is governed primarily by ground water and subsurface soil conditions, type of method used (either tunnel or trenchless), and workmanship. Based on the subsurface conditions encountered in our borings and the proposed invert depths (see Table 7 in Section 5.3.2.2 of this report), we anticipate that firm to very stiff fat clay (CH) will generally be encountered within the tunnel/auger zone along the siphon pipe alignment. Secondary features such as sand or silt clay seams/pockets/layers were also encountered within the cohesive soils, and could be significant at some locations. In addition, the type and property of subsurface soils are subject to change between borings, and may be different at locations away from the borings.

When granular soils are encountered during construction an unsupported tunnel/bore face (in case of tunneling by boring shield or dry bore/auger by trenchless method) can become unstable. Granular soils below ground water will tend to flow into the excavation hole; granular soils above the ground water level will generally not stand unsupported but will tend to ravel until a stable slope is formed at the face with a slope equal to the angle of repose of the material in a loose state. Thus, granular soils are generally considered unstable in an unsupported excavation face; uncontrolled flowing soil can result in large loss of ground. A MTBM with a slurry face should be able to support the tunnel face if saturated granular soils are encountered, even under groundwater. Similarly, a slurry auger/bore can also support the excavation face if saturated granular soils are encountered, even under groundwater. Similarly, a close-faced TBM should also be able to support the tunnel face within granular soils, although the groundwater level within the granular soil layers will need to be lowered first.

5.3.2.2 Anticipated Ground Behavior

Where granular or soft cohesive soils are encountered, provisions should be made to stabilize the tunnel or bore excavations. AEC notes that granular soils (not necessarily indicated on the boring logs) may be encountered between boring locations; subsurface conditions between boring locations should be verified against the boring logs and AEC notified if different soil conditions are encountered during construction so that additional recommendations can be provided as necessary.

The estimated ground surface settlements caused by volume loss from tunneling method using boring shield, mechanized close face TBM, or MTBM, as well as trenchless method using dry auger/bore or slurry auger/bore are presented in Table 7. The settlement estimates presented in Table 7 include both one pass and two path methods. For two pass method, AEC assumes that a 78 inch diameter steel casing (considering 72 inch diameter siphon pipes) will be used, although other primary liner options can also be considered.

If tunneling method using a boring shield or trenchless method using a dry auger are used, a Stability Factor, $N_t = (P_z - P_a)/C_u$ may be used to evaluate the stability of an unsupported tunnel/bore face in cohesive soils, where P_z is the overburden pressure to the tunnel/bore centerline; P_a is the equivalent uniform interior pressure applied to the face; and C_u is the soil undrained shear strength. For tunneling operations, no interior pressure is applied. Generally, N_t values of 4 or less are desirable as it represents a practical limit below which tunneling/augering may be accomplished without significant difficulty. Higher N_t values usually lead to large deformations of the soil around the tunnel/bore and problems associated with increased subsidence. It should be noted that the exposure time of the face is most important; with time, creep of the soil will occur, resulting in a reduction of shear strength. The N_t values will therefore increase when construction is slow. Estimated N_t values are presented on Table 7.

Note that the cohesive soils have secondary structures such as fissures, sand seams, and sand lenses which can cause the bore face to become unstable. Where granular or soft cohesive soils, if any, are encountered, the Contractor should make provisions for casing to stabilize the tunnel.

The settlement amounts estimated in Table 7 also assume the Contractor practices good workmanship during construction. AEC notes that if the Contractor practices poor workmanship during construction, the amount of settlement could be significantly larger than the amounts estimated in Table 7.
Secondary soil features present within the clay soils (which will be encountered within almost all

tunnel/bore zones) such as sand/silt seams, and slickensides can result local instabilities at the tunnel face.

Table 7. Anticipated Soil Types and Estimated Settlements along Tunnel/Bore Alignments

Soil Boring	Approx. Tunnel/Bore Invert Depth (ft)	Anticipated Soil Types in Tunnel/Bore Zone(2)	One Pass Method		Two Pass Method		Note/Suggestion
			Stability Factor N_t	S_{max}(1) (in)	Stability Factor N_t	S_{max}(1) (in)	
B-1	12.8 (elevation 19' above MSL)	4'-16': Firm to very stiff Fat Clay (CH)	1.5	MTBM or Mechanized TBM: 0.11 Boring Shield or Auger/Bore: 0.45	1.4	MTBM or Mechanized TBM: 0.12 Boring Shield or Auger/Bore: 0.49	Potential swelling ground due to high plasticity fat clay. Pressurized groundwater. Dewatering not required if MTBM or slurry bore/auger is used. Otherwise, dewatering operations will be necessary if boring shield, mechanized TBM, or dry auger/bore is used.
B-2 and G116-13 B-46	12.8/13.1 (elevation 19' above MSL)	4'-16': Firm to very stiff Fat Clay (CH)	1.2	MTBM or Mechanized TBM: 0.10 Boring Shield or Auger/Bore: 0.41	1.2	MTBM or Mechanized TBM: 0.11 Boring Shield or Auger/Bore: 0.45	Potential swelling ground due to high plasticity fat clay. Pressurized groundwater. Dewatering not required if MTBM or slurry bore/auger is used. Otherwise, dewatering operations will be necessary if boring shield, mechanized TBM, or dry auger/bore is used.

Note: (1) S_{max} = Estimated settlement along the tunnel/bore alignment due to volume loss only; not including consolidation settlement, collapse of voids, or lowering of groundwater table.
(2) Tunnel zone takes as one half waterline diameter above tunnel/bore crown to one half waterline diameter below tunnel invert.

AEC notes that the estimated settlements presented in Table 7 do not include settlement from dewatering operations (if a boring shield, mechanized TBM, or dry auger/bore is used), consolidation settlement, or settlement from collapse of voids within the soil around the tunnel/bore. As a result, the actual settlement at the tunnel/bore locations during construction could be more than estimated in Table 7. In addition, if dewatering operations are used in the vicinity of the tunnel/bore (if a boring shield, mechanized TBM, or dry auger/bore is used), or if dewatering is performed in the vicinity of tunnel access shafts, additional settlement will also occur due to increases in effective stress of the soil strata.

The information in this report should be reviewed so that appropriate tunneling/boring equipment and operation can be planned and factored into the construction plan and cost estimate. If tunneling method is used, Plate D-8 in Appendix D provides a general guideline for TBM selection.

5.3.2.3 Influence of Tunneling/Boring on Existing Structures

We estimated the resulting influence zone (extending from the centerline of the tunnel/bore, see Plate D-9, in Appendix D) to range from approximately 16.3 feet at Boring B-1 to 17.9 feet at Borings B-2 and G116-13 Boring B-46. The estimated maximum settlements [caused by volume loss only, not including consolidation settlement, collapse of voids (if any) within the soil, or settlement from dewatering] along the tunnel/bore alignment are included in Table 7 in Section 5.3.2.2 of this report.

AEC emphasizes that the size of the influence zone of a tunnel/bore is difficult to determine because several factors influence the response of the soil to tunneling/trenchless operations including type of soil, ground water level and control method, type of tunneling/boring equipment, tunneling/trenchless operations, experience of operator, and other construction in the vicinity. Methods to prevent movement and/or distress to existing structures will require the services of a specialty contractor.

5.3.3 Measures to Reduce Distress from Tunneling/Boring

If a one pass method will be used, AEC recommends the use of slurry bore for trenchless method or MTBM for tunnel method during construction. If a two pass method will be used, AEC recommends the use of a steel casing as a primary liner to support the tunnel/bore excavation during construction. Considering the ground conditions discussed in Table 7 in Section 5.3.2.2 of this report, if excessive voids occur during tunneling/boring, the Contractor should immediately and completely grout the annular space (when tunneling/boring in stiff to hard clays) between the steel casing and the ground at the tail of the machine, in accordance with Section 31 71 02.02 of the latest edition of the SJRA Construction Specifications. It should be noted that grouting may increase friction resistance while advancing the casing and the contractor will need to address this condition as part of their tunnel work plan. Grouting the annular space will most likely not be possible when tunneling in granular soils, or in weak/soft clays. Plate D-10, in Appendix D provides a general guideline for selection of grouting material. The tunneling machine selection, tunneling operation, and grouting (as necessary) will be the full responsibility of the Contractor.

To reduce the potential for the tunneling/boring to influence existing foundations or structures, we recommend that the outer edge of the influence zone of the tunnel/bore be a minimum of 5 feet from the outer edge of the bearing (stress) zone of existing foundations. The bearing (stress) zone is defined by a line drawn downward from the outer edge of an existing foundation and inclined at an angle of 45 degrees to the vertical.

We recommend that the following situations be evaluated on a case by case basis, where:

- tunneling/boring cannot be located farther than the minimum distance recommended above;
- tunneling/boring cannot be located outside the stress zone of the foundations for existing structures;
- unstable soils are encountered near existing structures;
- heavily loaded or critical structures are located close to the influence zone of the tunnels/bores;

As an option, existing structure foundations should be protected by adequate shoring or strengthened by underpinning or other techniques, provided that tunneling/boring cannot be located outside the stress zone of the existing foundations.

Disturbance and loss of ground from the tunneling/trenchless operation may create surface soil disturbance and subsidence which in turn may cause distress to existing structures (including underground utilities and pavements) located in the zone of soil disturbance. Any open-cut excavation in the proposed tunneling/boring areas should be adequately shored.

5.3.4 Monitoring Existing Structures

The Contractor should be responsible for monitoring existing structures nearby and taking necessary action to mitigate impact to adjacent structures. Existing structures located close to the proposed construction excavations should be surveyed prior to construction and pre-existing conditions of such structures and their vicinity be adequately recorded. This can be accomplished by conducting a pre-construction survey, taking photographs and/or video, and documenting existing elevations, cracks, settlements, and other existing distress in the structures. The monitoring should include establishment of elevation monitor stations, crack gauges, and inclinometers, as required. The monitoring should be performed before, periodically during, and after construction. The data should be reviewed by qualified engineers in a timely manner to evaluate the impact on existing structures and develop plans to mitigate the impact, should it be necessary.

5.4 Siphon Inlet/Outlet Structures

Based on the drawing provided by TWE, the dual 72 inch diameter siphon pipes will have a flowline invert elevation of 19 feet above MSL. The inlet/outlet structure headwalls will be supported on a mat foundation that will bear at an elevation of 16.5 feet above MSL.

Design of the siphon structure headwalls and wingwalls should consider the allowable bearing capacity of the foundation soils, sliding, and overturning stability. We recommend using a factor of safety (FS) of 2 for passive earth pressure, a FS of 1.5 for sliding, and a FS of 2 for overturning stability of the walls.

5.4.1 Allowable Bearing Capacity

Based on the drawing provided by TWE, the proposed inlet/outlet structures will be supported on a mat foundation that bears at an elevation of approximately 16.5 feet above MSL. Based on Borings B-1, B-2, and G116-13 Boring B-46, an inlet/outlet structure footing bearing at an elevation of approximately 16.5 feet above MSL can be designed for an allowable net bearing capacity of 1,400 pounds per square foot (psf) for sustained loads and 2,100 psf for total loads. These allowable bearing pressures include a minimum FS of 3 for sustained loads and 2 for total loads, whichever is more critical should be used for design.

Modulus of Subgrade Reaction: The modulus of subgrade reaction (k) is frequently used in the structural analysis of mat foundations. Based on the soil conditions encountered, we recommend using k = 25 pounds per cubic inch (pci) for a mat foundation founded at an elevation of 16.5 feet above MSL.

Foundation Settlement: AEC understands that TWE will perform settlement analysis of the inlet/outlet structure mat foundations. Design soil parameters for mat foundation settlement analyses are presented on Plate C-8, in Appendix C.

5.4.2 Lateral Earth Pressures

The inlet/outlet structure headwalls and wingwalls will be subjected to lateral earth pressures. The magnitudes of the lateral earth pressures will depend on the type and density of the backfill, surcharge on the backfill, and hydrostatic pressure. If the backfill is over-compacted or if highly plastic clays are placed behind the walls, the lateral earth pressure could exceed the vertical pressure from the weight of the backfill. Lateral pressure resulting from construction equipment, and traffic, or other surcharge on the top of the walls should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure. We recommend that at least 250 psf surcharge be considered for design of the walls. Hydrostatic pressure should also be included, unless adequate drainage is provided behind the walls (see Section 5.4.4 of this report).

The wall design should be based on whether or not the top of the walls will be allowed to move. If the walls will be allowed to move, then the wall design should be based on active earth pressure parameters. If the walls will not be allowed to move (i.e. considered fully restrained), then the wall design should be based on at-rest earth pressure parameters. Selection of which earth pressure condition to use (either active or at-rest) should be determined by the wall designer. Wall design should consider short-term and long-term conditions; whichever case is critical should be used for the design. Active earth pressure can be determined in accordance with Equation (6) in Section 5.2.3 of this report. The at-rest earth pressure at depth z can be determined by Equation (8). Design soil parameters that can be used for wall design are presented on Plate C-2, in Appendix C.

$$p_0 = (q_s + \gamma h_1 + \gamma' h_2) K_0 + \gamma_w h_2 \qquad \text{............Equation (8)}$$

where,

	p_0	=	at-rest earth pressure, psf.
	q_s	=	uniform surcharge pressure, psf.
	γ, γ'	=	wet and buoyant unit weights of soil, pcf.
	h_1	=	depth from ground surface to ground water table, feet.
	h_2	=	$z-h_1$, depth from ground water table to point under consideration, feet.
	z	=	depth below ground surface, feet.
	K_0	=	coefficient of at-rest earth pressure.
	γ_w	=	unit weight of water, 62.4 pcf.

The short-term and long-term lateral earth pressure design soil parameters presented on Plate C-2, in Appendix C should be used for head/wing walls design. The values of soil cohesion, c, and c', can conservatively be omitted for lateral earth pressure design. AEC notes that the coefficients of active, at-rest, and passive earth pressure provided in Plate C-2, in Appendix C are for level backfill behind the head/wing walls. Coefficients of active and passive earth pressure for sloped backfill can be determined using Plate C-3, in Appendix C.

Sliding Resistance: The resistance of a structure wall footing against sliding due to lateral loads is a combination of soil friction resistance, soil adhesion resistance (on the footing), plus passive pressure resistance in front of the wall (if any). Sliding resistance can be determined using Equation (9). Passive pressure resistance can conservatively be omitted from design. Otherwise, if passive earth pressure resistance is considered in the design, a FS of 2 should be applied to the passive pressure component. Passive earth pressure resistance can be determined using Equation (7) in Section 5.3.1 of this report. Design soil parameters for sliding resistance are presented on Plate C-2, in Appendix C.

$$\Sigma F_r = \Sigma V \times \tan(\delta) + B_f \times C_\alpha + P_p \qquad \text{...........Equation (9)}$$

where: ΣF_r = Sum of horizontal resisting forces
ΣV = Sum of vertical forces
δ = $2/3\ \phi$
ϕ = angle of internal friction
B_f = width of footing (ft)
C_α = soil adhesion (psf)
P_p = passive pressure resistance [see equation (7) in Section 5.3.1 of this report].

5.4.3 Hydrostatic Uplift

For hydrostatic uplift, AEC recommends the structure design consider a most conservative case of when the siphon pipes are empty, but the design water level is at the top of wall or 100-year flood elevation, whichever is more critical. If the dead weight of the structure (including the backfill on top of the pipe) and the frictional resistance between the wall and backfill are inadequate to resist uplift forces, the width of the wall footing and the wall thickness can be increased to provide additional uplift resistance. The buoyant unit weight of concrete can be taken as 90 pcf. The minimum recommended factors of safety against uplift should be 1.1 for concrete weight, 1.5 for soil weight and 3.0 for soil friction. Structure uplift design should consider short-term and long-term conditions, whichever is critical. Design soil parameters are presented on Plate C-2, in Appendix C. Recommended design criteria for uplift resistance are shown on Plate D-11, in Appendix D.

5.4.4 Drainage System

If possible, AEC recommends a 1-foot wide, vertical "bank sand chimney" drainage system or equivalent prefabricated drainage element be placed behind the head/wing walls. Bank sand can be composed of SP, SW, SP-SM, SW-SM, and some SM type soils in accordance with the USCS classification system, with less than 15 percent passing the No. 200 sieve, and amount of clay not exceeding 2 percent by weight. The entirety of the bank sand chimney should be wrapped in a geotextile filter fabric. In addition, a 12 to 24 inch thick select fill clay cap should be placed on top of the drainage chimney to prevent the infiltration of surface runoff into the drainage chimney. The select fill clay cap should be in accordance with Section 5.5 of this report. The drainage system should drain water through weepholes in the wall face, or into a perforated drainage pipe (also wrapped with a geotextile filter fabric) that connects to a sump or a storm sewer. If weepholes are used, we recommend at least 3-inch diameter weepholes installed at a spacing of 10 feet on center or less, both horizontally and vertically. The drainage system should be regularly maintained and repaired as necessary so that hydrostatic pressures do not develop behind the wall. If the drainage system is not used, then the wall designer should consider hydrostatic pressure in the wall design, as discussed in Section 5.4.2 of this report.

5.4.5 Siphon Inlet/Outlet Construction

Based on Borings B-1, B-2, and G116-13 Boring B-46, the top 15.5 (i.e. at an elevation of approximately 16.5 feet above MSL) feet of the subsurface soil conditions at the siphon inlet/outlet structures consist of firm to hard fat clay. Evaluation of foundation excavation, bottom stability, and shoring/bracing (if needed) should be performed in general accordance with the excavation stability guidelines presented in Section 5.2.2 of this report.

Based on the groundwater encountered in Borings B-1, B-2, and G116-13 Boring B-46, groundwater control may be required during siphon inlet/outlet construction. Dewatering guidelines are presented in Section 6.2 of this report.

AEC anticipates that the inlet/outlet structures will be installed by open cut method. Siphon inlet/outlet construction should be performed in general accordance with Section 31 23 16.16 of the latest edition of the SJRA Construction Specifications. After excavation, any soft or compressible materials and ponded water should be removed from the bottom of the excavation prior to placement of concrete; such materials should be replaced with compacted select fill. Select fill should be in accordance with Section 5.5 of this report.

Excavation Extents: For backfill purposes (see below), AEC recommends that the structure excavation extend an additional 2 feet horizontally past the edge of the mat foundation and slope upward at a minimum

H:V = 1:1 inclination to the ground surface. The excavation should also be benched as necessary; AEC recommends that the bench height not exceed 4 feet, and the bench width be at least 8 feet.

Backfill: AEC recommends compacted select fill be used as backfill behind the head/wing walls. Select fill criteria are presented in Section 5.5 of this report. Based on Borings B-1, B-2, and G116-13 Boring B-46, the top 18 feet of soils at the site are high plasticity fat clay. Due to its high expansive potential, AEC does not recommend excavated onsite soils to be reused as wall backfill, since the expansive clay can impart additional loading on the walls. Furthermore, the magnitude of swell pressures on the walls will be difficult to predict, and would depend on PI, moisture content, degree of compaction at the time of fill placement, as well as change in moisture content within the backfill throughout the life of the structure. Regardless, AEC does not recommend the onsite in-situ material be re-used as wall backfill.

5.5 Select Fill

"Select" Fill: It is AEC's experience that "select" fill material imported from sand and clay pits in the Greater Houston area is generally non-homogenous (i.e. composed of a mixture of sands, silts, and clays, instead of a homogenous sandy clay material) and of poor quality, and either contains too much sand or has large clay clods with high expansive potential. Use of this non-homogenous soil can result in poor long term performance of structures and pavements placed on top of the fill.

Select fill (whether imported from offsite or is already onsite) should consist of ***uniform***, non-active inorganic lean clays with a PI between 10 and 20 percent, and more than 50 percent passing a No. 200 sieve. Material intended for use as select fill shall not have clay clods with PI greater than 20, clay clods greater than 2 inches in diameter, or contain sands/silts with PI less than 10. Sand and clay mixtures/blends are unacceptable for use as select fill. Sand/silt with clay clods is unacceptable for use as select fill. Mixing sand into clay or mixing clay into sand/silt is also unacceptable for use as select fill. **Prior to construction, the Contractor should determine if he or she can obtain qualified select fill meeting the above select fill criteria. The testing lab shall *reject* any material intended for use as select fill that does not meet the PI, sieve, and clay clod requirements above, without exceptions.**

Lifts and Compaction: All material intended for use as select fill should be tested prior to use to confirm that it meets select fill criteria. The fill should be placed in loose lifts not exceeding 8 inches in thickness. Backfill within 3 feet of the siphon, headwalls, wingwalls, or other adjacent structures should be placed in loose lifts no more than 4-inches thick and compacted using hand tampers, or small self-propelled compactors.

Select fill should be compacted to a minimum of 95 percent of the ASTM D 698 (Standard Proctor) maximum dry unit weight at a moisture content ranging between optimum and 3 percent above optimum.

Testing: **If select fill will be used, at least one Atterberg Limits and one percent passing a No. 200 sieve test shall be performed for each 5,000 square feet (sf) of placed fill, per lift (with a minimum of one set of tests per lift), to determine whether it meets select fill requirements.** Prior to placement of pavement or concrete, the moisture contents of the top 2 lifts of compacted select fill shall be re-tested (if there is an extended period of time between fill placement and concrete placement) to determine if the in-place moisture content of the lifts have been maintained at the required moisture requirements.

5.6 SJRA Canal at Siphon 29

Plan and profile and cross section drawings along the SJRA canal immediately upstream and downstream of Siphon 29 were prepared by TWE and provided to AEC. The provided cross section locations are presented on Plate E-1 and the cross sections themselves are presented on Plate E-2, in Appendix E. Based on the drawings provided by TWE, the canal depth varies approximately from 7.0 to 7.4 feet and the existing canal interior slopes have an inclination varying from H:V = 2.3:1 to 2.9:1. According to TWE, the 100 year water surface elevation (WSE) is at an approximate elevation of 28 feet above MSL and the ordinary WSE is between 29 to 30 feet above MSL.

5.6.1 Slope Stability Analysis

AEC performed slope stability analysis on the interior east bank of the upstream cross section at Station 0+81.49 (see Plate E-2, in Appendix E), based on the soil conditions encountered in G116-13 Boring B-46. AEC performed the analysis based on three different conditions: short-term condition, long-term condition, and rapid drawdown condition. AEC performed the stability analyses in general accordance with the requirements of the December 2010 HCFCD Geotechnical Guidelines.

