AVILES ENGINEERING CORP.

# GEOTECHNICAL INVESTIGATION

## SAN JACINTO RIVER AUTHORITY
## WALLISVILLE SIPHON REPLACEMENT PROJECT
## HARRIS COUNTY, TEXAS

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

**by**

**Aviles Engineering Corporation**
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**REPORT NO. G108-18 (Revision 1)**

**August 2018**

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August 9, 2018

Ms. Victoria Foss, P.E., Owner
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**Reference:** **Geotechnical Investigation**
**San Jacinto River Authority**
**Wallisville Siphon Replacement Project**
**Harris County, Texas**
**AEC Report No. G108-18 (Revision 1)**

Dear Ms. Foss,

Aviles Engineering Corporation (AEC) is pleased to present this revised report of the results of our geotechnical investigation for the above referenced project. Notice to proceed for the Task 1 (for drilling and laboratory tests) and Task 2 (for engineering) of the project were provided by you on February 1, 2018 and on March 26, 2018, respectively. The project terms and conditions were in accordance with the Professional Services Agreement between Texas Water Engineering, PLLC (TWE) and AEC, dated February 1, 2018, based upon AEC proposal G2017-12-05R3, dated January 2, 2018. The contents of this report supersede AEC's previous reports.

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 Wallisville Siphon Replacement in Harris County, Texas (Houston/Harris County Key Map No. 461Q). Based on the Plan and Profile drawings (dated April 20, 2018) and the information provided by Texas Water Engineering, PLLC. (TWE), AEC understands that the siphon replacement will accommodate the needs for the Harris County East Wallisville Road Expansion Project with the future ROW of the roadway of 117.5 feet. The replacement of the siphon consists of: (i) installation of new dual 48 inch diameter centrifugally cast fiberglass reinforced polymer motar (CCFRPM) siphon pipes. The new siphons will likely be installed using open cut method at the areas of the intake/discharge structures within the SJRA easement, and then using pipe jacking method beneath Wallisville Road. The invert elevation of the new siphon pipes will be 14.5 above Mean Sea Level (MSL) (i.e. maximum invert depth of approximately 22 to 23 feet below the top of existing levee); (ii) installation of associated intake and discharge structures at both ends of the siphon pipes. The proposed flow line elevation of the canal at the intake/discharge structures will be +30 feet MSL; and (iii) demolition and removal of the existing 48-inch diameter reinforced concrete siphon pipe located outside the existing road ROW after installation of the new siphons and headwall structures, while the portion beneath Walliville Road will remain in place. AEC previously drilled two 30-foot deep borings and issued a Geotechnical Report G185-10, dated February 4, 2011. The contents of this revised report supersede AEC's previous report.

As requested by TWE, AEC drilled two borings (Borings B-3 and B-4) adjacent to the existing siphon to a depth of 40 feet below existing grade. The previously drilled boring logs (Borings B-1 and B-2) are also included in this report, as reference.

1. <u>Subsurface Soil Conditions:</u> Based on AEC's borings, the subsurface soil conditions in the vicinity of the siphon generally consist of soft to hard lean/fat clay (CL/CH) from the ground surface to depths of 33 to 36 feet; however, approximately 2 to 4 feet of loose silt/silty sand (ML/SM) are encountered at depths between 14 and 18 feet, and approximately 4 to 7 feet of medium dense clayey/silty clayey sand (SC/SC-SM) are encountered at a depth of 33 feet to the boring termination depth of 40 feet in Boring B-3, and at a depth of 36 feet to the boring termination depth of 40 feet in Boring B-4.

2. <u>Subsurface Soil Properties:</u> The subsurface cohesive soils encountered in the borings have slight to very high plasticity, with liquid limits (LL) ranging from 27 to 69, and plasticity indices (PI) ranging from 9 to 47. The cohesive soils encountered are classified as "CL" and "CH" type soils and granular soils were classified as "ML", "SM", "SC-SM", and "SC" type soils in accordance with ASTM D 2487.

3. <u>Groundwater Conditions:</u> Groundwater was initially encountered in Borings B-3 and B-4 at depths of 16 to 17 feet below grade during drilling, and subsequently rose to a depth between 6.9 to 9.0 feet approximately 15 minutes after the initial encounter. Groundwater was measured at 2.7 to 3.7 feet below grade approximately 3 to 7 days after drilling was completed. Groundwater levels encountered in the borings are summarized in Table 5 in Section 4.1 of this report. Based on the groundwater readings in Table 5, where encountered, groundwater at the site is likely pressurized.

## EXECUTIVE SUMMARY (CONTINUE)

4. Soil Dispersion Characteristics: AEC performed a total of 3 crumb tests to evaluate the dispersive characteristics of clay soils at the site; the results indicate that the tested soil samples in the channel zone are non-dispersive. However, AEC also performed 2 double hydrometer tests on selected soil samples, and the tested sample have percent dispersion of 14.3 to 30.4, which indicates the presence of non-dispersive to moderately dispersive clays.

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

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

AVILES ENGINEERING CORP.

**GEOTECHNICAL INVESTIGATION**

**SAN JACINTO RIVER AUTHORITY**
**WALLISVILLE SIPHON REPLACEMENT PROJECT**
**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 Wallisville Siphon Replacement in Harris County, Texas (Houston/Harris County Key Map No. 461Q). A vicinity map is presented on Plate A-1, in Appendix A. Based on the Plan and Profile drawings (dated April 20, 2018) and the information provided by Texas Water Engineering, PLLC. (TWE), AEC understands that the siphon replacement will accommodate the needs for the Harris County East Wallisville Road Expansion Project with the future ROW of the roadway of 117.5 feet. The replacement of the siphon consists of: (i) installation of new dual 48 inch diameter centrifugally cast fiberglass reinforced polymer motar (CCFRPM) siphon pipes. The new siphons will likely be installed using open cut method at the areas of intake/discharge structures within the SJRA easement, and then using pipe jack and bore method beneath Wallisville Road. The invert elevation of the new siphons will be 14.5 above Mean Sea Level (MSL) (i.e. maximum invert depth of approximately 22 to 23 feet below the top of existing levee); (ii) installation of associated intake and discharge structures at both ends of the siphon pipes. The proposed flow line elevation of the canal at the intake/discharge structures will be +30 feet MSL; and (iii) demolition and removal of the existing 48 inch diameter reinforced concrete siphon pipe located outside the existing road ROW after the installation of the new siphons and headwall structures, while the portion beneath Walliville Road will remain in place. AEC previously drilled two 30-foot deep borings and issued a Geotechnical Report G185-10, dated February 4, 2011. The contents of this revised report supersede AEC's previous reports.

### 1.2 Purpose and Scope

As requested by TWE, the purpose of this geotechnical investigation is to evaluate the subsurface soil and ground water conditions at the site and develop geotechnical engineering recommendations for design and construction of the siphon pipes and associated siphon intake/discharge structures. The scope of this geotechnical investigation is summarized below:

1. Drilling and sampling two geotechnical borings to a depth of 40 feet below existing grade;
2. Soil laboratory testing on selected soil samples;
3. Compare existing soil conditions in the new borings with the previously performed Borings B-1 and B-2, and determine whether an updated report for engineering analysis and recommendations is necessary. If TWE determines that an updated report is required, AEC will incorporate all borings to perform Items 4 through 7 below;
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 pipe jacking method, including tunnel access shafts, reaction walls, and tunnel stability;
6. Engineering analyses and recommendations for the headwall/wing walls of the siphon intake/discharge structures, including allowable bearing capacity and design soil parameters for lateral earth pressure; and
7. Construction recommendations for the siphon and intake/discharge structures.

Slope stability of the existing levee slopes and recommendations for the future roadway pavements of Wallisville Road Expansion Project are beyond AEC's scope of services.

## 2.0 SUBSURFACE EXPLORATION

As requested by TWE, AEC drilled two borings (Borings B-3 and B-4) adjacent to the existing siphon to a depth of 40 feet below the top of existing levee. The boring locations are shown on the Boring Location Plan on Plate A-2, in Appendix A. The previously drilled boring logs (Borings B-1 and B-2) are also included on Plates A-3 and A-4, in Appendix A of this report, as reference. After completion of drilling, the boring locations were surveyed by others and the approximate boring surface elevations are measured from the Plan and Profile drawing provided by TWE on April 20, 2018. Boring survey data for Borings B-3 and B-4 in State Plane *Grid* Coordinates (Texas South Central Zone) and the approximate boring surface elevations, as well as the corresponding siphon pipe inverts and elevations are summarized in Table 1.

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**Table 1. Summary of Borings and Invert Depths**

| Boring No. | Boring Depth (ft) | Northing (Grid, ft) | Easting (Grid, ft) | Approx. Boring Surface Elevation (ft) | Proposed 48-Inch Dia. Siphon Invert Elevation at Boring (ft) | Proposed Siphon Invert Depth at Boring (ft) |
|---|---|---|---|---|---|---|
| B-3 | 40 | 13868454.70 | 3243800.44 | 36 | 14.5 | 21.5 |
| B-4 | 40 | 13868630.82 | 3243831.82 | 37 | 14.5 | 22.5 |

Note: (1) Northing and Easting are *in Grid*, and converted from surface by multiplying a scale factor of 0.99990166 provided by the Surveyor.
(2) Approximate elevations and inverts are obtained from the Plan & Profile drawings provided by TWE on April 20, 2018.

The borings were generally advanced initially using dry auger method, and then using wet rotary method once groundwater was encountered, or the borings caved 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 to 7 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 typical samples to establish the index properties and confirm field classification of the subsurface soils. Strength properties of cohesive soils were determined by means of torvane (TV), unconfined compression (UC), and undrained-unconsolidated (UU) triaxial tests performed on undisturbed samples. The test results are presented on the boring logs. Details of the soils encountered in Borings B-3 and B-4 are presented on

Plates A-5 and A-6, 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-7 through A-10, in Appendix A. Sieve analysis results are presented on Plate A-11, in Appendix A.

Double Hydrometer Tests: To evaluate the dispersive characteristics of clayey soils at the siphon, two double hydrometer tests were performed on a selected soil sample in accordance with ASTM D 4221. The results of the double hydrometer tests are summarized in Table 2, and are presented on Plates A-12 and A-13, in Appendix A. When the percent dispersion equals 100, it indicates a completely dispersive clay-sized fraction. When the percent dispersion equals 0, it indicates completely non-dispersive clay.

$D_{50}$ Grain Size Analysis Tests: AEC also determine the $D_{50}$ (grain diameter, in mm, corresponding to 50 percent passing by weight) from double hydrometer tests on selected samples obtained from the borings as reference for scour analysis and selection of geofabric filter of riprap, if needed. The $D_{50}$ of the selected samples are also included in Table 2, and presented on Plates A-12 through A-13, in Appendix A.

**Table 2. Summary of Double Hydrometer Test Results**

| Sample ID and Description | Dispersion (%) | $D_{50}$ (mm) |
|---|---|---|
| B-3, 6’-8’, Fat Clay (CH) | 14.27 | 0.0125 |
| B-4, 12’-14’, Lean Clay (CL) | 30.36 | 0.0132 |

Crumb Tests: To evaluate the dispersive characteristics of clayey soils at the siphon, three crumb tests were performed on selected soil samples in the channel zone in accordance with ASTM D 6572, Method A. The results of the crumb tests are summarized on Table 3 and are presented on Plate A-14, in Appendix A.

**Table 3. Summary of Crumb Test Results**

| Sample ID and Description | Dispersive Grade | Dispersive Classification |
|---|---|---|
| B-3, 6’-8’, Fat Clay (CH) | 1 | Non-dispersive |
| B-4, 2’-4’, Fill: Lean Clay (CL) | 1 | Non-dispersive |
| B-4, 12’-14’, Lean Clay (CL) | 1 | Non-dispersive |

# 4.0 SITE CONDITIONS

## 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. A generalized subsurface profile is presented on Plate B-1, in Appendix B.

| Boring | Depth (ft) | Description of Stratum |
|---|---|---|
| B-1 | 0 - 8 | Very stiff to hard, Sandy Lean Clay (CL), with sand seams and roots |
| | 8 - 12 | Very stiff, Fat Clay with Sand (CH), with calcareous nodules |
| | 12 - 18 | Very stiff, Lean Clay (CL), with silt partings and calcareous nodules |
| | 18 - 30 | Stiff, Lean Clay (CL), with silt partings |
| B-2 | 0 - 6 | Very stiff to hard, Sandy Lean Clay (CL), with roots and calcareous nodules |
| | 6 - 10 | Firm to very stiff, Fat Clay with Sand (CH), with silt partings and calcareous nodules |
| | 10 - 14 | Very stiff, Fat Clay (CH), with calcareous nodules |
| | 14 - 16 | Stiff, Lean Clay (CL), with silt partings |
| | 16 - 18 | Soft to firm, Fat Clay (CH) |
| | 18 - 22 | Firm, Lean Clay (CL), with silt seams and partings |
| | 22 - 27 | Stiff, Fat Clay with Sand (CH), with silt partings |
| | 27 - 30 | Firm to stiff, Lean Clay (CL), with silt partings and silty sand seams |
| B-3 | 0 - 4 | Very stiff, Lean Clay (CL) |
| | 4 - 14 | Soft to very stiff, Fat Clay (CH), with ferrous nodules |
| | 14 - 16 | Sandy Silt (ML), with clayey sand pockets, wet |
| | 16 - 18 | Silty Sand (SM), wet |
| | 18 - 27 | Stiff to very stiff, Fat Clay (CH) |
| | 27 - 33 | Very stiff, Lean Clay with Sand (CL), with silty sand pockets, partings, and laminations |
| | 33 - 38 | Clayey Sand (SC), wet |
| | 38 - 40 | Medium dense, Silty Clayey Sand (SC-SM), wet |
| B-4 | 0 - 4 | Fill: firm to very stiff, Lean Clay (CL) |
| | 4 - 8 | Stiff to very stiff, Lean Clay (CL), with calcareous nodules |
| | 8 - 10 | Stiff to very stiff, Fat Clay (CH), with abundant calcareous nodules |
| | 10 - 16 | Firm to very stiff, Lean Clay (CL), with calcareous nodules |
| | 16 - 18 | Loose, Silt (ML), with fat clay pockets, wet |
| | 18 - 23 | Very stiff, Lean Clay with Sand (CL), with silty sand and fat clay pockets |
| | 23 - 33 | Stiff to very stiff, Fat Clay (CH), with slickensides |
| | 33 - 36 | Firm to stiff, Lean Clay with Sand (CL), with silty sand and silty clay pockets |
| | 36 - 40 | Clayey Sand (SC), with abundant silt and sand partings, wet |

Subsurface Soil Properties: The subsurface cohesive soils encountered in the borings have slight to very high plasticity, with liquid limits (LL) ranging from 27 to 69, and plasticity indices (PI) ranging from 9 to 47. The cohesive soils encountered are classified as "CL" and "CH" type soils and granular soils were classified as "ML", "SM", "SC-SM", and "SC" 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 was initially encountered in Borings B-3 and B-4 at depths of 16 to 17 feet below grade during drilling, and subsequently rose to a depth between 6.9 to 9.0 feet approximately 15 minutes after the initial encounter. Groundwater was measured at 2.7 to 3.7 feet below grade approximately 3 to 7 days after drilling was completed. Groundwater levels encountered in the borings are summarized in Table 5. Based on the groundwater readings in Table 5, where encountered, groundwater at the site is likely pressurized.

**Table 5. Groundwater Depths below Existing Ground Surface**

| Boring No. | Date Drilled | Boring Depth (ft) | Groundwater Depth (ft) | Boring Cave in Depth (ft) |
|---|---|---|---|---|
| B-1[(1)] | 12/13/10 | 30 | 18 (Drilling)<br>10.5 (12/14/10) | - |
| B-2[(1)] | 12/13/10 | 30 | 18 (Drilling)<br>9.3 (12/14/10) | - |
| B-3 | 2/16/18 | 40 | 16 (Drilling)<br>9.0 (15 min.)<br>2.7 (2/23/18) | 14.4 (Drilling)<br>9.7 (2/23/18) |
| B-4 | 2/20/18 | 40 | 17 (Drilling)<br>6.9 (15 min.)<br>3.7 (2/23/18) | 14 (Drilling) |

Note: (1) Borings were previously drilled in AEC Report No. G185-10

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 or siltstone fragments, 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

Based on the Plan and Profile drawings (dated April 20, 2018) and the information provided by TWE, AEC understands that the siphon replacement will accommodate the needs for the Harris County East Wallisville Road Expansion Project with the future ROW of the roadway of 117.5 feet. The replacement of the siphon consists of: (i) installation of new dual 48 inch diameter CCFRPM siphon pipes. The new siphons will likely be installed using open cut method at the areas of intake/discharge structures within the SJRA easement, and then using pipe jack and bore method beneath Wallisville Road. The invert elevation of the new siphons will be 14.5 above MSL (i.e. maximum invert depth of approximately 22 to 23 feet below the top of existing levee); (ii) installation of associated intake and discharge structures at both ends of the siphon pipes. The proposed flow line elevation of the canal at the intake/discharge structures will be +30 feet MSL; and (iii) demolition and removal of the existing 48 inch diameter reinforced concrete siphon pipe outside the roadway ROW after the installation of the new siphons and headwall structures, while the portion beneath Walliville Road will remain in place.

## 5.1 Geotechnical Parameters for Siphon Pipes

Recommended soil parameters for design of siphon pipes and headwalls/wing walls are presented on Plate C-1, in Appendix C. The design values are based on the results of field and laboratory test data on 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 Installation of Siphon Pipes 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 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):

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

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

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

where:
$W_e$ = trench fill load, in pounds per linear foot (lb/ft);
$C_d$ = trench load coefficient, see Plate C-2, in Appendix C;
$\gamma$ = 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,<br>0.1500 maximum for saturated top soil,<br>0.1300 maximum for ordinary clay,<br>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-3, 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);
- $\gamma$ = effective unit weight of soil over the conduit (pcf);
- $p_s$ = vertical pressure on conduit resulting from traffic and/or construction equipment (psf).

