AVILES
ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

SAN JACINTO RIVER AUTHORITY
12 INCH WATER LINE REPLACEMENT CROSSING
PANTHER BRANCH AT GROGAN'S POINT ROAD
THE WOODLANDS, TEXAS

Reported to

SJRA Technical Services Department

The Woodlands, Texas

by

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REPORT NO. G152-18

November 2018

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November 5, 2018

Mr. Aaron K. Schindewolf, P.E.
SJRA Technical Services Department
2436 Sawdust Road
The Woodlands, TX, 77380

Reference: **Geotechnical Investigation**
San Jacinto River Authority
12 inch Water Line Replacement crossing
Panther Branch at Grogan's Point Road
The Woodlands, Texas
AEC Report No. G152-18

Dear Mr. Schindewolf,

Aviles Engineering Corporation (AEC) is pleased to present this report of the results of our geotechnical investigation for the above referenced project. This investigation was authorized to proceed by you on September 4, 2018. The project terms and conditions are based upon the Professional Services Agreement (Contract No. 18-0039) between SJRA and AEC dated March 26, 2018, and the scope of work was performed in accordance with SJRA Work Order No. 2 dated August 13, 2018, based upon AEC Proposal No. G2018-07-07, dated July 20, 2018.

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

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

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THE WOODLANDS, TEXAS

1.0 INTRODUCTION

1.1 Project Description

This report presents the results of a geotechnical investigation performed by Aviles Engineering Corporation (AEC) for the proposed water line replacement crossing Panther Branch at Grogan's Point Road. The project is located to the northeast of the intersection of Grogan's Point Road and North Tranquil Path, in The Woodlands, Texas. A vicinity map is presented on Plate A-1, in Appendix A.

AEC understands that an existing San Jacinto River Authority (SJRA) 12 inch diameter water line crossing Panther Branch has been exposed and damaged due to severe erosion within Panther Branch. Based on the schematic drawings provided by SJRA Technical Services Department on June 14, 2018, approximately 760 linear foot of 12 inch diameter water line crossing beneath Panther Branch will be replaced. The replacement water line will be installed by horizontal directional drill (HDD) method. In addition, AEC anticipates that the replacement water line will require two pits to connect the new line with the existing water line on either end.

1.2 Purpose and Scope

The purpose of this geotechnical investigation is to evaluate the subsurface soil and ground water conditions at the project site and to develop geotechnical engineering recommendations for the replacement water line. The scope of this geotechnical investigation is summarized below:

1. Drilling and sampling two soil borings ranging from a depth of 40 to 50 feet below existing grade;
2. Performing soil laboratory testing on selected soil samples;
3. Engineering recommendations for the replacement water line to be installed by HDD, including design parameters, loadings on the water line, and face stability;
4. Engineering recommendations for the pits, including excavation, shoring, and backfill;
5. Construction guidelines for the water line and the pits, including dewatering recommendations.

Recommendations for restoration of Panther Branch at the water line crossing is beyond AEC's scope of service.

2.0 SUBSURFACE EXPLORATION

Subsurface conditions at the site were investigated by drilling two soil borings ranging from a depth of 40 to 50 feet below existing grade in the proximity of the proposed replacement water line. The approximate boring locations are shown on the Boring Location Plan on Plate A-2, in Appendix A. Boring survey data was not available at the time this report was prepared; however, AEC compared the GPS coordinates against as built drawings (dated February 1990) provided by SJRA in order to estimate the approximate surface elevation of the borings.

AEC notes that Borings B-1 and B-2 were drilled along the approximate alignment of the proposed replacement water line on the south side of Panther Branch only. Due to site access issues, borings were not able to be performed on the north side of Panther Branch. AEC notes that the soil and groundwater conditions along channels in the Greater Houston area vary significantly. It is possible that different soil and groundwater conditions could be encountered during construction of the water line on the north side of Panther Branch. AEC recommends that additional soil borings be performed on the north side of Panther Branch in order to provide additional coverage of the proposed replacement water line alignment. If additional soil borings are not performed, AEC will not be liable if changed soil or groundwater conditions are encountered at areas along the project alignment that are not currently covered by AEC’s current borings.

The borings were drilled using a truck-mounted drill rig and were advanced using dry auger method and then completed using wet rotary method once groundwater was encountered or the borings began to cave in. Undisturbed samples of cohesive soils were obtained from the borings by pushing 3-inch diameter thin-wall, seamless steel Shelby tube samplers in 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. 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. After completion of drilling, the boreholes were left open overnight so that a 24 hour groundwater reading could be obtained. Afterwards, the borings were backfilled with bentonite chips. Details of the soils encountered in the borings are presented on Plates A-3 and A-4, in Appendix A.

3.0 LABORATORY TESTING

Soil laboratory testing was performed by AEC personnel. Samples from the borings were examined and classified in the laboratory by a technician under 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 representative samples to establish the index properties and confirm field classification of the subsurface soils. Strength properties of cohesive soils were estimated by means of unconfined compression (UC) and Unconsolidated-Undrained (UU) triaxial tests performed on undisturbed samples. The test results are presented on their representative boring logs. 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-5 through A-8, in Appendix A. The results of the sieve analyses are presented on Plates A-9 through A-11, in Appendix A.

Double Hydrometer Tests: To evaluate the dispersive characteristics of clayey soil at Panther Branch, a double hydrometer test was performed on a selected soil sample in accordance with ASTM D 4221. The results of the double hydrometer test is summarized in Table 1, and is presented on Plate A-12, in Appendix A. When the percent dispersion is less than 30, it indicates that the soil is non-dispersive. When the percent dispersion is equal to 30 but less than 50, it indicates that the soil is intermediately dispersive. When the percent dispersion is greater than 50, it indicates that the soil is dispersive.

Table 1. Summary of Double Hydrometer Test Results at Panther Branch

Sample ID and Description	Dispersion (%)	Dispersion Classification
B-2, 10'-12', Sandy Lean Clay (CL)	40.3	Intermediately Dispersive

Crumb Dispersion Tests: To evaluate the dispersive characteristics of clayey soils at Panther Branch, two crumb tests were performed on selected soil samples in accordance with ASTM D 6572, Method A. The results of the crumb tests are summarized on Table 2 and are presented on Plate A-13, in Appendix A.

Table 2. Summary of Crumb Test Results at Panther Branch

Sample ID and Description	Dispersive Grade	Dispersive Classification
B-1, 4'-6', Silty Sand (SM)	2	Intermediate

Sample ID and Description	Dispersive Grade	Dispersive Classification
B-2, 10’-12’, Sandy Lean Clay (CL)	4	Highly dispersive

4.0 SITE CONDITIONS

Based on AEC’s site visit, the project area is a clearing along the south bank of Panther Branch. The existing banks of Panther Branch have experienced significant erosion. AEC understands that the existing broken water line will be abandoned and the replacement water line will be installed via HDD. The north side of Panther Branch could not be accessed by AEC’s drill rig at the time of our site visit.

4.1 Subsurface Conditions

Soil strata encountered in our borings are summarized below:

Boring	Depth (ft)	Description of Stratum
B-1	0 - 4	Fill: medium dense, Silty Sand (SM)
	4 - 16.5	Loose to medium dense, Silty Sand (SM)
	16.5 - 18	Very loose, Clayey Sand (SC), with fat clay pockets, wet
	18 - 21	Soft to firm, Fat Clay (CH)
	21 - 36	Medium dense to very dense, Silty Sand (SM), wet
	36 - 40	Dense, Clayey Sand (SC), with fat clay pockets and gravel, wet
B-2	0 - 2	Poorly Graded Sand with Silt (SP-SM), with roots
	2 - 8	Very loose to loose, Silty Sand (SM)
	8 - 12	Firm to stiff, Sandy Lean Clay (CL), with fat clay pockets
	12 - 14	Very loose, Clayey Sand (SC), wet
	14 - 33	Loose to medium dense, Poorly Graded Sand (SP), wet
	33 - 36	Medium dense, Clayey Sand (SC), with gravel, wet
	36 - 48	Very stiff to hard, Sandy Lean Clay (CL)
	48 - 50	Very stiff to hard, Fat Clay (CH), with lean clay pockets

Details of the soils encountered during drilling are presented on the boring logs. The cohesive soils encountered in the borings have a Liquid Limit (LL) of 38 and Plasticity Index (PI) of 23. The cohesive soils encountered are classified as “CL” and “CH” type soils and the granular soils are classified as “SM”, “SC”, “SP-SM” and “SP” 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 moisture contents. “CL” type soils with lower LL (less than 40) and PI (less than 20) generally do not undergo significant volume changes with changes in moisture content. However, “CL” soils with LL approaching 50 and PI greater than 20 essentially behave as “CH” soils and could undergo significant volume changes.

Groundwater: Groundwater levels and boring cave-in depths encountered during drilling are presented in Table 3. Based on Table 3, groundwater along portions of the alignment is likely to be pressurized.

Table 3. Groundwater Depths below Existing Ground Surface

Boring No.	Date Drilled	Boring Depth (ft)	Groundwater Depth (ft)	Boring Cave in Depth (ft)
B-1	9/21/2018	40	14 (Drilling) 10.4 (9/22/2018)	11.2 (Drilling)
B-2	9/20/2018	50	8 (Drilling) 5.4 (15 min.)	5.4 (Drilling)

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, ground water depths can vary from location to location, and (ii) at any given location, ground water depths can change with time. Ground water depths will vary with seasonal rainfall and other climatic/environmental events. Subsurface conditions may vary at locations away from the borings and in between borings.

Clay soils in the Houston area typically have secondary features such as slickensides, calcareous nodules, and contain sand/silt seams/lenses/layers/pockets. It should be noted that the information in the boring logs is based on 3-inch diameter soil samples and the soil samples were obtained continuously at intervals of 2 feet from the ground surface to a depth of 20 feet in the borings, 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 ENGINEERING ANALYSIS AND RECOMMENDATIONS

Based on the schematic drawings provided by SJRA on June 14, 2018, AEC understands that the proposed improvement is a replacement 760 linear feet 12 inch diameter water line, to be installed by HDD method. The replacement water line will require the excavation of two pits to connect to existing 12 inch diameter water line on either end. AEC anticipates that the entrance and exit pit will require a minimum depth matching the invert depth of the existing water line. Based on the provided schematic drawings, the existing water line invert depth at the entrance pit (i.e. south bank) is approximately 6 feet while the existing water line invert depth at the exit pit (i.e. north bank) is approximately 6.5 feet.