5.6.2 Design Soil Parameters and Profiles

Soil parameters used in the analyses include moist unit weights, UU shear strengths, effective stress shear strength (developed using total stress parameters and pore water pressure), and total stress shear strength. AEC selected the interior east bank at Station 0+81.49 along the canal at Siphon 29 as the 'most-critical' cross section to perform slope stability analyses, since it has the steepest interior slope amongst the provided cross sections. Design soil parameters for the slope stability analysis are presented on Plate E-3, in Appendix E.

Clay Desiccation Zone: In general, very high plasticity fat clay soils were encountered in the borings along the canal. Exposing these fat clays to the atmosphere and cycles of wetting-drying from seasonal moisture changes will result in desiccation, cracking, and progressive movement of these clays, and a reduction in their shear strengths. We considered the desiccation zone for fat clay from the top of levee or levee slope surface to the assumed seepage line in the existing levee. For fat clay within the desiccation zone, we used effective residual shear strengths of c'_r = 65 psf and ϕ'_r = 21 degrees to evaluate slope stability for long-term and rapid drawdown conditions. We also reduced the c' and c_{cu} of clay soils (with a PI greater than 20) within the non-desiccated zone based on our experience with similar levee projects in the Houston area based on a combination of methods by G. Mesri (1999) and S. Wright (2005) for both the long-term condition and rapid drawdown condition.

5.6.3 Conditions Analyzed for Slope Stability

We used the Simplified Bishop Method of Slices option in the GeoStudio 2018 computer program (SLOPE/W) to analyze slope stability for 2-dimensional limiting equilibrium. The program has the capability to compute pore water pressures based on a defined piezometric surface.

Groundwater Level: For the analyses, we considered different groundwater conditions for short term, long term, and rapid drawdown conditions. For the short term and long term conditions, we considered the water level within the canal to be at the ordinary WSE, and the groundwater outside to the canal to be equal to the seepage line where the groundwater has saturated the levee soil below the defined piezometric surface. For the rapid drawdown condition, we considered the water level within the canal to be at the ordinary WSE (30 feet above MSL), and the water level outside the canal to be at the ground surface. The rapid drawdown condition considers that the groundwater level rapidly changes from the ground surface/ordinary WSE to the bottom of the canal, such as a condition where the canal is drained for maintenance purposes.

Required Safety Factor: HCFCD requires a minimum Safety Factor (SF) of 1.3 for short-term conditions, 1.5 for long-term conditions, and 1.25 for rapid drawdown conditions. Stability analyses for the channel slopes were conducted for the short-term (end-of-construction), long-term, and rapid drawdown conditions. A brief description of these conditions is presented below:

1. Short Term (i.e. End-of-Construction Condition) - This condition models rapid construction loading taking place, so that there is no time for the induced excess pore water pressure to dissipate or for consolidation to occur during the loading period. Unconsolidated-undrained shear strength parameters were used for this analysis.

2. Long-Term Condition - This condition models long-term steady seepage through embankments and the long-term stability of slopes in clays. Consolidated-drained effective stress shear strength parameters (obtained from CU triaxial tests with pore water pressure measurements) were used for this analysis.

3. Rapid Drawdown Condition - The majority of slope failures in the Harris County/Houston area occur under rapid drawdown conditions. This condition models when the slope becomes fully saturated and consolidated and is at equilibrium with the existing stress system, then encounters rapid drawdown and simultaneously allows no drainage to occur. Consolidated-undrained total stress shear strength with pore pressures parameters modeling rapid drawdown conditions were used for this analysis.

AEC considered global slide (GS) and local slide (LS) conditions in the slope stability analysis. GS condition can be defined as a global slope failure where the failure circle passes through and/or below the slope toe, which can cause a catastrophic failure of the levee. For the LS condition, the slope failure occurs locally on the slope with a limited volume of slope movement. However, a LS condition can cause progressive slope movement and eventually become a global slide.

5.6.4 Slope Stability for East Bank of Upstream Section at Station 0+81.49

We performed slope stability analyses on the interior east bank of the upstream section at Station 0+81.49 based on the soil conditions encountered in G116-13 Boring B-46. The analyses were performed on the existing canal slopes. Design soil parameters used for the slope stability analyses are presented on Plate E-3, in Appendix E. A 300 psf construction surcharge was added to the top of bank for the short term condition.

The results of the slope stability analyses of the interior east bank of the canal levee at Station 0+81.49 under short-term, long-term, and rapid drawdown conditions are presented on Plates E-4 through E-9, in Appendix E. The SF of the canal levee slopes under short term, long term, and rapid drawdown conditions are in Table 8.

Table 8. Slope Stability Analysis Results for East Bank of Canal at Station 0+81.49

Slope Condition	Calculated Safety Factor (SF)		
	Short-Term	Long-Term	Rapid Drawdown
Existing Slope, Canal Side	6.05 (GS, Plate E-4) 24.6 (LS, Plate E-5)	2.27 (GS, Plate E-6) 2.88 (LS, Plate E-7)	1.30 (GS, Plate E-8) 1.79 (LS, Plate E-9)

Notes: (1) G.S. = global slide; (2) L.S. = local slide.

Based on the summary in Table 8, the calculated SF's of the existing canal levee interior slope meets HCFCD requirements under short term, long term, and rapid-drawdown conditions.

<u>Exterior Levee Slope:</u> According to the drawings provided by TWE, maximum height of the exterior slope on both sides of the existing levee is approximately 2.13 feet with a steepest slope inclination of H:V = 3.2:1. Based on the soils encountered in our borings, slope height and inclination, and our experience, the exterior levee slope should be stable. For reference, HCFCD does not typically require slope stability analyses for levees with a height of less than 3 feet.

5.7 Rip Rap

Slope failures due to local slides were observed on both of the banks of the canal near the siphon structures and displacement of the existing riprap was also seen southwest of Siphon 29 (see Plate B-1, in Appendix B). Distress of the canal levees include local slides, sloughing and undermining caused by scouring, and desiccation cracks in the soils on the top of the levee, which can result in progressive slope movement in the future and which might have an adverse impact on the headwalls/wingwalls. To protect the levee slopes, a minimum 24 inch thick layer of stone rip rap can be used in accordance with Section 31 37 00 of the latest edition of the SJRA Construction Specifications. An extra volume of rip rap should be included at the slope toes (ideally the rip rap toe should be trenched to below the depth of potential for rip rap degradation and contraction scour) to mitigate the potential for undermining of the rip rap toe due to wave action or flow at the siphon inlet/outlet areas. All stone rip rap should be underlain by a geotextile filter fabric that is suitable for mechanically separating the riprap and underlying soils. AEC also recommends that rip rap be provided around the inlet/outlet structure headwalls and wingwalls to mitigate scouring effects, especially where granular soils are present.

Synthetic Filter Fabric: We recommend that non-woven geotextile fabric be used and placed between the riprap and the underlying soils to prevent soil movement into or through the riprap. AEC recommends adding a chart to the construction drawings that provides the requirements for non-woven general filter fabric properties to be placed under all riprap. The contractor should submit proposed filter fabric design calculations and specifications for approval by the Construction Manager before installing the fabric under the riprap.

The design of geotextile filter fabric should address: (1) retention criterion - the geotextile must retain the soil; (2) permeability criterion - the geotextile opening size must allow water to pass through the geotextile; (3) clogging resistance criterion - over the life of the structure; and (4) survivability criterion - the geotextile must survive the installation process. The geotextile filter fabric should be in accordance with the requirements of Section 31 38 25 of the latest edition of the SJRA Construction Specifications. Additional filter fabric requirements are presented on Table 9 below.

Table 9. Filter Fabric Requirements

Physical Properties	Test Method	Type 2 Requirements
Fabric Weight, on an ambient temperature air-dried tension-free sample	--	12 oz/yd^2, minimum
Porosity	--	30%, minimum
Permittivity, 1/sec	ASTM D 4491	$K_{Fabric} > 10K_{soil}$
Tensile Strength, N	ASTM D 4632	890 N (200 lbs), minimum
Apparent Opening size	ASTM D 4751	80-120
Elongation at yield, %	ASTM D 4632	20-100
Trapezoidal Tear, N	ASTM D 4533	490 N (110 lbs), minimum

The geotextile filter fabric should be overlapped on the edges by at least 2 feet, and the anchor pins be spaced every 3 feet along the overlap. The upper and lower ends of the cloth should be buried a minimum of 12 inches below ground. Precautions should be taken to not damage the cloth by dropping the riprap. If damage occurs, the riprap should be removed, and the sheet repaired by adding another layer of filter fabric with a minimum overlap of 12 inches around the damaged area. Where large stones are to be placed, a 4-inch layer of fine sand or gravel is recommended to protect the filter cloth.

Riprap: The gradation and installation of riprap should be in accordance with Section 31 37 00 of the latest edition of the SJRA Construction Specifications. Placement of the riprap should follow immediately after placement of the filter. Riprap should be placed so that it forms a dense, well-graded mass of stone (or concrete) with minimum voids. Place riprap to its full thickness in one operation.

Riprap Maintenance: AEC recommends the riprap be inspected periodically for scour or dislodged stones. Missing or dislodged riprap should be replaced as soon as possible. Damaged filter fabric should also be patched and repaired.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Site Preparation

To mitigate site problems that may develop following prolonged periods of rainfall, it is essential to have adequate drainage to maintain a relatively dry and firm surface prior to starting any work at the site. Adequate drainage should be maintained throughout the construction period. Methods for controlling surface runoff and ponding include proper site grading, berm construction around exposed areas, and installation of sump pits with pumps.

6.2 Groundwater Control

The need for groundwater control will depend on the depth of excavation relative to the groundwater depth at the time of construction. In the event that there is heavy rain prior to or during construction, the groundwater table may be higher than indicated in this report; higher seepage is also likely and may require a more extensive groundwater control program. In addition, groundwater may be pressurized in certain areas of the alignment, requiring further evaluation and consideration of the excess hydrostatic pressures.

The Contractor should be responsible for selecting, designing, constructing, maintaining, and monitoring a groundwater control system and adapt his operations to ensure the stability of the excavations. Groundwater information presented in Section 4.1 and elsewhere in this report, along with consideration for

potential environmental and site variation between the time of our field exploration and construction, should be incorporated in evaluating groundwater depths. The following recommendations are intended to guide the Contractor during design and construction of the dewatering system.
In cohesive soils seepage rates are lower than in granular soils and groundwater is usually collected in sumps and either pumped out or channeled by gravity flow to storm sewers. If cohesive soils contain significant secondary features, seepage rates will be higher. This may require larger sumps and drainage channels, or if significant granular layers are interbedded within the cohesive soils, methods used for granular soils may be required. Where it is present, pressurized groundwater will also yield higher seepage rates.

Groundwater for excavations within saturated sands can be controlled by the installation of wellpoints. The practical maximum dewatering depth for well points is about 15 feet. When groundwater control is required below 15 feet, multiple staged wellpoints or eductors (for silts or silty sands) have generally proved successful. AEC recommends that the groundwater table be lowered to at least 3 feet below the excavation bottom to be able to work on a firm surface when water-bearing granular soils are encountered. Groundwater control should be in accordance with Section 01 57 23.02 of the latest edition of the SJRA Construction Specifications. Another groundwater control option is to use water-tight sheet pile cutoff walls to seal off water bearing sand/silt layers (see Section 5.3.1 of this report).

Extended and/or excessive dewatering can result in settlement of existing structures in the vicinity; the Contractor should take the necessary precautions to minimize the effect on existing structures in the vicinity of the dewatering operation. We recommend that the Contractor verify the groundwater depths and seepage rates prior to and during construction and retain the services of a dewatering expert (if necessary) to assist him in identifying, implementing, and monitoring the most suitable and cost-effective method of controlling groundwater.

For open cut construction in cohesive soils, the possibility of bottom heave must be considered due to the removal of the weight of excavated soil. In lean and fat clays, heave normally does not occur unless the ratio of Critical Height to Depth of Cut approaches one. In silty clays, heave does not typically occur unless an artificially large head of water is created through the use of impervious sheeting in bracing the cut. Guidelines for evaluating bottom stability are presented in Section 5.2.2 of this report.

6.3 Construction Monitoring

If applicable, excavation, bedding, and backfilling of underground utilities should be monitored by qualified geotechnical professionals to check for compliance with project documents and changed conditions, when encountered. AEC should be allowed to review the design and construction plans and specifications prior to release to check that the geotechnical recommendations and design criteria presented herein are properly interpreted.

6.4 Monitoring of Existing Structures

Existing structures in the vicinity of the project area should be closely monitored prior to, during, and for a period after excavation. Several factors (including soil type and stratification, construction methods, weather conditions, other construction in the vicinity, construction personnel experience, and supervision) may impact ground movement in the vicinity of the alignment. We therefore recommend that the Contractor be required to survey and adequately document the condition of existing structures in the vicinity of the proposed alignment.

7.0 LIMITATIONS

The information contained in this report summarizes conditions found on the dates the borings were drilled. The attached boring logs are true representations of the soils encountered at the specific boring locations on the dates of drilling. Reasonable variations from the subsurface information presented in this report should be anticipated. If conditions encountered during construction are significantly different from those presented in this report; AEC should be notified immediately.

This investigation was performed using the standard level of care and diligence normally practiced by recognized geotechnical engineering firms in this area, presently performing similar services under similar circumstances. This report is intended to be used in its entirety. The report has been prepared exclusively for the project and location described in this report. If pertinent project details change or otherwise differ from those described herein, AEC should be notified immediately and retained to evaluate the effect of the changes on the recommendations presented in this report, and revise the recommendations if necessary. The recommendations presented in this report should not be used for other structures located along these alignments or similar structures located elsewhere, without additional evaluation and/or investigation.

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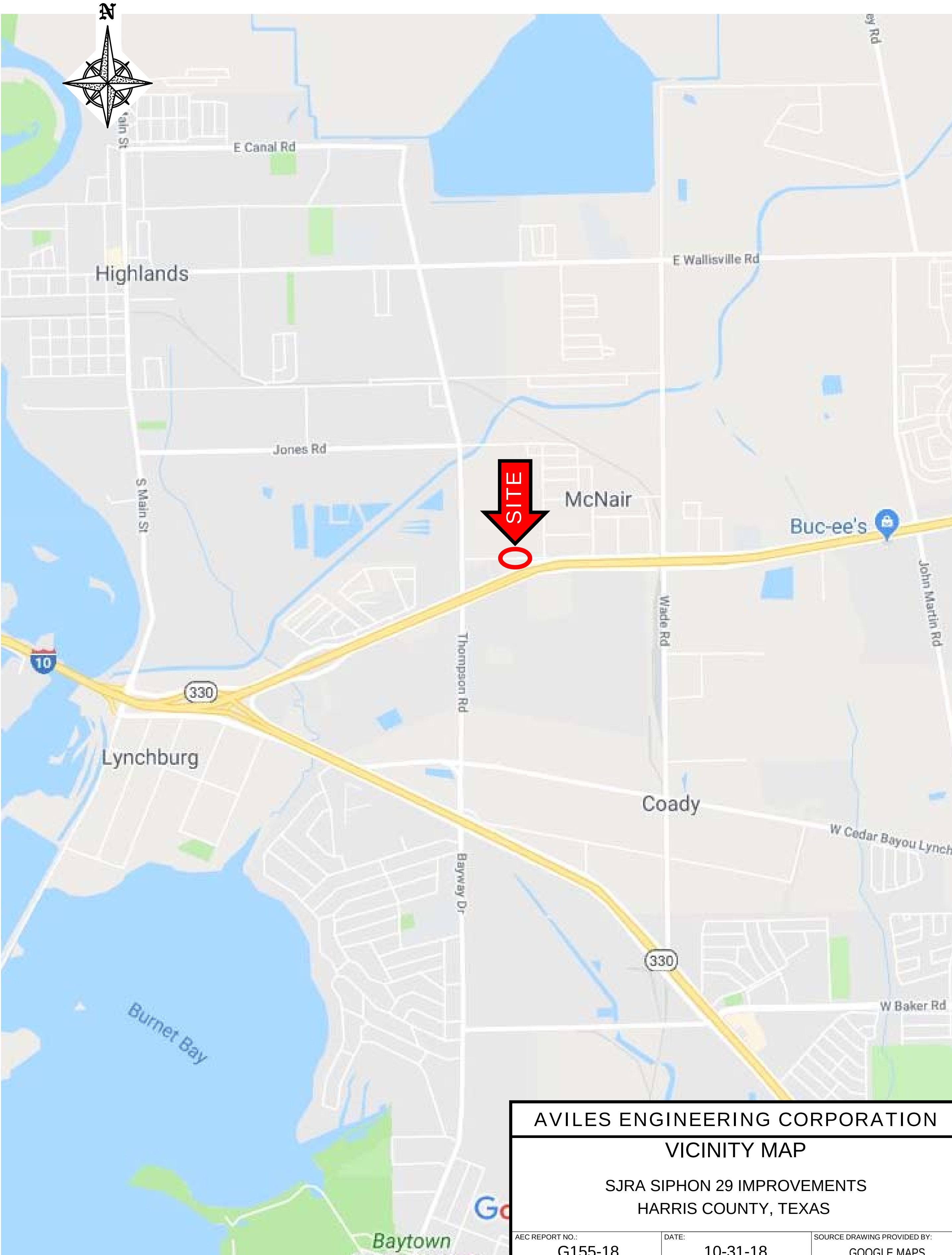
APPENDIX A



AVILES ENGINEERING CORPORATION

VICINITY MAP

SJRA SIPHON 29 IMPROVEMENTS
HARRIS COUNTY, TEXAS

AEC REPORT NO.: G155-18	DATE: 10-31-18	SOURCE DRAWING PROVIDED BY: GOOGLE MAPS
APPROX. SCALE: N.T.S.	DRAFTED BY: MRB	PLATE NO.: PLATE A-1

LEGEND:

B-# (X') BORING LOCATION (DEPTH IN FEET)

B-# (X') PZ-# (X') PREVIOUSLY DRILLED BORING AND PIEZOMETER (DEPTH IN FEET) AEC PROJECT NO. G116-13

NOTE: BORING LOCATIONS ARE APPROXIMATE.

AVILES ENGINEERING CORPORATION

BORING LOCATION PLAN

SJRA SIPHON 29 IMPROVEMENTS
HARRIS COUNTY, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DRAWING PROVIDED BY:
G155-18	09-05-18	GOOGLE EARTH
APPROX. SCALE:	DRAFTED BY:	PLATE NO.:
1" = 100'	CHL	PLATE A-2

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

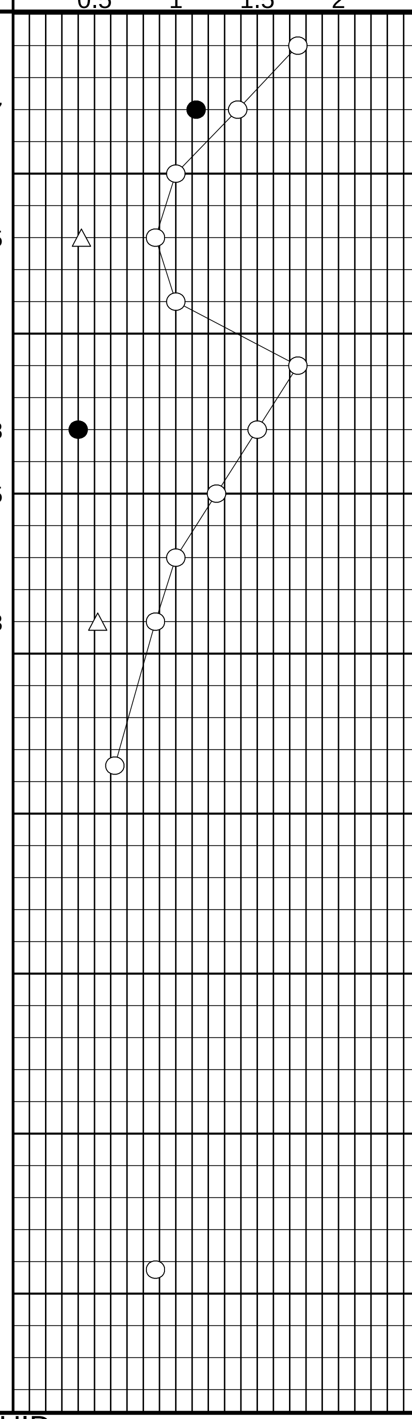
PROJECT: **Improvements of SJRA Siphon 29 at Ellis School** BORING **B-1**

DATE **9/7/18** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DESCRIPTION

GRID Coordinates (US Survey ft):
Texas State Plane Zone: 4204
Easting: 3,227,693.675
Northing: 13,859,924.290
Elevation: 31.769

SHEAR STRENGTH, TSF: △ Confined Compression; ● Unconfined Compression; ○ Pocket Penetrometer; □ Torvane (scale 0.5, 1, 1.5, 2)



DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: very stiff, dark olive and tan Fat Clay (CH), with shell -with calcareous nodules 0'-2' -dark brown 2'-4'		22			69	23	46
			26	95.7				
	Firm to very stiff, dark gray and dark olive-gray Fat Clay (CH), with slickensides		29					
6	-groundwater at 7.6' approx. 15 min. after initial encounter		33	89.5				
	-tan and gray 8'-18', with ferrous nodules 8'-10' and calcareous nodules 8'-16' -with ferrous stains 10'-14'		30		87	70	23	47
			22					
12			27	99.8				
			30	93.5		78	25	53
	-with sandy clay pockets and silty clay seams 16'-18'		23					
18	Stiff, tan Sandy Lean Clay (CL), with silty clay pockets		26	99.3				
	-with fat clay seams 23'-24'		22		56	33	18	15
24	Tan Silty Sand (SM), with fat clay seams, wet							
30	Stiff, tan and gray Sandy Lean Clay (CL), with calcareous nodules	12	21					
	Loose, tan Silty Sand (SM), with clayey sand pockets, wet	8	25		46			
36	Stiff, tan and grayish tan Fat Clay (CH), with ferrous stains	10	22					
42	Termination depth = 40 feet.							