5.2.2 Trench Stability

Cohesive soils in the Greater 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);
$\gamma$, $\gamma'$ = 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 for level ground;
$c$ = cohesion of clayey soils (psf); c can be omitted conservatively;
$\gamma_w$ = 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.

Based on our boring logs and the invert depths of the siphon pipes presented on Table 1 in Section 2.0 of this report, AEC anticipates that open cut excavations will likely encounter silty sand/silt (SM/ML) soils within the trench or pipe bedding zone. Based on the groundwater levels presented on Table 5 in Section 4.1 of this report, AEC anticipates that pressurized groundwater will likely be encountered within the trench or pipe bedding zone. 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 5 feet below the excavation in accordance with Section 01 57 23.02 of the latest edition of the SJRA Construction Specifications.

Calcareous nodules, silt/sand layers/seams/partings, and fat clays with slickensides were encountered in some of the 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.

### 5.2.3 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 Pipe Jacking

AEC understands that the siphon pipes beneath Wallisville Road will be installed by one-pass pipe jacking 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 as required. The following discussion provides general guidelines to the Contractor for reference purposes. The approximate tunnel invert depths, the most critical subsurface and groundwater conditions at the siphon are summarized in Table 6 below.

**Table 6. Subsurface and Groundwater Conditions within Tunnel Zone**

| Boring No. | Tunnel Invert Depth at Boring (ft) | Possible Soil Types Encountered within Tunnel Zone | Ground Water Depth below Existing Ground Surface (ft) |
|---|---|---|---|
| B-1 & B-4 | 22.6 | 14'-16': Firm to very stiff CL, with silty sand seams/partings<br>16'-18': Loose ML, wet<br>18'-23': Stiff to very stiff CL<br>23'-27': Stiff to very stiff CH/CL | 17 (Drilling)<br>3.7 (3 days) |
| B-2 & B-3 | 21.6 | 13'-14': soft to very stiff CH, with crawfish holes<br>14'-18': Wet ML/SM or soft to firm CL/CH<br>18'-26': Firm to very stiff CL/CH, with silt seams/partings | 16 (Drilling)<br>2.7 (7 days) |

Note: (1) Tunnel influence zone assumed between one pipe diameter above the top of pipe, and one pipe diameter below pipe invert.
(2) CH = Fat Clay, CL = Lean Clay, ML = Silt, SM = Silty Sand.

Pipe jacking and bore operations should comply with Section 33 05 23.19 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

Pipe jacking system should be constructed in accordance with Item 3.3G in Section 33 05 23.19; while tunnel access shaft should be constructed in accordance with Section 31 75 00 of the latest edition of the SJRA Construction Specifications. Based on Table 6 in Section 5.3 of this report, the tunnel access shafts on the siphon tunnel will most likely extend into water-bearing sand/silt. The access shaft walls can be supported by internally-braced, water-tight steel sheet piles.

AEC anticipates ground water control will be required for the tunnel shafts. Possible ground water control measures includes: (i) single- or multiple-stage well points or eductors (for silts and silty sands); and/or (ii) water-tight sheet pile cut-off walls. Generally, the groundwater depth should be lowered at least 5 feet below the excavation bottom in accordance with Section 01 57 23.02 of the latest edition of the SJRA Construction Specifications to be able to work on a firm surface when water-bearing granular soils are encountered. AEC notes that extended and/or excessive dewatering can result in settlement of existing structures, pavement, or utilities in the vicinity. One option to reduce the risk of settlement in these cases includes installing a series of reinjection wells around the perimeter of the construction area. General groundwater control recommendations are presented in Section 6.2 of this report. The options for dewatering 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 dewatering operation.

<u>Sheet Piling:</u> 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. We recommend that the steel sheet piles be driven in pairs. It is important that the sheet pile with the ball end be driven first. If the sheet pile with the socket end is driven first, it may clog with soil and make it difficult to drive the adjacent pile. Regular inspection of sheet pile tops should be performed to assess damage resulting from driving through relatively hard soils. 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) Ground water elevation at the top of the ground surface on the retained side;
(2) Ground water elevation 5 feet below the bottom of the access shaft excavation (assuming dewatering operations as recommended in Section 6.2 of this report) at shaft side;
(3) Neglect cohesion for active pressure determination, Equation (6) in Section 5.2.2 of this report;
(4) The design retained height should extend from the ground surface to the water line tunnel invert depth, plus 1 or 2 feet;
(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 and located at least 15 feet away from the sheet piles;
(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.

Bottom Stability: Recommendations for evaluating tunnel access shaft bottom stability are presented in Section 5.2.2 of this report.

Reaction Walls: Reaction walls (if used) will be part of the tunnel shaft walls; they will be rigid structures and support tunneling 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 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);
- $\gamma$ = wet unit weight of soil (pcf);
- $z$ = depth below ground surface for the point under consideration (ft);
- $K_p$ = coefficient of passive earth pressure for level ground/backfill;
- $c$ = 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 Face Stability during Construction

#### 5.3.2.1 General

The stability of a tunnel face is governed primarily by ground water and subsurface soil conditions, type of tunnel machine used, and workmanship. Based on the subsurface conditions encountered in our borings and the proposed invert depths (see Table 6 in Section 5.3 of this report), we anticipate that soft to very stiff lean/fat clay (CL/CH), and water bearing loose silt/silty sand (ML/SM) will likely encounter within the tunnel zone. Secondary features such as sand or silt partings/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 the tunnel face 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.

5.3.2.2 Anticipated Ground Behavior

A Stability Factor, $N_t = (P_z - P_a)/C_u$ may be used to evaluate the stability of an unsupported bore face in cohesive soils, where $P_z$ is the overburden pressure to the 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, assumed no interior pressure is applied. Generally, $N_t$ values of 4 or less are desirable as it represents a practical limit below which augering may be accomplished without significant difficulty. Higher $N_t$ values usually lead to large deformations of the soil around the 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. The $N_t$ values estimated for the cohesive soils encountered at the siphon tunnel is presented in Table 7.

Note that the cohesive soils have secondary structures such as fissures, sand/silt seams, and sand/silt lenses which can cause the bore face to become unstable. As indicated on Table 6 in Section 5.3 of this report, saturated silty sand/silt layers are encountered within the anticipated tunnel invert depths in the vicinity of Borings B-3 and B-4. Where granular or soft cohesive soils are anticipated to be encountered, the Contractor should make provisions of using microtunnel boring machine (MTBM) with slurry or earth pressure balance system that can balance soil and groundwater pressure to stabilize the tunnel face. The Contractor should not base their bid on the above information alone, since granular or soft cohesive soils may be encountered between boring locations; the Contractor should verify the subsurface conditions between boring locations or add a contingency.

AEC assumes that the maximum amount of allowable ground surface settlement at the tunnel crossing beneath Wallisville Road is very low. If pipe jacking method is to be used, AEC recommends that either a MTBM with pressurized slurry face, or a close chamber (bulkhead) face, i.e. Earth Pressure Balance (EPB) TBM be used along with the tunnel operation. These options can result in the least amount of settlement at the ground surface, since the EPB TBM will make a more precise tunnel cut, and will be able to maintain the groundwater level while cutting (without requiring dewatering), compared to a pipe-jacking with boring shield alone. If MTBM with slurry face or EPB TBM will be not used, dewatering will be required during the tunnel operation.

**AEC has reservations against excavation of the tunnels by hand (and allowing workmen to enter into the tunnels with an unsupported tunnel face),** including: (i) workman safety; (ii) potential buildup of toxic/noxious gases (if any); (iii) hand excavation will be a slower process compared to excavation with a TBM, which will in turn lengthen the period of dewatering, causing consolidation of the soils above the tunnel, will cause additional disturbance, and settlement of the ground surface/roadway; and (iv) digging by hand is less precise and there will be less control over the volume of soil removed compared to a TBM, which can increase the volume of excavated soil approximately 1 to 2 percent, resulting in more settlement.

The estimated maximum settlements ($S_{max}$) caused by volume loss from tunneling are presented in Table 7, using either with (i) a closed face EPB MTBM; or (ii) a pipe jacking with MTBM. The settlement amounts estimated in Table 7 also assume the tunneling contractor practices good workmanship during the tunnel construction.

**Table 7. Tunnel Face Stability Factor and Estimated Settlements at Siphon**

| Boring No. | Tunnel Invert Depth (ft) | Anticipated Soil Types in Tunnel Zone | Stability Factor $N_t$ | $S_{max}$ (in) | Note/Suggestion |
|---|---|---|---|---|---|
| B-1 & B-4 | 22.6 | 14'-16': Firm to very stiff CL, with silty sand seams/partings<br>16'-18': Loose ML, wet<br>18'-23': Stiff to very stiff CL<br>23'-27': Stiff to very stiff CH/CL | 1.6 | EPB MTBM or (Slurry Face): 0.23<br><br>Pipe Jacking with Shield: 0.06 | Mix ground and potential swelling ground. High plasticity clay with flowing silt in middle.<br>Suggest EPB MTBM if dewatering is not performed or Pipe-Jacking with boring shield and dewatering is performed. |
| B-2 & B-3 | 21.6 | 13'-14': soft to very stiff CH, with crawfish holes<br>14'-18': Wet ML/SM or soft to firm CL/CH<br>18'-26': Firm to very stiff CL/CH, with silt seams/partings | 4.5 | EPB MTBM or (Slurry Face): 0.21<br><br>Pipe Jacking with Shield: 0.35 | Mix ground and potential swelling ground. Soft high plasticity fat clay with flowing silt/silty sand in middle.<br>Suggest EPB MTBM if dewatering is not performed or Pipe-Jacking with boring shield and dewatering is performed. |

Note: (1) $S_{max}$ = Estimated settlement along the tunnel alignment due to volume loss if EPB MTBM is used; not including consolidation settlement.
(2) $S_{max}$ = Estimated settlement along the tunnel alignment due to volume loss if Pipe Jacking MTBM with shield is used; not including consolidation settlement and settlement due to dewatering.
(3) CH = Fat Clay, ML = Silt, CL = Lean Clay, SM = Silty Sand.

AEC notes that the estimated settlements presented in Table 7 do not include consolidation settlement or settlement from collapse of voids within the soil around the tunnel. The actual settlement at the tunnel locations during construction could be more than estimated in Table 7. In addition, if pipe jacking method is to be used, dewatering operations in the vicinity of the tunnels will cause additional settlement due to increases in effective stress of the soil strata. **AEC notes that if an EPB MTBM or a slurry face MTBM is not to be used along with the pipe jacking operation, or if the tunneling contractor practices poor workmanship during construction, the amount of settlement could be significantly larger than the amounts estimated in Table 7.**

The information in this report should be reviewed so that appropriate tunneling equipment and operation can be planned and factored into the construction plan and cost estimate. If the estimated settlement is too high, we suggest that the tunnel construction also consider the use of jet grouting to stabilize the saturated granular soils within the tunnel zone (in addition to using an EPB TBM). The choice of tunneling machine should be selected by the Contractor. Plate D-10 in Appendix D provides a general guideline for TBM selection. Pipe jacking operations should comply with Section 33 05 23.19 of the latest edition of the SJRA Construction Specifications.

5.3.2.3 Influence of Tunneling on Existing Structures

We estimated the resulting influence zones (extending from the centerline of the tunnel, see Plate D-9, in Appendix D) to range from approximately 20 to 21.5 feet based on our borings; although the values of tunnel influence zone presented are rough estimates. The estimated maximum settlements [caused by volume loss if a TBM is used] along the tunnel 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 is difficult to determine because several factors influence the response of the soil to tunneling operations including type of soil, ground water level and control method, type of tunneling equipment, tunneling 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 Pipe Jacking

Considering the ground conditions discussed in Table 7 in Section 5.3.2.2 of this report, AEC recommends that a EPB MTBM or a slurry face MTBM be used and keep the pressure at least equal to but not greater than the combined soil and groundwater pressure in the ground at the tunnel level. 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 to influence existing foundations or structures, we recommend that the outer edge of the influence zone of the tunnel 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 cannot be located farther than the minimum distance recommended above;
- tunneling 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;

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

Disturbance and loss of ground from the tunneling 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 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 Intake/Discharge Structures

Based on the Plan and Profile drawing provided by TWE, the footings of intake and discharge structures will bear at an elevation of 28.0 feet above MSL.

Design of the 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 Headwall/Wing Wall Foundations

Allowable Bearing Capacity: Based on our borings, headwall/wing wall footings bearing at a depth of approximately 8 to 9 feet below grade (i.e. at an elevation of approximately 28.0 feet above MSL), can be designed for an allowable net bearing capacity of 1,200 pounds per square foot (psf) for sustained loads and 1,800 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.

Headwall Footing Construction: Based on Borings B-1 through B-4, AEC anticipates that the headwall footings be supported on top of firm to very stiff lean clay/fat clay (CL/CH) layer at an elevation of approximately +28 feet MSL; however, weak layers such as soft fat clay that consists of abundant silt/sand seams or crawfish holes, and loose silt/silty sand (ML/SM) are encountered at depths of 10 to 16 (i.e. elevation of approximately +27 to +19 feel MSL) in Borings B-3 and B-4. Although these weak layers encountered below the proposed headwall foundations in the borings, if the silt/sand layer presents at the bottom of the headwall footing excavation during construction, AEC recommends that a minimum of 18 inches of subgrade soils be replaced with compacted gravel (wrapped with geofabric filter) or cement stabilized sand (CSS). CSS should be in accordance with Section 31 32 13.16 of the latest edition of the SJRA Construction Specifications.

Foundation construction and excavation should be protected by adequate shoring. Recommendations for excavation stability and interlocked sheet piles are presented in Sections 5.2.2 and 5.3.1 of this report, respectively. Dewatering guidelines are presented in Section 6.2 of this report. AEC also recommends that scour protection (such as riprap) be provided around the footings of headwalls and wingwalls.

Foundation Settlement: A detailed settlement analysis is beyond the scope of this investigation. Based on the soil conditions encountered, we estimate that the headwall and wingwall footings, designed and constructed as recommended in this report, will experience total settlements on the order of 1 inch.

### 5.4.2 Hydrostatic Uplift

For hydrostatic uplift, AEC recommends the structure design consider a most critical 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. Wall design should consider short-term and long-term conditions, whichever is critical. Design soil parameters are presented on Plate C-1, in Appendix C. Recommended design criteria for uplift resistance are shown on Plate D-8, in Appendix D.

### 5.4.3 Lateral Earth Pressures

The magnitudes of the lateral earth pressures on the headwalls/wing walls will depend on the type and density and configuration of the retained soils (or backfill), surcharge on the retained soils, and hydrostatic pressure, if any. If over-compacted or highly plastic clays are placed behind the walls, the lateral earth pressure could exceed the vertical pressure. Lateral pressure resulting from construction equipment, pavement and traffic, or other surcharge on the top of the wall should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure. AEC recommends that the followings be considered for in the headwalls/wing walls design:

1) If the existing highly expansive soils behind the wall within the active zone are not to be replaced (See “Replacement of Existing Backfill Behind Wall” in Section 5.4.4), a uniformly distributed swell pressure of at least 4,500 psf should be used in the design.
2) If wall drainage is not provided (See “Drainage System” in Section 5.4.5), hydrostatic pressure at the 100 year flood elevation or the top of the levee, whichever is more critical, should be considered behind the wall in the design.
3) We recommend that at a minimum 250 psf of traffic surcharge be considered for design of the wall.

Wall design should consider short-term and long-term conditions, whichever is critical. Based on the drawings provided by TWE, AEC anticipates that the concrete headwalls and wingwalls of the intake and discharge structures will not be allowed to move, and should be designed based on at-rest earth pressures.

At-Rest Earth Pressure: The at-rest earth pressure at depth z can be determined by Equation (8). Design soil parameters are presented on Plate C-1, in Appendix C.

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

where,
- $p_0$ = at-rest earth pressure, psf.
- $q_s$ = uniform surcharge pressure, psf.
- $\gamma, \gamma'$ = 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 for level backfill, see Plate C-1, in Appendix C.
- $\gamma_w$ = unit weight of water, 62.4 pcf.
- $p_s$ = swell pressure, at least 4,500 psf; if drainage is to be provided behind the wall and the existing backfill is to be replaced according to Sections 5.4.4 and 5.4.5 of this report, swell pressure can be omitted in the design.
- h = depth from the top of wall to the bottom of the wall, feet.

Active Earth Pressure: If the wingwall is allowed to move, the wall should be designed based on active earth pressures. The active earth pressure at depth z can be determined by Equation (9). Design soil parameters for the lateral earth pressure design are presented on Plate C-1, in Appendix C. The design of the wall should consider both short and long-term conditions, whichever condition is critical should be used for design.

$$p_a = (q_s + \gamma h_1 + \gamma' h_2)K_a - 2c(K_a)^{1/2} + \gamma_w h_2 + p_s h \qquad \text{.........Equation (9)}$$

where,
- $p_a$ = active earth pressure, psf.
- $K_a$ = coefficient of active earth pressure for level backfill, see Plates C-1, in Appendix C.
- c = cohesion of clayey soils (can be conservatively neglected for design), see Plate C-1, in Appendix C.
- $q_s$ = uniform surcharge pressure, psf.
- $\gamma, \gamma'$ = wet and buoyant unit weights of soil, pcf, see Plate C-1, in Appendix C.
- $h_1$ = depth from ground surface to groundwater table, feet.
- $h_2$ = z-$h_1$, depth from groundwater table to point under consideration, feet.
- z = depth below ground surface, feet.
- $\gamma_w$ = unit weight of water, 62.4 pcf.
- $p_s$ = swell pressure, at least 4,500 psf; if drainage is to be provided behind the wall and the existing backfill is to be replaced according to Section 5.4.4 of this report, swell pressure can be omitted in the design.
- h = depth (final grade) from the top of RWs to the bottom of the wall, feet.