5.1 Water Lines Installed by Horizontal Directional Drilling Method

We understand that the replacement water line will be installed by HDD beneath Panther Branch. Water line installation by HDD should be performed in accordance with Section 33 05 23.13 of the SJRA Construction Specifications. HDD method utilizes steerable drilling systems to install water lines in 2 steps: (i) a pilot hole is drilled with a diameter of 1 to 5 inches along the proposed design centerline; and (ii) the pilot hole is enlarged by backreaming to the desired diameter with high volume and high pressure bentonite slurry, which maintains the bore and prevents caving of the surrounding soils; the product pipe/conduit is also connected to the end of the drilling rod and backreamer assembly by a swivel and pulled back through the enlarged pilot hole.

The Contractor is responsible for selecting, designing, installing, maintaining and monitoring safe drilling systems and retaining professionals who are qualified and experienced to perform the tasks and who are capable of modifying the system, as required. The following discussion provides general guidelines to the Contractor. The information in this report should be reviewed so that appropriate drilling equipment and techniques can be planned and factored into the construction plan and cost estimate.

5.1.1 Geotechnical Design Parameters

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

5.1.2 Loadings on Water Line

The water line will support the weight of the soil and water above the crown.

Earth Loads: For underground utilities to be installed using HDD method, 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 B-2, in Appendix B;
γ = effective unit weight of soil over the water line, in pounds per cubic foot (pcf);
B_d = trench width at top of the water line $< 1.5\ B_c$ (ft);
B_c = outside diameter of the water line (ft);
H = variable height of fill (ft);
when the height of fill above the top of the water line $H_c > 2\ B_d$, $H = H_h$ (height of fill above the middle of the water line). When $H_c < 2\ B_d$, H varies over the height of the water line; and
$K\mu'$ = 0.1650 maximum for sand and gravel,
0.1500 maximum for saturated top soil,
0.1300 maximum for ordinary clay,
0.1100 maximum for saturated clay.

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

Uplift Resistance: AEC recommends that the water line designer determine if the depth of the water line is deep enough so that the soil overburden load and the dead weight of the water line are greater than the buoyant uplift force from the displaced volume of the water line. The potential loss of overburden pressure due to erosion in the channel scour zone should be included in the uplift resistance analysis. When determining uplift resistance, AEC also recommends that the water line designer consider the groundwater level to be at the top of the channel bank.

Lateral Loads: The lateral soil pressure p_l can be calculated from Equation (4); hydrostatic pressure should be added, if applicable.

$$p_l = K_0 (\gamma H_h + p_s) \qquad \text{...........Equation (4)}$$

where: K_0 = Coefficient of earth pressure, at-rest, see Plate B-1, in Appendix B;
H_h = height of fill above the center of the water line (ft);
γ = effective unit weight of soil over the water line (pcf);
p_s = vertical pressure on water line resulting from traffic and/or construction equipment (psf).

5.1.3 Drilling Face Stability during Construction

Based on AEC's borings, the HDD will be installed primarily through granular soils and potentially beneath groundwater. In order to maintain a stable HDD installation, AEC recommends that pressurized bentonite slurry be used to support the HDD pilot hole and backream against caving.

5.1.4 Influence of Drilling on Adjacent Structures

Based on the schematic drawings provided by SJRA, AEC notes that the replacement water line will cross beneath Panther Branch. Care should still be taken to ensure that the drilling/installation operations do not adversely affect any nearby structures or pavements (if any).

Ground Subsidence: Drilling in soft ground can induce some degree of settlement (ground subsidence) of the overlying ground surface if the volume and/or pressure of bentonite slurry is inadequate. If such settlement is excessive, it may cause distress/damage to existing structures and services located above and/or near the drilling zone.

Predicting the amount of loss of ground (or ground subsidence) due to drilling is very difficult, primarily because of the uncertainty involved in the analysis: such as heterogeneous soil properties, subsurface variability, or lack of information about proposed construction equipment and techniques.

Measures to Reduce Distress from Drilling: Impact to existing foundations and structures can be mitigated by following proper drilling procedures. Some methods to mitigate movement and/or distress to existing structures include:

- Supporting the drilling excavation with steel or rigid concrete casing or the pipe material itself, as soon as the excavation is advanced and at short intervals; and
- proper grouting of the annular spaces; the type of equipment and method chosen will require the services of a specialty contractor.

To reduce the potential for the drilling to influence existing structures, we recommend that the outer edge of the influence zone of the water line be a minimum of 5 feet from the outer edge of the bearing (stress) zone of any existing foundations of nearby structures. 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.

The drilling influence zone is assumed to extend a distance of about 2.5i from the center of the drilled tunnel, as shown on Plate C-8, in Appendix C. We emphasize that the size of the influence zone of the bore hole is difficult to determine because several factors influence the response of the soil to drilling operations including type of soil, ground water level, type of drilling equipment, volume and pressure of drilling fluid, experience of operator and other construction in the vicinity.

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

- drilling cannot be located farther than the minimum distance recommended above;
- drilling 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 bore holes;

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

Disturbance and loss of ground from the drilling operation may create surface soil disturbance and subsidence which in turn may cause distress to existing structures (including pavements) located in the zone of soil disturbance.

5.1.5 Pit Excavation

As noted in Section 5.0 of this report, AEC anticipates that entrance and exit pits will be used to connect the replacement water line to the existing water line on both ends. The Contractor should be responsible for designing, constructing and maintaining safe excavations. The excavations should be performed in a manner so as to not cause any distress to existing structures and pavements in the vicinity (if any).

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 B-1, in Appendix B. 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 C-1, in Appendix C. 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 C-2, in Appendix C.

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 C-3, in Appendix C. 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 (5). The design soil parameters for trench bracing design is presented on Plate B-1, in Appendix C.

$$p_a = (q_s + \gamma h_1 + \gamma' h_2)K_a - 2c\sqrt{K_a} + \gamma_w h_2 \qquad \text{............Equation (5)}$$

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

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

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 C-7, in Appendix C.

AEC assumes that the new water line will require an entrance and exit pit to match the invert depths of the existing water line. Based on the schematic drawings provided by SJRA, the invert depth of the water line at the entrance pit (i.e. south bank) is approximately 6 feet while the invert depth of the water line at the exit pit (i.e. north bank) is approximately 6.5 feet. AEC anticipates that open cut excavations will generally encounter granular soils throughout both pits.

Based on the groundwater levels described in Section 4.1 of this report, AEC anticipates that open cut excavations that are 6 to 6.5 feet (for entrance and exit pits) will generally encounter groundwater within the pit zone in the vicinity of Boring B-2. AEC does not anticipate that groundwater will be encountered within the pit excavation in the vicinity of Boring B-1; however, groundwater was encountered in Boring B-1 immediately below the anticipated pit bottom, and groundwater may be higher at the time of construction. Groundwater control recommendations are presented in Section 6.2 of this report, if required. 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.

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, AEC recommends that the groundwater table be lowered at least 3 feet below the bottom of the excavation. Groundwater control should be in accordance with Section 01 57 23.02 of the SJRA Construction Specifications. Groundwater control recommendations are presented in Section 6.2 of this report.

5.1.6 Backfill for Excavated Pits

Backfill for the entrance and exit pits should be in accordance with Section 31 21 33 of the SJRA Construction Specifications. Embedment material and backfill should be placed in loose lifts not exceeding 8 inches and compacted in accordance with Section 31 21 33 of the latest edition of the SJRA Construction Specifications.

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. Groundwater control should be in general accordance with Section 31 21 33 of the latest edition of the SJRA Construction Specifications.

The Contractor should be responsible for selecting, designing, constructing, maintaining, and monitoring a groundwater control system and adapt his operations to ensure the stability of the excavations. Groundwater information presented in Section 4.1 and elsewhere in this report, along with consideration for potential environmental and site variation between the time of our field exploration and construction, should be incorporated in evaluating groundwater depths. The following recommendations are intended to guide the Contractor during design and construction of the dewatering system.

In cohesive soils seepage rates are lower than in granular soils and groundwater is usually collected in sumps and 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, possible ground water control measures include: (i) multi-staged wellpoints; (ii) deep wells or turbines (in granular soils); (iii) ejectors or educators (for silts); or (iv) water-tight sheet pile cut-off walls. Generally, AEC recommends the groundwater depth be lowered at least 3 feet below the excavation bottom to be able to work on a firm surface when water-bearing granular soils are encountered.