BORING DRILLED TO **18** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **18** FEET WHILE DRILLING

WATER LEVEL AT **6.3** FEET AFTER **9/10/18**

DRILLED BY **V&S** DRAFTED BY **BpJ** LOGGED BY **BTC**

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Improvements of SJRA Siphon 29 at Ellis School** BORING **B-2**

DATE **9/7/18** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DESCRIPTION

GRID Coordinates (US Survey ft):
Texas State Plane Zone: 4204
Easting: 3,227,773.863
Northing: 13,859,804.270
Elevation: 31.842

Firm to hard, dark brown Fat Clay (CH), with slickensides
-with roots 0'-2'
-dark olive-gray and black 2'-4'
-dark gray 4'-8'
-with ferrous nodules 6'-10'
-grayish tan 8'-10', with calcareous nodules 8'-12'
-tan and gray 10'-18', with ferrous stains 10'-16'
-groundwater at 12.6' approx. 15 min. after initial encounter
-with calcareous nodules 14'-16'
-with silty clay seams 16'-18'

Very stiff, tan and gray Lean Clay (CL), with silty clay and fat clay pockets

Soft to stiff, tan Sandy Lean Clay (CL), with abundant silty clay pockets, wet
-with silty clayey sand seams 23'-25'
-tan and gray, with silty sand pockets 28'-30'
-with fat clay seams 33'-35'

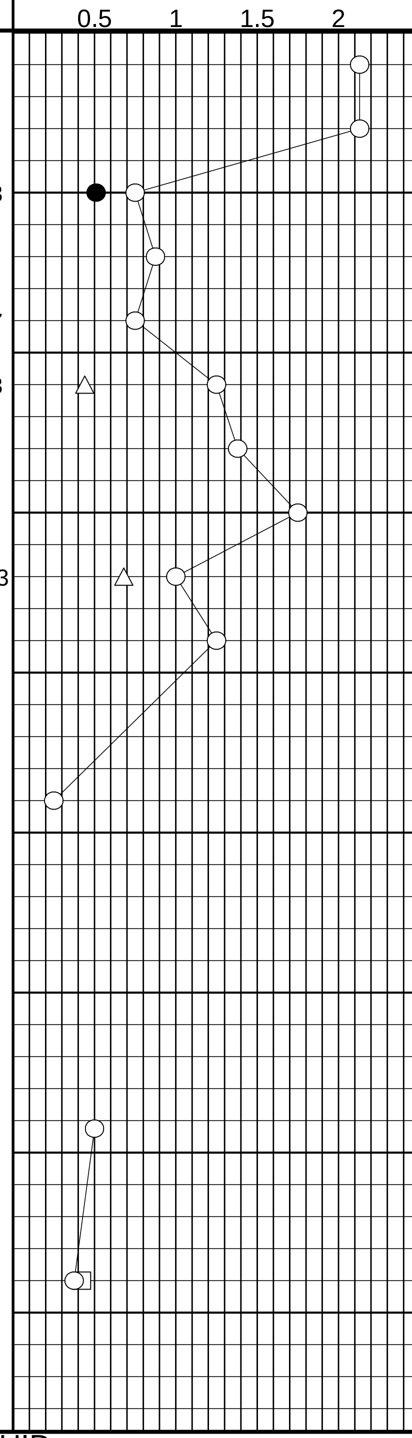
Tannish gray and gray Sandy Silt (ML), with silty clay seams, wet

Termination depth = 40 feet.

DEPTH IN FEET	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0		25					
		26			72	24	48
		29	93.3				
6		30					
		30	92.7		75	23	52
		27	96.3				
12		30					
		29			75	28	47
		24	102.3				
18		23		98			
24		24		57	28	18	10
30	5	30					
	7	25					
36		23		52			
42							

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **20** FEET WHILE DRILLING
WATER LEVEL AT **6.5** FEET AFTER **9/10/18**
DRILLED BY **V&S** DRAFTED BY **BpJ** LOGGED BY **BTC**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: SJRA Phase Ib, South Canal BORING B-46

DATE 6/4/13 TYPE 4" Dry Auger / Wet Rotary LOCATION See Boring Location Plan

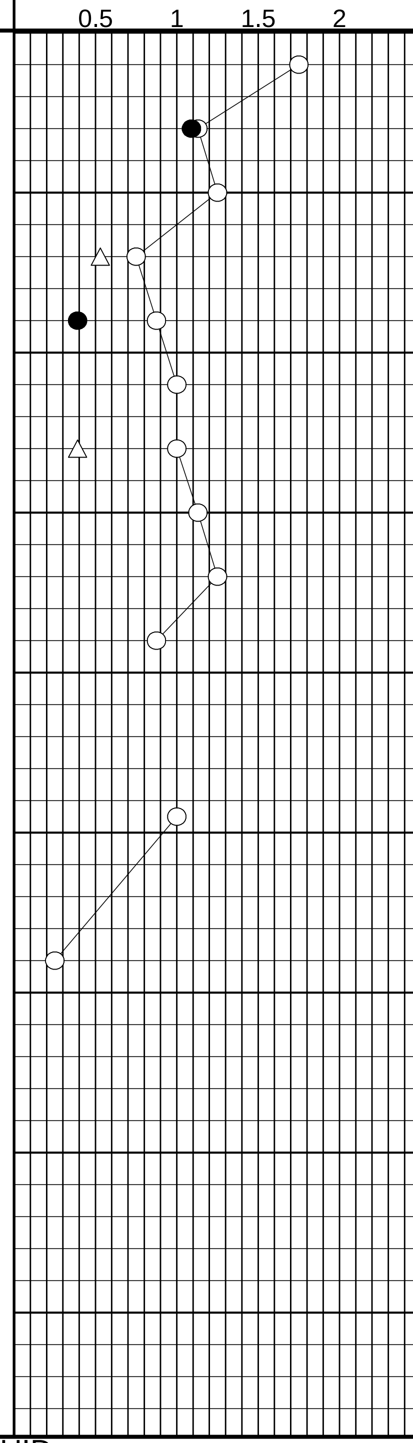
Approximate Surface Elevation (feet): 32.1

DEPTH IN FEET	DESCRIPTION
0	Fill: very stiff, tan and dark gray Fat Clay w/ Sand (CH), with calcareous nodules and roots -with shell 0'-2' -tan and gray 2'-4'
4	Firm to very stiff, dark gray Fat Clay (CH), with slickensides and ferrous stains -gray and reddish tan 8'-10' -reddish tan and light gray 10'-20', with roots 10'-12', and calcareous nodules 10'-14' -with sand pockets 16'-18'
18	Stiff, reddish tan and light gray Lean Clay (CL), with silt partings -borehole caved in at 21.7'
23	Reddish tan and light gray Silt (ML), wet
24	Stiff, reddish tan Fat Clay (CH), with sand pockets
28	Soft to stiff, reddish tan Lean Clay w/Sand (CL), with abundant silt partings -clay layer 34'-35'
35	Medium dense, reddish tan Silty Sand (SM), with clayey sand pockets, wet
40	Termination depth = 40 feet.

S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
	19	110	76			
	27	97		81	22	59
	27	95	98			
	31	91		80	21	59
	29	92	97			
	28	96				
	31	93	98			
	30	93				
	32	93	100			
	23	103	95	46	17	29
	23	104	95			
	22	105	77	29	15	14
15	29		71			
13	26		30			

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO 25 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 23.4 FEET WHILE DRILLING

WATER LEVEL AT 6.2 FEET AFTER 24 HRS

DRILLED BY V&S DRAFTED BY BpJ LOGGED BY RJM

PROJECT NO. G116-13

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	Fill
	High plasticity clay
	Low plasticity clay
	Silty sand
	Silt
Misc. Symbols	
	Water table depth during drilling
	Subsequent water table depth
○	Pocket Penetrometer
●	Unconfined Compression
△	Confined Compression
□	Torvane
Soil Samplers	
	Undisturbed thin wall Shelby tube
	Standard penetration test

ENGINEERING CORP.

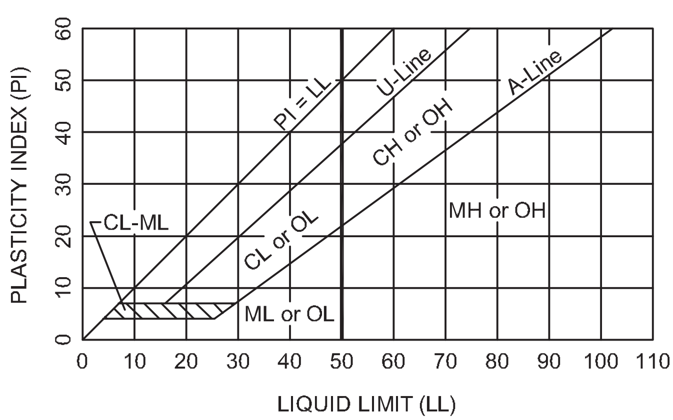
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

ASTM Designation D-2487

MAJOR DIVISIONS				GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (Less than 50% of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)		GW	Well-graded gravel, well-graded gravel with sand
				GP	Poorly-graded gravel, poorly-graded gravel with sand
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	GM	Silty gravel, silty gravel with sand
			Limits plot above "A" line & hatched zone on plasticity chart	GC	Clayey gravel, clayey gravel with sand
	SANDS (50% or more of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 sieve)		SW	Well-graded sand, well-graded sand with gravel
				SP	Poorly-graded sand, poorly-graded sand with gravel
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	SM	Silty sand, silty sand with gravel
			Limits plot above "A" line & hatched zone on plasticity chart	SC	Clayey sand, clayey sand with gravel
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS AND CLAYS (Liquid Limit Less Than 50%)			ML	Silt, silt with sand, silt with gravel, sandy silt, gravelly silt
				CL	Lean clay, lean clay with sand, lean clay with gravel, sandy lean clay, gravelly lean clay
				OL	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt
	SILTS AND CLAYS (Liquid Limit 50% or More)			MH	Elastic silt, elastic silt with sand, sandy elastic silt, gravelly elastic silt
				CH	Fat clay, fat clay with sand, fat clay with gravel, sandy fat clay, gravelly fat clay
				OH	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt

NOTE: Coarse soils between 5% and 12% passing the No. 200 sieve and fine-grained soils with limits plotting in the hatched zone of the plasticity chart are to have dual symbols.

PLASTICITY CHART



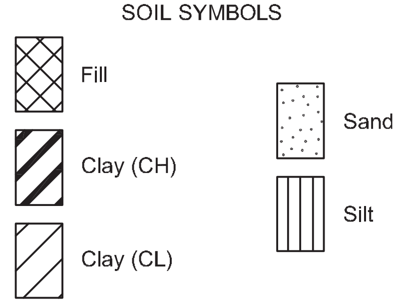
Equation of A-Line: Horizontal at PI=4 to LL=25.5, then PI=0.73(LL-20)

Equation of U-Line: Vertical at LL=16 to PI=7, then PI=0.9(LL-8)

DEGREE OF PLASTICITY OF COHESIVE SOILS

Degree of Plasticity	Plasticity Index
None	0 - 4
Slight	5 - 10
Medium	11 - 20
High	21 - 40
Very High	>40

SOIL SYMBOLS



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TERMS USED ON BORING LOGS

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

6" 3" 3/4" #4 #10 #40 #200

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

152 76.2 19.1 4.76 2.00 0.420 0.074 0.002

SOIL GRAIN SIZE IN MILLIMETERS

STRENGTH OF COHESIVE SOILS

Consistency	Undrained Shear Strength, Kips per Sq. ft.	SPT Blowcount
Very Soft	less than 0.25	< 2 bpf
Soft	0.25 to 0.50	2-4 bpf
Firm	0.50 to 1.00	4-8 bpf
Stiff	1.00 to 2.00	8-16 bpf
Very Stiff	2.00 to 4.00	16-32 bpf
Hard	greater than 4.00	>32 bpf

RELATIVE DENSITY OF COHESIONLESS SOILS FROM STANDARD PENETRATION TEST

Very Loose .... <4 bpf
Loose .... 5-10 bpf
Medium Dense .... 11-30 bpf
Dense .... 31-50 bpf
Very Dense .... >50 bpf

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows per Foot	Description
25	25 blows driving sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows driving sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows driving sampler 3 inches, during initial 6-inches seating interval.

NOTE: To avoid change to sampling tools, driving is limited to 50 blows during or after seating interval.

DRY STRENGTH ASTM D2488

None	Dry specimen crumbles into powder with mere pressure of handling
Low	Dry specimen crumbles into powder with some finger pressure
Medium	Dry specimen breaks into pieces or crumbles with considerable pressure
High	Dry specimen cannot be broken with finger pressure, it can be broken between thumb and hard surface
Very High	Dry specimen cannot be broken between thumb and hard surface

MOISTURE CONDITION ASTM D2488

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy. The degree of slickensidedness depends upon the spacing of slickensides and the easiness of breaking along these planes.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Friable	Crumbly, can be easily crushed with light pressure.
Blocky	Clays that have a block-like or polyhedral structure.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil types.
Interlayered	Soil sample composed of alternating layers of different soil types.
Intermixed	Soil sample composed of pockets of different soil types and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of calcium material.

ASTM & TXDOT DESIGNATION FOR SOIL LABORATORY TESTS

SOIL TEST	ASTM TEST DESIGNATION	TXDOT TEST DESIGNATION
Unified Soil Classification System	D 2487	Tex-142-E
Moisture Content	D 2216	Tex-103-E
Specific Gravity	D 854	Tex-108-E
Sieve Analysis	D 6913	Tex-110-E (Part 1)
Hydrometer Analysis	D 7928	Tex-110-E (Part 2)
Minus No. 200 Sieve	D 1140	Tex-111-E
Liquid Limit	D 4318	Tex-104-E
Plastic Limit	D 4318	Tex-105-E
Standard Proctor Compaction	D 698	Tex-114-E
Modified Proctor Compaction	D 1557	Tex-113-E
California Bearing Ratio	D 1883	-
Swell	D 4546	-
Consolidation	D 2435	-
Unconfined Compression	D 2166	-
Unconsolidated-Undrained Triaxial	D 2850	Tex-118-E
Consolidated-Undrained Triaxial	D 4767	Tex-131-E
Permeability (constant head)	D 5084	-
Pinhole	D 4647	-
Crumb	D 6572	-
Double Hydrometer	D 4221	-
pH of Soil	D 4972	Tex-128-E
Soil Suction	D 5298	-
Soil Sulfate	C 1580	Tex-145-E
Organics	D 2974	Tex-148-E

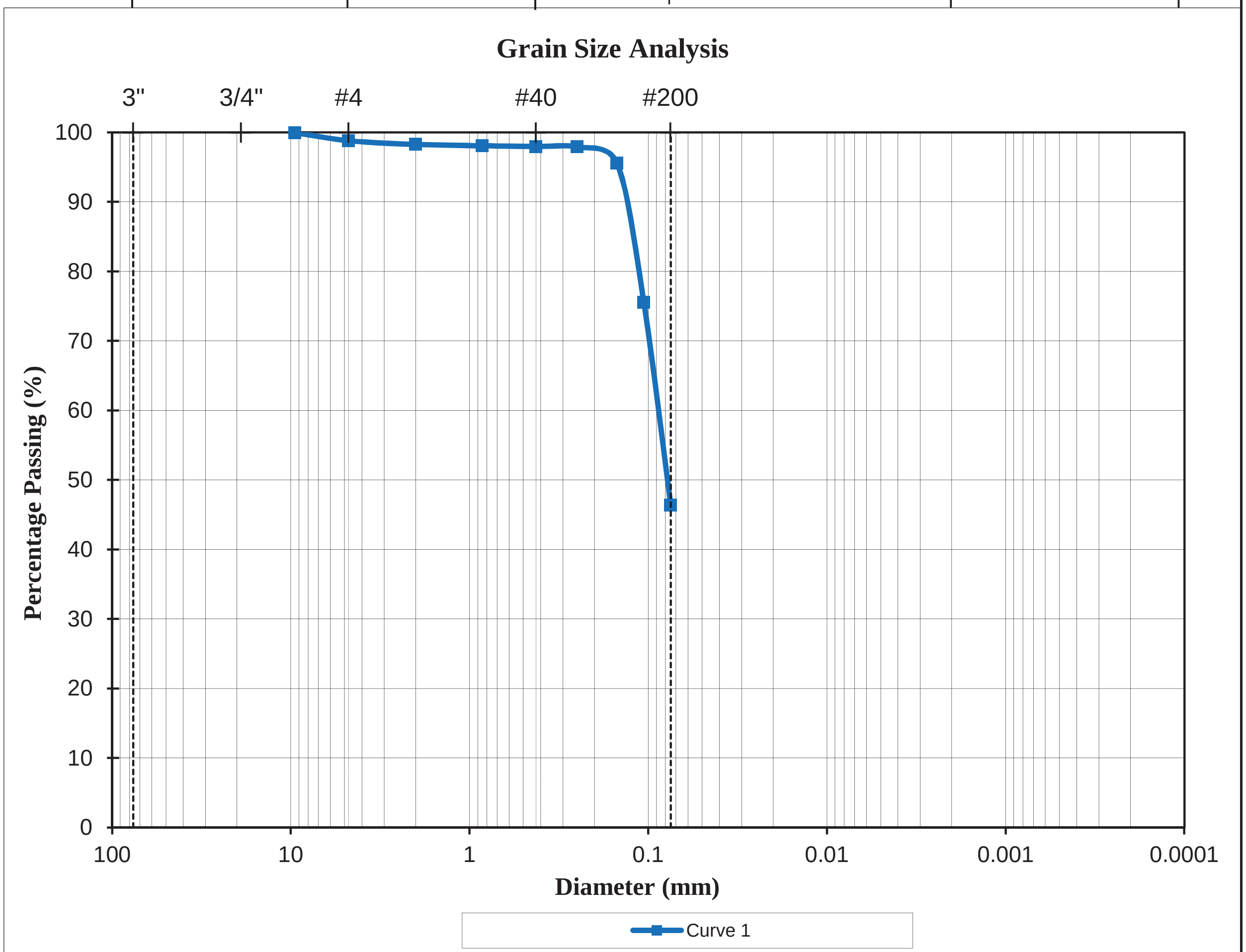
AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE

Project : Improvements of SJRA Siphon 29 at Ellis School Rd **Job No.:** G155-18
Location of Project: Harris County, Texas **Date of Testing:** 9/26/2018

	Gravel	Sand: Coarse to Medium	Sand: Fine	Silt	Clay	



Curve	Boring	Depth (ft)	Soil Description	Cu	Cc	D_{50} (mm)
1	B-1	33-35	Silty Sand (SM)	N/A	N/A	0.078

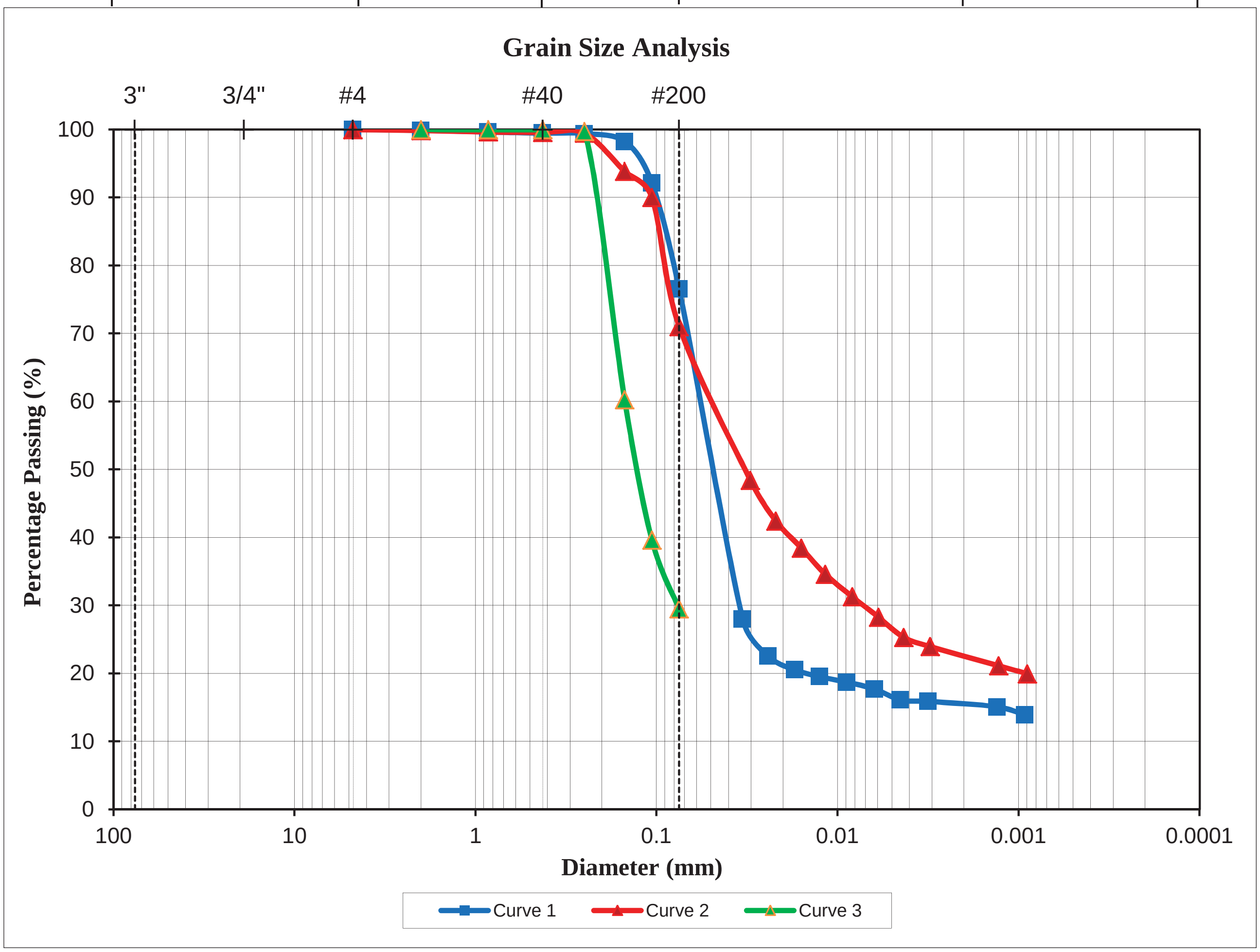
AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE & HYDROMETER

Project : SJRA Highlands System Assessment Study **Job No.:** G116-13
Project Section: SJRA Phase Ia South Canal **Date of Testing:** 6/6/2013

	Gravel	Sand		Silt	Clay
		Coarse to Medium	Fine		



Curve	Boring	Depth (ft)	Soil Description	D50 (mm)	Cu	Cc
1	B-46	28-30	Lean Clay (CL)	0.048	N/A	N/A
2	B-46	33-35	Lean Clay (CL)	0.032	N/A	N/A
3	B-46	38-40	Silty Sand (SM)	0.120	N/A	N/A

AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

RESULTS OF CRUMB TESTS (ASTM D 6572)

Project Name: Siphon 29 Improvements, Harris County, Texas

Project No.: G155-18 Test Date: 10/5/18

Boring Number	Depth, feet	2 Minutes		1 Hour		6 Hours	
		Grade	C (deg)	Grade	C (deg)	Grade	C (deg)
B-1	4-6	1	22.3	1	22.4	1	22.3
B-1	12-14	1	22.3	1	22.3	1	22.3
B-2	2-4	1	22.3	1	22.4	1	22.3
B-2	10-12	1	22.3	1	22.4	1	22.3

Grade Classification:

- Grade 1 Non-dispersive; No reaction
- Grade 2 Intermediate; Slight reaction
- Grade 3 Dispersive; Moderate reaction
- Grade 4 Highly Dispersive; Strong reaction

Interpretation:

Under normal conditions, use the 1 hour reading to determine dispersive grade.
However, if the dispersive grade changes from 2 to 3 or from 3 to 4 between the 1 and 6 hour readings, use the 6 hour reading instead.

AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

RESULTS OF CRUMB TESTS (ASTM D 6572)

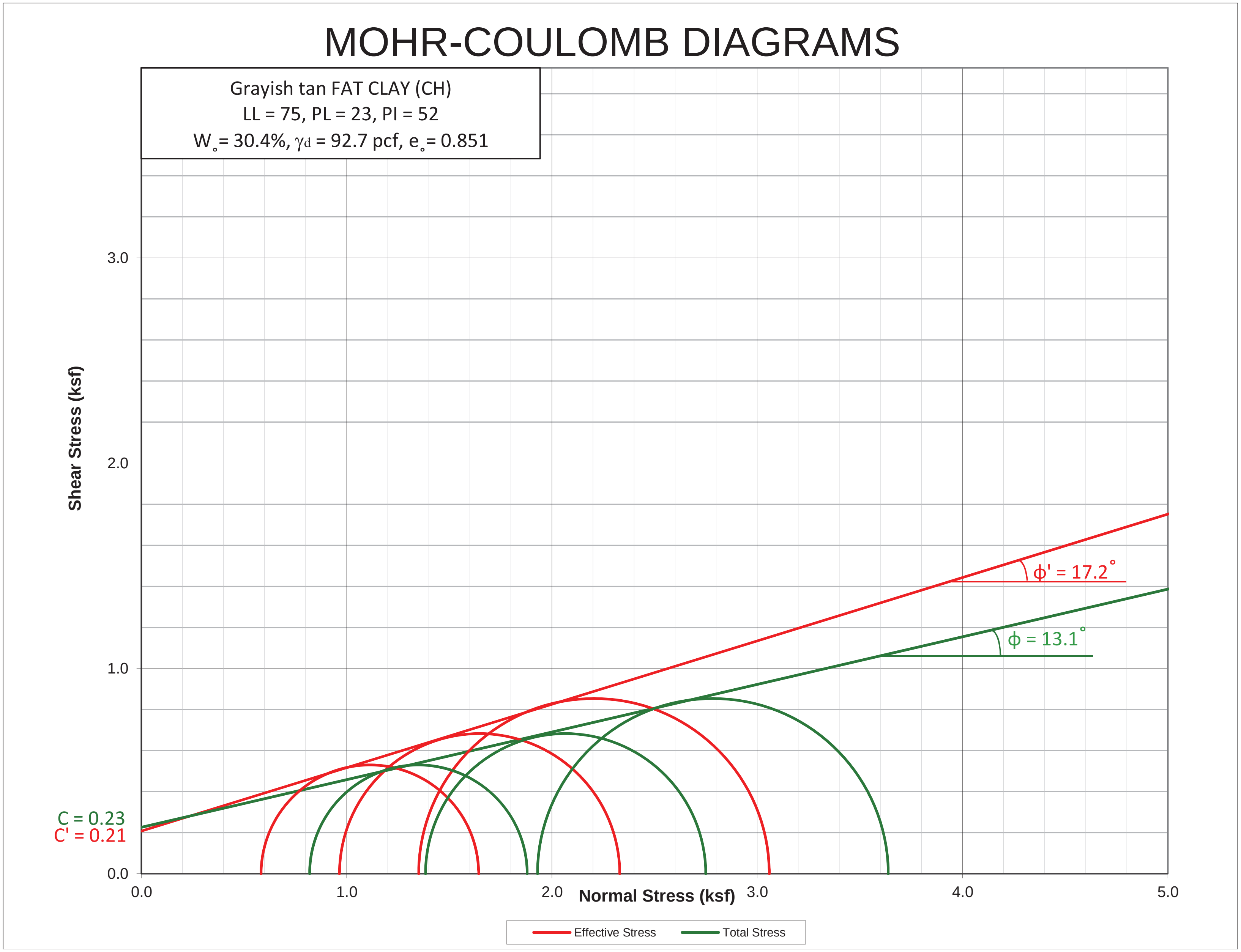
Project Name: Highlands System Assessment Study - SJRA Phase Ib South Canal
Project No.: G116-13 Test Date: 6/17/2013

Boring Number	Depth, feet	Crumbling Characteristics				Crumb Test Classification	
		Crumbles? (Y/N)	If soil crumbles, rate of crumbling			@15 minutes	@30 minutes
			Minor	Moderate	Immediate		
B-46	4-6	Y	•			1	1
B-46	23-25	Y		•		2	3
B-54	2-4	Y	•			1	1

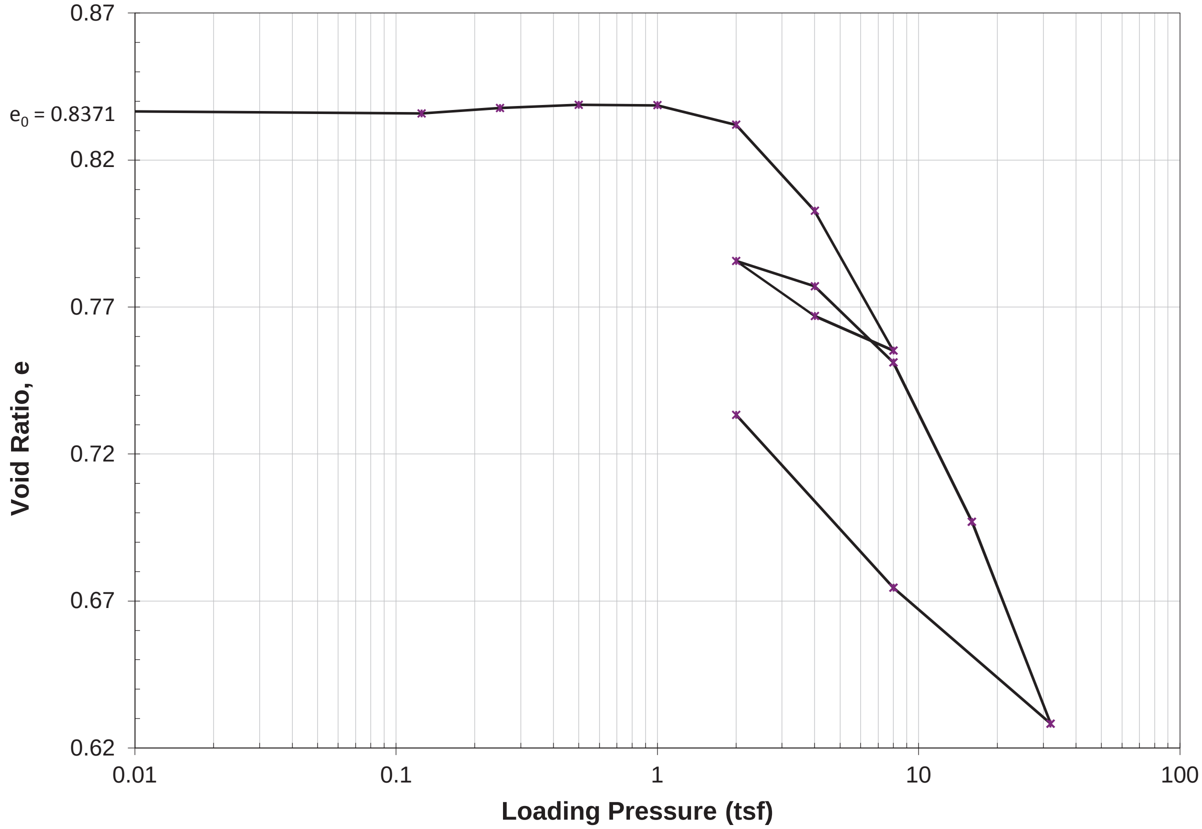
Results interpretation:

1 No sign of cloudy water caused by colloidal suspension
2 Bare hint of colloidal cloud formation at surface or soil crumb
3 Easily recognized colloidal cloud covering at least 1/4 to 1/2 of the bottom of the glass container
4 Strong reaction with colloidal cloud covering most of the bottom of the glass container

G155-18 B-2, 8'-10'



CONSOLIDATION TEST RESULTS



Project No.: G155-18

Project: Improvements of Siphon 29 at Ellis School Rd.

Sample ID: B-1, 14 to 16 feet

Sample Description: Tan and gray Fat Clay (CH)

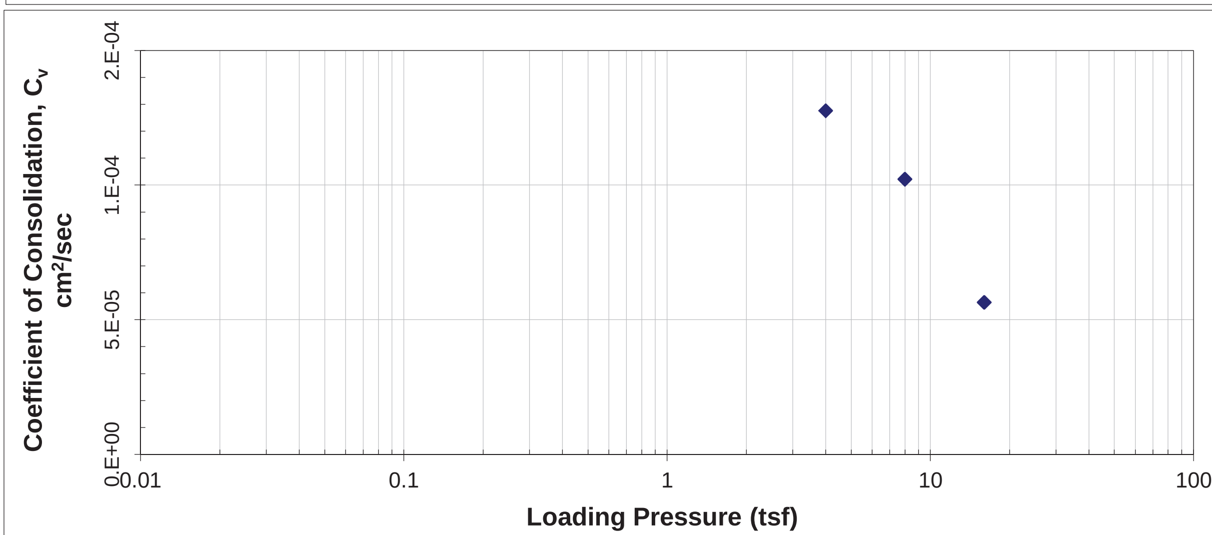
Dry Unit Weight (γ_d): 93.5 pcf

Estimated Consolidation Index (Cc): 0.2239

Estimated Recompression Index (Cr): 0.0301

Estimated OCR: 7.0

Estimated Preconsolidation Pressure (Pc): 4.4 tsf



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APPENDIX B

Plates B-1 and B-2 Site Photographs near Siphon 29

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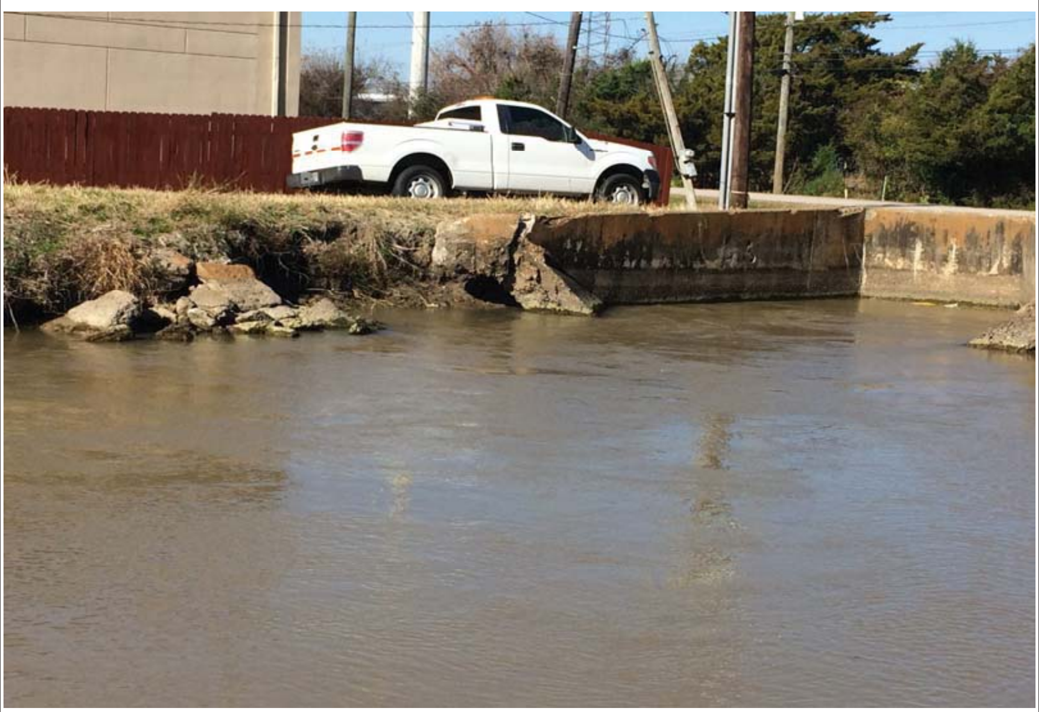
Major Feature	Undermining
Date	12/24/2018

Description:
Undermining due to erosion was observed at various locations behind the wing walls. Photo depicts the view to the southwest of Siphon No. 29, where a hole was observed beneath the wing wall due to undermining by erosion.



Major Feature	Local Slide & Steep Slope, and Displacement of existing Riprap
Date	12/24/2018

Description:
Slope failure due to local slides were observed on both the banks of the canal near the siphon structures and displacement of the existing riprap was also seen southwest of Siphon 29. Photo depicts the view to the southwest of Siphon No. 29, where a slope failure and displacement of the riprap was observed.

Major Feature	Crack on Wing Walls
Date	12/24/2018

Description:
Cracks on all the wing walls were observed. Photo depicts the view to the northeast of Siphon No. 29, where a severe crack on the wing wall was observed.

Major Feature	Crack and Movement of the Head Walls
Date	12/24/2018

Description:
Cracks and movement of both the head walls were observed. Photo depicts a view to the north of Ellis School Road at Siphon No. 29, where number of cracks on the head wall and its movement were observed.

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APPENDIX C

G155-18 SJRA SIPHON 29 IMPROVEMENTS, HARRIS COUNTY, TEXAS
SOIL PARAMETERS FOR UNDERGROUND UTILITIES

Boring	Depth (ft)	Soil Type	γ (pcf)	γ' (pcf)	OSHA Type	Short-Term					Long-Term				
						C (psf)	φ (deg)	K_a	K_0	K_p	C' (psf)	φ' (deg)	K_a	K_0	K_p
B-1	0-4	Fill: very stiff CH	121	59	C	1500	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
	4-10	Firm to very stiff CH	119	57	C	850	0	1.00	1.00	1.00	75	16	0.57	0.72	1.76
	10-18	Firm to very stiff CH	122	60	C	800	0	1.00	1.00	1.00	75	16	0.57	0.72	1.76
	18-24	Stiff CL	125	63	C* (18-20)	1000	0	1.00	1.00	1.00	100	18	0.53	0.69	1.89
	24-28	SM	120	58	N/A	0	26	0.39	0.56	2.56	0	26	0.39	0.56	2.56
	28-40	Stiff CL/loose SM/stiff CH	120	58	N/A	0	28	0.36	0.53	2.77	0	28	0.36	0.53	2.77
B-2	0-4	Hard CH	120	58	B	2500	0	1.00	1.00	1.00	250	16	0.57	0.72	1.76
	4-10	Firm to stiff CH	120	58	B (C*, 6-10)	1000	0	1.00	1.00	1.00	100	16	0.57	0.72	1.76
	10-20	Firm to very stiff CH/CL	122	60	C*	1100	0	1.00	1.00	1.00	100	16	0.57	0.72	1.76
	20-30	Soft to firm CL	115	53	N/A	250	0	1.00	1.00	1.00	25	18	0.53	0.69	1.89
	30-36	Firm to stiff CL	115	53	N/A	400	0	1.00	1.00	1.00	25	18	0.53	0.69	1.89
	36-40	ML	120	58	N/A	0	26	0.39	0.56	2.56	0	26	0.39	0.56	2.56
B-46	0-4	Fill: very stiff CH	123	61	C	1500	0	1.00	1.00	1.00	120	19	0.51	0.67	1.97
	4-8	Firm to very stiff CH	119	57	B (C*, 6-8)	1000	0	1.00	1.00	1.00	100	17	0.55	0.71	1.83
	8-18	Firm to very stiff CH	119	57	C	800	0	1.00	1.00	1.00	90	17	0.55	0.71	1.83
	18-23	Stiff CL	127	65	C (18-20)	800	0	1.00	1.00	1.00	130	21	0.47	0.64	2.12
	23-24	ML	128	66	N/A	0	26	0.39	0.56	2.56	0	26	0.39	0.56	2.56
	24-28	Stiff to very stiff CH	120	58	N/A	1000	0	1.00	1.00	1.00	120	17	0.55	0.71	1.83
	28-35	Soft to stiff CL	128	66	N/A	250	0	1.00	1.00	1.00	25	18	0.53	0.69	1.89
	35-40	Medium dense SM	120	58	N/A	0	28	0.36	0.53	2.77	0	28	0.36	0.53	2.77

(1) γ = Unit weight for soil above water level, γ' = Buoyant unit weight for soil below water level. E'n = Soil modulus for native soils;

(2) C = Soil ultimate cohesion for short term (upper limit of 3,000 psf for design purposes), φ = Soil friction angle for short term;

(3) C' = Soil ultimate cohesion for long term (upper limit of 300 psf for design purposes), φ' = Soil friction angle for long term;

(4) K_a = Coefficient of active earth pressure, K_0 = Coefficient of at-rest earth pressure, K_p = Coefficient of passive earth pressure;

(5) CL = Lean Clay, CH = Fat Clay, ML = Silt, SM = Silty Sand;

(6) OSHA Soil Types for soils in the top 20 feet below grade:

A: cohesive soils with qu = 1.5 tsf or greater (qu = Unconfined Compressive Strength of the Soil)

B: cohesive soils with qu = 0.5 tsf or greater

C: cohesive soils with qu = less than 0.5 tsf, fill materials, or granular soil

C*: submerged cohesive soils; dewatered cohesive soils can be considered OSHA Type B.

G155-18 SJRA SIPHON 29 IMPROVEMENTS, HARRIS COUNTY, TEXAS
LATERAL EARTH PRESSURE PARAMETERS FOR HEAD/WING WALLS

Backfill Type			γ (pcf)	γ' (pcf)	Short-Term						Long-Term					
					C (psf)	Cα (psf)	φ (deg)	K_a	K_0	K_p	C' (psf)	C'α (psf)	φ' (deg)	K_a	K_0	K_p
Select Fill			120	58	1600	900	0	1.00	1.00	1.00	180	125	22	0.45	0.63	2.20
Boring	**Depth (ft)**	**Soil Type**	γ (pcf)	γ' (pcf)	Short-Term						Long-Term					
					C (psf)	Cα (psf)	φ (deg)	K_a	K_0	K_p	C' (psf)	C'α (psf)	φ' (deg)	K_a	K_0	K_p
B-1	0-4	Fill: very stiff CH	121	59	1500	1000	0	1.00	1.00	1.00	150	100	16	0.57	0.72	1.76
	4-10	Firm to very stiff CH	119	57	850	550	0	1.00	1.00	1.00	75	50	16	0.57	0.72	1.76
	10-18	Firm to very stiff CH	122	60	800	500	0	1.00	1.00	1.00	75	50	16	0.57	0.72	1.76
	18-20	Stiff CL	125	63	1000	650	0	1.00	1.00	1.00	100	50	18	0.53	0.69	1.89
B-2	0-4	Hard CH	120	58	2500	1650	0	1.00	1.00	1.00	250	150	16	0.57	0.72	1.76
	4-10	Firm to stiff CH	120	58	1000	650	0	1.00	1.00	1.00	100	50	16	0.57	0.72	1.76
	10-20	Firm to very stiff CH/CL	122	60	1100	700	0	1.00	1.00	1.00	100	50	16	0.57	0.72	1.76
B-46	0-4	Fill: very stiff CH	123	61	1500	1000	0	1.00	1.00	1.00	120	80	19	0.51	0.67	1.97
	4-8	Firm to very stiff CH	119	57	1000	650	0	1.00	1.00	1.00	100	65	17	0.55	0.71	1.83
	8-18	Firm to very stiff CH	119	57	800	500	0	1.00	1.00	1.00	90	60	17	0.55	0.71	1.83
	18-20	Stiff CL	127	65	800	500	0	1.00	1.00	1.00	130	85	21	0.47	0.64	2.12

(1) γ = Unit weight for soil above water level, γ’ = Buoyant unit weight for soil below water level;

(2) C = Soil ultimate cohesion for short term (upper limit of 3,000 psf for design purposes), φ = Soil friction angle for short term.

(3) C' = Soil ultimate cohesion for long term (upper limit of 300 psf for design purposes), φ' = Soil friction angle for long term.

(4) Cα = Soil ultimate adhesion for short term.