Sliding Resistance: The sliding resistance of wall footings can be determined by the summation of the friction resistance between the wall footing and the underlying soil, the adhesion resistance between the wall footing and underlying soil, and passive earth pressure resistance in front of the wall (e.g. if a key is installed below the wall footing). The sliding resistance of the walls can be determined by Equation (10). Headwall/wing wall design should consider both short-term and long-term conditions; whichever condition is more critical should be used for design. The design soil parameters for headwall/wing wall design are presented on Plate C-1, in Appendix C.

$$\Sigma F_r = (\Sigma V)\tan(\delta) + B\,(C_\alpha) + P_p \qquad \text{...........Equation (10)}$$

where, $\Sigma F_r$ = sum of horizontal resistance forces, plf.
$\Sigma V$ = sum of vertical forces, including wall weight and soil weight above footing, plf.
$\delta$ = angle of friction between soil and footing; can be taken as 2/3 $\phi$.
$\phi$ = soil angle of internal friction, see Plate C-1, in Appendix C.
$B$ = width of wall footing, feet.
$C_\alpha$ = adhesion between soil and footing, psf, see Plate C-1, in Appendix C.
$P_p$ = passive earth pressure resistance, psf; see Equation (7), should be neglected conservatively.

<u>Passive Earth Pressure Resistance:</u> Due to limited backfill in front of the wall footing as well as the surface clays are prone to desiccation and can be eroded away, AEC recommends that passive earth pressure resistance be omitted for design. If passive earth pressure is to be considered in the wall design, it can be calculated using Equation (7) in Section 5.3.1 of this report. We recommend that a FS of 2.0 be applied to the passive earth pressure resistance if it is to be used in the design. Headwall/wing wall design should consider both short-term and long-term conditions; whichever condition is more critical should be used for design. The design soil parameters for headwall/wing wall design are presented on Plate C-1, in Appendix C.

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 wall. The soil conditions should be checked by geotechnical personnel to confirm the recommended soil parameters.

#### 5.4.4 Soil Replacement Behind Headwall/Wing Wall

Note that the top 10 feet of existing levee soils encountered in our borings consist of high to very high PI clays which are vulnerable to moisture change; they can swell significantly when wet and resulting in excessive swell pressure on wall (upto 4,500 psf based on our existing data on similar clayey soils from SJRA projects), which can have adverse impact on headwall/wing wall stability. If right of way allows, AEC recommends that the design engineer to consider that the existing high plasticity clayey soils behind the wall within the active zone be entirely removed and replaced with compacted select fill, and provide a 1-foot wide sand chimney (or equivalent prefabricated drainage element) behind the wall (See Section 5.4.5 of this report). The use of compacted select fill should slope upward from the footing heel with a slope of H:V = 1:1 (i.e. 45 degree with vertical). The select fill should be placed in loose lifts not exceeding 8 inches in thickness. Fill within 3 feet of columns or walls should be placed in loose lifts no more than 4-inch thick and compacted using hand tampers, or small self-propelled compactors. The 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. Compacted select fill requirements are presented in Section 5.6 of this report.

**Note that if the existing high plasticity clay backfill behind the headwalls/wing walls within the active zone cannot be entirely removed and replaced with compacted select fill due to the space limit, a lateral swelling pressure of 4,500 psf should still be applied to the area located inside the active zone where the high plasticity clay cannot be replaced.**

#### 5.4.5 Drainage System

We recommend the use of a drainage system behind the headwall/wing walls to prevent the buildup of excessive hydrostatic pressures on the wall faces. A 1-foot wide, vertical “bank sand” chimney wrapped with a geofabric filter (or equivalent prefabricated drainage system) should be placed behind the headwall/wing walls. Bank sand can be composed of SP, SW, or 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 sand chimney should be connected to a perforated drainage pipes connecting to weep holes. The top of the chimney should be covered by at least 24 inches of compacted impervious clay liner to prevent the infiltration of surface water/runoff into the drainage system. If weep holes are to be used, we recommend using at least 3-inch diameter weep holes 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.

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5.4.6 Riprap

AEC recommends that at least 24-inch thick HCFCD Grade 2 riprap be provided around the headwall/wing wall footings; ideally the rip rap toe should be trenched to below the depth of potential for riprap degradation and contraction scour to mitigate the potential for undermining of the riprap toe due to wave action or flow at the siphon intake/discharge structure areas. The riprap should be also underlain by a geotextile filter fabric to prevent the underlying soils from moving into or through the riprap.

Filter Blanket: A suitable filter may consist of a well-graded gravel or sand-gravel layer or a synthetic filter fabric manufactured for this purpose. A well-graded gravel or sand-gravel layer filter blanket should be at least 6-inches thick, which should have the following relationship for a stable design:

$d_{15\ filter}/d_{85\ base} \leq 5$ ............(Equation 11)

$5 < d_{15\ filter}/d_{50\ base} \leq 40$ ............(Equation 12)

and

$d_{50\ filter}/d_{50\ base} \leq 40$ ............(Equation 13)

where, $d_{15\ filter}$ = the diameter of soil particle corresponding to percent finer than 15% of the filter material;
$d_{85\ base}$ = the diameter of soil particle corresponding to percent finer than 85% of the base material;
$d_{50\ filter}$ = the diameter of soil particle corresponding to percent finer than 50% of the filter material; and
$d_{50\ base}$ = the diameter of soil particle corresponding to percent finer than 50% of the base material.

Filter refers to the overlying material while base refers to the underlying material. These relationships must hold between the base and filter and the filter and riprap to prevent migration of fines.

Synthetic Filter Fabric: If a geotextile filter fabric is to be used, 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. Placement of geotextile should be in accordance with

Section 31 38 25 of the SJRA Standard Specifications or Section 02379 of the HCFCD 2005 Standard Specifications Book, whichever is more stringent.

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 Section 4.1.2.6, "Filter Selection" of the 2009 HCFCD "Design, Installation & Maintenance Manual for Gabion Structures" (Final Draft). The non-woven filter fabric shall meet HCFCD Class 1 requirements, presented on Table 8 below.

**Table 8. 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 SJRA Standard Specifications or Section 02378 of the HCFCD 2005 Standard Specifications Book, whichever is more stringent. 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.

### 5.5 Demolition of Existing Siphon

AEC understands that the Client plans to demolish and remove the portion of the existing 48-inch diameter siphon located outside the roadway ROW and the associated innet/outlet structures after installing the new siphons.

Demolition and Backfill: Recommendations for open cut stability are presented in Section 5.2.2 of this report. The existing siphon and associated structures should be demolished and the concrete debris should be entirely removed. After removal, the trench should then be backfilled with CSS in accordance with Section 31 32 13.16 of the latest edition of the SJRA Construction Specifications.

Plug and Abandon In Place: For the portion of the existing siphon that will be abandoned in place, AEC recommends that flowable fill be used to backfill the existing siphon pipes. Flowable fill should be in general accordance with Section 02322 of the 2017 City of Houston Standard Construction Specification (COHSCS), or the corresponding SJRA Construction Specifications, whichever is more stringent.

### 5.6 Select Fill

AEC recommends that the fill material to be used for backfill or for replacement of existing material behind the intake/discharge structures meets the requirements of Section 02314, Item 2.1 “Imported Select Fill Material” of the 2005 HCFCD Standard Specifications or equivalent local standard. The select fill should be free from roots, trash, organic matter, and other objectionable materials. The select fill should be non-dispersive sandy lean clay or lean clay, with a maximum LL of 49, a PI between 15 and 30, and between 60 and 85 percent of the material passing a No. 200 sieve. The select fill should be placed in loose lifts not exceeding 8 inches in thickness. Heavy compaction equipment and excessive equipment passes should be avoided within 3 feet of the siphon or other adjacent structures. Backfill within 3 feet of structures should be placed in loose lifts no more than 4 inches thick and compacted using portable compaction equipment or hand tampers. The select fill should be compacted to a minimum of 95 percent of its ASTM D 698 (Standard Proctor) maximum dry unit weight, at a moisture content within optimum and 3 percent above optimum.

Prior to construction, the Contractor should determine if he or she can obtain qualified select fill meeting the above select fill criteria. All material intended for use as select fill should be tested prior to use to confirm that it meets select fill criteria.

## 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 or her 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 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. In accordance with Section 01 57 23.02 of the latest edition of the SJRA Construction Specifications, the groundwater depth should be lowered at least 5 feet below the excavation bottom to be able to work on a firm surface when water-bearing granular soils are encountered. 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

Excavation, bedding, and backfilling of underground utilities should be monitored by qualified geotechnical professionals to check for compliance with project documents and changed conditions, if 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 this alignment or similar structures located elsewhere, without additional evaluation and/or investigation.

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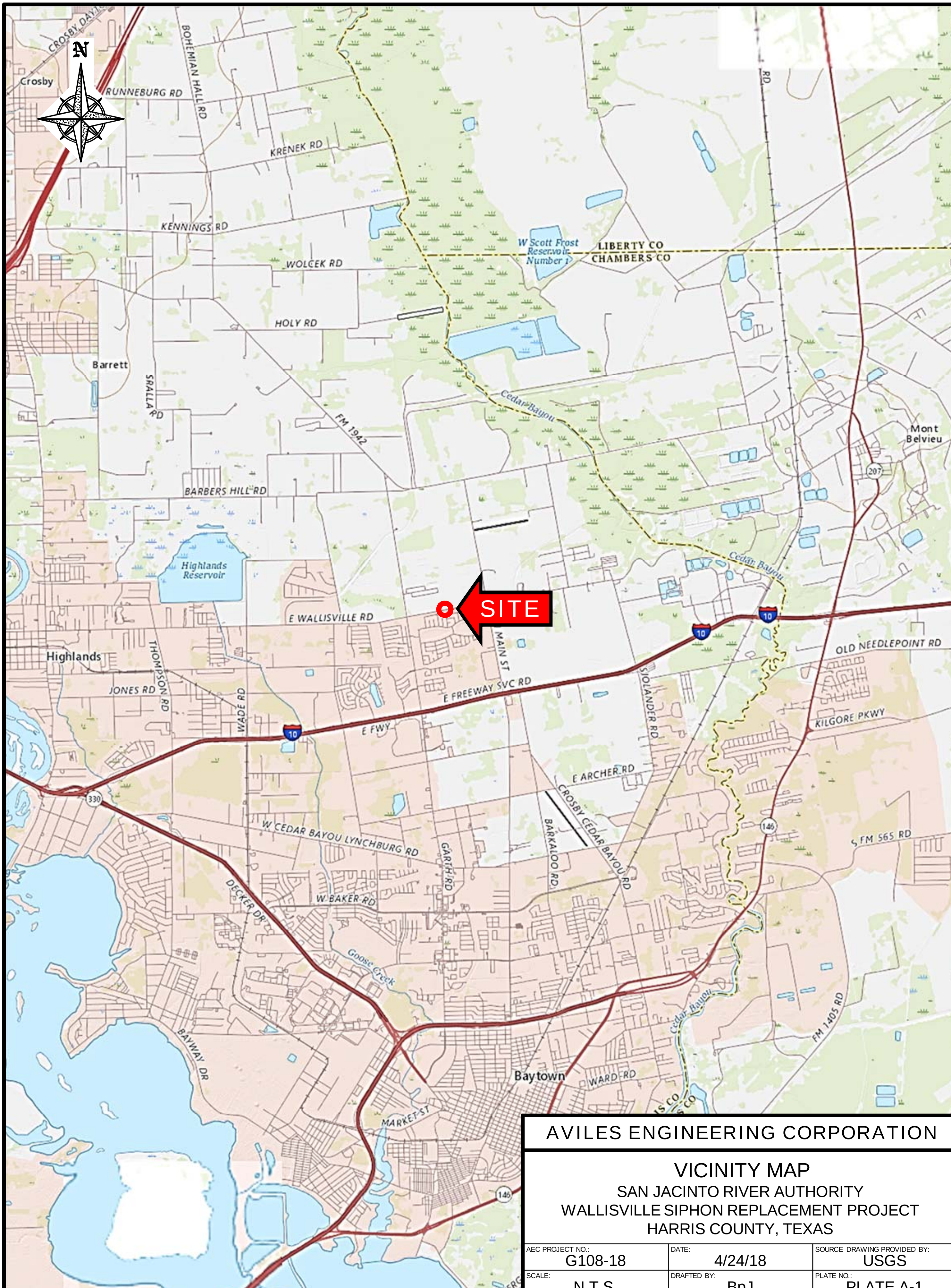
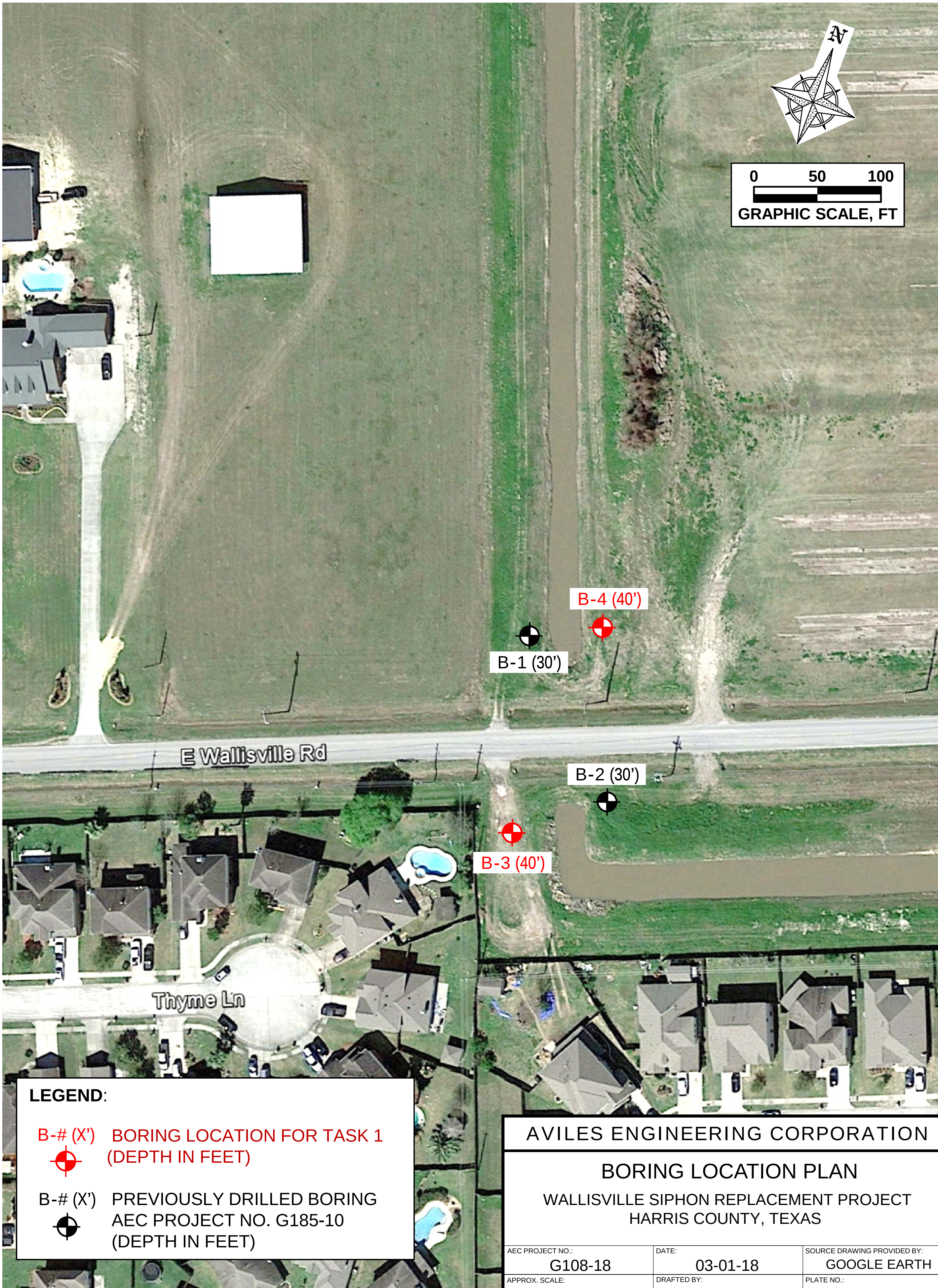
# APPENDIX A




AVILES ENGINEERING CORPORATION

VICINITY MAP
SAN JACINTO RIVER AUTHORITY
WALLISVILLE SIPHON REPLACEMENT PROJECT
HARRIS COUNTY, TEXAS

| AEC PROJECT NO.: | DATE: | SOURCE DRAWING PROVIDED BY: |
|---|---|---|
| G108-18 | 4/24/18 | USGS |
| SCALE: N.T.S. | DRAFTED BY: BpJ | PLATE NO.: PLATE A-1 |




**LEGEND**:

B-# (X') BORING LOCATION FOR TASK 1 (DEPTH IN FEET)

B-# (X') PREVIOUSLY DRILLED BORING AEC PROJECT NO. G185-10 (DEPTH IN FEET)

AVILES ENGINEERING CORPORATION

# BORING LOCATION PLAN

WALLISVILLE SIPHON REPLACEMENT PROJECT
HARRIS COUNTY, TEXAS

| AEC PROJECT NO.: | DATE: | SOURCE DRAWING PROVIDED BY: |
|---|---|---|
| G108-18 | 03-01-18 | GOOGLE EARTH |
| APPROX. SCALE: | DRAFTED BY: | PLATE NO.: |
| 1" = 100' | BpJ | PLATE A-2 |