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

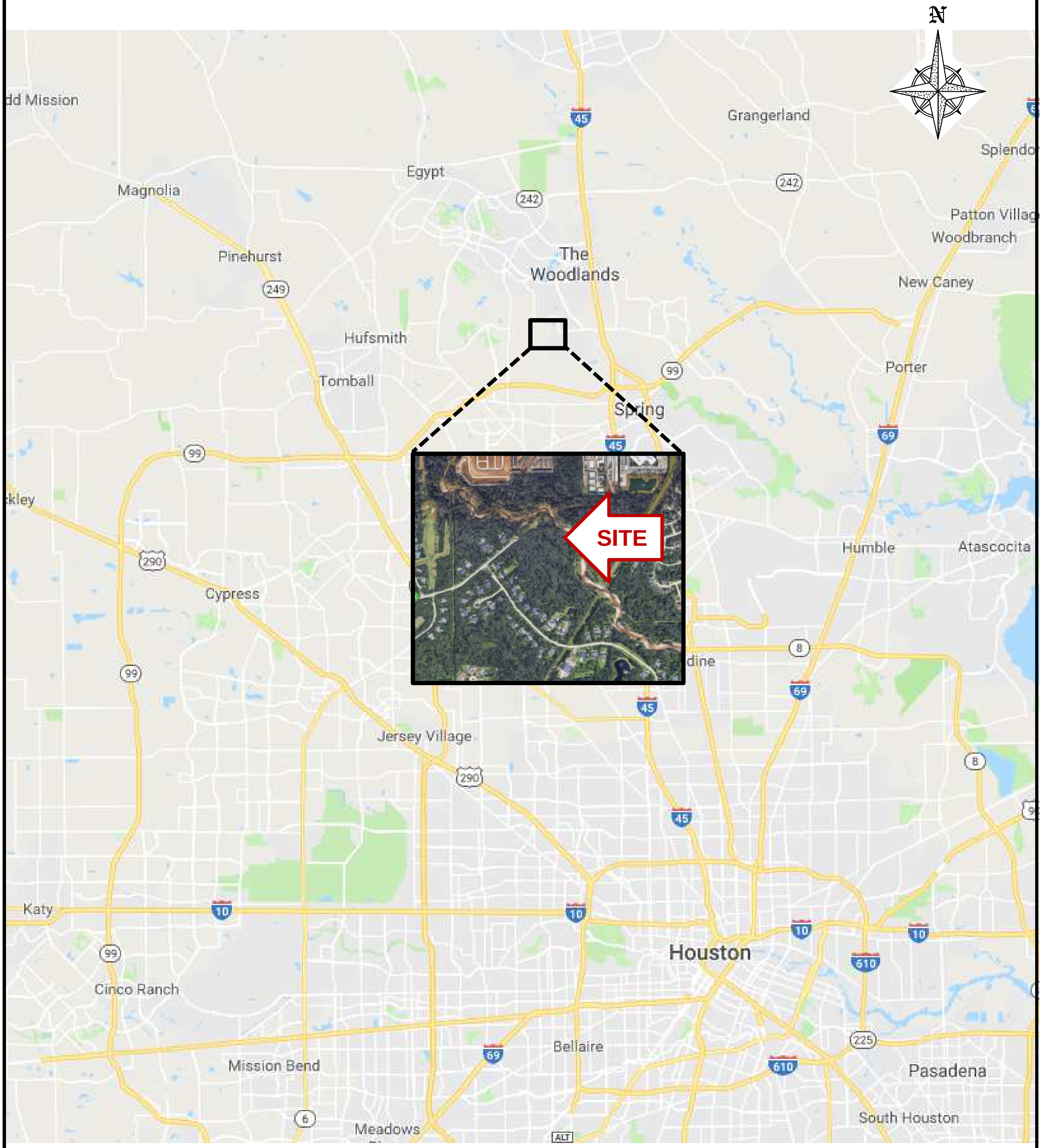
7.0 LIMITATIONS

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

AEC should be notified immediately.

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

APPENDIX A



AVILES ENGINEERING CORPORATION

VICINITY MAP

12” WATER LINE REPLACEMENT CROSSING PANTHER
BRANCH AT GROGAN’S POINT ROAD
THE WOODLANDS, TEXAS

AEC REPORT NO.: G152-18	DATE: 10-26-2018	SOURCE DRAWING PROVIDED BY: GOOGLE MAP
APPROX. SCALE: N.T.S.	DRAFTED BY: JG	PLATE NO.: PLATE A-1

LEGEND

B-# (X') ⌖	BORING NO. AND (DEPTH)

AVILES ENGINEERING CORPORATION

BORING LOCATION PLAN

12" WATER LINE REPLACEMENT CROSSING PANTHER
BRANCH AT GROGAN'S POINT ROAD
THE WOODLANDS, TEXAS

AEC PROJECT NO.: G152-18	DATE: 10-26-18	SOURCE DRAWING PROVIDED BY: GOOGLE EARTH
SCALE: 1" = 200'	DRAFTED BY: BpJ	PLATE NO.: PLATE A-2

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **12" Water Line Replacement at Panther Branch**

BORING **B-1**

DATE **09/21/18**

TYPE **4" Dry Auger / Wet Rotary**

LOCATION **See Boring Location Plan**

Approximate Surface Elevation (feet): 104

SHEAR STRENGTH, TSF:
- △ Confined Compression
- ● Unconfined Compression
- ○ Pocket Penetrometer
- ☐ Torvane

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	SHEAR STRENGTH, TSF (0.5, 1, 1.5, 2)	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Fill: medium dense, brown Silty Sand (SM) -with sandy clay pockets and roots 0'-2'		13						
	-with lean clay seams 2'-4'	30	11			41	16	14	2
5	Loose to medium dense, tannish gray Silty Sand (SM) -with clayey sand pockets 4'-6'	23	12						
	-light grayish tan 6'-10'	28	4						
	-with gravel 8'-10'	22	7			17			
10	-gray 10'-16', with clayey sand pockets 10'-12' ▼ -boring cave-in at 11.2' during drilling	15	14						
	▽	9	17						
15	-with gravel and clay pockets 14'-16'	28	16			14	NP	NP	NP
	Very loose, gray and dark gray Clayey Sand (SC), with fat clay pockets, wet	3	19						
	Soft to firm, yellowish tan Fat Clay (CH)	4	23						
20									
	Medium dense to very dense, light tan Silty Sand (SM), wet								
	-with clay pockets 23'-25' and gravel pockets 23'-30'	56	20			18	NP	NP	NP
25									
	-tan 28'-35', with fat clay pockets 28'-30'	30	20						
30									
		34	26						
35									

BORING DRILLED TO **14** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **14** FEET WHILE DRILLING ▽

WATER LEVEL AT **10.4** FEET AFTER **1 DAY** ▼

DRILLED BY **Van & Sons** DRAFTED BY **JG** LOGGED BY **BTC**

PROJECT NO. G152-18

PLATE A-3

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **12" Water Line Replacement at Panther Branch**   BORING **B-1**

DATE **09/21/18**   TYPE **4" Dry Auger / Wet Rotary**   LOCATION **See Boring Location Plan**

SHEAR STRENGTH, TSF
△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane
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
	▨	☒	Dense, tan Clayey Sand (SC), with fat clay pockets and gravel, wet	39	23			26			
40			Termination Depth = 40 feet								
45											
50											
55											
60											
65											
70											

BORING DRILLED TO **14** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **14** FEET WHILE DRILLING ▽
WATER LEVEL AT **10.4** FEET AFTER **1 DAY** ▼
DRILLED BY **Van & Sons**   DRAFTED BY **JG**   LOGGED BY **BTC**

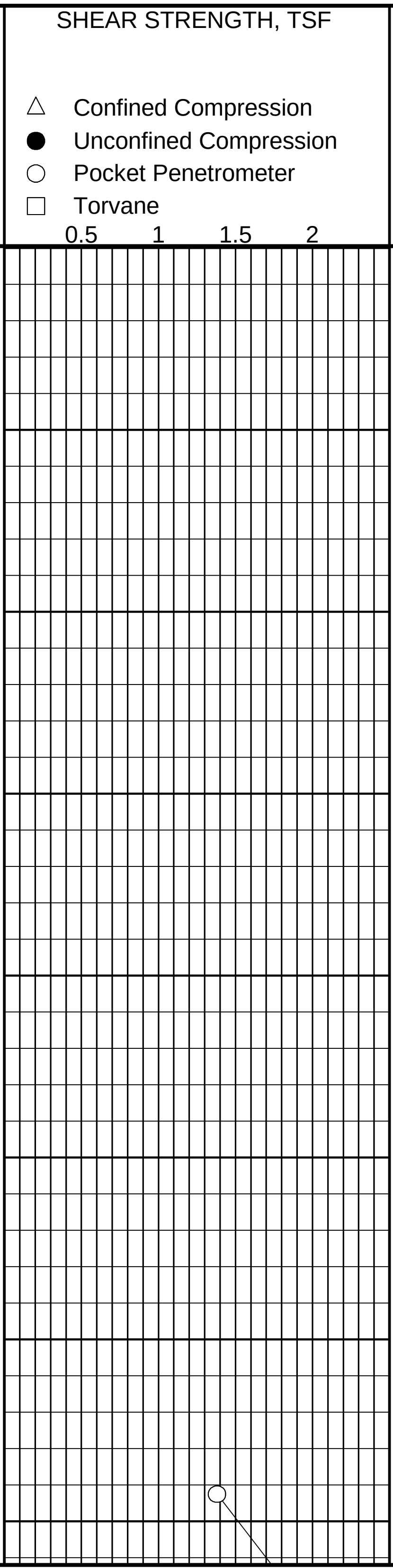
AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **12" Water Line Replacement at Panther Branch** BORING **B-2**

DATE **09/20/18** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

Approximate Surface Elevation (feet): 98

DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Tan Poorly Graded Sand with Silt (SM), with roots		7		5			
Very loose to loose, tan and gray Silty Sand (SM) -with clayey sand pockets 2'-4'	2	16					
-tannish gray 4'-6', with sandy clay seams 4'-8' -boring cave-in at 5.2' during drilling	2	22					
-gray 6'-8'	5	16		43	NP	NP	NP
Firm to stiff, gray Sandy Lean Clay (CL), with fat clay pockets -with organic pockets 8'-10'	5	27					
-dark brownish gray 10'-12'	12	35		58			
Very loose, dark brownish gray Clayey Sand (SC), wet	0	38		28	35	21	14
Loose to medium dense, gray and light gray Poorly Graded Sand (SP), wet -with gravel and sandy clay pockets 14'-16'	10	23					
-light gray and tan, with clayey sand pockets 16'-18'	15	21					
-tan, with gravel 18'-20'	22	21		4	NP	NP	NP
-reddish tan 23'-25'	28	17					
-tan, with clayey sand pockets 28'-30'	23	17					
Medium dense, light gray and tan Clayey Sand (SC), with gravel, wet	31	17		34	40	14	26



BORING DRILLED TO **8** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **8** FEET WHILE DRILLING ∇
WATER LEVEL AT **5.2** FEET AFTER **15 MIN** ▼
DRILLED BY **Van & Sons** DRAFTED BY **JG** LOGGED BY **BTC**

PROJECT NO. G152-18

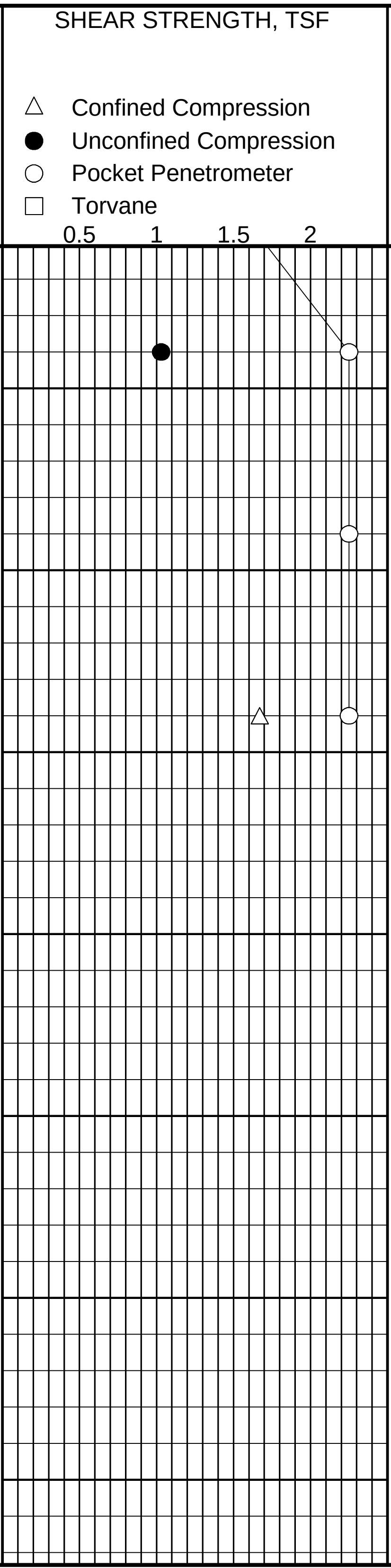
AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **12" Water Line Replacement at Panther Branch**
BORING **B-2**