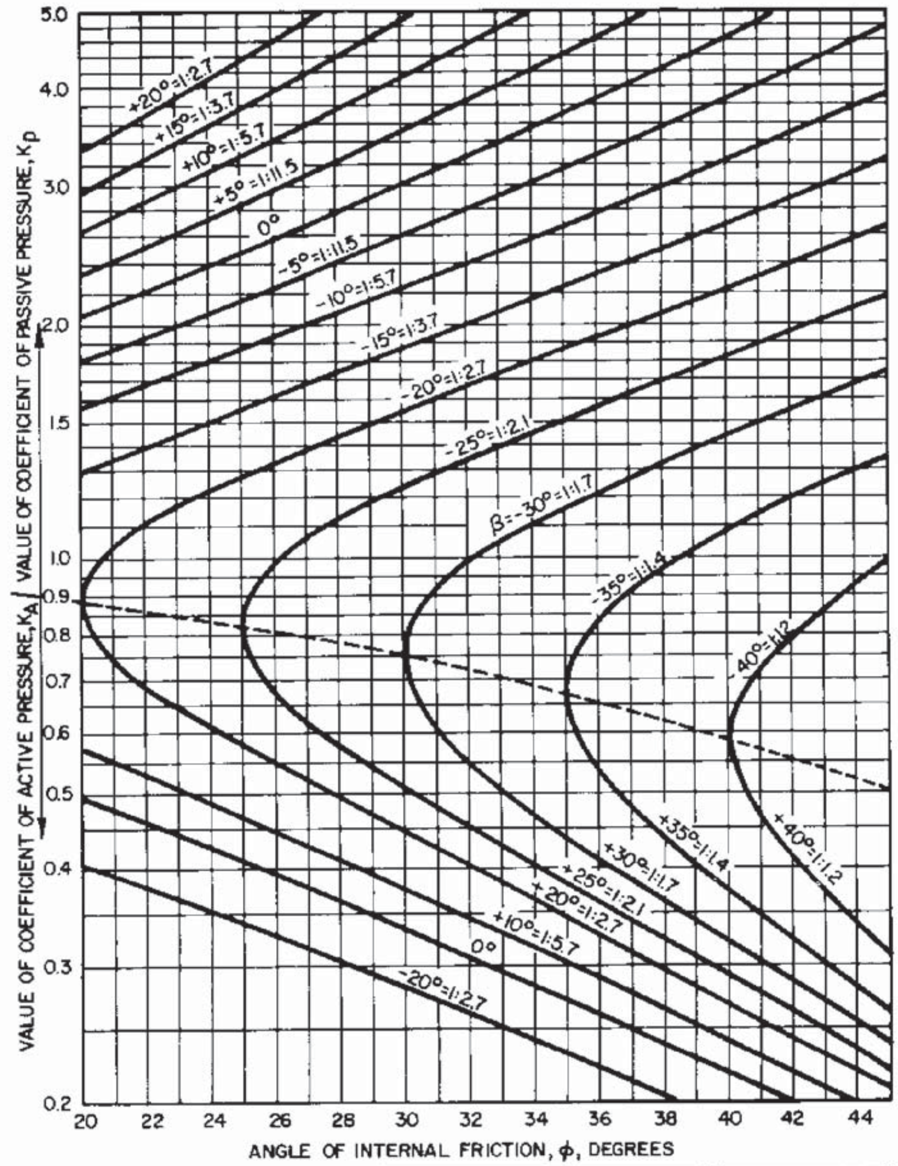
(5) C'α = Soil ultimate adhesion for long term.

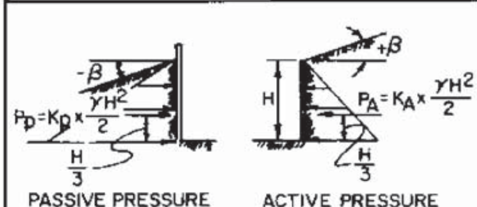
(6) δ = angle of friction between soil and footing for short term=2/3 of φ.

(7) δ' = angle of friction between soil and footing for long term=2/3 of φ'.

(8) Ka = coefficient of active earth pressure, Ko = coefficient of at-rest earth pressure, Kp = coefficient of passive earth pressure, for level backfill.

(9) AEC recommends the use of FS = 2 for passive pressure if it is to be used in the design.





$$K_A = \left[\frac{\cos\phi}{1+\sqrt{\sin\phi(\sin\phi-\cos\phi\tan\beta)}}\right]^2; \quad K_P = \left[\frac{\cos\phi}{1-\sqrt{\sin\phi(\sin\phi+\cos\phi\tan\beta)}}\right]^2$$

K_A & K_P = COEFFICIENTS FOR COULOMB'S EQUATION FOR ACTIVE AND PASSIVE EARTH PRESSURE (NO SHEAR STRESS ON VERTICAL PLANES).

P_A = ACTIVE RESULTANT

P_P = PASSIVE RESULTANT

γ = UNIT WEIGHT OF SOIL

H = HEIGHT OF WALL

ϕ = ANGLE OF INTERNAL FRICTION

β = SLOPE ANGLE

FIGURE 3
Active and Passive Coefficients, Sloping Backfill
(Granular Soils)

From US Naval Facilities Engineering Command (NAVFAC) Design Manual DM7.02 "Foundations and Earth Structures", September 1986

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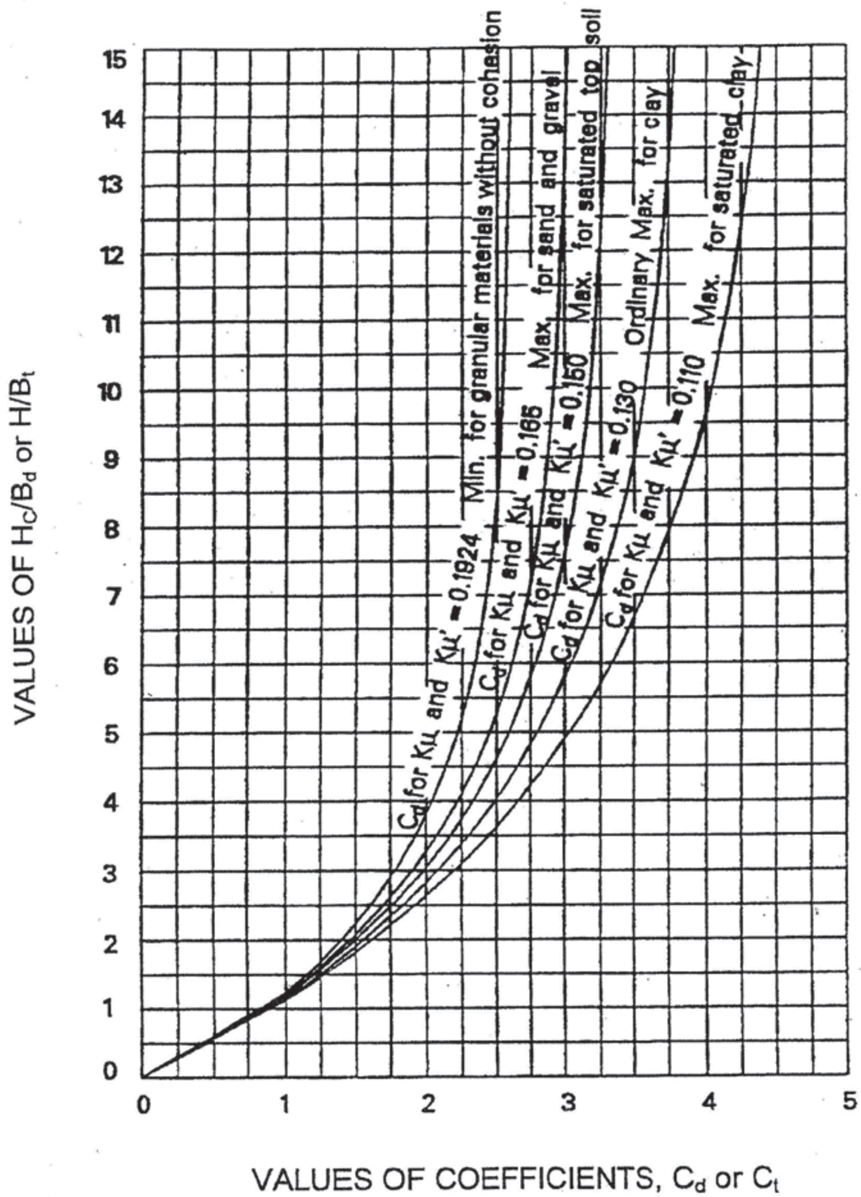
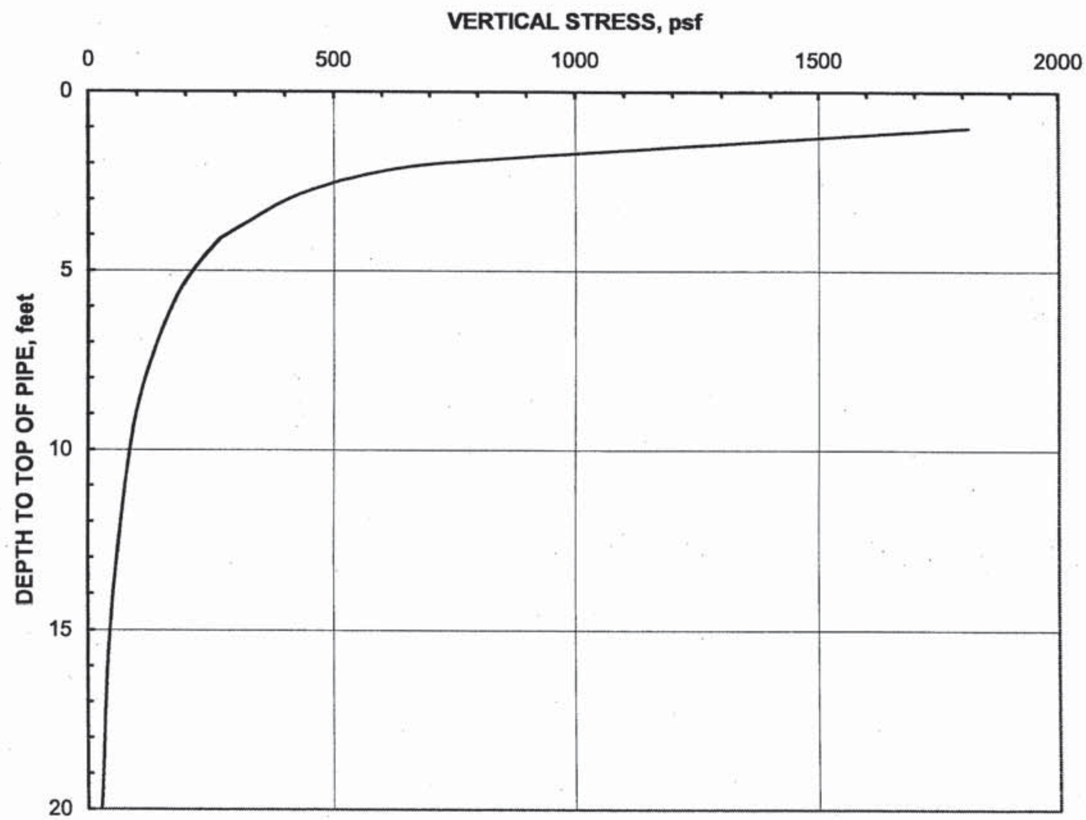


Reference: US Army Corps of Engineers Engineering Manual, EM 1110-2-2902, Oct. 31, 1997, Figure 2-5.



LIVE LOADS ON PIPE CROSSING UNDER ROADWAY

Note: 1. The vertical stress was estimated using AASHTO HS20 truck axle loadings on paved surfaces (Reference: ASCE 15-98, "Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standandard Installations").

2. Single truck passing.

TUNNEL SETTLEMENT CALCULATION FOR ONE PASS METHOD

Project Name: Siphon 29 Improvements — Date: 12/28/2018
Client: Texas Water Engineering — Project #: G155-18
Locations: Harris County, Texas

Tunnel Center Line

Tunnel Diameter, D = 6 ft,
Tunnel Radius, R = D/2 = 3 ft,
Z_0 = Depth of Tunnel Centerline, ft
σ_z = Overburden Pressure to Tunnel Centerline, psf

Tunnel Volume/ft, $V = \pi(R)^2$ = 28.27 ft^3/ft
Max Settlement, $S_{max} = V_L \pi R^2/(2.5i)$, ft
C_u = Undrained Shear Strength of Soil, psf
$N_t = \sigma_z/Cu$, Stability Factor

	B-1				B-2/B-46									
Tunnel Invert Depth, Z (ft)	12.7				13.3									
Tunnel Zone Soil Type	Firm to very stiff Fat Clay (CH)				Firm to very stiff Fat Clay (CH)									
Z_0 = Z-R, (ft)	9.7				10.3									
Unit Weight of Soil (pcf)	120.3				120.5									
σz (psf)	1167				1241									
Cu (psf)	800				1000									
N_t	**1.5**				**1.2**									
V_L (%)	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.00						
Volume Loss = V_L*V (ft^3/ft)	0.141	0.283	0.424	0.565	0.141	0.283	0.424	0.565						
$Z_0/(2R)$	1.62				1.72									
i/R	2.0				2.2									
i (ft)	6.0				6.6									
S_{max} (ft)	0.0094	0.0188	0.0283	0.0377	0.0086	0.0171	0.0257	0.0343						
S_{max} (in)	**0.113**	**0.226**	**0.339**	**0.452**	**0.103**	**0.206**	**0.308**	**0.411**						
0.5 Trough Width, 2.5i (ft)	15.0				**16.5**									
Note	Potential swelling ground due to high plasticity fat clay. Pressurized groundwater. Dewatering not required if MTBM or slurry bore/auger is used. Otherwise, dewatering operations will be necessary if boring shield, mechanized TBM, or dry auger/bore is used.				Potential swelling ground due to high plasticity fat clay. Pressurized groundwater. Dewatering not required if MTBM or slurry bore/auger is used. Otherwise, dewatering operations will be necessary if boring shield, mechanized TBM, or dry auger/bore is used.									

(Blue) Estimated settlement for using micro-tunnel boring machine (MTBM)/mechanized close face Tunnel Boring Machine (TBM)
(Yellow) Estimated Settlement for Boring Shield or Auger/Bore.

V_L and i/R from: J. Bickel, T. Kuesel, and E. King, 'Tunnel Engineering Handbook', 2nd Edition, 1996, Tables 6-8 and Figure 6-19, respectively.

TUNNEL SETTLEMENT CALCULATION FOR TWO PASS METHOD

Project Name: Siphon 29 Improvements | Date: 12/28/2018
Client: Texas Water Engineering | Project #: G155-18
Locations: Harris County, Texas

Tunnel Center Line

Tunnel Diameter (with liner/casing), D = 6.5 ft,
Tunnel Radius (with liner/casing), R = D/2 = 3.25 ft,
Z_0 = Depth of Tunnel Centerline, ft
σ_z = Overburden Pressure to Tunnel Centerline, psf

Tunnel Volume/ft (with liner plate/casing), $V = \pi(R)^2$ = 33.18 ft^3/ft
Max Settlement, $S_{max} = V_L \pi R^2/(2.5i)$, ft
C_u = Undrained Shear Strength of Soil, psf
$N_t = \sigma_z/Cu$, Stability Factor

	B-1				B-2/B-46									
Tunnel Invert Depth, Z (ft)	12.7				13.3									
Tunnel Zone Soil Type	Firm to very stiff Fat Clay (CH)				Firm to very stiff Fat Clay (CH)									
Z_0 = Z-R, (ft)	9.5				10.1									
Unit Weight of Soil (pcf)	120.3				120.5									
σz (psf)	1137				1211									
Cu (psf)	800				1000									
N_t	**1.4**				**1.2**									
V_L (%)	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0						
Volume Loss = V_L*V (ft^3/ft)	0.166	0.332	0.498	0.664	0.166	0.332	0.498	0.664						
$Z_0/(2R)$	1.45				1.55									
i/R	2.0				2.2									
i (ft)	6.5				7.2									
S_{max} (ft)	0.0102	0.0204	0.0306	0.0408	0.0093	0.0186	0.0278	0.0371						
S_{max} (in)	**0.123**	**0.245**	**0.368**	**0.490**	**0.111**	**0.223**	**0.334**	**0.446**						
0.5 Trough Width, 2.5i (ft)	**16.3**				**17.9**									
Note	Potential swelling ground due to high plasticity fat clay. Pressurized groundwater. Dewatering not required if MTBM or slurry bore/auger is used. Otherwise, dewatering operations will be necessary if boring shield, mechanized TBM, or dry auger/bore is used.				Potential swelling ground due to high plasticity fat clay. Pressurized groundwater. Dewatering not required if MTBM or slurry bore/auger is used. Otherwise, dewatering operations will be necessary if boring shield, mechanized TBM, or dry auger/bore is used.									

Estimated settlement for using micro-tunnel boring machine (MTBM)/mechanized close face Tunnel Boring Machine (TBM)
Estimated Settlement for Boring Shield or Auger/Bore.

V_L and i/R from: J. Bickel, T. Kuesel, and E. King, 'Tunnel Engineering Handbook', 2nd Edition, 1996, Tables 6-8 and Figure 6-19, respectively.

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Table 1. Soil Parameters for Settlement Analysis (Based on Boring B-1)

Depth (ft)	Soil type	γ (pcf)	γ' (pcf)	e_0	C_r	C_c	N	Material Properties	P_c (psf)
0-4	Fill: very stiff CH	121	59	0.7800	0.0319	0.1912	-	Cohesive	8800
4-18	Firm to very stiff CH	122	60	0.8371	0.0301	0.2239	-	Cohesive	8800
18-24	Stiff CL	125	63	0.6967	0.0219	0.1749	-	Cohesive	8800
24-28	SM	120	54	-	-	-	4	Cohesionless	-
28-31	Stiff CL	125	63	0.6357	0.0203	0.1621	-	Cohesive	8800
31-38	Loose SM	119	57	-	-	-	8	Cohesionless	-
38-40	Stiff CH	124	62	0.6559	0.0206	0.1650	-	Cohesive	8800

Table 2. Soil Parameters for Settlement Analysis (Based on Boring B-2)

Depth (ft)	Soil type	γ (pcf)	γ' (pcf)	e_0	C_r	C_c	N	Material Properties	P_c (psf)
0-18	Firm to hard CH	121	59	0.8511	0.0344	0.2062	-	Cohesive	8800
18-36	Soft to very stiff CL	115	65	0.6469	0.0274	0.1644	-	Cohesive	8800
36-40	ML	120	55	-	-	-	4	Cohesionless	8800

Note: (1) γ = wet unit weight of soil, γ' = buoyant unit weight of soil;
(2) e_0 = initial void ratio;
(3) C_c = compression ratio;
(4) C_r = recompression ratio, which is derived from the recompression curve within the stress range from 2 to 8 ksf;
(5) N = number of SPT blow counts per foot;
(6) p_c = preconsolidation pressure;
(7) CH = Fat Clay; CL= Lean Clay; ML = Silt; SM = Silty Sand.

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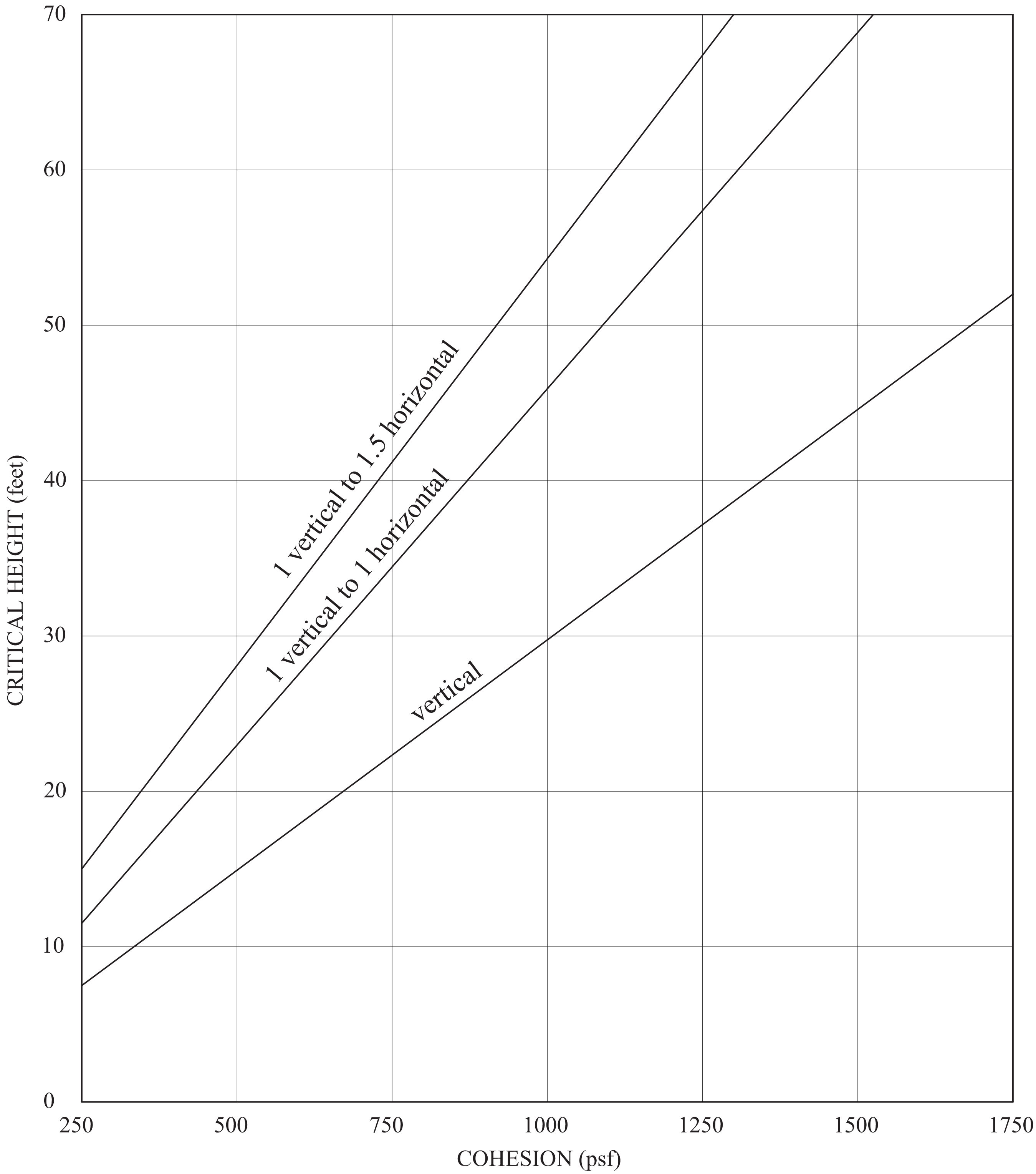
APPENDIX D

Critical Heights of Cut Slopes in Nonfissured Clays



Note: The charts are calculated based on NAVFAC DM7.1, Page 7.1-319, assuming the critical circles are toe circles, and wet unit weight of soils = 125pcf.