PROJECT: SJRA E Wallisville Rd Siphon

**AVILES ENGINEERING CORP.** GEOTECHNICAL ENGINEERS

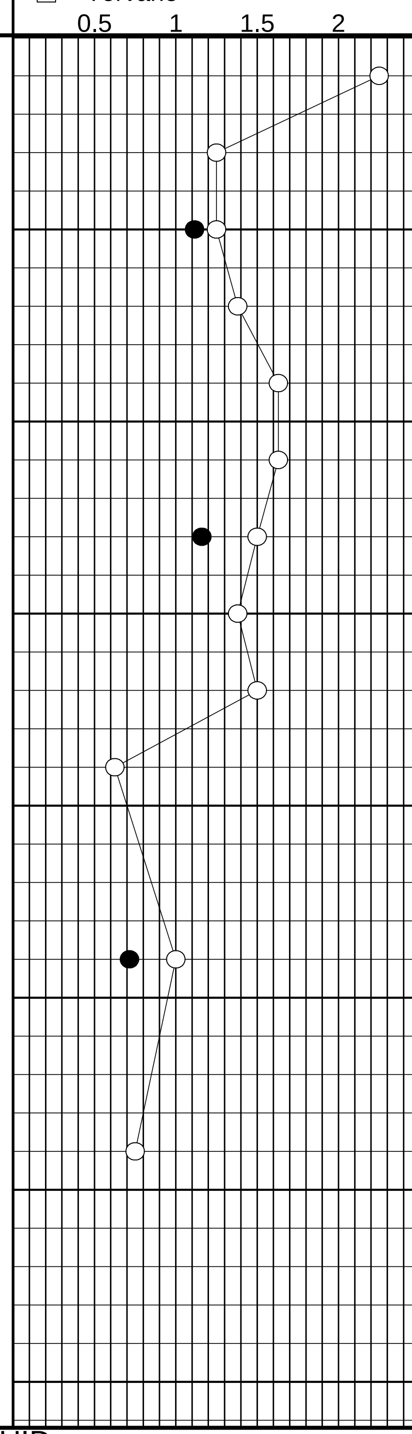
BORING B-1

DATE 12/13/10 TYPE 4" Dry Auger / Wet Rotary LOCATION See Boring Location Plan

| DEPTH IN FEET | DESCRIPTION | S.P.T. BLOWS / FT. | MOISTURE CONTENT, % | DRY DENSITY, PCF | -200 MESH | LIQUID LIMIT | PLASTIC LIMIT | PLASTICITY INDEX |
|---|---|---|---|---|---|---|---|---|
| 0 | Very stiff to hard, dark gray and tan Sandy Lean Clay (CL), with sand seams and roots 0'-2', and calcareous nodules | | 17 | | | | | |
| | -gray and tan 2'-8' | | 20 | | | 44 | 15 | 29 |
| 5 | | | 20 | 106 | | | | |
| | | | 18 | | | | | |
| | Very stiff, tan and light gray Fat Clay w/Sand (CH), with calcareous nodules | | 21 | | | | | |
| 10 | | | 20 | | | 51 | 15 | 36 |
| | Very stiff, tan and light gray Lean Clay (CL), with silt partings, and calcareous nodules (abundant 12'-14') | | 24 | 103 | | | | |
| 15 | -tan 14'-16' | | 26 | | | | | |
| | | | 20 | | | | | |
| | Stiff, tan Lean Clay (CL), with silt partings | | 27 | | 94 | 32 | 20 | 12 |
| 20 | | | | | | | | |
| | | | 24 | 99 | | 42 | 16 | 26 |
| 25 | | | | | | | | |
| | -tan and light gray 28'-30' | | 24 | | | | | |
| 30 | Termination depth = 30 feet. | | | | | | | |
| 35 | | | | | | | | |

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane




BORING DRILLED TO 18 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 18 FEET WHILE DRILLING
WATER LEVEL AT 10.5 FEET AFTER 24 HRS.
DRILLED BY V&S DRAFTED BY ACO LOGGED BY V&S

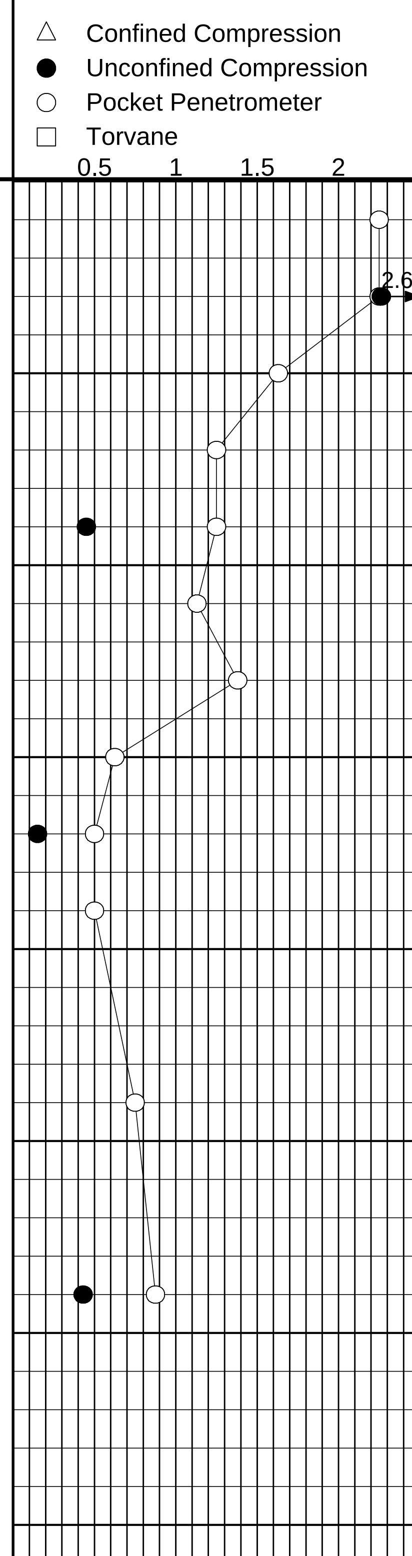
AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: SJRA E Wallisville Rd Siphon BORING B-2

DATE 12/13/10 TYPE 4" Dry Auger / Wet Rotary LOCATION See Boring Location Plan

| DEPTH IN FEET | DESCRIPTION | S.P.T. BLOWS / FT. | MOISTURE CONTENT, % | DRY DENSITY, PCF | -200 MESH | LIQUID LIMIT | PLASTIC LIMIT | PLASTICITY INDEX |
|---|---|---|---|---|---|---|---|---|
| 0 | Very stiff to hard, tan Sandy Lean Clay (CL), with roots and calcareous nodules 0'-2' | | 19 | | | | | |
| | -dark gray 2'-4' | | 18 | 109 | | | | |
| 5 | -dark gray and tan, with numerous calcareous nodules 4'-6' | | 21 | | | | | |
| | Firm to very stiff, tan and light gray Fat Clay w/Sand (CH), with silt partings and calcareous nodules | | 23 | | | 54 | 15 | 39 |
| | -with roots and numerous calcareous nodules 8'-10' | | 26 | 96 | | | | |
| 10 | Very stiff, tan and light gray Fat Clay (CH), with calcareous nodules 10'-12' | | 29 | | | | | |
| | | | 28 | | | | | |
| 15 | Stiff, tan and light gray Lean Clay (CL), with silt partings | | 26 | | | 36 | 19 | 17 |
| | Soft to firm, tan Fat Clay (CH) | | 32 | 85 | | | | |
| 20 | Firm, tan and light gray Lean Clay (CL), with silt seams and partings | | 28 | | | | | |
| 25 | Stiff, tan and light gray Fat Clay w/Sand (CH), with silt partings | | 27 | | | 52 | 17 | 35 |
| | Firm to stiff, tan and light gray Lean Clay (CL), with silt partings and silty sand seams | | 24 | 98 | | | | |
| 30 | Termination depth = 30 feet. | | | | | | | |
| 35 | | | | | | | | |

SHEAR STRENGTH, TSF




BORING DRILLED TO 18 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 18 FEET WHILE DRILLING

WATER LEVEL AT 9.3 FEET AFTER 24 HRS.

DRILLED BY V&S DRAFTED BY ACO LOGGED BY V&S

**AVILES ENGINEERING CORP.**
GEOTECHNICAL ENGINEERS

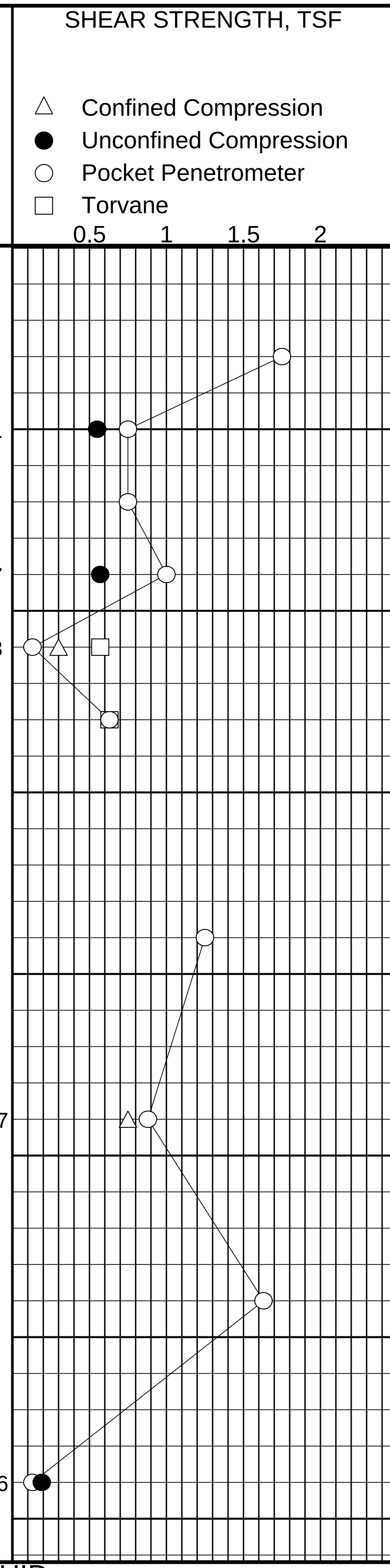
PROJECT: SJRA Wallisville Siphon Replacement Project    BORING B-3

DATE 02/16/18    TYPE 4" Dry Auger/Wet Rotary    LOCATION See Boring Location Plan

Approximate Surface Elevation (feet): 36

| DEPTH IN FEET | DESCRIPTION | S.P.T. BLOWS / FT. | MOISTURE CONTENT, % | DRY DENSITY, PCF | -200 MESH | LIQUID LIMIT | PLASTIC LIMIT | PLASTICITY INDEX |
|---|---|---|---|---|---|---|---|---|
| 0 | Very stiff, dark grayish brown Lean Clay (CL) | | 15 | | | | | |
| | -with lean clay pockets, silty sand pockets, and ferrous stains 2'-4' | | 19 | | 93 | 48 | 20 | 28 |
| 5 | Soft to very stiff, dark tannish gray Fat Clay (CH), with ferrous nodules | | 27 | 96.1 | | | | |
| | -with lean clay seams and crawfish holes 6'-8'<br>-tan and gray, with calcareous nodules 6'-10' | | 21 | | 89 | 61 | 18 | 43 |
| | | | 27 | 97.7 | | | | |
| 10 | -tan and gray, with lean clay seams 10'-14', and crawfish holes 10'-12' | | 43 | 80.3 | 97 | | | |
| | | | 26 | | | | | |
| 15 | Gray and tan Sandy Silt (ML), with clayey sand pockets, wet<br>-boring cave-in at 14.4' during drilling | | 22 | | 54 | | | |
| | Tan Silty Sand (SM), wet | | 29 | | | | | |
| | Stiff to very stiff, tan Fat Clay (CH)<br>-with silt partings 18'-20' | | 26 | | | 69 | 22 | 47 |
| 20 | | | | | | | | |
| | -tan and gray, with ferrous stains, and lean clay pockets 23'-25' | | 25 | 103.7 | | | | |
| 25 | | | | | | | | |
| | Very stiff, tan and gray Lean Clay with Sand (CL), with silty sand pockets, partings, and laminations | | 27 | | | 44 | 19 | 25 |
| 30 | | | | | | | | |
| | Tan Clayey Sand (SC), wet<br>-with silty sand layer 34.5'-35' | | 24 | 103.6 | 43 | | | |
| 35 | | | | | | | | |

SHEAR STRENGTH, TSF




BORING DRILLED TO 18 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 16 FEET WHILE DRILLING

WATER LEVEL AT 2.7 FEET AFTER 02/23/18

DRILLED BY GDT    DRAFTED BY JG    LOGGED BY BPJ

PROJECT NO. G108-18

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: SJRA Wallisville Siphon Replacement Project

BORING B-3

DATE 02/16/18 TYPE 4" Dry Auger/Wet Rotary LOCATION See Boring Location Plan

| DEPTH IN FEET | SYMBOL | SAMPLE INTERVAL | DESCRIPTION | S.P.T. BLOWS / FT. | MOISTURE CONTENT, % | DRY DENSITY, PCF | SHEAR STRENGTH, TSF (△ Confined Compression; ● Unconfined Compression; ○ Pocket Penetrometer; □ Torvane) 0.5 1 1.5 2 | -200 MESH | LIQUID LIMIT | PLASTIC LIMIT | PLASTICITY INDEX |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Clayey Sand (cont...) | | | | | | | | |
| | | | Medium dense, tan and gray Silty Clayey Sand (SC-SM), wet | 12 | 28 | | | | | | |
| 40 | | | Termination Depth = 40 feet | | | | | | | | |
| 45 | | | | | | | | | | | |
| 50 | | | | | | | | | | | |
| 55 | | | | | | | | | | | |
| 60 | | | | | | | | | | | |
| 65 | | | | | | | | | | | |
| 70 | | | | | | | | | | | |

BORING DRILLED TO **18** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **16** FEET WHILE DRILLING
WATER LEVEL AT **2.7** FEET AFTER **02/23/18**
DRILLED BY **GDT** DRAFTED BY **JG** LOGGED BY **BPJ**

ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

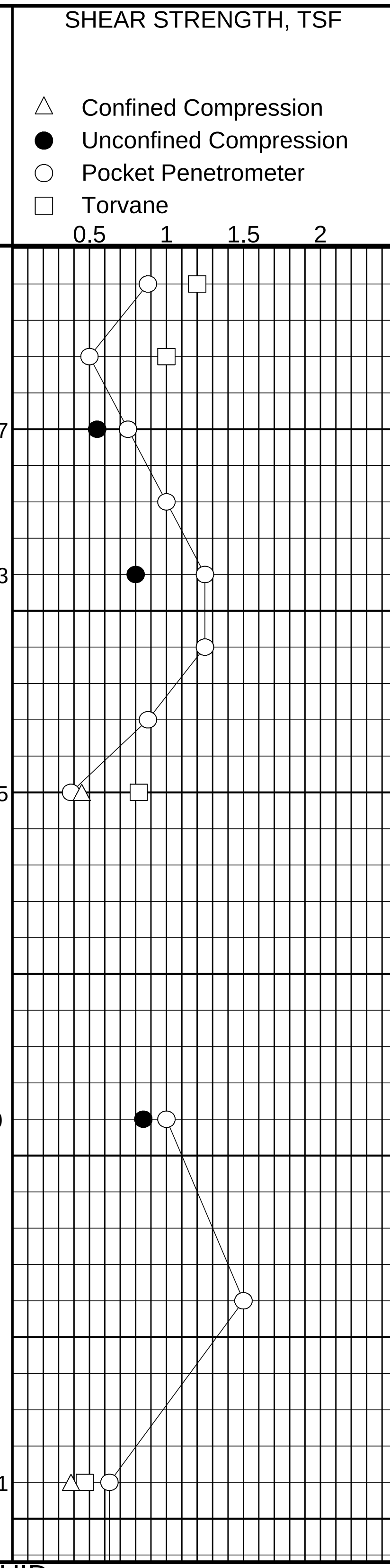
PROJECT: SJRA Wallisville Siphon Replacement Project

BORING B-4

DATE 02/20/18 TYPE 4" Dry Auger/Wet Rotary LOCATION See Boring Location Plan

Approximate Surface Elevation (feet): 37

| DEPTH IN FEET | DESCRIPTION | S.P.T. BLOWS / FT. | MOISTURE CONTENT, % | DRY DENSITY, PCF | -200 MESH | LIQUID LIMIT | PLASTIC LIMIT | PLASTICITY INDEX |
|---|---|---|---|---|---|---|---|---|
| 0 | Fill: firm to very stiff, olive green and tan Lean Clay (CL)<br>-with shell fragments 0'-2'<br>-with ferrous stains 2'-4' | | 21 | | | | | |
| | | | | | | 41 | 18 | 23 |
| | | | 20 | | | | | |
| 5 | Stiff to very stiff, gray and tan Lean Clay (CL), with calcareous nodules<br>-with roots and ferrous stains 4'-6' | | 22 | 102.7 | | | | |
| | | | 21 | | | | | |
| | Stiff to very stiff, tan and gray Fat Clay (CH), with abundant calcareous nodules and ferrous stains | | | | | 51 | 17 | 34 |
| | | | 18 | 114.3 | | | | |
| 10 | Firm to very stiff, tan and gray Lean Clay (CL), with calcareous nodules<br>-with ferrous nodules 10'-14'<br>-with fat clay pockets 12'-16'<br>-boring cave-in at 14' during drilling<br>-with silty sand seams 14'-16' | | 24 | | | | | |
| | | | | | 97 | 45 | 19 | 26 |
| | | | 22 | | | | | |
| 15 | | | 24 | 103.5 | | | | |
| | Loose, tan and gray Silt (ML), with fat clay pockets, wet | | | | 91 | | | |
| | | 10 | 27 | | | | | |
| | Very stiff, tan Lean Clay with Sand (CL), with silty sand and fat clay pockets | | | | 76 | 27 | 18 | 9 |
| | | 28 | 25 | | | | | |
| 20 | | | | | | | | |
| | Stiff to very stiff, tan and gray Fat Clay (CH), with slickensides<br>-with silty clay seams 23'-25' | | 29 | 97.0 | | | | |
| 25 | -with calcareous nodules 28'-30' | | 25 | | | | | |
| 30 | | | | | | | | |
| | Firm to stiff, tan and gray Lean Clay with Sand (CL), with silty sand and silty clay pockets | | | | 75 | 28 | 19 | 9 |
| 35 | | | 23 | 105.1 | | | | |