DATE **09/20/18**
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	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
			Very stiff to hard, light gray Sandy Lean Clay (CL)							
					17	115.7				
40										
					15		54	38	15	23
45										
			Very stiff to hard, light gray Fat Clay (CH), with lean clay pockets		28	98.0				
50			Termination Depth = 50 feet							
55										
60										
65										
70										

SHEAR STRENGTH, TSF



BORING DRILLED TO **8** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **8** FEET WHILE DRILLING
WATER LEVEL AT **5.2** FEET AFTER **15 MIN**
DRILLED BY **Van & Sons** DRAFTED BY **JG** LOGGED BY **BTC**

PROJECT NO. G152-18

PLATE A-4

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
▩	Fill
▥	Silty sand
▨	Clayey sand
▨	High plasticity clay
▦	Poorly graded sand with silt
▨	Low plasticity clay
░	Poorly graded sand
Misc. Symbols	
▽	Water table depth during drilling
▼	Subsequent water table depth
○	Pocket Penetrometer
●	Unconfined Compression
△	Confined Compression
Soil Samplers	
◨	Auger
⊠	Standard penetration test

Symbol	Description
■	Undisturbed thin wall Shelby tube

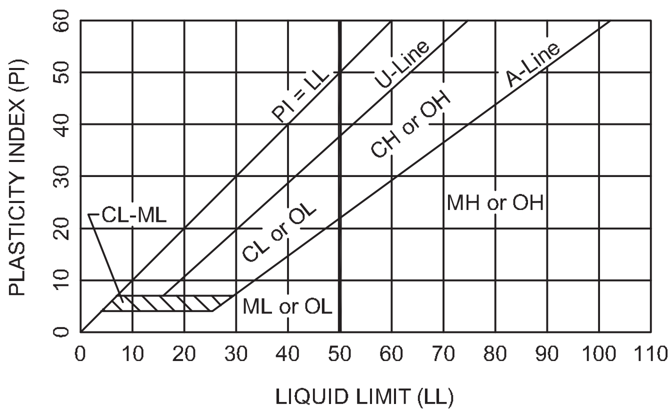
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

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

▩	Fill
▨	Clay (CH)
▨	Clay (CL)
░	Sand
▥	Silt

TERMS USED ON BORING LOGS

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

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

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

152 76.2 19.1 4.76 2.00 0.420 0.074 0.002

SOIL GRAIN SIZE IN MILLIMETERS

STRENGTH OF COHESIVE SOILS

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

RELATIVE DENSITY OF COHESIONLESS SOILS FROM STANDARD PENETRATION TEST

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

SPLIT-BARREL SAMPLER DRIVING RECORD

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

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

DRY STRENGTH ASTM D2488

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

MOISTURE CONDITION ASTM D2488

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

SOIL STRUCTURE

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

ASTM & TXDOT DESIGNATION FOR SOIL LABORATORY TESTS

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

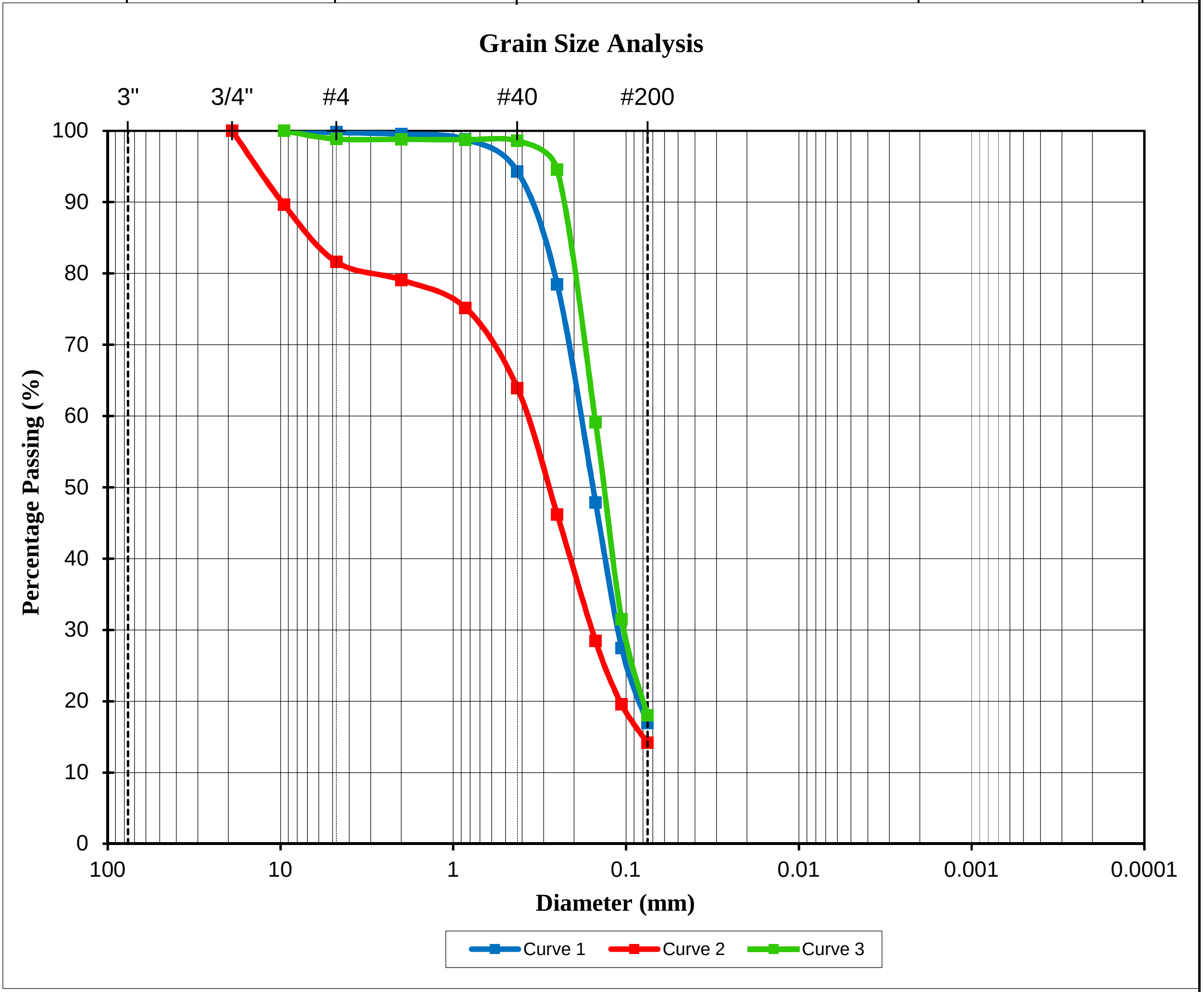
AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE

Project : 12" Water Line Replacement at Panther Branch
Location of Project: The Woodlands, Texas
Job No.: G152-18
Date of Testing: 10/1/2018

	Sand			
Gravel	Coarse to Medium	Fine	Silt	Clay



Curve	Boring	Depth (ft)	Soil Description	Cu	Cc	D_{50} (mm)
1	B-1	8-10	Silty Sand (SM)	N/A	N/A	0.155
2	B-1	14-16	Silty Sand (SM)	N/A	N/A	0.280
3	B-1	23-25	Silty Sand (SM)	N/A	N/A	0.134

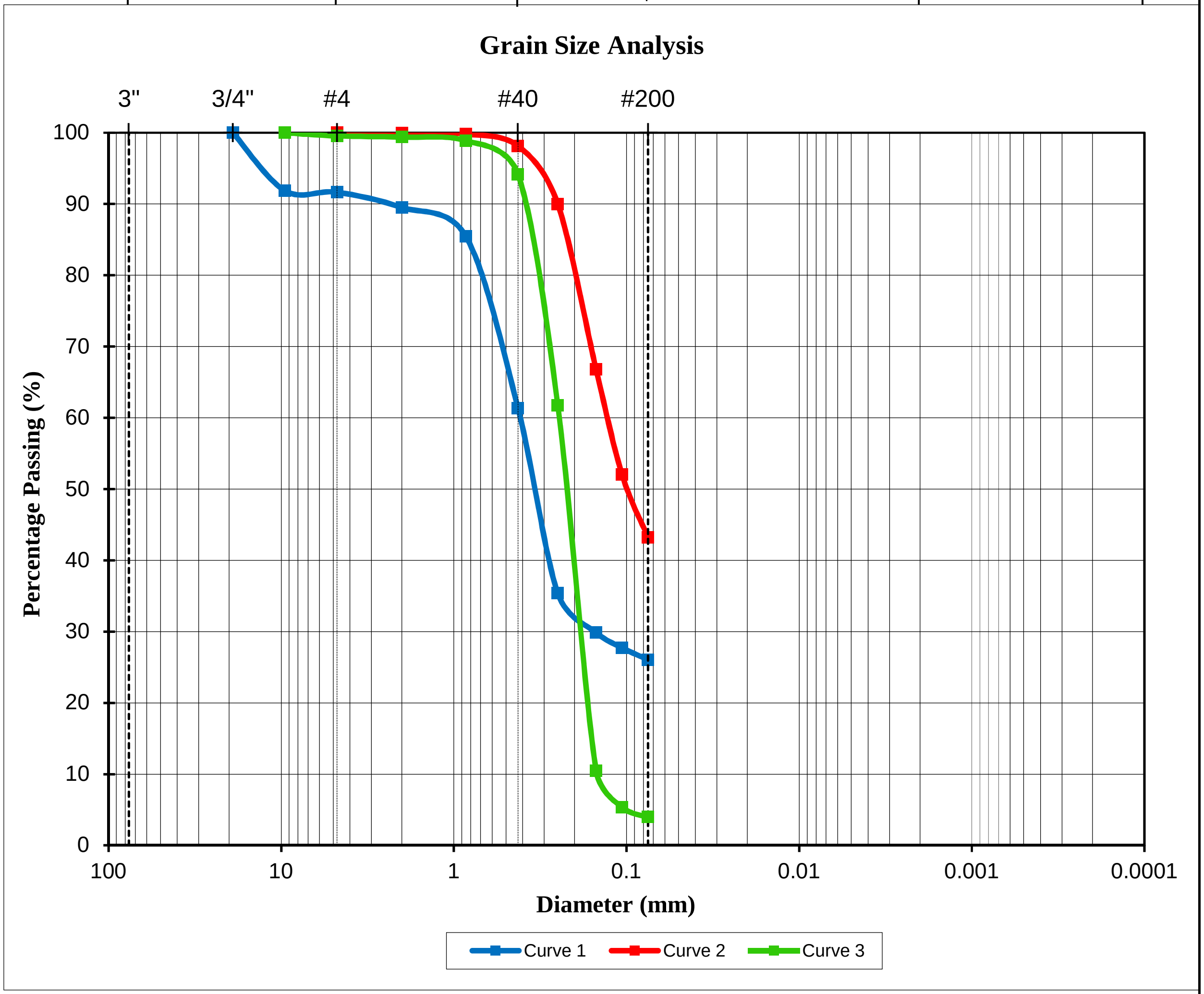
AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE