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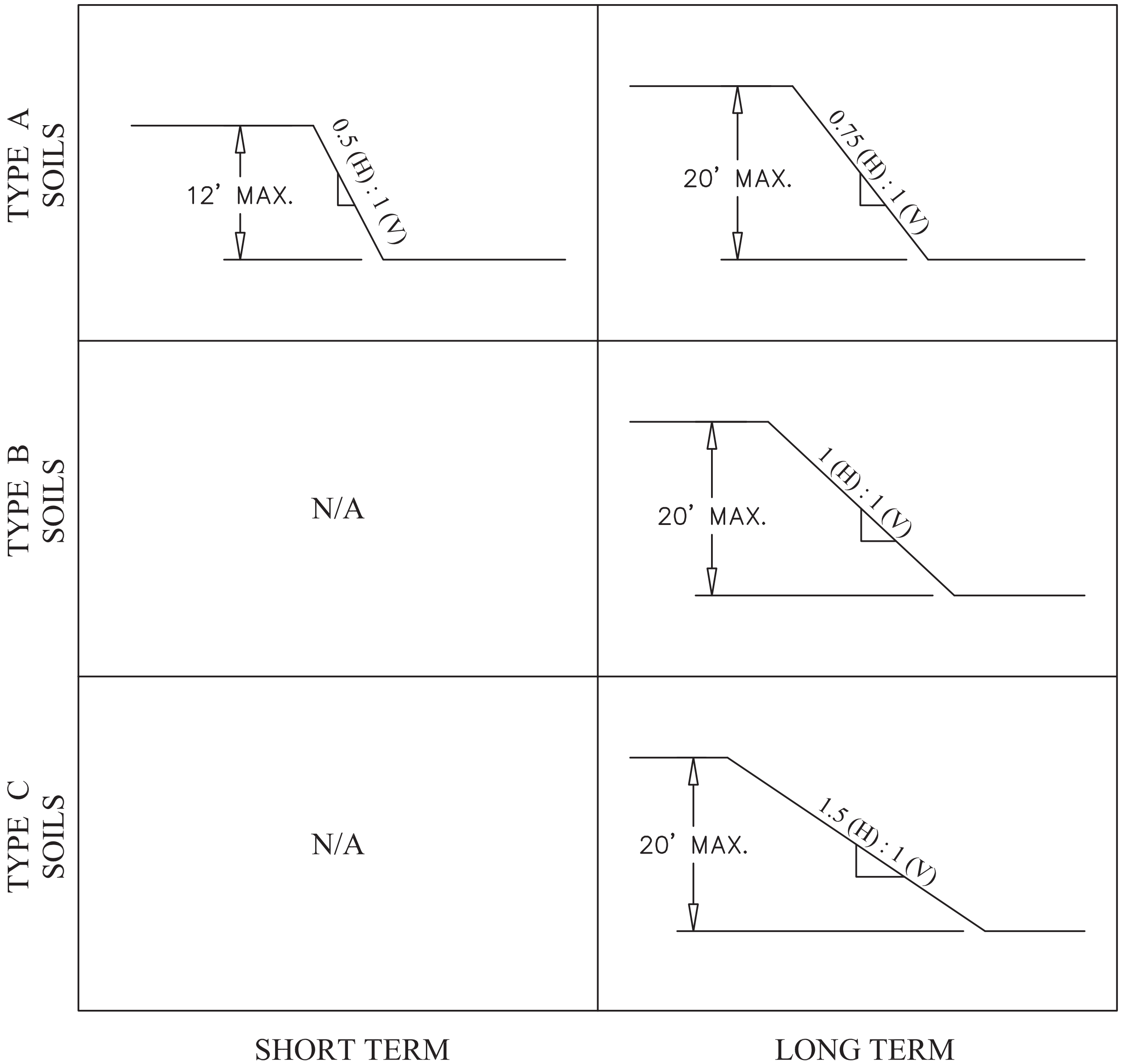
MAXIMUM ALLOWABLE SLOPES



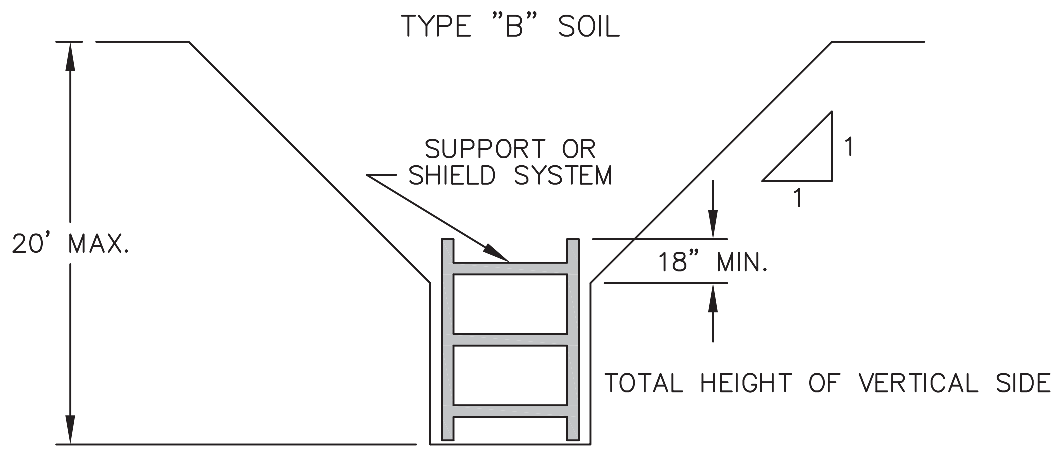
NOTES:

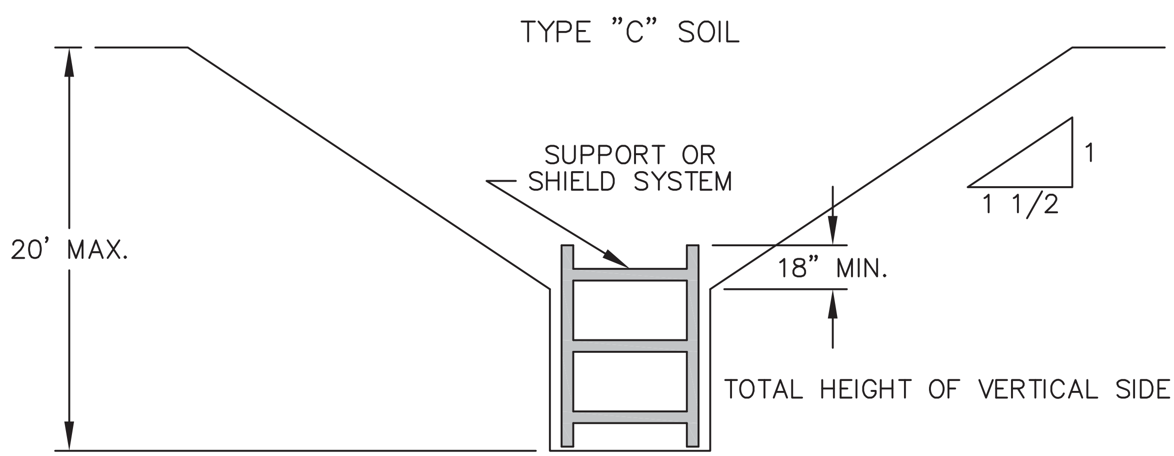
(1) For Type A soils, a short term maximum allowable slope of 0.5 (H) : 1 (V) is allowed in excavations that are 12 feet or less in depth; short term (24 hours or less) maximum allowable slopes for excavations greater than 12 feet in depth shall be 0.75 (H) : 1 (V).

(2) Maximum depth for above slopes is 20 feet. For slopes deeper than 20 feet, trench protection should be designed by the Contractor's professional engineer.

Reference: OSHA, Safety and Health Regulations for Construction, 1926 Subpart P.

A COMBINATION OF BRACING AND OPEN CUTS





Reference: OSHA, Safety and Health Regulations for Construction, 1926 Subpart P.

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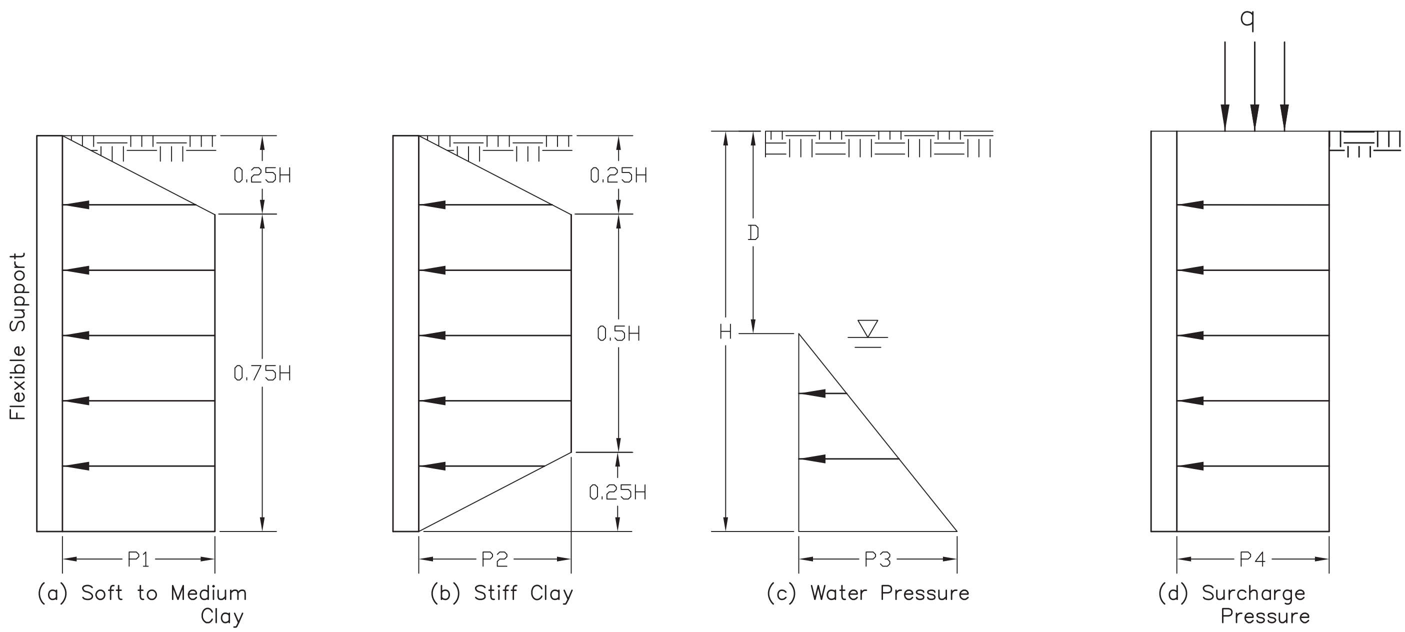
LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN COHESIVE SOIL - LONG TERM CONDITIONS



Empirical Pressure Distributions

Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = γH-4C, psf
P2 = Lateral earth pressure = 0.4γH, psf
P3 = Water pressure = γ_w (H-D), psf
P4 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
C = Drained shear strength or cohesion, psf
K_a = Coefficient of active earth pressure

Notes:
1. All pressures are additive.
2. No safety factors are included.
3. For use only during long term construction.
4. If γH/C < 4, use section (b),
   If 4 < γH/C < 6, use larger of section (a) or (b),
   If γH/C > 6, use section (a).

Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

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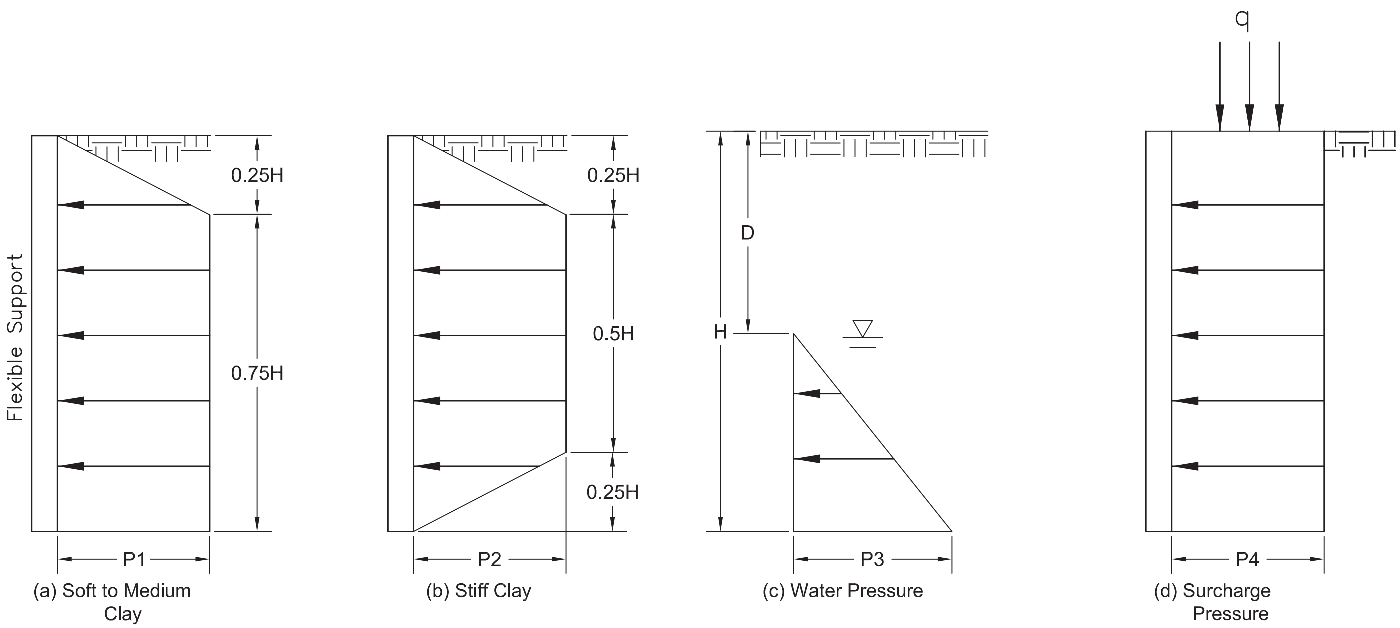
LATERAL PRESSURE DIAGRAMS

FOR OPEN CUTS IN COHESIVE SOIL - SHORT TERM CONDITIONS



Empirical Pressure Distributions

Where:

H = Total excavation depth, feet

D = Depth to water table, feet

P1 = Lateral earth pressure = $\gamma H\text{-}4S_u$, psf

P2 = Lateral earth pressure = $0.2\gamma H$, psf

P3 = Water pressure = γ_w (H-D), psf

P4 = Lateral earth pressure caused by surcharge = qK_a, psf

γ = Effective unit weight of soil, pcf

γ_w = Unit weight of water, pcf

S_u = Undrained shear strength = $q_u/2$, psf

q_u = Unconfined compressive strength, psf

K_a = Coefficient of active earth pressure

Notes:

1. All pressures are additive.
2. No safety factors are included.
3. For use only during short term construction.
4. If $\gamma H/S_u < 4$, use section (b),
   If $4 < \gamma H/S_u < 6$, use larger of section (a) or (b),
   If $\gamma H/S_u > 6$, use section (a).

Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

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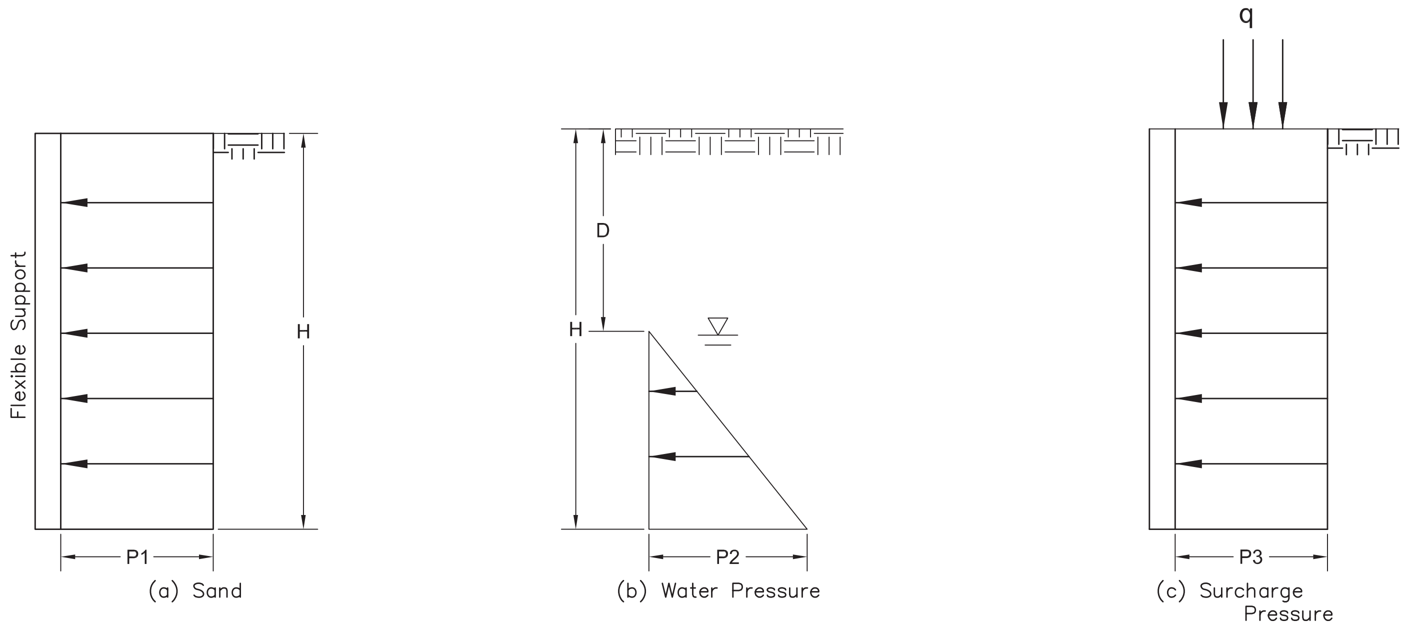
LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN SAND



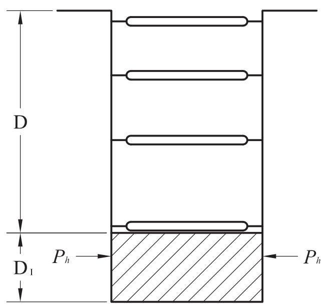
Empirical Pressure Distributions

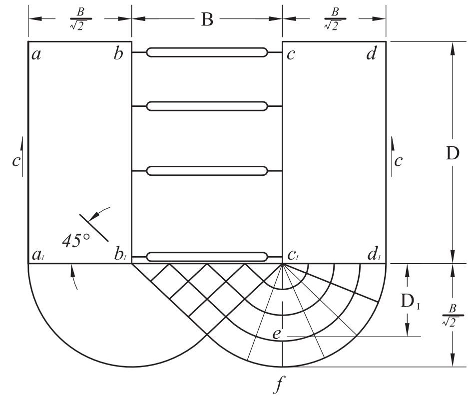
Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $0.65^{*}\gamma HK_a$, psf
P2 = Water pressure = γ_w (H-D), psf
P3 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
K_a = Coefficient of active earth pressure = $(1-\sin\varphi)/(1+\sin\varphi)$
φ = Drained friction angle

Notes:
1. All pressures are additive.
2. No safety factors are included.

Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

BOTTOM STABILITY FOR BRACED EXCAVATION IN CLAY





Factor of Safety against bottom of heave,

$$F.S = \frac{NcC}{(\gamma D+q)}$$

where, Nc = Coefficient depending on the dimension of the excavation (see Figure at the bottom)
C = Undrained shear strength of soil in zone immediately around the bottom of the excavation,
γ = Unit weight of soil,
D = Depth of excavation,
q = Surface surcharge.

If F.S < 1.5, sheeting should be extended further down to achieve stability

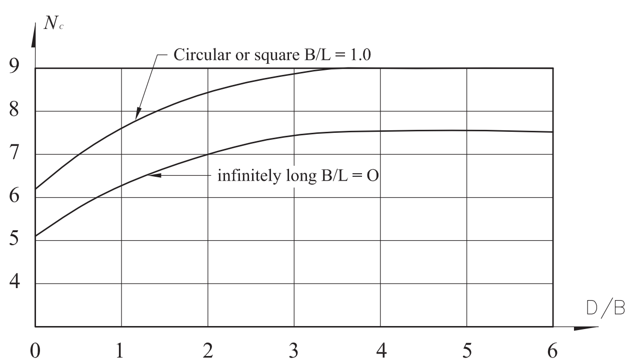
$$\text{Depth of Buried Length, } (D_1) = \frac{1.5(\gamma D+q)-NcC}{(C/B)-0.5\gamma} \;;\; D_1 \geqslant 5\ ft.$$

Pressure on buried length, P_h:

For $D_1 < 0.47B$; $P_h = 1.5\ D_1(\gamma D - 1.4\ CD/B - 3.14C)$

For $D_1 > 0.47B$; $P_h = 0.7\ (\gamma DB - 1.4\ CD - 3.14CB)$

where; B = width of excavation



N_c rectangular = $(0.84 + 0.16B/L)N_c$ square

Reference: Bjerrum, L. and Eide, O., Stability of Strutted Excavations in Clay, Geotechnique, 6, 32-47 (1956).