BORING DRILLED TO 18 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 17 FEET WHILE DRILLING

WATER LEVEL AT 3.7 FEET AFTER 02/23/18

DRILLED BY GDT/V&S DRAFTED BY JG LOGGED BY BPJ

**PROJECT NO. G108-18**

**PLATE A-6**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: SJRA Wallisville Siphon Replacement Project

BORING B-4

DATE 02/20/18 TYPE 4" Dry Auger/Wet Rotary LOCATION See Boring Location Plan

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

| DEPTH IN FEET | SYMBOL | SAMPLE INTERVAL | DESCRIPTION | S.P.T. BLOWS / FT. | MOISTURE CONTENT, % | DRY DENSITY, PCF | SHEAR STRENGTH, TSF | -200 MESH | LIQUID LIMIT | PLASTIC LIMIT | PLASTICITY INDEX |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Tan Clayey Sand (SC), with abundant silt and sand partings, wet | | 25 | 103.9 | | | | | |
| 40 | | | Termination Depth = 40 feet | | | | | | | | |
| 45 | | | | | | | | | | | |
| 50 | | | | | | | | | | | |
| 55 | | | | | | | | | | | |
| 60 | | | | | | | | | | | |
| 65 | | | | | | | | | | | |
| 70 | | | | | | | | | | | |

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

WATER ENCOUNTERED AT **17** FEET WHILE DRILLING

WATER LEVEL AT **3.7** FEET AFTER **02/23/18**

DRILLED BY **GDT/V&S** DRAFTED BY **JG** LOGGED BY **BPJ**

**PROJECT NO. G108-18**

# KEY TO SYMBOLS

| Symbol | Description |
|---|---|
| **Strata symbols** | |
| | Low plasticity clay |
| | High plasticity clay |
| | Silt |
| | Silty sand |
| | Clayey sand |
| | Poorly graded clayey silty sand |
| | Fill |
| **Misc. Symbols** | |
| | Water table depth during drilling |
| | Subsequent water table depth |
| | Torvane |
| | Pocket Penetrometer |
| | Unconfined Compression |
| | Confined Compression |
| **Soil Samplers** | |
| | Auger |

| Symbol | Description |
|---|---|
| | Undisturbed thin wall Shelby tube |
| | Standard penetration test |

ENGINEERING CORP.

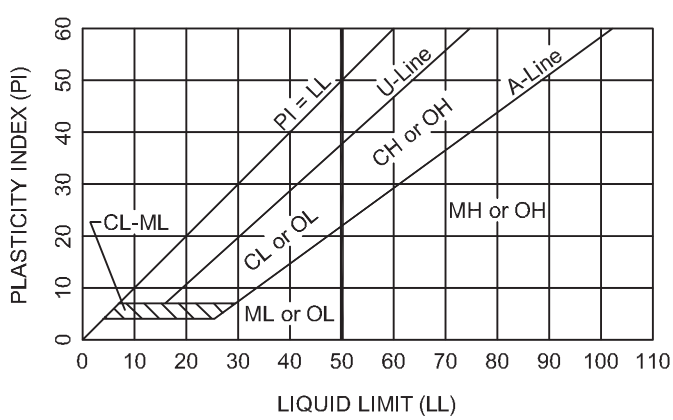
# CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

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



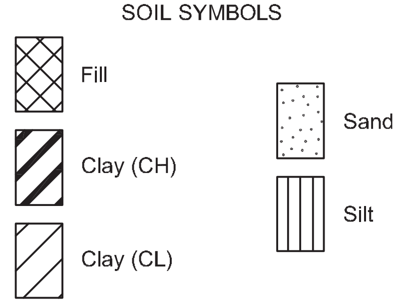

Equation of A-Line: Horizontal at PI=4 to LL=25.5, then PI=0.73(LL-20)

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



AVILES ENGINEERING CORP.

# 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. |
| 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. |

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## ASTM & TXDOT DESIGNATION FOR SOIL LABORATORY TESTS

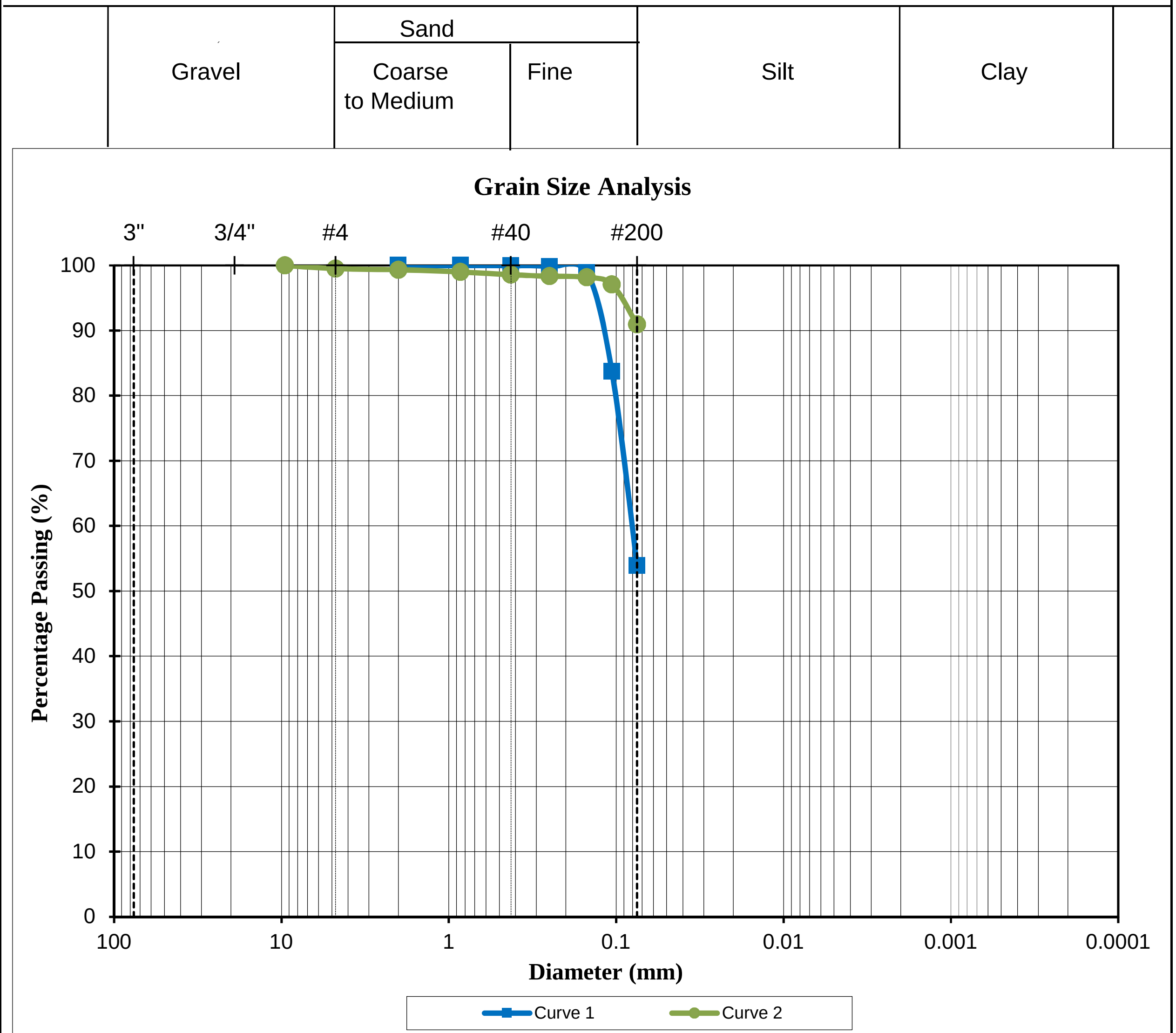
| SOIL TEST | ASTM TEST DESIGNATION | TXDOT TEST DESIGNATION |
|---|---|---|
| Unified Soil Classification System | D 2487 | Tex-142-E |
| Moisture Content | D 2216 | Tex-103-E |
| Specific Gravity | D 854 | Tex-108-E |
| Sieve Analysis | D 6913 | Tex-110-E (Part 1) |
| Hydrometer Analysis | D 7928 | 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 :** SJRA Wallisville Siphon Replacement Project **Job No.:** G108-18
**Location of Project:** Harris County, Texas **Date of Testing:** 2/19/2018




| Curve | Boring | Depth (ft) | Soil Description | D50 (mm) | Cu | Cc |
|---|---|---|---|---|---|---|
| 1 | B-3 | 14-16 | Sandy Silty Clay (CL-ML) | N/A | N/A | N/A |
| 2 | B-4 | 16-18 | Silt (ML) | N/A | N/A | N/A |

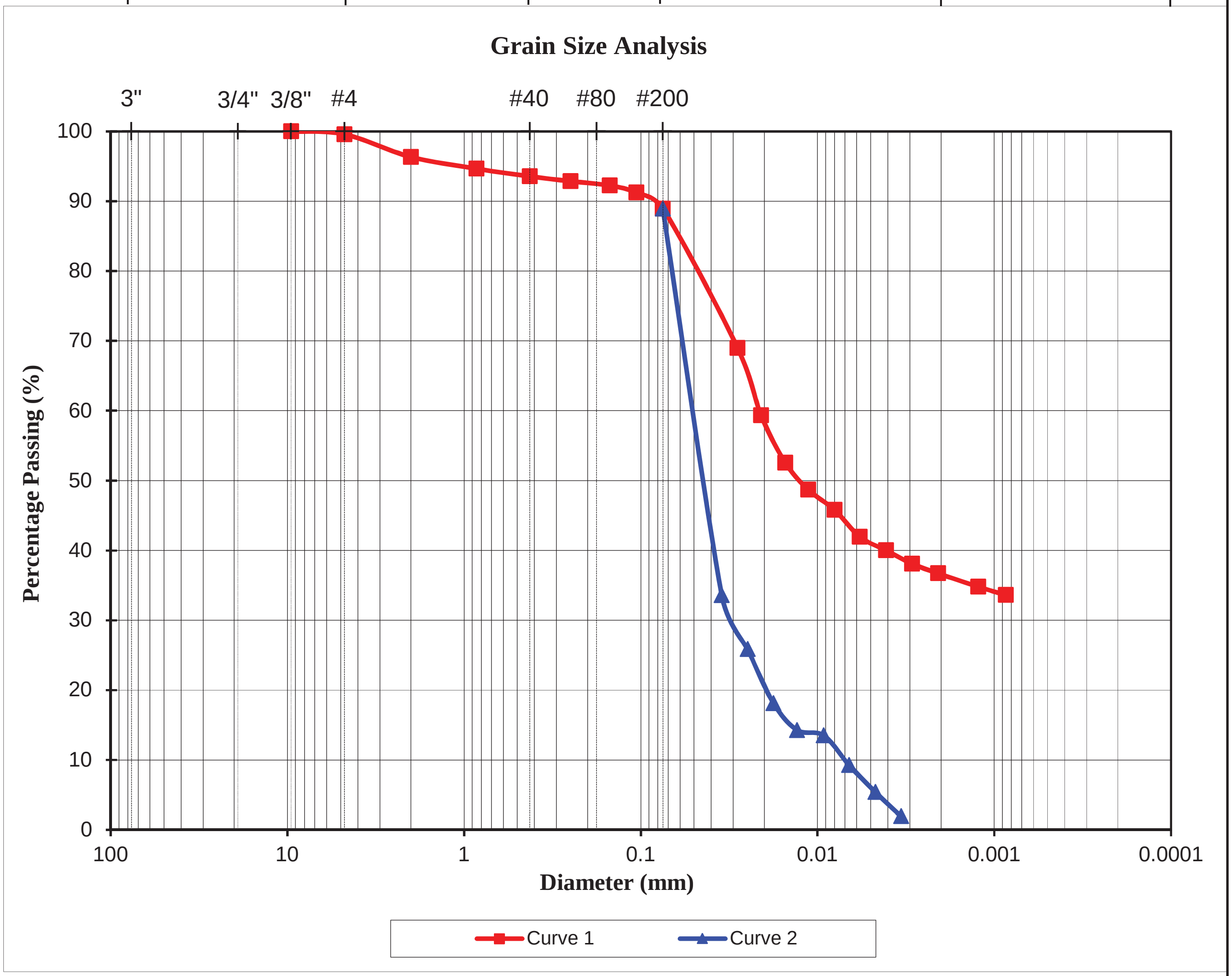
# AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

**SIEVE & DOUBLE HYDROMETER TESTS (ASTM D 4221)**

**Project :** SJRA Wallisville Siphon Replacement Project **Job No.:** G108-18
**Location of Project:** Harris County, Texas **Date of Testing:** 2/19/2018

| | Gravel | Sand | | Silt | Clay |
|---|---|---|---|---|---|
| | | Coarse to Medium | Fine | | |




| Curve | Boring | Depth (ft) | Soil Description | Cu | Cc | %-5µm | $D_{50}$ (mm) |
|---|---|---|---|---|---|---|---|
| 1 [(a)] | B-3 | 6-8 | Fat Clay (CH) | N/A | N/A | 41.05 | 0.0125 |
| 2 [(b)] | B-3 | 6-8 | Fat Clay (CH) | N/A | N/A | 5.86 | N/A |
| | | | | **% Dispersion =** | | 14.27% | |

Notes: (a) Hydrometer test with added dispersant
(b) Hydrometer test without added dispersant

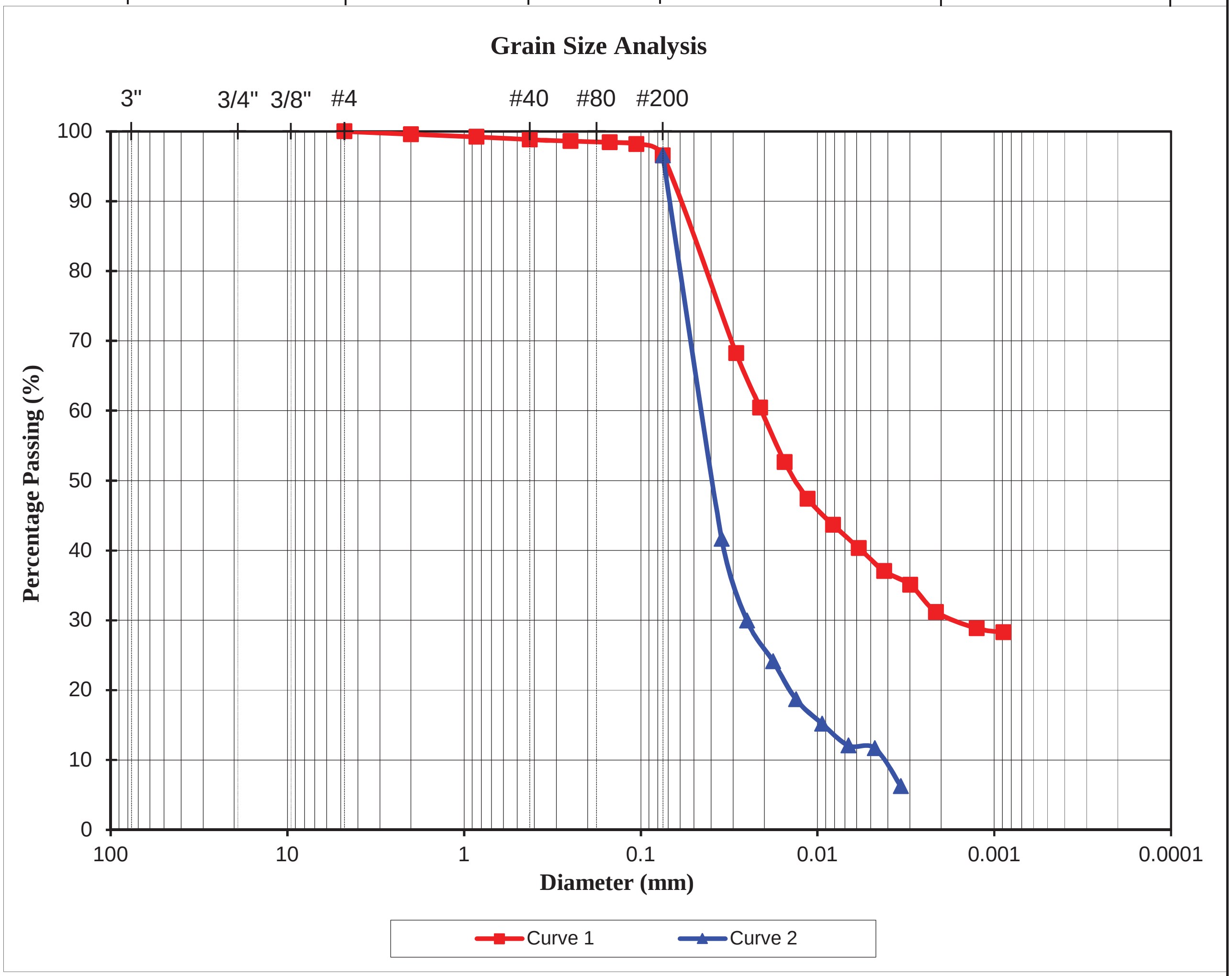
# AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