Project : 12" Water Line Replacement at Panther Branch
Location of Project: The Woodlands, Texas
Job No.: G152-18
Date of Testing: 10/1/2018

		Sand				
	Gravel	Coarse to Medium	Fine	Silt	Clay	



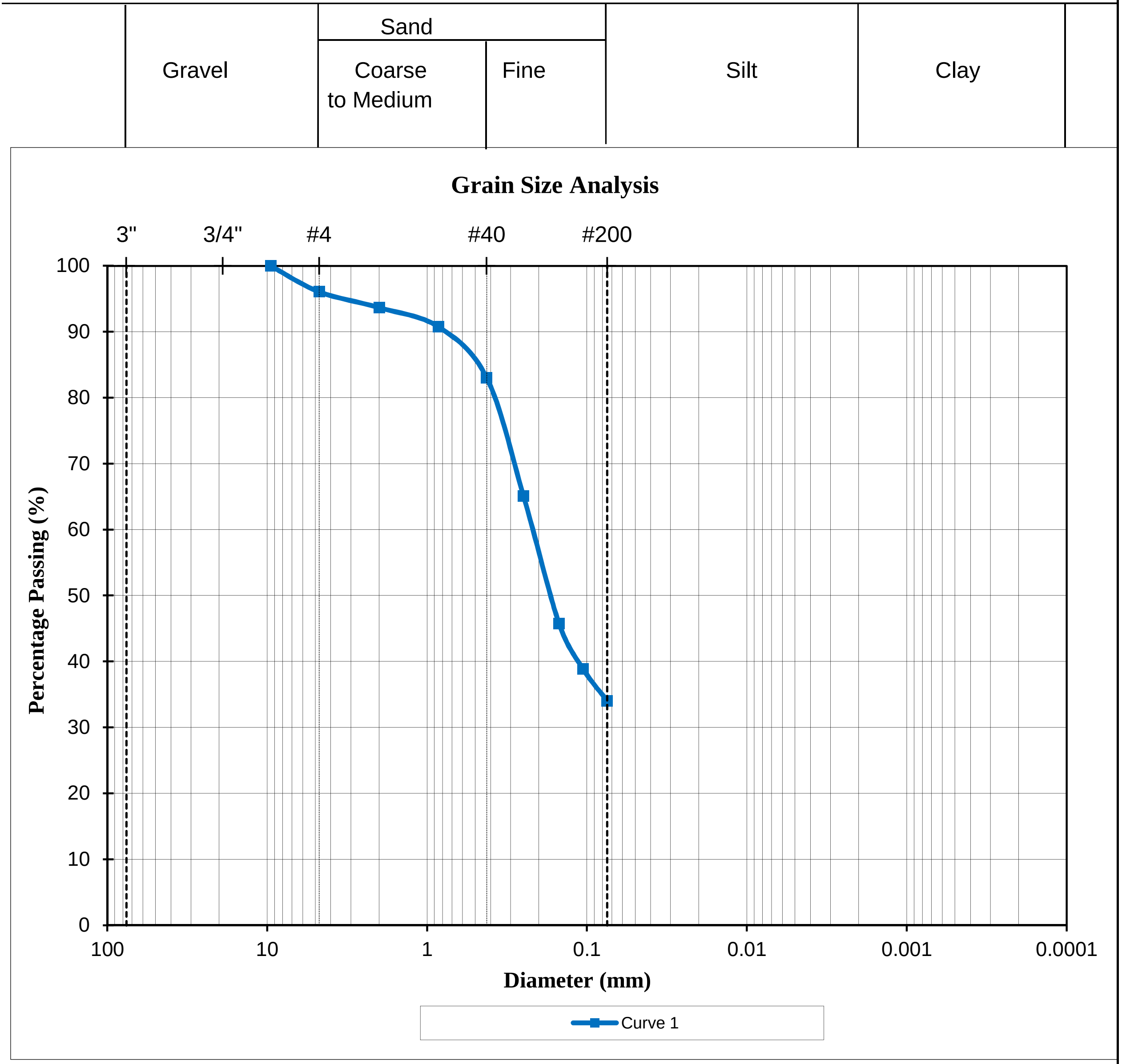
Curve	Boring	Depth (ft)	Soil Description	Cu	Cc	D$_{50}$ (mm)
1	B-1	38-40	Clayey Sand (SC)	N/A	N/A	0.337
2	B-2	6-8	Silty Sand (SM)	N/A	N/A	0.098
3	B-2	18-20	Poorly Graded Sand (SP)	1.69	0.93	0.222

AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE

Project : 12" Water Line Replacement at Panther Branch **Job No.:** G152-18
Location of Project: The Woodlands, Texas **Date of Testing:** 10/1/2018



Curve	**Boring**	**Depth (ft)**	**Soil Description**	**Cu**	**Cc**	**D_{50} (mm)**
1	B-2	33-35	Clayey Sand (SC)	N/A	N/A	0.168

AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

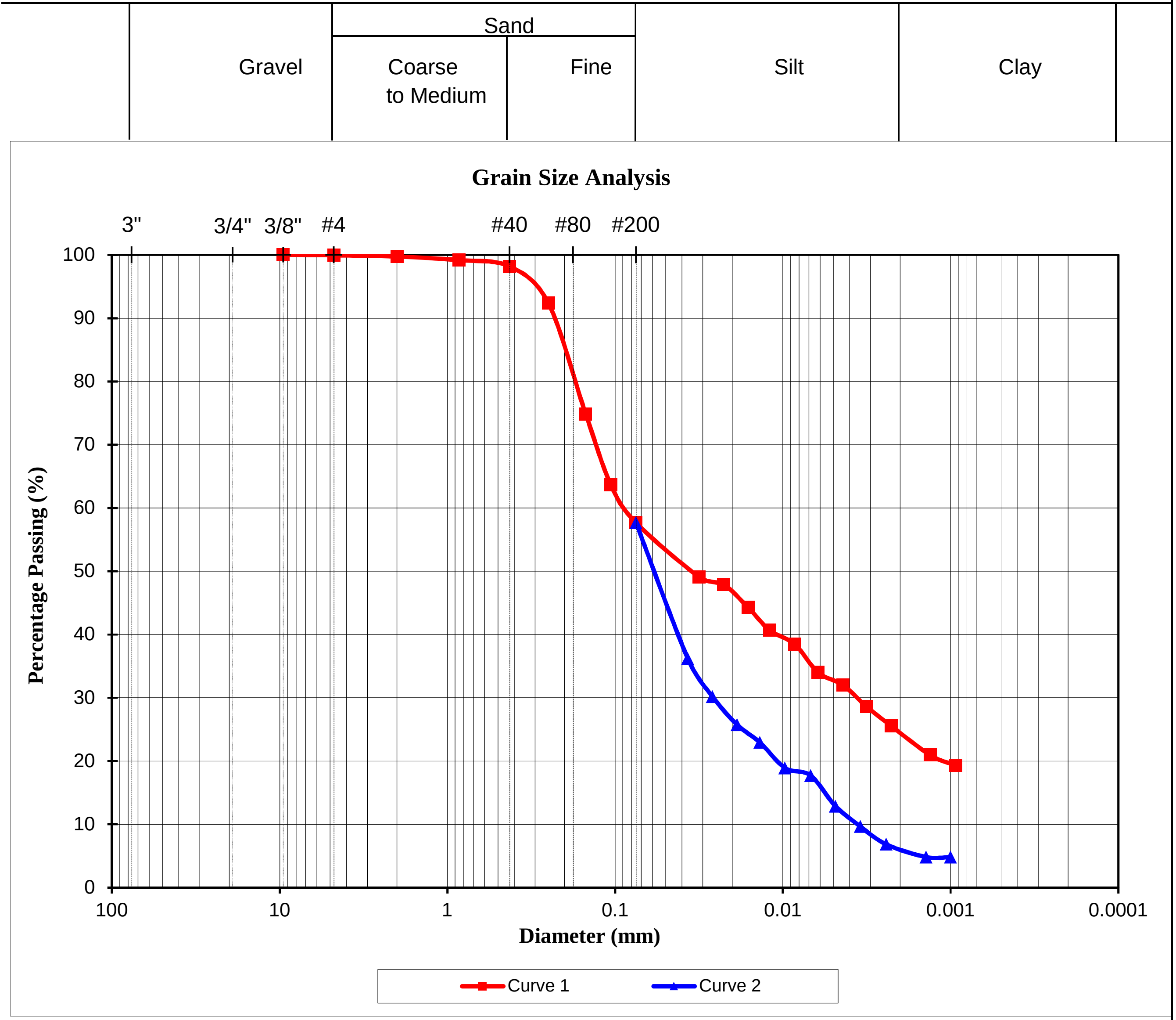
SIEVE & DOUBLE HYDROMETER TESTS (ASTM D 4221)

Project: 12" Water Line Replacement at Panther Branch

Location of Project: The Woodlands, Texas

Job No.: G152-18

Date of Testing: 10/1/2018



Curve	Boring	Depth (ft)	Soil Description	Cu	Cc	%-5µm	D_{50} (mm)
1	B-2	10-12 (a)	Sandy Lean Clay (CL)	N/A	N/A	32.68	0.035
2	B-2	10-12 (b)	Sandy Lean Clay (CL)	N/A	N/A	13.17	N/A
				% Dispersion =		40.29%	

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: 12" Water Line Replacement at Panther Branch
Project No.: G152-18 Test Date: 10/5/2018