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Tunnel Behavior: Sands and Gravels
(Terzaghi, 1977)

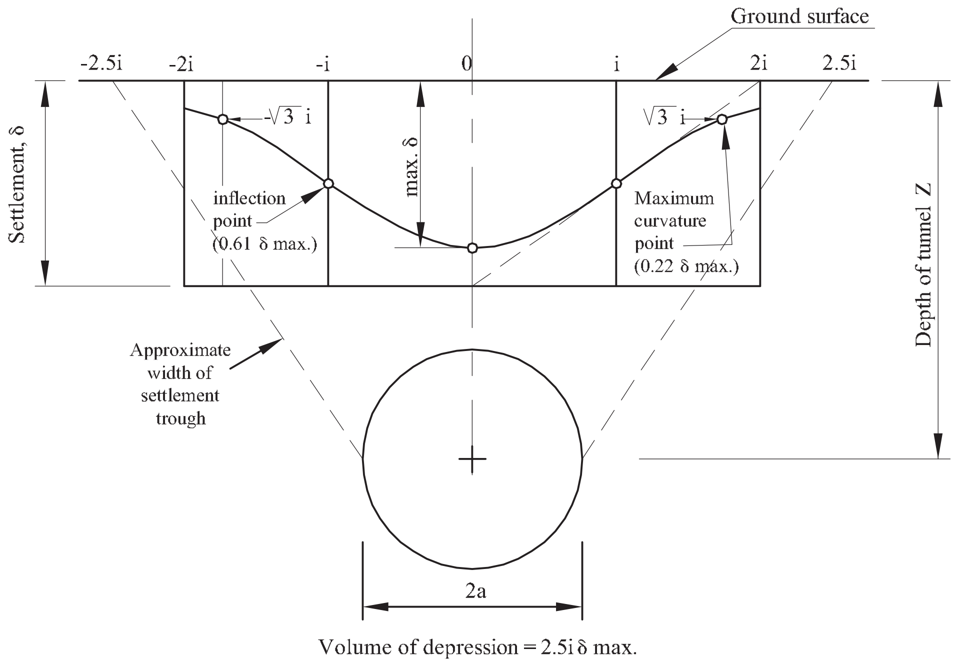
Designation	Degree of Compactness	Tunnel Behavior	
		Above Water Table	Below Water Table
Very Fine Clean Sand	Loose, $N \leq 10$	Cohesive Running	Flowing
	Dense, $N > 30$	Fast Raveling	Flowing
Fine Sand with Clay Binder	Loose, $N \leq 10$	Rapid Raveling	Flowing
	Dense, $N > 30$	Firm or Slowly Raveling	Slowly Raveling
Sand or Sandy Gravel with Clay Binder	Loose, $N < 10$	Rapid Raveling	Rapidly Raveling or Flowing
	Dense, $N > 30$	Firm	Firm/slow Raveling
Sandy Gravel and Medium to Coarse Sand		Running Ground. Uniform ($C_u < 3$) and loose ($N < 10$) materials with round grains run much more freely than well graded ($C_u > 6$) and dense ($N > 30$) ones with angular grains.	Flowing Conditions combined with extremely heavy discharge of water.

TBM FAMILY OF MACHINES
(From Kessler & Moore,)

Machine Type	Typical Machine Diameters	Ground Condition TBM is Best Suited For
Pipe Jacking Machines	Up to approx. 10 – 13 ft (3 - 4m)	Any ground
Small Bore Unit (SBU)	Up to 6.6 ft (2m)	Any ground
Shielded TBMs	6.6 – 46 ft (2 to 14m) plus	Soft ground above the water table
Mix Face TBMs	6.6 – 46 ft (2 to 14m) plus	Mixed ground above the water table
Slurry TBMs	6.6 – 46 ft (2 to 14m) plus	Coarse-grained soft ground below the water table
EPB TBMs	6.6 – 46 ft (2 to 14m) plus	Fine-grained soft ground below the water table
Hard Rock TBMs	6.6 – 46 ft (2 to 14m) plus	Hard rock
Reamer TBMs	Various	Hard rock
Multi-head TBMs	Various	Various

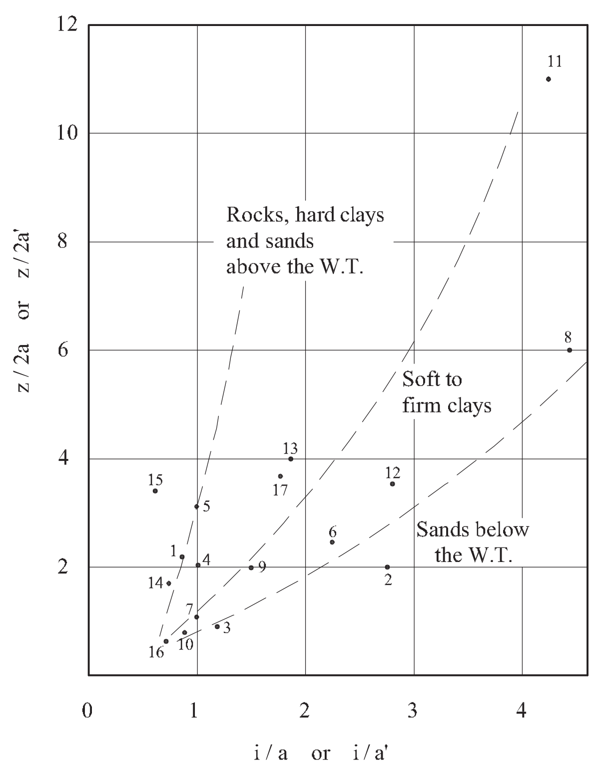
Reference: Dots Oyenuga (2004), "FHWA Road Tunnel Design Guidelines", Pages 8 and 10, published by U.S. Department of Transportation Federal Highway Administration, Report No. FHWA-IF-05-023, Washington DC.

Relation between the Width of the Surface Depression (i/a) and the Depth of the Cavity (z/a) for Tunnels



Volume of depression = 2.5i δ max.

(a)



(b)

Reference: Peck, R. B. (1969) "Deep Excavations and Tunneling in Soft Ground," Proceedings, Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico City, State of the Art Volume, pp. 225-290.

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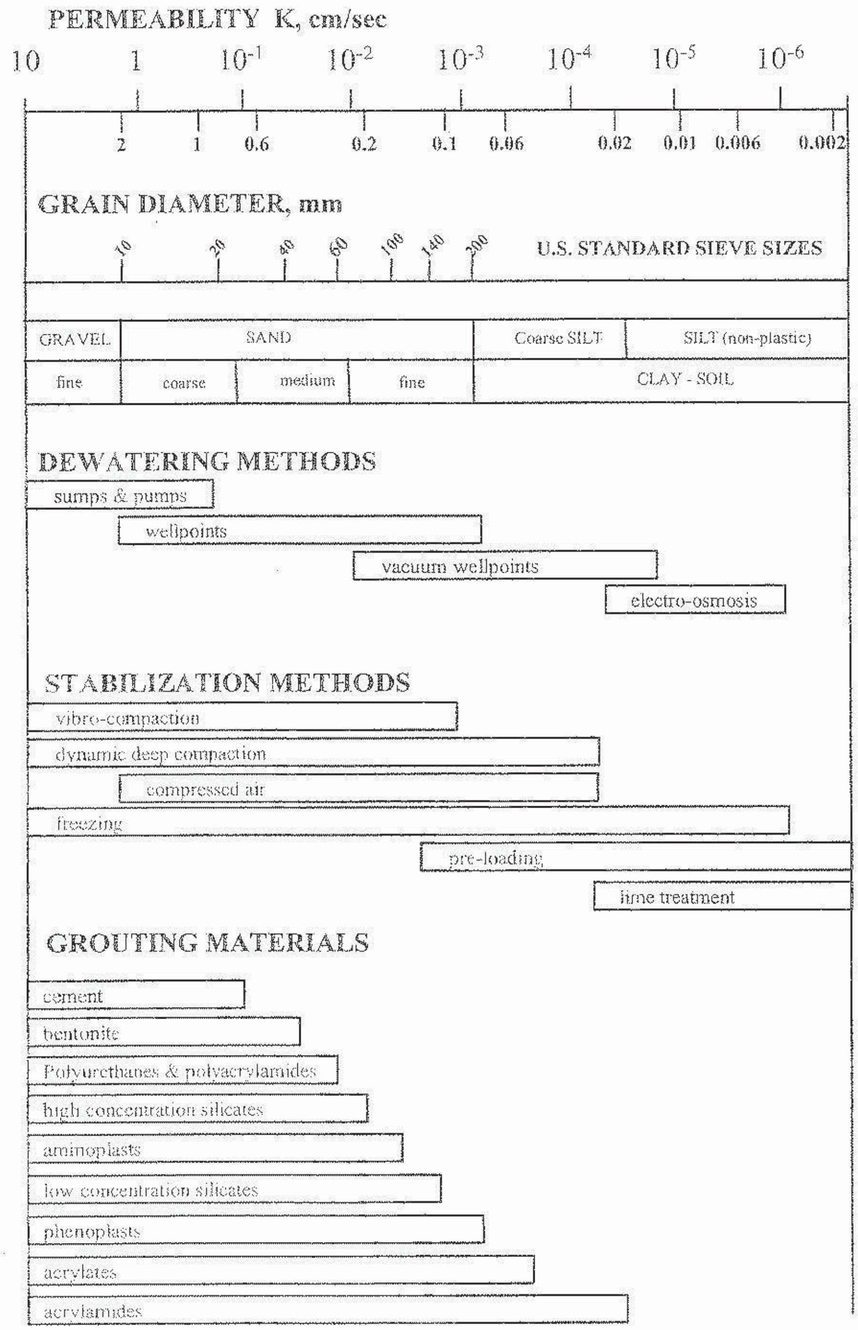
Methods of Controlling Groundwater
(after Karol, 1990)



Note: 1 cm/sec = 0.4 in/sec; 1 mm = 0.04 in.

Reference: Dots Oyenuga (2004), "FHWA Road Tunnel Design Guidelines", Page 9, published by U.S. Department of Transportation Federal Highway Administration, Report No. FHWA-IF-05-023, Washington DC.

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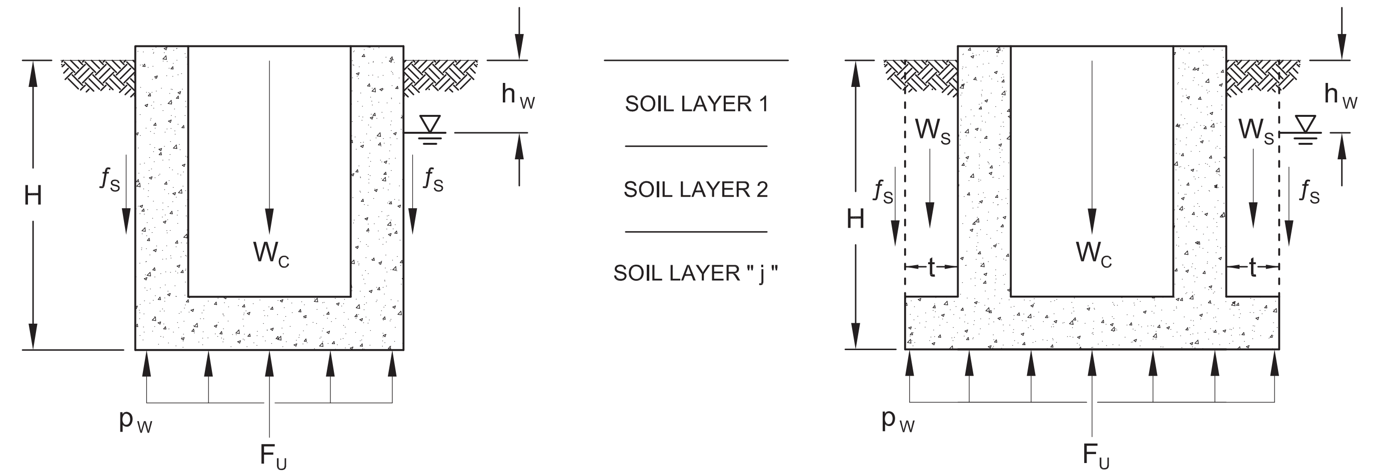
BUOYANT UPLIFT RESISTANCE FOR BURIED STRUCTURES

(a) WALL / SOIL FRICTION PLUS STRUCTURAL WEIGHT

(b) SOIL WEIGHT ABOVE BASE EXTENSION



(a)

cohesive soils: $f_{S_j} = \alpha \, c_j \leqq 3{,}000$ psf

cohesionless soils: $f_{S_j} = 0.75 \, K_S \, \sigma_{V_j} \tan\delta_j$

$$Q_S = P_S \sum f_{S_j} \, h_j$$

$$\frac{W_C}{S_{f_a}} + \frac{Q_S}{S_{f_b}} \geqq F_U$$

(b)

cohesive soils: $f_{S_j} = c_j \leqq 3{,}000$ psf

cohesionless soils: $f_{S_j} = 0.75 \, K_S \, \sigma_{V_j} \tan\Phi_j$

$$Q_S = P_S \sum f_{S_j} \, h_j$$

$$\frac{W_C}{S_{f_a}} + \frac{Q_S}{S_{f_b}} + \frac{W_S}{S_{f_c}} \geqq F_U$$

Where:

- A_B = area of base, sq. ft.
- H = buried height of structure, ft.
- h_W = depth to water table, ft.
- p_W = $\gamma_W (H - h_W)$, unit hydrostatic uplift, psf.
- γ_W = 62.4 pcf, unit weight of water
- F_U = $p_W A_B$, hydrostatic uplift force, lbs.
- f_{S_j} = unit frictional resistance of soil layer " j ", psf.
- c_j = undrained cohesion of soil layer " j ", psf.
- α = 0.55, cohesion factor between soil and structure wall
- σ_{V_j} = effective overburden pressure at midpoint of soil layer " j ", psf.
- δ_j = 0.75 Φ_j, friction angle between soil layer " j " and concrete wall, degrees
- Φ_j = internal angle of friction of soil layer " j ", degrees
- K_S = 0.4, coefficient of lateral pressure
- h_j = thickness of soil layer " j ", ft.
- j = 1, 2, ...
- P_S = perimeter of structure base, ft.
- Q_S = ultimate skin friction, lbs.
- W_C = weight of structure, lbs.
- W_S = weight of backfill above base extension, lbs.
- S_{f_a} = 1.1, factor of safety for dead weight of structure
- S_{f_b} = 3.0, factor of safety for soil / structure friction
- S_{f_c} = 1.5, factor of safety for soil weight above base extension
- t = width of base extension, ft.

NOTE: neglect f_S in upper 5 feet for expansive clay with a plasticity index > 20.

Reference:

1) *American Concrete Pipe Association, (1996), Manhole Floatation*
2) *O'Neill, M.W., and Reese, L.C., (1999), "Drilled Shafts: Construction Procedures and Design Methods", FHWA-IF-99-025*

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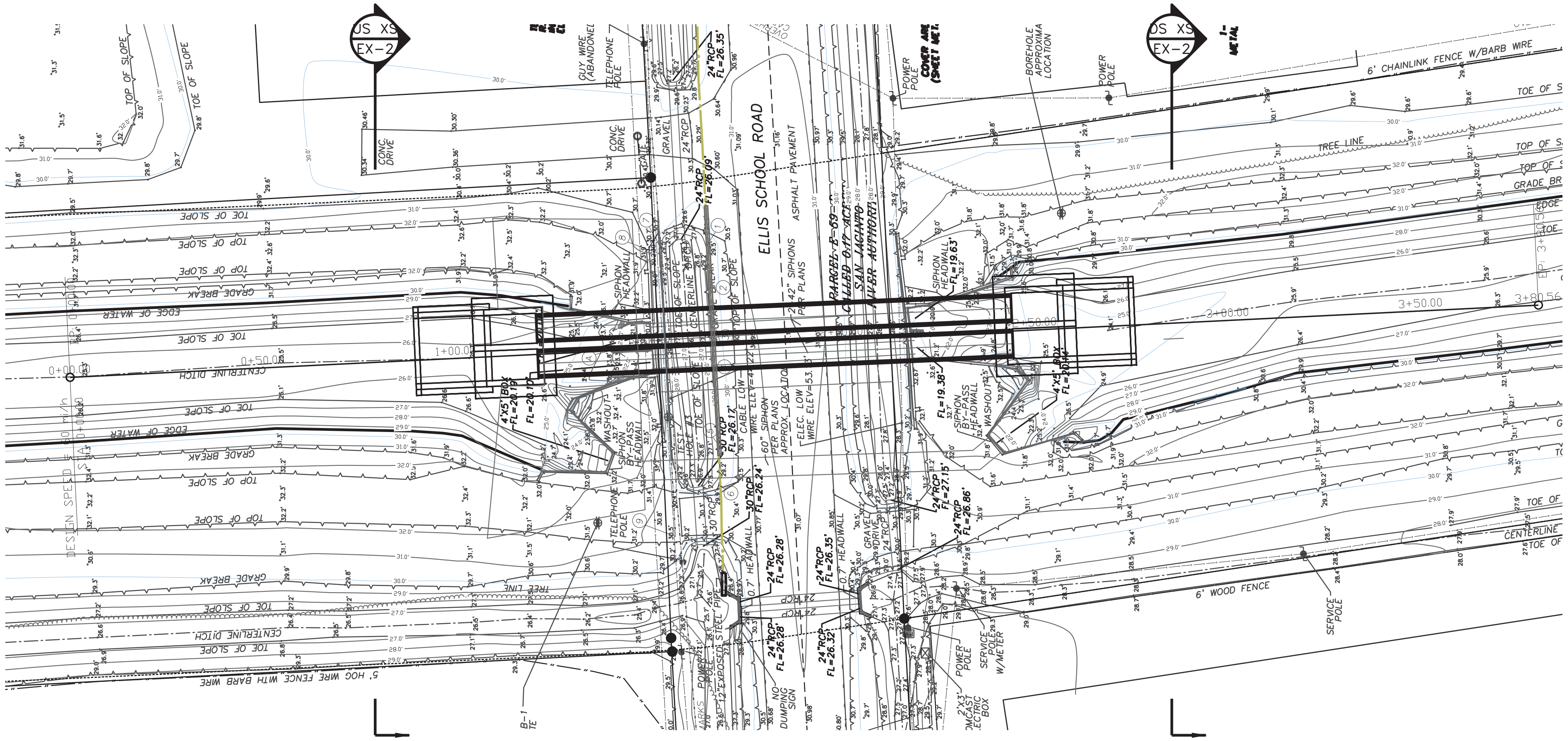
APPENDIX E

Plates E-1 and E-2	Plan and Profile, and Canal Cross Section Exhibits prepared by TWE, dated January 24, 2019
Plate E-3	Design Soil Parameters for Slope Stability Analyses
Plates E-4 to E-9	Slope Stability Analysis for East Bank of Upstream Cross Section at Station 0+81.49, based on Boring B-46

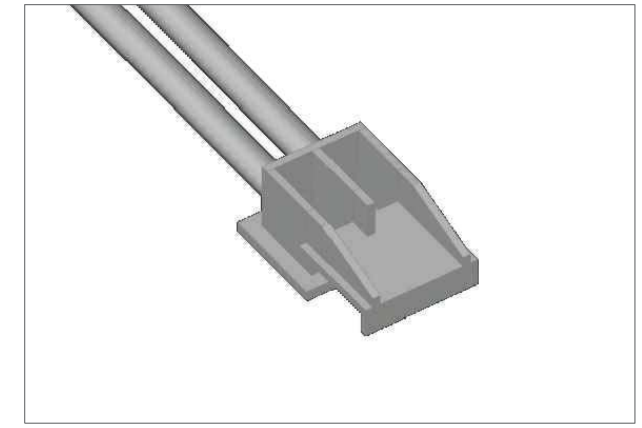
SCALE: 1"=20'

GRAPHIC SCALE IN FEET

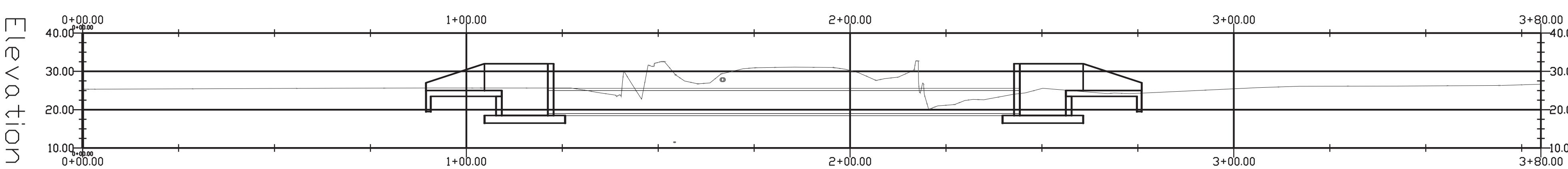
0 10 20 30 40



PLAN



SIPHON INTAKE/DISCHARGE STRUCTURE 3D RENDERING



PROFILE

TEXAS WATER ENGINEERING, PLLC.
Texas Registered Engineering Firm F-8482

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SAN JACINTO RIVER AUTHORITY HIGHLANDS DIVISION

SJRA HIGHLANDS SIPHON 29 IMPROVEMENTS

ISSUE	DATE	DESCRIPTION

SJRA PROJECT NO:
FILE NAME: Siphon-29-BASE.dwg
DRAWN BY: AC
CHECKED BY: VF
SCALE: AS SHOWN

SIPHON 29

PLAN AND PROFILE

SHEET EX-1

SEQ. 1 OF 2

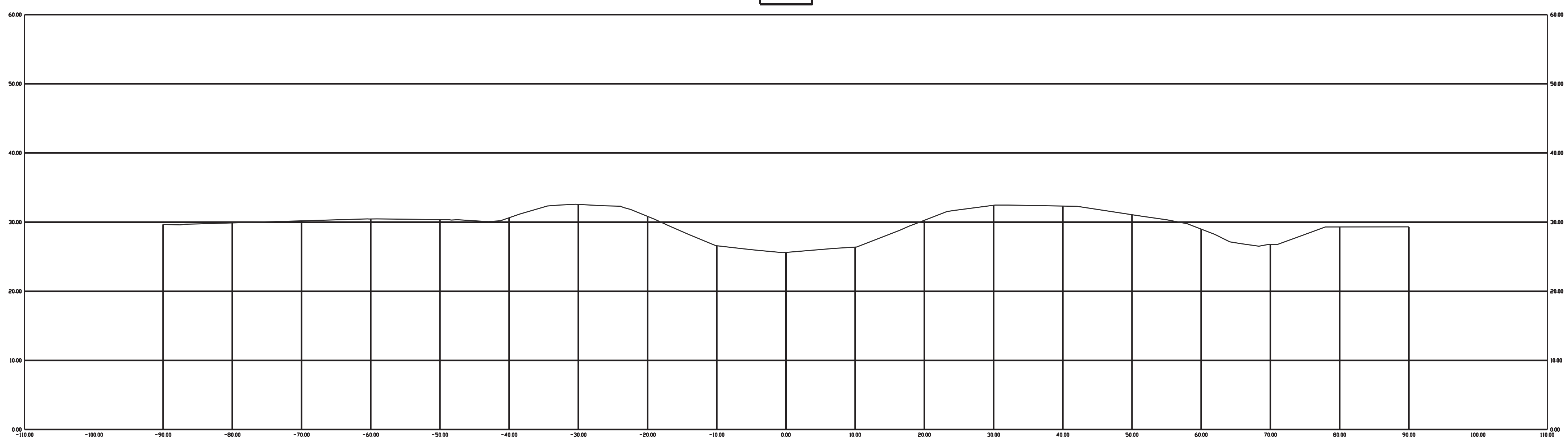
CONCEPTUAL EXHIBIT

FILE: C:\Users\Abby Crockett\Documents\Projects\SJRA Siphon 29\Drawings\Siphon-29-BASE.dwg LAYOUT: PNP DATE: 1/24/19 BY: ABBY CROCKETT