**SIEVE & DOUBLE HYDROMETER TESTS (ASTM D 4221)**

**Project :** SJRA Wallisville Siphon Replacement Project **Job No.:** G108-18
**Location of Project:** Harris County, Texas **Date of Testing:** 2/26/2018

| | Gravel | Sand: Coarse to Medium | Sand: Fine | Silt | Clay |
|---|---|---|---|---|---|




| Curve | Boring | Depth (ft) | Soil Description | Cu | Cc | %-5µm | $D_{50}$ (mm) |
|---|---|---|---|---|---|---|---|
| 1 [(a)] | B-4 | 12-14 | Lean Clay (CL) | N/A | N/A | 38.63 | 0.0132 |
| 2 [(b)] | B-4 | 12-14 | Lean Clay (CL) | N/A | N/A | 11.73 | N/A |
| | | | | | **% Dispersion =** | 30.36% | |

Notes: (a) Hydrometer test with added dispersant
(b) Hydrometer test without added dispersant

## AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

### RESULTS OF CRUMB TESTS (ASTM D 6572)

Project Name: SJRA Wallisville Siphon Replacement Project
Project No: G108-18 Test Date: 3/7/2018

| Boring Number | Depth, feet | 2 Minutes | | 1 Hour | | 6 Hours | |
|---|---|---|---|---|---|---|---|
| | | Grade | C (deg) | Grade | C (deg) | Grade | C (deg) |
| B-3 | 6-8 | 1 | 22.5 | 1 | 22.4 | 1 | 22.3 |
| | | | | | | | |
| B-4 | 2-4 | 1 | 22.5 | 1 | 22.4 | 1 | 22.3 |
| | | | | | | | |
| B-4 | 12-14 | 1 | 22.5 | 1 | 22.4 | 1 | 22.3 |
| | | | | | | | |
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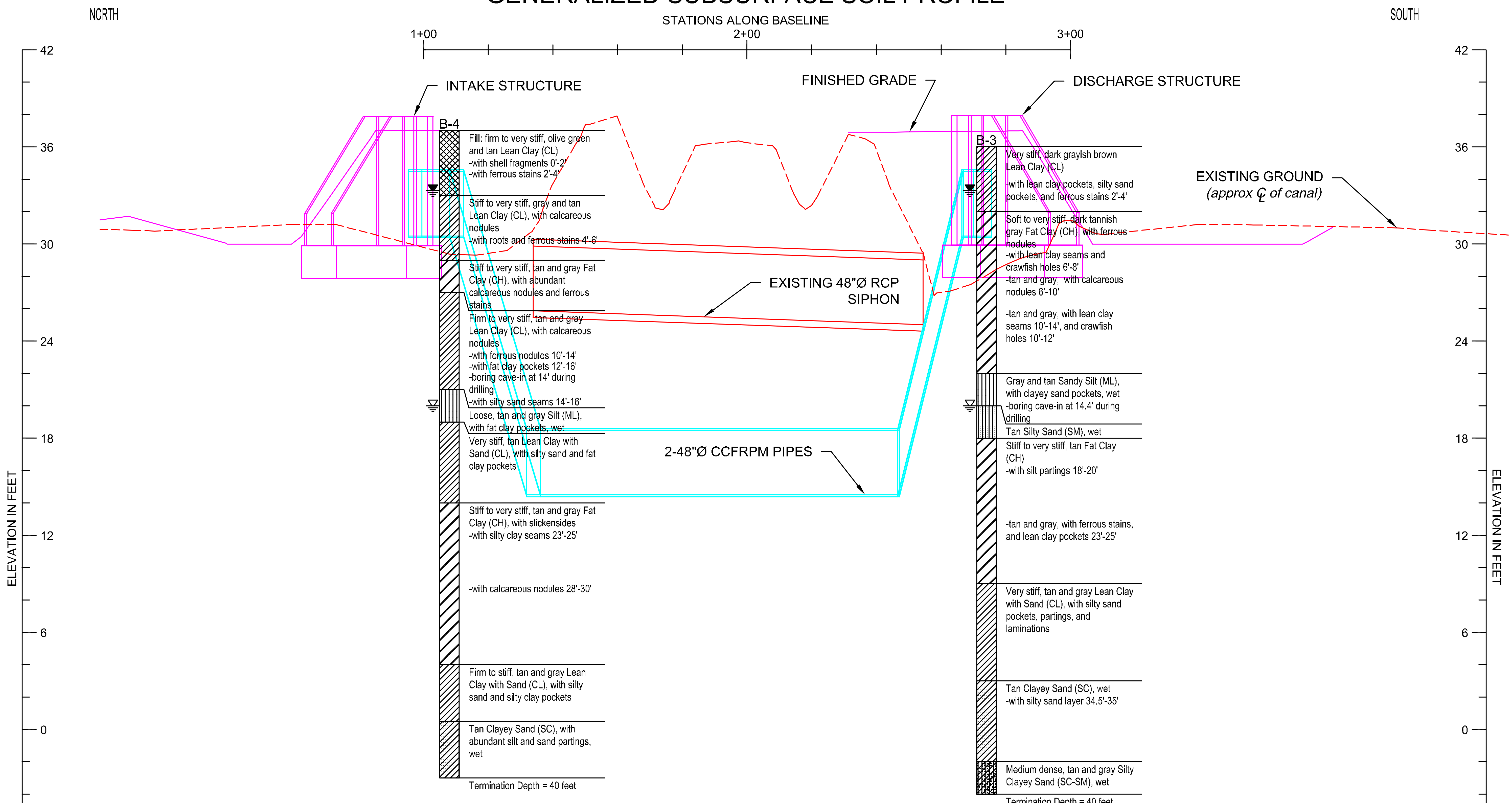
Grade Classification:
- Grade 1 Non-dispersive; No reaction
- Grade 2 Intermediate; Slight reaction
- Grade 3 Dispersive; Moderate reaction
- Grade 4 Highly Dispersive; Strong reaction

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.

**APPENDIX B**

| | |
|---|---|
| Plate B-1 | Generalized Soil Profile |

# GENERALIZED SUBSURFACE SOIL PROFILE




LEGEND:

- Paving
- Fill
- Low plasticity clay
- High plasticity clay
- Silt
- Silty sand
- Silty clayey sand
- Clayey sand
- Depth of water encountered during drilling
- Depth of water measured > 24 hours after drilling

NOTES:

1) See Plate A-2 in Appendix A for approximate location of borings.
2) Data concerning subsurface conditions have been obtained at boring locations only. Actual conditions between borings may differ from profile shown here.
3) See logs of boring for detailed description of soil encountered in each borehole.
4) Ground surface elevation at each boring location was estimated based on AutoCAD drawing data provided by Texas Water Engineering, PLLC.
5) Soil stratigraphy and secondary soil structure (such as seams, layers, or pockets of sands, silts, slickensides, fissures, and siltstones/claystones) that are different from what were identified in the actual borings may exist away from these borings.

**AVILES ENGINEERING CORPORATION**

GENERALIZED SOIL PROFILE
SAN JACINTO RIVER AUTHORITY
WALLISVILLE SIPHON REPLACEMENT PROJECT
HARRIS COUNTY, TEXAS

| AEC PROJECT NO.: G108-18 | DATE: 04-23-18 | SOURCE DRAWING PROVIDED BY: TEXAS WATER ENGINEERING |
|---|---|---|
| VERTICAL SCALE: 1" = 6' HORIZONTAL SCALE: 1" = 30' | DRAFTED BY: BpJ | PLATE NO.: PLATE B-1 |

AVILES ENGINEERING CORP.

## APPENDIX C

## G108-18 SJRA WALLISVILLE SIPHON REPLACEMENT PROJECT
## DESIGN SOIL PARAMETERS FOR LATERAL EARTH PRESSURE FOR SIPHONS AND HEADWALLS

| Boring | Depth (ft) | Elev. (ft MSL) | Soil Type | $\gamma$ (pcf) | $\gamma'$ (pcf) | OSHA Type | Short-Term | | | | | | Long-Term | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | C (psf) | $C_\alpha$ (psf) | $\varphi$ (deg) | $K_a$ | $K_0$ | $K_p$ | C' (psf) | $C_\alpha'$ (psf) | $\varphi'$ (deg) | $K_a$ | $K_0$ | $K_p$ |
| - | Varies | Varies | Cement Stabilized Sand | 120 | 58 | C | 0 | 0 | 30 | 0.33 | 0.50 | 3.00 | 0 | 0 | 30 | 0.33 | 0.50 | 3.00 |
| - | Varies | Varies | Select Backfill (CL) | 120 | 58 | C | 1600 | 900 | 0 | 1.00 | 1.00 | 1.00 | 180 | 120 | 22 | 0.45 | 0.63 | 2.20 |
| B-1 & B-4 | 0-4 | 37 to 33 | Fill: firm to very stiff CL | 120 | 58 | C | 1000 | 750 | 0 | 1.00 | 1.00 | 1.00 | 100 | 80 | 18 | 0.53 | 0.69 | 1.89 |
| | 4-8 | 33 to 29 | Stiff to very stiff CL | 127 | 65 | C* | 1100 | 800 | 0 | 1.00 | 1.00 | 1.00 | 100 | 80 | 18 | 0.53 | 0.69 | 1.89 |
| | 8-14 | 29 to 23 | Stiff to very stiff CH/CL | 135 | 73 | C* | 1600 | 900 | 0 | 1.00 | 1.00 | 1.00 | 150 | 100 | 16 | 0.57 | 0.72 | 1.76 |
| | 14-16 | 23 to 21 | Firm to very stiff CL | 128 | 66 | C* | 800 | - | 0 | 1.00 | 1.00 | 1.00 | 75 | - | 18 | 0.53 | 0.69 | 1.89 |
| | 16-18 | 21 to 19 | Loose ML | 115 | 53 | C | 0 | - | 26 | 0.39 | 0.56 | 2.56 | 0 | - | 26 | 0.39 | 0.56 | 2.56 |
| | 18-30 | 19 to 7 | Stiff to very stiff CL/CH | 125 | 63 | C* (18-20) | 1400 | - | 0 | 1.00 | 1.00 | 1.00 | 125 | - | 16 | 0.57 | 0.72 | 1.76 |
| B-2 & B-3 | 0-10 | 36 to 26 | Stiff to hard CL/CH | 124 | 62 | B, C*(3-10) | 1000 | 750 | 0 | 1.00 | 1.00 | 1.00 | 100 | 80 | 16 | 0.57 | 0.72 | 1.76 |
| | 10-14 | 26 to 22 | Soft to very stiff CH | 114 | 52 | C* | 600 | 500 | 0 | 1.00 | 1.00 | 1.00 | 60 | 50 | 16 | 0.57 | 0.72 | 1.76 |
| | 14-18 | 22 to 18 | ML/SM | 115 | 53 | C | 0 | 0 | 26 | 0.39 | 0.56 | 2.56 | 0 | 0 | 26 | 0.39 | 0.56 | 2.56 |
| | 18-22 | 18 to 14 | Soft to very stiff CL/CH | 120 | 58 | C (18-20) | 300 | - | 0 | 1.00 | 1.00 | 1.00 | 50 | - | 16 | 0.57 | 0.72 | 1.76 |
| | 22-27 | 14 to 9 | Stiff CH | 129 | 67 | N/A | 1500 | - | 0 | 1.00 | 1.00 | 1.00 | 150 | - | 16 | 0.57 | 0.72 | 1.76 |
| | 27-30 | 9 to 6 | Firm to very stiff CL | 122 | 60 | N/A | 800 | - | 0 | 1.00 | 1.00 | 1.00 | 75 | - | 18 | 0.53 | 0.69 | 1.89 |

Notes: (1) $\gamma$ = Unit weight for soil above water level, $\gamma'$ = Buoyant unit weight for soil below water level.

(2) C = Soil ultimate cohesion, $C_\alpha$ = Soil ahesion between soil and concrete, and $\varphi$ = Soil friction angle for short term.

(3) C' = Soil ultimate cohesion, $C_\alpha'$ = Soil ahesion between soil and concrete, and $\varphi'$ = Soil friction angle for long term.

(4) Friction angle between soil and concrete for short term and long term, $\delta = 2/3\ \varphi$ or $\delta' = 2/3\ \varphi'$, respectively.

(5) $K_a$ = Coefficient of active earth pressure, $K_0$ = Coefficient of at-rest earth pressure, $K_p$ = Coefficient of passive earth pressure, for level backfill.

(6) CL = Lean Clay, CH = Fat Clay, ML = Silt, and SM = Silty Sand.

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

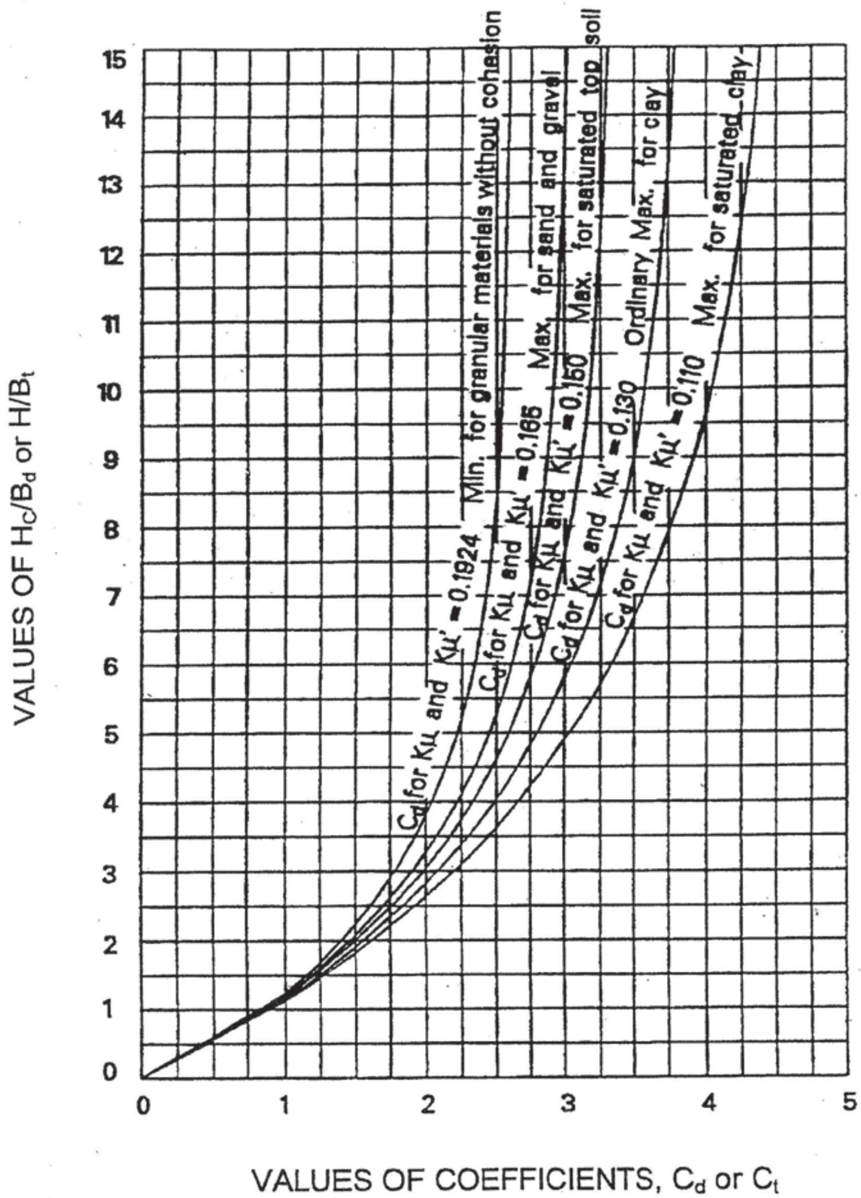
A: cohesive soils with qu = 1.5 tsf or greater (qu = Unconfined Compressive Strength of the Soil)

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

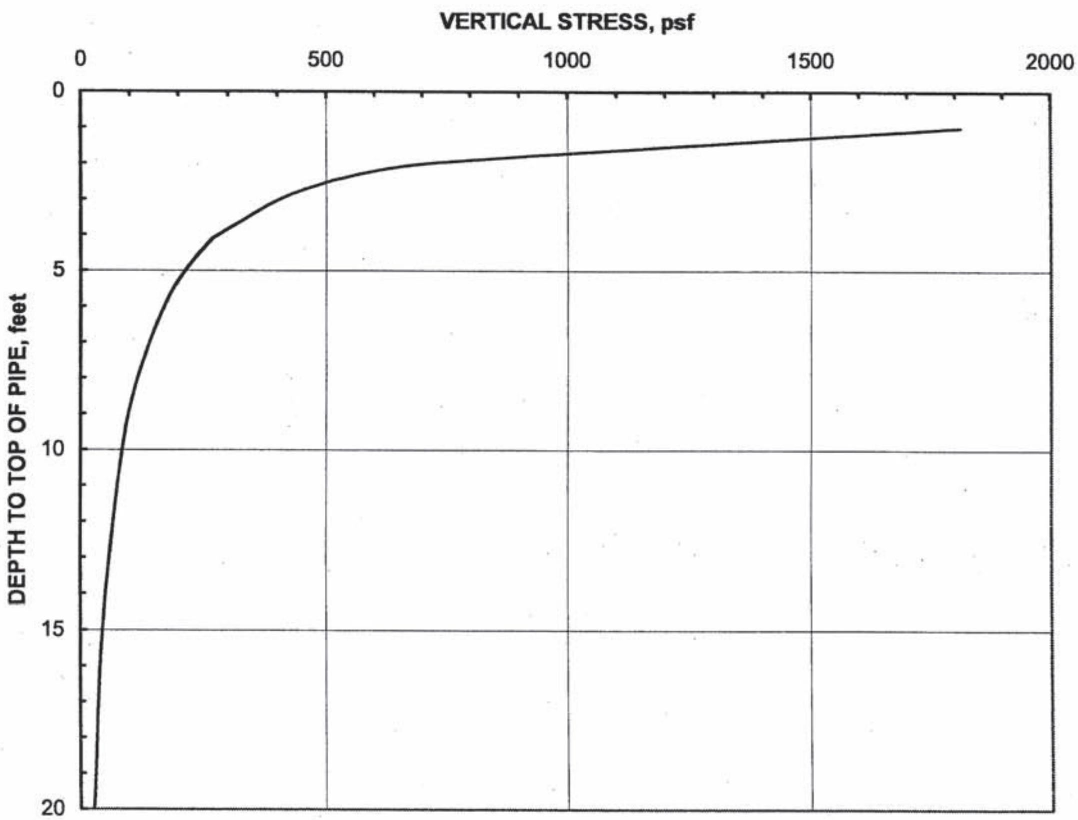
(C*) The above OSHA Soil Types were recommended on assumption that the excavations are dewatered; if the site is not dewatered, all submerged soils should be classified as OSHA Type C.