Boring Number	**Depth, feet**	**2 Minutes**		**1 Hour**		**6 Hours**	
		Grade	**C (deg)**	**Grade**	**C (deg)**	**Grade**	**C (deg)**
B-1	4-6	2	22.2	2	22.3	2	22.3
B-2	10-12	3	22.2	4	22.3	4	22.3

Grade Classification:

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

Interpretation:

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

APPENDIX B

G152-18 12" WATER LINE REPLACEMENT CROSSING PANTHER BRANCH, THE WOODLANDS, TEXAS
SOIL PARAMETERS FOR WATERLINE DESIGN AND CONSTRUCTION

Boring	**Depth (ft)**	**Soil Type**	**γ (pcf)**	**γ' (pcf)**	**OSHA Type**	**Short-Term**					**Long-Term**				
						C (psf)	**φ (deg)**	**K_a**	**K_0**	**K_p**	**C' (psf)**	**φ' (deg)**	**K_a**	**K_0**	**K_p**
B-1	0-2	Fill: SM	120	58	C	0	26	0.39	0.56	2.56	0	26	0.39	0.56	2.56
	2-4	Fill: medium dense SM	120	58	C	0	32	0.31	0.47	3.25	0	32	0.31	0.47	3.25
	4-10	Medium dense SM	120	58	C	0	30	0.33	0.50	3.00	0	30	0.33	0.50	3.00
	10-15	Loose to medium dense SM	115	53	C	0	28	0.36	0.53	2.77	0	28	0.36	0.53	2.77
B-2	0-8	Very loose to loose SP-SM/SM	115	53	C	0	26	0.39	0.56	2.56	0	26	0.39	0.56	2.56
	8-12	Firm to stiff CL	120	58	C	400	0	1.00	1.00	1.00	25	18	0.53	0.69	1.89
	12-14	Very loose SC	115	53	C	0	26	0.39	0.56	2.56	0	26	0.39	0.56	2.56
	14-15	Medium dense SP	120	58	C	0	28	0.36	0.53	2.77	0	28	0.36	0.53	2.77

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

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

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

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

(5) CL = Lean Clay, SM = Silty Sand, SP = Poorly Graded Sand;

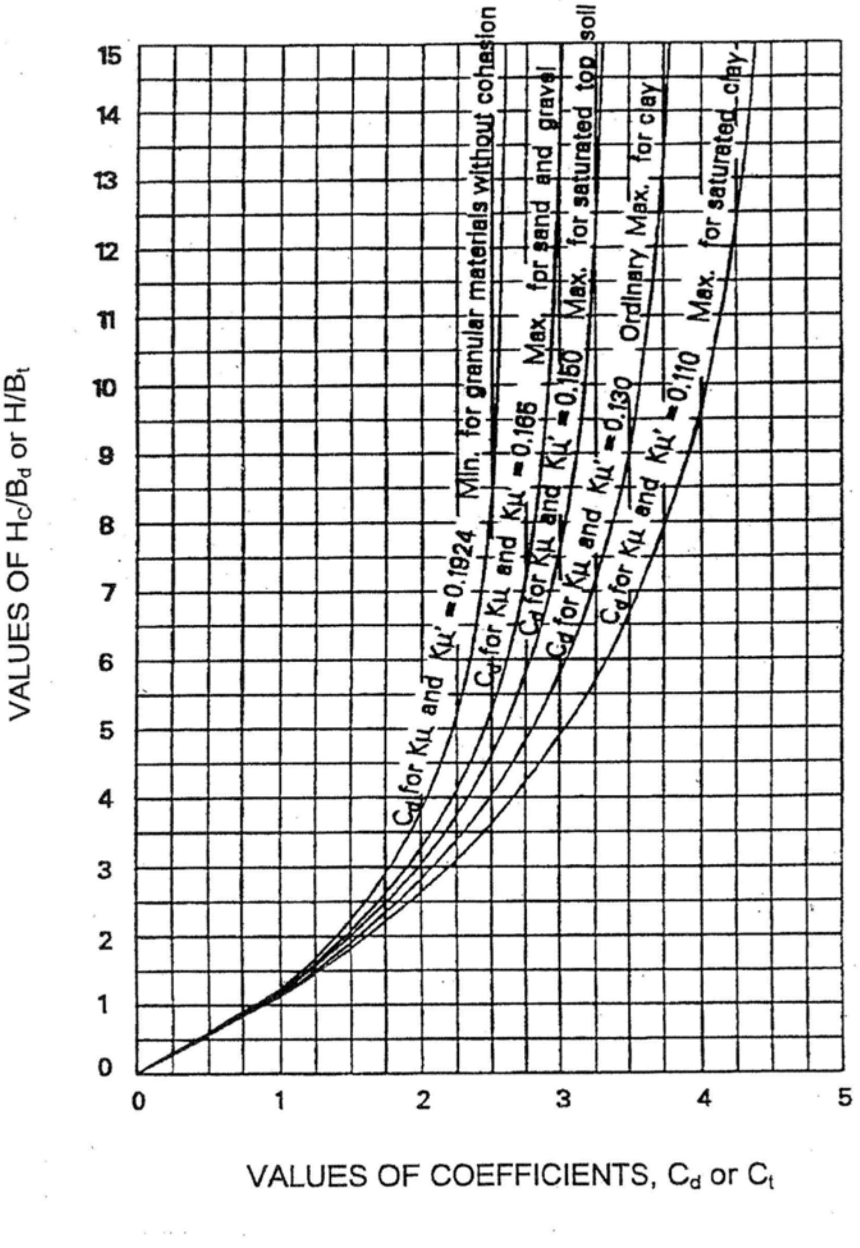
(6) OSHA Soil Types for soils in the top 20 feet below grade:

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

B: cohesive soils with qu = 0.5 tsf or greater

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

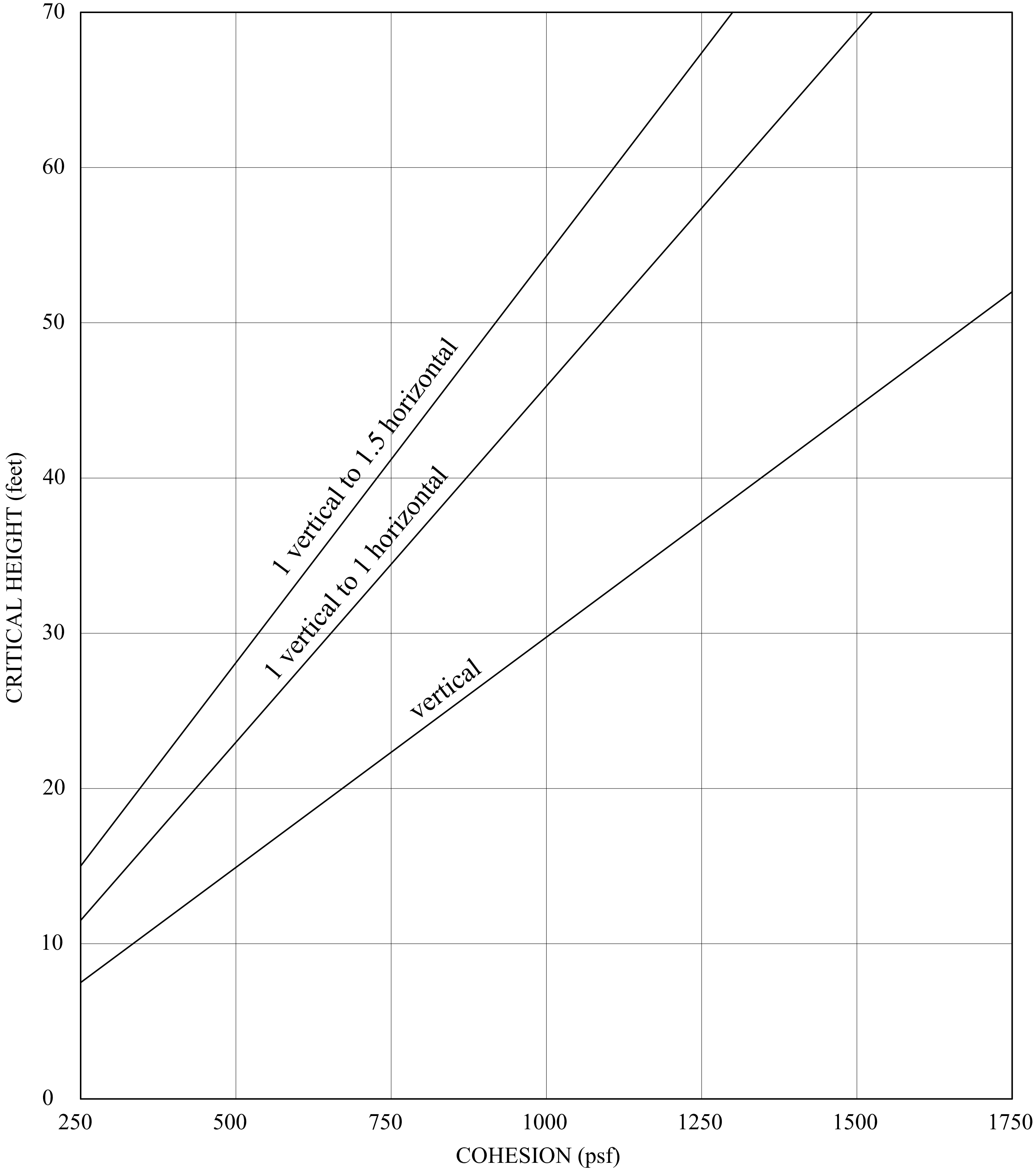
C*: submerged cohesive soils; dewatered cohesive soils can be considered OSHA Type B.