NOTES:

1. CROSS SECTIONS ARE LOOKING DOWNSTREAM/SOUTH.

0+81.49



UPSTREAM CROSS SECTION

2+88.43

DOWNSTREAM CROSS SECTION

SCALE: 1"=10'

GRAPHIC SCALE IN FEET
0 5 10 15 20

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SAN JACINTO RIVER AUTHORITY
HIGHLANDS DIVISION

SJRA HIGHLANDS
SIPHON 29
IMPROVEMENTS

ISSUE	DATE	DESCRIPTION

SJRA PROJECT NO:
FILE NAME: Siphon-29-BASE.dwg
DRAWN BY: AC
CHECKED BY: VF
SCALE: AS SHOWN

SIPHON 29
CROSS SECTIONS

SHEET EX-2
SEQ. 2 OF 2

CONCEPTUAL EXHIBIT

PLATE E-2

FILE: C:\Users\Abby Crockett\Documents\Projects\SJRA Siphon 29\Drawings\Siphon-29-BASE.dwg LAYOUT: XS DATE: 1/24/19 BY: ABBY CROCKETT

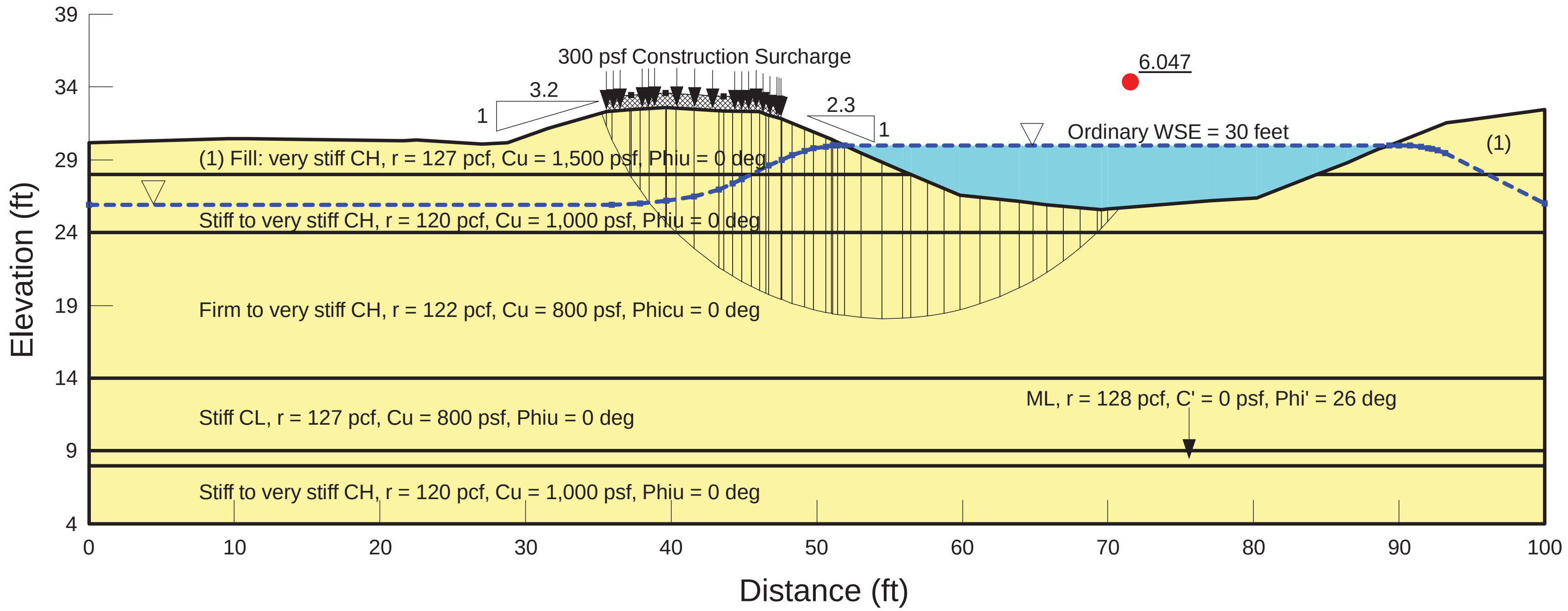
AVILES ENGINEERING CORP.

Design Soil Parameters for Slope Stability Analyses
East Bank of Upstream Cross Section at Station 0+81.49 (Based on Boring B-46)

Elevation (ft)	Soil Type	γ (pcf)	Short-Term (UU)		Long-Term (CD)		Rapid Drawdown (CU)	
			C_u (psf)	ϕ_u (deg)	C' (psf)	ϕ' (deg)	C_{cu} (psf)	ϕ_{cu} (deg)
32 to 28	Fill: very stiff CH	127	1,500	0	120 (C_r = 65)	19 (ϕ_r=21)	140 (C_r = 65)	16 (ϕ_r=21)
28 to 24	Stiff to very stiff CH	120	1,000	0	100 (C_r = 65)	17 (ϕ_r=21)	110 (C_r = 65)	13 (ϕ_r=21)
24 to 14	Firm to very stiff CH	122	800	0	90	17	100	13
14 to 9	Stiff CL	127	800	0	130	21	150	18
9 to 8	ML	128	0	26	0	26	0	26
8 to 4	Stiff to very stiff CH	120	1,000	0	120	17	140	14

Notes: (1) γ = wet unit weight of soil;
(2) C_u =undrained cohesion, ϕ_u = angle of internal friction, under short term conditions. UU = strength parameters that were determined from Unconsolidated-Undrained triaxial tests;
(3) C' =effective cohesion, ϕ' =effective friction angle, under long term condition; CD = Consolidated-Drained strength parameters that were determined from CU triaxial tests with pore pressure measurements;
(4) C_{cu} = cohesion, ϕ_{cu} = friction angle, under rapid drawdown condition; CU = strength parameters developed from Consolidated-Undrained triaxial tests;
(5) C_r = cohesion for desiccated fat clay, ϕ_r = friction angle for desiccated fat clay
(6) CH = fat clay, CL = Lean Clay, ML = Silt.

G155-18 HIGHLANDS SOUTH CANAL SIPHON 29 IMPROVEMENTS - WO#2
SLOPE STABILITY ANALYSIS FOR EAST BANK OF UPSTREAM CROSS SECTION AT STATION 0+81.49
SHORT TERM (GLOBAL SLIDE) CONDITION, BASED ON BORING B-46



G155-18 HIGHLANDS SOUTH CANAL SIPHON 29 IMPROVEMENTS - WO#2
SLOPE STABILITY ANALYSIS FOR EAST BANK OF UPSTREAM CROSS SECTION AT STATION 0+81.49
SHORT TERM (LOCAL SLIDE) CONDITION, BASED ON BORING B-46

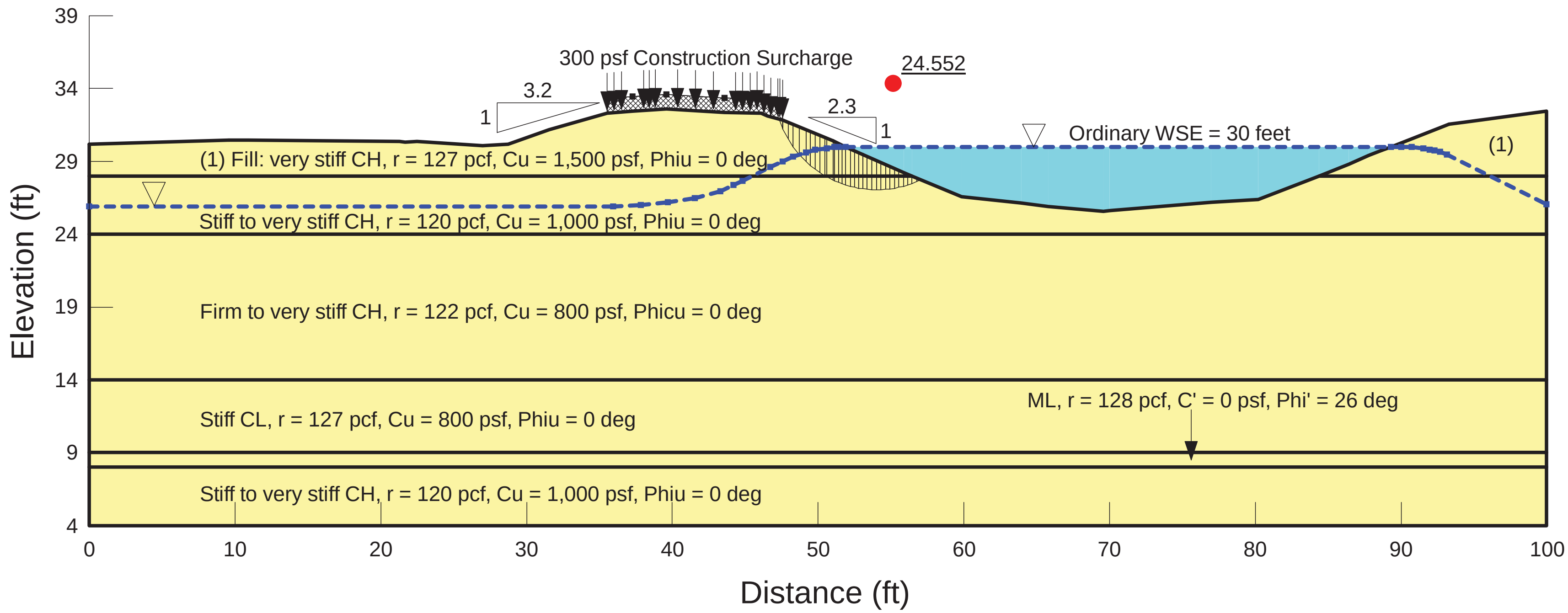


PLATE E-5

G155-18 HIGHLANDS SOUTH CANAL SIPHON 29 IMPROVEMENTS - WO#2
SLOPE STABILITY ANALYSIS FOR EAST BANK OF UPSTREAM CROSS SECTION AT STATION 0+81.49
LONG TERM (GLOBAL SLIDE) CONDITION, BASED ON BORING B-46

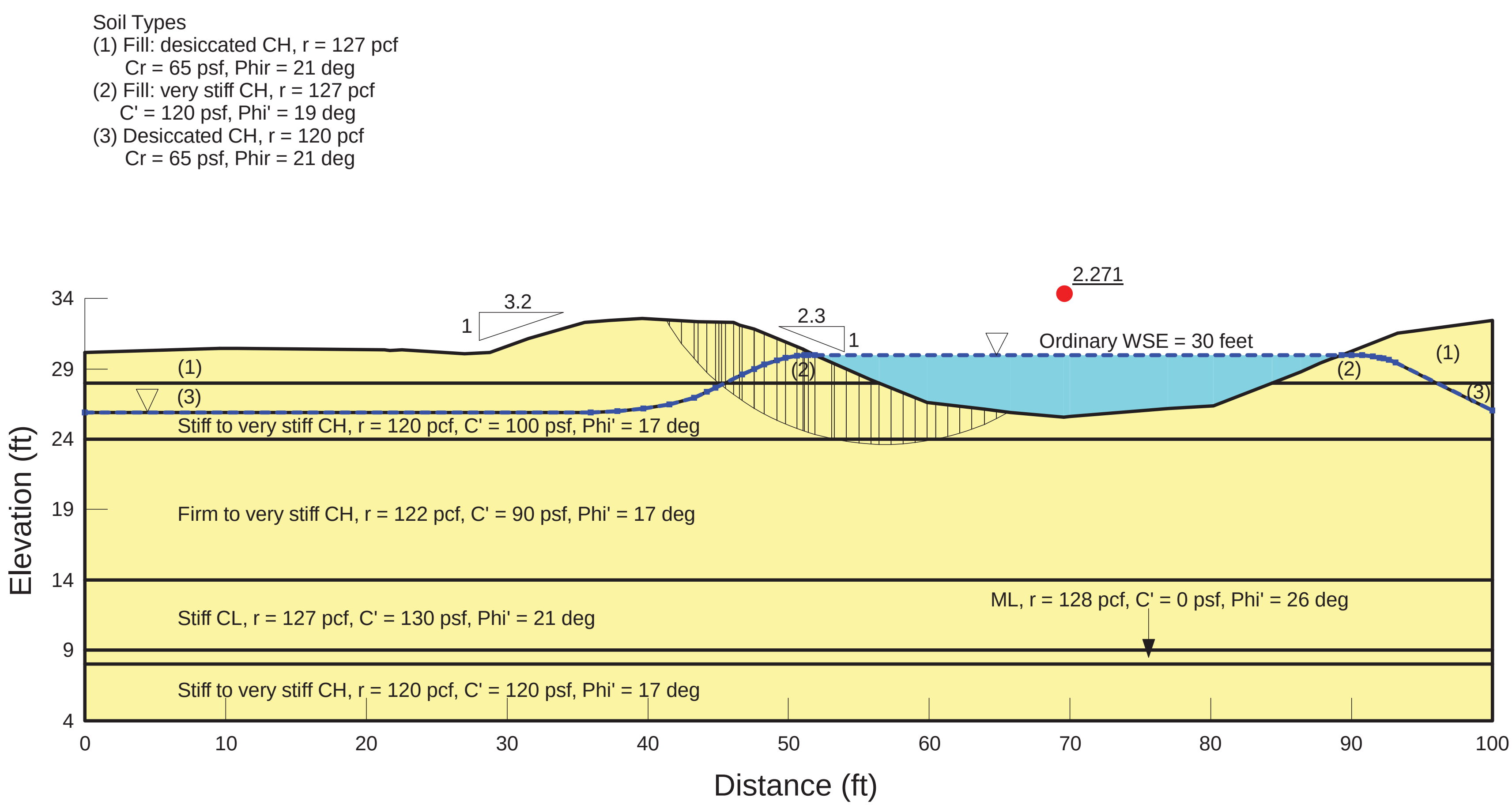
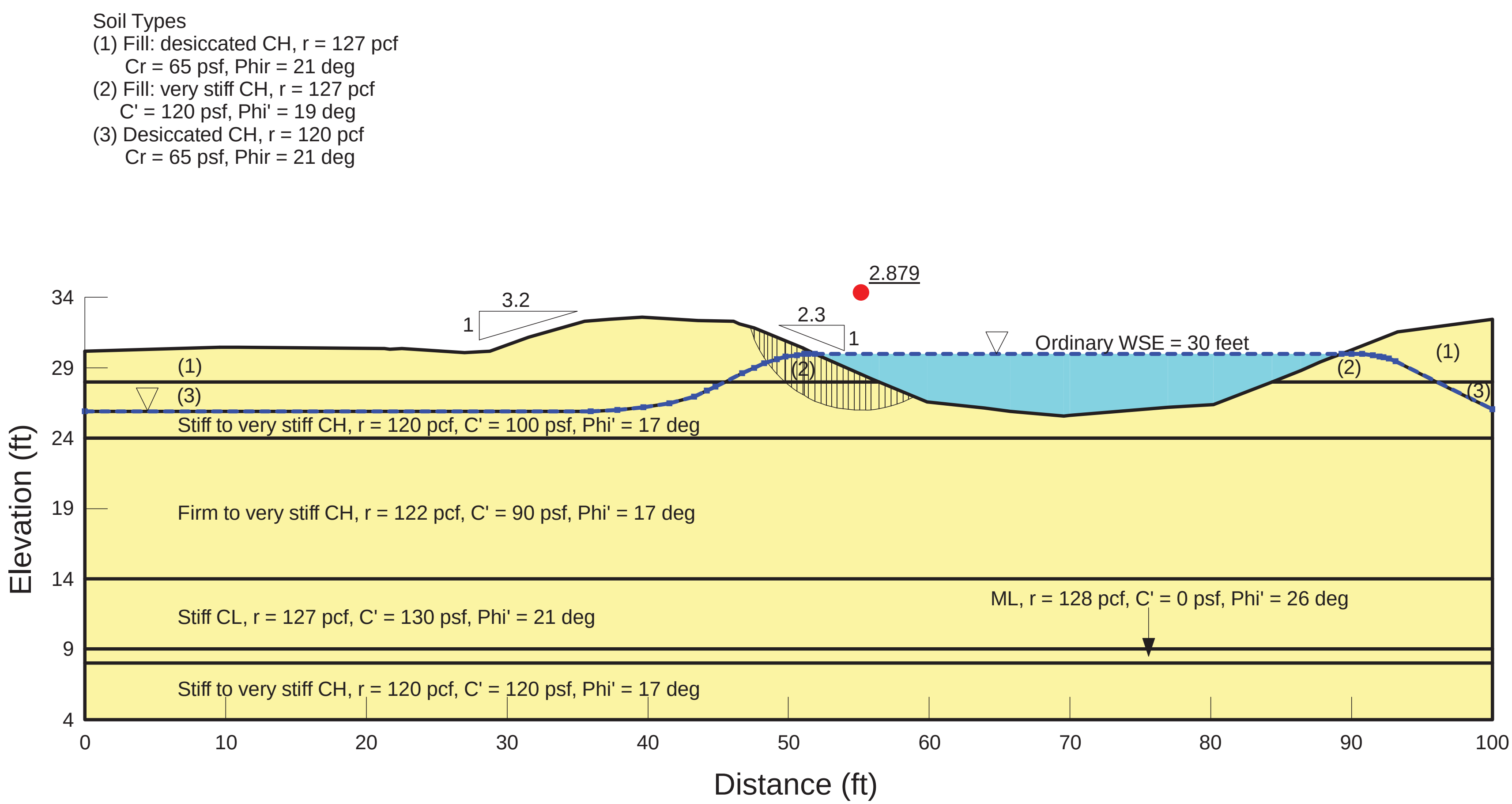


PLATE E-6

G155-18 HIGHLANDS SOUTH CANAL SIPHON 29 IMPROVEMENTS - WO#2
SLOPE STABILITY ANALYSIS FOR EAST BANK OF UPSTREAM CROSS SECTION AT STATION 0+81.49
LONG TERM (LOCAL SLIDE) CONDITION, BASED ON BORING B-46



G155-18 HIGHLANDS SOUTH CANAL SIPHON 29 IMPROVEMENTS - WO#2
SLOPE STABILITY ANALYSIS FOR EAST BANK OF UPSTREAM CROSS SECTION AT STATION 0+81.49
RAPID DRAWDOWN (GLOBAL SLIDE) CONDITION, BASED ON BORING B-46

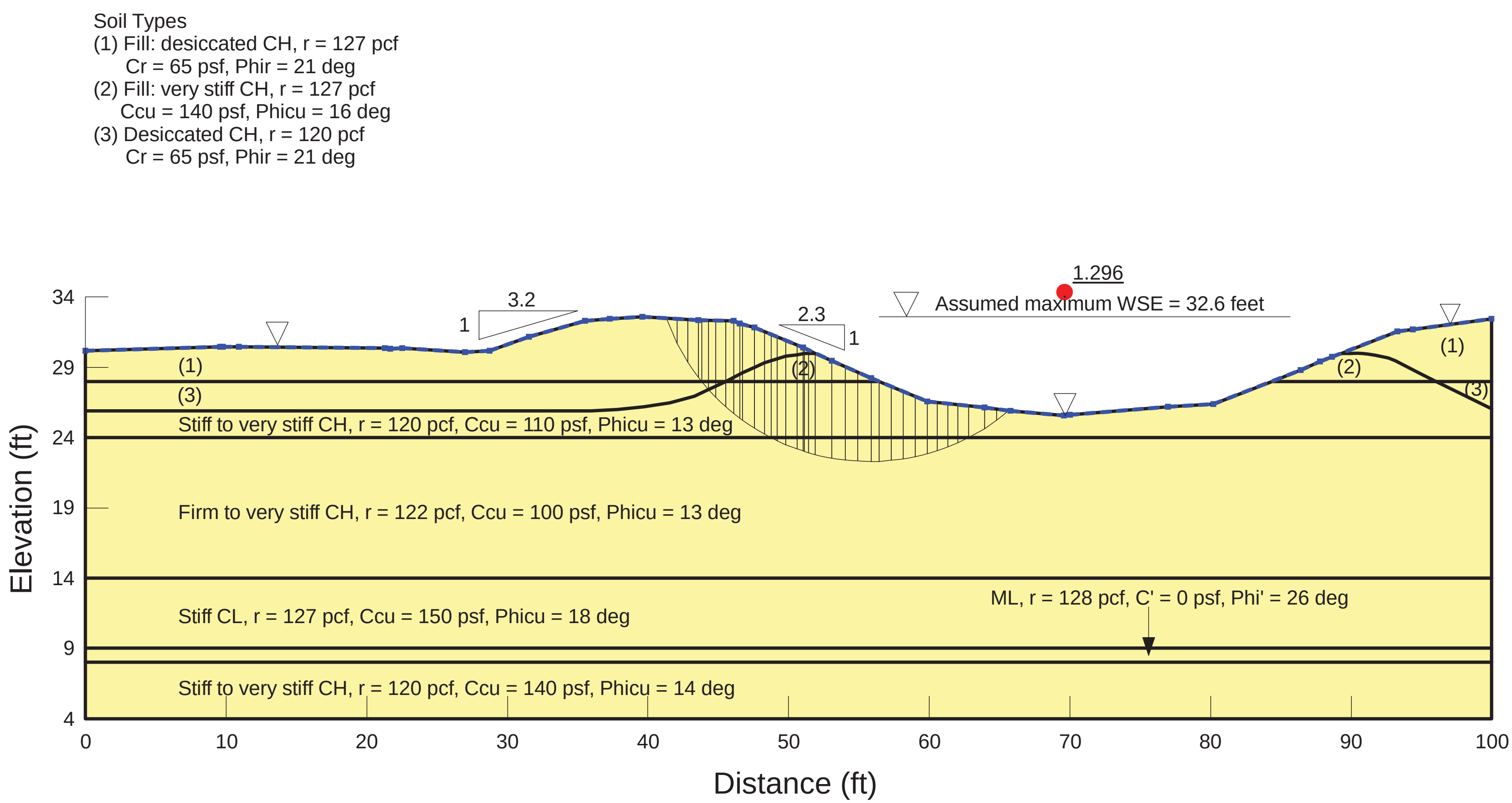


PLATE E-8

G155-18 HIGHLANDS SOUTH CANAL SIPHON 29 IMPROVEMENTS - WO#2
SLOPE STABILITY ANALYSIS FOR EAST BANK OF UPSTREAM CROSS SECTION AT STATION 0+81.49
RAPID DRAWDOWN (LOCAL SLIDE) CONDITION, BASED ON BORING B-46