AVILES ENGINEERING CORP.




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

AVILES ENGINEERING CORP.




**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.

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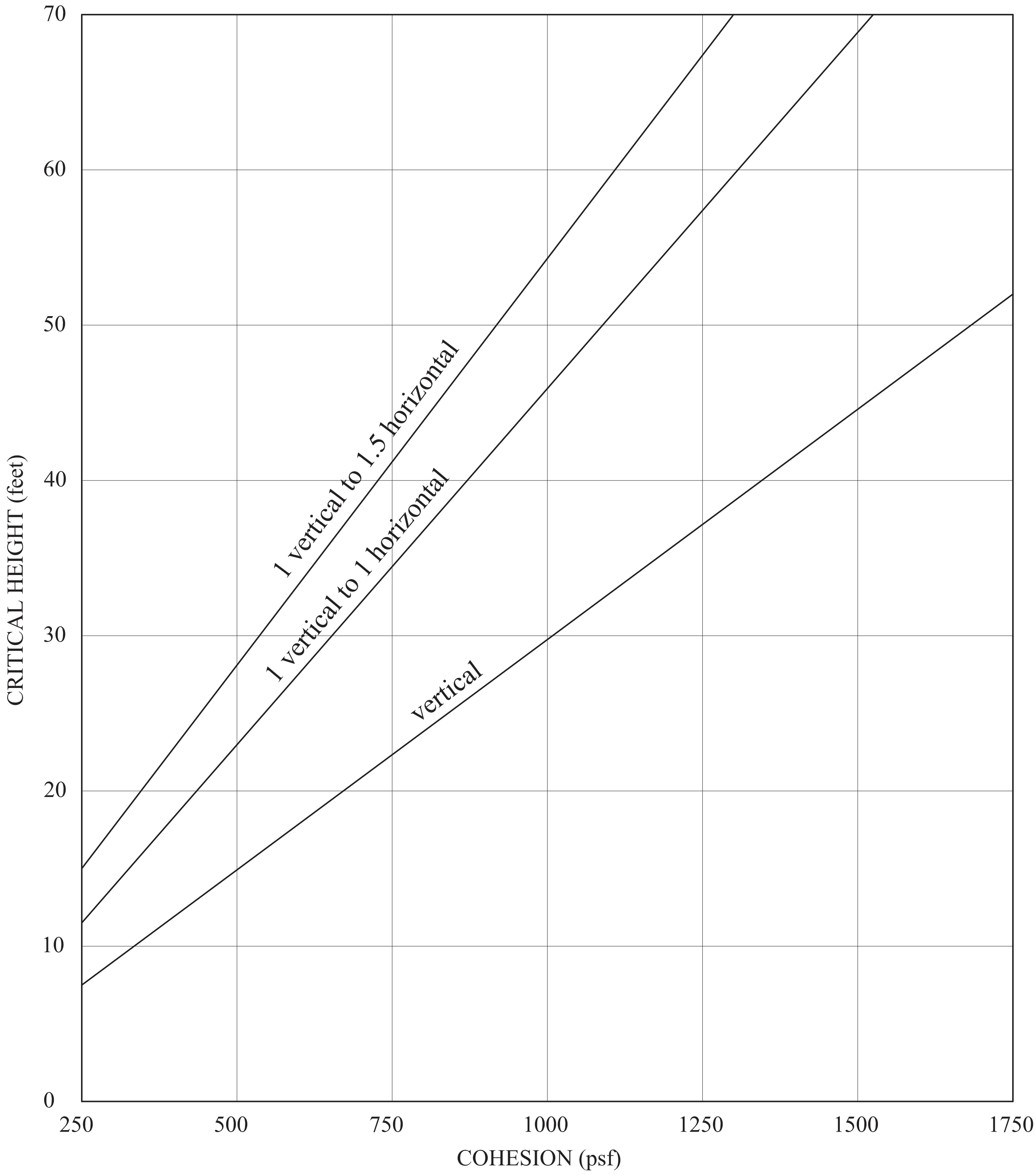
## APPENDIX D

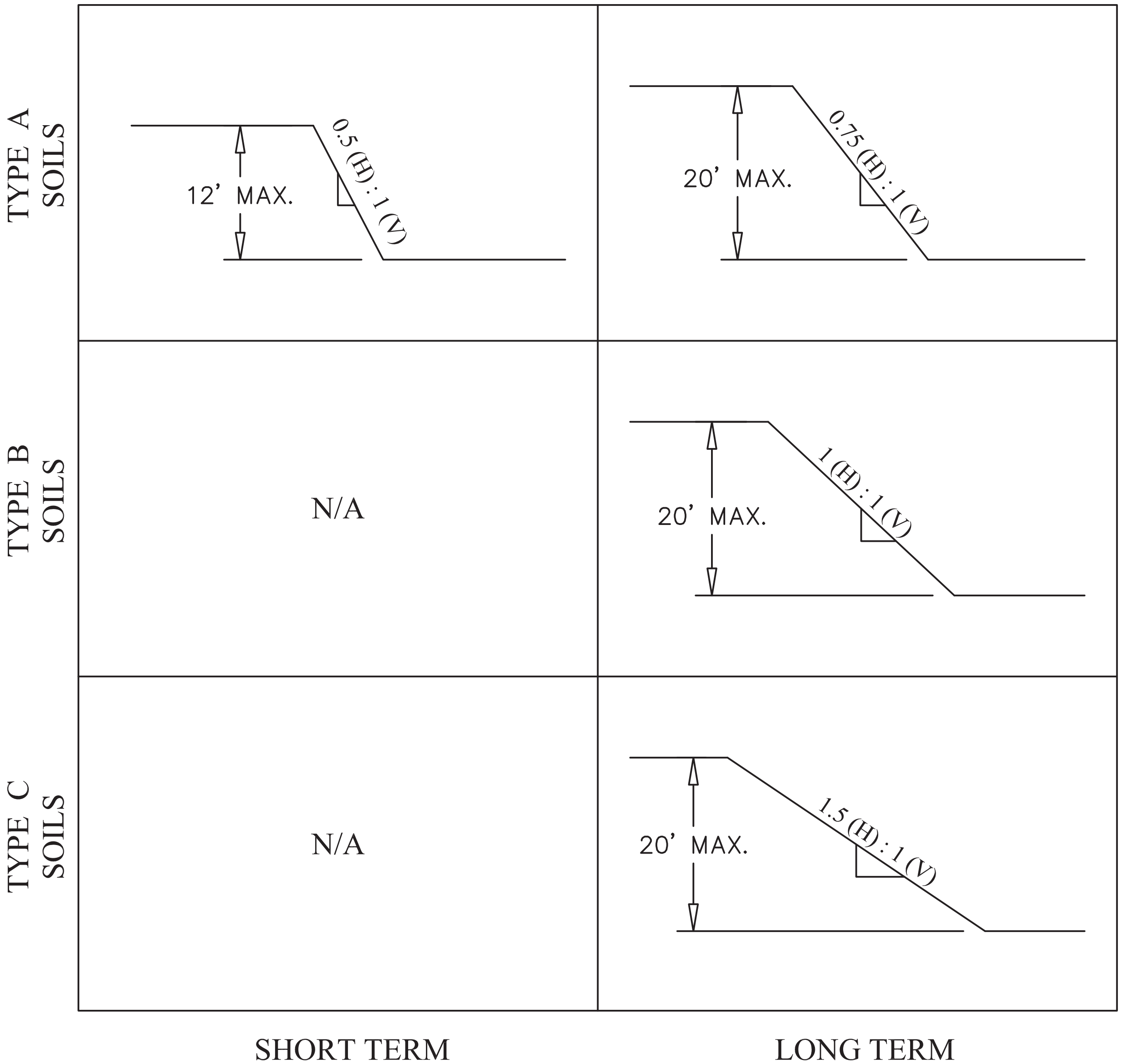
ENGINEERING CORP.

## 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.*

# MAXIMUM ALLOWABLE SLOPES



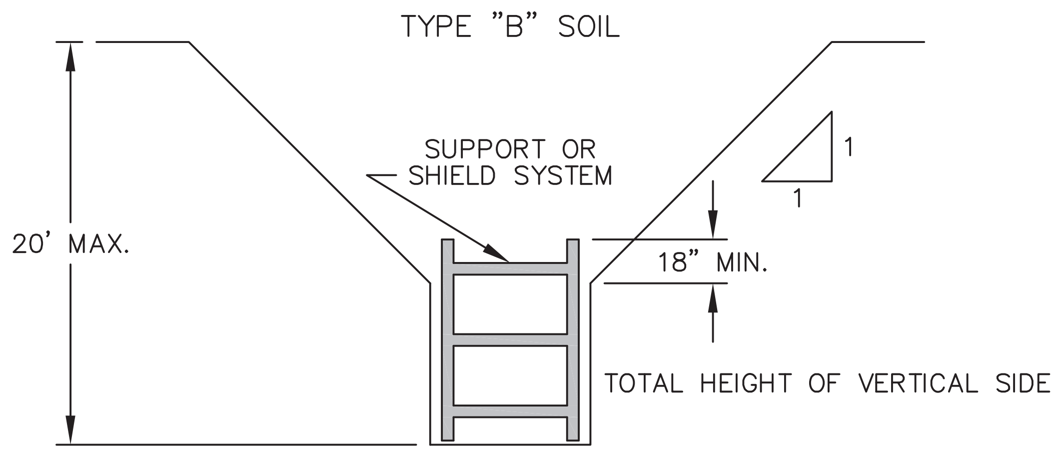

NOTES:

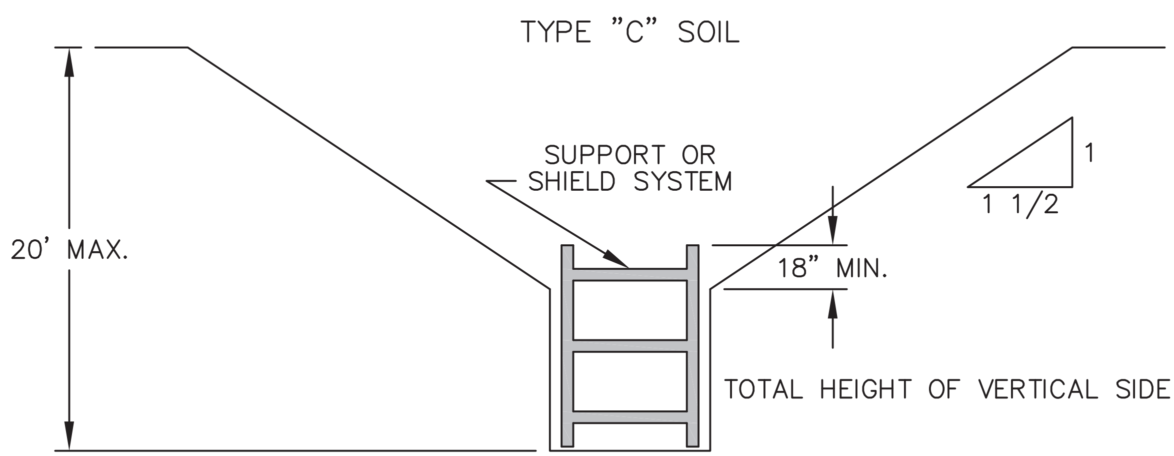
(1) For Type A soils, a short term maximum allowable slope of 0.5 (H) : 1 (V) is allowed in excavations that are 12 feet or less in depth; short term (24 hours or less) maximum allowable slopes for excavations greater than 12 feet in depth shall be 0.75 (H) : 1 (V).

(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



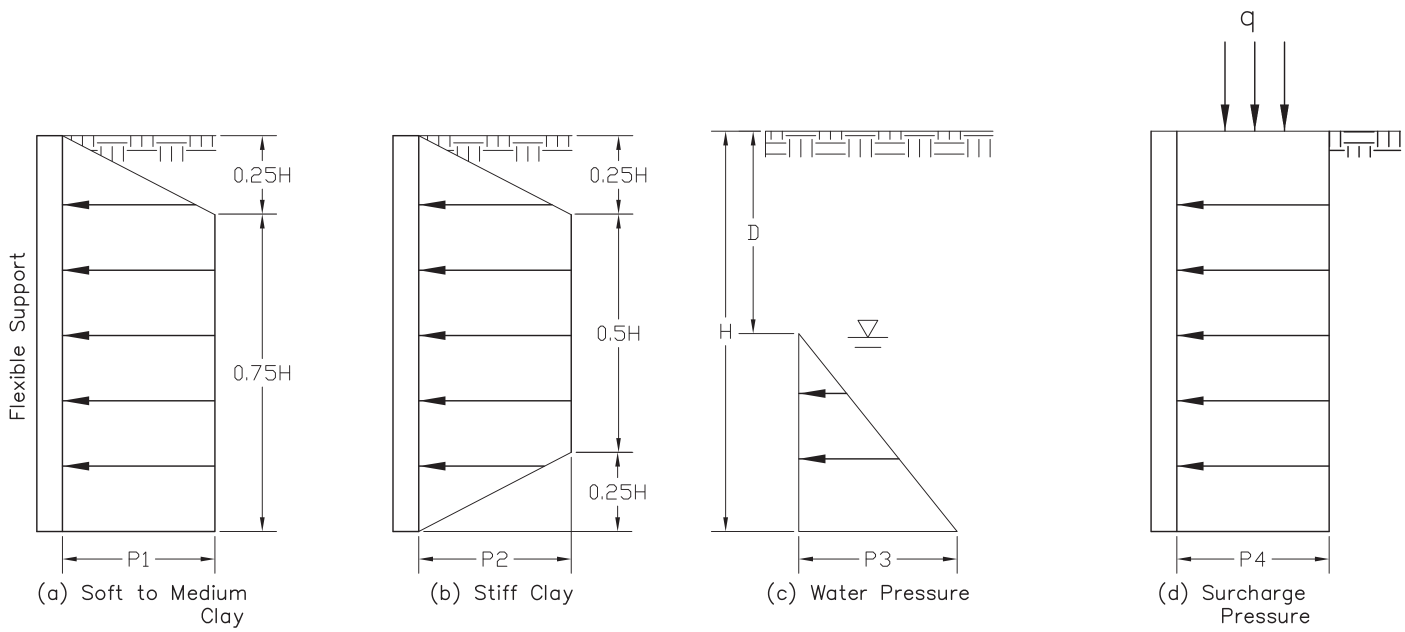




*Reference: OSHA, Safety and Health Regulations for Construction, 1926 Subpart P.*

AMLES ENGINEERING CORP.

## 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 = $\gamma$H-4C, psf
P2 = Lateral earth pressure = 0.4$\gamma$H, psf
P3 = Water pressure = $\gamma_w$ (H-D), psf
P4 = Lateral earth pressure caused by surcharge = q$K_a$, psf
$\gamma$ = Effective unit weight of soil, pcf
$\gamma_w$ = Unit weight of water, pcf
C = Drained shear strength or cohesion, psf
$K_a$ = Coefficient of active earth pressure

Notes:

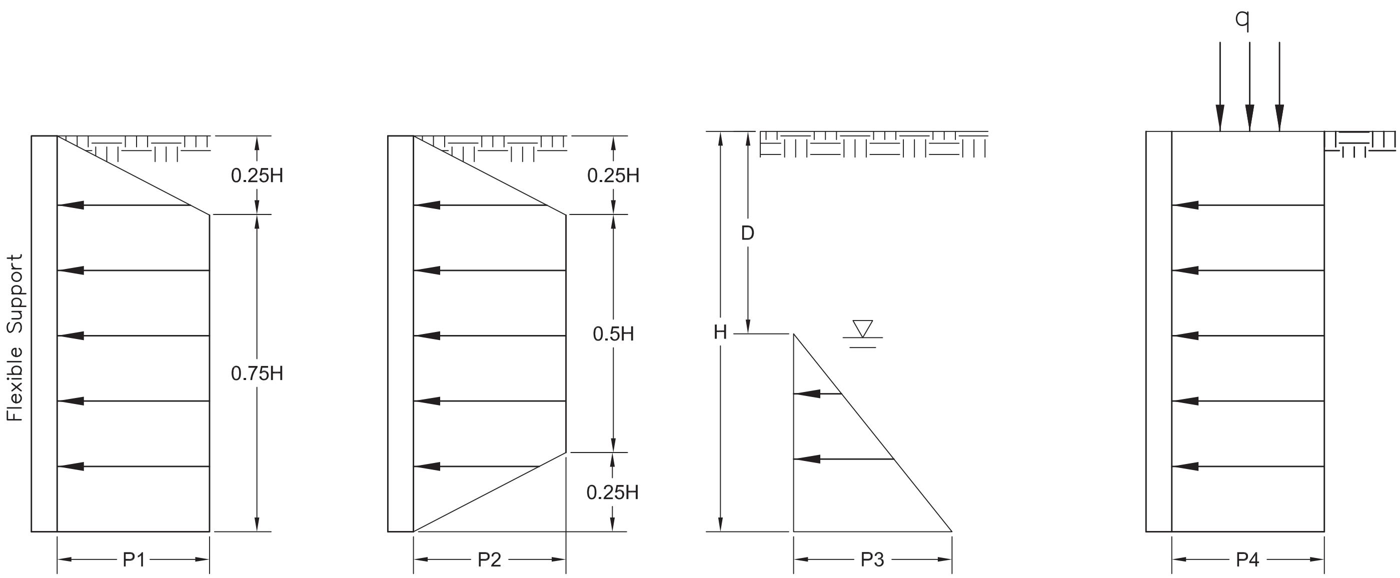
1. All pressures are additive.
2. No safety factors are included.
3. For use only during long term construction.
4. If $\gamma$H/C < 4, use section (b),
   If 4 < $\gamma$H/C < 6, use larger of section (a) or (b),
   If $\gamma$H/C > 6, use section (a).