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

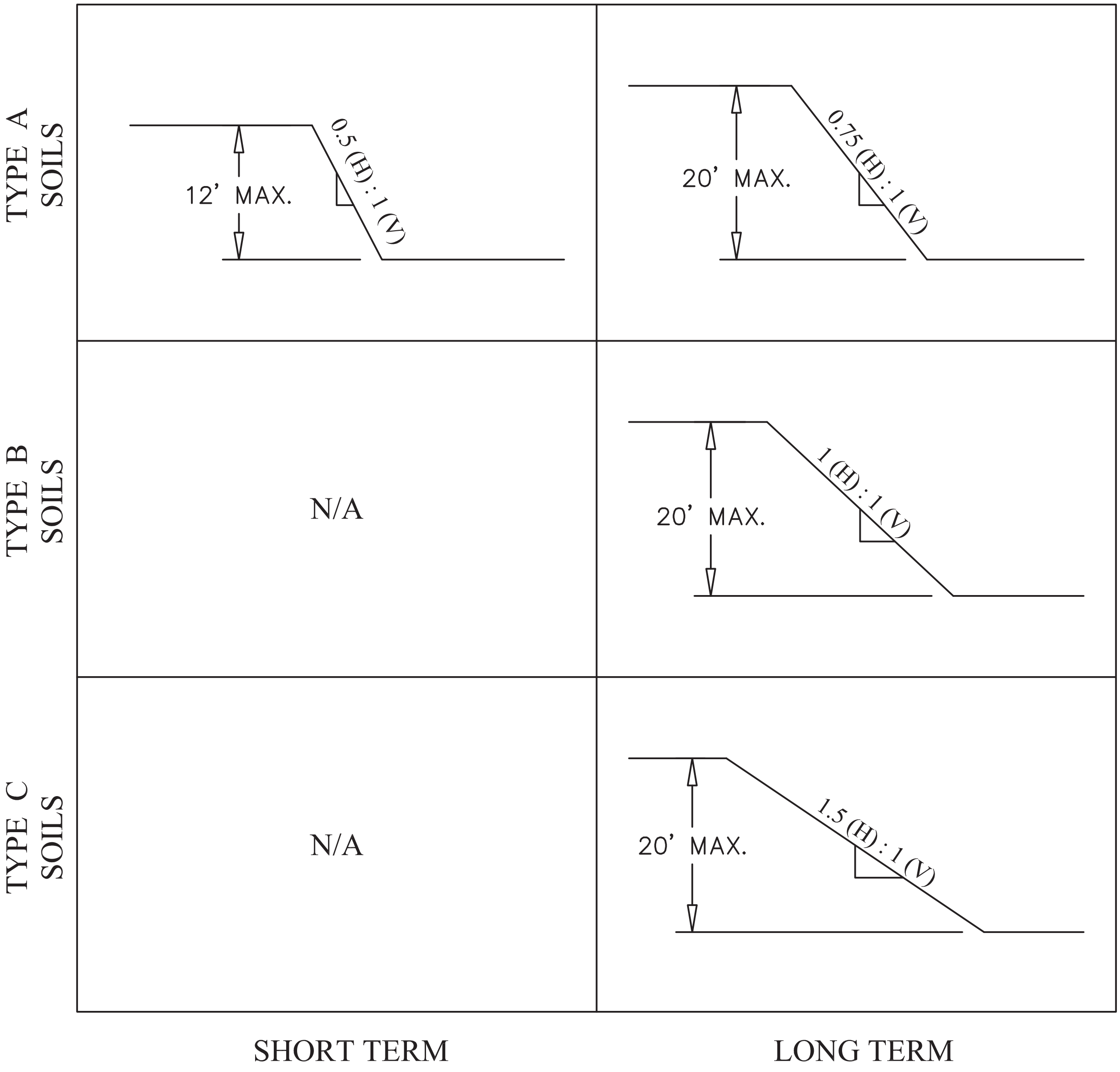
APPENDIX C

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



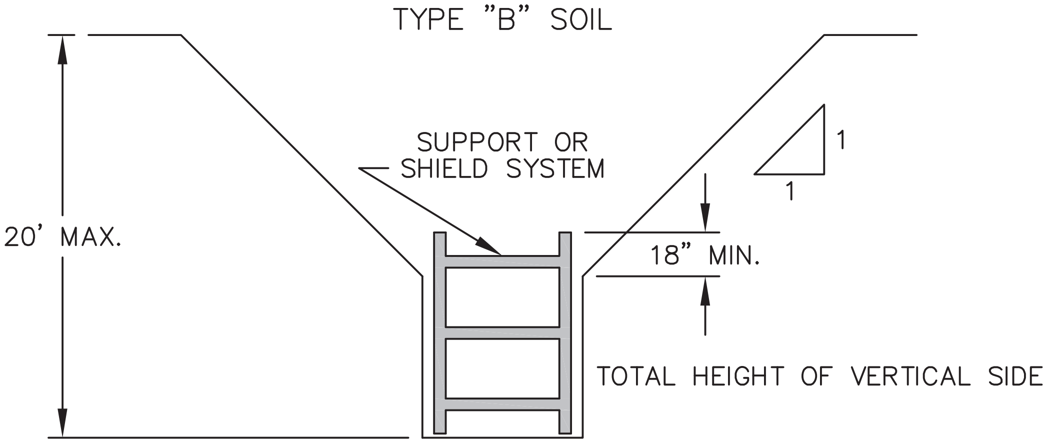
NOTES:

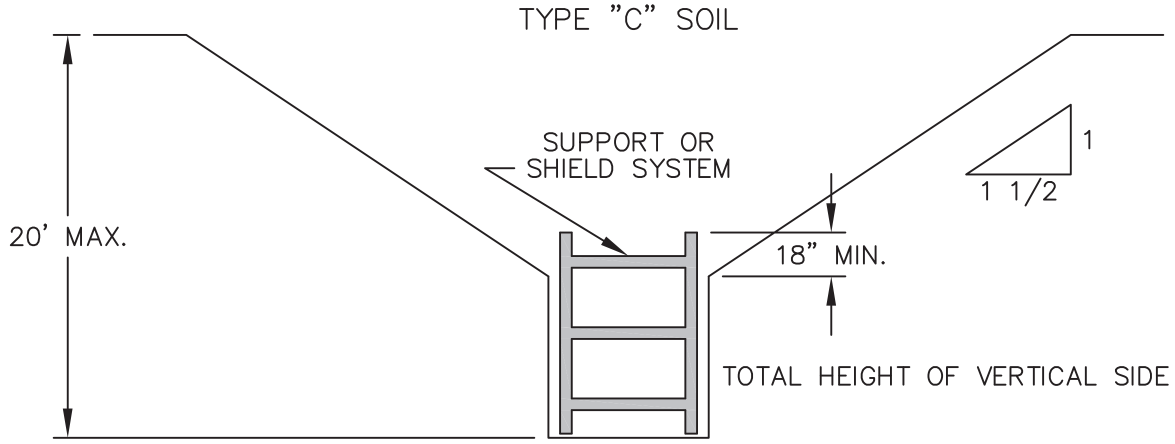
(1) For Type A soils, a short term maximum allowable slope of 0.5 (H) : 1 (V) is allowed in excavations that are 12 feet or less in depth; short term (24 hours or less) maximum allowable slopes for excavations greater than 12 feet in depth shall be 0.75 (H) : 1 (V).

(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.

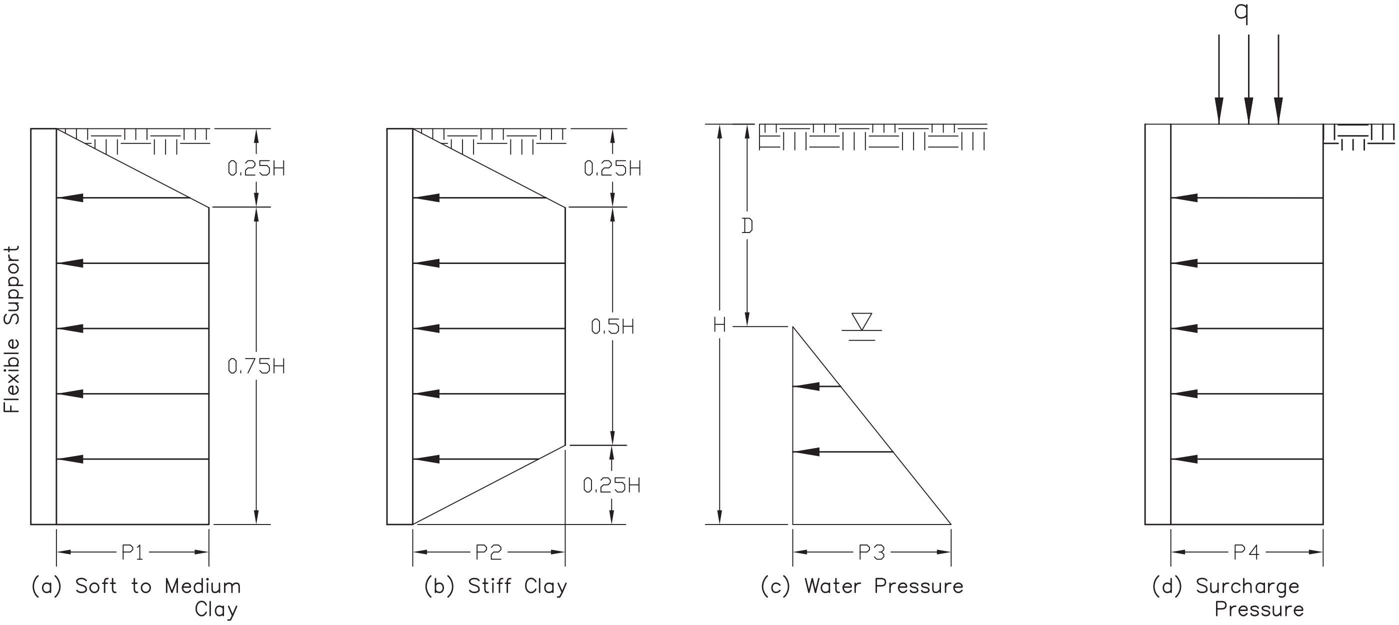
A COMBINATION OF BRACING AND OPEN CUTS





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

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



Empirical Pressure Distributions

Where:

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

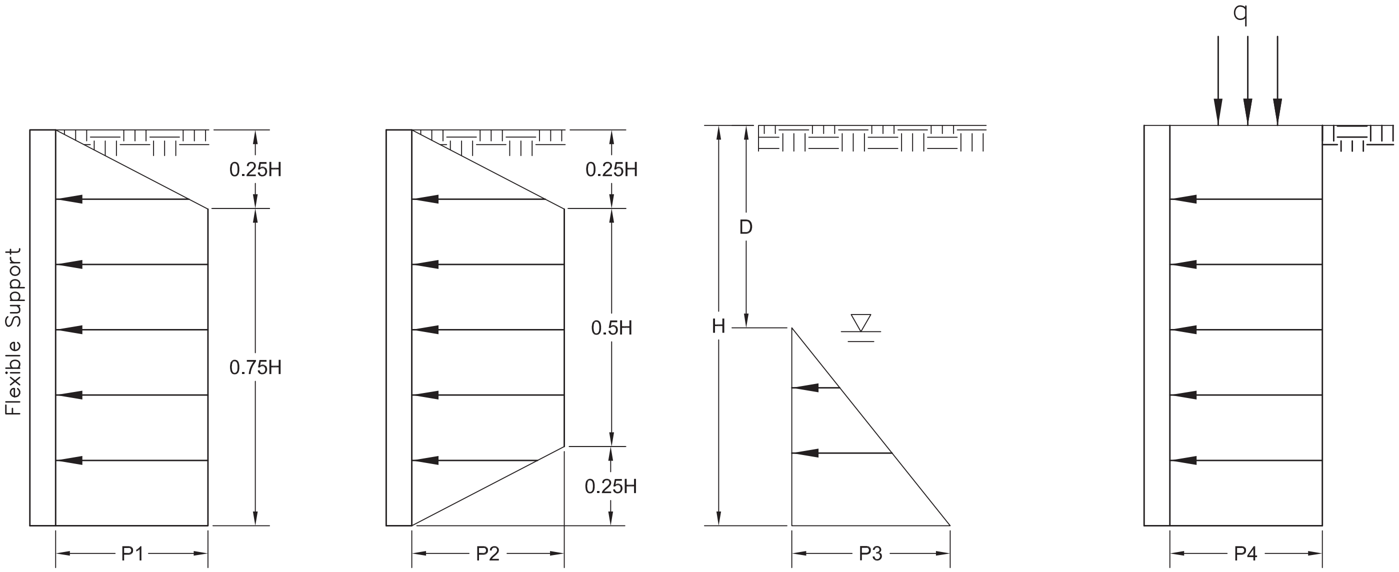
Notes:

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

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

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

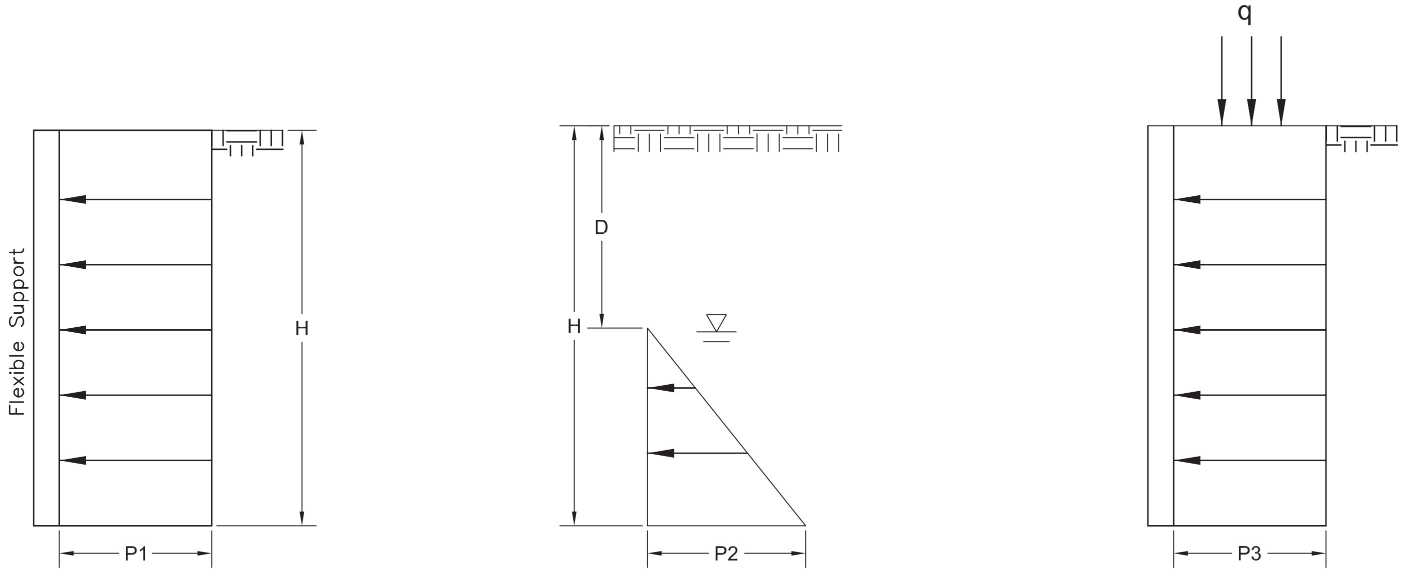
Where:
H = Total excavation depth, feet
D = Depth to water table, feet
$P1$ = Lateral earth pressure = $\gamma H\text{-}4S_u$, psf
$P2$ = Lateral earth pressure = $0.2\gamma H$, psf
$P3$ = Water pressure = γ_w (H-D), psf
$P4$ = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
S_u = Undrained shear strength = $q_u/2$, psf
q_u = Unconfined compressive strength, psf
K_a = Coefficient of active earth pressure

Notes:

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

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

LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN SAND



(a) Sand (b) Water Pressure (c) Surcharge Pressure

Empirical Pressure Distributions

Where:

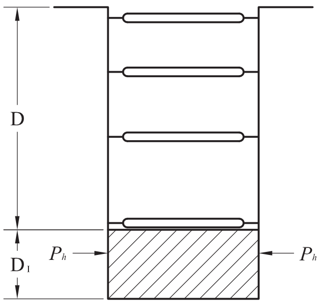
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $0.65*\gamma H K_a$, psf
P2 = Water pressure = $\gamma_w (H\text{-}D)$, psf
P3 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf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

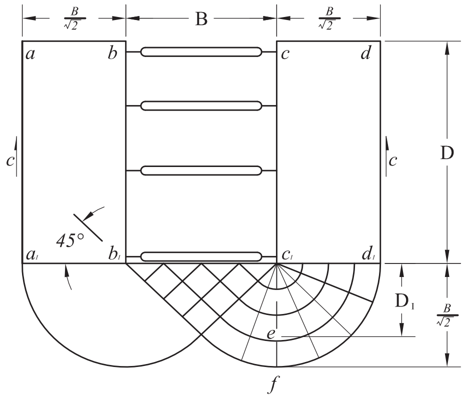
Notes:

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

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

BOTTOM STABILITY FOR BRACED EXCAVATION IN CLAY





Factor of Safety against bottom of heave,

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

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

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

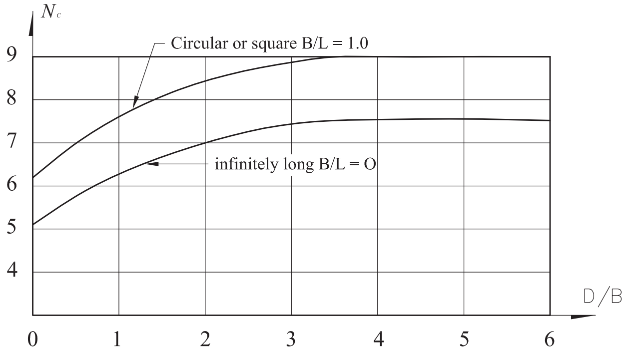
$$\text{Depth of Buried Length, } (D_1) = \frac{1.5(\gamma D+q)-NcC}{(C/B)-0.5\gamma}\ ;\ D_1 \geq 5\ ft.$$

Pressure on buried length, P_{h}:

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

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

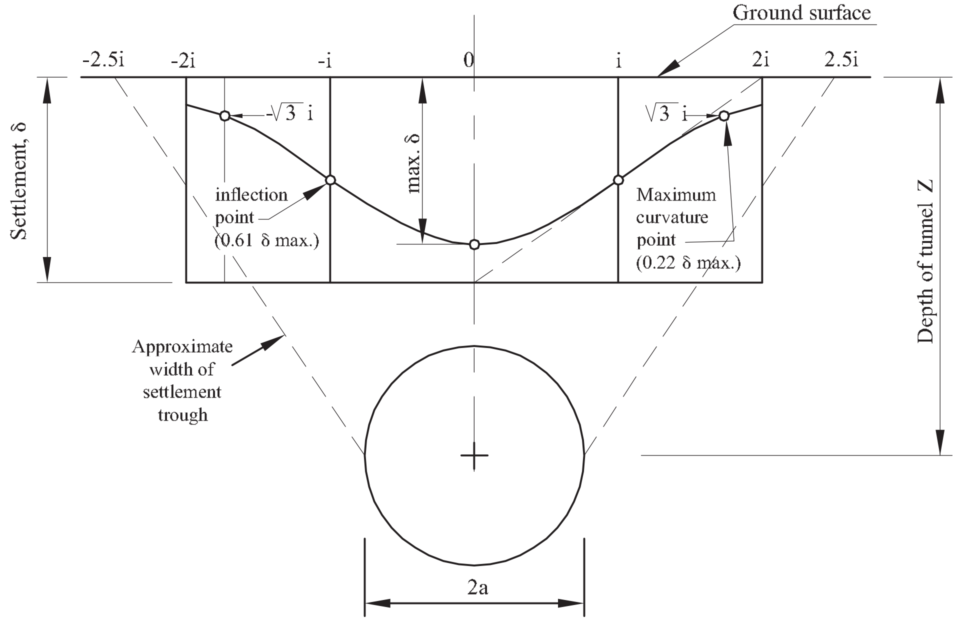
where; B = width of excavation



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

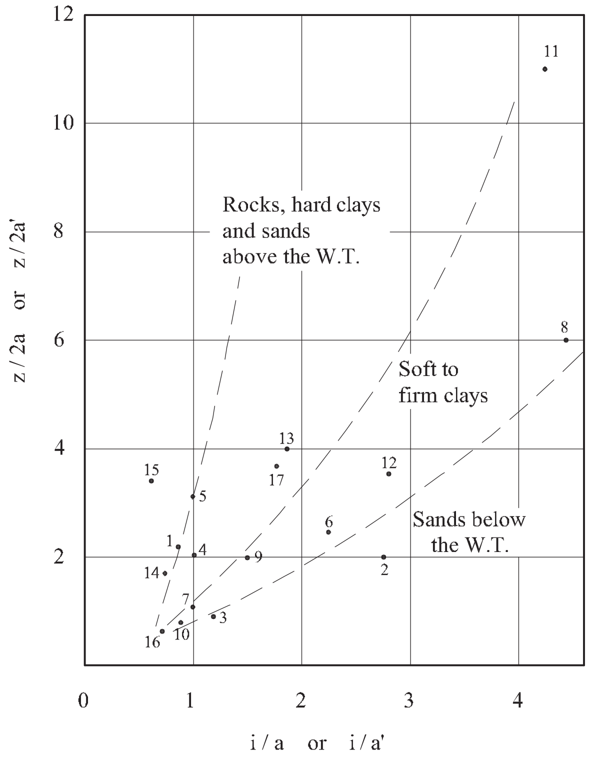
Reference: Bjerrum, L. and Eide, O., Stability of Strutted Excavations in Clay, Geotechnique, 6, 32-47 (1956).

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.