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

ENGINEERING CORP.

**LATERAL PRESSURE DIAGRAMS**

FOR OPEN CUTS IN COHESIVE SOIL - SHORT TERM CONDITIONS




(a) Soft to Medium Clay (b) Stiff Clay (c) Water Pressure (d) Surcharge Pressure

Empirical Pressure Distributions

Where:

- H = Total excavation depth, feet
- D = Depth to water table, feet
- P1 = Lateral earth pressure = $\gamma H-4S_u$, psf
- P2 = Lateral earth pressure = $0.2\gamma H$, psf
- P3 = Water pressure = $\gamma_w (H-D)$, psf
- P4 = Lateral earth pressure caused by surcharge = $qK_a$, psf
- $\gamma$ = Effective unit weight of soil, pcf
- $\gamma_w$ = 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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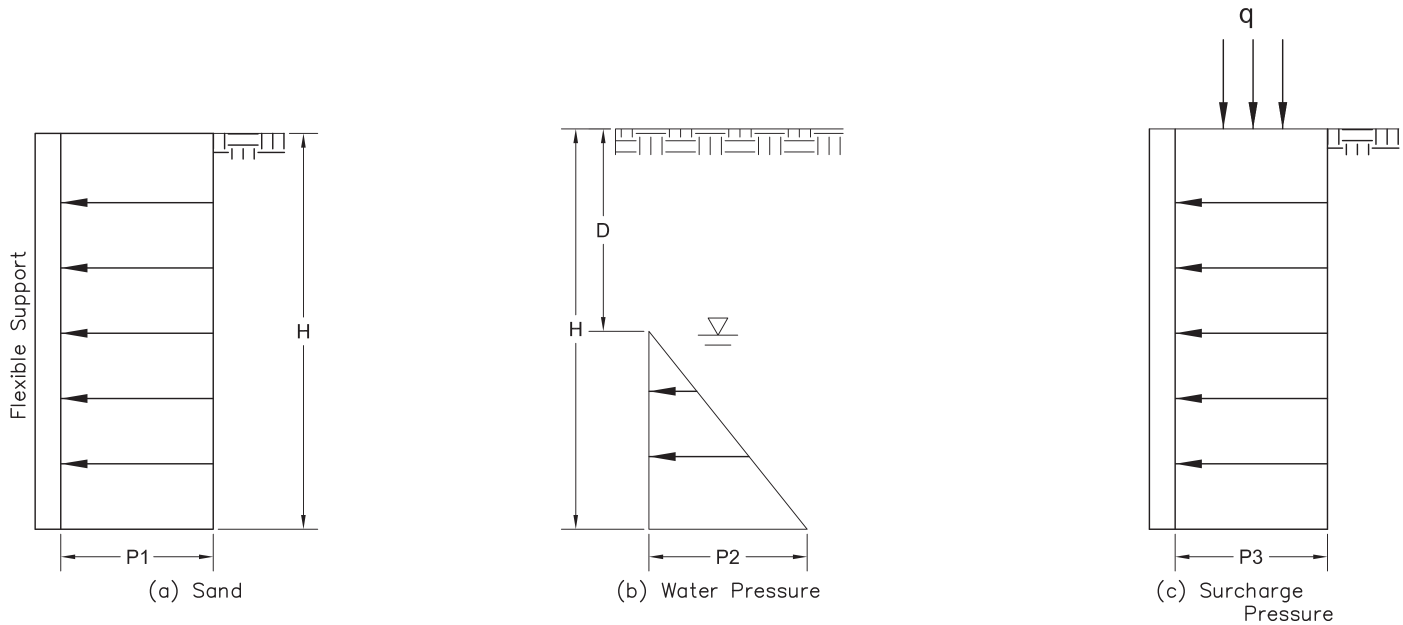
## LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN SAND



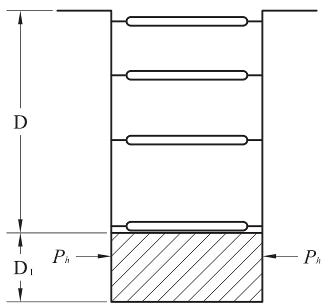

Empirical Pressure Distributions

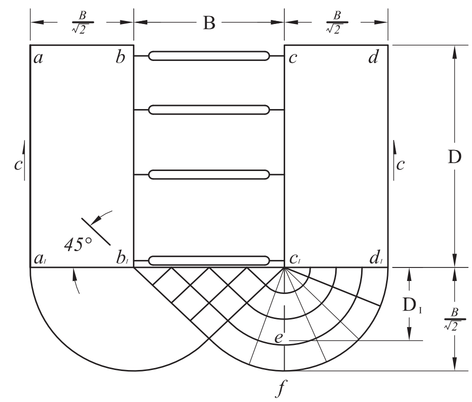
Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $0.65^{*}\gamma H K_a$, psf
P2 = Water pressure = $\gamma_w$ (H-D), psf
P3 = Lateral earth pressure caused by surcharge = $qK_a$, psf
$\gamma$ = Effective unit weight of soil, pcf
$\gamma_w$ = Unit weight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

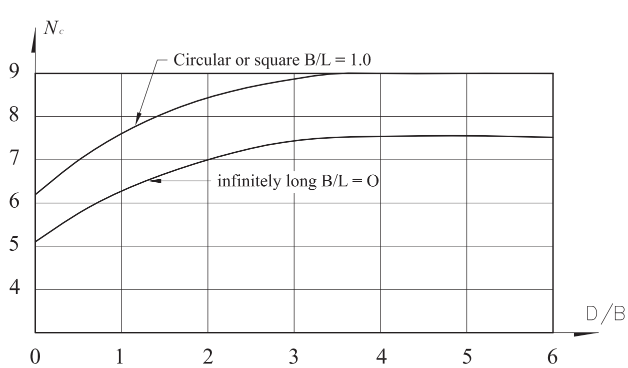
$$\text{Depth of Buried Length, } (D_1) = \frac{1.5(\gamma D+q)-NcC}{(C/B)-0.5\gamma} \;;\; D_1 \geqslant 5 \text{ ft.}$$

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

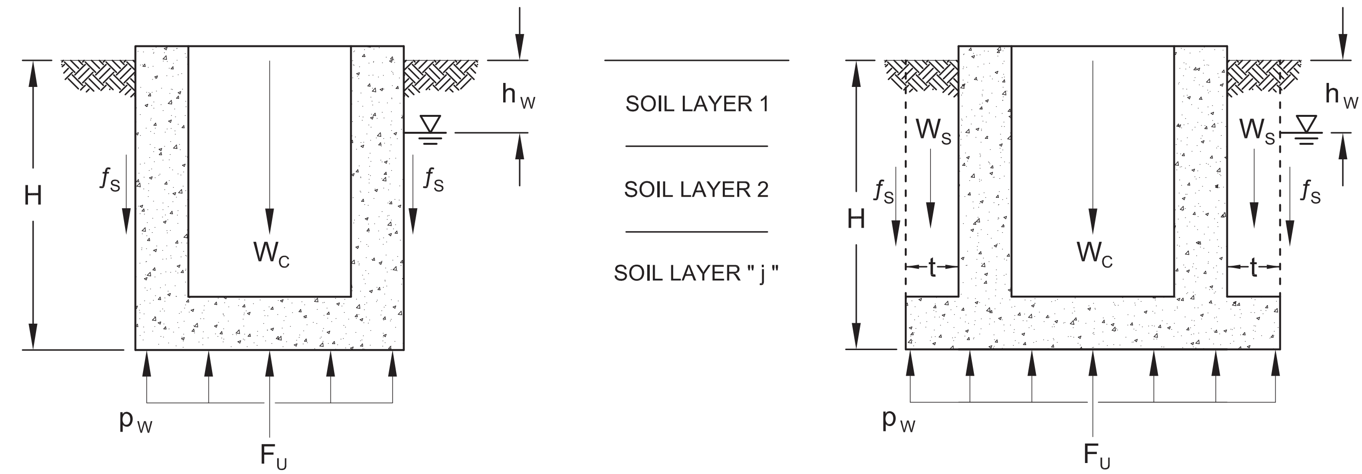
*Reference: Bjerrum, L. and Eide, O., Stability of Strutted Excavations in Clay, Geotechnique, 6, 32-47 (1956).*

ENGINEERING CORP.

# 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:

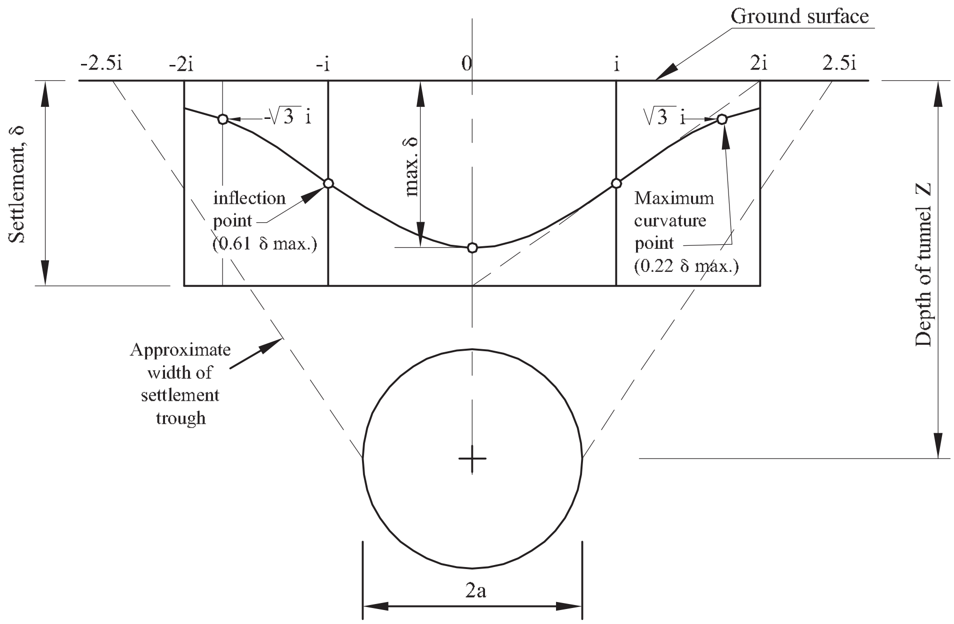
- $A_B$ = area of base, sq. ft.
- $H$ = buried height of structure, ft.
- $h_W$ = depth to water table, ft.
- $p_W$ = $\gamma_W (H - h_W)$, unit hydrostatic uplift, psf.
- $\gamma_W$ = 62.4 pcf, unit weight of water
- $F_U$ = $p_W A_B$, hydrostatic uplift force, lbs.
- $f_{S_j}$ = unit frictional resistance of soil layer " j ", psf.
- $c_j$ = undrained cohesion of soil layer " j ", psf.
- $\alpha$ = 0.55, cohesion factor between soil and structure wall
- $\sigma_{V_j}$ = effective overburden pressure at midpoint of soil layer " j ", psf.
- $\delta_j$ = 0.75 $\Phi_j$, friction angle between soil layer " j " and concrete wall, degrees
- $\Phi_j$ = 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:*

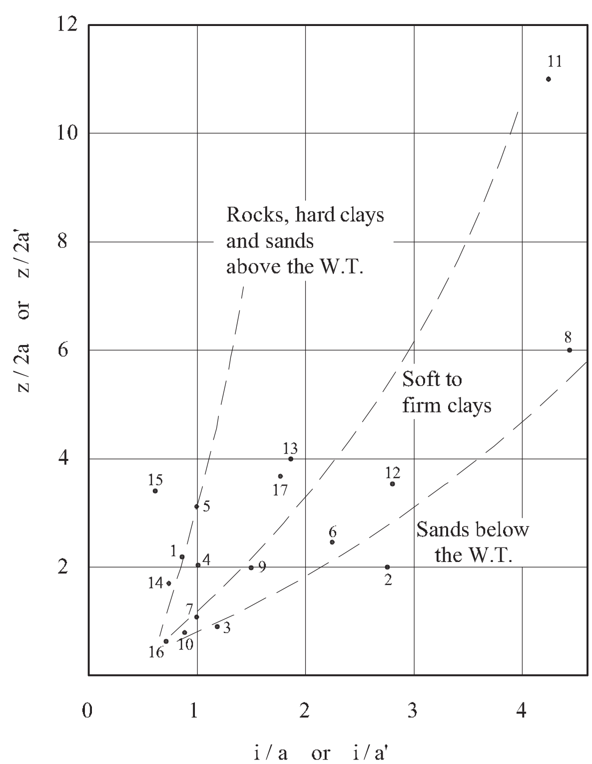
1) *American Concrete Pipe Association, (1996), Manhole Floatation*
2) *O'Neill, M.W., and Reese, L.C., (1999), "Drilled Shafts: Construction Procedures and Design Methods", FHWA-IF-99-025*

## 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.*

AVILES ENGINEERING CORP.

**Tunnel Behavior: Sands and Gravels**
(Terzaghi, 1977)

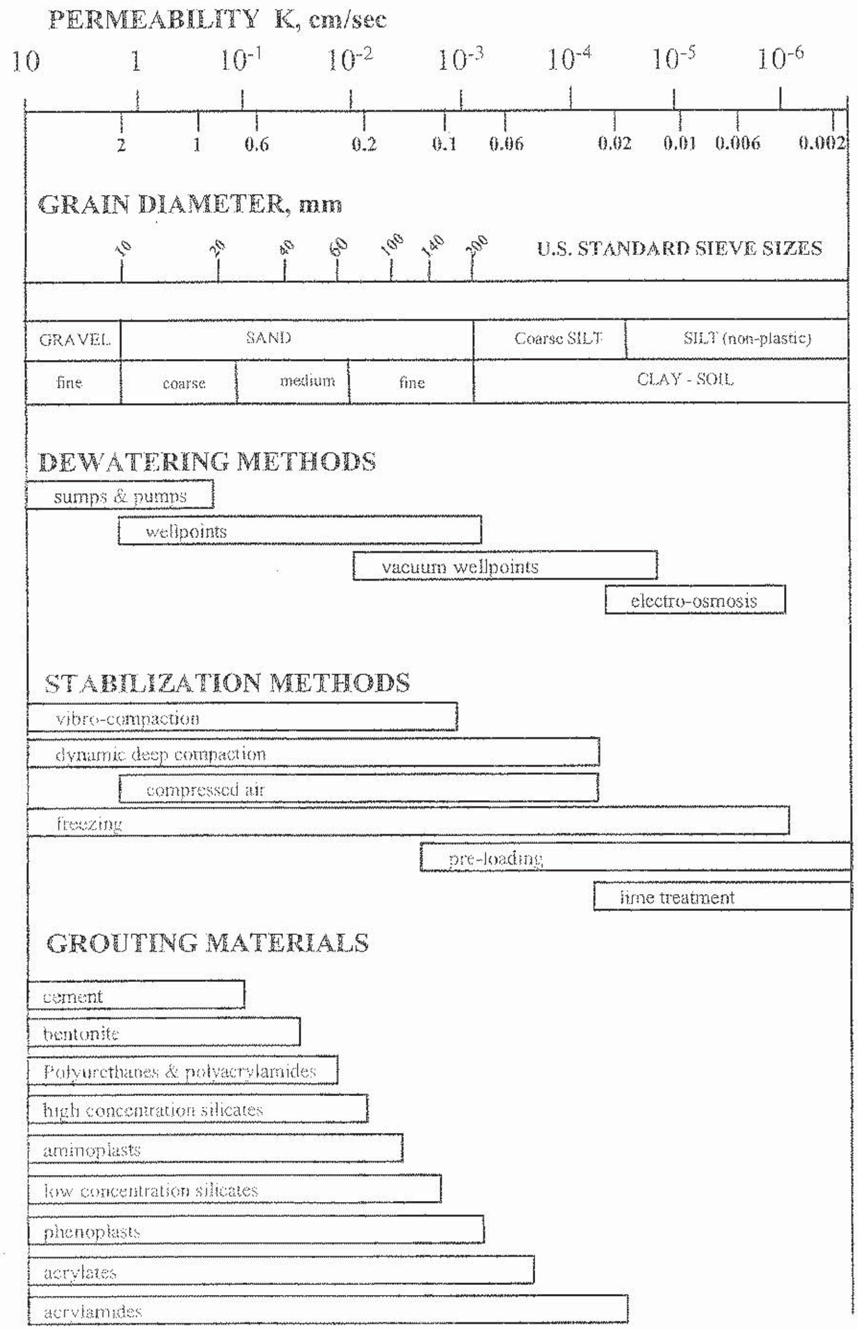
| Designation | Degree of Compactness | Tunnel Behavior | |
|---|---|---|---|
| | | Above Water Table | Below Water Table |
| Very Fine Clean Sand | Loose, N ≤ 10<br>Dense, N > 30 | Cohesive Running<br>Fast Raveling | Flowing<br>Flowing |
| Fine Sand with Clay Binder | Loose, N ≤ 10<br>Dense, N > 30 | Rapid Raveling<br>Firm or Slowly Raveling | Flowing<br>Slowly Raveling |
| Sand or Sandy Gravel with Clay Binder | Loose, N < 10<br>Dense, N > 30 | Rapid Raveling<br>Firm | Rapidly Raveling or Flowing<br>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.

AVILES
ENGINEERING CORP.

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.